Parry Sound Harbour, Ontario

Bay St. Wharf (#401)
Condition Assessment Report

Small Craft Harbours, Fisheries and Oceans Canada

Project number: 60719231

March 2024

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Executive Summary

Parry Sound Harbour is located on the east shore of Georgian Bay in the Town of Parry Sound, Ontario. Parry Sound Harbour is administered and maintained by Fisheries and Oceans Canada, Small Craft Harbours (SCH). The Town of Parry Sound manages the facility under a lease with SCH. The Bay St. Wharf (#401), also known as the Town Dock, was investigated as part of this assignment.

Record drawings indicate the Wharf was originally constructed in 1921/1922. Although originally constructed mainly using timber piles, pile caps, and decking, subsequent reconstructions over the 100-year history of the Wharf divided the structure into four distinct substructure groups. The four substructures of the Wharf (and their construction years) are timber cribs (1952), timber pile bents/caps/stringers (1921), steel sheet piling (1967), and timber sheet piling (1931).

AECOM carried out a site investigation of above water components and Watech Services Inc. carried out an underwater inspection using divers and a remotely operated vehicle (ROV).

Structure A was deemed to be in fair condition with localized deterioration. Structure B was deemed to be in fair condition with localized areas in poor condition. Structure C was deemed to be in fair to good condition. Structure D was deemed to be in fair condition with areas in poor condition.

Remaining Useful Residual Life (URL) for each component was calculated using available information on construction years. All three timber substructure groups are considered to have exceeded their remaining useful residual life. The steel sheet piling, constructed in 1967, is considered to have a URL of 23 years remaining. The concrete superstructures have all exceeded their URL.

Sheet pile capacity at Structure C was confirmed to be sufficient for the Island Queen V cruise ship to continue docking at this location. Structural analysis of the Structure D pile bents established a maximum ship berthing load of 200 kN, which must be compared with the berthing energies calculated for vessels and the energy deflection behaviour of the specific fendering system used. Based on completed calculations, bollards in the current condition cannot safely moor any cruise vessel considered, except for the Island Queen V. A summary of recommendations, preliminary cost estimates and recommended timing is presented below:

Structure	Description of Work	Estimated Cost (2024 Dollars)	Timing
Overall	►Install ladders along length of wharf.	**►$50,000**	**►Priority 1**
	►Localized concrete deck repairs	**►$150,000**	**►Priority 2**
Structure A 0+000.0 to 0+013.7	►Install curb rail	**►$10,000**	**►Priority 1**
	►Encapsulate Structure A	**►$300,000**	**►Priority 4**
Structure B 0+013.7 to 0+048.3	►Install of curb rail	**►$51,000**	**►Priority 1**
	►Concrete repair of cope wall		**►Priority 1**
	►Dredge lakebed at outfall		**►Priority 1**
	►Encapsulate Structure B	**►$520,000**	**►Priority 3**
Structure C 0+048.3 to 0+117.8	►Replace timber fenders	**►$30,000**	**►Priority 2**
	►Replace entire deck	**►$400,000**	**►Priority 4**
Structure D 0+117.8 to 0+273.3	►Repair concrete stairs	**►$85,000**	**►Priority 1**
	►Repair railings		**►Priority 1**
	►Install stair handrails		**►Priority 1**
	►Repair concrete at bollards	**►$445,000**	**►Priority 2**
	►Install fenders		**►Priority 2**
	►Encapsulate Structure D	**►$6,700,000**	**►Priority 4**

Priority 1: recommended immediately.
Priority 2: recommended for completion within 1 to 5 years.
Priority 3: recommended for completion within 6 to 10 years.
Priority 4: recommended for completion within 11 to 15 years.

1. Introduction

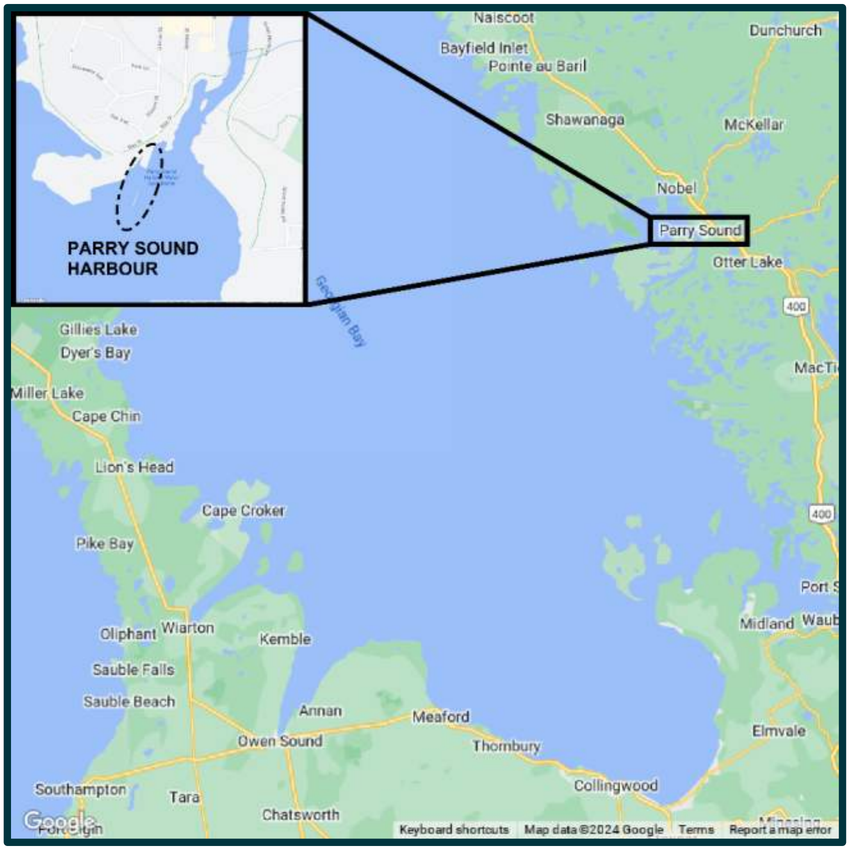
Parry Sound Harbour is located on the east shore of Georgian Bay in the Town of Parry Sound, Ontario. It is located approximately 225 km north of Toronto and is accessible by Bay Street.

Parry Sound Harbour is administered and maintained by Fisheries and Oceans Canada, Small Craft Harbours (SCH). The Town of Parry Sound manages the facility under a lease with SCH. The facility is a full-service marina intended as a recreational facility, though it also operates as a departure location for various sightseeing tours and cruises of the 30,000 Islands.

The Bay St. Wharf (#401) was investigated as part of this assignment. This report outlines a description of the structure, summarizes observations from the site investigations, outlines the structural evaluation and provides a condition assessment and recommendations including preliminary cost estimates.

The location of Parry Sound Harbour is shown in **Figure 1**.

Figure 1. Key Map



2. Background Information

2.1 General

SCH infrastructure at Parry Sound Harbour consists of the Bay St. Wharf (also referred to as the Town Dock). The Harbour is managed by the Town of Parry Sound. The facility generally accommodates pleasure craft and small to medium sized cruise and sightseeing ships. The harbour is also a popular area used by the public who enjoy walking along the length of the wharf.

Recently, the Town has found its cruise ship season become increasingly busy. Between May and October 2023, a record number of 33 cruise ship visits (6 ships involving 4 companies) were scheduled[1]. This number is a marked increase from the 20 visits logged in 2022, and the typical pre-pandemic numbers of 10 to 12 visits per season. The visits have led to increased economic growth for the Town, with local businesses providing tourism activities for docked cruise passengers[2].

The existing Wharf is divided into four main sections based on substructure types: stone-filled timber cribs, round timber pile bents with pile cap and stringers, Z-shaped steel sheet piling, and timber sheet piling. The wharf is oriented in a north-south direction, beginning north at the shore and extending 273.3 m south into Georgian Bay.

Record drawings for the construction of the Wharf date back to 1921. The original Wharf substructure consisted of round timber pile bents with pile caps and stringers, and the superstructure was constructed using timber decking. The 1921 drawings divide the wharf structure along its length into four main sections based on timber pile spacings:

1. Section AB was detailed as approximately 50.9 m long and 4.88 m wide. The first 4.88 m starting at the north shore was a stone-filled timber crib. The remaining substructure consisted of pile bents at 1.22 m spacings, alternating between two and three piles per bent.
2. Section CD was detailed as approximately 12.2 m long and 6.1 m wide. The substructure pile bents were placed at 1.524 m spacings, alternating between three and five piles per bent.
3. Section EF was detailed as approximately 46.33 m long. The first 21.3 m section was 6.1 m wide with similar substructure to Section CD. The remaining 24.4 m section was 9.14 m wide and alternating seven and three piles per bent.
4. The Outer Section was detailed as approximately 174.35 m long by 9.14 m wide. The substructure pile bents were spaced at 3.05 m. Each bent contained seven piles.

A previous wharf structure referred to as the “Slab Wharf” existed prior and is detailed in the 1921 drawings. The Slab Wharf coincided with the Bay St. Wharf along the western portion of sections AB through EF. During the original 1921 construction of the Bay St. Wharf, some piles were driven through the intersecting Slab Wharf structure.

Various repairs completed during the Wharf’s history have resulted in a redefinition of the Wharf sections, based on the existing substructures. Section AB was divided into two structures after a stone-filled timber crib substructure was constructed in 1952 along STA 0+000 to 0+013.7. The remaining section keeps the original 1921/1922 substructure from STA 0+013.7 to 0+048.3.

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[1] Clark, T. (2023, April 26). *Parry Sound could see 40 cruise visits per season in coming years, says town’s economic development officer*. ParrySound.com. Retrieved February 26, 2024, from https://www.parrysound.com/news/parry-sound-could-see-40-cruise-visits-per-season-in-coming-years-says-towns-economic/article_4f05e416-8284-5827-892a-37a2ba60ea16.html

[2] Kelly, L. (2023, May 19). *Parry Sound prepping for record number of cruise visits*. Northern Ontario Business. Retrieved February 26, 2024, from https://www.northernontariobusiness.com/industry-news/tourism/parry-sound-prepping-for-record-number-of-cruise-ship-visits-7018623

Sections CD and EF were grouped as a single section after 1967 repairs saw the original timber substructure removed and replaced with Z-shaped steel-sheet piling. The new section boundaries are STA 0+048.3 to 0+117.8.

Timber sheet piling was installed at the Outer Section in 1931 around the original 1921/1922 substructure, and additional timber piles were driven. This section remains as STA 0+117.8 to 0+273.3.

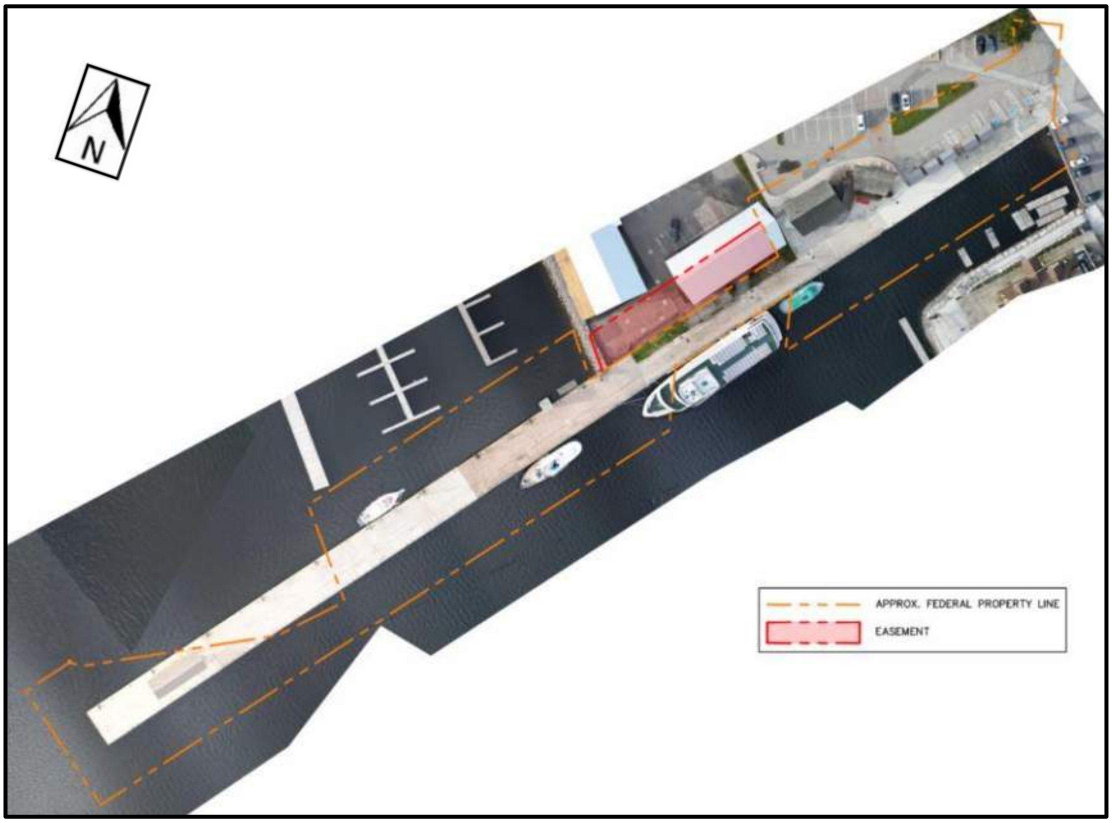
Table 1 summarizes the Bay St. Wharf (#401) structure types, construction history, and lengths of each structure investigated within the scope of work of this assignment.

Table 1. Summary of Structures

Stationing	Superstructure History	Substructure History	Approximate Length (m)
0+000.0 to 0+013.7 Structure A *(Formerly AB)*	►**1921/1922**: Timber decking. ►**1952**: Reinforced cast-in-place mass concrete slab and reinforced precast concrete footing blocks.	►**1921/1922**: Round timber pile bents with pile caps and stringers. ►**1952**: Stone-filled timber cribs.	13.7
0+013.7 to 0+048.3 Structure B *(Formerly AB)*	►**1921/1922**: Timber decking. ►**1952**: Reinforced cast-in-place mass concrete slab and reinforced precast concrete footing blocks.	►**1921/1922**: Round timber pile bents with pile caps and stringers.	34.6
0+048.3 to 0+117.8 Structure C *(Formerly CD & EF)*	►**1921/1922**: Timber decking. ►**1927/1928**: Reinforced cast-in-place concrete slab and reinforced precast concrete footing blocks. ►**1967**: New reinforced cast-in-place concrete slab and continuous reinforced concrete parapet.	►**1921/1922**: Round timber pile bents with pile caps, cross-bracing and stringers. ►**1931**: Additional timber piles driven. Timber sheet piling installed encapsulating 1921/1922 structure. ►**1967**: Z-shaped steel sheet piling. Removal of 1921/1922 structure.	69.5
0+117.8 to 0+273.3 Structure D *(Formerly Outer Section)*	►**1921/1922**: Timber decking. ►**1931**: Reinforced cast-in-place mass concrete slab and reinforced precast concrete footing blocks.	►**1921/1922**: Round timber pile bents with pile caps, cross-bracing and stringers. ►**1931**: Additional timber piles driven. Timber sheet piling installed encapsulating 1921/1922 structure.	155.5

A general aerial view identifying the four structures of the Bay St. Wharf (#401) is shown in **Figure 2**.

Figure 2. Aerial View



2.2 Additional Information

AECOM reviewed the following background data and reference information:

- Parry Sound Town Dock Proposed Alterations Plan (Public Works of Canada, 1921)
- Parry Sound Wharf Contract Plan (Public Works of Canda, 1922)
- Proposed Repairs to Town Wharf Plan (Public Works of Canada, 1928)
- Proposed Reconstruction of Outer 505 Lineal Feet of Wharf Plan (Public Works of Canada, 1931)
- Wharf Reconstruction Working Plan (Public Works of Canada, 1938)
- Wharf Reconstruction As Built Drawing (Public Works of Canada, 1952)
- Water Gauge Station Plan (Public Works of Canada, 1960)
- Repairs to Wharf Approach Drawing (Public Works of Canada, 1963)
- Harbour Repairs & Improvements, Wharf Reconstruction Plan (Public Works of Canada, 1967)
- Installation of Guard Rail Drawing (Public Works of Canada, 1968)
- Bay St. Wharf Harbour Inspection Report (MareTer Engineers, 1996)
- Engineering Investigation of Bay St. Wharf Report (Riggs Engineering, 2011)
- Parry Sound Harbour Inventory Asset Listing Drawing (Small Craft Harbours, 2012)
- Pile Bent Inspection of Bay St. Wharf Report (Riggs Engineering, 2014)
- Parry Sound Dock Inspection Report (Watech Services, 2023)

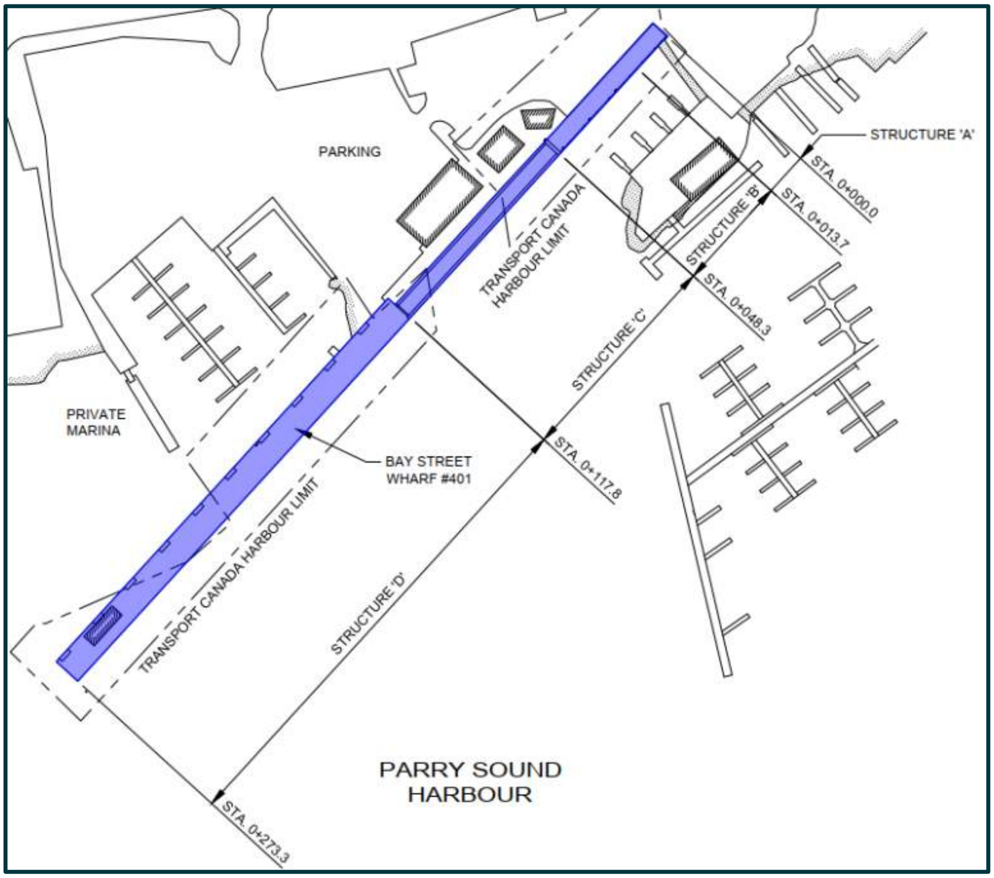
3. Inspection Methodology

3.1 General

The project team included staff from AECOM Canada Ltd (AECOM). General inspection of the above water components of the Bay St. Wharf (#401) was completed by Karol Chorostecki, P.Eng. and Aziz Younis, E.I.T. from the AECOM London office. Underwater Inspection was completed by Watech Services Inc (Watech).

For documentation of defects and details, reference baselines were established and temporarily marked with chalk along the deck. The reference stationing used during the inspection is shown in **Figure 3**.

Figure 3. Stationing – Bay St. Wharf (#401)



Station 0+000 of the Bay St. Wharf is located at the north end and progresses south, ending at 0+273.3. Structure A begins at station 0+000 and progresses south to 0+013.7. Structure B begins at station 0+013.7 and progresses south to 0+048.3. Structure C begins at station 0+048.3 on the and progresses south to 0+117.8. Structure D begins at station 0+117.8 and progresses south to the end of the wharf at 0+273.3.

3.2 Methodology

Existing information and documents were reviewed prior to the investigation to understand the site layout and composition of the structures. Base drawings of the facility were prepared with stationing to document conditions during the site visit.

The comprehensive site inspection was carried out in accordance with Section A3 of the “Guidelines for Inspection and Maintenance of Marine Facilities”, prepared by PWGSC and Transport Canada/DFO. The above water inspection consisted of a visual examination and written documentation of conditions of all the components of the Bay St. Wharf. A photographic record was undertaken of the Wharf.

A videographic scan of the above water concrete parapet of the wharf was performed using a GoPro mounted to a telescoping pole. The underwater inspection consisted of a four-person dive team undertaking a visual and tactile inspection. Equipment used in the underwater inspection included a 4.8 m dive vehicle, diving equipment, helmet-mounted LED lighting and an underwater camera system. Due to visibility and access limitations underwater, not all defects can be picked up with the underwater camera.

The nomenclature and classification of the element condition severity for material defects outlined in the Ontario Structural Inspection Manual (OSIM) was utilized for the inspection of components to ensure consistency in describing material defects. This approach will provide a good baseline of condition information that is repeatable and comparable for future investigations. Components have been rated ‘Excellent’, ‘Good’, ‘Fair’ and ‘Poor’ in accordance with OSIM methodology.

In addition to personal safety equipment, the tools utilized for the above water survey included measuring tapes, measuring wheel, hammers, sounding chain, cameras, flashlights, clipboards, and chalk.

3.3 Reference Documents

The following technical reference documents were applied to this assignment, as applicable:

- National Building Code of Canada (NBCC)
- PWGSC Guidelines Inspection and Maintenance of Marine Facilities
- Canada Occupational Health and Safety Regulations (SOR/86-304)
- Design of Concrete Structures (CSA-A23.3)
- Design of Steel Structures (CSA-S16)
- Engineering Design in Wood (CSA-O86)
- Maritime Works – Part 4: Code of Practice for Design of Fendering and Mooring Systems (BS 6349-4)
- Harbour Approach Channels Design Guidelines – Appendix C: Typical Ship Dimensions (PIANC Report No. 121-2014)
- Ontario Structural Inspection Manual (OSIM, Ontario Ministry of Transportation)

4. Description of Structures

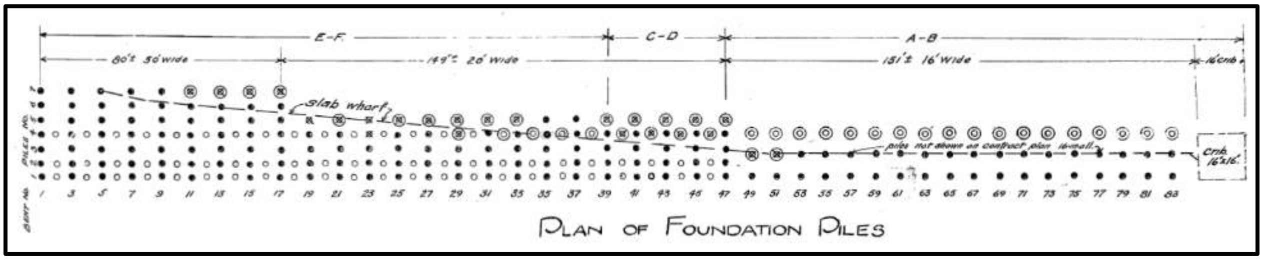
4.1 General

A general description of the Bay St. Wharf (#401) components investigated as part of this assignment at the Parry Sound Harbour facility is provided below. A general site plan is shown on **Figure A1** of **Appendix A**. The property line arrangement and general aerial view are shown on **Figure A2** of **Appendix A**.

The Bay St. Wharf (#401) is approximately 273.3 m long and is aligned in a north-south direction. The Wharf abuts to shore at the northern end. Starting from the north, a stretch of 126 m along the western edge of the wharf is also adjacent to land. The remaining length of the Wharf is open to water.

The original Bay St. Wharf structure was constructed in 1921/1922 above the existing town wharf, which was referred to as the Slab Wharf in record drawings. The Slab Wharf was built some time before 1921 and is buried under parts of the Bay St. Wharf sections AB, CD and EF (named Structure A, B and C, as part of this report). The original Bay St. Wharf consisted of timber decking above a round timber pile bent substructure, with some piles driven through the pre-existing Slab Wharf structure. The Slab Wharf location relative to the 1921 Bay St. Wharf timber piles can be seen in **Figure 4**.

Figure 4. Plan view of timber piles and pre-1921 Slab Wharf along STA 0+000 to 0+117.8 from 1921 Town Dock Proposed Alterations Plan, Public Works of Canada



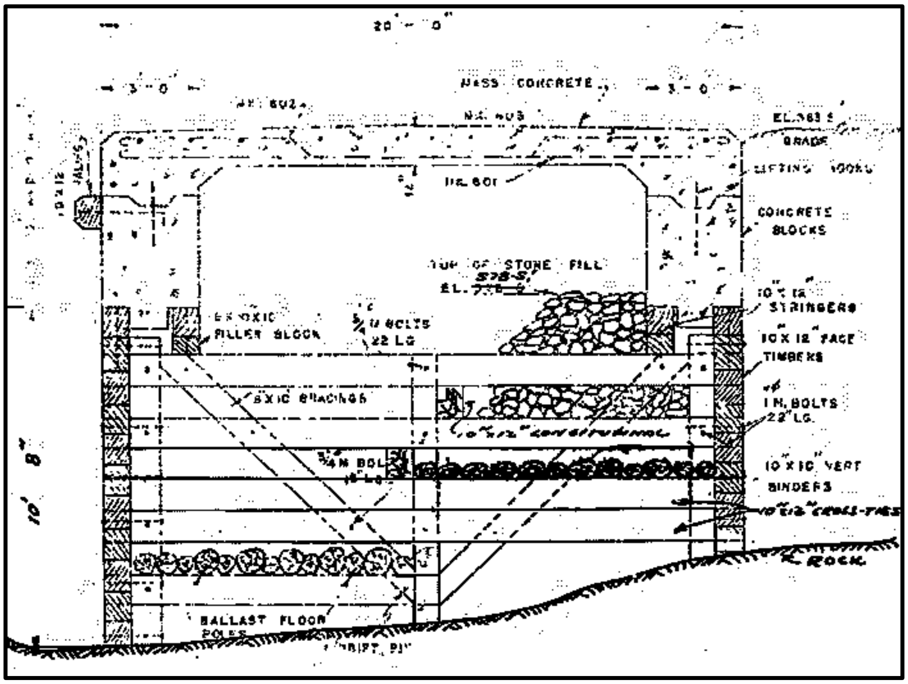
The original Bay St. Wharf underwent repairs and reconstructions at different times throughout the years after 1922, transforming the wharf into the arrangement present today. The following sections describe the structures as they are in present day.

4.2 Structure A, STA 0+000 to 0+013.7 – Stone-filled Timber Cribs (1952)

Structure A is approximately 13.7 m long and is the northmost component of the Bay St. Wharf. The structure was initially part of Section AB until reconstruction in 1952. During reconstruction, the substructure was converted into stone-filled timber cribwork. Additionally, the reconstruction replaced the original timber deck with a reinforced cast-in-place concrete deck supported by precast concrete footing blocks.

A typical cross-section of Structure A is provided in **Figure 5**.

Figure 5. Cross-section of Structure A from 1952 Wharf Reconstruction As-Built Drawing, Public Works of Canada



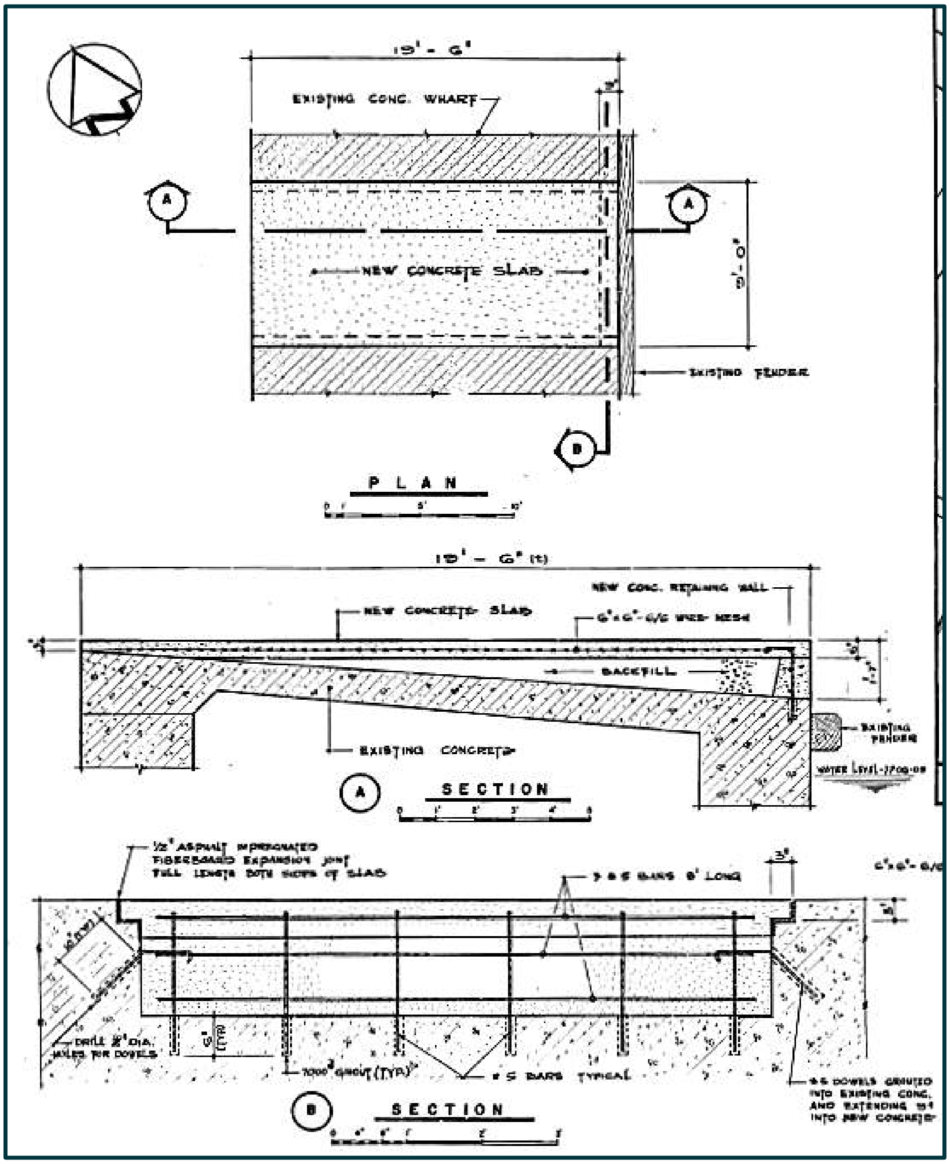
4.3 Structure B, STA 0+013.7 to 0+048.3 – Timber Pile Bents (1921/1922)

Structure B is located along STA 0+013.7 to 0+048.3 and is approximately 34.6 m long. The substructure maintains its original 1921 timber pile bents, with additional piles driven in 1928 at 1.22 m spacings. Between 1937 and 1938, a timber slipway ramp was added to the structure.

The timber deck was replaced in 1952 with a reinforced concrete slab supported by precast concrete footing blocks. The new concrete deck included a slipway ramp. The slipway ramp was filled in with concrete in 1977, although an investigation completed by Riggs Engineering Ltd (Riggs) in 2011 found that the repairs may not have been completed in accordance with the 1977 plan. Specifically, horizontal reinforcement was missing, and the vertical reinforcement does not appear to have been properly embedded. Riggs' 2011 report noted severely deteriorated concrete at this location.

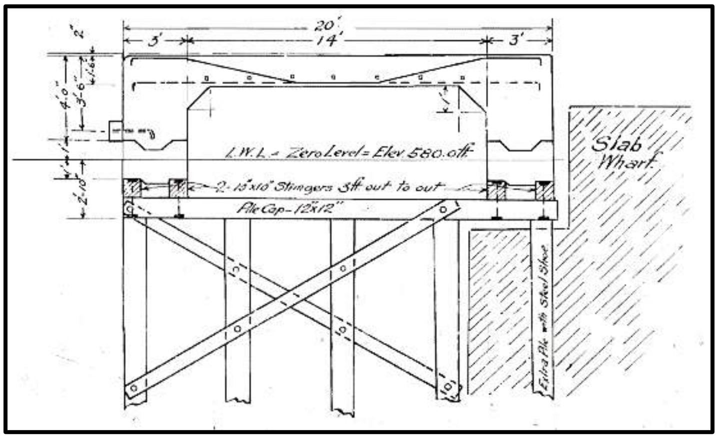
A cross-section of the slipway closure from the 1977 repair plan is provided in **Figure 6**.

Figure 6. Cross-section of Slipway Slab and Reinforcement from 1977 Slipway Closure Plan, Public Works of Canada



A typical cross-section of Structure B is provided in **Figure 7**. Although the cross-section is dated 1921, record drawings from 1938 and 1952 indicate the deck at this location was timber until its replacement with concrete in 1952. The 1952 cross-section for this area was not included in this report due to its illegible and faded condition.

Figure 7. Cross-section of Structure B from 1921 Town Dock Proposed Alterations Plan, Public Works of Canada



4.4 Structure C, STA 0+048.3 to 0+117.8 – Steel Sheet Piling (1967)

Structure C is located along STA 0+048.3 to 0+117.8 and is approximately 69.5 m long. Originally, this structure comprised Sections CD and EF during initial construction in 1921/1922. Following fire damage in 1927/1928, the timber deck at this location was replaced with a reinforced cast-in-place concrete deck supported by precast concrete footing blocks.

In 1967, the 1928 concrete deck was removed to allow installation of Z-shaped steel sheet piling. A new concrete deck was constructed with a continuous reinforced concrete parapet in place of the previous precast blocks.

A typical cross-section of Structure C is provided in **Figure 8**.

Two steel ladders were installed during the 1967 reconstruction. These ladders consist of steel pipes embedded across a recess formed in the concrete section. The recess aligns with the outline of the steel sheet piling, and six rungs are welded to the steel sheet piling at each ladder location.

Details and cross-sections of the ladders at this location are provided in **Figure 9**.

Figure 8. Cross-section of Structure C from 1967 Wharf Reconstruction Plan, Public Works of Canada

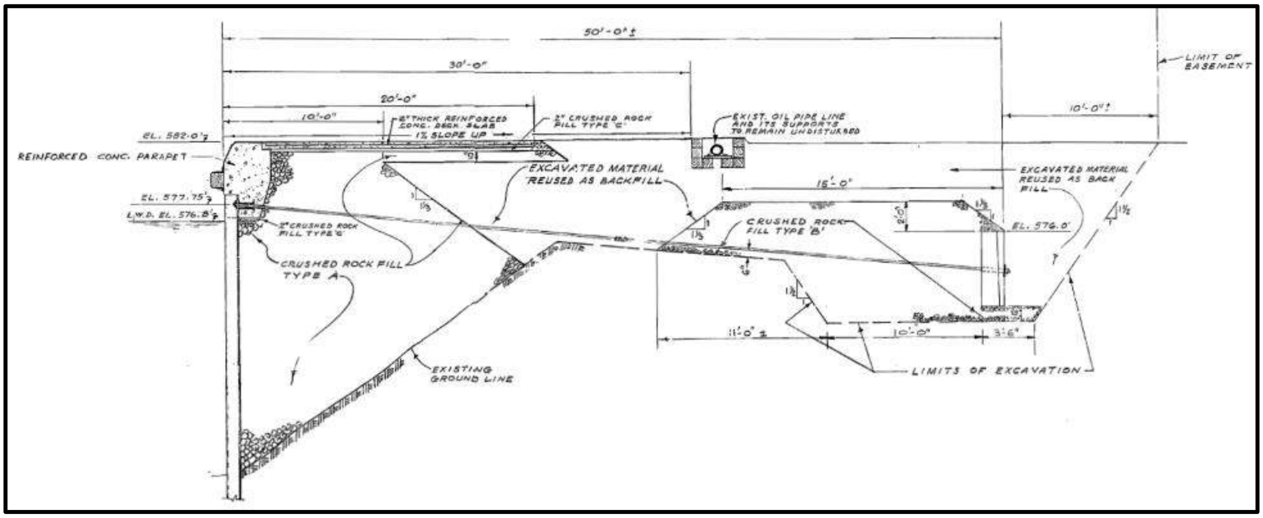
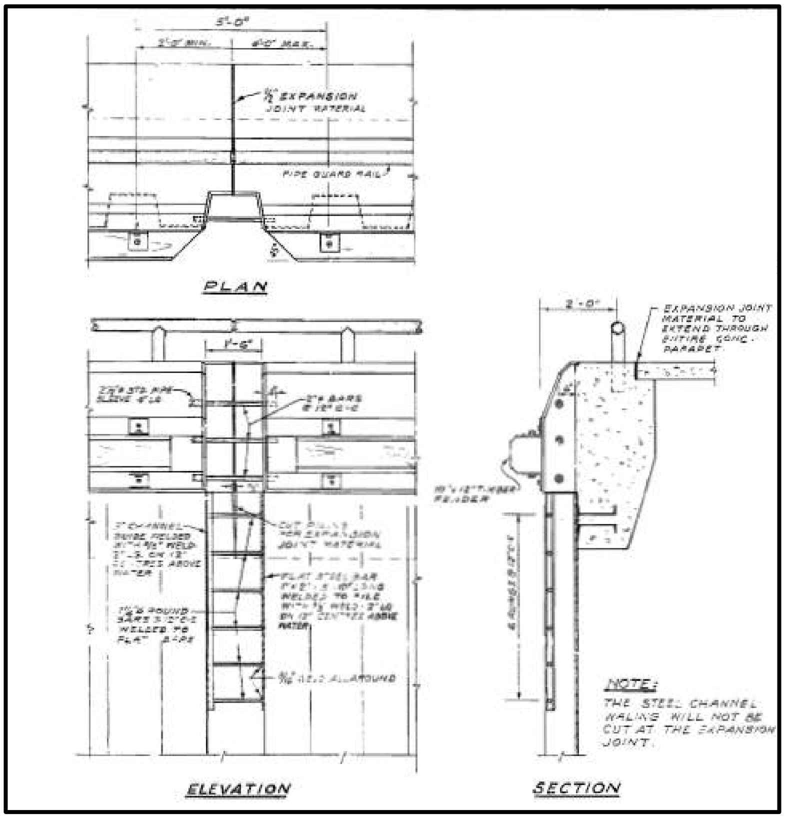


Figure 9. Details of Structure C Ladders from 1967 Wharf Reconstruction Plan, Public Works of Canada



4.5 Structure D, STA 0+117.8 to 0+273.3 – Timber Sheet Piling (1931)

Structure D is the largest component of the Bay St. Wharf, spanning approximately 155.5 m from STA 0+117.8 to 0+273.3. This section is referred to as the Outer Section in record drawings and was part of the original 1921/1922 construction.

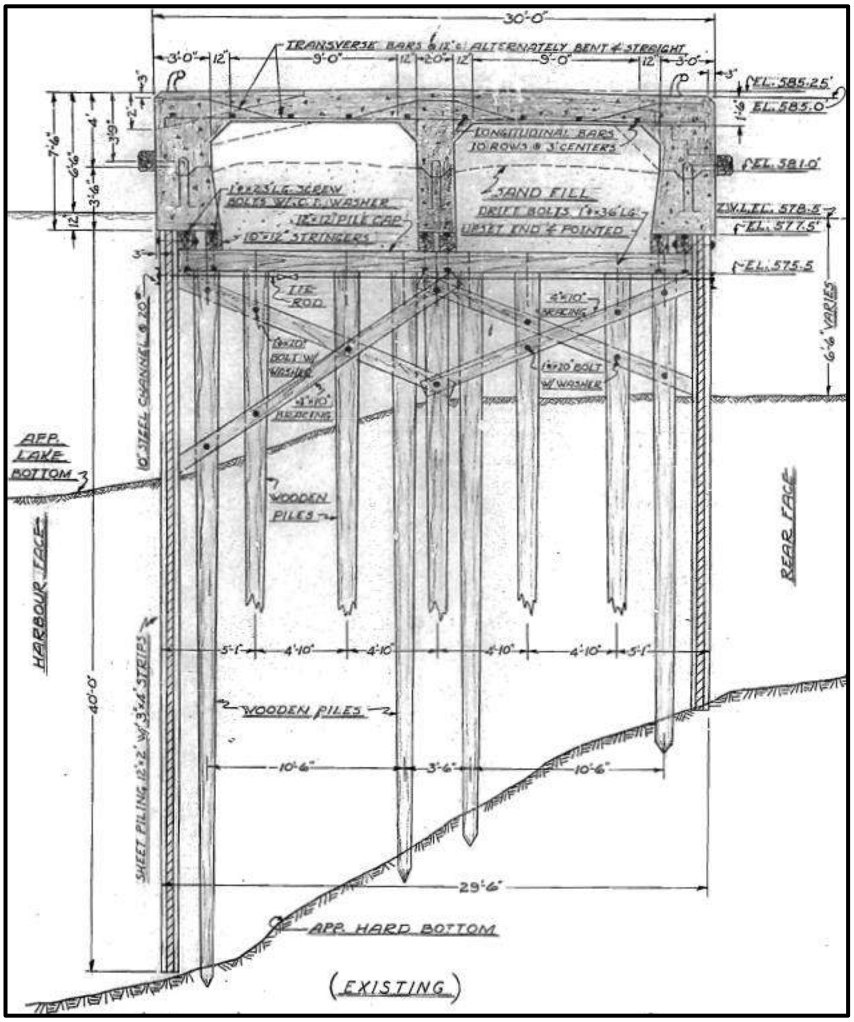
In 1931, the original timber substructure was encapsulated with timber sheet piling. All original timber piles along the outer perimeter of Structure D were removed to accommodate placement of the timber sheet piling. New timber piles were driven at a distance of 0.76 m from the timber sheet face as detailed in the 1931 reconstruction drawings.

Additional timber piles were also driven in pairs along the centerline of the structure, with additional stringers also installed. Reinforced precast concrete blocks were placed along the outer perimeter and centerline of the structure, and a reinforced concrete deck was poured above. The concrete deck included several 3.5 m wide descending stairs on the west side, and descending slipway ramps on the east side. The slipway ramps were later filled in with concrete, although the date of this construction is not clear.

In 1962, a water gauge station was installed overhanging the west edge of the deck at STA 0+143. A section of concrete measuring roughly 0.46 m wide by 2.15 m long was chipped out of the concrete deck at this location. Existing rebar was left intact to be incorporated into the water gauge station deck. The overhang is supported by two steel H-piles.

A typical cross-section of Structure D is provided in **Figure 10**.

Figure 10. Cross-section of Structure D from 1960 Water Gauge Station Plan, Public Works of Canada



5. Existing Conditions and Observations

5.1 General

General inspection of the facility was completed by AECOM on November 16, 2023. The weather condition was sunny, with a temperature of 12ºC. Additional observations were obtained on November 17, 2023. Watech and AECOM collected GoPro footage on December 7 and 8, 2023, from holes cored into the concrete deck. Diving inspections were conducted on December 7, 2023. ROV and additional diving inspections were completed on December 20 and 21, 2023.

This section summarizes the conditions observed during the above water field investigation and underwater inspection. Photographs of the field investigation are included in **Appendix B**, underwater inspection report is included in **Appendix C** and detailed inspection sheets are included in **Appendix D**.

5.2 Structure A, STA 0+000 to 0+013.7 – Stone-filled Timber Cribs (1952)

5.2.1 General

The first section of the Bay St. Wharf, identified as STA 0+000 to 0+013.7, measured approximately 6.06 m in width. The concrete deck at this location was found to be in fair condition. General observations noted light-to-medium scaling, efflorescence-stained cracking and delaminations on the concrete edge. A large area of narrow map cracking was found near STA 0+000, along with a severe spall at the eastern edge. A medium-to-wide transverse crack running approximately 75% the width of the deck was observed further along the section. Between STA 0+005 to 0+010, a large patch area with hairline map cracking was observed. A large, patched area was observed around the small bollard located between STA 0+010 to 0+013.7. Vegetation growth was noted at the control joint at STA 0+013.7.

5.2.2 Below Water Review

The water along this section of the wharf was quite shallow, limiting visibility below the water level. The concrete cope wall was in fair condition with efflorescence-stained cracking, localized spalling and delamination. Some shifting was noted on the underlying precast concrete blocks, along with medium scaling and spalling.

The condition of the timber cribs could not be assessed due to their position below the lakebed.

5.3 Structure B, STA 0+013.7 to 0+048.3 – Timber Pile Bents (1921/1922)

5.3.1 General

The second section of the Bay St. Wharf, identified as STA 0+013.7 to 0+048.3, measured approximately 6.06 m in width. The concrete deck at this location was found to be in fair condition, with localized areas in poor condition. General observations noted a large section of newer concrete between STA 0+026 to 0+048.3, various severe delaminations, light to medium scaling throughout, and cracks of varying severity. Coring through one location at this section indicated that the concrete deck was approximately 356 mm thick.

Within STA 0+013.7 to 0+025, localized patch locations were identified including a large patch around the bollard at STA 0+023. Two narrow to medium cracks were noted around STA 0+020, with some patchwork on each.

Four localized areas of medium to severe delaminations were observed at the transition to the newer concrete section around STA 0+026. The section of newer concrete ends around STA 0+048.3. Hairline to narrow pattern cracking was observed throughout the newer concrete, likely caused by light to medium alkali-aggregate reaction (AAR) cracking.

Between STA 0+030 and 0+035, a very severe spall measuring 0.4 x 1.0 m was observed at the eastern edge and a narrow to medium transverse crack around STA 0+031. The delaminated area around the spall is adjacent to a plastic 4-step ladder installed approximately at STA 0+032. This ladder is one of three along the eastern side of the Wharf.

Two very severe localized delaminations were noted between STA 0+035 and 0+040. A narrow transverse crack was also noted in the area around STA 0+037. Control joints are cut into the concrete at both STA 0+035 and 0+040. A bollard is located at the east edge at STA 0+036. Map cracking, likely caused by AAR, was identified around the bollard.

Between STA 0+040 to 0+048.3, the inspection identified three very severe delaminations ranging in size from 0.65 m x 0.5 m to 2.5 m x 0.75 m. A medium to wide crack was observed around STA 0+047. A control joint at STA 0+048.3 marks the end of the second section of the wharf. A very severe spall with exposed rebar was noted at the east face of STA 0+048.

5.3.2 Below Water Review

The internal timber structure could not be accessed by the diver or ROV due to the high level of fill underneath the deck. Watech noted the top of the timber piles to be consistently below water level, and the exterior timber structure was noted to be generally sound due to the lack of exposure to air.

Deterioration of the precast blocks along this section was less severe as compared to the precast blocks at the outer section (Structure D), likely due to the reduced wave action at the inner portion of the wharf. However, an area of serious concern was observed at the location of the filled-in concrete slipway at STA 0+39.5. The concrete slipway was filled in 1977, and the concrete wall at this location was reported to be severely deteriorated and spalled with exposed vertical reinforcement by Riggs Engineering in 2011. The present inspection found that further deterioration has occurred since 2011, with horizontal reinforcing steel bars being exposed and an outflow of fill material washing out of the structure.

To determine the cause of the washout, a core was taken through the deck at STA 0+039.5 for GoPro access. GoPro footage found the concrete at the outflow section to be fully eroded below water, with only vertical reinforcement remaining. A plastic pipe was discovered beneath the deck with its outfall directed towards the eroded section of concrete block. The pipe has likely contributed to the advanced deterioration of the block. The pipe is presumed to be still operational, as the divers observed water and additional material flowing from the pipe when firefighters were addressing a nearby fire.

Additional observations obtained from the GoPro footage at this location noted timber at the underside of the concrete deck. This timber is either the original deck or discarded formwork. The thickness of the concrete deck at the location of the core measured about 350 mm, which is 100 mm thinner than shown in the record drawing.

Timber penetration depth measurements of the timber piles at Structure B were taken at various locations along the length. Three readings were collected at each location by the diver, and the average was recorded. Marine growth was removed as necessary to obtain true results. Penetration measurements of the timber piles indicated an average penetration of 2.0 mm, with a maximum of 2.2 mm and a minimum of 1.8 mm.

5.4 Structure C, STA 0+048.3 to 0+117.8 – Steel Sheet Piling (1967)

5.4.1 General

The third section of the Bay St. Wharf, identified as STA 0+048.3 to 0+117.8, measured approximately 6.06 m in width. Curb railings begin at this section and continue throughout the east side of the wharf. Control joints running longitudinally between the concrete deck and east wall fascia are present only at this section. The Island Queen V (40.2 m long cruise ship) docks between STA 0+085 and 0+125. Curb railings have been removed from STA 0+087 to 0+093 presumably for passenger access to the Island Queen. Timber fenders are installed along the parapet from STA 0+051 to 0+110.

The concrete deck was found to be in fair to good condition. General observations highlighted two areas of settlement, settlement-induced cracking, and multiple areas of severe delaminations. Localized areas of light scaling were present throughout this section of the wharf. Town staff informed AECOM staff of repair work undertaken in recent years to mitigate the settlement issue. The repair work, totalling around $20,000, involved the injection of spray foam between STA 0+050 to 0+063 to address the settlement issue.

Severe settlement of the concrete deck was noted at STA 0+048.3 to 0+055. Two transverse and one longitudinal crack originate from the settled area. The transverse cracks are joined and span from the west edge of the concrete deck to the transverse control joint at 0+050. The longitudinal crack joins with the transverse cracks and extends to the location of a buried conduit at STA 0+069. Three areas of severe delaminations were noted around the settled area, and vegetation growth was noted at the transverse control joint at STA 0+050. Vegetation growth was also noted along the longitudinal fascia control joint. At STA 0+055, a buried conduit was noted with a drain on the west side of the deck.

Between STA 0+055 to 0+085, a narrow to medium transverse crack was identified near STA 0+056 and buried conduits were located at STA 0+064 and 0+069. East bollards at STA 0+057 and 0+076 appeared to be in good condition, with some hairline cracking observed at the STA 0+076 bollard. Concrete and patchwork over both conduits exhibited medium to wide cracking and severe delamination. Two areas of medium delamination were noted towards the west side of the deck at STA 0+070. Steel ladder rungs cast into a blocked-out section of the concrete parapet were located on the eastern side of the wharf at STA 0+069. This ladder is one of two detailed in the 1967 reconstruction drawings. The presence of a continuous timber fender across the ladder opening suggests that the ladder is currently not in use. The reason for its disuse could not be determined during the inspection. The timber fender is discontinuous at the second ladder, located at STA 0+097. The ladders looked in good condition above water, but the condition of the underwater portion of the ladder could not be determined.

A second area of settlement in the concrete deck was identified spanning from STA 0+085 to the transition at STA 0+117.8. The settlement has induced wide transverse and longitudinal cracks, the latter extending to STA 0+117.8. Two additional medium to wide transverse cracks were identified at STA 0+107 and 0+113. At the western side of the deck, a very severe delamination measuring 1.0 m x 0.7 m was noted at STA 0+085. The east bollards at STA 0+094 and 0+113 appeared to be in good condition. A buried conduit with delaminated cover was identified at STA 0+097.

One core was taken at the center of the deck at STA 0+113.0. Sand fill was present to the underside of concrete at the core location with no voids noted.

The timber fenders were in fair to poor condition with cracking, splitting, rotting and disintegration.

5.4.2 Below Water Review

Considerable marine growth was noted on the steel sheet piles which caused difficulties for visual inspection and assessment of overall condition.

Ultrasonic thickness measurements of the steel sheet pile substructure were taken at 2 m to 3 m intervals along Structure C. Marine growth was removed as needed to provide better accuracy for readings. Measurements could not be made along a 50 m stretch of the steel sheet pile due obstructions from large vessels preventing safe access for the diver. Thickness measurements of the steel sheet pile indicated an average steel thickness of 7.6 mm, with a maximum thickness of 8.9 mm and a minimum thickness of 5.4 mm. Over 90% of measurements recorded indicated thicknesses 6.2 and 8.9 mm.

5.5 Structure D, STA 0+117.8 to 0+273.3 – Timber Sheet Piling (1931)

5.5.1 General

The fourth section of the Bay St. Wharf, identified as STA 0+117.8 to 0+273.3, measured approximately 9.0 m in width. Cores taken through this section indicated the concrete deck to be approximately 508 mm. Curb railings continue at this section throughout the east and south sides of the wharf. Small lengths measuring about 200 mm of the curb railings were found to have been removed at STA 0+125 and 0+131. The reasoning behind this removal was not clear at the time of inspection.

The concrete deck at this location was found to be in fair condition. General observations noted multiple medium to wide cracks running transversely across the deck, medium to severe scaling throughout, and several instances of severe delamination especially at the east bollards. The concrete spalling and delamination around the bollards signify potential structural compromise in those areas. Another notable observation was the presence of a very wide transverse crack extending the entire width of the concrete deck at STA 0+270. The crack was found to terminate at the joints between the precast blocks on both sides of the wharf. The pattern of the crack suggests potential settlement issues.

Widespread hairline map cracking indicating light AAR was noted at the west side of the deck between STA 0+117.8 to 0+126. This concrete section appears to have been cast more recently compared to the sections adjacent STA 0+117.8 and 0+126. Along the boundary at STA 0+126, an area of severe delamination measuring 2.4 m x 0.1 m was observed.

Continuing past STA 0+126, medium to severe scaling and localized patch repairs were found to be typical throughout this area continuing to STA 0+273.3. An area of very severe spalling was found at the east concrete face of the east bollard at STA 0+129. The bollard plate and two studs are exposed due to the spall. A small section of the east curb rail was noted to have been cut and removed at STA 0+131 without clear purpose. An expansion joint with vegetation growth was noted at STA 0+136. The first set of stairs on the west side are present between STA 0+131 and 0+134. Light spalling was found on the stairs. The first of six buried slipways on the east side was found between STA 0+139 and 0+141.

Along the west side, the first of five west bollards was noted at STA 0+141. Vegetation growth was found in a longitudinal crack between the bollard and the west edge. Constructed in the 1960s, the water gauge station is also located along the west edge at STA 0+144. Areas of delamination and patch repairs were found in the concrete at the construction joint between the deck and the water gauge slab. Several narrow transverse cracks were noted between STA 0+142 and 0+150. The east bollard positioned at 0+144 appeared to be in good condition.

The second set of stairs was observed on the west side between STA 0+146 and 0+150, with narrow to medium cracking and severe delamination of 0.5 x 1.0 m on the lower step.

Vegetation growth was noted in the expansion joint along STA 0+151. At the east face, severe delamination was observed at 0+160 and medium delamination at STA 0+164. Severe delamination was also present at the concrete face by the east bollard at STA 0+159. The second of six buried slipways on the east side was found between STA 0+154 and 0+156, with a severe delamination of 0.6 x 0.2 m adjacent to its southeast corner. A control joint was observed at STA 0+161 and severe scaling noted at the east parapet between STA 0+160 and 0+161.

The third set of stairs are located on the west side between STA 0+161 to 0+164. Concrete disintegration showing exposed reinforcing steel bar was observed at the north corner of the stairs. The north set of railings were noted to be bent and deformed. Previous concrete patches were noted on the stairs.

From STA 0+165 to 0+190, expansion joints were noted at STA 0+167 and 0+182, with vegetation growth observed in the joints. Control joints were observed at STA 0+170, 0+175, and 0+185. The third buried slipway on the east side was located between STA 0+169 to 0+172. The second west bollard was found at STA 0+170. Severe scaling was noted at the east face between STA 0+167 and 0+169. A narrow transverse crack with patchwork was observed at STA 0+172. Severe scaling was noted at the east face between STA 0+175 and 0+178. The curb railing at STA 0+181 was found to be deformed, possibly due to an impact or collision. Severe delamination and spalling of the east face were observed at the east bollard at STA 0+175. More severe spalling was observed at another east bollard by STA 0+190, with the bollard plate exposed by spalled concrete. The fourth buried slipway on the east side was located between STA 0+185 to 0+188.

The fourth set of stairs was present on the west side between STA 0+176 and 0+180. The south railings were noted to be missing a post, and a wide transverse crack was found extending halfway across the deck from the north corner of the stair recess.

From STA 0+190 to 0+215, expansion joints were noted at STA 0+197 and 0+213, and one control joint was noted at STA 0+192. Medium delamination measuring 0.3 x 0.3 m was observed towards the middle of the deck at STA 0+192. Two medium to wide transverse cracks were observed at STA 0+203 and 0+207. Severe scaling was noted on the east side between STA 0+211 and 0+212. The third west bollard was located at STA 0+201. Light cracking was observed around the concrete adjacent to the east bollard at STA 0+206. The fifth buried slipway on the east side was located between STA 0+200 and 0+203. A medium to wide transverse crack extending between the south corner of the buried slipway and the west edge of the deck was identified at STA 0+203. Another medium to wide crack was identified at STA 0+207.

The fifth set of stairs was present on the west side between STA 0+192 and 0+196. Medium to severe scaling was noted on the stairs along with a large aggregate popout. The north corner of the stairs aligns with the control joint at STA 0+192.

The sixth set of stairs was present on the west side between STA 0+207 and 0+211. The medium to wide crack noted at STA 0+207 continues along the north edge of the stairs. Vegetation growth was noted in the crack, between the north edge of the stairs and the vertical wall.

From STA 0+215 to 0+240, one expansion joint was noted at STA 0+229. Vegetation growth and a light spall were observed at the west end of the joint. Narrow transverse cracks were observed at the west side of the deck at STA 0+221, and the east side at STA 0+223. Severe delamination was observed at the east face of the concrete by the bollard at STA 0+221. Severe scaling was observed at the east face at STA 0+224. Medium to severe scaling was also observed at the east side of the deck at STA 0+231, with a narrow transverse crack also noted at this location. A very severe delamination measuring 1.2 x 1.4 m was observed at the center of the deck at STA 0+228, with disintegrating patchwork on top. Another area of very severe delamination was observed at the west side at STA 0+235, and measured to be 0.7 x 1.0 m. The fourth west bollard was identified at STA 0+232, with a severe spall at the west face at this location. Light cracking was observed around the concrete adjacent to the east bollard at STA 0+236. The last of the six buried slipways on the east side was located between STA 0+216 and 0+218.

The seventh set of stairs was present on the west side between STA 0+222 and 0+226. Medium scaling was noted on the stairs.

The eighth set of stairs was present on the west side between STA 0+238 and 0+242. Two narrow cracks were observed at the north corner. Light localized scaling was noted on the lowest step.

From STA 0+240 to 0+273.3, expansion joints were noted at STA 0+244 and 0+259. Reconstructed in 2016, the Town Pavilion is situated between STA 0+246 and 0+258. Medium to wide transverse cracks were observed at STA 0+248 and medium transverse cracks were observed at STA 0+251. Severe delamination was noted on the vertical concrete face at both east bollards at STA 0+251 and 0+266.

The ninth set of stairs was present on the west side between STA 0+253 and 0+256. A short narrow crack was noted on the north corner. A medium crack extending through the stairs towards the center of the deck was also noted at STA 0+266. Medium to severe scaling was noted on the lowest step.

Severe scaling was present on the west side edge between STA 0+260 and 0+262. The final west bollard was located at STA 0+264, with severe scaling noted south of the bollard to STA 0+265. Medium delamination measuring 0.3 x 0.2 m was observed towards the center of the deck at STA 0+265. Most concerning in this section was the presence of a very wide transverse crack extending across the deck was noted by the west stairs at STA 0+271. Medium scaling was observed at the west section of the south edge of the wharf.

The tenth and final set of stairs was present on the west side between STA 0+268 to 0+272. Severe scaling was observed at the stairs. The very wide crack noted at STA 0+271 extends from the joint between the precast blocks below the stairs, through the stairs, and transversely across most of the deck. Patchwork completed on the deck has also developed fractures aligned with the crack. The severity of the crack suggests prior or potentially ongoing settlement or movement of the deck. Noting photographs in the 2011 Riggs report, in addition to SCH photographs from 2012, the crack appears to have grown, suggesting some of the settlement occurred in the last 10 years.

5.5.2 Below Water Review

Below water inspection of the substructure by the dive team and ROV found the exterior wharf face to consist of driven 350 x 350 mm square timber piles with additional timber pieces used to close the gap between square timber piles. A steel channel waler with 40 mm diameter tie rods spaced at 2.5 m to 3.0 m were noted at the top of the wharf face.

The tops of the timber piles were typically below the top of the water during the inspection. Since the piles are not exposed to air, they were found to generally be in good condition. The fill pieces were observed to be rotted and split. Timber penetration depth measurements of the timber piles at Structure D were taken at various locations, roughly every 5 m along the structure. Three readings were collected at each location by the diver, and the average was recorded. At STA 0+150, penetration was up to 51 mm due to splits in the timber pile. At STA 0+239, timbers were split all the way through. Besides STA 0+150 and 0+239, penetration measurements of the timber piles indicated an average penetration of 3.8 mm, with a maximum of 12.7 mm and a minimum of 1.3 mm. The tie rods were observed to be in fair to good condition with minor surface corrosion. The steel channel walers were observed to be in fair condition with corrosion and steel pitting. The connections were observed to be intact and in fair to good condition.

Observations of above water portions seen by the diving team and ROV noted the cast-in-place concrete copewalls to be in poor to fair condition, with numerous severe cracks and spalls throughout these components. Spalling and severe scaling was also noted to be typical on the precast concrete blocks beneath the copewalls, especially along the splash zone.

The ”I” beam at the water level gauge station was observed to have significant steel section loss and pitting.

An internal inspection was carried out using an ROV however, high backfill levels inside of the structure limited the effectiveness of movement throughout the interior structure and did not allow for confirmation of the pile structure. A total of seven cores were taken through the concrete deck at Structure D to provide access for the GoPro camera. Concrete deck thickness was measured to be approximately 508 mm at all core locations which is consistent with record drawings. Loss of sand fill was noted in all core locations, with most cores measuring fill level to be 2.2 m below top of concrete deck. This measurement corresponds to a void of about 1.7 m below the underside of the concrete deck which is significantly larger than the 0.6 m void shown in the original 1931 drawings.

Observations from core holes are presented in **Table 2**.

Table 2. Findings from Coring Concrete Deck at Structure D

Location: STA 0+129.7

Transverse Distance to Nearest Edge: 1.65 m to East

Concrete Deck Thickness: ± 508 mm

Fill Level to Top of Concrete: ± 1.5 m

Observations: Severe void due to deteriorating concrete on parapet.

Location: STA 0+198.6

Transverse Distance to Nearest Edge: 1.59 m to West

Concrete Deck Thickness: ± 508 mm

Fill Level to Top of Concrete: ± 2.2 m

Observations: Underside of south corner of buried slipway at 0+200.

Location: STA 0+228.6

Transverse Distance to Nearest Edge: 3.04 m to West

Concrete Deck Thickness: ± 508 mm

Fill Level to Top of Concrete: ± 2.2 m

Observations: Unidentified timber debris above fill.

Location: STA 0+248.2

Transverse Distance to Nearest Edge: 2.31 m to East

Concrete Deck Thickness: ± 508 mm

Fill Level to Top of Concrete: ± 2.2 m

Observations: Efflorescence, underside of medium crack at 0+251.

Location: STA 0+258.5

Transverse Distance to Nearest Edge: 2.71 m to West

Concrete Deck Thickness: ± 508 mm

Fill Level to Top of Concrete: ± 2.4 m

Observations: Top of Pile Cap.

Location: STA 0+261.7

Transverse Distance to Nearest Edge: 2.31 m to West

Concrete Deck Thickness: ± 508 mm

Fill Level to Top of Concrete: ± 2.4 m

Observations: Very wide crack at 0+271 noted in above water inspection. Visibility here indicates crack penetrates all the way through the concrete deck

Location: STA 0+261.7

Transverse Distance to Nearest Edge: 2.31 m to West

Concrete Deck Thickness: ± 508 mm

Fill Level to Top of Concrete: ± 2.4 m

Observations: Very wide crack at 0+271 noted in above water inspection. Also visible is the transition in haunch dimensions from stairs to copewall.

Location: STA 0+263.4

Transverse Distance to Nearest Edge: 2.44 m to West

Concrete Deck Thickness: ± 508 mm

Fill Level to Top of Concrete: ± 2.2 m

Observations: Underside of deck, typical.

6. Useful Residual Life

The remaining useful residual life (URL) is an asset management tool for assessing the estimated time, in years, that the asset is expected to continue serving its intended function. The URL is helpful when assessing rehabilitation versus replacement alternatives, and timing needs for a facility.

The empirical method published by Public Works Canada and Transport Canada "Guidelines for Inspection and Maintenance of Marine Facilities" (1985) was used to determine the URL for components of the Bay St. Wharf (#401) at Parry Sound Harbour. The assessment utilizes the theoretical useful life (TUL) with subjectively applied weighting coefficient (WC) related to environmental site conditions and a compensating factor (CF) related to actual physical condition. The actual age (AA) of the structure is later subtracted from the TUL to determine the URL.

The empirical formula for calculating URL is as follows:

URL = [TUL x (100 – WC) x CF] – AA

Where:

WC = 0 to 30 (dependent on use, exposure, ice, waves, foundations, etc.)
CF= 0.7 to 1.0 (dependent on structure condition)
AA= actual age of the structure in years

The useful residual life of the Bay St. Wharf (#401) is provided in **Table 3** and based on an age as of 2024. Values for the various parameters and calculations are provided in **Appendix E**.

Even with favourable conditions with weighting coefficients and compensating factors, a theoretical URL value of zero may be calculated based on the age and theoretical useful life of a component. However, this does not necessitate immediate replacement. In most cases, planning for the component rehabilitation or replacement may be initiated while the life of a component is extended and monitored.

Table 3. Useful Residual Life Values

Component	Year Constructed	TUL (yrs)	AA (yrs)	URL (yrs)	Comments*
Structure A (0+000 – 0+013.7)					
Timber Crib (Substructure)	1952	40	72	0	Actual Age of structure exceeds its TUL.
Concrete Deck (Superstructure)	1952	60	72	0	Actual Age of structure exceeds its TUL.
Structure B (0+013.7 – 0+048.3)					
Timber Pile (Substructure)	1922	40	102	0	Actual Age of structure exceeds its TUL.
Concrete Deck (Superstructure)	1952	60	72	0	Actual Age of structure exceeds its TUL.
Structure C (0+048.3 – 0+117.8)					
Steel Sheet Pile (Substructure)	1967	80	57	23	
Concrete Deck (Superstructure)	1967	60	57	0	
Structure D (0+117.8 – 0+273.3)					
Timber Pile (Substructure)	1931	40	93	0	Actual Age of structure exceeds its TUL.
Concrete Deck (Superstructure)	1931	60	93	0	Actual Age of structure exceeds its TUL.

**See additional notes below.*

Additional discussion is provided for components where the URL has been calculated to be zero, or near zero.

- **Timber crib substructures**: The timber crib of Structure A has exceeded its TUL. However, given the right conditions, it is recognized that timber can have significantly longer service lives. Given the structure condition and performance, structure competency cannot be relied on indefinitely. A safe service life of 10 to 15 years is expected with routine inspection. Replacement/encapsulation of the timber crib structure is recommended in the long term (11 to 15-year time period).
- **Timber pile substructures**: The timber pile substructure of Structure B and D have greatly exceeded their TUL. However, given the right conditions, it is recognized that timber can have significantly longer service lives. Given the structure condition and performance, structure competency cannot be relied on indefinitely. Replacement/encapsulation of the timber pile structure of Structure B is recommended in the medium term (6 to 10-year time period), while replacement/encapsulation of the timber crib structure of Structure D is recommended in the long term (11 to 15-year time frame).
- **Concrete Deck superstructure**: The concrete deck of Structures A, B and D have exceeded their TUL. While there are a number of observed defects, the concrete deck tops have been maintained with periodic repairs. Replacement of the concrete superstructures need not occur until replacement of the timber substructures is warranted.
- **Concrete Deck superstructure**: The concrete deck of Structures C has an actual age that is nearing its TUL, with a calculated URL of zero. While there are observed defects, the concrete deck top has been maintained with periodic repairs. Replacement of the concrete superstructure need not occur at this time.

7. Load Evaluation Assessment

This section presents the results of the load evaluation assessment conducted for the two critical sections of the Wharf at Structure C (STA 0+048.3 to 0+177.8) and Structure D (STA 0+117.8 to 0+273.3). The objective of the assessment was to confirm if Structure C has adequate capacity for the Island Queen Cruise ship, and determine the largest vessel that should be permitted to use Structure D.

The capacity of the bollards in both current and repaired conditions was calculated to determine which vessels can safely moor using the bollards. The steel sheet piling at Structure C was evaluated to determine if the Island Queen Cruise ship can continue to dock to the structure. The timber substructure at Structure D was modelled using finite-element analysis (FEA) software to determine the largest berthing force that can be resisted by the structure.

7.1 Material and Vessel Properties

7.1.1 Material Properties

No material strength properties were provided on original drawings of the Wharf. **Table 4** summarizes the material strength properties assumed in the various parts of this evaluation.

Table 4. Material Strength Properties

Material Property	Strength	Reference
Concrete Compressive Strength (f'_C) – *Deck*	20 MPa	CSA S6-19 14.7.4.3
Steel Yield Strength (f_{ya}) – *Bollard Anchors*	210 MPa	CSA S6-19 14.7.4.2, Table 14.1
Timber Pile Bending Strength (f_b) – *Timber Piles*	20.1 MPa	CSA O86-19 Table 13.1, D.-Fir
Timber Pile Shear Strength (f_v) – *Timber Piles*	1.4 MPa	CSA O86-19 Table 13.1, D.-Fir
Timber Pile Compressive Strength (f_c) – *Timber Piles*	18.7 MPa	CSA O86-19 Table 13.1, D.-Fir

Table 5 summarizes material weight properties used in the various parts of this evaluation.

Table 5. Material Weight Properties

Material	Weight (kN/m^3)	Reference
Reinforced Concrete	24.0	CHBDC S6-19 Table 3.4
Douglas Fir (softwood)	6.0	CHBDC S6-19 Table 3.4
Clear Stone	17.0	Assumed value
Sand & Silt	17.5	Assumed value

7.1.2 Section Properties

Based on the condition of timber assessed during the inspection, a reduction of cross-sectional properties was applied to the timber components in the finite-element model. Cross-sections were reduced by 20% along their local x- and y-axes.

7.1.3 Vessel Properties

Cruise ships which had visited Parry Sound in 2023 were used in the load evaluation and structural analysis. **Table 6** summarizes assumed dimensions and mass of each cruise vessel included in the calculations. Information regarding ship size and Gross Tonnage (GT) for each ship was obtained as available from either national vessel registers or maritime analytics databases. Mass values for specific ships could not be found and were instead estimated based on vessels of similar size and GT listed in PIANC Report No. 121-2014, Table C-1.

Table 6. Dimensions and Mass of Cruise Vessels which visited Parry Sound in 2023

			Dimensions (m)		
Vessel	**Gross Tonnage (gt)**	**Mass (t)**	***Length***	***Beam***	***Draft***
Viking Octantis*	30150	19000	205.0	23.5	6.0
Viking Polaris*	30150	19000	205.0	23.5	6.0
HANSEATIC Inspiration	15650	11500	138.0	22.0	5.6
Le Bellot - Ponant	9976	8000	131.5	18.0	4.6
Le Dumont-d'Urville - Ponant	9976	8000	131.5	18.0	4.6
Pearl Mist	5109	5000	99.1	16.8	3.7
Island Queen V	525.9	800	38.9	9.2	2.9

**NB: Viking Octantis and Viking Polaris currently anchor away from shore and do not dock at the Bay St. Wharf (#401).*

7.2 Loads and Modelling Approach

7.2.1 Background

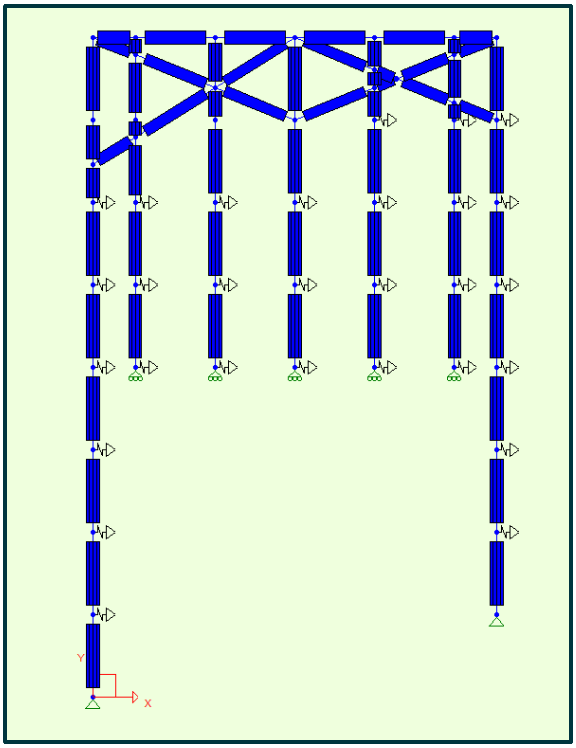
A linear static finite-element analysis (FEA) model of the main Structure D pile bent profile was created using Altair® S-FRAME 2023. The outer timber piles installed in 1931 were modelled as extending to bedrock as shown in record drawings. These piles were assumed to have light penetration into the bedrock, and as such were modelled with pin supports at the tip. The inner timber piles installed prior to 1931 were modelled as shorter lengths resting on roller supports, as their exact condition and bedrock penetration is not known.

Geotechnical information was not available for the timber piles at Structure D. Blow counts found in the 1967 Structure C drawings were used to estimate bearing capacity of the underlying layers of sand and gravel. Based on the bearing capacity, the subgrade modulus of the soil surrounding the timber piles was estimated. A 10,000 kN/m^2/m equivalent spring constant was applied to the pile element joints in the FEA model to simulate the soil.

Member properties were defined as discussed in Sections 7.1.1 and 7.1.2. Structural capacities of the timber piles were determined from hand calculations based on the strength values listed in **Table 4**.

Figure 11 provides a view of the FEA model geometry, including supports and spring locations.

Figure 11. Structure D Model Geometry



7.2.2 Berthing Energy and Resultant Force

Berthing energy refers to the energy applied by vessels as they contact fenders at the dock. The vessel's berthing energy is absorbed by the fender and a resultant force is applied on the wharf. The Canadian Highway Bridge Design Code (CSA S6) does not directly address berthing energies for docking vessels but provides methods for determining vessel collision energies in the context of ships colliding with bridge piers. Collision energies were calculated using CSA S6 for reference, however, berthing energies would be more relevant to consider in the present case. Since CSA S6 does not include methods for determining berthing energies, the British Standard Maritime Works – Code of Practice for Design of Fendering and Mooring Systems (BS 6349-4) was utilized.

Detailed calculations and berthing energy tables for different berthing angles and velocities can be found in **Appendix F**.

Since the fendering system used by cruise vessels when berthing to Structure D is not known, berthing loads of specific vessels could not be determined. The model was instead used to determine the maximum berthing load that may be applied to the wharf. The maximum berthing energy and maximum berthing load values can be used to select a suitable engineered fender for the wharf.

7.2.3 Loads

Four significant load groups were evaluated and applied in the model. Dead loads from the self-weight of the timber piles were defined through the material properties in the model. Dead loads from the self-weight of the concrete superstructure were applied at outer and center piles bearing locations. Two live load conditions were included in the evaluation: 12 kPa due to emergency and utility vehicle usage of the concrete deck, and 4.8 kPa for pedestrian occupancy during berthing of vessels. Ice load caused by ice impacting the wharf was calculated in accordance with CSA S6-19 Cl 3.12.

The ship berthing load was categorized as a vessel load separate from live load. This load was adjusted in the model to determine the maximum berthing load that may be applied to the structure.

A summary of load categories included in the model is provided in **Table 7**.

Table 7. Load Categories and Magnitudes for Structure D Model

Load	Category	Magnitude	Reference
Concrete Superstructure	Dead Load	*See Table 5*	CHBDC S6-19 Table 3.4
Timber Piles/Caps/Stringers	Dead Load	*See Table 5*	CHBDC S6-19 Table 3.4
Pedestrian/Vehicle Traffic on Deck	Live Load	12 kPa (emergency vehicle allowance) 4.8 kPa (during berthing operation)	NBCC 2020 Table 4.1.5.3
Ship Berthing Load	Vessel Load	Varies – determined in model	--
Ice Impact	Ice Load	143.1 kN	CHBDC S6-19 3.12

When lateral loads are applied to the concrete over a pile bent at Structure D, the concrete superstructure is expected to act as a diaphragm and distribute part of the applied load to adjacent pile bents. To represent this behaviour, a secondary FEA model was created. In this model, the concrete deck slab is defined as a beam with springs at the pile bent locations.

The spring coefficient of a single pile bent was approximated using the primary FEA model by determining the deflection produced by a 1 kN lateral load. An equivalent spring coefficient of 7,905 kN/m was calculated by using the 1 kN lateral load and associated lateral deflection.

In the secondary model, a 100 kN load was applied at a pile bent (spring) location. The results showed approximately 35% of the load is resisted by the pile bent where the load is applied. Each adjacent pile bent resists about 25% of the load. The remaining 15% of the load is distributed to other pile bents further away.

For simplicity, the diaphragm action of the concrete superstructure was assumed to distribute 40% of the load to the pile bent where the load is applied, and 30% each to both adjacent pile bents. Therefore, a lateral vessel load applied in the primary model is equivalent to 40% of the maximum load.

7.2.4 Load Combinations

From CSA S6-19, ULS Combinations 1 and 7 were considered relevant and applied to the FEA model. CSA S6-19 does not provide a load combination for berthing vessels, although ULS Combination 8 applies to vessel collisions. ULS Combination 8 is defined as:

1.2 x [Dead Load] + 0.5 x [Live Load] + 1.0 x [Vessel Collision]

The likely intent of ULS Combination 8 is to account for relatively infrequent cases where vessels may inadvertently collide with bridge piers, rather than the daily berthing of vessels to wharves. In the absence of a load combination specific to berthing in CSA S6-19, a modified iteration of ULS Combination 8 with larger factors for live and vessel loads was defined and analyzed in the FEA model.

Table 8 lists the load combinations considered in the finite-element analysis of Structure D.

Table 8. Load Combinations

Combination	Reference	Load Combination
ULS Combination 1	CSA S6-19, Tables 3.1-3	1.2 x [Dead Load] + 1.7 x [Live Load 12 kPa]
ULS Combination 7	CSA S6-19, Tables 3.1-3	1.2 x [Dead Load] + 1.3 x [Ice Load]
Mod. ULS Combination 8	--	1.2 x [Dead Load] + 1.0 x [Live Load: 4.8 kPa] + 1.5 x [Vessel Load]

7.2.5 Output Interpretation

Since CSA S6-19 and NBCC 2020 do not provide horizontal deflection limits for wharves or docks, a maximum deflection limit of L/240 (12.5 mm) was assumed, where L is the span between pile bents. Shear, moment, and axial loads obtained from analysis results were checked to confirm members can resist the imposed loads. Since significant axial loads and moments were developed in the piles, combined bending and axial interaction was checked to confirm piles can resist the combined load.

Analysis of ULS Combinations 1 and 7 found Structure D has adequate capacity to resist the loads imposed in those combinations. Analysis of Mod ULS Combination 8 found Structure D pile bents can withstand an applied vessel load of 80 kN, limited by deflection criteria. Recalling that this load is 40% of the total maximum as discussed in Section 7.2.3, the maximum allowable berthing force on Structure D is 200 kN.

7.3 Evaluation Results

7.3.1 Bollard Resistance and Loading

Bollard tensile and shear capacities for east and west bollards were determined in accordance with CSA A23.3. East bollards were found to be critical compared to west bollards, as west bollards have a larger edge distance and can withstand higher loads than the east bollards.

Shear resistance of a single east bollard without spalling was calculated to be 88 kN in the transverse direction (perpendicular to wharf) and 138 kN in the longitudinal direction (along wharf). Tensile breakout strength of the bollards in the vertical direction was calculated as 178.2 kN. East bollards subjected to an equal or greater load may experience delamination and spalling of concrete at the face, however, the bollard may remain usable. After failure of the concrete face ahead of the outer anchors, shear capacity of the bollards is reduced to 82 kN in the transverse direction resisted by the remaining anchors.

Critical loading experienced by the bollards is caused by wind and wave action on moored vessels. Values for wind speed over 10-year return period for Parry Sound were obtained from NBCC Appendix C. Wind forces and wave current forces acting on moored vessels was calculated using BS 6349-1. **Table 9** summarizes the calculated loads applied by vessels moored to the bollards. The calculated loads assume that both ends of the vessel are moored to the wharf and the vessel does not extend past the end of the wharf. Based on calculations, the only ship which can safely dock using the Bay St. Wharf bollards is the Island Queen V.

Despite its current condition, the bollard at STA 0+129 retains sufficient capacity for the Island Queen V to use. The Island Queen V should use this bollard to moor rather than the curb railing currently used, as the capacity of the curb railing is unknown and expected to be less than that of the bollard.

Table 9. Loads Exerted on Bollards by Moored Vessels

Vessel	Transverse Load on Bollard (kN)	Longitudinal Load on Bollard (kN)
Viking Octantis*	460	114
Viking Polaris*	460	114
HANSEATIC Inspiration	257	64
Le Bellot - Ponant	224	55
Le Dumont-d'Urville - Ponant	252	62
Pearl Mist	131	32
Island Queen V	25	6

**NB: Viking Octantis and Viking Polaris currently anchor away from shore and do not dock at the Bay St. Wharf (#401).*

Shear capacity of repaired east bollards may be increased if reinforcing steel (fully developed 20M) is appropriately installed parallel to the edge of deck during repair. East bollards repaired in this method are calculated to have a transverse shear resistance of 122 kN.

Repaired east bollards may be able to withstand higher loading if edge reinforcement (fully developed 20M) is installed, anchors are welded to the bollard plate, and minimum 35 MPa concrete is cast in the critical shear area surrounding the bollard. These combined repair methods may increase transverse shear capacity of the bollards to 266 kN and breakout strength in tension to 235 kN. Provided tensile loading on the bollards by mooring vessels is confirmed to be below the breakout capacity, this repair would allow for most of the vessels listed in **Table 9** to use the bollards apart from the two Viking ships.

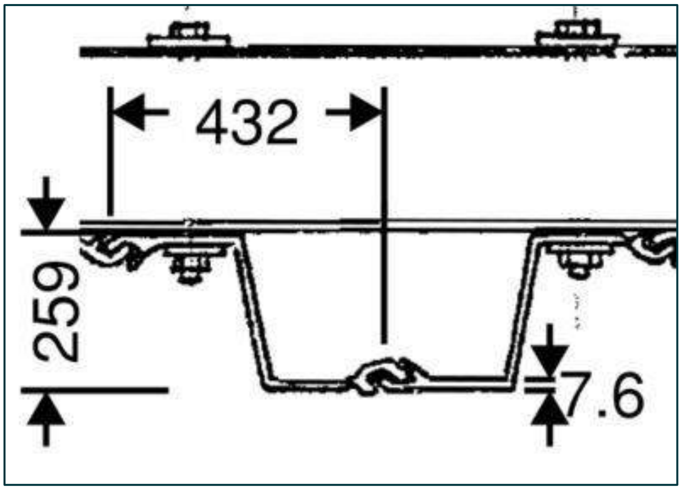
Detailed calculations of bollard capacities and loadings are provided in **Appendix G**.

7.3.2 Steel Sheet Pile Capacity (Structure C)

Steel sheet pile capacity was calculated using a safety factor of 1.5 at four representative locations with different dredge depths along the length of the substructure. The four locations as labelled in the 1967 record drawings are BH11, BH12, BH13, and BH2.

Record drawings do not indicate the exact sheet pile section used for the encapsulation in 1967, however, conservative estimates of the dimensions were made based on available data. **Figure 12** provides details of the dimensions of the steel sheet pile assumed for the evaluation.

Figure 12. Steel Sheet Pile Dimensions



The walers considered in the evaluation were two C250x30 channels, tie rods were 57 mm diameter (#18) steel tie rods and deadman anchors were 2.44 m x 1.524 m precast concrete anchor blocks, based on records drawings. The capacity of the concrete deadman anchor was calculated in accordance with CSA A23.3.

A 7.2 kPa surcharge load was applied to the sheet pile wall, along with a mooring load from the Island Queen vessel.

Based on the evaluation, Structure C has adequate capacity for the Island Queen V to continue docking at this location.

Detailed calculations for steel sheet pile capacity are provided in **Appendix H**.

7.3.3 Timber Pile Bent Substructure Capacity (Structure D)

As noted in section 7.2.5, maximum horizontal berthing force that may be applied to Structure D was determined using finite-element analysis to be 200 kN. This value is limited by the maximum deflection criteria selected to be L/240 or 12.5 mm. Shear, bending, and axial loads experienced by the timber piles under different load combinations were analyzed and confirmed to be well within the capacity of the piles. The combined bending and axial interaction on the timber piles was also checked and confirmed to satisfy the interaction equation in CSA O86-19 Cl 6.5.9.

The maximum vessel size that may safely berth to this section of the wharf is highly dependant on the type of fender system used. The current fender system(s) used by cruise ships docking at Parry Sound could not be determined, as the wharf does not have any engineered fenders installed and cruise ships are likely using their own onboard fender systems. Fender systems and berthing procedures must be reviewed to ensure fenders are able to safely absorb the docking vessel's berthing energy without applying a force exceeding 200 kN.

7.4 Summary of Findings

Sheet pile capacity at Structure C was confirmed to be sufficient for the Island Queen V cruise ship to continue docking at this location. Structural analysis of the Structure D pile bents established a maximum ship berthing load of 200 kN, which must be compared with the berthing energies calculated for vessels and the energy deflection behaviour of the specific fendering system used. No fender system could be identified at the wharf, aside from large truck tires used as makeshift fenders. With proper fendering and berthing procedures, including strict control of vessel berthing velocity and angle, an engineered fender system such as Trelleborg's AN Arch Fenders may potentially allow for vessels such as the HANSEATIC Inspiration to berth. However, the capacity of the current structure is limited by the condition of the bollards.

Based on completed calculations, bollards in the current condition cannot safely moor any cruise vessel considered, except for the Island Queen V.

Bollard capacity may be increased to allow larger cruise ships to dock, if a more in-depth and comprehensive repair project is undertaken. As a minimum, this repair project should involve the installation of edge reinforcement, welding of bollard anchors to the bollard plate, and replacement of surrounding concrete in the shear zone with new concrete of minimum 35 MPa compressive strength.

8. Code Compliance

A general review of the facility components was completed for compliance with the Canada Occupational Health and Safety Regulations (SOR/86-304) and the National Building Code of Canada. The following section outlines weaknesses of the facility with respect to legislative requirements.

8.1 Safety Ladders

For compliance with SOR/86-304 Section 12.15 'Protection Against Drowning' Paragraph (2) of the Canada Occupational Health and Safety Regulations, there is a requirement for ladders every 60 m along wharfs, docks, and piers. The ladder must extend two or more rungs below the water level.

Three safety ladders, two steel and one plastic, were observed along the length of the Bay St. Wharf. However, the plastic ladder is non-compliant as the rungs do not extend below water level, and one of the steel ladders was blocked by a timber fender.

Considering the 425 m perimeter of the wharf, it is recommended that a total of at least 7 safety ladders be installed for compliance.

8.2 Stairs

There are ten (10) stairways along the west side of Structure D, descending to the water level. These concrete stairways are approximately 3.5 m wide and consist of 4 risers. As a guideline for stairways and passageways used by the public as access to exit, National Building Code of Canada (NBCC) 3.4.6.5 requires that one handrail is provided on each side of stairways that are at least 1.1 m wide. No handrails are present at the stairways. Railing guards were observed at the top of the stairs, however, one railing post was missing and another railing was bent with perforations in the steel. In addition, some of the stair treads were found to be in fair to poor condition with concrete spalls, medium to severe scaling, and cracking.

Repair of the concrete stairs and existing railing guards, as well as installation of a handrail on both sides of each stairway is recommended.

9. Evaluations and Recommendations

AECOM reviewed the existing Bay St. Wharf component structures at Parry Sound, Ontario and the following section summarizes the evaluation and recommendations.

9.1 Structure A (STA 0+000 to 0+013.7)

Structure A was deemed to be in fair condition with light to medium scaling, areas of spalling and delamination, cracking and indication of shifting of precast blocks. The timber crib substructure has exceeded its Useful Residual Life (URL); however, it is recognized that timber can have significantly longer service lives. A safe service life of 10 to 15 years is expected with routine inspection. Replacement/encapsulation of the timber crib structure is recommended in the long term (11 to 15-year time period). The concrete deck superstructure has also exceeded its URL and while there are a number of observed defects, replacement is not recommended until replacement of the timber crib structure. Localized concrete repairs of the deck top are recommended to maintain a safe walking surface.

It was noted that this section of the wharf does not have public protection measures along the edge of the wharf and, it is recommended to install a curb rail. There were no safety ladders along this section of the wharf; however, this structure is a short section and may not require a safety ladder depending on the location of ladders along adjacent sections.

9.2 Structure B (STA 0+013.7 to 0+048.3)

Structure B was deemed to be in fair condition with localized areas in poor condition. The deck had areas of delamination, light to medium scaling and cracking. An area of severe deterioration and exposed reinforcing steel was observed in the cope wall at approximately STA 0+039.5. A pipe was observed discharging below the deck that has led to an accumulation of sand below and in front of the wharf structure. The discharge may have also contributed to advanced deterioration of the concrete. The pipe is suspected to discharge storm water, as water and material discharge was observed while firefighters were addressing a nearby fire at the time of diving inspection.

The timber pile substructure has greatly exceeded its URL; however, it is recognized that timber can have significantly longer service lives. A safe service life of 6 to 10 years is expected with routine inspection. Replacement/encapsulation of the timber pile structure is recommended in the medium term (6 to 10-year time period). The concrete deck superstructure has also exceeded its URL and while there are a number of observed defects, replacement is not recommended until replacement of the timber pile structure. Localized concrete repairs of the deck top are recommended to maintain a safe walking surface.

It was noted that this section of the wharf does not have public protection measures along the edge of the wharf and, it is recommended to review vessel use for consideration of curb rail installation. There was one plastic safety ladder along this portion of the wharf, however, this ladder is non-compliant and it is recommended to replace this ladder with one that is code compliant.

9.3 Structure C (STA 0+048.3 to 0+117.8)

Structure C was deemed to be in fair to good condition. The concrete deck had two areas of settlement, and settlement induced concrete cracking; however, one of the areas had repair work carried out recently to fill voids below the concrete deck with spray foam and the second area had one core hole taken in the area of settlement and no void was observed below the concrete deck. The two areas of settlement may have stabilized based on the observations; however, continued monitoring is recommended. Multiple areas of delamination and light scaling were also observed on the concrete deck. The timber fenders were in fair to poor condition with cracking, splitting, rotting and disintegration. It is recommended to replace the timber fenders.

Considerable marine growth was noted on the steel sheet pile below water level, making visual observation difficult. Ultrasonic measurement of the sheet pile indicated an average steel thickness of 7.6 mm with a maximum thickness of 8.9 mm and a minimum thickness of 5.4 mm. In addition, 90% of recorded measurements indicated a thickness between 6.2 and 8.9 mm. The thickness of the steel at the time of installation is unknown as available record drawings do not provide this information.

The steel sheet pile substructure has a calculated URL of 23 years, and the concrete deck has a URL of zero and the actual age is approaching the theoretical useful life. However, while there were observed defects, the concrete deck has been maintained with periodic repairs and replacement of the concrete deck is not recommended at this time. A safe service life of 10 to 15 years is expected for the concrete deck with routine inspection Replacement of the deck can be considered in the long term (11 to 15-year time period).

An evaluation of the steel sheet pile wall indicated that Structure C has adequate capacity for the Island Queen V to continue docking at this location.

There were two ladders noted along Structure C; however, one ladder was blocked by a horizontal timber fender and was not useable. It is recommended to review spacing of ladders and install new ladders that are accessible.

9.4 Structure D (STA 0+117.8 to 0+273.3)

Structure D was deemed to be in fair condition with areas in poor condition. The concrete deck had areas of medium to severe delamination, spalling and scaling, medium to wide cracking typical. The concrete around several bollards had severe spalling and delamination with exposed bollard plates and anchors. The stairs along the west side of the wharf had areas of scaling, cracking, spalling, delamination, disintegration with exposed reinforcing steel.

A very wide crack was noted at the south end of the deck, near approximately STA 0+271, extending from a joint in the precast blocks below the concrete deck and progressing transversely across the deck. The location and severity of the crack suggests a potential settlement or movement of the deck. It is recommended to monitor the crack width and elevation of the deck in the area of the crack to determine if settlement is occurring.

Curb rail was noted along the east and south side of the wharf, but not along the west side. In addition, the curb rail was observed to be cut and deformed in some areas.

Railing guards were observed at the top of the stairs, however, one railing post was missing and another railing was bent with perforations in the steel.

The timber piles were generally observed to be in good condition with typically low penetration depths recorded, with an average of 3.8 mm. The timber fill pieces between the timber piles were observed to be in poor condition with rotting and splitting typical. The investigation of the interior of the structure was limited based on difficult access, even with the ROV as high backfill levels inside the structure limited movement.

Cores taken through the concrete deck indicated that the fill directly below the deck has been lost and voids below the deck of approximately 1.7 m.

One of the “I” beams at the water level gauge station was observed to have significant section loss and pitting and may require some additional investigation and repair.

The timber pile substructure has greatly exceeded its URL; however, it is recognized that timber can have significantly longer service lives. A safe service life of 10 to 15 years is expected with routine inspection. Replacement/encapsulation of the timber crib structure is recommended in the long term (11 to 15-year time period). The concrete deck superstructure has also exceeded its URL and while there are a number of observed defects, replacement is not recommended until replacement of the timber pile structure. Localized concrete repairs of the deck top are recommended to maintain a safe walking surface.

A load evaluation was carried out on Structure D to determine the largest vessels that can be accommodated. Based on the bollard capacity, the only vessel that can safely dock using the Bay St. Wharf is the Island Queen V, as the other vessel considered impart too large of a force onto the bollards due to wind loading acting on the vessel when moored. If the bollards were repaired including edge reinforcement, welding of anchors to bollard plate and replacement of concrete with 35MPa concrete, there is any opportunity that larger vessels could be permitted to moor at the wharf; however, the fender system would need to be reviewed to determine the berthing force on the wharf.

The capacity of the timber pile substructure of Structure D was determined to be 200 kN of berthing force. The size of the vessel could not be determined as it is highly dependent on the fender system used by the vessels since the wharf does not have fixed fenders in place.

9.5 Summary of Recommendations:

A summary of key recommendations is provided below.

9.5.1 General

- Install ladders along the length of the wharf.
- Localized concrete deck repairs.
- Routine scheduled inspections over remaining service life.

9.5.2 Structure A (STA 0+000.0 to 0+013.7)

- Install curb rail for enhanced safety.
- Encapsulate Structure A with Steel Sheet Pile.

9.5.3 Structure B (STA 0+013.7 to 0+048.3)

- Install curb rail for enhanced safety.
- Dredge lakebed locally at outfall.
- Localized concrete repair of cope wall.
- Encapsulate Structure B with Steel Sheet Pile.

9.5.4 Structure C (STA 0+048.3 to 0+117.8)

- Replace timber fenders.
- Replace entire deck.

9.5.5 Structure D (STA 0+117.8 to 0+273.3)

- Restrict mooring at Structure D to vessels that are no larger than the Island Queen V, as governed by bollard capacity.
- Restrict berthing at Structure D to a fendering force of 200 kN.
- Reconstruct sections of deck around bollards, install fully developed 20M edge reinforcing and weld anchors to bollard plates to increase bollard capacity..
- Repair concrete stairs and existing railing guards, as well as installation of a handrail on both sides of each stairway.
- Monitor crack at south end of Structure D.
- Install proper fenders for vessels.
- Encapsulate Structure D with Steel Sheet Pile.

10. Cost Summaries and Priorities

Based on the recommendations developed in the previous section, preliminary cost estimates were calculated based on typical unit costs for work on similar structures. The cost estimates are in 2024 dollars, exclusive of taxes and include material supply, delivery and installation. Detailed costing of individual work is included in **Appendix I**. The following components were included in the preliminary cost estimates, based on a percentage of the capital cost subtotal:

- Contractor overhead, profit, bonds and insurance – 15%
- Preliminary estimating contingency – 20%

Costs for individually procured work items may vary from the quoted estimates according to various factors, such as local market conditions, economy of scale, season of work, requirements for engineering and other miscellaneous factors. A summary of the preliminary cost estimates is provided in **Table 10**.

Timing of recommendations are provided based on three priorities, as follows:

- Priority 1 items: recommended immediately.
- Priority 2 items: recommended for completion within 1 to 5 years.
- Priority 3 items: recommended for completion within 6 to 10 years.
- Priority 4 items: recommended for completion within 11 to 15 years.

Table 10. Summary of Preliminary Cost Estimates

Structure	Description of Work	Estimated Cost (2024 Dollars)	Timing
Overall	►Install ladders along length of wharf.	**►$50,000**	**►Priority 1**
	►Localized concrete deck repairs	**►$150,000**	**►Priority 2**
Structure A 0+000.0 to 0+013.7	►Install curb rail	**►$10,000**	**►Priority 1**
	►Encapsulation of Structure A	**►$300,000**	**►Priority 4**
Structure B 0+013.7 to 0+048.3	►Install curb rail ►Concrete repair of cope wall ►Dredge lakebed at outfall	**►$51,000**	**►Priority 1** **►Priority 1** **►Priority 1**
	►Encapsulate Structure B	**►$520,000**	**►Priority 3**
Structure C 0+048.3 to 0+117.8	►Replace timber fenders	**►$30,000**	**►Priority 2**
	►Replace entire deck	**►$400,000**	**►Priority 4**
Structure D 0+117.8 to 0+273.3	►Repair concrete stairs ►Repair railings ►Install stair handrails	**►$85,000**	**►Priority 1** **►Priority 1** **►Priority 1**
	►Concrete repairs at bollards ►Install fenders	**►$445,000**	**►Priority 2** **►Priority 2**
	►Encapsulate Structure D	**►$6,700,000**	**►Priority 4**

Appendix A

Site Plan Drawings

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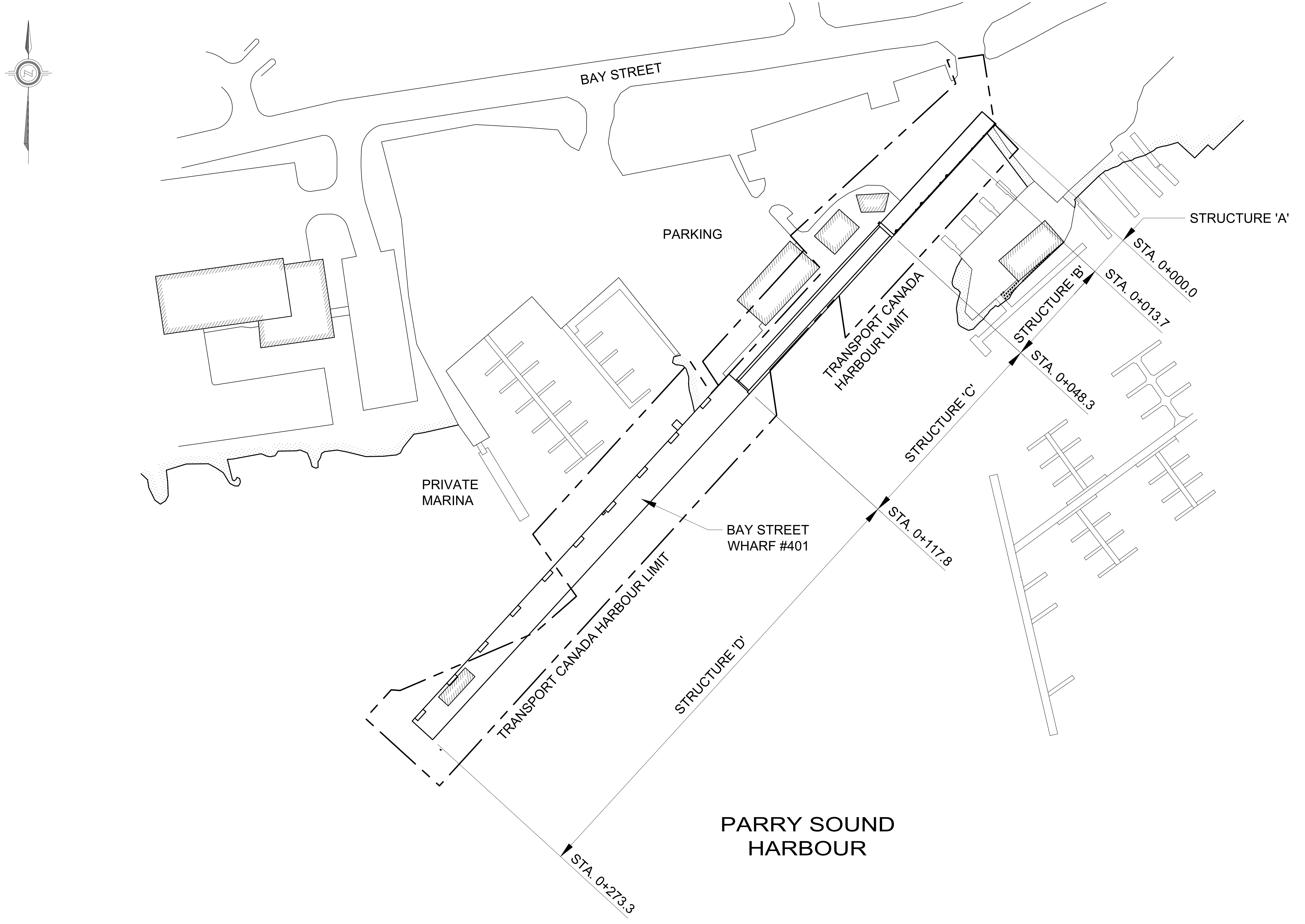


NOTES:
1. DIMENSIONS IN MILLIMETERS.
2. ELEVATIONS IN METERS.

GOVERNMENT OF CANADA GOUVERNEMENT DU CANADA
FISHERIES AND OCEANS

SMALL CRAFT HARBOURS

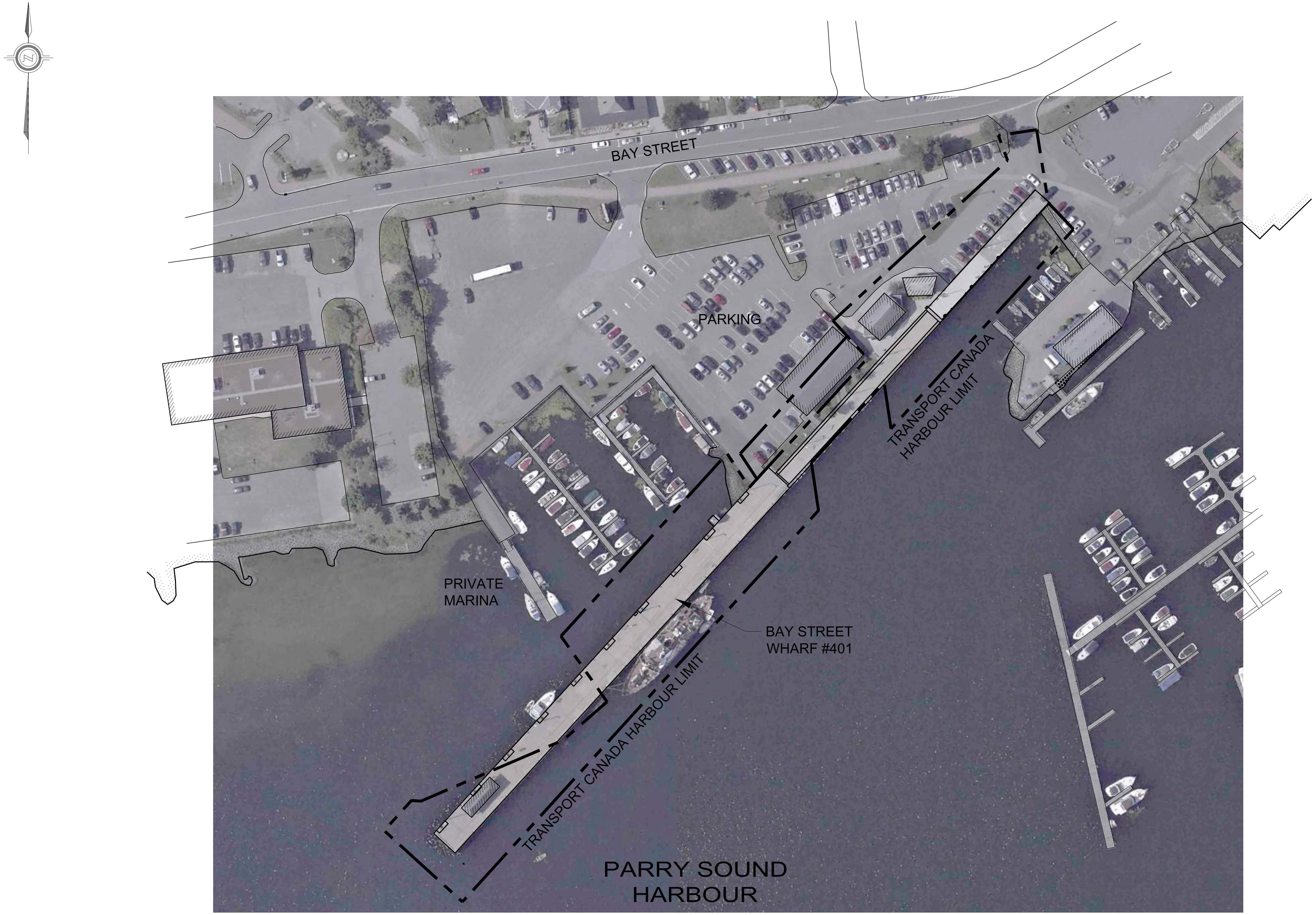
AECOM

PARRY SOUND HARBOUR

GENERAL ARRANGEMENT
SITE PLAN

FIGURE No.
FIGURE - A1

Date:
JANUARY 2024



NOTES:
1. DIMENSIONS IN MILLIMETERS.
2. ELEVATIONS IN METERS.

GOVERNMENT OF CANADA GOUVERNEMENT DU CANADA
FISHERIES AND OCEANS

SMALL CRAFT HARBOURS

AECOM

PARRY SOUND HARBOUR

PROPERTY PLAN

FIGURE No.
FIGURE - A2

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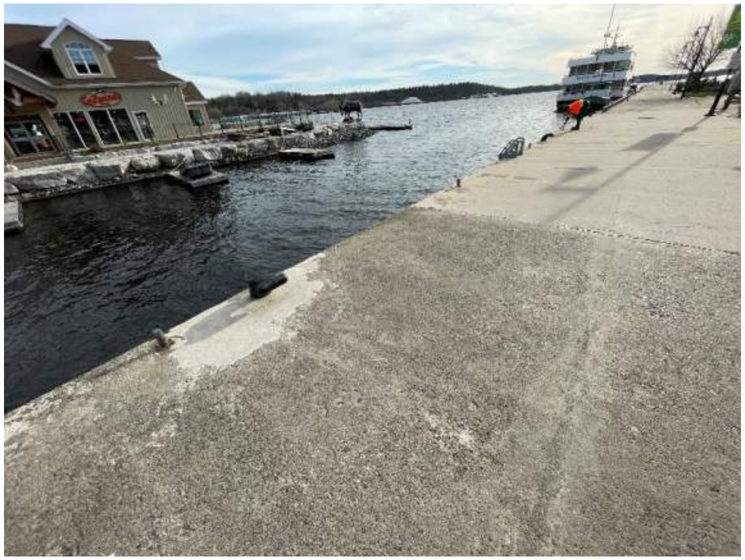
Appendix B

Photographs

1- Bay St Wharf – East Face (looking South from shoreline)

2- Structure A – East Face

3- Structure A – Concrete Deck



4- Structure B – East Bollard, Stn 0+023

5- Structure B – East Bollard, Stn 0+023

6- Structure B – Newer Concrete Deck Transition, Stn 0+026

7- Structure B – Plastic Ladder and Delamination, Stn 0+32

8- Structure B – Plastic Ladder, Stn 0+32

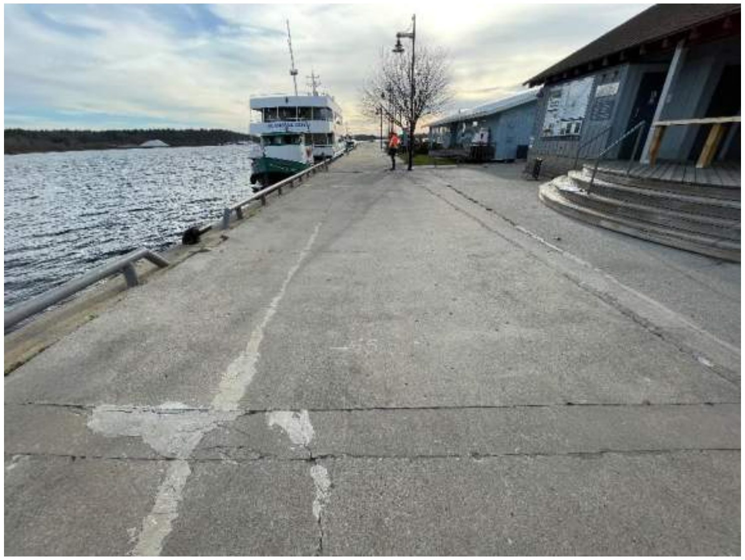
9- Structure B – East Bollard, Stn 0+036

10- Structure B – Map Cracking at Concrete Deck, Stn 0+045

11- Structure B – East Bollard, Stn 0+047

12- Structure B – Spall and Patch at Transition, Stn 0+048.3

13- Structure C – Settled Area, Stn 0+048.3 to 0+069



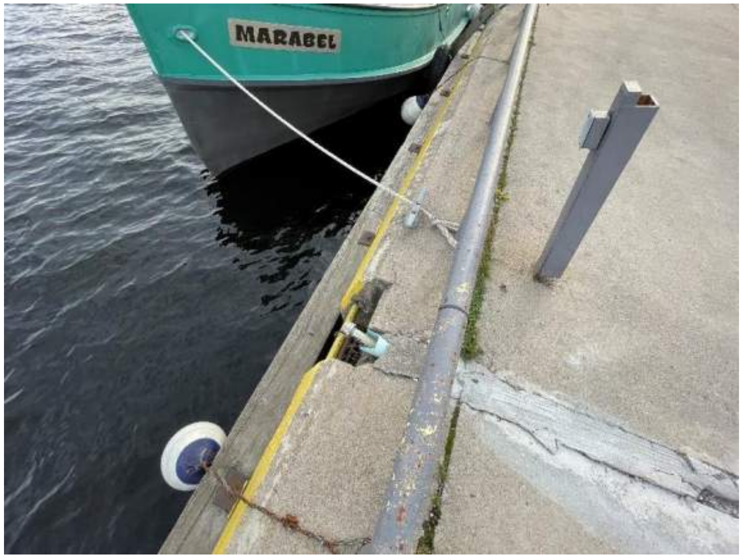
14- Structure C – Longitudinal Crack extending past Settlement

15- Structure C – East Bollard and N-M Crack, Stn 0+057

16- Structure C – East Bollard, Stn 0+057

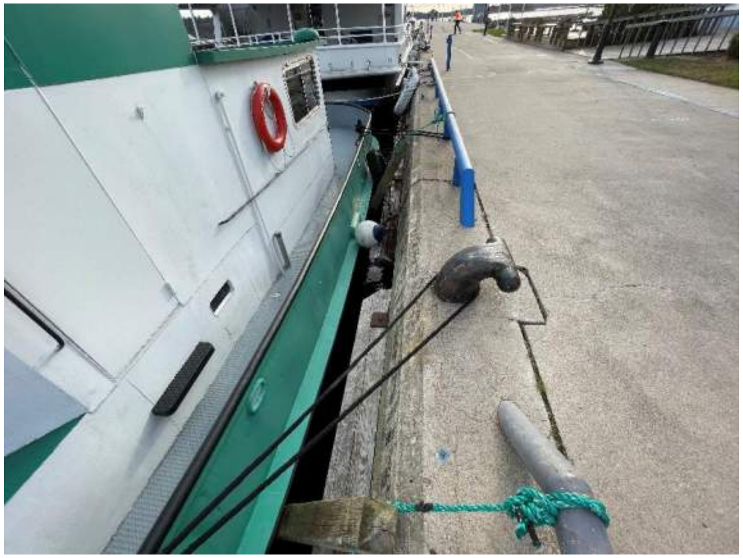
17- Structure C – Buried Conduit, Stn 0+064

18- Structure C – Buried Conduit, Stn 0+069



19- Structure C – Steel Ladder blocked by Timber, Stn 0+069

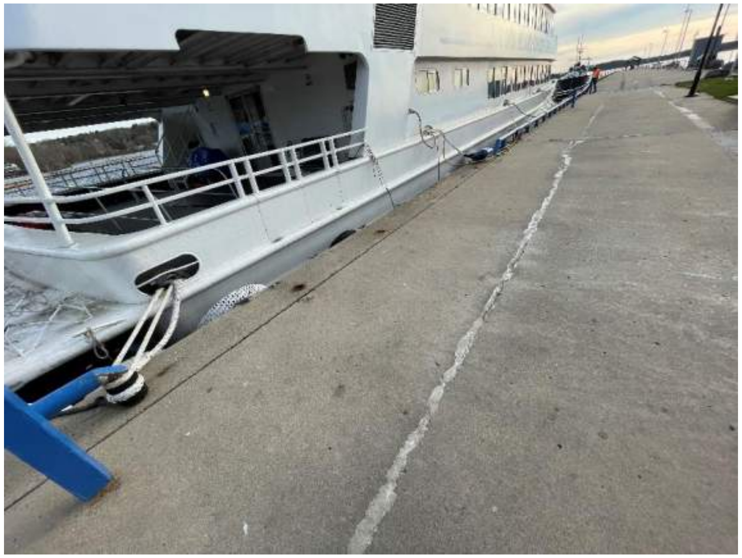
20- Structure C – Concrete Deck, Stn 0+075



21- Structure C – East Bollard, Stn 0+076

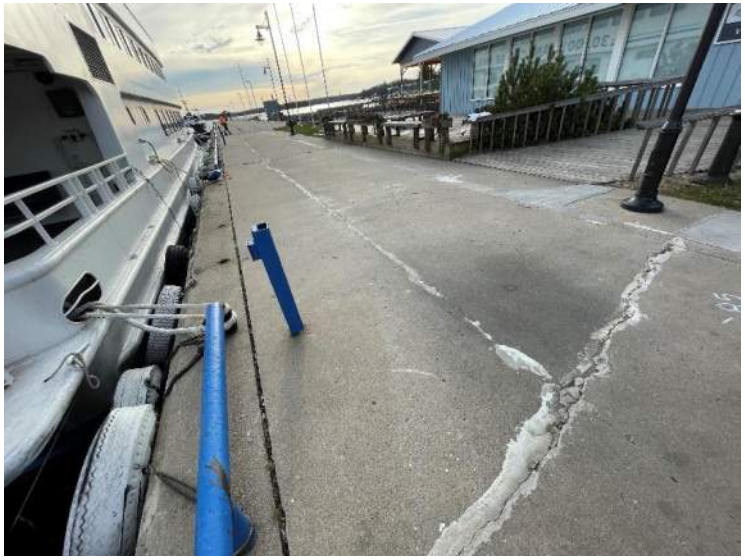
22- Structure C – East Bollard, Stn 0+076

23- Structure C – Island Queen Cruise Ship, Stn 0+085 to 0+125



24- Structure C – Removed Curb Railing, Stn 0+087 to 0+093

25- Structure C – Removed Curb Railing, Stn 0+087 to 0+093



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28- Structure C – Settled Area, Stn 0+095

29- Structure C – Fenders along Island Queen Cruise Ship

30- Structure C – East Bollard, Stn 0+94

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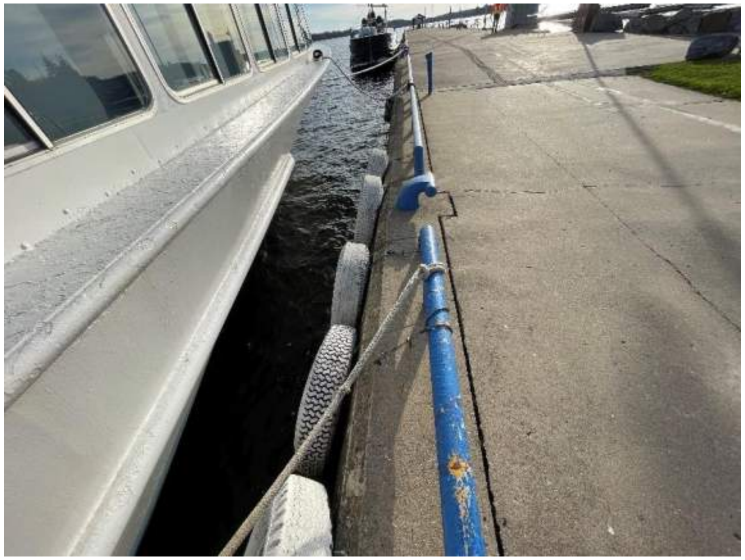
32- Structure C – Longitudinal Crack from Settlement, Stn 0+105



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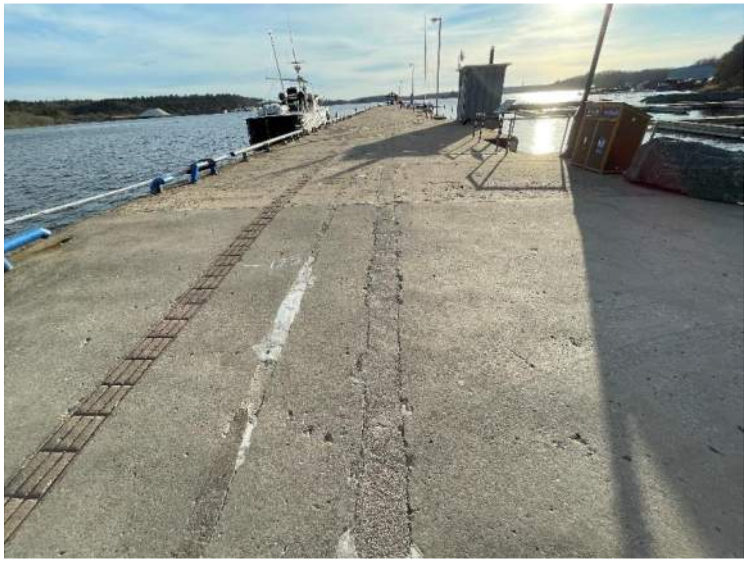

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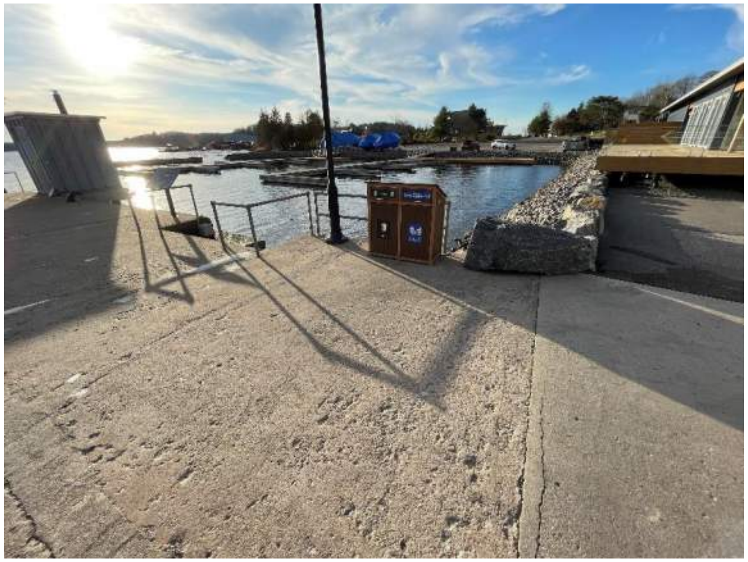

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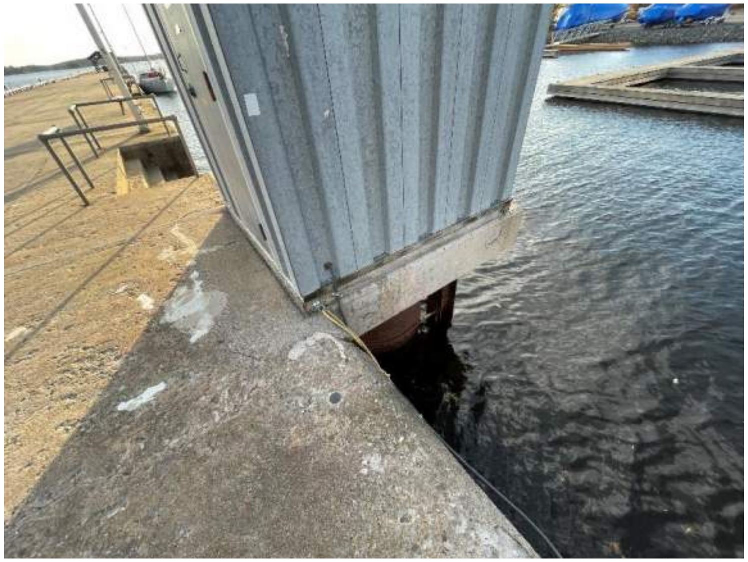

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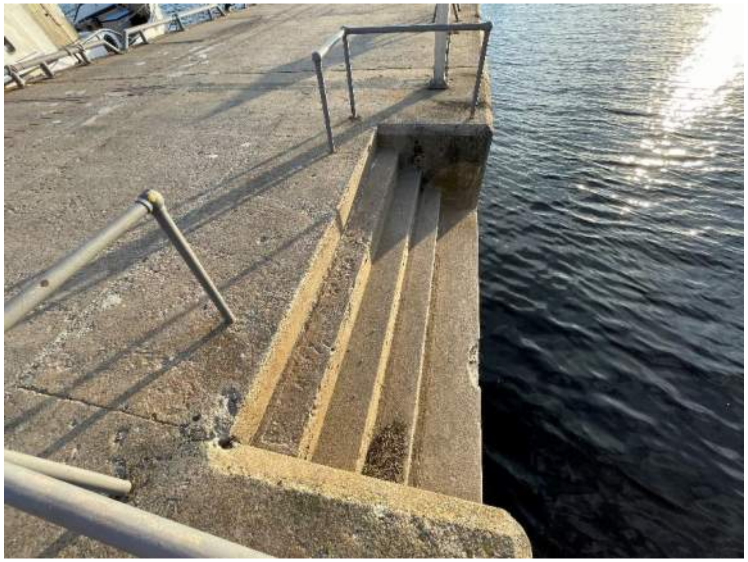

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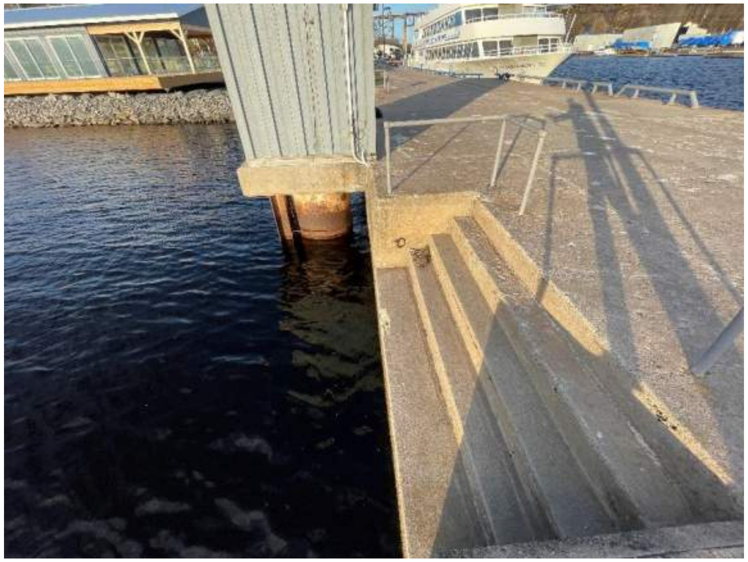

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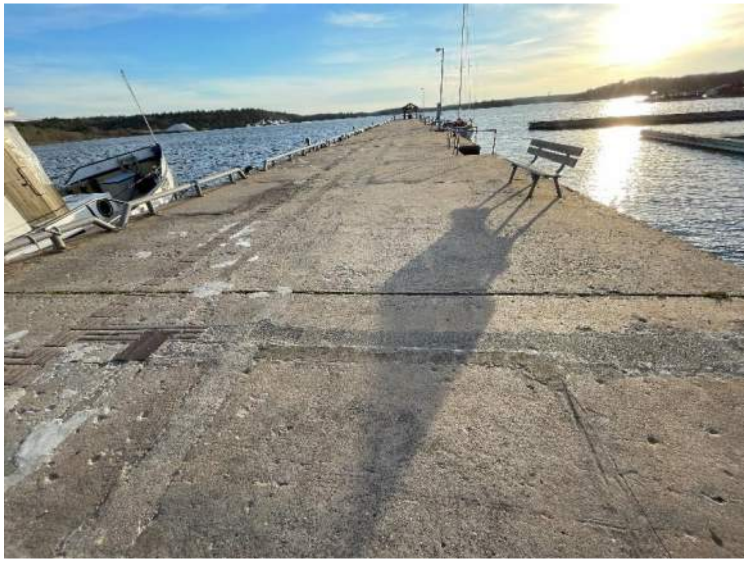

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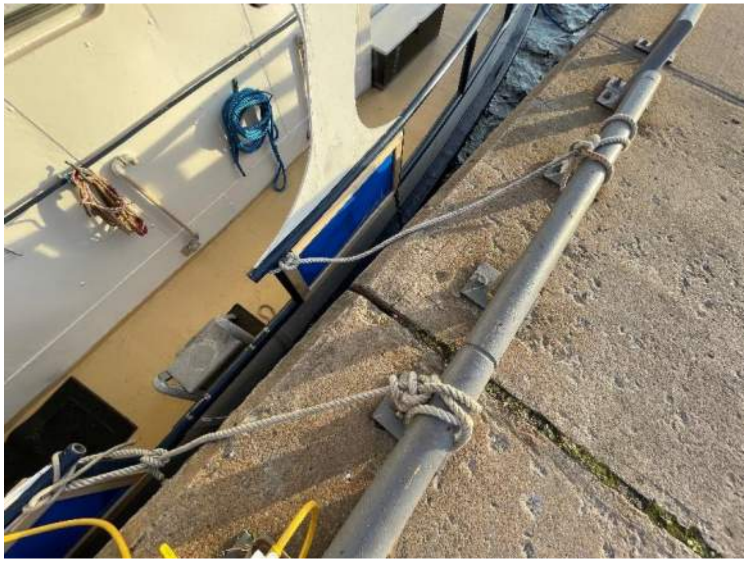

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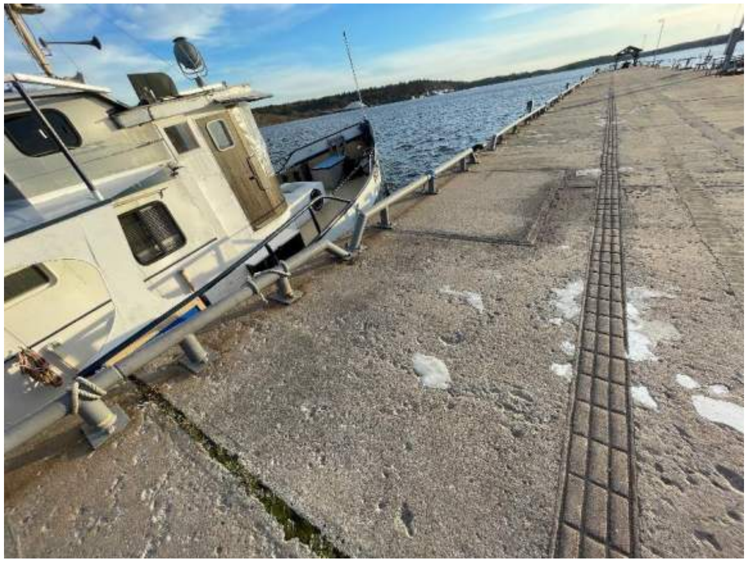

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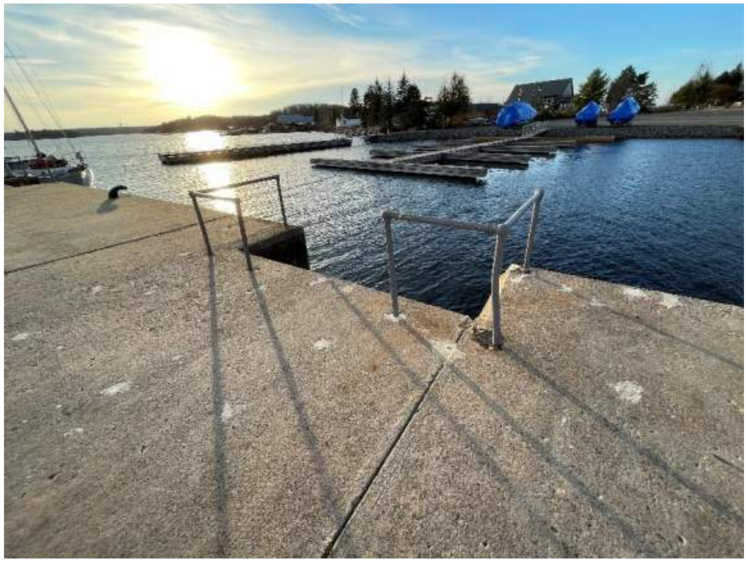

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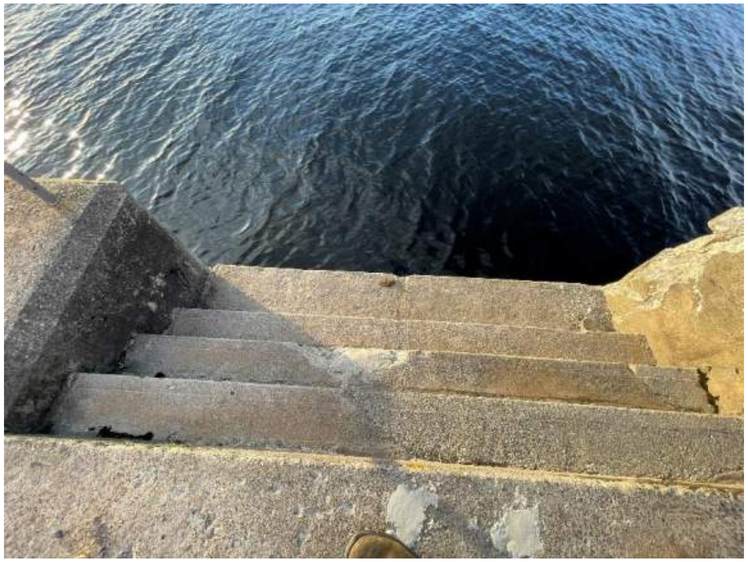

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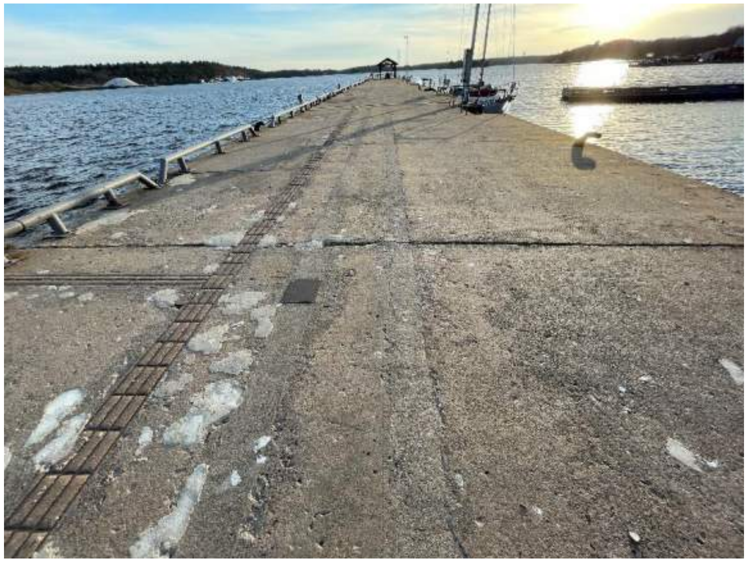

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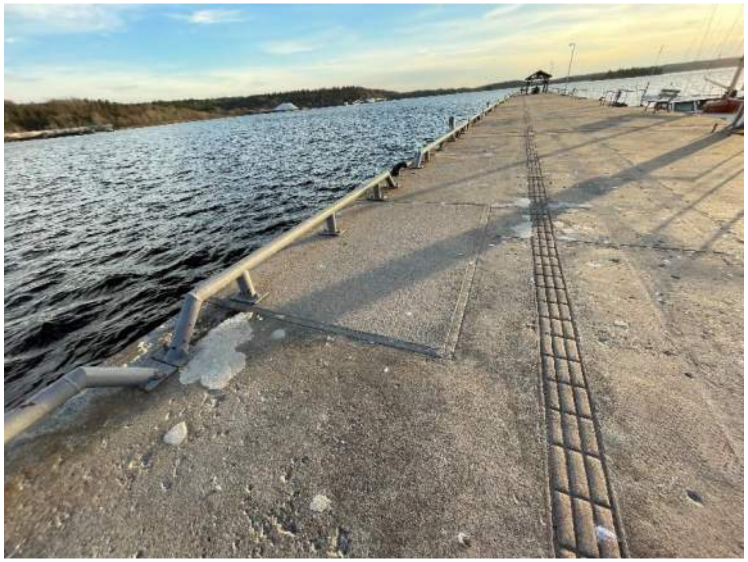

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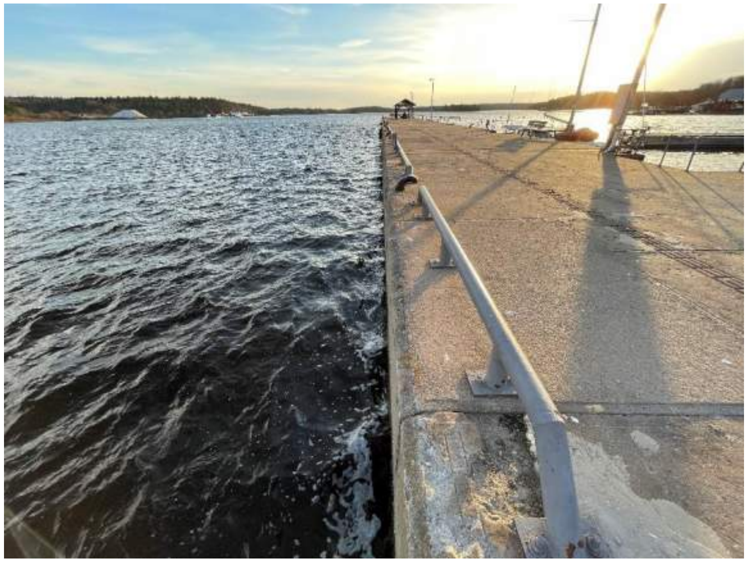
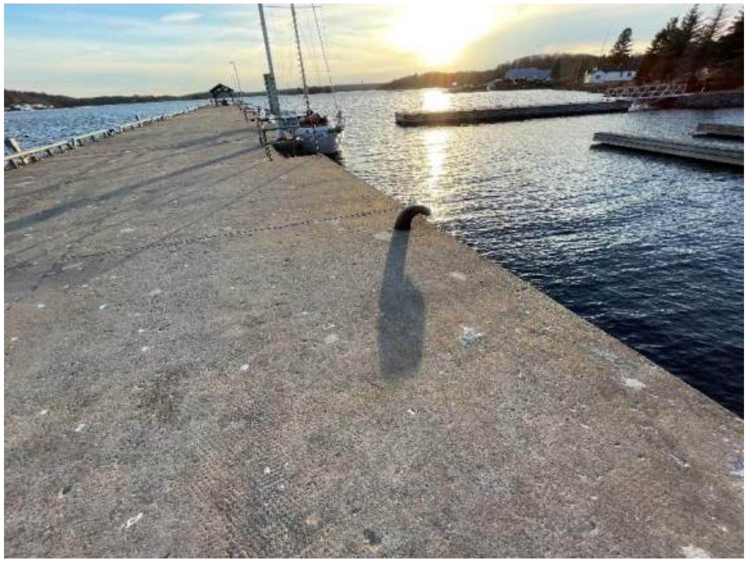


78- Structure D – Buried East Slipway, Stn 0+169 to 0+172



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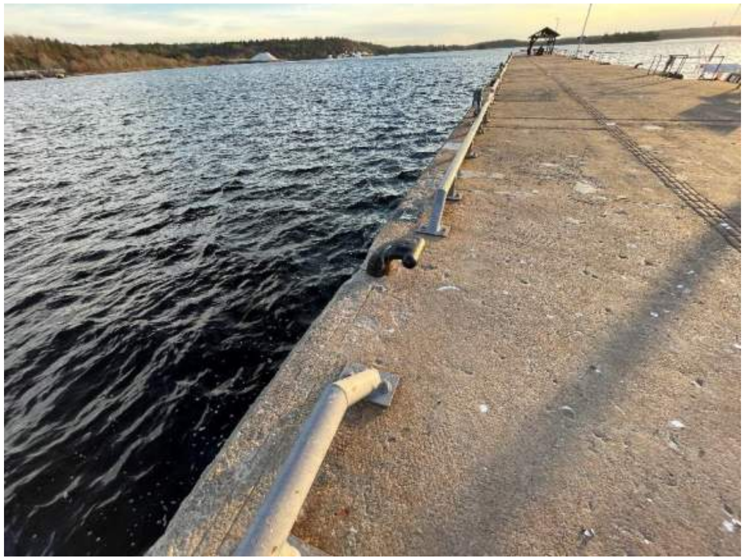

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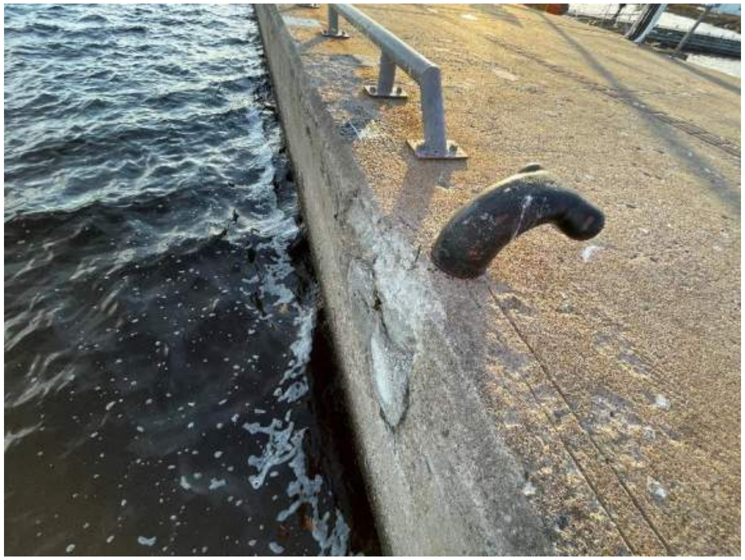

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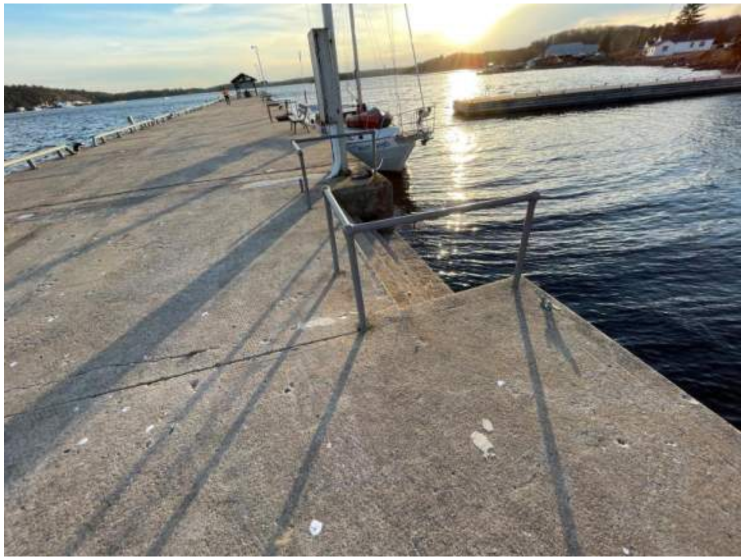

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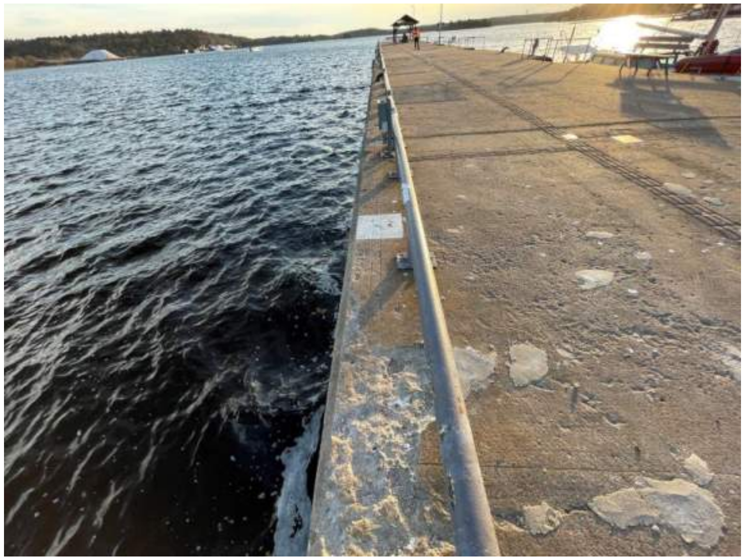

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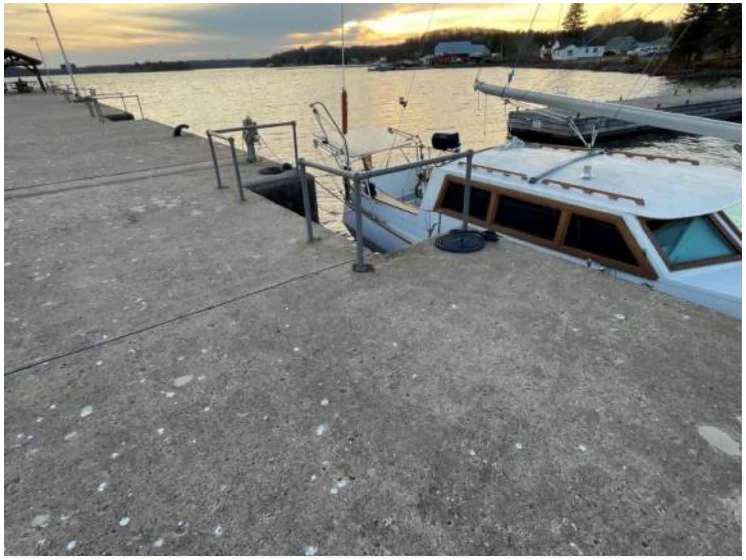

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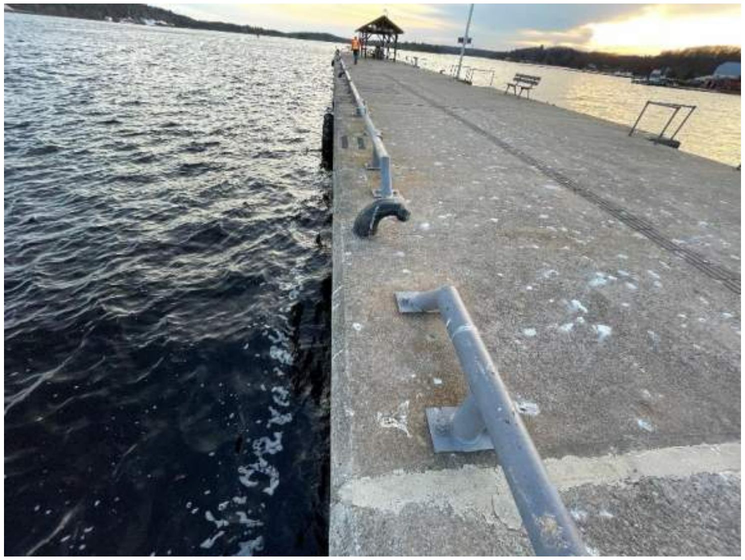

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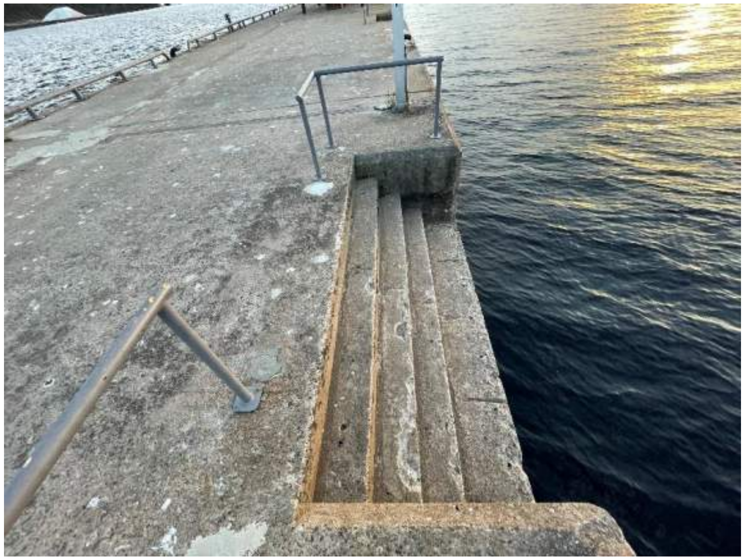

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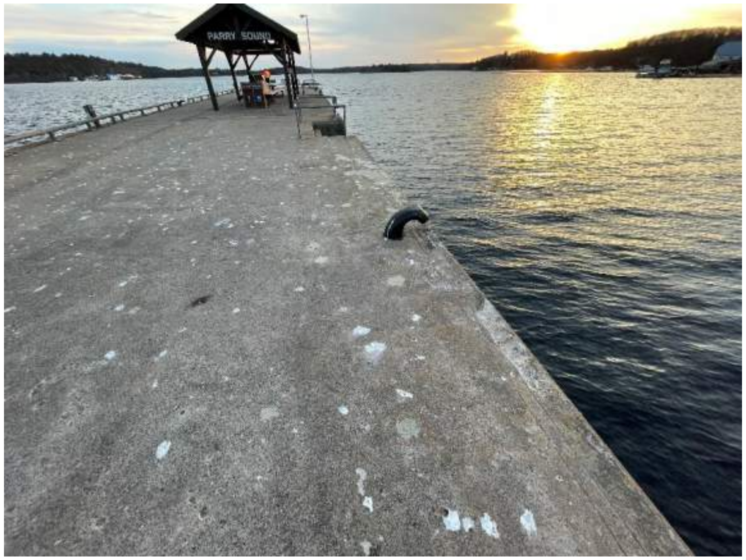

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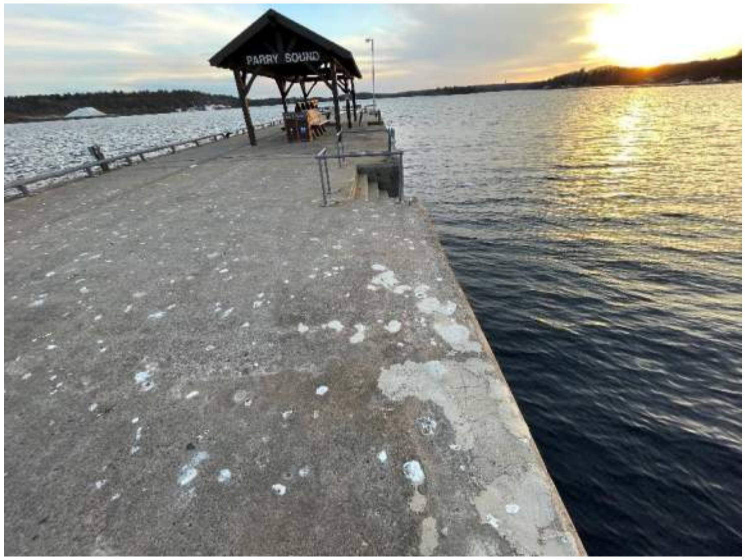

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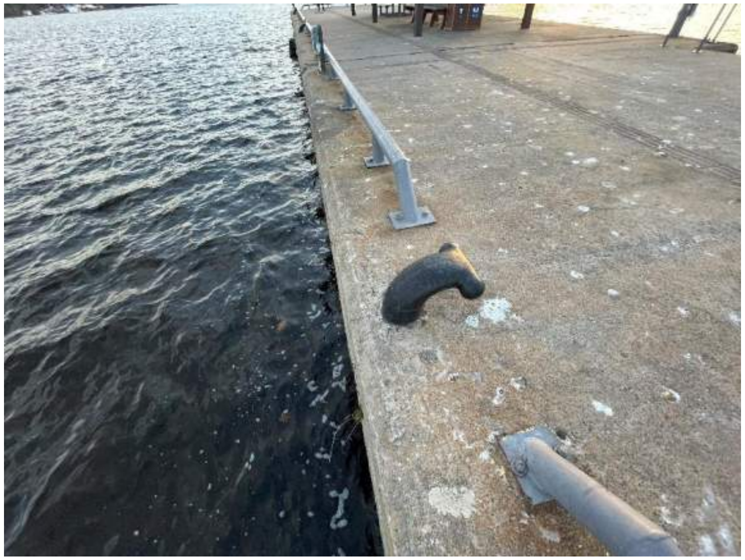
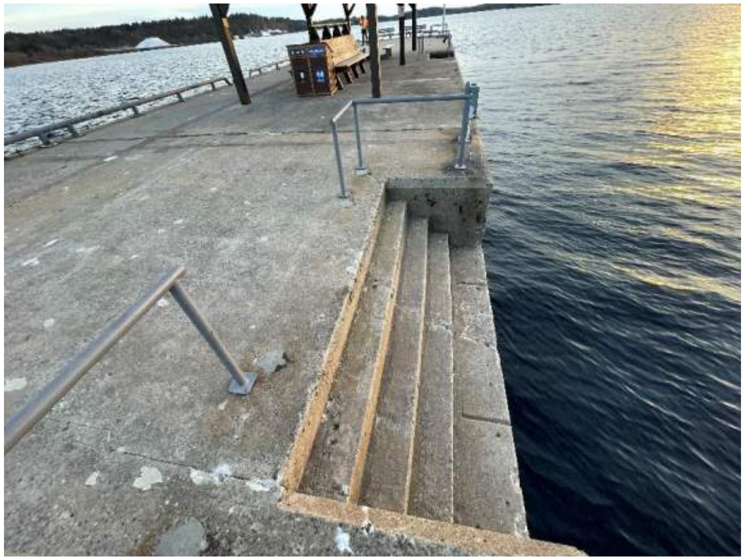


121- Structure D – East Bollard, Stn 0+236

122- Structure D – East Bollard, Stn 0+236



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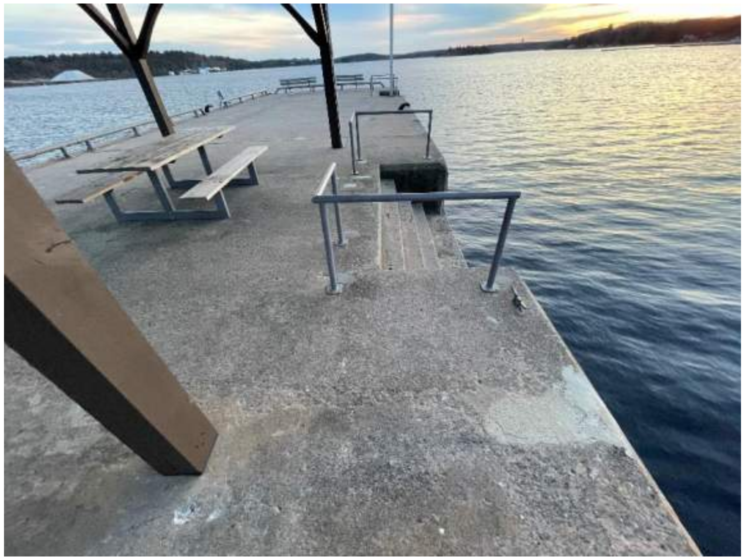

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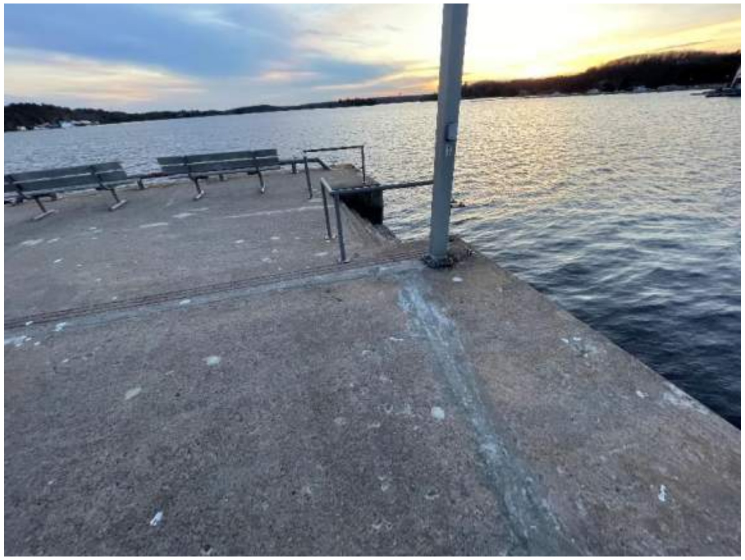

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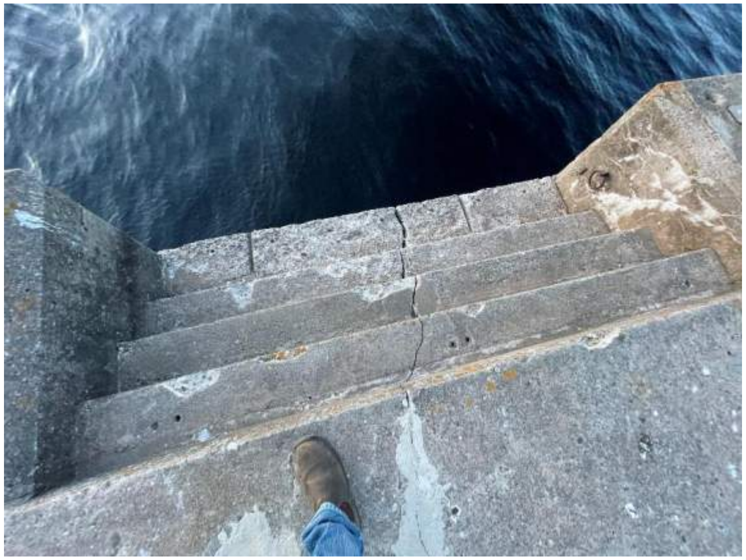

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Appendix C

Underwater Inspection Report

DOCK INSPECTION & EVALUATION
PARRY SOUND DOCK
PARRY SOUND, ONTARIO

DRAFT

Prepared for:
Aecom

Prepared by:
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479 Burbrook Street
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N5W 4B5

December 2023 WSI 23189

NOTES

1. INSPECTION COMPLETED IN DECEMBER 2023.

AECOM

WHARF INSPECTION

KEY MAP

23189

DECEMBER 2023

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1. INTRODUCTION

WATECH SERVICES INC. was retained by Aecom to carry out an inspection and evaluation of the existing dock structure at the Parry Sound Dock in Parry Sound, Ontario. The inspection involved above and below water inspections of the existing dock wall for a distance of approximately 425 metres.

2. PROCEDURE

2.1. General

Inspection Team:	4-person crew
Location:	Parry Sound, Ontario
Date:	December 20-21, 2023
Weather:	-10°C, Cloudy

2.2. Procedure

Prior to beginning the inspection, linear chainage was marked on the structure in order to provide location control for the inspection results. The north face of the structure at the rock wall shoreline was marked as Station 0+000 for the start of the inspection. Our inspection team then marked 10 metre intervals around the perimeter of the main wharf structure.

The maximum water depth at the time of the inspection was 5.7 metres.
Above water photographs were obtained by the inspection diver and by surface team members working from a workboat and from a diver floating on the water.

Water depth soundings were obtained at intervals along the face of the dock using a survey rod. The water level was referenced to the DFO Parry Sound Water Level Gauge located on the dock. The soundings shown on Figure 1 are referenced below chart datum.

The underwater inspection was completed by a robotic camera (ROV) to provide photo & video documentation of the wharf structure interior and exterior. A diver floating on the water with a digital still camera was also used to obtain photographs of the sides of the wharf structure where possible.

3. OBSERVATIONS AND INSPECTION RESULTS

3.1 General

The marine facility known as the “Town Dock” in Parry Sound, Ontario consists of a timber, sheet pile, and concrete structure for the docking of commercial and recreational vessels.

The wharf is constructed with a vertical driven timber face with horizontal walers and tie rods that extend from one side of the wharf to the other. Station 0+000 to -0+076 consists of sheet pile. The deck of the wharf structure is cast in place concrete.

3.2 Timber Wharf Face

The exterior wharf face consists of driven 350mm squared timber piles. A steel C channel waler with tie rods is noted near the top of the wharf face. The timbers are driven relatively close together with additional timber pieces used to close the gap between the piles. Within the timber face walls it is likely that timber pile bents are also in place. The spacing and configuration of the bents was only visible at one location where a timber face pile was missing (Station 0+018).

The top of the individual timber face piles are typically below the water by 0.6 to 1.3 metres depending on the water level. As the piles are not exposed to air the wood is generally sound. The smaller fill pieces between the piles are typically somewhat rotted and split.

The tie back waler consists of a 150mm steel C channel. Tie rods are spaced approximately 2.5 to 3.0 metres apart. The tie rods are 40mm in diameter and appear in fair to good condition with some corrosion noted. The C channel steel is somewhat pitted and corroded. The connections appear to be intact and in relatively fair to good condition.

Timber penetration depths were measured at random locations on the east face to document the condition of the timber piles that make up most of the dock structure. At each location, 3 readings were obtained, and the average of the readings was the recorded measurements. Marine growth was removed as necessary to obtain true results.

The following table indicates the station/location and the average reading.

Station	Average Measurement (mm)
0+000	1.6
0+005	1.4
0+010	1.5
0+016	1.5
0+024	Up to 51.0mm due to splits
0+035	1.3
0+046	1.4
0+052	12.7
0+060	1.6
0+069	6.4
0+075	1.7
0+082	1.8
0+092	1.6
0+104	1.6
0+113	Timbers split all the way though (101.6mm)
0+124	1.5
0+132	1.6
0+140	12.7
0+149	12.7

3.3 Concrete Deck

The deck of the wharf structure is constructed from cast in place concrete. The concrete deck rests on the face wall piles and is believed to be supported by internal timber pile bents. The deck is in fair condition. Minor but frequent cracking of the deck is noted throughout the structure. The deck appears to have been overlaid at some point and the cracks are reflective of underlying conditions. The cracks are generally tight, and no significant movement or displacement is noted across the cracks.

Minor spalling and chipping of the deck is noted at a few locations. In general, the patches that have been completed to date appear in fair condition and the deck is level with no significant trip hazards. Utility trenches and patches in the concrete deck are noted. The utility deck patches are level and also generally appear sound.

Two weeks prior to the inspection, nine (9) core holes were drilled into the deck to allow access to the underside of the deck for visual inspection. All 9 holes were patched following the drilling operations. The concrete cores determined that the deck is approximately 508mm thick between station 0+000 and 0+150 and at some point there was most likely an overlay of an older deck. The deck thickness between station 0+000 and -0+125 was approximately 350mm thick in accordance with previous reports. The Aecom representative on site obtained their own videos through the deck following the drilling operations.

3.4 Concrete Copewalls

The vertical face concrete copewalls consist of cast in place concrete constructed integrally with the concrete deck. The copewalls are in poor to fair condition. Numerous cracks and spalls are evident in the concrete copewalls. The corners of the wharf structure are notably cracked and eroded (see Photographs 165, 166 and 170 in the above water photographs).

The deterioration of the copewall face on the east side is the worst at the location of the tie up bollards (see Photographs 115-116, 122, 128-131, 141 to 148, 154-155, 160 and 163 in the above water photographs). The bollard concrete appears to have been stressed and is heavily cracked and spalled.

Significant cracks and spalls are also noted at the stairwells on the west side of the wharf (see Photographs 173, 175, 179-180, 183-185, 188-189, 191, 194-195, 198, and 204 in the above water photographs). The stair treads are generally in better condition and have been patched where required.

3.5 Ultrasonic Metal Thickness Measurements

Steel sheet pile makes up two sections of the dock. Station 0+000 to -0+076 along the east wall is made up of steel sheet pile with a concrete cap. The north wall perpendicular to the north end of the dock is also made up of sheet pile. This can be seen on Figure 1.

Ultrasonic metal thicknesses were measured at random locations to document the corrosion in the steel sheet piling. At each location, 3-5 readings were obtained, and the average of the readings was the recorded measurements. Marine growth was removed as necessary to obtain good contact with the transducer. Obtaining ultrasonic thickness measurements between station -0+002 and -0+052 was not possible due to large vessels blocking the remaining sheet pile.

The following table indicates the station/location and the average reading.

Location/Station	Average Thickness (mm)
0+000	8.0
-0+002	8.2
-0+052	8.5
-0+054	8.4
-0+057	8.5
-0+060	6.3
-0+062	5.4
-0+064	6.5
-0+067	8.9

-0+070	7.2
-0+072	7.5
North wall - east end	4.0
North wall - 7 metres from the east end	4.5
North wall - 7 metres from the west end	4.0
North wall - west end	3.9

3.6 Internal Inspection of Wharf Structure

The inspection could not confirm the dock pile bent section construction due to the high backfill level. Figures 2-4 show a typical section view of certain locations throughout the structure. The drawings show the high level of backfill on the south half of the dock.

Two holes were cut into the timbers at certain locations to allow the ROV to enter underneath the wharf structure to determine the construction of the bents. The best attempt to gain accurate information was made, but due to high levels of backfill and debris the ROV had limited effectiveness moving throughout the structure. The locations of the holes can be seen on Figure 1. Both holes were patched after the inspection work (see Photographs 131-132 in the underwater photographs).

3.7 Timber Bent Section (Station -0+076 to -0+125)

This section of the dock structure was constructed in 1921 to 1922 and is part of the original construction. This section is approximately 49 metres long and the superstructure is a mass concrete slab on pre-cast concrete footing blocks. The superstructure is supported by a round timber pile bent substructure.

The substructure consists of round timer pile bents with a timber pile cap and timber stringers. Assuming the north end of the dock to be typical construction throughout, we estimate the pile bents to be approximately 1.2 metres apart.

Timber penetration depths were measured at random locations in this section to document the condition of the timber piles that make up most of the dock structure. At each location, 3 readings were obtained, and the average of the readings was the recorded measurements. Marine growth was removed as necessary to obtain true results.

The following table indicates the station/location and the average reading.

Station	Average Measurement (mm)
-0+078	1.8
-0+087	2.2
-0+097	1.8
-0+105	2.1
-0+115	2.2
-0+124	1.9

3.8 Vessel Facilities

Potable water and electrical power is available on both sides of the wharf structure. There are 17 water and power stations on the wharf. The operation of each station was not confirmed; however, they all appear functional.

3.9 Water Level Gauge Station

The water level gauge station located at station 0+016 on the west side of the dock appears to be in generally good condition. A section of "I" beam in this area was noted to be deteriorated.

DOCK INSPECTION
PARRY SOUND, ONTARIO

AECOM

December 2023

Photographs (Above Water)

WATECH SERVICES INC.
WSI 23189

	PHOTO # 1 Overall of the dock structure in Parry Sound looking south
	PHOTO # 2 Looking south along the dock
	PHOTO # 3 Looking north along the dock

PHOTO # 4

Looking north along the dock

PHOTO # 5

0+000 on the west side

PHOTO # 6

Stairs at 0+005 on the west side

PHOTO # 7

Crack at 0+013 on the west side

PHOTO # 8

Crack at 0+013 on the west side

PHOTO # 9

Crack at 0+013 on the west side

PHOTO # 10

Crack at 0+019 on the west side

PHOTO # 11

Light at 0+023 on the west side

PHOTO # 12

0+030 on the west side

PHOTO # 13

Stairs at 0+037 on the west side

PHOTO # 14

Stairs at 0+038 on the west side

PHOTO # 15

Crack at 0+052 on the west side

PHOTO # 16

Crack at 0+052 on the west side

PHOTO # 17

Patch at 0+056 on the west side

PHOTO # 18

0+061 on the west side

PHOTO # 19

Corner of stairs at 0+067

PHOTO # 20

0+072 on the west side

PHOTO # 21

Crack at 0+074 on the west side

PHOTO # 22

Crack at 0+074 on the west side

PHOTO # 23

Crack at 0+077 on the west side

PHOTO # 24

Crack at 0+077 on the west side

PHOTO # 25

Crack at 0+077 on the west side

PHOTO # 26

Crack at 0+082 on the west side

PHOTO # 27

Crack at 0+082 on the west side

PHOTO # 28

Crack at 0+082 on the west side

PHOTO # 29

Crack at 0+098 on the west side

PHOTO # 30

Crack at 0+098 on the west side

PHOTO # 31

Stairs at 0+099 on the west side

PHOTO # 32

Stairs at 0+099 on the west side

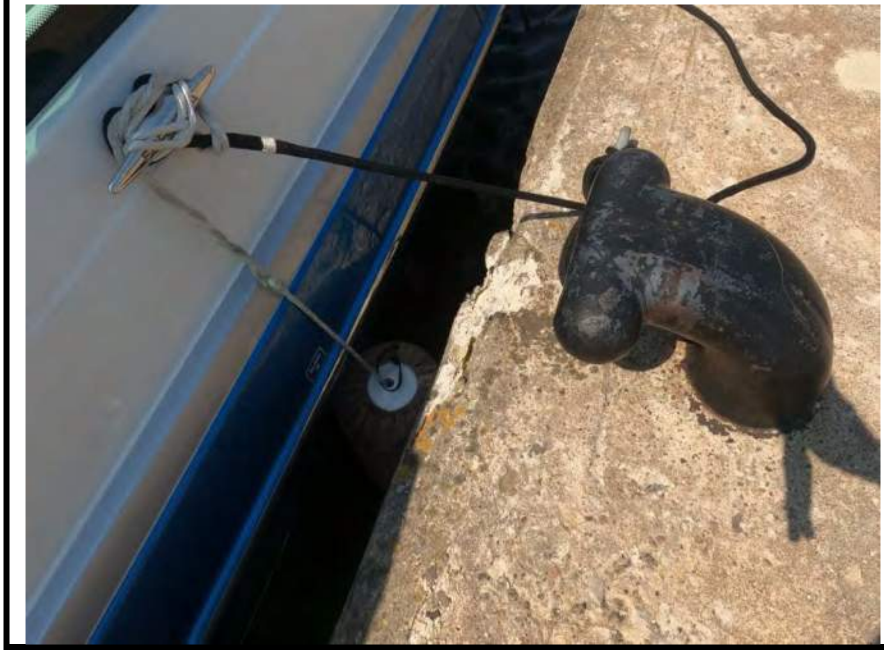


PHOTO # 33

Bollard at 0+107 on the west side

PHOTO # 34

Crack at 0+113 on the west side

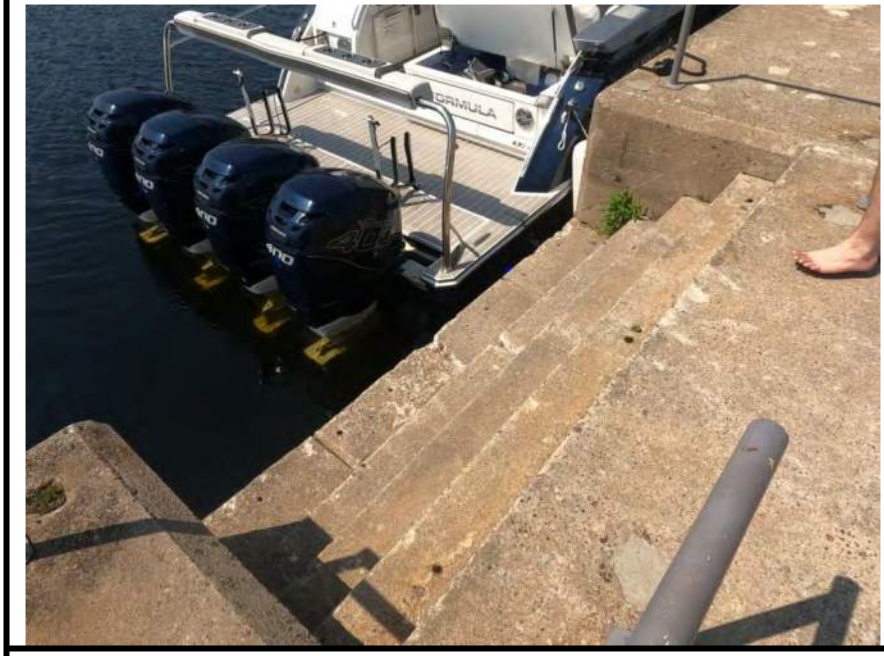


PHOTO # 35

Stairs at 0+114 on the west side

PHOTO # 36

0+118 on the west side

PHOTO # 37

Crack at 0+123 on the west side

PHOTO # 38

Crack at 0+123 on the west side

PHOTO # 39

Patch at 0+124 on the west side

PHOTO # 40

Patch at 0+124 on the west side

PHOTO # 41

Crack at 0+127 on the west side

PHOTO # 42

Crack at 0+127 on the west side

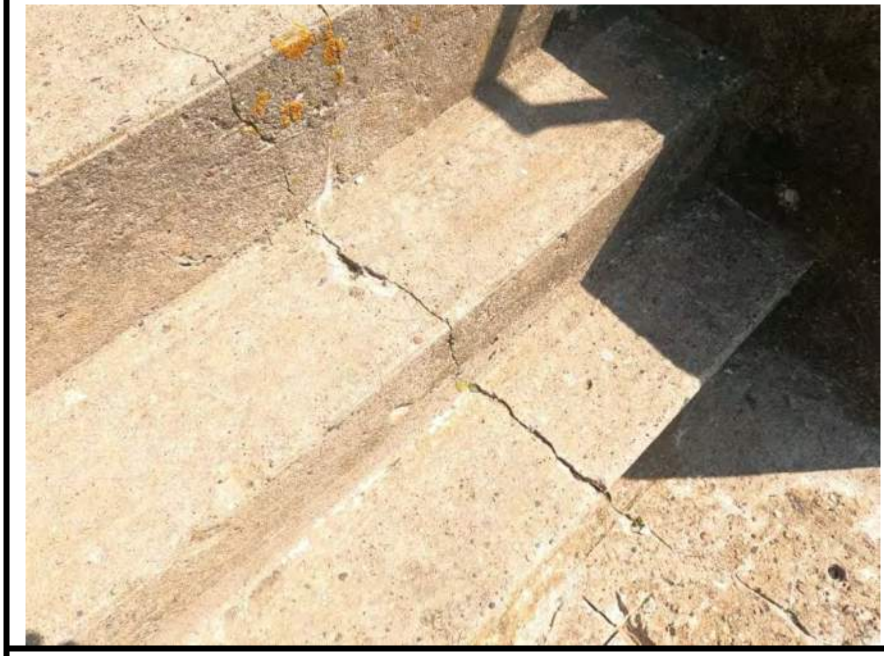


PHOTO # 43

Crack on stairs at 0+131 on the west side

PHOTO # 44

Crack on stairs at 0+131 on the west side

PHOTO # 45

Spalling at 0+132

PHOTO # 46

Spalling at 0+136 on the west side

PHOTO # 47

Spalling at 0+136 on the west side

PHOTO # 48

Spalling at 0+138 on the west side

PHOTO # 49

Bollard at 0+138 on the west side

PHOTO # 50

Patch at 0+141 on the west side

PHOTO # 51

Crack at 0+142 on the west side

PHOTO # 52

0+143 on the west side

PHOTO # 53

Calcite and cracking 0+144

PHOTO # 54

Crack at 0+144 on the west side

PHOTO # 55

Crack at 0+146 on the west side

PHOTO # 56

Crack at 0+146 on the west side

PHOTO # 57

Crack at 0+146 on the west side

PHOTO # 58

Crack at 0+146 on the west side

PHOTO # 59

0+149 on the west side

PHOTO # 60

Southeast corner

PHOTO # 61

Crack on deck at 0+0145 on the east side

PHOTO # 62

Bollard at 0+142 on the east side

PHOTO # 63

Looking north from 0+136 on the east side

PHOTO # 64

Typical core hole after patching

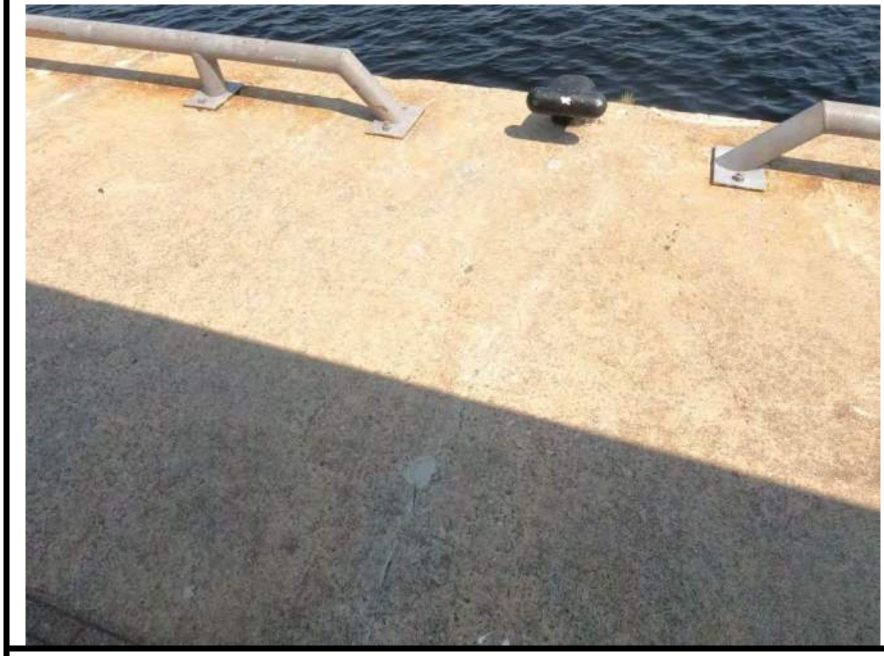


PHOTO # 65

Crack and bollard at 0+127 on the east side

PHOTO # 66

Crack at 0+124 on the east side

PHOTO # 67

Bollard at 0+112 on the east side

PHOTO # 68

Crack and patch 0+110 on the east side

PHOTO # 69

Patch at 0+102 service on the east side

PHOTO # 70

Bollard at 0+096 on the east side

PHOTO # 71

Crack at 0+097 on the east side

PHOTO # 72

Crack at 0+097 on the east side

PHOTO # 73

Spalling at 0+065 on the east side

PHOTO # 74

Spalling at 0+065 on the east side

PHOTO # 75

Patch at 0+051 on the east side

PHOTO # 76

Patch at 0+042 on the east side

PHOTO # 77

Patch at 0+041 on the east side

PHOTO # 78

Bollard at 0+035 on the east side

PHOTO # 79

0+018 on the east side

PHOTO # 80

Bollard at 0+001 on the east side

PHOTO # 81

Overview of the dock from the south looking north

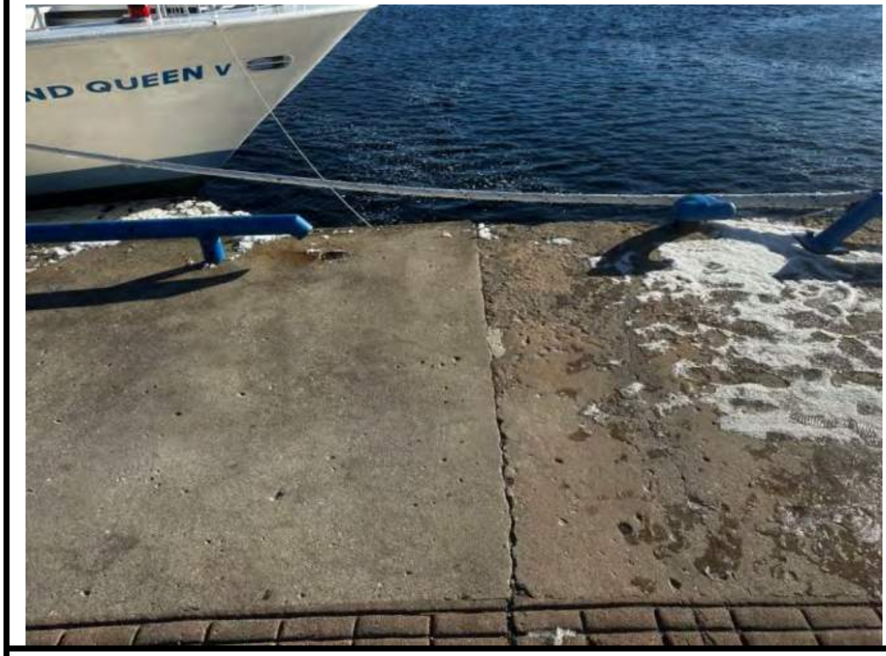


PHOTO # 82

Joint at 0+000

PHOTO # 83

Start of railing

PHOTO # 84

Hydro station #23 at
-0+007 looking north

PHOTO # 85

Typical core hole after patching

PHOTO # 86

Typical core hole after patching

PHOTO # 87

Bollard at -0+014

PHOTO # 88

Overview looking north from -0+014

PHOTO # 89

Cracking in concrete at -0+018

PHOTO # 90

Hydro station #23 at -0+019

PHOTO # 91

Concrete cracking at -0+029

PHOTO # 92

Bollard at -0+032

PHOTO # 93

Overview looking south at -0+032

PHOTO # 94

Hydro station #23 at -0+040

PHOTO # 95

Crack in concrete at -0+042

PHOTO # 96

Bollard at -0+051

PHOTO # 97

Life ring at -0+053

PHOTO # 98

Electrical box #26 at -0+057

PHOTO # 99

Bollard at -0+069

PHOTO # 100

Overview looking south from -0+069

PHOTO # 101

Overview looking north from -0+069

PHOTO # 102

Crack and spalling at -0+076

	PHOTO # 103 Bollard at -0+078
	PHOTO # 104 Spalling at -0+081
	PHOTO # 105 Bollard at -0+091

PHOTO # 106

Overview looking north at end of dock

PHOTO # 107

Northeast face of the sheet pile wall

PHOTO # 108

Typical condition of sheet pile wall

PHOTO # 109

Close up view of sheet pile wall

PHOTO # 110

North end of the dock from the east side

PHOTO # 111

North end of the dock from the east side

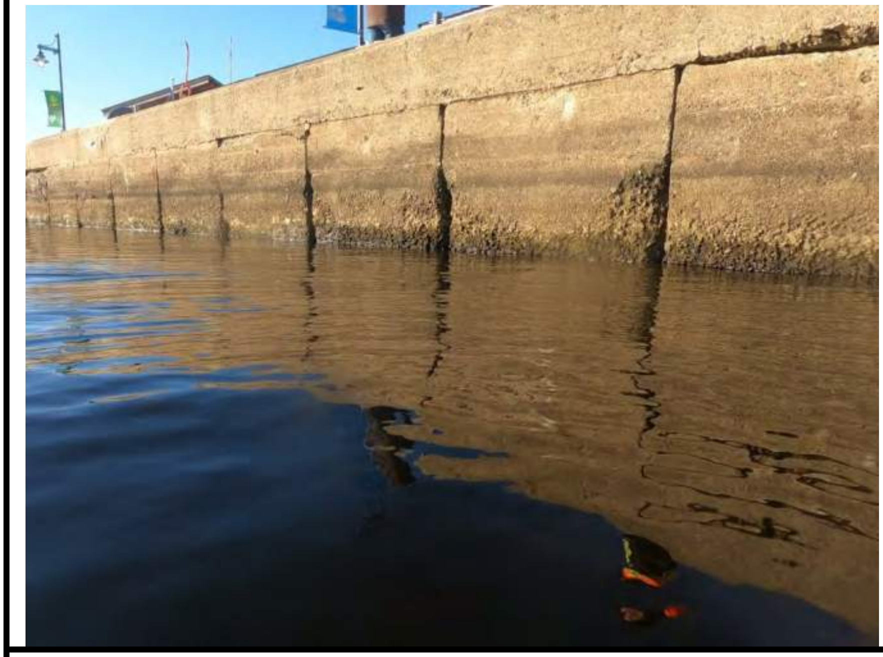


PHOTO # 112

Spalling at -0+109

PHOTO # 113

Spalling at -0+109

PHOTO # 114

Split in joint at -0+106

PHOTO # 115

Concrete repair underneath bollard at -0+103

PHOTO # 116

Spalling at -0+099

PHOTO # 117

Typical concrete condition at -0+098

PHOTO # 118

Typical concrete condition at -0+098

PHOTO # 119

Concrete delamination at -0+095.1

PHOTO # 120

Looking north from -0+094

PHOTO # 121

Looking south from
-0+094

PHOTO # 122

Spalling at 0+091

PHOTO # 123

Spalling at 0+090

PHOTO # 124

Typical concrete condition at -0+087

PHOTO # 125

Outfall at -0+083

PHOTO # 126

Outfall at -0+083

PHOTO # 127

Close up of outfall at -0+083

PHOTO # 128

Concrete spalling and delamination next to the outfall at -0+083

PHOTO # 129

Typical condition of concrete at -0+079

PHOTO # 130

Typical condition of concrete underneath bollard at -0+078

PHOTO # 131

Spalling at -0+076

PHOTO # 132

Opening in concrete at -0+076

PHOTO # 133

Looking inside opening at -0+076

PHOTO # 134

Looking inside opening at -0+076

PHOTO # 135

Typical condition of sheet pile at -0+073

PHOTO # 136

Typical condition of sheet pile at -0+069

PHOTO # 137

Typical condition of sheet pile at -0+063

PHOTO # 138

Typical condition of sheet pile at -0+063

PHOTO # 139

Ladder at -0+057

PHOTO # 140

Looking south showing boats blocking the inspection area

PHOTO # 141

Cracking and spalling at 0+000 on the east face

PHOTO # 142

Cracking and spalling at 0+010 on the east face

PHOTO # 143

Cracking and spalling at 0+014 on the east face

PHOTO # 144

Spalling at 0+036 on the east face

PHOTO # 145

Spalling at 0+040 on the east face

PHOTO # 146

Spalling at 0+049 on the east face

PHOTO # 147

0+054 on the east face

PHOTO # 148

Spalling at 0+060 on the east face

PHOTO # 149

0+066 on the east face

PHOTO # 150

Cracking and spalling at 0+072 on the east face

PHOTO # 151

Cracking and spalling at 0+077 on the east face

PHOTO # 152

0+082 on the east face

PHOTO # 153

0+088 on the east face

PHOTO # 154

Cracking and spalling at 0+092 on the east face

PHOTO # 155

Cracking and spalling at 0+096 on the east face

PHOTO # 156

Cracking and spalling at 0+099 on the east face

PHOTO # 157

Cracking and spalling at 0+102 on the east face

PHOTO # 158

Cracking and spalling at 0+112 on the east face

PHOTO # 159

Cracking and spalling at 0+118 on the east face

PHOTO # 160

Cracking and spalling at 0+127 on the east face

PHOTO # 161

Cracking and spalling at 0+127 on the east face

PHOTO # 162

Cracking and spalling at 0+135 on the east face

PHOTO # 163

Cracking and spalling at 0+142 on the east face

PHOTO # 164

Cracking and spalling at 0+146 on the east face

PHOTO # 165

Cracking and spalling at 0+149 on the east face

PHOTO # 166

Cracking and spalling at 0+150 on the southeast corner

PHOTO # 167

Cracking and spalling on the south face

PHOTO # 168

Cracking and spalling on the south face

PHOTO # 169

Cracking and spalling on the south face

PHOTO # 170

Cracking and spalling at 0+150 on the southwest corner

PHOTO # 171

Cracking at 0+150 on the southwest corner

PHOTO # 172

Cracking at 0+150 on the southwest corner

PHOTO # 173

Cracking at 0+149 on the west face

PHOTO # 174

Cracking on the west face at the stairs at 0+145

PHOTO # 175

Cracking at 0+144 on the west face

PHOTO # 176

Cracking and spalling at 0+142 on the west face

PHOTO # 177

Cracking and spalling at 0+138 on the west face

PHOTO # 178

Cracking and spalling at 0+135 on the west face

PHOTO # 179

Cracking and spalling at 0+132 on the west face

PHOTO # 180

Cracking and spalling at 0+128 on the west face

PHOTO # 181

Cracking and spalling at 0+123 on the west face

PHOTO # 182

Cracking at 0+118 on the west face

PHOTO # 183

Cracking and spalling at 0+116 on the west face

PHOTO # 184

Cracking at 0+101 on the west face

PHOTO # 185

Cracking and spalling at 0+097 on the west face

PHOTO # 186

Cracking at 0+092 on the west face

PHOTO # 187

Cracking and spalling at 0+088 on the west face

PHOTO # 188

Cracking and spalling at 0+086 on the west face

PHOTO # 189

Cracking and spalling at 0+080 on the west face

PHOTO # 190

Cracking and spalling at 0+075 on the west face

PHOTO # 191

Cracking and spalling at 0+052 on the west face

PHOTO # 192

Cracking and spalling at 0+047 on the west face

PHOTO # 193

Cracking and spalling at 0+040 on the west face

PHOTO # 194

Cracking and spalling at 0+037 on the west face

PHOTO # 195

Cracking and spalling at 0+035 on the west face

PHOTO # 196

Cracking at 0+030 on the west face

PHOTO # 197

Cracking and spalling at 0+026 on the west face

PHOTO # 198

Cracking and spalling at 0+018 on the west face

PHOTO # 199

Close up of spalling and looking behind water level gauge station at 0+017

PHOTO # 200

Cracking and spalling at 0+017 on the west face

PHOTO # 201

Water level gauge station at 0+016

PHOTO # 202

Cracking and spalling at 0+013 on the west face

PHOTO # 203

Cracking and spalling at 0+007 on the west face

PHOTO # 204

Cracking at 0+002 on the west face

DOCK INSPECTION
PARRY SOUND, ONTARIO

AECOM

December 2023

Photographs (Underwater)

WATECH SERVICES INC.
WSI 23189

PHOTO # 1

"I" beam at water level station (west face)

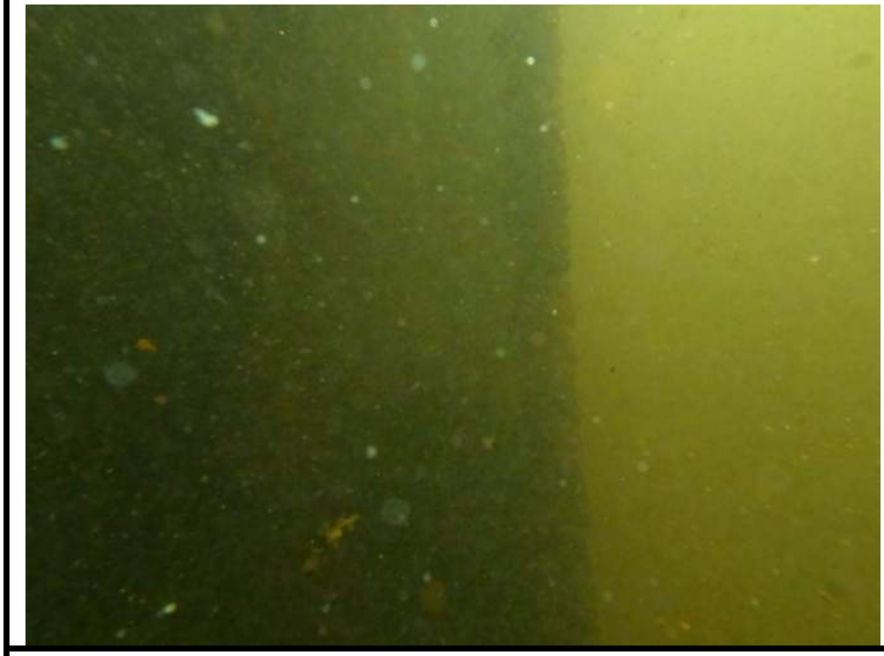


PHOTO # 2

"I" beam at water level station (west face)

PHOTO # 3

"I" beam at water level station (west face)

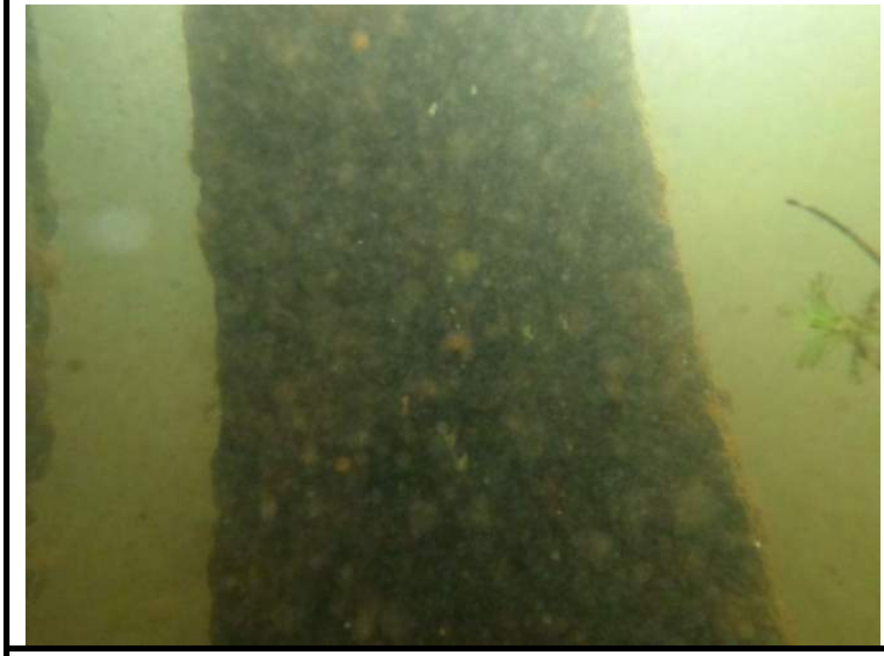


PHOTO # 4

"I" beam on water level station (west face)

PHOTO # 5

"C" channel at 0+020 (west face)

PHOTO # 6

"C" channel at 0+032 (west face)

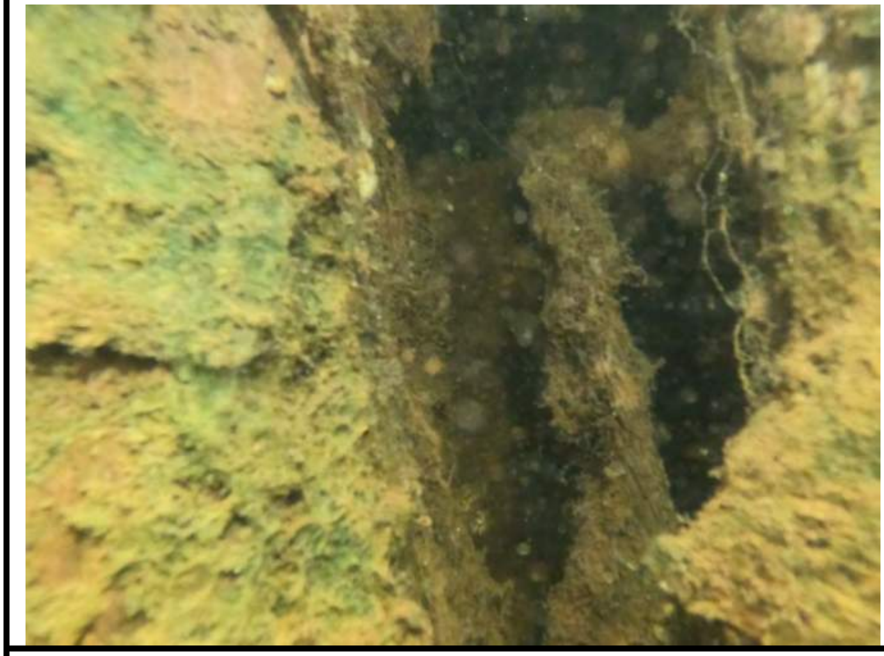


PHOTO # 7

Deteriorated timber at 0+035 (west face)

PHOTO # 8

Deteriorated timber at 0+035 (west face)

PHOTO # 9

"C" channel at 0+040 (west face)

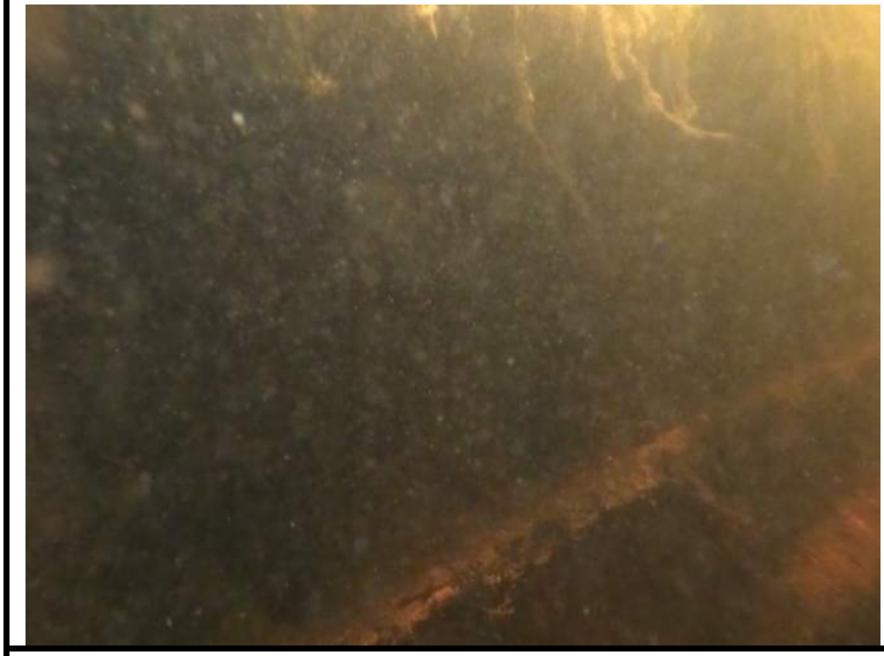


PHOTO # 10

"C" channel at 0+050 (west face)

PHOTO # 11

Piles at bottom at 0+050 (west face)

PHOTO # 12

Piles at bottom 0+050 (west face)

PHOTO # 13

Tie back bolt at 0+069 (west face)

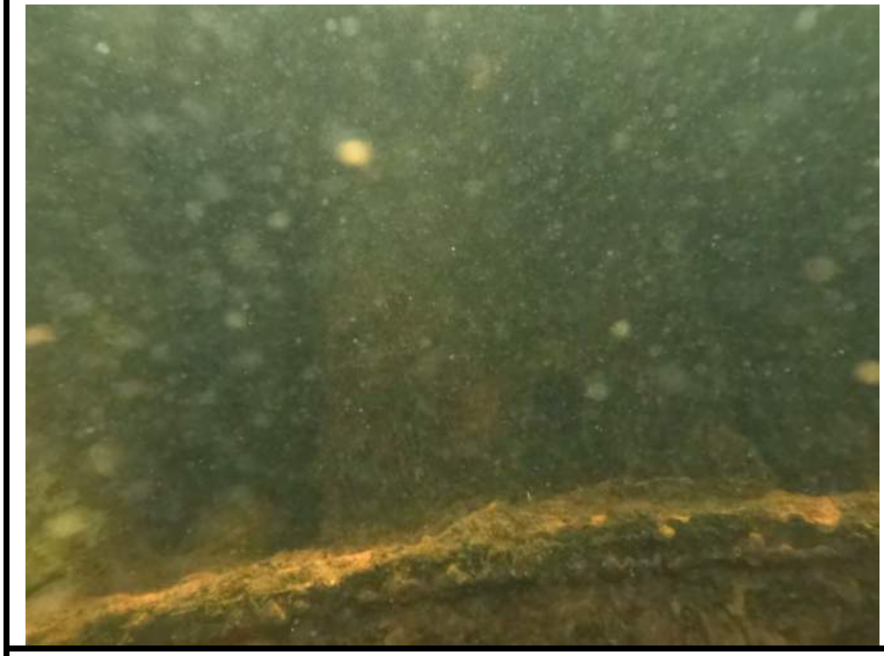


PHOTO # 14

Tie back bolt 0+069 (west face)

PHOTO # 15

"C" channel 0+074 (west face)

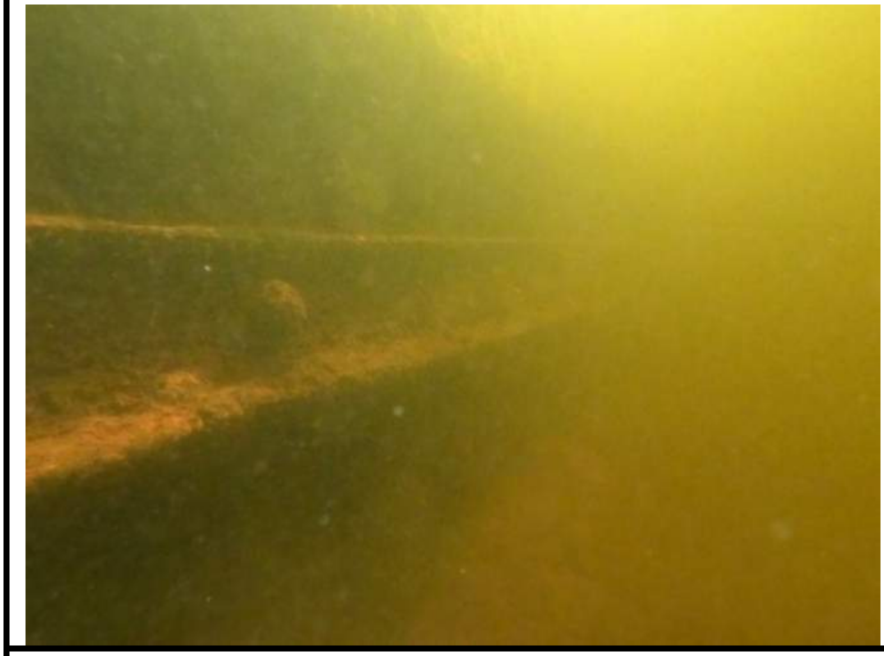


PHOTO # 16

"C" channel at 0+076 (west face)

PHOTO # 17

Pile meeting concrete cap at 0+077 (west face)

PHOTO # 18

Pile meeting concrete cap at 0+079 (west face)

PHOTO # 19

Piles meeting concrete cap at 0+080 (west face)

PHOTO # 20

Piles meeting concrete cap at 0+081 (west face)

PHOTO # 21

C channel at 0+082 (west face)

PHOTO # 22

"C" channel at 0+083
(west face)

PHOTO # 23

Piles at bottom at 0+083
(west face)

PHOTO # 24

Piles at bottom at 0+084
(west face)

PHOTO # 25

Piles at bottom at 0+085 (west face)

PHOTO # 26

"C" channel at 0+085 (west face)

PHOTO # 27

"C" channel at 0+086 (west face)

PHOTO # 28

"C" channel at 0+088
(west face)

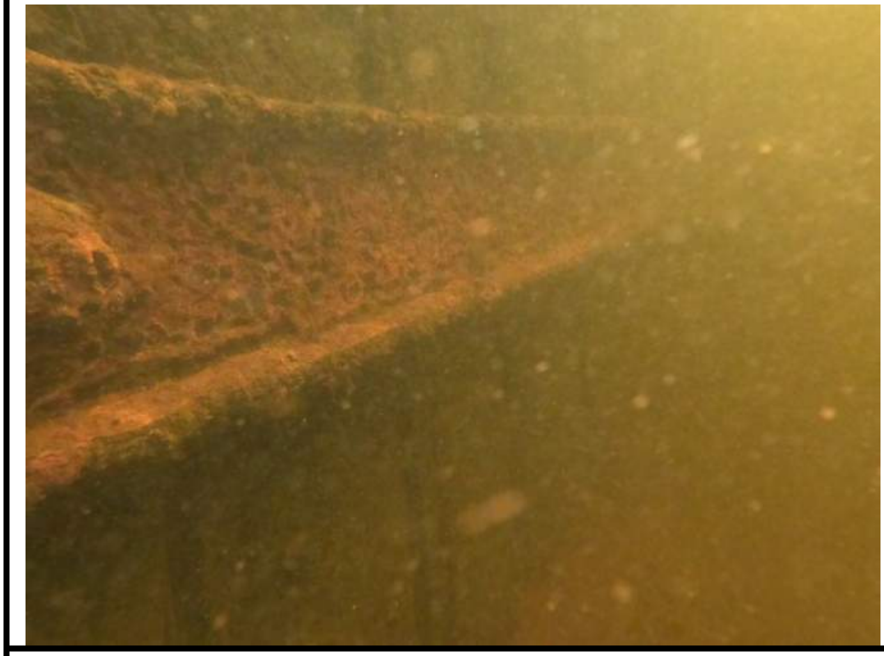


PHOTO # 29

"C" channel at 0+090
(west face)

PHOTO # 30

Piles at bottom at 0+091
(west face)

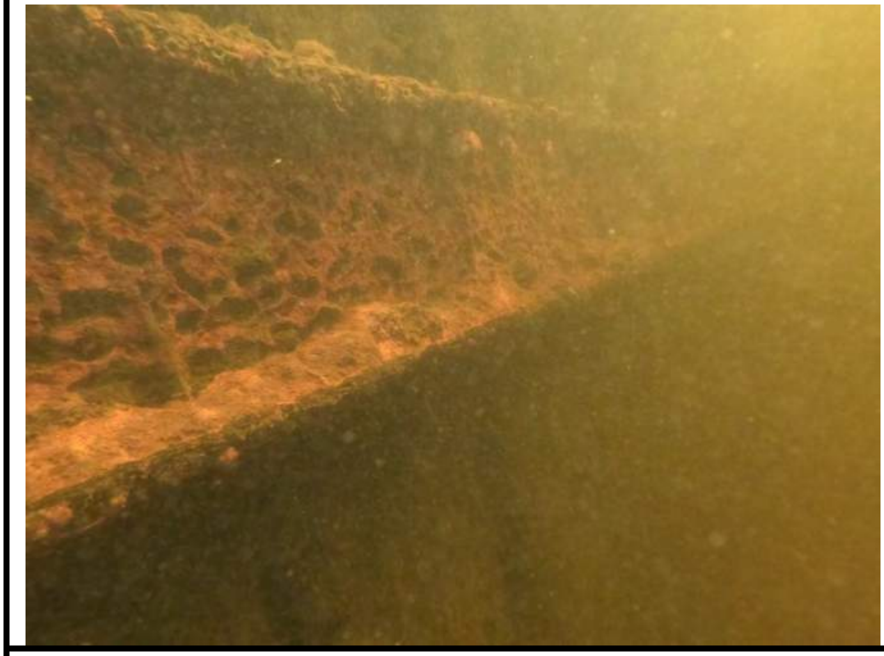


PHOTO # 31

"C" Channel at 0+092 (west face)

PHOTO # 32

Top of "C" channel 0+093 (west face)

PHOTO # 33

Typical timber condition at 0+095 (west face)

PHOTO # 34

"C" channel at 0+095 (west face)

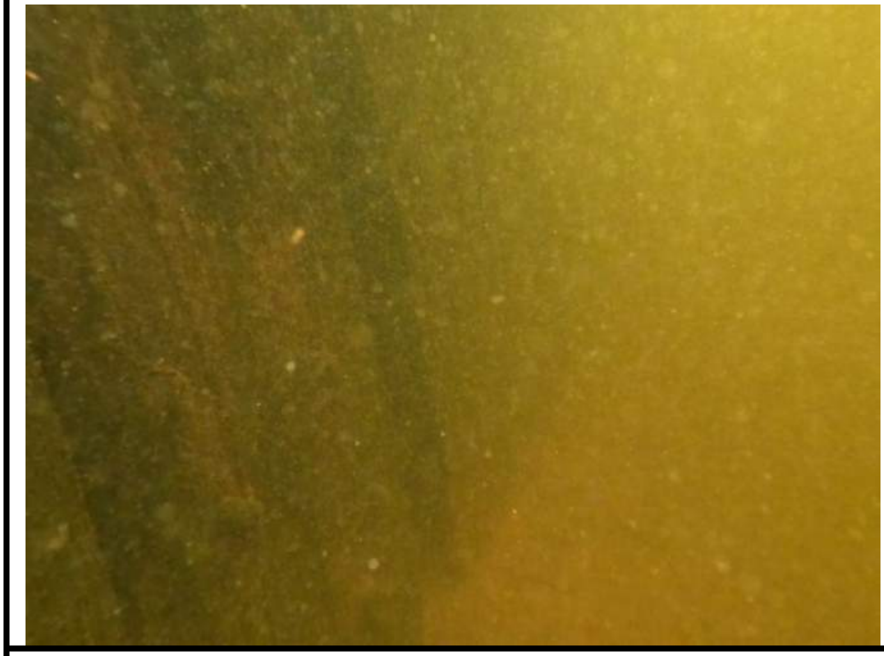


PHOTO # 35

Piles at bottom at 0+100 (west face)

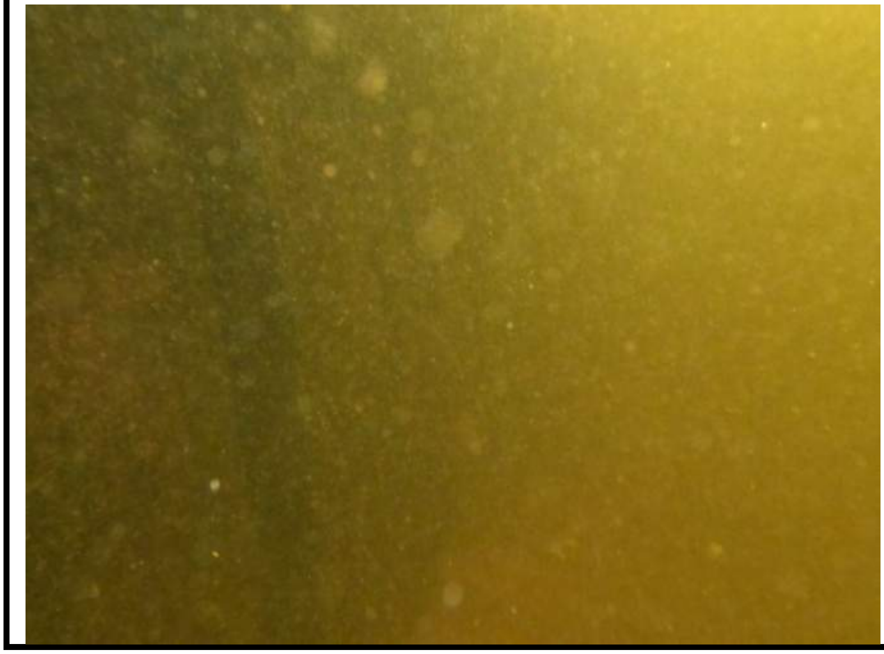


PHOTO # 36

Piles at bottom at 0+105 (west face)

PHOTO # 37

Tie back bolt at 0+108 (west face)

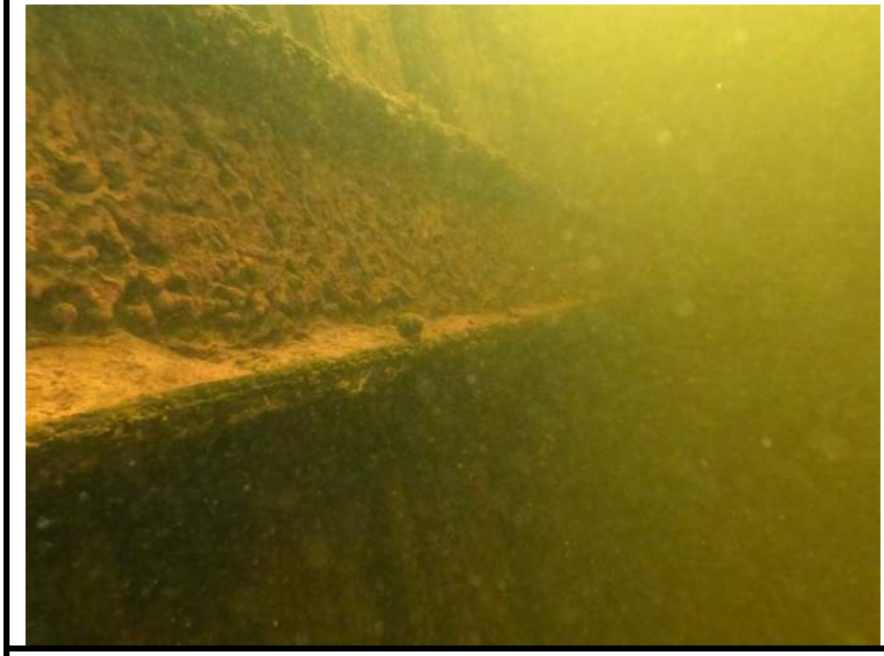


PHOTO # 38

"C" channel at 0+109 (west face)

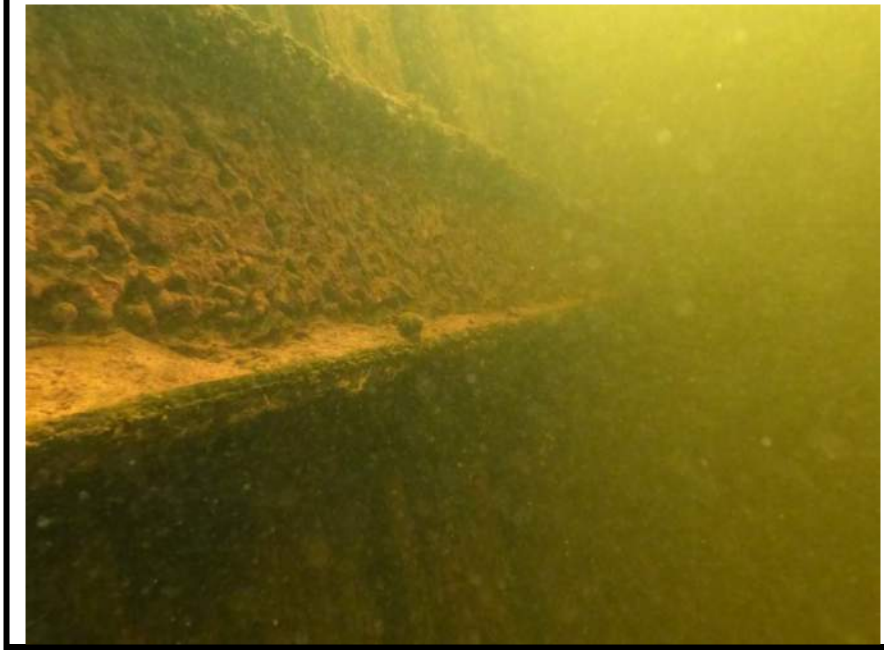


PHOTO # 39

"C" channel at 0+110 (west face)

PHOTO # 40

"C" channel 0+113 (west face)

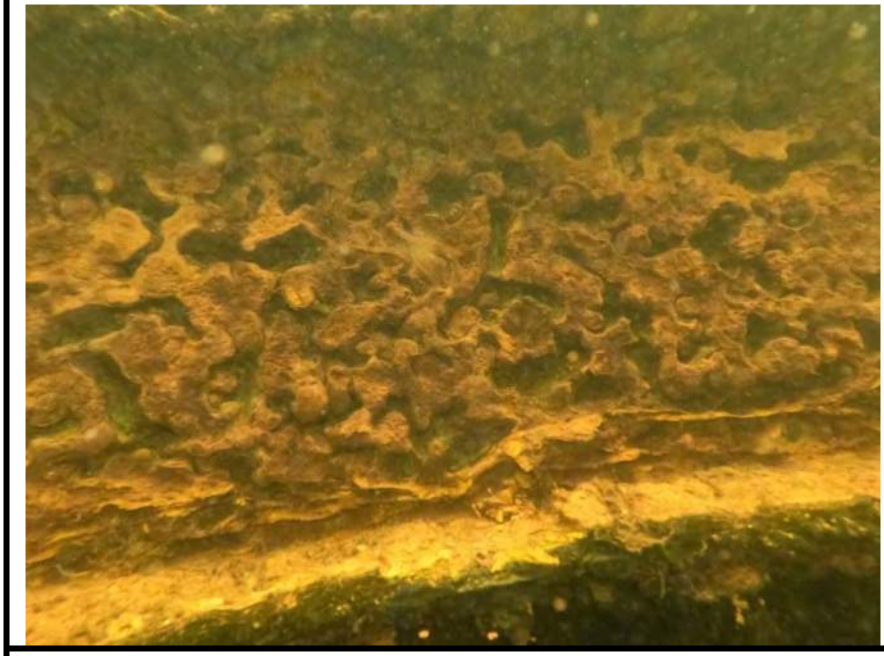


PHOTO # 41

"C" channel at 0+114 (west face)

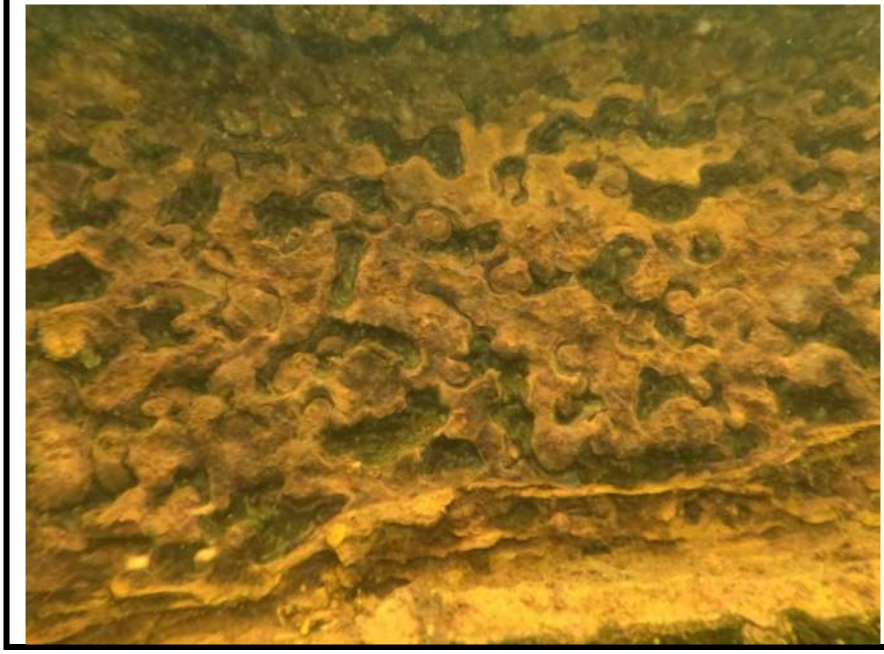


PHOTO # 42

Close up of typical condition of "C" channel at 0+118 (west face)

PHOTO # 43

Piles at bottom at 0+119 (west face)

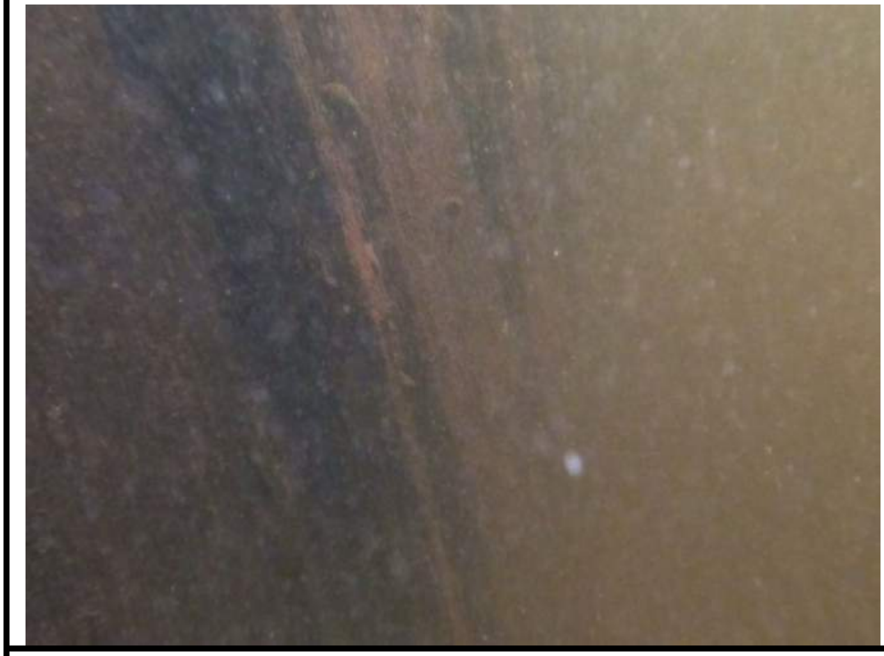


PHOTO # 44

Piles at bottom at 0+120 (west face)

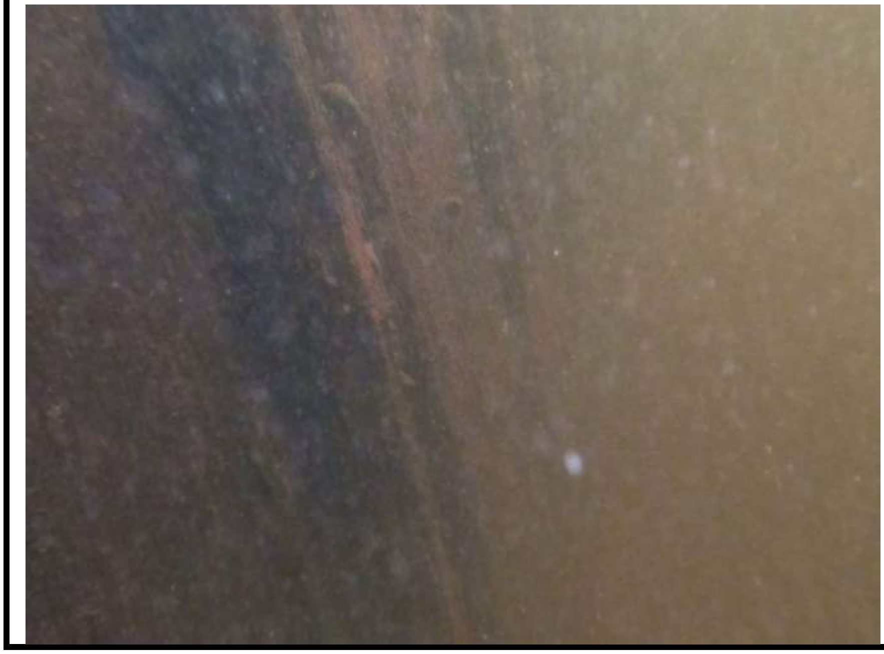


PHOTO # 45

Piles at bottom at 0+124 (west face)

PHOTO # 46

"C" channel at 0+135
(west face)

PHOTO # 47

"C" channel at 0+135
(west face)

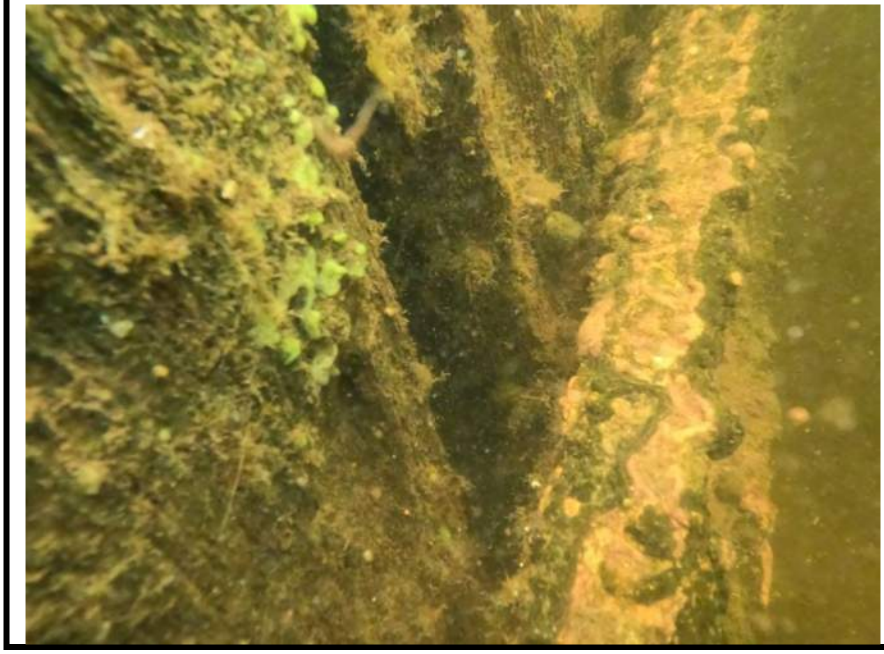


PHOTO # 48

"C" channel at 0+140
(west face)

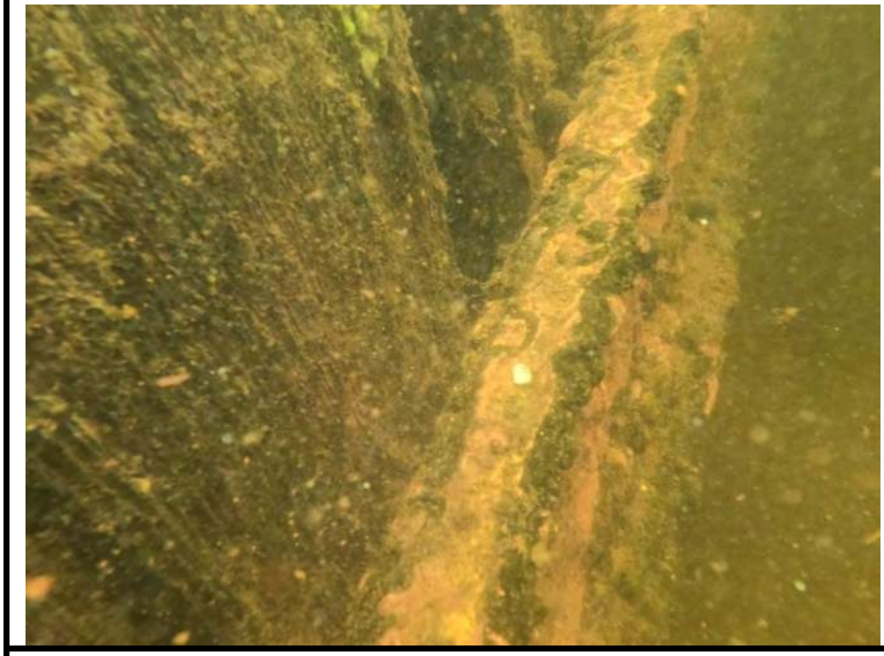


PHOTO # 49

"C" channel at 0+140
(west face)

PHOTO # 50

Piles at bottom at 0+148
(east face)

PHOTO # 51

Piles at bottom at 0+140
(east face)

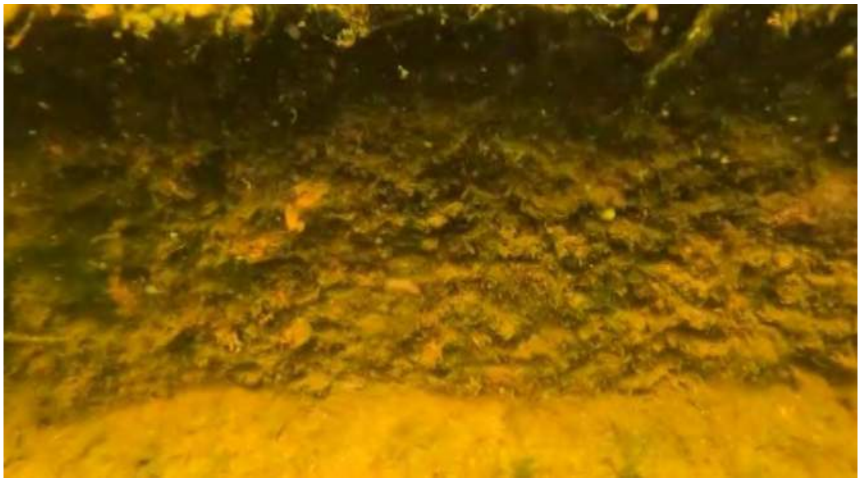


PHOTO # 52

Close up of typical condition of "C" channel at 0+137 (east face)

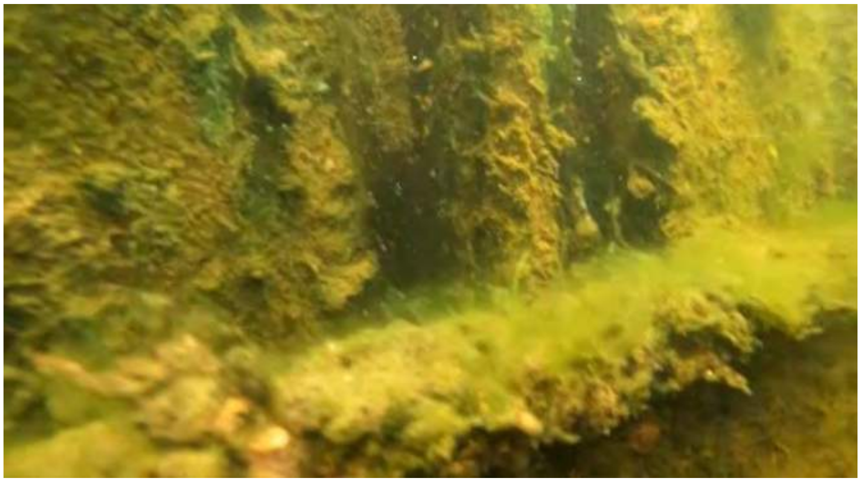


PHOTO # 53

Top of "C" channel 0+135 (east face)

PHOTO # 54

Piles at bottom at 0+133 (east face)

PHOTO # 55

Close up of typical condition of "C" channel at 0+130 (east face)

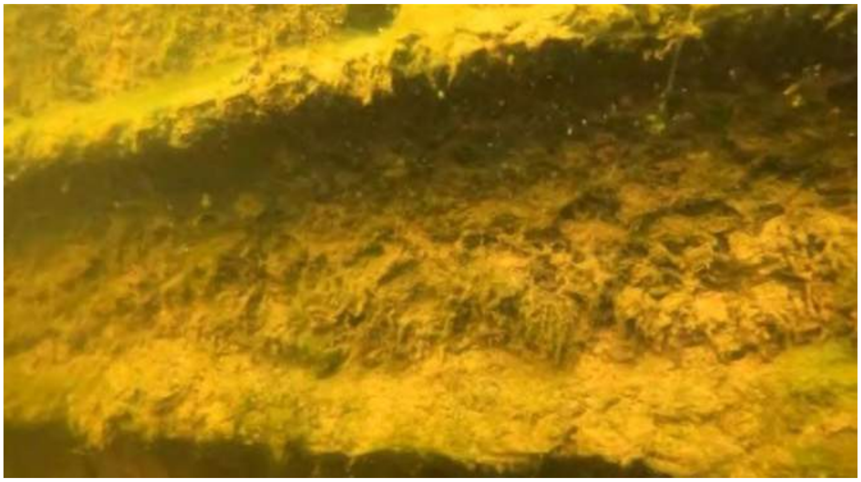


PHOTO # 56

Close up of typical condition of "C" channel at 0+128 (east face)

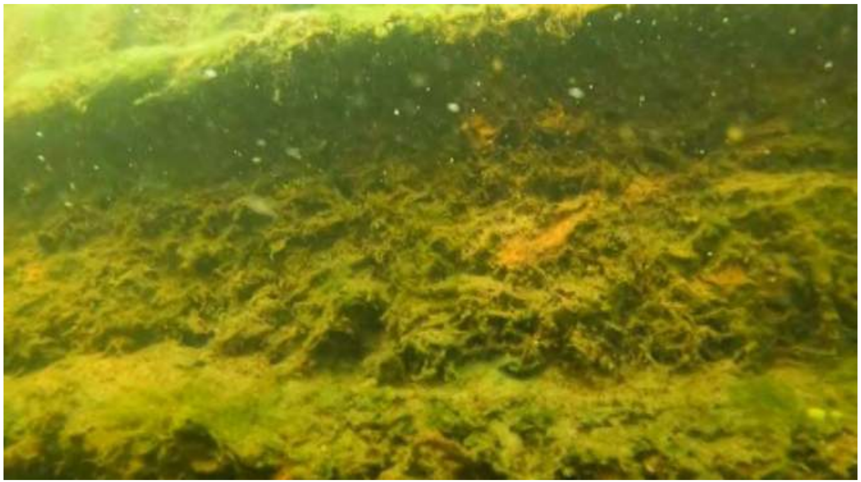


PHOTO # 57

Close up of typical condition of "C" channel at 0+125 (east face)

	PHOTO # 58 Piles at bottom at 0+123 (east face)
	PHOTO # 59 "C" channel at 0+120 (east side)
	PHOTO # 60 Piles at bottom at 0+116 (east face)

PHOTO # 61

Piles at bottom at 0+113 (east face)

PHOTO # 62

"C" channel at 0+112 (east side)

PHOTO # 63

Piles at bottom at 0+110 (east face)

PHOTO # 64

Piles at bottom at 0+106 (east face)

PHOTO # 65

Top of "C" channel 0+103 (east face)

PHOTO # 66

Close up of typical condition of "C" channel at 0+100 (east face)

PHOTO # 67

Top of "C" channel 0+100 (east face)

PHOTO # 68

"C" channel at 0+096 (east side)

PHOTO # 69

Close up of typical condition of "C" channel at 0+094 (east face)

PHOTO # 70

Piles at bottom at 0+090 (east face)

PHOTO # 71

Piles at bottom at 0+088 (east face)

PHOTO # 72

"C" channel at 0+084 (east side)

PHOTO # 73

Piles at bottom at 0+084 (east face)

PHOTO # 74

Top of "C" channel 0+082 (east face)

PHOTO # 75

Close up of typical condition of "C" channel at 0+080 (east face)

PHOTO # 76

Piles at bottom at 0+080 (east face)

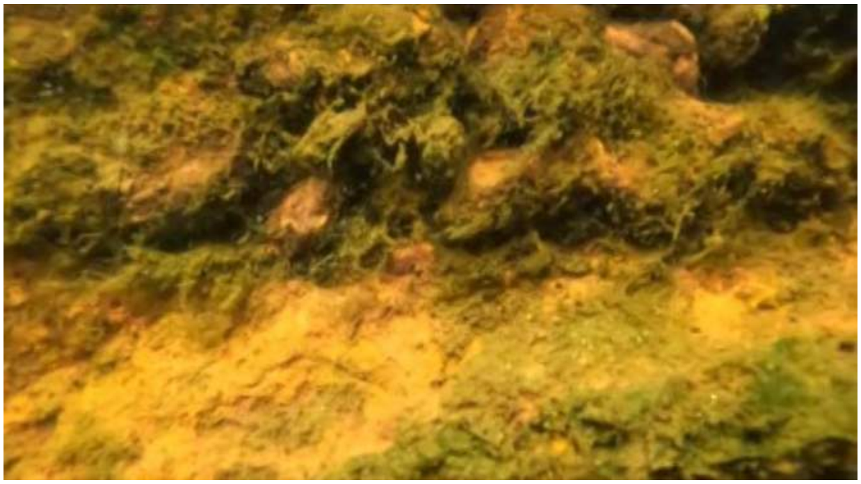


PHOTO # 77

Close up of typical condition of "C" channel at 0+075 (east face)

PHOTO # 78

Piles at bottom at 0+075 (east face)

PHOTO # 79

"C" channel at 0+068 (east side)

PHOTO # 80

Piles at bottom at 0+065 (east face)

PHOTO # 81

"C" channel at 0+060 (east side)

PHOTO # 82

Piles at bottom at 0+054 (east face)

PHOTO # 83

Close up of typical condition of "C" channel at 0+052 (east face)

PHOTO # 84

Piles at bottom at 0+050 (east face)

PHOTO # 85

Piles at bottom at 0+040 (east face)

PHOTO # 86

Top of "C" channel 0+040 (east face)

PHOTO # 87

Top of "C" channel 0+030 (east face)

PHOTO # 88

Piles at bottom at 0+024 (east face)

PHOTO # 89

Top of pile at 0+023 (east side)

PHOTO # 90

Top of "C" channel 0+016 (east face)

PHOTO # 91

Piles at bottom at 0+010 (east face)

PHOTO # 92

"C" channel at 0+010 (east side)

PHOTO # 93

Piles at 0+004 (east side)

PHOTO # 94

Typical steel sheet pile condition at 0+000

PHOTO # 95

Typical steel sheet pile condition at -0+002

PHOTO # 96

Typical steel sheet pile condition at -0+004

PHOTO # 97

Typical steel sheet pile condition at -0+005

PHOTO # 98

Typical steel sheet pile condition at -0+007

PHOTO # 99

Typical steel sheet pile condition at -0+009

PHOTO # 100

Typical steel sheet pile condition at -0+010

PHOTO # 101

Typical steel sheet pile condition at -0+012

PHOTO # 102

Typical steel sheet pile condition at -0+015

PHOTO # 103

Typical steel sheet pile condition at -0+016

PHOTO # 104

Typical steel sheet pile condition at -0+018

PHOTO # 105

Typical steel sheet pile condition at -0+020

PHOTO # 106

Typical steel sheet pile condition at -0+025

PHOTO # 107

Typical steel sheet pile condition at -0+027

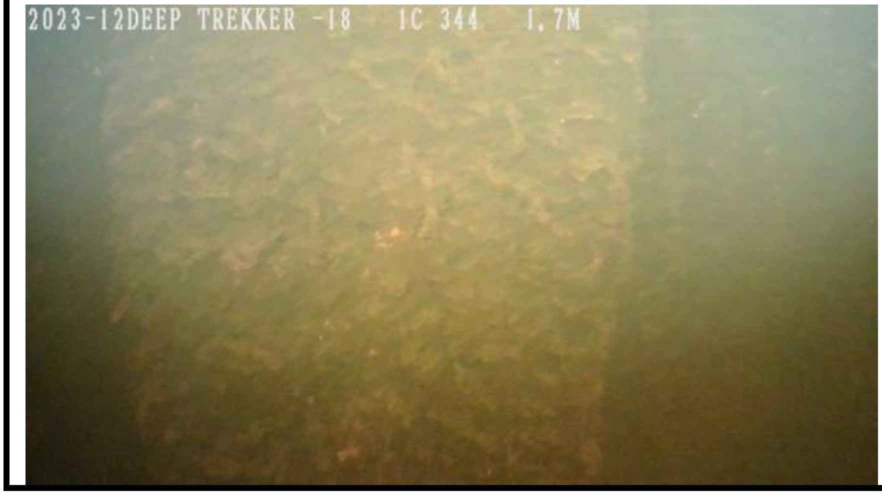


PHOTO # 108

Typical steel sheet pile condition at -0+029

PHOTO # 109

Typical steel sheet pile condition at -0+031

PHOTO # 110

Typical steel sheet pile condition at -0+034

PHOTO # 111

Typical steel sheet pile condition at -0+036

PHOTO # 112

Typical steel sheet pile condition at -0+038

PHOTO # 113

Typical steel sheet pile condition at -0+041

PHOTO # 114

Typical steel sheet pile condition at -0+043

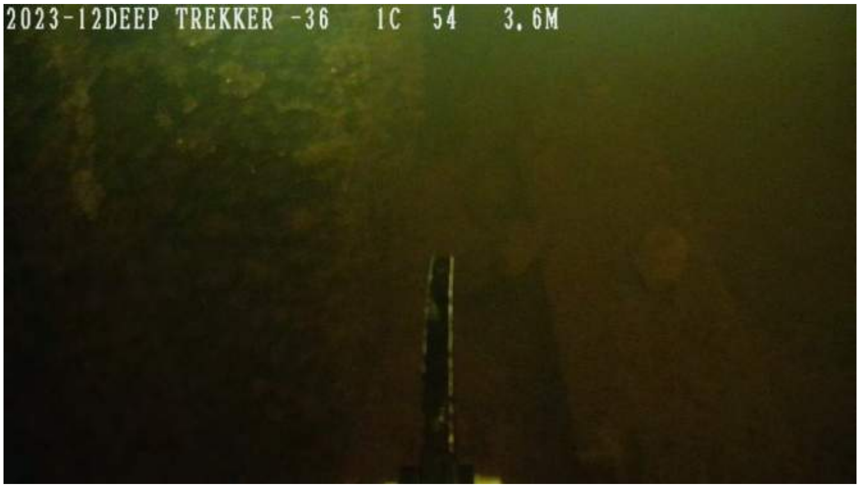


PHOTO # 115

Typical steel sheet pile condition at -0+045

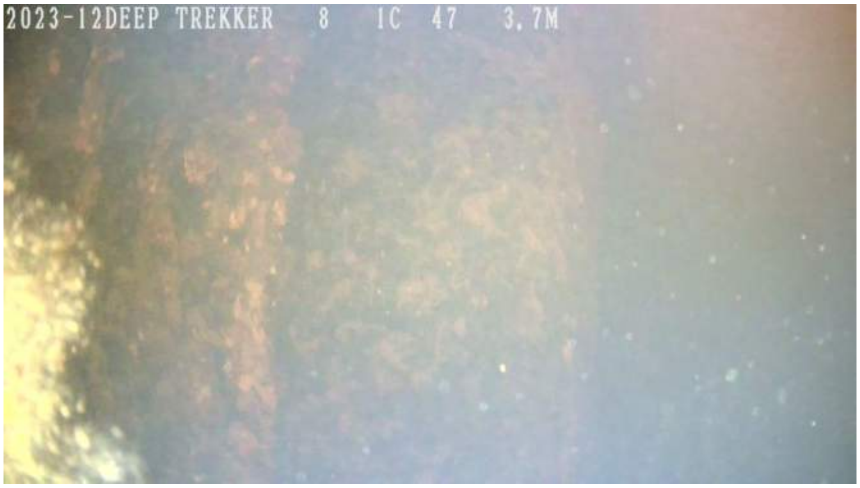


PHOTO # 116

Typical steel sheet pile condition at -0+051

PHOTO # 117

Typical steel sheet pile condition at -0+055

PHOTO # 118

Typical steel sheet pile condition at -0+060

PHOTO # 119

Typical steel sheet pile condition at -0+064

PHOTO # 120

Typical steel sheet pile condition at -0+070

PHOTO # 121

Where sheet pile ends and timber cribbing starts at -0+076

PHOTO # 122

Transition between sheet pile and timber cribbing at 0+076

PHOTO # 123

Transition between sheet pile and timber cribbing at 0+076

PHOTO # 124

Transition between sheet pile and timber cribbing at 0+076

PHOTO # 125

General condition of timber cribbing at -0+078

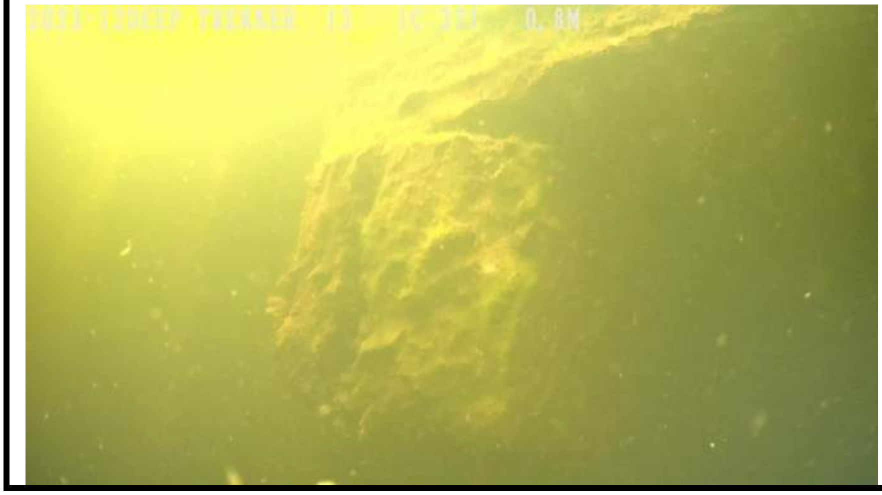


PHOTO # 126

General condition of timber cribbing at -0+078

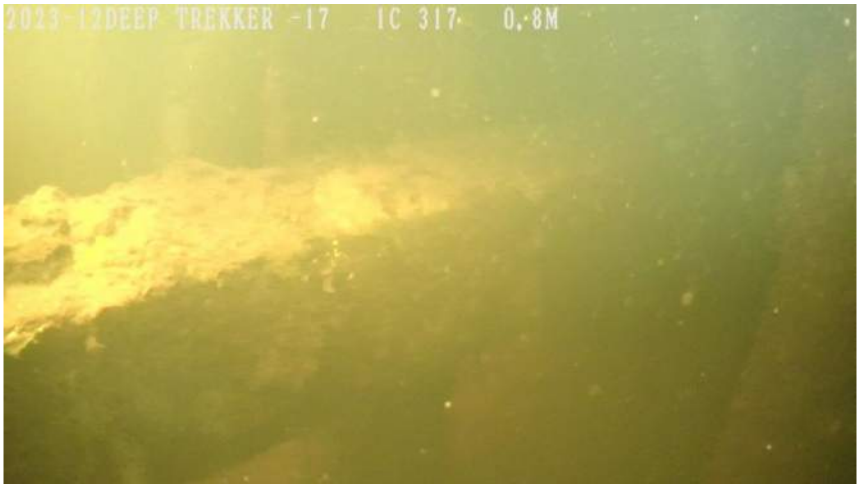
	PHOTO # 127 General condition of timber cribbing at -0+079
	PHOTO # 128 General condition of timber cribbing at -0+080
	PHOTO # 129 General condition of timber cribbing at -0+081

PHOTO # 130

General condition of timber cribbing at -0+082

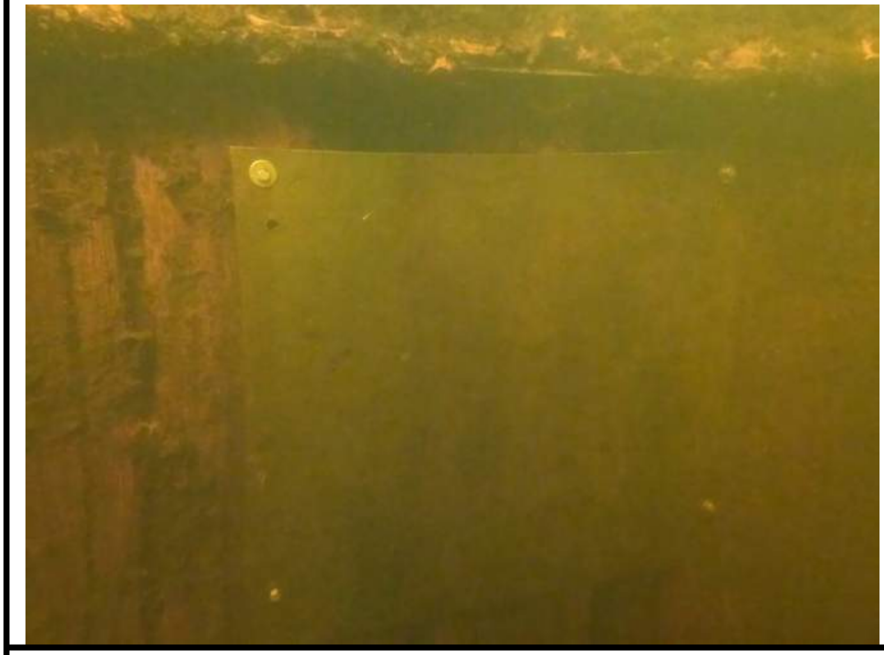


PHOTO # 131

General photo showing the hole patch in the timber

PHOTO # 132

General photo showing the hole patch in the timber

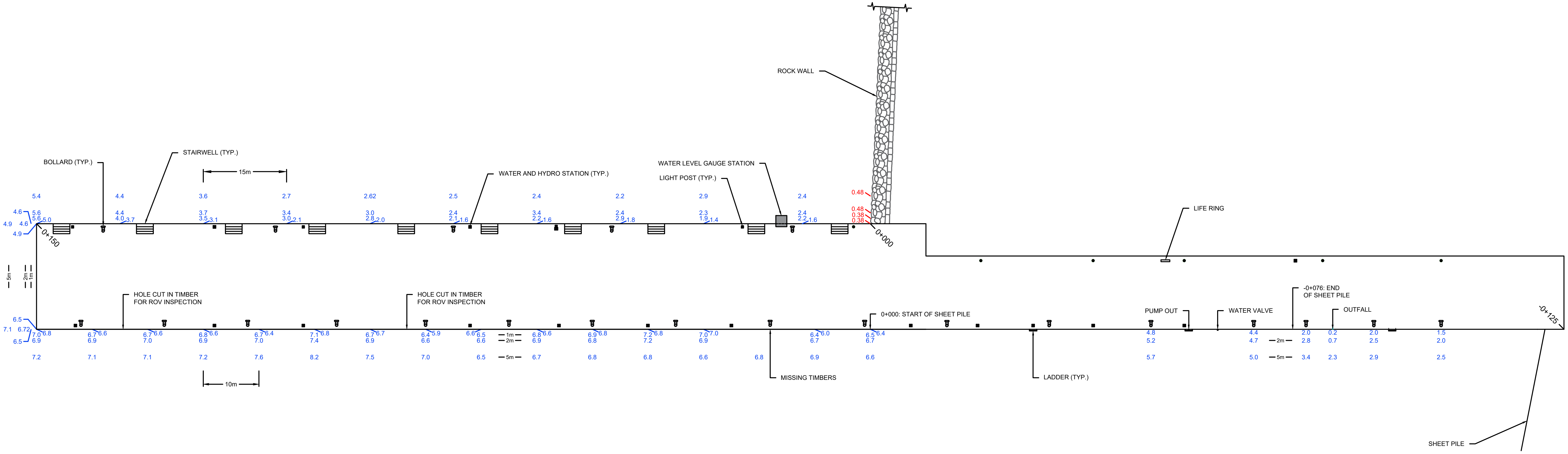
DOCK INSPECTION
PARRY SOUND, ONTARIO

AECOM

December 2023

Figures

WATECH SERVICES INC.
WSI 23189



SITE PLAN

NOTES

1. INSPECTION COMPLETED IN DECEMBER 2023.
2. BLUE TEXT REPRESENTS WATER DEPTH SOUNDINGS.
3. ALL MEASUREMENTS ARE SHOWN IN METRES.
4. ALL SOUNDINGS ARE REFERENCED TO BELOW CHART DATUM.
5. HOLES CUT IN TIMBER FOR ROV ACCESS WERE PATCHED AFTER THE INSPECTION.

SCALE:

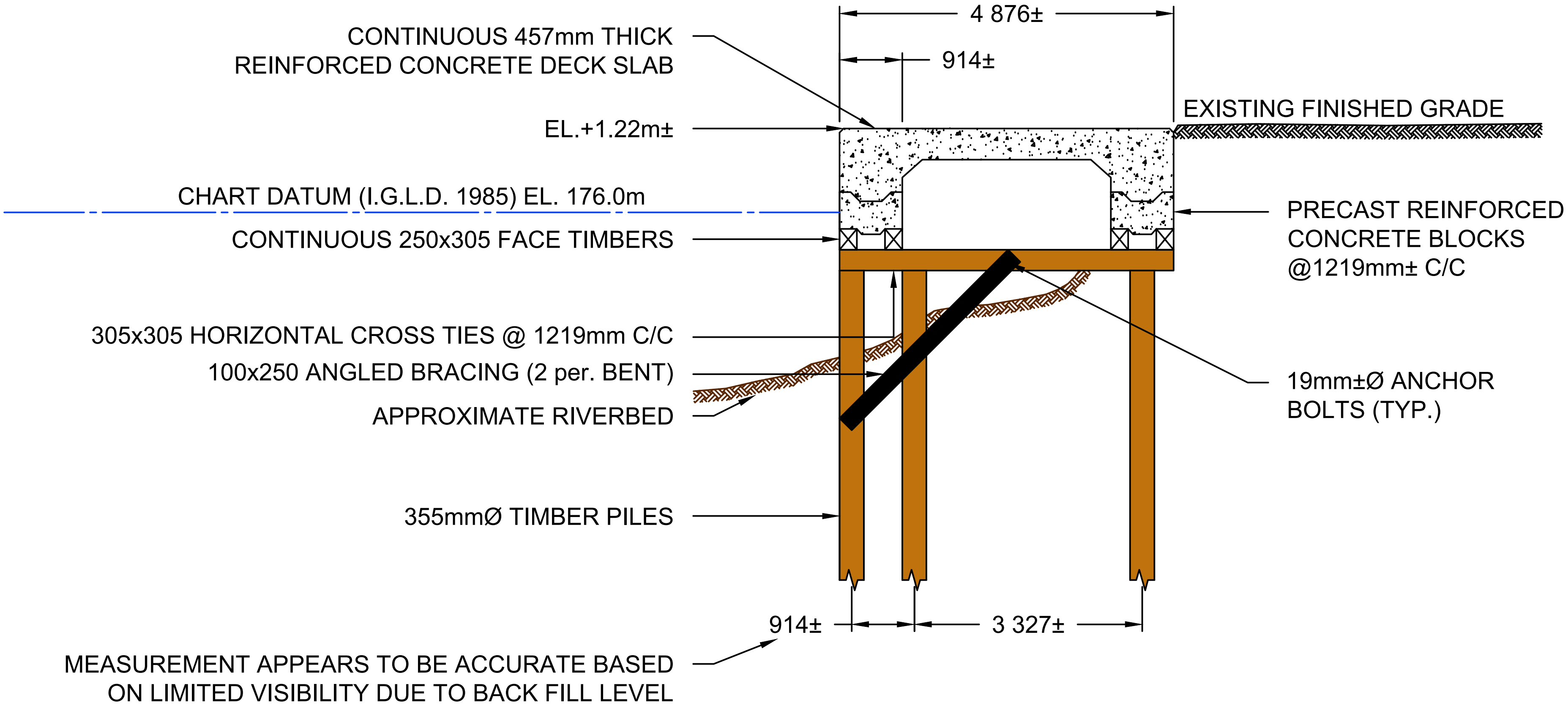
NTS

WATECH SERVICES INC

AECOM

DOCK INSPECTION
FIGURE 1
SITE PLAN

23189
DECEMBER 2023



NOTES

1. INSPECTION COMPLETED IN NOVEMBER & DECEMBER 2023.
2. ALL MEASUREMENTS ARE SHOWN IN MILLIMETERS.

SCALE:

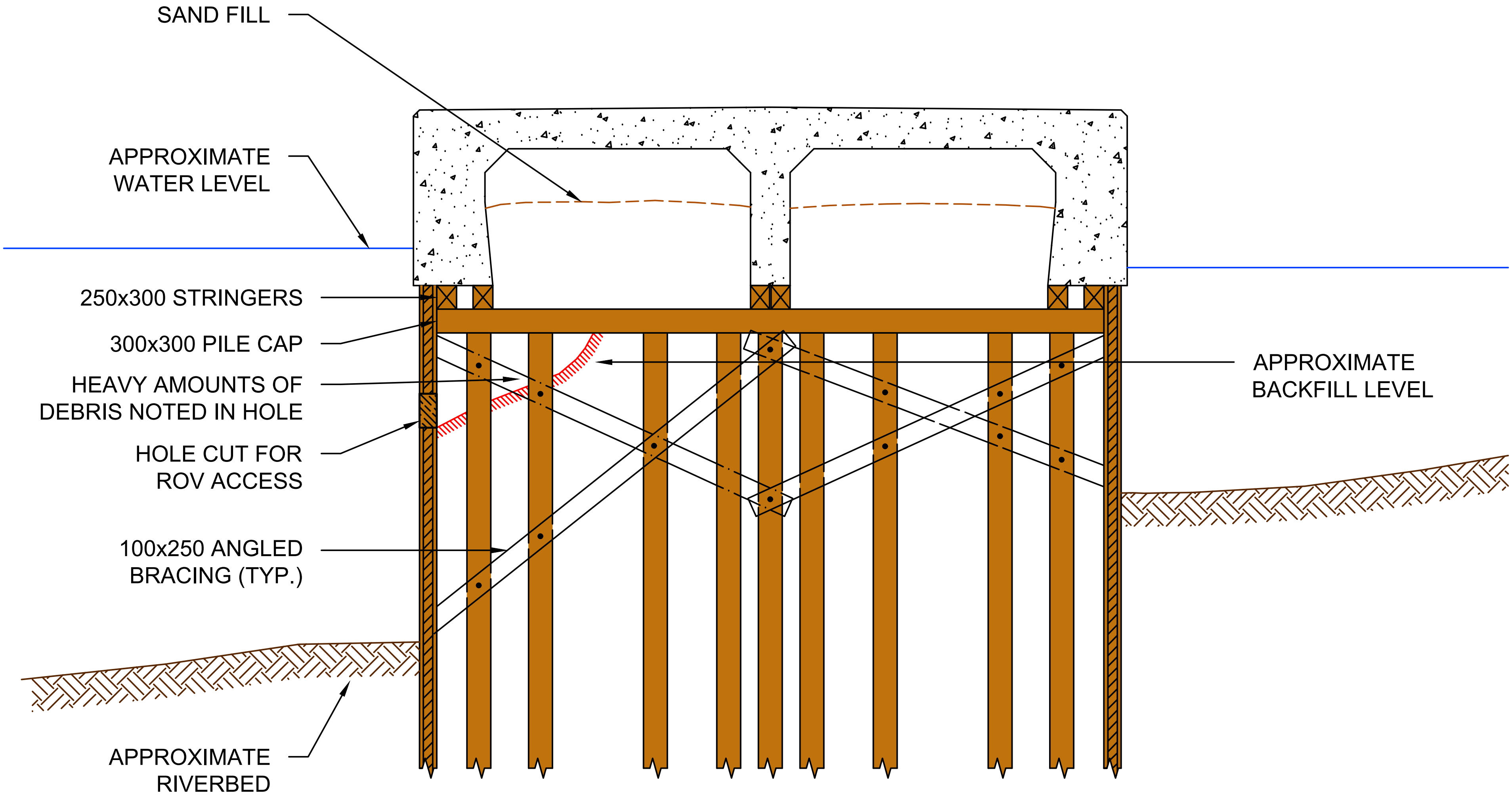
NTS

WATECH SERVICES INC.

AECOM

DOCK INSPECTION
FIGURE 2
TYP. SECTION AT STATION -0+080

23189
DECEMBER 2023



NOTES

1. INSPECTION COMPLETED IN NOVEMBER & DECEMBER 2023.
2. ALL MEASUREMENTS ARE SHOWN IN MILLIMETERS.
3. HOLES CUT IN TIMBER FOR ROV ACCESS WERE PATCHED AFTER THE INSPECTION.

SCALE:

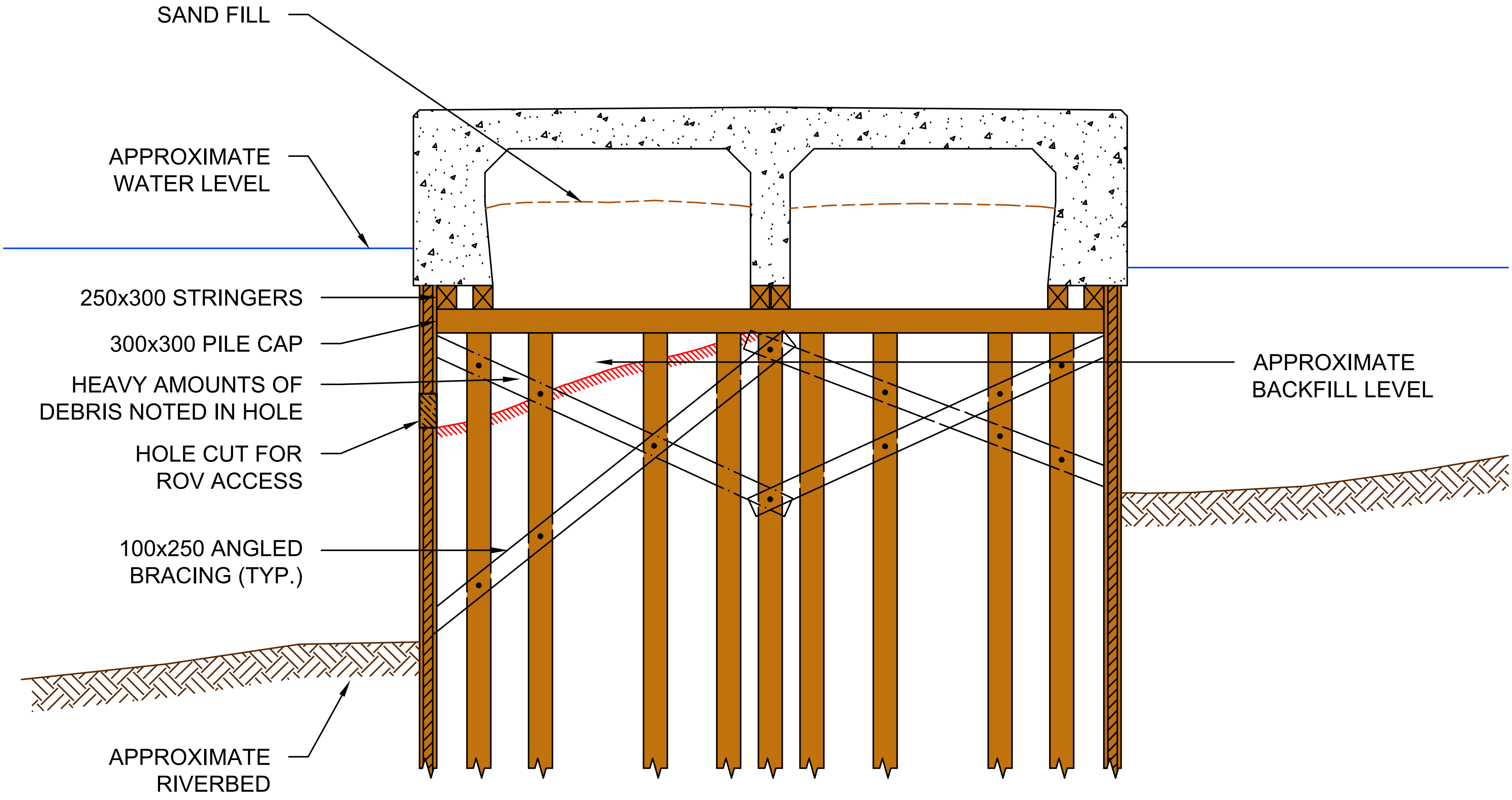
NTS

WATECH
SERVICES INC.

AECOM

DOCK INSPECTION
FIGURE 3
TYP. SECTION AT NORTH
ROV ACCESS HOLE

23189
DECEMBER 2023



NOTES

1. INSPECTION COMPLETED IN NOVEMBER & DECEMBER 2023.
2. ALL MEASUREMENTS ARE SHOWN IN MILLIMETERS.
3. HOLES CUT IN TIMBER FOR ROV ACCESS WERE PATCHED AFTER THE INSPECTION.

SCALE:

NTS

WATECH
SERVICES INC.

AECOM

DOCK INSPECTION
FIGURE 4
TYP. SECTION AT SOUTH
ROV ACCESS HOLE

23189
DECEMBER 2023

DOCK INSPECTION
PARRY SOUND, ONTARIO

AECOM

December 2023

Video

WATECH SERVICES INC.
WSI 23189

Appendix D

Detailed Inspection Sheets

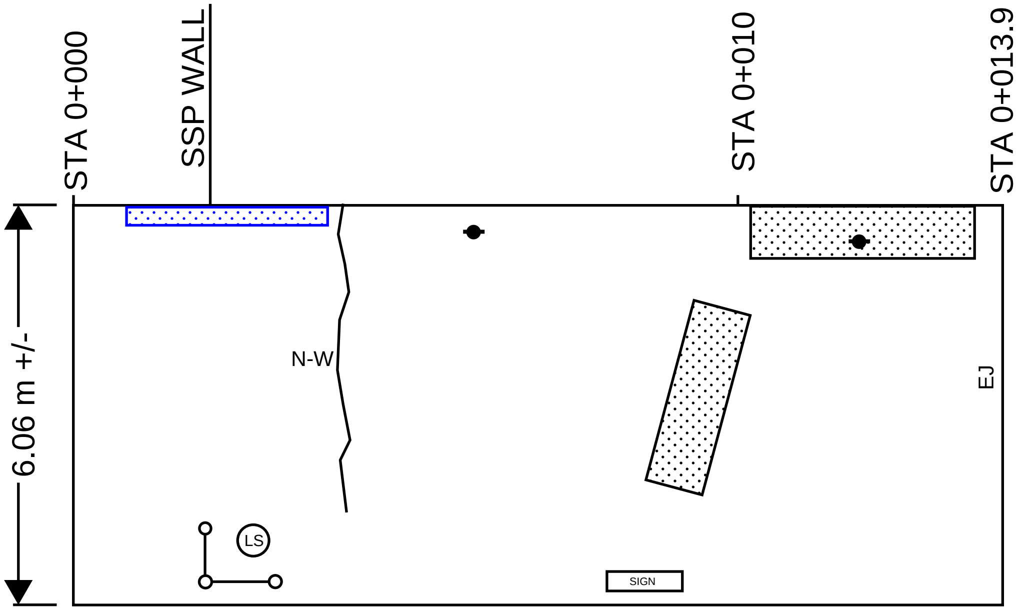
AECOM

JOB TITLE: PARRY SOUND HARBOUR - DETAILED INSPECTION SHEET
PROJECT NUMBER: 60719231
PREPARED BY: KC
DATE: NOVEMBER 16, 2023
WEATHER: SUNNY
TEMPERATURE: 12°C
SHEET NO. 1 OF 13

LEGEND:

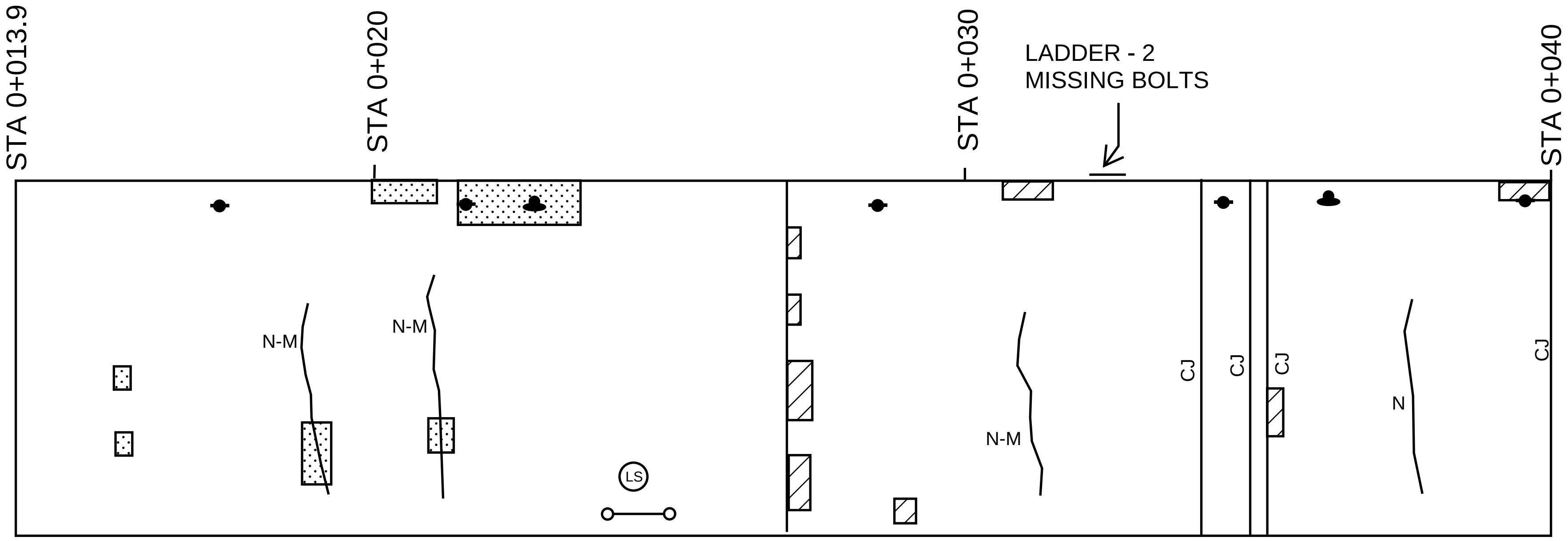
●	BOLLARD
•	2 1/4" DIA. BOLLARD
LS	LIGHT STANDARD
o—o	RAILING
—o—o—o—	CURB RAIL
┄┄┄	BURIED CONDUIT
▨	DELAMINATION
🟦	SPALL
🟩	SCALING
░	CONCRETE PATCH

JOB TITLE: PARRY SOUND HARBOUR - DETAILED INSPECTION SHEET
PROJECT NUMBER: 60719231
PREPARED BY: KC DATE: NOVEMBER 16, 2023
WEATHER: SUNNY TEMPERATURE: 12°C SHEET NO. 2 OF 13



NOTES:
- NARROW MAP CRACKING TYPICAL
- LOCALIZED AREAS OF DELAMINATION
- EFFLORESCENT STAINED CRACKING AND AREAS OF DELAMINATION ON CONCRETE EDGE (STA 0+002 TO 0+007).
- SOME SHIFTING OBSERVED IN PRECAST CONCRETE BLOCKS, AS WELL AS MEDIUM SCALING AND AREAS OF SPALLING.
- VEGETATION GROWTH IN EXPANSION JOINT

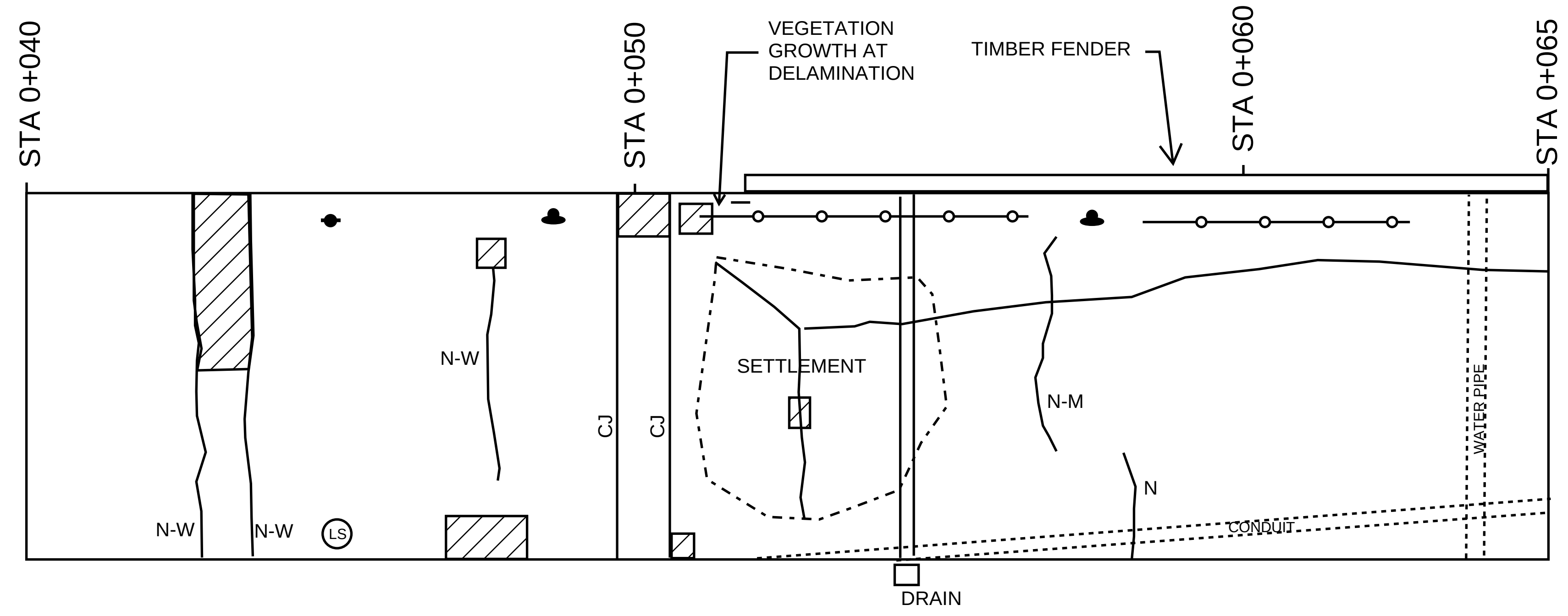
JOB TITLE: PARRY SOUND HARBOUR - DETAILED INSPECTION SHEET
PROJECT NUMBER: 60719231
PREPARED BY: KC DATE: NOVEMBER 16, 2023
WEATHER: SUNNY TEMPERATURE: 12°C SHEET NO. 3 OF 13



NOTES:
- LIGHT TO MEDIUM SCALING.
- NEWER CONCRETE FROM STA 0+027 TO 0+050.
- HAIRLINE MAP CRACKING TYPICAL IN SECTION OF NEWER CONCRETE.

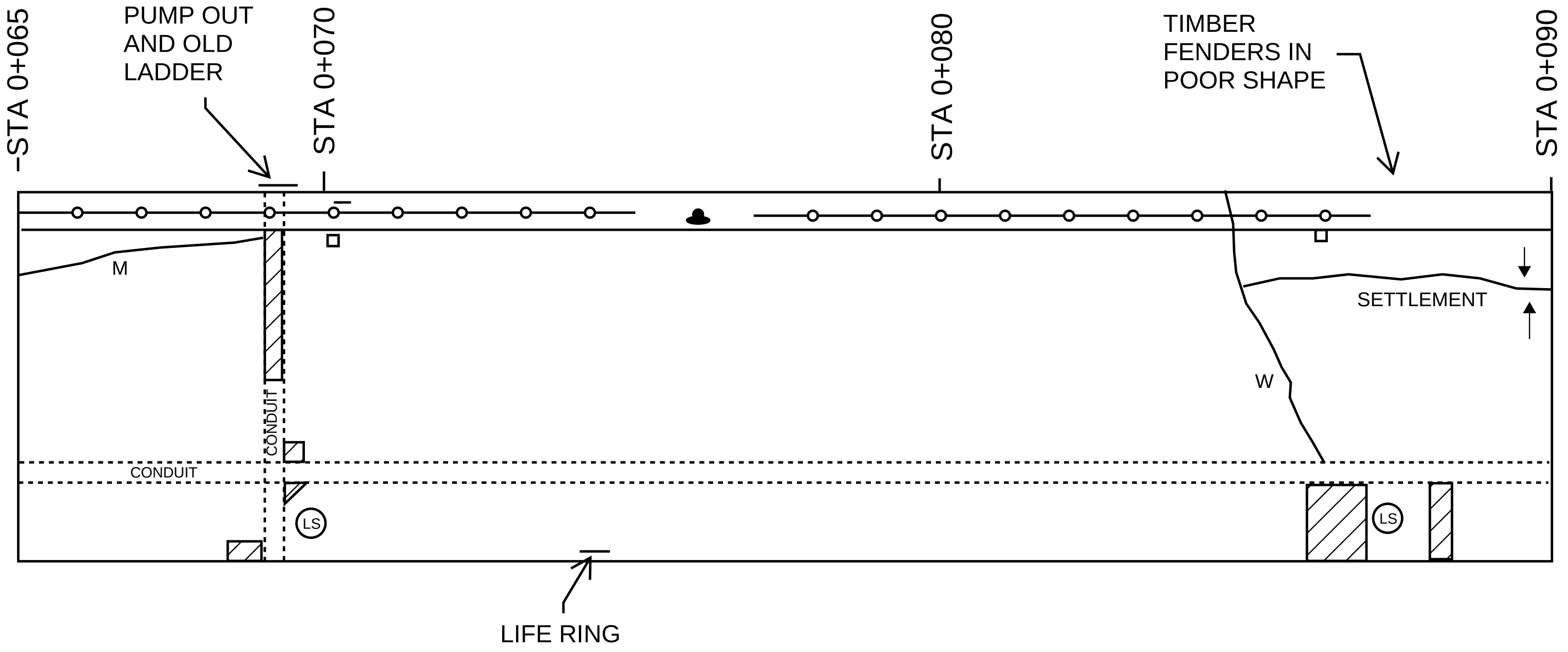
AECOM

JOB TITLE: PARRY SOUND HARBOUR - DETAILED INSPECTION SHEET
PROJECT NUMBER: 60719231
PREPARED BY: KC
DATE: NOVEMBER 16, 2023
WEATHER: SUNNY
TEMPERATURE: 12°C
SHEET NO. 4 OF 13



NOTES:
- VEGETATION GROWTH IN CONSTRUCTION JOINTS.
- NEWER CONCRETE FROM STA 0+027 TO 0+050.
- SPRAY FOAM WAS INSTALLED BELOW DECK FROM STA 0+050 TO 0+063 (APPROXMATELY).
- MAP CRACKING TYPICAL.
- CURB RAIL HAS LOCALIZED COATING LOSS AND SURFACE CORROSION

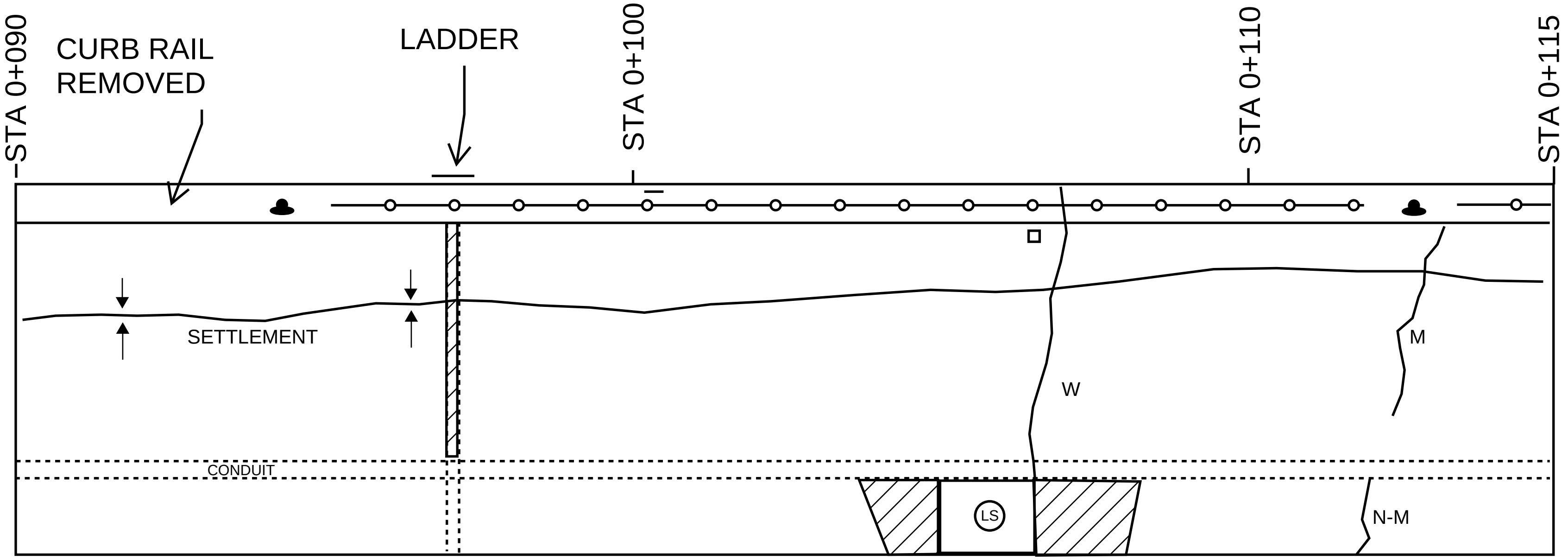
JOB TITLE: PARRY SOUND HARBOUR - DETAILED INSPECTION SHEET
PROJECT NUMBER: 60719231
PREPARED BY: KC
DATE: NOVEMBER 16, 2023
WEATHER: SUNNY
TEMPERATURE: 12°C
SHEET NO. 5 OF 13



NOTES:
- LIGHT SCALING OF CONCRETE SURFACE.
- VEGETATION GROWTH IN JOINT BETWEEN DECK AND FASCIA.

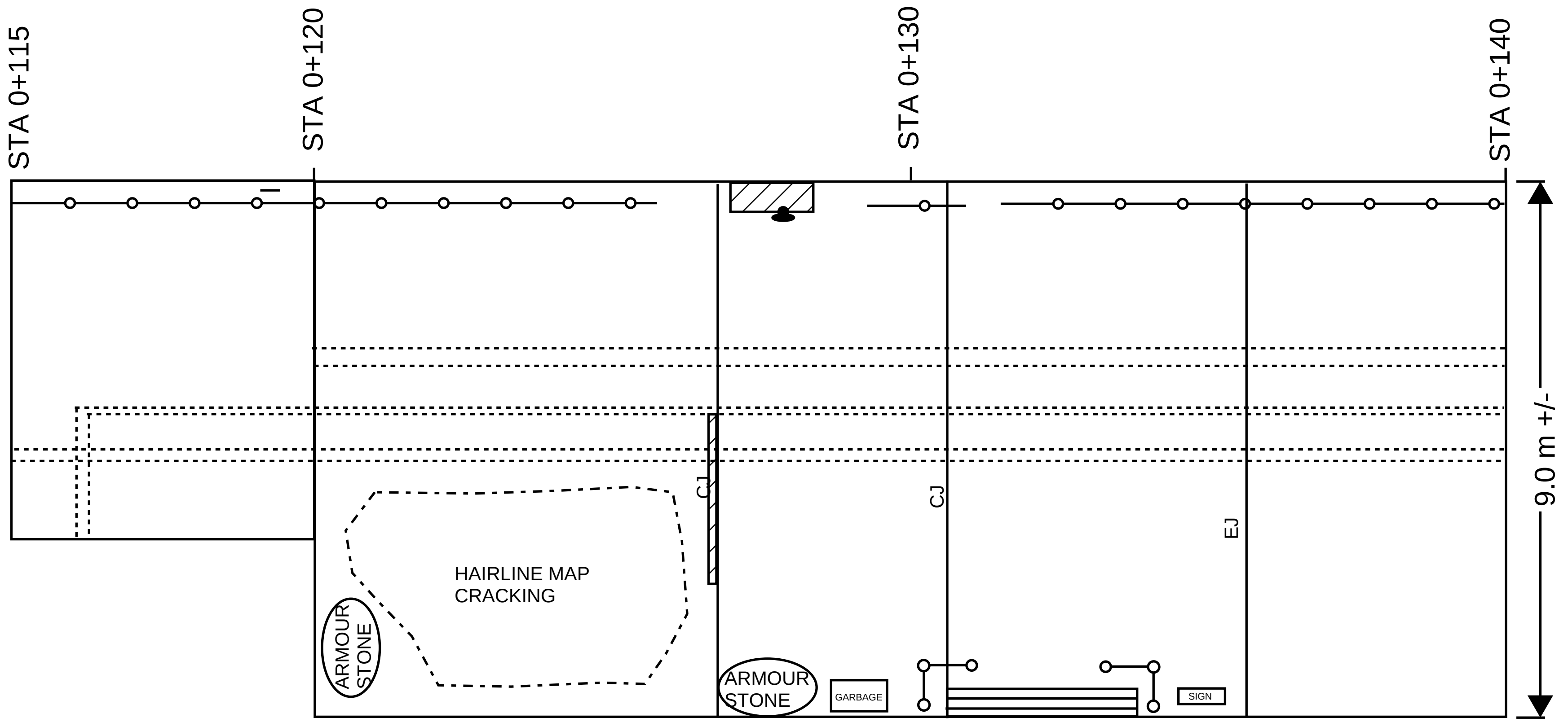
AECOM

JOB TITLE: PARRY SOUND HARBOUR - DETAILED INSPECTION SHEET
PROJECT NUMBER: 60719231
PREPARED BY: KC
DATE: NOVEMBER 16, 2023
WEATHER: SUNNY
TEMPERATURE: 12°C
SHEET NO. 6 OF 13



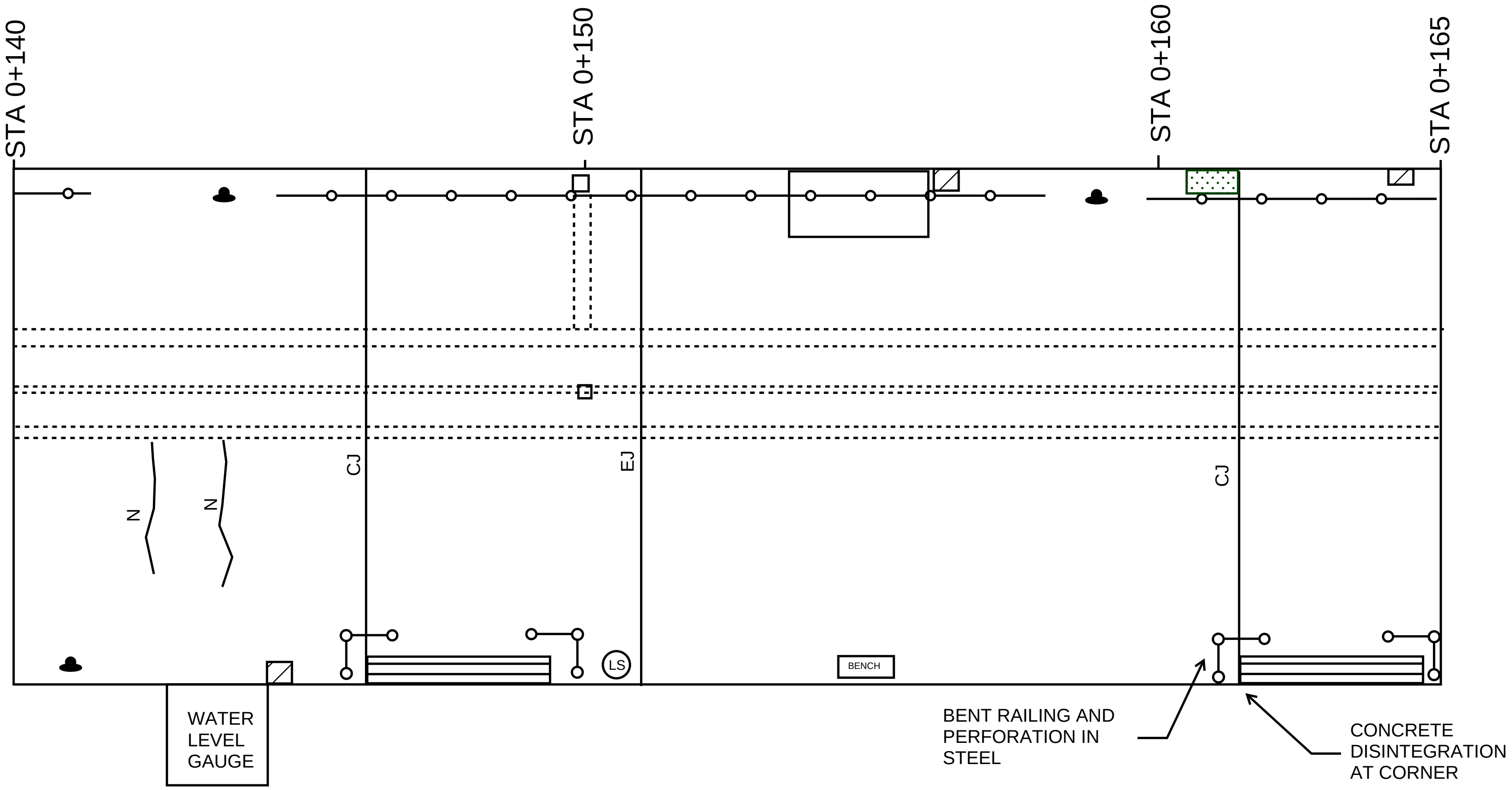
NOTES:
- AREAS OF LOCALIZED SCALING.
- DECK SOUNDS HOLLOW IN SETTLED AREAS.
- TIRES ON SIDE OF WHARF FOR ISLAND QUEEN.
- GAP AND VEGETATION GROWTH IN JOINT.

JOB TITLE: PARRY SOUND HARBOUR - DETAILED INSPECTION SHEET
PROJECT NUMBER: 60719231
PREPARED BY: KC DATE: NOVEMBER 16, 2023
WEATHER: SUNNY TEMPERATURE: 12°C SHEET NO. 7 OF 13



NOTES:
- VEGETATION GROWTH IN EXPANSION JOINT.
- LOCALIZED SPALLING ON STAIRS.
- MEDIUM TO SEVERE SCALING TYPICAL.
- PATCH REPAIRS TYPICAL.
- WATER LEVEL MEASURED AT 1.4m +/- BELOW TOP OF DECK.

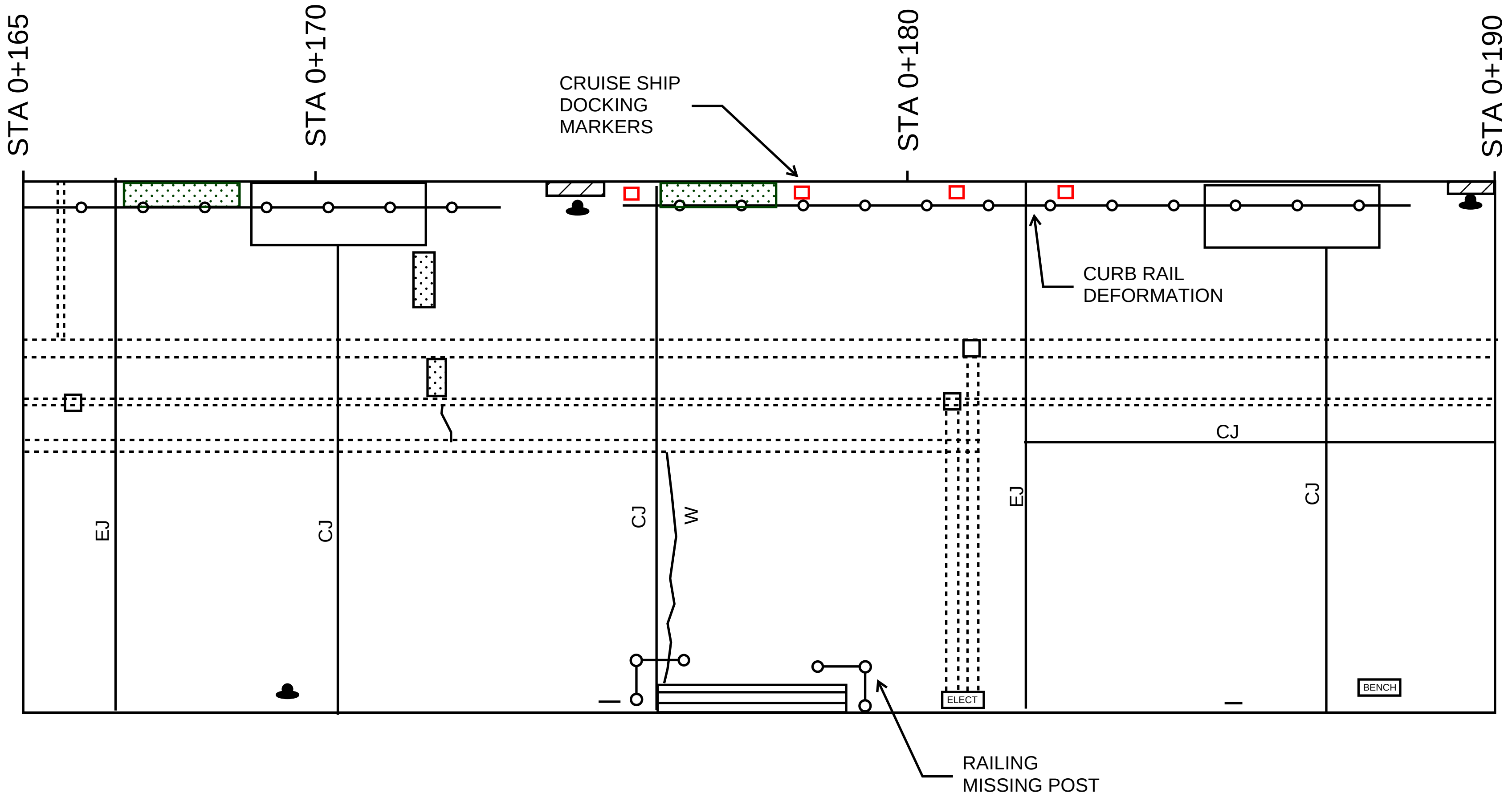
JOB TITLE: PARRY SOUND HARBOUR - DETAILED INSPECTION SHEET
PROJECT NUMBER: 60719231
PREPARED BY: KC
DATE: NOVEMBER 16, 2023
WEATHER: SUNNY
TEMPERATURE: 12°C
SHEET NO. 8 OF 13



NOTES:

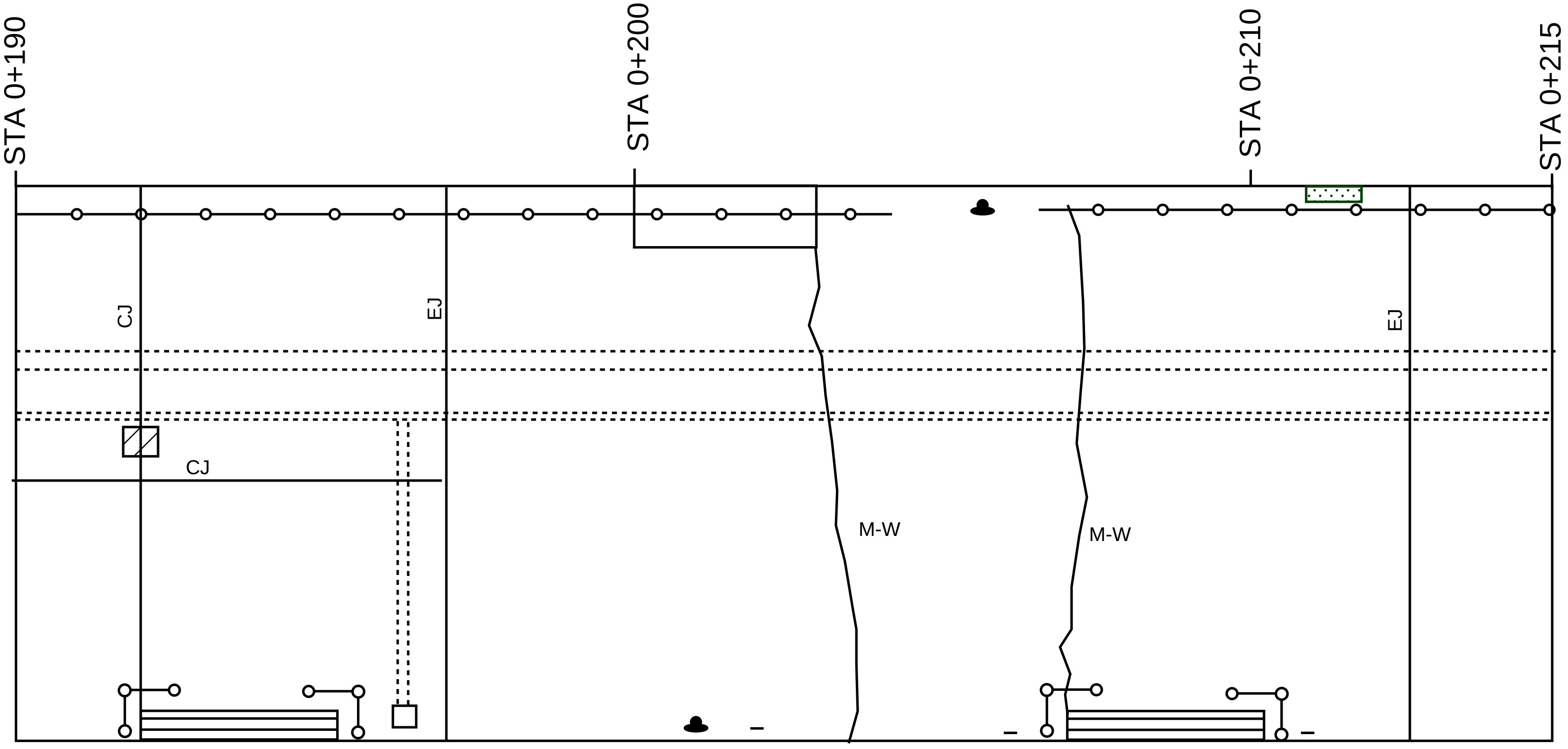
- DELAMINATION AND NARROW TO MEDIUM CRACKING ON STAIRS.
- PATCHES ON STAIRS TYPICAL.
- MEDIUM TO SEVERE SCALING TYPICAL.
- PATCH REPAIRS TYPICAL.

JOB TITLE: PARRY SOUND HARBOUR - DETAILED INSPECTION SHEET
PROJECT NUMBER: 60719231
PREPARED BY: KC
DATE: NOVEMBER 16, 2023
WEATHER: SUNNY
TEMPERATURE: 12°C
SHEET NO. 9 OF 13



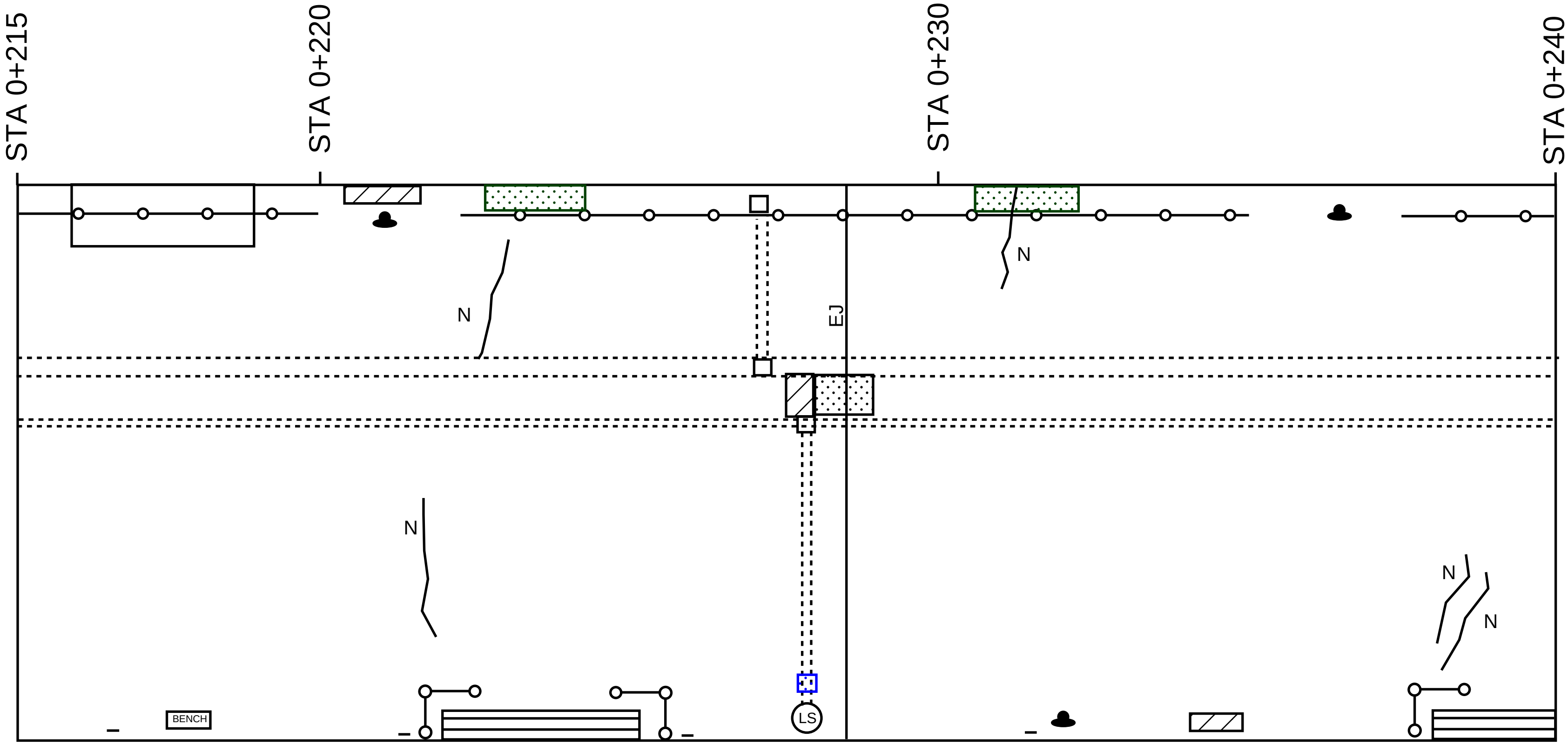
NOTES:
- MEDIUM SCALING TYPICAL.

JOB TITLE: PARRY SOUND HARBOUR - DETAILED INSPECTION SHEET
PROJECT NUMBER: 60719231
PREPARED BY: KC
DATE: NOVEMBER 16, 2023
WEATHER: SUNNY
TEMPERATURE: 12°C
SHEET NO. 10 OF 13



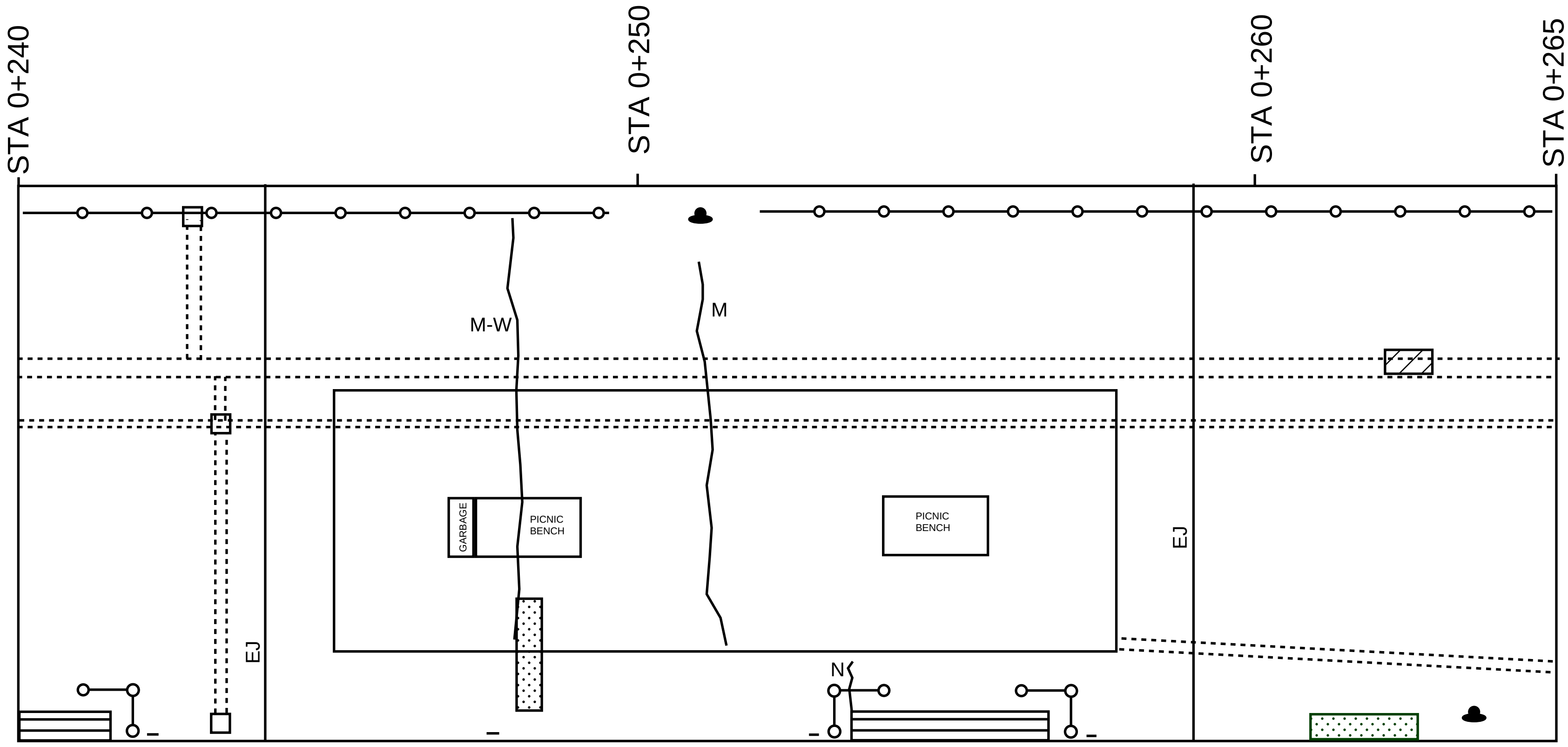
NOTES:
- MEDIUM TO SEVERE SCALING TYPICAL.

JOB TITLE: PARRY SOUND HARBOUR - DETAILED INSPECTION SHEET
PROJECT NUMBER: 60719231
PREPARED BY: KC
DATE: NOVEMBER 16, 2023
WEATHER: SUNNY TEMPERATURE: 12°C SHEET NO. 11 OF 13



NOTES:

JOB TITLE: PARRY SOUND HARBOUR - DETAILED INSPECTION SHEET
PROJECT NUMBER: 60719231
PREPARED BY: KC DATE: NOVEMBER 16, 2023
WEATHER: SUNNY TEMPERATURE: 12°C SHEET NO. 12 OF 13



NOTES:
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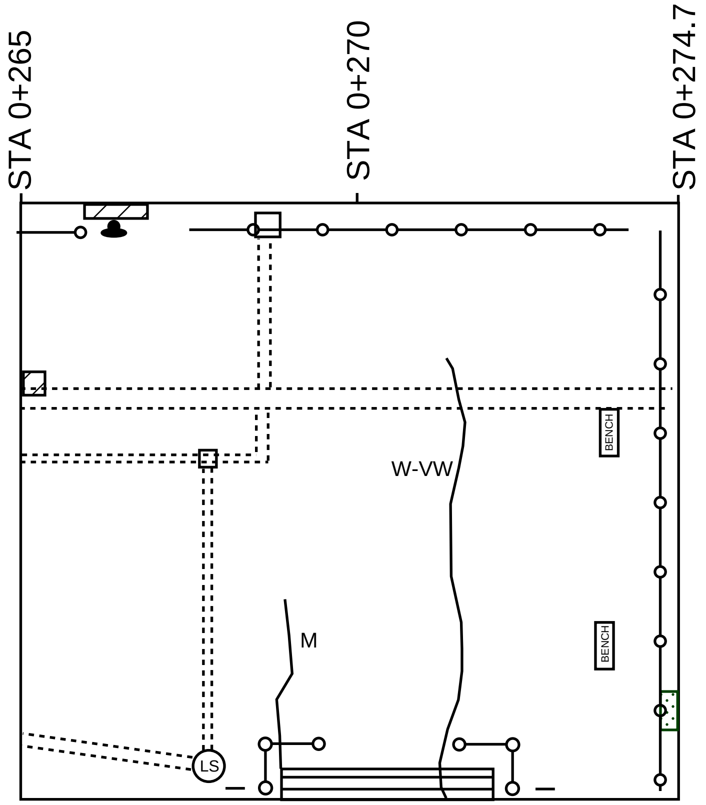
JOB TITLE: PARRY SOUND HARBOUR - DETAILED INSPECTION SHEET
PROJECT NUMBER: 60719231
PREPARED BY: KC DATE: NOVEMBER 16, 2023
WEATHER: SUNNY TEMPERATURE: 12°C SHEET NO. 13 OF 13



NOTES:
-

Appendix E

Useful Residual Life Calculations

Project: Parry Sound Harbour
Component: **Structure A**
Timber Crib Substructure

Project No: 60749231
Date: Jan-24

Year Constructed = 1952 years
Actual Age, AA = 72 years

Theoretical Useful Life, TUL = 40 years

Foundations	Steel Sheet Piles	80
	Steel Pipe Piles	80
	Timber Piles	40
Concrete on Timber Foundations		60
Concrete on Steel Foundations		60
Timber Superstructures		40
Pavement		20
Breakwaters / Rock Protection		100

Compensating Factor, Selected CF =

Severe deterioration	0.7
Considerable Deterioration	0.8
Average Deterioration	0.9
Normal Condition	1.0

Weighting Coefficient, calculated WC = 0

		Steel	Concrete	Timber	Rock	Selected
Use	Normal	0.0	0.0	0.0	0.0	
	Heavy	7.5	5.0	10.0	0.0	
	Abusive	15.0	10.0	20.0	0.0	
Exposure to Salinity and Polution	None	0.0	0.0	0.0	0.0	
	Alternating	10.0	10.0	2.5	0.0	
	Concentrated	25.0	20.0	5.0	0.0	
Sea Condition	Mild (0 to 3')	0.0	0.0	0.0	0.0	
	Average (3 to 6')	2.5	2.5	2.5	5.0	
	Severe (>6')	5.0	5.0	5.0	10.0	
Ice and Waves	Good	0.0	0.0	0.0	0.0	
	Fair	2.5	2.5	7.5	5.0	
	Inadequate	5.0	5.0	15.0	10.0	
Fender System	Good	0.0	0.0	0.0	0.0	
	Fair	2.5	2.5	5.0	0.0	
	Inadequate	5.0	5.0	10.0	0.0	
Foundation	Excellent	0.0	0.0	0.0	0.0	
	Fair	5.0	10.0	2.5	5.0	
	Problems	10.0	15.0	5.0	10.0	
Construction and Design	Excellent	0.0	0.0	0.0	0.0	
	Fair	5.0	5.0	5.0	15.0	
	Weak	10.0	10.0	10.0	30.0	
Biological Attack	None	0.0	0.0	0.0	0.0	
	Some	0.0	0.0	15.0	0.0	
	Advanced	0.0	0.0	30.0	0.0	
					Sum WC =	0.0

Notes:
1. AA > TUL, calculation not required

Remaining Useful Residual Life

URL = [TUL x (100 – WC) x CF] – AA

= 0 years

Project: Parry Sound Harbour

Project No: 60749231

Component: **Structure A**
Concrete Deck Superstructure

Date: Jan-24

Year Constructed = 1952 years
Actual Age, AA = 72 years

Theoretical Useful Life, TUL = 60 years

Foundations	Steel Sheet Piles	80
	Steel Pipe Piles	80
	Timber Piles	40
Concrete on Timber Foundations		60
Concrete on Steel Foundations		60
Timber Superstructures		40
Pavement		20
Breakwaters / Rock Protection		100

Compensating Factor, Selected CF =

Severe deterioration	0.7
Considerable Deterioration	0.8
Average Deterioration	0.9
Normal Condition	1.0

Weighting Coefficient, calculated WC = 0

		Steel	Concrete	Timber	Rock	Selected
Use	Normal	0.0	0.0	0.0	0.0	
	Heavy	7.5	5.0	10.0	0.0	
	Abusive	15.0	10.0	20.0	0.0	
Exposure to Salinity and Polution	None	0.0	0.0	0.0	0.0	
	Alternating	10.0	10.0	2.5	0.0	
	Concentrated	25.0	20.0	5.0	0.0	
Sea Condition	Mild (0 to 3')	0.0	0.0	0.0	0.0	
	Average (3 to 6')	2.5	2.5	2.5	5.0	
	Severe (>6')	5.0	5.0	5.0	10.0	
Ice and Waves	Good	0.0	0.0	0.0	0.0	
	Fair	2.5	2.5	7.5	5.0	
	Inadequate	5.0	5.0	15.0	10.0	
Fender System	Good	0.0	0.0	0.0	0.0	
	Fair	2.5	2.5	5.0	0.0	
	Inadequate	5.0	5.0	10.0	0.0	
Foundation	Excellent	0.0	0.0	0.0	0.0	
	Fair	5.0	10.0	2.5	5.0	
	Problems	10.0	15.0	5.0	10.0	
Construction and Design	Excellent	0.0	0.0	0.0	0.0	
	Fair	5.0	5.0	5.0	15.0	
	Weak	10.0	10.0	10.0	30.0	
Biological Attack	None	0.0	0.0	0.0	0.0	
	Some	0.0	0.0	15.0	0.0	
	Advanced	0.0	0.0	30.0	0.0	
					Sum WC =	0.0

Notes:

1. AA > TUL, calculation not required

Remaining Useful Residual Life

URL = [TUL x (100 – WC) x CF] – AA

= 0 years

Project: Parry Sound Harbour

Project No: 60749231

Component: **Structure B**
Timber Pile Substructure

Date: Jan-24

Year Constructed = 1922 years
Actual Age, AA = 102 years

Theoretical Useful Life, TUL = 40 years

Foundations	Steel Sheet Piles	80
	Steel Pipe Piles	80
	Timber Piles	40
Concrete on Timber Foundations		60
Concrete on Steel Foundations		60
Timber Superstructures		40
Pavement		20
Breakwaters / Rock Protection		100

Compensating Factor, Selected CF =

Severe deterioration	0.7
Considerable Deterioration	0.8
Average Deterioration	0.9
Normal Condition	1.0

Weighting Coefficient, calculated WC = 0

		Steel	Concrete	Timber	Rock	Selected
Use	Normal	0.0	0.0	0.0	0.0	
	Heavy	7.5	5.0	10.0	0.0	
	Abusive	15.0	10.0	20.0	0.0	
Exposure to Salinity and Polution	None	0.0	0.0	0.0	0.0	
	Alternating	10.0	10.0	2.5	0.0	
	Concentrated	25.0	20.0	5.0	0.0	
Sea Condition	Mild (0 to 3')	0.0	0.0	0.0	0.0	
	Average (3 to 6')	2.5	2.5	2.5	5.0	
	Severe (>6')	5.0	5.0	5.0	10.0	
Ice and Waves	Good	0.0	0.0	0.0	0.0	
	Fair	2.5	2.5	7.5	5.0	
	Inadequate	5.0	5.0	15.0	10.0	
Fender System	Good	0.0	0.0	0.0	0.0	
	Fair	2.5	2.5	5.0	0.0	
	Inadequate	5.0	5.0	10.0	0.0	
Foundation	Excellent	0.0	0.0	0.0	0.0	
	Fair	5.0	10.0	2.5	5.0	
	Problems	10.0	15.0	5.0	10.0	
Construction and Design	Excellent	0.0	0.0	0.0	0.0	
	Fair	5.0	5.0	5.0	15.0	
	Weak	10.0	10.0	10.0	30.0	
Biological Attack	None	0.0	0.0	0.0	0.0	
	Some	0.0	0.0	15.0	0.0	
	Advanced	0.0	0.0	30.0	0.0	
					Sum WC =	0.0

Notes:
1. AA > TUL, calculation not required

Remaining Useful Residual Life

URL = [TUL x (100 – WC) x CF] – AA

= 0 years

Project: Parry Sound Harbour
Component: **Structure B**
Concrete Deck Superstructure

Project No: 60749231
Date: Jan-24

Year Constructed = 1952 years
Actual Age, AA = 72 years

Theoretical Useful Life, TUL = 60 years

Foundations	Steel Sheet Piles	80
	Steel Pipe Piles	80
	Timber Piles	40
Concrete on Timber Foundations		60
Concrete on Steel Foundations		60
Timber Superstructures		40
Pavement		20
Breakwaters / Rock Protection		100

Compensating Factor, Selected CF =

Severe deterioration	0.7
Considerable Deterioration	0.8
Average Deterioration	0.9
Normal Condition	1.0

Weighting Coefficient, calculated WC = 0

		Steel	Concrete	Timber	Rock	Selected
Use	Normal	0.0	0.0	0.0	0.0	
	Heavy	7.5	5.0	10.0	0.0	
	Abusive	15.0	10.0	20.0	0.0	
Exposure to Salinity and Polution	None	0.0	0.0	0.0	0.0	
	Alternating	10.0	10.0	2.5	0.0	
	Concentrated	25.0	20.0	5.0	0.0	
Sea Condition	Mild (0 to 3')	0.0	0.0	0.0	0.0	
	Average (3 to 6')	2.5	2.5	2.5	5.0	
	Severe (>6')	5.0	5.0	5.0	10.0	
Ice and Waves	Good	0.0	0.0	0.0	0.0	
	Fair	2.5	2.5	7.5	5.0	
	Inadequate	5.0	5.0	15.0	10.0	
Fender System	Good	0.0	0.0	0.0	0.0	
	Fair	2.5	2.5	5.0	0.0	
	Inadequate	5.0	5.0	10.0	0.0	
Foundation	Excellent	0.0	0.0	0.0	0.0	
	Fair	5.0	10.0	2.5	5.0	
	Problems	10.0	15.0	5.0	10.0	
Construction and Design	Excellent	0.0	0.0	0.0	0.0	
	Fair	5.0	5.0	5.0	15.0	
	Weak	10.0	10.0	10.0	30.0	
Biological Attack	None	0.0	0.0	0.0	0.0	
	Some	0.0	0.0	15.0	0.0	
	Advanced	0.0	0.0	30.0	0.0	
					Sum WC =	0.0

Notes:
1. AA > TUL, calculation not required

Remaining Useful Residual Life

URL = [TUL x (100 – WC) x CF] – AA

= 0 years

Project: Parry Sound Harbour
Component: **Structure C**
Steel sheet pile

Project No: 60749231
Date: Jan-24

Year Constructed = 1967 years
Actual Age, AA = 57 years

Theoretical Useful Life, TUL = 80 years

Foundations	Steel Sheet Piles	80
	Steel Pipe Piles	80
	Timber Piles	40
Concrete on Timber Foundations		60
Concrete on Steel Foundations		60
Timber Superstructures		40
Pavement		20
Breakwaters / Rock Protection		100

Compensating Factor, Selected CF = 1

Severe deterioration	0.7
Considerable Deterioration	0.8
Average Deterioration	0.9
Normal Condition	1.0

Weighting Coefficient, calculated WC = 0

		Steel	Concrete	Timber	Rock	Selected
Use	Normal	0.0	0.0	0.0	0.0	0.0
	Heavy	7.5	5.0	10.0	0.0	
	Abusive	15.0	10.0	20.0	0.0	
Exposure to Salinity and Polution	None	0.0	0.0	0.0	0.0	0.0
	Alternating	10.0	10.0	2.5	0.0	
	Concentrated	25.0	20.0	5.0	0.0	
Sea Condition	Mild (0 to 3')	0.0	0.0	0.0	0.0	0.0
	Average (3 to 6')	2.5	2.5	2.5	5.0	
	Severe (>6')	5.0	5.0	5.0	10.0	
Ice and Waves	Good	0.0	0.0	0.0	0.0	0.0
	Fair	2.5	2.5	7.5	5.0	
	Inadequate	5.0	5.0	15.0	10.0	
Fender System	Good	0.0	0.0	0.0	0.0	0.0
	Fair	2.5	2.5	5.0	0.0	
	Inadequate	5.0	5.0	10.0	0.0	
Foundation	Excellent	0.0	0.0	0.0	0.0	0.0
	Fair	5.0	10.0	2.5	5.0	
	Problems	10.0	15.0	5.0	10.0	
Construction and Design	Excellent	0.0	0.0	0.0	0.0	0.0
	Fair	5.0	5.0	5.0	15.0	
	Weak	10.0	10.0	10.0	30.0	
Biological Attack	None	0.0	0.0	0.0	0.0	0.0
	Some	0.0	0.0	15.0	0.0	
	Advanced	0.0	0.0	30.0	0.0	
					Sum WC =	0.0

Remaining Useful Residual Life

URL = [TUL x (100 – WC) x CF] – AA

= 23 years

Project: Parry Sound Harbour

Project No: 60749231

Component: **Structure C**
Concrete Deck Superstructure

Date: Jan-24

Year Constructed = 1967 years
Actual Age, AA = 57 years

Theoretical Useful Life, TUL = 60 years

Foundations	Steel Sheet Piles	80
	Steel Pipe Piles	80
	Timber Piles	40
Concrete on Timber Foundations		60
Concrete on Steel Foundations		60
Timber Superstructures		40
Pavement		20
Breakwaters / Rock Protection		100

Compensating Factor, Selected CF = 0.9

Severe deterioration	0.7
Considerable Deterioration	0.8
Average Deterioration	0.9
Normal Condition	1.0

Weighting Coefficient, calculated WC = 0

		Steel	Concrete	Timber	Rock	Selected
Use	Normal	0.0	0.0	0.0	0.0	0.0
	Heavy	7.5	5.0	10.0	0.0	
	Abusive	15.0	10.0	20.0	0.0	
Exposure to Salinity and Polution	None	0.0	0.0	0.0	0.0	0.0
	Alternating	10.0	10.0	2.5	0.0	
	Concentrated	25.0	20.0	5.0	0.0	
Sea Condition	Mild (0 to 3')	0.0	0.0	0.0	0.0	0.0
	Average (3 to 6')	2.5	2.5	2.5	5.0	
	Severe (>6')	5.0	5.0	5.0	10.0	
Ice and Waves	Good	0.0	0.0	0.0	0.0	0.0
	Fair	2.5	2.5	7.5	5.0	
	Inadequate	5.0	5.0	15.0	10.0	
Fender System	Good	0.0	0.0	0.0	0.0	0.0
	Fair	2.5	2.5	5.0	0.0	
	Inadequate	5.0	5.0	10.0	0.0	
Foundation	Excellent	0.0	0.0	0.0	0.0	0.0
	Fair	5.0	10.0	2.5	5.0	
	Problems	10.0	15.0	5.0	10.0	
Construction and Design	Excellent	0.0	0.0	0.0	0.0	0.0
	Fair	5.0	5.0	5.0	15.0	
	Weak	10.0	10.0	10.0	30.0	
Biological Attack	None	0.0	0.0	0.0	0.0	0.0
	Some	0.0	0.0	15.0	0.0	
	Advanced	0.0	0.0	30.0	0.0	
					Sum WC =	0.0

Remaining Useful Residual Life

URL = [TUL x (100 – WC) x CF] – AA

= 0 years

Project: Parry Sound Harbour

Project No: 60749231

Component: **Structure D**
Timber Pile Substructure

Date: Jan-24

Year Constructed = 1931 years
Actual Age, AA = 93 years

Theoretical Useful Life, TUL = 40 years

Foundations	Steel Sheet Piles	80
	Steel Pipe Piles	80
	Timber Piles	40
Concrete on Timber Foundations		60
Concrete on Steel Foundations		60
Timber Superstructures		40
Pavement		20
Breakwaters / Rock Protection		100

Compensating Factor, Selected CF =

Severe deterioration	0.7
Considerable Deterioration	0.8
Average Deterioration	0.9
Normal Condition	1.0

Weighting Coefficient, calculated WC = 0

		Steel	Concrete	Timber	Rock	Selected
Use	Normal	0.0	0.0	0.0	0.0	
	Heavy	7.5	5.0	10.0	0.0	
	Abusive	15.0	10.0	20.0	0.0	
Exposure to Salinity and Polution	None	0.0	0.0	0.0	0.0	
	Alternating	10.0	10.0	2.5	0.0	
	Concentrated	25.0	20.0	5.0	0.0	
Sea Condition	Mild (0 to 3')	0.0	0.0	0.0	0.0	
	Average (3 to 6')	2.5	2.5	2.5	5.0	
	Severe (>6')	5.0	5.0	5.0	10.0	
Ice and Waves	Good	0.0	0.0	0.0	0.0	
	Fair	2.5	2.5	7.5	5.0	
	Inadequate	5.0	5.0	15.0	10.0	
Fender System	Good	0.0	0.0	0.0	0.0	
	Fair	2.5	2.5	5.0	0.0	
	Inadequate	5.0	5.0	10.0	0.0	
Foundation	Excellent	0.0	0.0	0.0	0.0	
	Fair	5.0	10.0	2.5	5.0	
	Problems	10.0	15.0	5.0	10.0	
Construction and Design	Excellent	0.0	0.0	0.0	0.0	
	Fair	5.0	5.0	5.0	15.0	
	Weak	10.0	10.0	10.0	30.0	
Biological Attack	None	0.0	0.0	0.0	0.0	
	Some	0.0	0.0	15.0	0.0	
	Advanced	0.0	0.0	30.0	0.0	
					Sum WC =	0.0

Notes:

1. AA > TUL, calculation not required

Remaining Useful Residual Life

URL = [TUL x (100 – WC) x CF] – AA

= 0 years

Project: Parry Sound Harbour
Component: **Structure D**
Concrete Deck Superstructure

Project No: 60749231
Date: Jan-24

Year Constructed = 1931 years
Actual Age, AA = 93 years

Theoretical Useful Life, TUL = 60 years

Foundations	Steel Sheet Piles	80
	Steel Pipe Piles	80
	Timber Piles	40
Concrete on Timber Foundations		60
Concrete on Steel Foundations		60
Timber Superstructures		40
Pavement		20
Breakwaters / Rock Protection		100

Compensating Factor, Selected CF =

Severe deterioration	0.7
Considerable Deterioration	0.8
Average Deterioration	0.9
Normal Condition	1.0

Weighting Coefficient, calculated WC = 0

		Steel	Concrete	Timber	Rock	Selected
Use	Normal	0.0	0.0	0.0	0.0	
	Heavy	7.5	5.0	10.0	0.0	
	Abusive	15.0	10.0	20.0	0.0	
Exposure to Salinity and Polution	None	0.0	0.0	0.0	0.0	
	Alternating	10.0	10.0	2.5	0.0	
	Concentrated	25.0	20.0	5.0	0.0	
Sea Condition	Mild (0 to 3')	0.0	0.0	0.0	0.0	
	Average (3 to 6')	2.5	2.5	2.5	5.0	
	Severe (>6')	5.0	5.0	5.0	10.0	
Ice and Waves	Good	0.0	0.0	0.0	0.0	
	Fair	2.5	2.5	7.5	5.0	
	Inadequate	5.0	5.0	15.0	10.0	
Fender System	Good	0.0	0.0	0.0	0.0	
	Fair	2.5	2.5	5.0	0.0	
	Inadequate	5.0	5.0	10.0	0.0	
Foundation	Excellent	0.0	0.0	0.0	0.0	
	Fair	5.0	10.0	2.5	5.0	
	Problems	10.0	15.0	5.0	10.0	
Construction and Design	Excellent	0.0	0.0	0.0	0.0	
	Fair	5.0	5.0	5.0	15.0	
	Weak	10.0	10.0	10.0	30.0	
Biological Attack	None	0.0	0.0	0.0	0.0	
	Some	0.0	0.0	15.0	0.0	
	Advanced	0.0	0.0	30.0	0.0	

Sum WC = 0.0

Notes:
1. AA > TUL, calculation not required

Remaining Useful Residual Life

URL = [TUL x (100 – WC) x CF] – AA

= 0 years

Appendix F

Berthing Energy Calculations and Tables

SHEET NO. 1 **OF** 7

AECOM

JOB TITLE PARRY SOUND/ BERTHING CALCULATIONS
PROJECT NO. 60719231
COMPUTED BY AY **DATE** January 11, 2024
VERIFIED BY **DATE**

1.0 Cruise Ships Visiting Parry Sound, ON (2023)

Vessel	**Mooring**	**Gross Tonnage (gt)**	**DWT**	**Dimensions (m)**			**Max Persons**
				Length	**Moulded Beam**	**Design Draft**	
Le Bellot - Ponant	Dock	9976	1359	131.5	18.0	4.6	302
Le Dumont-d'Urville - Ponant	Dock	9976	1377	131.5	18.0	4.6	302
HANSEATIC Inspiration	Dock	15650	1800	138.0	22.0	5.6	405
Pearl Mist	Dock	5109	700	99.1	16.8	3.7	290
Viking Polaris	Anchor	30150	4059	205.0	23.5	6.0	646
Viking Octantis	Anchor	30150	4059	205.0	23.5	6.0	646
Island Queen V	Dock	525.9	*Unknown*	38.9	9.2	2.9	550+

**Note: Above table values are an estimate based on publicly-available information.*

2.0 CHBDC (CSA S6-19) Method

2.1 Vessel Collision Energy

Although CSA S6-19 does not directly address berthing forces/energies for docking vessels, methods for calculating vessel collision energy and head-on ship-to-pier collision force are provided.

The CSA S6.1-19 Commentary notes that hydrodynamic mass coefficients used to calculate vessel collision energy are smaller than those generally used in berthing calculations.

Vessel Collision Energy (Cl A3.3.7):

$$KE = \frac{(C_H)(W)(V)^2}{2 \times 10^3}$$

The following values for C_H shall be used:

a) for large under-keel clearances (≥ 0.5*draft): 1.05
b) for small under-keel clearances (≤0.1*draft): 1.25

Values for C_H may be interpolated for intermediate under-keel clearances.
Depth sounding results not available at time of calculation, so both clearance cases will be calculated.

V = collision velocity, m/s
Berthing velocities depend on navigation conditions and vessel size. Typical values used in BS-6459 and available research range from 0.1 to 0.7 m/s for vessels with DWT ≤ 5000.

W = vessel displacement tonnage, t
Due to missing information, W is assumed as to be same as GT

AECOM

JOB TITLE PARRY SOUND/ BERTHING CALCULATIONS
PROJECT NO. 60719231
COMPUTED BY AY **DATE** January 11, 2024
VERIFIED BY **DATE**

The following table presents Vessel Collision Energy, KE, in MN*m for collision velocities ranging from 0.1 to 1.0 m/s:

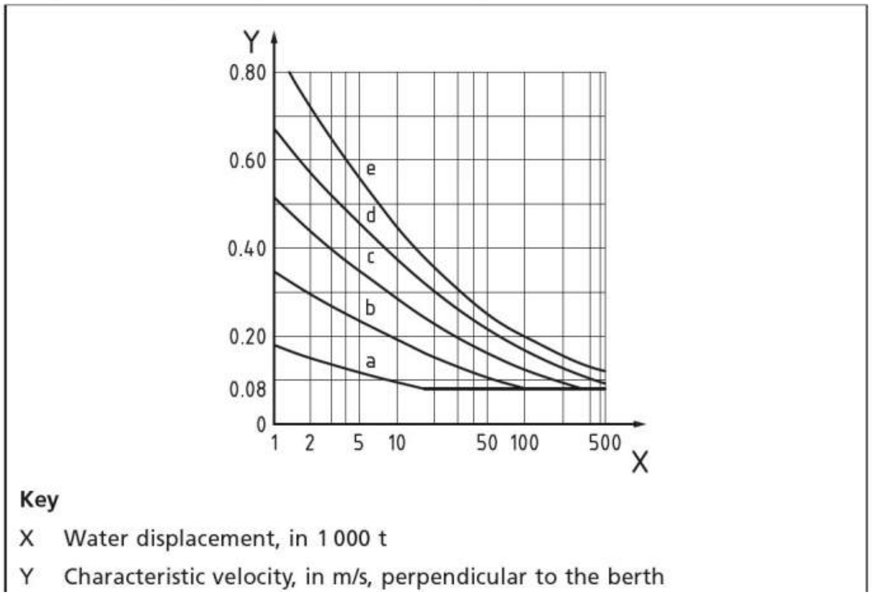
Vessel	**UKC[A] (m)**	**Mass[B] (t)**	**Vessel Collision Energy (MN*m)**					
			Collision Velocity (m/s)					
			0.1	**0.25**	**0.4**	**0.55**	**0.7**	**1.0**
*Assuming under-keel clearances ≥ 0.5 * draft , C_H = 1.05*								
Le Bellot - Ponant	≥ 2.3	8000	0.042	0.263	0.672	1.271	2.058	4.200
Le Dumont-d'Urville - Ponant	≥ 2.3	8000	0.042	0.263	0.672	1.271	2.058	4.200
HANSEATIC Inspiration	≥ 2.8	11500	0.060	0.377	0.966	1.826	2.958	6.038
Pearl Mist	≥ 1.85	5000	0.026	0.164	0.420	0.794	1.286	2.625
Viking Polaris	≥ 3	19000	0.100	0.623	1.596	3.017	4.888	9.975
Viking Octantis	≥ 3	19000	0.100	0.623	1.596	3.017	4.888	9.975
Island Queen V	≥ 1.435	800	0.004	0.026	0.067	0.127	0.206	0.420
*Assuming under-keel clearances ≤ 0.1 * draft , C_H = 1.25*								
Le Bellot - Ponant	≤ 0.46	8000	0.050	0.313	0.800	1.513	2.450	5.000
Le Dumont-d'Urville - Ponant	≤ 0.46	8000	0.050	0.313	0.800	1.513	2.450	5.000
HANSEATIC Inspiration	≤ 0.56	11500	0.072	0.449	1.150	2.174	3.522	7.188
Pearl Mist	≤ 0.37	5000	0.031	0.195	0.500	0.945	1.531	3.125
Viking Polaris	≤ 0.6	19000	0.119	0.742	1.900	3.592	5.819	11.875
Viking Octantis	≤ 0.6	19000	0.119	0.742	1.900	3.592	5.819	11.875
Island Queen V	≤ 0.287	800	0.005	0.031	0.080	0.151	0.245	0.500

[A] *: Under-Keel Clearance*

[B] *: Mass displaced is assumed based on values for similar ship sizes in PIANC WG 121 (2014), Table C-1.*

Note design berthing velocities from BS-6349-4:2014, Fig 9:

Figure 9 **Design berthing velocity as function of navigation conditions and size of vessel**



2.2 Ship Collision Force on Pier

Head-on ship collision force on pier (Cl A3.3.8):

$$P_S = (DWT)^{0.5}\ (V/8.4)$$

The following table presents Ship Collision Force on Pier, P_S, in MN for different collision velocities:

Vessel	Ship Collision Force (MN)							
	Collision Velocity (m/s)							
	0.1	0.25	0.4	0.55	0.7	1.0	2.0	3.0
Le Bellot - Ponant	0.4389	1.0972	1.7555	2.4138	3.0721	4.3886	8.777	13.17
Le Dumont-d'Urville - Ponant	0.4418	1.1044	1.7670	2.4297	3.0923	4.4176	8.835	13.25
HANSEATIC Inspiration	0.5051	1.2627	2.0203	2.7779	3.5355	5.0508	10.102	15.15
Pearl Mist	0.3150	0.7874	1.2599	1.7323	2.2048	3.1497	6.299	9.449
Viking Polaris	0.7585	1.8961	3.0338	4.1715	5.3092	7.5846	15.169	22.75
Viking Octantis	0.7585	1.8961	3.0338	4.1715	5.3092	7.5846	15.169	22.75
Island Queen V								

3.0 BS 6349-4:2014 Method

BS 6349-4 is the Maritime Works - Code of Practice for Design of Fendering and Mooring Systems, published by the British Standards Institution. The standard provides direct considerations for calculating berthing forces.

3.1 Characteristic Berthing Energy for Alongside Berthing

Cl 5.1 requires a berthing energy factor be established, with typical values of 1.5 for low-risk situations and 2.0 for high-risk situations. The characteristic berthing energy (E_C) should be multiplied by a berthing energy factor to determine the design berthing energy (E_D).

Characteristic berthing energy (Cl 5.2.1):

$$E_C = 0.5 C_M M_D (V_B)^2 C_E C_S C_C$$

3.1.1 Hydrodynamic Mass Coefficient, C_M :

$$C_M = 1 + \frac{2D_V}{B}$$

For under-keel distances greater than $0.1{*}D_V$

Where D_V = Draft
B = Beam

Vessel	**Draft D_V (m)**	**Beam B (m)**	**C_M**
Le Bellot - Ponant	4.6	18.0	1.51
Le Dumont-d'Urville - Ponant	4.6	18.0	1.51
HANSEATIC Inspiration	5.6	22.0	1.51
Pearl Mist	3.7	16.8	1.44
Viking Polaris	6.0	23.5	1.51
Viking Octantis	6.0	23.5	1.51
Island Queen V	2.9	9.2	1.62

(Typical values given as 1.3 to 1.9 in Cl 5.2.4)

3.1.2 Eccentricity Coefficient, C_E:

$$C_E = \frac{K^2 + R^2\cos^2\gamma}{K^2 + R^2}$$

where K is calculated from the formula:

$$K = (0.19C_b + 0.11)L_{BP}$$

R is the distance between the point of contact to the centre of mass of the vessel. Assume R = 0.5*L

Υ is the angle between the vessel's velocity vector and the line joining the point of impact to the vessel's centre of mass.

Typical ranges of C_b from BS 6349-1-1:2013 Table D.2:

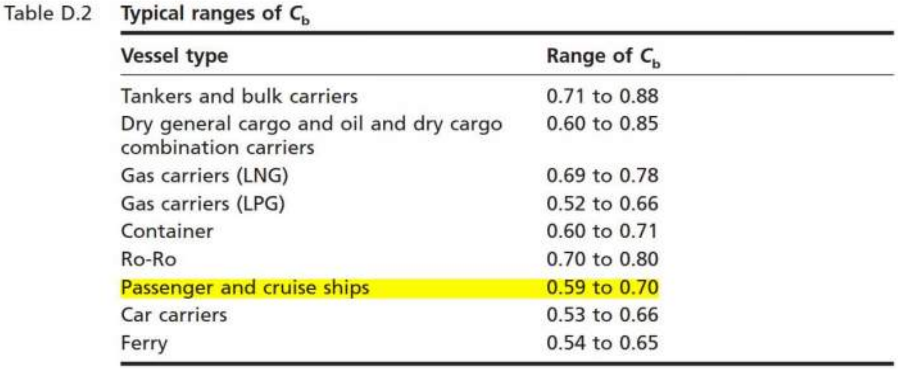


Table D.2 **Typical ranges of C_b**

Vessel type	**Range of C_b**
Tankers and bulk carriers	0.71 to 0.88
Dry general cargo and oil and dry cargo combination carriers	0.60 to 0.85
Gas carriers (LNG)	0.69 to 0.78
Gas carriers (LPG)	0.52 to 0.66
Container	0.60 to 0.71
Ro-Ro	0.70 to 0.80
Passenger and cruise ships	0.59 to 0.70
Car carriers	0.53 to 0.66
Ferry	0.54 to 0.65

$$C_b = \frac{M_D}{L_{BP}Bd\rho_W}$$

Vessel	**Assumed M_D (t)**	**Length L_{BP} (m)**	**Beam B (m)**	**Draft d (m)**	**Water density**	**C_b**
Le Bellot - Ponant	8000	131.5	18.0	4.6	1.0000	0.73
Le Dumont-d'Urville - Ponant	8000	131.5	18.0	4.6	1.0000	0.73
HANSEATIC Inspiration	11500	138.0	22.0	5.6	1.0000	0.68
Pearl Mist	5000	99.1	16.8	3.7	1.0000	0.81
Viking Polaris	19000	205.0	23.5	6.0	1.0000	0.66
Viking Octantis	19000	205.0	23.5	6.0	1.0000	0.66
Island Queen V	800	38.9	9.2	2.9	1.0000	0.78

Eccentricity Coefficient (C_E) for different Υ to test sensitivity:

Vessel	**K**	**Assumed *R***	**Eccentricity Coefficient (C_E)**					
			Υ =					
			85	**75**	**65**	**55**	**45**	**35**
Le Bellot - Ponant	32.82	65.8	0.2056	0.2531	0.3425	0.4628	0.5997	0.7366
Le Dumont-d'Urville - Ponant	32.82	65.8	0.2056	0.2531	0.3425	0.4628	0.5997	0.7366
HANSEATIC Inspiration	32.92	69.0	0.1916	0.2399	0.3309	0.4534	0.5927	0.7320
Pearl Mist	26.18	49.6	0.2242	0.2707	0.3579	0.4755	0.6091	0.7428
Viking Polaris	48.15	102.5	0.1870	0.2357	0.3271	0.4503	0.5904	0.7305
Viking Octantis	48.15	102.5	0.1870	0.2357	0.3271	0.4503	0.5904	0.7305
Island Queen V	10.01	19.4	0.2156	0.2626	0.3508	0.4697	0.6048	0.7400

3.1.3 Softness Coefficient, C_S :

Cl 5.2.6 notes that the softness coefficient or vessel flexibility factor (CS) should generally be taken as 1.0.

3.1.4 Berth Configuration Coefficient, C_C :

Cl 5.2.7 assigns a value of 0.9 for solid quay walls under parallel approach (berthing angles < 5 degrees) and underkeel clearances less than 15% of vessel draughts, and a value of 1.0 for all other cases.

Recall $E_C = 0.5 C_M M_D (V_B)^2 C_E C_S C_C$

Berthing Energy, E $_C$, for Berthing Velocity (V $_B$) = 0.1 m/s:

Vessel	**Berthing Energy, E_C (kN*m)**					
	ϒ =					
	85	**75**	**65**	**55**	**45**	**35**
Le Bellot - Ponant	12.43	15.30	20.70	27.98	36.25	44.53
Le Dumont-d'Urville - Ponant	12.43	15.30	20.70	27.98	36.25	44.53
HANSEATIC Inspiration	16.62	20.82	28.71	39.34	51.43	63.52
Pearl Mist	8.07	9.75	12.89	17.12	21.94	26.75
Viking Polaris	26.84	33.82	46.94	64.62	84.73	104.83
Viking Octantis	26.84	33.82	46.94	64.62	84.73	104.83
Island Queen V	1.40	1.70	2.27	3.05	3.92	4.80

Berthing Energy, E $_C$, for Berthing Velocity (V $_B$) = 0.15 m/s:

Vessel	**Berthing Energy, E_C (kN*m)**					
	ϒ =					
	85	**75**	**65**	**55**	**45**	**35**
Le Bellot - Ponant	27.96	34.42	46.58	62.95	81.57	100.18
Le Dumont-d'Urville - Ponant	27.96	34.42	46.58	62.95	81.57	100.18
HANSEATIC Inspiration	37.40	46.85	64.60	88.52	115.72	142.91
Pearl Mist	18.17	21.93	29.00	38.53	49.36	60.19
Viking Polaris	60.39	76.10	105.62	145.40	190.64	235.87
Viking Octantis	60.39	76.10	105.62	145.40	190.64	235.87
Island Queen V	3.15	3.83	5.12	6.85	8.82	10.80

Berthing Energy, E $_C$, for Berthing Velocity (V $_B$) = 0.2 m/s:

Vessel	**Berthing Energy, E_C (kN*m)**					
	ϒ =					
	85	**75**	**65**	**55**	**45**	**35**
Le Bellot - Ponant	49.70	61.20	82.80	111.91	145.00	178.10
Le Dumont-d'Urville - Ponant	49.70	61.20	82.80	111.91	145.00	178.10
HANSEATIC Inspiration	66.49	83.28	114.84	157.36	205.72	254.07
Pearl Mist	32.30	38.99	51.56	68.49	87.75	107.00
Viking Polaris	107.36	135.29	187.78	258.49	338.91	419.33
Viking Octantis	107.36	135.29	187.78	258.49	338.91	419.33
Island Queen V	5.59	6.81	9.10	12.18	15.69	19.19

SHEET NO. 7 **OF** 7

JOB TITLE PARRY SOUND/ BERTHING CALCULATIONS
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Berthing Energy, E_C, for Berthing Velocity (V_B) = 0.25 m/s:

Vessel	**Berthing Energy, E_C (kN*m)**					
	ϒ =					
	85	**75**	**65**	**55**	**45**	**35**
Le Bellot - Ponant	77.66	95.62	129.38	174.85	226.57	278.29
Le Dumont-d'Urville - Ponant	77.66	95.62	129.38	174.85	226.57	278.29
HANSEATIC Inspiration	103.89	130.13	179.44	245.88	321.43	396.98
Pearl Mist	50.47	60.92	80.56	107.02	137.10	167.19
Viking Polaris	167.74	211.38	293.40	403.90	529.55	655.21
Viking Octantis	167.74	211.38	293.40	403.90	529.55	655.21
Island Queen V	8.74	10.64	14.22	19.04	24.51	29.99

Berthing Energy, E_C, for Berthing Velocity (V_B) = 0.5 m/s:

Vessel	**Berthing Energy, E_C (kN*m)**					
	ϒ =					
	85	**75**	**65**	**55**	**45**	**35**
Le Bellot - Ponant	310.64	382.48	517.50	699.42	906.28	1113.14
Le Dumont-d'Urville - Ponant	310.64	382.48	517.50	699.42	906.28	1113.14
HANSEATIC Inspiration	415.57	520.52	717.77	983.53	1285.73	1587.94
Pearl Mist	201.87	243.67	322.22	428.06	548.41	668.76
Viking Polaris	670.98	845.54	1173.60	1615.59	2118.21	2620.83
Viking Octantis	670.98	845.54	1173.60	1615.59	2118.21	2620.83
Island Queen V	34.96	42.57	56.87	76.14	98.05	119.97

Appendix G

Bollard Capacity and Loading Calculations

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PROJECT/JOB NO. 60719231 CALCULATION NO.
COMPUTED BY AY DATE Jan 8/2024
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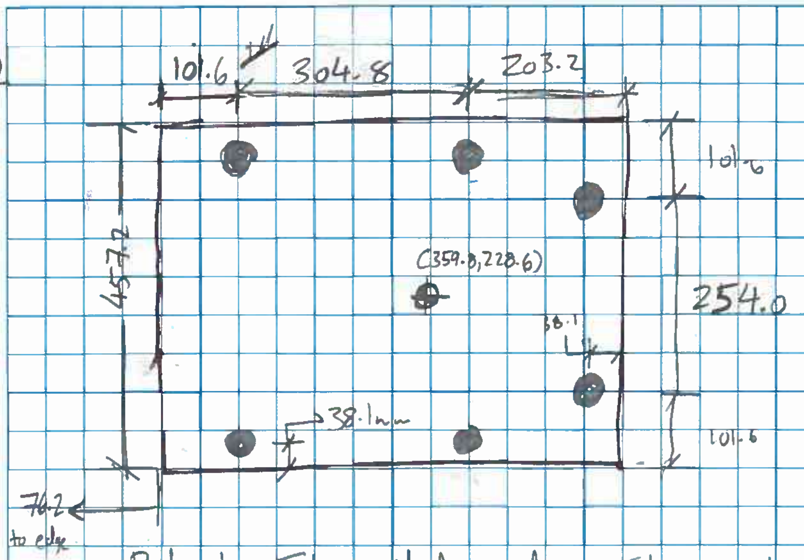
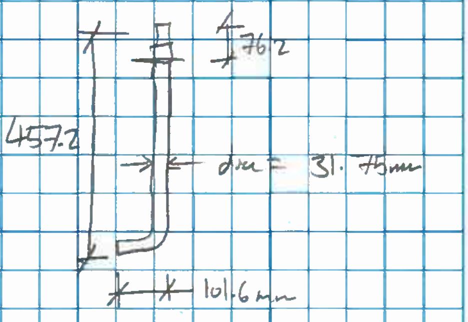
SCALE ~~NA~~ SHEET NO. 1 OF 10

1.0



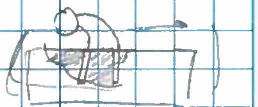
mm

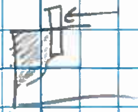
Centroid $x = 359.8$
$y = 228.6$



Relevant Failure Modes under Shear Loading:

- Steel failure preceded by concrete spall
- Concrete pryout for anchors far from free edge
- Concrete Breakout





↑ (See Clauses D.5.1.1 and D.7.2.1)

CSA A23.3 Appendix D.

2.0

2.1 Cl D.7.1.2 -b) Shear Resistance of hooked bolt anchor

$$V_{sar} = A_{se,V} \times \phi_s \times 0.6 f_{uta} \times R \qquad \phi_s = 0.85$$

Where, $A_{se,V}$ = effective cross-sectional area of a single anchor $= 791.7\,mm^2$
$f_{uta} \leq 1.9 f_{ya}$ or 860 MPa → assume $f_{ya} = 210$ MPa
R = 0.75 (gov. by ductile steel)
0.65 (gov by brittle steel)
1.00 (gov by concrete breakout)
} Shear.

$$\boxed{V_{sar,\ ductile\ steel} = 120.8\ kN}$$

$V_{sar,\ Brittle\ steel} = 104.7$ kN

$V_{cpr,\ Concrete\ Pr}$ = 160.9 kN

SHEAR CAPACITY OF SINGLE ANCHOR.

5x5 = 1 in

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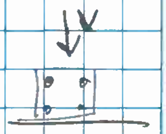
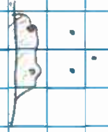
2.2 Concrete Breakout Resistance of Anchor in Shear:



2.2.1 Cl D.7.2.1 – b) Shear force perpendicular to edge on group

Assuming Front Anchors Critical $(V_{cbgr} = \frac{V_c}{2})$

$$V_{cbgr} = \frac{A_{vc}}{A_{vco}} \psi_{ec,v} \times \psi_{ed,v} \times \psi_{c,v} \times \psi_{h,v} \times V_{br}$$



Cl D.7.2.5 $\psi_{ec,v} = 1.0$ (no perpendicular eccentricity)

Cl D.7.2.6 $C_{a2} \geq 1.5 C_{a1}$ ✓

C_a (perpendicular edge) $= 76.1 + 101.6$
$= 177.7mm$
C_{a2} (other edge) $\gg 1.5 \times 177.7mm$

$\therefore \psi_{ed,v} = 1.0$

Cl D.7.2.7 For anchors in cracked concrete without reinforcement, (unrepaired) $\psi_{c,v} = 1.0$

For anchors in cracked concrete w/ 15M or greater reinforcement (repaired) $\psi_{c,v} = 1.2$

Cl D.7.2.8 $h_a \not< 1.5 C_{a1}$

$\therefore \psi_{h,v} = 1.0$

Cl D.7.2.2 V_{br} shall not exceed the smaller of a) or b):

$$a)\ V_{br} = 0.58 \left(\frac{l_e}{d_a}\right)^{0.2} \sqrt{d_a} \times \phi_c \times \lambda_a \times \sqrt{f'_c} \times C_a^{1.5} \times R$$

$d_a = 31.75mm$ $\phi_c = 0.65$
$l_e \leq 8d_a = 254mm$
$f'_c = 20$ MPa (assumed for safety)
$C_{a1} = 177.7mm$*
$R = 0.75$ (ductile) / 0.65 (Brittle) / 1.0 (concrete) (gov. by)
$\lambda_a = 1.0$ (normal density concrete)

$V_{br(a)} = 34.1$ kN $(R = 1)$
22.4 kN $(R = 0.65)$
25.6 kN $(R = 0.75)$

* Assumed 3" (76mm) cover to plate, per 1931 dwgs (after wharf reconstruction, Public Works of Canada)

5x5 = 1 in

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b) $V_{br} = 3.75 \times \lambda_a \times \phi_c \sqrt{f'_c} \times c_{a1}^{1.5} R$

$V_{br(b)} = 25.8 \text{ kN}$ $(R = 1)$
16.8 kN $(R = 0.65)$
19.4 kN $(R = 0.75)$

$V_{br(b)} < V_{br(c)}$

$\Rightarrow \underline{V_{br} = V_{br(b)}}$

Cl D.7.2.1 A_{Vco}

$A_{Vco} = 4.5 (C_{a1})^2$

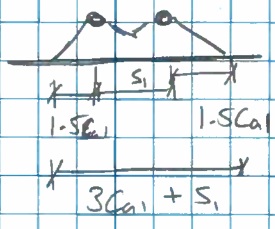
$\underline{A_{Vco} = 142098 \text{ mm}^2}$

Fig. D.13 c). A_{Vc}

2 anchors in breakout group.

$A_{Vc} = \text{Length} \times \text{Depth}$

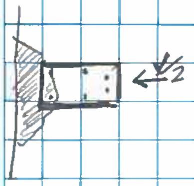
$= (3 C_{a1} + S_1) \times 1.5 C_{a1}$



$S_1 = 381 \text{ mm}$

$C_{a1} = 177.7$

$\underline{A_{Vc} = 243653 \text{ mm}^2}$



Recall: $V_{cbgr} = \frac{A_{Vc}}{A_{Vco}} \times \Psi_{ec,V} \times \Psi_{ed,V} \times \Psi_{c,V} \times \Psi_{h,V} \times V_{br}$

Case 1:
$V_{cbgr,1} = 44.3 \text{ kN}$
$R = 1.0$ (Cone B/O governs, no edge rft)
$\Psi_{c,V} = 1.0$ (no edge rft).

Case 2:
$V_{cbgr,2} = 28.8 \text{ kN}$
$R = 0.65$ (brittle steel governs)
$\Psi_{c,V} = 1.0$ (no edge rft)

Case 3
$V_{cbgr,3} = 33.2 \text{ kN}$
$R = 0.75$ (ductile steel governs)
$\Psi_{c,V} = 1.0$ (no edge rft)

Case 4:
$V_{cbgr,4} = 61.1 \text{ kN}$
$R = 1.15$ (Cone B/O governs, edge rft)
$\Psi_{c,V} = 1.2$ (15M edge rft, no stirrups)

Case 5
$V_{cbgr,5} = 34.5 \text{ kN}$
$R = 0.65$ (brittle steel governs)
$\Psi_{c,V} = 1.2$ (15M edge rft, no stirrups)

5x5 = 1 in

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Case 6:

$V_{cbgr,6} = 39.8 \text{ kN}$

$R = 0.75$ (ductile steel governs)
$\psi_{c,V} = 1.2$ (15M edge rft, no stirrups.

Case 7:

$V_{cbgr,7} = 71.3 \text{ kN}$

$R = 1.15$ (conc. B/O governs, 15M edge rft)
$\psi_{c,V} = 1.4$ (15M edge rft w/ stirrups)

Case 8:

$V_{cbgr,8} = 40.3 \text{ kN}$

$R = 0.65$ (brittle steel governs)
$\psi_{c,V} = 1.4$ (15M edge rft w/ stirrups)

Case 9:

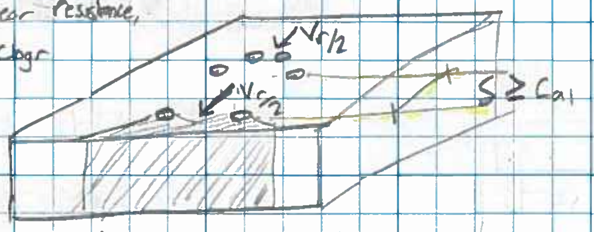
$V_{cbgr,9} = 46.5 \text{ kN}$

$R = 0.75$ (ductile steel governs)
$\psi_{c,V} = 1.4$ (15M edge rft w/ stirrups)

Note: for V_{cbgr} in direction parallel to edge, $V_{cbgr,\parallel} = 2 \times V_{cbgr,\perp}$. After accounting for eccentricity, $e'_{V,\parallel} = 76.$
$\psi_{ec,V} = 0.778$

$$\Rightarrow V_{cbgr,\parallel} = \underline{2 \times (0.778)} V_{cbgr,\perp} = 1.56 V_{cbgr,\perp}$$

NB: Total shear resistance,
$V_r = 2V_{cbgr}$



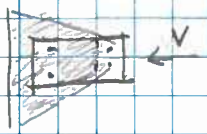
$S = 304.8$
$C_a = 177.7$

(From CSA A23.3 Appendix D Fig D.13)

2.2.2 (Cl D.7.2.1 - b) Shear force perpendicular to edge in group (rear anchor failure).

Assuming rear anchors critical

($V_{cbgr} = V_r$)



$$V_{cbgr} = \frac{A_{Vc}}{A_{Vco}} \overset{1}{\psi_{ec,V}} \times \overset{.0}{\psi_{ed,V}} \times \overset{.0}{\psi_{c,V}} \times \overset{1.0}{\psi_{h,V}} \times V_{br}$$

$$V_{br(a)} = 0.58\left(\frac{l_e}{d_a}\right)^{0.2}\sqrt{d_a} \times \phi_c \times \lambda_a \times \sqrt{f'_c} \times c_{a1}^{1.5} \times R$$

$c_{a1} = 76.2 + 101.6 + 304.8$
$\Rightarrow c_{a1} = 482.6 \text{ mm}$

$V_{br(a)} = R \times 152.7 \text{ kN}$

*Note: V_r values calculated here are for cases where the failure surface occurs at the rear anchor group. This is the case if anchors are welded to the bollard plate. These values assume an edge distance based on undamaged/spalled concrete. These values do NOT represent bollards in their current/damaged state.

5x5 = 1 in

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$$V_{br(b)} = 3.75 \times \lambda_a \times \phi_c \times \sqrt{f'_c} \times C_{a1}^{1.5} \times R$$

$$V_{br(b)} = R \times 115.6 \text{ kN}$$

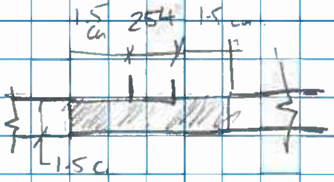
$$V_{br(b)} < V_{br(a)}$$

$$\therefore V_{br} = \phi_c \times R \times 217.8 \text{kN}$$

Recall $V_{cbgr} = \frac{A_{vc}}{A_{vco}} \times \Psi_{ec,V} \times \Psi_{ed,V} \times \Psi_{c,V} \times V_{b}$

$$A_{vc} = [(3 \times C_{a}) + 254](1.5 C_{a})$$

$$= 1,323,868 \text{mm}^2$$



$$A_{vco} = 4.5\, C_a^2$$

$$= 1,048,062$$

$$A_{vc} \leq 2 \times A_{vco}$$ ✓

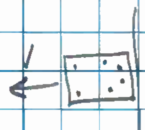
Rear Anchor Resistance (total bollard concrete breakout): * SEE NOTE ON PREVIOUS PAGE.

$V_{r,1} = 146.0$ kN (Concrete b/o governs, no edge rft)

$V_{r,2} = 201.5$ kN (Concrete b/o governs, 15M edge rft)

$V_{r,3} = 235.0$ kN (Concrete b/o governs 15M edge rft + stirrups)

2.3 Concrete Pryout Resistance for a group of anchors in Shear: (West side bollards)



Cl D.7.3-b) $V_{cpgr} = K_{cp} \times N_{cpgr}$

$$K_{cp} = 1.0 \text{ for } h_{ef} < 65\text{mm}$$
$$= 2.0 \text{ for } h_{ef} \geq 65\text{mm}$$

$h_{ef} > 65$ mm $\therefore$ $K_{cp} = 2.0$

$$N_{cpgr} = \underset{\text{Cl.D.6.2.1}}{N_{cbgr}} = \frac{A_{Nc}}{A_{Nco}} \times \Psi_{ec,N} \times \Psi_{ed,N} \times \Psi_{c,N} \times \Psi_{cp,N} \times N_{br}$$

Cl D.6.2.2 $N_{br} = K_c \phi_c \lambda_a \times \sqrt{f'_c} \times h_{ef}^{1.5} \times R_c$

$K_c = 10$ (cast-in)

Assume $f'_c = 20$ MPa

$h_{ef} = 350$mm (approx.)

$$N_{br} = R \times 190.3 \text{ kN}$$

5x5 = 1 in

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$$N_{cpgr} = N_{cbgr} = \frac{A_{Nc}}{A_{Nco}} \times \psi_{ec,N} \times \psi_{ed,N} \times \psi_{c,N} \times \psi_{cp,N} \times N_{br}$$

Cl D.6.2.4 $\psi_{ec,N} = \dfrac{1}{\left(1 + \dfrac{2e'_N}{3h_{ef}}\right)}$

Recalling centroid location on plate is (359.8, 228.6)

Bollard horn location on plate is (177.8, 228.6).

$e'_N = 359.8 - 177.8$
$e'_N = 182.0 \text{mm}$

$\psi_{ec,N} = 0.743$

Cl D.6.2.5 $\psi_{ed,N}$

If $C_{a,min} \geq 1.5 h_{ef}$. $\psi_{ed,N} = 1.0$

$C_{a,min} = 177.7$
$1.5 \times h_{ef} = 525$

$C_{a,min} < 1.5 h_{ef}$.

$\therefore \psi_{ed,N} = 0.7 + 0.3 \times \dfrac{C_{a,min}}{1.5 h_{ef}}$

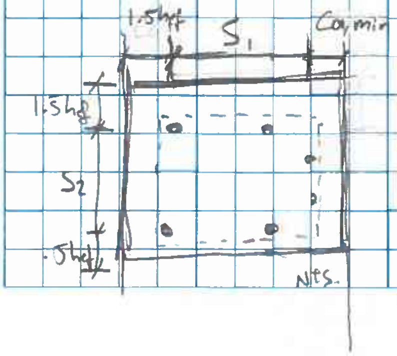
$\Rightarrow \psi_{ed,N} = 0.802$

Cl D.6.2.6 $\psi_{c,N} = 1.25$ Fr cast-in anchors (if analysis indicates no cracking at service)

use $\psi_{c,N} = 1.0$ (cracking @ Service).

Cl D.6.2.1 $A_{Nco} = 9 h_{ef}^2$

$\Rightarrow A_{Nco} = 1,102,500 \text{mm}^2$



$A_{Nc} = (C_{a,1} + S_1 + 1.5h_{ef})(1.5h_{ef} + 1.5h_{ef} + S_2)$

$C_{a,1} = 177.7, \quad S_1 = 508 \quad S_2 = 381$

$\Rightarrow A_{Nc} = 1,732,512 \text{ mm}^2$

5x5 = 1 in

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SCALE
SHEET NO. 7 OF 10

$$N_{cbgr} = \frac{A_{NC}}{A_{NCo}} \times \psi_{ec,N} \times \psi_{ed,N} \times \psi_{c,N} \times \psi_{cp,N} \times N_{br}$$

$N_{cbgr,1} = 178.2$ kN (R = 1.0, conc pryout governs)

$N_{cbgr,2} = 124.8$ kN (R = 0.7, brittle steel)

$N_{cbgr,3} = 142.6$ kN (R = 0.8, ductile steel)

Recall $V_{cpgr} = K_{cp} \times N_{cpg}$, $K_{cp} = 2.0$ for $h_{ef} \geq 65$mm

$V_{cpg} = 356.5$ kN (R = 1.0 conc. pryout governs)

$V_{cpgr,2} = 249.5$ kN (R = 0.7, brittle steel governs)

$V_{cpgr,3} = 285.2$ kN (R = 0.8, ductile steel governs).

(For West bollards).

5x5 = 1 in

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3.0 Tensile Resistance:

3.1 Resistance of a Anchor in tension:

Cl D.6.T.2 $N_{sar} = A_{se,N} \times \phi_s \times f_{uta} \times R$

$A_{se,N}$ = effective cross-sectional area of a single anchor = 791.7 mm²

$f_{uta} \leq 1.9 f_{ya}$ or 860 MPa

Assume f_{ya} = 210 MPa (CISC Hbk, Historic steel).

$\Rightarrow f_{uta} = 1.9 \times 210 MPa = 399$

N_{sar} = 268.5 kN (R = 1.0) Concrete BO/PO

$N_{sar,2}$ = 187.9 kN (R = 0.7) Brittle Steel.

$N_{sar,3}$ = 214.8 kN (R = 0.8) Ductile Steel.

Resistance of Single anchor in Tension.

3.2 Concrete Breakout Resistance of an anchor group in Tension

Cl D.6.2.1 $N_{cbg} = \frac{A_{Nc}}{A_{Nco}} \cdot \Psi_{ec,N} \times \Psi_{ed,N} \times \Psi_{c,N} \times \Psi_{cp,N} \times N_{br}$

Recall from section 2.3.

N_{cbgr} = 178.2 kN (R = 1.0, concrete BO/PO)

$N_{cbgr,2}$ = 124.8 kN (R = 0.7, brittle steel)

$N_{cbgr,3}$ = 142.6 kN (R = 0.8, ductile steel).

5x5 = 1 in

AECOM Delivering a better world

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SHEET NO. 9 OF 10

3.3 Pullout Resistance of Cast-in Anchor

Cl D.6.3.1 Th Factored pull-out resistance of a single cast-in anchor in tension shall not exceed:

$$N_{cpr} = \varphi_{c,p} \times N_{pr}$$

Cl D.6.3.6: $\varphi_{c,p} = 1.0$ (no analysis indicating no cracking at service)

Cl D.6.3.5 Factored pull-out resistance in tension of a single L-bolt:

$$N_{pr} = 0.9 \times \phi_c \times f'_c \times e_h \times d_a \times R$$

$\phi_c = 0.65$

$f'_c = 20$ MPa (assumed).

e_h = distance from inner surface of L-bolt shaft to tip.

$= 101.6 - 31.75$

$\Rightarrow e_h = 69.8$ mm

$d_a = 31.75$ mm

Eq^n Valid for $3d_a < e_h \leq 4.5\,d_a$

∴ Cannot be used to determine pull-out resistance in this situation, as the anchor hook length, e_h, is less than the minimum of $3d_a = 95.25$ mm

5x5 = 1 in

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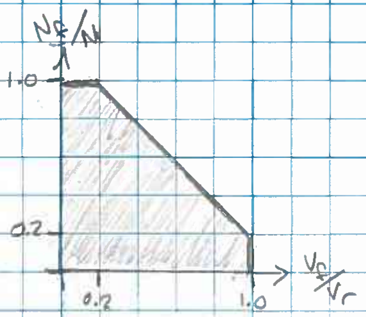
4.0 Interaction of Tensile and Shear Forces

If $\frac{V_f}{V_r} \leq 0.2$ $N_r > N_f$ (Full resistance in tension may be used).

If $\frac{N_f}{N_r} \leq 0.2$ $V_r \geq V_f$ (Full resistance in shear may be used).

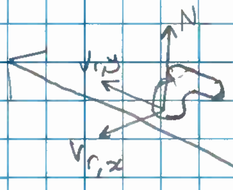
If $\frac{V_f}{V_r} > 0.2$ and $\frac{N_f}{N_r} > 0.2$, $\frac{N_f}{N_r} + \frac{V_f}{V_r} < 1.2$



5.0 Summary of Calculated Results

For front anchors only:

	W/out edge rft	w/ edge rft (15M min.)
$V_{r,x}$ =	88.6 kN	122.2 kN
$V_{r,y}$ =	138.2 kN	190.6 kN



For rear anchors: after front anchor spalling: $V_{r,x}$ = 82.2 kN

For rear anchors if welded to plate

	W/out edge rft	w/ edge rft (15M min.)
$V_{r,x}$ =	146.0 kN	201.5 kN
$V_{r,y}$ =	227.7 kN	314.3 kN

N_r = 178.2 kN

5x5 = 1 in

PROJECT: Parry Sound Harbour
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Bollard Capacity Summary

Values given in kN

Existing condition

	Failure Mode	Single Anchor		Anchor Group	
D.7.1	Steel strength in shear	Vsar =	120.8	Vsagr =	724.8
D.7.2	Concrete breakout strength in shear (outside anchors)	Vcbr =	44.3	Vcbgr =	88.6
	Concrete breakout strength in shear (inside anchors after outside spall)			Vcbgr =	82.2
D.7.3	Concrete pryout strength in shear			Vcpgr =	356.5
D.6.1	Steel strength in tension	Nsar =	214.8	Nsagr =	1288.8
D.6.2	Concrete breakout strength in tension	Nbr =	190.3	Ncbgr =	178.2
D.6.3	Pullout strength in tension	Pullout cannot be determined using Appendix D, however, it is not expected to govern			

Bollards are governed by Concrete Breakout strength in shear and in tension

Repaired condition (welded anchors)

	Failure Mode	Single Anchor		Anchor Group	
D.7.1	Steel strength in shear	Vsar=	120.8		724.8
D.7.2	Concrete breakout strength in shear			Vcbgr =	146.0
D.7.3	Concrete pryout strength in shear			Vcpgr =	356.5
D.6.1	Steel strength in tension	Nsar =	214.8	Nsagr =	1288.8
D.6.2	Concrete breakout strength in tension	Nbr =	190.3	Ncbgr =	178.2
D.6.3	Pullout strength in tension	Pullout cannot be determined using Appendix D, however, it is not expected to govern			

Bollards are governed by Concrete Breakout strength in shear and in tension

Repaired condition (with 15M edge rft)

	Failure Mode	Single Anchor		Anchor Group	
D.7.1	Steel strength in shear	Vsar =	120.8	Vsagr =	724.8
D.7.2	Concrete breakout strength in shear	Vcbr =	61.1	Vcbgr =	122.2
D.7.3	Concrete pryout strength in shear			Vcpgr =	356.5
D.6.1	Steel strength in tension	Nsar =	214.8	Nsagr =	1288.8
D.6.2	Concrete breakout strength in tension	Nbr =	190.3	Ncbgr =	178.2
D.6.3	Pullout strength in tension	Pullout cannot be determined using Appendix D, however, it is not expected to govern			

Bollards are governed by Concrete Breakout strength in shear

Repaired condition (with 15M edge rft and welded anchors)

	Failure Mode	Single Anchor		Anchor Group	
D.7.1	Steel strength in shear	Vsar =	120.8	Vsagr =	724.8
D.7.2	Concrete breakout strength in shear			Vcbgr =	201.5
D.7.3	Concrete pryout strength in shear			Vcpgr =	356.5
D.6.1	Steel strength in tension	Nsar =	214.8	Nsagr =	1288.8
D.6.2	Concrete breakout strength in tension	Nbr =	190.3	Ncbgr =	178.2
D.6.3	Pullout strength in tension	Pullout cannot be determined using Appendix D, however, it is not expected to govern			

Bollards are governed by Concrete Breakout strength in shear and in tension

Repaired condition (with 15M edge rft, welded anchors, and shear zone replaced with 35 MPa concrete)

	Failure Mode	Single Anchor		Anchor Group	
D.7.1	Steel strength in shear	Vsar =	120.8	Vsagr =	724.8
D.7.2	Concrete breakout strength in shear			Vcbgr =	266.5
D.7.3	Concrete pryout strength in shear			Vcpgr =	471.6
D.6.1	Steel strength in tension	Nsar =	214.8	Nsagr =	1288.8
D.6.2	Concrete breakout strength in tension	Nbr =	251.8	Ncbgr =	235.8
D.6.3	Pullout strength in tension	Pullout cannot be determined using Appendix D, however, it is not expected to govern			

Bollards are governed by Concrete Breakout strength in shear and in tension

Mooring Loads - Loading on Vessel

British Standards
BS 6349-1

Transverse Wind Force

$F_{TW} = C_{TW}\, \rho_a\, A_L\, V_w^2 \times 10^{-4}$ **267.2** kN

ρ_a = 1.221 kg/m3 mass density of air

A_L = 1220 m2 Longitudinal projected area of Pearl Mist

NBCC App C V_w = 21.5 m/s Wind Speed for Parry Sound for 10 year return

BS Fig G.1 $C_{TW,\ Aft}$ = 2.00 Transverse wind force drag coefficient, Aft

BS Fig G.1 $C_{TW,\ Forward}$ = 1.88 Transverse wind force drag coefficient, Forward

Longitudinal Wind Force

$F_{LW} = C_{LW}\, \rho_a\, A_L\, V_w^2 \times 10^{-4}$ **62.0** kN

BS Fig G.1 C_{LW} = 0.9 Longitudinal wind force drag coefficient

Vessel	Long Projected Area, A_y	Transverse Wind Force, F_{TW} (kN)	Longitudinal Wind Force, F_{LW} (kN)
Viking Octantis	4200	920	213
Viking Polaris	4200	920	213
Pearl Mist	1200	263	61
Le Dumont D'Urville	2300	504	117
Le Bellot	2050	449	104
Hanseatic Inspiration	2350	515	119
Island Queen	230	50	12

Current Loads on Vessel

Since the vessel is morred against the wharf, there would be no current loading in the transverse direction

British Standards
BS 6349-1

Longitudinal Current Force

$F_{LC} = C_{LC}\, C_{CL}\, \rho\, L_{BP}\, d_m\, V_c'^2 \times 10^{-3}$ **3.54** kN

Fig G.4 C_{LC} = 0.20 Longitudinal current drag force coefficient

Fig G.6 C_{CL} = 1.22 Depth Correction Factor for longitudinal current forces

ρ_w = 1000 kg/m3

L_{BP} = 99 m Length of vessel between perpendiculars

d_m = 3.66 m draft of vessel

V_c' = 0.2 m/s based on max recorded value on Seguin River (MNRF)

d = 7 m water depth

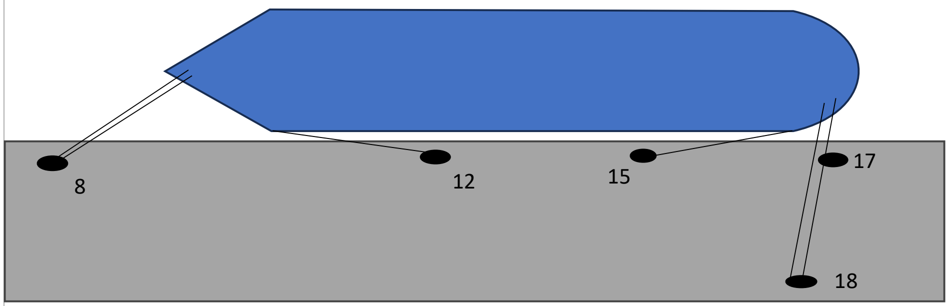
d/d_m = 1.9

Vessel	Length, L_{BP}	Draft ,dm	d/dm	C_{CL}	Long Current Force, F_{LC} (kN)
Viking Octantis	202.7	6.0	1.2	1.5	14.2
Viking Polaris	202.7	6.0	1.2	1.5	14.2
Pearl Mist	99	3.7	1.9	1.2	3.5
Le Dumont D'Urville	131.1	4.6	1.5	1.3	6.4
Le Bellot	131.1	4.6	1.5	1.3	6.4
Hanseatic Inspiration	138	5.6	1.3	1.5	9.3
Island Queen	40.2	1.8	3.8	1.0	0.6

Wave Loads on Vessel

In sheltered waters where piers and wharves are usually constructed, wave forces are not typically significant and may be ig

Typical Mooring Lines for Vessels:



Transverse Wind Load would be resisted by the double bow lines moored to bollard 8 and the double stern lines moored to
Since the four lines are connected to two bollards, it is assumed that the load would be distributed evenly between the two
Longitudinal load would be primarily resisted by the lines moored to bollards 12 and 15, with some contribution from bollar

As a worst case, bollards would be loaded with ½F_{TW} and ½ (F_{TW} + F_{LC})

Vessel	Transverse Wind Force, F_{TW} (kN)	Transverse Load on Bollard (kN)	Longitudinal Wind Force, F_{LW} (kN)	Long Current Force, F_{LC} (kN)	Longitudinal Force on Bollard (kN)
Viking Octantis	920	**460**	213	14.2	**113.8**
Viking Polaris	920	**460**	213	14.2	**113.8**
Pearl Mist	263	**131**	61	3.5	**32.2**
Le Dumont D'Urville	504	**252**	117	6.4	**61.6**
Le Bellot	449	**224**	104	6.4	**55.3**
Hanseatic Inspiration	515	**257**	119	9.3	**64.3**
Island Queen	50	**25**	12	0.6	**6.1**

Based on the capacity of the bollards and loading in the transverse direction, the only ship that would be safe to dock is the Island Queen

Vr = 82.2 kN

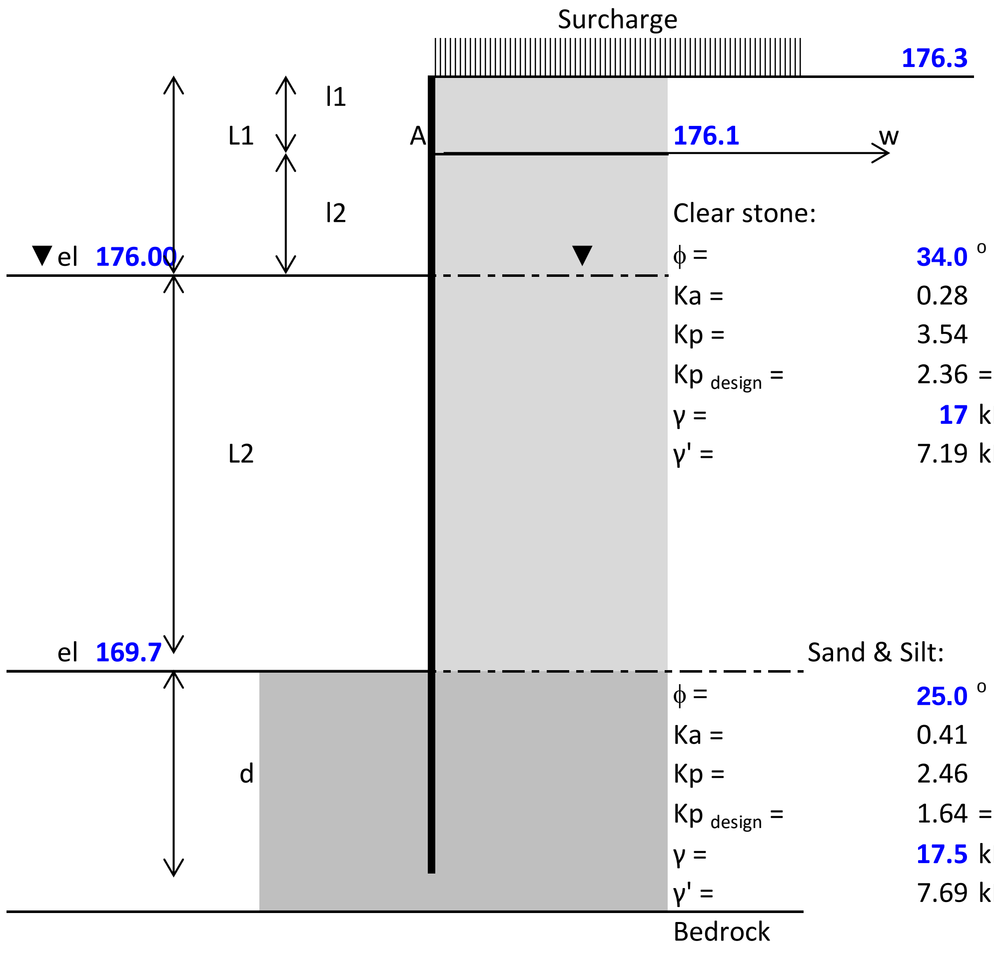
Appendix H

Steel Sheet Pile Analysis

PROJECT: Parry Sound Harbour
PN: 60719231
Date: January 2024

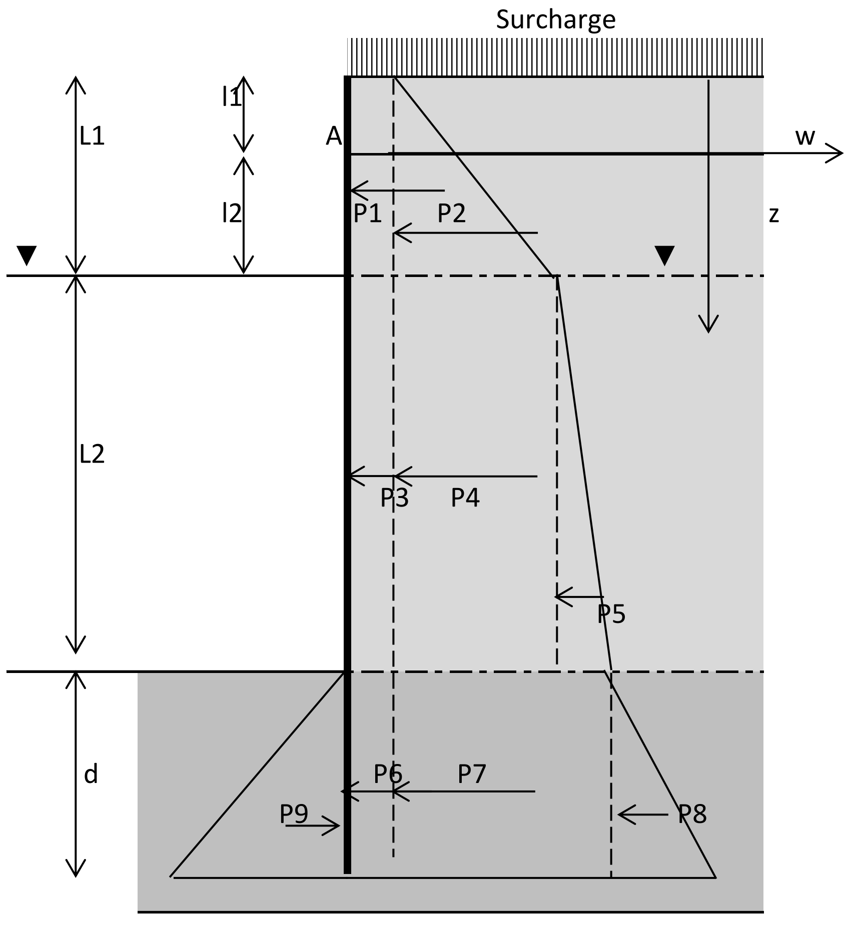
AECOM

Sheet Pile Wall - BH 11 6.30 Dredge Depth



Clear stone:
ϕ = 34.0 °
Ka = 0.28
Kp = 3.54
Kp $_{design}$ = 2.36 = Kp / FOS
γ = 17 kN/m3
γ' = 7.19 kN/m3

Sand & Silt:
ϕ = 25.0 °
Ka = 0.41
Kp = 2.46
Kp $_{design}$ = 1.64 = Kp / FOS
γ = 17.5 kN/m3
γ' = 7.69 kN/m3



Assumptions:
Wall Friction Angle δ = 0

Surcharge = 12 kPa
FOS = 1.5

L1 = 0.30 m
L2 = 6.30 m

l1 = 0.2 m
l2 = 0.10 m

Pressures

ρ1 =	Ka * Surcharge =	3.39	kN/m2
ρ2 =	Ka * γ * L1 =	1.44	kN/m2
ρ3 =	Ka * Surcharge =	3.39	kN/m2
ρ4 =	Ka * γ * L1 =	1.44	kN/m2
ρ5 =	Ka * γ' * L2 =	12.81	kN/m2
ρ6 =	Ka * Surcharge =	3.39	kN/m2
ρ7 =	Ka * γ * L1 + Ka * γ' * L2 =	14.25	kN/m2
ρ8 =	Ka * γ' * d =	14.63	kN/m2
ρ9 =	Kp * γ' * d =	59.22	kN/m2

Forces

P1 =	ρ1 * L1 =	1.02	kN/m
P2 =	½ ρ2 * L1 =	0.22	kN/m
P3 =	ρ3 * L2 =	21.37	kN/m
P4 =	ρ4 * L2 =	9.08	kN/m
P5 =	½ ρ5 * L2	40.34	kN/m
P6 =	ρ6 * d =	15.90	kN/m
P7 =	ρ7 * d =	66.80	kN/m
P8 =	½ ρ8 * d =	34.30	kN/m
P9 =	½ ρ9 * d =	138.81	kN/m

Moment Arm about A

z1 =	½ L1 - l1 =	-0.05	m
z2 =	⅔ L1 - l1 =	0.00	m
z3 =	½ L2 + l2 =	3.25	m
z4 =	½ L2 + l2 =	3.25	m
z5 =	⅔ L2 + l2 =	4.30	m
z6 =	L2 + l2 + ½d =	8.74	m
z7 =	L2 + l2 + ½d =	8.74	m
z8 =	L2 + l2 + ⅔d =	9.53	m
z9 =	L2 + l2 + ⅔d =	9.53	m

Moment

M1 =	-0.05
M2 =	0.00
M3 =	69.46
M4 =	29.52
M5 =	173.46
M6 =	139.07
M7 =	584.07
M8 =	326.70
M9 =	-1322.23

ΣM = 0.00

d = 4.69 m
w = 50.22 kN/m

Total Depth 11.29 m 165.01 m
37.03 ft

Point of Zero Shear
(Assume point of zero shear is at x between water level and dredge line)

6.24

ΣFx = 0.0000

P1 =	ρ1 * L1 =	1.02	kN/m
P2 =	½ ρ2 * L1 =	0.22	kN/m
P3 =	ρ3 * x =	14.47	kN/m
P4 =	ρ4 * x =	6.15	kN/m
P5 =	½ (Ka * γ' * x) * x =	28.37	kN/m
w =	-w	-50.22	kN/m

Force acting at:

z1 =	½ L1 + x =	4.41	m
z2 =	⅓ L1 + x =	4.36	m
z3 =	½ x =	2.13	m
z4 =	½ x =	2.13	m
z5 =	⅓ x =	1.42	m
z_T =	x + l2 =	4.36	m

Moment

M1 =	4.49
M2 =	0.94
M3 =	30.84
M4 =	13.11
M5 =	40.32
M_T =	-219.14

ΣM = -129.44 kNm / m

x = 4.26 m

Sheet Pile Check
Sheet Pile Section Modulus Required:

Mr = Φs S_r Fy
S_r = Mf / Φs Fy = FOS * M / 0.9 * 350
= 616 cm^3 / m

Existing SSP Section
S = 967.7 cm^3 / m Sr > S, ok
t = 7.6 mm

Waler and Tie Check
Tie Force w = 50.2 kN/m Span = 3.556 m
w_f = 75.3 kN/m
T_f = 295.4 kN *Adding force acting on bollard from Island Queen
Assuming simple span (ties spaced at 3.56m)
M_f = $w_f L^2 / 8$ = 119.1 kNm
V_f = $w_f L / 2$ = 133.9 kN

Waler - C250x30
Mr = 2 * 69.4 = 138.8 kNm Mr > Mf, ok
Vr = 2 * 435 = 870 kN Vr > Vf, ok

Tie Rod - #18 (2 1/4" diameter) Grade 75 Dywidag Tie Rod
T_r = 0.9 * 1423 = 1280.7 kN Tr > Tf, ok

PROJECT: Parry Sound Harbour
PN: 60719231
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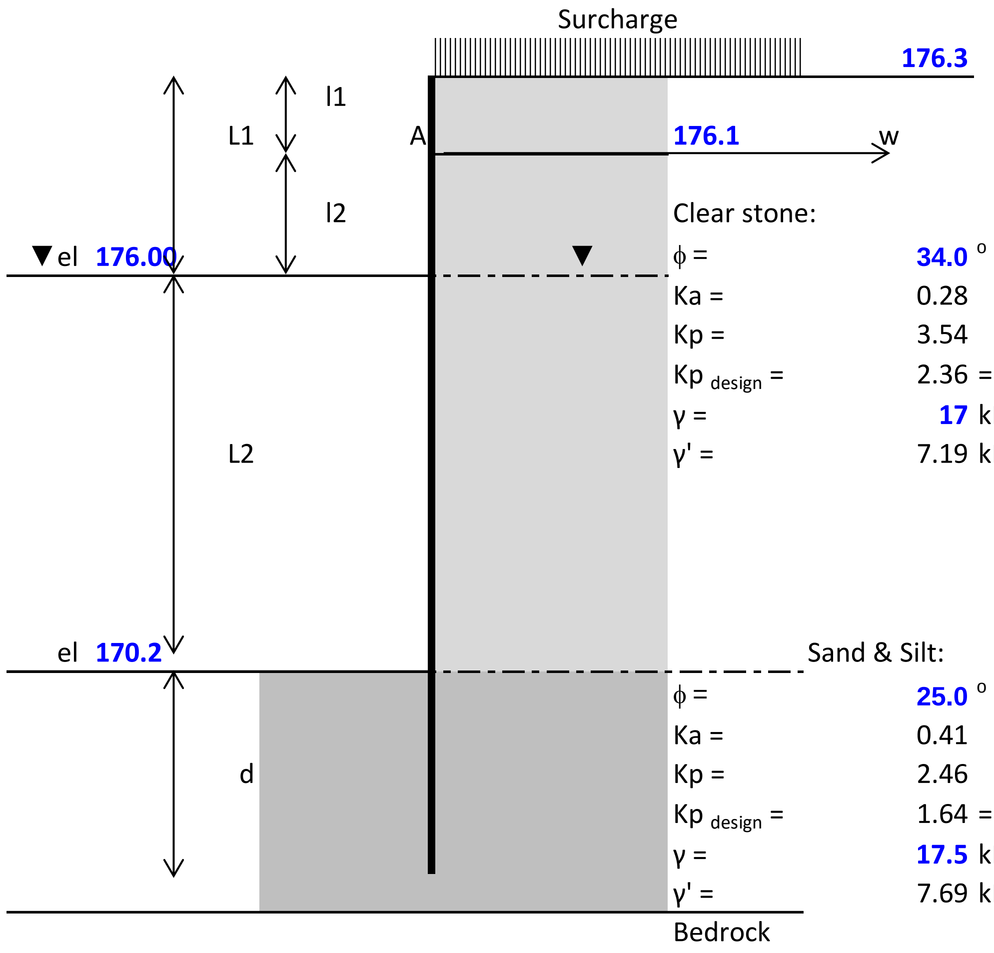
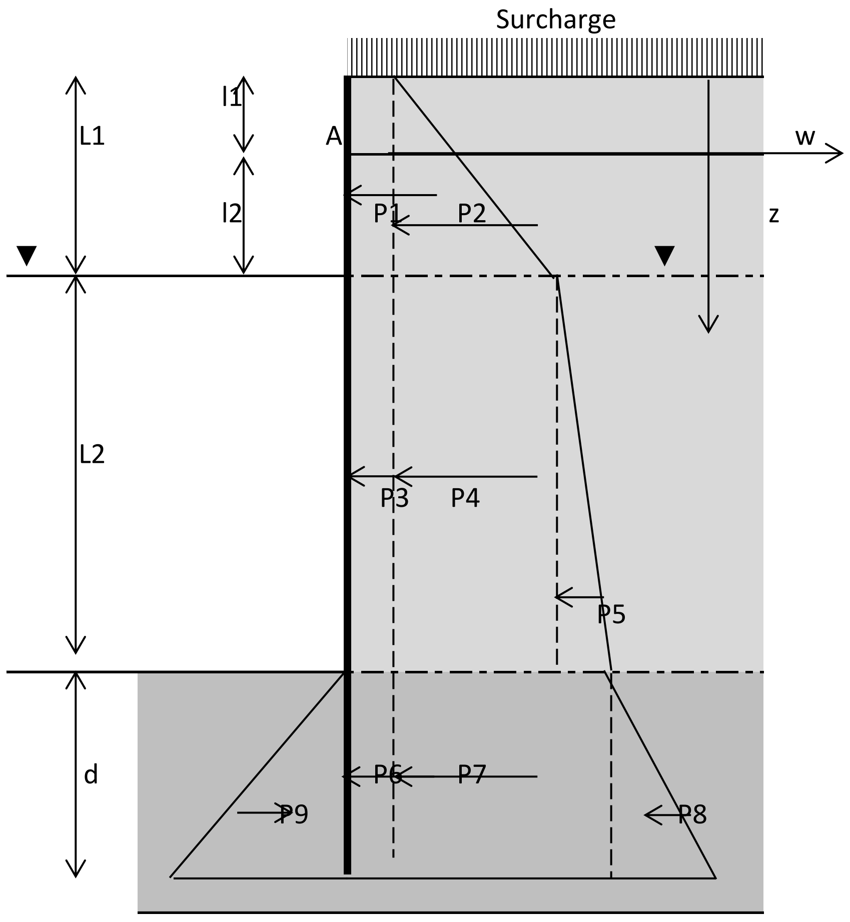
Sheet Pile Wall - BH 12 **5.80 Dredge Depth**



Clear stone:
ϕ = 34.0 °
Ka = 0.28
Kp = 3.54
Kp design = 2.36 = Kp / FOS
γ = 17 kN/m3
γ' = 7.19 kN/m3

Sand & Silt:
ϕ = 25.0 °
Ka = 0.41
Kp = 2.46
Kp design = 1.64 = Kp / FOS
γ = 17.5 kN/m3
γ' = 7.69 kN/m3



Assumptions:
Wall Friction Angle δ = 0

Surcharge = 12 kPa   L1 = 0.30 m   l1 = 0.2 m
FOS = 1.5   L2 = 5.80 m   l2 = 0.10 m

Pressures

ρ1 =	Ka * Surcharge =	3.39	kN/m2
ρ2 =	Ka * γ * L1 =	1.44	kN/m2
ρ3 =	Ka * Surcharge =	3.39	kN/m2
ρ4 =	Ka * γ * L1 =	1.44	kN/m2
ρ5 =	Ka * γ' * L2 =	11.79	kN/m2
ρ6 =	Ka * Surcharge =	3.39	kN/m2
ρ7 =	Ka * γ * L1 + Ka * γ' * L2 =	13.23	kN/m2
ρ8 =	Ka * γ' * d =	13.72	kN/m2
ρ9 =	Kp * γ' * d =	55.53	kN/m2

Forces

P1 =	ρ1 * L1 =	1.02	kN/m
P2 =	½ ρ2 * L1 =	0.22	kN/m
P3 =	ρ3 * L2 =	19.68	kN/m
P4 =	ρ4 * L2 =	8.36	kN/m
P5 =	½ ρ5 * L2	34.19	kN/m
P6 =	ρ6 * d =	14.91	kN/m
P7 =	ρ7 * d =	58.17	kN/m
P8 =	½ ρ8 * d =	30.16	kN/m
P9 =	½ ρ9 * d =	122.05	kN/m

Moment Arm about A

z1 =	½ L1 - l1 =	-0.05	m
z2 =	⅔ L1 - l1 =	0.00	m
z3 =	½ L2 + l2 =	3.00	m
z4 =	½ L2 + l2 =	3.00	m
z5 =	⅔ L2 + l2 =	3.97	m
z6 =	L2 + l2 + ½d =	8.10	m
z7 =	L2 + l2 + ½d =	8.10	m
z8 =	L2 + l2 + ⅔d =	8.83	m
z9 =	L2 + l2 + ⅔d =	8.83	m

Moment

M1 =	-0.05
M2 =	0.00
M3 =	59.03
M4 =	25.09
M5 =	135.62
M6 =	120.77
M7 =	471.03
M8 =	266.30
M9 =	-1077.79
ΣM =	**0.00**

d = **4.40** m
w = **44.65** kN/m

Total Depth 10.50 m 165.80 m
34.44 ft

Point of Zero Shear
(Assume point of zero shear is at x between water level and dredge line)

6.24

ΣFx = 0.0000

P1 =	ρ1 * L1 =	1.02	kN/m
P2 =	½ ρ2 * L1 =	0.22	kN/m
P3 =	ρ3 * x =	13.39	kN/m
P4 =	ρ4 * x =	5.69	kN/m
P5 =	½ (Ka * γ' * x) * x =	24.33	kN/m
w =	-w	-44.65	kN/m

Force acting at:

z1 =	½ L1 + x =	4.10	m
z2 =	⅓ L1 + x =	4.05	m
z3 =	½ x =	1.97	m
z4 =	½ x =	1.97	m
z5 =	⅓ x =	1.32	m
z_T =	x + l2 =	4.05	m

Moment

M1 =	4.17
M2 =	0.88
M3 =	26.44
M4 =	11.24
M5 =	32.02
M_T =	-180.75
ΣM =	**-106.01** kNm / m

x = **3.95** m

Sheet Pile Check
Sheet Pile Section Modulus Required:

$M_r = \Phi_s S_r F_y$

$S_r = M_f / \Phi_s F_y = FOS * M / 0.9 * 350$

= **505** cm³ / m

Existing SSP Section
S = 967.7 cm³ / m   Sr > S, ok
t = 7.6 mm

Walер and Tie Check

Tie Force   w = 44.6 kN/m   Span = 3.556 m
w_f = 67.0 kN/m
T_f = 238.2 kN

Assuming simple span (ties spaced at 3.56m)
M_f = $w_f L^2 / 8$ = 105.9 kNm
V_f = $w_f L / 2$ = 119.1 kN

Waler - C250x30
Mr = 2 * 69.4 = 138.8 kNm   Mr > Mf, ok
Vr = 2 * 435 = 870 kN   Vr > Vf, ok

Tie Rod - #18 (2 1/4" diameter) Grade 75 Dywidag Tie Rod
T_r = 0.9 * 1423 = 1280.7 kN   Tr > Tf, ok

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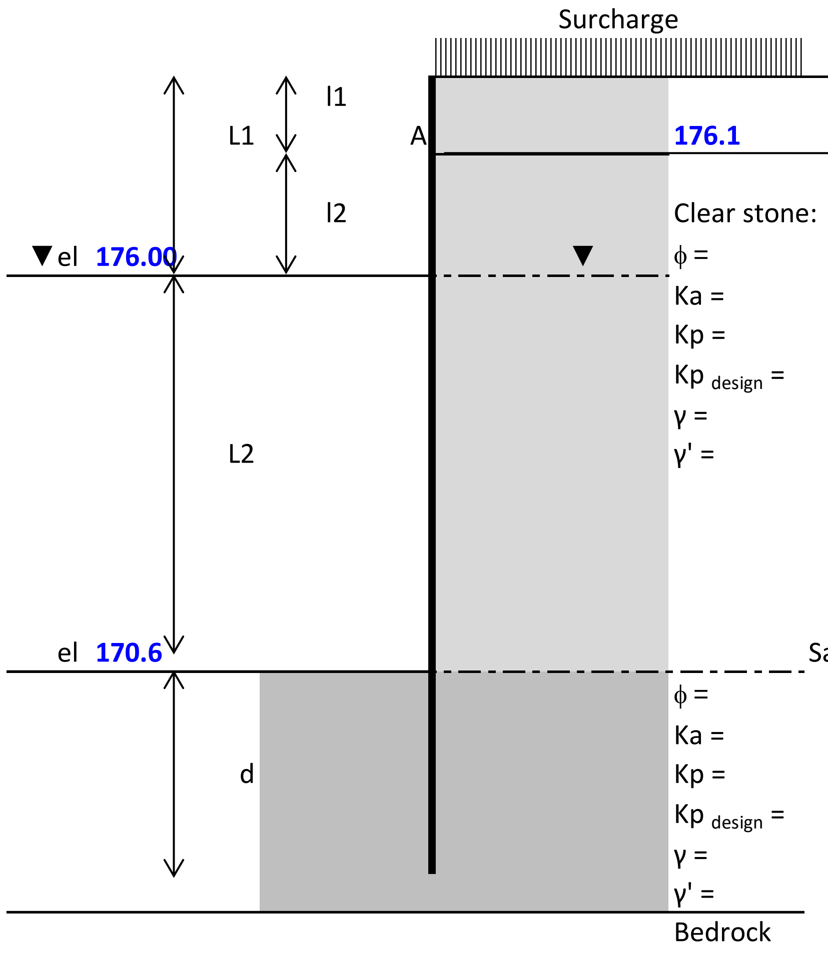
Sheet Pile Wall - BH 13

5.40 Dredge Depth

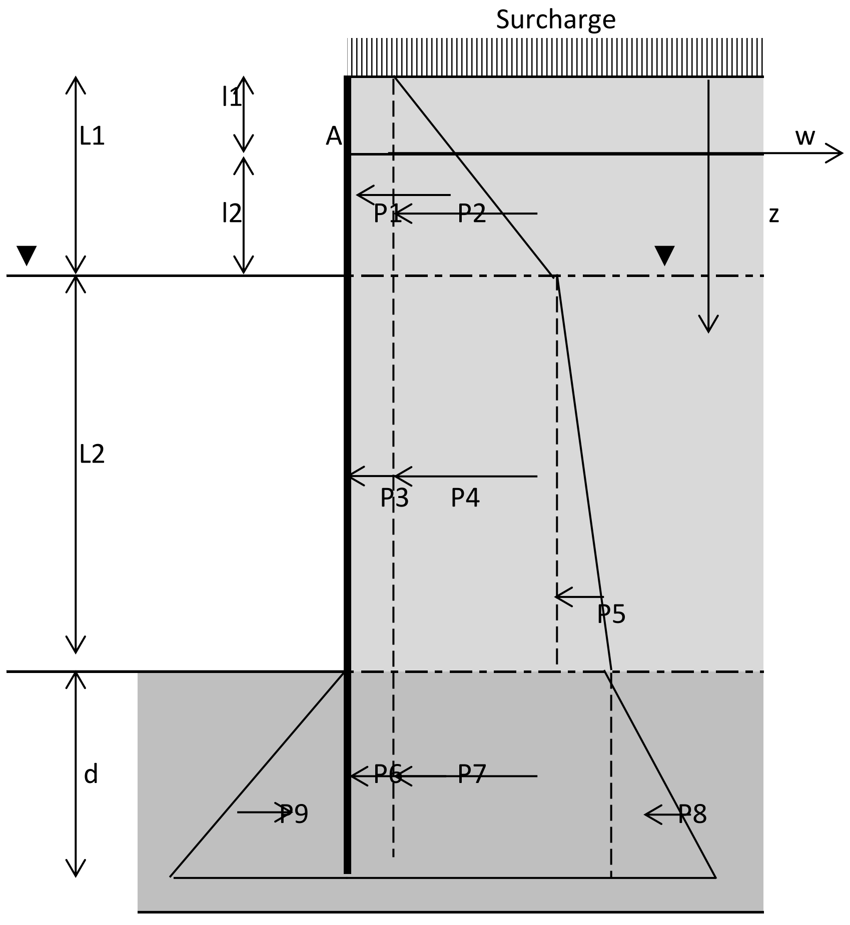


Clear stone:

ϕ =	**34.0** °
Ka =	0.28
Kp =	3.54
Kp design =	2.36 = Kp / FOS
γ =	**17** kN/m3
γ' =	7.19 kN/m3

Sand & Silt:

ϕ =	**25.0** °
Ka =	0.41
Kp =	2.46
Kp design =	1.64 = Kp / FOS
γ =	**17.5** kN/m3
γ' =	7.69 kN/m3



Assumptions:
Wall Friction Angle δ = 0

Surcharge =	**12** kPa	L1 =	0.30 m	l1 =	0.2 m
FOS =	**1.5**	L2 =	5.40 m	l2 =	0.10 m

Pressures

ρ1 =	Ka * Surcharge =	3.39 kN/m2
ρ2 =	Ka * γ * L1 =	1.44 kN/m2
ρ3 =	Ka * Surcharge =	3.39 kN/m2
ρ4 =	Ka * γ * L1 =	1.44 kN/m2
ρ5 =	Ka * γ' * L2 =	10.98 kN/m2
ρ6 =	Ka * Surcharge =	3.39 kN/m2
ρ7 =	Ka * γ * L1 + Ka * γ' * L2 =	12.42 kN/m2
ρ8 =	Ka * γ' * d =	12.99 kN/m2
ρ9 =	Kp * γ' * d =	52.57 kN/m2

Forces

P1 =	ρ1 * L1 =	1.02 kN/m
P2 =	½ ρ2 * L1 =	0.22 kN/m
P3 =	ρ3 * L2 =	18.32 kN/m
P4 =	ρ4 * L2 =	7.79 kN/m
P5 =	½ ρ5 * L2	29.64 kN/m
P6 =	ρ6 * d =	14.12 kN/m
P7 =	ρ7 * d =	51.68 kN/m
P8 =	½ ρ8 * d =	27.03 kN/m
P9 =	½ ρ9 * d =	109.40 kN/m

Moment Arm about A

z1 =	½ L1 - l1 =	-0.05 m
z2 =	⅔ L1 - l1 =	0.00 m
z3 =	½ L2 + l2 =	2.80 m
z4 =	½ L2 + l2 =	2.80 m
z5 =	⅔ L2 + l2 =	3.70 m
z6 =	L2 + l2 + ½d =	7.58 m
z7 =	L2 + l2 + ½d =	7.58 m
z8 =	L2 + l2 + ⅔d =	8.27 m
z9 =	L2 + l2 + ⅔d =	8.27 m

Moment

M1 =	-0.05
M2 =	0.00
M3 =	51.30
M4 =	21.80
M5 =	109.66
M6 =	107.04
M7 =	391.82
M8 =	223.66
M9 =	-905.22

ΣM = 0.00

d = **4.16** m
w = **40.41** kN/m

Total Depth 9.86 m 166.44 m
32.36 ft

Point of Zero Shear

(Assume point of zero shear is at x between water level and dredge line)

6.24

ΣFx = 0.0000

P1 =	ρ1 * L1 =	1.02 kN/m
P2 =	½ ρ2 * L1 =	0.22 kN/m
P3 =	ρ3 * x =	12.54 kN/m
P4 =	ρ4 * x =	5.33 kN/m
P5 =	½ (Ka * γ' * x) * x =	21.31 kN/m
w =	-w	-40.41 kN/m

Force acting at:

z1 =	½ L1 + x =	3.85 m
z2 =	⅓ L1 + x =	3.80 m
z3 =	½ x =	1.85 m
z4 =	½ x =	1.85 m
z5 =	⅓ x =	1.23 m
z_T =	x + l2 =	3.80 m

Moment

M1 =	3.91
M2 =	0.82
M3 =	23.17
M4 =	9.85
M5 =	26.25
M_T =	-153.39

ΣM = **-89.39** kNm / m

x = **3.70** m

Sheet Pile Check

Sheet Pile Section Modulus Required:

$Mr = \Phi s\ S_r\ Fy$

$S_r = Mf / \Phi s\ \ Fy = FOS * M / 0.9 * 350$

= **426** cm³ / m

Existing SSP Section
S = 967.7 cm³ / m Sr > S, ok
t = 7.6 mm

Waler and Tie Check

Tie Force w = 40.4 kN/m Span = 3.556 m
w_f = 60.6 kN/m
T_f = 215.6 kN

Assuming simple span (ties spaced at 3.56m)
M_f = $w_f L^2 / 8$ = 95.8 kNm
V_f = $w_f L / 2$ = 107.8 kN

Waler - C250x30
Mr = 2 * 69.4 = 138.8 kNm Mr > Mf, ok
Vr = 2 * 435 = 870 kN Vr > Vf, ok

Tie Rod - #18 (2 1/4" diameter) Grade 75 Dywidag Tie Rod
T_r = 0.9 * 1423 = 1280.7 kN Tr > Tf, ok

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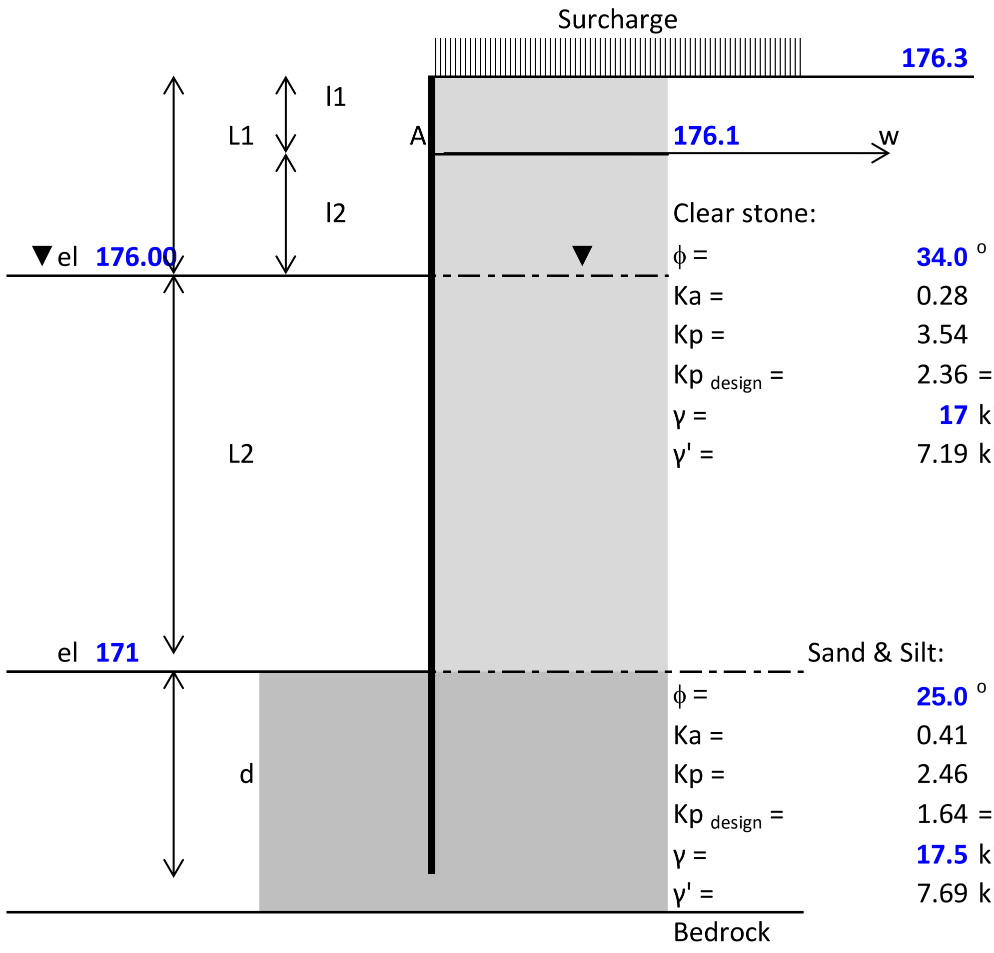
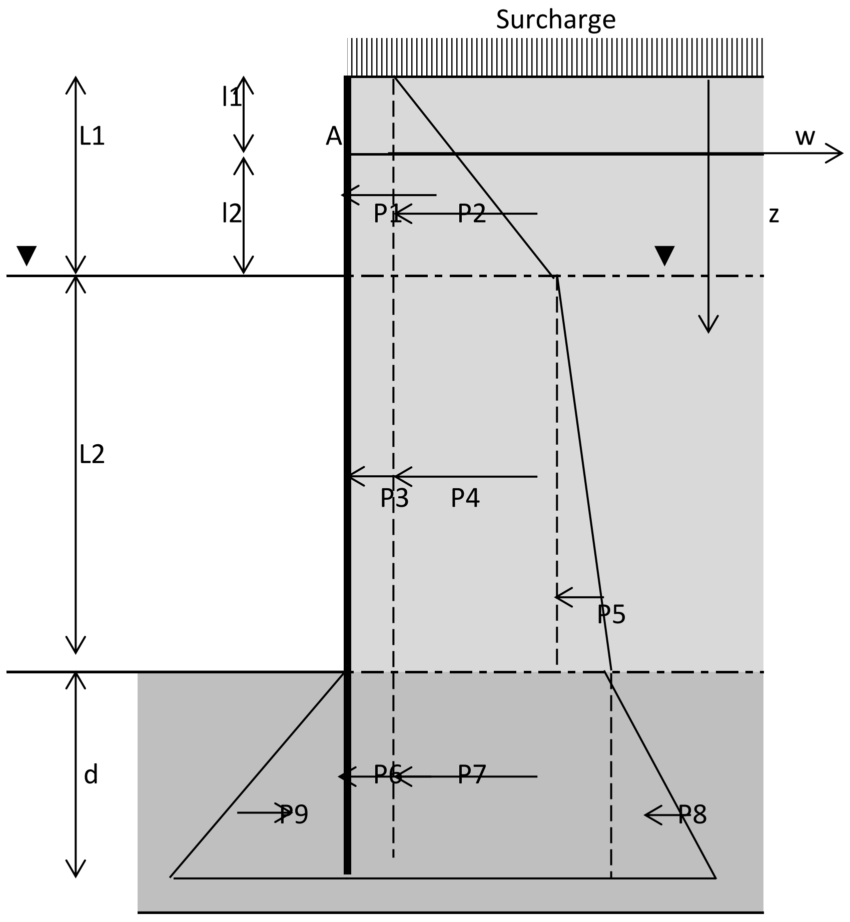
AECOM

Sheet Pile Wall - BH 2 **5.00 Dredge Depth**



Clear stone:
ϕ = 34.0 °
Ka = 0.28
Kp = 3.54
Kp design = 2.36 = Kp / FOS
γ = 17 kN/m3
γ' = 7.19 kN/m3

Sand & Silt:
ϕ = 25.0 °
Ka = 0.41
Kp = 2.46
Kp design = 1.64 = Kp / FOS
γ = 17.5 kN/m3
γ' = 7.69 kN/m3



Assumptions:
Wall Friction Angle δ = 0

Surcharge = 12 kPa
FOS = 1.5

L1 = 0.30 m
L2 = 5.00 m

l1 = 0.2 m
l2 = 0.10 m

Pressures

ρ1 =	Ka * Surcharge =	3.39 kN/m2
ρ2 =	Ka * γ * L1 =	1.44 kN/m2
ρ3 =	Ka * Surcharge =	3.39 kN/m2
ρ4 =	Ka * γ * L1 =	1.44 kN/m2
ρ5 =	Ka * γ' * L2 =	10.16 kN/m2
ρ6 =	Ka * Surcharge =	3.39 kN/m2
ρ7 =	Ka * γ * L1 + Ka * γ' * L2 =	11.61 kN/m2
ρ8 =	Ka * γ' * d =	12.26 kN/m2
ρ9 =	Kp * γ' * d =	49.61 kN/m2

Forces

P1 =	ρ1 * L1 =	1.02 kN/m
P2 =	½ ρ2 * L1 =	0.22 kN/m
P3 =	ρ3 * L2 =	16.96 kN/m
P4 =	ρ4 * L2 =	7.21 kN/m
P5 =	½ ρ5 * L2	25.41 kN/m
P6 =	ρ6 * d =	13.32 kN/m
P7 =	ρ7 * d =	45.58 kN/m
P8 =	½ ρ8 * d =	24.07 kN/m
P9 =	½ ρ9 * d =	97.42 kN/m

Moment Arm about A

z1 =	½ L1 - l1 =	-0.05 m
z2 =	⅔ L1 - l1 =	0.00 m
z3 =	½ L2 + l2 =	2.60 m
z4 =	½ L2 + l2 =	2.60 m
z5 =	⅔ L2 + l2 =	3.43 m
z6 =	L2 + l2 + ½d =	7.06 m
z7 =	L2 + l2 + ½d =	7.06 m
z8 =	L2 + l2 + ⅔d =	7.72 m
z9 =	L2 + l2 + ⅔d =	7.72 m

Moment

M1 =	-0.05
M2 =	0.00
M3 =	44.10
M4 =	18.74
M5 =	87.24
M6 =	94.12
M7 =	321.95
M8 =	185.78
M9 =	-751.88
ΣM =	0.00

d = 3.93 m
w = 36.37 kN/m

Total Depth 9.23 m 167.07 m
30.27 ft

Point of Zero Shear 6.24
(Assume point of zero shear is at x between water level and dredge line)
ΣFx = 0.0000

P1 =	ρ1 * L1 =	1.02 kN/m
P2 =	½ ρ2 * L1 =	0.22 kN/m
P3 =	ρ3 * x =	11.68 kN/m
P4 =	ρ4 * x =	4.96 kN/m
P5 =	½ (Ka * γ' * x) * x =	18.49 kN/m
w =	-w	-36.37 kN/m

Force acting at:

z1 =	½ L1 + x =	3.59 m
z2 =	⅓ L1 + x =	3.54 m
z3 =	½ x =	1.72 m
z4 =	½ x =	1.72 m
z5 =	⅓ x =	1.15 m
z_T =	x + l2 =	3.54 m

Moment

M1 =	3.66
M2 =	0.77
M3 =	20.10
M4 =	8.54
M5 =	21.22
M_T =	-128.85
ΣM =	-74.56 kNm / m

x = 3.44 m

Sheet Pile Check
Sheet Pile Section Modulus Required:
Mr = Φs S_r Fy
S_r = Mf / Φs Fy = FOS * M / 0.9 * 350
= 355 cm³ / m

Existing SSP Section
S = 967.7 cm³ / m Sr > S, ok
t = 7.6 mm

Waler and Tie Check
Tie Force w = 36.4 kN/m Span = 3.556 m
w_f = 54.6 kN/m
T_f = 194.0 kN
Assuming simple span (ties spaced at 3.56m)
M_f = w_f L² / 8 = 86.2 kNm
V_f = w_f L / 2 = 97.0 kN

Waler - C250x30
Mr = 2 * 69.4 = 138.8 kNm Mr > Mf, ok
Vr = 2 * 435 = 870 kN Vr > Vf, ok

Tie Rod - #18 (2 1/4" diameter) Grade 75 Dywidag Tie Rod
T_r = 0.9 * 1423 = 1280.7 kN Tr > Tf, ok

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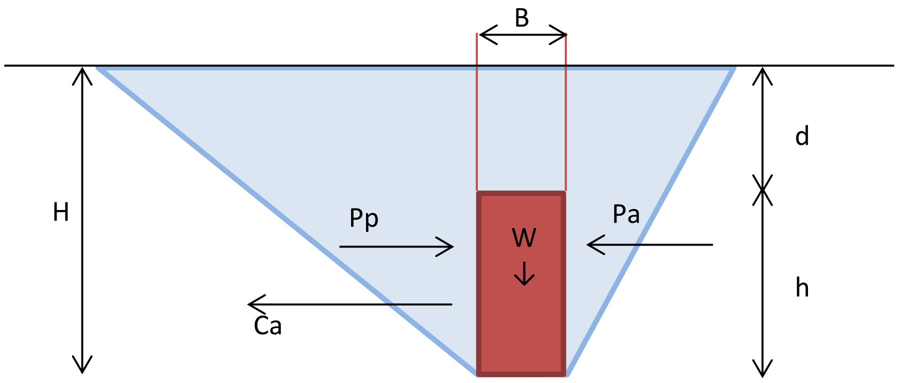
AECOM

Deadman Anchor Capacity

Concrete Deadman with Single Tie



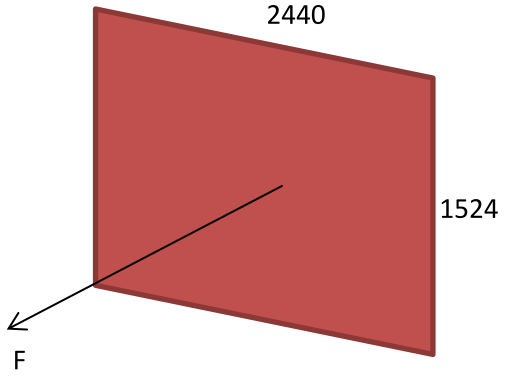
γ = **17** kN/m3
ϕ = **34.0** °
Ka = **0.28**
Kp = **3.54**
Kp/F = 2.36
Ko = **0.50**
B = 304.8 mm
L = 2440 mm
h = 1524 mm

1.4m below grade

d = 1400 mm
H = 2924 mm

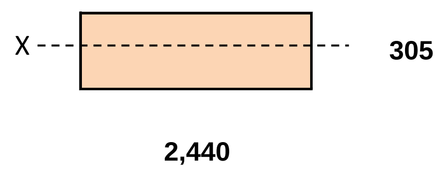
Pp = $\gamma H^2 K_p /2 =$ 171.5 kN/m
Pa = $\gamma H^2 K_a /2 =$ 20.3 kN/m

Ca = L (Pp - Pa) + 1/3 Ko γ (sqrt(Kp) + sqrt (Ka)) H^3 tan ϕ
= 467.51 kN
Ca > F - Sufficient Capacity



F = **295.3608659** kN
w = F / h = 193.81 kN / m
Vf = w h / 2 = **147.7** kN
Mf = $w h^2$ / 8 = **56.3** kNm

Concrete Deadman Anchor



Mr =	326	Pass
Vr =	535	Pass

Section Dimensions

	Per Meter Section	b =	2,440	mm	
	Thickness	t =	305	mm	
	Gross area of wall per running meter length	Ag =	743,712	mm^2	
	Concrete Cover	Cc =	60	mm	
	$b\ t^2 / 6$ =	Sx =	37,780.6	$x\ 10^3\ mm^3$	
	$b\ t^3 / 12$ =	Ix =	5,757.8	$x\ 10^6\ mm^3$	

Applied Factored Forces

	Maximum Factored Moment	M_f =	56	kNm	
	Factored Shear Force	V_F =	148	kN	
	Factored axial load normal to the cross-section	N_f =	0	kN	

Material Properties

	Concrete Strength	f'c =	20.0	MPa	
	Reinforcing Yield Strength	F_Y =	300	MPa	
A23.3-04 [8.4.2]	Concrete Resistance Factor	ϕc=	0.65		
A23.3-04 [8.4.3]	Reinforcing Steel Resistance Factor	ϕs=	0.85		
A23.3-04 [10.1.7]	Ratio of depth in Rect. Compr. Block	β_l =	0.92		= 0.97 - 0.0025 f'c > 0.67
A23.3-04 [10.1.7]	Ratio of Avg Stress in Compr. Block	α_l =	0.82		= 0.85 - 0.0015 f'c > 0.67
A23.3-04 [8.6.5]	Modification Factor for Normal Concrete Density	λ =	1.0		= 1.0, 0.85, 0.75
	max aggregate size	A_d =	20.0	mm	= Diameter
	Density of Concrete	γ_c =	2400.0	kg/m^3	
A23.3-04 [8.6.2]	Concrete Modulus of Elasticity	Ec =	23,086	MPa	= $(3300f'_c{}^{1/2}+6900) \times (\gamma_c / 2300)^{1.5}$
A23.3-04 [8.5.4.1]	Reinforcing Steel Modulus of Elasticity	Es =	200,000	MPa	
		n =	8.66		= Es / Ec
		ε_y =	0.002		= fy / Es

Reinforcing Steel Details

Tension Reinforcing

	Main flexural reinforcing steel	Rebar =	#7		
	Spacing of Steel	S =	150	mm	
	Rebar diameter	b_d =	22.2	mm	
	Rebar area	b_a =	387.0	mm^2	
	Number of bars per running meter length	N_b =	16.267		= b / S
	Area of tension face steel	As =	6295.2	mm^2	= N_b x b_a
A23.3-04 [7.8.1]	Minimum Area of Reinforcing	As,min =	1487.424	mm^2	= 0.002 Ag
A23.3-04 [8.6.4]	Modulus of rupture of concrete	f_r =	2.68		= $0.6\ \lambda\ f'_c{}^{1/2}$
A23.3-04 [10.5.1.3]		**$M_R > 1.33\ M_F$?**	**OKAY**		If "OKAY" then [10.5.1.1] may be disregarded
A23.3-04 [10.5.1.1]	Cracking moment	M_{CR} =	101.4	kNm	= fr x S_x x 10^{-6}
		1.2 x M_{CR} =	121.7	kNm	
		$M_r \geq 1.2M_{CR}$?	**OKAY**		

Compression Reinforcing

Main flexural reinforcing steel	Rebar =			
Spacing of Steel	S =	300	mm	
Rebar diameter	b_d =	#N/A	mm	
Rebar area	b_a =	#N/A	mm^2	
Number of bars per running meter length	N_b =	8.133		= b / S
Area of compression steel	As ' =	#N/A	mm^2	= N_b x b_a

Concrete Deadman Anchor

Shear Ties

	Stirrup	Stirrup =	-		
	Stirrup bar diameter	S_{bd} =	0.00	mm	

Reinforcing Location

	Depth of tension steel	d =	**233.7**	mm	= t - Cc - S_{bd} - b_d / 2
	Depth of Compression steel	d ' =	**#N/A**	mm	= Cc + S_{bd} ' + b_d ' / 2
A23.3-04 [2.3]	Effective shear depth	d_v =	**219.5**	mm	= max (0.9d, 0.72t)

Shear Check

Find Vc

A23.3-04 [11.3.6.4]		*Mf ' =*	*32.41*	kN	=V_f x dv
			Ok Mf > Mf '		
	Specified nominal size of coarse aggregate	a_g =	**20**	mm	= assume
A23.3-04 [11.3.6.3]	Crack spacing parameter	S_z =	219.5		= dv
A23.3-04 [11.3.6.3]	Equivalent value of Sz	S_{ze} =	**219**	mm	=35sz / (15 + a_g) ≥ 0.85sz
		N_f =	**0**	kN	
A23.3-04 [11.3.6.4]		ε_x =	0.0002		= (M_f / d_v + V_f ± 0.5 N_f) / 2 E_s A_s ≤ .003
A23.3-04 [11.3.6.4]		β =	0.344		= [0.40 / (1+1500ε_x)]x[1300 / (1000+s_{ze})]
A23.3-04 [11.3.6.4]		θ =	30.1	degrees	= 29 + 7000 ε_x
A23.3-04 [11.3.4]	Shear force resistance capacity of concrete	**V_c =**	**535**	**kN**	= $\beta \phi_c \lambda f_c'^{1/2} b_v d_v$

Find Vs

	Area of tie	At =	0.0	mm2	
	Tie Spacing longitudinally	Ts_L =	**0**	mm	
	Tie Spacing transverse	Ts_T =	**0**	mm	
	Number of bars per unit width	Nb_t =	***0.00***		= b / Ts_T
	Total tie area per section	Av =	***0.0***	mm2	= Nb_t x At
A23.3-04 [eq 11-7]	Stirrup contribution to shear	**Vs =**	**0**	**kN**	= Φs Av Fy dv cot Θ / Ts_L

Total Shear Capacity

A23.3-04 [11.3.5.1]		√f'c =	4.47		> 8 Mpa ***Pass***
A23.3-04 [11.3.3]	Maximum shear force resistance capacity of wall	V_{max} =	1740	kN	= 0.25 ϕ_c f'c b dv
		Vr =	**535**	**kN**	= Vc + Vs
		Vf =	**148**	**kN**	

Moment Capacity (Flexural Method)

Cc = $\alpha_1 \phi_c f'_c$ a b
Ts = $\phi_s A_s F_Y$
Ts = Cc

a =	61.7	mm	= $\phi_s A_s F_Y / (\alpha_1 \phi_c f'_c b)$
M_r =	**325.6**	**kNm**	= $\phi_s A_s F_Y$ (d - a / 2)
M_f =	**56**	**kNm**	
Moment Capacity =	**579%**		

Check rebar Yield

$$\frac{\varepsilon_s}{0.0035} = \frac{d-c}{c}$$

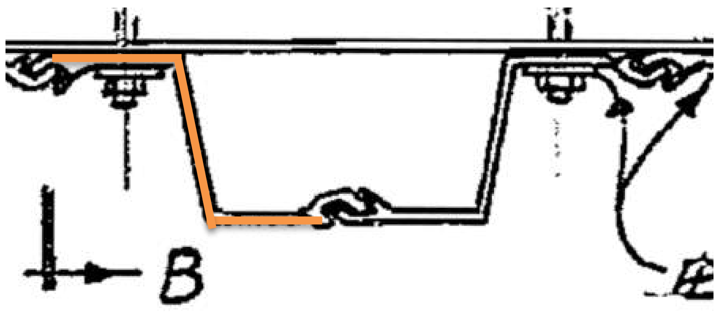
c =	67.1		= a / β1
ε_s =	0.009	***> 0.002 - Ok***	

Section Properties (SSP) :

SSP Parry Sound

	b (mm)	d (mm)	t (mm)	w (mm)	A (mm^2)	Y (mm)	A Y (mm^3)	AY^2 (mm^4)	$I_{x\text{-}x}$ (mm^4)	$I_{y\text{-}y}$ (mm^4)
Bottom	**203.2**	**-**	**7.6**		1,544.3	3.8	5.87E+03	2.23E+04	7.43E+03	5.31E+06
mid		**259.0**		**7.6**	1,968.4	110.5	2.18E+05	2.40E+07	1.10E+07	9.47E+03
Top	**203.2**		**7.6**		1,544.3	217.2	3.35E+05	7.29E+07	0.00E+00	0.00E+00
					5,057.0		5.59E+05	9.69E+07	1.10E+07	

Y_{bottom}	Y_{bot} =	110.5	mm	$= \sum AY / \sum A$
Moment of Inertia about x	Ix =	**46,174,780**	mm^4	$= \sum AY^2 + \sum I_o - Y_{bot}^2 \sum A$
Section Moduli at Bottom	Sx_{bot} =	**417,871**	mm^3	$= I_x / Y_{bot}$
	S =	**967.7**	cm^3 / m	



Appendix I

Cost Estimates

PARRY SOUND HARBOUR
Appendix I
PRELIMINARY COST ESTIMATE (2024 dollars)

General

Item #	Description	Unit	Estimated Quantity	Unit Price ($)	Total Price ($)
1	Ladders	ea	7	$5,000	$35,000
2	Localized concrete deck repairs	m^3	20	$5,500	$110,000
	Subtotal (rounded to nearest thousand)				**$145,000**
	General Contractor Adjustments (insurance, bonding, overhead, profit) (15%)				$21,750
	Preliminary Estimating Contingency (20%)				$29,000
	TOTAL COST (rounded up to nearest ten thousand)				**$200,000**

Structure A (STA 0+000 to 0+013.7)

Item #	Description	Unit	Estimated Quantity	Unit Price ($)	Total Price ($)
1	Curb Rail	m	14	$500	$7,000
	Subtotal (rounded to nearest thousand)				**$7,000**
	General Contractor Adjustments (insurance, bonding, overhead, profit) (15%)				$1,050
	Preliminary Estimating Contingency (20%)				$1,400
	TOTAL COST (rounded up to nearest thousand)				**$10,000**

ENCAPSULATE STRUCTURE A WITH STEEL SHEET PILE

Item #	Description	Unit	Estimated Quantity	Unit Price ($)	Total Price ($)
1	Removal of Superstructure	LS	1	$10,000	$10,000
2	Steel Sheet Pile (includes supply of driving equipment)	m^2	100	$600	$60,000
3	Tie Rods	m	60	$200	$12,000
4	Walers	m	14	$400	$5,600
5	Pile Cap	m	14	$450	$6,300
6	Deadman Anchors	ea	6	$3,000	$18,000
7	Concrete Slab including Reinforcing Steel	m^3	20	$1,400	$28,000
8	Fill	200	940	$80	$75,200
	Subtotal (rounded to nearest thousand)				**$215,100**
	General Contractor Adjustments (insurance, bonding, overhead, profit) (15%)				$32,265
	Preliminary Estimating Contingency (20%)				$43,020
	TOTAL COST (rounded up to nearest ten thousand)				**$300,000**

Structure B (STA 0+013.7 to 0+048.3)

Item #	Description	Unit	Estimated Quantity	Unit Price ($)	Total Price ($)
1	Curb Rail	m	35	$500	$17,500
2	Concrete Repair of Cope Wall	LS	1	$10,000	$10,000
3	Dredge at Outfall	LS	1	$10,000	$10,000
	Subtotal (rounded to nearest thousand)				**$37,500**
	General Contractor Adjustments (insurance, bonding, overhead, profit) (15%)				$5,625
	Preliminary Estimating Contingency (20%)				$7,500
	TOTAL COST (rounded up to nearest thousand)				**$51,000**

ENCAPSULATE STRUCTURE B WITH STEEL SHEET PILE

Item #	Description	Unit	Estimated Quantity	Unit Price ($)	Total Price ($)
1	Removal of Superstructure	LS	1	$20,000	$20,000
2	Steel Sheet Pile (includes supply of driving equipment)	m^2	240	$600	$144,000
3	Tie Rods	m	150	$200	$30,000
4	Walers	m	35	$400	$14,000
5	Pile Cap	m	35	$450	$15,750
6	Deadman Anchors	ea	15	$3,000	$45,000
7	Concrete Slab including Reinforcing Steel	m^3	42	$1,400	$58,800
8	Fill	t	700	$80	$56,000
	Subtotal (rounded to nearest thousand)				**$383,550**
	General Contractor Adjustments (insurance, bonding, overhead, profit) (15%)				$57,533
	Preliminary Estimating Contingency (20%)				$76,710
	TOTAL COST (rounded up to nearest ten thousand)				**$520,000**

Structure C (STA 0+048.3 to 0+117.8)

Item #	Description	Unit	Estimated Quantity	Unit Price ($)	Total Price ($)
1	Removal of Superstructure	LS	1	$40,000	$40,000
2	Concrete Slab including Reinforcing Steel	m^3	180	$1,400	$252,000
3	Timber Fenders	m	60	$400	$24,000
	Subtotal (rounded to nearest thousand)				**$316,000**
	General Contractor Adjustments (insurance, bonding, overhead, profit) (15%)				$47,400
	Preliminary Estimating Contingency (20%)				$63,200
	TOTAL COST (rounded up to nearest ten thousand)				**$430,000**

Structure D (STA 0+117.8 to 0+273.3)

Item #	Description	Unit	Estimated Quantity	Unit Price ($)	Total Price ($)
1	Concrete Removals at Bollard Locations	m^3	9	$7,000	$59,500
2	Concrete Repairs at Bollard Locations	m^3	9	$6,500	$55,250
3	Dowels	ea	170	$50	$8,500
4	Reinforcing steel	t	0.6	$10,000	$6,000
5	Replace two Railing Guards	ea	2	$1,500	$3,000
6	Handrails (two handrails at each of 10 stairs)	m	30	$2,000	$60,000
7	Fender Allowance	ea	40	$5,000	$200,000
	Subtotal (rounded to nearest thousand)				**$392,250**
	General Contractor Adjustments (insurance, bonding, overhead, profit) (15%)				$58,838
	Preliminary Estimating Contingency (20%)				$78,450
	TOTAL COST (rounded up to nearest ten thousand)				**$530,000**

ENCAPSULATE STRUCTURE D WITH STEEL SHEET PILE

Item #	Description	Unit	Estimated Quantity	Unit Price ($)	Total Price ($)
1	Removal of Superstructure	LS	1	$125,000	$125,000
2	Steel Sheet Pile (includes supply of driving equipment)	m^2	4800	$600	$2,880,000
3	Tie Rods	m	650	$200	$130,000
4	Walers	m	320	$400	$128,000
5	Pile Cap	m	320	$450	$144,000
6	Concrete Slab including Reinforcing Steel	m^3	280	$1,400	$392,000
7	Fill	t	14000	$80	$1,120,000
	Subtotal (rounded to nearest thousand)				**$4,919,000**
	General Contractor Adjustments (insurance, bonding, overhead, profit) (15%)				$737,850
	Preliminary Estimating Contingency (20%)				$983,800
	TOTAL COST (rounded up to nearest hundred thousand)				**$6,700,000**

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