Ontario's Experience With Composite Wood/Steel Bridges

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Longitudinally laminated prestressed decks for bridges gained prominence in the late 1970's and have evolved into an effective structural system. It was similarly recognized that combining such a deck with steel girders could offer further advantages for longer span bridges. The concept of a composite wood and steel bridge was first reported upon by the Ontario Ministry of Transportation in 1986. Using shear bulkheads a longitudinally laminated deck was made composite with steel girders. Ontario has now constructed three full scale bridges using this technique. A number of unique details and improvements to the original concept have been developed.

In 1986 MT0 developed and tested a steel-wood composite bridge configuration [1]. The concept consisted of a longitudinally laminated prestressed wood deck made composite with steel girders through the use of shear bulkheads. These bulkheads were created by coring holes through the stressed wood deck directly over top of girders, welding shear studs to the girders and then filling the holes with fibre reinforced cementitious grout. (fig 1).

Having confirmed the functionality of this concept it was desirable to construct full scale applications. Between 1991 and 1994 three steel/wood composite bridges were built in Ontario--North Pagwachuan R., Aubinadong R. and Hoiles Cr.-- each with unique characteristics.

Although the original research dealt effectively with the theory of composite steel/wood design many other design details unique to this structural form had to be developed and a number of significant improvements were made through the course of construction.

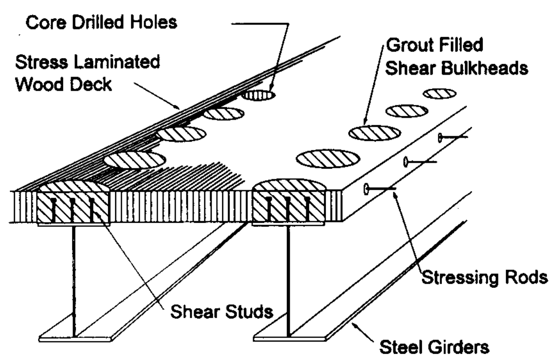

Figure 1 The concept of composite steel/wood design

North Pagwachuan River Bridge

Description

The North Pagwachuan River Bridge is located on Highway 11 in Northern Ontario approximately 400 km from the nearest major urban centre, Thunder Bay. The existing bridge had been scheduled for replacement by the Ministry of Transportation (MTO) due to its advanced state of deterioration. A problem encountered in the structural planning was that the fairly remote location of this bridge dictated that an alternative to traditional concrete and steel options be used.

It was decided that this project offered an excellent opportunity to employ the methods developed in the MT0 research [1] in a full scale prototype application for Ontario's first composite wood/steel bridge.

Soil conditions at the site precluded the use of intermediate piers resulting in a fairly long 50 m clear span. The bridge had a two lane cross section 12.4 m wide with 4% superelevation. The deck was 286 mm thick supported on 5--1800 mm deep girders. (Fig.2). It was constructed on the existing highway alignment necessitating a temporary detour and bridge to carry traffic during construction.

Superstructure Design

MTO's research [1] concluded that it was reasonable to assume that 50% of the deck acted compositely with the girders. This was of course dependant upon a number of factors including girder spacing, deck thickness and prestressing forces but was felt to be a reasonable assumption.

Because the system was a prototype and its long term effectiveness not proven, the superstructure was designed such that the girders acted compositely for serviceability loading only. At ultimate limit state however, the naked girder sections were designed to carry all loads irrespective of the deck. Conversely, as long as composite action existed the superstructure would be considerably over designed. It was recognized therefore that the true economies of the design could not be fully realized.

The bridge was designed using the methodology suggested in the research [l] and complied with the Ontario Highway Bridge Design Code (OHBDC) [2]. It was analyzed using the simplified methods of the code by idealizing the configuration as a two dimensional orthotropic plate. Plate properties were approximated based upon the material properties of the wood.

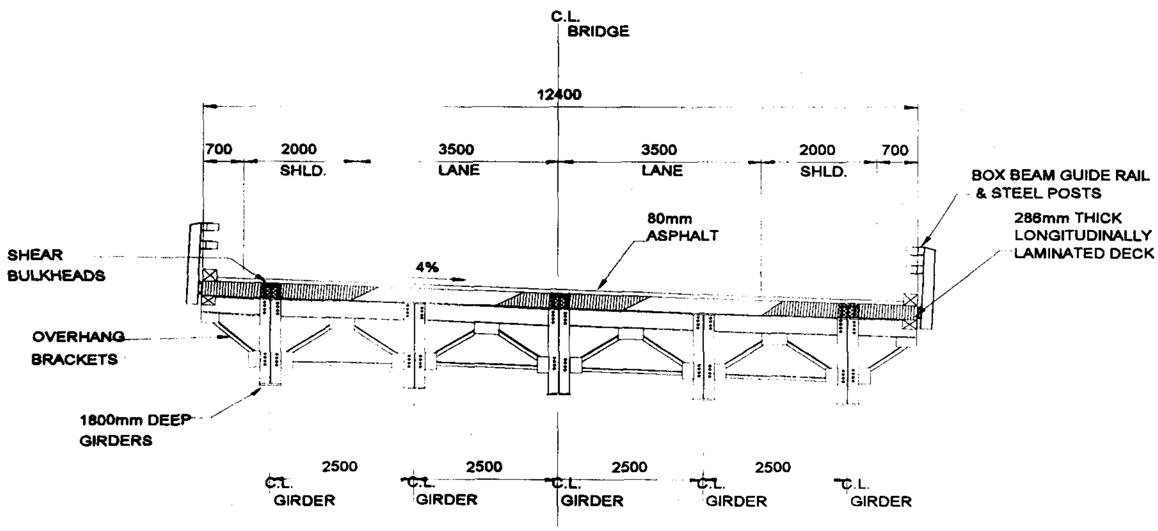

Figure 2 Cross section, North Pagwachuan River Bridge.

Decking
The deck itself was designed to span in the longitudinal direction supported on transverse diaphragms at 7.25 m spacings. No allowance was made for the support provided along the longitudinal girder lines because of the substantially lower transverse stiffness of the deck.

The material used for the deck laminates was 38 mm x 286 mm Douglas Fir, #2 or better. It originated in Oregon, was milled, fabricated and treated in Montreal, Quebec and shipped to Thunder Bay, Ontario. All sections were precut to lengths of up to 6 m and all stressing holes were predrilled prior to treatment.

It was necessary to minimize to number of butt splices in the deck and also to ensure that stress rod holes lined up after the deck was assembled. Therefore tolerances were built into all dimensions and the entire assembly pattern was laid out in plan form to assist the contractor during construction.

The deck was assembled on pre-erected girders by establishing a start point along the centre girder and attaching laminates in all directions from there. (The size of the bridge precluded pre-assembly.) The intention was to minimize the affects of dimensional tolerances at the edges since the final position of the deck relative to the girders was important. Still the deck width ended up being about 450 mm wider on each side than the theoretical width after it was assembled. This had been anticipated, and in order to further ensure that the deck ended up in the correct final location over the girders, one line of shear bulkheads was installed along the centre girder to act as an anchor prior to stressing (Fig 3). Also temporary support bracket extensions were attached to the permanent brackets along the edges to support the 450 mm overhangs (Fig. 4). All laminates were nailed together with 100 mm air driven nails at 600 mm spacings in order to assist construction.

In specifying a treatment for the wood it was decided that traditional creosote treatments were unacceptable in that creosote exudation which normally occurs would not be tolerated by environmental regulatory agencies. As a result it was decided to employ an experimental severe Reuping process to treat the wood. The objective was to obtain adequate penetration of the preservative but to minimize final retention.

At the time of treatment the process appeared to have had the desired effect [3]. However a delay in awarding the contract by one year necessitated

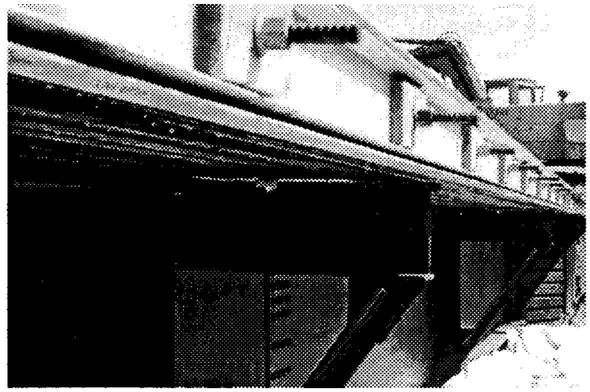

Figure 4 Overhang support brackets (after stressing).

storing the wood for one winter outside. The intense freezing weather had the effect of causing creosote to in fact exude the following year, when it was needed for construction. This made handling extremely difficult and uncomfortable in warm weather, thereby necessitating night time assembly of the deck.

A potential problem that became evident after the deck was assembled was that the longitudinal dimensional tolerance of the deck resulted in gaps of about 6 mm between all laminate butt splices. This was a concern because salt water would eventually percolate through the asphalt surface and flow between these gaps and onto the unprotected steel girders. A “quick fix” and effective solution was to have the contractor fabricate wood wedges out of the scrap material which could be drive into the gaps in order to plug them.

Shear Bulkheads
As had been anticipated the task of field core drilling 470--250 mm diameter holes for the shear bulkheads turned out to be arduous. A specially fabricated carbide tooth core bit was used but was found to generate considerable heat and slowed the process. The contractor found that the drilling time could be lessened and the bit life extended by drilling a serious of 25 mm diameter holes around the circumference of the bulkhead.

After coring The wood in the holes was subsequently treated with sprayed applications of creosote. While helpful, the long term effectiveness of this approach is uncertain. To further guard against ingress of water into these bulkheads a rubber membrane was bonded to the deck surface along each line of bulkheads.

Core drilling of the holes through the laminated deck had to take place after the first stage stressing (except along the centre girder) in order to ensure that they were located directly above the girders. Since there was already considerable transverse compressive stress in the deck prior to drilling of the holes the stress increase which resulted when material was removed at the holes caused some distortion and local crushing as well as unknown prestress loss.

After completing this process if became evident that the concept of field drilling the shear bulkhead holes was an area where considerable improvement could be made.

Deck Stressing

The OHBDC specifies demanding stressing cycles in order to mitigate prestress loss. This was however felt to be overly restrictive and costly for this project in that the stressing would have extended over a period of at least 5 weeks prior to which the final shear bulkheads could not be fabricated.

This requirement exists in order to mitigate prestress losses which occur shortly after initial stressing. These losses in turn decrease interlaminate shear in the deck which affects deck strength. Prestress losses are dependant upon a number of factors including wood creep related to the wood species and moisture content [4]. Notwithstanding, the sequence was felt to pose an unfair economic penalty on this system and would have to be relaxed were it to be developed into a competitive configuration. It was decided to alter the sequence such that only two stressing cycles were carried out--the initial (including a restress after 12 hours) and a final stress just ten days after the initial stressing. To counter the potential for prestress loss the initial force specified was higher than required and the wood was supplied with a very low moisture content. It was also recognized that should this approach not be effective the rods could be restressed in the future if necessary.

The presence of rigidly connected steel girders did complicate the potential restressability of the deck. Because the deck is rigidly fixed to the girders any compression of the deck due to restressing would be resisted by the diaphragms between the girders. Therefore to facilitate the level of movement necessary the diaphragm-to-girder bolted connections were made with slotted holes. In the event of the need to re-stress, the bolts could be loosened first.

A problem often encountered in stressed wood decks is the tendency of the deck to warp when stressed, due to the high transverse compression in the deck. This can be resisted by anchoring the deck to its support system. This potential problem was anticipated and anchors consisting of small clip angles brackets were included in the design. However, because of the higher than normal prestress forces, the problem turned out to be more severe than anticipated. Also, because the deck moved during stressing some hold downs rotated and therefor could not engage the diaphragm flanges properly. Some quick fix anchor designs had to be developed to overcome the problem. As well, at the approach ramps, where anchoring was difficult there was an even greater tendency to warp. The problem was eventually overcome by forcing the deck back down with a heavy weight and then securing it in place.

Railing System

Developing a railing system substantial enough to satisfy OHBDC [2] requirements proved challenging because of the relatively weak transverse strength of the deck. The code required the railing to sustain a static point load of 80 kn (performance level (PL) specifications did not come into effect in Ontario until later). It was again desirable to avoid the use of concrete. A wood railing was considered but rejected as a potential maintenance concern viz snow plows. A galvanized steel box beam railing was finally selected. It was mounted to steel W section posts which were fastened to the deck with a somewhat sophisticated arrangement of timbers, bolts and shear connectors modelled after a PL-1 wood railing connection [5]. A new innovation was the use of Ekki, a tropical hardwood, to both act as a snow plough “rub rail” curb and also anchor the railing posts and distribute forces throughout the deck.

The combination of potential out of plane transverse bending in the deck and the relative weak transverse strength necessitated special cantilever brackets on the outsides of the girders to support the 1.2 m deck overhang. (Fig. 4)

Substructure and Approaches

For the same reasons that concrete was being avoided in the superstructure it was also desirable to keep it out of the substructure. While the use of a crib type of substructure would have been preferred , very unstable soil necessitated the use of pile foundations. A fairly simple configuration of steel piles and timber lagging was developed (fig. 5).

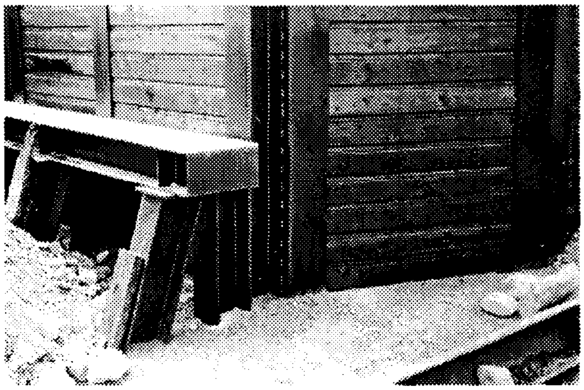

Figure 5 H-piles and timber lagging substructure.

The foundation piles were designed to function as a rigid A-frame which meant that thermal movement of the superstructure had to be accommodated. The girders were placed on elastomeric bearing pads and the wood deck was extended 6.8 m beyond the abutments to function as an approach ramp. Simple rubberized asphalt cycle control joints were placed at the end of the deck.

Aubinadong River Bridge

The Aubinadong River Bridge is located about 300 km north of Sault Ste Marie, Ontario and like North Pagwachuan is not close to a reliable source of concrete. The new bridge was located some 500 m downstream of the existing bridge at a location that required a clear span of 52 m.

Once again this presented an opportunity to use the composite wood/steel configuration. Having constructed North Pagwachuan the year before, a number of improvements were made to the design.

The basic concept remained unchanged, however significant modifications were made to a number of elements including the shear bulkheads, deck hold down system and stressing requirements.

Wood Deck

Instead of using Douglas Fir it was decided that this system would gain favour if it made use of a local species of wood, namely one that is native to Northern Ontario. The local treatable species with strength characteristics suitable for this kind of deck and in significant dimensions was Jack Pine. To that end a deck was designed using 38 mm x 235 mm laminates. The wood was harvested, fabricated and treated in Northern Ontario and shipped to the site. This time the material was treated in the same year as construction using the same reduced retention method that was employed at North Pagwachuan R. The end product was highly satisfactory in that there were no surface residues.

Shear Bulkheads

Some disadvantages of the drilled shear bulkhead included, concerns over long term durability, delays due to the slow field coring process and crushing of wood due to removal of large amounts of timber. It was felt that if the bulkhead openings were prefabricated then the shape, orientation and size could be designed to mitigate these problems. The experiences of North Pagwachuan showed that the amount of transverse deck movement that occurred due to elastic compression during stressing was very predictable. Of course a far greater amount of movement took place due to closing of spaces between laminates however since this totally disappeared during stressing it turned out to be irrelevant in determining where to locate the bulkheads. In theory therefore the bulkheads could be offset from the girders during deck assembly such that they were located directly over the girders, after stressing.

Since there were no longer any restrictions on bulkhead size and shape they could now be made short and wide in order to accommodate more shear studs while maximizing material between bulkheads. To further increase fabrication efficiency large dimension lumber was used between bulkheads instead of standard width laminations (fig. 6).

The new prefabricated bulkheads were now rectangular in shape measuring 380 mm wide by 250 mm long with up to 500 mm of material between them. There were also 12 shear studs in each bulkhead instead of 5 as at North Pagwachuan.

The new shear bulkhead design performed very well during construction. No crushing of wood occurred and the openings that were offset from the girders slid over into their predicted final positions during stressing. Finally, because the openings and therefore the wood along the bulkhead lines was all prefabricated, it had been creosoted in the plant and its longevity was no longer a concern.

Deck Hold Downs

Deck uplift had been a significant problem at North Pagwachuan and it was clear that more robust deck hold downs were required. The new ones were based on the same principals as had been used at North Pagwachuan however with the ability to grip into the wood in order to prevent twisting under movement. They were still attached to the diaphragms and facilitated lateral deck movement. These new hold downs performed well in that they seated properly and did not rotate during deck movement.

Serious uplift occurred at North Pagwachuan on the approach ramps where there was no simple way of anchoring the deck down during stressing. Since the uplift that occurred was largely a result of the high prestress forces (no uplift was observed until approaching final prestress force) it then occurred that a simple answer was simply to lower the forces at the bridge ends. This was possible because the approach ramps were actually supported by the granular subgrade underneath and therefore did not require a significant level of prestress.

The only drawback was a visual one in that because the wood did not compress as much there was a visual sweep along the edge of the deck. This essentially became hidden by the curb and guard rail system.

Deck Stressing

The stressing cycle was further modified in that the contractor was encouraged to monitor the stress level in the rods after stage one and undertake stage two once the level had dropped to about 50%. This reduced the 10 day wait period by a couple of days and was felt to not adversely affect the ultimate prestress loss.

It was still necessary to accommodate possible future restressing of the deck. However the use of slotted holes in all diaphragm connections was felt to be too costly and excessive given the minor elastic movements that were expected. It was decided that oversized holes in the diaphragms would allow adequate movement at a much lower cost.

Deck Drains

North Pagwachuan did not have deck drains and because of the cross fall and longitudinal profile this has not been a problem. Aubinadong R was constructed on a horizontal tangent with little vertical slope and as a result it was determined that deck drains were needed. A simple design was used which involve drilling an oversize hole through the deck and grouting in a steel pipe. The one drawback that is seen is the potential for premature decay due to the field drilling. A prefabricated opening would have been more desirable.

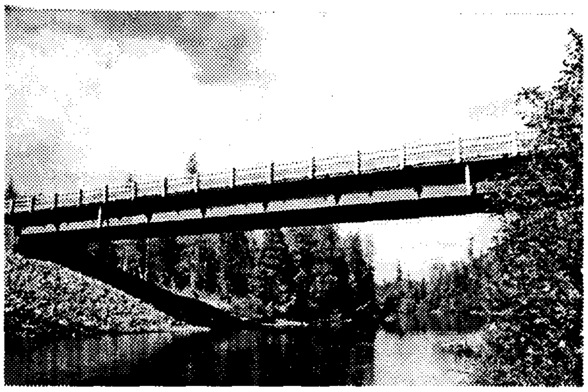

Figure 7 Completed Aubinadong River Bridge.

Hoiles Creek Bridge

The Hoiles Creek Bridge is located about 25 km east of the North Pagwachuan R. Bridge on Highway 11 and therefore once again it was desirable to avoid the use of concrete. It was replaced in 1994 at the same time as the Aubinadong R. Br. This structure was also a single span but was considerably smaller than the other two at 30 m. The cross section dimensions were the same as at North Pagwachuan.

Most of the modifications that were incorporated into the Aubinadong R. Br. design were also made to the Hoiles Cr. A major difference was that this site presented an opportunity to stage construct the bridge and determine the effectiveness of this system in avoiding a separate detour. It was estimated that savings of up to $250,000.00 could be realised.

Staged construction for plain stress laminated decks was first carried out by MT0 in 1980 at the Sioux Narrows Bridge [6] as well as a number of times since. The primary difference from traditional stressed deck construction is that a bulkhead is required at the centre of the bridge against which the first half of the bridge is stressed. This bulkhead must then be incorporated into the second half of the structure. Of course the added complication at Hoiles Cr. was that the deck had to be composite with girders after the first stage.

In previous designs a steel channel stressing bulkhead was used at the stage point, necessitating complicated and costly detailing. Ekki timber was felt to offer the same rigidity as the steel channels while allowing for simpler detailing (fig. 8,9).

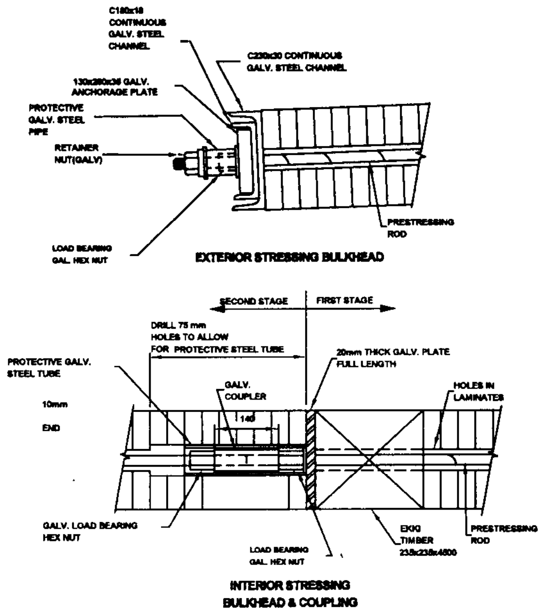

Figure 8 Stressing bulkhead details.

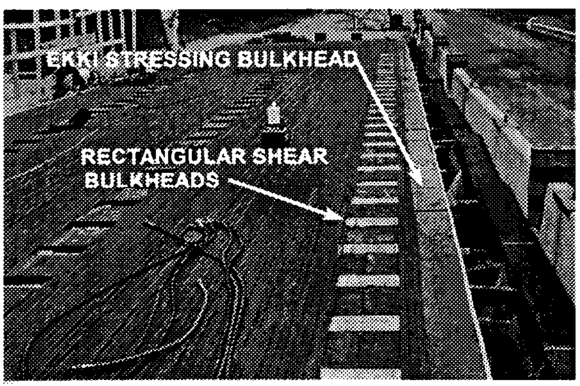

Figure 9 First Stage of deck prior to stressing. Note Ekki stressing bulkhead on right and rectangular shear bulkheads in deck.

Summary

Composite steel/wood was proven to be an effective configuration particularly at remote locations. Costs were found to be comparable with steel/concrete construction.

The design must be made more cfficicnt in order to achieve maximum benefit from the composite action and make the system competitive with more traditional bridge configurations. Full composite action at ULS must be accepted.

Changes made in the design of the shear bulkheads by prefabricating rectangular shapes have gone a long way towards improving their longevity and simplifying construction.

The system has proven itself as particularly adaptable to staged construction.

Improvements in stressing systems which minimize prestress loss would be of particular benefit here in that allowance for future re-stressing could be dispensed with.

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