Steel and composite tied-arch bridges: a conceptual approach to structural design

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Abstract

The behaviour of tied-arch bridges with composite decks (bowstring bridges) is overly sensitive to the different choices made by the designer in the first phases of the conceptual approach, mainly in the design of the arch and the deck. The geometry of the arch shape, the design of the arch-tie joint and the construction sequence can significantly modify the global behaviour in terms of stress state and deformed configuration. After a general discussion on the different parameters affecting the behaviour of bowstring bridges with steel-concrete composite decks, the consequences of choices made in the conceptual design phases are discussed, analysing the effects of each choice on the global behaviour of the bridge with particular focus on the potential of significant concrete slab cracking and on the reinforcement amount to be used in the deck slab for limiting the crack widths. Acting on geometry, materials and construction sequence, the bridge behaviour can be optimised, and examples of this approach are shown on two actual cases of bridges in Southern Italy.

Keywords: Arch, Tie, Bowstring, Bridges, Composite structures, Shrinkage, Slabs & plates, Cracking, Conceptual design

1 Introduction

The arch represented the optimal solution for bridges till the 18th century due to its structural efficiency, because when the axially rigid arch is designed following an anti-funicular curve of loads, its transverse sections are in overall compression under uniform loads. Hence, depending on the length to be crossed, in masonry solution, bridges were built with single or multi-span arches. Afterwards the advent of new materials, such as steel and concrete, together with innovative construction techniques, marked a turning point in the history of bridge construction. Although resistance to compression is fundamental to the arch working principle, in the case of arch bridges it is not possible to completely avoid bending moments, because the thrust line cannot always coincide with the geometric axis for all live load combinations, due to the variability of traffic loads. Further, the axial deformability of arch

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cross-sections implies an unavoidable thrust loss in redundant structures (2-hinge or clamped arches) which produces positive bending moments in the central part of the span even for the dead load considered in the construction of the anti-funicular axis curve. When traffic loads were not so big with respect to dead loads, as occurred in historical masonry bridges, arch sections were designed with the right thickness to maintain the thrust line in the central core of inertia and to avoid tensile stresses for all load combinations. In modern bridges, instead, the ratio between live loads and dead loads has increased, and arch sections are designed considering the effect of significant bending moments.

In any case the main condition to obtain arch behaviour is significant thrust at the footings, i.e. horizontal forces in the foundations. When the soil is not adequate to receive these forces, it is possible to compensate this through a tie placed between the arch footings; in this way solely vertical reaction forces can be obtained. Hence the whole tied-arch structure works like a simply supported beam, in which the arch is a curved compressed member and the tie is in tension (fig. 1a). In bridges with an upper arch, the tie can be provided by the deck itself. In such cases the deck is suspended from the arch by metallic hangers. This is the so-called bowstring structure, used either with concrete or steel arches. In the classical solution the hangers are vertical and contained in the arch and tie plane, but different arrangements can be found throughout the historical evolution of these bridges with inclined hangers in longitudinal and transverse directions. Longitudinally, the most recurrent solutions are parallel and radial arrangements or network arches (Fernandez Troyano, 2003; Tveit, 1980). In transverse directions inclined arches (and consequently inclined hangers) appeared in the 19th and 20th centuries with inward and outward solutions.

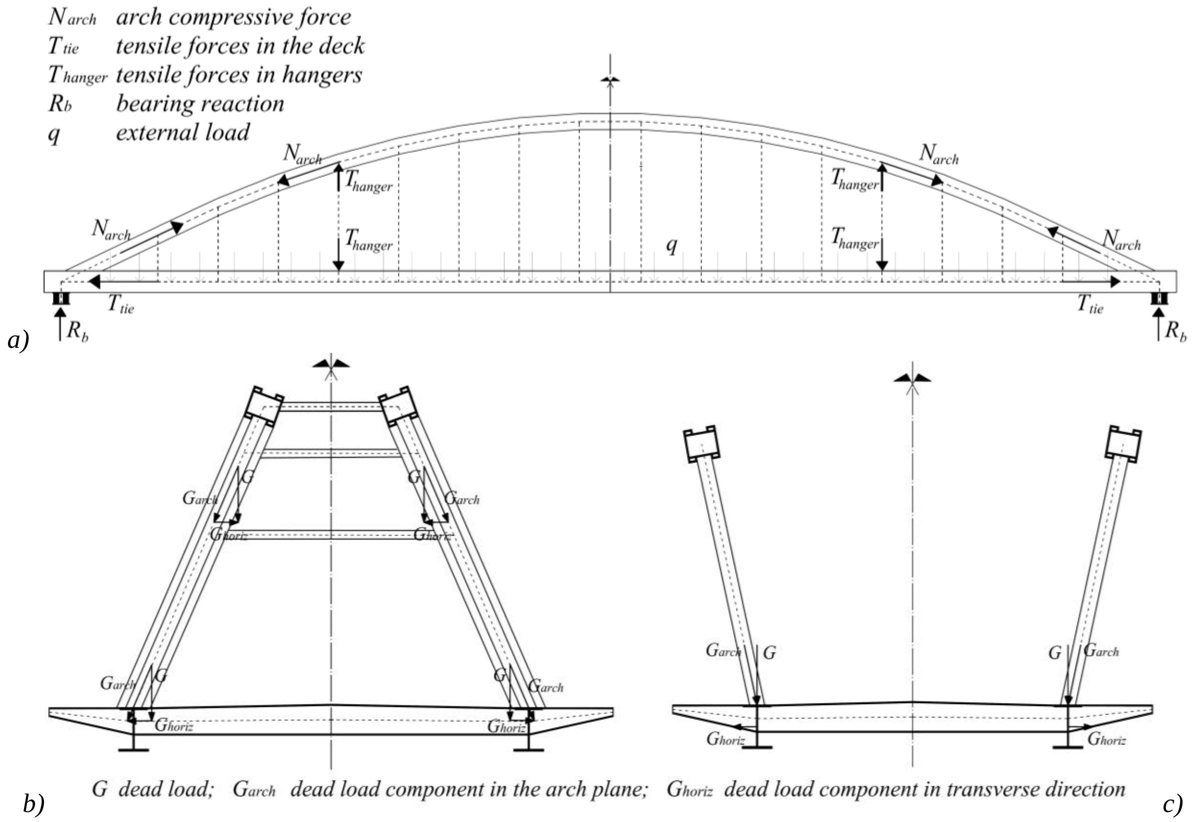
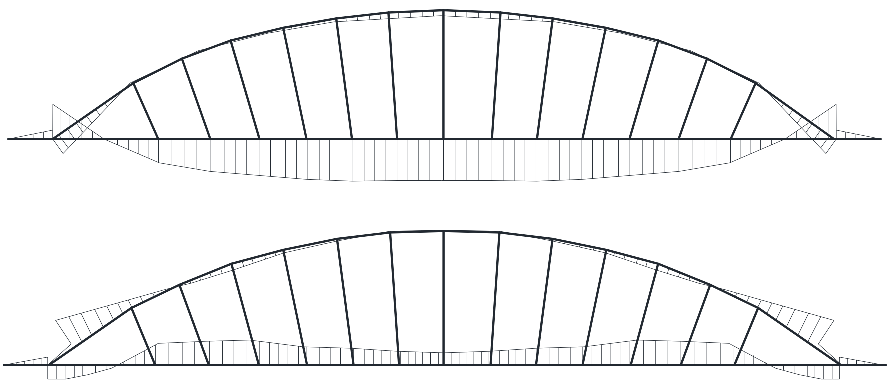


Figure 1. Variability of bowstring bridge solutions. a) arch-tie longitudinal behaviour in the arch plane; b) transverse behaviour with inward arches; c) transverse behaviour with outward arches.

Longitudinally the behaviour of the tied arch is similar to that of the Nielsen truss: a truss where the upper member is curved and compressed as an arch (Collings, 2013; Johnson, 2003). A difference between vertical and inclined hanger arrangements can be found in the behaviour for live loads because compressive forces may appear in inclined hangers; hence they have to be stiffened or pre-tensioned in order to avoid instability and fatigue failure. When hangers are formed from metallic bars, cables or tendons, i.e. members with negligible global bending stiffness, they can be pre-tensioned conveniently in the construction stages. Through pre-tensioned hangers the bending moment diagram in the arch and deck can be modified even for dead loads, because metallic hangers can be considered equivalent to elastic restraints in which the vertical force can be modified; hence the deck bending moment diagram depends strictly on the hanger forces. When instead rigid hangers are adopted with non-zero bending stiffness (Sun et al., 2020), pretension is generally impractical and the global behaviour is similar to that of a truss.

In the transverse direction the classical configuration is that of inward inclined arches which are stiffened with top transverse elements (fig. 1b); in this configuration it is common to adopt either rigid bars or pre-tensioned hangers. When the arches are inclined outward (fig. 1c) it would be beneficial to use hangers with transverse bending stiffness to improve the arch stability and pre-tension is difficult to be applied. In figure 2 the effect of hanger pre-tensioning (Granata et al., 2013a; Au et al., 2003) on the bending moment diagram is shown for a bridge without or with a construction pre-tensioning sequence. Hanger pre-tensioning in the construction stages modifies the stress pattern as well as the profile of the final configuration by controlling the displacements of the deck, especially in the case of segmental construction, together with the design pre-camber. An appropriate sequence of hanger pre-tensions avoids unsymmetrical distribution of stresses, overstress of elements or unsuitable behaviour at the service limit state (Hayashi et al., 2010).



a)

b)

Figure 2. Effect of hanger pre-tensioning for dead loads: a) Bending moment in the bridge without any pre-tension; b) bending moment at the end of construction with hanger pre-tensioning sequence.

The above brief description of the possible configurations of bowstring bridges can give an idea of the wide variety of structural parameters to be managed in the conceptual design stages by the engineer.

Arch and deck can be made of concrete or steel or combined (Collings, 2013). With concrete arches the deck can be made of prestressed concrete or composed of steel elements, in order to resist the tension of the tie effect. A commonly used configuration for spans of medium length is the steel arch with the tie made of a composite steel-concrete beam, which is the main subject of this paper. In this case the concrete slab is a fundamental element in the structural arrangement of the bridge for different reasons and in different phases of the construction and service life. The concrete slab can be part of the deck cross section or not (it can act compositely or not with the principal longitudinal beams); it can be totally cast in situ or partially prefabricated; it can be cast with or without temporary supports; it can be cast before or after the pretension of hangers. All these possibilities imply different arch-deck behaviours and different states of stress and strain of concrete and steel members.

In the following sections the conceptual design of tied-arch bridges with composite decks is presented with the aim of reducing the potential for significant concrete slab cracking and the amount of reinforcement to be used. By acting on the geometry of the arch and deck and on the construction sequence, the behaviour of the bridge can be optimized regarding both stress state and durability. The different parameters involved in the design phases and the consequences of each design choice are shown, focusing mainly on tied-arch bridges with stiff hangers, where hanger pre-tensioning is difficult to implement for modifying the global behaviour, in contrast to bridges with cable or flexible hangers.

In the first part of the paper, the evaluations are made through a theoretical and conceptual approach for evaluating the effect of different parameters on bridge behaviour; afterwards two actual cases experienced by the authors are selected and illustrated for inward and outward tied-arch bridges. The main aim is to apply the theoretical concepts to actual cases of existing bridges, testing the influence of the design choices on each aspect of conceptual design and evaluating the consequences on the structural behaviour.

2 Key conceptual design aspects of bowstring bridges

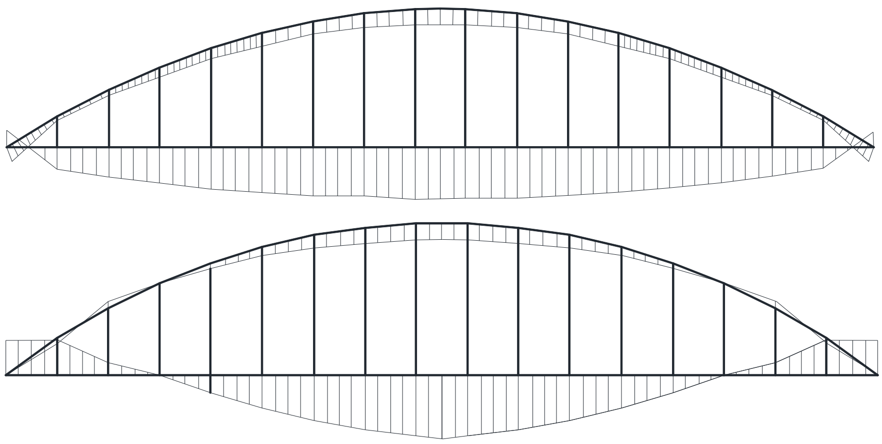
The preliminary design of a tied-arch bridge starts from definition of the correct geometry of elements together with mechanical and kinematic parameters that govern the behaviour of this complex system. A multistep design sequence has to be followed, by iteratively reviewing and refining the solution. After the dead load configuration is achieved, live loads, including traffic, have to be applied and analysed.

In this section two aspects of conceptual design of bowstring bridges are considered:

- the geometry of structural elements of the bridge and its restraints;
- the type and sequence of construction stages.

2.1 Geometry of the bridge

In many cases the geometry of the arch is chosen as the anti-funicular of dead loads acting on the bridge (self-weight and additional permanent loads due to pavement, guardrails, etc…). In this way, the arch is theoretically compressed with only axial force and no bending moments appear if the arch is considered axially rigid. The axial deformation of arch members and the deformability of the tie induce positive bending moments in the upper arch and in the lower beam, which is the effect of thrust loss. The choice of arch profile influences the bending moment diagram in the arch and in the deck (fig. 3a and fig. 3b).



a)

b)

Figure 3. Bending moments due to distributed permanent load: a) anti-funicular arch shape; b) circular shape.

With the aim of modifying the bending moment diagram due to permanent loads and of increasing positive moments in the deck midspan, it is possible to change the arch profile. Instead of plotting the anti-funicular of real loads weighing on the bridge, it is possible to fictitiously modify these loads for the sole purpose of tracing the axis curve.

If the tracing load is fictitiously increased in the central area only (fig. 4a), a negative moment is obtained in the midspan or in any case a reduction of the bending moment in the central part of the tie (unlike what would be expected in the case of a simple supported beam). The opposite occurs if the load is fictitiously increased in the two lateral end zones (fig. 4b).

To realize this effect, suppose that the tracing load of the anti-funicular shape is assumed for simplicity as a uniform load; in this case the anti-funicular shape is a 2^{nd}-order parabola (marked as a dashed line in figure 4). Assuming this shape, zero bending moments would be obtained for an axially rigid structure, if a uniform load were applied; actually the structure is axially deformable and therefore with the anti-funicular curve, due to axial deformation, positive moments arise on the arch and the lower girder.

If the tracing load is fictitiously increased in the central part, assuming the corresponding anti-funicular shape, a small negative moment in the centre and positive moments in the lateral areas would be obtained for the loads actually applied on the structure (fig. 4a). The opposite effect arises by fictitiously increasing the tracing load in the side regions (fig. 4b). Since tracing of the arch profile is carried out by iteration, it is possible to obtain the most advantageous shape and reach the desired objective by appropriately measuring out the load increase in the different areas of the structure.

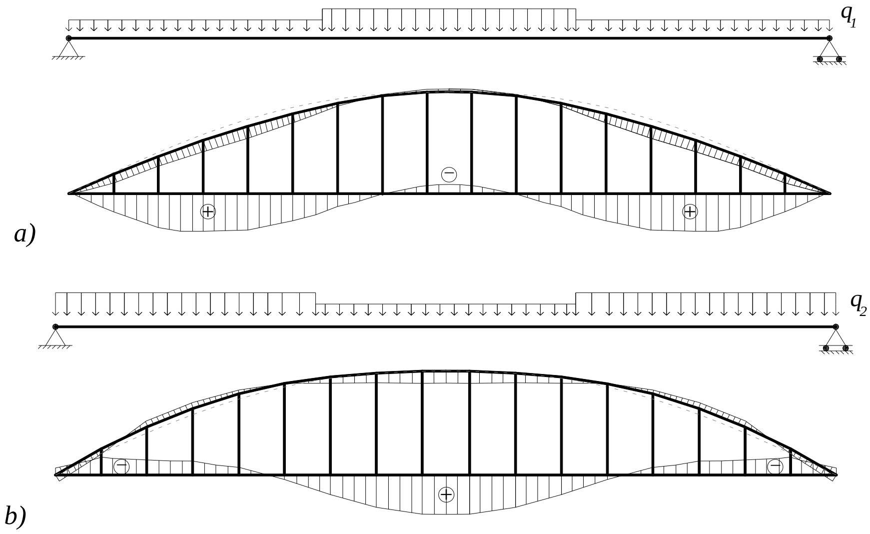


Figure 4. Bending moments due to distributed loads for two different cases. The anti-funicular curve of a uniformly distributed load on the entire span is shown with a dashed line

Hence, by modifying the arch shape (i.e. by lifting or lowering the arch profile with respect to the anti-funicular shape) it is possible to optimize the distribution of bending moments before application of live loads. Afterwards, moving loads will induce additional positive and negative bending moments in the elements. Moreover, for each load acting on the deck there is a variation of thrust and a correspondent variation of the tensile stress in the tie. If the structure was made entirely from steel this would not be significant when the arch has adequate stiffness against buckling, but the concrete slab of a composite deck can suffer significant cracking due to tensile stresses.

These stresses are produced by the following:

1. Tension from the tie-effect and due to additional permanent loads and live loads;
2. Tension in the upper fibres due to negative bending moments for the same loads;
3. Tension due to shrinkage and creep effects in the composite structure.

While the first aspect is intuitive, the second and third ones are less evident and they have to be considered with particular attention by the designer.

Let us consider the arch-beam joints and the position of the external supports. The correct geometry is generally the one in which three axes of arch, deck and external support meet in one point (fig. 5a), because in this way the joint of the steel members is clear and no eccentricities are introduced with secondary bending effects. When instead the arch and tie axes meet before the bearing axis (fig. 5b) or after it (fig. 5c) two equivalent eccentricities appear in the joint: e_1 (vertically) and e_2 (horizontally).

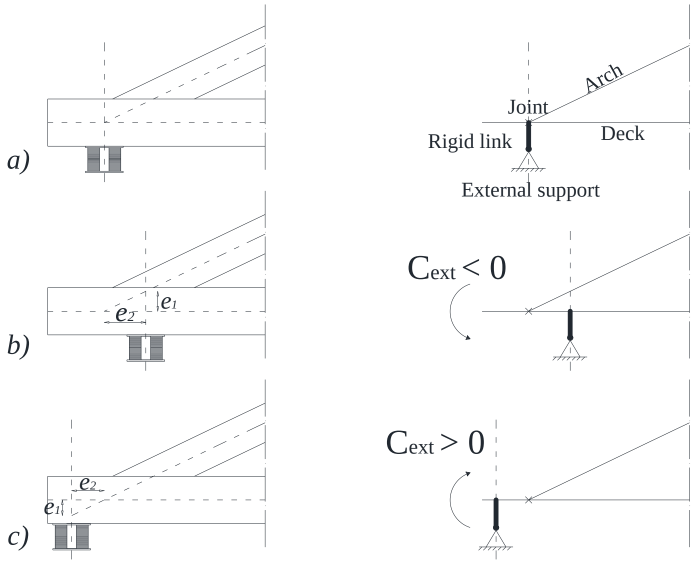


Figure. 5. Geometry of the arch-tie joint and F.E.M. modelling: a) the member axes meet in one point; b) the member axes meet before bearing; c) the member axes meet after bearing.

These modifications correspond to bending moments applied externally to the arch-tie system and to additional bending moments induced in the arch and in the tie, due only to the geometric configuration of this joint, associated with a small length of the tie subjected to constant shear forces. It could appear to the designer like a secondary effect but in some cases the consequences can be important. It is worth noting that this effect is significant for loads applied after hardening of the slab concrete but some level of eccentricity for composite decks is inevitable; in fact, when the principal longitudinal beam has a composite cross-section, the position of the centroid changes during construction and service life: at the beginning there is only the steel beam without slab, then the slab is cast on the beam and the centroid goes up, and afterwards the centroid goes down again due to time-dependent phenomena (creep and shrinkage). For this reason, the design choice can be establishing the joint configuration of fig. 5a for service life by considering time-dependent phenomena (shrinkage and creep at $t \rightarrow \infty$), avoiding secondary bending effects at the joint in the definitive configuration of service life.

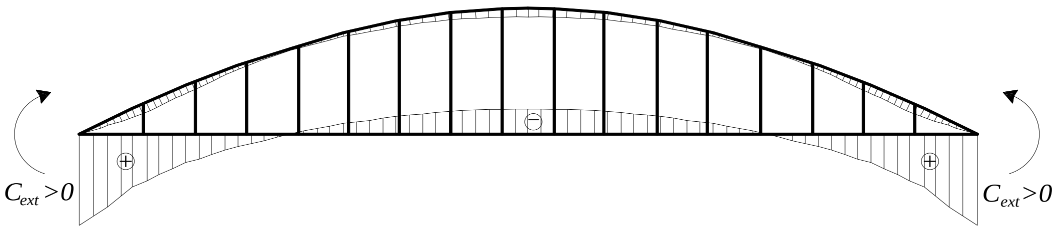


Figure 6. Bending moment due to a positive external bending moment applied to the tied arch.

The bending moment C_{ext} applied to the system induces bending moments in the two members with the same sign, but the distribution of these moments is not constant along the arch and the tie beam.

Figure 6 shows the bending moment diagram caused in the bridge members by a positive bending moment applied externally to the system, whose values depend on the flexural stiffness ratio between inclined arch section and tie-beam. The bending moment diagram is affected by the arch thrust H_c since the total bending moment $M(x)$ of the tie-arch system is:

$$M(x) = C_{ext} - H_c \cdot y(x) = M_{\mathrm{arch}}\,(x) + M_{\mathrm{tie}}\,(x) \qquad (1)$$

The effect of eccentricities could be significant, supplying a negative bending moment in the side areas of the structure (figs. 5 and 6).

The result of negative bending moments in the tie beam is the tensile stresses in the slab added to the one induced by the tensile axial force of the tie effect. By contrast, positive moments give compressive stresses in the concrete slab with a beneficial effect. For this reason in this kind of bridges, where the lower beam is composite, the tendency of conceptual design is to maintain high values of positive bending moments in order to partially compensate the tensile stresses of the tie effect with the compressive one induced in the slab by the moments. This is especially relevant in cases where the concrete has hardened prior to releasing the dead loads, i.e. constructed on falsework. The values of stresses due to permanent loads have to be added to those induced by moving loads (fig. 7), shrinkage and creep in the composite structure. It is worth noting that the tensile stresses in the upper concrete slab due to the negative moments for moving loads are temporary, differently than the ones due to permanent loads. The same is valid for the tension arising in the concrete slab through the local effect of axle traffic loads; the reinforcement located in the slab through the local effect resists temporary tensile stresses, while the reinforcement computed for the global effect in the composite section resists permanent stresses.

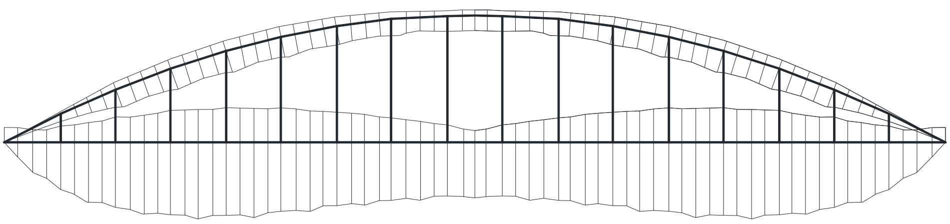


Figure 7. Maximum and minimum bending diagram due to moving loads

The influence of shrinkage and creep in the composite deck has two consequences: one affects the global behaviour and is linked to the hyperstatic structure (restrained deformations due to shrinkage and creep) and the second is linked to the so-called “creep transformed section”, i.e. to the change of the geometric characteristics of the homogenised steel-concrete section, whose principal effect is the change of the centroid position with increased tension in the bottom steel flange and increased compression in the upper one (in the case of a positive bending moment). For this reason, it is important to estimate the actual values of stresses in the flanges together with those in the concrete slab, avoiding any danger of instability of flange and slab cracking. This could be especially important in the case of principal beams with reduced inertia, when steel sections have been optimized; it is less important when the bridge has been built without temporary supports and the entire construction loads are faced by the steel sections; in the latter case the steel section is strong enough to face the effects of shrinkage and creep. A useful tool for the evaluation of stresses in the conceptual design phases can be the Busemann's creep fibre method (Rüsch et al., 2012; Shengping Zhang et al., 2019). The effect of shrinkage on the hyperstatic structure will be treated in section 3.

2.2 Construction stages of the bridge

About construction stages two cases are considered:

1. The construction of the bridge is made without intermediate temporary supports, and hence the steel structure alone resists the load of concrete casting (fig. 8a).
2. The construction of the bridge is made with intermediate temporary supports, and hence the steel structure is helped in facing the weight of the concrete slab until concrete is hardened and the composite section is completed (fig. 8b and 8c).

The choice between these two alternative methods is an important aspect of conceptual design because of the consequences in the stress state at the end of construction.

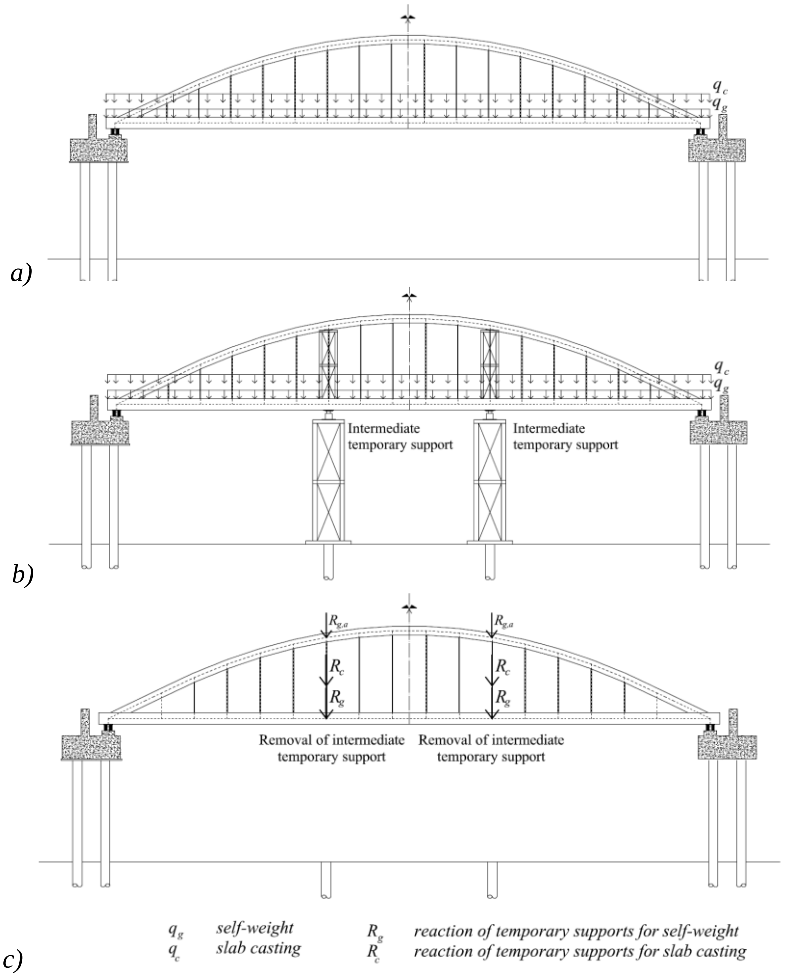


Figure 8. Type of construction stages: a) without temporary supports; b) with two temporary supports

(self-weight and slab casting) c) with two temporary supports (removal temporary supports).

The 2nd construction sequence implies the cost of temporary supports and their foundations; it can seem more complicated than the 1st method but it has the advantage of designing steel structures only for service life, reducing the height of the tie beam and the amount of steel in the arch and the beam because the composite structure is activated when permanent loads act with the total span free, after temporary support removal. On the other hand, this construction implies the presence of equivalent concentrated forces at support removal, moving away from the hypothesis of distributed loads generally considered for the construction of the anti-funicular arch shape. Moreover the 2nd construction solution gives negative bending moments to be evaluated (fig. 9), even though they are comparatively smaller because they only depend on the self-weight of the steel members and concrete slab on reduced spans.

In contrast, the 1st construction solution is more adherent to the distribution of loads considered in the design of the arch shape; positive bending moments are important for the dead load but the steel structure has to resist the entire stress state due to the slab casting.

From the sequence shown in fig. 9 in the case of intermediate temporary supports, considering the corresponding values of axial forces in the arch and tie, it is worth noting that the stage in fig. 9b gives positive moments in the central part of the span and negative moments near the ends when the concrete slab is already cast, due to the self-weight of the bridge. This can be important in the superposition of the stresses with the effects of negative bending moments in the central part of the span due to shrinkage (see section 3); hence a good conceptual approach is maintaining a high value of positive bending moments for permanent loads. Other evaluations can be made using these two different construction sequences for bowstring bridges accounting for the stressing sequence of the hangers, which is often an essential item in the choice of the construction method (Hayashi et al., 2010; Granata et al., 2013b).

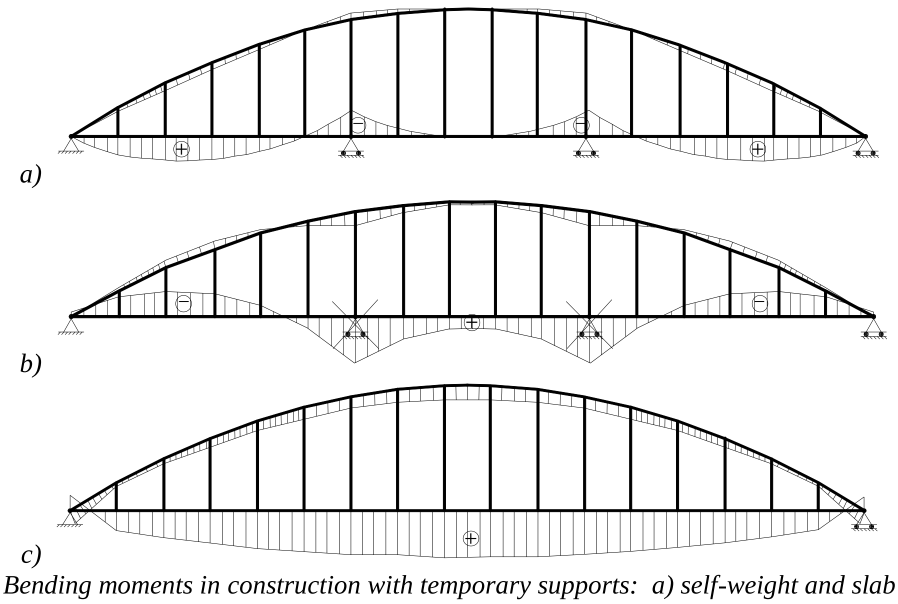


Figure 9. Bending moments in construction with temporary supports: a) self-weight and slab casting; b) removal of temporary supports; c) additional permanent loads.

Summarizing, some actions to modify the diagrams of bending moment and to reduce the negative moments on the composite beam have been discussed in order to contain the tensile stresses on the concrete slab. The optimal combination of the actions shown in Table 1, associated with the effects of shrinkage, explained in the next section, can lead to optimization choices of conceptual design in order to avoid excessive tension on the slab.

Action	*Type of action*	*Bending moment*		*Tensile force in the tie*
		Central area	*Side areas*	
1	Increase the tracing load of the anti-funicular curve in the central area	Negative moment	Positive moment	Increase
	Increase the tracing load of the anti-funicular curve in the side areas	Positive moment	Negative moment	Decrease
2	$C_{ext} > 0$	Negative moment	Positive moment	Increase
	$C_{ext} < 0$	Positive moment	Negative moment	Decrease
3	Removal of temporary supports after slab casting	Negative moment	Positive moment	-

Table 1. Summary of actions proposed for the conceptual design

Table 2 summarizes the effects of the shape modifications on moving loads.

Action	*Type of action*	*Central area*		*Side areas*	
		Positive moment	Negative moment	Positive moment	Negative moment
1	Increase the tracing load of the anti-funicular curve in the central area	Decrease	Increase	Increase	Decrease
	Increase the tracing load of the anti-funicular curve in the side areas	Increase	Decrease	Decrease	Increase

Table 2. Summary of effects of shape modification on bending moments due to moving loads

3 Slab concrete cracking in composite deck

The stress state at the end of construction due to permanent loads is affected by shrinkage and creep in the concrete slab. We will focus attention particularly on the effects of shrinkage in the upper slab.

By adopting the approximate method of Mörsch, the concrete slab is theoretically separated from the steel beam and fixed at the ends, evaluating the tension due to the shrinkage deformation (1st scheme, fig. 10a), and afterwards, the reaction at the ends is applied, for the equilibrium, to the composite beam (2nd scheme, fig. 10a) with the opposite sign, resulting in an eccentric compressive force; the superposition of these two effects supplies the distribution of stresses in the composite beam due to shrinkage deformation opposed by the steel connectors.

It is necessary to distinguish the cases of the isostatic structure and the redundant one. For an isostatic structure, the 2nd Mörsch scheme (eccentric compressive force) is always a beneficial effect on the composite beam. By contrast, in hyperstatic structures (like bowstring bridges) the eccentric force induces a compressive axial force F_{SHR} together with a bending moment C_{SHR} (fig. 10b) and consequently a bending moment diagram ($M_{Deck,SHR}$) which is partially positive and negative along the beam (fig. 10c).

This method can give useful indications on the effects induced on the combined arch-tie system, even though it is approximate because it does not consider the compatibility between the bottom concrete slab fibre and the upper steel flange, only respecting the equilibrium conditions, in the hypothesis of no slip between concrete and steel. The compressive force of the second Mörsch scheme is applied in this case to the combined arch-tie system: it is an eccentric force, and hence it leads to a beneficial compressive axial force in the tie and to an external positive bending moment. The result is the superposition of stresses due to tension in the slab for the 1st Mörsch scheme, compressive axial force for the 2nd Mörsch scheme, and bending moment distribution on the tie beam for the 2nd Mörsch scheme. The latter stresses are not of the same sign along the bridge because of the internal indeterminacy of the arch-hanger-tie system.

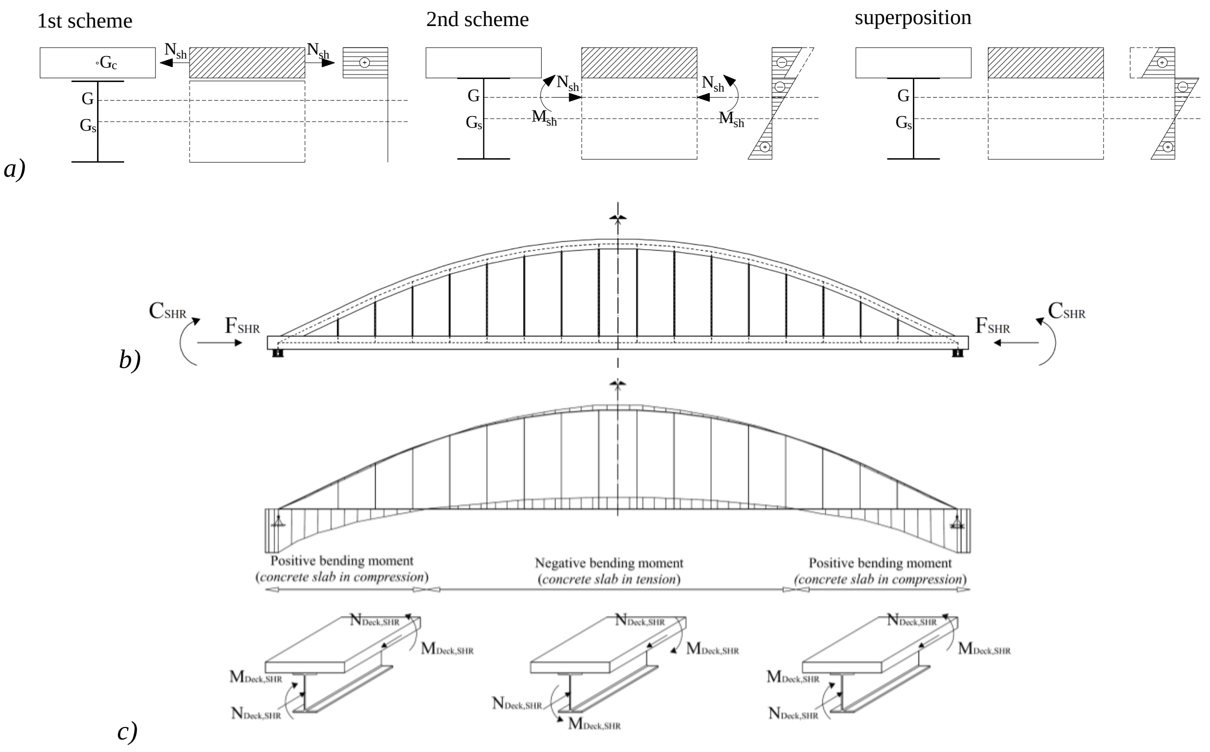


Figure 10. Mörsch method for evaluation of the shrinkage effect on the composite beam: a) 1st, 2nd scheme and superposition of stresses; b) external forces and moments applied to the system; c) bending moment diagram

The significant values of shrinkage deformations given by the Eurocode (CEN, 2005a; CEN, 2005b) and by North American codes (ACI, 1997; ACI, 2008) in geographic areas with dry and hot climates makes this effect very important, especially for medium-span bridges and low values of relative humidity (RH%), increasing the tensile stress in the slab with high likelihood of concrete cracking, resulting in significant reinforcement requirements to maintain an acceptable value of crack width in service life (Granata et al., 2013c) in order to avoid durability problems on the deck. The load

combination of permanent loads + shrinkage + moving loads can lead to large tensile forces in the slab, especially for the maximum values of negative bending moments due to moving loads.
Considering the stresses due to permanent and moving loads and the effects of the shrinkage deformations, that which are linked to the geometry of the bridge and to the type of construction stages, in order to reduce the tensile stresses and the likelihood and magnitude of concrete cracking in the upper slab of composite deck, it is important to check the values of stresses in the following sections (fig. 11):

- Section 1 near the external support, where negative moments could be due to the arch shape and to the geometry of the arch-tie joints.
- Section 2 where it is possible to find the maximum value of negative bending moment due to moving loads whose position and value are linked to the arch shape.
- Section 3 in the midspan with negative moment due to the 2^{nd} scheme of the Mörsch method in the evaluation of shrinkage effects for tied arches.

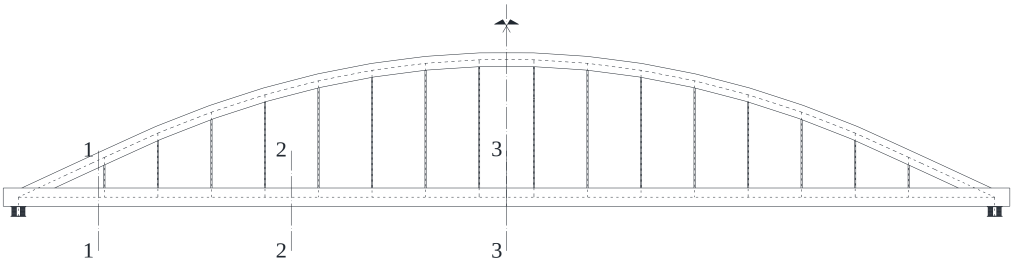


Figure 11. Sections to check for a possible high value of tensile stress in the upper concrete slab

Summarizing the aspects analysed in the previous sections, in the conceptual approach to the structural design of the bridge a few simple suggestions could be useful for optimizing bridge behaviour, focusing on the problem of the slab cracking. These suggestions are the following:

1. Adopting an arch shape close to the anti-funicular one or modified with slight adjustments; this gives an increased positive bending moment in the central or side parts of the bridge (action 1 of table 1) and an associated variation of tensile force in the tie. In this way it is also possible to reduce and to shift in a convenient way the value and the position of the minimum negative moments due to moving loads. The values of bending moments due to dead loads after construction is in fact the basic value of the stress state associated with the value of arch thrust and tie tensile stresses. The combination of this stress state with those due to the loads applied in the successive stages and to shrinkage effects supplies the final state of the steel elements and concrete slab.
2. Introducing appropriate eccentricity at the arch-tie joints (action 2 of table 1) which is in opposition to the effect of the 2^{nd} Mörsch scheme (negative external moment, shifting the bearing support towards the central span): in this way the bending moment diagrams are in opposition with each other and the effect of shrinkage is partially compensated, especially in

the central part of the span, considering the associated variation of tie force. Alternatively, it could be convenient to have perfect coincidence of the arch and beam axes with the support one, avoiding the external bending moments associated with eccentricities, especially when shrinkage does not cause problems for the slab.

3. Adopting the most appropriate construction sequence (action 3 of table 1): it appears that the method with intermediate supports could be convenient for the state of concrete slab stresses; moving the position of the intermediate temporary supports it is possible to modify in a convenient way the final bending moments due to self-weight and permanent loads. The beneficial effect is given only if the slab casting is made with temporary supports.
4. In geographic areas with dry climates it would be appropriate to reduce shrinkage deformations with additives in the concrete mixtures, adopting pre-cast slabs with minor parts of sections made of cast-in-situ concrete or adopting an appropriate cast sequence. This decreases the importance of shrinkage effects in the bridge behaviour, avoiding high tensile stresses in the 1st and 2nd Mörsch schemes. In addition to the above choices, when the concrete slab is too sensitive to time-dependent phenomena it could be appropriate to have principal longitudinal tie beams entirely made of steel and the concrete slab only on the transverse beams, i.e. to avoid the composite section for the principal beams (i.e. only the transverse beams act compositely with the concrete slab).

All these evaluations are made on medium span bridges and they have to be verified in each actual case. The objective of this paper is to focus on the main parameters that can influence the behaviour of the bridge and to illustrate how it is possible to improve and optimize it, by applying a few simple suggestions to avoid unexpected distributions of stresses, especially in the concrete slab for serviceability.

An uncontrolled combination of the effects of the parameters described above can lead to unexpected inappropriate structural behaviours; for this reason a conceptual approach in which choices are carefully made can give the structural designer useful suggestions for optimizing the global behaviour of the bridge.

4 Case study of a bridge with outward arches

A first case-study is shown on a bridge with outward arches, explaining the above evaluations of conceptual design aspects, applied to the actual case of a bridge over the San Leonardo river, in Southern Italy. The authors verified the design of this bridge under assignment of the Public Administration.

The bridge is made of two twin lateral steel arches with a slight outward inclination of 5°, span length 48 m, arch rise 6.10 m (f/l = 0.127); the deck is made of two longitudinal tie beams (double T, 1 m high) with a concrete slab 0.27 m thick, transverse beams at any hanger and spacing of 2.82 m between hangers, each one made of a flat steel plate 300x400 mm. The arch section is a hollow box 600x600 mm. Figure 12 shows the geometry of the bridge and the finite element model used for the design.

The tensile yield strength of the steel is f_{yk} = 355 MPa and the concrete strength is f_{ck} = 40 MPa. The elastic modulus of the concrete E_c = 30582 MPa and the elastic modulus of steel E_s = 210000 MPa. The relative humidity RH = 60%, the conventional thickness $h_0 = 2A_c/u$ = 380 mm and the total value of shrinkage deformation at $t \rightarrow \infty$ is ε_{cs} = 0.00029, following the EC2 model (CEN, 2005a).

For the driveway, the additional dead load G_2 = 2.40 kN/m^2. The weight of the barriers is 5.73 kN/m. For the sidewalks, the additional dead load G_2 = 7.03 kN/m along the beam and G_2 = 15.53 kN at the end of the beam (railing). The moving loads Q_{ML} are evaluated according to Eurocode (CEN, 2005b).

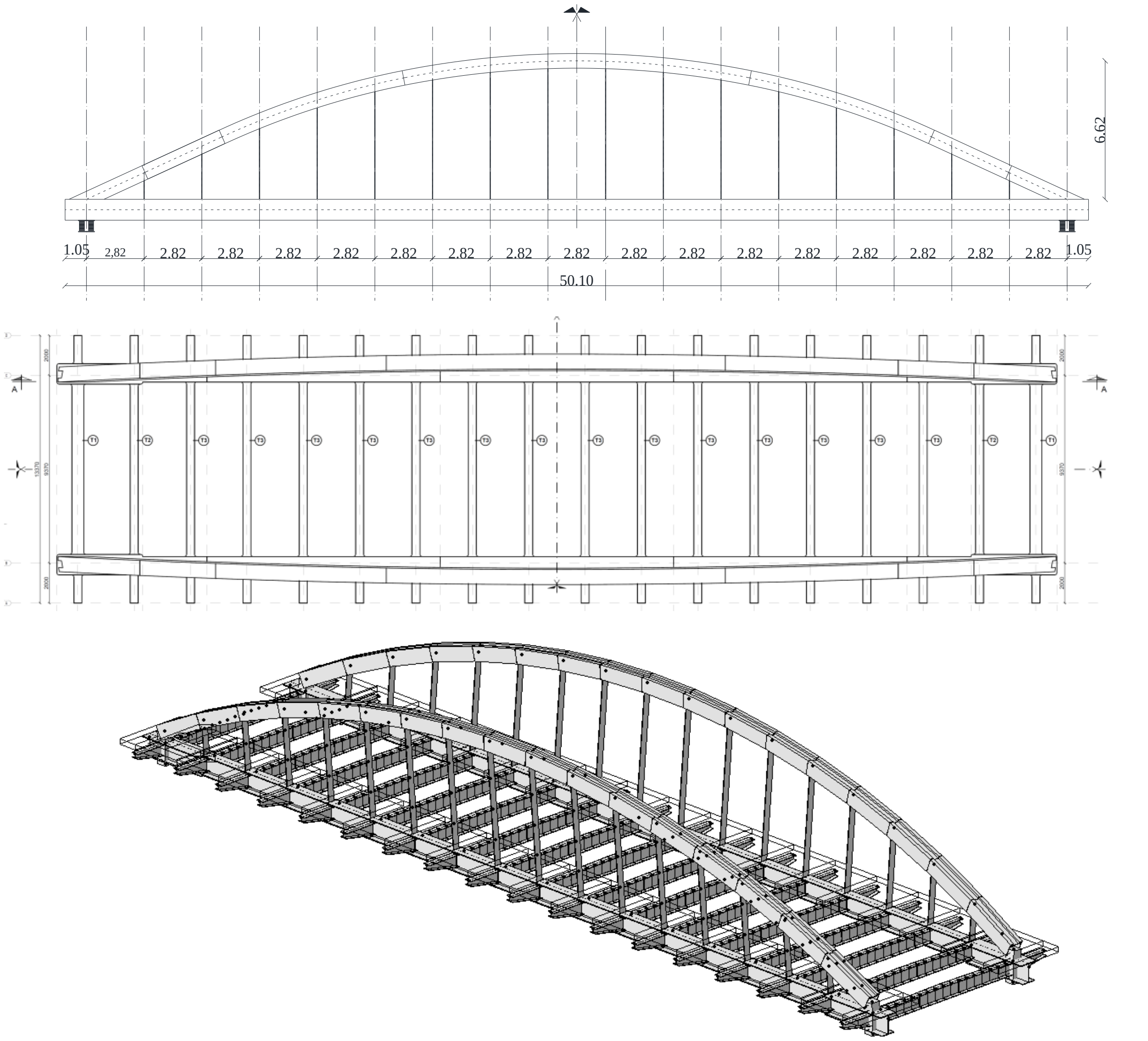


Figure 12. Geometry and F.E.M. model of the first case-study bridge with outward arches

The construction was made without intermediate supports, obtaining positive bending moments for additional permanent loads only, in the basic configuration at the end of construction. Afterwards the effects of shrinkage modify the distribution of stresses.

In the initial design the aspects analysed in the previous sections were not considered and no conceptual approach was applied to the design of the bridge, hence the project showed the following:

- the original arch shape was circular, varying significantly from the anti-funicular shape, with negative bending moments for self-weight and permanent loads (case of figure 3b);
- a high eccentricity value at the ends induced a high negative bending moment at the ends (case of figure 5b);
- there were high values of shrinkage deformations with unacceptable resultant tensile stresses in the concrete slab, especially at the ends of the deck, and an excessive amount of reinforcement bars to be placed in the cast-in situ slab;
- there were high values of negative bending moments due to moving loads.

All these elements lead to an unfavourable state of stresses of the slab for serviceability and to an unsuitable global behaviour of the bridge. The value of the shrinkage deformation is significant, inducing tensile stresses in the concrete slab of about 3.8 MPa for the 1st Mörsch scheme, which reduces to 2.5 ÷ 3 MPa applying the compressive force of the 2nd scheme. This value is added to the stresses due to permanent loads in the basic configuration of the bridge and to the effects of service loads (moving loads).

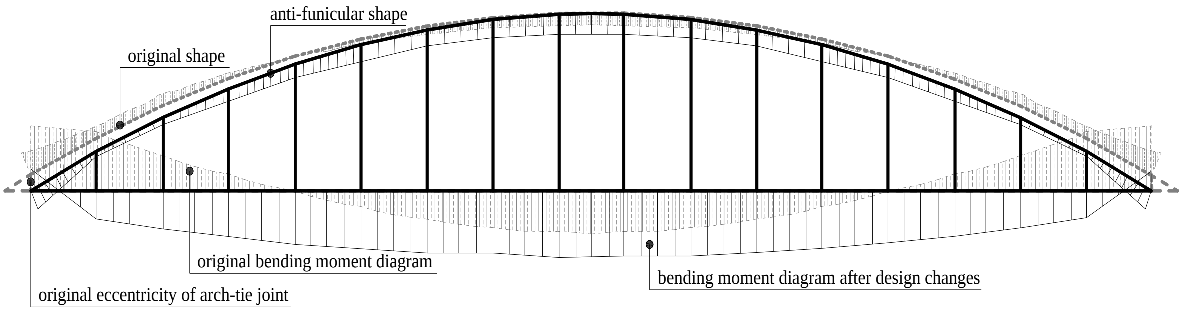


Figure 13. Comparison between the original and the modified solutions. Superposition of arch shape and bending moment diagrams for dead loads

In order to improve this behaviour the anti-funicular shape was adopted and the eccentricity at the ends was diminished, maintaining a low value of negative external bending moment to compensate for the shrinkage effect. Figure 13 shows the superposition of the arch before and after the design modifications together with the bending moment diagrams for dead loads. It is evident that the original shape of the arch was near the circular one and raised with respect to the anti-funicular curve (almost parabolic) and that the negative bending moments in the side areas were too high. The total value of shrinkage deformation was reduced by using additives in the mixture and a segmented cast sequence of the slab was adopted. It was also useful to increase the thickness of the precast slabs, decreasing the volume of cast-in situ concrete.

	INITIAL DESIGN (X_i)	CONCEPTUAL DESIGN (X_f)	$(X_f - X_i)/ X_i$
Tensile stress σ_t at the ends of the bridge (section 1)	5.0 MPa (1.43 f_{ctm})	2.0 MPa (0.57 f_{ctm})	- 60 %
Tensile stress σ_t at the midspan of the bridge (section 3)	4.5 MPa (1.29 f_{ctm})	3.0 MPa (0.86 f_{ctm})	- 33 %
The stresses are evaluated for the serviceability combination: $G_1 + G_2 + \psi_0 \cdot Q_{ML}$			

Table 3. Tensile stresses in the initial design and conceptual design

By applying these suggestions, the tensile stress was reduced with respect to the initial design of the bridge: from about 5.0 MPa at the ends of the bridge to 2.0 MPa and from 4.5 MPa to 3.0 MPa at the midspan (Table 3). In the same way the total amount of reinforcement was reduced, bringing the maximum value of tensile stress under the value of first cracking f_{ctm} = 3.50 MPa, for all load combinations in service life. It is worth noting that the original design showed values of tensile stress above the f_{ctm} value even for permanent loads, in the basic configuration without moving loads. This situation is unacceptable for serviceability and durability.

In this case the global effect on tensile stresses (due to dead loads) is comparable to the local one (due to axle live loads). In the original design the tension induced by the tie and the shrinkage on the slab doubled the reinforcement requirement in areas with negative bending moment; afterwards, with the actions applied for modifying the global behaviour, the reinforcement requirement decreased significantly and proportionally to the tensile stress shown in table 3. As mentioned above, the reinforcement evaluated for the global effect is always designed to resist permanent tension while the one evaluated for the local effect is designed to resist transient tension due to the moving load and therefore to a limitation of the temporary opening of the crack; this is a fundamental difference in the scale of importance of the consequences on the structural behaviour.

5 Case study of a bridge with inward arches

Another case-study is presented to show a different situation of an existing arch bridge with inward arches placed in Southern Italy, over the mouth of the river Arena in Mazara del Vallo. The authors were appointed to retrofit the bridge, which presents a high level of corrosion of the principal elements.

There are two circular arches inclined inwardly at 18° with a 700x900 mm steel box cross-section. The two arches are stiffened by 11 transverse beams. The bridge has longitudinal and transverse symmetry with 27 hangers at each bridge side for a total number of 54 hangers made of circular steel bars, 70 mm in diameter. The deck is composed of two longitudinal side beams, with double T section, 900 mm wide and 1750 mm high, emerging from the platform and inclined 18°, lying in the same plane as the arches and hangers. The longitudinal beams are stiffened by transverse double T beams 800 mm high with L diagonal bracing and an upper reinforced concrete slab 250 mm thick. The upper slab is connected only to the transverse beams through Nelson studs, but not to the longitudinal ones, which are almost independent and linked only to the steel elements of arch, hangers and deck. The deck supports a single carriageway and two side footways. The total length of the bridge is 87 m, and the span between the supports is 85.4 m. The arch rise is 16.5 m (f/l = 0.193) and the hanger spacing is 3.05 m. Each abutment is founded on 8 piles with 1.20 m of diameter. The geometry is shown in figure 14.

The tensile yield strength of the steel is f_{yk} = 355 MPa and the concrete strength is f_{ck} = 35 MPa. The elastic modulus of the concrete E_c = 30500 MPa and the elastic modulus of steel E_s = 210000 MPa. The relative humidity RH = 60%, the conventional thickness $h_0 = 2A_c/u$ = 290 mm (transverse beams) and the total value of shrinkage deformation at $t \rightarrow \infty$ is ε_{cs} = 0.00025, following the EC2 model (CEN, 2005a).

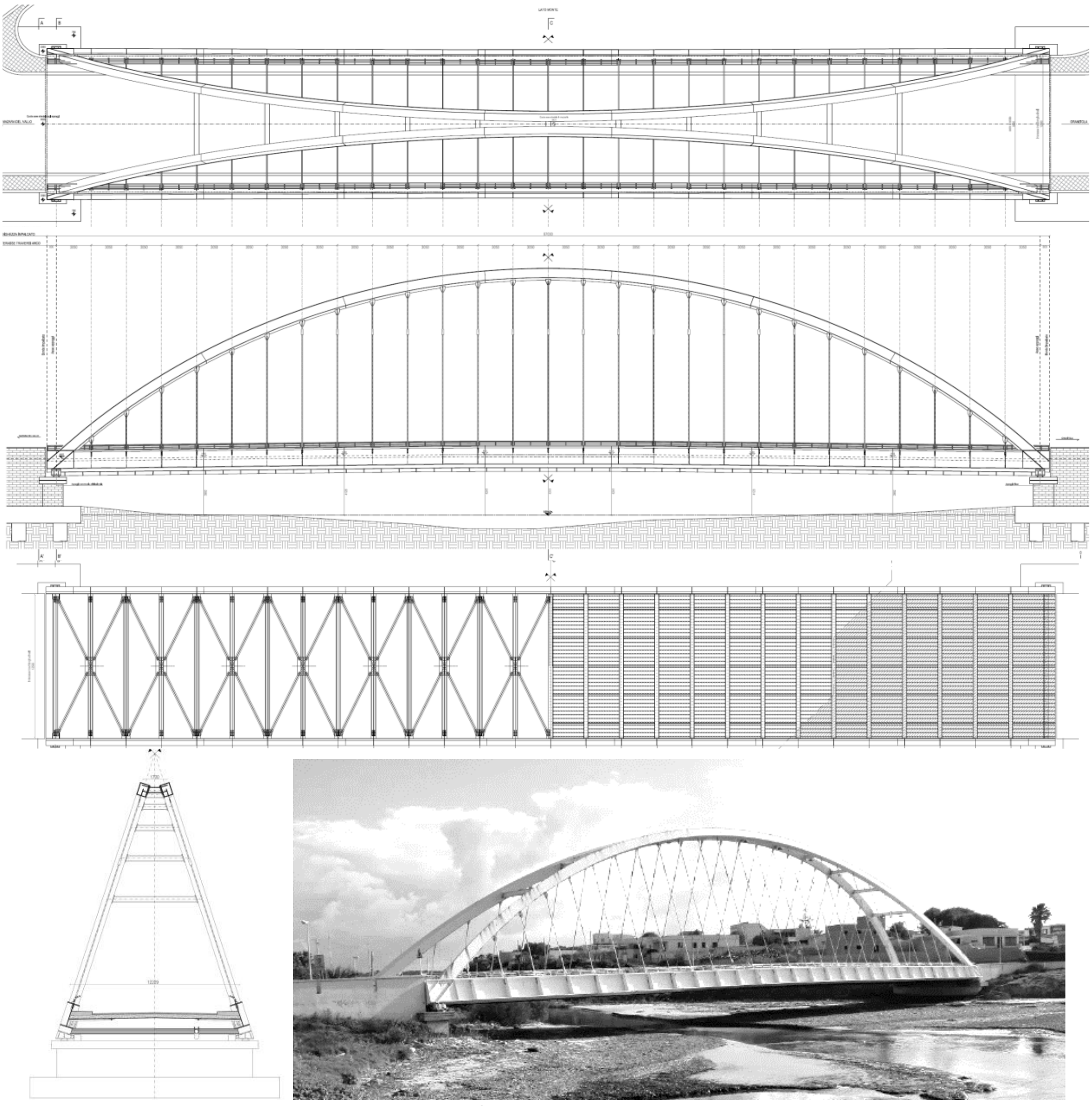


Figure 14. Geometry and picture of the second case-study bridge with inward arches

The characteristics of the bridge design are the following ones:

- the supports are perfectly aligned with the arch and beam deck (fig. 5a) and no eccentricities are present at the ends, avoiding the external moments of fig. 6;
- the arch shape is circular, leading to the situation of fig. 3b and 4b, with negative bending moments for dead loads for about 20 m from the ends and positive bending moments only in the central part of about 45 m;
- the longitudinal beam emerges from the deck slab and the latter is only connected to the transverse beams;
- the hangers are rigid bars and no pre-tension is given to them because in the original configuration the designer decided to connect them to the arch and beam through welded rigid connections: for this reason and due to the inward inclination, the hangers have a degree of bending associated with axial force which is not negligible;
- the construction was made with a central temporary pier but this was only used for assembling steel members till arch closure at the key, and was removed before slab casting; hence the steel structure supported the entire slab weight and the composite structure is only effective for additional permanent loads and live loads;
- the slab is composed of prefabricated elements 50 mm thick, with additional reinforcement and in-situ cast for the remaining 200 mm above, connected to the transverse beams: longitudinal and transverse reinforcement is well spaced and the amount is ϕ16/150 or ϕ16/200 mm, with increased diameter at the end for about 3 m, due to the presence of bridge joints.

The bending moment diagrams for dead loads and for the combination with moving loads are shown in figure 15. It is worth noting that the circular arch induces negative bending moments in the side areas of the deck and positive moments in the midspan. This is also reflected in the diagram of the maximum and minimum bending moments for moving loads, with significant minimum values at the ends of the tie beam.

Although the anti-funicular shape was not chosen by the designer and the construction made without temporary piers during the slab casting, the principal longitudinal beam was not connected to the concrete slab, and hence the negative bending moments do not significantly affect the slab but only the steel beam, which is strong enough. This reduces the effect of shrinkage too, associated with the partially precast elements, because the restrained shrinkage deformation is in the transverse direction, with a reduced secondary effect in the longitudinal direction. These aspects lead to a maximum tensile stress in the deck slab of 3.2 MPa, for all service loads, always maintaining it under the limit of first cracking with a medium value of 1.9 MPa.

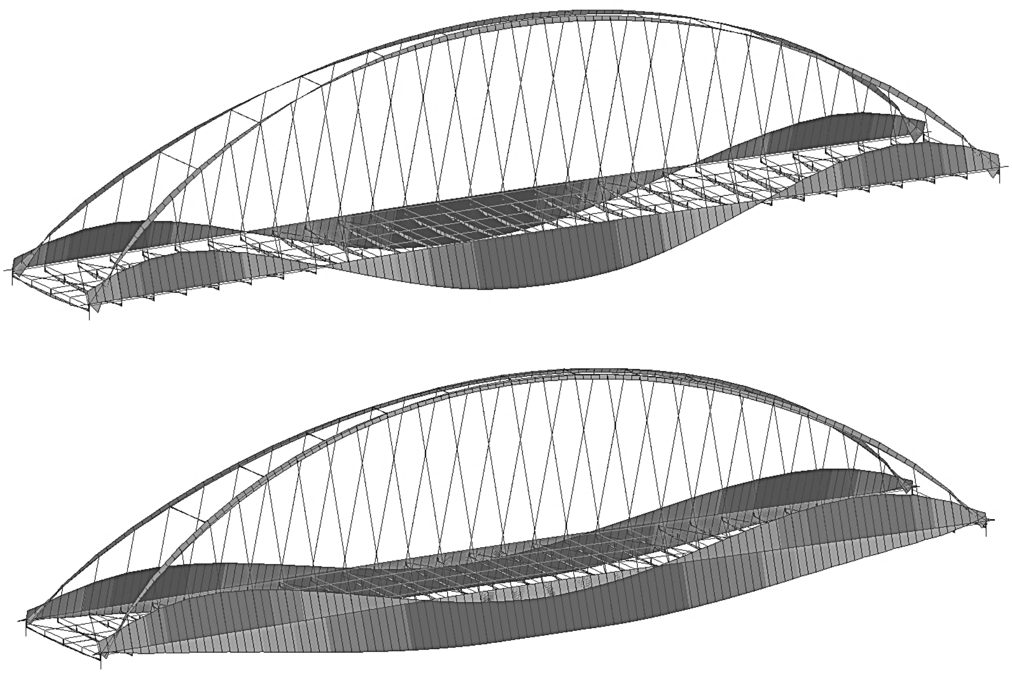


Figure 15. Bending moment diagrams for dead load and for the combination with moving loads (envelope)

A different choice for the arch shape (dead load anti-funicular one) and for the construction stages (slab casting with temporary piers) would supply more appropriate behaviour with reduced steel sections, saving steel quantities and total weight, maintaining the effectiveness already achieved for the concrete slab behaviour. Moreover, together with the anti-funicular shape, pre-tensioned bars would modify the dead load bending moment diagram appropriately during construction, enhancing the deformed configuration of the bridge for dead loads which is actually not so good due to a badly defined deck camber and to the rigid connections of hangers welded to the arch and longitudinal deck beams.

6 Conclusions

Aspects regarding the conceptual design of tied-arch bridges with steel-concrete composite decks have been analysed and evaluated, focusing on the geometry of the bridge and on the construction stages. The consequences of design choices could lead to an unfavourable state of stress, especially for the concrete deck slab, with a likelihood of significant cracking. The conceptual design approach of this type of bridges was discussed, giving a few practical suggestions. It is possible to summarize as follows:

1. The arch shape modifies the bending moment distribution in the arch and deck: by adopting the anti-funicular shape with a few adjustments it is possible to obtain positive bending

moments in areas where the upper concrete slab is in tension for service loads. It is also possible to decrease the maximum values of negative bending moments due to moving loads. Unfortunately, in the two case-studies examined the shape was circular, denoting the tendency of designers to disregard the importance of an appropriate arch shape, prioritizing the ease of construction at the expense of structural behaviour and denoting a serious error of conceptual design.

2. The arch-tie joints and the position of the external supports can introduce an external moment to the arch-tie system with additional bending moments in the arch and in the tie, due only to the geometric configuration of this joint. The effect of this eccentricity has to be evaluated with particular attention because in many cases it is not a secondary effect on the bridge behaviour. In some cases it could be conveniently used for counterbalancing the effect of shrinkage in terms of bending moment along the deck.
3. The construction sequence can affect the final value of the stress state in the composite beam; hence it is necessary to evaluate the most convenient construction sequence together with the concrete casting sequence. Intermediate temporary supports and segmented concrete casting can be useful to optimize the state of stress at the end of construction.
4. When the shrinkage deformation value is significant, it is appropriate to reduce the total shrinkage deformation through additives in the concrete mixtures. It can also be useful to reduce the volume of cast-in-situ concrete by adopting prefabricated elements. In fact the total shrinkage deformation supplied by the codes is significant, especially for low values of relative humidity (dry climate) and this point can become a fundamental issue for the tensile stress of concrete slabs. Hence, in some cases, it can be useful to eliminate the cooperation of steel and concrete in the deck longitudinal principal beams, adopting emerging principal beams and maintaining the composite section only in the transverse direction avoiding the tension due to the tie-effect in the concrete slab.

By considering the above suggestions and combining them in the most convenient way, a multi-step procedure can be implemented in the design with successive refinements in order to optimize the structural behaviour in a global view of conceptual design. A casual or uncontrolled superposition of stress states related to the items considered above and due to bad design choices can lead to inappropriate behaviours of the bridge, a high amount of reinforcements, slab cracking, overstressed elements and reduced performance in terms of durability and serviceability. By composing the effects of each parameter in an appropriate way it would be possible instead to achieve a good result for any actual case of a bowstring bridge in engineering practice, avoiding unexpected states of stress and deformation in the service configuration.

In this paper only the tied arch has been considered, with the focus on rigid hangers for which no pre-tension is given in the construction stages. For other cases with cable hangers, pre-tension is always an efficient way of controlling the stressed state, even in the concrete slab, together with a correct setting of the pre-camber. In other cases it could be appropriate to consider the alternative of a network arch arrangement because it generally allows a significant reduction of bending moments in the lower beam due to its structural efficiency and stiffness.

The application of these evaluations on two actual case-studies of bowstring bridges with inward and outward arches was presented in order to show the consequences of each aspect of the conceptual approach discussed here.

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Notation

A_c	area of concrete cross section
C_{ext}	moment externally applied to the structure
C_{SHR}	moment externally applied to the structure due to shrinkage
e_1,e_2	eccentricities of arch joint
f_{ctm}	maximum concrete tensile stress at first cracking
F_{SHR}	axial force externally applied to the structure due to shrinkage
H_c	arch thrust due to the moment C
h_0	conventional thickness of concrete section for shrinkage
$M(x)$	moment in the combined arch-tie structure
M_{arch}	bending moment in the arch
$M_{deck,SHR}$	bending moment in the deck due to shrinkage
M_{sh}	moment due to shrinkage
M_{tie}	bending moment in the tie
N_{arch}	arch compressive force
N_{sh}	axial force due to shrinkage
q	external distributed load
q_c	dead load due to slab casting
q_g	self-weight
R_b	bearing reaction
R_c	reaction due to slab casting
R_g	reaction due to self-weight
T_{tie}	tensile force in the deck
T_{hanger}	tensile force in the hanger
x	longitudinal axis
$y(x)$	arch rise
u	external perimeter of concrete section