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CONCEPTUAL DESIGN OF BOWSTRING BRIDGES WITH STRESS RIBBON TIE

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Parole chiave: ponte, nastro teso, arco, spinta, pretensione dei cavi, tirante

Abstract. *In this paper the conceptual design of tied-arch bridges is examined. A proposal for a particular kind of bowstring bridges, with an upper deck and a stress ribbon footbridge placed between the arch footings is discussed. The double structure allows to avoid horizontal forces into foundations when functional requirements lead to differentiate pedestrian and road traffic. After a general discussion about classical tied-arches, the main parameters involved in the design of the proposed solution are discussed and a case-study is presented.*

Sommario. *In questo studio viene sviluppato il conceptual design di ponti ad arco a spinta eliminata. In particolare si propone una soluzione innovativa di ponte bowstring con arco a via superiore e tirante inferiore costituito da una passerella a nastro teso in calcestruzzo. Tale doppia struttura, che risponde alle esigenze funzionali di avere il ponte carrabile e quello pedonale separati, ha come obiettivo principale l'eliminazione di forze orizzontali in fondazione. Dopo una discussione generale sulle configurazioni classiche dei ponti ad arco a spinta eliminata, vengono illustrati ed investigati gli aspetti fondamentali per il progetto della soluzione proposta e viene presentato un caso studio.*

1 INTRODUCTION

Along bridges history, arch represented the optimal solution for its structural efficiency, because when it is designed following the anti-funicular curve of loads, transverse sections are uniformly compressed. This efficiency is the reason of using arches made of materials with good compression strength and bad tensile properties. Till 19th century arches have been built only with stones or bricks and, depending on the length to be saved, they were single or multi-span bridges. Unfortunately in arch bridges it is not possible to avoid totally bending moments, because the thrust line cannot coincide with the geometric axis for all live load combinations, due to the variability of traffic loads. To solve this problem arch cross sections have the right thickness, in order to maintain the thrust line into the central core of inertia and

to avoid tensile stresses for all load combinations. The arch behaviour is established when a significant thrust at footings appears, which implies horizontal forces into foundations. When soil is not adequate to receive these forces, it is possible to compensate them through a tie placed between arch footings; in this way only vertical reaction forces can be obtained. So the whole tied-arch structure works as a simply supported beam, in which the arch is a curved compressed member and the tie is in tension. In bridges with an upper arch, the tie can be provided by the deck itself, that is the link member between arch footings. In this case the deck is suspended to the arch by a number of metallic hangers. This is the so-called bowstring structure, used either with concrete or steel arches. In the classical solution hangers are vertical, but different arrangements can be found throughout the historical evolution of these bridges, as harp arrangements or network arches [1]. Inclined hangers appeared in XIX and XX centuries following the Nielsen solution, in which the global behaviour of the bridge is exactly the same of a truss where the upper member is curved and compressed as an arch. Even if hangers are always in tension for dead loads, the difference between vertical and inclined arrangements can be found in the behaviour for live loads. High compressive forces could appear into inclined hangers for traffic loads combinations, so they need to be pre-tensioned at higher values with respect to vertical ones. Bowstring supports are always the same of a simply supported beam; they allow vertical reactions, so abutments as well as piers are exactly the same of a supported beam. Figure 1*a* shows a classical metallic tied arch bridge while fig. 1*b* shows a network arch bridge.

a b

Figure 1. a) Steel tied arch with vertical hangers (Griffin memorial bridge).
b) Network arch (Steinkjer bridge)

In tied-arch bridges designed by Juan José Arenas in Spain (Barqueta Bridge in Sevilla and Third Millennium Bridge in Zaragoza, fig. 2) the central upper arch is divided into two arms near the deck to achieve the shape of an “entering portal” [2,3]. Arch profile follows the anti-funicular of dead loads in both cases but materials and hanger arrangements are different.

a b

Figure 2. a) Barqueta bridge in Sevilla. b) Third Millennium Bridge in Zaragoza. (J. J. Arenas de Pablo)

While the Sevilla bridge (with a span of 165 m) is a steel arch with a composite steel-concrete deck and fan arranged hangers on a central plane, the Zaragoza bridge (with a span of 216 m) is a self-compacting high-strength concrete arch with a prestressed concrete deck and two planes of parallel hangers. These two examples can appear very similar; they show instead the variability of possible solutions for tied arch bridges. In both cases the end of arches are received by vertical piers. In the metallic arch, the steel deck is itself the tie. In the concrete arch instead, the tie is formed by a number of external prestressing tendons, placed inside the concrete box deck section, which connect the arch footings.

Even if different solutions can be found to attain a tied arch, an important consideration is common to all of them: because of arch axial elastic shortening and tie deformability, a significant thrust loss have to be considered in design phases. Moreover, for concrete arches, even if the structural profile follows the anti-funicular curve of dead loads, the increasing axial shortening due to creep and shrinkage in time amplifies thrust loss, inducing bending moments into the arch. For this reason the concrete arch of fig. 2*b* has been built using the Freyssinet technique of jacking key section in order to recover the thrust loss [3].

These two examples of bridges are also useful to show how construction techniques can be very different. The Sevilla bridge has been completely assembled on a river side and then rotated to its final position. The Zaragoza bridge instead has been built by incrementally launching the deck and then by positioning on it a continuous falsework to cast the arch (fig. 3). The deck has been suspended to the arch by vertical hangers, falsework has been removed and tie cables has been tensioned.

a b

Figure 3. Construction methods. a) Barqueta bridge built on a river side. b) Third Millennium Bridge built on site with the continuous falsework

During construction stages, for all methodologies, an important operation is always that of hangers pre-tensioning. First, in order to achieve the desired arch and deck profiles, pre-cambers are given to both the elements. Then, it is necessary to obtain the desired tensile forces into hangers. When construction is made by falsework or temporary piers on which arch and deck are assembled, hanger tensile forces are obtained directly by removing temporary supports, i.e. by applying all at one time the entire dead load of the deck. Then, for vertical hangers a little pre-tensioning value is sufficient. This is not true for fan arrangements or network arches. So, a pre-tensioning sequence needs, taking into account construction and service life loads.

Another problem in concrete bridges is related to time-dependent phenomena as creep and shrinkage, because they imply a variation in time of deformed configuration and possible redistribution of internal forces between different structural elements.

Another configuration of tied arches has often been used in XIX century, by giving to the tie an opposite curvature with respect to the arch; in this case the tie is quite rigid, as in the

multiple span bridges of figure 4, with upper arches and curved tie independent from the deck, below.

The same shape can be found in Pauli and Lohse solutions, built in those years in USA and Germany and called “lenticular bridges” for their particular configuration.

a b

Figure 4. Lenticular bridges. a) Quaquaga bridge, USA. b) Royal Albert Bridge, UK (I. K. Brunel).

In practice, they can be considered as truss bridges in which deck, tie and arch are geometrically separated and with different roles in the structural behaviour. Although this typology could appear nice to the observer, it has been completely forgotten.

Nowadays tied arch bridge has been reviewed in bridges with an upper deck: in this case the thrust is conveyed from arch footings to deck by inclined struts and the upper deck can be considered the tie. This is the solution used in the Arriondas Bridge over the Sella river, designed by J. Manterola Armisén (fig. 5). It works well only if extremes are rigid and no relative deformations are allowed between foundation and deck; moreover a horizontal displacement at both ends have to be accounted in order to transmit tensile forces into the tie.

All solutions shown have the common target to avoid horizontal forces into foundations.

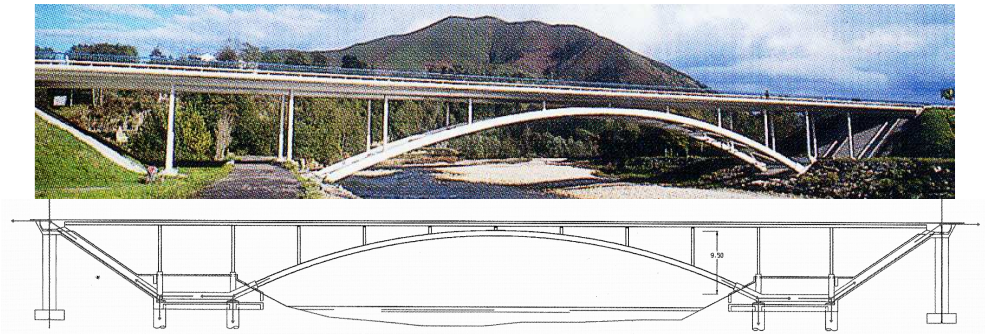

Figure 5. Arriondas Bridge over Sella river, Spain (J. Manterola Armisén, [1])

A different role can be assumed by the arch when it is used to stiffen a *stress ribbon footbridge* (called also *spannband*, with the German terminology). In this case the main structure is a footbridge with catenary shape, composed of earth-anchored cables upon which a concrete ribbon is put [5].

The deck is built by concrete elements which can be cast-in-situ or precast and simply assembled together on the bearing cables (fig. 6). The concrete deck is also prestressed by internal tendons to achieve the ribbon continuity and to prevent cracking in service life. In order to avoid excessive sag and to allow longer spans, an arch can be used as footbridge support and stiffener (fig. 7*a*). The concept is very similar to that of Arriondas bridge, in which the upper deck is now a stress ribbon footbridge (fig. 7*b*). A discussion about stress

ribbon footbridges stiffened by arches can be found in [6]. Examples of spannband footbridges can be found in references [7,8,9].

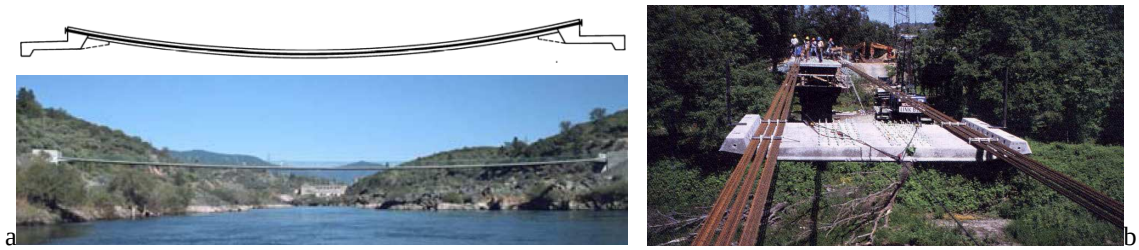
a b

Figure 6. a) Spannband scheme and Redding footbridge. b) Construction of Rogue river footbridge (J. Strasky)

On the basis of the historical development of tied arch bridges, a proposal of a different configuration is illustrated in this study. The idea is to use a stress ribbon footbridge as the tie of an arch bridge with an upper deck. It is a double bridge: a lower arch with an upper deck for road traffic and a footbridge below the arch connecting the ends.

This solution reviews the historical configuration with curved ties in a modern way (fig. 8). In this case the footbridge is covered by the arch and the two bridges maintain separate functions but they collaborate to avoid horizontal forces into foundations. The main aim is to achieve only vertical reactions at the extremal supports, as for a beam, exploiting the arch thrust for tensioning stress ribbon footbridge and maintaining this configuration since contruction till service life. In this way no anchor blocks need to compensate the high value horizontal forces into spannband, leading to a self-anchored arch-spannband structure.

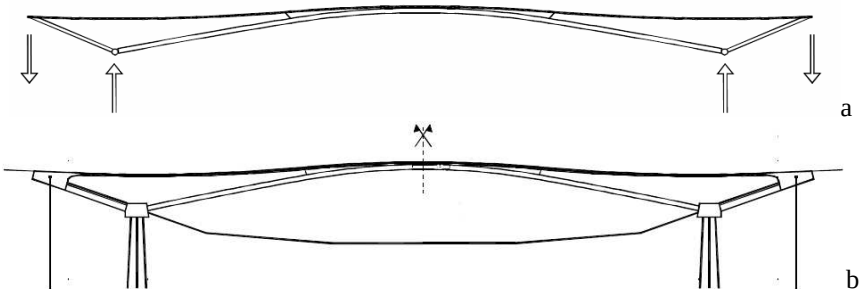

Figure 7. Stress ribbon footbridge stiffened by a lower arch (Kulhavy, [6])

The behaviour of this particular system (arch bridge and stress ribbon footbridge) is described here starting from the conceptual design, in order to give the first global characteristics in a preliminary design stage. External actions during construction and moving loads in service life are considered and their effects are discussed. Interaction between arch and stress ribbon is examined, considering the strongly non linear behaviour of spannband structure. In fact stress ribbon tie can be considered as a cable subject to transverse loads and tensioned at both ends, having a significant sag effect.

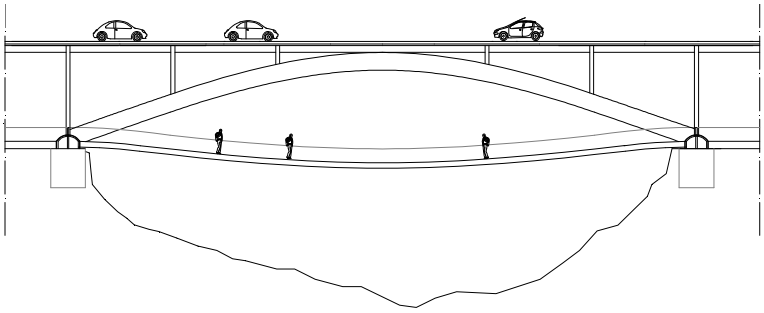

Figure 8. The concept of arch-spannband bridge

Main parameters to be considered in design of this kind of bridge are: arch rise and stress ribbon sag, pre-tensioning values of bearing cables, dead load values and temperature effects. Moreover different live load combinations have to be investigated.

Another important topic is also the dynamic behaviour of the bridges. Dynamics of arch bridges with an upper deck is well known, as well as dynamics of stress ribbon footbridges, that can be found in literature [5]. The combined system instead, never has been investigated. In fact the importance of crowd walking on the footbridge with possible lock-in phenomena has to be considered with particular attention in the design of this kind of bridge.

In this study only the static behaviour is considered. Items about the influence of time-dependent phenomena in concrete are also to be studied. They imply increased deformations in time and stress redistribution, depending on construction sequence, as well as changes of spannband profile with cables sag variation.

This particular solution of tied arch bridge has been investigated also in multi-span configuration, in which a sequence of tied arches with a lower stress ribbon footbridge can be useful to limit or avoid horizontal forces at bridge footings, due to asymmetrical and unbalanced traffic loads into the spans. With this target in mind, a solution studied for the preliminary design of the multi-span bridge over the Pertusillo lake, placed in the south of Italy, is presented and discussed, considering the importance of construction stages, in order to achieve the desired service life configuration and to fulfill the aim of avoiding thrust forces at piers and abutments in each arch span and in every stage.

2. CONCEPTUAL DESIGN OF ARCH-SPANNBAND BRIDGES

The preliminary design of a bowstring bridge, with a stress ribbon footbridge as tie, starts from the definition of the correct geometry of elements together with mechanical and kinematical parameters that govern the behaviour of this complex system and in consideration of the strong geometric non linearity of stress ribbon. A multistep design sequence has to be followed, by reviewing and refining the solution found in more times. After the dead load configuration is achieved, live loads, including moving ones due to traffic, have to be applied and analyzed.

A tied arch with two hinges at footings is considered here. The arch is axially rigid and the tie, put between the arch footings, is formed by a cable that is transversely loaded. Let L be the span length of the arch, f_a the arch rise and f_s the cable sag. Dead load on the cable is uniformly distributed in a first approximation, naming:

- q_a^d the arch dead load (including self weight and superstructure loads);
- q_s^d the load acting on the sagged spannband (fig. 9) and consisting of bearing cables, concrete deck segments, prestressing tendons and superstructure loads.

Let M_a^d be the bending moment at the midspan section of an equivalent simply supported

beam that is loaded by the same loads acting on the arch. The arch thrust and the horizontal tensile force into spannband are expressed by the following relations:

$$\mathrm{H_a} = \frac{M_\mathrm{a}^\mathrm{d}}{f_\mathrm{a}} \quad , \quad \mathrm{H_s} = \frac{q_\mathrm{s}^\mathrm{d}\, L^2}{8\, f_\mathrm{s}} \tag{1a,b}$$

Because of the equilibrium, the arch thrust and the horizontal tension must be equal, obtaining the equation:

$$q_\mathrm{s}^\mathrm{d} = \frac{8\, M_\mathrm{a}^\mathrm{d}}{L^2} \frac{f_\mathrm{s}}{f_\mathrm{a}} \tag{2}$$

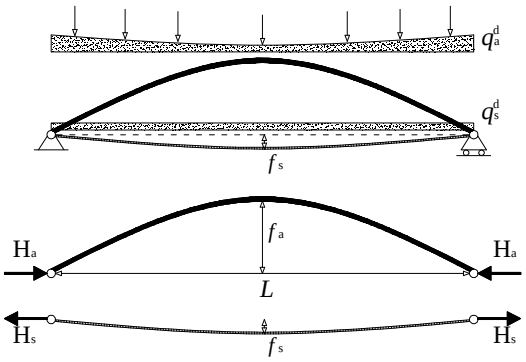

Figure 9. Arch thrust and stress ribbon tensile force

Relation (2) is useful to give the first dimensions to design the spannband-tie. In fact, if the arch characteristics are known it is possible to fix the value of the footbridge sag, due to pedestrians comfort, and then the dead load of spannband q_s^d can be found. It is directly related to the kind of deck segments, bearing cables and superstructure to be used. Consequently the order of magnitude of internal forces is known as well as geometric parameters, in a first approximation. This approximation depends on the real arch axial stiffness, because it cannot be considered as perfectly axially rigid, and also the tie is composed of a flexible cable. So a significant variation of horizontal forces has to be expected.

Consider now the real axial stiffness of the arch, being *s* the curvilinear abscissa along the arch geometric axis. When the arch is a redundant structure, even if the geometric shape has been designed coincident with the thrust line, the axial shortening due to the arch deformability modifies the real arch shape and the effect is a reduction of the thrust value as well as bending moments into the arch. Thrust loss [10] is given by the relation:

$$\Delta \mathrm{H_a^{def}} = -\frac{\mathrm{H_a^{rig}}}{\mathrm{u_1}} \int_{arch} \frac{ds}{E_\mathrm{a} A_\mathrm{a}} \tag{3}$$

in which $\mathrm{H_a^{rig}}$ is the rigid arch thrust; $\mathrm{u_1}$ is the horizontal displacement of the arch footing when it is longitudinally unrestrained with a horizontal unit force applied. The integral term contains all axial shortenings due to elastic strains of arch segments d*s* with area A_a and elastic modulus E_a.

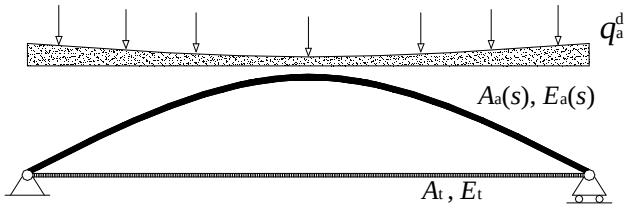

Figure 10. Tied arch concept with a flexible tie

Consider now the effect of a flexible tie with area A_t and elastic modulus E_t (fig. 10). It needs a horizontal free footing to work correctly and a further variation of the thrust into the arch, due to the tie elastic elongation, cannot be avoided. The total thrust loss is:

$$\Delta H_a^{def,tie} = -\frac{H_a^{rig}}{u_1}\left(\int_{arch}\frac{ds}{E_a A_a}+\frac{L}{E_t A_t}\right) \tag{4}$$

This loss can be significant, so the real thrust $H_a^{def,tie} = (H_a^{rig} + \Delta H_a^{def,tie})$ can differ a lot from the theoretical one of the axially rigid arch H_a^{rig}. As a consequence bending moments appear into the arch, depending on the arch and tie axial flexibilities. In this case the tie is composed of a spannbad footbridge whose deformability is very high and the value of thrust given by relation (4) is too much lower than the rigid one. Under these hypotheses the spannband could not be used as a tie because the displacement of the arch footing would be too large and the induced bending moment too high. This problem can be solved through a pre-tensioning of the tie, i.e. a pre-stress given to the bearing cables, during construction, that recovers all thrust losses. By applying an imposed shortening δ_t to bearing cables, the value of thrust becomes [10,11]:

$$H_a^{def,tie} = \frac{H_a^{rig}u_1+\delta_t}{u_1+\int_{arch}\frac{ds}{E_a A_a}+\frac{L}{E_t A_t}} \tag{5}$$

The shortening δ_t can be seen as the effect of a tensile force T given to the tie which recovers the thrust loss:

$$\delta_t = T\left(u_1+\int_{arch}\frac{ds}{E_a A_a}+\frac{L}{E_t A_t}\right) \tag{6}$$

and so the thrust value can be written by the following relation:

$$H_a^{def,tie} = \frac{H_a^{rig}u_1}{u_1+\int_{arch}\frac{ds}{E_a A_a}+\frac{L}{E_t A_t}} + T = H_a^{rig} \tag{7}$$

Relation (7) expresses the arch thrust when axial deformability is considered and the flexible tie is pretensioned by a force T applied at its ends, that is equivalent to the imposed cable shortening δ_t. From equation (7) it is possible to find the value of pretension T to be given in order to completely recover the thrust loss and to achieve the value of horizontal force H_a^{rig}, avoiding any bending moment into the arch in the dead load configuration. By imposing this condition, the value T is found:

$$H_a^{rig}\left(\int_{arch}\frac{ds}{E_a A_a}+\frac{L}{E_t A_t}\right) = T\left(u_1+\int_{arch}\frac{ds}{E_a A_a}+\frac{L}{E_t A_t}\right) \tag{8}$$

However, in this case the tie is a stress ribbon footbridge with a geometric non linear

behaviour due to the cable sag, so equation (8) is still valid but it implies an iterative procedure to find the correct value T. By considering the spannband as a cable of area A_s, elastic modulus E_s, subject to dead load q_s^d and with a fixed sag f_s, the horizontal force of bearing cables (that is equal to the arch thrust) can be computed by an iterative sequence. It consists of a successive application of equations (7) and (8) in order to find the value of T that assures the total recover of thrust loss. The procedure is illustrated in [12] by taking into account the Dischinger modulus of a sagged cable tensioned by a horizontal force at its ends.

3. SERVICE LIFE BEHAVIOUR OF ARCH-SPANNBAND BRIDGES

Consider now the arch-spannband system in its "zero condition" of dead load configuration with a funicular arch and a pretensioned tie that recovers all thrust losses. Construction stages of a spannband imply a change of stress ribbon cross section, by inserting prestressing tendons into the segments to connect them and to avoid concrete cracking in service life. In the initial phase the effective cross section area was only that of bearing cables A_s with concrete elements acting as an external load. After prestressing a different effective area A_e has to be considered, by homogenizing concrete, bearing cables and prestressing tendons (fig. 11). The effect of prestressing leads to a stiffened stress ribbon tie with a significant increment of section inertia and a little vertical upward displacement that modifies the real sag. Live loads must be considered associated with the effective section area:

$$A_e = A_s + A_p + (E_c/E_s)\, A_c \tag{9}$$

where E_c is the concrete elastic modulus, E_s is the steel one, A_p the internal tendons area and A_c the ribbon concrete area.

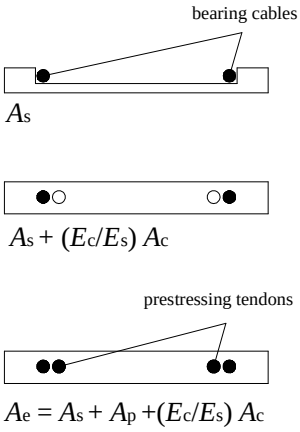

Figure 11. Modification of stress ribbon section during construction stages

The upward radial forces due to prestressing diminish the cable sag by a little. If it is neglected, the only variation of stress ribbon geometry and internal forces is due to further application of service loads.

Live loads are applied at the same time on the arch bridge and on the footbridge. It is important to evaluate the correct variation of arch thrust and horizontal tensile force into the spannband. In this analysis the arch maintains its linear behaviour and loads applied in different times can be superimposed. The effects of new loads acting on the spannband instead cannot be superimposed to the previous ones, just applied in the dead load configuration. It needs to calculate the new sag and the tension change into bearing cables, by applying all loads contemporarily and by re-computing the reduced Dischinger elastic

modulus in the changed configuration [13]. Now the stress ribbon can be considered as a cable of area A_e [5]. In conceptual design the effect of bending moments at the ends of stress ribbon, due to the increased stiffness (achieved after internal prestressing), is neglected, but it has to be taken into account in the successive design phases.

Traffic loads q_a^L on the arch and q_s^L on the footbridge have to be considered as well as uniform temperature variations Δt_1 on the arch and Δt_2 on the ribbon. Service loads modify the dead load configuration inducing a horizontal displacement of the movable footing, a variation of stress ribbon sag and bending moments into the arch. It is important to evaluate which is the worst distribution of variable loads that gives the maximum bending moment values into the arch.

By using the auxiliary restraint method, it is possible to find the effect of loads onto the structures (arch and stress ribbon). First the two structures are considered independent one from the other by temporarily restraining the movable footing to evaluate the horizontal reaction at the auxiliary restraint due to applied loads. After, the movable footing is unrestrained and the reaction found can be applied to the footing joint with the opposite sign. By superimposing forces in the scheme with the auxiliary restraint and those in the scheme with the moved end, the solution is found.

With this fact in mind, let consider first dead loads applied only on the arch with fixed footings. Live load position giving the maximum thrust value is that of a uniformly distributed load onto the entire span length (fig. 12*a*). Alternatively, the load distribution giving the maximum value of bending moments is an asymmetrical distributed load onto the half span length, with positive bending moments in the loaded half arch and negative ones in the other half arch (fig. 12*b*). In this last case the thrust is half of the maximum one, occurring in the previous configuration.

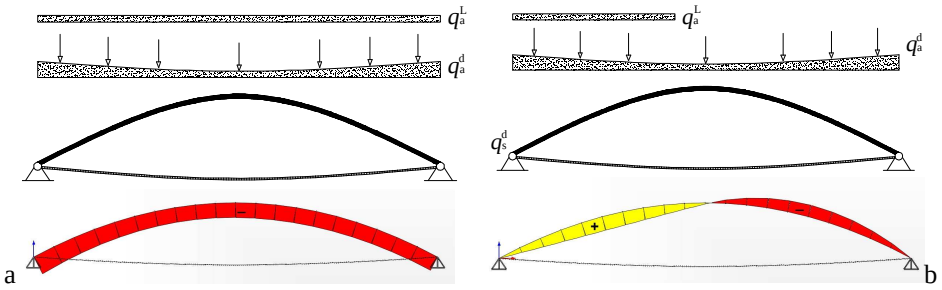

Figure 12. Internal forces in the arch alone. a) Max arch thrust (axial forces). b) Max bending moment.

This is valid for the arch alone, fixed at its ends. When the entire arch-spannband system is considered and the movable footing is unrestrained, it moves outside and positive bending moments, with lower fibres in tension, appears into the arch. In the case of live load distributed on the half arch, the effect of the tie is a reduction of negative moments and an increasing of positive ones, till the entire arch can have lower fibres in tension (fig. 13*a*).

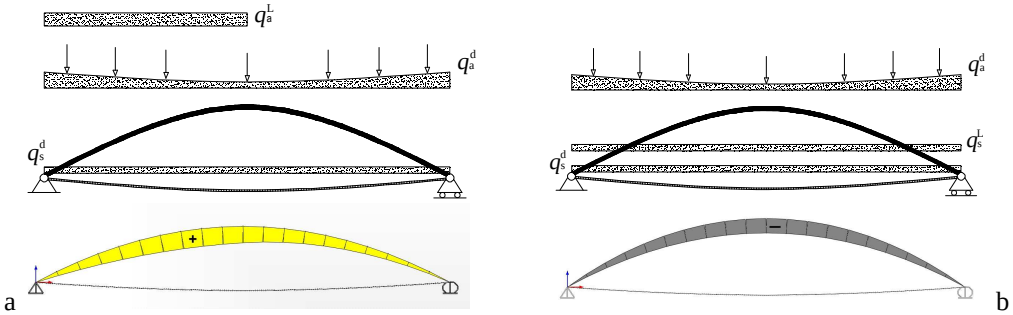

Figure 13. Bending moment diagrams in arch-spannband system. a) Max positive moments in the arch loaded. b) Max negative moments in the arch with stress ribbon loaded.

Consider now live loads onto the stress ribbon footbridge (fig. 14). When it has the movable footing restrained, the maximum value of horizontal force is obtained by applying the load onto the entire footbridge length. It diminishes when live load is distributed on the half length; in this case tensile force value is less than half of the maximum one, due to the non linear behaviour of ribbon. Uniform loads can have different effects: for an asymmetrical load on half span, horizontal force and sag increase and the maximum vertical displacement is under the loaded part (fig. 14*a*). For a symmetric centred load distributed on a length equal to half span instead, the increment of horizontal force is more significant and the maximum sag is in the midspan section (fig. 14*b*). These are the worst distributions of live loads onto the spannband, considered alone with the fixed ends.

When the combined system with the arch is considered and the end is unrestrained, live load on the spannband implies that arch footing moves inside and negative bending moments are expected into the arch, with upper fibres in tension (fig. 13*b*). This situation is a singularity of the arch-spannband system, because the arch alone has not this situation when live loads are applied on it. The values of negative bending moments are significant and the worst distribution is with the entire spannband loaded. It is important that the inertia of arch cross sections is dimensioned on this value of bending moments, to avoid undesirable upper cracking into the arch or too high stresses.

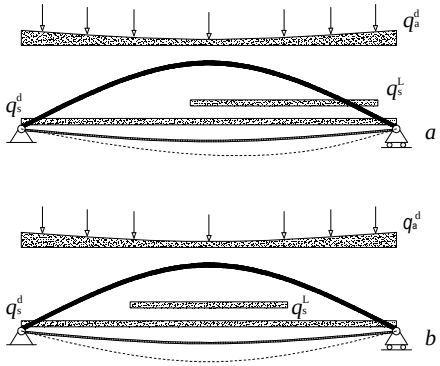

Figure 14. Live loads acting on the spannband

Another consideration has to be done on the ribbon sag: it depends on the geometric configuration (rigid displacements) and on the free end movements, related to elastic tie deformation and to arch stiffness. Because of live loads can act contemporaneously on the arch and the footbridge, it is important to consider different combinations, taking into account that the thrust value of the rigid arch reached in the initial dead load configuration is always the minimum value, which is increased for a live load applied onto one or both structures.

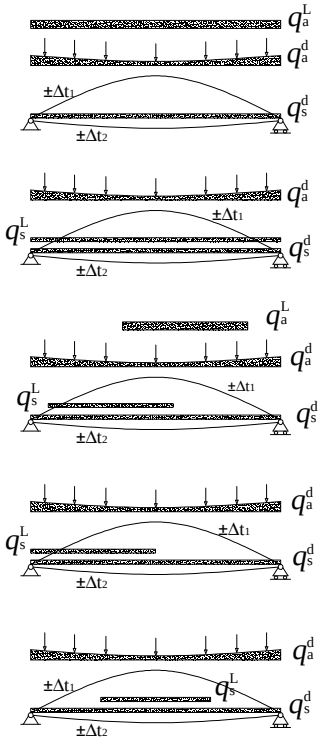

Figure 15. Live loads combination acting on the arch-spannband system

So an increased axial force is expected into the arch and into the bearing cables but bending moments can have different values and signs. In the particular case for which the thrust increased value due to arch loads is equal to the spannband horizontal increased force due to live loads acting on it, the result is that combined system has not horizontal displacements at the movable footing and the arch is a funicular one without bending moments and an increased value of thrust.

Figure 15 shows possible combinations of live loads applied on the global system, by considering also temperature variation. Every configuration change implies a displacement of horizontal footing, a variation of ribbon sag and bending moments into the arch.

In the case of multi-span tied arch bridges, the tie can be still a continuous spannband and it is very useful to avoid horizontal forces due to unbalanced live loads acting in different spans. This condition can be achieved by building each span as independent from the others and then connecting each tied arch to the successive one in service configuration. In this way the single span is a bridge in which the dead load configuration is that of a funicular rigid arch and it is achieved in the same way previously shown; after that, live loads are applied on the entire multi-span bridge (fig. 16). The way to evaluate maximum and minimum bending moments are still valid, by considering arch footing movements.

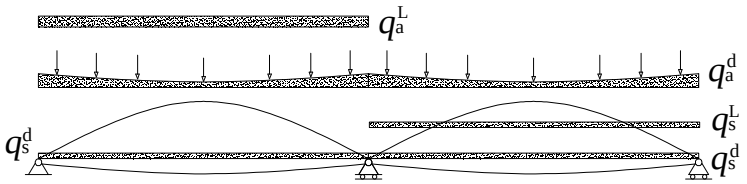

Figure 16. Multispan bridge

4 AN EXAMPLE OF A BOWSTRING BRIDGE WITH STRESS RIBBON TIE

A case-study of a multiple span bowstring bridge with a stress ribbon tie is presented. The bridge has been designed for the competition called by the Public Administration of "Regione Basilicata" in the south of Italy, regarding a new bridge crossing the Pertusillo lake. The idea is to solve the problem of a long bridge over a lake with the minimum impact on the environment, by assembling all prefabricated elements, without too invasive casting operations into water.

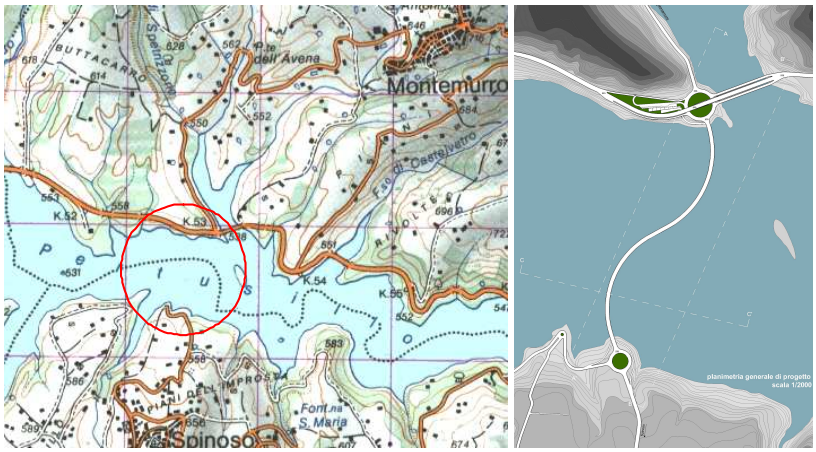

Figure 17. Geographical placement and general plane view

The total length has been divided into a number of subsequent spans covered by arches in sequence, each with a span L=48 meters, and a deck 10 m wide. The arch rise is f_a = 6.6 m while the initial spannband sag in the dead load configuration is f_s = 0.96 m. Arch footings rest on vertical piers whose foundations are positioned into water on the lake bed. Figure 17 shows the geographical position of the bridge while figure 18 shows the main geometric characteristics of the bridge.

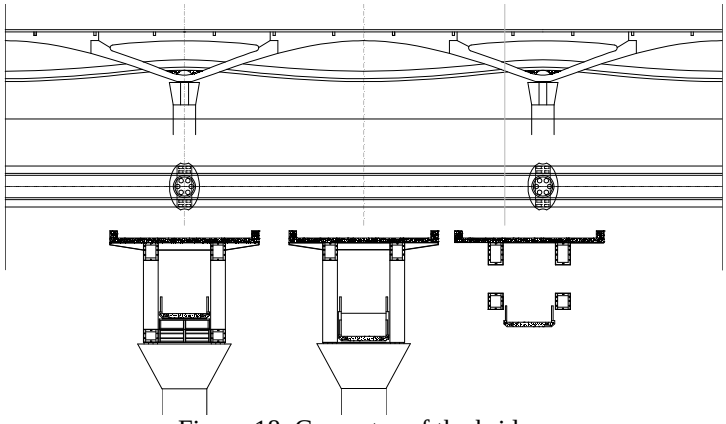
Figure 18. Geometry of the bridge

Every span is composed of two metallic parallel longitudinal arches with a boxed cross section. Each arch is assembled and formed by three elements longitudinally jointed: two cantilevers of triangular shape resting on the piers and a central beam supported by the triangle cantilevers through Gerber saddles. Elements to be assembled during construction are shown in fig. 19.

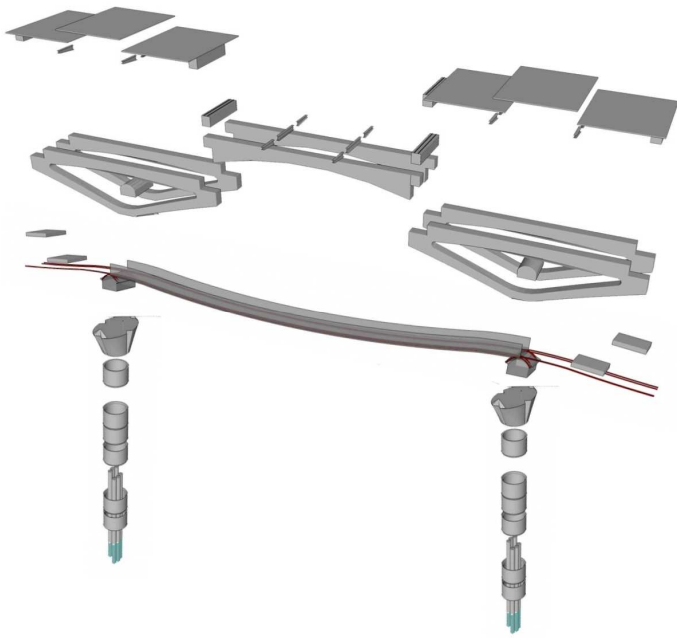

Figure 19. Elements to be assembled during construction stages

The construction sequence is shown in the following. First of all foundations have to be built without invasive works into water. It can be done by metallic driven piles that constitute the reinforcement of the piers successively composed of sunk precast circular hollow boxes. They are positioned from the bed to a predefined height over the maximum water level. The internal core is then filled by concrete in situ and the hollow box has the function of framework to avoid loss of material into water. After that, the pier top is completed by a prefabricated cap on which bridge supports are placed. On these supports the triangle cantilevers are put, temporarily fixed and then, between them, the central beam is placed. Transverse stiffeners link the two parallel arches which rest on piers as simply supported beams, longitudinally fixed.

After this first phase the bearing cables of the stress ribbon footbridge can be placed and tensioned between the supports over the piers and connected to the arches footings; deck concrete segments are put onto the bearing cables. At this point the movable end, previously fixed, can be horizontally released and the triangle elements are made independent as in the scheme shown in figure 20, by internal releases. In this way the arch, with the antifunicular shape of dead loads, supports the lateral elements which close the triangles. The central arch has the spannband between footings, working as a tie. The upper arch is completed with precast plates put on the metallic arch elements, in order to form the upper bridge slab and finishing works with superstructure elements are completed. In this phase the pre-tensioning force T is given to the bearing cables, by adjusting the tension through jacks in more phases, till the dead load configuration is achieved with the thrust of the rigid arch H_a^{rig}, by recovering all thrust losses.

In this way the dead load configuration is reached in a single span. The operation is then repeated for the subsequent spans by temporarily fixing the arch footing of the completed span. After all spans are completed, all ends except one are longitudinally unrestrained. Intermediate stiffened piers with fixed arch restraints need in order to avoid excessive displacements to be absorbed at the end of bridge and to limit the effects of earthquakes.

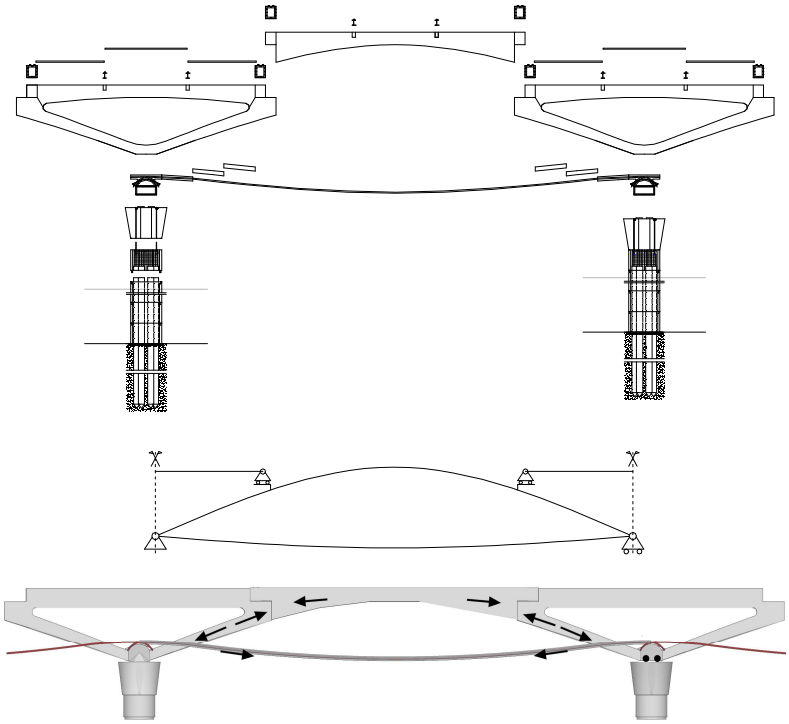

Figure 20. Construction sequence and static scheme for conceptual design

After dead load configuration is achieved into all spans the internal prestressing of stress ribbon footbridges can be done, before to apply live loads. In this stage the effective area of spannband changes and radial forces modify the spannband sag; the structure stiffness increases and secondary bending moments at the ends appear. Live loads due to traffic can be applied at this stage, achieving the service life condition. Time-dependent phenomena have to be considered for the final profile, evaluating the modification in time of stress ribbon deformed shape.

Figure 21 shows a render of the proposed solution of arch-spannband multiple bridge over the Pertusillo lake. The footbridge can be considered as a real walking on the water; it is covered by the upper arch and it is independent from the bridge road traffic.

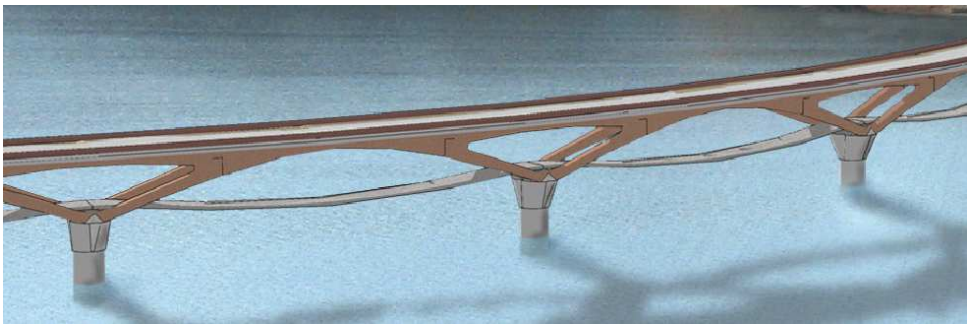

Figure 21. Render of the bridge

5 CONCLUSIONS

A proposal of a bowstring bridge, in which the tie of the arch with an upper deck is composed of a stress ribbon footbridge, has been investigated. The main parameters involved in the conceptual design of this particular solution has been shown and discussed, considering the strong non linear behaviour of the stress ribbon footbridge. Considerations about dead and

live loads are given as well as construction sequence and successive static schemes variation have been shown. A case-study of the conceptual design for the Pertusillo lake bridge has been presented, to explain the main items regarding the approach to the design of this kind of double bridge structure. Construction stages have been described underlining the importance of a correct sequence in order to achieve the main target of avoiding horizontal forces on foundations or over the piers, in every stage.

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