EVALUATION OF THE EFFECTIVE SLAB WIDTH FOR COMPOSITE CABLE-STAYED BRIDGE

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Extended Abstract

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ABSTRACT:
The present dissertation aims the evaluation of the effective slab width to be used in cable-stayed bridge longitudinal analysis, with a composite double girder steel-concrete deck, based on 3D finite element model results, and considering the construction stages, the live loads with different patterns and increments up to Ultimate Limit State.

Several finite element models were developed for the effective width definition, in order to account for different subsets of parameters such as the stay to deck anchorage points, the construction stages, load patterns and load levels. The effects of those parameters on the effective slab widths were assessed based on model results. Based on the impact assessment, a criterion for effective slab widths of composite decks studied was proposed.

KEYWORDS: Cable-stayed bridge; Composite deck; Effective slab width; Linear elastic analysis; Three-dimensional finite element model; Cantilever construction method.

1 INTRODUCTION
The continuous increase of the deck spans, together with the aesthetic concerns associated with bridges, has led to cross-sections slender and stronger. The great progress in materials that has been observed in the last decades, as well as the achieved experience itself, has shown that composite steel–concrete decks are a very efficient solution. The combination of the two materials allows taking advantage of their best features, allowing the deck to be light, strong and slender.

The most common composite cable-stayed deck is composed by two main steel main girders, continuously suspended by cables, and a concrete slab supported on a grid formed by this main girders and cross-beams. The main girders, spaced between stay-cables, support the longitudinal and transversal bending moments while the concrete slab resists to the local loads and also mainly supports the high axial compressions introduced on the deck by the stays anchorages.

For the design of these structures, often numerical models foe their analysis. These models are expected to be powerful, but at the same time should provide an easy use in order to achieve results. Beam elements are usually adopted to simulate steel girders as well as the concrete slab, of a composite deck. Therefore, for the slab it is necessary to define its effective width. The criteria included in the present codes are prepared for defining the effective width of composite beams subjected to bending and supported on rigid supports, or separately, submitted to concentrated forces applied on the slab.
Therefore the present codes cannot be directly used to the specific case of composite cable-stayed bridges, since the cables cannot be considered as rigid supports, and furthermore in this particular case, the deck is submitted to both axial compression and bending moments.

Considering the current problems of the cable-stayed bridges with composite cable-stayed decks according to the definition of effective slabs widths, it is intended, in this study, to define a single effective based on 3D finite element model results, and considering the construction stages, the live loads with different patterns and increments up to Ultimate Limit State.

2 MODELLING AND STRUCTURAL ANALYSIS
This chapter presents the modelling of the structural elements as well as their constitutive laws. Some simplifications are also presented, regarding the consideration of nonlinear behaviours and the time depended effects, since this study focuses on a linear elastic analysis of three-dimensional models.

The three-dimensional models used beam finite linear elements for the steel structures of the deck, and of the towers and cables. Finite elements of the slab, plans and four-nodes, based on the theory of thin slabs to represent the concrete slab. The consideration of finite elements already mentioned, provide information about the transverse distributions of stresses in the slab, therefore allowing for the definition of an effective slab width to be used when the slab is modelled as a beam in the longitudinal bridge numerical models.

The consideration of parallel finite elements to model the composite section of the deck, shows to be advantageous because it allows eliminating the disadvantages of the models whose modelling composite deck section is performed using a unique finite element. Furthermore, using different finite elements there is the advantage of carrying out a separated analysis of specific behaviours of each material, as well as allowing for the consideration of the construction stages, when the steel structure is often erected before the concrete slab.

3 MATERIALS MODELLING
The simulation of the structure of a composite cable-stayed bridges depends on the behaviour of the materials used, whereas it is needed a correct knowledge of their mechanical behaviour.

The test model is made of structural steel, concrete and fictitious very high stiffness material, referred as “rigid”. The towers and the piers are made of concrete, the cables are of steel and the section is of composite type. For intermediate piers and deck temporary supports on the towers the rigid material is used. The reinforcements are not simulated in deck and towers concrete cross-sections.

The study considers the construction stages and also the service phase, therefore materials tensions levels are always much lower than the respective material resistances. Thus, the materials exhibit mechanical linear elastic behaviours, characterized by its modulus of elasticity
E. *Table 3.1* summarizes the material properties used in the numerical models. These properties were determined according with the considerations set out in EC2 and EC3 [4; 5].

<table>
<thead>
<tr>
<th>Material discretion</th>
<th>E (GPa)</th>
<th>Fy (MPa)</th>
<th>ν</th>
<th>α (K1)</th>
<th>PP (KN/m³)</th>
<th>Analyses</th>
<th>Elements</th>
</tr>
</thead>
<tbody>
<tr>
<td>S355 NL</td>
<td>210</td>
<td>355</td>
<td>0.3</td>
<td>1.2E 05</td>
<td>77</td>
<td>linear</td>
<td>Girders</td>
</tr>
<tr>
<td>C45/55</td>
<td>44.17</td>
<td>53</td>
<td>0.2</td>
<td>1.0E 05</td>
<td>25</td>
<td>linear</td>
<td>Slab</td>
</tr>
<tr>
<td>C40/50</td>
<td>41.74</td>
<td>48</td>
<td>0.2</td>
<td>1.0E 05</td>
<td>25</td>
<td>linear</td>
<td>Towers</td>
</tr>
<tr>
<td>Cable</td>
<td>195</td>
<td>1770</td>
<td>0.3</td>
<td>1.0E 05</td>
<td>variable</td>
<td>linear</td>
<td>Cables</td>
</tr>
<tr>
<td>Rigid</td>
<td>1000x210</td>
<td>355</td>
<td>0.3</td>
<td>1.2E 05</td>
<td>-</td>
<td>linear</td>
<td>Temporary supports</td>
</tr>
<tr>
<td>Rigid</td>
<td>1000x210</td>
<td>355</td>
<td>0.3</td>
<td>1.2E 05</td>
<td>-</td>
<td>linear</td>
<td>Connection elements</td>
</tr>
</tbody>
</table>

*Table 3.1 – Material properties.*

For the concrete’s case, instead of the reduced properties proposed by the codes for design proposes, average properties of concrete are adopted, so that the behaviour of composite deck is as real as possible.

It is known that time dependent effects produce stress redistribution on the composite decks, and therefore may affect during the time life period of the composite cable-stayed bridge the effective slab widths. However, for the present work, these effects are not considered. Also for simplification, the cables sag effect is not considered directly, being used an equivalent elastic modulus, always constant and equal to the elastic modulus of the cables material [1].

### 4 NUMERICAL MODEL PRESENTATION

In the present work a case study of a composite deck bridge with a main span of 420 meters is used. The side spans of 194.4 meters are supported by two intermediate piers, and therefore forming three span lengths of 60.125+72.1875+72.1875 meters (*Figure 4.1*).

In service, it is considered that the bridge deck is supported at the piers but not on the towers. However, during the construction executed by the balanced cantilever construction scheme, the deck is supported at the towers level. The relevance of the towers support throughout the construction stages and the service phase, for the effective slab width, is evaluated.

*Figure 4.1 – Static scheme of the composite cable-stayed bridge used in the study.*
SAP2000 software is used for the structure numerical modelling. Since the structure is symmetrical in relation to the longitudinal axis, the 3D model simulates half of the structure (Figure 4.2). The steel beams, towers / piers and the cables are modelled using finite bar elements, while the slab is simulated by shell finite elements, plans and four-node, based on the theory of thin slabs. The beam elements were associated with the elements of slab considering a full connection.

The modelling of the stays anchorages is accomplished using auxiliary elements to assure the correct angles of the cables. These elements have high rigidity (Rigid – table 3.1).

The model used 2x32 elements for the stays and 2x26 elements for the towers. The deck used a total of 6020 elements, 5700 concerning the slab, 128 the longitudinal main girder and 192 the cross-beams.

5 CONSTRUCTION STAGES: APPLIED LOADS, INITIAL GEOMETRY AND CABLE FORCES

In the present work considers a simplified construction simulation. With this procedure, used by Pedro [7], the result of all sequences of construction (while the structure is a cantilever), is obtained by an unique "intermediate equilibrium position". From that "intermediate equilibrium position" only 4 more construction stages are needed until it the end of bridge construction. These stages are: 1) the addition of the closing elements, 2) the addition of the intermediate pier supports, 3) the execution of the finishing works and 4) the final re-tensioning of the cables.

The stressing values and corresponding applied forces in each cable, at the "intermediate equilibrium position", and the overall re-tensioning values used in this work, were established by Pedro [7].

The permanent loads applied in the structure correspond to the self-weight and of the finishing works, were also the same as defined by Pedro [7]. The live load applied over the slab is 4 kN/m². Figure 5.1 presents the two patterns of live load considered and the increment loads up to Ultimate Limit Stages (ULS) are the following ones:

<table>
<thead>
<tr>
<th>Increment (CP+SC1) applied along all the deck</th>
<th>Increment (SC2) in the central span</th>
</tr>
</thead>
<tbody>
<tr>
<td>CP+SC1</td>
<td>CP+SC2</td>
</tr>
<tr>
<td>1.5(CP+SC1) = λ₁ (CP+SC1); λ₁=1.5</td>
<td>1.5(CP+SC2) = CP+λ₃SC2; λ₃=3.083</td>
</tr>
</tbody>
</table>
6 EFFECTIVE SLAB WIDTH DEFINITION

By definition the slab effective width consists in the region of the concrete slab that works together with the steel beam in resisting the external forces applied, and in which the stress distribution is said to be uniform and equal to the maximum stress applied along the width of the slab. This width is usually determined in accordance with the regulations and legislation such as the AASHTO (1994), German Deutsche Norm (DIN 1075) (1981).

In Portugal Eurocodes have been used in the last years, and for the composite bridges EC4-2 code should be adopted [6]. However, for the specific case of composite cable-stayed bridges this code can not be directly applied. In fact, the deck slab presents at the same time bending moments and axial compression, which is not a situation predicted in the codes. Recent studies have given more information on the definition of a single width effective. In a study made by Byers [2] cable-stayed bridges, with the bridge deck supported on the towers and also in the retention piers, is presented a proposal for calculating the effective slab width of composite cable-stayed bridges in service. The constructive stages and the time dependent effects were not considered by this author.

The evaluation of the slab effective width in a given section requires the knowledge of the total resultant force \( Q \) that is applied in this section, as well as the maximum stress applied \( (\sigma_{\text{max}}) \).

Once you know the values of \( Q \) and \( \sigma_{\text{max}} \), the calculation of effective width is done through the following expression:

\[
b_{\text{eff}} = \frac{Q}{(\sigma_{\text{max}}.t)}
\]  \( (6.1) \)

The calculated value of \( Q \) is done using an integration of tension along the of the slab width. The use of finite elements transforms the applied stress in the section of the slab in a discrete system. Thus, determining the value of the total force, \( Q \), results from the sum of the resultant forces obtained in each element (eq. (6.2))

\[
Q = \sum_{i=1}^{n} \sigma_i \cdot t_i \cdot b_i
\]  \( (6.2) \)
The resultant of forces acting on each element is determined from numerical integration of the normal stress using the trapezoid rule. Since the element thicknesses are constant, the resultant force on an element corresponds to the average nodes stresses multiplied by the thickness and by the length of the side of the element. Evaluating $\sigma_{med}^{max}$ for each cross-section, then $b_{eff}$ is obtained from the following expressions:

$$Q_i = \sigma_{med}^{max} (b_0 + b_{eff,0} e_2)$$

$$b_{eff,0} = \frac{Q_i}{\sigma_{med}^{max} e_2}$$

$$b_{eff} = b_0 + b_{eff,0}$$

Where $b_0$ and $e_1$ are, respectively, the total width and thickness of the slab cantilever, and $b_{eff,0}$ and $e_2$ are, respectively, the effective width and thickness of the slab between the main girders.

### 7 EFFECTIVE SLAB WIDTH EVALUATION

This chapter presents the longitudinal distributions of axial forces and the evaluation of the corresponding effective slabs width for several different three-dimensional models. The ten 3D models are geometrically similar; however, they present certain differences regarding the construction stages, the type of connection between the stay-cable and the bridge deck, and the live load patterns (Table 7.1). The relevance of these issues in the longitudinal distribution of the slab effective width is studied.

<table>
<thead>
<tr>
<th>Model</th>
<th>Stays anchorages</th>
<th>State</th>
<th>Towers temporary sponsors</th>
<th>Adding intermediate support</th>
<th>Load patent</th>
</tr>
</thead>
<tbody>
<tr>
<td>Mod.A</td>
<td>GIRDER</td>
<td>Construction S.</td>
<td>remov. on stage 2</td>
<td>stage 2</td>
<td>cp</td>
</tr>
<tr>
<td>Mod.B</td>
<td>SLAB</td>
<td>Construction S.</td>
<td>remov. on stage 2</td>
<td>stage 2</td>
<td>cp</td>
</tr>
<tr>
<td>Mod.C</td>
<td>SLAB</td>
<td>Construction S.</td>
<td>Stay on the strut.</td>
<td>stage 2</td>
<td>cp</td>
</tr>
<tr>
<td>Mod.D</td>
<td>SLAB</td>
<td>Construction S.</td>
<td>Stay on the strut.</td>
<td>stage 1</td>
<td>cp</td>
</tr>
<tr>
<td>Mod.E</td>
<td>SLAB</td>
<td>Service S.</td>
<td>Stay on the strut.</td>
<td>stage 2</td>
<td>cp+sc1</td>
</tr>
<tr>
<td>Mod.G</td>
<td>SLAB</td>
<td>Service S.</td>
<td>Stay on the strut.</td>
<td>stage 2</td>
<td>1.5(cp+sc1)</td>
</tr>
<tr>
<td>Mod.F</td>
<td>SLAB</td>
<td>Service S.</td>
<td>Stay on the strut.</td>
<td>stage 2</td>
<td>cp+sc2</td>
</tr>
<tr>
<td>Mod.I</td>
<td>SLAB</td>
<td>Service S.</td>
<td>Stay on the strut.</td>
<td>stage 2</td>
<td>1.5(cp+sc2)</td>
</tr>
<tr>
<td>Mod.H</td>
<td>SLAB</td>
<td>Service S.</td>
<td>Stay on the strut.</td>
<td>stage 2</td>
<td>2.0(cp+sc1)</td>
</tr>
<tr>
<td>Mod.J</td>
<td>SLAB</td>
<td>Service S.</td>
<td>Stay on the strut.</td>
<td>stage 2</td>
<td>2.0(cp+sc2)</td>
</tr>
</tbody>
</table>

*Table 7.1 – Identification and properties of the models studied*

The values of effective width present local amplitude oscillations in the longitudinal direction, in all models, that decrease with the approach of the towers. The settings and amplitudes of these oscillations vary from model to model. These oscillations are the result of local abrupt variations...
of tension in the slab, resulting from the entry of successive compressions of the horizontal cables and by the effects of local bending moments between the cables.

These oscillations and changes in the stresses can be observed in (Figure 7.1), which is presented, to some sections of the model Mod.B, the actual distribution of stresses, the corresponding effective widths, and, in dashed, its variation in length. The values of effective width increase in all sections, except in sections before and after of the cables alignment, which they decrease.

The horizontal component of the cables when inserted into the slab is responsible for the appearance of concentrations of high efforts on lateral sides slab, an effect that is responsible for the reduction of the effective width. It is also noteworthy that in these areas, of the bridge deck the flexion is mostly negative, so this effect is also responsible for reducing the effective width of the slab, first due to the concentration of efforts on the beam and second by the appearance of tractions on the slab level.

In other sections, the spread of the efforts peaks by other areas of the slab also with the accumulation of global concentrations and the existence of positive flexion on the bridge deck leads to the occurrence of increased values of effective width.

The amplitude decrease in the oscillations of the towers occurs because of the accumulation of the global efforts compressions and the progressive reduction of horizontal components introduced in the bridge deck.

From the obtained results in Mod C and Mod D we can identify that the waterways supports are important in the distribution of the effective width values. The local moment bending that occurs on this support usually leads to decreased levels of effective slab width. On this support, the negative bending moment is responsible for the appearance of tractions at the slab level leading to the reduction of global compression and efforts concentrations in the side areas of the slab.

On the other hand these local bending over the supports depend on the applied load level, geometry of load application, and involvement degree of cables in the balancing of the load bridge deck. All these factors contribute to the level of negative bending that arises on the intermediate support.

The increased level of applied load over the entire bridge deck causes an increase of the negative bending moment over the piers and also an increase of the reduction of the effective slab width over the supports. Thus, one can notice that the reductions in the values of $b_{eff}$ over this supports are much higher when the deck is much loaded. However, the effective width reduction is not proportional to the increase of applied load, ie, to higher load levels, the same increase in load results in smaller reductions in the $b_{eff}$ values, and ultimately $b_{eff}$ values tend to stabilize.
Figure 7.1 – Slab longitudinal stress for dead loads and correspondent effective widths
At the span level, the load increments do not significantly change the $b_{\text{eff}}$ value, which means that the increase of the applied loads only causes a uniform increase of the compression diagrams.

Regarding the load pattern, when live loads are applied in the entire bridge deck they do not cause major changes in the expected behaviour of the structure, however, as expected; it causes an increase of axial forces that are acting on the slab. Only at supports lower values of slab effective widths are obtained, due to the increasing in the bending moments.

When the live loads are applied only at the main span, important changes in structure behaviour are observed, due to the "imbalance" of loads between spans. This effect leads to an upward displacement of the lateral bridge spans. The restriction of vertical displacements of the bridge, by the inner retention piers causes the occurrence of positive local bending moments over these supports, and so the concentration of compression efforts in the side areas of the slab (and therefore the effective slab width reduction). For ELU load levels this effect leads to cracking of the slab between the piers P1-P2 and therefore to nil values of $b_{\text{eff}}$.

It was made a comparative study of theses results with those proposed by Byers [3], and proposed a rule to define the effective width of the slab from the test models (Figure 7.2).

Given that the models of Byers and the ones that are being submitted to different conditions, for comparison only considered the results for models Mod.C, Mod and Mod.F in order to compare only those models that have more similarities among them (Figure 7.3). Since that is not referred any information about the value of effective width to be considered on the inner supports, it was the same criterion used for the support of the towers.

As it can be observed, longitudinal distribution of effective width proposed by Byers does not feat completely to the model studied in this work. Byers criteria were inadequate to support spans where there are inner supports. The failure to take into consideration the construction stages means also that at the middle sections of the main span the values obtained were quite different from the criteria defined by Byers.

Introducing some adjustments in the rules proposed by Byers, according with the obtained results, Figure 7.4 proposes a possible distribution of slab effective width, to be considered in longitudinal analysis and design of the composite cable-stayed bridges studied in this work.

It should finally be noted that this proposed distribution is valid when the cantilever construction method is adopted with the sequence of installation and re-stressing used, and for the live load patterns considered in this work. However, it represents an improvement on the Byers proposal as it considers the effects of construction stages, and so approaching to the reality of the construction of long cable-stayed bridges with composite decks.
Figure 7.2 – Comparison of effective width proposed by Byers with the results obtained in some studied models.

Figure 7.3 – Effective slab width proposed and results from the studied models.
8 CONCLUSIONS

The studies done led to the following main conclusions:

1) The 3D model results allowed us to highlight the importance of the stay to deck anchorage points, the construction stages, load patterns and load levels for effective slab widths of composite decks.

2) The stays anchorages has an important contribution to the values of effective slab width, partly because of the direct effect on the deck bending moment, and the other due to significant axial forces, that introduce on the deck.

3) The occurrence of negative bending moment on the deck, usually leads to a reduction to of the effective slab width.

4) The consideration of the intermediate pier supports since the beginning of the construction stages leads, in general, on the supports whether to obtain lower values on effective slab width.

5) The load patterns and load levels affect directly the tensions applied on the slab and therefore the values of effective slab width.

6) Construction stages of the deck were considered and proved they are important it slab effect width definition, mainly at mid-span section of the main span and extremities of the deck.

7) Based on the results obtained, a criterion for effective slab widths of composite decks studied was proposed. This proposal is valid for the parameters used on its definition.
9 BIBLIOGRAPHY


