PARAMOUNT ENGINEERING

DESIGN RECOMMENDATIONS: ST. LAWRENCE MARINE TERMINAL

ENGI 8700 FINAL REPORT

Submitted to Dr. Bruneau
Apr. 5, 2010

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April 5, 2010

Mr. Ray Bailey, P.Eng. & Mr. Nick Gillis, P.Eng.
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Subject: *Civil Engineering 8700 Final Project Report*

Dear Mr. Bailey and Mr. Gillis,

The following document is the final project report for the design of the St. Lawrence Marine Terminal as part of the Civil Engineering Project course at Memorial University of Newfoundland.

Contained in the report is:

- Preliminary design and cost analysis of three recommended structural designs;
- Detailed design and cost analysis of the recommended optimal design;
- Design calculations;
- Design drawings.

If there are any questions or concerns regarding any aspect of the report please inform us and we will address the issue.

Thank you for selecting Paramount Engineering to undertake the project design.

Yours Sincerely,

Paramount Engineering:
Peter Collins
Andrew Small
Steven Greeley
Robert Hunt

cc: Steve Bruneau, Ph.D, P.Eng

SUMMARY

This report presents the process followed to determine the most optimal structural design option for a dry bulk marine terminal for a mining operation in an Arctic environment. Various structure types were investigated and ranked based on project specific criteria. The ranking of the structural options was most heavily influenced by the effects of arctic sea-ice and subsea bed rock conditions which slope at 30 °. The ranking procedure produced three options for consideration at a conceptual design stage:

- Cellular sheet piles,
- Concrete caissons
- Steel pipe piles.

A preliminary design was performed for each alternative where global stability checks were performed to obtain a general sizing and associated cost for each option. The most economical design was the cellular sheet pile option. Case histories where cellular sheet piles have been applied in Arctic environments served to increase confidence in the selected design.

The cellular sheet pile concept was finalized into a detailed design which included:

- Ice strengthening panels,
- Slab-on-grade,
- Foundation details,
- Cope wall,
- Fenders,
- Mooring devices.

A detailed cost analysis was performed and total cost of construction was estimated at $40,281,160.

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1 INTRODUCTION

1.1 OBJECTIVE OF REPORT

This document serves as the final report for the St. Lawrence Marine Terminal Project, and is prepared for review by the client (BAE-Newplan) and course instructor (Steve Bruneau, Ph.D, P.Eng.).

The objective of this report is to:

- Develop and compare structural alternatives for the construction of a dry bulk terminal in an arctic environment;
- Evaluate and rank the economic merits and construction risks associated with each alternative;
- Provide recommendation for a type of wharf structure;
- Detail the design of the recommended structural option.

1.2 BACKGROUND INFORMATION

The St. Lawrence Marine Terminal will be a near-shore, single berth, dry bulk terminal located in the Canadian Arctic. The exact location of the project is a matter of confidentiality and St. Lawrence, NL as been chosen by the client as a mock location for the terminal. The proposed site is in a sheltered bay location subject to sea ice conditions as shown in Figure 1 (next page). The names shown in the figure are fictional.

The main function of the port will be for the export of iron ore (20 Million tons per annum). Secondary usages will be for the import of fuel, spare parts, and other necessities. The berthing facilities will be capable of accommodating the various ranges of vessels that will be required for the various cargos. The port layout and equipment will be based on three (3) types of vessels: [1]

- dry bulk carriers for ore export (220,000 DWT)
- general cargo vessels (15,000 DWT)
- bulk oil tankers (30,000 DWT)

The construction phase of the project requires a temporary dock on site to facilitate the landing and unloading of barges containing construction equipment, materials, and supplies. The shipping of these items will occur during the ice free summer season. [1]

The length of the shipping season is very sensitive in the arctic environment and varies on an annual basis. Significant downtime due to summer storms and wind conditions can be experienced during the short summer season. [1]

Table 1 shows the historic dates of ice break up and freeze up and the average available shipping days based on ice conditions. The duration of the shipping season typically occurs when the waters are relatively ice free. This usually occurs two (2) to three (3) weeks after break up and one (1) week before freeze up. It will not be possible to meet the required iron ore exports during the ice free shipping season, thus requiring year round shipping. [1]

Average Break Up	Average Freeze Up	Average Shipping Days
May 15	November 30	180

Table 1 - Typical Arctic Shipping Season

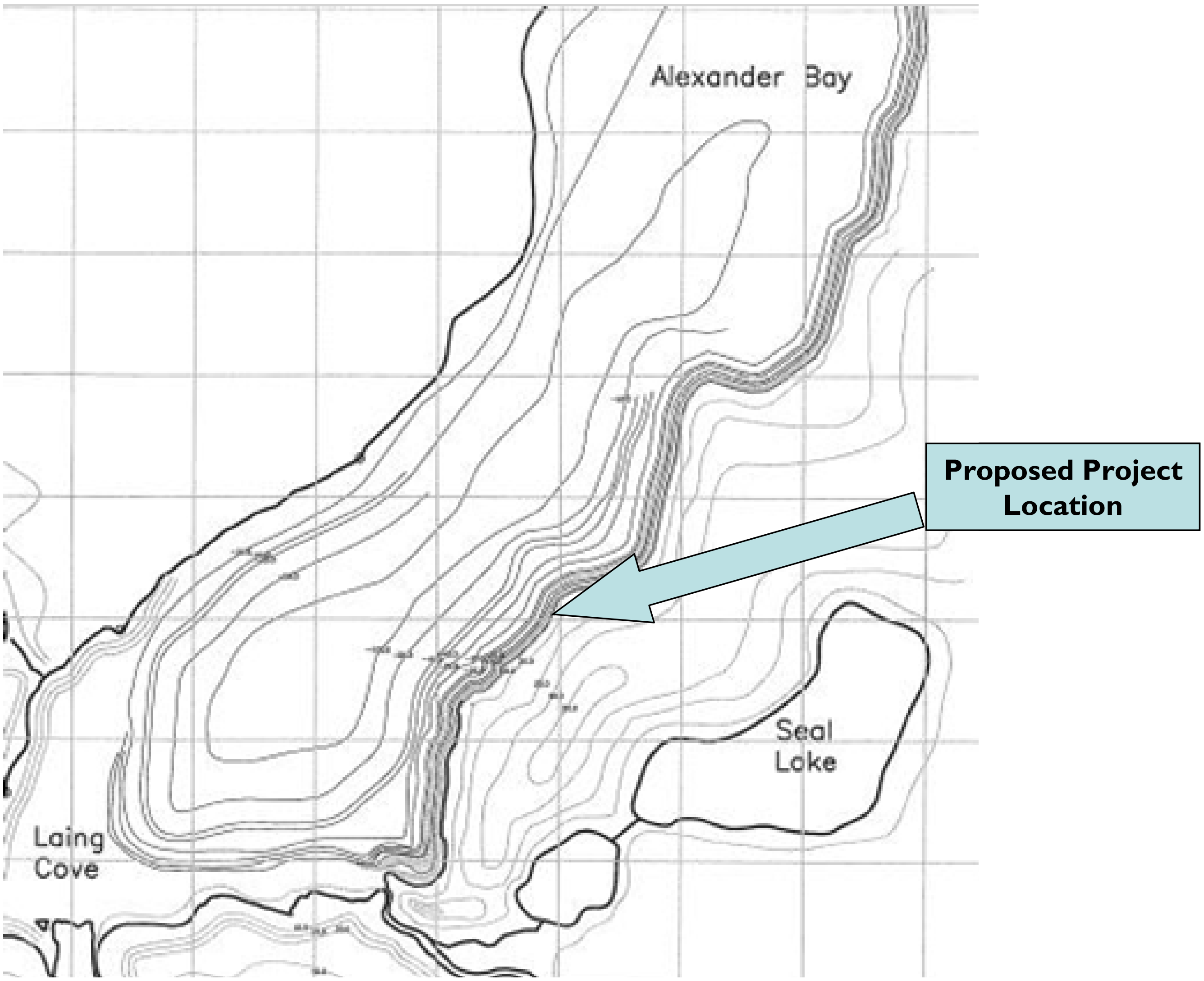

Figure 1 - Proposed Project Location

2 SITE CONDITIONS

Site conditions pertain to information regarding climatic, environmental, geotechnical, and operational conditions. Unless noted otherwise, all data has been provided by the client.

2.1 CLIMATIC CONDITIONS

2.1.1 Wind

The wind in the area mainly blows from north (N) towards the south (S) during winter and from east (E) and the north-east (NE) towards west (W) and south-west (SW) during the summer or period of open water. [1]

The design wind speed for the project location is 36 m/s. [1]

2.1.2 Snow

Given the infrequency of snow in the arctic, the loading due to snow on the structure deck has been neglected. Operational loads will exceed any expected snow surcharge and it is anticipated that snow clearing will be done in an effective manner preventing the build up of snow on deck.

2.2 ENVIRONMENTAL CONDITIONS

2.2.1 Tides

A 2.0 m range of tidal fluctuations exist at the project site. [1]

In addition to the tidal range it has been estimated to allow an additional 0.5 m increase in water level as a global warming allowance to account for ice melt conditions during the structure's serviceable life. [1]

2.2.2 Waves

A significant wave height, H_s, of 1.5 m has been estimated for the proposed site. [1]

Assuming a fetch length of approximately 7 km based on the geography of the harbor, and the estimated value for H_s, the period, T, wavelength, L, and celerity, C, of the design wave may be determined using wave forecasting techniques set out in the U.S. Army Corps of Engineers (USACE) Shore Protection Manual (SPM). [2]

The design wave conditions are:

- Period, T = 4.0 s
- Wavelength, L = 25 m
- Celerity, C = 6.25 m/s

The detailed calculations and the excerpt from the SPM are located in Appendix A.

2.2.3 Current

Currents, both parallel and transverse to the planned terminal, are estimated at 1 knot (0.51 m/s). [1]

2.2.4 Ice

Arctic ice conditions are the primary factor affecting the functional and cost aspects of the marine facilities. The ice conditions place constraints on the length of the shipping season, the shipping schedule, the type of vessel required for year round shipping and the associated shipping costs, and the type of terminal and construction practices employed. [1]

The bay is largely ice-covered for six (6) months of the year. First year ice begins to form in mid to late November and grows from the coastline seaward. The bay is covered in a thin layer of first year ice by the end of November. Break-up of the first year ice commences in May. Multi-year ice is not present in the bay. [1]

A maximum ice thickness of 1.5 m is experienced by March. [1]

2.2.5 Seismicity

The client has instructed to negate any effects from seismicity so it has not been included in any aspect of design.

2.3 GEOTECHNICAL CONDITIONS

The subsea terrain slopes from the shoreline at an angle of 30 degrees and exposed bedrock conditions exist throughout the site.

There was no geotechnical information made available. As a result, assumptions and the selection of geotechnical parameters were made with consultation to data presented in Principles of Geotechnical Engineering [3]

It was assumed that in situ and backfill soil parameters were equivalent and its consistency was comparable to that of a dense angular-grained silty sand.

The following soil parameters were used throughout the design:

- Internal friction angle, $\varphi = 30°$ (assumed equal to seabed slope)
- Dry unit weight, $\gamma_d = 19$ kN/m^3
- Saturated unit weight, $\gamma_{sat} = 21.8$ kN/m^3
- Coulomb's active pressure theory coefficient, $K_a = 0.2973$

2.4 OPERATIONAL CONDITIONS

Operational conditions include the surcharge expected from ship loading equipment and other vehicles operating on deck. A value of 20 kPa has been assigned as a uniform surcharge load to account for all loading on the terminal deck. [1]

Other operational forces include those generated by the berthing and mooring of the design vessel which is further examined in Section 6 of this report.

3 SELECTION OF DESIGN VESSEL

The marine terminal was designed to accommodate the berthing and loading of a 220,000 DWT dry bulk carrier.

A mean statistical analysis approach was used to calculate the design vessel's main dimensions based on data provided in Planning and Design of Ports and Marine Terminals[4] and Design of Marine Facilities for the Berthing, Mooring, and Repair of Vessels. [5]

The vessel's main dimensions of interest and their influence on the terminal's structural design are: [4]

- **Length overall, LOA:** is the vessel's total length which governs the length and layout of single-berth terminals, and influences the length of the loading platform.
- **Beam or Breadth, B:** is the vessel's maximum width and governs the required reach of cargo handling equipment.
- **Draught, D:** is the distance from the vessel's water line to the bottom of its keel. There are two types of draught; loaded and light (ballast) draught conditions. Gaythwaite recommends that light draughts are typically 30%-50% of loaded draught conditions. [5] For the purpose of this design an average of 40% was used. Draught influences the water depth along the berth.
- **Depth, D_s:** is the depth of the vessel's hull measured from the deck to the bottom of the keel. The vessel depth influences the area over which wind forces act on the vessel.

Figure 2 shows a typical dry bulk carrier and highlights the dimensions listed above.

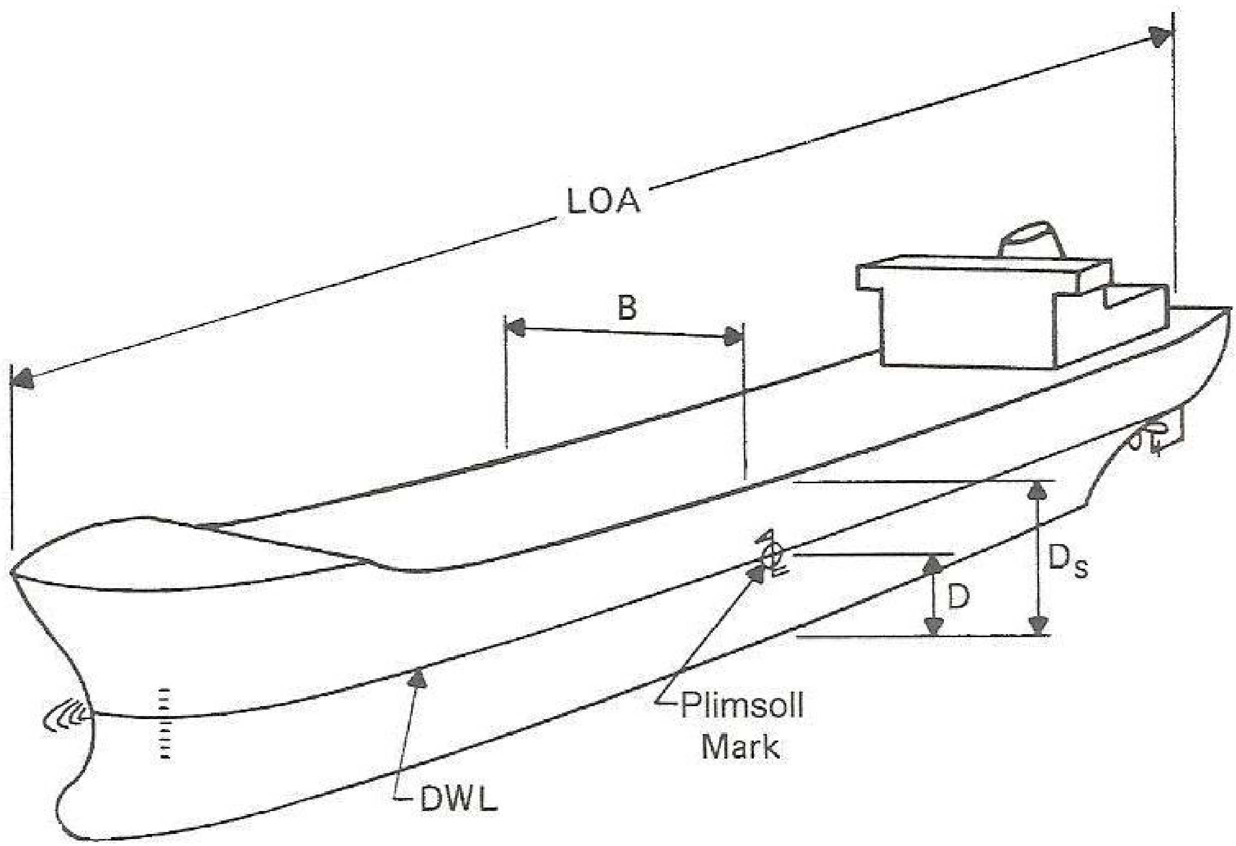

Figure 2 – Vessel Dimension Definition

The following dimensions were calculated and confirmed by the client. Appendix B contains the detailed calculations resulting in the values shown in Table 2.

Dimension	Value
Length Overall, LOA	310 m
Beam, B	50 m
Draught, D (Loaded)	18.5 m
Draught, D (Ballast)	7.5 m
Depth, D_s	27.5 m

Table 2 - Design Vessel Dimensions

4 SHIP LOADING EQUIPMENT

In order to transport the ore from the mine location to the marine terminal a conveying belt will be used. From this point a ship loader capable of loading the design vessel at a rate of 10,000 tph (tones per hour) will be employed. The desired ship loader is a longitudinally travelling device that can traverse the wharf on a rail system, servicing the vessel without needing to swivel or turn, to reach all hatches of the bulk carrier vessel.

A number of online resources were referenced to select a suitable ship loader. The dimensions and weight shown below in Table 3 are based on information provided from a typical ship loader used on Sandvik's Finucane Island project, in Australia, [6] and a KRUPP ship loader and deck conveyor that was previously installed in Los Angeles, CA. [7]

Component	Value
Design Vessel	220,000 DWT
Required Capacity Rate	10,000 tph
Ship Loader Weight	Approx. 860 metric tonnes
Boom Length (Extended)	46 m
Boom Length (Retracted	14 m
Rail Gauge	19 m
Ship Loader Length	25 m
Belt width	1.8 m

Table 3 - Design Ship Loader Data

Figure 3 shows the ship loader traversing a 265 m loading platform along guide rails separated by 19 m and accessing the hatches of the design vessel.

In consultation with the client it was assumed that the hatches extend for approximately 70% of the overall length of the vessel (Hatch Length = 310 m x 70% ≈ 220 m). Due to the fact that the design ship loader does not turn or swivel, the wharf length will have to accommodate an extra half of a ship loader length on either end to ensure that all hatches on the bulk carrier can be accessed. In addition to this, there should also be some form of blocking mechanism so that the loader is prevented from travelling off the platform. Although the design of the blocking mechanism is not part of the project scope, it has been estimated that two (2) blocks of 5 m in width be added to each end of the wharf. It was also assumed that the ore conveying belt is 10 m in width.

The required length of our docking structure is equal to:

L_{dock} = 220 m + 2 x 12.5 m + 2 x 5 m + 10 m = 265 m.

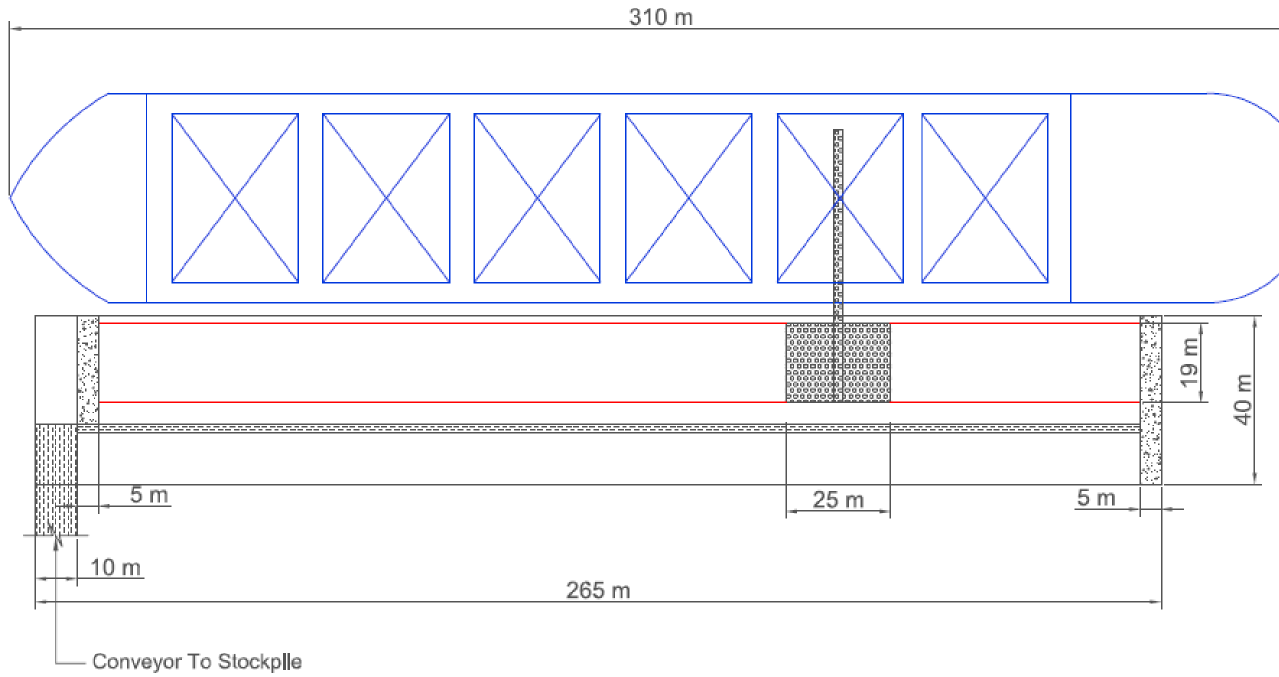

Figure 3 - Plan of Loading Platform

5 DETERMINATION OF DECK ELEVATION

The deck elevation is a function of the loaded draught of the ship, under keel clearance, and environmental and climatic conditions.

Wave runup must be considered for both vertical face and open structures since at the water line level each structural configuration will possess a vertical element to provide for the berthing capacity of the vessel. Wave runup, δ, is determined using Sainflou's (1928) formula for fully reflected regular waves as shown below: [8]

$$\delta = \frac{\pi H^2}{L}\coth\left(\frac{2\pi z}{L}\right) = \frac{\pi (1.5\,m)^2}{25\,m}\coth\left(\frac{2\pi (22\,m)}{25\,m}\right) = 0.3\,m$$

The combination of 2 m tides, a significant wave height of 1.5 m, a wave runup of 0.3 m, a minimum freeboard of 1.0 m and a global warming allowance of 0.5 m required the structure to have an elevation of +5.3 m above low natural tide (LNT). A structure elevation of 7.0 m above LNT has been recommended to protect electrical equipment on the deck form wave overtopping occurring during storm events, and also to permit easier accessibility to vessel hatches while docked at the terminal.

The loaded draught of the ship is 18.5 m and Handbook: Quay Walls[9] recommends an under keel clearance equal to 10% of the draught, therefore equal to 1.85 m. These values combined require a water depth of 20.35 m. A design water depth of -22.0 m below LNT was conservatively selected to safeguard against irregularities on the sea floor at the berth face such as small humps, or against the build up of sediments from material wastage during the ship loading process. As a result, the terminal design will have a retaining height of 29.0 m.

In order to achieve the design water depth at LNT the berthing face of the structure must be located 40 m from the shoreline.

Figure 4 shows a generalized cross sectional view of the berthing arrangement displaying water levels, deck elevation, water depth, and the proximity of the berthing face to shore.

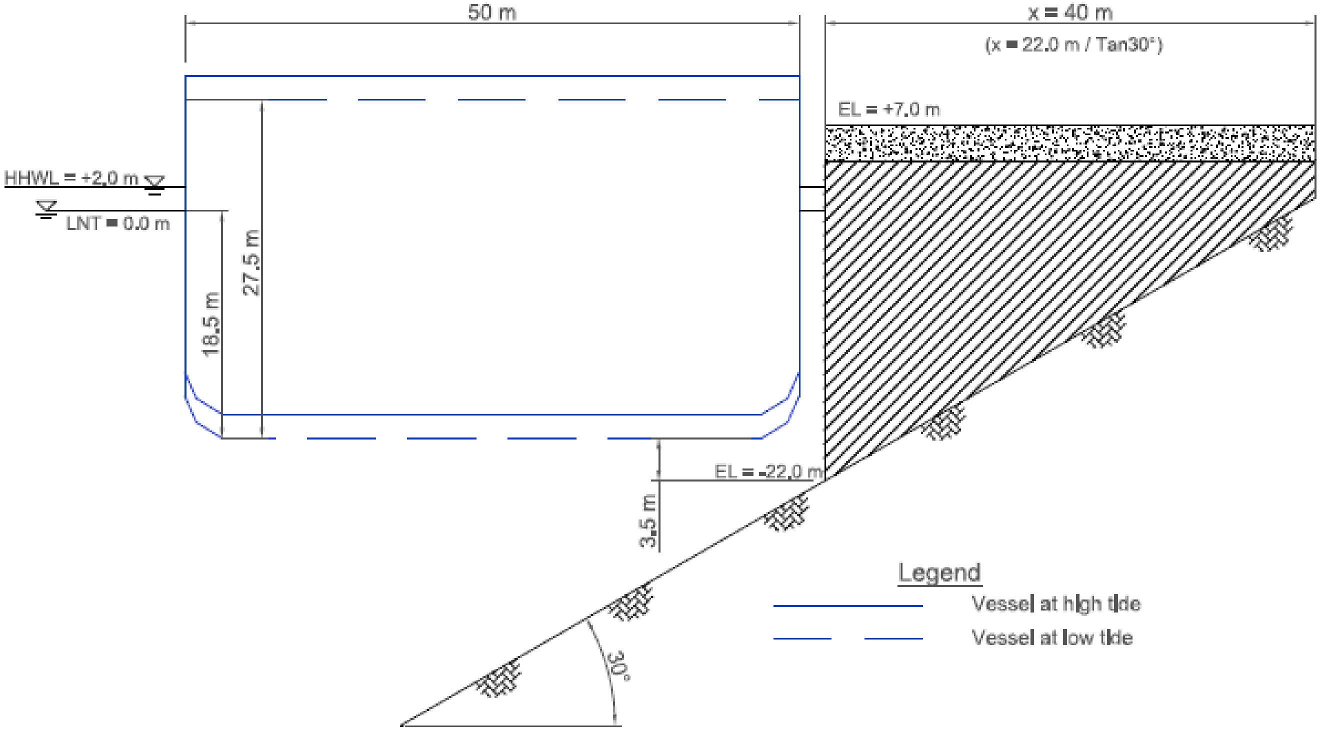

Figure 4 - Berthing Arrangement Cross Section

6 STRUCTURE LOADS

In addition to the operational loads, the terminal will experience a variety of lateral forces. These lateral forces consist of berthing and mooring loads and loads due to environmental conditions. Considering the arctic location of the structure ice forces were also considered and are examined in Section 12, as they are largely dependent on the type of structure employed at site.

6.1 BERTHING FORCES

The normal berthing energy was calculated based on a design formula and recommended coefficients as stated by Gaythwaite in Design of Marine Facilities for the Berthing, Mooring, and Repair of Vessels. [5]

$$E_N = W/2g \times V^2 \times C_m \times C_c \times C_e \times C_s = \frac{\left(4/3 \times 220{,}000 DWT\right)}{2 \times 9.81 m/s^2} \times \left(0.15 m/s\right)^2 \times 1.8 \times 0.5 \times 1.0 \times 1.0$$
$$= 302\, ton - m$$

The terms in the above equation are:

- W = vessel displacement
- V = vessel velocity normal to pier
- g = acceleration due to gravity
- C_m = virtual mass coefficient, accounts for entrained water
- C_c = configuration coefficient, accounts for pier type and geometry
- C_e = eccentricity coefficient, accounts for vessel rotation
- C_s = softness coefficient, accounts for relative stiffness of vessel and fender system

The vessel displacement was calculated as 4/3 of the vessel's DWT. The approach velocity is dependent upon the exposure of the berth and the size of the vessel. Since the vessel is very large, greater than 200,000 DWT, and is berthing in a relatively sheltered location, a berthing velocity of 0.15 m/s was selected. [5]

The virtual mass coefficient, C_m, is a function of the ratio of under keel clearance to the vessel's draught. It accounts for the body of water carried along with the vessel as the ship moves sideways while approaching the quay. Typical values usually range from 1.3 to 1.8. The under keel clearance was selected as 10% of the vessel draught which corresponds to a C_m value of 1.8. [5]

This added water mass also has the effect of cushioning the berth of the vessel and dissipating some of the berthing energy. The configuration coefficient, C_c, ranges from 0.8 to 1 depending whether the structure is closed, semi closed, or open. However, if the effects of under keel clearance were accounted for in C_m, a value of 1 is typically assigned to C_c. [5]

The eccentricity coefficient, C_e, allows for the energy dissipated by rotation of the ship about its point of impact with the fenders. Assuming a quarter-point berthing case, Gaythwaite recommends using a value of 0.5 for the eccentricity coefficient, C_e. [5]

It is common practice to assume that soft fenders will be employed, which corresponds to a softness coefficient, C_s, of 1. [5]

The berthing energy acts as an impact force to the structure and will also be used to design and select typical fenders to be installed on site.

6.2 MOORING FORCES

The mooring forces affect the selection of bollards and the required mooring arrangement. A number of different loading cases were examined based on the provided environmental data to determine the maximum mooring loads that will be experienced. Both wind-generated and current-generated mooring forces were calculated.

The various loading scenarios account for:

- Wind direction (from waterside or landside),
- Tidal range (high tide vs. low tide),
- Draught condition (loaded vs. ballast).

The combination of these factors created different loading conditions because for each combination a different area of the vessel hull is exposed to either winds or currents.

The calculations were completed in accordance with British Standards Design Manual, BS6349, Part 4 - Maritime structures. Code of practice for design of fendering and mooring systems. [10]

The calculations are based on the drag force equation:

$F_D = \rho \times C_D \times V^2 \times A_P$

The terms in the above equation are:

- ρ = density of fluid (air or water)
- C_D = drag coefficient
- V = velocity of fluid relative to object
- A_P = projected area normal to direction of flow

The forces were calculated separately for wind and current and then combined to get a total force. Longitudinal and transverse wind forces were calculated for wind speeds at a direction of 90 degrees, 45 degrees, and 0 degrees. Current forces were calculated based on currents in both the longitudinal and transverse directions. The different coefficients for each case were determined using figures and graphs provided in the standard. The figures and graphs used as well as the detailed design calculations are found in Appendix C.

A summary of the mooring force calculation results are shown in Table 4. Reference Figure 5 to explain the notations used in Table 4.

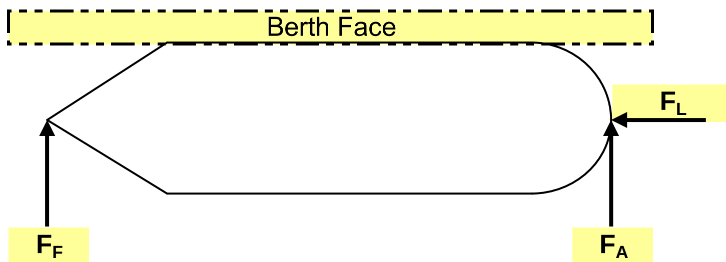

Figure 5 - Mooring Force Schematic

Condition			Forces (kN)		
Tide	**Draught**	**Wind**	**F_F**	**F_A**	**F_L**
LLWL	Ballast	Ocean	2780	3279	1481
LLWL	Loaded	Ocean	1279	1984	1341
LLWL	Ballast	Ocean	2741	3235	1481
LLWL	Loaded	Ocean	740	1383	1207
HHWL	Ballast	Land	1969	2321	1039
HHWL	Loaded	Land	856	1192	678
HHWL	Ballast	Land	1930	2278	1039
HHWL	Loaded	Land	316	591	544

Table 4 - Mooring Force Results

6.3 ENVIRONMENTAL FORCES

Environmental forces consider the pressure on the structure due to either wind, current, or wave loading. The loads from vessel berthing, and the anticipated ice loading due to the arctic environment will govern the lateral loading of the structure and as a result environmental forces have been neglected in design.

7 SELECTION OF STRUCTURAL ALTERNATIVES

The uniqueness of the project facilitated a need to research multiple structural solutions and to assess their suitability to the given site conditions and loading environment. A ranking matrix was developed to assist in the selection of structural alternatives for preliminary design and to ensure a systematic approach to the selection process was followed.

Due to the structure's location in a sheltered bay, static ice loading must be considered. Static ice loading includes thermal stresses and wind and current actions on ice sheets adhering to the structure. Ice movements around the structure can also cause material abrasion. In vertical face structures, ice formation can obstruct berthing operations.

7.1 MATERIAL CONSIDERATIONS

Three types of material have been considered; concrete, steel and timber.

7.1.1 Concrete

In general, concrete has long-term durability in ocean environments. However, serious concrete deterioration can occur on the face of a structure depending on the exposure conditions. In the uppermost zone of a structure that is only exposed to the atmosphere, cracking can occur due to corrosion of the reinforcement. In the splash zone concrete is exposed to spray, frost action, solar radiation and rapid evaporation. Below the splash zone is the tidal zone where the structure is exposed to repeated cycles of wetting and drying, freezing and thawing, ice abrasion and wave action. These two zones are most vulnerable to deterioration as concrete cracking and spalling can occur. In the submerged zone, loss of concrete strength can occur due to chemical reactions between seawater and hydration products in the cement. [11]

7.1.2 Steel

Steel degradation occurs by corrosion where electrons flow from an anode to a cathode. Steel corrosion is intense in the ocean environment because dissolved salts greatly increase water conductivity and hence its corrosiveness. Ice conditions cause removal of all corrosion products and effectively expose totally bare steel every spring. Other factors influencing corrosion include water temperature, oxygen concentration, pH value and water salinity. [11]

7.1.3 Timber

Timber structures have a satisfactory performance in marine environments. Historical applications show that sea ice can cause catastrophic structural damage to timber structures. Also, timber structures are susceptible to degradation by marine organisms such as bacteria, fungi, mollusks and crustaceans. Mollusks and crustaceans are particularly destructive as they will bore and destroy timber structures. [11]

7.2 STRUCTURAL CANDIDATES

A number of structural options have been considered for the selection process. The candidates may be grouped into the following types of structures:

- Gravity walls
- Sheet pile walls
- Open pile structures
- Floating structures
- Timber crib

Bottom mounted marine structures are classified as either flexible or rigid. For example, gravity-type structures are classified as rigid and open pile structures are classified as flexible. The classification of a structure determines the force interaction and approach for analysis. Forces on rigid structures can induce some vibrations, but in general they can be treated as static, whereas forces on flexible structures can cause dynamic effects. In a dynamic analysis, the mass, stiffness and damping characteristics of the structure must be considered along with damping effects due to water, foundation and friction. In some cases of dynamic analysis, model tests may be required to accurately determine forces and reactions. [11]

7.2.1 Gravity Walls

Gravity walls are earth retaining structures. They provide an alongside berthing arrangement and have a bearing capacity capable of carrying loads such as the weight of ship loading equipment. The stability of a gravity wall is obtained from the self-weight of the structure and the imposed weight of any soil lying above. In order to prevent excess pore pressure build-up behind the structure, drainage is necessary to remove excess rainwater. Different types of gravity walls include block wall, L-wall, caisson wall, and cellular wall. [9]

Block Wall

A block wall is the simplest type of gravity wall and achieves stability from the self-weight of its large blocks. The blocks are made of either concrete or natural stone and are piled on top of each other. The blocks are placed from the waterside on a layer of gravel or crushed stone and covered with a reinforced concrete cap. Block walls require good bearing material such as very firm sand or rock. Retaining heights above 20 m are possible and the wide joints between blocks allow adequate drainage. Block walls require a high amount of material however they are not labour intensive. [9]

L-wall

An L-wall achieves stability from the weight of concrete plus the weight of the surcharge material that rests on it. The slim construction of an L-wall is applicable where the bearing capacity of the soil is insufficient to handle the weight of a block wall or where there may be cost savings from the lower material requirement. The large structural elements of L-walls may be pre-fabricated elsewhere and placed on a gravel bed on the site using heavy lifting equipment. As an alternative, L-walls can be constructed on site using a large building pit and de-watering system. [9]

Caisson Wall

Caissons are large hollow cellular concrete structures that are filled with soil or other material of sufficient mass to provide stability. Good bearing material is required to support the weight of the structure. Caissons are economical in material use but tend to be labour intensive. Caissons are usually pre-fabricated on a construction dock, floated to site and sunk into place. After filling, the construction of the superstructure can be completed above water. [9]

Cellular Wall

A cellular wall consists of interlocking steel web profiles driven to form cylindrical cells. The cylindrical cells consist of material enclosed by steel rings. The cells rest on the seabed and require good bearing material. The walls are relatively thin and vulnerable to damage if collisions occur. Cellular wall structures are economical in material use but tend to be labour intensive. [9]

7.2.2 Sheet Pile Walls

Sheet pile walls obtain their soil retaining function and stability from the fixation capacity of soil. They consist of interlocking vertical steel elements driven into the subsoil. They are useful in areas where the soil has poor bearing capacity and as a fundamental requirement, the subsoil must be easily penetrable. Sheet pile walls may be free standing or anchored where higher retaining heights are required. A drainage system is required to reduce excess pore pressure from rain water. Sheet pile wall systems include single, combined and cofferdam walls. [9]

Single sheet piles can have U, H or Z profiles. If a higher retaining height and load capacity is required, combined sheet piles may be used. These include heavy steel elements at a specified spacing in addition to single sheet piles. A cofferdam wall includes two sheet pile walls with soil filling the space between the walls. The walls are connected by anchors and work as a single unit to transfer horizontal and vertical loads to the subsoil. [9]

7.2.3 Open Pile Structures

Open pile structures consist of a deck supported by vertical and inclined piles. A pile structure is useful when relatively poor subsoil conditions exist. The piles used in open structures can be steel or concrete and may be prefabricated. In order to support deck loading, such as the rails for a ship loader, the piles can be spaced accordingly. The underside of the deck is difficult to access making long-term maintenance more complex and the structure is easily damaged by collisions. [9]

7.2.4 Floating Port

Floating ports can be of one pontoon or several pontoons linked together and can be built of concrete or steel. Floating ports are applicable for remote areas only accessible by water where components can be prefabricated and floated to site. Past experience has shown that for specific conditions, such as short periods suitable for construction or unfavorable soil conditions, floating piers would be less costly and less time consuming. In areas of heavy ice flow and corrosive environments floating ports may incur higher operation and maintenance costs. So as not to impede the proper function of loading equipment they should be used in areas where waves generated seldom exceed 1 m to 1.5 m in height. [12]

7.2.5 Timber Crib

Timber cribs are a rectangular lattice of logs or heavy timbers used to retain rocks or rubble. They are typically used in areas where bottom conditions prevent driving piles deep enough to give lateral stability. Cribs are built on land and floated into position where they are then filled with rock. [13]

7.3 RANKING CRITERIA

A list of 15 items was generated as criteria influencing the process of selecting viable design options. Table 5 contains a list of all criteria and the associated performance requirements. Provided below is a description of the criteria considered:

Subsurface soil conditions: The depth to bedrock influences the fixity of certain structure types to the seafloor and the bearing capacity of the ocean floor affects the suitability of structural options.

Profile of seafloor: The bathymetric steepness affects the ability of the structure to rest on the seafloor. Gravity type structures require a flat resting surface whereas piles and sheet piles are independent of the seafloor. However, open-piled structures are complicated by the sloping of the seafloor because it creates irregularities in the pile driving process.

Water depth: Structures located in deep water are more susceptible to buckling or bending (if earth material is retained). Steel is more suitable for greater water depths along with large robust structures in comparison to slender structures.

Construction material requirements: Pre-fabricated units are ideal because the material can be shipped ready for installation. Similar options which contain additional material requirements may have enhanced stability but at the expense of cost associated with materials.

Material degradation: Steel is better than concrete which is likewise better than timber in arctic environments. Although all materials are suspect to some degree of degradation, the mitigations to prevent degradation of steel are primarily allowances for corrosion. Concrete however must account for corrosion of reinforcing steel as well as the degradation of the concrete itself by means of chemical deterioration for instance. Open pile structures were deemed more susceptible to degradation then their vertical face counterparts of the same material. This is because a greater percentage of the surface area is exposed in open pile structures as compared to sheet piles.

Accommodation of ship loader foundation: A higher performance rating was granted to structures whose inherent design could potentially provide support to the ship loader.

Historical arctic applications: Higher values were awarded if there is a widespread application of the option under consideration. However, a reduction in score was made if the option was prone to potential problems.

Pore pressure build up: The ability of the structure to drain excess pore pressures built up on the land side of the structure. The more open the structure the higher the performance rating.

Constructability: Constructability consisted of three (3) options that were averaged to give the performance score. This category includes labor requirements, the ease of construability, especially in an arctic environment, and familiarity with the method, as well as the available construction season associated with each. For instance, some options may permit using the ice to the advantage of an employed construction method thereby increasing the available construction season.

Load and impact resistance: Rigid structures are the most robust, followed by flexible structures. Floating structures and timber structures display the least impact resistance.

Long term maintenance: Considers the ease and accessibility to providing maintenance to the structure.

Resistance to ice abrasion: Steel has a greater resistance to ice abrasion than concrete which is in turn better than timber. Open pile structures are deemed better then their similar material vertical face counterparts because ice will break up around the pile creating an area of less ice behind the pile. Therefore a lower percentage of its surface area is exposed to ice abrasion.

Freeze-thaw durability: Steel is more durable to the process or repeated freeze and thaw than concrete which is more durable than timber. Open pile structures were deemed more susceptible to freeze-thaw cycles then their vertical face counterparts of the same material. This is because a greater percentage of the surface area is exposed in open pile structures as compared to sheet piles

Berthing ice control: Vertical face structures require some form of process or device to eliminate the build up of ice along the berth face which can restrict the docking of vessels. Open face structures permit the passage of ice under the deck reducing the need for an ice control mechanism. Some structures my have a moderate ability to deflect the build up of ice but will have a greater reliance on an ice control technique than open pile structures.

Susceptibility to the dynamics effects of ice: Bottom fixed units are the least susceptible to the dynamic loading of ice because their shear magnitude and rigidity offer great resistance to vibrations. The dynamic interaction between flexible structures and ice is more pronounced because studies show that the natural frequency of flexible structures closely matches that of dynamic ice loading. [11]

		Performance Rating				
#	Criteria	1	2	3	4	5
1	Subsurface soil conditions	Most difficulty penetrating (ex. sheet piles)	N/A	Difficulty penetrating (piles)	N/A	Beneficial or does not apply (gravity & floating
2	Profile of seafloor	Require flat surface	N/A	Complicates design	N/A	Independent of profile
3	Water depth	Timber	Long & slender (ex: L-wall)	Box gravity units	Sheet piling	Open-piled structures
4	Construction material requirements	High quantity of materials requires (ex: block walls)	Floating (lots of items)	Open piles or combined steel options	Single sheet piling	Pre-fabricated concrete units
5	Material degradation	Timber	Concrete (open piles, greater exposure)	Concrete (vertical face)	Steel (open piles, greater exposure)	Steel (vertical face)
6	Accommodation of ship loader foundation	No inherent support (open piles)	Rubble-filled structures with interfering anchors	Rubble-filled structures that can accommodate placement of a foundation	Semi-support already provided (large concrete caps, cell arcs)	Possibility of using structure as support (ex: caissons)
7	Historical arctic applications	No information or failed applications	Limited examples	Some examples	Many applications	Many applications with desirable results
8	Pore pressure build-up	No release of pore pressure	Interlocking piles	Links between sections permit escape of pore pressures but filter required to keep material retained	Links between sections permit escape of pore pressures	No pore pressure build-up
9	Constructability	Short season, difficult construction practice, labor intensive	Between 1 & 3	Moderate season, construction method, and labor requirements	Between 3 & 5	Long season, easy construction method, little labor required
10	Load and impact resistance	Not bottom-fixed	Timber	Flexible (open)	Rigid, thin materials	Rigid, vertical-face, thick material
11	Long term maintenance	Timber	Not bottom-fixed	Open structures	Gravity walls	Sheet piling
12	Resistance to ice abrasion	Timber	Vertical face concrete	Piled concrete & vertical face steel	Piled steel	Not bottom-fixed
13	Freeze thaw durability	Timber	Piled concrete	Vertical face concrete	Piled steel	Vertical face steel
14	Berthing ice control	Vertical face	N/A	Some allowance for ice diversion	N/A	Open structures
15	Susceptibility to dynamic effects of ice	Timber	Flexible structures	Not bottom-fixed	Bottom-fixed linked units	Bottom-fixed single units

Table 5 - Performance Criteria Breakdown

7.4 SELECTION OF STRUCTURAL ALTERNATIVES

Figure 6 shows a schematic of the selection matrix utilized in the selection process. The matrix was created to rank the alternatives according to a point system. The point system is based on a weighted value and rank method.

Weight (W) is a function of the critical nature of the criteria with five (5) being the most critical items. Performance (P) is the ability of an option to meet the criteria in question with five (5) being the most capable. The product of the weight and performance equals the amount of points (PTS) awarded to an option for that specified criteria. The sum of the points for all 15 criteria provides the score for that option. The options are then ranked according to their score with the highest rank awarded to the option with the highest score.

The criteria shown in the figure is the actual criteria considered in the process as described in Section 7.3. The weight shown adjacent to the criteria is the weight associated to that criteria item. A breakdown of the weight distribution is shown in Table 6.

Due to the amount of options considered, the schematic contains only three (3) fictional options labeled one (1) through three (3) to help explain the functionality of the schematic. The performance rating attributed to each option for each of the criteria items is also fictional.

Weight considers the critical nature of the criteria:
5 >> most critical
1 >> least critical

Performance is the capability of the option to meet the criteria:
5 >> most capable
1 >> least capable

#	Criteria	Weight (W)	Option 1 Performance (P)	Option 1 Points (PTS)	Option 2 Performance (P)	Option 2 Points (PTS)	Option 3 Performance (P)	Option 3 Points (PTS)
1.	Subsurface soil conditions	5	4	20	1	5	4	20
2.	Profile of seafloor	5	3	15	2	10	4	20
3.	Water depth	4	2	8	1	4	5	20
4.	Construction material requirements	3	1	3	3	9	3	9
5.	Material degradation	2	5	10	4	8	2	4
6.	Accomodation of ship loader foundation	2	4	8	4	8	1	2
7.	Historical arctic applications	2	5	10	5	10	1	2
8.	Hydraulic conditions	3	5	15	3	9	2	6
9.	Constructability	3	3	9	4	12	4	12
10.	Load and impact resistance	4	2	8	2	8	5	20
11.	Long term maintenance	1	4	4	1	1	3	3
12.	Resistance to ice abrasion	3	5	15	4	12	2	6
13.	Freeze thaw durability	3	5	15	3	9	1	3
14.	Berthing ice control	2	2	4	5	10	2	4
15.	Susceptibility to dynamic effects of ice	4	5	20	2	8	5	20
	TOTAL SCORE			164		123		151
	RANK			1		3		2

W x P = PTS
3 x 5 = 15

Ranking is based on the total score:
1 >> most desired option
3 >> least desired option

Figure 6 - Selection Matrix Schematic

Weight	Description
5	Client specified, major impact on design conditions.
4	Affects structural stability of option.
3	Affects structural integrity of option.
2	Miscellaneous items affecting design.
1	Maintenance; post design criteria.

Table 6 - Weight Distribution

7.5 SELECTION RESULTS

Based on the execution of the selection design matrix, the following three (3) options were selected to be further analyzed:

- Caisson wall
- Steel cellular wall
- Floating hybrid

The results of the selection process are contained in Appendix D.

Upon submission to the client of our selected design options, they recommended replacing the floating hybrid option with an open pile design. An open pile design was the next highest rated alternative in our selection process as all sheet pile systems scored the same. Based on our selection methodology it was determined that steel piles will behave better than concrete piles given the site constraints and thus an open pile steel design will be compared with the concrete caisson wall, and the circular sheet pile cell wall.

8 STRUCTURE ALTERNATIVES

8.1 GENERAL

Three (3) wharf structure forms have been deemed suitable and are considered for implementation at the site. Option 1 is a circular sheet pile cell design, option 2 is a concrete caisson structure, and option 3 is steel open pile structure.

The first two (2) options are solid-fill gravity structures. These structures rely primarily on their weight and friction on the foundations to resist any of the possible adverse load combinations. Gravity structures can withstand very high lateral loads from vessel impact without sustaining damage and may be subjected to overload conditions from vessel collisions without collapse or irreparable damage. [5]

Piles are used extensively in marine environments to carry structural loads through the water column to the underlying foundation. End-bearing piles derive their main resistance from the bearing capacity of the hard or dense soil or rock to which the piles are driven. Due to the exposed bedrock conditions located on site end-bearing piles will be examined. [5]

A more detailed description of each alternative will follow in addition to a breakdown of the preliminary design process of each.

8.2 OPTION 1: SHEET PILE CELLS

This option is the construction of a sheet pile cofferdam with straight web sheet piles driven in the form of interconnecting circles to form circular sheet pile cells. A typical layout of this structural option is shown in Figure 7. The advantage of circular type construction is that each individual circular cell can act as an independent structure, thus greatly facilitating construction. [5]

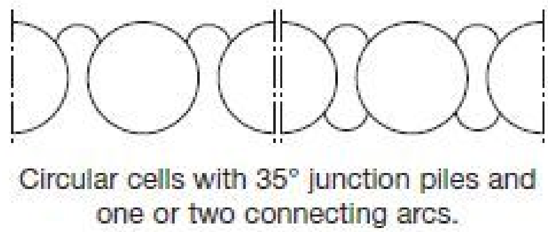

Figure 7 - Typical Circular Sheet Pile Cell Arrangement

The continuity of the wall is achieved by the intermediate arcs which connect the circular structures by means of fabricated junction piles. Typical junction piles are shown in Figure 8.

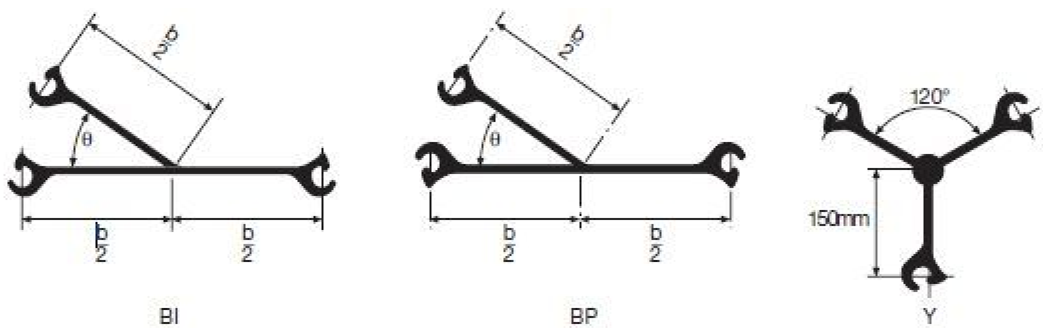

Figure 8 - Sheet Pile Cell Junction Piles

Proper alignment during driving is critical to achieve closure of the cells and connection with adjacent cells. As a result driving templates are utilized to ensure cell configuration. Pile penetration will assist in the resistance of any lateral loads occurring during the construction phase and in the vulnerable period before the fill has been placed and the cell becomes inherently unstable. [14]

The sheet pile structural option under consideration is based on nine (9) 28.82 m diameter cells interconnected with arcs on both the exterior and interior of the wharf to provide a continuous berth face for the vessel. The sides of the soil retained wharf are constructed with identical sized cells flared at an angle of 40° from the berthing face. The flaring of the sides is preferred in comparison to constructing the sides perpendicular to the structure face as it would dissipate the frictional forces caused by ice adhering to the structure. [15]

A layout and cross section of the proposed circular sheet pile option is shown in Figures 9 and 10 respectively.

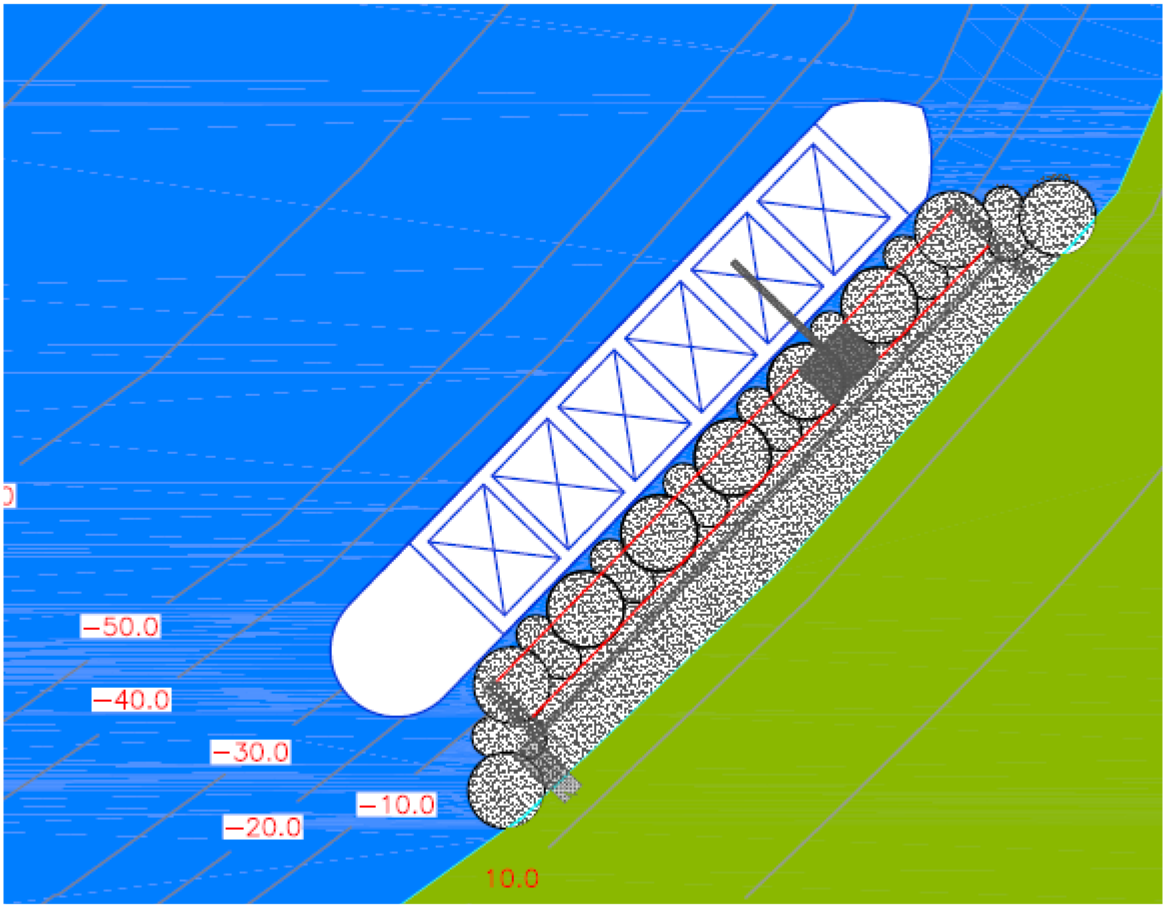

Figure 9 - Sheet Pile Cell Site Plan

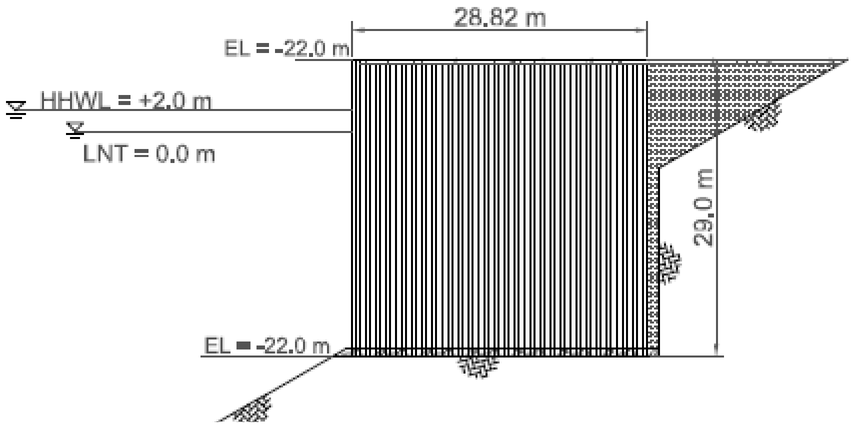

Figure 10 - Sheet Pile Cell Cross Section

8.2.1 Installation of Cellular Sheet Pile Structures

Since there are exposed bedrock conditions at site, there is no overburden material for the sheet piles to penetrate into. In these situations it is recommended that approximately eight (8) feet (about 2.5 m) of free-draining cell fill material be placed to provide a toe for the sheet piling in the installation operation as shown in Figure 11. This blanket of free-draining material provides stability to the cells during the setting and driving operations. Placing this blanket first is often more economical than filling the cell after the sheet piles have been driven. [16]

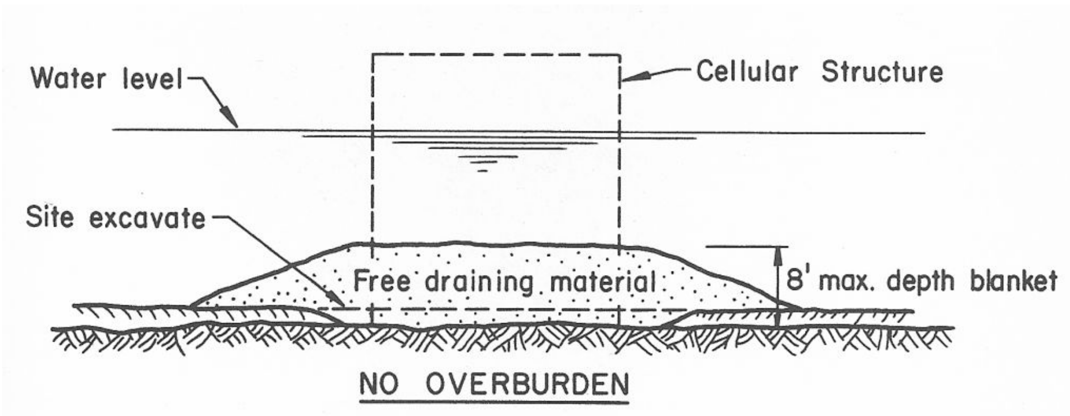

Figure 11 - Sheet Pile Cells at No Overburden Site

A template is required to set sheet piles correctly in a cellular structure. The template should be built to a diameter of about 0.25 m smaller than the driving line dimensions of the sheet pile. This permits ample room to set and swing or rotate the sheet pile to the correct orientation and prevents the template from binding during removal. [16]

Once the site is prepared the installation of the sheet piling follows six (6) general steps.

Step 1

- Installation of template and supporting piles.

Step 2

- Positioning of four or more isolated sheet piles (usually the special junction piles).
- Verification of the verticality of the sheet piles.
- Threading of adjacent sheet piles.

Step 3

- Closing of cells between special junction piles.
- Threading of arc piles (2 or 4).

Step 4

- Driving of piles using staggered driving method after closing of the cell.

Step 5

- Partial filling of the cell.
- Removal of template and working platforms.

Step 6

- Extraction of supporting piles.
- Backfilling to the top of cell.

Figure 12 shows the construction of a cellular sheet pile structure in an environment comparable to the proposed location of the St. Lawrence Marine Terminal.

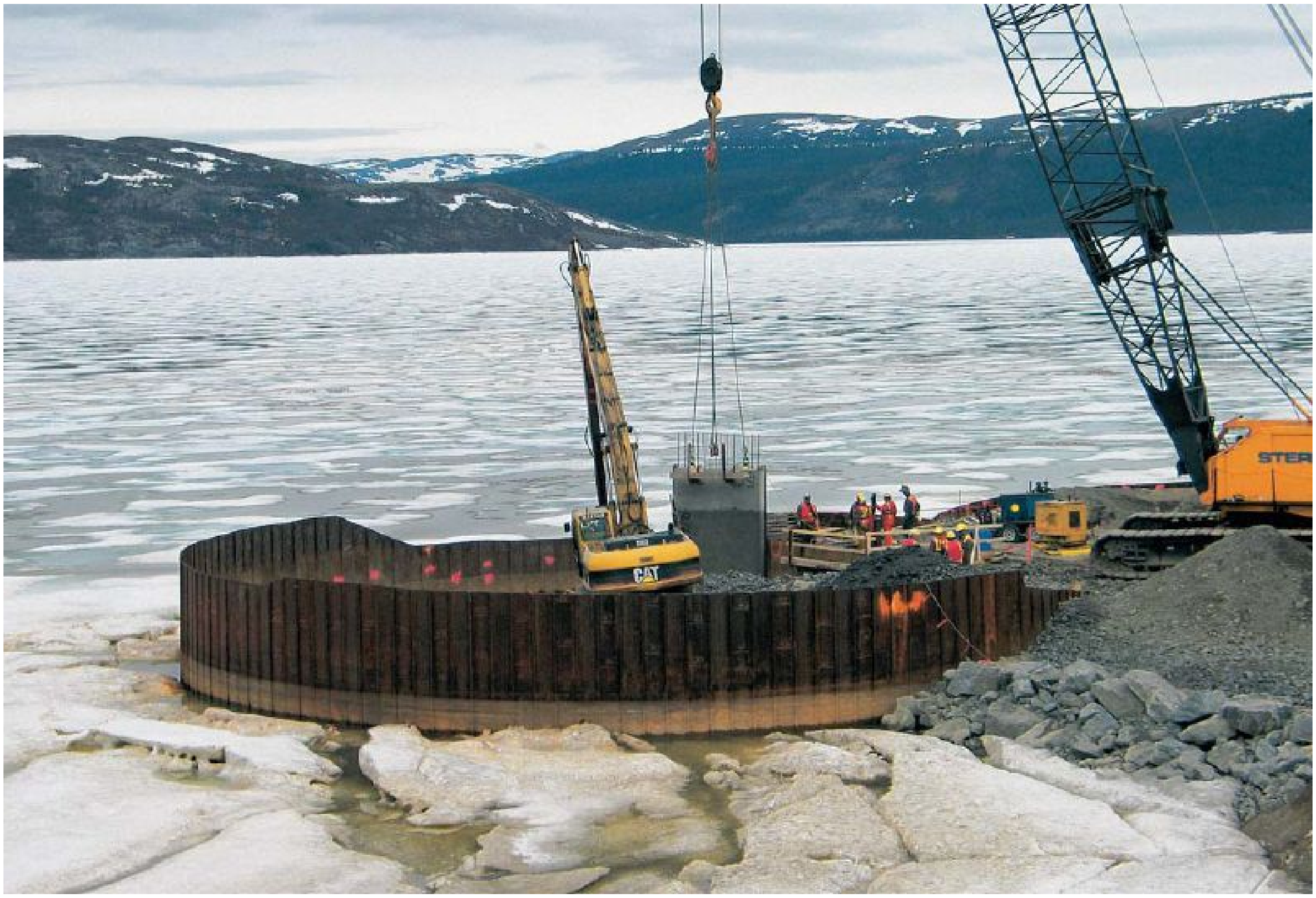

Figure 12 - Sheet Pile Installation in Arctic Environment

8.3 OPTION 2: CONCRETE CAISSON

Concrete caissons are composed of a row of prefabricated reinforced concrete shells built on land, transported to site, floated into position, and ballasted to a prepared rubble mattress. The caissons are then filled with a granular rock backfill. Concrete caissons require the sea floor to be as level as possible for its final resting position. This necessitates blasting of the sea floor and the construction of the rubble mattress foundation.

Difficulties arise in attaining precise alignment during placement, requiring filters or grout to be placed between adjacent units to prevent washout of backfill. Also, a cast-in-place concrete cope wall forms the upper part of the dock face, allowing true alignment and grade, as well as providing attachment for fender systems, cleats, railings, and other hardware. [5]

The caisson wharf structure proposed for St. Lawrence Marine Terminal will be comprised of ten (10) individual units measuring 28 m in length by 21 m wide. Each individual caisson will have four (4) cells along its length plus three (3) cells in the direction of its width for a total of twelve (12) cells per unit. The total height of the structure will be 29 m.

A layout and cross section of the proposed concrete caisson option is shown in Figures 13 and 14 respectively.

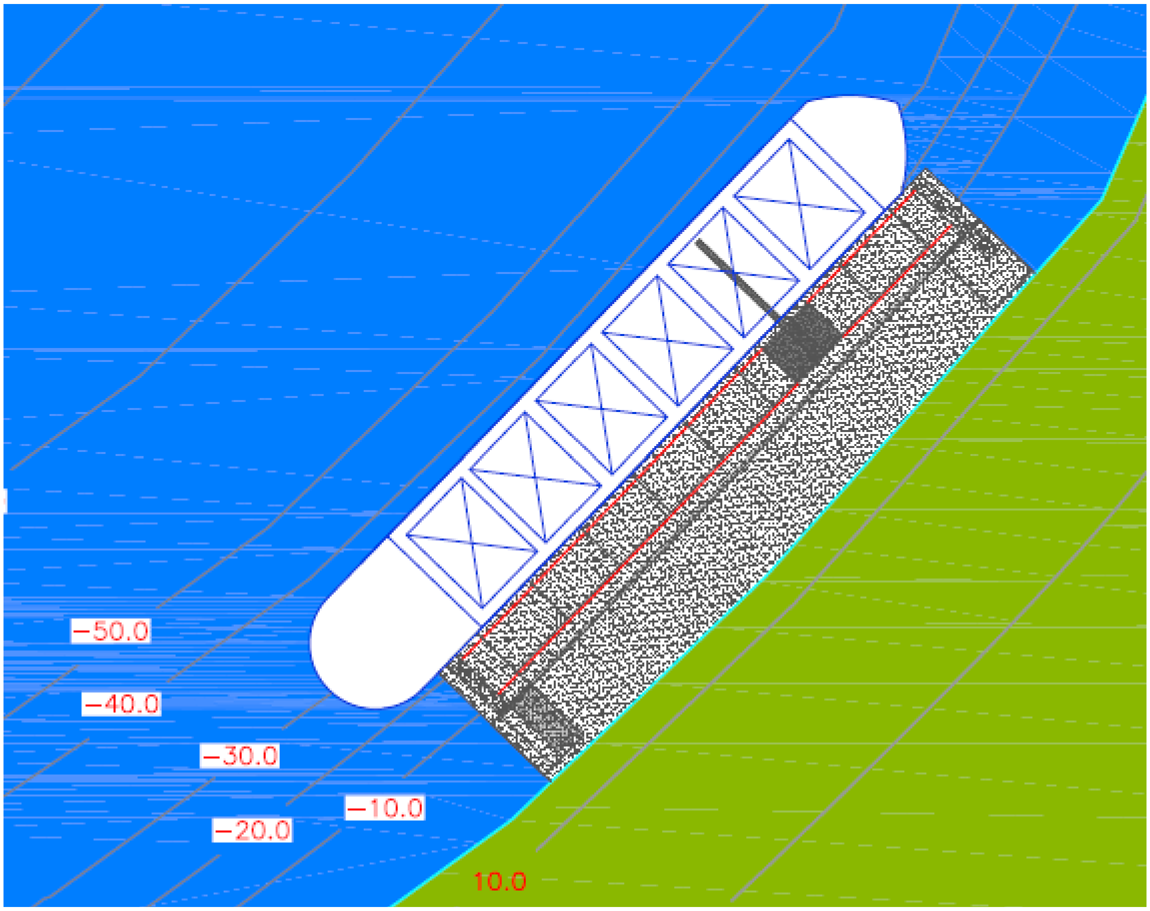

Figure 13 - Concrete Caisson Site Plan

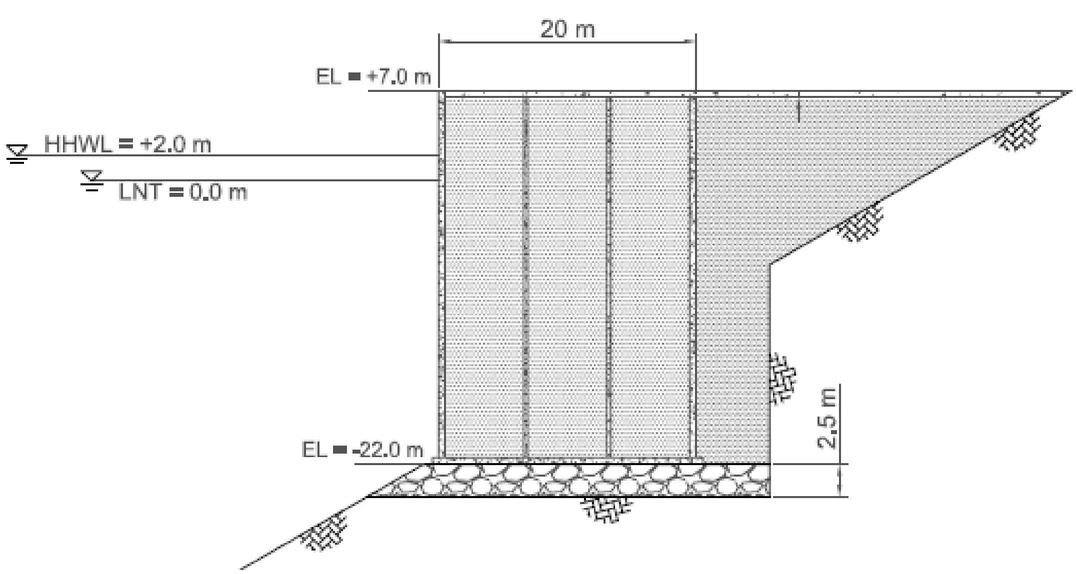

Figure 14 - Concrete Caisson Cross Section

8.3.1 Concrete Caisson Construction

The general principles governing the construction and installation of concrete caissons are described by Gregory P. Tsinker in Handbook of Port and Harbor Engineering: Geotechnical and Structural Aspects. [17] One of the advantages of concrete caissons is that the majority of construction can be performed on land for ease of access. As a result, construction delays due to weather and wave conditions are much less prevalent.

There are several construction options available which include:

- On-site fabrication using a purpose built graving dock or launching slipway.
- On-site fabrication using a semi-submersible barge.
- Off-site fabrication at suitable existing facilities and being towed or ferried to site.

Given the remote location for the proposed structure it is highly likely that a slipway launch will be utilized.

It is viable to construct the concrete caissons close to shore given the large amount of vacant land in proximity to the shore on-site. Due to the water depth at the berthing face, the caissons will only be partially constructed on land.

The concrete for caisson structures in marine applications is typically normal weight structural concrete, with a minimum crushing strength of 35 MPa after 28 days. A concrete mix with a water-cement ratio of 0.4 or less by weight is typical.

The structure will consist of cast-in-place reinforced concrete. The slip forms used during the dry construction phase will remain on the structure during transport to the structure’s final position.

Employing the slipway launch method requires the caisson structure to be constructed on a tilting platform adjacent to the shoreline. After dry construction has ceased, the platform is tilted and the caisson slides into the water when the slip forming is complete.

The caisson is subsequently towed to its deployment position where it is ballasted with water onto the prepared rubble mattress foundation. The towing of the caisson can be a complex marine operation and should only be conducted when environmental conditions, such as wind, waves, and currents, are acceptable for such a practice.

Excessive swaying can occur throughout the towing process, so it is essential to ensure a sufficient freeboard at all times to prevent flooding of the cells.

Furthermore, the forces experienced during the caisson launch and the towing phase should be checked to ensure they do not exceed the loads for which the structure was designed for. This loading can be hydrostatic or motion-induced and is to be examined if this option is recommended for detailed design.

It is unlikely that the caisson will be set down in its first attempt within an acceptable deviation. In such an event, some water should be pumped out of the cells so the structure floats minimally permitting re-alignment. This process is repeated until the structure’s position is within acceptable limits.

After the caisson is in position the water ballast is replaced with solid ballast.

Figure 15 shows the various stages described during the construction and launch of a concrete caisson. From the picture in the top left rotating clockwise: 1. The tilting platform with the future slipway located at the right; 2. The dry construction of the caisson resting on the tilting platform; 3. The caisson accelerating down the slipway during launch with slip forms still in tact; 4. The structure beginning its towing phase while maintaining sufficient freeboard to prevent sinking the unit.

Figure 15 - Concrete Caisson Construction and Launch

8.4 OPTION 3: STEEL PILES

Marine piles are often only partially embedded, leaving much of the pile material exposed to severe environmental conditions. The design of piles consequently must consider corrosion, abrasion, impact, ice damage, and cyclic and dynamic loading. [5]

There are several types of piles but steel pipe piles were selected given their advantages in resistance to ice damage as opposed to other materials and pile types. Steel pipe piles may be concrete-filled or not for structural purposes, but often are for corrosion considerations. Similarly thicker pipe walls may also be used to combat corrosion. [5]

Concrete filled steel pipes behave as end-bearing displacement type piles, which can be driven either open or close-ended. An open-ended option is likely since it permits a grouted anchor to be installed through the pile to resist large uplift loads which may potentially exist due to the high lateral loads expected on the structure. [5]

Additionally steel pipe piles provide the ability to be spliced easily using full penetration butt welds all around which will be necessary given the length of piles required near the berthing face. Unspliced pile lengths typically range from 20 to 25 m; however piles of 30 m and greater are needed. [5]

From a structural engineering point of view, piles of circular cross sections are generally preferred because of their efficiency as long columns, high torsional strength, and because the same strength properties in all directions. [5]

The option considered for implementation at site consists of 196 concrete filled steel pipe piles, 1300 mm in diameter with a wall thickness of 30 mm. anchored in 5 m deep sockets drilled into the bedrock. The piles come in varying lengths due to the sloping seabed conditions.

A layout and cross section of the proposed open pile option is shown in Figures 16 and 17 respectively.

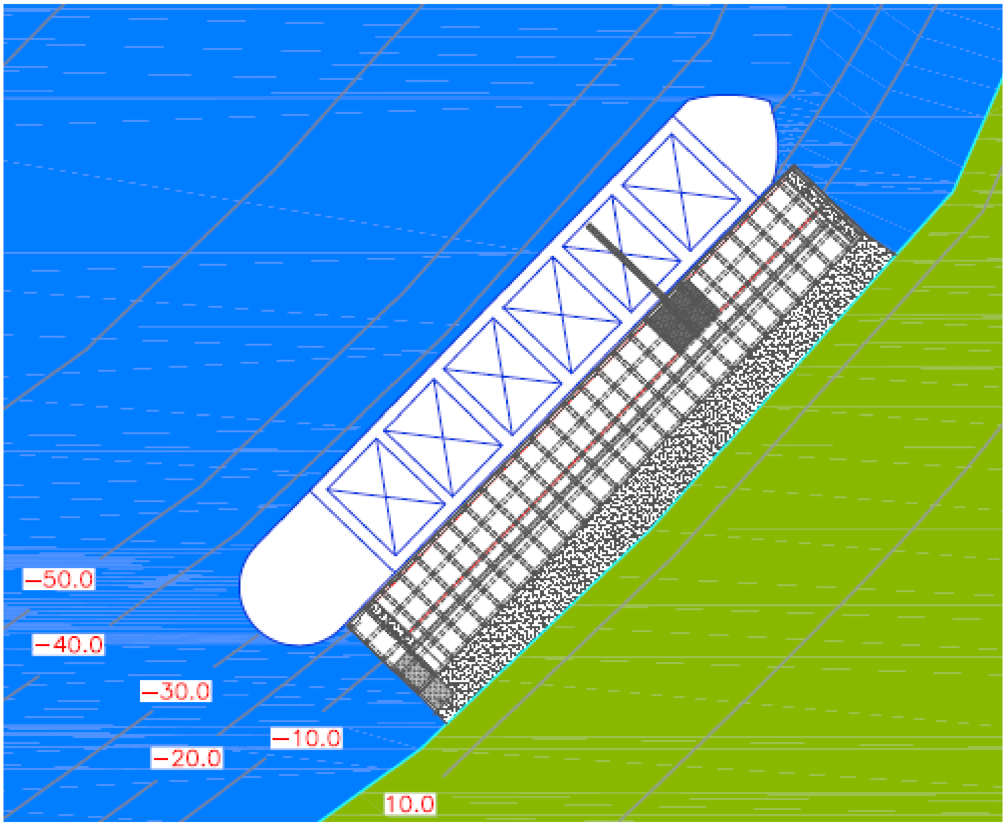

Figure 16 -Steel Pipe Pile Site Plan

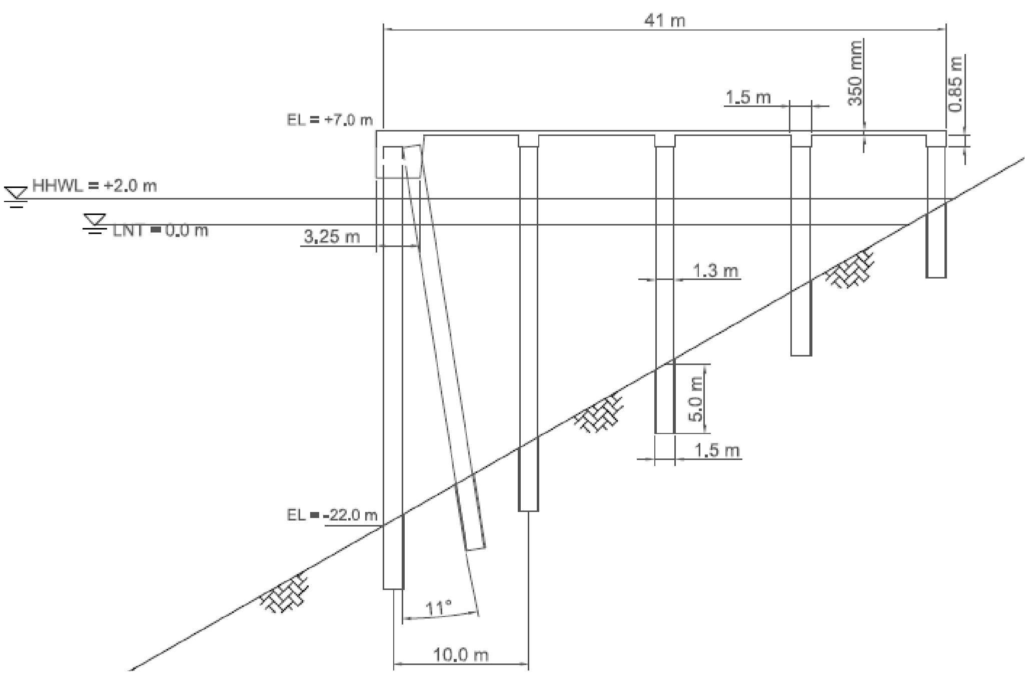

Figure 17 -Steel Pipe Pile Cross Section

8.4.1 Open Pile Installation

Marine piles must be installed to provide bearing uplift and lateral resistance. Tubular steel piles are prefabricated as rolled plates with longitudinal seams. Each rolled plate can be 20 to 25 m long and where piles are spliced the longitudinal seam is rotated by 90°. Piles are then loaded onto a barge and towed to site where large derrick barges are used for installation. Where piles are to be mobilized in bedrock, boreholes can be drilled ahead of the pile. The pile is then set in place with grout or concrete. The concrete may need to be reinforced to transfer load. [18]

9 PRELIMINARY STRUCTURAL DESIGN

At the preliminary stage in the design process, each alternative was examined based on a static analysis against global failure to determine the approximate dimensions required for each option to satisfy required stability checks.

The design of the circular sheet pile cells, concrete caissons, and steel piles are further detailed in the following sections.

9.1 CELLULAR SHEET PILE DESIGN

As a result of the exposed bedrock conditions on site, the sheet piles will be resting directly bedrock. This situation can be modeled as a cofferdam design placed directly on bedrock. The design was based on guidelines presented in the Pile Buck Steel Sheet Piling Design Manual[19], and the Handbook of Port and Harbour Engineering: Geotechnical and Structural Aspects[17].

A variety of failure mechanisms must be satisfied in the design of cellular sheet pile structures. They are sliding, overturning, slipping between sheeting and cell fill, vertical shear, busting and sheet pile interlock failure, and horizontal shear. A factor of safety of 1.5 is required for each stability check. Each failure mechanism is analyzed in detail throughout the subsequent subsections.

The design of circular sheet pile cells interconnected by sheet pile arcs are simplified by equating an effective width to allow simplified expressions to be used in the stability checks. The Arcelor-Mittal Piling Handbook[14] was referenced during the design which provided properties of sheet piles and possible arrangements of sheet pile cell orientation, each with a distinct effective width. Multiple arrangements were examined and compared and consideration was given to the required amount of piles as wells as the number of driving templates needed.

It was determined that the optimal option consisted of nine (9) 28.82 m diameter cells interconnected with arcs by means of a 35° junction pile. The proposed design option has an effective width, w_e = 24.72 m. The geometric orientation and arrangement of circular sheet pile cells is based on material provided in Chapter 9 of the Arcelor-Mittal Pilling Handbook which is contained in Appendix E.

9.1.1 Stability Against Sliding, Overturning, and Slipping

Figure 18 shows the forces acting on the sheet pile cell considered in the stability check against sliding, overturning, and slipping.

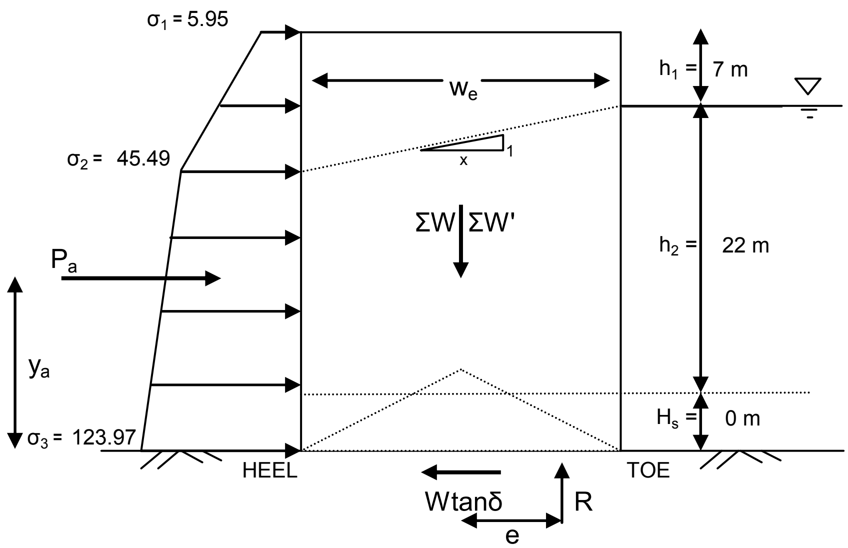

Figure 18 - Sliding, Overturning, and Slipping Stability Check

The driving force, P_a, acting on the structure is a result of the surcharge load on the wharf deck plus the pressure generated by the retained material. P_a is the resultant of the pressure diagram shown to the left of the structure.

The highlighted points of the pressure diagram are calculated as follows:

$\sigma_1 = k_a \text{ x } q_{sur.} = 0.2973 \text{ x } 20 \text{ kPa} = 5.95 \text{ kPa}$

$\sigma_2 = k_a \text{ x } (\gamma_d * h_1) + \sigma_1 = 0.2973 \text{ x } (19 \text{ kN/m}^3 \text{ x } 7 \text{ m}) + 5.95 \text{ kPa} = 45.49 \text{ kPa}$

$\sigma_3 = k_a \text{ x } \gamma' \text{ x } (h_2 + H_s) + \sigma_2 = 0.2973 \text{ x } 12 \text{ kN/m}^3 \text{ x } (22 \text{ m}) + 45.49 \text{ kPa} = 123.97 \text{ kPa}$

The pressure resultant is:

$P_a = [\sigma_1 \text{ x } (h_1 + h_2 + H_s)] + [0.5 \text{ x } (\sigma_2 – \sigma_1) \text{ x } h_1] + [(\sigma_2 – \sigma_1)(h_2 + H_s)] + [0.5 \text{ x } (\sigma_3 - \sigma_2) \text{ x } (h_2 + H_s)]$

$= [6.0 \text{ x } 29] + [0.5 \text{ x } (45.5 – 6.0) \text{ x } 7] + [(45.5 – 6.0) \text{ x } (22)] + [0.5 \text{ x } (124.0 – 45.5) \text{ x } (22)]$

$= 2044 \text{ kN/m}$

The resultant acts through the centroid of the pressure diagram occurring at $y_a = 10.65$ m from the base of the structure.

9.1.1.1 Sliding on the Foundation

The resistance to sliding is the weight of material retained by the structure multiplied by the coefficient of friction between sand and rock. The coefficient of friction is the tan of the internal friction angle of the retained material, however this value is conservatively assumed as 0.5. The weight neglects the weight of the structure itself. The driving force is the active pressure P_a generated by the soil.

The $FS_{sliding}$ = Resisting Forces / Driving Forces > 1.5

The resistance to sliding is calculated as:

$RS = W \text{ x } \tan\varphi = w_e \text{ x } [(\gamma_d \text{ x } h_1) + \gamma' \text{ x } (h_2 + H_s)] \text{ x } \tan\varphi$

$= (24.72 \text{ m}) \text{ x } [(19 \text{ kN/m}^3 \text{ x } 7 \text{ m}) + 12 \text{ kN/m}^3 \text{ x } (22 \text{ m})] \text{ x } 0.50 = 4907 \text{ kN/m}$

Therefore, the factor of safety against sliding is 4907 / 2044 = 2.40 > 1.5 (ok).

9.1.1.2 Overturning Stability

The factor of safety against overturning is the ratio of resisting moment to overturning moment.

$FS_{overturning}$ = Resisting Moment / Overturning Moment > 1.5.

The overturning moment is the product of the active resultant force, P_a, and the moment arm, y_a.

$M_{overturning} = P_a \times y_a = 2044$ kN/m x 10.65 m = 21770 kN-m/m

The resisting moment is the product of the weight of retained material and the moment arm, $w_e/2$. Tsinker (1997) proposes a method which neglects a prism of submerged fill within the cell structure. This assumption is conservative in nature and was adopted in this design check.

$M_{resisting} = W' \times w_e/2 = [W - (0.25 \times w_e^2 \times \gamma')] \times w_e/2$

$= [4907 \text{ kN/m} / 2 - (0.25 \times (24.72 \text{ m})^2 \times 12 \text{ kN/m}^3)] \times 24.72 \text{ m} / 2 = 98640$ kN-m/m

Therefore, the factor of safety against overturning is 98640 / 21770 = 4.53 > 1.5 (ok).

Furthermore, although the structure is resting on bedrock and in theory will have infinite bearing capacity, the structure itself cannot take negative pressures at its base. To avoid this situation, the resultant base pressure R, must act through the middle one-third of the base.

R acts at a distance x_1 from the toe of the structure.

$x_1 = \Sigma M - \Sigma W = (M_r - M_o)/W'$ = (98640 kN-m/m – 21770 kN-m/m) / 7981 kN/m = 9.6 m.

Therefore, the eccentricity, $e = w_e/2 - x_1 = 2.73$ m < $w_e/6 = 4.12$ m, so the resultant is in middle one-third of the base and structure experiences only positive bearing pressure.

9.1.1.3 Slipping Between Sheeting and Cell Fill

The structure must also be stable against a situation in which the seaward face of the cell pile structure may lift and the retained material slips from underneath the toe of the structure into the ocean. The friction force created between sheeting and cell fill acts downward to combat this lifting action.

The $FS_{slipping} = w_e \times \tan\delta / y_a$ = 24.72 m x 0.4 x 2 / 10.65 m = 1.86 > 1.5 (ok).

The value, 2, in the above equation, is to account for friction forces acting on both sides of the cell wall since material is retained both within the cells as well as the material retained outside the cell wall comprising the wharf. Tan δ is the coefficient of friction between sand and steel and is taken as 0.4.

9.1.2 Internal Stability at Cell Centerline

The overturning force acting on the structure creates a shearing force, Q, acting along the cell centerline. The internal stability of the cell is achieved when the sum of the shear resistance along the centerline of the cell and the interlock tension is greater than Q.

$FS_{cell\ centerline} = (R_s + T) / Q > 1.5$

Where;

R_s = shear resistance along cell centerline;

T = resistance to shear due to friction in cell interlocks.

Figure 19 shows the forces contributing to the internal stability of the cell centerline.

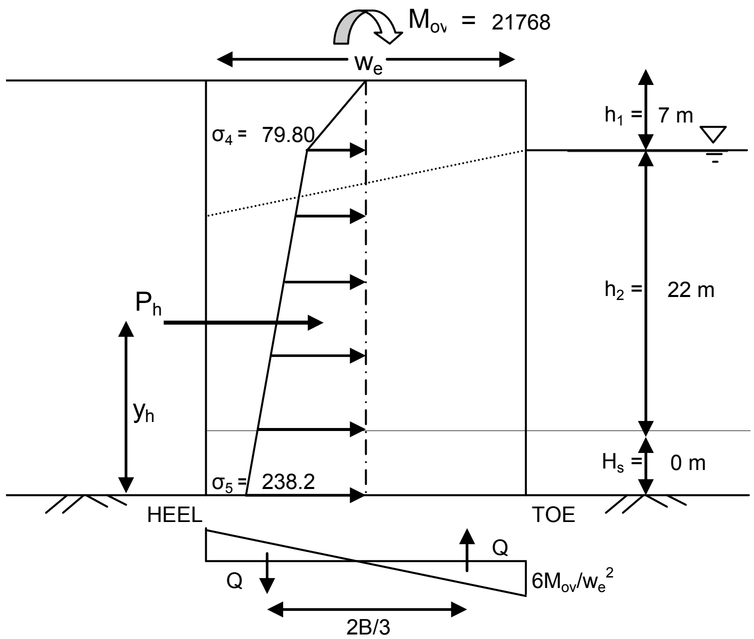

Figure 19 - Internal Stability at Cell Centerline

The total shearing force, Q, is the area of the pressure diagram acting at the base of the structure.

$Q = 0.5 \times (w_e/2) \times (6 \times M_{ov}/w_e^2) = 0.5 \times (24.72\ m\ /2) \times (6 \times 21768\ /\ (24.72\ m)^2) = 1321\ kN/m$

The resisting shear force within the cell, $R_s = P_h \times \tan\varphi$.

P_h is the horizontal soil pressure within the cell. The horizontal soil pressure coefficient, K is:

$$K = \frac{\cos^2\varphi}{2-\cos^2\varphi} = \frac{\cos^2(30^\circ)}{2-\cos^2(30^\circ)} = 0.600$$

The highlighted points on the pressure diagram in Figure 14 are calculated as follows:

$\sigma_4 = k \times (\gamma_d * h_1) = 0.600 \times (19\ kN/m^3 \times 7\ m) = 79.80\ kPa$

$\sigma_5 = k \times \gamma' \times (h_2 + H_s) + \sigma_4 = 0.600 \times 12\ kN/m^3 \times (22\ m) + 79.80\ kPa = 238.20\ kPa$

The pressure resultant is:

$P_h = [0.5 \times \sigma_4 \times h_1] + [\sigma_4 \times (h_2 + H_s)] + [0.5 \times (\sigma_5 - \sigma_4) \times (h_2 + H_s)]$

$= [0.5 \times 79.80 \times 7] + [79.80 \times (22)] + [0.5 \times (238.20 - 79.80) \times (22)]$

$= 3777\ kN/m$

The resultant acts through the centroid of the pressure diagram occurring at $y_h = 10.29$ m from the base of the structure.

The resistance to shear due to friction in the cell interlocks, T, is based on the given formula:

$T = 2 \times T' \times f_i / L$

Where; f_i = coefficient of friction for steel on steel = 0.3.

$T' = P_i \times R = P_a \times D/2$

L = 39.82 m (the length between cell centerlines)

The calculation for T is:
T = (2 x 2044 kN/m x 28.82 m / 2 x 0.3) / 39.82 m = 444 kN/m.
As a result, the $FS_{cell\ centerline}$ = (3777 x tan(30°) + 444) / 1321 = 1.99 > 1.5 (ok).

9.1.3 Bursting Stability Check

Bursting stability checks are largely dependent upon the selection of sheet pile. Therefore the minimum size sheet piling was selected to satisfy our desired factor of safety of 1.5.

Figure 20 shows the pressure diagram contributing the bursting force to the sheet piles. The pressure diagram is the resultant of forces acting within the cell and thus surcharge is neglected.

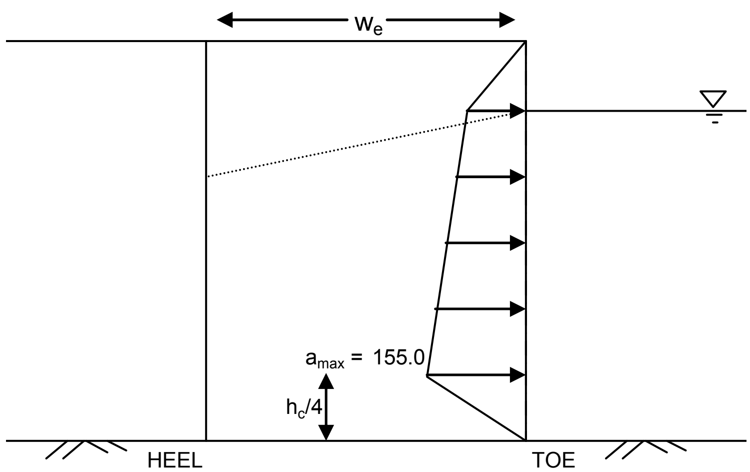

Figure 20 - Sheet Pile Cell Bursting Stability

The maximum pressure for cell bursting occurs at a point one-quarter of the cell height from the bedrock. For the current situation, h_c / 4 = 29 m / 4 = 7.25 m.

The value of maximum pressure, $a_{max} = k_i \times [(\gamma_d \times h_1) + (\gamma' \times (h_c – h_1 - h_c / 4))]$, where k_i is a coefficient of soil lateral pressure taken as 0.5 based on empirical values provided by Tsinker (1997).

a_{max} = 0.5 x [(19 kN/m^3 x 7 m) + (12 kN/m^3 x (29 m – 7 m – 7.25 m))] = 155 kPa.

The tension experienced by the sheet piles due to this pressure is checked in the main cell itself as well as in the connections. Arcelor recommends a safety factor of 1.5 against interlock resistance in the connections, and a value of 1.5 for yielding of the web.

The sheet pile required against web yielding must withstand a maximum tension of,

$T_{max} = a_{max}$ x R = 155 kPa x 28.82 m / 2 = 2234 kN/m.

In addition the sheet pile must withstand a maximum tension of,

$T_{max} = a_{max}$ x 0.5 x L / cosβ =155 kPa x 0.5 x 39.82 m / cos(35°) = 3563 kN/m in the interlocks.

Based on values provided in Arcelor's Piling Handbook, AS 500-12.5, sheet pile cells are required to satisfy bursting conditions. These piles have a strength of 5500 kN/m thereby offering a factor of safety if 2.46 against web yielding and 1.54 against interlock tension.

9.1.4 Horizontal Shear Check

Horizontal shear forces acting on a sheet pile structure can cause the cell to tilt. The check for horizontal shear was completed following the Cummings' method. The method is depicted in Figure 21

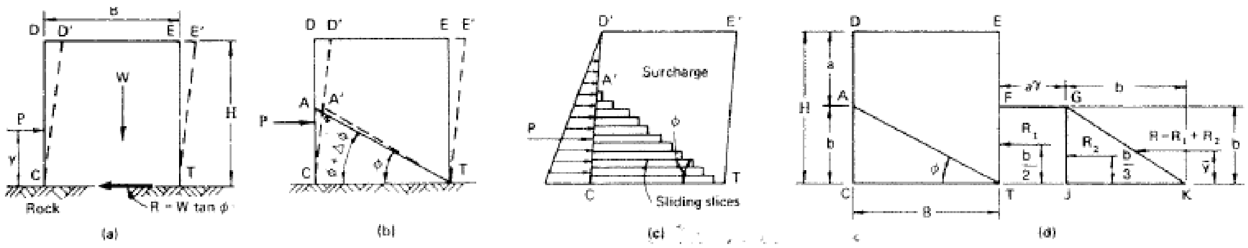

Figure 21 - Horizontal Shear (Cummings') Method

The resistance to horizontal shear is a combination of a resisting moment based on the pressure resultants, R_1 and R_2, plus a moment due to interlock friction.

The resisting moment due to R_1 and R_2 is:

$M_r = R_1 \times b/2 + R_2 \times b/3 = [\gamma' \times a \times b \times b/2] + [\gamma' \times b^2 \times b/3]$

$= [12\ kN/m^3 \times 14.7\ m \times 14.3\ m \times 14.3\ m/2] + [12\ kN/m^3 \times (14.3\ m)^2 \times 14.3\ m / 3]$

= 29628 kN-m/m.

The moment generated by interlock friction is, $M_i = 2 \times P_a \times f \times w_e$.

$M_i = 2 \times 2044\ kN/m \times 0.3 \times 24.72\ m = 30318$ kN-m/m.

The factor of safety against this tilting motion is the ratio of these two resisting moments to the overturning moment acting on the structure.

$FS_{tilt} = (M_r – M_i) / M_{ov} = (29628 + 30318) / (21770) = 2.75 >1.5$ (ok).

9.1.5 Cellular Sheet Pile Design – Summary

The design of the cellular sheet pile option requires nine (9) 28.82 m diameter cells interconnected by eight (8) intermediate arcs along the front and back. The berthing face will be comprised of seven (7) interconnected cells. Based on the properties of the selected cell dimensions the total berthing length of the structure will equal approximately 268 m.

$L_{total\ berthing} = 6 \times L + D = 6 \times 39.82\ m + 28.82\ m = 267.74$ m.

Each pile shall be 29 m in length and the cells are to be constructed form a sheet pile of grade AS 500-12.5 or better.

The total amount of piles required is shown in Table 7.

Pile Type	Length (m)	# Req'd
L	29	1062
M	29	522
S	29	36
N	29	688
Total:		2308

Table 7 - Cellular Sheet Pile Requirements

The typical geometric orientation and arrangement of the sheet pile cells employed at site are shown in Figure 22.

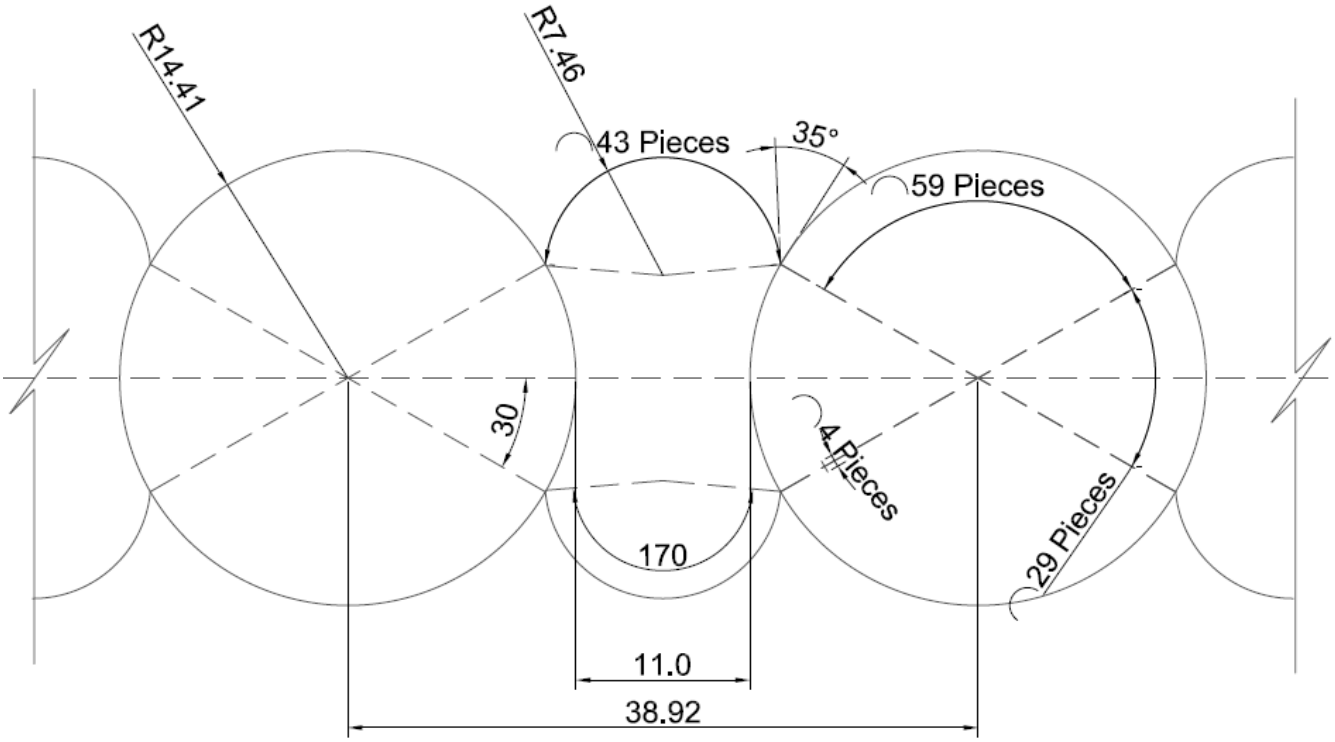

Figure 22 - Typical Sheet Pile Cell Arrangement

A summary of the preliminary design results is shown in Table 8.

Failure Mechanism	Target FS	Actual FS
Sliding	1.50	2.40
Overturning	1.50	4.53
Slipping	1.50	1.86
Internal stability at cell centerline	1.50	1.99
Web Yielding	1.50	2.46
Busting of Interlocks	1.50	1.54
Tilting	1.50	2.75

Table 8 - Cellular Sheet Pile Stability Summary

The preliminary design of circular sheet piles is found in Appendix F.

9.2 CONCRETE CAISSON DESIGN

The design of the concrete caisson option was based on a methodology presented in Handbook of Port and Harbour Engineering: Geotechnical and Structural Aspects. [17]

Although the structure will likely be floated-in at an incomplete phase of its construction, the preliminary design does not account for the stability checks required during the installation phase. The structure is designed based on an in-place stability perspective. The structure is sized to satisfy the failure mechanisms of sliding, overturning, bearing pressure on the structure, and bearing on the rubble mattress.

The caisson structure will be comprised of ten (10) identically sized caisson units, 28 m in length by 21 m in width placed end-to-end comprising a structure face totalling 265 m. There will be eight (8) units along the berth face and two (2) corner units extending down either side of the wharf. There is also a necessity for an additional unit on each wharf end to tie the wharf back to the coastline. This unit will be sized during the detail design process if required. Each caisson will be box type units consisting of individual cells with rock fill.

Figure 23 shows the active pressure exerted on the structure once it is in place.

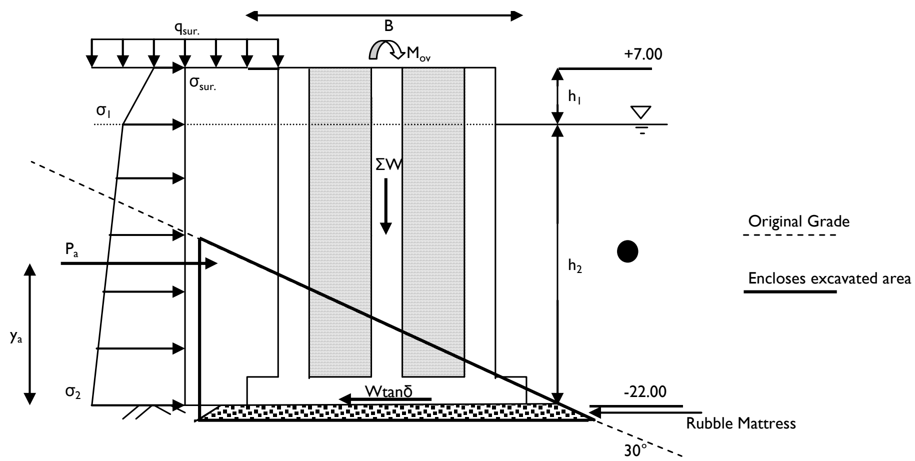

Figure 23 - Option 2 - Concrete Caisson

Multiple arrangements and sizes of units were considered in order to select an option to proceed with for preliminary design and to serve on a comparative basis in a preliminary cost estimate. The selection of these caisson dimensions plus the following calculations are shown in Appendix G.

9.2.1 Sliding Stability

Similar to the cellular sheet pile cell design, the driving force acting on the structure is calculated in the same way.

$\sigma_{sur.}$= 5.95 kPa, σ_1= 45.49 kPa, σ_2 = 123.97 kPa, therefore the pressure resultant P_a = 2044 kN/m acting through a point, y_a = 10.65 m from the base.

The resistance to sliding is a combination of the structure weight and the friction between the caisson unit and the soil surface. A spreadsheet was used to determine the weight of the structure due to the complexity of the calculation; taking into account the volume of interior walls, base slab, and multiple other components of the caisson. The total weight was determined to be 8317 kN/m based on the selected dimensions. The coefficient of friction between the soil and concrete, equal to the tan of the internal friction angle is conservatively assumed to be 0.5. As a result the total resistance to sliding, Wtanδ, is 4158 kN/m.

The $FS_{sliding}$ = Wtanδ / P_a = 2.03 > 1.5 (ok).

9.2.2 Overturning Stability

The overturning moment, M_{ov}, is the product of P_a x y_a = 2044 kN/m x 10.65 m = 21768 kN-m/m.

The resisting moment, M_r, is the weight of the structure W, multiplied by one-half width of the structure, B/2. M_r = 8316 kN/m x 21 m / 2 = 87326 kN-m/m.

Therefore, the $FS_{overturning}$ = M_r / M_{ov} = 4.01 > 1.5 (ok).

9.2.3 Contact Stresses at Base

The structure base cannot take negative pressure so the structure must be sized such that the resultant contact pressure acts through the middle one-third of the base. The desired bearing distribution is shown in Figure 24. An allowable pressure, σ_{allow}, of 1000 kPa is assumed since the structure foundation is bedrock.

The pressure resultant acts at a distance, x_1 = ΣM / ΣW from the toe of the structure.

x_1 = (87326 kN-m/m – 21768 kN-m/m) / 8316 kN/m = 7.88 m.

The resulting eccentricity, e = B/2 – x_1 = 21 m / 2 – 7.88 m = 2.62 m < B/6 = 3.50 m. Therefore, the resultant acts through the middle one-third of the base.

$\sigma_{B\ (toe)}$ = ΣW / B x (1 + 6e/B) = (8316 / 21) x (1 + 6 x 2.62 / 21) = 692 kPa < 1000 kPa.

$\sigma_{B\ (heel)}$ = ΣW / B x (1 - 6e/B) = (8316 / 21) x (1 + 6 x 2.62 / 21) = 100 kPa < 1000 kPa.

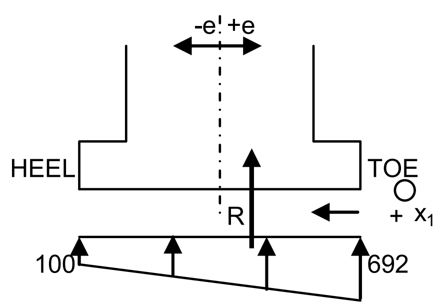

Figure 24 - Contact Pressures at Base

9.2.4 Stresses at Mattress-Soil Interface

Concrete caissons require the construction of a suitable foundation mattress to support the structure, as well as to provide a level seating for the structure. The rubble mattress also has an outer scour protection layer to protect the core of the mattress from being washed away by propeller wash.

The material selected for the rubble mattress at the site is dense gravel which has an allowable pressure of approximately 600 kPa under extreme loading conditions, according to Tsinker (1997). It is assumed that the density of the rubble mattress material is 1950 kg/m^3.

The minimum mattress thickness, h_{min} is governed by the following equation provided by Tsinker (1997).

$$h_{m,\min} = \frac{2\sigma_f - \gamma_r B}{4\gamma_r} - \left[\left(\frac{2\sigma_f - \gamma_r B}{4\gamma_r}\right)^2 - \frac{B(\sigma_{\max} - \sigma_f)}{2\gamma_r}\right]^{0.5}$$

$$= \frac{2\times 600kPa - 9.32kN/m^3 \times 21m}{4\times 9.32kN/m^3} - \left[\left(\frac{2\times 600kPa - 9.32kN/m^3 \times 21m}{4\times 9.32kN/m^3}\right)^2 - \frac{21m(692kPa - 600kPa)}{2\times 9.32kN/m^3}\right]^{0.5}$$

$$= 2.00m$$

A value of 2.50 m was conservatively selected as it is near impossible to achieve a uniform mattress thickness in subsea conditions. Based on this value, the stresses experienced at the heel and toe locations of the mattress can be calculated. The pressure occurring at this mattress-soil interface resembles the distribution shown in Figure 24.

$$\sigma'_{B(toe)} = \sigma_{B(toe)}\left(\frac{B}{B+2h_m}\right) + \gamma_r \times h_m = 692kPa\left(\frac{21m}{21m + 2\times 2.50m}\right) + 9.32kN/m^3 \times 2.50m = 582kPa$$

$$\sigma'_{B(heel)} = \sigma_{B(heel)}\left(\frac{B}{B+2h_m}\right) + \gamma_r \times h_m = 100kPa\left(\frac{21m}{21m + 2\times 2.50m}\right) + 9.32kN/m^3 \times 2.50m = 104kPa$$

9.2.5 Concrete Caisson Design – Summary

The design of the concrete caisson option is comprised of ten (10) individual box type caissons, 28 m x 21 m x 29 m. The interior cells measuring 6.325 m x 6.200 m will be filled with solid ballast with a dry density of 1937 kg/m^3, and a submerged density of 1223 kg/m^3.

The concrete used in the caisson will be of normal density with a density of 2400 kg/m^3, and a minimum crushing strength of 35 MPa.

The total volume of material used in this structural option is shown in Table 9.

Material	Volume (m^3)		Weight (kN)	
	Per unit	Total	Per unit	Total
Concrete	2,076	20,760	48,873	488,730
Ballast	13,412	134,120	183,997	1,839,970

Table 9 - Concrete Caisson Material Summary

A summary of the results yielded from the completed stability analysis is shown in Table 10.

Failure Mechanism	Target	Actual
Sliding	1.50	2.03
Overturning	1.50	4.01
Contact Pressure at Wall Base	1000kPa	692 kPa
Contact Pressure at Soil-Mattress Interface	600kPa	582 kPa

Table 10 - Concrete Caisson Stability Summary

A plan view of a typical caisson unit is shown in Figure 25.

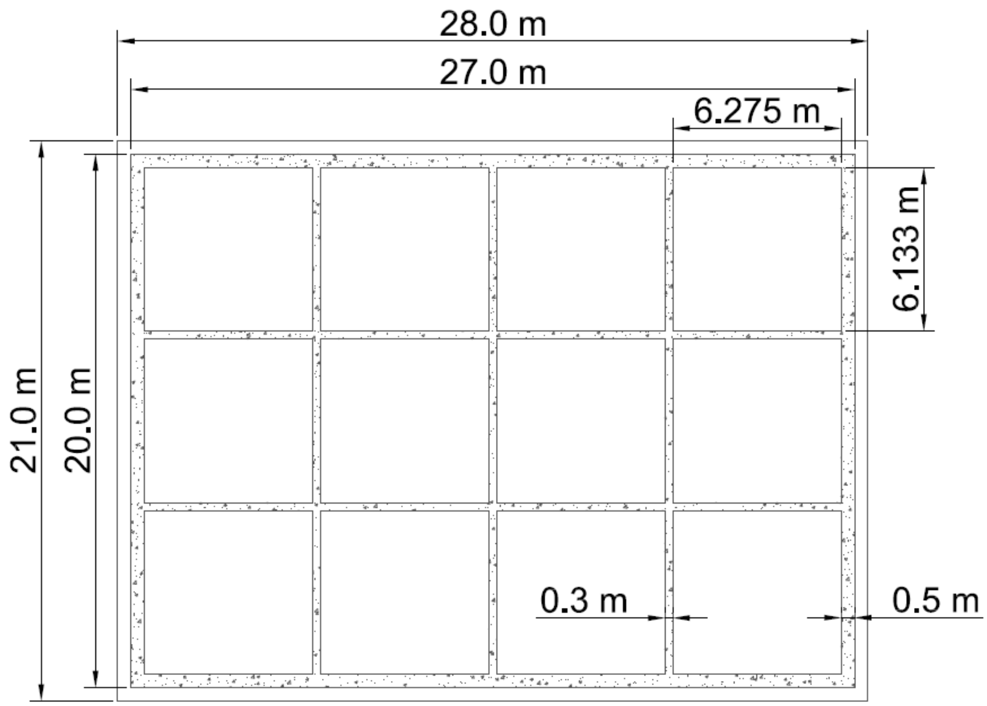

Figure 25 - Plan of Typical Caisson Unit

For sliding and overturning the value represents a factor of safety which we desire to be greater than the target. For contact pressure, the actual pressure must be lower than the target.

9.3 TUBULAR STEEL PILE DESIGN

The design for tubular steel piles was carried out with reference to Pile Design and Construction Practice[20]. This reference was used as a guide for pile sizing and spacing.

A variety of failure mechanisms must be checked in the design of tubular steel pile structures. These include axial capacity, slenderness, bearing, pullout and lateral resistance.

Given the port dimensions a pile spacing of 10 m was selected in both horizontal directions as shown in Figure 26.

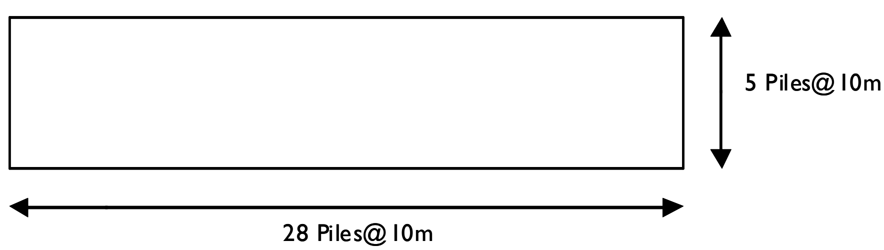

Figure 26 - Pile Spacing

9.3.1 Axial Capacity

Based on existing structures with similar layouts as provided by Tomlinson[20], steel tubular piles 1300mm in diameter and 30 mm thick were selected. The governing load included an operational surcharge of 20 kPa and ship loader self weight of 860 tons. These loads combined to create a maximum axial force of approximately 4,000 kN. The axial resistance of the assumed piles was well above the required value.

9.3.1.1 Slenderness Effects

The longest piles required at the deep end of the pier were 30 m as shown in Figure 30. These were checked for a fixed-“free” condition using Euler’s buckling formula:

$$F_r = \pi^2 EI \big/ (KL)^2 = \pi^2 \times 200000MPa \times 2.415\times10^{10}\, mm^4 \big/ (1.5\times 30000mm)^2 = 24000kN$$

In order to account for the fixed-“free” condition, a value of 1.5 was used for K. The slenderness calculation showed that the selected pile size was adequate for slenderness effects of a 30 m long pile.

9.3.2 Bearing and Pullout Check

Boreholes of 1.5m diameter filled with concrete were selected to anchor the piles. Between the steel pile and exposed bedrock, a concrete annulus was used to provide skin friction resistance for bearing and uplift. A cross section of the pile and borehole connection is shown in Figure 27. A unit skin friction value for bedrock of 1000 kPa was taken from Pile Design and Construction Rules of Thumb (2008). [21]

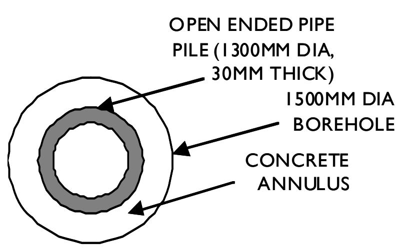

Figure 27 - Pile Borehole Connection

9.3.2.1 Bearing

The borehole depth required for adequate bearing capacity was calculated assuming 90% of the axial load will be resisted by skin friction as shown in Figure 28. The required length was determined using the following equation:

$$L = F_{sk} \big/ \pi D_{borehole} f = 0.9\times 4000kN \big/ \pi \times 1500mm \times 1000kPa = 0.75m$$

A conservative value of 1 m was taken for bearing length.

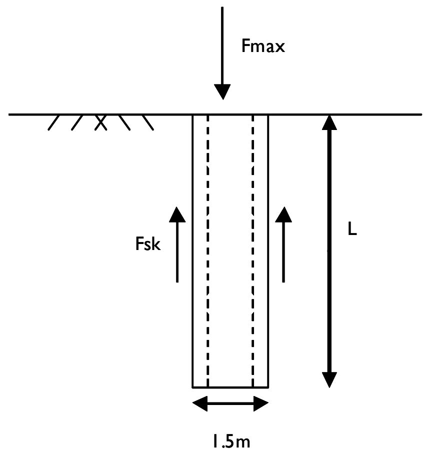

Figure 28 - Pile Bearing Schematic

9.3.2.2 Pullout

The borehole depth required for adequate pullout capacity was calculated assuming all anchorage will be provided by skin friction. Figure 29 shows how the action of pullout is resisted. Based on a rule of thumb, the pullout length was taken as five times the bearing length. Pullout capacity was calculated as approximately 24,000 kN using the following equation:

$$F_{pullout} = \pi DLf = \pi \times 1.5m \times 5m \times 1000kPa = 23562kN$$

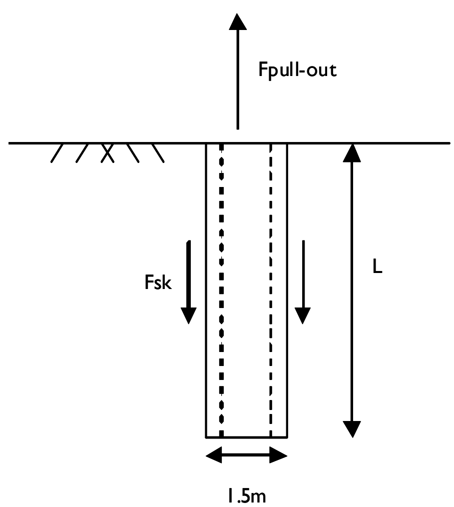

Figure 29 - Pullout Resistance Schematic

9.3.3 Lateral Resistance

The major lateral force to be resisted by the piles will be due to a berthing energy of 302 ton-m as calculated in Section 6.1. A fender deflection was determined based on a SCK 2500 fender with a rated deflection of 52.5%. This amounted to a displacement of 1.3 m:

Displacement, δ = 0.525 x 2500 mm =1.3 m

Thus the total berthing force, F, was calculated to be 232 ton or 2070kN as shown in the following equation:

Force, F = Energy,E / Displacement, δ = 302 ton-m / 1.3m= 232 ton = 2070kN.

The berthing force was applied to the pier as shown in Figure 30.

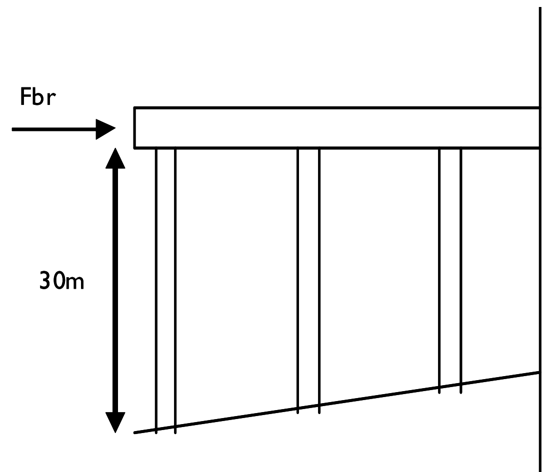

Figure 30 - Lateral Forces on Pier

The resulting bending moment was found to exceed the elastic bending capacity of a pile. Therefore batter piles were added along the front row of piles to provide lateral support and reduce bending effects. Batter piles are shown in Figure 31.

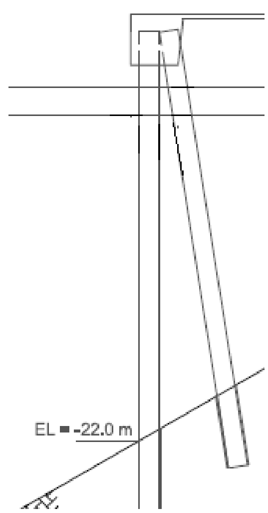

Figure 31 - Batter Piles

9.3.4 Tubular Steel Pile Design – Summary

The design requires 140 vertical piles arranged in a 10x10 grid and 56 batter piles arranged along the front row. The pile lengths required range from 12 m to 35 m. See

Figure 32 for a plan view of the pile arrangement. The total amount of piles required is shown in Table 11.

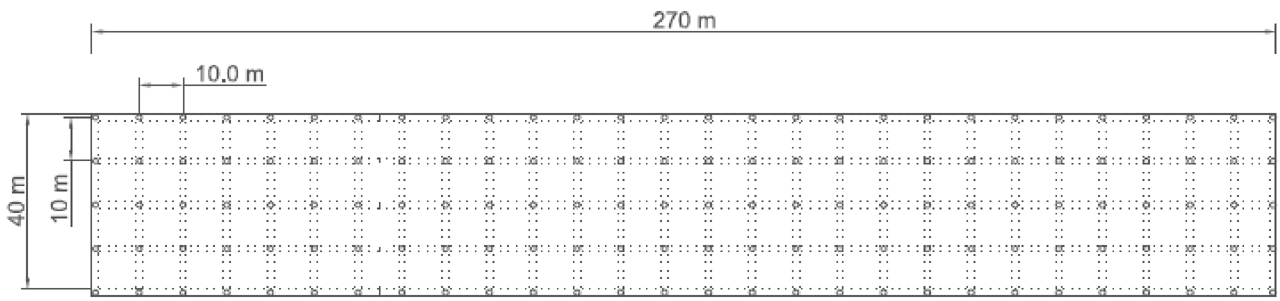

Figure 32 - Plan View of Pile Arrangement

Pile Type	Length (m)	# Req'd
Vertical:	35	28
	30	28
	24	28
	18	28
	12	28
Batter:	30	56
	Total:	196

Table 11 – Tubular Steel Pile Requirements

The preliminary design of tubular steel piles is found in Appendix H.

10 PRELIMINARY COST ESTIMATES

A comparison of the preliminary cost estimate for each alternative is shown in Table 12. This table summarizes the major elements of each structure. A more detailed breakdown of each estimate is provided in Appendix I: Preliminary Cost Estimates.

The preliminary cost estimate was performed using a unit rate analysis based on data provided by both the client and in consultation with Steve Bruneau, PhD, P. Eng.

Due to member experience, availability of resources, and time constraints the preliminary cost estimate for both gravity options were completed with a higher level of confidence than the steel pipe pile option. It is recommended that a steel pipe pile option be given further consideration in an attempt to refine its preliminary estimate to a level of confidence equivalent to that of both the sheet pile option and the concrete caisson option.

ITEM	ESTIMATED COST
Circular Sheet Pile Cells	**$37,409,000**
Civil Works	$15,409,000
Sheet Pile Structure	$22,000,000
Concrete Caisson	**$44,878,000**
Civil Works	$10,418,000
Concrete Caisson Structure	$34,460,000
Steel Pipe Pile Option	**$48,808,500**
Civil Works	$612,500
Supply and Installation of Pipe Piles	$39,946,000
Concrete and Associated Works	$8,250,000

Table 12 - Preliminary Cost Estimate Summary

In reviewing the preliminary estimates above, it is important to note that the estimates are comparative only and do not include all cost components associated with the work. The following items are not included as they are considered to be the same for both options:

- General conditions including management costs, establishment of site office, etc.
- Mechanical and electrical services associated with the ship loader.

Based on the preliminary cost analysis it is recommended that the circular sheet pile cell option be implemented at site as it is the cheapest option in addition to receiving the highest performance score in our pre-assessment ranking.

11 CASE HISTORIES

There are several examples of cellular sheet pile cells that have been constructed in cold regions. The conditions of these areas are similar to that of the location of this project. The following two case studies strengthen the judgment of recommending cellular sheet pile cells for this project.

11.1 WHARF AT NANISIVIK, BAFFIN ISLAND

In Nanisivik, Baffin Island a wharf was constructed using cellular sheet pile cells to accommodate a 50 000 DWT bulk carrier. Arctic conditions existed at this site with a mean temperature of -14 °C and sea ice thickness up to 2 m. The ice loads on the structure reached 27 kPa at high water level. Three (3) cells of 21.3 m diameter where aligned in a straight line, spaced 38.1 m center-to-center to form a berthing face of approximately 100m. The cells where filled with coarse granular material and topped with a reinforced concrete slab 460mm thick. The fenders used were rubber tires suspended from a guard rail. During the ice season between September and July, the ice was used as a construction platform to assist in the driving of individual sheet piles.[11]

11.2 VOISEY'S BAY MARINE TERMINAL, LABRADOR

A marine terminal was designed for Voisey's Bay Nickel Company in order to import mine consumables and export nickel concentrate. A detailed design and analysis report for the project completed by Westmar Consultants is provided in Appendix J.

The port location was in a remote area on the north-east coast of Labrador in the Canadian Arctic. The berth face was approximately 100m consisting of four cells and six connecting arcs. The cells used were 24.7 m in diameter spaced at 27.2 m center-to-center. Due to extremely hard soil conditions, AS 500 sheet piles were selected. These piles do not require embedment into lower soil. A plan view of the cell arrangement constructed at site is shown in Figure 33. [22]

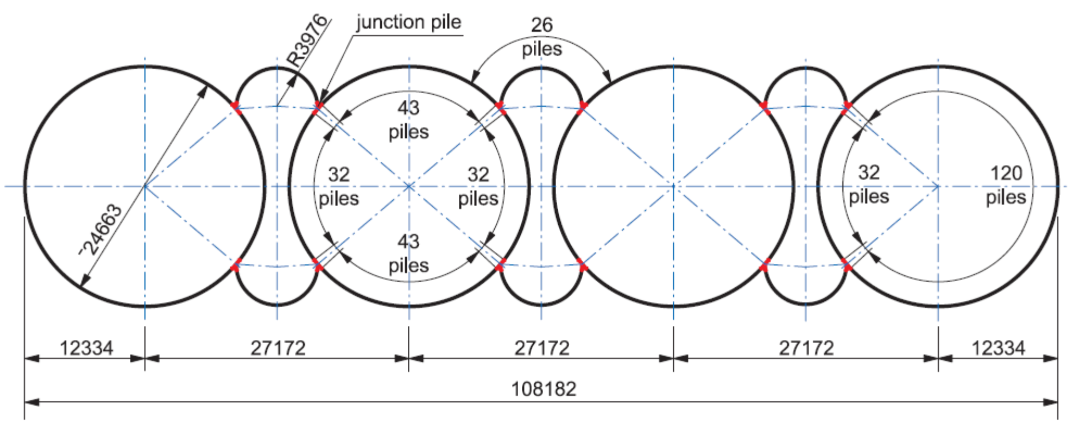

Figure 33 - Plan View of Cell Arrangement at Voisey's Bay Marine Terminal

Ice conditions in the region were due to freeze-up (fast ice) during the winter months. The maximum average thickness for the region was 1.2 m. In order to determine ice loading, limit momentum loads and limit force load conditions were considered. Limit momentum loading considers energy transfer during ice impact where limit force loading includes forces created by driving forces behind the ice feature. A global ice pressure of 500 kPa was used in the design of the structure. In order to supplement the strength of the sheet piles, pre-cast concrete ice impact panels were used. These panels were installed directly behind the sheet piles to provide increased resistance.[23] Further details on the construction of the structure are presented in a report in Appendix J.

12 ICE FORCES

For ice-structure interaction, two types of limiting environmental forces were considered as provided by Cammaert and Muggeridge in Ice Interaction With Offshore Structures. These loads include limit momentum loads and limit force loads. [24]

12.1 LIMIT MOMENTUM LOADS

The limit momentum load is the force required to bring an ice feature to rest after it impacts a structure as shown in Figure 34. As the ice feature slows down momentum is absorbed by the structure.

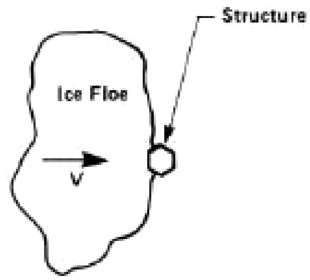

Figure 34 - Limit Momentum Load

The amount of load is the product of the maximum width of contact, local ice thickness and effective ice pressure. An analytical model for calculating the maximum impact force for static analysis has been proposed by Cammaert and Tsinker (1981):

$$F_{1m} = 1.82h(p_e L V_i)^{0.67}([1+C_m]R_s \rho_i)^{0.33}$$

Where;

$h =$ Ice thickness (m)

$p_e =$ Effective crushing pressure (Pa)

$L =$ Length of ice (m)

$V_i =$ Ice speed (m/s)

$C_m =$ Added mass factor

A value for effective crushing pressure was assumed as 2.0 MPa. The ice speed was conservatively takes as the 5 m/s which is less than the wave velocity experienced at site. The limit momentum load calculated from the above equation was 40.8 MN. This equates to an ice pressure value of 700 kPa by dividing the load over the contact area. The contact was assumed to be the product of the ice thickness and the length of impact. The length of impact was taken as the system length of the cellular sheet pile arrangement which is 40 m. Further details regarding the calculation of ice loads is provided in Appendix K.

12.2 LIMIT FORCE LOADS

The limit force load is caused by ridge-building pressures exerted by pack ice on the ice feature as shown in Figure 35.

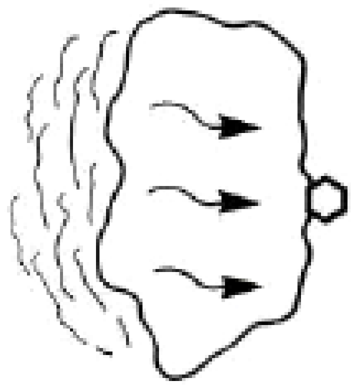

Figure 35 - Limit Force Load

The load is determined from the combined effects of pressures exerted on the ice feature in contact with the structure and drag forces caused by current and wind. A formula for this loading condition has been proposed by Croasdale (1980):

$$F_{2m} = (C_a \rho_a V_a^2 A) + (0.5 C_w \rho_w V_w^2 A) + (wL)$$

Where;

$A =$ Floe area (m^2)

$\rho_a =$ Mass density of air (kg/m^3)

$V_a =$ Air velocity (m/s)

$C_a =$ Drag coefficient - air

$C_w =$ Drag coefficient - water

$\rho_w =$ Mass density of water (kg/m^3)

$V_w =$ Current velocity (m/s)

$w =$ Average pack ice pressure (kN/m)

$L =$ Width of flow (m)

An average pack ice pressure of 50 kN/m was assumed based on an energy model.

The limit force load calculated was 2,000 kN which is equivalent to an ice pressure of 33 kPa. Further detail on this calculation is provided in Appendix K.

The governing load condition for design is the limit momentum load. Expressed in terms of ice crushing stress on the structure, this load has a value of 700 kPa. In the design of the Voisey's Bay marine terminal, a global ice pressure of 500 kPa was used for design. Given the higher average ice thickness expected to occur at the St. Lawrence Marine Terminal site, the value of 700 kPa relative to 500 kPa was deemed appropriate.

This additional ice pressure necessitated the design of reinforced concrete ice strengthening panels which are to be installed directly behind the sheet piles.

13 DETAILED DESIGN

The detailed design of the sheet pile structure required design and analysis of additional components. These components include:

- Ice strengthening panels which provide extra resistance to ice loading;
- Slab-on-grade of the marine terminal deck;
- Foundation design for support of ship loader;
- Concrete cope wall which provides a means of supporting the fender units and bollards;
- Fender design including the selection of fender type, sizing of the fender panel, and selection of chains;
- Selection of bollards;
- Design of a cathodic protection corrosion system.

13.1 ICE STRENGTHENING PANELS

The sheet pile interlocks that form the cellular structure are highly susceptible to ice-loading damage. Ice loading is transferred to the face of the sheet piles and then transferred to the cell fill. Through the transfer of these forces the interlocks may lose strength or fail. For the Voisey's Bay wharf, reinforced concrete panels were used to withstand the ice impact forces. A similar system of reinforced concrete panels has been designed for this project. [23]

13.1.1 DESIGN DETAILS

The governing ice load was found to be 700 kPa as calculated in Section 12. Given the tidal range of 2 m, a 1.5 m thick ice feature acts over a range of 3.5 m. The design was based on 3 m wide pre-cast panels. This panel width allowed an even distribution along the exposed arc length as illustrated in Figure 36.

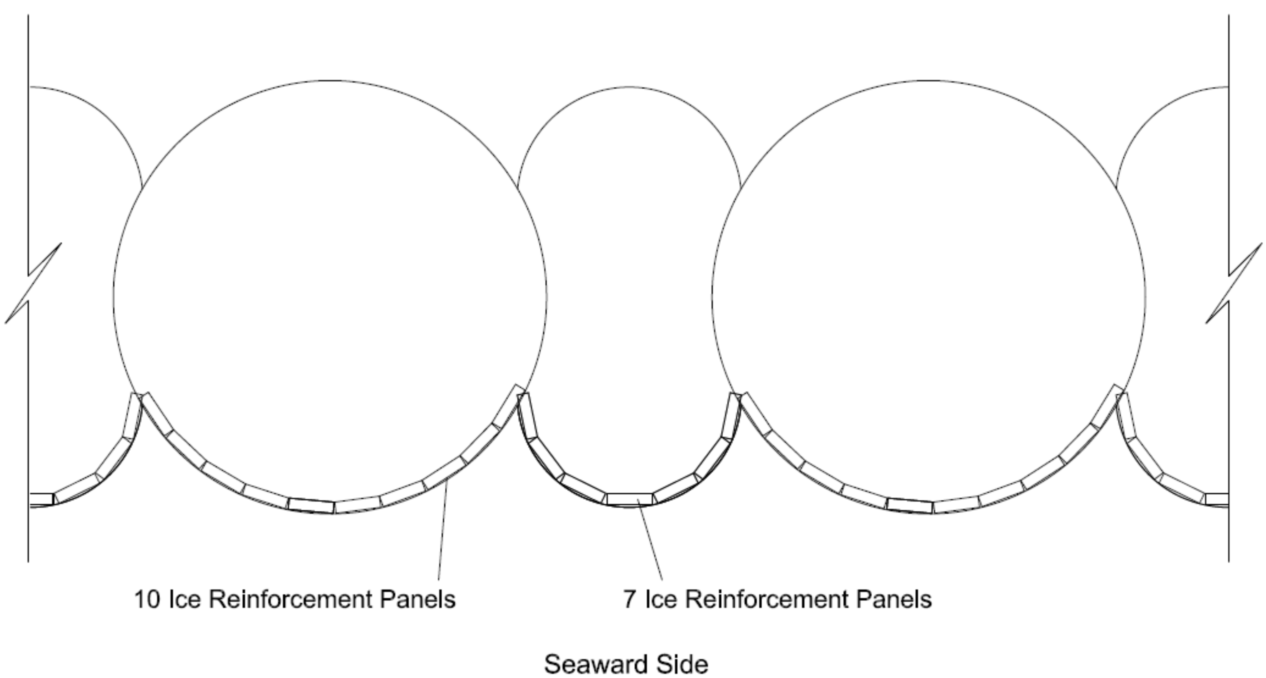

Figure 36 - Ice Panel Layout

An interference check showed that a 3 m panel width permitted the idealization of the panels as plain rectangular sections. Therefore the panels were designed to resist the load over a 3 m x 3.5 m contact area.

The panels were treated as vertical footings subject to surcharge loading for the design of panel reinforcement. Bending moments were calculated and adequate reinforcement was provided as governed by the Canadian Concrete Standard CSA-A23.3-04.[25] The details of the reinforcement were determined with the aid of the Voisey's Bay design report. [23]

In addition to the main tension reinforcement, hooks and dowels were included in the panel detailing. The calculations for the design of the reinforced ice strengthening panels are contained in Appendix L.

13.2 SLAB-ON-GRADE

Following the guidance of the client, the wharf deck was designed to withstand the maximum highway loading of a recognized standard. The deck of the marine terminal was designed as a slab-on-grade using a publication from the Portland Cement Association (PCA), Concrete Floors on Ground.[26] Based on guidelines presented by PCA the required slab-on-grade thickness was calculated to be 300 mm.

This slab was designed to withstand a transport truck load of 80 kips which is the maximum load allowed under US regulations for most interstates. A design check was also done to ensure the slab could withstand an assumed maximum design distributed load of 134,400 lbs which is equivalent to the weight of two 40 ft dry freight containers stacked together.

The slab was designed with a compressive strength of 35 MPa and a flexural strength of about 4.3 MPa. The backfill was assumed to have a sub-grade modulus of 27.1 MPa/m.

Reinforcement of 3-20M bars per meter was specified to avoid cracking of the slab due to temperature and shrinkage effects.

See Appendix M for calculations.

13.3 SHIP LOADER SUPPORT

The ship-loader weight of 860 tons was distributed over four (4) 8 m wheel sets to give a uniform live load of 240 kN/m per wheel set. This distributed load was increased by 10% to account for additional operating loads imposed during ship loading.

The supporting rails were selected to be heavy duty 171-CR rails with a width of 150 mm as shown in Figure 37. These type of rails are commonly used for marine ship loaders.[5]

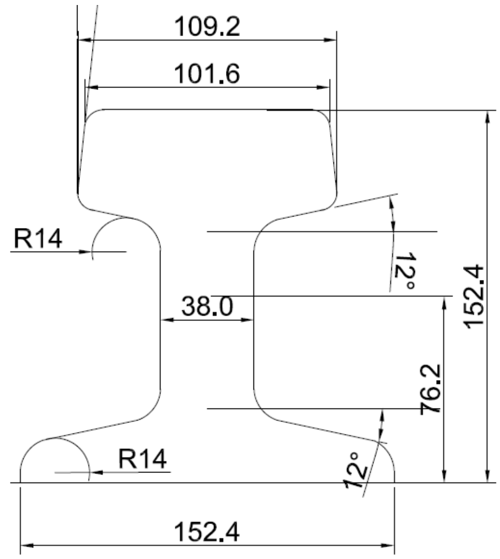

Figure 37 - Crane Rail Detail

The foundation for the rails was modeled as a wall foundation. Based on CSA A23.3-04 a section 300 mm thick and 1500 mm wide with 3-20M bars in each direction was designed. Figure 38 shows the foundation support section, its connection to the overlying slab-on-grade, and the embedded crane rail.

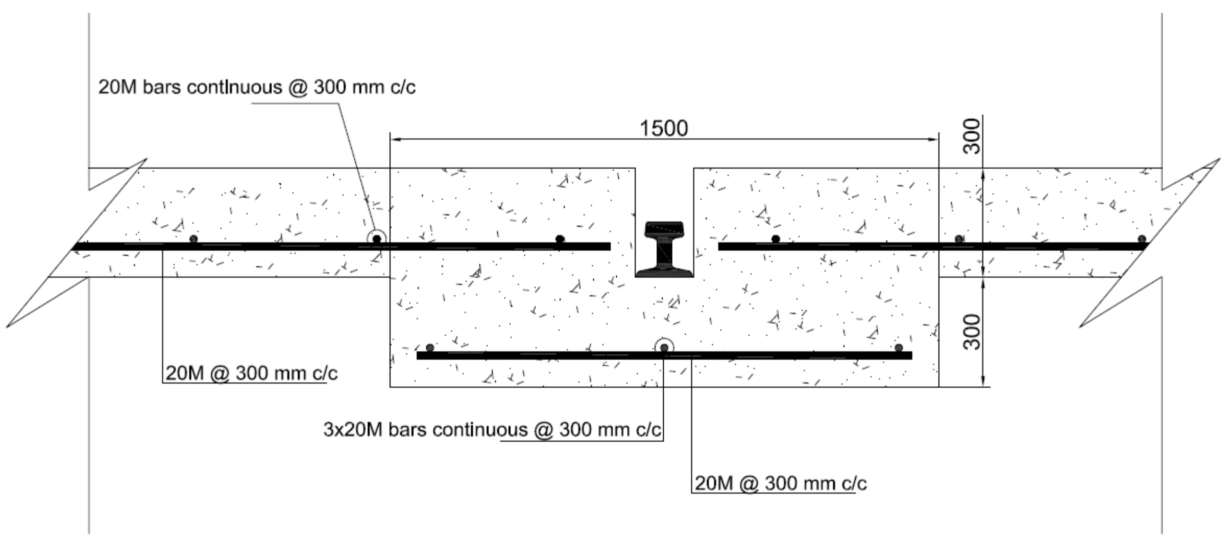

Figure 38 - Crane Rail Foundation Detail

Appendix N contains detailed design calculations for the ship loader support.

13.4 COPE WALL

For design idealization, the cope wall was assumed to act as a cantilevered beam subject to two (2) load cases. The first loading case included the self weight of the fender and panel with no berthing force present. The second loading case was conservatively assumed as the force of berthing against the fender only, neglecting the self-weight of the panel.

The design process yielded a cope wall section with a height and width of 3.5 m and 1.0 m respectively.

Maximum moments were calculated for the two loading cases mentioned above. The maximum moment for case 1 was 390 kN-m and the maximum moment for case 2 was 3500 kN-m in the opposite sense.

Appropriate checks were made for minimum reinforcement, rebar spacing, shear failure, and development as per CSA A23.3-04. [25] See Appendix O for design calculations.

13.5 FENDERS

Fenders are installed to prevent direct contact between quay and vessel during berthing and while the vessel is moored alongside berth. Fenders protect the structure by absorbing berthing impact energy. [4]

Fender design for bulk carriers must account for the following: [5]

- The fender must permit close berthing of the vessel so ship loader outreach is not exceeded;
- Large change of draught between laden and empty conditions;
- Require low contact pressures unless belted.

The normal berthing energy calculated in Section 6.1 was equal to 302 ton-m. This value is used for the design and selection of fenders and their components. Design was based on guidelines presented in the Trelleborg Marine Systems Safe Berthing and Mooring Catalogue[27]. Excerpts from the catalogue and design calculations are found in Appendix P.

The fender is designed for the abnormal berthing energy, E_A. Abnormal impacts arise when the normal energy is exceeded. This may result due to human error, malfunctions, exceptional weather conditions, or a combination of these factors. [27]

The abnormal energy to be absorbed by the fender can be calculated as:

$$E_A = FS \times E_N$$

Trelleborg recommends a factor of safety, FS of 1.25 for large dry bulk carriers based on values provided by Pianc (2002). The selected fender must be capable of absorbing an energy of 377.5 ton-m, which is equivalent to 3367 kN-m.

Super Cone (SCN) fenders were selected on the basis that they are Trelleborg's most optimal and efficient design. The conical body shape of the SCN fender makes them very stable at large compression angles, and provides excellent shear strength. They also have an installed overload stop which gives them an added resistance to over compression. [27]

SCN 1800 E3.0 fenders were selected based on their efficiency to absorb the impact from berthing and to support the static weight of the fender panel. The properties and geometry of the SCN 1800 E3.0 fenders are shown in Table 13. A schematic of the fender is shown in Figure 39.

	E (kN-m)	R (kN)	Eff. (E/R) (kN-m/kN)	Weight (kg)
SCN 1800 E3.0	3530	3775	0.932	6618

Table 13 - Fender Properties

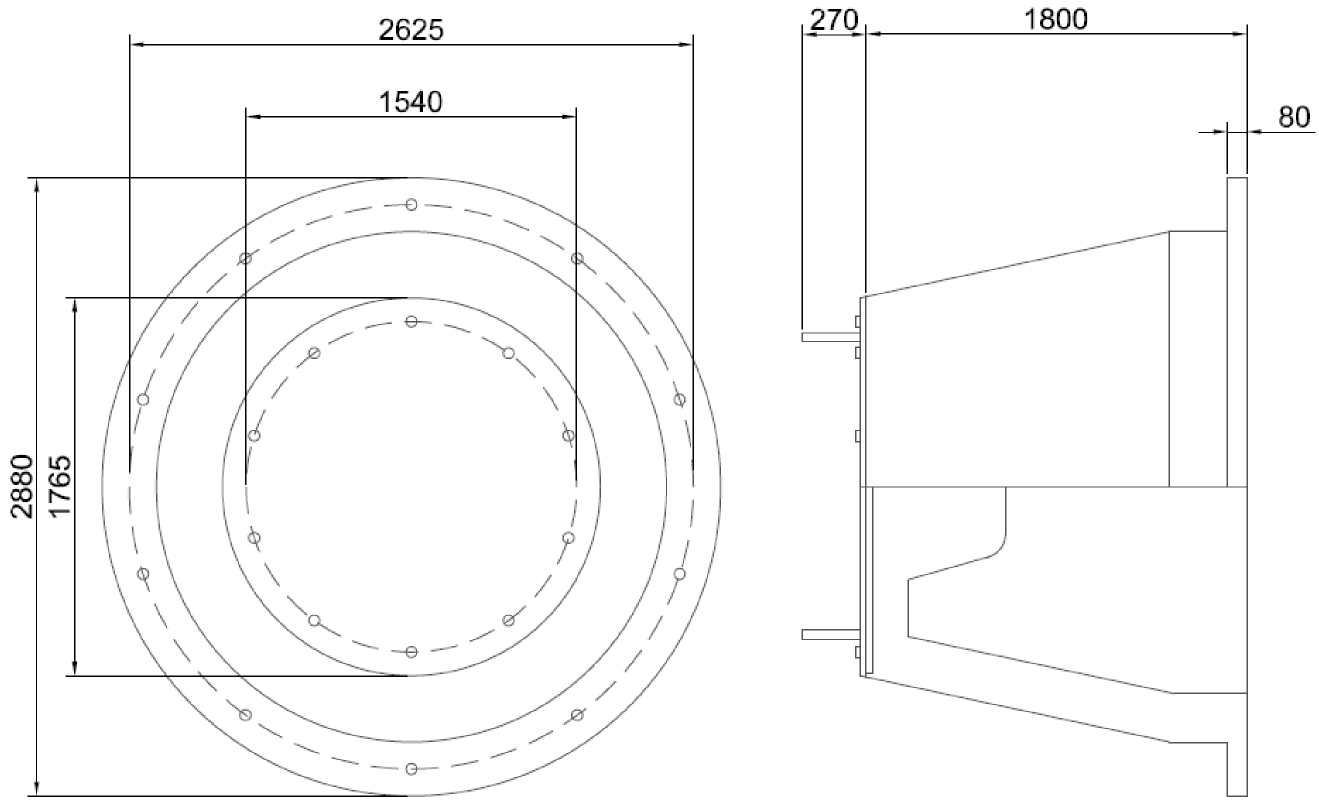

Figure 39 - SCN 1800 E3.0 Detail

13.5.1 Fender Panel

The main function of the front panel is to distribute the reaction forces from the fender units into the ship's hull. The loads and stress loads exerted on the front panel will depend on the type of ship, berthing mode, characteristic of the rubber fender, and tidal range. Fender panels for dry bulk carriers are designed to reduce the hull pressure to a value less than 200 kN/m^2.

$$A_{req'd} = R/P = \frac{E/eff.}{P} = \frac{3367kNm / 0.932kNm/kN}{200kN/m^2} = 3613kN / 200kN/m^2 = 18.06m^2$$

Fender panels, 4.5 m squared were selected. This provides a panel area of 20.25 m^2 which reduces the pressure on the hull to 178 kN/m^2.

It was assumed the selected panel is heavy duty which typically weighs 350 kg/m^2. The fender is exposed on one face which requires a steel thickness of approximately 10 mm.

$$Weight_{panel} = [350 + 0.01m(7850)]kg/m^2 \times 20.25m^2 = 8677kg$$

SCN 1800 E3.0 fenders can support a static weight (weight of fender panel) of:

$$W_{static} = n \times 1.5 \times W_{fender} = 1 \times 1.5 \times 6618kg = 9927kg \geq 8677kg \quad \therefore OK$$

UHMW-PE fender panels have been recommended because of their robustness in extreme climates. UHMW-PE panels are of a polyethylene material. It is a low friction facing material which helps reduce friction and supporting chain requirements.[27]

Corrosion prevention for fender panels will consist of specialized paint coatings which will need periodic applications over the service life of the terminal. Paint coatings will comply with ISO EN 12944, a widely used international standard defining the durability of corrosion protection systems in various environments. The C5-M class paint applies to marine coastal, offshore and high salinity locations and is considered the most applicable to fenders. [27]

Figure 40 shows a typical fender and panel arrangement.

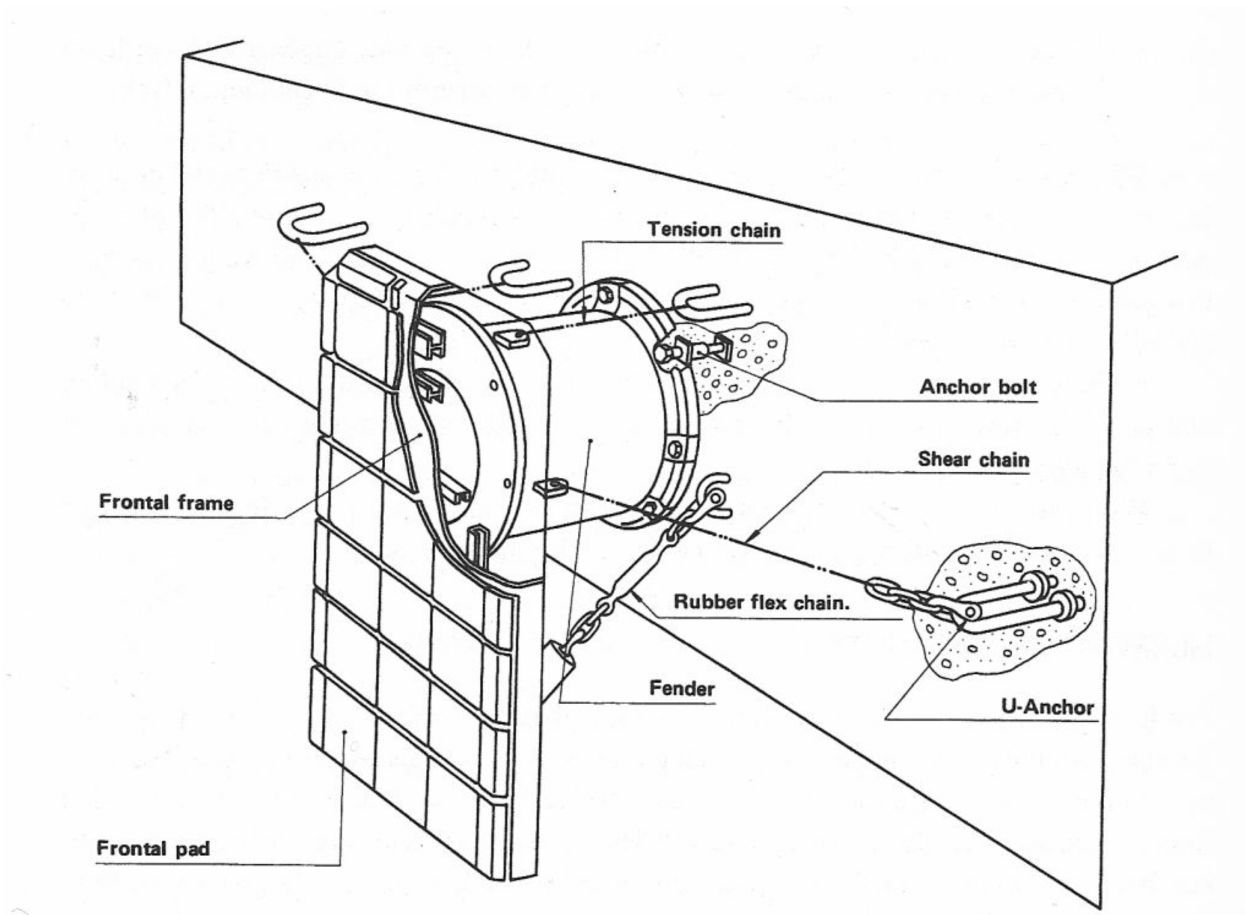

Figure 40 - Typical Fender and Panel Arrangement

13.5.2 Fender Pitch

Fenders are spaced accordingly based on the largest vessel anticipated to berth at site such that the vessel will not come in contact with the wharf as shown in Figure 41.

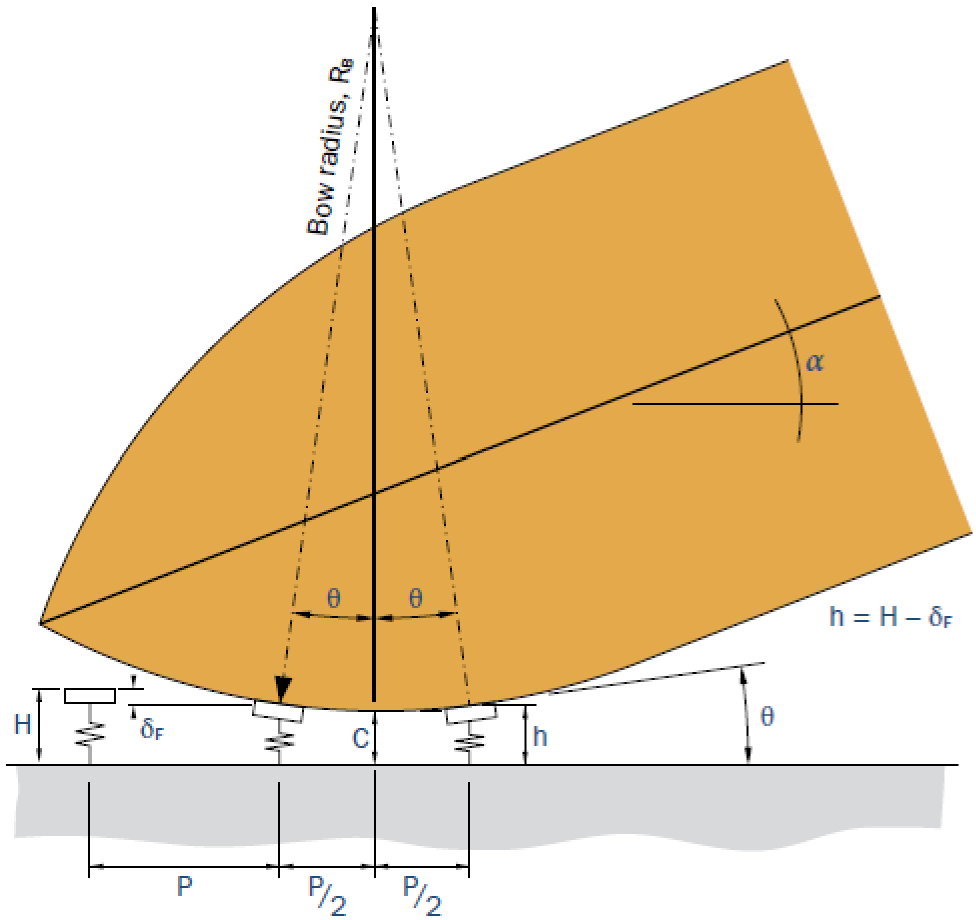

Figure 41 - Fender Pitch Considerations

The maximum pitch, P, between suitable fenders on a continuous wharf is governed by the formula below:

$$P \le 2\sqrt{R_B^2 - (R_B - h + C)^2}$$

The terms in the above equation are:

$$R_B = bow\ radius \approx \frac{1}{2}\left[\frac{B}{2} + \frac{LOA^2}{8B}\right] = \frac{1}{2}\left[\frac{50m}{2} + \frac{(310m)^2}{(8 \times 50m)}\right] = 132.625m$$

$$h = \ fender\ \ projection\ when\ compressed$$

$$= H - \delta_f \ = 1800mm - 0.72 \times 1800mm = 504mm$$

$$C = clearence\, between\ vessel\ and\ dock$$

$$\approx 0.15H \ge 300mm = 0.15 \times 1800mm = 270mm \therefore C = 300mm$$

Based on the preceding calculations the minimum fender pitch required for the largest anticipated vessel is:

$$P \le 2\sqrt{(132.65m)^2 - (132.65m - 0.504m + .300m)^2} \le 14.7m$$

BS6349: Part 4:1994 also recommends that fender spacing does not exceed $0.15L_s$, where L_s is the length of the smallest ship. Employing a mean statistical analysis similar to the method used for the selection of the design vessel, the smallest expected vessel is 144 m in length. This requires a pitch, P ≤ 0.15 x 144 m = 21.6 m.[10]

Twenty-one (20) fenders will be installed along the 267.74 m berth face at a center-to-center spacing of 13.387 m.

13.5.3 Chain Design

Chains are used to restrain the movements of fenders during compression and to support static loads. Three (3) types of chains used in fender design are:

Tension Chain: protects the fender from damage while under compression.
Weight Chain: is used to support the weight of the frontal and face panel.
Shear Chain: protects the fender from damage while in shear deflection.

It was assumed that two (2) chains will be used in each mechanism for a total of six (6) chains. The chains should have minimum breaking loads in each mechanism as shown in Table 14.

Type of Chain	Minimum Breaking Load, F_M (kN)
Tension	104
Weight	11
Shear	92.5

Table 14 - Fender Chain Requirements

Corrosion prevention for chains and bolts will use hot dip galvanizing with a thickness of 85 μm. Hot dip galvanizing coats steel parts in zinc, and when exposed to sea water the zinc acts as an anodic reservoir protecting the steel underneath. Galvanizing is a finite application which necessitates re-application during the serviceable life of the terminal. [27]

13.6 BOLLARDS

A typical alongside berthing and mooring arrangement is shown in Figure 42.

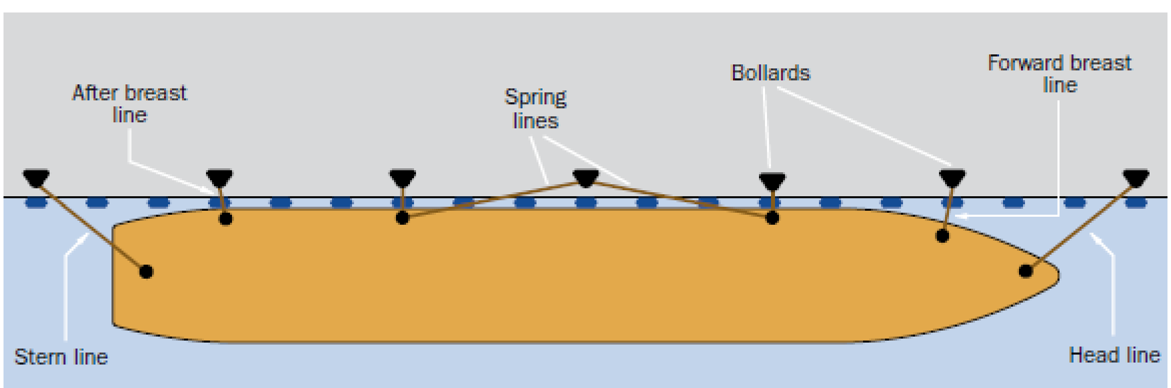

Figure 42 - Alongside Berthing Arrangement

The vessel is held securely in position during berthing by head and stern lines, spring lines, and breasting lines which are secured to the bollards. Trelleborg recommends that for vessels with greater than 200,000 tonnes displacement, a 200 tonne bollard is required. [27] Tee bollards are recommended since they are capable of 200 tonnes and permit the steepest vertical angle of the rope. The largest vertical angle occurs for light draught vessel berthed at high tide conditions. This scenario results in the vessel deck at an elevation 15 m above the wharf deck elevation. Based on the bollard arrangement this relates to a maximum vertical angle of approximately 75 °, but it is recommended that this scenario be avoided.

A sketch of the selected tee bollard is shown in Figure 43. Given the high loads the bollards have been recessed which helps prevent the bollard working loose on its bolts or cracking the grout bed. [27]

The bollards are installed on the underlying concrete cope wall by use of 1 m long embedded anchor bolts. The bolts are not to be fully tightened until the grout has reached full strength. A total of eight (8) bollards equally spaced along the top of the cope wall are to be provided.

Ductile cast iron (spheroidal graphite) has been recommended as opposed to grey cast iron or cast steel bollards due to its many benefits.

These benefits include:

- Lowest service life cost;
- High strength;
- Good impact resistance;
- High corrosion resistance.

The bollards shall be produced to the following material specifications as shown in Table 15. International standards are to be held in compliance where possible.

Material	Standard	Grade
Ductile Cast Iron	ASTM A 536	80-55-6
Anchor bolts (galvanized)	ISO 898	Gr 8.8 (galvanized)
Blasting (high performance)	ISO 12944	SA2.5
Paint (high performance)	ISO 12944	Class C5M

Table 15 - Bollard Material Specifications

13.7 CORROSION PROTECTION

Deterioration caused by corrosion is a factor in determining service life and as a result corrosion potential must be evaluated. In marine environments there are two general types of corrosion; uniform corrosion and pitting attack. Uniform corrosion is a general roughing of the metal surface and frequently occurs in low resistivity natural waters. Pitting corrosion is a more localized attack and often occurs in saline waters.

For the case of this particular project design was taken to protect against pitting. A galvanic anode system was selected because it is relatively easy to install and maintain and it requires no external source of power.

With the aid of Handbook of Corrosion Protection for Steel Pile Structures in Marine Environments[28] an anode system was designed using 1600 aluminum anodes (4” x 4” x 15”) to provide protection for an estimated design life of 15 years. See Appendix Q for design calculations.

14 DETAILED COST ESTIMATE

The detailed cost estimate was performed by building on the items included in the preliminary cost estimate. Quantity take offs were performed during detailed design and costs were obtained using unit rate data obtained from the client and RS Means data. A summary of the detailed cost estimate is shown below in Table 16. The total cost was estimated to be $40,281,160. A thorough breakdown of the detailed cost estimate is provided in Appendix R.

ITEM	ESTIMATED COST
Civil Works	$15,409,000
Sheet Pile Cells	$22,000,000
Ice Strengthening Panels	$976,590
Crane Support	$41,570
Slab on Grade	$696,850
Corrosion Protection	$458,500
Fenders	$100,000
Mooring Devices	$9,600
TOTAL	**$40,281,160**

Table 16 - Detailed Cost Summary

15 DESIGN DRAWINGS

All design drawings are found in Appendix S of the report.

16 ACKNOWLEDGEMENTS

Paramount Engineering would like to acknowledge the following individuals for their support and input throughout the design course:

- Steve Bruneau, Ph.D, P.Eng. (Course Instructor)
- Ray Bailey, P.Eng. (Client)
- Nick Gillis, P.Eng. (Client)
- Amgad Hussein, Ph.D, P.Eng. (Faculty Professor)

17 CONCLUSIONS & RECOMMENDATIONS

The recommended structural option to be constructed for the St. Lawrence Marine Terminal is a circular sheet pile cell structure. The sheet pile structure acts as a cofferdam and is constructed form AS 500 12.0 sheet piles. The individual sheet piles are to be driven in a circular fashion to form 28.82 m diameter cells that are interconnected by arcs on both the exterior and interior face of the cells.

The structure is comprised of nine (9) cells with sixteen (16) connecting arcs (8 along front, 8 along back). The total berthing length of the wharf is 267.74 m. The berthing face of the wharf is located approximately 40 m from shore. The deck is elevated to +7.0 m above LNT and the water depth along the berth face is -22.0 m below LNT. The sides of the terminal are constructed from identically sized cells that are flared at an angle of 40° from the berth face to minimize an increased load build up due to the frictional effects of ice.

The cellular sheet pile option is a gravity based rigid structural solution and was chosen on the basis that its magnitude and weight make it highly resistant to large lateral loads which are anticipated at site due to impact from a 220,000 DWT design vessel and from loads acting on the structure due to ice pressure.

Reinforced pre-cast concrete panels, 3.5 m high by 3 m wide x 750 mm thick are installed directly behind the exposed face of the sheet pile structure to provide added resistance to the harmful effects of ice loading.

A steel option was preferred over other materials due to its enhanced durability to the harmful effects imposed by the harsh arctic environment such as freeze-thaw cycles, ice abrasion, susceptibility to dynamic effects of ice, and its low maintenance requirements.

Exposed bedrock conditions render driving of the sheet piles impermissible. Therefore, blasting of the sea floor is required to achieve a level surface so the structure may be seated properly. Prior to installation of the sheet piles, a 2.5 m thick free-draining rubble foundation is to be installed at site to permit some form of overburden for the driven sheet piles.

The wharf is to be outfitted with twenty (20) SCN 1800 E3.0 fenders separated by 13.387 m center-to-center. The fenders support UHMW-PE fender panels measuring 4.5 m x 4.5 m. The fenders are rigidly attached to a 3.5 m high x 1.5 m wide continuous concrete cope wall that extends the entire berthing length and is cast in place with 35 MPa concrete. The cope wall is built integrally with a 300 mm slab-on-grade which serves as the deck of the proposed port. The slab on grade is thickened by 300 mm for a 1.5 m width to support a typical ship loader. A typical ship loader to be employed at this site is capable of a loading rate of 10,000 tph and weighs approximately 860 tons.

Eight (8) 200 tonne tee bollards are to be recessed equally spaced along the top of the cope wall. The bollards are to be embedded to the cope wall with seven (7) 1 m long anchor bolts.

The sheet pile structure is to be equipped with a galvanic anode cathodic protection system to protect it form the corrosive environment of sea water.

The recommend option is expected to cost $40,281,160.

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18 LIST OF RESOURCES

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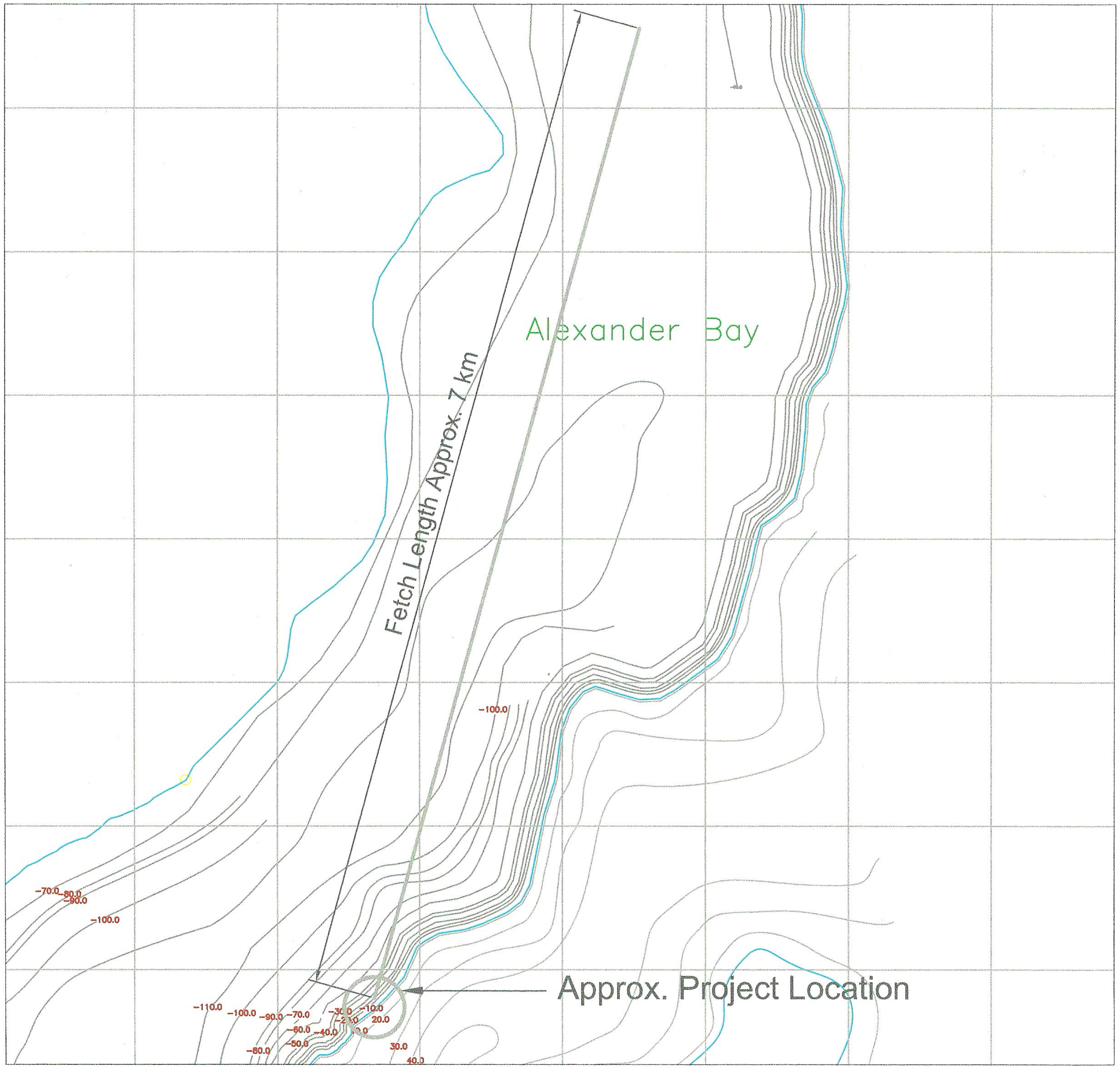
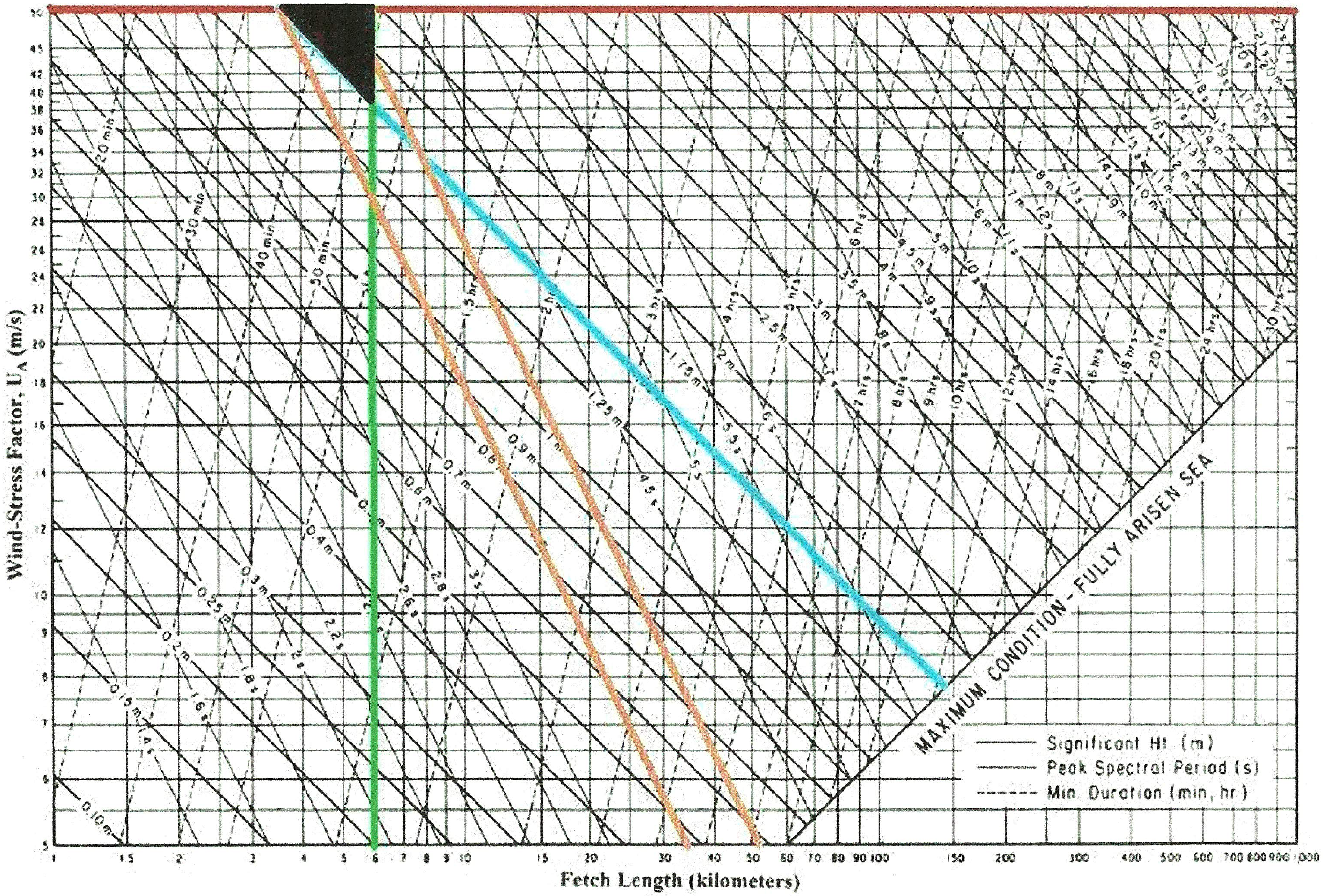
APPENDIX A: DESIGN WAVE CALCULATIONS



Alexander Bay
Fetch Length Approx. 7 km
Approx. Project Location
-100.0
-70.0
-80.0
-90.0
-100.0
-110.0
-100.0
-90.0
-70.0
-60.0
-50.0
-40.0
-80.0
-10.0
20.0
30.0
40.0


Wind-Stress Factor, U_A (m/s)
Fetch Length (kilometers)
MAXIMUM CONDITION - FULLY ARISEN SEA
Significant Ht. (m)
Peak Spectral Period (s)
Min. Duration (min, hr)

» Based on procedure outlined in USACE SPM (1984)

↳ Wind stress factor (U_A)

$U_A = 0.71(U_{10})^{1.23}$ assume provided velocity of wind is at 10m reference elevation ∴ $U_{10} = 36m/s$

$U_A = 0.71(36m/s)^{1.23}$
$U_A = 58m/s$

Note Fetch chart tops out @ 50, however based on the spacing of divisions along the y-axis near $U_A = 50$ we will assume to take a line for $U_A = 58m/s$.

↳ $H_s = 1.5m$ (client provided)

↳ Fetch length = 7km (based on scaling of site photo - see next)

* Using nomograph for deepwater wave prediction based on (3) known values

$T = 3.5$ to $4.0s$ (assume $T = 4.0s$)

→ ∴ deepwater wave

$L = L_0 = \frac{gT^2}{2\pi} = \frac{9.81(4.0s)^2}{2\pi} \approx 25m$

$C = L/T = 25m/4.0s = 6.25m/s$

Equations found in CEM 1110-2-1100 (see next)

→ check deepwater wave assumption

$d/L = \frac{22m}{25m}$ ← based on structure layout ∴ $= 0.88 > 0.5$ ∴ OK

Relative Depth	Shallow Water $\frac{d}{L} < \frac{1}{25}$	Transitional Water $\frac{1}{25} < \frac{d}{L} < \frac{1}{2}$	Deep Water $\frac{d}{L} < \frac{1}{2}$
1. Wave profile	Same As >	$\eta = \frac{H}{2}\cos\left[\frac{2\pi x}{L} - \frac{2\pi t}{T}\right] = \frac{H}{2}\cos\theta$	< Same As
2. Wave celerity	$C = \frac{L}{T} = \sqrt{gd}$	$C = \frac{L}{T} = \frac{gT}{2\pi}\tanh\left(\frac{2\pi d}{L}\right)$	$C = C_0 = \frac{L}{T} = \frac{gT}{2\pi}$
3. Wavelength	$L = T\sqrt{gd} = CT$	$L = \frac{gT^2}{2\pi}\tanh\left(\frac{2\pi d}{L}\right)$	$L = L_0 = \frac{gT^2}{2\pi} = C_0 T$
4. Group velocity	$C_g = C = \sqrt{gd}$	$C_g = nC = \frac{1}{2}\left[1 + \frac{4\pi d/L}{\sinh(4\pi d/L)}\right]C$	$C_g = \frac{1}{2}C = \frac{gT}{4\pi}$
5. Water particle velocity			
(a) Horizontal	$u = \frac{H}{2}\sqrt{\frac{g}{d}}\cos\theta$	$u = \frac{H}{2}\frac{gT}{L}\frac{\cosh[2\pi(z+d)/L]}{\cosh(2\pi d/L)}\cos\theta$	$u = \frac{\pi H}{T}e^{\left(\frac{2\pi z}{L}\right)}\cos\theta$
(b) Vertical	$w = \frac{H\pi}{T}\left(1+\frac{z}{d}\right)\sin\theta$	$w = \frac{H}{2}\frac{gT}{L}\frac{\sinh[2\pi(z+d)/L]}{\cosh(2\pi d/L)}\sin\theta$	$w = \frac{\pi H}{T}e^{\left(\frac{2\pi z}{L}\right)}\sin\theta$
6. Water particle accelerations			
(a) Horizontal	$a_x = \frac{H\pi}{T}\sqrt{\frac{g}{d}}\sin\theta$	$a_x = \frac{g\pi H}{L}\frac{\cosh[2\pi(z+d)/L]}{\cosh(2\pi d/L)}\sin\theta$	$a_x = 2H\left(\frac{\pi}{T}\right)^2 e^{\left(\frac{2\pi z}{L}\right)}\sin\theta$
(b) Vertical	$a_z = -2H\left(\frac{\pi}{T}\right)^2\left(1+\frac{z}{d}\right)\cos\theta$	$a_z = -\frac{g\pi H}{L}\frac{\sinh[2\pi(z+d)/L]}{\cosh(2\pi d/L)}\cos\theta$	$a_z = -2H\left(\frac{\pi}{T}\right)^2 e^{\left(\frac{2\pi z}{L}\right)}\cos\theta$
7. Water particle displacements			
(a) Horizontal	$\xi = -\frac{HT}{4\pi}\sqrt{\frac{g}{d}}\sin\theta$	$\xi = -\frac{H}{2}\frac{\cosh[2\pi(z+d)/L]}{\sinh(2\pi d/L)}\sin\theta$	$\xi = -\frac{H}{2}e^{\left(\frac{2\pi z}{L}\right)}\sin\theta$
(b) Vertical	$\zeta = \frac{H}{2}\left(1+\frac{z}{d}\right)\cos\theta$	$\zeta = \frac{H}{2}\frac{\sinh[2\pi(z+d)/L]}{\sinh(2\pi d/L)}\cos\theta$	$\zeta = \frac{H}{2}e^{\left(\frac{2\pi z}{L}\right)}\cos\theta$
8. Subsurface pressure	$p = \rho g(\eta - z)$	$p = \rho g\eta\frac{\cosh[2\pi(z+d)/L]}{\cosh(2\pi d/L)} - \rho gz$	$p = \rho g\eta\, e^{\left(\frac{2\pi z}{L}\right)} - \rho gz$

Figure II-1-9. Summary of linear (Airy) wave theory - wave characteristics

assumption may be questionable. A third dimensionless parameter, which may be used to replace either the wave steepness or relative water depth, may be defined as the ratio of wave steepness to relative water depth. Thus,

$$\frac{H/L}{d/L} = \frac{H}{d} \qquad \text{(II-1-66)}$$

APPENDIX B: SELECTION OF DESIGN VESSEL

PARAMOUNT	DESIGN CALCULATIONS SHEET	DATE:	Feb 2/2010
		PAGE:	1 of 2

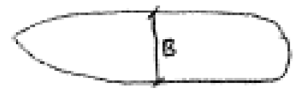
PROJECT:	St.Lawrence Marine Terminal	DOCUMENT NO.:	DC1-8700-07-001
PROJECT NO.:	8700-07	REVISION:	0
ITEM:	Design Vessel Dimensions	PREPARED BY:	Steven Greeley

» Client instructed the design vessel to be 220,000 DWT

» Two (2) resources used to determine vessel's main dimensions:

① LOA (length overall)
② B (breadth)
③ D (draught)

(x-section)



(PLAN)

* Resource #1 - PLANNING AND DESIGN OF PORTS AND MARINE TERMINALS (2nd ed.)

→ pp.55-57 list dimensions of dry-bulk carrier and the frequency of ships that size in operation
→ performed mean statistical analysis on vessels (200,000-225,000 DWT) (28 Total)

① LOA:
(4) @ 295-299m
(2) @ 300-304m
(6) @ 305-309m
(13) @ 310-314m
(2) @ 315-319m
(1) @ 320-324m
28

take ½ way value in each range

$\therefore$ LOA: $\frac{8646m}{28}$ = 308.8m ⇒ 310m

② Breadth (B):
(26) @ 49-50m
(2) @ 53-54m
28

$\therefore$ B = $\frac{1394m}{28}$ = 49.8m ⇒ 50m

③ Draught (D):
(1) @ 16.5-16.99m
(2) @ 17.5-17.99m
(18) @ 18.0-18.49m
(2) @ 18.5-18.99m
(5) @ 19.5-19.99m
28

$\therefore$ D = $\frac{517m}{28}$ = 18.5m ⇒ 18.5m

PARAMOUNT	DESIGN CALCULATIONS SHEET	DATE:	Feb 2/2010
		PAGE:	2 of 2

PROJECT:	St. Lawrence Marine Terminal	DOCUMENT NO.:	DC1-8700-07-001
PROJECT NO.:	8700-07	REVISION:	D
ITEM:	Design Vessel Dimensions	PREPARED BY:	Steven Greeley

* Resource #2: DESIGN OF MARINE FACILITIES FOR THE BERTHING, MOORING, AND REPAIR OF VESSELS

→ figure 2.6, p.30 provides a table w/ dimensions for avg. size bulk carriers
→ the values calculated are an average from 200,000DWT & 250,000DWT vessels

① LOA: $\frac{327+344}{2} = 335.5m \Rightarrow 336m$ (8% higher than resource #1)

② B: $\frac{52m+54m}{2} = 53.0m$ (6% higher than resource #1)

③ D: $\frac{19.1+21.0m}{2} = 20.0m$ (7.5% higher than resource #1)

⇒ Consulted w/ client on final Vessel Dimensions...
Selected ⇒

LOA = 310m

Breadth (B): 50m

Draught (D): 18.5m

→ Vessel depth (only information contained in resource #2)

$D_s = \frac{27+28.5}{2} = 27.75m$ (Take 27.5m)

→ Light draft condition (based on section 2.1 of resource #2)

Tankers/Bulk carriers = 30% to 50% of full load displacement for "in ballast" conditions

∴ $0.3(D) \rightarrow 0.5(D) = 5.55m \rightarrow 9.25m$ (7.4m average)

∴ Light Draft → select 7.5m as light draft

APPENDIX C: MOORING FORCE CALCULATIONS

PROJECT:	St. Lawerence Marine Terminal	DOCUMENT NO.:	DC1-8700-07-003
PROJECT NO.:	8700-07	REVISION:	0
ITEM:	Mooring Forces	PREPARED BY:	Robert Hunt

INPUT

Vessel Information:

Length overall:	LOA =	310	m
Length BP:	L_{BP} =	310	m
Beam:	B =	50	m
Molded Depth:	MD =	27.5	m
Loaded Draft:	LD =	18.5	m
Light Draft:	D =	7.5	m
Superstructure Area:	A_{ss} =	1000	m^2

Environmental Information:

Wind @ 90 deg:	$V_{w,90}$ =	36	m/s
Wind @ 45 deg:	$V_{w,45}$ =	36	m/s
Wind @ 0 deg:	$V_{w,0}$ =	36	m/s
Density of air:	ρ_a =	1.31	kg/m^3
Ice thickness:	t_{ice} =	1.5	m
Wave height:	H_s =	1.5	m
Current (longitudinal):	$V_{c,L}$ =	0.51	m/s
Current (transverse):	$V_{c,T}$ =	0.51	m/s
Density of sea-water:	ρ_w =	1025	kg/m^3
Highest High Water Level:	HHWL =	2	m
Lowest Water Level:	LLWL =	0	m

Site Information:

Deck Elevation:	El =	7	m
Water Depth (@LLWL):	d =	-22	m

PROJECT:	St. Lawerence Marine Terminal	DOCUMENT NO.:	DC1-8700-07-003
PROJECT NO.:	8700-07	REVISION:	0
ITEM:	Mooring Forces	PREPARED BY:	Robert Hunt

Force Coefficients *(independent of tidal position)*

Draft	Unloaded (Ballast)			Loaded			
Angle (α)	0	45	90	0	45	90	
Coefficient							Figure
$C_{TW,FWD}$	0	2	2.2	0	0.85	1.15	27
$C_{TW,AFT}$	0	1.4	2.6	0	1.4	2.15	27
C_{LW}	1.2	0.4	-0.4	1.8	1.1	-0.06	27
$C_{TC,FWD}$	0	1.16	1.4	0	1.16	1.4	25
$C_{TC,AFT}$	0	0.68	1.56	0	0.68	1.56	25
C_{LC}	0.4	-0.48	0	0.4	-0.48	0	25

Water Depth Correction Factors

At HHWL:

Draft	Unloaded (Ballast)			Loaded			
d/d_m =	(d+HHWL)/D = 3.20			(d+HHWL)/LD = 1.30			
Angle (α)	0	45	90	0	45	90	
Coefficient							Figure
C_{CT}	1	1	1	5	3.42	2.92	29
C_{CL}	1	1	1	1.33	1.33	1.33	30

At LLWL:

Draft	Unloaded (Ballast)			Loaded			
d/d_m =	(d+LLWL)/D = 2.93			(d+LLWL)/LD = 1.19			
Angle (α)	0	45	90	0	45	90	
Coefficient							Figure
C_{CT}	1.92	1.67	1.75	10	4.5	4.2	29
C_{CL}	1	1	1	1.5	1.5	1.5	30

PARAMOUNT	DESIGN CALCULATIONS SHEET	DATE:	14-Feb-10
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PROJECT:	St. Lawerence Marine Terminal	**DOCUMENT NO.:**	DC1-8700-07-003
PROJECT NO.:	8700-07	**REVISION:**	0
ITEM:	Mooring Forces	**PREPARED BY:**	Robert Hunt

Area Calculations

Superstructure Area: $A_{ss} =$ 1000 m^2

Ballast Condition

Wind from ocean side:

$A_{wind} =$ 6200 m^2 → $\mathbf{A_t =}$ **7200** $\mathbf{m^2}$

Wind from land side:

$A_{wind} =$ 4030 m^2 → $\mathbf{A_t =}$ **5030** $\mathbf{m^2}$

From current:

$A_{current} =$ **2325** m^2

Loaded Condition

Wind from ocean side:

$A_{wind} =$ 2790 m^2 → $\mathbf{A_t =}$ **3790** $\mathbf{m^2}$

Wind from land side:

$A_{wind} =$ 620 m^2 → $\mathbf{A_t =}$ **1620** $\mathbf{m^2}$

From current:

$A_{current} =$ **5735** m^2

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PROJECT:	St. Lawerence Marine Terminal	DOCUMENT NO.:	DC1-8700-07-003
PROJECT NO.:	8700-07	REVISION:	0
ITEM:	Mooring Forces	PREPARED BY:	Robert Hunt

Wind Force

Wind from:	Ocean Side				Land Side			
Tide	LLWL		HHWL		LLWL		HHWL	
Draft / Force	Ballast	Loaded	Ballast	Loaded	Ballast	Loaded	Ballast	Loaded
$F_{TW,90,FWD}$	2688.4	739.7	2688.4	739.7	1878.2	316.2	1878.2	316.2
$F_{TW,45,FWD}$	2444.0	546.8	2444.0	546.8	1707.4	233.7	1707.4	233.7
$F_{TW,0,FWD}$	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
$F_{TW,90,AFT}$	3177.2	1383.0	3177.2	1383.0	2219.7	591.1	2219.7	591.1
$F_{TW,45,AFT}$	1710.8	900.6	1710.8	900.6	1195.2	384.9	1195.2	384.9
$F_{TW,0,AFT}$	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
$F_{LW,90}$	-488.8	-38.6	-488.8	-38.6	-341.5	-16.5	-341.5	-16.5
$F_{LW,45}$	488.8	707.6	488.8	707.6	341.5	302.4	341.5	302.4
$F_{LW,0}$	1466.4	1157.9	1466.4	1157.9	1024.5	494.9	1024.5	494.9

Current Force

Tide	LLWL		HHWL	
Draft / Force	Ballast	Loaded	Ballast	Loaded
$F_{TC,90,FWD}$	151.9	899.0	86.8	0.0
$F_{TC,45,FWD}$	120.1	798.1	71.9	0.0
$F_{TC,0,FWD}$	0.0	0.0	0.0	0.0
$F_{TC,90,AFT}$	169.2	1001.8	96.7	0.0
$F_{TC,45,AFT}$	70.4	467.9	42.1	0.0
$F_{TC,0,AFT}$	0.0	0.0	0.0	0.0
$F_{LC,90}$	0.0	0.0	0.0	0.0
$F_{LC,45}$	-29.8	-110.1	-29.8	-97.6
$F_{LC,0}$	24.8	305.0	24.8	81.3

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PROJECT:	St. Lawerence Marine Terminal	**DOCUMENT NO.:**	DC1-8700-07-003
PROJECT NO.:	8700-07	**REVISION:**	0
ITEM:	Mooring Forces	**PREPARED BY:**	Robert Hunt

Force Summary

Conditions: Wind from ocean side
Tide: LLWL
Draft: Ballast

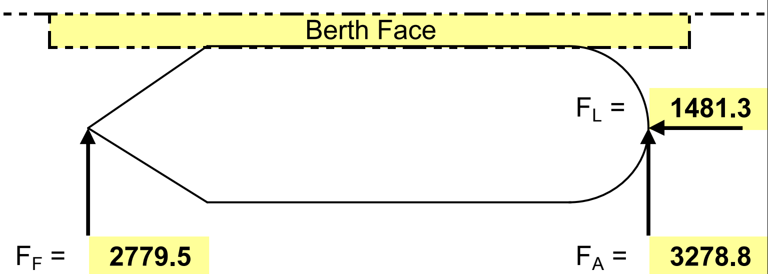
Total Transverse Force: [kN]

$F_{T,90,FWD} = F_{TW,90,FWD} + F_{TC,90,FWD} =$ **2779.5**
$F_{T,90,AFT} = F_{TW,90,AFT} + F_{TC,90,AFT} =$ **3278.8**
$F_{T,45,FWD} = F_{TW,45,FWD} + F_{TC,45,FWD} =$ **2516.1**
$F_{T,45,AFT} = F_{TW,45,AFT} + F_{TC,45,AFT} =$ **1753.1**
$F_{T,0,FWD} = F_{TW,0,FWD} + F_{TC,0,FWD} =$ **0.0**
$F_{T,0,AFT} = F_{TW,0,AFT} + F_{TC,0,AFT} =$ **0.0**

Total Longitudinal Force: [kN]

$F_{L,90} = FL_{W,90} + F_{LC,90} =$ **-488.8**
$F_{L,45} = F_{LW,45} + F_{LC,45} =$ **471.0**
$F_{L,0} = F_{LW,0} + F_{LC,40} =$ **1481.3**

$F_L := \max(F_{L,90}, F_{L,45}, F_{L,0})$
$F_F := \max(F_{T,90,FWD}, F_{T,45,FWD}, F_{T,0,FWD})$
$F_A := \max(F_{T,90,AFT}, F_{T,45,AFT}, F_{T,0,AFT})$



Moment about stern: [kN-m]

$M_{stern} := (F_F - F_A) * L_{BP} =$ **-154757.9**

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PROJECT:	St. Lawerence Marine Terminal	DOCUMENT NO.:	DC1-8700-07-003
PROJECT NO.:	8700-07	REVISION:	0
ITEM:	Mooring Forces	PREPARED BY:	Robert Hunt

Force Summary

Conditions: Wind from ocean side
Tide: LLWL
Draft: Loaded

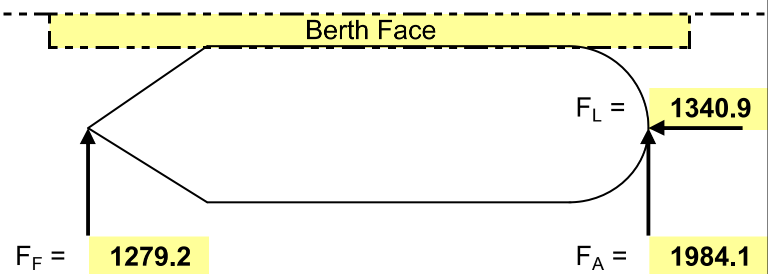
Total Transverse Force: [kN]

$F_{T,90,FWD} = F_{TW,90,FWD} + F_{TC,90,FWD} =$ **1279.2**
$F_{T,90,AFT} = F_{TW,90,AFT} + F_{TC,90,AFT} =$ **1984.1**
$F_{T,45,FWD} = F_{TW,45,FWD} + F_{TC,45,FWD} =$ **1025.6**
$F_{T,45,AFT} = F_{TW,45,AFT} + F_{TC,45,AFT} =$ **1181.3**
$F_{T,0,FWD} = F_{TW,0,FWD} + F_{TC,0,FWD} =$ **0.0**
$F_{T,0,AFT} = F_{TW,0,AFT} + F_{TC,0,AFT} =$ **0.0**

Total Longitudinal Force: [kN]

$F_{L,90} = FL_{W,90} + F_{LC,90} =$ **-38.6**
$F_{L,45} = F_{LW,45} + F_{LC,45} =$ **641.5**
$F_{L,0} = F_{LW,0} + F_{LC,40} =$ **1340.9**

$F_L := \max(F_{L,90}, F_{L,45}, F_{L,0})$
$F_F := \max(F_{T,90,FWD}, F_{T,45,FWD}, F_{T,0,FWD})$
$F_A := \max(F_{T,90,AFT}, F_{T,45,AFT}, F_{T,0,AFT})$



Moment about stern: [kN-m]

$M_{stern} := (F_F - F_A) * L_{BP} =$ **-218519.8**

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PROJECT:	St. Lawerence Marine Terminal	DOCUMENT NO.:	DC1-8700-07-003
PROJECT NO.:	8700-07	REVISION:	0
ITEM:	Mooring Forces	PREPARED BY:	Robert Hunt

Force Summary

Conditions: Wind from ocean side
Tide: HHWL
Draft: Ballast

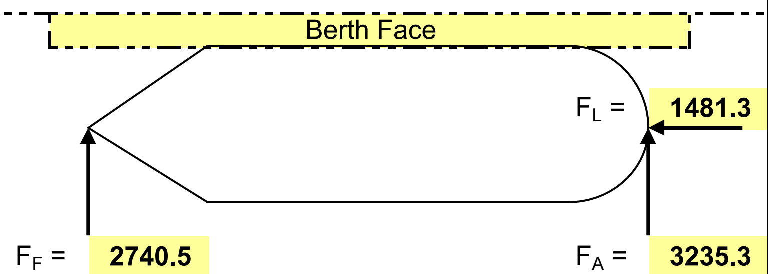
Total Transverse Force: [kN]

$F_{T,90,FWD} = F_{TW,90,FWD} + F_{TC,90,FWD} =$ **2740.5**
$F_{T,90,AFT} = F_{TW,90,AFT} + F_{TC,90,AFT} =$ **3235.3**
$F_{T,45,FWD} = F_{TW,45,FWD} + F_{TC,45,FWD} =$ **2487.2**
$F_{T,45,AFT} = F_{TW,45,AFT} + F_{TC,45,AFT} =$ **1736.1**
$F_{T,0,FWD} = F_{TW,0,FWD} + F_{TC,0,FWD} =$ **0.0**
$F_{T,0,AFT} = F_{TW,0,AFT} + F_{TC,0,AFT} =$ **0.0**

Total Longitudinal Force: [kN]

$F_{L,90} = FL_{W,90} + F_{LC,90} =$ **-488.8**
$F_{L,45} = F_{LW,45} + F_{LC,45} =$ **471.0**
$F_{L,0} = F_{LW,0} + F_{LC,40} =$ **1481.3**

$F_L := \max(F_{L,90}, F_{L,45}, F_{L,0})$
$F_F := \max(F_{T,90,FWD}, F_{T,45,FWD}, F_{T,0,FWD})$
$F_A := \max(F_{T,90,AFT}, F_{T,45,AFT}, F_{T,0,AFT})$



Moment about stern: [kN-m]

$M_{stern} := (F_F - F_A) * L_{BP} =$ **-153374.4**

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ITEM:	Mooring Forces	PREPARED BY:	Robert Hunt

Force Summary

Conditions: Wind from ocean side
Tide: HHWL
Draft: Loaded

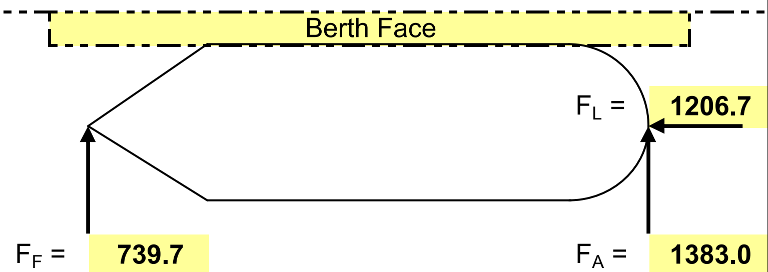
Total Transverse Force: [kN]

$F_{T,90,FWD} = F_{TW,90,FWD} + F_{TC,90,FWD} =$	**739.7**
$F_{T,90,AFT} = F_{TW,90,AFT} + F_{TC,90,AFT} =$	**1383.0**
$F_{T,45,FWD} = F_{TW,45,FWD} + F_{TC,45,FWD} =$	**546.8**
$F_{T,45,AFT} = F_{TW,45,AFT} + F_{TC,45,AFT} =$	**900.6**
$F_{T,0,FWD} = F_{TW,0,FWD} + F_{TC,0,FWD} =$	**0.0**
$F_{T,0,AFT} = F_{TW,0,AFT} + F_{TC,0,AFT} =$	**0.0**

Total Longitudinal Force: [kN]

$F_{L,90} = FL_{W,90} + F_{LC,90} =$	**-38.6**
$F_{L,45} = F_{LW,45} + F_{LC,45} =$	**649.0**
$F_{L,0} = F_{LW,0} + F_{LC,40} =$	**1206.7**

F_L:=max($F_{L,90}$,$F_{L,45}$,$F_{L,0}$)
F_F:=max($F_{T,90,FWD}$,$F_{T,45,FWD}$,$F_{T,0,FWD}$)
F_A:=max($F_{T,90,AFT}$,$F_{T,45,AFT}$,$F_{T,0,AFT}$)



Moment about stern: [kN-m]

M_{stern}:= $(F_F-F_A)*L_{BP}$ = **-199408.9**

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PROJECT NO.:	8700-07	REVISION:	0
ITEM:	Mooring Forces	PREPARED BY:	Robert Hunt

Force Summary

Conditions: Wind from land side
Tide: LLWL
Draft: Ballast

Total Transverse Force: [kN]

$F_{T,90,FWD} = F_{TW,90,FWD} + F_{TC,90,FWD} =$ **1969.3**

$F_{T,90,AFT} = F_{TW,90,AFT} + F_{TC,90,AFT} =$ **2321.2**

$F_{T,45,FWD} = F_{TW,45,FWD} + F_{TC,45,FWD} =$ **1779.5**

$F_{T,45,AFT} = F_{TW,45,AFT} + F_{TC,45,AFT} =$ **1237.4**

$F_{T,0,FWD} = F_{TW,0,FWD} + F_{TC,0,FWD} =$ **0.0**

$F_{T,0,AFT} = F_{TW,0,AFT} + F_{TC,0,AFT} =$ **0.0**

Total Longitudinal Force: [kN]

$F_{L,90} = FL_{W,90} + F_{LC,90} =$ **-341.5**

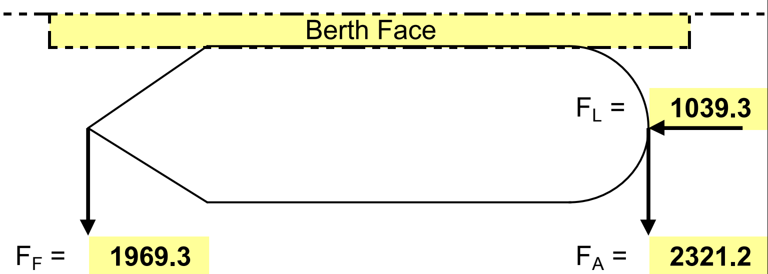
$F_{L,45} = F_{LW,45} + F_{LC,45} =$ **323.6**

$F_{L,0} = F_{LW,0} + F_{LC,40} =$ **1039.3**

$F_L := \max(F_{L,90}, F_{L,45}, F_{L,0})$

$F_F := \max(F_{T,90,FWD}, F_{T,45,FWD}, F_{T,0,FWD})$

$F_A := \max(F_{T,90,AFT}, F_{T,45,AFT}, F_{T,0,AFT})$



Moment about stern: [kN-m]

$M_{stern} := (F_F - F_A) * L_{BP} =$ **-109088.5**

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PROJECT NO.:	8700-07	REVISION:	0
ITEM:	Mooring Forces	PREPARED BY:	Robert Hunt

Force Summary

Conditions: Wind from land side
Tide: LLWL
Draft: Loaded

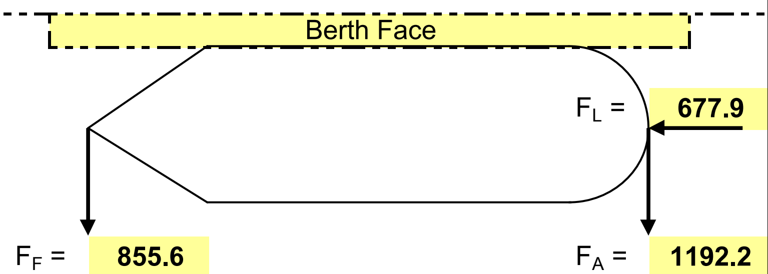
Total Transverse Force: [kN]

$F_{T,90,FWD} = F_{TW,90,FWD} + F_{TC,90,FWD} =$ **855.6**
$F_{T,90,AFT} = F_{TW,90,AFT} + F_{TC,90,AFT} =$ **1192.2**
$F_{T,45,FWD} = F_{TW,45,FWD} + F_{TC,45,FWD} =$ **712.6**
$F_{T,45,AFT} = F_{TW,45,AFT} + F_{TC,45,AFT} =$ **665.7**
$F_{T,0,FWD} = F_{TW,0,FWD} + F_{TC,0,FWD} =$ **0.0**
$F_{T,0,AFT} = F_{TW,0,AFT} + F_{TC,0,AFT} =$ **0.0**

Total Longitudinal Force: [kN]

$F_{L,90} = FL_{W,90} + F_{LC,90} =$ **-16.5**
$F_{L,45} = F_{LW,45} + F_{LC,45} =$ **236.4**
$F_{L,0} = F_{LW,0} + F_{LC,40} =$ **677.9**

F_L:=max($F_{L,90}$,$F_{L,45}$,$F_{L,0}$)
F_F:=max($F_{T,90,FWD}$,$F_{T,45,FWD}$,$F_{T,0,FWD}$)
F_A:=max($F_{T,90,AFT}$,$F_{T,45,AFT}$,$F_{T,0,AFT}$)



Moment about stern: [kN-m]

M_{stern}:= $(F_F-F_A)*L_{BP}$ = **-104346.3**

PROJECT:	St. Lawerence Marine Terminal	DOCUMENT NO.:	DC1-8700-07-003
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Force Summary

Conditions: Wind from land side
Tide: HHWL
Draft: Ballast

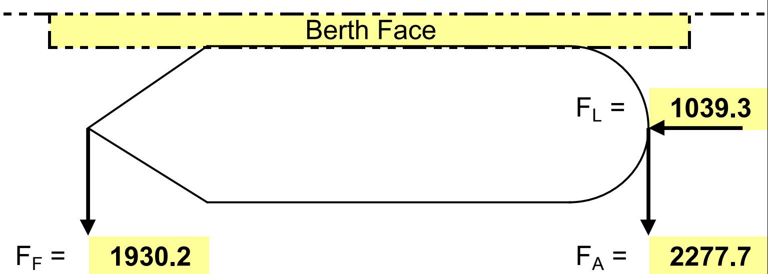
Total Transverse Force: [kN]

$F_{T,90,FWD} = F_{TW,90,FWD} + F_{TC,90,FWD} =$ **1930.2**
$F_{T,90,AFT} = F_{TW,90,AFT} + F_{TC,90,AFT} =$ **2277.7**
$F_{T,45,FWD} = F_{TW,45,FWD} + F_{TC,45,FWD} =$ **1750.6**
$F_{T,45,AFT} = F_{TW,45,AFT} + F_{TC,45,AFT} =$ **1220.5**
$F_{T,0,FWD} = F_{TW,0,FWD} + F_{TC,0,FWD} =$ **0.0**
$F_{T,0,AFT} = F_{TW,0,AFT} + F_{TC,0,AFT} =$ **0.0**

Total Longitudinal Force: [kN]

$F_{L,90} = FL_{W,90} + F_{LC,90} =$ **-341.5**
$F_{L,45} = F_{LW,45} + F_{LC,45} =$ **323.6**
$F_{L,0} = F_{LW,0} + F_{LC,40} =$ **1039.3**

$F_L := \max(F_{L,90}, F_{L,45}, F_{L,0})$
$F_F := \max(F_{T,90,FWD}, F_{T,45,FWD}, F_{T,0,FWD})$
$F_A := \max(F_{T,90,AFT}, F_{T,45,AFT}, F_{T,0,AFT})$



Moment about stern: [kN-m]

$M_{stern} := (F_F - F_A) * L_{BP} =$ **-107705.0**

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ITEM:	Mooring Forces	PREPARED BY:	Robert Hunt

Force Summary

Conditions: Wind from land side
Tide: HHWL
Draft: Loaded

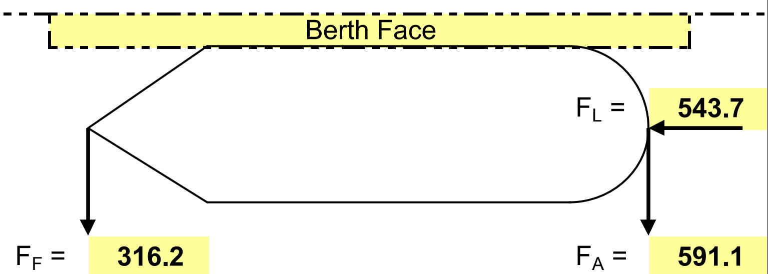
Total Transverse Force: [kN]

$F_{T,90,FWD} = F_{TW,90,FWD} + F_{TC,90,FWD} =$ **316.2**
$F_{T,90,AFT} = F_{TW,90,AFT} + F_{TC,90,AFT} =$ **591.1**
$F_{T,45,FWD} = F_{TW,45,FWD} + F_{TC,45,FWD} =$ **233.7**
$F_{T,45,AFT} = F_{TW,45,AFT} + F_{TC,45,AFT} =$ **384.9**
$F_{T,0,FWD} = F_{TW,0,FWD} + F_{TC,0,FWD} =$ **0.0**
$F_{T,0,AFT} = F_{TW,0,AFT} + F_{TC,0,AFT} =$ **0.0**

Total Longitudinal Force: [kN]

$F_{L,90} = FL_{W,90} + F_{LC,90} =$ **-16.5**
$F_{L,45} = F_{LW,45} + F_{LC,45} =$ **243.9**
$F_{L,0} = F_{LW,0} + F_{LC,40} =$ **543.7**

$F_L := \max(F_{L,90}, F_{L,45}, F_{L,0})$
$F_F := \max(F_{T,90,FWD}, F_{T,45,FWD}, F_{T,0,FWD})$
$F_A := \max(F_{T,90,AFT}, F_{T,45,AFT}, F_{T,0,AFT})$



Moment about stern: [kN-m]

$M_{stern} := (F_F - F_A) * L_{BP} =$ **-85235.5**

14 9 18·
13 9 15

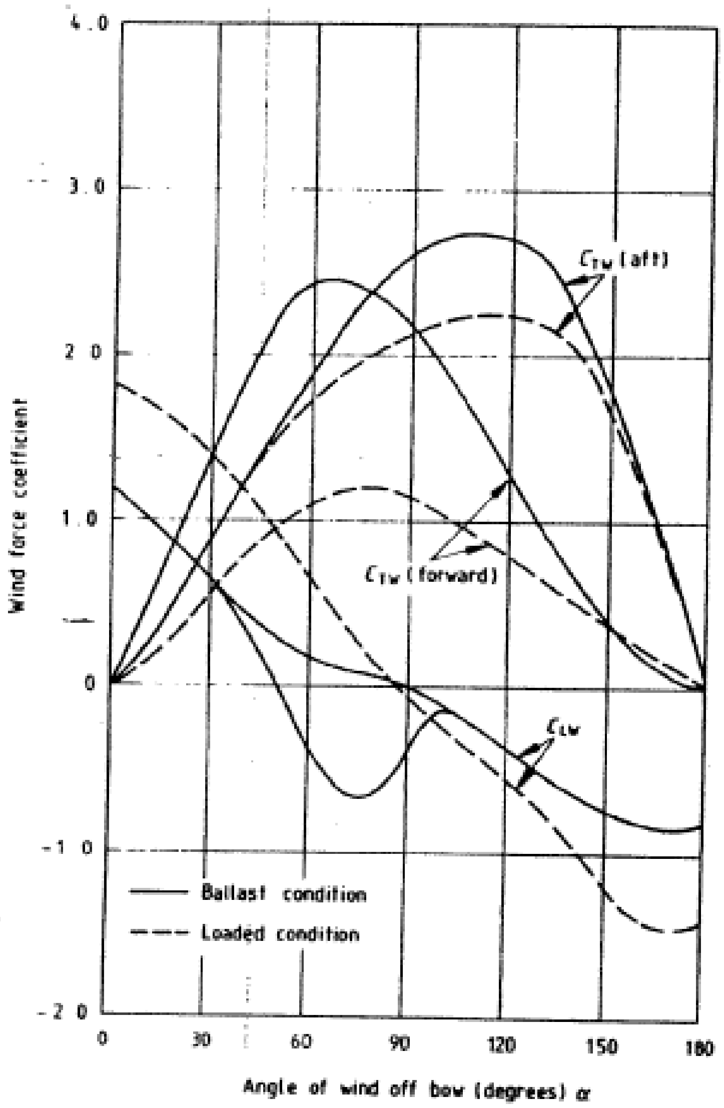


Figure 27. Wind force coefficients for very large tankers with superstructures aft

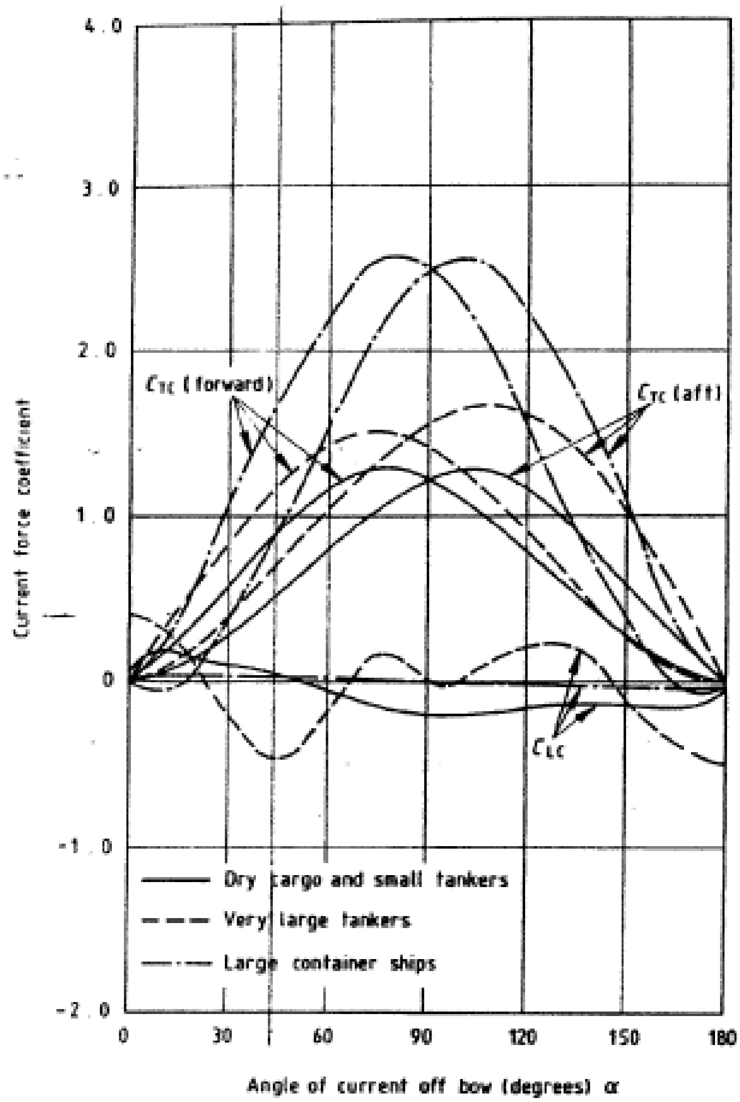


Figure 25. Current force coefficients, all ships, deep water case

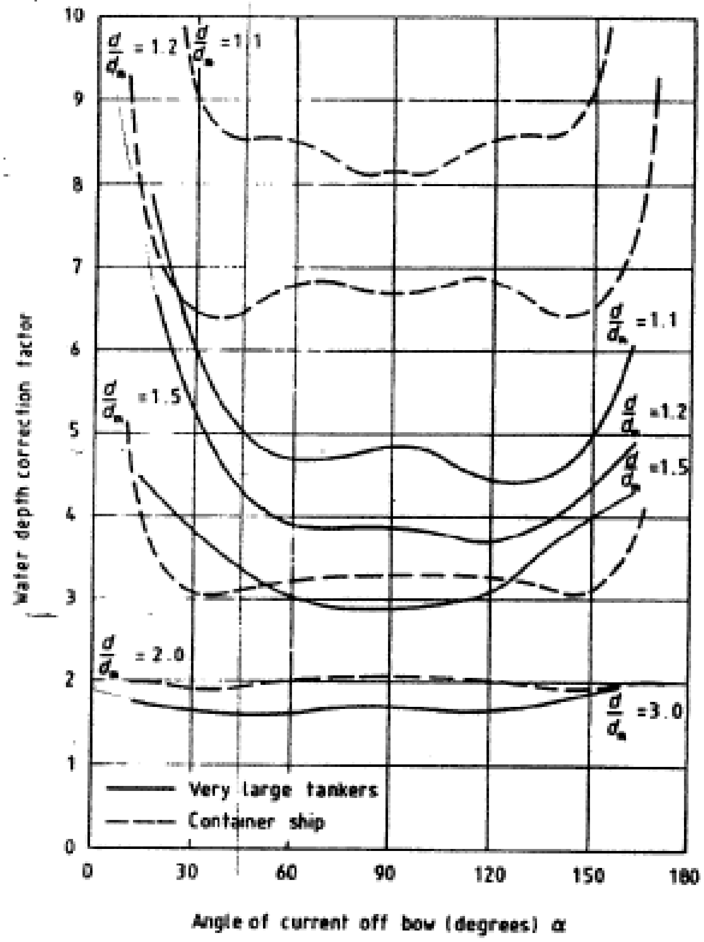


Figure 29. Water depth correction factors for lateral current forces

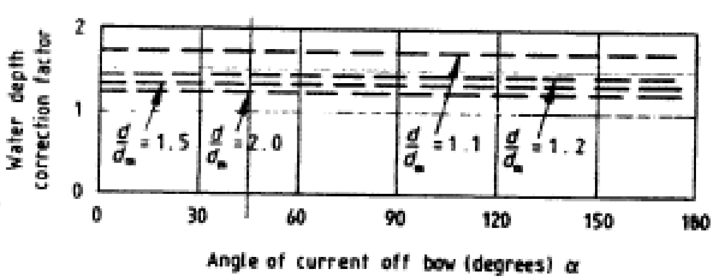


Figure 30. Water depth correction factor for longitudinal current forces on container ships

APPENDIX D: SELECTION MATRIX RESULTS

#	Criteria	Weight (W)	Gravity Walls: Block Wall Performance (P)	Block Wall Points (PTS)	L-Wall Performance (P)	L-Wall Points (PTS)	Caisson Wall Performance (P)	Caisson Wall Points (PTS)	Cellular Wall Performance (P)	Cellular Wall Points (PTS)	Sheet Pile Walls: Single Performance (P)	Single Points (PTS)	Combined Performance (P)	Combined Points (PTS)	Cofferdam Performance (P)	Cofferdam Points (PTS)	Open Pile Structures: Steel Piles Performance (P)	Steel Piles Points (PTS)	Concrete Piles Performance (P)	Concrete Piles Points (PTS)	Floating Hybrid Performance (P)	Floating Hybrid Points (PTS)	Timber Crib Performance (P)	Timber Crib Points (PTS)
1.	Subsurface soil conditions	**5**	5	**25**	5	**25**	5	**25**	5	**25**	1	**5**	1	**5**	1	**5**	3	**15**	3	**15**	5	**25**	5	**25**
2.	Profile of seafloor	**5**	1	**5**	1	**5**	1	**5**	1	**5**	5	**25**	5	**25**	5	**25**	3	**15**	3	**15**	5	**25**	1	**5**
3.	Water depth	**4**	3	**12**	2	**8**	3	**12**	3	**12**	4	**16**	4	**16**	4	**16**	5	**20**	5	**20**	5	**20**	1	**4**
4.	Construction material requirements	**3**	1	**3**	5	**15**	5	**15**	3	**9**	4	**12**	3	**9**	3	**9**	3	**9**	3	**9**	2	**6**	5	**15**
5.	Material degradation	**2**	3	**6**	3	**6**	3	**6**	5	**10**	5	**10**	5	**10**	5	**10**	4	**8**	2	**4**	5	**10**	1	**2**
6.	Accomodation of ship loader foundation	**2**	4	**8**	3	**6**	5	**10**	4	**8**	2	**4**	2	**4**	2	**4**	1	**2**	1	**2**	5	**10**	1	**2**
7.	Historical arctic applications	**2**	1	**2**	1	**2**	4	**8**	5	**10**	1	**2**	2	**4**	2	**4**	3	**6**	3	**6**	1	**2**	1	**2**
8.	Hydraulic conditions	**3**	3	**9**	4	**12**	4	**12**	2	**6**	2	**6**	2	**6**	2	**6**	5	**15**	5	**15**	5	**15**	4	**12**
9.	Constructability	**3**	3	**9**	3	**9**	3	**9**	4	**12**	4	**12**	3	**9**	3	**9**	3	**9**	3	**9**	3	**9**	2	**6**
10.	Load and impact resistance	**4**	5	**20**	5	**20**	5	**20**	4	**16**	4	**16**	5	**20**	5	**20**	3	**12**	3	**12**	1	**4**	2	**8**
11.	Long term maintenance	**1**	4	**4**	4	**4**	4	**4**	4	**4**	5	**5**	5	**5**	5	**5**	3	**3**	3	**3**	2	**2**	1	**1**
12.	Resistance to ice abrasion	**3**	2	**6**	2	**6**	2	**6**	3	**9**	3	**9**	3	**9**	3	**9**	4	**12**	3	**9**	5	**15**	1	**3**
13.	Freeze thaw durability	**3**	3	**9**	3	**9**	3	**9**	5	**15**	5	**15**	5	**15**	5	**15**	4	**12**	2	**6**	5	**15**	1	**3**
14.	Berthing ice control	**2**	3	**6**	1	**2**	1	**2**	3	**6**	1	**2**	1	**2**	1	**2**	5	**10**	5	**10**	3	**6**	1	**2**
15.	Susceptibility to dynamic effects of ice	**4**	4	**16**	5	**20**	5	**20**	5	**20**	5	**20**	5	**20**	5	**20**	2	**8**	2	**8**	3	**12**	1	**4**
	TOTAL SCORE			**140**		**149**		**163**		**167**		**159**		**159**		**159**		**156**		**143**		**176**		**94**
	RANK			**10**		**8**		**3**		**2**		**4**		**4**		**4**		**7**		**9**		**1**		**11**

APPENDIX E: ARCELOR PILING DESIGN MANUAL: CHPATER 9 –CIRCULAR CELL CONSTRUCTION DESIGN & INSTALLATION

ArcelorMittal

1 Product information

2 Sealants

3 Durability

4 Earth and water pressure

5 Design of sheet pile structures

6 Retaining walls

7 Cofferdams

8 Charts for retaining walls

9 Circular cell construction design & installation

10 Bearing piles and axially loaded sheet piles

11 Installation of sheet piles

12 Noise and vibration from piling operations

13 Useful information

Circular cell construction design & installation

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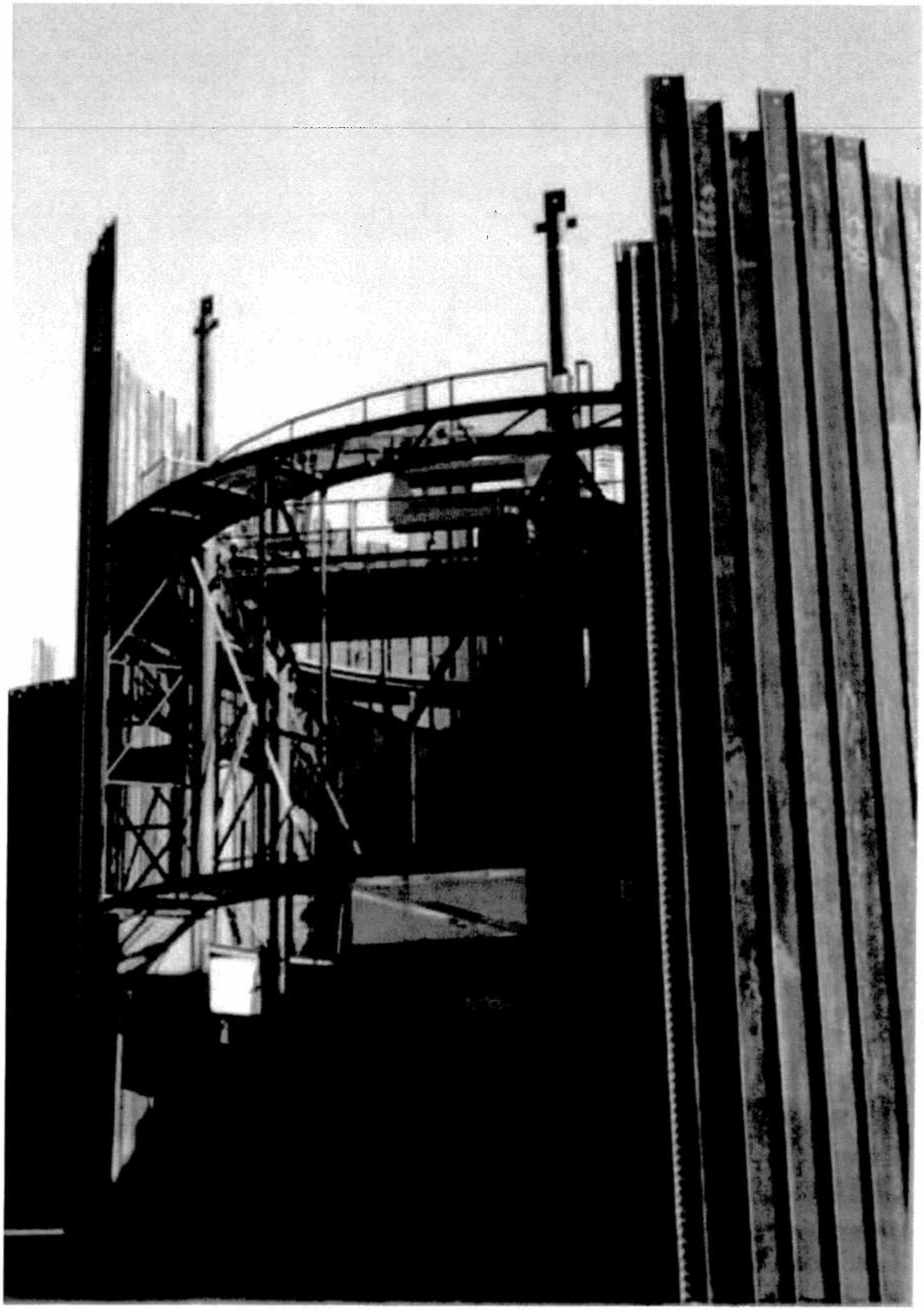
9.1	**Introduction**	1
9.2	**Straight web piling**	1
9.2.1	Dimensions and properties for AS 500 straight web piles	1
9.3	**Interlock strength**	2
9.4	**Junction piles**	3
9.5	**Types of cell**	3
9.6	**Bent piles**	3
9.7	**Equivalent width and ratio**	4
9.8	**Geometry**	5
9.8.1	Circular cells	5
9.8.2	Diaphragm cells	6
9.9	**Handling straight web piles**	7

Circular cell construction design & installation



Circular cell construction design & installation

9.1 Introduction

Cellular cofferdams are self-supporting gravity structures constructed using straight web sheet piles to form various shapes. The piles are interlocked and driven to form closed cells which are then filled with cohesionless material. To achieve continuity of the wall, the circular cells are connected together using fabricated junction piles and short arcs.

Provided that the material on which they are to be founded is solid they require only nominal penetration to be stable. Pile penetration will assist in the resistance of any lateral loads occurring during the construction phase in the vulnerable period before the fill has been placed and the cell has become inherently stable.

Cellular cofferdam structures are used to retain considerable depths of water or subsequently placed fill. They are commonly used as dock closure cofferdams, or to form quay walls and breakwaters. The straight web pile section and particularly the interlocks have been designed to resist the circumferential tension which is developed in the cells due to the radial pressure of the contained fill and at the same time permit sufficient angular deflection to enable cells of a practical diameter to be formed. In cellular construction no bending moments are developed in the sheet piles which enables the steel to be disposed in such a manner that the maximum tensile resistance is developed across the profile. The sections have therefore very little resistance to bending and are not suitable for normal straight sheet pile wall construction. Walings and tie rods are not required.

The design and construction of cellular cofferdams is very complex and further information is available from our Technical Department.

Circular cell construction design & installation

9.2 Straight web piling

Table 9.2 Tolerances for straight web piles to EN 10248 Part 2

Tolerances	AS 500
Mass	±5%
Length	±200mm
Height	-
Thickness	t, s > 8.5mm: ±6%
Width single pile	±2%
Width double pile	±3%
Straightness	0.2% of the length
Ends out of square	2% of pile width

9.2.1 Dimensions and properties for AS 500 Straight Web piles

Fig 9.2.1

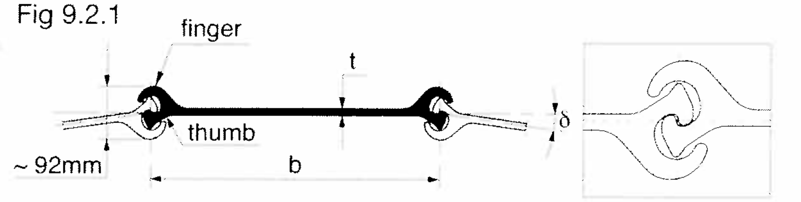


Table 9.2.1

Section	Nominal width* b mm	Web thickness t mm	Deviation angle δ °	Perimeter of a single pile cm	Steel section of a single pile cm^2	Mass per m of a single pile kg/m	Mass per m^2 of wall kg/m^2	Moment of inertia of a single pile cm^4	Section modulus of a single pile cm^3	Coating area*** m^2/m
AS 500-9,5	500	9.5	4.5**	138	81.3	63.8	**128**	168	46	0.58
AS 500-11,0	500	11.0	4.5**	139	90.0	70.6	**141**	186	49	0.58
AS 500-12,0	500	12.0	4.5**	139	94.6	74.3	**149**	196	51	0.58
AS 500-12,5	500	12.5	4.5**	139	97.2	76.3	**153**	201	51	0.58
AS 500-12,7	500	12.7	4.5**	139	98.2	77.1	**154**	204	51	0.58

Note: all straight web sections interlock with each other.
* The **effective width** to be taken into account for design purposes (lay-out) is **503 mm** for all AS 500 sheet piles.
** Max. deviation angle 4.0° for pile length > 20 m.
*** On both sides, excluding inside of interlocks.

9.3 Interlock strength

The interlock complies with EN 10248. Following interlock strength F_{max} can be achieved with a steel grade S 355 GP. However, higher steel grades are available.

Sheet pile	F_{max} [kN/m]
AS 500 - 9.5	3,000
AS 500 - 11.0	3,500
AS 500 - 12.0	5,000
AS 500 - 12.5	5,500
AS 500 - 12.7	5,500

For verification of the strength of piles, both yielding of the web and failure of the interlock should be considered. The allowable tension force T in the pile may be obtained by applying a safety factor, for example:

$$T = \frac{1}{\eta} R.$$

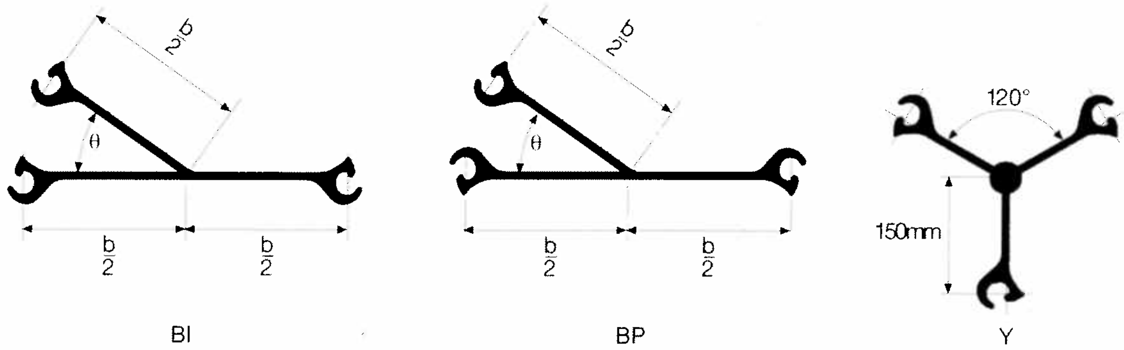
Circular cell construction design & installation

The magnitude of the safety factor depends on the calculation method and assumptions, the installation method and the function of the structure. When two different sections are used in the same section of wall, the lowest allowable tension force is to be taken into account. The value of $\eta = 2.0$ is currently used.

9.4 Junction piles

In general junction piles are assembled by welding in accordance with EN 12063.

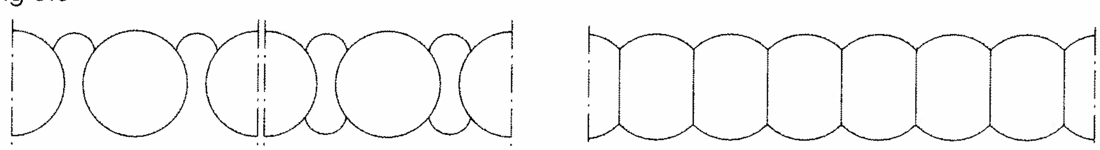
Fig 9.4



The connecting angle θ should be in the range from 30° to 45°.

9.5 Types of cell

Fig 9.5



Circular cells with 35° junction piles and one or two connecting arcs.

Diaphragm cells with 120° junction piles.

9.6 Bent piles

If deviation angles exceeding the values given in table 9.2.2 have to be attained, piles pre-bent in the mill may be used.

Fig 9.6

Circular cell construction design & installation

9.7 Equivalent width and ratio

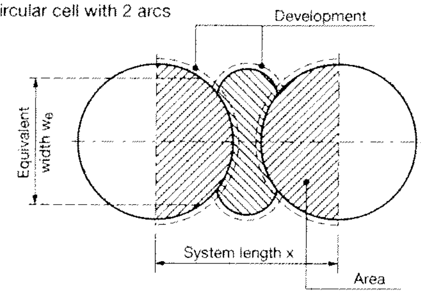
Fig 9.7

The **equivalent width w_e** which is required for stability verification, determines the geometry of the chosen cellular construction.

- for circular cells

$$w_e = \frac{\text{Area within 1 cell + Area within 2 (or 1) arc(s)}}{\text{System length x}}$$

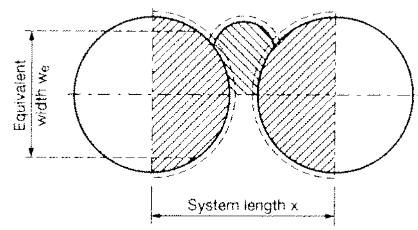
Circular cell with 2 arcs



The **ratio** shown on tables indicates how economical the chosen circular cell will be.

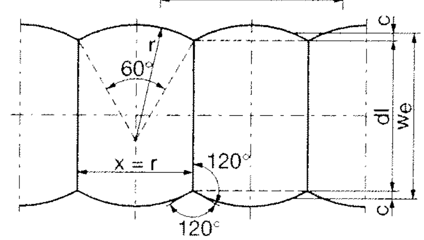
$$\text{Ratio} = \frac{\text{Development 1 cell + Development 2 (or 1) arc(s)}}{\text{System length x}}$$

Circular cell with 1 arc



- for diaphragm cells

w_e = diaphragm wall length (dl) + 2 · c



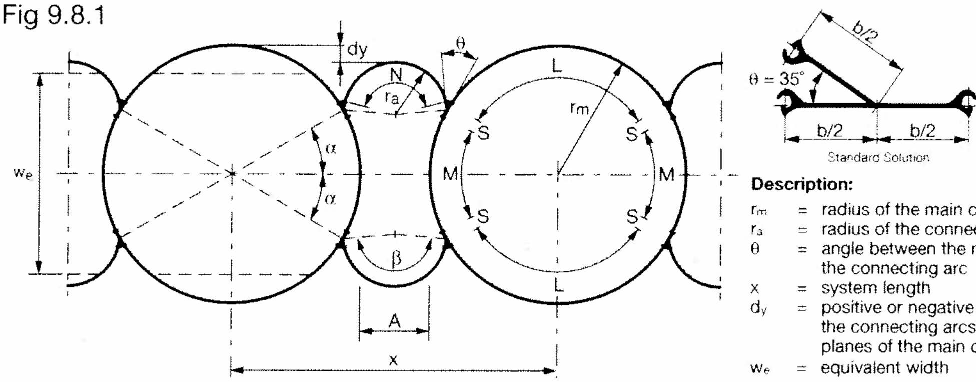
Circular cell construction design & installation

9.8 Geometry

9.8.1 Circular cells

Once the equivalent width has been determined, the geometry of the cells is to be defined. This can be done with the help of tables or with computer programs. Several solutions are possible for both circular and diaphragm cells with a given equivalent width.

Fig 9.8.1



Description:

- r_m = radius of the main cell
- r_a = radius of the connecting arcs
- θ = angle between the main cell and the connecting arc
- x = system length
- d_y = positive or negative offset between the connecting arcs and the tangent planes of the main cells
- w_e = equivalent width

Junction piles with angles θ between 30° and 45°, as well as θ = 90°, are possible on request. The following table shows a short selection of solutions for circular cells with 2 arcs and standard junction piles with θ = 35°.

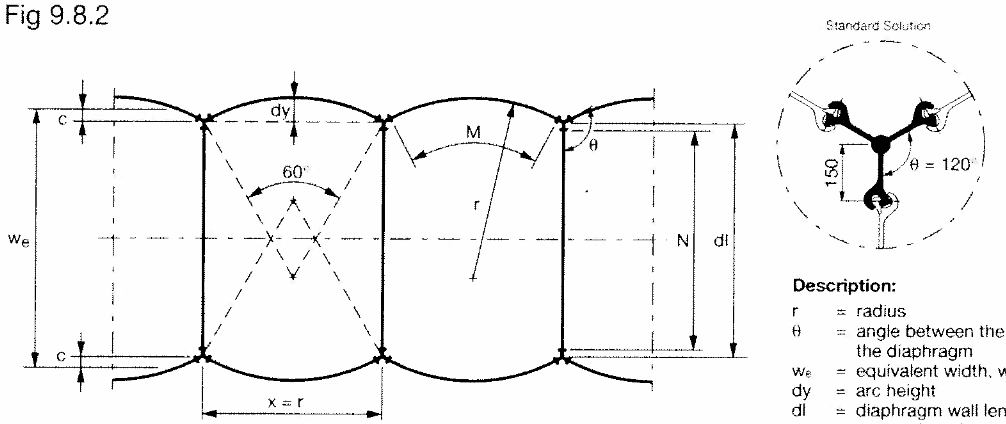
Table 9.8.1

Number of piles per						Geometrical values						Interlock deviation		Design values	
Cell				Arc	System							cell	arc	2 arcs	2 arcs
pcs.	L pcs.	M pcs.	S pcs.	N pcs.	pcs.	$d=2 \cdot r_m$ m	r_a m	x m	d_y m	α °	β °	δ_m °	δ_a °	w_e m	ratio
100	33	15	4	25	150	16.01	4.47	22.92	0.16	28.80	167.60	3.60	6.45	13.69	3.34
104	35	15	4	27	158	16.65	4.88	24.42	0.20	27.69	165.38	3.46	5.91	14.14	3.30
108	37	15	4	27	162	17.29	4.94	25.23	0.54	26.67	163.33	3.33	5.83	14.41	3.27
112	37	17	4	27	166	17.93	4.81	25.25	0.33	28.93	167.86	3.21	6.00	15.25	3.35
116	37	19	4	27	170	18.57	4.69	25.27	0.13	31.03	172.07	3.10	6.15	16.08	3.42
120	39	19	4	29	178	19.21	5.08	26.77	0.16	30.00	170.00	3.00	5.67	16.54	3.38
124	41	19	4	29	182	19.85	5.14	27.59	0.50	29.03	168.06	2.90	5.60	16.82	3.35
128	43	19	4	31	190	20.49	5.55	29.09	0.53	28.13	166.25	2.81	5.20	17.27	3.32
132	43	21	4	31	194	21.13	5.42	29.11	0.33	30.00	170.00	2.73	5.31	18.10	3.39
136	45	21	4	33	202	21.77	5.82	30.61	0.36	29.12	168.24	2.65	4.95	18.56	3.35
140	45	23	4	33	206	22.42	5.71	30.62	0.17	30.86	171.71	2.57	5.05	19.39	3.42
144	47	23	4	33	210	23.06	5.76	31.45	0.50	30.00	170.00	2.50	5.00	19.67	3.39
148	47	25	4	35	218	23.70	5.99	32.13	0.00	31.62	173.24	2.43	4.81	20.67	3.44
152	49	25	4	35	222	24.34	6.05	32.97	0.34	30.79	171.58	2.37	4.77	20.95	3.42
156	49	27	4	35	226	24.98	5.94	32.98	0.15	32.31	174.62	2.31	4.85	21.76	3.48
160	51	27	4	37	234	25.62	6.33	34.48	0.17	31.50	173.00	2.25	4.55	22.23	3.44
164	53	27	4	39	242	26.26	6.72	35.98	0.20	30.73	171.46	2.20	4.29	22.69	3.41
168	55	27	4	41	250	26.90	7.12	37.48	0.23	30.00	170.00	2.14	4.05	23.15	3.38
172	55	29	4	41	254	27.54	7.00	37.49	0.03	31.40	172.79	2.09	4.11	23.98	3.43
176	57	29	4	41	258	28.18	7.06	38.32	0.37	30.68	171.36	2.05	4.08	24.26	3.41
180	59	29	4	43	266	28.82	7.46	39.82	0.40	30.00	170.00	2.00	3.86	24.72	3.39
184	59	31	4	43	270	29.46	7.35	39.83	0.20	31.30	172.61	1.96	3.92	25.54	3.43
188	61	31	4	45	278	30.10	7.74	41.33	0.23	30.64	171.28	1.91	3.72	26.00	3.41

Circular cell construction design & installation

9.8.2 Diaphragm cells

Fig 9.8.2



Description:
r = radius
θ = angle between the arc and the diaphragm
w_e = equivalent width, with $w_e = dl + 2 \cdot c$
dy = arc height
dl = diaphragm wall length
x = system length

The two parts of the following table should be used separately depending on the required number of piles for the diaphragm wall and the arcs.

Table 9.8.2

Geometry of the diaphragms		Geometry of the arcs				
Number of piles N	Wall length dl m	Number of piles M	System length x m	Arc height dy m	c m	Interlock deviation arc δa °
11	5.83	11	5.57	0.75	0.51	5.17
13	6.84	13	6.53	0.87	0.59	4.41
15	7.85	15	7.49	1.00	0.68	3.85
17	8.85	17	8.45	1.13	0.77	3.41
19	9.86	19	9.41	1.26	0.86	3.06
21	10.86	21	10.37	1.39	0.94	2.78
23	11.87	23	11.33	1.52	1.03	2.54
25	12.88	25	12.29	1.65	1.12	2.34
27	13.88	27	13.26	1.78	1.20	2.17
29	14.89	29	14.22	1.90	1.29	2.03
31	15.89	31	15.18	2.03	1.38	1.90
33	16.90	33	16.14	2.16	1.46	1.79
35	17.91	35	17.10	2.29	1.55	1.69
37	18.91	37	18.06	2.42	1.64	1.60
39	19.92	39	19.02	2.55	1.73	1.52
41	20.92	41	19.98	2.68	1.81	1.44
43	21.93	43	20.94	2.81	1.90	1.38
45	22.94	45	21.90	2.93	1.99	1.32
47	23.94	47	22.86	3.06	2.07	1.26
49	24.95	49	23.82	3.19	2.16	1.21
51	25.95	51	24.78	3.32	2.25	1.16
53	26.96	53	25.74	3.45	2.33	1.12
55	27.97	55	26.70	3.58	2.42	1.08

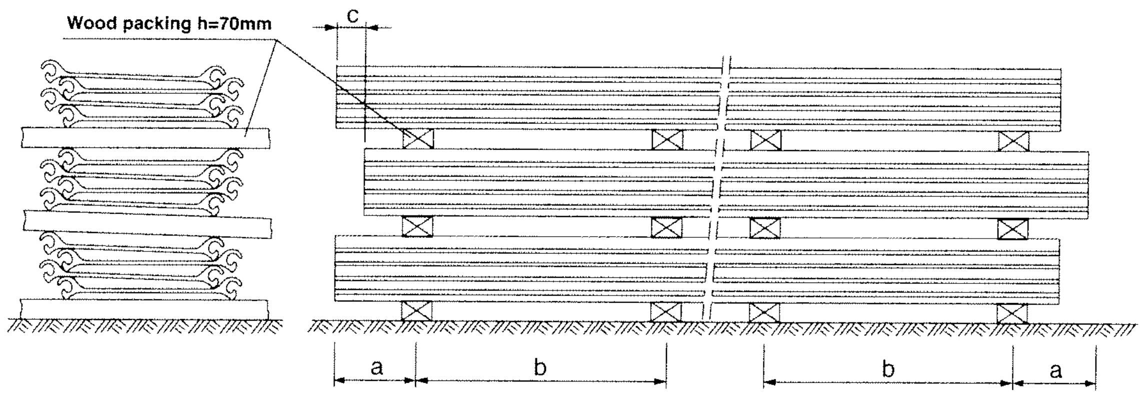
Circular cell construction design & installation

9.9 Handling straight web piles

Unlike piles designed to resist bending moments, straight-web sheet piles have low flexural stiffness, which means that care must be taken over their handling.

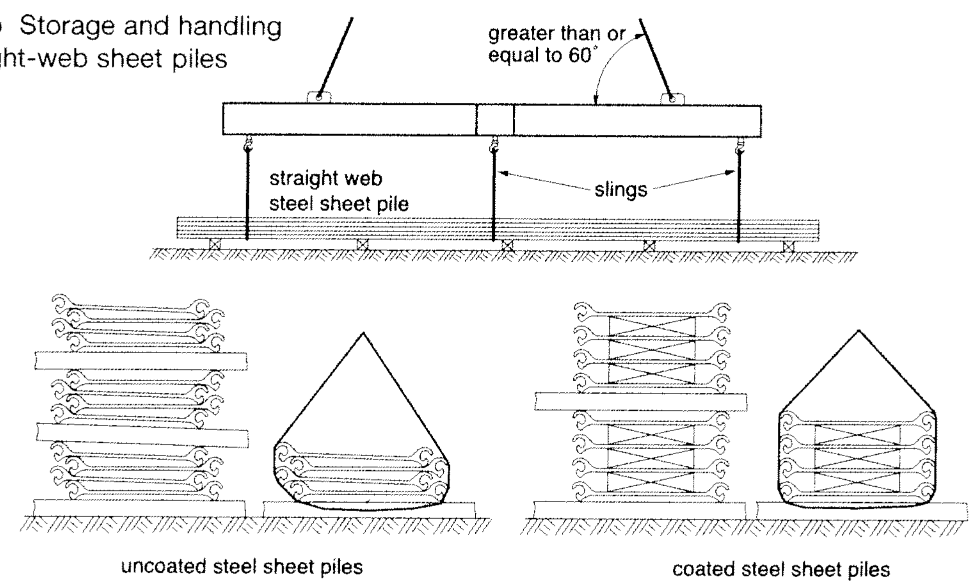
Incorrect storage could cause permanent deformation, making interlock threading difficult if not impossible. It is therefore vital to have a sufficient number of wooden packing pieces between each bundle of stacked sheet piles, and to position these pieces above each other to limit the risk of deformation.

Fig 9.9a Storage of straight-web sheet piles



max. bundle weight: 7.5 tonnes
Overhang "a" less than 1.5 m
Spacing of packings "b" less than 4.0 m
Offset of bundle "c" not less than 0.15 m
Wood packings to be aligned in the vertical plane

Fig 9.9b Storage and handling of straight-web sheet piles



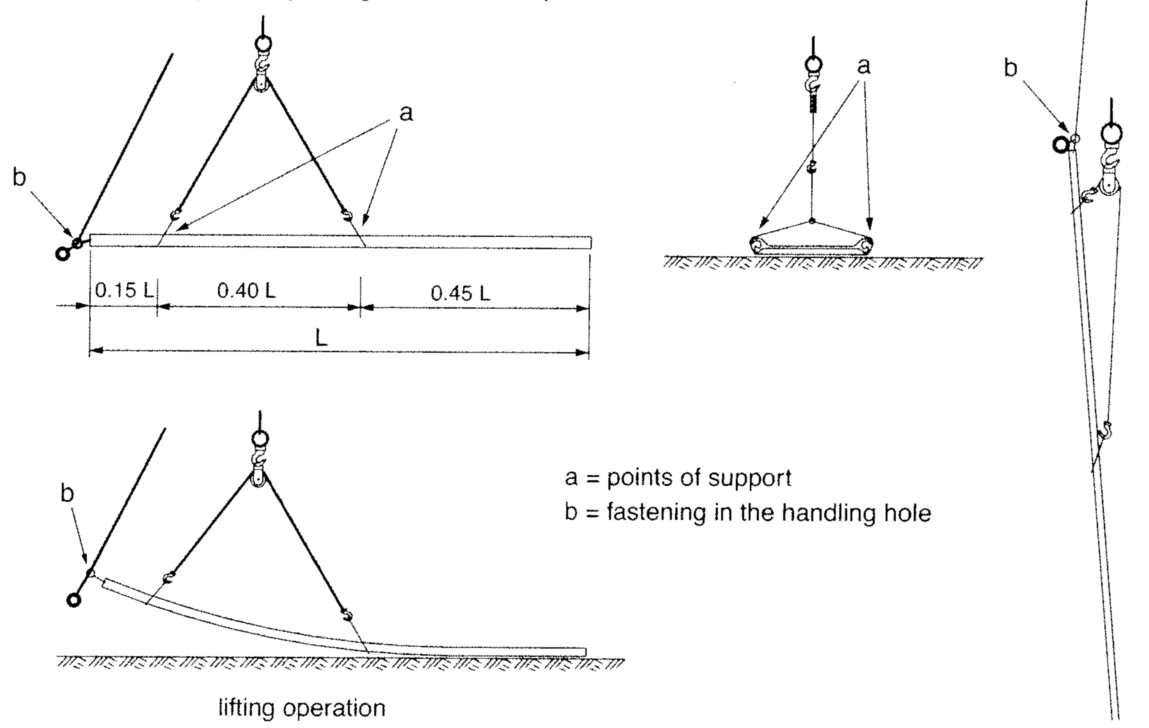
Circular cell construction design & installation

When sheet piles have to be moved from the horizontal storage position to another storage location, lifting beams or brackets made from pile sections threaded into the interlocks prior to lifting should be used.

When pitching piles up to 15 m long into the vertical position, only one point of support near the top (the handling hole) is necessary.

Straight-web sheet piles more than 15 m long should be lifted at two or even three points, in order to avoid plastic distortion.

Fig 9.9c Lifting of long straight-web sheet piles



APPENDIX F: PRELIMINARY DESIGN: CIRCULAR SHEET PILE CELLS

PARAMOUNT	DESIGN CALCULATIONS SHEET	DATE:	Mar 15/2010
		PAGE:	1 of 13

PROJECT:	St. Lawrence Marine Terminal	DOCUMENT NO.:	DC1-8700-07-008
PROJECT NO.:	8700-07	REVISION:	0
ITEM:	Sheet Pile Cell Option - Global Stability	PREPARED BY:	Steven Greeley

» Preliminary design based on global stability to determine sizing of structure

» Design is based on guidelines presented in:
① Pile Buck Steel Sheet Piling Design Manual (1987)
② Handbook of Port and Harbour Engineering: Geotechnical & Structural Aspects (Tsinker, 1997)
③ Acelor Mittal Piling Handbook, 8th ed. (2008)

» Sheet Pile Properties based on material provided by Profilarbed
- General Steel Sheet Piling Catalogue (2004)

» Global stability checks
① sliding
② overturning
③ slipping between fill & cell
④ shear at cell centerline
⑤ bursting
⑥ horizontal shear

» No Geotechnical data available other than bedrock conditions exist throughout site
- assume backfill material is a dense-angular grained silty sand
- "Principles of Geotechnical Engineering" 6th ed. Braja M. Das (2006)

$\gamma_d = 19\,kN/m^3$ (Table 3.2)
$e = 0.4$

$\rho_d = 19kN/m^3 \times 1kg/9.81N \times 1000N/kN = 1937\,kg/m^3$
$G_s = \frac{\rho_d(1+e)}{\rho_w} = \frac{1937(1+0.4)}{1000} = 2.71$

$\rho_{sat} = \frac{(G_s+e)\rho_w}{1+e} = \frac{(2.71+0.4)\,1000\,kg/m^3}{(1+0.4)} = 2223\,kg/m^3$
$\gamma_{sat} = 2223kg/m^3 \times 9.81N/kg \times 1kN/1000N = 21.8\,kN/m^3$

$\therefore \gamma_d = 19kN/m^3$; $\gamma_{sat} = 21.8kN/m^3$; $\gamma_w = 9.8kN/m^3$; $\gamma' = 12.0kN/m^3$

» $\phi' = 30°$ (internal friction angle) - assumed equal to sea bed slope
$\delta = 20°$ (Friction angle between steel & soil) = $2/3\phi'$
$\theta = 0°$ (Slope of vertical face of sheet pile cofferdam)
$\alpha = 0°$ (Slope of retained material)

» Ka - calculated using EQN 12.19 from Das (Coulomb's Active Pressure Theory)
- using Table 12.5 for $\phi' = 30°$ & $\delta' = 20°$ → Ka = 0.2973

» $\tan\phi = 0.577$ → Pile Buck Manual recommends for conservative reasons to take this value as 0.5

» excerpts from DAS are found on page 2

Table 3.2 Void Ratio, Moisture Content, and Dry Unit Weight for Some Typical Soils in a Natural State

Type of soil	Void ratio, e	Natural moisture content in a saturated state (%)	Dry unit weight, γ_d lb/ft^3	Dry unit weight, γ_d kN/m^3
Loose uniform sand	0.8	30	92	14.5
Dense uniform sand	0.45	16	115	18
Loose angular-grained silty sand	0.65	25	102	16
Dense angular-grained silty sand	0.4	15	121	19
Stiff clay	0.6	21	108	17
Soft clay	0.9–1.4	30–50	73–93	11.5–14.5
Loess	0.9	25	86	13.5
Soft organic clay	2.5–3.2	90–120	38–51	6–8
Glacial till	0.3	10	134	21

$$\text{Density} = \rho = \frac{(1+w)G_s\rho_w}{1+e} \qquad (3.21)$$

$$\text{Dry density} = \rho_d = \frac{G_s\rho_w}{1+e} \qquad (3.22)$$

$$\text{Saturated density} = \rho_{sat} = \frac{(G_s+e)\rho_w}{1+e} \qquad (3.23)$$

$$K_a = \frac{\cos^2(\phi' - \theta)}{\cos^2\theta\cos(\delta' + \theta)\left[1 + \sqrt{\dfrac{\sin(\delta' + \phi')\sin(\phi' - \alpha)}{\cos(\delta' + \theta)\cos(\theta - \alpha)}}\right]^2} \qquad (12.69)$$

Table 12.5 Values of K_a [Eq. (12.69)] for $\theta = 0°$, $\alpha = 0°$

↓ ϕ' (deg)	δ' (deg) → 0	5	10	15	20	25
28	0.3610	0.3448	0.3330	0.3251	0.3203	0.3186
30	0.3333	0.3189	0.3085	0.3014	0.2973	0.2956
32	0.3073	0.2945	0.2853	0.2791	0.2755	0.2745
34	0.2827	0.2714	0.2633	0.2579	0.2549	0.2542
36	0.2596	0.2497	0.2426	0.2379	0.2354	0.2350
38	0.2379	0.2292	0.2230	0.2190	0.2169	0.2167
40	0.2174	0.2089	0.2045	0.2011	0.1994	0.1995
42	0.1982	0.1916	0.1870	0.1841	0.1828	0.1831

PARAMOUNT	DESIGN CALCULATIONS SHEET	DATE:	Mar 15/2010
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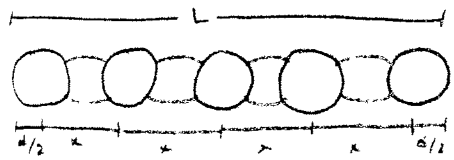
PROJECT:	St. Lawrence Marine Terminal	DOCUMENT NO.:	DC1-8700-07-008
PROJECT NO.:	8700-07	REVISION:	0
ITEM:	Sheet Pile Cell option - Global Stability	PREPARED BY:	Steven Greeley

» Determined smallest diameter cell satisfying global stability checks was:

$d = 23.70m$
$X = 32.13m$
$We = 20.67m$

↳ req'd length = 265m



$\therefore L_{req'd} = \# X + d \Rightarrow \# = \frac{L_{req'd} - d}{x} = \frac{265m - 23.70m}{32.13m} = 7.51$ ∴ (8) x's → 9 cells & 8 arcs

$L_{TOTAL} = 8x + d = 8(32.13m) + 23.70m = 280.74m$

Piles/cell

cell ⇒	L ⇒ 94	× 9 cells	846
	M ⇒ 50		450
	S ⇒ 4		36
Arc ⇒	N ⇒ 70	× 8 arcs	560
			1892 piles

→ find most economical solⁿ

d	x	We	# = (Lreq'd − d)/x	# Piles	
24.34	32.97	20.45	7.30 (7<#<8) ∴ 23.70m better b/c less piles		
24.98	32.98	21.76	7.28 " " " " "		
25.62	34.48	22.33	6.94 ∴ $L_{TOTAL} = 7(34.48) + 25.62 = 266.98m$	L⇒102; M⇒54; S⇒4; N⇒74	= 1798 < 1892
26.26	35.98	22.69	6.63 ∴ No Good ⇒ 8 cells w/more pcs.		
26.90	37.48	23.15	6.35 " " " " "		
27.54	37.49	23.98	6.33 " " " " "		
28.18	38.32	24.26	6.18 " " " " "		
28.82	39.82	24.72	5.93 ∴ $L_{TOTAL} = 6(39.82) + 28.82 = 267.74m$	L⇒118; M⇒58; S⇒4; N⇒86	= 1776 < 1798
29.46	39.83	25.54	5.91 ∴ No Good ⇒ 7 cells w/more pcs		
30.10	41.33	26.00	5.68 " " " " "		

PARAMOUNT	DESIGN CALCULATIONS SHEET	DATE:	Mar 15/2010
		PAGE:	4 of 13

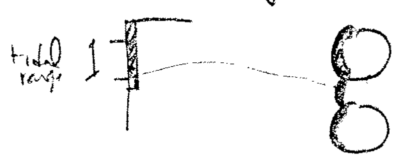
PROJECT:	St. Lawrence Marine Terminal	DOCUMENT NO.:	DC1-8700-07-008
PROJECT NO.:	8700-07	REVISION:	0
ITEM:	Sheet Pile Cell Option - Global Stability	PREPARED BY:	Steven Greeley

» Client Meeting - March 9/2010

- (2) options presented - preferred larger diameter cells
  - ↳ less total piles
  - ↳ less template moves although larger template

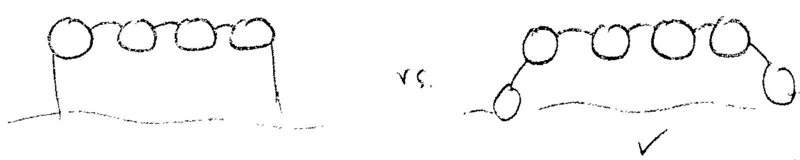
  ∴ 28.82m dia. cells selected

- raised questions on how to incorporate impact loading from ice
  - ↳ Voisey's used steel beams
  - ↳ concrete lining in tidal range



→ at detailed design stage

- whether structure should wraparound or use armor layer sides

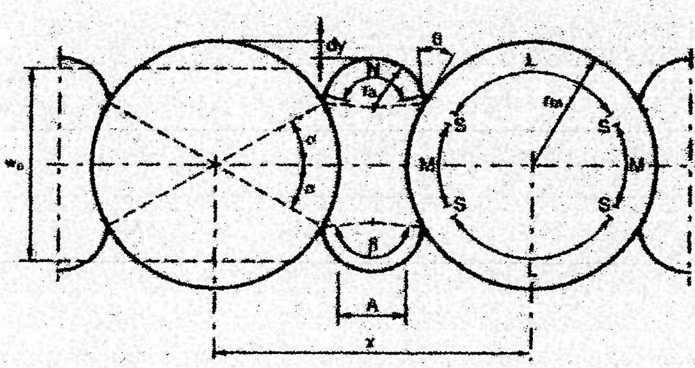


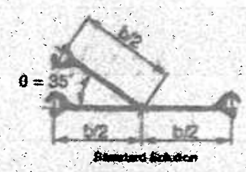
→ flaring reduces high frictional forces due to ice (consulted with Dr. Bruneau)

STRAIGHT WEB SECTIONS

CIRCULAR CELLS

Once the equivalent width has been determined, the geometry of the cells is to be defined. This can be done with the help of tables or with computer programs. Several solutions are possible for both circular and diaphragm cells with a given equivalent width.





Description:

r_m = radius of the main cell
r_a = radius of the connecting arcs
θ = angle between the main cell and the connecting arc
x = system length
d_y = positive or negative offset between the connecting arcs and the tangent planes of the main cells
W_e = equivalent width

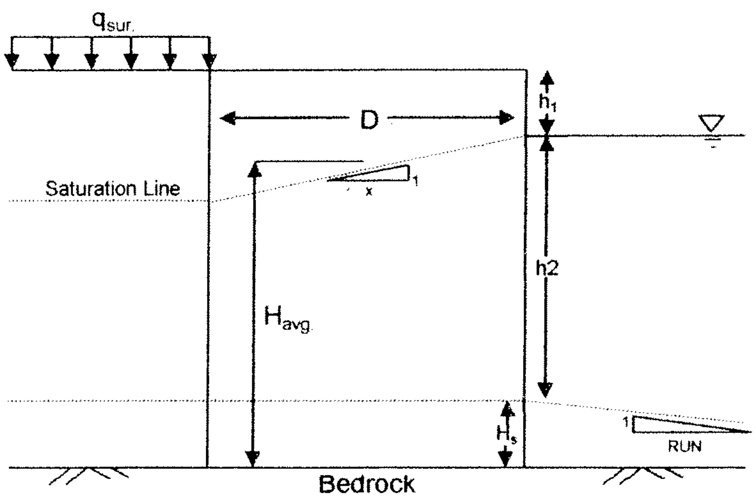
Junction piles with angles θ between 30° and 45°, as well as θ = 90°, are possible on request. The following table shows a short selection of solutions for **circular cells with 2 arcs** and standard junction piles with θ = 35°.

Nb. of piles per						**Geometrical values**								**Design values**	
												Interlock deviation			
Cell				Arc	System							cell	arc	**2 arcs**	2 arcs
	L	M	S	N		$d=2 \cdot r_m$	r_a	x	d_y	α	β	δ_m	δ_a	w_e	ratio
pcs.	pcs.	pcs.	pcs.	pcs.	pcs.	m	m	m	m	°	°	°	°	m	
100	33	15	4	25	150	16.01	4.47	22.92	0.16	26.80	167.60	3.60	6.45	**13.69**	3.34
104	35	15	4	27	158	16.65	4.88	24.42	0.20	27.69	165.38	3.46	5.91	**14.14**	3.30
108	37	15	4	27	162	17.29	4.94	25.23	0.54	26.67	163.33	3.33	5.83	**14.41**	3.27
112	37	17	4	27	166	17.93	4.81	25.25	0.33	28.93	167.86	3.21	6.00	**15.25**	3.35
116	37	19	4	27	170	18.57	4.69	25,27	0.13	31,03	172.07	3.10	6.15	**16.08**	3.42
120	39	19	4	29	178	19.21	5.08	26.77	0.16	30.00	170.00	3.00	5.67	**16.54**	3.38
124	41	19	4	29	182	19.85	5.14	27.59	0.50	29.03	168.06	2.90	5.60	**16.82**	3.35
128	43	19	4	31	190	20.49	5.65	29.09	0.53	28.13	166.25	2.81	5.20	**17.27**	3.32
132	43	21	4	31	194	21.13	5.42	29.11	0.33	30.00	170.00	2.73	5.31	**18.10**	3.39
136	45	21	4	33	202	21.77	5.82	30.61	0.36	29.12	168.24	2.65	4.95	**18.56**	3.35
140	45	23	4	33	206	22.42	5.71	30.62	0.17	30.86	171.71	2.57	5.05	**19.39**	3.42
144	47	23	4	33	210	23.06	5.76	31.45	0.50	30.00	170.00	2.50	5.00	**19.67**	3.39
148	47	25	4	35	218	23.70	5.99	32.13	0.00	31.62	173.24	2.43	4.81	**20.67**	3.44
152	49	25	4	35	222	24.34	6.05	32.97	0.34	30.79	171.58	2.37	4.77	**20.95**	3.42
156	49	27	4	35	226	24.98	5.94	32.98	0.15	32.31	174.62	2.31	4.85	**21.76**	3.48
160	51	27	4	37	234	25.62	6.33	34.48	0.17	31.50	173.00	2.25	4.55	**22.33**	3.44
164	53	27	4	39	242	26.26	6.72	35.98	0.20	30.73	171.46	2.20	4.29	**22.69**	3.41
168	55	27	4	41	250	26.90	7.12	37.48	0.23	30.00	170.00	2.14	4.05	**23.15**	3.38
172	55	29	4	41	254	27.54	7.00	37.49	0.03	31.40	172.79	2.09	4.11	**23.98**	3.43
176	57	29	4	41	258	28.16	7.06	38.32	0.37	30.68	171.36	2.05	4.06	**24.26**	3.41
180	59	29	4	43	266	28.82	7.46	39.82	0.40	30.00	170.00	2.00	3.86	**24.72**	3.39
184	59	31	4	43	270	29.46	7.35	39.83	0.20	31.30	172.61	1.96	3.92	**25.54**	3.43
188	61	31	4	45	278	30.10	7.74	41.33	0.23	30.64	171.28	1.91	3.72	**26.00**	3.41

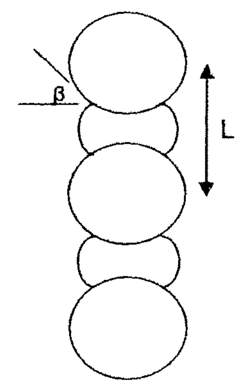
PROJECT:	St. Lawrence Marine Terminal	DOCUMENT NO.:	DC1-8700-07-008
PROJECT NO.:	8700-07	REVISION:	0
ITEM:	Sheet Pile Cell Option - Global Stability	PREPARED BY:	Steven Greeley

Design of Cofferdams Placed Directly on Bedrock

Profile



Plan



Data Input

Symbol	Value	Units	Description
γ_d	19.0	kN/m^3	Dry unit weight of soil
γ_{sat}	21.8	kN/m^3	Saturated unit weight of soil
γ_w	9.8	kN/m^3	Unit weight of water
γ'	12.0	kN/m^3	Effective unit weight of soil
φ'	30	°	Internal friction angle
δ'	20	°	Friction angle between sand and steel
θ	0	°	Slope of batter on retainig wall
α	0	°	Slope of retained earth
K_a	0.297		Coulomb's Active Pressure Coefficient
$\tan\varphi'$	0.50		Cofficient of Friction - sand on rock
$\tan\delta'$	0.40		Cofficient of Friction - sand on steel
f	0.30		Cofficient of Friction - steel on steel
h_1	7.0	m	Cofferdam height above water
h_2	22.0	m	Design water depth at cofferdam face
H_s	0.0	m	Depth of embedment
H_{avg}	22.0	m	Average height of saturated soil within sheet pile cell
D	28.82	m	Cell Diameter
x	0		Corresponding run for a unit drop in saturation level
L	39.82	m	Distance between cell centroids (from Profilarbed)
w_e	24.72	m	Effective width of cells (from Profilarbed)
q_{sur}	20	kPa	Surcharge

Summary

	FS against:	Target	Actual
Global Stability	Sliding	1.5	2.40
	Overturning	1.5	4.53
	Slipping	1.5	1.86
	Cell Centerline	1.5	1.99
	Bursting (main cell)	1.5	1.50
	Bursting (connections)	1.5	1.50
	Horizontal Shear	1.5	2.75

Note: The bursting stability is satisfied by selcting a sheet pile with sufficient strength.

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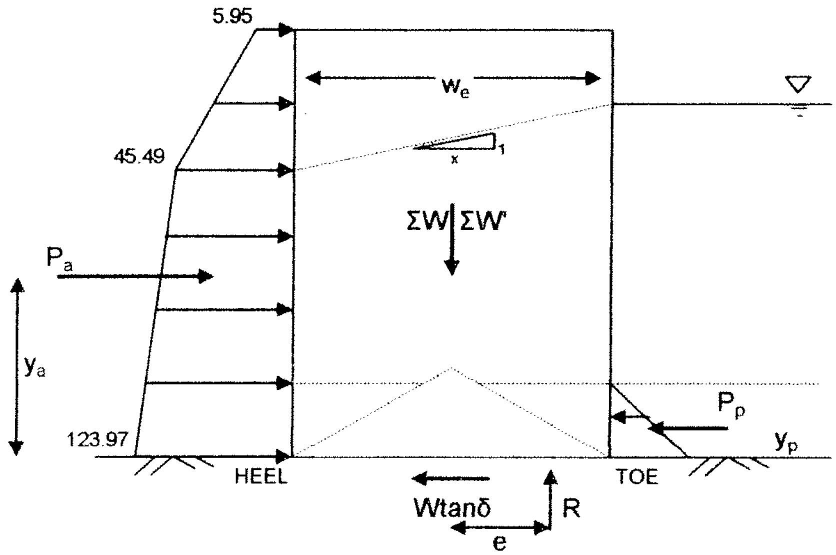
PROJECT:	St. Lawrence Marine Terminal	DOCUMENT NO.:	DC1-8700-07-008
PROJECT NO.:	8700-07	REVISION:	0
ITEM:	Sheet Pile Cell Option - Global Stability	PREPARED BY:	Steven Greeley

Design of Cofferdams Placed Directly on Bedrock



Notes:

1. ΣW' omits the weight of the prism shown in the figure.
2. Passive Pressures are currently omitted from calculations.
3. All FS checks are currently done neglecting load factors.
4. Neglect weight of sheet piles.

$FS_{sliding}$

Item	Value	
P_a	2044	kN/m
y_a	10.65	m
Wtanδ	4907	kN/m
FS	**2.40**	

$FS_{overturning}$ & Bearing Check

Item	Value	
M_o	21768	kN-m/m
ΣW'	7981	kN/m
M_r	98640	kN-m/m
FS	**4.53**	
e	2.73	m
$\sigma_{B\,(toe)}$	537	kPa
$\sigma_{B\,(heel)}$	109	kPa

$FS_{slipping}$

Item	Value
FS	**1.86**

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PROJECT:	St. Lawrence Marine Terminal	DOCUMENT NO.:	DC1-8700-07-008
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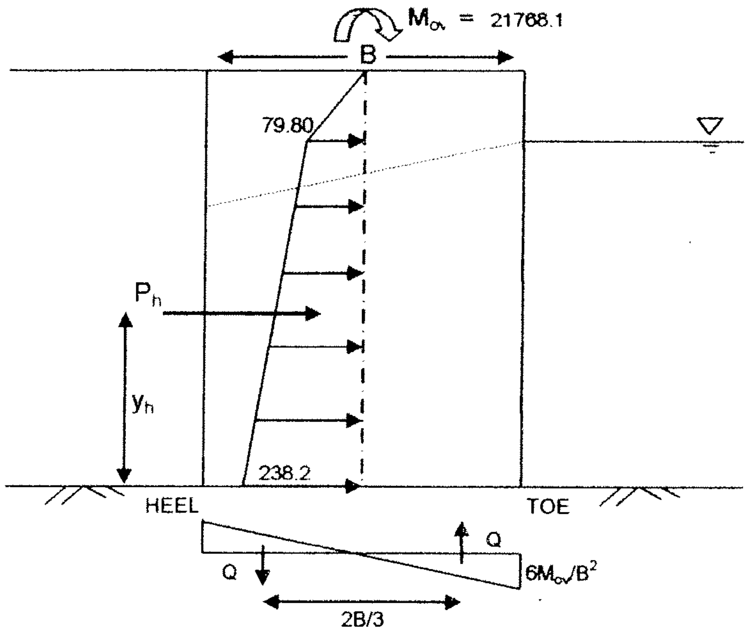
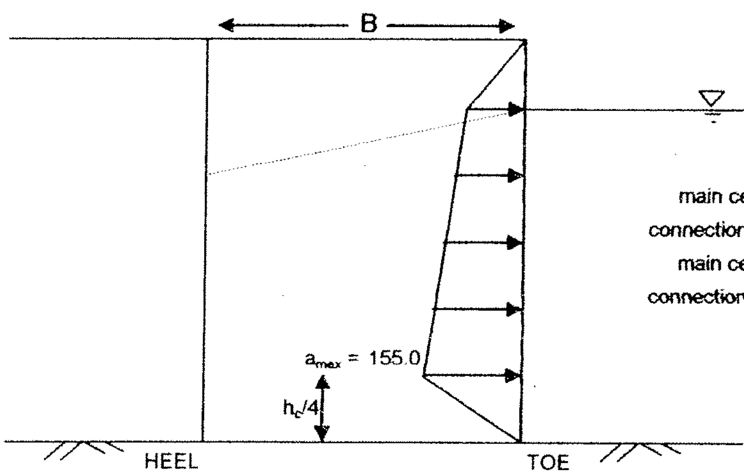
ITEM:	Sheet Pile Cell Option - Global Stability	PREPARED BY:	Steven Greeley

Design of Cofferdams Placed Directly on Bedrock



$FS_{cell\ centerline}$

Item	Value	
Q	1321	kN/m
K	0.600	
P_h	3777	kN/m
y_h	10.29	m
R_s	2181	kN/m
P_i	2044	kN/m
T	444	kN/m
FS	**1.99**	



$FS_{bursting}$

	Item	Value	
	K_i	0.50	
	hc/4	7.25	m
main cell	T_{max}	2233.55	kN/m
connections	T_{max}	3563.46	kN/m
main cell	Min.	AS 500-11.0	
connections	Min.	AS 500-12.5	

Arcelor (Profilarbed catalogue) recommends a safety factor of ~~2~~ 2.5 for interlock resistance and 1.5 for yielding of the web.

Pile Section	Interlock Strength
AS 500-9.5	3000
AS 500-11.0	3500
AS 500-12.0	5000
AS 500-12.5	5500
AS 500-12.7	5500

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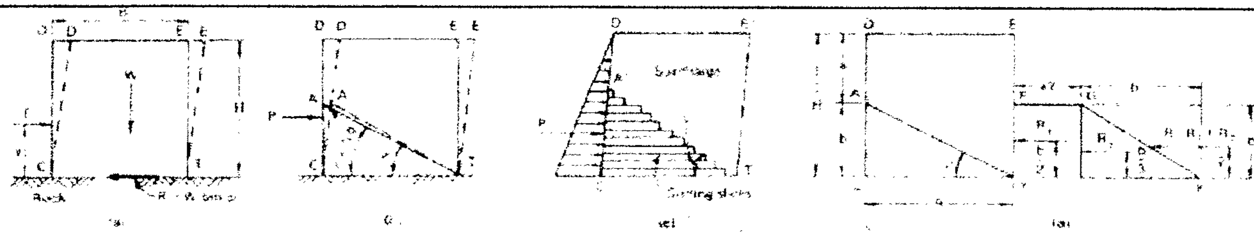
PROJECT:	St. Lawrence Marine Terminal	**DOCUMENT NO.:**	DC1-8700-07- 008
PROJECT NO.:	8700-07	**REVISION:**	0
ITEM:	Sheet Pile Cell Option - Global Stability	**PREPARED BY:**	Steven Greeley



FS $_{\text{cell centerline}}$

Item	Value	
a	14.73	m
b	14.272	m
R_1	2522.38	kN/m
R_2	2444.31	kN/m
M_r	29628.30	kN-m/m
$M_{i\,(a)}$	36571.30	kN-m/m
$M_{i\,(b)}$	30317.89	kN-m/m
FS	**2.75**	

Note:
There are two methods for calculating M_i in Tsinker however they both do not give the same result. The value which generates the lowest FS has been used.

$R = W\tan\phi = \gamma_1 B H \tan\phi$ where $H = a + b$, $B = B/\tan\phi$

$\therefore R = \gamma'(ab + b^2) \rightarrow b = B\tan\phi = W_e\tan\phi = 24.72\,m \tan 30 = 14.27\,m$

$a = H - b = 29\,m - 14.27\,m = 14.73\,m$

$R_1 = \gamma' a b = 12\,kN/m^3 (14.73\,m)(14.27\,m) = 2522.38\,kN/m$

$R_2 = \gamma' b^2 = 12\,kN/m^3 (14.27\,m)^2 = 2444.31\,kN/m$

$\Rightarrow M_r = R_1(b/2) + R_2(b/3)$
$= 2522.38\,kN/m(14.27\,m/2) + (2444.31\,kN/m)(14.27\,m/3)$
$= 29{,}628.30\,kN{\cdot}m/m$

$\Rightarrow M_i$ = Moment due to interlock friction

$M_{i(a)} = 2T'B/L = 2(2044.10\,kN/m)(14.41\,m)(24.72\,m)/39.82\,m = 36{,}571.30\,kN{\cdot}m/m$

$M_{i(b)} = 2P_a f W_e = 2(2044.10\,kN/m)(0.3)(24.72\,m) = 30317.89\,kN{\cdot}m/m$

$$\gg FS_{TILT} = \frac{M_r + M_i}{M_o} = \frac{(29{,}628.30 + 30{,}317.89)\,kN{\cdot}m/m}{21{,}769.67\,kN{\cdot}m/m} = 2.75 > 1.5 \quad (\therefore OK)$$

PARAMOUNT	DESIGN CALCULATIONS SHEET	DATE:	Mar 15/2010
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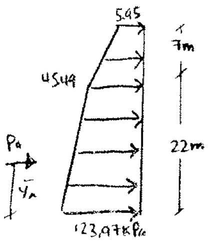
PROJECT:	St. Lawrence Marine Terminal	DOCUMENT NO.:	DCJ-8700-07-008
PROJECT NO.:	8700-07	REVISION:	0
ITEM:	Sheet Pile Cell option - Global Stability	PREPARED BY:	Steven Greeley

① Sliding On foundation

Driving Forces: $q_{sur} K_a + \gamma_d h_1 K_a + \gamma'(h_2 + H_s) K_a = K_a[q_{sur} + \gamma_d h_1 + \gamma'(h_2 + H_s)]$

$= 0.2973[20\,kPa + 19\,kN/m^3(7m) + 12\,kN/m^3(22m)]$

$= 123.97\,kPa$



$P_a = 5.95\,kPa(29m) + \tfrac{1}{2}(45.49\,kPa - 5.95\,kPa)(7m) + (45.49\,kPa - 5.95\,kPa)(22m) + \tfrac{1}{2}(123.97\,kPa - 45.49\,kPa)(22m)$

$= 172.55\,kN/m + 138.39\,kN/m + 869.88\,kN/m + 863.28\,kN/m$

$= 2044.10\,kN/m$

$\bar{y}_A = \dfrac{\Sigma yA}{\Sigma A} = \dfrac{(29/2\,m)(172.55\,kN/m) + (22m + 7/3\,m)(138.39\,kN/m) + (22m/2)(869.88\,kN/m) + (\frac{22m}{3})(863.28\,kN/m)}{2044.10\,kN/m}$

$= \dfrac{2501.98\,kN + 3367.49\,kN + 9568.68\,kN + 6330.72\,kN}{2044.10\,kN/m}$

$= 21768.87\,kN / 2044.10\,kN/m$

$\bar{y}_A = 10.65m$

Resisting Forces: $W\tan\delta = W_e(\gamma_d h_1 + \gamma'(h_2 + H_s))\tan\delta$

$= 24.72m\,(19\,kN/m^3(7m) + 12\,kN/m^3(22m))\,(\tan 30° \approx 0.5)$

$= 24.72m(397\,kN/m^2)(0.5)$

$W\tan\delta = 4906.92\,kN/m$

$\Rightarrow FS_{sliding} = \dfrac{W\tan\delta}{P_a} = \dfrac{4906.92\,kN/m}{2044.10\,kN/m} = 2.40 > 1.5$ (∴ OK)

② Overturning Stability

Overturning Moment, $M_o = P_a \bar{y}_A = 2044.10\,kN/m\,(10.65m) = 21{,}769.67\,kN \cdot m/m$

Resisting moment, $M_r = \Sigma W'(W_e/2)$ where $\Sigma W' = \Sigma W - \tfrac{1}{4} W_e^2 \gamma' = \dfrac{4906.92\,kN/m}{0.5} - \dfrac{1}{4}(24.72m)^2(12\,kN/m^3)$

$\Sigma W' = 7980.60\,kN/m$

$M_r = 7980.60\,kN/m\,(24.72m/2) = 98{,}640.28\,kN \cdot m/m$

$\Rightarrow FS_{overturning} = \dfrac{M_r}{M_o} = \dfrac{98{,}640.28\,kN \cdot m/m}{21{,}769.67\,kN \cdot m/m} = 4.53 > 1.5$ (∴ OK)

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PROJECT:	St. Lawrence Marine Terminal	DOCUMENT NO.:	DC1-8700-07-008
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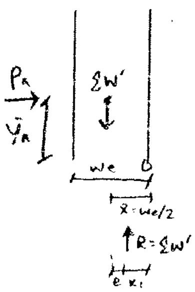
ITEM:	Sheet Pile Cell Option - Global Stability	PREPARED BY:	Steven Greeley

③ Bearing Check

* want to avoid negative pressures at structure base so ensure $\Sigma W'$ lies within middle 1/3 of base



$$\circlearrowleft + \Sigma M_O = -P_a\bar{y}_a + \Sigma W'\bar{x} - R x_1 = 0 \Rightarrow x_1 = \frac{\Sigma W'\bar{x} - P_a\bar{y}_a}{R} = \frac{M_e - M_o}{\Sigma W} = \frac{\Sigma M}{\Sigma W'}$$

$$\therefore x_1 = \frac{(98{,}640.28 - 21{,}769.67)\ kN\cdot m/m}{7980.60\ kN/m}$$

$$x_1 = 9.63m$$

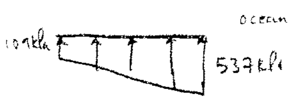
$$\therefore e = W_e/2 - x_1 = 24.72m/2 - 9.63m = 2.73m$$

$$W_e/6 = 24.72m/6 = 4.12m > e \quad \therefore R \text{ lies in middle one-third}$$

$$\sigma_{B(toe)} = \frac{\Sigma W'}{W_e}\left[1 + \frac{6e}{W_e}\right] = \frac{7980.60kN/m}{24.72m}\left[1 + \frac{6(2.73m)}{24.72m}\right] = 537kPa$$

$$\sigma_{B(heel)} = \frac{\Sigma W'}{W_e}\left[1 - \frac{6e}{W_e}\right] = \frac{7980.60kN/m}{24.72m}\left[1 - \frac{6(2.73m)}{24.72m}\right] = 109kPa$$

* Structure rests on bedrock so bearing pressure not an issue



④ Slipping Between Sheet Piling and Cell Fill

* Fill material located both inside and outside pile material $\therefore$ force acts on 2 sides so result is multiplied by 2.

$$FS_{slipping} = \frac{W_e P_a \tan\delta}{P_a \bar{y}_a} = \frac{W_e \tan\delta}{\bar{y}_a} = \frac{24.72m(0.4)}{10.65m}(2) = 1.86 > 1.5 \ (\therefore OK)$$

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PROJECT:	St. Lawrence Marine Terminal	DOCUMENT NO.:	DC1-8700-07-008
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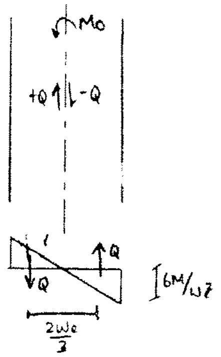
ITEM:	Sheet Pile Cell Option - Global Stability	PREPARED BY:	Steven Greeley

⑤ Internal Stability at Cell Centerline



Q = total shearing force per length of structure

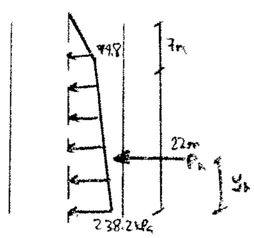
$Q = \frac{1}{2}(W_e/2)(6M/W_e^2) = \frac{3M_o}{2W_e} = \frac{3(21{,}769.67\,kN{\cdot}m/m)}{2(24.72m)} = 1321.98\,kN/m$

Rs = vertical resisting shear within cell

$R_s = P_n \tan\phi$, P_h = soil horizontal pressure within cell (neglect surcharge)
ϕ = angle of internal friction

↳ P_h calculation

$K = \frac{\cos^2\phi}{2-\cos^2\phi} = \frac{(\cos 30)^2}{2-(\cos 30)^2} = 0.600$



$\sigma_h = K(\gamma_d h_1 + \gamma'(h_2+H_s)) = 0.6(19\,kN/m^3(7m) + 12\,kN/m^3(22m)) = 238.2\,kPa$

$P_h = \frac{1}{2}(79.8kPa)(7m) + (79.8kPa)(22m) + (\frac{1}{2})(238.2-79.8)(22m)$
$= 279.3\,kN/m + 1755.6\,kN/m + 1742.4\,kN/m$
$= 3777.30\,kN/m$

$\bar{y}_h = \frac{\Sigma yA}{\Sigma A} = \frac{(22m + 7/3m)(279.3kN/m) + (22m/2)(1755.6kN/m) + (22m/3)(1742.4kN/m)}{3777.30\,kN/m}$

$= \frac{6796.30\,kN + 19311.60\,kN + 12777.60\,kN}{3777.30\,kN/m}$

$= 10.29m$

$\therefore R_S = 3777.30\,kN/m(\tan 30°) = 2180.83\,kN/m$

↳ T = resistance to shear due to friction in cell interlocks

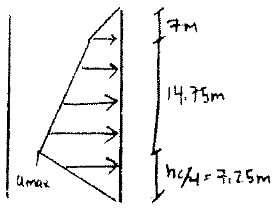
$T = \frac{2T'f_i}{L}$ $\quad f_i = 0.3$ (friction coefficient - steel on steel)
$T' = P_i R \rightarrow P_i = P_a = 2044.10\,kN/m$
$R = D/2 = 28.82m/2 = 14.41m$
$L = 39.82m$

$T = 2(2044.10kN/m)(14.41m)(0.3)/39.82m = 443.83\,kN/m$

>> $FS_{cell\ centerline} = \frac{R_s + T}{Q} = \frac{2180.83\,kN/m + 443.83\,kN/m}{1320.98\,kN/m} = 1.99 > 1.5$ ($\therefore$ ok)

PROJECT:	St. Lawrence Marine Terminal	DOCUMENT NO.:	DC1-8700-07-008
PROJECT NO.:	8700-07	REVISION:	0
ITEM:	Sheet Pile Cell Option - Global Stability	PREPARED BY:	Steven Greeley

6 Bursting Stability



h_c = height of cell = 29m ∴ $h_c/4$ = 7.25m

* only soil weight is included in pressure diagram
— surcharge is neglected

» $a_{max} = \gamma_d(h_1) + \gamma'(h_c - h_1 - h_c/4) = 19 kN/m^2 (7m) + 12 kN/m^2 (14.75m) = 310 KPa$

» K_i = coefficient of soil lateral pressure (taken as 0.5)

T_{max} (main cell) $= k a_{max} R = 0.5(310 KPa)(28.82m/2) = 2233.55 kN/m \times \overset{FS}{1.5} = 3350 kN/m$

T_{max} (connections) $: K_i a_{max} (0.5) L / \cos\alpha = 0.5(310 KPa)(0.5)(39.82m)/\cos 30 = 3563.46 kN/m \times 1.5$
$= 5345 kN/m$

» Select from Profilarbed catalogue : Pile AS 500-12.5 → Strength = 5500 kN/m

APPENDIX G: PRELIMINARY DESIGN: CONCRETE CAISSONS

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		PAGE:	1 of 6

PROJECT:	St. Lawrence Marine Terminal	DOCUMENT NO.:	DC1-8700-07-007
PROJECT NO.:	8700-07	REVISION:	0
ITEM:	Concrete Caisson Option - Global Stability	PREPARED BY:	Steven Greeley

» Preliminary design based on global stability to determine approximate sizing of structure

» Design is based on guidelines presented in:
① Handbook of Port & Harbor Engineering: Geotechnical + Structural Aspects (Tsinker, 1997)

» Global Stability checks:
① Sliding
② Overturning
③ bearing on foundation
④ bearing/contact pressure @ rubble mattress

» used same geotechnical data as that for sheet pile cells which was obtained from DAS (2006)

↳ assumptions: concrete density, $\rho_c = 2400 kg/m^3$
rubble mattress density, $\rho_d = 1950 kg/m^3$

» General Design Considerations
↳ wraparound of structure since slope very steep for armor protected slope
↳ can't flare caissons
↳ try to accommodate ship loader or not (rail gauge reduced to 19m)
↳ extended base slab 0.5m all the way around cells
↳ for preliminary purposes assume: interior walls - 0.3m thick
exterior walls - 0.4m thick
exterior walls to accommodate shiploader - 0.5m thick

» The following stability calculations were performed on (5) preliminary options
↳ A quick cost comparison was conducted to determine most efficient soln

* Structure Length: 265m
* distance from shore: 40m
* retaining height - 29m

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PROJECT:	St. Lawrence Marine Terminal	DOCUMENT NO.:	DC4-8700-07-007
PROJECT NO.:	8700-07	REVISION:	0
ITEM:	Concrete Caisson Stability - Global Stability	PREPARED BY:	Steven Greeley

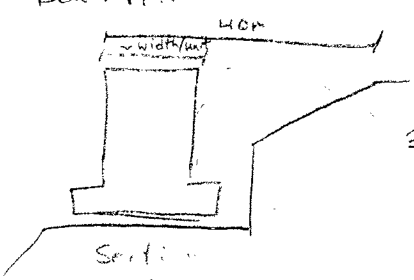
Volume Calculations

Concrete: Concrete (in m³/m) [from spreadsheet] × $\frac{\text{length}}{\text{unit}}$ × # units

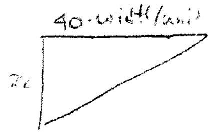
$\left[\frac{\#\text{ units}}{\text{along } L} + 2 + \frac{40 - \text{length/unit}}{\text{length/unit}} \times \ldots\right]$

Fill in Cells: Interior cell volume (in m³) [from spreadsheet] × # units

Backfill:

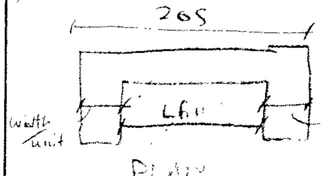


$\cong$

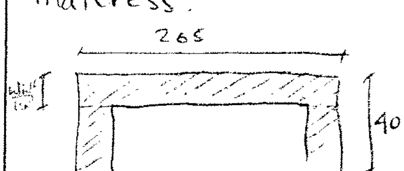


A = 1/2 bh = $\frac{1}{2}$(40 - width/unit)(22)

∴ V = AL = $\frac{1}{2}$ × 22m × (40 - width/unit) × (265 - 2 × width/unit)



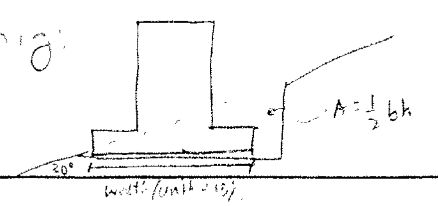
Mattress:



V = mattress thickness × width/unit × (265 + 2 × (40 - width/unit))

* values for options are found on next sheet - the quantities are placed in a preliminary cost sheet

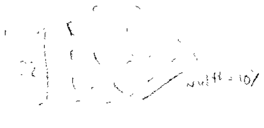
Blasting:



tan 30 = $\frac{opp}{adj}$ ⇒ opp = adj tan 30 = 1.1 width tan 30

∴ V_1 = $\frac{1}{2}$ (1.1 width/unit)² tan 30 × 365m (along front)

gouge in sides:



V_2 = $\frac{1}{2}$ (40 - 1.1 width/unit) · 22 × (1.1 width/unit ...)

V_T = ...

PARAMOUNT	DESIGN CALCULATIONS SHEET	DATE:	Mar 17 / 2010
		PAGE:	3 of 6

PROJECT:	St. Lawrence Marine Terminal	DOCUMENT NO.:	DC1-8700-07-007
PROJECT NO.:	8700-07	REVISION:	0
ITEM:	Concrete Caisson Option - Global Stability	PREPARED BY:	Steven Greeley

							Volumes (m^3)			
Opt.	Length / unit	Width / unit	# units along L	L total	Int. Cell Volume	Matress thickness	Concrete	Fill in cells	Backfill	Matress
1	31	24.5	7	265	17825	0.37	23166	170775	36828	2683
2	28	21	8	265	13412	2	22496	145616	46607	12726
3	25	20.5	9	265	11503	2.27	23485	140337	48048	14147
4	25	20.5	9	265	11274	2.37	26230	137543	48048	14770
5	21	17.5	11	265	7875	4.03	23325	116625	56925	21863

	Blasting (m^3)	Blasting (t)	Fill in cells (t)	Backfill (t)	Matress (t)
1	59430	126614	363833	78461	5756
2	45115	96116	310232	99295	27297
3	43228	92097	298984	102365	30345
4	43228	92097	293032	102365	31681
5	32741	69755	248467	121278	46896

Based on Preliminary costing (ignoring the cost of elements which do not change)

Option	Cost ($)
1	$26,739,888
2	$25,913,524
3	$26,686,228
4	$28,975,854 * accomodates shiploader as foundation
5	$26,300,660

* option 2 - appears the best but difference is [illegible] negligible
* try option 2 as basis for design but try to accomodate foundation for ship loader.

PARAMOUNT	DESIGN CALCULATIONS SHEET	DATE:	Mar 17 / 2010
		PAGE:	4 of 6

PROJECT:	St. Lawrence Marine Terminal	DOCUMENT NO.:	DC1-8700-07-007
PROJECT NO.:	8700-07	REVISION:	0
ITEM:	Concrete Caisson Option - Global Stability	PREPARED BY:	Steven Greeley

Material Densities

Concrete (ρ_c)	2400 kg/m^3
Caisson ballast (dry density) (ρ)	1937 kg/m^3
Caisson ballast (submerged density) (ρ')	1223 kg/m^3
Water (ρ_w)	1000 kg/m^3
Rubble Matress (ρ_r)	1950 kg/m^3

Material Unit Weights

Concrete (γ_c)	23.54 kN/m^3
Caisson ballast (unit weight) (γ)	19.00 kN/m^3
Caisson ballast (submerged unit weight) (γ')	12.00 kN/m^3
Water (γ_w)	9.81 kN/m^3
Rubble Matress (buoyant unit weight) (γ_r)	9.32 kN/m^3

Design Loads

Vertical Loads

Surcharge ($q_{sur.}$) 20 kPa

Horizontal Loads

Wind

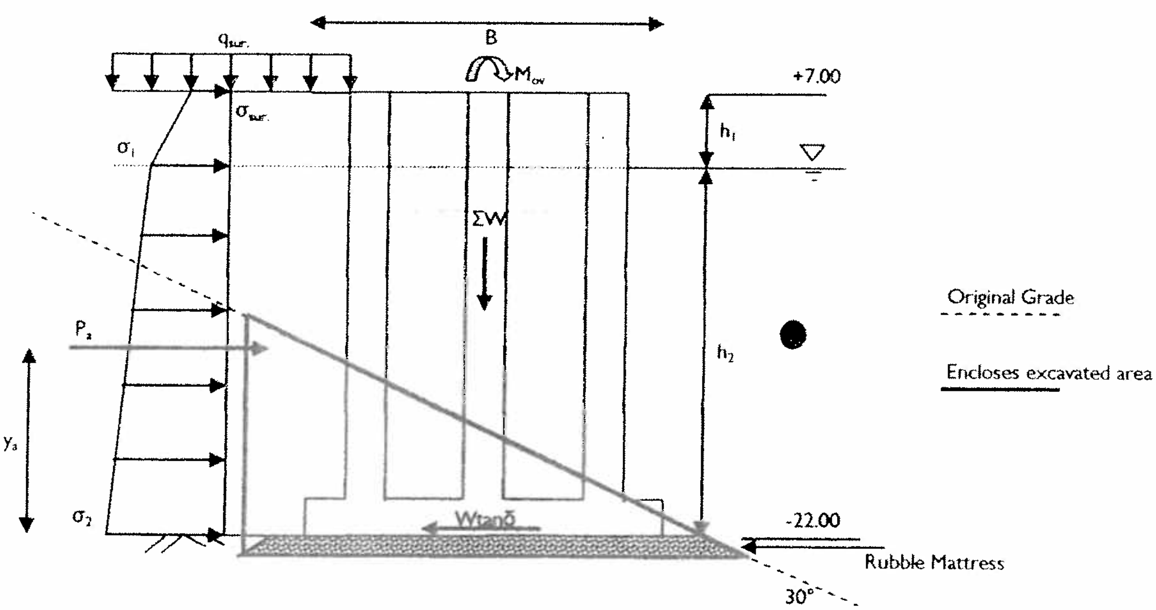
Wave

Earthquake

Berthing

Other Design Criteria

Total Height	29 m	
(Width of Base)$_{initial}$	24.5 m	Based on past project history: $(H/w)_{avg} = 1.20$
K_a	0.297	Based on Coulomb's Active Pressure Theory.
$\tan\delta$	0.5	Coefficient of friction for a soil-concrete interface



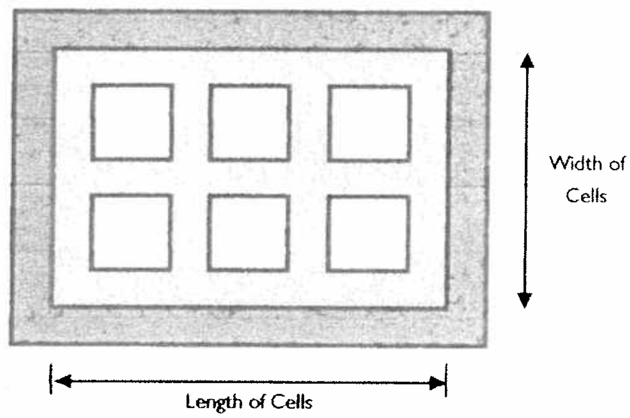
Note: Ignore bedrock behind caisson and assume material to have same properties as contained backfill.

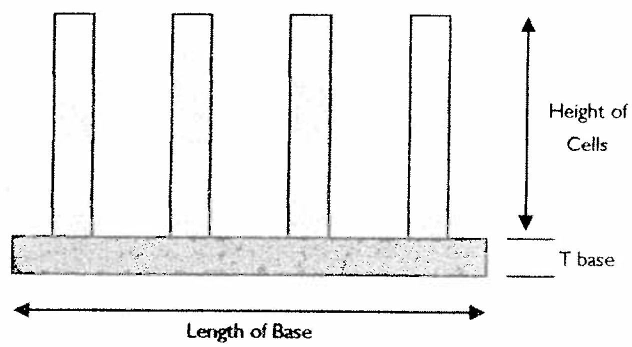
PROJECT:	St. Lawrence Marine Terminal	**DOCUMENT NO.:**	DC1-8700-07- 007
PROJECT NO.:	8700-07	**REVISION:**	0
ITEM:	Concrete Caisson Option - Global Stability	**PREPARED BY:**	Steven Greeley

CAISSON GEOMETRY

(input red values)

Number of cells along length	4	
Number of cells along width	3	
Base slab projection along sides	0.5	m
Base slab projection along ends	0.5	m
Base Slab		
Length of base	28	m
Width of base (B)	21	m
Thickness of base slab	0.5	m
Volume of base slab	**294**	m^3
Weight of base slab	**6922**	kN
Bouyancy of base slab	**2884**	kN
Exterior Cell Dimensions		
Overall length of cells	**27**	m
Overall width of cells	**20**	m
Overall height of cells	23.5	m
Interior Cell Dimensions		
Dimension along length	**6.325**	m
Dimension along width	**6.200**	m
Volume of interior cells	**13412**	m^3
Interior Walls		
Thickness of interior walls	0.3	m
Volume of concrete	**701**	m^3
Weight of interior walls	**16507**	kN
Exterior Walls		
Thickness of exterior walls	0.4	m
Volume of concrete	**1053**	m^3
Weight of exterior walls	**24800**	kN
Gussets		
Gussets offset	0.25	m
Volume of gussets	**27.36**	m^3
Weight of gussets	**644**	kN





Total Weights

Total volume of concrete	**2076**	m^3
Total caisson weight (concrete)	**48873**	kN
Total Weight of rock fill in caisson	**183997**	kN
Total Weight of structure	**232870**	kN

Total Weights per meter length

Total volume of concrete	**74**	m^3/m
Total caisson weight (concrete)	**1745**	kN/m
Total Weight of rock fill in caisson	**6571**	kN/m
Total Weight of structure (ΣW)	**8317**	kN/m

PARAMOUNT	DESIGN CALCULATIONS SHEET	DATE:	Mar 17 / 2010
		PAGE:	6 of 6

PROJECT:	St. Lawrence Marine Terminal	DOCUMENT NO.:	DC1-8700-07-007
PROJECT NO.:	8700-07	REVISION:	0
ITEM:	Concrete Caisson Option - Global Stability	PREPARED BY:	Steven Greeley

Stability Checks

1. Sliding

FS = 2.03 > 1.5 **Ok**

σ_{sur} = 5.95 kPa $= \boxed{k_a \times q_{sur}}$

σ_1 = 45.49 kPa $= \boxed{\sigma_{sur} + \left(k_a \times \gamma \times h_1\right)}$

σ_2 = 123.97 kPa $= \boxed{\sigma_1 + \left(k_a \times \gamma' \times h_2\right)}$

P_a = 2044 kN/m $= \boxed{\sigma_{sur}(h_1 + h_2) + \tfrac{1}{2}(\sigma_1 - \sigma_{sur})(h_1) + (\sigma_1 - \sigma_{sur})(h_2) + \tfrac{1}{2}(\sigma_2 - \sigma_1)(h_2)}$

y_a = 10.65 m $= \boxed{\dfrac{\sigma_{sur}(h_1 + h_2)\left(\frac{(h_1 + h_2)}{2}\right) + \frac{1}{2}(\sigma_1 - \sigma_{sur})(h_1)\left(h_2 + \frac{h_1}{3}\right) + (\sigma_1 - \sigma_{sur})(h_1)\left(\frac{h_1}{2}\right) + \frac{1}{2}(\sigma_2 - \sigma_1)(h_2)\left(\frac{h_2}{3}\right)}{P_c}}$

Wtanδ = 4158 kN/m $= \boxed{\Sigma W \tan\delta}$

2. Overturning

FS = 4.01 > 2.0 **Ok**

M_r = 87326 kN-m/m $= \boxed{\Sigma W \times B/2}$

M_{ov} = 21768 kN-m/m $= \boxed{P_a \times y_a}$

3. Contact stresses at wall base

σ_{max} = 692 < σ_{all} **Ok**

x_1 = 7.88 m $= \boxed{(M_r - M_{ov})/\Sigma W}$

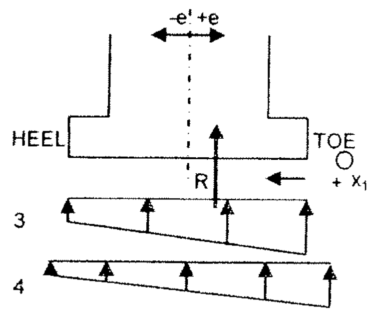
e = 2.62 m $= \boxed{B/2 - x_1}$

B/6 = 3.50 m Middle 1/3 of base

$\sigma_{B\,(toe)}$ = 692 kPa $= \boxed{\Sigma W/B\left(1 + 6e/B\right)}$

$\sigma_{B\,(heel)}$ = 100 kPa $= \boxed{\Sigma W/B\left(1 - 6e/B\right)}$

σ_{all} = 1000 kPa Bedrock assumed 1000 kPa has infinite bearing capacity.



4. Stresses at mattress-soil interface

σ'_{max} = 582 < σ_{all} **Ok**

h_m = 2.50 m $= \boxed{\geq hm,\min}$

Matress thickness

$\sigma'_{B\,(toe)}$ = 582 kPa $= \boxed{\sigma_{B(toe)}\left(\dfrac{B}{B + 2h_m}\right) + \gamma_r \times h_m}$

$\sigma'_{B\,(heel)}$ = 104 kPa $= \boxed{\sigma_{B(heel)}\left(\dfrac{B}{B + 2h_m}\right) + \gamma_r \times h_m}$

σ_{all} = 600 kPa Based on Table 5.2 form Tsinker (1997) for dense gravel beddings

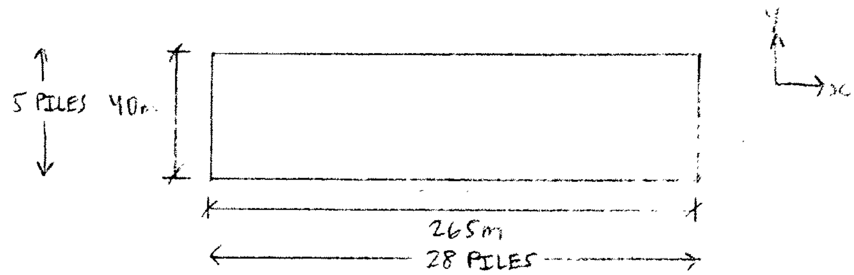
$h_{m,min}$ = 2.00 m $= \boxed{\dfrac{2\sigma_f - \gamma_r B}{4\gamma_r} - \left[\left(\dfrac{2\sigma_f - \gamma_r B}{4\gamma_r}\right)^2 - \dfrac{B(\sigma_{max} - \sigma_f)}{2\gamma_r}\right]^{0.5}}$

APPENDIX H: PRELIMINARY DESIGN: TUBULAR STEEL PILES

PARAMOUNT	DESIGN CALCULATIONS SHEET	DATE:	MAR 15/2010
		PAGE:	1 of 5

PROJECT:	ST. LAWRENCE MARINE TERMINAL	DOCUMENT NO.:	DC1-8700-07-009
PROJECT NO.:	8700-07	REVISION:	0
ITEM:	STEEL PIPE PILE DESIGN	PREPARED BY:	ANDREW SMALL

PORT DIMENSIONS:



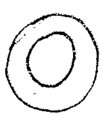
* ASSUME 10M SPACING FOR PILES

$\# SPACES\ (y\text{-}DIR'N) = \frac{40}{10} = 4 \rightarrow \# PILES = \# SPACES + 1 = 5$

$\# SPACES\ (x\text{-}DIR'N) = \frac{265}{10} = 27 \rightarrow \# PILES = 27 + 1 = 28$

* ASSUME D = 1300 mm // TAKEN FROM WORKED EXAMPLE FOR BREASTING DOLPHIN
t = 30 mm

AXIAL CAPACITY:



$A_{steel} = \frac{\pi}{4}(1300^2 - 1240^2) = 119\,695\ mm^2$

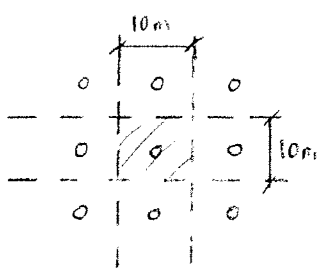
* ASSUME $F_y = 350\ MPa$

AXIAL RESIST. $F_r = 350\ N/mm^2 \times 119\,695\ mm^2 = 41\,893\ kN$

AXIAL LOAD:

1. OPERATIONAL SURCHARGE (F_o)
2. SHIP-LOADER (F_s)

1. OPERATIONAL SURCHARGE



$q = 20\ kPa$

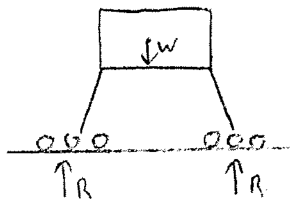
$F_o = 20\ \frac{kN}{m^2} \times 10m \times 10m = 2\,000\ kN$

PARAMOUNT	DESIGN CALCULATIONS SHEET	DATE:	MAR 15/2010
		PAGE:	2 of 5

PROJECT:	ST. LAWRENCE MARINE TERMINAL	DOCUMENT NO.:	DC1-8700-07-009
PROJECT NO.:	8700-07	REVISION:	0
ITEM:	STEEL PIPE PILE DESIGN	PREPARED BY:	ANDREW SMALL

2. SHIP-LOADER

W = 860 tons



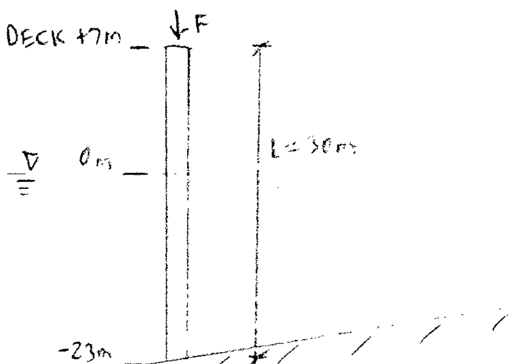
* TREAT AS CONCENTRATED RXNS

$W = 860 \text{ tons} \times 2000 \text{ lb/ton} \times 1 \text{ kg}/2.2 \text{ lb} \times 9.81 \text{ N/kg} \times 1 \text{ kN}/1000 \text{ N} = 7670 \text{ kN}$

$R = \dfrac{W}{2 \text{ rails} \times 2 \text{ rxns}} = 1918 \text{ kN} = F_s$

MAX FORCE ON PILE, $F_{max} = F_o + F_s = 2000 + 1918 = 3918 \text{ kN} < F_R$ (OK)

SLENDERNESS CHECK:



CONDITION	K
PIN - PIN	1.0
FIX'D - FIX'D	0.5
FIX'D - FREE	2.0
FIX'D - "FREE"	1.5 *

* ASSUME PARTIAL RESTRAINT

BEDROCK

$F_r = \dfrac{\pi^2 E I}{(KL)^2}$

$I = \dfrac{\pi}{4}(r_o^4 - r_i^4) = \dfrac{\pi}{4}\left(\left(\dfrac{1300}{2}\right)^4 - \left(\dfrac{1250}{2}\right)^4\right) = 2.415 \times 10^{10} \text{ mm}^4$

* ASSUME E = 200 000 MPa

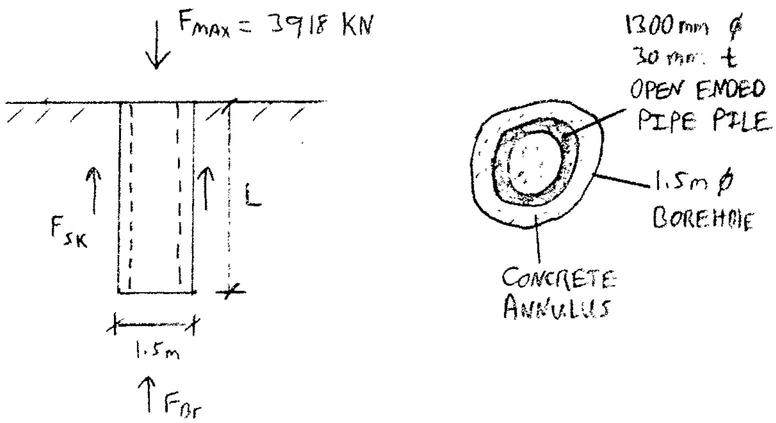
$F_r = \dfrac{\pi^2 (200\,000 \text{ N/mm}^2)(2.415 \times 10^{10} \text{ mm}^4)}{(1.5 \times 30\,000 \text{ mm})^2} \times \dfrac{1 \text{ kN}}{1000 \text{ N}}$

$= 23\,541 \text{ kN} > F_{max}$ (OK)

PARAMOUNT	DESIGN CALCULATIONS SHEET	DATE:	MAR 15 /2010
		PAGE:	3 of 5

PROJECT:	ST. LAWRENCE MARINE TERMINAL	DOCUMENT NO.:	DC1-8700-07-009
PROJECT NO.:	8700-07	REVISION:	0
ITEM:	STEEL PIPE PILE DESIGN	PREPARED BY:	ANDREW SMALL

BEARING CHECK:



* ASSUME 1.5 m BOREHOLE

* ASSUME 90% OF LOAD CARRIED BY SKIN FRICTION ($F_{SK} = 0.9 \times 3918 = 3526$ KN)

$$F_{SK} = \pi D_{borehole} \times L \times f$$

(f =) UNIT SKIN FRICTION

* ASSUME $f = 1000$ kPa FOR BEDROCK

$$L = \frac{F_{SK}}{\pi D_{borehole} f}$$

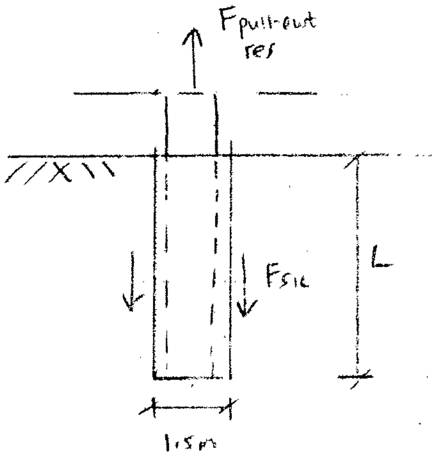
$$= \frac{3526 \text{ KN}}{\pi (1.5\text{m})(1000 \text{ kN/m}^2)}$$

$L = 0.75$ m

PARAMOUNT	DESIGN CALCULATIONS SHEET	DATE:	MAR 15/2010
		PAGE:	4 of 5

PROJECT:	ST. LAWRENCE MARINE TERMINAL	DOCUMENT NO.:	DCI - 8700 - 07 - 009
PROJECT NO.:	8700 - 07	REVISION:	0
ITEM:	STEEL PIPE PILE DESIGN	PREPARED BY:	ANDREW SMALL

PULL-OUT CHECK:



* ASSUME $L_{pullout} = 5m$

$F_{pullout} = \pi D L f = \pi (1.5m)(5m)(1000\ kN/m^2)$

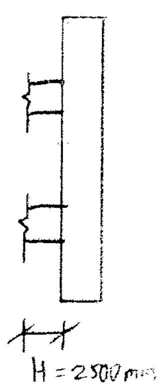
$= 23\ 562\ kN$ (OK)

BERTHING FORCE:

BERTHING ENERGY = 302 ton·m

SCK FENDER (TRELLEBORG CATELOGUE) 2500 →

→ RATED DEFLECTION = 52.5%



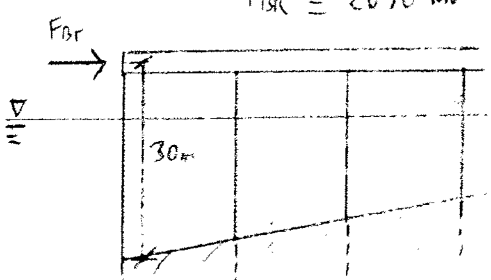
DEFLECTION, $\Delta = 0.525 \times 2500 = 1.3m$

BERTHING ENERGY = FORCE × DIST

→ BERTHING FORCE, $F_{BR} = \frac{302\ ton \cdot m}{1.3m}$

$= 232\ ton \times \frac{2000\ lb}{ton} \times \frac{1kg}{2.2\ lb} \times \frac{9.8N}{kg} \times \frac{1kN}{1000N}$

$F_{BR} = 2070\ kN$

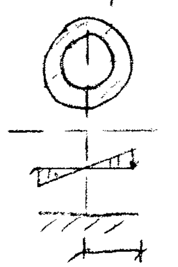


PARAMOUNT	DESIGN CALCULATIONS SHEET	DATE:	MAR 15/2010
		PAGE:	5 of 5

PROJECT:	ST. LAWRENCE MARINE TERMINAL	DOCUMENT NO.:	DC1-8700-07-009
PROJECT NO.:	8700-07	REVISION:	0
ITEM:	STEEL PIPE PILE DESIGN	PREPARED BY:	ANDREW SMALL

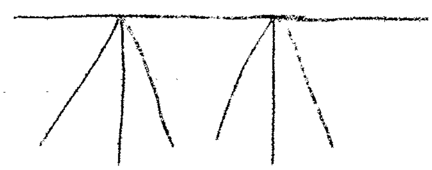
BENDING MOMENT = $F_{Br} \times 30\,m$ = 62 100 kN·m

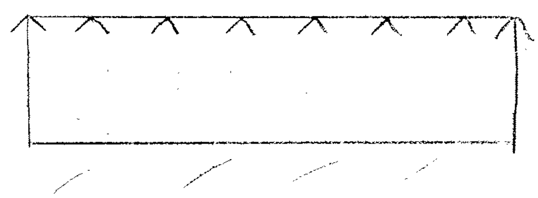
$$M_r = \frac{\sigma_{max} I}{y} = \frac{(350\ MPa)(2.415 \times 10^{10}\ mm^4)}{(650\ mm)}\ N \cdot mm \times \frac{1\,kN}{1000\,N} \times \frac{1\,m}{1000\,mm}$$



$= 13\,000\ kN \cdot m < M_f$

∴ ADD BATTER PILES ALONG FRONT ROW





APPENDIX I: PRELIMINARY COST ESTIMATE

PRELIMINARY COST ESTIMATE	Project:	St. Lawrence Marine Terminal
STEEL PIPE PILES	Project #:	8700-07
	Document #:	CE1-8700-07-001
	Page #:	1 of 3

ITEM	QUANTITY	UNITS	UNIT RATE	TOTAL
CIVIL WORKS				
SITE WORK & SITE GRADING				
Mass rock excavation	0	M^3		$0.00
Drill holes for pile placement	1,750	M^3	$350.00	$612,500.00
TOTAL				**$612,500**
STEEL PIPE PILES				
SUPPLY AND INSTALLATION OF STEEL PIPE PILES	5,700	tonne	$4,000.00	$22,800,000.00
RENTAL OF PILE DRIVING EQUIPMENT	1	LS	$7,500,000.00	$7,500,000.00
CONCRETE FOR PILES				
Concrete (Pipe Fill)	5,200	M^3	$1,800.00	$9,360,000.00
Concrete (Bore Hole Annulus)	450	M^3	$1,800.00	$810,000.00
TOTAL				**$40,470,000**
CONCRETE AND ASSOCIATED WORKS				
CONCRETE ACCESSORIES				
Concrete Deck	4,000	M^3	$1,500.00	$6,000,000.00
Concrete for Wharf Bollards	0	M^3	$600.00	$0.00
TOTAL				**$6,000,000**
GRAND TOTAL				**$47,082,500**

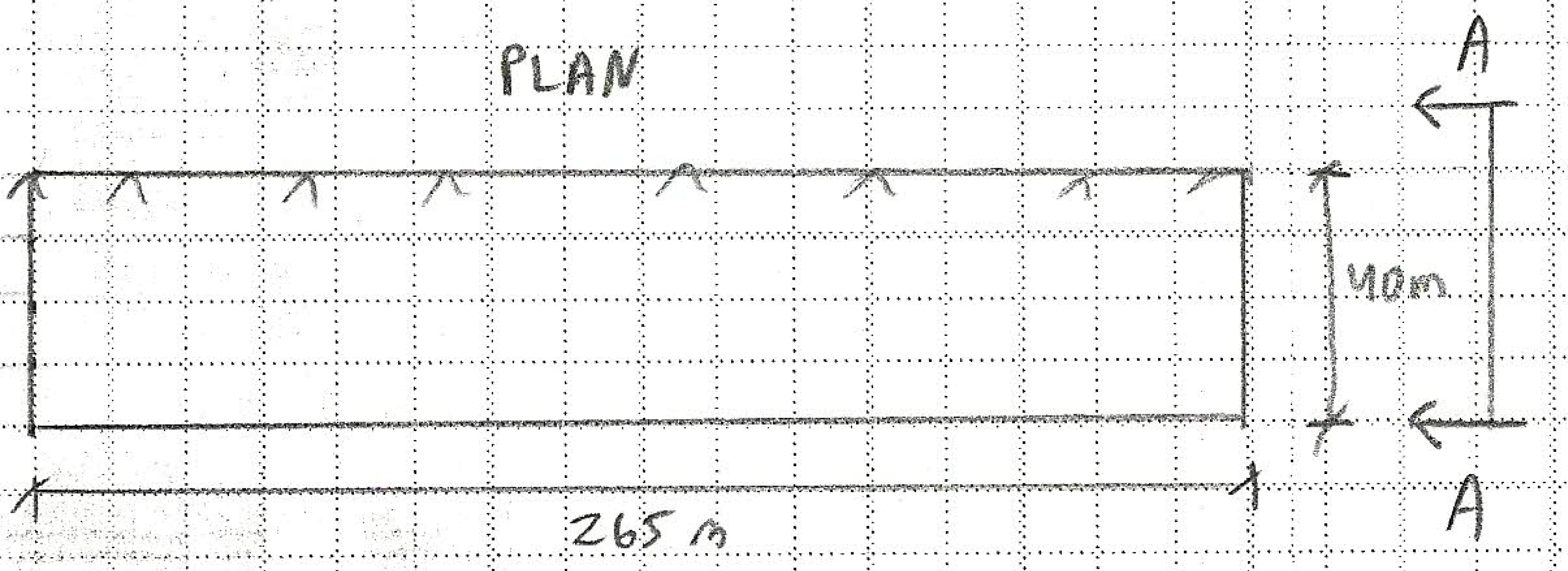
PARAMOUNT

PARAMOUNT	DESIGN CALCULATIONS SHEET	DATE:	MAR 23/2010
		PAGE:	2 of 3

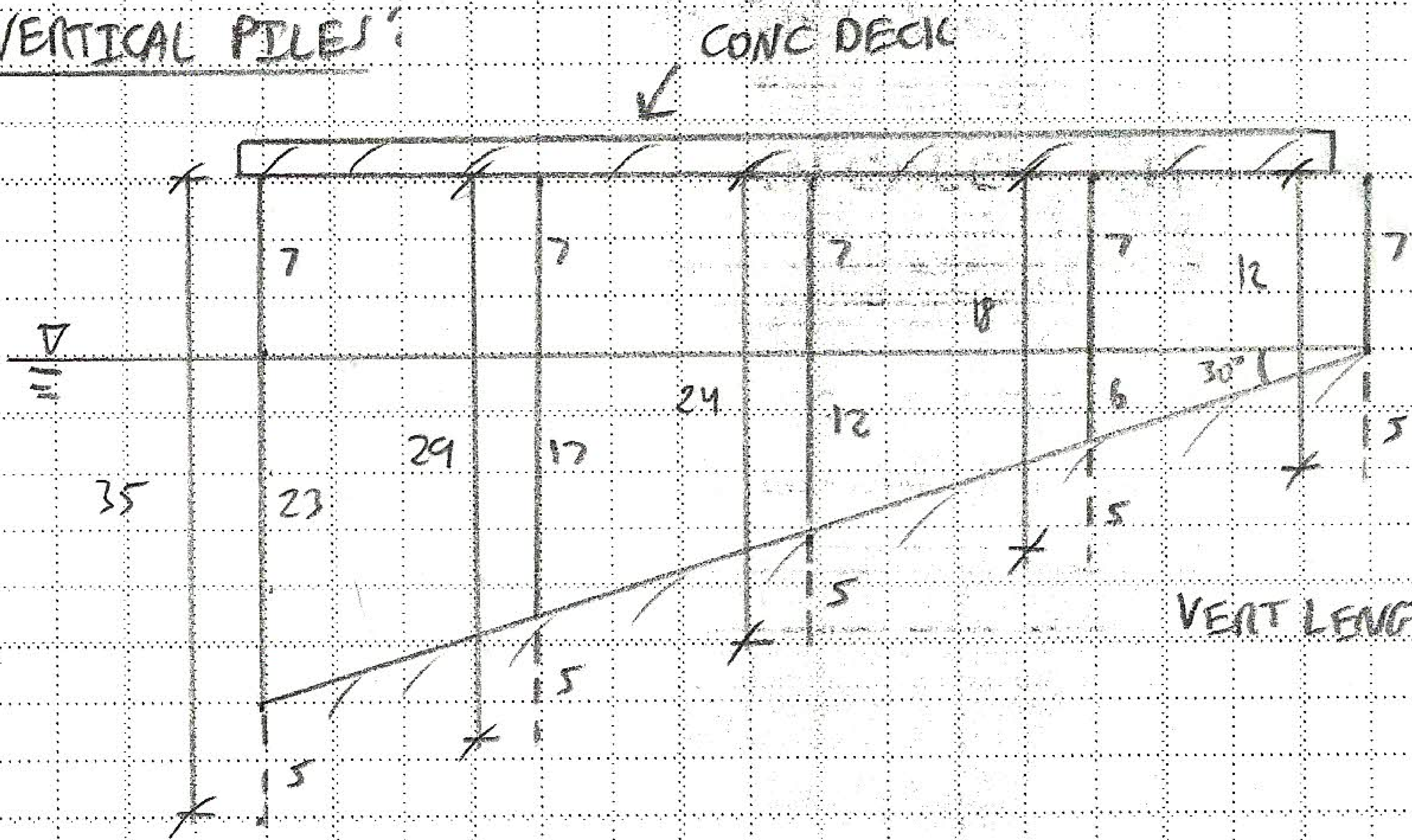
PROJECT:	ST. LAWRENCE MARINE TERMINAL	DOCUMENT NO.:	CE1-8700-07-001
PROJECT NO.:	8700-07	REVISION:	0
ITEM:	STEEL PIPE PILES - PRELIM. TAKE-OFF	PREPARED BY:	ANDREW SMALL

ROW	# VERT. PILES	# BATTER PILES
1	28	56
2	28	–
3	28	–
4	28	–
5	28	–

PLAN



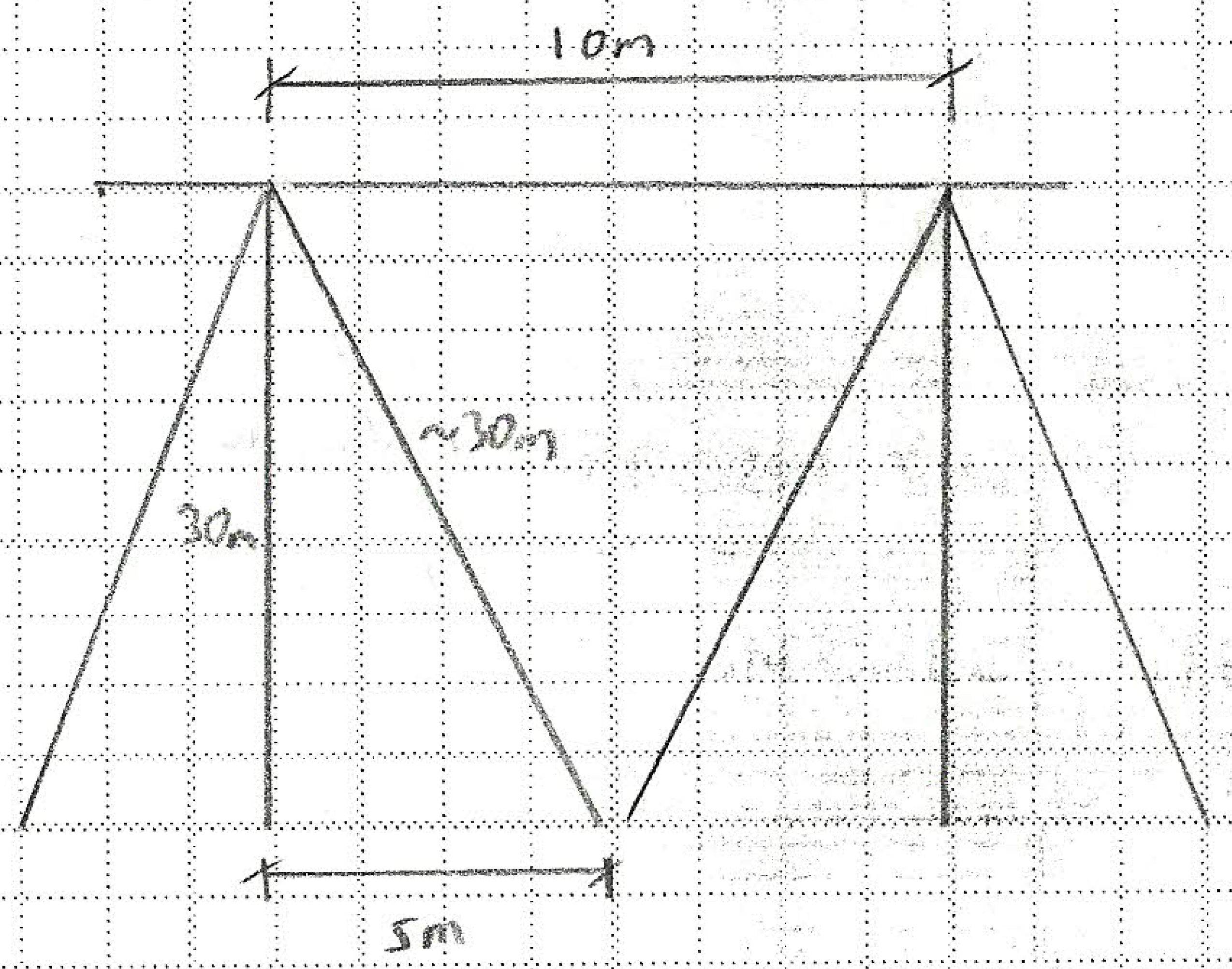
VERTICAL PILES:



$\tan 30 = \frac{y}{x}$

VERT LENGTH = 118m

BATTER PILES:



#BATTER = 28x2 = 56 PILES

BATTER PILE LENGTH = 56 x 30 = 1680m

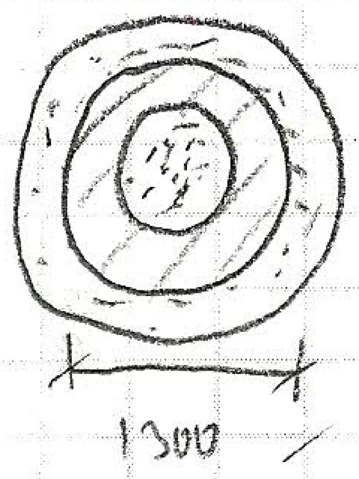
TOTAL LENGTH OF PILES = $\left(118\,\text{m}/\text{row} \times 28\ \text{rows}\right)$ + 1680 m (batter) = 4984m

PARAMOUNT	DESIGN CALCULATIONS SHEET	DATE:	MAR 23/2018
		PAGE:	3 of 3

PROJECT:	ST. LAWRENCE MARINE TERMINAL	DOCUMENT NO.:	CE1-8700-07-001
PROJECT NO.:	8700-07	REVISION:	0
ITEM:	STEEL PIPE PILES - PRELIM. TAKE-OFF	PREPARED BY:	ANDREW SMALL

$$\text{TOTAL PILE WT} = 4984\,m \times 0.12\,m^2 \times \frac{7850\,kg}{m^3} \times \frac{2.2\,lb}{kg} \times \frac{1\,ton}{2000\,lb} = 5164\ ton$$

$$A_{st} = \frac{\pi}{4}\left(1300^2 - 1240^2\right) = 0.12\,m^2$$



CONCRETE FILL:

$$A_{conc} = \frac{\pi(1.24)^2}{4}$$

$$V_{conc} = A_{conc} \times \underset{4984m}{\underset{\uparrow}{L_{piles}}} = 6019\,m^3$$

BORE HOLES:

$$V = \frac{\pi(1.5)^2}{4} \times 5\,m \times 196\ \text{HOLES} = 1732\,m^3$$

$$\#\text{holes} = \underset{\text{VERTICAL}}{(5 \times 28)} + \underset{\text{BATTER}}{56} = 196$$

CONCRETE ANNULUS:

$$A_{conc} = \frac{\pi}{4}\left(1.5^2 - 1.3^2\right)$$

$$V_{conc} = A_{conc} \times \#\text{HOLES} \times 5\,m$$

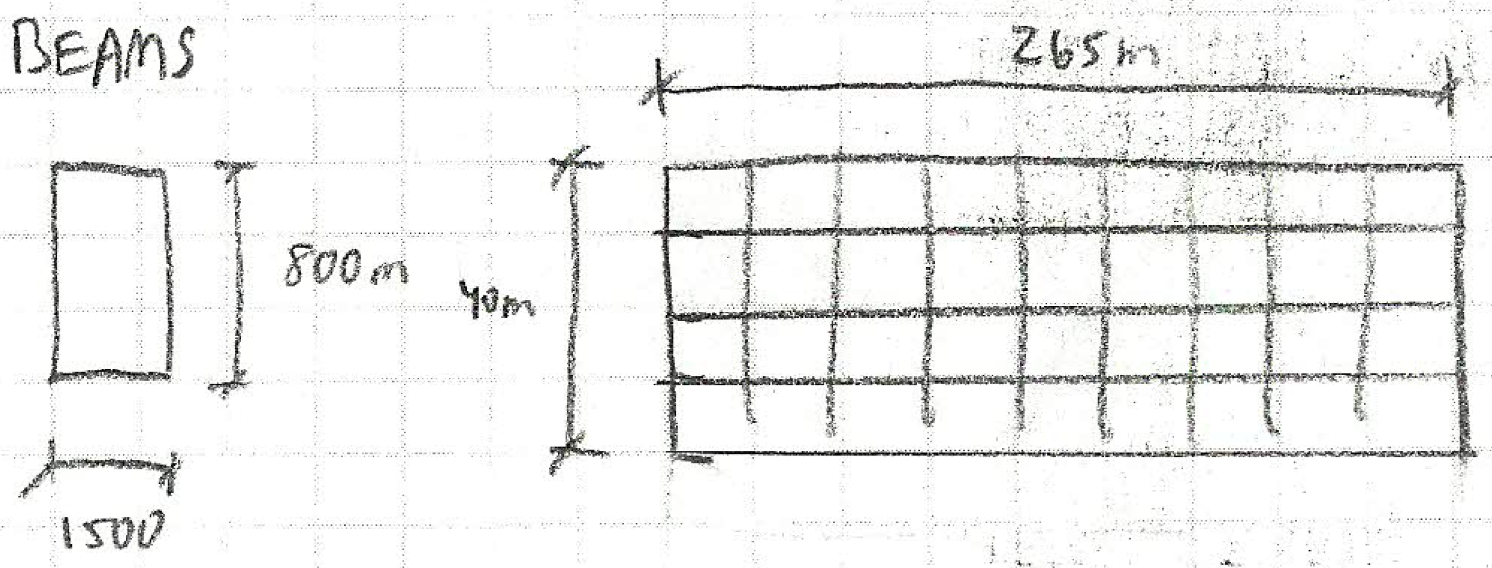
$$= \frac{\pi}{4}\left(1.5^2 - 1.3^2\right) \times (196) \times 5$$

$$= 431\,m^3$$

CONCRETE DECK

$$\text{SLAB} \rightarrow V_{conc} = 0.350 \times 40 \times 265 = 3710\,m^2$$

BEAMS



$$L_{beam} = (5 \times 265\,m) + (28 \times 5\,m)$$

$$= 1450\,m$$

$$V_{conc} = 1.5 \times 0.8 \times 1450$$

$$= 1740\,m^3$$

$$\text{TOTAL } V_{conc} = 5450\,m^3$$

PRELIMINARY COST ESTIMATE		
SHEET PILE CELLS	Project:	St. Lawrence Marine Terminal
	Project #:	8700-07
	Document #:	CE1-8700-07-002
	Page #:	1 of 4

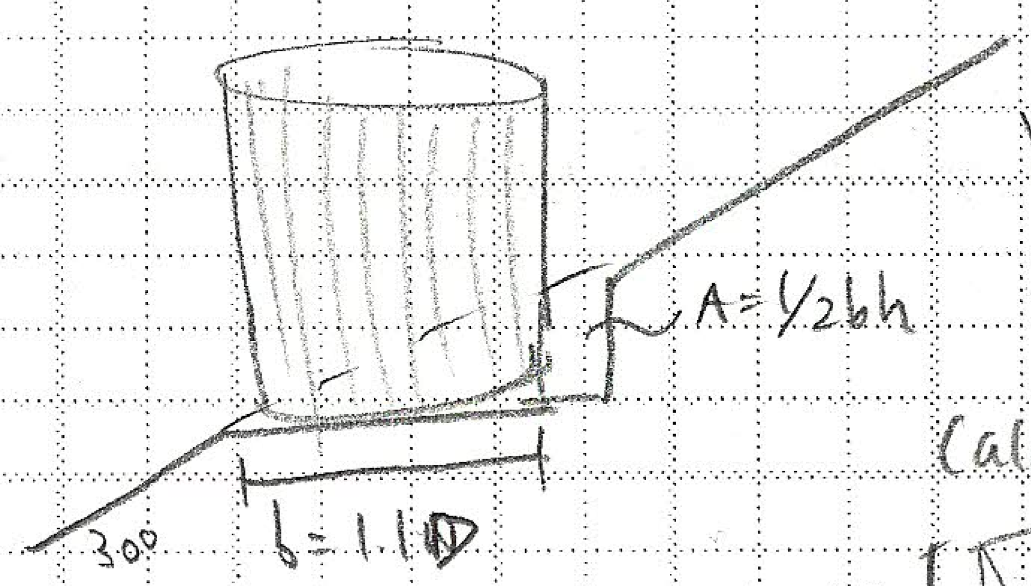
ITEM	QUANTITY	UNITS	UNIT RATE	TOTAL
CIVIL WORKS				
SITE WORK & SITE GRADING				
Mass rock excavation	0	M^3		$0.00
Drill & blast rock for placement of cells	178,000	tonne	$18.00	$3,204,000.00
Place rock fill behind wharf structure	113,000	tonne	$15.00	$1,695,000.00
ROCK SUB-MATTRESS / MATRESS				
Supply and place rock matress	65,000	tonne	$20.00	$1,300,000.00
SLAB ON GRADE	2,200	M^3	$600.00	$1,320,000.00
SSP CELL BALLAST				
Supply & Placement of Rock Ballast in SSP Cells				
Rock Ballast - (100 minus)	526,000	tonne	$15.00	$7,890,000.00
TOTAL				**$15,409,000**
STEEL SHEET PILE CELLS				
SUPPLY OF STEEL SHEET PILING	5,500	tonne	$3,000.00	$16,500,000.00
INSTALLATION OF STEEL SHEET PILING				
Driving Template	1	LS	$250,000.00	$250,000.00
Cells	24,000	M^2	$150.00	$3,600,000.00
Arcs	11,000	M^2	$150.00	$1,650,000.00
TOTAL				**$22,000,000**
GRAND TOTAL				**$37,409,000**

PARAMOUNT

PARAMOUNT	DESIGN CALCULATIONS SHEET	DATE:	March 23/2010
		PAGE:	2 of 4

PROJECT:	St. Lawrence Marine Terminal	DOCUMENT NO.:	CE7-8700-07-002
PROJECT NO.:	8700-07	REVISION:	0
ITEM:	Sheet Piles - Prelim. Takeoff	PREPARED BY:	Steven Greeley

① Drill + Blast Rock for Placement of Sheet Piles

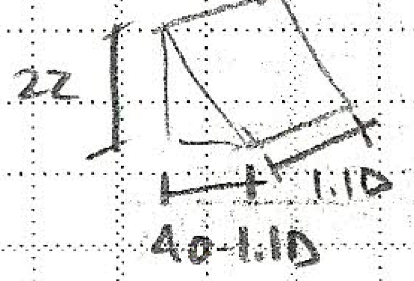


$V_{blast, along\ front} = \frac{1}{2}(1.1 \times 28.82m)^2 \tan 30 \times 267m$

$= 77,463\ m^3$

(along sides) $+ (2)(½)(40 - 1.1(28.82))(22m)(1.1 \times 28.82)$

$= 5787\ m^3$



$\therefore V_{TOTAL} = 83,250 m^3$

$\times\ 1937\ kg/m^3$

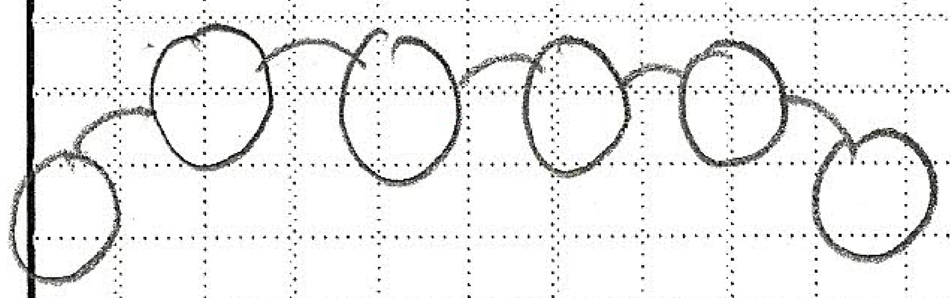
$\times\ 2.2 lbs/kg$

$\times\ 1\ ton/2000 lbs$

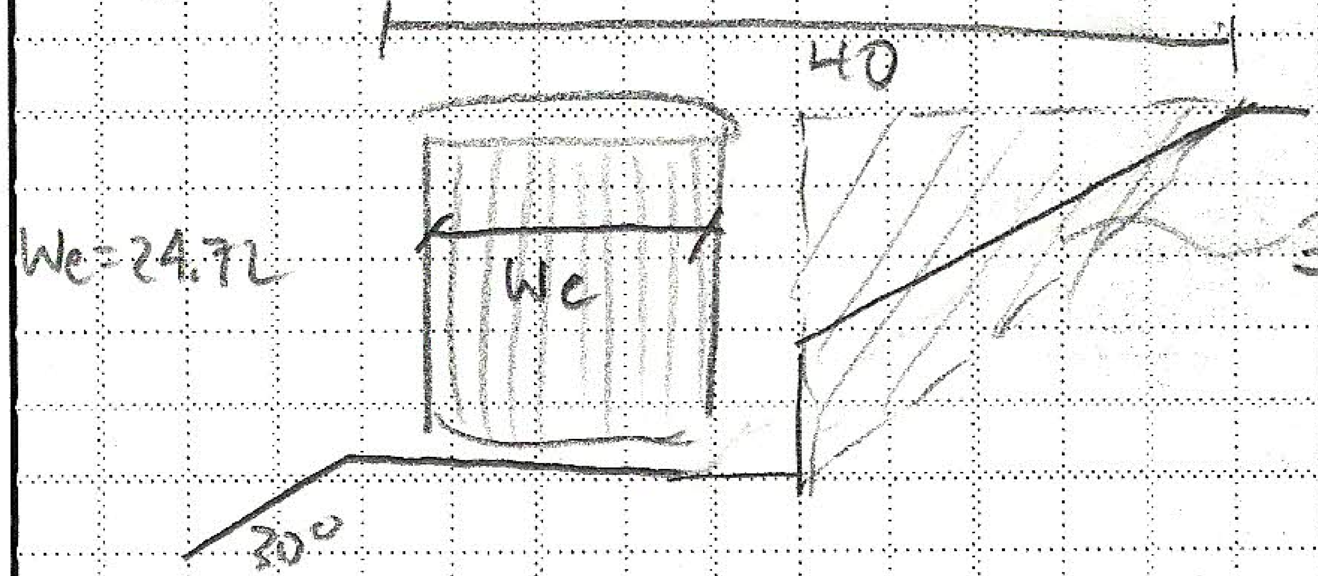
177,381t

↓

178,000t



② Backfill behind Structure



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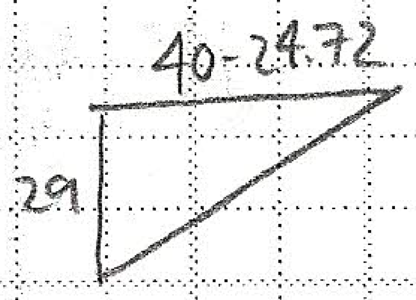


$A = \frac{1}{2}(40 - 24.72)(29) = 221.56 m^2$

$V = AL = 221.56(267 - 2(24.72))$

$= 48,203 m^3$

+ 10% (due to Flaring)

$V_{Backfill} = 53,023 m^3$

↓

112,976 t

↓

113,000t

PARAMOUNT	DESIGN CALCULATIONS SHEET	DATE:	March 23/2010
		PAGE:	3 of 4

PROJECT:	St. Lawrence Marine Terminal	DOCUMENT NO.:	CE1-8700-07-002
PROJECT NO.:	8700-07	REVISION:	0
ITEM:	Sheet Piles - Prelim. Takeoff	PREPARED BY:	Steven Greeley

③ Supply rock mattress

$V_{mattress} = [(267m \times 28.82m) + 2(40m)(28.82m)] \times \underset{\text{assumed}}{3m}$

$= 30002m^3 \times 1950kg/m^3 \times \frac{2.2lbs}{1kg} \times \frac{1t}{2000lbs} = 64,353t \rightarrow 65,000t$

④ Slab-on-grade

$267m \times 40m \times 0.2m = 2136m^3 \rightarrow 2200m^3$

⑤ Cell Ballast

$A_{cell} = \frac{\pi D^2}{4} = \frac{\pi (28.82m)^2}{4} = 652.35m^3 \times 9\ cells = 5871m^3$

$+ A_{arcs} = 328.73m^2$ (from CAD) $= 328.73m^2 \times 8\ arcs = 2630m^3$

$8501m^3$

$\times 29m$ (height)

$V_{ballast} = 246524m^3$

$\therefore 246,524m^3 \times 1937\frac{kg}{m^3} \times \frac{2.2lbs}{1kg} \times \frac{1ton}{2000lbs} = 525,269t \rightarrow 526,000t$

⑥ # Piles: $9cells \times (108+62+4) = 1566$ piles

$+8arcs \times (86) = 688$ piles

2254 piles $\times 29m$ (length)

$= 65,366m$ (pile)

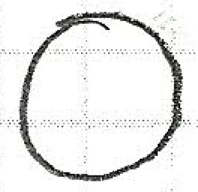
Pile Type: AS 500-12.5 = 76.3kg/m

$\therefore M_{total} = 65,366m \times 76.3kg/m \times \frac{2.2lbs}{1kg} \times \frac{1ton}{2000lbs} = 5486t \rightarrow 5500t$

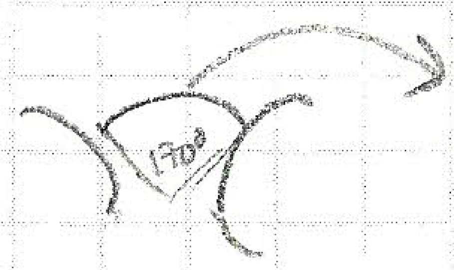
PARAMOUNT	DESIGN CALCULATIONS SHEET	DATE:	March 23/10
		PAGE:	4 of 4

PROJECT:	St. Lawrence Marine Terminal	DOCUMENT NO.:	CE7-8700-07-002
PROJECT NO.:	8700-07	REVISION:	0
ITEM:	Sheet Piles - Prelim. Takeoff	PREPARED BY:	Steven Greeley

7 Installation of Sheet Piling ($/m^2$ pile)



$\Rightarrow \pi d \times h = \pi(28.82m) \times 29m = 2626m^2 \times 9$ cells $= 23,631m^2 \downarrow 24,000m^2$



$\left(\frac{170°}{360°}\right)\pi(14.92m)(29m) \times 8$ arcs $\times \underset{\text{front \& back}}{2} = 10,270m^2 \downarrow 11,000m^2$

PRELIMINARY COST ESTIMATE	Project:	St. Lawrence Marine Terminal
CONCRETE CAISSONS	Project #:	8700-07
	Document #:	CE1-8700-07-003
	Page #:	1 of 3

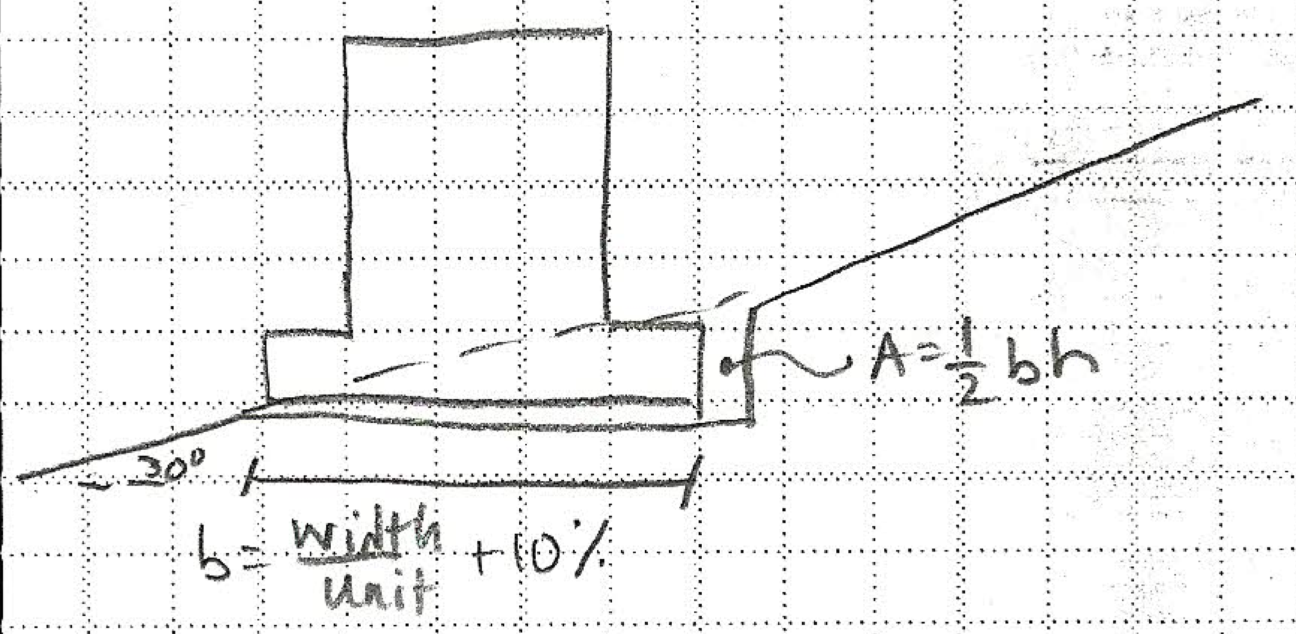
ITEM	QUANTITY	UNITS	UNIT RATE	TOTAL
CIVIL WORKS				
SITE WORK & SITE GRADING				
Mass rock excavation	0	M^3		$0.00
Drill & blast rock for placement of caissons	106,000	tonne	$18.00	$1,908,000.00
Place rock fill behind wharf structure	131,000	tonne	$15.00	$1,965,000.00
ROCK SUB-MATTRESS / MATRESS				
Supply and place rock matress	28,000	tonne	$20.00	$560,000.00
SLAB ON GRADE	2,200	M^3	$600.00	$1,320,000.00
CAISSON BALLAST				
Supply & Placement of Rock Ballast in Caissons				
Rock Ballast - (100 minus)	311,000	tonne	$15.00	$4,665,000.00
TOTAL				**$10,418,000**
CONCRETE AND ASSOCIATED WORKS				
CONCRETE CAISSONS				
Construction of Slipway	1	LS	$250,000.00	$250,000.00
Concrete for Caissons	22,500	M^3	$1,500.00	$33,750,000.00
Transport and Place Caissons	12	each	$20,000.00	$240,000.00
Concrete Joint Panels				
Supply and Install Joint Panels	11	each	$20,000.00	$220,000.00
Install Joint Panels	11	each	$0.00	$0.00
CONCRETE ACCESSORIES				
Concrete for Wharf Bollards	0	M^3	$600.00	$0.00
TOTAL				**$34,460,000**
GRAND TOTAL				**$44,878,000**

PARAMOUNT

PARAMOUNT	DESIGN CALCULATIONS SHEET	DATE:	March 23/10
		PAGE:	2 of 3

PROJECT:	St. Lawrence Marine Terminal	DOCUMENT NO.:	CE1-8700-07-003
PROJECT NO.:	8700-07	REVISION:	0
ITEM:	Concrete Caisson - Prelim. Take Off	PREPARED BY:	Steven Greeley

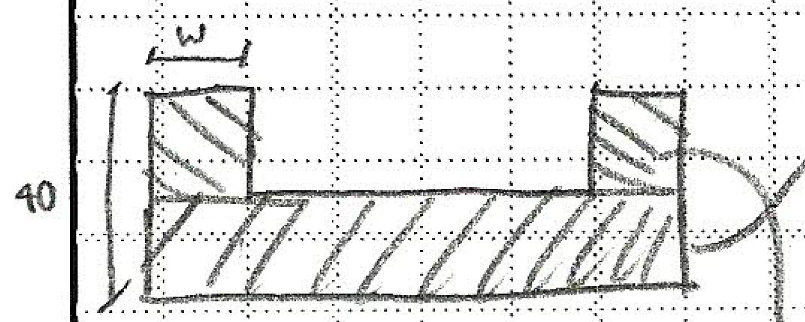
① Drill & Blast Rock for Placement of Caissons



$\tan 30 = \frac{opp}{adj} = \frac{h}{b} \therefore h = b\tan 30$

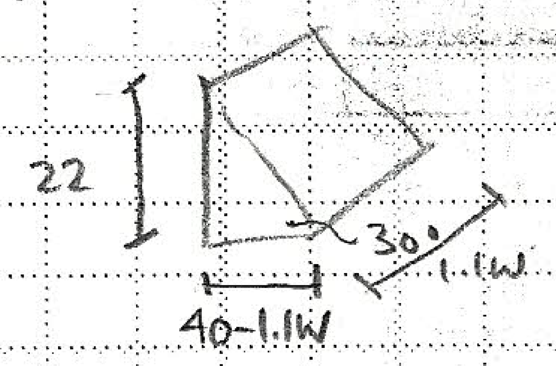
$\therefore V_{blast,\,along\,front} = \frac{1}{2}(1.1\,width)(1.1\,width)\tan 30 \times 265m$

$= \frac{1}{2}(1.1(21m))^2 \tan 30 \times 265m$



$= 40{,}821\ m^3$

volume into sides



$\therefore V_{blast,\,sides} = (2) \times \frac{1}{2}(40 - 1.1(21m))(22m)(1.1 \times 21m)$

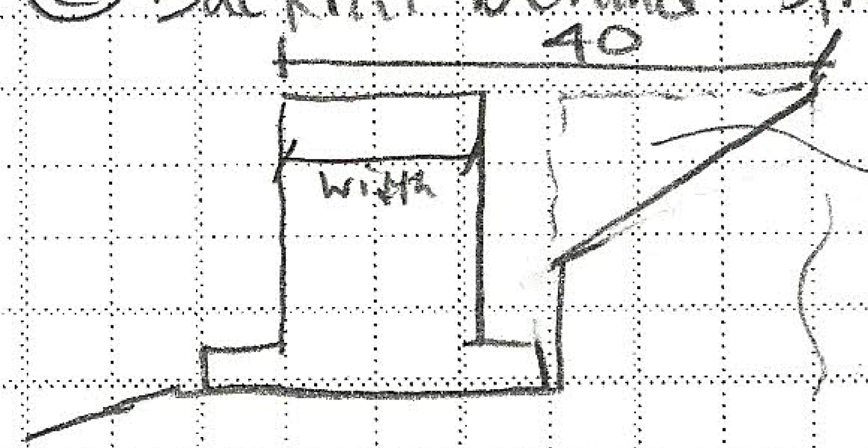
$= 8{,}589\ m^3$

$V_{TOTAL,\,Blast} = 49{,}410\ m^3 \times \frac{1937\,kg}{m^3} \times \frac{2.2\,lbs}{1\,kg} \times \frac{1\,ton}{2000\,lbs} = 105{,}263\ t$

$\downarrow$

106,000 t

② Backfill behind structure



$A = \frac{1}{2}(bh) = \frac{1}{2}(40m - 21m)(29m) = 275.5\ m^2$

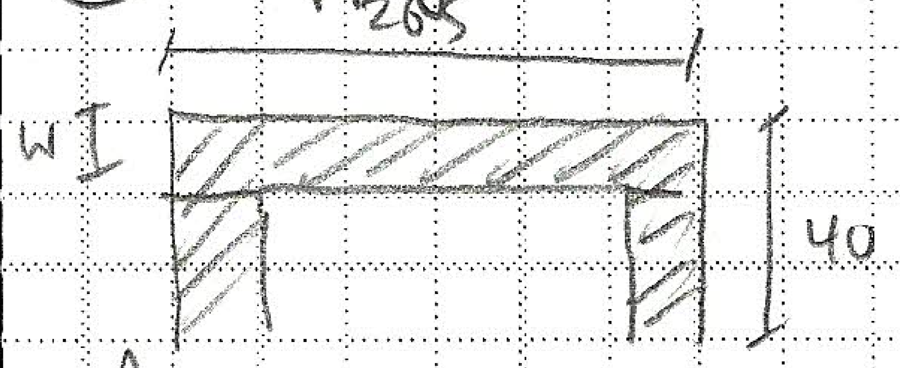
$V = AL = 275.5(265 - 2(21)) = 61{,}437\ m^3$

$\therefore V_{Rockfill} = 61{,}437\ m^3 = 130{,}889\ t \rightarrow 131{,}000\ t$

PARAMOUNT	DESIGN CALCULATIONS SHEET	DATE:	March 23/10
		PAGE:	3 of 3

PROJECT:	St. Lawrence Marine Terminal	DOCUMENT NO.:	CE7-8700-07-003
PROJECT NO.:	8700-07	REVISION:	0
ITEM:	Concrete Caissons - Prelim. Take-off	PREPARED BY:	Steven Greeley

③ Supply Rock Mattress



Req'd $t_{mattress}$ = 2m
γ_r = 1950 kg/m³

$V_{mattress} = [(265 \times 21m) + 2(40m - 2(m)(21m)] \times 2m$
$= 12,726 m^3$
$= 27,297 t \rightarrow 28,000 t$

④ Slab-on-grade

$265m \times 40m \times 0.2m = 2,120 m^3 \rightarrow 2200 m^3$

⑤ Caisson Ballast

$V_{cells} = 145,616 m^3 = 310,264 t \rightarrow 311,000 t$

⑥ Concrete For Caissons

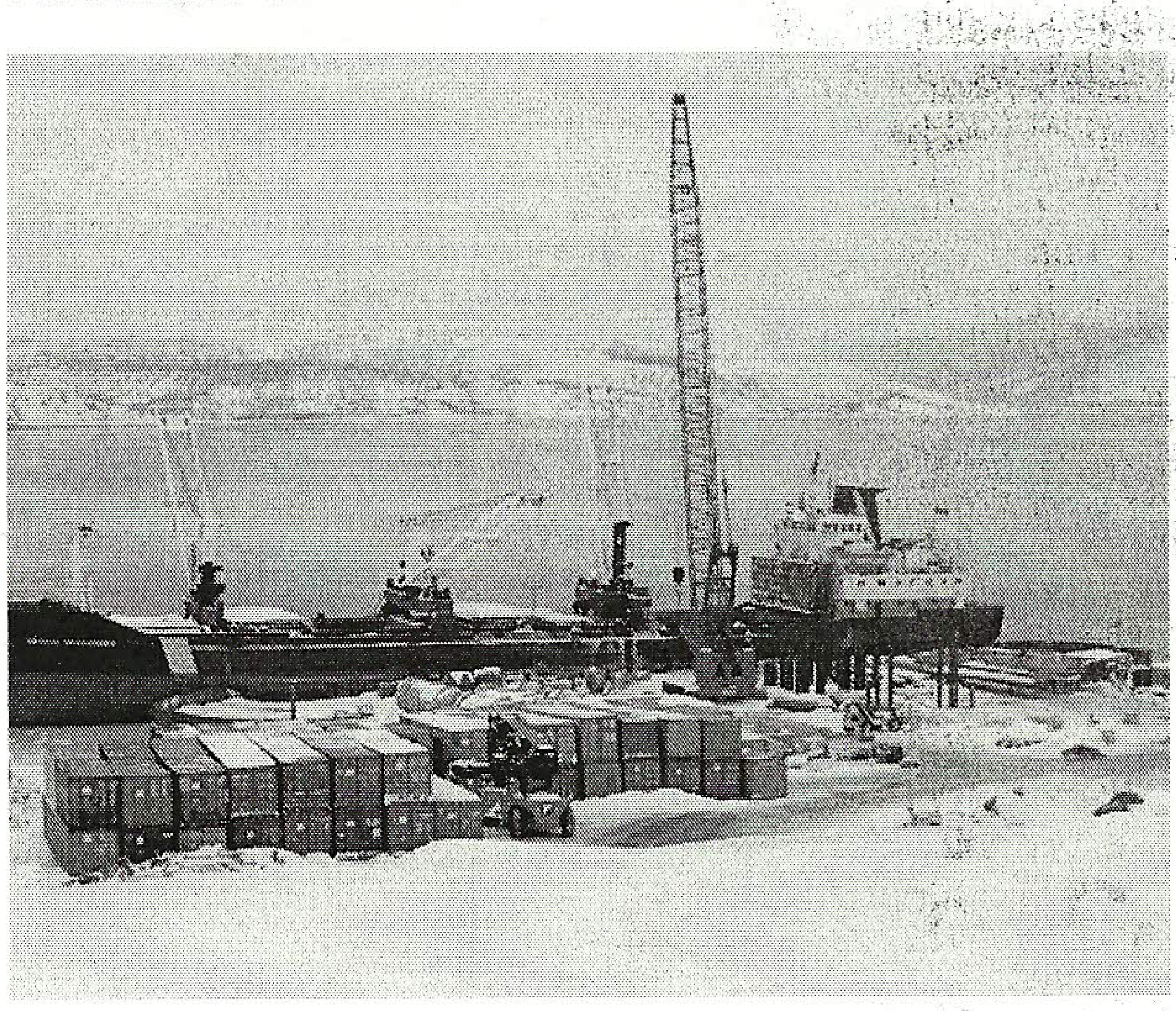
$V = \frac{74m^3}{m} \times 28m/unit \times [\underset{\text{along L}}{8 units} + \underset{\text{along sides}}{2 units} + \underset{\text{to tie to ground}}{\left(\frac{40-21}{21}\right)2}] = 22,496 m^3 \rightarrow 22,500 m^3$

from spreadsheet

APPENDIX J: REFERENCE REPORTS

Voisey's Bay, Labrador, CANADA

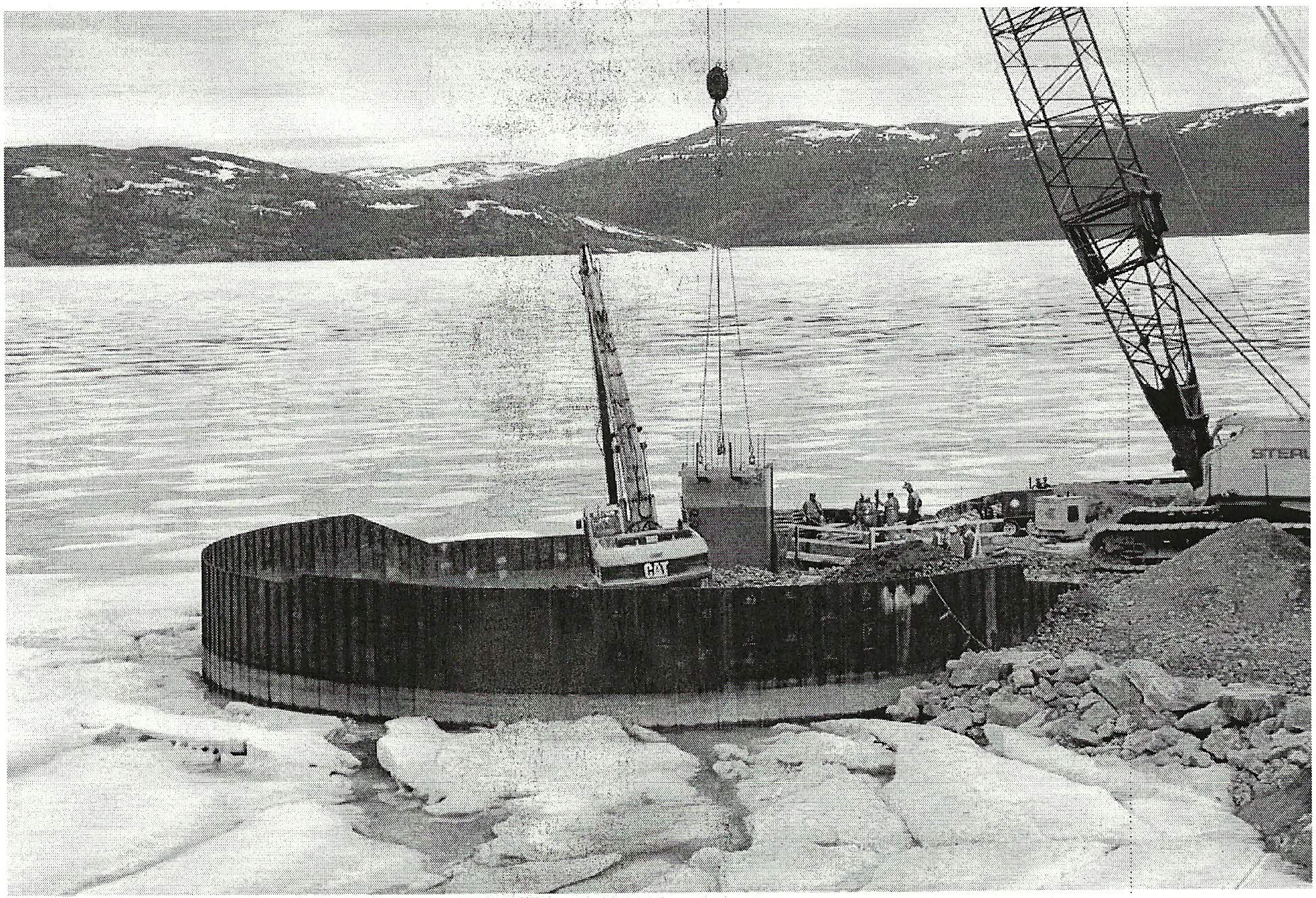
Construction of permanent port facilities



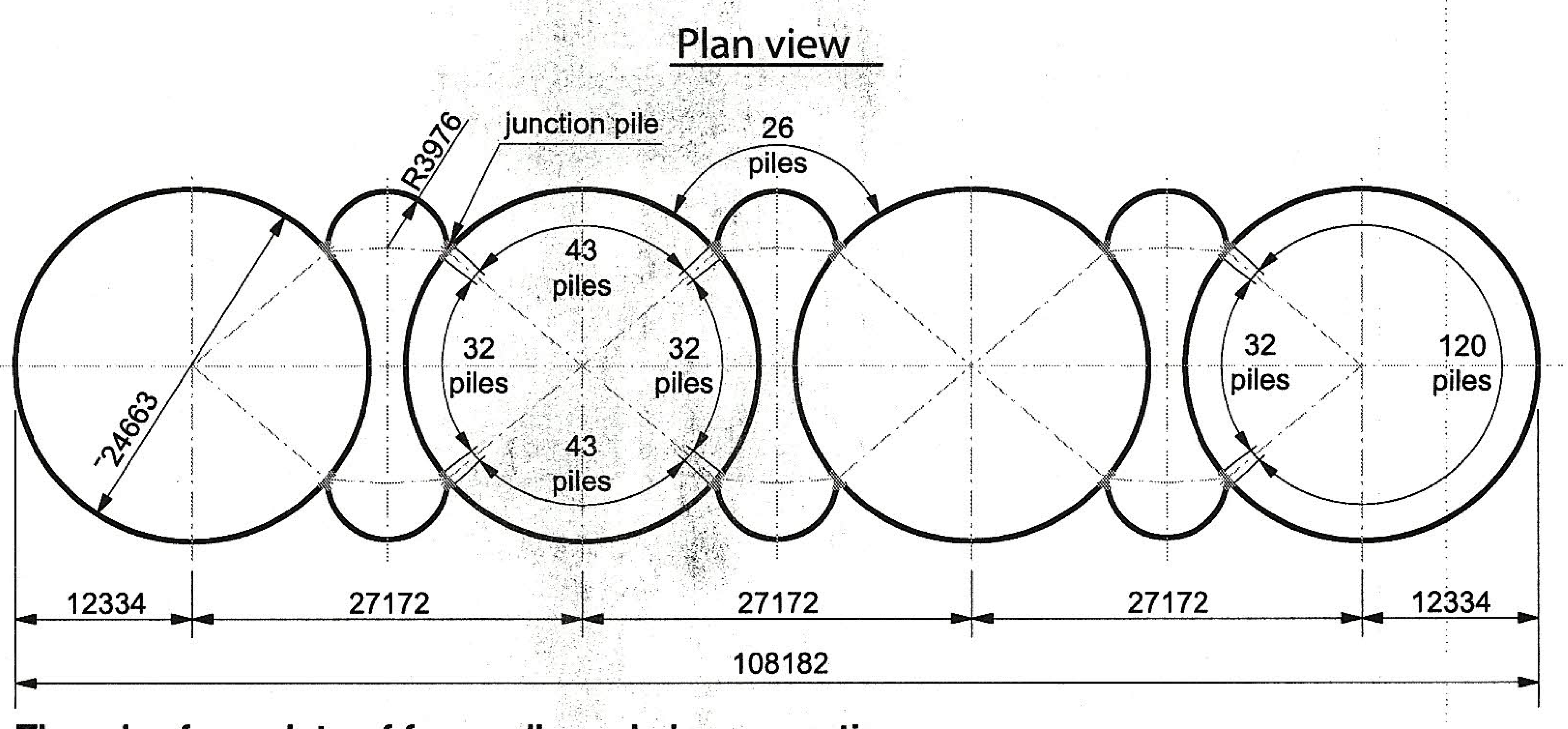
Supply ship docking at the sheet pile wharf in Voisey's Bay

Voisey's Bay is situated in a remote area on the north-east coast of Labrador, in the Canadian Arctic. One of the richest nickel-copper-cobalt finds in the world, the Voisey's Bay deposit was discovered in 1993, some 350 km north of Happy Valley-Goose Bay.

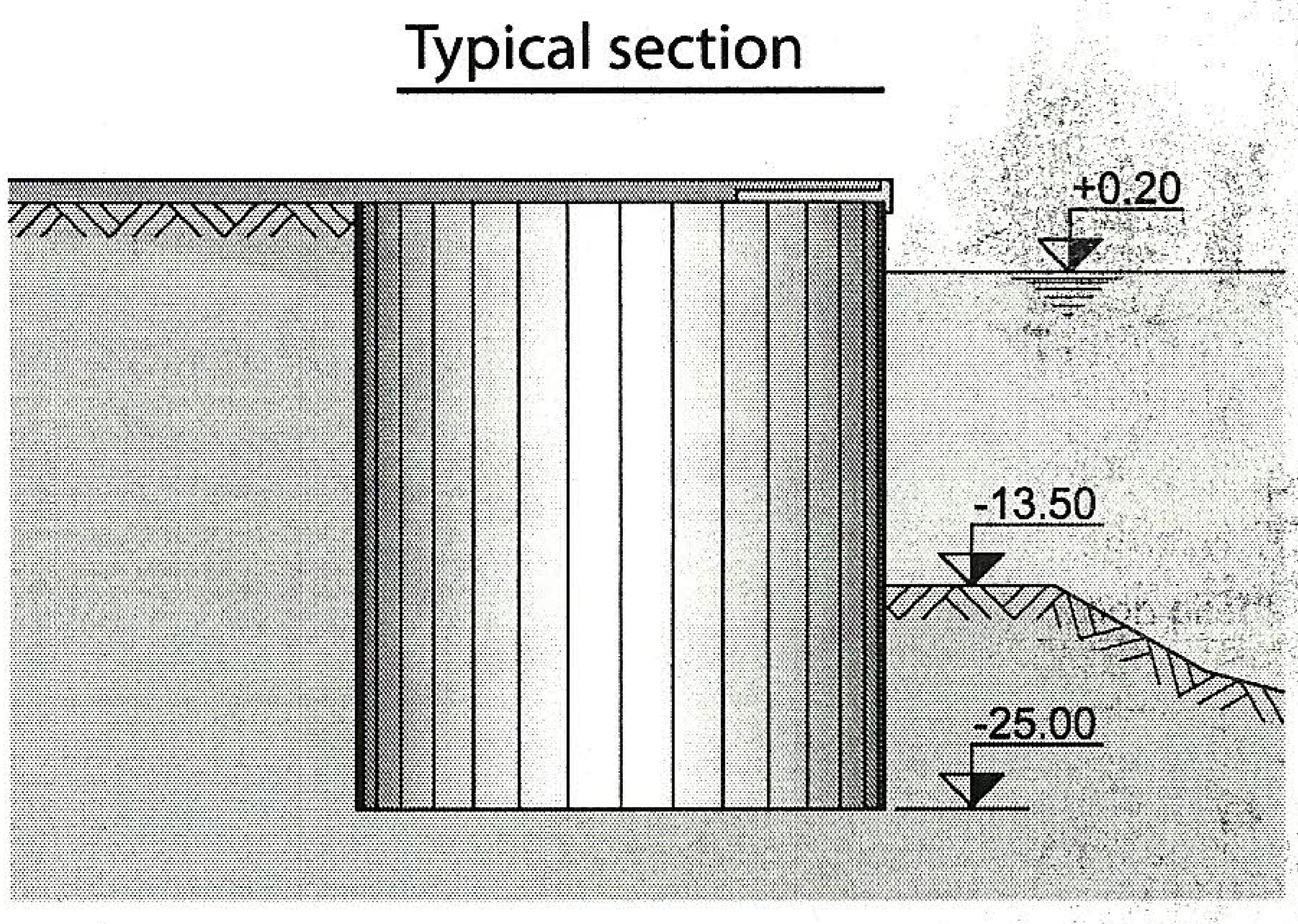
The Voisey's Bay Nickel Company (VBNC) built an integrated mine at the site which is now in operation. A harbour was required in order to import mine consumables and export the nickel concentrate. Construction of the wharf in nearby Anaktalak Bay began in summer 2004, and the main structure was completed in December 2004, with some ancillary work completed in late spring 2005. The new deep-sea wharf received its first ship on schedule in November 2005. The dock has an approximately 100-metre berthing face with a minimum draught of 13.5 metres. Westmar Consultants Inc. (marine structural design) and Jacques Whitford (geotechnical design) jointly submitted the design for a new deep-sea wharf in Anaktalak Bay in order to accommodate up to six supply ships and concentrate carriers per month.



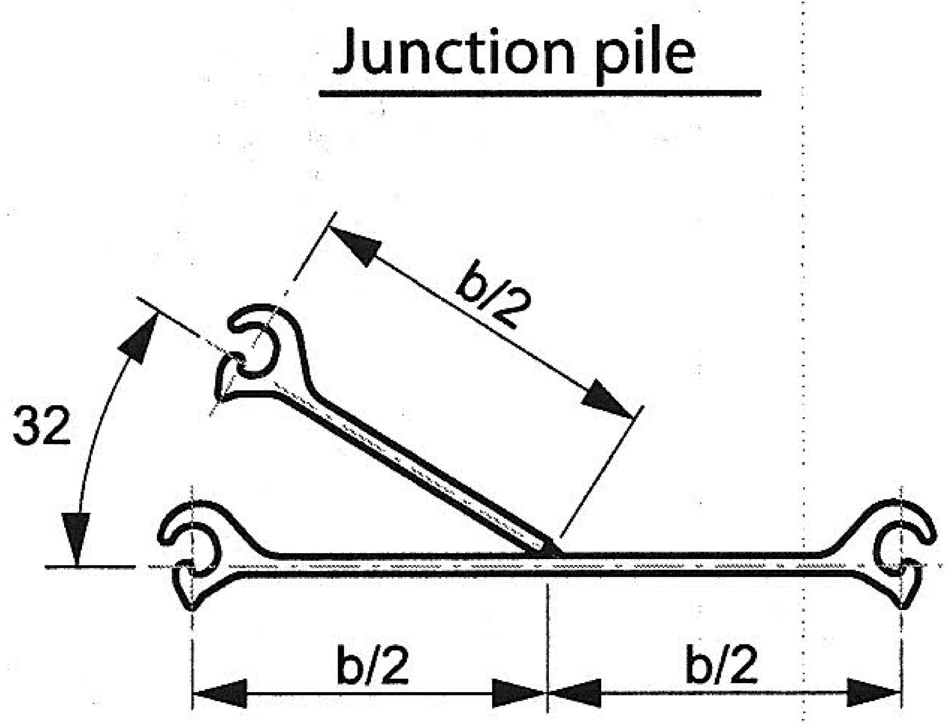
The design of the sheet pile cells had to take account of ice loads



The wharf consists of four cells and six connecting arcs



Cross-section of the AS 500 sheet pile cell



The junction piles were produced by one of Arcelor's subcontractors

A circular steel sheet pile gravity structure was selected as the main structure. Individual sheet pile cells were driven into the predominantly dense sand/gravel soil. Since the AS 500 sheet pile system does not require embedment into lower soil layers for statical reasons, it is a standard solution for extremely hard soil conditions. The design of the wharf was particularly challenging due to the fact that its construction had to be completed within one short ice-free season. Several geotechnical boreholes were not completed before the installation of the first sheet piles.

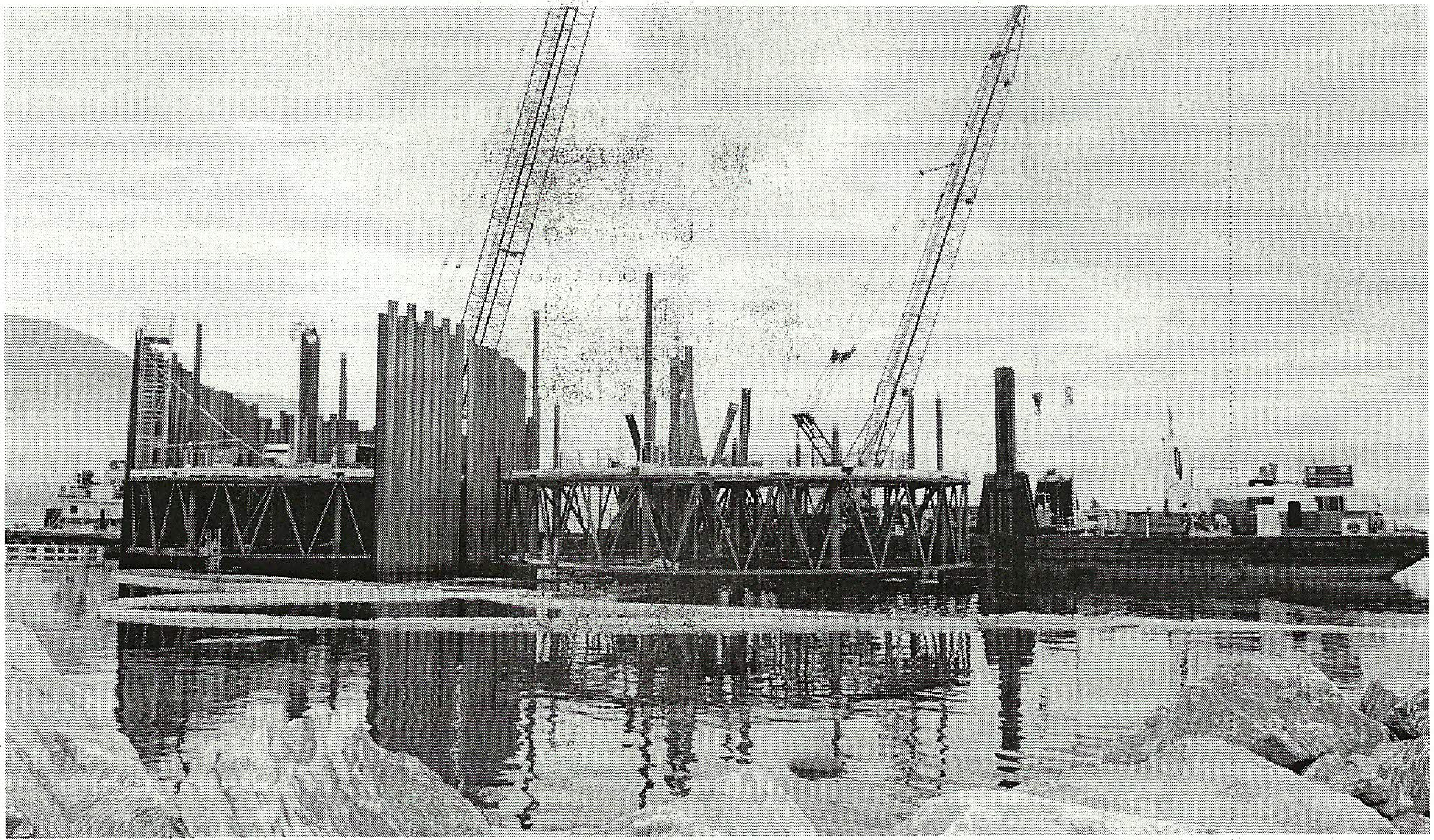
The geology of the Anaktalak Bay site can be simplified into three distinct soil layers overlying bedrock. The surface is characterised by a significant zone of soft to firm clay overlying a sandy layer containing cobbles and boulders. The rockfill for the cells and the backfill consist of well-graded angular material. Dredging of the very soft sediments had to be avoided. This led to the development of a state-of-the-art instrumentation plan to continuously monitor stability during construction. A set of curved precast concrete ice impact panels with a reinforced cope beam system supplemented the strength of the main structure. A variety of failure mechanisms (overturning, sliding, interlock failure, as well as horizontal and vertical shear failure) were analysed in the design of the sheet pile structure. Ice loads were an essential design consideration due to extreme winters with temperatures dropping as low as -40°C (-40°F). Special ice-impact beams were installed to take the horizontal loads.

The face of the marginal wharf is made up of four AS 500 cells joined together with six arcs. The sheet pile cells forming the face of the wharf also act as a retaining structure for backfill material. Scour protection was placed in front of the cells and the sheet piles were driven into it. Once the cells were placed, the area behind the wharf was backfilled with dredged soil.

For the construction of the four cells and six arcs, the following numbers of sheet piles were delivered by Arcelor's Canadian agent, Skyline Canada:

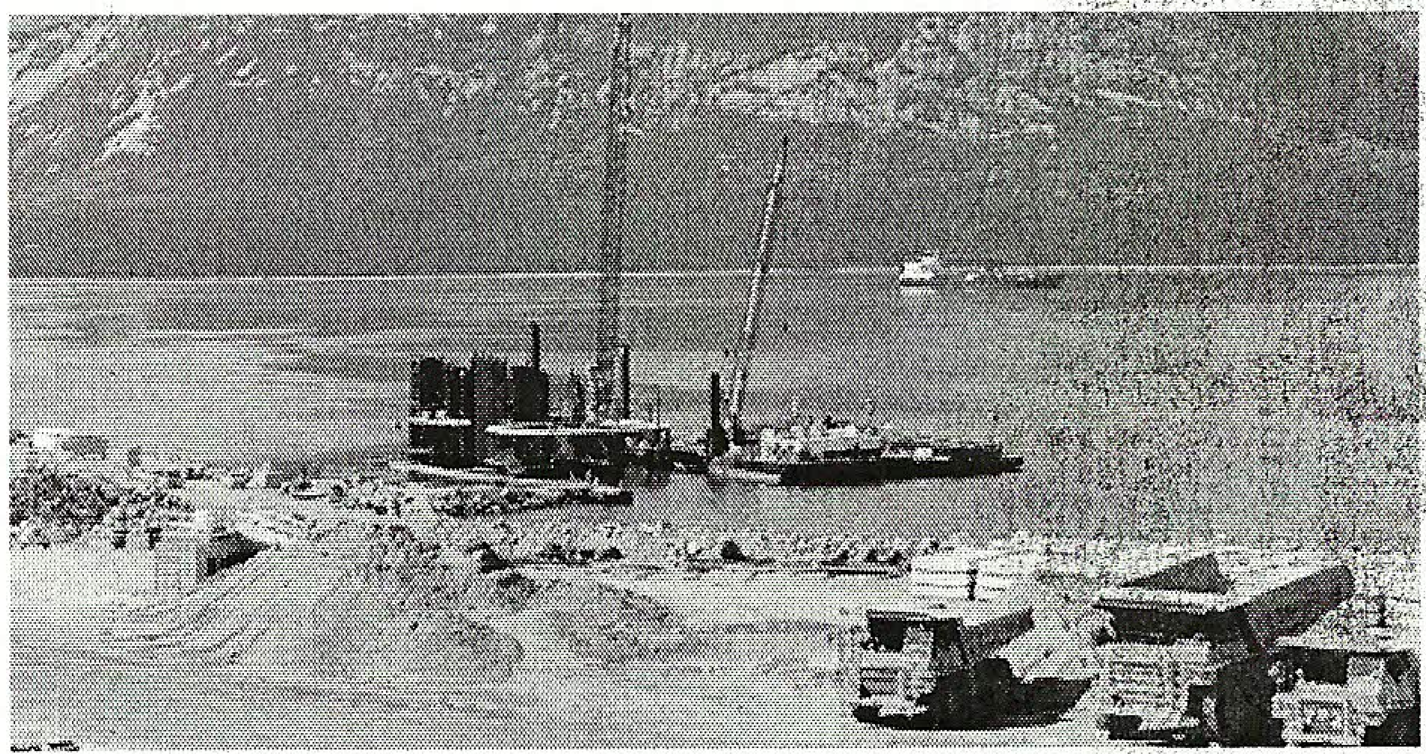
- 680 straight-web sheet piles
- 72 straight-web sheet piles bent by 7°
- 12 straight-web junction sheet piles.

Each main cell made of 150 AS 500 straight-web sheet piles and 4 junction piles has a diameter of 24.7 m. Each of the six arcs is made of 14 normal AS 500 piles and 12 bent piles in alternate positions. Secometal, a subcontractor of Arcelor, fabricated the bent piles. All the AS 500 sheet



The sheet piles were installed with the help of a template

piles are 26.7 m long and 12.7 mm thick. The piles have a guaranteed minimum interlock strength of 5,500 kN per running metre of interlock. Skyline Canada additionally delivered 19 spare piles including single, bent and junction piles. The new wharf design received an Award of Engineering Excellence from the Consulting Engineers of British Columbia in 2006.



Piles were driven with barge-mounted equipment

Owner:
Voisey's Bay Nickel Company (VBNC)

Contractor:
IKC-Borealis

Designer:
Westmar Consultants Inc and Jacques Whitford

Sheet piles:
AS 500-12.7

Pile length:
26.7 m

Steel grade:
S 355 GP

Total quantity of sheet piles:
1,640 metric tons

General installation procedure for AS 500 cells:

Step 1
- Installation of template and supporting piles
- Temporary positioning of top/lower platform as high/low as possible above/below water level

Step 2
- Positioning of four or more isolated sheet piles (usually the special junction piles)
- Verification of verticality, then fixing by tack welding to upper platform
- Threading of adjacent sheet piles

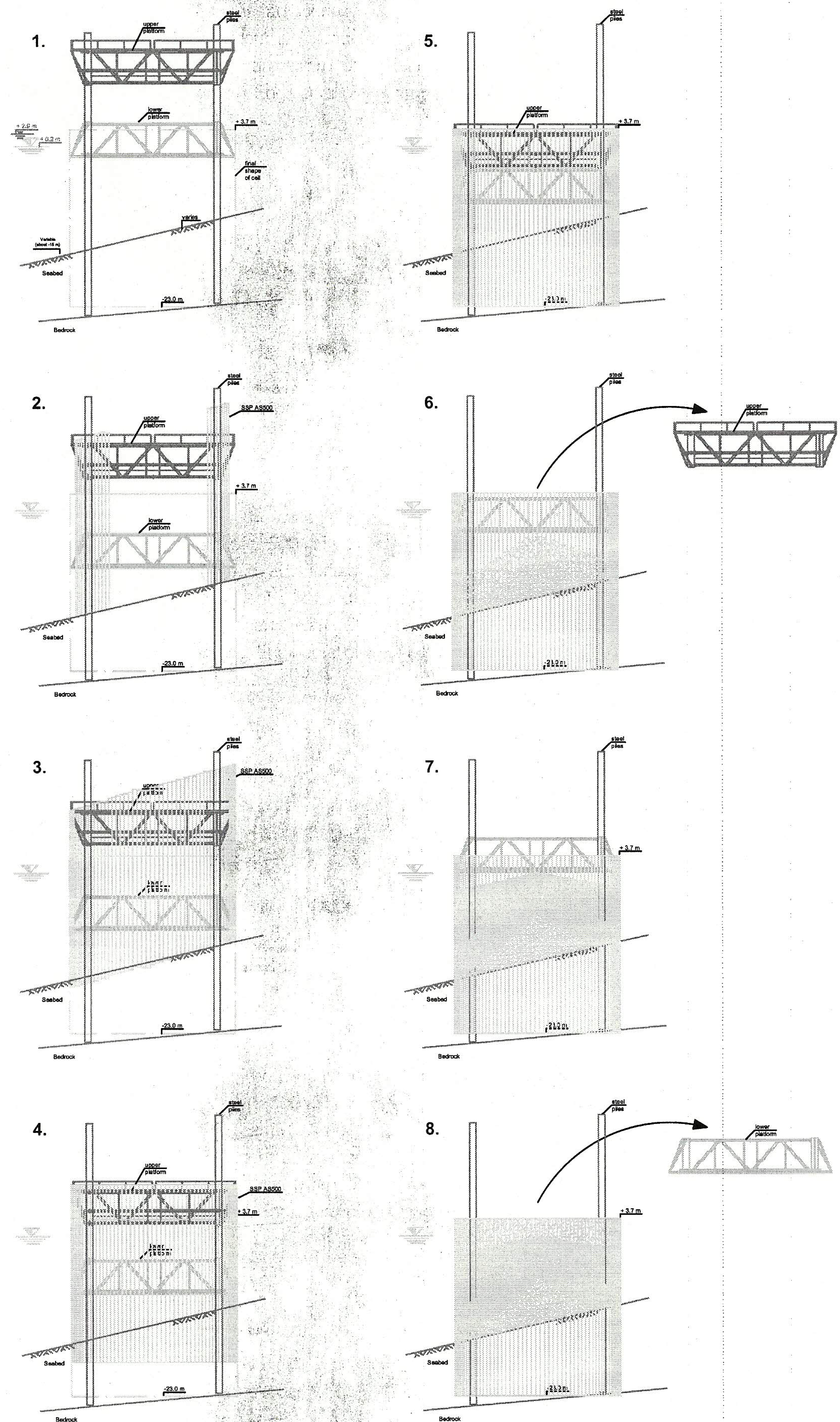
Step 3
- Closing of cells between special junction piles
- Threading of arc piles (2 or 4)

Step 4
- Driving of piles using staggered driving method after closing of the cell

Step 5
- Lowering of upper platform and driving of piles to design level

Step 6 & 7
- Filling of the cell
- Raising/Removal of platforms at appropriate times

Step 8
- Backfilling to the top of the cell
- Extraction of supporting piles ■


1.
2.
3.
4.
5.
6.
7.
8.
steel piles
upper platform
lower platform
+ 3.7 m
final shape of cell
varies
Seabed
Bedrock
-23.0 m
SSP AS500

Detailed Design of Marine Terminal in Edward's Cove, Labrador

Ryan A. MacPherson
Westmar Consultants Inc.
North Vancouver, BC, Canada

Harald G. Kullmann
Westmar Consultants Inc.
North Vancouver, BC, Canada

ABSTRACT

In 2003, Westmar Consultants Inc. undertook the detailed design of Voisey's Bay Nickel Company's marine terminal in Edward's Cove in northern Labrador. To facilitate year-round shipping, the design had to withstand the harsh ice conditions at the site. The design was complicated by environmental regulations which limited dredging of the weak seabed clay layer, resulting in difficult foundation conditions. The construction of the terminal was completed in July 2005 and the first ship arrived in November 2005. The terminal serves as a transportation hub for re-supply cargo and nickel concentrate export from mine operations.

KEY WORDS: VBNC; SSPC; cell; interlock; monitor; instrumentation; ice strengthening.

NOMENCLATURE

B = effective width of section
k_{01} = basic subgrade modulus
k_h = horizontal subgrade modulus
k_v = size corrected subgrade modulus
L = centre-to-centre distance of main cell and arc cell
n_h = constant
P = lateral earth pressure inside cell
r = radius of cell
t = interlock stress in main cell
t_{cw} = interlock stress in cross wall
Z = depth
θ = angle from SSPC axis to connecting pile

INTRODUCTION

Voisey's Bay Nickel Company's (VBNC) marine terminal is located in Edward's Cove, Anaktalak Bay, on the northern coast of Labrador. The terminal serves as the transportation hub for re-supply cargo and nickel concentrate export for VBNC's mine operations. The terminal is used for year-round shipping and the bay is subject to ice conditions.

As is the case for many Arctic locations, the terminal is located in an environmentally sensitive area. The resulting limitations on the scope of the in-water work proved to be challenging for the design of the wharf structure; in addition, the existing seabed comprises soft sediments which provide low bearing capacity for foundations.

Construction of the VBNC wharf structure incorporated conventional, proven structural solutions with innovative design and analysis to realize an effective solution for this difficult site. Analysis results were coupled with a monitoring program to assist in the construction process.

SITE CONDITIONS

Geotechnical Conditions

The geotechnical site investigation was carried out by Jacques Whitford Company Inc. (JWC) of St. John's, NL (formerly Newfoundland Geosciences Ltd.). The investigation showed that the seabed consisted of the following stratification:

- Clayey sand (1.0 m to 6.5 m) overlying;
- Clay (1.0 m to 3.8 m) overlying;
- Sand with gravel (2.5 m to 8.9 m) overlying;
- Bedrock.

Basic soil properties were provided by JWC (presented in Table 1).

Table 1. Basic Seabed Soil Properties

Stratum	Effective Friction Angle (deg.)	Effective Cohesion (kPa)	Total Unit Weight (kN/m^3)	Effective Unit Weight (kN/m^3)
Drained Analysis				
Clayey Sand	30	0	20	10
Clay	27	10	19	9
Sand With Gravel	32	0	20	10
Undrained Analysis				
Clay	0	40	19	9

The results of lab testing indicated that the clay layer had relatively low shear strengths and was soft in comparison to the other layers overlying the bedrock. Table 2 presents the design parameters developed by JWC.

Table 2. Geotechnical Design Parameters

Stratum	Lateral Earth Pressure Co-efficients		
	Active Pressure (K_a)	At Rest Pressure (K_o)	Passive Pressure (K_p)
Drained Analysis			
Clayey Sand	0.33	0.50	1.50
Clay	0.37	0.55	1.20
Sand With Gravel	0.30	0.47	1.70
Undrained Analysis			
Clay	1.00	1.00	1.00

Environmental Conditions

The Canadian Arctic is a pristine and sensitive environment. The directive for the project was to avoid dredging to the maximum extent practical. Environmental approval was provided on the basis that dredging would not be required.

Ice Conditions

Anaktalak Bay is subject to freeze-up (fast ice) during the winter months. Fast ice data used for developing loads on the wharf structures were obtained from others and are summarized as follows:

- Extreme maximum level ice thickness is 1.45 m (Dickins, 1977);
- Maximum average level ice thickness is 1.2 m (Dickins, 1977);
- Ice thickness at break-up is 0.8 m (Ice Center, 1992).

In addition to fast ice, the wharf structure is subject to loads generated by an ice bustle which forms at the face of the wharf. The ice bustle develops from tidal action and the repeated wetting and freezing of the top surface of the ice. Technical information on ice bustles in the Arctic was obtained from three papers by Robert Frederking et al. (1977, 1980 and 1988). The ice bustles were estimated to grow at a rate proportional to the square root of the freezing degree days (Frederking, personal communication, Sep 16, 2002; Dickins, personal communication, Sep 19, 2002). Based on this information, the following design parameters were calculated for the ice bustle at VBNC's wharf:

- Average ice bustle width is 3.75 m;
- Extreme ice bustle width is 4.5 m;
- Average ice bustle thickness is 4.3 m;
- Extreme ice bustle thickness is 5.2 m.

Ice loading on the structure was developed using methods defined in CAN/CSA-S471 (1992) and API RP 2N (1995), and is based on the following limits:

- Limit stress (where the load is determined by failure/crushing of the ice against the structure);
- Limit energy (where the load is limited by the momentum of an isolated floe);
- Limit force (where the load is limited by driving forces behind the ice feature).

Thermal expansion and contraction of the ice, floe impact and loads from ice-breaking ships at the wharf were considered in the design. Loading was estimated as follows:

- Thermal: 250 kN/m;
- Floe impact: 3.4 MN or 400 kN/m;
- Ice-breaking ship: 4.9 MN or 200 kN/m.

Design ice pressures, based on CAN/CSA-S6 (2000) for ice at break-up, and CAN/CSA-S471 (1992) for pressure versus area curves, were used as follows:

- Global pressure: 500 kPa;
- Local pressure (0.8 m by 0.8 m): 2100 kPa;
- Local pressure (0.32 m by 0.32 m): 7000 kPa.

TERMINAL EQUIPMENT

The terminal's main equipment is a fixed slewing, luffing and shuttling shiploader, with a design peak loading rate of 1,500 tonnes per hour. The terminal is also used for loading and unloading general cargo using a Manitowoc 2250 Series 2 crawler crane, exerting a peak ground pressure of near 300 kPa. Containers on the wharf are handled with a top lift container handler with a peak design axle load of approximately 120 tonnes.

Use of the crawler crane and container handling equipment required that a large working (back-up) area be provided behind the wharf. The equipment also required unrestricted access to the wharf face to limit the crane's reach to within acceptable levels.

To provide the greatest flexibility at the wharf, all areas of the structure were designed for the heavy equipment and a uniform storage load of 35 kPa.

FOUNDATION SOLUTION

Gravity structures provide a means for retaining back-up fills and are generally capable of withstanding large lateral forces. Therefore, a gravity structure was considered for VBNC's wharf. A piled structure was not considered viable since they are generally not suitable for resisting the large ice forces generated in Arctic regions. Typical gravity structures found in the Canadian Arctic include both steel sheet pile cells (SSPC) and concrete cribs.

Foundation Limitations

The design of the wharf structure was made difficult by the soft clay layer coupled with dredging restrictions. Conventional practice would be to remove these soft sediments. As such, the design solution needed to satisfy the existing conditions.

The design solution also needed to be easily constructed during the months when ice is not present at the site: approximately six months between ice break-up in mid-June, and freeze-up in mid-December. As such, on-site construction time had to be limited.

Substructure Selection

Without removing the soft clay layer beneath the seabed, calculations indicated that the seabed material would not have adequate strength to withstand the large pressures from concrete cribs. In addition, concrete cribs would require the placement of a mattress layer to provide a level foundation surface. Installation of this mattress layer would increase the required construction time.

Based on proven performance and past success in Arctic locations, Westmar selected a SSPC structure for the wharf, with a freestanding cell height of approximately 19 m (see Fig. 1). This type of structure can be easily constructed and is well suited for use on native soils of varying conditions. However, SSPC structural stability is contingent on managing sheet pile interlock stresses, typically by ensuring that interior lateral pressures are minimized. High quality engineered fill is generally used to keep these internal pressures low.

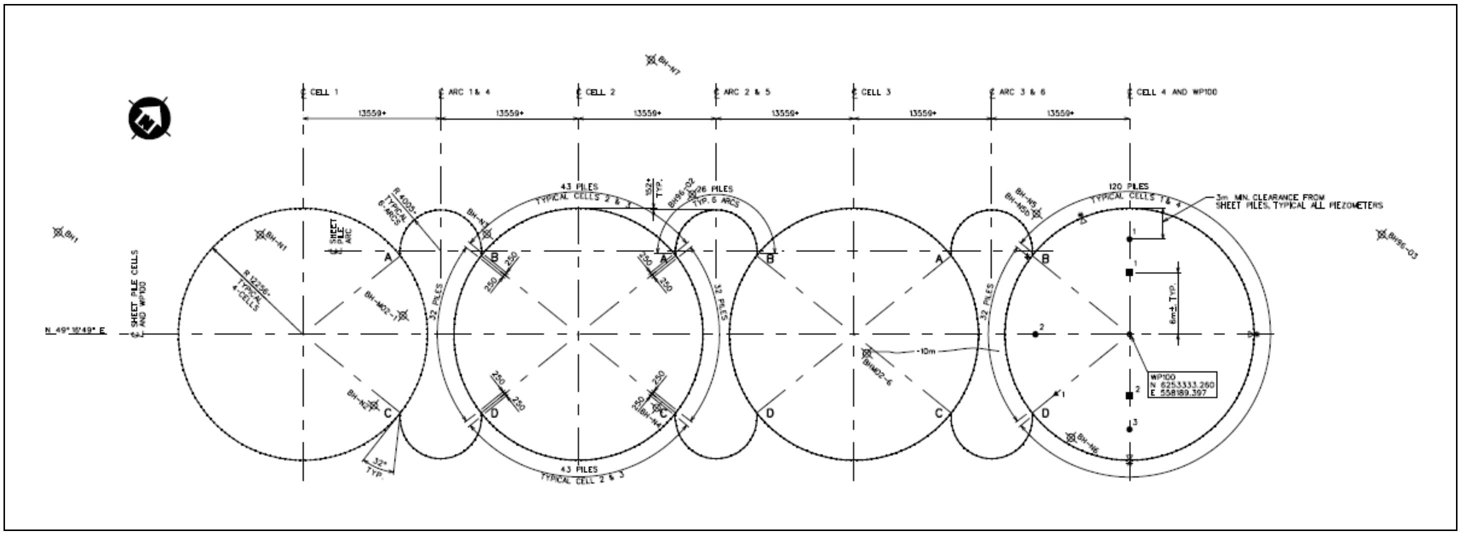


Fig. 1 Plan of SSPC Layout

Analysis

SSPC structures are subject to the following primary failure modes:

- Bursting (failure) of the sheet pile interlocks;
- Slip on vertical center plane;
- Slip on horizontal plane;
- Sliding instability;
- Bearing failure.

In addition to the above, overall slope stability was considered and the analysis completed by JWC.

Structural analysis was completed using the geotechnical recommendations from JWC, and the varying seabed soil conditions found at the site. A typical section through the SSPC structure is shown in Fig. 2.

Bursting of the Interlocks SSPC structures rely on interlock/hoop tension for overall cell stability. Internal pressures are translated into hoop tensions, much like a barrel. These tensions are transmitted between adjacent sheets by means of the interlocks, and failure of the interlocks results in an opening in the cells and loss of cell fill. A zipper effect can occur whereby the entire interlock fails, which ultimately would lead to cell failure. To guard against this phenomenon, an adequate factor of safety is required. AS 500-12.7 straight web sheets from Profil ARBED were selected for the cells, with interlock strength of 5,500 kN/m.

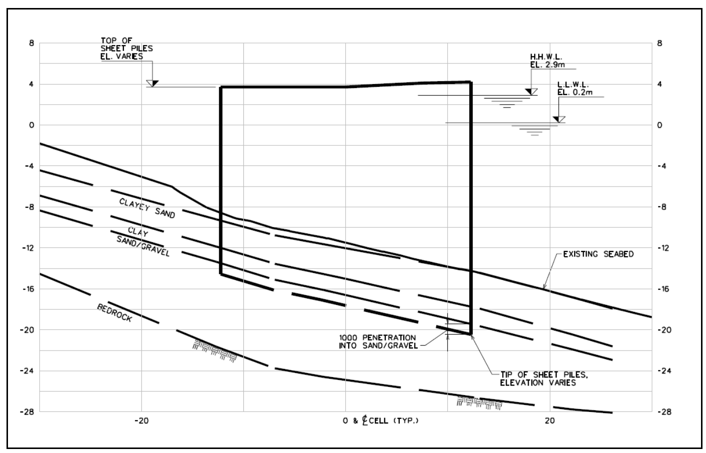


Fig. 2 Typical Section through SSPC Structure

To review interlock forces, a spreadsheet-based program was developed. The force on the interlocks is shown in Eq. 1 for main or arc cells and in Eq. 2 for cross walls.

$$t = Pr \quad (1)$$
$$t_{cw} = PL / \cos \theta \quad (2)$$

A typical profile of internal cell pressures (P) for the undrained soil condition is shown in Fig. 3. Pressures are based on at-rest (K_o) soil pressures acting on the inside of the cell, and mobilized passive pressures acting on the outside of the cell.

The steep increase in lateral pressures beneath the seabed represents the soft clay layer. As can be seen in Fig. 3, the result of the soft clay layer is a substantial increase in loading. Passive pressures acting on the outside of the cell are insufficient to overcome the internal pressure at the clay layer.

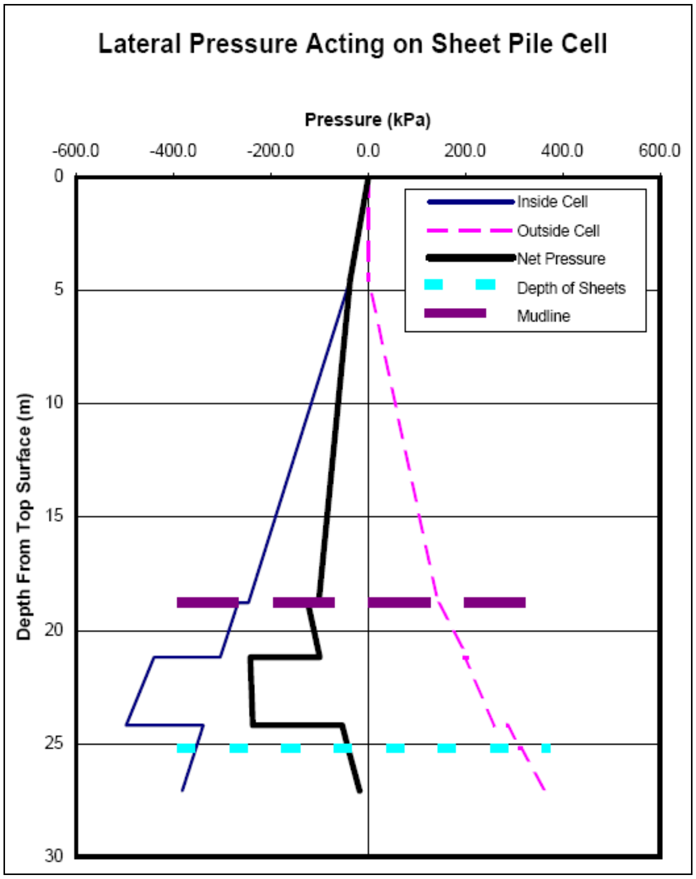


Fig. 3 Typical Profile of Undrained Lateral Pressure Acting on a Cell

In general, a factor of safety of 2.0 is considered adequate for sustained interlock tensions. Under infrequent loading, a factor of safety of 1.5 is considered acceptable.

The analysis for interlock strength indicated that for the drained condition, the factor of safety against failure was within acceptable levels. For the undrained condition, the factor of safety for a fully loaded cell was as low as 1.2. However, the undrained condition that exists during construction is a temporary condition which is relieved as pore water pressures dissipate.

The low factor of safety for a fully loaded cell in the undrained condition suggested that it would be necessary to complete the cell filling process in stages. A sensitivity analysis for the undrained condition verified that an adequate factor of safety was dependent on the fill rate, the water levels within the cells and the dissipation of pore pressure within the clay layer. These results suggested that the strength of the cells was highly dependent on the rate of cell filling and sequence of construction.

Slip on Vertical and Horizontal Planes Checking for slip on vertical and horizontal planes within the fill is necessary to verify overall cell performance. Fill slippage can result in excessive movement and potential collapse. Analysis of both failure mechanisms was completed with factors of safety found to be within acceptable levels. The soft clay layer did not pose a significant problem to the internal stability of the cells.

Sliding Instability Sliding stability of SSPC structures, like other gravity structures, is gained by base friction and mobilized passive pressure acting on the embedded portions of the structure. The SSPC structure is located on a sloping surface, and sliding calculations were completed by resolving driving and resisting forces into their normal and perpendicular components.

Initial calculations indicated that backfilling behind the wharf and over the existing soft seabed would result in unacceptably low factors of safety. Therefore, sensitivity cases varying the amount of native material removal behind the wharf were analyzed. It was determined that removing approximately two meters of native material was necessary to ensure a factor of safety of 1.5 or greater. This removal was contrary to the directive to eliminate dredging, but it was determined to be acceptable because the removal could occur after the SSPC structure was installed, which would maximize turbid water containment.

Bearing Failure Concrete cribs were ruled out as a possible solution for VBNC's wharf, since the soft clay layer did not have adequate strength and could not be dredged out. This soft clay layer also posed a problem for founding the sheet pile cells. To overcome this problem and ensure that a bearing failure of the SSPC structure could not occur, the sheet piles were driven to the dense sand and gravel layer below the soft clay. Driving the sheet piles into this layer resulted in the effective founding layer being beneath the soft clays.

Encapsulating the clay layer within the cells did have the potential for excessive settlement of the cell fill. This potential settlement was addressed by phasing construction, as discussed later.

MONITORING PROGRAM

Analysis of the SSPC structure indicated that interlock tensions within the cells were highly sensitive to fill levels and rates, and pore water pressures within the cells for the undrained, temporary condition. Rapid filling of the cells would lead to overstress of the interlocks and unacceptably low factors of safety against interlock failure.

To combat the potential for overstressing during cell filling, a program was developed to monitor interlock stresses using vibrating wire type strain gauges located at varying heights and plan locations of the SSPC structure.

In order to correlate actual interlock stresses with theoretical values and identify any variations, earth pressure cells and piezometers were used to verify actual vertical earth and water pressures. Actual vertical pressures were used in calculations to determine anticipated interlock stresses, which were then used for comparison to actual measured values.

Inclinometers were installed to monitor cell movement and sheet pile deflection. This information was used to further correlate actual interlock stresses with anticipated values, as well as to validate overall stability with respect to cell movement.

Only one of the four cells forming the SSPC structure was instrumented (strain gauges, earth pressure cells, piezometers and inclinometers) and monitored during construction. This cell was the first constructed, and was used to determine the rate of filling for the subsequent cells.

SUPERSTRUCTURE

Unlike many wharf structures, a straight wharf face was necessary for VBNC's operations. Cranes and container handling equipment needed unrestricted access to the berth in order to limit reach for general cargo loading and unloading operations. To construct a straight face, it was necessary to fill the area between the cells and the arcs.

The superstructure designed for the surface of the SSPC structure consists of a cast-in-place reinforced concrete integral cope wall and slab system (see Fig. 4). The waterside cope wall serves as a structural beam for supporting a suspended slab between the arcs and cells. The entire structure is designed for supporting the large pressures exerted by the crawler crane.

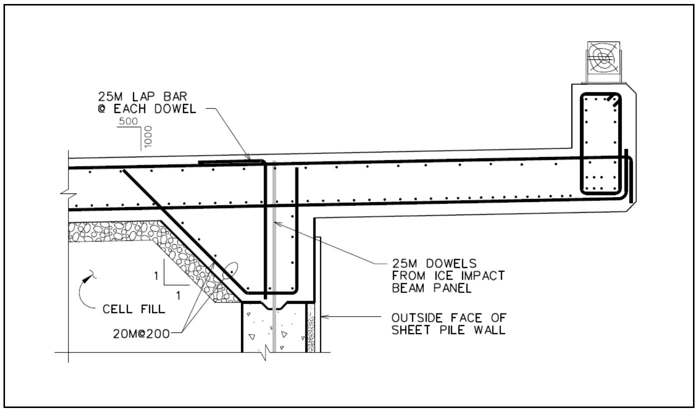


Fig. 4 Typical Section through Wharf Superstructure

The superstructure is supported directly on the cell fill and is designed as a slab-on-grade. Computer software was used to develop a structural model, which included slab and beam elements supported on elastic springs (see Fig. 5). A base subgrade modulus of $k_{01} = 100$ MN/m^3 for a 305 mm diameter plate was recommended by JWC. The base value was corrected for the size effects of the slab in accordance with Eq. 3:

$$k_v = k_{01}\,(B + 1 / 2B)^2 \qquad (3)$$

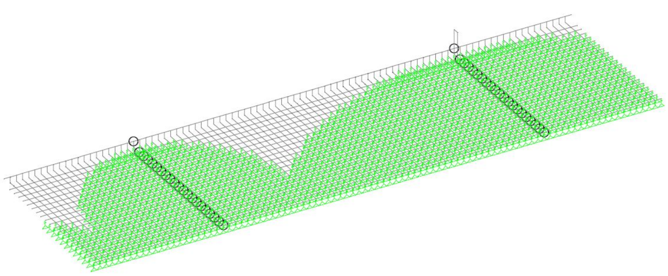


Fig. 5 Structural Model for Wharf Superstructure

In addition to supporting the anticipated vertical loads from the terminal equipment, the cope wall and slab superstructure incorporated the mooring bollards and supporting hardware for the fender system.

Due to the presence of the soft clay layer, differential settlement between adjacent cells and arcs was anticipated. As such, movement joints which enable rotation were incorporated into the superstructure. These joints were provided at all cell and arc centerlines.

ICE STRENGTHENING

The SSPC structure selected to form the wharf structure at VBNC's terminal was an economical solution to satisfy the poor ground conditions at the site. However, the sheet pile interlocks comprising the structure are highly susceptible to ice-loading damage. Therefore, a means of strengthening the cell sheet piles was needed.

System Development

High localized ice pressures and forces generated from ice-breaking ships and floe impacts are imparted onto the SSPC structure. These forces are transmitted to the face of the sheet piles and then transferred to the cell fill. Through this transmission of forces, there is the potential for damage to the interlocks which would result in the loss of interlock strength and the potential for interlock failure.

One alternative considered for strengthening the ice impact zone was to create a grouted rock mass directly behind the sheet piles within the cell fill. This grouted mass would have a higher lateral strength than the cell fill, and would provide greater support to the sheet piles. This alternative would be effective, but required considerable volumes of grout and extensive quality control procedures to ensure success. In addition, the method would be time consuming. For these reasons, this alternative was considered problematic.

The strengthening system developed for VBNC's wharf was installation of a number of precast reinforced concrete panels designed to withstand ice impact forces (see Fig. 6). These precast panels were installed directly behind the sheet piles to provide increased resistance. The panels were constructed off-site, transported to the wharf, and easily and quickly set in place prior to final filling operations.

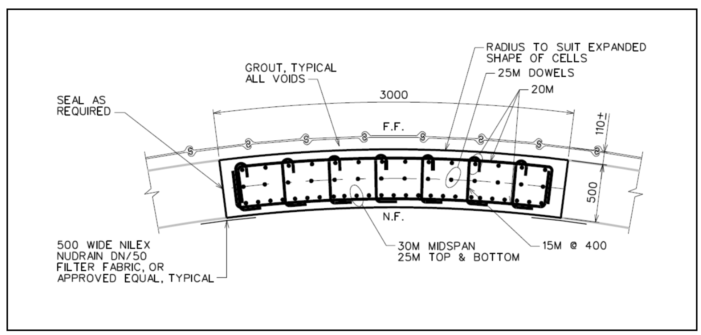


Fig. 6 Typical Section of Precast Ice Strengthening Panel

Design

The ice strengthening panels were designed using traditional methods for a slab on an elastic foundation. Computer software was used to develop a structural model of the panels. The model consisted of slab elements set on elastic springs to represent the foundation support from the internal cell fill (see Fig. 7). As recommended by JWC, springs were developed based on Eq. 4 for a linearly increasing horizontal subgrade modulus.

$$k_h – n_h\, Z / B \qquad (4)$$

A value of $n_h = 3{,}000$ kN/m^3 was used.

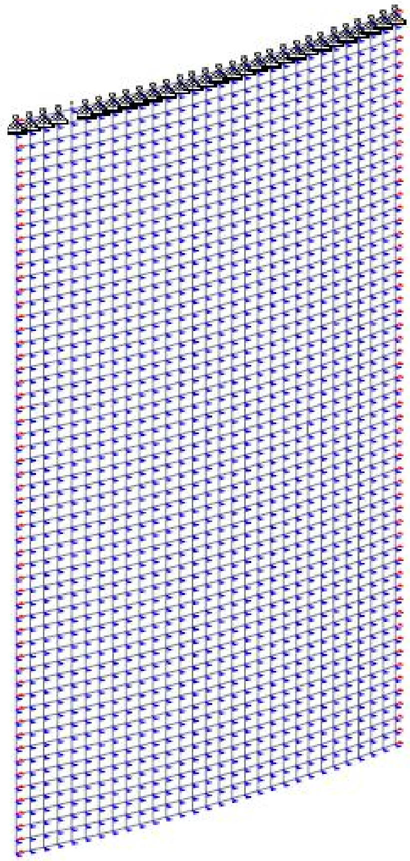


Fig. 7 Structural Model for Ice Strengthening Panel

The panels were constructed in 3.0 m lengths and curved to be 110 mm smaller than the inside radius of the expanded cells. The space between the cells and the panels was filled with grout to ensure uniform bearing between the panels and the sheets. The panels were tied into the wharf superstructure by means of projecting dowels and a shear key at the upper end of the panel.

SHIPLOADER FOUNDATION

The terminal's fixed shiploader required a strengthened single point foundation. The most economical solution for this type of foundation in competent soils is a spread footing. Although the structural cell fill was capable of supporting the anticipated loads, settlement of the seabed beneath the cells was a concern. Designing the shiploader for the anticipated settlement would have been cost prohibitive; therefore, the shiploader foundation was set on H-piles driven into the underlying bedrock.

CONSTRUCTION

Construction of VBNC's wharf began in the summer of 2004 and the first ship arrived at the facility in November 2005. The limited construction season of only six months precluded the completion of the wharf in one season.

Construction was initiated with the installation of the SSPC structure, starting with the instrumented cell. Each cell was constructed using a ring template secured to temporary piles. Once all the sheet piles were set around the template and aligned within tolerance, pile driving was completed on pairs of piles in stages around the cell.

After all piles had been driven at least one meter into the dense sand and gravel layer, filling operations began. Filling was completed in successive lifts, with the template being raised as required. On the first cell, instrumentation was continually monitored in order to ensure that interlock stresses were within acceptable levels. If interlock stresses were found to be exceeding design levels, filling operations were suspended until stresses dissipated. Results from the monitoring program set the rate of filling operations for the remaining three cells.

Upon completion of the SSPC structure, the ice strengthening panels were installed. Completing the installation of these panels marked the end of the first construction season.

In the summer of 2005, excavation and backfilling behind the wharf was completed, followed by construction of the cast-in-place cope wall and suspended slab system, and separate shiploader foundation. The lag between the completion of the SSPC structure and the superstructure ensured that most of the settlement within the cells and the underlying soft clay layer had occurred.

Construction was successfully completed on time in the fall of 2005.

CONCLUSIONS

SSPC structures have proven performance in Arctic conditions. Through extensive analysis and innovative design solutions, application of this conventional structure was successful at a site with difficult foundation conditions. Unique methods for ice strengthening minimized on-site construction time, while ensuring long-term structural performance. A load monitoring program incorporated as part of the construction phase of the project ensured that structural stability was maintained, and assisted in minimizing the construction duration. VBNC's wharf structure at Edward's Cove was successfully completed on time and well in advance of the first ship arriving in November 2005.

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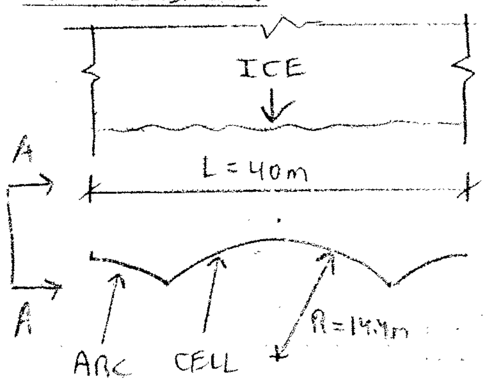
APPENDIX K: ICE FORCES

PARAMOUNT	DESIGN CALCULATIONS SHEET	DATE:	MAR 20/2010
		PAGE:	1 of 2

PROJECT:	ST. LAWRENCE MARINE TERMINAL	DOCUMENT NO.:	DC1-8700-07-04
PROJECT NO.:	8700-07	REVISION:	1
ITEM:	ICE LOADS - CELLULAR PILES	PREPARED BY:	PETER COLLINS/ANDREW SMALL

CELL DIMENSIONS:



LIMIT-MOMENTUM LOAD (IMPACT LOAD):

$$F_{lm} = 1.82\,h\,(p_e L V_i)^{0.67}\,([1+C_m]\,R_s\,\rho_i)^{0.33}$$

h = ICE THICKNESS = 1.5m

p_e = EFFECTIVE CRUSHING PRESSURE *ASSUME $p_e = 2.0 \times 10^6$ Pa

L = LENGTH OF ICE = 40 m

V_i = ICE SPEED * ASSUME ICE SPEED = WAVE SPEED = 5 m/s

C_m = ADDED MASS FACTOR = $\dfrac{0.9h}{(2z - 0.9h)} = 0.0317$

z: WATER DEPTH = 22m

R_s = RADIUS OF STRUCTURE * ASSUME $R = \frac{L}{2} = 20m$

ρ_i = DENSITY OF ICE = 980 kg/m³

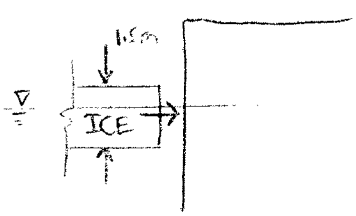
$F_{lm} = 1.82\,(1.5m)\,(2.0\times10^6\,Pa \times 40m \times 5\,m/s)^{0.67}\,([1+0.0317]\,20m \times 920\,kg/m^3)^{0.33}$

= 40.8 MN

ICE PRESSURE, $q_{ICE} = \dfrac{F_{lm}}{A_{impact}} = \dfrac{41\times10^6 N}{(1.5m \times 40m)} \times \dfrac{1kN}{1000N}$

= 683 kN/m²

≈ 700 kPa * GOVERNS



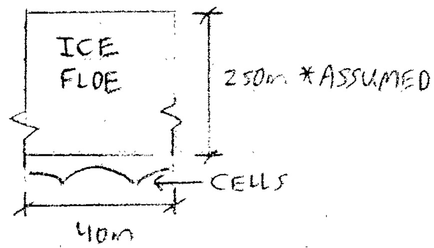
PARAMOUNT

DESIGN CALCULATIONS SHEET

DATE:		PAGE:	2 of 2
PROJECT:	ST. LAWRENCE MARINE TERMINAL	DOCUMENT NO.:	DC1-8700-07-04
PROJECT NO.:	8700-07	REVISION:	1
ITEM:	ICE LOADS - CELLULAR PILES	PREPARED BY:	PETER COLLINS / ANDREW SMALL

LIMIT - FORCE LOAD (RIDGE BUILDING PRESSURE):

$F_{2m} = (C_a \rho_a V_a^2 A) + (0.5 C_w \rho_w V_w^2 A) + (wL)$ $\quad \frac{kg}{m^3} \cdot \frac{m^2}{s^2} \cdot m^2$ [N]



A = FLOE AREA = 250 m x 40m = 10 000m^2
ρ_a = MASS DENSITY OF AIR = 1.34 kg/m^3
V_a = AIR VELOCITY @ 10m = 36 m/s
C_a = DRAG COEFF. = 3×10^{-3}
C_w = " " = 5.5×10^{-3}
ρ_w = 1025 kg/m^3
V_w = CURRENT VELOCITY = 0.51 m/s
w = AVERAGE PACK ICE PRESSURE = 50 kN/m * ASSUME
L = WIDTH OF FLOE = 250m

$F_{2m} = (3\times10^{-3} \times 1.34\,kg/m^3 \times (36m/s)^2 \times 10\,000\,m^2) + (0.5 \times 5.5\times10^{-3} \times 1025\,kg/m^3 \times (0.51m/s)^2 \times 10\,000\,m^2)$
$+ (50\,000\,N/m \times 40m)$

$= 52\,099 + 7332 + 2\,000\,000$ [N]

$F_{2m} \simeq 2$ MN

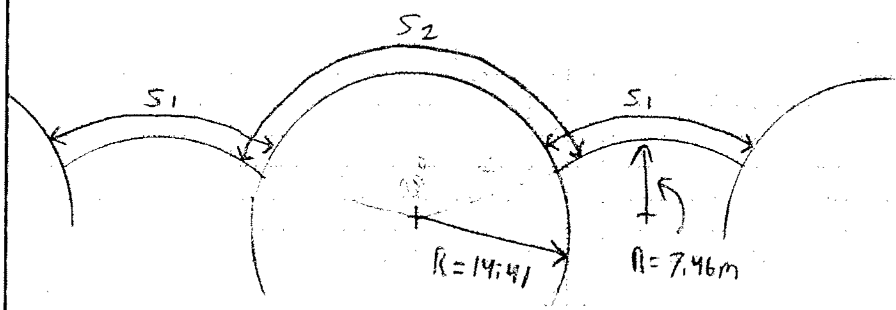
ICE STRESS, $\sigma_{ICE} = \frac{F_{2m}}{A} = \frac{2\times10^6 N}{(1.5m \times 40m)} \times \frac{1kN}{1000N} = 33$ kPa

APPENDIX L: DETAIL DESIGN: ICE STRENGTHENING PANELS

PARAMOUNT	DESIGN CALCULATIONS SHEET	DATE:	MAR 26/2010
		PAGE:	1 of 4

PROJECT:	ST. LAWRENCE MARINE TERMINAL	DOCUMENT NO.:	DC1-8700-07-012
PROJECT NO.:	8700-07	REVISION:	0
ITEM:	ICE STRENGTHENING PANELS	PREPARED BY:	ANDREW SMALL

CELL DIMENSIONS



$S_1 = 22.13m$ ✓
$S_2 = 30.18m$ ✓

* ASSUME 3m PANELS → S2 = 10 panels × 3m = 30m (OK)
S1 = 7 panels × 3m = 21m (OK)

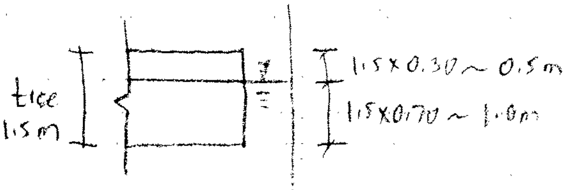
LOADING:

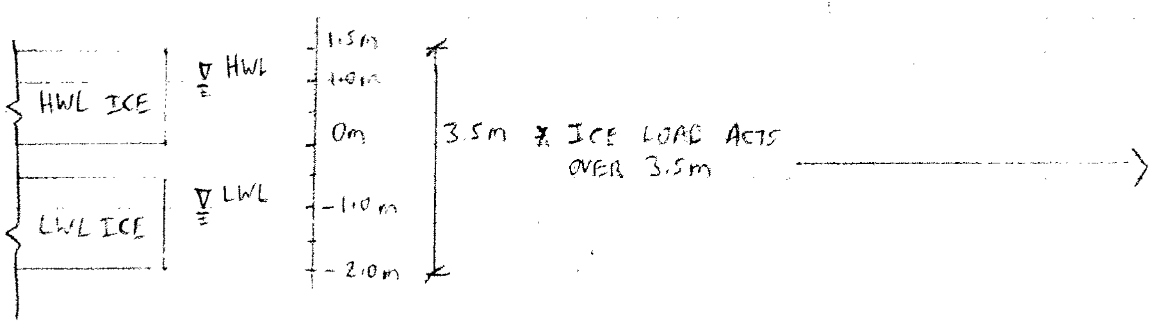
FROM ICE LOADS (DOC NO. DC1-8700-07-004 - REV. 2):

$q_{max} = 700$ KPa

TIDES = ± 1.0m
t_{ice} = 1.5m

* ASSUME 70% OF ICE SUBMERGED

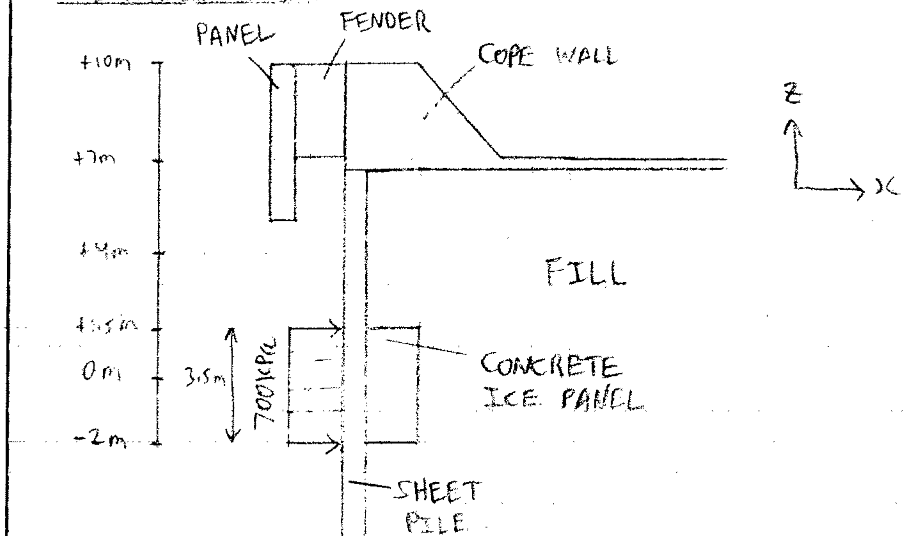




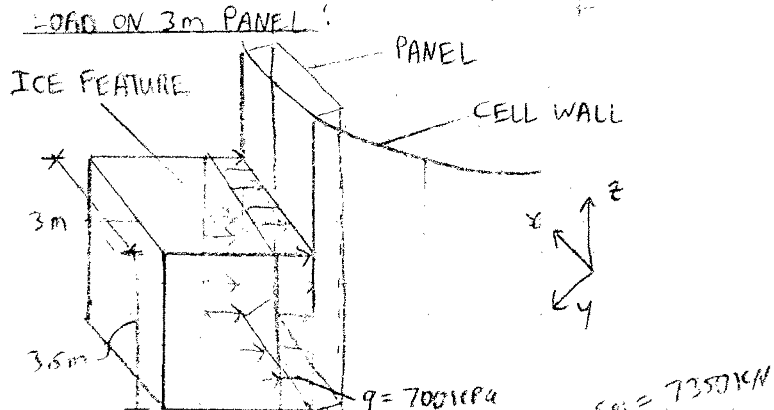
PARAMOUNT	DESIGN CALCULATIONS SHEET	DATE:	
		PAGE:	2 of 4

PROJECT:	ST. LAWRENCE MARINE TERMINAL	DOCUMENT NO.:	DC1-8700-07-012
PROJECT NO.:	8700-07	REVISION:	0
ITEM:	ICE STRENGTHENING PANELS	PREPARED BY:	ANDREW SMALL

ELEVATION DIMS:

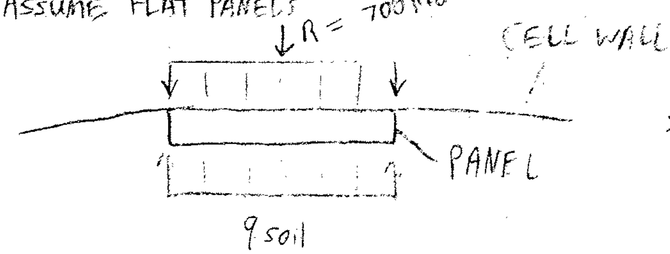


LOAD ON 3m PANEL:



* ASSUME FLAT PANELS

$R = 700\text{kPa} \times 3\text{m} \times 3.5\text{m} = 7350\text{kN}$



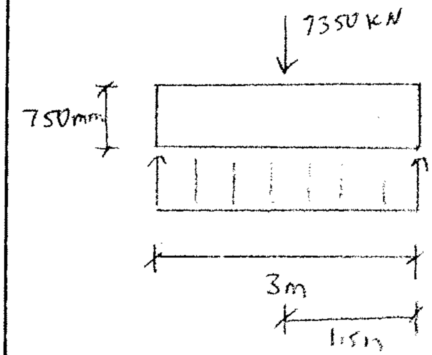
* TREAT AS SPREAD FOOTING →

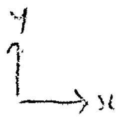
PLAN

PARAMOUNT	DESIGN CALCULATIONS SHEET	DATE:	
		PAGE:	3 of 4

PROJECT:	ST. LAWRENCE MARINE TERMINAL	DOCUMENT NO.:	DC1-8700-07-012
PROJECT NO.:	8700-07	REVISION:	0
ITEM:	ICE STRENGTHENING PANELS	PREPARED BY:	ANDREW SMALL

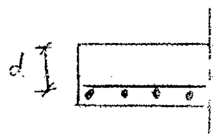
BENDING ABOUT Z-AXIS:





$$q_{sf} = \frac{\text{FORCE}}{\text{CONTACT AREA}} = \frac{7350\ kN}{3m \times 3.5m} = 700\ kPa$$

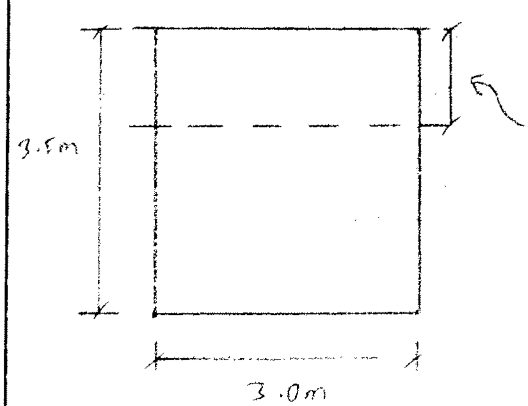
* ASSUME RESISTANCE $\geq$ 700 kPa OF FILL MATERIAL

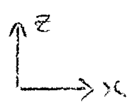


$$M_f = \left(700\ \frac{kN}{m^2} \times 1m\right) \times 1.5m \times \frac{1.5m}{2} = 788\ kN \cdot m$$

$$d = 750 - 75mm - 25.2 - \frac{25.2}{2} = 637.2\ mm$$

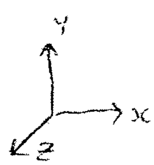
CLEAR COVER — USE 25M BARS

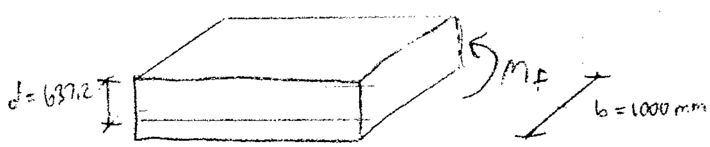




1m DESIGN STRIP

(b = 1000 mm)





CSA-A23.3-04, TABLE 2.1 (?)

let $M_r \geq M_f \rightarrow K_r = \frac{M_r}{bd^2} \times 10^6 = \frac{788\ kN \cdot m}{(1000 \times 637.2^2)} \times 10^6 = 1.94$

$f_c' = 30\ MPa$ } $\rho = 0.68$

PARAMOUNT	**DESIGN CALCULATIONS SHEET**	DATE:	
		PAGE:	4 of 4

PROJECT:	ST. LAWRENCE MARINE TERMINAL	DOCUMENT NO.:	DC1-8700-07-012
PROJECT NO.:	8700-07	REVISION:	0
ITEM:	ICE STRENGTHENING PANELS	PREPARED BY:	ANDREW SMALL

$A_s = \rho bd = 0.006 \times 1000 \times 637.2 = 3823\ mm^2$

25M bars $\rightarrow$ $A = 500\ mm^2$ $\rightarrow$ # BARS $= \frac{3823}{500} \approx$ 8 BARS/1m strip

$A_s = 8 \times 500\ mm^2 = 4\,000\ mm^2$

CLAUSE 7.8.1 : $A_{s,min} = 0.002 A_g = 0.002 \times 1000 \times 750 = 1500\ mm^2$ (OK)

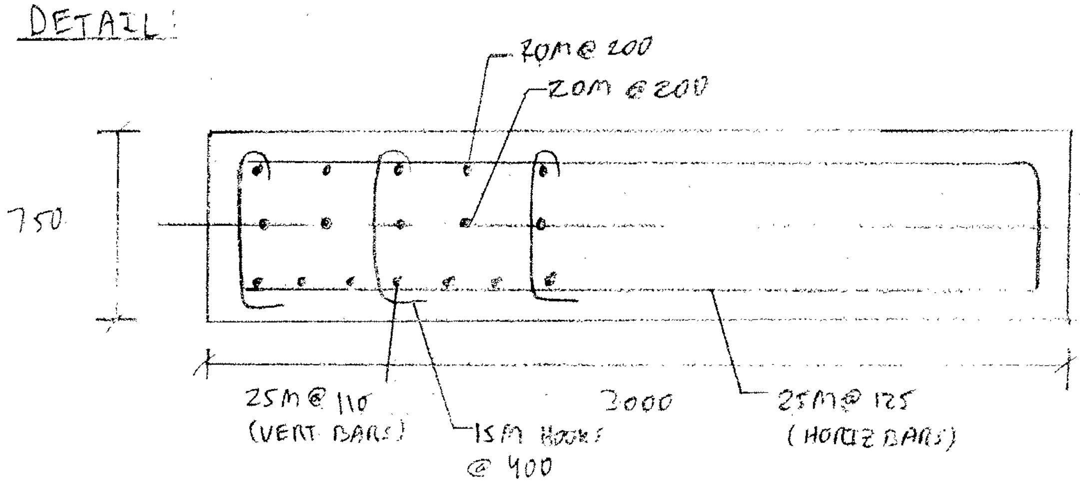
FOR 3.5 m STRIP $\rightarrow$ # BARS $= 8 \times 3.5m = 28$ BARS

BAR SPACING, $s \approx \frac{3500\ mm}{28} = 125\ mm$

* USE SAME REINFORCEMENT FOR BENDING ABOUT y-AXIS

** ADD HOOKS & DOWELS AS PER VOISEY'S BAY CASE STUDY (ATTACHED)

DETAIL:



APPENDIX M: DETAIL DESIGN: SLAB-ON-GRADE

PARAMOUNT	DESIGN CALCULATIONS SHEET	DATE:	mar 18/2010
		PAGE:	1 of 4

PROJECT:	St. Lawrence Marine Terminal	DOCUMENT NO.:	DC7-8700-07-006
PROJECT NO.:	8700-07	REVISION:	0
ITEM:	Prelim. Slab-on-Grade	PREPARED BY:	Robert Hunt

40' Dry Freight Container:

max gross weight: 67,200 lbs
payload: 59,417 lbs

Length: 40'
width: 8'
height: 8'6"

Tractor trailer
gross weight limit (US): 80,000 lbs
assume tire pressure: 120 psi
assume dual spacing: 15 in

Vehicle loads

- assume gross weight divided equally on 4 rear axles: 20 kips/axle
- assume dual wheel spacing to be: 15 in
- assume wheel assembly spacing: 40 in
- 4 wheels per axle.
- Tire inflation pressure assume: 120 psi

$$\text{Tire contact area} = \frac{\text{wheel load}}{\text{inflation pressure}}$$

$$= \left(\frac{\frac{20{,}000}{4}}{120}\right) = 41.7 \text{ sq.in} \approx \text{assume } 50 \text{ in}^2 \text{ for convinence}$$

Subgrade & Concrete data

Subgrade modulus, k 27.1 MPa, 100 pci (assumed value)

Using $f'_c = 35$ MPa, Concrete Flexural strength, MR = 4.3 MPa (640 psi)

PROJECT:	St. Lawrence Marine Terminal	DOCUMENT NO.:	DC1-8700-07-006
PROJECT NO.:	8700-07	REVISION:	0
ITEM:	Prelim. Slab-on-grade	PREPARED BY:	Robert Hunt

1. Safety factor:

Assuming the truck load is experienced 2-3 times a week. Over a 40-year design life there will be about 5760 load repetions. From Table 5.1 the stress ratio is ≈ 0.66 and the safety factor is 1.5.

2. Joint Factor:

Joint spacing will be greater than 4.5m so we will use a joint factor of 1.6.

3. Concrete working stress:

$$WS = \left(\frac{MR}{SF \times JF} \right) = \frac{640}{1.5 \times 1.6} = 267 \text{ psi}$$

4. Using figure 5.5 with dual spacing of 15in and contact area of 50 in^2 & a trial slab thickness of 8in the equivalent load factor F = 0.775 The equivalent single-wheel axle load = 0.775 × 20 = 15.5 kips

5. Slab Stress per 1,000 lb of axle load:

$$= \left(\frac{WS}{\text{axle load, kip}} \right) = \left(\frac{267}{15.5} \right) = 17.2 \text{ psi}$$

6. From Figure 5.4, stress of 17.2 psi, contact area 50 in^2, wheel spacing of 40 in, and a subgrade k of 100 pci gives an approximate thickness of 7.4 in. This is close to 8in calculated with dual axles so will go will 8in slab.

8in = 203 mm

PARAMOUNT	DESIGN CALCULATIONS SHEET	DATE:	Mar 18/2010
		PAGE:	3 of 4

PROJECT:	St. Lawrence Marine Terminal	DOCUMENT NO.:	DC7-8700-07-006
PROJECT NO.:	8700-07	REVISION:	0
ITEM:	Prelim. Slab-on-grade	PREPARED BY:	Robert Hunt

Distributed Loads

design for two containers stacked.

Total weight: 134,400 lbs

Contact Area: 40 ft x 8 ft = 320 ft^2

Force = $\frac{134,400}{320}$ = 420 psf

Using Table 5-3. and assuming a Concrete flexural strength of 650 psi, subgrade K=100 pci a slab thickness of 5 in carries an allowable load of 900 psf, more than enough.

Quantities Check

thickness = 200 mm

terminal dimensions: L = 265 m, W = 40 m

Area: 265 x 40
= 10,600 m^2

To account for sides coming back on an angle, additional Area:

A = ½ x 40 x 20
= 400 m^2

Concrete: 10,600 m^2 x 0.2 = 2,120 m^3
400 m^2 x 0.2 = 80 m^3

2,200 m^3

PARAMOUNT	DESIGN CALCULATIONS SHEET	DATE:	MAR 18/2018
		PAGE:	4 of 4

PROJECT:	ST. LAWRENCE MARINE TERMINAL	DOCUMENT NO.:	DC1-8700-07-006
PROJECT NO.:	8700 - 07	REVISION:	0
ITEM:	Slab on grade reinforcement	PREPARED BY:	ROBERT HUNT

From the publication of Concrete Floors on Ground by James A. Farny Published by Portland Cement Association it discusses how it is recommended that a minimum of 0.1% reinforcing steel based on the cross sectional area of the slab, even higher for harsher climates. For Continuously Reinforced slabs where there are no contraction joints the percentage of reinforcement is 0.5% - 0.7% the slab cross-sectional area.

We will bump our slab thickness to 300mm. Using 1m section:

Cross-sectional Area: 300000 mm^2

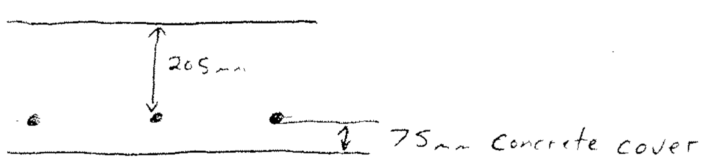
reinforcement = 0.4% × 300000 mm^2

= 1200 mm^2

Use 3-20M bars/meter

An assumption of 0.4% reinforcement would be used which would meet minimum requirements along with contraction joints. (BOTH DIRECTIONS)

Spacing:



APPENDIX N: DETAIL DESIGN: SHIP LOADER FOUNDATION

PARAMOUNT	DESIGN CALCULATIONS SHEET	DATE:	April 2/10
		PAGE:	1 of 3

PROJECT:	St. Lawrence Marine Terminal	DOCUMENT NO.:	DCI-8700-07-011
PROJECT NO.:	8700-07	REVISION:	0
ITEM:	Crane foundation design	PREPARED BY:	Robert Hunt

weight of Crane: 860 tons
weight per leg: 1917 kN

assume force is over a span of 8m for rail wheels.

- distribution load of $1917/8 = 240$ kN/m
- increase 10% to account for ore load ∴ 264 kN/m
- design as a wall foundation with a width of 1m
- use 171-CR rails for the crane (often used for heavy duty cranes)
- width is 6 inches ≙ 145mm

$f'_c = 30$ MPa $a_g = 20$mm, $f_y = 400$ MPa

Factored soil pressure

$$q_{sf} = P_f/A = \frac{1.5 \times 264}{1} = 396 \text{ kN/m}^2$$

- assume this is less than the ultimate limit state resistance

Footing depth

Factored shear: $V_f = q_{sf} \times b(427.5 - d_v)$

$$= 396(427.5 - d_v)$$

shear resistance $V_c = \phi_c \lambda \beta \sqrt{f'_c}\, b_w d_v$

- assume $\beta = 0.21$, where overall thickness is not greater than 350mm

$$V_c = 0.65 \times 1 \times 0.21 \times \sqrt{30} \times 1000 \times d_v$$

$$= 747.6\, d_v$$

equating $V_f < V_c$: $396(427.5 - d_v) = 747.6\, d_v$

$$d_v = \frac{427.5}{2.89}$$

$$d_v = 148\text{mm}$$

PARAMOUNT	DESIGN CALCULATIONS SHEET	DATE:	April 2/10
		PAGE:	2 of 3

PROJECT:	St. Lawrence Marine Terminal	DOCUMENT NO.:	DCI-8700-07-011
PROJECT NO.:	8700-07	REVISION:	
ITEM:	Crane Foundation design	PREPARED BY:	Robert Hunt

$d = d_v / 0.9 = 165\,mm$

using clear cover 75 mm & using 20M bars

$h = 165 + 75 + 20/2 = 250\,mm$

take $h = 300\,m$ $\therefore d = 300 - 75 - 20/2 = 215\,mm$

Design Moment

Maximum moment at Face:

$M_F = 0.396 \times 1000 \times 427.5 \times 427.5/2 = 36.2 \times 10^6\,Nmm$

$M_F = K_r b d^2 \times 10^{-6}$

$K_r = \dfrac{36.2 \times 10^6}{1000 \times 215^2}$

$= 0.78$

From Table 2.1 Concrete handbook : $\rho \approx 0.23\%$

$As_{required} = 0.0023 \times 1000 \times 215$

$= 495\,mm^2$

$As_{min} = 0.2\%\,A_g$

$= 0.002 \times 1000 \times 300$

$= 600\,mm^2$

Select 3-20M bars at $S = 333\,mm$ spacing. * USE 3-20M BARS IN BOTH DIRECTIONS

$S_{max} = (3h_s$ or $500\,mm)$, $S < S_{max}$ $\therefore$ OK

Development

using table 9.10 development length $l_d = 580\,mm$

Increase footing to 1.5 m

PARAMOUNT	DESIGN CALCULATIONS SHEET	DATE:	April 2/10
		PAGE:	3 of 3

PROJECT:	St. Lawrence Marine Terminal	DOCUMENT NO.:	DC1-8700-07-011
PROJECT NO.:	8700-07	REVISION:	
ITEM:	Crane Foundation design	PREPARED BY:	Robert Hunt

Re-check thickness

$d_v = 177\,mm$

$d = 197\,mm$

$h = 282 \therefore$ take $h = 300$ & $d = 215\,mm$

Moment

$M_f = 60.7 \times 10^6\ N\cdot mm$

$K_r' = 1.31 \Rightarrow \rho \approx 0.4\%$

$As_{required} = 0.004 \times 1000 \times 215$
$= 860\,mm^2$

$As_{min} = 600\,mm^2$

Select 3-20M bars at $S = 333$ $\therefore$ OK

Development

$678 - 79 = 602\,mm$ $\therefore$ OK

Connection

Column bearing: $B_r = 0.85\,\phi_c\,f'_c A_1 + \phi_s F_y A_{dowel}$

$B_r = 0.85 \times 0.65 \times 30 \times 145 \times 1000 \times 10^{-3} = 2403\ KN$

$B_r > P_f$ $\therefore$ OK

Footing bearing: $B_r = 0.85\,\phi_c\,f'_c A_1 \sqrt{A_2/A_1} + \phi_s f_y A_{dowel}$, $\sqrt{A_2/A_1} \leq 2$

$A_1 = 145 \times 1000 = 145 \times 10^3\,mm^2$
$A_2 = 1500 \times 1000 = 1500 \times 10^3\,mm^2$

$\sqrt{A_2/A_1} = 3.2 \leq 2 \Rightarrow$ use 2.

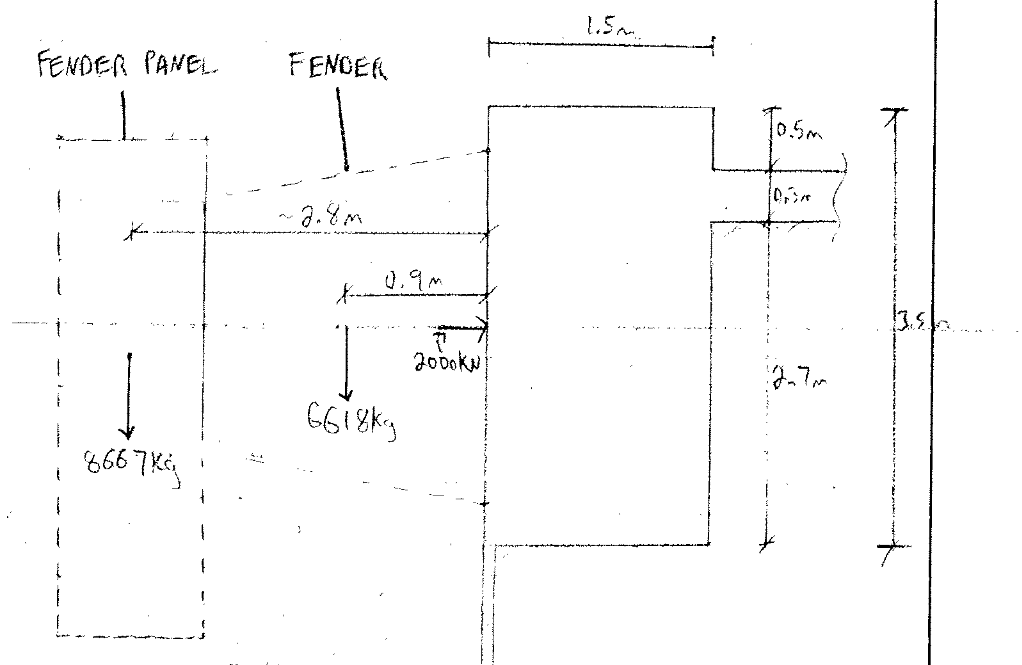
$B_r = 0.85 \times 0.65 \times 30 \times .145 \times 10^6 \times 2 = 4807\ KN > P_f$ $\therefore$ OK

APPENDIX O: DETAIL DESIGN: COPE WALL

PARAMOUNT	DESIGN CALCULATIONS SHEET	DATE:	Apr. 4 /2010
		PAGE:	1 of 5

PROJECT:	ST LAWRENCE MARINE TERMINAL	DOCUMENT NO.:	DC1-8700-07-013
PROJECT NO.:	8700-07	REVISION:	0
ITEM:	COPE WALL	PREPARED BY:	PETER COLLINS

Forces



Load Case 1: Berthing Load only

$P = 2000\,kN$

Take $P = 3500\,kN$

$M = Pl$
$= (3500)(1.75) = \underline{6125\ kN \cdot m}$

Load Case 2: No Berthing Load, self weight only

$$M = (0.9)\frac{(6618)(9.81)}{1000} + (2800)\frac{(8667)(9.81)}{1000} = 238.1\ kN \cdot m$$

Take = $\underline{357\ kN \cdot m}$

PARAMOUNT	DESIGN CALCULATIONS SHEET	DATE:	Apr. 4/2010
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PROJECT:	St. Lawrence Marine Terminal	DOCUMENT NO.:	DCI-8700-07-013
PROJECT NO.:	8700-07	REVISION:	0
ITEM:	Cope Wall	PREPARED BY:	Peter Collins

Flexure

Load Case 1:

Assume $f_c' = 35\,MPa$
$A_g = 20mm$
$f_y = 400\,MPa$

Assume 35M bars in two layers

Spacing: $1.4 d_b = 50mm \Leftarrow$ Governs
$1.4 A_g = 28mm$
$30mm$

75mm Clear Cover

$d = 1500 - 75 - 35.7 - (50/2) = 1360mm$

For 1m design strips ($b = 1000mm$)

Letting $M_r = M_f$

$M_r = K_r b d^2 \times 10^{-6}$ $\quad K_r = \dfrac{M_r}{bd^2 \times 10^{-6}} = \dfrac{6125}{(1000)(1360)^2 \times 10^{-6}} = 3.3$

From Table 2.1, $\rho = 1.08\%$

$A_s = \rho b d = (1.08)(1000)(1360) = 14688\,mm^2$

For Symmetry, use 16-35M bars (8 per layer)

$A_{s,actual} = 16{,}000\,mm^2$ (per meter)

$Spacing_{,actual} = \dfrac{1000 - (8)(35.7)}{7} = 102mm > 50mm \therefore OK$

Load Case 2:

Assume 1 layer of 25M bars

$d = 1500 - 75 - \left(\frac{25.2}{2}\right) = 1410\,mm$

$M_F = 357\,kN\cdot m$

$K_r = \frac{M_F}{bd^2 \times 10^{-6}} = \frac{357}{(1000)(1410)^2 \times 10^{-6}} = 0.18$

Kr Value is so small, assume $A_{s,min}$ will govern.

let $b_t = 600\,mm$

$A_{s,min} = \frac{0.2\sqrt{f'_c}\,b_t\,h}{f_y} = \frac{(0.2)(\sqrt{35})(600)(1500)}{400}$

$= 2662\,mm^2$

Use 6-25M bars per meter ($A_{s,actual} = 3000\,mm^2$)

$Spacing = \frac{1000 - (6)(25.2)}{5} = 170\,mm > 1.4d_b\ (36\,mm)\ \therefore\ ok$

Shear Reinforcement (using Load Case 1)

$d_v = 0.9d = (0.9)(1360) = 1224\,mm$ ⇐ Governs
or $= 0.72h = (0.72)(1500) = 1080\,mm$

$\beta = \frac{230}{1000 + d_v} = 0.103$

$\lambda = 1$ (normal weight Concrete)

$V_c = \phi_c \lambda \beta \sqrt{f'_c}\,b_w d_v = (0.65)(1)(0.103)(\sqrt{35})(1000)(1224) = 485\,kN$

$V_r = V_c + V_s \quad \therefore\ V_s = V_r - V_c = 3500 - 485 = \underline{3015\,kN}$

$V_s = 3015\,kN$

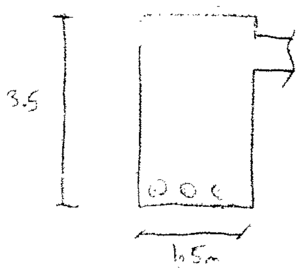
Find spacing; Assume 20M ties, ∴ $A_v = 600\,mm^2$

$\cot\theta = 1.43$

$$S = \frac{\phi_s A_v f_y d_v \cot\theta}{V_s} = \frac{(0.85)(600)(400)(1224)(1.43)}{3015000}$$

$S = 118.4 \Rightarrow S = 115\,mm$

Since some areas of Cope Wall will be unsupport underneath. Thus design for flexure in this case.



Self weight
$(1.5)(3.5)(2400) = 12.6\ kN/m$

largest span 10.0m

$$M_{f_1} = \frac{w l^2}{8} = \frac{(12.6)(10)^2}{8} = 158\,kN\cdot m$$

account if a Fender is attached on this gap:

$$\frac{(6618 + 8667)(9.81)}{1000} = 150\,kN \quad , \quad M_{f_2} = \frac{(150)(10)}{2} = 750\,kN\cdot m$$

$M_F = 910.0\,kN\cdot m$

$d = 3500 - 75 - \frac{35.7}{2} = 3407.2\,mm$

$$K_r = \frac{M_r}{b d^2 \times 10^{-6}} = \frac{(910)}{(1500)(3407)^2 \times 10^{-6}} = 0.052$$ Very small, use $A_{s,min}$

PARAMOUNT	DESIGN CALCULATIONS SHEET	DATE:	Apr. 4/2010
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PROJECT:	St. Lawrence Marine Terminal	DOCUMENT NO.:	DCI - 8700-07-013
PROJECT NO.:	8700-07	REVISION:	0
ITEM:	Cope Wall	PREPARED BY:	Peter Collins

$$A_{s,min} = \frac{0.2\sqrt{f_c'}\,b_t h}{f_y} = \frac{(0.2)(\sqrt{35})(1500)(3000)}{400} = 13311.2\ mm^2$$

Use 14-35M bars $\therefore A_{s\,actual} = 14000\ mm^2$

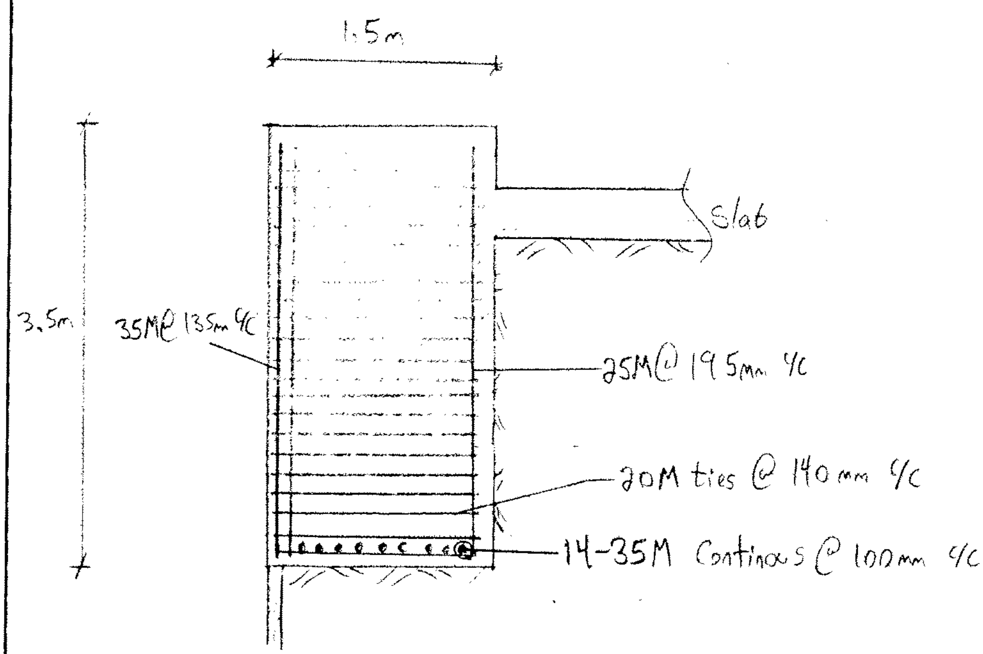
$$Spacing = \frac{1500-(2)(75)-(14)(35.7)}{13} = 65mm$$

$$S_{req.} = 1.4d_b = (1.4)(35.7) = 50mm$$

$S_{actual} > S_{req}$ $\therefore$ OK

From Table 3.1 & 3.8, using $f_c' = 35$ MPa, all development lengths will not be of issues.

Reinforcement Detail



APPENDIX P: FENDER DESIGN AND SELECTION

PARAMOUNT	DESIGN CALCULATIONS SHEET	DATE:	Apr 2/2010
		PAGE:	1 of 5

PROJECT:	St. Lawrence Marine Terminal	DOCUMENT NO.:	DC7-8700-07-005
PROJECT NO.:	8700-07	REVISION:	1
ITEM:	Fender Design + Selection	PREPARED BY:	Steven Greeley

» Berthing Energy, E_N = 302 ton·m (see DC7-8700-07-

↳ abnormal energy, $E_A = F_S E_N = 1.25(302 \text{ ton·m}) = 377.5$ ton·m

↳ F_S for large dry bulk carriers recommended by Pianc (2002)

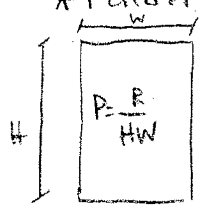
$$E_A = 377.5 \text{ ton·m} \times \frac{2000\text{lbs}}{\text{ton}} \times \frac{1\text{kg}}{2.2\text{lbs}} \times \frac{9.81\text{N}}{1\text{kg}} \times \frac{1\text{kN}}{1000\text{N}} = 3367 \text{ kN·m}$$

* need to select fender for E_A = 3367 kN·m
* want to use (1) fender to eliminate concrete req'ments

Try Super Cone fenders - most optimal type

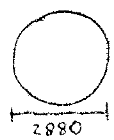
↳ Options: SCN 1800 E 2.9 or higher
or SCN 2000 E 1.5 or higher

* Fender Panel - hull pressure for dry bulk carrier < 200 kN/m²



$$E/R = 0.932 \frac{\text{kNm}}{\text{kN}} \Rightarrow R = \frac{3367 \text{ kNm}}{0.932 \frac{\text{kNm}}{\text{kN}}} = 3613 \text{ kN}$$

↳ SCN 1800 E 2.9 → R = 3613 kN ∴ $HW = \frac{R}{P} = \frac{3613\text{kN}}{200 \text{ kN/m}^2} = 18.06 \text{ m}^2$



∴ Try 4.5 × 4.5 fender → Area = 20.25 m²

∴ P = 3613/20.25 = 178 kN/m²

» assume heavy duty panels ~ 350 kg/m²
one faced exposed ∴ tst = 10mm

$$\text{Panel Weight} = \left[350 + \frac{10\text{mm}}{1000\frac{\text{mm}}{\text{m}}}(7850 \text{ kg/m}^3)\right] \times 20.25\text{m}^2 = 8677 \text{ kg}$$

PARAMOUNT	DESIGN CALCULATIONS SHEET	DATE:	Apr 2/2018
		PAGE:	2 of 5

PROJECT:	St. Lawrence Marine Terminal	DOCUMENT NO.:	DC1-8700-07-605
PROJECT NO.:	870007	REVISION:	1
ITEM:	Fender Design & Selection	PREPARED BY:	Steven Greeley

* Can fender support static weight of panel?

$W_H \le n \times 1.3 \times W$

$8677\,kg \le (1) \times 1.3 \times 6618\,kg$

$8677\,kg \nleq 8603\,kg$

↳ Try E3.0

$W_H \le n \times 1.5 \times W$

$\le 1 \times 1.5 \times 6618\,kg$

$8677\,kg \le 9927\,kg$ ∴ ok → Select SCN1800H E3.0 fender

* # Fenders

$P \le 2\sqrt{R_B^2 - (R_B - h + C)^2}$

Where; $R_B \approx \frac{1}{2}\left[\frac{B}{2} + \frac{L_{OA}^2}{8B}\right] = \frac{1}{2}\left[\frac{50m}{2} + \frac{(310m)^2}{8(50m)}\right] = 132.65m$

h = fender projection when compressed

$= H - \delta_f$ ← rated deflection

$= 1800mm - (0.72)(1800mm)$

$= 504mm$

C = clearance between vessel & dock

$= 0.15H \ge 300mm$

$= 0.15(1800mm) = 270mm \ge 300mm$

$C = 300mm$

∴ $P \le 2\sqrt{(132.65m)^2 - (132.65m - 0.504m + 0.3m)^2}$

$P \le 14.7m$

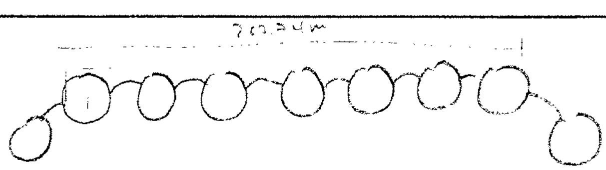
» BS6349: Part 4: 1994 recommends a fender spacing $< 0.15 L_s$

↑ length of smallest vessel → (see sheet 1)

$P \le 0.15(144m) \le 21.6m$

∴ use $P \le 14.7m$

» Actual spacing ⇒ Select (21) Fenders @ 13.387m



spacings

267.74/20 = 13387m

PARAMOUNT	**DESIGN CALCULATIONS SHEET**		DATE:	Apr 2/2010
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PROJECT:	St. Lawrence Marine Terminal	DOCUMENT NO.:	DC 7-8700-07-005
PROJECT NO.:	8700-07	REVISION:	1
ITEM:	Fender Designs Selection	PREPARED BY:	Steven Greeley

≫ Smallest vessel : 15,000 DWT (general cargo)

↳ From Agerschou (mean statistical analysis)

→ For general cargo vessels (10,000-15,000 DWT)

4 @ 105-109 m = 428 m
6 @ 110-114 m = 672 m
11 @ 115-119 m = 1287 m
23 @ 120-124 m = 2806 m
76 @ 125-129 m = 9652 m
61 @ 130-134 m = 8052 m
71 @ 135-139 m = 9727 m
93 @ 140-144 m = 13206 m
92 @ 145-149 m = 13524 m
143 @ 150-154 m = 21736 m
73 @ 155-159 m = 11461 m
73 @ 160-164 m = 11826 m
5 @ 165-169 m = 835 m
1 @ 170-174 m = 172 m
1 @ 175-179 m = 177 m

733 take ½ way value for each range 105561 m

$L_{avg} = \frac{105561\,m}{733}$

$L_{avg} = 144\,m$

PARAMOUNT	DESIGN CALCULATIONS SHEET	DATE:	Apr 2/2010
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ITEM:	Fender Design + Selection	PREPARED BY:	Steven Greeley

Chain Design

$h_1 \cong L \sin\phi_1$ $\quad \phi_1 = 15\text{-}25° \rightarrow$ select $20°$

$L \cong \frac{h_1}{\sin\phi_1} = \frac{1800mm}{\sin 20°} = 5263mm$

$D \approx 0.72h_r = 0.72(1800mm) = 1296mm$

rated deflection

$h_2 \cong h_1 - D = 1800 - 1296mm = 504mm$

$\phi_2 = \sin^{-1}\left[\frac{h_1 - D}{L}\right] = \sin^{-1}\left[\frac{1800mm - 1296mm}{5263mm}\right] = 5.5°$

F_L (safe working Load) [N]

$F_L = \frac{\mu(\Sigma R) + W}{9.81 n \cos\phi_2}$ $\quad$ assume $n = 2$ for each chain design

$\mu = 0.2$ (friction coefficient for UHMW-PE panels)

① Tension Chain: $F_L = \frac{(0.2)(3{,}613{,}000N) + 8677 kg(9.81N/kg)}{9.81(2)\cos(5.5°)}$

$F_L = 41{,}359 N$

$F_M = F_S F_L = 2.5(41.4kN) = 104kN$

② Weight Chain: $F_L = \frac{8677kg(9.81N/kg)}{9.81(2)\cos 5.5°} = 4358N$

$F_M = 2.5(4.4kN) = 11kN$

③ Shear Chain: $F_L = \frac{(0.2)(3{,}613{,}000N)}{9.81(2)\cos 5.5} = 37{,}000N$

$F_M = 2.5(37kN) = 92.5kN$

PARAMOUNT	DESIGN CALCULATIONS SHEET	DATE:	Apr 2/2010
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PROJECT:	St. Lawrence Marine Terminal	DOCUMENT NO.:	DC1-8700-07-005
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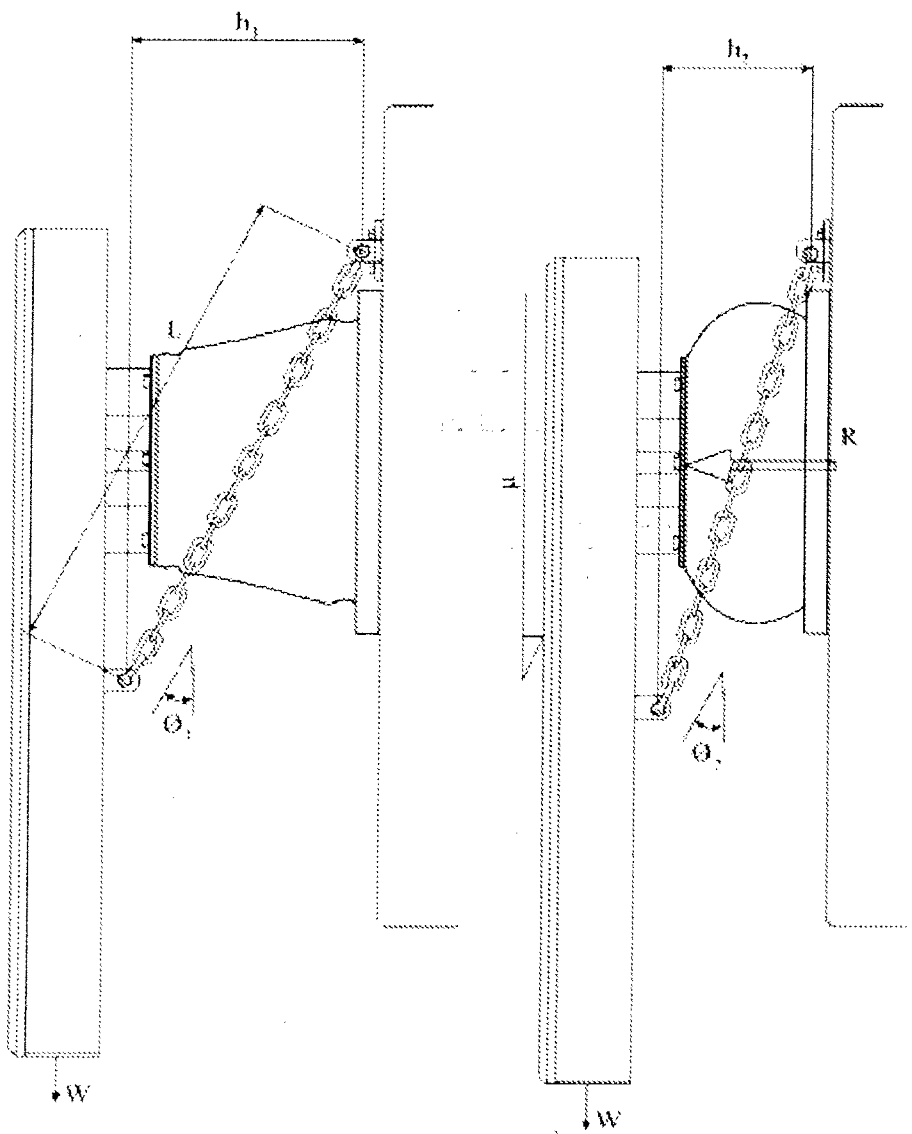
ITEM:	Fender Design + Selection	PREPARED BY:	Steven Greeley



$$h_1 \cong L \cdot Sin(\phi_1)$$

$$\phi_2 \cong arcSin\left[\frac{h_1 - D}{L}\right]$$

$$h_2 \cong h_1 - D$$

$$F_L \cong \frac{\mu \cdot \left(\sum R\right) + W}{9.81 \cdot n \cdot Cos(\phi_2)}$$

$$F_M \cong F_S \cdot F_L$$

SUPER CONE FENDERS

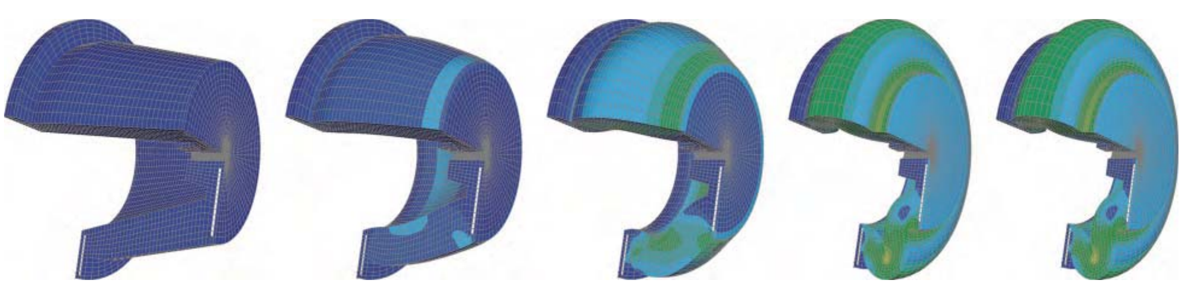
Super Cones are the latest generation of 'cell' fender, with optimal performance and efficiency. The conical body shape makes the SCN very stable even at large compression angles, and provides excellent shear strength. With overload stops the Super Cone is even more resistant to over-compression.

Features

- Highly efficient geometry
- No performance loss even at large berthing angles
- Stable shape resists shear
- Wide choice of rubber compounds

Applications

- General cargo berths
- Bulk terminals
- Oil and LNG facilities
- Container berths
- RoRo and cruise terminals
- Parallel motion systems
- Monopiles and dolphins

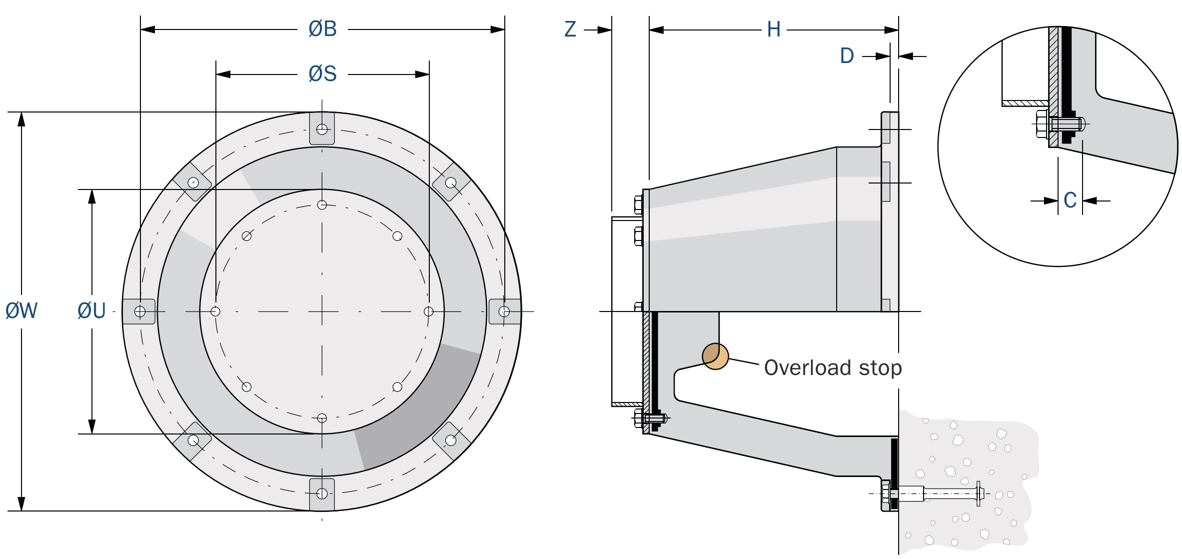


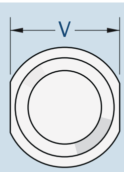
TRELLEBORG

SUPER CONE FENDERS

	H	ØW	V	ØU	C	D	ØB	ØS	Anchors/ Head bolts	Z_{min}	Weight
SCN 300	300	500	–	295	27–37	20–25	440	255	4 × M20	45	40
SCN 350	350	570	–	330	27–37	20–25	510	275	4 × M20	52	50
SCN 400	400	650	–	390	30–40	20–28	585	340	4 × M24	60	76
SCN 500	500	800	–	490	32–42	30–38	730	425	4 × M24	75	160
SCN 550	550	880	–	540	32–42	30–38	790	470	4 × M24	82	210
SCN 600	600	960	–	590	40–52	35–42	875	515	4 × M30	90	270
SCN 700	700	1120	–	685	40–52	35–42	1020	600	4 × M30	105	411
SCN 800	800	1280	–	785	40–52	35–42	1165	685	6 × M30	120	606
SCN 900	900	1440	–	885	40–52	35–42	1313	770	6 × M30	135	841
SCN 950	950	1520	1440	930	40–52	40–50	1390	815	6 × M30	142	980
SCN 1000	1000	1600	–	980	50–65	40–50	1460	855	6 × M36	150	1125
SCN 1050	1050	1680	–	1030	50–65	45–55	1530	900	6 × M36	157	1360
SCN 1100	1100	1760	–	1080	50–65	50–58	1605	940	8 × M36	165	1567
SCN 1200	1200	1920	–	1175	57–80	50–58	1750	1025	8 × M42	180	2028
SCN 1300	1300	2080	–	1275	65–90	50–58	1900	1100	8 × M48	195	2455
SCN 1400	1400	2240	2180	1370	65–90	60–70	2040	1195	8 × M48	210	3105
SCN 1600	1600	2560	2390	1570	65–90	70–80	2335	1365	8 × M48	240	4645
SCN 1800	1800	2880	2700	1765	75–100	70–80	2625	1540	10 × M56	270	6618
SCN 2000	2000	3200	–	1955	80–105	90–105	2920	1710	10 × M56	300	9560

[Units: mm, kg]





Some SCN sizes have a modified flange for reduced shipping dimensions.

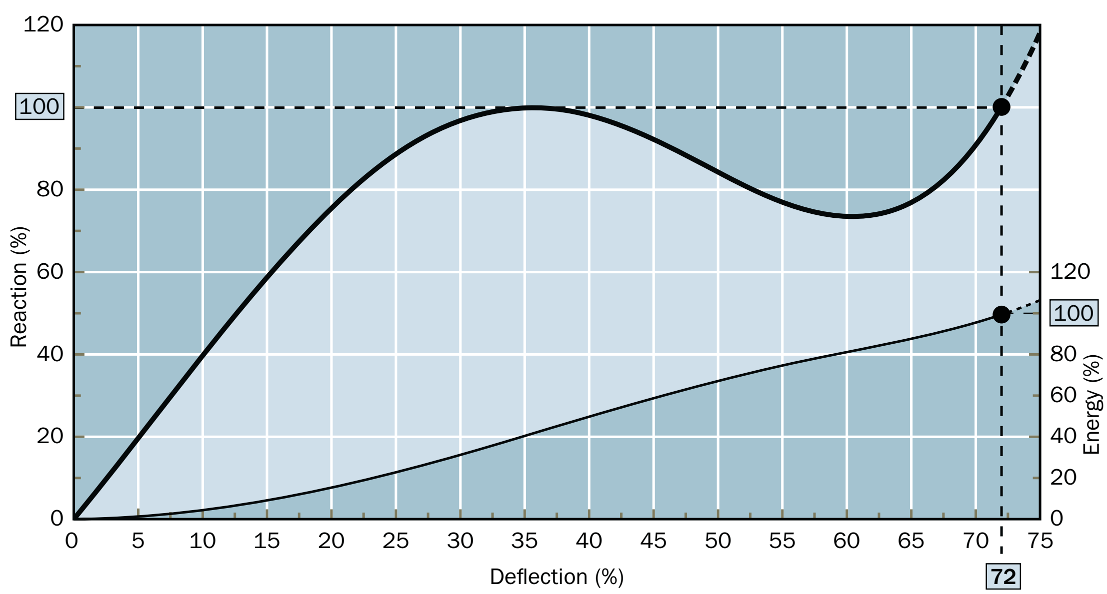
TRELLEBORG

SUPER CONE FENDERS

Rated Performance Data (RPD)*

		E0.9	E1.0	E1.1	E1.2	E1.3	E1.4	E1.5	E1.6	E1.7	E1.8	E1.9	E2.0
SCN 300	E_R	7.7	8.6	8.9	9.2	9.5	9.8	10.1	10.4	10.6	10.9	11.2	11.5
	R_R	59	65	67	68	70	72	74	75	77	79	80	82
SCN 350	E_R	12.5	13.9	14.4	14.8	15.3	15.7	16.2	16.7	17.1	17.6	18	18 5
	R_R	80	89	91	93	96	98	100	102	104	107	109	111
SCN 400	E_R	18.6	20.7	21.4	22.1	22.8	23.5	24.2	24.8	25.5	26.2	26.9	27.6
	R_R	104	116	119	122	125	128	131	133	136	139	142	145
SCN 500	E_R	36.5	40.5	41.9	43.2	44.6	45.9	47.3	48.6	50	51.3	52.7	54
	R_R	164	182	187	191	196	200	205	209	214	218	223	227
SCN 550	E_R	49	54	56	58	59	61	63	65	67	68	70	72
	R_R	198	220	226	231	237	242	248	253	259	264	270	275
SCN 600	E_R	63	70	72	74	76	78	80	82	84	86	88	90
	R_R	225	250	257	263	270	276	283	289	296	302	309	315
SCN 700	E_R	117	130	134	137	141	144	148	151	155	158	162	165
	R_R	320	355	365	374	384	393	403	412	422	431	441	450
SCN 800	E_R	171	190	196	201	207	212	218	223	229	234	240	245
	R_R	419	465	478	490	503	515	528	540	553	565	578	590
SCN 900	E_R	248	275	282	289	296	303	310	317	324	331	338	345
	R_R	527	585	601	617	633	649	665	681	697	713	729	745
SCN 950	E_R	291	322	331	339	348	356	364	373	381	390	398	407
	R_R	588	653	671	688	706	724	742	759	777	795	813	830
SCN 1000	E_R	338	375	385	395	405	415	425	435	445	455	465	475
	R_R	653	725	745	764	784	803	823	842	862	881	901	920
SCN 1050	E_R	392	435	447	458	470	481	493	504	516	527	539	550
	R_R	720	800	822	843	865	886	908	929	951	972	994	1015
SCN 1100	E_R	450	500	514	527	541	554	568	581	595	608	622	635
	R_R	788	875	899	923	947	971	995	1019	1043	1067	1091	1115
SCN 1200	E_R	585	650	668	685	703	720	738	755	773	790	808	825
	R_R	941	1045	1073	1101	1129	1157	1185	1213	1241	1269	1297	1325
SCN 1300	E_R	743	825	847	869	891	913	935	957	979	1001	1023	1045
	R_R	1103	1225	1258	1291	1324	1357	1390	1423	1456	1489	1522	1555
SCN 1400	E_R	927	1030	1058	1085	1113	1140	1168	1195	1223	1250	1278	1305
	R_R	1278	1420	1459	1497	1536	1574	1613	1651	1690	1728	1767	1805
SCN 1600	E_R	1382	1535	1577	1618	1660	1701	1743	1784	1826	1867	1909	1950
	R_R	1670	1855	1905	1955	2005	2055	2105	2155	2205	2255	2305	2355
SCN 1800	E_R	1967	2185	2244	2303	2362	2421	2480	2539	2598	2657	2716	2775
	R_R	2115	2350	2413	2476	2539	2602	2665	2728	2791	2854	2917	2980
SCN 2000	E_R	2700	3000	3080	3160	3240	3320	3400	3480	3560	3640	3720	3800
	R_R	2610	2900	2978	3056	3134	3212	3290	3368	3446	3524	3602	3680

*in accordance with PIANC. [Units: kNm, kN]



Standard manufacturing and performance tolerances apply (see pages 12–36 to 12–39)

SUPER CONE FENDERS

Rated Performance Data (RPD)*

		E2.1	E2.2	E2.3	E2.4	E2.5	E2.6	E2.7	E2.8	E2.9	E3.0	E3.1	E/R (ε)
SCN 300	E_R	11.8	12.1	12.4	12.7	13.0	13.3	13.5	13.8	14.1	14.4	15.9	0.138
	R_R	84	86	89	91	93	95	97	100	102	104	114	
SCN 350	E_R	19	19.4	19.9	20.3	20.8	21.3	21.7	22.2	22.6	23.1	25.4	0.163
	R_R	114	117	120	123	126	129	132	135	138	141	155	
SCN 400	E_R	28.3	29	29.7	30.4	31 1	31.8	32.5	33.2	33.9	34.6	38.1	0.186
	R_R	149	153	157	161	165	169	173	177	181	185	204	
SCN 500	E_R	55.4	56.7	58.1	59.4	60.8	62.2	63.5	64.9	66.2	67.6	74.4	0.232
	R_R	233	239	246	252	258	264	270	277	283	289	318	
SCN 550	E_R	74	76	77	79	81	83	85	86	88	90	99	0.256
	R_R	283	290	298	305	313	320	328	335	343	350	385	
SCN 600	E_R	93	96	99	102	105	108	111	114	117	120	132	0.290
	R_R	324	332	341	349	358	366	375	383	392	400	440	
SCN 700	E_R	169	173	177	181	185	189	193	197	201	205	226	0.364
	R_R	462	474	486	498	510	522	534	546	558	570	627	
SCN 800	E_R	252	258	265	271	278	284	291	297	304	310	341	0.414
	R_R	606	621	637	652	668	683	699	714	730	745	820	
SCN 900	E_R	355	364	374	383	393	402	412	421	431	440	484	0.466
	R_R	765	785	805	825	845	865	885	905	925	945	1040	
SCN 950	E_R	418	429	440	451	463	473	485	496	507	518	570	0.492
	R_R	853	875	897	919	941	963	986	1008	1030	1052	1158	
SCN 1000	E_R	488	501	514	527	540	553	566	579	592	605	666	0.518
	R_R	945	969	994	1018	1043	1067	1092	1116	1141	1165	1282	
SCN 1050	E_R	565	580	595	610	625	640	655	670	685	700	770	0.544
	R_R	1042	1069	1096	1123	1150	1177	1204	1231	1258	1285	1414	
SCN 1100	E_R	652	669	686	703	720	737	754	771	788	805	886	0.571
	R_R	1145	1174	1204	1233	1263	1292	1322	1351	1381	1410	1551	
SCN 1200	E_R	847	869	891	913	935	957	979	1001	1023	1045	1150	0.622
	R_R	1361	1396	1432	1467	1503	1538	1574	1609	1645	1680	1848	
SCN 1300	E_R	1074	1102	1131	1159	1188	1216	1245	1273	1302	1330	1463	0.674
	R_R	1597	1638	1680	1721	1763	1804	1846	1887	1929	1970	2167	
SCN 1400	E_R	1341	1376	1412	1447	1483	1518	1554	1589	1625	1660	1826	0.725
	R_R	1853	1901	1949	1997	2045	2093	2141	2189	2237	2285	2514	
SCN 1600	E_R	2003	2056	2109	2162	2215	2268	2321	2374	2427	2480	2728	0.830
	R_R	2418	2480	2543	2605	2668	2730	2793	2855	2918	2980	3278	
SCN 1800	E_R	2851	2926	3002	3077	3153	3228	3304	3379	3455	3530	3883	0.932
	R_R	3060	3139	3219	3298	3378	3457	3537	3616	3696	3775	4153	
SCN 2000	E_R	3904	4008	4112	4216	4320	4424	4528	4632	4736	4840	5324	1.039
	R_R	3778	3876	3974	4072	4170	4268	4366	4464	4562	4660	5126	

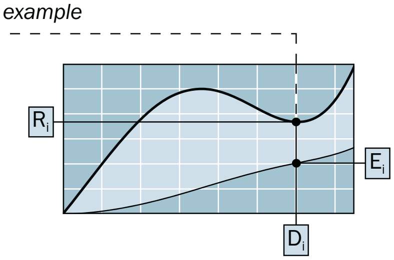
*in accordance with PIANC. [Units: kNm, kN]

Intermediate deflections

D_i(%)	0	5	10	15	20	25	30	35	40	45	50	55	60	65	70	**72**	75
E_i(%)	0	1	4	8	15	22	31	40	50	59	67	75	82	89	96	**100**	106
R_i(%)	0	19	39	59	75	89	97	100	98	92	84	77	73	77	91	**100**	118

Nominal rated deflection may vary at RPD. Refer to p12–35.

example



PIANC factors (from 3rd party witnessed Type Approval testing)

Angle factor

Angle (°)	AF
0	**1.000**
3	1.039
5	1.055
8	1.029
10	1.000
15	0.856
20	0.739

Temperature factor

Temperature (°C)	TF
50	0.882
40	0.926
30	0.969
23	**1.000**
10	1.056
0	1.099
-10	1.143
-20	1.186
-30	1.230

Velocity factor

Time (seconds)	VF
1	1.050
2	1.020
3	1.012
4	1.005
5	1.000
6	1.000
8	1.000
≥10	1.000

For steady state deceleration, the compression time is:

$$t \text{ (seconds)} = \frac{2d}{V_i}$$

d = fender deflection (mm)

V_i = impact speed (mm/s)

If compression time t<4s, please ask.
Refer to page 1–2 for further information.

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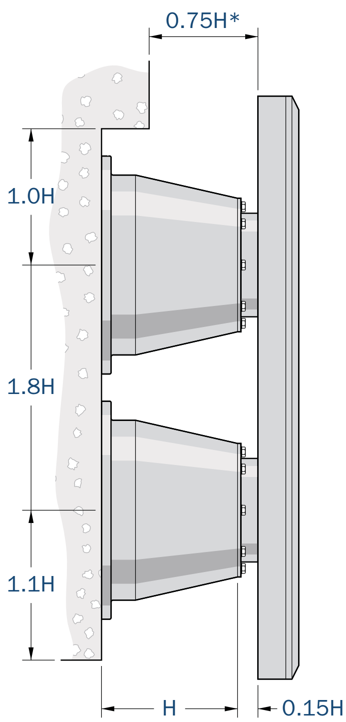
SUPER CONE FENDERS

Clearances

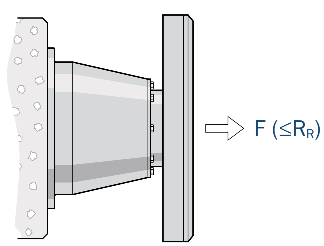


** does not allow for bow flares*

There must be enough space around and between Super Cone fenders and the steel panel to allow them to deflect without interference.

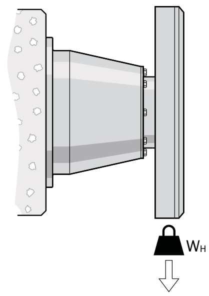
Distances given in the above diagram are for guidance. If in doubt, please ask.

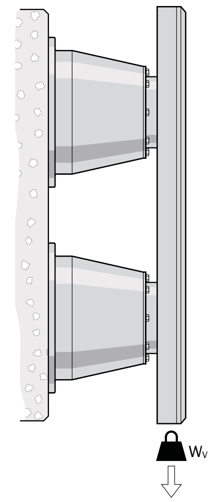
Tension



If the tensile load exceeds the rated reaction then tension chains may be required. Please ask for advice on the design of tension chains.

Weight support





Super Cone fenders can support a lot of static weight. The table is a guide to the permitted weight of front panel before additional support chains may be required.

SCN	Panel weight (kg)	
	Single or multiple horizontal ($n \geq 1$)	Multiple vertical ($n \geq 2$)
E1	$W_H \leq n \times 1.0 \times W$	$W_V \leq n \times 1.25 \times W$
E2	$W_H \leq n \times 1.3 \times W$	$W_V \leq n \times 1.625 \times W$
E3	$W_H \leq n \times 1.5 \times W$	$W_V \leq n \times 1.875 \times W$

n = number of Super Cones. W = Super Cone weight

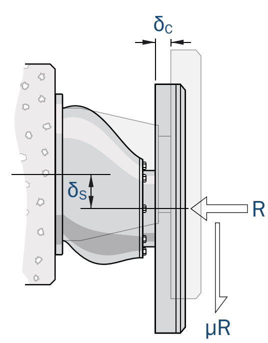
W_H = panel weight – single or multi-horizontal

W_V = panel weight – single or multi-vertical

Interpolate for other grades.

Refer to TMS when Super Cone direction is reversed.

Shear



Super Cones are very stable in shear. The table is a guide to maximum shear deflections (δ_S) for different shear coefficients (μ) and rubber grades.

δ_S	Friction coefficients (μ)			
	0.15	0.2	0.25	0.3
E1	7%	9%	11%	14%
E2	9%	11%	14%	17%
E3	11%	17%	18%	22%

δ_S (max) usually occurs at δ_C=0.3H to 0.35H.

For $\delta_S \geq 20\%$, refer to TMS.

SUPER CONE FENDERS

Proven in practice

TRELLEBORG

APPENDIX Q: DETAIL DESIGN: CORROSION PROTECTION

PARAMOUNT	DESIGN CALCULATIONS SHEET	DATE:	April 4/10
		PAGE:	1 of 2

PROJECT:	St. Lawrence Marine Terminal	DOCUMENT NO.:	DCI-8700-07-014
PROJECT NO.:	8700-07	REVISION:	
ITEM:	Corrosion Protection	PREPARED BY:	Robert Hunt

Steel Area Exposed

Surface area in soil zone

$450 m \times 2.5 m = 1125 m^2 = 12,109.4 Ft^2$

Surface area in submerged zone

$450 m \times 19.5 m = 8775 m^2 = 94,453.3 ft^2$

Surface area in tidal zone

$450 m \times 2 m = 900 m^2 = 9687.5 ft^2$

Current Requirement Calculations:

- current densities from Table 8.2 of Handbook of Corrosion Protection:

- Submerged zone: 32 mA/sq ft to achieve polarization
  8 mA/sq ft to maintain protection.

Soil zone: 5 mA/sq ft to achieve polarization.
  1.3 mA/sq ft to maintain protection.

Current required to polarize tidal zone:

$I_{pol} = 9,687.5 ft^2 \times 32 mA/sq ft \times \frac{1A}{1000 mA}$
$= 310 A$

Submerged zone:

current required to polarize: $I_{pol} = 94,453.3 ft^2 \times 32 mA/sq ft$
$= 3020 A$

current required to maintain: $I_{pol} = 94,453.3 \times 8 mA/ft^2$
$= 756 A$

PROJECT:	St. Lawrence Marine Terminal	DOCUMENT NO.:	DC1-8700-07-014
PROJECT NO.:	8700-07	REVISION:	
ITEM:	Corrosion Protection	PREPARED BY:	Robert Hunt

Soil zone:

To polarize: $I_{pol} = 12{,}109.4 \times 5\ mA/ft^2$
$= 61A$

To maintain: $I_{prot} = 12{,}109.4 \times 1.3\ mA/ft^2$
$= 16A$

Total Current Requirements:

$I_{T\text{-polarizing}} = 31\times10^4 + 30.2\times10^5 + 6.1\times10^4$
$= 33.9\times10^5$ amp X

$I_{T\text{-after polarization}} = 31\times10^4 + 75.6\times10^4 + 1.6\times10^4$
$= 1080$ A

$I_{T\text{-protection (15 yrs)}} = 1080$ A

Life Expectancy of the System:

- system will be designed for a 15-year life. The minimum number of anodes required:

$N = \cancel{C \times Y \times I} / W \times U \times E$

C = consumption rate of aluminum alloy = 6.8 lb/amp yr.
Y = Life expectancy of system = 15 yrs
I = Average total current required;

$\dfrac{I_{after\ polarization} + I_{protection\ (15\ yrs)}}{2} = 1080A$

W = weight of one anode (4in × 4in × 57in) = 90 lb
U = Utilization Factor = 0.85
E = Efficiency factor = 0.90

$N = \dfrac{6.8 \times 15 \times (1080)}{90 \times 0.85 \times 0.90} = 1600$ Anodes

APPENDIX R: DETAIL COST ESTIMATE

DETAILED COST ESTIMATE	Project: St. Lawrence Marine Terminal
SHEET PILE CELLS	Project #: 8700-07
	Document #: CE1-8700-07-0024
	Page #: 1 of 9

ITEM	QUANTITY	UNITS	UNIT RATE	TOTAL
CIVIL WORKS				
SITE WORK & SITE GRADING				
Mass rock excavation	0	M^3		$0.00
Drill & blast rock for placement of cells	178,000	ton	$18.00	$3,204,000.00
Place rock fill behind wharf structure	113,000	ton	$15.00	$1,695,000.00
ROCK SUB-MATTRESS / MATRESS				
Supply and place rock matress	65,000	ton	$20.00	$1,300,000.00
SLAB ON GRADE	2,200	M^3	$600.00	$1,320,000.00
SSP CELL BALLAST				
Supply & Placement of Rock Ballast in SSP Cells				
Rock Ballast - (100 minus)	526,000	ton	$15.00	$7,890,000.00
TOTAL				**$15,409,000**
STEEL SHEET PILE CELLS				
SUPPLY OF STEEL SHEET PILING	5,500	ton	$3,000.00	$16,500,000.00
NSTALLATION OF STEEL SHEET PILING				
Driving Template	1	LS	$250,000.00	$250,000.00
Cells	24,000	M^2	$150.00	$3,600,000.00
Arcs	11,000	M^2	$150.00	$1,650,000.00
TOTAL				**$22,000,000**
ICE STRENGTHENING PANELS				
PRE-FAB REINFORCED CONCRETE PANELS				
Reinforcement	156	ton	$890.00	$138,840.00
Concrete	1,150	M^3	$250.00	$287,500.00
Grout between cell and panel	1,550	M^2	$355.00	$550,250.00
TOTAL				**$976,590**
CRANE SUPPORT				
REINFORCED CONCRETE FOOTINGS				
Reinforcement	13	ton	$890.00	$11,570.00
Concrete	120	M^3	$250.00	$30,000.00
TOTAL				**$41,570**
SLAB ON GRADE				
CAST IN PLACE CONCRETE				
Reinforcement	165	ton	$890.00	$146,850.00

DETAILED COST ESTIMATE	Project: St. Lawrence Marine Terminal
SHEET PILE CELLS	Project #: 8700-07
	Document #: CE1-8700-07-0024
	Page #: 2 of 9

ITEM	QUANTITY	UNITS	UNIT RATE	TOTAL
Concrete	2,200	M^3	$250.00	$550,000.00
TOTAL				**$696,850**
COPE WALL				
REINFORCED CONCRETE				
Reinforcement	270	ton	$890.00	$240,300.00
Concrete	1,395	M^3	$250.00	$348,750.00
TOTAL				**$589,050**
CORROSION PROTECTION				
ALUMINUM ANODES				
Suppy Anodes	65,500	kg	$4.50	$294,750.00
Install Anodes	65,500	kg	$2.50	$163,750.00
TOTAL				**$458,500**
FENDERS				
BERTHING FENDERS				
Primary Berth	20	EA	$5,000.00	$100,000.00
TOTAL				**$100,000**
MOORING DEVICES				
WHARF BOLLARDS				
200t Wharf Bollards	8	EA	$1,200.00	$9,600.00
TOTAL				**$9,600**
GRAND TOTAL				**$40,281,160**

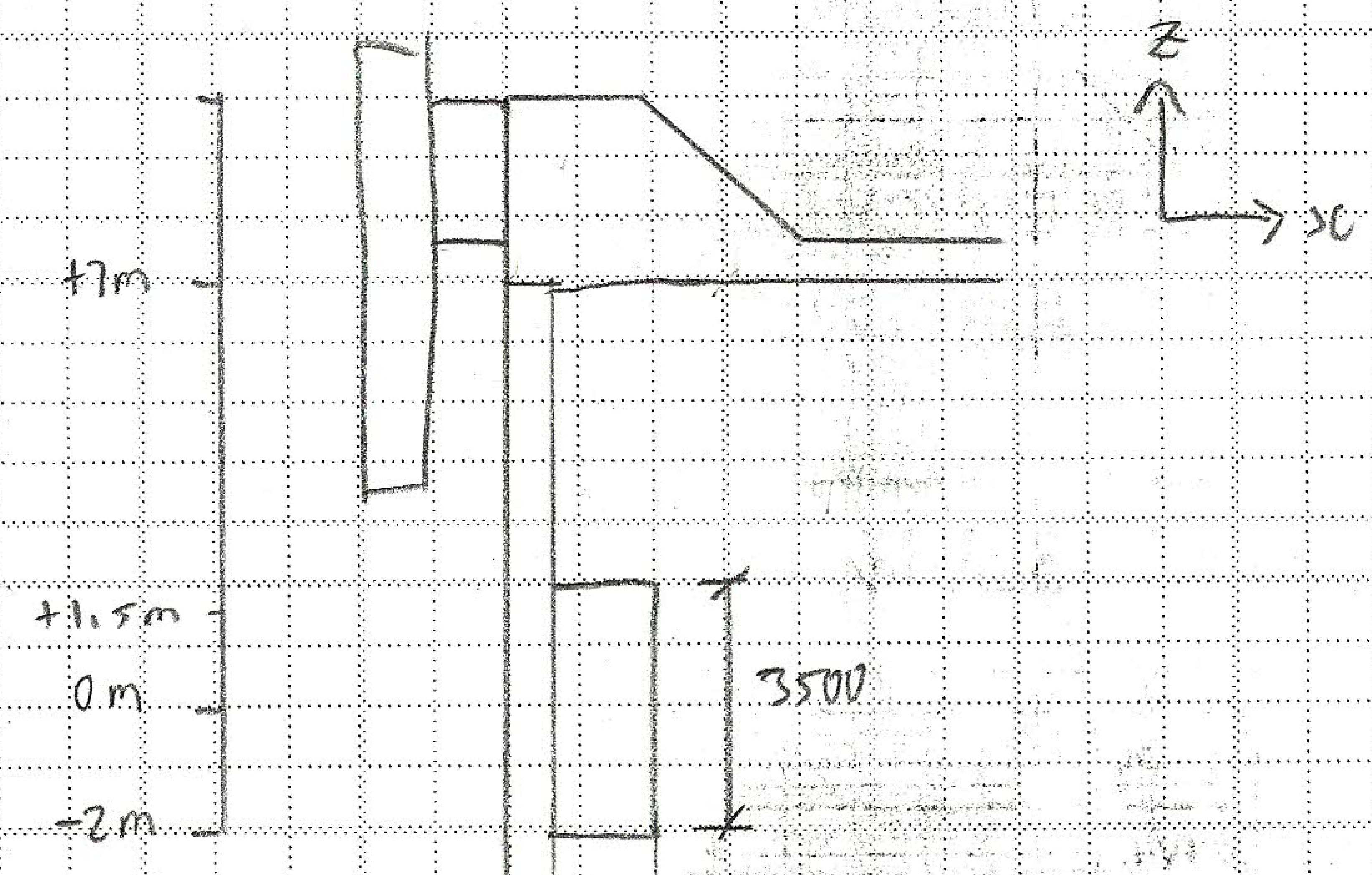
PARAMOUNT

PARAMOUNT	DESIGN CALCULATIONS SHEET	DATE:	MAR 26/2010
		PAGE:	3 of 9

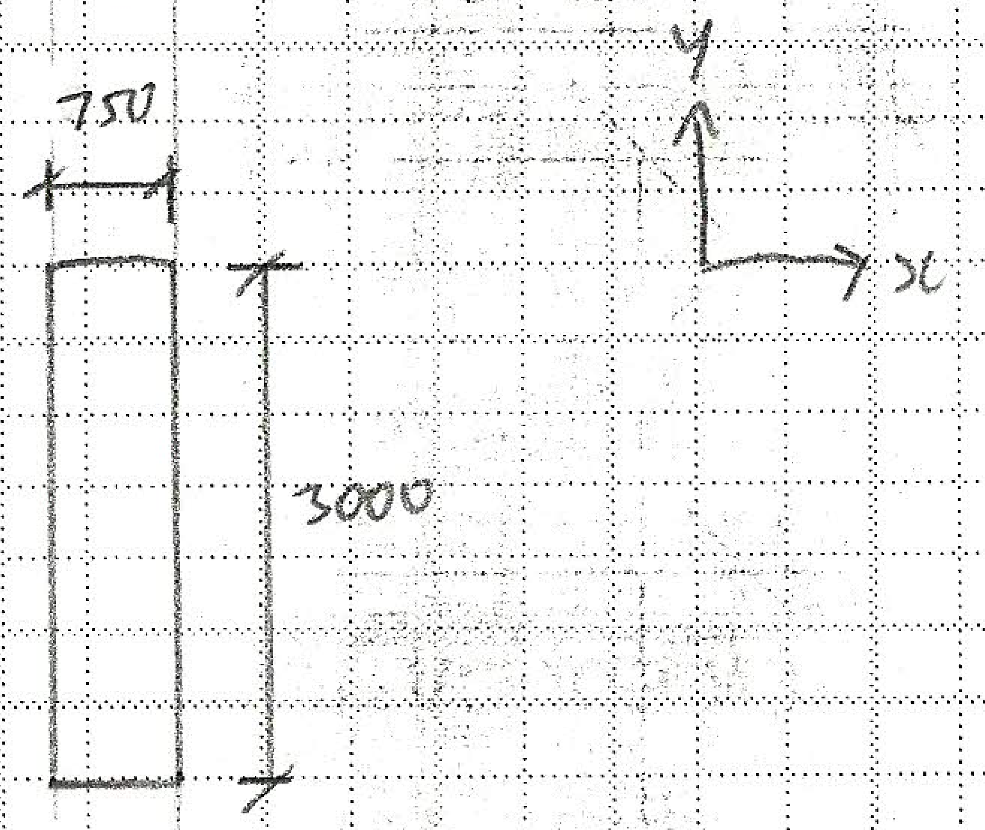
PROJECT:	ST. LAWRENCE MARINE TERMINAL	DOCUMENT NO.:	CEI-8700-07-004
PROJECT NO.:	8700-07	REVISION:	0
ITEM:	SHEET PILES DETAIL TAKE OFF	PREPARED BY:	ANDREW SMALL

ICE STRENGTHENING PANELS:

PANEL DIMENSIONS:



ELEVATION

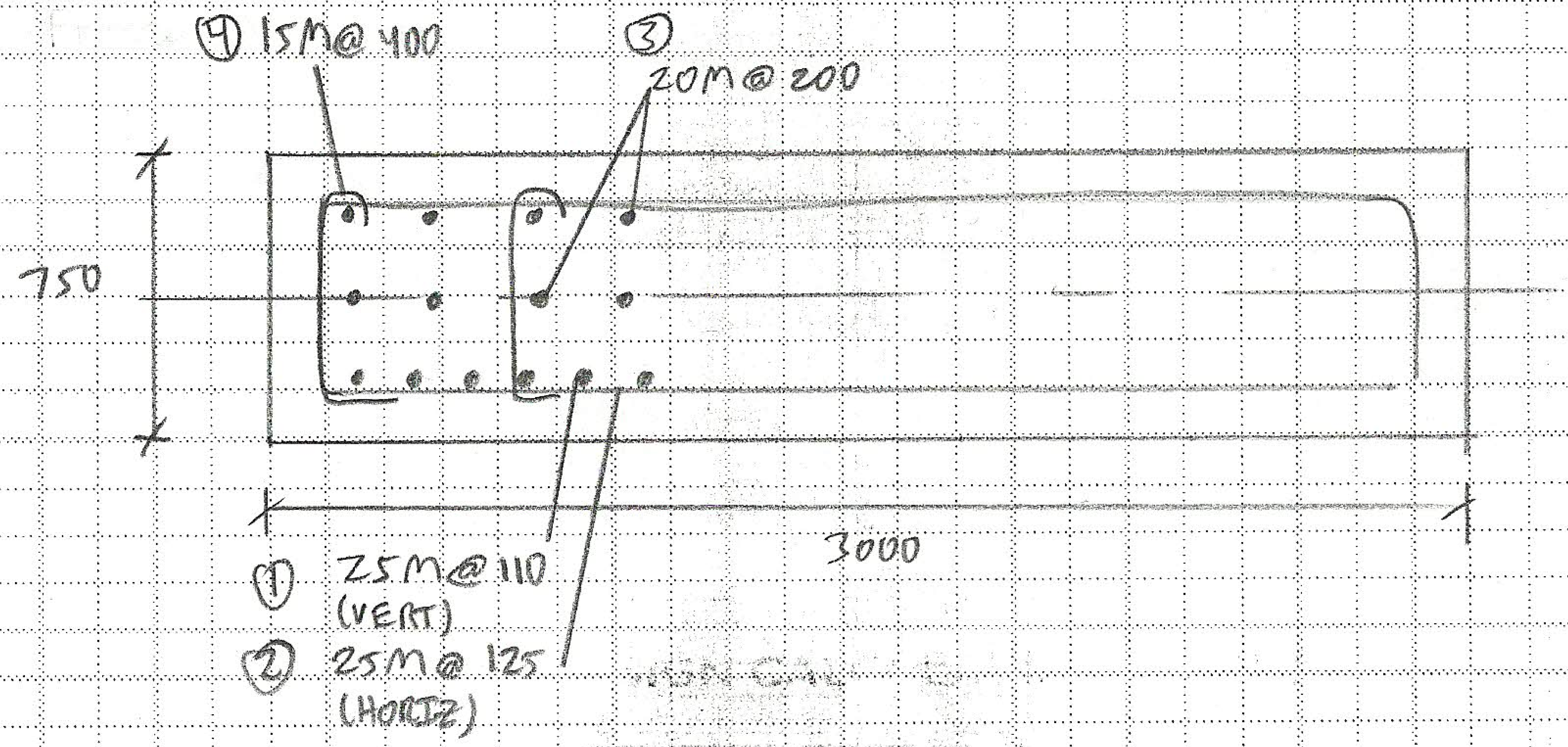


PLAN

$$V_{concrete} = (3500 \times 750 \times 3000) \times \left(\frac{1m}{1000mm}\right)^3 = 7.875\ m^3/\text{PANEL}$$

PARAMOUNT	DESIGN CALCULATIONS SHEET	DATE:	MAR 26/2011
		PAGE:	4 of 9

PROJECT:	ST. LAWRENCE MARINE TERMINAL	DOCUMENT NO.:	CE1-8700-07-004
PROJECT NO.:	8700-07	REVISION:	0
ITEM:	SHEET PILES DETAIL TAKEOFF	PREPARED BY:	ANDREW SMALL



① 25M@110 : $\frac{3000}{110}$ = 28 Bars x 3.5m = 98m

② 25M@125 : $\frac{3500}{125}$ = 28 Bars x 3.0m = 84m

$\}$ 182m

③ 20M@200 : $\frac{3000}{200}$ = 15 Bars x 3.5m x 2 = 104m

④ 15M@400 : $\frac{3000}{400}$ = 8 bars x 750 = 6m

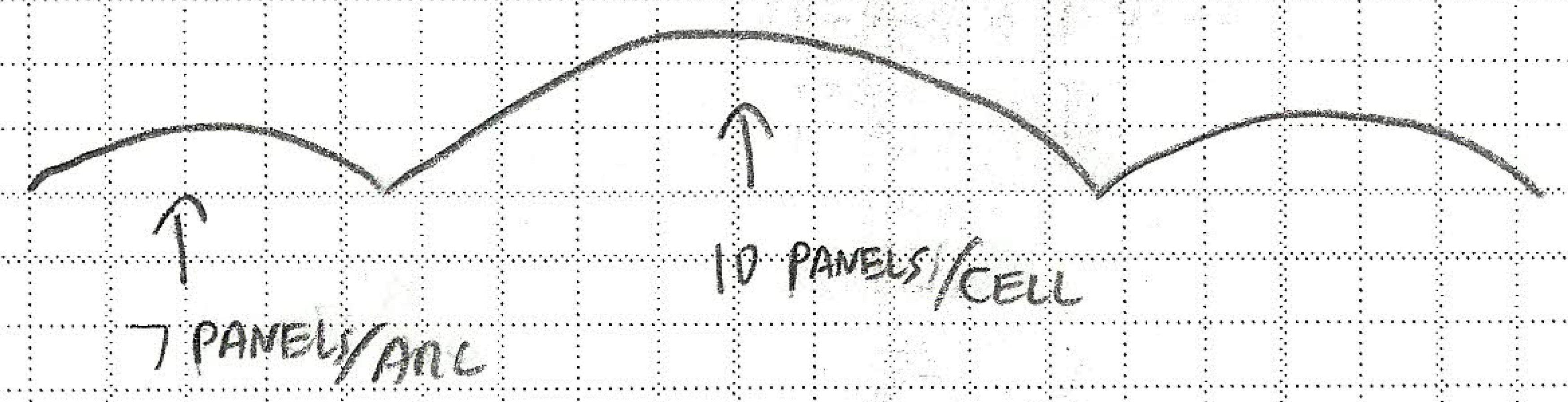
BAR	LENGTH (m)	LINEAR DENSITY (kg/m)	WEIGHT (kg)
25M	182	3.925	714.35
20M	104	2.355	244.92
15M	6	1.570	9.42

$968.69\,kg \times \frac{2.2\,lb}{kg} \times \frac{1\,ton}{2000\,lb}$

= 1.07 ton / PANEL

PARAMOUNT	DESIGN CALCULATIONS SHEET	DATE:	MAR 26/2010
		PAGE:	5 of 9

PROJECT:	ST. LAWRENCE MARINE TERMINAL	DOCUMENT NO.:	CEI-8700-07-004
PROJECT NO.:	8700-07	REVISION:	0
ITEM:	SHEET PILES - DETAIL TAKEOFF	PREPARED BY:	ANDREW SMALL



\# ARC = 8

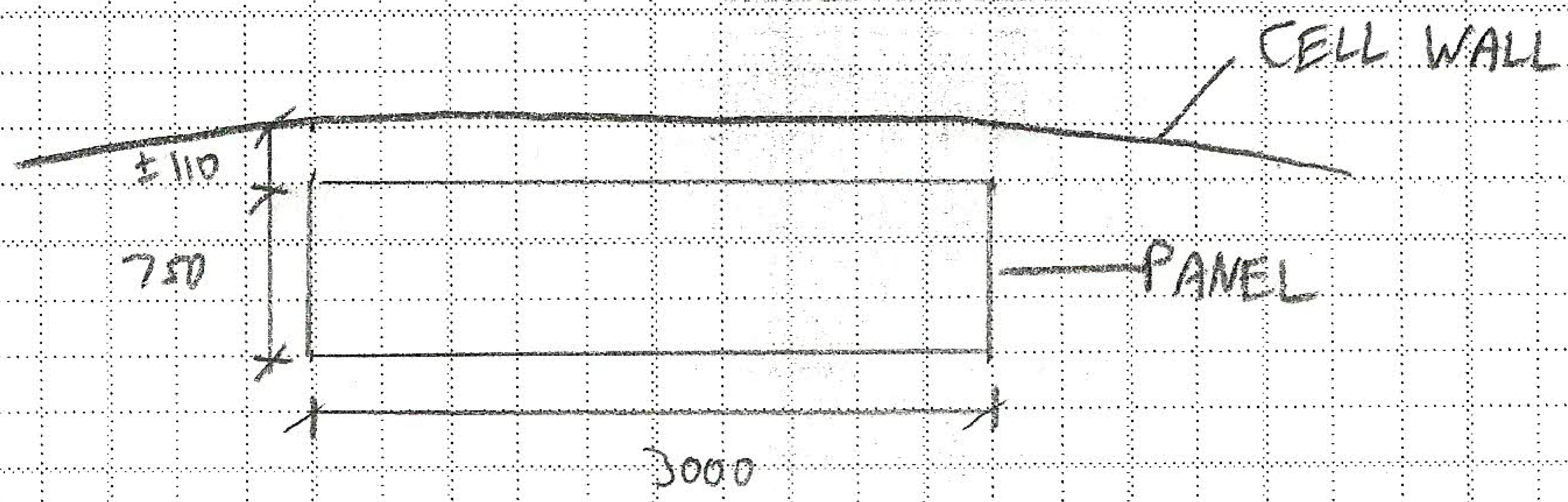
\# CELLS = 9

TOTAL # PANELS = $(7 \times 8) + (10 \times 9)$ = 146 PANELS

$V_{CONCRETE} = 7.875 \ m^3/\text{PANEL} \times 146 \text{ PANELS} = 1149.75 \sim 1150 \ m^3$

$W_{steel} = 1.07 \ \text{ton}/\text{PANEL} \times 146 \text{ PANELS} = 156 \ \text{ton}$

GROUT BETWEEN PANEL + CELL WALL:

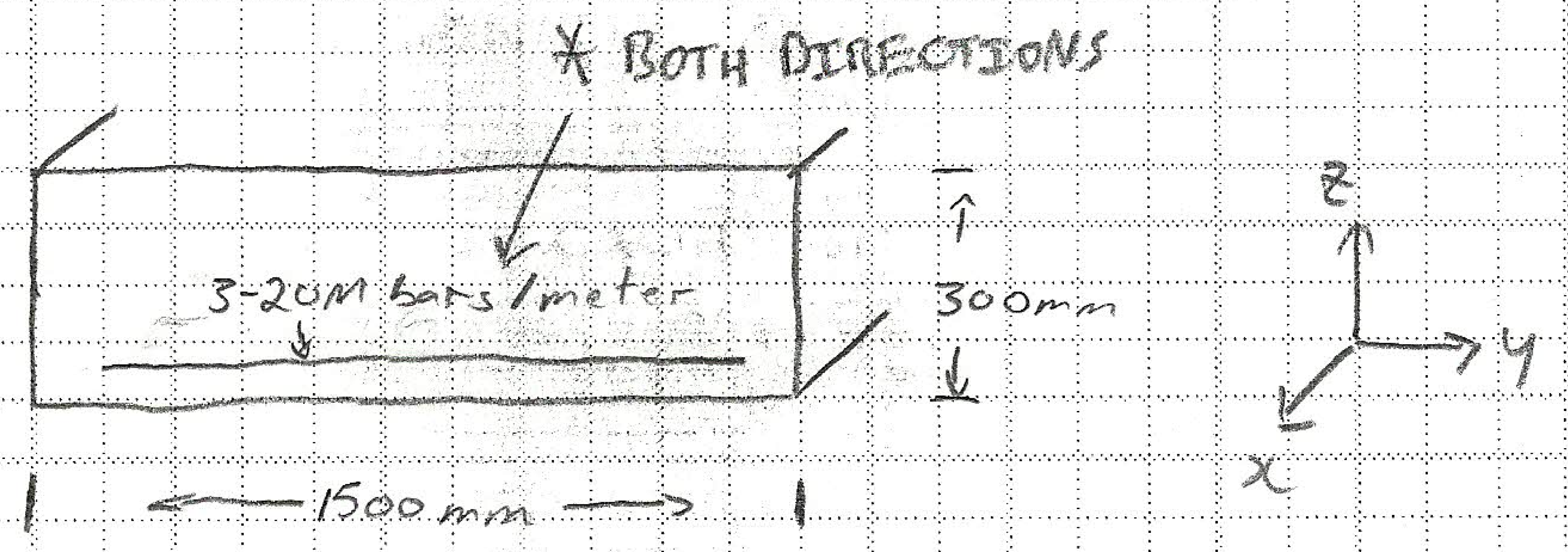


$\text{Vol Grout} = (3000 \times 3500) \times \left(\frac{1m}{1000mm}\right)^2 = 10.5 \ m^2/\text{Panel} \times 146 \text{ PANELS}$

$= 1533 \ m^2$

PARAMOUNT	DESIGN CALCULATIONS SHEET	DATE:	
		PAGE:	8 of 9

PROJECT:	ST. LAWRENCE MARINE TERMINAL	DOCUMENT NO.:	CEI-8700-07-004
PROJECT NO.:	8700-07	REVISION:	0
ITEM:	Crane Foundation TAKEOFF	PREPARED BY:	ROBERT HUNT



Volume of Concrete

$267.64m \times 1.5m \times 0.3m = 120.44\ m^3$

2 Beams

$2 \times 120.44 = 241\ m^3$ Total

Rebar (y-DIRN)

$3\ bars/m \times 265m \times 1.5m = 1193m \times 2 = 2385m$

CRANE RAILS

REBAR (x-DIRN)

$3\ BAR/m \times 1.5m = 5\ BARS \times 265m = 1325m \times 2 = 2650m$

CRANE RAILS

TOTAL BAR LENGTH

$= 5035\ m$

Weight (total)

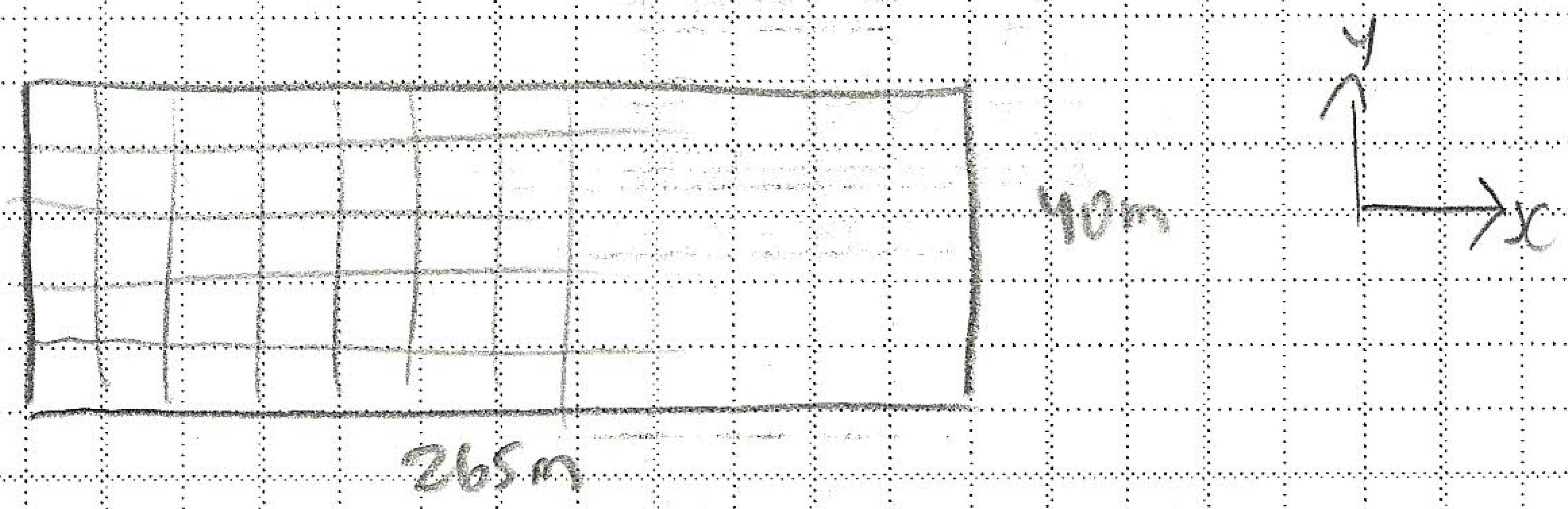
$\pm\ 5035\ m \times 2.355\ kg/m = 11857\ kg \times \frac{2.2lb}{1kg} \times \frac{1ton}{2000lb} = 13\ ton$

PARAMOUNT	DESIGN CALCULATIONS SHEET	DATE:	
		PAGE:	7 of 9

PROJECT:	ST. LAWRENCE MARINE TERMINAL	DOCUMENT NO.:	CEI-8700-07-004
PROJECT NO.:	8700-07	REVISION:	0
ITEM:	SLAB ON GRADE TAKE OFF	PREPARED BY:	ANDREW SMALL

$V_{CONCRETE} = 2200\ m^3$ // SEE DCI-8700-07-006, P3 of 4

REBAR



x-dim : 3-20M/m x 265m = 795 bars x 40m = 31800 m

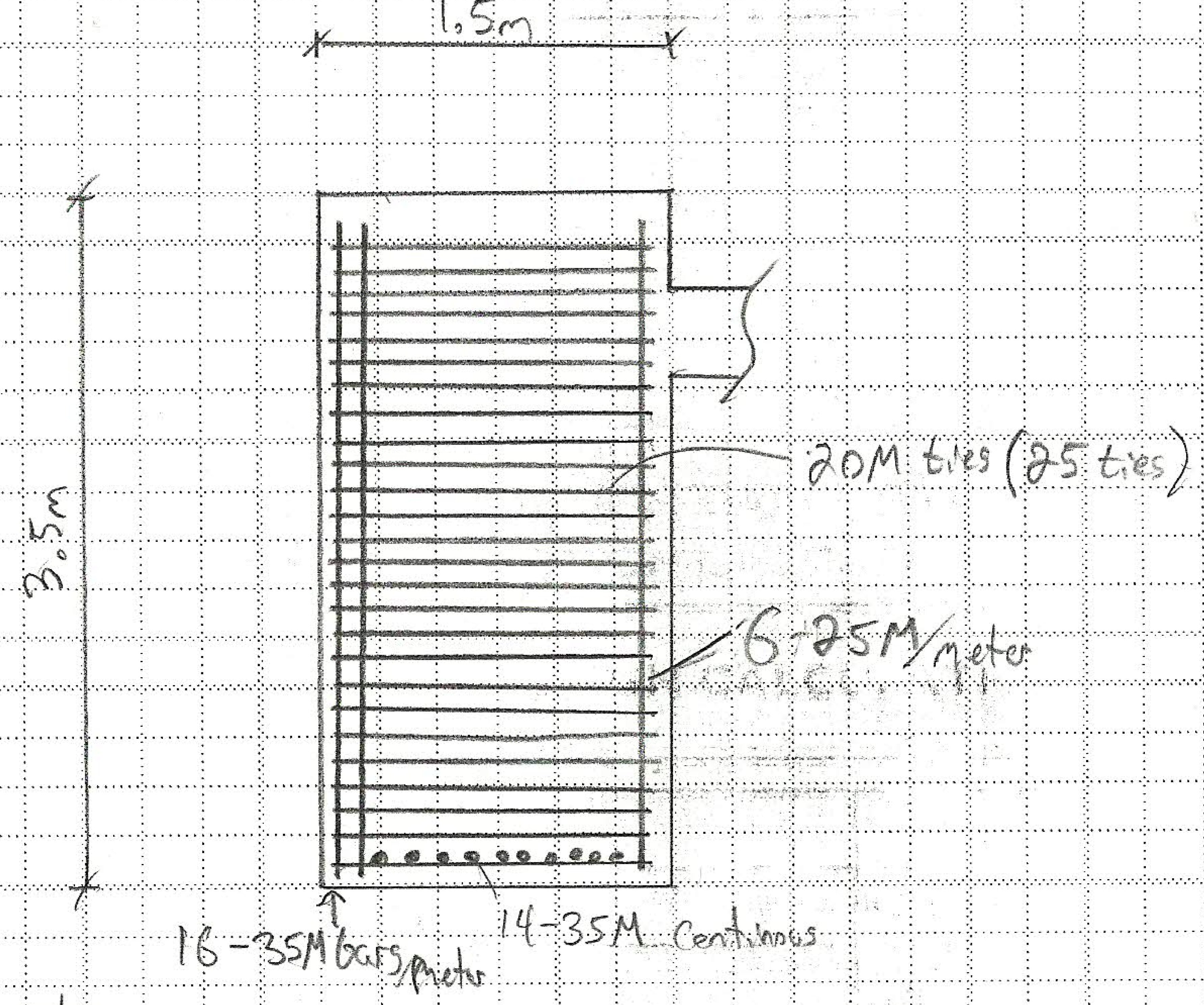
y-dim : 3-20M/m x 40m = 120 bars x 265m = 31800m

63600m

$Wt = 63600m \times 2.355\ kg/m = 149778\ kg \times \frac{2.2lb}{kg} \times \frac{1\ ton}{2000lb} = 165\ ton$

PARAMOUNT	DESIGN CALCULATIONS SHEET	DATE:	Apr. 4/2010
		PAGE:	8 of 9

PROJECT:	St. Lawrence Marine Terminal	DOCUMENT NO.:	CEI-8700-07-004
PROJECT NO.:	8700-07	REVISION:	0
ITEM:	COPE WALL TAKE-OFF	PREPARED BY:	Peter Collins



Concrete Vol. Concrete = (1.5)(3.5)(265) = 1391.3 m³

Steel

length of 35M: (3.5)(16)(265) = 14840m

(14)(265) = 3710 m

total: 18550m × 7.85kg/m = 145617.5kg

↳ 160.5 ton

length of 25M: (6)(3.5)(265) = 5565m

5565m × 3.925 kg/m = 21842.6 kg

↳ 24.1 ton

length of 20M ties: [(2)(1.5+1)] 265 = 33125m

33125m × 2.355kg/m = 78009.4 kg

↳ 86.0 ton

PARAMOUNT	DESIGN CALCULATIONS SHEET	DATE:	April 4/10
		PAGE:	9 of 9

PROJECT:	St. Lawrence Marine Terminal	DOCUMENT NO.:	CE1-8700-07-004
PROJECT NO.:	8700-07	REVISION:	0
ITEM:	Corrosion Protection	PREPARED BY:	Robert Hunt

Cost estimate:

1600 anodes weighing 90 lbs each.

= 144000 lb

// SEE DCI-8700-07-014 Page 2 of 2 FOR # ANODES

APPENDIX S: DESIGN DRAWINGS

Project

St. Lawrence Marine Terminal

April 5, 2010

Consultants

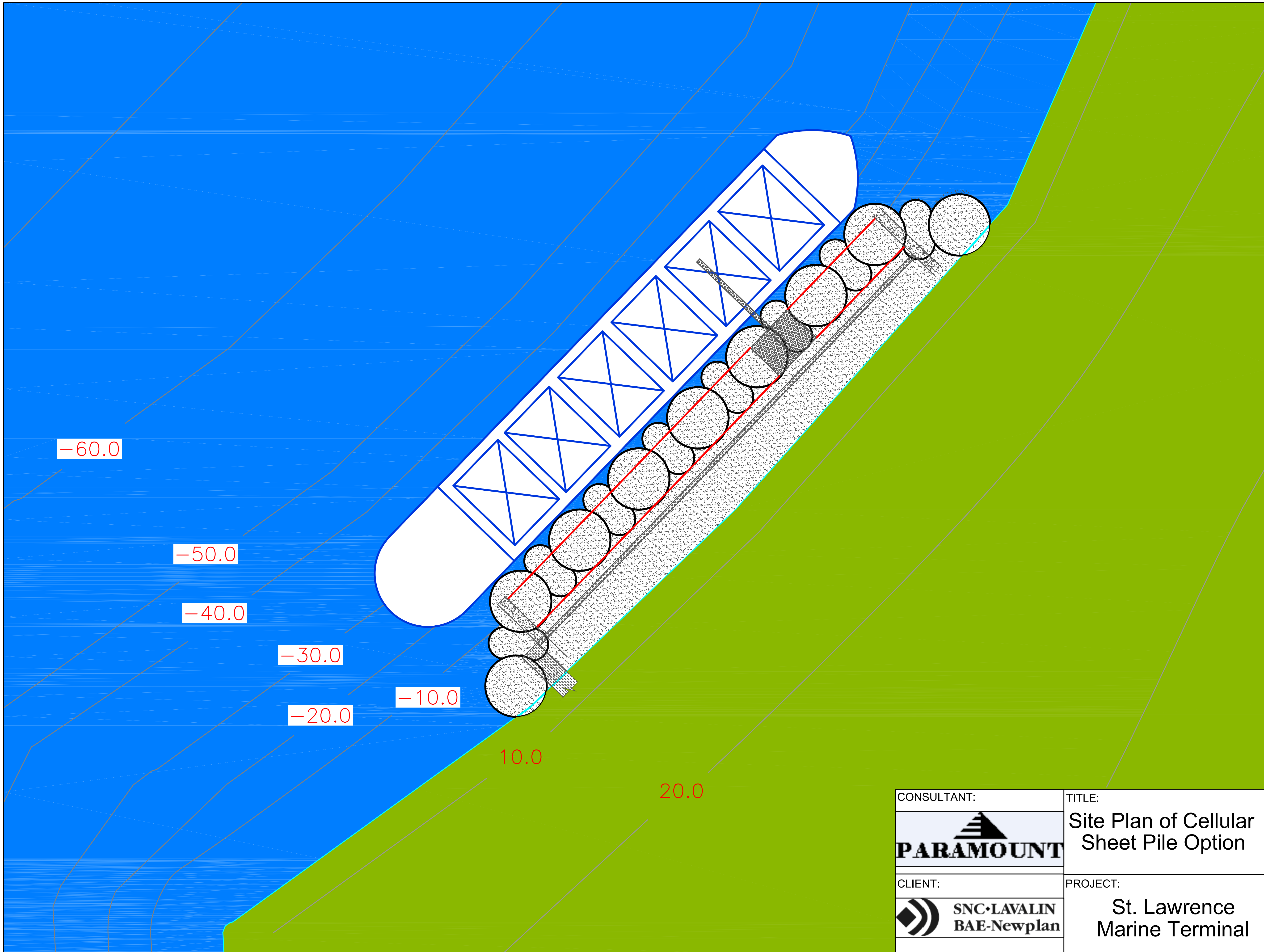
PARAMOUNT

Consultants

SNC·LAVALIN
BAE-Newplan

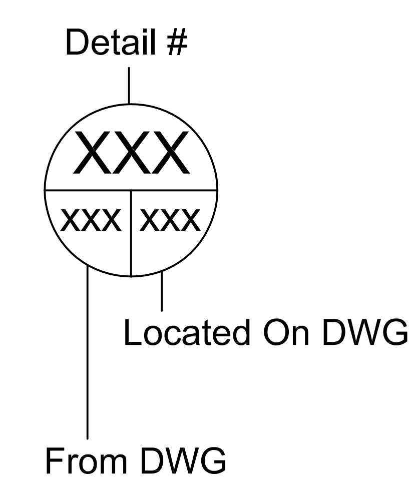
List of Drawings

PO1-01	Site Plan of Cellular Sheet Pile Option
PO1-02	Circular Sheet Pile Plan and Cross Section
PO1-03	Circular Sheet Pile Details
PO2-01	Site Plan of Concrete Caisson Option
PO2-02	Caisson Plan, Cross Section and Details
PO2-03	Concrete Caisson Section
PO3-01	Site Plan of Open Pile Option
PO3-02	Open Pile Plan and Elevation
PO3-03	Open Pile Section
001	Cellular Sheet Pile Detailed Section
002	Slab and Crane Reinforcement Details
003	Cope Wall Details
004	Ice Reinforcement Details
005	Heavy Duty Wheel Stop Detail
006	Crane Rail Details
007	Fender Plan and Elevations
008	Fender Details
009	Fender Panel Details
010	Tee Bollard Details

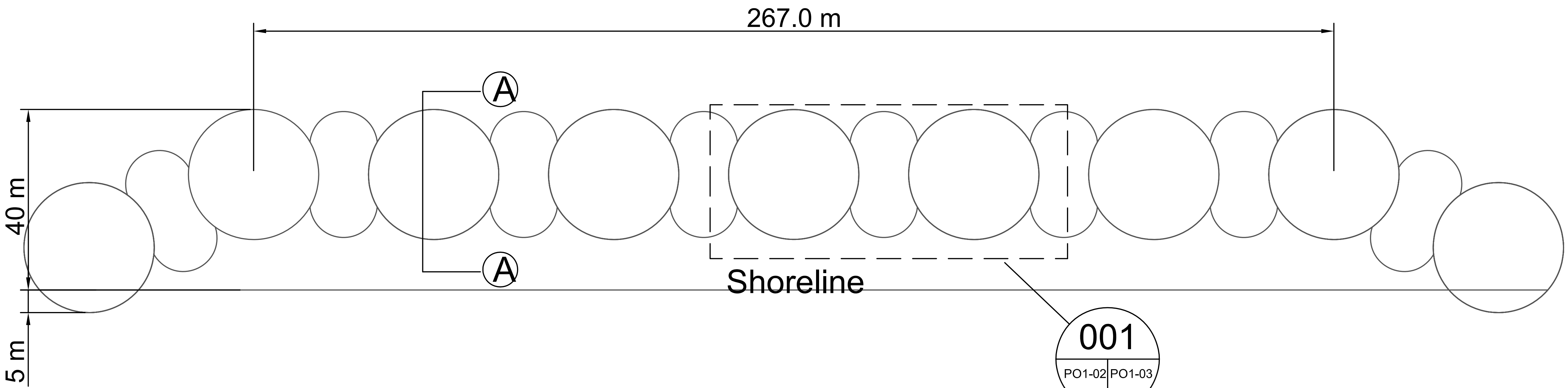


Notes:

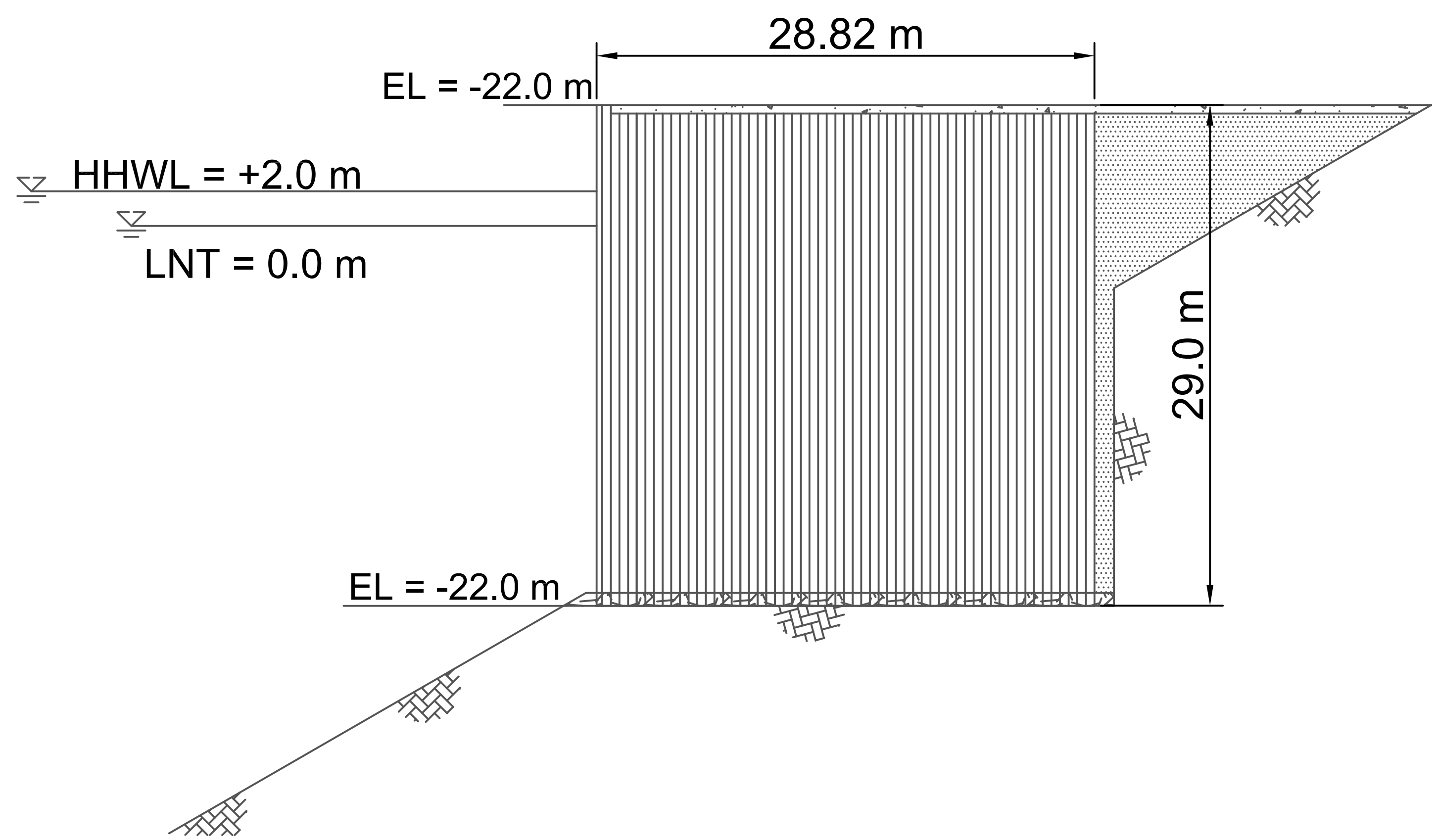
Legend:



CONSULTANT: PARAMOUNT	TITLE: Site Plan of Cellular Sheet Pile Option	DRAWING #: DW1-8700-07-PO1-01	DRAWN BY: PWC CHECKED BY: AGS
CLIENT: SNC·LAVALIN BAE-Newplan	PROJECT: St. Lawrence Marine Terminal	PROJECT #: 8700-07	APPROVED BY: SRG DATE: Mar. 15, 2010 SCALE: NTS



Plan View



Section A-A

Notes:

Legend:

Detail #

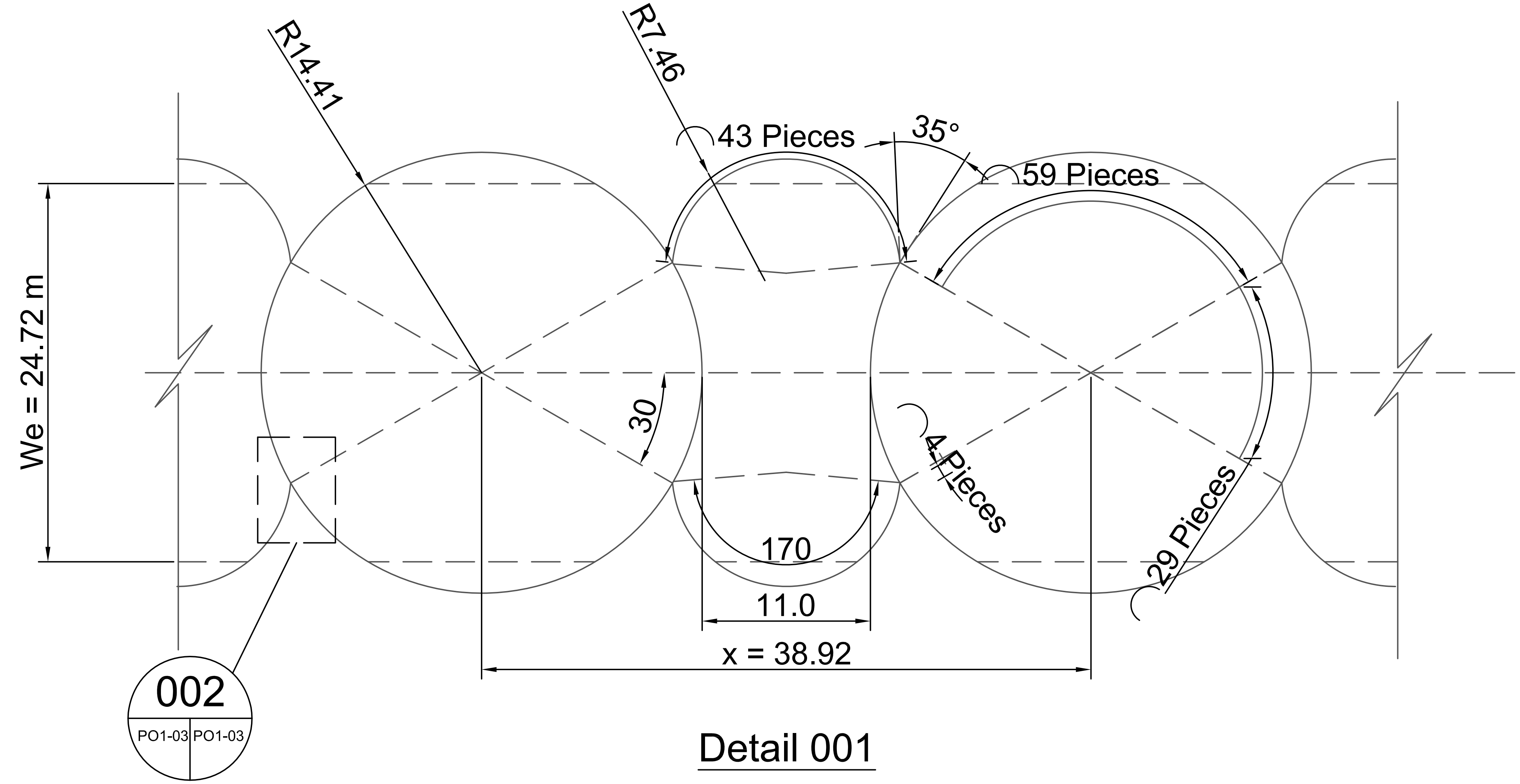
XXX

xxx xxx

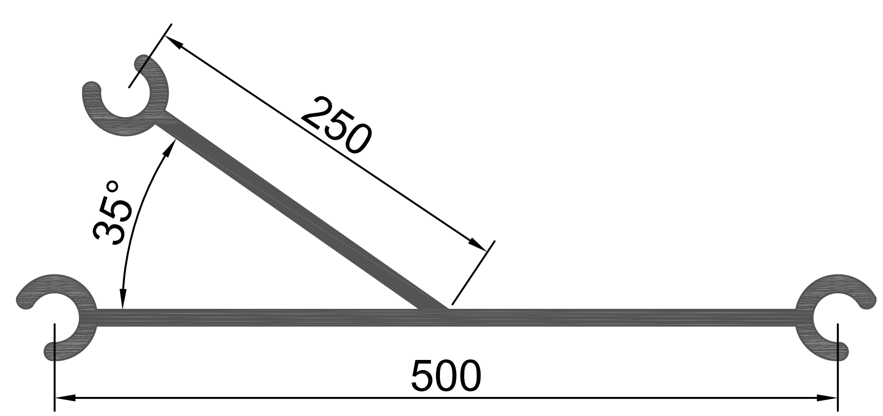
Located On DWG

From DWG

CONSULTANT: PARAMOUNT	TITLE: Circular Sheet Pile Plan & Cross Section	DRAWING #: DW1-8700-07-PO1-02	DRAWN BY: PWC CHECKED BY: AGS APPROVED BY: SRG
CLIENT: SNC•LAVALIN BAE-Newplan	PROJECT: St. Lawrence Marine Terminal	PROJECT #: 8700-07	DATE: Mar. 15, 2010 SCALE: NTS



Detail 001

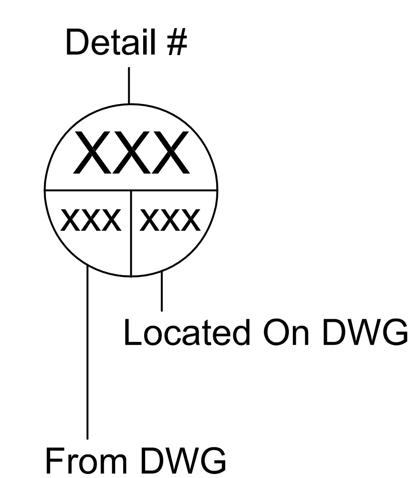


Detail 002

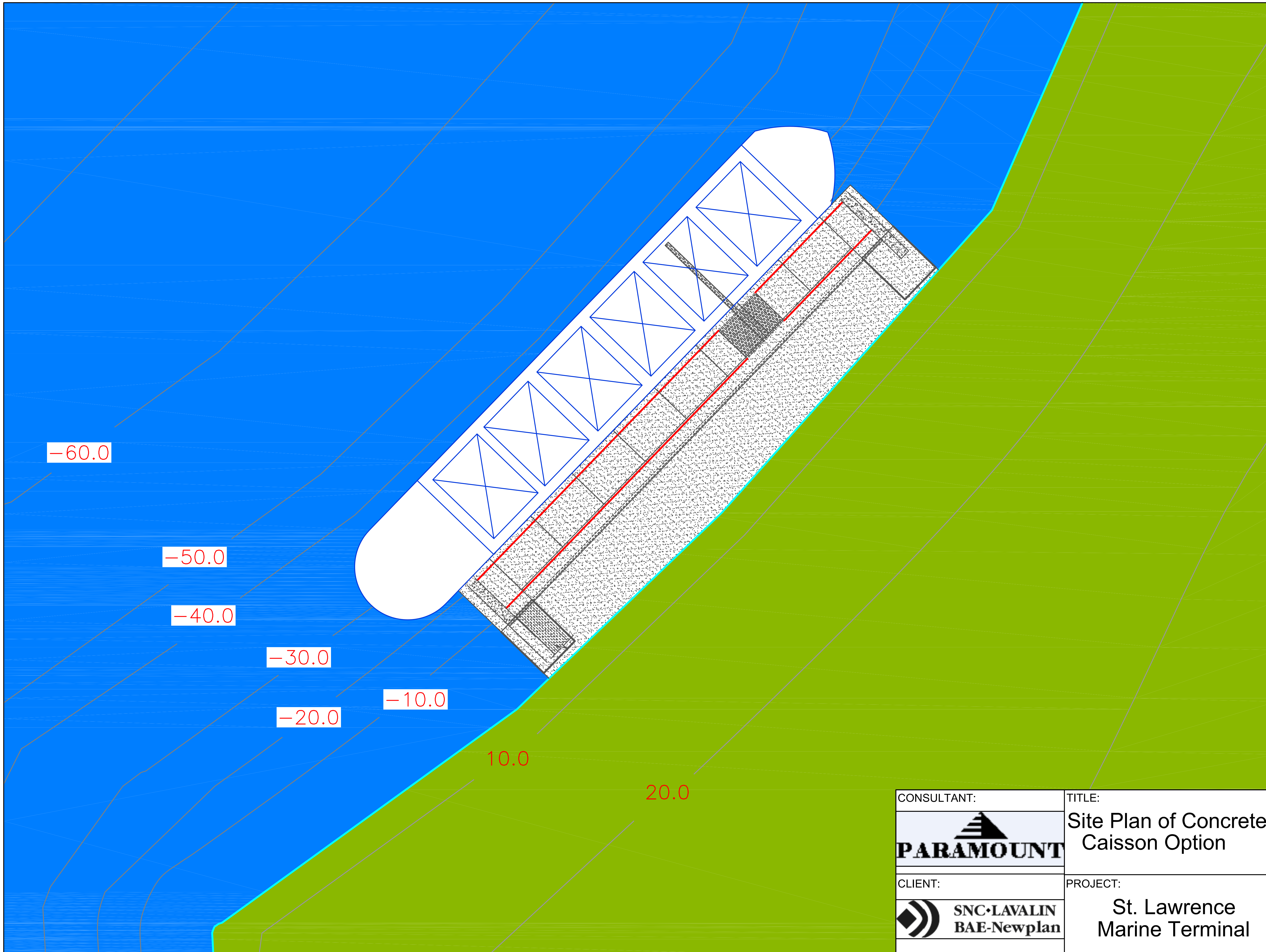
Notes:

1. Cells constructed from AS 500-12.0 sheet piles.

Legend:

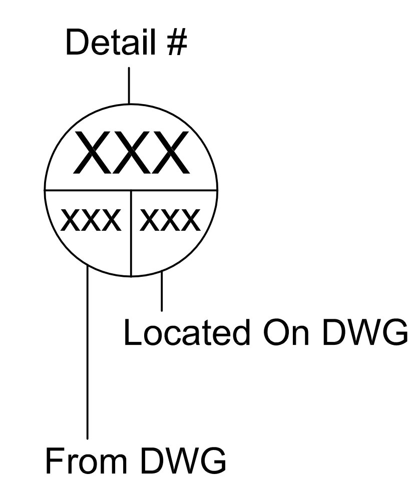


CONSULTANT:	TITLE:	DRAWING #:	DRAWN BY:
PARAMOUNT	Circular Sheet Pile Details	DW1-8700-07-PO1-03	PWC
			CHECKED BY: AGS
CLIENT:	PROJECT:	PROJECT #:	APPROVED BY: SRG
SNC•LAVALIN BAE-Newplan	St. Lawrence Marine Terminal	8700-07	DATE: Mar. 15, 2010
			SCALE: NTS

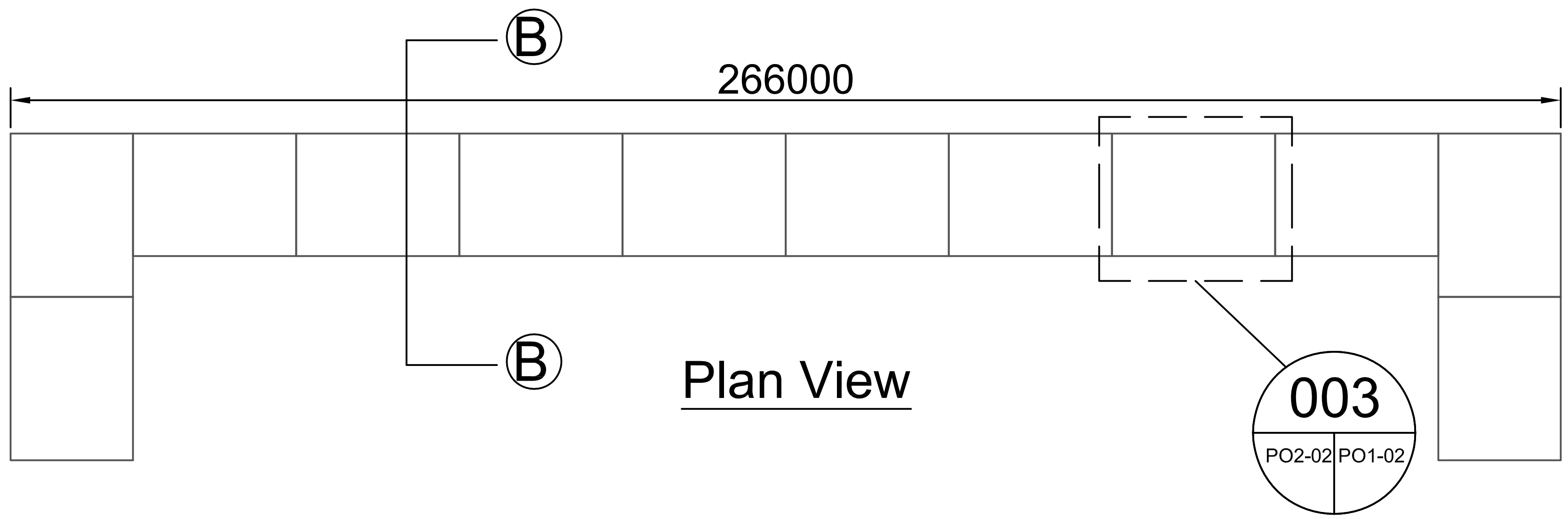


Notes:

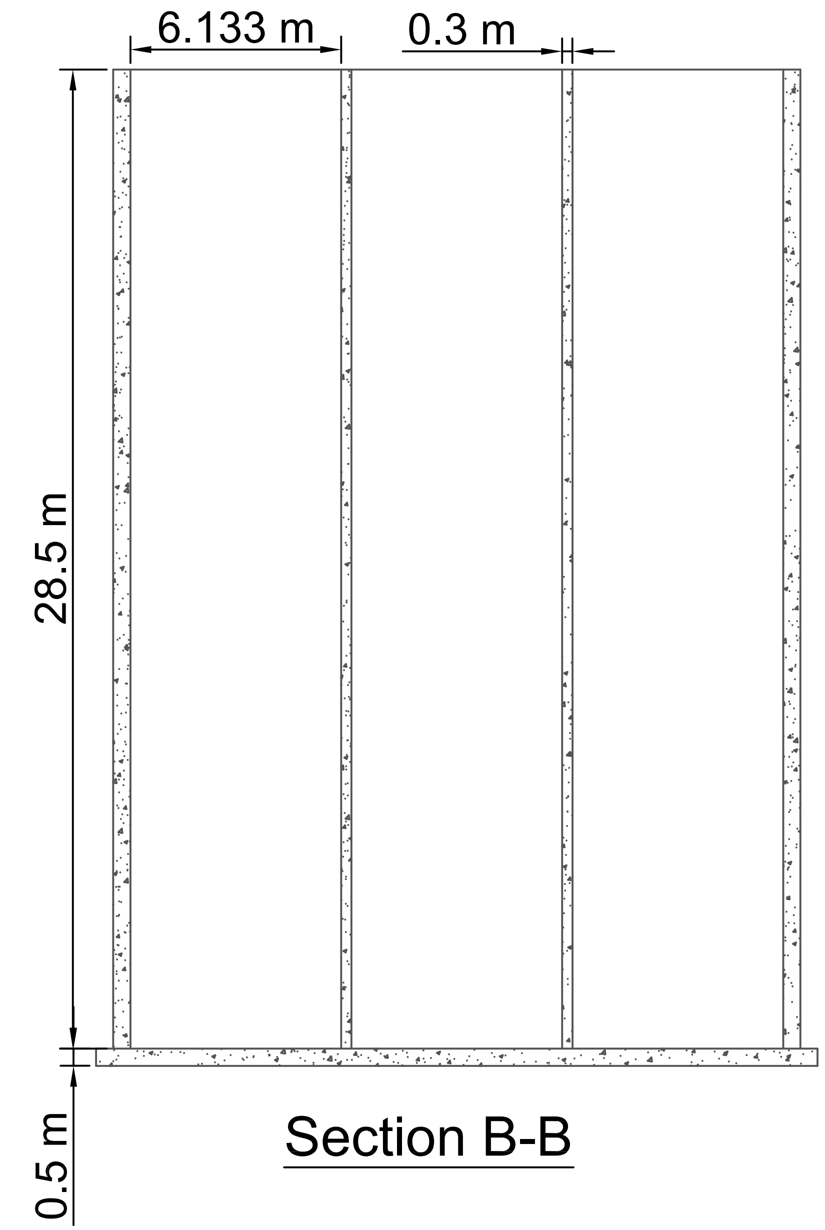
Legend:



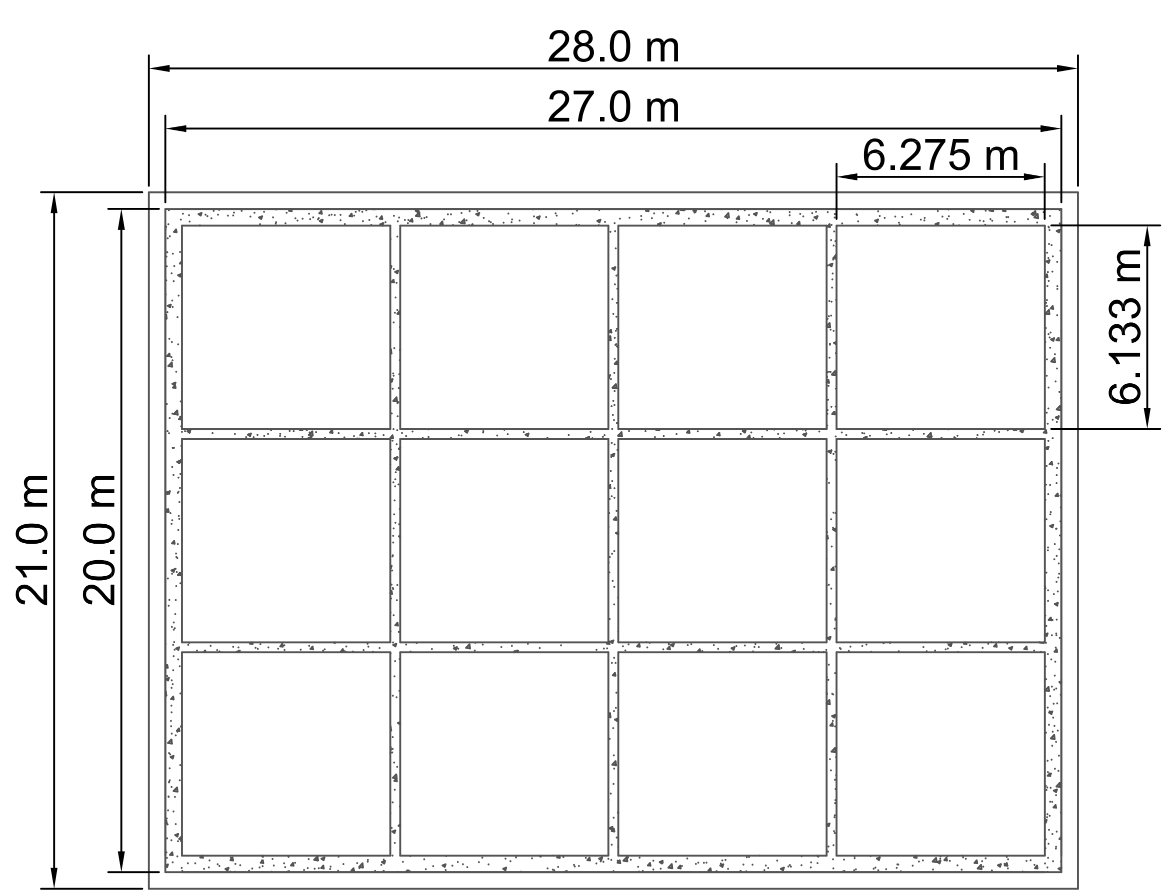
CONSULTANT: PARAMOUNT	TITLE: Site Plan of Concrete Caisson Option	DRAWING #: DW1-8700-07-PO2-01	DRAWN BY: PWC CHECKED BY: AGS
CLIENT: SNC·LAVALIN BAE-Newplan	PROJECT: St. Lawrence Marine Terminal	PROJECT #: 8700-07	APPROVED BY: SRG DATE: Mar. 16, 2010 SCALE: NTS



Plan View



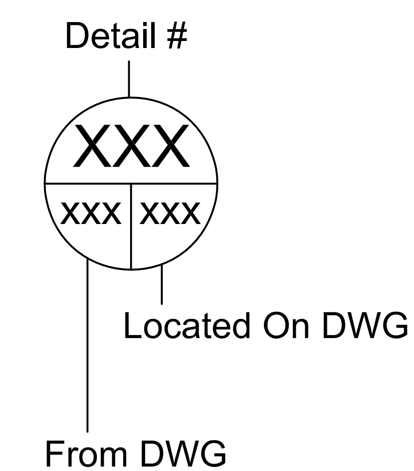
Section B-B



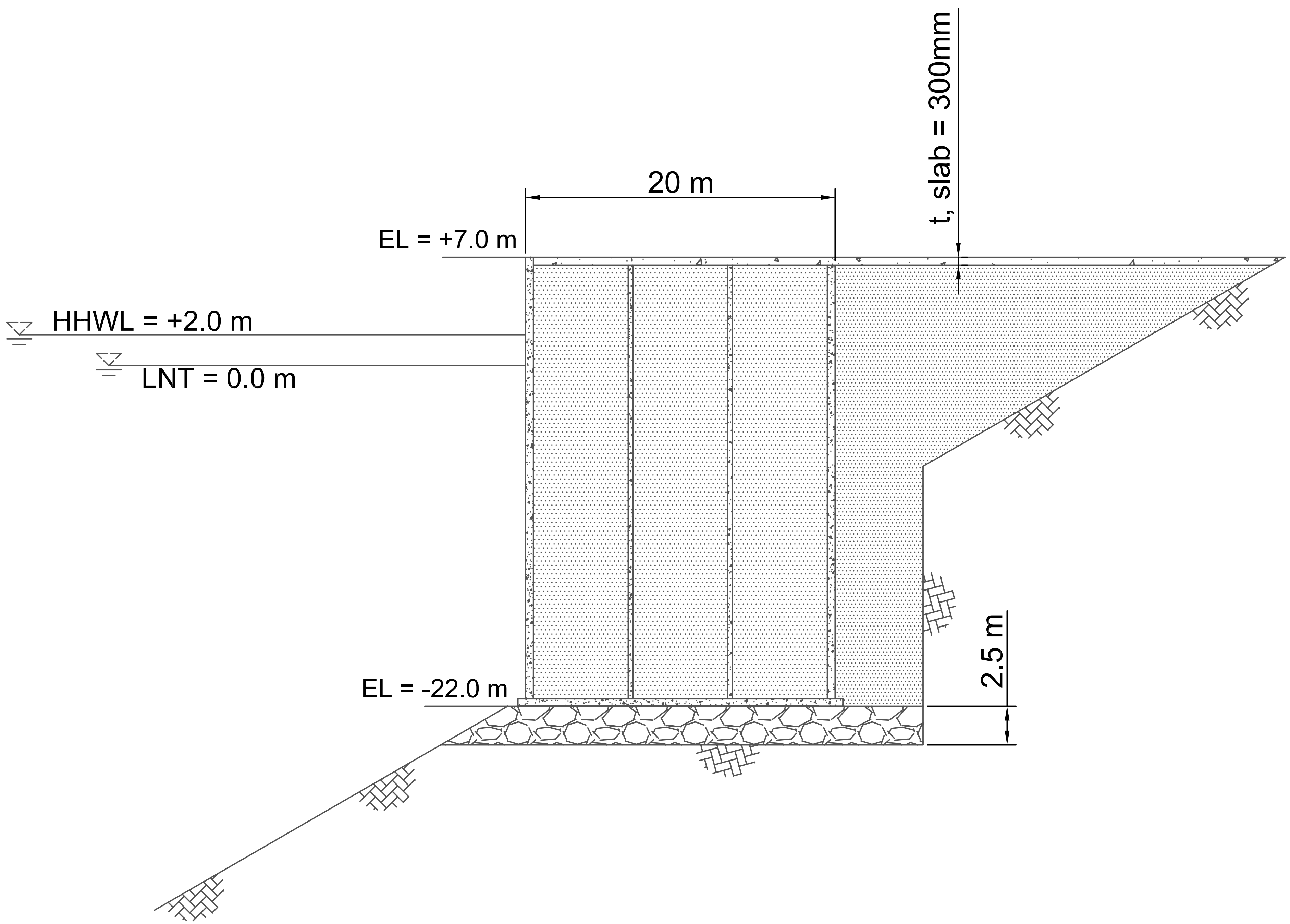
Detail 003

Notes:

Legend:



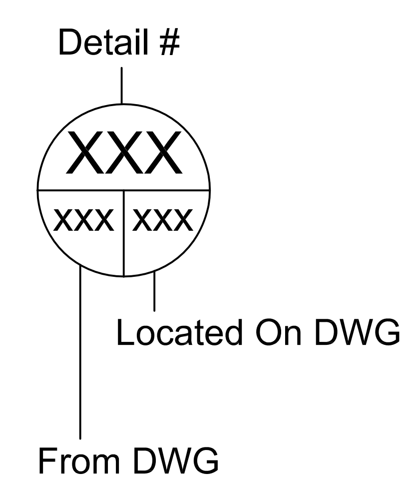
CONSULTANT: PARAMOUNT	TITLE: Caisson Details Plan & Cross Section	DRAWING #: DW1-8700-07-PO2-02	DRAWN BY: PWC CHECKED BY: AGS APPROVED BY: SRG
CLIENT: SNC•LAVALIN BAE-Newplan	PROJECT: St. Lawrence Marine Terminal	PROJECT #: 8700-07	DATE: Mar. 16, 2010 SCALE: NTS



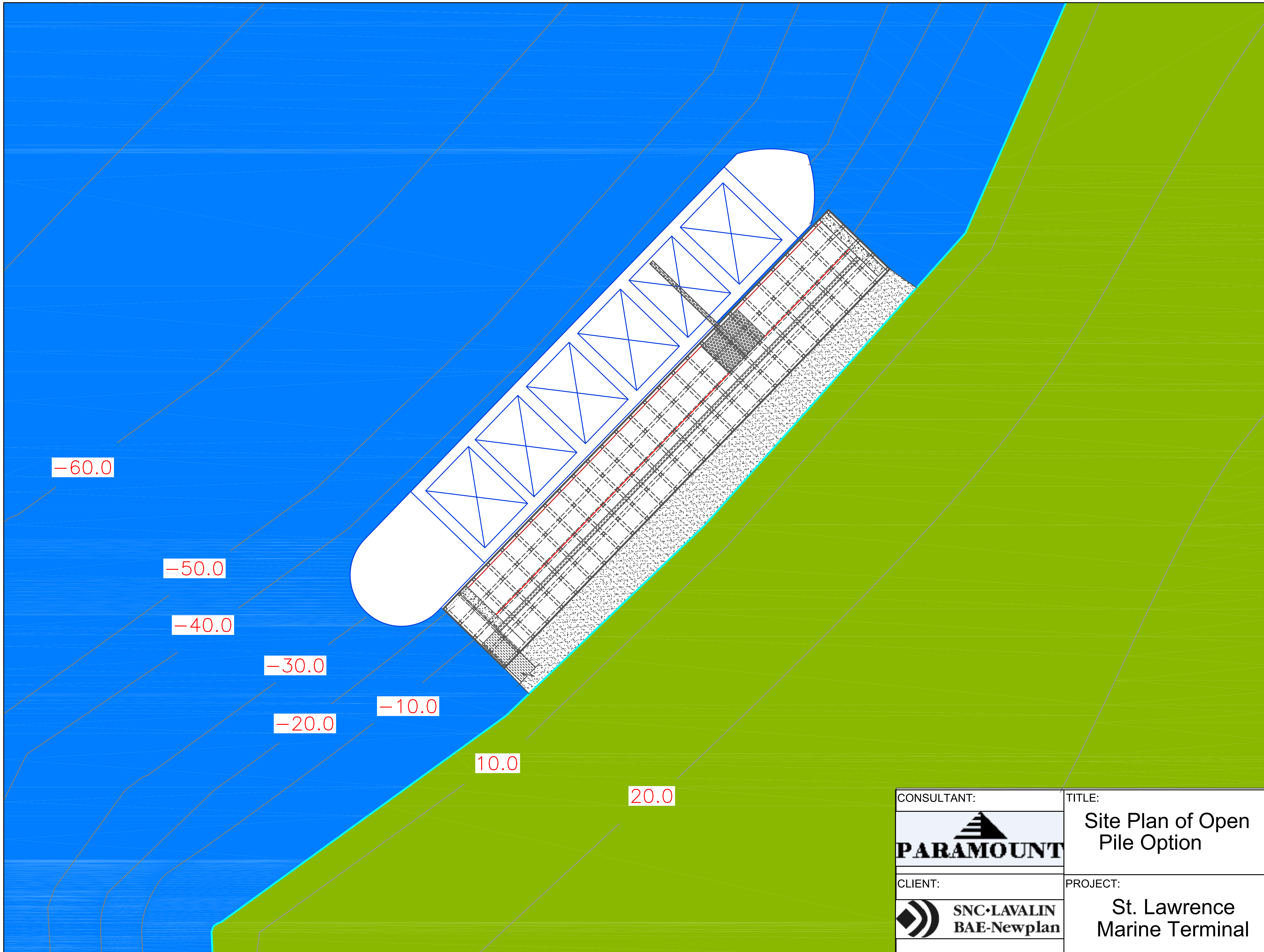
Section B-B

Notes:

Legend:

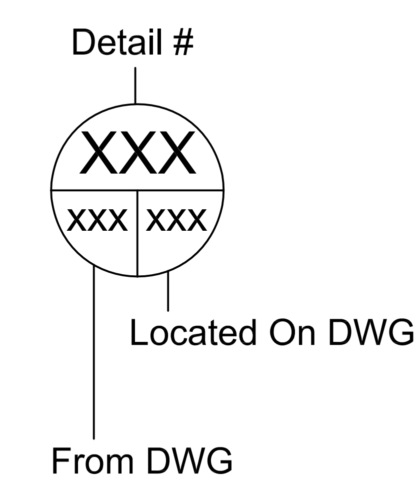


CONSULTANT:	TITLE:	DRAWING #:	DRAWN BY:
PARAMOUNT	Concrete Caisson Section	DW1-8700-07-PO2-03	PWC
			CHECKED BY: AGS
CLIENT:	PROJECT:	PROJECT #:	APPROVED BY: SRG
SNC•LAVALIN BAE-Newplan	St. Lawrence Marine Terminal	8700-07	DATE: Mar. 17, 2010
			SCALE: NTS

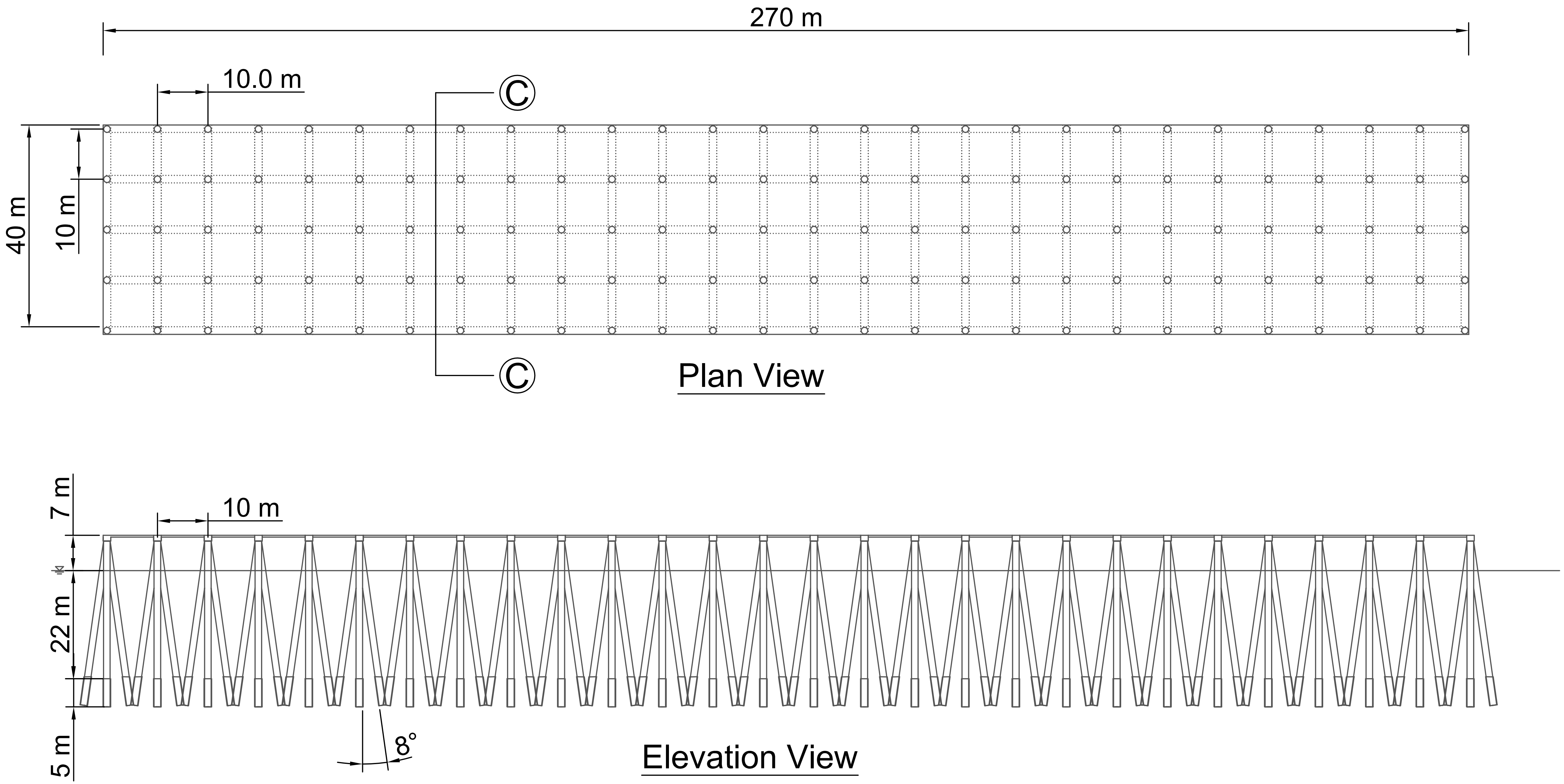


Notes:

Legend:



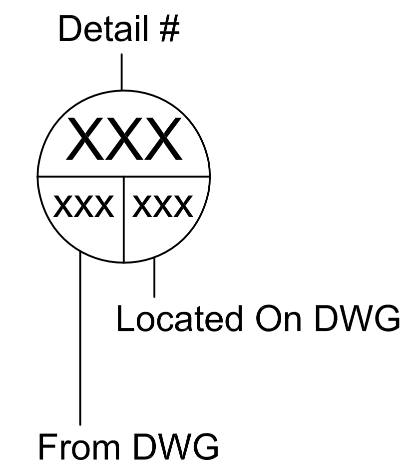
CONSULTANT: PARAMOUNT	TITLE: Site Plan of Open Pile Option	DRAWING #: DW1-8700-07-PO3-01	DRAWN BY: PWC CHECKED BY: AGS APPROVED BY: SRG
CLIENT: SNC·LAVALIN BAE-Newplan	PROJECT: St. Lawrence Marine Terminal	PROJECT #: 8700-07	DATE: Mar. 23, 2010 SCALE: NTS



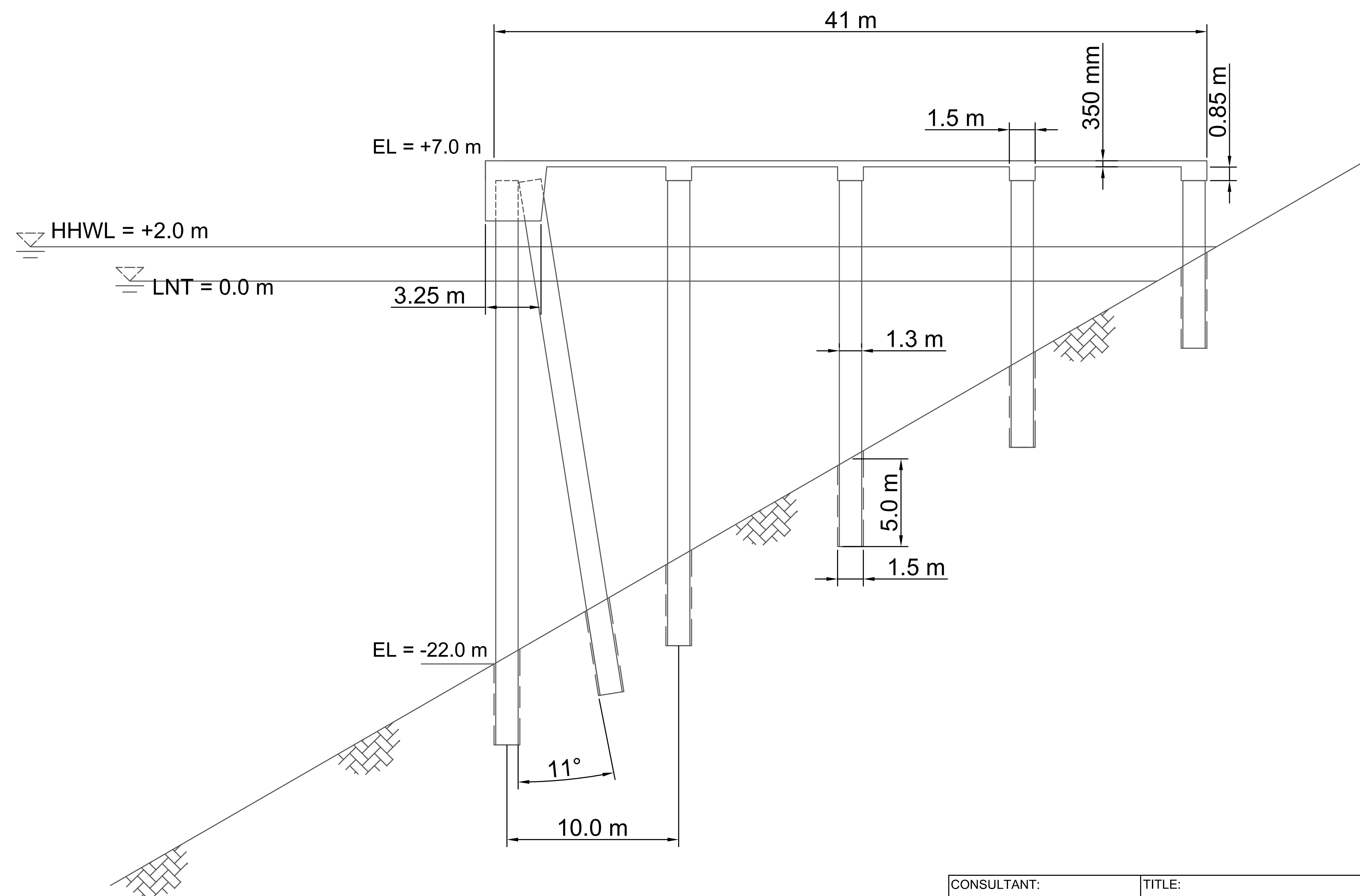
Notes:

1. Pipe piles are 1300 mm diameter with 30 mm wall thickness.
2. Pipe piles are to be concrete filled.

Legend:



CONSULTANT: PARAMOUNT	TITLE: Open Pile Plan and Elevation	DRAWING #: DW1-8700-07-PO3-02	DRAWN BY: PWC CHECKED BY: AGS APPROVED BY: SRG
CLIENT: SNC•LAVALIN BAE-Newplan	PROJECT: St. Lawrence Marine Terminal	PROJECT #: 8700-07	DATE: Mar. 23, 2010 SCALE: NTS



Section C-C

Notes:

Legend:

Detail #

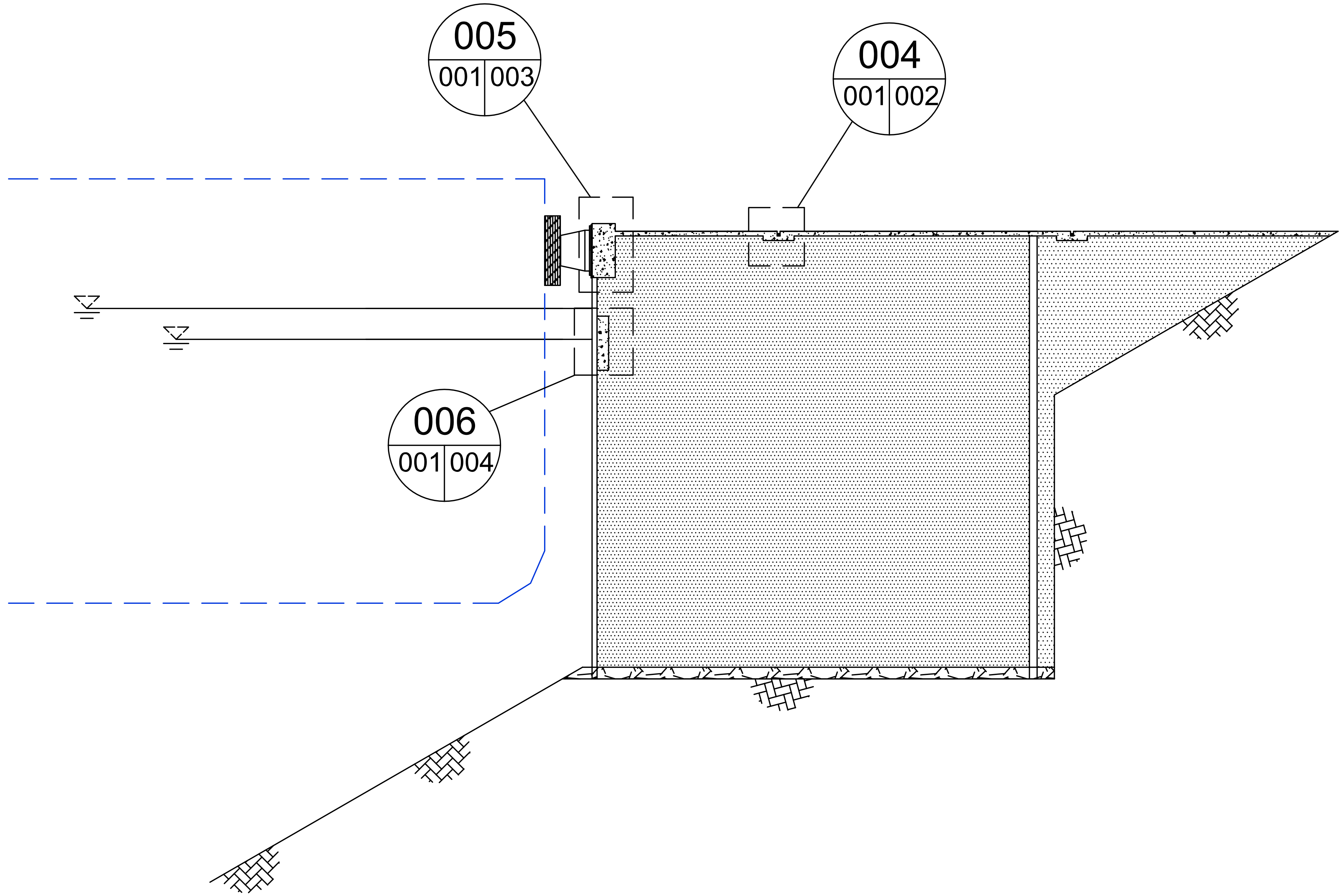
XXX

xxx xxx

Located On DWG

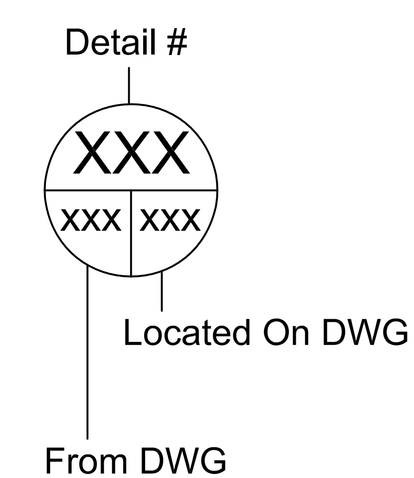
From DWG

CONSULTANT: PARAMOUNT	TITLE: Open Pile Section	DRAWING #: DW1-8700-07-PO3-03	DRAWN BY: PWC CHECKED BY: AGS APPROVED BY: SRG
CLIENT: SNC•LAVALIN BAE-Newplan	PROJECT: St. Lawrence Marine Terminal	PROJECT #: 8700-07	DATE: Mar. 23, 2010 SCALE: NTS



Notes:

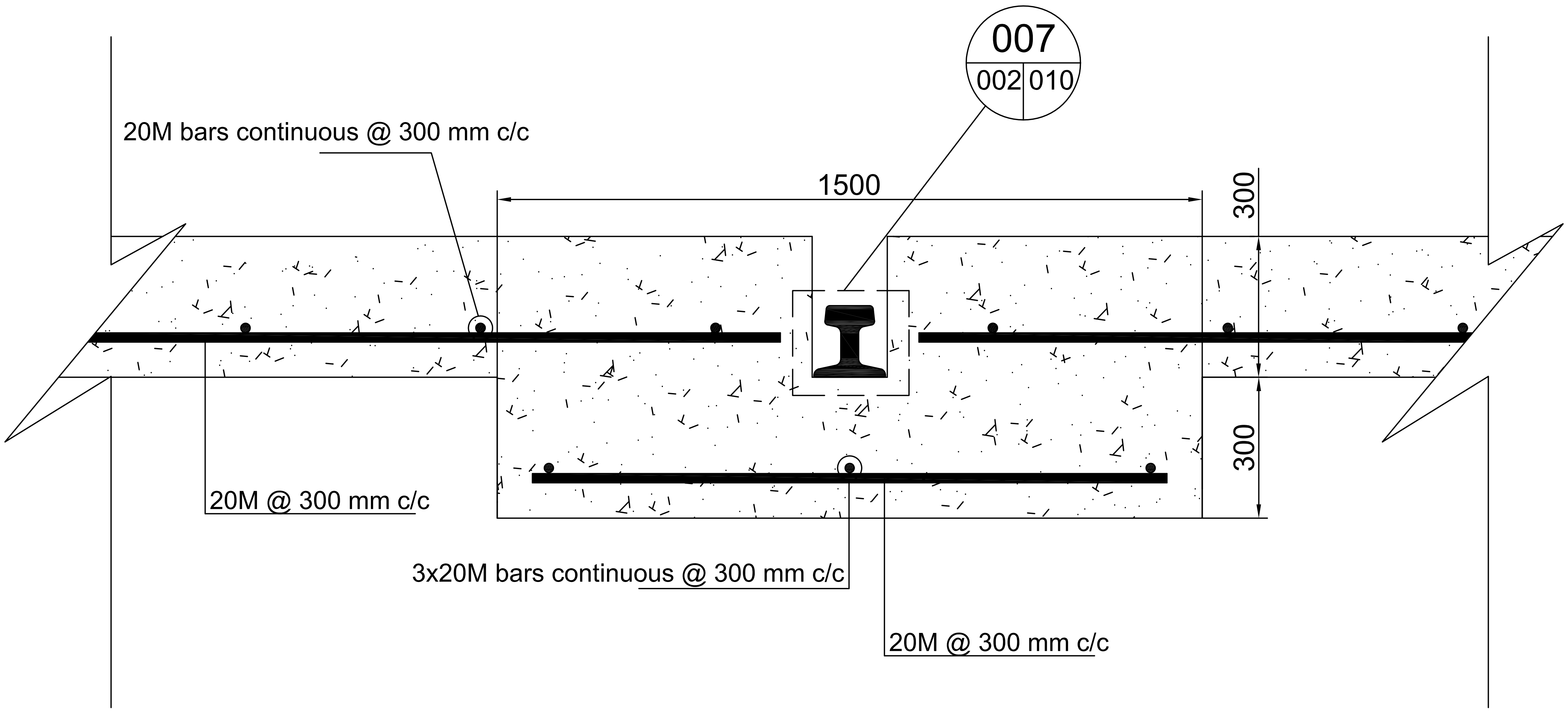
Legend:



CONSULTANT: PARAMOUNT	TITLE: Cellular Sheet Pile Detailed Section	DRAWING #: DW1-8700-07-001	DRAWN BY: PWC
			CHECKED BY: AGS
CLIENT: SNC·LAVALIN BAE-Newplan	PROJECT: St. Lawrence Marine Terminal	PROJECT #: 8700-07	APPROVED BY: SRG
			DATE: Mar. 26, 2010
			SCALE: NTS

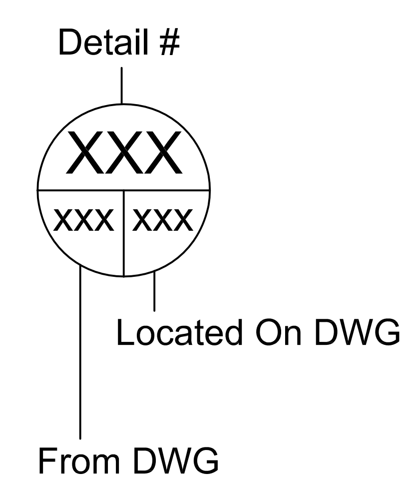
Notes:

1. All dimensions in millimeters
2. Concrete is 35 MPa at 28 days.

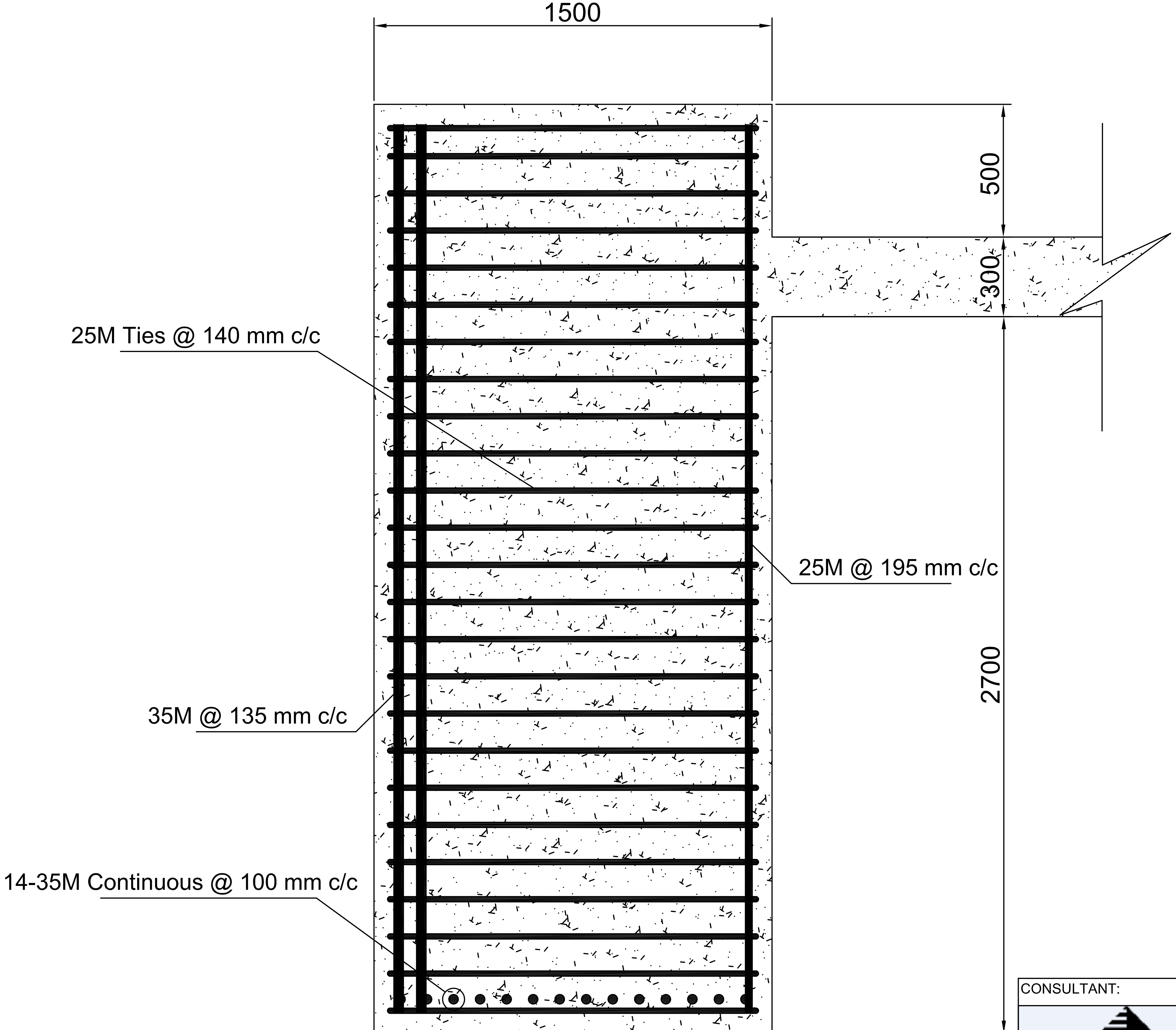


Detail 004: Slab and Crane Reinforcement Detail

Legend:



CONSULTANT: PARAMOUNT	TITLE: Slab and Crane Reinforcement Details	DRAWING #: DW1-8700-07-002	DRAWN BY: PWC	CHECKED BY: AGS
CLIENT: SNC•LAVALIN BAE-Newplan	PROJECT: St. Lawrence Marine Terminal	PROJECT #: 8700-07	APPROVED BY: SRG	DATE: Apr. 2, 2010 SCALE: NTS



Detail 005: Cope Wall Detail

Notes:

1. All dimensions in millimeters
2. Concrete is 35 MPa at 28 days.

Legend:

Detail #

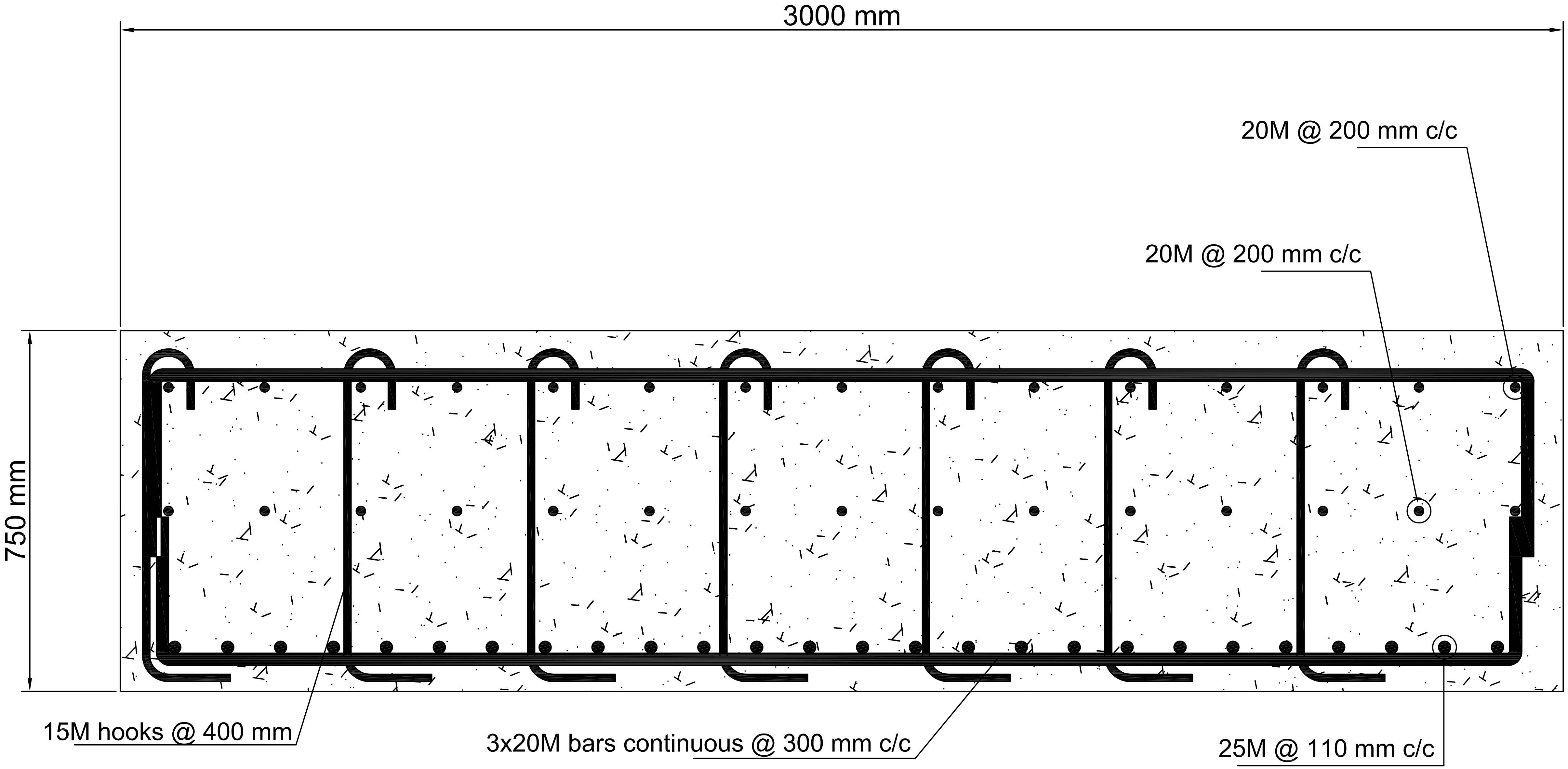
XXX

xxx | xxx

Located On DWG

From DWG

CONSULTANT:	TITLE:	DRAWING #:	DRAWN BY: PWC
PARAMOUNT	Cope Wall Details	DW1-8700-07-003	CHECKED BY: AGS
CLIENT: SNC•LAVALIN BAE-Newplan	PROJECT: St. Lawrence Marine Terminal	PROJECT #: 8700-07	APPROVED BY: SRG
			DATE: Apr. 4, 2010
			SCALE: NTS

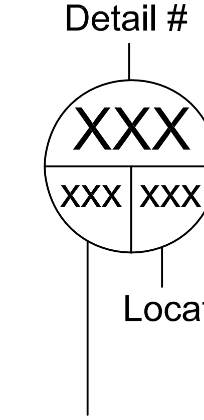


Detail 006: Plan view of Ice Reinforcement Plan View

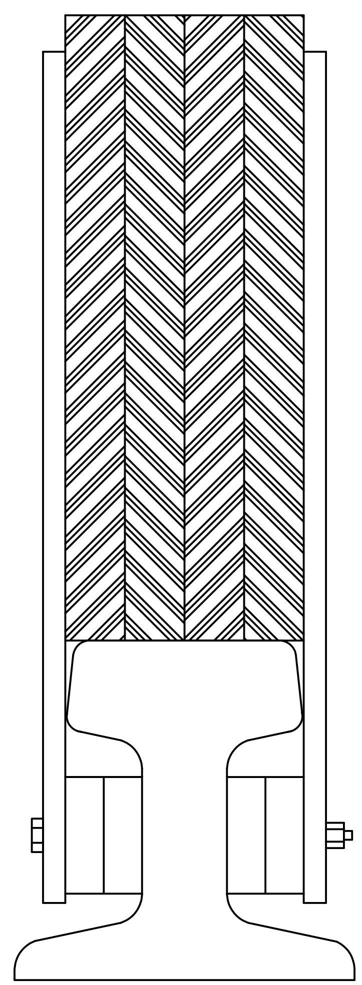
Notes:

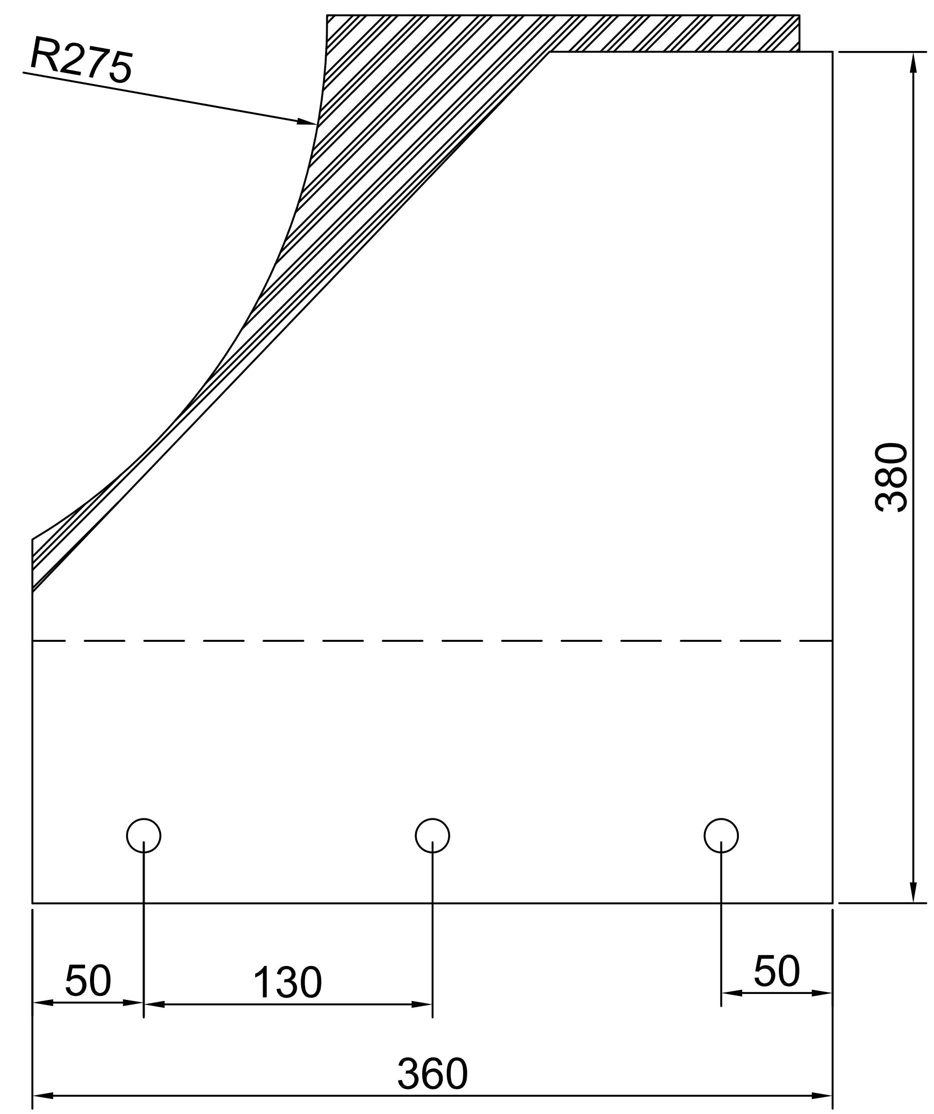
1. All dimensions in millimeters
2. Concrete is 30 MPa at 28 days.
3. Panels are pre-cast.

Legend:



CONSULTANT: PARAMOUNT	TITLE: Ice Reinforcement Details	DRAWING #: DW1-8700-07-004	DRAWN BY: PWC CHECKED BY: AGS APPROVED BY: SRG
CLIENT: SNC•LAVALIN BAE-Newplan	PROJECT: St. Lawrence Marine Terminal	PROJECT #: 8700-07	DATE: Apr. 3, 2010 SCALE: NTS

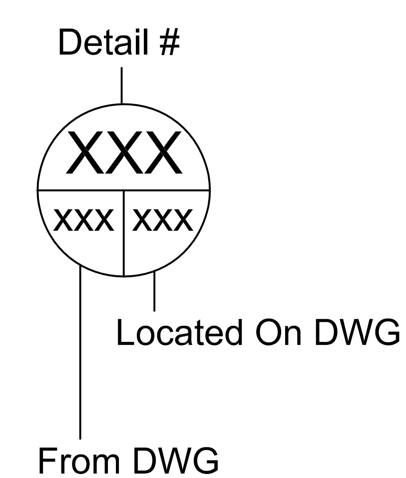




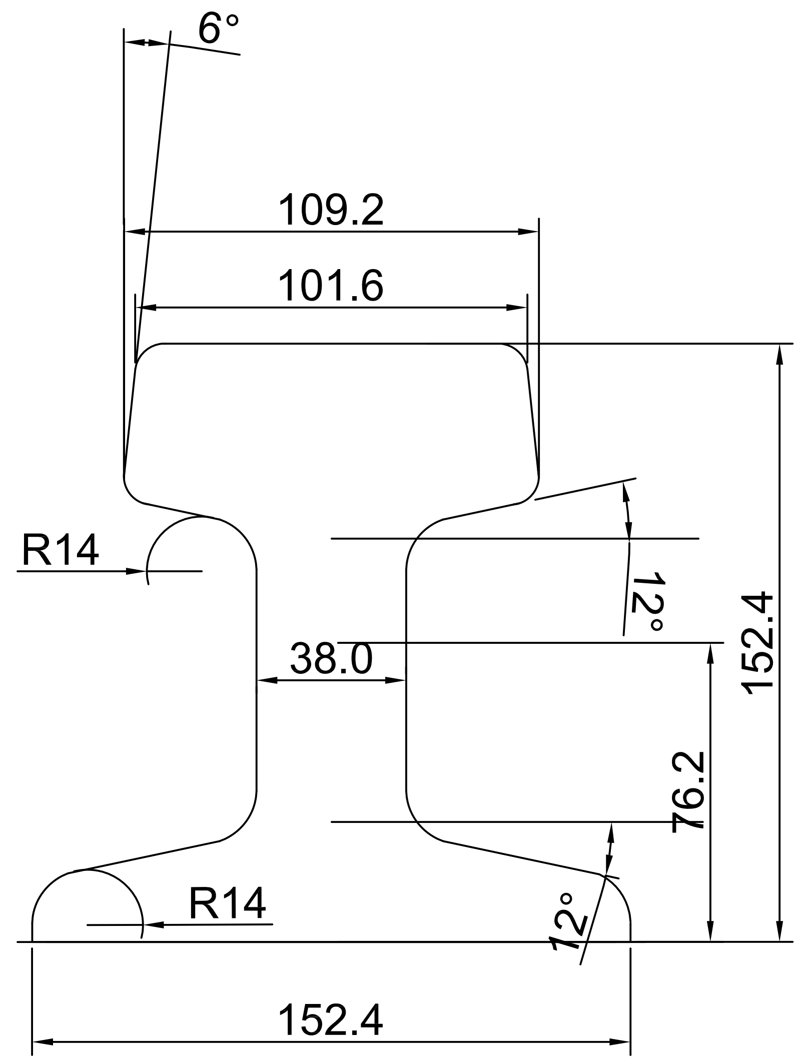
Notes:

1. Rail is CR-171.
2. All dimensions in millimeters

Legend:



CONSULTANT: PARAMOUNT	TITLE: Heavy Duty Wheel Stop Detail	DRAWING #: DW1-8700-07-005	DRAWN BY: PWC CHECKED BY: AGS APPROVED BY: SRG
CLIENT: SNC•LAVALIN BAE-Newplan	PROJECT: St. Lawrence Marine Terminal	PROJECT #: 8700-07	DATE: Apr. 4, 2010 SCALE: NTS

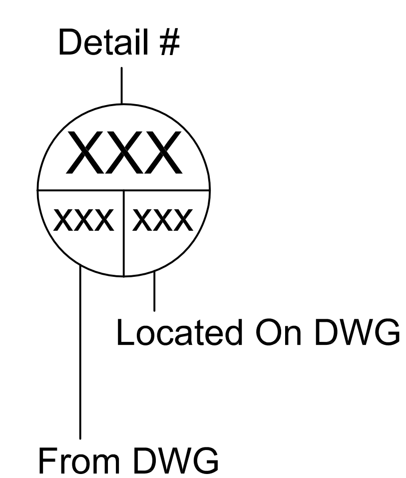


Detail 007: Crane Rail Details

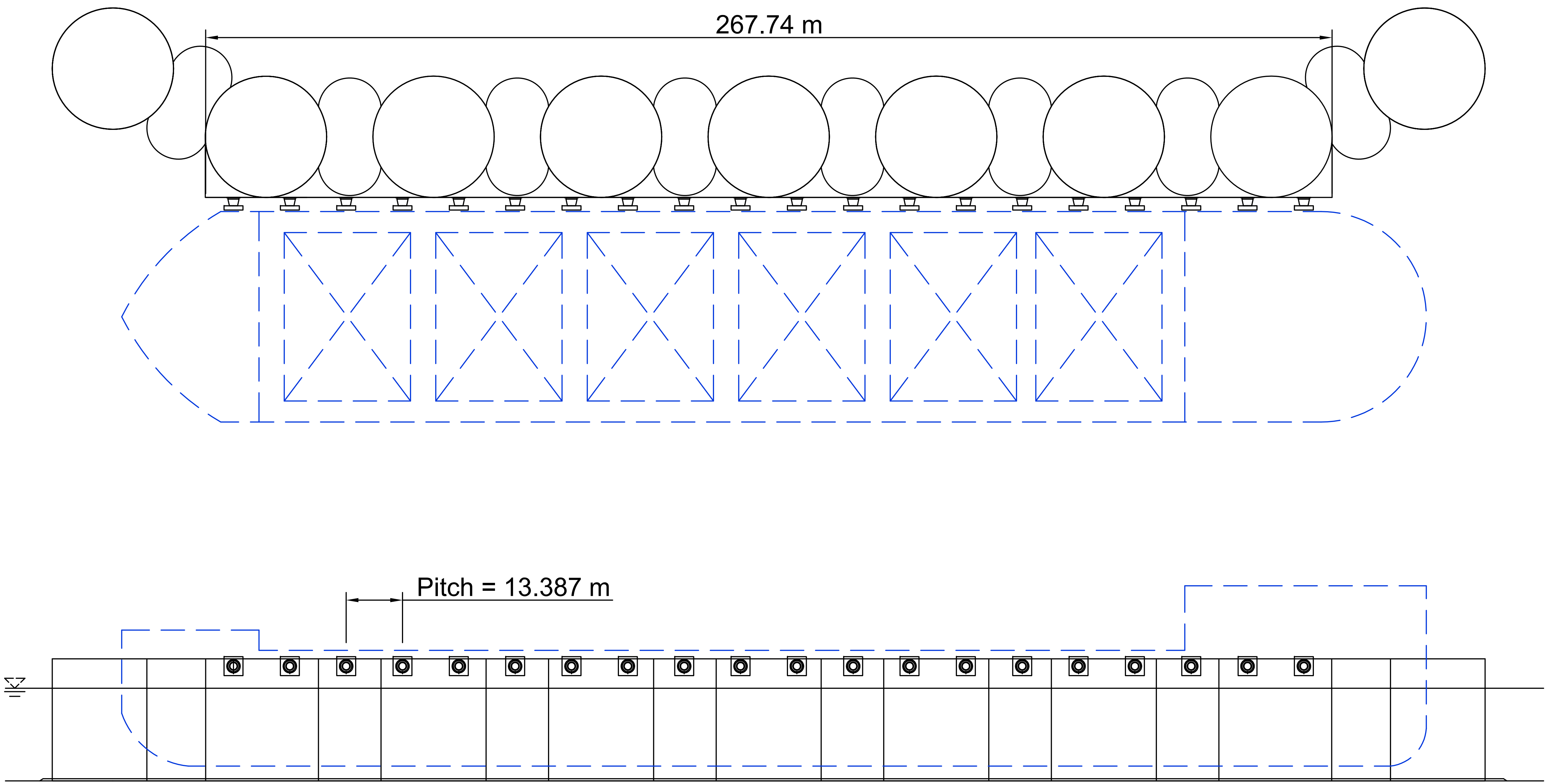
Notes:

1. Rail is CR-171.
2. All dimensions in millimeters

Legend:



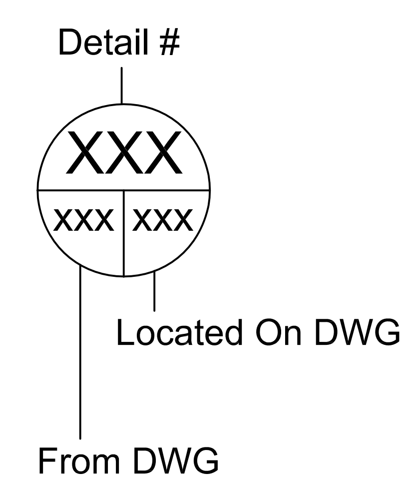
CONSULTANT: PARAMOUNT	TITLE: Crane Rail Details	DRAWING #: DW1-8700-07-006	DRAWN BY: PWC CHECKED BY: AGS APPROVED BY: SRG
CLIENT: SNC·LAVALIN BAE-Newplan	PROJECT: St. Lawrence Marine Terminal	PROJECT #: 8700-07	DATE: Apr. 4, 2010 SCALE: NTS



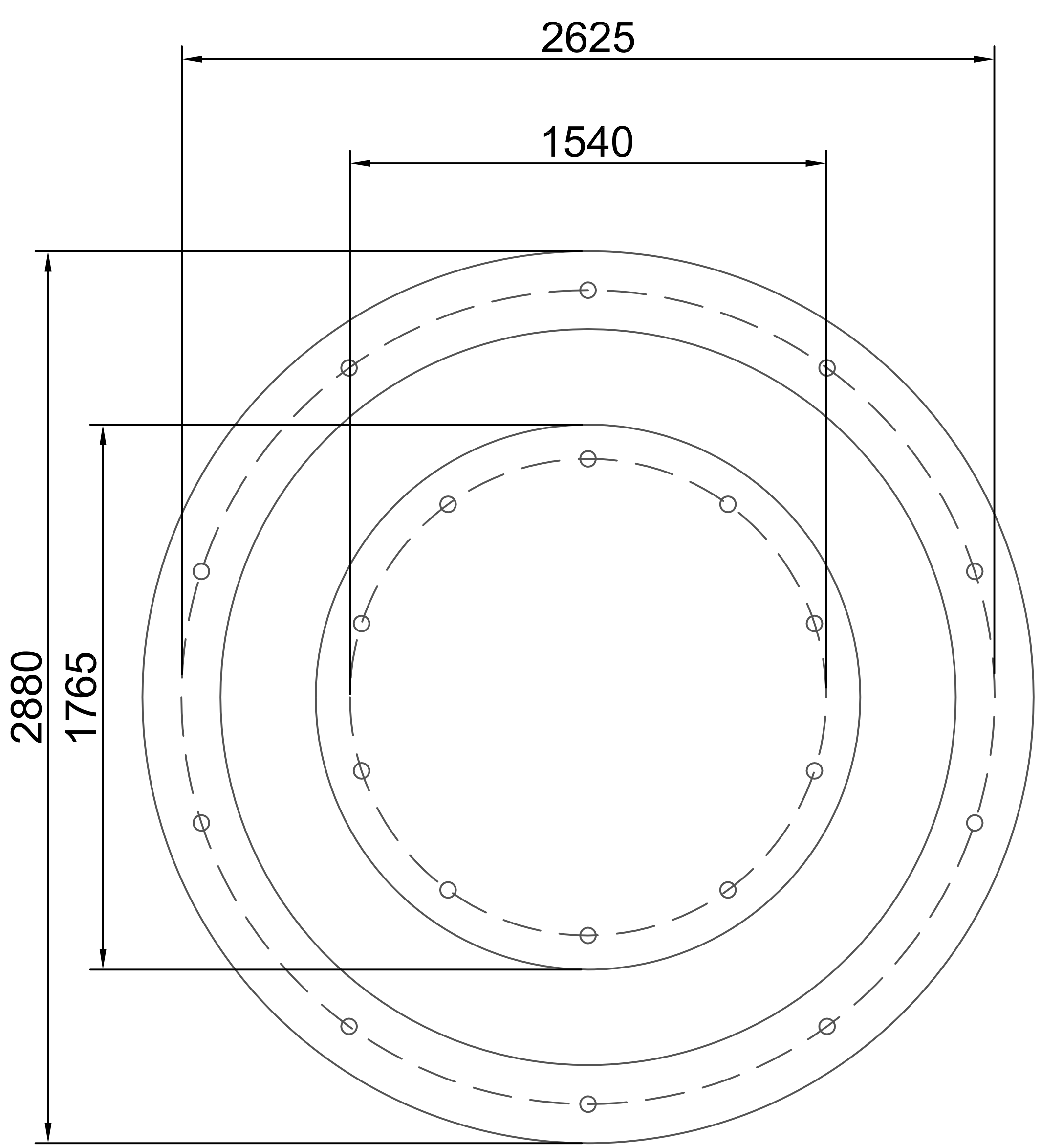
Notes:

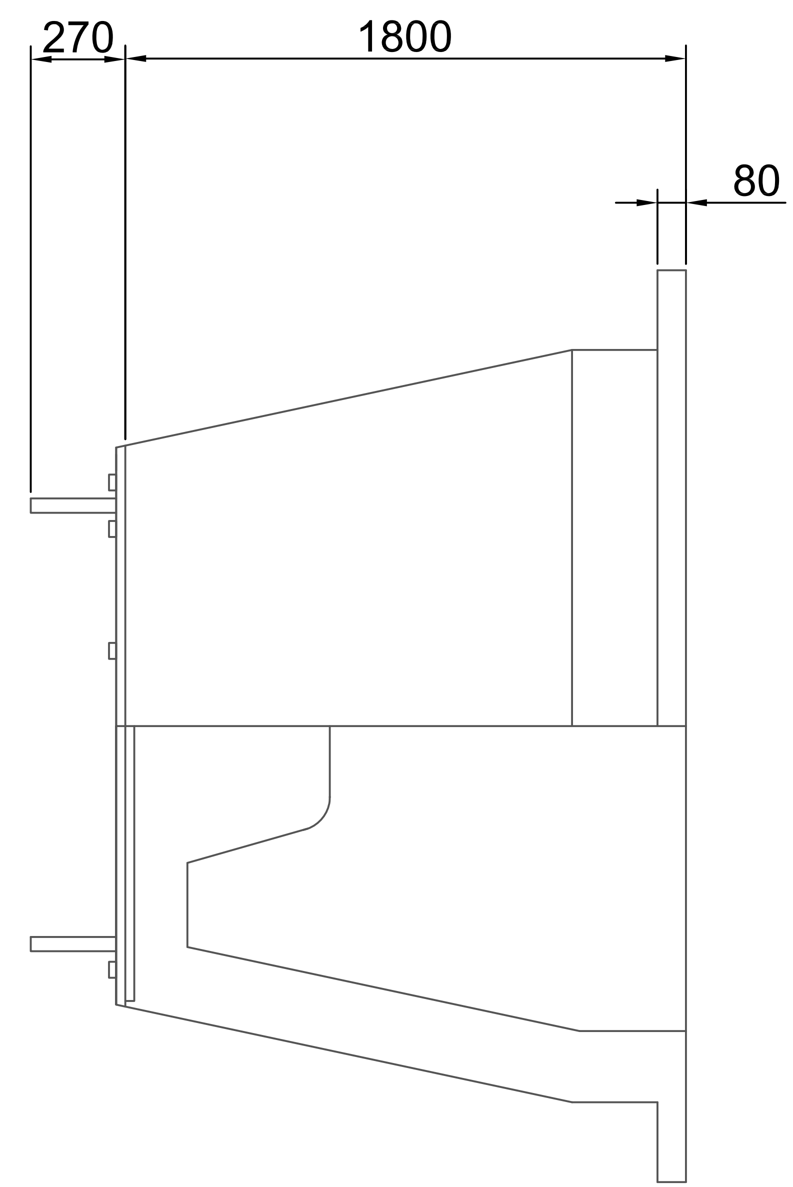
1. 20 fenders in total.
2. Fender pitch is equal to 13.387 m.

Legend:



CONSULTANT: PARAMOUNT	TITLE: Fender Plan and Elevations	DRAWING #: DW1-8700-07-007	DRAWN BY: PWC CHECKED BY: AGS APPROVED BY: SRG
CLIENT: SNC•LAVALIN BAE-Newplan	PROJECT: St. Lawrence Marine Terminal	PROJECT #: 8700-07	DATE: Mar. 31, 2010 SCALE: NTS





Notes:

1. Selected fender is SCN1800 E3.0
2. Fender secured to concrete cope wall with 10-M56 anchor bolts.
3. All dimensions in millimeters

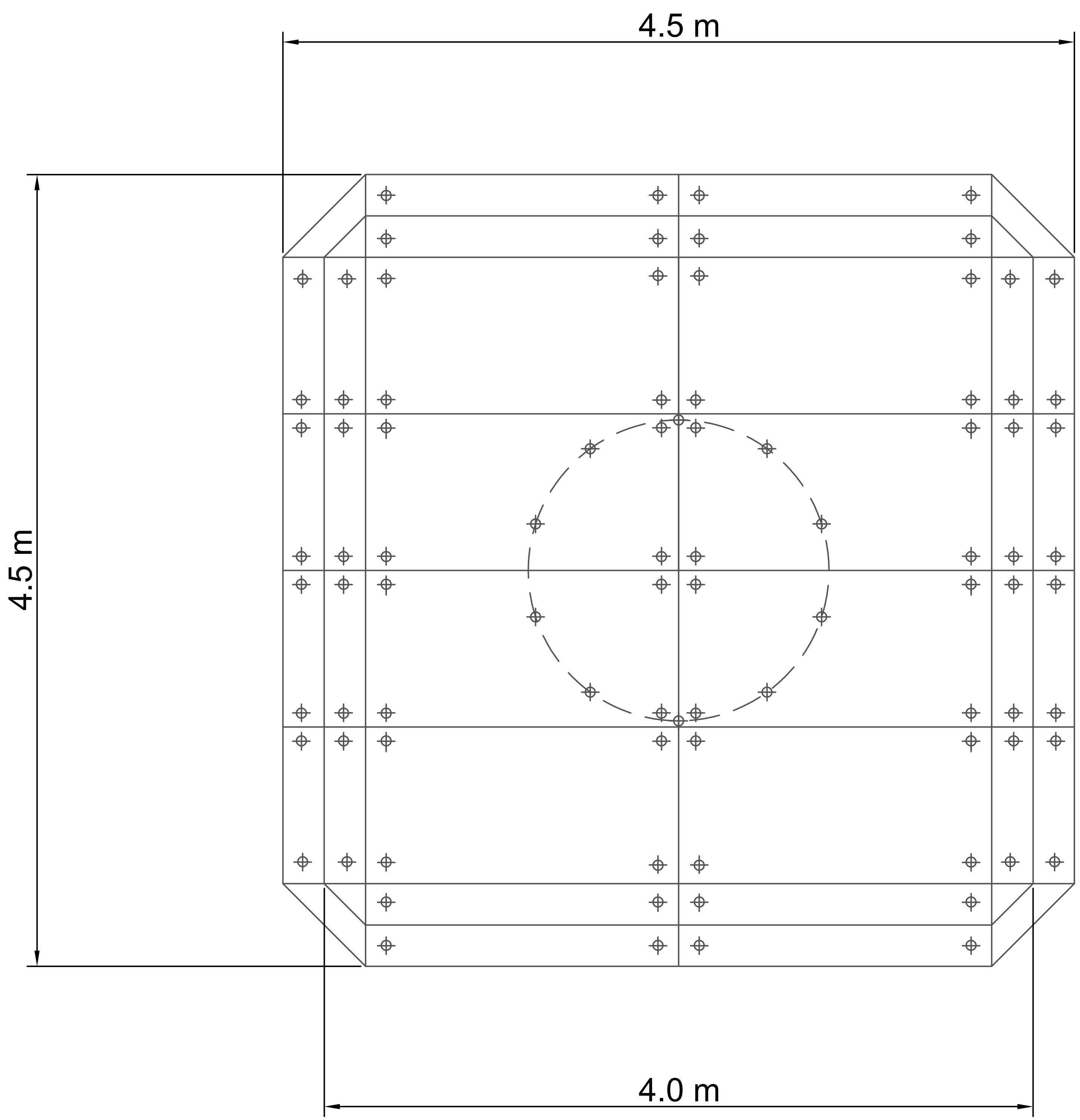
Legend:

Detail #
XXX
XXX XXX
Located On DWG
From DWG

Fender Properties

E	R	Eff. (E/R)	Weight
(KN-m)	(KN)	(KN-m/KN)	(kg)
3530	3775	0.932	6618

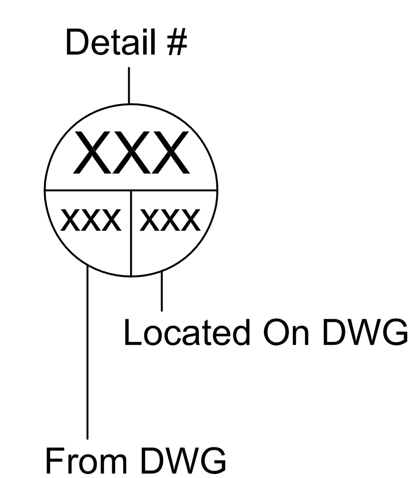
CONSULTANT: PARAMOUNT	TITLE: Fender Details	DRAWING #: DW1-8700-07-008	DRAWN BY: PWC CHECKED BY: AGS APPROVED BY: SRG
CLIENT: SNC·LAVALIN BAE-Newplan	PROJECT: St. Lawrence Marine Terminal	PROJECT #: 8700-07	DATE: Apr. 4, 2010 SCALE: NTS



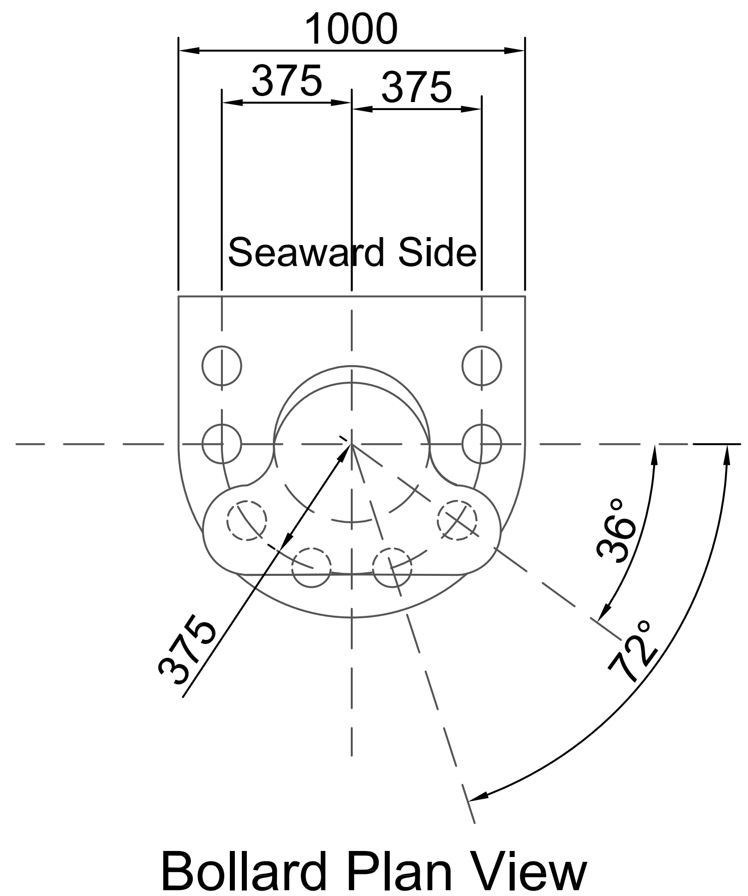
Notes:

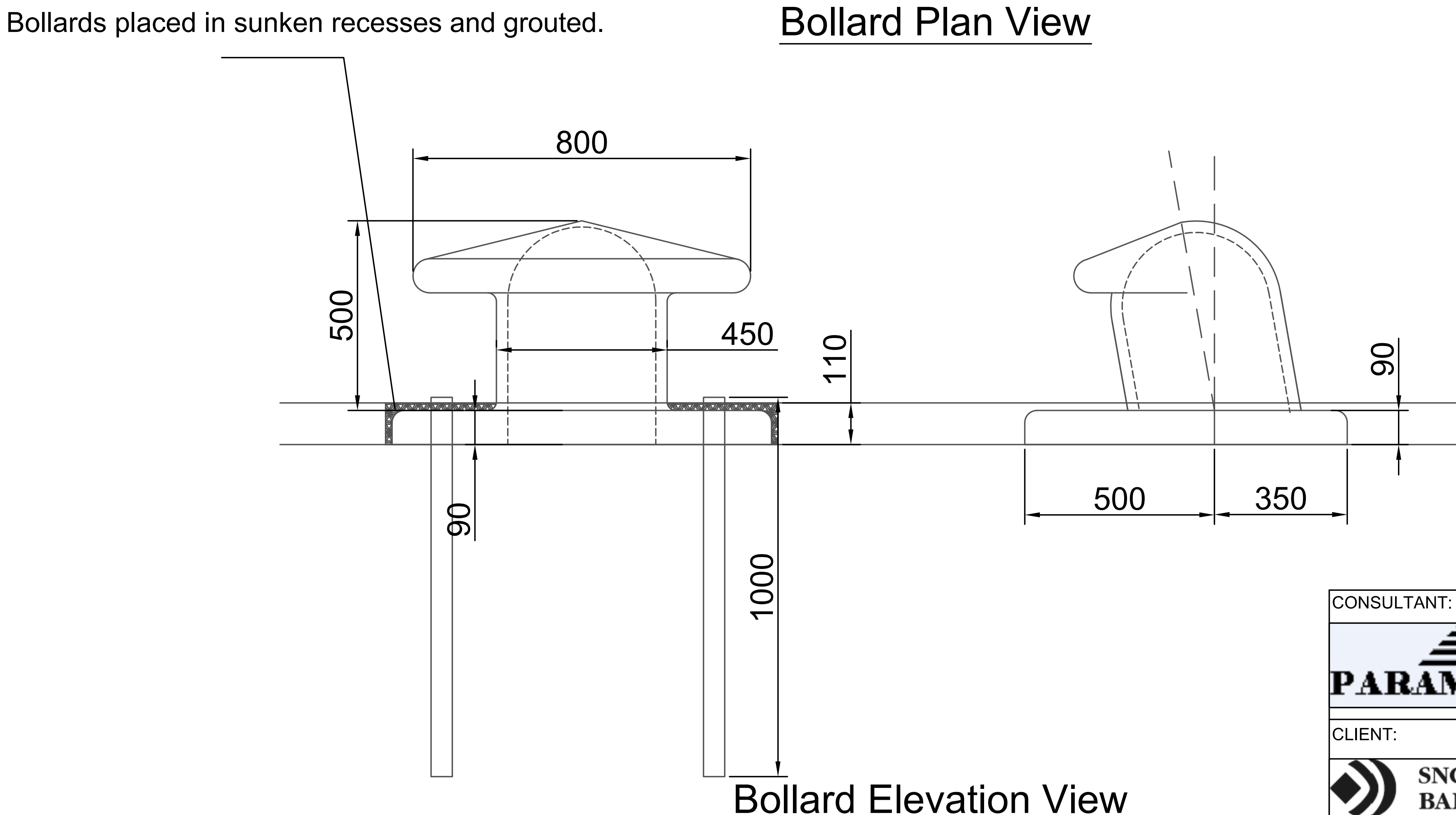
1. Fender panel material is UHMW-PE.
2. Steel panel thickness is 10mm.
3. Paint coating complies with ISO EN 12944.
4. C5-M class paint.

Legend:



CONSULTANT:	TITLE:	DRAWING #:	DRAWN BY:
PARAMOUNT	Fender Panel Details	DW1-8700-07-009	PWC
			CHECKED BY: AGS
			APPROVED BY: SRG
CLIENT: SNC•LAVALIN BAE-Newplan	PROJECT: St. Lawrence Marine Terminal	PROJECT #: 8700-07	DATE: Apr. 4, 2010
			SCALE: NTS

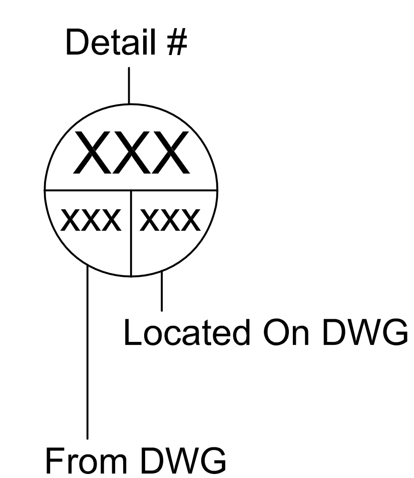




Notes:

1. Bollards are 80-55-6 Grade Ductile Cast Iron.
2. A325 M56 Anchor Bolts length of 1.0 m.
3. SA2.5 Grade Blasting Materials
4. Class C5M Paint.

Legend:



CONSULTANT: PARAMOUNT	TITLE: Tee Bollard Details	DRAWING #: DW1-8700-07-010	DRAWN BY: PWC CHECKED BY: AGS APPROVED BY: SRG
CLIENT: SNC•LAVALIN BAE-Newplan	PROJECT: St. Lawrence Marine Terminal	PROJECT #: 8700-07	DATE: Apr. 4, 2010 SCALE: NTS

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