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Prototype Prestressed Wood Bridge

R.J. TAYLOR and H. WALSH

ABSTRACT

The transverse prestressing of wood was conceived of in 1976 as a method for rehabilitating nailed laminated wood decks. Using high-strength prestressing steel, a permanent pressure is introduced normal to the direction of the laminations to provide high interlaminate shear strength and improved load distribution. The success of this new concept in rehabilitation resulted in its becoming the subject of a major research and development program conducted by the Ontario Ministry of Transportation and Communications (MTC). The extensive work performed by MTC over the past 7 years has led to the formulation of a set of comprehensive design specifications for prestressed wood. The objective of this paper is to outline the design, construction, and load testing of the world's first new prestressed wood bridge. The bridge was designed by MTC and constructed by the Ontario Ministry of Natural Resources (MNR) over the West River, on a logging access road, near Espanola, Ontario, in 1981. The design process with reference to the new design specifications, which have since been adopted by the Ontario Highway Bridge Design Code, is discussed. The field construction is outlined highlighting the prefabrication and assembly of the prestressed wood superstructure. The load testing of the bridge in 1982 and the subsequent evaluation of the test results are described. The MNR determined that the West River bridge cost only two-thirds of the steel structure originally proposed for that site. The load testing and subsequent evaluation indicated that this prestressed wood bridge is an extremely rigid structure with considerable reserve strength.

The transverse prestressing of laminated wood decks was conceived of in 1976 (1) as a method of rehabilitating existing nailed decks. The success of this new concept in rehabilitation resulted in a major research and development program (2) conducted by the Ontario Ministry of Transportation and Communications (MTC). Extensive research and development work led to the formulation of a comprehensive set of design specifications (3,4) devoted entirely to the design of prestressed wood decks. These new specifications have been included in the 1983 edition of the Ontario Highway Bridge Design Code (OHBDC) (4).

To evaluate the effectiveness of these new specifications, MTC and the Ontario Ministry of Natural Resources (MNR) designed and constructed the first new prestressed wood bridge in 1981. The objective of this paper is to outline the design, construction, and load testing of this prototype prestressed wood bridge.

The design analysis, with reference to the new OHBDC specification, and several computer analysis techniques are described in this paper. The fabrication and erection procedures are also outlined with particular emphasis on the field construction conducted by the MNR field construction crew. The load testing and subsequent evaluation of the completed bridge, performed by MTC in 1982, are also summarized.

STRUCTURAL DESCRIPTION

The main objective of the structural selection was to optimize the use of the prestressed wood concept while minimizing on-site construction requirements. The use of this prototype to demonstrate the design flexibility of the prestressed wood system was of secondary importance.

The bridge is located on the MNR Fox Lake logging access road near Espanola, Ontario. It is believed this bridge, which spans the West River near the southern highway entrance, will be subjected to some of Ontario's heaviest commercial loadings.

The site, shown in Figure 1, consists of about a 7.0-m-wide fast-moving waterway surrounded by large bedrock formations. A typical elevation at the narrowest crossing is shown in Figure 2 along with the proposed structural form. To avoid the costly removal of the surrounding bedrock and limit the fill requirements the economic structure length would have to be about 13 meters. To satisfy the site requirements, while demonstrating the flexibility of the prestressed wood system, the wood frame structure shown in Figure 2 was proposed. This structure uses inclined legs providing a clear opening of 7.7 m. It maintains the required bridge elevation with minimum foundation requirements. It also takes advantage of two naturally formed rock ledges situated symmetrically on either side of the waterway.

The legs and the deck were to be constructed as an integrated, prestressed, laminated system. The individual leg laminations were to be prespliced to the deck laminations using galvanized nail-plate connectors. The prefabricated frames were then to be shipped to the site and assembled, and the entire structure was to be prestressed to form a continuous, rigid-frame structure.

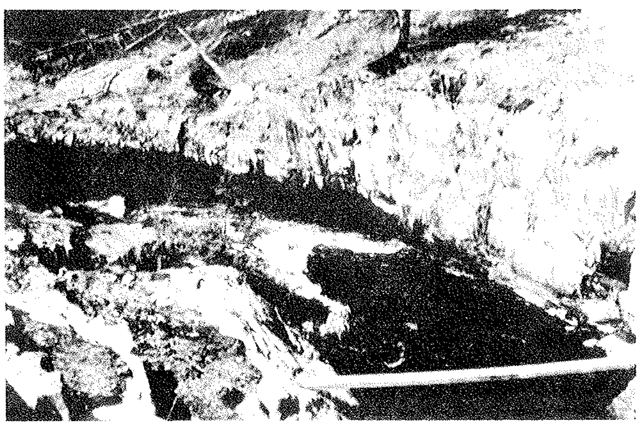


FIGURE 1 Proposed West River crossing.

DESIGN

The OHBDC specifications do not cover some of the requirements for the design of this frame structure, particularly those of load distribution. Therefore a number of computer analyses are presented as well as some laboratory testing of the deck-leg connection.

A preliminary evaluation indicated that using Ontario red pine graded No. 2 and better would require laminations of 38 mm x 292 mm for the deck and 38 mm x 190 mm for the legs. In addition, only one-half of the deck laminations would require full leg supports in terms of moment and axial capacity. The latter was to be achieved by spacing groups of four laminate frames as shown in Figure 3. Some additional construction details are provided in Figure 4.

The deadline imposed for MNR construction resulted in the commencement of the bridge, based only on a preliminary analysis. Because of its availability Douglas fir was substituted for red pine, so the following analyses and evaluation are based upon Douglas fir graded No. 2 and better.

Analysis

Because of the structure's low span-to-depth ratio it was believed that simplified static analysis would not properly represent the distribution of load in the structure. However, the application of a costly three-dimensional analysis was not considered practical. Instead, two complementary two-dimensional analyses were performed.

The first, a two-dimensional frame analysis program developed by R.K. Ayres as an MTC research project in 1975, represented the structure in elevation and so did not consider any lateral distribution of load. This provided maximum moments, shears, and reactions per line of wheels of the design vehicle.

The second analysis represented the deck in plan as an orthotropic plate (5) and simulated the legs as flexible columns. This type of analysis had

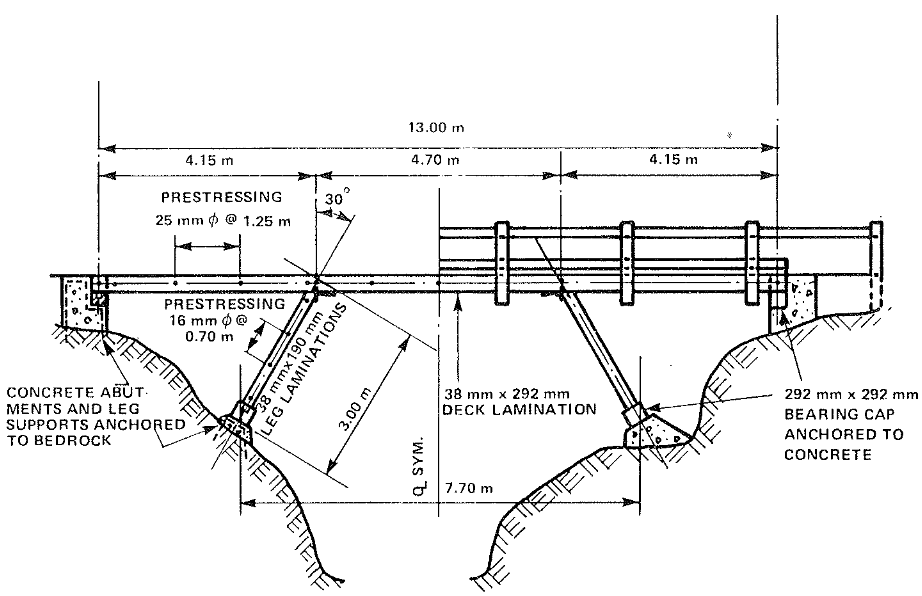


FIGURE 2 Elevation of Fox Lake Road bridge.

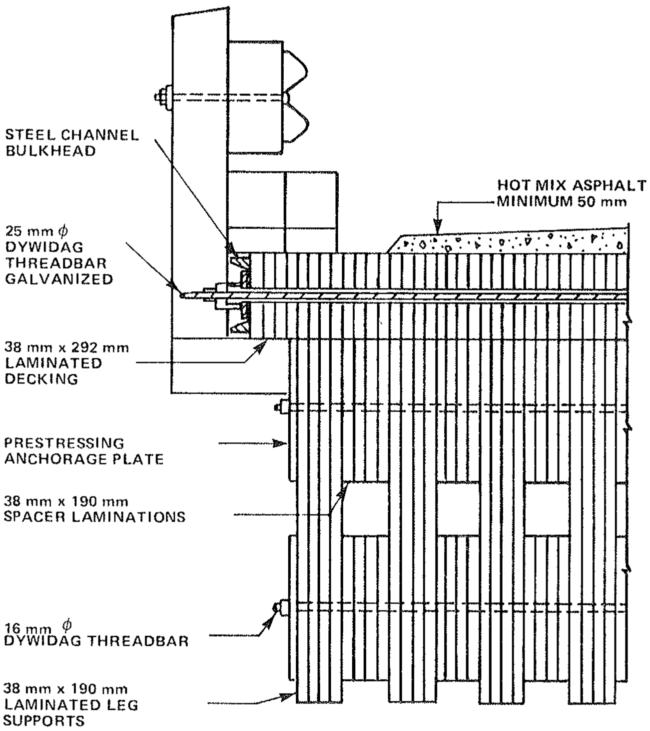


FIGURE 3 Partial section at leg.

already proven to be a fairly accurate representation of prestressed wood decks (1,2) and so was used to derive the distribution criteria needed to produce the design forces.

Frame Analysis

Figure 5 shows the frame arrangement used to represent the structure for analysis. The analysis was performed producing influence lines under the effects of a 100 kN wheel load. The influence lines were later used to determine the maximum forces under the effects of the OHBDC design vehicle.

This analysis was also used to determine dead load forces and vertical movement of the deck at the leg support under live load. The latter information enabled the determination of a representative support flexibility for use in the subsequent orthotropic analysis, where the leg supports were simulated by flexible columns.

Orthotropic Analysis

The orthotropic analysis represents the prestressed deck as a two-dimensional plate with flexural and torsional properties that may be different in two directions. The determination of rigidities for the analysis was based on the idealizations presented in the analysis section of the OHBDC (4).

Figure 6 shows the geometry and boundary conditions used to represent the structure in the orthotropic analysis. The abutment supports were repre-

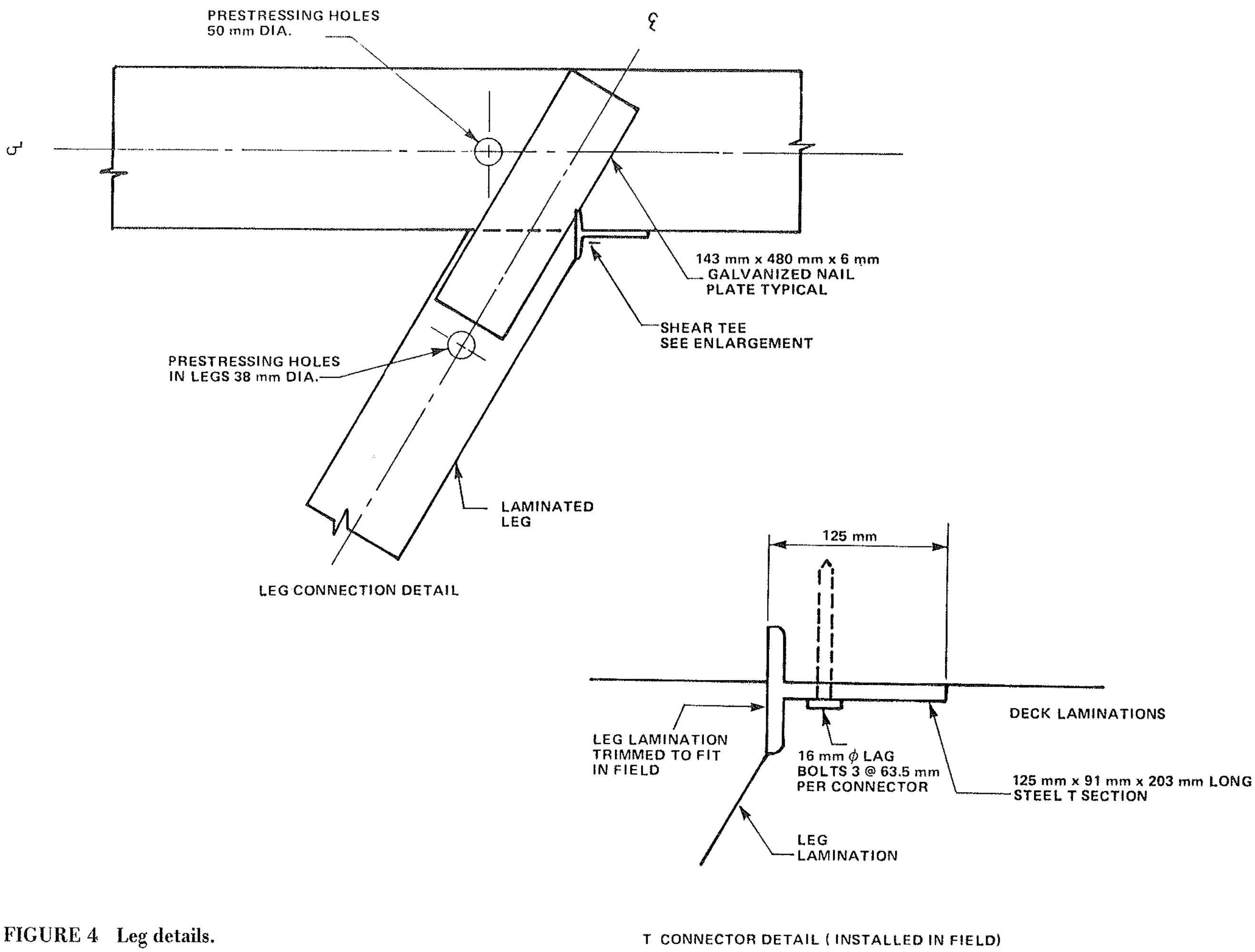


FIGURE 4 Leg details.

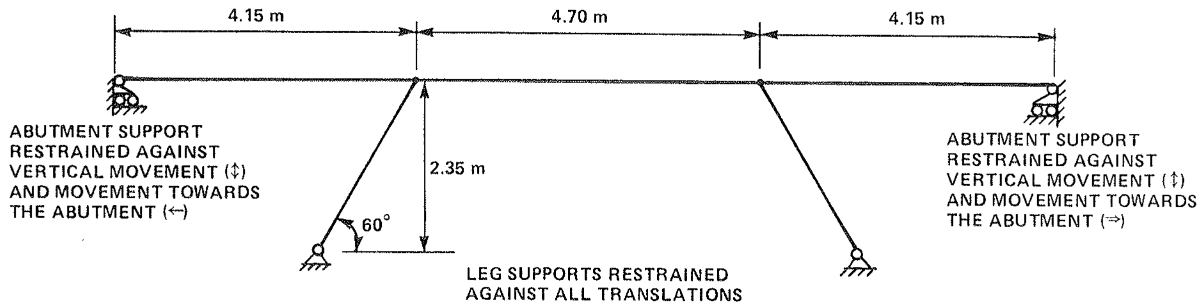


FIGURE 5 Geometry assumed for frame analysis.

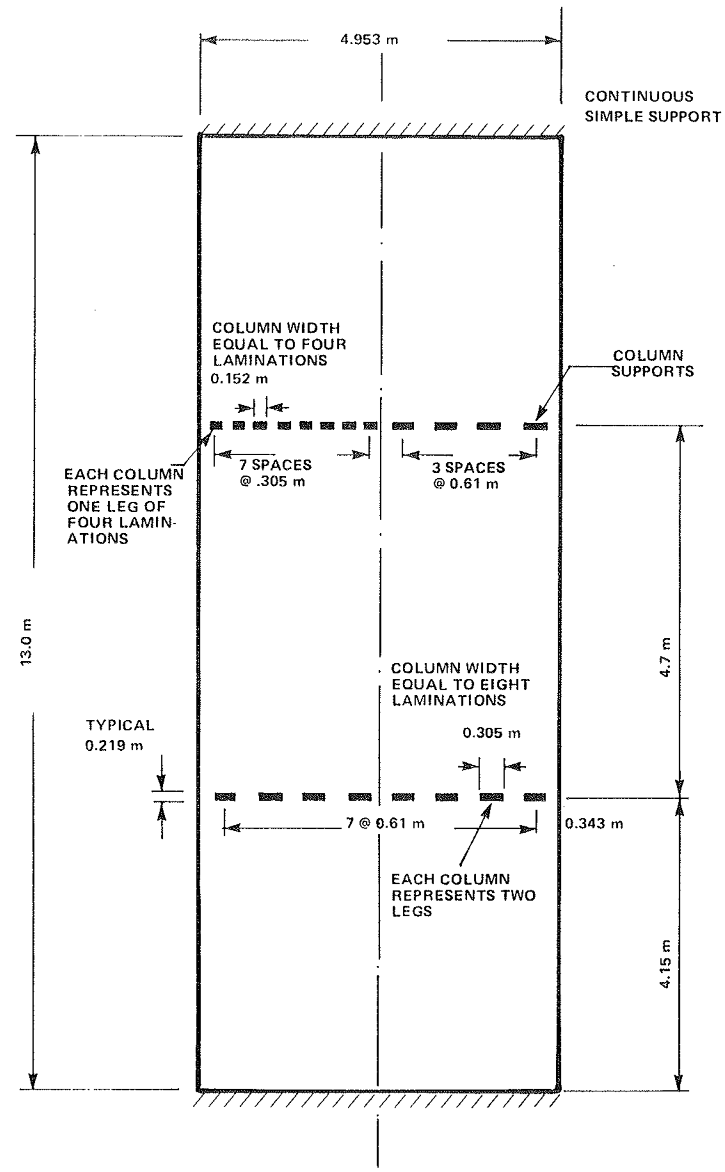


FIGURE 6 Orthotropic deck layout.

sented by line supports and the legs by flexible columns. Because of the limited number of columns that can be given in this program, only one-half of one support was represented in detail. Each leg, consisting of four laminations, was simulated by a column with dimensions equal to the horizontal projection of the leg-deck connection. All other columns represented two legs combined.

The orthotropic analysis was used to produce influence lines as a means of determining the critical vehicle placement. The unit load was a 100 kN axle with dimensions representing the OHBDC design vehicle. All the force modes considered in the design of the bridge are listed in the following table.

Structural Component	Force Mode
Deck	Moment[a], vertical shear, interlaminate shear, axial force[a], and bearing
Leg	Moment[a] and axial force[a]
Leg-deck connection	Moment[a], horizontal shear[a], and bearing

[a]Evaluated as combined effects

Because the analysis did not properly represent the leg supports, the determination of moments and shears in the leg was based on the following assumptions. The distribution of load used to determine moment at the top of the leg was assumed to be the same as for negative moment in the deck over the leg, as determined from the orthotropic analysis. The distribution of load used to determine horizontal shear in the leg-deck connection was assumed to be the same as that for the maximum column reaction determined from the orthotropic analysis. These distribution widths were then applied to the undistributed leg moment and horizontal shear as determined from the frame analysis program.

OHBDC Specifications

MTC Report SRR-83-03 (3) details the design of prestressed wood bridges using the OHBDC specifications. Except for the case of the deck-leg connections, which will be discussed separately, all design modes for this bridge followed a similar design process. Therefore only the design for maximum positive moment is presented as an example of the design process using the new OHBDC requirements.

Design for Maximum Positive Moment

The following is a sample of the design calculations for maximum positive moment based on the OHBDC specifications (3). (OHBDC clause references are given in parentheses for possible future reference.)

Factored moment capacity is expressed:

$Mu = f_{bu}\ S\ k_d\ k_m\ k_s$ (13.22.6.)

ϕ = performance factor = 0.9 (13.4.4.)

f_{bu} = specified strength = 10.0 MPa (13.11.2(a).)

S = section modulus = $bd^2/6$ = 0.75 $[(1000 \text{ mm} \times 292^2)/6]$ mm

The maximum moment was assumed to occur at a transverse line of butt joints. Therefore the section modulus considers every fourth lamination to be discontinuous.

$S = 10.66 \times 10^6 \text{ mm}^3$

k_d = duration of load effect = 1.0 (13.5.3.)

k_m = load sharing effect = 1.5 (13.5.6.)

k_s = size effect = 1.0 (13.6.2.1.)

Therefore

$Mu = 0.9(10.0)\ (10.66 \times 10^6)\ (1.5)$

and

factored moment capacity $Mu = 144 \times 10^6$ N•mm/m width

Total factored load effect is expressed:

Maximum unfactored live load moment M_{LL}

M_{LL} = 64.6 kN•m/m width

Corresponding dead load moments M_{DL}

asphalt M_{DL1} = 2.96 kN•m/m width

wood M_{DL2} = 2.91 kN•m/m width

Load factors at ULS α (Table 2.5.1.b)

live load and dynamic load allowance $\alpha_L = 1.4$

dead load (asphalt) $\alpha_A = 1.5$

dead load (wood) $\alpha_W = 1.2$

For dynamic load allowance, DLA, a single axle governs maximum positive moment. Therefore

DLA = 0.4 (2.4.3.2.3.)

DLA must be adjusted by a factor of 0.7 for wood components (2.4.3.2.10). Therefore

DLA = 0.7(0.4) = 0.28

Total factored load effect M_{TOT}

$$M_{TOT} = \alpha\ M_{DL1} + \alpha_W\ M_{DL2} + \alpha_L\ (M_{LL})\ (1 + DLA)$$
$$= 1.5(2.96) + 1.2(2.91) + 1.4(64.6)\ (1.28)$$

M_{TOT} = 124 kN•m/m width

The factored moment capacity, Mu = 144 kN•m/m width, is 16 percent greater than the total factored load effect, M_{TOT} = 124 kN•m/m width.

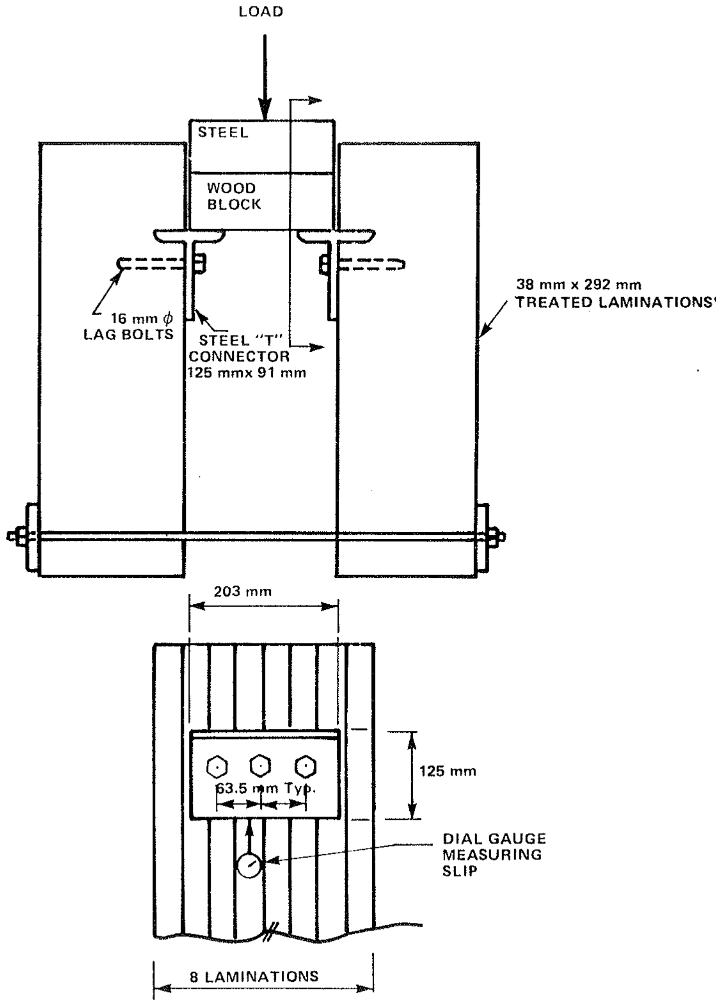
Deck-Leg Connection

The as-built leg connection is shown in Figure 4. Originally only the galvanized nail plate was to be used. According to the analysis and subsequent evaluation using the OHBDC specifications, the moment and shear capacity of the nail-plate connectors were more than adequate at the ultimate limit state. However, concerns were raised about the durability of the nail plates alone under repeated loadings. Therefore the additional steel T shear connector shown in Figure 4 was proposed. This type of connector had already proven to be resilient to the effects of repeated loads in part of another MTC research project for the development of a steel-wood composite (6).

Because of the deadline for construction and the uniqueness of this particular bridge, extensive development of the connection detail was not considered practical. However, some ultimate static testing was performed in an effort to establish the mode of failure and the ultimate strength of the proposed connection.

The first test was performed on the steel T section alone, as shown in Figure 7. The typical load-versus-slip response is shown in Figure 8 indicating linear performance up to about 200 kN. This would represent 100 kN per steel connector, one of which is used per leg. The total factored horizontal shear from the design analysis is only 48 kN per leg; therefore the connector is more then adequate.

Subsequently a full-size connector test was con-



* Test Material Obtained from Actual Bridge Supply

FIGURE 7 Test set-up for shear connector.

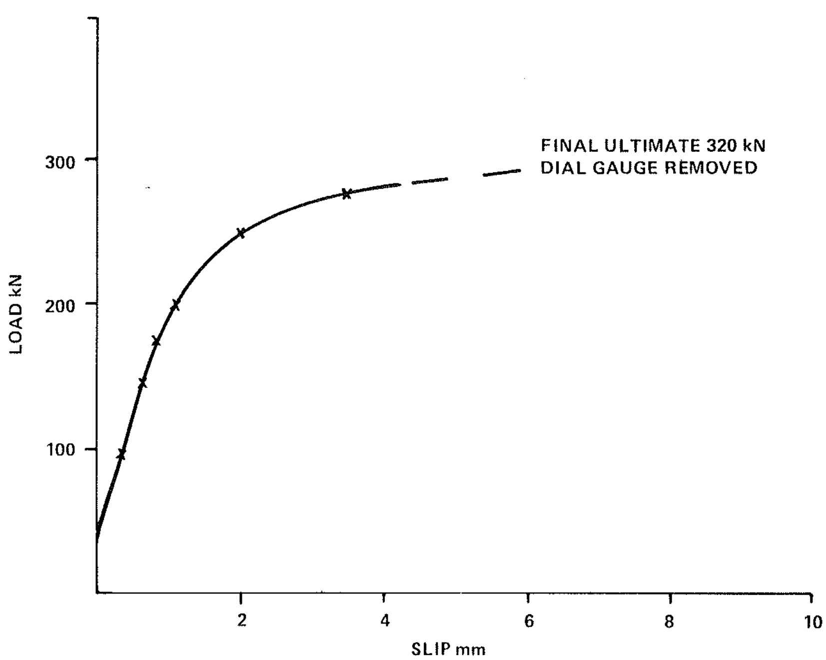


FIGURE 8 Load versus slip of shear connector.

ducted with both the nail plate and the steel T section installed. The ultimate failure mode was in bearing perpendicular to the grain of the wood deck as shown in Figure 9. The load at which the first sign of failure occurred was 385 kN per leg, and the ultimate crushing load was 508 kN per leg. No measurable slip movement of the steel T connectors occurred. The 385 kN would represent a horizontal shear capacity of about 190 kN per leg and 330 kN per leg in bearing--a value far in excess of that required.

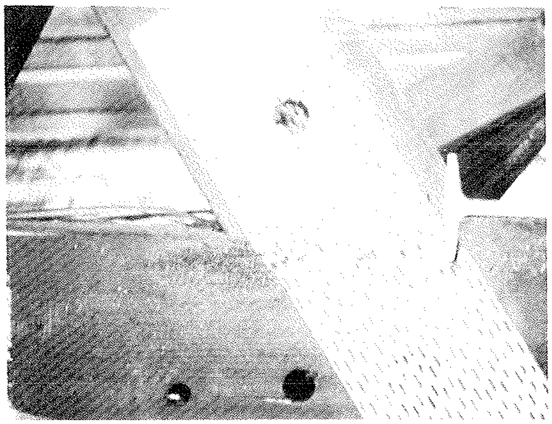


FIGURE 9 Failure of leg connection.

Scaling down the leg connection for more efficient material use was not considered justified. The nail-plate connectors were already as thin as practical, and a reduction in the steel T would not represent any significant savings because of the small number used.

Summary

The moment capacity as determined in the design example accounts for the occurrence of butt joints in the deck. Actually, these joints could be ignored because they have been spliced with nail plates and because they are not in the highest moment region. This would increase the total factored moment capacity from 144 kN•m to 192 kN•m. This is over 50 percent greater than the total factored load effect. Because of the substitution of Douglas fir for the originally proposed red pine, all of the design modes were determined to be conservative.

CONSTRUCTION

The fabrication and construction of the bridge were carried out by the MNR field construction crew under occasional supervision of the MTC design engineer. The crew consisted of three experienced construction people. This crew was, for a period of several weeks, supplemented by as many as three additional men who were available from other local MNR forces.

All steel hardware, including the nail-plate connectors and prestressing materials, were hot-dip galvanized for protection. All the wood materials, including the curbs, posts, and bearing caps, were cut and drilled before undergoing pressure preservative treatment with creosote. Only the holes in the deck, necessary to attach the deck to the supports and the curbs and posts to the deck, were drilled on site. These areas were surface treated according to the requirements of the OHBDC specifications.

Foundations

The abutments and footings were cast in place on sound bedrock and were anchored to the bedrock by grouted reinforcing tendons. This enables the structure to resist the negative reaction forces and high longitudinal forces that this light frame structure exhibits. Its dead load is very low so, in contrast to continuous structures of steel and concrete, the negative live load reactions are not overcome by gravity.

Solid sawn 290 mm x 290 mm bearing caps were tied down to the concrete supports as shown in Figure 10. This provided flexibility for the tie-down of the laminated deck and legs with lag bolts after prestressing had been performed.

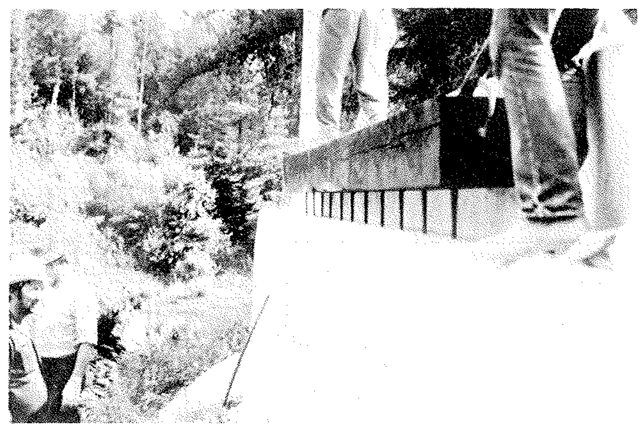


FIGURE 10 Wood bearings anchored to concrete supports.

Fabrication and Erection

The deck and leg laminations were fabricated at a nearby MTC yard to form the required frame geometry. First the deck laminations were spliced to form full-length laminations equivalent to the overall bridge length. Subsequently the legs were attached using a marked template to ensure proper locations and angle. When completed the individual laminate frames, ready for assembly, were shipped to the site. The prefabricated units were lifted into place by crane and nailed together to maintain alignment before prestressing. The completed wood frame structure is shown in Figure 11 with the prestressing bars and steel bulkheads installed and ready for prestressing.

Prestressing was performed using a new 24-jack hydraulic system assembled by MTC. This enabled the prestressing of the entire bridge to be done at the same time. The completed bridge, shown in Figure 12, was opened to traffic in the fall of 1981 and has since been used consistently by the local heavy logging traffic.

LOAD TESTING AND EVALUATION

In 1982 MTC performed a live load test on the bridge as part of the evaluation of its structural performance. The primary objective was to develop a better understanding of structural response in order to evaluate the analytic model used in the design. In addition, using MTC's special load testing vehicles, the bridge would be proof tested to a static load of more than two and one-half times the legal load.

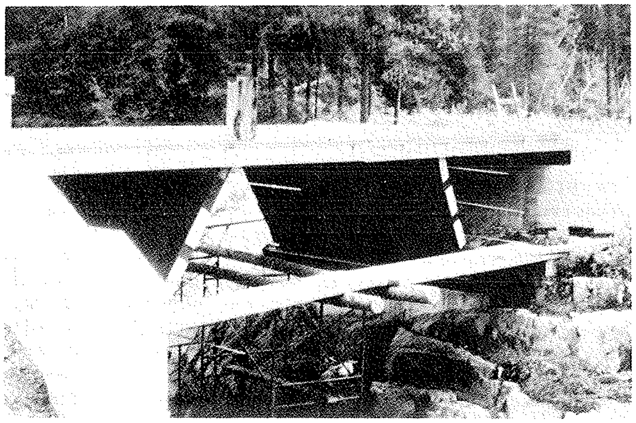


FIGURE 11 Assembled structure before prestressing.

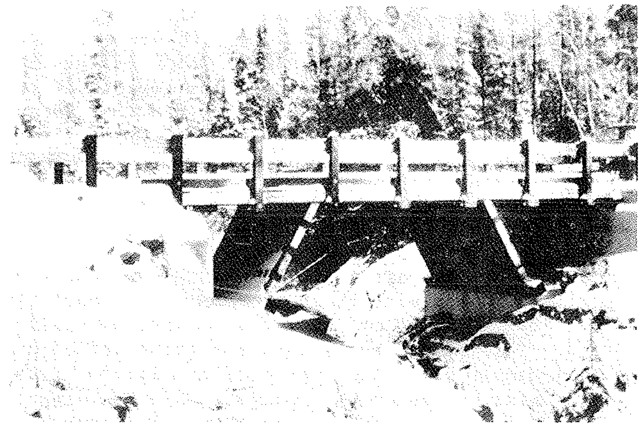


FIGURE 12 Completed structure opened in 1982.

Instrumentation

The bridge was instrumented primarily with linear displacement transducers, located as shown in Figure 13, to monitor the vertical displacement contour of the deck under load. A number of transducers were also installed to measure relative movement of the deck at the abutments and at the leg-deck connections, but these gauges registered no appreciable movements.

Nine demountable strain gauges developed at MTC's Testing and Instrumentation Laboratory (7) were also used to measure extreme fiber flexural strain in the deck. These were located immediately adjacent to the nine displacement transducers in the center span of the bridge.

Vehicle Loading

Using the influence lines produced by the design analysis, a number of critical vehicle positions were selected for one of MTC's load testing vehicles. The vehicle was moved through two transverse positions that represented an extreme eccentric lane loading and a concentric lane loading. In each lane the vehicle stopped at five longitudinal positions, providing a total of ten different static positions of the vehicle. At each of these static positions, all the instrument readings were recorded.

Load Testing

Load testing of the bridge was conducted without major difficulties and a maximum tandem loading of 42.5 kg (425 kN) was successfully applied. However, during the test it became apparent from visual observation that the leg supports were not resting evenly on the bearing caps. In fact some movement of leg was required before the support legs became fully engaged. Four additional displacement transducers were installed at the bottom of the legs to monitor this movement. Although the movement was only about 3 mm, its importance, as will be discussed in the next section, is reflected in the fact that the overall maximum vertical deflection of the bridge was only 9 mm.

Evaluation of Load Test Results

A preliminary review of the load test results indi-

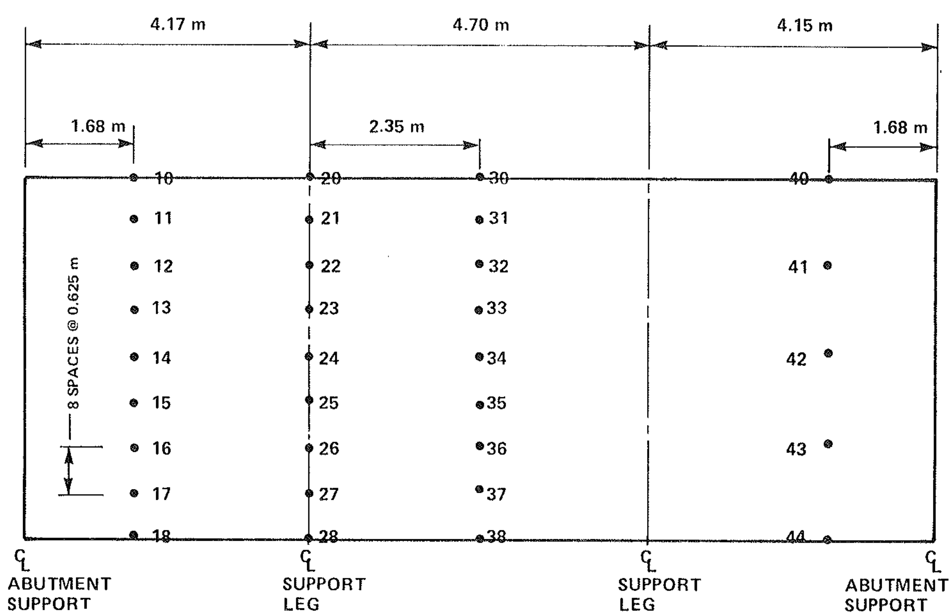


FIGURE 13 Plan layout of deflection transducers.

cated that the vertical deflection contour and the flexural strains would provide excellent data for evaluating the analytic model used in design. However, the movement of the legs would affect the overall deflection contour and subsequently the distribution of load. Therefore it would be necessary to account for this movement in the analytic model. This, in turn, would require better knowledge of the material stiffness.

Modulus of Elasticity

Usually, when comparing the deflection contours of experimental data with those produced by an analytic model, the comparison is made using normalized curves. This procedure offsets the error introduced by assuming a value for the modulus of elasticity, but it also assumes the structure behaves elastically. Because of the movement of the leg supports, this structure did not perform elastically. Therefore it was necessary to consider the measured leg displacements in the analysis and use a realistic value for the modulus of elasticity.

A sample of 17 full-size deck laminations from the actual treated material supplied for the bridge was retrieved at the time of construction. This made possible the determination of a representative modulus of elasticity for use in the analysis. The average modulus of elasticity, which was utilized in the subsequent computer analysis, was about 13 300 MPa. This average ignores the highest and the lowest measured values. The coefficient of variation was about 27 percent.

When reviewing the subsequent deflection and strain comparisons, this wide variation in material property should be kept in mind. The analytic model assumes uniform properties where in reality the distribution and magnitude of the material properties are quite varied.

Analytic Design Models

The bridge was initially reanalyzed using the same orthotropic analysis that was used in the original design described earlier. Only the loading was changed to represent the actual test vehicle used in the field. This analysis was designated orthop 1, and some results compared with the experimental data are shown in Figures 14 and 15. These figures display deflection contours at the midspan cross section under the eccentric and the concentric loading condition. In both cases the experimental results are of greater magnitude than those of the orthop 1 analysis. Though the analysis provides for some elastic shortening of the leg support, it does not account for the magnitude of movement that was measured in the field. Therefore the apparent deflections, as demonstrated by the experimental results, are much greater. The same is true for the longitudinal deflection contour shown in Figure 16.

The orthotropic analysis was then repeated, introducing prescribed settlements for the columns based upon the deflections measured at the west leg. This analysis was designated orthop 2. Because only the west leg was fully instrumented, as shown in Figure 13, it was not possible to represent all of the load positions. However, for the symmetrical loading with the tandem axle at the center of the

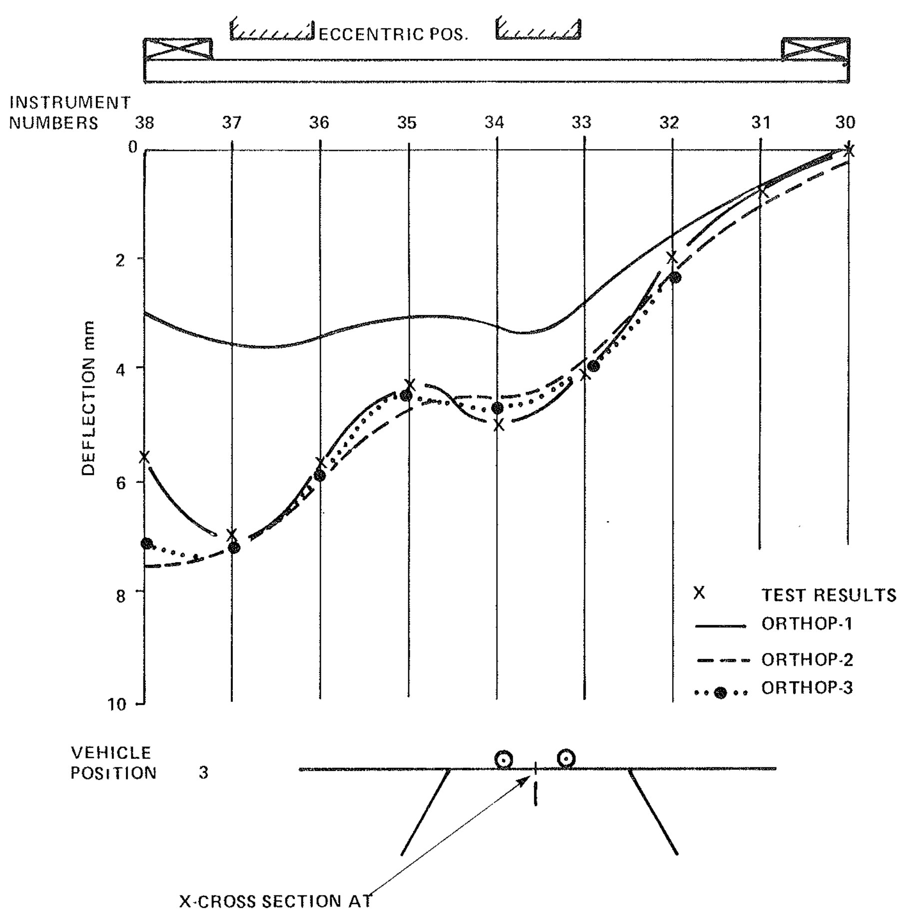


FIGURE 14 Vertical displacements, center span, eccentric loading.

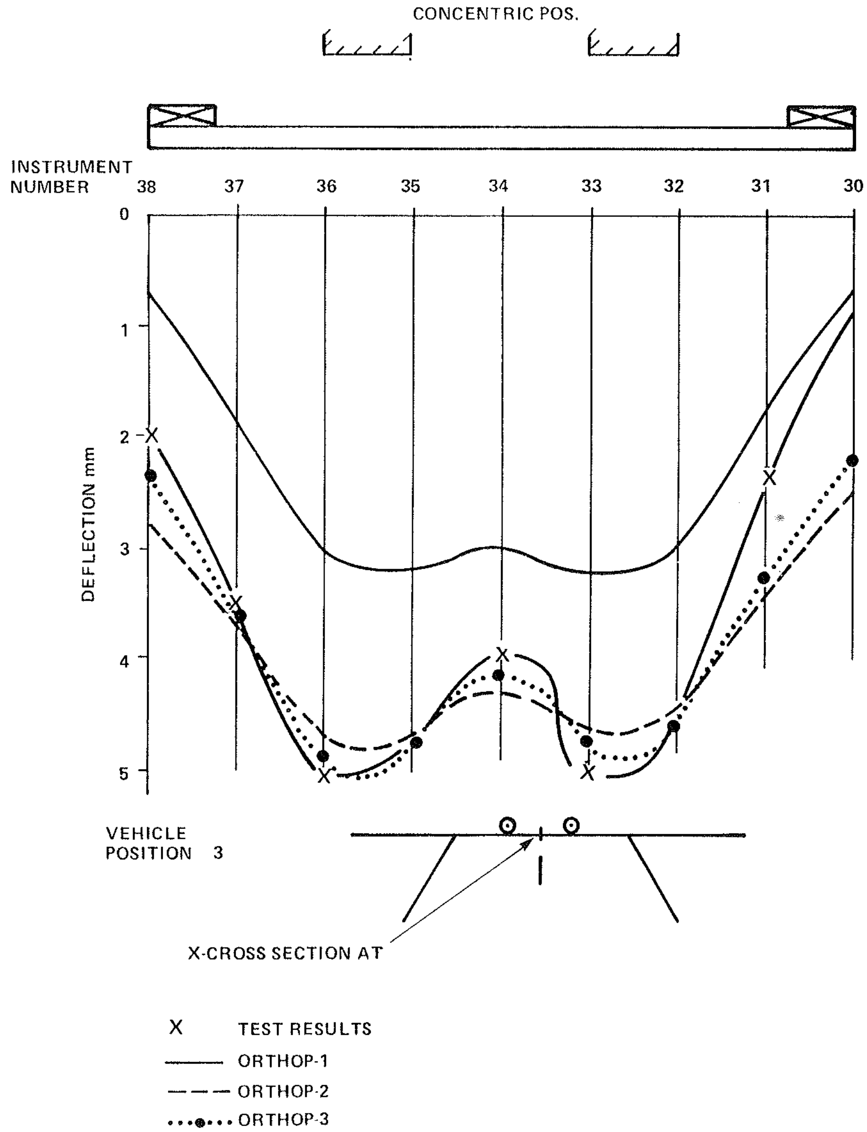


FIGURE 15 Vertical displacements, center span, concentric loading.

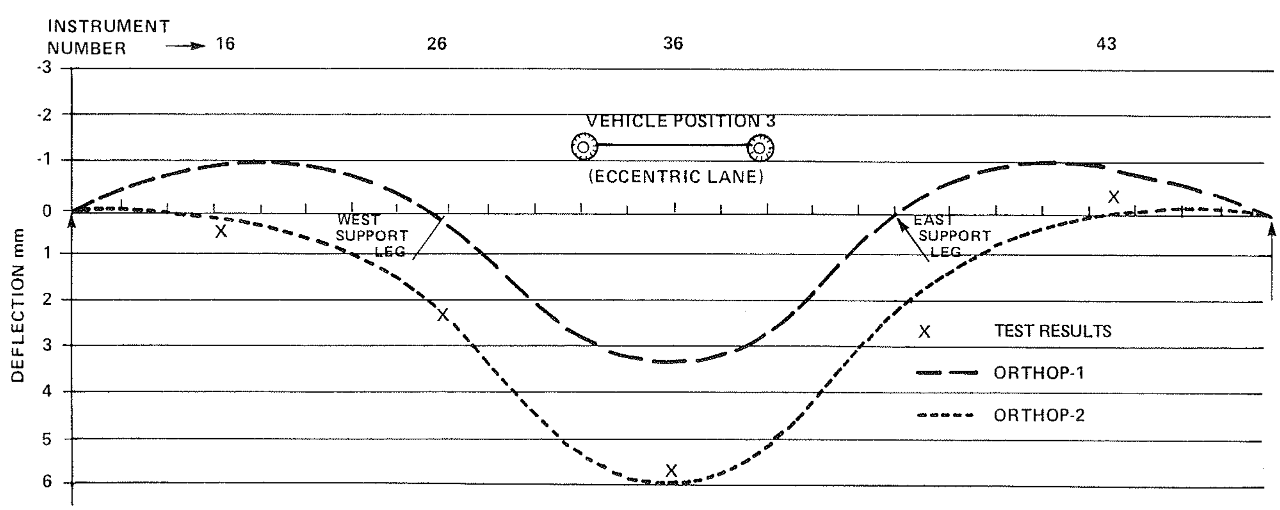


FIGURE 16 Vertical displacements, vehicle position 3, eccentric loading.

middle span, the east leg movement was assumed to be the same as that measured for the west leg.

The results of the analysis are shown in Figures 14, 15, and 16. The orthop 2 analysis provided deflections that were surprisingly close to the experimental deflection contours. The major discrepancy was at the outside edge of the deck. However, the orthop 2 analysis did not consider the increased edge stiffness caused by the curb, guardrail, and steel prestressing bulkhead. One additional, orthotropic analysis was performed that is designated orthop 3. It was performed primarily to demonstrate the sensitivity of the structural performance to variations in the material property assumptions.

The OHBDC specifies the shear modulus G_{LT} at $0.065E_L$, and the transverse modulus of elasticity E_T at $0.05E_L$ where E_L is the longitudinal modulus of elasticity. These values were used in the orthop 1 and orthop 2 analyses. However, a recent investigation (2) indicated that these structural properties for prestressed wood decks are much lower. Based upon the results of that investigation new values of $G_{LT} = 0.055E_L$ and $E_T = 0.037E_L$ were selected.

In addition to the material property changes, the orthop 3 analysis included the stiffness of the wood curb as an edge beam, in an attempt to better represent the real edge conditions.

The results of the orthop 3 analysis are shown in Figures 14, 15, and 16, and although the changes are not dramatic they do indicate a trend. The reduction in torsional and transverse stiffness has caused the transverse curvatures to become more pronounced, and the new deflected shapes are closer to the experimental results.

The increase in edge stiffness reduced the edge deflections bringing them closer to the experimental results. If the edge stiffness of the steel channel prestressing bulkhead could also be accommodated the representation would become even better.

Flexural Strains

In addition to the vertical displacement comparisons several strain comparisons were made with the analytic models. The transverse line of demountable strain gauges used at the center of the bridge provided a contour of the flexural strain on the underside of the deck. Figures 17 and 18 compare the experimental strains with those derived from the moments produced by the analytic models. In this derivation the average experimental modulus of elasticity was used and a uniform deck thickness was assumed.

Given the variation in the modulus of elasticity that exists between the actual laminations, the comparison between analytic and experimental results is considered fair. The applicability of demountable gauges to wood structures was still under investigation at the time of this test, and this particular application was considered part of that investigation.

Figures 17 and 18 also illustrate a very important consideration with respect to the three analytic models: Flexural strain distributions directly reflect moment distributions. According to the figures, these distributions changed very little

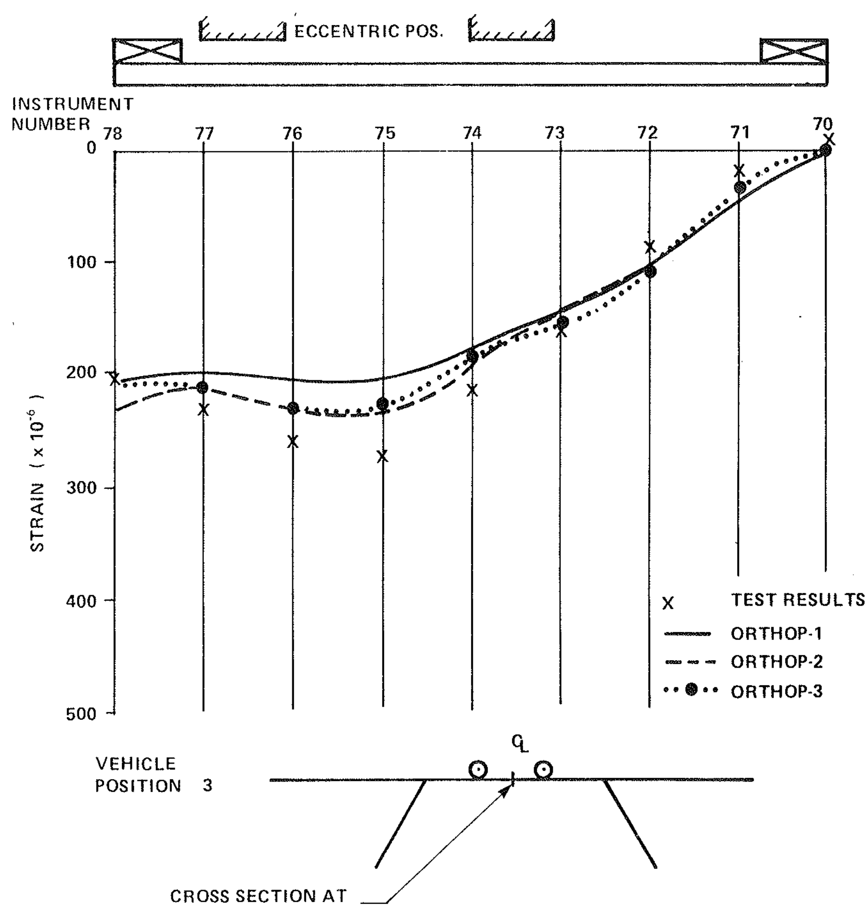


FIGURE 17 Flexural strains, vehicle position 3, eccentric loading.

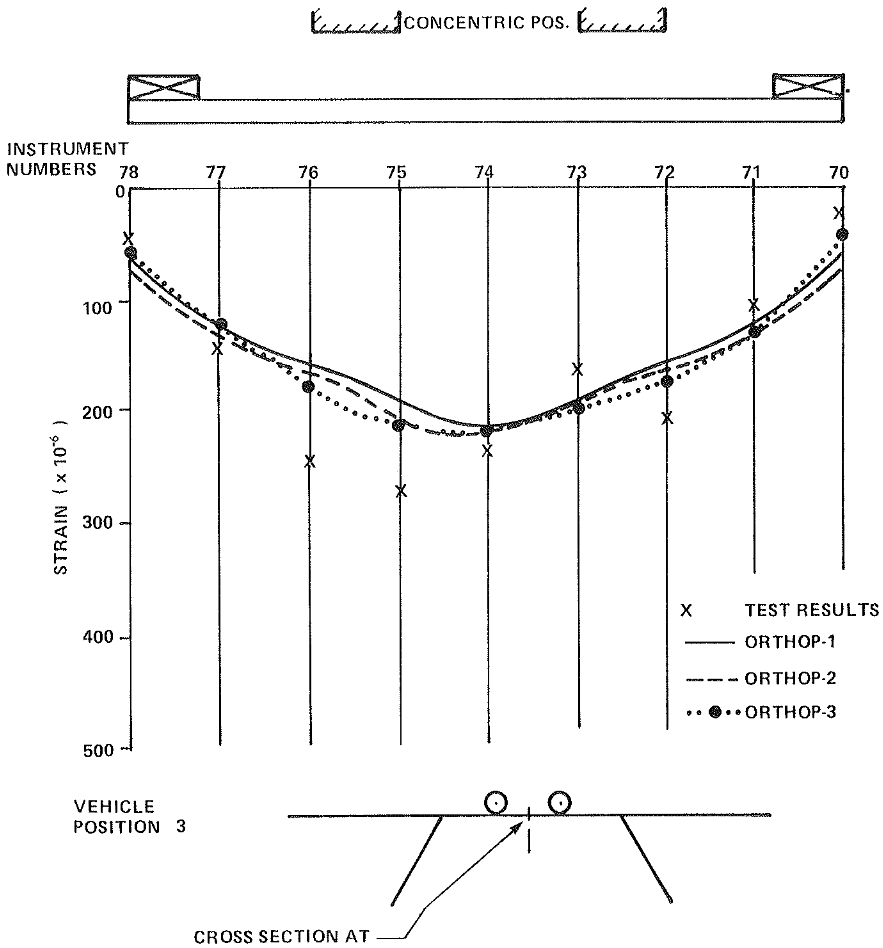


FIGURE 18 Flexural strains, vehicle position 3, concentric loading.

between the original design model and the introduction of leg movements in orthop 2 and orthop 3. Even the magnitude did not change appreciably, considering that the positive moment at the center of the bridge should increase when the intermediate supports undergo settlement. The increase in maximum positive moment in the center span was only 15 percent from the original analytic design model, orthop 1, to the final model, orthop 3, as shown in Figure 17.

Summary

The use of orthotropic analysis to represent the structural response of a prestressed wood deck system appears to be fairly accurate for displacement contours. The use of lower torsional and transverse flexural stiffness should be considered in design, although they apparently have limited effect on the distribution.

In this particular bridge, the leg movements do not appear to affect the force distributions appreciably. According to the analytic models only the maximum positive moment in the center span is increased by the settlement of the leg supports. All other design force modes were reduced. The apparent 15 percent increase in the positive design moment is comfortably accommodated by the moment capacity, which was determined to be 16 percent larger than the original factored moment determined earlier.

Further modification of the analytic model was not considered justified because small changes in the parameters do not appear to change the structural response to any appreciable degree.

Conclusions

This prototype bridge successfully carried MTC's maximum load testing vehicle that represents over two and one-half times the legal load. Under this load the maximum vertical displacement of the deck was around 9 mm.

The structural performance of the bridge was found to be representative of an orthotropic plate where the leg supports could be simulated as flexible columns. The analytic model indicated that the apparent leg movements have no adverse effects on the capacity of this structure to carry the full design load.

SUMMARY AND CONCLUSIONS

The design and construction of this prototype wood bridge demonstrated the flexibility in application of the prestressed wood system. The resulting structure is a strong and durable rigid frame that has a life expectancy of more than 50 years (8).

The bridge was constructed by what may be considered semiskilled labor. Very little special equipment, other than MTC's hydraulic prestressing system and a light construction crane, was required for construction of the prototype. A minimum of on-site work was required because the majority of the fabrication was performed before transportation to the site.

The MNR estimated the cost of this prototype bridge to be only two-thirds the cost of the originally proposed steel structure (9). According to the load testing and the subsequent analytic evaluation, the capacity of the bridge is more than adequate to carry the full OHBDC design loads.

It is believed that the prestressed wood system can now be considered an economical alternative for new short-span bridges (2) as well as a method for rehabilitation (1). The system is being evaluated for use as a transverse laminated wood deck on steel girder bridges to replace the old nailed deck system. Two prototypes have been designed and were expected to be implemented in the fall and winter of 1983.

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