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Autor(en): **Csagoly, Paul F. / Taylor, Raymond J.**

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A Structural Wood System for Highway Bridges

Ponts routiers en bois précontraint

Ein Holzsystem unter Spannung für Autobahnbrücken

Paul F. CSAGOLY
Principal Research Officier

Raymond J. TAYLOR
Research Engineer

Ministry of Transportation and Communications
Downsview, Ontario, Canada

SUMMARY

The Ontario Ministry of Transportation and Communications is developing a new family of timber bridges based on the transverse post-tensioning of longitudinally laminated decks. The paper highlights only some of the findings. The development program is aimed at rehabilitating wood as a viable and competitive structural material for bridges. The system permits low grade materials, consumes less than 10% of the energy required by steel bridges and can be built any time the year by unskilled labour.

RÉSUMÉ

Le ministère des Transports et des Communications de l'Ontario est en train de mettre au point une nouvelle série de ponts en bois avec des tabliers laminés en longueur soumis à une contrainte transversale. Le document ne donne qu'un aperçu général de certains résultats. Le but de ce programme de mise au point est de réintégrer le bois comme matériau viable et compétitif pour la construction des ponts. Grâce à cette méthode, on peut utiliser un matériau de moindre qualité; on consommera moins de 10% de l'énergie nécessaire pour les ponts en acier et les structures peuvent être construites en toute saison par une main-d'œuvre non qualifiée.

ZUSAMMENFASSUNG

Das Ministerium der Provinz Ontario für Transport- und Verkehrswesen arbeitet an der Entwicklung einer neuen Art von Holzbrücken, die auf dem nachträglichen transversalen Verbund der längslaufenden Lamellen der Fahrban beruhen. Die Abhandlung betont lediglich einige der Untersuchungsergebnisse. Das Entwicklungsprogramm zielt auf die Rehabilitierung von Holz als entwicklungs- und konkurrenzfähiges Baumaterial für Brücken. Das System lässt Materialien von geringem Wert zu, nimmt weniger als 10% der bei Stahlbrücken benötigten Energie in Anspruch und kann zu jeder beliebigen Jahreszeit von ungelernten Arbeitskräften gebaut werden.

1 INTRODUCTION

This paper describes a part of the timber bridge development program that is being carried out by the Ontario Ministry of Transportation and Communications (MTC). It deals essentially with the rehabilitation and improvement of bridges constructed using longitudinally laminated, nailed decks supported by pile bents as illustrated in Figure 1. Hundreds of these bridges exist in the province of Ontario, and there are probably several thousand of them throughout Canada.

The maximum permissible single axle weight in Ontario is 10 000 kg (98.06 kN), but on logging and mining roads where many of these bridges are located, weights of up to 20 000 kg (196.12 kN) have been observed. This observation is reflected in the new Ontario Highway Bridge Design Code [1] which specifies a 200 kN axle for the design of bridge decks.

Under such loads bridge decks often fail. The failure is precipitated by the bending of the nails and the crushing of the adjacent wood. Initially, the holding resistance of the nails deteriorates because of repeated use of the bridge by heavy commercial vehicles. In addition, incompressible materials accumulate between the laminates, as shown in Figure 2; these are driven down by heavy wheel loads, further separating the laminations. This allows water to enter between the laminations and to penetrate to the untreated heartwood of the laminations through the enlarged nail holes. The combined effect of wood decay and decreasing transverse load distribution leads to local deck failures after only a relatively short service life.

Another weakness of these decks is the absence of longitudinal continuity of the laminates, which are traditionally butt-ended without any phsyical connection. A recent test was carried out for MTC by the University of Toronto on a two-span, nailed strip consisting of 24 lines of laminates. The test indicated that the lack of continuity caused a total disintegration of the structural system at less than 60% of predicted ultimate load. Figure 3 clearly illustrates this disintegration, although the laminations show no signs of flexural failure.

The frequent failures of timber bridges in the past convinced engineers that these structures cannot be expected to last more than 40 years. This is less than the period of time generally specified by most jurisdictions. Consequently, timber bridges have been largely eliminated from professional education, and their application in present times has been restricted to parks, resort areas, etc., primarily for architectural reasons. The gradual disappearance of wood of large structural sizes, the availability of steel of guaranteed characteristics, and improved concrete construction signaled the end of a structural material that had dominated the construction of bridges for millennia.

Normally bridges that displayed local failures in the past were replaced. However, due to the deteriorating economy and the high cost of replacements, MTC directed its continuous bridge testing program towards wood structures in 1973. This testing program is aimed at determining the actual load-carrying capacity of existing highway bridges.

Since that time numerous wood bridges have been tested. The tests involved wood trusses, sawn timber stringers, glue-laminated girders, laminated decks with or without concrete overlay and wood piling. Many of the bridges tested were close to 50 years old, but were in remarkably good condition and have displayed load-carrying capacities in excess of the minimum requirements. Clearly, the 40-year rule could be challenged.

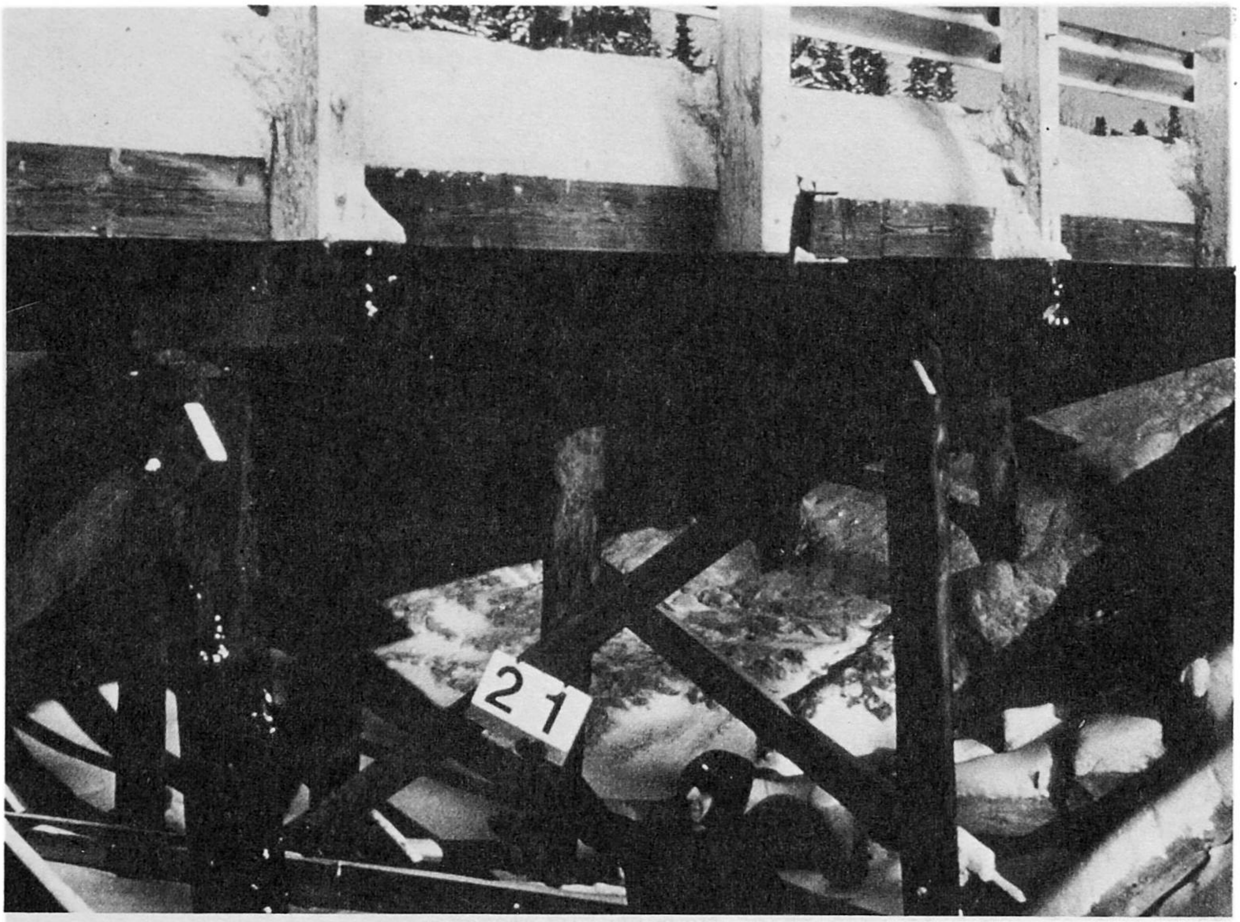


Figure 1 Side View of McCarthy Creek Bridge. A Typical Laminated Deck on Pile Bents

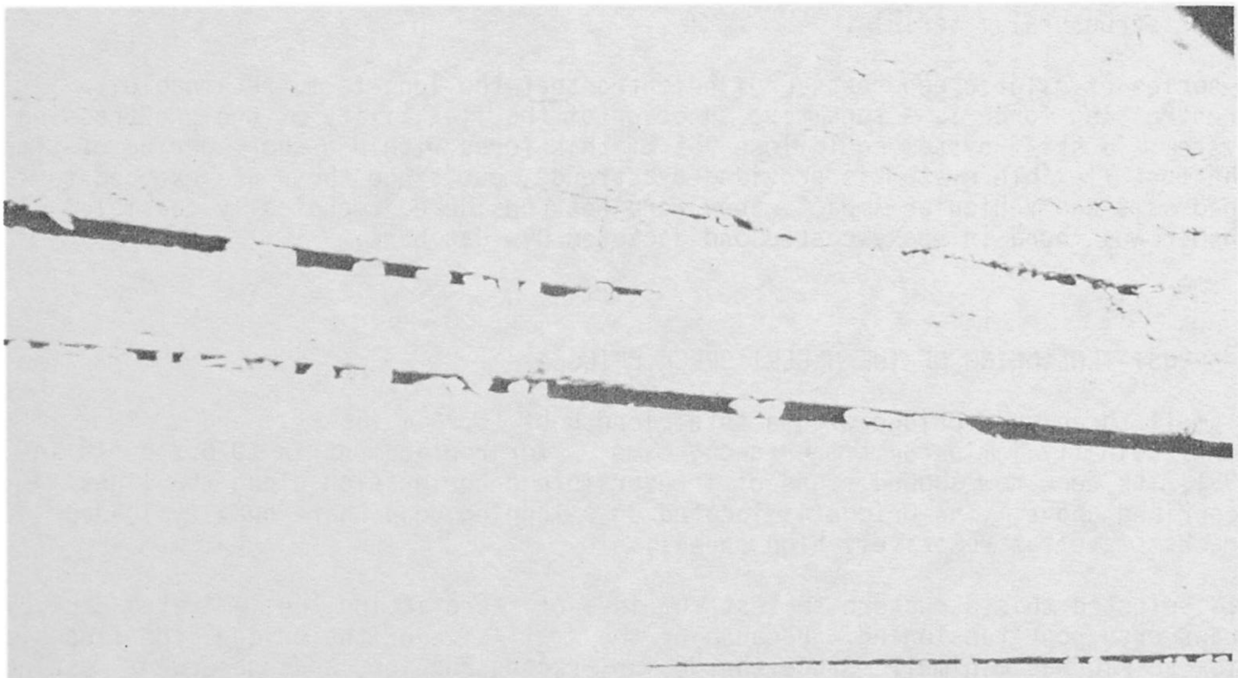


Figure 2 Separation of Laminated Timber Decking Due to Incompressible Materials

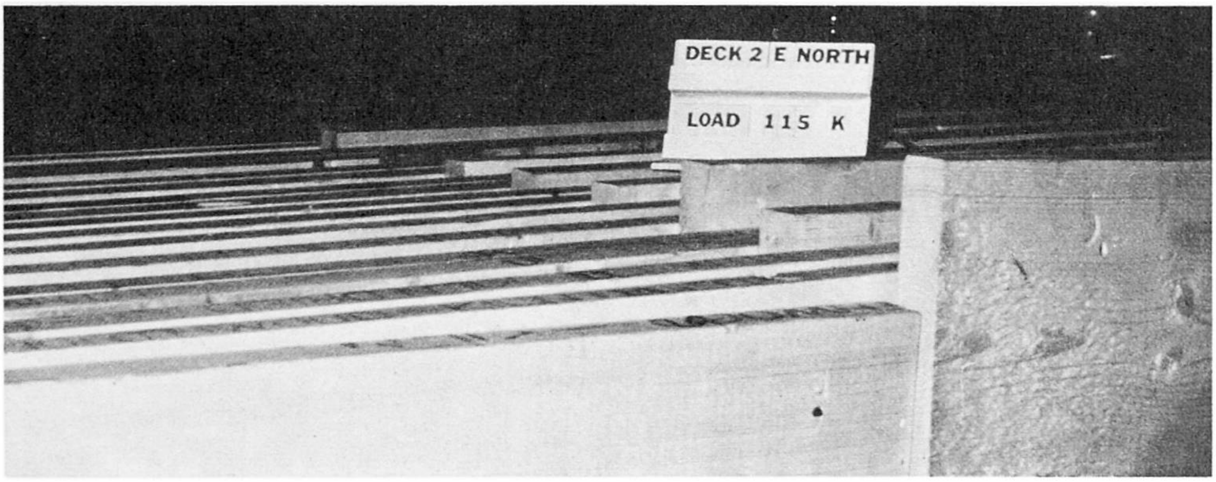


Figure 3 Disintegration of Unspliced Deck Structure

Specifically, the load carrying capacity and the service life of the longitudinally laminated decks were identified as being a direct function of the "tightness" of the structural system. ("Tightness" is defined as the ability of the structure to withstand relative interface movement and the access of foreign materials between adjoing components). Figure 4 shows an elementary method of arresting the separation of the laminates in a longitudinally laminated wood deck. It consists simply of pairs of mild steel bars anchored in plates.

Essentially, transverse post-tensioning of laminated decks is an engineered version of this idea. The first and certainly the most important problem was to determine the permanency of the prestressing force applied perpendicular to the grain in wood. Wood can be easily crushed by pressure applied perpendicular to its grain. It also reacts to changes in temperature and humidity at a much higher rate than do other structural materials.

A series of pilot creep tests [2] indicated that the long-term, retainable prestressing force is a sensitive function of the flexibility of the prestressing system; a stiff system could lose 95% of this force within a short period of time. The most flexible system is provided by strands, but since these are exposed to road salt and vehicular impact, they were not considered technically feasible. The answer was found in epoxy-coated and jacketed Dywidag bars.

2 POST-TENSIONING OF THE HEBERT CREEK BRIDGE

A small three-span bridge with a total length of 16.78 m (55 ft.) and a longitudinally laminated timber deck, came up for replacement in 1976. Built in 1951, its deck now showed signs of irreversible deterioration along the lines described above. The bridge is located on a logging road where heavily loaded trucks travel at relatively high speeds.

MTC selected this structure to test the idea of retrofitting the laminated deck by transverse post-tensioning. Because of the small size of the bridge, the financial risk to MTC was minimal. Since the design, reconstruction, and subsequent testing have been described in detail elsewhere [2], only the highlights will be repeated here.

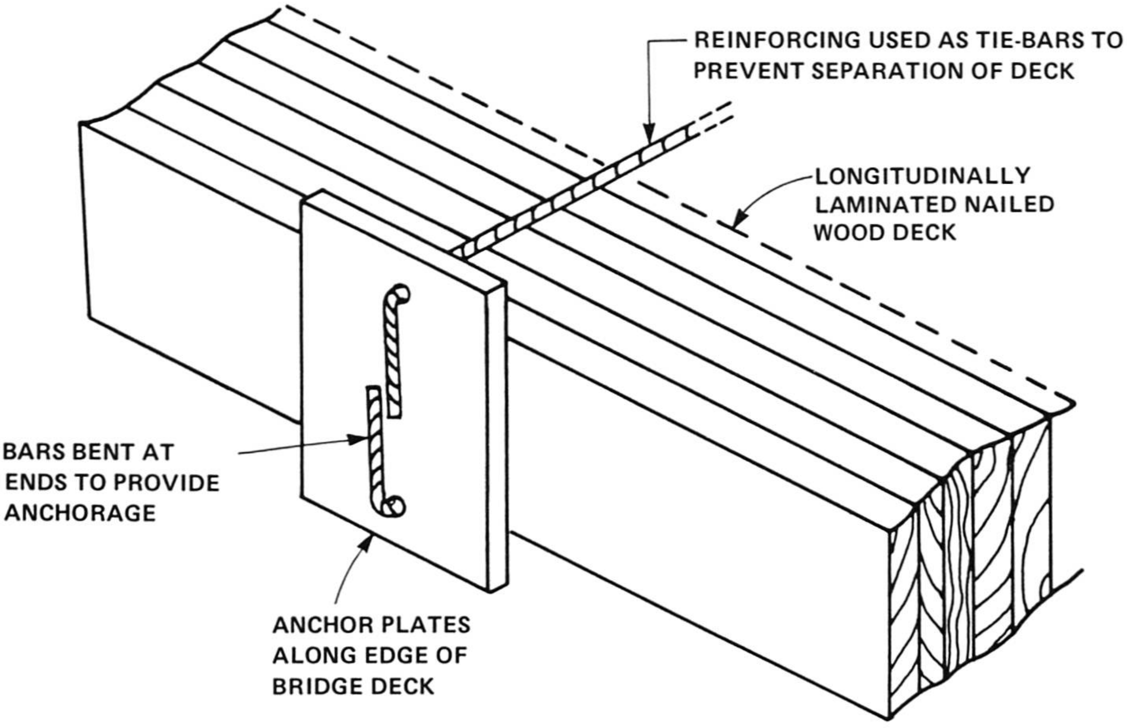


Figure 4 Elementary Tie-Bar System

Figure 5 Anchor Plate on Hebert Creek Bridge

The prestressing system employed consists of pairs of 16 mm (5/8 in.) diameter Dywidag bars at 0.915 m (3.0 ft.) centers anchored in steel plates along the sides of the bridge as shown in Figure 5. The bars were protected by enclosing them in grease-filled PVC pipes. The top bars were layed into transverse troughs cut into the asphalt wearing surface, while the bottom bars were attached to the wood deck by brackets and wood screws. Under the anchor plates a new 76 mm (3 in.) thick continuous B.C. fir bulkhead was installed along both sides of the bridge deck in order to better spread the prestressing force in the deck and to compensate for the expected shortening of the width of the deck due to prestressing.

As shown in Figure 6, the prestressing of pairs of bars was carried out in a sequential manner by using two jacks at a time. This operation had to be repeated several times as the width of the deck shortened by as much as 460 mm (18 in.) before the specified average prestressing stress of 0.69 mPa had been attained. Approximately 82% of this shortening was related to straightening the laminates and closing the spaces between them. Very little prestressing force was built up during this period. After prestressing, efforts to insert razor blades between the laminates failed, and during a rainfall it was observed that the deck became practically watertight.

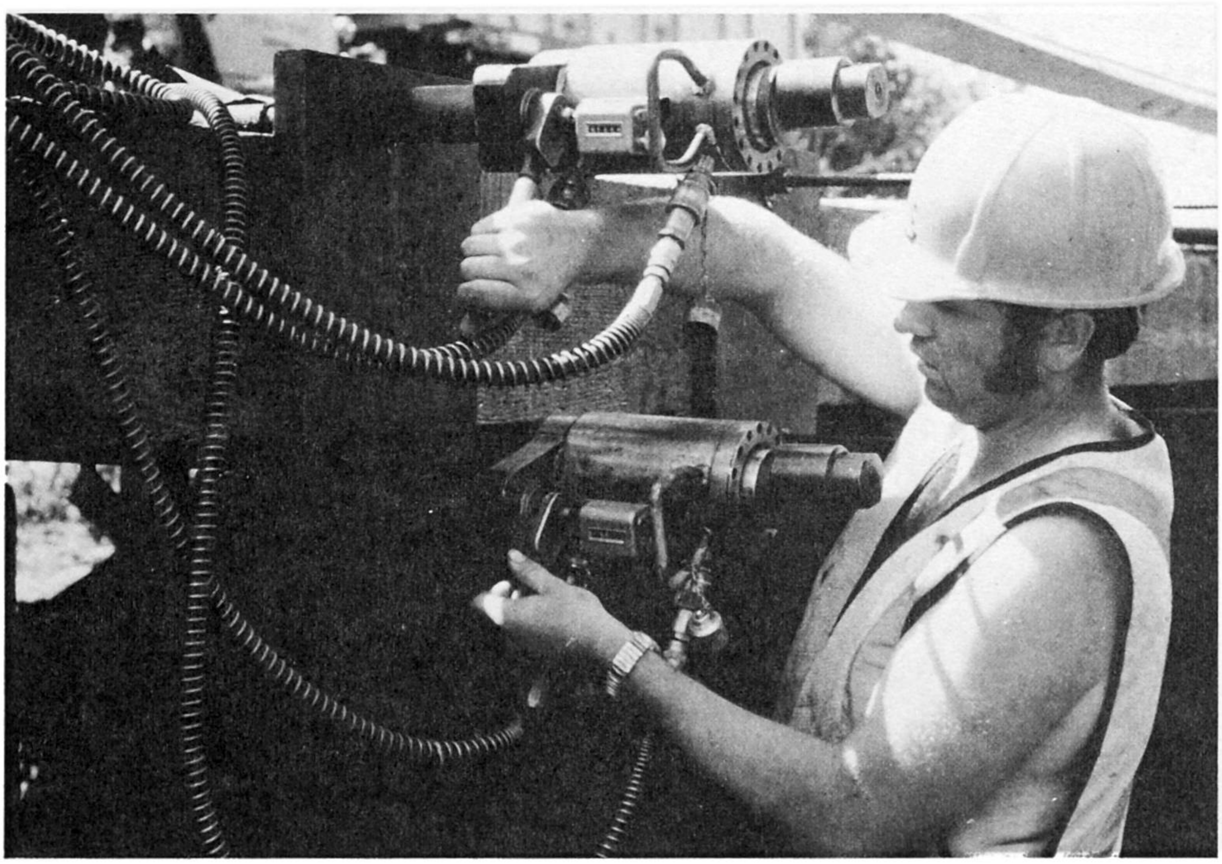


Figure 6 Prestressing of Hebert Creek Bridge

The bridge was load-tested before and after the application of prestressing, using one of the two MTC testing vehicles. The maximum gross weight of these vehicles is 890 kN each and any bridge that is not substandard is expected to support both of these vehicles at the same time without any signs of distress. Due to the nature of this bridge only one vehicle was employed. The deck indicated a local failure at the 795 kN load level prior to prestressing.

After prestressing, the bridge was retested and it sustained the full specified load without any difficulty.

The improvement in behaviour was so great that two months later a third test was carried out. Figure 7 illustrates, in terms of measured deflections across the width of the deck, the improvement in load distribution. The improvement was about 100% and the difference between the second and third test is apparently due to the application of an additional layer of asphalt which slightly increased transverse distribution. The bridge has been monitored on a continuous basis since then, and no further deterioration of the deck has been observed.

During the summer of 1979, two other structures were retrofitted by transverse post-tensioning. One is on a country road in Southern Ontario; the other - a major bridge - is at Prince Rupert, British Columbia. Both operations were successful.

3 LOAD-SHARING AT ULTIMATE LIMIT STATES

The Ontario Highway Bridge Design Code is based on limit states principles. The ultimate flexural capacity of a wood component is defined as follows:

$$M_\mu = \phi k_m k_\upsilon f_{bu} S$$

where:

M_μ = moment capacity at ultimate limit states,

ϕ = performance factor,

k_m = factor for load duration and load sharing,

k_υ = factor for size effect,

f_{bu} = 5th percentile ultimate flexural strength,

S = sectional modulus.

In this section of the paper, only factor k_m will be dealt with. The Code specifies k_m = 0.7 for dead load and earth pressure, k_m = 1.0 for live load, and k_m = 1.20 for live load acting on a "load-sharing" system. Load-sharing is a construction composed of three or more essentially parallel members so arranged or connected that they mutually support the load; in case of the failure of one member, the system retains the capacity to support the load. The 20% increase in k_m is a conservative estimate based on observation and engineering logic but not on experimentation.

In order to create a data base, MTC contracted the Western Forest Product Laboratory (WFPL) of Vancouver, B.C., to undertake the testing of a large number of wood specimens both in individual and load-sharing modes. The work is described in detail in reference 3, only the highlights are reported here. Three species were included, namely B.C. hem fir, Ontario red pine and Ontario white pine. All samples were 51 mm x 254 mm rough- sawn boards, 4.88 m long. These were purchased green and kiln-dryed to an approximate moisture content of 19%.

After retaining only that material graded as #2 and better [4], each sample size was reduced to about 420 specimens. Sixty of each species were tested individually for elasticity and modulus of rupture. The rest were tested for load-sharing by combining together either 6, 12, or 18 laminates per specimen by transverse post-tensioning. Each of the three groups contained 10 beam units for each species, totalling 90 post-tensioned wood specimens.

Each beam was made up by assembling the boards side-by-side in groups and post-tensioning them transversely to a contact pressure of approximately 0.35 mPa. The post-tensioning was accomplished by placing 16 mm diameter Dywidag bars in pairs above and below the beam, at 710 mm centers along the beam. In each test, the loading was applied at the third points of the 4.32 m span and at a displacement control rate of 10.99 mm/second.

Table 1 is a summary of all tests. All 5th percentile values are related to an assumed lognormal distribution. It can be seen that the average values, regardless of the number of specimens in a beam, hardly fluctuate, but there is a dramatic increase in ultimate strength for the load - sharing systems. The increase from individual specimens to an 18-laminates beam is 82.5, 74.9, and 50.8% for white pine, red pine and hem fir, respectively.

Table 1 Western Forest Product Laboratory
(Stress Values Given in mPa)

Description of Test Mode	Number Tested	Designation	White Pine	Red Pine	Hem Fir
Individual specimens	60	mean	15.98	27.98	45.92
		5th percentile	8.75	14.87	25.90
Beams made of 6 specimens	10	mean	19.40	27.52	46.89
		5th percentile	15.02	18.99	35.63
Beams made of 12 specimens	10	mean	21.17	29.02	43.89
		5th percentile	19.00	22.76	39.56
Beams made of 18 specimens	10	mean	18.31	28.78	44.83
		5th percentile	15.97	25.99	39.07
Ratio of 5th percentile 18 speciments to individual	-	-	1.825	1.749	1.508

Zahn [5] and Reardon [6] investigated the load-sharing characteristics of wood in the early seventies. Both studies found that the advantage in system strength due to load-sharing was minimal despite the high variability in wood strength.

This is because:

- the conceptual models define collapse to be the occurrence of the failure of the first element, and

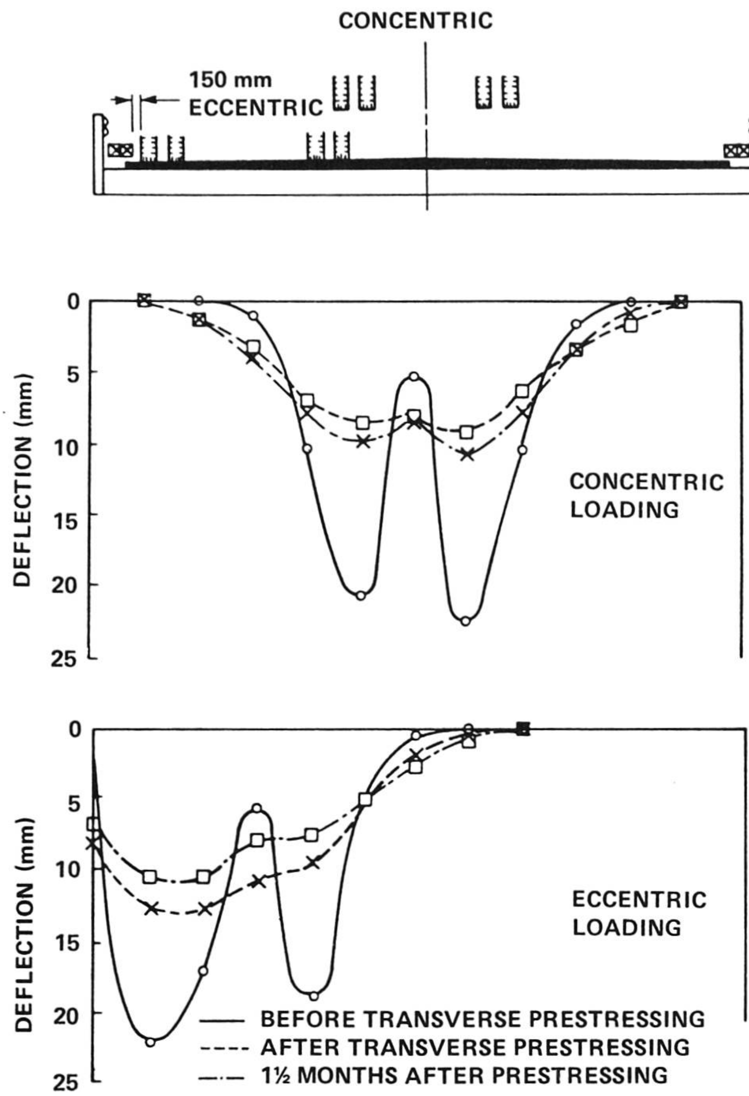


Figure 7 Midspan Deflections. Hebert Creek Bridge Test

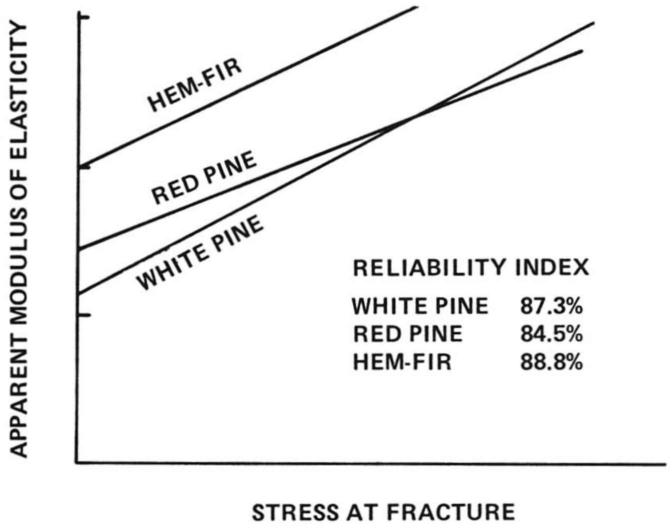


Figure 8 Modulus of Elasticity as a Function of Modulus of Rupture

- mechanical assistance of adjacent members to zones of local weakness is negligible in the configurations examined.

The first question relates to structural philosophy. In this respect the Ontario Bridge Code [1] does not consider partial component failures, but does consider the maximum capacity at ultimate limit states. More important, however, is the presence of transverse post-tensioning which, by interface friction, assures the mutual cooperation of the participating elements, resulting in a marked enhancement of strength as indicated by the last line of Table 1.

It was observed during the WFPL tests in Vancouver, (and in similar tests carried out at the University of Toronto and at MTC's own laboratory in Downsview) that the ultimate load carrying capacity is attained usually after about 25% of the laminates are broken. In addition, it was evident that laminates weakened by internal discontinuities (knots) and by interrupted or misaligned grains failed first.

The WFPL tests included the measurement of modulus of elasticity, in addition to modulus of rupture. This permitted the construction of a statistical model by which load distribution among the laminates at any stage of the failure sequence could be estimated.

As a result of the individual specimen tests, the stiffness versus strength relationships were plotted separately for the three different species, and straight-line, best-fit analyses were carried out. Figure 8 shows the three functions obtained along with a reliability index defined as 100 (1.0 - C_v), where C_v is the coefficient of variation. It can be seen that the two Ontario species indicate similar relationships in spite of the difference in their strength. The line for the far superior hem fir is higher, but essentially parallel to the others. The high reliability indices confirm that the relationship between stiffness and strength exists and can be represented by a single function for each species.

Figures 9(a) through 9(c) consist of a normal probability distribution curve, the modulus of elasticity function and the expected failure sequence. The following argument is advocated. If the number of laminates in a beam is sufficiently large, the normal curve (Gaussian) is believed to describe the probability distribution of the strength of individual laminates in the beam with a reasonable degree of accuracy. This can be visualized by subdividing the area under the curve, which is equal to 1.0, into a number of identical sub-areas equal to the number of laminates in the beam. The dividing line between jth and $(j + 1)$th sub-areas signifies the stress where the jth laminate would fail. Making use of the modulus of elasticity versus strength function as shown in Figure 9b, the load distributed to the jth laminate, in terms of moment, can be estimated as:

$$m_{\mu j} = M_{\mu j}\ E_j \Big/ \sum_{j}^{N} E_i$$

where:

$m_{\mu j}$ = ultimate moment capacity of laminate "j",

$M_{\mu j}$ = ultimate moment capacity of the system when laminate "j" breaks,

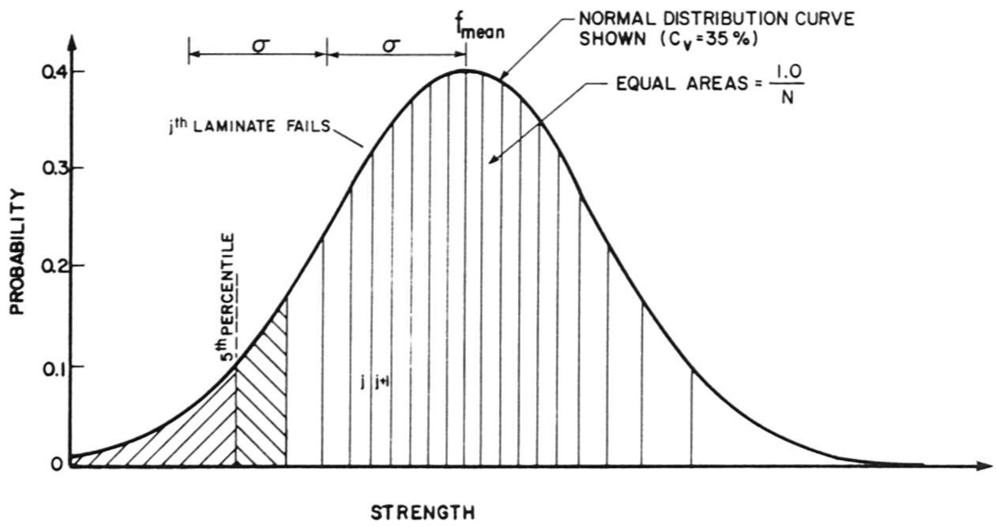


Figure 9(a) Normal Strength Distribution Representing the Strength of a 20 Laminate Beam

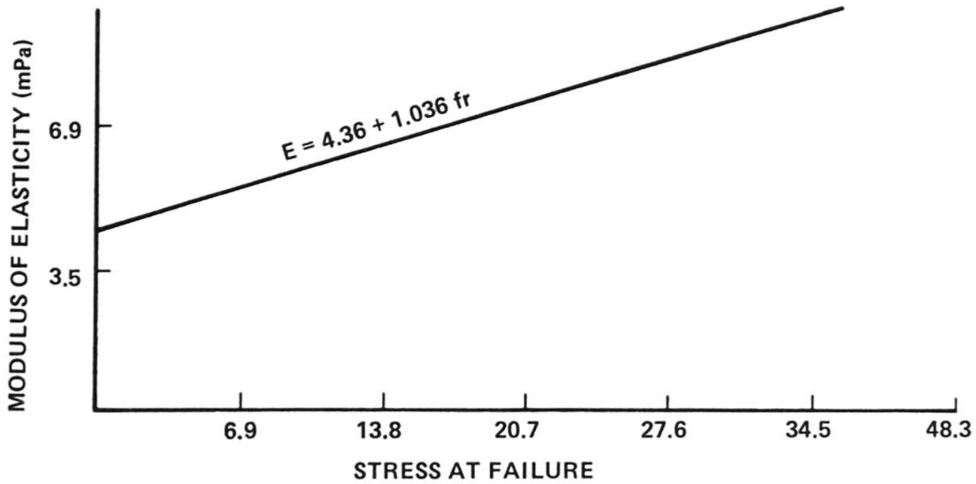
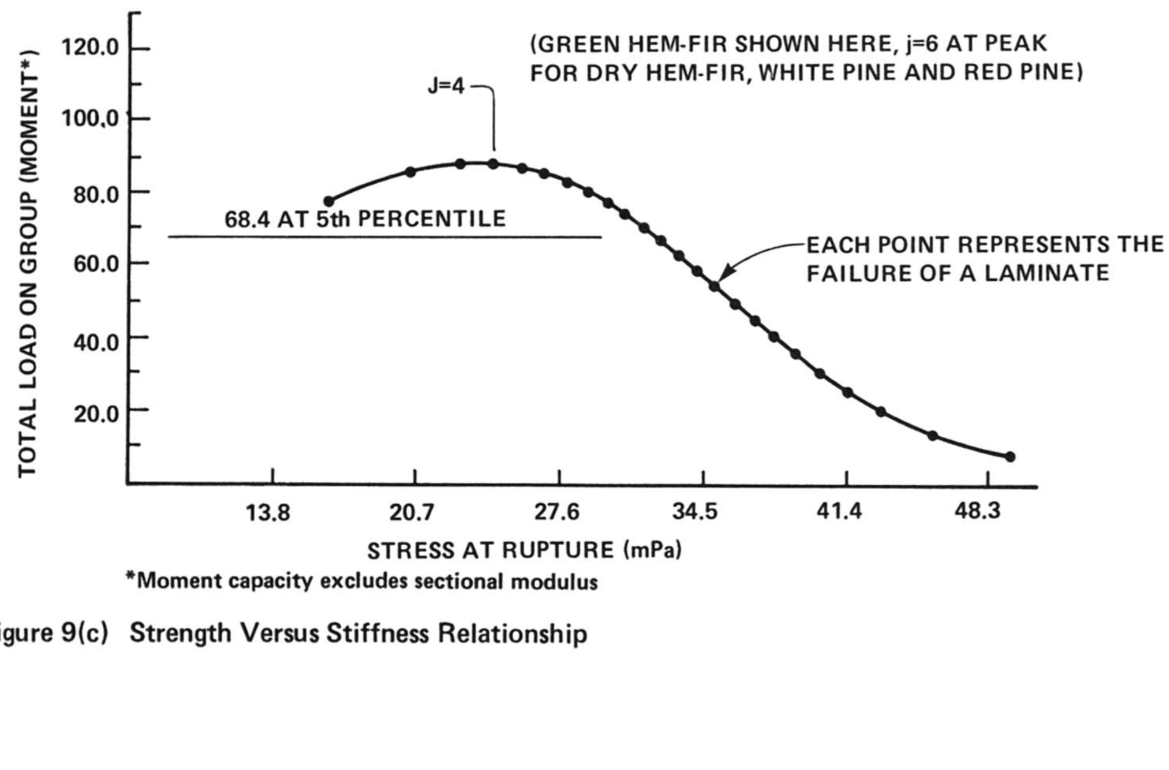


Figure 9(b) Failure Sequence of a 24 Laminate Beam



*Moment capacity excludes sectional modulus

Figure 9(c) Strength Versus Stiffness Relationship

E_j = calculated modulus of elasticity of laminate "j",

and also:

where:

S = sectional modulus of a laminate, and

f_j = ultimate strength of laminate "j".

After rearranging these two expressions, the ultimate moment capacities of the beam can be calculated as:

$$M_{\mu j} = S f_j \sum_j^N E_i / E_j$$

The process includes exponential functions and consequently does not lend itself to a closed format manipulation. As shown in Figure 9(c), $M_{\mu j}$ values can be plotted and the peak value established. For the purpose of this derivation, M^{max} values can be compared with the traditionally calculated moment capacity at the 5th percentile.

$$M_{\mu 5} = Sf_{5th} \; N$$

The calculations for Figure 9c were carried out for N = 24, a number that provides for the width of a bridge deck within which deformations are uniformly distributed. It is interesting to note that for all three species, $M^{max}_{\mu j}$ was obtained for j = 6 (after six pieces failed), a phenomenon that was confirmed by several laboratory tests carried out on full-size laminates.

Table 2 Statistical Analysis of Load-Sharing System (mPa)

Hem	White Pine	Red Pine	Hem Fir
f_{5th} - lognorm. distr.	8.75	14.87	25.90
f_{5th} - normal distr.	6.85	11.58	21.52
$M_{\mu 5}$/ f5th - normal distr.	164	278	516
$M^{max}_{\mu j}$/f5th - normal distr.	259	443	742
$M^{max}_{\mu j}/M_{\mu 5}$	1.580	1.596	1.437
Ratio from Table 1 (R)	1.825	1.749	1.508
$R/M^{max}/M_{\mu 5}$	1.155	1.096	1.049

Table 2 gives the $M_{\mu 5}$ and $M^{max}_{\mu j}$ values normalized (divided) by the sectional modulus "S", and subsequently the ratio between them. Comparing this ratio with that from Table 1, using actual test results, it can be observed that this statistical manipulation explains the majority but not the total strength increase obtained by transverse post-tensioning.

It can be seen that the statistical analysis yields strength ratios approaching those obtained by actual testing. It is believed, however, that if the sequence of laminate failures were selected by the Monte Carlo or another method, the result could be somewhat less than those indicated in Table 2, (i.e. the difference between the theoretical and actual could be larger for all three species considered).

4 SIZE EFFECT ON THE STRENGTH OF LAMINATES

The testing project described in the previous section was aimed at structural specimens that are tall and narrow in cross section. Simultaneously, in preparing the Ontario Bridge Code, MTC contracted the University of British Columbia to carry out in-grade testing of large sawn-timber beams. These are used primarily as stringers in wood bridges. The findings were reported in reference 7.

To destruct a large number of full size beams is a financially unacceptable proposition, consequently, a strategy different from the one applied to the load-sharing project was devised. In order to establish the 5th percentile strength of a population of beams, it is statistically adequate to test to failure about 10% of the population involved. Thus an arrangement was made with a sawmill in the vicinity of Vancouver such that MTC would pay only for those beams actually destroyed.

The project included sections having widths of both 152 and 203 mm. However, because of the incomplete state of the data base developed, only the results of the 152 mm wide sections are considered here. Sections with three different heights were included, namely 203 mm, 305 mm and 406 mm. All wood was rough-sawn B.C. fir with moisture content between 21 and 44%. Regardless of the length of the specimen, the test span was taken as 17 times the height. Loading was always applied at the third points and no control on load or deflection rate was undertaken. The testing program included Select Structural and No. 1 materials only. Altogether 452 specimens of 152 mm width were tested, of which 48 were broken. Table 3 is a summary of the project.

Table 3 Summary of the Beam Test Project (mPa)

Size (mm)	Number Tested	Number Broken	Mean	Standard Deviation	f_{5th}
152 x 203	200	12	56.33	15.27	31.21
152 x 305	202	22	48.90	13.22	27.15
152 x 406	50	14	34.42	9.65	18.55

The statistical evaluation process consists of taking the standard normal density function $\phi = \mathbf{EXP}\,(-t^2/2)/2\pi$, subdividing the area under the curve into N(total number of specimens tested) identical sub-areas, from which t_i values for every broken specimen of a strength of f_i can be established. Since $t_i = (f_i - \mu)/\delta$, a straight line approximation using least square analysis to the family of t_i, f_i pairs can be attempted in the form of $t = af - b$.
The process yields "a" and"b" values with an acceptable level of certainty, of which the mean (μ) and standard deviation (δ) can be calculated. The 5th percentile strength is established by:

$$f_5 = \mu - 1.6458\delta$$

The purpose of presenting these values is to confirm the existence of "size effect", ("shape effect" would perhaps be a better nomenclature). The plotting of the specified strength (f_5) versus height-to-width ratio (r) , as shown on Figure 10, leads to a curve that tends to have a local maximum in the vicinity r = 1.0. However, this maximum is only slightly larger than the value obtain for r = 1.33 (203 mm). The total reduction from 203 mm to 406 mm material is astonishing 40.4%. Lacking complementary test data, it is difficult to suggest that this trend is present for all cases that may occur, but it appears to be significant.

It can be speculated that the answer to this question should perhaps be sought in the macro-structure of the wood proper. When a close-to-square section is formed, it likely includes the whole log, cut to eliminate the circumferential material. This leaves the macro-structure symmetrical, both ways, to the center of gravity of the section. When two or more sections with (r) much less than 1.0 are cut from a log, say 51 mm x 305 mm material used in laminated decks, the chances are that none or only one section will end up with a symmetrical macro-structure. This lack of symmetry causes the shear center of the section to move away from the center of gravity, resulting in torsional moments under vertical loading. Figure 11 illustrates some of the more frequently occurring non-symmetrical macro-structures of structural softwood.

In addition to torsional shear stresses, the probability of having discontinuities (knots, etc.) penetrating the whole width of narrow sections are believed to account for the apparent dramatic decrease in strength of tall, narrow sections. The theory of shifting shear centers seems to be reinforced by the observed high flexural strength of the transversely prestressed systems. These systems internally eliminate the torsional moments due to the random orientation and do not permit the development of torsional stresses due to the close to absolute confinement.

It is also statistically obvious that discontinuities are randomly distributed in a laminated deck, i.e., they do not constitute a line of weakness. The transverse post-tensioning mobilizes longitudinal friction of considerable magnitude among the laminates by which longitudinal stresses can bypass the discontinuities. This "bridging", along with the elimination of torsional stresses, is believed to be responsible for the part of strength increase, not explainable by the statistical process described.

The significance of this argument is that for the basic ultimate moment limit states equation the following modifications can be made:

k_m: increased to between 1.40 and 1.60 as determined by testing,

K_υ : to be taken as 1.0 for transversely post-tensioned decks, and

f_{bu}: the 5th percentile strength of close-to-square sections.

For traditional nailed, laminated decks the AASHTO Specifications [8], prescribe a width of 0.81 m (32 in.) upon which one line of wheel loads can be uniformly distributed for design purposes. The results of the Hebert Creek Bridge Test [2], indicate a largely improved width of 1.65 m (66 in.).

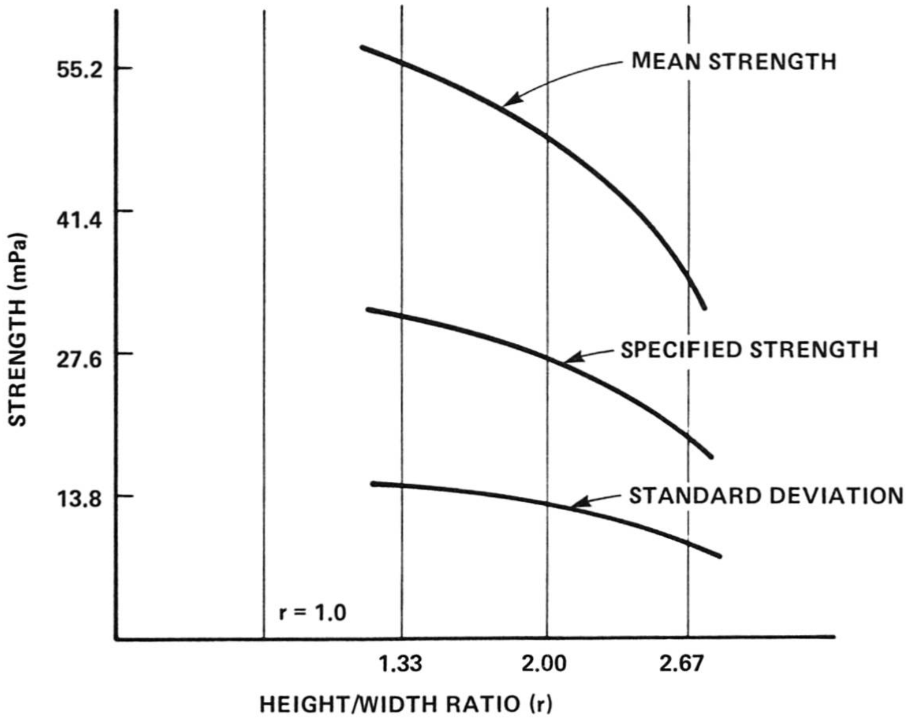


Figure 10 Strength as a Function of Size

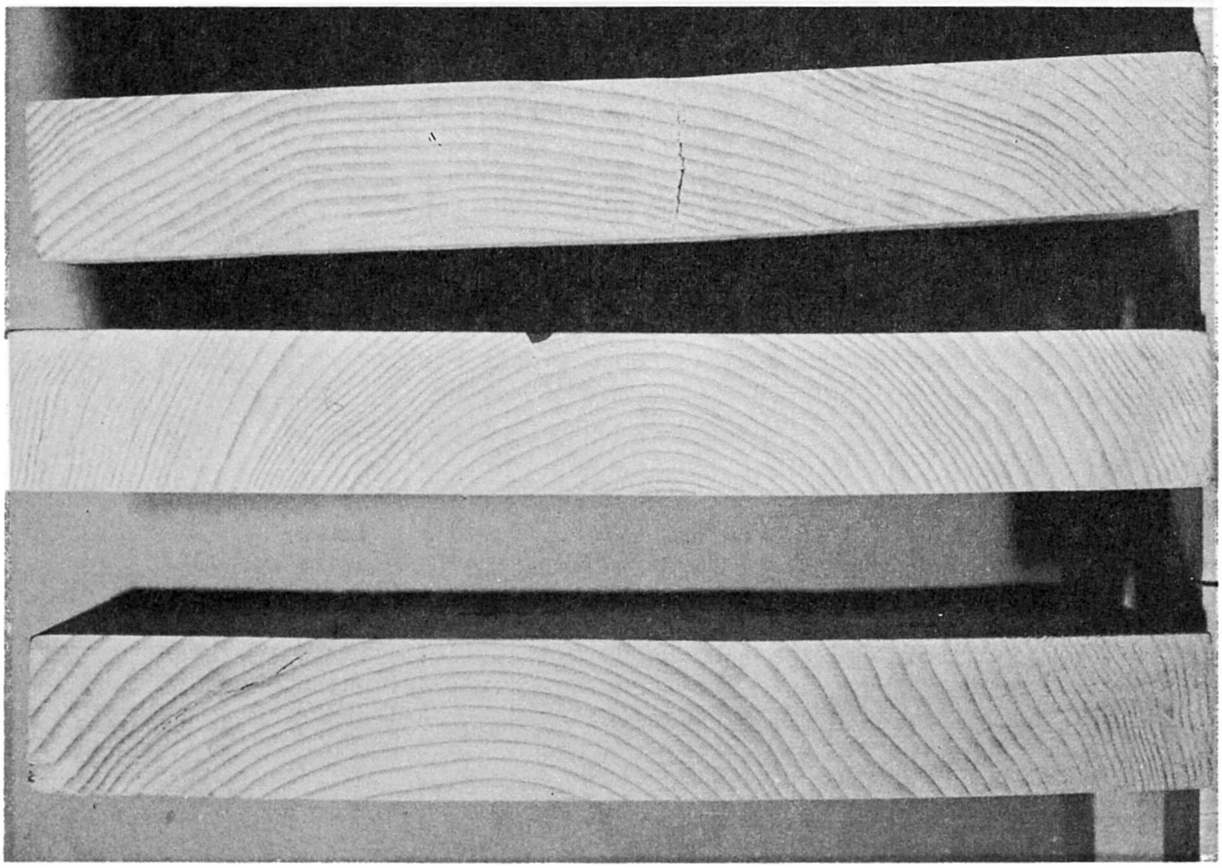


Figure 11 Typical Non-Symmetrical Macro-Structure of Softwood

It appears that transverse post-tensioning upgrades the load- carrying capacity of laminated decks in the following three ways:

- by forcing the system into load sharing action,
- by preventing torsional shear stresses and developing internal "bridging", and
- by improving the lateral distribution of wheel loads.

The degree of potential improvement in comparison with the traditional nailed system can be expressed quantitatively by multiplying the ratio of the 5th percentile strength of the 18 laminate beams and the single laminates (Table 1), by the ratio of distribution widths between post-tensioned and nailed decks. For the three species tested [3], the following values can be obtained:

White pine: $\left[\frac{15.97}{8.75} \times \frac{1.65}{0.81} - 1\right] \times 100 = 272$ percent

Red pine: $\left[\frac{25.99}{14.87} \times \frac{1.65}{0.81} - 1\right] \times 100 = 256$ percent

Hem fir: $\left[\frac{39.07}{25.90} \times \frac{1.65}{0.81} - 1\right] \times 100 = 207$ percent

5 LONGITUDINAL CONTINUITY OF LAMINATES

Sections 3 and 4 of this paper deal with full length, uninterrupted laminates and beams. In present designs the maximum span length is about 6.0 m, however, the continuous deck may exceed 100 m. Traditionally, a deck is constructed in multiples of three, such that one laminate is butt-jointed over the pier and the other two at alternating third points of the span. These third points have proved to be lines of considerable weakness, since the laminates move independently, as illustrated in Figure 3. There is no physical continuity of the laminates provided at the butt-joints other than two nails by which the ends are connected to the adjacent laminates. The test at the University of Toronto, referred to earlier, indicated a disastrous effect: the nails split the ends of the laminates and the structure disintegrated as shown in Figure 12.

In order to eliminate this weakness, a variety of methods of correction were considered. The one that was found to be most feasible involves the "nailplate" or "gang-nail," a commercially available fastener being used in prefabricated wood trusses for residential construction. The gang- nail is manufactured from galvanized sheet metal by punching out the teeth with a machine die. These plates are then pressed into the wood using hydraulic jacks and are used to connect two components together. The sheet comes in various thicknesses and is made of mild steel with an ultimate tensile strength of 320 mPa.

From pilot tests it appears that a 14 gauge (1.90) plate would match the strength of red pine at their mutual 5th percentile level. However, since the strength coefficient of variation of steel is only 8% while that for red pine is 36%, it is assured that the plates would rupture first.

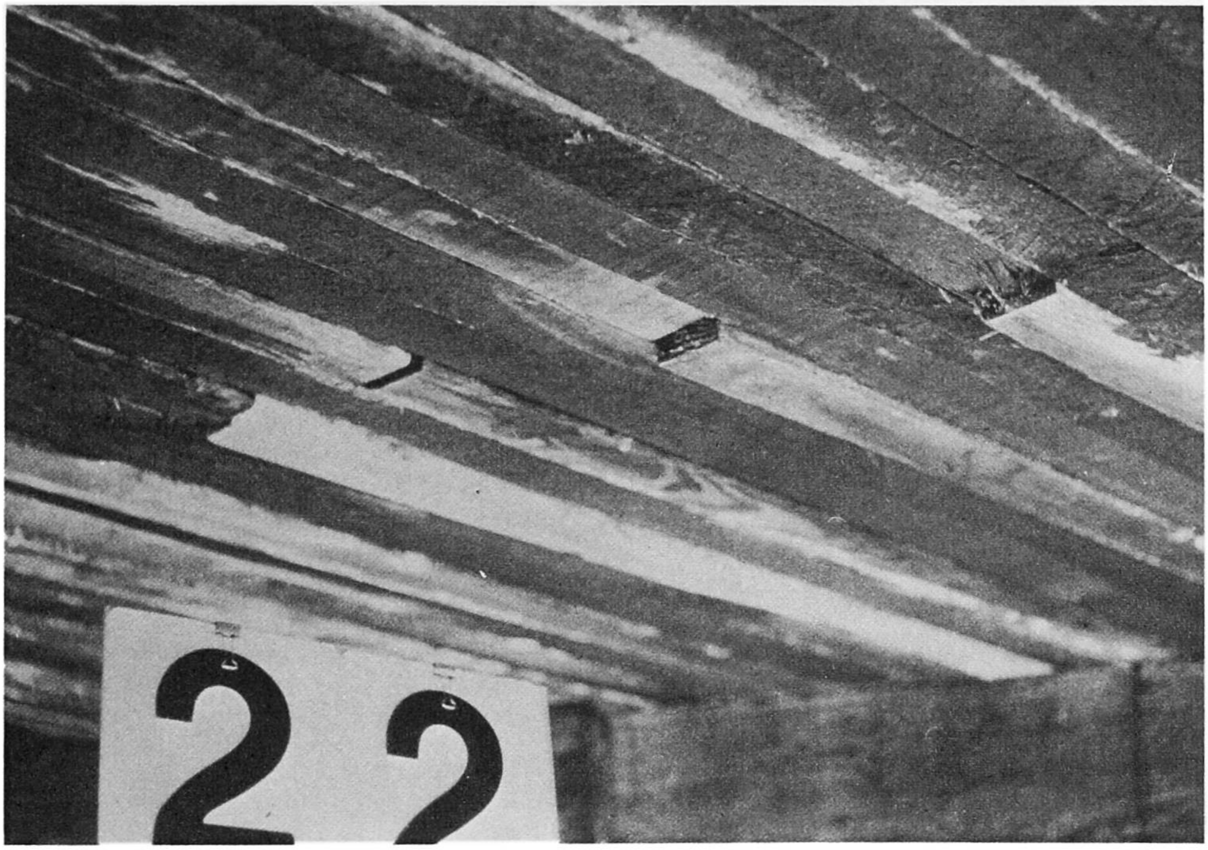


Figure 12 "Piano-Key" Failure of Nailed Decks with Butt-Joints

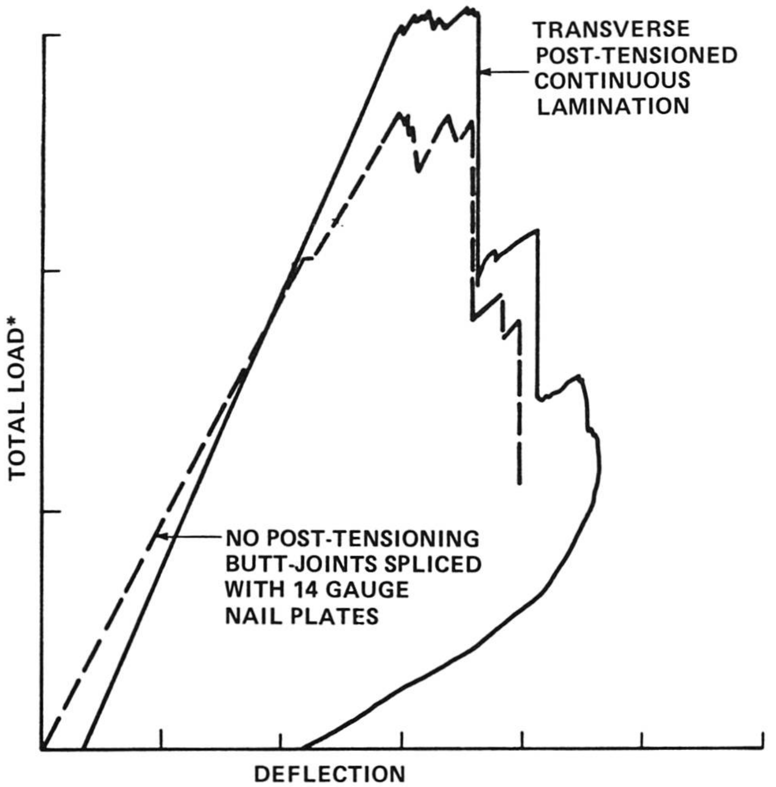


Figure 13 Comparative Ductilities of Laminated Beams

It has been observed in various tests, not exhibited here, that these nail-plates ensure the integrity of the system by preventing piano-key type failures. In addition, the use of the plates on nailed decks introduces ductility, a feature that has only previously been observed in the post-tensioned deck system. Figure 13 displays the load-deflection curves of two independent tests, one belongs to a post-tensioned deck and the other to a nailed deck with joints spliced using nail-plates. Both curves display the same sabre-tooth type of configuration that is associated with the sequential failure of laminations, and both exhibit relatively large plateaus of ductility, a highly desirable structural feature.

The Ontario Bridge Code specifies that all steel connectors and accessories attached to wood bridges are to be hot-dip galvanized. A concern was expressed regarding the long term service life of these relatively thin steel connectors as they are expected to be partially exposed to road salt and and water. The sheet metal comes galvanized, but the punching process exposes bare metal and it is believed that regalvanizing might be required. At present, MTC is carrying out standard corrosion tests to substantiate this probable requirement.

6 DESIGN CONSIDERATIONS

While at present the maximum span of laminated wood decks of any species is limited to about 6.0 m, it is obvious from the previous derivations that the unnailed but tooth-plated and transversely post-tensioned system is capable of spanning considerably more. In this section, the flexural capacity and maximum span lengths of bridge decks made from the three species tested [3] are explored. Where applicable, the provisions of the Ontario Bridge Code [1] are used.

In a traditional bridge deck, the absence of connections at the butt joints reduces the sectional capacity to 67% of the nominal value. But due to structural disintegration of longer spans observed in testing and in some prototype bridges, a reduction factor of only 0.60 is recommended. The post-tensioned system is now to be considered with transverse lines of butt-joints spaced at 1.2 m along the bridge. Therefore a factor of 0.90 appears to be safe, although somewhat conservative. The Ontario Bridge Code specifies a performance factor of $\phi = 0.90$ for wood in flexure. Accordingly, the usable flexural capacity of a lane of 3.30 m (2 x 1.65 m) width of the standard 305 mm (12 in.) thick deck can be determined for the three species tested:

$$S = 3.30 \times 305^2/6 = 51{,}164 \text{ mm}^2\text{.m}$$

$$M_\mu = \phi \times \text{Reduction Factor} \times \text{5th percentile of group strengths} \times S$$

White pine: $M_\mu = 0.9 \times 0.9 \times 15.97 \times 51{,}165 \times 10^{-3}$
$= 661.8$ kN.m/lane

Red pine: $M_\mu = 0.9 \times 0.9 \times 25.99 \times 51{,}164 \times 10^{-3}$
$= 1077.1$ kN.m/lane

Hem fir: $M_\mu = 0.9 \times 0.9 \times 39.07 \times 51{,}165 \times 10^{-3}$
$= 1619.2$ kN.m/lane

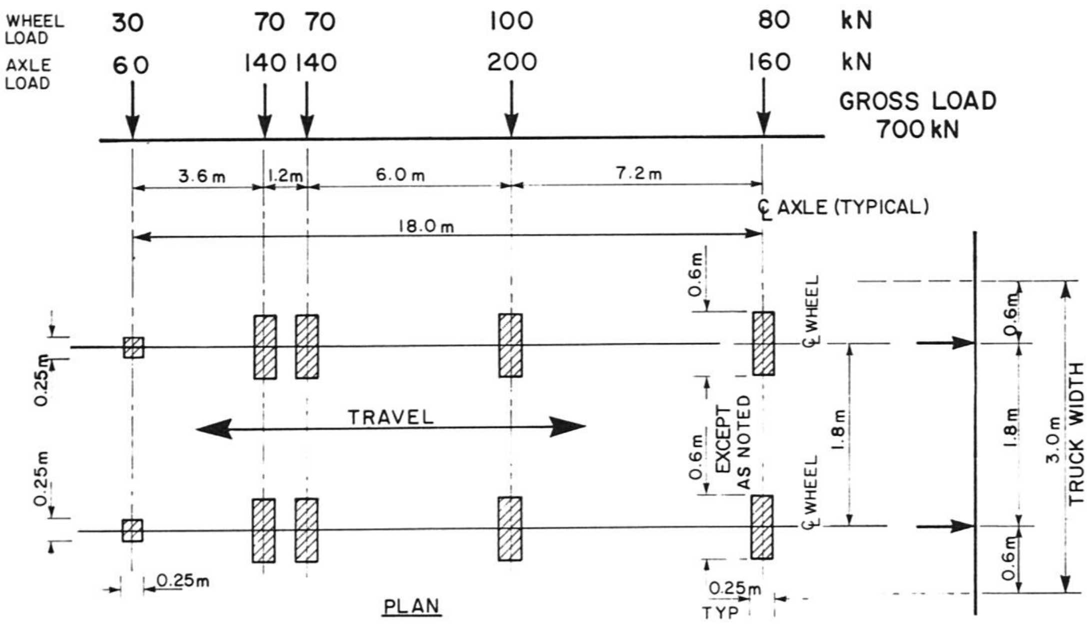


Figure 14 The Ontario Highway Bridge Design Truck

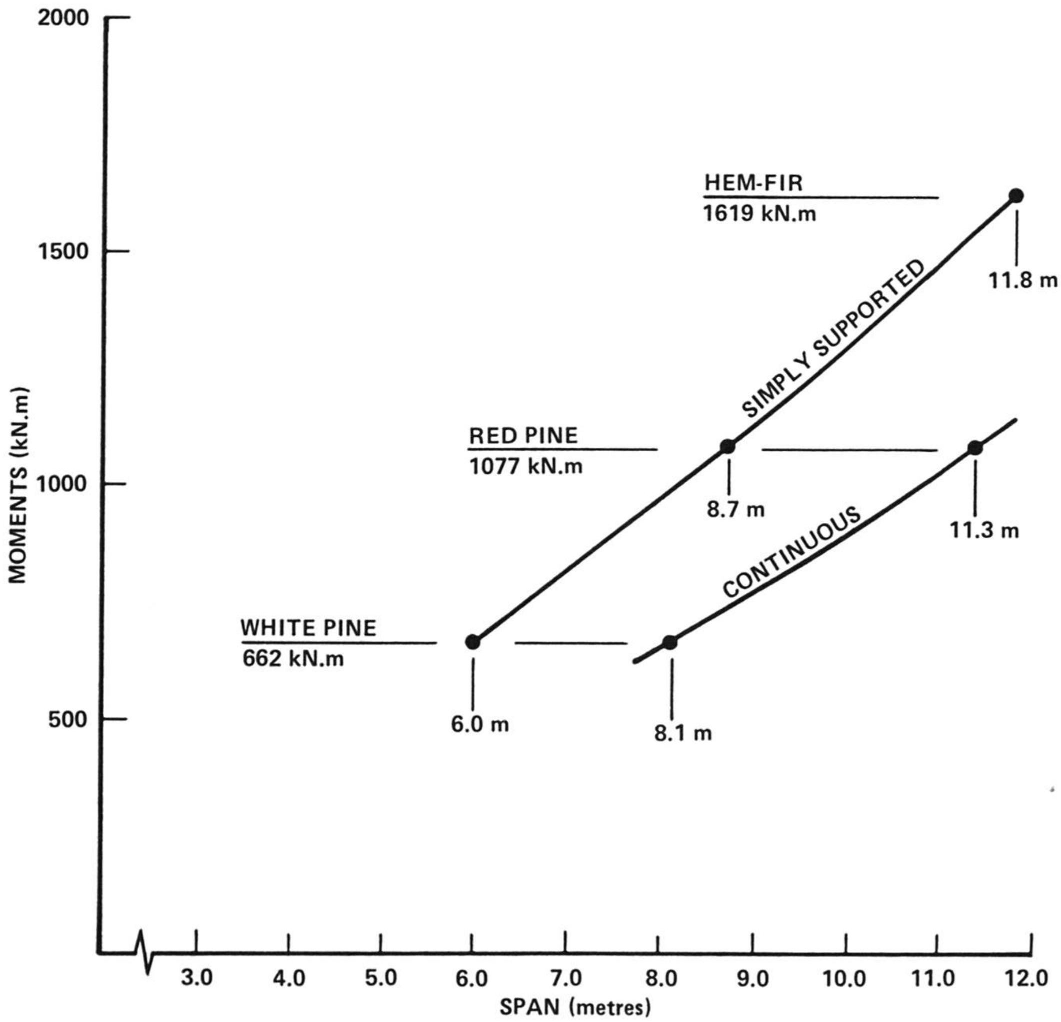


Figure 15 Comparison of Simple and Continuous Span Moments

Loads:

Wood: $7.85 \times 3.30 \times .305 = 7.90$ kN/m/lane

Asphalt (76 mm): $23.50 \times 3.30 \times .076 = 5.89$ kN/m/lane

Applying the appropriate load factors as per the code:

$W_{DL} = 1.20 \times 7.90 + 1.50 \times 5.89 = 18.31$ kN/m/lane

Live load factor: 1.40

dynamic allowance: 0.2205

and $1.4 \times 1.2205 = 1.709$

The Ontario Highway Bridge Design Truck is displayed in Figure 14. It is expected that in the span range considered the 2 x 140 kN tandem with or without the 200 kN single axle will always govern. The calculated maximum live load moments are exhibited in Figure 15. It can be seen that the maximum simply supported spans are:

White pine: 6.0 m (19.7 ft.)

Red pine: 8.7 m (28.5 ft.)

Hem fir: 11.8 m (38.7 ft.)

In general, the Ontario Bridge Code does not specify criteria for deflection, however, a reasonable continuity of the highway profile has to be maintained. Taking the average modulus of elasticity [3], deflections can be calculated under the unfactored truck load (service load) for the maximum spans obtained.

White pine: E_a = 5,725 mPa (.830.3 Ksi) d_{max} = 27 mm

Red pine: E_a = 7,155 mPa (1037.9 Ksi) d_{max} = 67 mm

Hem fir: E_a = 11,050 mPa (1602.6 Ksi) d_{max} = 109 mm

While the maximum deflection of the white pine structure is tolerable, those obtained for red pine and hem fir are certainly not; therefore, it is obvious that their design is controlled by deformation, rather than strength.

The effect of continuity on both strength and deflection has also been explored for white and red pines. For this calculation, a side-span to central span ratio of 0.9 was assumed, and this arrangement gave approximately equal maximum deflections. Figure 15 demonstrates the maximum bending moments obtained and indicates that from the strength point of view, maximum central spans of 8.1 m (26.5 ft.) and 11.3 m (37.0 ft.) can be attained for white pine and red pine, respectively.

Further calculations revealed that maximum unfactored deflections associated with these maximum spans are 29 mm and 64 mm, respectively. It appears, therefore, that on account of the dramatic enhancement in load-carrying capacity due to transverse post-tensioning, only those tree species that have strength characteristics close to white pine can be used economically. In summary, the limits for white pine are as follows:

Simply supported:	maximum span	6.0 m (19.7 ft.)
Continuous:	maximum side span	7.3 m (23.9 ft.)
	maximum central span	8.1 m (26.5 ft.)

It is obvious from previous discussions that, for the above span lengths, the strength of the deck does not govern. In order to make more use of the available strengths of the transversely post-tensioned deck, it can be combined with other structural systems as explained in the next section.

7 A PROPOSED STRESSED WOOD SYSTEM (SWS)

The development of modern bridge engineering has gone through a number of distinctive phases. In the first phase, the deck was supported by the superstructure without intentional interaction between the two; in some cases certain measures were taken to prevent such interaction. In the second phase, the deck was constructed monolithic with the girders or was connected to them by various interface devices, generally referred to as "shear connectors". In the third phase, which is now the common practice, the top flange of cellular structures is being used as the roadway.

It has been demonstrated in the previous sections that the enhancement in strength of the laminated wood deck by transverse post-tensioning is so great that only the lowest grade of timber can be used economically and that deformation always governs the design. In this section, three structural systems are described by which the stiffness of the deck can be improved. Though stiffening elements are attached to and/or integrated with the deck, the latter remains the primary component of the bridge. From the development point of view, this could be treated as a distinctive fourth phase that conceptually illustrates the fact that the purpose of the bridge is to provide a continuous roadway.

7.1 Continuous Decks with Struts

Figure 16a exhibits a three-span continuous deck stiffened by struts at the internal piers. This structure has struts only at every fourth (or as required) laminate. The thickness of the strut and the laminate being the same, the connection can be prefabricated in a shop using the galvanized nail plates. In every other respect the construction is the same as that of the normal prestressed deck, which is shown in Figure 17.

Figure 16b is a comparison between the moment diagrams obtained in strutted and unstrutted decks for a centrally located concentrated load. A reduction can be observed in both positive and negative moments in the strutted version. Calculations indicate that maximum deflections, which are proportional in general to the square of the moments, are reduced by approximatley 40%. For a structure, in which the side span is 75% of the central span and the horizontal projection of the strut is one third of the side span, the 305 mm thick deck provides the following limits by deflection in central spans:

White pine:	9.5 m (31.3 ft.)
Red pine:	10.3 m (33.7 ft.)
Hem fir:	11.9 m (39.0 ft.)

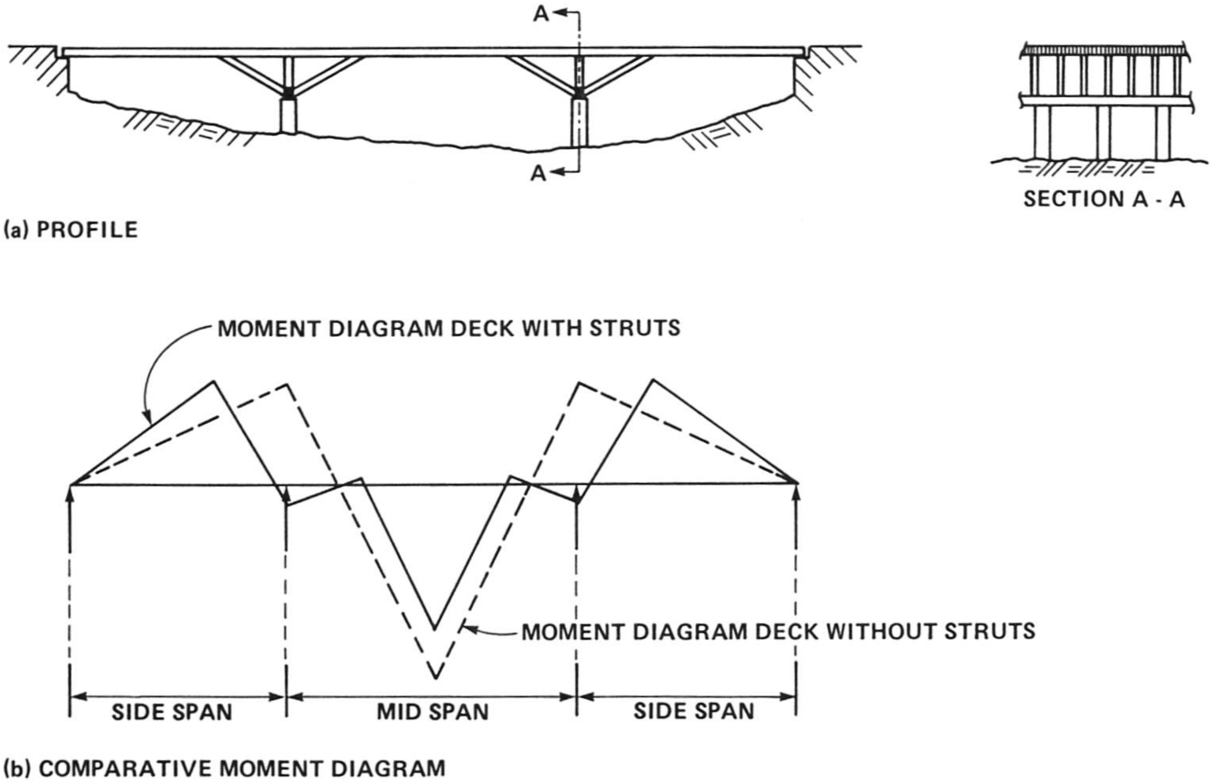


Figure 16 Continuous Deck with Struts. Comparative Moment Diagram

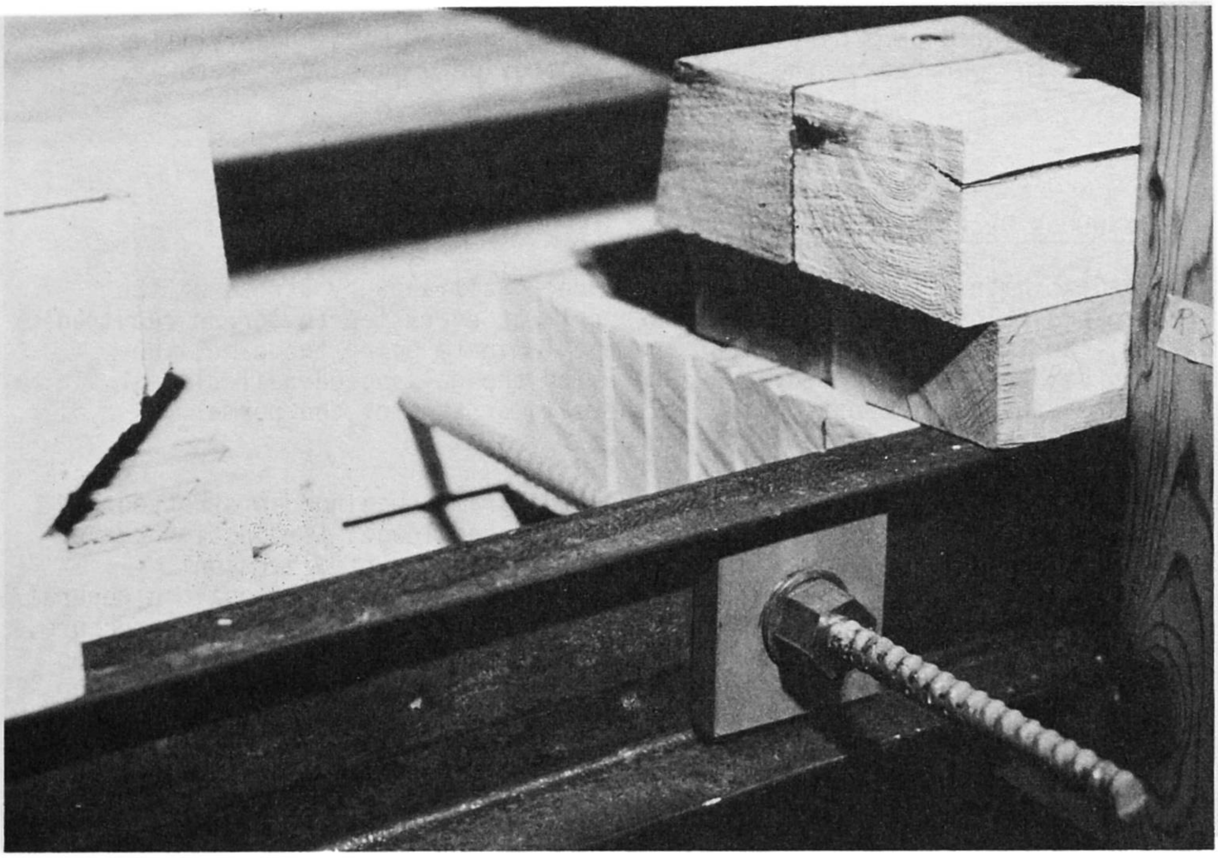


Figure 17 The Regular Post-Tensioned Wood System

The idea of stiffening by struts is by no means new. As a matter of fact, it must have been a standard method in ancient times, leading to the development of modern truss bridges. One such example is exhibited in Figure 18. The bridge is located in Kashmir at the toe of the Himalayas at a location that is hard to access and that has a climate comparable to that of Canada.

7.2 The Integrated Deck Truss

Figure 19 illustrates schematically an integrated deck truss system, that is (in principle) very similar to the strutted deck, except that a number of laminates are turned into full trusses. The trusses can be prefabricated in segments - up to 27.50 m (90 ft.) - and spliced together by bolts or galvanized nail plates on site. The system is believed to permit simply supported spans 75 m (245 ft) and internal spans 100 m (300 ft.) for continuous bridges.

This type of structure has never been built. Model studies are presently under way to evaluate constructional feasibility, strength, and appearance. The lateral stability of the bottom chords would be provided by transversely prestressing solid wood spacer blocks at every bottom panel point. This would result in a torsionally compact and stiff structure.

7.3 Deck Stiffened by Tie Bars

Many of the existing wood truss bridges have their cross beams stiffened and strengthened by the addition of bars as illustrated in Figure 20. The system has an excellent record in Canada. Longitudinal bars can be used to stiffen the transversely post-tensioned laminated decks as shown schematically in Figure 21.

Exploratory calculations indicate that the behaviour of this system with respect to both stress and deformation is only marginally different from the three-span continuous structures discussed in the previous sections.

The only difference appears to be the presence of the horizontal compressive force is only about 2.0 mPa (300 p.s.i.) but its presence is beneficial to the deck as it creates additional confinement in the longitudinal direction. As well, it takes advantage of the additional strength available in these decks. Using an internal span ratio of 0.9, the following maximum spans can be permitted:

White pine: l_{max} = 22.7 m (74.4 ft.)

Red pine: l_{max} = 24.5 m (80.3 ft.)

Hem fir: l_{max} = 27.5 m (90.2 ft.)

The basic deck system could, of course, permit a number of other possible structural permutations, but the scope of this paper does not permit a complete discussion of them.

8 COST, ENERGY AND OTHER CONSIDERATIONS

The precision drilling of holes in the laminates and their splicing by galvinized nail-plates assume the availability of certain manufacturing facilities. These do not yet exist as no new bridge has even been built using the systems proposed here.

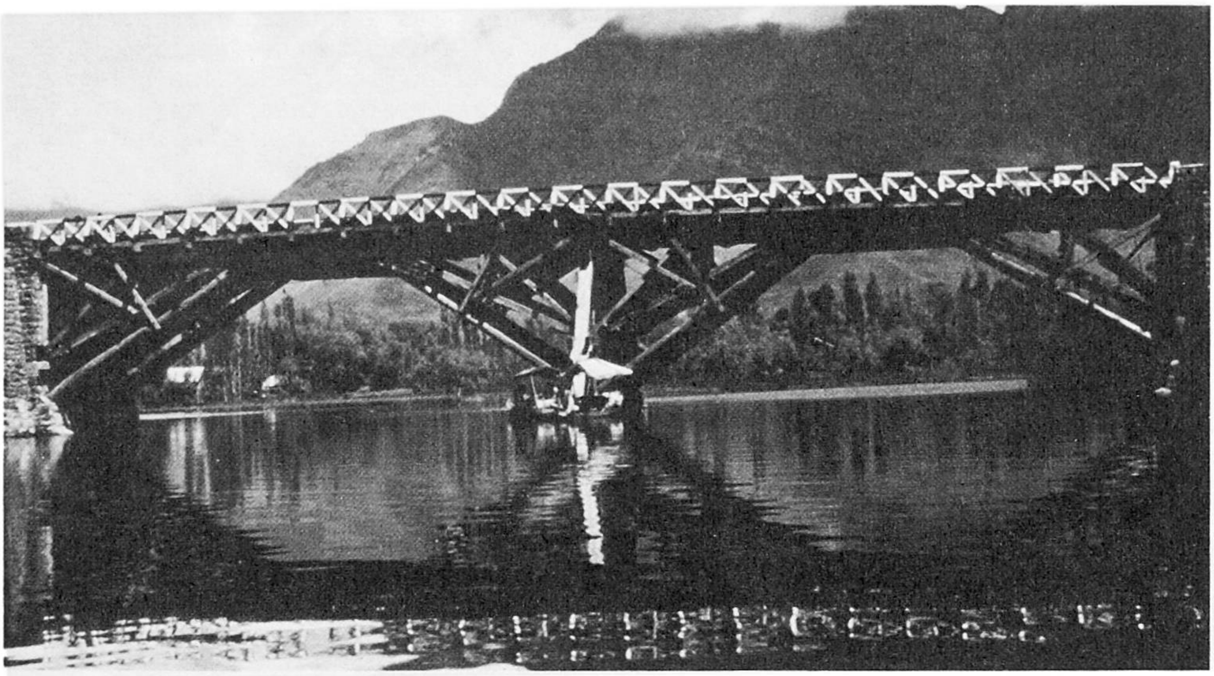
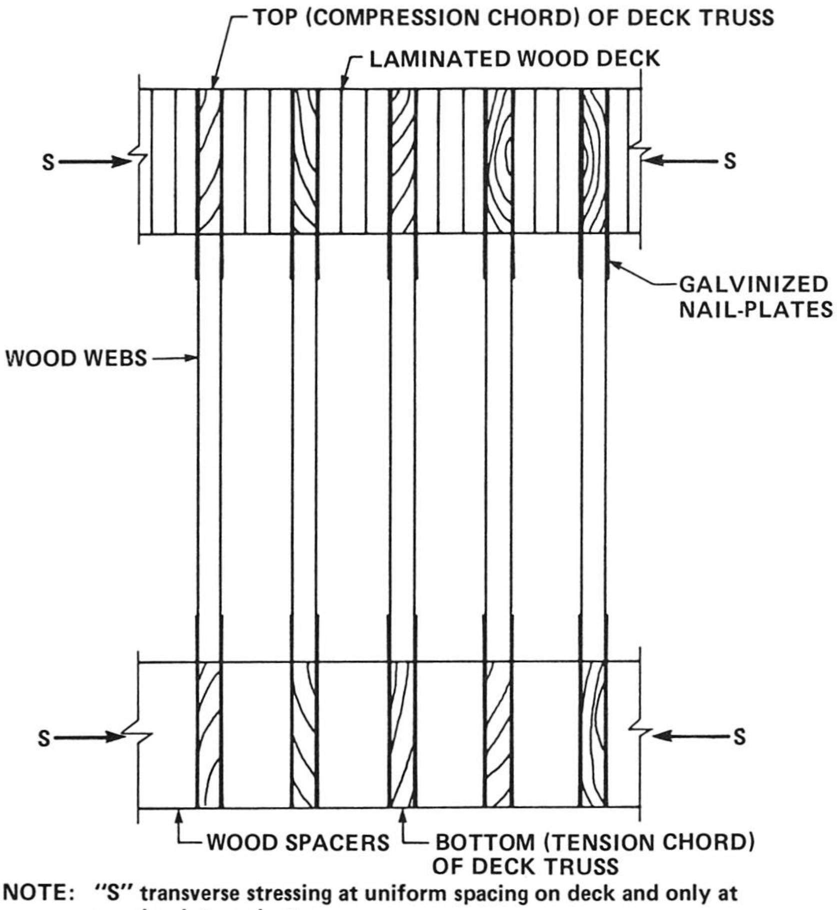


Figure 18 A Bridge in Dal Lake, Kashmir, India



NOTE: "S" transverse stressing at uniform spacing on deck and only at panel points on bottom

Figure 19 Partial Section of a Fully Integrated Deck Truss

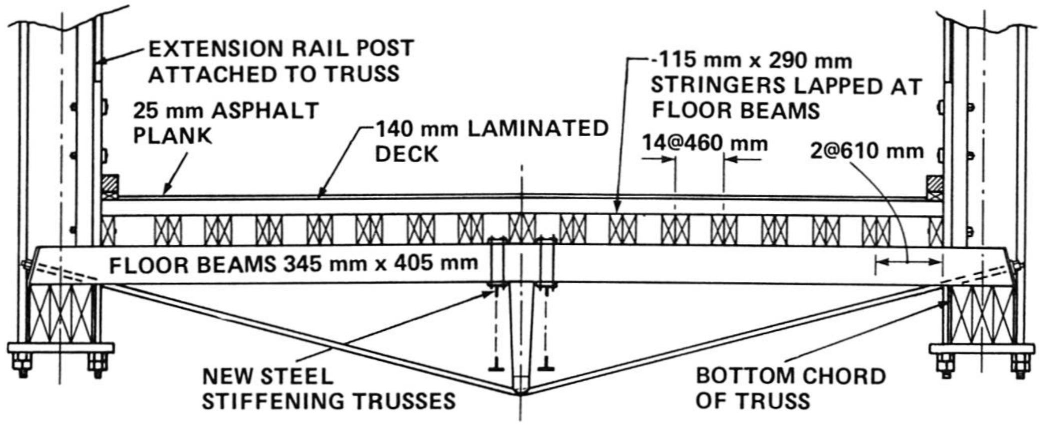


Figure 20 **Cross Section of Truss Span at Sydney Creek Bridge. Traditional Cross Beam Design with Tie-Bars**

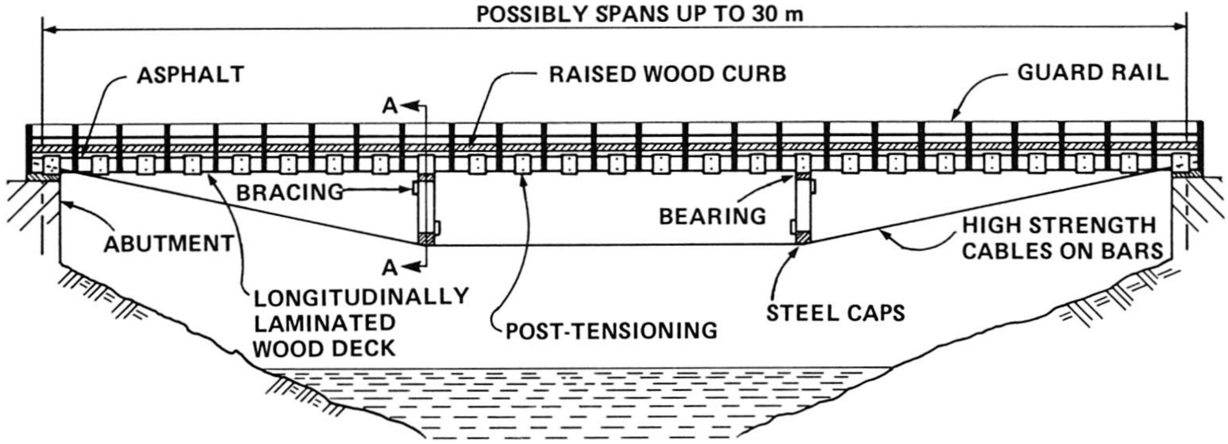


Figure 21 **Long-Span, Transverse Post-Tensioned Deck System Using Tie-Bars**

But from the three bridges that have been rehabilitated by transverse post-tensioning it appears that the cost (in Canadian dollars) of a new bridge would not exceed \$325/m^2 (\$30/sq foot) of deck area, while the rehabilitation including new bulkheads and the stripping of existing wearing surface and subsequent resurfacing, is about \$95/m^2 (\$9/sq. foot).

The construction of these bridges is entirely mechanical and therefore could be done any time of the year. Since the bridges are built up from relatively small elements, in most cases the presence of a crane may not be required. When larger prefabricated components are involved, such as those of the integraed deck-truss systems, a crane would be necessary, but these members are still rather light in comparison with steel or concrete components of comparable size.

Observation of these transversely post-tensioned decks shows them to be extremely tight and waterproof. In addition, the prestressing forces do not permit the development of cracks when heated, and therefore resist the escape of flammable gases. As a result, the deck is believed to be essentially fireproof. The loss of prestressing is easily measured and the bars could be retightened, if necessary, at certain time intervals. The epoxy coating is guaranteed to protect the Dywidag bars for approximately 35 years. The system permits the removal of the bars, one at a time, without loss of strength and interruption of vehicular traffic, for examination and replacement if so required.

Wood is a renewable structural material and it has been shown consistently throughout this paper that the enhancement by transverse post-tensioning makes feasible the application of the lowest grade species for bridge construction.

The system requires an extremely low amount of energy, an aspect that will have major consequences in the years to come. Preliminary energy calculations, including the mining of ores to the galvanizing of the nail plates, indicate the superiority of wood structures. The energy requirement of different materials have been compared for a 20 m (60 ft.) span, two lane, standard AASHTO bridge with the following results:

Steel girders with reinforced concrete deck	2 100 000 KW hours
Reinforced concrete T-beams	245 000 KW hours
Glued-laminated girders	185 000 KW hours

As the proposed system makes use of rough-cut, unplaned wood without nailing or gluing, the expected energy requirement for a comparable size of structure is not expected to exceed 100 000 KW hours.

The construction of these bridges is relatively simple and can be done by local people under the supervision of a trained foreman. Thus the import of expensive craftsmen can be avoided and local employment can be stimulated. The maximum use of locally available material, softwood or hardwood, can also be attained by the application of the proposed system.

Limited tests carried out so far indicate that these decks have low dynamic responses, a remarkable resilience against cyclic loading, and outstanding characteristics with regard to structural ductility.

The Ontario Bridge Code requires that all structural wood be pressure treated in order to prevent decay. To reduce the volumetric change caused by varying ambient humidity and, therefore, to reduce changes in the post-tensioning forces, oil-borne chemicals like pentachlorophenol and creosote are preferred. The effect of road salt on wood and these chemicals, which have been used to preserve the wood is not fully known. However, wood members which were properly treated have been observed on structures built in the 1930's.

The Bridge Code also specifies hot-dip galvanizing for all steel elements and attachments. In rural areas where the application of road salt is expected to be non-existant or minimal, the channel bulkhead could conceivably be made of self-protecting steel and left unpainted. The brownish colour of the steel combined with the dark wood provides a rustic look which will pleasantly blend into a natural environment.

9 CONCLUSIONS

The transversely post-tensioned wood bridge decks offer a superior load carrying capacity, derived primarily not from the strength of the material employed, but from structural confinement. These decks are capable of sustaining the loads specified by the Ontario Bridge Code, which are some of the heaviest in existence.

The decks make use of low grade timber that is available in most northern countries. Since wood is a renewable resource, its application requires minimum amount of constructional energy. Thus, it is now time to consider wood for higher applications.

This proposed stressed wood system (SWS) operates contrary to the traditional structural engineering approach. Until now, for the most part designers would isolate discrete linear elements from the overall structure in order to simplify their calculations. The stressed wood system, on the other hand, derives most of its strength from the confinement of discrete, linear elements into a structural continuum.

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