CI C0067183

TUC Wharf Access Bridge Replacement

REDACTED

August 5, 2025

Nova Scotia
POWER
An Emera Company

REDACTED (CONFIDENTIAL INFORMATION REMOVED)

Nova Scotia POWER An Emera Company

NSEB APPROVAL SHEET

Project Title: TUC Wharf Access Bridge Replacement

CI Number: C0067183 **Date:** August 5, 2025

Expenditure Profile		
Year	Budget Amount	Project Estimate
2024		107,386
2025		2,319,283
Total		$2,426,669

Type of Filing

- [X] Capital Project Authorization
  - [X] Unforeseen and Unbudgeted (U&U)
  - [ ] Planned & Advanced (P&A)
  - [ ] Subsequent Approval Item
- [ ] Authorization to Overspend (ATO)
- [ ] Scope Change
- [ ] Final Cost (FIN)

COMMENTS

Submitted on behalf of NOVA SCOTIA POWER INCORPORATED	Approved on behalf of NOVA SCOTIA ENERGY BOARD
Authorized Signatory Dave Pickles Chief Operating Officer DATE August 5, 2025	DATE

CI Number: C0067183

Title: TUC Wharf Access Bridge Replacement

Start Date: 2024/06
In-Service Date: 2025/12
Final Cost Date: 2026/06
Function: Steam
Forecast Amount: $2,426,669

DESCRIPTION:

This project is for the replacement of the wharf access bridge at Tufts Cove, which was originally constructed in 1977. The existing bridge spans East to West from the shore to the unloading dock. The bridge consists of a structural steel frame divided into three equal spans, timber decking, and concrete foundation. The bridge is supported at the ends by the manifold-cell concrete structure and the shore-side cell structure, with two concrete piers in between. The main purpose of the bridge is to provide access for people, vehicles and materials to/from the wharf, and to support pipe, cables and other fixed assets that are used in the offloading and transfer of heavy fuel oil from tanker vessel to storage tank. Tufts Cove oil offloading and storage facilities are used to supply heavy fuel oil to Tufts Cove, Lingan, Trenton and Point Tupper.

Summary of Related CIs +/- 2 years:
Pursuant to Section 11.2 of the CEJC, related CIs for Steam projects include "Work completed on the same asset (turbine, boiler, etc.) and on the same unit (Lingan Unit #3, for example)."

- 2023 – C0060384 TUC Wharf Upgrades 2023 - $319,993

Depreciation Class: Steam Production Plant - Tufts Cove - Common

Estimated Life of the Asset: 30-50 years

Retirement Information:

- Categorization of Retirement: Accounting Policy 6420 - Retirement and Disposal of Capital Assets
- Percentage of Asset Pool: 0.40%

JUSTIFICATION:

Justification Criteria: Thermal

Sub Criteria: Equipment Replacement

Why do this project?

The wharf must be operable to accept deliveries of heavy fuel oil. In addition to fuel used at Tufts Cove, this facility accepts delivery of the heavy fuel oil that is used at Lingan, Trenton, and Point Tupper plants. The current condition of the wharf is no longer suitable for safe operation and requires replacement.

Why do this project now?

Visual Inspection and Non-Destructive Examination (NDE) using Ultrasonic Testing (UT) thickness measurements, which were conducted in 2024, showed deterioration in the condition of both the connections and the steel structural elements of the platform. As a result, the replacement of the joists, braces, and beams must be completed in 2025. Due to the deteriorated condition, the capacity of the wharf platform has been derated to only allow six (6) people on the wharf at any time. The wharf must be replaced prior to receiving the next delivery of heavy fuel oil, currently anticipated in the first quarter of 2026. A delay in the wharf replacement could significantly impact the ability to unload heavy fuel oil, which would in turn affect fuel supply across the thermal generation fleet, including Tuft's Cove, Lingan, Trenton, and Point Tupper. Ensuring the wharf is operational ahead of the next scheduled delivery is critical to maintaining reliable fuel logistics and uninterrupted generation capability.

Why do this project this way?

Two preliminary design alternatives were developed to replace the existing wharf access bridge structure, with both designs retaining the substructure, including the piles and pile caps (please refer to Partially Confidential Attachment 1). The alternatives were developed to ensure that the design would be able to accommodate the telehandler that NS Power uses during regular offloading operations.

The first preliminary design alternative considered is a steel structure with timber deck, similar to the current structure. The second alternative is precast concrete girders and deck. This option uses three precast concrete girders per span. During preliminary design, the structural consultant provided a cost estimate for each option. The estimated costs for each option consider the required rehabilitation and refurbishments that are necessary at the piers and pile caps, as well as the removal of the existing superstructure. Option 1 – steel framing with a timber deck – was selected as it was the more economical option and met all load requirements for regular offloading operations.

Contingency Statement:

Contingency for this project was determined using a combination of internal subject matter expert judgement (professional engineers and third-party consultants) and the non-binding contingency guidelines. Consistent with the maturity matrix, this project is being filed as a Class 3 estimate with a contingency of 15%. Risks intended to be covered by the contingency are related to the concrete scope, as the extent of the concrete refurbishment is unknown until the steel structure is removed.

Capital Project Detailed Estimate

Location: Steam
CI# : C0067183
Title: TUC Wharf Access Bridge Replacement
Execution Year: 2025

Description	Unit	Quantity	Unit Estimate	Total Estimate	Cost Support Reference	Completed Similar Projects (FP#'s)
Regular Labour						
Electrician	PD	20	$ 403	$ 8,066		
Engineering	PD	30	$ 457	$ 13,706		
Maintenance Trades	PD	80	$ 411	$ 32,847		
Utilityworker	PD	80	$ 286	$ 22,879		
			Sub-Tota	$ 77,499		
OT Labour						
Maintenance Trades	PD	40	$ 821	$ 32,847		
Utilityworker	PD	40	$ 572	$ 22,879		
			Sub-Tota	$ 55,727		
Term Labour						
Maintenance Trades	PD	0.87	$ 411	$ 356		
			Sub-Tota	$ 356		
OT Term Labour						
Maintenance Trades	PD	0.19	$ 821	$ 156		
			Sub-Tota	$ 156		
Materials						
Wharf Fabrication	lot	1	[redacted]		Attachment 2	
Dry Ice	lot	1	$ 12,500	$ 12,500		
Piping	lot	1	[redacted]			
Welding Consumables	lot	1	$ 2,500	$ 2,500		
			Sub-Tota	$ 697,356		
Contracts						
Wharf Install Mobilization	lot	1	[redacted]		Attachment 2	
Deck Removal	lot	1	[redacted]		Attachment 2	
Steel Substructure Removal	lot	1	[redacted]		Attachment 2	
Installation of New Wharf	lot	1	[redacted]		Attachment 2	
NDE Pipework	lot	1	$ 10,000	$ 10,000		
Crane	lot	1	$ 60,000	$ 60,000		
Diving Support	lot	1	$ 15,000	$ 15,000		
Concrete Refurbishment	lot	1	$ 100,000	$ 100,000		
Inspection Support / Pipe Removal & Disposal	lot	1	$ 64,055	$ 64,055		
			Sub-Tota	$ 978,399		
Consulting						
Wharf Replacement Design Drawings	lot	1	[redacted]		Attachment 3, Page 2 "Fees and Deliverables"	
Structure Inspection Report	lot	1	$ 33,000	$ 33,000		
Concrete Inspection	lot	1	$ 25,000	$ 25,000		
			Sub-Tota	[redacted]		
Meals						
Meals	lot	1	$ 39	$ 39		
			Sub-Tota	$ 39		
Other Goods & Services						
Contingency	%	15%	$ 1,940,332	$ 291,050		
			Sub-Tota	$ 291,050		
Interest Capitalized						
AFUDC	lot	1	$ 35,448	$ 35,448		
			Sub-Tota	$ 35,448		
Administrative Overhead						
Labour AO	lot	1	$ 38,757	$ 38,757		
Contractor AO	lot	1	$ 121,082	$ 121,082		
			Sub-Tota	$ 159,839		
			SUB-TOTAL (no AO, AFUDC)	$ 2,231,382		
			TOTAL (AO, AFUDC included)	$ 2,426,669		
Original Cost				$ 442,711		

Note 1: The labour figures noted above are an average of salaries across a variety of jobs within similar classifications including fringe, and are used solely for budgeting purposes.
Note 2: Small differences in totals are attributable to rounding.

Nova Scotia POWER An Emera Company

Instructions: Select option in Column B. Columns C through G will auto-populate.

Document No: XXX-XX-XXXX-XXXX

Project Cost Estimate Input Checklist and Maturity Matrix

Process Industries	Required fields:	Estimate Classification				
Project Name: TUC Wharf Access Bridge Replacement	**Started or Preliminary**	**Class 5**	**Class 4**	**Class 3**	**Class 2**	**Class 1**
Maturity Level of Project Definition Deliverables	**Defined or Complete**					
General Project Data						
A. Scope						
Non-Process Facilities (Infrastructure, Ports, Pipeline, Power Transmission)	Not Applicable (NA)					
Project Scope of Work Description	Defined (D)	P	P	D	D	D
Byproduct and Waste Disposal	Defined (D)	NR	P	D	D	D
Site Infrastructure (Access, Construction Power, Camp, etc.)	Defined (D)	NR	P	D	D	D
B. Capacity						
Plant Production/Facility (includes power facilities)	Not Applicable (NA)					
Electrical Power Requirements (when not the primary capacity driver)	Defined (D)	NR	P	D	D	D
C. Project Location						
Plant and Associated Facilities	Defined (D)	P	P	D	D	D
D. Requirements						
Codes and/or Standards	Defined (D)	NR	P	D	D	D
Communication Systems	Not Applicable (NA)	NR				
Fire Protection and Life Safety	Not Applicable (NA)	NR				
Environmental Monitoring	Defined (D)	NR	NR	P	P	D
E. Technology Selection						
Process Technology	Not Applicable (NA)					
F. Strategy						
Contracting/Sourcing	Defined (D)	NR	P	D	D	D
Escalation	Not Applicable (NA)	NR				
G. Planning						
Logistics Plan	Preliminary (P)	P	P	P		
Integrated Project Plan	Defined (D)	NR	P	D	D	D
Project Code of Accounts	Defined (D)	NR	P	D	D	D
Project Schedule	Defined (D)	NR	P	D	D	D
Regulatory Approval & Permitting	Preliminary (P)	NR	P			
Risk Register	Preliminary (P)	NR	P			
Stakeholder Consultation/Engagement/Management Plan	Defined (D)	NR	P	D	D	D
Work Breakdown Structure (WBS)	Defined (D)	NR	P	D	D	D
Start-up & Commissioning Plan	Not Applicable (NA)	NR				
H. Studies						
Environmental Impact/Sustainability Assessment	Not Applicable (NA)	NR				
Environment/Existing Conditions	Defined (D)	NR	P	D	D	D
Soils & Hydrology	Not Applicable (NA)	NR				
Technical Deliverables (Specifications and/or Drawings)						
Block Flow Diagrams	Not Applicable (NA)					
Equipment Data Sheets	Not Applicable (NA)	NR/S				
Equipment Lists: Electrical	Not Applicable (NA)	NR/S				
Equipment Lists: Process/Utility/Mechanical	Not Applicable (NA)	NR/S				
Heat & Material Balances	Not Applicable (NA)	NR				
Process Flow Diagrams (PFDs)	Not Applicable (NA)	NR				
Utility Flow Diagrams (UFDs)	Not Applicable (NA)	NR				
Design Specifications	Complete (C)	NR	S/P	C	C	C
Electrical One-Line Drawings	Not Applicable (NA)	NR				
General Equipment Arrangement Drawings	Not Applicable (NA)	NR				
Instrument List	Not Applicable (NA)	NR				
Piping & Instrumentation Diagrams	Not Applicable (NA)	NR				
Plot Plans/Facility Layouts	Complete (C)	NR	S/P	C	C	C
Construction Permits	Not Applicable (NA)	NR				
Civil/Site/Structural/Architectural Discipline Drawings	Complete (C)	NR	S/P	P	C	C
Demolition Plans and Drawings	Complete (C)	NR	S/P	P	C	C
Erosion Control Plan & Drawings	Not Applicable (NA)	NR				
Fire Protection & Life Safety Drawings & Details	Not Applicable (NA)	NR				
Electrical Schedules	Not Applicable (NA)	NR	NR/S			
Instrument & Control Schedules	Not Applicable (NA)	NR	NR/S			
Instrument Data Sheets	Not Applicable (NA)	NR	NR/S			
Piping Schedules	Complete (C)	NR	NR/S	P	P/C	C
Piping Discipline Drawings	Complete (C)	NR	NR/S	S/P	C	C
Spare Parts Listings	Not Applicable (NA)	NR	NR			
Electrical Discipline Drawings	Not Applicable (NA)	NR	NR			
Facility Emergency Communication Plan & Drawings	Not Applicable (NA)	NR	NR			
Information Systems/Telecommunication Drawings	Not Applicable (NA)	NR	NR			
Instrumentation/Control System Discipline Drawings	Not Applicable (NA)	NR	NR			
Mechanical Discipline Drawings	Complete (C)	NR	NR	S/P	P/C	C
	Total # Deliverables for this Project	3	20	22	21	21
	% Defined/Complete towards Class Estimate	100%	100%	92%	92%	88%

Comments:

This project is being filed as a AACE Class 3 estimate. The defined deliverables for this project indicate that 92% of Class 3 deliverables are completed. AACE recommends applying between 10% and 30% contingency to estimates in this class. A contingency value of 15% contingency was selected to account for specific risks asssociated with the project. The overall project scope is demolition, concrete pier refurbishment, and installation of the new wharf spans. However, the extent of the concrete repair work cannot be determined until the steel structure is removed. As a result, the concrete repair work is the main driver of contingency on this project.

Legend:

The maturity level is an approximation of the completion status of the deliverable. The completion is indicated by the following descriptors:

General Project Data:

- Not Required (NR): May not be required for all estimates of the specified class, but specific project estimates may require at least preliminary development.
- Preliminary (P): Project definition has begun and progressed to at least an intermediate level of completion. Review and approvals for its current status has occurred.
- Defined (D): Project definition is advanced, and reviews have been conducted. Development may be near completion with the exception of final approvals.

Technical Deliverables:

- Not Required (NR): Deliverable may not be required for all estimates of the specified class, but specific project estimates may require at least preliminary development.
- Started (S): Work on the deliverable has begun. Development is typically limited to sketches, rough outlines, or similar levels of early completion.
- Preliminary (P): Work on the deliverable is advanced. Interim, cross-functional reviews have usually been conducted. Development may be near completion except for final reviews and approvals.
- Complete (C): The deliverable has been reviewed and approved as appropriate.

Inclusions

For the purposes of this document, the term *process industries* is assumed to include firms involved with the manufacturing and production of chemicals, petrochemicals, and hydrocarbon processing. The common thread among these industries (for the purpose of estimate classification) is their reliance on process flow diagrams (PFDs), piping and instrument diagrams (P&IDs), and electrical one-line drawings as primary scope defining documents. These documents are key deliverables in determining the degree of project definition, and thus the extent and maturity of estimate input information. This RP applies to a variety of project delivery methods such as traditional design-bid-build (DBB), design-build (DB), construction management for fee (CM-fee), construction management at risk (CM-at risk), and private-public partnerships (PPP) contracting methods.

Estimates for process facilities center on mechanical and chemical process equipment, and they have significant amounts of piping, instrumentation, and process controls involved. As such, this recommended practice may apply to portions of other industries, such as pharmaceutical, utility, water treatment, metallurgical, converting, and similar industries.

Most plants also have significant electrical power equipment (e.g., transformers, switchgear, etc.) associated with them. As such, this RP also applies to electrical substation projects, either associated with the process plant, as part of power transmission or distribution infrastructure, or supporting the power needs of other facilities. This RP excludes power generating facilities and high-voltage transmission.

219 Waverley Road, Suite 200
Dartmouth, NS, Canada B2X 2C3
T: +1 902 405 4696

MEMO

Date:	2025-Feb-3	**File No.:**	241188
To:	Connor Lebans. P.Eng.	**From:**	Thomas McNutt, P.Eng.
	Mechanical Engineer		
Address:	Tufts Cove		

Subject: **Tufts Cove Wharf –Evaluation**

Harbourside Engineering Consultants (Harbourside) were retained by Nova Scotia Power Incorporated (NSPI) to complete an inspection and evaluation of the Tufts Cove Wharf in order to assess its capacity and determine if modifications or rehabilitation would be required to allow NSPI to use the wharf structure this fall. The existing superstructure consists of 3 – 18.2m spans that are supported on concrete pile caps at the pier locations, and the ends of the spans are supported on the shore-side and manifold cell structures. The structure is a steel frame with a timber deck. The wharf is used as an access bridge for equipment and personnel to access the manifold cell and loading platform as well as to support the 0.5 m diameter HFO line that runs between the shore-side cell and manifold cell. The wharf is oriented on an East-West axis. Information pertaining to the structure was obtained through drawings and previous inspection reports provided by NSPI.

Each of the three spans includes two 18.2 m long girders on the North and South sides of the wharf. The size of the girders was not specified in the provided drawings; however, previous inspections have identified them as W760x173 sections. Six W410x46 floor beams, span between the main girders and are 5.0 m in length. They are connected to the main girder webs by a welded end plate and bolted connection. The floor beams have a typical spacing of 3.7 m center to center. Two of the floor beams in a span are end beams which are located over top of either the pile caps or shore or manifold cell structures which support the ends of the girders. The timber deck is supported on four stringers which connect to the floor beams. The 3.7m long stringers were specified as W12x27 sections on the design drawings which is equivalent to the metric W310x39 section. The typical stringer spacing is as follows: the first stringer is offset 0.4 m from the South girder and the subsequent stringers are spaced 1 m apart leaving a 1.7 m space between the final stringer and the North girder. The lateral load resistance of the structure is provided by horizontal cross bracing using L127x127x11 sections. This information was not specified in the provided drawings but was identified in a previous inspection report.

The timber deck is 0.15 m x 0.3 m timber planks running parallel to the floor beams. The timber planks are sawn timber and are pressure treated No.1 and No.2 grade hemlock. The deck is supported by 0.15 m x 0.2 m nailers on top of the stringers. The deck also includes 0.2 m x 0.2 m timber guards that are located along the edges of the deck and are on top of 0.1 m x 0.2 m x 0.6 m timber chucks spaced 2.4 m apart on centre. The foundation is concrete with two concrete pile caps dividing the three spans. Each pile cap is on two 0.8 m diameter concrete filled steel pipe piles. The top portion of each pile is encased by a concrete jacket.

Background

NSPI previously engaged Stantec to complete an inspection and evaluation of the structure and Stantec produced a report stating that due to excessive deterioration of the stringers and floor beams the structure requires significant rehabilitation to allow it to be used. They specified a maximum of 1 person to be allowed on the wharf at a time.

NSPI looked into the option of rehabilitating or replacing the structure this year to address the deficiencies noted in the Stantec report, however, it was determined through discussions with various steel fabricators and contractors that completing the rehabilitation or replacement before the end of this calendar year would be unlikely.

Inspection

Harbourside completed an inspection of the wharf structure during two visits on June 24, 2024, and July 5, 2024. Access for the inspection was coordinated through NSPI, and Connor's Diving provided a boat to access and inspect the underside of the existing superstructure. The stringers were not surveyed during the inspection due to their poor condition noted in the Stantec inspection report. The floor beams were inspected, and UT measurements were taken at the worst-case locations for deterioration which were visually established. No UT measurements were taken at the girders, but they were noted to be in fair condition without any significant section loss.

Harbourside also completed a visual inspection of the substructure components that were visible above the waterline during the site visits. Cracking of the concrete pile caps was observed as well as some areas of spalling around the tops of the piles. The Site Visit Report summarizing the findings of the inspection, and can be found in Appendix A.

In addition to the inspection work completed by Harbourside NSPI engaged Connor's Diving to complete a sub-sea inspection of the wharf substructure. The results of this inspection are also provided in Appendix A.

Evaluation Methodology

Harbourside's first step in determining if NSPI is able to use the wharf for this year's offloading of fuel in the fall, was to confirm the capacity of the existing structure. To do this, an S-Frame model of the superstructure was developed, and the results confirmed by hand calculations. The section properties were modified to reflect the findings from the inspection of the members. The analysis of the structure was completed in accordance with CSA S6-19, Section 14 (Evaluation). Dead loads for the structure were calculated and are summarized in Table 1. Dead load factors of 1.05 and 1.1 were used for the steel and timber components respectively based on CSA S6-19 Section 14.

Table 1: Calculated Dead Loads

Component	Dead Load (kN)
Timber Deck	370
Stringers	92
Floor Beams	45
Girders	205
Braces	33
Total Dead Load	**745**

The following details the assumptions and information used in the evaluation:

- The yield strength of steel was assumed to be 300 MPa based on the time of design and construction of the wharf.
- The telehandler used by NSPI during their operations on the wharf is the model Genie® GTH™-5519 which weighs 4700 kg unloaded. The telehandler has a maximum capacity of 2495 kg.
- The weight of the telehandler is assumed to be distributed such that the front wheels support 75% of the telehandler's weight and the back wheels support the remaining 25% while carrying a load. The spacing the front and rear wheels of the telehandler is 2.29 m and the lateral spacing is 1.5 m. The wheels have a width of 0.3 m.
- Four personnel are required in addition to the telehandler at a mass of 100 kg/person.
- If the telehandler is not used for offloading, the number of personnel required to be on the wharf at a time is recommended to be 6.
- The weight of the HFO pipe when full is 330 kg/m as indicated by NSPI.
- The location of the HFO pipe is halfway between the North edge of the timber deck and the North girder.
- A section loss of 50% is applicable to the flange and web thicknesses of the stringers. This was based on the results of the Stantec inspection report.
- The worst-case locations for deterioration along the floor beams that were visually established during the inspection were conservatively assumed to be representative and thus applied to the entire length of the member.
- The primary girders are in good condition with only minimal local deterioration, so their cross section was not modified or reduced.

Load cases were developed based on expected loading during operation and anticipated repair alternatives. A live load factor of 1.7 was used in each case as per CSA S6-19 Section 3.5. Section 14 allows for a live load factor of 1.35 for this structure, however due to the uncertainty regarding the movement of personnel on the structure, it was decided to stick with a live load factor of 1.7 for the analysis.

Load Combination #1

The first load combination considered assumed that the telehandler would be used for offloading operations. The live loads considered were a telehandler, personnel, and the HFO pipe that runs along the structure near the North side. The telehandler was assumed to be carrying 80% of its maximum load which is 1996 kg. The location of the telehandler was skewed toward the South girder. 75% of the telehandler load was evenly distributed between the front wheels as point loads on a single floor beam and the remaining 25% evenly distributed between the rear wheels applied as point loads on the next floor beam. Based on the observed condition of the stringers during the Stantec inspection, it was assumed that the stringers had deteriorated beyond a condition that could safely support the telehandler load. As such, Harbourside proposed utilizing a jump bridge system that spanned between floor beams to allow the telehandler to travel along the bridge. Therefore, an additional dead load was considered in this load combination which accounted for the weight of the rail system. The additional weight of these beams was 279 kN, in addition to the deadload outlined in Table 1, giving an overall total dead load of 1024 kN.

Load Combination #2

The second load combination is the same as the first load combination but assumed that the telehandler was only carrying 50% of its maximum load which is 1248 kg.

Load Combination #3

The third load combination was based on a scenario where the telehandler would not be used. The NBCC gives a live load of 4.8 kPa for a footbridge. This was reduced for the analysis based on the decreased number of people anticipated to be on the bridge during offloading compared to a typical pedestrian footbridge. Therefore, a uniform personnel live load of 2.4 kPa for the personnel was used. The weight of the full HFO pipe was also applied.

Load Combination #4

The fourth load combination was similar to load case #3, however, the personnel live load was further reduced to 1.2 kPa to represent the control measures (maximum of 6 people on the bridge at a time) that will be in place to limit the number of personnel that will be permitted on the structure during the offload operations.

Analysis

For the load combinations not considering the telehandler (LC 3 and 4), the stringers were not bypassed and were assessed for the following condition: self-weight, live load, and the load from the timber deck for the assumed 50% section loss. Based on the connection detail of the nailer, the stringers were assumed to be fully laterally supported. Given the poor condition of some stringers, the stringers were also checked assuming that a single stringer in a given bay had to be skipped due to excessive deterioration and the increase in loading was shed to the adjacent stringers either side of the deteriorated stringer. Based upon the assumed section loss, properties of the stringers were calculated to meet class 3 requirements as per S6-19. Given that there were locations of significant section loss in certain stringers, a minimum web height was also determined to resist the maximum shear, and this was checked against the observed inspection conditions per the Stantec inspection report. Additionally, to ensure that load could be shed to the neighboring stringers either side of a deteriorated stringer, the timber deck was also checked to ensure it was capable to carrying the loading for the increased span between stringers. The assessment was performed in accordance with CSA O86-14.

The floor beams and girders were checked for the load combinations noted previously, using various unbraced lengths that were selected based on the geometry, and the current conditions of the structure. An additional buckling analysis of the girders was completed to evaluate if there is any additional lateral restraint offered to the girders from the existing floor beams. The girder to floor beam assembly was assessed similar to a pony truss system to calculate the amount of lateral stiffness the floor beam provided to the top flange of the girder. This was completed to check that the top flange of the girders have sufficient lateral torsional restraint in the current configuration without modifying the current structure or connections to reduce the current unbraced length.

Due to the large location of full section loss in the web of FB2, additional analysis was competed for this floor beam. At the location of full section loss, the moment was decoupled and applied to the top flange as a compression force. The compression flange was treated as a plate with fixed-fixed ends acting over the unbraced length of 500 mm which was the length of the section loss measured during the inspection. It was checked for lateral buckling and torsional buckling as a compression member. The reduced height of the web was also checked in shear.

Evaluation Results

Girders

If the girders are assumed to be laterally unsupported along their entire, they do not have sufficient capacity for most of the load combinations considered. The calculated moment resistance for a fully laterally unsupported length would 294 kN-m. However, based on the additional buckling analysis completed to determine the more accurate lateral support conditions, it was determined that the unbraced length of the girder could be reduced, resulting in a higher bending capacity. This capacity is approximately 1050 kN-m which is sufficient to resist each of the four load combinations considered. A summary of the maximum factored moment in the girder under each of the four load cases and the utilization ratio, or demand over capacity d/c ratio, for the girders in bending is presented in Table 2.

Table 2: Comparison of Moment Resistance and Moment Capacity of the Girders

Mr (kN-m)	LC #1		LC #2		LC #3		LC #4	
	M_f (kN-m)	d/c	M_f (kN-m)	d/c	M_f (kN-m)	d/c	M_f (kN-m)	d/c
1050	753	72%	720	69%	870	83%	664	63%

Floor Beams

The capacities of the floor beams were calculated for unbraced lengths of 2.5 m and 1.7 m as well as for completely laterally unbraced. Based on the connection details between the stringers and the floor beams and the condition of the stringers, it was determined that a conservative assumption is to use a maximum unbraced floor beam length of 2.5 m. These results are summarized in Table 3, which account for the measured section loss in each floor beam. This summary also includes the utilization in bending for each of the floor beams. Utilization ratios less than or equal to 85% are indicated in green, those between 85% and 100% are highlighted in yellow, and utilizations above 100% are in red. It should be noted that although there are 18 floor beams, the analysis does not include values for the floor beams at the ends of each span as they were inaccessible during the inspection. These floor beams are located above either the pile caps or the manifold or shore-side cell structures.

Table 3: Comparison of Moment Resistance and Moment Capacity of the Floor Beams for an Unbraced Length of 2.5 m

Member	Mr (kN-m)	LC #1		LC #2		LC #3		LC #4	
		M_f (kN-m)	d/c	M_f (kN-m)	d/c	M_f (kN-m)	d/c	M_f (kN-m)	U
FB2*	210	128	61%	120	57%	78	37%	55	26%
FB3	153	128	84%	120	78%	78	51%	55	36%
FB4	211	128	61%	120	57%	78	37%	55	26%
FB5	208	128	62%	120	58%	78	38%	55	26%
FB8	198	128	65%	120	61%	78	39%	55	28%
FB9	179	128	72%	120	67%	78	44%	55	31%
FB10	155	128	83%	120	77%	78	50%	55	35%
FB11	198	128	65%	120	61%	78	39%	55	28%
FB14	114	128	112%	120	105%	78	68%	55	48%
FB15	129	128	99%	120	93%	78	60%	55	43%
FB16	178	128	72%	120	67%	78	44%	55	31%
FB17	165	128	78%	120	73%	78	47%	55	33%

*Note: The moment resistance value in Table 3**Error! Reference source not found.** for FB2 corresponds to the UT measurement taken in the beam at a location other than where there is the most severe section loss.

To check FB2 at its location of full section loss, an additional local check was performed where the web had a developed a hole. The moment at the location was decoupled and the top flange was checked as an unbraced compression member for the length of the section loss. The location of full section loss in FB2 is shown in Figure 1. It is between the North girder and the Northernmost stringer on FB2.

Figure 1: Full Section Loss in Web of FB2

The additional analysis for FB2 found that under load combination 3, the moment at the location of full section loss was 73 kN-m and under load case 4 it was 52 kN-m. These moments correspond to compression forces of 186 kN and 133 kN in the top flange, respectively. Assuming that the flange can be treated as fixed-fixed, the member is adequate under both loading scenarios. However, a conservative approach would be to assume that it may not be a true fixed-fixed member, resulting in a larger k value in the analysis. A typical k value for a fixed-fixed member is 0.5 and for a pinned-pinned member is 1.0. So, the analysis was also completed for a k value of 0.7. In this scenario, the 2.4 kPa live load exceeds the capacity of FB2 at the location of complete section loss, but the 1.2 kPa live load is still a conservative estimation of the load being applied during operations and FB2 is able to achieve the strength required for this scenario. A summary of the results for the analysis of FB2 is in Table 4, including the compressive resistance for the flange plate, the moment at the location of section loss, the corresponding decoupled compression force in the top flange, and the utilization.

Table 4: Summary of Results for FB2

C_r (kN)	LC3			LC4		
	M_f (kN-m)	C_f (kN)	d/c	M_f (kN-m)	C_f (kN)	d/c
179	73	186	104%	52	133	74%

For load combination 1 and 2, which use the telehandler for offloading, floor beams 14 and 15 are unable to remain under the 85% utilization ratio for this unbraced length due to the observed deterioration. The telehandler and beams required to act as a jump bridge for the telehandler add considerable load to the structure. These floor beams would have to be repaired to support these additional loads. However, under the unbraced length of 2.5 m, all floor beams are able to achieve adequate capacity and remain under the 85% utilization for load combinations 3 and 4, which don't use the telehandler.

Stringers

The stringers were found to be severely deteriorated. Typical stringers were assumed to have a 50% reduced section which was in line with findings in the Stantec inspection report. The locations of the worst section loss on the stringers are shown in Figure 2.

(a) Stringer 3

(b) Stringer 38 and Stringer 39

(c) Stringer 51 (d) Stringer 58

Figure 2: Section Loss in Stringers

Reduced section properties were calculated based on the assumption of 50% section loss. The capacities in bending were calculated and compared with load combinations 3 and 4 as they would not be utilized under the assumptions of load combinations 1 and 2. A summary of the typical stringer bending capacity and applied factored moment under load cases 3 and 4 is as follows:

Table 5: Comparison of Moment Resistance and Moment Capacity of the Stringers

Mr (kN-m)	LC #3		LC #4	
	M_f (kN-m)	d/c	M_f (kN-m)	d/c
54	19.6	36%	12.8	24%

Typical stringers were found to have adequate capacity to meet load combinations 3 and 4 in both flexure and shear based on the estimated 50% reduced section. Where there are locations of excessive section loss, such as in stringer 58, shown in Figure 2d, the adjacent stringers were also checked for their ability to carry the additional load from the deck, in the event a single stringer had deteriorated to the point it did not have any reasonable structural capacity.

Recommendations

As repairing or replacing all the stringers is unlikely to be feasible given the time frame of the offloading operations, two temporary alternatives were developed to accommodate the offloading operations for this season on the bridge and were evaluated in the load combinations previously discussed.

Option #1

The first option that was considered was to install discretely positioned jump bridge beams along the top of the timber deck to be used by the telehandler as analyzed in load combinations 1 and 2. This would ensure that the telehandler load was distributed to the floor beams, and bypassed the deteriorated stringers. The rails would be made from three steel HP310x79 sections per rail welded together along their top flanges to ensure they were wide enough to fit a telehandler wheel and allow for maneuverability. Angle sections would be attached to the edges of the top flange to act as wheel guards.

The jump bridge beams would be skewed to South side of the bridge, to minimize the bending produced in the floor beams. This option would require some local removal of the decking at the beam support points along the floor beams for the connection of the rails to the existing structure. Additionally, there were three floor beams that were heavily deteriorated, and would either need to be locally reinforced, or attempted to be by passed by the temporary modification. Floor beams 14 and 15 are next to each other and had utilization ratios of 112% and 99% respectively for load combination 1. Additionally, floor beam 2 had an area of full section loss as previously discussed, and would also be required to be bypassed, or temporarily repaired.

This option requires the supply, fabrication and installation of jump bridge beams to bypass the existing stringers. The cost of this option is not insignificant and is estimated to be in the range of $[redacted]

Option #2

Given the tight timeframe until the offloading activities, Option #1 may not be feasible. Additionally, given the cost of purchasing, and fabricating the rail assemblies, as well as performing any necessary repairs to the structure. As such, an additional option was considered consisting of a winch and sheeve assembly which could aid in the pulling the hose assembly out to the manifold cell and back to the shore cell as required. This would remove the requirement of the using the telehandler to drag the hose assembly along the bridge, as well as the new temporary rail assemblies that would be required to spread the weight of the telehandler. NSPI also has informed Harbourside, that the hose assembly already has a hose cradle complete with wheels, that they anticipate can be pulled/maneuvered by personnel. The only requirement would be to install some form of blocking out at the manifold cell to ensure that winch/personnel have a straight line back to shore to pull out the hose assembly in the travel cradles.

Based on the buckling analysis of the girders and considering the connection details of the stringers to the floor beams, this option would require no additional repair work for the wharf prior to the unloading of the hose. The connection between the floor beams and girders as well as the stiffeners on the girders were found to provide adequate lateral stability to prevent buckling and achieve a strength of approximately 1050 kN-m in the girders. The floor beams can be conservatively assumed to be laterally braced at midspan and therefore even the most deteriorated floor beam is able meet the strength requirements for this loading scenario and remain below the 85% utilization limit for bending. The stringers and timber deck are also able to meet the necessary strength in this scenario.

The loads for this option are limited to the following:

- Self weight of the hose and transportation cradles,
- Full weight of the HFO pipe along the north edge of the bridge
- Maximum of 6 personnel on the bridge at a time, with a maximum of 4 in a single span (between piers)

Replacement vs Rehabilitation

The recommendations outlined above are related to the short-term use of the wharf for offloading operations during the fall of 2024 and should not be considered as a long-term solution. For future operations additional work will be required to the wharf in order to restore the original capacity or provide additional capacity to meet NSPI requirements. Harbourside understand that NSPI have contacted some steel fabricators to discuss the option of repairing the existing steel components in-situ, but both fabricators indicated that they would recommend replacement of the structure rather than in-situ repairs.

The in-situ repair of the wharf structure will require the following operations:

- Removal of timber decking to access the steel framework below.
- Installation of temporary bracing to provide additional stability during repair work.
- Repair the concrete piers where concrete has spalled, and the reinforcing has started to deteriorate.
- Install containment and hoarding for rehabilitation work.
- Removal and replacement of all stringers.
- Replacement or local repair of the floor beam members.
- Remove and replace coatings on all steel members that remain.
- Re-install the timber decking

The replacement of the wharf will still require repairs to the concrete pier structures, and replacement of the wharf superstructure, but simplifies the construction by removing all of the old components before starting installation of the new. It is anticipated that the contractor could potentially sequence the work so that they start with the replacement of the first span, closest to the shore, while working on the shore. They can then move equipment out onto the new span to access the second span, and so on, until all three spans have been replaced. It should be noted that the crane loads have not yet been considered in the preliminary design, and the detailed design will further consider the erection methodology and its impact on the design. Alternative erection methods include erecting the structure from a barge, or partially launching the structure one span at a time.

Repairs to the substructure will be developed during the detailed design phase, but are anticipated to include repairs to areas where concrete has spalled, replacement or installation of reinforcing bars where there has been significant deterioration and epoxy crack injection to extend the life of the existing structure. Additionally, the potential installation of cathodic protection to slow the corrosion of the pipe piles will be explored during detailed design.

Replacing the wharf superstructure also provides NSPI with the opportunity to more easily access the shoreline below the wharf, which is being rehabilitated with a rock revetment as the existing steel sheet pile cells have severely deteriorated. Accessing the area below the wharf superstructure without removing the steel framing members will be very challenging, and it will be difficult for the contractor to place the armour stone in the correct locations. The design of the new wharf superstructure can also take into account the sea level rise considerations and potentially raise the level of the wharf to meet current design code requirements. Rehabilitating the existing components does not allow for raising of the wharf.

Therefore, based on the above considerations, Harbourside recommends that NSPI replace the existing wharf superstructure and consider the effects of sea level rise, and the rock revetment project that is currently being designed by Harbourside.

Preliminary Design Alternatives

Two preliminary design alternatives were developed to replace the existing wharf structure once this offloading season has completed. The preliminary designs were completed assuming that the substructure, including the piles and pile caps, is to remain. The alternatives were developed based on a 2.4 kPa personnel live load, which is typical of pedestrian bridges, the full HFO pipe, and the design maintenance vehicle from Section 3 of CSA S6-19, which has a maximum axle load of 56 kN. This ensures that it would be able to accommodate the telehandler that NSPI uses during regular offloading operations.

The first preliminary design alternative is a steel structure with timber deck, similar to the current structure. For this, W920x201 girders would be used, with W410x54 floor beams, and W310x39 stringers. The girders and floor beams were increased in size over their current member sizes to meet serviceability requirements. The timber deck would be made from 304 mm x 152 mm timber members. The second alternative is precast concrete girders and deck. This option uses three precast concrete girders per span. The girders considered are NEBT1000, which have a depth of 1000 mm and 1200 mm top flange width. The concrete deck would be 225 mm thick and would cover the entire bridge.

Estimated construction costs for each preliminary design alternative were produced. The costs for each option consider the required rehabilitation and repairs that are necessary at the piers and pile caps as well as the removal of the existing superstructure. The cost estimates for each alternative are shown in Table 6. A [redacted] contingency has been applied to the costs to reflect this stage of the design. Unit rates were selected based on historical cost information. A breakdown of the two alternatives is in Appendix B.

Table 6: Cost Comparison of Design Alternatives

Alternative	Estimated Construction Cost
Steel framing and timber deck	$[redacted]
Precast concrete girders and deck	$[redacted]

Conclusion

Harbourside has completed an inspection and evaluation of the Tufts Cove Wharf structure to determine its current capacity and develop alternatives for the offloading operations. Based on the current condition of the wharf, no repair or rehabilitation is required to complete this year's offloading provided that it is completed using Option 2 with 6 people maximum as outlined above, without the use of a telehandler. It should also be noted that Stantec indicated that there were some broken handrail attachments along the wharf. Therefore, NSPI personnel on the wharf during offloading should remain at least the minimum required distance from the edge per NSPI for working at the edge of water or be wearing PFDs.

Following the offload operations during the fall of 2025, Harbourside recommends that NSPI replace the existing wharf superstructure and consider the effects of sea level rise, and the rock revetment project that is currently in the design phase. Based on the two preliminary design alternatives considered, Harbourside recommends replacing the current superstructure with a steel structure and timber deck, similar to what is currently in place, as it is more cost effective than the precast concrete girders and deck alternative.

Sincerely,

Tim Matthews, P. Eng.
Senior Structural Engineer
O 902 405 4696
E tmatthews@harboursideengineering.ca

Harbourside Engineering Consultants
A Division of the Harbourside Engineering Group

Thomas McNutt, P. Eng., P.E., M.A.Sc.
Senior Structural Engineer | Partner
O 902 405 4696 M 902 452 6378
E tmcnutt@harboursideengineering.ca

Harbourside Engineering Consultants
A Division of the Harbourside Engineering Group

APPENDIX A – Site Visit Record

HARBOURSIDE
Engineering Consultants

219 Waverley Road, Suite 200
Dartmouth, NS, Canada B2X 2C3
T: +1 902 405 4696

SITE VISIT RECORD

Date:	2024-Sep-19	**File No.:**	241188
To:	Connor Lebans. P.Eng. Mechanical Engineer	**From:**	Tim Matthews, P.Eng.
Address:	Tufts Cove		

Subject: **Tufts Cove Wharf – Record of Site Inspection**

Harbourside Engineering Consultants (Harbourside) were retained by Nova Scotia Power Incorporated (NSPI) to complete an inspection and evaluation of the Tufts Cove Wharf to assess the capacity of the structure and determine if modifications or rehabilitation may be required to allow NSPI to use the wharf structure this fall for their offloading operations. The existing superstructure consists of 3 – 18.2m spans that are supported on concrete pile caps at the pier locations, and the ends of the spans are supported on a shore-side cell and manifold cell and unloading dock. Each span consists of 2 longitudinal exterior girders, 6 equally spaced transverse floor beams, and 20 longitudinal stringers which frame into the floor beams. The deck consists longitudinal timber nailers attached to the top flange of the strings, and a timber deck which runs perpendicular to the span.

Structural drawings for the existing structure can be found in Appendix B.

Background

NSPI previously engaged Stantec to complete and inspection and evaluation of the structure and Stantec produced a report that determined the structure has deteriorated to the point where it should not be used.

NSPI investigated the option of rehabilitating or replacing the structure this year to address the deficiencies noted in the Stantec report, however it was determined through discussions with various steel fabricators and contractors that completing the rehabilitation or replacement before the end of this calendar year would be unlikely.

Inspection

Harbourside completed an inspection of the wharf structure on June 28 and July 5, 2024. Access for the inspections was coordinated through NSPI, and Connor's Diving provided a boat to access and inspect the underside of the existing superstructure. During the inspection, both the steel superstructure, and the exposed concrete substruction that was visible above the waterline at the time of the site visit were inspected.

The site inspection performed by Harbourside is meant to supplement the site inspection performed by Stantec, and to provide Harbourside with an overall visual of the conditions of the structure and allow Harbourside to focus on the areas that were considered critical for a potential temporary repair / modifications to allow NSPI to access the structure to perform their required operations, while a more permanent solution was being considered. The Inspection Report produced by Stantec can be found in Appendix C. Additionally, there was a dive inspection performed by Connor's Diving on the substructure below the water line. The inspection report can be found in Appendix D.

Inspection - General Information

The general overall conditions for the entire steel superstructure (girders, floor beams, stringers, and plan bracing), apart from the primary girders, showed signs of significant corrosion. As previous mentioned, the inspection primarily focused on the areas that Harbourside had deemed crucial for a potential repair / modification to the be considered viable. Based upon the findings in the Stantec Inspection report, the deck stringers, and plan bracings were excluded due to the conditions observed and the number of members that were identified as having full section loss. A general sketch of the structure along with the approximate inspection data location can be found in Appendix A. It should be noted that there was no inspection performed directly of the timber deck as access on to the structure was restricted at the time of inspection. However, the condition of timber deck was observed from below the structure and visibly appeared to be in good condition. Based upon the information provided to Harbourside (ref. Appendix B), it appears that the deck was replaced in 2002. The deck can be seen in general area photos taken from below the structure and can be found in Photos 2-4 below.

Access to the underside of the wharf structure was provided by Connor's Diving and coordinated by NSPI. Connor's Diving was on site for underwater inspections of the piers, and they were able to provide Harbourside with a ride on their dive boat to complete the inspection to the underside of the wharf structure. The inspection had to be coordinated with the tides to allow access to reach the underside of the wharf structure, but due to the time restraints, the focus of the inspection was on the primary girders and the floorbeams. The condition of the stringers was observed, but there was more information on these components in the Stantec report, which will be considered in the evaluation.

Photo 1: General photo of the configuration of the wharf structure showing the South primary girders and piers 1 and 2.

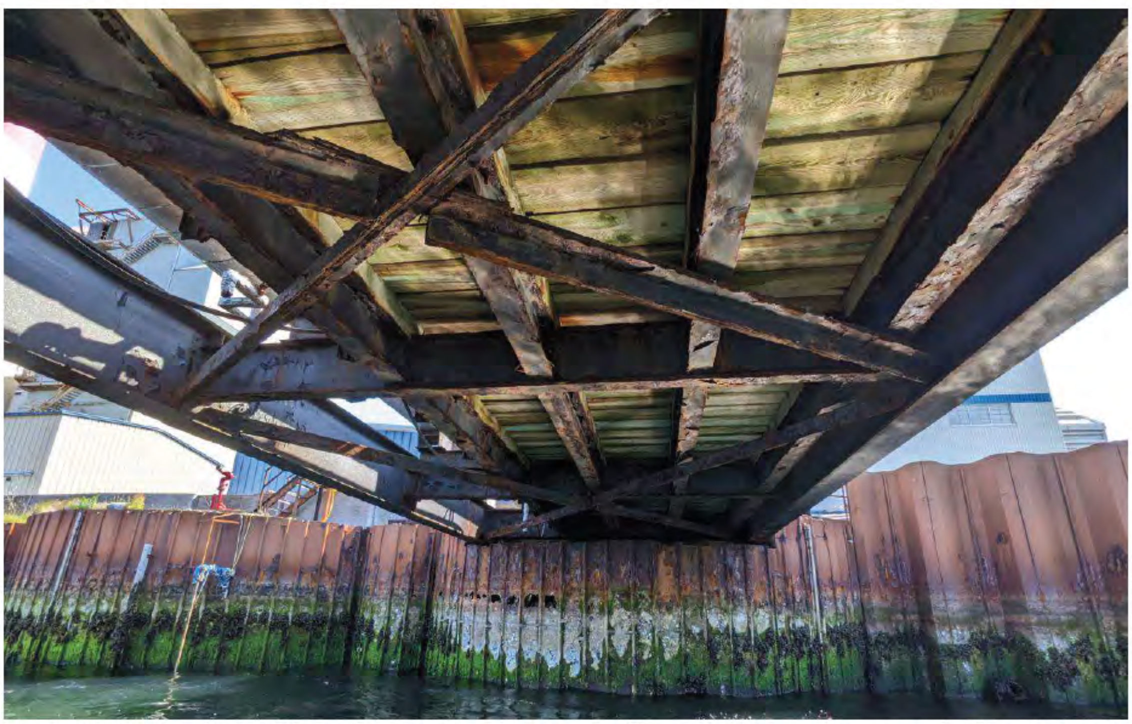


Photo 2: General condition of the superstructure in Span 1. As can be seen in the photo, the stringers and plan bracing have coatings breakdown in various areas over the length of the members. Corrosion is visible, with areas of delamination and flaking steel.

Photo 3: General condition of the superstructure in Span 2. As can be seen in the photo, the stringers and plan bracing have coatings breakdown in various areas over the length of the members. Corrosion is visible, with areas of delamination and flaking steel.

Photo 4: General conditions of the superstructure in Span 3. As can be seen in the photo, the stringers and plan bracing have coatings breakdown in various areas over the length of the members. Corrosion is visible, with areas of delamination and flaking steel.

Inspection – Primary Girders

The inspection of the primary girders for the structure showed that they were in good condition. The exterior face of the girder had coatings that were in good condition, with some localized breakdowns in areas along the webs and flanges. Generally, the coatings were in worse condition on the interior face of the webs. Minor corrosion is visible locally throughout the girders, with minimal thickness loss observed during the inspection.

Photo 5: General photo showing the exterior of Girder 1 as identified in the Inspection Layout Sketch found in Appendix A. The coatings appear to be in good condition, with some local breakdown around the corners of the flange, and some local blistering on the webs. Local corrosion observed in various locations along the girder.

Photo 6: General photo showing the exterior of Girder 2 as identified in the Inspection Layout Sketch found in Appendix A. The coatings appear to be in good condition, with some local breakdown around the corners of the flange, and some local blistering on the webs. Local corrosion observed in various locations along the girder.

Photo 7: General photo showing the exterior of Girder 3 as identified in the Inspection Layout Sketch found in Appendix A. The coatings appear to be in good condition, with some local breakdown around the corners of the flange, and some local flaking on the webs and stiffeners. Local corrosion observed in various locations along the girder.

Photo 8: General photo showing the exterior of Girder 6 as identified in the Inspection Layout Sketch found in Appendix A. The coatings appear to be in good condition, with some local breakdown around the corners of the flange, and some local breakdown on the webs, flanges, and stiffeners. Local corrosion observed in various locations along the girder.

Photo 9: General photo showing the exterior of Girder 5 as identified in the Inspection Layout Sketch found in Appendix A. The coatings appear to be in good condition, with some local breakdown around the corners of the flange, and some local breakdown on the webs, flanges, and stiffeners. Local corrosion observed in various locations along the girder.

Photo 10: General photo showing the exterior of Girder 4 as identified in the Inspection Layout Sketch found in Appendix A. The coatings appear to be in good condition, with some local breakdown around the corners of the flange, and some local breakdown on the webs, flanges, and stiffeners. Local corrosion observed in various locations along the girder.

Photo 11: General photo showing the coatings breakdown along the interior side of the girder web. Localized corrosion can be observed in areas where the coatings have broken down further.

Inspection – Floor Beams

The inspection of the floor beams for the structure showed that coatings are failing along the length of the members, there were visible signs of corrosion, localized material loss and delamination of the flanges, as well as localized area of full section loss in a localised area of the web for one of the floor beams. Due to the restricted access to the underside of the bridge due to the tidal action, limited inspection data was recorded to ensure the entire structure had some level of inspection. Some locations along floor beams, if they appeared better than other locations were not checked with the UT gauge.

	Thickness			
Location	Top Flg (mm)	Bottom Flg (mm)	Web (mm)	Notes
Floor Beam 1	-	-	-	Floor beam located above the Shore-side Cell, no measurements recorded
Floor Beam 2	-	-	6.8	Localized full Section Loss in the web (210mm high x 500mm long)
Floor Beam 3	-	5	6.8	
Floor Beam 4	-	5	5.9	
Floor Beam 5	-	-	6.5	
Floor Beam 6	-	-	-	Floor beam located above Pier 1, no measurements recorded
Floor Beam 7	-	-	-	Floor beam located above Pier 1, no measurements recorded
Floor Beam 8	-	-	5.1	
Floor Beam 9		7	7	
Floor Beam 10	10	7	7	
Floor Beam 11	-	-	5.1	
Floor Beam 12	-	-	-	Floor beam located above Pier 2, no measurements recorded
Floor Beam 13	-	-	-	Floor beam located above Pier 2, no measurements recorded
Floor Beam 14	7	5	-	Local corrosion along the web, flanges delaminated
Floor Beam 15	6	9	7	
Floor Beam 16	10	8	6.8	
Floor Beam 17	8	8	-	
Floor Beam 18	-	-	-	Floor beam located above the manifold Cell, no measurements recorded

Table 1: A record of the UT readings taken along the floor beams of the structure during the inspection.

Photo 12: Photo showing the general floor beam condition. Overall conditions are similar between spans. General coatings breakdown, general corrosion observed, both flanges starting to delaminate locally in locations.

Photo 13: Photo showing the condition of Floor Beam 2. As can be seen in the photo above, a hole has developed in the web of the beam, and both the top and bottom flanges are heavily corroded.

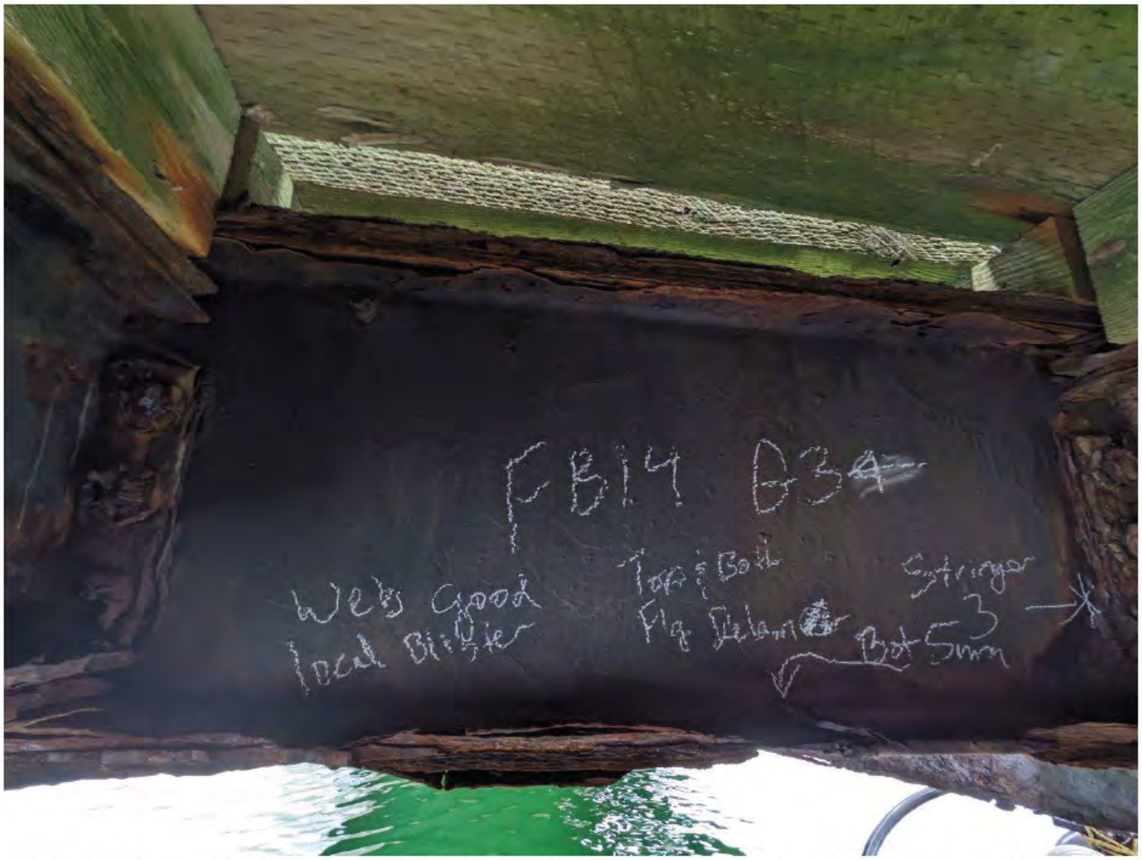


Photo 14: Photo showing the condition of Floor Beam 14. As can be seen in the photo, the coating along the web appears to be in fair condition. There is coating breakdown as visible corrosion, and delamination on the top and bottom flanges.

Inspection – Substructure Above Water Line

Photo 15: Photo showing the condition of Shore-side Cell. As can be seen in the photo above, there is marine growth over the wetted area of the steel sheet pile wall. There are no signs of coatings, corrosion is visible throughout, with areas of delamination in the steel sheet pile, and full section loss at locations.

Photo 16: Photo showing the condition of pile cap at Pier 1. Visible cracking throughout the pier cap can be seen, as well as efflorescence.

REDACTED (CONFIDENTIAL INFORMATION REMOVED)

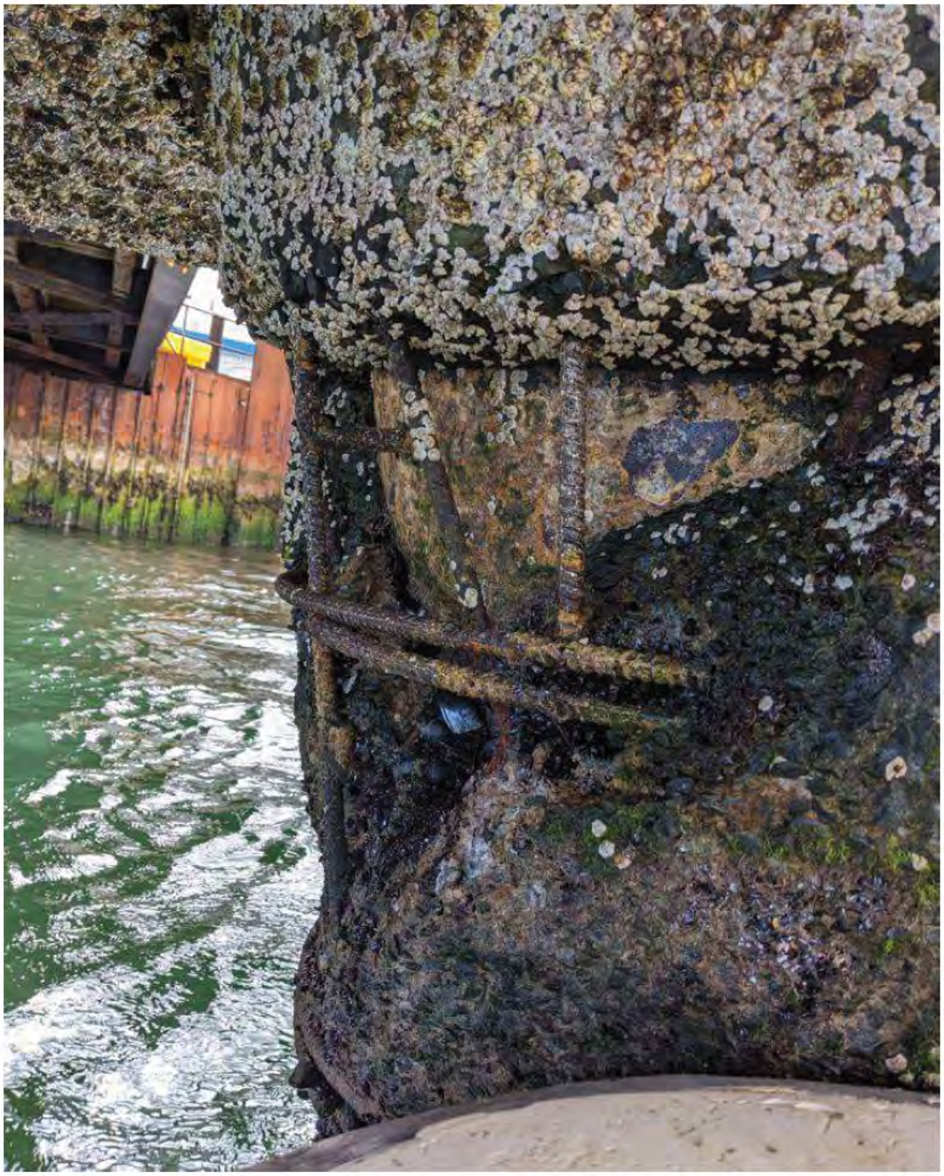


Photo 17: Photo showing the one of the piles at Pier 1. The concrete cover for the steel pile has spalled off locally and the rebar is exposed and has started to corrode.

Photo 18: Photo showing the condition of pile cap at Pier 2. Visible cracking throughout the pier cap can be seen, as well as efflorescence.

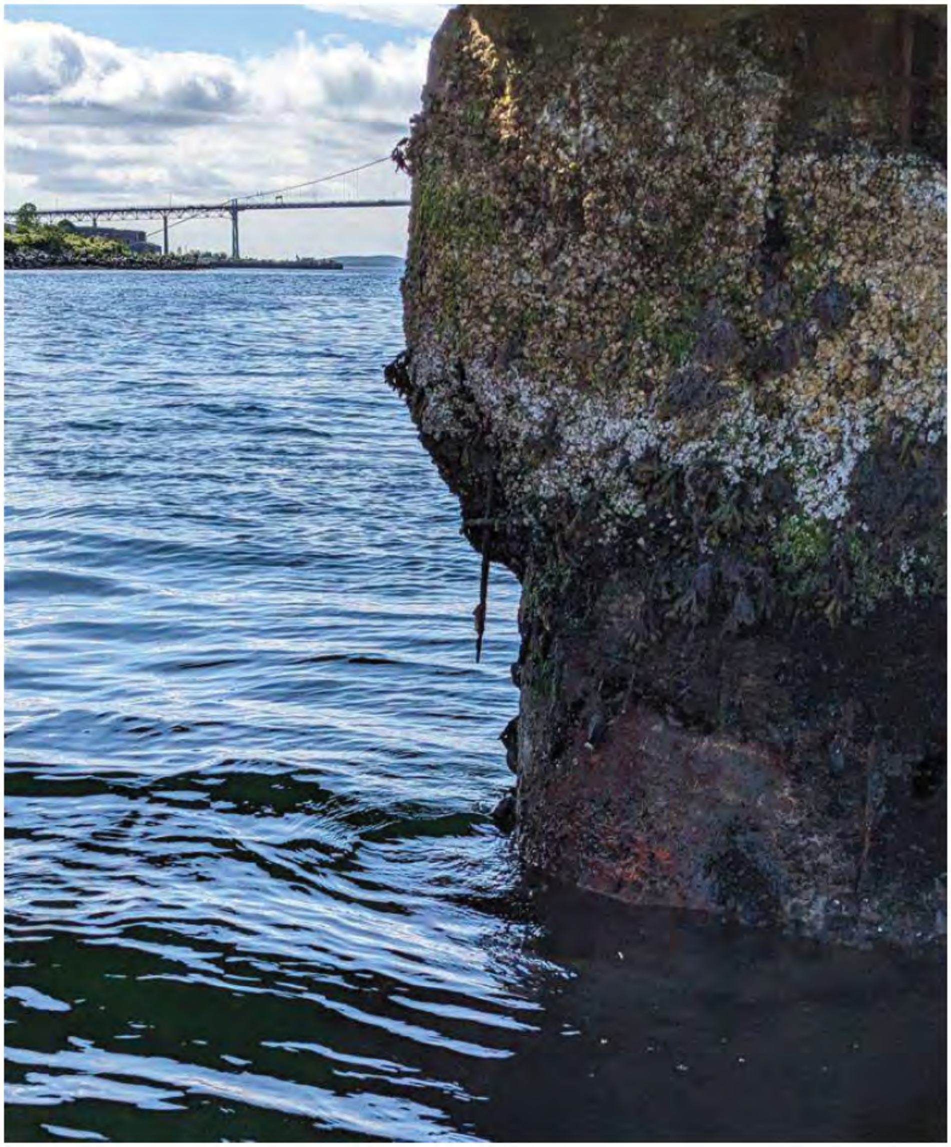


Photo 19: Photo showing the one of the piles at Pier 2. The concrete cover for the steel pile has spalled off locally and the rebar is exposed, and several bars have fully corroded through. The exposed steel pile has started to show signs of corrosion.

Photo 20: Photo showing the manifold cell structure. Marine growth was observed above the water level at the time of the inspection.

Photo 21: Photo showing the manifold cell structure. Marine growth was observed above the water level at the time of the inspection. There were local cracks and efflorescence visible along the structure.

Conclusions and Recommendations

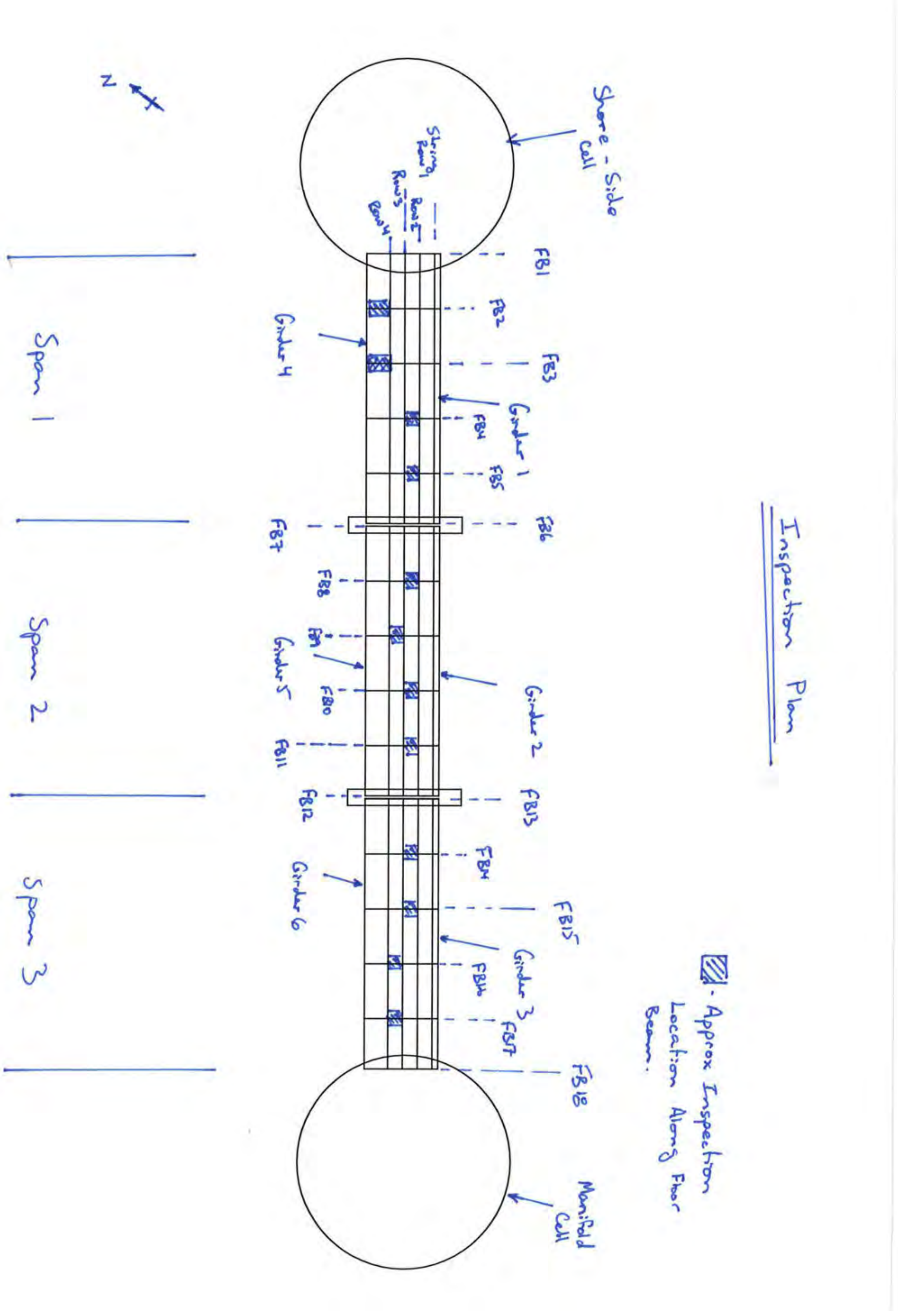
The findings for the inspection of the steel super structure will be used in conjunction with the other inspection data from the other reports will for the bases of the assessment performed by Harbourside on the steel superstructure. Generally, the primary girders were in good condition, however, the remaining steel structure has local areas with full section loss, visible corrosion throughout, and delamination in the flanges of the floor beams, and stringers. Additionally, the timber decking appeared in good condition and looks to have been replaced in 2002.

Sincerely,

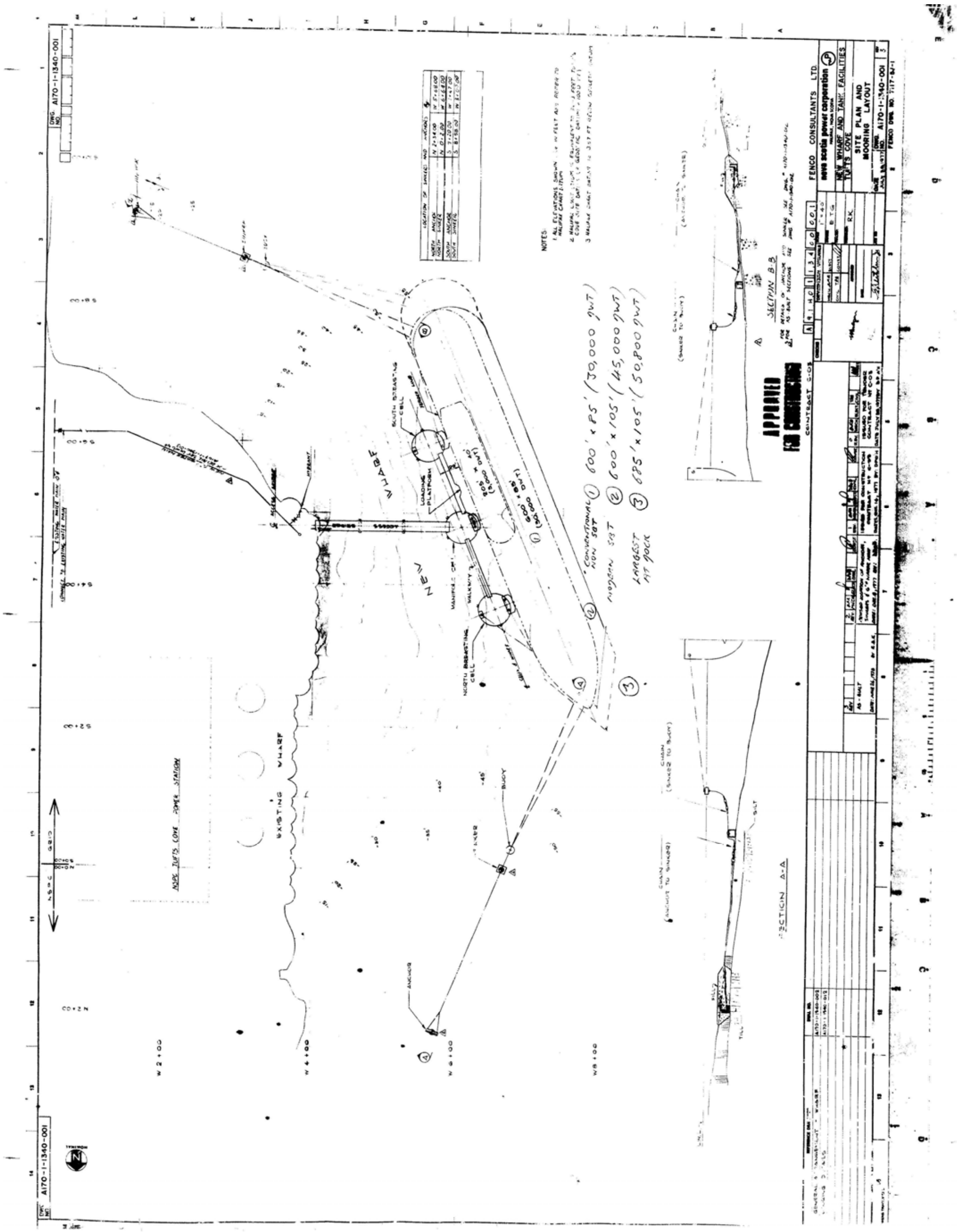
Tim Matthews, P. Eng.
Senior Structural Engineer
O 902 405 4696
E tmatthews@harboursideengineering.ca

Harbourside Engineering Consultants
A Division of the Harbourside Engineering Group

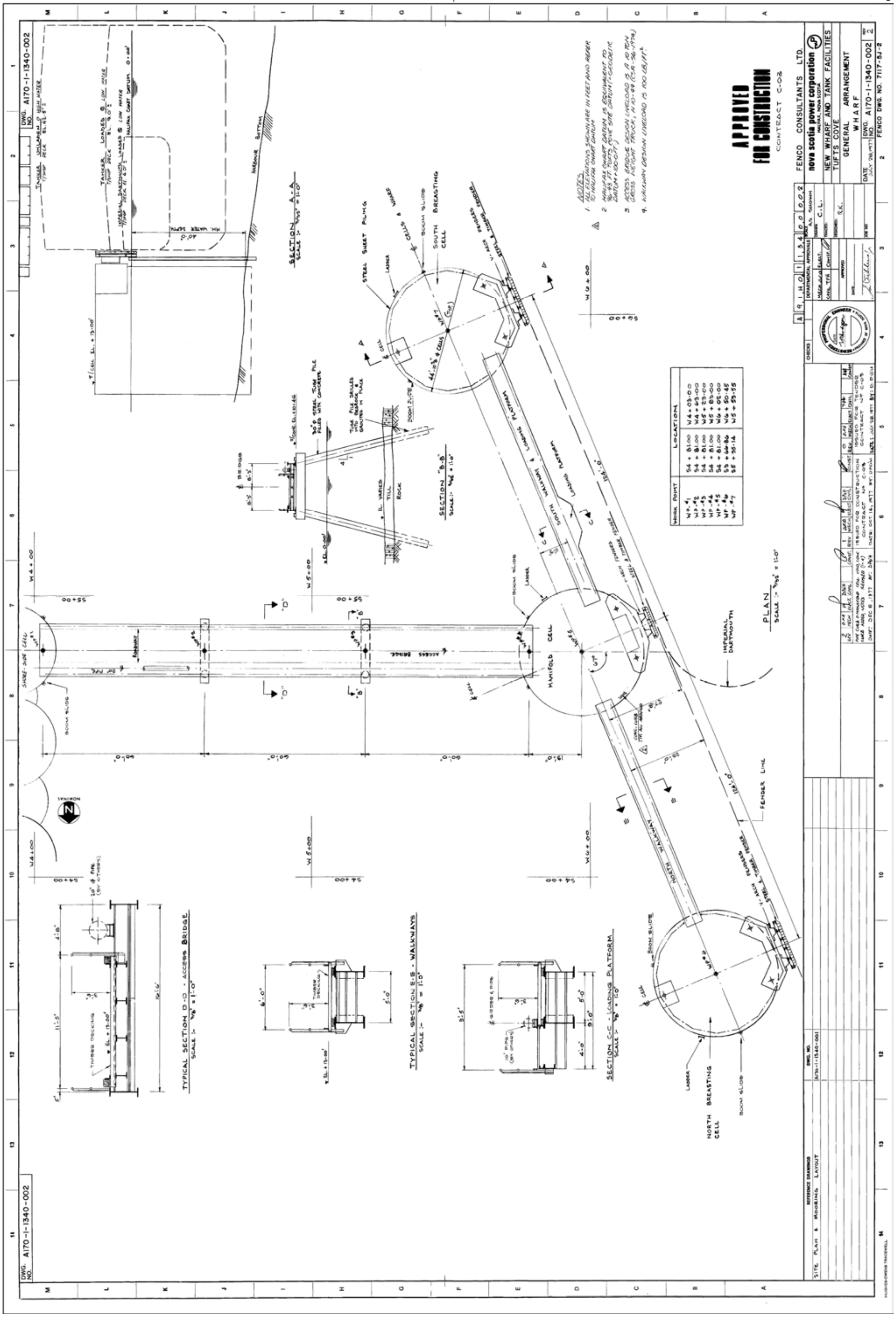
APPENDIX A – Inspection Plan


Inspection Plan
Approx Inspection Location Along Floor Beam.
Shore - Side Cell
Manifold Cell
N
String Row 1
Row 2
Row 3
Row 4
FB1
FB2
FB3
FB4
FB5
FB6
FB7
FB8
FB9
FB10
FB11
FB12
FB13
FB14
FB15
FB16
FB17
FB18
Girder 1
Girder 2
Girder 3
Girder 4
Girder 5
Girder 6
Span 1
Span 2
Span 3

APPENDIX B – Original Design Drawings

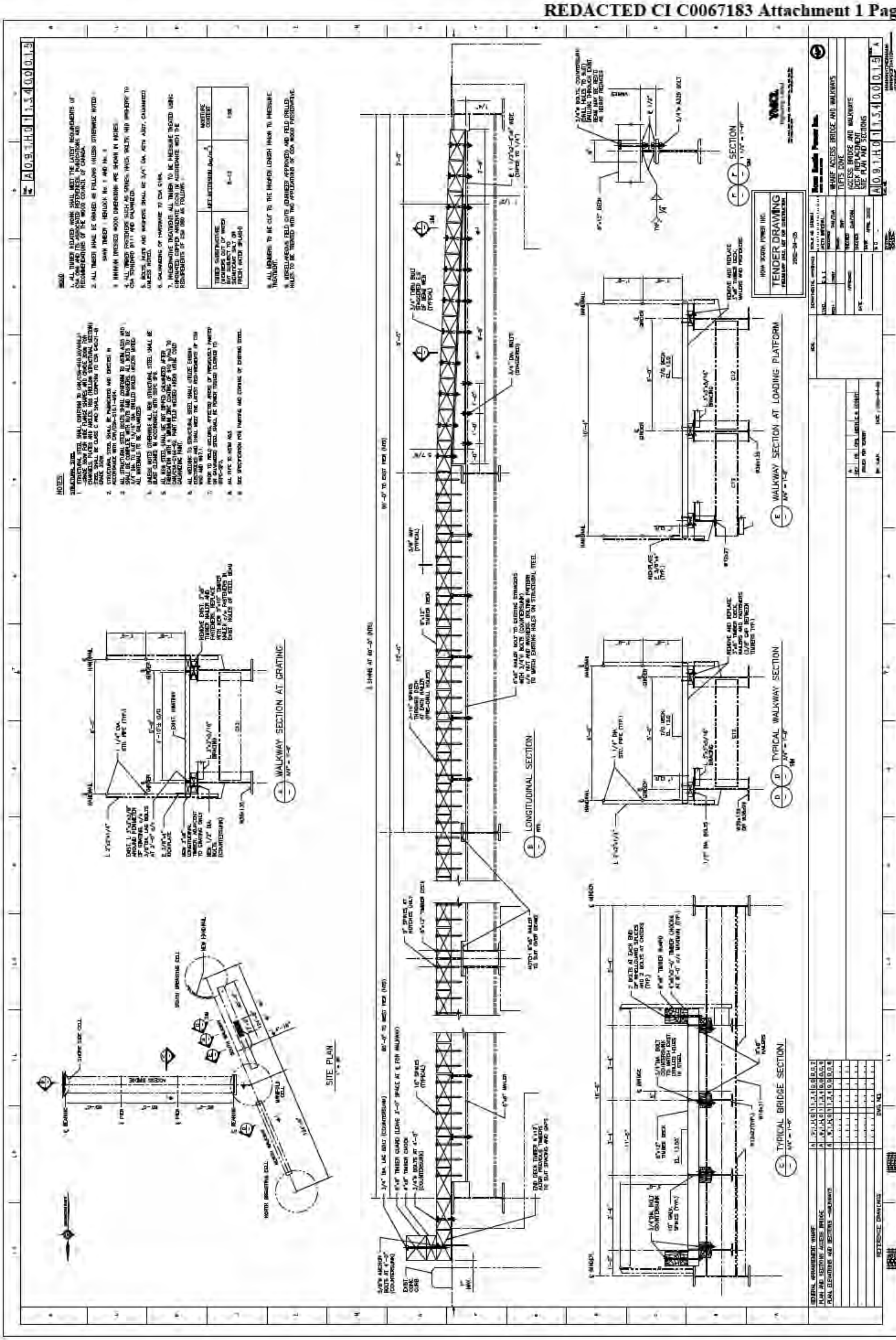


A170-1-1340-001
FENCO CONSULTANTS LTD.
nova scotia power corporation
NEW WHARF AND TANK FACILITIES TUFTS COVE
SITE PLAN AND MOORING LAYOUT
APPROVED FOR CONSTRUCTION
NEW WHARF
EXISTING WHARF
NSPC TUFTS COVE POWER STATION
SECTION A-A
SECTION B-B
NOTES:
SOUTH BREASTING CELL
NORTH BREASTING CELL
LOADING PLATFORM
BUOY
ANCHOR

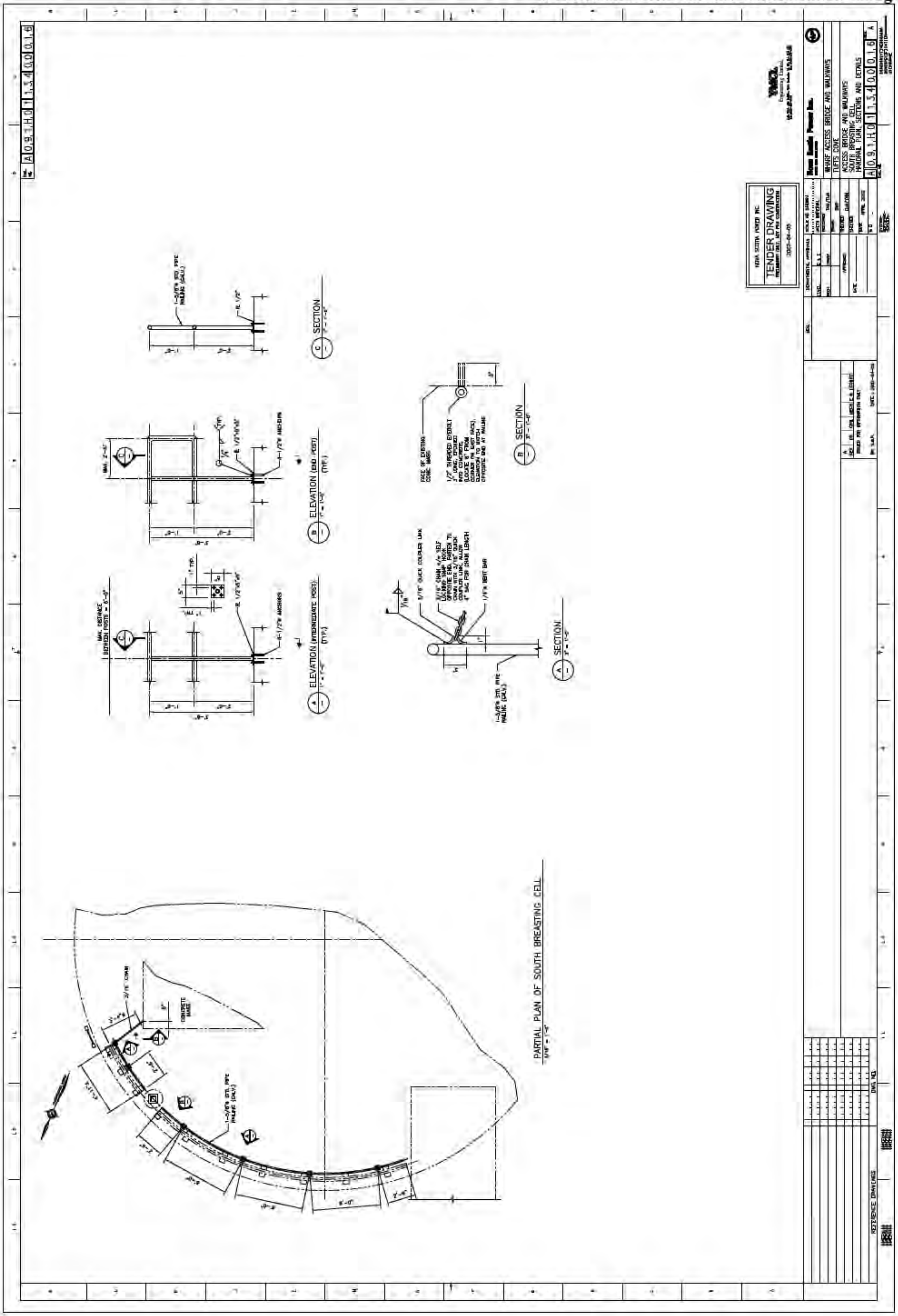


APPROVED FOR CONSTRUCTION
CONTRACT C-03
FENCO CONSULTANTS LTD.
nova scotia power corporation
NEW WHARF AND TANK FACILITIES
TUFTS COVE
GENERAL ARRANGEMENT
WHARF
DWG. NO. A170-1-1340-002
SECTION A-A
SECTION "B-B"
PLAN
TYPICAL SECTION D-D - ACCESS BRIDGE
TYPICAL SECTION E-E - WALKWAYS
SECTION C-C - LOADING PLATFORM
SOUTH BREASTING CELL
MANIFOLD CELL
NORTH BREASTING CELL
FENDER LINE
IMPERIAL DARTMOUTH
HARBOUR BOTTOM

REDACTED (CONFIDENTIAL INFORMATION REMOVED)



REDACTED (CONFIDENTIAL INFORMATION REMOVED)



APPENDIX C – Stantec Inspection Report

Stantec
Stantec Consulting Ltd.
102-40 Highfield Park Drive
Dartmouth NS B3A 0A3

May 31, 2024
File: 133432608

Attention: Connor Lebans, Mechanical Engineer – Tufts Cove and CTs
Nova Scotia Power Inc.
1223 Lower Water St
Halifax NS B3J 3S8

Dear Connor,

Reference: Condition Assessment of Access Bridge TUC, Nova Scotia

1.0 INTRODUCTION

Stantec Consulting Ltd. (Stantec) was requested by Nova Scotia Power Inc. (NSPI) to complete a non-destructive testing survey of one of the steel platforms located at their Tufts Cove facility in Dartmouth NS, to investigate the impact of corrosion on the structural integrity of the steel frame. The bridge provides access to the manifold cell and to the loading platform on the south walkway.

2.0 BACKGROUND INFORMATION

The existing platform spans East to West from the shore-side cell to the manifold cell and unloading dock. It was constructed in 1977. The platform consists of a structural steel frame, timber decking and concrete foundation. The steel frame is divided into three (3) equal spans. Each span is 18.2 m (60 ft) long and 5 m (16.5 ft) wide. Each span consists of two (2) girders spanning longitudinally, and six (6) beams, spaced at 3.6 m (11.8 ft), spanning perpendicularly to the girders. In between the beams, there are four (4) joists, spaced at 0.3 m (1 ft) or 1.7 m (5.6 ft). The lateral resistance of the structure is provided by horizontal bracing connected to the girders and located below the joists. The platform is supported by two piles caps dividing the platform into three (3) equal spans. At the ends, the bridge is supported by the manifold-cell concrete structure and the shore-side cell structure. There was no information about these two structures in the drawings received. The decking of the platform consists of timber decking running perpendicular to the main girders. The main purposes of the platform are to support the 20” dia. pipe which runs from the unloading dock to shore and to provide access for a 10 tons vehicle to the manifold cell. Plan views of the steel structure were prepared and can be found in Appendix A.

A site plan was provided by NSPI prior to the site visit. After the site visit, NSPI provided additional drawings for the timber decking replacement which included the sizes of some of the main steel framing members. The drawings received are included in Appendix D.

A set of drawings showing the layout of the platform was provided by NSPI prior to the site visit. Additional drawings were received after the site investigation which included the replacement of the timber decking of the platforms.

3.0 SITE VISIT

The site visit was conducted on April 24th, 2024 by Chi Phung and Daniel del Castillo Buiza, P. Eng of Stantec. The purpose of this visit was to complete a visual investigation of the steel frame of the wharf’s main platform, perform ultrasonic testing (UT) of the structural members to measure the corrosion levels, and to collect the information missing from the existing documentation received from NSPI. The visual inspection of the platform was focused on identifying locations of deterioration of the structural elements. UT measurements were taken at those locations where the deterioration was considered to have a potentially detrimental impact on the structural integrity of the steel members. Measurements of steel sections at intact locations were also taken to confirm the information included in the drawings.

The structural investigation was limited to the steel framing components of the access bridge. Other structural elements such as pile caps, piles and sheet piles, decking and non-structural items were not included in this investigation.

4.0 OBSERVATIONS

The overall dimension of the access bridge and member sizes were measured where accessible and were observed to be in general conformance with the information included in the drawings received. A summary of the member sizes can be found in Table 1, which includes the dimensions measured on site and the UT measurements taken on the structural members at locations not impacted by corrosion. It was observed that some of the member sizes indicated in the drawings are no longer available from Canadian mills. The general layout and location of bracing was not included in the drawings provided by NSPI. This information was collected during the site investigation. Information regarding the bearing plates and anchorage of the girders to the pile caps was also missing in the drawings received. These areas were not accessible during the inspection. The drawings in Appendix A include the location and magnitude of the UT measurements taken during the site investigation and the observed deficiencies.

Table 1 - Summary of Measurements in mm and Member Sizes

	Girders	Beams	End beams	Joists	Braces
Depth (d)	765	405	530	310	130
Flange Width (b)	265	145	330	170	130
Flange Thickness (t)	24.7	12.5	23.9	11.44	12.7
Web Thickness (w)	15.3	9.3	Not accessible	8.57	12.7
Metric Designation	W760x173	W410x46	Not able to provide	W310x39	L127x127x11
Imperial Designation	W30x116	W16x31	Not able to provide	W12x26	L5x5x1/2
Designation in provided drawings	Not provided	W16x31	Not provided	W12x27*	Not provided

Note: * W12x27 is no longer available.

Metric Designation and Imperial Designation refer to closest section size selected based on the measurements taken on site.

The steel structure is within the splash zone of the wharf. As a result, the steel frame is intermittently exposed to salt water and air. These cycles accelerate the deterioration of the coating protection leaving the steel unprotected against corrosion.

The girders were observed to be in fair condition. Surface corrosion was observed at multiple locations along the web and bottom flange of the girders. Based on the results of the structural analysis included in Section 5, this corrosion is not considered to have significant impact on the overall capacity of the girders. However, larger thickness losses were found at isolated locations. It is recommended to reinforce the girders at these locations. Coating failures were also observed at multiple locations on all girders. Re-coating operations are recommended to prevent corrosion from developing, which could negatively impact the structural integrity of the girders. Other elements, such as stiffeners, presented a similar condition and should be subjected to remediation actions as well.

The beams were observed to be in poor condition. Advanced corrosion was observed primarily on the flanges. UT measurements taken on these elements showed that the steel thickness was significantly reduced at these locations, having a negative impact on the structural capacity of the beams. The bolted connections between the joists and the beams were also in poor condition. Full material loss of the bolts was observed at most of these connections. Based on the condition observed on the beams, it is recommended to replace these members.

The joists were observed to be in poor condition with severe corrosion at multiple locations reaching full section loss (FSL) of the web on 16 joists throughout the platform. Due to the extent of the corrosion, repair operations are not considered to be suitable. Full replacement of the joists is recommended.

Similar condition was observed on the bracing elements. Severe corrosion was observed throughout the full length of the braces making it UT measurements on these elements unreliable. Due to the extent of the deficiencies, repair operations are not considered to be feasible. Full replacement of the bracing members is recommended.

Based on the overall condition of the structural members it is considered that the platform is not safe to use. Access to the platform shall be restricted for vehicles and pedestrians until remediation and replacement operations have been completed.

A summary of observations, recommendations, and priority of remediation measures are presented in Table 2. Priority for remedial measures is categorized as follows:

- I: immediate action/repair/replace.
- II: short-term repair or repair within the next year.
- III: monitoring and maintenance.

Reference: **Condition Assessment of Access Bridge TUC, Nova Scotia**

Table 2 - Summary of Observations

Location/ Label	Observation/Condition	Recommended Measures	Priority
Span A			
North Girder	• Localized reduction in section due to corrosion. • Connections welds show signs of corrosion. • Coating failure observed at multiple locations.	• Surface cleaning and re-coating. • Continue monitoring condition of the girder until repairs are completed.	II
South Girder	• Localized reduction in section due to corrosion. • Coating failure observed at multiple locations.	• Surface cleaning and re-coating. • Continue monitoring condition of the girder until repairs are completed.	II, III
Beams	• Severe corrosion in web and flanges at multiple locations. • Severe corrosion of connections to girders.	• Full replacement	I
Joists	• Severe corrosion in web and flanges. • Full section loss on multiple joists. • Severe corrosion of connections to beams.	• Full replacement	I
Bracing Members	• Severe corrosion throughout the full length of the braces. • Severe corrosion of connections.	• Full replacement	I
Stiffeners	• Severe section loss due to corrosion on multiple elements.	• General surface cleaning and re-coating • Full replacement of severely corroded stiffeners.	II
Span B			
North Girder	• Minor reduction in section due to corrosion.	• Surface cleaning and re-coating. • Continue monitoring condition of the girder until repairs are completed.	II, III
South Girder	• Minor reduction in section due to corrosion.	• Surface cleaning and re-coating. • Continue monitoring condition of the girder until repairs are completed.	II, III
Beams	• Severe corrosion in web and flanges at multiple locations. • Severe corrosion of connections to girders.	• Full replacement	I

Reference: **Condition Assessment of Access Bridge TUC, Nova Scotia**

Location/ Label	Observation/Condition	Recommended Measures	Priority
Joists	• Severe corrosion in web and flanges. • Full section loss on multiple joists. • Severe corrosion of connections to beams.	• Full replacement	I
Bracing Members	• Severe corrosion throughout the full length of the braces. • Severe corrosion of connections.	• Full replacement	I
Stiffeners	• Severe section loss due to corrosion on multiple elements.	• General surface cleaning and re-coating • Full replacement of severely corroded stiffeners.	II
Span C			
North Girder	• Minor reduction in section due to corrosion.	• Surface cleaning and re-coating. • Continue monitoring condition of the girder until repairs are completed.	II, III
South Girder	• Minor reduction in section due to corrosion.	• Surface cleaning and re-coating. • Continue monitoring condition of the girder until repairs are completed.	II, III
Beams	• Severe corrosion in web and flanges at multiple locations. • Severe corrosion of connections to girders.	• Full replacement	I
Joists	• Severe corrosion in web and flanges. • Full section loss on multiple joists. • Severe corrosion of connections to beams.	• Full replacement	I
Bracing Members	• Severe corrosion throughout the full length of the braces. • Severe corrosion of connections.	• Full replacement	I
Stiffeners	• Severe section loss due to corrosion on multiple elements.	• General surface cleaning and re-coating • Full replacement of severely corroded stiffeners.	II

5.0 STRUCTURAL ANALYSIS OF EXISTING GIRDERS

After the site visit, a structural analysis was performed to assess the impact of corrosion on the structural capacity of the platform. Based on the observations made on site and the results from the structural analysis, recommendations to repair or replace the structural members are provided. Detail design of the repairs is not included in the scope of this investigation.

Due to the observed deficiencies and condition of the platform, the existing capacity of the structure is considered to be provided solely by the existing girders. To complete the analysis on the capacity of the girders, the following assumptions have been considered:

- No material grade was included in the drawings received. The yield strength of the steel was assumed to be 300 MPa based on the time when the platform was constructed.
- The initial capacity of the beam was calculated to be 1650kNm in bending and 1955kN in shear.
- The remaining structural capacity of the girder was calculated based on the overall surface corrosion observed. Due to the fair condition of the girder, the impact on the capacity due to the corrosion observed is minor as large reduction in thickness was not generally found. However, the severe deterioration of the secondary structural members has reduced the lateral stability of the girders. This lack of lateral support has reduced the bending capacity of the girders to 300kNm, which is 18% of their initial resistance. The shear capacity of the girders is not impacted by the reduction in lateral support.
- The remaining moment and shear capacity of the girders were also calculated at those locations where higher corrosion levels were detected. The following measurements obtained from the ultrasonic testing were used for this calculation:
  - The web thickness of the beam was reduced to 6.4 mm based on UT readings.
  - The flange thickness of the beam was reduced to 12.7 mm based on UT readings.

  The remaining moment and shear capacity were calculated to be 215 kNm, and 306kN respectively. The capacity of the girders has been reduced to 13% in bending and 16% in shear. The value obtained for bending accounted for the lack of lateral support due to the poor condition of the secondary steel members.
- The maximum surface load that would result in the girders reaching their maximum capacity was calculated to be 1.99 kPa (40 psf). However, due to the condition of the intermediate structural members, this capacity cannot be relied upon as the beams and joists will not be able to transfer the load from the decking system to the girders. A load rating cannot be provided based on the current condition of the platform.

These results were obtained based on the data obtained through UT readings. This method only provides information at an isolated location on the steel members, this is being done at the discretion of the engineers. However, the member condition can be worse than what was measured on site based on the non-uniform deterioration of the steel. UT measuring could not be completed on all members due to limited accessibility.

6.0 RECOMMENDATIONS

Based on the observations made on site and the results from the structural analysis the following are recommended:

- Vehicles and pedestrians access to platform shall be restricted until repairs and/or replacement operations have been made.
- Evidence of corrosions and/or coating failures were observed on both girders during the investigation. It is recommended that the girders and supports be cleaned, and protecting coatings reapplied to prevent further deterioration of the girders. Reinforcement of the girders at the locations where larger thickness loss was observed. Multiple connections between beams and girders were observed to be severely corroded. It is recommended that at the location of the connections, the steel to be cleaned to not further damage the girders.
- The bearing plates were not accessible at the time of inspection. It is recommended that the bearing plates be inspected.
- Implementation of a monitoring plan to include the inspection of the girders every 5 years after the completion of the repairs to confirm no additional deterioration occurs.
- The current condition of the braces, joists, and beams do not satisfy the loading requirements. It is recommended that these members be replaced.

7.0 ADDITIONAL OBSERVATIONS AND RECOMMENDATIONS

During the site investigation, observations were also made on non-structural elements and other structural elements not included in the scope of this investigation. These are briefly explained below along with recommendations for these elements:

- The North handrails along the North side were supported by the joists and were observed to be in poor condition. The connection bracket to the joist showed full section loss at multiple locations. The condition of handrails can pose a safety risk for platform users. It is recommended that the handrail is replaced immediately.
- Efflorescence and cracking were observed in the pile caps. Based on life span of the structure, it is recommended that an investigation of the condition of the pile caps and battered piles be completed.

8.0 CONCLUSION

The condition assessment and UT measurements show severe deterioration in the condition of the connections and the steel structural elements of the platform. Further investigation to examine the condition of the foundations and bearing plates is recommended. It is recommended that the replacement of the joists, braces, and beams is completed as soon as reasonably possible if the intent is to maintain the platform. Surface cleaning and recoating of the girders should be completed along with the replacement of the rest of the structural members. Until the remediation operations are completed, vehicle access is restricted and pedestrian access should be limited to one person until the completion of the repair/replacement operations.

9.0 CLOSING

This document entitled Condition Assessment of Access Bridge, TUC, Nova Scotia was prepared by Stantec Consulting Ltd. ("Stantec") for the account of NSPI (the "Client"). Any reliance on this document by any third party is strictly prohibited. The material in it reflects Stantec's professional judgment in light of the scope, schedule and other limitations stated in the document and in the contract between Stantec and the Client. The opinions in the document are based on conditions and information existing at the time the document was published and do not take into account any subsequent changes. In preparing the document, Stantec did not verify information supplied to it by others. Any use which a third party makes of this document is the responsibility of such third party. Such third party agrees that Stantec shall not be responsible for costs or damages of any kind, if any, suffered by it or any other third party as a result of decisions made or actions taken based on this document.

Regards,

Stantec Consulting Ltd.

Chi Phung E.I.T.
Structural Engineer in Training
Phone: (902) 403-4769
chi.phung@stantec.com

Daniel del Castillo Buiza M.Eng., P.Eng..
Structural Engineer
Phone: (902) 707-4324
daniel.delcastillobuiza@stantec.com

Nathan Guy

Nathan Guy P.Eng.
Team Lead - Structural
Senior Structural Engineer
Phone: (902) 237-4972
nathan.guy@stantec.com

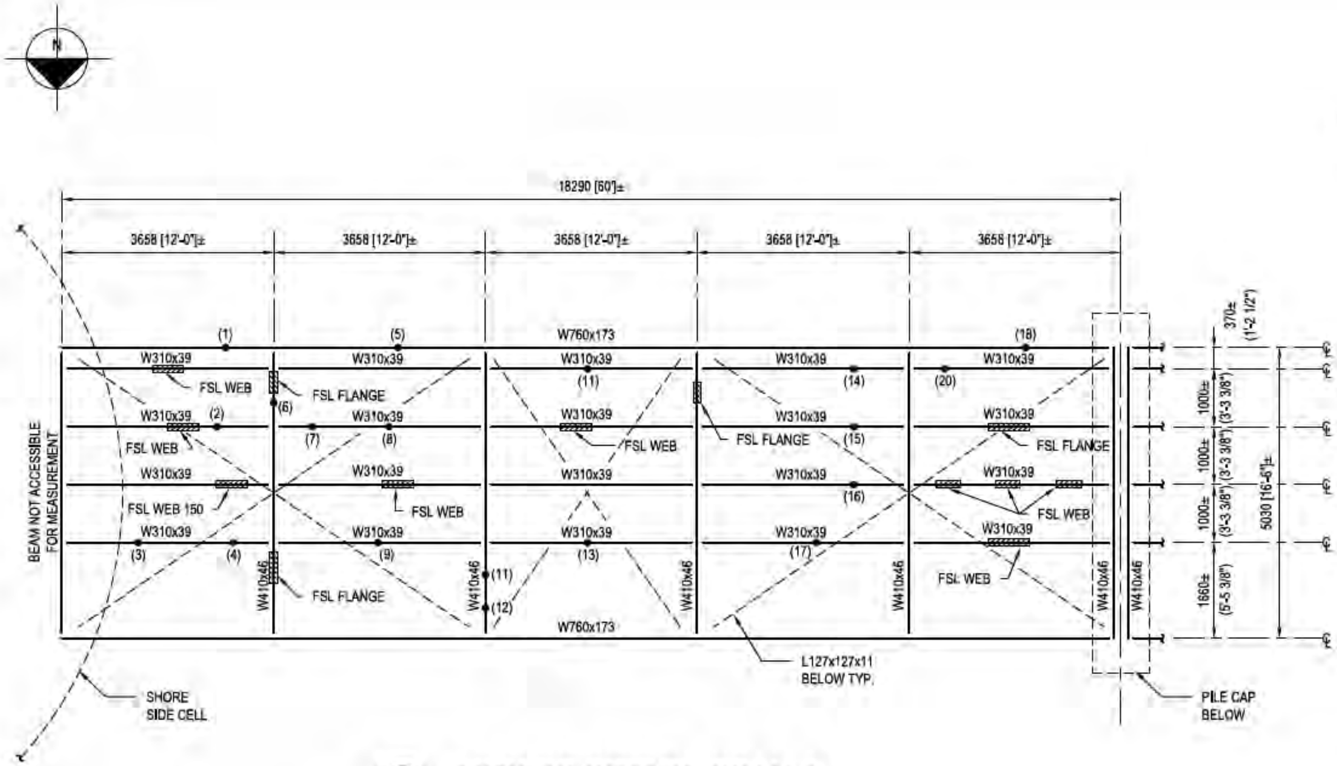
Attachment: Appendix A: Drawings: Observed Condition and UT Measurements
Appendix B: Site Photos
Appendix C: UT Equipment Reports
Appendix D: Provided Drawings

May 31, 2024
Connor Lebans

Reference: **Condition Assessment of Access Bridge TUC, Nova Scotia**

APPENDIX A

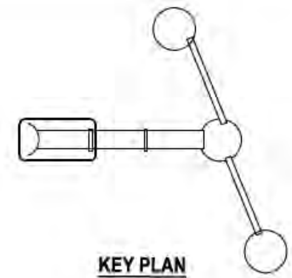
Drawings: Observed Condition and UT Measurement



ACCESS BRIDGE SPAN A

1 / -

SCALE: 1:75



KEY PLAN

LOCATION	UT READING WEB	UT READING BOT. FLANGE
(1)	6.4	12.7
(2)	3.7	
(3)	5.9	
(4)	5.1	
(5)	15.3	24.7
(6)	9.3	
(7)	7.3	
(8)	8.6	11.4
(9)	3.5	
(10)	6.8	
(11)	6.7	
(12)	6.2	7.8
(13)	3.0	
(14)	5.2	
(15)	6.7	
(16)	7.1	
(17)	5.8	
(18)	21.7	
(19)	5.2	

MAY 2024

U:\133432608_working_files\03_structural\02_cad\03_sheets\NSPI-Jetty Inspection - 133432608.dwg
2024/05/15 5:03 PM By: Lantey, Paul

Legend

FSL - FULL SECTION LOSS

- FULL SECTION LOSS OBSERVED

● (#) - LOCATION OF UT MEASUREMENT TAKEN

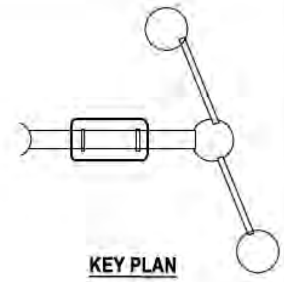
Stantec

102 - 40 Highfield Park Drive
Dartmouth NS
www.stantec.com

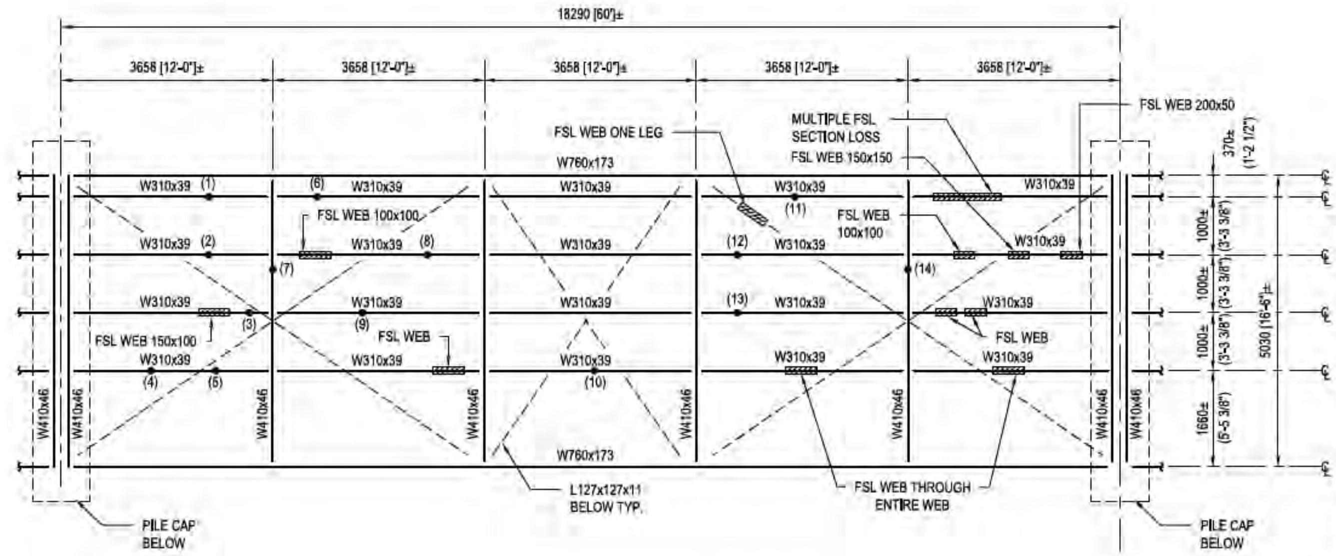
Client/Project
NOVA SCOTIA POWER INC.
TUFTS COVE WHARF INSPECTION

Figure No.
S01

Title
SPAN A SITE OBSERVATIONS AND U.T. MEASURMENTS



LOCATION	UT READING WEB	UT READING BOT. FLANGE
(1)	6.2	
(2)	5.7	
(3)	4.4	
(4)	4.4	
(5)	5.2	50% SECTION LOSS
(6)	3.5	
(7)	5.4	
(8)	3.6	
(9)	5.7	
(10)	4.9	
(11)	6.7	
(12)	4.9	
(13)	4.7	
(14)	5.5	



2 ACCESS BRIDGE SPAN B

SCALE: 1:75

U:\133432608\c_working_files\03_structural\02_cad\03_sheets\NSPI-Jetty Inspection - 133432608.dwg
2024/05/15 5:04 PM By: Lantey, Paul

MAY 2024

Stantec

102 - 40 Highfield Park Drive
Dartmouth NS
www.stantec.com

Legend

FSL - FULL SECTION LOSS

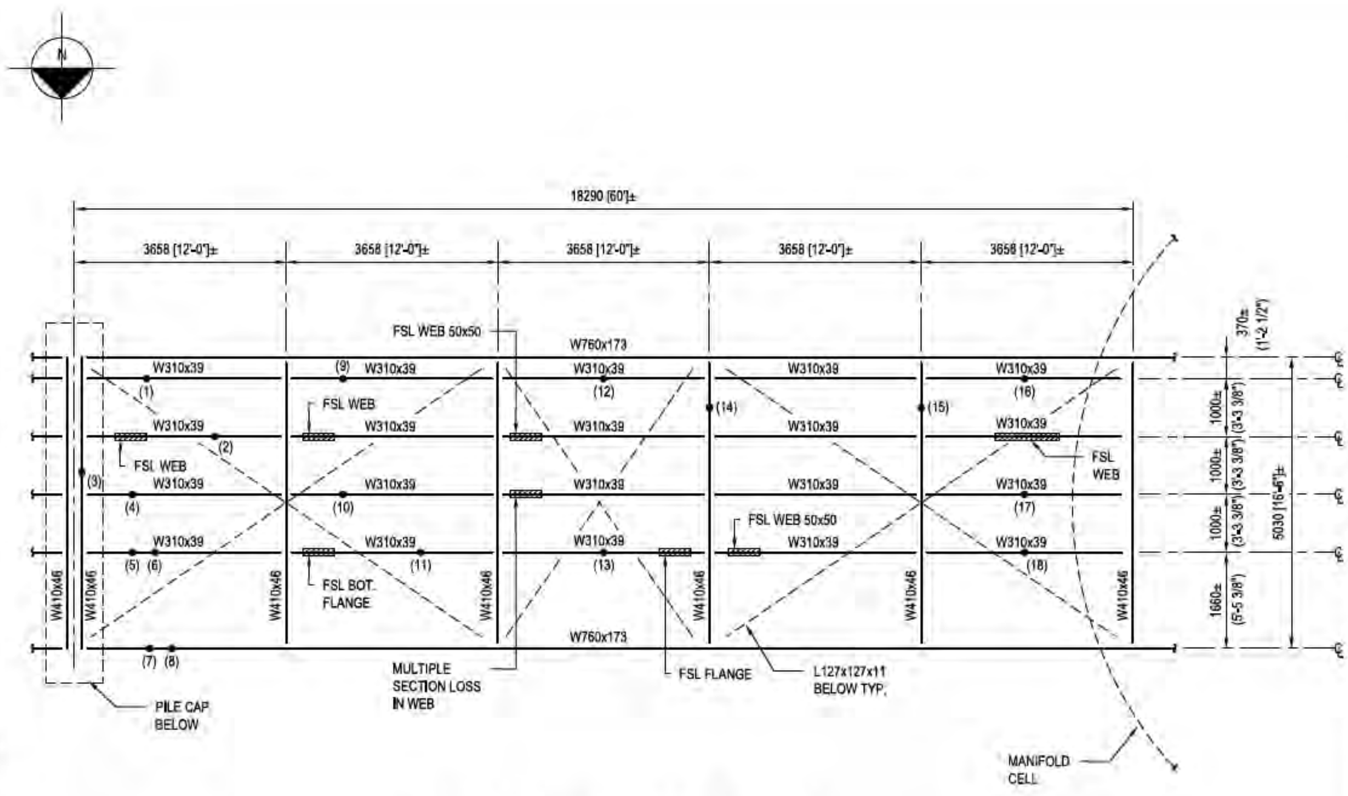
- FULL SECTION LOSS OBSERVED

(#) - LOCATION OF UT MEASUREMENT TAKEN

Client/Project
NOVA SCOTIA POWER INC.
TUFTS COVE WHARF INSPECTION

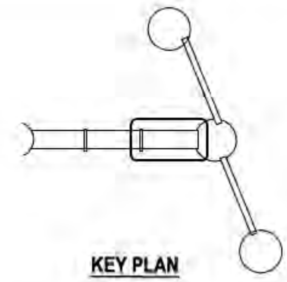
Figure No.
S02

Title
SPAN B SITE OBSERVATIONS AND U.T. MEASURMENTS



3 ACCESS BRIDGE SPAN C

SCALE: 1:75



KEY PLAN

LOCATION	UT READING WEB	UT READING BOT. FLANGE
(1)	5.3	
(2)	4.0	
(3)	10.1	
(4)	3.7	
(5)	3.3	
(6)	3.2	
(7)	12.5	22.6
(8)		20.9
(9)	5.7	
(10)	4.9	
(11)	4.9	
(12)	4.2	
(13)	4.7	
(14)	6.2	
(15)	8.3	
(16)	3.6	
(17)	6.7	
(18)	5.4	

MAY 2024

U:\133432608\1_working_files\03_structural\02_cad\03_sheets\NSPI-Jetty Inspection - 133432608.dwg
2024/05/15 5:08 PM By: Lantey, Paul

Stantec

102 - 40 Highfield Park Drive
Dartmouth NS
www.stantec.com

Legend

FSL - FULL SECTION LOSS

▨ - FULL SECTION LOSS OBSERVED

• (#) - LOCATION OF UT MEASUREMENT TAKEN

Client/Project
NOVA SCOTIA POWER INC.
TUFTS COVE WHARF INSPECTION

Figure No.
S03

Title
SPAN C SITE OBSERVATIONS AND U.T. MEASURMENTS

May 31, 2024
Connor Lebans

Reference: **Condition Assessment of Access Bridge TUC, Nova Scotia**

APPENDIX B

Site Photos

May 31, 2024
Connor Lebans

Reference: **Condition Assessment of Access Bridge TUC, Nova Scotia**

Photo 1 - South Girder at Span A showing localized corrosion in the web.

Photo 2 - South Girder at Span A showing corrosion on the flange and coating failure.

Photo 3 - North Girder at Span A showing corrosion on the flange and web with coating failure.

Photo 4 - North Girder at Span A and typical bolted beam to girder connection.

May 31, 2024
Connor Lebans

Reference: **Condition Assessment of Access Bridge TUC, Nova Scotia**

Photo 5 - Full section loss of through the web of joists.

Photo 6 - 2 full section loss through web of joists measuring 18" in total length.

Photo 7 - Full section loss at web of joist measuring 6” long.

Photo 8 - Full section loss at web of joist extend through the full height of the web and significant section loss in flange.

May 31, 2024
Connor Lebans

Reference: **Condition Assessment of Access Bridge TUC, Nova Scotia**

Photo 9 - Braces with severe corrosion damage.

Photo 10 - Braces with severe corrosion damage prior to corroded area being removed.

Photo 11 - Beam with corrosion damage.

Photo 12 - Beam with full section loss through the flange between the joists.

May 31, 2024
Connor Lebans

Reference: **Condition Assessment of Access Bridge TUC, Nova Scotia**

Photo 13 - Coating on girder alligatoring (South girder).

Photo 14 - Coating of girder cracking and rusting (North girder).

Photo 15 - South Girder at Span C in fair condition.

Photo 16 - South Girder at Span B in fair condition.

May 31, 2024
Connor Lebans

Reference: **Condition Assessment of Access Bridge TUC, Nova Scotia**

Photo 17 - North Girder at Span C in fair condition with minimal section loss due to corrosion.

Photo 18 - North Girder (South side) at Span C in fair condition.

Photo 19 - Presence of efflorescence of observed in pile caps.

May 31, 2024
Connor Lebans

Reference: Condition Assessment of Access Bridge TUC, Nova Scotia

Photo 20 - Full section loss of North handrail to the structure.

Photo 21 - Full section loss of North handrail to the structure.

Photo 22 - Full section loss of North handrail to the structure.

Photo 23 - Full section loss of North handrail to the structure.

May 31, 2024
Connor Lebans

Reference: **Condition Assessment of Access Bridge TUC, Nova Scotia**

Photo 24 - Poor condition observed in connection at multiple locations.

Photo 25 - Poor condition observed in connection at joist to beam connection.

May 31, 2024
Connor Lebans

Reference: **Condition Assessment of Access Bridge TUC, Nova Scotia**

Photo 26 - Connection different from the rest of the structure (South side of North girder on Span A).

Photo 27 - Connection different from the rest of the structure (North side of North girder on Span A).

Photo 28 - Bearing plate and anchorage to pile caps observed to be in fair condition.

Photo 29 - Bearing plate and anchorage to pile caps observed to be in fair condition.

May 31, 2024
Connor Lebans

Reference: **Condition Assessment of Access Bridge TUC, Nova Scotia**

Photo 30 - Coating failure and superficial corrosion observed at the welds at splice location in the girders and the girder bearing plate.

Photo 31 - Coating failure and superficial corrosion observed at the welds at splice location in the girders.

Photo 32 - South side of North Girder

Photo 33 - Girders connection at pile caps.

May 31, 2024
Connor Lebans

Reference: **Condition Assessment of Access Bridge TUC, Nova Scotia**

APPENDIX C

UT Equipment Reports

Acuren Group Inc.
61 Raddall Avn. Unit O
Dartmouth, NS, Canada
www.acuren.com
Phone: 902.434.4405
Toll Free: 800.252.1774

A Higher Level of Reliability

ACUREN

NONDESTRUCTIVE EXAMINATION

CLIENT: **NSPI- TUC**

PAGE: 1 of 1

DATE: April 24 -2024
ACUREN JOB #: 180-J134917
REPORT #: UT-CONJER-240424-01
CONTRACT/PO: NA WO: NA

ATTENTION: **N.S Power** WORK LOCATION: Dartmouth, N.S
PROJECT: TUC Wharf Structural Inspection
ITEM(S) EXAMINED: Structural Steel I- beams and Angle Iron.

PART #: NA MATERIAL: C.S THICKNESS: Various
SCOPE: Perform Ultrasonic spot checks on select locations on Underside of Wharf structure.
TYPE OF INSPECTION: Ultrasonic

TEST DETAILS:

ACCEPTANCE STANDARD: Client's Information REVISION: N/A
PROCEDURE/TECHNIQUE: CAN-UT-14T001 REVISION: 9
TYPE: METHOD:
INSTRUMENT: Olympus MODEL: 38 DL Plus S/N: 100080110 CAL DUE: Sept 20-2024
CAL. BLOCK: Step Block S/N: 0587 22 CABLE-TYPE: Coaxial LENGTH: 5'
CAL. BLOCK: S/N: COUPLANT: Sono 600

Probe & Technique Details:

	Test Angle (°)	Probe Type	Crystal Size	Freq. (MHz)	Serial Number	Damping Ω	Test From	Reference Reflector	Transfer Value	Reference		Scan dB	Range
										dB	% FSH		
1	0	D790	.500	5	1402266	N/A	Surface	Backwall	N/A	61	80	As req.	50-100mm

TEST SURFACE CONDITION: Clean TEST SURFACE TEMPERATURE: 7-25C

RESULTS:

Ultrasonic spot checks were performed on select locations on Underside of Wharf at the discretion of attending engineer.

*U.T readings in mm's.

CLIENT: ______ CLIENT PRINTED NAME ______ CLIENT SIGNATURE ACCEPTED & ACKNOWLEDGED BY

DTR No.: N/A

ACUREN TECHNICIAN: Jeremy Connor
1st Technician
CGSB MT/PT/ UT 2
CGSB Reg. #13535

2nd Technician

REVIEWER:

APPENDIX D – Connors Diving Substructure Inspection Report

Tufts Cove Marine Terminal

Underwater Visual & NDT Inspection

Dartmouth, NS

Inspection Report

Prepared for:

Connor Lebans, P.Eng.
Mechanical Engineer
Nova Scotia Power Inc.
Dartmouth, NS
Tel:902-237-2452
Connor.lebans@nspower.ca

Prepared by:

Cole Scarfe
Connors Diving Services Ltd
11- 2 Lakeside Park Drive
Halifax NS Canada B3T 1L7
Tel: 1.902.) 876.7078
Fax: 1.902. 876.7079

Conducted On:
June 25, 2024

REDACTED (CONFIDENTIAL INFORMATION REMOVED)

TABLE OF CONTENTS

General Information

Task:	Marine Terminal Inspection	
Reference:	Nova Scotia Power – Connor Lebans	
Personnel	Diving Supervisor	Jon Noiles
	Diver #1	Harry Demarjian
	Diver #2	Matt Chisolm
	Diver #3	Brennen Travis
Date/Time Commenced:	0700 – June 25, 2024	
Date/Time Completed:	1600 – June 26, 2024	
Location:	Dartmouth, NS	
Weather	Calm – Light Breeze	
Sea Conditions	Light	
Visibility - Surface	Unlimited	
Visibility – U/W	< 5'	

Introduction

As Directed by Connor Lebans of Nova Scotia Power, Connors Diving Services Carried out a structural inspection of the Tufts Cove Marine Terminal Including Support Piles and Docking Cells.

Procedure

As per the Nova Scotia Occupational Diving Regulations a four-man dive team is required. Upon arriving, a hazard assessment of the area was conducted. This, in conjunction with the dive plan was reviewed by the dive team and NSP representative prior to the start of any work and all LOTO permits were reviewed and signed off by all members. In addition, the scope of the work and procedures are discussed during the toolbox meeting.

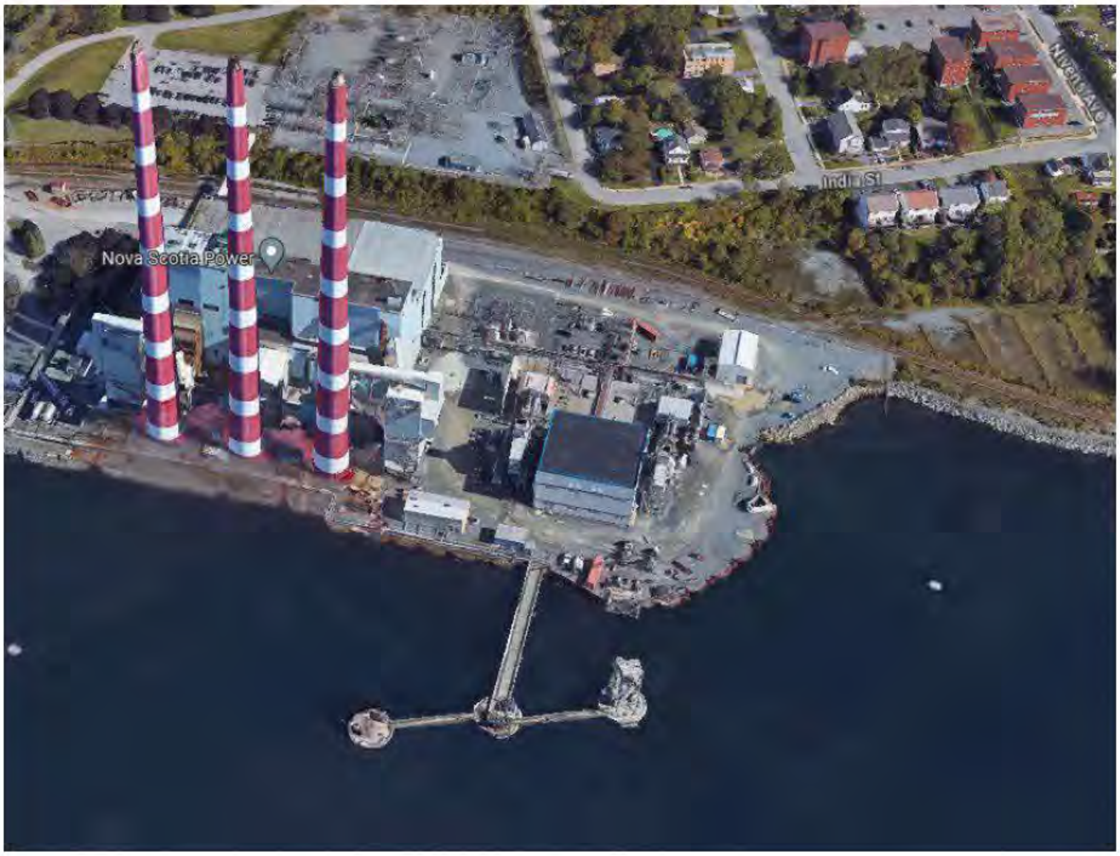
Divers used surfaced supplied diving equipment (SSDE) and closed circuit T.V. (CCTV). This gives the supervisor and client representative a real-time look at what the diver is experiencing. This ability also allows questions to be asked during the dive, eliminating the chance of missing valuable data.

The Marine Terminal consists of three Steel Sheet Pile cells, and an approach way from the shoreline supported by Steel Pipe Piles.

The focus of this survey was to identify any areas in need of repair and carry out NDT testing at specific locations throughout the Terminal as well as a general survey of the entire Terminal for other structural defects or anomalies found.

Nova Scotia Power

Tufts Cove Marine Terminal Inspection – Focal Points
Dartmouth, NS



Marine Terminal – Focal Points

1. **Middle Dock Cell** – Cope Wall Deterioration/Loss of Concrete/Exposed Rebar
2. **Middle Dock Cell –** Steel Sheet Pile Deterioration/Contact Damage/NDT Testing
3. **Shoreline SSP Cell -** Steel Sheet Pile Deterioration/Contact Damage/NDT Testing
4. **Approach way Piles –** Concrete Jacket Deterioration/Loss of Concrete/Exposed Rebar
5. **Approach way Piles –** Steel Pipe Pile Deterioration/Contact Damage/NDT Testing

Nova Scotia Power
Tufts Cove Marine Terminal Inspection – Inspection Areas
Dartmouth, NS

Middle Dock Cell (Steel Sheet Pile)
The Middle Dock Cell inspection consisted of a Visual Inspection of the Concrete Cope Wall, a Visual Inspection of the Steel Sheet Pile, and Ultrasonic Testing of the Steel Sheet Pile.

Main Approach way (Steel Pipe Piles)
The Main Approach way Steel Pipe Pile Inspection consisted of a Visual Inspection of the Concrete Pile Jackets, a Visual Inspection of the Steel Piles, and Ultrasonic Testing of the Steel Pipe Piles.

Shoreline Cell (Steel Sheet Pile)
The Shoreline Dock Cell inspection consisted of a Visual Inspection of the Steel Sheet Pile, and Ultrasonic Testing of the Steel Sheet Pile.

Middle Dock Cell (Steel Sheet Pile)
The Middle sheet pile cell was found to have 100% coverage of soft marine growth over a layer of surface corrosion which covers the entirety of the SSP.

Eight (8) Sheet Pile Pans were cleaned from the bottom of the concrete cope wall to the seabed at cardinal locations (North, Northeast, East, Southeast, South, Southwest, West, and Northwest). At each Cardinal location Ultrasonic Steel Thickness Testing was retrieved at 6 pre-determined elevations.

The diver inspected all the cleaned areas as well as carrying out a visual inspection of the uncleaned areas looking for any signs of large deficiencies in the sheet pile.

The Middle Dock Cell appeared to be in relatively good condition with minor pitting throughout the faces of the SSP inspected.

The Results of the Ultrasonic Thickness (UT) readings, and their locations are denoted on the table below.

Note: all values are expressed in millimeters

Location	Directly below cope wall	1m Below	2m Below	Midway	1m above Mudline	Mudline	Water depth at Mudline
North	8.80	9.80	10.15	10.30	10.80	10.40	38′
Northeast	10.50	9.25	10.45	10.70	10.70	10.70	33′
East	8.30	9.75	9.65	9.85	10.40	10.25	32′
Southeast	8.85	8.60	11.05	11.20	9.30	11.20	33′
South	9.75	10.35	10.25	10.80	10.05	10.50	38′
Southwest	8.30	8.95	9.05	9.70	10.75	11.10	40′
West	10.10	9.70	9.10	9.50	10.75	10.40	41′
Northwest	8.80	9.40	8.15	10.65	11.05	10.60	41′

Shoreline Cell (Steel Sheet Pile)

The Shoreline sheet pile cell was found to have 100% coverage of soft marine growth over a layer of surface corrosion which covers the entirety of the SSP.

Every third Sheet Pile Pan starting from the South Cell Edge was cleaned from the waterline to the seabed (i.e. #1, #4, #7, etc.). At each location Ultrasonic Steel Thickness Testing was retrieved at 3 pre-determined elevations.

The Diver inspected all the cleaned areas as well as carrying out a visual inspection of the uncleaned areas looking for any signs of large deficiencies in the sheet pile.

The Shoreline Dock Cell appeared to be in poor condition with visible holes and heavy corrosion present in the Tidal Zone.

The Results of the Ultrasonic Thickness (UT) readings, and their locations are denoted on the table below.

Note: all values are expressed in millimeters

Location (Sheet Pile Pan #)	Tidal Zone	Mid Water	Seabed
1	8.25	7.80	9.30
4	7.35	8.50	8.55
8	8.05	8.35	7.05
10	8.10	6.15	6.45
13	6.45	7.35	9.40
16	7.00	7.55	8.30
19	9.80	8.95	9.50
22	7.10	6.95	8.85
25	7.45	8.45	9.80

Main Approach way (Steel Pipe Piles)

The Four (4) Steel Approach way Piles were found to have 100% Soft Marine Growth Coverage throughout the entirety of the Pile.

Piles are Denoted as NW, NE, SW, and SE. The Diver inspected Both the Concrete Pile Jackets in the Tidal Zone, as well as the Exposed Pipe Piles running to the Seabed.

The Results of the Visual Inspections, Ultrasonic Thickness (UT) readings, and their locations are denoted on the tables below.

Southwest Steel Pipe Pile

Location	Directly below Jacket	1m Below	Midway	1m above Mudline	Mudline
North	7.95	7.45	6.20	5.25	6.15
East	6.55	6.15	7.45	7.95	5.95
South	8.85	7.00	7.05	6.15	7.25
West	6.90	8.20	7.95	5.15	8.25

Southwest Concrete Pile Jacket

Location of Deficiency	Notes/Area of Concern
West Face	Area of Spalling w/ Exposed Rebar (5.5'w x 3'h)
South Face	Heavy Spalling/Washout (7'w x 2.5'h)
East Face	Concrete Loss at Base of Jacket (4.5'w x 1'h)
North Face	Concrete Loss at Base of Jacket (3'w x 1'h)

Northwest Steel Pipe Pile

Location	Directly below Jacket	1m Below	Midway	1m above Mudline	Mudline
North	5.55	6.90	8.55	8.85	8.45
East	9.45	10.45	6.95	8.75	9.45
South	7.45	7.50	7.35	8.85	10.85
West	9.10	10.10	10.20	9.55	9.86

Northwest Concrete Pile Jacket

Location of Deficiency	Notes/Area of Concern
North Face	Area of Spalling w/ Exposed Rebar (6'w x 3'h)
East Face	Heavy Spalling/Washout (6.5'w x 3.5'h)

The Four (4) Steel Approach way Piles were found to have 100% Soft Marine Growth Coverage throughout the entirety of the Pile.

Piles are Denoted as NW, NE, SW, and SE. The Diver inspected Both the Concrete Pile Jackets in the Tidal Zone, as well as the Exposed Pipe Piles running to the Seabed.

The Results of the Visual Inspections, Ultrasonic Thickness (UT) readings, and their locations are denoted on the tables below.

Northeast Steel Pipe Pile

Location	Directly below Jacket	1m Below	Midway	1m above Mudline	Mudline
North	10.25	8.55	9.05	n/a	10.85
East	10.80	9.90	11.60	n/a	10.65
South	10.90	10.25	10.65	n/a	10.35
West	8.10	10.55	9.55	n/a	11.20

Northeast Concrete Pile Jacket

Location of Deficiency	Notes/Area of Concern
East Face	1" Wide Vertical Crack throughout Jacket Height

Southeast Steel Pipe Pile

Location	Directly below Jacket	1m Below	Midway	1m above Mudline	Mudline
North	12.85	12.80	12.45	n/a	10.80
East	12.85	12.75	12.80	n/a	8.75
South	12.65	10.25	9.15	n/a	10.20
West	10.50	11.30	12.45	n/a	8.20

Southeast Concrete Pile Jacket

Location of Deficiency	Notes/Area of Concern
West Face	Concrete Loss at Base of Jacket (9'w x 1'h)
South Face	Light Area of Spalling (6" x 6")
East Face	Area of Spalling w/ Exposed Rebar (6'w x 1'h)
North Face	Area of Spalling w/ Exposed Rebar (2.5'w x 3'h) Concrete Loss at Base of Jacket (6'w x 2'h)

REDACTED (CONFIDENTIAL INFORMATION REMOVED)

It was a pleasure working with Mr. Connor Lebans of the Nova Scotia Power Tufts Cove Generating Facility, and we look forward to working with you again in the future.

If you have any further questions, please feel free to contact me at any time,

Cole Scarfe
Connors Diving Services Limited
11-2 Lakeside Park Dr. Lakeside NS Canada
T 902.876.7078 • M 902.219.2815
cole@connorsdiving.com • www.connorsdiving.com

APPENDIX B – Cost Comparison Breakdown

Option 1: Steel structure with timber deck

Item	Total Quantity	Unit	Unit Rate	Cost
Steel girders, floor beams, stringers, and bracing	39.4	t	[redacted]	
Timber deck, chocks, and wheelguard	41.0	m^3		
Bearings	12	Ea.		
Railings/Barriers	110	m		
Rehab of pile caps and piles	1	L.S.		
Removal of existing superstructure	1	L.S.		
Sub-total construction cost estimate				
Contingency [redacted] of sub-total)				
	Total Construction Cost Estimate:			

Option 2: Precast concrete girders and deck

Item	Total Quantity	Unit	Unit Rate	Cost
NEBT 1000 precast concrete girders	165	m	[redacted]	
Reinforced concrete deck and concrete curb	105	m^3		
Bearings	18	Ea.		
Railings	110	m		
Rehabilitation of existing pile caps, piles, and sheet pile	1	L.S.		
Removal of existing superstructure	1	L.S.		
Sub-total construction cost estimate				
Contingency [redacted] of sub-total)				
	Total Construction Cost Estimate:			

CI C0067183 Attachment 2 has been removed due to confidentiality.

REDACTED (CONFIDENTIAL INFORMATION REMOVED)

HARBOURSIDE
Engineering Consultants

219 Waverley Road, Suite 200
Dartmouth, NS, Canada, B2X 2C3
T: +1 902 405 4696

Proposal

Date:	2025-Feb-7	**File No.:**	241188
To:	Connor Lebans, P.Eng.	**From:**	Thomas McNutt P.Eng.
Address:	Tufts Cove Power Generating Station		

Subject: **Tufts Cove Wharf Bridge Rehabilitation Proposal**

Harbourside Engineering Consultants (Harbourside) are pleased to provide the following proposal for the detailed design of the Tuft's Cove Wharf Bridge Rehabilitation for Nova Scotia Power Inc. (NSPI). The existing wharf bridge is a three span structure that provides access from the shore to the berthing wharf that is used to offload fuel for the power generating station. The existing bridge structure consists of steel girders, floorbeams and stringers that support a timber deck. The timber deck is in fair condition, but the steel superstructure shows signs of severe deterioration and is no longer able to support the required loads. Based on evaluations completed during 2024 access to the wharf bridge structure has been restricted and has an impact on NSPI operations.

<u>Scope of work</u>

Harbourside propose to complete the following scope of work for the detailed design:

- Kick off meeting with NSPI to confirm operational requirements for the wharf, modifications from the existing layout and any access restrictions that would impact the construction.
- Detailed design of the wharf superstructure to replace the existing steel and timber wharf bridge.
- Detailed design of repairs to the wharf substructure to address deterioration that was observed during the visual inspections.
- Submittal of 90% design drawings to NSPI for review and comment.
- Regular bi-weekly meetings with NSPI to discuss progress and if there are any issues that arise through the design.
- Consider the construction methodology that will be used during the rehabilitation of the wharf bridge structure.
- Issue stamped and sealed design drawings for the substructure rehabilitation and the superstructure replacement. All notes required for the construction of the wharf bridge will be provided on the design drawings, therefore separate specifications will not be provided.

Assumptions and exclusions:

- Harbourside have not made any allowance for geotechnical investigations. The intent of the design will be to avoid changing the overall loading on the wharf bridge.
- The scope of work includes the review of one round of shop drawings and technical support (assumed to be 4 RFI responses). Additional support during construction will be charged on a time and materials basis, using the rates outlined in the Master Services Agreement.
- Harbourside will develop a proposed construction sequence for the rehabilitation of the wharf bridge, but have not allowed for the detailed phasing of the construction. The construction details will remain the responsibility of the contractor.
- NSPI will provide the design loads for the wharf bridge structure, along with any required changes to the design during the initial kick-off meeting.

Schedule

Harbourside will commence the detailed design of the wharf rehabilitation upon receiving the notice to proceed and the load information required to complete the design. The 90% drawings will be issued within 8 weeks of receiving the required information, and then assuming 1 week of review by NSPI and no major comments are provided, the Issued for Construction Drawings will be issued one week after receiving NSPI comments.

Fees and Deliverables

Harbourside propose to provide the detailed design drawings for NSPI review at 90% completion, then incorporate any comments and issue sealed construction drawings. The drawings will include comments required for the wharf bridge rehabilitation such as code requirements, materials, proposed construction sequence, etc.

Harbourside propose to complete this project for the lump sum of [redacted] excluding HST.

We trust the information provided in this proposal is satisfactory to NSPI. If there are any questions, please do not hesitate to contact the undersigned.

Sincerely,

Thomas McNutt

Thomas McNutt, P. Eng., P.E., M.A.Sc.
Senior Structural Engineer | Partner
O 902 405 4696 M 902 452 6378
E tmcnutt@harboursideengineering.ca

Harbourside Engineering Consultants
A Division of the Harbourside Engineering Group