QCEC

Quality Civil Engineering Consultants

Duckworth Street Retaining Wall
Final Report
ENGI 8700

Prepared for: Acuren & Dr. Bruneau
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QCEC

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April 3, 2013

Emad Rizk
Acuren Group Inc.
112 Forest Rd
St. John's, NL, A1A 1E6

Dear Mr. Rizk,

Please see the enclosed document for details on the design and construction of the Duckworth Street temporary retaining wall in St. John's, Newfoundland.

The final project report details the design selection process followed by QCEC to select the most appropriate temporary retaining wall for construction of the Duckworth Street parking garage and condominium complex. The detailed design process is also discussed in depth for the selected retaining wall as well as the drafting and cost estimate. Finally, by request of the client, a construction plan and schedule has been developed and included in the following report.

If there are any questions regarding the project report, please feel free to contact the undersigned.

Sincerely,

Quality Civil Engineering Consultants

Chantel Nicolle

Erica Soucy

Qiong Zhang

Chenel Waight

cc. Dr. Steve Bruneau

Table of Contents

1.0 PROJECT DESCRIPTION

Quality Civil Engineering Consultants (QCEC) was contracted by Acuren Group Inc. to design a temporary retaining wall. A new parking garage and condominium complex is in development on Duckworth Street and a temporary retaining wall is required for support along Henry Street to allow for construction of the building (Figure 1.1).

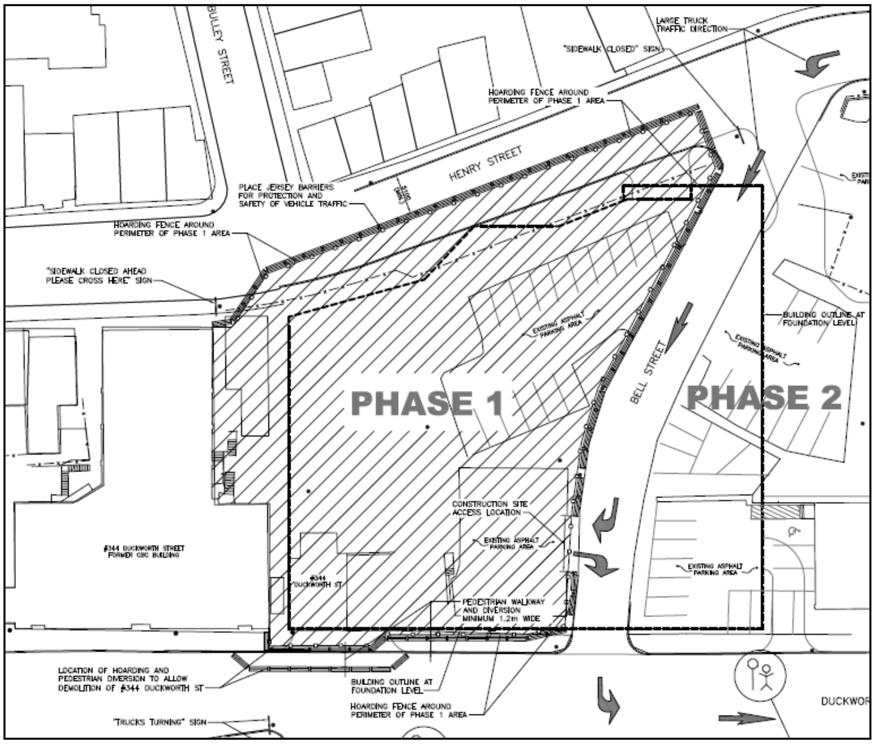

Figure 1.1: Site Location

The retaining wall will have a life of approximately three years then it will be backfilled and the new building will continue to provide long term support.

A geotechnical investigation was completed on the site in 2011 and a topographical survey of the site is also available. The developers of the parking garage also provided architectural drawings.

2.0 APPROACH

Initially, a soil profile of the retaining wall area was developed based on the provided geotechnical investigation. The soil profile includes soil details along the face of the retaining wall location as well as three cross sections along the length. QCEC used the soil profile to assist in selecting an appropriate retaining wall for the soil type on site. Site conditions and the soil profile are discussed in more detail in Section 4.0.

Several retaining wall types were researched in an effort to select the best option for the current site conditions and restraints. Potential solutions that were researched include:

- Mechanically Stabilized Earth (MSE) walls
- Earth backfilling
- Prefabricated modular or rigid gravity walls
- Reinforced concrete cantilever walls
- Soil nailing
- Partially embedded soldier piles
- Continuous sheet pile walls.

These retaining wall types were summarized by advantages and disadvantages then ranked in a decision table based on four criteria: applicability to the project, cost, ease of construction and construction time. Based on the decision table, two options (soldier pile and soil nail retaining wall) were selected for the Duckworth Street retaining wall as detailed in Section 3.0.

A preliminary design and cost estimate for both designs were completed to determine if one option was more cost efficient. Sections 5.1.2 and 5.2.2 outline the cost estimates based on preliminary design of each option.

Upon the completion of the preliminary design, detailed design and drafting was completed for the soldier pile wall (selected design). The cost estimate was then completed using data from RSMeans, local supply stores and unit prices provided by the client. At the request of the client, Acuren, QCEC also developed a construction plan and schedule. The construction plan is outlined in Section 8.0 and includes the required actions to construct the soldier pile retaining wall as well as a schedule (Appendix K) that details approximate time required to complete each task.

3.0 LITERATURE REVIEW

Potential solutions for the temporary Duckworth Street retaining wall are outlined below in Table 3.1.

Table 3.1: Wall Type Comparison

Wall Type	Advantages	Disadvantages
Mechanically Stabilized Earth (MSE) Walls	• Easy to install • Quick construction time • Low labour costs	• Height limited to 8.5m • Wide base (~70% of retaining wall height)
Earth Backfilling	• Low cost (only equipment and labour costs since method uses existing soil) • Easy to construct	• Not suitable for high walls and large loads/pressure • Wide base
Prefabricated modular or rigid gravity walls	• Can be used for depths >8.5m • Reduced construction time due to prefabrication • Potentially high cost	• Wide base (~50-70% of the retaining wall height) • Costly if not prefabricated with concrete or timber cribs/bins • Can be difficult to construct depending on the design
Reinforced Concrete Cantilever Wall	• Can be used for depths >8.5m • Smaller than a concrete gravity wall	• Economical up to ~7.5m • Wide base (~50-60% of retaining wall height)
Soil Nailing	• Economical alternative to traditional retaining walls • Can be used for narrow spaces	• Not applicable for loose soil.
Partially Embedded Soldier Piles	• Narrow base • Less labour required • Lower labour costs • Can be used for building pits and areas with underground facilities	• Increased costs if concrete or steel panel lagging is used • A tie back system may be required to support increased heights and loading
Continuous Sheet Pile Walls	• Narrow base	• Ideal height <6m • A tie back system may be required to support increased heights (up to ~24m)

Based on the advantages and disadvantages outlined above, it was evident that some options will not be feasible for the location. The limited space for the retaining wall due to the construction of the parking garage and condominium is a major factor in choosing a retaining wall. The critical height to be used in the design of the retaining wall is approximately 8.5 meters. Therefore, options with a wide base (50%-70% of wall height) such as MSE walls, gravity walls and reinforced concrete cantilever walls will not be applicable. Also, the earth backfilling would not be suitable for the height required of the retaining wall. However, a decision table based on weighted factors was developed to determine the best option for the site's criteria (Table 3.2).

Table 3.2: Retaining Wall Decision Table

Wall Type	Applicability	Cost	Ease of Construction	Construction Time	Total	Rank
Weighting Factor	70%	15%	10%	5%	**100%**	-
Mechanically Stabilized Earth (MSE) Walls	3	2	2	2	**2.7**	5
Earth Backfilling	3	1	2	2	**2.6**	4
Prefabricated modular or rigid gravity walls	3	2	2	2	**2.7**	5
Reinforced Concrete Cantilever Wall	3	3	2	3	**2.9**	7
Soil Nailing	1	2	2	2	**1.3**	2
Partially Embedded Soldier Piles	1	1	2	2	**1.2**	1
Continuous Sheet Pile Walls	2	2	3	2	**2.1**	3

Applicability (based on height & space)	**Cost**	**Ease of Construction**	**Construction Time**
1-Most Applicable	1-Low	1-Easy	1-Relatively Short
2-Possibly Applicable	2-Medium	2-Easy to Difficult	2-Short to Long
3-Least Applicable	3-High	3-Difficult	3-Relatively Long

QCEC determined four factors to be considered when choosing the retaining wall: applicability, cost, ease of construction and construction time. These factors were weighted based on QCEC's understanding of the client's needs with regards to the chosen retaining wall. The retaining wall's applicability to the site conditions (e.g. limited space for the base of the wall) was weighted with a factor of 70%. This factor was particularly high because if the base of the retaining wall is not narrow enough it cannot be considered an option. Cost was given a factor of 15% because if two options were considered equal in all regards the most cost efficient option would be chosen. Ease of construction and construction time were ranked the lowest with 10% and 5% respectively. Ease of construction was ranked slightly higher because the retaining wall is in a residential area, therefore minimal interruptions and noise due to construction methods would be ideal.

Based on the results of Table 3.2, a soldier pile wall would be the best option for the Duckworth Street temporary retaining wall and soil nailing could be a second possible alternative.

4.0 SITE CONDITIONS

In 2011, exp Services Inc. conducted a geotechnical subsurface investigation including a total 13 test pits and seven boreholes. Four boreholes (BH02, BH04, BH05 and BH06) and three test pits (TP08, TP12 and TP13) were located in the vicinity of the proposed retaining wall (Figure 03 of Appendix C). The report detailing the investigation was provided to QCEC by Acuren Group Inc. for reference throughout the design of the retaining wall (Appendix A).

QCEC analyzed the borehole and test pit records to create a soil profile for the benefit of the design process (Appendix C). Four cross sections were developed by linear interpolation of these records as shown in Figures 01 to 04 of Appendix C. The soil on site consists mainly of granular fill and till which extends from 0.3 to 8.41 meters below ground surface. The report describes the fill as greyish brown or dark grey to black gravelly sand with some silt and occasional cobbles and boulders. The till is described as brownish-grey to grey gravelly sand to a sand and gravel with traces of some silt and occasional cobbles and boulders. Bedrock is classified as very severely fractured to fractured medium grey sandstone.

Based on the geotechnical analysis, exp Services Inc. recommended soil parameters for design which are outlined below in Table 4.1. These are the parameters used by QCEC throughout the design of the retaining wall.

Table 4.1: Recommended Geotechnical Parameters

From Table 2 of the Geotechnical Report (Appendix A) Proposed Parking Garage Duckworth Street at Bell Street St. John's, Newfoundland and Labrador	
Parameter	**Compacted Engineered Fill**
Total Unit Weight, KN/m3	20.5
Buoyant Unit Weight, KN/m3	10.5
Effective Friction Angle, degrees	36
Coefficient of Active Earth Pressure, K_a	0.26
Coefficient of Passive Earth Pressure, K_p	3.8
Coefficient of Earth Pressure at Rest, K_o	0.41

As requested by the client, a one meter space must be maintained between the retaining wall and the building construction line to ensure accessibility for construction. This request placed size restrictions on the type of retaining wall that can be implemented in the limited space. However, based on the retaining wall location in Figure 05 of Appendix C, the space between the retaining wall and construction line is 5.09 meters on the south end (soil height of 2.78 meters) and 4.4 meters on the north end (soil height of 7.33 meters).

In addition, soil will be removed for the construction of the retaining wall and the bedrock will be excavated in steps to provide a level construction surface (Figure 02 of Appendix D).

5.0 PRELIMINARY DESIGN

Based on the literature review and the site conditions, two potential options were explored by QCEC. The top two ranked options in Table 3.2 were soldier pile wall and soil nailing wall. QCEC developed a preliminary design and cost estimate of each option to determine which option would be more efficient for the Duckworth Street Retaining Wall.

5.1 SOLDIER PILE RETAINING WALL

5.1.1 DESIGN

The preliminary design of the soldier pile retaining wall includes HP piles and timber lagging along the 65 meter length of the proposed retaining wall location. Typical values were found through literature review and communication with the client and used to complete the preliminary design . The piles are spaced at 3 meters, therefore, the number of piles required was determined by dividing the length of the wall by the spacing plus an additional pile at the end. The number of steel piles required using this design method is 22. The timber lagging placed between the steel piles have an assumed typical dimension 100x250 millimeters. The amount of timber required for the design was determined based on an assumed retaining wall height of 7 meters. The holes for placing the piles are drilled and and based on a typical diameter of 550 mm (steel casing) and will need to be done for each pile (assumed embedment depth of 2 meters). Tiebacks are also required if the wall exceeds a height of 4.5m [1], therefore, the preliminary design assumes that at least one tieback will be required for each pile. Tiebacks include steel rods, grouting and drilling which has also been considered in the preliminary design. Typical assumed lengths for grouting are 4.5 meters with a diameter of 168 millimeters (Appendix G) therefore QCEC assumed a total tieback length of 8 meter for each pile.

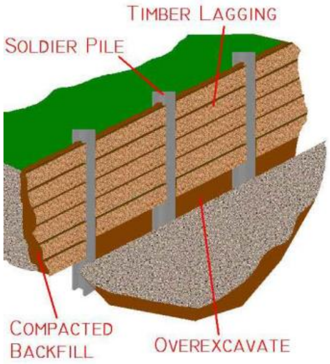

Figure 5.2: Soldier Pile Retaining Wall

5.1.2 COST

The soldier pile wall has typical components as determined in the preliminary design that must be considered for the cost estimate. The steel sections, timber, labour and equipment are the main factors to be considered for the initial cost estimate. Based on the preliminary drawings and design for the soldier pile retaining wall, 22 pile are required. The piles are placed by drilling holes into the ground, therefore equipment and labour is taken into account for this as well. The client has provided QCEC with some typical cost values that can be used for steel (kg), concrete (m^3), drilling in soil and bedrock (m) and grout (m^3), which include the materials, equipment, labour (Table 5.1). The holes are based on a 550 millimeter diameter for steel casing and the cost is determined using lineal meters of drilling required for both soil and bedrock. The concrete is required for the embedment of the piles and a volume can be determined based on the hole diameter and assumed embedment depth. Considering the height of the wall, QCEC implemented an estimate of one tieback per soldier pile. The tieback cost includes the anchor (steel – kg), the grouting (m^3) and the drilling (m).

Table 5.1 - Unit Prices

Item	Unit	Unit Price
Excavation (Soil)	m3	$15.70
Excavation (Rock)	m3	$39.25
Steel	m3	$3.50
Concrete	m3	$650.00
Grout	m3	$400.00
Drilling (Soil)	m	$180.00
Drilling (Rock)	m	$550.00

The timber that is placed between each steel section is 250x100 millimeters and 3 meters long. Based on these timber dimensions and an average wall height of approximately 7 meters, QCEC calculated the quantity of timber required for the design. The cost of timber and labour required (two labourers) can be calculated with data from RSMeans.

The preliminary cost estimate for the soldier pile retaining wall is outlined below in Table 5.2.

Table 5.2: Preliminary Cost Estimate for Soldier Pile Retaining Wall

SOLDIER PILE WALL				
ITEM	**QUANTITY**	**UNIT**	**UNIT PRICE**	**TOTAL**
PILES:				
STEEL	24200	kg	$3.50	$84,700.00
CONCRETE @ BASE	10.454	m3	$650.00	$6,794.87
DRILLING (SOIL) (550mm diam)	176	m	$180.00	$31,680.00
DRILLING (ROCK) (550mm diam)	44	m	$550.00	$24,200.00
TIMBER LAGGING (RS Means)	588.9	ea	N/A	$27,326.87
TIE-BACKS:				
RODS	1126.4	kg	$3.50	$3,942.40
GROUT	2.1945	m3	$400.00	$877.82
DRILLING (168mm diam)	176	m	$65.45	$11,520.00
TOTAL:				**$191,041.96**

5.2 SOIL NAIL RETAINING WALL

Soil nail retaining walls consist of soil nails installed and grouted into the soil then sprayed with shotcrete and connected to a concrete face (cast-in-place) along the soil to be retained [2]. In Figure 5.2-a below, two soil nails are already installed and sprayed with shotcrete and a third soil nail is in the process of installation. In Figure 5.2-b, the layer of shotcrete is sprayed on the third soil nail. Figure 5.2-c, all three soil nails are installed with their layer of shotcrete.

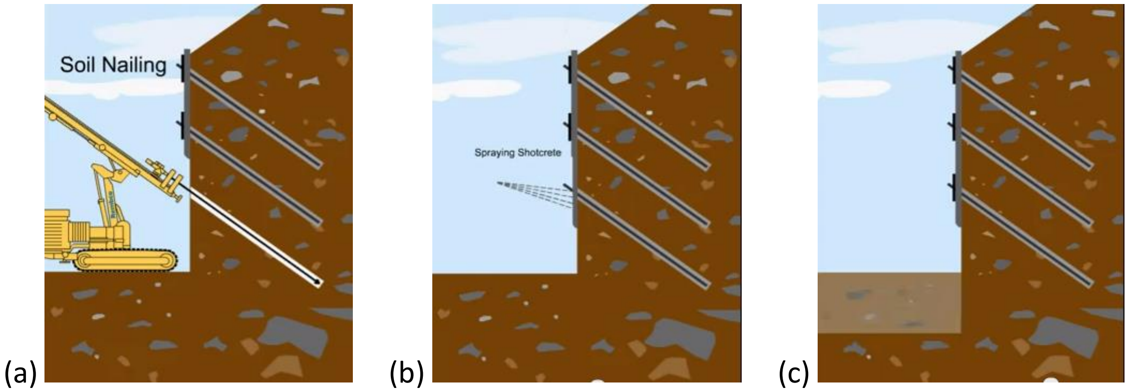

Figure 5.2: Soil Nail Retaining Wall [3]

The soil nails are composed of steel bars that are drilled and grouted at a specified angle to the horizontal [2]. These retaining walls require a "prequalified Anchored Wall Contractor" for construction which is unavailable in Newfoundland, therefore a contractor would have to relocate to Newfoundland for the duration of construction.

5.2.1 DESIGN

The preliminary design is based on standard design dimensions [2]. Shotcrete should be a minimum of 4 inches thick and the concrete facing should be a minimum of 8 inches thick. The concrete face should also extend 6 inches above the soil grade. Drain strips are also placed a maximum of 10 feet horizontally [2]. Typically, soil nails are installed with a drill hole diameter of 6 to 8 inches at an angle of 20 degrees to the horizontal. Preliminary drawings for the soil nail retaining wall are in Appendix E.

5.2.2 COST

Soil nail retaining wall quantity take-offs include the soil nails (anchors), shipping of the anchors, grouting, shotcrete and steel mesh. The cost of these items were provided by the client based on typical values used for cost estimates (Table 5.1). The preliminary design includes approximately 149 nails which is the main factor when completing the cost estimate. However, the material for the soil nails must be shipped from out of province. The exact location is unclear but QCEC assumed Toronto, Ontario for the preliminary estimate. RSMeans supplies a cost per kilometer for shipping; therefore using the travel distance from Toronto to St. John's (3075 kilometers) QCEC determined an approximate shipping cost.

The preliminary cost estimate for the soil nail retaining wall is outlined in Table 5.3 below.

Table 5.3: Preliminary Cost Estimate for Soil Nail Retaining Wall

SOIL NAILING				
ITEM	**QUANTITY**	**UNIT**	**UNIT PRICE**	**TOTAL**
Nails				
STEEL	5523.90	kg	$3.50	$19,333.66
SHIPPING (RS Means)	3075	km	$1.26	$3,874.50
GROUT	28.0044	m3	$400.00	$11,201.75
DRILLING	863.11	m	$65.45	$56,494.46
STEEL MESH/Concrete	332.3	m2	$430.60	$143,071.84
TOTAL:				**$233,976.21**

In addition, due to the fact that a contractor experienced with soil nail retaining walls is not located in Newfoundland, additional costs would incur to relocate a contractor for the duration of the project. These costs were not calculated because the soil nail retaining wall (Table 5.3) is already considerably more expensive than the soldier pile retaining wall option (Table 5.2). For that reason, QCEC elected to design a soldier pile retaining wall for the construction of the Duckworth Street parking garage and condominium complex.

6.0 GENERAL DESIGN AND SKETCHING

6.1 GENERAL

Based on the preliminary design and cost estimate for the soil nail and soldier pile retaining wall, the soldier pile wall is the most feasible and economical option.

However, as discussed in Section 4.0, the south end of the proposed retaining wall location has an available space of approximately 5 meters to the construction line. In this area it is not necessary to have a retaining wall with a narrow base. Therefore, combining a rigid gravity retaining wall with a soldier pile wall is a cost saving alternative. Rigid gravity walls are constructed of unreinforced concrete which is readily available in Newfoundland. Conversely, steel H-pile sections are uncommon in Newfoundland and must be delivered from the Nova Scotia. This adds additional shipping expenses (included in unit prices in Table 5.1) which will be reduced by implementing a concrete rigid gravity retaining wall.

6.2 RIGID GRAVITY RETAINING WALL

Rigid gravity walls are suitable to retain up to 8.5 meters of soil with a base width of approximately 60-70% of the wall height. Although its large dimensions make it an unsuitable choice for the entire retaining wall due to limited available space, the south end of the retaining wall has approximately 5 meters available to the construction line and 3 meters of soil to retain. This available space makes the rigid gravity wall a good alternative to the soldier pile wall due to its lower cost. Therefore, QCEC proposed to construct a 15 meter long rigid gravity wall starting from the south end of the retaining wall location.

As shown in Figure 6.1-a below, the final design of the rigid gravity wall includes a width of 0.5 meters at the top, 2 meters at the bottom, and height of 4 meters. The retaining wall is designed to run parallel to the construction line of the proposed parking garage and maintain a distance of 1 meter to the construction line.

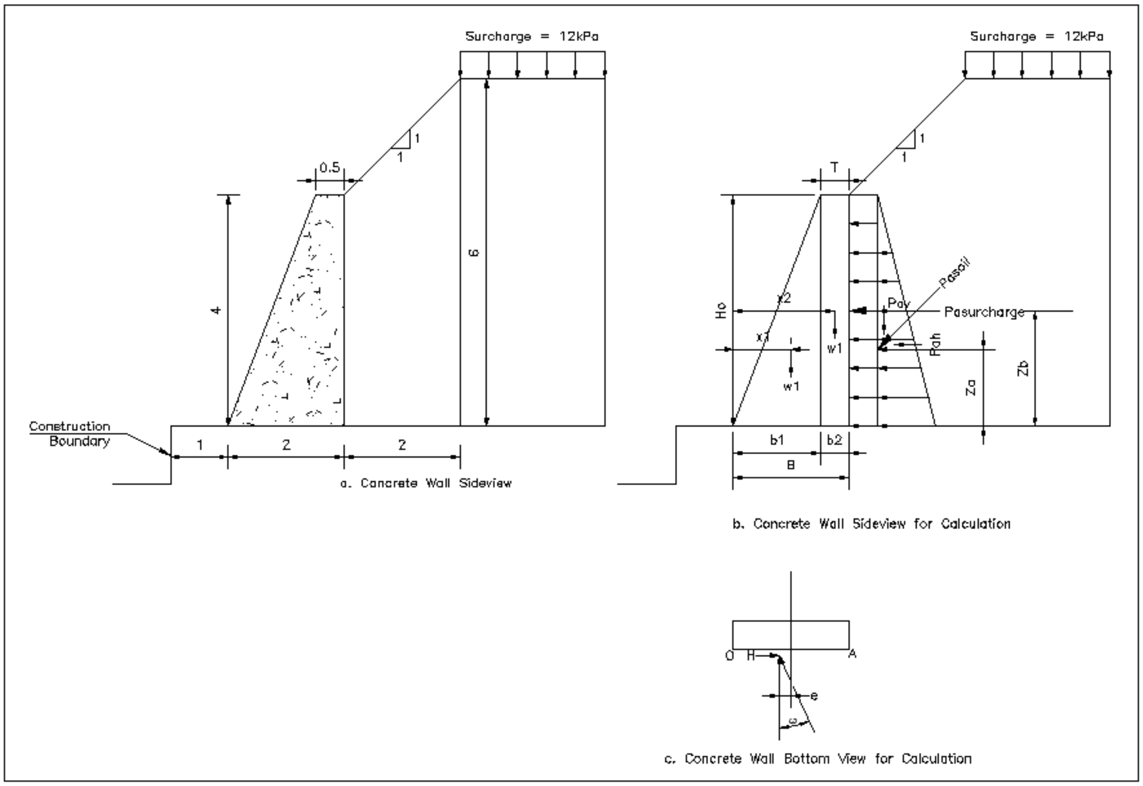

Figure 6.1 - Rigid Gravity Retaining Wall

Due to the excavation of the bedrock (Figure 02 of Appendix D), the design height for the rigid gravity wall is 6 meters. The retaining wall is designed for a strength of 24 MPa cast-in-place concrete. Other assumptions for the rigid gravity wall design include:

- Natural backfill (excavated soil is used as backfill)
- Backfill slope of 1:1
- Groundwater level is below the base of the rigid gravity retaining wall
- Wall friction is zero
- Effective friction angle, ϕ', is used as both the critical state friction angle ϕ'_{cs} and peak friction angle ϕ'_p.
- 12 KPa design surcharge load

6.2.1 LOADING

The rigid gravity wall must be designed to resist three types of failure: translational failure, rotational failure, and bearing failure. The initial step is to determine the lateral forces applied on the wall by the retained soil. The active soil pressure from the retained soil is calculated by using Rankine's Method:

$$P_{asoil} = \frac{1}{2} K_a \gamma {H_o}^2 \qquad \textbf{(Equation 6.2-1)}$$

where:

K_a = the active earth pressure coefficient

γ = total unit weight of the soil (kN/m^3)

H_o = height of the wall

The active pressure created by the design surcharge load must also be calculated using:

$$P_{asurcharge} = K_a q H_o \qquad \textbf{(Equation 6.2-2)}$$

Due to the sloped backfill, the concentrated active load caused by the retained soil is acts at one third of the wall height at the same angle as the backfill slope (1:1). The concentrated active load caused by the surcharge acts horizontally at one half of the wall height. In an effort to simplify the calculations, the active loads applied on the retaining wall can be transformed into horizontal and vertical components with the following equations:

$$P_{ah} = P_{asoil} \times \frac{b}{c} + P_{asurcharge} \qquad \textbf{(Equation 6.2-3)}$$

$$P_{av} = P_{asoil} \times \frac{a}{c} \qquad \textbf{(Equation 6.2-4)}$$

where:

a = 1 unit meter horizontal distance of backfill

b = 1 unit meter vertical distance of backfill

c = inclined distance of backfill for 1:1 slope

6.3 FAILURE MODES

The design of the gravity retaining wall should resist all three modes of failure: translational failure, rotational failure and bearing failure.

6.3.1 TRANSLATIONAL FAILURE

For the retaining wall to resist translational failure, the base should sufficiently resist the lateral forces applied to the wall by the retained soil. The typical factor of safety used for translation is 1.5.

In an effort to simplify calculations, QCEC divided the retaining wall into a triangular and rectangular shape based on its geometry (Figure 6.1-b). The weight of each components are calculated using:

$$W_1 = \frac{1}{2} \times b_1 \times H_o \times \gamma_c$$ **(Equation 6.3-1)**

$$W_2 = b_2 \times H_o \times \gamma_c$$ **(Equation 6.3-2)**

$$W = W_1 + W_2$$ **(Equation 6.3-3)**

where:

b_1, b_2 = Dimension of each component

H_o = Height of the retaining wall

γ_c = Unit weight of concrete

W = Total weight of retaining wall

The total moment on the retaining wall is calculated from Equation 6.3-4 below using dimensions shown in Figure 6.1-b:

$$M_o = W_1 \times x_1 + W_2 \times x_2 + P_{av} \times B - \left(P_{ah} - P_{asurcharge}\right) \times Z_a - P_{asurcharge} \times Z_b$$

The location of the vertical resultant force at the base of the retaining wall can be calculated with:

$$\bar{x} = \frac{M_O}{R_Z} \qquad \textbf{(Equation 6.3-5)}$$

where:

M_o = Total moment on retaining wall

R_z = Resultant vertical force ($W+P_{av}$)

The base resistance can then be calculated:

$$T = R_z \times \tan(\phi'_b) \qquad \textbf{(Equation 6.3-6)}$$

where:

R_z = Vertical resultant force

ϕ'_b = Base resistance factor

The factor of safety for translation is determined by:

$$(FS)_T = \frac{T}{P_{ah}} \qquad \textbf{(Equation 6.3-7)}$$

where:

T = Horizontal resistance of the base

P_{ah} = Horizontal force from active soil pressure

As shown in Appendix I, the final design has a factor of safety against translation of 1.568, which is greater than the minimum value of 1.5. Therefore, the design is safe against translational failure.

6.3.2 ROTATIONAL FAILURE

The rigid gravity wall must also resist rotational failure. If the vertical resultant force, R_z ,is located between one third and two thirds of the base width, the wall is considered safe. The eccentricity of R_z is calculated with:

$$e = \left|\frac{B}{2} - \bar{x}\right| \quad \textbf{(Equation 6.3-8)}$$

where:

B = Width of the base

$\bar{x}$ = Location of vertical resultant force

If the eccentricity is less than $B/6$ (0.333 meters), the wall is considered safe against rotation. The final design has an eccentricity of 0.007 meters, therefore the design is safe against rotational failure.

6.3.3 BEARING FAILURE

Finally, the rigid gravity wall must also resist bearing failure which means the allowable soil bearing capacity should be greater than the maximum pressure caused by the gravity wall on the soil. Since the wall is safe against rotational failure, no tensile forces will develop in the soil. The maximum stress occurs at point A in Figure 6.1-c and can be calculated from:

$$\sigma_{max} = \frac{R_z}{A} \times (1 + 6 \times \frac{e}{B}) \quad \textbf{(Equation 6.3-9)}$$

where:

R_z = Vertical resultant force

A = Area of base

e = Eccentricity

B = Width of the base

The vertical resultant force, Rz, is inclined to the vertical and eccentric (Figure 6.1-c). Therefore, a bearing capacity equation considering the inclined load should be used:

$$\omega = \tan^{-1}\frac{H}{V_n}$$ **(Equation 6.3-10)**

where:

$H = P_a$

$V_n = R_z$

The base of the retaining wall can be treated as a strip footing; therefore the B/L is approaching zero. The allowable soil capacity can be calculated using the following equation:

$$q_u = 0.5\gamma B' N_\gamma i_\gamma$$ **(Equation 6.3-11)**

where:

$B' = B - 2e$

$N_\gamma = 0.1054e^{(9.6\phi'_p)}$

$i_\gamma = (1 - \frac{H}{V_n})$

The factor of safety against bearing can be calculated by:

$$(FS)_B = \frac{q_u}{\sigma_{max}}$$ **(Equation 6.3-12)**

Typically, a factor of safety of 3 for bearing is considered safe. In the final design of the rigid gravity wall, a factor of safety of 4.29 was obtained which shows that the design is sufficient against bearing failure (Appendix I).

6.4 DRAINAGE

Precipitation may increase the groundwater level and subsequently increase the water content of the backfilled soil. This can decrease the stability of the retaining wall and cause it to fail. Weep holes have been implemented into the design to provide drainage for excess water. As shown in Figure 04 of Appendix D, weep holes with a 75 millimeter diameter were used in the concrete retaining wall with a horizontal and vertical spacing of 1.5 meters.

6.5 SOLDIER PILE RETAINING WALL

Prior to detailed design of the soldier pile retaining wall, an excavation plan was developed for the reshaping of the bedrock in the location of the retaining wall (Figure 02 of Appendix B). Based on the drawing, three different retaining wall heights were determined based on each 'step.' The critical section (8.41 meters) is located on the lowest step and will have a retaining wall height of 8.5 meters. Other sections of the soldier pile retaining wall will have a height of 7.5 meters or 6.5 meters based on the excavation plan.

In general, soldier pile walls exceeding 4.5 meters in height require the use of tiebacks [1]. When the soldier pile wall heights were finalized, it was evident that at least one or two tiebacks would be required at each height. In an effort to determine the number of tiebacks required, three retaining wall models were developed: cantilever soldier piles, soldier piles with one tieback and soldier piles with two tiebacks.

6.5.1 TIEBACK LAYOUT

Typically tiebacks are installed at a slight angle (15°) from the horizontal [1] to limit additional axial forces on the soldier piles. However, for the Duckworth Street retaining wall, underground obstructions had to be considered when designing the tiebacks. According to the client, underground sewer facilities are located approximately 2 meters below the surface of Henry Street along the length of the retaining wall location. For this reason, the first tiebacks are located 2 meters below the top of the soldier piles. This placement combined with the angle of the tiebacks proves sufficient for avoiding the underground sewer facilities. Based on suggested guidelines, the second anchor will be installed 3 meters below the first anchor [4]. In addition, residential housing is located across Henry Street, therefore the horizontal distance of the tiebacks cannot exceed the width of the street (approximately 8.5 meters) to avoid conflicts with housing. Subsequent to an evaluation of several tieback angles, an angle of 30° to the horizontal was selected to avoid the housing across Henry Street. The final layout of the soldier pile and tieback system is shown in Figure 6.2 below.

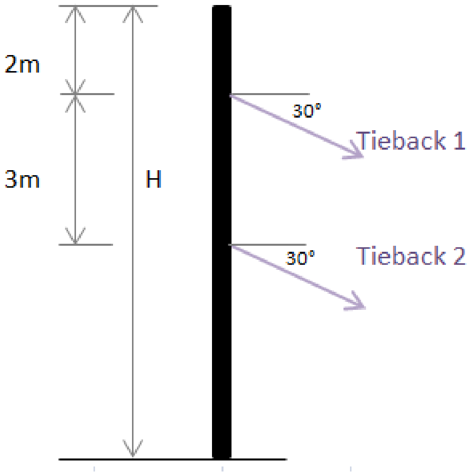

Figure 6.2: Layout of the Selected Tieback System

6.5.2 LOADING

The retaining wall was designed to resist the lateral earth pressure of the soil and a uniform surcharge load (q) applied along the surface of the retained soil (Figure 6.3). In the case of the Duckworth Street retaining wall, it is assumed that the water table is not encountered and sufficient drainage is provided in the design, therefore water pressure is neglected.

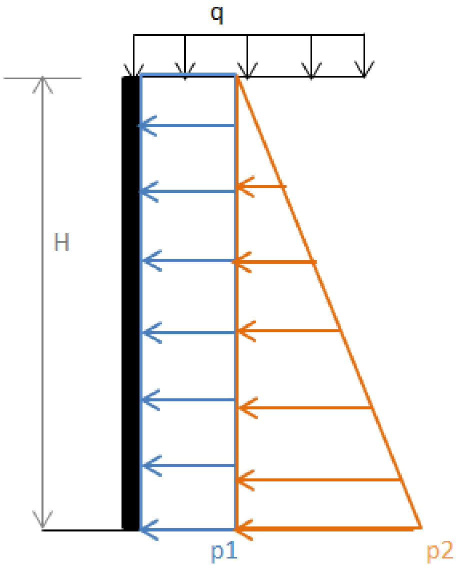

Figure 6.3: Lateral Earth Pressures for Retaining Wall Design

The active earth pressure (p2) acts along the height of the wall:

$$p2 \ = K_a \gamma H$$ **(Equation 6.5-1)**

where:

K_a = the active earth pressure coefficient

γ = Total unit weight of the soil (kN/m^3)

H = Height of the wall

The surcharge load creates a uniform load along the length of the retaining wall (p1) which is calculated with Equation 6.3-2 shown below. Typically for design purposes, the surcharge load is taken as 12kPa [5].

$$p1 = K_a q$$ **(Equation 6.5-2)**

where:

K_a = Active earth pressure coefficient

q = Uniform surcharge load (12kPa)

6.5.3 SOLDIER PILE ANALYSIS

Soldier piles are usually H-pile sections due to their equivalent dimensions and typically spaced at 2-3 meters. In an effort to reduce the number of piles required along the length of the wall, QCEC utilized a spacing of 3 meters in the design of the soldier pile retaining wall. S-FRAME models were developed for the three soldier pile design cases at 8.5 meters, 7.5 meters, and 6.5 meters. The pile loads were determined by multiplying p1 and p2 (Figure 6.3) by the tributary area (3 meters) and can be reviewed in Appendix F.

When the models were run through S-STEEL, the program suggested a HP310x79 section for the 8.5 meter soldier piles with two tiebacks. However, in the interest of safety, QCEC chose to use the slightly larger section of HP310x110 for the soldier piles. This section was evaluated using S-STEEL for all S-FRAME models and a summary of its results are detailed below in Table 6.1. Based on the section checks from the S-STEEL analysis, two tiebacks are required for the 8.5 meter soldier piles and one tieback could potentially be sufficient for the 7.5 and 6.5 meter piles.

Table 6.1: HP310X110 Section Check Summary Table

Wall Type	N_f (KN)	V_f (KN)	M_f (KN)	Code Check Results	Comment
8.5 m Cantilever	0	657	1975	> 1.0	Not OK
8.5 m Anchored -1 Tieback	132	429	490	> 1.0	Not OK
8.5 m Anchored -2 Tiebacks	238	246	155	0.575	OK
7.5 m Cantilever	0	520	1388	> 1.0	Not OK
7.5 m Anchored -1 Tieback	113	325	313	0.726	OK
7.5 m Anchored -2 Tiebacks	209	159	78	0.507	OK
6.5 m Cantilever	0	399	930	> 1.0	Not OK
6.5 m Anchored -1 Tieback	96	233	183	0.44	OK
6.5 m Anchored -2 Tiebacks	181	107	40	0.44	OK

Each wall height has a maximum allowable deflection of 0.005H based on guidelines outlined by the US Army Corps of Engineers. Table 6.2 below summarizes the maximum deflection of each soldier pile model and the maximum allowable deflection for comparison. The implementation of tiebacks reduces the maximum deflections at each height to an acceptable value.

Table 6.2: Deflection Check Summary Table

Wall Type	Δ_{max} (mm)	Δ_{Allow} (mm)	Comment
8.5 m Cantilever	633.19	42.5	Not OK
8.5 m Anchored -1 Tieback	18.38	42.5	OK
8.5 m Anchored -2 Tiebacks	2.19	42.5	OK
7.5 m Cantilever	348.74	37.5	Not OK
7.5 m Anchored -1 Tieback	8.73	37.5	OK
7.5 m Anchored -2 Tiebacks	1.27	37.5	OK
6.5 m Cantilever	177.08	32.5	Not OK
6.5 m Anchored -1 Tieback	3.62	32.5	OK
6.5 m Anchored -2 Tiebacks	1.27	32.5	OK

The embedment depth of the soldier piles is based on the shear force at the base of the critical section (8.5 meter height). As shown in Table 6.1 above, the shear force for the 8.5 meter pile with two tiebacks is 246 kN. In an effort to reduce the required embedment depth, the shear force at the base of the 7.5 and 6.5 meter piles must be less than 246 kN. Therefore, one tieback for the 7.5 meter soldier piles will not be sufficient (325 kN) and for that reason, two tiebacks have been implemented. However, one tieback is sufficient for the 6.5 meter soldier piles (233kN).

The required depth of the soldier pile is calculated based on the Wang-Reese equations for ultimate passive resistance of cohesionless soils [6]. The passive force, F_p, resists the shear force that is created at the base of the wall (Figure 6.4).

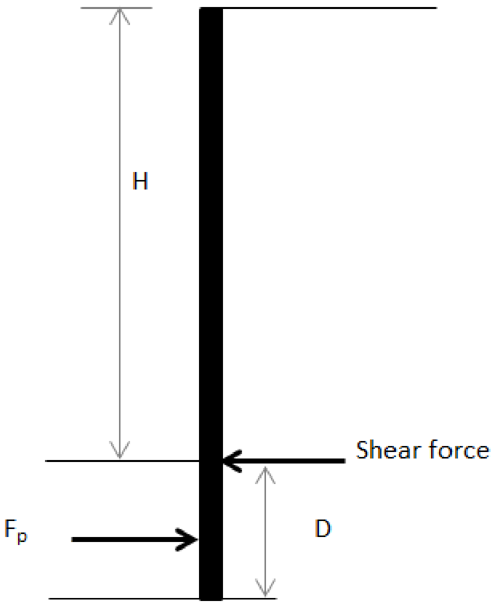

Figure 6.4 - Passive Force and Shear Force on Soldier Pile

F_p is calculated from **Equation 6.5-3** [6] below:

$$F_p = \gamma d^2 \left[\frac{K_0 tan\varphi' sin\beta}{\tan(\beta - \varphi')\, cos\alpha} + \frac{tan\beta}{\tan(\beta - \varphi')} \left(\frac{b}{2} + \frac{d}{3} tan\beta \; tan \propto \right) + \frac{K_0 d \; tan\beta}{3} (tan\; \varphi' sin\beta - tan\alpha) \right]$$

Where:

γ = total unit weight of the soil (kN/m^3)

d = embedment depth

K_0 = at-rest pressure coefficient

ϕ' = drained friction angle of soil

β = 45 + ϕ'/2

α = ϕ' (dense sands)

b = diameter of soldier pile

After the embedment depth is determined from Equation 6.5-3 it is multiplied by a factor of safety of 1.5. This results in a safe embedment depth of 3.0 meters which is implemented for all soldier piles.

6.5.4 TIEBACK DESIGN

Tiebacks have to be designed to sustain the axial forces of the tie members from the S-FRAME analysis (Table 6.3).

Table 6.3: Axial Tensile Forces in Tiebacks

Wall Height (m)	Tieback	Axial Tensile Force (kN)
8.5	Upper Tieback	146
	Lower Tieback	330
7.5	Upper Tieback	157
	Lower Tieback	261
6.5	Upper Tieback	192

Based on the S-FRAME results, the tiebacks must be designed for 330 kN. The capacities of various tie rod diameters were established from DYWIDAG product information [7]. A diameter of 32 millimeters will provide a yield load of 402 kN which is sufficient for the maximum tensile force in the tiebacks. The tiebacks are also designed in two other portions: the unbonded length and the bonded length.

The unbonded length must extend a distance of H/5 or 1.5 meters beyond the soil's failure plane [6]. The failure plane is located at an angle of $45° - \frac{\varphi}{2}$ from the base of the retaining wall (Figure 6.5).

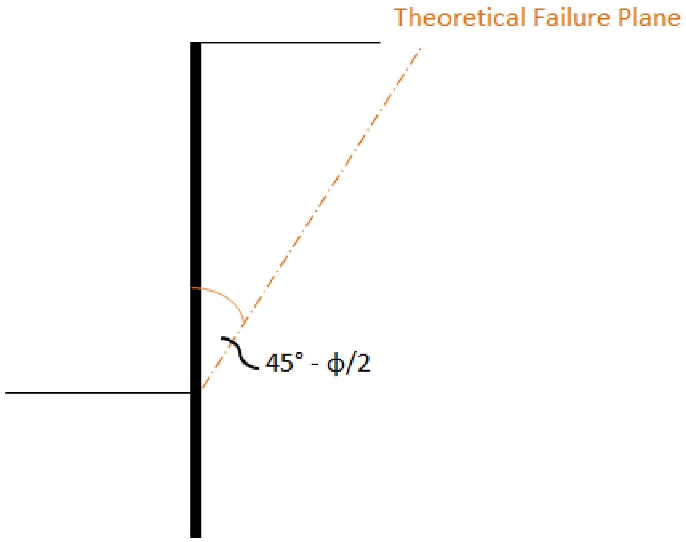

Figure 6.5: Theoretical Failure Plane

The interception points for the tiebacks and the theoretical failure plane were determined by plotting the failure plane and both tiebacks for each wall height (Appendix D). The top tieback of the 8.5 meter section governs the unbonded length design with a length of 4.5 meters.

The bonded length of the tiebacks is determined from the Design Anchor Capacity Estimates table provided to QCEC by the client (Appendix G). Based on the geotechnical report, QCEC determined the anchors would be bonded in either severely fractured bedrock or fractured sandstone bedrock. The provided table shows that severely fractured bedrock provides lower anchor capacities. Therefore, bond length was determined based on the criteria provided for the anchors bonded in severely fractured bedrock. Based on the load tested capacities, a bond length of 4.5 meters will provide 475 kN which will be sufficient to resist the maximum tensile force in the anchors.

The final tieback design includes a 32 millimeter diameter rod and a total length of 9 meters with a bonded length of 4.5 meters.

6.5.5 TIMBER LAGGING DESIGN

The lagging used for the soldier pile wall is untreated timber with a service life of three years. Since the retaining wall is temporary, all timber lagging will be untreated. The initial design of the timber lagging is based on the recommended thicknesses from the US Army Corps of Engineers [6].

The soil is considered to be a 'competent' soil with a clear span of 3 meters and a critical height of 8.5 meters. Based on these criteria, a thickness of 100 millimeters is recommended up to approximately 7.5 meters and a 125 millimeter thickness for timber deeper than 7.5 meters. The client informed QCEC that typical sizes of timber available have dimensions of 250x75 and 250x100 millimeters. Therefore, based on these recommendations, QCEC would suggest to use 250x100 millimeter sized timber for the retaining wall up to a depth of 7.5 meters then doubling the same sized timber for the deeper sections.

This design was verified by the wood design manual from the Canadian Wood Council. First, the factored load applied to the timber by the retained soil is calculated for different depths. The critical design depth used is 8.5 meters and the pressure is established every 0.5 meters. Assumptions relating to the calculations included:

- Untreated saw timber
- Members are intended to support permanent loads
- Wet service condition
- Timber is Spruce-Pine-Fir (S-P-F) Grade No.1
- Members are treated as simply support beams with uniform loading
- Timber is not notched
- No live load

The factored load which the soil applied to the timber can be calculated use equation:

$$w = K_a \times \gamma \times \left(H - \frac{b}{2}\right) + K_a \times q) \times b \times 0.6 \times 1.25$$ **(Equation 6.5-4)**

where:

w = Normally distributed load on the timber lagging

K_a = Active earth pressure coefficient

γ = Total unit weight of soil

H = Depth measured from the ground surface

b = Width of the timber lagging

q = Uniform surcharge load (12KPa)

0.6 = Factor used to compensate for soil arching behind lagging

1.25 = Dead load safety factor

The maximum moment is then calculated:

$$M = \frac{w \times l^2}{8}$$ **(Equation 6.5-5)**

where:

w = Normally distributed load on the timber lagging

l = clear span of the timber lagging

The section modulus of each section is calculated:

$$S = \frac{b \times d^2}{6}$$ **(Equation 6.5-6)**

where:

b = Width of the timber lagging

d = Thickness of the timber lagging

The bending stress is calculated:

$$\sigma = \frac{M}{S}$$ **(Equation 6.5-7)**

where:

M - Maximum bending moment

S - Section modulus

Based on the Wood Design Manual, 2005, the bending strength at the extreme fibre of S-P-F Grade No. 1 is 11.0 MPa. This number is reduced by a factor of 0.77 because the load is applied to the wider face of the timber. The calculated bending stress can then be compared to the bending strength. If the bending stress is less than the bending strength then the time section is considered adequate.

Based on the calculations (Appendix H), QCEC recommends 250x100 mm S-P-F Grade No. 1 timber for the first 2.5 meters then double the 250x100 mm timber for the remaining depth (i.e. 250x200mm).

This final timber lagging design differs from the initial design outlined by the US Army Corps of engineers because S-P-F is a relatively weaker timber species.

In addition, the timber members are spaced 25 millimeters to allow for drainage.

6.6 DRAFTING

There are three sets of AutoCAD drawings prepared for Duckworth Retaining Wall: Soil Profile, Construction Drawings for Soldier Pile and Rigid Gravity Retaining Wall, and Preliminary Drawings for Soil Nailing Retaining Wall. These drawings are outlined in Table 6.4 below.

Table 6.4: List of Drawings

Appendix	Drawing Number	Sheet	Description
C	Figure 01	S01	Borehole and test pit locations relevant to developing the soil profile
	Figure 02	S02	Locations of four cross sections
	Figure 03	S03	Cross section 1-1'
	Figure 04	S04	Cross sections 2-2', 3-3' and 4-4'
	Figure 05	S05	Proposed retaining wall location
D	Figure 01	A01	Layout of the retaining wall site
	Figure 02	A02	Stepped excavation profile for retaining wall
	Figure 03	A03	Plan and elevation view of soldier pile and rigid gravity retaining wall
	Figure 04	A04	Soldier pile and rigid gravity retaining wall details
E	Figure 01	A01	Plan and elevation view of soil nailing retaining wall
	Figure 02	A02	Side view of soil nailing retaining wall

The soil profile drawings in Appendix C are intended to show type and depths of soil in the retaining wall location. The geotechnical information in these drawings were linearly interpolated from the provided geotechnical report (Appendix A).

The construction drawings in Appendix D show the layout and detailed design of the soldier pile and rigid gravity (south end) retaining wall (Figure 03). Drawings include the plan and elevation view of the retaining wall and details of the timber and tieback design (Figure 04). The bedrock in the location of the retaining wall is sloped 12 degrees on the south side and 4 degrees on the north side of the critical depth (8.41 meters). The excavation profile (Figure 02) was developed in 0.5 to 1.0 meter steps to provide a level soil foundation for construction of the retaining wall.

Appendix E contains preliminary design drawings for the soil nailing retaining wall option. Drawings illustrate the plan, elevation and side view of the retaining wall. All dimensions used in this drawing are based on typical soil nailing designs.

7.0 COST ESTIMATE

7.1 QUANTITY TAKE-OFF

Based on the construction drawings for the rigid gravity and soldier pile retaining wall (Appendix D), QCEC performed a quantity take off for final cost estimate. Table 7.1 shows the various items that have been taken-off to complete the cost estimate. The quantity take-offs were initially completed by calculating the amount of each material required and then converted into a weight or volume based on dimensions. For example, steel was evaluated as a total mass in kilograms due to the provided unit prices (Table 5.1). Other quantities use simple calculations to determine the number of items required (ie. timber).

Table 7.1 – Quantity Take Off

Quantity Take-Off			
ITEM	**DESCRIPTION**	**QUANTITY**	**UNIT**
Steel	Hp Beams - 310x310 - 11 Each	13255	kg
Concrete	At pile embedment depths	7.84	m3
Steel Casings	Used for drilling into soil - 11 Each - Rented	44826	kg
Timber Lagging	Placed between piles	524	each
Timber Blocking	To brace timber between steel flange - 2x4x8 Lumber	221	pieces
Tie-Back Rods	19 Each	1094.4	kg
Grout	Used for grouted length of tie-backs	2.68	m3
L-angles	Used to support tie-backs at steel flange	4.57	kg
Misc Steel	Used to brace steel and timber	4.39	kg
Concrete	Concrete for rigid gravity wall	75	m3
Formwork	Plywood	134.08	m2
PVC Pipe	Used to form the weep holes in the rigid gravity wall - 3" diameter - approximately 1.5 m length	6	pieces

7.2 LABOUR AND MATERIALS

Table 7.2 - Cost Summary – Soldier Pile Wall

SOLDIER PILE WALL				
ITEM	**QUANTITY**	**UNIT**	**UNIT PRICE**	**TOTAL**
EXCAVATION (SOIL)	21741.00	m3	$15.70	$341,333.72
EXCAVATION (ROCK) (@concrete wall)	46.14	m3	$39.25	$1,811.00
PILES:				
STEEL	13255	kg	$3.50	$46,392.50
CONCRETE @ BASE	7.840	m3	$650.00	$5,096.15
DRILLING (SOIL) (550mm diam)	86.73	m	$180.00	$15,611.40
DRILLING (ROCK) (550mm diam)	39.25	m	$550.00	$21,587.50
CASING (Rented-10% of New Cost)	44826	kg	$3.50	$15,689.10
TIMBER LAGGING	523.5	ea	N/A	$31,266.20
BLOCKING (LUMBER)	221	pc	$2.79	$617.01
TIE-BACKS:				
RODS	1094.4	kg	$3.50	$3,830.40
GROUT	2.6861	m3	$400.00	$1,074.42
DRILLING (200mm diam)	171	m	$65.45	$11,192.73
L-ANGLE	4.5725	kg	$3.50	$16.00
MISC STEEL	4.3912	kg	$3.50	$15.37
CONCRETE WALL:				
CONCRETE	75.0	m3	$650.00	$48,750.00
FORMWORK	134.08	m2	N/A	$5,420.71
WEEP HOLES (PVC pipe)	6.00	ea	$13.49	$80.94
TOTAL				$549,785.15
10% extra-misc items				$54,978.52
TOTAL:				**$604,763.67**
TOTAL W/ MARK UP (10%)				**$665,240.03**

The cost estimate for the soldier pile retaining wall was calculated based on prices from RSMeans, local building suppliers (Kent Building Supplies), and typical unit prices supplied by the client. Items that used costs supplied by the client included:

- Excavation (both in soil and bedrock)
- Steel
- Drilling (both in soil and bedrock)
- Concrete/Grout

The prices for these items used standard costs that were provided by the client (Table 5.1), these prices include the material, equipment and labour. The prices are provided as per unit costs; therefore the quantities that are taken off are multiplied by per unit cost to give the total.

RSMeans was also used to complete the cost estimate of the timber lagging and formwork for the concrete rigid gravity wall. The timber lagging design requires the timber to double at a specific depth (greater than 2.5m), therefore each section would have a height varying timber thickness. The timber section used in the design is 250x100 millimeters and its cost was determined from RSMeans. The daily output, bare material cost, and crew from RSMeans were used to develop a cost for materials and labour. A crew of two labourers was used for the installation of the timber at a cost of \$283.60 per day per labourer. The quantity take-off for timber (Table 7.2) is multiplied by a bare material rate of \$451.20/m^3 from RSMeans. The per unit price for timber lagging is not shown in Table 7.2 as it includes both labour and materials and were calculated in another spreadsheet located in Appendix J.

The formwork for the rigid gravity wall is calculated using a price for plywood from RSMeans. The bare material price, \$22.38/m^2, is multiplied by the total area required to complete the formwork (Table 7.1). The labour used to complete this was taken as three labourers and one carpenter at a rate of \$283.60/labourer/day and \$359.20/carpenter/day. The time to complete this work should be a maximum of two days. The cost shown in Table 7.2 has been calculated in a separate spreadsheet as it includes the labour and material, similar to the timber lagging.

While most costs were found using typical per unit values or RSMeans, some smaller costs were found using local building supply stores. The blocking for the timber lagging and the weep holes located in the rigid gravity wall have are based on costs from Kent Building Supplies. The blocking for the timber requires four blocks (2x4 lumber) located between the steel flange and the timber lagging. The length required for all blocking was calculated and divided by 8 feet (length of lumber) then multiplied by the cost for each piece. The price found from a local supplier (Kent Building Supplies) was found at \$2.79/piece.

Similarly, the cost of the weep hole prices were determined by using PVC pipe as the material and a cost from Kent Building Supplies. Approximately 6 PVC pipes are required at a material cost of $13.49/piece and a total cost of $80.94.

After the cost estimate was completed, the final amount was increased by 10% to accommodate any additional costs (client recommended). In addition, the cost was increased by another 10% as a profit mark up.

8.0 CONSTRUCTION

8.1 METHODOLOGY

For the Duckworth Street Retaining Wall, QCEC designed a soldier pile wall with a 15 meter rigid gravity wall located on the south end (Figure 03 of Appendix D). The construction of the retaining wall is expected to be completed within 52 days (Appendix K).

The equipment for the soldier piles are first mobilized to site and set up to begin construction of the soldier pile wall. The equipment includes drilling equipment, a crane, and an excavator. The following steps explain how to construct the soldier pile wall with timber lagging

Soldier Piles and Lagging

1. Holes are drilled into the soil using 550 millimeter casing to ensure the soil does not collapse (Figure 8.1). The holes are drilled at different depths, the most critical depth being 8.5 meters.
2. The HP piles are then placed in the hole and held there while the concrete is poured. After the concrete is poured the casings are removed.
3. When all piles have been installed across the length of the wall, excavation will begin in order to install the timber lagging. The excavation is done in one meter increments.
4. At each one meter increment, the timber lagging is placed behind the H-beams and braced with lumber blocking (Figure 8.2)
5. The soldier piles are also designed with tiebacks to prevent high deflections. Each soldier pile will have a minimum of one tieback with most of them having two. The tiebacks are installed as the excavation occurs, the first one at a 2 meter depth and the second 3 meters below the first.

Figure 8.1: Drilling of Steel Casing for Installation of HP Sections

Figure 8.2: Bracing of Timber Lagging

Tiebacks:

The tiebacks are installed using the following procedure:

a. The holes for the tiebacks must be drilled for the required length and diameter at an angle of 30 degrees to the horizontal. There will be an unbonded length and a bonded length. The unbonded and bonded length will be the same for all tiebacks (4.5 meters). The diameter required for the bonded area of the tieback is 168mm, this will be the diameter for all drilled holes.
b. The steel bars must be placed in the hole, and then the primary grout is applied.
c. Performance and proof tests must be carried out on each bar to ensure sufficient installation.
d. The bars are then stressed and locked off.
e. Add secondary grout

The steel flange must be cut in order to drill holes and install the tiebacks. When the tiebacks are installed and tested, steel plates are welded back onto the flange for support. An L200x100x13 angle is also attached at the tieback as a steel connection to provide support.

6. As the excavation continues, the timber lagging and tiebacks will continue to be installed until it has reached the final depth.

Gravity Wall:

The gravity wall will begin construction on the south end when the desired depth of excavation is reached. It has a height of four meters and a length of approximately 15 meters. The base of the gravity wall is 2 meters thick and the top is 0.5 meters thick. PVC piping for the weep holes are to be installed within the concrete to allow for drainage.

The bedrock is excavated in order to place the formwork and begin the construction of the gravity wall. The formwork is constructed and placed in the correct location and then the concrete is poured. The piping for the weep holes is installed with the formwork to allow the concrete to form around them without seeping into the pipe. The concrete will take 14 days to cure and set and at that time the formwork can be removed. The soil behind the gravity wall will be sloped at a 45 degree angle to ensure that the soil does not fail.

7. Once both the soldier pile wall and gravity wall are complete, the soldier pile wall will be backfilled with the excess soil from the original excavation.
8. The final step is the demobilization of the site; all equipment will be removed.

9.0 CONCLUSION

Based on requirements outlined by Acuren, QCEC designed the most cost efficient solution for the Duckworth Street Retaining Wall.

Initially, QCEC determined a soldier pile retaining wall would be the most cost efficient option in comparison to a soil nail retaining wall. After a thorough analysis of the site conditions, it was determined that a rigid gravity wall could be implemented on the south end of the retaining wall location to reduce costs. The rigid gravity wall eliminated approximately 5 required soldier piles (large expense) and replaced them with unreinforced concrete. The materials required for the 15 meter length of the rigid gravity wall are much more cost efficient than 15 meters of additional soldier pile retaining wall.

At the request of the client, QCEC also developed a construction plan and schedule. The construction schedule details the actions required to construct the wall in 52 days. The construction of the rigid gravity and the soldier pile retaining wall have been scheduled to overlap for a period of the construction duration. This allows for efficient use of construction time and will limit the amount of disruption to the surrounding residential area.

10.0 REFERENCES

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- Handbook of Steel Construction, CSA S16-09, 10th ed., Canadian Institute of Steel Construction, Willowdale, ON, 2011.
- Concrete Design Handbook, 3rd ed., Cement Association of Canada, Ottawa, ON, 2006.
- RSMeansOnline version 5.0.1, Reed Construction Data Inc, 2013.

Appendix A

Geotechnical Report

Letter-form Report:

GEOTECHNICAL SUB-SURFACE INVESTIGATION

PROPOSED PARKING GARAGE AND CONDOMINIUMS

DUCKWORTH STREET AT BELL STREET

ST. JOHN'S, NEWFOUNDLAND AND LABRADOR

August 16, 2011

Project No. SJN-00021594-A0

Henry Bell Development Ltd.

Geotechnical Sub-surface Investigation

Type of Document
Letter-form Report

Project Name
Proposed Parking Garage and Condominiums
Duckworth Street at Bell Street
St. John's, Newfoundland and Labrador

Project Number
SJN-00021594-A0

Prepared by: **William G. Melendy, M.A.Sc., P.Eng.**

exp Services Inc.
60 Pippy Place, Suite 200
St. John's, NL A1B 4H7
Canada

Date Submitted
August 16, 2011

exp. *The new identity of **ADI Limited***

August 16, 2011

exp File SJN-00021594-A0
(27-6628-001.1)

Henry Bell Development Ltd.
12 Caldwell Place
St. John's, Newfoundland Labrador
A1E 6A4

Attention: Mr. William Clarke

Dear Sirs:

RE: **Geotechnical Sub-surface Investigation**
Proposed Parking Garage and Condominiums
Duckworth Street at Bell Street
St. John's, Newfoundland and Labrador

Acting on the request of Mr. William Clarke, representing *Henry Bell Development Ltd.,* **exp** Services Inc., the new identity of *ADI Limited*, has completed additional boreholes to supplement the previously-completed geotechnical sub-surface investigation at the Duckworth Street site. Specifically, three additional boreholes were advanced on Henry Street, and one borehole was advanced at the west side of Bell Street. The purpose of the investigation was to determine subsurface conditions, and update previous recommendations for the new parking garage/condominium project, as presented in the previously-completed reports, as follows:

> Geotechnical Sub-Surface Investigation, Proposed Parking Garage, Duckworth at Bell Street, St. John's, NF, final report dated February 14, 2011
>
> Geotechnical Sub-Surface Investigation, Proposed Parking Garage, Duckworth at Bell Street, St. John's, NF, letter-form report dated February 14, 2011.

1.0 FIELD PROGRAM

The field work for the additional borehole program was completed during the period July 5 to July 8, 2011, and consisted of four boreholes, advanced using a *CME-55* drill rig operated by *Logan Geotech Inc.* of Dieppe, New Brunswick. The boreholes were located in the field by **exp** in discussion with Henry Bell Development Ltd.. Final borehole locations are shown on *Figure 1: Borehole and Test Pit Location Plan.*

Conditions encountered in the boreholes are described below and on the Borehole Records attached. *Table 1: Summary of Borehole Data* presents the summarized findings of the investigation.

Table 1: SUMMARY OF BOREHOLE DATA
Proposed Parking Garage and Condominums - Duckworth Street at Bell Street
St. John's, Newfoundland and Labrador

Borehole No.	Ground Surface Elevation (m)	Bottom of Fill Elevation/Top of Till (m)	Top of Bedrock Elevation (m)	End of Borehole Elevation (m)	Ground-water Elevation (m)
BH01	17.77	16.0	16.0	13.7	16.9
BH02	21.63	19.7	16.2	12.6	19.2
BH03	22.76	21.7	21.7	13.0	21.3
BH04	25.09	19.4	18.4	14.8	-
BH05	25.71	22.6	17.3	15.6	-
BH06	26.68	23.5	22.9	19.3	-
BH07	19.94	17.6	17.0	13.9	-

NOTES:
Datum – Geodetic
BH01 to BH03 completed January 23-23, 2011
BH04 to BHo7 completed July 5-8, 2011
N/E: not-encountered

Asphalt

A 50 mm to 100 mm thick asphalt layer was encountered at the surface in BH04 to BH07.

Granular Fill/Fill

A Granular Fill or Fill layer was encountered beneath the asphalt at BH04, BH05, BH06, and BH07. The Granular Fill/Fill layer extended to depth ranging from 2.3 m to 5.7 m below the existing ground surface. The composition of the Fill is variable, but may generally be described as greyish-brown or dark grey to black gravelly Sand with some silt and occasional cobbles and boulders. Note that a 200 mm to 500 mm thick Class A and Class B granular layer was encountered in BH04, BH05, and BH06.

Based on N-values ranging from five to 42, encountered during the performance of Standard Penetration Tests, the Fill may be classified as loose to dense in terms of relative density

Tilll

A Till layer was encountered beneath the Fill in BH04, BH05, BH06, and BH07, at depths ranging from 2.3 m to 5.7 m below the existing ground surface, extending to depths ranging from 2.9 m to 8.4 m below the existing ground surface. The composition of the Till is variable, but may generally be described as brownish-grey to grey gravelly Sand, to a Sand and Gravel, with traces to some silt and occasional cobbles and boulders.

Based on N-values greater than 50, encountered during the performance of Standard Penetration Tests, the Till may be classified as very dense in terms of relative density.

Bedrock

Bedrock was encountered beneath the Till layer at BH04, BH05, BH06, and BH07 at depths ranging from 2.9 m to 8.4 m below the existing ground surface. Boreholes were terminated in the bedrock at depths ranging from 6.0 m to 10.3 m below the existing ground surface.

Bedrock is variable, but may be described as a medium grey sandstone. Rock quality designations (RQD) of retrieved core samples ranged from zero to 76, and on this basis, the bedrock may be classified as very-severely-fractured to fractured. Note that the RQD encountered in BH04, BH05, and BH07 was generally zero, a very-severely-fractured bedrock.

Published geology for the area indicates the bedrock consists of thin lenticular-bedded, dark grey sandstone and minor shale of the Renews Head Formation, St. John's Group.

2.0 DISCUSSION AND RECOMMENDATIONS

The following comments are provided relative to geotechnical aspects of design and construction for the proposed new parking garage/condominium building. It is understood that the proposed structure is intended to be of slab-on-grade construction, with conventional concrete strip footings, column footings, and foundation walls.

2.1 Site Development

Recommendations for site development within the proposed structure footprint, from a geotechnical viewpoint, were presented in the report, Geotechnical Sub-Surface

Investigation, Proposed Parking Garage, Duckworth at Bell Street, St. John's, NL, final report dated February 14, 2011, and will not be further discussed in this submission.

2.2 Building Foundations

Bearing Capacity of Footings
Bedrock encountered at the site, in general, is classified as very-severely-fractured to severely-fractured in terms of rock quality. Based on the bedrock quality, bedrock removal using a hydraulic rock buster may be achieved at the site. Bedrock quality typically encountered in the upper 1.6 m to 3.1 m is considered very-severely-fractured, becoming more sound with depth.

Exp is recommending that all building foundations be founded on bedrock with an RQD >30. For foundations placed on competent bedrock, design loads may be based on a net allowable bearing pressure of 800 kPa. Note that the recommended allowable bearing capacity of 800 kPa is conservative for competent bedrock. Higher bearing capacity values for competent bedrock could be discussed, if required, during detailed structural design. For fractured bedrock, design loads may be based on a net allowable bearing pressure of 400 kPa.

Foundation Preparation
Prior to placement of footings, excavation and removal of Fill, Till, and very-severely-fractured bedrock to required founding elevation, is required.

Floor slabs placed on structural blast-rock fill, compacted as previously outlined, should be cast on a free-draining layer of Department of Transportation and Works Class "A" aggregate at least 125 mm thick. The Class "A" should be compacted to 98 percent of the aggregate's Standard Proctor dry density.

Groundwater/drainage recommendations are presented in the report, Geotechnical Sub-Surface Investigation, Proposed Parking Garage, Duckworth at Bell Street, St. John's, NL, final report dated February 14, 2011, and the letter-form report, Geotechnical Sub-Surface Investigation, Proposed Parking Garage, Duckworth at Bell Street, St. John's, NL, letter-form report dated February 14, 2011, and will not be discussed further in this submission.

It is recommended that founding levels be inspected by a qualified geotechnical engineer/technologist prior to placement of footings, and during sub-slab placement of structural fill and/or blast-rock fill where required, to ensure that specified bearing capacities have been attained.

2.3 Geotechnical Parameters for Design

The recommended geotechnical parameters for design of foundations acting as retaining walls, as provided in the February 2011 geotechnical report, are summarized below in *Table 2: Recommended Geotechnical Parameters*. It should be noted that the following earth pressure coefficients are based on an assumed horizontal engineered backfill, placed and compacted in accordance with the recommendations of the previously submitted report. If inclined backfill or a different type of backfill is to be placed behind walls, the geotechnical engineer should be consulted for the appropriate earth pressure coefficients for design.

Table 2: RECOMMENDED GEOTECHNICAL PARAMETERS
Proposed Parking Garage and Condominiums - Duckworth Street at Bell Street
St. John's, Newfoundland and Labrador

Parameter	Compacted Engineered Fill
Total Unit Weight, kN/m^3	20.5
Buoyant Unit Weight, kN/m^3	10.5
Effective Friction Angle, degrees	36°
Coefficient of Active Earth Pressure, K_a	0.26
Coefficient of Passive Earth Pressure, K_p	3.8
Coefficient of Earth Pressure at Rest, K_o	0.41

The recommended geotechnical parameters for design of foundations acting as retaining walls for severely-fractured bedrock situations are summarized below in *Table 3: Recommended Geotechnical Parameters*. It should be noted that the following earth pressure coefficients are based on an assumed horizontal engineered backfill, placed and compacted in accordance with the recommendations of the previously-submitted report. Backfill material should consist of a well-graded, angular material with low percentage of fines (maximum 2 percent), such as a 4" minus blast rock. If inclined backfill or a different type of backfill is to be placed behind walls, the geotechnical engineer should be consulted for the appropriate earth pressure coefficients for design.

Table 3: RECOMMENDED GEOTECHNICAL PARAMETERS
Proposed Parking Garage and Condominiums - Duckworth Street at Bell Street
St. John's, Newfoundland and Labrador

Parameter	Severely Fractured Bedrock
Total Unit Weight, kN/m^3	24.5
Buoyant Unit Weight, kN/m^3	14.5
Effective Friction Angle, degrees	38°
Coefficient of Active Earth Pressure, K_a	0.24
Coefficient of Passive Earth Pressure, K_p	4.20
Coefficient of Earth Pressure at Rest, K_o	0.38

The following parameters may be used for concrete on sound bedrock interface:

- interface friction angle – 35°
- friction factor – 0.7
- geotechnical resistance factor, sliding – 0.8.

2.4 Site Classification for Seismic Site Response

In general, the ground profile at the site within the top 30 m may be considered as rock. Based on this, and per the requirements of the 2010 National Building Code of Canada (NBC 2010), the site classification for seismic site response, per Table 4.1.8.4.A of NBC 2010, Division B, Part 4, Structural Design, the site may be considered a Class "B" site classification.

3.0 CLOSURE

A subsurface investigation is a limited sampling of conditions at a particular site. Should conditions be encountered which differ from those described in this report, we require immediate notification in order to permit a re-evaluation of our recommendations.

We trust this submission meets your current requirements. Should you have any questions or require clarification on any aspect of this report, please do not hesitate to contact our office.

Yours very truly,

exp Services Inc.

WILLIAM MELENDY
Aug 17/2011

William G. Melendy, M.A.Sc., P.Eng.
Group Manager,
Geotechnical/Environmental Engineering

WGM:dgn

Attachments: Figure 1: Borehole and Test Pit Location Plan

Symbols and Terms Used on Borehole, Test Pit and Monitor Well Records

Borehole Records (4)

Geotechnical Sub-Surface Investigation, Proposed Parking Garage, Duckworth at Bell Street, St. John's, NL, final report dated February 14, 2011, prepared by ADI Limited

Geotechnical Sub-Surface Investigation, Proposed Parking Garage, Duckworth at Bell Street, St. John's, NL, letter-form report dated February 14, 2011, prepared by ADI Limited

PROVINCE OF NEWFOUNDLAND AND LABRADOR
PEG Newfoundland and Labrador
PERMIT HOLDER
This Permit Allows
EXP SERVICES INC.
To practice Professional Engineering in Newfoundland and Labrador.
Permit No. as issued by PEGNL D0215
which is valid for the year 2011

ATTACHMENTS

Figure 1: Borehole and Test Pit Location Plan

LEGEND

⊕ - TEST PIT LOCATION (MAY 2010)

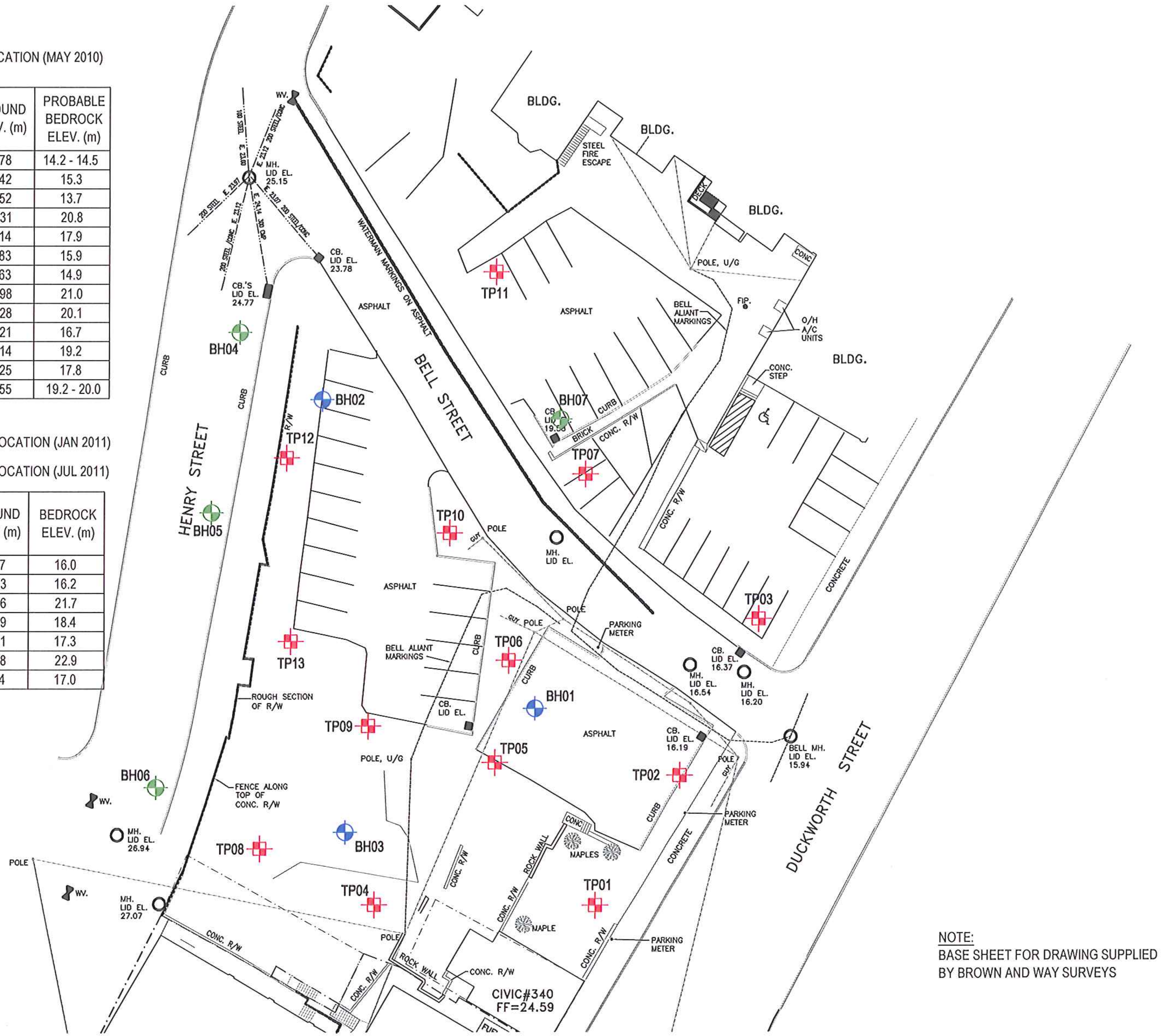
TEST PIT No.	GROUND ELEV. (m)	PROBABLE BEDROCK ELEV. (m)
TP01	15.78	14.2 - 14.5
TP02	16.42	15.3
TP03	16.52	13.7
TP04	23.31	20.8
TP05	18.14	17.9
TP06	18.83	15.9
TP07	18.63	14.9
TP08	23.98	21.0
TP09	20.28	20.1
TP10	20.21	16.7
TP11	21.14	19.2
TP12	22.25	17.8
TP13	21.55	19.2 - 20.0

LEGEND

⊕ - BORE HOLE LOCATION (JAN 2011)

⊕ - BORE HOLE LOCATION (JUL 2011)

BOREHOLE No.	GROUND ELEV. (m)	BEDROCK ELEV. (m)
BH01	17.77	16.0
BH02	21.63	16.2
BH03	22.76	21.7
BH04	25.09	18.4
BH05	25.71	17.3
BH06	26.68	22.9
BH07	19.94	17.0



NOTE:
BASE SHEET FOR DRAWING SUPPLIED BY BROWN AND WAY SURVEYS

No.	Issue	Date

1	BH04,BH05,BH06,BH07	W.G.M	AUG 17/2011
No.	Revision	Ckd. By	Date

Const. North

Drawn By: T.S.

Dwg. Standards Ckd. By:

Designed By: B.D.C.

Date Printed AUG, 2011

Dwg. Design Ckd. By:

exp Services Inc.

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60 Pippy Place, Suite 200
St. John's, NL A1B 4H7
CANADA
www.exp.com

Project Title

GEOTECHNICAL INVESTIGATION
PROPOSED PARKING GARAGE
DUCKWORTH STREET & BELL STREET
ST. JOHN'S, NL

Dwg. Title

BOREHOLE AND TEST PIT
LOCATION PLAN

Project No. SJN-0021594-A0

Dwg. No. FIGURE 1

Rev. No.

Scale 1 : 400

This drawing is not to be scaled

Offices located in:
Charlottetown, Moncton, Saint John, Truro, Halifax, Sydney
Port Hawkesbury, St.John's, Fredericton and Salem, NH

Symbols and Terms Used on the Borehole, Test Pit and Monitor Well Records

SYMBOLS AND TERMS USED ON THE BOREHOLE, TEST PIT, AND MONITOR WELL RECORDS

SOIL DESCRIPTION

Behavioural properties (i.e. plasticity, permeability) take precedence over particle gradation in describing soils.

Terminology describing soil structure:

Desiccated	-	having visible signs of weathering by oxidation clay minerals, shrinkage, cracks, etc.
Fissured	-	having cracks, and hence a blocky structure
Varved	-	composed of regular alternating layers of silt and clay
Stratified	-	composed of alternating layers of different soil types, e.g. silt and sand or silt and clay
Well-graded	-	having wide range in grain sizes and substantial amounts of all intermediate particle sizes
Uniformly-graded	-	predominantly of one grain size.

Terminology used for describing soil strata based upon proportion of individual particle sizes present:

Trace, or occasional	-	less than 10%
Some	-	10% to 20%
Adjective (e.g. silty or sandy)	-	20% to 35%
And (e.g. silt and sand)	-	35% to 50%

The standard terminology to describe cohesionless soils include the relative density, as determined by laboratory test or by the Standard Penetration Test N-value: the number of blows of 140 pound (64 kg) hammer falling 30 inches (760 mm), required to drive a 2-inch (50.8 mm) O.D. split-spoon sampler 1 foot (305 mm) into the soil. On the records, where complete sampler penetration is not achieved and an N-value cannot be reported, the total number of blows are shown over actual penetration in millimetres (eg. 75/180).

Relative Density	N-value	Relative Density %
Very Loose	<4	<15
Loose	4 - 10	15 - 35
Compact	10 - 30	35 - 65
Dense	30 - 50	65 - 85
Very Dense	>50	>85

The standard terminology to describe cohesive soils include the consistency, which is based on undrained shear strength as measured by in situ vane tests, penetrometer tests, unconfined compression tests, or occasionally by Standard Penetration Tests.

Consistency	Undrained Shear Strength		N-value
	Kips/sq.ft.	kPa	
Very Soft	< 0.25	< 12.5	< 2
Soft	0.25 to 0.5	12.5 to 25	2 to 4
Firm	0.5 to 1.0	25 to 50	4 to 8
Stiff	1.0 to 2.0	50 to 100	8 to 15
Very Stiff	2.0 to 4.0	100 to 200	15 to 30
Hard	> 4.0	> 200	> 30

SAMPLES

SS	Split-spoon sample (obtained by performing the Standard Penetration Test)
AS	Auger sample
ST	Shelby tube or thin-wall tube
PS	Piston sample

BK	Bulk sample
WS	Wash sample
RC	Rock core AXT, BXL, etc.

Rock core samples obtained with the use of standard diamond drilling bits.

OTHER TESTS

G	Specific Gravity
H	Hydrometer Analysis
S	Sieve Analysis
MC	Moisture Content
y	Unit Weight
C	Consolidation
CD	Consolidated drained triaxial

CU	Consolidated undrained triaxial with pore pressure measurements
UU	Unconsolidated undrained triaxial
RCC	Rock Core Compression
DS	Direct Shear
P	Field Permeability
TPH	Total Petroleum Hydrocarbons (ppm)
ND	Below Detection Limit

ROCK DESCRIPTION

The description of rock is based on the rock quality designation (RQD).

The classification is based on a modified core recovery percentage in which all pieces of sound core over 100 mm long are counted as recovery. The smaller pieces are considered to be due to close shearing, jointing, faulting, or weathering in the rock mass and are not counted. In most cases, RQD is run on NXL core; however, it can be used on different core sizes if the bulk of the fractures caused by drilling stresses are easily distinguishable from normal in situ fractures.

RQD	Rock Quality
90 to 100	excellent quality
75 to 90	good quality
50 to 75	fair quality
25 to 50	poor quality
< 25	very poor quality

CLASSIFICATION OF ROCK WITH REGARD TO STRENGTH

STRENGTH		FIELD IDENTIFICATION METHOD	RANGE OF UNCONFINED COMPRESSIVE STRENGTH (MPa)
Grade	Classification		
RO	Extremely weak	Indented by thumbnail	< 1
R1	Very weak	Crumbles under firm blows of geological hammer; can be peeled with a pocket-knife	1 - 5
R2	Weak rock	Can be peeled by a pocket-knife with difficulty; shallow indentations made by a firm blow with point of geological hammer	5 - 25
R3	Medium strong	Cannot be scraped or peeled with a pocket-knife; specimen can be fractured with a single firm blow of geological hammer	25 - 50
R4	Strong	Specimen requires more than one blow of geological hammer to fracture	50 - 100
R5	Very strong	Specimen requires many blows of geological hammer to fracture	100 - 250
R6	Extremely strong	Specimen can be chipped by geological hammer	> 250

STRATA PLOT


Asphalt

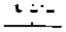

Concrete


Organics


Fill


Cobbles &
Boulders

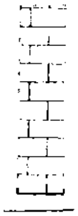

Gravel


Sand


Silt


Clay

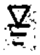

Igneous
Bedrock


Sedimentary
Bedrock


Metamorphic
Bedrock

WATER LEVEL MEASUREMENT


Borehole,
Standpipe or
Monitor Well
(at time of drilling)


Borehole,
Standpipe or
Monitor Well
(static conditions)

WELL CONSTRUCTION


Flush-mounted
Well-head
Enclosure


End/Top Cap


Solid
PVC Pipe


Slotted
PVC Pipe


Bentonite


Cave-in

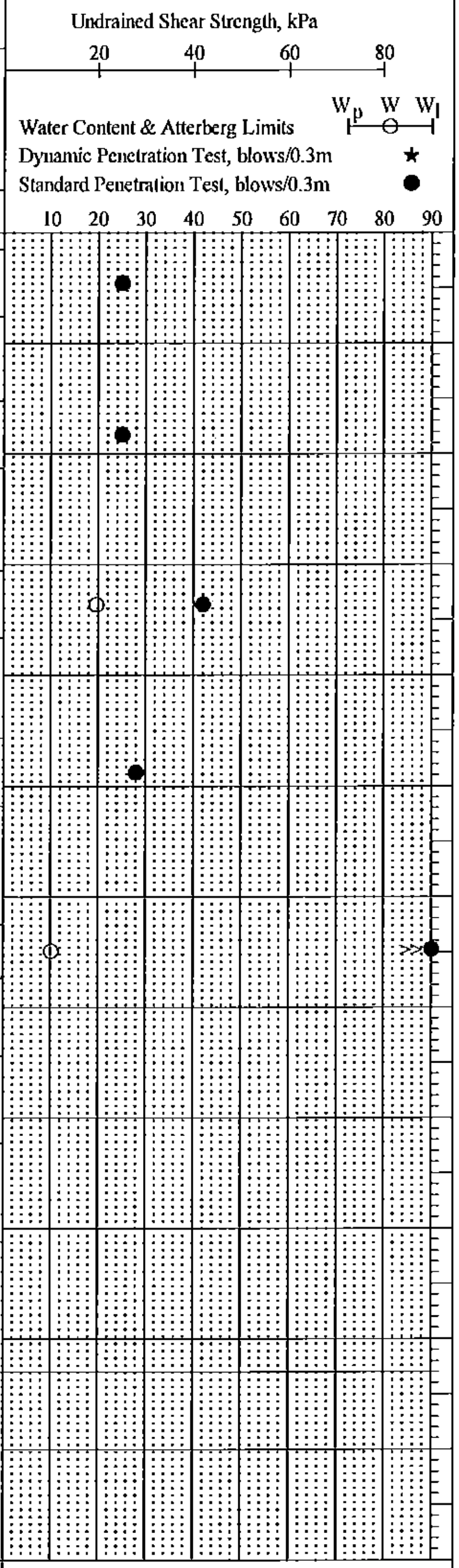
Borehole Records (4)

BOREHOLE RECORD

CLIENT Henry Bell Development Ltd.
PROJECT No. SJN-21594-A0
LOCATION Proposed Parking Garage - Duckworth Street at Bell Street, St. John's, NL
BOREHOLE No. BH 04
DATES (dd-mm-yy): BORING 07-07-11
WATER LEVEL 07-07-11
DATUM Geodetic

ELEV. (m)	DESCRIPTION
25.09	
25.0	**ASPHALT:**
24.8	**GRANULAR FILL:** Loose brownish grey GRAVEL and SAND; trace silt [Granular base (Class "A") and sub-base (Class "B") materials]; moist.
	FILL: Compact to dense greyish brown to brownish grey gravelly SAND; some silt; occasional cobbles and boulders; moist.
19.4	**TILL:** Very dense brownish grey to grey SAND and GRAVEL; trace silt; cobbles and boulders; moist.
18.4	**BEDROCK:** Very severely fractured SANDSTONE; some mud seams.
14.8	End of Borehole

TYPE	NUMBER	RECOVERY mm	N-VALUE OR RQD	OTHER TESTS
SS	01	355	25	
SS	02	305	25	
SS	03	430	42	MC, S
RC	04	305	28	
SS	05	355	123	MC, S
RC	06	100%	0	
RC	07	39%	13	
RC	08	0%	0	



DEPTH (m): 0 1 2 3 4 5 6 7 8 9 10 11 12

NOTES:

1) Bedrock encountered at 6.7 m depth.

exp Services Inc.
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St. John's, NL, A1B 4H7
Tel 709.579.2027 Fax 709.579.7115

Technologist: B. Cameron
Reviewed By: Wm. Melendy
Contractor: Logan Geotech Inc.
Equipment: CME-55

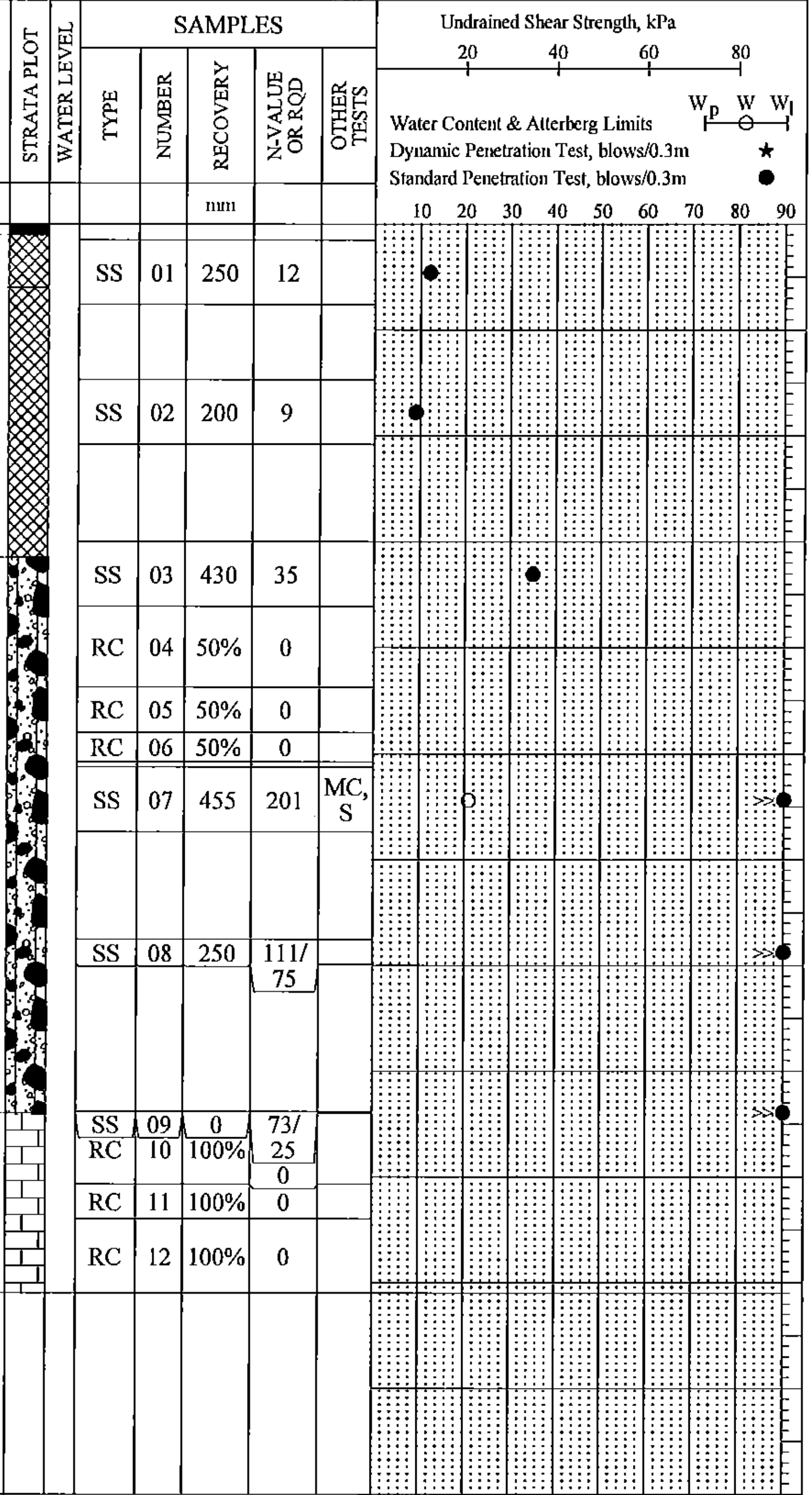
△ Unconfined Compression Test
▼ Water Level at Time of Drilling/Excavation
▽ Static Water Level
☐ Field Vane Test ■ Remoulded

BOREHOLE RECORD

CLIENT Henry Bell Development Ltd.
PROJECT No. SJN-21594-A0
LOCATION Proposed Parking Garage - Duckworth Street at Bell Street, St. John's, NL
BOREHOLE No. BH 05
DATES (dd-mm-yy): BORING 06-07-11 WATER LEVEL 06-07-11
DATUM Geodetic

ELEV. (m)	DESCRIPTION	TYPE	NUMBER	RECOVERY (mm)	N-VALUE OR RQD	OTHER TESTS
25.71						
25.6	**ASPHALT:**					
25.1	**GRANULAR FILL:** Compact brownish grey GRAVEL and SAND; trace silt [Granular base (Class "A") and sub-base (Class "B") materials]; moist.	SS	01	250	12	
	FILL: Loose to compact dark grey to black to brown gravelly SAND; some silt; occasional cobbles and boulders; moist.	SS	02	200	9	
22.6	**TILL:** Dense to very dense brownish grey to grey gravelly SAND; some silt; some cobbles and boulders - frequency decreasing with depth; moist.	SS	03	430	35	
		RC	04	50%	0	
		RC	05	50%	0	
		RC	06	50%	0	
		SS	07	455	201	MC, S
		SS	08	250	111/ 75	
17.3	**BEDROCK:** Very severely fractured SANDSTONE.	SS	09	0	73/ 25	
		RC	10	100%	0	
		RC	11	100%	0	
		RC	12	100%	0	
15.6	End of Borehole					

NOTES:
1) Bedrock encountered at 8.4 m depth.



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△ Unconfined Compression Test
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▽ Static Water Level
☐ Field Vane Test ■ Remoulded

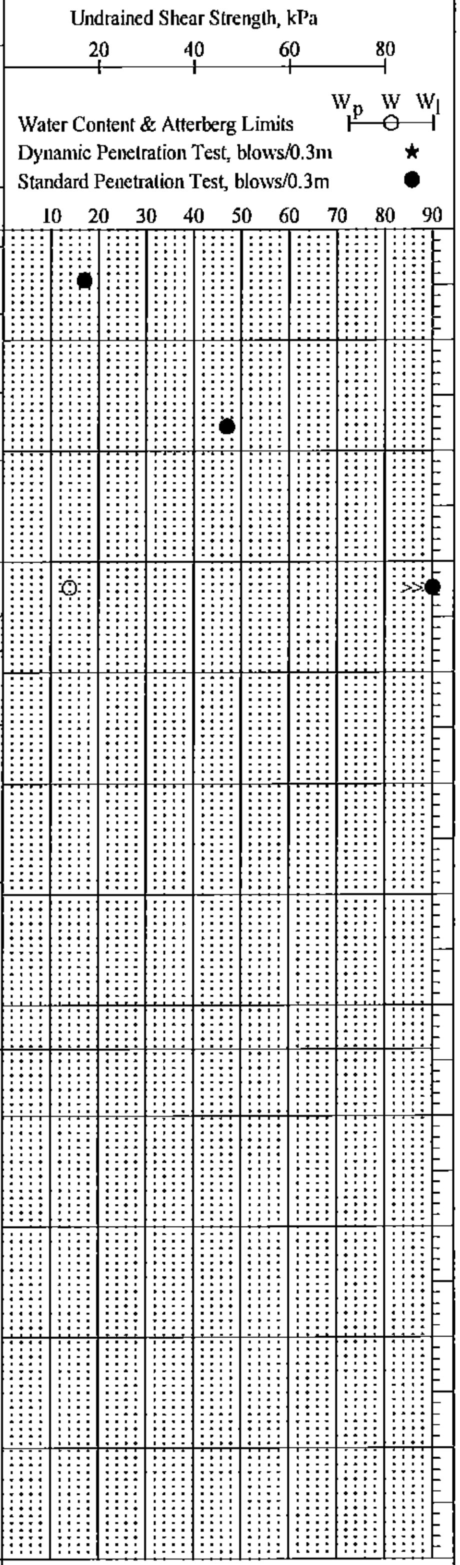
BOREHOLE RECORD

CLIENT Henry Bell Development Ltd.
PROJECT No. SJN-21594-A0
LOCATION Proposed Parking Garage - Duckworth Street at Bell Street, St. John's, NL
BOREHOLE No. BH 06
DATES (dd-mm-yy): BORING 05-07-11
WATER LEVEL 05-07-11
DATUM Geodetic

DEPTH (m)	ELEV. (m)	DESCRIPTION	STRATA PLOT	WATER LEVEL	SAMPLES TYPE	NUMBER	RECOVERY	N-VALUE OR RQD	OTHER TESTS
							mm		
0	26.68								
	26.6	**ASPHALT:**							
	26.2	**GRANULAR FILL:** Compact brownish grey GRAVEL and SAND; trace silt [Granular base (Class "A") and sub-base (Class "B") materials]; moist.			SS	01	200	17	
		FILL: Compact to dense greyish brown to brownish grey gravelly SAND; some silt; occasional cobbles and boulders; moist.			SS	02	510	47	
3	23.5	**TILL:** Very dense brownish grey to grey gravelly SAND; some silt; occasional cobbles and boulders; moist.			SS	03	305	153	MC, S
	22.9	**BEDROCK:** Very severely fractured to fractured SANDSTONE.			RC	04	100%	0	
					RC	05	100%	45	
					RC	06	100%	76	
	19.3	End of Borehole							

NOTES:
1) Bedrock encountered at 3.8 m depth.

Depth scale (m): 0, 1, 2, 3, 4, 5, 6, 7, 8, 9, 10, 11, 12



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Technologist: B. Cameron
Reviewed By: Wm. Melendy
Contractor: Logan Geotech Inc.
Equipment: CME-55

△ Unconfined Compression Test
▼ Water Level at Time of Drilling/Excavation
▽ Static Water Level
☐ Field Vane Test ■ Remoulded

BOREHOLE RECORD

CLIENT: Henry Bell Development Ltd.

LOCATION: Proposed Parking Garage - Duckworth Street at Bell Street, St. John's, NL

DATES (dd-mm-yy): BORING 08-07-11

WATER LEVEL 08-07-11

PROJECT No. SJN-21594-A0

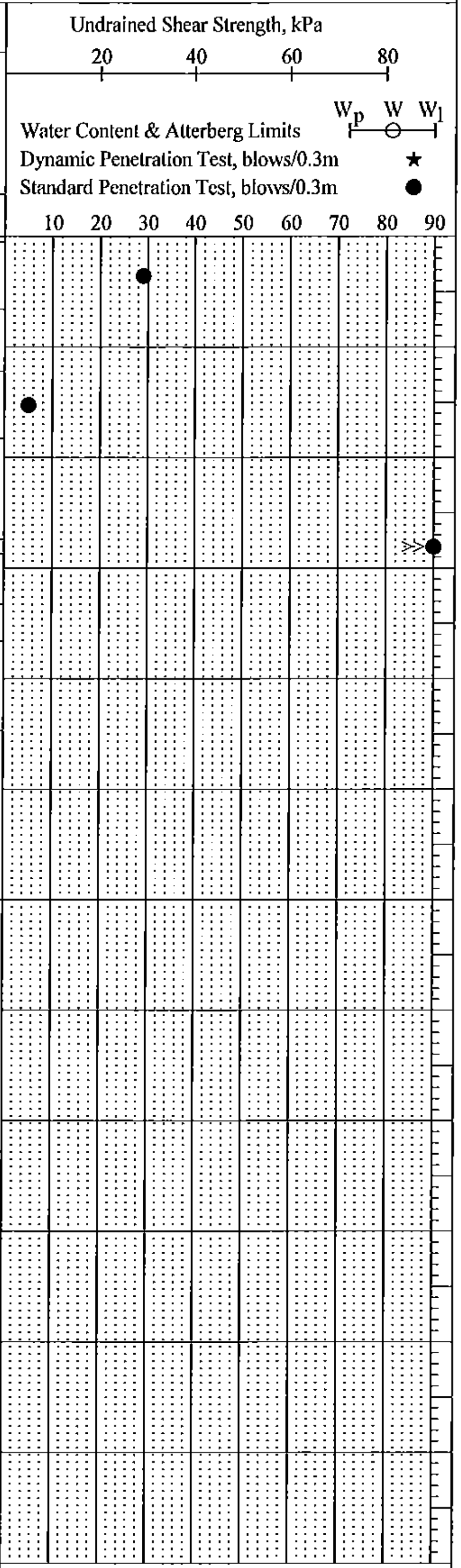
BOREHOLE No. BH 07

DATUM Geodetic

DEPTH (m)	ELEV. (m)	DESCRIPTION	TYPE	NUMBER	RECOVERY (mm)	N-VALUE OR RQD	OTHER TESTS
0	19.94 19.9	**ASPHALT:**					
		FILL: Compact to loose greyish brown to brownish grey gravelly SAND; some silt; occasional cobbles and boulders; moist.	SS	01	75	29	
			SS	02	75	5	
	17.6	**TILL:** Very dense brownish grey to grey gravelly SAND; some silt; occasional cobbles and boulders; moist.	SS	03	125	65/125	
	17.0	**BEDROCK:** Very severely fractured SANDSTONE; some mud seams.	RC	04	50%	0	
			RC	05	50%	0	
			RC	06	50%	0	
			RC	07	50%	0	
6	13.9	End of Borehole					

NOTES:

1) Bedrock encountered at 2.9 m depth.



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Contractor: Logan Geotech Inc.
Equipment: CME-55

△ Unconfined Compression Test
▼ Water Level at Time of Drilling/Excavation
▽ Static Water Level
☐ Field Vane Test ■ Remoulded

ADI Limited Final Report:

Geotechnical Sub-surface Investigation
Proposed parking Garage
Duckworth at Bell Street
St. John's, Newfoundland and Labrador
Date: February 14, 2011

Final Report:

GEOTECHNICAL SUB-SURFACE INVESTIGATION
PROPOSED PARKING GARAGE
DUCKWORTH STREET AT BELL STREET
ST. JOHN'S, NEWFOUNDLAND AND LABRADOR
Prepared for: Henry Bell Development Ltd.

ADI LIMITED
FILE: 27-6628-001.1
DATE: February 2011

ADI Limited
A Trow Global Company

Architecture
Engineering
Consulting
Procurement
Project Management

February 14, 2011

Air File 27-6628-001.1

Henry Bell Development Ltd.
12 Caldwell Place
St. John's, Newfoundland and Labrador
A1E 6A4

Attention: Mr. William Clarke

Dear Sirs:

RE: **Geotechnical Sub-surface Investigation**
Proposed Parking Garage
Duckworth Street at Bell Street
St. John's, Newfoundland and Labrador

Enclosed are three copies of our final report containing the findings of our geotechnical sub-surface investigation at the above-noted site, for your perusal.

We trust this submission meets your current requirements, and thank you for the opportunity of providing our services.

Yours very truly,

ADI Limited

Blair Cameron

Blair D. Cameron, P.Tech.
Sr. Geo-Enviromental Tech.

William G. Melendy, M.A.Sc., P.Eng.
Group Manager,
Geotechnical/Environmental Engineering

BDC/WGM:dgn

Enclosure: Report in triplicate

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o:/#Project Files/#Client 6601 to 7000/6628/6628001 1/Reports/Final Geotechnical Report Feb'11

Disclaimer Statement

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TABLE OF CONTENTS

ADI Quality System Checks

Project No.:	27-6628-001.1	Date:	2011 02 14 *[yr/mo/da]*
Issue Status:	Final Report	Revision No.:	0
Prepared by:	Blair D. Cameron, P.Tech., William G. Melendy, M.A.Sc., P.Eng.	Blair Cameron	*[Signature]*
Reviewed by:	William G. Melendy, M.A.Sc., P.Eng.		*[Signature]*

1.0 INTRODUCTION

Further to your request, *ADI Limited* has completed a geotechnical sub-surface investigation for a proposed parking garage located on Duckworth Street at Bell Street in St. John's, Newfoundland and Labrador. The intent of the investigation was to confirm soil conditions in the area of the proposed parking garage, and provide recommendations.

2.0 SITE DESCRIPTION

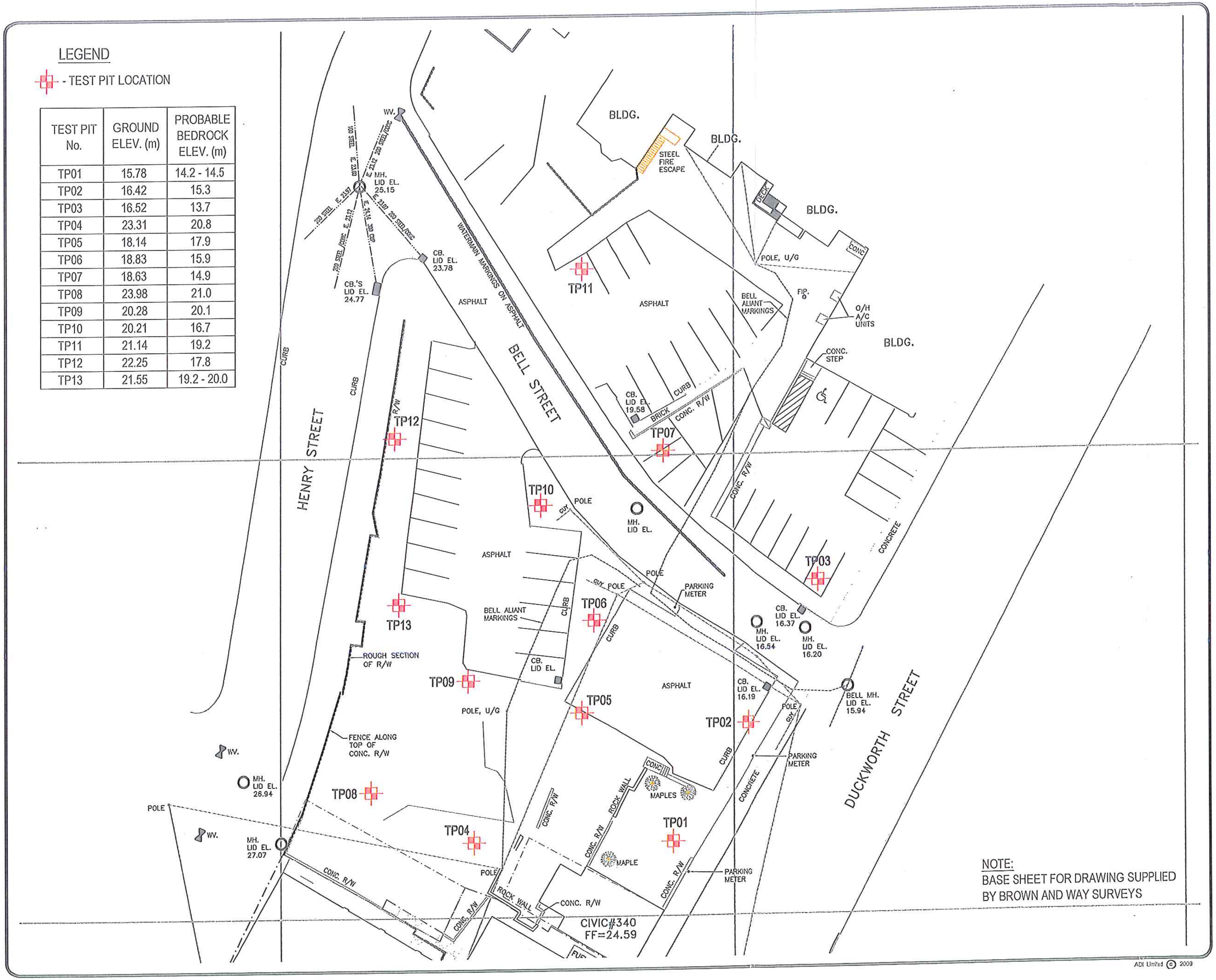
The proposed parking garage is to be located on the northern side of Duckworth Street at Bell Street, in St. John's, Newfoundland and Labrador. The property along the area of the proposed parking garage consists of Bell Street, along with several paved parking and landscaped areas. We understand that Bell Street is to be re-aligned to accommodate the proposed parking garage. Access to the site is via Duckworth Street from the south and Henry Street from the north. Site topography is generally sloping from north to south. The area is provided water and sewer services by the City of St. John's.

We understand that the proposed project will consist of slab-on-grade construction with conventional strip and column footings.

3.0 FIELD PROGRAMME

Field work was completed on May 29, 2010 and comprised the mechanical excavation of 13 test pits (TP), advanced using a *Cat 420E IT* extend-a-hoe operated by *Roley Construction Limited* of St. John's, Newfoundland and Labrador. The test pits were located in the field by *ADI Limited* through discussions with Mr. Richard Cooke of *RJC Services* and Mr. William Clarke of *Henry Bell Development Ltd.* Final test pit locations are shown on *Figure 1: Test Pit Location Plan.*

The test pits generally penetrated an organic or asphalt layer and a fill and till deposit. Bedrock was encountered at all 13 test pit locations. Groundwater was not encountered at any of the test pit locations.



LEGEND

⌖ - TEST PIT LOCATION

TEST PIT No.	GROUND ELEV. (m)	PROBABLE BEDROCK ELEV. (m)
TP01	15.78	14.2 - 14.5
TP02	16.42	15.3
TP03	16.52	13.7
TP04	23.31	20.8
TP05	18.14	17.9
TP06	18.83	15.9
TP07	18.63	14.9
TP08	23.98	21.0
TP09	20.28	20.1
TP10	20.21	16.7
TP11	21.14	19.2
TP12	22.25	17.8
TP13	21.55	19.2 - 20.0

NOTE:
BASE SHEET FOR DRAWING SUPPLIED
BY BROWN AND WAY SURVEYS

No.	Issue	Date

No.	Revision	Ckd. By	Date

Const. North

Drawn By: T.S.

Dwg. Standards Ckd. By:

Designed By: B.D.C.

Date Printed JUNE, 2010

Dwg. Design Ckd. By:

ADI Limited
St. John's, NL, Canada
Engineering, Consulting, Procurement and Project Management

Project Title
GEOTECHNICAL INVESTIGATION
PROPOSED PARKING GARAGE
DUCKWORTH STREET & BELL STREET
ST. JOHN'S, NL

Dwg. Title
TEST PIT LOCATION PLAN

Project No. 27-6628-001.1

Dwg. No. FIGURE 1

Rev. No.

Scale 1 : 400
This drawing is not to be scaled

Offices located in:
Charlottetown, Moncton, Saint John, Truro, Halifax, Sydney
Port Hawkesbury, St.John's, Fredericton and Salem, NH

"FILE"
MODEL "MODELNAME"
PLOTTED BY: "USER" DATE "DATE" "TIME"

Conditions encountered in test pits are described below and on Test Pit Records in the Appendix. *Table 1: Summary of Test Pit Data,* presents the summarized findings of the investigation.

TABLE 1: SUMMARY OF TEST PIT DATA
PROPOSED PARKING GARAGE
Duckworth Street at Bell Street
St. John's, Newfoundland and Labrador

Test Pit No.	Elevation (m)	Bottom of Organic/Asphalt Layer Elevation (m)	Top/Bottom of Fill Elevation (m)	Top of Sand and Gravel/ Till Elevation (m)	Top of Probable Bedrock Elevation (m)	End of Test Pit Elevation (m)	Ground-Water Elevation (m)
01	15.78	15.7	15.7 / 14.2	N/E	14.2 - 14.5	14.2	N/E
02	16.42	16.4(1)	16.4 / 15.3	N/E	15.3	15.2	N/E
03	16.52	16.5(1)	16.5 / 13.7	N/E	13.7	13.7	N/E
04	23.31	22.9	22.9 / 22.4	22.4	20.8	20.8	N/E
05	18.14	17.9	N/E	N/E	17.9	17.9	N/E
06	18.83	18.7	18.7 / 15.9	N/E	15.9	15.9	N/E
07	18.63	18.6(1)	18.6 / 16.6	16.6	14.9	14.9	N/E
08	23.98	N/E	24.0 / 23.0	23.0	21.0	21.0	N/E
09	20.28	N/E	20.3 / 20.1	N/E	20.1	19.8	N/E
10	20.21	20.0	20.0 / 17.6	17.6	16.7	16.7	N/E
11	21.14	21.1(1)	21.1 / 19.5	19.5	19.2	19.2	N/E
12	22.25	22.1	22.1 / 21.1	21.1	17.8	17.8	N/E
13	21.55	21.4	21.4 / 21.1	21.1	19.2 - 20.0	19.2	N/E

NOTES:
Elevations supplied by Brown and Way Surveys
N/E - not-encountered
(1) - Asphalt surface

3.1 Organics

A 100 mm to 400 mm thick Organic (grassmat) layer was encountered at ground surface in test pits TP01, TP04, TP05, TP06, TP10, and TP13.

3.2 Asphalt

A 30 mm to 70 mm thick Asphalt layer was encountered at ground surface in test pits TP02, TP03, TP07, and TP11.

3.3 Granular Fill

A 0.35 m to 0.45 m thick Granular Fill layer was encountered beneath the asphalt layer in TP02, TP03, TP07, and TP11. The granular fill extended to depths ranging from 0.4 m to 0.5 m below the existing ground surface. The composition of the Granular Fill is variable, but may generally be described as brownish-grey Gravel and Sand with traces of silt (granular sub-base and base - Class "B" and Class "A" materials).

Based on observation of the backhoe performance, the Granular Fill material is classified as loose at all test pit locations in terms of relative density.

3.4 Fill

A Fill layer ranging in thickness ranging from 0.2 m to 2.75 m was encountered at ground surface in TP08 and TP09; beneath the Organic layer in TP01, TP04, TP06, TP10, TP12, and TP13; and beneath the Granular Fill in TP02, TP03, TP07, and TP11. The Fill layer terminated on bedrock at depths ranging from 0.2 m to 2.9 m below the existing ground surface in TP01, TP02, TP03, TP06, and TP09. The Fill layer extended to depths ranging from 0.5 m to 2.6 m below the existing ground surface in TP04, TP07, TP08, TP10, TP11, TP12, and TP13. The composition of the Fill is variable, but may generally be described as a greyish-brown to brownish-grey Gravel and Sand with traces of silt and occasional to some cobbles and boulders. Some organics (roots and rootlets) were encountered in TP01, TP04, and TP08. Some debris (pieces of bricks, metal, concrete, rags, steel rods, wood, bottles, and old telephone cables) were encountered in the 12 test pits where Fill was encountered except TP11, TP12, and TP13. It should be noted that a 0.6 m and 0.2 m thick layer of rock fill was intermixed with Gravel and Sand on the surface of the Fill layer in TP10 and TP11, respectively.

Based on observation of the backhoe performance, the Fill is classified as loose to compact in terms of relative density.

3.5 Sand and Gravel

A 0.7 m thick Sand and Gravel layer was encountered beneath the Fill layer in TP12 at a depth of 1.2 m below the existing ground surface. The composition of the Sand and Gravel is variable, but may generally be described as a tannish-brown Sand and Gravel with traces of silt, and occasional cobbles and boulders.

Based on observation of backhoe performance, the Sand and Gravel is classified as compact in terms of relative density.

3.6 Till

A Till layer was encountered beneath the Fill layer in TP04, TP07, TP08, TP10, TP11, and TP13 at depths ranging from 0.5 m to 2.6 m below the existing ground surface, and beneath the Sand and Gravel layer in TP12 at a depth of 1.9 below the existing ground surface. The Till layer extended to depths ranging from 1.9 m to 4.5 m below the existing ground surface. The composition of the Till layer is variable, but may generally be described as a brownish-grey to grey Sand and Gravel to Gravel and Sand; with traces of silt and occasional to some cobbles and boulders.

Particle size analyses was completed on four representative sample of the Till deposit, with the following results:

Gravel:	40.4 percent to 61.6 percent	(average: 50.5 percent)
Sand:	32.9 percent to 51.3 percent	(average: 41.8 percent)
Silt:	5.5 percent to 8.5 percent	(average: 7.7 percent).

The natural moisture content of the Till samples tested ranged from 5.3 percent to 7.9 percent, with an average natural moisture of 6.5 percent.

Based on observation of the backhoe performance, the Till is classified as compact to dense at TP04, TP08, TP10, and TP11 and as compact to very dense at TP07, TP12, and TP13 in terms of relative density.

3.7 Bedrock

Bedrock was encountered in all 13 test pit locations. All test pits were terminated in Bedrock at depths ranging from 0.2 m to 4.5 m below the existing ground surface. Based on published geology for the area, Bedrock consists of thin, lenticular-bedded, dark grey sandstone and minor shale of the Renews Head Formation, St. John's Group (A.F. King, Map 90-120).

3.8 Groundwater

Groundwater was not encountered at any of the 13 test pit locations. It is noted that the groundwater table may generally be expected to fluctuate seasonally and in response to extended heavy rainfall events.

4.0 DISCUSSION AND RECOMMENDATIONS

The following comments are provided relative to geotechnical aspects of design and construction for the proposed new structure and parking areas. It is understood that the proposed structure is intended to be of slab-on-grade construction, with conventional concrete strip footings, column footings, and foundation walls.

4.1 Site Development

Site development within the proposed structure footprint and parking areas, from a geotechnical viewpoint, will require attention to the following aspects:

- In the areas of all exterior and interior footing construction, all Fill and native Till are to be removed to the surface of the underlaying Bedrock.

- Note that the excavated Fill is not suitable for re-use as structural fill, and may be used in landscaped areas only.

- Placement and compaction of imported pit-run fill, as/if required to attain proposed sub-grade elevations, should comprise a well-graded aggregate with less than 10 percent silt content, and all particles larger than 200 mm screened off. Structural fill should be placed in maximum loose lifts of 450 mm and compacted through use of a 10- to 12-tonne vibratory roller to minimum 98 percent of the material's Standard Proctor dry density. Should space restraints prevent the use of a 10- to 12-tonne vibratory roller, loose lift thickness should be reduced to a maximum thickness of 150 mm and compacted using a heavy plate tamper such as a *BOMAG* vibrating diesel plate tamper, or equivalent, to obtain the required compaction results.

- Finish grades around the structure should be sloped to promote positive drainage away from the structure.

- Structural fill beneath floor-slab areas, where required, should be placed in loose lifts not exceeding 450 mm in thickness and compacted at optimum moisture content to 98 percent of the material's Standard Proctor dry density.

- Imported fill (where required), as previously described for structural fill, to be used in access and/or parking areas, should be placed in maximum loose lifts of 450 mm and compacted through use of a 10- to 12-tonne vibratory roller to a minimum 98 percent of the material's Standard Proctor dry density.

- We anticipate that de-watering, as may be required during construction works, can be addressed by conventional sump and pump methods, based on the groundwater conditions encountered at the time of the investigation.

- Should the use of blast-rock be selected instead of a pit-run fill, it should comprise material of 200 mm maximum particle size with less than five percent smaller than 25 mm. The rock shall be placed in loose lift thicknesses as noted for pit-run, and compacted through a minimum eight passes of a 10- to 12-tonne vibratory roller.

4.2 Building Foundations

Bearing Capacity of Footings

ADI is recommending that all building foundations be founded on bedrock. For foundations placed on competent bedrock, design loads may be based on a net allowable bearing pressure of 800 kPa. Note that the recommended allowable bearing capacity of 800 kPa is conservative for competent bedrock. Higher bearing capacity values for competent bedrock could be discussed, if required, during detailed structural design. For fractured bedrock, design loads may be based on a net allowable bearing pressure of 400 kPa.

Foundation Preparation

Prior to placement of footings, excavation and removal of Organics, Fill, Till, and Bedrock to required founding elevation is required.

Floor slabs placed on structural blast-rock fill, compacted as previously outlined in Section 4.1, should be cast on a free-draining layer of Department of Transportation and Works Class "A" aggregate at least 125 mm thick. The Class "A" should be compacted to 98 percent of the aggregate's Standard Proctor dry density.

Sieve analyses have confirmed a silt content for the native site soils ranging from 5.5 percent to 8.5 percent. Where there is a high silt content (greater than ten percent), the potential for erosion problems increases. If soil becomes soft during construction activities, the soil must be removed from the work area and replaced with structural blast-rock or suitable Fill, compacted in lifts to the satisfaction of the geotechnical engineer. Founding areas must be void of surface water and free of any loose soil prior to placement of concrete.

We anticipate that de-watering, as may be required during construction works, can be addressed by conventional sump and pump methods, based on the groundwater conditions encountered at the time of the investigation.

It is recommended that founding levels be inspected by a qualified geotechnical engineer/ technologist prior to placement of footings, and during sub-slab placement of structural fill and/or blast-rock fill where required, to ensure that specified bearing capacities have been attained.

Groundwater/Drainage Recommendations

We recommend that all perimeter walls below grade receive a water-proofing membrane and free-draining backfill be installed immediately against foundation walls.

As noted previously, groundwater was not encountered at any of the test pit locations. Project drawings dated June 12, 2010, Typical Building Section, indicate a proposed finish Level 1 floor elevation of 14.3 m. Bedrock was encountered at Elev. 21.0 m (TP08), Elev. 17.8 m (TP12), and Elev. 19.2 m (TP13), on the Henry Street side of the development. Based on this, up to approximately 6.7 m of bedrock will have to be removed to achieve the Level 1 floor slab elevation of 14.3 m. *ADI* recommends that drainage piping should be constructed immediately adjacent all perimeter footings, with positive drainage into the municipal storm system.

Given the size of the proposed structure, the removal of up to 6.7 m of bedrock, and the potential for groundwater flow from fractures in the bedrock, an under-drain system may be required beneath the slabs at the lowest levels. Typically, this would consist of a series of perforated drainage pipes placed just above the bedrock surface and backfilled with free-draining gravels. The perforated piping should be connected to a non-perforated header pipe(s) to direct the flow out of the building. This requirement should be evaluated at the time of bedrock removal at the site, taking into the groundwater conditions encountered.

If required, we recommend that PVC piping be used for drainage works and that the system be designed to ensure positive flow to the municipal storm system. A minimum slope of one percent is recommended for perimeter and under-drain piping. The invert of all sections of drainage piping should be at least 300 mm below the top of slab level. Traps should be installed for future inspection and cleaning if required, and backwater flow valves should be installed to prevent water from entering the system from outside sources.

It is recommended that founding levels be inspected by a qualified geotechnical engineer prior to placement of footings, and during placement of structural fill and blast-rock fill, to ensure that specified bearing capacities have been attained.

4.3 Geotechnical Parameters For Design

The recommended geotechnical parameters for design of foundations acting as retaining walls are summarized below in *Table 2: Recommended Geotechnical Parameters*. It should be noted that the following earth pressure coefficients are based on an assumed horizontal engineered backfill, placed and compacted in accordance with the recommendations above. If inclined backfill or a different type of backfill is to be placed behind walls, the geotechnical engineer should be consulted for the appropriate earth pressure coefficients for design.

TABLE 2: RECOMMENDED GEOTECHNICAL PARAMETERS
Proposed Parking Garage
Duckworth Street at Bell Street
St. John's, Newfoundland and Labrador

Parameter	Compacted Engineered Fill
Total Unit Weight, kN/m^3	20.5
Buoyant Unit Weight, kN/m^3	10.5
Effective Friction Angle, degrees	36°
Coefficient of Active Earth Pressure, K_a	0.26
Coefficient of Passive Earth Pressure, K_p	3.8
Coefficient of Earth Pressure at Rest, K_o	0.41

4.4 Site Classification for Seismic Site Response

In general, the ground profile at the site within the top 30 m may be considered as very dense soil and soft rock. Based on this, and per the requirements of the 2005 National Building Code of Canada (NBC 2005), the site classification for seismic site response, per Table 4.1.8.4.A of NBC 2005, Division B, Part 4, Structural Design, the site may be considered a Class C site classification.

4.5 Parking and Drive-Lanes

In the areas of all parking and drive-lane construction, as a minimum, the compacted Fill is to be proof-rolled and, in situations where the material becomes soft, the soft material is to be removed and replaced with imported pit-run fill or blast-rock. It is recommended that the excavated elevation be inspected by a geotechnical engineer/technologist prior to proof-rolling to confirm suitability of the sub-grade layer for proof-rolling, and to monitor the proof-rolling activities.

Structural fill, as required to attain sub-grade design elevations, should be placed on the re-compacted Till only after it has been proof-rolled and re-compacted, as noted previously. Structural fill should be placed and compacted in 450 mm lifts to 98 percent of the material's Standard Proctor dry density. Materials with more than 10 percent silt and considered frost-susceptible should not be used as sub-grade structural fill.

Driving/parking surfaces for light vehicular traffic should comprise, as a minimum, a 50 mm thick asphalt course over properly-compacted base and sub-base granulars. All asphalt should be compacted to a minimum 97 percent of Marshall density. Granular sub-base (Class "B") and base (Class "A") granulars should comprise 150 mm and 100 mm, respectively, compacted to 100 percent of Standard Proctor dry density.

5.0 CLOSURE

A subsurface investigation is a limited sampling of conditions at a particular site. Should conditions be encountered which differ from those described in this report, we require immediate notification in order to permit a re-evaluation of our recommendations.

APPENDIX

Symbols and Terms Used on the Borehole, Test Pit, and Monitor Well Records

SYMBOLS AND TERMS USED ON THE BOREHOLE, TEST PIT, AND MONITOR WELL RECORDS

SOIL DESCRIPTION

Behavioural properties (i.e. plasticity, permeability) take precedence over particle gradation in describing soils.

Terminology describing soil structure:

Desiccated	-	having visible signs of weathering by oxidation clay minerals, shrinkage, cracks, etc.
Fissured	-	having cracks, and hence a blocky structure
Varved	-	composed of regular alternating layers of silt and clay
Stratified	-	composed of alternating layers of different soil types, e.g. silt and sand or silt and clay
Well-graded	-	having wide range in grain sizes and substantial amounts of all intermediate particle sizes
Uniformly-graded	-	predominantly of one grain size.

Terminology used for describing soil strata based upon proportion of individual particle sizes present:

Trace, or occasional	-	less than 10%
Some	-	10% to 20%
Adjective (e.g. silty or sandy)	-	20% to 35%
And (e.g. silt and sand)	-	35% to 50%

The standard terminology to describe cohesionless soils include the relative density, as determined by laboratory test or by the Standard Penetration Test N-value: the number of blows of 140 pound (64 kg) hammer falling 30 inches (760 mm), required to drive a 2-inch (50.8 mm) O.D. split-spoon sampler 1 foot (305 mm) into the soil. On the records, where complete sampler penetration is not achieved and an N-value cannot be reported, the total number of blows are shown over actual penetration in millimetres (eg. 75/180).

Relative Density	N-value	Relative Density %
Very Loose	<4	<15
Loose	4 - 10	15 - 35
Compact	10 - 30	35 - 65
Dense	30 - 50	65 - 85
Very Dense	>50	>85

The standard terminology to describe cohesive soils include the consistency, which is based on undrained shear strength as measured by in situ vane tests, penetrometer tests, unconfined compression tests, or occasionally by Standard Penetration Tests.

Consistency	**Undrained Shear Strength**		**N-value**
	Kips/sq.ft.	**kPa**	
Very Soft	< 0.25	< 12.5	< 2
Soft	0.25 to 0.5	12.5 to 25	2 to 4
Firm	0.5 to 1.0	25 to 50	4 to 8
Stiff	1.0 to 2.0	50 to 100	8 to 15
Very Stiff	2.0 to 4.0	100 to 200	15 to 30
Hard	> 4.0	> 200	> 30

SAMPLES

SS	Split-spoon sample (obtained by performing the Standard Penetration Test)
AS	Auger sample
ST	Shelby tube or thin-wall tube
PS	Piston sample

BK	Bulk sample
WS	Wash sample
RC	Rock core AXT, BXL, etc. Rock core samples obtained with the use of standard diamond drilling bits.

OTHER TESTS

G	Specific Gravity
H	Hydrometer Analysis
S	Sieve Analysis
MC	Moisture Content
y	Unit Weight
C	Consolidation
CD	Consolidated drained triaxial

CU	Consolidated undrained triaxial with pore pressure measurements
UU	Unconsolidated undrained triaxial
RCC	Rock Core Compression
DS	Direct Shear
P	Field Permeability
TPH	Total Petroleum Hydrocarbons (ppm)
ND	Below Detection Limit

ROCK DESCRIPTION

The description of rock is based on the rock quality designation (RQD).

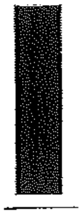
The classification is based on a modified core recovery percentage in which all pieces of sound core over 100 mm long are counted as recovery. The smaller pieces are considered to be due to close shearing, jointing, faulting, or weathering in the rock mass and are not counted. In most cases, RQD is run on NXL core; however, it can be used on different core sizes if the bulk of the fractures caused by drilling stresses are easily distinguishable from normal in situ fractures.

RQD	Rock Quality
90 to 100	excellent quality
75 to 90	good quality
50 to 75	fair quality
25 to 50	poor quality
< 25	very poor quality

CLASSIFICATION OF ROCK WITH REGARD TO STRENGTH

STRENGTH		FIELD IDENTIFICATION METHOD	RANGE OF UNCONFINED COMPRESSIVE STRENGTH (MPa)
Grade	Classification		
RO	Extremely weak	Indented by thumbnail	< 1
R1	Very weak	Crumbles under firm blows of geological hammer; can be peeled with a pocket-knife	1 - 5
R2	Weak rock	Can be peeled by a pocket-knife with difficulty; shallow indentations made by a firm blow with point of geological hammer	5 - 25
R3	Medium strong	Cannot be scraped or peeled with a pocket-knife; specimen can be fractured with a single firm blow of geological hammer	25 - 50
R4	Strong	Specimen requires more than one blow of geological hammer to fracture	50 - 100
R5	Very strong	Specimen requires many blows of geological hammer to fracture	100 - 250
R6	Extremely strong	Specimen can be chipped by geological hammer	> 250

STRATA PLOT


Asphalt

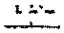

Concrete


Organics


Fill


Cobbles & Boulders


Gravel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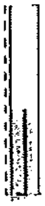
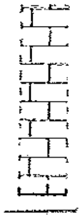

Silt


Clay


Igneous Bedrock


Sedimentary Bedrock


Metamorphic Bedrock

WATER LEVEL MEASUREMENT

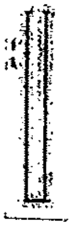

Borehole,
Standpipe or
Monitor Well
(at time of drilling)


Borehole,
Standpipe or
Monitor Well
(static conditions)

WELL CONSTRUCTION


Flush-mounted Well-head Enclosure


End/Top Cap


Solid PVC Pipe


Slotted PVC Pipe


Bentonite


Cave-in

Test Pit Records

TEST PIT RECORD

CLIENT Henry Bell Development Ltd.
PROJECT No. 27-6628-001.1
LOCATION Proposed Parking Garage - Duckworth Street @ Bell Street, St. John's, NL
TEST PIT No. TP 01
DATES (dd-mm-yy): DUG 29-05-10
WATER LEVEL 29-05-10
DATUM Geodetic

DEPTH (m)	ELEV. (m)	DESCRIPTION	STRATA PLOT	WATER LEVEL	SAMPLES TYPE	SAMPLES NUMBER	SAMPLES OTHER TESTS
0	15.78 15.7	**ORGANICS:** Grassmat and Topsoil.					
		FILL: Loose to compact greyish brown GRAVEL and SAND; trace silt; some organics (roots and rootlets); some debris (pieces of bricks, metal, concrete, rags, steel rods, bottles and an old telephone wire); some cobbles and boulders; moist.					
1							
	14.2	End of Test Pit					
2							
3							
4							
5							

NOTES:
1) Test pit terminated between 1.3 m (southern end) and 1.6 m (northern end) depths on probable bedrock.
2) Test pit dry at time of excavation.

Undrained Shear Strength, kPa
20 40 60 80

Water Content & Atterberg Limits W_p W W_l

10 20 30 40 50 60 70 80 90

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Technologist: B. Cameron
Reviewed By: Wm. Melendy
Contractor: Roley Construction Limited
Equipment: Cat 420E IT Extend-A-Hoe

△ Unconfined Compression Test
▼ Water Level at Time of Drilling/Excavation
▽ Static Water Level

TEST PIT RECORD

CLIENT Henry Bell Development Ltd.
LOCATION Proposed Parking Garage - Duckworth Street @ Bell Street, St. John's, NL
DATES (dd-mm-yy): DUG 29-05-10 WATER LEVEL 29-05-10

PROJECT No. 27-6628-001.1
TEST PIT No. TP 02
DATUM Geodetic

DEPTH (m)	ELEV. (m)	DESCRIPTION	STRATA PLOT	WATER LEVEL	SAMPLES TYPE	SAMPLES NUMBER	SAMPLES OTHER TESTS
0	16.42 16.4	**ASPHALT:**					
	15.9	**GRANULAR FILL:** Loose brownish grey GRAVEL and SAND; trace silt [Granular base (Class "A") and sub-base (Class "B") materials]; moist.					
1	15.3	**FILL:** Loose to compact greyish brown GRAVEL and SAND; trace silt; some debris (pieces of bricks and steel rods); some cobbles and boulders; moist.					
	15.2	**BEDROCK:** Shaley bedrock.					
		End of Test Pit					
2							
3							
4							
5							

NOTES:
1) Test pit terminated at 1.2 m depth in bedrock.
2) Test pit dry at time of excavation.
3) Concrete slab encountered at 0.7 m depth.

Undrained Shear Strength, kPa
20 40 60 80

Water Content & Atterberg Limits W_p W W_l

10 20 30 40 50 60 70 80 90

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Equipment: Cat 420E IT Extend-A-Hoe

△ Unconfined Compression Test
▼ Water Level at Time of Drilling/Excavation
▽ Static Water Level

TEST PIT RECORD

CLIENT Henry Bell Development Ltd.
LOCATION Proposed Parking Garage - Duckworth Street @ Bell Street, St. John's, NL
DATES (dd-mm-yy): DUG 29-05-10 WATER LEVEL 29-05-10
PROJECT No. 27-6628-001.1
TEST PIT No. TP 03
DATUM Geodetic

DEPTH (m)	ELEV. (m)	DESCRIPTION	STRATA PLOT	WATER LEVEL	SAMPLES TYPE	SAMPLES NUMBER	SAMPLES OTHER TESTS	Undrained Shear Strength, kPa 20 40 60 80 Water Content & Atterberg Limits W_p W W_l 10 20 30 40 50 60 70 80 90
0	16.52							
	16.5	**ASPHALT:**						
	16.1	**GRANULAR FILL:** Loose brownish grey GRAVEL and SAND; trace silt [Granular base (Class "A") and sub-base (Class "B") materials]; moist.						
1 2	13.7	**FILL:** Loose to compact greyish brown to brownish grey GRAVEL and SAND; trace silt; some debris (pieces of bricks, plastic, wood, metal, pipes, and concrete beams); occasional cobbles and boulders; moist.						
3 4 5		End of Test Pit						

NOTES:
1) Test pit terminated at 2.8 m depth on probable bedrock.
2) Test pit dry at time of excavation.
3) Concrete foundation wall encountered in northwestern corner of test pit.

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△ Unconfined Compression Test
▼ Water Level at Time of Drilling/Excavation
▽ Static Water Level

TEST PIT RECORD

CLIENT Henry Bell Development Ltd.
PROJECT No. 27-6628-001.1
LOCATION Proposed Parking Garage - Duckworth Street @ Bell Street, St. John's, NL
TEST PIT No. TP 04
DATES (dd-mm-yy): DUG 29-05-10
WATER LEVEL 29-05-10
DATUM Geodetic

DEPTH (m)	ELEV. (m)	DESCRIPTION	STRATA PLOT	WATER LEVEL	SAMPLES TYPE	SAMPLES NUMBER	SAMPLES OTHER TESTS
0	23.31	**ORGANICS:** Grassmat; Turf; and Topsoil.					
	22.9	**FILL:** Loose to compact greyish brown GRAVEL and SAND; trace silt; some organics (roots and rootlets); some debris (pieces of bricks, plastic, bottles and a chip bag); some cobbles and boulders; moist.					
	22.4	**TILL:** Compact to dense brownish grey to grey sandy GRAVEL; trace silt; occasional cobbles and boulders; moist.			BK	01	
	20.8	End of Test Pit					

Undrained Shear Strength, kPa: 20 40 60 80

Water Content & Atterberg Limits: W_p W W_l

10 20 30 40 50 60 70 80 90

Depth scale (m): 0 1 2 3 4 5

NOTES:
1) Test pit terminated at 2.5 m depth on probable bedrock.
2) Test pit dry at time of excavation.

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△ Unconfined Compression Test
▼ Water Level at Time of Drilling/Excavation
▽ Static Water Level

TEST PIT RECORD

CLIENT Henry Bell Development Ltd.
LOCATION Proposed Parking Garage - Duckworth Street @ Bell Street, St. John's, NL
DATES (dd-mm-yy): DUG 29-05-10 WATER LEVEL 29-05-10

PROJECT No. 27-6628-001.1
TEST PIT No. TP 05
DATUM Geodetic

DEPTH (m)	ELEV. (m)	DESCRIPTION	STRATA PLOT	WATER LEVEL	SAMPLES TYPE	SAMPLES NUMBER	SAMPLES OTHER TESTS
0	18.14	**ORGANICS:** Grassmat and Topsoil.	▨				
	17.9	End of Test Pit **NOTES:** 1) Test pit terminated at 0.2 m depth on probable bedrock. 2) Test pit dry at time of excavation.					
1							
2							
3							
4							
5							

Undrained Shear Strength, kPa: 20 40 60 80

Water Content & Atterberg Limits W_p W W_l

10 20 30 40 50 60 70 80 90

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Technologist: B. Cameron
Reviewed By: Wm. Melendy
Contractor: Roley Construction Limited
Equipment: Cat 420E IT Extend-A-Hoe

△ Unconfined Compression Test
▼ Water Level at Time of Drilling/Excavation
▽ Static Water Level

TEST PIT RECORD

CLIENT Henry Bell Development Ltd.

PROJECT No. 27-6628-001.1

LOCATION Proposed Parking Garage - Duckworth Street @ Bell Street, St. John's, NL

TEST PIT No. TP 06

DATES (dd-mm-yy): DUG 29-05-10

WATER LEVEL 29-05-10

DATUM Geodetic

DEPTH (m)	ELEV. (m)	DESCRIPTION	STRATA PLOT	WATER LEVEL	SAMPLES TYPE	SAMPLES NUMBER	SAMPLES OTHER TESTS	Undrained Shear Strength, kPa 20 40 60 80 Water Content & Atterberg Limits Wp W Wl 10 20 30 40 50 60 70 80 90
0	18.83	**ORGANICS:** Grassmat and Topsoil.						
	18.7	**FILL:** Loose to compact greyish brown to brownish grey to grey GRAVEL and SAND; trace silt; some organics (roots and rootlets) and some debris (pieces of bricks, metal, pipe, and wood) to 1.5 m depth; some cobbles and boulders; moist to wet.	▨					
1			▨					
2			▨		BK	01		
	15.9	End of Test Pit						
3								
4								
5								

NOTES:
1) Test pit terminated at 2.9 m depth on probable bedrock.
2) Test pit dry at time of excavation.
3) A PVC storm sewer pipe was encountered at 1.3 m depth.

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Equipment: Cat 420E IT Extend-A-Hoe

△ Unconfined Compression Test
▼ Water Level at Time of Drilling/Excavation
▽ Static Water Level

TEST PIT RECORD

CLIENT Henry Bell Development Ltd.
LOCATION Proposed Parking Garage - Duckworth Street @ Bell Street, St. John's, NL
DATES (dd-mm-yy): DUG 29-05-10 WATER LEVEL 29-05-10
PROJECT No. 27-6628-001.1
TEST PIT No. TP 07
DATUM Geodetic

DEPTH (m)	ELEV. (m)	DESCRIPTION	STRATA PLOT	WATER LEVEL	SAMPLES TYPE	SAMPLES NUMBER	SAMPLES OTHER TESTS	Undrained Shear Strength, kPa (20 40 60 80) Water Content & Atterberg Limits Wp W Wl (10 20 30 40 50 60 70 80 90)
0	18.63 18.6	**ASPHALT:**						
	18.2	**GRANULAR FILL:** Loose brownish grey GRAVEL and SAND; trace silt [Granular base (Class "A") and sub-base (Class "B") materials]; moist.						
1		**FILL:** Loose to compact greyish brown GRAVEL and SAND; trace silt; some debris (pieces of bricks, plastic, metal, wood, rags, pipes, old wires, ash/soot and concrete beams); occasional cobbles and boulders; moist.						
2	16.6	**TILL:** Compact to very dense brownish grey to grey GRAVEL and SAND; trace silt; occasional cobbles and boulders; moist.						
3					BK	01	MC, S	○
	14.9	End of Test Pit						
4		**NOTES:** 1) Test pit terminated at 3.7 m depth on probable bedrock. 2) Test pit dry at time of excavation.						
5								

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Contractor: Roley Construction Limited
Equipment: Cat 420E IT Extend-A-Hoe

△ Unconfined Compression Test
▼ Water Level at Time of Drilling/Excavation
▽ Static Water Level

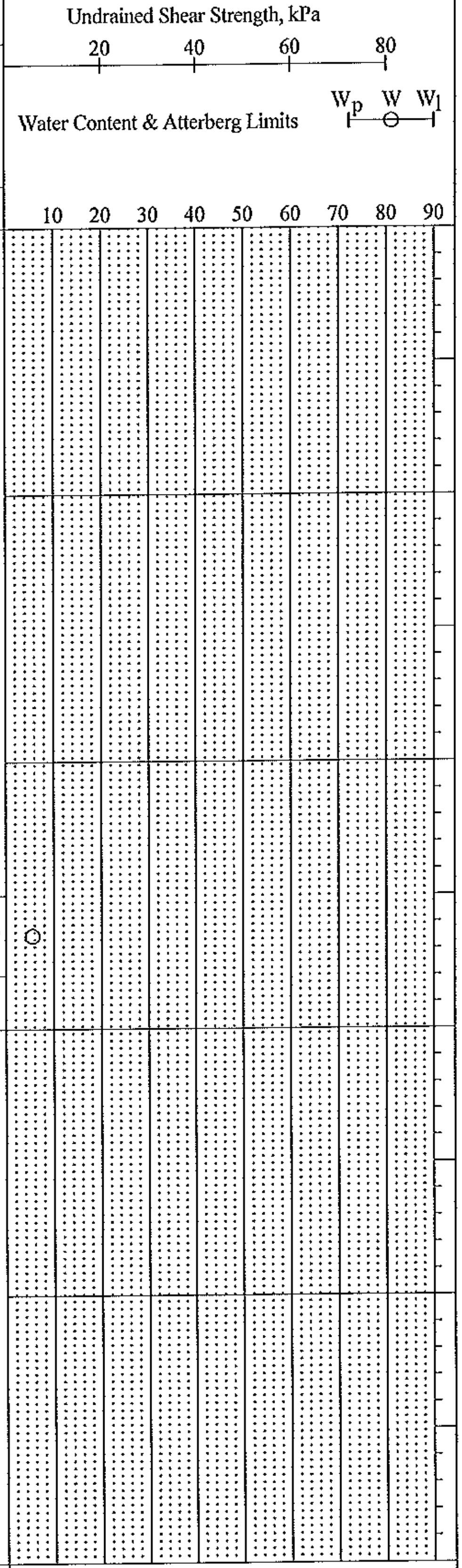
TEST PIT RECORD

CLIENT Henry Bell Development Ltd.
LOCATION Proposed Parking Garage - Duckworth Street @ Bell Street, St. John's, NL
DATES (dd-mm-yy): DUG 29-05-10 WATER LEVEL 29-05-10

PROJECT No. 27-6628-001.1
TEST PIT No. TP 08
DATUM Geodetic

DEPTH (m)	ELEV. (m)	DESCRIPTION	STRATA PLOT	WATER LEVEL	SAMPLES TYPE	SAMPLES NUMBER	SAMPLES OTHER TESTS
0	23.98	**FILL:** Loose to compact greyish brown GRAVEL and SAND; trace silt; some organics (roots and rootlets); some debris (pieces of bricks, metal, and concrete); some cobbles and boulders; moist.	▨				
1	23.0	**TILL:** Compact to dense brownish grey to grey sandy GRAVEL; trace silt; occasional cobbles and boulders; moist.					
					BK	01	MC, S
3	21.0	End of Test Pit					

NOTES:
1) Test pit terminated at 3.0 m depth on probable bedrock. Shaley bedrock encountered at 1.9 m depth at western end of test pit.
2) Test pit dry at time of excavation.



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Contractor: Roley Construction Limited
Equipment: Cat 420E IT Extend-A-Hoe

△ Unconfined Compression Test
▼ Water Level at Time of Drilling/Excavation
▽ Static Water Level

TEST PIT RECORD

CLIENT Henry Bell Development Ltd.
LOCATION Proposed Parking Garage - Duckworth Street @ Bell Street, St. John's, NL
DATES (dd-mm-yy): DUG 29-05-10 WATER LEVEL 29-05-10

PROJECT No. 27-6628-001.1
TEST PIT No. TP 09
DATUM Geodetic

DEPTH (m)	ELEV. (m)	DESCRIPTION	STRATA PLOT	WATER LEVEL	SAMPLES		
					TYPE	NUMBER	OTHER TESTS
0	20.28						
	20.1	**FILL:** Loose to compact greyish brown GRAVEL and SAND; trace silt; trace debris (pieces of bricks); occasional cobbles and boulders; moist.			BK	01	
	19.8	**BEDROCK:** Shaley bedrock.					
		End of Test Pit					
1							
2							
3							
4							
5							

NOTES:
1) Test pit terminated at 0.5 m depth in bedrock.
2) Test pit dry at time of excavation.

Undrained Shear Strength, kPa
20 40 60 80

Water Content & Atterberg Limits W_p W W_l

10 20 30 40 50 60 70 80 90

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Contractor: Roley Construction Limited
Equipment: Cat 420E IT Extend-A-Hoe

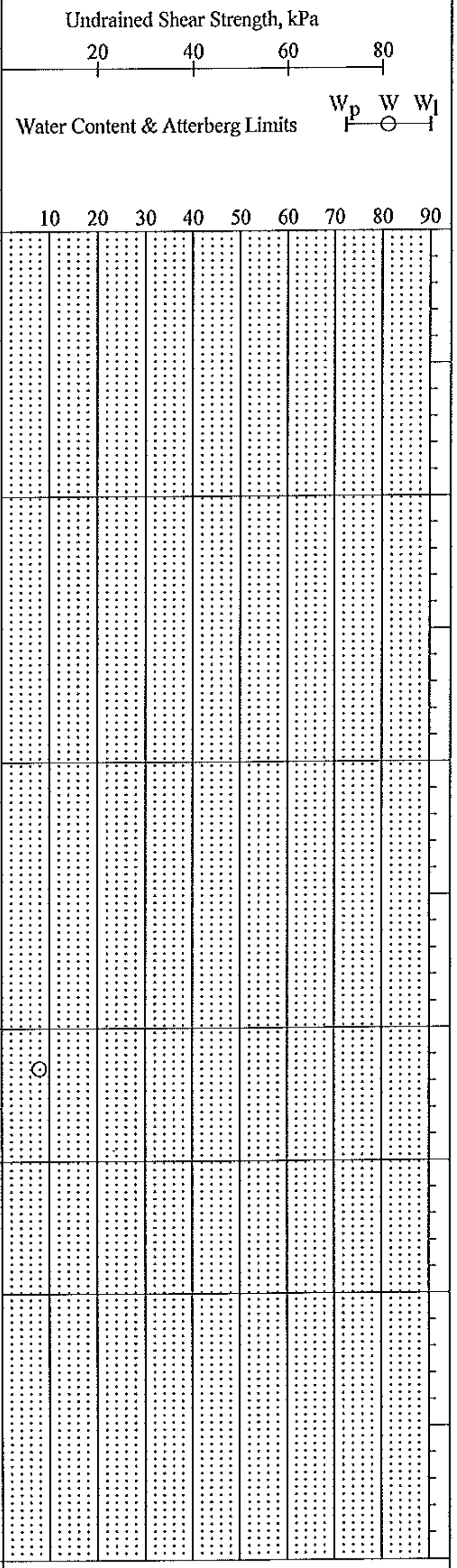
△ Unconfined Compression Test
▼ Water Level at Time of Drilling/Excavation
▽ Static Water Level

TEST PIT RECORD

CLIENT Henry Bell Development Ltd.
LOCATION Proposed Parking Garage - Duckworth Street @ Bell Street, St. John's, NL
DATES (dd-mm-yy): DUG 29-05-10
WATER LEVEL 29-05-10
PROJECT No. 27-6628-001.1
TEST PIT No. TP 10
DATUM Geodetic

DEPTH (m)	ELEV. (m)	DESCRIPTION	TYPE	NUMBER	OTHER TESTS
0	20.21	**ORGANICS:** Grassmat and Topsoil.			
	20.0	**FILL:** Loose greyish brown COBBLES (rock fill) intermixed with GRAVEL and SAND; trace silt; some debris (pieces of bricks and bottles and a piece of old sewer pipe from a former house); moist.			
	19.4	**FILL:** Loose to compact greyish brown GRAVEL and SAND; trace silt; some cobbles and boulders; moist.			
	17.6	**TILL:** Compact to dense brownish grey to grey GRAVEL and SAND; trace silt; occasional cobbles and boulders; moist.	BK	01	MC, S
	16.7	End of Test Pit			

NOTES:
1) Test pit terminated at 3.5 m depth on probable bedrock.
2) Test pit dry at time of excavation.



Technologist: B. Cameron
Reviewed By: Wm. Melendy
Contractor: Roley Construction Limited
Equipment: Cat 420E IT Extend-A-Hoe

△ Unconfined Compression Test
▼ Water Level at Time of Drilling/Excavation
▽ Static Water Level

TEST PIT RECORD

CLIENT Henry Bell Development Ltd.
LOCATION Proposed Parking Garage - Duckworth Street @ Bell Street, St. John's, NL
DATES (dd-mm-yy): DUG 29-05-10 WATER LEVEL 29-05-10

PROJECT No. 27-6628-001.1
TEST PIT No. TP 11
DATUM Geodetic

SAMPLES: TYPE | NUMBER | OTHER TESTS

Undrained Shear Strength, kPa: 20, 40, 60, 80

Water Content & Atterberg Limits: W_p W W_l

10 20 30 40 50 60 70 80 90

DEPTH (m)	ELEV. (m)	DESCRIPTION
0	21.14	
	21.1	**ASPHALT:**
		GRANULAR FILL: Loose brownish grey GRAVEL and SAND; trace silt [Granular base (Class "A") and sub-base (Class "B") materials]; moist.
	20.7	
	20.5	**FILL:** Loose greyish brown to brownish grey COBBLES (rock fill) intermixed with GRAVEL and SAND; trace silt; moist.
		FILL: Loose to compact greyish brown GRAVEL and SAND; trace silt; some cobbles and boulders; moist.
1		
	19.5	
		TILL: Compact to dense brownish grey to grey GRAVEL and SAND; trace silt; occasional cobbles and boulders; moist.
	19.2	
2		End of Test Pit
3		
4		
5		

NOTES:

1) Test pit terminated at 1.9 m depth on probable bedrock.
2) Test pit dry at time of excavation.

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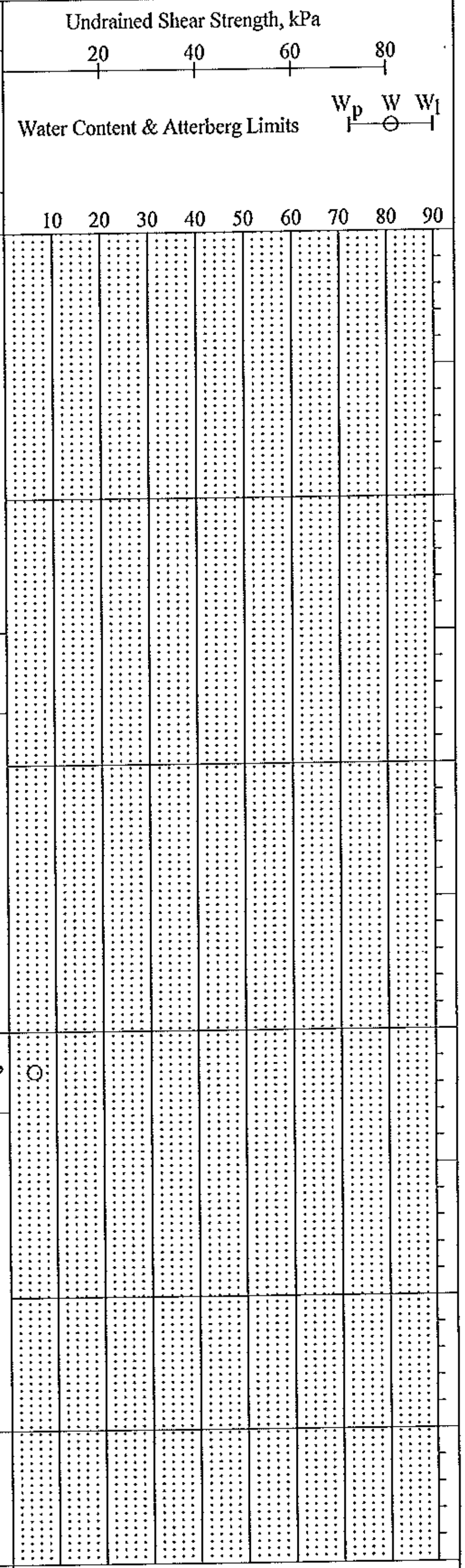
△ Unconfined Compression Test
▼ Water Level at Time of Drilling/Excavation
▽ Static Water Level

TEST PIT RECORD

CLIENT Henry Bell Development Ltd.
LOCATION Proposed Parking Garage - Duckworth Street @ Bell Street, St. John's, NL
DATES (dd-mm-yy): DUG 29-05-10 WATER LEVEL 29-05-10

PROJECT No. 27-6628-001.1
TEST PIT No. TP 12
DATUM Geodetic

DEPTH (m)	ELEV. (m)	DESCRIPTION	SAMPLES TYPE	SAMPLES NUMBER	SAMPLES OTHER TESTS
0	22.25	**ORGANICS:** Grassmat and Topsoil.			
1	22.1	**FILL:** Loose greyish brown GRAVEL and SAND; trace silt; some cobbles and boulders; moist.			
	21.1	**SAND and GRAVEL:** Compact tannish brown SAND and GRAVEL; trace silt; occasional cobbles and boulders; moist.	BK	01	
2 3 4	20.4	**TILL:** Compact to very dense brownish grey to grey SAND and GRAVEL; trace silt; occasional cobbles and boulders; moist.	BK	02	MC, S
5	17.8	End of Test Pit **NOTES:** 1) Test pit terminated at 4.5 m depth on probable bedrock. 2) Test pit dry at time of excavation.			



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Equipment: Cat 420E IT Extend-A-Hoe

△ Unconfined Compression Test
▼ Water Level at Time of Drilling/Excavation
▽ Static Water Level

GEOTECH 6628001.GPJ ADI.GDT 8/6/10

TEST PIT RECORD

CLIENT Henry Bell Development Ltd.
LOCATION Proposed Parking Garage - Duckworth Street @ Bell Street, St. John's, NL
DATES (dd-mm-yy): DUG 29-05-10 WATER LEVEL 29-05-10

PROJECT No. 27-6628-001.1
TEST PIT No. TP 13
DATUM Geodetic

DEPTH (m)	ELEV. (m)	DESCRIPTION	STRATA PLOT	WATER LEVEL	SAMPLES TYPE	SAMPLES NUMBER	SAMPLES OTHER TESTS	Undrained Shear Strength, kPa 20 40 60 80 / Water Content & Atterberg Limits Wp W Wl 10 20 30 40 50 60 70 80 90
0	21.55	**ORGANICS:** Grassmat and Topsoil.						
	21.4	**FILL:** Loose greyish brown GRAVEL and SAND; trace silt; some debris (pieces of bricks); occasional cobbles and boulders; moist.						
	21.1	**TILL:** Compact to very dense brownish grey to grey SAND and GRAVEL; trace silt; occasional cobbles and boulders; moist.						
1								
					BK	01		
2								
	19.2	End of Test Pit **NOTES:** 1) Test pit terminated at 2.4 m depth on probable bedrock. Knob of bedrock encountered at 1.6 m depth on west side of test pit. 2) Test pit dry at time of excavation.						
3								
4								
5								

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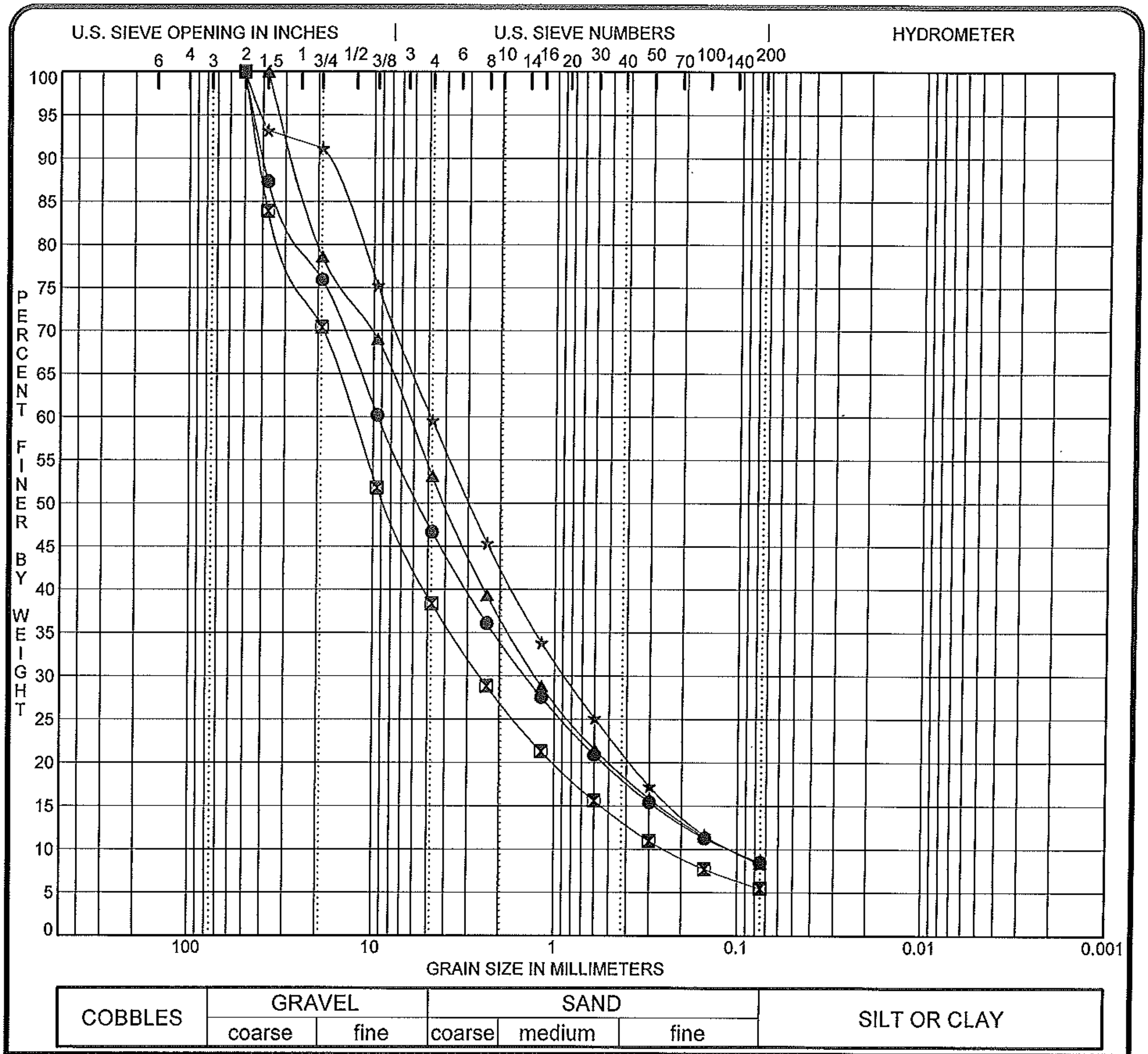
Reviewed By: Wm. Melendy
Contractor: Roley Construction Limited
Equipment: Cat 420E IT Extend-A-Hoe

△ Unconfined Compression Test
▼ Water Level at Time of Drilling/Excavation
▽ Static Water Level

GEOTECH 66280011.GPJ ADI.GDT 22/6/10

Gradation Curves



	Location	Depth (m)	Classification (USCS)	MC%	LL	PL	PI	Cc	Cu
●	**TP 07**	**3.2**	**TILL: GRAVEL and SAND; trace silt (GW-GM)**	**7.3**				**2.03**	**86.5**
☒	**TP 08**	**2.7**	**TILL: Sandy GRAVEL; trace silt (GW-GM)**	**5.5**				**2.09**	**52.9**
▲	**TP 10**	**3.2**	**TILL: GRAVEL and SAND; trace silt (GW-GM)**	**7.9**				**2.35**	**59.4**
★	**TP 12**	**3.2**	**TILL: SAND and GRAVEL; trace silt (SW-SM)**	**5.3**				**1.48**	**45.1**

	Location	Depth (m)	D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	**TP 07**	**3.2**	**50.00**	**9.39**	**1.437**	**0.1085**	**53.3**	**38.2**	**8.5**	
☒	**TP 08**	**2.7**	**50.00**	**12.89**	**2.564**	**0.2435**	**61.6**	**32.9**	**5.5**	
▲	**TP 10**	**3.2**	**37.50**	**6.40**	**1.275**	**0.1078**	**46.8**	**44.8**	**8.4**	
★	**TP 12**	**3.2**	**50.00**	**4.83**	**0.875**	**0.1071**	**40.4**	**51.3**	**8.3**	

PROJECT **Henry Bell Development Ltd. - Proposed Parking Garage - Duckworth Street @ Bell Street, St. John's, NL**

JOB NO. **27-6628-001.1**
DATE **02-06-10**

GRADATION CURVES

ADI Limited
St. John's, NL

ADI Limited Letter-form Report:

Geotechnical Sub-surface Investigation
Proposed parking Garage
Duckworth at Bell Street
St. John's, Newfoundland and Labrador
Date: February 14, 2011

FILE COPY

ADI Limited
A Trow Global Company

Architecture
Engineering
Consulting
Procurement
Project Management

February 14, 2011

ADI File 27-6628-001.1

Henry Bell Development Ltd.
12 Caldwell Place
St. John's, Newfoundland Labrador
A1E 6A4

Attention: Mr. William Clarke

Dear Sirs:

RE: **Geotechnical Sub-surface Investigation**
Proposed Parking Garage
Duckworth Street at Bell Street
St. John's, Newfoundland and Labrador

Acting on the request of Mr. William Clarke of *Henry Bell Development Ltd.*, *ADI Limited* has completed a follow-up geotechnical sub-surface investigation to determine the quality of the bedrock encountered during the initial geotechnical test pit sub-surface investigation completed on May 29, 2010, and reported in *ADI's* report dated June 15, 2010.

Field Program

Field work was completed on January 22 and 23, 2011, and consisted of three boreholes (BH), advanced using a *CME-55* drill rig operated by *Logan Geotech Inc.* of Dieppe, New Brunswick. The boreholes were located in the field by *ADI Limited* in discussion with Mr. Clarke. Final borehole locations are shown on *Figure 1: Borehole and Test Pit Location Plan.* Twenty-five-millimeter-diameter PVC stand-pipes were installed in each borehole to allow for static groundwater level measurements.

Conditions encountered in the boreholes are described below and on the Borehole Records attached. *Table 1: Summary of Borehole Data* presents the summarized findings of the investigation.

Bedrock

Bedrock was encountered beneath the overburden layer (Fill or Till) in BH01, BH02, and BH03 at depths of 1.8 m, 5.4 m, and 1.1 m, respectively, below the existing ground surface. All boreholes were terminated in the bedrock at depths ranging from 4.1 m to 9.8 m below the existing ground surface (approximate elevation 13.0 m geodetic).

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Table 1: SUMMARY OF BOREHOLE DATA
Geotechnical Sub-surface Investigation
Proposed Parking Garage - Duckworth Street at Bell Street
St. John's, Newfoundland and Labrador

Borehole No.	Ground Surface Elevation (m)	Bottom of Overburden Elevation (m)	Top of Bedrock Elevation (m)	End of Borehole Elevation (m)	Ground-Water Elevation (m)
BH01	17.77	16.0	16.0	13.7	16.9
BH02	21.63	16.2	16.2	12.6	19.2
BH03	22.76	21.7	21.7	13.0	21.3

NOTES:
Elevations provided by *Brown and Way Surveys*
N/E: not-encountered

Publisher geology for the area indicates the bedrock consists of thin lenticular-bedded, dark grey sandstone and minor shale of the Renews Head Formation, St. John's Group. Bedrock is variable, but may be described as a medium grey sandstone. Rock quality designations (RQD) ranged from 0 to 68 and, on this basis, the bedrock may be characterized as very-severely-fractured to fractured.

Compressive testing was completed on nine selected bedrock core samples. Results are presented in *Table 2: Bedrock Compressive Strength Results.*

Table 2: BEDROCK COMPRESSIVE STRENGTH RESULTS
Proposed Parking Garage - Duckworth Street at Bell Street
St. John's, Newfoundland and Labrador

Borehole Location	Sample Depth (m)	Compressive Strength (MPa)
BH01-01	3.0 m to 3.2 m	Rock core broke prematurely
BH01-02	3.6 m to 3.8 m	18.4
BH02-01	3.5 m to 3.8 m	35.2
BH02-02A	8.2 m to 8.5 m	39.1
BH03-02B	8.5 m to 8.8 m	39.7
BH03-01	3.2 m to 3.5 m	25.9
BH03-02	4.7 m to 5.0 m	61.5
BH03-03A	7.6 m to 7.9 m	46.1
BH03-03B	7.9 m to 8.2 m	35.1
BH03-04	9.1 m to 9.3 m	27.1

Groundwater

Twenty-five-millimetre-diameter stand-pipes were installed in each of the three boreholes. Depth to groundwater obtained on January 26, 2011, were as follows: BH01, 1.0 m; BH02, 2.5 m; BH03, 1.8 m, below the existing ground surface. Note that each borehole location (stand-pipe) was purged and allowed to obtain its static level prior to measurement.

Discussion

Groundwater was not encountered in test pits excavated during the June 2010 geotechnical investigation. Recent boreholes installed at the site have confirmed groundwater elevations ranging from 16.9 m to 21.3 m. Project drawings dated June 12, 2010, Typical Building Section, indicate a proposed finish Level 1 floor elevation of 14.3 m. Based on the findings of the follow-up borehole program, *ADI* recommends that drainage piping be constructed immediately adjacent all perimeter footings, with positive drainage into the municipal storm system, and an under-drain system be installed beneath the slabs at the lowest levels. Typically, this would consist of a series of perforated drainage pipes placed just above the bedrock surface, and backfilled with free-draining gravels. The perforated piping should be connected to a non-perforated header pipe(s) to direct the flow out of the building. It is recommended that PVC piping be used for drainage works and that the system be designed to ensure positive flow to the municipal storm system. A minimum slope of one percent is recommended for perimeter and under-drain piping. The invert of all sections of drainage piping should be at least 300 mm below the top of slab level. Traps should be installed for future inspection and cleaning, and backwater flow valves should be installed to prevent water from entering the system from outside sources.

Bedrock encountered at the site is classified as very-severely-fractured to fractured in terms of rock quality. Based on the bedrock quality, bedrock removal using a hydraulic rock buster should be achieved at the site. Bedrock quality typically encountered in the upper 2.0 m to 3.6 m is considered very-severely-fractured, becoming more sound with depth.

A subsurface investigation is a limited sampling of conditions at a particular site. Should conditions be encountered which differ from those described in this letter-report, we require immediate notification in order to permit a re-evaluation of our recommendations.

We trust this submission meets your current requirements. Should you have any questions or require clarification on any aspect of this report, please do not hesitate to contact our office.

Yours very truly,

ADI Limited

Blair D Cameron

Blair D. Cameron, P.Tech.
Sr. Geo-Environmental Tech.

William G. Melendy, M.A.Sc., P.Eng.
Group Manager,
Geotechnical/Environmental Engineering

BDC/WGM:dgn

Attachments: Figure 1: Borehole and Test Pit Location Plan
Borehole Records (3)

o:/#Project Files/#Client 6601 to 7000/6628/6628001I/Reports/Rock Core Letter-Report

LEGEND

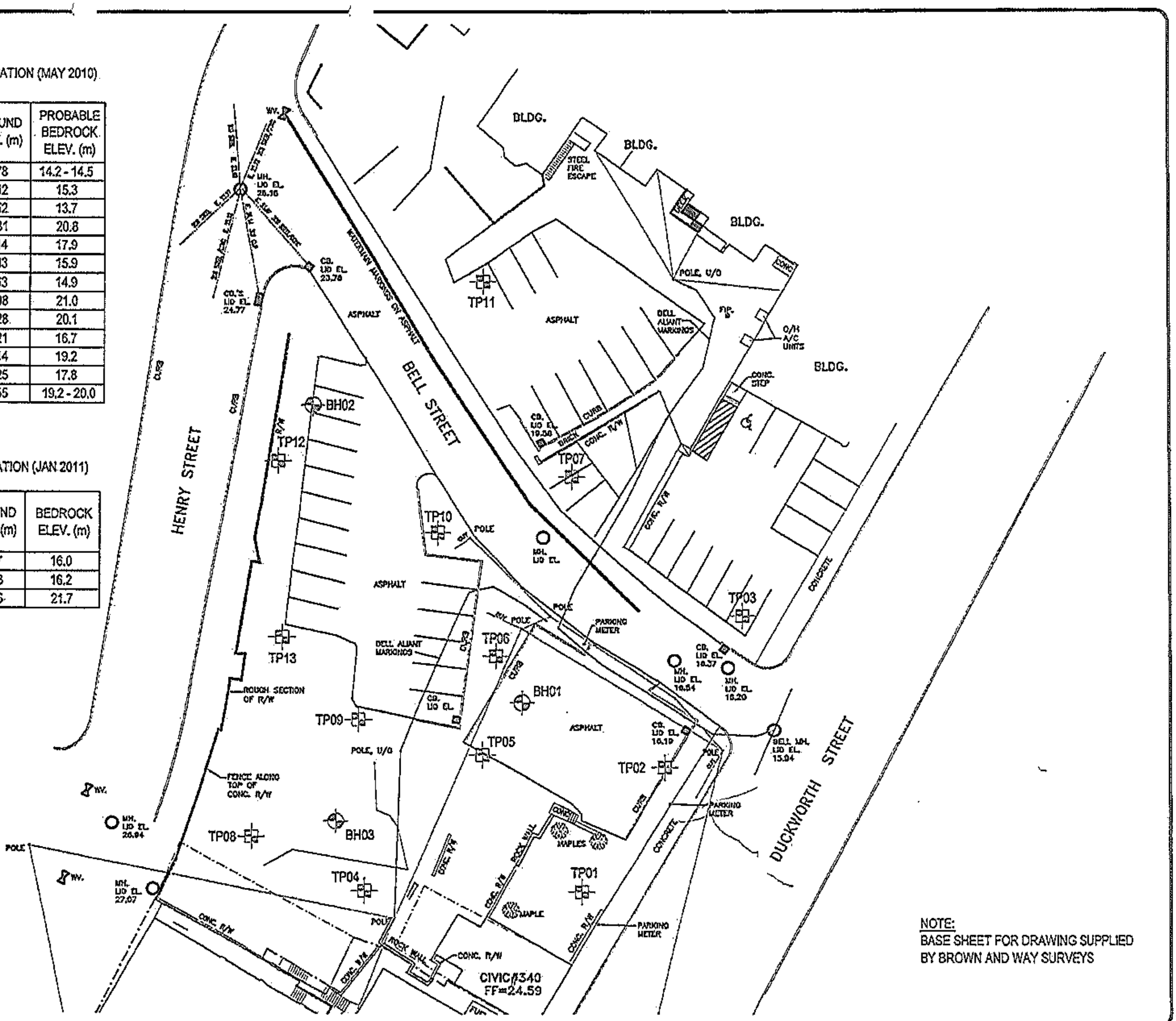
- TEST PIT LOCATION (MAY 2010)

TEST PIT No.	GROUND ELEV. (m)	PROBABLE BEDROCK ELEV. (m)
TP01	15.78	14.2 - 14.5
TP02	16.42	15.3
TP03	16.52	13.7
TP04	23.31	20.8
TP05	18.14	17.9
TP06	18.83	15.9
TP07	18.63	14.9
TP08	23.98	21.0
TP09	20.28	20.1
TP10	20.21	16.7
TP11	21.14	19.2
TP12	22.25	17.8
TP13	21.55	19.2 - 20.0

LEGEND

- TEST PIT LOCATION (JAN 2011)

BOREHOLE No.	GROUND ELEV. (m)	BEDROCK ELEV. (m)
BH01	17.77	16.0
BH02	21.63	16.2
BH03	22.76	21.7



NOTE:
BASE SHEET FOR DRAWING SUPPLIED
BY BROWN AND WAY SURVEYS

ADI Limited © 2011

No.	Issue	Date

No.	Revision	Ckd. By	Date

Const. North

Drawn By: T.S.
Dwg. Standards Ckd. By:
Designed By: B.D.C.
Date Printed JAN, 2011
Dwg. Design Ckd. By:

ADI Limited
St. John's, NL, Canada
Engineering, Consulting, Procurement
and Project Management

Project Title
GEOTECHNICAL INVESTIGATION
PROPOSED PARKING GARAGE
DUCKWORTH STREET & BELL STREET
ST. JOHN'S, NL

Dwg. Title
BOREHOLE AND TEST PIT
LOCATION PLAN

Project No. 27-6628-001.1
Dwg. No. FIGURE 1
Rev. No.
Scale 1 : 400
This drawing is not to be scaled

Offices located in:
Charlottetown, Moncton, Saint John, Truro, Halifax, Sydney
Port Hawkesbury, St.John's, Fredericton and Salem, NH

BOREHOLE RECORD

CLIENT ______

LOCATION Proposed Parking Garage - Duckworth Street at Bell Street, St. John's, NL

DATES (dd-mm-yy): BORING 22-1-11 WATER LEVEL 11-1-26

PROJECT No. 27-6628-001.1

BOREHOLE No. BH 01

DATUM Geodetic

DEPTH (m)	ELEV. (m)	DESCRIPTION	STRATA PLOT	WATER LEVEL	SAMPLES				CHEMICAL ANALYSES (mg/kg)					
									Laboratory					
					TYPE	NUMBER	RECOVERY	N-VALUE OR RQD	Benzene	Toluene	Ethyl Benzene	Xylene	TPH	Headspace Levels (ppm)
	17.77						mm							
0	17.7	**ASPHALT:**	▨											
	17.3	**GRANULAR FILL:** Loose brownish grey GRAVEL and SAND; trace silt [Granular base (Class "A") and sub-base (Class "B") materials]; moist.	▨	▼ ▽										
1	16.0	**FILL:** Loose to compact greyish brown GRAVEL and SAND; trace silt; some cobbles and boulders; moist.	▨											
2		**BEDROCK:** Very severely fractured to severely fractured SANDSTONE.	▤		RC	01	100%	0						
			▤		RC	02	100%	14						
3			▤		RC	03	100%	38						
4	13.7	End of Borehole												

NOTES:

1) Soil samples not collected. Soil strata and depths obtained from TP02 excavated on May 29, 2010.

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ADI Limited
60 Pippy Place, Suite 200
A1B 4H7
Tel 709.579.2027
Fax 709.579.7115

Technologist: B. Cameron
Reviewed By: Wm. Melendy
Contractor: Logan Geotech Inc.
Equipment: CME-55

▼ Water Level at Time of Drilling/Excavation

▽ Static Water Level

BOREHOLE RECORD

CLIENT
PROJECT No. 27-6628-001.1
LOCATION Proposed Parking Garage - Duckworth Street at Bell Street, St. John's, NL
BOREHOLE No. BH 02
DATES (dd-mm-yy): BORING 21-1-11 WATER LEVEL 11-1-26
DATUM Geodetic

DEPTH (m)	ELEV. (m)	DESCRIPTION	STRATA PLOT	WATER LEVEL	SAMPLES TYPE	SAMPLES NUMBER	SAMPLES RECOVERY mm	SAMPLES N-VALUE OR RQD	CHEMICAL ANALYSES (mg/kg) Laboratory Benzene	Toluene	Ethyl Benzene	Xylene	TPH	Headspace Levels (ppm)
0 1	21.63	**FILL:** Loose greyish brown GRAVEL and SAND; trace silt; some cobbles and boulders; moist.	▨											
	20.4	**SAND and GRAVEL:** Compact tannish brown SAND and GRAVEL; trace silt; occasional cobbles and boulders; moist.	░											
2 3 4 5	19.7	**TILL:** Compact to very dense brownish grey to grey SAND and GRAVEL; trace silt; some cobbles and boulders - becoming cobbley and bouldery below 2.5 m depth; moist.	▒	▼										
	16.2	**BEDROCK:** Very severely fractured to fractured SANDSTONE.	▦		RC	01	100%	0						
6			▦		RC	02	100%	0						
7			▦		RC	03	100%	21						
8			▦		RC	04	100%	68						
9 10 11 12	12.6	End of Borehole **NOTES:** 1) Soil samples not collected. Soil strata and depths obtained from TP12 excavated on May 29, 2010.												

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Reviewed By: Wm. Melendy
Contractor: Logan Geotech Inc.
Equipment: CME-55

▼ Water Level at Time of Drilling/Excavation
▽ Static Water Level

BOREHOLE RECORD

CLIENT ______
LOCATION Proposed Parking Garage - Duckworth Street at Bell Street, St. John's, NL
DATES (dd-mm-yy): BORING 22-1-11 WATER LEVEL 11-1-26
PROJECT No. 27-6628-001.1
BOREHOLE No. BH 03
DATUM Geodetic

DEPTH (m)	ELEV. (m)	DESCRIPTION	STRATA PLOT	WATER LEVEL	SAMPLES				CHEMICAL ANALYSES (mg/kg) Laboratory					
					TYPE	NUMBER	RECOVERY	N-VALUE OR RQD	Benzene	Toluene	Ethyl Benzene	Xylene	TPH	Headspace Levels (ppm)
	22.76						mm							
0	22.4	**ORGANICS:** Grassmat; Turf; and Topsoil.												
	21.7	**FILL:** Loose to compact greyish brown GRAVEL and SAND; trace silt; moist.		▼										
1		**BEDROCK:** Very severely fractured to fractured SANDSTONE.		▽										
2					RC	01	100%	38						
3					RC	02	100%	0						
4					RC	03	100%	12						
5					RC	04	100%	30						
6														
7					RC	05	100%	65						
8					RC	06	100%	43						
9					RC	07	100%	62						
	13.0	End of Borehole												
10		**NOTES:** 1) Soil samples not collected. Soil strata and depths obtained from TP04 excavated on May 29, 2010.												
11														
12														

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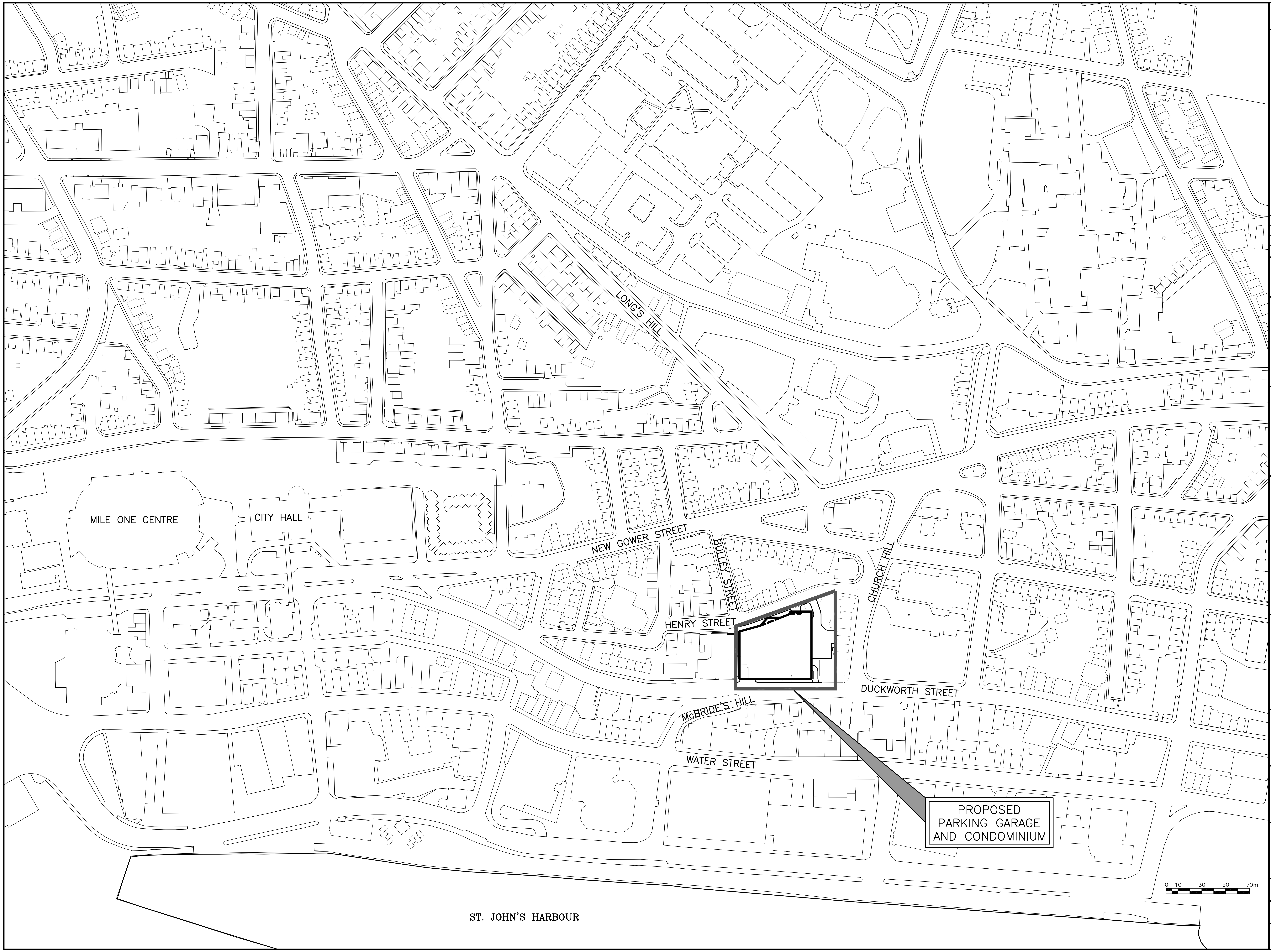
Technologist: B. Cameron
Reviewed By: Wm. Melendy
Contractor: Logan Geotech Inc.
Equipment: CME-55

▼ Water Level at Time of Drilling/Excavation
▽ Static Water Level

Appendix B

Drawings Received from Clients



GENERAL NOTES

No.	REVISIONS	BY	DATE
C	REVISED FOR APPROVALS	D.J.M.	02/17/12
B	ISSUED FOR PHASE 1 CONSTRUCTION	D.J.M.	11/30/11
A	ISSUED FOR CLIENT REVIEW	D.J.M.	08/31/11

A — PLAN, SECTION, ELEVATION, OR DETAIL No.
B — No. OF DRAWING WHERE ABOVE IS DRAWN

REFERENCE:

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PETER A. HUTTON
SIGNATURE
02/17/12
DATE

PERMIT STAMP:
PROVINCE OF NEWFOUNDLAND
PERMIT HOLDER
CLASS "A"
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PETER A. HUTTON
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Permit No. as issued by APEGN D0084
which is valid for the year 2012

DYNAMIC
DEVELOPMENT
SERVICES LTD.

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E-MAIL: djmadden@nl.rogers.com

ENGINEER/ARCHITECT:

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322 WATERFORD BRIDGE ROAD
ST. JOHN'S, NF
A1E 1E9

TEL. /FAX. (709) 368-8547 CELL (709) 685-3671

OWNER/CLIENT NAME:
HENRY BELL
DEVELOPMENTS LTD.

PROJECT TITLE:
DUCKWORTH STREET
PARKING GARAGE
AND CONDOMINIUM

DRAWING TITLE:
LOCATION PLAN

DRAWN/DESIGNED BY: D.J.M.
DATE: JULY 2010
APPROVED BY: P.A.H.
SCALE: AS SHOWN
PROJECT No.: 10208
DRAWING No.: C1
REV.: C

LEGEND

—●—	NEW MANHOLE
—○—	EXISTING MANHOLE
—■—	NEW CATCH BASIN
—□—	EXISTING CATCH BASIN
—⋈—	NEW GATE VALVE
—⋈—	EXISTING GATE VALVE
—Ⓒ—	CURB STOP
—+—	CORPORATION STOP
—┴—	TEE
——	WATER MAIN
——	SANITARY SEWER MAIN
— —	STORM SEWER MAIN
—x—x—	FENCE
—W—W—	RETAINING WALL
—C—C—	U/G CABLE
100.00	EXISTING ELEVATION

GENERAL NOTES

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3. CONTRACTOR SHALL CONFIRM LOCATION, ALIGNMENT, INVERTS AND TYPE OF MATERIAL OF ALL UNDERGROUND SERVICES AND SHALL NOTIFY THE ENGINEER/OWNER PRIOR TO COMMENCING ANY WORK.

No.	REVISIONS	BY	DATE
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ST. JOHN'S, NF
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TEL. /FAX. (709) 368-8547 CELL (709) 685-3671

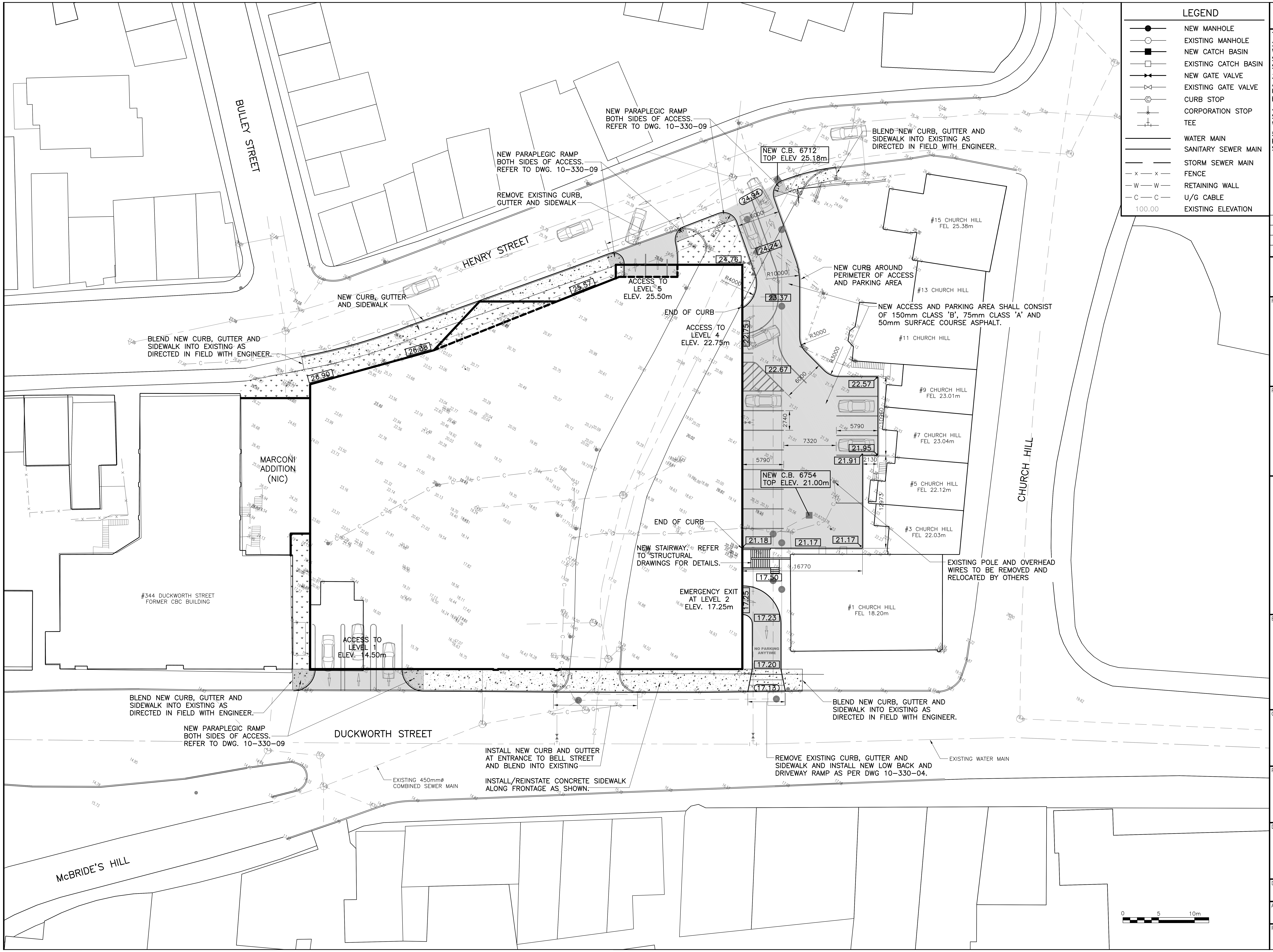
OWNER/CLIENT NAME:
HENRY BELL
DEVELOPMENTS LTD.

PROJECT TITLE:
DUCKWORTH STREET
PARKING GARAGE
AND CONDOMINIUM

DRAWING TITLE:
EXISTING SITE PLAN

DRAWN/DESIGNED BY:	DATE:
D.J.M.	JULY 2010
APPROVED BY: P.A.H.	SCALE: AS SHOWN

PROJECT No.:	DRAWING No.:	REV.:
10208	C2	C



LEGEND

●	NEW MANHOLE
○	EXISTING MANHOLE
■	NEW CATCH BASIN
□	EXISTING CATCH BASIN
⧓	NEW GATE VALVE
⋈	EXISTING GATE VALVE
Ⓒ	CURB STOP
┼	CORPORATION STOP
┴	TEE
———	WATER MAIN
———	SANITARY SEWER MAIN
—— ——	STORM SEWER MAIN
— x — x —	FENCE
— W — W —	RETAINING WALL
— C — C —	U/G CABLE
100.00	EXISTING ELEVATION

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4. ANY DISTURBED AREAS DURING CONSTRUCTION SHALL BE REPLACED WITH NEW AS DIRECTED IN FIELD WITH ENGINEER.
5. STREET EXCAVATION PERMIT MUST BE OBTAINED PRIOR TO STARTING ANY WORK WITHIN THE PUBLIC STREET RIGHT-OF-WAY.

No.	REVISIONS	BY	DATE
C	REVISED FOR APPROVALS	D.J.M.	02/17/12
B	ISSUED FOR PHASE 1 CONSTRUCTION	D.J.M.	11/30/11
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OWNER/CLIENT NAME:

HENRY BELL
DEVELOPMENTS LTD.

PROJECT TITLE:

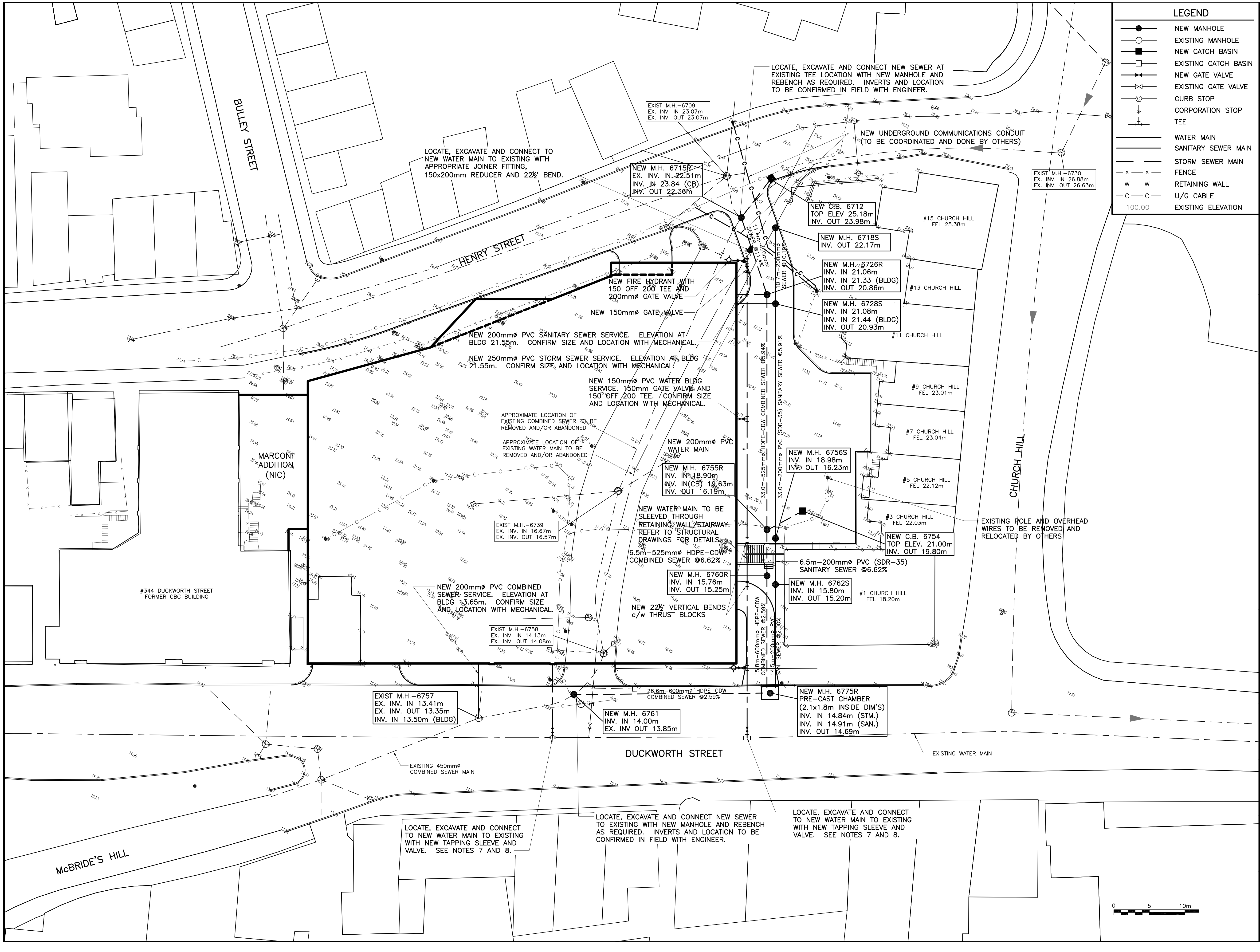
DUCKWORTH STREET
PARKING GARAGE
AND CONDOMINIUM

DRAWING TITLE:

SITE DEVELOPMENT PLAN

DRAWN/DESIGNED BY: D.J.M.	DATE: JUNE 2010
APPROVED BY: P.A.H.	SCALE: AS SHOWN

PROJECT No.:	DRAWING No.:	REV.:
10208	C3	C



LEGEND

●	NEW MANHOLE
○	EXISTING MANHOLE
■	NEW CATCH BASIN
□	EXISTING CATCH BASIN
⋈	NEW GATE VALVE
⋈	EXISTING GATE VALVE
©	CURB STOP
+	CORPORATION STOP
⊥	TEE
―	WATER MAIN
―	SANITARY SEWER MAIN
―	STORM SEWER MAIN
—x—x—	FENCE
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100.00	EXISTING ELEVATION

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4. MINIMUM WIDTH OF ASPHALT REINSTATEMENT FOR THE TRENCH SHALL BE 3.0m. ASPHALT REINSTATEMENT MUST BE PERFORMED IN ACCORDANCE WITH THE REQUIREMENTS OF ITEM NO. 352, FULL DEPTH ASPHALT PATCH, OF THE CITY OF ST. JOHN'S SPECIFICATIONS BOOK.
5. STREET EXCAVATION PERMIT MUST BE OBTAINED PRIOR TO STARTING ANY WORK WITHIN THE PUBLIC STREET RIGHT–OF–WAY.
6. A PERMIT TO CONNECT MUST BE OBTAINED PRIOR TO PERFORMING ANY SERVICING WORK.
7. ALL WORK ON EXISTING WATER MAINS SHALL BE DONE BY CITY FORCES.
8. CONNECTION TO EXISTING WATER MAINS SHALL BE DONE AS PER SECTION 230, IN PARTICULAR ITEM 230.17, OF THE CITY OF ST. JOHN'S SPECIFICATION BOOK.

No.	REVISIONS	BY	DATE
D	REVISED FOR APPROVALS	D.J.M.	04/05/12
C	REVISED FOR APPROVALS	D.J.M.	02/17/12
B	ISSUED FOR PHASE 1 INFORMATION	D.J.M.	11/30/11
A	ISSUED FOR CLIENT REVIEW	D.J.M.	08/31/11

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REFERENCE:

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SIGNATURE
04/05/12
DATE

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OWNER/CLIENT NAME:
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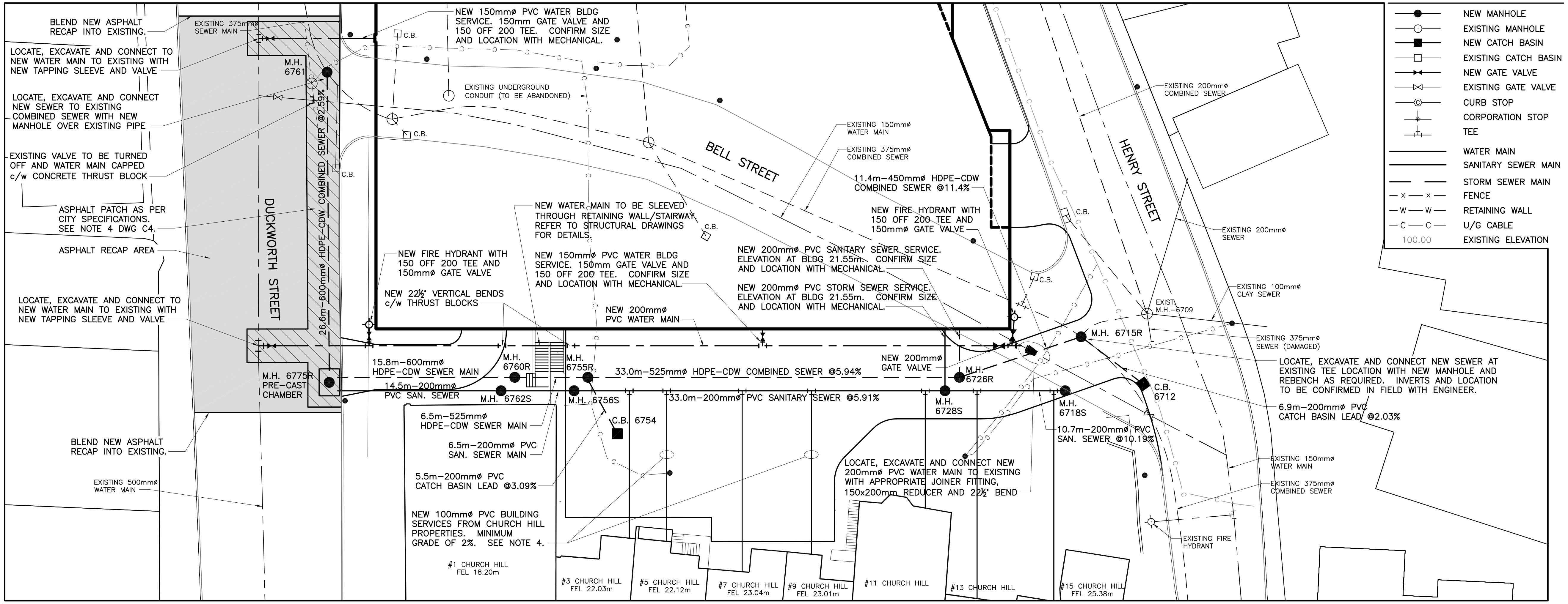
PROJECT TITLE:
DUCKWORTH STREET
PARKING GARAGE
AND CONDOMINIUM

DRAWING TITLE:
SITE SERVICES PLAN

DRAWN/DESIGNED BY: D.J.M.	DATE: JUNE 2010
APPROVED BY: P.A.H.	SCALE: AS SHOWN

PROJECT No.:	DRAWING No.:	REV.:
10208	C4	D

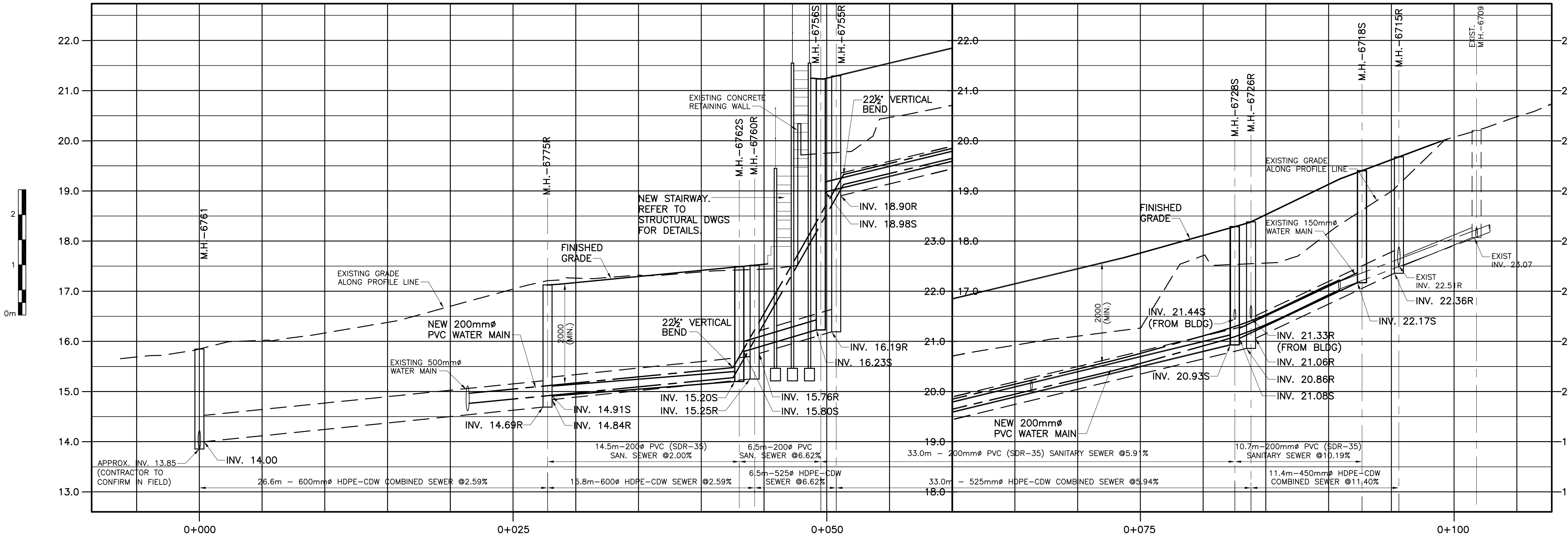


LEGEND

─●─	NEW MANHOLE
─○─	EXISTING MANHOLE
─■─	NEW CATCH BASIN
─□─	EXISTING CATCH BASIN
─▶◀─	NEW GATE VALVE
─▷◁─	EXISTING GATE VALVE
─©─	CURB STOP
─┼─	CORPORATION STOP
─┴─	TEE
────	WATER MAIN
────	SANITARY SEWER MAIN
── ──	STORM SEWER MAIN
— x — x —	FENCE
— W — W —	RETAINING WALL
— C — C —	U/G CABLE
100.00	EXISTING ELEVATION

PLAN STA. 0+000 TO 0+096

0 5 10m



PROFILE STA 0+000 TO STA 0+096

0 5 10m

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2. ALL WORK MUST BE PERFORMED IN ACCORDANCE WITH THE REQUIREMENTS OF THE APPLICABLE SECTIONS OF THE CITY OF ST. JOHN'S SPECIFICATIONS BOOK.
3. CONTRACTOR SHALL CONFIRM LOCATION, ALIGNMENT, INVERTS AND TYPE OF MATERIAL OF ALL UNDERGROUND SERVICES AND SHALL NOTIFY THE ENGINEER/OWNER PRIOR TO COMMENCING ANY WORK.
4. ALL EXISTING SERVICES FROM BUILDINGS ON CHURCH HILL THAT CONNECT TO EXISTING SEWER ON BELL STREET TO BE REPLACED WITH NEW FROM BUILDING FOUNDATION TO CONNECTION TO NEW SEWER. ALL NEW SERVICES SHALL HAVE A CLEAN–OUT LOCATED OUTSIDE OF EXISTING FOUNDATION AS PER CITY SPECIFICATIONS. ALL WORK TO BE DONE AS DIRECTED IN FIELD WITH ENGINEER.
5. STREET EXCAVATION PERMIT MUST BE OBTAINED PRIOR TO STARTING ANY WORK WITHIN THE PUBLIC STREET RIGHT–OF–WAY.
6. A PERMIT TO CONNECT MUST BE OBTAINED PRIOR TO PERFORMING ANY SERVICING WORK.
7. ALL WORK ON EXISTING WATER MAINS SHALL BE DONE BY CITY FORCES.
8. CONNECTION TO EXISTING WATER MAINS SHALL BE DONE AS PER SECTION 230, IN PARTICULAR ITEM 230.17, OF THE CITY OF ST. JOHN'S SPECIFICATION BOOK.

No.	REVISIONS	BY	DATE
D	REVISED FOR APPROVALS	D.J.M.	04/05/12
C	REVISED FOR APPROVALS	D.J.M.	02/17/12
B	ISSUED FOR PHASE 1 INFORMATION	D.J.M.	11/30/11
A	ISSUED FOR CLIENT REVIEW	D.J.M.	09/06/11

A — PLAN, SECTION, ELEVATION, OR DETAIL No.
B — No. OF DRAWING WHERE ABOVE IS DRAWN

REFERENCE:

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PETER A. HUTTON
SIGNATURE
04/05/12
DATE

PERMIT STAMP:

PROVINCE OF NEWFOUNDLAND
PERMIT HOLDER
CLASS "A"
This Permit Allows
PETER A. HUTTON
To practice Professional Engineering in Newfoundland and Labrador
Permit No. as issued by APEGN D0084
which is valid for the year 2012

DYNAMIC DEVELOPMENT SERVICES LTD.
CELL: (709) 682-3488
PHONE: (709) 368-1669
FAX: (709) 368-0318
P.O. Box 293
ST. JOHN'S, NL
A1S 1G4
E-MAIL: djmadden@nl.rogers.com

ENGINEER/ARCHITECT:

PETER HUTTON, P. ENG.
ADVANTAGE MANAGEMENT LTD.
322 WATERFORD BRIDGE ROAD
ST. JOHN'S, NF
A1E 1E9
TEL. /FAX. (709) 368–8547 CELL (709) 685–3671

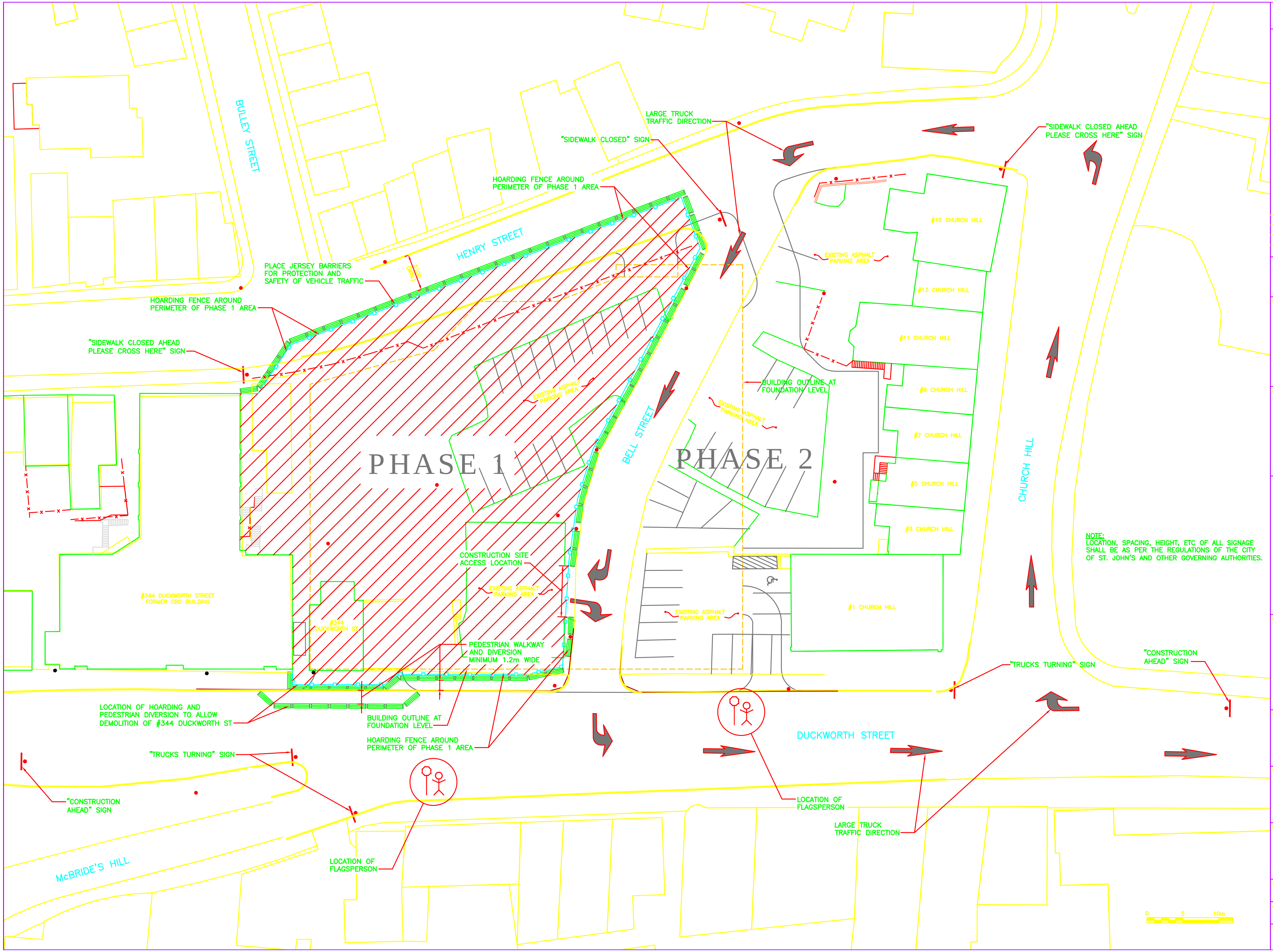
OWNER/CLIENT NAME:
HENRY BELL DEVELOPMENTS LTD.

PROJECT TITLE:
DUCKWORTH STREET PARKING GARAGE AND CONDOMINIUM

DRAWING TITLE:
PLAN – PROFILE
STA. 0+000 TO 0+096

DRAWN/DESIGNED BY: D.J.M.	DATE: JULY 2010
APPROVED BY: P.A.H.	SCALE: AS SHOWN

PROJECT No.:	DRAWING No.:	REV.:
10208	C5	D



NOTE:
LOCATION, SPACING, HEIGHT, ETC OF ALL SIGNAGE SHALL BE AS PER THE REGULATIONS OF THE CITY OF ST. JOHN'S AND OTHER GOVERNING AUTHORITIES.

GENERAL NOTES

1. DO NOT SCALE FROM DRAWINGS.
2. ALL WORK MUST BE PERFORMED IN ACCORDANCE WITH THE REQUIREMENTS OF THE APPLICABLE SECTIONS OF THE CITY OF ST. JOHN'S SPECIFICATIONS BOOK.
3. CONTRACTOR SHALL CONFIRM LOCATION, ALIGNMENT, INVERTS AND TYPE OF MATERIAL OF ALL UNDERGROUND SERVICES AND SHALL NOTIFY THE ENGINEER/OWNER PRIOR TO COMMENCING ANY WORK.
4. CONTRACTOR IS RESPONSIBLE FOR ALL TRAFFIC COORDINATION, SIGNAGE AND OTHER REQUIREMENTS AS PER THE REGULATIONS OF THE CITY OF ST. JOHN'S AND OTHER GOVERNING AUTHORITIES. REFER TO DIVISION 7 FOR SIGNAGE DETAILS.
5. SITE HOARDING FOR EACH PHASE TO BE DONE BY OTHERS.
6. DURING CONSTRUCTION OF PHASE 1, BELL STREET AND PARKING AREAS LOCATED OUTSIDE OF PHASE 1 BOUNDARY SHALL BE OPERATIONAL AT ALL TIMES.
7. SLOPE STABILIZATION FOR ALL EXCAVATIONS SHALL BE THE RESPONSIBILITY OF THE CONTRACTOR. ALL METHODS USED SHALL REQUIRE ACCEPTANCE FROM GEOTECHNICAL ENGINEER AND ALL HEALTH AND SAFETY REGULATIONS.

No.	REVISIONS	BY	DATE
C	ISSUED FOR PHASE 1 CONSTRUCTION	D.J.M.	11/30/11
B	REVISED FOR APPROVALS	D.J.M.	11/09/11
A	ISSUED FOR CLIENT REVIEW	D.J.M.	08/31/11

A – PLAN, SECTION, ELEVATION, OR DETAIL No.
B – No. OF DRAWING WHERE ABOVE IS DRAWN

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PERMIT STAMP:

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SERVICES LTD.

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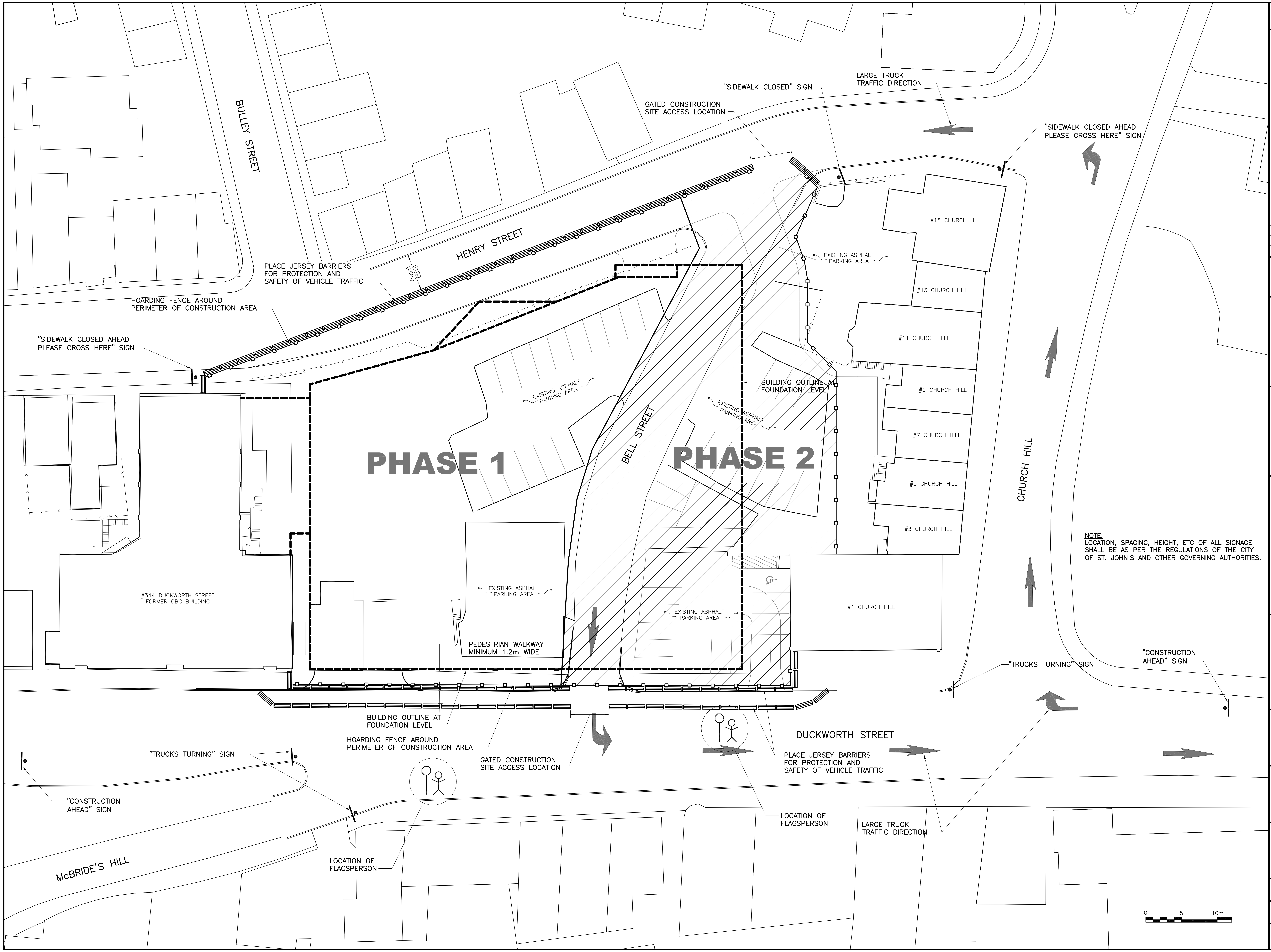
ST. JOHN'S, NF
A1E 1E9
TEL. /FAX. (709) 368-8547 CELL (709) 685-3671

OWNER/CLIENT NAME:
HENRY BELL
DEVELOPMENTS LTD.

PROJECT TITLE:
DUCKWORTH STREET
PARKING GARAGE
AND CONDOMINIUM

DRAWING TITLE:
HOARDING AND
TRAFFIC CONTROL PLAN
PHASE 1

DRAWN/DESIGNED BY: D.J.M.
DATE: AUGUST 2011
APPROVED BY: P.A.H.
SCALE: AS SHOWN
PROJECT No.: 10208
DRAWING No.: C6
REV.: C



NOTE:
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GENERAL NOTES

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5. SITE HOARDING FOR EACH PHASE TO BE DONE BY OTHERS.
6. SLOPE STABILIZATION FOR ALL EXCAVATIONS SHALL BE THE RESPONSIBILITY OF THE CONTRACTOR. ALL METHODS USED SHALL REQUIRE ACCEPTANCE FROM GEOTECHNICAL ENGINEER AND ALL HEALTH AND SAFETY REGULATIONS.

A	ISSUED FOR APPROVALS	D.J.M.	02/17/12
No.	REVISIONS	BY	DATE

A – PLAN, SECTION, ELEVATION, OR DETAIL No.
B – No. OF DRAWING WHERE ABOVE IS DRAWN

REFERENCE:

STAMP:
PETER A. HUTTON
SIGNATURE
02/17/12
DATE

PERMIT STAMP:
PROVINCE OF NEWFOUNDLAND
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PETER HUTTON, P. ENG.
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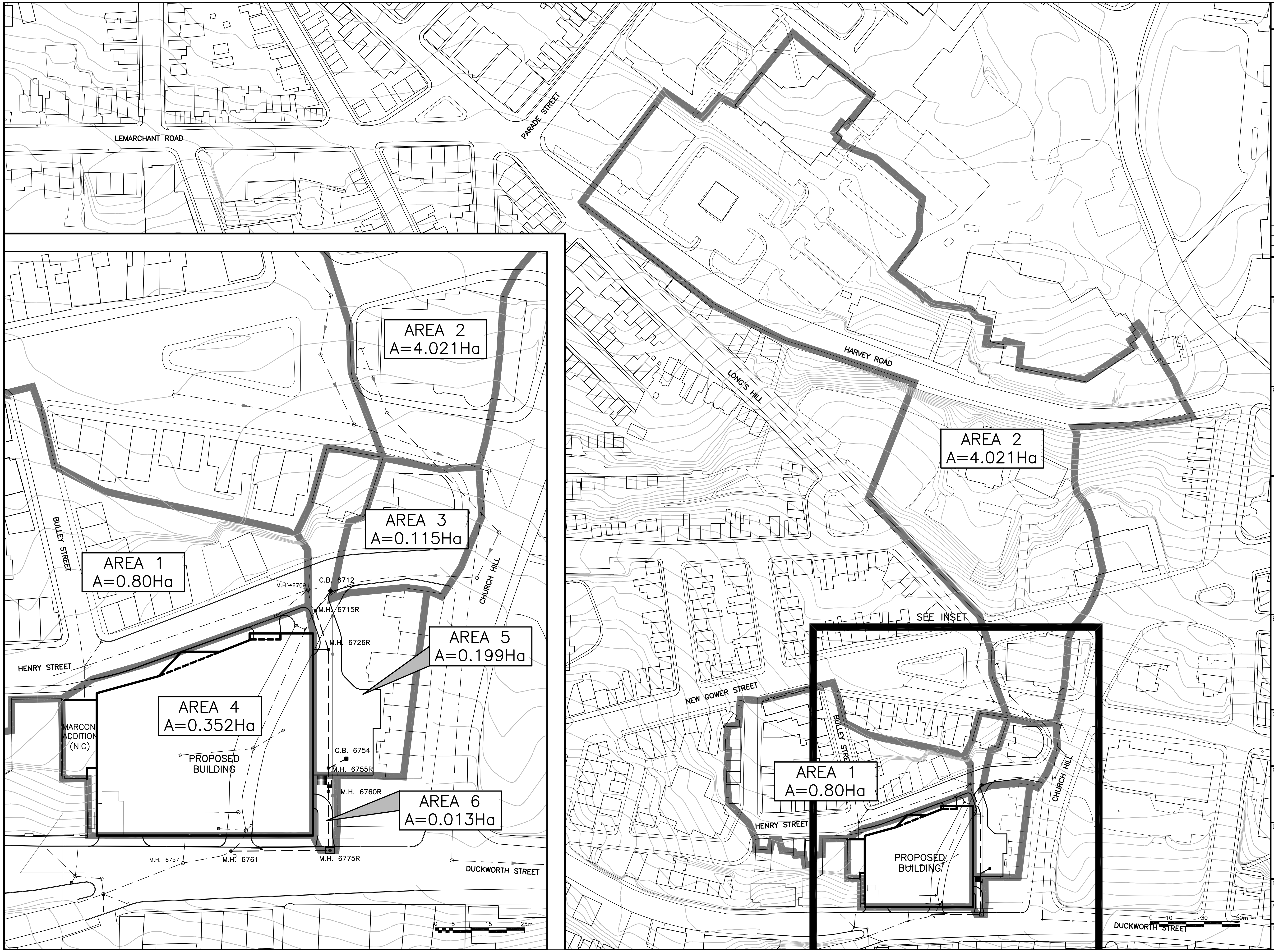
322 WATERFORD BRIDGE ROAD
ST. JOHN'S, NF
A1E 1E9
TEL. /FAX. (709) 368-8547 CELL (709) 685-3671

OWNER/CLIENT NAME:
HENRY BELL
DEVELOPMENTS LTD.

PROJECT TITLE:
DUCKWORTH STREET
PARKING GARAGE
AND CONDOMINIUM

DRAWING TITLE:
HOARDING AND
TRAFFIC CONTROL PLAN
PHASE 2

DRAWN/DESIGNED BY: D.J.M.
DATE: FEBRUARY 2012
APPROVED BY: P.A.H.
SCALE: AS SHOWN
PROJECT No.: 10208
DRAWING No.: C7
REV.: A



GENERAL NOTES

1. DO NOT SCALE FROM DRAWINGS.

No.	REVISIONS	BY	DATE
B	REVISED FOR APPROVALS	D.J.M.	04/05/12
A	ISSUED FOR APPROVALS	D.J.M.	02/17/12

A – PLAN, SECTION, ELEVATION, OR DETAIL No.
B – No. OF DRAWING WHERE ABOVE IS DRAWN

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STAMP:

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SIGNATURE
04/05/12
DATE

PERMIT STAMP:

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ENGINEER/ARCHITECT:

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322 WATERFORD BRIDGE ROAD
ST. JOHN'S, NF
A1E 1E9
TEL. /FAX. (709) 368-8547 CELL (709) 685-3671

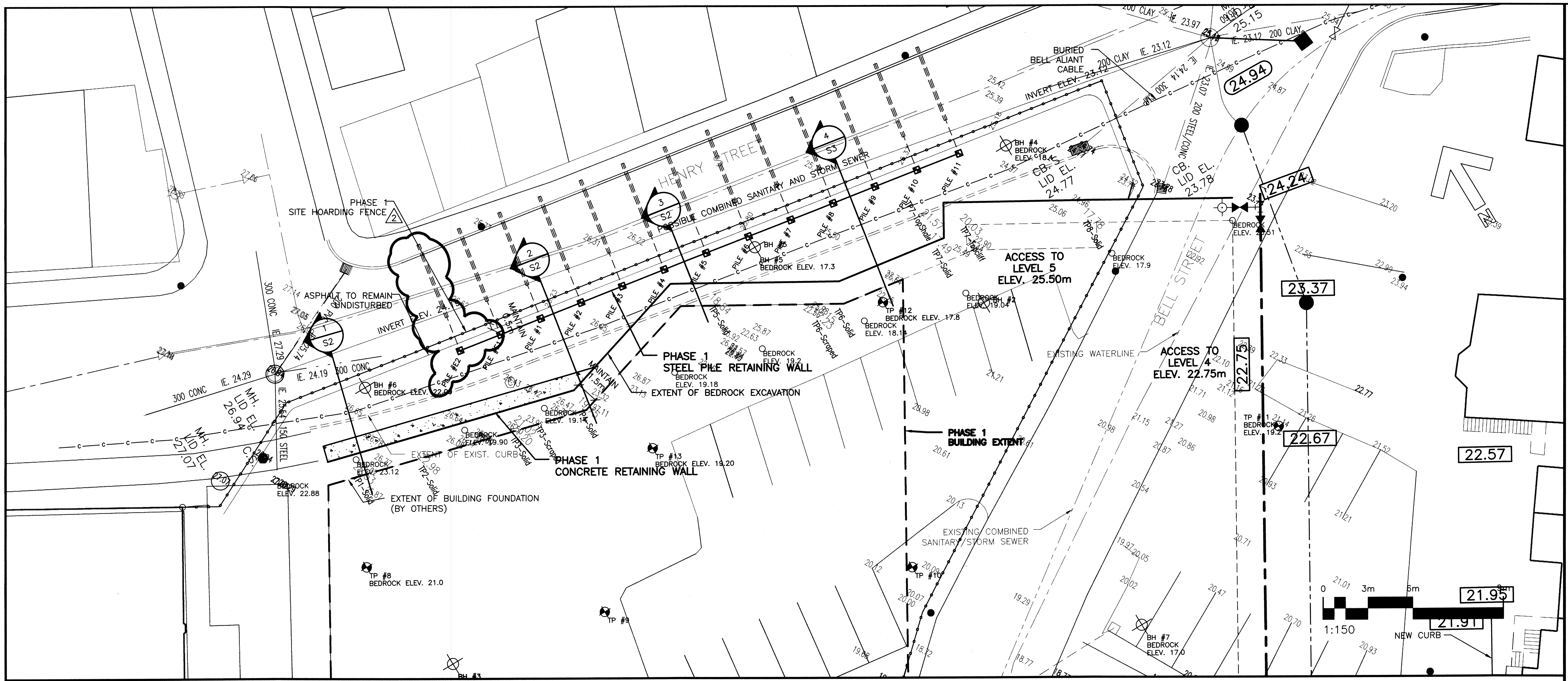
OWNER/CLIENT NAME:
HENRY BELL
DEVELOPMENTS LTD.

PROJECT TITLE:
DUCKWORTH STREET
PARKING GARAGE
AND CONDOMINIUM

DRAWING TITLE:
COMBINED SEWER
DRAINAGE AREA PLAN

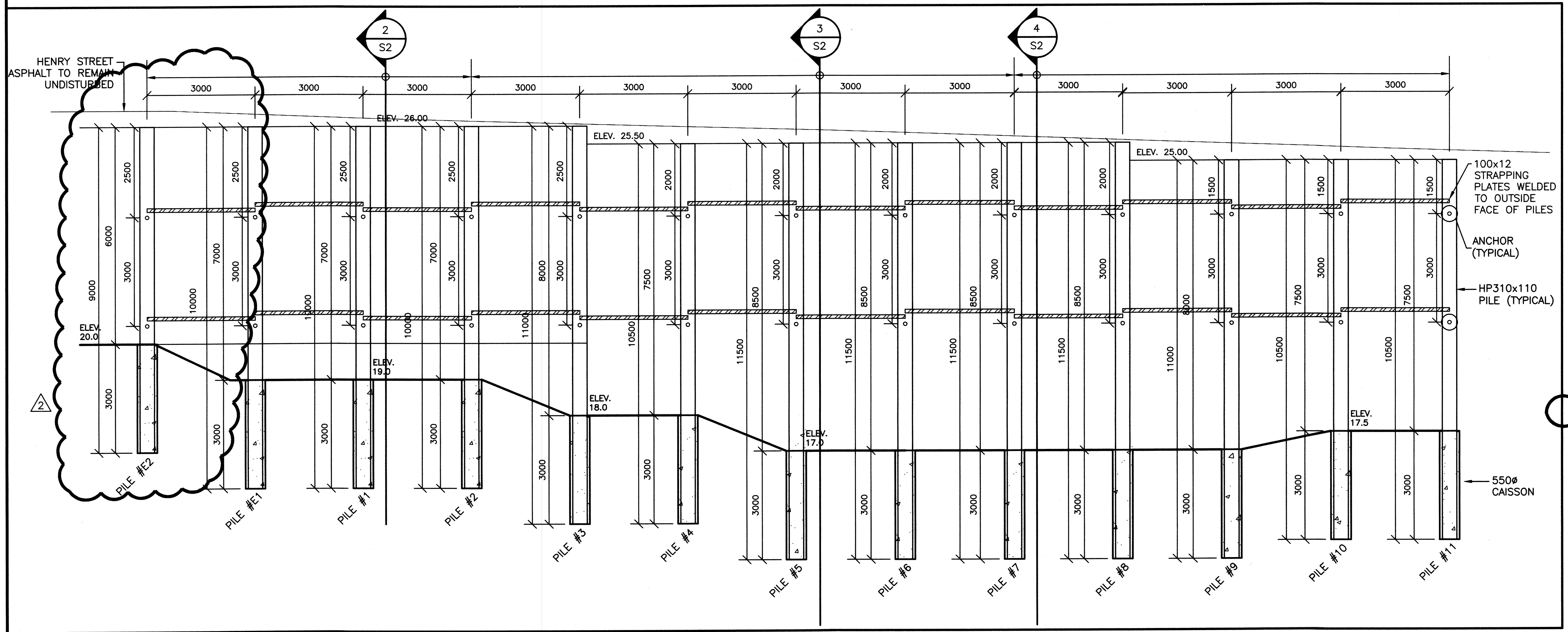
DRAWN/DESIGNED BY: D.J.M.	DATE: FEBRUARY 2012
APPROVED BY: P.A.H.	SCALE: AS SHOWN

PROJECT No.:	DRAWING No.:	REV.:
10208	C8	B



PHASE 1 HENRY STREET RETAINING WALL LOCATION PLAN

1:150



PHASE 1 HENRY STREET STEEL PILE RETAINING WALL PROFILE

1:75

PILE LOCATIONS

PILE #	COORDINATES	DISTANCE TO HENRY ST.	DISTANCE TO EXCAVATION
1	5 269 481.975 326 701.771	0.5m	5.225m
2	5 269 484.937 326 702.268	0.5m	3.993m
3	5 269 487.895 326 702.768	0.5m	2.538m
4	5 269 490.851 326 703.261	0.5m	1.065m
5	5 269 493.813 326 703.758	0.5m	2.154m
6	5 269 496.768 326 704.254	0.5m	3.296m
7	5 269 499.727 326 704.751	0.5m	4.432m
8	5 269 502.690 326 705.255	0.5m	4.956m
9	5 269 505.644 326 705.744	0.5m	4.913m
10	5 269 508.602 326 706.241	0.5m	4.877m
11	5 269 511.566 326 706.745	0.5m	3.425m
E1	5 269 479.290 326 701.320	0.5m	4.954m
E2	5 269 476.438 326 700.841	0.5m	3.599m

2

NOTES

CONTRACTORS TO VERIFY ALL DIMENSIONS ON THE SITE BEFORE PROCEEDING WITH THIS WORK.

"THESE DESIGN DOCUMENTS ARE PREPARED SOLELY FOR THE USE OF HENRY BELL DEVELOPMENTS LTD.WITH WHOM ACUREN GROUP INC. (ACUREN) HAS ENTERED INTO A CONTRACT & THERE ARE NO REPRESENTATIONS OF ANY KIND MADE BY ACUREN TO ANY PARTY WITH WHOM ACUREN HAS NOT ENTERED INTO A CONTRACT."

NO.	DESCRIPTION	YY.MM.DD DATE
2	PILE ADDED BY CONTRACTOR	12.07.03
1	PILE ADDED – SITE CONDITIONS	12.05.24
0	ISSUED FOR CONSTRUCTION	12.02.21
D	ISSUED FOR MATERIAL PURCHASE	12.02.13
C	ISSUED FOR REVIEW	12.02.07
B	ISSUED FOR REVIEW	12.01.17
A	ISSUED FOR DISCUSSION	12.01.13

ISSUES

REGISTERED PROFESSIONAL ENGINEER
K. GREGORY BARTON
SIGNATURE
4.Jul.2012
DATE
PROVINCE OF NEWFOUNDLAND

PROVINCE OF NEWFOUNDLAND AND LABRADOR
PEG Newfoundland and Labrador PROFESSIONAL ENGINEERS AND GEOSCIENTISTS
PERMIT HOLDER
This Permit Allows
ACUREN GROUP INC.
To practice Professional Engineering in Newfoundland and Labrador.
Permit No. as issued by PEGNL D0056

ACUREN
Acuren Group Inc.
112 Forest Rd. St. John's, NL A1A 1E6
Tel: (709)753–2100 / Fax: (709)753–7011
email: receptionnl@acuren.com

HENRY BELL DEVELOPMENTS LTD.

DRAWN BY: MARY PUMPHREY — DATE: YY.MM.DD 12.01.05
CHECKED BY: GREG BARTON — DATE: YY.MM.DD 12.02.13
APPROVED BY: GREG BARTON — DATE: YY.MM.DD 12.02.13
SCALE: AS NOTED

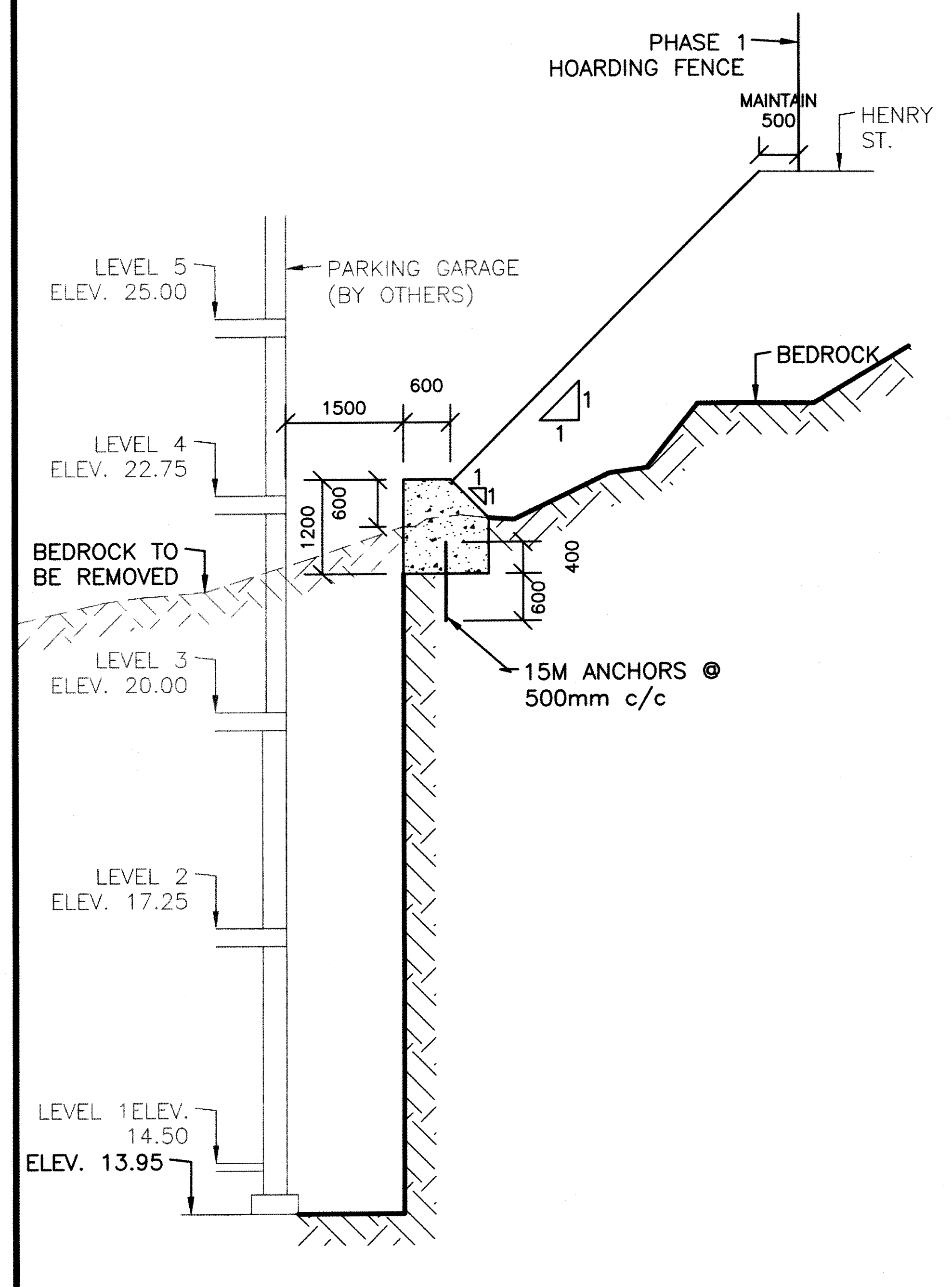
A DETAIL NO.
B DRAWING NO. WHERE DETAILED

JOB TITLE
MARCONI BUILDING HENRY STREET RETAINING WALL PHASE 1

DRAWING TITLE
SITE PLAN INDICATING RETAINING WALL LOCATION

PROJECT NO. 10030–01
SHEET NO. S1
REV. 2

K:\Current Projects\10030 Henry Bell Developments Ltd\-01 Duckworth Street Development\Drawing\REV 2\S1-S4 rev2, S5rA,S6rA,S7rA.dwg, S1r2, 7/3/2012 3:58:24 PM, mpumphrey, 1:1



SECTION ① S2
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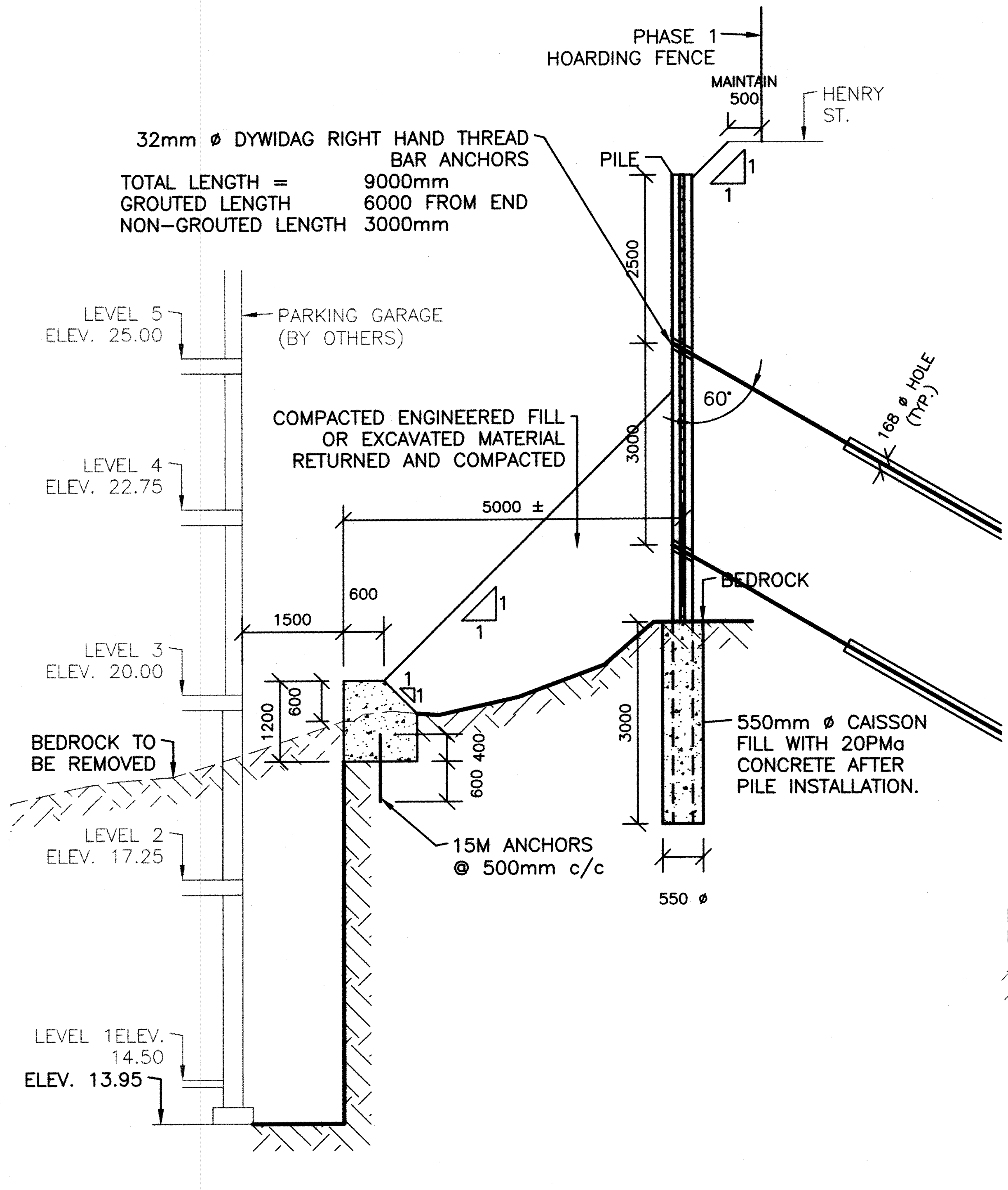

SECTION ② S2
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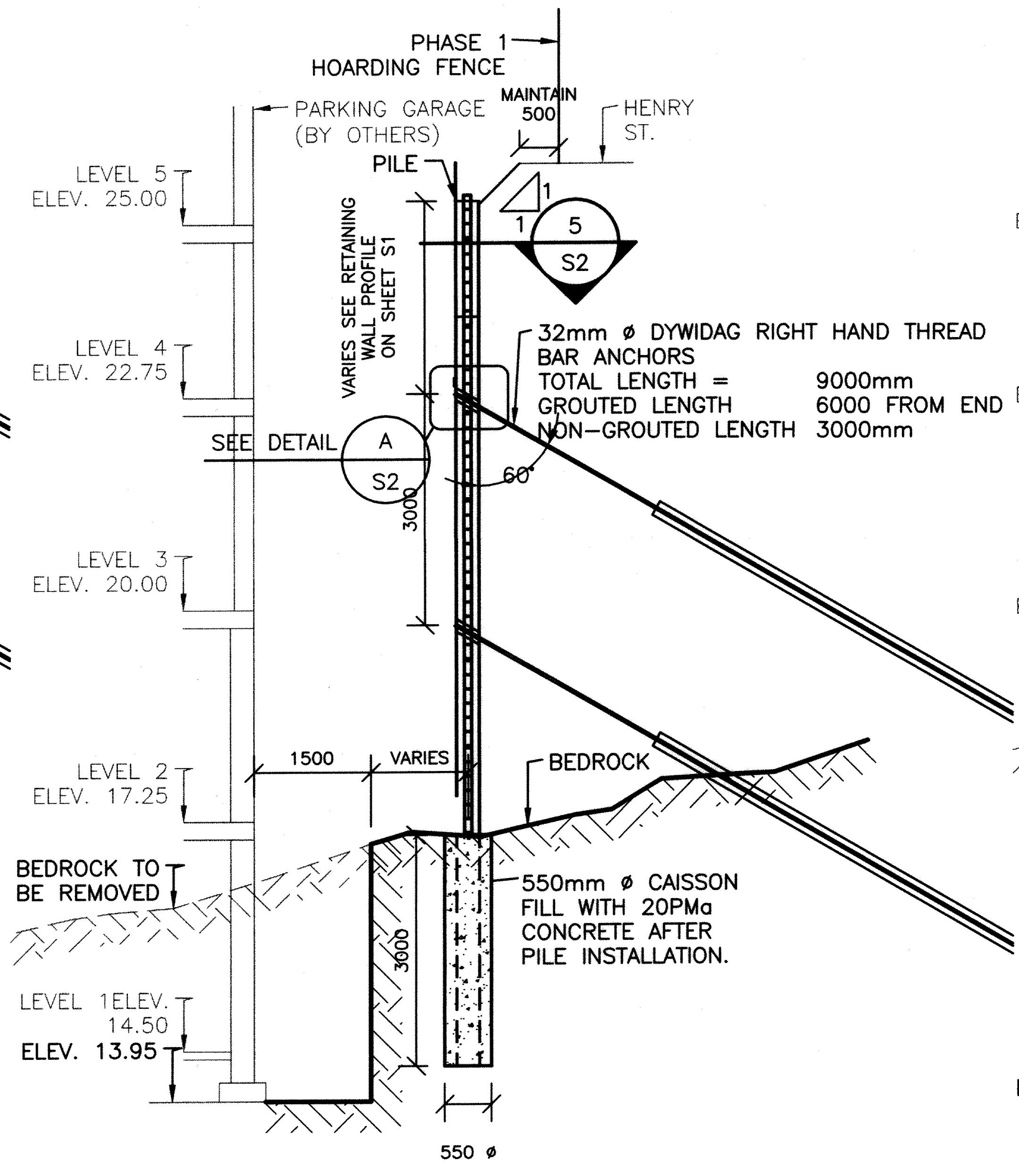

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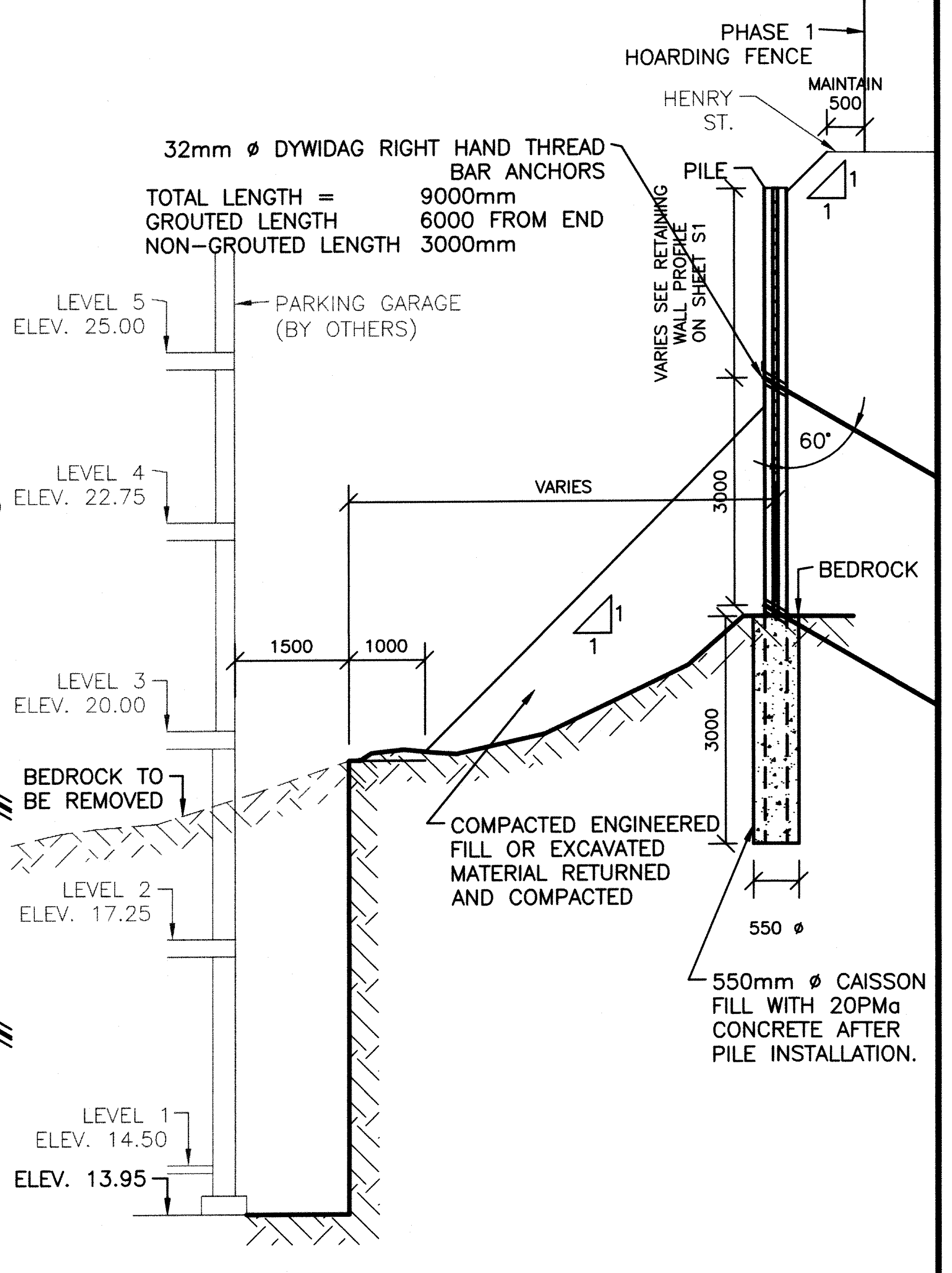

SECTION ④ S2
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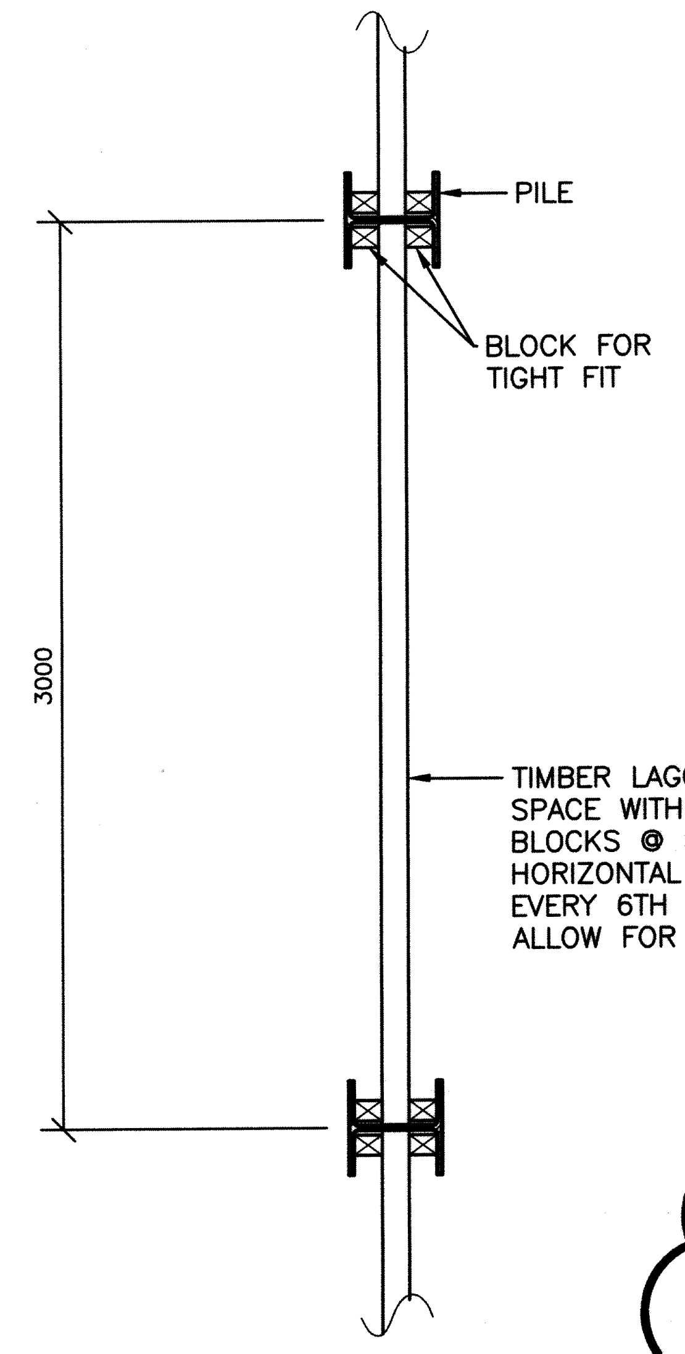
NOTE: ALL DYWIDAG THREADBARS TO BE A MINIMUM OF GRADE 75 (Fy = 517MPa).



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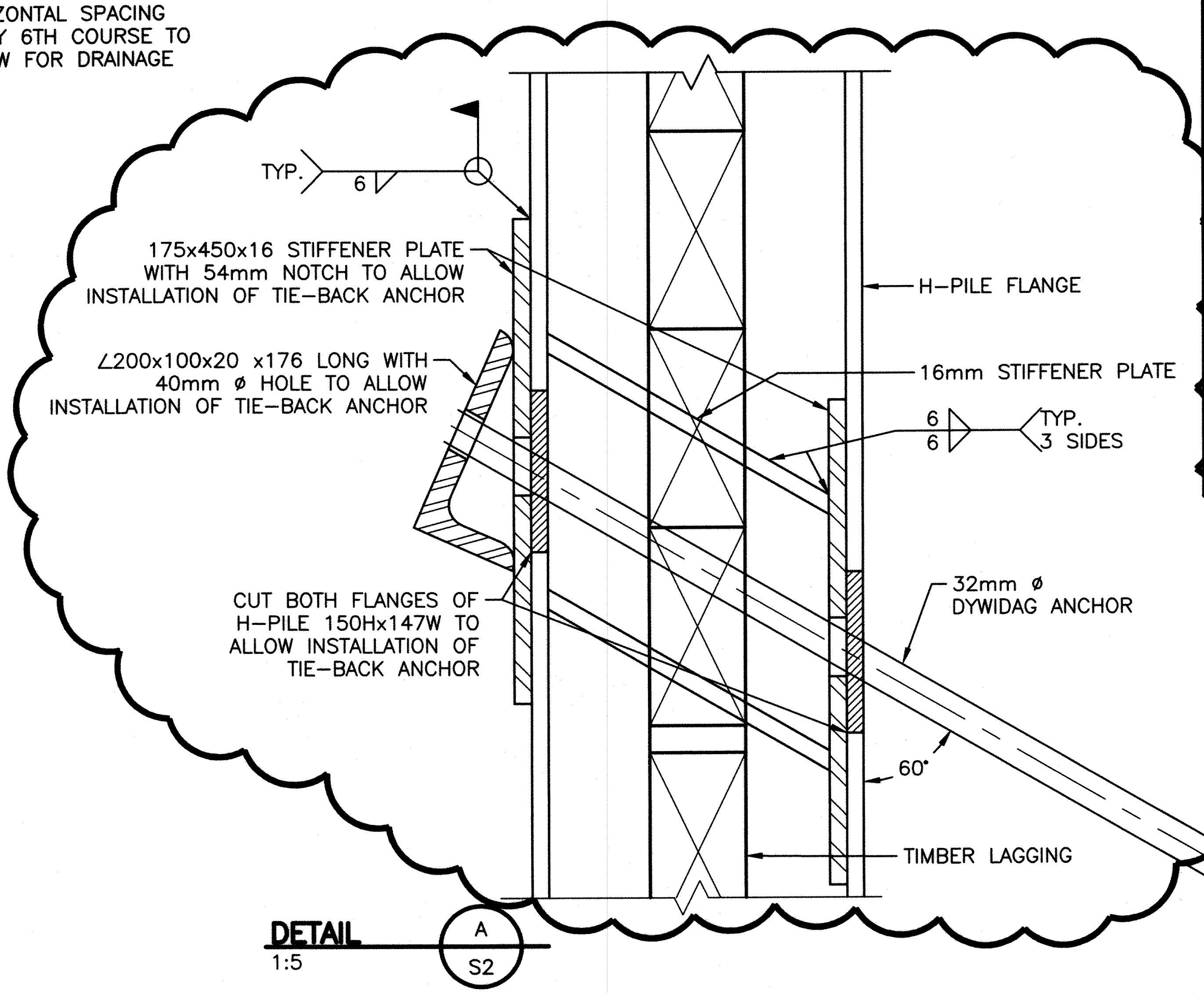

DETAIL Ⓐ S2
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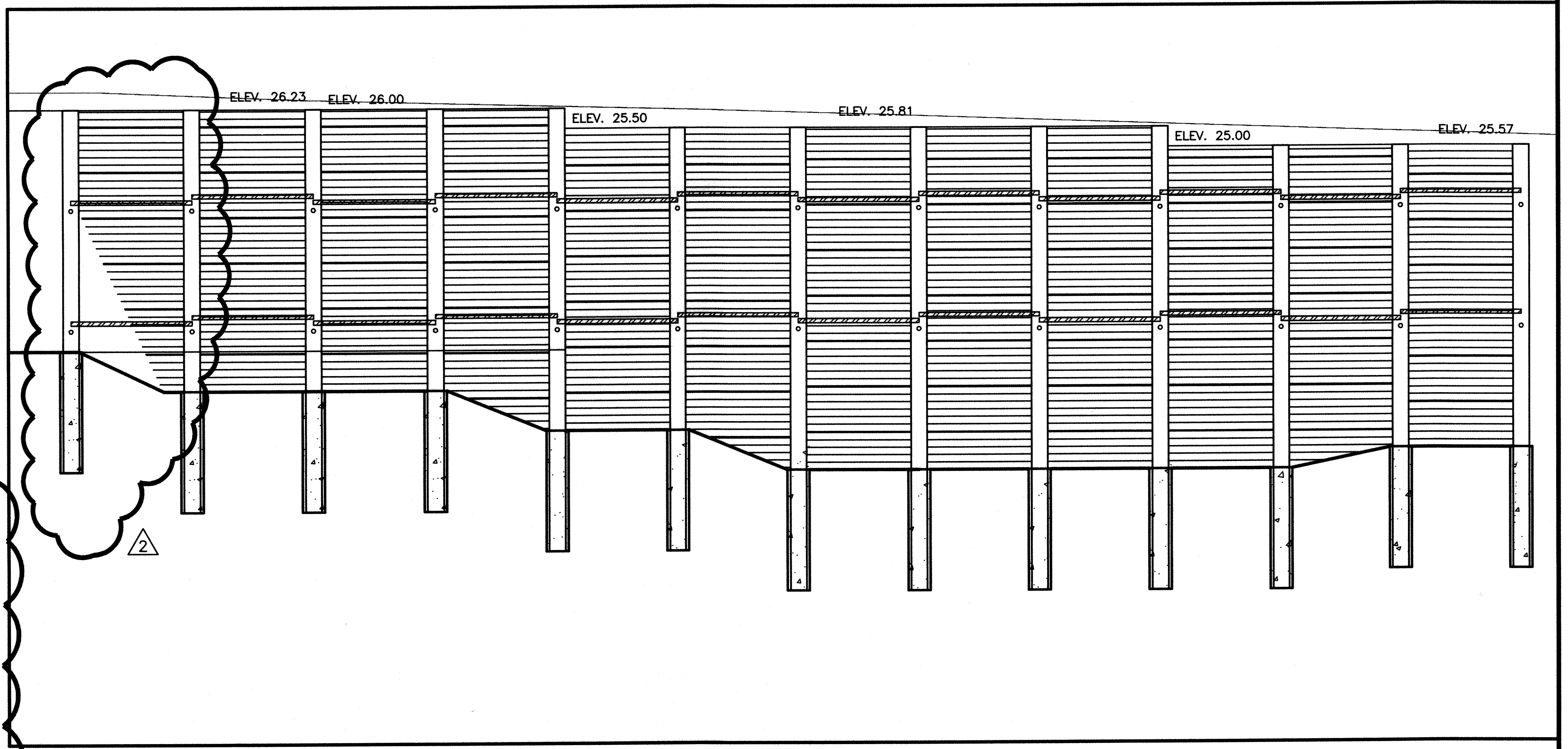

PHASE 1 HENRY STREET STEEL PILE RETAINING WALL PROFILE – FIGURE 1
1:100 (SHOWING TIMBER LAGGING)

NOTES

CONTRACTORS TO VERIFY ALL DIMENSIONS ON THE SITE BEFORE PROCEEDING WITH THIS WORK.

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B	ISSUED FOR REVIEW	12.01.17
A	ISSUED FOR DISCUSSION	12.01.13

ISSUES

REGISTERED PROFESSIONAL ENGINEER
K. GREGORY BARTON
SIGNATURE
4-Jul-2012
DATE
PROVINCE OF NEWFOUNDLAND

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Tel: (709)753–2100 / Fax: (709)753–7011
email: receptionnl@acuren.com

HENRY BELL
DEVELOPMENTS LTD.

DRAWN BY: MARY PUMPHREY — DATE: YY.MM.DD 12.01.05
CHECKED BY: GREG BARTON — DATE: YY.MM.DD 12.02.13
APPROVED BY: GREG BARTON — DATE: YY.MM.DD 12.02.13
SCALE: 1:50

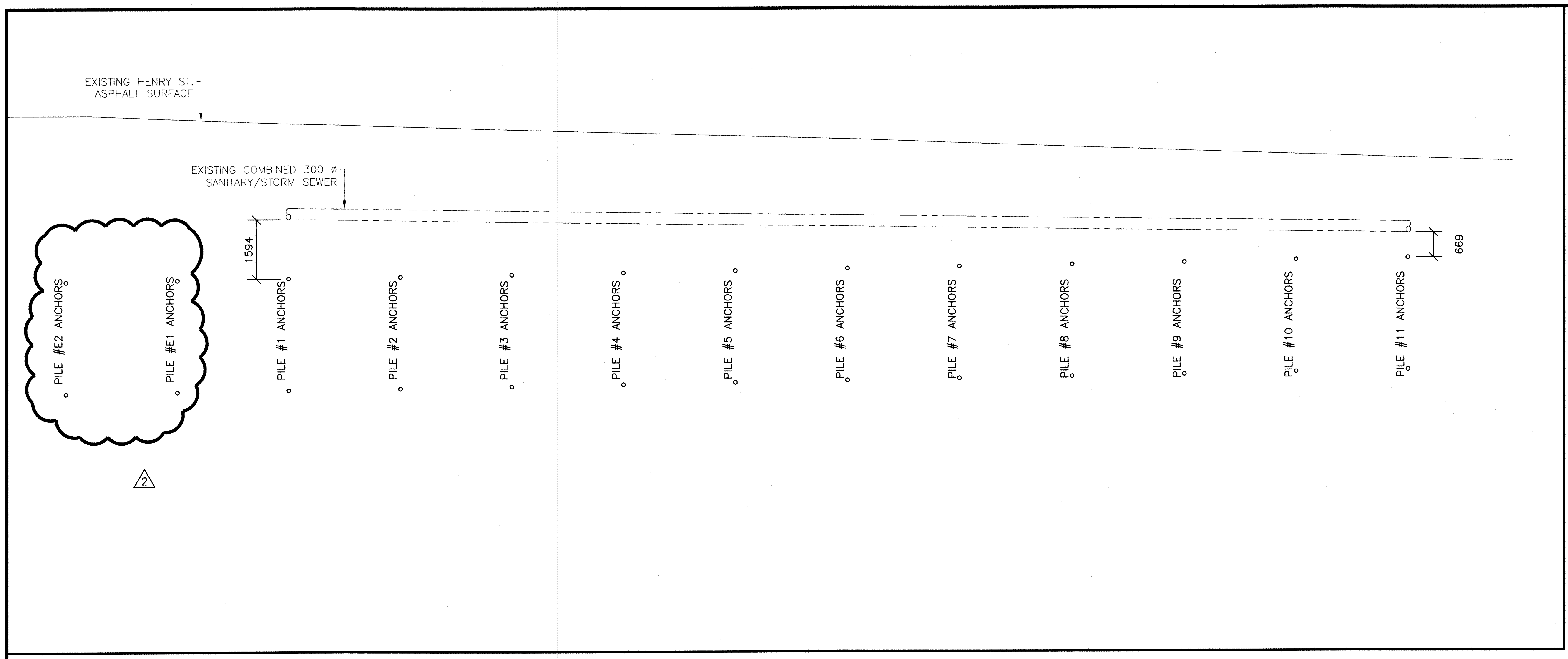
Ⓐ/Ⓑ A DETAIL NO. B DRAWING NO. WHERE DETAILED

JOB TITLE
MARCONI BUILDING
HENRY STREET RETAINING WALL
PHASE 1

DRAWING TITLE
RETAINING WALL SECTIONS

PROJECT NO.	SHEET NO.	REV.
10030–01	S2	2

K:\Current Projects\10030 Henry Bell Developments Ltd\-01 Duckworth Street Development\Drawing\REV 2\S1-S4 rev2, S5rA,S6rA,S7rA.dwg, S2r2, 7/3/2012 3:58:16 PM, mpumphrey, 1:1



PHASE 1 HENRY STREET RETAINING WALL ANCHORS AND SAN/STORM SEWER PROFILE
1:75

NOTES

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2	2 PILES ADDED	12.07.03
1	ISSUED FOR CITY APPROVAL	12.03.01
NO.	DESCRIPTION	YY.MM.DD DATE

ISSUES

K. GREGORY BARTON
SIGNATURE
4-Jul-2012
DATE

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email: receptionnl@acuren.com

HENRY BELL
DEVELOPMENTS LTD.

DRAWN BY: MARY PUMPHREY — DATE: YY.MM.DD 12.01.05
CHECKED BY: GREG BARTON — DATE: YY.MM.DD 12.02.13
APPROVED BY: GREG BARTON — DATE: YY.MM.DD 12.02.13
SCALE: AS NOTED

A DETAIL NO.
B DRAWING NO. WHERE DETAILED

JOB TITLE

2

MARCONI BUILDING
HENRY STREET RETAINING WALL
PHASE 1

DRAWING TITLE

PHASE 1 HENRY STREET
RETAINING WALL ANCHORS AND
SAN/STORM SEWER PROFILE

PROJECT NO.	SHEET NO.	REV.
10030-01	S4	2

K:\Current Projects\10030 Henry Bell Developments Ltd\-01 Duckworth Street Development\Drawing\REV 2\S1-S4 rev2, S5rA,S6rA,S7rA.dwg, S4r2, 7/3/2012 3:58:41 PM, mpumphrey, 1:1

Appendix C

Soil Profile Drawings

Project No.: 0001

Project Name: Duckworth Street Retaining Wall

Document: **Soil Profile Drawings**

To:

Acuren Group Inc.

112 Forest Rd

St. John's, NL, A1A 1E6

From:

Erica Soucy, Chenel Waight, Chantel Nicolle, Qiong Zhang

Group N - QCEC

Memorial University of Newofundland

St. John's, NL, A1B 3X5

Submission Date: April 03, 2013

Drawing Lists:

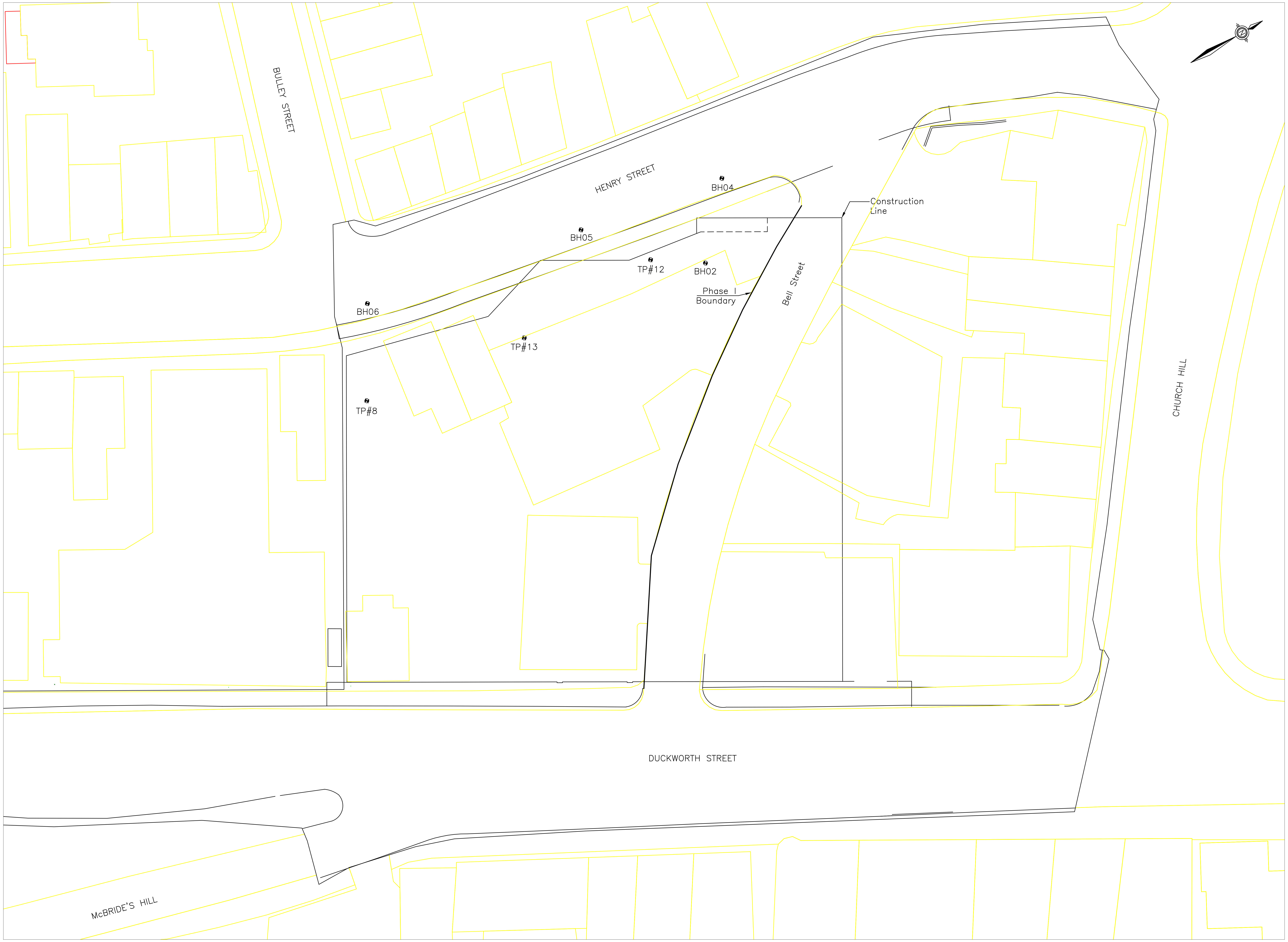

Figure 01

Site Plan with Borehole and Testpit Locations

General Notes

No.	Revision/Issue	Date

Firm Name and Address
Quality Civil Engineering Consultants
Memorial University of Newfoundland
St. John's, NL, A1B 3X5

Project Name and Address
Duckworth Street Retaining Wall
St. John's, NL

QCEC

Project
0001

Date
03.04.2013

Scale

Sheet
S-01

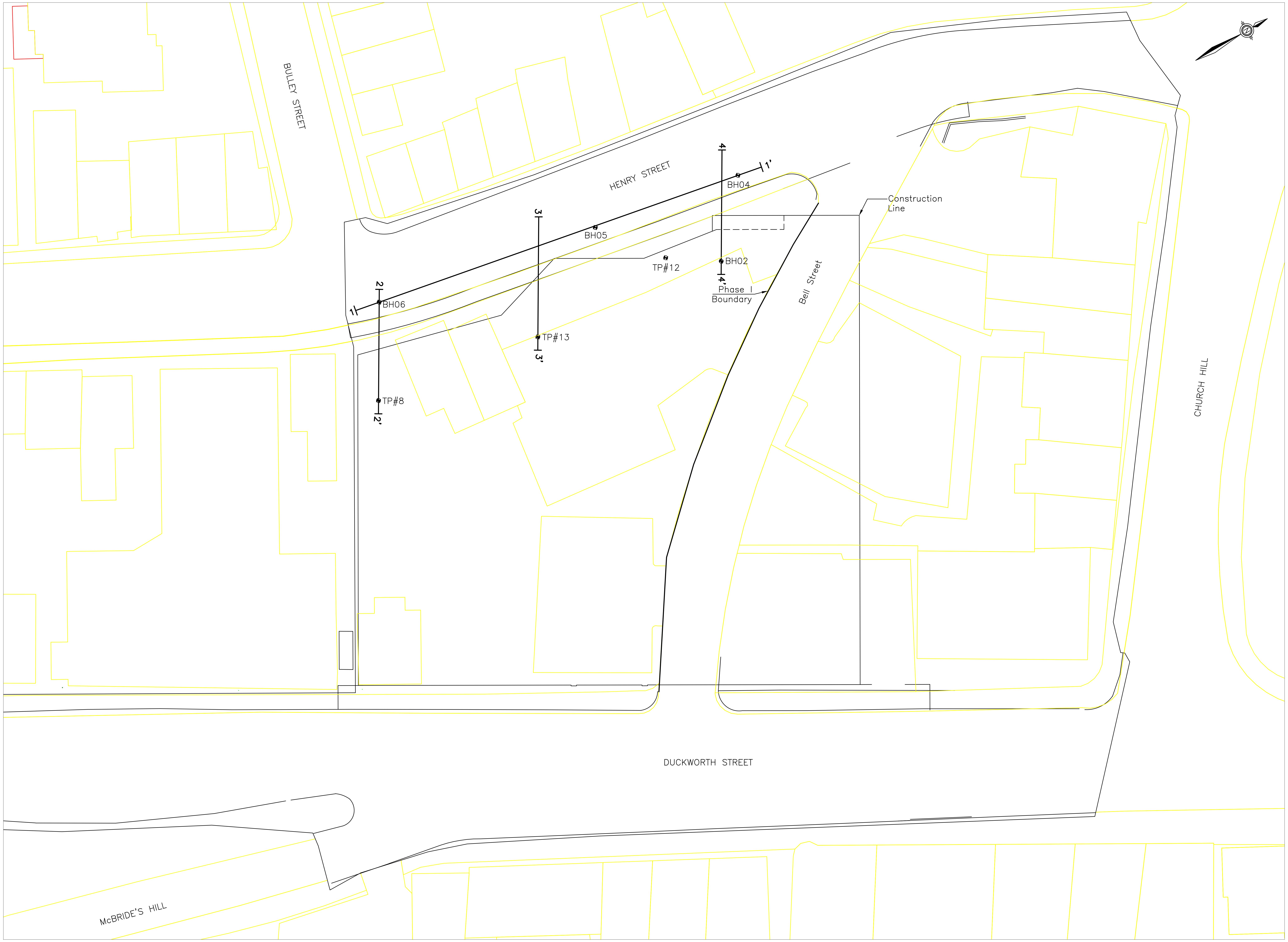

Figure 02 | Cross Section Locations

General Notes

No.	Revision/Issue	Date

Firm Name and Address
Quality Civil Engineering Consultants
Memorial University of Newfoundland
St. John's, NL, A1B 3X5

Project Name and Address
Duckworth Street Retaining Wall
St. John's, NL

QCEC

Project
0001

Date
03.04.2013

Scale

Sheet
S-02

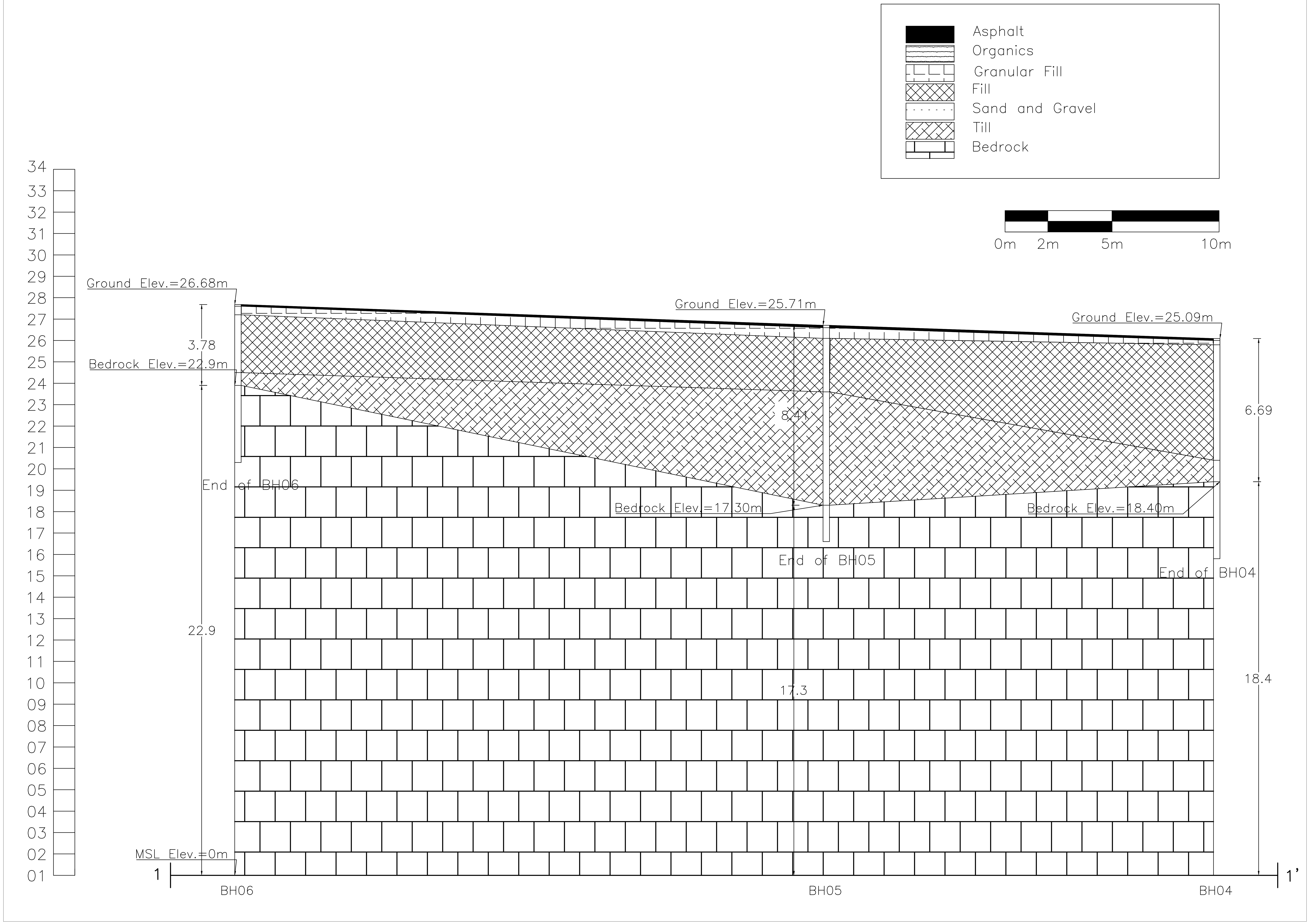

Figure 03	Soil Profile for Cross Section 1–1'

General Notes

No.	Revision/Issue	Date

Firm Name and Address
Quality Civil Engineering Consultants
Memorial University of Newfoundland
St. John's, NL, A1B 3X5

Project Name and Address
Duckworth Street Retaining Wall
St. John's, NL

QCEC

Project
0001

Date
03.04.2013

Scale

Sheet
S–03

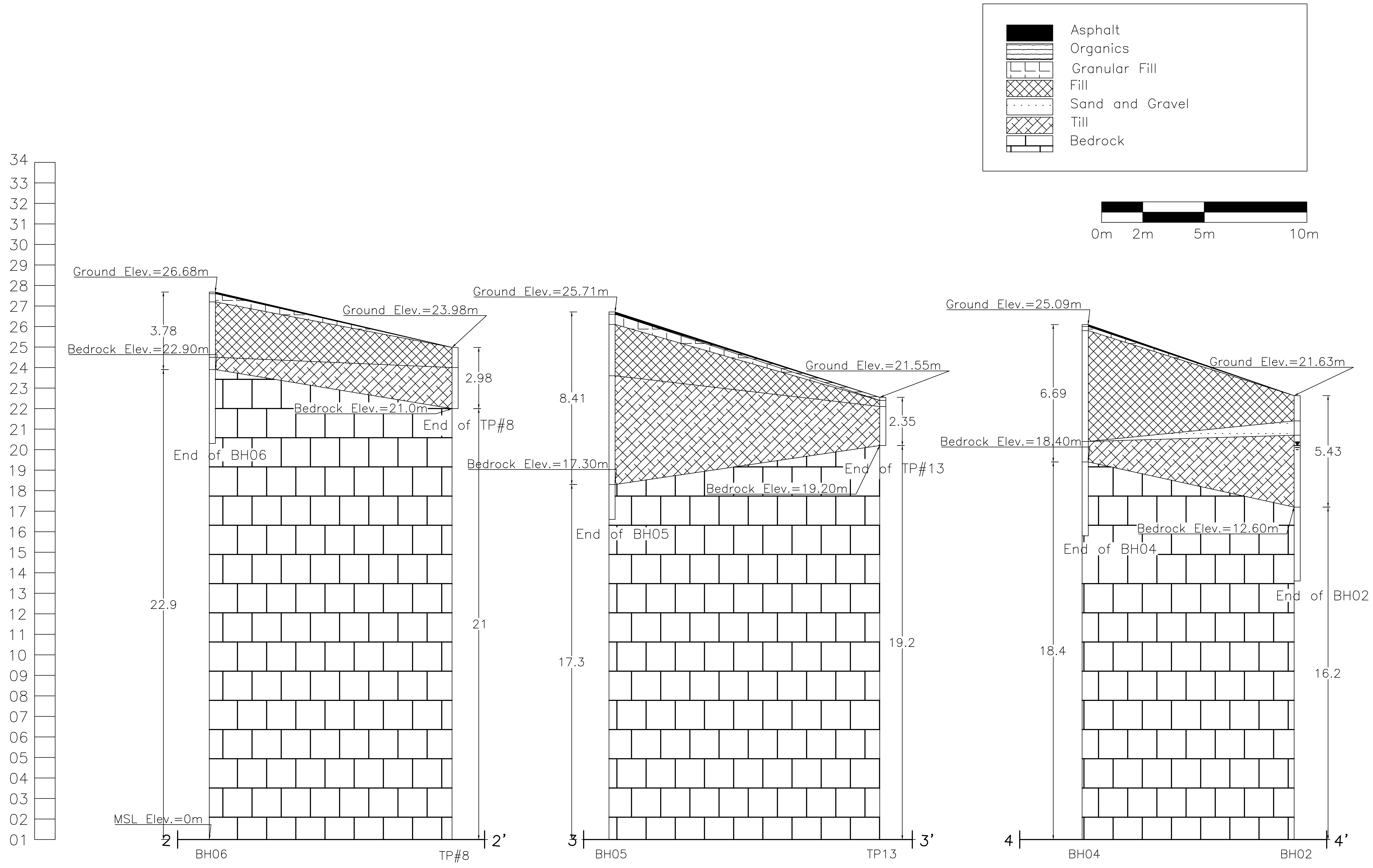

Figure 04

Soil Profile for Cross Section 2–2', 3–3', and 4–4'

General Notes

No.	Revision/Issue	Date

Firm Name and Address
Quality Civil Engineering Consultants
Memorial University of Newfoundland
St. John's, NL, A1B 3X5

Project Name and Address
Duckworth Street Retaining Wall
St. John's, NL

QCEC

Project
0001

Date
03.04.2013

Scale

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S–04

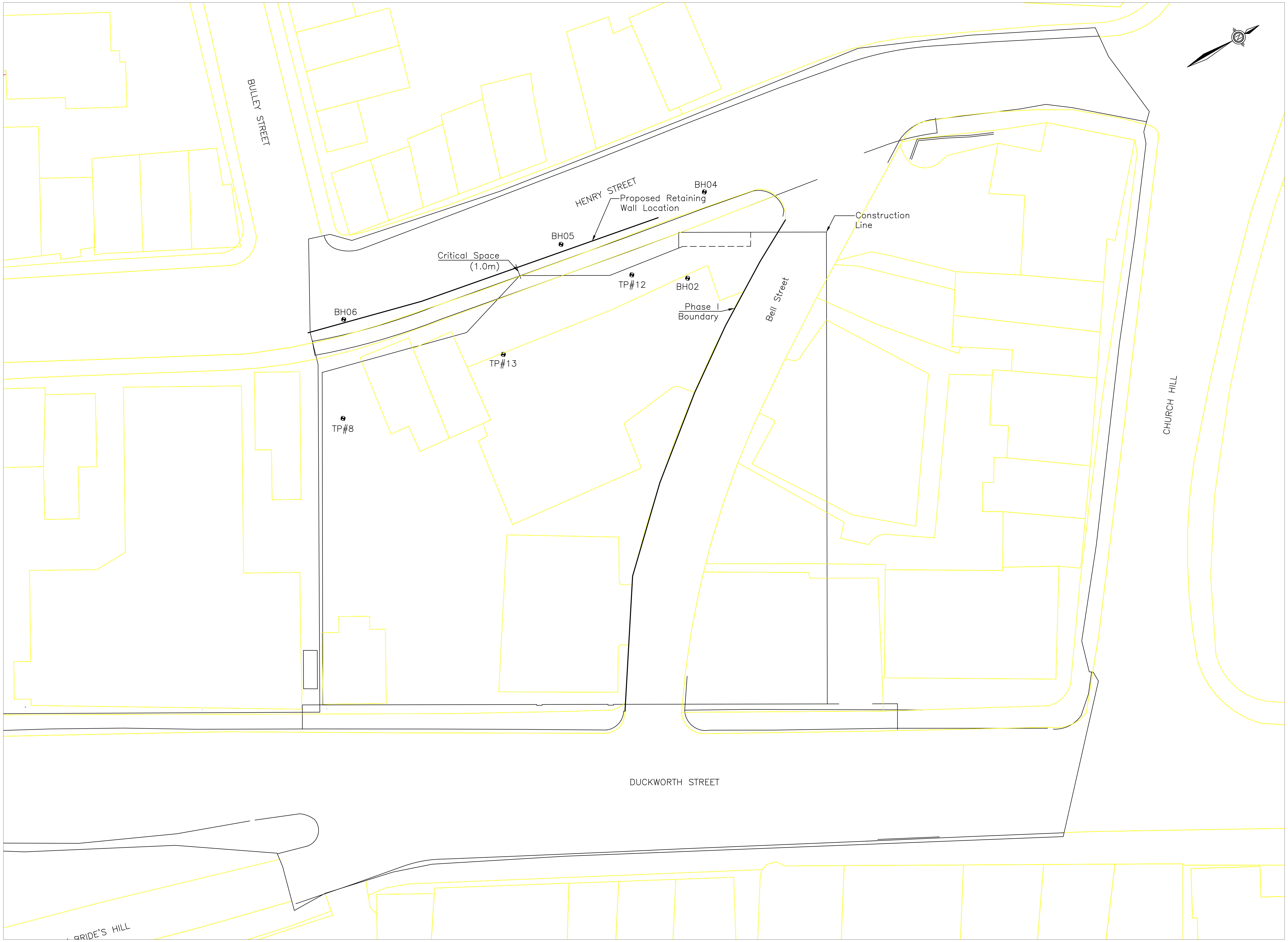


Figure 05 Site Plan with Retaining Wall Location

General Notes

No.	Revision/Issue	Date

Firm Name and Address
Quality Civil Engineering Consultants
Memorial University of Newfoundland
St. John's, NL, A1B 3X5

Project Name and Address
Duckworth Street Retaining Wall
St. John's, NL

QCEC

Project
0001

Date
03.04.2013

Scale

Sheet
S-05

Appendix D

Soldier Pile and Rigid Gravity Retaining Wall Drawings

Project No.: 0001

Project Name: Duckworth Street Retaining Wall

Document: **Soldier Pile Retaining Wall Detailed Drawing**

To:

Acuren Group Inc.

112 Forest Rd

St. John's, NL, A1A 1E6

From:

Erica Soucy, Chenel Waight, Chantel Nicolle, Qiong Zhang

Group N - QCEC

Memorial University of Newofundland

St. John's, NL, A1B 3X5

Submission Date: April 03, 2013

Drawing Lists:

Figure 01 | **Site Layout**

General Notes

No.	Revision/Issue	Date

Firm Name and Address

Quality Civil Engineering Consultants
Memorial University of Newfoundland
St. John's, NL, A1B 3X5

Project Name and Address

Duckworth Street Retaining Wall
St. John's, NL

QCEC

Project: 0001
Date: 03.04.2013
Scale:
Sheet: A-01

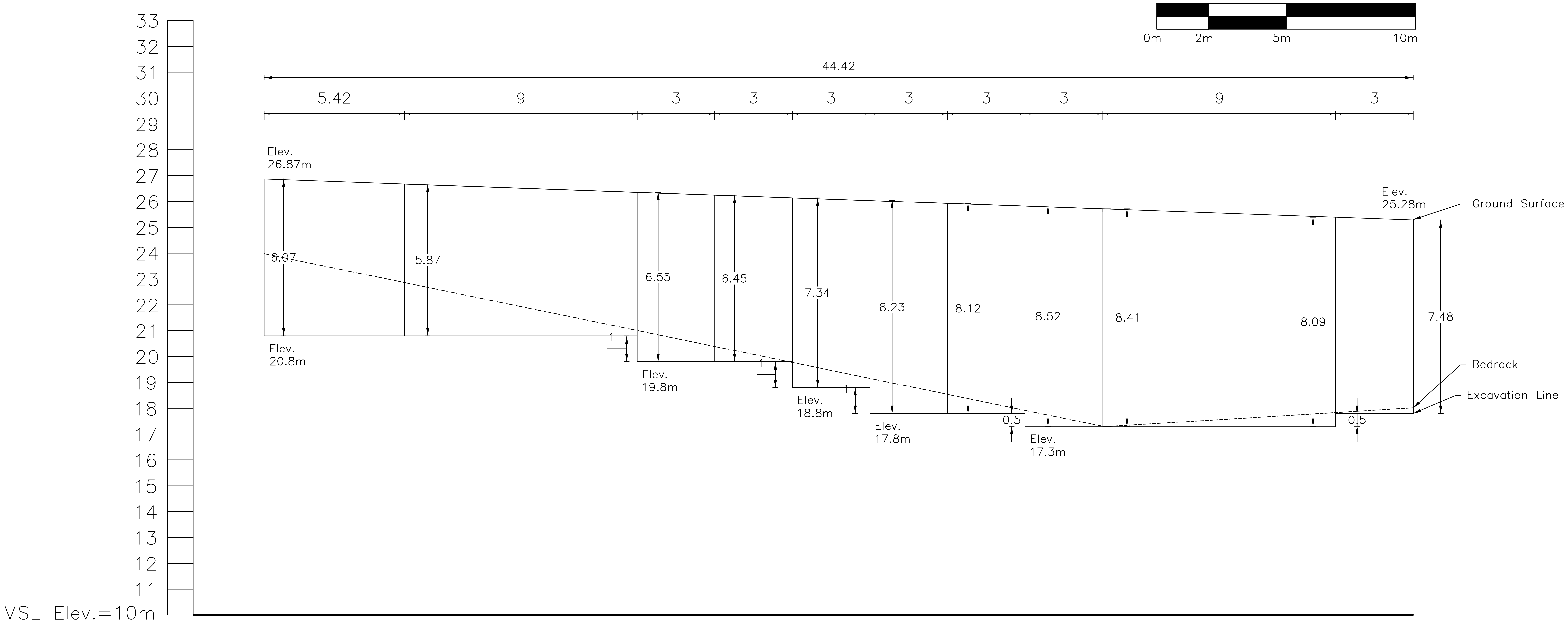


Figure 02	Excavation Profile

General Notes

No.	Revision/Issue	Date

Firm Name and Address
Quality Civil Engineering Consultants
Memorial University of Newfoundland
St. John's, NL, A1B 3X5

Project Name and Address
Duckworth Street Retaining Wall
St. John's, NL

QCEC

Project 0001
Date 03.04.2013
Scale
Sheet A-02

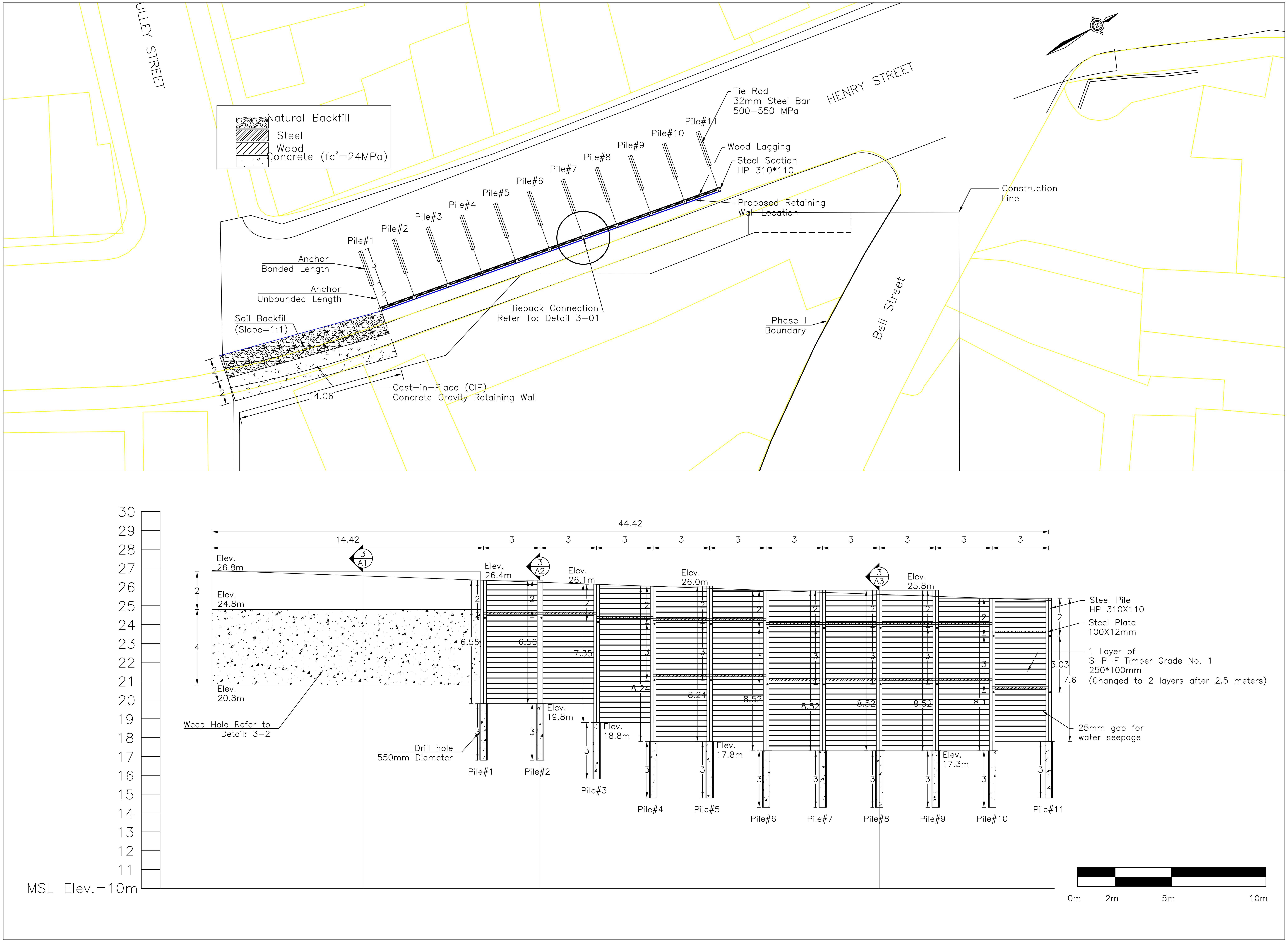


Figure 03 Soldier Pile Retaining Wall Plan View (Top) and Elevation View (Bottom).

General Notes

No.	Revision/Issue	Date

Firm Name and Address
Quality Civil Engineering Consultants
Memorial University of Newfoundland
St. John's, NL, A1B 3X5

Project Name and Address
Duckworth Street Retaining Wall
St. John's, NL

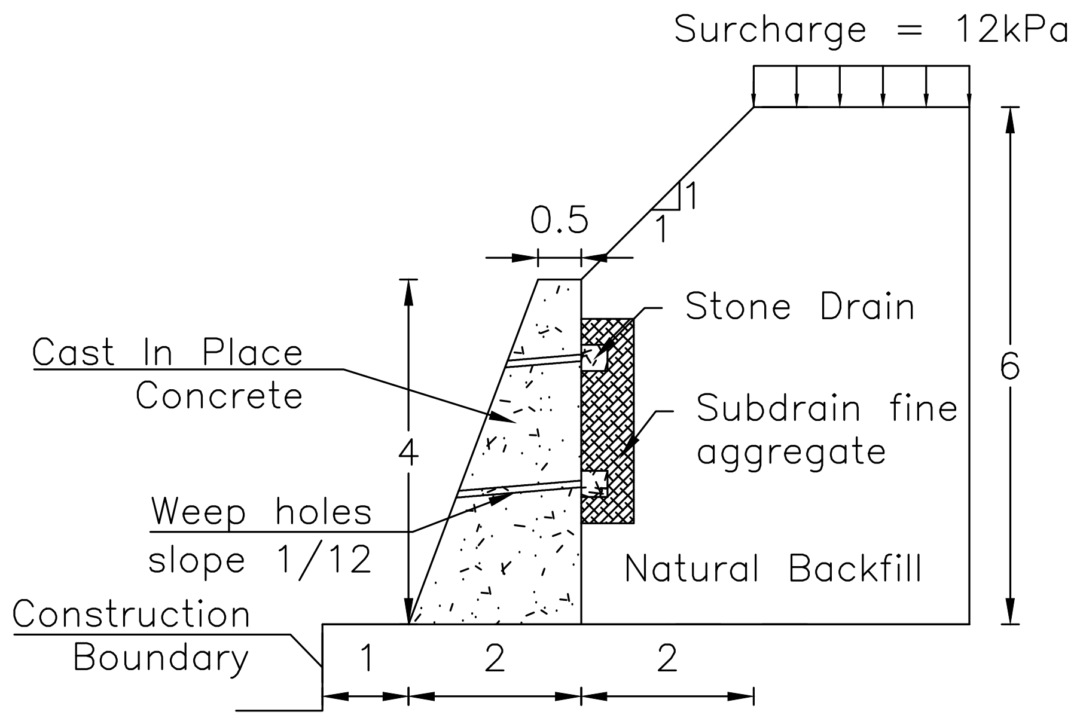
QCEC

Project
0001

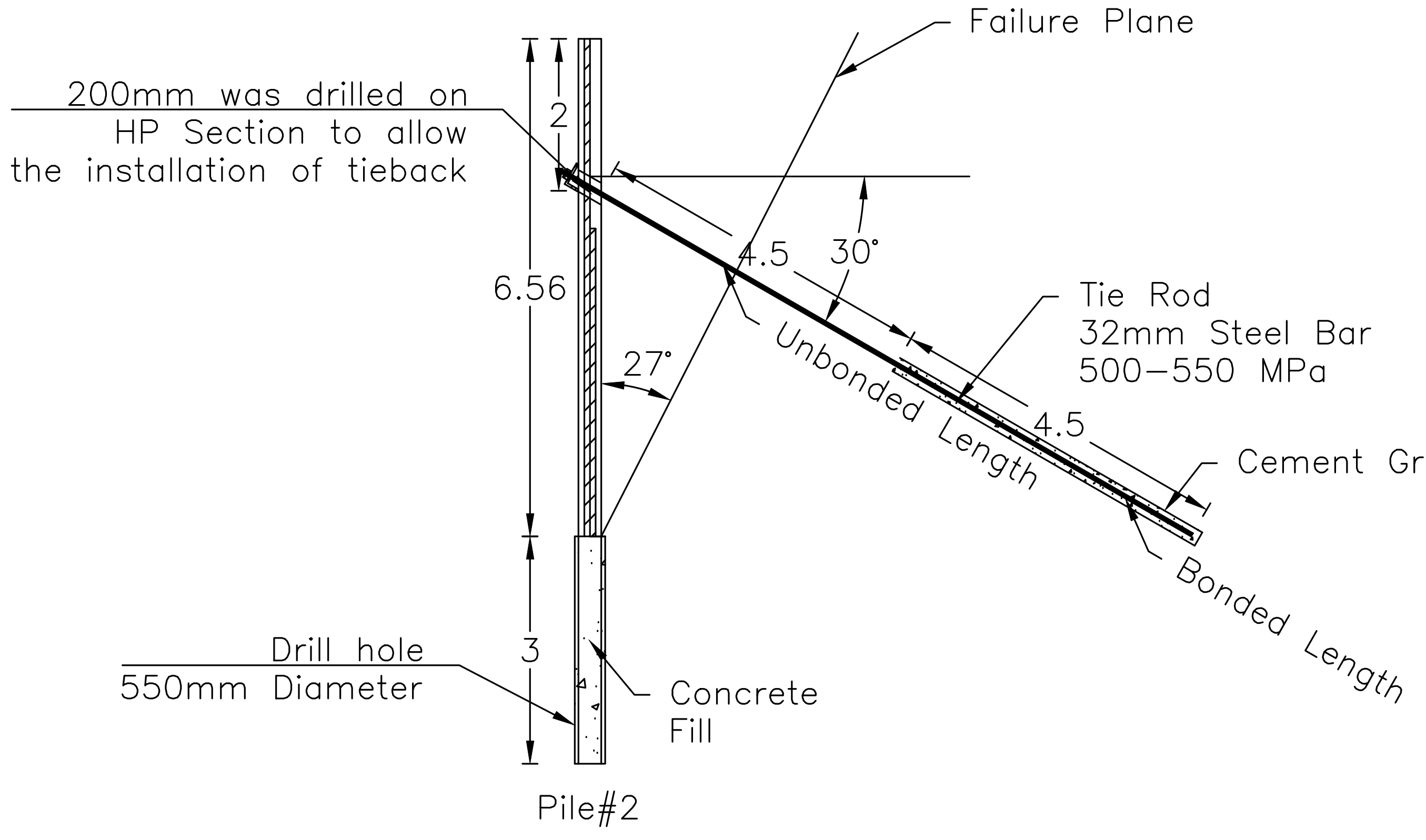
Date
03.04.2013

Scale

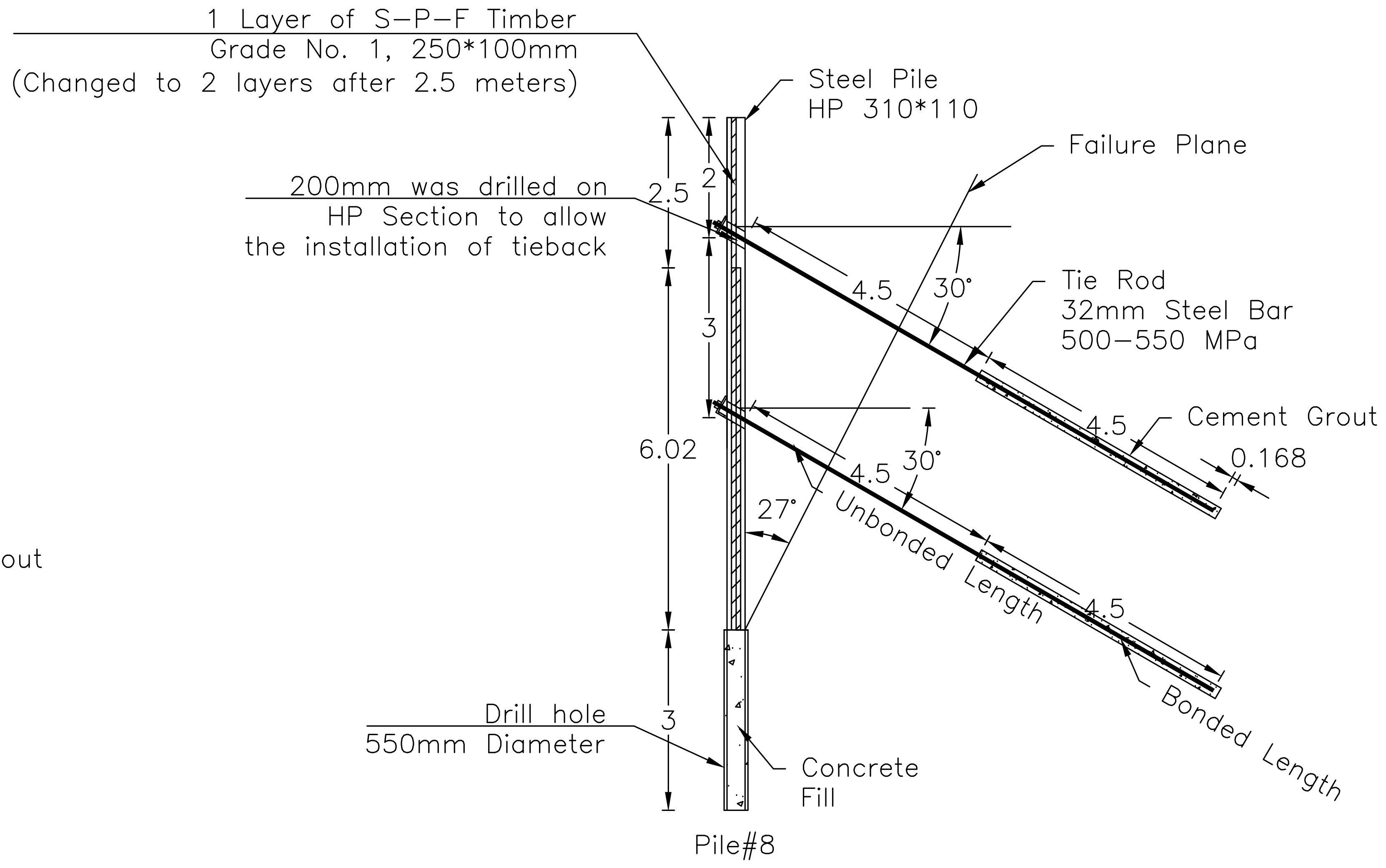
Sheet
A–03



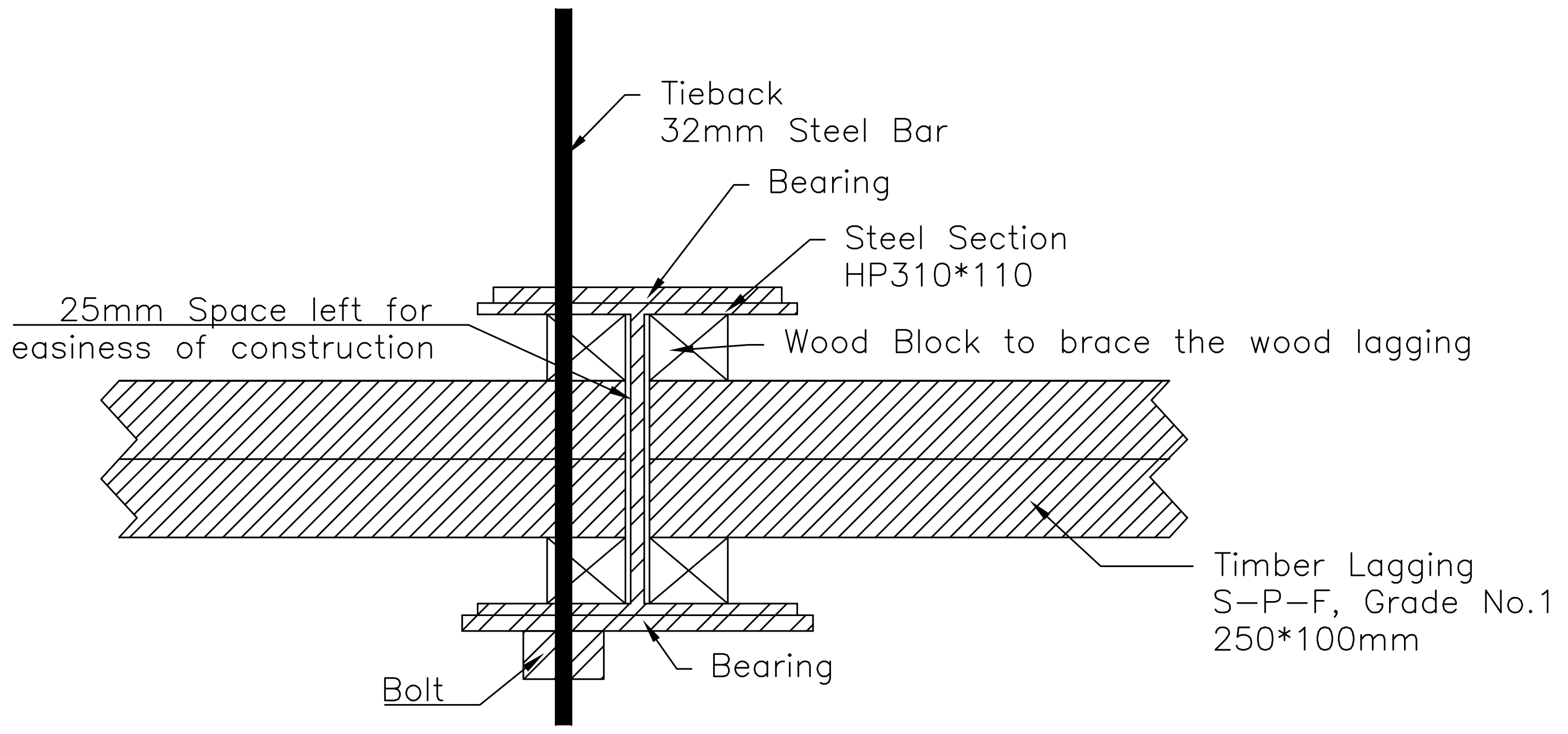
3 A1 Concrete Wall Side View



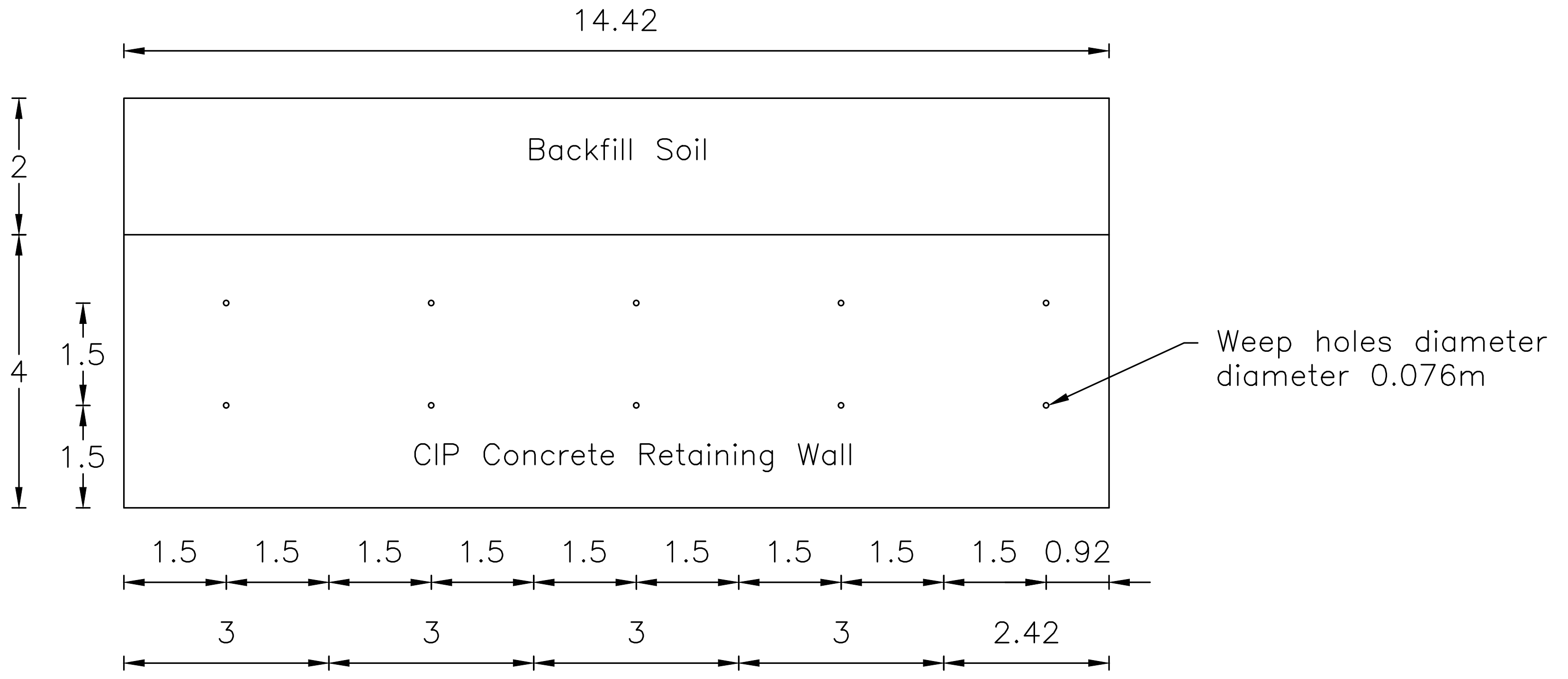
3 A2 Pile#2 Side View



3 A3 Pile#8 Side View



Detail: 3–01 Tieback Connection Detail



Detail: 3–02 Concrete Wall Front View – Weep Hole Locations

Figure 04

Soldier Pile Retaining Wall Detailed Drawings

General Notes

No.	Revision/Issue	Date

Firm Name and Address
Quality Civil Engineering Consultants
Memorial University of Newfoundland
St. John's, NL, A1B 3X5

Project Name and Address
Duckworth Street Retaining Wall
St. John's, NL

QCEC

Project
0001

Date
03.04.2013

Scale

Sheet
A–04

Appendix E

Preliminary Soil Nailing Retaining Wall Drawings

Project No.: 0001

Project Name: Duckworth Street Retaining Wall

Document: **Soil Nailing Retaining Wall Preliminary Drawing**

To:

Acuren Group Inc.

112 Forest Rd

St. John's, NL, A1A 1E6

From:

Erica Soucy, Chenel Waight, Chantel Nicolle, Qiong Zhang

Group N - QCEC

Memorial University of Newofundland

St. John's, NL, A1B 3X5

Submission Date: April 03, 2013

Drawing Lists:

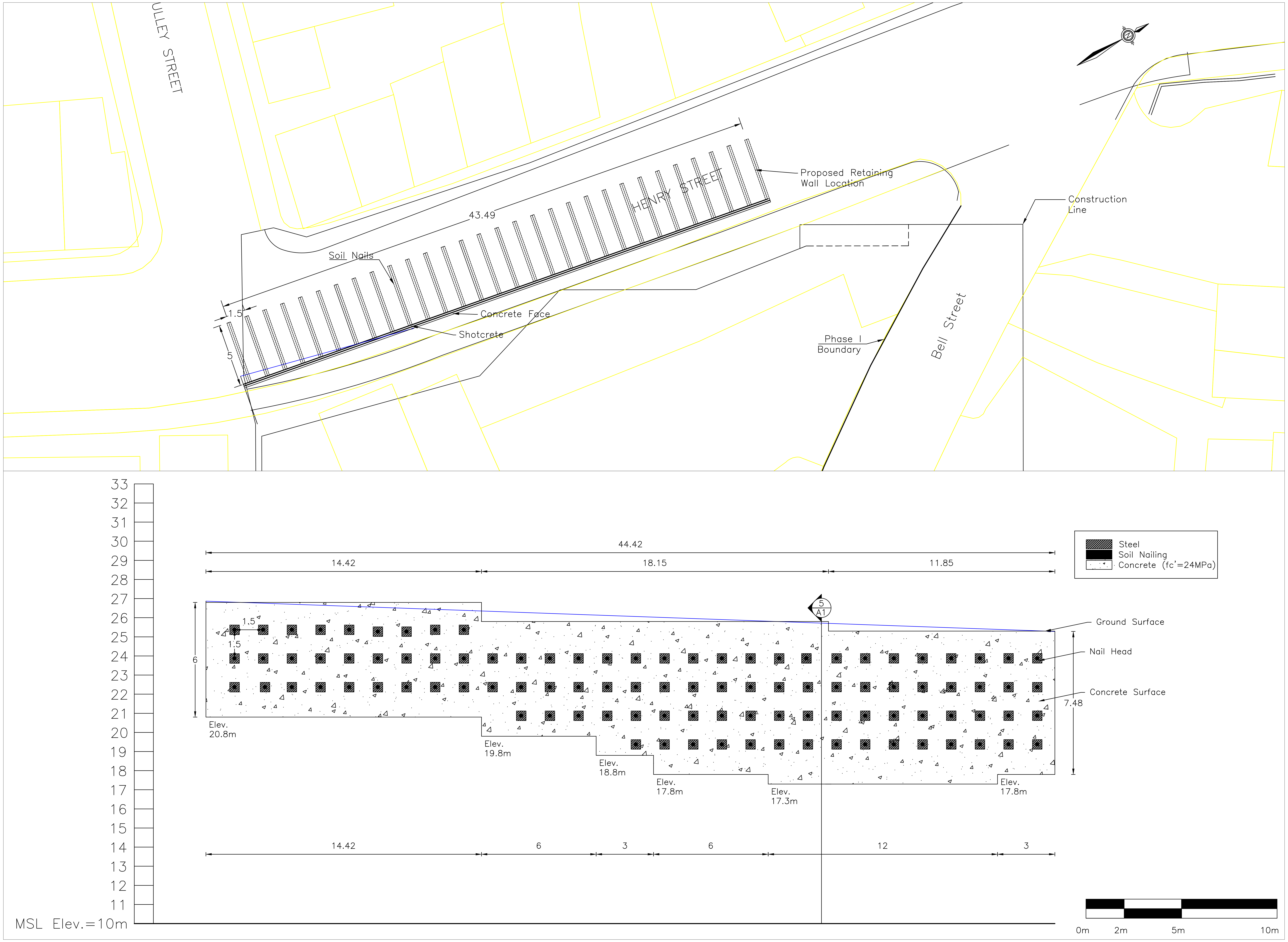


Figure 01

Soil Nailing Retaining Wall Plan View (Top) and Elevation View (Bottom)

General Notes

No.	Revision/Issue	Date

Firm Name and Address
Quality Civil Engineering Consultants
Memorial University of Newfoundland
St. John's, NL, A1B 3X5

Project Name and Address
Duckworth Street Retaining Wall
St. John's, NL

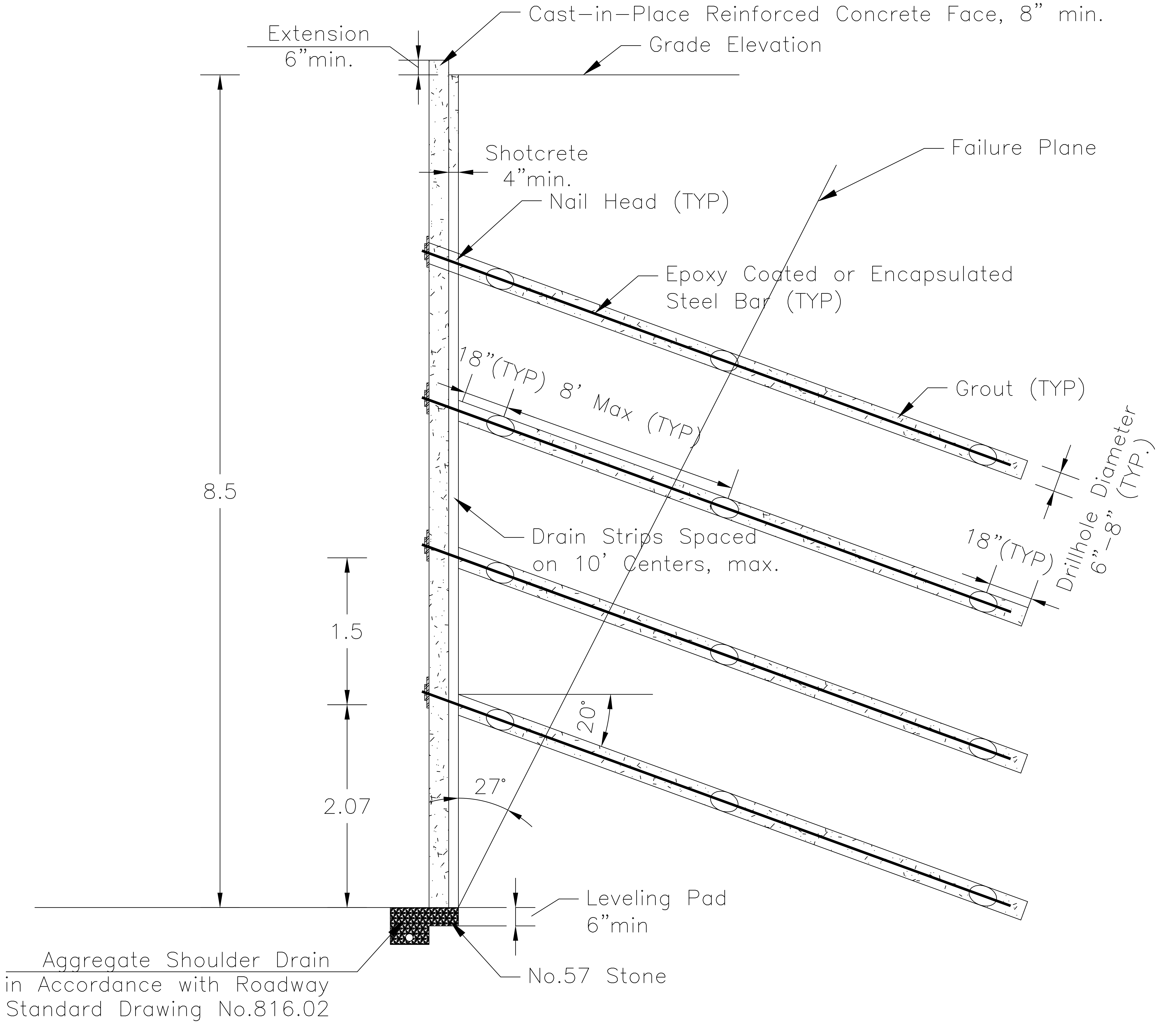
QCEC

Project
0001

Date
03.04.2013

Scale

Sheet
A–01



5
A1
Soil Nail Retaining Wall Side View

Figure 02	Soil Nail Retaining Wall Side View

General Notes

No.	Revision/Issue	Date

Firm Name and Address
Quality Civil Engineering Consultants
Memorial University of Newfoundland
St. John's, NL, A1B 3X5

Project Name and Address
Duckworth Street Retaining Wall
St. John's, NL

QCEC

Project 0001
Date 03.04.2013
Scale
Sheet A-02

Appendix F

S-FRAME and S-STEEL Results

QCEC
S.J. Carew Building
St. John's, NL, A1B 3X5

Retaining Wall Calculations

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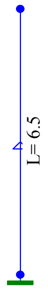
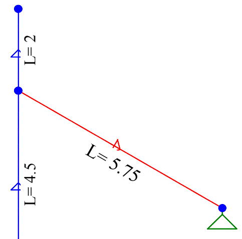
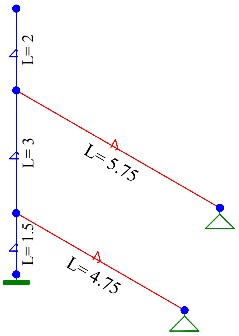
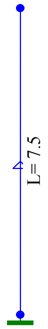
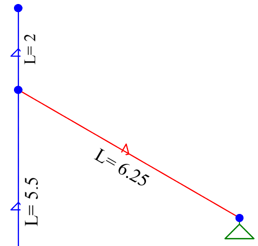
Description: Duckworth Street Retaining Wall
Engineer: Chenel C. Waight

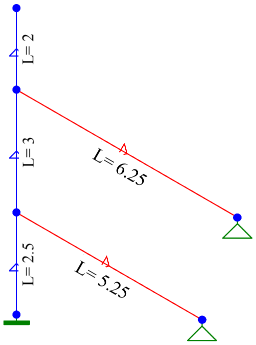


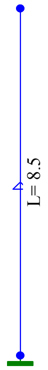


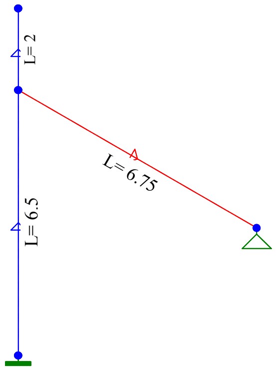


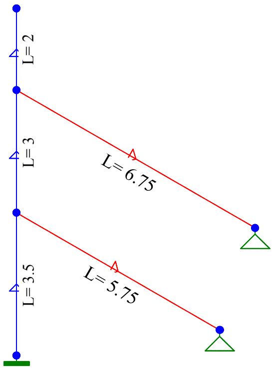












S-FRAME
Educational Version 10.0

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S.J. Carew Building
St. John's, NL, A1B 3X5

Retaining Wall Calculations

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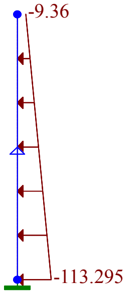
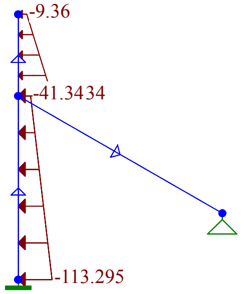
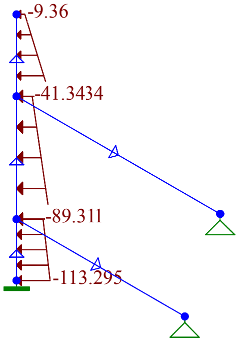
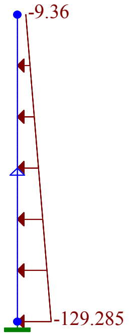
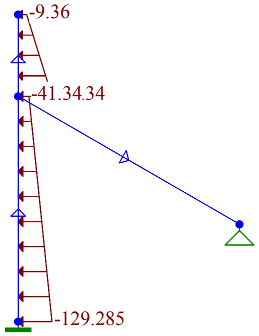
Load Case 1:
Engineer: Chenel C. Waight

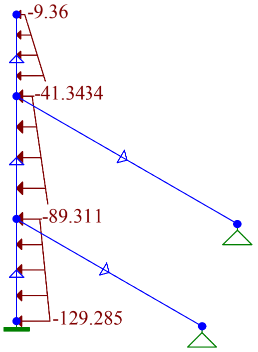


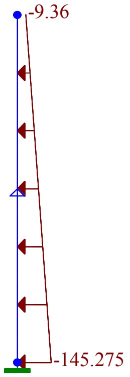


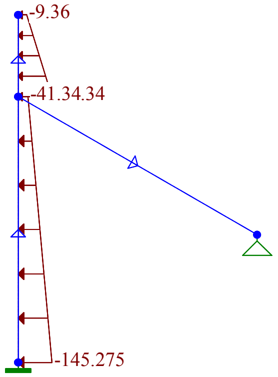


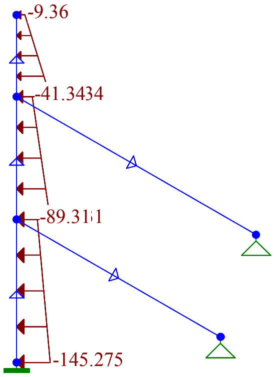












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S.J. Carew Building
St. John's, NL, A1B 3X5

Retaining Wall Calculations

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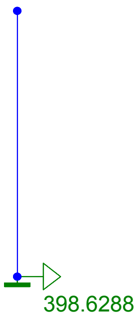
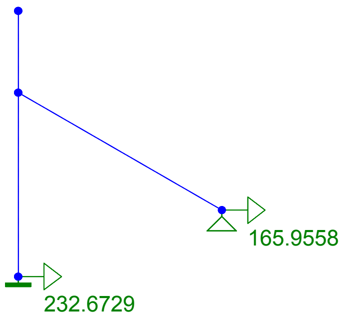
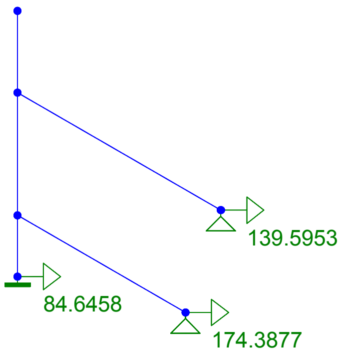
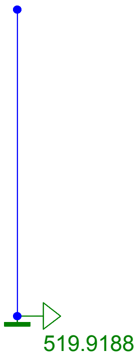
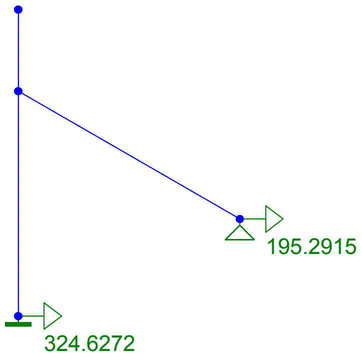
Engineer: Chenel C. Waight

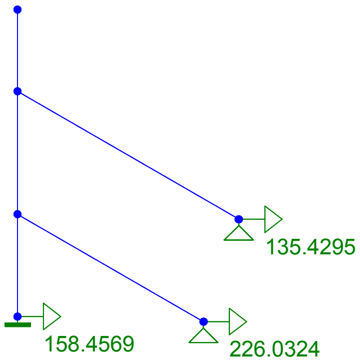


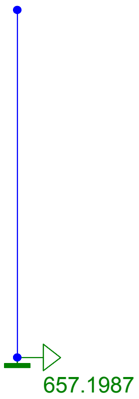


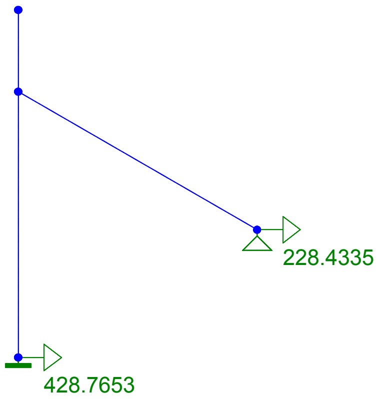


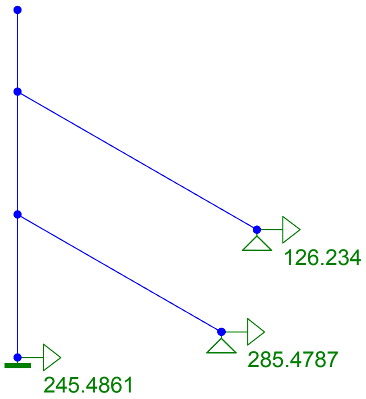












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S.J. Carew Building
St. John's, NL, A1B 3X5

Retaining Wall Calculations

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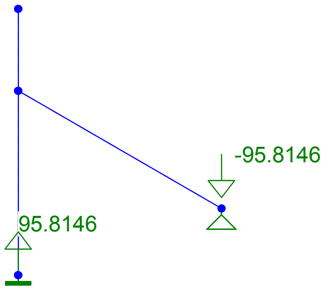
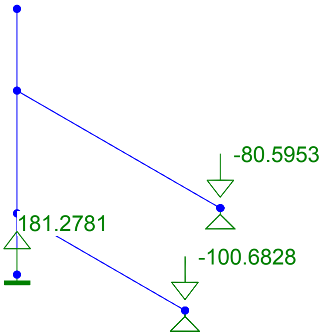
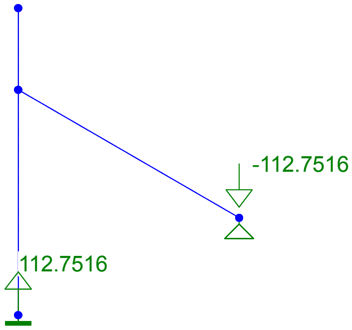
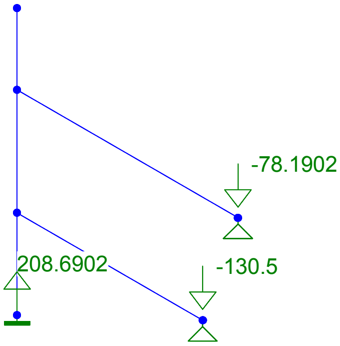
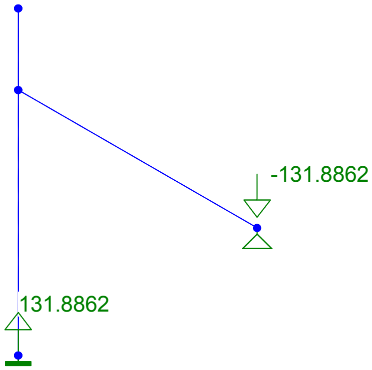
Engineer: Chenel C. Waight

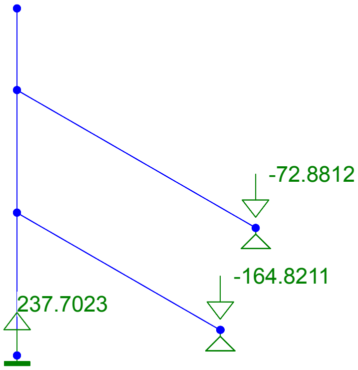












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S.J. Carew Building
St. John's, NL, A1B 3X5

Retaining Wall Calculations

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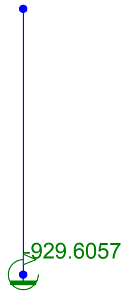
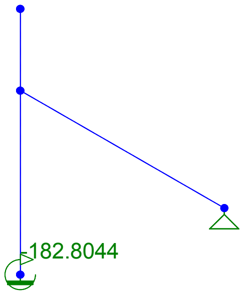
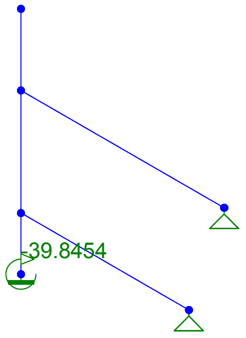
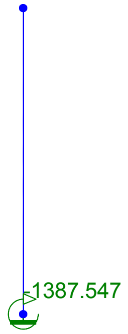
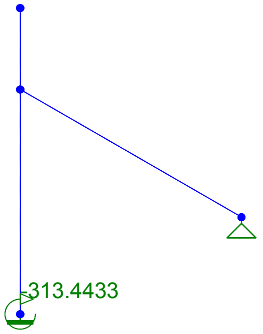
GRAPHICAL RESULTS - Ld Case 1 <Untitled> ->Mz Reaction (kN-m)
Engineer: Chenel C. Waight

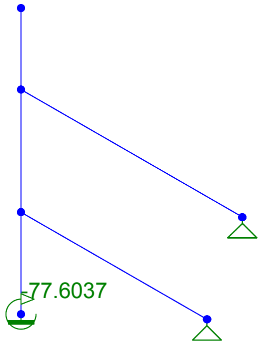


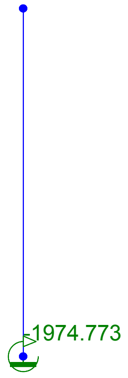


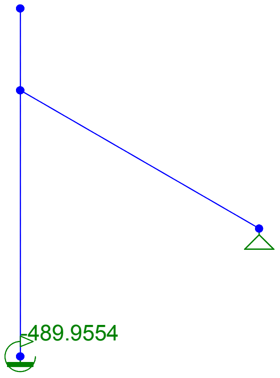


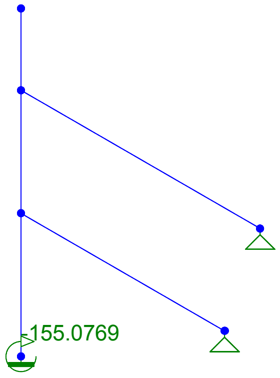












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S.J. Carew Building
St. John's, NL, A1B 3X5

Retaining Wall Calculations

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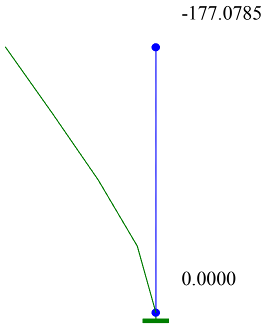
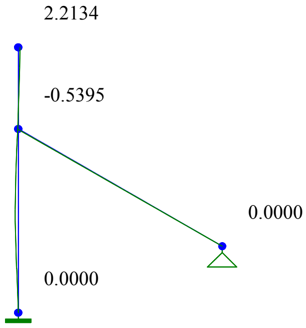
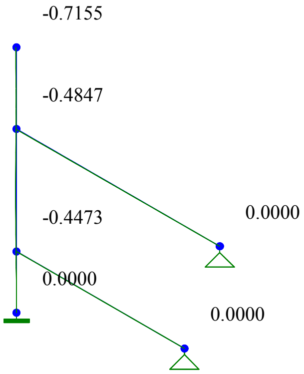
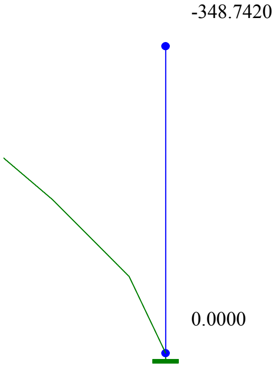
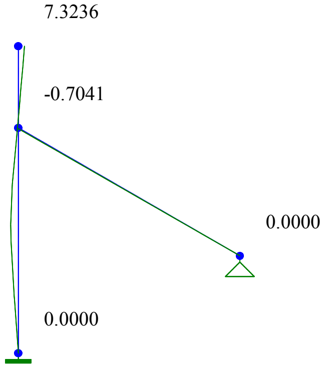
Engineer: Chenel C. Waight

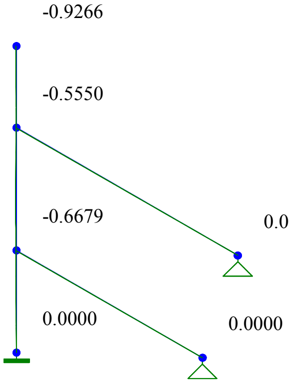


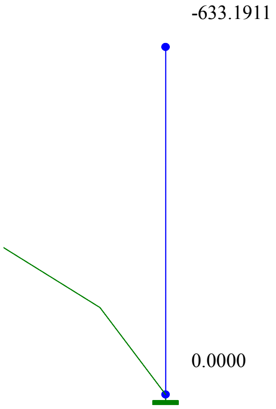


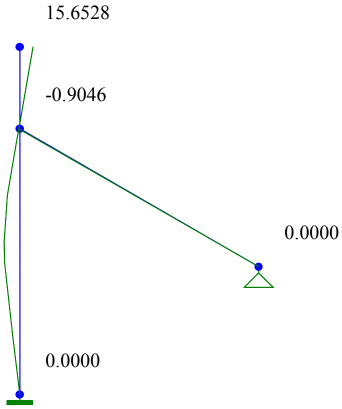


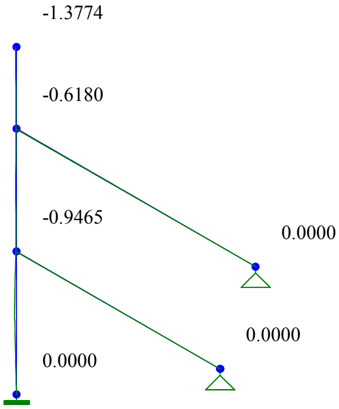












S-FRAME

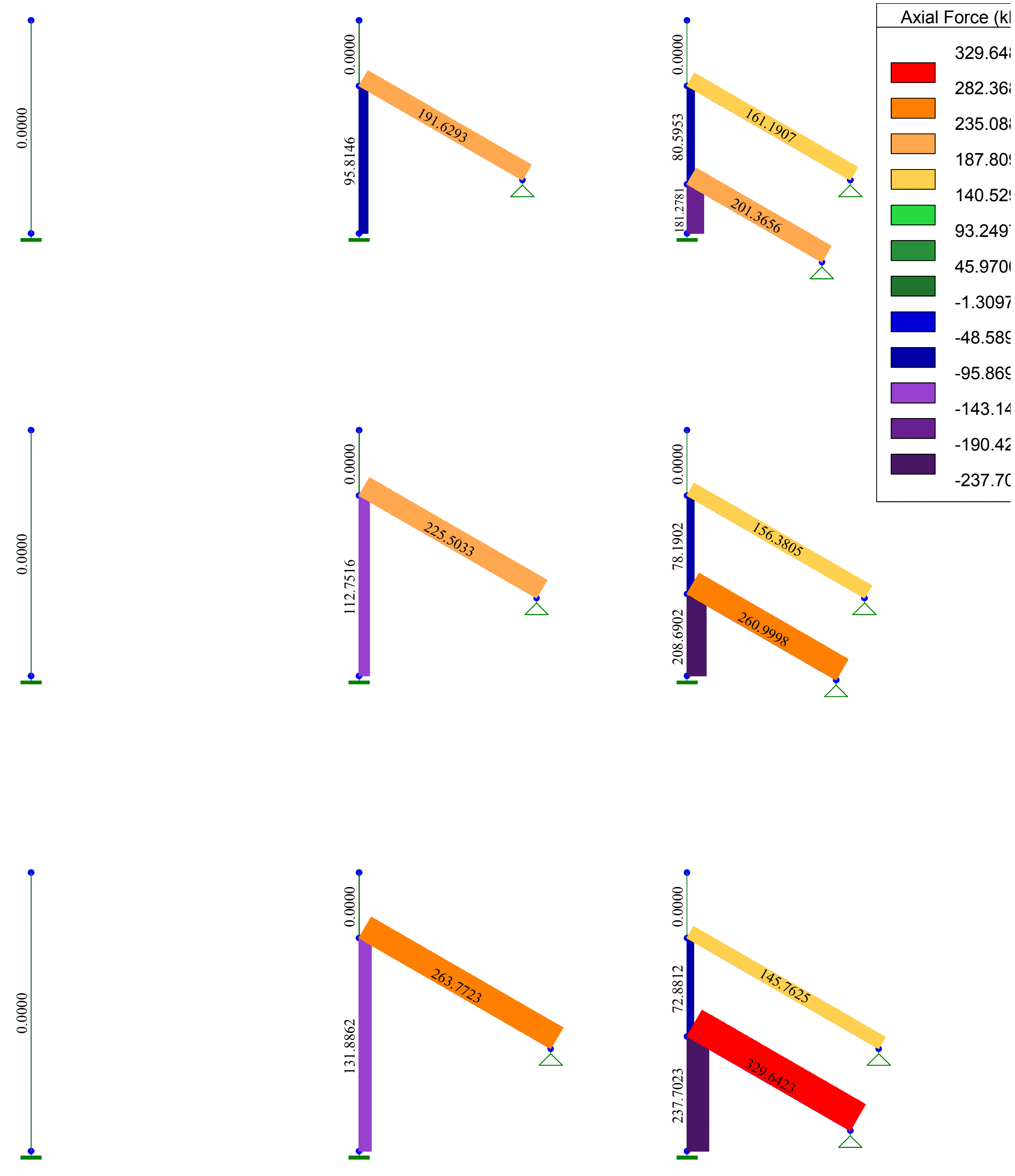
Educational Version 10.0

QCEC

S.J. Carew Building
St. John's, NL, A1B 3X5

Retaining Wall Calculations

Filename: G:\8700\S-Frame - Retaining Wall\Retaining Wall.TEL
GRAPHICAL RESULTS - Ld Case 1 <Untitled> ->Axial Force (kN)
Engineer: Chenel C. Waight



S-FRAME

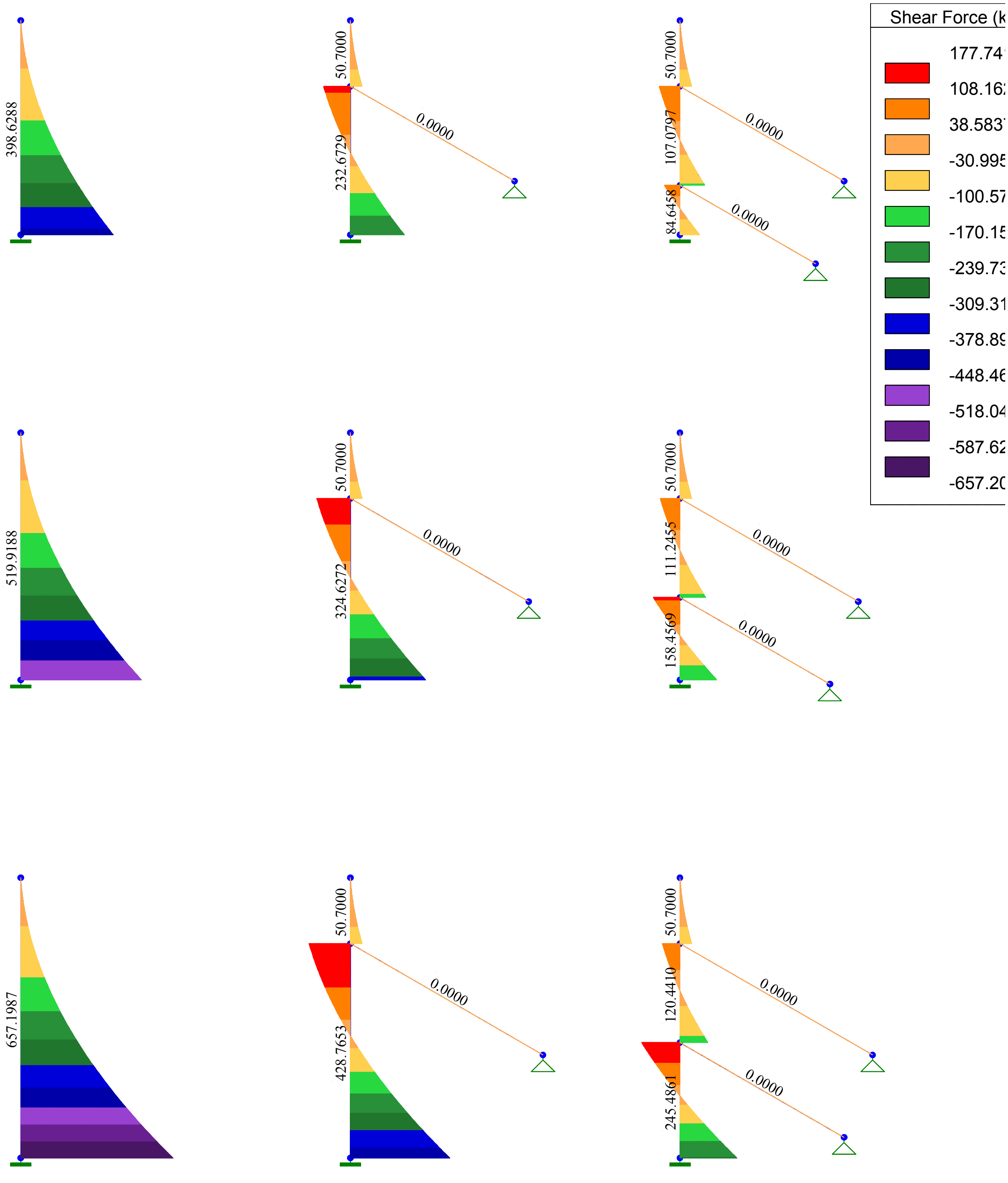
Educational Version 10.0

QCEC

S.J. Carew Building
St. John's, NL, A1B 3X5

Retaining Wall Calculations

Filename: G:\8700\S-Frame - Retaining Wall\Retaining Wall.TEL
GRAPHICAL RESULTS - Ld Case 1 <Untitled> ->Shear Force (kN)
Engineer: Chenel C. Waight



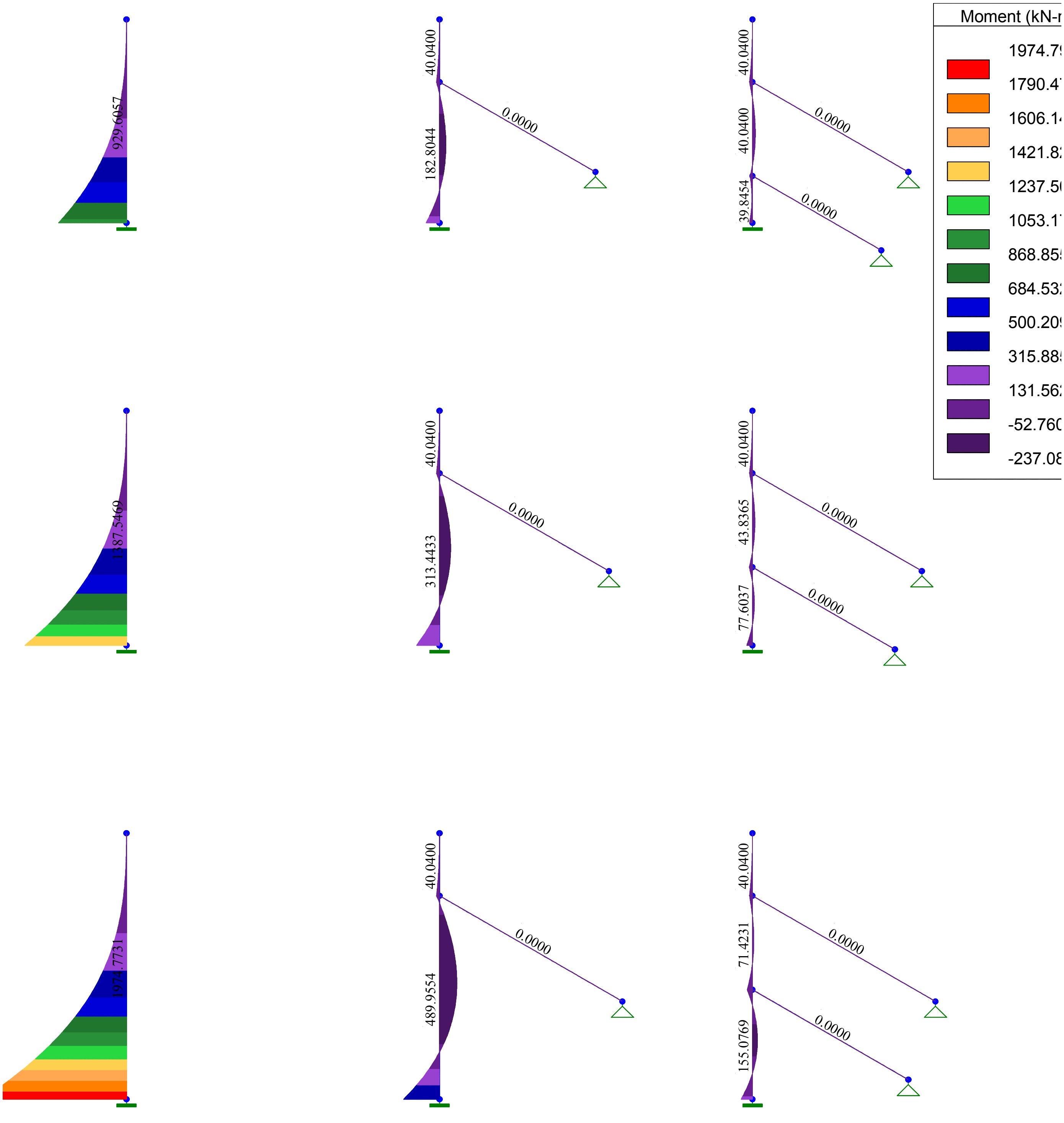
S-FRAME

Educational Version 10.0

QCEC
S.J. Carew Building
St. John's, NL, A1B 3X5

Retaining Wall Calculations

Filename: G:\8700\S-Frame - Retaining Wall\Retaining Wall.TEL
GRAPHICAL RESULTS - Ld Case 1 <Untitled> ->Moment (kN-m)
Engineer: Chenel C. Waight



S-FRAME
Educational Version 10.0

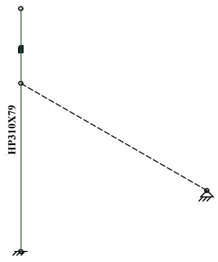
QCEC

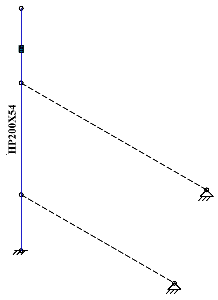
S.J. Carew Building
St. John's, NL, A1B 3X5

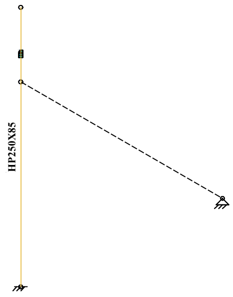
Print Screen

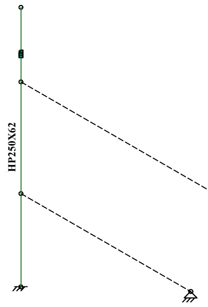
Project: Retaining Wall Calculations
Structure: Duckworth Street Retaining Wall
Filename: G:\8700\S-FRAME - RETAINING WALL\RETAINING WALL.TEL
Engineer: Chenel C. Waight

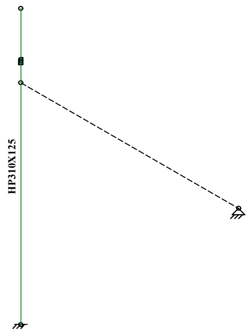
Page: 1
Date:25/03/2013

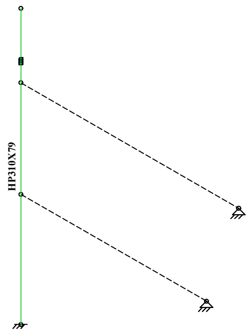










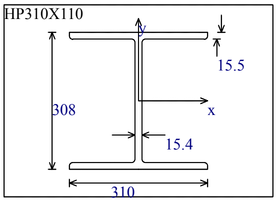


Design Code: CAN/CSA S16-09
Steel Table : Canadian (CISC)
Analysis Program: S-FRAME (Linear static analysis)

S-STEEL

Version 10.00.1 (Lite Version)

Member: 28 S-FRAME Section is HP310X110
Member is part of group: Section 1
Note: Neglecting: axial<1.0 kN, shear<1.0 kN, moment<1.0 kNm
Note: Member in braced frame(s). *Angle Gamma is -90.0 degrees*

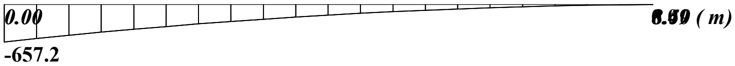


Load Case 1 (Bending)

Section classification (f_y=345 MPa); Section Class = **3**

Clause 11

Strong Axis Shear - (kN)

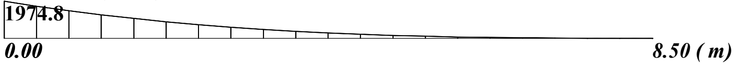


Strong axis shear strength check

A_w= 4743 mm^2; $\frac{V_{fy}}{\phi\ A_w\ F_s} = \frac{V_{fy}}{\phi\ A_w\ 0.66F_y} = \frac{657}{972} =$ **0.676**

Clause 13.4.1.1(a)

Strong Axis Moment - (kN-m)

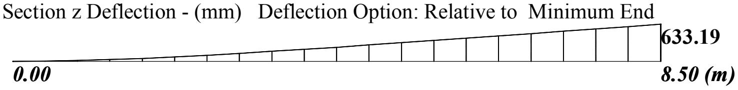


Bending Stability Check

L_u=8.50 m; ω_2=2.500; $\frac{M_{fx}}{M_{rx}} = \frac{1975}{478} =$ **4.130**

Clause 13.6(b)
> 1.00; FAIL

S-FRAME Section z Deflection - (mm) Deflection Option: Relative to Minimum End

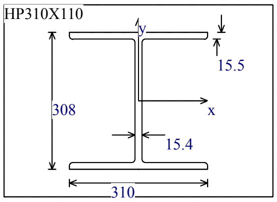


Deflection check

Max deflection =633.19mm; $\frac{(L/Deflection)_{Limit}}{L/(Deflection)_{Max}} = \frac{200}{13} =$ **> 10**

Clause 6.3.1
> 1.00; FAIL

Member (continuous): [29-30] S-FRAME Section is HP310X110
Member is part of group: Section 1
Note: Neglecting: axial<1.0 kN, shear<1.0 kN, moment<1.0 kNm
Note: Member in braced frame(s). *Angle Gamma is -90.0 degrees*



Load Case 1 (Bending + Compression)

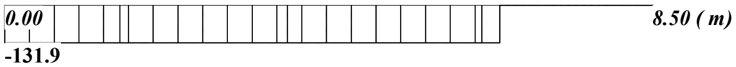
Section classification (f_y=345 MPa); Section Class = **3**

Clause 11

Governing geometrical slenderness ratio
k_x=1.00; k_y=1.00; k_xL/r_x=65.6; $\frac{k_y L/r_y}{200} = \frac{115}{200} =$ **0.575**

Clause 10.4.2.1

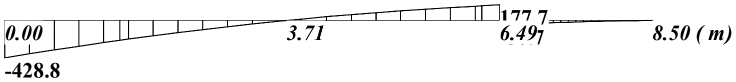
Axial Load - (kN)



Factored Compressive Resistance Check
n=1.34; λ_y=1.520 $\frac{C_f}{C_{ry}} = \frac{C_f}{\phi A F_y (1+\lambda^{2n})^{-1/n}} = \frac{C_f}{\phi A\ (121\ \text{MPa})} = \frac{132}{1536} =$ **0.086**

Clause 13.3.1

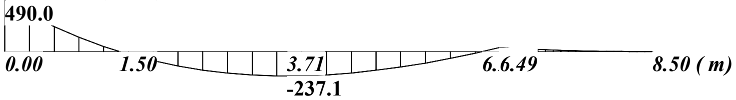
Strong Axis Shear - (kN)



Strong axis shear strength check
A_w= 4743 mm^2; $\frac{V_{fy}}{\phi\ A_w\ F_s} = \frac{V_{fy}}{\phi\ A_w\ 0.66F_y} = \frac{429}{972} =$ **0.441**

Clause 13.4.1.1(a)

Strong Axis Moment - (kN-m)



Bending Stability Check
L_u=8.50 m; ω_2=2.432; $\frac{M_{fx}}{M_{rx}} = \frac{490}{478} =$ **1.025**

Clause 13.6(b)
> 1.00; FAIL

Axial Compression and Bending cross-sectional Strength Check
ω_{1x}=1.00; U_{1x}=1.02; $\frac{C_f}{\phi\ A\ F_y} + \frac{M_{fx}}{\phi\ S_x\ F_y} =$ **1.076**

Clause 13.8.3(a)
> 1.00; FAIL

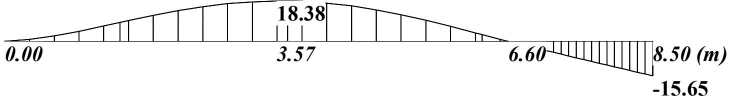
Axial Compression and Bending overall member Strength Check
ω_{1x}=1.00; U_{1x}=1.02; $\frac{C_f}{C_{rx}} + \frac{U_{1x}\ M_{fx}}{\phi\ S_x\ F_y} =$ **1.090**

Clause 13.8.3(b)
> 1.00; FAIL

Axial Compression and Bending lateral torsional buckling strength check
ω_{1x}=1.00; U_{1x}=1.02; $\frac{C_f}{C_{ry}} + \frac{U_{1x}\ M_{fx}}{M_{rx}} =$ **1.132**

Clause 13.8.3(c)
> 1.00; FAIL

S-FRAME Section z Deflection - (mm) Deflection Option: Relative to Minimum End



Deflection check
Max deflection =18.38mm; $\frac{(L/\text{Deflection})_{Limit}}{L/(\text{Deflection})_{Max}} = \frac{200}{463} =$ **0.432**

Clause 6.3.1

QCEC

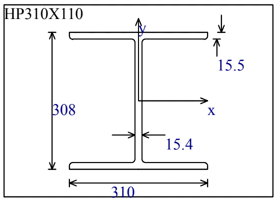
S.J. Carew Building
St. John's, NL, A1B 3X5

Code Details

Project: Retaining Wall Calculations
Structure: Duckworth Street Retaining Wall
Filename: G:\8700\S-FRAME - RETAINING WALL\RETAINING WALL.TEL
Engineer: Chenel C. Waight

Page: 1
Date:25/03/2013

Member (continuous): [31-32-33] S-FRAME Section is HP310X110
Member is part of group: Section 1
Note: Neglecting: axial<1.0 kN, shear<1.0 kN, moment<1.0 kNm
Note: Member in braced frame(s). *Angle Gamma is -90.0 degrees*



Load Case 1 (Bending + Compression)

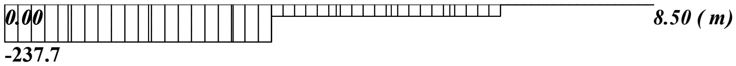
Section classification (f_y=345 MPa); Section Class = **3**

Clause 11

Governing geometrical slenderness ratio
k_x=1.00; k_y=1.00; $k_x L/r_x$=65.6; $\frac{k_y L/r_y}{200} = \frac{115}{200} =$ **0.575**

Clause 10.4.2.1

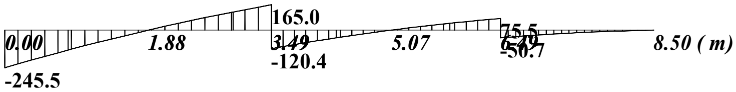
Axial Load - (kN)



Factored Compressive Resistance Check
n=1.34; λ_y=1.520 $\frac{C_f}{C_{ry}} = \frac{C_f}{\phi A F_y (1+\lambda^{2n})^{-1/n}} = \frac{C_f}{\phi A\ (121\ \text{MPa})} = \frac{238}{1536} =$ **0.155**

Clause 13.3.1

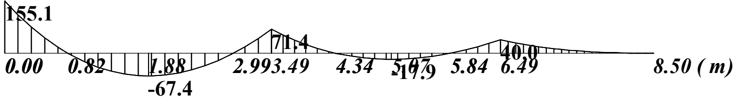
Strong Axis Shear - (kN)



Strong axis shear strength check
A_w= 4743 mm^2; $\frac{V_{fy}}{\phi A_w F_s} = \frac{V_{fy}}{\phi A_w 0.66 F_y} = \frac{245}{972} =$ **0.253**

Clause 13.4.1.1(a)

Strong Axis Moment - (kN-m)



Bending Stability Check
L_u=8.50 m; ω_2=2.500; $\frac{M_{fx}}{M_{rx}} = \frac{155}{478} =$ **0.324**

Clause 13.6(b)

Axial Compression and Bending cross-sectional Strength Check
ω_{1x}=1.00; U_{1x}=1.04; $\frac{C_f}{\phi A F_y} + \frac{M_{fx}}{\phi S_x F_y} =$ **0.391**

Clause 13.8.3(a)

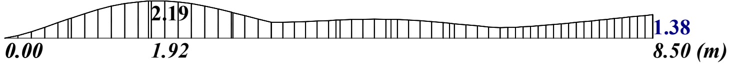
Axial Compression and Bending overall member Strength Check
ω_{1x}=1.00; U_{1x}=1.04; $\frac{C_f}{C_{rx}} + \frac{U_{1x} M_{fx}}{\phi S_x F_y} =$ **0.417**

Clause 13.8.3(b)

Axial Compression and Bending lateral torsional buckling strength check
ω_{1x}=1.00; U_{1x}=1.04; $\frac{C_f}{C_{ry}} + \frac{U_{1x} M_{fx}}{M_{rx}} =$ **0.491**

Clause 13.8.3(c)

S-FRAME Section z Deflection - (mm) Deflection Option: Relative to Minimum End



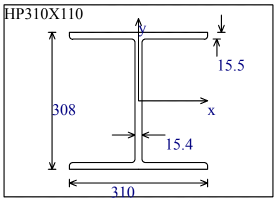
Deflection check
Max deflection =2.19mm; $\frac{(L/\text{Deflection})_{Limit}}{L/(\text{Deflection})_{Max}} = \frac{200}{3876} =$ **0.052**

Clause 6.3.1

Design Code: CAN/CSA S16-09
Steel Table : Canadian (CISC)
Analysis Program: S-FRAME (Linear static analysis)

S-STEEL
Version 10.00.1 (Lite Version)

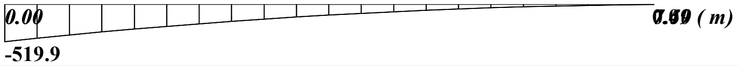
Member: 7 S-FRAME Section is HP310X110
Member is part of group: Section 1
Note: Neglecting: axial<1.0 kN, shear<1.0 kN, moment<1.0 kNm
Note: Member in braced frame(s). *Angle Gamma is -90.0 degrees*



⇨ **Load Case 1 (Bending)**

Section classification (f_y=345 MPa); Section Class = **3** — Clause 11

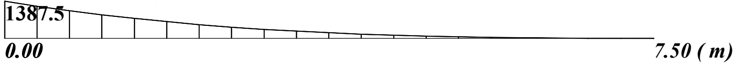
Strong Axis Shear - (kN)



Strong axis shear strength check — Clause 13.4.1.1(a)

A_w= 4743 mm^2; $\frac{V_{fy}}{\phi\ A_w\ F_s} = \frac{V_{fy}}{\phi\ A_w\ 0.66F_y} = \frac{520}{972} =$ **0.535**

Strong Axis Moment - (kN-m)

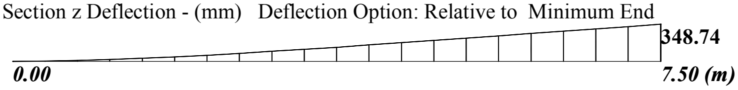


Bending Stability Check — Clause 13.6(b)

L_u=7.50 m; ω_2=2.500; $\frac{M_{fx}}{M_{rx}} = \frac{1388}{478} =$ **2.902**

> 1.00; FAIL

S-FRAME Section z Deflection - (mm) Deflection Option: Relative to Minimum End



⇨ **Deflection check** — **Clause 6.3.1**

Max deflection =348.74mm; $\frac{\mathbf{(L/Deflection)_{Limit}}}{\mathbf{L/(Deflection)_{Max}}} = \frac{200}{22} =$ **9.300**

> 1.00; FAIL

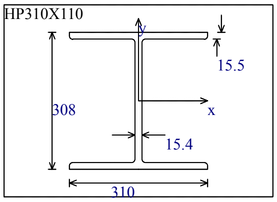
QCEC S.J. Carew Building St. John's, NL, A1B 3X5	*Code Details* Project: Retaining Wall Calculations Structure: Duckworth Street Retaining Wall Filename: G:\8700\S-FRAME - RETAINING WALL\RETAINING WALL.TEL Engineer: Chenel C. Waight	Page: 1 Date:25/03/2013

Member (continuous): [8-9] S-FRAME Section is HP310X110

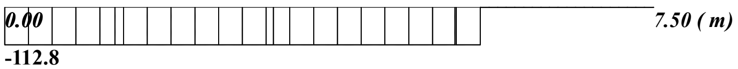
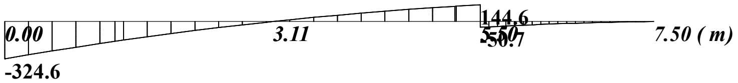
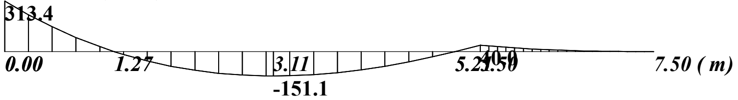
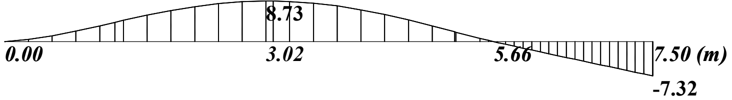
Member is part of group: Section 1

Note: Neglecting: axial<1.0 kN, shear<1.0 kN, moment<1.0 kNm

Note: Member in braced frame(s). *Angle Gamma is -90.0 degrees*



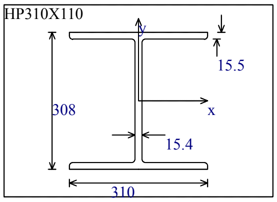
⇨ **Load Case 1 (Bending + Compression)**

Section classification (f_y=345 MPa); Section Class =	**3**	Clause 11
Governing geometrical slenderness ratio k_x=1.00; k_y=1.00; k_xL/r_x=57.8; $\frac{k_y L/r_y}{200} = \frac{101}{200} =$	**0.507**	Clause 10.4.2.1
Axial Load - (kN) 		
Factored Compressive Resistance Check n=1.34; λ_y=1.341 $\frac{C_f}{C_{ry}} = \frac{C_f}{\phi A F_y(1+\lambda^{2n})^{-1/n}} = \frac{C_f}{\phi A\ (145\ \text{MPa})} = \frac{113}{1840} =$	**0.061**	Clause 13.3.1
Strong Axis Shear - (kN) 		
Strong axis shear strength check A_w= 4743 mm^2; $\frac{V_{fy}}{\phi A_w F_s} = \frac{V_{fy}}{\phi A_w 0.66F_y} = \frac{325}{972} =$	**0.334**	Clause 13.4.1.1(a)
Strong Axis Moment - (kN-m) 		
Bending Stability Check L_u=7.50 m; ω_2=2.464; $\frac{M_{fx}}{M_{rx}} = \frac{313}{478} =$	**0.656**	Clause 13.6(b)
Axial Compression and Bending cross-sectional Strength Check ω_{1x}=1.00; U_{1x}=1.01; $\frac{C_f}{\phi A F_y} + \frac{M_{fx}}{\phi S_x F_y} =$	**0.690**	Clause 13.8.3(a)
Axial Compression and Bending overall member Strength Check ω_{1x}=1.00; U_{1x}=1.01; $\frac{C_f}{C_{rx}} + \frac{U_{1x} M_{fx}}{\phi S_x F_y} =$	**0.699**	Clause 13.8.3(b)
⇨ **Axial Compression and Bending lateral torsional buckling strength check** **ω_{1x}=1.00; U_{1x}=1.01;** $\frac{C_f}{C_{ry}} + \frac{U_{1x} M_{fx}}{M_{rx}} =$	**0.726**	**Clause 13.8.3(c)**
S-FRAME Section z Deflection - (mm) Deflection Option: Relative to Minimum End 		
Deflection check Max deflection =8.73mm; $\frac{(L/\text{Deflection})_{Limit}}{L/(\text{Deflection})_{Max}} = \frac{200}{859} =$	**0.233**	Clause 6.3.1

Design Code: CAN/CSA S16-09
Steel Table : Canadian (CISC)
Analysis Program: S-FRAME (Linear static analysis)

S-STEEL
Version 10.00.1 (Lite Version)

Member (continuous): [10-11-12] S-FRAME Section is HP310X110
Member is part of group: Section 1
Note: Neglecting: axial<1.0 kN, shear<1.0 kN, moment<1.0 kNm
Note: Member in braced frame(s). *Angle Gamma is -90.0 degrees*



Load Case 1 (Bending + Compression)

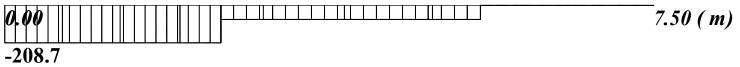
Section classification (f_y=345 MPa); Section Class = **3**

Clause 11

Governing geometrical slenderness ratio
k_x=1.00; k_y=1.00; k_xL/r_x=57.8; $\frac{k_y L/r_y}{200} = \frac{101}{200} =$ **0.507**

Clause 10.4.2.1

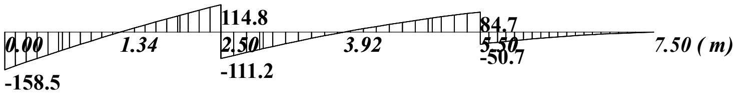
Axial Load - (kN)



Factored Compressive Resistance Check
n=1.34; λ_y=1.341 $\frac{C_f}{C_{ry}} = \frac{C_f}{\phi A F_y (1+\lambda^{2n})^{-1/n}} = \frac{C_f}{\phi A\ (145\ \text{MPa})} = \frac{209}{1840} =$ **0.113**

Clause 13.3.1

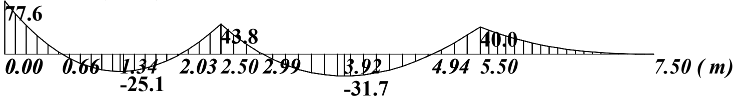
Strong Axis Shear - (kN)



Strong axis shear strength check
A_w= 4743 mm^2; $\frac{V_{fy}}{\phi A_w F_s} = \frac{V_{fy}}{\phi A_w 0.66 F_y} = \frac{158}{972} =$ **0.163**

Clause 13.4.1.1(a)

Strong Axis Moment - (kN-m)



Bending Stability Check
L_u=7.50 m; ω_2=2.343; $\frac{M_{fx}}{M_{rx}} = \frac{78}{478} =$ **0.162**

Clause 13.6(b)

Axial Compression and Bending cross-sectional Strength Check
ω_{1x}=1.00; U_{1x}=1.03; $\frac{C_f}{\phi A F_y} + \frac{M_{fx}}{\phi S_x F_y} =$ **0.214**

Clause 13.8.3(a)

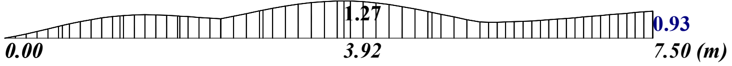
Axial Compression and Bending overall member Strength Check
ω_{1x}=1.00; U_{1x}=1.03; $\frac{C_f}{C_{rx}} + \frac{U_{1x} M_{fx}}{\phi S_x F_y} =$ **0.231**

Clause 13.8.3(b)

Axial Compression and Bending lateral torsional buckling strength check
ω_{1x}=1.00; U_{1x}=1.03; $\frac{C_f}{C_{ry}} + \frac{U_{1x} M_{fx}}{M_{rx}} =$ **0.280**

Clause 13.8.3(c)

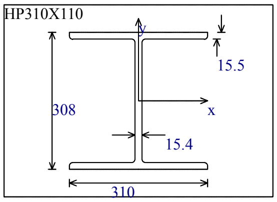
S-FRAME Section z Deflection - (mm) Deflection Option: Relative to Minimum End



Deflection check
Max deflection =1.27mm; $\frac{(L/\text{Deflection})_{Limit}}{L/(\text{Deflection})_{Max}} = \frac{200}{5886} =$ **0.034**

Clause 6.3.1

Member: 13 S-FRAME Section is HP310X110
Member is part of group: Section 1
Note: Neglecting: axial<1.0 kN, shear<1.0 kN, moment<1.0 kNm
Note: Member in braced frame(s). *Angle Gamma is -90.0 degrees*

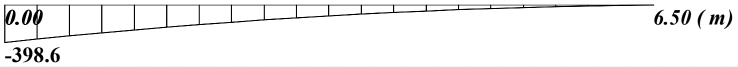


Load Case 1 (Bending)

Section classification (f_y=345 MPa); Section Class = **3**

Clause 11

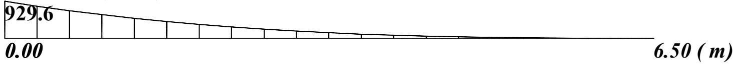
Strong Axis Shear - (kN)



Strong axis shear strength check
A_w= 4743 mm^2; $\frac{V_{fy}}{\phi A_w F_s} = \frac{V_{fy}}{\phi A_w 0.66F_y} = \frac{399}{972} =$ **0.410**

Clause 13.4.1.1(a)

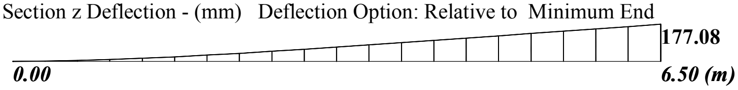
Strong Axis Moment - (kN-m)



Bending Stability Check
L_u=6.50 m; ω_2=2.500; $\frac{M_{fx}}{M_{rx}} = \frac{930}{478} =$ **1.944**

Clause 13.6(b)
> 1.00; FAIL

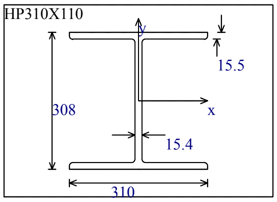
S-FRAME Section z Deflection - (mm) Deflection Option: Relative to Minimum End



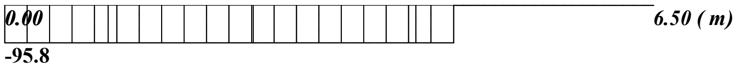
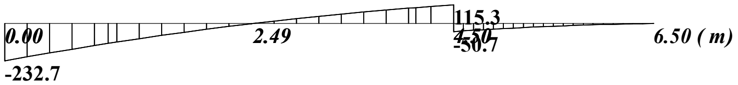
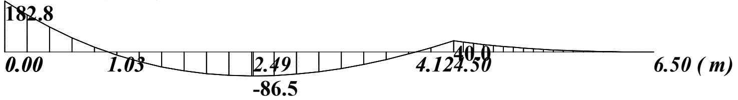
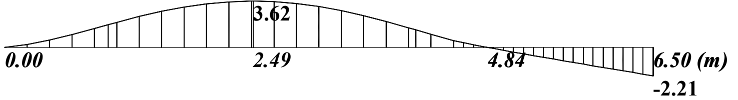
Deflection check
Max deflection =177.08mm; $\frac{(L/Deflection)_{Limit}}{L/(Deflection)_{Max}} = \frac{200}{37} =$ **5.449**

Clause 6.3.1
> 1.00; FAIL

Member (continuous): [14-15] S-FRAME Section is HP310X110
Member is part of group: Section 1
Note: Neglecting: axial<1.0 kN, shear<1.0 kN, moment<1.0 kNm
Note: Member in braced frame(s). *Angle Gamma is -90.0 degrees*



⇨ **Load Case 1 (Bending + Compression)**

Section classification (f_y=345 MPa);	Section Class =	3	Clause 11
⇨ **Governing geometrical slenderness ratio** **k_x=1.00; k_y=1.00; k_xL/r_x=50.1;**	$\frac{k_y\ L/r_y}{200} = \frac{88}{200} =$	**0.440**	**Clause 10.4.2.1**
Axial Load - (kN)  			
Factored Compressive Resistance Check n=1.34; λ_y=1.162	$\frac{C_f}{C_{ry}} = \frac{C_f}{\phi AF_y(1+\lambda^{2n})^{-1/n}} = \frac{C_f}{\phi A\ (174\ MPa)} = \frac{96}{2212} =$	0.043	Clause 13.3.1
Strong Axis Shear - (kN)  			
Strong axis shear strength check A_w= 4743 mm^2;	$\frac{V_{fy}}{\phi\ A_w\ F_s} = \frac{V_{fy}}{\phi\ A_w\ 0.66F_y} = \frac{233}{972} =$	0.239	Clause 13.4.1.1(a)
Strong Axis Moment - (kN-m)  			
Bending Stability Check L_u=6.50 m; ω_2=2.500;	$\frac{M_{fx}}{M_{rx}} = \frac{183}{478} =$	0.382	Clause 13.6(b)
Axial Compression and Bending cross-sectional Strength Check ω_{1x}=1.00; U_{1x}=1.01;	$\frac{C_f}{\phi\ A\ F_y} + \frac{M_{fx}}{\phi\ S_x\ F_y} =$	0.408	Clause 13.8.3(a)
Axial Compression and Bending overall member Strength Check ω_{1x}=1.00; U_{1x}=1.01;	$\frac{C_f}{C_{rx}} + \frac{U_{1x}\ M_{fx}}{\phi\ S_x\ F_y} =$	0.413	Clause 13.8.3(b)
Axial Compression and Bending lateral torsional buckling strength check ω_{1x}=1.00; U_{1x}=1.01;	$\frac{C_f}{C_{ry}} + \frac{U_{1x}\ M_{fx}}{M_{rx}} =$	0.429	Clause 13.8.3(c)
S-FRAME Section z Deflection - (mm) Deflection Option: Relative to Minimum End  			
Deflection check Max deflection =3.62mm;	$\frac{(L/Deflection)_{Limit}}{L/(Deflection)_{Max}} = \frac{200}{1795} =$	0.111	Clause 6.3.1

QCEC

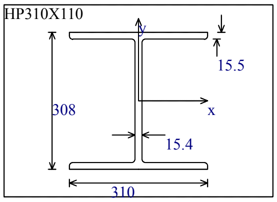
S.J. Carew Building
St. John's, NL, A1B 3X5

Code Details

Project: Retaining Wall Calculations
Structure: Duckworth Street Retaining Wall
Filename: G:\8700\S-FRAME - RETAINING WALL\RETAINING WALL.TEL
Engineer: Chenel C. Waight

Page: 1
Date:25/03/2013

Member (continuous): [16-17-18] S-FRAME Section is HP310X110
Member is part of group: Section 1
Note: Neglecting: axial<1.0 kN, shear<1.0 kN, moment<1.0 kNm
Note: Member in braced frame(s). *Angle Gamma is -90.0 degrees*

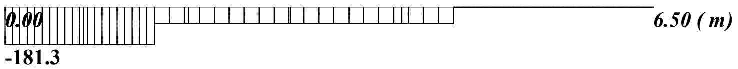


Load Case 1 (Bending + Compression)

Section classification (f_y=345 MPa); Section Class = **3** — Clause 11

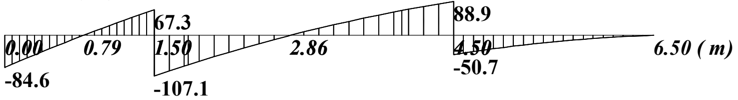
Governing geometrical slenderness ratio — **Clause 10.4.2.1**
k_x=1.00; k_y=1.00; k_xL/r_x=50.1; $\frac{k_y\ L/r_y}{200} = \frac{88}{200} =$ **0.440**

Axial Load - (kN)



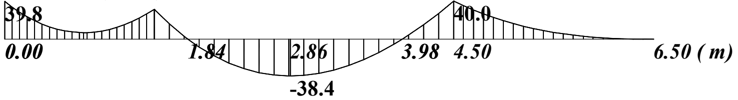
Factored Compressive Resistance Check — Clause 13.3.1
n=1.34; λ_y=1.162 $\frac{C_f}{C_{ry}} = \frac{C_f}{\phi A F_y (1+\lambda^{2n})^{-1/n}} = \frac{C_f}{\phi A\ (174\ \text{MPa})} = \frac{181}{2212} =$ **0.082**

Strong Axis Shear - (kN)



Strong axis shear strength check — Clause 13.4.1.1(a)
A_w= 4743 mm^2; $\frac{V_{fy}}{\phi\ A_w\ F_s} = \frac{V_{fy}}{\phi\ A_w\ 0.66F_y} = \frac{107}{972} =$ **0.110**

Strong Axis Moment - (kN-m)



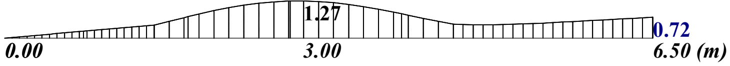
Bending Stability Check — Clause 13.6(b)
L_u=6.50 m; ω_2=1.399; $\frac{M_{fx}}{M_{rx}} = \frac{40}{476} =$ **0.084**

Axial Compression and Bending cross-sectional Strength Check — Clause 13.8.3(a)
ω_{1x}=1.00; U_{1x}=1.02; $\frac{C_f}{\phi\ A\ F_y} + \frac{M_{fx}}{\phi\ S_x\ F_y} =$ **0.127**

Axial Compression and Bending overall member Strength Check — Clause 13.8.3(b)
ω_{1x}=1.00; U_{1x}=1.02; $\frac{C_f}{C_{rx}} + \frac{U_{1x}\ M_{fx}}{\phi\ S_x\ F_y} =$ **0.136**

Axial Compression and Bending lateral torsional buckling strength check — Clause 13.8.3(c)
ω_{1x}=1.00; U_{1x}=1.02; $\frac{C_f}{C_{ry}} + \frac{U_{1x}\ M_{fx}}{M_{rx}} =$ **0.168**

S-FRAME Section z Deflection - (mm) Deflection Option: Relative to Minimum End



Deflection check — Clause 6.3.1
Max deflection =1.27mm; $\frac{(L/\text{Deflection})_{\text{Limit}}}{L/(\text{Deflection})_{\text{Max}}} = \frac{200}{5103} =$ **0.039**

Design Code: CAN/CSA S16-09
Steel Table : Canadian (CISC)
Analysis Program: S-FRAME (Linear static analysis)

S-STEEL
Version 10.00.1 (Lite Version)

Appendix G

Design Anchor Capacity Estimates

exp.

Design Anchor Capacity Estimates - New Parking Garage and Condominium
Henry Bell Developments Limited, St. John's, NL

Geotechnical Resistance Factor = ϕ

Ultimate Anchor Capacity $P_{ult} = \alpha_{bond} \times \text{PI} \times \text{Diam}_{socket} \times \text{bond-length}$

Factored Ultimate Anchor Capacity $\phi P_{ult} = \phi \times \alpha_{bond} \times \text{PI} \times \text{Diam}_{socket} \times \text{bond-length}$

$\phi_{\text{Axial Tension - empircal data}} =$ 0.3 Current analyses - designing using ultimate bond stress based on rock and soil types

$\phi_{\text{Axial Tension - load test}} =$ 0.4 Alternative approach - design to be confirmed prior to permanent anchor construction by load testing

Basis For Geotechnical Resistance Factor	Bond Diameter mm	Bond Length m	Ultimate Bond Stress α_{bond}, kPa	Ultimate Anchor Capacity P_{ult}, kN	Factored Ultimate Anchor Capacity ϕP_{ult}, kN	Material Type Over Bonded Length
Empirical Design Approach $\phi = 0.3$	168	3.0	300	475	143	Anchor bonded in random **sand and gravel fill**
	168	4.5	300	713	214	
	168	3.0	500	792	238	Anchor bonded in **till or severely fractured bedrock**
	168	4.5	500	1188	356	
	168	3.0	800	1267	380	Anchor bonded in **fractured sandstone bedrock**
	168	4.5	800	1900	570	
Design Confirmed by Load Testing Prior To Production Anchor Construction $\phi = 0.4$	168	3.0	300	475	190	Anchor bonded in random **sand and gravel fill**
	168	4.5	300	713	285	
	168	3.0	500	792	317	Anchor bonded in **till or severely fractured bedrock**
	168	4.5	500	1188	475	
	168	3.0	800	1267	507	Anchor bonded in **fractured sandstone bedrock**
	168	4.5	800	1900	760	

January 30, 2012

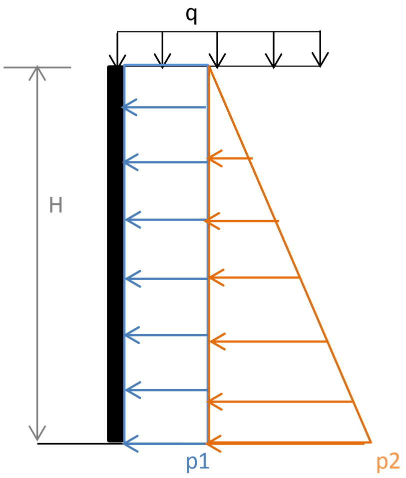
exp Services Inc.
B. Walker

Appendix H

Soldier Pile Retaining Wall Calculations

Wall Pressure Calculations

ϒ=	20.5	KN/m^3
Ka=	0.26	
ϕ =	36	°
q =	12	kPa
H =	8.5	m

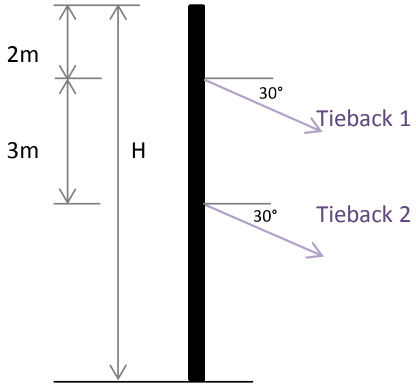


Find pressure on retaining wall from surcharge (q):

p1 = Ka*q = 3.12 kPa

Find active soil pressure on retaining wall:

p2 = Ka* ϒ * H = 45.305 kPa

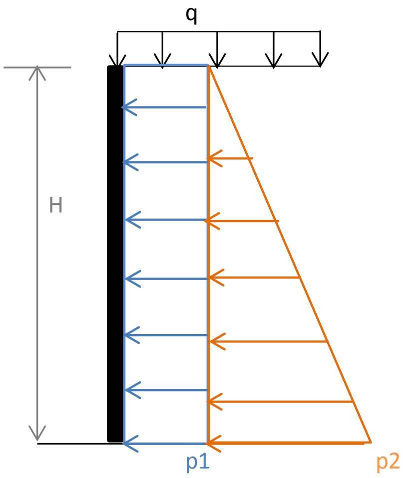


MAX allowable deflection (.005H) = 42.5 mm

Reference: US Army Corps of Engineers

Wall Pressure Calculations

Υ= 20.5 KN/m^3
Ka= 0.26
ϕ = 36 °
q = 12 kPa
H = 7.5 m

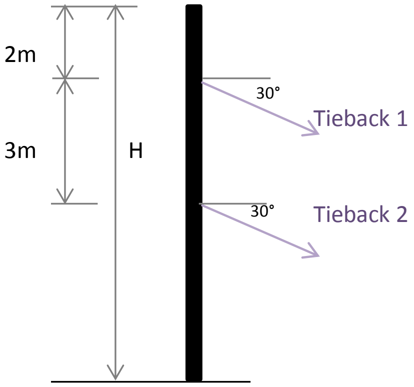


Find pressure on retaining wall from surcharge (q):

p1 = Ka*q = 3.12 kPa

Find active soil pressure on retaining wall:

p2 = Ka* ϒ * H = 39.975 kPa

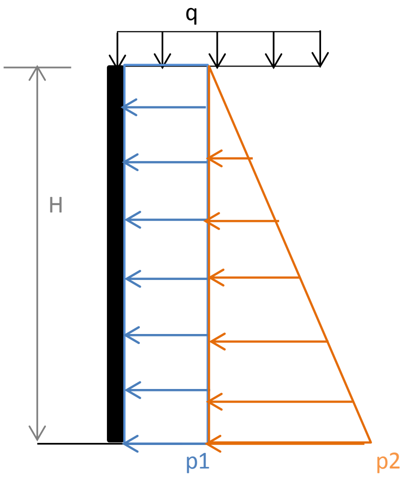


MAX allowable deflection (.005H) = 37.5 mm

Reference: US Army Corps of Engineers

Wall Pressure Calculations

ϒ= 20.5 KN/m^3
Ka= 0.26
ϕ = 36 °
q = 12 kPa
H = 6.5 m

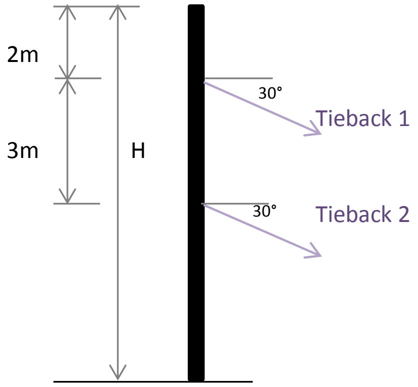


Find pressure on retaining wall from surcharge (q):

p1 = Ka*q = 3.12 kPa

Find active soil pressure on retaining wall:

p2 = Ka* ϒ * H = 34.645 kPa



MAX allowable deflection (.005H) = 32.5 mm

Reference: US Army Corps of Engineers

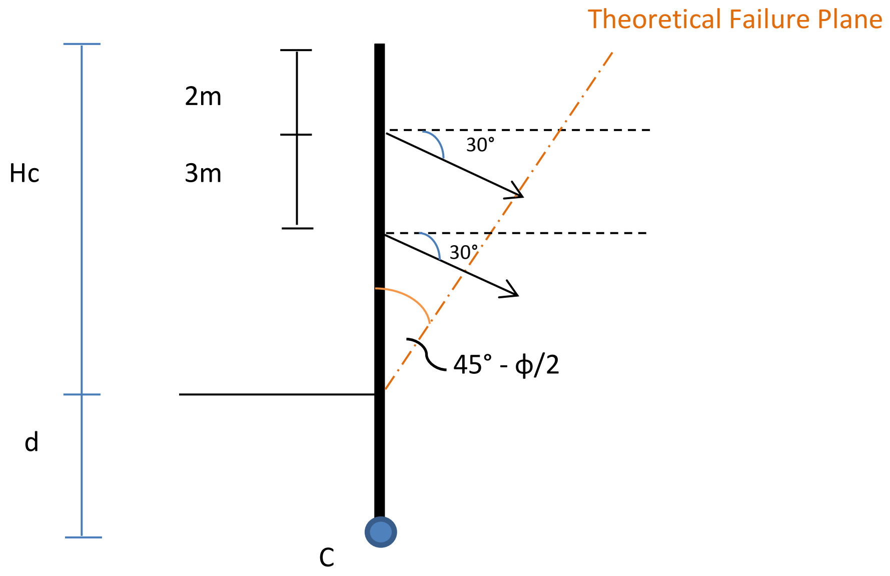
Tieback Design Calculations

Assumption:

1. Non-cohensive soil
2. Groundwater level is neglectable
3. Non-inclined backfill
4. Use ϕ' as ϕ'_{cs} and ϕ'_{p}

Notes:

1. Using Rankine's Equation
2. Critical design depth equals to 8.41 meters
3. Using Safety Factor equals to 1.5
4. Drilled hole diameter = 550mm



Reference: US Army Corps of Engineers

For Hc = 8.5m

27 deg = 0.471239 rad

failure plane	
x (m)	y
0	0
1	1.962611
2	3.925221
3	5.887832
4	7.850442
5	9.813053

Tieback Design Calculations

First Tieback 30 deg = 0.523599 rad

x (m)	y (m)
0	6.5
1	5.92265
2	5.345299
3	4.767949
4	4.190599
5	3.613249

Second Tieback 30 deg = 0.523599

x (m)	y (m)
0	3.5
1	2.92265
2	2.345299
3	1.767949
4	1.190599
5	0.613249

From graph:

First tieback intercepts failure plane at:

x = 2.55 m

y = 5 m

Length of tieback from face of wall to theoretical failure plane = 2.9 m

Second tieback intercepts failure plane at:

x = 1.4 m

y = 2.7 m

Length of tieback from face of wall to theoretical failure plane = 1.6 m

Minimum anchor free length

The greater of:

i) length of tieback from face of wall to theoretical failure plane + H/5

ii) length of tieback from face of wall to theoretical failure plane + 1.5

Reference: US Army Corps of Engineers

First Tieback

i) 2.9m + 8.5m/5 = 4.6 m

ii) 2.9m + 1.5m = 4.4 m

Anchor free length = 4.5 m **

Second Tieback

i) 1.7m + 8.5m/5 = 3.3 m

ii) 1.7m + 1.5m = 3.1 m

Anchor free length = 3.5 m

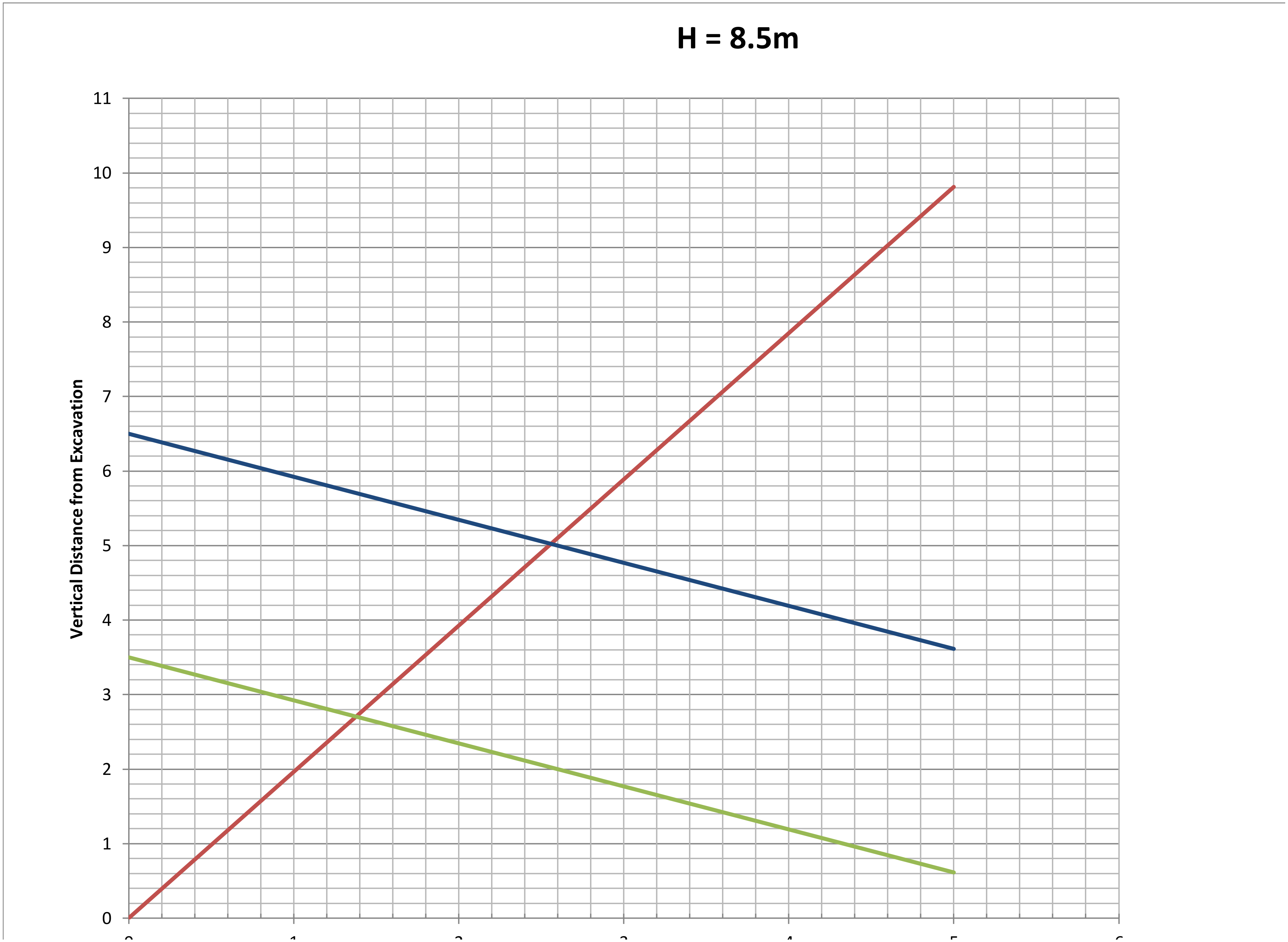
Tieback Design Calculations

Lengths in the:

				X-dir (m)	Y-dir (m)
Bond length is either 3.0m or 4.5m depending on required capacity:					
Total anchor length (First Tieback) =		7.5	m	6.5	3.8
	OR	9	m	7.8	4.5
Total anchor length (Second Tieback) =		6.5	m	5.6	3.3
	OR	8	m	6.9	4.0

NOTE: Available horizontal distance from soldier pile wall to buildings across Henry St. Is approximately 8.5m

Therefore, the maximum tie lengths will be ok for the site


H = 8.5m
Vertical Distance from Excavation
11
10
9
8
7
6
5
4
3
2
1
0

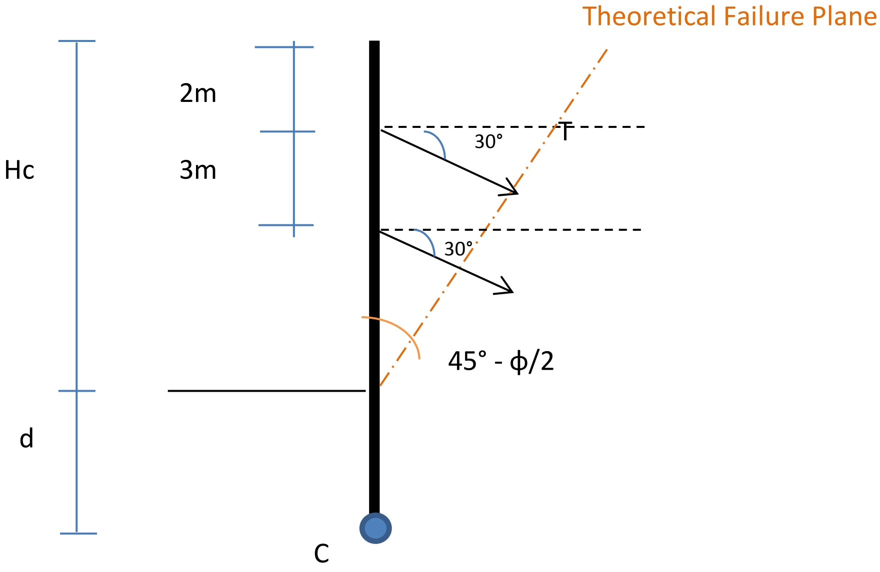
Tieback Design Calculations

Assumption:

1 Non-cohensive soil
2 Groundwater level is neglectable
3 Non-inclined backfill
4 Use ϕ' as ϕ'_{cs} and ϕ'_p

Notes:

1 Using Rankine's Equation
2 Critical design depth equals to 8.41 meters
3 Using Safety Factor equals to 1.5
4 Drilled hole diameter = 550mm



Reference: US Army Corps of Engineers

For Hc = 7.5m

27 deg = 0.471239 rad

failure plane

x (m)	y
0	0
1	1.962611
2	3.925221
3	5.887832
4	7.850442
5	9.813053

First Tieback 30 deg = 0.523599 rad

x (m)	y (m)
0	5.5
1	4.92265
2	4.345299
3	3.767949

4	3.190599
5	2.613249

Second Tieback 30 deg = 0.523599

x (m)	y (m)
0	2.5
1	1.92265
2	1.345299
3	0.767949
4	0.190599
5	-0.38675

From graph:

First tieback intercepts failure plane at:

x = 2.15 m

y = 4.2 m

Length of tieback from face of wall to theoretical failure plane = 2.5 m

Second tieback intercepts failure plane at:

x = 1 m

y = 1.9 m

Length of tieback from face of wall to theoretical failure plane = 1.2 m

Minimum anchor free length

The greater of:

i) length of tieback from face of wall to theoretical failure plane + H/5

ii) length of tieback from face of wall to theoretical failure plane + 1.5

Reference: US Army Corps of Engineers

First Tieback

i) 2.5m + 7.5m/5 = 4.0

ii) 2.5m + 1.5m = 4.0

Anchor free length = 4 m

Second Tieback

Anchor free length = 3 m

i) 1.1m + 7.5m/5 = 2.7

ii) 1.1m + 1.5m = 2.7

Lengths in the:

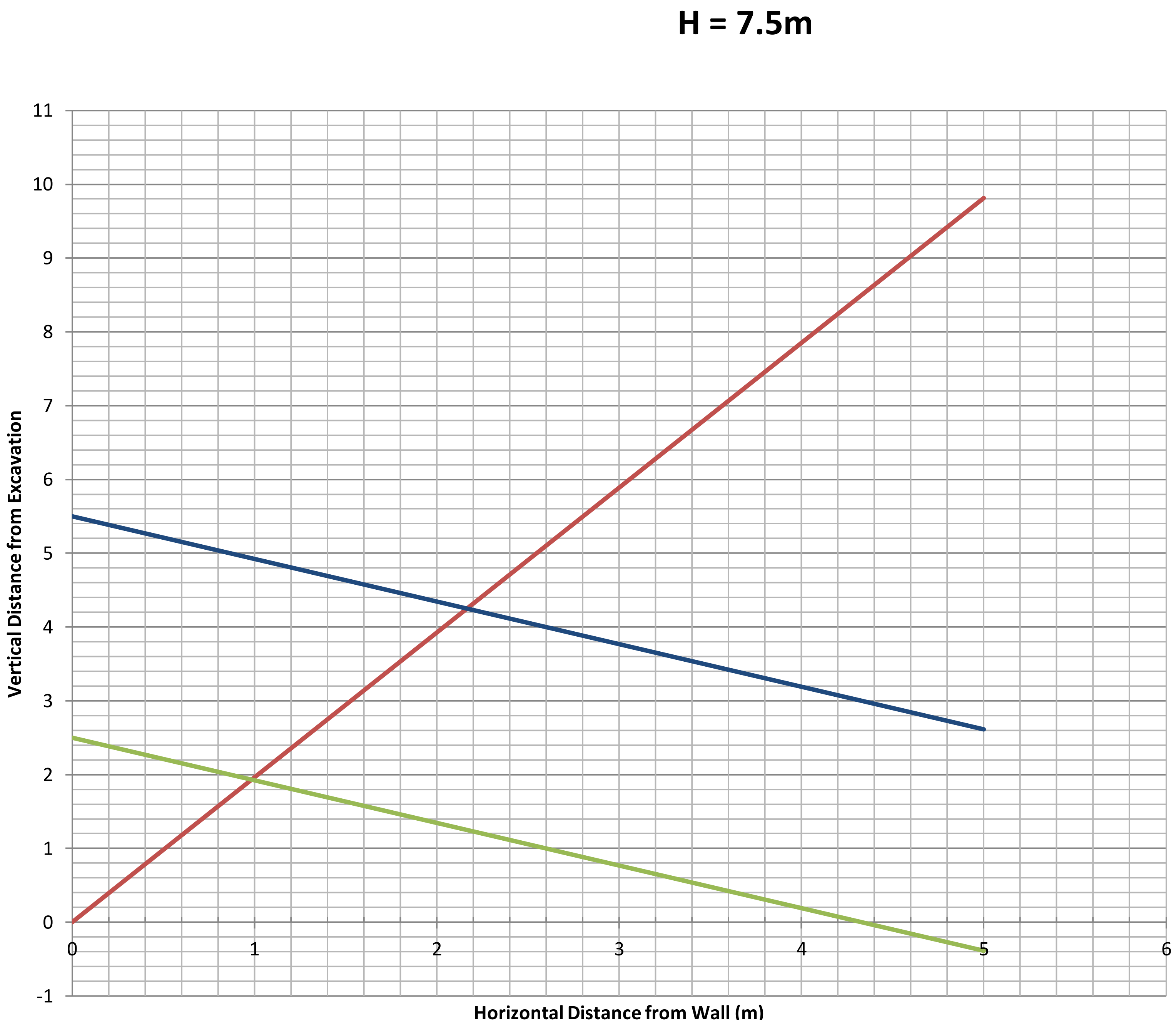
Bond length is either 3.0m or 4.5m depending on required capacity:

		X-dir (m)	Y-dir (m)
Total anchor length (First Tieback) =	7 m	6.1	3.5
OR	8.5 m	7.4	4.3

Total anchor length (Second Tieback) =		6	m	5.2	3.0
	OR	7.5	m	6.5	3.8

NOTE: Available horizontal distance from soldier pile wall to buildings across Henry St. Is approximately 8.5m

Therefore, the maximum tie lengths will be ok for the site


H = 7.5m
Vertical Distance from Excavation
Horizontal Distance from Wall (m)
11
10
9
8
7
6
5
4
3
2
1
0
-1
0
1
2
3
4
5
6

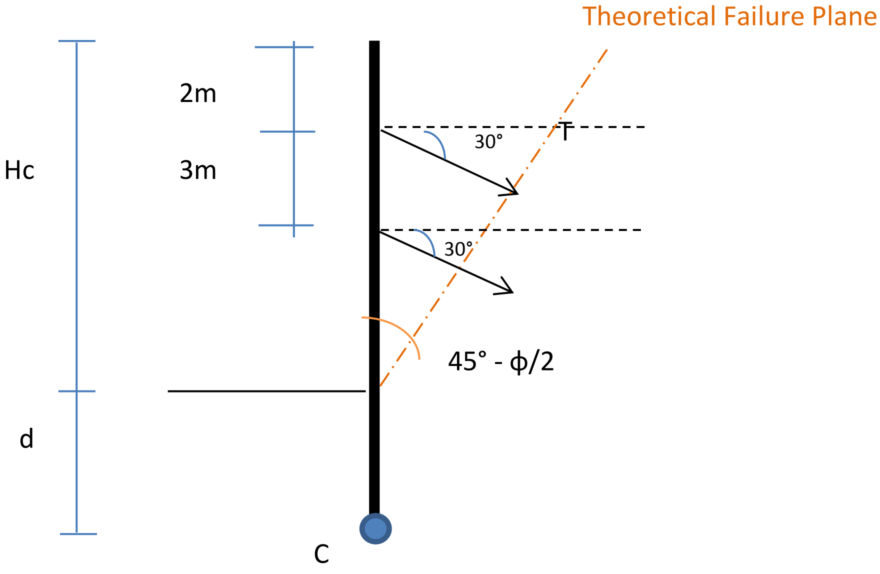
Tieback Design Calculations

Assumption:

1. Non-cohensive soil
2. Groundwater level is neglectable
3. Non-inclined backfill
4. Use ϕ' as ϕ'_{cs} and ϕ'_{p}

Notes:

1. Using Rankine's Equation
2. Critical design depth equals to 8.41 meters
3. Using Safety Factor equals to 1.5
4. Drilled hole diameter = 550mm



Reference: US Army Corps of Engineers

For Hc = 6.5m

27 deg = 0.471239 rad

failure plane

x (m)	y
0	0
1	1.962611
2	3.925221
3	5.887832
4	7.850442
5	9.813053

First Tieback

30 deg = 0.523599 rad

x (m)	y (m)
0	4.5
1	3.92265
2	3.345299
3	2.767949

4	2.190599
5	1.613249

Second Tieback 30 deg = 0.523599

x (m)	y (m)
0	1.5
1	0.92265
2	0.345299
3	-0.23205
4	-0.8094
5	-1.38675

From graph:

First tieback intercepts failure plane at:

x = 1.8 m

y = 3.2 m

Length of tieback from face of wall to theoretical failure plane = 2.1 m

Second tieback intercepts failure plane at:

x = 0.6 m

y = 1.2 m

Length of tieback from face of wall to theoretical failure plane = 0.7 m

Minimum anchor free length

The greater of:

i) length of tieback from face of wall to theoretical failure plane + H/5

ii) length of tieback from face of wall to theoretical failure plane + 1.5

Reference: US Army Corps of Engineers

First Tieback

i) 2.1m + 6.5m/5 = 3.4

ii) 2.1m + 1.5m = 3.6

Anchor free length = 3.5 m **

Second Tieback

Anchor free length = 2.5 m

i) 0.7m + 6.5m/5 = 2.0

ii) 0.7m + 1.5m = 2.2

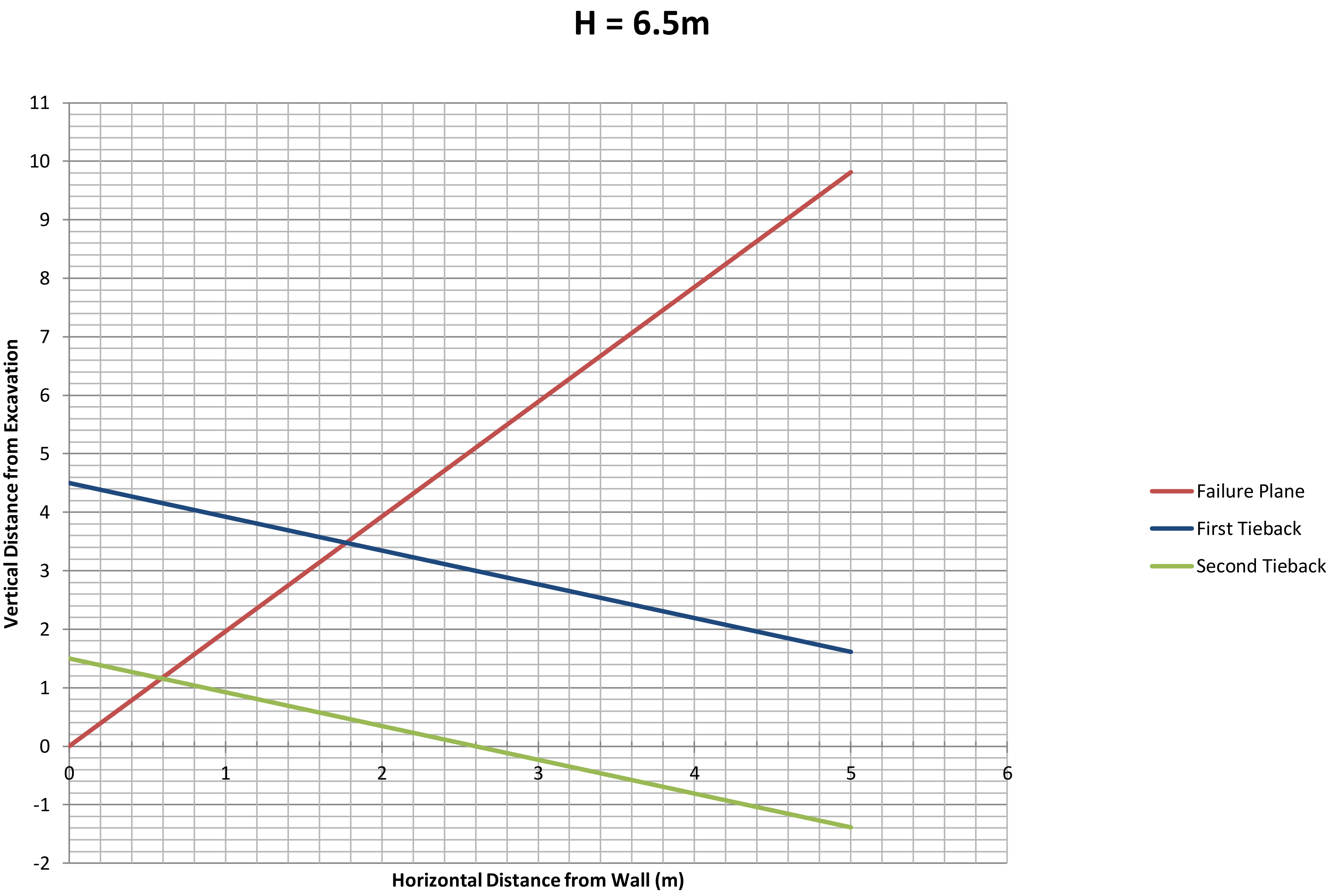
Bond length is either 3.0m or 4.5m depending on required capacity:

		Lengths in the: X-dir (m)	Y-dir (m)
Total anchor length (First Tieback) =	6.5 m	5.6	3.3
OR	8 m	6.9	4.0

Total anchor length (Second Tieback) =		5.5	m	4.8	2.8
	OR	7	m	6.1	3.5

NOTE: Available horizontal distance from soldier pile wall to buildings across Henry St. Is approximately 8.5m

Therefore, the maximum tie lengths will be ok for the site


H = 6.5m
11
10
9
8
7
6
5
4
3
2
1
0
-1
-2
0
1
2
3
4
5
6
Vertical Distance from Excavation
Horizontal Distance from Wall (m)
Failure Plane
First Tieback
Second Tieback

Embedment Depth Calculations

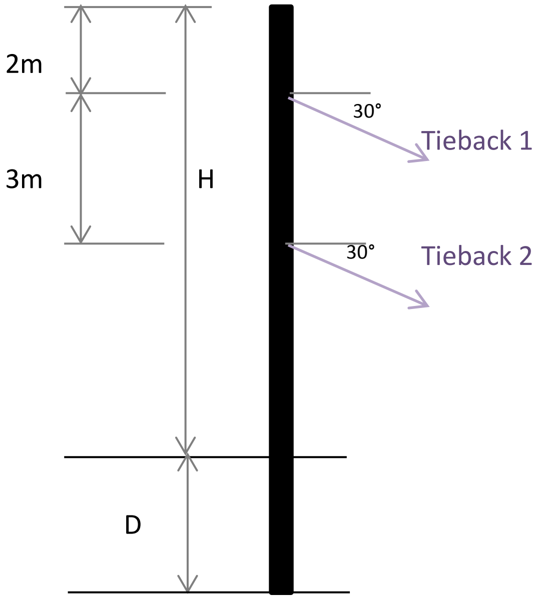
Determine Embedment Depth from S-Frame Analysis

NOTE:
Use critical section (8.5m) to determine ebedment depth
Embedment depth is based on the shear force at the base of the retaining wall

The equation below is from the US Army Corps of Engineers State of the Practice in the Design of Tall, Stiff, and Flexible Tieback Retaining Walls

$$F_p = \gamma d^2\left[\frac{K_0 d \tan\phi' \sin\beta}{\tan(\beta-\phi')\cos\alpha} + \frac{\tan\beta}{\tan(\beta-\phi')}\left(\frac{b}{2}+\frac{d}{3}\tan\beta\tan\alpha\right) + \frac{K_0 d \tan\beta}{3}\left(\tan\phi'\sin\beta - \tan\alpha\right)\right]$$

Fp = 245.4861 kN (From S-Frame Model - 2 ties at 8.5m)
Υ= 20.5 kN/m^3
Ko= 0.41
ϕ = 36 °
β = 45 + ϕ/2 63.0 °
α = ϕ = 36.0 ° (for dense soils)
b = 0.55 m (soldier pile diameter)
FS = 1.50 (factor of safety)



Solve the above equation for d:

D= **1.57** m

Embedment Depth Calculations

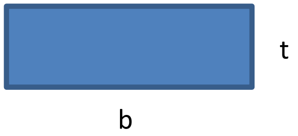
D x FS = **2.4** m

Therefore, use an embedment depth of 3.0 meters for all soldier piles

Timber Lagging Design

Assumptions

1. Use sawn timbers as material
2. Members intend to support permannent loads
3. Single member configuration
4. Wet service condition
5. Untreated wood
6. Species Indentification is SPF Grade No.1 and No.2
7. Members are restricted against lateral displacement and rotation at their end
8. Not notched
9. Simple supproted beam with a normal distribted load
10. Uniform distributed load, no intermediate support
11. No live load is considered



Geotechnical Parameters for design:

ϒ=	20.5	KN/m3
ϕ'=	0.628	rad
Ka=	0.26	
Kp=	3.8	
Ko=	0.41	
q=	12	kPa

36 Degree

Wood Lagging Design Criteria:

Length (L)	3	m
Width (d)	0.25	m

0.0254 in

	Unit
Factored Load: w=(Ka*ϒ*(H-d/2)+Ka*q)*d*0.6*1.25	KN/m
Moment: M=w*L^2/8	KN.m
Section Modulus: S=b*t^2/6	m^3
Bending stress: σ=M/S	KPa

Load Analysis			250*75				250*100				250*150				250*200			
			b=	0.25	m		b=	0.25	m		b=	0.25	m		b=	0.25	m	
			t=	0.075	m		t=	0.1	m		t=	0.15	m		t=	0.2	m	
Depth (m)	w	M	S	σ/1000	fb	Check	S	σ/1000	Fr		S	σ/1000	Fr	Check	S	σ/1000	Fr	Check
0.50	0.96	1.08	0.00	4.61	8.47	Pass	0.00	2.59	8.47	Pass	0.00	1.15	8.47	Pass	0.00	0.65	8.47	Pass
1.00	1.46	1.64	0.00	7.01	8.47	Pass	0.00	3.94	8.47	Pass	0.00	1.75	8.47	Pass	0.00	0.99	8.47	Pass
1.50	1.96	2.20	0.00	9.40	8.47	Fail	0.00	5.29	8.47	Pass	0.00	2.35	8.47	Pass	0.00	1.32	8.47	Pass
2.00	2.46	2.77	0.00	11.80	8.47	Fail	0.00	6.64	8.47	Pass	0.00	2.95	8.47	Pass	0.00	1.66	8.47	Pass
2.50	2.96	3.33	0.00	14.20	8.47	Fail	0.00	7.99	8.47	Pass	0.00	3.55	8.47	Pass	0.00	2.00	8.47	Pass
3.00	3.46	3.89	0.00	16.60	8.47	Fail	0.00	9.34	8.47	Fail	0.00	4.15	8.47	Pass	0.00	2.33	8.47	Pass
3.50	3.96	4.45	0.00	19.00	8.47	Fail	0.00	10.69	8.47	Fail	0.00	4.75	8.47	Pass	0.00	2.67	8.47	Pass
4.00	4.46	5.01	0.00	21.40	8.47	Fail	0.00	12.04	8.47	Fail	0.00	5.35	8.47	Pass	0.00	3.01	8.47	Pass
4.50	4.96	5.58	0.00	23.79	8.47	Fail	0.00	13.38	8.47	Fail	0.00	5.95	8.47	Pass	0.00	3.35	8.47	Pass
5.00	5.46	6.14	0.00	26.19	8.47	Fail	0.00	14.73	8.47	Fail	0.00	6.55	8.47	Pass	0.00	3.68	8.47	Pass
5.50	5.96	6.70	0.00	28.59	8.47	Fail	0.00	16.08	8.47	Fail	0.00	7.15	8.47	Pass	0.00	4.02	8.47	Pass
6.00	6.46	7.26	0.00	30.99	8.47	Fail	0.00	17.43	8.47	Fail	0.00	7.75	8.47	Pass	0.00	4.36	8.47	Pass
6.50	6.96	7.83	0.00	33.39	8.47	Fail	0.00	18.78	8.47	Fail	0.00	8.35	8.47	Pass	0.00	4.70	8.47	Pass
7.00	7.46	8.39	0.00	35.79	8.47	Fail	0.00	20.13	8.47	Fail	0.00	8.95	8.47	Fail	0.00	5.03	8.47	Pass
7.50	7.96	8.95	0.00	38.19	8.47	Fail	0.00	21.48	8.47	Fail	0.00	9.55	8.47	Fail	0.00	5.37	8.47	Pass
8.00	8.46	9.51	0.00	40.58	8.47	Fail	0.00	22.83	8.47	Fail	0.00	10.15	8.47	Fail	0.00	5.71	8.47	Pass
8.50	8.95	10.07	0.00	42.98	8.47	Fail	0.00	24.18	8.47	Fail	0.00	10.75	8.47	Fail	0.00	6.04	8.47	Pass

Recommendation:
QCEC recommend to use 250*100mm SPF Grade No.1 Timber for the first 2.5 meters and change to double 250*100 SPF Grade No.1 Timber for depth deeper than 2.5 meters.

Appendix I

Rigid Gravity Retaining Wall Calculations

Massive Rigid Retaining Wall Design

Note Design for the left hand side of the retaining wall only. The wall is designed to be 15 meters (14.42m) long. Specification refers to the "Design Criterias". The design height was used as 6 meters for the whole length.

Assumptions:

1 Natural backfill
2 Groundwater level is seasonal
3 Groundwater level is below the base of the gravity retaining wall
4 Wall friction = 0
5 Effective friciton angle ϕ' provided in the geotechnical report can be used as bothe critical state friction angle ϕ'_{cs} and peak friction angle ϕ'_p
6 θ'_b=2/3*θ'_{cs} (should be between 1/2*θ'cs to 2/3*θ'cs)

Notes:

1 Using Rankine's Method
2 Factor of safety for translation equals to 1.5
3 Factor of safety for bearing capacity equals to 3
4 With wall friction, the factor of safety against translation is greater than without friction

Geotechical Parameters for design:

ϒ=	20.5	KN/m^3		ϒbase=	20.5	KN/m^3					
							36	Degree		36	Degree
ϕ'=	0.628	rad	36	Degree		ϕ'cs=	0.628	rad	ϕ'p=	0.628	rad
Ka=	0.26										
Kp=	3.8										
Ko=	0.41										

Design Criterias:											
	Design Length=15m										
q=	12	kPa									
a/b=	1.00		a	b	c						
			1	1	1.414						
T=	0.5	m	Unit Thickn	1	m						
Ho=	4	m	b1=	1.5	m	x1=	1.000	m	θ'_b=	0.419	rad
B=	2	m	b2=	0.5	m	x2=	1.750	m	Min.$(FS)_T$=	1.5	
ϒc=	24	KN/m^3	A=	2	m^2	Za=	1.333333	m	Min.$(FS)_B$=	3	
Calculations:						Zb=	2	m			

1 Determine the laterial force

Pasoil: $1/2*Ka*\Upsilon*Ho^2$ = 42.640 KN

Pasurc Ka*q*Ho = 12.480 KN

Pah= Pasoil*b/c+Pasurcharge = 42.631 KN

Pav= Pasoil*a/c = 30.151 KN

2 Determing the wall stability

Consider a unit lenth of wall

W1= $1/2*b1*Ho*\Upsilon c$ = 72.000 KN

W2= $b2*Ho*\Upsilon c$ = 48.000 KN

W= W1+W2 = 120.000 KN

Mo= W1*x1+W2*x2+Pav*B-(Pah-Pas = 151.141 kNm

Rz= W+Pav = 150.151 KN

xbar= Mo/Rz = 1.007 m

3 Base resistance

T= $Rz*\tan(\theta'_b)$ = 66.852 KN

4 Factor of Safety for Translation

$(FS)_T$= T/Pah = 1.568 **Safe against translation**

5 Checking for rotational stability

e= |B/2-xbar| = 0.007 m

B/6= B/6 = 0.333 m **Safe against rotation**

6 Checking for bearing capacity

Notes:

1 Since the resultant vertical force is located within the middle one-third, tension will not be developed in the soil.

2 The maximum stress occurs at A

3 The groundwater level is below B=2m from the base, so groundwater would have no effect on the bearing capacity.

σ_{max}= Rz/A*(1+6*e/B) = 76.560 Kpa

H= Pah = 42.631 KN

Vn= Rz = 150.151 KN

ω= $\tan^{-1}(H/Vn)$ = 3.427 rad

4 The base of the wall can be taken as a strip, surface foundation, that is B/L approaching 0, and Df=0.

B'= B-2e = 1.987 m

B'/L'= B'/L' = 0.000

n= (2+B'/L')/(1+B'/L') = 2.000

i_Υ= (1-H/Vn)^(n+1) = 0.367

N_Υ= $0.1054*e^\wedge(9.6*\phi'_p)$ = 43.898

q_u= $0.5*\Upsilon*B'*N_\Upsilon*i_\Upsilon$ = 328.252 KPa

$(FS)_B$= q_u/σ_{max} = 4.287518 **Safe against bearing**

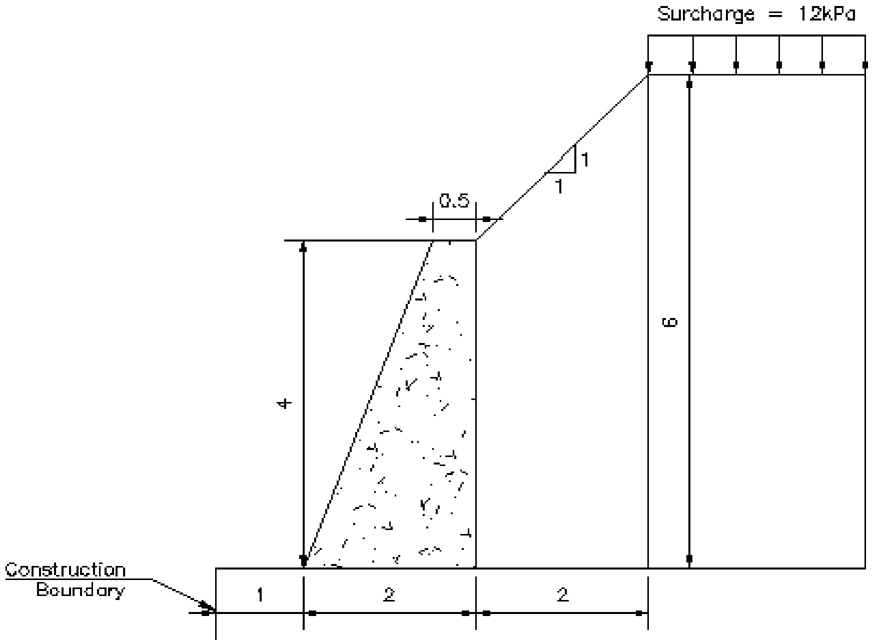


Figure 1. Concrete Wall Sideview

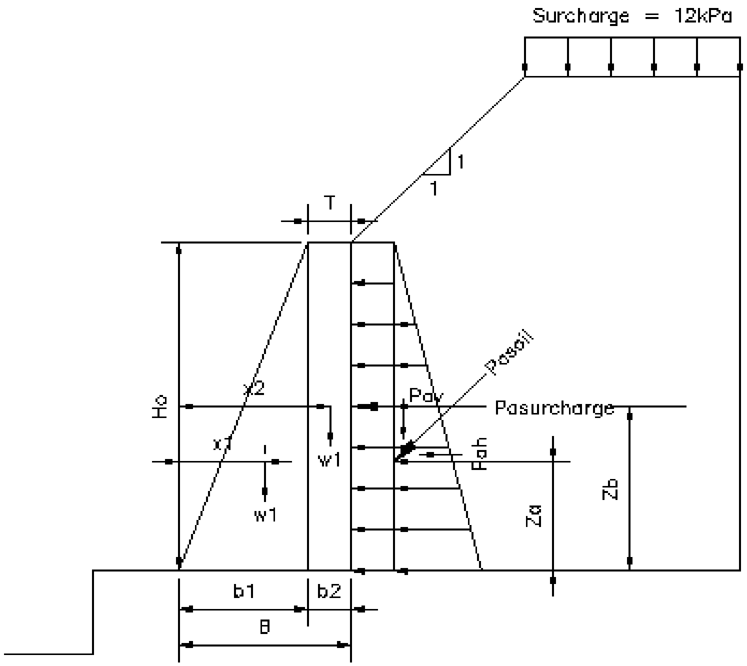


Figure 2. Concrete Wall Sideview for Calculation

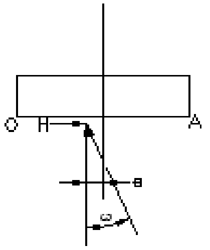


Figure 3. Concrete Wall Bottom View for Calculation

Appendix J

Cost Estimate

STEEL - PRELIMINARY

Wall Length	63	m		**Drilling (Soil) Length**	176	m
# of Piles	22			**Drilling (Rock) Length**	44	m
Lenth of Pile (avg)	10	m		Drilling - Soil Price	$180.00	/m
Total Pile Length	220	m		Drilling - Rock Price	$550.00	/m
Weight of Steel	110	kg/m		**Drilling Cost-Soil:**	**$31,680.00**	
Total Weight of Steel	24200	kg		**Drilling Cost-Rock:**	**$24,200.00**	
Steel Weight Price:	$3.50	/kg		**Tie-Back**		
Total Steel Cost:	**$84,700.00**			Length (un-grouted)	3.5	m
				Length (grouted)	4.5	m
Concrete (@ base)				Drilled Diameter	168	mm
Diameter	550	mm		Bar Diameter	32	mm
Height	2	m		# of Tie-Backs:	22	ea
Volume/hole	0.475166	m3		Total Tie-Back Length	176	m
# of holes	22			Drilling Price:	$65.45	/m
Total Volume	**10.454**	**m3**		**Drilling Cost:**	**$11,520.00**	
Concrete Price	$650.00	/m3		Weight of Tie-back (DYWIDAG)	6.40	kg/m
Total Concrete Cost:	**$6,794.87**			Total Weight:	1126.4	kg
				TIE-BACK ROD PRICE:	**$3,942.40**	
				Grout Volume	0.09975185	m3
				Total Grout Volume	**2.19454070**	**m3**
				Grout Price	$400.00	/m3
				Total Grout Price	**$877.82**	

TIMBER-PRELIMINARY

# Piles:	22	
Average Height:	7.1	m
Timber Dimensions:		
Width	0.076	m
Height	0.254	m
Length	3.049	m
Timber @ 1 Section	28	
TOTAL TIMBER:	**589**	ea
Vol. Per 1 Timber	0.0590	m3
Total Volume of Timber:	34.7693	m3
Daily Output	2.120	m3/Day
Timber Installed per Day	35.91	ea/day
Days to Install Timber	**16.4**	**days**
Labour Cost (2 labouer):	$567.20	/day
Labour Price:	**$9,302.44**	
Material Cost:	$451.20	/m3
Material Price:	**$15,687.93**	
TOTAL PRELIMINARY TIMBER PRICE:	**$24,990.37**	

SOIL NAILING - PRELIMINARY

Number of Nails	149	ea
Nail Length	5.79	m
Total Length	863.11	m
Weight	6.40	kg/m
Total Weight	**5523.90**	**kg**
Material Price	$3.50	/kg
Total Nail Material Price	**$19,333.66**	
Diameter	0.203252033	m
Shotcrete Thickness	0.101626016	m
Area for Mesh	332.2616	m2
Shotcrete w/ Mesh	$430.60	/m2
Shotcrete w/ Mesh Cost:	**$143,071.84**	
Grout Volume	0.187948746	m3
Total Grout Volume	28.0043632	m3
Grout Price	400	/m3
Grout Cost:	**$11,201.75**	
Drilled Diameter	200	mm
Total Drilled Length	863.1097561	m
Drilling Price (% of greater diam drilled)	$65.45	/m
Drilling Cost:	**$56,494.46**	
Dist from Toronto to St. John's	3075	km
price per distance (km)	$1.26	/km
Shipping Cost:	**$3,874.50**	

Cost to pay a contractor to come from the mainland and do the project will increase the cost greatly

EXCAVATION		
Excavation in Soil		
EXCAVATION (WHOLE CONSTRUCTION AREA)	2898.8	m2
VOLUME EXCAVATED (avg height=7.5m)	21741.00	m3
Excavation in Rock		
VOLUME EXCAVATED (under concrete wall)	46.14	m3
COSTS		
Price in Soil	$15.70	/m3
Cost-Soil	**$341,333.72**	
Price in Rock (soil price x 2.5)	$39.25	/m3
Cost-Rock	**$1,811.15**	

DRILLING

Diameter :	550	mm
Pile #	Length in Soil (m)	Length in Rock (m)
1	6.56	4.000
2	6.56	3.500
3	7.35	4.000
4	8.24	4.250
5	8.24	3.500
6	8.52	3.500
7	8.52	3.000
8	8.52	3.250
9	8.52	3.500
10	8.1	3.500
11	7.6	3.250
Total (m)	**86.73**	**39.25**
Price (soil)	$180.00	/m
Total Cost:	**$15,611.40**	
Price (rock)	$550.00	
Total Cost:	**$21,587.50**	

STEEL PILES

Pile #	Height (m)	Unit
1	9.5	m
2	9.5	m
3	10.5	m
4	11.5	m
5	11.5	m
6	11.5	m
7	11.5	m
8	11.5	m
9	11.5	m
10	11.5	m
11	10.5	m
Total Length of Steel	120.5	m
Weight of Steel	110	kg/m
Total Weight:	**13255**	**kg**
Material Price	$3.50	/kg
Material Cost:	**$46,392.50**	

**price confirmed by client*

**accounts for any shipping

Concrete @ embedment depth

Diameter	550	mm
Height	3	m
Volume/hole	0.712749	m3
# of holes	11	
Total Volume	**7.840**	**m3**
Concrete Price	650	/m3**
Concrete Cost:	**$5,096.15**	

***price confirmed by client*

Steel Casing

Rented

# of Casings	11	
Length	120.5	m
weight of casing	372	kg/m
Total Weight	44826	kg
Material Price	$3.50	/kg
Total Cost (New Material)	$156,891.00	
Rented Cost (10% New)	**$15,689.10**	

TIMBER

TIMBER (4"x10"x10')	WALL HEIGHT	HEIGHT SECTIONS (m)	# of Sections	QTY	W	H	L
TIMBER 1 (< 2.5m)	8.5	2.5	6	59.04	0.10	0.25	3.05
TIMBER 1 (> 2.5m)		6	6	283.4	0.10	0.25	3.05
TIMBER 2 (< 2.5m)	7.5	2.5	2	19.68	0.10	0.25	3.05
TIMBER 2 (> 2.5m)		5	2	78.72	0.10	0.25	3.05
TIMBER 3 (< 2.5m)	6.5	2.5	2	19.68	0.10	0.25	3.05
TIMBER 3 (> 2.5m)		4	2	62.98	0.10	0.25	3.05

Option 2: is to use 250*100mm for first 2.5 meters,
and use 2*250*100mm for depth greater than 2.5 meters.

TOTAL TIMBER:	**523.488**	**ea**
Total Volume of Timber:	41.21	m3

Volume of Each Timber	0.08	m3	
Daily Output	2.36	m3	
Timber Placed/Day	29.98	ea/day	
Days to Place All Timber	**17.5**	**Days**	
Labourer(2)	$567.20	/Day	2 Labourers
Labour Cost:	**$9,903.92**		
Materials:	$451.20	/m3	
Material Cost:	**$18,593.10**		
Total Timber Cost:	**$28,497.01**		

Blocking	W	H	L
Dimensions (m)	0.051	0.102	2.44
Total Timber	523.488		
Blocking/Timber	4	ea	
Blocking Length (section w/ 1 timber)	420	mm	
Blocking Length (section w/ 2 timber)	220	mm	
Blocking Amount (1 timber section)	393.6		
Blocking Amount (2 timber section)	1700.35		
Total Length (1)	165.31	m	
Total Length (2)	374.08	m	
# of pieces of lumber:	221	pc	
Price of Lumber 2x4x8)	$2.79	/pc	
Blocking Price:	**$617.01**		

TIMBER + BLOCKING:	**$29,114.02**

Tie-Back

Length (un-grouted)	4.5	m
Length (grouted)	4.5	m
Drilled Diameter	168	mm
Bar Diameter	32	mm
# of Tie-Backs:	19	ea
Total Tie-Back Length	171	m
Drilling Price (**% of larger drilling cost)	$65.45	/m
Drilling Cost	**$11,192.73**	
Weight of Tie-back (DYWIDAG)	6.40	kg/m
Total Weight:	1094.4	kg
Steel Price	$3.50	/kg
TIE-BACK ROD PRICE:	**$3,830.40**	
Grout Volume	0.14137167	m3
TOTAL GROUT VOLUME	**2.68606172**	**m3**
Grout Price	$400.00	/m3
Grout Cost:	**$1,074.42**	
Steel Beam Cut/Replace		
Steel Strip (100mmx12mm)	3	m
Amount of Steel Strip	10	ea
Total Length of Strip	30	m
Weight of Strip	94.2	kg/m2
Weight/Strip	0.11304	
Total Strip Weight	**4.3912**	**kg**
Steel Price	$3.50	/kg
Total Strip Cost:	**$15.37**	
L - angle (L200x100x13) (to secure tie-back)	29.5	kg/m
Length of angles	155	mm
total weight of angles:	**4.5725**	KG
Steel Price	$3.50	/kg
Total Strip Cost:	**$16.00**	

RIGID GRAVITY WALL

DESCRIPTION		**UNIT**
CONCRETE WALL		
WALL LENGTH	15	m
WALL HEIGHT	4	m
TOP WIDTH	0.5	m
BASE WIDTH	2	m
CONCRETE VOLUME	75	m3
Concrete Price:	650	/m3
Concrete Cost:	**$48,750.00**	
WEEP HOLES	10	ea
DIAMETER	0.08	m
0.5 < L < 2 M (say 1.5m length)	1.5	m
PVC PRICE-KENTS.CA	$13.49	/pc
4" diameter - 10' length		
say 2 pipes from one piece (*plus one just in case?*)	6.00	
PVC PRICE:	$80.94	
Material Cost:	**$80.94**	
Labour-Drill holes in formwork and place pipe - can be included with formwork price		

confirmed by client

FORMWORK			TOTAL	UNIT
BACK AREA	1	60	60	m2
SIDE AREA	2	5	10	m2
FRONT AREA	1	64.08	64.08	m2
TOTAL FORMWORK AREA			134.08	m2
say 3 labourers and 1 carp				
and 2 days				
Days for Formwork	2.00			
Labour Price	$1,210.00	/day		
Labour Cost	**$2,420.00**			
Materials:	$22.38	/m2		
Material Cost	**$3,000.71**			
Total Formwork Cost:	**$5,420.71**			

TOTAL CONCRETE WALL COST:	**$54,251.65**

RSMEANS VALUES

TIMBER

	LINE #	DESCRIPTION	UNIT	CREW	DAILY OUTPUT	LABOUR HOURS	BARE MATERIAL	BARE LABOUR	BARE EQUIPMENT	BARE TOTAL	TOTAL O&P
0	61323100270	Framing, heavy mill timber, beams, built from 100mm lumber, multiple 100mm x 250mm	m3	2 Carp	2.36	6.781	518.4	252.24		770.64	959.89

FORMWORK

	LINE #	DESCRIPTION	UNIT	CREW	DAILY OUTPUT	LABOUR HOURS	BARE MATERIAL	BARE LABOUR	BARE EQUIPMENT	BARE TOTAL	TOTAL O&P
0	31113854900	Retaining Wall,battered, job-built plywood, over 2.4 to 4.8m high, 1 use	m2CA	C2	22.3	2.153	22.38	79.24	0	101.62	146.86

SOIL NAILING

	LINE #	DESCRIPTION	UNIT	CREW	DAILY OUTPUT	LABOUR HOURS	BARE MATERIAL	BARE LABOUR	BARE EQUIPMENT	BARE TOTAL	TOTAL O&P
	313236160060	Grouted soil nailing material delivery add $1.14 to $1.26 per truck km for shipping									

Crew Prices	Daily
1 Laborer	$283.60
1 Carpenter	359.2

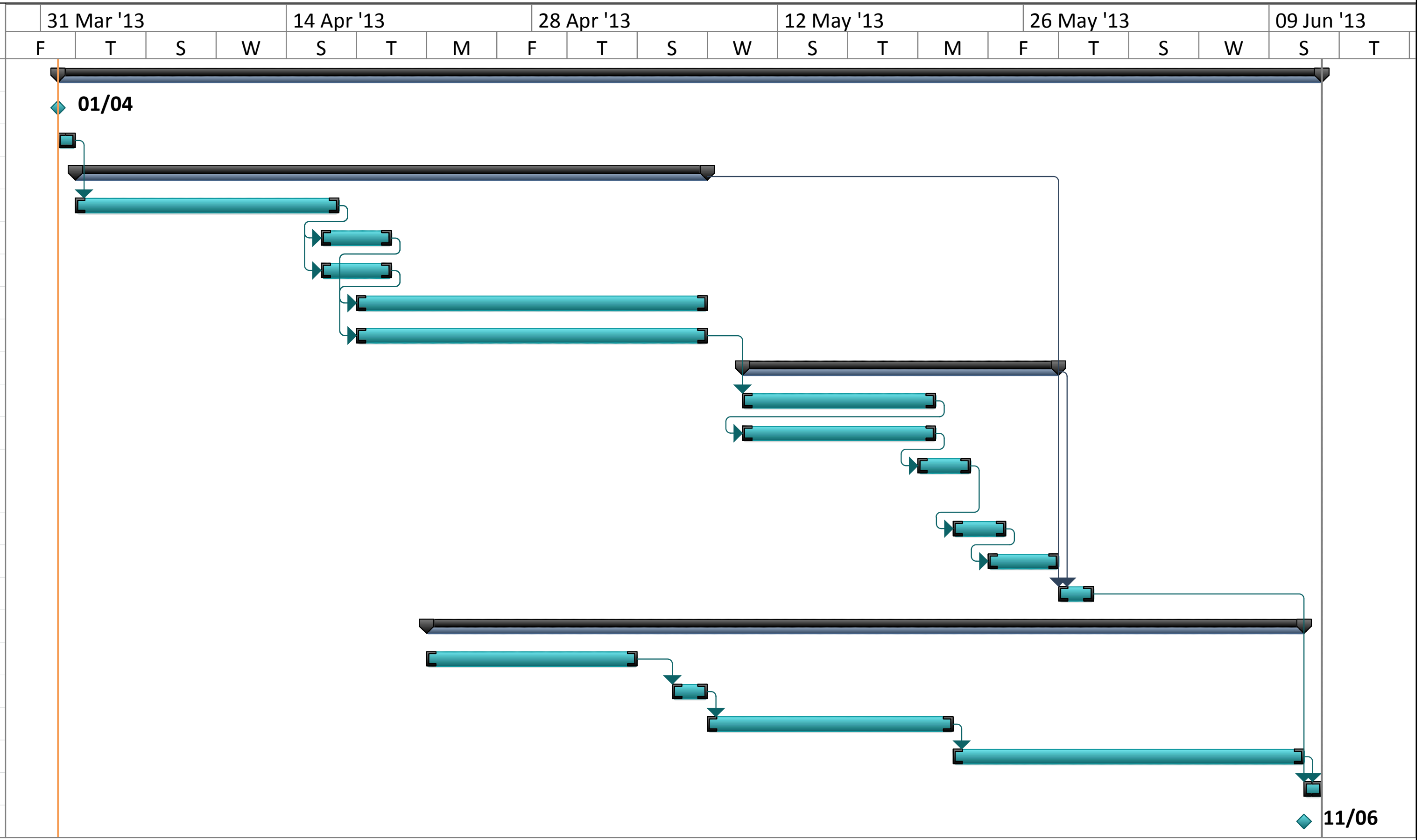
Appendix K

Construction Schedule

Duckworth Street Retaining Wall Construction Scehdule

QCEC

ID	Task Mode	Task Name	Duration	Start	Finish
1	📌	**Project Length**	**52 days**	**Mon 01/04/13**	**Tue 11/06/13**
2	📌	Start	0 days	Mon 01/04/13	Mon 01/04/13
3	📌	Mobilization	1 day	Mon 01/04/13	Mon 01/04/13
4	📌	**Soldier Pile Installation**	**26 days**	**Tue 02/04/13**	**Tue 07/05/13**
5	📌	Drill Boreholes (~11)	11 days	Tue 02/04/13	Tue 16/04/13
6	📌	Install Piles/Steel Beams	4 days	Tue 16/04/13	Fri 19/04/13
7	📌	Pour Embedding Concrete	4 days	Tue 16/04/13	Fri 19/04/13
8	📌	Excavate Soil & Bedrock	14 days	Thu 18/04/13	Tue 07/05/13
9	📌	Install Lagging	14 days	Thu 18/04/13	Tue 07/05/13
10	📌	**Tieback Installation**	**12 days**	**Fri 10/05/13**	**Mon 27/05/13**
11	📌	Drill Tieback Holes	7 days	Fri 10/05/13	Mon 20/05/13
12	📌	Install & Grout Bar	7 days	Fri 10/05/13	Mon 20/05/13
13	📌	Complete Performance and Proof Tests	3 days	Mon 20/05/13	Wed 22/05/13
14	📌	Lock-off and stress	3 days	Wed 22/05/13	Fri 24/05/13
15	📌	Place secondary grout	2 days	Fri 24/05/13	Mon 27/05/13
16	📌	Backfill and Compact Soil	2 days	Tue 28/05/13	Wed 29/05/13
17	📌	**Gravity Wall Installation**	**36 days**	**Mon 22/04/13**	**Mon 10/06/13**
18	📌	Excavation	10 days	Mon 22/04/13	Fri 03/05/13
19	📌	Formwork Installation	2 days	Mon 06/05/13	Tue 07/05/13
20	📌	Pour/Place Concrete Wall	10 days	Wed 08/05/13	Tue 21/05/13
21	📌	Cure/Set Concrete	14 days	Wed 22/05/13	Mon 10/06/13
22	📌	Demobilize	1 day	Tue 11/06/13	Tue 11/06/13
23	📌	End	0 days	Tue 11/06/13	Tue 11/06/13



Project: Construction Plan1
Date: Mon 01/04/13

Task	▬	External Milestone	◆	Manual Summary Rollup	▬	Baseline Milestone	◇
Split	┅	Inactive Task	▭	Manual Summary	▬	Baseline Summary	▬
Milestone	◆	Inactive Milestone	◇	Start-only	[	Progress	▬
Summary	▬	Inactive Summary	▭	Finish-only	]		
Project Summary	▬	Manual Task	▬	Deadline	⇩		
External Tasks	▬	Duration-only	▬	Baseline	▬		

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