Design of timber-concrete composite (TCC) bridges with under-deck stay cables

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Abstract

Timber-concrete composite (TCC) bridges represent an attractive structural system due to the synergistic use of its wood and reinforced-concrete constituent components. However, their relatively large flexibility limits their application for larger spans. This paper presents an alternative solution for TCC bridges involving the implementation of post-tensioned under-deck tendons. Based on a series of design and numerical studies, the advantages of the newly proposed system for 30-m, 60-m and 90-m spans are evaluated. This paper shows that the incorporation of under-deck post-tensioning changes the critical limit states governing the design of TCC bridges, and allows for a significant increase in their slendernesses at medium and long spans. Timber’s shear-deformation contribution to the vertical deflection of TCC bridges is significant and should be accounted for, especially when the span/depth ratio \(l/h\) is less than 20. However, this additional deformation can be neglected when stay cables are implemented, especially for bridges with medium and long spans. In order to achieve a more efficient structure, it is proposed that shear connection with an efficiency coefficient, \(\gamma\), greater than 0.8 be used. Finally, the best practical eccentricity of the under-deck tendons and the best location of the deviators are determined on the basis of parametric analyses.

Keywords: Timber-concrete composite, bridges, under-deck, cable-stayed, shear deformations

1. Introduction

A timber-concrete composite (TCC) bridge deck combines timber beams with a concrete slab by means of different connection types. Such structure exploits the best properties of both materials since bending and tensile forces are resisted mainly by the timber and compression is carried by the concrete. The application of TCC systems in bridge construction was first studied by Seiler and Keeney [1], as a cheaper and more sustainable solution than reinforced concrete for the erection of bridges with longer service lives than pure timber bridges. With the development of Engineered Wood Products (EWP) and improvements in construction technology, the total number of TCC bridges has now increased. The Vihantasalmi Bridge built in 1999 in Finland, a five-span king-post truss bridge with two end spans of 21 m and three main spans of 42 m, is a prime example of a TCC bridge.

Three main additional phenomena, specific to TCC bridges, should be considered in their design: (1) the partial composite action between beam and slab due to the flexibility of the connection, (2) the time-dependent properties of the materials [2], and (3) the shear deformation of timber [3]. Given the first phenomenon, the composite cross section does not remain plane due to the slip between the concrete slab and the timber beam. Two approaches are usually followed to deal with this issue in design: the linear-elastic method [4, 5, 6] and the elastoplastic method [7]. The linear-elastic method, also known as the gamma method, considers an effective bending stiffness \(EI_{ef}\) to account for the semi-rigidity of the connection between timber and concrete, assuming that all the materials (concrete, timber and connection) behave within the linear-elastic range before failure of the composite beam. This method is widely employed in practice. On the other hand, the elastoplastic method is particularly appropriate for situations where the composite beam fails after yielding of the connectors. Connectors with low strength, low stiffness and large ductility

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[2, 8, 9, 10] are particularly suited for an elastic-plastic design. However, the gamma method is widely used in the design of TCC structures.

Different approaches have been proposed to account for shrinkage of the material, as well as creep, thermal and hygroscopic related strains. Shrinkage of concrete leads to internal longitudinal shear forces caused by the restraints of the composite action. These internal shear forces, which result in tensile stresses at the bottom fibre of the concrete slab and compression stresses at the top timber fibre, are directly proportional to the stiffness of the connection. To deal with this issue, Yeoh et al. [11] recommended the use of low-shrinkage concrete and prefabrication technologies. On the other hand, Fragiaco and Ceccoti [12] suggested closed-form expressions to analyse the influence of environmental strains and concrete drying shrinkage on TCC systems. Some formulae have been proposed by Shänzlin [13] to simplify the design procedure, transforming environmental strains and concrete drying shrinkage into equivalent uniformly distributed loads. Besides, Ceccoti [6], Shänzlin [13] and Schänzlin and Fragiaco [14] suggested the utilization of an effective modulus to consider the creep effects of different materials. In this approach, effective moduli are introduced by dividing the elastic moduli of the materials by one plus their corresponding effective creep coefficients at a given time. In Eurocode 5 [15], a coefficient named $k_{def}$ is introduced to consider the additional long-term deflections.

Similarly, timber shear deformation is expected to have a significant effect on beam deflections. This is particularly important in timber beams since, unlike other construction materials, timber’s shear modulus is only $\frac{1}{16}$ of its corresponding Elastic modulus. Accordingly, the contribution of timber shear deformation to the total deflection is not negligible (i.e. larger than 15% for sections with depth to span ratios of more than $h/l = 1/10$). However, there is a dearth of information on the effect of timber’s shear deformation on TCC bridges.

Slab bridges and girder bridges with composite decks are the two main types of TCC bridges. These bridge types are typically employed for spans of 5-15 m and 10-30 m, respectively [16]. However, the TCC system seems to be less competitive at longer spans because of its relatively large flexibility. A potentially more efficient solution comes through the implementation of external post-tensioning tendons with high eccentricity. The benefits of this solution, in terms of strength, capacity and deflection control, have been revealed in concrete and steel-concrete composite bridges [17, 18]. Similarly, the advantages of longitudinally post-tensioned timber beams have been highlighted through experimental and numerical investigations [19]. An attractive type of longitudinal post-tensioning, known as under-deck cable-stayed system, was introduced into bridge engineering in 1978 by Leonhardt in the Weitingen viaduct over the Neckar River in Germany [20]. In the last two decades, significant research has been conducted on the application of post-tensioned under-deck tendons to concrete and steel-concrete composite bridges [21, 22, 23, 24, 25]. However, little work has been done so far to promote the development of under-deck cable-stayed tendon systems in TCC bridges [19, 26].

This paper examines various aspects of the structural design of under-deck post-tensioned TCC bridges and evaluates their potential benefits. Focus is placed on the identification of the governing limit states (and their associated failure modes) for different span lengths. Only simply-supported bridges are considered at this stage. Three spans, namely 30 m, 60 m and 90 m, are chosen representing short-, medium- and long-span for TCC bridges, respectively. These spans are consistent with the most efficient span range identified in previous research [21, 24]. It is shown that the post-tensioned under-deck tendons can significantly increase the slenderness of TCC bridges, and change the critical limit states governing their design. In addition, it is demonstrated that the design criteria of these bridges differs from those of under-deck post-tensioned bridges with concrete and composite decks. The influence of concrete shrinkage, timber shear deformation and semi-rigid composite action are studied. Furthermore, sensitivity analyses are conducted on key design parameters of post-tensioned under-deck tendons and the best practical values of stay eccentricity and location of deviation struts are identified.
2. Design of TCC bridges with and without post-tensioned under-deck tendons

2.1. Bridge geometries and cross section

Figure 1 shows views of TCC bridges without tendons, where \( l \) represents the span length, \( b \) the section width and \( h \) the section depth. The simply-supported TCC bridges considered consist of one concrete slab and two timber beams. Similarly, Figure 2 shows views of post-tensioned TCC bridges incorporating post-tensioned tendons. Sections A and B correspond to the locations where the deviation forces of the tendons are introduced into the deck through the struts (as shown in Figure 2). Hence the struts divide the total span into three sub-spans when the permanent loads are applied. The post-tensioned under-deck tendons are anchored at the concrete cross beams (diaphragms) located at the support sections over the abutments through conventional anchoring systems for tendons. The tendons are deviated at the struts through saddles [27]. These connection types have been used in built under-deck cable-stayed bridges. There is no direct connection between the tendons and the timber beams.

![Diagram of TCC bridges without tendons](image)

(a) Elevation view

![Diagram of cross-sectional view at mid-span section](image)

(b) Cross-sectional view at mid-span section

**Fig. 1.** TCC bridges without tendons

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2.2. Material properties and loads

C35 concrete [28] was assumed for the concrete slab while GL28h glulam [15, 29] timber beams were adopted. Permanent loads as well as shrinkage and traffic loads (EN 1990:2002 [30]) were taken into consideration. Permanent loads include the self-weight of the timber beams, concrete slab, pavement and barriers. Based on the EN 1992-1-1:2004 provisions [28], the total shrinkage strains consist of the free-strains plus those strains required to guarantee compatibility at section and structural level. The shrinkage free strains can be calculated by assuming a given relative humidity, while plane cross-section was assumed for the calculation of final strains for simplicity. Traffic loads included two lane double-axle concentrated loads and uniformly distributed loads as specified in EN 1991-2:2003 [31]. The pedestrian load was represented by two uniformly distributed loads of 5 kN/m² applied on the two sidewalks in accordance with EN 1991-2:2003 [31].

2.3. Timber-concrete connection system

The connection between the concrete slab and timber beams is a crucial component of a TCC bridge since it transfers longitudinal shear forces between the different deck components. A suitable connection system can lead to high composite action, which can significantly reduce the depth of the beam and increase the composite sectional bending strength.
2.3.1. Overview of timber-concrete connection types

A wide range of connection systems have been put forward by researchers and engineers in the last century to ensure a reliable joint between the concrete slab and timber beams. The first engineered shear connectors for composite timber beams can be traced back to the 1930s [32, 33] with further developments made in the 1970s [34, 35]. A steel screw or dowel with notches cut into the timber beam is usually considered as the best connection for timber-concrete composite structures due to its favourable shear strength and stiffness [36, 37, 38]. The length of the notch, the depth of penetration into the timber and the presence of lag screws are the most significant factors in the design of this connection, with the former two affecting the stiffness and strength of the connection and the latter determining its ductility and post-peak behaviour [39, 40]. Compared with notched connections, mechanical connectors like nailplates do not require timber cutting [41], while the slip modulus of these connections depends mainly on the flexibility of the fastener and the nearby timber [42, 43]. Some studies have also been conducted on connections that rely on glue and epoxy resin [44, 45, 46, 47] or inclined screws [48].

Glued and long notch doweled connections have high stiffness and strength but low ductility, while dowel type connections have high ductility [2, 49], but low capacity. In this study, dowel type connections and glued-in steel plate connections are adopted as representative of the most rigid and most flexible connector responses in order to reflect the lower and upper bounds of composite action and their effects on the design of TCC bridges. A schematic view of both connection types is presented in Figure 3.

2.3.2. Degree of composite action

The gamma method, which was developed by Mohler [50] and implemented in EN 1995-1 [15], is employed herein to quantify the degree of composite action between the timber beams and concrete slab. In this method, all the materials (concrete, timber and steel) are assumed to behave within the linear-elastic range before failure of the composite beam. The method relies on an effective bending stiffness, \( EI_{ef} \), to account for the effects of the flexibility of the timber-to-concrete connection. This effective bending stiffness can be calculated as:

\[
EI_{ef} = E_t I_0 + \gamma E_t (I_{id} - I_0)
\]  

(1)

where \( E_t \) is timber elastic modulus; \( I_0 \) and \( I_{id} \) refer to the inertia of the equivalent pure-timber fully unconnected and fully-connected sections, respectively; and \( \gamma \) is the shear connection efficiency coefficient that quantifies the level of
composite action, and ranges from 0 for no composite action to 1 for full composite action. $I_0$ and $\gamma$ can be calculated from Eq. 2 and Eq. 3:

$$I_0 = I_t + nI_c$$

$$\gamma = \left[1 + \frac{\pi^2 E_t (I_{id} - I_0)}{d_G^2 G} \cdot \frac{s}{KL^2}\right]^{-1}$$

where $I_t$ and $I_c$ are the inertia of the timber section and the concrete section, with respect to each of the centroids of the timber and the concrete sections considered independently, respectively; $n$ is the ratio of the concrete elastic modulus to the timber elastic modulus; $s$ the connector longitudinal spacing; $d_G$ the distance between the centroids of the timber and concrete sections; $l$ the bridge span; and $K$ the slip modulus of the connector.

### 2.4. Importance of shear deformations

In order to investigate the relative importance of considering timber shear deformation in the estimation of overall deflections of TCC beams, a series of three-dimensional finite-element models were constructed in Abaqus [51]. Simply-supported TCC bridges, with spans ($h_t$) of 30 m, 60 m, and 90 m, and different depths of timber beams ranging from 0.3 m to 5.5 m, were examined. However, as will be explained later in this paper, practical beam configurations are usually associated with beam depths of 2 m or less. The width of the concrete slab ($b$), width of the timber beam ($w_t$) and the thicknesses of the concrete slab over the beams ($h_{c1}$) and at the centre of the cross section ($h_{c2}$) were assumed to be 14 m, 2 m, 0.5 m and 0.3 m, respectively. Steel studs with a diameter of 16 mm and spaced at 200 mm intervals, both in the longitudinal and transversal directions, were employed to connect the concrete slab and timber beams. 4-node linear shell elements with variable thickness and 3D beam elements were employed to model the concrete slab and timber beams, respectively. Moreover, Eulerian elements were used for the concrete slab. On the other hand, the timber beams were represented by means of Eulerian elements in one case and Timoshenko beams in the other with the aim of evaluating the effects of a formal consideration of shear deformations versus a pure flexural formulation. Linear (anisotropic) material constitutive models with effective properties were employed for the timber beams. Elastic spring elements in longitudinal, vertical, transversal and torsional directions were used to simulate the connectors. The stiffnesses of these springs were calculated from Eqs 4 to 6 based on the slip modulus of a single steel stud [15] and the arrangement of the connection.

$$K_p = 2n_{en} \cdot \rho^{1.5} \frac{d}{23}$$

Fig. 3. Schematic views of the dowel-type (left) and glued-in steel plate (right) connections
with $K_p$ being the spring stiffness in the longitudinal and transversal directions of the bridges, expressed in [N/mm]; while $K_z$ (in [N/mm]) and $K_r$ (in [N·mm/rad]) are the vertical and torsional stiffnesses, respectively; $n_{cn}$ the number of connectors on each beam in every transverse section; $\rho$ the density of studs expressed in [kg/m$^3$]; $d$ stud diameter in [mm]; $E_{cn}$ the connector elastic modulus; $h_{cn}$ the stud height in [mm]; and $e_i$ the eccentricity (from the connection surface) of each connector to the centroid of corresponding timber beam. A concentrated load at mid-span and a uniform load along the whole span were applied on all the models to investigate their responses.

A mesh size of 0.1 m was employed following mesh sensitivity studies. The additional deflection associated with shear deformations in the timber beams was quantified by comparing the mid-span deflections estimated from a fully Eulerian TCC beam representation with those coming from the models incorporating a Timoshenko timber beam idealisation. Focus was placed on the increase of the mid-span vertical deflections since timber shear deformation is expected to affect the vertical displacement to a greater extent. This difference is illustrated in Figure 4a as a function of $(h/l)^2$. As expected, the increment of deflection is proportional to $(h/l)^2$, and it is larger under concentrated loads than for distributed loads of equal overall magnitude.

Figure 4b shows the relationship between the deflection increment and deck slenderness $(l/h)$. It can be observed from Figure 4b that, for TCC bridges with average material properties (C35 concrete and GL28h glulam), timber shear deformation significantly increase the mid-span vertical deflection when the value of $l/h$ is less than 20. These measurements range from around 8% to 30%. Alternatively, a deflection increment of no more than 5% is observed if the value of $l/h$ is larger than 30, while the effect of timber’s shear deformation is negligible when $l/h$ reaches 70. As a result, the relative contribution of shear deformations to the overall bridge deflections needs to be accounted for during design and assessment of TCC bridges with $l/h < 20$.

2.5. Under-deck cable-stayed tendon system

The under-deck cable-stayed tendon system is highly efficient leading to slender bridge decks with up to a two thirds reduction of material quantities in comparison with conventional bridges without stay cables, producing more sustainable designs [21]. The reduction of the amount of materials may not have a direct implication in terms of overall cost, as there are also construction aspects that have to be considered. Past work has largely concentrated on concrete or concrete-steel composite bridges. In recent years, longitudinal post-tensioning technology has started to be used in short-to-medium timber bridges, and a comparison between new and traditional solutions has been made by Palermo et al. [19] based on the valuation of long-term post-tensioning losses, cost and sustainability considerations. Nonetheless, more work is required in order to fully understand the driving design considerations and ranges of applicability of post-tensioned timber bridges. This section describes a detailed study into the application of under-deck cable-stayed tendon systems on TCC bridges.

2.5.1. Key design parameters of tendon system

The number of strands per tendon, number of tendons, initial pre-stress, number of struts and eccentricity as well as the side to medium sub-span ratio are five key parameters for the design of post-tensioned under-deck tendons. In this study, only two struts have been considered, as an ideal solution for this bridge type. Steel tendons, made of 7 pre-stressed strands consisting of 7 wires with an area of 140 mm$^2$ each were considered in this study. A stay eccentricity at mid-span of $l/10$ was recommended by Ruiz-Teran and Aparicio [52] for post-tensioned concrete slabs. Similar results were obtained by Madrazo-Aguirre et al. [24] for concrete-steel composite bridges. On this basis, an initial eccentricity of $l/10$ was adopted herein for design. As discussed above, timber shear deformations have a remarkable
Fig. 4. Mid-span deflection increment due to shear deformation in TCC bridges without under-deck cable stay systems. Deflection increment due to shear deformation in the timber beams versus $(h/l)^2$

Deflection increment due to shear deformation in the timber beams versus deck slenderness

Influence in the structural performance of TCC bridges, while the conventional concrete, steel or composite bridges (where the struts tend to be located at around the thirds of the span) are governed by other design considerations. Therefore, the position of the two struts in TCC bridges with under-deck cable system need to be closer to the support sections in order to get smaller shear forces in the deck at cross sections close to the supports, owing to the larger contribution provided by the vertical component of the axial force in the stay cables. Side to middle sub-span ratios of 0.8 were recommended by Ruiz-Teran and Aparicio [21] for concrete slabs, and a value of 1 was suggested by Madrazo-Aguirre et al. [24] for steel-concrete composite decks. A value of 0.5 was initially adopted for the designs of TCC bridges in this paper due to reasons clearly explained later.

The initial pre-stress of the tendons depends on the number of strands and the value of initial deviation force required from under-deck cable system. In this design, the vertical deflection at section A (or B in Figure 2) resulting from
post-tensioning \( (f_p) \) is assumed to be such that it overcomes the initial permanent-load induced deformations \( (f_{\text{dead}}) \). Therefore, the relationship \( f_{\text{dead}} + f_p = 0 \) is employed for the calculation of the value of the initial deviation force.

The calculation of the number of pre-stress strands is based on the verification of the ULS of fatigue at the stay anchorages and the following design criteria was adopted: (i) if a conventional external-prestressing anchoring system (EAS) is used, the maximum tensile stress under characteristic load combination in the tendons should be less than or equal to 65\% of their corresponding strength, while stress variation in the tendons caused by the frequent live load should not exceed 80 MPa; or (ii) if a stay-cable anchoring system (SAS) is used, the maximum tensile stress in the tendons due to characteristic load combination should be less than or equal to 45\% of their corresponding strength, while stress variation in the tendons caused by the frequent live load is kept below 200 MPa. These values of 80 and 200 MPa depend on the fatigue capacity of the anchorages used and are representative of the current technology, which can be provided by most suppliers [53, 54].

2.5.2. Design approaches

Figures 5 and 6 show the procedures followed for the design of TCC bridges without and with under-deck tendons, respectively. All designs were performed according to European specifications [5, 15, 30, 31, 28]. Concrete slabs of 14 m wide were employed in this study. The slabs incorporate two 2-m sidewalks, two 1.5-m shoulders, and two 3.5 m lanes. The serviceability limit state of crack width was checked after assuming an initial concrete slab thickness. This was verified at a later stage and the thickness revised when needed. Finally, a concrete slab thickness of 0.5 m at the beams reducing to 0.3 m towards the centre of the cross-section was chosen in the design.

All required limit state verifications [5, 30] were conducted while systematically altering the width \( (w_t) \) and depth \( (h_t) \) of the timber beams. These verifications included the ultimate limit state of normal stresses in concrete, the ultimate limit state of shear and normal stresses in timber, the serviceability limit state of shear force on connectors, the serviceability limit state of deflections as well as the provision of tension and crack control steel reinforcement. These verifications included the ultimate limit state of normal stresses in concrete, the ultimate limit state of shear and normal stresses in timber with due consideration of the grain direction, the serviceability limit state of shear force on connectors, the serviceability limit state of deflections as well as the provision of tension and crack control steel reinforcement. Similarly, all losses of post-tension, including immediate losses (losses due to friction in the saddles at the struts, elastic deformation of deck and wedge draw-in of the anchorage devices) and time-dependent losses (losses due to the creep of timber and concrete, concrete shrinkage and steel relaxation), were considered. The support, A/B sections, and mid-span sections were chosen as critical sections since they are associated with the largest concrete compression, timber shear or timber combined normal stresses. A total of 270 different designs of TCC bridges with and without post-tensioning were examined. On this basis, the best practical designs of 30-m, 60-m and 90-m span TCC bridges with and without tendons were identified as discussed in the following section.

2.6. Design results and discussion

2.6.1. Best practical design parameters

A number of best practical TCC bridge configurations can be identified based on the extensive database of designs generated with the procedure outlined above. In this paper, the best practical design is defined as the most slender configuration that satisfies all structural design checks as well as external-prestressing anchoring system (EAS) or stay-cable anchoring system (SAS) requirements. Those best practical configurations are summarised in Table 1. In this table, \( n_p \) is the number of strands, and \( \beta \) is a parameter that measures the efficiency of the under-deck tendon system, it represents the ratio of the external moment resisted by the tendons (axial force times eccentricity) to the total sectional moment without post-tensioning. Also, the level of composite action, \( \gamma \) factors, adopted follow those observed in the technical literature [55, 56]. The subscripts ‘e’ and ‘s’ refer to external-prestressing anchoring system (EAS) and stay-cable anchoring system (SAS), respectively.
Fig. 5. Design procedure of TCC bridges without tendons
Set span arrangement and bridge materials
Assume thickness of concrete slab and diameter of transverse and longitudinal reinforced steel bar used in concrete slab
Determine loads and calculate SLS combinations of design actions for transverse check
Transversal crack check for concrete slab Fail
Assume connector spacing
Assume the width and depth of timber beams
Calculate the shear connection reduction factor γ and effective section properties
Assume key parameters of post-tensioned tendons (number of strands, eccentricity and the location of the struts)
Calculate the initial deviation force using the criteria that the deformation caused by post-tensioning can cover that induced by dead loadings, and determine the number of strands based on criteria under two conditions
Calculate ULS and SLS combinations of design actions for longitudinal checks
Check the ultimate limit states of the structure Fail
Check concrete compression stresses, timber combined normal stresses and Timber shear stresses
Pass
Check the serviceability limit states of the structure Fail
Longitudinal crack check and determine the spacing of reinforced steel bar Fail
Check connector shear force Pass
Check deflection Fail
Change the number of strands
Check the maximum stresses of tendons and the stress variation of tendons induced by frequent live loads Fail
Pass
Pass
Pass
Fail
Fig. 6. Design procedure of post-tensioned TCC bridges
As shown in Table 1, the stiffer (glued-in steel plate) connector almost doubles the connection spacing of its dowel counterpart. On the other hand, the application of the post-tensioning results in an increase of the connection spacing by approximately 2.5 times. For TCC bridges without tendons, the most appropriate ratio between the section depth and the span, \( h/l \), varies from 1/16.7 to 1/17.6. The latter value can be considered as the practical limit for simply-supported TCC bridges since \( h/l \) remains constant when the span \( l \) is increased from 60 m to 90 m. By contrast, when compared with other bridges such as steel and steel-concrete composite bridges that can achieve \( h/l \) ratios of 1/20 to 1/30 [57], the section depth of TCC bridges without tendons appears to be less competitive. However, when under-deck tendons are incorporated, the bridge depth span ratio, \( h/l \), decreases significantly almost by half, especially for bridges with 60 and 90 m spans. This is mainly because the post-tensioned tendons and the deck resist a significant part of the bending moment through axial response (tension of the tendons and compression of the deck), thus largely reducing the value of associated flexural stresses on the slab-beam system. The increase of the slenderness can also contribute to less impact of timber shear deformation, nearly negligible for 60 m and 90 m span bridges (Figure 4b). Both \( n_p \) and \( \beta \) increase with the span. A SAS needs more strands since the stress level of a single SAS strand is lower than in an EAS (as the fatigue verification is governed by the maximum stress rather than by the stress variation), but it has higher post-tensioning efficiency (as a consequence of the larger stay area).

2.6.2. Critical limit states

Figure 7 summarises the stress to strength ratios of TCC bridges without tendons when dowel type connections are employed. The numbers on the bars represent how much the parameter (i.e. concrete compression, timber shear or timber normal stress, deflection, etc.) would vary if a glued-in steel plate connection is used rather than a dowel connection. It should be noted that significant levels of tensile forces were sometimes induced in the reinforced-concrete slab for which appropriate quantities of reinforcement will be required. \( R_{\text{design}}/R_{\text{allowance}} \) in Figure 7 represents the structural demand under combined design loading conditions over the corresponding capacity allowance. It is observed from the chart that, for the short span bridge (30 m) with dowel connection, the critical items are timber shear stresses at the support section and timber combined normal stresses at mid-span. For the medium span bridge (60 m) with dowel connections, the concrete compression stresses at mid-span and the timber normal stresses at mid-span are the critical design consideration. Similarly, for the long span bridge (90 m) with dowel connections, the concrete compression stresses at mid-span and the timber normal stresses at mid-span are the critical design consideration. As expected, shear failure becomes more significant as the span reduces, while bending failure becomes dominant for longer spans. Importantly, when changing the connection system into a stiffer one (i.e. using a glue connection rather than a dowel one), the value of \( R_{\text{design}}/R_{\text{allowance}} \) for the connector increases by around 13%, reflecting the larger shear forces transferred by the connectors at the interface. The glued-in steel plate connections also increase the spacing of reinforced steel bars in the concrete slab, thus reducing the required steel quantities, almost 15% for 30-m-span bridges.
<table>
<thead>
<tr>
<th>Span l (m)</th>
<th>Tendons</th>
<th>Shear connection type</th>
<th>Timber beam depth $h_t$ (m)</th>
<th>Section depth $h$ (m)</th>
<th>Depth span ratio $h/l$</th>
<th>Composite action level $y$</th>
<th>Number of strands $n_{pe}$</th>
<th>Stay efficiency $\beta_e$</th>
<th>Number of strands $n_{ps}$</th>
<th>Stay efficiency $\beta_s$</th>
<th>Maximum connection spacing (mm)</th>
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<tbody>
<tr>
<td>30</td>
<td>no</td>
<td>Dowel</td>
<td>1.3</td>
<td>1.8</td>
<td>1/16.7</td>
<td>0.87</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>115</td>
</tr>
<tr>
<td>30</td>
<td>no</td>
<td>Glued-in Plate</td>
<td>1.3</td>
<td>1.8</td>
<td>1/16.7</td>
<td>0.95</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>225</td>
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<tr>
<td>30</td>
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<td>Dowel</td>
<td>0.7</td>
<td>1.2</td>
<td>1/25.0</td>
<td>0.92</td>
<td>49</td>
<td>0.79</td>
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<td>0.82</td>
<td>275</td>
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<tr>
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<td>1.2</td>
<td>1/25.0</td>
<td>0.97</td>
<td>49</td>
<td>0.79</td>
<td>77</td>
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<td>60</td>
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<td>Dowel</td>
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<td>3.4</td>
<td>1/17.6</td>
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<td>-</td>
<td>-</td>
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<td>3.4</td>
<td>1/17.6</td>
<td>0.98</td>
<td>-</td>
<td>-</td>
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<tr>
<td>60</td>
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<td>Dowel</td>
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<td>1.8</td>
<td>1/33.3</td>
<td>0.96</td>
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<td>154</td>
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<td>1.8</td>
<td>1/33.3</td>
<td>0.99</td>
<td>105</td>
<td>0.88</td>
<td>154</td>
<td>0.91</td>
<td>435</td>
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<tr>
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<td>Dowel</td>
<td>4.6</td>
<td>5.1</td>
<td>1/17.6</td>
<td>0.96</td>
<td>-</td>
<td>-</td>
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<tr>
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<td>Glued-in Plate</td>
<td>4.6</td>
<td>5.1</td>
<td>1/17.6</td>
<td>0.99</td>
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<td>-</td>
<td>-</td>
<td>-</td>
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<tr>
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<tr>
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<td>2.4</td>
<td>1/37.5</td>
<td>0.99</td>
<td>161</td>
<td>0.91</td>
<td>238</td>
<td>0.93</td>
<td>450</td>
</tr>
</tbody>
</table>

* All timber beams have width of 2.0 m.
Figure 8 reports the critical parameters in the design of under-deck post-tensioned TCC bridges with dowel and glued-in steel plate connections. The values for glued connection are reported in a similar manner than in Figure 7 at the top of the bars. 'Deflection EAS' and 'Deflection SAS' are the maximum deflections due to characteristic load combinations in the EAS and SAS tendon conditions, respectively. It is found that, for the short span bridge (30 m) with dowel connection, timber shear stresses at the section A/B and deflection in the EAS are the governing design considerations. For the medium span bridge (60 m) with dowel connection, the critical items are timber shear stresses at the section A/B and deflection for the EAS. For the long span bridge (90 m) with dowel connection, the critical design factors are timber shear stresses at the section A/B, concrete compression stresses at the mid-span and deflection for the EAS, with the latter two being more critical. An important difference between TCC bridges with and without post-tensioning is that the critical section for shear in the post-tensioned structures is now at section A/B rather than at the support. This is due to the fact that the pre-stressing layout reduces the design shear forces in the first and third sub-spans, but has no effect on the middle sub-span. Consequently, the shear force at the section where the deck is supported by the struts and the deflections at mid-span are the two main aspects governing the design of short and medium span post-tensioned bridges. Conversely, bending failure becomes more important when the span increases to 90 m. The use of glued-in steel plate connection has little effect on the results since both post-tensioned TCC bridges with dowel type connection and glued connection achieve remarkably high levels of composite action.

Figure 9 depicts the distribution of sectional stresses at mid-span in the best practical TCC bridges. In this figure, solid lines represent total design stresses including those induced by shrinkage; dash-dotted lines refer to sectional stresses assuming a fully rigid composite action but without considering those stresses caused by shrinkage; and dashed lines depict section stresses without accounting for those induced by shrinkage under real composite action. For the short-span (30 m) bridge, tensile normal stresses are developed at the concrete-timber interface, and shrinkage increases the stress difference between the concrete and timber fibres. However, for longer span bridges, both concrete and timber normal stresses at the interface are compressive, thus making the effect of the concrete shrinkage very limited. On the other hand, at the support section, all normal stresses were found to be caused by shrinkage. It follows that, concrete shrinkage only needs to be considered in short-span (30 m) TCC bridges and at the support sections in medium- (60 m) and long-span (90-m) TCC bridges. In addition, the stress difference due to semi-rigid composite action in 30-m-span bridges is more significant than in 60-m and 90-m-span bridges, as will be seen in the following section.

### 2.6.3. Influence of semi-rigid connection on TCC bridges

The relationship between \( \gamma \) and \( l/h \) is shown in Figure 10. The \( \gamma \) factor, which represents the level of composite action between timber beams and concrete slab, increases with the span and the slenderness \( (l/h) \). Importantly, as expressed by Equation 3, lower levels of composite action are achieved for shorter bridges (i.e. 30 m span as opposed to 90 m span) even when the bridge slenderness \( (l/h) \) is kept constant. This is a consequence of the quadratic relationship between \( \gamma \) and span \( (l) \) described by Equation 3. It follows that a more efficient composite action is achieved in slenderer and longer bridges.

Figures 11 and 12 depict the relationship between the maximum tendon stresses (resulting from the permanent and live loads) and the tendon stress variation due to frequent live load as functions of structural slenderness \( (l/h) \). For stay-cable anchoring systems (SAS), the maximum tendon stress dominates the design, while the design is controlled by tendon stress variation when the value of \( l/h \) exceeds 40 in external-prestressing anchoring systems (EAS). In order to achieve a more efficient use of cables, it is preferred that the design of tendons in post-tensioned TCC bridges is dominated by the maximum stress. Therefore it is recommended that the structural slenderness \( (l/h) \) be less than 40 when an external-prestressing anchoring system (EAS) is employed.

As illustrated in Figure 13, the 'maximum stress amplification', calculated as the maximum difference between the stresses in the timber beams and concrete slab with real composite action and the corresponding stresses with ideally rigid composite action, varies significantly with \( \gamma \). It can be seen from Figure 13 that the maximum stress amplification decreases linearly with increasing \( \gamma \) in TCC bridges without tendons, but non-linearly in post-tensioned TCC bridges. Importantly, for 90-m-span post-tensioned TCC bridges, the value of \( \gamma \) should be larger than 0.91 if a maximum stress
Fig. 7. Design/allowance ratios for timber-concrete composite bridges without tendons
Fig. 8. Design/allowance ratios in under-deck post-tensioned TCC bridges
Fig. 9. Typical distribution of normal stresses in best practical TCC bridges
Fig. 10. Relationship between $\gamma$ and $l/h$

Fig. 11. Relationship between maximum tendon stresses and $l/h$
amplification of less than 5% is required, while the corresponding value of $\gamma$ would be 0.99 for 30-m-span TCC bridges.

The relationship between the most appropriate slenderness and $\gamma$ is indicated in Figure 14. It is observed that the structural slendernesses raise almost linearly when $\gamma$ increases from 0.0 to around 0.8, after which higher values of $\gamma$ have no effect on the most appropriate bridge slenderness. This is mainly because when $\gamma$ is less than 0.8, the composite action is partial, making the concrete compression stresses, the timber combined normal stresses and the deflections at mid-span, the critical design verifications for TCC bridges. However, when the value of $\gamma$ exceeds 0.8, the timber shear stresses start to gradually control the design of TCC bridges. Conversely the concrete compression stresses, the timber combined normal stresses and the deflections at mid-span will decrease with larger $\gamma$. Therefore, a $\gamma$ of 0.8 marks the limit at which the TCC bridges change from a connection-controlled systems to a shear-controlled one. In light of the above, it is recommended that TCC bridges be designed for a composite action level compatible with $\gamma \geq 0.8$ since the shear-controlled system favours a more efficient use of the material.

3. Sensitivity analyses

In all previous designs, the two key design parameters of the tendon system (eccentricity $z$ and side to medium sub-span ratio $l_p/(l - 2l_p)$) were assumed to be fixed to simplify the design and discussion. To determine the best practical values of these two parameters, additional sensitivity analyses were carried out and are presented in this section.

3.1. Stay eccentricity

The stay eccentricity ($z$ in Figure 2) was systematically varied between 0.001$l$ and 0.25$l$, in order to investigate its influence on key structural properties. The same bridge geometries, cross sections, material properties and loading conditions described before were employed for these analyses. Similarly, two tendon anchoring conditions (EAS and SAS) were considered.

To make the TCC bridge an efficient structure, the concrete slab and timber beams should be mainly subjected to compression and tensile stresses, respectively. Figure 15 shows the relationship between the tension rate in timber
Fig. 13. Relationship between stress amplification and $\gamma$, with a zoom-in as put for 5%.

Fig. 14. Relationship between the most appropriate slenderness $l/h$ and connection efficiency factor $\gamma$. 
beams (fraction of the timber beam height in tension) and the stay eccentricity, $z/l$, at mid-span for 30-m, 60-m and 90-m-span post-tensioned TCC bridges. It can be appreciated from this figure that when the eccentricity is very small, in order to provide enough deviation force to meet the $f_p + f_{dead} = 0$ requirement at sections A and B, the axial component is so large that all the section is under compression stresses (i.e. the tension rate in the timber beam is zero).

After the eccentricity, $z/l$, exceeds 0.040, 0.073 and 0.085 for the 30-m, 60-m and 90-m-span post-tensioned TCC bridges, respectively, the tension rate in timber beams reach their corresponding plateaus. Therefore, eccentricities smaller than these values are not recommended due to efficiency considerations.

![Fig. 15. Relationship between the tension rate in the timber beams and $z/l$.](image)

Figures 16 and 17 depict the number of strands and total amount of stay cables required versus $z/l$. The total amount of stay cables is the total weight of tendons used in each case. It is evident from these figures that, as the eccentricity increases, the number of strands and the total amount of stay cables are reduced and their rates of reduction also decrease. The trends for the number of strands and for the total amount of material in the stay cables are almost the same even though the total length increases with larger eccentricities. This is due to the fact that the number of strands (or the area of tendons) rather than the total tendon length dominates the design. However, a more inefficient design will be obtained if the slope of the curve is very close to 0 where a very large increase of the eccentricity is required to achieve a very minor reduction in the number of strands. Figure 18 presents the variation of the ratio of total stay cables weight over the bridge weight ($W_{tendon}/W_{total}$) versus $z/l$. This figure shows that the stay cables weigh less than 2% of the bridge weight when the stay eccentricity $z$ is higher than 0.1 $l$.

Figure 19 presents the deflection at mid-span in the 30-m, 60-m and 90-m span bridges as a function of their eccentricity $z/l$. When the eccentricity is small, in order to provide enough deviation force to meet the $f_p + f_{dead} = 0$ requirement at sections A and B, the number of strands is very large resulting in unexpectedly large bending stiffness and very small deflections. Similarly, after reaching their corresponding peak values (or plateaus), the vertical deflections at mid-span start to decrease with increasing eccentricities. Ideally, the design eccentricity needs to be larger than the value associated with peak deflections.

Figure 20 shows the relationship between stay efficiency, $\beta$, and $z/l$. For 30-m span bridges with EAS or SAS tendon conditions, the stay efficiency increases significantly with stay eccentricity. For 60-m and 90-m span bridges, on the other hand, the stay efficiency comes to a stage where $\beta$ remains almost unaltered regardless of the increase of stay eccentricity for values of $z/l \geq 0.10$. In order to achieve a high level of stay efficiency, values of no less than 0.10
Fig. 16. Relationship between number of strands and $z/l$

Fig. 17. Relationship between total amount of stay cables and $z/l$
Fig. 18. Relationship between the weight of stay cables (\(W_{	ext{tendon}}\)) over the total weight of the bridge (\(W_{	ext{total}}\)) and \(z/l\).

Fig. 19. Relationship between deflection and \(z/l\).
are recommended for stay eccentricity $z/l$. Results also show that the differences of stay efficiency between bridges with EAS and SAS are limited when $z/l$ is larger than 0.10. However, since the total amount of stay cables required (Figure 17) is smaller and the anchorages are significantly cheaper, bridges with EAS are recommended in design. When considering the ratio of the total weight of stay cables over the bridge weight ($W_{tendon}/W_{total}$ in Figure 18), it is found that, when the stay eccentricity $z$ is higher than 0.1, the stay cables take over 80% of the live load while their weight amounts to less than 2% of the total, this shows the high efficiency of the under-deck tendon system.

The relationships between the maximum tendon stress and stress variation due to the frequent live loads with eccentricity $z/l$ are demonstrated in Figure 21 and Figure 22, respectively. It can be appreciated from these figures that, the maximum tendon stress dominates the design over the full range of $z/l$ when stay-cable anchoring system (SAS) is used, but it only controls the design for short span bridges (30 m) when external-prestressing anchoring system (EAS) is employed. For external-prestressing anchoring system (EAS), $z/l = 0.15$ (60-m span) and $z/l = 0.125$ (90-m span) are the eccentricity levels that mark the change from a design governed by maximum tendon stress to stress variation. In order to make full use of the tendons and avoid anchorage fatigue, a maximum-stress-controlled system rather than stress-variation-controlled design is preferred. Therefore, eccentricities $z/l$ of less than 0.15 and 0.125 are preferred for medium-span (60 m) and long-span (90 m) bridges with external-prestressing anchoring system (EAS), respectively.

Figure 23 shows the strut axial force as a function of the eccentricity $(z/l)$. When the value of $z/l$ is less than 0.025, the total amount of stay cables is large enough so that the vertical component of strut axial force (deviation force) has to support not only the dead load of the composite beam but also the unexpected weight of the tendons, thus making the force in the strut unexpectedly large. The strut axial force reaches the smallest values within the range of $0.025 \leq z/l \leq 0.2$, afterwards this force increases non-linearly and the buckling lengths become larger, leading to stockier elements. Consequently, from the point of view of the axial force in the strut, the best practical value of the eccentricity $z/l$ should range between 0.025 and 0.2.

Table 2 summarises the feasible ranges of the stay eccentricity taking account of all the influenced indicators analysed above. The optimal solution in each case is a compromise solution. With the aims of reducing the amount of material and the vertical deflection, and as well as increasing the structural efficiency ($h_{tension}/h_t$ and $\beta$), a higher value should be chosen for the stay eccentricity. While, in order to reduce the axial forces in the struts and increase their stability, smaller values in a range from 0.025 to 0.2 for $z/l$ are more reasonable. Finally, when taking account of both the
Fig. 21. Relationship between maximum stress in strands and $z/l$

Fig. 22. Relationship between stress variation in strands due to the frequent live load and $z/l$
Fig. 23. Relationship between strut axial force and \( z/l \)

Table 2. Summary of the best practical values of the stay eccentricity

<table>
<thead>
<tr>
<th>Indicators</th>
<th>30-m-EAS*</th>
<th>30-m-SAS</th>
<th>60-m-EAS</th>
<th>60-m-SAS</th>
<th>90-m-EAS</th>
<th>90-m-SAS</th>
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<tr>
<td>( h_{\text{tension}}/h_t )</td>
<td>( z/l \geq 0.040 )</td>
<td>( z/l \geq 0.073 )</td>
<td>( z/l \geq 0.085 )</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>30-m-EAS*</td>
<td>30-m-SAS</td>
<td>60-m-EAS</td>
<td>60-m-SAS</td>
<td>90-m-EAS</td>
<td>90-m-SAS</td>
<td></td>
</tr>
<tr>
<td>( f )</td>
<td>( z/l \geq 0.037 )</td>
<td>( z/l \geq 0.025 )</td>
<td>( z/l \geq 0.097 )</td>
<td>( z/l \geq 0.056 )</td>
<td>( z/l \geq 0.068 )</td>
<td>( z/l \geq 0.068 )</td>
</tr>
<tr>
<td>( \beta )</td>
<td>( z/l \geq 0.1 )</td>
<td>( z/l \leq 0.15 )</td>
<td>( z/l \leq 0.125 )</td>
<td>( z/l \leq 0.125 )</td>
<td>( z/l \leq 0.2 )</td>
<td></td>
</tr>
<tr>
<td>( \sigma_p )</td>
<td>( z/l \leq 0.15 )</td>
<td>( z/l \leq 0.125 )</td>
<td>( z/l \leq 0.125 )</td>
<td>( z/l \leq 0.2 )</td>
<td></td>
<td></td>
</tr>
<tr>
<td>( \Delta \sigma_p )</td>
<td>( z/l \leq 0.15 )</td>
<td>( z/l \leq 0.125 )</td>
<td>( z/l \leq 0.125 )</td>
<td>( z/l \leq 0.2 )</td>
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</tr>
<tr>
<td>( F_s )</td>
<td>0.025 ( \leq z/l \leq 0.2 )</td>
<td>0.025 ( \leq z/l \leq 0.2 )</td>
<td>0.025 ( \leq z/l \leq 0.2 )</td>
<td>0.025 ( \leq z/l \leq 0.2 )</td>
<td>0.025 ( \leq z/l \leq 0.2 )</td>
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* 30-m-EAS represents 30-m-span under-deck post-tensioned TCC bridges with EAS tendon condition.

The relationship between some key structural indicators against the side to medium sub-span ratio for 30 m, 60 m and 90 m span bridges is depicted in Figure 24. These indicators include slenderness (maximum \( l/h \)), the number of post-tensioned strands (\( n_p \)), total stay efficiency (\( \beta \)) and stay efficiency per strand (\( \beta/n_p \)). In terms of slenderness (\( l/h \)), the slenderest decks have side to middle sub-span ratios between 0.5 and 1.2. Smaller side to medium span ratios lead to a smaller number of strands with a lower efficiency. On one side, reducing the number of strands is beneficial with positive economic and sustainable consequences. On the other hand, larger efficiencies of the cable-staying system are preferred, as they enhance the axial, versus the flexural, behaviour of the whole system. Therefore, the stay efficiency per strand (\( \beta/n_p \)) is introduced to evaluate the structural performance. This newly-proposed coefficient is found to be more sensitive to small changes of side to medium sub-span ratios when the maximum \( l/h \) remains constant (i.e. side to middle sub-span ratios range from 0.5 to 1.2, 0.8 to 1.2 and 1.0 to 1.2 for 30-m, 60-m and 90-m-span bridges, respectively), showing that best practical side to middle sub-span ratios range from 0.5 to 0.8, with the optimality
leaning to 0.5 for TCC bridges with spans of 30 and 60 m and moving towards 0.8 for 90 m. For small spans when shear forces are critical, smaller ratios of side to middle sub-span increase the shear contribution of the pre-stressing, providing the best practical configuration. For larger spans, when bending is critical, side to middle sub-span ratios of 0.8 provide the best practical configuration (as expected for end-spans in continuous beams). However, when considering the cost of stays, a lower value of side to middle sub-span ratio should be adopted since a smaller number of stays is needed. Therefore, a side to middle sub-span ratio of 0.5 is recommended for under-deck post-tensioned TCC bridges.

It also can be found that, although the total stay efficiency ($\beta$) in longer span bridges is higher than that in shorter span bridges (Figure 24c), the stay efficiency per strand ($\beta/n_p$) in bridges with shorter span has higher value. The newly proposed coefficient, $\beta/n_p$, allows the consideration of efficiency and amount of material at the same time when searching for a compromise solution between increasing efficiency and limiting the amount of material used in the stay cables. This coefficient is proposed to be more suitable to carry out the comparison of post-tensioned TCC bridges alternatives in sensitivity analysis of side to middle sub-span ratio. Furthermore, since $\beta/n_p$ is sensitive to small changes of key design parameters in the under-deck tendon system, it might be an adequate indicator in other studies when associated with this tendon system.

4. Conclusion

Under-deck post-tensioned TCC bridges are efficient structures that combine post-tensioning technology and TCC systems. A total of 270 designs of TCC bridges with and without post-tensioning have been examined in this study. It has been shown that, the application of post-tensioned under-deck tendons significantly changes the critical limit states governing the design of TCC bridges. For small-span post-tensioned under-deck TCC bridges, shear failure is critical, while bending failure becomes dominant for longer spans. The under-deck tendons also significantly increase the slenderness and efficiency of TCC bridges with the depth to span ratio ($h/l$) decreasing almost by half, especially for bridges with spans of 60 and 90 m in comparison with conventional TCC configurations. Concrete shrinkage has limited influence in the design of this newly-proposed bridges, and only needs to be considered in short-span bridges and at the support sections in middle and long span structures. Finally, sensitivity analyses have been conducted to explore the best practical configurations. The following recommendations can be offered from the analyses presented:

1. It was demonstrated that for the bridges under considerations, shear deformations in the timber beams can lead to up to 30% greater deformations when the value of slenderness $l/h$ is less than 20, while the deflection increment could be less than 5% for $l/h$ values larger than 30. Therefore, although the consideration of timber’s shear deformation is of paramount importance for a realistic estimation of vertical deflections in TCC bridges without stay cables, it can be neglected when stay cables are implemented and larger slenderness are achieved.

2. The use of stiffer concrete-to-timber connections leads to larger shear forces being transferred at the slab-beam interface. However, for 90-m-span under-deck post-tensioned TCC bridges, semi-rigid connections will not induce important amplification of section stresses if the composite action coefficient ($\gamma$) is larger than 0.91, while the corresponding value of $\gamma$ required for 30-m-span TCC bridges would be 0.99.

3. A composite action coefficient ($\gamma$) of 0.8 marks the transition between a connection-controlled TCC bridge and a shear-controlled system. The shear-controlled system is recommended in the design of TCC bridges to achieve a more efficient structure.

4. External-prestressing anchoring systems (EAS) are recommended in the design of under-deck post-tensioned TCC bridges. For this anchorage type, the amount of material used in the stays is smaller, the anchorages are significantly cheaper, and the stay efficiency is very similar for the same stay eccentricity, especially when the stay eccentricity is larger than 0.1l. Furthermore, in order to make the structures controlled by maximum tendon stress rather than by the stress variation, a structural slenderness ($l/h$) of less than 40 is proposed.
Fig. 24. Relationship between some key design parameters and side to middle sub-span ratio.
5. With the aims of reducing the amount of material and the vertical deflection, as well as increasing the structural efficiency ($h_{\text{tension}}/h_{\text{l}}$ and $\beta$), a higher value should be chosen for the stay eccentricity. While, in order to reduce the axial forces in the struts and increase their stability, smaller values in a range from 0.025 to 0.2 for $z/l$ are more reasonable. Taking account of all the indicators above as well as the aesthetical appearance of the whole structure, a value of $z/l = 0.1$ is recommended for under-deck post-tensioned TCC bridges.

6. A side to middle sub-span ratios of 0.5 is proposed for under-deck post-tensioned TCC bridges.

7. A new coefficient, the stay efficiency per strand ($\beta/n_p$), is introduced to evaluate the structural performance. This newly-proposed coefficient is more sensitive to small changes in the design parameters and allows the simultaneous consideration of efficiency and amount of material when performing design comparisons.

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References


