KICHI SIPI BRIDGE TO CROSS LAKE

INNOVATIONS IN DEEP FOUNDATION DESIGN AND CONSTRUCTION

Eric B. Loewen, P. Eng.,
of Earth Tech (Canada) Inc.;

James S. Betke, P. Eng.,
of Earth Tech (Canada) Inc.;

Fred Kemp, P. Eng.,
of Earth Tech (Canada) Inc.;

Rados Eric, P. Eng.,
of Earth Tech (Canada) Inc.;

and Ruth J. Eden, P. Eng., M.Sc.,
of Manitoba Transportation & Government Services

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ABSTRACT

The Kichi Sipi Bridge, as part of Manitoba's Northern Development Strategy, represents an investment in both transportation infrastructure in the region and in the well-being of the Pimicikamak Cree Nation and Cross Lake communities. The Kichi Sipi Bridge is a 260 metre long, four-span structure with an innovative concrete foundation system supporting steel plate girders and a composite concrete deck. The Kichi Sipi, which means "Great River" in Cree, replaces temporary, seasonal ferry service and a winter ice road across the Nelson River on PR 374 to provide year-round access to the remote northern Manitoba community. The project provided many challenges, including the remote location, high cost of construction, and degree of difficulty of the site. Water depths reach up to 20 metres, the bedrock is a very hard basalt/granite with sudden elevation changes, and the ice is up to 1 metre thick with a high crushing strength. These factors result in a tremendous load on the tall piers, and a conventionally constructed foundation system would have added enormous cost to the project.

A unique foundation system was developed to minimize the effect of the ice loads. The foundation system also reduced risk during construction and the initial capital cost of the project. Each pier consists of a group of six relatively small diameter battered rock-socketed concrete caissons that extend into the bedrock, and are tied together with a submerged pile cap. The pile cap supports a narrow, sloped pier shaft, which results in minimal ice contact area with the piers and greatly reduced loads. The system provides stable foundations, reduced time and costs for drilling smaller caissons, and reduced concrete quantities and costs. In addition, the system provided redundancy and enhanced constructability.

The foundation system required innovative construction techniques that were investigated during the design phase to ensure constructability. This included the use of a steel-tub form, which acted as both the caisson template and the submerged cofferdam. Cost and schedule dictated that coring the caissons for quality assurance was impractical. As a result Crosshole Sonic Logging (CSL), a non-destructive testing method, was specified. During construction this method identified a major defect near the bottom of one caisson. A repair method was developed to adequately restore the caisson capacity and to allow the repair to proceed while construction of the substructure and placement of the girders continued above.

Significant knowledge was gained on the Kichi Sipi Bridge project through the success of several design and construction innovations, as well as through the remediation of the defective caisson. Recommendations and lessons learned include the following: (1.) Redundancy for high risk elements is critical. With the innovative foundation design developed, redundancy reduced risk and allowed for the defective caisson to be remediated without delaying the project schedule. (2.) Submission of specific tremie concrete procedures by the Contractor should be made a requirement, even if detailed specifications are provided by the Owner. (3.) On this project the only locally available aggregate source was the rock blasted and crushed for the project, however it would have been beneficial to import smooth aggregate specifically for the caisson concrete to provide a more pumpable mix. (4.) CSL testing was very effective in identifying a major caisson defect and provided sufficient information to allow acceptance of the final repair. (5.) The use of hydrodemolition to remove poor concrete in a submerged condition was ineffective on this project. (6.) Grouting the defective concrete with preplaced aggregate in a tremie condition proved to be highly successful as evidenced by above-ground trials, coring and final CSL testing. In conclusion, despite the many challenges encountered, this project was completed successfully, within the project budget, and the bridge was opened ahead of schedule.

PROJECT BACKGROUND

Introduction

The Kichi Sipi Bridge is a 260 metre long bridge over the Nelson River in northern Manitoba. The owner of the bridge is Manitoba Transportation and Government Services (MTGS), the prime consultant was Earth Tech (Canada) Inc., and the General Contractor was Kiewit Management Co. Ltd. The bridge was constructed in 2003/2004 to provide year-round access to the community of Cross Lake, replacing a seasonal combination of a ferry crossing and ice bridge. The new bridge is a four-span structure with an innovative concrete foundation system supporting steel plate girders and a composite concrete deck. This complex project provided many challenges, including the remote location, increased cost of construction, deep water, and hard basaltic bedrock.

The conceptual design phase included investigation of several crossing locations over the Nelson River, as well as a route location study to ensure cost effective access to the community of Cross Lake. The recommended solution included construction of the 260 metre long bridge, located approximately 1000 metres downstream from the existing ferry crossing, as well as construction of approximately six kilometres of new roadway for relocation of highway PR 374.

Selection of the final design solution involved engineering evaluations and risk analysis of constructability in a remote location, future maintenance considerations, environmental impacts, transportation economics, and expectations and future visions for the Cross Lake community. Tasks included a detailed geotechnical investigation of the site to establish foundation design criteria and identify construction concerns; a hydrologic and hydraulic assessment of the river to identify water levels and design loads for the structure; and a preliminary design which investigated many bridge and foundation types to produce the most suitable final design concept.

Geotechnical Considerations

The geotechnical engineer for the project, Dyregrov Consultants, was retained to conduct a detailed geotechnical investigation of the site and to develop design criteria for the foundations. Several key issues were identified through the geotechnical investigation which included:

- The bedrock was a very hard basalt/granite (moderately to strongly siliceous mafic metavolcanic rock). The very hard bedrock would result in challenging drilling conditions.

- Channel bottom conditions varied from virtually no overburden at a water depth of 20 metres, to several metres of bouldery overburden at some proposed pier locations, to 18 metres of very soft silty overburden at other locations. This would provide challenges in selecting a single, simple foundation type to meet the varied conditions.

- The uneven bedrock elevation varied by up to three metres within the width of a pier footprint. This would further complicate construction, potentially create constructability issues, and increase risks during foundation construction.

The geotechnical recommendations included the following:

- The most appropriate foundations were assessed to be cast-in-place rock-socketed concrete caissons.

- Special techniques would be required to properly seat the steel caisson sleeves due to the thin, bouldery overburden and uneven bedrock surface. Sealing the caisson sleeves would be extremely difficult and tremie concrete pours would likely be required for caisson installation.

- Battering the caissons could help to accommodate the lateral loads on the piers with batters not to be greater than one horizontal to six vertical.

Hydrologic and Hydraulic Considerations

The hydrologic and hydraulic investigation for the project identified several key issues which had a significant impact on the design of the bridge. These included the following:

- As a result of the northern location and local conditions, the crushing strength of the ice for the extreme event ice load was determined to be 1.0 MPa, with a corresponding ice thickness of 1.0 metre. This results in a tremendous overturning moment on the tall bridge piers.

- Due to control structures on adjacent channels of the Nelson River, the fluctuation in water levels at the site was not very large, typically limited to 1.5 metres in a total water depth of up to 20 metres. As a result there was no seasonal period of relatively shallow water depth that would have otherwise facilitated construction.

- Despite relatively low flow velocities of 0.5 m/s, the significant water depth results in a significant cumulative effect acting on the construction works.

Foundation Design Considerations

The remote location, deep water, hard basaltic bedrock, variable overburden, and hydraulic conditions at the site, as well as the high cost of construction in northern Manitoba resulted in a significant challenge for design of the bridge foundations. An innovative design was required to reduce risks associated with constructability and safety during the construction of the piers, minimize the ice loads on the piers, minimize the cost of construction, and transfer loads to the bedrock in the most efficient manner. Constructability was affected due to the depth of water, difficult bedrock conditions, and variable channel bottom characteristics. High construction costs were driven in a large part by the very high cost of concrete in this region. As a result the volume of concrete required for any pier option had a significant impact. With a high estimated cost per pier, it was also understood that minimizing the number of piers would be significant in reducing the overall bridge costs. As the foundation design progressed, it became apparent that the foundation system also needed to minimize the ice load in order to achieve a reasonable level of efficiency and cost.

DESIGN

Preliminary Design

Several bridge types were investigated in the preliminary design phase. These included options with precast concrete NU girders, steel box and I-Girders, a tied arch structure, a precast balanced cantilever option, and various span lengths, pier types and pier locations. Pier types included: traditional, and relatively massive, cast-in-place piers; precast pier segments post-tensioned to the bedrock; and large diameter caissons with a pier cap to support the girders. A traditional pier type with a massive concrete foundation and pier was found to be extremely costly due to the high concrete volume, and difficult to construct due to the water depths, difficulty of installing cofferdams, and inherent safety issues of such deep cofferdams.

An innovative option with precast pier segments post-tensioned to the bedrock had significant merit; however the water depth, variable bedrock elevation, and requirement for a level bearing area to place the first segment made this option impractical to construct.

Several options utilizing large diameter caissons, up to 1.8 metres in diameter, were investigated. Drilling large diameter shafts straight into the bedrock, and extending them up through the water to support a pier cap would be a very practical and straight-forward pier option in many situations. Several site conditions however made this option undesirable. Firstly, the varying bedrock elevation would make seating a large diameter sleeve prior to drilling problematic and difficult to achieve. Secondly, the cost of large diameter custom drill bits was prohibitive, and the risk and impact of breakdowns were significant for custom equipment utilized in a remote location with a short construction season. It should be noted that the very hard bedrock eliminated the option of coring the bedrock. Thirdly, a foundation system relying on two or three large diameter caissons, which would act as relatively flexible members under the extreme loading conditions, were not seen as having an acceptable level of redundancy. Finally, and of the greatest significance, the large diameter caissons had the effect of attracting even more ice load than more traditional, narrow pier cross sections. Increased cross-sections to resist the high ice load resulted in even greater ice load, producing a design with “diminishing returns”, and a relatively inefficient structure.

Detailed Design

The selected bridge design consisted of both steel and precast concrete girder options for a four-span, continuous structure with spans of 59-71-71-59 metres, which effectively reduced and optimized the number of river piers. The steel girder option was ultimately constructed, and the Kichi Sipi Bridge is now the second largest bridge in Manitoba in terms of span lengths and girder depths. The water depth of up to 20 metres is approximately three times that of most major river crossings in Canada.

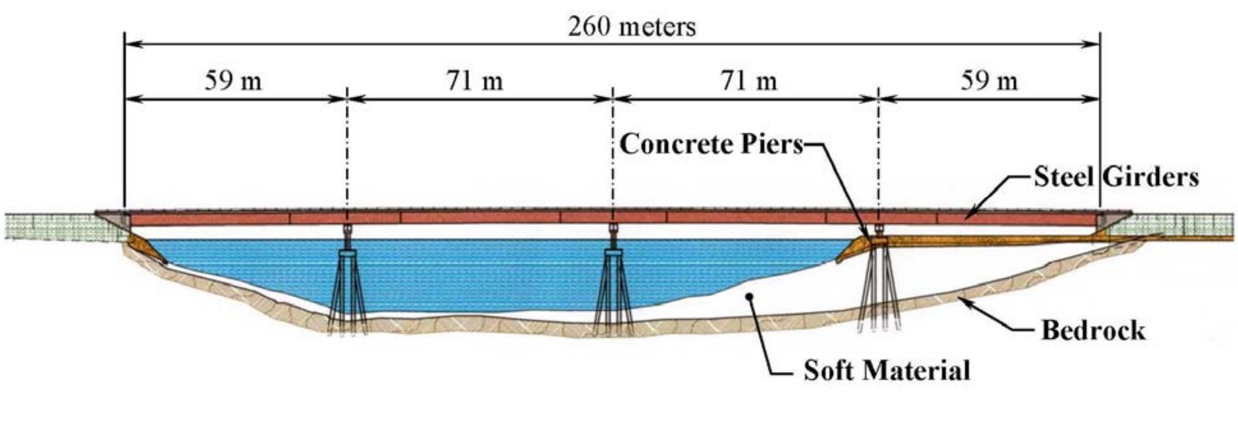

Figure 1: General elevation of the Kichi Sipi Bridge to Cross Lake.

Earth Tech developed a unique foundation system to minimize the effects of the ice, minimize risk and cost during construction, improve constructability, and provide the necessary stability for the structure. Each pier consists of six 900 mm diameter battered rock-socketed caissons that extend into the bedrock and are tied together with a submerged pile cap. The submerged pile cap supports a narrow, sloped pier shaft at each pier location. The pier shaft extends out of the water to support the superstructure above. This results in a minimal ice contact area with the pier, a wide stable foundation with minimal concrete volume and cost, and reduced risk, time and cost for drilling large diameter caissons. The pier and foundation system were modeled and analyzed using STAAD-Pro to determine stresses and deflections for detailed design of the pier components.

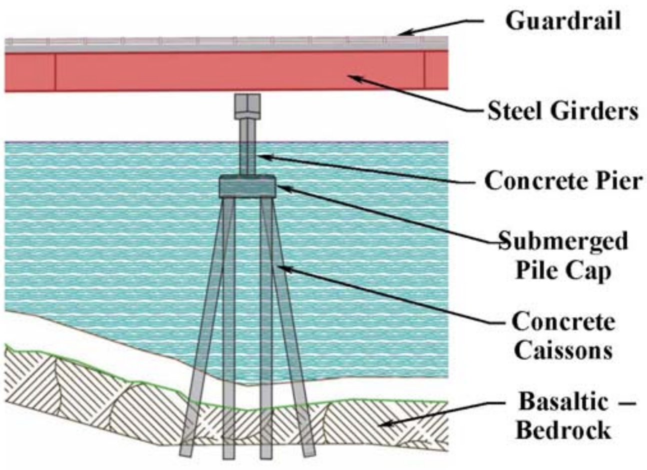

Figure 2: Upstream view of a typical river pier with innovative deep foundation system.

The six caisson design also provides a redundant system, as the structure would not collapse should any one of six caissons fail at an individual pier. This redundancy is critical in part because the caissons are relatively long and slender, and do not have the same characteristics inherent with a typical massive pier foundation. This system also provides flexibility during construction, as specific bedrock conditions at a single caisson location could require slight relocation of the caisson at the bedrock elevation, without a significant effect on the entire foundation. (As a comparison, relocating a large diameter caisson could have a major impact on construction of the pier, and adverse effects on the overall project cost and schedule.)

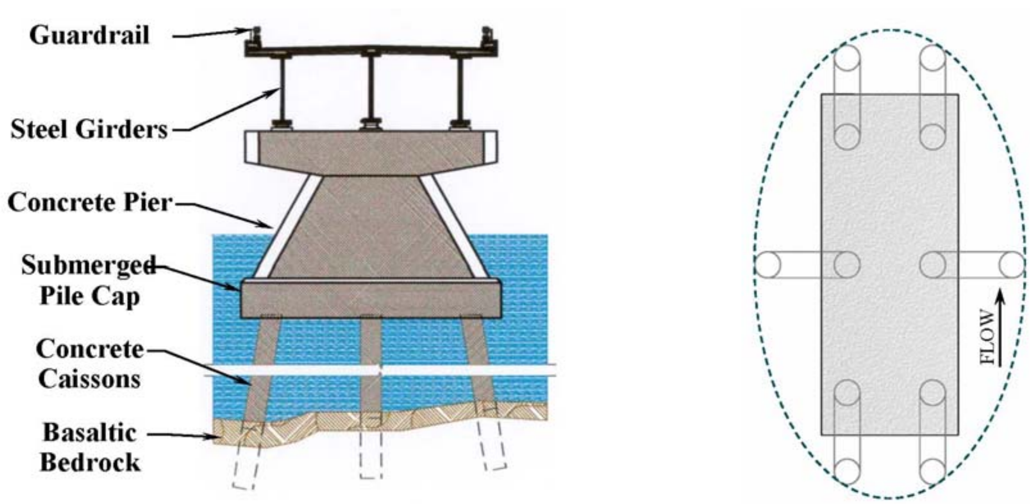

Figure 3: Pier geometry, underwater pile cap and caisson layout for a typical river pier.

The battered caissons provide lateral and longitudinal load capacity similar to battered piles on typical bridge foundations. The caissons were designed with fixed moment connections between the top of the caissons and the pile cap; and the rock-socketed portion provides fixity to transfer bending into the bedrock. As a result the system also acts as a moment frame, which further increases lateral load capacity and stiffness.

The narrow, sloped pier shaft extending up out of the water provides a relatively small ice contact area, and based on Earth Tech's detailed analysis the resulting ice load on the substructure was reduced by approximately 50% compared to a pier with larger diameter caissons extending out of the water. In addition, the selected geometry allowed the pier nosing to be sloped 30° to promote ice break-up, which further reduced ice loads by 30%. The resulting pier shape above the water appears to be quite traditional, and gives little indication of the unique system below the water level.

Photos 1 & 2: View of typical river piers illustrating the narrow, sloped pier shaft.

Due to the relatively constant water depth throughout the year the Contractor could not take advantage of any seasonal low water levels to aid construction. However, the minimal water level fluctuation was utilized in the foundation design, as the submerged pile cap is located safely below the ice level, which also provides the necessary clearance for seasonal boat traffic. A final benefit of this foundation system was that the need for a cofferdam in 20 metres of water was eliminated, and approximately only a five metre deep cofferdam was required to construct the submerged pile cap.

CONSTRUCTION

River Pier Foundations

The river piers were constructed from barges and the Contractor utilized a steel tub to act as both a template for caisson installations and as a form for the underwater pile cap. The steel tub also acted as a temporary cofferdam that could be dewatered, thus enabling the Contractor to construct the submerged pile caps and pier shafts in the dry. The steel tubs were later successfully removed.

Photos 3 & 4: River pier foundations in the Nelson River constructed from barges utilizing a steel tub form.

The submerged pile cap was designed with the intent of using a steel tub form that could be dewatered, though the details were the responsibility of the Contractor. The use of the tub form as a template to place the caissons was an innovation on the part of the Contractor. This verified that constructability issues of the submerged pile cap had been properly addressed during design, yet the specifications and drawings allowed the Contractor enough freedom to develop his own innovative techniques.

Photos 5 & 6: The steel tub form acted as both a template for the caisson installations, as well as a temporary cofferdam, which allowed construction of the underwater pile cap to be carried out in the dry.

Drilling Equipment

The Contractor encountered significant difficulty in attempting to seat and seal the steel pipe sleeves for the caissons in the very hard bedrock. Drilling for the 900 mm diameter rock sockets was initially attempted with a tricone rotary drill. The tricone drill was found to be ineffective as the poor seal allowed infiltration of the river bottom till material which would plug the drill head and inhibit removal of rock cuttings. The Contractor then attempted to advance the steel sleeve into the bedrock to achieve a better seal by means of chipping the rock with a gravity drop steel churn. Production was slow, however caissons for the two river piers were constructed using the combination of the steel churn and a tricone drill to seat the sleeves and drill the rock sockets.

An alternate method of caisson installation was undertaken for the third pier. This method utilized a coring sleeve to provide a seal at the bedrock interface. The rock socket was then drilled utilizing both a down-the-hole hammer and the rotating tricone method. This proved to be the most cost-effective method for the caisson installation.

Caisson Tremie Pour

The Contractor utilized a concrete pump and a series of solid and flexible pipe sections for tremie pouring the concrete caissons. Tremie procedures included using a sponge plug or "pig" followed by approximately 1 m^3 of grout to lubricate the lines and to prevent loss of cementitious material on the interior of the pipe. The intent was that the pig would prevent segregation, and the cement rich grout would facilitate placement of the concrete, and would be displaced by quality tremie concrete, which followed behind. Typically this procedure worked well, however line plugs were encountered and cleared often throughout the project.

Photos 7 & 8: View of pumping and tremie placement operations for the river pier caissons.

Quality Assurance

Rock sockets for each of the caissons extended a minimum of three metres into sound bedrock to provide bottom fixity of the caisson and to allow for inconsistencies in rock quality. Quality assurance testing following the caisson tremie pours consisted of ultrasonic testing by the Crosshole Sonic Logging (CSL) method. Testing of each caisson was considered critical due to the nature of the foundation design. This was in part due to the relatively tall and slender caisson section, laterally unsupported, extending up to 16 metres through the water, as well as the fact that the caissons are likely to never be inspected in the future due to the remote bridge location, zero visibility in the water, significant water depth, and exterior steel sleeves. The CSL testing method was selected by Earth Tech in order to reduce the high cost and lengthy duration associated with full depth coring. Testing of all six caissons at each pier location was carried out by the CSL test method in accordance with ASTM D6760-02.

CSL testing is based on the theory that sound waves travel at different speeds through different media. Four steel tubes were cast into each caisson, as illustrated in the following photo and figure (Photo 9 and Figure 4). Variations in the apparent wave speeds between tube pairings identify potential anomalies in the caisson, or lower density concrete, which may be due to poor quality or excessively voided concrete.

Robert Miner Dynamic Testing Inc. (RMDT) conducted the CSL testing for the project. At the time of testing, the age of the concrete in the caisson was typically three to four days old. CSL measurements were logged at approximately 0.03 m intervals as the “transmitter” and “receiver” probes were simultaneously pulled upwards from the bottom of the access tubes. Individual scans were identified by the tube pairs, as shown in Figure 4, with a total of six possible tube pairings in each caisson, which provided two diagonal and four perimeter scans between the four CSL tubes.

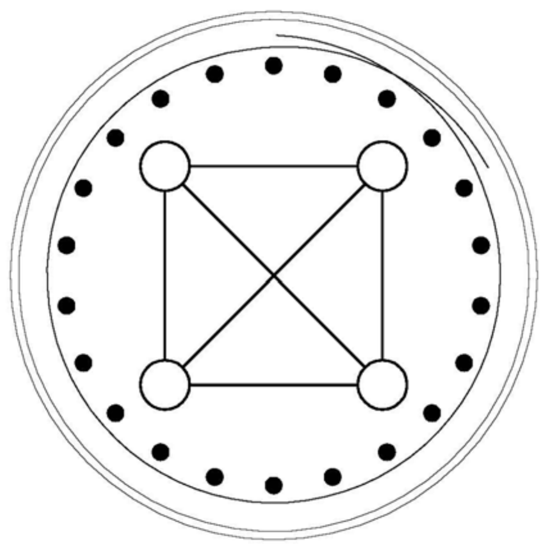

Photo 9 & Figure 4: Typical view of the caisson reinforcing cage illustrating the steel tubes for Crosshole Sonic Log (CSL) testing. Note the conduit layout facilitated two diagonal and four perimeter scans between the four tubes.

This was the first use of CSL testing by the Province of Manitoba and the speed, accuracy, and cost-effectiveness of the technology was proven on this project. CSL testing confirmed an acceptable quality at the first seventeen caissons at the three pier locations. However, testing of the eighteenth and final caisson indicated poor quality concrete in a significant portion of the lower section of the caisson. The following section describes the investigation, design, and remediation of the resulting caisson repair.

CAISSON REPAIR

Pour Observations

The final caisson of eighteen on the project had a line plug early in the pour. It is believed that the tremie pipe may not have been fully re-inserted into the already placed tremie concrete following clearing of the line plug. During the time that the Contractor encountered difficulties placing the tremie concrete in the caisson in question, it was observed that the water in the caisson was becoming white and foamy, a potential indicator that some of the admixtures had washed out of the concrete. The attached photo documents this observation; however the nature of the deficiencies in the concrete was not known until after the Crosshole Sonic Log testing was completed.

Photos 10 & 11: Close-up view of tremie placement at an individual caisson. Note the milky colour and foam in the water of the caisson that later required remediation.

Crosshole Sonic Logging Test Results

Results from all six tube pairings tested in the caisson identified two potential anomalies. The first anomaly was evident between approximately 16.1 and 17.3 metres depth and appeared to be a zone of somewhat softer (low density) material. The second anomaly was evident between approximately 18.2 and 21.5 metres depth and appeared to be a zone of very soft (very low density) or voided material. The remainder of the concrete above 16.1 metres appeared to be consistent with sound and uniform conditions, similar to all previous caissons. A sketch illustrating the defective zones is provided in Figure 5.

The anomalies were later verified by a full-depth 75 mm diameter core, and the core results verified that the bottom 6.83 metres of caisson concrete was of questionable quality, including, in descending order, a zone of primarily grout with a trace of coarse aggregate, a zone of voided and honeycombed or “popcorn” concrete, and a zone where no core recovery was possible. This lowest zone was assumed to be a loosely bound coarse aggregate matrix. The average compressive strength of samples taken in the upper grout zone was approximately 7 MPa.

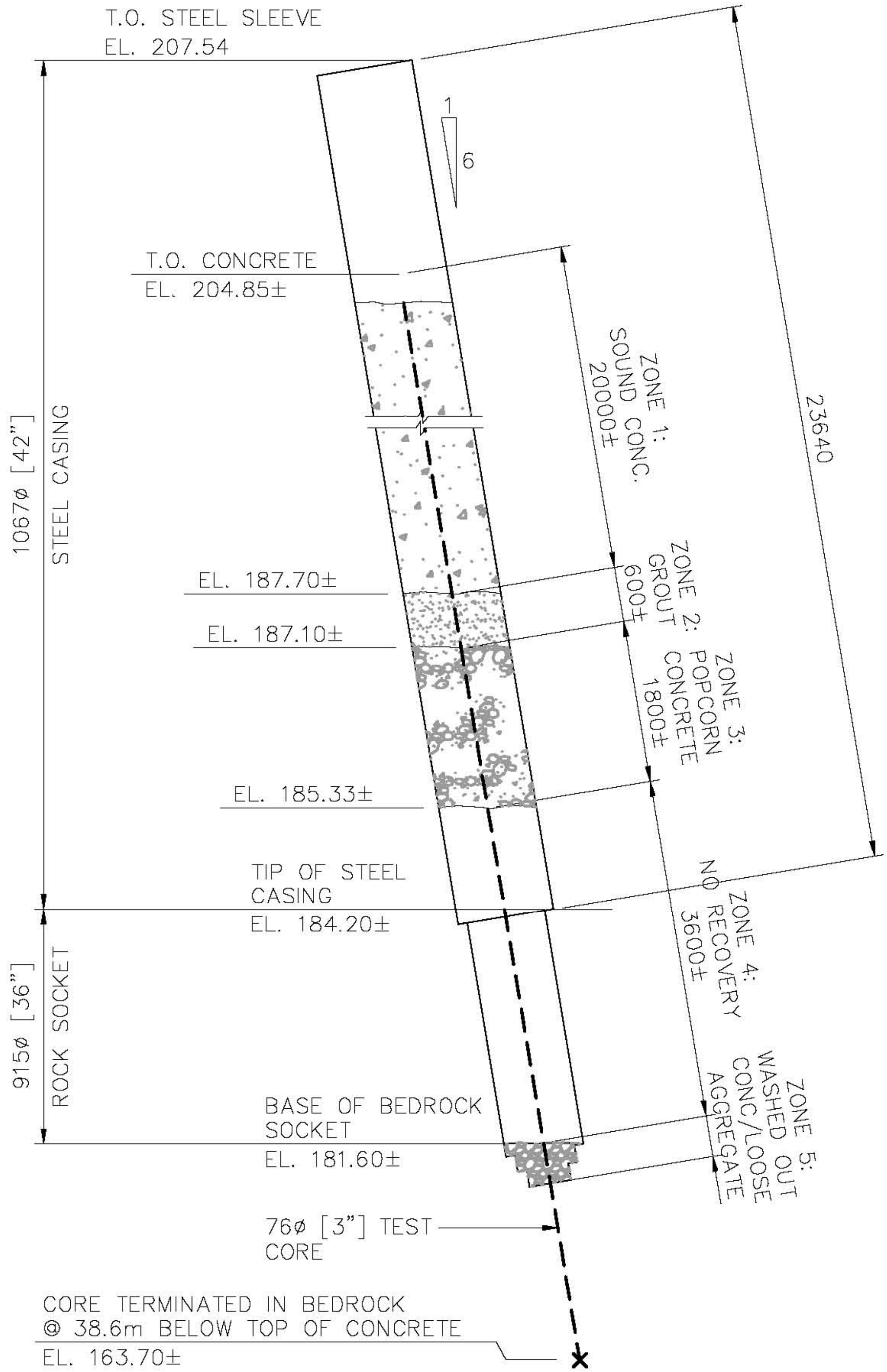

Figure 5: Sketch showing the defective concrete zones as identified by the CSL testing and verified by coring.

Remediation Design

Based on the test results, it was assumed that the caisson fixity in the rock socket was lost and that a substantial portion of the caisson was incapable of transferring vertical, shear, and bending loads to the underlying bedrock. Several 150 mm diameter full depth cores were taken to allow access to the bottom of the caisson. Remote cameras were lowered to determine the nature of the deficiency, which confirmed the presence of major voids and material defects in the concrete.

Several repair and replacement options were discussed, and a decision was made to hydrodemolish and airlift the questionable concrete in the bottom 6.8 metres of the caisson, add pre-placed aggregate, and pressure grout the repair zone from the bottom up to fill the voids in the aggregate with a cement/sand paste. Pre-placed aggregate was to be backwashed prior to pressure grouting to facilitate bond. This method had never been attempted before in Manitoba and raised a number of concerns that were noted and discussed prior to initiating the repair.

Concerns included that hydrodemolition would not be successful in cleaning the surface of the rock socket behind the reinforcing cage, and thus the concrete caisson would be unable to engage this surface in developing the skin friction required to re-establish fixity and vertical load transfer. In addition there were concerns that the airlift would not be able to completely remove all of the debris from the repair zone. Finally, there was a concern that the very bottom of the caisson below the end of the CSL tubes could not be effectively repaired and/or have the repair verified to provide end bearing.

To address these concerns three mini-piles were designed and installed at the base of the caisson to transfer vertical and lateral load into the bedrock below. Since the skin friction between the concrete caisson and the rock socket and end bearing could not be relied on, the vertical load capacity would be provided by extending three mini-piles 5, 10, and 15 metres into the bedrock beyond the bottom of the existing rock socket. Since re-establishing fixity could not be verified, the resulting caisson was re-analyzed using the STAAD model utilized for detailed design assuming a pinned connection at the bedrock level. As noted earlier, the lateral ice loads on the piers are resisted by a combination of two methods: through the battered configuration of the caissons; and by the caissons and pile cap acting as a moment frame due to the fixed connections at the top and bottom of the caissons. The component resisted by the fixity of the top and bottom of the six caissons was thus based on twelve fixed connections, of which one was now assumed to be ineffective. The result of assuming one of these connections as pinned was found not to be significant as verified by the subsequent analysis.

The original foundation system was governed by lateral deflection due to extreme ice loads. Provided that the vertical capacity and lateral shear capacity could be established, it was determined that this single pinned connection would be considered acceptable and it did not significantly increase lateral deflections.

Pile Cap Modifications

Due to the tight construction schedule and continued remediation of the caisson, the Contractor requested permission to proceed with construction of the submerged pile cap and the pier, prior to completion of the remediation works. During design the caissons had been located outside the footprint of the pier shaft above to improve constructability (reduce congestion of reinforcing) but within half the distance of the pile cap depth to eliminate the demand for shear reinforcing. (A nominal amount of shear reinforcing was still included in the pile cap.) As a result, the caisson steel sleeve could be left intact, protruding up through the water and future ice level to maintain access to the caisson, while constructing the pile cap around this caisson. Also, in order to proceed with this work, pile cap modifications were required to allow access to the caisson, yet ensure transfer of loads from the pile cap to the caisson in the final configuration. Nelson studs were welded to both the exterior, and later the interior of the steel caisson sleeve to allow load transfer from the pile cap concrete through the steel sleeve into the caisson concrete.

Photos 12 and 13: (Left) View of the steel Nelson studs welded onto the exterior of the sleeve as part of the modifications to provide fixity between the caisson and the pile cap. (Right) View of heated enclosure to facilitate coring, hydrodemolition and grouting operations, as well as dywidag installations off the ice in cold weather.

These modifications not only allowed for ongoing caisson remediation during construction of the remainder of the pier, but also allowed for girder erection and placement of the precast deck panels to continue. These pile cap modifications during the caisson repair allowed the Contractor to maintain a tight project schedule and allowed for girder erection to proceed from the ice bridge as planned.

Coring

Caisson remediation began with installation of 122 mm diameter core holes to provide access to the repair area. Full depth core holes at various stages of the repair process also facilitated hydrodemolition of the defective concrete zone, placement of pre-placed aggregate and grout, and installation of the dywidags. The coring process was quite problematic. Due to the batter of the caissons and the reduced diameter of the rock socket, the core holes for the mini-piles converged, thus increasing the level of difficulty for installing the three dywidag anchors. Steel casing and other coring components were either left or lost in the caisson during coring, which compounded the problem even further. Core holes were also misaligned and hit either CSL tubes or rebar at various depths and therefore had to be abandoned. Core holes also converged and the steel casing from previous cores was damaged resulting in the loss of one or both holes in some cases.

Hydrodemolition

Hydrodemolition was chosen by the Contractor as the preferred method to break-up and remove the defective concrete in the bottom six metres of the caisson. A 300 hp triplex water pump and high-pressure line to operate a self-rotating, swivel nozzle was used, which was fed down a core hole to the defective zone of concrete. The nozzle included 4-6 high-pressure tips at various angles that run at a specified theoretical pressure of 10,000 psi (70 MPa) each and a corresponding flow capacity of 50 gallons per minute. The intent was that the water jet nozzle would break down all loose or incompetent material within the defective zone. An airlift would then be inserted into the hole and all loose material removed. The discharge from the airlift was to be collected in order to quantify the effectiveness of the hydrodemolition process. This process was to be repeated as required until a cavity down the length of the defective zone in the pile was evident and the caisson was ready for placement of aggregate and pressure grouting. However, the following difficulties were encountered during the hydrodemolition process that reduced its overall effectiveness:

- The hydrodemolition operations were performed under water, which appeared to significantly reduce the effectiveness of the water jet nozzle. Even higher-pressure nozzles up to 15,000 psi (105 MPa) did not appear to be effective.

- Use of the airlift to "aerate" the water during hydrodemolition did not improve the effectiveness of the water jets.

- Airlifting operations removed only a minimal quantity of coarse aggregate from the caisson.

In general, hydrodemolition and airlifting operations were unable to break up and remove a significant portion of the defective concrete in the bottom 6.5 metres of the caisson. It appeared that the hydrodemolition was only effective at removing some loose sand and cement, as subsequent video inspections indicated a relatively "clean" defective area, free of loose material.

It is important to note that a water jet pipe was advanced to near the bottom of the zone of no recovery; however the video camera could not be advanced into this zone. This indicates that this zone at least partially consisted of loose aggregate which could be displaced with the pressure of the waterjet, but could not be removed by air-lifting operations. This was important in understanding the nature of the material in the lowest zone.

<u>Video Inspections</u>

From the onset of the repair, video inspection was considered critical to monitor and verify the results of the hydrodemolition, and airlifting operations. Observations of the taped video inspection were as follows:

- In the zone of good concrete (Zone 1 as per Figure 5) the smooth walls of the core hole were visible.

- In the zone of primarily grout (Zone 2) there was no aggregate visible in the matrix. Various voids were visible.

- In the zone of "popcorn" concrete (Zone 3) various voids were visible, including larger open voids and areas of honeycombing. Darker aggregate appeared to be visible as part of the concrete matrix. It also appeared that though the hydrodemolition did not remove the concrete as intended, at a minimum it removed any loose cement/sand mix adjacent to the core. In general the voids seemed to be relatively large in size, with larger voids interconnected and surrounded by what appeared to be a "solid" concrete matrix. In this region various CSL tubes and reinforcing bars were also visible among the voids, which assisted in orientating the view and providing a sense of scale.

- The camera could not enter Zone 4.

As a result of the above observations the Contractor proceeded with placing the aggregate and grouting the caisson.

Pre-Placed Aggregate and Grouting

It was decided to add pre-placed aggregate and pressure grout the repair zone from the bottom up to fill the voids in the aggregate with a cement/sand paste. Due to the high importance of a successful repair, a prototype was set up on dry land as a test. The purpose of the prototype was to evaluate the proposed grouting methodology, determine set and curing requirements, and to obtain cores and cubes for compressive strength testing prior to the actual grouting operation. Three prototypes were set up, including the following:

1. A 150 mm tube, three metres in height filled with preplaced aggregate to determine if the selected grout could be pumped a significant distance up through the aggregate. The aggregate was wetted prior to grouting in this test.

2. A 400 mm tube, 1200 mm in height to determine if the grout would disperse laterally as well as vertically through the preplaced aggregate when grouted from a point source. The aggregate was wetted prior to grouting in this test.

3. A third tube of equal dimension and grouted in a similar manner to the second prototype but with the aggregate left dry prior to grouting.

In general grouting of the preplaced aggregate in the prototypes went well. Though none of the tests were done in a tremie condition, it was shown that the pre-wetted aggregate facilitated the grouting operation. In both cases with the prewetted aggregate the grout dispersed throughout the tubes both vertically and laterally, pumping from the bottom up did not create any difficulty, and no voids were found following grouting. A small number of voids were found in the test without the aggregate pre-wetted, however the grout still dispersed well throughout the aggregate matrix. All indications were that grouting of preplaced aggregate in the caissons had a good probability of success. The repair zone was then grouted as follows:

- Two 16 mm (5/8”) PVC tubes were installed to the base of the repair zone to act as primary and back-up grout tubes. The back-up tube was to be used only if the primary tube became plugged during the grouting operation.

- Sealed 32 mm (1 ¼ ”) PVC tubes were inserted into the intact CSL tubes to ensure these tubes would not become plugged during the grouting operation, and to enable final CSL testing for determining the effectiveness of the grouting operations. (Some damage had occurred to the CSL tubes in the prior coring operations.)

- The 20 mm (3/4”) aggregate was slowly placed into the caisson through the 122 mm (5”) core holes. Pressurized air was injected into the caisson through the 16 mm PVC tubes during placement to facilitate consolidation of the aggregate.

- Micro-silica grout was pumped into the caisson from the bottom up under tremie conditions so that the water was displaced from the voids and replaced by the cement paste. Grouting progress was closely monitored to allow for early detection of leakage outside the steel casing into the river. In the event leakage was detected, a contingency plan involving a secondary grouting procedure was also developed.

It was noted that the use of pre-placed aggregate in all open spaces of the repair would reduce the volume of grout required to fill the void, reduce the heat of hydration in the caisson repair area, and minimize the potential for shrinkage. Microsil Anchor Grout was utilized, and the supplier noted that the grout contained silica fume and was designed to be used at a low water/cement ratio between 0.27 and 0.31. It was therefore resistant to water washout and could be used in tremie conditions where water was not flowing. Furthermore, the Microsil grout also resists bleeding and segregating under pressure and was pumpable within the range of water/cement ratios noted above.

In general, the pre-placed aggregate and grouting operations appeared to effectively repair the zone of defective concrete as intended. Core samples obtained from subsequent coring/drilling operations required for dywidag installations were visually inspected and tested for compressive strength to verify the effectiveness of the repair. Video inspection and final CSL testing provided further evidence as to the quality of the repair. Final coring following placement of the mini-piles and dywidag rods was not done due to the high risk of damaging these installations, and due to the reasonable level of confidence in the repairs gained from the previous cores, video inspection and final CSL testing.

Dywidag Installations

Three dywidag anchors were installed in the mini-piles to transfer the vertical and lateral loads to the bedrock as a pinned connection. Figure 6 indicates the design requirements for the dywidag installations, as well as the as-built conditions.

As noted previously the initial design consisted of three dywidag installations in 122 mm diameter mini-piles. The three dywidags were to be embedded a minimum of 1.5 metres into the caisson and extend 5, 10, and 15 metres respectively into the bedrock below the base of the rock socket. Due to difficulties encountered during coring and hydrodemolition, these installation requirements were revised during caisson remediation to best suit the present conditions.

Early coring efforts resulted in losses of steel casing and core barrels within the repair zone, which further reduced an already limited target zone at the base of the rock socket and increased the complexity of the dywidag installations. For these reasons, the Contractor was only able to install dywidag anchors to depths that extended 4, 10.5, and 12.5 metres into the bedrock below the base of the rock socket. It was determined that the overall skin friction provided by the three mini-piles as installed was adequate for the required vertical load transfer.

Based on site observations during the caisson remediation, hydrodemolition did not appear to effectively remove any defective concrete from the caisson's rock socket, and as a result the repair in this portion of the defective zone would not allow for adequate transfer of the vertical load from the caisson into the dywidag anchors. To ensure that adequate vertical load transfer was provided two of the three dywidag anchors were extended up into the sound caisson concrete above the defective zone of concrete, effectively "bridging" the repair zone. Final analysis of the completed installations verified that vertical load transfer was provided irrespective of the quality of the concrete repair in the bottom six metres of the caisson. In effect, the pre-placed aggregate and grout was required only to encase and provide lateral support to the dywidag bars within the repair zone.

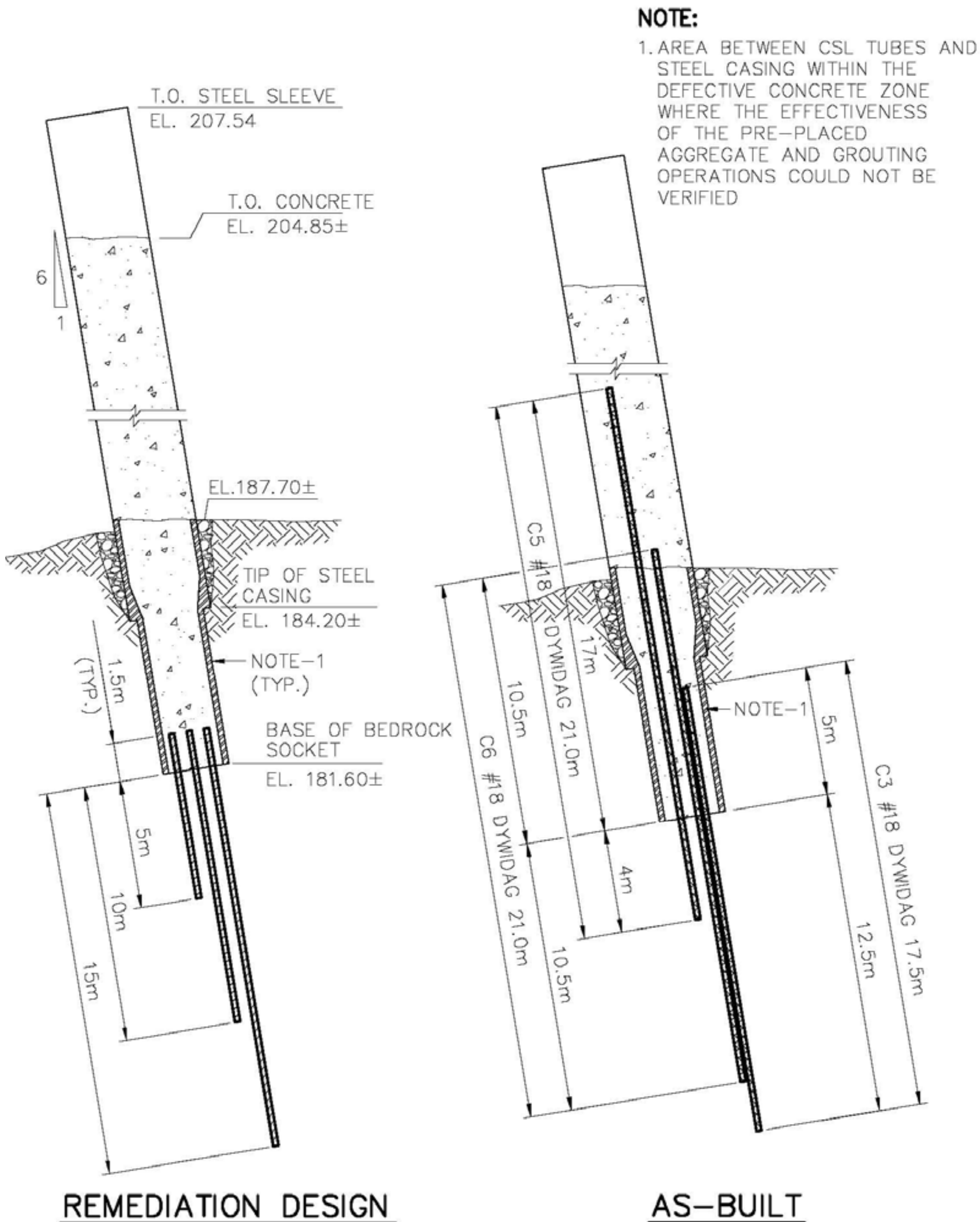

Figure 6: Dywidag installation details to transfer vertical and lateral loads from the sound concrete in the caisson to the underlying bedrock.

Final Testing and Acceptance

At the conclusion of the investigative and remedial work on the caisson, it was determined that two access tubes were still available to use for final CSL testing. The two tubes allowed for one set of data to be obtained diagonally across the repair zone. Based on comparison of the results before and after the repair, a very substantial increase in both the wave speed and material uniformity in the bottom six metres of the caisson was achieved. Furthermore, the apparent wave speeds measured in the remediated zone were comparable to those measured in the upper sound material.

Coring undertaken upon completion of the hydrodemolition and grouting operations provided further verification of the quality of the repair. A uniform, sound matrix of grout and aggregate was observed in all subsequent cores extracted from the caisson during the dywidag installations.

The final remediation of the caisson was accepted based on the information obtained from as-built records, final CSL testing and core test results, which indicated that the loose aggregate and voids had been replaced by a grouted aggregate, and successful installation of the dywidag anchors.

RECOMMENDATIONS

Significant knowledge was gained on the Kichi Sipi Bridge project through the successes of several design and construction innovations, as well as through the remediation of a defective caisson. Recommendations are provided regarding the prevention and mitigation of similar caisson installation issues on subsequent projects, and with respect to the specific quality assurance and repair methods utilized on this project. With respect to the prevention and mitigation of similar situations, recommendations are as follows:

1. This project demonstrates that structural designs benefit from a level of redundancy in the event of installation problems with key elements. Redundancy was specifically addressed in the design of the foundations which allowed for complete removal of one of six caissons at any pier without collapse of the structure. (This was assumed to occur in some catastrophic future event, not due to a construction issue.) This design, as well as the configuration of the caissons to be outside of the pier shaft area, allowed for pier construction and girder erection to continue during the caisson repair with only minor modifications at the pier cap. The steel caisson sleeves were also assumed to act only as formwork and were not relied upon in the original design, which added an additional safety factor throughout the repair, as well as for the future. In general for high-risk installation elements this redundancy and “forgiveness” in the design is strongly recommended.

2. The project specifications included a substantial section for tremie concrete, and this specification has been successfully utilized for many years in Manitoba. However, this project did identify opportunities for improvements. Specific parameters for materials, equipment and installation procedures should be developed further in consultation with the construction industry and be incorporated in the contract documents, including a formal review and approval process during construction. Issues to be considered include improvements in the concrete mix designs to facilitate pumping operations (reduce line plugs), tremie concrete methods that identify methods of controlling the vertical movement of the tremie pipe, limitations on pipe geometry, and improved methods of determining the level of concrete in the pile at all times. Specific to this project, the only available local aggregate source was the rock blasted and crushed for the project. It would have been beneficial to import smooth aggregate specifically for the caisson concrete to provide a more pumpable mix. In general the importance of the above issues is also magnified for higher risk elements.

3. Maintaining embedment of the tremie pipe in a concrete tremie pour is a fundamental requirement for tremie concrete work. Had the tremie pipe been fully reinserted into the concrete following the line plug significant repair costs would have been avoided. The

importance of carefully monitoring the progress of the concrete placement throughout the tremie pour cannot be overstated.

With respect to the quality assurance and repair methods for the defective caisson, recommendations are provided as follows:

1. Crosshole sonic logging was very effective in identifying the zone of poor concrete, and the final CSL testing also provided sufficient information to allow acceptance of the repair without the risk of further coring damaging the repairs. Augmenting the CSL testing with cores where necessary was also very effective in verifying the initial CSL test results.

2. The use of hydrodemolition to remove poor concrete in this caisson had very little effect, other than possibly cleaning the existing voids from a weak sand and cement mixture. Even with aeration of the water, the pressure was still not adequate to remove even the weak 7 MPa grout mix. This does not mean that a higher pressure system would not be effective; however success was not achieved on this project when attempting to perform hydrodemolition in a submerged condition and in deep water.

3. The ability to produce a televised video inspection down an adequately sized core hole was very significant to assess the condition of the repair zone, beyond the confines of the core itself.

4. The testing of and use of grout with preplaced aggregate in a tremie condition appeared to be highly successful as evidenced by the above-ground tests, coring and final CSL testing. Estimates and accurate measurement of the volume of voids, aggregate to be and actually placed, and volume of grout to be and actually placed is important to help monitor and verify that the desired repair has been achieved.

5. Coring the full length of relatively narrow and long caissons can be extremely difficult, especially if the caissons are battered. The cores tended to intersect each other and cause damage to reinforcing, CSL tubes and the casing sleeves of previous cores. Most of the cores tended to wander to the lower side of the batter. This may be attributed to the entire caisson having a slight bend in it from the original installation, rather than the core actually wandering. This could not be verified. The difficulty in obtaining cores straight down the middle of the caisson also verified that CSL testing was a better option than coring for general quality control for the caisson work.

In general the construction of the Kichi Sipi Bridge to Cross Lake provided many challenges which were overcome through various innovative design and construction techniques, including modifications following problems in the field. The final product showed that with proper considerations in the original design and a committed effort by all parties during construction, very challenging situations can be effectively overcome. Despite the many challenges encountered, this project was completed successfully, within the project budget, and the bridge was opened ahead of schedule.