

TRANSVERSE POST-TENSIONING OF LONGITUDINALLY LAMINATED TIMBER BRIDGE DECKS

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Longitudinally, nail-laminated timber bridge decks are used extensively in northern Canada, where the cold and dry climate discourages the activities of fungi and termites. The load-carrying capacity of these structures is dependent upon their ability to effectively distribute wheel loads among the laminates; this being a function of the friction and the holding power of the nails. Due to overloads, volumetric changes and environmental effects, the holding power tends to diminish resulting in a subsequent reduction in the load-carrying capacity of the bridge. Construction of these nail-laminated systems involves extensive labour, since thousands of nails have to be driven into hundreds of laminates. Quality control is quite difficult as it requires continuous supervision to ensure that all the nails are properly placed and driven. This report describes the application of transverse post-tensioning to an existing longitudinally nail-laminated timber deck structure in Ontario. This 3-span continuous bridge portrayed the delaminating problem, and as such, presented an appropriate test site for a post-tensioning system. Load testing was done before and after post-tensioning to determine its effects on the structure's response. An evaluation of the test results indicated that the structure's load-carrying capacity was increased by at least 100%. Transverse post-tensioning, in effect, replaces the need for nailing and hence reduces the labour required for construction. Quality control extends only to ensuring that adequate post-tensioning forces exist to provide the friction necessary for load distribution, allowing for anticipated losses due to creep. This confined deck system exhibits better resistance to the environment, as it eliminates the penetration of foreign materials between the laminations.

The Ontario Ministry of Transportation and Communications is conducting an ongoing load testing program, in which a varying number of bridges are analyzed and tested each year. The primary objective of the program is to establish a safe load-carrying capacity, while increasing knowledge of the behaviour of bridges under load.

Prior to the start of a load testing season, an inspection is undertaken to determine which structures would most benefit from load testing. It is during these inspections that many inherent problems related to specific structural designs become apparent. One such problem concerns the delamination of Longitudinally Nail-Laminated Timber Deck structures (L.N.L.).

Since the L.N.L. system depends extensively on its ability to distribute wheel loads transversely, delamination of the system can cause a severe reduction in the capacity of the structure. In some instances, where some of the nails no longer perform their function, the capacity of the structure rests upon those laminates directly under the applied load.

Hebert Creek Bridge displayed the characteristic problems of a delaminated L.N.L. structure. The bridge was posted for a reduced load of 15 tons (133 kN) and identified for replacement in 1976. This bridge was selected as a test site for the development and testing of a transverse post-tensioning system. This system would create adequate friction to ensure load sharing of the laminations, while closing the deck tightly to prevent further deterioration.

This report is primarily concerned with the design and installation of the post-tensioning system for the Hebert Creek Bridge and the load testing of the structure before and after the system was stressed.

Site Description

Hebert Creek Bridge is a 2-lane structure located on Highway 539 about 5 km (3 mi.) north of Highway 64, near Field, Ontario. The 3-span continuous structure was designed and built for an H20 truck load in 1951.

The 16.76 m (55 ft.) long structure consists of two side spans of 5.33 m (17.5 ft.) and one center span of 6.10 m (20 ft.), as shown in Figure 1. The 51 mm x 305 mm (2 in. x 12 in.) timber laminations were placed on edge and oriented longitudinally, as illustrated in Figure 2. The timbers were so arranged to produce longitudinal discontinuities of the laminates at the one third span points and support points only. In addition, for any one transverse section the joints would occur at every third lamination.

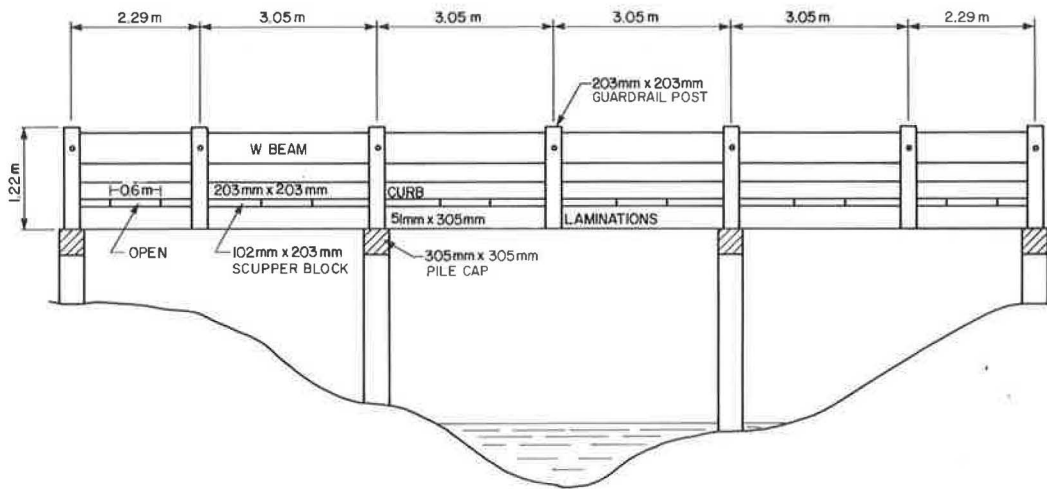


Figure 2. Hebert Creek cross section of deck at pier.

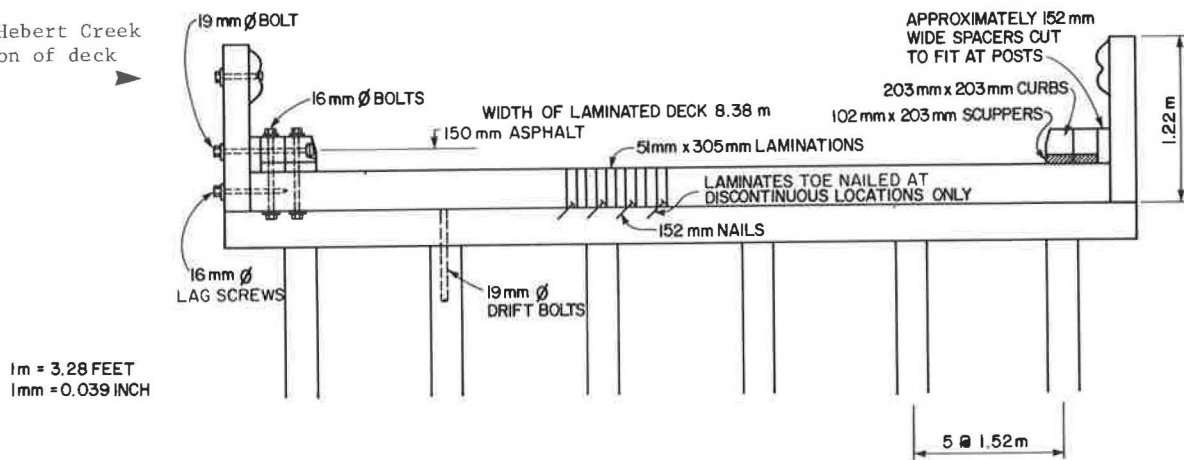
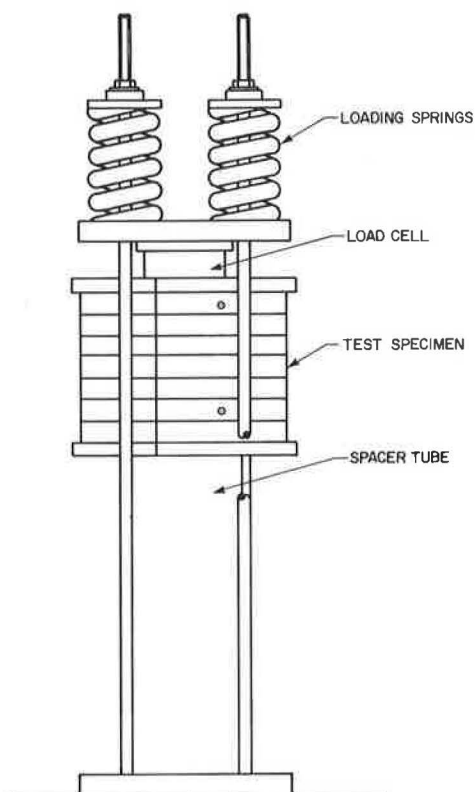


Figure 3. Creep testing machine.



Pre-Testing Evaluation

The structure was initially evaluated according to AASHTO (1) Specifications and the original H20 design loading was confirmed. A precise computer analysis of the structure, prior to post-tensioning, was not feasible due to the non-linear mechanical nature of the deteriorated nail-laminated system.

However, a computer analysis was performed which was intended to represent the post-tensioned structure. The analysis was conducted using a program called ORTHOP (3) assuming that the transverse transfer of load is performed by shear, with no transverse flexural stiffness. The analysis predicted a maximum longitudinal flexural stress of 6200 kPa (900 psi) under the maximum load of a 417 kN (93,800 lb.) tandem of the test vehicle. The maximum transverse vertical shear stress was established as 138 kPa (20 psi) which was incorporated in the design of the post-tensioning system. The maximum anticipated deflection of the post-tensioned structure under the test vehicle was 10 mm (0.4 in.), assuming a modulus of elasticity of 10.3×10^6 kPa (1.5×10^6 psi).

Allowances for support movements were not incorporated in the analysis, therefore, the actual displacements were expected to be higher.

Post-Tensioning

Creep Tests

Creep is considered to be one of the most important considerations in post-tensioning timber perpendicular to the grain. Under sustained stress the timber will creep considerably, reducing the initial stress that was introduced into the system. To evaluate the effects of creep, an investigation was conducted prior to the actual design of a post-tensioning system.

Two test specimens, composed of seven pieces of timber approximately 279 mm x 279 mm x 41 mm (11 in. x 11 in. x 1 5/8 in.), were constructed and installed into the testing apparatus as displayed in Figure 3. Both specimens were stressed to 1034 kPa (150 psi), however, the load springs were removed from one specimen to introduce a variation in the stiffness of the system. Both load and strain measurements were taken over a period of 2000 h.

The results of these tests are displayed in Figures 4 and 5 and indicate that an equilibrium was reached at approximately 1000 h. Plotting the ratio of the initial to the final stress against the stiffness of the test apparatus produces the two points on the curve given in Figure 6. With these points and the two known boundary conditions, an approximate solution to the curve was developed. This relationship was used to account for the effects of creep in the design of the post-tensioning system.

Further creep tests are being performed to gain an understanding of the effects of the environment on the test specimens under sustained load. This is being performed as a separate investigation since it involves a considerable amount of time and special equipment.

Design

An important aspect of the design is the evaluation of the minimum permanent stress required between the timber laminations, to ensure proper load transfer. An analysis was performed using a

program called ORTHOP (3), as described in the pre-testing analysis. Under the maximum tandem load of the test vehicle, 417 kN (93,800 lb.), the maximum transverse shear stress was calculated to be 138 kPa (20 psi). Assuming a conservative coefficient of friction between the timber surfaces of 0.5 (4), the required minimum prestress would be 276 kPa (40 psi). Using the relationship for creep developed in the preceding section, and assuming an initial stress of 1034 kPa (150 psi) on the timber, the stiffness of the required post-tensioning system should be about 70 000 kN/m (400,000 lb./in.).

The selection of the post-tensioning system used at Hebert Creek was based upon the following considerations:

1. Ductility - The system must be ductile enough to compensate for losses due to creep. A minimum design strength of 793 MPa (115 ksi) was required to provide a stiffness of 70 000 kN/m (400,000 lb./in.) based upon a modulus of elasticity of steel of 200×10^6 kPa (29×10^6 psi).
2. Adjustability - Since re-prestressing may be a necessary maintenance procedure, it should be entirely readjustable.
3. Protection - Since the system would not be embedded in concrete or grouted, as is the case with concrete design, environmental protection should be provided.
4. Practicality - The system should be such that it can be handled easily by local Ministry personnel and could be installed with little disruption to traffic flow.

After evaluating various systems available, a post-tensioning bar manufactured by Dywidag Canada Ltd. was selected. This system incorporates a high strength bar with spiral deformations, which use anchorage nuts containing complementary deformations. This system can be quickly re-adjusted and can be handled easily by the local Ministry personnel. As well, the 16 mm (5/8 in.) diameter bars have an ultimate strength of 1082 MPa (157 ksi) and can be safely loaded to the required stress of 793 MPa (115 ksi).

The layout of bars shown in Figure 7, with two bars at each station, provides an initial transverse stress of 1034 kPa (150 psi) on the timber decking. The anchorage bulkhead, as shown in Figure 8, consists of new 76 mm x 305 mm (3 in. x 12 in.) select structural Douglas Fir placed the full length of both sides of the timber deck and secured in place with 178 mm (7 in.) spikes. Steel anchorage plates, 305 mm x 457 mm x 38 mm (12 in. x 18 in. x 1 1/2 in.), bear directly against this timber at each post-tensioning station. Protection of the bars is provided by enclosing the bars in grease filled P.V.C. piping, a method which has already been used by Dywidag Canada Ltd. Steel sleeves are welded onto the steel plates and, under relative movement, telescope into the P.V.C. piping to maintain an enclosed environment. Plastic caps are placed over the anchorage nuts and filled with grease to prevent deterioration of the anchorage system.

To facilitate installation, the guardrail posts and curbs were removed from the structure. The top P.V.C. pipes, with the bars inside, were placed into transverse grooves cut in the asphalt. The bottom P.V.C. pipes with bars inside were secured to the underside of the timber deck with standard metal pipe brackets and wood screws, supported at every 0.91 m (3 ft.) along their length. Installation of the system, including the replacement of the curbs and posts but not including the stressing operation, took just over two working days with

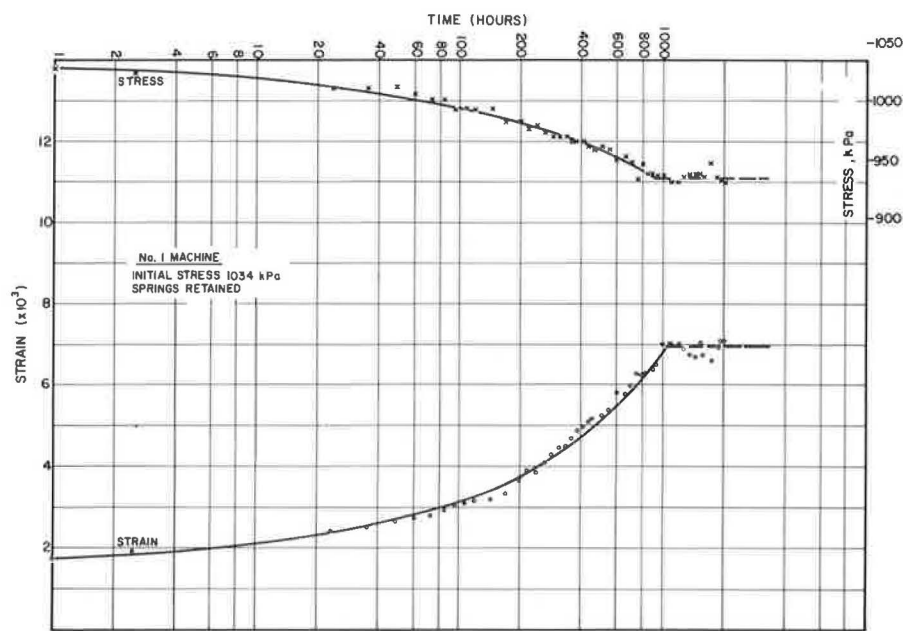


Figure 4. Results of creep tests, 2000 h.

Figure 5. Results of creep tests, 2000 h.

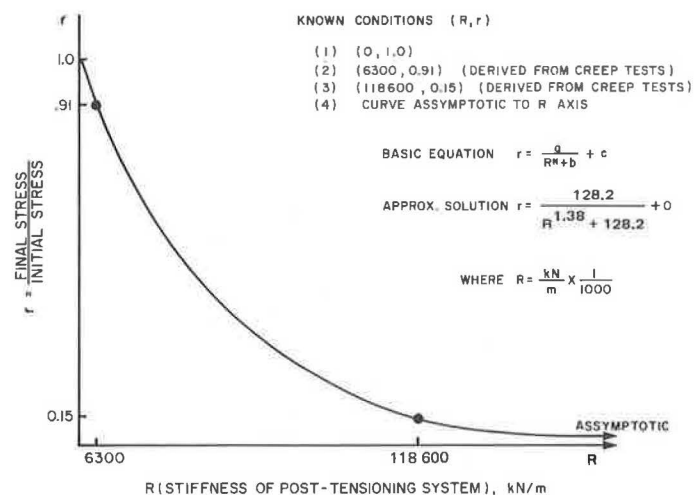
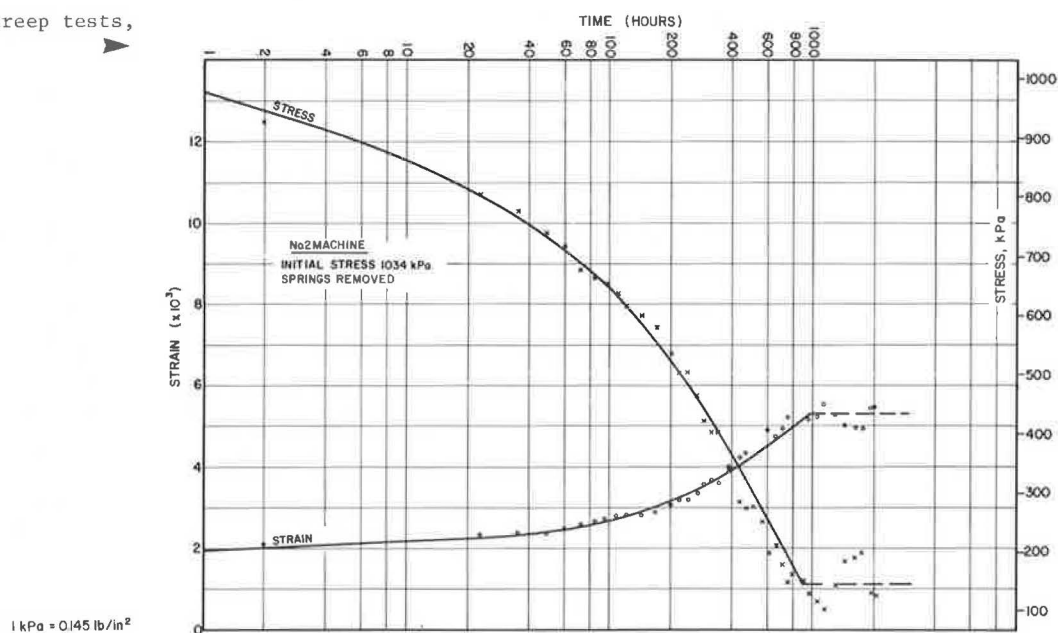


Figure 6. Ratio of final to initial stress versus stiffness of post-tensioning system.

four men (approximately 70 man-hours).

Stressing

Two jacks were used in tandem at one post-tensioning location to stress both the top and bottom bars simultaneously and to maintain a uniform pressure on the steel-timber contact surface.

Each location along the length of the bridge was stressed in turn to one half of the required load. Five complete passes were necessary to maintain some amount of tension at all of the post-tensioning locations. After a further seven passes an average load of 222 kN (50,000 lb.) was obtained at each post-tensioning location, which provided a normal pressure of 827 kPa (120 psi) on the timber deck. Further stressing was not possible as the rotation of some steel plates was causing local crushing of the timber bulkhead.

Though narrowing of the deck was anticipated it was not expected to exceed 150 mm (6 in.). However, a maximum of 450 mm (18 in.) occurred across some locations of the bridge.

It is estimated that about 60% of the movement of the structure occurred during the first five stressing operations. These took a total of 2 full working days to perform. The remaining 40% occurred in the last seven passes which also took two full working days. The stressing operation required the constant participation of at least three men.

Instrumentation and Testing

The instrumentation layout, as shown in Figure 9, consisted of 44 linear displacement transducers used to measure vertical displacements across the center of the north end span and center span, where maximum deflections were anticipated. The remaining transducers were positioned to record the vertical movement of the timber piles at the north abutment and the two interior piers. The south abutment was monitored with dial gauges.

The demac points were used to measure extreme fiber strain on the underside of the timber deck at the location of maximum moment occurring at 2.13 m (7 ft.) from the north abutment. The points were equally spaced from the west edge of the structure to the centerline, and each was oriented longitudinally in the center of the bottom face of a lamination.

The structure was tested in July 1976 using the Ministry's test vehicles, before and after post-tensioning. Testing was repeated 2½ months later in October when the majority of the post-tensioning losses was believed to have occurred. The vehicle was loaded to produce six increasing vehicle weights as indicated in Table 1, and was positioned in 18 static locations, as shown in Figure 10. Dial gauges at the south abutment were monitored whenever the vehicle was placed in the fifth or sixth longitudinal position of each lane.

Few problems were encountered while testing the structure apart from deformation of the asphalt pavement under the heavy wheel loads, which is a common occurrence during load testing.

Post Testing Evaluation

Test Results

Prior to post-tensioning, the load testing was terminated during the application of load level 5.

Under this loading, the failure of a timber lamination occurred at vehicle position 2 of lane 1 (Figure 10). Just prior to failure, the maximum deflection and maximum strain were 25 mm (1 in.) and 1470 micro strain, respectively. However, measurements were not taken at the actual time of the failure as the vehicle was being removed to prevent further damage to the bridge. It is believed that the deflections and strains would have been much higher at the time of failure, as visual observation displayed a sudden vertical movement of the laminations directly under the wheels of the vehicle. This vertical movement suggests that a loss of transverse shear capacity occurred which resulted in the overloading of several laminates and the failure of one.

Immediately after post-tensioning was completed, the structure was re-tested and the maximum load level, as shown in Table 1, was reached. No signs of structural deficiency were observed and the maximum deflections and strain were 16 mm (0.62 in.) and 940 micro strain, respectively. These maxima again occurred under the same vehicle position as in the previous test. In comparison to the unstensioned structure the maximum measurements under load level 5 were 14 mm (0.56 in.) and 750 micro strain of deflection and strain, respectively.

Figures 11 and 12 compare some transverse deflection curvatures from the load tests. These curves illustrate clearly the dramatic effects of the structure's response due to post-tensioning and indicate that the maximum deflection was reduced by 40% immediately after post-tensioning. The figures also show an additional 12% reduction in deflections under the final testing of the post-tensioned structure. It is believed that the new asphalt, which adds slightly to the stiffness and increases the load distribution, may account for the additional reduction in vertical displacements.

Table 2 displays the strain results obtained from the load testing of the structure in both cases. Strains were not measured during the final testing when the post-tensioning losses had occurred. Under load level 5 the maximum strain measured was reduced by 50% when the structure was post-tensioned. This reflects directly on the flexural capacity of the structure. Table 3 shows the relative changes in normal stresses in the timber laminates over a period of 1½ years. The table indicates that the normal stress has not changed significantly since the bridge was last tested in October 1976.

Post Testing Analysis

The action of the post-tensioned structure was not correctly represented by the computer analysis performed before testing. Under post-tensioning forces the structure performed similar to an orthotropic plate, with a very small transverse flexural rigidity. An orthotropic (3) analysis was performed assuming the transverse to longitudinal stiffness ratio to be approximately 1/20, based upon figures from the Wood Handbook (4). A shear modulus was estimated at 1/16 (4) of the longitudinal modulus of elasticity. The results of this analysis are displayed in Figures 13 and 14 compared to the test results of July and October 1976. Although the overall displacement diagrams are similar to the test data, there exists definite discrepancies in curvature between the points of load application. Variation of the modulus of rigidity and Poisson's ratio had little apparent effect on the curvatures. However, reducing the

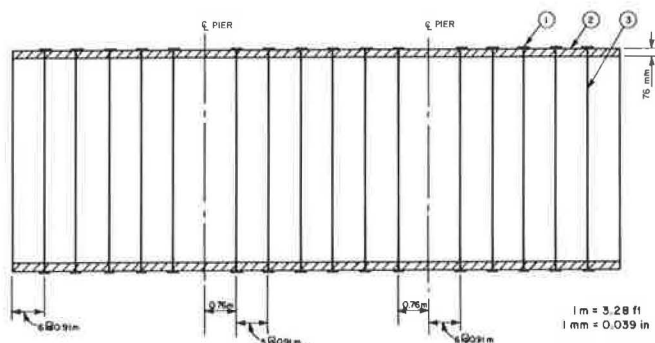


Figure 8. Details of anchorage and protection.

Figure 7. Plan view post-tensioning.

- 1 Steel Plates 38 mm x 457 mm x 305 mm
 - at 16 post tensioning stations
 - secured by 152 mm spikes
- 2 Select Structural 76 mm x 305 mm Wood Bulkhead
 - each side comes in 3 pieces
 - 2 @ 5.5 m and 1 @ 6.10 m
 - secured by 178 mm spikes
- 3 Bars 16 mm ϕ in 51 mm P.V.C. Sleeve
 - top bars below asphalt
 - bottom bars secured by clamps to underdeck

DETAILS OF ANCHORAGE SYSTEM

- ① METAL SLEEVE ATTACHED TO PLATE
- ② 51 mm P.V.C. SLEEVE FITTED OVER STEEL SLEEVE FILLED WITH GREASE.

1 m = 3.28 ft.
1 mm = .039 in.

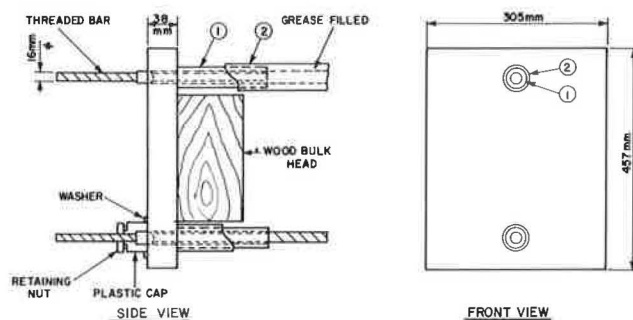


Figure 9. Plan showing instrumentation locations.

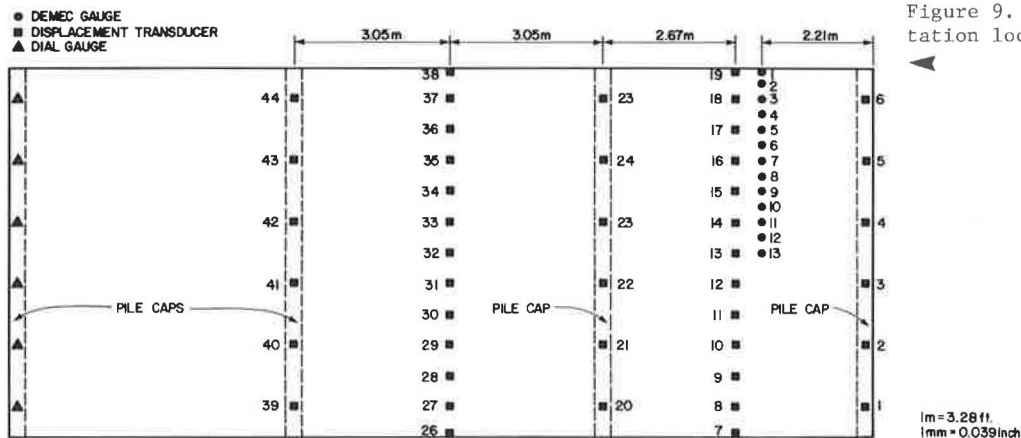
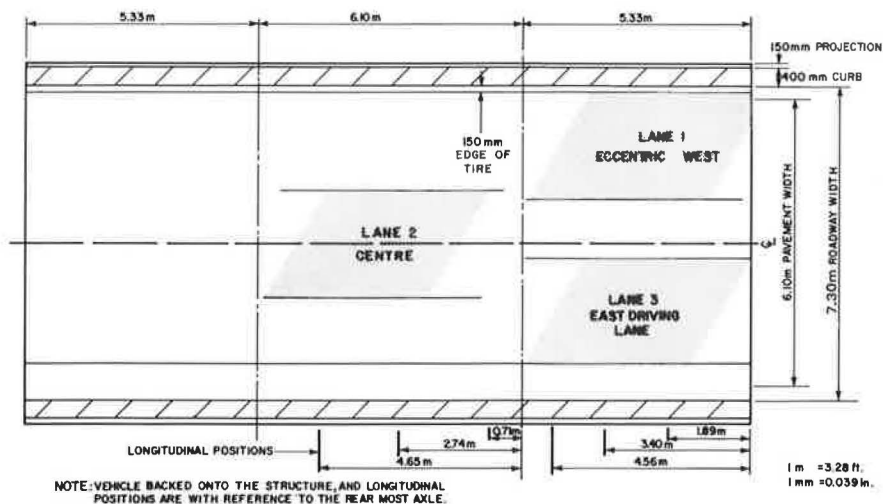


Figure 10. Load vehicle positions.



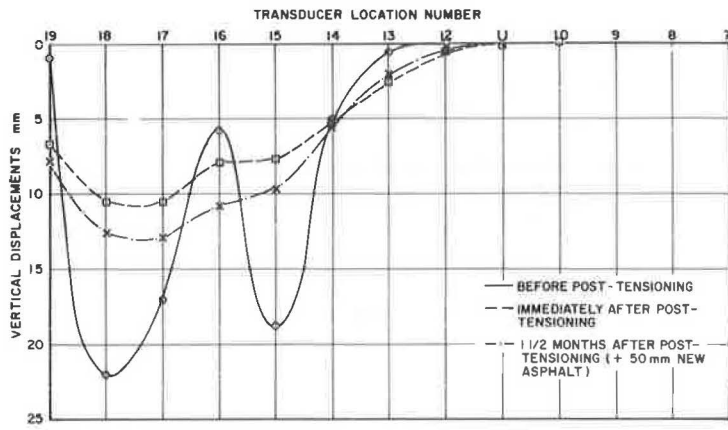


Figure 11. Vertical displacements at L/2 of north end span, load level 4, lane 1, longitudinal position 2.

Figure 12. Vertical displacements at L/2 of north end span, load level 4, lane 2, longitudinal position 2.

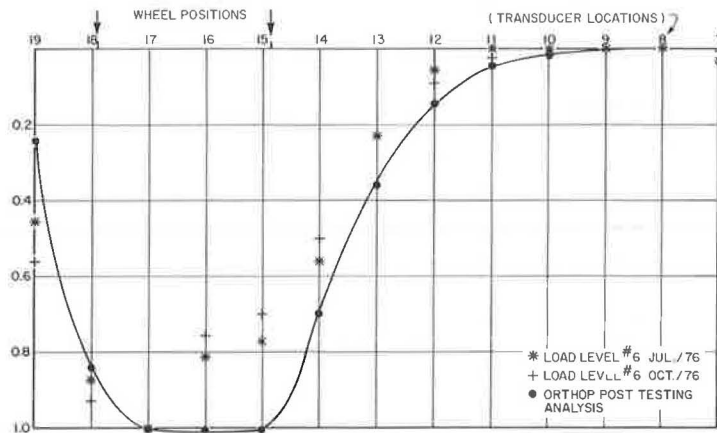
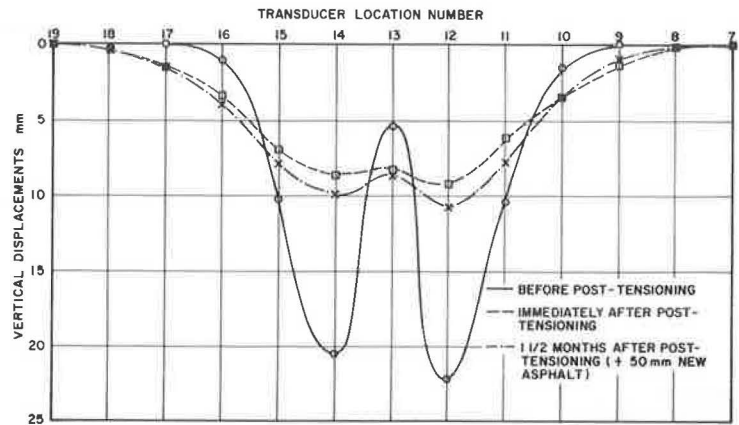


Figure 13. Normalized vertical displacements compared to an orthotropic analysis at L/2 of north end span, load level 6, lane 1, longitudinal position 2.

Figure 14. Normalized vertical displacements compared to an orthotropic analysis at L/2 of north end span, load level 6, lane 2, longitudinal position 2.

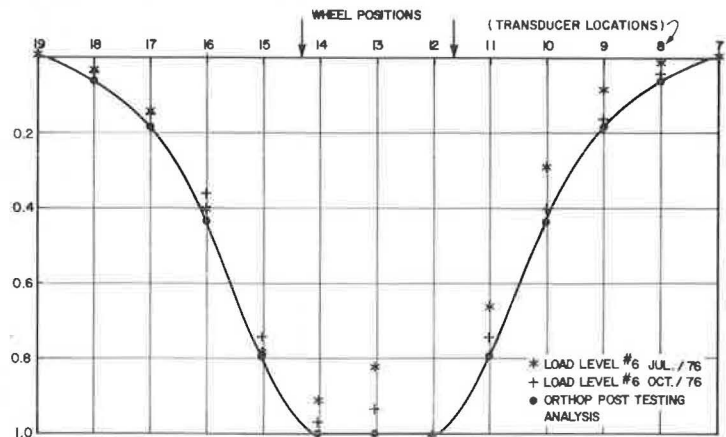


Table 1. Tandem weights of vehicle for testing.

Load Level	Number of Blocks	Vehicle Loading	
		Weight Rear Tandem (kg)	Weight Front Tandem (kg)
1	8	12400	12400
2	22	18900	19100
3	36	25700	25600
4	52	33100	33300
5	62	37800	37900
6	72	42600	42600

1 kg = 2.20 lb.

1 kg = 9.81 N

Table 2. Test results. Strain ($\times 10^{-6} \pm 10$) vehicle in longitudinal position 2 of lane 1.

Load Level	Demac Location					
	1	2	3	4	5	6
Before Post-Tensioning						
3	20	210	710	970	60	180
4	40	280	910	1140	180	270
5	30	310	1080	1470	—	—
After Post-Tensioning						
3	180	230	320	520	330	280
4	270	280	420	670	420	370
5	270	340	490	750	640	470
6	320	330	520	940	530	520

Table 3. Post-tensioning stress (kPa) on timber laminates during survey to date.

Survey	Station Number															
	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16
(1 day after)																
July 1976																
Cloudy																
21°C (70°F)	870	760	870	760	760	680	650	710	490	760	680	760	850	850	810	810
Oct. 1976																
Overcast																
16°C (60°F)	520	490	490	460	460	490	520	490	490	520	490	540	520	540	490	490
Jan. 1977																
Snowing																
-29°C (-20°F)	410	430	430	410	380	410	410	410	410	410	410	410	430	490	460	430
April 1977																
Very Wet																
18°C (65°F)	460	520	540	520	490	460	490	490	490	460	460	490	490	520	490	490
August 1977																
Dry																
21°C (70°F)	410	430	430	410	410	390	410	410	410	410	410	410	430	460	430	380
Nov. 1977																
Cloudy																
13°C (55°F)	410	430	430	410	420	410	420	420	420	410	410	430	460	480	460	410

1 kPa = 0.145 psi

transverse flexural stiffness considerably tended to produce closer agreement with the test results. It is believed that the transverse flexural rigidity is dependent upon the magnitude of the applied transverse moment. When high enough, this moment will begin to separate the laminations as the post-tensioning stress is overcome. This creates the need for a non-linear analysis which not only accounts for this changing transverse flexural stiffness, but also considers the longitudinal discontinuities produced by the butt joints. However, a non-linear analysis is considered beyond the scope of this report.

Although not acceptable for design, the orthotropic analysis does indicate that the actual longitudinal modulus of elasticity of the Hebert Creek Bridge was drastically overestimated in the pre-testing analysis. A more realistic value of 7.6×10^6 kPa (1.1×10^6 psi), as compared to the former value of 10.3×10^6 kPa (1.5×10^6 psi), was needed to produce the overall magnitude of the deflections that were obtained from the field test data. As well, the orthotropic analysis indicates that the maximum transverse shear stress, which was used in the post-tensioning design, was conservative.

Conclusions

The post-tensioning performed on the Hebert Creek Bridge was more than adequate to establish the structure as capable of safely carrying the Ministry's testing vehicle, which represents the heaviest commercial vehicle weights in the province. The flexural capacity, as indicated by the maximum strains in Table 2, was almost doubled. The maximum strain of 940 micro strain provides a stress of 7170 kPa (1040 psi), assuming a modulus of elasticity of 7.6×10^6 kPa (1.1×10^6 psi) as indicated in the previous section. This stress corresponds to an assumed distribution width of at least 1650 mm (66 in.) for flexural design of one line of wheels. This is over twice the design width of 810 mm (32 in.) specified by AASHTO (1),(2). Coupled with this, the maximum stress of 7170 kPa (1040 psi) is less than the original design strength of the timber of 8300 kPa (1200 psi), and yet was induced by a vehicle load which produces a moment 60% larger than the AASHTO design moment.

Existing nail-laminated structures can be adequately upgraded by post-tensioning with a system equivalent to that provided on Hebert Creek. The fact that the structure now becomes overdesigned cannot be accommodated. Only by increasing the spans of the structures, which would only be practical in new bridges, could the extra strength be utilized.

Using this system in new structures, the additional capacity could be utilized by increasing spans and maintaining the present deck thicknesses. This would be most beneficial where multi-span structures are encountered, as it would decrease the number of supports required.

Confinement of the deck system in Hebert Creek and the reduced deflections has virtually eliminated the passage of moisture through the deck. This was confirmed by observing the structure under various climatic conditions during the collection of the data given in Table 3. The reduced deflections lower the tendency of the asphalt to break up which resists the penetration of moisture to the deck below.

Future Research

One of the most important considerations is the development of an analytical understanding of the post-tensioned structure. Presently, an investigation is being formulated, in cooperation with Queen's University in Kingston, Ontario, to establish an acceptable design procedure for new bridges. The program includes the testing of small and large-scale models to establish a better understanding of the flexural and torsional properties of the post-tensioned deck system. As well, further creep tests are to be performed to determine the effects of environmental changes on the post-tensioning forces.

The local failures of the timber bulkhead due to rotation of the steel anchorage plates indicate that a practical design is required which ensures dimensional stability and resists rotation. It is hoped that a continuous anchorage system can be developed which does not require extensive labour or machinery for installation. Along with this, investigations are underway to determine whether a high strength tendon can be manufactured which would not require external protection. This would require less work to install and give better peace of mind, as the physical protection provided at Hebert Creek is never totally dependable.

Finally, tests are to be performed to produce acceptable friction coefficients for design of various timbers. The tests will subject the timber to different environmental conditions and preservative treatments aimed at representing the field conditions.

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