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STRUCTURAL HEALTH MONITORING OF LINDQUIST BRIDGE

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Abstract

Many forestry bridges in Canada are typically single-lane, single span structures with two steel plate girders and a deck comprising of precast reinforced concrete panels. The concept of arching in deck slabs was utilized in the steel-free precast panels used in the Lindquist Bridge in British Columbia, Canada. The panels were completely devoid of tensile reinforcement and transverse confinement to the panels was provided by external steel straps. After the bridge was constructed in 1998, electrical strain gauges were installed on the girders and straps. Static and dynamic load tests were performed. The cracks on the top and bottom of the deck were mapped in 1999 and 2003. In 2006, a load test and crack mapping were performed on the bridge. The strain readings in the straps were compared with the data obtained 8 years prior. After analysis of the cracks and strain gauge readings, conclusions were drawn on the performance of the bridge. The cracks were formed to accommodate arching action and it was concluded that the bridge is still performing as it was designed.

INTRODUCTION

The steel free concrete deck slab concept was developed in Canada during the past 18 years [1]. Traditional steel reinforced concrete bridge decks are designed to resist loads in flexure. Past research has shown that bridge decks subjected to concentrated wheel loads behave in arching. The unreinforced concrete deck slab design takes advantage of the internal arching action by removing the internal reinforcement and utilizing transverse steel straps between girders. Thus, the steel free deck system fails by punching shear at substantially higher loads than the flexural design load. Also, by eliminating conventional internal steel reinforcement, concrete deterioration due to corrosion is minimized. The concept of the steel free deck slab has been applied to several bridges across Canada over the past 10 years.

Forestry bridge superstructures in Canada are comprised of conventionally reinforced precast concrete panels supported by a pair of steel girders. Precast panels are used for ease of installation in remote areas. In 1997, the Lindquist Bridge (Figure 1a) in British Columbia was constructed with unreinforced precast concrete panels [2]. The 24 m single span bridge is primarily used by heavy logging trucks (Figure 1b).

a) b)

Figure 1. The Lindquist Bridge: (a) a view of the bridge; (b) a logging truck on the bridge.

The Lindquist Bridge was designed for a simply supported span of 23.1 m and consists of a total of eight externally restrained precast concrete arch panels on two steel plate girders, which rest upon steel piles. Each panel was made with polypropylene fibre reinforced concrete and measured 3.0 x 4.3 m in plan. Three steel straps were embedded at their ends in the precast panels at a spacing of 1.0 m. The 25 x 50 mm studded steel straps provide the transverse confinement to the panels. A cross-section of the bridge is shown in Figure 2. Stiffeners on the web of the girders are located every 3.0 m and cross-frames between the two girders are spaced at 6.0 m.

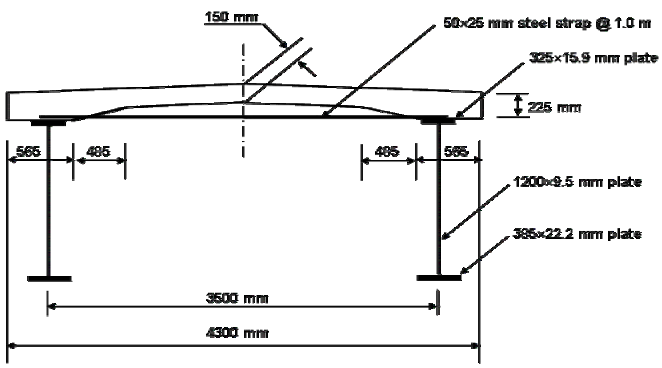


Figure 2. Cross-section of Lindquist Bridge.

INSTRUMENTATION AND TEST DETAILS

In the spring of 1998, load tests were performed on the Lindquist Bridge using a controlled vehicle load to observe the behavior of the bridge. The strains on the midspan straps and girders were monitored during the test and were then analyzed to confirm that the bridge was behaving according to bridge design assumptions [3]. Researchers returned to the bridge in the summer of 2006 to perform similar tests using a similar controlled vehicle load.

The orientation of the Lindquist Bridge is roughly in the east-west direction. A plan of the bridge is shown in Figure 3 with sections AA to DD denoting the locations where electrical resistance strain gauges were installed. A summary of the sensor locations is found in Table 1.

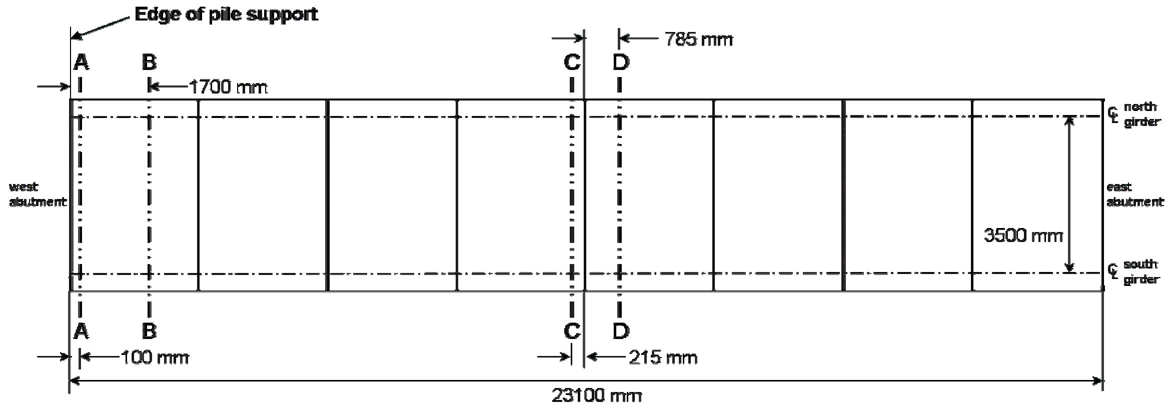


Figure 3. Locations of instrumented cross-sections.

Table 1. Sensor locations.

Identification	Section	Sensor Location
S1	A-A	stiffener on south girder
S2	A-A	top of bottom flange of south girder
S3	B-B	top of bottom flange of south girder
S4	C-C	top of bottom flange of north girder
S5	C-C	dummy gauge on north girder
S6	C-C	bottom of top flange of north girder
S7	C-C	bottom of strap
S8	C-C	bottom of top flange of south girder
S9	D-D	bottom of strap

The test vehicle consisted of a gravel dump truck carrying a front end loader and hauling a trailer transporting an excavating machine (Figure 4). The axle weights of the fully loaded test vehicle were measured on a weighing scale where the two axles of each tandem were weighed collectively. The total weight of the vehicle was 43.9 kN.

Figure 4. Test vehicle.

A plan view of the Lindquist Bridge is shown in Figure 5, where each precast deck panel has three steel straps. Electrical resistance strain gauges were installed on two transverse steel straps near mid-span and labeled S7 and S9, respectively.

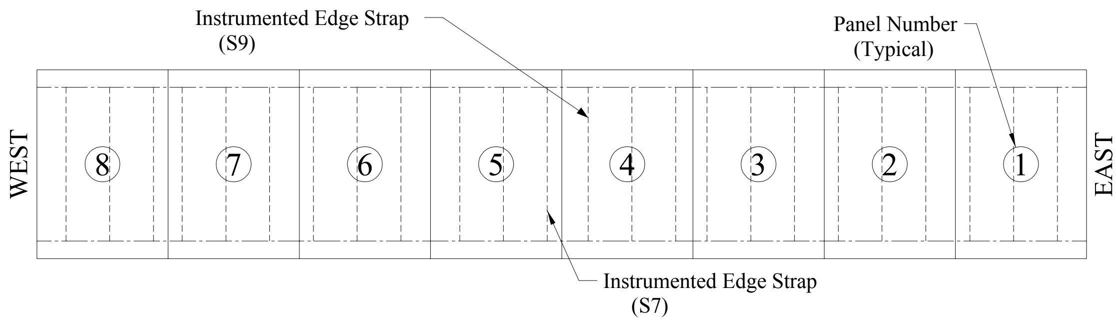


Figure 5. Locations of instrumented steel straps.

The tests involved the vehicle traveling over the bridge in the centre transverse position at four speeds: crawling, slow, medium, and fast. The test vehicle traveled over the bridge towards the east abutment during the crawling speed tests and traveled towards the west abutment at the remaining speeds. During the crawling speed tests, the weights of the driving axle, 1st tandem, and 2nd tandem were 35.2, 232.3, and 171.0 kN, respectively. During the dynamic tests, the weights of the driving axle, 1st tandem, and 2nd tandem were 40.8, 213.07, and 184.04 kN, respectively. Measurements from the electrical strain gauges were recorded by a data acquisition system.

STRUCTURAL HEALTH MONITORING

Composite Action

One of the main objectives of the first monitoring of the Lindquist Bridge in 1998 was to determine if the unreinforced deck slab was acting compositely with the girders. At that time, strain gauges on the girders at the mid-span were installed only on the bottom flanges. The presence of composite action between the deck slab and the girders was then confirmed indirectly by noting that these strain readings on the bottom flanges at mid-span were smaller than those based on calculations for composite action. Thus, to confirm this assessment, strain gauges were installed at both the top and bottom flanges of the north girder at the mid-span during the second monitoring in 2006.

The top and bottom strains at the mid-span of the north girder are plotted against time in Figure 6 as the test vehicle traveled at a crawling speed. It can be seen in this figure that the magnitude of the tensile strains in the bottom flange are always larger than the magnitude of the compressive strains in the top flange. This confirms the presence of the composite action between the deck slab and the girders.

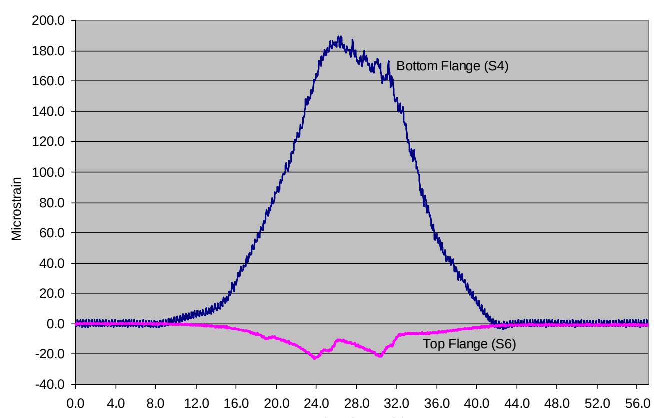
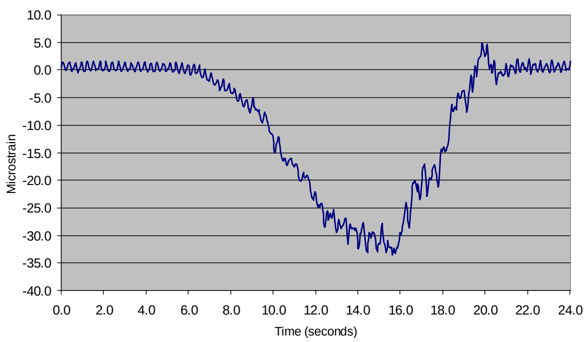


Figure 6. Longitudinal strains at mid-span in North girder.

Bearing Restraint

During the 1998 test of the Lindquist Bridge, the longitudinal strains due to the test loads at the bottom of the girders were about 24% smaller than the corresponding strains calculated by assuming full composite action between the deck slab and the girders [3]. As a result, it was suggested that the bearing restraint forces, combined with the moment resistance of passive earth pressure on the girder/ballast wall connection, was one of the factors responsible for the discrepancy between the observed and analytical girder strains.

In order to investigate this further in the 2006 monitoring, a strain gauge was installed on the bottom flange of the south girder at Section A-A (Figure 3), which is located only 100 mm from the edge of the pile support. In a truly simply supported bridge, the strains at this location would have been tensile and very small in magnitude. The bottom flange strains induced by the test vehicle moving at a crawling speed near the support are plotted in Figure 7a while a photograph of a girder support is shown in Figure 7b. It can be seen in Figure 7a that the strains at the bottom girder flange near the support are negative, i.e. compressive. The maximum magnitude of these compressive strains is about 34 με and is not small compared to the maximum tensile girder strain at the mid-span which is about 200 με (Figure 6). Thus, it is confirmed that the girders of the Lindquist Bridge are not simply supported, and have substantial bearing restraint.



a)

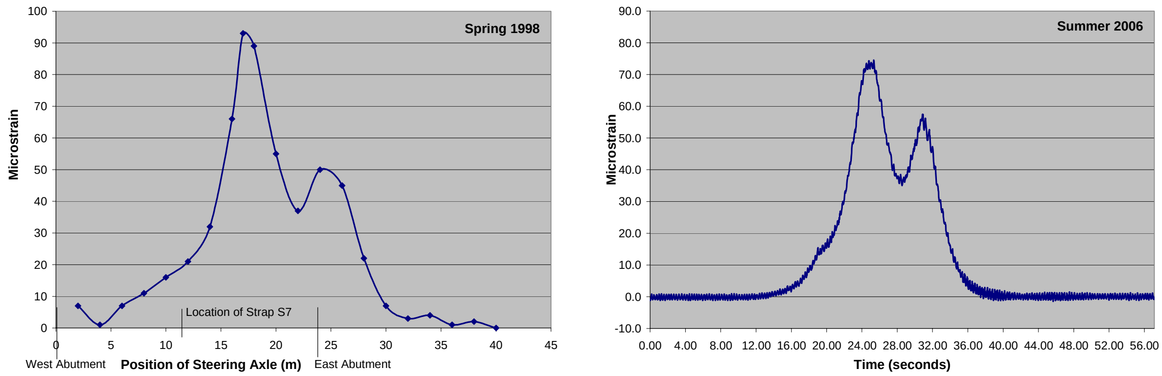
b)

Figure 7. a) Bottom flange strains at Section A-A; b) Girder support.

Strap Strains under Quasi Static Loads

As the test vehicle traveled across the bridge at a crawling speed, measurements from the strain gauges were recorded. The test vehicle crossed the bridge towards the east abutment in the central transverse position. After clearing the bridge, the vehicle reversed over the bridge towards the west abutment. In 1998, the strains in strap S7 were 93 and 50 με (Figure 8a) [2], in the 1st and 2nd tandems, respectively. In 2006, as the test vehicle crawled forwards towards the east, the strains in Strap S7 are plotted against time in Figure 8b.

In order to compare the strain readings in the strap to the 1998 measurements, the ratios are determined using the vehicle loads in both tests. The ratio of the loads on the 1st tandems of the 1998 and 2006 test vehicles is 1.31, and the corresponding ratio for the 2nd tandems is 0.90. Using the ratios, the expected strain readings in the 2006 test are 71 and 56 με. It can be seen in Figure 8b that the maximum strain in Strap S7, induced mainly by the 1st tandem of the test truck was about 73 με and the strain induced by the 2nd tandem was 57 με. This suggests that under the same loads, the strap strains have hardly increased since 1998.



a) b)

Figure 8. Strain in Strap S7: a) Spring 1998; b) Summer 2006.

Strap Strains under Dynamic Loads

Three dynamic tests were conducted on the Lindquist Bridge where each test involved the test truck crossing the bridge in the central transverse position towards the west abutment. The test vehicle crossed the bridge at three different speeds: approximately 15, 25 and 29 km/hr. The strains in Straps S7 and S9 during the slow speed test are plotted against time in Figure 9.

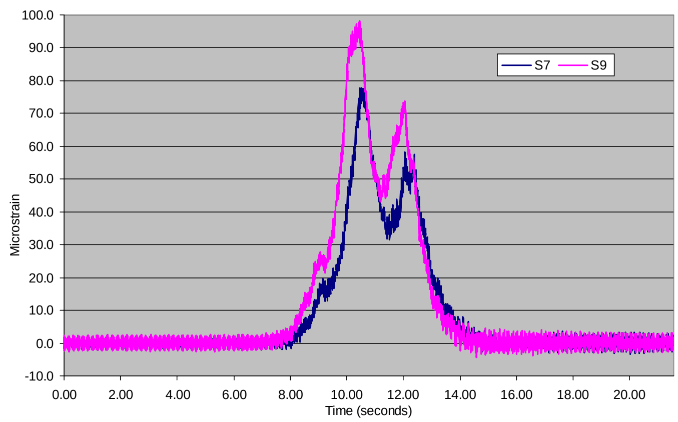


Figure 9. Strains in Straps S7 and S9 with test vehicle traveling at about 15km/hr.

The plots of the strains in Straps S7 and S9 during the medium and fast speed tests are similar to Figure 9. It can be seen that the strains in the strap closer to a cross-frame (ie. S7) are consistently lower than those in the other strap, thus confirming that the top horizontal component of the cross-frames also participates in restraining the deck slab in the transverse direction.

CRACK PATTERNS

Another method of monitoring the health of the Lindquist Bridge involves observing the behavior of crack propagation in the precast unreinforced concrete arch panel deck slabs. The main reason for crack progression is fatigue. In 1999, 2003 and 2006, the precast deck slab panels of the Lindquist Bridge were inspected very carefully. Before all inspections, debris on the bridge deck was shoveled and swept with hand brooms. Care was taken not to allow debris from the deck to fall into the creek. A truck with a high-pressure water pump was used to wash the deck surface. After the deck was cleaned, visual observations were taken of cracks on both the top and bottom of the deck slab. The length of each crack was measured using a tape measure where it did not terminate at the edge of the concrete. The widths of the cracks were measured with a pair of digital calipers.

In general, the measurements indicate that some of the crack widths appear to be smaller in 2006 than they were in 2003. However, measurements in 2003 were taken during the fall, while in 2006 they were taken during the summer. Thus, there was likely a temperature difference of between 10 to 20ºC. If one considers the change in length of the top lateral steel bracing and compares the difference in length between the transverse bracing (in plan) and the diagonal bracing (in plan), there would be a difference of change in length, transversely, of approximately 0.13 mm to 0.26 mm. Relatively speaking, the diagonal brace would therefore resist full contraction in the transverse direction whereas the slab would be free to contract, thus accounting for the cracks in cooler temperatures being wider. This was a general trend noticed for the longitudinal cracks in the centre of the panels.

Longitudinal Cracks Between Girders

In 1999, the main cracks observed were bottom longitudinal cracks midway between the girders. Upon the 2nd visual inspection of the Lindquist Bridge in 2003, corresponding top longitudinal cracks midway between the girders was observed in six panels. Finally, in 2006, it was observed that very few additional cracks formed since 2003.

The Lindquist Bridge is considered a 1st generation externally restrained deck slab with no internal crack control reinforcement. It was observed that the longitudinal cracks in 1st generation deck slabs extend the full depth of the slab. As a result, there was some concern raised about possible failure in these types of slabs due to shear forces caused by wheel loads traveling on one of the cracks. However, further research has removed any concern about the safety of externally restrained deck slabs with full-depth longitudinal cracks, and without crack control reinforcement [4].

It is noted that the 2nd generation of externally restrained deck slabs are provided with a crack control grid of glass fibre reinforced polymer (GFRP) bars. As specified in the ACI document [5] and the Canadian Highway Bridge Design Code [6], the requirement for the crack control grid will ensure that the unsightly cracks observed in the deck slab of the Lindquist Bridge are not formed. However, it is emphasized that the requirement for the crack control grid was introduced for aesthetic, rather than safety, reasons.

Longitudinal Cracks Over Girders

The precast deck slab of the Lindquist Bridge is made composite with steel girders by means of shear bulkheads consisting of 250 mm diameter corrugated galvanized steel pipe bulkheads. Each bulkhead contains seven shear studs (Figure 10a) and site-mixed grout. The shear bulkheads are also referred to as grout pockets. The grout poured in the shear bulkhead and in transverse joints between the precast panels was mixed at site using heated water. The installation of the grout was not of the highest quality, and was found missing in one location as shown in Figure 10a, in which the heads of the studs are exposed.

During the years 1999 and 2003, longitudinal cracks developed between these shear bulkheads (Figure 10b). It is important to note that such cracks were not observed in the fatigue tests on the same type of deck slab panel as employed in the Lindquist Bridge [7].

a) b)

Figure 10. a) Exposed shear studs in one shear bulkhead; b) Longitudinal cracks between shear bulkheads

A possible explanation of why these longitudinal cracks were formed involves the lateral forces experienced by the bulkheads. As seen in Figures 8 and 9, the tensile forces in straps are highly localized and are highest when a truck axle is directly above a strap. As a result, the top flanges of the girders are pushed out laterally at the locations of the shear bulkheads by forces of different magnitude. In Figure 11, a case is shown where only one shear bulkhead is subjected to a lateral force acting towards the outer edge of the bridge, while the adjacent bulkheads are not subjected to any lateral forces. Clearly, in such a case, the transverse tensile stresses will crack the unreinforced slab at the section with the smallest area of cross-section, which is the longitudinal section between the shear bulkheads.

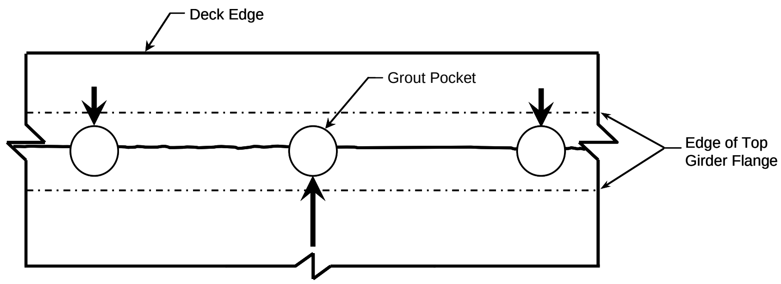
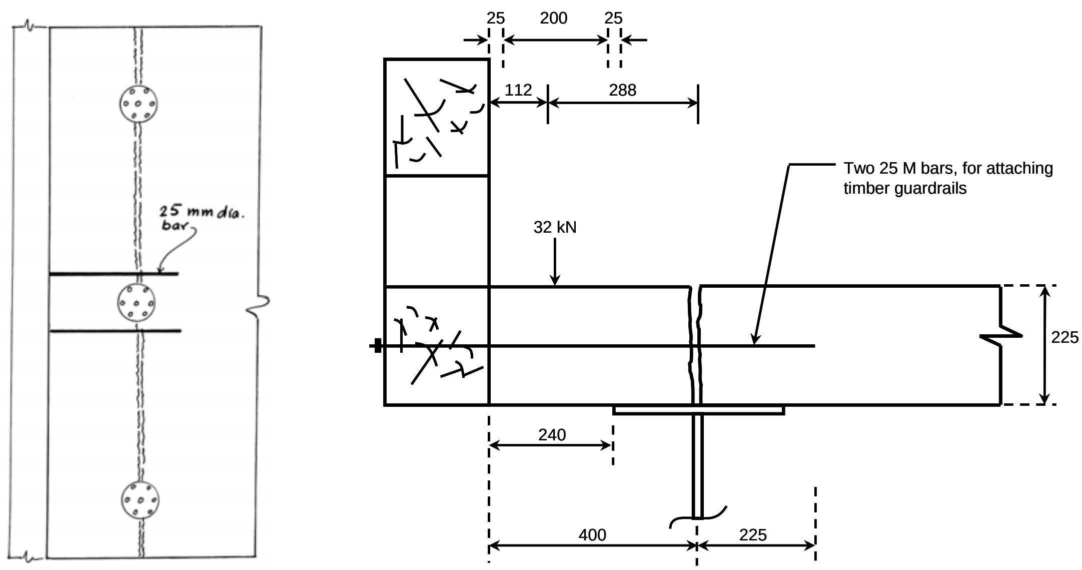


Figure 11. Example of only one shear bulkhead subjected to lateral force.

At a cursory glance, the longitudinal cracks in the precast panels above the girders (Figure 10) give a cause for concern about the stability of the cantilever portion of the unreinforced deck slab, lying beyond these cracks and the anchorage of the steel strap. However, upon reflection, it becomes clear for several reasons that the cantilever portions of the deck slab have little danger of being detached from the main deck slab.

A careful inspection by several professional engineers confirmed that the longitudinal cracks stop at the shear bulkhead and do not appear to go around the bulkheads when viewed from the top of the deck. If the longitudinal cracks were developed by transverse negative moments (ie. due the weight of the vibrating timber guard rail), the cracks would have gone around the perimeter of the shear bulkheads.

The shear bulkheads are formed with corrugated steel pipes (see Figure 10a) which provide a mechanical connection in the vertical and longitudinal directions between the corrugated pipe and the surrounding concrete. The cantilever portions of the deck slab beyond the longitudinal cracks cannot break off without engaging the corrugated steel pipe. Since only a single-tire wheel of the most eccentric vehicle can come on the cantilever portion of the slab, for the L75 design vehicle, the maximum weight of this wheel is 32 kN. As seen in Figure 11a, there are two 25mm diameter x 625 long steel reinforcing bars installed midway along the exterior edge of each interior deck slab panel and project approximately 125 mm beyond the cracks. These rods are provided to secure the timber guard rails to the deck slab. In the event that longitudinal cracks between the shear bulkheads are joined by circumferential cracks around the bulkheads, these rods would be required to restrain the detached portions of the deck slab. The transverse moments in the deck slab due to live loads are expected to reach their peak at the edge of the girder flange, beyond where their intensity should remain nearly constant up to the location of the girder web. It was found that the total transverse moment intensity at the edge of the top flange due to factored dead loads and factored 32 kN wheel, including 40% impact, is 9.9 kN.m per 3 metre length of the panel. The factored moments of resistance of the deck provided mainly by the two 25 M bars varies from zero at the extremities of the bars to 15.3 kN.m per 3 metre length of the deck slab. It is therefore concluded that there is little danger of the cantilever portion of the deck slab from being detached.



a) b)

Figure 11. a) Cracks in precast panels over girder; b) A single-tire wheel of the most eccentric vehicle

CONCLUSION

This paper has described the periodic structural health monitoring involving load testing and crack observations of a forestry bridge which was constructed with unreinforced precast concrete deck panels. The second monitoring of the Lindquist Bridge has provided researchers with information to confirm the conclusions drawn from the first monitoring in 1998. The following conclusions are drawn from the inspection and second monitoring of the Lindquist Bridge.

a) The live load strains in the straps have hardly increased over the past eight years and give no cause for concern because their magnitude is very small.
b) There is full composite action between the deck slab and the girders.
c) The bearing and ballast wall restraint forces in the girders are quite large, thus making the bridge safer than assumed in the design.
d) Longitudinal cracks between the shear bulkhead do not compromise the safety of the cantilever portions of the deck slab. Since these portions contain transverse steel rods for connecting the timber guard rails to the slab, they have more than ample flexural resistance to sustain the negative transverse moments due to the dead loads.
e) The longitudinal cracks between the girders are small in width. The fatigue life of the deck slab will be of concern only when these cracks become much wider.

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