EFFECTS IN SERVICE OF THE STAGGERED CONSTRUCTION OF CABLE-STAYED BRIDGES BUILT ON TEMPORARY SUPPORTS

Jose Antonio Lozano-Galant[1]✉, Ignacio Payá-Zaforteza[2], Jose Turmo[3]

[1]Dept of Civil Engineering, Castilla-La Mancha University, Avda. Camilo José Cela, s/n, 13071 Ciudad Real, Spain
[2]ICITECH, Dept of Contruction Engineering, Universitat Politècnica de València,
Camino de Vera s/n, 46023 Valencia, Spain
[3]Dept of Construction Engineering, Universitat Politècnica de Catalunya, BarcelonaTech,
c/ Jordi Girona 1–3, C1, 08034 Barcelona, Spain
E-mails: [1]joseantonio.lozano@uclm.es; [2]igpaza@upvnet.upv.es; [3]jose.turmo@upc.edu

Abstract. Cable-stayed bridges can be rarely built on a single construction stage and staggered construction is commonly used. The effects of this staggered construction are not only economical as they might also play an important role in the structural behaviour in service. Despite of this importance, these effects are rarely included into the definition of the structural response in service. In order to fill this gap, this paper deals with the effects in service of the staggered erection of steel cable-stayed bridges built on temporary supports. To do so, a criterion based on the minimization of the bending energy in terms of stay forces is applied to several cable-stayed bridges. This study shows the importance of the existence of the pylon-deck connection as well as the number and location of both construction joints and temporary supports during staggered erection.

Keywords: cable-stayed bridge, temporary supports, construction joint, optimization, objective service stage.

1. Introduction

Cable-stayed bridges stand as one of the most efficient, economical and aesthetic long-span bridge types. Nevertheless, because of the audacity of their designs and their lightness, they are especially sensitive to dynamic and static loads. Furthermore, the erection process of the superstructure and the tensioning operations used to transfer the loads to the stays are of primary importance to assure that the target state of stresses for which the structure was designed, known as the Objective Service Stage (*OSS*), is achieved in service. This stage is characterized by a set of target forces in the stays (Lozano-Galant *et al.* 2012a).

Mid- and long-span cable-stayed bridge superstructures are rarely built in a single operation. In these structures, staggered erection is to accelerate construction and minimize the cost in environmentally sensitive and difficult to access locations. Nevertheless, the importance of the erection process of the superstructure is not only economical as it has also a great influence on the geometry and stress state of the structure during construction and in service. Examples of these are the changes in the longitudinal structural system when concrete or steel segments are successively assembled in cantilever (Wang *et al.* 2004) or over temporary supports (Lozano-Galant *et al.* 2012b,

2013; Lozano-Galant, Turmo 2014a), or in the transverse structural system when the cross sections of wide decks are evolutionary assembled. The linkage of longitudinal and/or transverse segments by casting, welding or bolting results in construction joints that create planes of weakness. In general, a poorly designed, installed, or maintained deck joint becomes to the premature replacement of the bridge or become a dangerous safety hazard to the public as shown, e.g. by the collapse of the Hasselt Road Bridge over the Albert Canal in Belgium in 1938 (Åkesson 2008).

Many researchers (Janjic *et al.* 2003) have stated the importance of including the effects of the erection process into the definition of the *OSS*. Unfortunately, this is rarely the case in current practice as the stress state of the structure in the *OSS* is usually defined in early stages of design, when the construction process is not conceived in detail yet. Furthermore, and despite its importance, the effects of the staggered erection of cable-stayed bridge superstructure in the stress state of the structure has received little attention. Only a few criteria to include the effects of the staggered erection of cable-stayed bridges into the stress state of the structure during construction and in service have been proposed (Lozano-Galant *et al.* 2014; Lozano-Galant, Turmo 2014b).

Additionally, research on one of the main parameters of the staggered erection of the superstructure, the construction joints, is incomplete. Work has been done on some areas such as the influence of on-site defects, residual welding stresses, seismic behaviour (Veletzos, Restrepo 2011) or fatigue resistance (Li *et al.* 2010; Zhu *et al.* 2012). Also, some general recommendations related to the use and location of construction joints have been published in Montana Structures Manual – Part II by Montana Dept of Transportation (USA) in 2002. However, important questions such as "How does the number and location of the construction joints affect the stress state of a cable-stayed bridge in the *OSS*?", remain unanswered. Additionally, if the cable-stayed bridge is built using the temporary support erection technique, new questions such as "How does the number of temporary supports affect the stress state in the *OSS*?" appear.

This paper aims to answer these questions by studying the effects of the main parameters of the staggered erection of the superstructure, the construction joints and the temporary supports, on the stress state of cable-stayed bridges. To do so, several structures, with and without pylon deck connection, erected by construction processes with different number and location of construction joints and temporary supports, are analysed. This analysis shows the important role that the pylon-deck connection plays in the structural behaviour of the structure in service.

This study is focused on:

1) the effects of the structural parameters in the *OSS*;

2) the construction of steel structures;

3) the temporary support erection technique;

4) deck construction with only longitudinal staggered erection, without considering evolutionary cross sections;

5) on linear static analyses, so geometrical or mechanical non-linearities are not taken into account;

6) bridges where the construction joints are placed over a temporary support;

7) bridges where time-dependent phenomena, such as steel cable relaxation, can be neglected.

This loss is traditionally neglected when the ratio of the initial prestress to the yield strength of the steel is lower than 55% (Cluley, Shepherd 1996).

This paper is organized as follows. In Section 2, a criterion to include the effects of the staggered erection of the superstructure into the stress state of the *OSS* by mean of a stay forces analysis is presented. In Section 3, the effects of the number and the location of the construction joints and the temporary supports in the *OSS* of two simplified examples are studied. In Section 4, the conclusions obtained by the analysis of the simplified examples are validated in the model of an actual cable-stayed bridge. Furthermore, the effects of the number of temporary supports over which the superstructure is erected are studied. Finally, in Section 5, some conclusions are drawn.

2. Simulation of the stress state in the *OSS* including the effects of staggered erection

One of the main criteria to define the stay forces in the *OSS* consists on minimizing the bending energy of the structure. The main trade off of this method is the necessity of a numerical integration. In order to avoid this numerical integration a simplified criterion was presented in Lozano-Galant *et al.* (2013). Unlike the minimal bending energy criterion, this method is based on the analysis of the stay cable forces. A number of criteria are found in the literature to define the set of stay target forces that minimize the bending energy in the *OSS* (N^{OSS}). An example of these criteria is the rigidly continuous beam criterion (Lazar *et al.* 1972). According to this criterion, these target forces are calculated as the projection into the stay cable directions of the vertical reactions of an equivalent fictitious beam. Vector N^{OSS} is defined as the sum of a passive set of forces (NP) and an active one (NA) as presented in Fig. 1. On the one hand (NP) is obtained by transferring the permanent loads to the stay system. This vector includes the effect of the evolutionary erection of the superstructure in the reactions of the temporary supports. On the other hand (NA) is defined by the product of an Influence Matrix ($[\Delta N]$) that shows how the axial forces in all the stays vary when a unitary strain is introduced into each stay, and a vector of target imposed strains in the stays (ε) as follows:

$$N^{OSS} = NP + NA = NP + \left[\Delta N \right] \cdot \varepsilon. \qquad (1)$$

The only unknown of Eq (1) is the set of stay strains required to minimize the bending energy of the structure, ε. This vector is directly calculated by mean of the inverse of $[\Delta N]$, $[\Delta N]^{-1}$ as presented in the following equation:

$$\varepsilon = \left[\Delta N \right]^{-1} \cdot \left(N^{OSS} - NP \right). \qquad (2)$$

It is important to highlight that the obtained energy does not always correspond with the minimal possible (the one of an equivalent continuous beam) as higher energy estates are obtained. This is the case of structures with pylon-deck connection, in which the minimal bending energy depends on the construction process of its superstructure. In this case, ε enables the simulation of the structural response in service that minimizes the bending energy. For example, this vector is used to simulate the bending moments in M^{OSS} as presented in Fig. 1 and Eq (3).
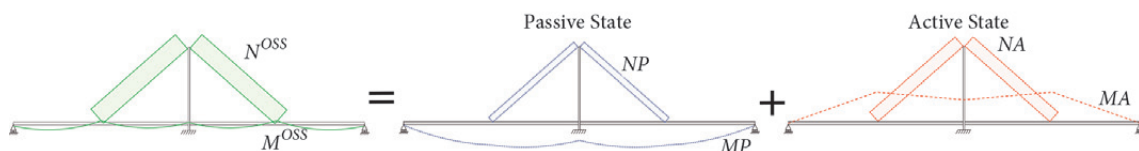

Fig. 1. Bending moment diagrams and stay forces at *OSS*

$$M^{OSS} = MP + MA = MP + \left[\Delta N\right] \cdot \varepsilon, \qquad (3)$$

where MP – sum of a vector of passive bending moments, kN·m; MA – vector of active bending moments, kN·m. The latter vector might be expressed in terms of an influence matrix of the prestressing operations in terms of bending moments ($[\Delta M]$) and ε. It is to notice that M^{OSS} is not a target as it represents the structural response pursued by the simulation. This response is obtained when the value of an adequate set of variables (e.g. strains) is fixed.

The structural response obtained by the minimal bending energy criterion does not depend on the mechanical properties of the superstructure. For example, the more flexible the deck, the higher the deformations and therefore, the higher the passive forces in NP. As target forces N^{OSS} are kept constant, in this case lower active forces NA are required. Nevertheless, this is not the case of the superstructure erection process as the structural response might be affected by the number and location of the construction joints and the temporary supports. These effects depend, to a great extent, on the pylon-deck connection type. To illustrate the effects of different locations of construction joints and temporary supports, several examples with different pylon-deck connections are analysed in the following section.

3. Location of the joints and temporary supports

The number and the location of the temporary supports over which the deck is erected and the number and the location of the construction joints of the deck play an important role in the geometry and stress state of the cable-stayed bridge both during erection and in the OSS. To study the effect of both factors in the stress state of the OSS, two cable-stayed bridges erected by several construction processes are analysed in this section.

3.1. Simplified examples

In this section the two simplified examples presented in Lozano-Galant *et al.* (2013) and named B_3 and B_2 are analysed. These structures are described in Fig. 2, respectively. The differences between both structures are as follow:

1) different number of stays (3 stays for B_3 and 2 for B_2);
2) different type of pylon-deck connection (no connection in B_3 and vertically simply supported in B_2). Both examples are symmetric.

For both structures, Young Modulus is assumed as 206 GPa for deck and pylon and 195 GPa for stays. Deck and pylon area and inertia are 1 m^2 and 1 m^4. Stays area is 0.003 m^2. Inertia of the stays is neglected.

The stay forces in the OSS of both bridges are defined by projecting the vertical reaction of the supports of equivalent beams into the stays direction. The obtained values for a target permanent load of 120 kN/m are presented in Table 1 and Table 2.
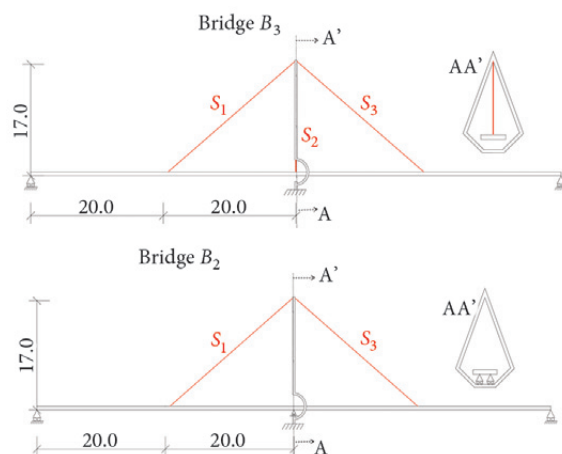

Fig. 2. Simplified examples of bridges B_2 and B_3 (dimensions in metres). *Note:* bridge B_2 has only 2 stays named S_1 and S_3, and stay S_2 in bridge B_3 is aligned with the bridge mast

Table 1. Obtained values for bridge B_2

| Joint	Stay	N^{OSS}, MN	NP_{2-2}, MN	$\varepsilon_{2-2} \cdot 10^{-3}$	NP_{2-3}, MN	$\varepsilon_{2-3} \cdot 10^{-3}$
$x = 10$ m	1	4.24	0.58	−7.34	0.07	−7.32
	2	4.24	0.58	−7.34	0.07	−7.32
$x = 20$ m	1	4.24	0.08	−7.30	0.09	−7.27
	2	4.24	0.08	−7.30	0.09	−7.27

Note: B_2 is erected with 2 (B_{2-2}) or 3 (B_{2-3}) construction joints

**Table 2.** Obtained values for bridge B_3

Joint	Stay	N^{OSS}, MN	NP_{3-2}, MN	$\varepsilon_{3-2} \cdot 10^{-3}$	NP_{3-3}, MN	$\varepsilon_{3-3} \cdot 10^{-3}$
	1	4.24	0.96	−7.40	0.87	−7.87
$x = 10$ m	2	2.22	3.22	−4.05	2.90	−5.75
	3	4.24	0.96	−7.40	0.87	−7.87
	1	4.24	0.98	−7.40	0.96	−7.54
$x = 20$ m	2	2.22	3.19	−4.28	3.31	−4.74
	3	4.24	0.98	−7.40	0.96	−7.54

Note: B_3 is erected with 2 (B_{3-2}) or 3 (B_{3-3}) construction joints

To study the effect of the location of the temporary supports for different number of construction joints, two different staggered construction processes have been considered in structures B_2 and B_3. These erection processes have been named as B_{2-2}, B_{2-3} for B_2 and B_{3-2}, B_{3-3} for B$_3$. The difference between both processes refers to the number of construction joints in the deck: B_{2-2} and B_{3-2} include two construction joints spaced a distance x from abutments. The construction process of these bridges is as follow: firstly, three simply supported segments, x, 80–2x and x m length, are placed over the temporary supports. Then, both construction joints are welded to provide continuity to the deck. Finally, the tensioning sequence is carried out to transfer the load from temporary supports to the stay system. In addition to these two joints, construction processes B_{2-3} and B_{3-3} include an extra construction joint placed at the pylon-deck connection. In these structures the construction process includes four simply supported segments with x, 40–x, 40–x, x m length respectively.
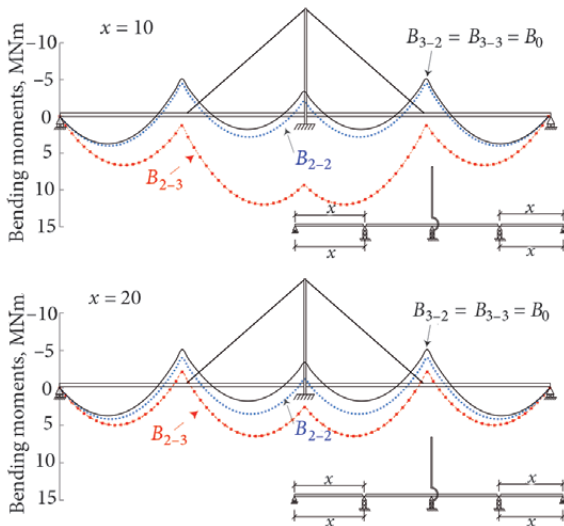
Fig. 3. Bending moments in the deck of structures B_{2-2}, B_{2-3}, B_{3-2}, B_{3-3} and B_0 or different location of the temporary supports and construction joints (x): a – x = 10 m; b – x = 20 m
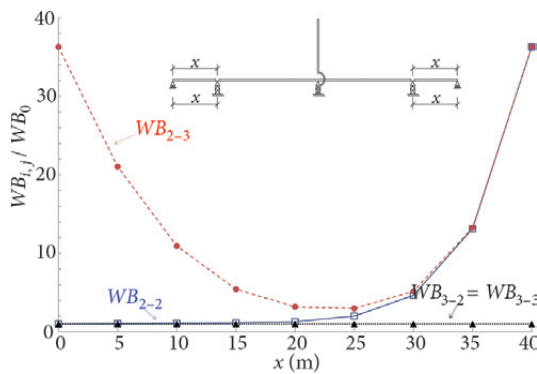
Fig. 4. Ratio between the bending energy of different structures with stays and construction joints and the bending energy of the continuous beam for different locations of the temporary supports and the construction joints

3.2. Study of the location of the temporary supports and construction joints

In this section all the segments described in the preceding section (B_{2-2}, B_{2-3} B_{3-2} and B_{3-3}) are erected over three temporary supports. For the sake of equilibrium, construction joints have to be located over a temporary support and therefore, the location of 2 of these temporary supports has to vary symmetrically with length x (Fig. 3). In addition to these, an additional temporary bent located at the pylon-deck connection is introduced.

The stay forces in the OSS (N^{OSS}), the passive forces in the stays (NP_{2-2}) and (NP_{2-3}) and the calculated target strains (ε_{2-2}) and (ε_{2-3}) for each stay of the different construction processes of B_2 are summarized in Table 1. Table 1 includes the results obtained when construction joints are spaced 10 m and 20 m from abutments (x = 10 m and x = 20 m). Results obtained in the analysis of B_3 are summarized in Table 2. These tables show that the number and the location of the construction joints influence the passive state and consequently, the target strains in the active state. Despite the fact that this procedure assures the achievement of the target forces in the stays in the OSS, changes in the stress state of the OSS usually occur. The bending moments in the OSS obtained by Eq (3) in several construction processes of B$_2$ and B$_3$ are presented in Fig. 3.

The analyses of Fig. 3 show that, independently of the temporary supports location, structures without pylon-deck connection, such as B_{3-2} and B_{3-3}, have the same bending moment diagram as the existing in the rigidly continuous beam B_0. Therefore, it is concluded that in this kind of bridges stress redistribution produced by staggered erection of the superstructure is corrected by prestressing conveniently the stays. Nevertheless, this is not the case in bridges with pylon-deck connection, such as B_{2-2} and B_{2-3}, as the effects of the staggered erection of the superstructure cannot be corrected by stay prestressing and therefore higher sagging bending moments are obtained. The maximum differences between B_0 and the obtained moments in B_{2-3} (5.9 MNm for x = 10 m) and B_{2-2} (2.2 MNm for x = 20 m) are found at the pylon-deck connection. This implies an increase in bending moments of 172.7% and 65.4% compared with the continuous beam. It is to highlight that in B_{2-3}, sagging bending moments are obtained in the vicinity of the pylon-deck connection instead of the hogging bending moments of a continuous beam.

To illustrate the importance that the effects of the location of the temporary supports and the construction joints produce in the stress state of the structure in the OSS of different bridges with pylon-deck connection, Fig. 4 is presented. Fig. 4 presents the ratio between the bending energy of structures with i stays and j construction joints, WB_{i-j} and the bending energy of a continuous beam (WB_0) for different x lengths. The bending energy (WB) of the girder (G) has been calculated numerically from Eq (4):

$$WB = \int_G \frac{M(s)^2}{E(s) \cdot I(s)} \cdot ds, \qquad (4)$$

where $M(s)$ – bending moment at the section s of the girder, kN·m; $E(s)$ – Young's modulus, kN/m^2; $I(s)$ – inertia.

The analysis of Fig. 4 shows the following:

1) The placement of a construction joint in the deck-pylon connection (B_{2-3}) increases significantly the bending energy of the deck. The maximum energy is obtained when the structure is erected without construction joints between pylon and abutment, that is, $x = 0$ and $x = 40$ m. This energy is 35.30 times higher than the minimal bending energy (2.06 kJ). Such energy is explained by the fact that the obtained bending moment diagram has greater sagging bending moments than those of the equivalent continuous beam. As presented in Fig. 3, this increase produces a change of the sign of the bending moment in the surroundings of the pylon-deck connection with the consequent increase of the bending energy of the structure.

2) In this particular example, the optimal location of the lateral construction joints in B_{2-3} is close to 5/8 of the span. In this case, the bending energy 6.20 kJ is 2.99 times higher than the minimal one.

3) To define more accurately the optimal length x, a geometrical optimization is advisable, especially if the number of joints increases.

4) The bending energy of the deck in B_{2-2} depends on x. The bigger x, the larger the bending energy is. Therefore, to minimize the bending energy in this construction process, the construction joints need to be placed as near as possible to the abutments. This requirement has to be compatible with other construction constraints, such as maximum allowable lengths of segments.

5) The bending energy of cable-stayed bridges with no pylon-deck connection (B_{3-2} and B_{3-3}) does not depend on the location of the temporary supports nor on the number of construction joints. In these cases the bending energy of the continuous beam is always achieved.

3.3. Study of the number of temporary supports

To study the effect of the number of temporary supports over which the deck is erected, two different temporary support distributions, every 20 m and 10 m, are analysed for the construction processes B_{i-j} described in Section 3.1. In these temporary supports distributions a number of $t = 3$ and $t = 7$ temporary supports are used during construction (Fig. 5). It is to highlight that in this section, and unlike the preceding one, length x only represents the distance between abutments and lateral joints. Therefore, the location of the temporary supports does not varies with length x.

The bending moments obtained when the lateral construction joint are spaced 20 m from abutments ($x = 20$ m) for both temporary support distributions are presented in Fig. 5. The analysis of Fig. 5 shows that the number of temporary supports is of primary importance

in the bending moment distribution of the structure in the OSS. Independently of the number of temporary supports, the bending moments of a continuous beam are achieved in the OSS of structures without deck-pylon connection, such as B_{3-2} and B_{3-3}. Nevertheless, this is not the case in structures with pylon-deck connection such as B_{2-2} and B_{2-3}. In these structures, increasing the number of temporary supports reduces the differences with the bending moment of a continuous beam. In this case, maximum differences (located at the pylon-deck connection) are reduced from 2.2 MNm to 0.2 MNm in B_{2-2} and from 5.9 to 0.4 MNm in B_{2-3} when the number of temporary supports is increased from 3 to 7. This implies reductions from 65.4% to 5.9% and from 172.7% to 11.7%.

To analyse the effect of the location of the construction joint in both temporary supports distribution Fig. 6 is presented. Fig. 6 shows the ratio between the bending energy (Eq (3) in the OSS for a certain construction process, WB_{i-j}, and that of a continuous beam, WB_0, for different locations of the construction joints (x) with 3 and 7 temporary supports.

The analysis of Fig. 6 shows the following:

1) Independently of the number of temporary supports the minimal bending energy WB_0 is always achieved in structures without pylon-deck connection, such as B_{3-2} and B_{3-3}.

2) In structures with pylon-deck connection, such as B_{2-2} and B_{2-3}, the number of temporary supports is of primary importance in the bending energy of the structure in the OSS. Independently of the erection process, the higher the number of temporary supports, the closer the obtained bending energy to WB_0. In this particular example the ratio of the bending energy is reduced from 1.306 to 1.002 in B_{2-2} and from 3.198 to 1.008 in B_{2-3}.

3) Independently of the number of temporary supports, the minimal bending energy in bridges
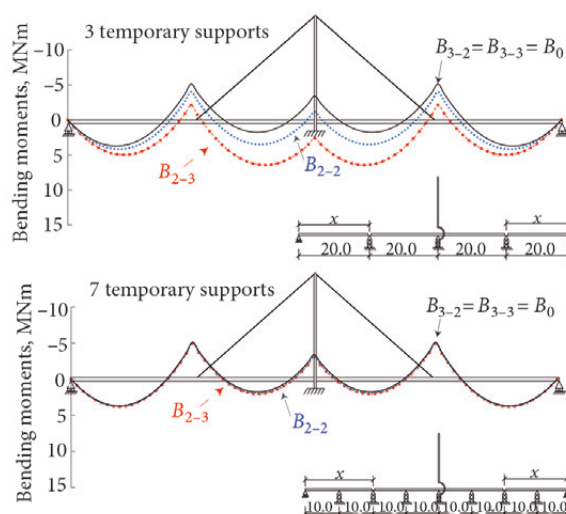
Fig. 5. Bending moment diagrams in the deck of structures B_{2-2}, B_{2-3}, B_{3-2}, B_{3-3} and B_0 built with two 20 m long segments ($x = 20$ m)
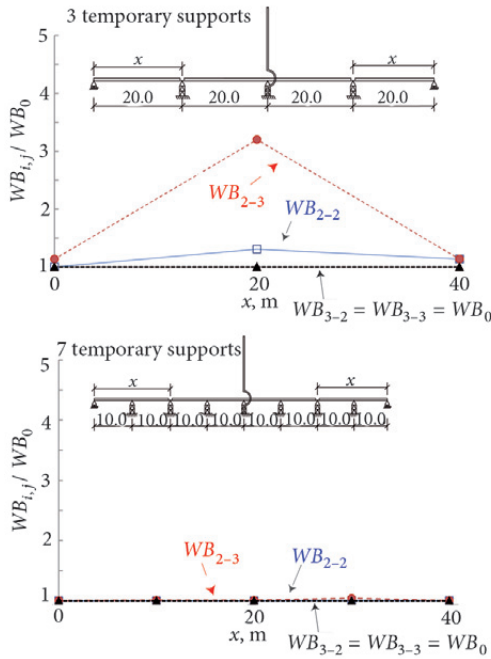
Fig. 6. Ratio between the bending energy of a structure with stays and construction joints and the bending energy of the equivalent continuous beam for different locations of the construction joints and number of temporary supports
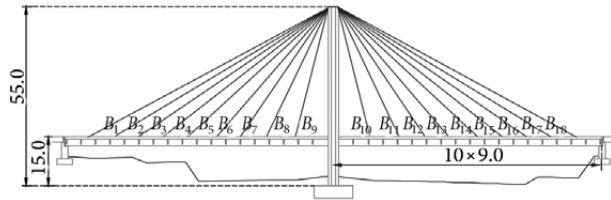

Fig. 7. Cable-Stayed bridge in Wuxi (dimensions in metres)

Table 3. Mechanical properties of the Finite Element Model

Element	E, GPa	I, m^4	A, m^2
Girder	206	4.220	1.719
Pylon	206	11.300	1.220
Stays	195	0.000	0.007

with deck-pylon connection is only be achieved when there are no construction joints in the deck. This is the case of B_{2-2} with $x = 0$ m. It is important to highlight that if a higher number of temporary supports are introduced (e.g. $t = 15$ with temporary support every 5 m) no significant variations with case $t = 7$ are obtained.

4. Cable-stayed bridge in Wuxi

In this section the cable-stayed bridge with 18 stays, B_{18}, presented in Fig. 7 is analysed. This bridge is a simplified model for a project of a cable-stayed bridge in the city of Wuxi in China. The bridge has a 55 m high steel pylon, a 180 m long steel box girder deck and the stays are arranged in a fan symmetrical form. The deck is vertically linked with the pylon. The dead loads of the girder and the pylon are 135 kN/m and 95 kN/m respectively. The mechanical properties in this example are presented in Table 3. The stay forces in the *OSS* (N^{OSS}) are calculated by the Rigidly Continuous Beam Criterion for a target load of 205.5 N/m.

The cable-stayed bridge is studied including the effect of different construction processes of its superstructure. All these construction processes include three construction joints: 2 of these joints are spaced 45 m from both abutments and 1 is located at the pylon-deck connection (Fig. 9). Therefore, the deck is divided into 4 steel segments of 45 m that are transported on site. These segments are firstly simply supported on the temporary supports. In this stage only the self-weight (135 kNm) is applied. Then, they are welded to provide continuity to the deck. Finally, a tensioning process is used to transfer the load of the temporary supports to the stay system. The imposed strains of this tensioning process are calculated by Eq (2).When the rest of the permanent load (70.5 kNm) is applied, the *OSS* is achieved.

As in the case of the analysed simplified examples, the presence of the construction joints influences the stress state of the structure in the *OSS*. This is appreciable in Fig. 8 where the bending moments obtained in the OSS when the superstructure with 3 construction joints is built
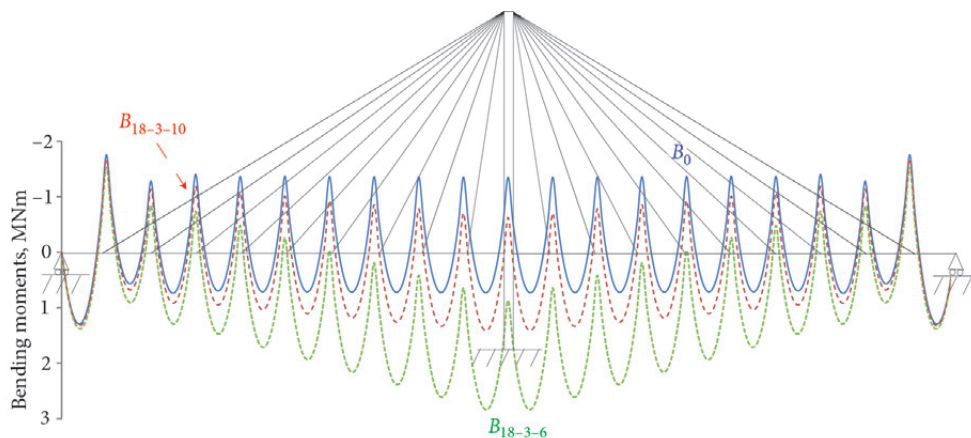

Fig. 8. Comparison between the target bending moments and the bending moments obtained when the structure includes 3 construction joints and is built on 6 or 10 temporary supports

on a set of $t = 6$ and $t = 10$ equidistant temporary supports (B_{18-3-t}) are presented. Fig. 8 shows that the maximum differences with the continuous beam (2.29 MNm in B_{18-3-6} and 0.76 MNm in $B_{18-3-10}$) are found at the pylon deck connection.

The number of temporary supports has a great influence in the stresses in the deck in the *OSS* of cable-stayed bridges with deck-pylon connection. In this structure the possible temporary supports distributions include $t = 6$, $t = 10$, $t = 14$, $t = 18$, $t = 22$, $t = 26$, $t = 30$ and $t = 34$. The effects of the number of temporary supports are especially appreciable in the Bending Moment at the Pylon-Deck connection ($PDBM_{18-3-t}$). This bending moment differs significantly from the target one of the continuous beam $PDBM_0$. The ratio between $PDBM_{18-3-t}$ and $PDBM_0$ is analysed in Fig. 9. As $t = 30$ and $t = 34$ presented negligible differences with $t = 26$, their results are not included in Fig. 9. In this way, analysed cases presented in Fig. 9 are $t = 6$, $t = 10$, $t = 14$, $t = 18$, $t = 22$, and $t = 26$. The analysis of Fig. 9 shows the following:

1) The number of temporary supports influences the sign of the bending moment at the pylon-deck connection. This is appreciable for $t = 6$ where a ratio of –0.61 is obtained. This is explained by the fact that at the pylon-connection a sagging bending moment of 870.11 kNm, is obtained instead of the hogging bending moment of 1387.82 kNm of the continuous beam. The modification of the bending moment sign produces some safety problems if the cross sections of the segments located in the vicinity of the pylon-deck connection as it is not possible to counterbalance sagging bending moments.

2) The higher the number of temporary supports, the lower the differences between the $PDBM_{18-3-t}$ and $PDBM_0$ and therefore, the closer their ratio to 1. For example, this ratio varies from 0.45 for $t = 10$ to 0.97 for $t = 30$.

3) The higher the number of temporary supports, the lower the marginal benefit of adding additional temporary supports. For example, the marginal benefit of the ratio of passing from $t = 14$ to $t = 18$ temporary supports is 0.15, while from passing from $t = 18$ to $t = 22$ and from $t = 22$ to $t = 30$ is reduced to 0.06 and 0.02, respectively.

5. Conclusions

This paper studies the effects in service of the staggered erection of the superstructure of steel cable stayed bridges built on temporary supports. To do so, a criterion based on the minimization of the bending energy in terms of stay cable forces is applied to several simplified and actual examples. In all these examples, the construction joints correspond with a certain temporary support. From the results of these examples, the following conclusions were obtained:

1. The analysis of the simplified cable-stayed bridges illustrates the important role that the pylon-deck connection plays in the bending energy in service. In cable-stayed bridges without pylon-deck connection the bending energy does not depend on the superstructure erection process. In this case, the minimal possible bending energy (the one of a continuous equivalent beam) is always achieved. Nevertheless, this is not the case of structures with pylon-deck connection in which higher minimal bending energies are usually obtained. This unfavourable increase of energy depends on how the superstructure is erected. These results show the convenience of structures without pylon-deck connection. In those cases when a pylon-deck connection is required a detailed study of the construction joints and the temporary supports is required to minimize the unfavourable effect of the staggered construction of the deck.

2. The simplified examples show the effects of the deck construction joints located at the pylon level in cable-stayed bridges with pylon-deck connection. In this case, these joints increase the bending energy in service. For this reason, whenever it is possible, it is advised to avoid them. Nevertheless, in those cases where they cannot be avoided, their locations are of primary importance. In these structures, they are advised to be placed at the proximities of the mid-span. The effects of the central joints are minimized by increasing the number of temporary supports.

3. The analysis of cable stayed bridges with pylon-deck connection shows the important role of the number of temporary supports. In this case, the higher the number of temporary supports, the lower the bending energy in service. Furthermore, the higher the number of temporary supports, the lower the marginal benefit of adding an additional temporary support.
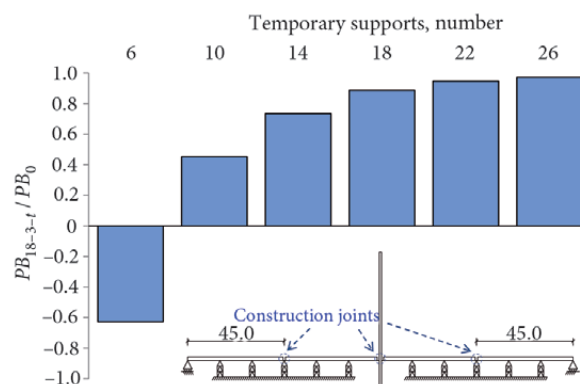
Acknowledgements

Fig. 9. Ratio between the bending moment at the deck-pylon connection of the bridge built with 3 construction joints and the target bending moment at that location for different number of temporary supports

References

Åkesson, B. 2008. *Understanding Bridge Collapses*. CRC Press. 280 p. ISBN 978-0-415-43623-6.

Cluley, N. C.; Shepherd, R. 1996. Analysis of Concrete Cable-Stayed Bridges for Creep, Shrinkage and Relaxation effects, *Computers & Structures* 58 (2): 337–350.
http://dx.doi.org/10.1016/0045-7949(95)00131-Y

Janjic, D.; Pircher, M.; Pircher, H. 2003. Optimization of Cable Tensioning in Cable-Stayed Bridges, *Journal of Bridge Engineering* 8(3): 131–137.
http://dx.doi.org/10.1061/(ASCE)1084-0702(2003)8:3(131)

Lazar, B. E.; Troitsky, M. S.; Douglas, M. C. 1972. Load Analysis Balancing of Cable Stayed Bridges, *Journal of the Structural Division* 92(8): 1725–1740.

Li, L. G.; Ma, Z. J.; Oesterle, R. G. 2010. Improved Longitudinal Joint Details in Decked Bulb Tees for Accelerated Bridge Construction: Fatigue Evaluation, *Journal of Bridge Engineering* 15(5): 511–522.
http://dx.doi.org/10.1061/(ASCE)BE.1943-5592.0000097

Lozano-Galant, J. A.; Ruiz-Ripoll, L.; Paya-Zaforteza, I. J.; Turmo, J. 2014. Modification of the Stress-State of Cable-Stayed Bridges due to Staggered Construction of Their Superstructure, *The Baltic Journal of Road and Bridge Engineering* 9(4): 241–250. http://dx.doi.org/10.3846/bjrbe.2014.30

Lozano-Galant, J. A.; Turmo, J. 2014a. An Algorithm for Simulation of Concrete Cable-Stayed Bridges Built on Temporary Supports and Considering Time Dependent Effects, *Engineering Structures* 79: 341–353.
http://dx.doi.org/10.1016/j.engstruct.2014.08.018

Lozano-Galant, J. A.; Turmo, J. 2014b. Creep and Shrinkage Effects in Service Stresses of Concrete Cable-Stayed Bridges, *Computers and Concrete* 13(4): 483–399.
http://dx.doi.org/10.12989/cac.2014.13.4.483

Lozano-Galant, J. A.; Dong. X; Paya-Zaforteza, I. J.; Turmo, J. 2013. Direct Simulation of the Tensioning Process of Cable-Stayed Bridges, *Computers & Structures* 121: 64–75.
http://dx.doi.org/10.1016/j.compstruc.2013.03.010

Lozano-Galant, J. A.; Paya-Zaforteza, I. J.; Dong. X; Turmo, J. 2012a. Analysis of the Construction Process of Cable-Stayed Bridges Built on Temporary Supports, *Engineering Structures* 40: 95–106. http://dx.doi.org/10.1016/j.engstruct.2012.02.005

Lozano-Galant, J. A.; Paya-Zaforteza, I. J.; Dong. X; Turmo, J. 2012b. New Algorithm to Model the Construction Process of Cable-Stayed Bridges Built on Temporary Supports, *Engineering Structures* 40: 119–130.
http://dx.doi.org/10.1016/j.engstruct.2012.02.022

Veletzos, M. J.; Restrepo, J. I. 2011. Modeling of Jointed Connections in Segmental Bridges, *Journal of Bridge Engineering* 16(1): 139–147.
http://dx.doi.org/10.1061/(ASCE)BE.1943-5592.0000112

Wang, P. H.; Tang, T.; Zheng, H. 2004. Analysis of Cable-Stayed Bridges during Construction by Cantilever Method, *Computers & Structures* 82(4–5): 329–346.
http://dx.doi.org/10.1016/j.compstruc.2003.11.003

Zhu, P.; Ma, Z. J.; French, C. E. 2012. Fatigue Evaluation of Longitudinal U-Bar Joint Details for Accelerated Bridge Construction, *Journal of Bridge Engineering* 17(2): 191–200.
http://dx.doi.org/10.1061/(ASCE)BE.1943-5592.0000257