Structural Engineering and Mechanics, Vol. 45, No. 5 (2013) 703-722
DOI: http://dx.doi.org/10.12989/sem.2013.45.5.703

Concrete arch bridges built by lattice cantilevers

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(Received August 29, 2012, Revised January 25, 2013, Accepted February 19, 2013)

Abstract. In this paper a study about concrete arch bridges built by lattice cantilevers is presented. Lattice cantilevers are partial structures composed of deck, arch, piers and provisional steel diagonals, organized as reticular cantilever girders, in order to build arch bridges without the use of centrings, supports or temporary towers. Characteristics of this construction methodology with its variants are explained together with their implications in the erection sequence. Partial elastic scheme method is implemented in order to find initial forces of temporary cables and a forward analysis is carried out to follow the actual sequence of construction, by extending a procedure already applied to concrete cable–stayed bridges and to arches built by the classical suspended cantilever method. A numerical application on a case–study of a concrete arch bridge is performed together with a comparison between different methodologies followed for its construction sequence. Differences between erection by lattice cantilevers and cable-stayed cantilevers, are discussed. Results can be useful for designers in conceptual design of concrete arch bridges.

Keywords: arch bridge; lattice cantilever; partial elastic scheme; staged construction; cable-stayed

1. Introduction

Concrete arches have traditionally been built by means of centrings and temporary supports. In the last century, in order to avoid these expensive provisional structures, cantilever construction of arches spread around the world, starting from the idea of J. Eads for the St. Louis steel bridge in 1874. The same method was used by Eiffel for the Maria Pia and Garabit steel bridges; Eugene Freyssinet applied this methodology for the first time to concrete arches (Fernandez Troyano 2003). Today cantilever construction of arches is a widely used method, particularly for concrete bridges with the deck above the arch (Fig. 1(a)). It is similar to that used for cable-stayed bridges, the two half-arches being suspended from auxiliary temporary stays; after key closure, the stays are removed and deck is built on the completed arch. This method was used for example in the construction of the Bloukrans Bridge, South Africa, in 1984 (Sirolli and Capitanio 1986). Suspended cantilevers need two temporary towers, in order to obtain efficient inclined stays,

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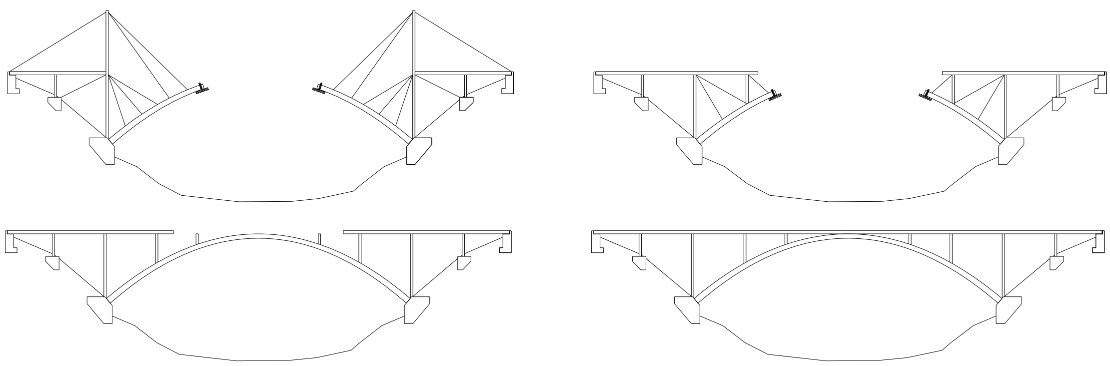
especially for construction of midspan arch segments. An interesting alternative, used for long-span concrete arches, is the lattice cantilever method in which the reticular girder is composed of deck and arch segments connected by piers and temporary diagonals (Fig. 1(b)).

The latter methodology comes directly from the construction of cantilever bridges, like the Rip one in Brisbane, built in 1974, a cantilever truss in which concrete diagonals remain as definitive elements, after construction end (Ranalli 1976, Leonhardt 1986). Arch bridges can be built in the same way but steel diagonals are removed after completion of the bridge. This method was used for the first time in the erection of the Hokawatsu arch, in 1978 (Manterola Armisen 2006, Fernandez Troyano 2003), almost simultaneously to the Krk bridges, built in Croatia; afterwards it was applied to many arches (Fernandez Troyano 2004). The Krk bridges (Stojadinovic and Huet 1981), respectively world span records of 244 and 390 m, were built as cantilevers in which the compressed bottom chord is the arch itself while the top chord is supplied by steel cables; provisional steel diagonals are put between the top of a pier and the bottom of the next one, at the joint with the arch segments. After key closure, the deck was built on the piers and provisional steel members (upper cables and diagonals) were removed.

The Los Tilos arch bridge on La Palma island (Pérez Fadón *et al*. 2005), with a span of 255 m, was built in a similar way, but in this case the top tensile members were supplied by steel beams, assuring greater axial and bending stiffness of the link elements between the tops of concrete piers. Steel provisional elements were incorporated into the definitive deck girder after completion of the arch, as longitudinal and transverse structural pieces of the deck cross-section.



(a) cable-stayed cantilevers (b) lattice cantilevers

Fig. 1 Cantilever construction of an arch bridge

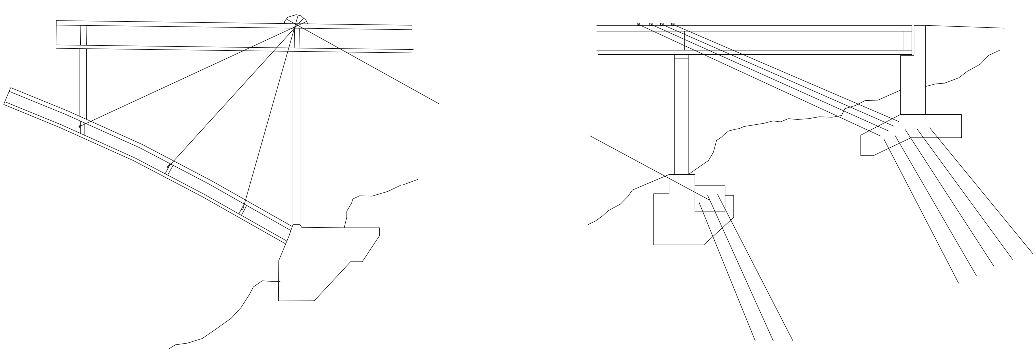
Truss members were supplied by stiff longitudinal beams and temporary rigid diagonals in the construction of the Rio Almonte bridge in Spain, with a span of 184 m (Llago Acero 2006).

La Regenta Ana Ozores arch, with a span of 194 m, which is part of the Pintor Ferrios viaduct in Spain, was built with the same methodology (Arenas *et al*. 1997). In this case the top chord is the bridge deck itself, and the construction proceeded with the contemporary erection of arch and deck, connected by the definitive vertical piers and auxiliary steel diagonals. After key closure and the completion of the upper deck, the provisional steel elements were removed (Pepponi 2000). The bridge was completed in 1996.

At the same time, in 1995, a steel-concrete composite arch bridge, with a span of 168 m, was built in Ricobayo, Spain, with the same methodology (Pérez Fadón and Herrero Beneítez 1999). The concurrent construction of deck and arch was also used in the La Peña bridge, Spain, an arch with a span of 148 m, in which the upper prestressed concrete deck was the top chord of the partial truss cantilevers.

Recently the Infante Dom Henrique bridge in Porto was built with the same method (Adão da Fonseca and Millanes Mato 2005), by proceeding with deck and arch together. This particular bridge has a span of 280 m, a sag/span ratio of 1/11 and an arch thickness of 1.50 m, which makes it a very low and slender arch. It is a deck-stiffened arch bridge, in which the slender arch shows great sensitiveness to second-order effects. This happens not only in service life but also during construction, when excessive deflections due to dead loads and time-dependent phenomena like creep can induce large differences between theoretical and actual arch geometry. In this bridge, the lattice cantilever method was effectively adopted in order to avoid the loss of arch shape, by creating stiff reticular partial structures composed of deck, arch, vertical piers and provisional pre-tensioned diagonal cables. In this way, by acting on these pre-tensioned cables, deflections and construction errors could be minimized, allowing engineers to control the entire process with good precision and to limit arch and deck displacements during the cantilever stages of construction (Adão da Fonseca and Bastos 2004). In this bridge two temporary concrete piers were used in order to reduce the cantilever span during construction; these piers were removed after bridge completion together with auxiliary diagonal cables.

Lattice cantilevers have the disadvantage of longer construction times compared to the traditional suspended cantilever method. This is because it is necessary to complete the triangular system arch-deck-pier in order to have an efficient cantilever truss, before starting the construction of the subsequent arch segments. On the other hand, in suspended cantilevers, the arch is completed before the deck and the girder can be assembled upon the arch, after pier construction, for example by incremental launching technique (Arici and Granata 2007, Granata *et al*. 2013a).



(a) provisional diagonals (b) backstays and ground anchorages

Fig. 2 Details of anchorages

In lattice cantilevers joint connections between temporary diagonals, arch and piers are details of fundamental importance for the entire construction sequence. They must ensure an adequate value of joint stiffness and also their design is often conditioned by the search for a simple way to give the required value of pre-tension forces to stays. When rigid steel profiles are used as diagonal members instead of cables, the anchorage point can be complex and has to be studied specifically for the case. When diagonals are made of cables, a stiff anchorage is incorporated in the arch, fixed during segment casting. The desired value of pretension can be given by acting with a jack, often placed in an active anchorage above or inside the deck. If diagonals are made of steel bars, a stretcher can be put in a convenient position in order to obtain the right pre-tension force by giving a relative displacement between diagonal ends. Regarding the anchorage of backstays and upper retain cables in the ground, it is generally solved by an adequate foundation, in which the anchorage point is placed inside an appropriate chamber. In most cases the high values of tensile forces have to be compensated for ties deeply anchored to the ground (Fig. 2), as in the construction of the Dom Henrique bridge (Adão da Fonseca and Bastos 2004).

In all these methodologies, the layout of provisional cables or rigid diagonal members and their tensile stress values are key elements in order to ensure the effectiveness of the implemented construction sequence. The evaluation of stay pre-tension forces is a common problem of arch bridges built by cantilevering and concrete cable-stayed bridges, with the fundamental difference given by the curvilinear arch shape. The effects of creep in the staged erection of concrete cable-stayed bridges were investigated by Arici *et al*. (2011), while arch bridges built by the suspended cantilever method were discussed in Granata *et al*. (2012a). In order to find the initial force value in pre-tensioned steel members, the Partial Elastic Scheme (PES) method has been used. This method is based on the implementation of a zero-displacement procedure on elastic schemes for each erection stage (Wang *et al*. 2004, Arici *et al*. 2011). The same method is applied here to the study of the construction sequence of arches erected by lattice cantilevers with or without the simultaneous presence of the deck as the top chord. The method is explained, in order to show its effectiveness and the wide range of related applications. Implications due to different methodologies of arch erection are considered, based on the actual cases of recently built bridges. A numerical application is presented on a concrete arch, with a span of 198 m, built by the lattice cantilever method. The PES method is used in order to find the initial cable forces, by implementing a forward staged construction on a Finite Element (FE) model. A comparison between different methodologies investigated for arch bridge construction is explained and discussed, the characteristics of each method and the differences between them being evaluated. The methodologies are the following: lattice cantilevers with arch and upper deck built simultaneously, lattice cantilevers with upper retain cables, and cable-stayed cantilevers. The target is to investigate the implications of the PES method applied in these different situations and the consequences of performing forward staged construction analysis. The results in terms of deformed shape, state of stress and cable forces can be useful to designers, particularly in the earlier phases of a project when structural choices have to be made regarding the conceptual design of arch bridges.

2. Partial elastic scheme method for concrete arch bridges

In staged construction analysis two kinds of procedure can be implemented: the backward and the forward ones. In the first case the bridge is analysed in its final desired state and then dismantled by following the inverse path with respect to erection (Chen and Duan 1999). In the second case the actual sequence of construction is followed. Backward analysis is used today mainly in order to achieve the initial cable forces at each stage for stays in cantilever construction of cable-stayed and arch bridges (Manterola Armisen 2006). Then a forward analysis is always implemented by introducing the initial value of pre-tension forces and their adjustments, found by the backward procedure, stage by stage. Moreover a forward analysis needs always when time-dependent phenomena as creep and shrinkage in concrete have to be taken into account (Arici *et al*. 2011, Granata *et al*. 2013b) or when non-linear effects such as cable sag and beam-column effect, occur (Wang *et al*. 2004). Initial cable forces are found in this work by an alternative method with respect to backward analysis, based on partial elastic schemes of every stages.

Let us consider an arch bridge, built by the suspended cantilever method in N_s stages; the k-th partial scheme, related to the k-th construction stage, is shown in Fig. 3. Let $\mathbf{t}_{(k)}$ be the array of pre-tension forces $t_{(k)i}$ of n stays to be pre-tensioned in that stage. Let $\boldsymbol{\delta}_{(k)}$ be the array of displacements $\delta_{(k)j}$ of m control points, conveniently established in the joints of cables and structure related to the n stays, and due to loads applied in the reference elastic scheme. The influence matrix $\mathbf{D}_{(k)}$ ($m \times n$) can be assembled, which gives displacement $d_{(k)ji}$ of control point j due to the unitary value of cable force $t_{(k)i}$.

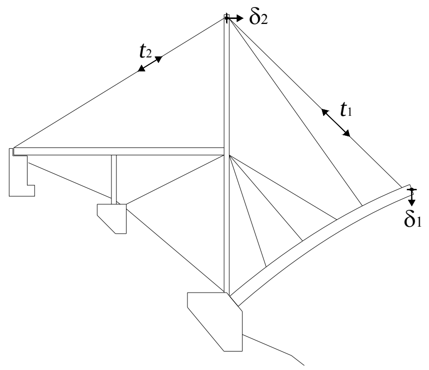


Fig. 3 *k*-th partial elastic scheme in suspended cantilever construction of an arch bridge

By applying the zero-displacement method to the partial elastic scheme, values of cable forces $\mathbf{t}_{(k)}$ can be found, in order to make null control points displacements

$$\mathbf{D}_{(k)}\,\mathbf{t}_{(k)} + \boldsymbol{\delta}_{(k)} = 0 \tag{1}$$

If a pre-camber has to be imposed in such points by a higher value of cable pre-tension, Eq. (1) can be changed into

$$\mathbf{D}_{(k)}\,\mathbf{t}_{(k)} + \boldsymbol{\delta}_{(k)} = \boldsymbol{\delta}_{(k)}^{*} \tag{2}$$

in which $\boldsymbol{\delta}_{(k)}^{*}$ is the array of desired control point displacements. Eqs. (1) and (2) can be simply solved when the number of control points is exactly the same of pre-tensioned stays ($n = m$); in this case cable forces are found by the following relation

$$\mathbf{t}_{(k)} = \mathbf{D}_{(k)}^{-1}\,(\boldsymbol{\delta}_{(k)}^{*} - \boldsymbol{\delta}_{(k)}) \tag{3}$$

If $m \neq n$, $\mathbf{D}_{(k)}$ is not a square matrix and a good approximation of the solution can be found instead by the relation

$$\mathbf{D}_{(k)}^{\mathrm{T}}\,\mathbf{D}_{(k)}\,\mathbf{t}_{(k)} + \mathbf{D}_{(k)}^{\mathrm{T}}\,(\boldsymbol{\delta}_{(k)} - \boldsymbol{\delta}_{(k)}^{*}) = \mathbf{0} \tag{4}$$

in which $\mathbf{D}_{(k)}^{T}\,\mathbf{D}_{(k)}$ is a square and symmetric matrix (Recupero and Granata 2013).

Solution of Eq. (2) or (4) gives the array $\mathbf{t}_{(k)}$ of pre-tension forces in the k-th reference elastic scheme. In this way it is evident that each elastic scheme, in which the zero-displacement algorithm is applied, is composed of a partial structure referred to a construction stage and it is analyzed by applying permanent loads and prestressing forces related only to that stage. If stays are attached to the structure for the first time in that scheme, $t_{(k)i}$ represents the initial cable force, otherwise it represents an adjustment of a previously attached cable.

In Fig. 3 only the new stay attached is stressed in the reference scheme, but it is possible to adjust other stays previously attached only when it can be advantageous, by increasing the dimensions of matrix $\mathbf{D}_{(k)}$. The choice of pre-tensioning one or more stays at each stage is an important design parameter in order to implement a convenient sequence of stay tensioning during construction and it is strictly related to the forward staged construction analysis to be performed in the model. Sequences with one or two stays pre-tensioned at each stage have been considered by authors for cable-stayed bridges (Arici *et al*. 2011), in which a two-phase stressing procedure can be useful in order to reduce effects of creep in cable-stayed bridges with concrete deck. Moreover it is useful to achieve a better precision in the geometric shape and a more convenient bending moment diagram for composite cable-stayed bridges, in the final dead load configuration (Granata *et al*. 2012b). In some cases a single stay stressing procedure does not permit to achieve the exact desired geometric profile but on the other hand a multiple stressing of each stay in different phases, implies technological problems, because it is necessary to shift the stressing equipment stay by stay. For cable-stayed bridges too many stress adjustments are not convenient because they imply that cable free length is marked many times, gripping strands in areas where marks exist from previous wedge seating, with the consequence of a local strength reduction.

A stressing sequence in which one stay is stressed at each stage has been considered by Granata *et al*. (2012a) for arch bridges built by cable-stayed cantilevers for which the PES method has been implemented. The same choice of a single stressing operation, has been taken for lattice cantilevers, for which the k-th reference partial elastic scheme is shown in figure 4. In arch bridges some authors prefer to re-stress many stays at each stage (Missbauer 1981, Brenni and Dazio 1987), but it is not necessary when a convenient value of initial cable force is found and a forward analysis is implemented in design phases, as it has been verified in previous applications of PES method.

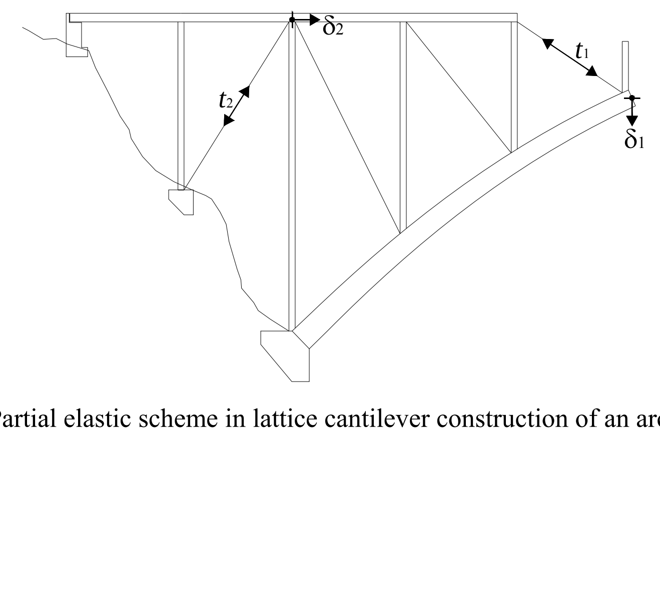


Fig. 4 Partial elastic scheme in lattice cantilever construction of an arch bridge

If steel diagonals are made of cables, a pre-tension force can be given at each stage only to the new cable attached. Backstays instead need to be adjusted many times during construction, because they have the function of anchorage to the ground. By solving the related *k*-th elastic scheme (fig. 4) and by applying the zero-displacement procedure by means of Eq. (1), one obtains

$$\begin{bmatrix} \mathrm{d}_{11} & \mathrm{d}_{12} \\ \mathrm{d}_{21} & \mathrm{d}_{22} \end{bmatrix}_{(k)} \begin{bmatrix} t_1 \\ t_2 \end{bmatrix}_{(k)} = -\begin{bmatrix} \delta_1 \\ \delta_2 \end{bmatrix}_{(k)} \tag{5}$$

where d_{ji} are elements of the influence matrix $\mathbf{D}_{(k)}$ while δ_j are the vertical and horizontal displacements of control points due only to dead loads applied in the *k*-th reference scheme. From Eq. (5) values of t_1 and t_2 are found; while t_1 represents the initial force for the new stay attached, t_2 is the adjustment force of backstay at the reference stage. Eq. (5) has to be applied for all $\mathrm{N_s}$ partial elastic schemes and the values found have to be introduced into the forward procedure. Nowadays modern software packages implemented the automatic assemblage of the requested influence matrix, the staged construction and the time-dependent analysis. Forward procedure gives the final result in terms of displacements and internal forces, after the entire sequence of construction has been analyzed. The engineer can choice, in design phases, how many times the stay has to be stressed in the sequence and which stay force and control point has to be considered into Eq. (5) at each stage. Usually it is convenient to stress every cables only once.

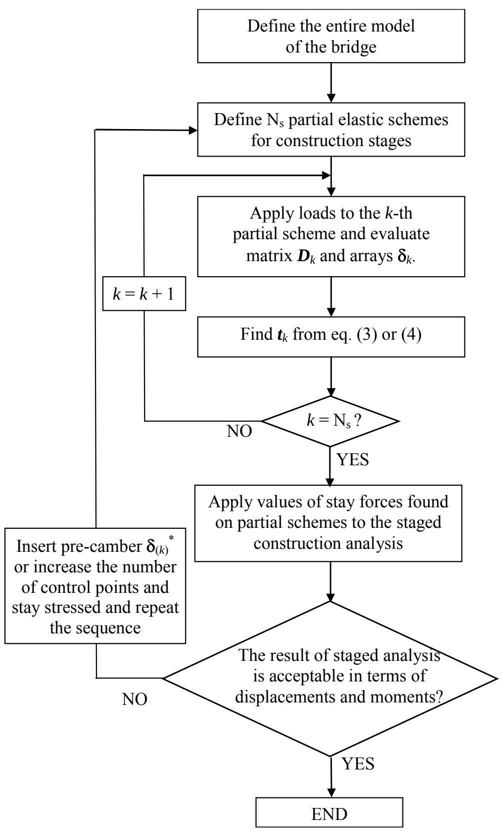


Fig. 5 Flow chart of PES method

Adão da Fonseca and Bastos (2004) applied the influence matrix method in order to get the initial cable forces stage by stage. They implemented an iterative procedure based directly on cable forces and their mutual effects instead of the zero-displacement procedure applied here in the reference elastic scheme. Corres Peiretti *et al.* (2001) found values of stay forces by referring to the theoretical geometric arch shape, for the Burguillo project.

In the Ricobayo bridge (Pérez Fadón and Herrero Beneítez 1999) a forward sequence has been applied in which initial cable forces are found by a backward analysis; the pre-tensioned diagonal cables allow designers to reduce the cantilever tip vertical displacement from 31 cm to 7 cm during the assemblage of the steel arch. After the completion of steel elements, an adjustment of that cables has been performed during the operations of arch concrete filling and deck slab casting. The importance of pre-camber has been underlined by these authors; it has been achieved in that bridge by applying the technique of Freyssinet through an imposed strain in the arch key, in order to recover the thrust loss. A pre-camber can be considered already in design phases by introducing it in the partial elastic scheme, by means of non-zero values given to array $\delta_{(k)}^*$ of Eq. (4). It can be done in the intermediate stages, in order to minimize deflections due to permanent actions and at the end of construction, in order to recover displacements due to time-dependent phenomena. In Fig. 5 flow chart which describes the procedure adopted with the PES method is given.

In the following a numerical application is explained to make clear the proposed method and to compare different ways of erection for the same arch bridge structure.

3. Numerical application

An arch bridge, taken from a real case-study (Arenas *et al.* 1997), composed of a stiff arch with a bi-cellular boxed cross-section and an upper deck with a single-cell cross-section, is presented here. The methodology previously described is applied to the bridge, by implementing the PES method. The geometric characteristics of the bridge are shown in Fig. 6.

The concrete arch has a span of 198 m and a rise of 50 m, giving the rise/span ratio $f/l = 0.25$.

The arch section has a constant width of 10 m and a variable height, from 4 m at arch footings to 2.5 m at the key segment. The 9 m wide upper deck is composed of 21 spans each being 18 m long; only 11 spans are resting on the arch, the remaining ones being supported by piers and

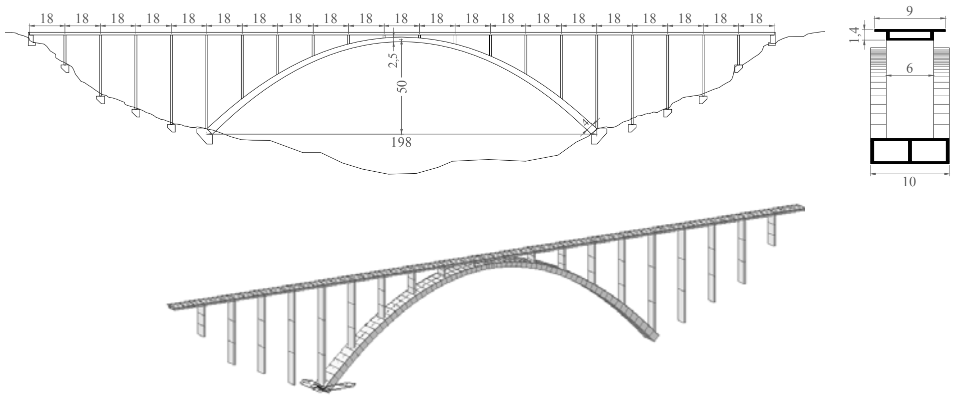


Fig. 6 Geometric characteristics of the bridge and finite element model view [m]

abutments. The stays have an equivalent steel diameter of ϕ_s = 0.0353 m, while the backstays have a diameter of ϕ_b = 0.0462 m. The concrete strength is f_{ck} = 45 MPa while the stay steel strength is f_{ptk} = 1860 MPa.

Following three possible construction methodologies were studied for this bridge:

1) lattice cantilevers with the simultaneous construction of arch and deck;

2) lattice cantilevers for arch construction with upper retain cables to compensate for tensile stresses of the top chord, and the subsequent construction of the deck on the completed arch;

3) cable-stayed cantilevers for arch construction and subsequent erection of deck on the completed arch.

A linear elastic forward analysis was performed by taking into account creep and shrinkage effects on deformations and stress redistribution in time. The FE model is composed of 2D frame elements for arch, deck and piers and other additional truss elements for cables.

The analysis takes into account tangent displacement during erection, in order to limit the effects of deformed configuration on the staged construction. As a matter of fact, when a new segment is built by the cantilever method, by assembling it with the previous one already completed and by stressing the related stay, it is necessary to choose the right position of the new segment by following the tangent direction with respect to the tip of the previous segment, already deformed (Granata *et al*. 2012a). The same procedure has to be implemented in cable-stayed bridges, in order to avoid the rise of discontinuities between geometrical axes of segments.

The arch is fixed to footings. The deck is continuous over piers and simply supported on the abutments.

Creep and shrinkage effects were considered in the analysis. Among the different possibilities given by the specialized literature and international codes to take into account time-dependent phenomena in concrete, fib Model Code 2010 (Fib 2012) was selected in order to evaluate long-term deflections in the present application. In the United States the creep model of ACI209 is still used (ACI 1997), though it has been recently updated by ACI 209 Committee, by inserting more recent models (ACI 2008).

The parameters used to perform forward analysis and to consider creep and shrinkage effects are the following:

- relative humidity: RH=70%;
- notional size of the cast element (equivalent fictitious thickness): $h = 2A_c/u$ = 700 mm, where A_c is the mean concrete area of the arch cross-section and u its external perimeter which comes into contact with the external environment;
- cement type: normal, moist cured.

With these parameters, the creep model gives $\varphi(10000, 7) = 1.64$, in which $\varphi(t, t_0)$ is the creep coefficient at time t (days) due to a sustained load applied at time t_0. The data refer to a loading time of 7 days for each segment and a time of 10000 days for the final step of the analysis. The shrinkage model gives the total shrinkage strain at the final time of analysis: $\varepsilon_{sh}(10000) = 2.36 \cdot 10^{-4}$. The construction chronology is the following:

- side spans: 90 days, total duration 90 days;
- two symmetric half arches by cantilevering with piers: 7 days for each segment and 14 days for each pier, total duration 200 days;
- key closure: 7 days, total duration 207 days;
- removal of auxiliary stays: 7 days, total duration 214 days;
- when the deck is built above the arch: 90 days, total duration 304 days;
- final time for creep and shrinkage: 10000 days.

A comparison of the forward analysis results is presented and discussed in order to evaluate the differences between the three construction methodologies as well as the effectiveness of the PES method, for the case in which the arch is much stiffer than the deck.

3.1 Arch bridge built by lattice cantilevers with upper deck

A first analysis was performed by considering the lattice cantilever composed of arch, deck with its definitive section, piers and temporary diagonal cables. In this case the FE model is composed of 181 joints, 191 frame elements and 30 truss elements for stays and the forward analysis is performed on 18 stages. The PES method was implemented on $N_s = 14$ elastic schemes; examples of partial elastic schemes are reported in Fig. 7.

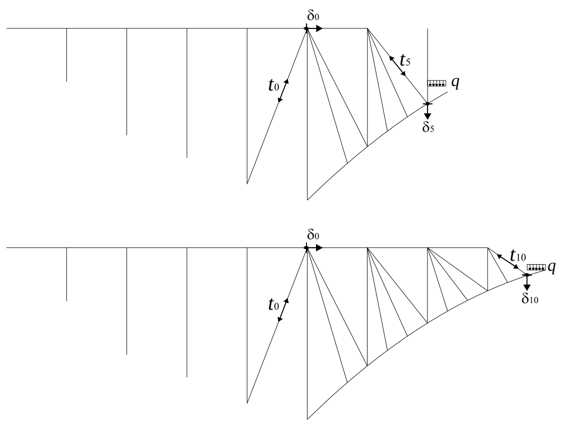


Fig. 7 Examples of elastic schemes for lattice cantilevers with upper deck

Figs. 8(a) and 8(b) show the deformed shape respectively of the last cantilever stage and at the end of construction, due to permanent actions applied during the entire sequence. A table with displacement values and a comparison between the results is shown in section 3.4.

At any rate the contribution of the upper deck in limiting deflections is evident: this kind of construction allows engineers to have a cantilever truss with stiff top and bottom chords (the deck itself and the arch). In this case, the pre-stress values given to diagonal cables have the main aim of maintaining the cantilever geometric profile close to the expected one, minimizing the relative displacements between deck and arch in the construction stages.

Fig. 9 shows the diagram of maximum and minimum bending moments during the entire sequence. This methodology of construction avoids centerings or provisional supports but implies different behaviour of concrete segments between construction stages and the completed arch: in intermediate phases the structure is mainly subjected to bending moments, while after key closure the arch takes on its mainly axial behaviour. The bending moment diagram is very useful for designers, because it directly refers to the dimensioning of reinforcements in the arch sections. Moreover, it allows engineers to check whether concrete sections crack or not; indeed, section cracking can be avoided by limiting bending tensile stresses in the cantilever stages, during the entire construction sequence, when compressive axial force due to the arch effect is not yet available.

Fig. 8 Lattice cantilevers with upper deck. (a) Deformed shape at the last cantilever stage. (b) Deformed shape at the end of construction [cm]

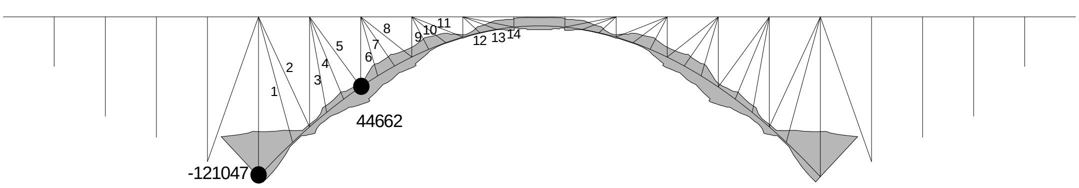


Fig. 9 Lattice cantilevers with upper deck. Max-min bending moment diagram in the arch for the entire construction sequence [kNm].

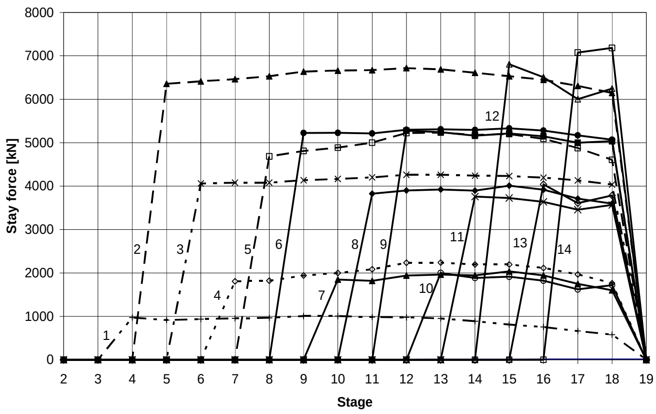


Fig. 10 Lattice cantilevers with upper deck. Stay forces diagram for the entire construction sequence

Due to the construction methodology, even if the arch shape has been designed as the anti-funicular of dead loads, bending moments appear in the cantilever stages and residual values remain in the arch after key closure and stays removal. These values of bending moments have to be superimposed on those due to live loads on the bridge in service life (particularly moving loads), so reinforcements increase in the arch with the increase in bending moment residual values due to construction.

Fig. 10 shows the diagram of the cable forces for each stay in the entire construction sequence.

3.2 Arch bridge built by lattice cantilevers with upper tensioned cables

The second analysis performed considers the lattice cantilever composed of arch, piers, temporary diagonal cables and temporary upper rectilinear retain cables, instead of the deck. This construction methodology is similar to that used for the erection of the Krk bridges (Stojadinovic and Huet 1981).

In this case the FE model is composed of 193 joints, 191 frame elements and 40 truss elements for stays and the forward analysis is performed in 19 stages. The PES method was implemented on N_s=15 elastic schemes; examples of these schemes are reported in Fig. 11.

Figs. 12(a) and 12(b) show the deformed shape respectively of the last cantilever stage and at the end of construction, due to dead loads applied during the entire sequence. A greater value of displacement and a more evident bending influence on the deformed shape were found with respect to the previous construction sequence. This is due to the deformability of the piers and upper cables; in order to compensate for dead load displacements, the upper cable must be tensioned to high values and pier deformations increase because a stiff link between their tops is not supplied. The final result shows less precision in achieving the required geometric shape of the arch and consequently an inaccurate positioning of the upper deck.

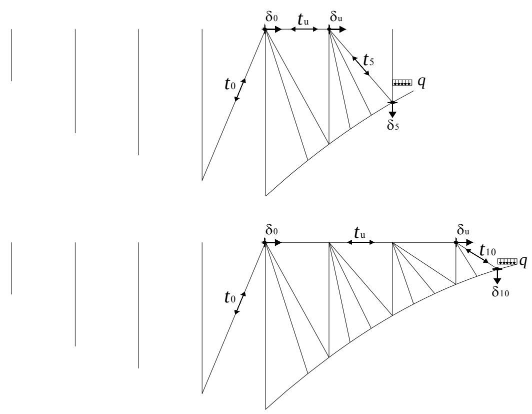


Fig. 11 Examples of elastic schemes for lattice cantilevers with upper cables

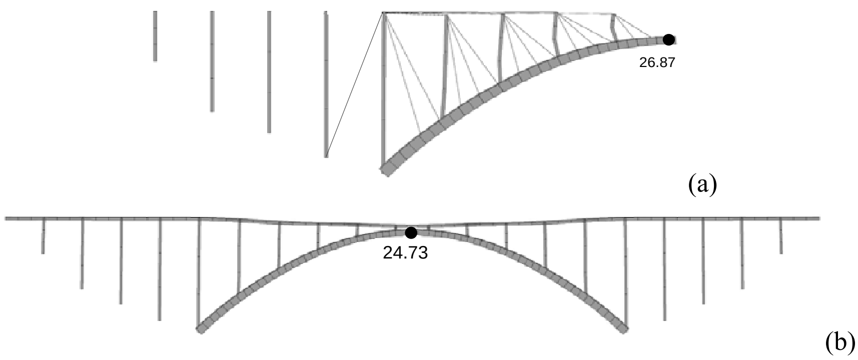


Fig. 12 Lattice cantilevers with upper cables. (a) Deformed shape at the last cantilever stage (b) Deformed shape at the end of construction [cm]

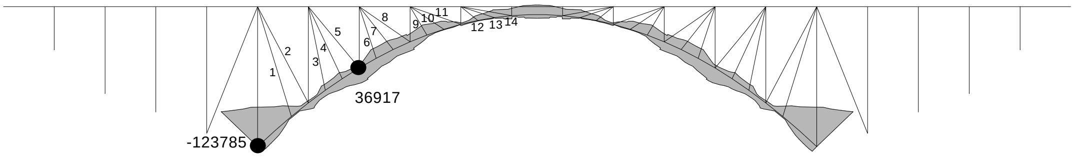


Fig. 13 Lattice cantilevers with upper cables. Max-min bending moment diagram in the arch for the entire construction sequence [kNm]

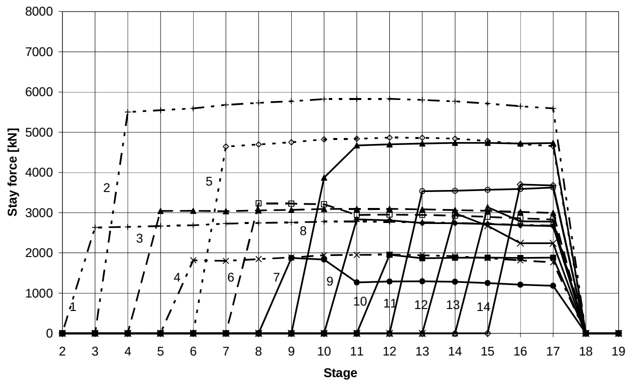


Fig. 14 Lattice cantilevers with upper cables. Stay forces diagram in provisional diagonal cables for the entire construction sequence

Fig. 13 shows the diagram of maximum and minimum bending moments during the entire construction sequence for this second methodology. A similar result with the previous solution was found with lower values of maximum bending moments.

Fig. 14 shows the diagram of stay forces for the entire construction sequence. The results for provisional diagonals are similar to the previous case with lower maximum values.

3.3 Arch bridge built by cable-stayed cantilevers

The last analysis regards the classical cable-stayed cantilever construction; in this case a temporary tower, from which the longer stays start, was added to the FE model, which is composed of 183 joints, 193 frame elements and 34 truss elements for stays. The forward analysis is performed on 19 stages and the PES method was implemented on N_s = 15 elastic schemes (Fig. 15).

Figs. 16(a) and 16(b) show the deformed shape respectively of the last cantilever stage and at the end of construction, due to dead loads applied during the entire sequence.

The difference from the previous construction sequences due to the role assumed by stays is evident: in this case the cables literally suspend the arch till its completion, while in the previous cases provisional diagonals have the role of activating and completing the behavior of the cantilever, as a truss made of concrete and steel elements. These distinct roles imply different values of prestressing forces and also a different way to control the arch shape during construction. As a consequence the maximum vertical displacement is not found at the cantilever tip but at an intermediate arch section. Fig. 17 shows the diagram of maximum and minimum bending moments during the entire construction sequence for the third methodology. A similar result was found to the second one, with a smaller value of the minimum moment.

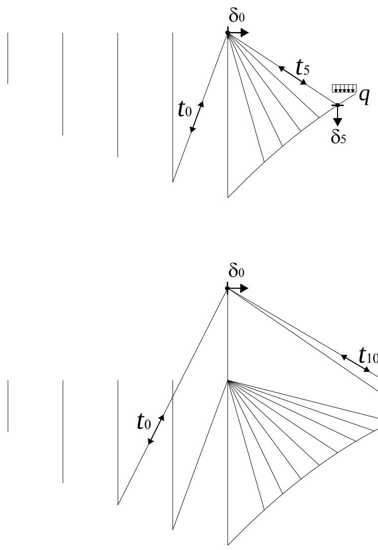


Fig. 15 Examples of elastic schemes for cable-stayed cantilevers

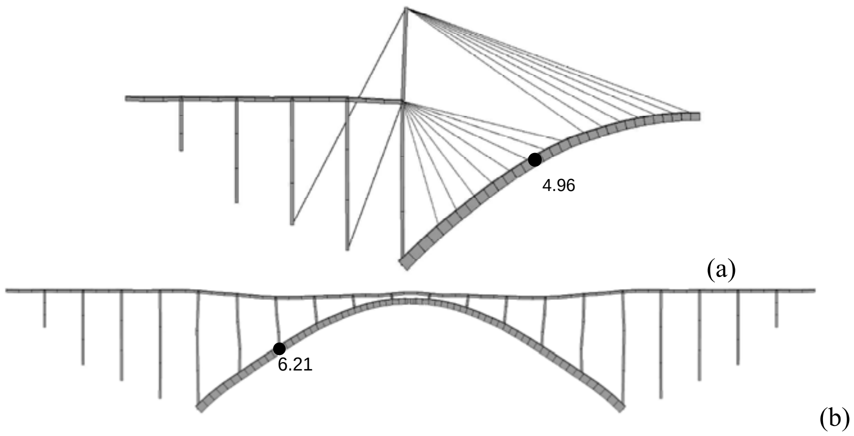


Fig. 16 Cable-stayed cantilevers. (a) Deformed shape at the last cantilever stage. (b) Deformed shape at the end of construction [cm]

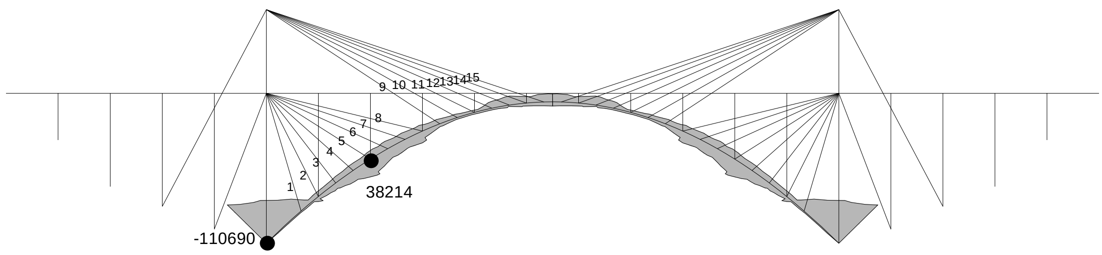


Fig. 17 Cable-stayed cantilevers. Max-min bending moment diagram in the arch for the entire sequence [kNm]

Fig. 18 shows the diagram of stay forces for the entire construction sequence. In this case lower values of axial forces were found, though they refer to longer stays attached to provisional towers, which are not present in the previous methodologies. This implies different costs for this methodology compared to the other ones, due to the provisional tower and the longer stays; besides, a more precise control of arch shape can be achieved thanks to the supporting function of the free length cables placed between tower and arch as in cable-stayed bridges. In the previous methodologies, prestressed cables are less effective because their efficiency depends strictly on deck, piers and arch stiffness.

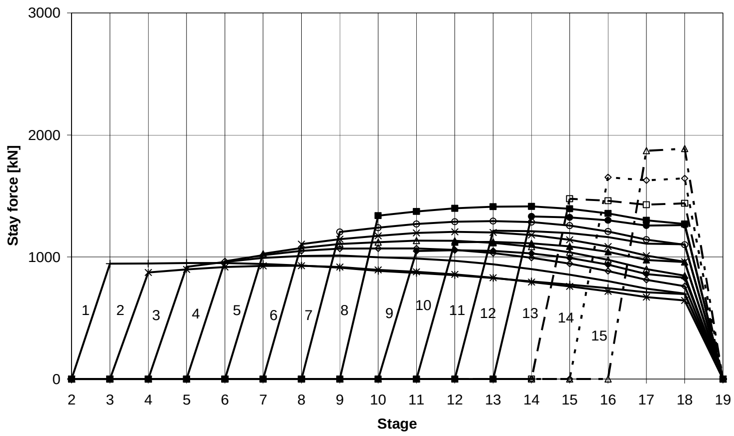


Fig. 18 Cable-stayed cantilevers. Stay forces diagram for the entire construction sequence

A gradual variation in the stay prestressing force can be observed in Fig. 18. For each stage the new stay attached shows a small increment in axial force compared to the previous one and this value does not vary significantly in successive stages. This is a good result of the PES method applied to this construction methodology, as pointed out by the authors in a previous paper (Granata *et al*. 2012a). As reported in the literature, other authors found more variable trends of stay axial forces with large increasing and decreasing cable stress fluctuations, during construction, due to different approaches in finding initial cable forces (Corres Peiretti *et al*. 2001, Sirolli and Capitanio 1986).

3.4 Comparison and discussion of results

A comparison of numerical values for maximum vertical displacement results of the three solutions is reported in Table 1. It is evident, from this table, that the cable-stayed cantilever solution gives the best results in terms of deformed shape at the end of construction and at the final time of analysis. In the case of lattice cantilevers with upper deck and cable-stayed cantilevers, the influence of creep and shrinkage significantly increases the value of the final vertical displacement with respect to erection end. Negative values of displacement due to shrinkage in cantilever phases depend on the behavior of the system arch-deck-piers for which the cantilever tip has upward displacements for the effect of shrinkage alone. For lattice cantilevers with upper cables creep gives the maximum downward displacement at the construction end due to the influence of cantilever stages in which the behavior is mainly governed by bending. After that, creep increases downward displacements of intermediate sections of the arch while slightly decreases the displacement of the key section.

Table 2 shows a comparison of maximum and minimum bending moments in the entire construction sequence. For residual bending moments after construction the best result is also given by the cable-stayed cantilever method.

Table 1 Comparison of maximum vertical displacements for the three analyses performed [cm]

	Lattice cantilevers with upper deck			Lattice cantilevers with upper cables			Cable-stayed cantilevers		
Stage	Total	Creep	Shrinkage	Total	Creep	Shrinkage	Total	Creep	Shrinkage
Last cantilever	4.18	1.27	-0.20	26.87	5.51	0.35	4.96	1.42	0.02
Construction end	7.56	2.10	-0.03	24.73	5.25	0.43	6.21	1.60	0.04
Final time of analysis (t = 10000 days)	12.41	4.88	1.38	26.72	4.70	2.98	9.44	3.56	1.37

Table 2 Comparison of maximum and minimum bending moments [kNm]

Lattice cantilevers with upper deck		Lattice cantilevers with upper cables		Cable-stayed cantilevers	
Max	Min	Max	Min	Max	Min
44662	-121047	36917	-123785	38214	-110690

Regarding stay forces, an evaluation of the diagrams in Figs. 10, 14 and 18 makes it clear that the stays are less stressed in cable-stayed cantilever construction, so the third methodology appears to be the most satisfactory one from the structural point of view, though it implies the construction of temporary towers. The PES method gives a good result in terms of displacements and residual bending moments at erection end, in all construction methodologies for the stiff arch investigated. In a previous paper (Granata *et al*. 2012a) the PES method was implemented with good results on a bridge with different arch geometrical characteristics and rise/span ratio, built by the suspended cantilever method.

Lattice cantilevers appear more satisfactory when the deck is built simultaneously to the arch, in order to reduce the effects of pier bending deformability. On the other hand, the simultaneous construction of deck and arch makes the erection sequence more complex. In this methodology the presence of deck during construction implies a higher value of dead loads and a consequent higher value of prestress force in cables. In order to eliminate this drawback, the top chord of lattice cantilevers can be supplied by steel beams linking tops of piers, as in the Los Tilos arch bridge (Pérez Fadón *et al*. 2005), but it is a rational choice only if these beams remain as definitive members of the deck cross-section. In this case, the top tensile member has to be compensated by anchorages in the ground which also serve to fix the deck, preventing horizontal displacements of the cantilever top chord.

Lattice cantilevers with rectilinear upper cables seem to be the worst method because cables are too deformable to give good results in terms of displacements and geometric arch shape. The use of cables could be convenient if a final adjustment of upper and diagonal cables stress is made in the stage of key arch closure.

A pre-camber is convenient and advisable in all cases by imposing a higher profile of arch and deck, in order to recover downward displacements due to construction sequence and time-dependent phenomena. This can be done by modifying the imposed values of displacements in partial elastic schemes (array $\delta_{(k)}^*$ of Eq. (2)). The pre-camber displacement values can be given by the following procedure:

1) the forward analysis is performed a first time on the FE model as done in the previous sections, so that the partial elastic scheme method is implemented by imposing $\delta_{(k)}^*=0$ in Eq. (3) or (4) and displacements in the final stage are found;

2) downward displacements found by this analysis are inserted with the opposite sign in array $\delta_{(k)}^*$ and a new iteration is done by performing the forward analysis.

In this way, a convenient value of pre-camber can be applied and an improved result can be found in terms of arch and deck geometric shapes. The desired value of pre-camber can be inserted in array $\delta_{(k)}^*$ for each elastic scheme and deflections due to permanent actions can be recovered in every construction stage.

Another technique making it possible to provide a pre-camber is the Freyssinet method, giving an imposed strain in the arch key segments by jacks (Pérez Fadón and Herrero Beneítez 1999). In concrete bridges, creep reduces the effectiveness of the imposed strains, because of the stress relaxation due to the principles of linear viscoelastic theory (Chiorino 2005).

In suspended cantilever construction, another alternative is to stress the last cables attached to a higher tensile value, recovering the thrust loss due to the arch elastic shortening. In this way the arch key can be closed at a higher position with respect to the design one: afterwards elastic shortening and time-dependent phenomena will bring the arch into the required position (Corres Peiretti *et al*. 2001).

Even though the analysis performed in this study concerns only the static behavior, the dynamics of arch bridges has to be considered in the design stage. Creep effects on the dynamic behavior of arch bridges were considered by Ma *et al*. (2011), while dynamic tests for concrete arch assessment are referred by Ozden Caglayan *et al*. (2012)

Design choices depend not only on the structural behavior but also on costs; in the bridge examined the increment in steel weight for cables in suspended cantilever construction is about 80% compared to lattice cantilevers with upper deck and about 40% compared to lattice cantilevers with upper retain cables. Moreover, the costs of additional temporary towers must be considered for cable-stayed cantilevers. These aspects must be considered by designers together with construction times, because lattice cantilevers need longer times for arch segment concrete casting, while suspended cantilevers grow with high speed. Based on these considerations, lattice cantilevers with upper retain cables seem not to be satisfactory from both the structural and economical points of view, because they imply long construction times with the delayed construction of deck, high values of steel weight for cables and anchorages and unsatisfactory structural behavior.

Regarding evaluation of the three methodologies investigated, it is important to underline that the present results were found on a bridge in which the arch bending stiffness is greater than the deck stiffness. This implies that the arch has sufficient stiffness in the cantilever stages to face bending moments. Different results could be expected when the arch is much slenderer than the deck. For deck-stiffened arch bridges, in which the arch is very slender with an upper stiff girder, lattice cantilevers with the simultaneous construction of arch and deck can be the most advantageous methodology; it takes advantage of a stiff top chord that renders the system much more stable. In deck-stiffened bridges, the arch is generally dimensioned to face axial forces and it is assisted by the deck to face bending moments, so cable-stayed construction may not be the optimal one, involving the risk of concrete cracking or arch instability. A bridge with a low value of rise/span ratio was built by the suspended cantilever method in Spain and evaluations of construction and monitoring are reported by Corres Peiretti *et al*. (2001).

When these bridges also have lower values of rise/span ratios, the thrust increases significantly and the danger of geometric shape loss can become very important, leading to the need for nonlinear analyses (Adão da Fonseca and Millanes Mato 2005). A very interesting comparison of different characteristics of concrete bridges built by suspended and lattice cantilevers since 2000, can be found in Corres Peiretti *et al*. (2001), with useful information for designers.

Another parameter that has to be taken into account, for arches to be built by lattice cantilever, is pier spacing. In a bridge with many piers connecting arch and deck, the funicular shape is close to a curve: in this case the lattice cantilever is a stiffer truss due to the efficiency of reduced triangles in the arch-deck-pier system. Otherwise in a bridge with a few piers (an increased pier spacing/arch span ratio) the funicular shape is a polygonal curve and the cantilever truss is more deformable and less effective, when cables are used as temporary diagonals. In the latter case it could be a good choice to have diagonals made of steel profiles, because they present an increased area and hence increased axial stiffness with respect to cables (Llago Acero 2006).

4. Conclusions

Construction of concrete arch bridges by the lattice cantilever method has been presented. Lattice cantilevers are partial reticular structures composed of deck, arch, piers and provisional steel diagonals, for which arches are built without the use of centrings, supports or temporary towers. The partial elastic scheme method was implemented in order to find the initial forces of temporary cables and to perform a forward analysis following the actual construction sequence, by extending a procedure already applied to concrete cable-stayed bridges and to arches built by the suspended cantilever method. A numerical application on a case-study of a concrete arch bridge is performed with a comparison between different methodologies for its construction sequence: lattice cantilevers with upper deck, lattice cantilevers with upper retain cables and cable-stayed cantilevers. The results show that the suspended cantilever method gives the most precise response in terms of deformed shape, residual bending moments in the arch and stay forces, especially for stiff arches. This methodology implies different costs and construction equipments, so a cost-benefit evaluation has to be carried out. The fact is that lattice cantilevers with upper deck have the advantage of avoiding provisional towers with a good stiffness value between piers during the erection sequence, while lattice cantilevers with upper cables show too much deformability of the elements and the disadvantage of delayed deck construction. Evaluations of the characteristics of each methodology and the results presented here can be useful for engineers in the conceptual design of concrete arch bridges.

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