The PRINCESS MARGARET BRIDGE – Fredericton, N.B.

Innovative Solutions for a Major Rehabilitation

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Paper prepared for presentation

at the **Bridges – Innovations Session**

of the 2012 Conference of the
Transportation Association of Canada
Fredericton, New Brunswick

Princess Margaret Bridge Project

Abstract

The Princess Margaret Bridge (the “Bridge”) was built from 1957-59 as one of the largest and most modern bridges in New Brunswick, to serve as part of the Trans Canada Highway system, connecting the north and south sides of the Saint John River in Fredericton. It was officially inaugurated in 1959 by Princess Margaret. The Trans Canada Highway in the Fredericton area was bypassed by the Fredericton-Moncton Highway Project in 2001, and the Princess Margaret Bridge became a part of the Route 8 arterial highway system.

Figure 1: **Bridge under construction circa 1958.**

The 1.1 km long, 23-span steel structure is composed of several structural systems, and is supported on 14 land piers and 8 water piers.

The New Brunswick Department of Transportation & Infrastructure (“NBDTI”) determined in 2008 that the bridge was at the end of its useful service life due to age, an increase in live load due to truck volumes as well as size, combined with the extensive use of deicing salts to keep the bridge operational during Canada’s harsh winter conditions, and deferred maintenance. A major rehabilitation or complete bridge replacement was required due to the age and condition if the bridge.

The Princess Margaret Bridge Project (the “Project") was developed to partner with industry to find a solution that would provide both innovation and value for money to the Province. Since 2010, the 50-year old Princess Margaret Bridge structure has undergone a major rehabilitation by SNC-Lavalin.

This paper describes the Project objectives, procurement process, business model, and details the innovative structural system designed and constructed by SNC-Lavalin that revealed to be a cost-effective solution for the Province of New Brunswick.

1. Background

The Princess Margaret Bridge provides an elevated crossing of the Saint John River and flood plain at Fredericton, NB which measures 1097.1 m in overall length. A photograph of the bridge in profile is shown in Figure 2. It is a high-level structure supported on tall slender piers in order to provide the required 23± m vertical clearance at the navigation span and to match the topography of the steeply sloping hillside at the bridge's west abutment. The majority of the bridge superstructure has a concrete deck slab supported on traverse floor beams at 2800± mm centers. These floor beams are in turn supported by two main steel carrying trusses or girders, with the exception of the through truss span, where the floor beams are spaced at 6400 mm centre to centre. The roadway is supported by 9 deck truss spans, one through-truss navigation span, 7 plate girder spans and 6 rolled beam approach spans. Support for the steel superstructure is provided by 8 main river piers, 14 land based piers and 2 abutments.

Figure 2: Bridge Profile

The bridge was originally designed for an H20-S16: 32,500 kg design vehicle. For many years, the bridge had been carrying much heavier vehicles than originally designed for. The *Motor Vehicle Act* in New Brunswick allows vehicles < 62,500 kg on this route.

Routine inspections in 2008 revealed significant deterioration of both steel and concrete. NBDTI immediately completed a condition assessment discovering half of the bridge's rail posts were defective (see Figure 3) and half of the sidewalk supports and floor beams showed significant corrosion damage. Expansion joint drainage troughs and downspouts were leaking badly. There was vehicular collision damage and many seized bearings. Additionally, the concrete piers had suffered severe damage, and through an extensive substructure coring program, it was determined that it was caused alkali silica reaction and rebar corrosion. The deck and floor beams were deteriorated beyond repair. Pieces of the side walk had started to fall off the bridge and temporary nettings had been installed to protect pedestrian and motorists below.

The large movement finger plate expansion joints were in poor condition because of corrosion and misalignment. The box seal joints were replaced in a previous rehabilitation operation but were leaking. The main cause of deterioration in the steel superstructure was corrosion.

Figure 3: **Deteriorated Railpost and Pier.**

A structural condition assessment of the bridge in the fall of 2008 confirmed several structural deficiencies. Preliminary analysis indicated that certain steel members of the bridge superstructure were over-stressed when fully loaded. Immediately following the condition assessment, the bridge was weight restricted by NBDTI to a maximum 43,500 kg. It became necessary for affected trucks to take alternate routes to cross the Saint John River.

There was an urgent need to restore the desired truck carrying capacity of the bridge, and address structural deficiencies either though bridge rehabilitation or replacement, and it was fundamental that it be achieved with the least amount of closure time. The bridge carried 20,000 vehicles per day, and the only other river crossing in Fredericton was at capacity.

An immediate contract to repair critical steel members and replace some of the defective bearings (Phase 1) was completed in 2009 while the NBDTI studied bridge rehabilitation and replacement options under various procurement models.

2. Project Development and Procurement

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The New Brunswick Department of Transportation & Infrastructure put in place a project team to assess available options to meet its objectives and initiate the procurement process. Business models were developed for both traditional and design-build procurements for the major rehabilitation as well as for various replacement structure options. NBDTI ultimately decided to pursue a Design-Build contract for the rehabilitation of the bridge, with the objective of achieving a CL-625 live load carrying capacity and an additional 50 years of service life. The new bridge options were rejected at the time due to the exorbitant capital requirements, as well as the excessive delay inherent in that option for the users to have the truck loading reinstated. That option was estimated to add several years to the schedule for the completion of the necessary environmental review processes and the construction effort. A Design-Build model was chosen over traditional build and design procurements primarily for the purpose of schedule acceleration.

The Design-Build project delivery method had a three-stage procurement process. Between January and June 2010 there was a Request for Expressions of Interest ("RFEOI"), followed by a Request for Qualifications ("RFQ") and finally a Request for Proposals ("RFP") issued by NBDTI. Five RFQ respondents were qualified to proceed to the RFP stage. Three RFP proponents ultimately submitted detailed technical proposals, financial proposals as well as value engineering submissions in separate envelopes.

The scope of the rehabilitation work that was contracted for included:

- Removal and replacement of sidewalks, traffic barriers, deck slabs, and floor beams;
- replacement of bearings;
- refurbishment of steel superstructure;
- sandblasting and painting of existing steelwork;
- refurbishment of piers and abutments by encapsulation, and;
- provision of a new electrical system, inspection walkway and access ladders.

The contract required the work to be designed and built to a CL-625 live loading (63,700kg), and to provide a service life of 50 years. The rehabilitated bridge was to be designed without a sidewalk, to take maximize roadway width.

NBDTI drafted detailed technical and management system requirements in the project agreement. Technical specifications and standards limited deck replacement options to one of either cast-in-place concrete, precast post-tensioned concrete panels, or a steel orthotropic system.

A Request for Proposals process for the complete rehabilitation of the bridge (Phase 2) concluded in January 2010 with SNC-Lavalin Inc. as the Preferred Proponent. The Design-Build Agreement was signed on February 17, 2010 for a total project price of $77.4 M. SNC-Lavalin's design concept included a full deck replacement with a system of precast pre-stressed post-tensioned concrete deck panels, the design of which is in unique in Canada. To complete the rehabilitation, the bridge was fully closed to traffic for a total of 32 weeks in the 2010 and 2011 construction seasons. The bridge re-opened to traffic on November 10, 2011.

NBDTI worked cooperatively with the City of Fredericton to develop a Traffic Management Plan and identify mitigation strategies to minimize the congestion that was predicted would result from the bridge closure. A comprehensive communications strategy was developed, encouraging motorists to alter their habits by taking advantage of Park and Go and Park and Ride options that were put in place, as well as flexible work arrangements and the extensive recreational trail system throughout both sides of the Saint John River within the Fredericton City limits. Minor capital improvements were also identified and completed at key intersections in advance of the 2010 closure. Both the City and NBDTI kept a website, and links to live traffic cameras were available to commuters to monitor the situation before attempting to cross the alternate city bridge.

3. Design Concept and Structural Assessment

SNC-Lavalin proposed a creative solution: precast deck panels that are made composite with the trusses. To our knowledge, this is the first time trusses were made composite with a precast concrete deck. Not only did this solution accelerate construction, but it also allowed a significant in the amount of structural steel strengthening required in the trusses.

Dealing with an old steel truss bridge such as the Princess Margaret Bridge required a lot of creativity and innovation. SNC-Lavalin employed innovative design and construction techniques throughout the project to accelerate the schedule.

The construction phase of deck replacement presented an interesting challenge. The bridge trusses could not support the crane that was to be used to remove the old deck and install the new one. Custom built mobile lifting units were commissioned from Europe for the removal and replacement of the deck panels. They were electronically controlled machines that performed the work much faster and more effectively than a manually operated crane through their ability to rotate loads 360 degrees.

The assessment of the bridge steel superstructure members were categorized by letters. The first group is category A in which most of the bridge members were classified. Almost 90% of the members were in this category. Category A+ members had losses less than 5%, and Category A members had losses between 5 and 10%. This was followed by category B, a category in which almost 8% of the bridge members were in it. The members in this category suffered loss in section from 15 to 25%. This was followed by categories C and D in which almost 2% of the bridge members were in. The members in this category suffered loss in section over 25%.

The steel members were rated and tabulated. Both the capacity and the factored loads were based on the Canadian Highway Bridge Design Code.

Most of the bottom chord members as well as the diagonals using CL625 design truck and the heavier live load requirement had a rating less than 1.0 in some members. This necessitated the strengthening of those members. The top chord members were made composite with the new deck. This composite action tremendously increased the capacity of the top chord members and consequently increased their member rates significantly.

All bearings not already replaced by NBDTI, with the exception of the fixed spherical bearings needed to be replaced, especially the rocker bearings that were misaligned.

4. Seismic Analysis

Seismic analysis was performed using a 5% return period in 50 years which is an event every 1000 years. The bridge is located in Fredericton NE which is seismic zone 2. The bridge is classified as an emergency route bridge based in the project specifications. Because of the nature of the bridge, a Multi Mode Spectral Method was performed.

Accelerations versus time period were obtained from the Canadian national resources. Two main Finite Element Models were developed for the structure. The first was developed based on the lower bound of the soil parameter and the second was developed based on the upper parameter. Each of the above FEM has two models using 35% and 70% of the pier gross stiffness to account for the pier cracking moment of inertia. All results were bounded to get the most conservative values for the design and assessment

The soil was classified as soil type IV. The response spectrum were constructed and applied to the FEM of the structure. Displacements of the roller bearings were obtained. The bearing and the pedestals were designed to prevent potential unseating. Reactions of the fixed bearing were obtained as well to make sure that the bearings had enough strength to sustain those forces.

5. Bridge Strengthening: Composite Solution

A composite section was created between the deck slab and the steel members. This composite action tremendously improved the capacity of those members and therefore reduced the need for steel strengthening.

Composite action was created between the deck panels and the following steel bridge members:

1 - In the Deck Truss area between the top chord and the deck panels [*first time to be implemented worldwide to the knowledge of the authors*]

2 - In the Plate Girder area between the top flange and the deck panels

3 - In the Through Truss area between deck panels and the floor beams

Deck Truss panels were pre-tensioned transversally and post-tensioned longitudinally, which created a challenge to create the composite action. The composite action to the steel members was achieved as follows:

For both deck truss and plate girder, a spine beam was detailed to connect either the Plate Girder top flange or the Deck Truss top chord to the deck panels after the post-tensioning operations, as shown in Figure 4.

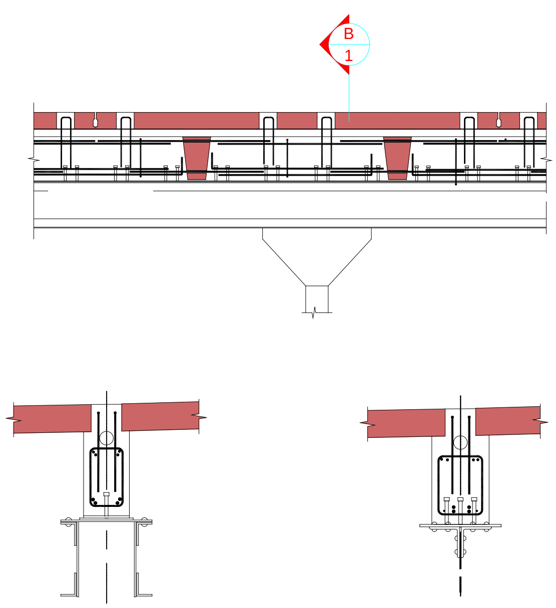

Figure 4: **Composite Action between the Precast Deck and either the Plate Girder Top Flange or the deck Truss Top Chord.**

For Through Truss, the composite action was created through a shear key between the panels in which shear studs are welded to the floor beams, as shown in Figure 5.

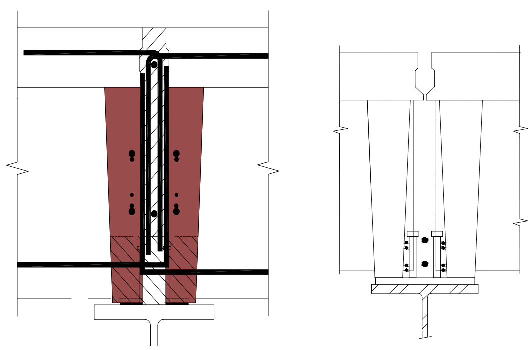

Figure 5: **Composite Action between the Precast Deck and the Through Truss Floor Beams**

6. New Design For Bridge Deck Reconstruction: Deck Trusses And Plate Girders

The deck design at the Deck Trusses and Plate Girders consist of precast double tee panels with the pre-tensioned ribs oriented transverse the direction of traffic. Deck panel geometry is shown in Figure 6. The typical double tee width is 4.5 m and length 9.66 m. the ribs were orientated in the transverse directions to comprise the floor beams and to span between the two main girder lines as shown in Figure 7. A unique system of post-tensioning the double tee slab parallel to the direction of traffic eliminated all duct coupling and possible beams misalignment has been implemented as shown in Figure 8.

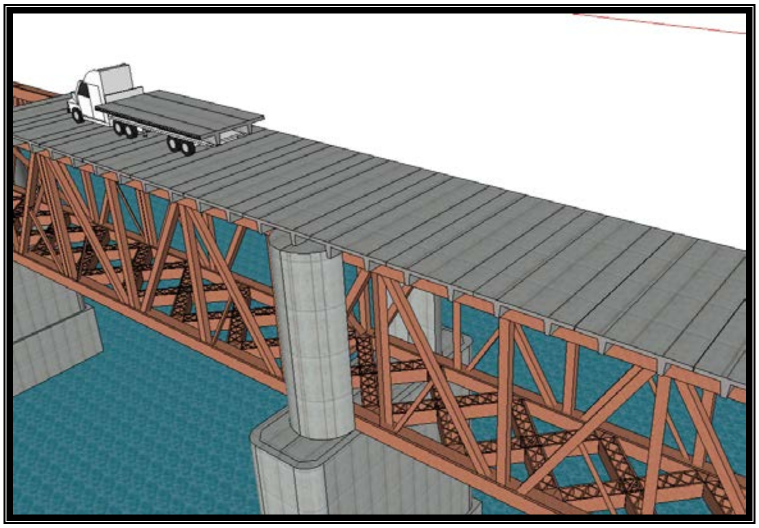

Figure 6: Deck panel geometry

Figure 7: Installation of typical precast panel

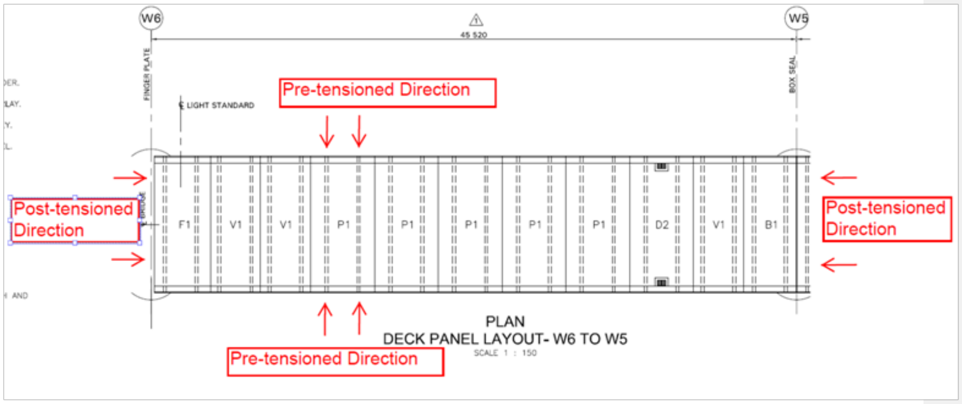

Figure 8: **Deck Truss and Plated Girder Deck Panel Design**

For deck truss and plate girder spans, the typical pre-tensioned profile and the post-tensioning layout is shown in Figures 9 and Figure 10.

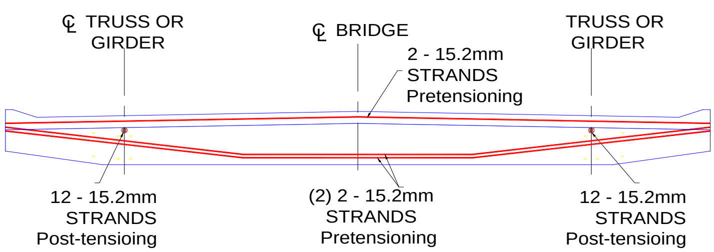

Figure 9: **Deck Truss and Plated Girder Deck Panel Pre-tensioning profile**

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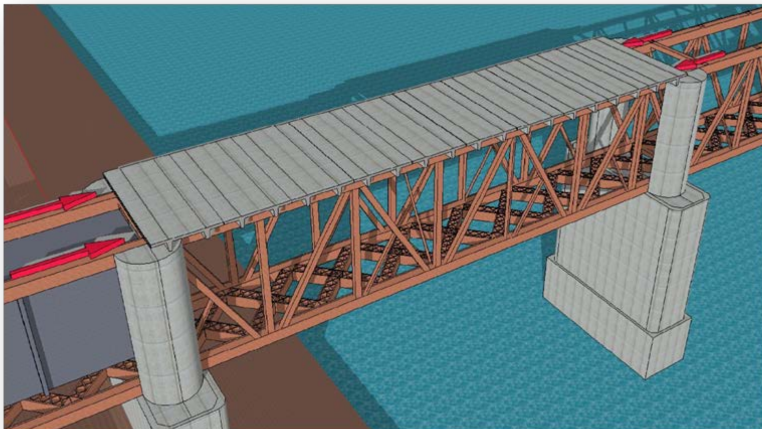

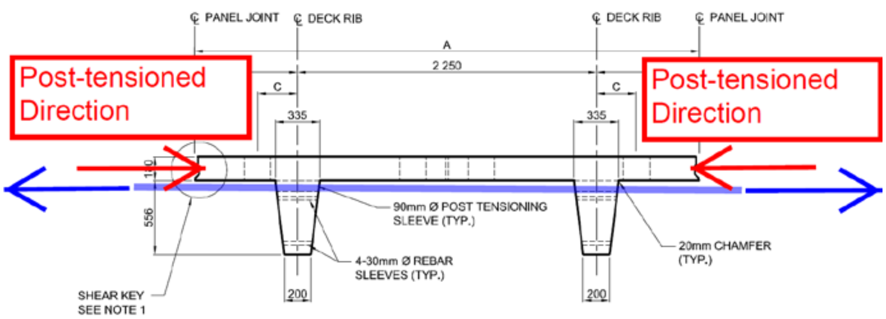

Figure 10: Deck Truss and Plated Girder Deck Panels Post-tensioning

7. Through truss

The Through Truss (Figure 11) measures 98.9 m in length.

Figure 11: **Through truss**

The floor beams in the Through Truss were in a fairly good shape and spaced at 9.6 m. The panel steel forms used for the Deck Truss and Plate Girder panels were utilized to form the Through Truss deck panels. Twenty deck panels were used.

The deck panels' ribs in the Through Truss were oriented to span between the floor beams. A second stage cast were added to the though truss panels to facilitate the panel installations as well to form the connecting beam above the floor beams see Figure 12.

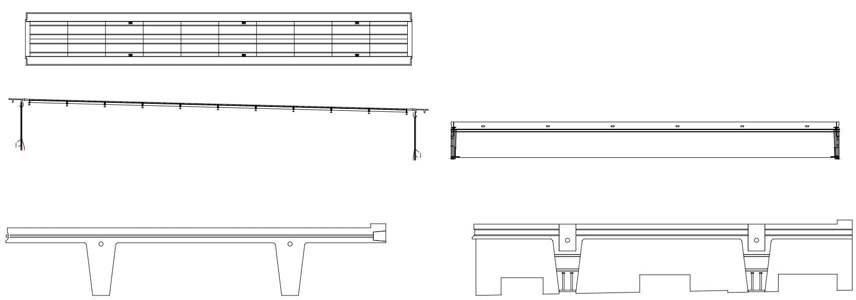

Figure 12: **Through Truss Deck Panel**

The panels were pre-tensioned in the ribs to carry the positive moment demand between the floor beams. The ten panels along the length of the 98.9 m span through truss were post-tensioned longitudinally to reinforce the negative moment over the floor beams (panel supports). The longitudinal post-tension was made straight to reduce all the possible friction losses in the 10 panels. Two panels were used side by side, for the 9.0 meter through truss roadway width. The two panels were post-tensioned transversally.

8. The Integral System, East Approach Spans

The east approach consists of two series of 3-continuous spans separated by a finger plate expansion joint, as shown in Figure 13. A single panel per span and three panels per cross section were used. A total of 18 panels with dimensions of 18 m by 3.22 m each were used.

Each panel is a precast concrete double tee oriented such that the ribs spans between the piers. Each double tee was post-tensioned with 9- 15.2 mm strands in the ribs at the precast yard. When the panels installed on the piers, another level of pos-tensioning duct was applied to supplement the first post-tensioning stage as well as reinforce the negative moment section over the piers.

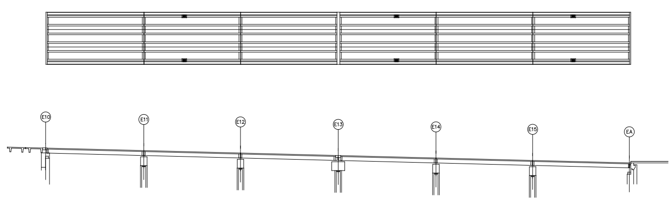

Figure 13: East Approach Deck Panel Design

The three panels in the cross section were post-tensioning transversally with flat plastic ducts in the 180 mm thick slab as shown in Figure 14.

Figure 14: East Approach Deck Panel

9. Deck Panels Fabrication And Construction

The fabrication of the deck panels was planned in a very efficient way. All the deck panels are double tees. The total numbers of panels fabricated was closed to 200 panels.

The forms of the deck panels can be divided into two groups. The first group is the majority of the panels, 173 panels. Of those, 153 panels were intended for the deck truss and the plate girders. The remaining 20 panels were for the through truss.

The 173 panels all had the same rib spacing (2.25 m), and cross section. The panel foot print is 4.5 x 9.66 m. Only one steel form geometry was used for the 173 panels.

The 153 Deck Truss and Plate Girder panels had a 2% crown. The ribs and the slab at the mid length of the rib were crowned to match the bridge cross section. Only one form geometry was designed to form the typical panel, the variable width panel and the span end panel. A concrete block out was used in the panel for the deck drains.

The distance between the two truss lines or girder lines in the whole bridge was 6.4 m and the overall bridge width is 9.66 m. Therefore, the 1.63 m overhang that was initially designed to have pre-stressing in the top of the ribs needed to be efficiently designed and detailed. This would have required two strand hold down devices as well as two strand hold up devices. This solution was investigated and found to be achievable but costly. Only the two hold down devices were used and the strands were sloped to reinforce the overhang in order to carry negative moment

The post-tensioning in this system was most innovative and was unprecedented to the knowledge of the author. The post-tensioning ducts were straight and longitudinal to traffic, crossing only the ribs. The top of the ducts was the soffit of the 180 mm slab. This eliminated all possible duct misalignment and tremendously reduced the friction losses.

The panel reinforcement details are shown in Figure 15.

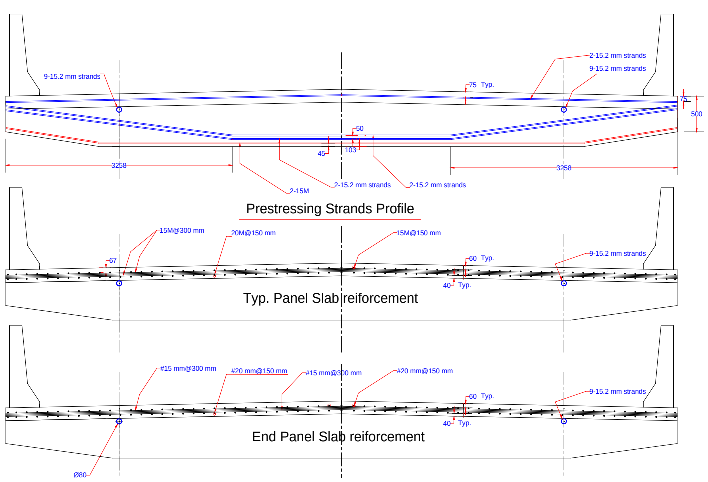

Figure 15: Deck Truss and Plate Girder Panel Details

The 20 Through Truss double tee panels were oriented in the longitudinal direction which necessitated reversing the crown direction in the panels. The same steel forms were utilized again. However the crown form was reversed using wood forms in the 180 mm slab. Reinforcement details are shown on Figure 16.

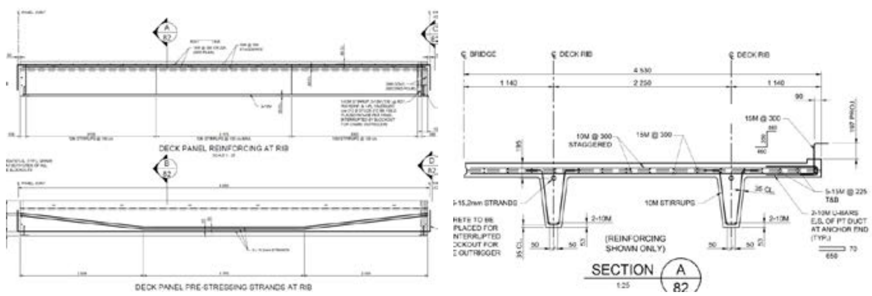

Figure 16: **Through Truss Panel Details**

The rest of the 18 panels which was the second group were used in the east approach. These panels were formed using wood forms. Rib spacing was 1.6 m and the total width was 3.2 m. The panel length was 18 m to span between piers. The panel reinforcement is shown in Figure 17.

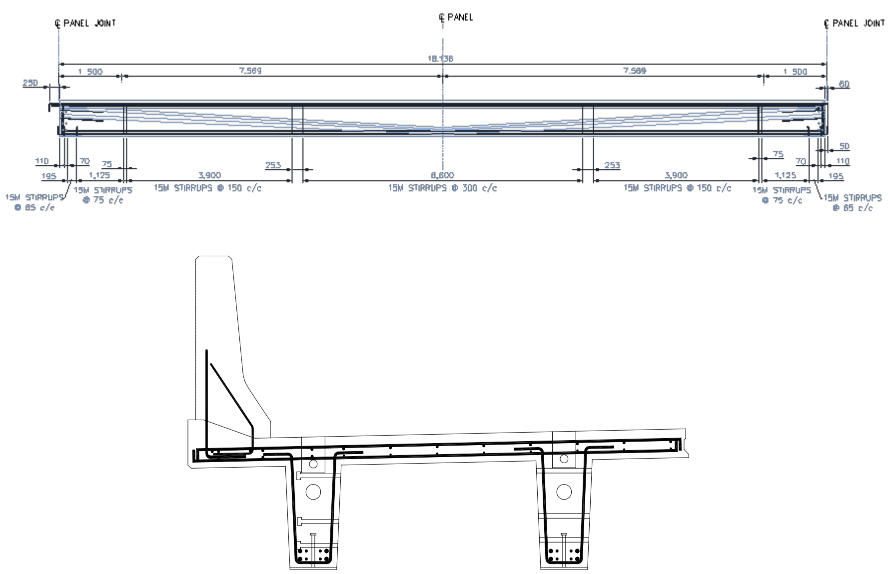

Figure 17: **East Approach Panel Details**

For the Deck Truss, Plate Girder and Through Truss panels, the plan was to install the panels using 110 Ton Crane. The panels were only installed using this Crane for one span.

After that, SNC-Lavalin came up with a very unique light instrument to install the panels much faster and without the cost of strengthening the top chord, as would have been necessary to carry the temporary 110 Ton crane loads. The East approach panels were installed using a 200 Ton crane from the ground. Two storage yards were provided by NBDTI near the each end of the bridge site to stockpile the precast panels, as shown in Figure 18.

Figure 18: **Precast panels stockpiled in storage yard**

10. Piers and abutments repair

Most of the bridge substructure components were showing signs of severe deterioration, which took the form of scaling, map cracking, spalling, and delamination of the concrete surface, and corrosion of the embedded steel reinforcement. The deterioration was the result of the combined action of alkali-silica reaction (ASR), cyclic freezing and thawing, and the chloride-induced corrosion of steel. The encapsulation method was chosen to repair the piers and abutments. This method entailed the installation of a stainless steel reinforced concrete jacket around and over the structure. This encapsulation tasks included the following:

- Remove concrete to a minimum of 250 mm from the original profile of pier, with all reinforcement, as shown in Figure 19.
- Install a reinforced concrete jacket around and over the pier to completely encapsulate the exposed surface, and using stainless steel for corrosion resistance, as shown in Figure 20.

Figure 19: **Concrete removal**

Figure 20: **Installation of stainless steel reinforcement**

11. Scope of Work Summary

A breakdown for the scope of work is shown on Figure 21.

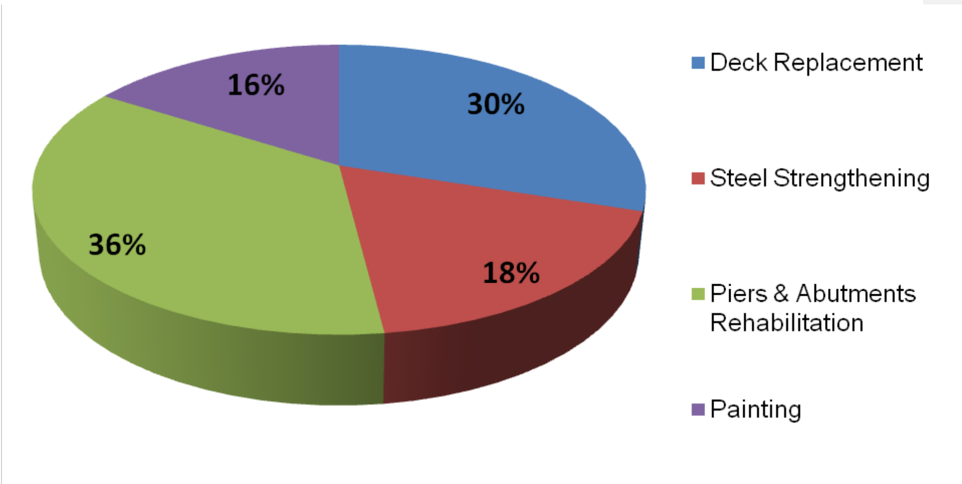

Figure 21: **Scope of work**

The quantities that have been used for the Princess Margaret Bridge rehabilitation work are show in Table 1.

Bridge Deck Prefabrication	
Deck truss and Plate Girder Spans	199 panels
Through Truss Span	20 panels
Approach Spans	18 panels
Material used for Precast Panels	
High Performance Concrete	2 900 m^3
Reinforcing steel	645 000 kg
Pre-stressing strands	45 000 m
Material used for structural steel and substructure	
Steel strengthening and repair	260 000 kg
Stainless reinforcing steel	383 000 kg
Conventional reinforcing steel	111 000 kg
Concrete for pier encapsulation	2 800 m^3
Bearing Replacement	22 units

Table 1: **Material Quantities**

12. Conclusion

This design build project has been innovatively designed and constructed for the Province of New Brunswick to ensure that the Princess Margaret Bridge will have the required live load carrying capacity as well as service life for the next 50 years. Total Completion is scheduled at the end of the 2012 construction season.

Based on the experience gained from this project, the following conclusions can be drawn:

- The use of precast deck panels that are composite with the trusses proved to be a very efficient deck replacement strategy, significant reducing the requirement for steel strengthening.
- The innovative system of post-tensioning the precast panels parallel to the direction of traffic, using the spine beam concept, eliminated all duct coupling and possible misalignment.
- The unique light equipment to install the deck panels was instrumental in achieving the required productivity which allowed the project to be completed within the two-year allotted timeframe.

Acknowledgments

The New Brunswick Department of Transportation & Infrastructure wish to acknowledge the efforts of Eastern Designers and Gemtec Consulting Engineers for their respective roles in the completion of the original structural condition assessment and substructure investigation for the Princess Margaret Bridge, as well as for their continued efforts as technical advisors through the project development, procurement and Design-Build phases. NBDTI would also like to acknowledge “exp” for their role in the development of the traffic management plan and associated mitigation strategies. The author would also like to express appreciation for the contributions of the entire Project Team at Partnerships NB and for the support and guidance that they received from the management team at the New Brunswick Department of Transportation & Infrastructure.

SNC-Lavalin wish to express their appreciation to the effort and guidance of Dr. Maher K. Tadros for his valuable contribution to the success of the project. The authors also wish to acknowledge the contributions and collaboration of the numerous firms and individuals who contributed to the success of the project and in particular the engineering team and construction project managers of SNC-Lavalin Inc.

Both the New Brunswick Department of Transportation & Infrastructure and SNC-Lavalin would like to thank the City of Fredericton and its residents for their continued support and cooperation in advance of and throughout the construction of the Project.

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