Development of a Slab-on-Girder Wood-Concrete Composite Highway Bridge

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Summary

This paper examines the development of a slab-on-girder wood-concrete composite highway bridge superstructure. The proposed bridge makes use of a slender ultra-high performance fibre-reinforced concrete (UHPFRC) deck made partially-composite in longitudinal bending with glued-laminated wood girders. Longitudinal external unbonded post-tensioning is utilized to increase span capabilities. Prefabrication using double-T modules minimizes the need for cast-in-place concrete on-site. Durability is realized through the highly impermeable UHPFRC deck slab that protects the girders from moisture. Results show that the system can span up to 30 m while achieving span-to-depth ratios equivalent to or better than competing slab-on-girder bridges.

Keywords: wood-concrete composites, timber bridges, short span bridges, external post-tensioning

1. Introduction

1.1 Scope

Research was recently completed at the University of Toronto to develop a single-span wood-concrete composite slab-on-girder bridge superstructure for use on secondary highways, by making use of glued-laminated (glulam) girders, an ultra-high performance fibre-reinforced concrete (UHPFRC) deck slab, and external unbonded prestressing [1]. This superstructure is referred to as the "proposed concept".

This paper focuses exclusively on superstructure design, for spans ranging from 10 m to 30 m. Fatigue is not considered, nor are lateral loads due to wind and seismic loading.

The design of the proposed concept is based on CAN/CSA-S6-06 Canadian Highway Bridge Design Code (CHBDC). Since the proposed concept is not explicitly recognized by the CHBDC, it was necessary to utilize other design standards, research papers, and engineering principles to complete the design. This paper presents some of the relevant design information that would be required in a design standard for the proposed concept, and introduces potential future research concepts for wood-concrete composite bridges.

1.2 Wood-Concrete Composites

Wood-concrete composites (WCCs) are a structural configuration wherein a concrete slab is made composite in longitudinal bending with either wood girders or a wood deck beneath. Composite action is achieved by means of a mechanical shear connector. The key benefits of WCCs are improved strength and stiffness. Figure 1 illustrates the qualitative interlayer slip, longitudinal deflections, and elastic strain distributions for non-composite, partially-composite, and fully-composite WCC systems under the same midspan load.

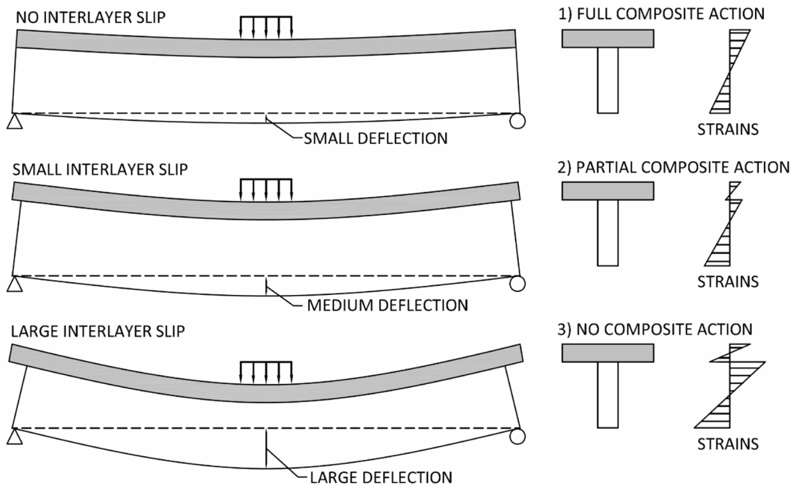

Figure 1 Varying degrees of composite action in a wood-concrete composite system (adapted from [2])

Large interlayer slip and longitudinal deflections develop in a wood-concrete system with no composite action because both the wood and concrete components undergo bending independently. The interlayer slip and deflections decrease when partial composite action is introduced. Furthermore, partial composite action reduces the strains in both materials, with the concrete tending towards compressive strain and the wood tending towards tensile strain under positive moment. There is no interlayer slip in a fully-composite system. The deflections are also smaller relative to partially-composite systems, and especially non-composite systems. In a fully-composite system, the concrete experiences predominantly or entirely compressive strains, while the wood experiences predominantly or entirely tensile strains. Due to flexibility of the shear connectors, all WCCs known by these authors, including the proposed concept, undergo partial-composite action.

WCCs combine wood and concrete to capitalize on the structural attributes of each material [3]. Concrete performs best in compression, and has relatively low tensile strength. Concrete is well-suited at the top of a WCC section, where it can be designed to resist only compressive stresses under partial or full composite action. The wood girders provide a very lightweight form of tensile resistance when compared to an equivalent all-concrete section [4].

1.3 The Need to Develop a Wood-Concrete Composite Bridge

In Ontario, non-composite slab-on-wood-girder bridges have been historically limited to spans of 15 m, beyond which the wood girders became too large for practical purposes [5]. The guidance offered by the CHBDC for WCCs is restricted to a specific slab-on-deck WCC system that is limited to spans up to 8 m [6]. As a result, WCC systems are rarely utilized in Ontario, or any other part of Canada.

Forestry is the second-largest industry sector in Ontario, with over 26 million of Ontario’s 71 million hectares of forested land being sustainably harvested in commercial logging ventures [7]. However, wood is an underutilized material in Ontario bridge design. Only approximately 250 of the 2800 provincially-owned bridges are made primarily of wood [5]. With the Ministry of Transportation of Ontario having been a driver of wood bridge innovation in Ontario [5] [8] [9], it is seemingly unlikely that the municipal bridge inventory contains a higher percentage of wood bridges than does the provincial bridge inventory. The need for a wood bridge system in Ontario that is cost-competitive with conventional solutions is apparent given the scarcity of wood bridges, in spite of a sizeable forestry sector.

An examination of civil engineering undergraduate curricula in Canadian universities indicates that most schools offer no more than one course on the design of wood structures. Many schools do not even offer such a course. At best, students might be taught wood design over a few weeks in a general structural design class. Engineers who are familiar and comfortable with wood design have typically studied the subject at the graduate level, or learned it informally on the job. This educational limitation has left young engineers lacking in technical knowledge and ability to design wood structures. The proposed bridge design concept in this paper presents an opportunity to incorporate wood more commonly in infrastructure design.

2. The Proposed Concept

2.1 The Structural System

The proposed concept is a two-lane slab-on-girder bridge consisting of a 110 mm-thick UHPFRC slab and eight 365 mm-wide glulam girders. The cross-sectional dimensions have been standardized for all spans of the proposed concept. Only the glulam girder depths vary with span. Figure 2 illustrates a plan, longitudinal section, and cross-sections of the proposed concept for a 30 m span. The layouts would be similar for spans ranging from 10 m to 25 m.

UHPFRC was chosen rather than conventional concrete because its high tensile strength precluded the need for passive reinforcement in the deck slab, except in the cantilevers where reinforcing steel is necessary to resist barrier impact loads. The use of UHPFRC also allows for a slender slab that is much lighter than traditional deck slabs, thereby improving the live load capacity of the system. UHPFRC has very low permeability and high freeze-thaw resistance that enhances system durability. Although expensive, it is anticipated that the premium paid for the UHPFRC will be offset by these benefits over the life of the bridge.

Each glulam girder is prestressed by external unbonded longitudinal tendons located on each side of each girder. The tendons are deviated along a polygonal profile at the span fifth-points by U-shaped steel saddle deviators. The inclined tendons reduce longitudinal shear demand, flexural demand, and deflections of the glulam girders. The tendons are anchored inside full-width UHPFRC end buttresses that are poured monolithically with the deck slab. The buttresses house the prestressing anchorages and protect the ends of the glulam. The buttresses absorb the high forces within the anchorage zone, and distribute the anchorage forces between the slab and girders. The buttresses can be constructed with a shear key to transmit lateral loads from the superstructure to the substructure.

The proposed concept comprises two interior and two exterior full-span prefabricated double-T modules. Each module consists of two glulam girders made partially-composite with a UHPFRC deck slab by longitudinal shear connectors. The modules are match-cast so that their deck fasciae are geometrically compatible when abutted. Transverse post-tensioning tendons within the diaphragms and end buttresses are used to connect the modules after they have been erected.

Each module has several 200 mm-wide UHPFRC diaphragms that are cast monolithically with the deck slab. Notches are cut into the compression zones of the glulam girders to achieve continuous full-width diaphragms. No wood in tension is to be removed. The diaphragms improve live load distribution between girders and house the transverse post-tensioning that connects adjacent modules. Sufficient live load distribution occurs when the 15-30 m spans have diaphragms at their span tenth-points, and when the 10 m span has diaphragms at its fifth-points.

2.2 Fabrication & Construction

Prefabrication of the double-T modules affords many benefits. Prefabrication improves quality control by providing the best curing conditions for the UHPFRC, and results in a higher level of durability [10]. Prefabrication also significantly reduces the on-site duration of construction, thus reducing traffic impacts associated with any construction closures. The prismatic shape of the girders and slab allows for straightforward manufacture of the modules. The modules are light and small enough to fit on conventional flatbed trucks for transportation to site. Stability during erection is provided by the use of two-girders per module, with the UHPFRC slab and diaphragms stabilizing the glulam girders against buckling.

Prefabrication begins with the manufacture of the glulam girders. Two narrow longitudinal slots are cut along the entire top face of each girder for the longitudinal shear connectors. The girders are then pressure-treated with a wood preservative, if necessary. Two longitudinal shear connectors are glued into the slots. Two girders are placed in a casting bed for construction of the UHPFRC slab, diaphragms, and end buttresses. All longitudinal and transverse post-tensioning hardware is placed before casting. The technique of match-casting is utilized to preclude the need for on-site cast-in-place concrete closure joints.

The modules are erected on-site by a mobile crane. The transverse tendons are stressed to connect the modules together. The longitudinal tendons are then stressed. The curbs, steel PL-2 barrier, and asphalt wearing surface can be constructed after all stressing operations. Since the curb is cast on site, conventional high-early strength concrete could be used as the curb material rather than UHPFRC. A life-cycle cost analysis would be required to determine the best alternative for the curb material.

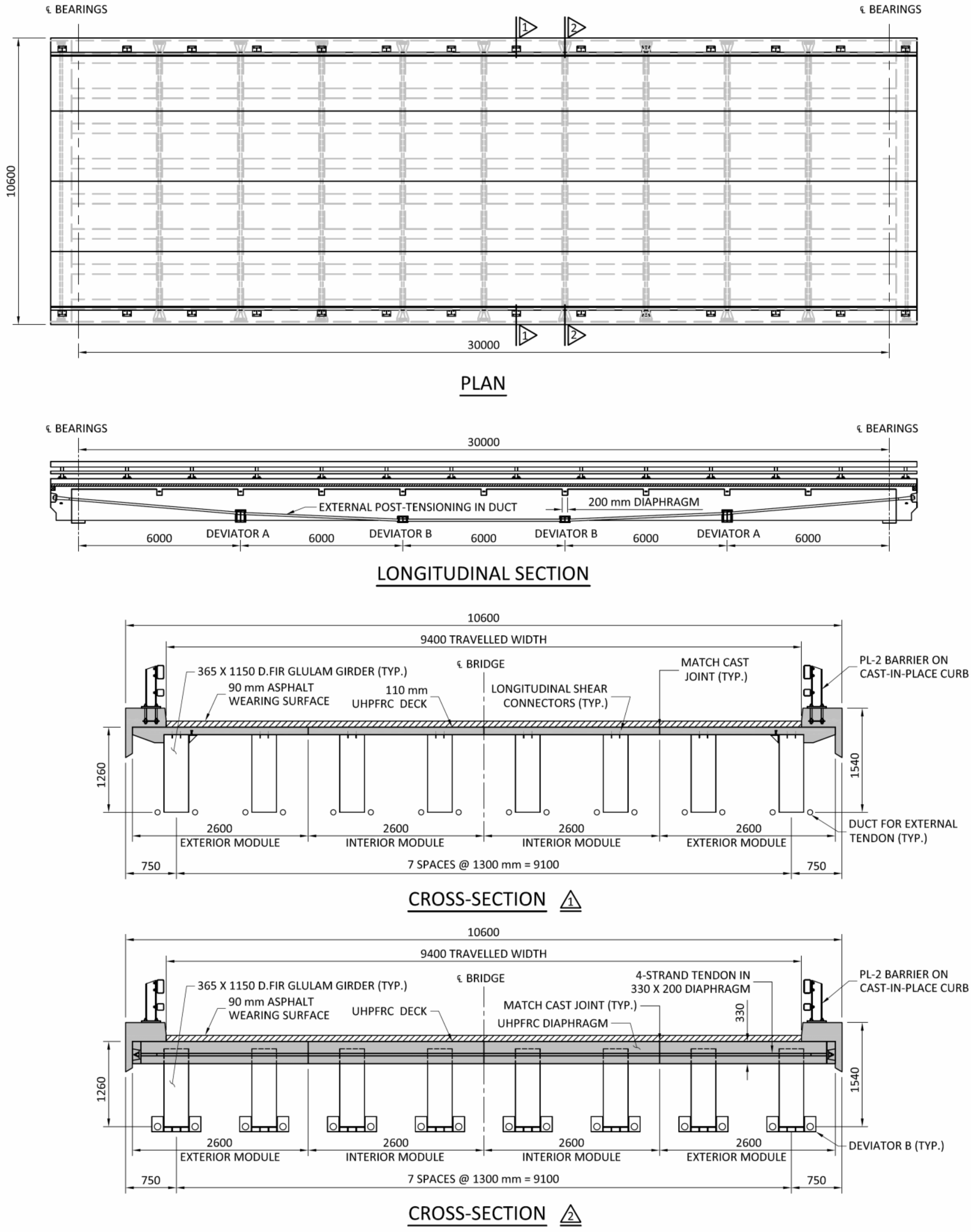

Figure 2 Plan, longitudinal section, and cross-sections for a 30 m span

2.3 Materials

The glulam girders are assumed to be Douglas-fir 24f-EX bending stress grade commonly manufactured in Canada.

The assumed properties for the UHPFRC are based on a mix design developed and tested at the University of Toronto [11] [12]. The peak compressive stress is 120 MPa, elastic modulus is 42065 MPa, cracking stress is 7 MPa, peak tensile stress is 14.6 MPa, long-term creep coefficient is 0.8, and total shrinkage strain is 552 microstrain after 50 days.

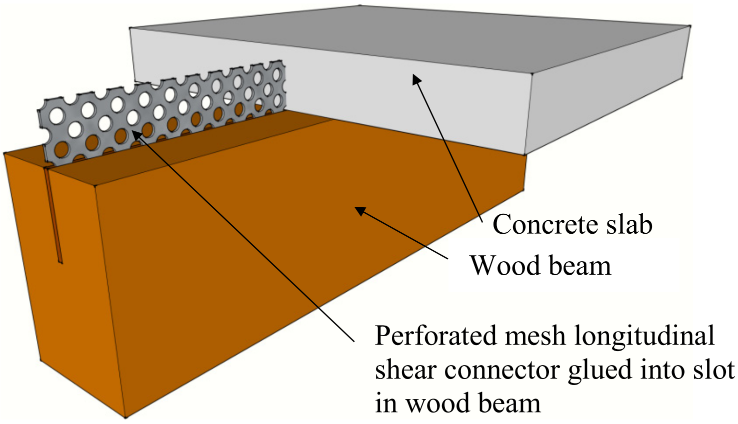

Figure 3 Perforated mild steel mesh longitudinal shear connector (adapted from [13])

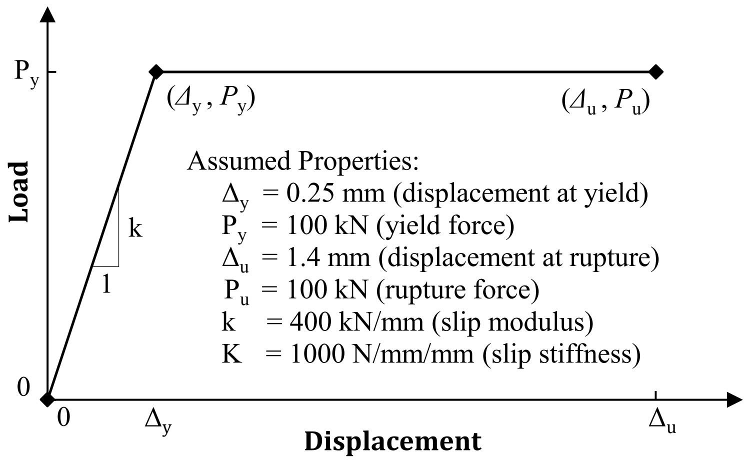

Figure 4 Idealized load-displacement curve for assumed longitudinal shear connector

Mild steel perforated mesh longitudinal shear connectors are assumed, per Bathon and Clouston [13], as shown in Figure 3. The 100 mm-deep connectors are embedded 50 mm into the glulam girders, and 50 mm into the UHPFRC slab, over the full length of the girders, with local discontinuities at diaphragm locations. This particular connector was chosen because it has well-developed, experimentally validated mechanical properties that are superior to most other shear connectors presented in academic literature, particularly with respect to stiffness and ductility. A different connector could be utilized, but the system behaviour would require re-evaluation.

The proposed shear connector was developed at the University of Applied Sciences, Wiesbaden, Germany [14] [15]. Figure 4 illustrates the idealized load-displacement curve of the connector. The behaviour is linear-elastic until yield, and perfectly-plastic until rupture. The observed plastic region is important because it indicates that the connector will allow slip between the wood and concrete as it approaches its own capacity, without reducing the shear force transferred between the wood and concrete components. This slip will result in system deflections that should alert users of the impending system failure brought on by the impending connector rupture.

2.4 Durability

Bridge durability can be enhanced by the use of WCCs. Controlling the ingress of moisture into wood is the most important aspect of preventing wood decay [16]. A WCC slab-on-girder bridge provides protection of the wood girders by means of the concrete deck slab. The deck can greatly increase the durability of the wood girders by preventing contact with moisture that originates on the deck surface [14]. Due to composite action and prestressing, the deck slab can be kept under compression, thereby precluding tensile cracks that could expose the wood girders to moisture from above [4]. Using UHPFRC for the deck slab is more effective than a deck slab made of conventional concrete because the material has very low permeability and is extremely resistant to freeze-thaw cycles [11]. For these reasons, it is anticipated that the moisture content of the glulam can be kept below its fibre saturation point, making wood preservation unnecessary for prevention of decay by moulds or fungi (note that preservatives may be desirable for preventative purposes and/or as a deterrent to insect attack). The prestressing tendons will be protected in greased ducts to reduce the risk of corrosion.

3. Analysis of the Proposed Concept

3.1 The γ-Method

The CHBDC does not address partially-composite slab-on-girder WCCs. Consequently, the system was analysed as single T-girders by use of the "γ-method" ("gamma-method") that is described in Annex B of Eurocode 5: Design of Timber Structures [17]. The effective slab width was determined to be the centre-to-centre glulam girder spacing, implying that the entire slab participates in resisting loads acting on the longitudinal system. Transverse live load distribution was determined by a grillage analysis. The key assumptions of the γ-method are as follows:

- the wood, concrete, and longitudinal shear connectors behave linearly-elastically;
- the wood and concrete are connected by a mechanical fastener with constant slip stiffness *K*;
- the beam is simply-supported over a length L and transversely loaded to produce a shear force $V = V(x)$ and bending moment $M = M(x)$ that varies sinusoidally or parabolically;
- plane sections do not remain plane for the cross-section as a whole due to inter-component slip, but plane sections remain plane in each component.

The γ-method estimates the effective bending stiffness, as well as the normal stresses due to combined bending moments and axial forces within the system. The following explanation of the γ-method is best understood in conjunction with an examination of Figure 5.

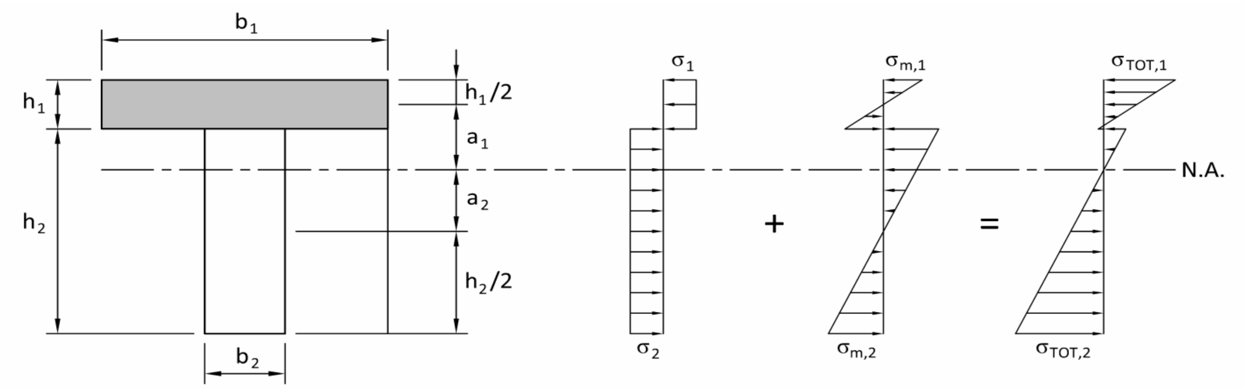

Figure 5 Cross-sectional geometry and stresses according to the γ-method

The effective bending stiffness $(EI)_{eff}$ of a simply supported WCC beam of length$L$, and constant slip stiffness *K*, can be calculated using equations 1 to 7,

$$(EI)_{eff} = \sum_{i=1}^{2}(E_i I_i + \gamma_i E_i A_i a_i^2) \tag{1}$$

$$A_i = b_i h_i \tag{2}$$

$$I_i = \frac{b_i h_i^3}{12} \tag{3}$$

$$\gamma_1 = \frac{1}{1 + \frac{\pi^2 E_1 A_1}{K L^2}} \tag{4}$$

$$\gamma_2 = 1 \tag{5}$$

$$a_1 = \frac{h_1 + h_2}{2} - a_2 \tag{6}$$

$$a_2 = \frac{\gamma_1 E_1 A_1 (h_1 + h_2)}{2(\gamma_1 E_1 A_1 + \gamma_2 E_2 A_2)} \tag{7}$$

where E is the modulus of elasticity, I is the moment of inertia, A is the cross-sectional area, a is the distance between the centroid of a component and the partially-composite neutral axis, γ is the shear coefficient, b is the component width, and h is the component depth. Subscripts 1 and 2 refer to the concrete and wood components, respectively. The maximum axial stress σ_i and maximum bending stress $\sigma_{m,i}$ for a given internal bending moment M, are calculated according to equations 8 and 9.

$$\sigma_i = \mp \frac{\gamma_i E_i a_i}{(EI)_{eff}} M \quad (-\text{ for } i = 1, + \text{ for } i = 2) \tag{8}$$

$$\sigma_{m,i} = \mp \frac{0.5 E_i h_i}{(EI)_{eff}} M \quad (-\text{ for the top fibres}, + \text{ for the bottom fibres}) \tag{9}$$

3.2 Effect of Prestressing

The tensile force in the unbonded tendons is a function of their elongation. It is proposed to ignore the additional strain in the tendons brought about by overall system deformations. Neglecting this additional strain is conservative and greatly simplifies the calculation of the prestressing force. The simplification is made by considering prestressed glulam analogous to prestressed concrete in an uncracked state. Before cracking, the latter is known to experience negligible additional strains in the tendons [18]. Noteworthy additional strains occur only after cracking. Since flexural cracking of concrete is analogous to the flexural rupture of glulam, it is posited that the additional strain in the tendons of the proposed concept will be negligible, even at the ultimate limit state. With flexural rupture of the glulam being taken as system failure, the additional strain in the tendons incurred beyond such rupture need not be calculated. Therefore, the prestressing force can be treated as a series of static loads applied at the anchorages and deviators.

The total loss of prestress, at and after transfer, is assumed to be 20% of the ultimate tendon stress. This assumption is based on a review of prestressed wood literature [19] [20].

4. Performance of the Proposed Concept

A slab-on-girder system using standardized girders from the Canadian Precast Prestressed Concrete Institute (CPCI) is the most common bridge superstructure in many parts of Canada for spans up to 45 m. Figure 6 illustrates a typical cross-section of this superstructure. It consists of parallel, prestressed, precast, concrete I-girders made composite with a cast-in-place concrete deck.

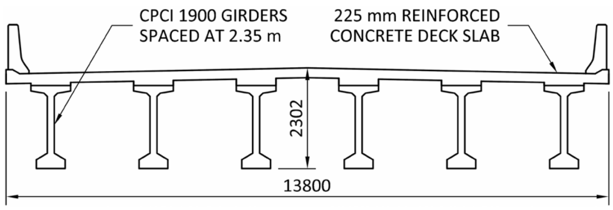

Figure 6 Cross-section of a typical slab-on-CPCI-girder bridge

The current popularity of the CPCI girder system stems from its large degree of standardization, resulting in relatively straightforward design and construction techniques. The precast girders provide an obvious construction time savings; although, this savings is somewhat undermined by the cumbersome process of casting the deck slab. This non-prestressed

Table 1 Single-span CPCI girder bridges used for comparative purposes

Span	Deck Width	# of Girders	Girder Type
16.6 m	13.5 m	7.0	CPCI 900
22.0 m	14.2 m	7.0	CPCI 1200
31.5 m	20.5 m	8.0	CPCI 1600

slab tends to crack in service, consequently compromising its durability. These cracks allow for the penetration of de-icing chemicals that can corrode the reinforcing steel. Nonetheless, the CPCI girder system remains popular in many parts of Canada.

Given its popularity and given that it is a slab-on-girder system, the CPCI girder system is a suitable benchmark for comparison with the proposed concept. Specifically, comparisons can be made with regard to superstructure weight, volume of cast-in-place concrete, and slenderness. The three single-span CPCI girder bridges presented in Table 1 have been chosen for comparison to the proposed concept. Three is a small sample; however, given the large degree of standardization among CPCI girder bridges, it is expected that the variance in geometrical and structural properties is rather small between any two CPCI girder bridges of similar span and width.

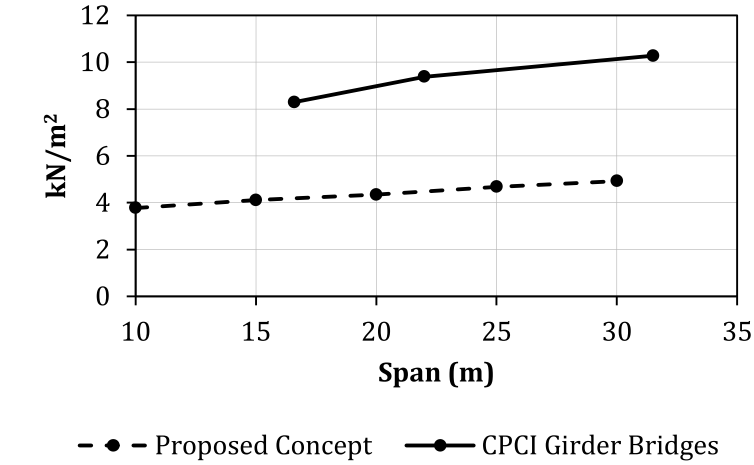

Figure 7 Superstructure weight per unit deck area

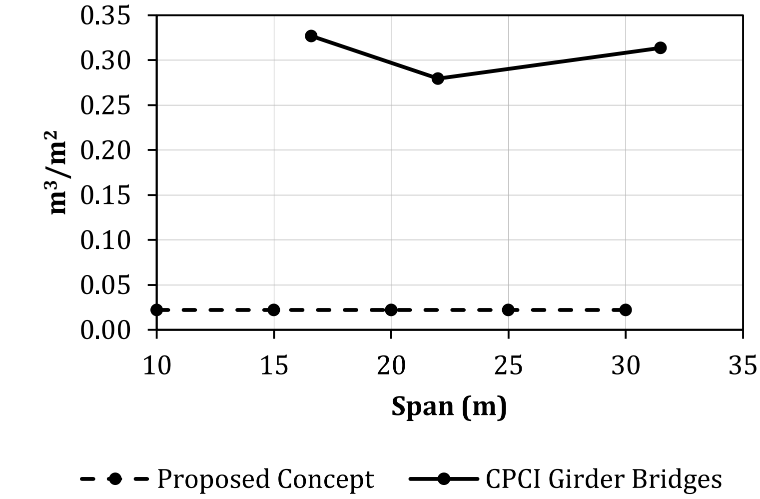

Figure 8 Volume of cast-in-place concrete per unit deck area

Before making comparisons to the CPCI girder system, recall that the proposed concept is valid for spans ranging from 10 m to 30 m. Also, recall that the proposed concept has been designed to carry two lanes of traffic loaded by the CL-625-ONT design truck that is the standard design vehicle for highway bridge design in Ontario.

A comparison of superstructure weights in Figure 7 shows that the proposed concept is about half as heavy as the CPCI girder system, suggesting that the latter is carrying a substantial amount of weight in its structural elements that does not enhance the structural capacity of the system. This weight disparity is also notable because it suggests that proposed concept will not require as large or as expensive foundations when compared to the CPCI girder system. A similar argument can made with regard to transportation of all constituent elements and/or materials to site. Finally, the significantly lesser weight of the proposed concept means that it will experience lesser inertial forces in a seismic event, which could result in savings throughout the superstructure and foundations.

Precasting concrete elements reduces the duration of on-site construction. Since both systems incorporate prefabricated elements, which require very little time for on-site erection, the volume of cast-in-place concrete is a useful metric in estimating which system will likely require a longer duration of on-site construction. Figure 8 illustrates such a comparison, where it is apparent that the CPCI girder system requires much more cast-in-place concrete than does the proposed concept.

The implication in Figure 8 is that the proposed concept will not require as much on-site labour or time to construct as does a CPCI girder bridge. To be fair, the proposed concept requires post-tensioning operations that the CPCI girder system does not require. Even so, the post-tensioning operations should not take more than a few days, ultimately having little effect on the duration of on-site construction.

The volume of cast-in-place concrete is also a useful metric in evaluating durability because the controlled environment of a precasting facility affords the appropriate conditions for optimal curing. Conversely, cast-in-place concrete offers less quality control, as the curing of the concrete is largely at the mercy of on-site environmental conditions. For these reasons, precast concrete is often assumed to be more durable than cast-in-place concrete. In view of that, the proposed concept is likely more durable than the CPCI girder system, given its lower volume of cast-in-place concrete.

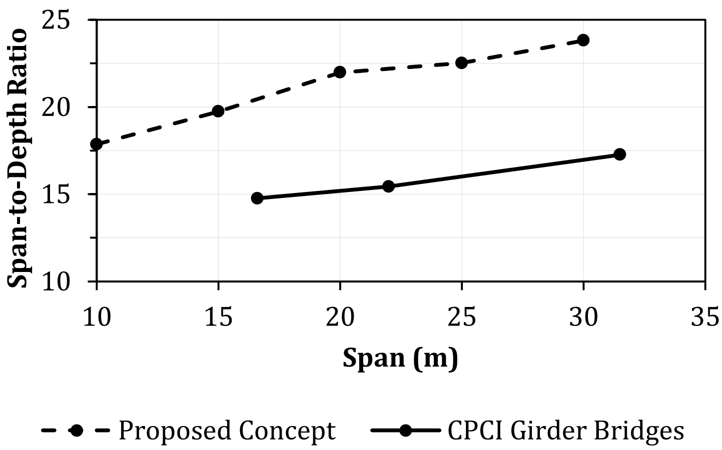

Figure 9 Comparative slenderness

Slender bridges tend to be lighter, more aesthetically pleasing, and more efficient in bending than less slender bridges [18]. Slenderness is commonly measured by the span-to-depth ratio, where the depth excludes all elements that do not explicitly contribute to longitudinal system strength (i.e. barrier walls). Figure 9 shows that the proposed concept is much more slender than the CPCI girder system. This observation is not surprising given the superstructure weight disparity shown in Figure 7. The implication is that the CPCI girder bridges are less efficient in bending than the proposed concept, which could also signify inferior economic performance.

Another implication is that the approaches for the CPCI girder system need to be higher than those for the proposed concept. Thus, the all concrete CPCI girder system requires additional fill material for the approaches and an increase in the height of any structures that retain approach fill.

5. Conclusions

The proposed concept is a feasible concept that can be included in the sub-set of spanning solutions for Canadian bridge engineers. It uses glued-laminated timber to capitalize on the sizeable forestry industry in Ontario that is underutilized in bridge construction. The following remarks can be made about its structural performance relative to the commonly constructed CPCI girder superstructure:

- The proposed concept is significantly lighter, likely affording it advantages with regard to foundation design, seismic design, and transportation of constituent elements to site;
- The proposed concept utilizes far less cast-in-place concrete, suggesting that it should be more durable and likely capable of being constructed more quickly;
- The proposed concept is much more slender, implying greater structural efficiency and requiring shallower approach fill and associated retaining structures.

6. Acknowledgements

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