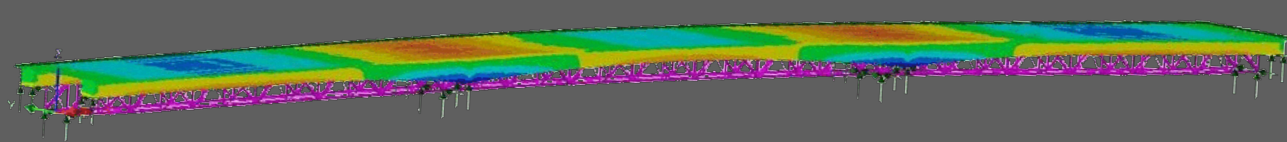
Hillsborough Bridge Structural Review

Final Report



142613.00 ● Final Report ● January 2016

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Prepared for:

Prince Edward Island Department of Transportation and Infrastructure Renewal

Prepared by:

CBCL LIMITED
Consulting Engineers

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Solving today's problems with tomorrow in mind

January 25, 2016

Darrell Evans, P.Eng.
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Charlottetown, PE C1A 7N8

Dear Mr. Evans:

RE: Hillsborough Bridge Structural Review - Final Report

Enclosed is a copy of the Hillsborough Bridge Structural Review for your review and comments. The superstructure has been assessed during our field work and evaluated according to the project scope. The substructure has been included in the condition assessment and is supplemented by the dive inspection of the piers performed by Diversified Divers Incorporated and Harbourside Engineering.

This revision of the report has several more changes. An additional loading scenario, "Case C" (see Section 1.1) has been included. This load case was previously studied and issued as an addenda. Additionally, updated interaction ratios for truss members which incorporate results from the structural testing by Bridge Diagnostics Limited, and some updated discussion related thereto, have also been included.

Please advise if there are comments or questions.

Yours very truly,

CBCL Limited

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Project No: 142613.00

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CHAPTER 1 **INTRODUCTION**

1.1 General

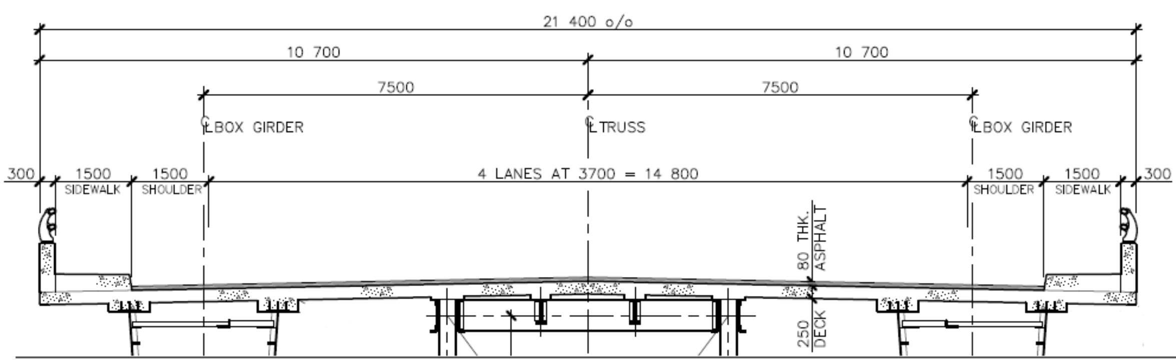
CBCL Limited (CBCL) was engaged by Prince Edward Island Department of Transportation and Infrastructure Renewal (PEITIR) to conduct a structural evaluation of the Hillsborough Bridge.

The principal objectives of the bridge review were to:

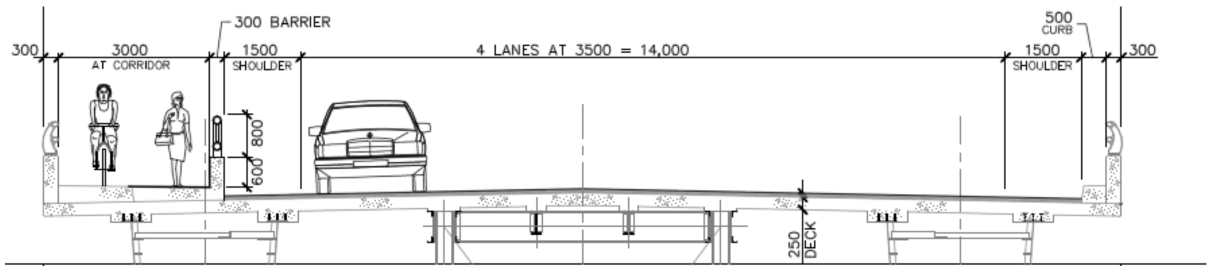
1. Conduct a desktop study of previous engineering reports, drawings, condition surveys, and material test data for the structure. This information includes an evaluation completed by CBCL in 1994, a 1997 steel coupon test report by Geoconn Atlantic, and a 2012 Ontario Structures Inspection Manual (OSIM) Inspection performed for PEITIR.
2. Conduct a condition survey of the bridge to verify criteria required for evaluation.
3. Carry out necessary structural evaluations to determine the structural capacity of the bridge in accordance with Section 14 of CAN/CSA-S6-06, the Canadian Highway Bridge Design Code (CHBDC).
4. Identify members overstressed by both existing and proposed conditions (Case A, B, and C) and provide a load posting, if necessary.

The evaluation considers three conditions, hereafter described as Case A, Case B, or Case C:

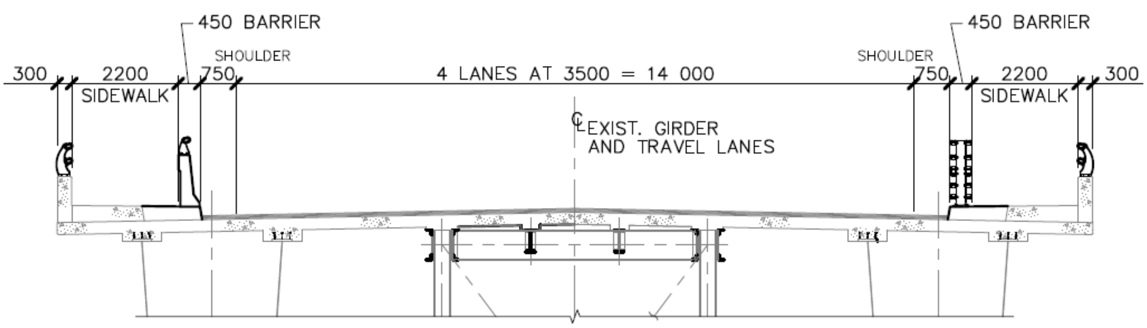
Case A – The bridge in its existing state: Four lanes with sidewalks on each side.



Case B – A proposed condition whereby the sidewalk is removed from the west side such that an active transportation trail is located on the east side and the four traffic lanes will be shifted to the west. Further, a sewerage force main will be supported from lower members of the box truss (not depicted below).



Case C – A proposed condition whereby active transportation trails are added to both sides of the bridge. Two different interior barriers (both shown) were considered for this case, Concrete (F-shaped PL-3 Concrete Parapet), and steel (modified 4 rail standard PL-2 New Brunswick Barrier). A sewerage force main will be supported from lower members of the box truss in this scenario as well.



1.2 History

The first Hillsborough Bridge was constructed circa 1905 (**Photo 1.1**) to replace a seasonal ferry service between Charlottetown and Mutch's Point (present day Stratford). The bridge was assembled of spans previously in service in New Brunswick. Due to increased load demands in NB, the bridge was disassembled and barged to PEI. The total length of the bridge was approximately 770 m between abutments with approach fills of 400 m on the Charlottetown side and 150 m at Mulch's Point. Twelve masonry piers supported spans up to 65 m each. The through truss structure carried a single lane/railway track and had a swing span in the navigation channel to allow ships to pass up the Hillsborough River (**Photo 1.2**).

Photo 1.1: Bridge Site before 1905

As part of the development of the Trans-Canada Highway network, a new bridge opened in 1962 with two travel lanes. The new bridge was a 3-span continuous steel box truss 7.3 m deep and approximately 247 m in length. Two approach spans were located on either side of the truss and were approximately 11.5 m each for a total structure length of 270 m (**Photo 1.3**). The structural steel arched truss is still in service today.

Photo 1.2: Old Hillsborough Bridge 1907

In 1996 the bridge was widened to accommodate four lanes as a part of the "Hillsborough River Bridge Improvement Project" (Bridge Widening). This was achieved by adding two variable depth trapezoidal steel box girders to each side of the existing truss (**Photo 1.4**). The new box girders were supported by widened piers and abutments. At the time of the bridge widening, an evaluation was also performed identifying several truss members to be strengthened during the bridge widening. The widening/rehabilitation also upgraded the highway loading from the PEI "B" Train to the CS600 which was current to the CAN/CSA S6-88, Canadian Highway Bridge Design Code (CHBDC).

Photo 1.3: Hillsborough Bridge before 1996 Upgrades

1.3 Scope of Work

1.3.1 Desktop Study

The primary sources of information for the desktop study were:

1. Information filed at the PEITIR offices.
2. Project files associated with CBCL's role in the 1996 widening of the Hillsborough Bridge.

Photo 1.4: Erection of Box Girders during Bridge Widening

1.3.2 Field Work/Condition Survey

1.3.2.1 SUPERSTRUCTURE

The scope of superstructure field work included:

1. Visual observations of the structural condition of members and connections;
2. Recording section loss of various members ;
3. A limited dimensional inspection to verify existing information (from record drawings and inspection reports) and establish principal member sizes;
4. A "walk-through" of the Trapezoidal Box to identify obvious variations from the design drawings, as well as any section loss; and
5. A photographic survey to record representative conditions.

1.3.2.2 Substructure

The scope of the substructure field work included:

1. A visual observations of structural condition from the access platform;
2. A photographic survey to record representative conditions; and
3. A dive inspection of the piers performed by Diversified Divers Incorporated and Harbourside Engineering Consultants and provided to CBCL by PEITIR.

Access to the truss was via the access platform supported by the lateral bracing at the bottom chords. “Hands-on” access to the vertical truss members was achieved by walking on the bottom chords while supported by fall arrest harness and lanyards as planned and supervised by safety consultant SEM Partnerships.

Access to the trapezoidal boxes was achieved by ladder through the underside access hatches on each end of the bridge. Inspectors followed confined space entry procedures and were equipped with gas monitors. A rescue plan was developed and supervised by safety consultant SEM Partnerships.

1.3.3 Structural Analysis/Evaluation

The scope of the structural analysis/evaluation work included:

1. Create a finite element model to represent the bridge superstructure for Case A, Case B, and Case C;
2. Develop the loads associated with Case A, Case B, and Case C;
3. Design evaluation criteria to all members in the superstructure;
4. Determine the capacity of each member in Case A, Case B, and Case C in accordance with the 2006 CHBDC; and
5. Provide a summary of overstressed members in Case A, Case B, and Case C.

CHAPTER 2 DESKTOP STUDY

2.1 Existing Documentation

CBCL has been involved in numerous projects involving the Hillsborough Bridge, most notably the Bridge Widening. Records of the bridge design and construction were reviewed at CBCL offices, including an evaluation of the box truss structure that was prepared for the Bridge Widening using a grillage methodology. This evaluation was useful in validating the result of the finite element model developed for this project. Steel coupon testing was also contained with the information in CBCL files and is discussed further in Section 2.3.1.

PEITIR have an extensive collection of documents and correspondence from the Bridge Widening project. Most importantly are documents regarding the GEWI pile installation in the piers. During construction the contractor encountered conditions that required changes to the pile design resulting in a reduction in piles. This information was useful in determining the as-built conditions.

2.2 Maintenance and Inspection History

Documentation regarding the maintenance history on the Hillsborough Bridge was not readily available; however, CBCL understands that the bridge had a fairly regular painting program until the late 1980's. This is exhibited by the very good condition of most of the original truss members. At the time of the Bridge Widening new truss plates and reinforcing members appear to have been painted; however, no documentation was discovered through the desktop study to confirm this. Documented maintenance/repairs are outlined below:

- 1962 – Construction of new truss bridge;
- 1978 – Bearing replacement; and
- 1996 – Hillsborough River Bridge Improvement Project widening the bridge by adding two variable depth box girders and strengthening the existing truss.

In 2008 PEITIR initiated a biennial inspection program based on the Ontario Structures Inspection Manual (OSIM) procedures. It is believed that an OSIM based inspection has been performed every two years since, with the latest being carried out in 2012.

2.3 Materials

2.3.1 Steel

Around the time of the bridge widening, 14 steel coupons were taken from the box truss and submitted to Geocon Atlantic (now SNC Lavalin). The sample results are summarized in **Table 2.1**.

Table 2.1: Samples Submitted to Geocon Atlantic for Evaluation Prior to the Bridge Widening

Sample #	Member Type	Element	Yield Strength, KSI [MPa]	Equivalent Carbon Content, CE (%)
1	Diagonal	Cover Plate	45.11 [304.1]	0.271
2	"	"	44.63 [307.7]	0.271
3	"	"	40.70 [280.6]	0.266
4	"	"	41.86 [288.6]	0.265
5	Vertical Post	WF	50.13 [345.7]	0.438
6	"	"	49.39 [340.5]	0.470
15	"	"	50.19 [346.1]	0.470
16	"	"	48.32 [333.2]	0.414
7	Bottom Chord	Channel	41.52 [286.3]	0.422
8	"	"	37.80 [260.6]	0.335
12	"	"	39.99 [275.7]	0.356
13	"	“"	37.72 [260.1]	0.325
9	Diagonal	Cover Plate	41.16 [283.8]	0.263
10	"	"	42.01 [289.7]	0.262
11	"	"	40.08 [276.4]	0.264
14	"	"	41.04 [283.0]	0.265

The general notes on Drawing S1, “Fieldwork to Existing Jacking Girders at Piers 1 & 4” dated January 23, 1970 state that original steel is ASTM A7 with an Fy = 33ksi (227 MPa). It is unclear as to whether this applies to all members or simply the gusset plates shown on that particular drawing.

In accordance with CHBDC Section 14, in lieu of original construction documents, strength of materials not showing signs of deterioration are determined based on test samples, date of construction or an approved method. As such, steel strengths were assumed based on the following hierarchy:

1. Results based on coupon specimen “equivalent” yield strength evaluated in accordance with Section A14.1.1 of the CHBDC. This applies to the channels, cover plates and WF sections whose strengths are summarized in **Table 2.2**. Note that ultimate strengths for these members were not tested and were assigned based on the construction documents.
2. Steel Grade based on construction documents:
    a. All sections of original construction (other than those described above) will be assumed to be ASTM A7 steel (Fy = 230MPa, Fu = 410MPa), from the aforementioned Drawing S1.
    b. Truss reinforcing shall be CAN/CSA – G40.21 M92, Grade 300W based on Drawing 25 of the Hillsborough River Bridge Improvement Project, “Existing Truss Reinforcing Elevation and Details”.

c. Box girders shall be comprised of G40.21M – 350AT Category 1 based on Drawing 13 of the Hillsborough River Bridge Improvement Project, "Box Girder Plan Layouts".

Table 2.2: Summary of Calculated Yield Strengths in Accordance with A14.1.1 of the CHBDC

Element	Mean	Standard Deviation	Coefficient of Variation	K_s, From Table A14.1.1 [# of samples]	Yield Strength (MPa)
WF Shapes	341.4	6.0	.018	2.34 [4]	297
Cover Plates	289.2	11.2	.039	1.45 [8]	243
Channels	270.7	12.7	.047	2.34 [4]	211

As the truss members were built up sections, effective yield strength was determined based on a weighted average of the member's composition. This also accounts for the new plates and threadbars used to reinforce the truss during the bridge widening. Yield strengths of the members are presented in **Appendix B**.

2.3.2 Concrete Deck

The concrete deck was completely replaced during the Bridge Widening. From the construction drawings a 28 day compressive concrete strength of 45MPa is utilized although cylinder tests reported values of 55MPa. Steel reinforcing is as detailed on the drawings and has a yield strength of 400MPa. The unit weight of reinforced concrete as per **Table 3.3** of the CHBDC is 24 kN/m^3.

2.3.3 Ashpalt Surfacing

The thickness of asphalt was not measured during the condition assessment but a nominal thickness of 90 mm was assumed based on Clause 14.8.2.1 of the CHBDC. A unit weight of 23.5 kN/m^3 was applied based on **Table 3.3** of the CHBDC.

2.4 Record Information

Record drawings for the Hillsborough River Bridge Improvement Project were not developed following construction and were prepared concurrently with this evaluation. Sketches and supporting documentation of discrepancies between the construction drawings and the as found conditions are included in **Appendix A**.

2.4.1 Superstructure

Discrepancies between the construction drawings and the as found condition are summarized below but are not considered to be critical to the superstructure evaluation.

- Cast in place deck over box girder was replaced with precast panels;
- Temporary tie beams at diaphragms over abutment and pier were left in place; and
- Struts were added to inside of box girders.

2.4.2 Substructure

The bridge substructure involved four piers sequentially numbered from north (Charlottetown side) to south (Stratford side). Discrepancies between the construction drawings and the as found conditions are summarized below:

- Slight dimensional changes to Pier #2;
- Additional reinforcing and revised stress pockets to Pier #3;
- Revisions to the abutment base; and
- Modifications to the Gewi pile system.

The modifications to the Gewi pile system is the only discrepancy considered relevant from an evaluation perspective. During drilling of the pile shafts on Pier #2, large inflows of water at most shaft locations was experienced. A two-stage grouting process was undertaken to ensure the water did not adversely impact the quality of the grout around the Gewi pile. This entailed drilling and grouting the full length of shaft, re-drilling the shaft, installing the Gewi pile and re-grouting the pile itself. The original construction drawings assumed a single drilling, pile installation and grouting.

Further, at Pier #3 the lack of adequate cement in the existing caisson resulted in the upper regions of a drilled hole breaking away, sloughing down on top of the drill bit. It was concluded that as a result of high sand content that the grout was not able to penetrate the surrounding body of the concrete caisson. It was determined that the piles could only be installed by casing the hole with a heavy walled pipe sleeve. A drill rig capable of installing a casing compatible with a #20 Gewi pile could not be mobilized; therefore a high grade #18 Gewi pile system was substituted.

When substituting the smaller Gewi pile, the depth of embedment was increased to compensate for the decreased drilled shaft diameter into the rock, such that the rock-grout bond strength exceeded the capacity of the Gewi pile. Load tests carried out on selected piles in each pier caisson established a working pile capacity at 800 kN per pile. These pile capacities were higher than what the construction drawings were based on, therefore in the final condition the total number of piles was reduced from 16 to 14 in Pier #2 and 32 to 24 in Pier #3.

CHAPTER 3 **CONDITION ASSESSMENT**

3.1 General

CBCL's condition assessment was performed from June 2 to 6, 2014. The intent of the assessment was to confirm the findings of the 2012 OSIM inspection.

CBCL's findings are in general agreement with the condition assessment report produced by the OSIM bridge inspection for PEITIR in 2012, which is comprehensive, accurate, and valid in its detail.

3.2 Nomenclature

Truss node and member numbers are adopted from the original truss drawings (1959) and are graphically presented in **Figure 3.1**. Numbering of abutment, piers, trusses and members are labelled from north to south with north being the Charlottetown side and south being the Stratford side. Further, in keeping with the convention of the OSIM inspection, references to left and right are to be interpreted while standing with one's back to the Charlottetown Abutment (i.e., left = east, right = west).

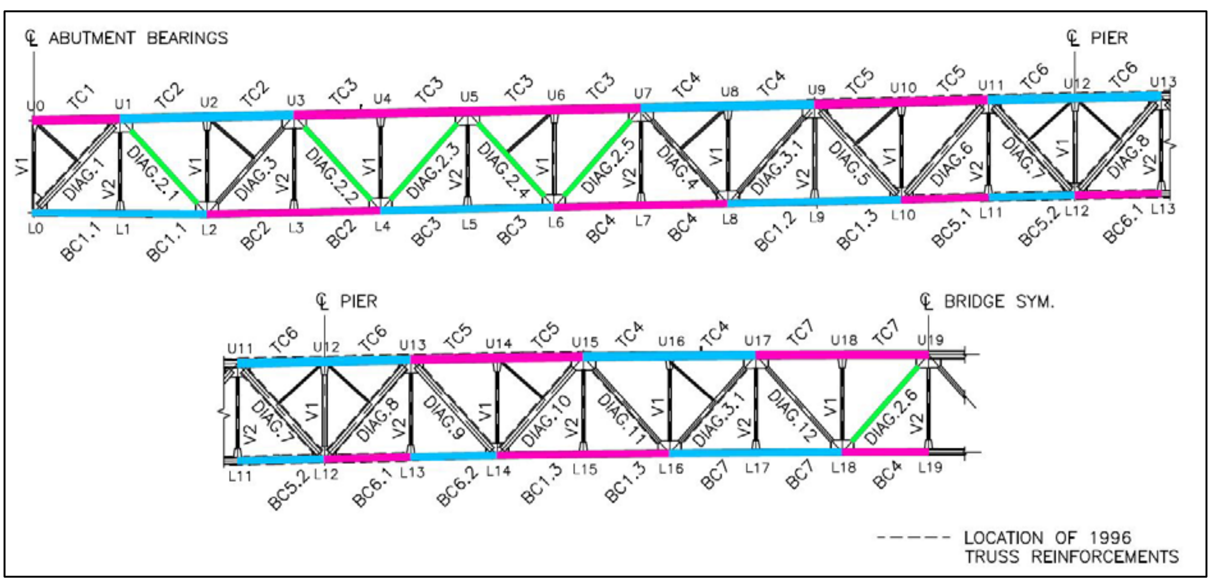


Figure 3.1: Member Groups for the Existing Truss

3.3 Superstructure

3.3.1 Truss System

In general, for the age of the structure, the truss is in very good condition. There are areas of localized corrosion that are outlined below.

Photo 3.1: Typical Corrosion at Channel/Web Plate Edge

3.3.1.1 BOTTOM CHORDS

The bottom chords are typically 18” deep back to back channels. The channels are built up in several different configurations and are presented in **Appendix B**. Further, several bottom chords are reinforced by cover plates as well as the addition of pre-stressed Gewi-bars.

Locations with web plates typically exhibited corrosion at the exposed interface of the channel webs and web plates (**Photo 3.1**). This amount of section loss is not considered to be critical in determining the member capacity.

Photo 3.2: Severe Corrosion of Bottom Chords from L39-L37 Right

The top flange from L39 to L37 (**Photo 3.2**) on the right truss was severely corroded and was found to have approximately 30% section loss on the top flanges and approximately 15% section loss in the webs. This section loss will be considered in the resistance calculations. At the same location the connection of the post tensioned ducts (L37) appeared to have corrosion in the welded connection (**Photo 3.3**). Access was limited to this connection, but a 15% loss in capacity will be assumed.

Approximately 5% section loss was observed in the bottom chord at the bottom flange and web of L2-L3 left (**Photo 3.4**). This is a localized area and will be considered if deemed critical during the evaluation.

Photo 3.3: Corroded Anchorage of Post Tensioned Reinforcing at L37 Right.

Photo 3.4: Corrosion of the Exterior Bottom Flange at Bottom Chord from L2-L3 Left

3.3.1.2 VERTICALS

The verticals were typically found to be the original members (14” deep wide flange columns). Generally the verticals were found to be good condition. It should be noted that a “hands on” inspection was only performed on the lower two metres of the verticals.

Vertical U16-L16 left exhibited approximately 5-10% section loss on the web and flanges (**Photo 3.5**) as did U8-L8 right. Other minor areas of corrosion were noted at U8-L8 left, U2-L2 left and U14-L14 right.

A steel coupon appears to have been taken from U37-L37 right and the patch is exhibiting lighting corrosion less than 5% (**Photo 3.6**).

Photo 3.5: Corrosion of Webs and Flanges of Vertical at U16-L16 Left

Photo 3.6: Coupon Taken from Vertical at U37-L37 Right

3.3.1.3 DIAGONALS

There are a number of different types of diagonals comprising wide flange shapes and back to back channels orientated vertically as well as horizontally. The back to back channel sections were also stabilized through a number of combinations of perforated plates, battens and lacing. Further, several reinforcements of the diagonals took place as part of the Bridge Widening. It should be noted that a “hands on” inspection was only performed on the lower 2 m of the diagonals.

In general light corrosion was found throughout. The coatings would typically be in poor condition, specifically on the new cover plates, where the patina of the weathering steel is preventing durability of the coating (**Photo 3.7**). The appearance of the member is much worse than the actual section loss.

New intermediate diagonals were added at several locations. From a distance coating flaking was observed and a section loss of 5% was assumed (**Photo 3.7**).

Photo 3.7: Typical Flaking of Coating on Cover Plates

Diagonal U13-U14 left exhibited the worst corrosion at approximately 5% (see **Photo 3.8**).

Photo 3.8: Corrosion on Diagonal U13-L14 Left

3.3.1.4 TOP CHORDS

Typically the top chords are comprised of four angles combined with web plates, top plates and bottom perforated plates. The top chord was also reinforced during the Bridge Widening.

From the truss catwalk a limited inspection of the top chords was performed (**Photo 3.9**). Light corrosion was observed throughout but there were no areas of section loss of note to be considered in the evaluation. This correlates well with the 2012 OSIM inspection.

Photo 3.9: Limited View of Top Chord from Access Platform

3.3.1.5 FLOOR BEAMS AND STRINGERS

From the truss catwalk a limited inspection of the floor beams and stringers was performed. Light corrosion was observed throughout but there were no areas of section loss of note to be considered in the evaluation.

3.3.1.6 LATERAL BRACING SYSTEM

The lateral bracing systems were observed to match the conditions reported in the 2012 OSIM Inspection. Detailed measurements of section loss were not collected as it was deemed not critical the live load evaluation. This is discussed further in Chapter 4.

3.3.1.7 DECK

From inside the box girders transverse cracking was noticed in the deck soffit throughout. There were many locations where the cracks had been injected with sealant. It is understood that this remediation occurred immediately after the deck construction during the Bridge Widening project. A detailed inspection of the deck top was not performed as the 2012 OSIM Inspection reported it to be in good to excellent condition.

3.3.2 Box Girder System

The assessment of the steel box girders took place from inside the box. Light to medium corrosion was noticed throughout but was difficult to distinguish from the flaking patina of the AT steel (weathering steel). Areas of medium corrosion were often located near drains. Transverse cracks were noticed in the concrete deck which coincided with some medium corrosion and staining on the steel below (**Photo 3.10**). In general the girders are considered in good to excellent condition with no allowance for section loss being considered in the evaluation.

Photo 3.10: Medium Corrosion and Staining Near Cracks in Deck

3.4 Substructure

3.4.1 Abutments

Each abutment is comprised of reinforced concrete pile cap supported on 30 HP piles. The truss sits on a bearing seat directly on the pile cap while the two steel box girders are supported on reinforced concrete pedestals. The approach embankment is supported by a mechanically stabilized earth (MSE) retaining wall.

In general the abutment appears to be in very good condition with light scaling and cracking throughout.

It was noticed that the MSE wall on the Stratford side (Abutment 2) appears to be bulging outward at mid-height (**Photo 3.11**). CBCL understands that, outside of this study, the original wall manufacturer will be providing an assessment report to PEITIR regarding the movement noted in the abutments.

Photo 3.11: Apparent Bulge of MSE Wall at Stratford Abutment

3.4.2 Piers

From the surface the piers appear in fair to good condition. Somewhat conspicuous are the medium vertical cracks found underneath the box girders in the flared corbel portion of the pier (**Photo 3.12**). While the size of cracks does not indicate a strength issue, durability may be of concern in this heavily reinforced and post tensioned location.

Photo 3.12: Medium Vertical Cracks at Flared Corbel, Typical

Beneath the surface, the piers are founded on steel sheet pile caissons supported by a series of piles as described in Section 2.4.2. The condition of the pier beneath water surface is discussed in the following chapter.

3.4.3 Dive Inspection

A dive inspection was conducted for PEITIR by Diversified Divers Incorporated and Harbourside Engineering Consultants. The report was provided to CBCL by PEITIR and is located in **Appendix D**.

CHAPTER 4 ANALYSIS

4.1 General

A structural analysis of the bridge superstructure was conducted in order to determine the existing capacity of the structure (Case A). Additionally, the structural impact of modifying the structure use through the addition of an active transportation trail and sewerage forcemain (Case B and Case C), as discussed in Section 1.1 of this report, was investigated.

The analysis was conducted using the finite element analysis software package LUSAS Bridge, and accounts for the original trusses, the retrofitted adjacent box girders, and the intricate interactions between them. The state of computer modeling technology has advanced significantly over the preceding 20 years since the original structure was designed, and it was felt that the complex nature of this bridge benefits significantly from this more sophisticated investigation. A screenshot taken from LUSAS showing a cutaway of the Hillsborough Bridge is shown in **Figure 4.1**.

Primary truss members were evaluated at the ultimate limit state, including the bottom and top chords, diagonals and verticals. Local checks were performed on the floor beams, stringers, and bottom chord bracing. The box girders were evaluated at the ultimate limit state for bending and shear. Interaction ratios were obtained for Case A, Case B and Case C for each of these members.

This analysis has considered dead loads and live loads only. Provisions outlined in Section 14 of CAN/CSA-S6-06 CHBDC were strictly adhered to, providing updated load and resistance factors, among other recommendations. The structure was not assessed for wind loads, as discussed in Section 4.3.3. The effects of ice accretion in conjunction with maximum traffic loads were also not considered.

4.2 Analysis Methods

The structural analysis performed for this project, using beam and shell elements, was significantly more sophisticated than the original analysis, using a grillage approximation. Such a beam and shell model would not have been possible 20 years ago due to limitations in both the modelling user interface and computing power. The model was linear elastic, which was felt to be more than adequate for this structure. Three dimensional beam and shell elements with six degrees of freedom per node were utilized. Both beam and shell elements accounted for shear deformations. A summary of element types used is described below:

- Truss members – beams elements;
- Trapezoidal box girder (web plates, bottom flange, diaphragm) – Shell elements;
- Trapezoidal box girder (top flanges, bracing, T-stiffeners, web stiffeners) – Beam Elements; and
- Reinforced concrete deck – Shell elements.

In general, the concrete deck acts compositely with the truss and box girder through the presence of shear studs. However, notably over the six bays at each pier over the truss, shear connectors were not utilized between the slab and truss. To correctly model this, the nodal connection between the slab and truss was "broken" and replaced with 5 mm tall "beams" only capable of transferring vertical forces. This provided a more accurate load distribution near the piers. Additionally, the stiffness of the reinforced concrete deck elements in the negative moment zones over the piers was reduced such that it represented the rebar alone. This was done to correctly portray the cracked section properties.

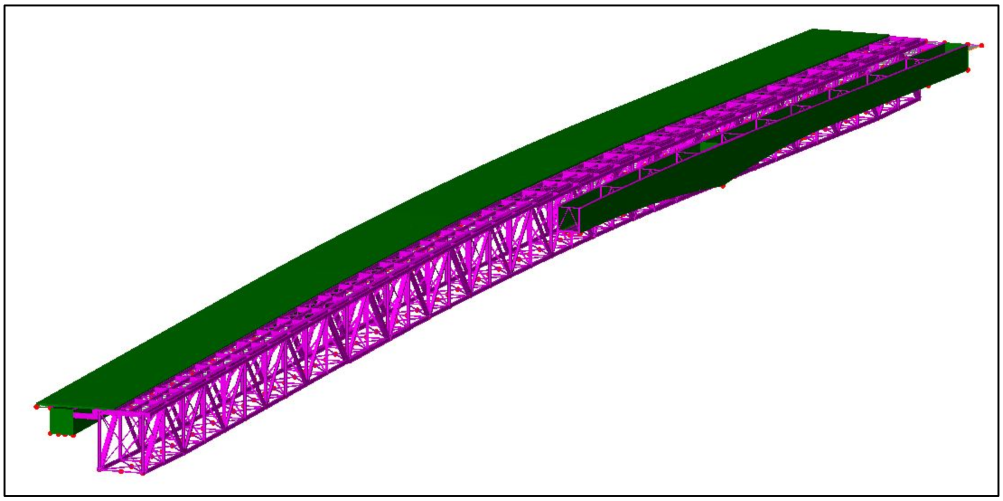


Figure 4.1: Cutaway of Hillsborough Bridge LUSAS Finite Element Model

As discussed in the CAN/CSA-S6-06 CHBDC commentary, at bridge spans exceeding 75 meters loads are dominated not by single heavy trucks, but by lanes of traffic consisting of a variety of vehicles. Thus for this bridge, the live load case was governed by lane loads, with an additional load provided by 80% of the CL-625 design truck. Lane loads were place in the spans which maximized load effects for individual members. **Table 3.4** of CAN/CSA-S6-06 CHBDC dictates that the structure should be designed for a five design lanes. However, Section 14 allows for the structure to be evaluated for the actual number of driving lanes, thus four lanes were used for this evaluation.

4.2.1 Trusses

As previously discussed, truss elements were represented using 3D "thick" beam elements with six degrees of freedom per node. This allowed them to accurately portray axial forces, weak and strong axis shear, in and out of plane bending, as well as torsion. All truss elements were represented in the model, including the primary truss elements evaluated in the report, as well as all top chord bracing, cross bracing, and bottom chord bracing. These members are important for the model to resist torsional forces due to eccentric loading.

Most of the primary truss elements were constructed using built up members consisting of cut plates, perforated plates, batons, channels and angles. The cross section of each unique built up member was drawn using AutoCAD and imported into LUSAS' "Arbitrary Section Property Calculator" for input into the model. For the properties used in the model, an effort was made to properly represent the gross area (Ag) of built up members by not including elements such as batons. Additional sections were also drawn and used to the determine section properties such as out of plane moment of inertia (Iy), torsion constant (J), and the warping constant (Cw). Similar members were grouped as specified in **Figure 3.1**.

Two models were constructed, a "naked" model, consisting of the truss without the concrete deck, and a model with the partially composite bridge deck. The naked model was loaded with truss dead loads as well as the weight of the wet concrete slab. The partially composite model was loaded with superimposed dead loads, (asphalt surfacing at 90 mm thickness, bridge barriers and sidewalks), and live loads.

4.2.2 Box Girders

The box girder bottom flange, web plates, and diaphragms were modelled using 3-D shell elements capable of representing in-plane membrane axial and shear forces as well as out of plane shear and bending/twisting forces. Top flanges, web stiffeners, T-stiffeners, and bracing were modelled using 3-D beam elements.

As in the truss evaluation, load cases included both dead and live load forces in two different configurations, a "naked girder" model and a model with a fully composite deck. Naked girder loads were comprised of steel self-weight as well as the wet slab. The partially composite model was loaded with superimposed dead loads, (asphalt surfacing at 90 mm thickness, bridge barriers and sidewalks), and live loads.

To determine usable structural forces from beam and shell models, rather than simply stresses, LUSAS contains a unique "slice tool". The tool allows the user to cut through beams and shells for a given load case and integrates the stresses over the element areas while automatically accounting for differences in the stiffness of the materials. This allows for the output of three forces (strong axis shear, weak axis shear, and axial) and three bending moments (strong axis bending, weak axis bending, and torsion). A screenshot taken from LUSAS showing the slices considered for this evaluation is shown in **Figure 4.2**.

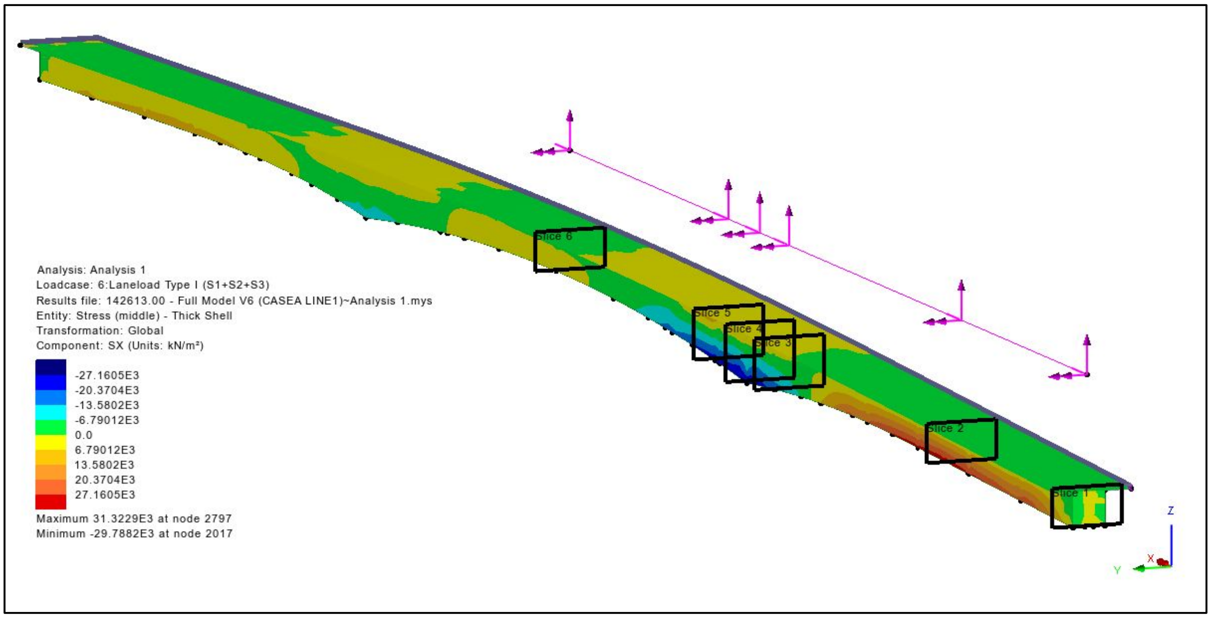


Figure 4.2: Screenshot of Slice Tool in LUSAS

4.2.3 Deck System

Over the box girders, the deck was considered longitudinally through the box girder resistance. Transverse moments and shears were considered not critical and, as such, were not included in the resistance checks. This decision was based on design experience with 250 mm thick deck slabs with comparable transverse spans and similar reinforcing.

A separate model was used to determine the forces acting on the floor beams and stringers. Lane loads were not critical for these members as their spans were much shorter than the total truss spans. Therefore, the CL-625 design truck, including dynamic load allowance, was used instead.

4.3 Loadings

4.3.1 Dead Loads

For the truss, dead loads were comprised of the original members as shown on the As-Built Drawings as well the truss reinforcements that occurred during the Bridge Widening. The reinforcements, as detailed on the drawings, were confirmed in the field.

Due to the large number of built up members, rivets, and connection/gusset plates, determination of an accurate steel self-weight (which accounts for a significant portion of the total truss load) was very difficult. Ultimately, “gravity” loads were applied to members, applied by LUSAS directly according to the cross sectional area of members and density of materials, which were scaled up to the match total bridge weight as ascertained from our desktop study.

Member self-weight for the box girders were much easier to calculate in comparison to the trusses, and were calculated through LUSAS' gravity loads and some manual calculations. The results compared favorably with previous estimates prepared by CBCL during the initial bridge widening.

4.3.2 Live Loads

As discussed in Section 1.1, three cases were investigated. Case A considers the current existing bridge condition. Case B and Case C each considered the proposed rearrangement of the deck to accommodate active transportation corridor(s) while reconfiguring placement of traffic lanes. These cases also include the addition of a sewerage force main. In an effort to reduce computational effort several assumptions were made:

1. The most critical stresses in the box girders would occur while the live load lanes were placed at the extreme edge of the bridge.
2. For all cases, the most critical stresses in the truss would occur by placing the truck within the specified lanes pushed towards one of the trusses.
3. For spans of this length, the critical live load scenario is generally governed by the alternative loading of Clause 14.9.1.6 of the CHBDC. This utilizes 80% of the evaluation vehicle (with no dynamic load allowance) and a superimposed lane load. An 8 kN/m lane load is associated with a Class B highway and is what was used for the evaluation.
4. Effects of wheels on the sidewalk (Clause 3.8.4.4) is not considered in the longitudinal effects of the box girders. Sidewalks loading was also not considered, as it was unlikely to occur coincident with maximum traffic loading (Clause 14.9.5.1).

The highway class is determined in accordance with Table 1.1 of CAN/CSA-S6-06 CHBDC. According to the 2013 bridge traffic counts, the AADT for the bridge is 32,609 vehicles with 9.45 percent trucks. This equates to average daily traffic (ADT) per lane in the order of 8,152 and average daily truck traffic (ADTT) in the order of 770. Based on ADT, the Hillsborough Bridge would be considered a Class A highway as ADT exceeds 4000. However, the truck traffic on the bridge falls into the range of 250-1000 vehicles per lane or a Class B Highway. Therefore for the purpose of determining truck loading frequency and intensity, the Class B highway loading of an 8 kN/m uniformly distributed lane load (UDLL), was utilized for this evaluation.

4.3.3 Wind Loads

There are no anecdotal records of the superstructure showing signs of distress during extreme wind events. Further, the addition of the box girders provides shielding and additional stiffness to the interior truss members that was not accounted for in the original design. Wind load is therefore not considered to govern the live load capacity of the bridge.

4.4 Fatigue

Fatigue evaluation of existing steel bridges that have outlived their design life is a difficult subject.

Miner's concept is generally accepted as one the best methods for evaluating cumulative fatigue damage under repeated loads. Fatigue evaluation can be conducted through the use of traffic counts and applying Miner's concept to the theoretically calculated stress ranges. In many cases this approach shows bridges to have exhausted their useful fatigue life.

To begin, it would be difficult to plot a detailed stress history for this structure since it has already given 50+ years of service. Although current traffic volumes data exists, plotting accurate stress ranges over the entire life span of the structure would be speculative at best.

The majority of connections on this structure are bolted or pinned, which generally perform much better than welded connections in terms of fatigue life. It should also be considered that there are no signs of cracking following an OSIM inspection in 2012 and the condition assessment associated with this evaluation.

Research and field testing results suggest that in many cases, when a theoretical approach predicts only a few years of remaining fatigue life, the structure or detail/connection may not display fatigue cracking until many years after initial prediction.

Considering these points, a detailed evaluation to determine the remaining fatigue life of the structure has not been undertaken. Through regular inspection, signs of fatigue crack initiation and propagation should be monitored.

CHAPTER 5 EVALUATION RESULTS

5.1 General

The results of the structural analysis were generally favorable. Most elements of the structure were found to be under their factored design stress, with interaction ratios of individual primary structural elements varying from 13% to 97% for Case A, 18% to 105% for Case B, and 13% to 103% for Case C.

5.2 Reliability Index to Determine Dead and Live Load Factors

Section 14 of the CAN/CSA-S6-06 CHBDC contains provisions for using a more refined probabilistic framework to define load and resistance factors based on a wide variety of factors including past bridge performance, the condition of members, the mode of failure, and the importance of the member in the overall behaviour of the structure. This is accomplished using the target reliability index, β. In general for a new structure, the target reliability is generally the same for all members. During a bridge evaluation, the engineer is able to set different values for each element. The target reliability index is determined by assigning a value, 1 – 3, for each of the following categories:

- System Behavior (S1-S3) – The effect of the element's failure on the entire structure;
- Element Behavior (E1-E3) – Consideration for the element's ductility; and
- Inspection Level (INSP1-INSP3) – Consideration of the level of inspection on the element.

Once the target reliability has been determined it is used to compute refined dead and live load factors, α_D and α_L. The target reliability is also used to modify the existing code material resistance factors, ϕ. A list of structure elements and their associated factors are presented with the evaluation results in **Appendix C**.

5.3 Truss

Classical truss theory states that all bolted connections are rotationally free and loads are carried exclusively through axial member forces; tension and compression. While predominantly true, trusses which span continuously over supports such as the Hillsborough Bridge are often subject to significant bending moments, particularly in the bottom chord at the piers. Bending moments are also present in top chords due to bending under dead and live loads spanning between the panel points. As such, the following scenarios were evaluated for each member:

1. Maximum compression with associated moment;

2. Maximum tension with associated moment; and
3. Maximum moment with associated axial force.

These scenarios were investigated for Case A (existing) and Case B (AT trail one side only), and Case C (AT trail both sides, concrete and steel barriers) and are summarized in **Table 5.1** and graphically displayed in **Figure 5.1**. Members with interactions at or exceeding their capacity (great than or equal to 1) have been highlighted.

Table 5.1: Truss Member Capacity Utilization for Case A, Case B and Case C

Member	Interaction (% of Capacity)			
	Case A	Case B	Case C	
			Concrete Bar.	Steel Bar.
BC1	0.9	0.94	0.89	0.89
BC2	0.79	0.79	0.83	0.81
BC3	0.79	0.80	0.83	0.81
BC4	0.74	0.74	0.73	0.73
BC5.1	0.84	0.87	0.89	0.87
BC5.2	0.62	0.64	0.65	0.64
BC6.1	0.64	0.66	0.67	0.66
BC6.2	0.97	1.01	1.03	1.01
BC7	0.72	0.72	0.72	0.71
TC1	0.22	0.22	0.22	0.22
TC2	0.61	0.67	0.65	0.64
TC3	0.61	0.66	0.62	0.62
TC4	0.64	0.67	0.64	0.64
TC5	0.61	0.61	0.61	0.61
TC6	0.68	0.69	0.68	0.68
TC7	0.46	0.50	0.48	0.47
Diag1	0.93	1.05	0.98	0.97
Diag2	0.92	0.97	0.97	0.95
Diag3	0.79	0.93	0.84	0.82
Diag3.1	0.75	0.78	0.73	0.73
Diag4	0.7	0.79	0.76	0.74
Diag5	0.73	0.81	0.78	0.76
Diag6	0.95	0.95	0.94	0.93
Diag7	0.7	0.75	0.73	0.72
Diag8	0.66	0.81	0.68	0.67
Diag9	0.9	0.92	0.89	0.89
Diag10	0.62	0.68	0.65	0.64
Diag11	0.85	0.89	0.87	0.85
Diag12	0.7	0.71	0.74	0.73
Vert1	0.64	0.74	0.68	0.67
Vert2	0.13	0.16	0.13	0.13
Vert3	0.58	0.67	0.60	0.60

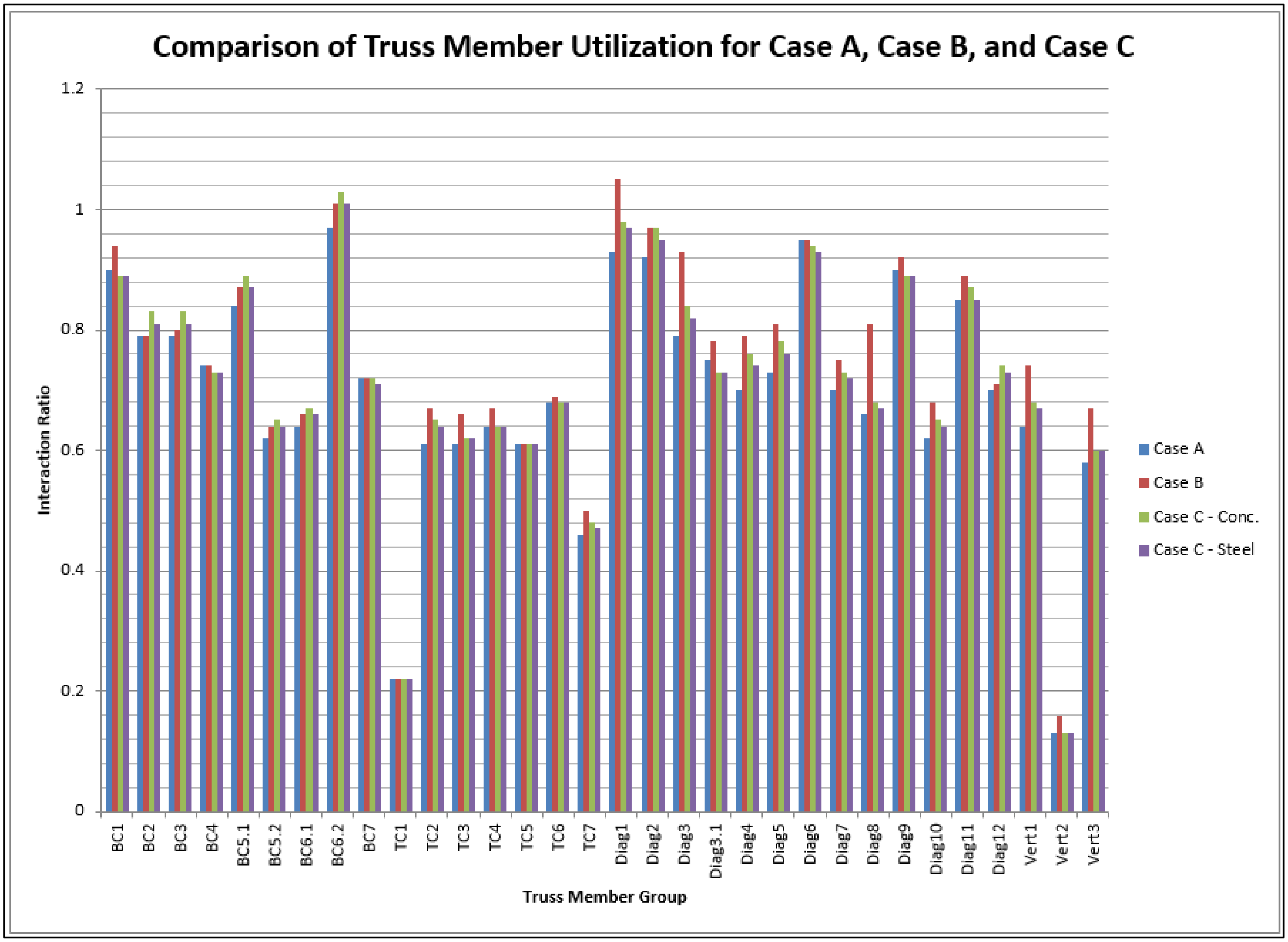


Figure 5.1: A Comparison of Truss Member Interactions between Case A, Case B, and Case C

Member resistances were generally calculated at the location of least section for a group of members with a uniform cross section. The location of least section is based on section loss observed during the condition assessment.

5.4 Box Girder

The ULS bending moment and shear capacity of the steel box girders was evaluated at various critical sections and compared to the results obtained from the LUSAS model. Results are summarized for Case A and B in **Table 5.2**. Following is a list of critical sections that were evaluated and rationale behind the determination:

1. At the abutment, considered to be the highest shear on the end span;
2. At the middle of the end span, considered to be the highest positive moment on the end span;
3. At field splice #3, using the thinner of adjacent sections, considered to be a critical combination of negative moment and shear on the end span;
4. Over the pier, considered to be the highest negative moment;

5. At field splice #4, using the thinner of adjacent sections, considered to be a critical combination of negative moment and shear in the center span; and
6. At the middle of the center span, considered to be the highest positive moment in the center span.

Table 5.2: Utilization of Box Girder Capacity for Case A and Case B

Station (m)	Description	Interaction (% of Capacity)		% Increase
		Case A	Case B	
0	At Abutment	0.43	0.47	4%
30	Max. Moment, End Span	0.27	0.29	2%
71.2	Neg. Moment, End Span – Splice 3	0.29	0.31	2%
78.2	Neg. Moment at Pier	0.26	0.31	5%
85.7	Neg. Moment Mid Span – Splice 4	0.25	0.30	5%
123.6	Max. Moment Mid Span	0.17	0.19	2%

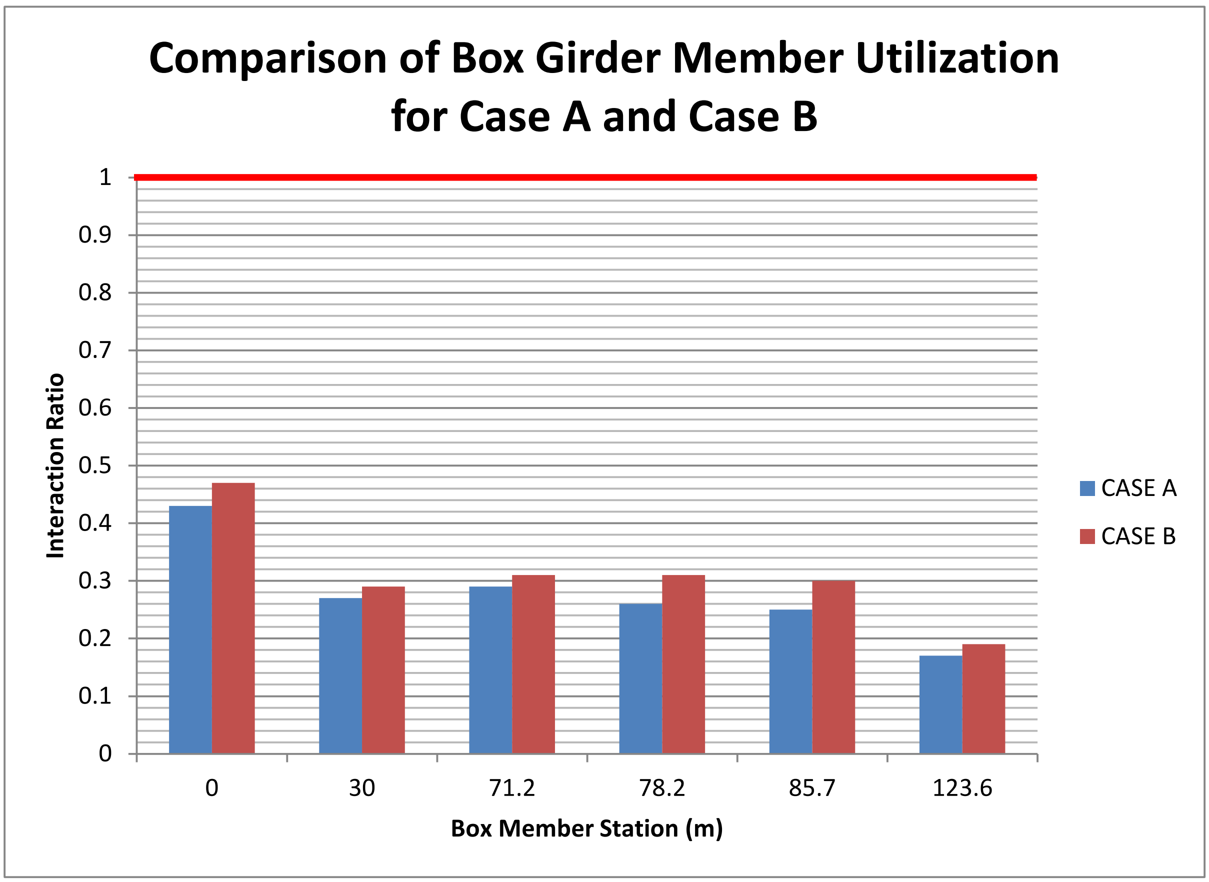


Figure 5.2: A Comparison of Box Girder Interactions

5.5 Local Checks

Local strength checks were performed on the following members:

- Floor beams and stringers, (under ULS truck loads with dynamic load allowances);
- Cantilever deck overhang (under Case B); and
- Bottom chord truss bracing, (Case A and Case B).

The floor beams and stringers were not adequately evaluated in the global model due to the lane loads not being applicable for the small spans. A local model was constructed and these members were loaded using the CL-625 design truck without lane loads, and with appropriate dynamic load allowances. The results indicated that both of these members have adequate capacity. It seems likely that the original designers of the bridge, having no access to computer analysis software, used more traditional rational analysis methods to design these members. It was found that the forces acting on these members were much lower than capacity, and generally the loads were transferred to the truss panel points and primary top chords.

The cantilevered deck overhang from the box girder was deemed important to evaluate particularly for Case B, as the truck line was able to move directly to the edge of the bridge where previously there was a sidewalk. The distance of the overhang was modest, at 1750 mm, and was more than adequately reinforced. The capacity of this element was adequate.

Bottom chord truss bracing was comprised of two angles separated by baton plates, forming essentially a shallow 450 mm deep truss. The structural system was evaluated as a truss, with the 89 x 76 x 7.6 angles forming the top and bottom chords and batons forming the diagonals. Case A involved the self-weight of the truss elements as well as the maintenance walkway, and a small live load for maintenance workers. Case B was evaluated using all of these loads, plus the addition of the 600 mm diameter sewerage forcemain located at mid-span directly between the primary trusses. The braces were found to be adequate under Case A loads. For Case B, the load on these elements was increased by nearly five times, and the bottom chord bracing failed in bending.

5.6 Substructure

The report from the diving inspection indicated the piers and abutments are in generally good condition (refer to **Appendix D** for details); however, there were concrete delaminations and spalls identified on the top surface of the concrete base (inspection report refers to the concrete base as the transfer cap). The concrete base was part of the bridge widening and is anchored with several Gewi piles. Local punching shear design checks were performed for the Gewi piles and the results indicate the concrete strength has adequate capacity when compared to the factored design load of the Gewi pile.

CHAPTER 6 CONCLUSIONS AND RECOMMENDATIONS

6.1 Evaluation Conclusions

The deep sections of the box girders were designed to match the stiffness of the truss to reduce differential displacement under live loading. As a result the sections have considerable reserve capacity. Although the new bridge configurations are generally more detrimental to the box girders, the interaction ratio is only increased by 2-5% overall. Ultimately the box girders are structurally adequate, well detailed and are not a cause for concern regardless of the bridge configuration.

The existing condition of the truss finds no members overstressed, however with interaction ratios at 97%, they are very close. As it was found that section loss played very little role in the governing interactions, this overstress can mainly be attributed to two things:

1. Increased truck loading from CS-600 (S6-88) to a CL625 (S6-00); and
2. Decreased yield strength based on the evaluation of coupon tests in accordance with Section 14 of S6-06.

Case B and Case C were found to be more detrimental to the truss elements than the boxes, although the response varied depending on the member being examined. Generally, Case C, utilizing a steel internal barrier, exhibited the lowest increase in load demand of the proposed options.

6.2 Recommendations

Under existing conditions the truss is nearly at the capacity, especially for bottom chord members at the pier locations and some diagonals. In service the truss is showing no signs of distress and, as such, it is the opinion of CBCL that the bridge does not require a load posting under Case A. The truss is in remarkably good condition considering its age. This can be attributed to a regular maintenance program early in its service life. It is recommended that a regular maintenance program (i.e. painting) be reinstated to preserve the condition of the structure.

If it is decided to proceed with implementing an active transportation trail and supporting a force main with the truss, it is recommended that:

1. The force main be suspended as close as possible to the bottom chord. The condition of the bottom lateral bracing is generally in poor condition and is required to transfer torsion from eccentric live

loading as well as resist lateral loads. To support the force main from these members would require a more detailed analysis.

2. If the installation of active transportation trails is pursued, we recommend Case C with steel barriers to reduce the overall load demand on the truss. Strengthening should be considered for those members at or near capacity.

Prepared by:	Reviewed by:
Luke MacDonald, M.A.Sc., B.Eng., P.Eng. Structural Engineer	Colin Jim, B.Sc., B.Eng., P.Eng. Structural Engineer

APPENDIX A

Supplemental Record Information

HISTORY OF FINAL INSPECTION.

Nov. 7,1997	CBCL inspected Box Girder
April 16, 1998	Site inspection revealed extensive cracking of deck that was brought to SCI'S attention.
April 22, 1998	SCI"S response to cracking.
June 16, 1998	The Bridge was inspected following heavy rain, but before waterproofing was installed.
	Report recorded extensive cracking (after cracks were " sealed ") and water penetration in boxes. See photografic record.
July 10,1998	Detailed visual inspection was carried out jointly with SCI and CBCL.
July 10, 1998	B.Farago's report.
July 16, 1998	SCI'S report.
Dec. 3, 1998	B.Farago's memo contains list of items SCI have not complied with.
Jan. 4, 1999	PEI'S memo to SCI Re: Outstanding issues.

Prince Edward Island
Department of Transportation & Public Works
Highway Maintenance Division
P.O. Box 2000, Charlottetown P.E.I., C1A 7N8

Fax Transmission

To: Phil Lockwood **Destination Fax No. 403-~~244~~ 228-8643**

From: Steve MacLean P.Eng.
Director
Phone: (902) 368-5103 Fax: (902) 368-6244

Date: January 4/99

No. of Pages = 2

Subject: Outstanding Issues/Deliverables Hillsborough Project

Further to our conversation in December, I forward a "punch list" of remaining issues from my perspective.

1) Deficiency List and Corrective Actions

On July 16, we had a definitive meeting on the subject wherein all major and minor deficiencies we recorded.

I would like to have confirmation in writing:
a) Each deficiency
b) The specifics of; date and course of action for each
c) What remains.
All relevant correspondence in your Job Files should be included.

Key items within the list are a) Bearings, b) Deck cracks and leaks to interior of girder c) Retaining wall system.

At the July meeting, CBCL promised a written opinion that the overall extent of deck cracking and the remedial treatment provided by SCI 1) does not limit the laod capacity of the bridge, and 2) does not present a long-term durability concern. This has not been received. I should add that I expect the scope of the opinion letter to include the consequential issue of leaks to the girder interior and the potential for accelerated corrosion and reduced life expectancy.

2) Warranties:

There should be a warranty file wherein all components are listed complete with warranty terms/conditions. Nothing rec'd.

3) Design Information:

The Department expects to receive copies of all design calculations which are relevant as reference in the future when TPW needs to do a) normal maintenance, bearings etc. b) over-weight permitting c) increased highway loading, d) other design checks etc. As you know, Bert F. has given TPW advice in this area.

Nothing has been rec'd yet.

As a minimum, we expect to receive a) detailed DL and LL calcs. b) the Reactions (detailed) at bearings c) Capacity at jacking points d) Max. bending-moment and shear diagrams for SLS and ULS. e) calculated movements at joints and bearings f) calculated capacity of truss members as surveyed and after strengthening together with SLS and ULS forces in members due to DL, LL and other effects. g) I.D. of fatigue critical members of the boxes and truss, giving category and stress level.

4) Inspection Manual:

By agreement, TPW is responsible to the lessor to carry out regular inspections and normal maintenance. We need to agree on what the inspection program should be. It is also a very useful step, and one that we had informal agreement that SCI would supply.

SCM.

S.C.M.

cc: Bert Farago, Farago Associates

B. FARAGO & ASSOCIATES INC.
CONSULTING ENGINEERS

TELEPHONE:(416) 444-5035
FAX: (416) 444-8791

83 Larkfield Drive,
Don Mills, Ontario
M3B 2H5

MEMORANDUM.

To: Steve McLean
Re: Hillsborough Bridge
Date: December 3, 1998
Project:97127

This is to report to you on the work I performed in your office on December 2 and 3, 1998.

1. Review of files submitted by SC1 :
    a.) Checked off the files submitted against the transmittal slip.
    The files are substantially complete except they had a separate file on deficiencies (their terminology is different) which we reviewed in July.
    I found some sheets spread over different files, but would be good to have it in one place.
    b.) I propose to develop an index binder containing a complete list of all files and drawings for future reference.
    c.) Drawing tubes are now numbered 1 to 19. Their content list will be in the binder. (After receiving list from SC1)

    d.) Binders are divided into four boxes (according to importance of the material contained) and numbered.
    Their contents will also be in the index binders.

2. The following material, that was discussed at the meeting on July 16, 1998 with McGinn and Stroltz is still outstanding.

    a.) There is a need for a hand over file:
    This should contain, the deficiency list from July 16 meeting (our prepared by Don and another by myself.
    I also propose to include in this " deficiency " file the photographic records of cracks that you took.

    b.) There is need to record, with dates, what action was taken what was corrected and what is still outstanding.
    c.) This file perhaps should contain the correspondence on key items:

        I.) Bearings.
        ii.) Leakage in boxes due to poor quality construction joints.

MEMORANDUM CONTINUED........
Re: Hillsborough Bridge

iii.) Identifying and recording continuing wet spots (and corrosion) inside boxes.
iv.) Deficiencies in the retaining wall systems.

This might be the right place for storing copies of warranties (none received to date.)

b.) Design information for the Owner.
In earlier correspondence I suggested that it is not unreasonable for an Owner to get copies of all design calculations for future reference.

If CBCL is reluctant to provide this as a minimum requirement I believe, the Owner should have sufficient information for designing future maintenance (such as bearing replacement) and for checking the bridge for over-weight permit application,or future increase in highway loading:

1. Detailed dead and live load calculations. Reactions (detailed) at berings Also capacity of jacking points.
2. Max. bending - moment and shear diagrams for SLS and ULS.
3. Temperature and other movements at joints and bearings.
4. Calculated capacity of every truss member as surveyed and after strengthening ,together with SLS and ULS forces in members due to dead , live and other effects.
5. Identification of fatigue critical members of the boxes and truss , giving category and stress level.

c.) Inspection manual.
If I recall Mr. Strolz promised to put together inspection and maintenance guide for the Owner.This is very important, as the PEIDOT as lessee is responsible to the lessor (HBDI) to carry out regular inspection and maintain the bridge in good condition, during the lease term and " to maintain complete record of the history of the bridge. "

d.) When we discussed the significant number of cracks, many of them not fully grouted, Mr. Strolz promised to provide a written statement that these cracks have not reduced the load capacity of the bridge and it's longterm durability. This you have not received.

Issues to deal with.:
In addition to the many relatively minor deficiency items, listed in a1.) , many of which

MEMORANDUM CONTINUED........
Re: Hillsborough Bridge

no doubt have already been resolved, there are three key issues on which the long-term durability of the bridge depends and therefore require speedy resolutions:

1. Bearing.
   This I understand , is being reviewed by all the parties, involved.
   For reference it would be useful to have a survey done on all bearings including direction of tipping (incl. on replaced bearing) any gap between guide -bar and slot.
2. Cracks and open joints in the precast deck with resulting leakage into the boxgirders.

   This is critical, because

   a.) It results in continuing corrosion and eventual loss of section, in the the box girders.

   b.) It could lead to corrosion of the reinforcing steel and stud shear connectors on which the whole structure depends.

3. Corrosion of the weathering steel.

   It will be necessary to record conditions where moisture can get into the box (cracks in the slab , above , splice -plate locations etc.) to see if further sealing of openings is warranted.

Hillsborough Bridge Improvement Project

Bridge Inspection Report

Recd: Aug 3-98

Strait Crossing Inc.

July 16, 1998

1. Introduction

An inspection of the Hillsborough Bridge was undertaken on July 12, 1998 with representatives of Strait Crossing Inc (Contractor) and CBCL (Engineer) on hand. The purpose of the inspection was to identify deficiencies (work still to be done, work done incorrectly, additional work required) in order to reach total completion of work.

Additionally, the DOT & PW project representative and an external, independent, review engineer contracted by DOT & PW were in attendance to discuss deficiencies and the work in general.

Attendance:

Gerry Strolz -	CBCL
Don McGinn -	SCI
Steve Pletch -	SCI
Steve MacLean-	DOT & PW
Bert Ferago -	Ferago Consultants

2. Marine Piers

An inspection of the piers was performed via boat. The following was noted.

.1 North face of Pier No. 2 has three cracks at water level which are wide enough to be injected with epoxy resin. The cracks originated from the shrinkage of the new pier around the old.

.2 Other shrinkage cracks were observed on both piers but were considered of no consequence.

.3 Previously injected cracks looked good.

.4 Stressing pockets looked tight.

.5 A few rust spots associated with form ties have come through. Although not of structural concern they should be chipped back and repaired.

.6 In general the piers look fine.

.7 Final settlement of piers is to be recorded. (survey)

.8 Re-caulk holes in sides of pier. (These holes are to be recorded on as-built drawings)

.9 Corbel tie beams to be painted.

.10 Some of the vertical post-tensioning pockets need their perimeter sealed with epoxy.

3. Abutment.

An inspection of the abutment area yielded the following improvements to be done.

.1 Remove black tar stain from lower portion of SE abutment. ✓

.2 Repair undercut (concrete washed out) in old (existing)abutment crossbeam. ✓

.3 Remove nails from abutment footings. ✓

.4 Landscape area so water drains away from truss bearing seats and pick up debris. ✓

.5 Remove silt fence. ✓

.6 Fence off access to bridge ✓

.7 Remove legs of old MECL tower ✓

.8 Wash discoloration of side of abutment ✓

.9 Tidy tie hole patching in new abutment. ✓

.10 Clean off all bearing seats. ✓

.11 Chip concrete around footings of North abutment. ✓

.12 Fix undercut in concrete at seat of NE bearing of the truss. ✓

4. **Retaining Walls**

There are two types of retaining structures. Both appear to be working correctly. Settlements have been noted which in accordance with the design.

The following work remains to be completed on the retaining walls:

.1 **RECO Walls**

.1 Clean grout off face. ✓

.2 Resolve discoloration of face of panels. ✓

.3 Repair broken edges. where ?

.4 Construct concrete cap beam. (coping) ✓

.5 Erect rail at top of wall. ✓

.6 Bury all straps minimum of 6" deep. ✓

.7 Remove board stuck between RECO wall and abutment. ✓

.8 Add top soil above RECO wall and seed. ✓

.9 Remove boards behind abutment cross beam (in front of RECO wall). ✓

.2 **Maccaferri Wall**

.1 Cut the grass ✓

.2 Ensure grass is growing in deficient areas ✓

.3 Plant bushes. ✓

.4 Install rail along top.

4. New Steel Box Girder

The new steel box girders were carefully examined from the inside and outside. Performance and overall quality is excellent. The following notes were made:

.1 Outside

Some un-even weathering has occurred generally at locations where water has come down from the deck level. The girders should gradually become uniform in appearance.

The outside faces have not matured as much as the inner and underside.

.1 At Pier No. 2, east girder, outer face there is a whitish patch. This may be grout

.2 Straps at location of splices where ultrasonic testing couplant grease has no weathered. This is not a problem.

.3 Locations of steel girder lift points for erection were to be filled with a bolt with nut on both sides of flange. Some of the nuts have been removed. This is acceptable.

.2 Inside.

.1 Complete clean up of girder floor using a stiff broom (no wire brush) followed by a vacuum to be done.

.2 All drainage holes and fillets at stiffeners etc. are to be checked to allow for drainage.

.3 Conduit holes at girder ends which are not being used are to be screened.

.4 Water may be entering girders at splice points. Accelerated weathering has occurred at some of the joints. They are to be monitored if sealing of the joints should be considered.

.5 Evidence of previous leaking into the girder through the deck is apparent. Deck waterproofing has resolved this problem.

.6 Water infiltration seems to be causing uneven weathering of the steel. There is always a degree of inconsistency in steel and horizontal areas where dust and moisture collects will have accelerated weathering.

CBCL has recommended a representative of CISC who is knowledgeable in quality and inspection view the girder interiors to ensure weathering rates/manner are acceptable.

.7 Electrical boxes at girder entrance/exit have pulled out due to expansion/contraction.

6. Bearings

.1 Box Girder Bearings (new)

Similar problems with two bearings (guidebar) since the last inspection seems to have occurred.

.1 Guide bar is twisted, indicative of broken bolts, at both guided bearings east and west boxes at the south abutment. Bearing supplier (Watson Bowman) is to supply response.

.2 Offsets of bearings to be recorded.

.3 Bearings were recently repainted. The paint specification is to be reviewed.

.2 Truss Bearings (old)

.1 Clean up debris and vacuum dirt from bearing area.

.2 Inspect all bearings for signs of wear and/or improper behavior.

.3 Replace rubber protective sheath.

.4 Record bearing offsets.

7. Steel Truss

Strengthening of the steel truss was completed, mainly with pre-painted steel plates. It was noted that additional weathering of the truss paint system has occurred since the project began.

.1 Box girder ties (bracket) have resulted in some areas requiring touch-up paint. ✓

.2 Replacement pieces in handrail to be painted. ✓

.3 Misc. old fixtures and cable of Island Tel are to be removed. ✓

.4 Miscellaneous tie wire on lower chord cross bracing is to be removed. ✓

.5 Debris (from demolition) in corner areas of lower chord is to be swept out ✓

.6 Handrail over top of diaphragm at pier areas to be painted. Unpainted steps are alright as they are. ✓

.7 Finish fastening railing in P2 area. ✓

.8 Burn marks on underside of transverse beams from attachment of studs need to be touched up. ✓

.9 Rope hanging out of bird screen between abutment and Pier 2 to be removed. ✓

.10 Grout on lower chord (west) between Pier 2 and abutment (bay # 4) to be removed. ✓

.11 Sign warning anchor wires to be removed. ✓

8. Concrete Deck, Barrier, Sidewalk, & Expansion Joints

The concrete deck was already waterproofed and asphalt applied so mainly the barriers and sidewalks were inspected.

.1 Evidence of previous leaking of the deck throughout the winter was apparent.

.2 Leaks were mainly at precast component construction joints, however there were some leaks associated with unplugged tie holes and in the negative moment area some deck cracks. This type of cracking was expected and is not of structural concern.

to be kept under obs.

.3 Water proofing has resolved construction joint and shrinkage crack leaks.

.4 Joints and large cracks in the sidewalk are also leaking into the girders. The cracks are not to be touched. The joints are to be routed and filled with a sealant.

.5 There are some shinkage cracks (approx. six locations) in the east barrier which requires treatment. Epoxy sealant should be tied and the locations monitored.

.6 The east sidewalk cover plates do not fit properly.

Re.

.7 Cracks were noted in the concrete at the expansion joints in areas where transverse strengthening plates are located. This is apparently typical. The cracks have already been epoxied and are of no consequence.

.8 The seal strips are still to be installed and tested.

FAX TRANSMISSION

PEI DEPARTMENT OF TRANSPORTATION & PUBLIC WORKS
HIGHWAY MAINTENANCE DIVISION
P.O. BOX 2000, PARK STREET
CHARLOTTETOWN, PE C1A 7N8
TEL: (902) 368-4750
FAX: (902) 368-6244

TO: Bert Farago **DATE:** August 3, 1998

FAX #: 1-(416)-444-8791 **PAGES: 12 (INCLUDING THIS COVER SHEET)**

FROM: Steve MacLean

SUBJECT: Hillsborough Bridge Project - Inspection Report

COMMENTS:

For your information and review, the attached is a copy of the inspection report.

Call Nick
Re Watson Bowman bearing. W/ Steve
A 574

Call Steve
steel material
Designer: letter re cracks.

B. FARAGO & ASSOCIATES INC.
Consulting Engineers

TEL: (416) 444-5035
FAX: (416) 444-8791

83 Larkfield Drive,
Don Mills, Ontario
M3B 2H5

MEMORANDUM.

TO: STEVE MACLEAN
RE: HILLSBOROUGH BRIDGE
Substantial Completion Inspection
DATE: July 10, 1998

During our detailed inspection of the project the following issues were discussed (most of these, no doubt, will appear in Don McGinn deficiency list):

1. Piers.

   Three cracks were identified on the North face of Pier # 2 from the horizontal construction joint down below the high water. These shall be pressure injected with epoxy.
   Both piers have a number of " shrinkage " cracks above the construction joint and were caused by the restraining effect of the old pier on the freshly placed concrete. Due to the radial prestressing these cracks cannot get wider and have no adverse effect on the structural strength and durability of the piers.

2. Steel boxes- outside surfaces.

   Generally in good condition with weathering well developed except there are a few streakings where water from the deck was flowing down the web. The underside of the boxes is of lighter colour with colour variations due to incomplete weathering.

   Near the south abutment on both, upstream and downstream sides there are two bright narrow stripes, apparently due to radiography.
   A few bright coloured spots were found on downstream side (grout or bird droppings ?). No action required.
   Large bright spot over pier 2 upstream side to be investigated.

3. Steel boxes -inside surfaces -
   Where water was dripping prior to waterproofing the following were observed.:
   a.) At the top flange and web, discolouration of steel.
   b.) On horizontal surfaces (where water was standing) some from of " alligator cracking " with white specs, also loose rust in some places.

c.) Near the bottom of the web, the steel colour is darker, probably due to moisture being absorbed from the floor. (vick effect)

d.) At some of the splices (but not all) where there is an open gap near the top and bottom of the web corrosion appear to be continuing.

The following decisions were made;

a.) Farago to meet informally with CISC representatives to discuss the unusual surface texture. It would be helpful if an expert would visit the bridge.
b.) All flaky and dusty corrosion particles shall be removed from steel surfaces by brushing , following which all dust, bolts, nuts concrete droppings, etc. shall be swept and vacuumed clean.
c.) No decision was made whether the open gaps at splices shall be sealed check with CISC.
d.) All drain-holes and rat-holes at stiffener locations shall be checked for any possible blockage.
e.) The underside of the deck appeared to be dry, but according to Don, where sidewalk construction joints are located over the box, but beyond the waterproofing, there is still leakage. This SCI will try to stop by caulking but if it is not affective al coating of the steel might be required. (Nova Scotia practice)

4. Bearings.

One guided bearing was replaced because the guide failed. During inspection of the bearings two more were found where the guide-bar is twisted (proof that the bolts failed.)

Further investigation is needed to determine (confirm) that :
a.) The original bearing design was inadequate.
b.) Can the failed guided bearings be left in place or not?
c.) What steps have to be taken to transfer the lateral restraint?
d.) Issue must be resolved before " final completion " (i.e. in approx. 4 weeks time.)

5. Concrete deck.

a.) At the June 14 inspection by MacLean and Farago,(following overnight heavy rains and prior to the installation of the waterproofing) numerous leaky joints were found, namely,
I.) Most infill joints between precast panels

ii.) Construction joints at blockouts for deck-drains and for shear connectors over the flanges.

iii.) Improperly plugged tie-rod holes

iv.) Many transverse crack lines in the cast in place deck.

v.) Along the infill strips, more pronounced along the " high side " (i.e. towards the insitu concrete)

vi.) Transverse cracks at regular 1200-1500 mm inervals along the full length of the two infill joints..

vii.) A few transverse cracks were found within the precast panels in the negative moment region

b.) During the present inspection 99% of the cracks (after water proofing and following a dry-spell) appeared to be dry with only a few wet spots . (see item 3e) *

c.) As long as the waterproofing is effective, the durability of the deck is not affected, however the many cracks could influence structural performance of the deck (primerily in the " infill " joints. *
Gerry Strolz promised to respond to this issue in writing.

d.) The large number of cracks, in my personal opinion, will reduce the long term durability of the deck, primarily after the waterproofing starts to fail.

6. Truss strengthening.
Generally satisfactory. Remove any debris left over. Re-attach hand-rail posts over Pier 2.

7. Sidewalks and Parapets.
Problems and solutions.

I.) All construction joints in sidewalks will be sawcut including along curb-face and sealed by caulking. *

ii.) Cracks in sidewalks and parapets generally half-way between construction joints are defects , but no action will be taken to seal them. Will be kept under observation during warranty period and fixed later if warranted.

iii.) Irregular cracks: where sidewalk joint is offset from parapet joint and there is an irregular crack between the two, will not be acted on.
As a deficiency to be kept under observation during warranty.

MEMORANDUM CONTINUED.......
RE: HILLSBOROUGH BRIDGE

iv.) Where (mainly on upstream side) there is a crack, approx . 1"-2" from a construction joint the crack shall be epoxy injected. Also a few wider than usual shrinkage cracks in the parapet (upstream side) shall be epoxy injected.

v.) At north-end west side rails do not line up, needs correction.

vi.) Two sidewalk cover plates were fabricated too short.

8. Asphalt and waterproofing.
Waterproofing quality assurance quality control was left to the Sub- contractor.
During warranty period keep areas of 3% slope under observation for slippage of asphalt or ripple effect.
Cause of one vehicle tearing asphalt and waterproofing 2 days after paving need to be further investigated.
Liquid asphalt seal along curb-face still has to be applied. (Was supposed to be done within 24-hours of paving.

9. Expansion Joints.

Slabs will be installed next week.
Farago prefers leakage tests as it quickly shows up defects, primarily voids behind the armouring and rarely along the seal, but it is a problem under traffic.
Leaks will show- up hopefully within 2 years, but then it will be more difficult to rectify under traffic.

10. Points raised by Farago during discussions: In inspection report include list and hand-over dates for:

a.) Design
Design notes by CBCL shall be passed to the Department for future reference. This applies to the steel boxes as well as the original and increased capacity of the truss.

b.) Drawings.
" As built " drawings.
Shop drawings of rebar
struvtural -steel
strengthening
bearings
exp. joints
wiring

c.) Construction records (test and inspection) :
Including: piling
concrete strength, placing; steel: mill certificates
fabrication and erection reports.

d.) Suppliers and Subcontractors warranties
bearings
exp. joints
retaining walls
Waterproofing

MARENCO CONSULTING AND TESTING

West Royalty Industrial Park
Charlottetown, PEI
CIE 1B0

TEL. 902 629-1245
FAX. 902-628-6547

Post-it® Fax Note 7671

Date Feb 2/97	# of pages ▶ 4
To STEVE MacLEAN	From
Co./Dept. PEI DOT	Co.
Phone #	Phone #
Fax # 368-5425	Fax #

December 09, 1994
Job 5036

Jacques Whitford
4G Walker Drive
Charlottetown, P.E.I.
C1A 8S6

Attention: Mr. George Zafiris, P.Eng.

Dear Mr. Zafiris:

Re: Hillsborough Bridge - Piers 2 & 3
Underwater Inspection

Per your request we have carried out an underwater inspection on the above piers. The work was started on December 06, 1994 but cancelled because of a lack of visibility. The work was completed on December 08, 1994 when the visibility had improved, although it was still poor.

All of the submerged portions of the structure and the seabed are covered with a heavy growth of mussels which is very difficult to remove and which limited the detail of the inspection. It was possible to determine the following.

Pier No. 2
The steel sheet piling is straight, does not appear to have any holes and is covered in marine growth. It is our experience that submerged steel sheet piling will be covered with a dense iron rich layer (magnetite) which limits corrosion to about .05 mm/year/per exposed face.

The top of the steel piling extends about 0.6 m above the base slab inside the caisson. There was no sign of any scour or mud flows in the vicinity of the caisson.

Pier No. 3
The embankment around the pier extends above the top of the steel sheet piling and is much flatter than indicated on the sketches. The embankment is covered with a very heavy growth of mussels and the condition of the rip-rap could not be determined. There was no sign of any mud flows or scour around the embankment.

.../2

December 09, 1994

Jacques Whitford
Attn: Mr. George Zafiris, P.Eng.
Re: Hillsborough Bridge-Piers 2&3
Page 2

The limited time available for the inspection did not allow us to determine the elevation of the top of the embankment or the steel sheet piling.

Though limited by visibility and the heavy marine growth, the inspection did not find any indication of structural problems or other conditions which would make the caissons unsuitable for incorporation in a new structure.

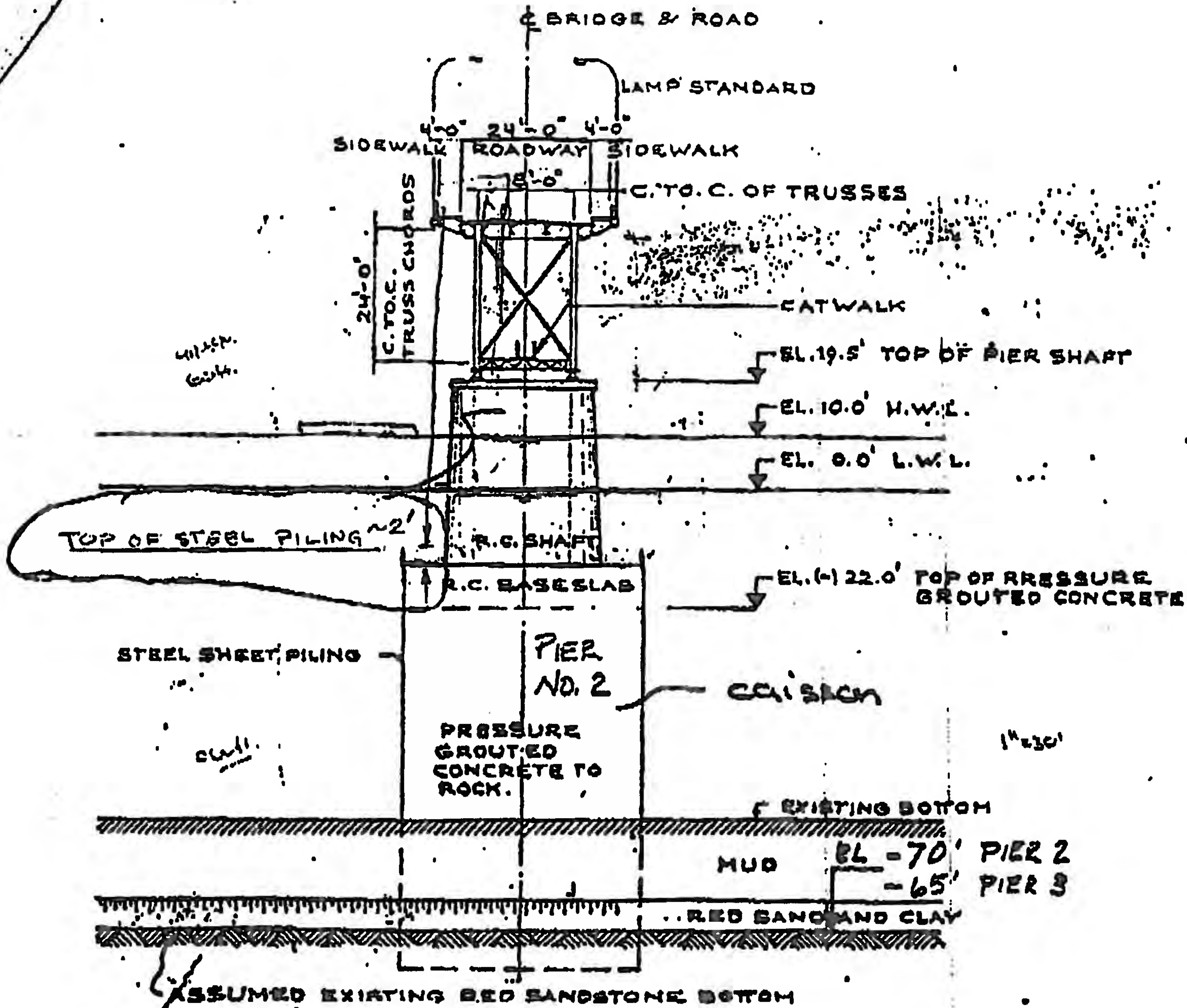
Before a final decision to use the caissons is taken, we suggest a more thorough inspection including cleaning of selected area of the caisson and ultrasonic thickness measurements on the steel piling. Planning for such an inspection should take into account that the current is very strong and diving is practical for less than one-half hour at each tide change. Visibility is very poor after a rain or high winds and would be best in the spring, shortly after the ice leaves.

If you have any questions on the above or if we can be of further assistance, please do not hesitate to give me a call at your convenience.

Sincerely yours,
MARENCO CONSULTING AND TESTING

W.J. MacDonald, P.Eng.

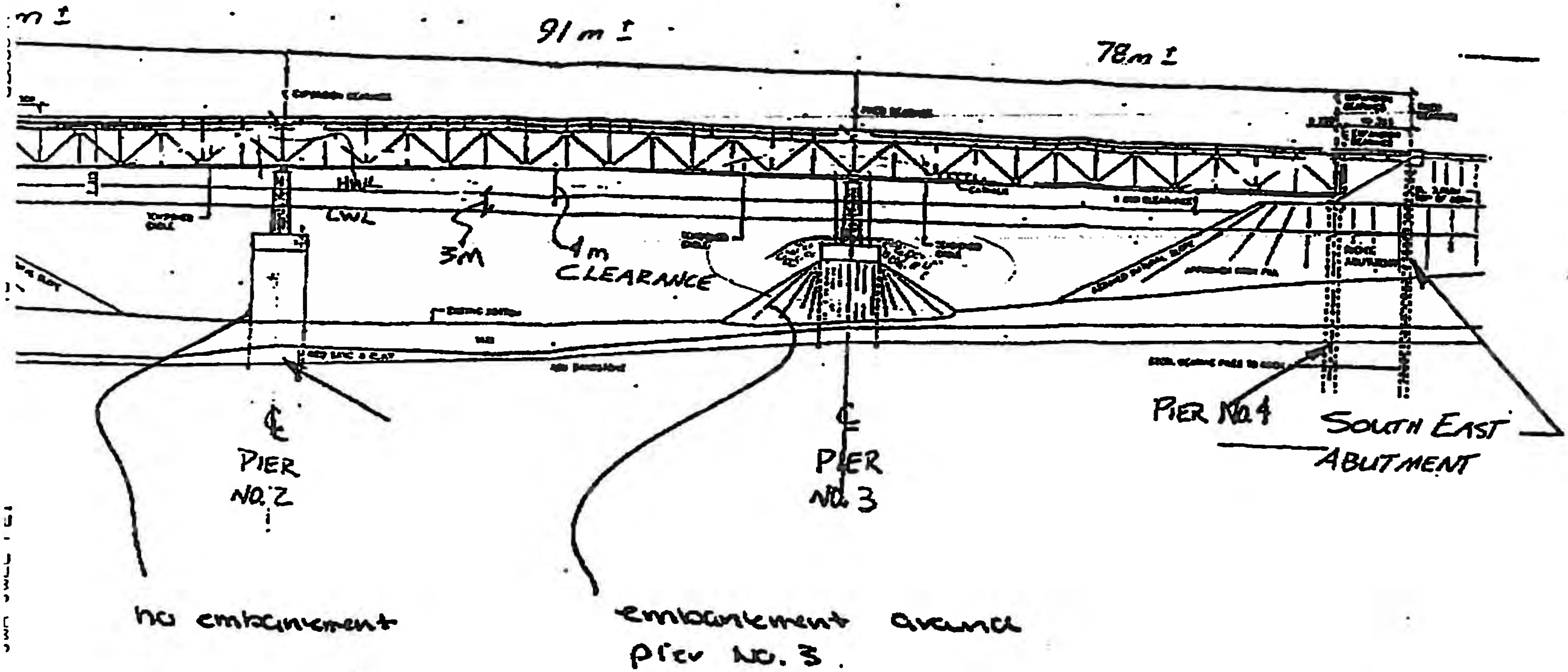
WJM/bm
attachments



SECTION A - A

Dec. 08/94

CH'TN.



Dec. 08/94

◆ Strait Crossing Inc. ◆

10 Horton Drive Charlottetown, PEI, C1A 7G3
Phone: (902) 569-1566 Fax: (902) 569-1820

FAX COVER SHEET

DATE: January 28, 1997

COMPANY: **CBCL**

ATTENTION: **G. Strolz**

FAX NO.: (902) 423-3938

FROM: D. McGinn , P.Eng.

TOTAL NUMBER OF PAGES: **8**
(including cover sheet)

Original to follow? Yes No ✓

RE: Hillsborough River Bridge Improvement
GEWI pile coring program

Gerry,

It appears that the purpose for coring of the caisson concrete was suggested prior to GEWI piles being selected as a means to gain additional capacity. Could you please review and confirm.
Attachment: Minutes of meeting January 27, 1997.

Thank-you,

Strait Crossing Inc.

Steve Plett (for)

Donald McGinn
Project Manager

DM/am

Hillsborough Bridge Improvements
GEWI Pile Installation Meeting No. 1
Monday January 27, 1997
Page 1

Attendance:	G. Zafiris-JWA J. Brown-JWA	G. Kast-DSI S. Pletch	R. Cantin P. Colucci D. McGinn
Distribution:	P. Lockwood-SCI G. Strolz-CBCL		

1. Program To Date

- S. Pletch provided description of works done to date.

Summary

a. Piles 9, 10, 13 and 16 on pier had casing installed.

b. Pile # 16 was drilled using Drill Rig C - A rotary percussion type rig with down the hole hammer.

c. Drilling log for this pile was reviewed.

d. To seal the hole, it was grouted and planned to be redrilled the next day.

e. Storm arrived and took out the cofferdam (i.e. hole has not been redrilled).

2. Coring Program

a. CBCL and JWA found not much geotechnical information exists from 1965 when bridge was installed. Only 3 to 4 bore holes were done.

JWA has maintained that more cores are required. SCI proposed to do coring at time piles are being installed. At that time SCI was considering equipment that could HQ-4" core.

It is there, not to verify the GEWI pile, but for the foundations overall.

b. JWA needs NQ (only slightly under 3") not the 4" specified. This should be done by coring specialist; Long Year or Logan for example.

With DSI equipment we might get 1m/hr coring Vs 6m/hr for correct.

Long Year's equipment is larger.

Hillsborough Bridge Improvements
GEWI Pile Installation Meeting No. 1
Monday January 27, 1997

Coring con't

Action:

It would be more efficient to use DSI equipment for geotechnical core recovery.

Action:

JWA to review number of cores required at each pier. (1wk)

c. Coring the concrete is more of a problem. DSI GEWI pile drilling log will provide some information.

Action:

It appears that coring requirements through the concrete originated before the GEWI piles were selected. CBCL to confirm. Need established in 1994 . Tadros/CBCL/JWA

3. Pile Optimization

a. Drilling of 32 piles in Pier 3 is going to be difficult due to working space limitations.

b. In a 6" hole more than one bar could fit. In BC there is a job with 1x63+ 2x45mm bars. But couplers make this impossible.

c. Larger holes are possible but

i) drill bits much more expensive
ii) casing is more expensive

d. Length of socket could be increased to increase capacity of pile.

e. Based upon DSI experience, the pile length is borderline for test load.

- DSI recommends doing one pile test to failure and JWA concurs. Results may reduce number of P3 pile required.
- DSI recommends pile test be done before drilling rest of piles.
- Winter work of drilling is very difficult. Everything needs to be hoarded and heated.

f. SCI to review P3 pile locations and see where preferred moved locations would be.

Pile Optimization con't

g. Drilling some piles from existing bridge deck with truck mounted rig would be difficult.

4. Testing Program

- DSI testing program is acceptable to JWA.
- JWA wants representative to observe pile load test
- JWA representative to review with DSI drilling engineer requirements and methods of coring.

5. Attachments

a) Drilling Log

b) Drilling Method

c) Test Procedure

d) Drill Starter Detail

DYWIDAG SYSTEMS INTERNATIONAL CANADA

DRILL LOGS

PROJECT HILLSBOROUGH BRIDGE	JOB #	ANCHOR # 16
GENERAL CONTRACTOR STRAIT CROSSING INC.	BORE HOLE DIA. 6½" / 5½"	ANGLE OF HOLE FROM HORIZONTAL 90
	DRILL RIG TYPE	DRILL FLUID AIR
DRILLER RAY ANDERSON	DRILL METHOD DTH and 133 casing + T45	DEPTH OF HOLE 37 m.

DATE	TIME	DEPTH m.	STRATIGRAPHY	REMARKS
1996				
Dec 16	10:55	0 – 7.8	Inside pipe extension	6½" DTH w/ 133 drill pipe
		7.8 – 8.2	new concrete	dry
		8.2 – 9.0	old concrete	– very wet, shells + starfish & seaweed + old bolt, pebbles and misc marine stuff in cuttings
		9.0	steel rebar	
	11:20	9.4	old concrete	add 2m length 133 drill pipe
	11:33	10.3		burst air line
	12:18	10.3		resume drilling – v. wet
	12:30	11.0	steel rebar	
	12:48	11.4		add 2m length Estimate 50 gpm water return
	1:05			coffee break
	1:23	12.6	old concrete	resume drilling
	1:33	13.4		add 2m length
	1:42	14.4	softer concrete	
	1:47	15.4	old concrete	add 2m length
	1:51	15.7		water return reduced, bubbles seen in river.
	1:58	17.0		stop drilling – repair.
	2:08	17.0		resume drilling
	2:10	17.4		add 2m length. Lots of water in return – no bubbles in river.
	2:22	19.4	old concrete	add steel
	2:34	21.4		add 2m length
	2:50	23.4	old concrete	add 2m length
	3:05	25.4		add 2m length High tide – 1.9m. Reduced water return, bubbles seen in river almost immed. full water return est 100 gpm
	3:30	27.4		add 2m length
	3:47	29.4	softer concrete	add 2m length
	4:03	30.8	RED SANDSTONE	Large amt. bubbles in river.
	4:14	31.4		add 2m length. No bubbles.
	4:36	32.0		Drilling too slow. Decided to pull out
	5:05			Hammer out of hole. The 6½" bit has lost all but one of the inner buttons.

2 of 2

DYWIDAG SYSTEMS INTERNATIONAL CANADA

DRILL LOGS

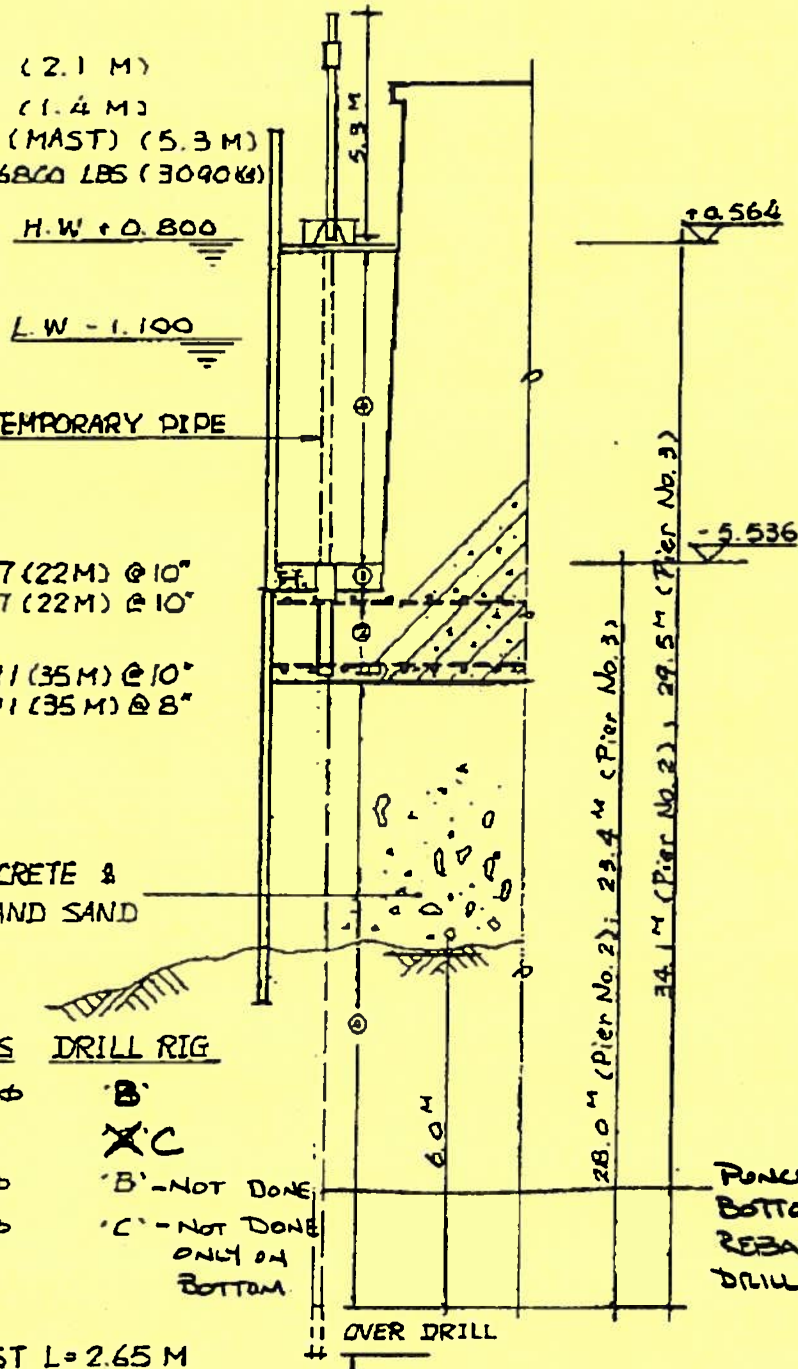
PROJECT HILLSBOROUGH BRIDGE	JOB #	ANCHOR # 16
GENERAL CONTRACTOR STRAIT CROSSING INC.	BORE HOLE DIA. 6½ / 5½"	ANGLE OF HOLE FROM HORIZONTAL 90
	DRILL RIG TYPE	
	DRILL METHOD	DRILL FLUID AIR
DRILLER RAY ANDERSON	DTH and 133 casing + T.45	DEPTH OF HOLE 37m

DATE	TIME	DEPTH	STRATIGRAPHY	REMARKS
				DTH w/ 6½" bit + 133 casing ;
				Ran in loading poles to 30.5.
				Cleaned out hammer screens.
Dec 17	9:12		pink water return	Running in drill pipe - bubbles
				seen in river.
	9:57	32.00		Took a long time to reach the
				bottom of the hole
	10:28	32.4		using top hammer intermittently
	10:48	32.8		very slow drilling (bit is no good)
				bubbles appear only when bit
				is raised off the bottom.
	10:58	33.1		Abandoned drilling - too slow.
				Pulled out.
				Ran in loading poles to 29 m.
				Decided to switch method to
				133 casing w/ 5½" crown bit
				and T.45 steel.
	1:16	32.0		Start to drill
	1:18	33.0	Red water return	
	1:24	34.0		add 2m length
	1:36	36.0		add 2m length Delay to get
				drill supplies.
	1:43	36.0		resume drilling
	1:50	37.0		Bottom of hole.
	1:53			Tripping out
	2:07			All out
	3:10		start consolidation grouting - Type 30 0.45 w:c	
			12 bags filled the casing. Pulling casing and	
	5:00		added a further 16 bags.	
			Tide water = 3m. Depth of river water = 20m.	
Dec 18			Plumbed grout level - 26m.	
	9:45		Tremied in 5 bags - grout leakage seen at bottom of	
			sheet pile. Pumped a further 4 bags and added 80L	
	10:30		gravel at top of pipe. It appears that the leak is sealed.	

DRILL RIG 'C' :
L = 7'-0" (2.1 M)
W = 4'-6" (1.4 M)
H = 17'-4" (MAST) (5.3 M)
WEIGHT : 6800 LBS (3090 kg)

REINFORCEMENT
TOP MAT :
TOP : No. 7 (22M) @ 10"
BOT : No. 7 (22M) @ 10"
BOT MAT :
TOP : No. 11 (35M) @ 10"
BOT : No. 11 (35M) @ 8"

MIX OF LEAN CONCRETE & LOOSE GRAVEL AND SAND



DRILLING STEPS	DRILL RIG
① CORING 210 Φ	'B'
② DTH - 150 Φ	~~X~~ C
③ CORING 145 Φ	'B' - NOT DONE
④ CASING 140 Φ	'C' - NOT DONE ONLY ON BOTTOM

PUNCHED THRU BOTTOM MAT OF REBAR WITH DRILL RIG 'C'

DRILL RIG 'B' :
LENGTH OF MAST L = 2.65 M
WEIGHT : W = 200 KG
FEATURES : ROTARY / PERCUSSION DTH HAMMER ; CORING

DRILL RIG 'A' :
STANDARD CORE DRILL :
WORK PERFORMED BY LOCAL CORING CONTRACTOR.

DSI
DYWIDAG SYSTEMS INTERNATIONAL CANADA LTD.

HILLSBOROUGH RIVER BRIDGE P.E.I

DSI PROPOSAL DRILLING

09/23/96
A.C
G.K
DWG. No. SK-01

MARK-UP FROM JAN 27th MEETING

General Notes.

1. Material
   1.1 Gewi-Pile
   #20 Grade 75 Ksi Dywidag Threadbar by Dywidag - Systems International.

   Yield Capacity = 368 Kips
   Ultimate Capacity = 491 Kips
   Max. Design Load = 0.6 X Yield Capacity (220 Kips)

   1.2 Grout
   Type 30 Cement, w/c = 0.5(max), fc' = 30 MPa (min) at 7 days

2. Drilling
   Drill hole for anchor by using percussion drilling method with casing liner if required.

3. Installation and Grouting
   3.1 Water testing the borehole (if required).
   3.2 Install the pre-assembled threadbar into the bore hole.
   3.3 Trimie grout the bore hole with the grout line attached to the bottom of the anchor.
   3.4 For the test pile only, two stages grouting shall be required and bond breaker shall be installed on top of the bonded length of the pile.

4. Testing (Gewi-Piles shall be tested in tension: one in every four piles)

   4.1 Performance Test (As Per PTI Recommendations)
   Performance Tension Test on TWO (2) production Gewi-piles only. (One at each pier)

   Procedure as follow:

   Design Load P = Working Load
   Max. Test Load = 60% Yield of Bar (220 Kips)
   Alignment Load = 10% Design Load

   Testing Procedure:

   Stress anchor to Alignment Load and dial guage will be set for zero at this datum load.
   The test shall be made by incrementally loading and unloading starting from the Alignment Load to the following maximum cycle loads:

   0.25 P
   0.5 P
   0.75 P
   1.0 P
   1.33 P / Max. Test Load

   At each increment, the movement of the anchor shall be recorded to the nearest 0.001 inches.The creep readings at the maximum test load shall be read at 0, 1, 2, 3, 4, 5, 10. 15, 20, 25, 30, 45, and 60 minutes. Unload the anchor to 100%, 75%, 50%, and 25% of the Design Load and then back to the Alignment Load and measure the anchor extension at each load.

   4.2 Proof Test (remaining anchors)

   Max. Test Load = 60% Yield of Bar (220 Kips)
   Alignment Load = 10% Design Load

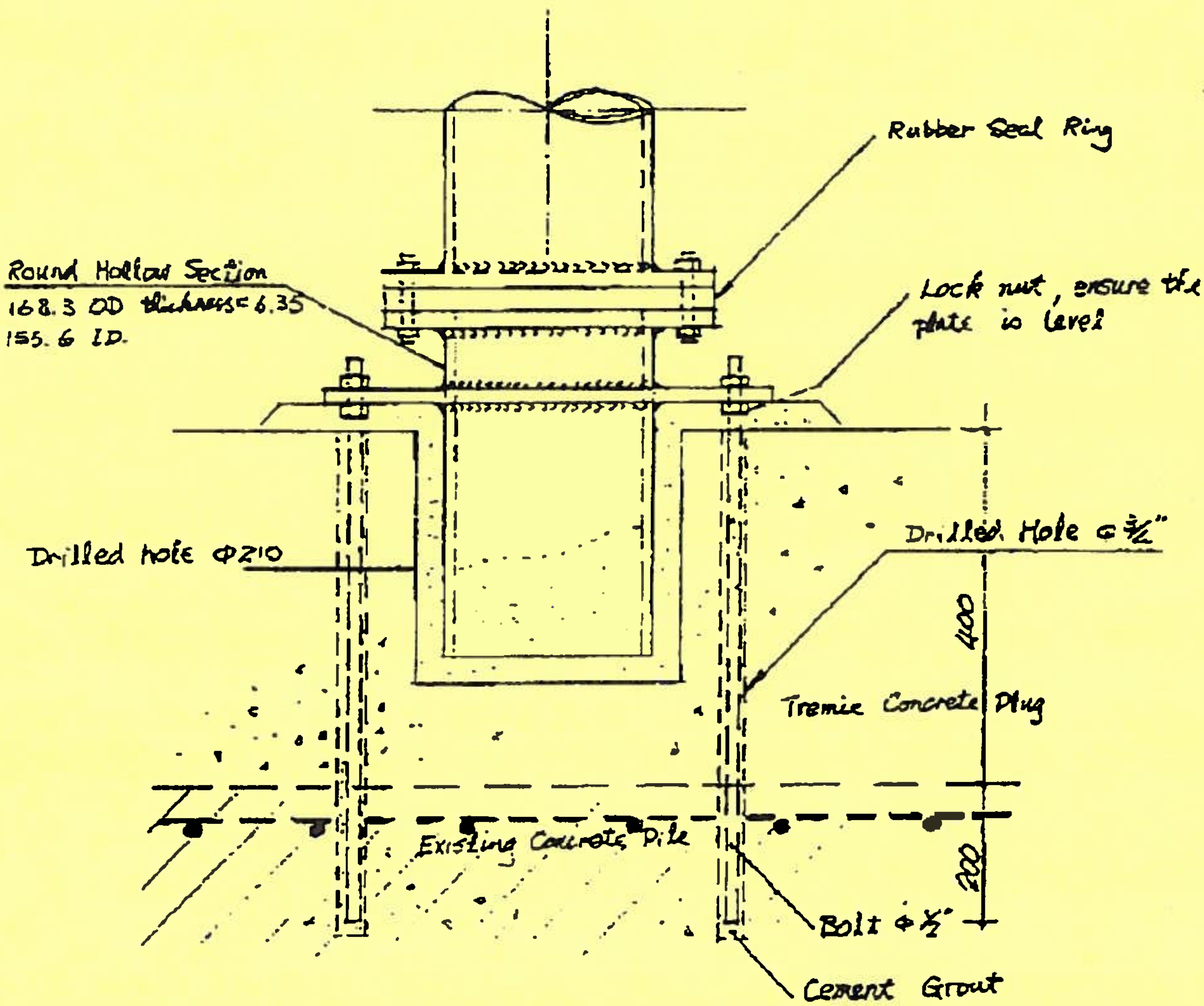
   Testing Procedure:
   Stress anchor to Alignment Load and dial guage will be set for zero at this datum load. Continue stressing anchor incrementally to 25%, 50%, 75%, 100% of the Design Load. At each increment the movement of the anchor shall be recorded to the nearest 0.001 inches. Stress anchor to 100% of the test load and hold for 10 minutes, the elongation shall be measured at 1, 2, 3, 4, 5 and 10 minutes Unload the anchor to 100%, 75%, 50%, 25% and 10% of the design load and measure the anchor extension at each load.

FACTORED LOAD = 900 kN/PILE

SCALE: N.T.S.	APPROVED BY:	DRAWN BY A.C
DATE: DEC. 11 1996		REVISED
Hillsborough River Bridge		
Gewi Pile Installation		DRAWING NUMBER HRB-004

PRINTED ON NO. 1000H CLEARPRINT ®

This program is acceptable to JWA.



DSI

DYWIDAG SYSTEMS INTERNATIONAL CANADA LTD.

HILLSBOROUGH RIVER
BRIDGE P.E.I

DSI PROPOSAL

09/23/96
A.C
G.K
DWG. No. SK-03

Strait Crossing Inc.

7th Floor, 1177 - 11th Avenue S.W. Calgary, Alberta T2R 1K9
Phone (403) 244-9090 Fax (403) 228-8643

FAX COVER SHEET

DATE: February 3, 1997

COMPANY: PEIDOT

ATTENTION: Mr. Steve MacLean, P.Eng.

FAX NO: 902-368-5425

FROM: Phillip Lockwood

TOTAL NUMBER OF PAGES: 1
(including cover sheet)

Original to follow? Yes No /

Dear Sir:

Re: Changed Condition – Pier #3
Hillsborough River Bridge Improvement Project

With reference to our many discussions on this subject over the past several months, we would confirm that the conditions encountered at Pier #3 are not as represented on the O.J McCullogh drawings for this structure. We have found that the top of the caisson has sheetpile which extends well above those cut-off elevations which could be reasonably expected and that the top of the caisson has been covered with non-structural fill material which has then been topped with sections of low strength concrete. Additionally, the centerline of the caisson is not concentric with the centerline of the pier shaft. A video has been taken to record the condition of the caisson so that those "above water" might better understand the extent of the problem and determine an appropriate solution. The video is available at our site office for your review. Similar to the changed conditions as were encountered at Pier #2, SCI views the costs of the additional work necessary to prepare the caisson for the installation of our cofferdam to be extra to our contract. The Project office will be able to provide you with full details of the steps and anticipated costs that will be incurred to "restore" the caisson to expected as-built conditions.

Yours sincerely,

Phillip Lockwood, P.Eng.

kc: SCI – Steve Pletch / Donald McGinn

◆ Strait Crossing Inc. ◆

10 Horton Drive Charlottetown, PEI, C1A 7G3
Phone: (902) 569-1566 Fax: (902) 569-1820

FAX COVER SHEET

DATE: **June 26, 1997**

COMPANY: PEI Department of Transportation and Public Works

ATTENTION: **Mr. Steve MacLean, P.Eng**

FAX NO.: **368-5425**

FROM: Donald J. McGinn

TOTAL NUMBER OF PAGES: **2**
(including cover sheet)

Original to follow? No

RE: Hillsborough River Bridge Improvement Project – GEWI Pile Installation – Pier No.3

Dear Sir:

As you are aware, we commenced drilling operations at Pier No.3 on June 16, 1997.

To date we have attempted drilling 23 locations. Two of these locations have had to be abandoned because of encountering significant steel within the caisson, which can not be drilled through. Steel has been encountered in some other locations, but the drilling has managed to mill through.

In general we are encountering a sand/gravel layer between 17 to 24m from the drill deck. The sand is dense enough that consolidation grouting is not successful – the grout does not penetrate the layer. There have been a couple of voids encountered also. The extent of the sand/gravel zone is more regular and over a larger area than indicated in the previous information provided. The amount of sand in the layer was unexpected.

We have asked Dr. John Brown (Geotechnical Engineer) to review our results to date. Dr. Brown has confirmed the sand/gravel layer should still be able to provide the lateral support required to the GEWI, in other words, the GEWI pile concept is still valid.

The inability to consolidate grout the sand/gravel layer results in the requirement to install GEWI piles through a casing and withdraw the casing while grouting. This has led to the difficulty that the drill rig is not capable of drilling a large enough hole for the specified No.20 bar to be installed within the casing.

Alternative drilling equipment is not a viable option. It would require a completely different platform design. In addition, it would probably not be capable of working in the tight confines around the pier. The selection of a larger drill operation would be difficult with present hoisting capabilities. The time to mobilize new equipment and become familiar with it is also significant.

We have reviewed the problem and have found that a high grade No.18 bar can provide the required pile capacity and can be installed within the casing of the present drill system. These bars have been ordered and are scheduled to arrive at site Friday, June 27. A longer bond length in the sandstone bedrock, 10m instead of the 6m indicated on drawings.

will be provided to keep the bond stress within acceptable limits. We will review if the No.20 bars already on site may be incorporated into the truss stengthening.

Installation of bars within the casing requires that the casing be removed after the pile is installed, while grouting, such that the grout will fill the column around the bar. The drill rig is required to withdraw the casing. The inability to do the two operations (drill and grout) at same time, lengthens the overall time required to install the GEWI piles.

Commencing Friday, we will have the piling work proceeding continuously (night, day, weekend, holiday) until completion. Providing no problems arise, piling may be completed by July 7th (late date required for Forming Sub-Contractor to complete pier by August 8 for Steel Girder Erection) and no effect to the overall Project Schedule will be incurred.

We will continue to keep you informed of our progress as it occurs.

Kind Regards,

Strait Crossing Inc.

Donald McGinn, P.Eng
Project Manager

cc P. Lockwood
S. Pletch

◆ Strait Crossing Inc. ◆

10 Horton Drive Charlottetown, PEI, C1A 7K7
Phone: (902) 569-1566 Fax: (902) 569-1820

TRANSMITTAL SHEET

COMPANY: PEIDOT **DATE:** December 9, 1997

REF: Change Order- P2 and P3

ATT: Mr. Steve MacLean

We are submitting (under separate cover), the following documents which are:

( ) Approved
( ) Approved as noted
( ) Not Approved
(x) For your approval
( ) For your records
( ) Preliminary
( ) Revised
( ) As built
(x) For your use

Number of Copies		Title or Description
1	Cover Fax & Binder	GEWI Pile Claim

Delivered by hand

PER: P. Lockwood

cc: D. McGinn

Strait Crossing Inc.

7th Floor, 1177 - 11th Avenue S.W. Calgary, Alberta, T2R 1K9
Phone: (403) 244-9090 Fax: (403) 228-8643

FAX TRANSMITAL SHEET

DATE: December 4, 1997

COMPANY: **PEIDOT**

ATTENTION: Mr. Steve MacLean, P.Eng.

FAX NO: 902-368-5425

FROM: Mr. Phillip Lockwood

TOTAL NUMBER OF PAGES: 4 + Attachments
(Including cover sheet)

Original to follow? Yes No

Dear Sir:

Re: Change Order– Piers #2 and #3
Hillsborough River Bridge Improvement Project

We reference our earlier communications concerning the problems encountered by SCI during the installation of the Gewi piles at Piers #2 and #3.

By way of brief recap, the condition and integrity of the existing caissons at Piers #2 and #3 has always been in question. During the early stages of the development and prior to SCI entering into a contract for the construction of the Hillsborough Bridge Project with the Province, it was mutually agreed between the Parties that it made little economic sense to mobilize an independent drilling operation to ascertain the condition of the interior of the caissons. SCI was confident in the overall proposed Gewi pile foundation solution for the piers, however, there were unknowns in the number of Gewi piles that would be ultimately required and the drilling/installation conditions that would be encountered. It would have taken the discovery of a very serious problem however, before the Gewi design solution itself was abandoned. The discovery of problems (as a result of the pre-contract independent drilling program) which would make the installation of the Gewi piles more difficult would only serve to raise the cost of the project as at Financial Closing.

It was believed by each of DOT and SCI that a more cost effective way (as opposed to conducting the pre-contract drilling program) of dealing with the "unknowns" at Pier #2 and #3 was to note the potential of the "unknown" in the contract document and to deal with any problem as it arose. To accommodate this arrangement and intent, a section of the Sale of Bridge Agreement was developed, addressing the notion of "Pre-Existing Conditions".

It is important to point out that in choosing to not perform the investigative drilling program at the pre-contract stage, SCI/DOT were not "shooting in the dark" as to the probable success of the Gewi pile foundation solution. A series of "As-Built" drawings were available to the project which had been prepared by O.J. McCulloch during the original construction of the bridge foundation. These drawings reported that the caisson foundation for Pier #2 was constructed of a pressure grouted concrete (concrete formed by pressure grouting previously placed aggregate) while Pier #3 was of a poor quality tremie concrete. From the data available, the design of the Gewi pile foundation solution was viable and the correct solution from both an engineering and economic standpoint.

The installation of the Gewi piles at Pier #2 did not proceed exactly as planned. During the drilling of the pile shafts, SCI experienced large inflows of water at a number (most) of the shaft locations. To ensure that the water infiltration did not adversely impact the quality of the grout around the Gewi pile, a two stage grouting process was undertaken. The installation of the Gewi pile now required the drilling of the shaft, full length grouting of the shaft (to seal the water seams around the pile shaft), re-drilling of the shaft, installation of the Gewi pile rod and finally, the re-grouting of the Gewi pile itself. Through adopting this procedure, the amount of drilling required at the caisson effectively doubled. In addition to these problems, the Gewi crews encountered several pieces of embedded steel during their drilling operations, requiring the abandonment of two pile locations.

The installation of the Gewi piles at Pier #3 encountered even more severe difficulties. As mentioned earlier, the McCulloch "As-Built" drawings reported that the Pier #3 caisson was constructed of a "poor quality tremie concrete". SCI fairly considered that the words "poor quality" implied "low strength". For the purposes of the installation of the Gewi piles, "low strength" was not a terribly significant issue. However, once drilling was commenced at the pier, it became evident that as opposed to the tremie being of "low strength", there was virtually no cement matrix in large zones of the caisson. Certainly, it was not reasonable to call this material "concrete" as had been reported on the "As-Built" drawings.

The direct impact of this lack of a cement matrix in the Pier #3 caisson was that the drilled shaft would not "stay open", that is, the upper regions of a drilled hole would break away and slough down onto the top of the drill bit, tending to bind and trap the bit. Through this material falling into the hole and on top of the drilling bit, the drill would become wedged into the hole and thereafter require great effort (and some luck) to break it free and allow the bit to be extracted from the hole. When these conditions were encountered, it would be necessary to grout the hole in an effort to secure the spalling areas of the shaft and then to commence the re-drilling operation.

After repeated attempts, it became clear that the material contained within the Pier #3 caisson could not be effectively grouted. It was concluded that as a result of a high sand content, the grout was not able to penetrate into the surrounding body of the "concrete"caisson.

For a number of the early piles attempted at Pier #3, the Gewi pile installation crews would painstakingly go through a drilling, grouting, re-drilling, re-grouting re-drilling, re-grouting, re-drilling process in an attempt to complete a single hole. Ultimately, not a single "free-standing" hole could be completed at Pier #3.

Despite all best efforts, the Gewi pile shafts required at Pier #3 could not be achieved using the drill/grout process. It became evident that the piles could only be installed by casing the hole, a process wherein a heavy walled pipe sleeve is used during the drilling process, thereby preventing the sand material from raveling and falling into the partially drilled hole. The installation of a shaft casing requires considerable machine power as compared to the power required for a straight drilling operation. Due to space limitations, the structural capacity of the drill work platform and the limited lifting capacity available from either the water or bridge, a larger drill rig as would be needed to install the size of casing compatible with the #20 Gewi rods could not be readily mobilized. The decision was made to attempt the installation of the casing using the current Gewi pile drilling equipment. As the machine did not have the power necessary to install the large diameter casing as needed for the #20 Gewi rods, a smaller diameter (#18) Gewi rod had to be purchased.

The decision to use the #18 Gewi piles in lieu of the earlier detailed #20 Gewi piles could not be decided by SCI alone. It was necessary for both CBCL (the structural engineer) and JWA (geotechnical engineer) to review the suggested design modifications and to provide concurrence. It was necessary for each of the designers to perform a through re-evaluation of the caisson foundation design, based upon the latest results of rock strengths and structure performance.

The installation of #18 Gewi piles at Pier #3 was completed to the satisfaction of the Engineers utilizing the casing technique. The installation and removal of the casings was also however, not without its problems. On a number of occasions, the loose sand of the caisson would flow down the outside of the casing and firmly wedge the casing in place. By the time the work had been completed, a total of five strings of casing had to be abandoned.

The final installation of the Gewi piles at Pier #3 has been reviewed by each of the structural and geotechnical design engineers. They are satisfied that sufficient additional foundation capacity has been provided at the pier in accordance with their design requirements.

As a direct result of the delays which occurred to the Project Schedule due to the extra work required at Pier #3, the erection of the structural steel bath-tub girders had to be delayed. This resulted in extra costs being incurred by Cherubini and Cherubini's erector, Marid. To minimize the delay to the erection program and the cost being incurred as a result of that delay, SCI ordered certain elements of the pier formwork contractor's work to be accelerated (Lancor).

Clearly, the conditions at Piers #2 and #3 were not as contemplated at the time of Financial Closing for the Project. SCI has incurred considerable additional expense in achieving the installation of the Gewi piles, and as provided by the terms of the contract, we are to be compensated for same.

Our additional costs are summarized on the following page. Supporting documents showing the derivation of the various daily rates used in the calculations are provided as an attachment to this document, as are copies of the various invoices for contractor delay and additional engineering costs.

A summary of the additional costs are as follows:

Pier #2	
1) Cost of Drilling Gewi Shafts	
(22 Days – 2 Day Test – 2 Day Core – 2 Day Learning) @ $9,078 / Day	$145,248
Less: Drilling Cost from Work Breakdown Structure	73,943
Total Additional Cost of Drilling at Pier #2	**$71,305**
Pier #3	
1) Cost of Drilling Gewi Shafts	
(48 Days – 1 Day Test – 1 Day Core – 1 Day No Work) @ $9,078 / Day	$408,510
Less: Drilling Cost from Work Breakdown Structure	117,912
Total Additional Cost of Drilling at Pier #3	**$290,598**
2) Cost of Additional Materials and Supplies	
Gewi Bars #18	$47,072
Casing Lost in Hole	17,775
Total Additional Cost of Material at Pier #3	**$64,847**
3) Cost of Schedule Acceleration and Delay	
Cherubini	$71,610
Lancor	6,431
Total Additional Cost of Schedule Implications at Pier #3	**$78,041**
4) Cost of Additional Engineering	
CBCL	$10,063
JWA	6,140
Total Additional Cost of Engineering at Pier #3	**$16,203**
SUBTOTAL OF DIRECT ADDITIONAL COST FOR GEWI PILE INSTALLATION	$520,994
Mark-up for Overhead and Profit @ 15%	78,149
TOTAL ADDITIONAL COST OF GEWI PILES AT PIERS	**$599,143**

Please provide confirmation of your agreement with the above such that we can prepare the necessary Change Order for your execution.

Yours truly,

Phillip Lockwood, P.Eng.

xc: SCI – Donald McGinn

Strait Crossing Inc.
7th Floor, 1177 - 11th Avenue S.W. Calgary, Alberta, T2R 1K9
Phone: (403) 244-9090 Fax: (403) 228-8643
E-mail: capital@cpgi.com

FAX TRANSMITAL SHEET

DATE: May 20, 1998

COMPANY: PEIDOT

ATTENTION: Mr. Steve MacLean, P.Eng.

FAX NO: 902-368-6244

FROM: Phillip Lockwood

TOTAL NUMBER OF PAGES: 8
(Including cover sheet)

Original to follow? Yes No •

Dear Sir:

Re: Submission for Equitable Adjustment
Installation of Gewi Piles at Piers #2 and #3
Hillsborough River Bridge Improvement Project

Please find attached the following:

1) Letter from DSI establishing their interpretation of what "poor quality tremie concrete" implied.
2) Extract from Sowers & Sowers defining the range of aggregates for which cement grout would be considered effective. This range of aggregate clearly lies within the boundaries of a concrete mix design as would be anticipated for a "mass concrete".

I would appreciate your further review of the Gewi Pile installation issue. As has been presented to you earlier, we feel very strongly that the drilling conditions encountered at the Project could not have been reasonably anticipated from the descriptions provided on the McCullogh drawings. Based upon the drilling conditions which were reasonably anticipated, the drilling methods proposed by DSI for the work were correct and should have been effective.

As discussed with you, representatives of both DSI and SCI will travel to Charlottetown to meet with you at 9:00 AM on May 22nd. We appreciate you breaking away from your new responsibilities and taking the time to meet with us. We look forward to resolving this issue with you on Friday.

Yours sincerely,

Phillip Lockwood, P.Eng.

xc: SCI – Donald McGinn / Steve Pletch

DYWIDAG-SYSTEMS INTERNATIONAL

DYWIDAG-SYSTEMS INTERNATIONAL
#103-19433-96th Avenue
Surrey, BC V4N 4C4
Ph: (604) 888-8818
Fax (604) 888-5008

Fax Total number of pages: 2

To: Strait Crossing Group Inc.
Attn.: Phillip Lockwood, P.Eng.
Phone:
Fax: 403-228-8643
CC:

From: Gary Kast
Date: May 20, 1998
Ref. No.:

Original to follow: ☐ Yes ☐ No

Subject: Hillsborough River Bridge - Gewi-Pile Installation

Dear Sir:

We are still waiting for payment on the balance of your account for this project which we understand you will not release until the escalation of the budgeted cost estimate has been explained.

We based our proposal to SCI on drawings prepared by O.J. McCullogh. The drawings indicate that we would be installing Gewi-piles through poor quality tremie concrete, into the underlying sandstone bedrock.

In my 30 years of experience in the drilling business, my interpretation of "poor quality" tremie concrete, is a compact concrete of low to medium strengths, which may contain occasional seams or pockets of washed out aggregate. Based on this information, we selected the appropriate drilling and installation method and related equipment, for the pricing of the Gewi-pile installation program, which we submitted to you in various proposals.

Once out in the field, we found the drilling conditions to be substantially different from those we had anticipated. Our drilling crews were not able to keep the boreholes open, a condition you would not expect to encounter in a tremie concrete structure. The presence of aggregate pockets in tremie concrete may somewhat slow down the drilling operation but not to the extent that the hole cannot be drilled by an open percussion drilling method.

From our drilling records on this project, it is evident that the drilling time for a borehole in a typical tremie concrete formation, was drilled from the platform level through the heavily reinforced pier cap to the top of the bedrock at an average time of 2-1/2 hours. Drilling of the 8 meter deep socket in the sandstone, took less than 40 minutes. These drilling rates were achieved only in Pier #2 where drilling conditions were considerably better than in Pier #3, despite encountering loose aggregate. Only 3 out of 16 holes in Pier #2 had to be cased, consolidation grouted and redrilled several times, requiring very large quantities of grout (up to 82 bags of cement per hole). Throughout the project, all the holes were drilled at least once, then consolidation grouted over their full length and redrilled again. The situation was particularly bad on Pier #3 where we started the drilling operation with the same method and equipment as in Pier #2.

At Pier #3, the initial drilling through the reinforced pier cap (4 layers of rebar), into the tremie concrete, progressed very well, until a massive zone of aggregate was encountered. A production of up to 7 holes per day was achieved for the first 10 meters of the boreholes, when the fine aggregate stopped any further penetration with the open hole drilling method.

Consequently, the entire drilling operation had to be changed over to a technically much more demanding cased rotary percussion method. Additional equipment and consumables had to be mobilized. In the first phase of casing drilling, we advanced the casing approx. 1.5 meters into the bedrock, pressure grouted the entire depth of the pier foundation by injecting cement grout through the casing, while withdrawing the casing simultaneously. The grout take varied from borehole to borehole and was as high as 44 bags of cement and 27 bags of sand per hole, in the first phase of consolidation grouting.

Anticipating that the consolidation grout would penetrate the sandy gravelly zones sufficiently to restore tremie concrete conditions, we attempted to redrill by the open hole method, but were unsuccessful. Obviously, the fine sands in the loose aggregate, prevented any effective grout penetration.

In general, the aggregate in the pier foundation was found in various layers, ranging in thickness from a few centimeters to several meters, with interbedded concrete lenses. During the second phase of redrilling, the borehole was generally advanced to its final depth, and the casing was withdrawn after grouting.

Unfortunately, at several holes, at the east side of Pier #3, the casing locked in during the withdrawal operation and could not be recovered, even when pulling at a force of 100 tons. Altogether, in total, we lost 98 meters of casing. Much of the casing had to be brought in by hot shot delivery from the USA since we were not prepared to deal with these conditions.

Since we were able to withdraw the full length of each string of casing during phase 1 drilling at Pier 3, we concluded that fine aggregates, mobilized by the change of tidal water levels built up behind the casing and caused the casing to wedge in. Despite working our powerful drilling equipment to the limit, using torque, percussion and pulling action all at the same time, we were unable to break the casing loose. Equipment breakdown and high repair costs were the result of our fruitless attempt to retrieve the casing.

I must emphasize that this was a very unusual situation based upon our experience of drilling through poor quality concrete. In our experience, the first phase of consolidation grouting should have been sufficiently effective in stabilizing the loose aggregates allowing us to drill to the required depths without a problem. Instead, many of the holes on Pier #3 required 2 or more drilling and grouting operations to achieve the required results.

We appreciate your efforts in arranging a meeting with PEIDOT which gives us an opportunity to explain and resolve the outstanding issues.

Yours truly,

Gary Kast
General Manager

I TRIED TO SEND THIS E-MAIL BUT FOR UNKNOWN REASON IT DID NOT GO THROUGH BERT

scmaclean@gov.pe.ca, Hillsborough Bridge-Site visit

To: scmaclean@gov.pe.ca
From: Bert Farago <bfarago@aracnet.net>
Subject: Hillsborough Bridge-Site visit June 16, 1998
Cc:
Bcc:
X-Attachments:

Following our joint inspection of both boxes and the underside of the deck
I have decided to summarize my observations as follows:
No attempt was made to map all the cracks, because it would have been a
lengthy exercise and in any case it is part of SCI'S tasks to prepare it
and issue as part of their non-conformance report.

A. Underside of deck inside both boxes.
The structure consists of 6m long precast panels with cast-in-place
transverse infill joints.
It is evident that there are many discontinuities along the
precast /infill interfaces.
These include:
Transverse joints between precast panels.
Around the perimeter of blockouts at most deck-drains.
Along the haunches over the top flanges where a cut-out
for shear connectors was imperfectly filled.This is
more pronounced along the lower side (i.e. along
the curb-face catching the water.)
A few of the grouted bolt holes leak water.
In the downstream box, northspan the junction box of
the electrical system is dripping water.

B. Steel boxes of weathering steel.
Boxes are generally clean with construction debris removed.
At every leak there is discolouration and evidence of continuing
corrosion.
At the abutment, particularly on the north side, water was standing up
to 1/4" deep near the end diaphragm.
There is some dampness at the open gaps (top and bottom) of web

splice plates, but this area can dry out quickly.

C. Underside of the deck between the two boxes.
The deck here is cast-in -place high-performance concrete,cast seperate from the precast decks on either side, followed by the casting of an approx. 1,000 mm.wide in- fill panel

a.) Infill panels.
There are shrinkage cracks at regular 1200 mm to 1500 mm interwalls along the full length. this can be expected due to restrained shrinkage.

b.) Longitudinal joints between decks and infill concrete.
The low-side (precast-side) is generallygood, but the high side (adjacent to the cast-in-place concrete) there is a continuous crack at almost full length.
It is my personal opinion, that the wet concrete tends to flow towards the low point, causing a line of weakness at the high end.

c.) Cast in -place high performance concrete.
There are numerous near transverse cracks along the full width of the deck. The spacing of cracks is fairly close south of the south pier,it is much fewer between the piers and increasing again north of the north pier.

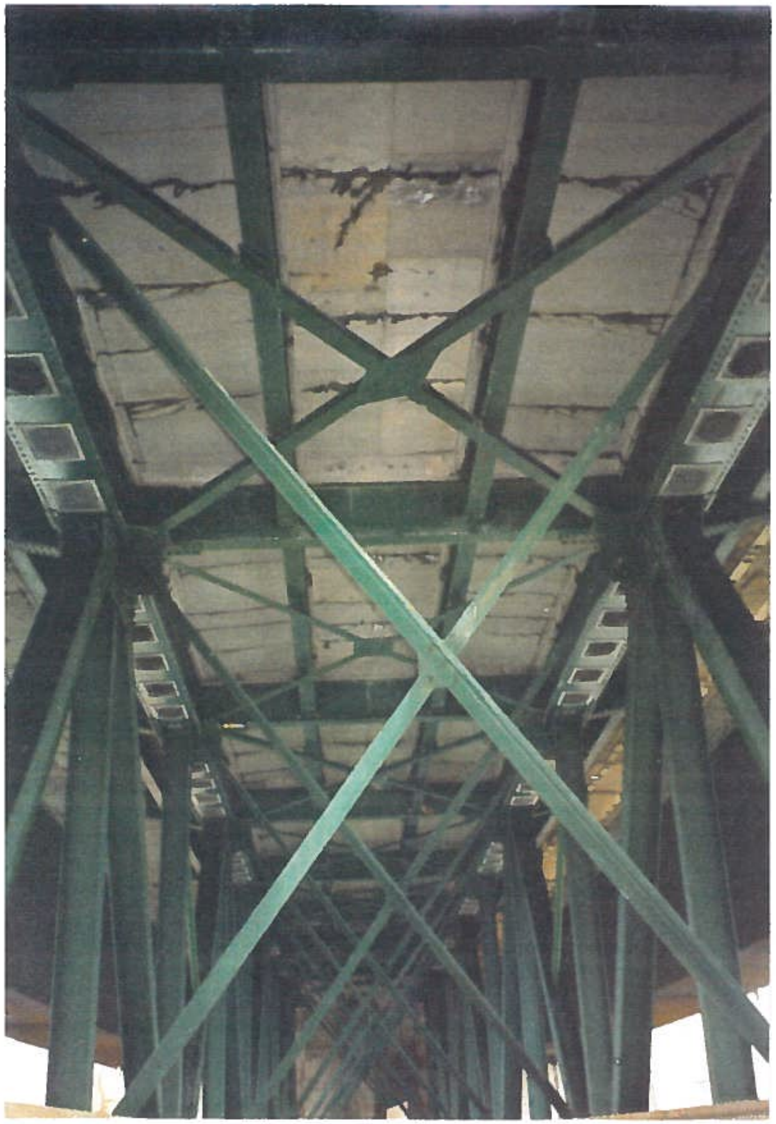
CONCLUSIONS.

SCI's quality assurance programm, I believe, requires that the presence of a large number of cracks, wider than 0.15mm shall be reported to Head Office.

I am convinced that all through cracks fall into this category.
I would like to see comments from CBCL and SCI (Tadros) how the presence of these cracks affect the structural performance as well as the future durability of the deck.
Certainly the low permeability of the high-performance concrete was partly lost through the cracks.
Sealing of the cracks by gravity feeding was not successful.

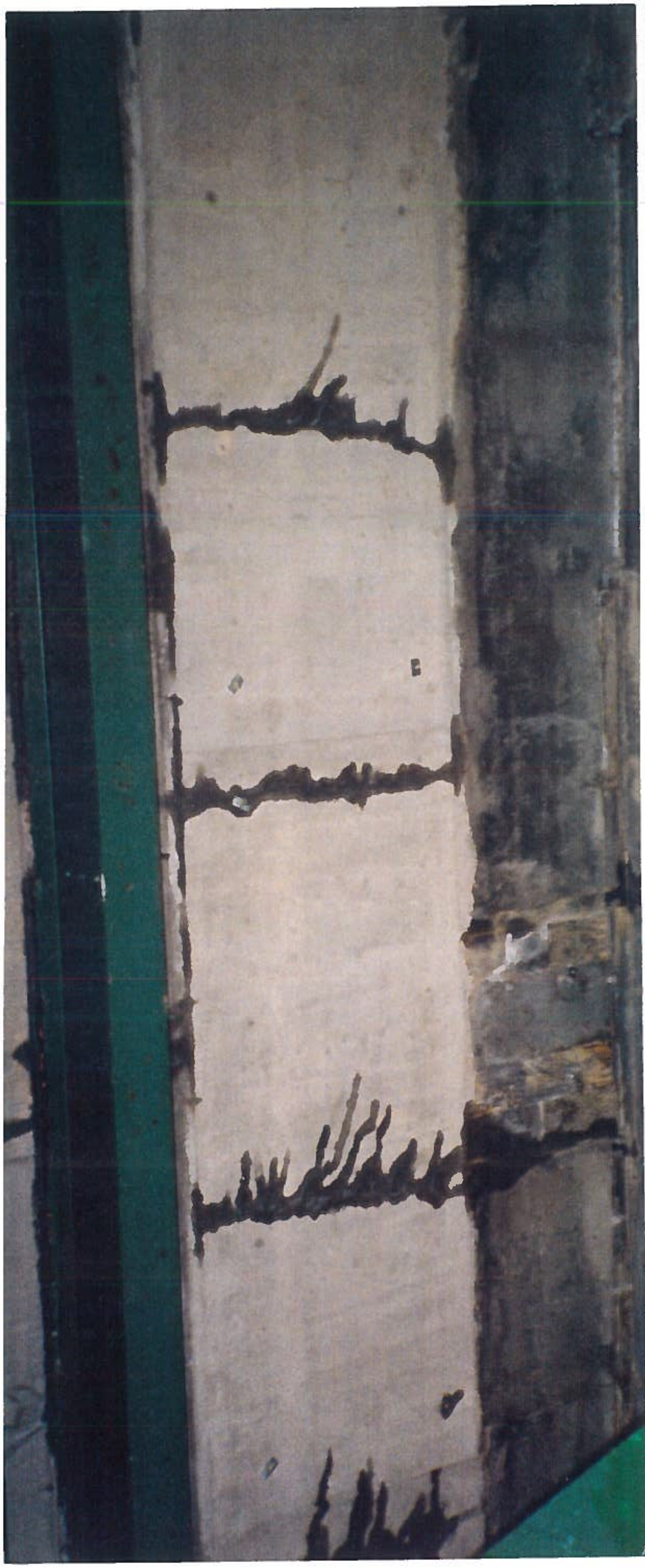
I wonder wheather the waterproofing (scheduled to start on Monday, June 22nd)
should not be delayed until the crack issue is fully resolved. The significantly differing stiffnesses of the adjacent structures puts greater strain on the performance of the infill strip.

There is loose rust and at some places dampness in the loose rust, at most leakage points. Following completion of the waterproofing, I recommend that all
loose flaky and powdery rust be removed by effective brushing and vacuming to leave the steel surface clean and uniform. This is essential for the future monitoring of the performance of the deck.



Cracks in cast-in-place deck-slab.

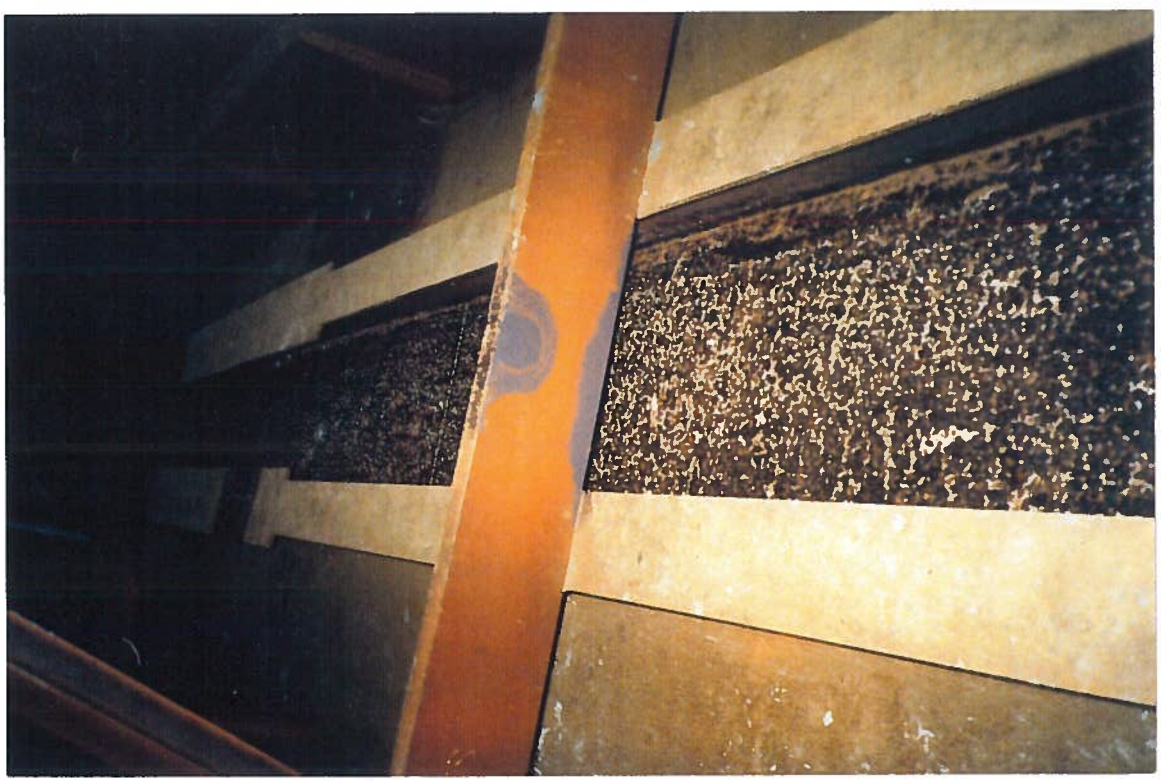
Cracks in in-fill strips.




Cracks in in-fill strips

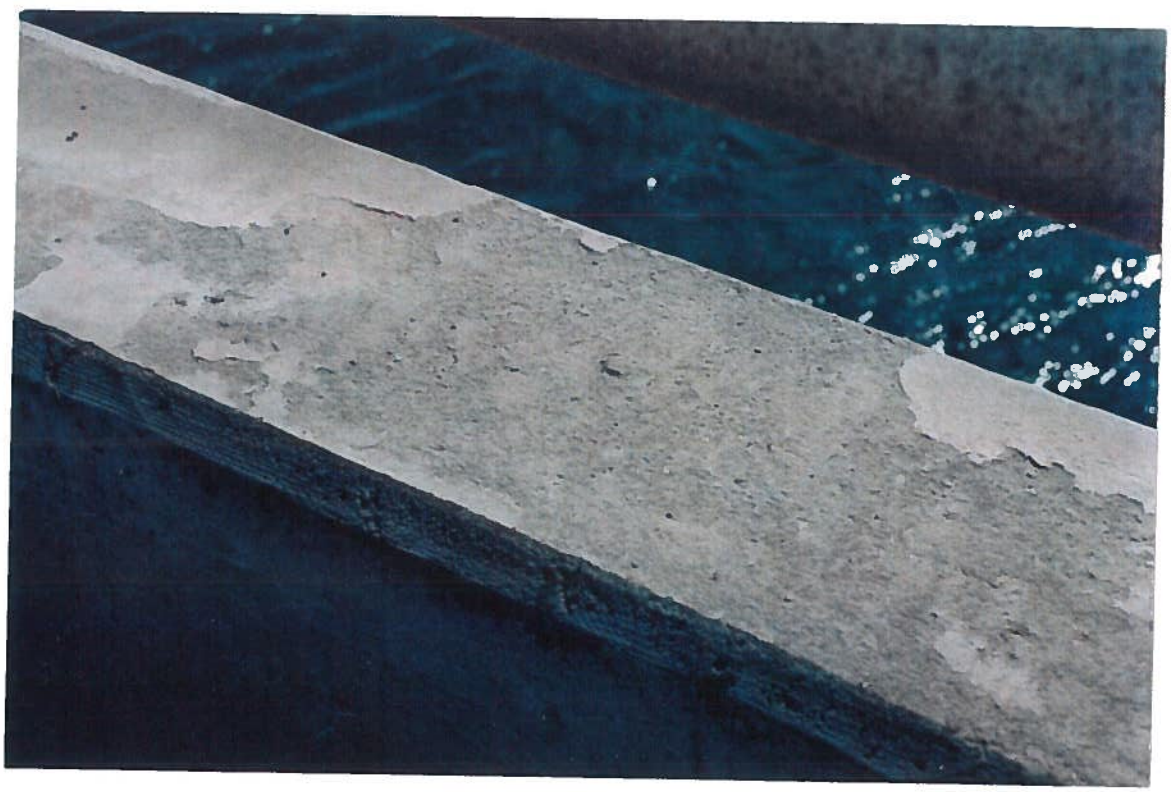
Cracks in precast panels at joints

Water at curbface and interior of box



Interior of box

Cracks in parapet close to joints.



Crack near joint and poor finishing.

FAXED Apr 22/98

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◆ Strait Crossing Inc. ◆

10 Horton Drive Charlottetown, PEI, C1A 7G3
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FAX COVER SHEET

DATE: April 22, 1998

COMPANY: Department of Transportation & Public Works

ATTENTION: Steve MacLean, P.Eng.

FAX NO.: 368-6244

FROM: Donald McGinn

TOTAL NUMBER OF PAGES:
(including cover sheet)

Original to follow? Yes No ✓

Re: Hillsborough Bridge Project- Surface Finish.

Dear Sir;

With regard to your inquiry of cracking of the recently poured concrete deck over top of the existing truss structure I can report the following:

1. The deck has been poured using silica fume concrete. The concrete is giving 60-65MPa strength, which far exceeds the 40MPa specified.

2. High performance silica fume concrete has a much lower permeability than normal concrete and also a higher electrical resistivity which makes it excellent for chloride resistance. Due to the phasing of the work, a membrane water proofing will be used nonetheless.

3. This concrete has a very low water/cement ratio, making curing critical. The curing used has been achieved by the application of a monomolecular curing compound within minutes of screeding and tarping of the deck at the end of the day (once the pour is completed and the concrete has set enough to support the tarps).

   Water cure has not been considered desirable because of the cool evening weather. Typical convention would consider that when the ambient temperature is less than 10°C curing is not required because the rate of surface evaporation is minimal.

4. Cracking of the concrete deck is minimal.

5. "Tears" within the concrete surface have been noted. Tearing of the concrete from station 180 to station 210 of the Stratford end (pour #1) and from station 40 to station 60 of the Charlottetown end (pour #2).

The tears in the concrete consist of an opening of the surface of the concrete while it is still plastic. The openings are generally 0.75mm to 2.0mm wide and extend 400-600mm in a transverse direction. There is approximately 50 tears in pour #1 and 8 tears in pour #2 (each pour was 493m^2 deck).

6. We have experienced similar tears in the concrete while constructing the Confederation Bridge. The openings have been found to extend to the top layer of reinforcement only. They are closely related to curing practice.

7. The tears are being gravity filled with Sikadur 52 epoxy in accordance with work procedure WP-A03 Concrete Repairs, which was developed during construction of the Confederation Bridge.

We have reviewed our procedures to see if we can further limit the extent of tearing encountered. We have considered the following:

1. The flow of wind may be responsible, in part. Nothing practical can be done to break the wind.
2. The deck will be covered at an earlier age.
3. The concrete placement temperature will be reduced from 18°C to 15°C.

We trust the above addresses your inquiry sufficiently. Please call if you require further details.

Yours truly,

Strait Crossing Inc.

Donald McGinn, M.Sc., P.Eng.
Project Manager

cc: S. Peach
P. Colucci

This message is intended for the use of the Addressee only.
If you have received this communication in error, please notify Alison immediately by telephone at (902) 569-1566. Thank You

HP OfficeJet LX
Personal Printer/Fax/Copier

Fax Log Report for
Strait Crossing Inc.
902-569-1820
Apr-22-98 09:56 AM

Identification	Result	Pages	Type	Date	Time	Duration	Diagnostic
STEVE MACLEAN	OK	02	Sent	Apr-22	09:54A	00:01:12	0024c5030022

7.4.0

Chapter 1 **Inspection of Box Girders and Bearings**

A visual inspection of the box girders was carried out by G. Strolz of CBCL Limited on 07 November 1997, in the company of Mr. McGinn, P.Eng., Project Manager and Mr. S. Pletch, both of Strait Crossing Inc.

The method of inspection was visual only and was carried out by walking through the interior of both boxes to view the field connections, rain water down pipes, underside of precast concrete deck and cast-in-place concrete closure strips. Exterior inspection of the boxes and the bearings was carried out from the existing truss structure.

The upstream side and the downstream side of the boxes could only be viewed from the approach causeways at each of the abutments.

1.1 Observations - Satisfactory

- All bolting of field splices is complete and appears to be satisfactory, no bolts missing.
- Downpipes of deck drains all installed and satisfactory.
- Underside of precast concrete deck panels are all satisfactory and only a few shrinkage cracks were observed.
- Cast-in-place closure strips all appear satisfactory and only minimal shrinkage cracking was noticed.
- Pier and abutment bearings appears to be satisfactory with the exceptions noted in "Observations - Action Required".
- Exterior surfaces are generally satisfactory with the exceptions noted in "Observations - Action Required".
- Patch of mill scale, approximately 250 mm square was noted on the exterior web of the upstream side box, located between the 9th and 10th field splice from the north abutment.

1.2 Observations – Action Required

- Clean floor of boxes at locations of field splices where debris has accumulated.
- Clean out small drain holes adjacent to the abutment and pier diaphragms for egress of water.
- Remove concrete splatter from the longitudinal floor and web stiffeners.
- Some greasy footprints were observed on the inside web of the upstream side box. Grease is to be removed from the surface of the web plate.

2.

- The rotational alignment of the bearings at the north abutment of the upstream side box are to be checked by SCI and reviewed with the bearing manufacturer.
- Minor rust spots have appeared local to the bearings at the field welds of the upstream side box at the south abutment. Clean surface and re-coat.
- The guide bar of the guided bearing on the upstream side box on Pier #2 is twisted within the guide slot. SCI to review the condition with the manufacturer of the bearings.
- The longitudinal alignment of the multi-directional bearing at Pier #2 of the downstream side box needs to be checked. Apparent excessive overhang of the top plate relative to the sliding surface.
- Remove a piece of temporary erection bracing cut for installation of the rain water down pipe at the south end of the upstream side box.

Chapter 2 Concrete Inspections

A visual inspection of completed work and work in progress was carried out by G. Strolz of CBCL Limited on 17 November 1997, in the company of Mr. D. McGinn, P.Eng., Project Manager and Mr. S. Pletch, both of Strait Crossing Inc.

The following observations were made:

- The concrete work on the piers is complete and appears to be satisfactory. Minor shrinkage cracking was observed but the cracks are inconsequential. No cracking was observed at the flared concrete section supporting the box girders.
- Abutment concrete work is complete (with the exception of the Phase II work) and appears to be satisfactory. The finished appearance of the concrete is very good.
- The deck slab, sidewalk and parapet concrete work on the upstream side box is complete. With the exception of some minor blemishes the surface finishes are acceptable. However, the vertical joints in the parapet are to be sawed through to provide a clear gap at each joint.
- On the downstream side box the concrete deck slabs, the cast-in-place infill sections and the sidewalk are complete and the work appears to be satisfactory. The parapet has yet to be completed.
- Work on the Reinforced Earth Wall System is complete at both abutments. Small differential panel movement was observed, but this is consistent with the system as it takes up the load due to earth pressure. The surface smoothness of the individual panels is good with the exception of some cracked corners, but the colouring of the panels is botchy.
- 2" x 4" timber blocking between cast-in-place concrete abutments and the reinforced earth wall panels are to be removed.

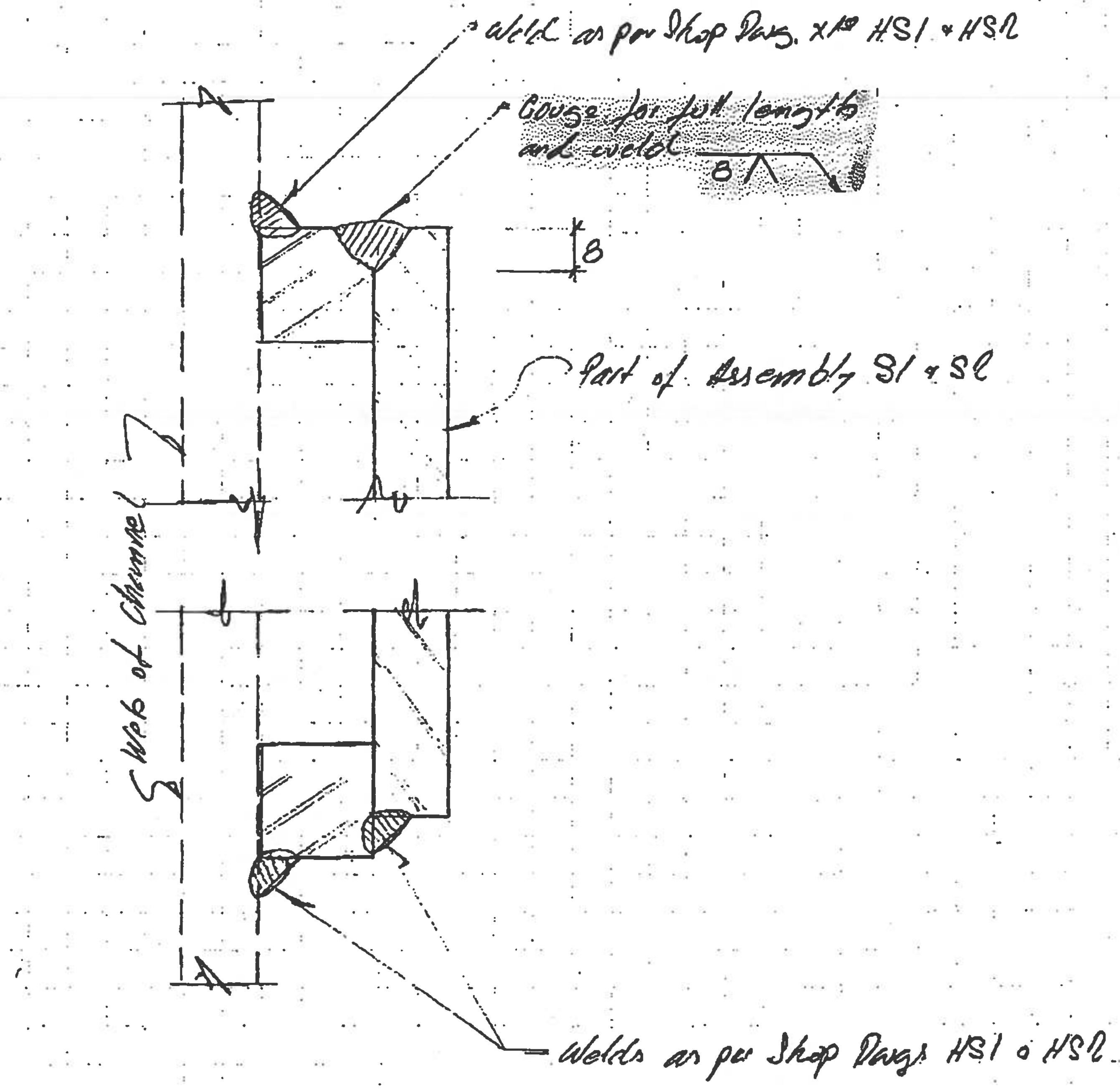
Chapter 3 **Inspection of Truss Strengthening**

A visual inspection of the work in progress of truss strengthening was carried out by G. Strolz of CBCL Limited on 07 November 1997, in the company of Mr. D. McGinn, P.Eng., Project Manager for SCI.

Work on reinforcing truss members is in progress and appears to be generally satisfactory. A final visual inspection by CBCL Limited will be carried out upon completion of the work.

The following conditions requiring action were noted, namely:

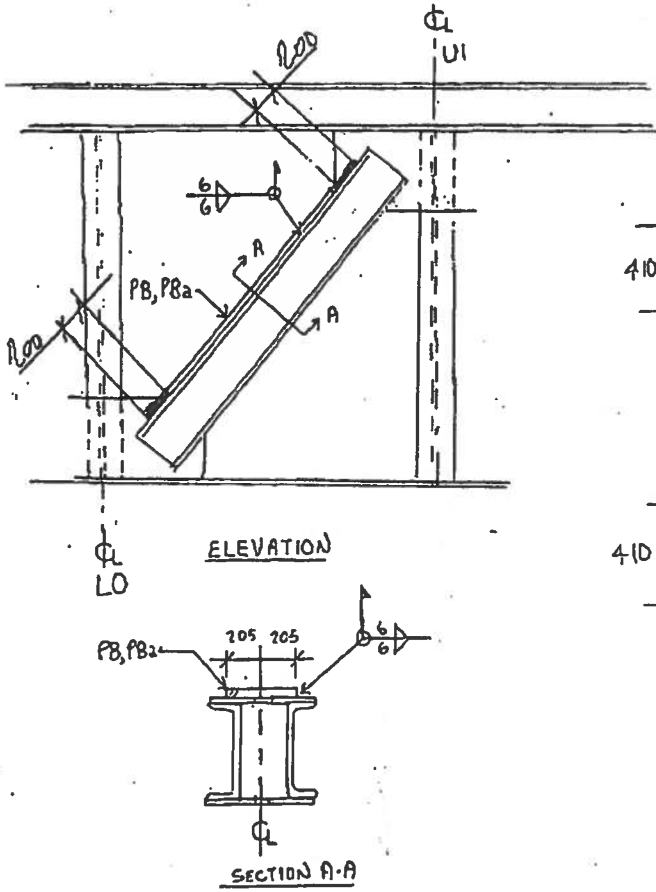
- Welding of Dywidag anchor assemblies to the bottom chord members of the truss is not in accordance with the shop drawings. The existing tack welds (spacer bar to anchor assembly) will require modification. Refer to facsimile transmittal of 17 November 1997, copy attached.
- At a number of locations the new reinforcing plates on the diagonals do not extend sufficiently into the gusset plate joints. Plates are to extend 200 mm into the joint as agreed on site, 11 September 1997, copy attached.
- The coverplate with saddle (refer to detail 5/25 on drawing 25 and shop drawing HS4) as installed is of insufficient size to cover the slotted hole at the bottom of the vertical truss members. Additional plating will be required.
- Caulking between the existing perforated plates on the diagonals and the new reinforcing plates is to be examined at all such locations to ensure a complete seal.



Attachment of Assembly S1 and S2
to Bottom Chord Channels

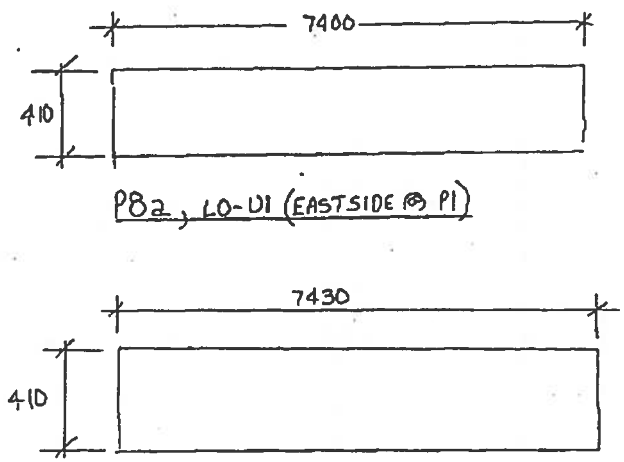
MATERIAL:	WELDING PROCEDURE:	PAINT:	NOTES:
CAN/CSA-G40.21-M92 300W	CSA W47 & W59	PRIMER DEVOE CATHA COAT 302 2.5-3.5 mils	

JOB DISC: HILLSBOROUGH BRIDGE PROJECT



ELEVATION

SECTION A-A



PBa, LO-U1 (EASTSIDE @ P1)

P8, LO-U1 (3 LOCATIONS)

Agreed on site 11/Sept/97
to extend our plates 200 mm
into gusset and weld to
gusset plates on each side
All reinforced PL's

PIECE MARKS	QTY	DESCRIPTION		LENGTH	WEIGHT	GRADE	REMARKS
PB	3	PL 16 x 410	(5/8" PLATE)	7430			
PBa	1	PL 16 x 410		7400			

DATE TO SHOP: PAGE NO: DETAILED: DATE:

REV.	DESCRIPTION	BY	DATE

◊SCI◊
STRAIT CROSSING INC.

DRAWN BY: JT DATE: JULY 15 1997 DRAWING: HS12

◆ Strait Crossing Inc. ◆

10 Horton Drive Charlottetown, PEI, C1A 7G3
Phone: (902) 569-1566 Fax: (902) 569-1820

FAX COVER SHEET

DATE: July 30, 1998

COMPANY: Department of T [illegible]

ATTENTION: Steve MacLean, P.Eng

FAX NO.: 368-6244

FROM: Steve Pletch

TOTAL NUMBER OF PAGES: 11
(including cover sheet)

Original to follow? Yes No ✓

Re: Hillsborough Bridge Project-Inspection Report

Steve,

Please find attached the bridge inspection report. We are using this report to identify all outstanding items required to reach total project completion. Should you have any comments concerning this report please direct them to the undersigned immediately.

Could you also please provide a letter of confirmation of concurrence that substantial completion has been achieved.

Kind regards,

Steve Pletch

Steve Pletch
Project Coordinator

cc: D. McGinn- SCL
G. Strolz- CBCL
P. Lockwood -CGY
B. Ferago

①
Guide-Bar failure, 2ND one in failure.
②. General - steel | Steel.
③. - bearings - Donnie /
- cracks. | CBCL satisfaction.
put in paper.
- Steel delivery slips. (Mexican Steelco?).
④ Fabricator should be brought in..

Hillsborough Bridge Improvement Project

Bridge Inspection Report

Strait Crossing Inc.

July 16, 1998

1. Introduction

An inspection of the Hillsborough Bridge was undertaken on July 12, 1998 with representatives of Strait Crossing Inc (Contractor) and CBCL (Engineer) on hand. The purpose of the inspection was to identify deficiencies (work still to be done, work done incorrectly, additional work required) in order to reach total completion of work.

Additionally, the DOT & PW project representative and an external, independent, review engineer contracted by DOT & PW were in attendance to discuss deficiencies and the work in general.

Attendance:

Gerry Strolz -	CBCL
Don McGinn -	SCI
Steve Pletch -	SCI
Steve MacLean-	DOT & PW
Bert Ferago -	Ferago Consultants

2. Marine Piers

An inspection of the piers was performed via boat. The following was noted.

.1 North face of Pier No. 2 has three cracks at water level which are wide enough to be injected with epoxy resin. The cracks originated from the shrinkage of the new pier around the old.

.2 Other shrinkage cracks were observed on both piers but were considered of no consequence.

.3 Previously injected cracks looked good.

.4 Stressing pockets looked tight.

.5 A few rust spots associated with form ties have come through. Although not of structural concern they should be chipped back and repaired.

.6 In general the piers look fine.

.7 Final settlement of piers is to be recorded. (survey)

.8 Re-caulk holes in sides of pier. (These holes are to be recorded on as-built drawings)

.9 Corbel tie beams to be painted.

.10 Some of the vertical post-tensioning pockets need their perimeter sealed with epoxy.

3. Abutment.

An inspection of the abutment area yielded the following improvements to be done.

1. Remove black tar stain from lower portion of SE abutment
2. Repair undercut (concrete washed out) in old (existing) abutment crossbeam.
3. Remove nails from abutment footings.
4. Landscape area so water drains away from truss bearing seats and pick up debris.
5. Remove silt fence.
6. Fence off access to bridge
7. Remove legs of old MFCI tower
8. Wash discoloration of side of abutment
9. Tidy tie-hole patching in new abutment
10. Clean off all bearing seats.
11. Chip concrete around footings of North abutment.
12. Fix undercut in concrete at seat of NE bearing of the truss.

4. Retaining Walls

There are two types of retaining structures. Both appear to be working correctly. Settlements have been noted which in accordance with the design.

The following work remains to be completed on the retaining walls

1 RECO Walls

1 Clean grout off face.

2 Resolve discoloration of face of panels.

3 Repair broken edges.

4 Construct concrete cap [illegible]

5 Erect rail at top of wall.

6 Bury all straps minimum of 6" deep.

7 Remove board stuck between RECO wall and abutment.

8 Add top soil above RECO wall and seed.

9 Remove boards behind abutment cross beam (in front of RECO wall).

2 Maccaferri Wall

1 Cut the grass

2 Ensure grass is growing in deficient areas

3 Plant [illegible]

4 Install [illegible] along top

4. New Steel Box Girder

The new steel box girders were carefully examined from the inside and outside. Performance and overall quality is excellent. The following notes were made.

.1 Outside

Some un-even weathering has occurred generally at locations where water has come down from the deck level. The girders should gradually become uniform in appearance.

The outside faces have not matured as much as [illegible] underside.

1) At Pier No. 3, east side, outer face of web has whitish patch. This may be grout.

2) Straps at [illegible] where [illegible] coating [illegible] has not weathered. This is not a problem.

3) [illegible] steel girder [illegible] to be [illegible] with a bolt with nut [illegible] flange. [illegible]

.2 Inside

1. Complete cleaning of each floor using a stiff broom (no wire brush) followed by a vacuum to be done.

2. All drainage holes and fillets at stiffeners etc. are to be checked to allow for drainage.

3. Conduit hole [illegible] ends which are not being used are to be screened.

4. Water may be entering girders [illegible] joints. Accelerated weathering has [illegible] some of the [illegible]. They are to be [illegible] of the joints should be considered.

5. Evidence of past [illegible] deck is apparent. Deck waterproofing has resolved this problem.

6. Water infiltration seems to be causing uneven weathering [illegible] inconsistency in [illegible] weathering.

[illegible] recommends [illegible] acceptable.

7. Electrical boxes at girder entrance [illegible]

6 Bearings

.1 Box Girder Bearings (new)

Similar problems with two bearings (guide bar) since the last inspection seems to be corrected.

1 Guide bar is twisted. [illegible] guided bearings east and west [illegible] at the [illegible] bolts. Bearing supplier (Watson Bowman [illegible]) [illegible]

2 Offsets of bearings to be recorded.

3 Bearings were recently repainted. The paint specification [illegible] to be [illegible]

.2 Truss Bearings (old)

1 Clean [illegible] and vacuum dust [illegible] bearing areas.

2 Inspect all bearings for signs [illegible]

3 Remove [illegible]

4 [illegible]

[illegible]

Strengthening of the steel truss was [illegible] of the steel truss. It was noted that additional weathering of the truss paint system has occurred since the project began.

1. [illegible] bracket [illegible] touch up paint
2. Replacement pieces to handrail to be painted
3. Misc. [illegible] fixtures and cable [illegible]
4. Miscellaneous [illegible] wire on lower chord [illegible] removed
5. [illegible]
6. [illegible] are as tight as they were
7. Finish fastening [illegible]
8. [illegible]
9. [illegible]
10. [illegible] to be painted
11. [illegible]

[illegible] Deck, Barriers, Sidewalks [illegible]

[illegible] deck was [illegible] the [illegible] and sidewalks were inspected.

[illegible]

[illegible]

[illegible]

[illegible]

[illegible]

[illegible]

[illegible]

[illegible]

Transmittal/Fax Cover Sheet

CBCL LIMITED
Consulting Engineers

1489 Hollis Street
PO Box 606
Halifax, Nova Scotia
Canada B3J 2R7

Telephone: 902 421-7241
Fax: 902 423-3938
E-mail: info@cbcl.ca
URL: http://www.cbcl.ca

FAXED BY
Initials

To: Strait Crossing Inc

Date: 02 Sept. 1998

No. of Pages (Incl. Cover): Four

Attn.: Steve Pletch / Don McGinn

From: Garry Sproule

Sent to Fax #: 569-1820 ( )

RE: Hillsborough River Bridge

Project No.: 94846-J

Message/Remarks:

Forwarding • Confirmation of Insurance Certificate for Errors and Omissions 1997 to 1998

• Diagram #1 ~ Box Truss Member Forces

We are a professional team working together, to provide quality services that satisfy our customers, and contribute to our mutual success.

We are sending by	the following	for	Kindly	Originals to be
☒ FAX	☐ Message	☐ Information	☐ Acknowledge	☐ Mailed
☐ Courier	☐ Prints	☐ Correction	☐ Resubmit	☐ Not Applicable
☐ Priority Post	☐ Specifications	☐ Action	☐ Return	☐ ________
☐ Mail	☐ Shop Drawing(s)	☐ Approval	☐ ________	
	☐ Copy of Letter	☐ ________		
	☐ Information			

ISO 9001
Registered Company

AON

Aon Reed Stenhouse Inc.

Insurance Brokers
Risk Consultants

To Whom It May Concern:

This letter will confirm that CBCL Limited, Consulting Engineers presently carry the following Liability coverages for the term December 31, 1997 to 1998:

Commercial General Liability Policy No. 60313598/A
Royal Insurance Company of Canada
Limit - $1,000,000.00 inclusive

Umbrella Liability Policy No. 6072774/A
Royal Insurance Company of Canada
Limit - $4,000,000.00 excess of $1,000,000.00 Primary

Professional Liability Policy No. L61100, Certificate No. TEN305611
Issued by Encon Managers Inc. On behalf of
Insurers: The Royal Insurance Company of Canada
Continental Casualty Company
Non-Marine Underwriters at Lloyd's under
Contract No. ENC5-97

Limit $3,000,000.00 - per claim
$3,000,000.00 - aggregate

Policy includes Pollution Liability Endorsement for an annual Aggregate sublimit of $2,000,000.00.

We trust the above will suffice for your file.

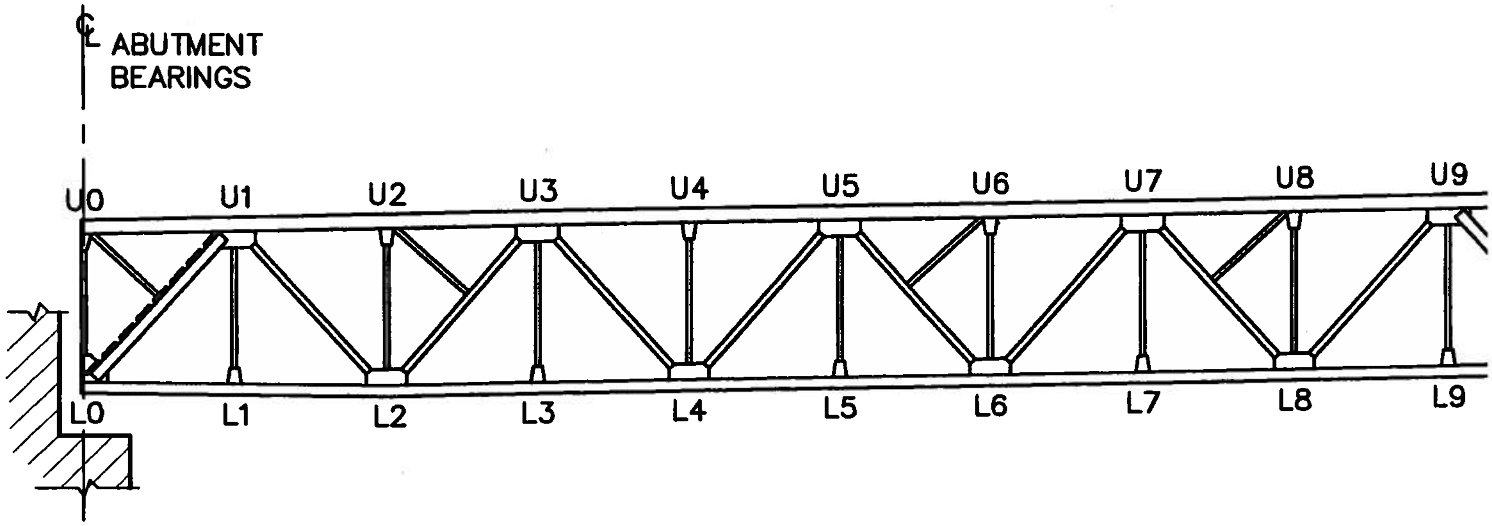
Yours sincerely,

C. Sampson

Colleen M. Sampson, F.I.I.C.
Vice President

CS/cds
w:\let\rep\cbclcmp.doc

P. O. Box 1010 • Purdy's Wharf Tower I, Suite 701
1959 Upper Water Street • Halifax, Nova Scotia B3J 3N2 • tel: (902) 429-7310 • fax: (902) 429-9087



TOP CHORD

MEMBER REFERENCE	C/T	FACTORED LOAD FORCE (kN)	FACTORED LOAD MOM. (kN–m)	CAPACITY FACTOR	CAPACITY CRITERION
$U_0 - U_1$	C	4	471	0.41	STABILITY
$U_1 - U_3$	C	4444	353	0.93	–
$U_3 - U_5$	C	6512	326	0.96	–
$U_5 - U_7$	C	6235	361	0.95	–
$U_7 - U_9$	C	3619	476	0.94	–
$U_9 - U_{11}$	T	3360	515	0.87	STRENGTH
$U_{11} - U_{13}$	T	9621	842	0.99	STRENGTH
$U_{13} - U_{15}$	T	3486	514	0.92	STRENGTH
$U_{15} - U_{17}$	C	2571	505	0.88	STABILITY
$U_{17} - U_{19}$	C	4813	401	0.93	STABILITY

BOTTOM CHORD

MEMBER REFERENCE	C/T	FACTORED LOAD FORCE (kN)	FACTORED LOAD MOM. (kN–m)	CAPACITY FACTOR	CAPACITY CRITERION
$L_0 - L_2$	T	2566	–	–	–
$L_2 - L_4$	T	5791	–	–	–
$L_4 - L_6$	T	6698	–	–	–
$L_6 - L_8$	T	5239	–	–	–
$L_8 - L_{10}$	T	1527	36	–	–
	C	1207	36	0.47	STABILTY
$L_{10} - L_{12}$	C	6110	204	0.98	–
$L_{12} - L_{14}$	C	6218	205	0.99	–
$L_{14} - L_{16}$	C	1552	33	0.59	–
	T	692	33	–	–
$L_{16} - L_{18}$	T	4011	–	–	–
$L_{18} - L_{19}$	T	5135	–	–	–

DIAGONALS

MEMBER REFERENCE	C/T	FACTORED LOAD FORCE (kN)	FACTORED LOAD MOM. (kN–m)	CAPACI FACTOR
$L_0 - U_1$	C	3856	40	0.99
$U_1 - L_2$	T	2908	12	0.91
$L_2 - U_3$	C	2180	12	0.88
$U_3 - L_4$	T	1273	9	0.43
$L_4 - U_5$	C	511	7	0.50
	T	297	6	0.13
$U_5 - L_6$	C	1093	6	0.60
$L_6 - U_7$	T	1847	11	0.60
$U_7 - L_8$	C	2764	17	0.93
$L_8 - U_9$	T	3460	18	0.97
$U_9 - L_{10}$	C	4405	65	0.98
$L_{10} - U_{11}$	T	5109	56	0.99
$U_{11} - L_{12}$	C	5918	109	0.97
$L_{12} - U_{13}$	C	5651	110	0.99
$U_{13} - L_{14}$	T	4833	54	0.89
$L_{14} - U_{15}$	C	4146	70	0.97
$U_{15} - L_{16}$	T	3179	13	1.00
$L_{16} - U_{17}$	C	2520	19	0.92
$U_{17} - L_{18}$	T	1567	6	0.83
$L_{18} - U_{19}$	C	888	7	0.83

C = COMPRESSION
T = TENSION

MEMBER CAPACITY FACTOR = FACTORED LOADS /

MEMBER CAPACITIES ARE CALCULATED ON A REDU
ALLOW FOR PRESENT LEVEL OF CORROSION AND E
CORROSION LOSS OVER 30 YEARS

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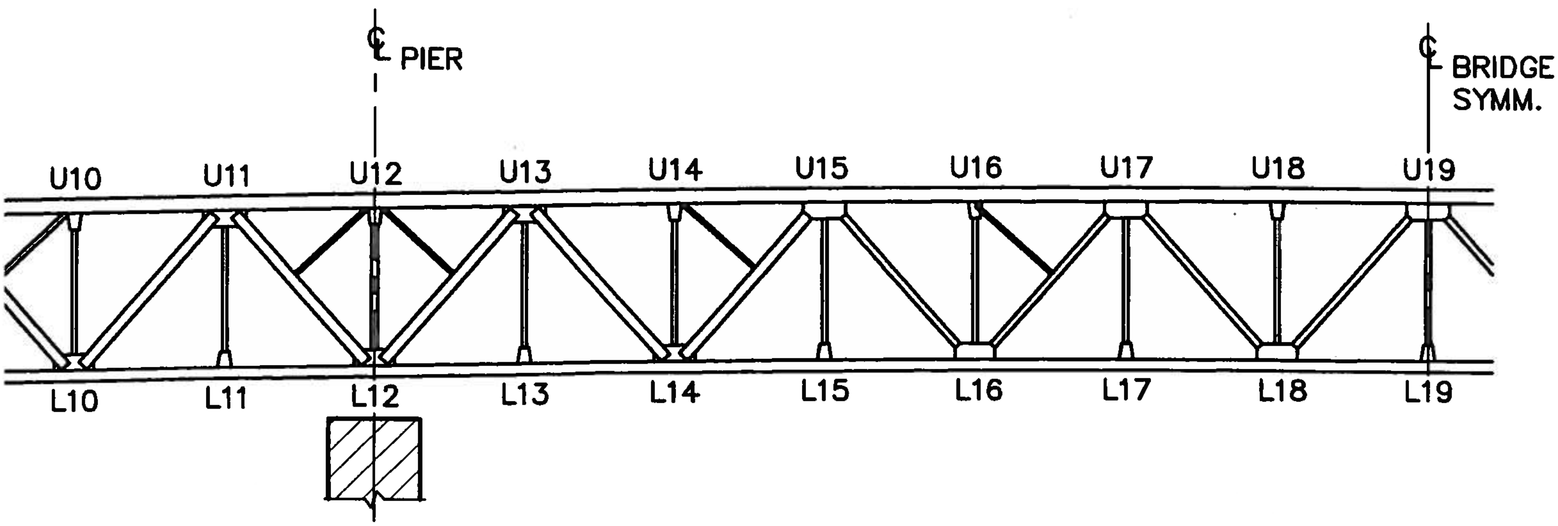

CAPACITY CRITERION
STABILITY
STRENGTH
STABILITY
STRENGTH
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STRENGTH
STABILITY
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STRENGTH
STABILITY
STRENGTH
STABILITY

VERTICALS

MEMBER REFERENCE	C/T	FACTORED LOAD		CAPACITY FACTOR	CAPACITY CRITERION
		FORCE (kN)	MOM. (kN–m)		
$L_0 - U_0$	C	434	15	0.52	STABILITY
$L_1 - U_1$	T	13	5	0.11	STRENGTH
$L_2 - U_2$	C	654	11	0.73	STABILITY
$L_3 - U_3$	T	18	3	0.07	STRENGTH
$L_4 - U_4$	C	669	3	0.67	STABILITY
$L_5 - U_5$	T	20	1	0.03	STRENGTH
$L_6 - U_6$	C	667	5	0.69	STABILITY
$L_7 - U_7$	T	20	3	0.09	STRENGTH
$L_8 - U_8$	C	649	10	0.71	STABILITY
$L_9 - U_9$	T	21	5	0.12	STRENGTH
$L_{10} - U_{10}$	C	655	11	0.73	STABILITY
$L_{11} - U_{11}$	T	20	6	0.13	STRENGTH
$L_{12} - U_{12}$	C	873	3	0.68	STABILITY
$L_{13} - U_{13}$	T	22	6	0.15	STRENGTH
$L_{14} - U_{14}$	C	645	11	0.72	STABILITY
$L_{15} - U_{15}$	T	22	5	0.12	STRENGTH
$L_{16} - U_{16}$	C	637	12	0.72	STABILITY
$L_{17} - U_{17}$	T	25	5	0.12	STRENGTH
$L_{18} - U_{18}$	C	644	6	0.68	STABILITY
$L_{19} - U_{19}$	T	22	1	0.02	STRENGTH

IMATE MEMBER STRENGTH

SECTION SIZE TO
ATED FUTURE

Diagram No. 1

Owner:
Prince Edward Island
Department Of Transportation
And Public Works
Charlottetown, P.E.I.

Client:
Hillsborough Bridge
Development Inc.
Charlottetown, P.E.I.

Manager:
Strait Crossing Inc.
Calgary, Alberta

Engineer:
CBCL
CBCL Limited
Consulting Engineers
Halifax, Nova Scotia

HILLSBOROUGH RIVER BRIDGE
IMPROVEMENT

BOX TRUSS MEMBER FORCES

Scale	Designed	Date
AS NOTED	G. STROLZ	JUNE 1995
Checked A. PERRY	Drawn W. MORROW	Contract No 94846
Approved	Revision △	Drawing No —
Sheet of –/–		

Jacques Whitford
Consulting Engineers
Environmental Scientists

3 Spectacle Lake Drive
Dartmouth, Nova Scotia
Canada B3B 1W8
Tel: 902 468 7777

Facsimile Transmission

Company	Attention	Fax Number
SCI	Don McGinn	902 569 1820
cc. CBCL	Gerry Strolz	902 423 3938
cc. JWA	George Zafiris	902 566 2004

If not well received please call (902) 468-0431.
We are transmitting a total of 1 pages, including this page.

FROM: John D. Brown **DATE:** 18 July 1997 **OUR REF:** 10271

RE:- Hillsborough River Bridge

I have reviewed the rock core and the pile load test sent to me in recent days, both from Pier No. 3, and have the following summary comments.

Rock Core

Core was obtained from Holes 12 and 31. The rock is sandstone, generally very good quality with good recovery and high RQDs. In each hole there is a minor mudseam less than 3 inches thick. We are performing unconfined compression tests for confirmation, but I am sure that the strengths will be very acceptable in terms of our objectives.

Pile Load Test, Pile No 6.

The test was taken to 282 kips (1254kN) with near linear behaviour and about 0.5 inches (13 mm) permanent set after unloading. This is similar to the tests on Pier No. 2. For the stated bond length of 6 m, the average bond stress at maximum load was 90 psi (630kPa). These are encouraging results.

Regards

PS. I will be in Newfoundland 21-22 July. If you wish to contact me, please try 709-228-2600.
jdbdmg03.fax

Jacques Whitford
Consulting Engineers
Environmental Scientists

3 Spectacle Lake Drive
Dartmouth, Nova Scotia
Canada B3B 1W8
Tel: 902 468 7777
Fax: 902 468 9009

Facsimile Transmission

Company	Attention	Fax Number
CBCL	Gerry Strolz	
cc. JWA Charlottetown	George Zafiris	

If not well received please call (902) 468-0431.
We are transmitting a total of 2 pages, including this page.

FROM: John D. Brown **DATE:** 17 July 1997 **OUR REF:** 10271

RE:- Hillsborough Bridge, Pier No. 3

Further to our meeting on 14 June, this summarizes various points regarding the GEWI piles.

1. As per Don McGinn's letter, installation difficulties encountered by Dywidag will probably result in not more than 24 GEWI piles being installed at this pier, and SCI have asked JWA and CBCL to review the design implications.

2. The Gewi piles will be #18 bar @ grade 75 (75 ksi yield stress). Dywidag's published design values are as follows:

   Yield stress, f_y = 1335 kN
   Maximum allowable bar load = 0.6 x f_y = 801 kN [Is this a working stress or ULS value? I suspect it is for working stress method of analysis.]

3. The drill used to drill the rock has a bit diameter of 105 mm, hence the hole diameter for the socket design is 105 mm.

4. The geotechnical capacity of the socket for the #18 bar should be made equal to the *maximum allowable bar load*, to achieve the most efficient design.

5. The geotechnical capacity is governed by the bond capacity which can be developed between the socket wall and the grout. This is calculated using an empirical equation which relates it to the unconfined strength of the rock. Our earlier calculations assumed that the rock strength was 15 MPa, and we computed an ultimate bond capacity of 770 kPa. On the basis of 10 unconfined tests on the rock from Pier No. 2, the average strength is 25 MPa, which could

permit us to make some increase in bond capacity. However, there may not be a need to invoke this, as you will see from the next items.

6. The factored socket capacity is given by

$$Q = \text{bond capacity} \times \text{resistance factor} \times \text{hole circumference} \times \text{length}$$

$$Q = 770 \times 0.5 \times 3.14 \times 0.105 \times L$$

$$Q = 127 \times L$$

The length should be selected so that the factored capacity equals the allowable bar load

$$L = 801/127 = 6.3 \text{ m}$$

The drilled length should exceed this by an allowance for weathered rock at the interface (currently judged to be less than 0.5 m) and overdrilling to allow for some debris accumulation in the bottom of the socket (say 0.5 m). Thus the nominal drilled length should be 7.3 m. Provided this length is achieved, the limit on the Gewi pile capacity is governed by yield of the bar itself, and we can't do anything about that.

7. The tests performed at Pier No. 2 resulted in a maximum bond stress of 500 kPa, with no evidence of failure. This is less than the ultimate capacity of 770 kPa given above, but more than the factored bond resistance of 385 kPa. This gives some comfort.

8. Pile deformation is usually computed at the service load. Maybe this needs to be discussed further. Based on the tests at Pier No. 2, settlement of the pile caused by rock deformation due to the service load can not be calculated very closely, but it seems that it will be less than about 13 mm. In addition, the pile will undergo elastic shortening for the length from the cap to the rock. Treating it as a grout-encased steel bar with lateral support, carrying a service load of 600 kN, I have computed about 18 mm compression for a 25-m bar length. Thus settlements under a service load of 600 kN would be in the region of 30 mm.

We need to discuss this further, but I think that this implies some load transfer to the 30 inch piles to maintain strain compatibility.

9. On a related matter, I have just finished examining the core taken from holes 12 and 31, Pier No. 3, and I am pleased that the rock is a good sandstone with only a few mud seams less than 3 inches thick.

Jacques Whitford
Consulting Engineers
Environmental Scientists

3 Spectacle Lake Drive
Dartmouth, Nova Scotia
Canada B3B 1W8
Tel: 902 468 7777
Fax: 902 468 9009

Facsimile Transmission

70626
GEWI PILES

Company	Attention	Fax Number
CBCL	Gerry Strolz	
cc. JWA Charlottetown	George Zafiris	

If not well received please call (902) 468-0431.
We are transmitting a total of 2 pages, including this page.

FROM: John D. Brown **DATE:** 17 July 1997 **OUR REF:** 10271

RE:- Hillsborough Bridge, Pier No. 3

Further to our meeting on 14 June, this summarizes various points regarding the GEWI piles.

1. As per Don McGinn's letter, installation difficulties encountered by Dywidag will probably result in not more than 24 GEWI piles being installed at this pier, and SCI have asked JWA and CBCL to review the design implications.

2. The Gewi piles will be #18 bar @ grade 75 (75 ksi yield stress). Dywidag's published design values are as follows:

   Yield stress, f_y = 1335 kN
   Maximum allowable bar load = 0.6 x f_y = 801 kN [Is this a working stress or ULS value? I suspect it is for working stress method of analysis.]

3. The drill used to drill the rock has a bit diameter of 105 mm, hence the hole diameter for the socket design is 105 mm.

4. The geotechnical capacity of the socket for the #18 bar should be made equal to the *maximum allowable bar load*, to achieve the most efficient design.

5. The geotechnical capacity is governed by the bond capacity which can be developed between the socket wall and the grout. This is calculated using an empirical equation which relates it to the unconfined strength of the rock. Our earlier calculations assumed that the rock strength was 15 MPa, and we computed an ultimate bond capacity of 770 kPa. On the basis of 10 unconfined tests on the rock from Pier No. 2, the average strength is 25 MPa, which could

permit us to make some increase in bond capacity. However, there may not be a need to invoke this, as you will see from the next items.

6. The factored socket capacity is given by

$$Q = \text{bond capacity} \times \text{resistance factor} \times \text{hole circumference} \times \text{length}$$

$$Q = 770 \times 0.5 \times 3.14 \times 0.105 \times L$$

$$Q = 127 \times L$$

The length should be selected so that the factored capacity equals the allowable bar load

$$L = 801/127 = 6.3 \text{ m}$$

The drilled length should exceed this by an allowance for weathered rock at the interface (currently judged to be less than 0.5 m) and overdrilling to allow for some debris accumulation in the bottom of the socket (say 0.5 m). Thus the nominal drilled length should be 7.3 m. Provided this length is achieved, the limit on the Gewi pile capacity is governed by yield of the bar itself, and we can't do anything about that.

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8. Pile deformation is usually computed at the service load. Maybe this needs to be discussed further. Based on the tests at Pier No. 2, settlement of the pile caused by rock deformation due to the service load can not be calculated very closely, but it seems that it will be less than about 13 mm. In addition, the pile will undergo elastic shortening for the length from the cap to the rock. Treating it as a grout-encased steel bar with lateral support, carrying a service load of 600 kN, I have computed about 18 mm compression for a 25-m bar length. Thus settlements under a service load of 600 kN would be in the region of 30 mm.

We need to discuss this further, but I think that this implies some load transfer to the 30 inch piles to maintain strain compatibility.

9. On a related matter, I have just finished examining the core taken from holes 12 and 31, Pier No. 3, and I am pleased that the rock is a good sandstone with only a few mud seams less than 3 inches thick.

70626

◆ Strait Crossing Inc. ◆

10 Horton Drive Charlottetown, PEI, C1A 7G3
Phone: (902) 569-1566 Fax: (902) 569-1820

FAX COVER SHEET

DATE: **July 9, 1997**

COMPANY: Jacques Whitford and Associates

ATTENTION: **John Brown**

FAX NO.: **1-902-468-9009**

FROM: Donald McGinn

TOTAL NUMBER OF PAGES: 7
(including cover sheet)

Original to follow? No

RE: Hillsborough River Bridge – GEWI Pile Installation – Pier No. 3

Dear Mr. Brown,

Attached is an update sheet of progress to date of installing GEWI piles at Pier No.3. Only 5 piles have successfully been installed to date.

I spoke with Gerry Strolz regarding progress and the fact that there is now locations that piles can not be installed because of steel within the caisson or because of casing stuck which has had to abandoned. Gerry clarified that there is no point in installing more in one quadrant than another; ie if there is a quadrant that six piles can be installed, then the total piles should be six per quadrant, or 24 in total. This is less than originally requested.

We discussed if the 24 piles would be sufficient. One of the item for consideration is the existing 30" diameter piles and if we have obtained further information about them that increases our level confidence concerning them and their capacity. The extra information we now have is:

a) The caisson does in fact have a large gravel layer (5 to 7m thick) and the decision not to count on the caisson to transfer vertical loads and instead install the 30" diameter piles was probably wise.
b) Samples of bedrock have been recovered indicating relatively sound sandstone with a couple of mudstone streaks. This is probably close to, maybe slightly better than anticipated. What are results of unconfined compressive tests? Does this lead confirm anticipated pile capacity?
c) Pieces of steel sheet pile have been recovered. They seem to confirm that corrosion rates are very low at these depths. This should also be true for the pile jacket. (see attached Marenco Report regarding corrosion)

d) Settlements associated with pouring the concrete of the footing and pier stem (approx 250cm of concrete, or 6.25MN of weight) have been less then 1mm.

Attached is an excerpt from a letter in 1995 regarding the large diameter pile capacity. Could you please comment further now that we have obtained some additional information. At the same time as this, Gerry will be reconsidering the loadings, with the clarification of ice loadings, to see if any improvement in required capacity may be obtained.

This issue is extremely serious to the project. We are at the point now of possible delay because of the difficulties encountered. If the work of installing the 24 GEWI piles is for not, that an additional form of strengthening will be required, then we need to make that call immediately. The effects on other stages of the work (steel girder erection, etc) has serious consequences.

Perhaps we could determine to proceed with what we hope to obtain, recognizing that there exists a risk, albeit reduced, that there may be some settlement at some point in time that would require resetting and realignment of the bridge bearings.

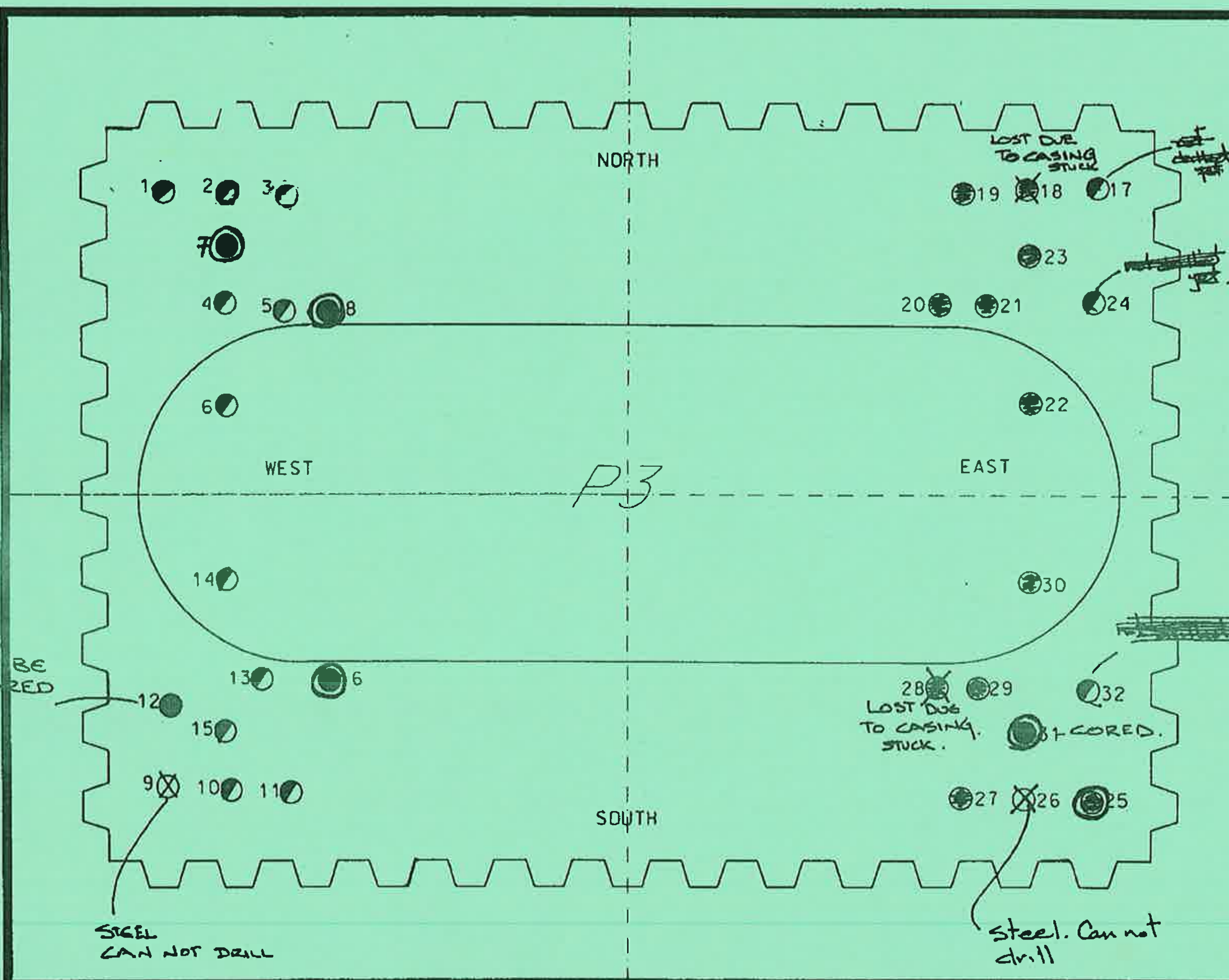
Kind Regards,

Donald McGinn, P.Eng
Project Manager

cc. S Pletch
P. Lockwood/ G. Tadros
G. Strolz – CBCL
George Zafiris 566-2004

DRILLED 1/2 WAY
#18 BAR INSTALLED

UPDATED
JULY 8/97.



◊SCI◊
STRAIT CROSSING INC.
HILLSBOROUGH BRIDGE PROJECT

DRAWING TITLE:
PIER #3 GEWIS NUMBERING SYSTEM

PROJECT AREA : HILLSBOROUGH BRIDGE	CONTRACT :

SCALE: N.T.S.

	DAY	MTH	YEAR
SURVEYED :			
DRAWN :	17	06	97
APPROVED :			
DRAWING NO. :		REV.	

70626

◆ Strait Crossing Inc. ◆

10 Horton Drive Charlottetown, PEI, C1A 7G3
Phone: (902) 569-1566 Fax: (902) 569-1820

FAX COVER SHEET

DATE: **June 25, 1997**

COMPANY: Jacques Whitford and Associates

ATTENTION: **George Zafiris**

FAX NO.: **566-2004**

FROM: Donald McGinn

TOTAL NUMBER OF PAGES: 2
(including cover sheet)

Original to follow? No

RE: Hillsborough River Bridge – GEWI Pile Installation

George,

Attached is a tentative schedule which indicates coring operations tentatively scheulded for Saturday, June 28 and Wednesday, July 2nd. There was some confusion over logs on the last pier cores so it may be beneficial if you had someone present or at least stop by twice each day during the operation.

Please keep in contact with Mr. Pat Colucci at 569-1566 regarding when the core recovery is scheduled.

Kind Regards,

Donald McGinn, P.Eng
Project Manager

cc. S Pletch
D. Cote
P. Colucci

GEWI PILE SCHEDULE

JOB	TUE 24	WED 25	THU 26	FRI 27	SAT 28	SUN 29	MON 30	TUE 1	WED 2	THU 3	FRI 4	SAT 5	SUN 6	MON 7
	D N	D N	D N	D N	D N	D N	D N	D N	D N	D N	D N	D N	D N	D N
6" DTH (REBAR)	6	5	5	1										
CONSOLIDATE GROUT			7	4										
LB SYSTEM	[illegible]													
CORED HOLE					1			1	1					
INSTALL + GROUT					3	4	5	3	3	5	4	3		
TEST								1	1	1	1	1		1
MOVE DRILL			✓					✓				✓		
LOW SLACK TIDE				11a.m	12noon	1:00pm	2:00pm	3:00	4:00	5:00	6:00			

[illegible] - DS.
June 24/97 △

This assumes no major problems.

△ Ammended June 25/97
7:00 a.m.

◆ Strait Crossing Inc. ◆

10 Horton Drive Charlottetown, PEI, C1A 7G3
Phone: (902) 569-1566 Fax: (902) 569-1820

FAX COVER SHEET

DATE: May ~~9,~~ 1997 June 25

COMPANY: Jacques Whitford and Associates

ATTENTION: **John Brown**

FAX NO.: **1-902-468-9009**

FROM: Donald McGinn

TOTAL NUMBER OF PAGES: ~~25~~ 25
(including cover sheet)

Original to follow? No

RE: Hillsborough River Bridge – GEWI Pile Installation – Pier No. 3

Dear Mr. Brown,

We have resumed drilling operations at Pier 3.

Attached are drilling and grouting logs. In general, we are encountering a gravel/sand layer between 17 to 24 from the drill deck. The sand is dense enough that consolidation grouting is not successful – the grout does not penetrate the layer. There have been a couple of voids encountered also.

It is our understanding following our discussion yesterday, that the sand/gravel layer will still be able to provide the lateral support required to the GEWI pile; in other words, the GEWI pile concept is still valid.

The inability to consolidate grout the sand gravel layer results in the requirement to install GEWI piles through the casing and withdraw the casing while grouting. This has led to the difficulty that the drill rig is not capable of drilling a large enough hole.

We have ordered #18 Grade 75 bars to be used as GEWI piles for this pier (in lieu of the #20 bars). The bar will be installed in a 105mm diameter hole. The hole will be lengthened to 10.0m thus keeping the bond stress the same as at Pier No.2. (1100/1400)x (145/105) x 9.2m= 10.0m

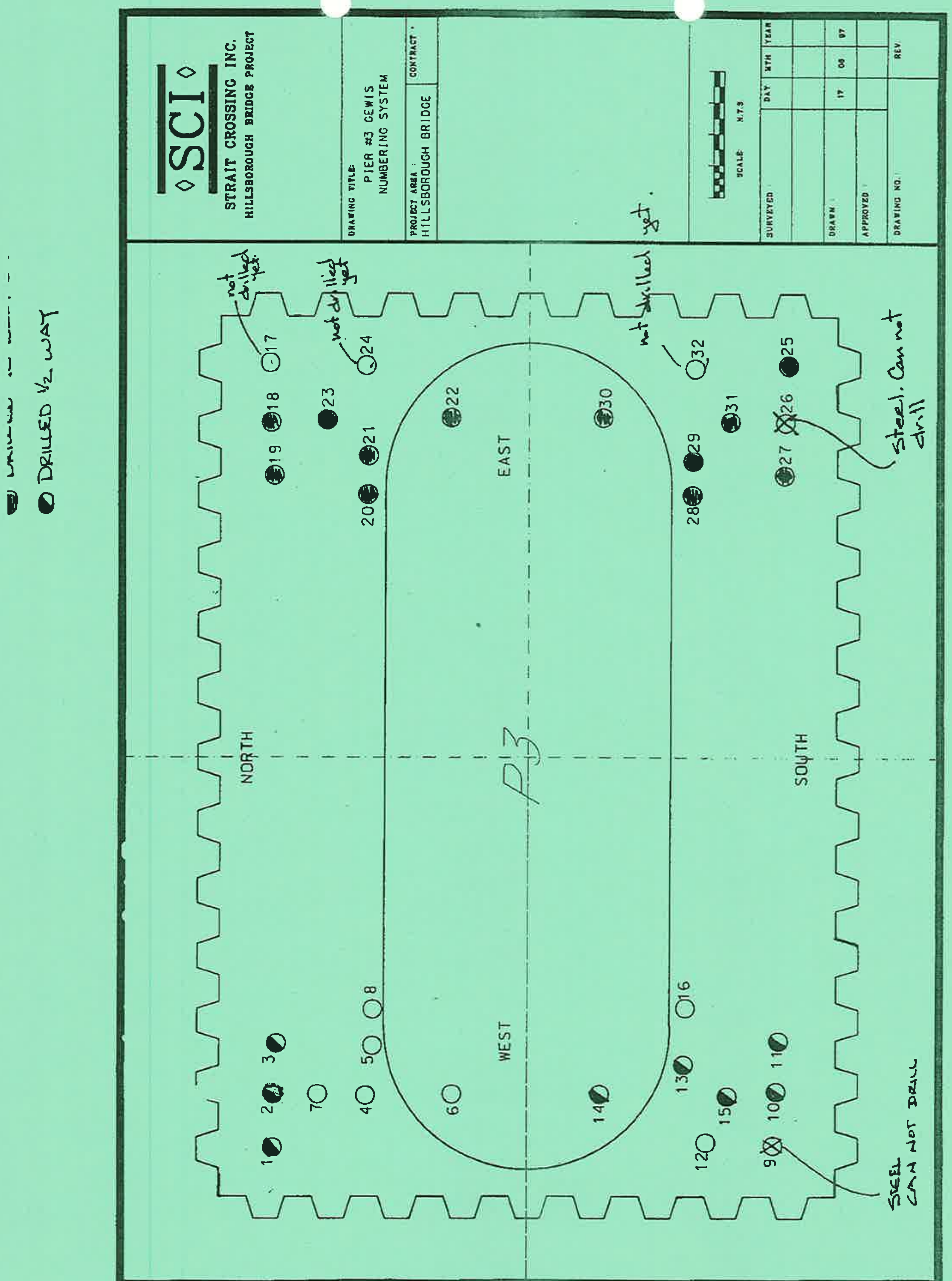
Regarding capacities, the original 900kN per pile was increased to 1400kN based upon a higher capacity bar and longer bond area. The #18 grade 75 bars have a capacity of about 1100 kN. To date two holes have had to be abandoned because of steel. We planning to install all that can be installed (ie potentially 30 of the 32 piles). We will keep you informed of progress and/or any changes in conditions.

I trust we have informed you sufficiently. Call immediately if you have any concerns.

Kind Regards,

Donald McGinn, P.Eng
Project Manager

cc. S Pletch -cover - Pat
P. Lockwood/ G. Tadros – fax cover
G. Strolz – CBCL
George Zafiris 566-2004


DRILLED 1/2 WAY
SCI
STRAIT CROSSING INC.
HILLSBOROUGH BRIDGE PROJECT
DRAWING TITLE:
PIER #3 GEWIS
NUMBERING SYSTEM
CONTRACT
PROJECT AREA:
HILLSBOROUGH BRIDGE
SCALE N.T.S
SURVEYED
DAY MTH YEAR
DRAWN 17 06 97
APPROVED
DRAWING NO.
REV
not drilled yet.
not drilled yet
not drilled yet
not drilled yet.
steel, Can not drill
STEEL CAN NOT DRILL
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SOUTH
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1.0.2
Read

Hillsborough Bridge Improvements
DSI/GEWI Pile Meeting No. 2
Monday June 23, 1997
Page 1

Attendance:	Sheldon Kapeller-DSI	S. Pletch
	Terry Fright- DSI	D. McGinn
Distribution:	P. Colucci	D. Cote

1. Hole 28 has 12m of casing stuck.
   - Need to jack it out. Jacking equipment is being mobilized.

2. All other holes are 1 1/2m into rock by casing.

3. At 17.5m open hole, holes are collapsing when being re-drilled open with the open hole method.

4. No voids in the South East quadrant.

5. North East had inter-connected layers/voids at 15m.

6. Grout take is low. Only slightly more than theoretical. 0.55 w/c to try to get it to travel.

7. Coupler is 4" dia. Drilling with casing 145mm O.D.. Inner bit is only 105mm 0. Therefore need to use into borehole. There is risk of getting struck in the hole.

8. Other alternative is to run 'stratex' system. 5 1/2" I.D.
   - A. In Vancouver.
   - B. It is 5 days away at least.
   - C. It is more casing.

9. If we case there is a good chance of losing the casing. If we lose it, we lose the hole and $18,000 of casing.

10. Gravel goes from 17m – 24m.

11. Option is to use #18 bar instead of #20 bar. The couplers will be smaller. May need more piles .

12. To go to double shift system DSI has enough personnel here. SCI would need to provide 2 helpers each shift. A night boatman would also be required. On this basis, production should be about 4 holes per day.

DYWIDAG THREADBARS - TECHNICAL DATA [IMPERIAL UNITS]

[STATUS: April, 1997, DSI, Surrey, BC]

STEEL GRADE f_y/f_u lb/in², [ksi]		NOMINAL DIAMETER inch		STEEL AREA A_s in²	YIELD LOAD $P_y=f_y{*}A_s$ kip	ULT. LOAD $P_u=f_u{*}A_s$ kip	UNIT WT. lb/ft	MAX BAR Ø inch	IMPACT VALUES [CHARPY -NOTCH] °F	ft-lb
120/150 ksi	**ASTM A722-90**	1.00	1"	0.85	102.00	127.50	3.01	1.201	-9.00	6.00
Hot-rolled, Proof Stressed & Stress Relieved Post-tensioning Threadbar		1¼	1¼"	1.25	150.00	187.50	4.39	1.457	-20.00	5.00
PT Ground Anchors	Post-Tensioning	1⅜	1⅜"	1.58	189.60	237.00	5.56	1.630		
60/90 ksi	**ASTM A615-87**	¾	#6	0.44	26.40		1.50	0.862		
Hot-rolled Reinforcing Threadbar		⅞	#7	0.60	36.00		2.04	0.996		
Tie Rods		1.00	#8	0.79	47.40		2.67	1.122		
Hanging Rods, Gewi-Piles		1⅛	#9	1.00	60.00		3.40	1.268	-15.00	3-7
Anchor Bolts, Concrete Reinforcing		1¼	#10	1.27	76.20		4.30	1.433		
Seismic Anchors, Rock Bolts		1⅜	#11	1.56	93.60		5.31	1.614		
Ground Anchors, Soil Nailing		1¾	#14	2.25	135.00	ORIGINAL DESIGN	7.65	1.862		
Precast Connections		2¼	#18	4.00	240.00		13.60	2.504		
75/100 ksi	**ASTM A615-87**	¾	#6	0.44	33.00		1.50	0.862		
Hot-rolled Reinforcing Threadbar		1.00	#8	0.79	59.30		2.67	1.122	-15.00	3-7
		1⅜	#11	1.56	117.00	PIER 3	5.31	1.614		
[SAME AS ABOVE]		2¼	#18	4.00	300.00		13.60	2.504		
80/106 ksi		2½	#20	4.91	393.00	PIER 2.	16.70	2.72		
73/80 ksi	**ASTM A706**	1.00	25T	0.76	55.30		2.58	1.120		
Quenched & Tempered Weldable Threadbar		1⅛	28T	0.95	69.20		3.24	1.265	-4.00	100.00
Gewi-Piles, Seismic Anchors		1¼	32T	1.25	90.40		4.23	1.430	-40.00	20.00
Concrete Reinforcing, Seismic Restrainers		1$^{9}/_{16}$	40T	1.95	141.20		6.62	1.745		
Cold Region Anchors		2.00	50T	3.00	220.80		10.34	2.200		
130/160 ksi	**Hot-Rolled Threadbar (HT)**	⅝"	⅝"	0.27	35.10	43.20	0.99	0.693	-4.00	7-15
Form-ties	Soil Nailing	¾"	¾"	0.49	63.70	78.40	1.74	0.900	-40.00	7.00
115/130 ksi	**Cold-rolled Threadbar (DRC)**	⅝"	⅝"	0.29	33.70	38.20	0.98	0.688		
Form-ties	Soil Nailing	⅞"	⅞"	0.52	60.70	67.50	1.68	0.864		
70/87 ksi	**DSI-MAI**	R25N 12 mm core	1"		34.00	45.00	1.74	1.000		
Hollow-core Injection Anchors		R32N 18 mm core	1¼"		51.70	63.00	2.35	1.250		
		R32S 15 mm core	1¼"		63.00	81.00	2.82	1.250		
Shoring, Soil Nailing		R38N 19 mm core	1½"		96.70	112.00	4.03	1.500		
Micro - Pile, Installation Without Casing		R51N 34 mm core	2"		141.60	180.00	6.46	2.00		

pproximate Modulus of Elasticity: E=205,000 MPa

pproximate Relaxation Values: Gr. 413/620, Gr. 500/550 & Gr. 517/690: 7% @ 0.6 f_y & 9% @ 0.9 f_y

Gr. 835/1030, Gr. 900/1100: 2.6% @ 0.6 f_u

or permanent anchors, please inquire about our Double Correction Protection system

1-02
Read.

Hillsborough Bridge Improvements
DSI/GEWI Pile Meeting No. 3
Tuesday June 24, 1997
Page 1

Attendance:	S. Pletch D. McGinn	Terry Fright- DSI
Distribution:	P. Colucci	D. Cote

1. Hole # 28 has casing stuck. Value of casing is $800/length x 12 lengths. Jacking equipment is being mobilized.

2. Stratex system is still being tracked down.

3. Geotechnical Engineer will accept # 18 bar instead of a # 20 bar. Just more bars will be required.

4. # 18 bars to be ordered immediately.

5. Availability of extra casing to be tracked down.

RECEIVED
JUN 23 1997
HBDI
CHARLOTTETOWN, P.E.I.

DISTRIBUTION	HBDI	I	A	R
FM				
CM				
ENG				
PURCH				
ACCT				
SUP				
D. McGinn			X	
S. Platek			X	
READ				
3.020.12.0				

DYWIDAG SYSTEMS INTERNATIONAL
GROUT REPORT
PROJECT HILLS BOROUGH BRIDGE P.E.I. (PIER # 3)

DATE	ANCHOR #	BAR LENGTH METERS	BOND LENGTH METERS	DRILL LENGTH METERS	BORE HOLE	GROUT ZONE METERS	TREMIE GROUT (BAGS)	TOP UP (BAGS)	NO. OF BAGS USED	REMARKS
JUNE 16/97	A31	CONSOLIDATE		17.3 M	6 1/8"	8.7 M	14		14	THEORICAL VOLUME 5BAGS
	A 23	CONSOLIDATE		19 M	6 1/8"	10.7 M	34		34	THEORICAL VOLUME 6 BAGS
										GROUT LEVEL CAME UP 1 FT. BIG VOID
JUNE 17/97	A 23	CONSOLIDATE		19 M	6 1/8"	10.7 M	10			GROUT LEVEL NOT COMING UP
							20			ADD 27 BAGS OF SAND LEVEL CAME UP 3 FT.
							5		35	ADD 10 BAGS OF SAND LEVEL CAME UP 1 FT.
	A 27	CONSOLIDATE		12 M	6 1/8"	3.4 M	6		6	THEORICAL VOLUME 2 BAGS
	A 25	CONSOLIDATE		17.3 M	6 1/8"	8.7 M	15		15	THEORICAL VOLUME 5BAGS
	A 28	CONSOLIDATE		17.3 M	6 1/8"	8.7 M	7		7	THEORICAL VOLUME 5BAGS
JUNE 18/97	A 23	CONSOLIDATE		16.7 M	6 1/8"	8.2 M	24		24	ADD 18 BAGS OF SAND LEVEL CAME UP TO 8.5 M
	A 30	CONSOLIDATE		14.4 M	6 1/8"	5.8 M	5		5	THEORICAL VOLUME 3.5 BAGS
										Grout level 7.8m
JUNE 19/97	A 23	CONSOLIDATE		27.7 M	145MM	27.7M	9			THEORICAL VOLUME 8 BAGS
		CASING SEATED AT 24 M				24 - 22		1		
START 12:20 PM						22 - 20		1		
								50 PSI 3 BAG		
						20 - 18		7		GROUT AT 30 FT.
						18 - 16		6		GROUT AT 28 FT.
			FINISH 1:20 PM			16 - 14		0		GROUT AT 28 FT. PULL ALL CASING OUT
								TOTAL	27 BAGS	
										4:00 PM CHECK GROUT LEVEL AT 50 FT.
										PUMP 4BAGS GROUT LEVEL AT 32 FT.

DYWIDAG SYSTEMS INTERNATIONAL
GROUT REPORT
PROJECT HILLS BOROUGH BRIDGE P.E.I. (PIER # 3)

DATE	ANCHOR #	BAR LENGTH METERS	BOND LENGTH METERS	DRILL LENGTH METERS	BORE HOLE	GROUT ZONE METERS	TREMIE GROUT (BAGS)	TOP UP (BAGS)	NO. OF BAGS USED	REMARKS
JUNE 19/97	A 25	CONSOLIDATE								
START 3:20 PM				27.7 M	145MM	27.7M	9			THEORICAL VOLUME 9.5 BAGS
CASING SEATED AT 24 M						24 - 22		0		
								120 PSI .5 BAG		
						22 - 20		0.5		
								100 PSI 1 BAG		
						20 - 18		1		
								50 PSI 4 BAGS		
						18 - 16		0		GROUT LEVEL AT 28 FT.
			FINISH AT 4:10 PM			16 - 14		0		GROUT LEVEL AT 26 FT. PULL ALL CASING OUT
							TOTAL	15 BAGS		Grout level 9.6 m.
JUNE 19/97	A 29	CONSOLIDATE								
START 6:00 PM				27.7 M	145MM	27.7M	9			THEORICAL VOLUME 8 BAGS
CASING SEATED AT 24 M						24 - 22		2		
								90 PSI 3.5 BAGS		
						22 - 20		1		
								90 PSI 1 BAG		
						20 - 18		1		
								90 PSI 1.5 BAGS		
						18 - 16		3		
								50 PSI 5 BAGS		
						16 - 14		0		GROUT LEVEL AT 15 FT.
						14 - 12		0		GROUT LEVEL AT 17 FT. PULL ALL CASING OUT
							TOTAL	27 BAGS		Grout level 8.4 m.
JUNE 19 /97	A 22	CONSOLIDATE								
START 1:35 PM				17 M	6 1/8"	8.4 M	44		44 BAGS	THEORICAL VOLUME 5BAGS
										GROUT LEVEL AT 28 FT.
JUNE 19 /97	A 20	CONSOLIDATE		15 M	6 1/8"	6.4 M.	21		21 BAGS	THEORICAL VOLUME 3.5 BAGS
										GROUT LEVEL AT 28 FT.
										GROUT TRAVELED TO A19 AND A 18

DYWIDAG SYSTEMS INTERNATIONAL

GROUT REPORT

PROJECT HILLS BOROUGH BRIDGE P.E.I. (PIER # 3)

DATE	ANCHOR #	BAR LENGTH METERS	BOND LENGTH METERS	DRILL LENGTH METERS	BORE HOLE	GROUT ZONE METERS	TREMIE GROUT (BAGS)	TOP UP (BAGS)	NO. OF BAGS USED	REMARKS
JUNE 19 /97	A 18	CONSOLIDATE		15 M	6 1/8"	6.4 M.	3		3 BAGS	GROUT TRAVELED FROM A 20
										GROUT LEVEL AT 28 FT.
JUNE 19 /97	A 19	CONSOLIDATE		15 M	6 1/8"	6.4 M.	1		1 BAG	GROUT TRAVELED FROM A 20
										GROUT LEVEL AT 28 FT.
JUNE 20/97	A 27	CONSOLIDATE								
				27.7 M	145MM	27.7 M	9			THEORICAL VOLUME 9.5 BAGS
CASING SEATED AT 24 M						24 - 22		2		
								90 PSI .5 BAG		
						22 - 20		0.5		
								90 PSI .5 BAG		
						20 - 18		0.5		
								90 PSI .5 BAG		
						18 - 16		0.5		
								90 PSI .5 BAG		
						16 - 14		0		GROUT LEVEL AT 15 FT.
						14 - 12		0		GROUT LEVEL AT 18 FT.
								TOTAL	14 BAGS	Grout level 9.5m.
JUNE 20/97	A 30	CONSOLIDATE		27.7 M	145MM	27.7 M	9			THEORICAL VOLUME 9.5 BAGS
CASING SEATED AT 25 M						25 - 24		1.5		
						24 - 22		1.5		
								90 PSI 2 BAGS		
						22 - 20		1		
								90 PSI 1 BAG		
						20 - 18		0.5		
								90 PSI 1.5 BAGS		
						18 - 16		0		GROUT LEVEL AT 5 FT.
						16 - 14		0		GROUT LEVEL AT 8 FT.
						14 - 12		0		GROUT LEVEL AT 10 FT. PULL ALL CASING OUT
								TOTAL	18 BAGS	Grout level 7.8m

DYWIDAG SYSTEMS INTERNATIONAL
GROUT REPORT
PROJECT HILLS BOROUGH BRIDGE P.E.I. (PIER # 3)

DATE	ANCHOR #	BAR LENGTH METERS	BOND LENGTH METERS	DRILL LENGTH METERS	BORE HOLE	GROUT ZONE METERS	TREMIE GROUT (BAGS)	TOP UP (BAGS)	NO. OF BAGS USED	REMARKS
JUNE 20/97	A 18	CONSOLIDATE		28.8 M	145MM	28.8 M	9			THEORICAL VOLUME 8.5 BAGS
						22 -20 M		1		
CASING SEATED AT 22 M								90 PSI .5 BAG		
						20 - 18 M		1.5		
								90 PSI .5 BAG		
						18 - 16		1		
								90 PSI 2 BAGS		
						16 - 14		0		GROUT LEVEL AT 8 FT.
						14 - 12		0		GROUT LEVEL AT 10 FT. PULL OUT ALL CASING
								TOTAL	15.5 BAGS	
JUNE 20/97	A 20	CONSOLIDATE		27.8 M	145MM	27.8 M	8			THEORICAL VOLUME 8.5 BAGS
						22 - 20		1		
CASING SEATED AT 22 M						20 -18		0.5		
								90 PSI .5 BAG		
						18 - 16		0.5		
								90 PSI 2 BAGS		
						16 - 14		0.5		
								90 PSI .5 BAG		
						14 - 12		0		GROUT LEVEL AT 5 FT. PULL ALL CASING OUT
								TOTAL	13.5 BAGS	

DYWIDAG SYSTEMS INTERNATIONAL
GROUT REPORT
PROJECT HILLS BOROUGH BRIDGE P.E.I. (PIER # 3)

DATE 1997	ANCHOR #	BAR LENGTH METERS	BOND LENGTH METERS	DRILL LENGTH METERS	BORE HOLE mm	GROUT ZONE METERS	TREMIE GROUT (BAGS)	TOP UP (BAGS)	NO. OF BAGS USED	REMARKS
JUNE 21	A 21	CONSOLIDATE		27.8	145	27.8	8 1/2			CASING SEATED AT 22 m.
						20 - 22		1/2		
						18 - 20		1/2		
								press 1/2	~~90psi~~	90 psi
						16 - 18		1/2 + 1/2	~~100~~	100 psi
						14 - 16		1/2 + 0	~~120~~	120 psi
									12 bags	GROUT LEVEL Theor 10 bags
JUNE 21	A 31	CONSOLIDATE		26.0	145	26.0	9			CASING SEATED AT 24 m.
						22 - 24		4		
								5		90 psi
						20 - 22		1/2		
								1/2		90 psi
						18 - 20		1/2		
								1/2		90 psi
									20 bags	Theoretical 10 bags. Grout level 9.3 m
JUNE 21	A 28	CONSOLIDATE							14 bags	CASING SET AT 25 M
										PULLED 1 M, 24 m STUCK IN HOLE
										Tremie grouted twice and flushed out twice.
JUNE 22	A 19	CONSOLIDATE		27.8	145	0 - 27.8	8			
						22 - 24		1		
						20 - 22		1/2		
								1/2		90 psi
						18 - 20		1		
								1		90 psi
						16 - 18		1/2	12 1/2	Theoretical 8 bags

DYWIDAG SYSTEMS INTERNATIONAL
GROUT REPORT
PROJECT HILLS BOROUGH BRIDGE P.E.I. (PIER # 2)

DATE 1997	ANCHOR #	BAR LENGTH	BOND LENGTH	DRILL LENGTH METERS	BORE HOLE	GROUT ZONE METERS	TREMIE GROUT (BAGS)	TOP UP (BAGS)	NO. OF BAGS USED	REMARKS
JUNE 22	22	CONSOLIDATE		8.6 - 24	145					
				24.0-27.8	108		8			
						22 - 24		1/2		
						20 - 22		1		
								1/2		90 psi
						18 - 20		1 1/2		
								1/2		90 psi
						16 - 18		1/2		
								1/2	13	90 psi Theoretical 8 bags.

DSI DYWIDAG SYSTEMS INTERNATIONAL

DRILL LOGS

PROJECT **HILLSBOROUGH BRIDGE**	JOB #	ANCHOR # **A 18**
GENERAL CONTRACTOR **STRAIT CROSSING INC.**	BORE HOLE DIA. **6 1/8"** DRILL RIG TYPE **KANI**	ANGLE OF HOLE FROM HORIZONTAL **90**
DRILLER **CHRIS KAPELLER**	DRILL METHOD **DTH HAMMER**	DRILL FLUID **AIR** DEPTH OF HOLE

DATE	TIME	DEPTH	STRATIGRAPHY	REMARKS
JUNE 18/97		0 - 7.6 M	OPEN PIPE	
START 6:15 PM		7.6 - 8.6 M	GRAVEL BAG	
		8.6 - 9.6 M	CONCRETE (DENSE)	
		9.6 - 9.7 M	STEEL	
		9.7 - 11.6 M	CONCRETE (LESS DENSE)	
		11.6 - 11.7 M	STEEL	
		11.7 - 11.8M	CONCRETE (LESS DENSE)	
		11.8 - 11.9 M	STEEL	
		11.9 - 15.5 M	OLD GROUT	
		15.5 - 16M	VOID	LOTS OF WATER
		6:45PM	**STOP DRILLING**	
		REDRILL		
JUNE 20/97		**133 CASING SYSTEM**		
START 12:50 PM		0 - 9 M	OPEN HOLE	
		9 - 15 M	GROUT	
		15 - 16 M	SAND AND GRAVEL	
		16 - 17 M	GROUT	
		17 - 20 M	SAND AND GRAVEL	
		20 - 27.3 M	CONCRETE AND GRAVEL	CASING SEATED AT 22M
FINISH 3:30 PM		27.3 - 28.8 M	SAND STONE (RED)	

DSI DYWIDAG SYSTEMS INTERNATIONAL

DRILL LOGS

PROJECT	JOB #	ANCHOR # **A 19**
HILLSBOROUGH BRIDGE	BORE HOLE DIA. **6 1/8"**	ANGLE OF HOLE FROM
GENERAL CONTRACTOR	DRILL RIG TYPE **KANI**	HORIZONTAL **90**
STRAIT CROSSING INC.	DRILL METHOD	DRILL FLUID **AIR**
DRILLER **CHRIS KAPELLER**	**DTH HAMMER**	DEPTH OF HOLE

DATE	TIME	DEPTH	STRATIGRAPHY	REMARKS
JUNE 18/97				
START 4:30 PM		**0 - 8.2 M**	**OPEN PIPE**	
		8.2 - 8.6 M	**GRAVEL BAG**	
		8.6 - 9.5 M	CONCRETE (DENSE)	
		9.5 - 9.6 M	CONCRETE (LESS DENSE)	
		9.6 - 9.7 M	**STEEL**	
		9.7 - 11 M	CONCRETE (LESS DENSE)	
		11 - 11.6 M	CONCRETE AND GRAVEL	
		11.6 - 11.8 M	**STEEL**	
		11.8 - 12 M.	CONCRETE AND GRAVEL	
		12 - 12.1 M	**STEEL**	
		12.1 - 15M	OLD GROUT	
		15 - 15.5 M	VOID	LOTS OF WATER
		5:50 PM	**STOP DRILLING**	
JUNE	22/97	0 - 8.6m	OPEN	133 system
START	8-30am	8.6 - 15.0	GROUT	
		15.0 - 19.0	GRAVEL + SAND W/ WATER	
		19.0 - 20.0	CONCRETE (LESS DENSE)	
		20.0 - 27.3	CONCRETE (DENSE)	CASING SEATED AT 22M.
		27.3 - 28.8	RED SDST.	

DSI DRILL LOGS SYSTEMS INTERNATIONAL

PROJECT **HILLSBOROUGH BRIDGE**	JOB #	ANCHOR # **A 20**
GENERAL CONTRACTOR **STRAIT CROSSING INC.**	BORE HOLE DIA. **6 1/8"** DRILL RIG TYPE **KANI**	ANGLE OF HOLE FROM HORIZONTAL **90**
DRILLER **CHRIS KAPELLER**	DRILL METHOD **DTH HAMMER**	DRILL FLUID **AIR** DEPTH OF HOLE

DATE	TIME	DEPTH	STRATIGRAPHY	REMARKS
JUNE 18 /97		0 - 8.4 M	OPEN PIPE	
START 2:55 PM		8.4 - 8.8 M	GRAVEL BAG	
		8.8 - 9.6 M	CONCRETE (DENSE)	
		9.6 - 9.7 M	STEEL	
		9.7 - 10 M	CONCRETE (LESS DENSE)	
		10 - 10.5 M	CONCRETE (DENSE)	
		10.5 - 11.6 M	CONCRETE (LESS DENSE)	
		11.6 - 11.7 M	STEEL	
		11.7 - 11.9 M	CONCRETE AND GRAVEL	
		11.9 - 13 M	CONCRETE (LESS DENSE)	
		13 - 14.5M	CONCRETE AND GRAVEL	
		14.5 - 14.8 M	GRAVEL	HOLE COLAPSING
		3:50 PM	CAN'T DRILL ANY FURTHER	
		REDRILL		
JUNE 20/97		133 CASING SYSTEM		
START 5:30 PM		0 - 9 M	OPEN HOLE	
		9 - 14.5 M	GROUT	
		14.5 - 19.5 M	SAND AND GRAVEL	
		19.5 - 26.6 M	CONCRETE AND GRAVEL	CASING SEATED AT 22 M
FINISH 6:30 PM		26.6 - 27.8 M	SAND STONE (RED)	

DSI DYWIDAG SYSTEMS INTERNATIONAL

DRILL LOGS

PROJECT **HILLSBOROUGH BRIDGE**	JOB #	ANCHOR # **A 21**
GENERAL CONTRACTOR **STRAIT CROSSING INC.**	BORE HOLE DIA. **6 1/8"**	ANGLE OF HOLE FROM HORIZONTAL **90**
	DRILL RIG TYPE **KANI**	DRILL FLUID **AIR**
DRILLER **CHRIS KAPELLER**	DRILL METHOD **DTH HAMMER**	DEPTH OF HOLE

DATE	TIME	DEPTH	STRATIGRAPHY	REMARKS
JUNE 19/97				
START 7:15 AM		**0 - 8.2 M**	**OPEN PIPE**	
		8.2 - 8.5 M	**GRAVEL BAG**	
		8.5 - 9.7 M	**CONCRETE (DENSE)**	
		9.7 - 11 M	**CONCRETE (LESS DENSE)**	
		11 - 11.7 M	**CONCRETE (DENSE)**	
		11.7 - 11.8 M	**STEEL**	
		11.8 - 11.9 M	**CONCRETE AND GRAVEL**	
		11.9 - 13.3 M	**CONCRETE (LESS DENSE)**	Ca:
		9:55 AM	**STOP DRILLING**	
June 21				
START 7:15 a.m		133 start		
		13.3 - 15	DENSE CONCRETE	
		15 - 16	SAND & GRAVEL	
		16 - 17	DENSE CONCRETE	
		17 - 20	SAND & GRAVEL	
		20 - 26.6	DENSE CONCRETE	CASING SEATED AT 22m
		26.6 - 27.8	RED SANDSTONE	

DSI DYWIDAG SYSTEMS INTERNATIONAL

DRILL LOGS

PROJECT **HILLSBOROUGH BRIDGE**	JOB #	ANCHOR # **A 22**
GENERAL CONTRACTOR **STRAIT CROSSING INC.**	BORE HOLE DIA. **6 1/8"**	ANGLE OF HOLE FROM HORIZONTAL **90**
DRILLER **CHRIS KAPELLER**	DRILL RIG TYPE **KANI**	DRILL FLUID **AIR**
	DRILL METHOD **DTH HAMMER**	DEPTH OF HOLE

DATE	TIME	DEPTH	STRATIGRAPHY	REMARKS
JUNE 18/97				
START 2:00 PM		0 - 8.4 M	OPEN PIPE	
		8.4 - 8.8 M	GRAVEL BAG	
		8.8 - 10 M	CONCRETE (DENSE)	
		10 - 10.5 M	CONCRETE (LESS DENSE)	
		10.5 - 10.7 M	CONCRETE AND GRAVEL	
		10.7 - 11.6 M	CONCRETE (LESS DENSE)	
		11.6 - 11.7 M	STEEL	
		11.7 - 13 M	CONCRETE (LESS DENSE)	
		13 - 16 M	CONCRETE AND GRAVEL	POSSIBLE VOID AT 15 M.
		16 - 17 M	GRAVEL	HOLE COLAPSED TO 16 M.
		2:30 PM	CAN'T DRILL ANY FURTHER	
JUNE	22/97			
START	11.10a.m	0 - 8.6m	OPEN	133 system
		8.6 17.0	GROUT	
		17.0 - 20.0	GRAVEL SAND w/ WATER	
		20.0 - 22.5	CONCRETE (LESS DENSE)	
		22.5 - 26.6	CONCRETE (DENSE)	CASING SEATED AT 24M
		26.6 - 27.8	RED SDST	

DSI DYWIDAG SYSTEMS INTERNATIONAL

DRILL LOGS

PROJECT **HILLSBOROUGH BRIDGE**	JOB #	ANCHOR # **A 23**
	BORE HOLE DIA. **6 1/8"**	ANGLE OF HOLE FROM HORIZONTAL **90**
GENERAL CONTRACTOR **STRAIT CROSSING INC.**	DRILL RIG TYPE **KANI**	
	DRILL METHOD	DRILL FLUID **AIR**
DRILLER **CHRIS KAPELLER**	**DTH HAMMER**	DEPTH OF HOLE

DATE	TIME	DEPTH	STRATIGRAPHY	REMARKS
JUNE 16/97				
START 10:00 AM		0 - 8.2 M	OPEN PIPE	
		8.2 - 9.2 M	GRAVEL BAG	
		9.2 - 11.5 M	CONCRETE (DENSE)	
		11.5 - 11.6 M	STEEL	
		11.6 - 12.5 M	CONCRETE AND STEEL	
		12.5 - 13 M	CONCRETE (LESS DENSE)	
		13 - 15 M	CONCRETE (DENSE)	
		15 - 15.5 M	CONCRETE AND GRAVEL	HOLE COLAPSING
		15.5 - 15.7 M	CONCRETE AND GRAVEL	AIR BLOWING INTO WATER
		15.7 - 19 M.	CONCRETE AND GRAVEL	LOTS OF GRAVEL AND AIR BLOW INTO
		12:00 PM	**CAN'T DRILL ANY FURTHER**	WATER.
JUNE 19/97		**REDRILL**		
START 7:35 AM		0 - 8.3 M	OPEN HOLE	
		8.3 - 14.6 M.	GROUT	
		14.6 - 15.3 M	SAND AND GROUT	
		8:05 AM	**STOP DRILLING**	
JUNE 19/97		**REDRILL**		
START 10:00 AM		**133 CASING SYSTEM**		
		0 - 15 M	OPEN HOLE	
		15 - 18.5 M	SAND AND GROUT	
		18.5 - 20 M	SAND AND GRAVEL	
		20 - 22 M	CONCRETE (LESS DENSE)	
		22 - 25.6 M	SAND AND GRAVEL	
		25.6 - 26.6M	CONCRETE AND GRAVEL	CASING SEATED AT 24 M
		26.6 - 27.7M	SAND STONE (RED)	
FINISH 11:10 AM				

DSI DYWIDAG SYSTEMS INTERNATIONAL

DRILL LOGS

PROJECT **HILLSBOROUGH BRIDGE**	JOB #	ANCHOR # **A 25**
GENERAL CONTRACTOR **STRAIT CROSSING INC.**	BORE HOLE DIA. **6 1/8"**	ANGLE OF HOLE FROM HORIZONTAL **90**
DRILLER **CHRIS KAPELLER**	DRILL RIG TYPE **KANI**	DRILL FLUID **AIR**
	DRILL METHOD **DTH HAMMER**	DEPTH OF HOLE

DATE	TIME	DEPTH	STRATIGRAPHY	REMARKS
JUNE 17/97				
START 2:15 PM		0 - 8.3 M	OPEN PIPE	
		8.3 - 9 M	GRAVEL BAG	
		9 - 10 M	CONCRETE (DENSE)	
		10 - 10.3 M	CONCRETE AND GRAVEL	HOLE COLAPSE
		10.3 - 13 M	CONCRETE (LESS DENSE)	
		13 - 14.5 M	CONCRETE AND GRAVEL	HOLE COLAPSE
		14.5 - 17 M	CONCRETE (LESS DENSE)	
		17 - 17.3 M	GRAVEL	HOLE COLAPSE
		3:30 PM	**CAN'T DRILL ANY FURTHER**	
JUNE 18/97		**REDRILL**		
START 7:30 AM		0 - 8.4 M	OPEN HOLE	
		8.4 - 16.8 M	GROUT	
		16.8 - 17 M	CONCRETE AND GRAVEL	
		17 - 17.5 M	GRAVEL	HOLE COLAPSING
		8:15 AM	**CAN'T DRILL ANY FURTHER**	
JUNE 19/97		**REDRILL**		
START 1:30 PM		**133 CASING SYSTEM**		
		0 - 17 M	OPEN HOLE	
		17 - 24 M	SAND AND GRAVEL	DRILL DOWN WITH NO INNER ROD
		24 - 26.6 M	CONCRETE AND GRAVEL	CASING SEATED AT 24 M
FINISH 2:50 PM		26.6 - 27.7 M	SAND STONE (RED)	

DSI DYWIDAG SYSTEMS INTERNATIONAL

DRILL LOGS

PROJECT **HILLSBOROUGH BRIDGE**	JOB #	ANCHOR # **A 26**
GENERAL CONTRACTOR **STRAIT CROSSING INC.**	BORE HOLE DIA. **6 1/8"**	ANGLE OF HOLE FROM HORIZONTAL **90**
	DRILL RIG TYPE **KANI**	DRILL FLUID **AIR**
DRILLER **CHRIS KAPELLER**	DRILL METHOD **DTH HAMMER**	DEPTH OF HOLE

DATE	TIME	DEPTH	STRATIGRAPHY	REMARKS
JUNE 17/97				
START 4:00 PM		0 - 8.3 M	OPEN PIPE	
		8.3 - 9.5 M	CONCRETE (DENSE)	
		9.5 - 10.2 M	STEEL (POSSIBLE PIPE PILE)	
		5:40 PM	STOP DRILLING	

DSI DYWIDAG SYSTEMS INTERNATIONAL

DRILL LOGS

PROJECT **HILLSBOROUGH BRIDGE**	JOB #	ANCHOR # **A 27**
GENERAL CONTRACTOR **STRAIT CROSSING INC.**	BORE HOLE DIA. **6 1/8"**	ANGLE OF HOLE FROM HORIZONTAL **90**
DRILLER **CHRIS KAPELLER**	DRILL RIG TYPE **KANI**	DRILL FLUID **AIR**
	DRILL METHOD **DTH HAMMER**	DEPTH OF HOLE

DATE	TIME	DEPTH	STRATIGRAPHY	REMARKS
JUNE 17/97				
START 8:00 AM		0 - 8.1 M	OPEN PIPE	
		8.1 - 9 M	GRAVEL BAG	
		9 - 10 M	CONCRETE (DENSE)	
		10 - 10.3 M	CONCRETE (LESS DENSE)	
		10.3 - 11.5 M	CONCRETE AND GRAVEL	
		11.5 - 11.6 M	STEEL	
		11.6 - 12 M	CONCRETE, GRAVEL AND STEEL	HOLE COLAPSING
		10:26 AM	**CAN'T DRILL ANY FURTHER**	CONSOLDATE GROUT
JUNE 18/97		**REDRILL**		
START 8:30 AM		0 - 8.4 M	OPEN HOLE	
		8.4 - 12 M	GROUT	
		12 - 12.1 M	STEEL	
		12.1 - 14.5 M	CONCRETE WITH GRAVEL SEAMS	
		14.5 - 16 M	CONCRETE (LESS DENSE)	
		16 - 16.8M	GRAVEL	HOLE COLAPSING
		9:50 AM	**CAN'T DRILL ANY FURTHER**	
JUNE 20/97		**REDRILL**		
START 7:30 AM		**133 CASING SYSTEM**		
		0 - 15 M	OPEN HOLE	
		15 - 23.5 M	SAND AND GRAVEL	
		23.5 - 26.5 M	CONCRETE AND GRAVEL	CASING SEATED AT 24 M
FINSH 9:35 AM		26.5 - 27.7 M	SAND STONE (RED)	

DSI DYWIDAG SYSTEMS INTERNATIONAL

DRILL LOGS

PROJECT **HILLSBOROUGH BRIDGE**	JOB #	ANCHOR # **A 28**
GENERAL CONTRACTOR **STRAIT CROSSING INC.**	BORE HOLE DIA. **6 1/8"**	ANGLE OF HOLE FROM HORIZONTAL **90**
	DRILL RIG TYPE **KANI**	DRILL FLUID **AIR**
DRILLER **CHRIS KAPELLER**	DRILL METHOD **DTH HAMMER**	DEPTH OF HOLE

DATE	TIME	DEPTH	STRATIGRAPHY	REMARKS
JUNE 17 /97				
START 11:00 AM		0 - 8.3 M	OPEN PIPE	
		8.3 - 9 M	GRAVEL BAG	
		9 - 9.5 M	CONCRETE (DENSE)	
		9.5 - 15 M	CONCRETE, SEAMS OFGRAVEL	
		15 - 17 M	CONCRETE (LESS DENSE)	
		17 - 17.3 M	GRAVEL	HOLE COLAPSING
		12:30 PM	**CAN'T DRILL ANY FURTHER**	
JUNE	21	0 - 9	OPEN PIPE	
		9 - 17	GROUT	
		17 - 23.8	GRAVEL SAND + WATER	
		23.8 - 24.5	CONCRETE (LESS DENSE)	
		24.5 - 26.6	CONCRETE (DENSE)	CASING SEATED AT 25 M.
		26.6 - 27.8	RED SANDSTONE	

DSI DYWIDAG SYSTEMS INTERNATIONAL

DRILL LOGS

PROJECT **HILLSBOROUGH BRIDGE**	JOB #	ANCHOR # **A 29**
GENERAL CONTRACTOR **STRAIT CROSSING INC.**	BORE HOLE DIA. **6 1/8"**	ANGLE OF HOLE FROM HORIZONTAL **90**
DRILLER **CHRIS KAPELLER**	DRILL RIG TYPE **KANI**	DRILL FLUID **AIR**
	DRILL METHOD **DTH HAMMER**	DEPTH OF HOLE

DATE	TIME	DEPTH	STRATIGRAPHY	REMARKS
JUNE 18/97		0 - 8.2 M	OPEN PIPE	
START 12:20 PM		8.2 - 10 M	CONCRETE (DENSE)	
		10 - 11 M	CONCRETE (LESS DENSE)	
		11 - 11.7 M	CONCRETE AND GRAVEL	
		11.7 - 12	STEEL	
		12 - 16.6 M	CONCRETE AND GRAVEL	
		16.6 - 17 M	GRAVEL	HOLE COLAPSING
		1:35 PM	CAN'T DRILL ANY FURTHER	
JUNE 19/97		**REDRILL**		
START 4:00 PM		133 CASING SYSTEM		
		0 - 15.5 M	OPEN HOLE	
		15.5 - 18.5 M	SAND AND GRAVEL	
		18.5 - 18.8M	CONCRETE	
		18.8 - 24 M	SAND AND GRAVEL	
		24 - 26.5 M	CONCRETE AND GRAVEL	CASING SEATED AT 24 M
FINISH 5:20 PM		26.5 - 27.7 M	SAND STONE (RED)	

DSI DYWIDAG SYSTEMS INTERNATIONAL

DRILL LOGS

PROJECT **HILLSBOROUGH BRIDGE**	JOB #	ANCHOR # **A 30**
GENERAL CONTRACTOR **STRAIT CROSSING INC.**	BORE HOLE DIA. **6 1/8"**	ANGLE OF HOLE FROM HORIZONTAL **90**
	DRILL RIG TYPE **KANI**	
	DRILL METHOD	DRILL FLUID **AIR**
DRILLER **CHRIS KAPELLER**	**DTH HAMMER**	DEPTH OF HOLE

DATE	TIME	DEPTH	STRATIGRAPHY	REMARKS
JUNE 17/97				
START 6:05 PM		0 - 8.3 M	OPEN PIPE	
		8.3 - 8.8 M	GRAVEL BAG	
		8.8 - 9.6 M	CONCRETE (DENSE)	
		9.6 - 9.7 M	STEEL	
		9.7 - 11.8 M	CONCRETE (LESS DENSE)	
		11.8 - 11.9 M	STEEL	
		11.9 - 13.3 M	CONCRETE (LESS DENSE)	
		13.3 - 14 M	CONCRETE AND GRAVEL	
		14 - 14.4 M	GRAVEL	HOLE COLAPSING
		6:55 PM	**CAN'T DRILL ANY FURTHER**	
JUNE 20/97		**REDRILL**		
START 9:45 AM		**133 CASING SYSTEM**		
		0 - 9 M	OPEN HOLE	
		9 - 14.5 M	GROUT	
		14.5 - 17 M	CONCRETE AND GRAVEL	
		17 - 25 M	SAND AND GRAVEL	CASING SEATED AT 25 M
		25 - 26.5M	CONCRETE AND GRAVEL	
FINISH 11:20 AM		26.5 - 27.7 M	SAND STONE (RED)	

DSI DYWIDAG SYSTEMS INTERNATIONAL

DRILL LOGS

PROJECT **HILLSBOROUGH BRIDGE**	JOB #	ANCHOR # **A 31**
GENERAL CONTRACTOR **STRAIT CROSSING INC.**	BORE HOLE DIA. **6 1/8"**	ANGLE OF HOLE FROM HORIZONTAL **90**
DRILLER **CHRIS KAPELLER**	DRILL RIG TYPE **KANI**	DRILL FLUID **AIR**
	DRILL METHOD **DTH HAMMER**	DEPTH OF HOLE

DATE	TIME	DEPTH	STRATIGRAPHY	REMARKS
JUNE 16/97				
START 1:25 PM		0 - 8.3 M	OPEN PIPE	
		8.3 - 8.6 M	GRAVEL BAG	
		8.6 - 11.5 M	CONCRETE (DENSE)	
		11.5 - 11.6 M	STEEL	
		11.6 - 12.5 M	CONCRETE WITH STEEL	
		12.5 - 13 M	CONCRETE (LESS DENSE)	HOLE COLLAPSING.
		13 - 14 M	CONCRETE (LESS DENSE)	
		14 - 14.5 M	CONCRETE (LESS DENSE)	HOLE COLLAPSING.
		14.5 - 15 M	CONCRETE (LESS DENSE)	
		15 - 17 M	CONCRETE (LESS DENSE)	HOLE COLLAPSING.
		17 - 17.3 M	GRAVEL	HOLE COLLAPSING.
		3:00 PM	**CAN'T DRILL ANY FURTHER**	
JUNE 18/97		**REDRILL**		6 1/8" dia
START 10:50 AM		0 - 8.3 M	OPEN HOLE	
		8.3 - 17 M	GROUT	
		17 - 17.3 M	GRAVEL	HOLE COLLAPSING.
		11:15 AM	**CAN'T DRILL ANY FURTHER**	
June 21 start	9:35am	REDRILL		T.45 + 133 Casing
		0 - 17.	OPEN HOLE	
		17 - 23.7	SAND & GRAVEL	WATER
		23.7 - 25.5	CONCRETE (LESS DENSE)	
		25.5 - 26.5	CONCRETE (DENSE)	CASING SEATED 24M. AFTER SEATING THE CASING AT 24m AND CONTINUING DRILLING W/ T45 + DTH THE HOLE CAVED. LARGE CHUNKS OF GRAVEL WAS WASHED OUT AND BUBBLES SEEN IN THE WATER. THE CASING COULD NOT BE DRIVEN IN FURTHER. CONTINUED DRILLING W/ T45 + DTH WITH GROUT FLUSH TO 26.5 m.
JUNE 22 start	3:35pm	0 - 9.25	OPEN HOLE	6" IR HAMMER
		9.25 - 17.5	GROUT W/ SAND	HOLE CAVING W/ SOME WATER AT 17.5M. HOLE ABANDOMED

70626

Jacques Whitford
Consulting Engineers
Environmental Scientists

3 Spectacle Lake Drive
Dartmouth, Nova Scotia
Canada B3B 1W8
Tel: 902 468 7777

Facsimile Transmission

Company	***Attention***	***Fax Number***
SCI	D. McGinn	902 569 1820
JW	G. Zafiris	902 566 2004
CBCL	G. Strolz	902 423 3938

If not well received please call (902) 468-0431.
We are transmitting a total of 2 pages, including this page.

FROM: John D. Brown **DATE:** June 4, 1997 **OUR REF:** 10271

RE:-GEWI Pile Tests, Pier No. 2, Hillsborough River Bridge

I have reviewed the data sent to me on the initial three pull-out tests, and have the following comments:

1. The pile tested on 16 May is identified as both A11 and A16. I assume that it is A16, but request confirmation.

2. For all three tests [A16 (or A11), A10 and A6] the behaviour under the test load to 370 kips is essentially linear with increasing load.

3. Making a correction for the fact that the dial gauges were set at zero under a load of 28 kips, I find that the permanent set of the pile head after unloading from the maximum load is as follows:

Pile A6	set = 0.52 inches
Pile A10	set = 0.45 inches
Pile A16	set = 0.55 inches

4. I have summarized the drill log information and test data for the tests, as tabled below. The apparent free length of the pile was found by fitting the elastic deformation curve for the GEWI bar to the straight-line loading curve.

Pile No.	Drilled Length in Bedrock ft (m)	Grouted Length ft (m)	Measured Free Length ft (m)	Apparent Free Length ft (m)	Average Bond Stress at Maximum Load psi (kPa)
A6	28.5 (8.7)	34.5 (10.5)	74.7 (22.8)	89.3 (27.2)	46 (320)
A10	30.2 (9.2)	22.2 (6.8)	94.0 (28.7)	95.0 (29.0)	72 (500)
A16	between 20' and 28' (6.1 to 8.5m)	30 (9.1)	82 (25.0)	89.7 (27.3)	53 (370)

5. It is noted that the grouted length in Piles A11 and A16 apparently extends up into the pier. Therefore, as a test of the bedrock/grout bond, A10 is the best result. For the revised factored load of 1400 kN and a socket length of 30 ft (9.1 m), the average factored bond stress is 45 kPa. This test represents a nominal overload of 72/45 = 1.6 times the factored load.

6. The above results are encouraging, and indicate that the piles are acceptable.

Regards

70626

Strait Crossing Inc.

10 Horton Drive Charlottetown, PEI, C1A 7G3
Phone: (902) 569-1566 Fax: (902) 569-1820

FAX COVER SHEET

DATE: **May 30, 1997**

COMPANY: Jacques Whitford and Associates

ATTENTION: **George Zafiris** **John Brown**

FAX NO.: **566-2004** **1-902-468-9009**

FROM: Donald McGinn

TOTAL NUMBER OF PAGES: 12
(including cover sheet)

Original to follow? No

RE: Hillsborough River Bridge – GEWI Pile Installation

Gentlemen,

Please find attached DSI analysis on the first GEWI pile test to ultimate. Two other piles have also been tested to this value with similar results. Two core recoveries have been made (one each side). These have been forwarded to JWA via G. Zafiris.

Operations at Pier No.2 are complete. The footing will be poured around the piles mid-next week. Fourteen of the sixteen proposed piles were installed. Two piles could not be installed because of steel mid-depth in the hole. These piles have been deleted (with CBCL's approval) with the recognition that the remaining piles have 1400kN capacity, which is greater than the original design of 900kN.

Complete drilling and installation records will be sent out via courier this afternoon.

We will commence drilling operations about June 12 at Pier No.3.

Kind Regards,

Donald McGinn, P.Eng.
Project Manager

cc. S Pletch
P. Lockwood/ G. Tadros
G. Strolz - CBCL 1-902-423-3938

This message is intended for the use of the Addressee only.
If you have received this communication in error, please notify Alison immediately by telephone at (902) 569-1566. Thank You

DYWIDAG-SYSTEMS INTERNATIONAL

DSI

Fax

Total number of pages: 6

(DSI Correspondence No 55)

DYWIDAG-SYSTEMS INTERNATIONAL
#103-19433-96th Avenue
Surrey, BC V4N 4C4
Ph: (604) 888-8818
Fax: (604) 888-5008

Hermin

To: Strait Crossing Inc.
7th Floor 1177 - 11th Ave. S.W.
Calgary, AB T2R 1K9

From: Joe Li

Attn.: Mr. Donald McGinn,P.Eng.

Date: May 16, 1997

Phone: 403-244-9090 902-569-1566

Ref. No.: 710268

Fax: 403-228-8643 Site: 902-569-1820

CC: G. Kast

Original to follow: ☐ Yes ☐ No

Subject: **Hillsborough Bridge, PEI - Pier No. 2 - Pile Test No. 1** — All

Dear Mr. McGinn:

Further to your telephone discussion with our Mr. Gary Kast, please find attached our evaluation of the first pile test at pier No. 2 as follows:

Table #1	Calculation of upper and lower limit for bar elongation;
Graph #1	Plot of elongation vs. Test loads at different load stages ;
	Bar elongation at different load stages are well within the tolerance limits.
Table #2	Measured bar elongation at different load cycles;
Graph #2	Plot of pile head movements vs. Test load;
Table #3	Creep values at 133% test load - 60 minute creep test
Graph #3	Plot of creep vs. Time:
	Calculated Ks per log cycle is : [0.488" - 0.478"] / log [T2/T1]
	=0.006" per log cycle; (0.15mm per long cycle)
	well within the allowable limit of 2mm/log cycle

RECEIVED
MAY 16 1997
HBDI
CHARLOTTETOWN, P.E.I.

If you have any questions please call Joe or Gary at 604-888-8818.

Regards,

Joe Li

Joe Li, P.Eng.

Manager, Geotechnical Division

Copy.
Steve. P.
Gamil.

Table #1 ~ A //

bond L ft	Free L ft	Jack & Frame ft	Bar size	Grade	Steel Area per bar In^2	E KSI	Design Load (kips)	278
30	82	3.5	#20 (63.5Ø)	80 ksi	4.91	29700		

Load stage %	kip	** Theoretical (in)	** Minimum (in)	** Maximum(in)
10	27.82	0.00	0.00	0.00
25	69.55	0.29	0.23	0.35
50	139.10	0.78	0.63	0.92
75	208.65	1.27	1.02	1.50
100	278.20	1.76	1.41	2.07
110	306.01	1.96	1.57	2.30
120	333.83	2.15	1.72	2.53
133	370.00	2.41	1.93	2.83

Load Stage %	Test Load kip	** Min. Elongation in	Measured Elongation (in) gauge #1	gauge #2	Measured average	** Max. Elongation in
10	27.82	0.000	0.000	0.000	0.000	0.00
25	69.55	0.235	0.237	0.238	0.238	0.35
50	139.10	0.626	0.777	0.749	0.763	0.92
75	208.65	1.018	1.282	1.222	1.252	1.50
100	278.20	1.409	1.800	1.724	1.762	2.07
110	306.01	1.566	1.992	1.910	1.951	2.30
120	333.83	1.722	2.178	2.093	2.136	2.53
133	370.00	1.926	2.480	2.387	2.434	2.83

Note:

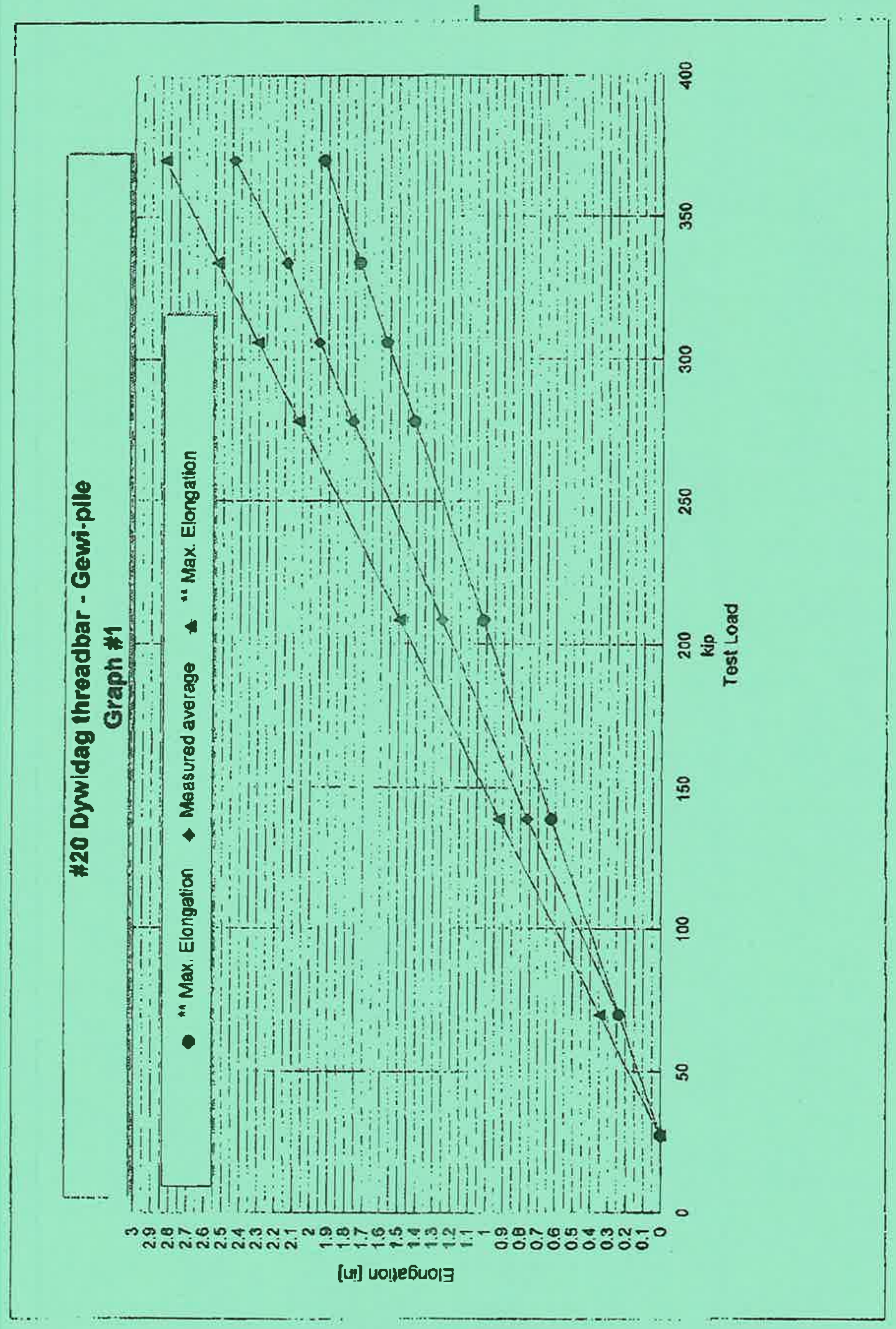
** Calculation based on (test load less 10% design load)
Use tow dial gauges to measure bar elongation
Set both dial gauges to zero at 10% of design load
30 minute Creep test at 133% design load = 370 kips

TESTED By: ____________

DATE: ____________

Date: 05/16/97

Hillsborough Bridge P.E.I.

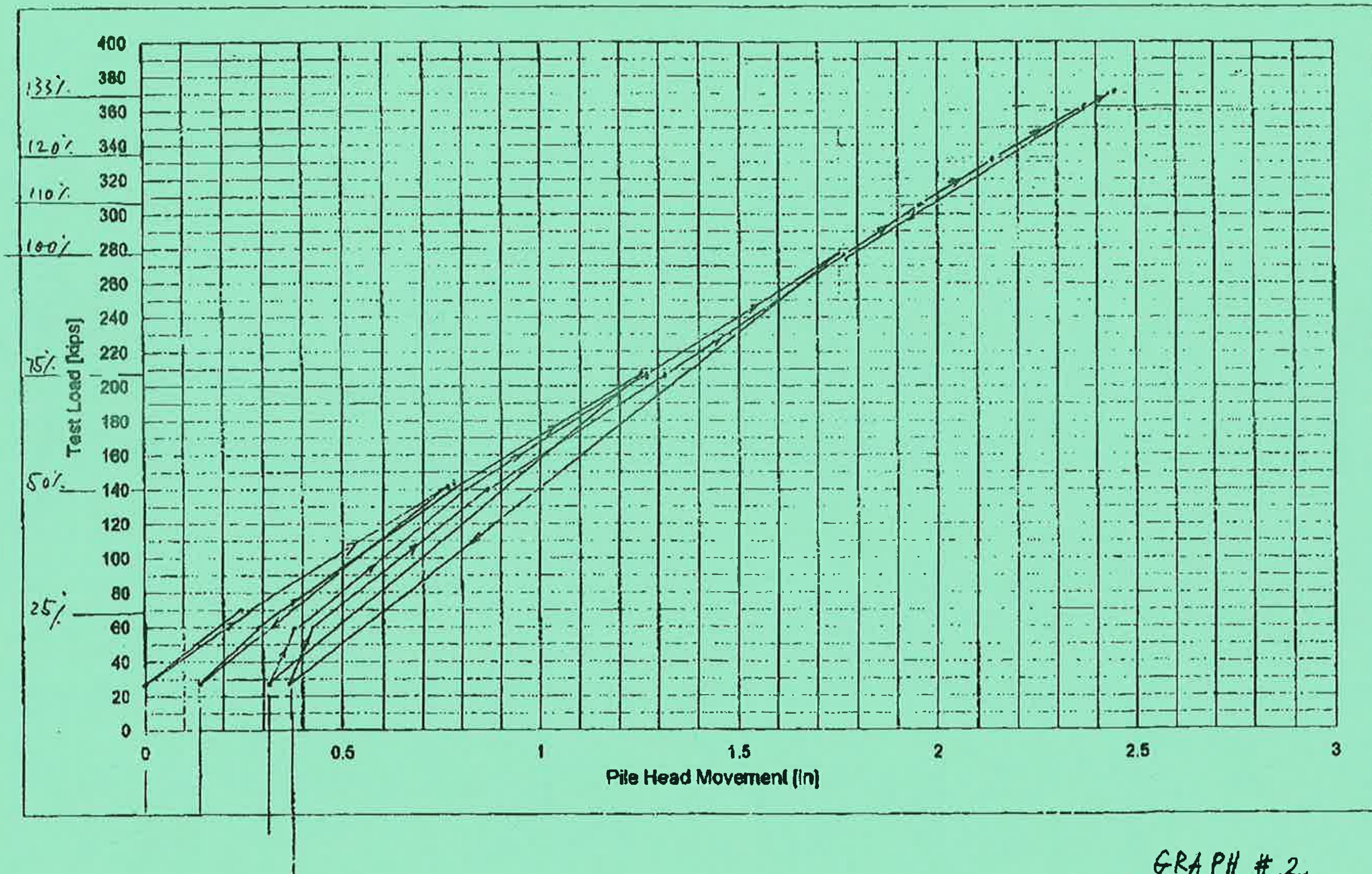


Pile head movement due to cycle loading **Table #2** **Design load = 278 kips**

(kips)	AL 27.8	25% 69.5	50% 139.0	75% 208.5	100% 278.0	110% 305.8	120% 333.6	130% 361.4	133% 369.7
gauge 1	0	0.237							
gauge 2	0	0.238							
average	0.000	0.238							
gauge 1	0	0.25	0.777						
gauge 2	0	0.252	0.749						
average	0.000	0.251	0.763						
gauge 1	0.006	0.25	0.787	1.282					
gauge 2	0.268	0.312	0.756	1.222					
average	0.137	0.281	0.772	1.252					
gauge 1	0.339	0.407	0.84	1.297	1.8				
gauge 2	0.288	0.338	0.778	1.234	1.724				
average	0.314	0.373	0.809	1.266	1.762				
gauge 1	0.403	0.465	0.906	1.354	1.82	1.992	2.178	2.42	2.478
gauge 2	0.32	0.387	0.832	1.28	1.743	1.91	2.093	2.332	2.387
average	0.362	0.436	0.869	1.317	1.782	1.951	2.136	2.376	2.433

TABLE #2

Pile head movement due to cycle loading

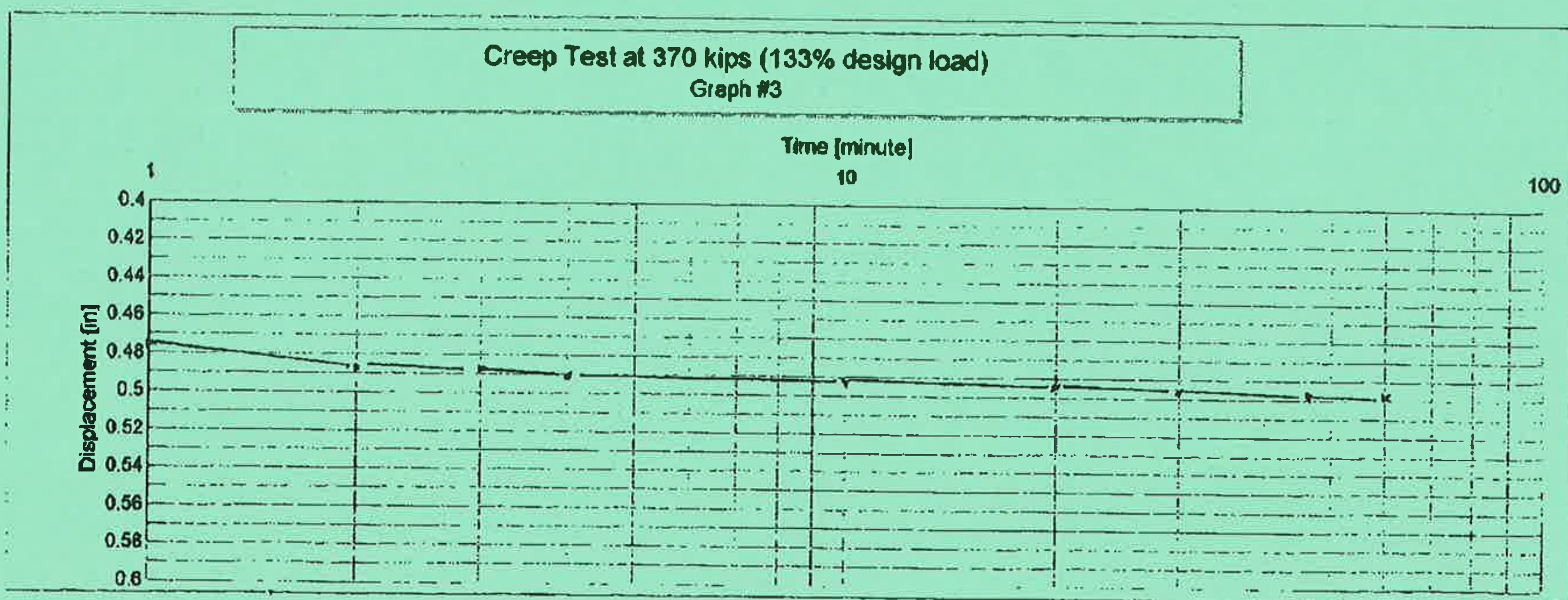


GRAPH # 2

Date: 05/16/97 Hillsborough Bridge P.E.I, Page5

TABLE #3

Creep [minute]	gauge 1		gauge 2	Average [in]
1	0.478		0.477	0.4775
2	0.488		0.478	0.484
3	0.489		0.479	0.484
4	0.49		0.48	0.485
5	0.491		0.481	0.486
10	0.491		0.482	0.487
15	0.492		0.482	0.487
20	0.492		0.482	0.487
25	0.492		0.483	0.488
30	0.493		0.483	0.488
45	0.493		0.483	0.488
60	0.493		0.483	0.488



Graph #3

DSI DYWIDAG SYSTEMS INTERNATIONAL

PROJECT: HILLSBOROUGH BRIDGE P.E.I
ANCHOR #: A10
JACK #: HJ-11
TEST LOAD (MAX): 370 KIP
BAR SIZE #20 (63.5 ⌀)

DATE: May 28/97
Pier No. 2

GAUGE # A 103
BOND LENGTH
FREE LENGTH

PERFORMANCE TEST

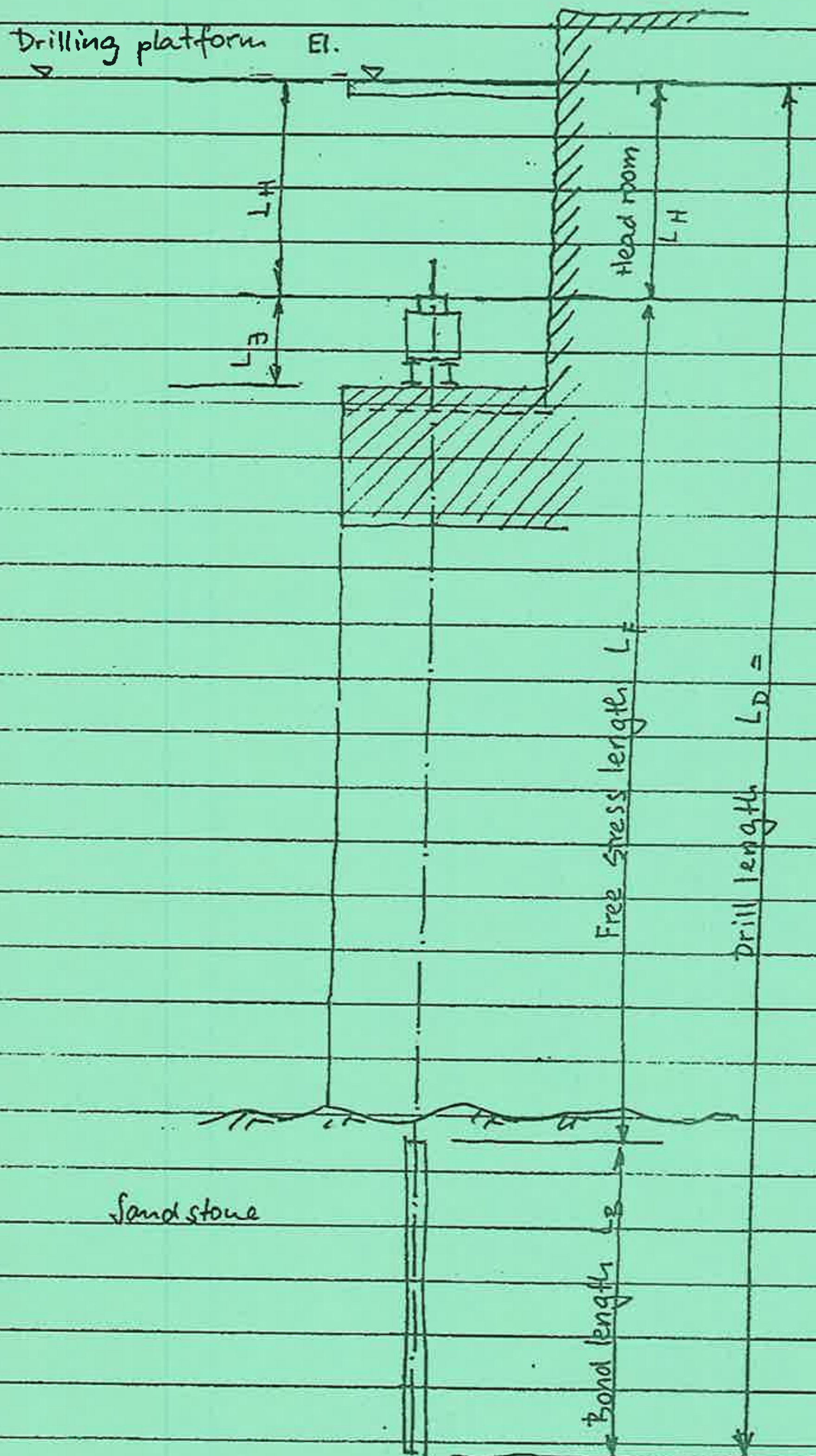
TESTED BY: ED SHATULA

TEST LOAD / % OF TEST LOAD			DIAL GAUGE	DIAL GAUGE READINGS	START AT / FINISH AT			INSPECTOR: [redacted]			
	KIPS	PSI		0 MIN	1MIN	2MIN	3MIN	4MIN	5MIN	10MIN	15MIN
AL	27.8	550	D1	0.000							
			D2	0.000	← Set to zero						
25%	69.5	900	D1	0.246							
			D2	0.245							
AL	27.8	550	D1	0.147							
			D2	0.146							
25%	69.5	900	D1	0.248							
			D2	0.247							
50%	139	1700	D1	0.793							
			D2	0.796							
AL	27.8	550	D1	0.185							
			D2	0.184							
25%	69.5	900	D1	0.298							
			D2	0.297							
50%	139	1700	D1	0.795							
			D2	0.801							
75%	208	2475	D1	1.315							
			D2	1.322							
AL	27.8	550	D1	0.213							
			D2	0.213							
25%	69.5	900	D1	0.331							
			D2	0.332							
50%	139	1700	D1	0.826							
			D2	0.833							
75%	208	2475	D1	1.323							
			D2	1.331							
87.50%	243	2800	D1	1.548							
			D2	1.554							
100%	278	3250	D1	1.853	1.860	1.860	1.860	1.862	1.863		
			D2	1.859	1.864	1.864	1.864	1.866	1.867		
AL	27.8	550	D1	0.251							
			D2	0.253							
25%	69.5	900	D1	0.377							
			D2	0.379							
50%	139	1700	D1	0.884							
			D2	0.889							
75%	208	2475	D1	1.365							
			D2	1.371							
100%	278	3250	D1	1.873							
			D2	1.879							
1.10%	305.8	3525	D1	2.063							
			D2	2.070							
1.20%	333.6	3825	D1	2.264							
			D2	2.271							
1.30%	361.4	4175	D1	2.519							
			D2	2.525	1MIN	2MIN	3MIN	4MIN	5MIN	10MIN	15MIN
1.33%	370	4250	D1	2.605	2.610	2.613	2.614	2.615	2.617	2.626	2.627
			D2	2.612	2.615	2.618	2.619	2.620	2.622	2.631	2.632

				20 MIN.	25 MIN.	30 MIN.	45 MIN.	60 MIN.			
			D1	2.628	2.629	2.631					
			D2	2.632	2.634	2.636					
UNLOADING											
100%	270	3250	D1			2.259					
			D2			2.265					
75%	208	2475	D1			1.736					
			D2			1.740					
50%	139	1700	D1			1.226					
			D2			1.228					
25%	69.5	900	D1			0.679					
			D2			0.679					
AL	27.8	550	D1			0.418					
			D2			0.416					

PEI - Hillsborough River Bridge - Gewi Piles

97-05-97



PILE # A10

PIER No. 2

L_D = 133'-6" (40.70 m)

L_F = 78'-9" (24.00)

L_H = 20'-4" (6.20 m)

L_B = 34'-5" (10.50 m)

L_J = 4'-0"

Drilling platform elevation
at Pier #2 : El = +3.070 m
Pier #3 : El = +

L_B = 40.70 - 24.00 - 6.20
=

Date : 28-05-97

By : GK

DSI Canada Ltd.

28-05-97 (PEI) ①

TEST REPORT OF PILE 'A10' LOAD TEST (TESTED ON 28-05-97 by G. Kast and Ed Shatula)

1. Theoretical Elongation Calculation

Theor. elongation ($\Delta_{1.33\ theo.}$) at $1.33\ P_w = 370\ k$, calculated from actual field measurement of $L_{F_{field}} = 94'$-0 (1,128")

$$\Delta_{1.33\ theo} = \frac{P_T \times L_F}{A_s \times E_s} = \frac{370{,}000 \times 1{,}128''}{4.91 \times 29{,}700{,}000} = \underline{\underline{2.862''}}$$

2. Actual Elongation (as measured during testing) $\Delta_{act.}$

2.1 Measurement at $0.1\ P_w$ (alignment load ≐ 28k) → $\Delta_{0.1} = 0.000''$

2.2 Measurement at $1.33\ P_w$ (max. test load = 370k) → $\Delta_{1.33} = 2.636''$

2.3 Deduct residual elongation, as measured → $\Delta_{res} = -0.417$

2.4 Actual elongation measured $\Delta_{act.} = 2.219''$

(from $0.1\ P_w$ ~~to $1.33\ P_w$~~) = 342.0 k

2.5 Prorate elongation from $P_w = 0$ to $1.33\ P_w = 370\ k$

$$\frac{2.219''}{342\ k} = \frac{\Delta_e}{370}$$

$$\Delta_e = \frac{2.219 \times 370}{342}$$

78'-9 (24.0,

$$\Delta_e = \underline{2.400''} \text{ (actual field elongation)}$$

3. Calculate L_F from actual field elongation

$$L_F = \frac{\Delta_e \times A_s \times E_s}{P_T} = \frac{2.4 \times 4.91 \times 29{,}700{,}000}{370{,}000} = 946'' \ (78'\text{-}9''$$

DSI DYWIDAG SYSTEMS INTERNATIONAL

PROJECT: HILLSBOROUGH BRIDGE P.E.I.
ANCHOR #: A6
JACK #: HJ-11
TEST LOAD (MAX): 370 KIP
BAR SIZE #20 (63.5 ⌀)

DATE: 27-05-97
GAUGE # A 103
BOND LENGTH
FREE LENGTH

North Pier
(Pier No. 2)

PERFORMANCE TEST

TESTED BY: ED SHATULA

TEST LOAD 370 K

START AT
FINISH AT

INSPECTOR : GEORGE

% OF TEST LOAD	KIPS	PSI	DIAL GAUGE	0 MIN	1MIN	2MIN	3MIN	4MIN	5MIN	10MIN	15MIN
AL	27.8	550	D1	0							
			D2	0							
25%	69.5	900	D1	0.253							
			D2	0.236							
AL	27.8	550	D1	0.203							
			D2	0.188							
25%	69.5	900	D1	0.256							
			D2	0.239							
50%	139	1700	D1	0.768							
			D2	0.753							
AL	27.8	550	D1	0.267							
			D2	0.256							
25%	69.5	900	D1	0.305							
			D2	0.292							
50%	139	1700	D1	0.769							
			D2	0.753							
75%	208	2475	D1	1.262							
			D2	1.247							
AL	27.8	550	D1	0.290							
			D2	0.283							
25%	69.5	900	D1	0.335							
			D2	0.325							
50%	139	1700	D1	0.795							
			D2	0.778							
75%	208	2475	D1	1.261							
			D2	1.242							
87.50%	243	2800	D1	1.462							
			D2	1.442							
100%	278	3250	D1	1.811		1.812		1.813	1.813		
			D2	1.790		1.792		1.795	1.795		
AL	27.8	550	D1	0.330							
			D2	0.318							
25%	69.5	900	D1	0.391							
			D2	0.378							
50%	139	1700	D1	0.855							
			D2	0.838							
75%	208	2475	D1	1.318							
			D2	1.299							
100%	278	3250	D1	1.869							
			D2	1.849							
1.10%	305.8	3525	D1	1.950							
			D2	1.930							
1.20%	333.6	3825	D1	2.140							
			D2	2.118							
1.30%	361.4	4175	D1	2.371							
			D2	2.348	1MIN	2MIN	3MIN	4MIN	5MIN	10MIN	15MIN
1.33%	370	4250	D1	2.430	2.454	2.456	2.457	2.459	2.461	2.463	2.464
			D2	2.421	2.426	2.428	2.429	2.431	2.433	2.434	2.435

			DIAL GAUGE	20 MIN.	25 MIN.	30 MIN.	45 MIN.	60 MIN.			
			D1	2.465	2.466	2.466					
			D2	2.436	2.436	2.436					
UNLOADING											
100%	270	3250	D1			2.237					
			D2			2.199					
75%	208	2475	D1			1.655					
			D2			1.630					
50%	139	1700	D1			1.266					
			D2			1.231					
25%	69.5	900	D1			0.712					
			D2			0.681					
AL	27.8	550	D1			0.464					
			D2			0.437					

measured free length L_F = 74'-8
+ 17'-4
92'-0

127'-0
− 92'-0
L_B = 35'-0

Actual elong.
D1: 2.466
− 0.464
2.002"

D2: 2.436
− 0.437
1.999"

C.C. SCI

DSI

DYIDAG SYSTEMS INTERNATIONAL

WIRELINE CORE DRILL LOGS

PROJECT **HILLSBOROUGH BRIDGE**	**PIER # 2 EASTSIDE**	ANCHOR # **A 2**
GENERAL CONTRACTOR **STRAIT CROSSING INC.**	BORE HOLE DIA. **3"**	ANGLE OF HOLE FROM HORIZONTAL **90**
DRILLER **CHRIS KAPELLER**	DRILL RIG TYPE **KANI**	DRILL FLUID **WATER**
	DRILL METHOD **NQ CORING 5FT. SAMPLES**	DEPTH OF HOLE **134.7 FT.**

DATE	TIME	DEPTH	STRATIGRAPHY	REMARKS
MAY 17/97				
	7:00 - 11:00 AM		SET UP AND RUN NW CORE RODS TO 98 FT. (OPEN HOLE)	
START	11:00 AM	0 - 98 FT.	OPEN HOLE DRILLED WITH 6 1/8" DTH HAMMER DRILL	
		98 - 98.3 FT.	SEAT NW CASING INTO CONCRETE	
		98 - 102 FT.	CONCRETE	
		102 - 104 FT. (APPROX.)	NO RECOVERY	DRILLING STILL DENSE THREW THIS ZONE.
		104 - 125 FT.	SAND STONE	
		125 - 129.7 FT.	SANDSTONE WITH LAYERS OF SILTSTONE AND CLAY.	LOST APRROX. 2FT. OF RECOVERY IN THE CLAY SEAMS (WATER INJECTION SEEMS TO WASH THE CLAY) ALL DRILLING IS CONSISTENT THREW THE LAYERS (DENSE) .
FINISH 4:00 PM.		129.7-134.7	SANDSTONE WITH LAYERS OF SILTSTONE AND CLAY.	GOOD RECOVERY
			4:00 - 5:00 PM	PULL DRILL RODS OUT OF HOLE

◆ Strait Crossing Inc. ◆

10 Horton Drive Charlottetown, PEI, C1A 7G3
Phone: (902) 569-1566 Fax: (902) 569-1820

FAX COVER SHEET

DATE: May 13, 1997

COMPANY: Jacques Whitford Environment Ltd.

ATTENTION: G. Zafiris J. Brown 902-468-9009

FAX NO.: **566-2004**

FROM: Donald McGinn

TOTAL NUMBER OF PAGES: **16**
(including cover sheet)

Original to follow? Yes No ✓

Re: Hillsborough Bridge Improvement

P2 drilling records to date.

cc: G. Strolz- CBCL 902-423-3938

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MAY 13 1997

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CHARLOTTETOWN, P.E.I.

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DYIDAG SYSTEMS INTERNATIONAL

DRILL LOGS

PROJECT **HILLSBOROUGH BRIDGE**	JOB #	ANCHOR # **A 12**
	BORE HOLE DIA. **6 1/8"**	ANGLE OF HOLE FROM
GENERAL CONTRACTOR **STRAIT CROSSING INC.**	DRILL RIG TYPE **KANI**	HORIZONTAL **90**
	DRILL METHOD	DRILL FLUID **AIR**
DRILLER **CHRIS KAPELLER**	**DTH HAMMER**	DEPTH OF HOLE

DATE	TIME	DEPTH	STRATIGRAPHY	REMARKS
MAY 12/97	5:00 PM	0 - 8.2 M.	PIPE SLEEVE	
		8.2 - 9.3	CONCRETE	WATER COMING INTO HOLE
		9.3 - 9.35M.	STEEL	
		9.35 - 11.4M.	CONCRETE	
		11.4 - 11.5M	STEEL	
		11.5 - 31.2M	CONCRETE	
		31.2 - 31.5M.	CONCRETE WITH SILT	LOTS OF WATER
	7:03 PM	31.5 - 33.5	SAND STONE (RED)	
				DRILLED HOLE WITH API AND DTH
	7:30 PM	FINISH PULLING DRILL RODS		HAMMER 6 1/8" BIT
				WHEN CLEANING BORE HOLE LOTS
				OF BIG PCS. OF BROKEN CONCRETE
				AND ROCKS CAME OUT OF HOLE
				CHECK DEPTH OF BORE HOLE
				HOLE OPEN TO BOTTUM
			7:45 - 8:20 PM	CONSOLIDATE GROUT HOLE 23 BAGS
				GROUT LEVEL AT APPROX. 17 FT
				FROM DECK

DSI DYWIDAG SYSTEMS INTERNATIONAL

DRILL LOGS

PROJECT	JOB #	ANCHOR # **A 13**
HILLSBOROUGH BRIDGE	BORE HOLE DIA. **6 1/8"**	ANGLE OF HOLE FROM
GENERAL CONTRACTOR	DRILL RIG TYPE **KANI**	HORIZONTAL **90**
STRAIT CROSSING INC.	DRILL METHOD	DRILL FLUID **AIR**
DRILLER **CHRIS KAPELLER**	**DTH HAMMER**	DEPTH OF HOLE

DATE	TIME	DEPTH	STRATIGRAPHY	REMARKS
MAY 11/97	5:15 PM	0 - 8.5 M.	PIPE SLEEVE	
		8.5 - 9.2 M.	CONCRETE	HOLE DRILLED DRY TO 22.6M.
		9.2 - 9.4M.	STEEL	
		9.4 - 11.2 M.	CONCRETE	
		11.2 - 11.4 M.	STEEL	
		11.4 - 22.6 M.	CONCRETE	
6:40 - 7:00 PM		22.6 M.	STEEL IN CONCRETE	DRILL APPROX. 2" INTO STEEL
				END OF SHIFT.
MAY 12/97			**MAY 12 /97 7:00 - 10:30 AM.**	DRILL APPROX. 1 FT. INTO CONCRETE
				AND STEEL
				DECIDE TO PULL OUT CHECK DRILL
				BIT STILL IN GOOD SHAPE
			10:30 AM	TALKED TO DON INFORM OF PROBLEM
				HIS SUGGESTION TO CON' T DRILLING
				FOR ANOTHER 2 HRS TO SEE IF WE
				CAN GET THREW THE STEEL
			10:30 - 11:30 AM	REPAIR DRILL FOOT (REWELD)
			11:40 am - 12:40 pm	START DRILLING AGAIN NO MOVEMENT
			1:00 - 4:30 PM	PULL OUT OF HOLE CHANGE DRILL OVER
				TO CORING HEAD RUN DOWN HOLE
				TRY TO CORE STEEL BROKE CORE
				BARREL OF DOWN THE HOLE
				DECIDE TO MOVE TO A NEW HOLE.

DYWIDAG SYSTEMS INTERNATIONAL
GROUT REPORT
PROJECT HILLS BOROUGH BRIDGE P.E.I. (PIER # 2)

DATE	ANCHOR #	BAR LENGTH	BOND LENGTH	DRILL LENGTH METERS	BORE HOLE	GROUT ZONE METERS	TREMIE GROUT (BAGS)	TOP UP (BAGS)	NO. OF BAGS USED	REMARKS
										THEORETICAL GROUT VOLUME 15.6 BAGS
MAY 11/97	A 14	CONSOLIDATE		33.5 M.	6 1/8"	33.5M.	17			GROUT CAME TO TOP OF CASING
		GROUT				32 M.				GROUT LEVEL AT 9 FT. FROM DECK
						30 M.				GROUT LEVEL AT 15 FT. FROM DECK
START 3:30 PM						28 M.				GROUT LEVEL AT 42 FT. FROM DECK
						26 M.				GROUT LEVEL AT 46 FT. FROM DECK
FINISH 4:50 PM						24 M.		4		GROUT LEVEL AT 26 FT. FROM DECK
						22 M.				GROUT LEVEL AT 28 FT. FROM DECK
						20M.				GROUT LEVEL AT 29 FT. FROM DECK
						18 M.				GROUT LEVEL AT 30 FT. FROM DECK
						16 M.				GROUT LEVEL AT 30 FT. FROM DECK
						14 M.		3		GROUT LEVEL AT 17 FT. FROM DECK
						12 M.				GROUT LEVEL AT 18 FT. FROM DECK
						10 M.				GROUT LEVEL AT 20 FT. FROM DECK
						TOTAL	17	7	24	
										PULL ALL CASING OUT OF HOLE
										GROUT LEVEL AT 25 FT.
MAY 12/97	A 12	CONSOLIDATE		33.5 M.	6 1/8"	33.5 -8.2M.	23 BAGS		23	THEORICAL GROUT VOLUME FOR 25M. 15 BAGS
		GROUT								
										TIE MK 1 TO POLE RUN TO BOTTUM OF HOLE
START	7:45 PM									AND TREMIE GROUT
FINISH	8:20 PM									GROUT LEVEL APPROX. 17FT. FROM DECK
								MAY 13/97 7:00AM		MEASURE GROUT LEVEL AT 25 FT. FROM DECK

D. McGinn
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DYWIDAG SYSTEMS INTERNATIONAL

DATE MAY 11/97

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CHARLOTTETOWN, P.E.I.

CONTRACTOR: STRAIT CROSSING INC.

LOCATION: HILLSBOROUGH BRIDGE PROJECT P.E.I. (PIER # 2)

LABOUR NAME	CLASSIFICATION	WORK DESCRIPTION	HOURS REG.	O.T.
SHELDON KAPELLER	SUPERVISOR	CHECK GROUT LEVEL A11 AT 45FT.		12
		DRILL A9 TO 17M. HIT STEEL CAN'T		
CHRIS KAPELLER	DRILLER	DRILL THREW		12
		DRILL A14 TO 33.5M CONSOLIDATE		
PETER OED	DRILLER	GROUT		12
		DRILL A13 TO 22.6M (END OF SHIFT)		
ED SHATULA	GROUTER			12
		MAKE GROUT PAD FOR A 16 TEST		
* NOTE		ANCHOR		
ALL DEPTHS ARE		CHECK FOR TESTING BEAMS		
RECORDED FROM				
DRILL DECK				

EQUIPMENT DESCRIPTION	HOURS	FOOTAGE DRILLED
MP 4000 GROUT PLANT WITH HYD POWER PACK	2	
RUBBER HYD DRILL / 6"DTH HAMMER	12	240 FT.
LUMAS MINI DRILL		
TESTING EGUIPMENT		
WATER TESTING EQUIPMENT (AIR PACKER & FLOW METER)		

MATERIALS DESCRIPTION	QUANITY	
# 20 BARE BAR		
# 20 CUPLERS		
TYPE 30 CEMENT FOR CONSOLIDATION GROUTING	24	BAGS
TYPE 30 CEMENT FOR ANCHOR GROUTING		

DAILY TIME TICKET

DSI REPRESENTATIVE: [signature]

CLIENT APPROVAL: [signature]

DATE: May 12/97

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DYWIDAG SYSTEMS INTERNATIONAL

DATE MAY 10 /97

CONTRACTOR: STRAIT CROSSING INC.

LOCATION: HILLSBOROUGH BRIDGE PROJECT P.E.I. (PIER # 2)

LABOUR NAME	CLASSIFICATION	WORK DESCRIPTION	HOURS REG.	O.T.
SHELDON KAPELLER	SUPERVISOR	CHECK GROUT LEVEL A16 95 FT.		11.5
		A11 30FT.		
CHRIS KAPELLER	DRILLER			10.5
		REDRILL A 11 TO 33.5 M HOLE		
PETER OED	DRILLER	COLAPSE AT 25.6M. RUN 133MM		11.5
		CASING CONSOLIDATE GROUT A11		
ED SHATULA	GROUTER	24 BAGS		10.5
* NOTE		BROKE FEED CABLE ON DRILL 2 HRS.		
ALL DEPTHS ARE		TO REPAIR		
RECORDED FROM				
DRILL DECK				

EQUIPMENT DESCRIPTION	HOURS	FOOTAGE DRILLED
MP 4000 GROUT PLANT WITH HYD POWER PACK	2	
RUBBER HYD DRILL / 6"DTH HAMMER	10.5	110 FT.
LUMAS MINI DRILL		
TESTING EGUIPMENT		
WATER TESTING EQUIPMENT (AIR PACKER & FLOW METER)		

MATERIALS DESCRIPTION	QUANITY	
# 20 BARE BAR		
# 20 CUPLERS		
TYPE 30 CEMENT FOR CONSOLIDATION GROUTING	24	BAGS
TYPE 30 CEMENT FOR ANCHOR GROUTING		

DAILY TIME TICKET

DSI REPRESENTATIVE: [signature]

CLIENT APPROVAL: [signature]

DATE: May 12/97

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DYIDAG SYSTEMS INTERNATIONAL

DATE MAY 9/97

CONTRACTOR: STRAIT CROSSING INC.

LOCATION: HILLSBOROUGH BRIDGE PROJECT P.E.I. (PIER # 2)

LABOUR NAME	CLASSIFICATION	WORK DESCRIPTION	HOURS REG.	O.T.
SHELDON KAPELLER	SUPERVISOR	DRILL A16 TO 40M. (131.2 FT.)	8	4.5
CHRIS KAPELLER	DRILLER	WATER TEST A 16 WITH PACKER	8	4.5
PETER OED	DRILLER	INSTALL 130 FT. #20 BAR & GROUT	8	4.5
		BOND ZONE.		
ED SHATULA	GROUTER		8	4.5
		DRILL A11 TO 16M (52.5 FT.)		
* NOTE	6:00 PM	COMPRESSOR RUN OUT OF FEUL		
ALL DEPTHS ARE		CONSOLIDATE GROUT HOLE		
RECORDED FROM				
DRILL DECK				

EQUIPMENT DESCRIPTION	HOURS	FOOTAGE DRILLED
MP 4000 GROUT PLANT WITH HYD POWER PACK	3	
RUBBER HYD DRILL / 6"DTH HAMMER	7	183.7 FT.
LUMAS MINI DRILL		
TESTING EGUIPMENT		
WATER TESTING EQUIPMENT (AIR PACKER & FLOW METER)	1	

MATERIALS DESCRIPTION	QUANITY	
# 20 BARE BAR	130 FT.	
# 20 CUPLERS	6	PCS
TYPE 30 CEMENT FOR CONSOLIDATION GROUTING	6	BAGS
TYPE 30 CEMENT FOR ANCHOR GROUTING	6	BAGS

DAILY TIME TICKET

DSI REPRESENTATIVE: [signature]

CLIENT APPROVAL: [signature]

DATE: MAY 10/97

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DYIDAG SYSTEMS INTERNATIONAL

DATE MAY 8/97

CONTRACTOR: STRAIT CROSSING INC.

LOCATION: HILLSBOROUGH BRIDGE PROJECT P.E.I. (PIER # 2)

LABOUR NAME	CLASSIFICATION	WORK DESCRIPTION	HOURS REG.	O.T.
SHELDON KAPELLER	SUPERVISOR	TAKE 6" DTH APART PISTON RUSTED	8	4
		IN BARREL (REPAIR)		
CHRIS KAPELLER	DRILLER		8	4
		WELD EXTRA PLATES IN CONERS		
PETER OED	DRILLER	OF DECK AND GUARD RAIL POST	8	4
		AND RUN CABLES FOR RAILING		
ED SHATULA	GROUTER		8	4
		1:00PM SET DRILLING EQUIP ON DRILL		
		DECK AND SET UP ON A16		
		REPAIR LEAKS IN AIR LINE		

EQUIPMENT DESCRIPTION	HOURS	FOOTAGE DRILLED
MP 4000 GROUT PLANT WITH HYD POWER PACK		
RUBBER HYD DRILL / 6"DTH HAMMER		
LUMAS MINI DRILL		
TESTING EGUIPMENT		
WATER TESTING EQUIPMENT (AIR PACKER & FLOW METER)		

MATERIALS DESCRIPTION	QUANITY	
# 20 BARE BAR		
# 20 CUPLERS		
TYPE 30 CEMENT FOR CONSOLIDATION GROUTING		
TYPE 30 CEMENT FOR ANCHOR GROUTING		

DAILY TIME TICKET

DSI REPRESENTATIVE: [signature]

CLIENT APPROVAL: [signature]

DATE: MAY 10/97

DSI

DYIDAG SYSTEMS INTERNATIONAL

DATE MAY 7/ 97

CONTRACTOR: STRAIT CROSSING INC.

LOCATION: HILLSBOROUGH BRIDGE PROJECT P.E.I. (PIER # 2)

LABOUR NAME	CLASSIFICATION	WORK DESCRIPTION	HOURS REG.	O.T.
SHELDON KAPELLER	SUPERVISOR	CHECK OVER EQUIP	8	2.5
CHRIS KAPELLER	DRILLER	MOVE CONTAINERS AND GROUT	8	0.5
		PLANT TO NORTH END OF BRIDGE		
PETER OED	DRILLER		8	0.5
		MOVE DRILL AND DRILL RODS		
ED SHATULA	GROUTER	OVER TO LOADING DOCK FOR	8	0.5
		BARGE		
		REVIEW OLD REPORTS AND LAST		
		YEARS DRILLING INFO.		

EQUIPMENT DESCRIPTION	HOURS	FOOTAGE DRILLED
MP 4000 GROUT PLANT WITH HYD POWER PACK		
RUBBER HYD DRILL / 6"DTH HAMMER		
LUMAS MINI DRILL		
TESTING EGUIPMENT		
WATER TESTING EQUIPMENT (AIR PACKER & FLOW METER)		

MATERIALS DESCRIPTION	QUANITY	
# 20 BARE BAR		
# 20 CUPLERS		
TYPE 30 CEMENT FOR CONSOLIDATION GROUTING		
TYPE 30 CEMENT FOR ANCHOR GROUTING		

DAILY TIME TICKET

DSI REPRESENTATIVE: [signature]

CLIENT APPROVAL: [signature]

DATE: MAY 10/97

DSI

DYIDAG SYSTEMS INTERNATIONAL

DATE MAY 6 / 97

CONTRACTOR: STRAIT CROSSING INC.

LOCATION: HILLSBOROUGH BRIDGE PROJECT P.E.I. (PIER # 2)

LABOUR NAME	CLASSIFICATION	WORK DESCRIPTION	HOURS REG.	O.T.
SHELDON KAPELLER	SUPERVISOR	TRAVEL TO P.E.I.	8	
CHRIS KAPELLER	DRILLER		8	
PETER OED	DRILLER		8	
ED SHATULA	GROUTER		8	

EQUIPMENT DESCRIPTION	HOURS	FOOTAGE DRILLED
MP 4000 GROUT PLANT WITH HYD POWER PACK		
RUBBER HYD DRILL / 6"DTH HAMMER		
LUMAS MINI DRILL		
TESTING EGUIPMENT		
WATER TESTING EQUIPMENT (AIR PACKER & FLOW METER)		

MATERIALS DESCRIPTION	QUANITY	
# 20 BARE BAR		
# 20 CUPLERS		
TYPE 30 CEMENT FOR CONSOLIDATION GROUTING		
TYPE 30 CEMENT FOR ANCHOR GROUTING		

DAILY TIME TICKET

DSI REPRESENTATIVE: [signature]

CLIENT APPROVAL: [signature]

DATE: May 10/97

DSI DYWIDAG SYSTEMS INTERNATIONAL

DRILL LOGS

PROJECT **HILLSBOROUGH BRIDGE**	JOB #	ANCHOR # **A 9**
GENERAL CONTRACTOR **STRAIT CROSSING INC.**	BORE HOLE DIA. **6 1/8"**	ANGLE OF HOLE FROM HORIZONTAL **90**
	DRILL RIG TYPE **KANI**	
DRILLER **CHRIS KAPELLER**	DRILL METHOD **DTH HAMMER**	DRILL FLUID **AIR**
		DEPTH OF HOLE

DATE	TIME	DEPTH	STRATIGRAPHY	REMARKS
MAY 11/97	7:50 AM	0 - 6.5 M.	PIPE SLEEVE	
		6.5 - 7.5 M.	CONCRETE	WATER COMING INTO BORE HOLE
		7.5 - 7.55 M.	STEEL	
		7.55 - 9.3 M.	CONCRETE	
		9.3 - 9.5 M.	STEEL IN CONCRETE	
		9.5 - 11.1 M.	CONCRETE	
		11.1 - 11.2 M.	STEEL	
		11.2 - 17M.	CONCRETE	
9:00 - 9:45 AM		17 M.	STEEL	LOTS WATER COMING INTO HOLE
				DRILL MAYBE 1/4" INTO STEEL
				SEEMS LIKE BIG PCS OF STEEL
				LITTLE FLAKES OF METAL COMING
				OUT.
			9:45 - 10:05 AM	PUT DRILL RODS OUT OF HOLE CHECK
				BIT IS IN GOOD SHAPE
				ONLY WAY TO CON'T IS CORE STEEL

DSI DYWIDAG SYSTEMS INTERNATIONAL

DRILL LOGS

PROJECT **HILLSBOROUGH BRIDGE**	JOB #	ANCHOR # **A 14**
GENERAL CONTRACTOR **STRAIT CROSSING INC.**	BORE HOLE DIA. **6 1/8"**	ANGLE OF HOLE FROM HORIZONTAL **90**
	DRILL RIG TYPE **KANI**	DRILL FLUID **AIR**
DRILLER **CHRIS KAPELLER**	DRILL METHOD **DTH HAMMER**	DEPTH OF HOLE

DATE	TIME	DEPTH	STRATIGRAPHY	REMARKS
MAY 11/97	10:20 AM	0 - 6.4 M.	PIPE SLEEVE	
		6.4 - 7.3 M.	CONCRETE	
		7.3 - 7.4 M.	STEEL IN CONCRETE	WATER COMING INTO BORE HOLE
		7.4 - 9.2 M.	COMCRETE	
		9.2 - 9.4M	STEEL IN CONCRETE	
		9.4 - 33.8 M.	CONCRETE	
FINISH	1:10 PM	33.8 - 32 M.	CONCRETE WITH SILT	LOTS WATER COMING INTO BORE
		32 - 33.5 M	SAND STONE (RED)	AT INTERFACE OF ROCK
		1:10 - 1:50 PM	WASH BORE HOLE AND	26 - 32M. CONCRETE HAS BROKEN
			PULL OUT DRILL RODS	SEAMS AND FALLING INTO HOLE
				BUT STILL ABLE TO CONTINUE DRILLING
			CHECK BORE HOLE WITH POLES HOLE COLAPSE AT 28 M.	
		2:30 - 3:30 PM	RUN 133MM CASING TO 33.5 M. AND CLEAN OUT WITH T-45	

DYWIDAG SYSTEMS INTERNATIONAL
GROUT REPORT
PROJECT HILLS BOROUGH BRIDGE P.E.I. (PIER # 2)

DATE	ANCHOR #	BAR LENGTH	BOND LENGTH	DRILL LENGTH METERS	BORE HOLE	GROUT ZONE METERS	TREMIE GROUT (BAGS)	TOP UP (BAGS)	NO. OF BAGS USED	REMARKS
MAY 9/97	A16	130 FT.	30 FT.	40 M.	6 1/8"		6		6	THEORICAL VOLUME TO FILL 30 FT. BOND IS
										4.8 BAGS
			INSTALL 6 PCS. 20FT. AND 1 PCS. 10 FT. # 20 BARE BAR							MAY 10/97 CHECK GROUT LEVEL IN BORE HOLE
										WE HAVE 35 FT. OF GROUT
										ANCHOR HAS BOND BREAKER AT 30 FT.
MAY 9 /97	A 11	CONSOLIDATE GROUT		16 M.	6 1/8"	8.3- 16M.	6 BAGS		6	THEORICAL GROUT VOLUME IS 4.45 BAGS
										GROUT LEVEL AT 31 FT. FROM DECK
										MAY 10 /97 GROUT LEVEL AT 30 FT. FROM DECK

DSI DYWIDAG SYSTEMS INTERNATIONAL

DRILL LOGS

PROJECT **HILLSBOROUGH BRIDGE**	JOB #	ANCHOR # **A 11**
GENERAL CONTRACTOR **STRAIT CROSSING INC.**	BORE HOLE DIA. **6 1/8"** DRILL RIG TYPE **KANI**	ANGLE OF HOLE FROM HORIZONTAL **90**
DRILLER **CHRIS KAPELLER**	DRILL METHOD **DTH HAMMER**	DRILL FLUID **AIR** DEPTH OF HOLE

DATE	TIME	DEPTH	STRATIGRAPHY	REMARKS
MAY 9/97	4:20 PM	0 - 8.3M	PIPE SLEEVE	
		8.3 - 8.8M.	NEW TREME CONCRETE	
		8.8 - 9.2 M	OLD CONCRETE	Water coming into hole at interface new and
		9.2 - 9.3M.	STEEL	old concrete
		9.8 - 11.2M	OLD CONCRETE	
		11.2 - 11.3M	STEEL	
		11.3 - 14.5M	OLD CONCRETE	
		14.5 - 14.6	STEEL	
	6:00 PM	14.6 - 16	OLD CONCRETE	
		6:35 -7 PM	CONSOLIDATE GROUT HOLE 6 BAGS OF CEMENT GROUT LEVEL	
			APPROX. 30 FT. FROM TOP OF DECK	
		REDRILL		
MAY 10/97	8:35 AM	0 - 9M	OPEN HOLE	GROUT AT 9M (30 FT.)
		9 -16 M.	GROUT	
		16 - 31.5M	CONCRETE	LOTS WATER COMING INTO BORE HOLE
				20 - 28M. CONCRETE HAS BROKEN
				SEAMS AND FALLING INTO HOLE
				BUT STILL ABLE TO CONTINUE DRILLING
		31.5 - 32M	CONCRETE WITH SILT	LOTS OF WATER AT THIS ZONE
	11:45 AM	32 - 33.5	SAND STONE (RED)	
		11:45 - 11:48 AM	WASH AND FLUSH BORE HOLE	
		12:00 - 1:30 PM	FEED CABLE BROKE (REPAIR)	
		1:30 - 2:30 PM	TRIP DRILL RODS OUT	
		2:30 3:00PM	LUNCH	
		3:00 - 4:30 PM	CHECK HOLE WITH LOADING POLE HOLE COLAPSED IN AT	
			25.6M. RUN 133 MM CASING TO 33.5M CLEAN OUT CASING	
			WITH T - 45 DRILL STEEL	
		4:30 - 5:48 PM	CONSOLIDATE GROUT 24 BAGS OF TYPE 30 GROUT AT 25 FT.	
MAY 11/97	7:20 AM		CHECK GROUT LEVEL AT 45 FT. APPROX. 2 FT. OF SOFT GROUT ON	
			TOP	

DSI DYWIDAG SYSTEMS INTERNATIONAL

DRILL LOGS

PROJECT **HILLSBOROUGH BRIDGE**	JOB # BORE HOLE DIA. **6 1/8"**	ANCHOR # **A16** ANGLE OF HOLE FROM
GENERAL CONTRACTOR **STRAIT CROSSING INC.**	DRILL RIG TYPE **KANI** DRILL METHOD	HORIZONTAL **90** DRILL FLUID **AIR**
DRILLER **CHRIS KAPELLER**	**DTH HAMMER**	DEPTH OF HOLE

DATE	TIME	DEPTH	STRATIGRAPHY	REMARKS
			* NOTE (HOLE HAD BEEN DRILLED TO 37M. AND GROUTED DEC./96)	
MAY 9/97	7:45 AM	0 - 8M	PIPE SLEEVE	
		8 -11M.	CONCRETE & GROUT	WATER COMING INTO BORE
		11M.	POSSIBLE REBAR	
		11 - 31.5M.	CONCRETE & GROUT	
		31.5 - 34M.	GROUT & SAND STONE (RED)	
	11:19 AM	34 - 40M	SAND STONE	
	11:19 - 11:45 AM		WASH AND BLOW HOLE 5 TIME TO CLEAN BORE HOLE	
	11:45 AM - 12:45 PM		TRIP DRILL STEEL OUT FILL HOLE WITH WATER AS COMING OUT.	
	1:45 - 2:40 PM		WATER TEST HOLE WITH AIR PACKER AND FLOW METER	
			TEST AT NEW TREME CONCRETE LEVEL	5 PSI 20 LTS/MIN
			TEST AT 2 FT. INTO ROCK LEVEL	7 PSI 3 LTS/ MIN
				7 PSI 7LTS/ 5MINS
				7 PSI 13.5 LTS/ 10MINS
			DRILL HOLE IS ACCEPTABLE TO INSTALL ANCHOR	

DYWIDAG SYSTEMS INTERNATIONAL
GROUT REPORT
PROJECT HILLS BOROUGH BRIDGE P.E.I. (PIER # 2)

DATE	ANCHOR #	BAR LENGTH	BOND LENGTH	DRILL LENGTH METERS	BORE HOLE	GROUT ZONE METERS	TREMIE GROUT (BAGS)	TOP UP (BAGS)	NO. OF BAGS USED	REMARKS
MAY 10/97	A 11	CONSOLIDATE		34 M.	6 1/8"	34 M.	10			THEORICAL GROUT VOLUME FOR 25.7M IS 15 BAGS
START 4:30 PM		GROUT				32 M.		4		GROUT LEVEL AT 54 FT. FROM DECK
FINISH 5:48 PM						30M.		4		GROUT LEVEL AT 36 FT. FROM DECK
						28 M.				GROUT LEVEL AT 40 FT. FROM DECK
						26 M.		4		GROUT LEVEL AT 26 FT. FROM DECK
						24 M.				GROUT LEVEL AT 30 FT. FROM DECK
						22 M				GROUT LEVEL AT 33 FT. FROM DECK
						20 M.				GROUT LEVEL AT 36 FT. FROM DECK
						18 M.				GROUT LEVEL AT 38 FT. FROM DECK
						16M.		2		GROUT LEVEL AT 22 FT. FROM DECK
						14 M.				GROUT LEVEL AT 25 FT. FROM DECK
						12 M.				GROUT LEVEL AT 25 FT. FROM DECK
						10 M.				GROUT LEVEL AT 25 FT. FROM DECK
						TOTAL	10	14	24	
										PULL ALL CASING OUT OF HOLE
										GROUT LEVEL AT 28 FT. FROM DECK
										MAY 11/97 7:30AM MEASURE GROUT AT 45FT
										FROM DECK APPROX. 2 FT. OF SOFT GROUT ON
										TOP

TRANSMISSION VERIFICATION REPORT

TIME : 04/25/1997 10:50
NAME : JACQUES WHITFORD
FAX : 1-902-566-2004
TEL : 1-902-566-2866

DATE,TIME	04/25 10:49
FAX NO./NAME	DELCOM
DURATION	00:00:38
PAGE(S)	01
RESULT	OK
MODE	STANDARD

◆ Strait Crossing Inc. ◆

10 Horton Drive Charlottetown, PEI, C1A 7G3
Phone: (902) 569-1566 Fax: (902) 569-1820

FAX COVER SHEET

DATE: **May 9, 1997**

COMPANY: Jacques Whitford and Associates

ATTENTION: **George Zafiris** **John Brown**

FAX NO.: **566-2004** **1-902-468-9009**

FROM: Donald McGinn

TOTAL NUMBER OF PAGES: 1
(including cover sheet)

Original to follow? No

RE: Hillsborough River Bridge – GEWI Pile Installation

Gentlemen,

We have resumed drilling and pile installation.

We have drilled out the hole that was started last year and are in the process of installing the first pile. This pile will be load tested to ultimate on Tuesday. It is understood, that as per our meeting of January 27, an increase in capacity will result in a reduction in the number of piles required.

We also plan to begin core recovery operations on Tuesday. Please contact our office on Monday for confirmation

Kind Regards,

Donald McGinn, P.Eng
Project Manager

cc. S Pletch
P. Lockwood/ G. Tadros
G. Strolz - CBCL

◆ Strait Crossing Inc. ◆

10 Horton Drive Charlottetown, PEI, C1A 7G3
Phone: (902) 569-1566 Fax: (902) 569-1820

FAX COVER SHEET

DATE: January 30, 1997

COMPANY: **Jacques Whitford Environment Ltd.**

ATTENTION: **G. Zafiris/J. Brown** ← *

FAX NO.: **566-2004**

FROM: Steve Pletch

TOTAL NUMBER OF PAGES: **8**
(including cover sheet)

Original to follow? Yes No ✓

Re: Hillsborough Bridge River Improvement

- **GEWI pile coring program**

Hillsborough Bridge Improvements
GEWI Pile Installation Meeting No. 1
Monday January 27, 1997

Attendance:	G. Zafiris-JWA	G. Kast-DSI	R. Cantin
	J. Brown-JWA	S. Pletch	P. Colucci
			D. McGinn

Distribution:	P. Lockwood-SCI
	G. Strolz-CBCL

1. Program To Date

- S. Pletch provided description of works done to date.

Summary

a. Piles 9, 10, 13 and 16 on pier had casing installed.

b. Pile # 16 was drilled using Drill Rig C - A rotary percussion type rig with down the hole hammer.

c. Drilling log for this pile was reviewed.

d. To seal the hole, it was grouted and planned to be redrilled the next day.

e. Storm arrived and took out the cofferdam (i.e. hole has not been redrilled).

2. Coring Program

a. CBCL and JWA found not much geotechnical information exists from 1965 when bridge was installed. Only 3 to 4 bore holes were done.

JWA has maintained that more cores are required. SCI proposed to do coring at time piles are being installed. At that time SCI was considering equipment that could HQ-4" core.

It is there, not to verify the GEWI pile, but for the foundations overall.

b. JWA needs NQ (only slightly under 3") not the 4" specified. This should be done by coring specialist; Long Year or Logan for example.

With DSI equipment we might get 1m/hr coring Vs 6m/hr for correct.

Long Year's equipment is larger.

Coring con't

Action:

It would be more efficient to use DSI equipment for geotechnical core recovery.

Action:

JWA to review number of cores required at each pier. (1wk)

c. Coring the concrete is more of a problem. DSI GEWI pile drilling log will provide some information.

Action:

It appears that coring requirements through the concrete originated before the GEWI piles were selected. CBCL to confirm.

3. **Pile Optimization**

a. Drilling of 32 piles in Pier 3 is going to be difficult due to working space limitations.

b. In a 6" hole more than one bar could fit. In BC there is a job with 1x63+ 2x45mm bars. But couplers make this impossible.

c. Larger holes are possible but
   i) drill bits much more expensive
   ii) casing is more expensive

d. Length of socket could be increased to increase capacity of pile.

e. Based upon DSI experience, the pile length is borderline for test load.
- DSI recommends doing one pile test to failure and JWA concurs. Results may reduce number of P3 pile required.
- DSI recommends pile test be done before drilling rest of piles.
- Winter work of drilling is very difficult. Everything needs to be hoarded and heated.

f. SCI to review P3 pile locations and see where preferred moved locations would be.

Pile Optimization con't

g. Drilling some piles from existing bridge deck with truck mounted rig would be difficult.

4. Testing Program

- DSI testing program is acceptable to JWA.
- JWA wants representative to observe pile load test
- JWA representative to review with DSI drilling engineer requirements and methods of coring.

5. Attachments

a) Drilling Log

b) Drilling Method

c) Test Procedure

d) Drill Starter Detail

DYWIDAG SYSTEMS INTERNATIONAL CANADA

DRILL LOGS

PROJECT HILLSBOROUGH BRIDGE	JOB #	ANCHOR # 16
	BORE HOLE DIA. 6½" / 5½"	ANGLE OF HOLE FROM
GENERAL CONTRACTOR STRAIT CROSSING INC.	DRILL RIG TYPE	HORIZONTAL 90
	DRILL METHOD	DRILL FLUID AIR
DRILLER RAY ANDERSON	DTH and 133 casing + T.45	DEPTH OF HOLE 37 m.

DATE	TIME	DEPTH m.	STRATIGRAPHY	REMARKS
1996				
Dec 16	10:55	0 – 7.8	Inside pipe extension	6½" DTH w/ 133 drill pipe
		7.8 – 8.2	new concrete	dry
		8.2 – 9.0	old concrete	– very wet, shells + starfish + seaweed
		?		+ old bolt, pebbles and misc
	1.	–		marine stuff in cuttings
		9.0	steel rebar	
	11:20	9.4	old concrete	add 2m length 133 drill pipe
	11:33	10.3		burst air line
	12:18	10.3		resume drilling – v. wet
	12:30	11.0	steel rebar	
	12:48	11.4		add 2m length Estimate 50 gpm
				water return
	1:05			coffee break
	1:23	12.6	old concrete	resume drilling
	1:33	13.4		add 2m length
	1:42	14.4	softer concrete	
	1:47	15.4	old concrete	add 2m length
	1:51	15.7		water return reduced, bubbles
				seen in river.
	1:58	17.0		stop drilling – repair.
	2:08	17.0		resume drilling
	2:10	17.4		add 2m length. Lots of water
				in return – no bubbles in river.
	2:22	19.4	old concrete	add steel
	2:34	21.4		add 2m length
	2:50	23.4	old concrete	add 2m length
	3:05	25.4		add 2m length High tide
				– 1.9m. Reduced water
				return, bubbles seen in river
				almost immed full water return
				est 100 gpm.
	3:30	27.4		add 2m length
	3:47	29.4	softer concrete	add 2m length
	4:03	30.8	RED SANDSTONE	Large amt. bubbles in river.
	4:14	31.4		add 2m length. No bubbles.
	4:36	32.0		Drilling too slow. Decided to
				pull out
	5:05			Hammer out of hole. The
				6½" bit has lost all but one
				of the inner buttons.

✓

2 of 2

DYWIDAG SYSTEMS INTERNATIONAL CANADA

DRILL LOGS

PROJECT HILLSBOROUGH BRIDGE	JOB #	ANCHOR # 16
GENERAL CONTRACTOR STRAIT CROSSING INC.	BORE HOLE DIA. 6½ / 5½	ANGLE OF HOLE FROM HORIZONTAL 90
	DRILL RIG TYPE	
	DRILL METHOD	DRILL FLUID AIR
DRILLER RAY ANDERSON	DTH and 133 casing + T.45	DEPTH OF HOLE 37m

DATE	TIME	DEPTH	STRATIGRAPHY	REMARKS
				DTH w/ 6½" bit + 133 casing:
				Ran in loading poles to 30.5.
				Cleaned out hammer screens.
Dec 17	9:12		pink water return	Running in drill pipe - bubbles seen in river.
	9:57	32.00		Took a long time to reach the bottom of the hole.
	10:28	32.4		using top hammer intermittently
	10:48	32.8		very slow drilling (bit is no good) bubbles appear only when bit is raised off the bottom.
	10:58	33.1		Abandoned drilling - too slow. Pulled out.
				Ran in loading poles to 29 m.
				Decided to switch method to 133 casing w/ 5½" crown bit and T.45 steel.
	1:16	32.0		Start to drill
	1:18	33.0	Red water return	
	1:24	34.0		add 2m length
	1:36	36.0	" " "	add 2m length Delay to get drill supplies.
	1:43	36.0		resume drilling
	1:50	37.0	" " "	Bottom of hole.
	1:53			Tripping out
	2:07			All out.
	3:10		start consolidation grouting - Type 30 0.45 w:c 12 bags filled the casing. Pulling casing and	
	5:00		added a further 16 bags.	
	—		Tide water - 3m. Depth of river water = 20m.	
Dec 18			Plumbed grout level - 26m.	
	9:45		Tremied in 5 bags - grout leakage seen at bottom of sheet pile. Pumped a further 4 bags and added 80L	
	10:30		gravel at top of pipe. It appears that the leak is sealed.	

V

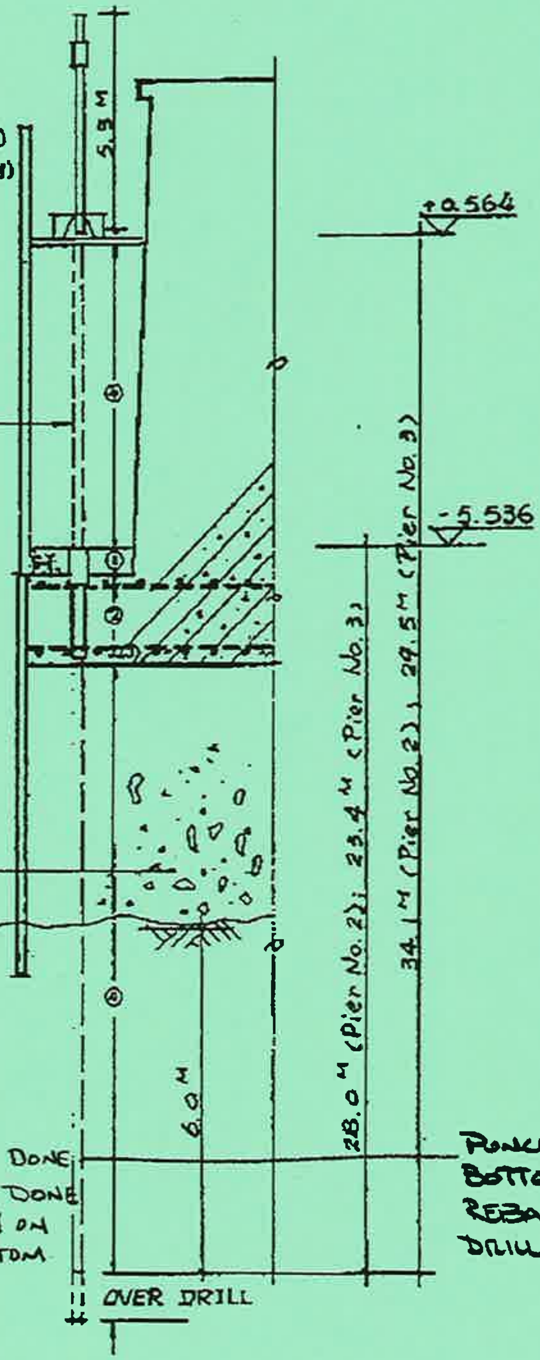
DRILL RIG 'C' :

L = 7'-0" (2.1 M)
W = 4'-6" (1.4 M)
H = 17'-4" (MAST) (5.3 M)
WEIGHT : 6800 LBS (3090 kg)

H.W. + 0.800

L.W. - 1.100

TEMPORARY PIPE



REINFORCEMENT
TOP MAT :
TOP : No. 7 (22M) @ 10"
BOT : No. 7 (22M) @ 10"
BOT MAT :
TOP : No. 11 (35M) @ 10"
BOT : No. 11 (35M) @ 8"

MIX OF LEAN CONCRETE &
LOOSE GRAVEL AND SAND

DRILLING STEPS		DRILL RIG
① CORING	210 Φ	'B'
② DTH - 150	Φ	~~'B'~~ C
③ CORING	145 Φ	'B' - NOT DONE
④ CASING	140 Φ	'C' - NOT DONE ONLY ON BOTTOM

PUNCHED THRU BOTTOM MAT OF REBAR WITH DRILL RIG 'C'

DRILL RIG 'B':
LENGTH OF MAST L = 2.65 M
WEIGHT : W = 200 KG
FEATURES : ROTARY / PERCUSSION
DTH HAMMER ; CORING

DRILL RIG 'A'
STANDARD CORE DRILL :
WORK PERFORMED BY LOCAL
CORING CONTRACTOR.

DSI
DYWIDAG SYSTEMS INTERNATIONAL CANADA LTD.

HILLSBOROUGH RIVER
BRIDGE P.E.I

DSI PROPOSAL
DRILLING

09/25/96
A.C
G.K
DWG. No. SK-01

MARK-UP FROM JAN 27th MEETING

FACTORED LOAD = 900 kN/PILE

General Notes.

1. Material

1.1 Gewi-Pile
#20 Grade 75 Ksi Dywidag Threadbar by Dywidag - Systems International.

Yield Capacity = 368 Kips
Ultimate Capacity = 491 Kips
Max. Design Load = 0.6 X Yield Capacity (220 Kips)

1.2 Grout
Type 30 Cement, w/c = 0.5(max), fc' = 30 MPa (min) at 7 days

2. Drilling

Drill hole for anchor by using percussion drilling method with casing liner if required.

3. Installation and Grouting

3.1 Water testing the borehole (if required).
3.2 Install the pre-assembled threadbar into the bore hole.
3.3 Trimie grout the bore hole with the grout line attached to the bottom of the anchor.
3.4 For the test pile only, two stages grouting shall be required and bond breaker shall be installed on top of the bonded length of the pile.

4. Testing (Gewi-Piles shall be tested in tension: one in every four piles)

4.1 Performance Test (As Per PTI Recommendations)
Performance Tension Test on TWO (2) production Gewi-piles only. (One at each pier)

Procedure as follow:

Design Load P = Working Load
Max. Test Load = 60% Yield of Bar (220 Kips)
Alignment Load = 10% Design Load

Testing Procedure:

Stress anchor to Alignment Load and dial guage will be set for zero at this datum load.
The test shall be made by incrementally loading and unloading starting from the Alignment Load to the following maximum cycle loads:

0.25 P
0.5 P
0.75 P
1.0 P
1.33 P / Max. Test Load

At each increment, the movement of the anchor shall be recorded to the nearest 0.001 inches. The creep readings at the maximum test load shall be read at 0, 1, 2, 3, 4, 5, 10. 15, 20, 25, 30, 45, and 60 minutes. Unload the anchor to 100%, 75%, 50%, and 25% of the Design Load and then back to the Alignment Load and measure the anchor extension at each load.

4.2 Proof Test (remaining anchors)

Max. Test Load = 60% Yield of Bar (220 Kips)
Alignment Load = 10% Design Load

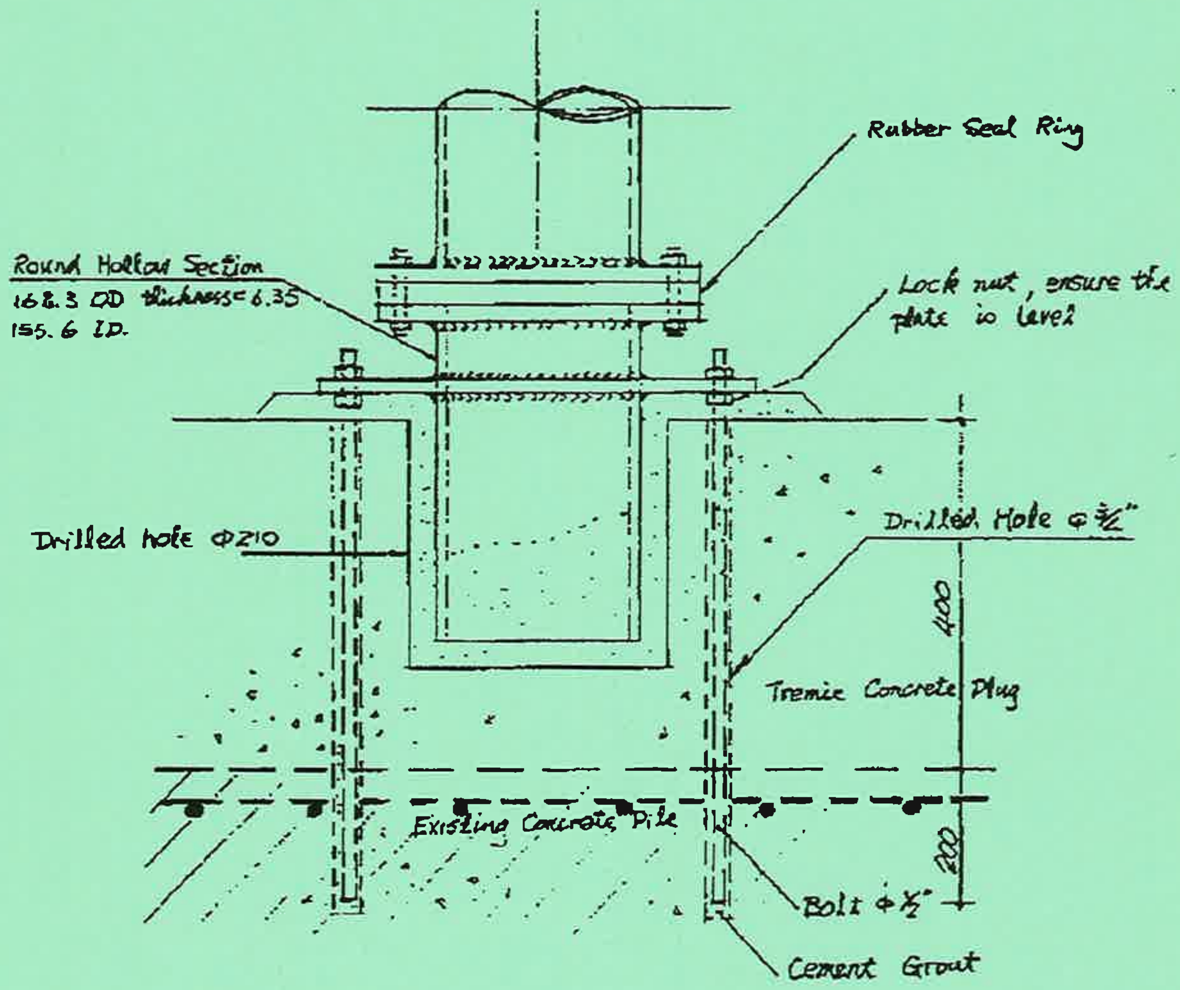
Testing Procedure:
Stress anchor to Alignment Load and dial guage will be set for zero at this datum load. Continue stressing anchor incrementally to 25%, 50%, 75%, 100% of the Design Load. At each increment the movement of the anchor shall be recorded to the nearest 0.001 inches. Stress anchor to 100% of the test load and hold for 10 minutes, the elongation shall be measured at 1, 2, 3, 4, 5 and 10 minutes. Unload the anchor to 100%, 75%, 50%, 25% and 10% of the design load and measure the anchor extension at each load.

DSI

SCALE: N.T.S.	APPROVED BY:	DRAWN BY A.C
DATE: DEC. 11 1996		REVISED
Hillsborough River Bridge		
Gewi Pile Installation		DRAWING NUMBER HRB-004

11 X 17 PRINTED ON NO. 1000H CLEARPRINT ®

This program is acceptable to JWA.



DSI
DYWIDAG SYSTEMS INTERNATIONAL CANADA LTD.

HILLSBOROUGH RIVER
BRIDGE P.E.I

DSI PROPOSAL

09/23/96	
A.C	
G.K	
DWG. No. SK-05	

◆ Strait Crossing Inc. ◆

10 Horton Drive Charlottetown, PEI, C1A 7G3
Phone: (902) 569-1566 Fax: (902) 569-1820

FAX COVER SHEET

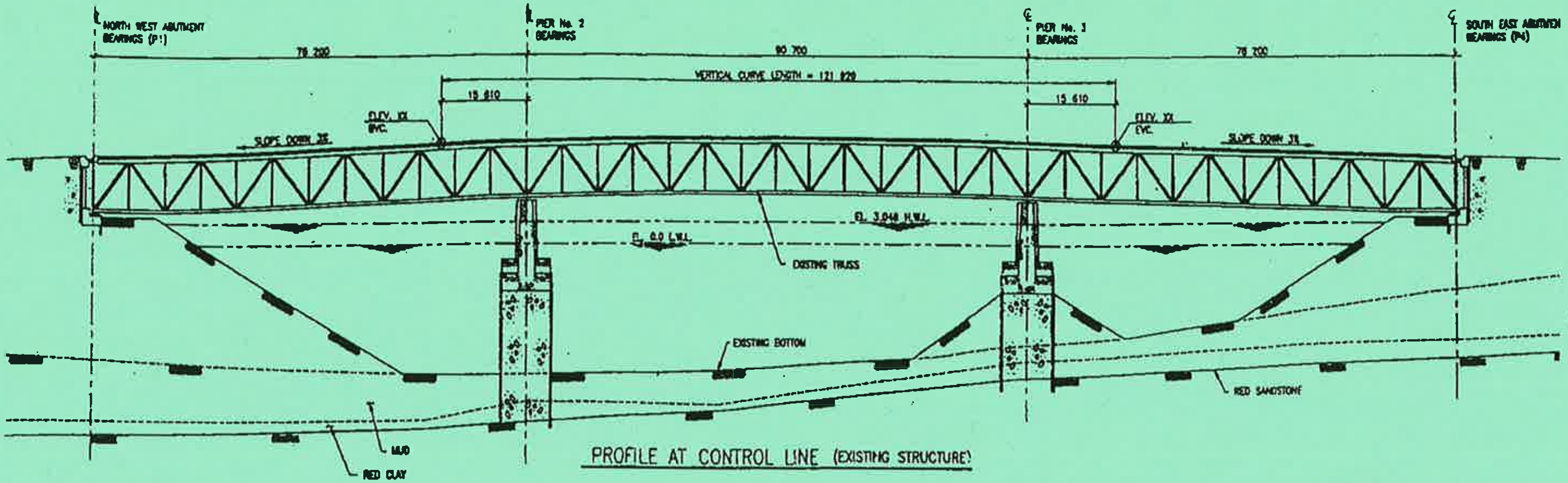
DATE:	December 13, 1996	
COMPANY:	**CBCL**	**Jacques Whitford**
ATTENTION:	**Mr. Gerry Strolz**	**Mr. George Zafiris**
FAX NO.:	**(902) 423-3938**	**566-2004**
FROM:	Mr. Steve Pletch	

TOTAL NUMBER OF PAGES: 5
(including cover sheet)

Original to follow? Yes No ✓

Subject: Hillsborough Bridge - Gewi Plie Installation Drawings

For Your Information.



PROFILE AT CONTROL LINE (EXISTING STRUCTURE)

DSI

SCALE: N.T.S	APPROVED BY:	DRAWN BY A.C.
DATE: DEC 11 1996		REVISED

Hillsborough River Bridge

Gewi Pile Installation

DRAWING NUMBER HRB-001

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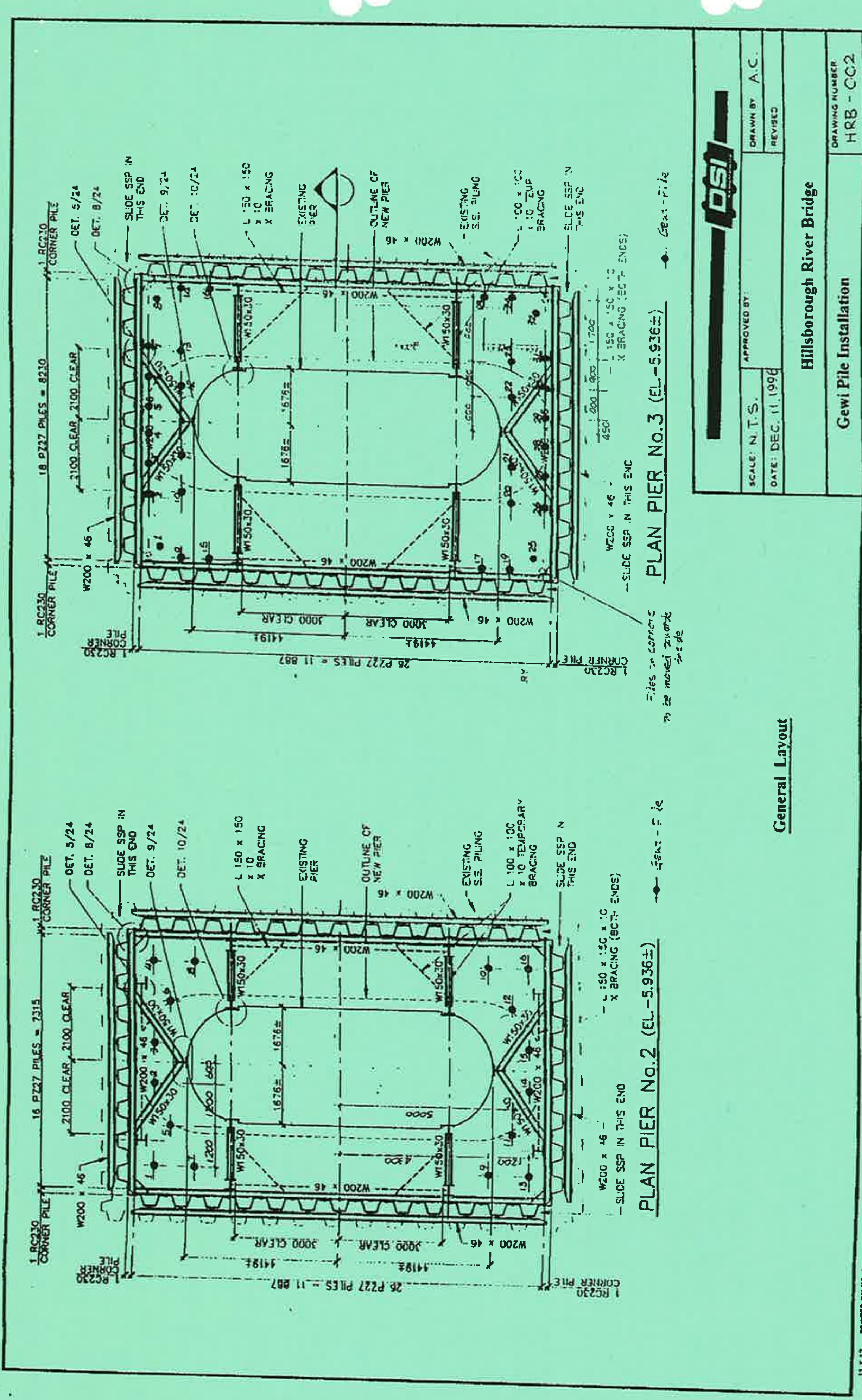
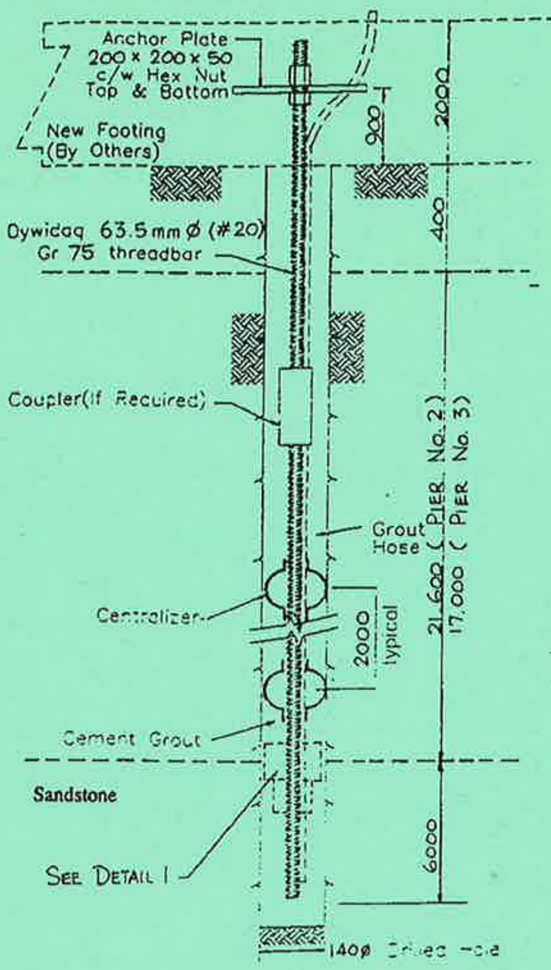
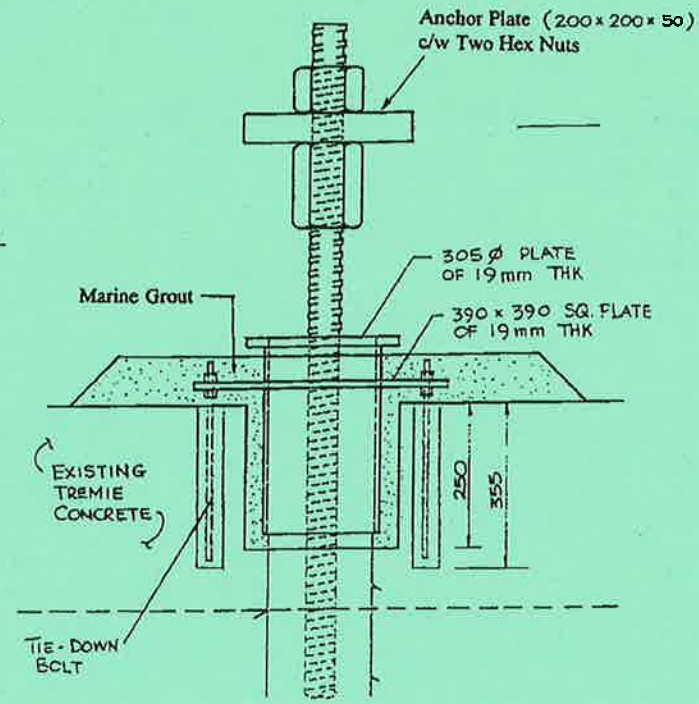
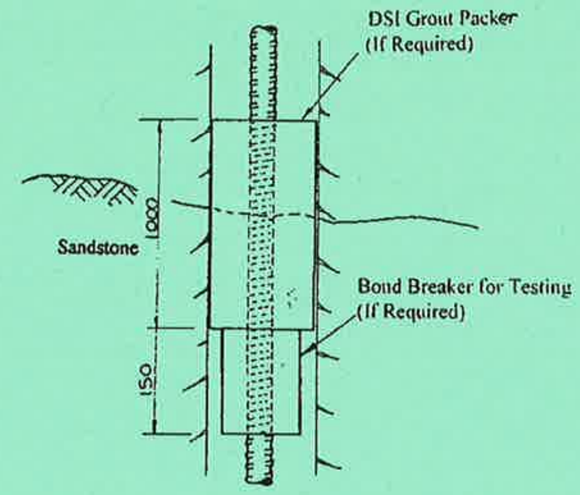

PLAN PIER No.3 (EL-5.936±)
PLAN PIER No.2 (EL-5.936±)
18 PZ27 PILES = 8230
16 PZ27 PILES = 7315
26 PZ27 PILES = 11 887
1 RC230 CORNER PILE
W200 x 46
W150x30
Gewi - Pile
General Layout
Hillsborough River Bridge
Gewi Pile Installation
SCALE: N.T.S.
DATE: DEC. 11, 1996
DRAWN BY A.C.
APPROVED BY:
REVISED
DRAWING NUMBER
HRB - CC2



Gewi - Pile Details



Pile Head Detail



Detail I

DSI

SCALE: N.T.S.	APPROVED BY:	DRAWN BY A.C.
DATE: DEC. 11, 1996		REVISED

Hillsborough River Bridge

Gewi Pile Installation

DRAWING NUMBER HRB-003

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General Notes.

1. Material
   1.1 Gewi-Pile
   #20 Grade 75 Ksi Dywidag Threadbar by Dywidag - Systems International.

   Yield Capacity = 368 Kips
   Ultimate Capacity = 491 Kips
   Max. Design Load = 0.6 X Yield Capacity (220 Kips)

   1.2 Grout
   Type 30 Cement, w/c = 0.5(max), fc' = 30 MPa (min) at 7 days

2. Drilling
   Drill hole for anchor by using percussion drilling method with casing liner if required.

3. Installation and Grouting
   3.1 Water testing the borehole (if required).
   3.2 Install the pre-assembled threadbar into the bore hole.
   3.3 Trimie grout the bore hole with the grout line attached to the bottom of the anchor.
   3.4 For the test pile only, two stages grouting shall be required and bond breaker shall be installed on top of the bonded length of the pile.

4. Testing (Gewi-Piles shall be tested in tension: one in every four piles)

   4.1 Performance Test (As Per PTI Recommendations)
   Performance Tension Test on TWO (2) production Gewi-piles only. (One at each pier)

   Procedure as follow:

   Design Load P = Working Load = 220
   Max. Test Load = 60% Yield of Bar (220 Kips)
   Alignment Load = 10% Design Load

   .1 x 220 = 22 kips

Testing Procedure:

Stress anchor to Alignment Load and dial guage will be set for zero at this datum load.
The test shall be made by incrementally loading and unloading starting from the Alignment Load to the following maximum cycle loads:

0.25 P
0.5 P
0.75 P
1.0 P
1.33 P / Max. Test Load 300 kips
292.6

At each increment, the movement of the anchor shall be recorded to the nearest 0.001 inches.The creep readings at the maximum test load shall be read at 0, 1, 2, 3, 4, 5, 10. 15, 20, 25, 30, 45, and 60 minutes. Unload the anchor to 100%, 75%, 50%, and 25% of the Design Load and then back to the Alignment Load and measure the anchor extension at each load.

4.2 Proof Test (remaining anchors)

Max. Test Load = 60% Yield of Bar (220 Kips)
Alignment Load = 10% Design Load

220 kips

Testing Procedure:
Stress anchor to Alignment Load and dial guage will be set for zero at this datum load. Continue stressing anchor incrementally to 25%, 50%, 75%, 100% of the Design Load. At each increment the movement of the anchor shall be recorded to the nearest 0.001 inches. Stress anchor to 100% of the test load and hold for 10 minutes, the elongation shall be measured at 1, 2, 3, 4, 5 and 10 minutes Unload the anchor to 100%, 75%, 50%, 25% and 10% of the design load and measure the anchor extension at each load.

900 kN = 200 kips

DSI		
SCALE: N.T.S	APPROVED BY:	DRAWN BY A.C
DATE: DEC 11 1996		REVISED
Hillsborough River Bridge		
Gewi Pile Installation		DRAWING NUMBER HRB-004

11 X 17 PRINTED ON NO. 1000H CLEARPRINT

ASPECTS OF THE DESIGN AND CONSTRUCTION OF THE NEW HILLSBOROUGH RIVER BRIDGE IMPROVEMENT SCHEME

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Gerry STROLZ, M.I.Struct.E., C.Eng.
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Strait Crossing Inc.
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Calgary, AB T2R 1K9

Summary

This paper describes the widening, strengthening, and re-decking of the Hillsborough River Bridge in Charlottetown, PEI, from an original two lane configuration to accommodate four lanes of traffic and sidewalks on both sides of the bridge. The paper describes the installation of two new steel box girders placed one on each side of the existing box truss structure, the deck replacement and the associated widening and strengthening of the existing pier and abutment substructures.

INTRODUCTION

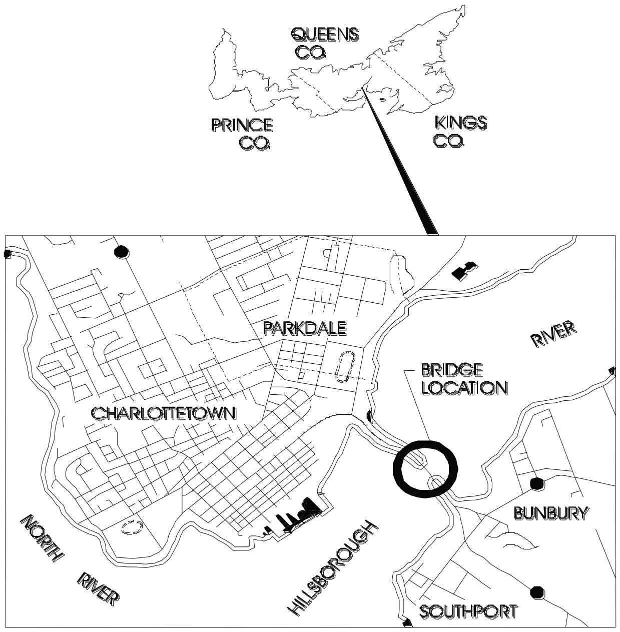

Figure 1

The first bridge across the Hillsborough River between Charlottetown and Mutch's Point, the present day community of Stratford, was completed in 1905. The single vehicle lane/railway track was carried on a series of steel "through-truss" structures supported on 12 masonry piers spanning up to 65 m per span; a swing span in the navigation channel allowed ships to pass up the Hillsborough River. The total length of the bridge was approximately 770 m between abutments, with approach fills 400 m in length on the Charlottetown side and 150 m long at Mutch's Point, extended from the river embankments (Figure 1).

In the late fifties, the old bridge was replaced. The existing causeways were extended along a new alignment toward the center navigation channel and connected by a new two lane bridge structure approximately 270 m in overall length. The new two lane bridge and approach roads were completed in 1961. The new bridge consisted of a three span continuous steel box truss 7.3 m deep and 247.2 m in length between the abutment expansion joints, and two 11.6 m long approach spans, one at either end. The total length of the new structure was 270.4 m.

At the present time the bridge carries between 19,000 and 26,000 vehicle trips on a normal day. During rush hours, traffic intensity increases to 1600 vehicle trips per hour in one direction, while at the peak of the tourist season, the daily vehicle trips carried by the bridge can exceed 30,000 vehicles. This volume of traffic is expected to continue to increase following the completion of the Confederation Bridge between New Brunswick and Prince Edward Island, which was opened in May of 1997. Because of the importance of the Hillsborough Bridge as a major road connection it was imperative that the traffic route would be kept open during any replacement or widening of the structure. A preliminary design study to widen the existing structure from two lanes to three was carried out by CBCL Limited in 1992, however the scheme, which involved the replacement of the original cast-in-place deck with new precast concrete deck units and stay cable strengthening of the truss structure, was not implemented as a final design.

CONCEPT

In 1994, Hillsborough Bridge Development Inc., on behalf of the PEI Department of Transportation and Public Works, engaged CBCL Limited to develop a number of alternative schemes for widening or replacing the existing structure to provide for four lanes of traffic and to establish budget costs for each of the schemes investigated. The three primary schemes reviewed were:

1. Total replacement of the existing structure with a new four lane bridge on a new but parallel alignment adjacent to the existing structure.

2. Construction of a new two lane bridge on new foundations adjacent to the existing structure, followed by replacement of the deteriorated concrete deck and refurbishment and strengthening of the existing box truss structure. Several sub-schemes were developed for this scenario, including jacking the existing structure, sliding it laterally onto new bearings supported on widened and strengthened pier and abutment superstructures, on extended abutment and pier substructure foundations. The two structures would therefore share the existing pier and abutment substructure foundations.

3. Construction of two new single lane structures, one on each side of the existing box truss structure, on widened and strengthened pier and abutment structures, as per the previous scheme, but with the three structural elements integrated with a common concrete deck slab for the purposes of load sharing.

The final scheme selected as being the least costly and also meeting all of the design requirements, namely uninterrupted two lane traffic flow at all times, and upgrading of the highway loading from the existing PEI "B" Train Loading to CS600 loading, was the placement of a steel box girder on either side of the existing box truss, with the three structural elements connected by a new but common reinforced concrete deck slab to accommodate the required four lanes of traffic. The new and the existing structures would then also share the existing foundations which would be modified to accommodate the increase in width and dead weight of the structure as well as the increase in live loading (Figure 2).

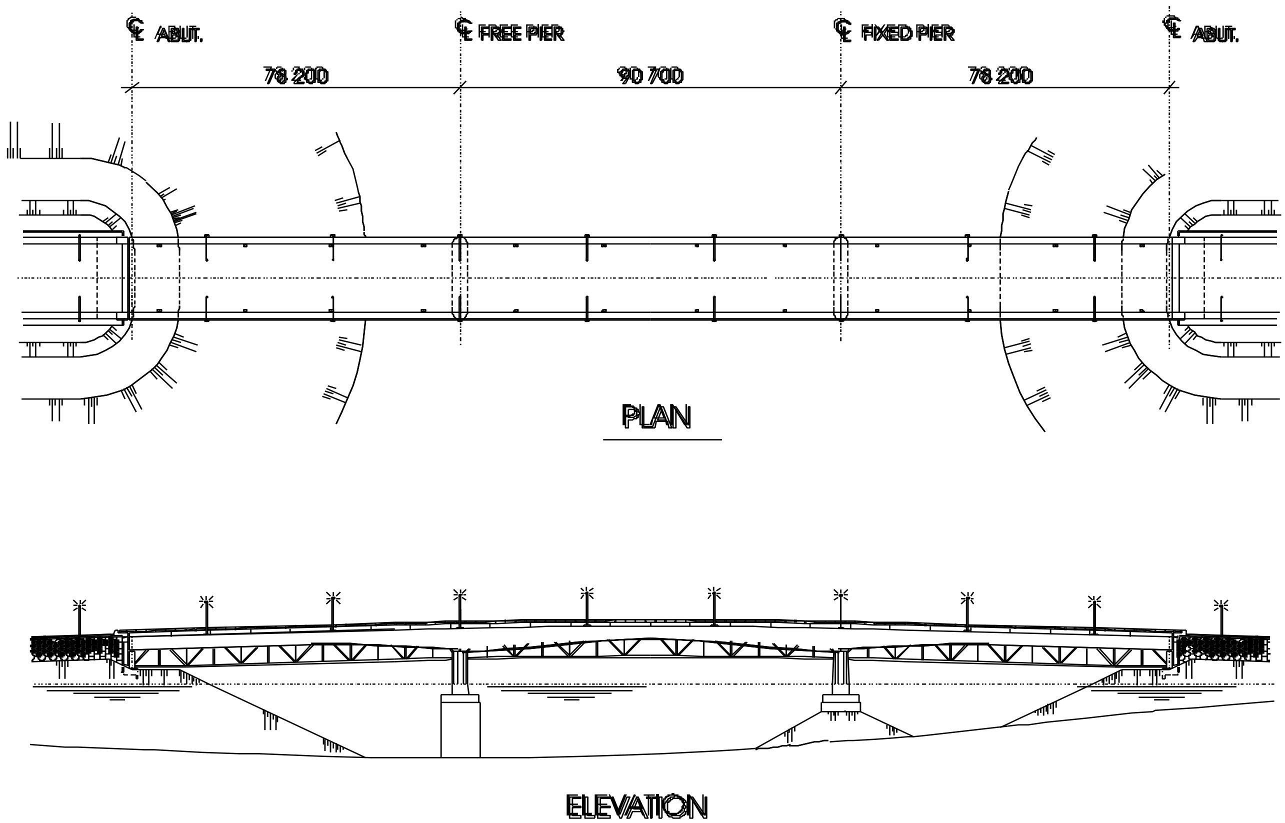

Figure 2

SUPERSTRUCTURE

The new modified superstructure retains the existing 7.3 m deep, 78.2 m, 90.7 m, 78.2 m three span continuous box truss structure, suitably strengthened, but with the two 11.6 m approach spans eliminated and replaced with new abutment structures constructed immediately adjacent to the two 78.2 m side spans. Two new variable depth steel box girders were placed, one on either side of the truss to carry the widened portions of the new deck and sidewalks. Considerable care was taken in establishing the underside profile of the boxes to obtain an aesthetically pleasing form to complement the existing box truss structure. It was also vital to maintain compatibility between the longitudinal profile of the new and strengthened portions of the structure whilst minimizing the additional weight of the new combined superstructure system.

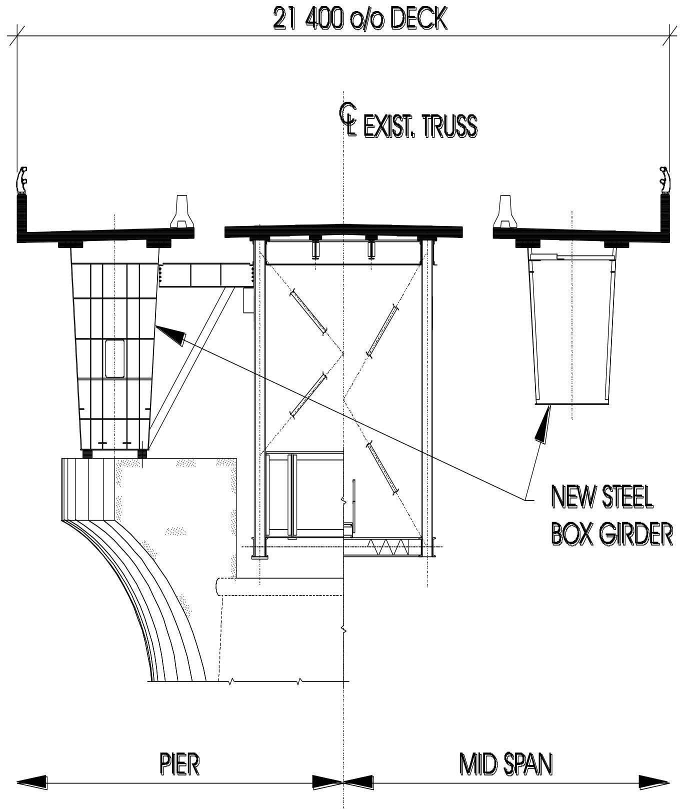

Figure 3

The construction of the superstructure was planned to be carried out in three phases. Following construction of the modified pier and abutment structures, the steel box girders were to be erected and a 250 mm deep composite concrete deck slab, 5.8 m wide, was to be cast-in-place on the girders. Cast-in-place outside permanent barrier walls would be constructed, and a temporary asphalt wearing surface laid on the deck. It was intended that traffic would then be re-routed from the old box truss structure onto the new box girder structures, enabling the design objective of maintaining traffic flow over the bridge without lane disruption or closure to be achieved. Jersey barriers were placed on the inside of the deck to channelize the traffic flow and provide a temporary barrier for safety purposes (Figure 3).

During the second phase of construction it was planned that the old concrete deck on the box truss would be removed, the box truss strengthened and a new variable depth composite concrete deck slab, 7.8 m wide, would be cast-in-place on the box truss following strengthening. During the third and final phase, it was planned that the 1.0 m wide longitudinal closure strips between the three previously cast concrete decks would be poured, temporary barriers removed, concrete sidewalks installed, and a new and final layer of asphalt would be placed on the now completed and continuous deck.

The design of the steel box girders was carried out for two distinct loading conditions. In the initial stage the composite concrete and steel box girders were required to carry a portion of the deck slab, the permanent barrier wall, the temporary concrete barrier and a single lane of traffic as a unique structural element. Analysis of the box girders was carried out to determine deflection, moments and shears. In the final stage, the box girders were integrated to work through the participation of the concrete deck slabs in conjunction with the box truss to carry the superimposed dead load due to asphalt and the upgraded CS600 traffic live loading.

A grillage analysis was used to model the behaviour of the structure, in particular the distribution of loading between the truss and box girder structures, and to confirm the deflected behaviour of the structure at various stages of construction. The basic stiffness of the truss structure resulted in some shedding of the box girder lane loading and this, combined with the upgrade in live loading, required selective strengthening of the existing truss structure. The sequence of the closure of the longitudinal joint between the new box girder bridge deck and the replaced bridge deck supported by the box truss, combined with the stiffness of the variable depth box girder structures, 5.0 m deep at the piers and 3.0 m deep at mid-span, was used to limit the transfer of live loading and minimize the truss strengthening required.

In like manner, the existing steel box truss was re-analyzed for two load conditions; in the initial stage for the dead load due to self weight, and the new composite concrete deck slab, and in the final stage as an integrated unit with the two box girders for superimposed dead load and for the CS600 live loading.

Truss strengthening was achieved by a combination of plating of the existing web and chord members, and by the introduction of additional bracing members to improve the slenderness ratio of certain compression elements. Top chord compression strength was improved by the addition of shear studs to provide composite action with the new concrete deck. It was planned that strengthening of the truss members would be carried out after the removal of the existing concrete deck slab, to take advantage of lower initial stress levels in the members prior to strengthening. A

Dywidag bar arrangement was used to apply an active force, equal to the initial stress level in the truss structure in the unloaded condition, to reinforce the centre span region of the bottom chord of the truss. The Dywidag arrangement was configured to act concentrically with the bottom chord member to avoid the introduction of secondary flexural stresses due to eccentricity of the applied load.

In re-analyzing the truss member capacities, an allowance to account for the existing levels of corrosion in truss members was introduced. A corrosion evaluation report had previously established a current upper bound value of 10% of material loss from the cross-sectional areas of members. This 10% figure was used in calculating the residual capacity of members for the program of strengthening. For the purposes of providing for future corrosion, an additional 6% of material loss was taken into account.

During construction a number of design changes were made to the original design to facilitate construction schedule improvements. The composite cast-in-place deck slab on the box girders was replaced with 17 tonne, full depth (250 mm) site precast concrete units 6.45 m long, 5.5 m wide, with 700 mm wide transverse joints arranged between adjacent precast panels. The panels were conventionally reinforced (no prestressing) and were cast using a nominal 45 MPa silica fume concrete; cylinder results indicated a minimum strength of 55 MPa at 28 days. Ports were arranged within the precast panels to ensure composite girder-deck action. The reinforcing was arranged and distributed within the precast units to facilitate reinforcement interlock and to avoid interference during deck placement between the longitudinal continuity reinforcement located in adjacent panels. After all the precast units were in place over the full length of the box girders, the 700 mm transverse joint strips between the precast units and the shear stud ports were closed with the 45 MPa silica fume concrete. The precast units were shimmed to a predetermined profile to reflect the final alignment of the integral box girder and box truss deck elevations and grouted through the shear stud ports with the same silica fume concrete.

Truss strengthening of the bottom chords and the web members was also commenced for schedule reasons whilst the box truss structure was still in full operational use. The strengthening details were therefore modified to accommodate the change in the residual stresses in the web members. The Dywidag bar reinforcement of the bottom chord was installed but not tensioned until the box truss deck had been removed, illustrating the flexibility of the reinforcement strategy selected. Similarly, a precast deck system over the box trusses to replace the cast-in-place deck system was considered to simplify working over water but was found to be uneconomic and therefore the original cast-in-place deck scheme was implemented.

SUBSTRUCTURE/ABUTMENTS

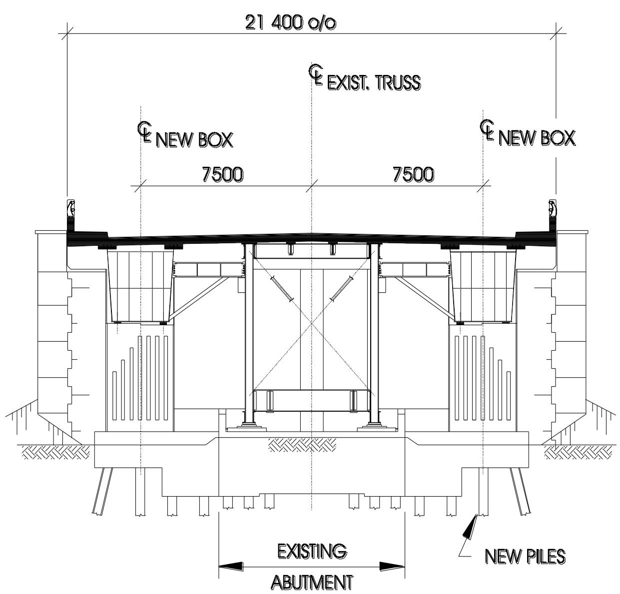

Figure 4

The original abutment foundations for the box truss were each carried on fourteen steel H-piles driven to refusal in fractured sandstone at an ultimate resistance of 150 tons per pile. An additional sixteen HP310 x 132 kg/m steel piles were installed at each abutment to carry the increase in dead and live loads originating from the new box girder and deck structures. A new concrete pile cap was designed to mobilize the additional piling and encapsulate the existing pile cap in order to redistribute the additional and existing loads as evenly as possible over the whole thirty pile group. The new pile cap was designed to encourage uniformity of deflection and settlement as the additional superstructure loads were applied. Two new concrete pilasters, each 2.6 m by 3.0 m wide, were installed on each side of the box truss to carry the new steel box girders. The back of the pilasters were extended up to deck level to pick up new concrete approach slabs, expansion joints and wing walls. This work was phased to accommodate the uninterrupted traffic flow design requirement (Figure 4).

The original approach spans at each abutment were eliminated from the new scheme; this was achieved by placing a mechanically stabilized earth wall structure immediately behind the extended pile cap system. Installation of the M.S.E. wall system was phased to accommodate construction of the widened roadway approach fills and to allow the existing approach span, between the original and subsequently abandoned bank seat abutment and the box truss, to remain in place until traffic was re-routed onto the steel box girders. Erection of the precast wall face elements was completed to the underside of the approach span in one operation (Figures 5 and 6). Temporary partial height M.S.E. (expanded metal) walls were installed on either side of the box truss approach span to facilitate its subsequent removal without disrupting the integrity of the previously placed approach fills and impeding the traffic already re-routed to the flanking box girder structures.

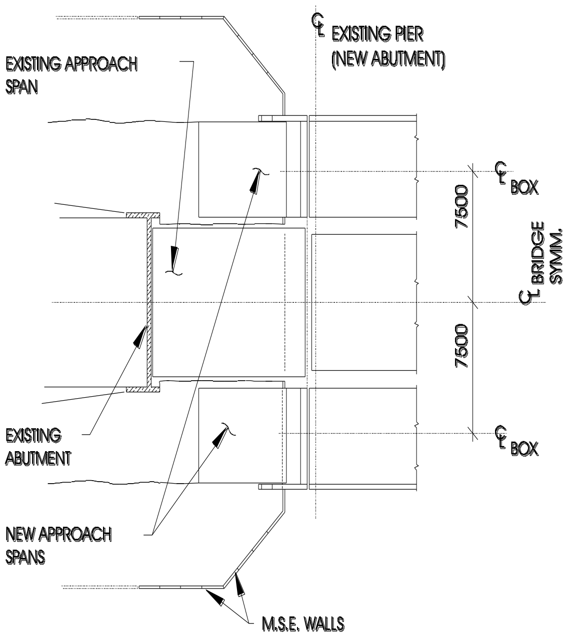

Figure 5

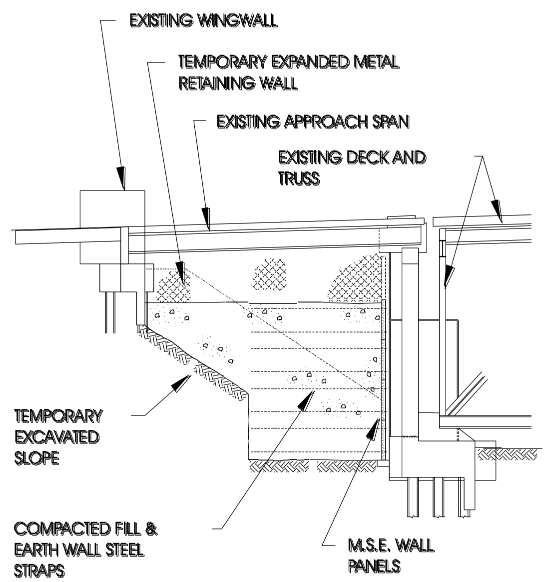

Figure 6

SUBSTRUCTURE/PIERS

The two existing pier substructures are constructed of rectangular steel sheet pile single cell caissons, founded on the sandstone bedrock material. The caissons are filled with a somewhat incompletely pressure grouted concrete at the free pier and a poorly consolidated tremie concrete at the fixed pier and are founded at an elevation up to 6.7 m below the low tide level. A 2.4 m thick reinforced concrete transfer slab carries a 10.0 m high by 8.8 m wide by 3.4 m deep reinforced concrete pier shaft at the transfer slab elevation, tapering to 8.2 wide by 2.5 m deep at the truss bearings elevation.

The fixed pier caisson is 14.0 m long by 10.1 m wide by 14.6 m deep and was reinforced at the time of construction, because of its poorly consolidated tremie concrete content, by ten 760 mm diameter piles and further stabilized with a rock berm on the four sides of the caisson. The dimension from the underside of this caisson to the top of the pier shaft is 27.4 m. The free pier caisson is slightly smaller in plan area, being 14.0 m long by 9.2 m wide and is 19.4 m deep. The dimension from the underside of this caisson to the top of the pier shaft is 32.0 m. The top of both pier shafts is at an elevation 6.0 m above the low tide mark.

The existing pier shafts were modified geometrically to accommodate the new steel box girder on each side of the existing truss structure, hence the shape of the new pier structure. It was also imperative that the increase in size of the pier shafts be kept to a minimum dimension below the high tide mark in order to limit the increase in the ice forces acting on the piers and caissons. These requirements were achieved by encapsulating the existing pier within an enlarged pier shaft, 11.8 m long by 4.0 m wide, below the high tide mark. Above the high tide level the ends of the new pier shaft were flared outwards to provide seating for the girder bearings. The ends of the pier shaft were rounded to minimize ice loads on the shaft, and the rounded profile was extended up to blend into the flared area under the bearings, resulting in an aesthetically pleasing pier profile. The bottom of the new encapsulating pier shaft was anchored to a new pier base slab cast on top of the existing transfer slab. The new pier base also encapsulates the ends of the Gewi-Pile reinforcement which extends through the existing caissons into rock sockets drilled into the sandstone bedrock below. The pier shaft was circumferentially post-tensioned to resist tensile shrinkage splitting stresses created by the encapsulation of the existing pier shaft. The enlarged pier flares were reinforced and vertically post-tensioned to counteract the tensile stresses created in the corbel required to support the new variable depth box girders at an elevation approximately 3.0 m higher than that of the existing box truss (Figure 7).

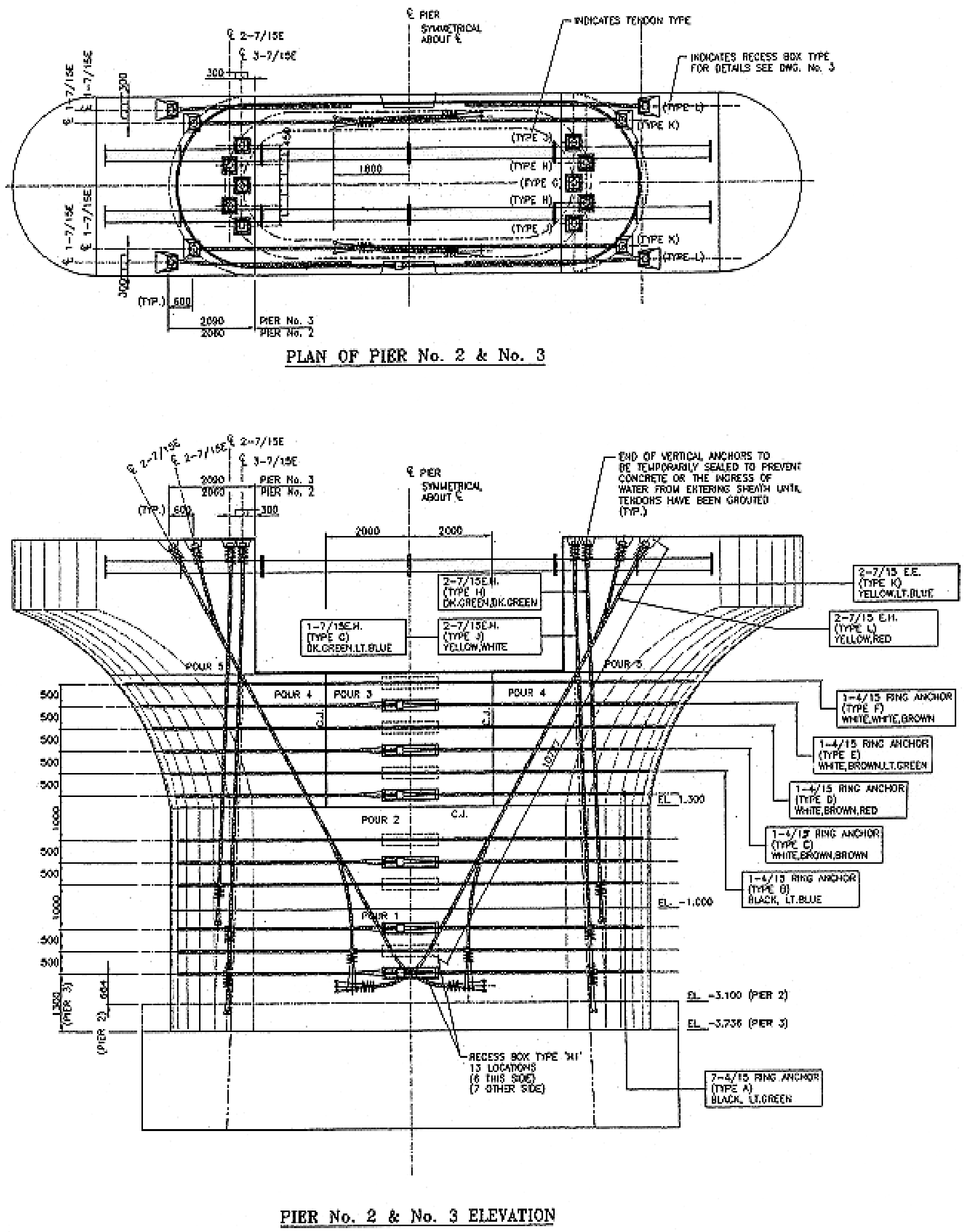

Figure 7

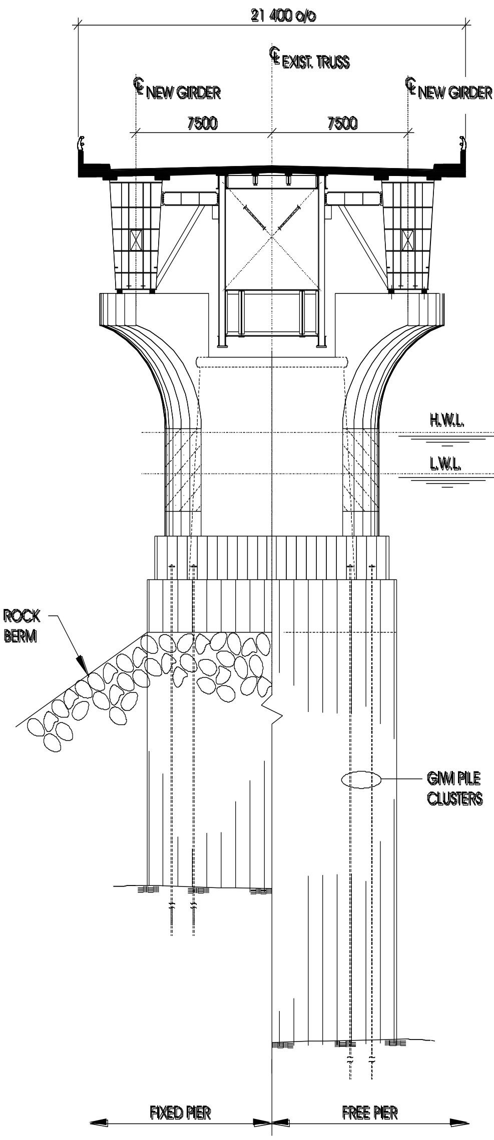

Figure 8

The corbel post-tensioning was supplemented by a pair of 275 mm diameter, 25 mm wall thickness pipe assemblies located symmetrically on either side of the box truss vertical bearing chord. The pipe assemblies were provided with sufficient clearance so as not to impede movement of the box truss under varying temperature conditions at the free pier location and were embedded beyond the vertical post-tensioning anchors in the pier corbel concrete. A snug tension was introduced into the pipe assemblies by tensioning abutting pipe flanges at a location coincident with the centre line of the box truss (Figures 8).

Headroom and access limitations required that strengthening of the pier sub-structure caissons be achieved using the Dywidag Gewi-pile System. Gewi–pile clusters consisting of 8 piles per cluster for the fixed pier and 4 piles per cluster for the free pier were installed in the four corner regions of the existing caissons to improve their axial load carrying capacity and overturning stability. The Gewi-piles were bored through the pressure grouted caisson of the free pier and through the poorly consolidated tremie concrete of the fixed pier, with the piles being terminated 6.0 m below the underside of the caisson in the sandstone bedrock strata. The rock socket dimensions facilitated the development of full Gewi-pile capacity under ultimate load conditions. Each pile in the cluster consisted of a #20 Dywidag bar installed inside a 145 mm diameter hole and subsequently grouted over its entire length. The top of the Gewi-piles were anchored into the new pier base cast over the top of the original transfer slabs. Load tests carried out on selected piles in each pier caisson established the working pile capacity at 800 KN per pile. These pile capacities were higher than was originally calculated from the preliminary and previously established bedrock data. The original design had envisioned a requirement for a greater number of Gewi-piles to reinforce the load transfer capacity of the caissons at the caisson/bedrock interface. However, as data from the pile boring logs became available, confirming the condition of the pressure grouted and tremie placed concrete within the caissons and also the results of the pile pull-out tests, it proved possible to reduce the number of Gewi-piles required to be installed to meet the loading requirements. In the final condition, a total of 14 piles were successfully installed in the free pier caisson and a total of 24 piles were successfully installed in the fixed pier caisson.

CONSTRUCTION CHALLENGES

The bridge is located at the tip of two causeways that were constructed as part of an earlier crossing project. Access to the bridge abutments was limited to use of the existing causeway road surface; the traffic management plan therefore had to recognize the limitations in how this access could be used. The bridge itself is located in the tidal region of the Hillsborough River and the causeway constriction causes water to flow through the narrows with a speed greater than 2m/s, which limits access by water to slack tide conditions (one hour every six).

Due to the very deep water (30m+) the design concentrated on the objective of performing strengthening and increasing foundation capacity by working through and within the confines of the existing pier substructures. A scheme to mount a cofferdam on top of the existing pier and to drill strengthening Gewi-piles within the pier substructure plan dimensions into the bedrock below therefore required significant planning. The fast flowing currents, the tidal conditions and occasional violent storms imposed a severe operational environment on the cofferdam structures as well as considerably impeding barge positioning during girder erection.

Figure 9

The need to erect large and heavy girder components around the bridge was further hampered by the fact that the clearance under the existing bridge is only 4 m at high tide. As marine erection was the only means available, selection of a specialty crane (a 300t modified Lima) mounted on a barge proved to be the best solution. By fully ballasting the barge the crane was passed under the bridge at low tide with only 450 mm of clearance remaining (Figure 9).

The increase in traffic that led to the decision to improve and widen the structure also posed perhaps the greatest construction challenge. As observed in the introduction, traffic volumes of up to 26,000 vehicles per day routinely utilize the Hillsborough River Bridge to cross between Charlottetown and Stratford. Alternate routes to cross the river involve a 46km detour on secondary roads. With this in mind, phasing of the work was devised to keep the bridge in operation throughout the construction period. Work was therefore strictly limited during peak morning and evening traffic flow periods, and reduction of the structure to a single lane was only allowed between 11:00 p.m. and 7:00 a.m. each day.

In planning to rehabilitate and improve an older structure such as the Hillsborough box truss, as much information as possible pertaining to the original construction, as well as a current condition survey had to be gathered at the outset. Notwithstanding this, continual assessment of existing conditions had to occur on an ongoing basis throughout the work.

The project, being a Public Private Partnership initiative was construction focussed, which allowed for a team approach between the Contractor and Consultant. This cooperation was vital to quickly assess new information concerning existing conditions as soon as it became available, in order that the impact on construction, in the form of additional work or delays, would be minimized. This working relationship also permitted the development of alternative approaches throughout the construction process, which allowed schemes jointly devised by the design and construction team to be implemented, and enabled the benefits derived, in the form of cost savings or schedule improvements to be realized. This cooperative approach also enabled the inevitable problems encountered during construction to be speedily addressed and resolved.

The bridge improvement scheme was estimated, at the time of sub-trade tender, to amount to $12 million for the bridge superstructure and substructure works alone.

ACKNOWLEDGEMENTS

The bridge works were designed by CBCL Limited for Hillsborough Bridge Development Inc., with conceptual and detailed design input being provided by Dr. Gamil Tadros of Strait Crossing Inc. Strait Cross Inc., on behalf of Hillsborough Bridge Development Inc., was also responsible for the planning, construction, and financing of the project for the PEI Department of Transportation and Public Works. The following were also significant contributors to the success of the project and their contribution is acknowledged:

Structural Steel Box Girders – Cherubini Group of Companies
Steel Erection – Marid Industries Limited
Post-Tensioning – Harris P.T., VSL Canada Ltd.
Gewi-Piles – Dywidag Systems International

HILLSBOROUGH BRIDGE
CHARLOTTETOWN.

The docummentation for the construction of this bridge was handed over by Strait Crossing Inc. to the Department on completion of the project.

The following list of documents was developed to assist with the retrieval of required information in future.

The material as a whole is divided into drawings (filed in 18 tubes) and binders (contained in 4 boxes.)

Drawing files.

TUBES	Drawings prepared by	Information	Pages
1-4	O.J. McCulloch.	Existing Bridge Drawings from 1959-1960	3-5
5	Harland Associates Locus Surveys.	Approach road drawings.	6
6-7	Harland Associates.	Roadway improvement drawings- as built. (Plan and profile, cross-sections ,details, traffic signals, force-main)	7
8-9	Harland Associates. Shaw Pipe.	Roadway improvement drawings Plan and profile. (8) Cross sections Shop drawings (9)	8
10	Harland Associates. Watson Bowman, Friman Steel.	Pedestrian Underpass drawings.	9
11	Harland Associates. EFCO. Watson Bowman.	Bridge Improvement Misc. Shop-drawings. Bearings	10
12	HARRIS.	Reinforcing placing drawings	

13-14	CBCL	Bridge Improvement,Design drawings	11
15	CBCL	Bridge Improvement As- Built drawings.	12
16-17	CHERUBINI	Bridge improvement,shop- drawings	13
18	CHERUBINI	Pier Temp. Works	14

FOR CONTENTS OF BOX FILES SEE PAGE 15.

STRAIT CROSSING INC.

Bridge Improvements

Design Drawings

Drawing List TUBES 13-14

Consulting Engineer	Drawing Name	DWG #	DWG Date	Rev #	Rev Date
CBCL Limited	**CONTRACT 94846**				
CBCL Limited	General Arrangement Sheet No. 1	1	June 1995		
CBCL Limited	General Arrangement Sheet No. 2	2	June 1995		
CBCL Limited	Abutment Details	3	June 1995	1	June 1995
CBCL Limited				2	Sept 16/96
CBCL Limited				3	Sept 23/96
CBCL Limited				4	Feb 11/97
CBCL Limited	Pier Details	4	June 1995		
CBCL Limited	Pier post tensioning & misc substructure details	5	June 1995		
CBCL Limited	Deck Plan & Details	6	June 1995		
				1	Dec 19/97
CBCL Limited	Deck Elevations & Details	7	June 1995		
				1	Dec 19/97
CBCL Limited	Expansion Joints Details	8	June 1995		
					Apr 18/97
CBCL Limited	Municipal & Bridge Services	9	June 1995		
					Apr 21/97
CBCL Limited	Abutment Reinforcing	10	June 1995		
CBCL Limited				1	Sept 16/96
CBCL Limited				2	Sept 23/96
CBCL Limited				3	Feb 11/97
CBCL Limited	Pier Reinforcing	11	June 1995		
CBCL Limited	Deck Reinforcing	12	June 1995		
				1	Feb 06/98
CBCL Limited	Box Girder Plan Layout	13	June 1995	1	June 24/96
CBCL Limited	Box Girder Elevations	14	June 1995		
CBCL Limited	Box Girder Bottom Plate Plans	15	June 1995		
CBCL Limited	Box Girder Sections	16	June 1995		
CBCL Limited	Box Girder Sections & Details	17	June 1995		
CBCL Limited	Box Girder Splice Details Sheet No. 1	18	June 1995		
CBCL Limited	Box Girder Splice Details Sheet No. 2	19	June 1995		
CBCL Limited	Bearing Details - New Box Girders	20	June 1995		
CBCL Limited	Abutment Staging	21	June 1995		
CBCL Limited				1	Sept 16/96
CBCL Limited				2	Sept 23/96
CBCL Limited				3	Feb 11/97
CBCL Limited	Sub Structure Phasing Sheet No. 1	22	June 1995		
CBCL Limited	Super Structure Phasing Sheet No. 2	23	June 1995		
CBCL Limited	Pier Temporary Works	24	June 1995	1	June 24/96
CBCL Limited	Existing Truss Reinforcing Elevation & Details	25	June 1995	1	June 11/97
CBCL Limited	Pier No. 2 Temporary Works Modifications	26	Mar. 13/97		
CBCL Limited	Panel Layout	27	July 1997	1	July 18,1997
CBCL Limited	Panel Layout	28	July 1997		

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◆ STRAIT CROSSING INC. ◆

Bridge Improvements

As-built Drawings

Drawing List - TUBE 15

Consulting Engineer	Drawing Name	DWG #	DWG Date	Rev #	Rev Date
CBCL Limited	**CONTRACT 94846**				
CBCL Limited	General Arrangement Sheet No. 1	1	June 1995	As-built	
CBCL Limited	General Arrangement Sheet No. 2	2	June 1995	As-built	
CBCL Limited	Abutment Details	3	June 1995	As-built	
CBCL Limited	Pier Details	4	June 1995	As-built	
CBCL Limited	Pier post tensioning & misc substructure details	5	June 1995	As-built	
CBCL Limited	Deck Plan & Details	6	June 1995	As-built	
CBCL Limited	Deck Elevations & Details	7	June 1995	As-built	
CBCL Limited	Expansion Joints Details	8	June 1995	As-built	
CBCL Limited	Municipal & Bridge Services	9	June 1995	As-built	
CBCL Limited	Abutment Reinforcing	10	June 1995	As-built	
CBCL Limited	Pier Reinforcing	11	June 1995	As-built	
CBCL Limited	Deck Reinforcing	12	June 1995	As-built	
CBCL Limited	Box Girder Plan Layout	13	June 1995	As-built	
CBCL Limited	Box Girder Elevations	14	June 1995	As-built	
CBCL Limited	Box Girder Bottom Plate Plans	15	June 1995	As-built	
CBCL Limited	Box Girder Sections	16	June 1995	As-built	
CBCL Limited	Box Girder Sections & Details	17	June 1995	As-built	
CBCL Limited	Box Girder Splice Details Sheet No. 1	18	June 1995	As-built	
CBCL Limited	Box Girder Splice Details Sheet No. 2	19	June 1995	As-built	
CBCL Limited	Bearing Details - New Box Girders	20	June 1995	As-built	
CBCL Limited	Abutment Staging	21	June 1995	As-built	
CBCL Limited	Sub Structure Phasing Sheet No. 1	22	June 1995	As-built	
CBCL Limited	Super Structure Phasing Sheet No. 2	23	June 1995	As-built	
CBCL Limited	Pier Temporary Works	24	June 1995	As-built	
CBCL Limited	Existing Truss Reinforcing Elevation & Details	25	June 1995	As-built	
CBCL Limited	Pier No. 2 Temporary Works Modifications	26	Mar. 13/97	As-built	
CBCL Limited	Panel Layout	27	July 1997	As-built	
CBCL Limited	Panel Layout	28	July 1997	As-built	

◆ STRAIT CROSSING INC. ◆

Bridge Improvements
Structural Steel Drawings
Drawing List – TUBES 16–18

Consulting Engineer	Drawing Name	DWG #	DWG Date	Date Rec'd	Rev #	Rev Date
CHERUBINI						
Cherubini	Box Girders G1N & G1S	1	Oct 4/96	Jan 14/97	2	Dec 20/96
Cherubini	Box Girders G2N & G2S	2	Oct 4/96	Jan 14/97	2	Dec 20/96
Cherubini	Box Girders G3N & G3S	3A	Oct 4/96	Jan 14/97	2	Dec 20/96
Cherubini	Box Girders G3N & G3S	3B	Oct 4/96	Jan 14/97	2	Dec 20/96
Cherubini	Box Girders G4N & G4S	4A	Oct 4/96	Jan 14/97	2	Dec 20/96
Cherubini	Box Girders G4N & G4S	4B	Oct 4/96	Jan 14/97	2	Dec 20/96
Cherubini	Box Girders G5N & G5S	5A	Oct 4/96	Jan 14/97	2	Dec 20/96
Cherubini	Box Girders G5N & G5S	5B	Oct 4/96	Jan 14/97	2	Dec 20/96
Cherubini	Box Girders G6N & G6S	6	Oct 4/96	Jan 14/97	2	Dec 20/96
Cherubini	Box Girders G7N & G7S	7A	Nov 27/96	Jan 14/97	1	Dec 20/96
Cherubini	Box Girders G7N & G7S	7B	Nov 27/96	Jan 14/97	1	Dec 20/96
Cherubini	Box Girders G8N & G8S	8A	Nov 27/96	Jan 14/97	1	Dec 20/96
Cherubini	Box Girders G8N & G8S	8B	Nov 27/96	Jan 14/97	1	Dec 20/96
Cherubini	Box Girders G9N & G9S	9A	Nov 27/96	Jan 14/97	1	Dec 20/96
Cherubini	Box Girders G9N & G9S	9B	Nov 27/96	Jan 14/97	1	Dec 20/96
Cherubini	Box Girders G10N & G10S	10	Nov 27/96	Jan 14/97	1	Dec 20/96
Cherubini	Box Girders G11N & G11S	11	Nov 27/96	Jan 14/97	1	Dec 20/96
Cherubini	Web Welding Details	12A	Oct 24/96	Jan 14/97	3	Dec 20/96
Cherubini	Web Welding Details	12B	Oct 24/96	Jan 14/97	3	Dec 20/96
Cherubini	Web Cutting Details	12C	Oct 24/96	Jan 14/97	3	Dec 20/96
Cherubini	Web Cutting Details	12D	Oct 24/96	Jan 14/97	3	Dec 20/96
Cherubini	Web Cutting Details	12E	Oct 24/96	Jan 14/97	3	Dec 20/96
Cherubini	Web Welding Details	12F	Nov 04/96	Jan 14/97	2	Dec 20/96
Cherubini	Top Flanges Cut & Weld.	13A	Nov 14/96	Jan 14/97	1	Dec 20/96
Cherubini	Top Flanges Cut & Weld.	13B	Nov 08/96	Jan 14/97	1	Dec 20/96
Cherubini	T's Web Cutting Details	14A	Nov 07/96	Jan 14/97	2	Dec 20/96
Cherubini	T's Web & Flanges Cutting	14B	Nov 08/96	Jan 14/97	2	Dec 20/96
Cherubini	Tie-Beams Ass. & Details	15	Dec 02/96	Jan 14/97	1	Jan 02/97
Cherubini	Box Girders Details	16A	Oct 28/96	Jan 14/97	2	Dec 20/96
Cherubini	Box Girders Details	16B	Oct 28/96	Jan 14/97	3	Dec 20/96
Cherubini	Box Girders Details	16C	Oct 28/96	Jan 14/97	2	Dec 20/96
Cherubini	Box Girders Acc. & Details	17	Nov 04/96	Jan 14/97	4	Jan 07/97
Cherubini	Splice 1 Ass. & Details	18A	Nov 20/96	Jan 14/97	1	Jan 02/97
Cherubini	Splice 2 Ass. & Details	18B	Nov 22/96	Jan 14/97	1	Jan 02/97
Cherubini	Splice 3 Ass. & Details	18C	Nov 25/96	Jan 14/97	1	Jan 02/97
Cherubini	Splice 4 Ass. & Details	18D	Nov 26/96	Jan 14/97	1	Jan 02/97
Cherubini	Splice 5 Ass. & Details	18E	Nov 27/96	Jan 14/97	1	Jan 02/97
Cherubini	Field Splice Plates Details	18F	Nov 19/96	Jan 14/97	1	Jan 02/97
Cherubini	Key Plan	19A	Nov 08/96	Jan 14/97	1	Jan 02/97

TUBE 16

TUBE 11

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◆ **STRAIT CROSSING INC.** ◆

Bridge Improvements

Pier Temporary Works

Drawing List

TUBE 18

Consulting Engineer	Drawing Name	DWG #	DWG Date	Date Rec'd	Rev #	Rev Date
Cherubini	Camber Coordinates	20	Dec 03/96	Jan 14/97	1	Jan 02/97
Cherubini	Erection from barge	271502	Jan 09/97	Apr 10/97	2	
Cherubini	Pier Temporary Works	E1		Aug-96		
Cherubini	Shop Details	1		Aug-96		
Cherubini	Shop Details	2		Aug-96		
Cherubini	Shop Details	3		Aug-96		
Cherubini	Shop Details	4		Aug-96		
Cherubini	Shop Details	5		Aug-96		
Cherubini	Shop Details	6		Aug-96		
Cherubini	Shop Details	7		Aug-96		
Cherubini	Shop Details	8		Aug-96		
Cherubini	Shop Details	9		Aug-96		
Cherubini	Shop Details	10		Aug-96		
Cherubini	Shop Details	11		Aug-96		
Cherubini	Shop Details	12		Aug-96		
Cherubini	Shop Details	13		Aug-96		
Cherubini	Shop Details	14		Aug-96		
Cherubini	Shop Details	15		Aug-96		
Cherubini	Shop Details	16		Aug-96		
Cherubini	Shop Details	17		Aug-96		
Cherubini	Shop Details	18		Aug-96		
Cherubini	Shop Details	19		Aug-96		
Cherubini	Shop Details	20		Aug-96		
Cherubini	Shop Details	21		Aug-96		
Cherubini	Shop Details	22		Aug-96		
Cherubini	Shop Details	23		Aug-96		
Cherubini	Shop Details	24		Aug-96		
Cherubini	Shop Details	25		Aug-96		
Cherubini	Shop Details	26		Aug-96		
Cherubini	Shop Details	27		Aug-96		
Cherubini	Shop Details	28		Aug-96		
Cherubini	Shop Details	29		Aug-96		
Cherubini	Shop Details	30		Aug-96		
Cherubini	Shop Details	31		Aug-96		
Cherubini	Shop Details	32		Aug-96		
Cherubini	Shop Details	33		Aug-96		
Cherubini	Shop Details	34		Aug-96		
Cherubini	Shop Details	35		Aug-96		
Cherubini	Shop Details	36		Aug-96		
Cherubini	Shop Details	37		Aug-96		

BINDER FILES.

BOX 1. CONTAINS INFORMATION THAT MIGHT BE REQUIRED TO REFER TO IN FUTURE.

		PAGES
Quality Assurance Books of SCI	1to 7	16 - 18
CBCL-Survey information including sketches.		19
Reinforced earth files.		
Steel truss strengthening.		20

BOX 2. Contains information not likely that reference will be made in future.

Daily work reports.
Road works daily quantities.
Daily Logs -SC1
Inland coastal daily work record.
Concrete Cylinder test reports.
Ped. Underpass daily logs.

BOX 3. Contains information less likely that reference will be made in future.

CHERUBINI METAL WORKS.
Quality Assurance Books 1 to 5. 21-23

BOX 5. SITE PROGRESS PHOTOS.

Volumes 1 to 11.

Hillsborough River Bridge Project
Quality Assurance
Documentation Package
Index to Box 1.

Existing Bridge Deck

Pours 1-5	Pour Release Test NCR
Closure Pours	Pour Release Test NCR
Expansion Joints	Contract & Specifications Correspondence Drawings

Section 4 **Bridge Strengthening** **Book 4**

Weld Procedures
Stiffner Welding Records
Tie Beam & Brace Connection Procedure
Dywidag Bar Installation
Weld Certification & Mill Reports

General
On Site Inspections
Paint
Shop Drawings
Correspondence

Section 5 **Retaining Wall & Gabion Baskets** **Book 5**

Contract & Specifications
RECO Wall Correspondence
RECO Wall Drawings & Sketches
Maccaferri Gabion Baskets Correspondence
Maccaferri Gabion Baskets Drawings & Sketches
NCRS
Tests

Section 6 **Concrete Delivery Slips** **Book 6**

Pier 1 Concrete delivery slips
Pier 2 Concrete delivery slips
Pier 3 Concrete delivery slips
Pier 4 Concrete delivery slips

Section 6A **Concrete Delivery Slips** **Book 6A**

Precast Panels- Concrete delivery slips
Barrier, Sidewalks & Precast Closure C.D.S.

Existing Bridge Deck
Existing Bridge Closure Pours

Section 7 **Roadworks** **Book 7**

Soil Compaction
Class 'A' Granular Analysis
Hot Mix Analysis

Hillsborough River Bridge Project

Quality Assurance
SURVEY INFORMATION AND SKETCHES

Index to Box 1.

Section

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STEEL TRUSS STRENGTHENING

Cherubini Metal Works Limited

Quality Assurance
Documentation Package

Index to Box 3.

Section 1	Certificate of Compliance / Inspection Release	Book 1
Section 2	Welding Procedures	Book 1
Section 3	Welder Certifications	Book 1
Section 4	Coating Inspection Reports	Book 1
Section 5	Hillsborough Bridge Pot Bearings / Material Certificates / Certificates of Compliance	Book 1
Section 6	Torque Testing Report	Book 1
Section 7	Hardware Mill Test Certificates	Book 1
Section 8	G1 North - South Heat Number List / Mill Reference List / Inspection Report / Non- Destructive Testing Map / As Built Drawings	Book 1
Section 9	G2 North - South Heat Number List / Mill Reference List / Inspection Report / Non- Destructive Testing Map / As Built Drawings	Book 1
Section 10	G3 North - South Heat Number List / Mill Reference List / Inspection Report / Non- Destructive Testing Map / As Built Drawings	Book 1

Cherubini Metal Works Limited

Quality Assurance
Documentation Package

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Cherubini Metal Works Limited

Quality Assurance
Documentation Package

Index

◆ STRAIT CROSSING INC. ◆

Existing Hillsborough Bridge

Drawing List - TUBES 1 TO 4

Consulting Engineer	Drawing Name	DWG #	DWG Date	Rev Date
O.J. McCulloch & Co.	GENERAL PLAN	1A File # 7-53.1-1	April 15/58	June 9/59
O.J. McCulloch & Co.	GENERAL PLAN	1 of 4 File # 7-53.1-2	July 25/58	Jan 12/62
O.J. McCulloch & Co.	TEST PILE DATA AND BORING LOGS	2 of 4 File # 7-53.1-2	July 25/58	
O.J. McCulloch & Co.	ABUTMENT & PIERS No. 1 & No. 4	3 of 4 File #7-53.1-2	July 25/58	Jan 12/62
O.J. McCulloch & Co.	PIERS No. 2 & No. 3	4 of 4 File #7-53.1-2	July 25/58	Jan 15/62
O.J.McCulloch & Co.	PROTECT PLATES SHAFTS PIERS No. 1 & 3	5 File # 7-53.1-2	Sept 21/61	Jan 8/62
O.J. McCulloch & Co.	GENERAL ARRANGEMENT	1 File # 7-53.1-3	Jan 16/59	Jan 16/62
O.J. McCulloch & Co.	STRESS SHEET	2 File # 7-53.1-3	Jan 16/59	Jan 23/78
O.J. McCulloch & Co.	GEOMETRY, X-SECTIONS AND PIER MEM	3 File # 7-53.1-3	Jan 16/59	May 8/87
O.J. McCulloch & Co.	HANDRAIL, EXP JOINTS AND MISC STEEL	4 File # 7-53.1-3	Jan 16/59	Jan 12/62
O.J. McCulloch & Co.	DECK SLAB	1 File # 7-53.1-4	Feb 15/61	
O.J. McCulloch & Co.	ASPHALT PAVEMENTS	2 File # 7-53.1-4	Feb15/61	Dec 13/61
O.J. McCulloch & Co.	LIGHTING AND APRON SLAB	3 File # 7-53.1-4	Feb 15/61	Dec 13/61
O.J. McCulloch & Co.	FIELD WORK TO EXIST JACK GIRDER 1&4	S1 File # 7-77.25	Jan 23/78	
O.J. McCulloch & Co.	FIELD WORK TO EXIST GIRDER No. 2	S2 File # 7-77.25	Jan 23/78	
O.J. McCulloch & Co.	POT TYPE BEARING AT PIER No. 1 & 4	S3 File # 7-77.25	Jan 23/78	
O.J. McCulloch & Co.	POT TYPE BEARING AT PIER No. 2	S4 File # 7-77.25	Jan 23/78	
O.J. McCulloch & Co.	Substructure-Pier 3- Log of 30" Drill	Job Dwg. A	Dec, 1960	
O.J. McCulloch & Co.	Substructure-Pier 3- Elevations of Tremie	Job Dwg. C	Dec, 1960	
O.J. McCulloch & Co.	Substructure-Pier 3- Drill Holes for Grouting	Job Dwg. D	Dec, 1960	
O.J. McCulloch & Co.	S.S.P. Layout for Pier 2&3	Job Dwg. G	May 14,1959	
O.J. McCulloch & Co.	Substructure-Pier 3- Log of Diamond Drill Holes in Tremie Concrete	Job Dwg. 1		
O.J.McCulloch & Co.	Substructure-Pier 3- Log of Diamond Drill Holes in Tremie Concrete	Job Dwg. 2		
O.J. McCulloch & Co.	Substructure-Pier 3- Cofferdam location			
O.J. McCulloch & Co.	Substructure-Pier 3-Sheetpile Depths			
O.J. McCulloch & Co.	Substructure-Pier 2-Test Holes in Grout		June 6,1959	
O.J. McCulloch & Co.	Mudwave Soundings from Existing		1957	
O.J. McCulloch & Co.	Soundings Across Channel			
O.J. McCulloch & Co.	Substructure-Pier 3- Record of Pile Driving			
Foundation Martime Ltd	Sections Showing Position of Cell No.3		July 20,1959	
Foundation Martime Ltd	Strenghing of Pier No.3	H-10597-E	Jun 15 1960	
O.J. McCulloch & Co.	Substructure-N.Abut- Log of H-Piles	Sketch		
O.J. McCulloch & Co.	Pier 3- Showing Soundings	Sketch		
O.J. McCulloch & Co.	Substructure-S.Abut.-H-Piles As-Driven	Sketch		
O.J. McCulloch & Co.	Fill Profile	Sketch	Jan 28,1959	
O.J. McCulloch & Co.	Substructure-Pier 3- Record of Portion of H-Piles Driven	Sketch		
O.J.McCulloch & Co.	Tide Velocity Readings	Sketch	June 8,1959	
O.J. McCulloch & Co.	Substructure-H-Piles in Pier No.4	Sketch		
O.J. McCulloch & Co.	Profile of Channel	Sketch	Nov 26,1960	
O.J. McCulloch & Co.	Mudwave Soundings	Sketch		
O.J. McCulloch & Co.	Boreholes N.Abutment	Sketch		
O.J. McCulloch & Co.	Profile of North Fill	Sketch	Nov 15,1959	
O.J. McCulloch & Co.	Test Hole in Fill - North Approach	Sketch		
O.J. McCulloch & Co.	Sections North Approach	Sketch	Feb 5,1959	
Foundation Martime Ltd	Details of Pier 1 & 4	H-10597-J-3	June 16,1959	

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◆ STRAIT CROSSING INC. ◆

Existing Hillsborough Bridge

Drawing List

Consulting Engineer	Drawing Name	DWG #	DWG Date	Rev Date
O.J. McCulloch & Co.	GENERAL PLAN	1A File # 7-53.1-1	April 15/58	June 9/59
O.J. McCulloch & Co.	GENERAL PLAN	1 of 4 File # 7-53.1-2	July 25/58	Jan 12/62
O.J. McCulloch & Co.	TEST PILE DATA AND BORING LOGS	2 of 4 File # 7-53.1-2	July 25/58	
O.J. McCulloch & Co.	ABUTMENT & PIERS No. 1 & No. 4	3 of 4 File #7-53.1-2	July 25/58	Jan 12/62
O.J. McCulloch & Co.	PIERS No. 2 & No. 3	4 of 4 File #7-53.1-2	July 25/58	Jan 15/62
O.J.McCulloch & Co.	PROTECT PLATES SHAFTS PIERS No. 1 & 3	5 File # 7-53.1-2	Sept 21/61	Jan 8/62
O.J. McCulloch & Co.	GENERAL ARRANGEMENT	1 File # 7-53.1-3	Jan 16/59	Jan 16/62
O.J. McCulloch & Co.	STRESS SHEET	2 File # 7-53.1-3	Jan 16/59	Jan 23/78
O.J. McCulloch & Co.	GEOMETRY, X-SECTIONS AND PIER MEM	3 File # 7-53.1-3	Jan 16/59	May 8/87
O.J. McCulloch & Co.	HANDRAIL, EXP JOINTS AND MISC STEEL	4 File # 7-53.1-3	Jan 16/59	Jan 12/62
O.J. McCulloch & Co.	DECK SLAB	1 File # 7-53.1-4	Feb 15/61	
O.J. McCulloch & Co.	ASPHALT PAVEMENTS	2 File # 7-53.1-4	Feb15/61	Dec 13/61
O.J. McCulloch & Co.	LIGHTING AND APRON SLAB	3 File # 7-53.1-4	Feb 15/61	Dec 13/61
O.J. McCulloch & Co.	FIELD WORK TO EXIST JACK GIRDER 1&4	S1 File # 7-77.25	Jan 23/78	
O.J. McCulloch & Co.	FIELD WORK TO EXIST GIRDER No. 2	S2 File # 7-77.25	Jan 23/78	
O.J. McCulloch & Co.	POT TYPE BEARING AT PIER No. 1 & 4	S3 File # 7-77.25	Jan 23/78	
O.J. McCulloch & Co.	POT TYPE BEARING AT PIER No. 2	S4 File # 7-77.25	Jan 23/78	
O.J. McCulloch & Co.	Substructure-Pier 3- Log of 30" Drill	Job Dwg. A	Dec, 1960	
O.J. McCulloch & Co.	Substructure-Pier 3- Elevations of Tremie	Job Dwg. C	Dec, 1960	
O.J. McCulloch & Co.	Substructure-Pier 3- Drill Holes for Grouting	Job Dwg. D	Dec, 1960	
O.J. McCulloch & Co.	S.S.P. Layout for Pier 2&3	Job Dwg. G	May 14,1959	
O.J. McCulloch & Co.	Substructure-Pier 3- Log of Diamond Drill Holes in Tremie Concrete	Job Dwg. 1		
O.J.McCulloch & Co.	Substructure-Pier 3- Log of Diamond Drill Holes in Tremie Concrete	Job Dwg. 2		
O.J. McCulloch & Co.	Substructure-Pier 3- Cofferdam location			
O.J. McCulloch & Co.	Substructure-Pier 3-Sheetpile Depths			
O.J. McCulloch & Co.	Substructure-Pier 2-Test Holes in Grout		June 6,1959	
O.J. McCulloch & Co.	Mudwave Soundings from Existing		1957	
O.J. McCulloch & Co.	Soundings Across Channel			
O.J. McCulloch & Co.	Substructure-Pier 3- Record of Pile Driving			
Foundation Martime Ltd	Sections Showing Position of Cell No.3		July 20,1959	
Foundation Martime Ltd	Strenghing of Pier No.3	H-10597-E	Jun 15 1960	
O.J. McCulloch & Co.	Substructure-N.Abut- Log of H-Piles	Sketch		
O.J. McCulloch & Co.	Pier 3- Showing Soundings	Sketch		
O.J. McCulloch & Co.	Substructure-S.Abut.-H-Piles As-Driven	Sketch		
O.J. McCulloch & Co.	Fill Profile	Sketch	Jan 28,1959	
O.J. McCulloch & Co.	Substructure-Pier 3- Record of Portion of H-Piles Driven	Sketch		
O.J.McCulloch & Co.	Tide Velocity Readings	Sketch	June 8,1959	
O.J. McCulloch & Co.	Substructure-H-Piles in Pier No.4	Sketch		
O.J. McCulloch & Co.	Profile of Channel	Sketch	Nov 26,1960	
O.J. McCulloch & Co.	Mudwave Soundings	Sketch		
O.J. McCulloch & Co.	Boreholes N.Abutment	Sketch		
O.J. McCulloch & Co.	Profile of North Fill	Sketch	Nov 15,1959	
O.J. McCulloch & Co.	Test Hole in Fill - North Approach	Sketch		
O.J. McCulloch & Co.	Sections North Approach	Sketch	Feb 5,1959	
Foundation Martime Ltd	Details of Pier 1 & 4	H-10597-J-3	June 16,1959	

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◆ **STRAIT CROSSING INC.** ◆

Existing Hillsborough Bridge

Drawing List #2

Consulting Engineer	Drawing Name	DWG #	DWG Date
O.J. McCulloch & Co.	Substructure-Pier 3- Log of 30" Drill	Job Dwg. A	Dec, 1960
O.J. McCulloch & Co.	Substructure-Pier 3- Elevations of Tremie	Job Dwg. C	Dec, 1960
O.J. McCulloch & Co.	Substructure-Pier 3- Drill Holes for Grouting	Job Dwg. D	Dec, 1960
O.J. McCulloch & Co.	S.S.P. Layout for Pier 2&3	Job Dwg. G	May 14,1959
O.J. McCulloch & Co.	Substructure-Pier 3- Log of Diamond Drill Holes in Tremie Concrete	Job Dwg. 1	
O.J.McCulloch & Co.	Substructure-Pier 3- Log of Diamond Drill Holes in Tremie Concrete	Job Dwg. 2	
O.J. McCulloch & Co.	Substructure-Pier 3- Cofferdam location		
O.J. McCulloch & Co.	Substructure-Pier 3-Sheetpile Depths		
O.J. McCulloch & Co.	Substructure-Pier 2-Test Holes in Grout		June 6,1959
O.J. McCulloch & Co.	Mudwave Soundings from Existing		1957
O.J. McCulloch & Co.	Soundings Across Channel		
O.J. McCulloch & Co.	Substructure-Pier 3- Record of Pile Driving		
Foundation Martime Ltd	Sections Showing Position of Cell No.3		July 20,1959
Foundation Martime Ltd	Strenghing of Pier No.3	H-10597-E	Jun 15 1960

◆ **STRAIT CROSSING INC.** ◆

Approach Road Drawings

Contract # 1

Drawing List – TUBE 5.

Consulting Engineer	Drawing Name	DWG #	DWG Date	Rev #	Rev Date
Harland Associates	Plan & Profile STA. 0+200 To STA. 0+500	C-1	July 9, 1996	1	July 10, 1996
Harland Associates	Plan & Profile STA. 0+500 To STA. 0+820	C-2	July 9, 1996	1	July 10, 1996
Harland Associates	Plan & Profile STA. 1+070 To 1+400	C-3	July 9, 1996	1	July 10, 1996
Harland Associates	Road x-sections	C-4	July 9, 1996	1	July 10, 1996
Harland Associates	Road x-sections	C-5	July 9, 1996	1	July 10, 1996
Harland Associates	Curve Data & Typical Details	C-6	July 9, 1996	1	July 10, 1996
Locus Surveys	Addition & Clarification of Services @ N&S Approaches		March 11/96		
Locus Surveys	Addition & Clarification of Services @ N&S Approaches	1 of 7	March 11/96		
Locus Surveys	Addition & Clarification of Services @ N&S Approaches	2 of 7	March 11/96		
Locus Surveys	Addition & Clarification of Services @ N&S Approaches	3 of 7	March 11/96		
Locus Surveys	Addition & Clarification of Services @ N&S Approaches	4 of 7	March 11/96		
Locus Surveys	Addition & Clarification of Services @ N&S Approaches	5 of 7	March 11/96		
Locus Surveys	Addition & Clarification of Services @ N&S Approaches	6 of 7	March 11/96		
Locus Surveys	Addition & Clarification of Services @ N&S Approaches	7 of 7	March 11/96		

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◆ **STRAIT CROSSING INC.** ◆

Roadway Improvements Drawings

As-built Drawings

Drawing List TUBES 6-7.

Consulting Engineer	Drawing Name	DWG #	DWG Date	Rev #	Rev Date
Harland Associates	Plan & Profile STA. 0+100 To STA. 0+310	C-1	Feb 15/97	As-built	
Harland Associates	Plan & Profile STA. 0+310 To STA. 0+620	C-2	Feb 15/97	As-built	
Harland Associates	Plan & Profile STA. 0+620 To Bridge	C-3	Feb 15/97	As-built	
Harland Associates	Plan & Profile STA. Bridge To 1+260	C-4	Feb 15/97	As-built	
Harland Associates	Plan & Profile STA. 1+260 To 1+480	C-5	Feb 15/97	As-built	
Harland Associates	Plan & Profile STA. 1+480 To 1+780	C-6	Feb 15/97	As-built	
Harland Associates	Plan & Profile STA. 1+780 To 2+040	C-7	Feb 15/97	As-built	
Harland Associates	Plan & Profile STA. 2+040 To 2+261	C-8	Feb 15/97	As-built	
Harland Associates	Plan & Profile STA. 3+080 To 3+300 Stratford Road	C-9	Feb 15/97	As-built	
Harland Associates	Plan & Profile STA. 3+300 To 3+560 Hopeton Road	C-10	Feb 15/97	As-built	
Harland Associates	Plan & Profile STA. Bunbury Merge Lane	C-11	Feb 15/97	As-built	
Harland Associates	Signalized Intersection Layout	C-12	Feb 15/97	As-built	
Harland Associates	Layout Information	C-13	Feb 15/97	As-built	
Harland Associates	Plan of Intersection with Storm Sewer Layout	C-14	Feb 15/97	As-built	
Harland Associates	Road Cross Sections STA: 0+160 To 0+320	XS-1	Jun 06/97	As-built	
Harland Associates	Road Cross Sections STA: 0+360 To 0+760	XS-2	Jun 06/97	As-built	
Harland Associates	Road Cross Sections STA: 0+800 To 1+120	XS-3	Jun 06/97	As-built	
Harland Associates	Road Cross Sections STA: 1+160 To 1+540	XS-4	Jun 06/97	As-built	
Harland Associates	Road Cross Sections STA: 1+740 To 2+240	XS-5	Jun 06/97	As-built	
Harland Associates	Road Cross Sections STA: 3+120 To 3+310	XS-6	Jun 06/97	As-built	
Harland Associates	Road Cross Sections STA: 3+380 To 3+520	XS-7	Jun 06/97	As-built	
Harland Associates	Road Cross Sections Bunbury Merge	XS-8	Jun 06/97	As-built	
Harland Associates	Miscellaneous Details	DT-1	Jun 06/97	As-built	
Harland Associates	Miscellaneous Details	DT-2	Jun 06/97	As-built	
Harland Associates	Overhead Road Sign Elevations & Details	DT-3	Jun 06/97	As-built	
Harland Associates	Stratford Traffic Signal-Signal & Wiring Layout	E1	Dec 02/96	As-built	
Harland Associates	Stratford Traffic Signal-Signal Construction	E2	Dec 02/96	As-built	
Harland Associates	Bridge and Intersection Lighting	E3	Dec 02/96	As-built	
Harland Associates	Mechanical Stabilized Earth System	SP1	Apr. 22/97	As-built	
CBCL Limited	Forcemain Plan and Profile Sta. 3+223 to 3+353	1 of 2	Mar.1997	As-built	
CBCL Limited	Miscellaneous Details	2 of 2	Mar.1997	As-built	

TUBE 6

TUBE 7

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◆ STRAIT CROSSING INC. ◆

Roadway Improvements Drawings

Contract # 3

Drawing List – TUBES 8 - 9.

Consulting Engineer	Drawing Name	DWG #	DWG Date	Rev #	Rev Date
Harland Associates	Plan & Profile STA. 0+100 To STA. 0+310	C-1	Feb 15/97	2	Feb 19/97
Harland Associates	Plan & Profile STA. 0+310 To STA. 0+620	C-2	Feb 15/97	2	Feb 19/97
Harland Associates	Plan & Profile STA. 0+620 To Bridge	C-3	Feb 15/97	2	Feb 19/97
Harland Associates	Plan & Profile STA. Bridge To 1+260	C-4	Feb 15/97	2	Feb 19/97
Harland Associates	Plan & Profile STA. 1+260 To 1+480	C-5	Feb 15/97	2	Feb 19/97
Harland Associates	Plan & Profile STA. 1+480 To 1+780	C-6	Feb 15/97	2	Feb 19/97
Harland Associates	Plan & Profile STA. 1+780 To 2+040	C-7	Feb 15/97	2	Feb 19/97
Harland Associates	Plan & Profile STA. 2+040 To 2+261	C-8	Feb 15/97	2	Feb 19/97
Harland Associates	Plan & Profile STA. 3+080 To 3+300 Stratford Road	C-9	Feb 15/97	2	Feb 19/97
Harland Associates	Plan & Profile STA. 3+300 To 3+560 Hopeton Road	C-10	Feb 15/97	2	Feb 19/97
Harland Associates	Plan & Profile STA. Bunbury Merge Lane	C-11	Feb 15/97	2	Feb 19/97
Harland Associates	Signalized Intersection Layout	C-12	Feb 15/97	2	Feb 19/97
Harland Associates	Layout Information	C-13	Feb 15/97	2	Feb 19/97
Harland Associates	Plan of Intersection with Storm Sewer Layout	C-14	Feb 15/97	2	Feb 19/97
Harland Associates	Road Cross Sections STA: 0+160 To 0+320	XS-1	Jun 06/97	2	Feb 19/97
Harland Associates	Road Cross Sections STA: 0+360 To 0+760	XS-2	Jun 06/97	2	Feb 19/97
Harland Associates	Road Cross Sections STA: 0+800 To 1+120	XS-3	Jun 06/97	2	Feb 19/97
Harland Associates	Road Cross Sections STA: 1+160 To 1+540	XS-4	Jun 06/97	2	Feb 19/97
Harland Associates	Road Cross Sections STA: 1+740 To 2+240	XS-5	Jun 06/97	2	Feb 19/97
Harland Associates	Road Cross Sections STA: 3+120 To 3+310	XS-6	Jun 06/97	2	Feb 19/97
Harland Associates	Road Cross Sections STA: 3+380 To 3+520	XS-7	Jun 06/97	2	Feb 19/97
Harland Associates	Road Cross Sections Bunbury Merge	XS-8	Jun 06/97	2	Feb 19/97
Harland Associates	Miscellaneous Details	DT-1	Jun 06/97	2	Feb 19/97
Harland Associates	Miscellaneous Details	DT-2	Jun 06/97	2	Feb 19/97
Harland Associates	Overhead Road Sign Elevations & Details	DT-3	Jun 06/97	2	Feb 17/97
Harland Associates	Stratford Traffic Signal-Signal & Wiring Layout	E1	Dec 02/96	2	Jan 27/97
Harland Associates	Stratford Traffic Signal-Signal Construction	E2	Dec 02/96	2	Jan 27/97
Harland Associates	Bridge and Intersection Lighting	E3	Dec 02/96	2	Jan 27/97
Harland Associates	Mechanical Stabilized Earth System	SP1	Apr. 22/97	2	Jan 27/97
L&A Metalworks	O/H Sign Structure	4711-1	Apr 29/97	2	Jun / 97
Shaw Pipe	Shop DWGS – Catch Basin # 7	1 of 20			
Shaw Pipe	Shop DWGS – Catch Basin # 7	2 of 20			
Shaw Pipe	Shop DWGS – Catch Basin # 13	3 of 20			
Shaw Pipe	Shop DWGS – Catch Basin # 15	4 of 20			
Shaw Pipe	Shop DWGS – Catch Basin # 17	5 of 20			
Shaw Pipe	Shop DWGS – Catch Basin # 19	6 of 20			
Shaw Pipe	Shop DWGS – Catch Basin # 21	7 of 20			
Shaw Pipe	Shop DWGS – Catch Basin # 32	12 of 20			
Shaw Pipe	Shop DWGS – Catch Basin # 34	13 of 20			
Shaw Pipe	Shop DWGS – Catch Basin # 36	14 of 20			
Shaw Pipe	Shop DWGS – Catch Basin # 38	15 of 20			
Shaw Pipe	Shop DWGS – Catch Basin # 13	16 of 20			
Shaw Pipe	Shop DWGS – Catch Basin # 14	17 of 20			
Shaw Pipe	Shop DWGS – Catch Basin # 15	18 of 20			
Shaw Pipe	Shop DWGS – Catch Basin # 20	19 of 20			
Shaw Pipe	Shop DWGS – Catch Basin # 21	20 of 20			

TUBE 8

TUBE 9

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◆ STRAIT CROSSING INC. ◆

Pedestrian Underpass Drawings

Contract # 2

Drawing List – TUBE 10.

Consulting Engineer	Drawing Name	DWG #	DWG Date	Date Rec'd	Rev #	Rev Date
Harland Associates	Site Plan & Details	S1 of 6	June 6, 1996	Feb. 13/97	2	Jan. 27, 1997
Harland Associates	Plan, Sections, & Details	S2 of 6	June 6, 1996	Feb. 13/97	2	Jan. 27, 1997
Harland Associates	Sections & Details	S3 of 6	June 6, 1996	Feb. 13/97	2	Jan. 27, 1997
Harland Associates	Curb Rebar & Details	S4 of 6	June 6, 1996	Feb. 13/97	2	Jan. 27, 1997
Harland Associates	Abutment Section & Details	S5 of 6	June 6, 1996	Feb. 13/97	2	Jan. 27, 1997
Harland Associates	Abutment Rebar & Details	S6 of 6	June 6, 1996	Feb. 13/97	2	Jan. 27, 1997
Harland Associates	Deck slabs	SD-2	Mar 31/97	Apr. 9/97		
Harland Associates	Abutments (4 units)	SD-4	Mar 31/97	Apr. 9/97		
Harland Associates	Abutments (4 units)	SD-3	Mar 31/97	Apr. 9/97		
Harland Associates	Approach slabs	SD-1	Mar 31/97	Apr. 9/97		
Harland Associates	Reinforcing bar schedule- abutment slabs		Mar 31/97	Apr. 9/97		
Harland Associates	Reinforcing bar schedule- deck slabs		Mar 31/97	Apr. 9/97		
Harland Associates	Reinforcing bar schedule- abutment (4 units)		Mar 31/97	Apr. 9/97		
Harland Associates	Expansion Joints	1 of 1	Apr 23/97	Apr. 23/97		
Harland Associates	Installation Procedures (Compression Seal)	B-9574	Feb. 26/97	Apr. 23/97		
Harland Associates	Upturn & Downturn Procedure (Compression Seal)	B-9577	Feb. 26/97	Apr. 23/97		
Arrow Construction	Compression Seal Ex. Joint Expansions	1 of 1	Apr 23/97	Apr 23/97		
Watson Bowman	Compressive Seal	B-9574	Feb 26/97			
Watson Bowman	Compressive Seal	B-9577	Feb 26/97	Apr 23/97		
Friman Steel	Deck Slabs (2 units)	SD-2	Mar 31/97	Apr 9/97		
Friman Steel	Abutments (4 units)	SD-4	Mar 31/97	Apr 9/97		
Friman Steel	Abutment (4 units)	SD-3	Mar 31/97	Apr 9/97		
Friman Steel	Approach Slabs	SD-1	Mar 31/97	Apr 9/97		
Watson Bowman	Laminated Bearing Pads	E-6547	Apr 97			
Arrow Construction	Bearing Details	1 of	Apr 17/97			

◆ **STRAIT CROSSING INC.** ◆

Bridge Improvements

Misc. Shop Drawings

Drawing List – TUBE 11.

Consulting Engineer	Drawing Name	DWG #	DWG Date	Date Rec'd	Rev #	Rev Date
EFCO	Pier Column	E89-1	Oct 28/96	Nov 5/96	0	
Aluma Systems	Abutment Overhang Plan	TOP 4964-01	Nov 25/96	Dec16/96	2	Dec11/96
Harland Associates	Pier P3 Drill Platform Beam & Column Modifications General Configuration		Jun 03/97	Jun 04/97	0	
Harland Associates	Pier 3 Drill Platform Beam & Column Modifications Details 2		Jun 03/97	Jun 04/97	0	
Harland Associates	Pier 3 Drill Platform Beam & Column Modifications Details		May 30/97	Jun 04/97		
Harland Associates	Concrete Formwork Support W200 Wale Stiffening Details	S1 of 2	Jun 24/97	Jun 24/97	0	
Harland Associates	Concrete Formwork Support W200 Wale Stiffening Details	S2 of 2	Jun 24/97	Jun 24/97	0	
EFCO	Pier Forms	E115-1	Jun 19/97	Jun 25/97		
EFCO	Elevation Showing Reinforcing Rods	E115-4	Jun 20/97	Jun 25/97		
EFCO	Reinforced Panel Locations	E115-3	Jun 19/97	Jun 25/97		
L & A Metalworks	Details of Railing	1	Aug /97	Sep 2/97		
MB Consultant Ltd.	Pier Flare Form	FW-1	Jun 3/97	Jul 3/97	1	Jul 2/97
MB Consultants Ltd.	Pier Flare Form	FW-2	Jun 3/97	Jul 3/97	1	Jul 2/97
CBCL	Girder Lift Points Details	S1	May 21/97	Jul 8/97	0	
Watson Bowman Acme	WPF-4900 Fixed Pot Bearings	E-6527-1	Dec/96	Apr 8/97	1	Mar 26/97
Watson Bowman Acme	WPG-1700 Uni-Directional Post Bearing	E-6523-1 (1 of 2)	Dec/96	Apr 8/97	1	Mar 27/97
Watson Bowman Acme	WPG-1700 Uni-Directional Post Bearing	E-6523-1 (2 of 2)	Dec/96	Apr 8/97	1	Mar 27/97
Watson Bowman Acme	WPM-4900 Multi-Directional Pot Bearing	E-6528-1 (1 of 2)	Dec/96	Apr 8/97	1	Mar 26/97
Watson Bowman Acme	WPM-4900 Multi-Directional Pot Bearing	E-6528-1 (2 of 2)	Dec/96	Apr 8/97	1	Mar 26/97
Watson Bowman Acme	WPM-1700 Multi-Directional Pot Bearing	E-6526-1 (1 of 2)	Dec/96	Apr 8/97	1	Mar 28/97
Watson Bowman Acme	WPM-1700 Multi-Directional Pot Bearing	E-65261- (2 of 2)	Dec/96	Apr 8/97	1	Mar 28/97
Watson Bowman Acme	WPG-4900 Uni-Directional Pot Bearing	E-6525-1 (1 of 2)	Dec/96	Apr 8/97	1	Mar 27/97
Watson Bowman Acme	WPG-4900 Uni-Directional Pot Bearing	E-6525-1 (2 of 2)	Dec/96	Apr 8/97	1	Mar 27/97

APPENDIX B

Truss Member Summaries and Yield Strengths

Original Steel Yield Strengths			
	Old Plate	242.9	MPa
	New Plate	300	MPa
	Channel	210.5	MPa
	Threadbar	413.7	MPa

Member	**Cross Sectional Areas (mm^2)**					
	Old Plate	**New Plate**	**Channel/Angle**	**Threadbar**	**Total**	**Yield**
BC1	0	0	16291	2581	18872	238.3
BC2	8445	0	22100	5162	35707	247.5
BC3	14224	0	22100	5162	41486	246.9
BC4	8445	0	19776	5162	33383	250.1
BC5.1	7550	4100	24738	0	36388	227.3
BC5.2	7550	20500	24738	0	52788	249.9
BC6.1	7550	20500	26072	0	54122	248.9
BC6.2	7550	0	26072	0	33622	217.8
BC7	0	0	22100	2581	24681	231.7
TC1	25560	0	7416		32976	235.6
TC2	22195	0	8560		30755	233.9
TC3	27293	0	7416		34709	236.0
TC4	20420	0	7416		27836	234.3
TC5	42404	0	7416		49820	238.1
TC6	17550	13500	7416		38466	256.7
TC7	25467	7940	7416		40823	248.1
Diag1	3773	3773	12900		20446	233.0
Diag2	-	-	-	-	-	297.0
Diag3	-	-	-	-	-	297.0
Diag3.1	-	-	-	-	-	297.0
Diag4	-	-	-	-	-	297.0
Diag5	8624	10250	15218		34092	245.6
Diag6	10668	4100	17454		32222	232.6
Diag7	17024	20500	16292		53816	254.8
Diag8	15882	20500	16292		52674	255.1
Diag9	8382	4920	17454		30756	233.6
Diag10	7588	10250	15218		33056	245.7
Diag11	-	-	-	-	-	297.0
Diag12	-	-	-	-	-	297.0
Vert1	-	-	-	-	-	297.0
Vert2	-	-	-	-	-	297.0
Vert3	-	-	-	-	-	297.0

APPENDIX C

Truss Members Resistance Summary

Hillsborough Bridge Truss Resistance Results Summary

Date: 23-Dec-14

Prepared by: CBCL Limited

Evaluation Scenario

1 -Maximum Compression and Associated Moment

2 - Maximum Tension and Associated Moment

3 - Max Moment and Associated Axial Force

Member Label	Member Location	Member Description	Interaction (% of Capacity)					
			Case A - Existing Bridge			Case B - Propose AT trail and force main		
			1	2	3	1	2	3
Top Chord 1	U0-U1	Built up member with cover plate on top and perforated plate on bottom	0.08	0.36	0.37	0.08	0.39	0.39
Top Chord 2	U1-U3	Built up member with cover plate on top and perforated plate on bottom	0.74	0.42	0.74	0.83	0.38	0.82
Top Chord 3	U3-U5, U5-U7	Built up member with cover plate on top and perforated plate on bottom	0.65	0.52	0.68	0.71	0.53	0.73
Top Chord 4	U7-U9, U15-U17	Built up member with cover plate on top and perforated plate on bottom	0.50	0.24	0.60	0.54	0.25	0.63
Top Chord 5	U9-U11, U13-U15	Built-up member with coverplate on top and perforated plate on bottom	0.37	0.50	0.70	0.37	0.53	0.75
Top Chord 6	U11-U13	Built-up member with coverplate on top and perforated plate on bottom	0.43	0.56	0.53	0.43	0.59	0.55
Top Chord 7	U17-U19	Built-up member with coverplate on top and perforated plate on bottom	0.45	0.35	0.46	0.50	0.35	0.50
BC1A	L0-L2	Back to back channels with tie plates (battens) on top and bottom with threadbar	0.86	0.89	1.00	0.87	0.97	1.06
BC1B	L8-L9	Back to back channels with lacing on top and bottom with threadbar	0.53	0.57	0.60	0.56	0.62	0.64
BC1C	L9-L10 & L14-L16	Back to back channels with lacing on top and bottom	0.53	0.56	0.57	0.58	0.59	0.59
Bottom Chord 2	L2-L4	Back to back channels with tie plates (battens) on top and bottom and threadbar	0.87	0.87	0.88	0.93	0.89	0.95
Bottom Chord 3	L4-L6	Back to back channels with tie plates (battens) on top and bottom and threadbar	0.85	0.84	0.89	0.92	0.91	0.95
Bottom Chord 4	L6-L8, L18-L19	Back to back channels with tie plates (battens) on top and bottom and threadbar	0.56	0.82	0.82	0.60	0.89	0.88
Bottom Chord 5.1	L10-L11	Back to back channels with coverplate on top and perforated plate on bottom	0.75	0.77	0.78	0.81	0.81	0.84
Bottom Chord 5.2	L11-L12	Back to back channels with coverplate on top and bottom	0.49	0.59	0.61	0.60	0.58	0.66
Bottom Chord 6.1	L12-L13	Back to back channels with coverplate on top and bottom	0.56	0.63	0.66	0.62	0.67	0.72
Bottom Chord 6.2	L13-L14	Back to back channels with perforated plate on top and bottom	0.96	0.95	0.97	1.05	1.01	1.03
Bottom Chord 7	L16-L18	Back to back channels with lacing on top and lacing on bottom and threadbar	0.62	0.64	0.67	0.65	0.67	0.71
Diagonal 1	L0-U1	Back to back channels with cover plate and perforated plate	1.03	1.10	1.14	1.21	0.96	1.33
Diagonal 2A	U1-L2	W14x78 (no metric equivalent)	1.00	1.00	1.00	0.99	1.07	1.07
Diagonal 2B	U3-L4	W14x78 (no metric equivalent)	0.49	0.48	0.48	0.49	0.50	0.51
Diagonal 2C	L4-U5	W14x78 (no metric equivalent)	0.44	0.42	0.42	0.51	0.41	0.47
Diagonal 2D	U5-L6	W14x78 (no metric equivalent)	0.76	0.76	0.80	0.82	0.76	0.80
Diagonal 2E	L6-U7	W14x78 (no metric equivalent)	0.62	0.62	0.62	0.37	0.49	0.50
Diagonal 2F	L18-U19	W14x78 (no metric equivalent)	0.59	0.57	0.57	0.61	0.57	0.59
Diagonal 3	L2-U3	W14x87 (no metric equivalent)	0.86	0.86	0.89	1.00	0.81	1.03
Diagonal 3.1	L8-U9, L16-U17	W14x87 (no metric equivalent) reinforced with 380 x 10 thick plate on side	1.03	0.78	0.87	1.16	0.84	0.95
Diagonal 4	U7-L8	W14x95 (no metric equivalent) reinforced with 380 x 10 thick side plate	0.74	0.70	0.79	0.84	0.72	0.89
Diagonal 5	U9-L10	Back to back channels with cover plate and perforated plate	0.75	0.75	0.78	0.85	0.77	0.88
Diagonal 6	L10-U11	Back to back channels with tie plates and 410 x 10 thick side plate	0.81	0.93	0.94	0.92	0.93	1.01
Diagonal 7	U11-L12	Back to back channels with 410 x 25 thick plate each side	0.64	0.65	0.73	0.71	0.66	0.82
Diagonal8	L12-U13	Back to back channels reinfroced with 410 x 25 thick plate each side	0.60	0.52	0.66	0.66	0.54	0.73
Diagonal 9	U13-L14	Back to back channels with tie plates and reinforced with 410 x 12 thick side plate	0.71	0.73	0.77	0.72	0.78	0.82
Diagonal 10	L14-U15	Back to back channels with perforated plate and reinforced with 410 x 25 thick plate	0.42	0.45	0.46	0.43	0.45	0.50
Diagonal 11	U15-L16	W14x84 (no metric equivalent) reinforced with 380 x 10 thick side plate	0.97	0.94	0.96	1.09	0.94	1.08
Diagonal12	U17-L18	W14x78 (no metric equivalent)	0.66	0.64	0.64	0.68	0.68	0.68
Verticals 1	L0-U0, L2-U2, L4-U4, L6-U6, L8-U8, L10-U10, L14-U14, L18-U18	W360x91 (W14x61)	0.69	0.31	0.65	0.79	0.32	0.72
Verticals 2	L1-U1, L3-U3, L5-U5, L7-U7, L9-U9, L11-U11, L13-U13, L15-U15, L17-U17, L19-U19	W360x64 (W14x43)	0.14	0.05	0.10	0.17	0.09	0.10
Verticals 3	L12-U12	W360x110 (W14x74)	0.60	0.60	0.60	0.64	0.61	0.61

APPENDIX D

Dive Inspection Report

Department of Transportation and Infrastructure Renewal, PEI

Hillsborough Bridge Piers, Engineering Dive Assessment

Diversified Divers Incorporated and Harbourside Engineering Consultants
10/28/2014

Table of Contents

1.0 INTRODUCTION

Diversified Divers Incorporated (DDI) was engaged by the Department of Transportation and Infrastructure Renewal of Prince Edward Island (TIR) to perform an "Engineering Dive Assessment" on the 2 in-water piers at the Hillsborough Bridge in Charlottetown, Prince Edward Island. It is understood that the purpose of the assessment is to gather information regarding the condition state of the piers to aid in an overall assessment of the bridge to accept additional loads. It is understood that the contemplated additional loads arise from the addition of an active transportation lane and/or a possible sewage forcemain(s). Based on the original drawings and site inspections, Pier 3 (Stratford pier) is the longitudinally fixed pier and Pier 2 (Charlottetown Pier) is a longitudinally free pier.

The following are the reference materials used to perform the inspection and prepare the report.

- 1996 Hillsborough Bridge Improvement drawings
- 2013 Bi-annual Bridge Inspection Report

DDI has engaged Harbourside Engineering Consultants (HEC) to be involved with the inspection process and prepare this report.

2.0 SCOPE OF INSPECTION

The scope of the engineering dive assessment was specified in the terms of reference provided by TIR.

Originally the scope of work included a dive inspection down to 18.3m (60') depth below the low water elevation. During a meeting on September 23rd, 2014, it was agreed that the scope be increased to the 30.5m (100' depth) or sea bed, whichever is shallower. Further, it was agreed that some ultrasonic thickness readings should be taken on the steel sheet pile. The number of readings was left at the discretion of the inspectors.

Refer to the general arrangement and elevation drawings (SK-01, 02, and 03) in Annex A. Pier number 2 is the north pier (Charlottetown pier) and pier number 3 is the south pier (Stratford pier). A tide board was setup using the top of the east corbel on Pier 3 as a datum. The elevation of the top of corbel was pulled off of the 1996 expansion drawings (El. 7.304 m) and its as-built elevation was assumed to be correct.

The terminology used during the inspection and in this report is based on the Ontario Bridge Inspection Manual and PEITIR inspection standards. The inspection was completed to OSIM standards where possible.

The engineering dive assessment scope and report scope are described below.

2.1. ENGINEERING DIVE ASSESSMENT SCOPE

1. Inspection of steel sheet piling (SSP) down to 30.5m (100') depth below low water level or sea bed (whichever is shallower).
    a. Diver swam vertically down to bottom at 8 locations (stations) around each pier.
    b. The diver cleaned the steel sheet pile (SSP) of marine growth at a vertical spacing of 3.00m to expose the steel. An area of approximately 500mm x 500mm was cleaned and a portion of the out-pan, web, and in-pan were exposed. There were a total of 59 locations inspected on Pier 2 and 33 locations inspected on Pier 3.
    c. U/T readings were taken at 42 locations on Pier 2 and 36 locations on Pier 3. The readings were taken at 2 stations on Pier 2 from the top to the sea floor including the out-pan, in-pan and web at each location. The readings were taken at 4 stations on Pier 2 from the top to sea floor including shots on the out-pan, in-pan, and web.
    d. The diver maintained an overall visual of the wall as they descended to inspect for major defects as holes or deformations.
2. Inspection of concrete works under water.

   a. Diver inspected two horizontal concrete ledges; one at the interface between the transfer cap and the original SSP, the other is the top of the new transfer cap. The diver cleaned specific areas of the concrete (over the GEWI piles in particular) of marine growth to get a visual on the concrete.
   b. Diver inspected the vertical face of the pier wall up to and including the high tide mark.
3. Inspection of concrete elements above high tide mark.
   a. HEC inspected the pier wall as high as they can reach from the boat.
   b. HEC then accessed the catwalk under the bridge. The top of the pier was inspected using fall arrest equipment. The sides were inspected down as far as could be reached safely. The rest was inspected visually.
4. Inspection of the GEWI piles.
   a. This inspection was be inferred from the condition of the horizontal section of the concrete transfer cap.
5. Inspection of the steel plate ice shield.
   a. The ice shield was inspected by the diver. Two locations were cleaned of marine growth, one at the top of the ice shield (El. ±1.00m) and one at the bottom (El. ±-2.50m).
   b. Two U/T readings were taken at the nose of the ice shield on the west side of Pier 3 (at the cleaned locations noted above). The top reading represents a reading in the splash zone and the bottom reading represents a reading in the anodic zone.
6. Measure the overall geometry of the pier.
   a. Diver measured the overall in-plan dimensions of the SSP structure below water on each pier.
   b. HEC measured the gross in-plan dimensions of the pier wall from the boat.
   c. Approximate elevations were measured for the concrete transfer cap and the interface between the transfer cap and the SSP below.

2.2. *REPORT SCOPE*

The report was to include a summary of the overall condition state of each element of the piers (SSP, concrete, piles, ice shield) and provide recommendations for possible further investigation or repairs for TIR's review and consideration.

3.0 INSPECTION RESULTS

The inspections took place over a period from October 8th to October 15th, 2014. There was a cleaning program (pressure washing of the elements to be inspected) that took place beforehand beginning on September 29th. Diving had to be timed accurately so that during the inspection of the north and south sides of the piers, the tide was at a slack (either at high tide or low tide) and the current was at a minimum. On a rising or falling tide, the current under the bridge is very strong making diving operations very difficult.

Refer to Annex B for detailed inspection notes for all elements. Below is a summary of the inspection results.

3.1. STEEL SHEET PILING

The steel sheet piling (SSP) is broken into 2 sub-groups:

1. Original SSP from elevation -5.936 to sea floor (BZ IV N sheet piling).
2. Transfer cap SSP (PZ 27 sheet piling).

It appears that the original SSP could be a critical structural member for Pier 2 which contains a "pressure grouted concrete" material as explained in the 1996 drawings. Pier 3 has a "tremie concrete foundation" (as explained in the 1996 drawings) however it also states that it is in "poor quality" therefore the SSP could also be a critical structural member.

Ultrasonic thickness readings of the SSP were taken at certain locations around both piers. Two stations were chosen on Pier 2 and 4 stations were chosen on Pier 3. Thickness readings were taken at the 3.00 m vertical intervals where the SSP had been cleaned. At each interval, the out-pan, web and in-pan were measured where possible.

The original thicknesses of the SSP are as follows:

- Pier 2, Charlottetown Pier Transfer Cap (installed in 1996)
  - Out-pan 9.5 mm
  - Web 9.5 mm
  - In-Pan 9.5 mm
- Pier 2, Charlottetown Pier Original SSP (installed in 1959)
  - Out-pan 14.0 mm
  - Web 10.0 mm
  - In-Pan 14.0 mm
- Pier 3, Stratford Pier Transfer Cap (installed in 1996)

  - Out-pan 9.5 mm
  - Web 9.5 mm
  - In-Pan 9.5 mm
- Pier 3, Stratford Pier Original SSP (installed in 1959)
  - Out-pan 14.0 mm
  - Web 10.0 mm
  - In-Pan 14.0 mm

The following tables (Table 1 and Table 2) summarize the thickness readings and calculate the percentage of the original thickness of the steel.

Table 1: UT Measurements on Pier 2 (Charlottetown Pier)

	Elevation (m)	Location	Station 10.00 m Measured Thickness (mm)	Station 10.00 m Corrosion Rate (mm/year)	Station 26.75 m Measured Thickness (mm)	Station 26.75 m Corrosion Rate (mm/year)
Transfer Cap, PZ 27 SSP (Built 1996)	Top of Transfer Cap	Outpan	7.05	0.14	9.85	-0.02
		Web	No Reading		8.20	
		Inpan	6.05	0.19	9.40	0.01
	-5.436	Outpan	9.75	-0.01	9.80	-0.02
		Web	8.20	0.07	8.70	0.04
		Inpan	9.55	0.00	9.75	-0.01
Original SSP (Built 1959)	-5.936	Outpan	12.35	0.03	10.45	0.06
		Web	8.05	0.04	6.30	0.07
		Inpan	11.75	0.04	10.10	0.07
	-8.936	Outpan	13.25	0.01	13.15	0.02
		Web	5.05	0.09	8.70	0.02
		Inpan	13.35	0.01	13.20	0.01
	-11.936	Outpan	13.45	0.01	13.00	0.02
		Web	9.05	0.02	8.35	0.03
		Inpan	13.40	0.01	13.20	0.01
	-14.936	Outpan	13.60	0.01	13.15	0.02
		Web	8.50	0.03	8.55	0.03
		Inpan	13.25	0.01	13.20	0.01
	-17.936	Outpan	13.80	0.00	N/A	
		Web	No Reading		N/A	
		Inpan	13.45	0.01	N/A	
	Bottom	Outpan	13.70	0.01	13.30	0.01
		Web	No Reading		8.60	0.03
		Inpan	No Reading		12.99	0.02
Notes:	Negative corrosion rates indicate the measurement was thicker than the original thickness.					

Table 2: UT Measurements on Pier 3 (Stratford Pier)

	Elevation (m)	Location	Station 0.00 m Measured Thickness (mm)	Station 0.00 m Corrosion Rate (mm/year)	Station 4.00 m Measured Thickness (mm)	Station 4.00 m Corrosion Rate (mm/year)	Station 12.5 m Measured Thickness (mm)	Station 12.5 m Corrosion Rate (mm/year)	Station 18.0 m Measured Thickness (mm)	Station 18.0 m Corrosion Rate (mm/year)
Transfer Cap, PZ 27 SSP (Built 1996)	Top of Transfer Cap	Outpan	6.30	0.18	8.35	0.06	7.15	0.13	7.75	0.10
		Web	4.85	0.26	7.45	0.11	3.05	0.36	6.30	0.18
		Inpan	8.80	0.04	8.05	0.08	6.30	0.18	6.75	0.15
	-5.436	Outpan	8.35	0.06	9.15	0.02	8.30	0.07	7.65	0.10
		Web	8.15	0.08	8.25	0.07	8.15	0.08	8.80	0.04
		Inpan	8.30	0.07	9.05	0.03	8.15	0.08	8.50	0.06
Original SSP (Built 1959)	-5.936	Outpan	N/A		11.80	0.04	N/A		11.35	0.05
		Web	N/A		6.55	0.06	N/A		6.90	0.06
		Inpan	N/A		11.20	0.05	N/A		11.80	0.04
	-8.936	Outpan	N/A		N/A		N/A		12.15	0.03
		Web	N/A		N/A		N/A		8.70	0.02
		Inpan	N/A		N/A		N/A		12.05	0.04
	Bottom	Outpan	11.70	0.04	13.30	0.01	12.05	0.04	12.20	0.03
		Web	8.75	0.02	8.45	0.03	8.75	0.02	8.80	0.02
		Inpan	12.35	0.03	12.65	0.02	12.90	0.02	11.60	0.04
Notes:	Negative corrosion rates indicate the measurement was thicker than the original thickness.									

There was marine growth on all of the SSP with the heaviest growth at the sea floor and becoming less dense at higher elevations. There was evidence of Microbiologically Induced Corrosion (MIC) throughout both piers. This is characterized by a bright orange, iron rich layer followed by a dense black layer followed by bright and shiny steel surface underneath (see Photo 1 below) and subsequent pitting of the steel. The MIC ranged from 3 mm to 6 mm thick (orange and black layers combined). As shown in the tables above, the most severe corrosion rate measured on both piers was 0.36mm/year but on average it was around 0.15 to 0.20 mm/year. The worst corrosion was on the east and west faces of Pier 3 at the top of transfer cap SSP which is in the anodic zone. The anodic zone is typically within 1.0 or 1.5 m below the low water mark.

Photo 1: Piece of MIC brought to surface.

It is noted that approximately 50% of the transfer cap SSP is peeling away from the concrete transfer cap. It is unclear whether this occurred during construction or if it has been happening over a long period of time. There are gaps between the SSP and concrete of as much as 150 mm.

3.2. CONCRETE WORKS UNDERWATER

The concrete underwater was broken into 3 sub-elements.

1. Horizontal ledge between original SSP and transfer cap (built 1959).
2. Horizontal surface at the top of transfer cap (built 1996).
3. Vertical face of the pier wall up to high water elevation (built 1996).

The ledge between the original SSP and the transfer cap has localized spalling in certain areas but is generally in a good condition state. The spalling appears to be adjacent to the original SSP. The concrete appeared to be heavily scaled but considering it age, this would be expected. See Photo 2 below.

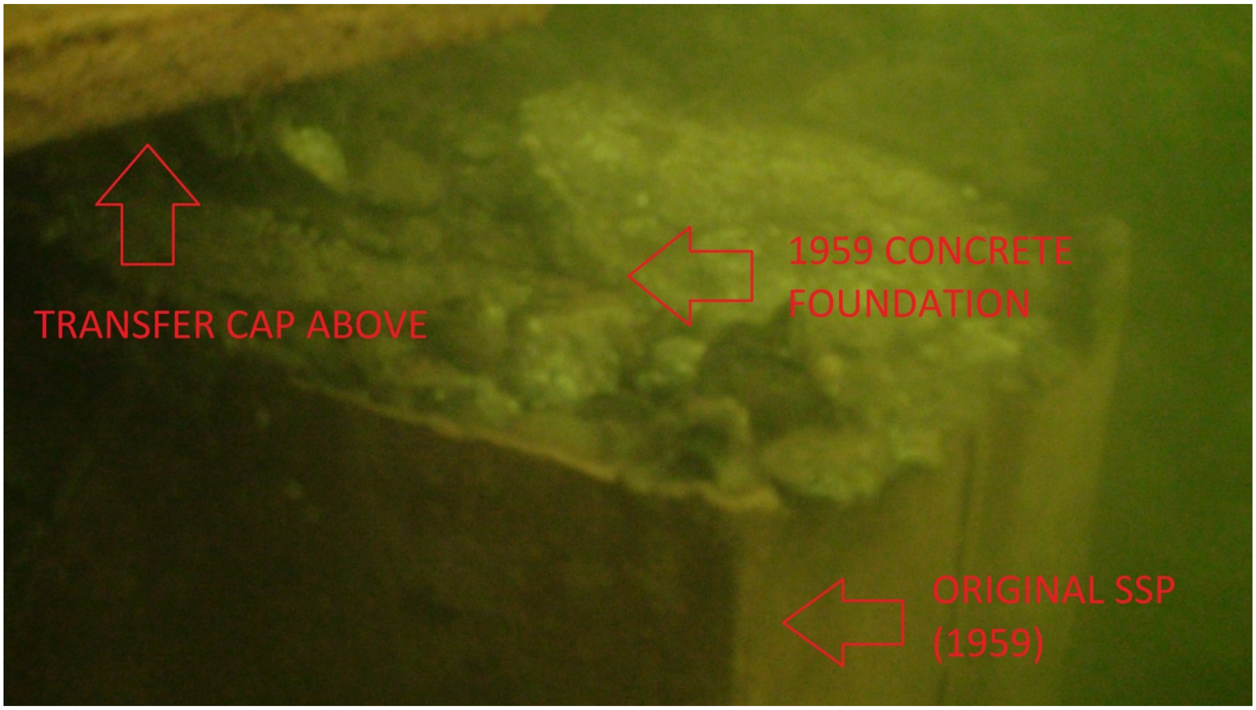


Photo 2: Typical condition of original concrete foundation.

The top of transfer cap is generally in a good condition state. There are some local areas that have some spalling and delaminations, specifically at Stations 1.5 m and 12.5 m on Pier 3. Both piers have several HP sections protruding vertically from the concrete. It appears these were used for formwork/cofferdam for the pier wall during the 1996 construction. These sections were typically protruding 200 – 300 mm from the concrete.

The vertical face of the pier wall from the transfer cap to high tide is in similar condition to the pier wall above high tide. There are several narrow cracks throughout but there does not appear to be any spalls.

3.3. CONCRETE WORKS ABOVE WATER

The concrete works above water was inspected from a boat at high tide and from above with the use of fall protection equipment.

There were several narrow cracks with wet stains on both piers. Cracks varied from horizontal to vertical cracks (with other orientations between horizontal and vertical). There was approximately 128 m of narrow cracks and 5 m of medium cracks on Pier 2. There was approximately 52 m of narrow cracks and 4 m of medium cracks on Pier 3. This is consistent with the bi-annual bridge inspection report submitted in 2013.

Each corbel has 9 post tensioning block-outs on the top face adjacent to the girder. Three of the block-outs on Pier 2 have delaminated and 2 block-outs on Pier 3 have delaminated. There were localized delaminations around the bearings for the truss on Pier 2 for a total area of 1.1 m^2. There were delaminations and spalling (150 mm in height on average) at the top edge of the ice shields on both piers. On the east side of Pier 2, the ice shield had a hollow sound when tapped with the hammer.

3.4. GEWI PILES

The inspection of the GEWI piles was limited due to their encasement in concrete at the transfer cap. The concrete above the GEWI piles appeared to be in good condition with the exception of 2 areas where spalling and delaminations were observed. These locations were on the south east and south west corners of Pier 3. Refer to Section 3.2.

3.5. ICE SHIELDS

The ice shields appear to be in good condition. The original thickness of the ice shields was 12mm and it was galvanized steel (according to the 1996 expansion drawings). UT readings were taken on the Pier 3 west ice shield. Readings were as follows:

- Bottom of ice shield (El. ±-2.5 m) : 10.65 mm
    - Corrosion rate = 0.08 mm/year
- Top of ice shield (El. ±1.0 m) : 9.60 mm
    - Corrosion rate = 0.13 mm/year

3.6. OVERALL GEOMETRY OF THE PIER

The outer dimensions of the SSP were measured underwater. Refer to Annex A, SK-02 for as-built measurements. All measurements were greater than the dimensions provided on the 1996 drawings.

Elevations were measured at sea bed by using the diver’s pneumofathometer gauge which is a depth measuring device used to monitor the divers depth at all times. This measurement is not accurate enough for a bathymetric survey however it an approximate depth at each vertical dive location. Any elevations indicated on the drawings in Annex A and the field notes were taken from the 1996 design drawings.

4.0 DISCUSSION

The condition of the SSP on both piers was very similar. There was evidence of MIC on the SSP at most locations. It is anticipated that the MIC has caused the accelerated corrosion rates. Besides the MIC, the condition of the SSP was good. There were no major holes or deformations to be noted. The marine growth was heaviest at the bottom elevations. There was a horizontal HP section located on Pier 2 at approximately elevation -14.336 m. It is expected that this was used as a tension tie but it is not indicated on the 1996 design drawings. It could have been installed for construction purposes when the original SSP was installed in 1959.

The concrete below water was in an overall good condition state. The only area of concern is at stations 1.25 and 12.5 on Pier 3. The concrete on the top of the transfer cap has delaminated and spalled in these areas which are located above the GEWI piles. It is unclear whether the spalling was caused by the piles but one would expect this type of failure if there was a bearing or punching shear failure of the transfer cap around the 200x200x40 thick bearing plate on top of the GEWI piles. The remaining areas with localized spalling on the ledge between the original SSP and transfer cap was likely due to settlement of the concrete foundation relative to the SSP which caused the concrete to chip around the edges of the original SSP (before the 1996 construction).

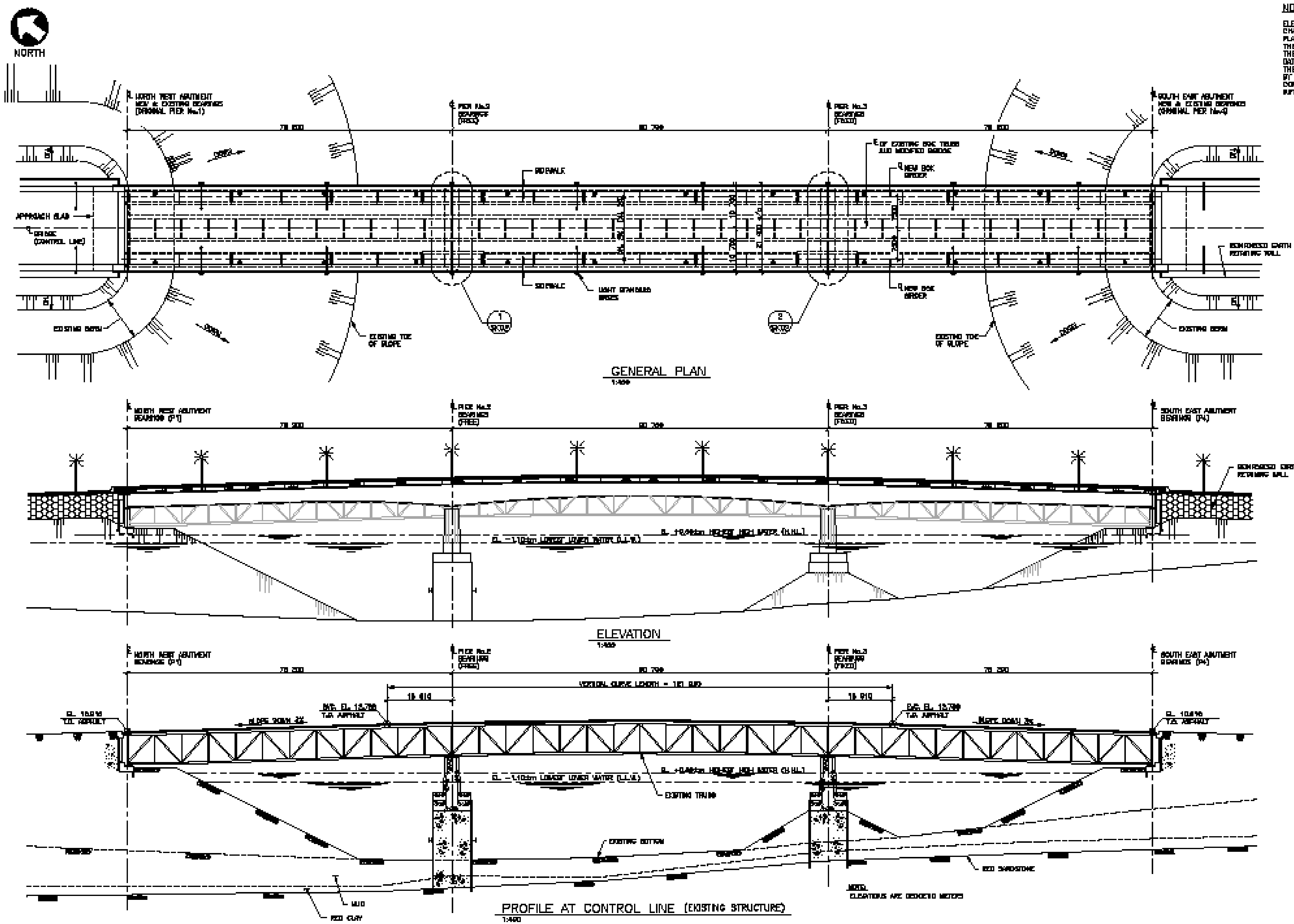
The concrete above water was in an overall good condition state. This is consistent with the previous bi-annual bridge inspection report completed in 2013. There are several narrow cracks throughout both piers however they are characteristic of a concrete structure of this size and thickness. The delamination of the grout filled post tensioning block-outs was likely caused by the combination of the infiltration of water and a freeze thaw cycle. It is unclear as to what caused the delaminations above the ice shields. It appears that these delaminations have been repaired at one time but the repairs have also delaminated.

5.0 RECOMMENDATIONS/ CONCLUSION

Based on this inspection, there are no material defects that are alarming from a structural point of view. The only area of concern would be the spalling on the top of Pier 3 transfer cap in the vicinity of the GEWI piles. It is difficult to ascertain, without the benefit of knowledge of the loads in these elements or the specifics of the overall analysis of the bridge, how serious this issue is. We are willing to participate with TIR in an assessment of this issue if required.

Upon review of this report, should the reader have any questions or concerns, please don't hesitate to contact Greg MacDonald (gmacdonald@harboursideengineering.ca) or Ronald Keefe (rkeefe@harboursideengineering.ca) to discuss.

Annex A Bridge Plan, Elevations and Pier Details



NOTE:

ELEVATIONS ARE REFERENCED TO CHART DATUM. CHART DATUM IS, BY INTERNATIONAL AGREEMENT, A PLANE BELOW WHICH THE TIDE WILL SELDOM FALL. THE CANADIAN HYDROGRAPHIC SERVICE HAS ADOPTED THE PLANE OF LOWEST NORMAL TIDE AS CHART DATUM. AS THE RISE AND FALL OF TIDES VARY DAILY, THE CANADIAN TIDE AND CURRENT TABLES, AS ISSUED BY THE CANADIAN HYDROGRAPHIC SERVICE, SHOULD BE CONSULTED FOR PREDICTIONS AND OTHER TIDAL INFORMATION RELATING TO THE WORK.

No.	Description	mm/dd/yy	By
A	ISSUED FOR REPORT	10/28/14	[illegible]

ISSUE or REVISION

Client

HARBOURSIDE
Engineering Consultants

Project
HILLSBOROUGH BRIDGE
CHARLOTTETOWN, P.E.I.

FOUNDATION DIVE REVIEW

Title
GENERAL ARRANGEMENT

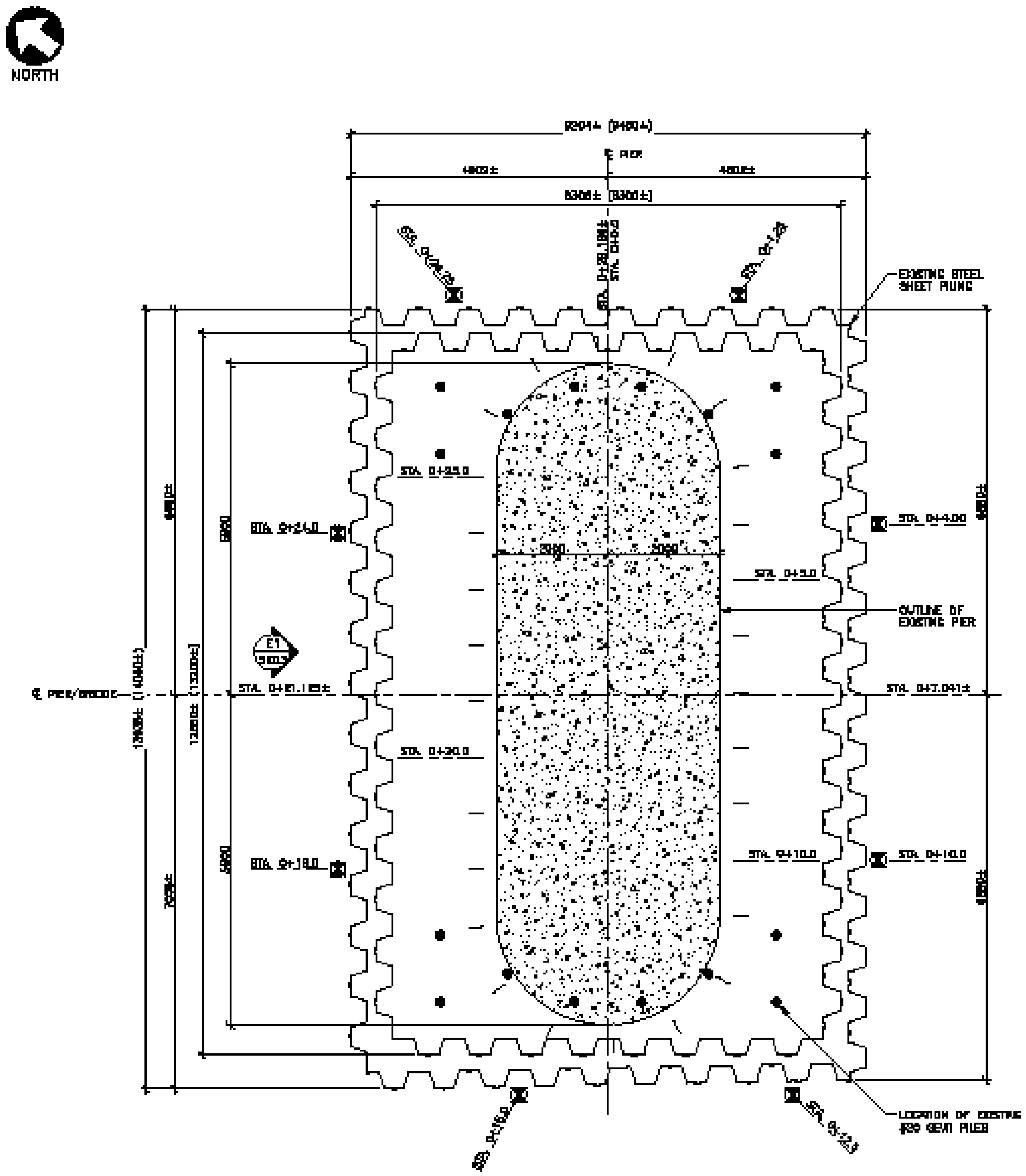
Designed	Checked	Approved	Drawn
[illegible]	-	[illegible]	J.BENNETT

Scale	Date
AS NOTED	10/14/14

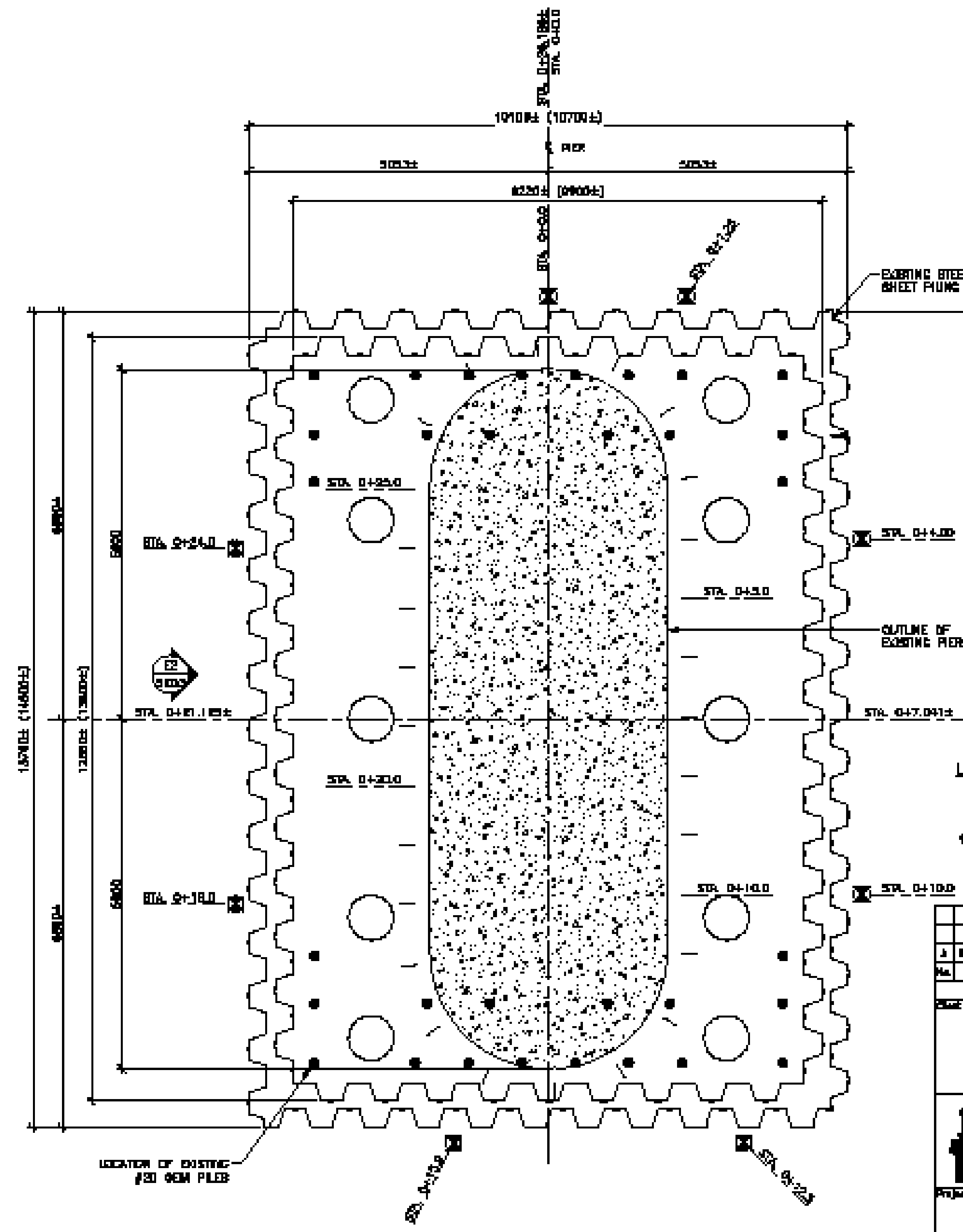
Contract No. 14129 — Rev. A

Drawing No.
SK01

DO NOT SCALE DRAWINGS



1 SK01 DETAIL 1:50 EXISTING PIER No.2 CHARLOTTETOWN



2 SK01 DETAIL 1:50 EXISTING PIER No.3 STRATFORD

LEGEND:

: VERTICAL DIVE LOCATION

9090 [9010] : THEORETICAL MEASUREMENT [AS-BUILT MEASUREMENT]

No.	Description	mm/dd/yy	By
A	ISSUED FOR REPORT	10/15/14	[illegible]

ISSUE or REVISION

Client

HARBOURSIDE Engineering Consultants

Project

HILLSBOROUGH BRIDGE
CHARLOTTETOWN, P.E.I.

FOUNDATION DIVE REVIEW

Title

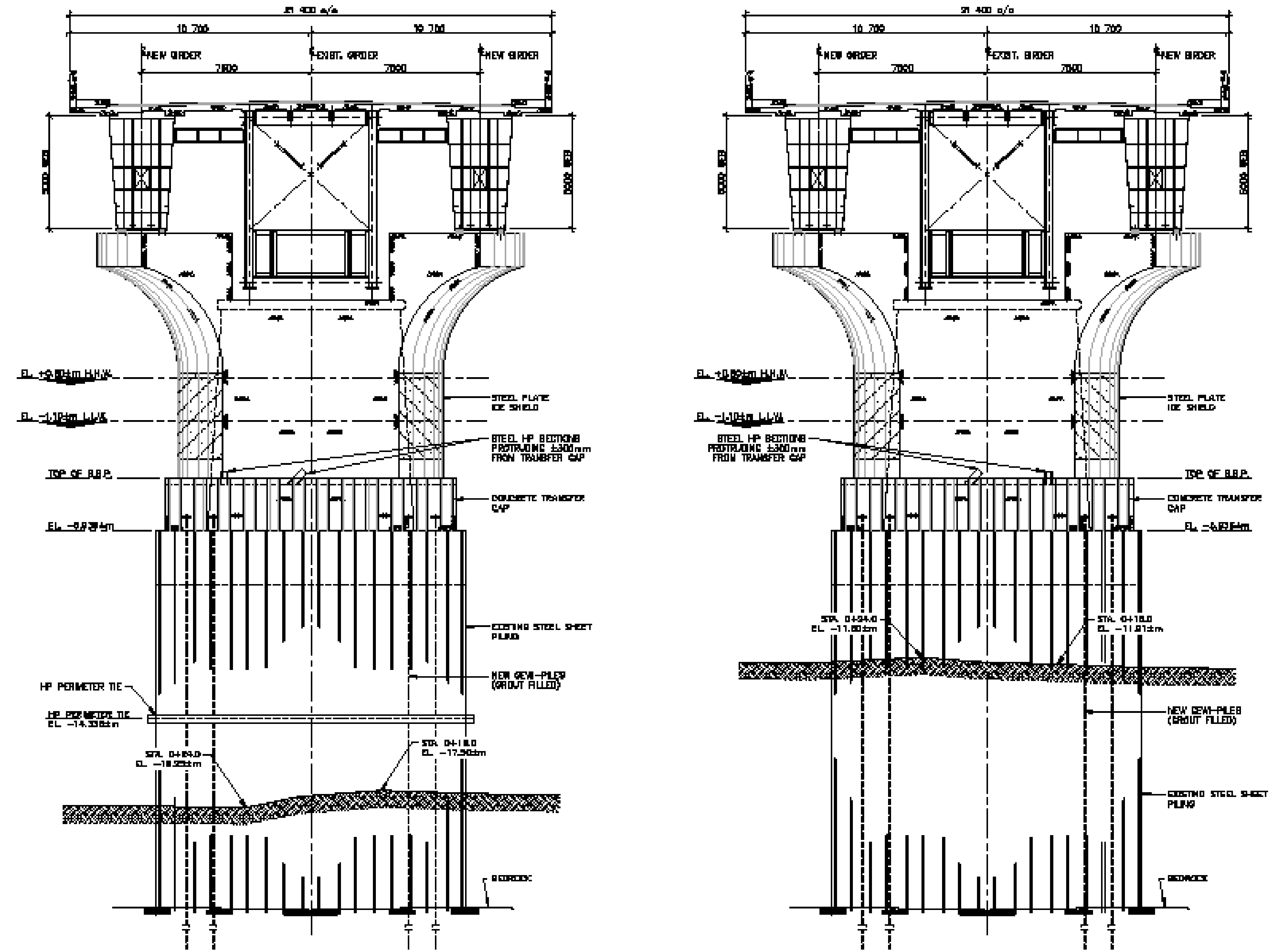
DETAILS
PIER No. 2 AND PIER No. 3

Designed	Checked	Approved	Drawn
[illegible]	-	[illegible]	J.BENNETT
		Scale: AS NOTED	Date: [illegible]
		Contract No.: 14159	Rev: A
		Drawing No.: SK02	

DO NOT SCALE DRAWINGS

NOTE:

ELEVATIONS ARE REFERENCED TO CHART DATUM. CHART DATUM IS, BY INTERNATIONAL AGREEMENT, A PLANE BELOW WHICH THE TIDE WILL SELDOM FALL. THE CANADIAN HYDROGRAPHIC SERVICE HAS ADOPTED THE PLANE OF LOWEST NORMAL TIDE AS CHART DATUM. AS THE RISE AND FALL OF TIDES VARY DAILY, THE CANADIAN TIDE AND CURRENT TABLES, AS ISSUED BY THE CANADIAN HYDROGRAPHIC SERVICE, SHOULD BE CONSULTED FOR PREDICTIONS AND OTHER TIDAL INFORMATION RELATING TO THE WORK.



E1 / SK02 — ELEVATION 1:100 — EXISTING PIER No.2 — CHARLOTTETOWN

E2 / SK02 — ELEVATION 1:100 — EXISTING PIER No.3 — STRATFORD

DO NOT SCALE DRAWINGS

No.	Description	mm/dd/yy	By
A	ISSUED FOR REPORT	10/[illegible]/14	[illegible]

ISSUE or REVISION

Client:

HARBOURSIDE Engineering Consultants

Project: HILLSBOROUGH BRIDGE CHARLOTTETOWN, P.E.I.

FOUNDATION DIVE REVIEW

Title: ELEVATIONS PIER No. 2 AND PIER No. 3

Designed	Checked	Approved	Drawn
[illegible]	-	[illegible]	J.[illegible]

Scale	Date
AS NOTED	[illegible]/14

Contract No.: 14[illegible] — Rev: A

Drawing No.: SK03

Annex B Detailed Inspection Notes

The following are the detailed inspection notes for both piers. The stations referred to are as per the stations identified on drawing SK-02 and elevations referenced are as per the datum provided on the 1996 expansion drawings. Both elevations and stations are approximate.

Steel Sheet Piling

Pier 2 (Charlottetown)

Station (m)	Elevation (m)	Notes
1.25	Top of Transfer Cap	Medium pitting. No major holes. Local severe corrosion for top 3" (very local). SSP is peeling away from concrete.
	-5.436	Shiny steel. Light to medium pitting. No major holes or severe corrosion. Evidence of MIC.
	-5.936	Shiny steel. Medium pitting. No holes or severe corrosion. Evidence of MIC.
	-8.936	Shiny steel. Medium pitting. No holes or severe corrosion. Evidence of MIC.
	-11.936	Shiny steel. Medium pitting. No holes or severe corrosion. Evidence of MIC.
	-14.936	Shiny steel. Light pitting. No holes or severe corrosion.
	-22.750 (Bottom)	Shiny steel. Light pitting. No holes or severe corrosion.
4.00	Top of Transfer Cap	Shiny Steel. Light pitting. Evidence of MIC.
	-5.436	Light Pitting. Evidence of MIC. There is a 4" angle running horizontally.
	-5.936	Light Pitting. Evidence of MIC with shiny steel underneath.
	-8.936	Shiny steel. Light pitting. Evidence of MIC.
	-11.936	Shiny steel. Light pitting. Evidence of MIC 6mm thick. Heavy growth, 3-4" thick.
	-14.936	Light Pitting. Evidence of MIC with shiny steel underneath.
	-17.936	Light Pitting. Evidence of MIC with shiny steel underneath.
	-21.000 (Bottom)	Evidence of MIC. Steel in good condition.
10.00	Top of Transfer Cap	SSP is peeling away from the concrete. Heavy pitting and shiny steel.

	-5.436	Evidence of MIC. 4" angle running horizontally.
	-5.936	Light pitting. Appears to be in good condition.
	-8.936	Light pitting.
	-11.936	Light pitting. Heavy growth.
	-14.936	Light Pitting. Evidence of MIC.
	-17.936	Light pitting. Evidence of MIC.
	-20.750 (Bottom)	Light pitting.
12.50	Top of Transfer Cap	Evidence of MIC approximately 3mm thick. Light Marine growth 6 to 12mm.
	-5.436	Severe pitting. Steel is not smooth.
	-5.936	Steel in good condition.
	-8.936	Steel in good condition.
	-11.936	Steel in good condition. No evidence of MIC.
	-14.936	Steel in good condition.
	-17.936	Steel in good condition.
	??? (Bottom)	Couldn't measure bottom due to severe conditions.
15.00	Top of Transfer Cap	SSP is peeling away from concrete. Medium pitting. No holes. Evidence of MIC.
	-5.436	Medium pitting. Evidence of MIC.
	-5.936	Light pitting. Visibility was bad at this location. Steel appears to be in good condition. Heavy Growth.
	-8.936	Light pitting. Clean shiny steel.
	-11.936	Steel is in good condition. Light pitting.
	-14.936	Light pitting. Steel is in good condition.
	-19.750 (Bottom)	Steel is in good condition. Light pitting. Shiny steel under growth.
18.00	Top of Transfer Cap	Shiny steel underneath MIC. There is a hole which was likely from a burning operation to cut the SSP. The web has a hole 2-3" in diameter.
	-5.436	Shiny steel underneath MIC. No holes. Light pitting.
	-5.936	Shiny steel. Evidence of MIC 6mm thick.
	-8.936	Shiny steel underneath MIC. No holes. Very light pitting. Heavy growth.
	-11.936	Very light pitting. Diver peeled a piece of MIC off the wall.

	-14.936	Smooth steel. No pitting. Steel in good condition. Heavy growth in this area.
	-17.500 (Bottom)	Evidence of MIC. There is debris from construction on the sea floor. Sea bed is relatively level in all directions.
24.00	Top of Transfer Cap	Holes in web from corrosion. Steel on in-pan is smooth.
	-5.436	Evidence of MIC. Light to medium pitting. Shiny steel behind MIC.
	-5.936	Shiny steel with minor pitting. Heavy growth in this area.
	-8.936	Shiny steel with minor pitting.
	-11.936	Shiny steel with no pitting.
	-14.936	Shiny steel with no pitting.
	-17.936	Shiny steel with no pitting. Heavy growth.
	-18.25 (Bottom)	Shiny steel with no pitting. Heavy growth.
26.75	Top of Transfer Cap	Medium pitting. No holes.
	-5.436	Light pitting. No holes.
	-5.936	Light pitting. Small hole likely from construction.
	-8.936	Light pitting. No holes.
	-11.936	Light pitting. No holes.
	-14.936	Light pitting. No holes.
	-19.00 (Bottom)	Light pitting. No holes. Shiny steel.

Pier 3 Stratford

Station (m)	Elevation (m)	Notes
0.00	Top of Transfer Cap	Medium pitting. Steel seems to be in good condition. Shiny steel.
	-5.436	Medium pitting. Steel seems to be in good condition. Shiny steel. No holes.
	-5.936	Light pitting. Shiny steel. No holes.
	-6.400 (Bottom)	Light pitting. Shiny steel. No holes.

2.00	Top of Transfer Cap	Severe pitting. Severe corrosion on top edge. No holes.
	-5.436	Medium pitting. No holes.
	-5.936	Light pitting. No holes. Shiny steel.
	-7.000 (Bottom)	Light pitting. No holes. Shiny steel.
4.00	Top of Transfer Cap	Light pitting. Steel in good condition.
	-5.436	Light pitting.
	-5.936	Shiny steel.
	-8.936	No pitting. Steel in good condition. Shiny steel.
	-10.000 (Bottom)	No pitting. Steel in good condition. Shiny steel.
10.00	Top of Transfer Cap	Light pitting. Steel in good condition.
	-5.436	Shiny steel. Evidence of MIC under marine growth.
	-5.936	Steel is in good condition. Shiny steel.
	-8.936	Heavy marine growth. Steel in good condition. Shiny steel.
	-11.600 (Bottom)	Heavy marine growth. Steel in good condition. Shiny steel. Grade drop off away from pier. Sea bed is armor stone.
12.50	Top of Transfer Cap	Light pitting. No holes. Shiny steel.
	-5.436	Light pitting. Steel in good condition. Shiny steel. Evidence of MIC 6mm thick.
	-5.936	Light pitting. No holes. Shiny steel.
	-7.650 (Bottom)	Light pitting. Shiny steel. 2 holes through SSP. 1 hole seems to have concrete behind SSP. There is a 12 to 25mm gap until the concrete which is likely the thickness of the SSP.
15.00	Top of Transfer Cap	Severe pitting. No holes. Evidence of MIC.
	-5.436	Medium pitting. No holes. Evidence of MIC. Shiny steel.
	-5.936	No holes. Evidence of MIC.
	-7.250 (Bottom)	Light pitting. No holes. Steel is in good condition.

18.00	Top of Transfer Cap	Evidence of MIC. Light pitting. Steel appears to be in good condition.
	-5.436	Light pitting. Steel is in good condition. Shiny steel.
	-5.936	Light pitting. Evidence of MIC 6mm thick.
	-8.936	Smooth steel. Evidence of MIC 6mm thick. No holes.
	-11.910 (Bottom)	Smooth steel. Evidence of MIC 6mm thick. No holes.
24.00	Top of Transfer Cap	Shiny steel. Medium pitting.
	-5.436	Shiny steel. Light pitting. Evidence of MIC 6mm thick.
	-5.936	Light pitting. No holes.
	-8.936	Light pitting. No holes.
	-11.600 (Bottom)	Light pitting. No holes. Light marine growth.

Concrete Ledge at Elevation -5.936 m

Pier 2 (Charlottetown)

Station (m)	Elevation (m)	Notes
1.25	-5.936	No spalls or visible cracks. Concrete appeared to be in good condition.
4.00	-5.936	Concrete in good to fair condition. There is localized spalling.
10.00	-5.936	Concrete appears to be in good condition. There are no major spalls or cracks.
12.50	-5.936	Concrete appears to be in good condition. There are localized spalls closer to the outside edge adjacent to the SSP.
15.00	-5.936	There is a HP section protruding from the concrete. There are no cracks or spalls. Concrete seems to be in good condition.
18.00	-5.936	There are additional angles and plates which cover the concrete ledge. There are also some sandbags (or concrete bags) at this location. It appears they were used as a form for the newer transfer cap.
24.00	-5.936	There are additional angles and plates which cover the concrete ledge. There is also some sandbags (or concrete bags) at this location. It appears they were used as a form for the newer transfer cap.
26.75	-5.936	Concrete appears to be in good condition. There are no visible cracks, spalls or delaminations.

Pier 3 (Stratford)

Station (m)	Elevation (m)	Notes
0.00	-5.936	There is some spalling at this location. No visible signs of exposed reinforcing. No visible cracks.
2.00	-5.936	There is some spalling at this location. No visible signs of exposed reinforcing. No visible cracks.
4.00	-5.936	Concrete is in good condition at this location.
10.00	-5.936	Concrete has some localized spalls but otherwise in good condition.
12.50	-5.936	Concrete has no visible cracks, spalls of delaminations.
15.00	-5.936	Concrete has no visible cracks, spalls of delaminations.

18.00	-5.936	Localized spalling but no signs or corroding reinforcing. Remaining concrete is in good condition.
24.00	-5.936	Concrete is in good condition at this location.

Top of Concrete Transfer Cap

Pier 2 (Charlottetown)

Station (m)	Elevation (m)	Notes
1.25	Top of Transfer Cap	No spalls. No delaminations. Concrete is 8-10" below the top of the transfer cap SSP. SSP is peeling away from the concrete at this location.
4.00	Top of Transfer Cap	Concrete appears to be in good condition.
10.00	Top of Transfer Cap	Concrete appears to be in good condition. There is a HP section protruding vertically from the concrete.
12.50	Top of Transfer Cap	Concrete appears to be in good condition.
15.00	Top of Transfer Cap	There is a HP protruding vertically from the top of concrete. Concrete appears to be in good condition. There are no cracks or spalls.
18.00	Top of Transfer Cap	Concrete appears to be in good condition.
22.00	Top of Transfer Cap	There is a HP section protruding at an angle from the top of the concrete. It is angle east to west at the top. Concrete appears to be in good condition.
24.00	Top of Transfer Cap	There is a HP protruding vertically from the top of concrete. Concrete appears to be in good condition. There are no cracks or spalls.
26.75	Top of Transfer Cap	Concrete appears to be in good condition.

Pier 3 (Stratford)

Station (m)	Elevation (m)	Notes
0.00	Top of Transfer Cap	
1.25	Top of Transfer Cap	There is a HP section protruding vertically from the top of concrete approximately 12" from the SSP. It appears the concrete has delaminations in this area. There are several pieces of concrete that have spalled off.
2.00	Top of Transfer Cap	There appears to be a 3-4" pipe protruding from the top of the concrete. Diver suspects this was a tremie pipe during construction.

5.00	Top of Transfer Cap	There is a HP section protruding vertically from the top of concrete.
6.30	Top of Transfer Cap	There are 2 HP sections protruding at an angle from the concrete which are leaning towards one another at the top. Concrete has spalled adjacent to the HP sections.
11.75	Top of Transfer Cap	There is a HP section protruding vertically from the top of concrete.
12.00	Top of Transfer Cap	There appears to be a 3-4" pipe protruding from the top of the concrete. Diver suspects this was a tremie pipe during construction.
12.50	Top of Transfer Cap	Concrete has severe spalls at this location.
13.00	Top of Transfer Cap	There is a HP section protruding vertically from the top of concrete. There is also a piece of rebar protruding from the concrete that appears to be 1 to 1-1/4" in diameter.
16.50	Top of Transfer Cap	There is a HP section protruding vertically from the top of concrete. There is some spalling adjacent to the HP section.
18.00	Top of Transfer Cap	Concrete appears to be in good condition. No spalls or delaminations.
19.50	Top of Transfer Cap	There is a HP section protruding vertically from the top of concrete.
21.00	Top of Transfer Cap	There is a HP section protruding at an angle from the top of concrete. The section is tilted east to west at the top.
24.00	Top of Transfer Cap	

Concrete Pier Walls

Pier 2 (Charlottetown)

Location	Notes
East Corbel	2 of 9 grouted post tensioning block-outs have delaminated from the concrete. 2.5 m narrow crack with wet stains noted on the south face running from the top down to the waterline. There are 7- vertical cracks at 4.0 meters each which extend across the top of the corbel approximately 1.5 m throughout.
West Corbel	1 of 9 post tensioning block-outs have delaminated from the concrete. 2.5m narrow crack with wet stains noted on the south face and north face starting at the top and running towards the water line. Cracks are located directly under the box girders.
Center section	Delamination noted on the horizontal surface adjacent to bearings. West side has a delamination 1000x200 and the east side has 2 delaminations; 1500x300 and 1000x100. There is a narrow crack propagating from the west bearing of the truss towards the south face approximately 1.5m long. There are 2-4.0 meter cracks between the bearings of the truss on the top surface. There are several horizontal, vertical and angled narrow cracks on north and south faces. There are approximately 40 meters of narrow cracks with wet stains on the south face down to the low water mark. There are approximately 32 meters of narrow cracks with wet stains and 3 meter of medium cracks with wet stains on the north face down to the low water mark. Concrete just above ice shield on both east and west sides was delaminated and concrete has spalled off. The ice shield on the east side has a hollow sound when tapped with the hammer for 50% of its width at the top. Concrete behind the ice shield may be delaminated.

Pier 3 (Stratford)

Location	Notes
East Corbel	1 of 9 grouted post tensioning block-outs have delaminated from the concrete and grout has crumbled. There is an exposed post tensioning strand which had severe corrosion. The stressing head was not visible. 2.0 m narrow crack with efflorescence noted on the north face running from the top down to the waterline. 7m of narrow horizontal and vertical cracks on the north face. There are 10m of narrow vertical cracks on the south face.

West Corbel	1 of 9 post tensioning block-outs have delaminated from the concrete. 2.5m narrow crack with wet stains noted on the south face and north face starting at the top and running towards the water line. 6 m of narrow vertical cracks between the bearings on the north face. Cracks are located directly under the box girders. There are 2-300 long narrow crack propagating from the south corbel tie on the inner horizontal face.
Center section	There are 8.0m narrow cracks with efflorescence on the south face. There are 4m of medium cracks and 2m of narrow cracks on the horizontal face running longitudinally with the bridge. There are 2-4.0m medium cracks between the bearings of the truss. There is a delamination 300x300 adjacent to the east bearing on the horizontal face. There is a 5m long vertical crack on the south face. Concrete just above ice shield on both east and west sides was delaminated and concrete has spalled off.

Annex C Inspection Photos

Photo 3: Typical condition of the SSP.

Photo 4: Heavy pitting on SSP.

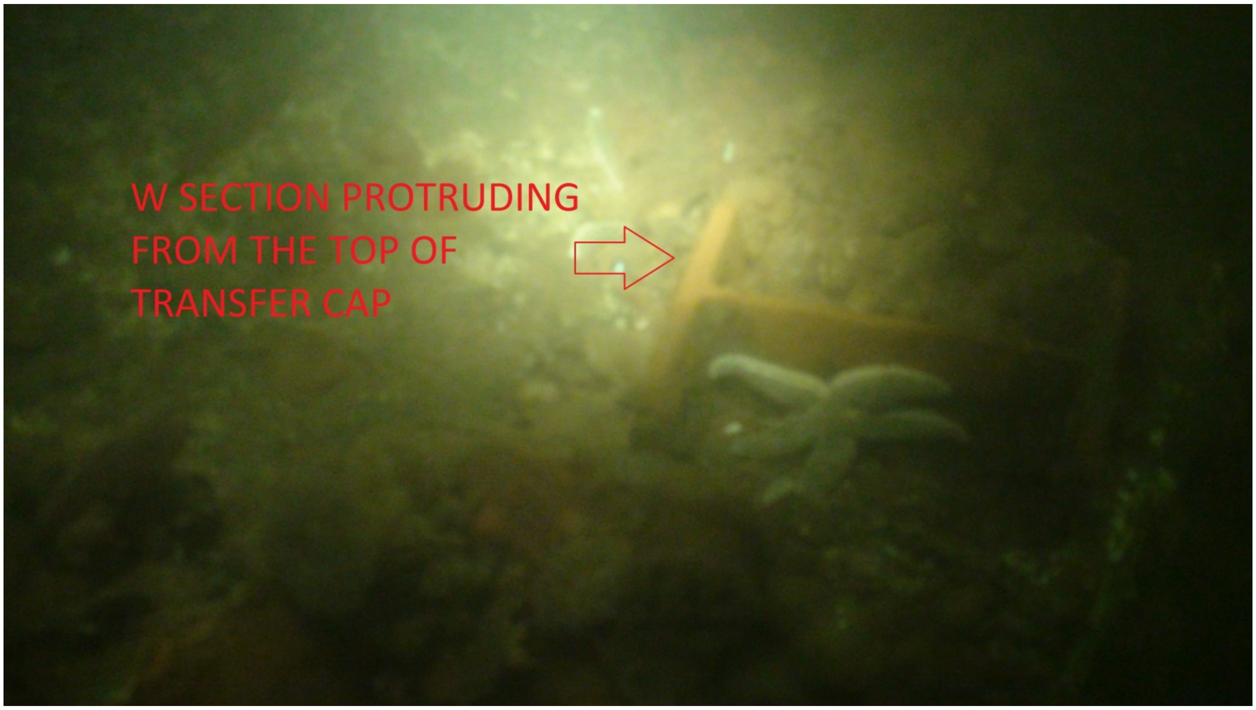


Photo 5: W-section protruding from transfer cap.

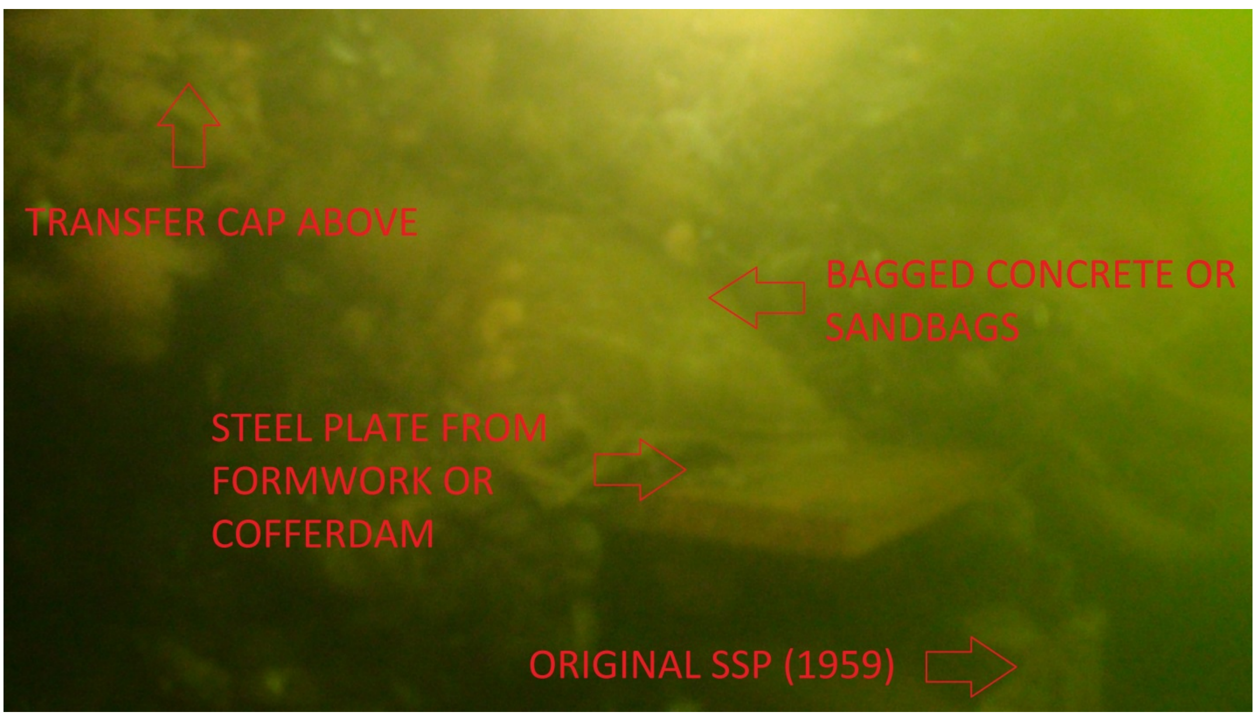


Photo 6: Additional formwork at the base of transfer cap.

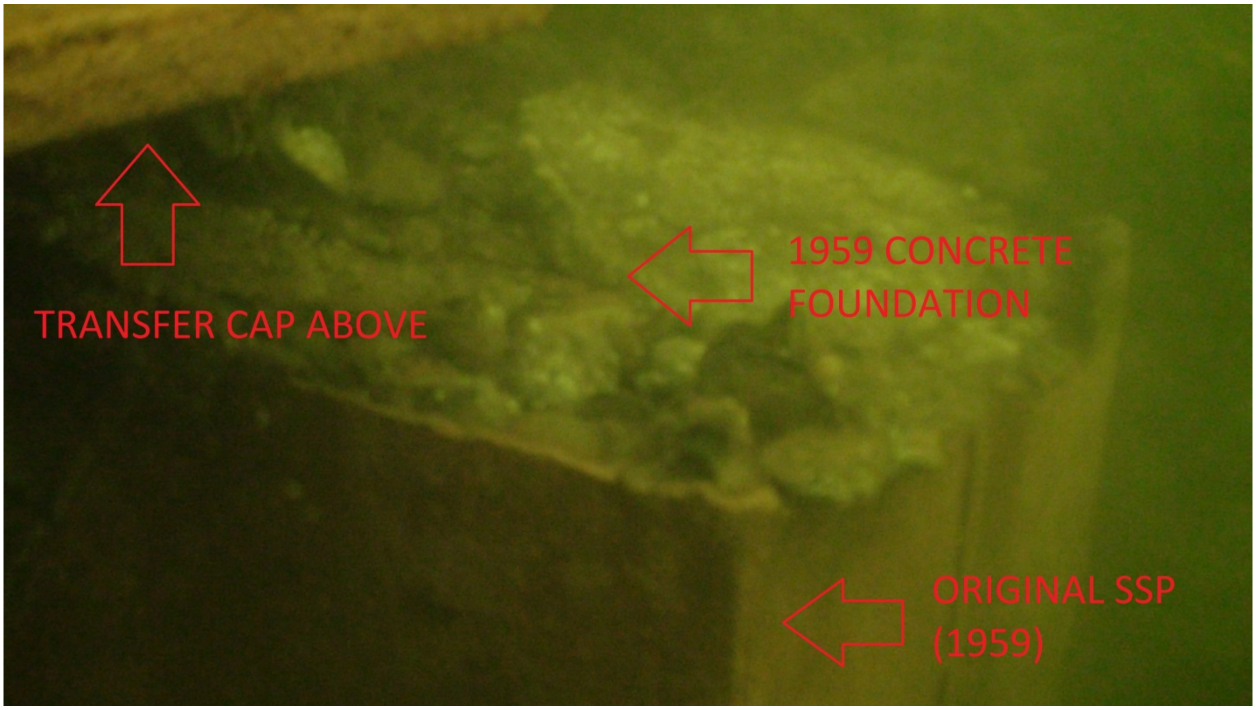


Photo 7: Typical condition of the concrete between original SSP and transfer cap.

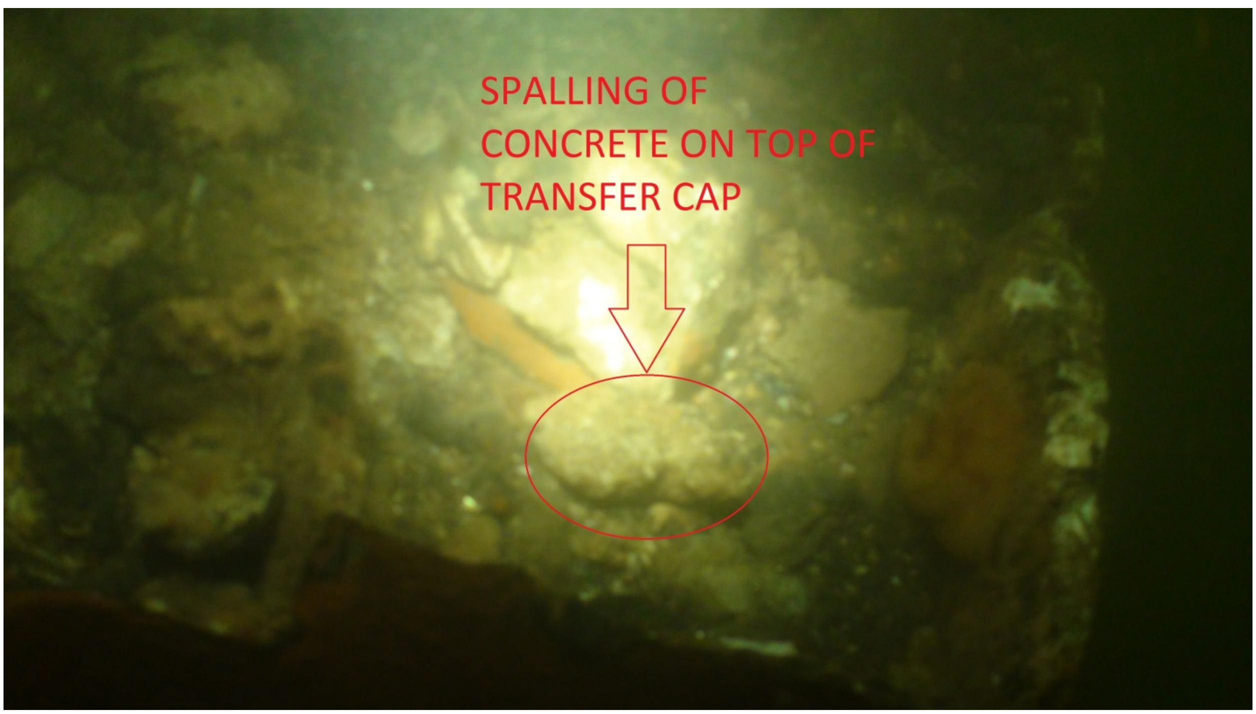


Photo 8: Spalling of concrete on the top of transfer cap.

Photo 9: Typical narrow cracking on pier wall.

Photo 10: Typical cracking on pier wall.

Photo 11: Repair of concrete above ice shields.

Photo 12: Typical spalling above ice shields.

Photo 13: Transverse cracks between truss bearings.

Photo 14: Spalling of grout in post-tensioning block-out.

Photo 15: Exposed PT strand.

Photo 16: Delamination adjacent to truss bearing.

Photo 17: Narrow cracks below box girders.