

## **Bridge strengthening by conversion to network arch. Design criteria and economic validation**

Matías A. Valenzuela\* and Joan R. Casas

*Public Works Department, Santiago, Chile – Universitat Politecnica de Catalunya BarcelonaTech, Barcelona, Catalunya-Spain*

Morandé 71, matias.valenzuela@mop.gov.cl, phone: +56956374031

# **Bridge strengthening by conversion to network arch. Design criteria and economic validation**

The paper presents a new strengthening methodology in bridges over river beds affected by scour and erosion at piers. The proposal is part of an innovative concept, implementing a structural change of the original bridge, without the need to strengthen the substructure, thanks to the construction of an upper arch with network and vertical hanger's. The vertical hangers are responsible to lift the deck from the piers, generating a single span (simply-supported tied arch). The paper describes the construction phases, considering the conditions and difficulties in their implementation due to multi-objective targets and additional boundary conditions and requirements. Additionally, it gives a description of the structural behavior and a layout of each element, to use as criteria or guideline for future applications of the method. Finally, a validation is made through a comparative analysis in a Chilean bridge, between a traditional method of strengthening by additional piling in the foundations and the proposal here presented. Additionally to the feasibility of application, the example shows that the new solution is cheaper.

Keywords: network arch; strengthening; scour;hanger; lifting

## **1.Introduction**

One of the major problems in bridges over riverbeds is the scour in the foundations. The consequences are of varying severity, but certainly the most important is the total collapse of the structure, generating high human and economic costs. As a result, they have been implemented several measures of inspection, monitoring and maintenance, not properly applied at underwater infrastructure conditions. In addition, the action of accidentals loads (earthquake) and the poor support conditions produce a highly vulnerable and risky scenario for these bridges.

In Andean countries such as Chile, one of the most common problems is the undermining of the piers (Muñoz, 2006; Seaurz, 2006). The repair and strengthening are frequent and generate high economic losses. The damage caused by the flow stream in Andean rivers is important because they are of torrential nature, able to move large and heavy rocks. These factors induce a

high risk in the piers, which are eroded in their foundations and can be themselves fully destroyed by the passage of the water flow.



Figure 1. Scour at bridge piers.

The condition of the foundation because of the scour, leaving the piles out of the ground as shown in Figure 1, is extremely dangerous in countries with high seismic activity, requiring a constant monitoring, repair and strengthening by additional piles (steel or concrete) and screeds. For this reason, the Ministry of Public Works of Chile, through the Department of Bridges has promoted the study and development of an alternative strengthening method for this type of damage.

Mitigation mechanisms have been developed, which often extend the service life of the structure. However, many of them present difficulties in their implementation (require qualified personnel and are very expensive). Among the traditional repair methods are those used for the protection of the channel and bridge elements. Frequent techniques correspond to: the armoring protection around the piers and the river bed (Figure 2), dam construction, the increase of the span of the bridge and constant monitoring of the scour. These techniques, in emergency cases, improve the conditions of the bridge. However, they present the disadvantage of physical changes to the river flow, and the requirement to work in complex access conditions, deriving on high costs of materials and labor.



Figure 2. Protected armoring. [Courtesy of B. Moya]

Other protection techniques are the improvement of resistance of elements and structural support. As an example, the concrete screed, mortars using mixed cement with sand, gravels and aggregates (bagged concrete or fluid micro-concrete) improve the strength and structural support at the base level (Figure 3). When the foundation is under water, the application is through pipe systems or injecting a concrete grout to fill any existing hole. The main drawback is the geometrical increase of the area exposed to the river current and the change in flow patterns, inducing future erosion problems.



Figure 3. Concrete screeds.[Courtesy of B. Moya]

Finally, to improve the bearing capacity, the most commonly technique used is the strengthening by additional piles (Figure 4). These enable to transfer the loads to a capable layer of soil when it is at a considerable depth. It is a common technique which requires a detailed study of soil stratigraphy and geology, looking for a good support layer. Despite its good

performance and wide variety of techniques (kneeling, percussion, excavated) and materials (steel, concrete, and wood) this method presents a number of drawbacks, including:

- Difficulty of implementation (limited access of the equipment, vertical clearance, inclined piles).
- Depth of foundations (Placing reinforcement and concrete joints, false rejections in the driving, presence of thin hard layers).
- Incompatibility between construction method and soil type (e.g. driving in the presence of coarse gravel, on-site method in locations not permitting shoring and movement of material).
- Durability of the material. (Steel corrosion or wood rotteness in unsaturated zone).



Figure 4. Strengthening by piles.[Courtesy of B. Moya]

Another technique is to strengthen by micro-piles. They are used in areas where is not possible to implement conventional piles by difficulty in the access. Among its advantages, is the fast implementation and minimal discomfort at worksite. However, the micro-pile bearing capacity is lower than that of the piles, requiring a large number of them, thus increasing the cost. Also, it is not usable in very deep foundations, and does not provide a good response to lateral actions in the presence of a seismic event. Normally, the strengthening sequence is as

follows: After the collapse of the piers and placement of provisional supports, the superstructure is released from the piers, and it is lifted by jacks. Then, the damaged part is demolished, and the foundations strengthened by piles or driving caissons. Finally, the pier is rebuilt and the superstructure is placed again into place (Valenzuela, 2012). This technique improves the bearing capacity of the piers and locates the foundation level below the general and local scour, delivering an optimum configuration. However, the interference of current flow is not eliminated, requiring future inspections and monitoring of the area.

In Malerba (2014), some examples of problems in bridges due to scour and the techniques used to strengthen the foundations are presented.

## **2. New strengthening methodology**

A new strengthening method of short and medium-span bridges, with several continuous spans is proposed. The method performs the modification of the static scheme (from continuous beam to tied-arch) by building an upper-deck arch with a network hanger arrangement, in which the existing deck becomes the tie of the system. This allows the removal of the intermediate supports, generating a single span, thereby eliminating any future scour problems (Valenzuela, 2012; Valenzuela & Casas, 2013). The main characteristic of network arch bridges is that the inclined hangers cross each other at least twice. They have demonstrated an excellent performance for the construction of new bridges (Tveit, 2007; Valenzuela, 2012; Pircher et al., 2013). In the present case, the network arch is used in the strengthening of an existing bridge.

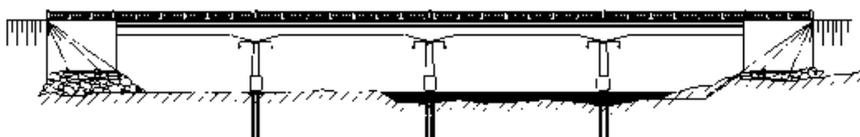
This new method of strengthening is sustainable due to the decrease of the repair costs, avoiding repetitive and expensive repairs and strengthenings in the foundations and piers. Additionally, it delivers a new significance to the repair concept, providing an enhanced aesthetic solution. The proposal is part of a philosophy of repair and strengthening of bridges with emphasis on the reuse of components, allowing structural and economic benefits and providing additional aesthetic value.

Based on the analysis of the construction process, the method proposes a verification associated to the definition of an upper and lower bound and an acceptance bandwidth based on these bound values. The method can be applied to bridges with little initial information, i.e., lack of detailed drawings, steel reinforcement, and types of materials, through the study of admissible ranges of stress (MAB) (Valenzuela, 2012; Valenzuela & Casas, 2013). The range corresponds to threshold values obtained by the analysis of the actual stresses in the bridge under analysis. To this end, based on the actions present in the existing bridge, the actual state of stress in the deck is obtained. These maximum and minimum limits in the service state can not be overpassed by the existing deck during the different phases of the strengthening application and later on in the new service phase of the strengthened bridge.

The initial step in the strengthening methodology corresponds to the calculation of the maximum stress state in the original deck over piers due to the actual loading conditions and feasible load combinations. If the information gathered is insufficient, a geometric survey and material coring and testing may be required to up-date the limited available information. The proposed new strengthening procedure is divided into three phases, as follows:

### ***2.1. Preparation Phase.***

During this phase is evaluated the need to improve the deck geometry according to current standards (e.g., incorporating an extension of the carriageway, enlarging the width of the existing lanes,...) or because of the additional space required by the strengthening process itself, for instance, the reconstruction of the carriageway with the necessary dimensions to deploy the arch at the center of the cross-section. It should be also assessed the scour condition and resistance capacity of the abutments to accommodate the additional vertical force transmitted in the new static scheme. (Figure 5.a).





After post-tensioning the deck with the external tendons, the temporary towers can be removed, avoiding excessive strain in the arch. Later on are deployed two groups of passive hangers (network and retention), and the vertical hangers without applying any tensioning (Figure 5.d).

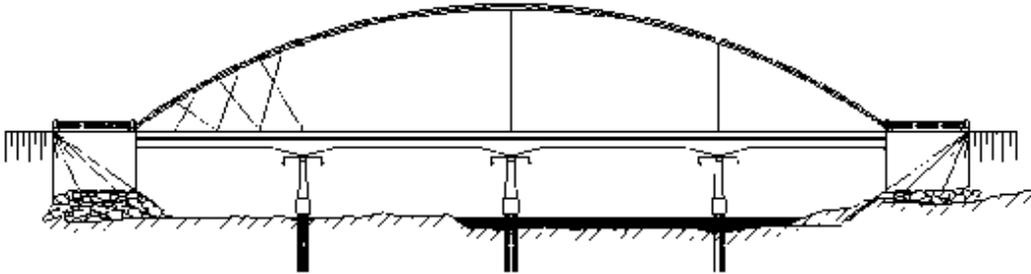


Figure 5.d. Deployment of network hangers (d).

The network hangers follow a configuration such that they cross each other at least twice (Tveit, 2007). The retention hangers correspond to the set of hangers placed near the junction between deck and arch, to regulate the negative bending moments in the deck at this point.

After the placement of the passive hangers, the first phase of the construction is completed. It should be noted that in these early stages, the requirement to be fulfilled is: "avoid that the stress at any point, in any of these stages, exceeds the limits set by the MAB". Following this recommendation, the critical point is normally located in the deck, since this is the zone where the reference stressbandwidth (MAB) is more restrictive.

**2.2. Lifting Phase.**

The second phase corresponds to the tensioning of the vertical hangers, which are responsible for gradually lifting the deck from the piers, as shown in (Figure 5.e).

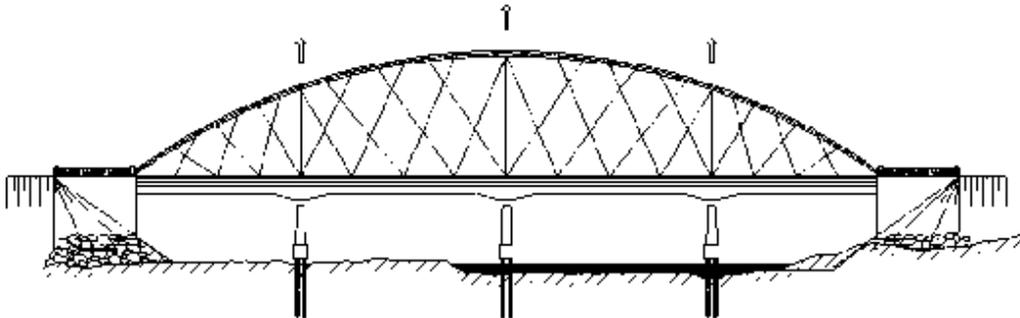


Figure 5.e. Tensioning of vertical hangers (e)

The proposed sequence of tensioning considers the introduction of the total force in the vertical hangers by only one jacking operation, reducing the number of tensioning operations to a minimum to optimize the cost. As a basic arrangement, the vertical hangers are placed over each original pier. At this point, the sequence and magnitude of the tension in each vertical hanger to achieve adequate lift of the deck should be defined. The structural analysis starts from the stress-strain state obtained at the end of the preparation phase, and follows the guidelines of the criterion of MAB (Valenzuela, 2012; Valenzuela & Casas, 2013).

The lifting phase of the deck is the most complex and delicate operation throughout the process of bridge strengthening. It involves a large number of parameters and variables that affect the response of the structure and define the success of the operation, considering all the stress changes in the deck due to the introduction of the force in the hangers. In this stage is when, in fact, the change of the structural longitudinal configuration (from continuous beam to arch) occurs. For this reason, this phase is considered fundamental in the strengthening process. The major computational efforts are present in this phase, as the tension introduced in the hangers must lift the deck and leave it in an allowable stress state during both the construction and the in-service phases. Therefore, it is necessary to obtain safe and acceptable values of the jacking sequence and the magnitude of the jacking force in the hangers, through an optimization tool, solving a multi-objective optimization problem by means of genetic algorithms (Valenzuela, 2012; Valenzuela & Casas, 2013). The algorithm is formulated through the combination of discrete and continuous variables, without an explicit objective function. It

optimizes two variables: order of tensioning and magnitude of the force to be introduced in the vertical hangers. The optimization seeks to reduce the differences between the original stress on the deck and those after each tensioning phase to lift the superstructure, allowing the lifting of the deck over each pier and maintaining a state of allowable stresses in all bridge elements (old and new). In addition, the optimization also searches to use the minimum amount of materials and minimum number of jacketing operations (economical constraints) (Valenzuela & Casas, 2011; Goldberg, 2002).

### ***2.3. Network phase.***

If necessary, in this phase the tensioning process in each hanger network is defined. Only the network hangers that require an initial tensioning will be stressed. This is defined according to the structural analysis of the bridge during the in-service operation. The location of hangers corresponds to a spacing not greater than 4 meters at the arch and a symmetrically configuration. Considering this, the tension of these hangers meets the needs of an efficient structural performance in the service stage (Valenzuela, 2012).

Finally, when the deck is completely supported by the hangers, the piers are demolished (Figure 5.f) and the new network arch configuration is obtained.

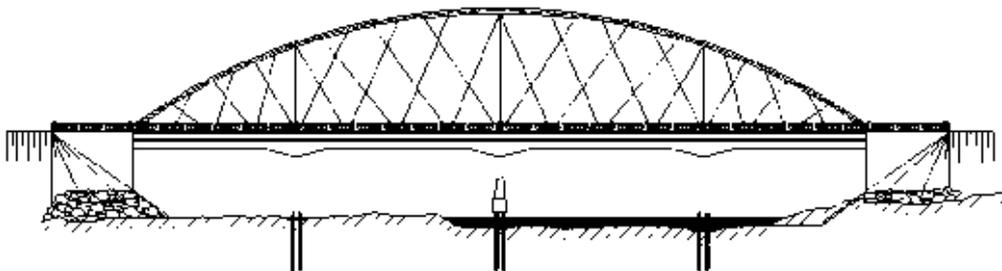


Figure 5.f. Removal of the piers (f).

### **3. Design criteria**

The basic criteria to define the elements used in the strengthening method include the arch, hangers, and the modification of the deck. Based on a great number of calculations and

parametric studies (Valenzuela, 2012), it has been possible to deliver a summary of these design criteria. According to the significance coefficients obtained from multivariate studies (Valenzuela, 2012; Valenzuela & Casas, 2013), the greatest impact upon the correct performance of the strengthening comes from the type of profile used in the arch, the external prestressing layout and the weight of the deck. In the following, the proposed design criteria based on a parametric study are presented. Further information on how they are derived can be seen in (Valenzuela, 2012; Valenzuela & Casas, 2013).

### ***3.1. Arch***

#### **(1) Behavior**

The arch is the main design element. It is proposed the use of a single arch centered and placed over the longitudinal beam, seeking for a reduction of material and number of operations. The internal forces in the arch are composed of staggered axial forces and bending moments in the plane, similar to the behavior of a continuous beam, i.e., it is composed by negative peaks at the points of application of the vertical hangers. However, these peaks have a decrease along length due to the secondary effects of alternation of the bending moments provoked by the network's hangers.

The bending moments in the plane of the arch are similar or slightly greater than the out-plane ones. Thus, it is recommended to use symmetrical profiles. The minimum square profile recommended is higher than the profiles used on a new construction projects. From this, the relationship between edge arch profile and total span is higher than for new construction projects (Valenzuela, 2010; Rongish, 2011).

When the deck has a significant width, it is necessary to use a double arch. In these cases, the inclination of the arches is a subject to study in the future due to the fact that it induces an axial effort in the transverse elements of the deck.

The rise-to-span ratio is relatively high, with a moderate participation in the lifting process and in the reduction of the effects of embedment between arch and edge of the deck.

## (2) Design Criteria

Based on the explained behavior and the results of the parametric study, the following design criteria were obtained:

- When using a single centered arch centered, it is recommended a symmetrical profile (same inertia in the two principal directions). When using a double arch solution, it is recommended to use arch profiles with greater inertia in the plane of the arch.
- The ratio between the inertia modulus of the arch and the longitudinal beams of the deck should not exceed 75. This improves the ability to lift the deck without inducing a failure.

$$R_{B/A} = \frac{Inertia_{Beam}}{Inertia_{Arch}} \leq 75 \quad (\text{Equation 1})$$

- A circular arch shape is recommended.
- A rise-to-span ratio in the order of 0.14 is recommended in those cases where the wind effects are very important. However, in the cases where the criterion has been the profile optimization, the rise-to-span ratio should be in the range 0.16 to 0.17. It is important to never exceed the value 0.2.
- Due to the selected profiles, the relation cross-section depth / span should not be less than 1/127.

With these design criteria, the buckling of the arch both in and out of plane is avoided, as shown in the analysis carried out by a Finite Element Model of the bridge (Valenzuela & Casas 2013).

### **3.2. Deck**

#### **(1) Behavior**

In the context of strengthening, the deck is not a design variable, as it corresponds to the unique active reused element of the original bridge, considering a static system that includes intermediate diaphragm at mid-span. The deck section studied corresponds to continuous longitudinal beams with transverse diaphragms, thus, the strengthening should preserve the stress states and internal forces to those of a continuous beam bridge, with negative moments in the areas of vertical hangers and positive in bays, with slight modification provoked by the network hangers. This induces a decrease of maximum values in the lifting phase respect to established limits.

The deck acts as edge beam or tie element of the system, thanks to the addition of the external prestressing. The transverse beams are actively involved in the stress distribution between the longitudinal beams and to face the transverse moments.

The embedment effect is a consequence of implementing the network structure conditions, where the arch and deck are linked generating negative moment at the longitudinal edge of the deck. Since the adequate behavior is defined as similar to continuous beam bridges, these effects are not desired. Therefore, it is recommended to modify the geometry of the arch or the curvature at the base of the arch; to use retention hangers or consider a new hinge joint between arch and deck, trying to reduce or eliminate these efforts. This connection is composed by a horizontal plate under the pin connection, in order to distribute the stress, and a vertical plate for prestress tendon joint (Figure 6).

The incidence of the embedment effect is negligible in the beams that do not support the arch, and when the arch has small profiles. Initially, the negative moment at the edge of the deck is not a problem for the admissibility. However, it is advisable to use mechanisms that reduce the effects of embedment. When the requirement of maximum allowable internal force is not

complicated using a hinge joint at the base of the arch, it requires a study of the ductility of the deck and its ability to form plastic hinges.

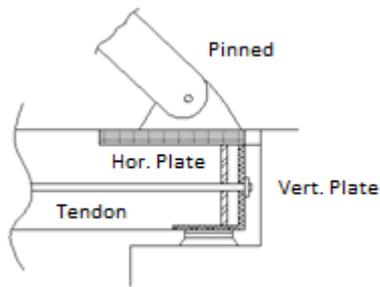


Figure 6. Arch-deck pinned connection.

The support system in the abutment corresponds to a simple support (fixed and unidirectional sliding), with "pot" bearing devices (neoprene confined). These are used due to the need to replace the original bearings, because of the increase of the vertical loads (due to removal of the piers and the additional weight of the arch). Additionally, the problem of the dimensions of the bearings can be reduced by the placement of more than one device under the arch support, considering as minimum one at each beam. This also improves the behavior because it reduces the transverse moments of the arch and produces a greater stability of the bridge against lateral loads. The analysis of the strengthened bridge, carried out by a Finite Element Model, shows a low distribution of the vertical loads to the beams that do not support the arch (4 - 7%). This requires to choose between the use of a stiffening system at the edge transverse beam to homogenize load on the bearing devices; or simply, to use bearings with different characteristics.

## (2) Design Criteria

- It is recommended to check the ductility of the deck in the link with the arch to see the feasibility of development of plastic hinges at this point and the ability to redistribute the

stresses in the structure. If enough redistribution capacity is guaranteed then it is recommended to place a hinge connection between the arch and the deck.

- Internal forces at the edge of the deck.
  - (a) It is not recommended the change in geometry of the arch based on the efforts at the edge of the deck.
  - (b) However, if the stresses at the edges exceed the MAB, and the bearing capacity and ductility of the deck is poor, it is justified the application of passive solutions (change of the curve of the arch, retention hanger, etc.).
- For the external prestressing it is recommended to use a straight layout placed at the center of gravity of the deck. It is allowed an eccentricity within the range -0.1 to 0 meters, looking for reactions and adequate reductions of the axial distribution of the network and vertical hangers.

In some cases, if the transverse beams (diaphragms) are not sufficiently designed to accommodate the internal forces in the new static system, the local strengthening of these elements should be also carried out.

### ***3.3. Vertical hangers.***

#### **(1) Behavior**

The vertical hangers are the active elements in the process of lifting the deck and responsible to induce a stress state similar to the original state of the deck, replacing the original piers. They are placed over each original pier. These hangers are fundamental in the lifting stage and the in-service state too. They never should lose tension, nor reduce it to levels that create stress states outside the MAB. In the same manner, the maximum tension should not exceed their material resisting capacity or induce stress states, in the deck and arch, higher than the design criteria.

From the jacking operation in the lifting stage, the normal behavior of these hangers is in tension. The maximum tension occurs in the service condition due to traffic load. The minimum stress occurs associated with vertical seismic loads in the cases when the top and bottom anchorages approach themselves.

## (2) Design Criteria

- The parameter "Ad" represents a limit criteria defined by the ratio between deck weight and its flexural resistance. This parameter is defined in the following way:

$$Ad = \frac{Pt_{EQ}}{M(-)_{MAX}} \quad (\text{Equation 2})$$

Where:

$Pt_{EQ}$ : Weight per stage equivalent, defined as  $Pt_{EQ} = \frac{Pt_i + Pt_{i+1}}{2}$

$Pt_i$ : Weight per span (i)

$M(-)_{MAX}$ : Maximum negative bending moment.

- The value of this parameter is used prior to the analysis of the strengthening process, such that if  $Ad > 1.2 \text{ m}^{-1}$  the iteration process of tensioning must begin with a higher number of vertical hangers or deferred lifting loads must be considered (by a second group of tension hangers or a multi-step tension operation after verification of the ultimate limit state).
- Tension force and order of tensioning of the vertical hangers.
  - In the multi-objective optimization, the algorithm used in this study is based on the theory of genetic algorithms (Valenzuela, 2012) being the tension force and the order of tensioning the variables. Therefore, a set of possible solutions (value of the tension

force and order of tensioning) is obtained at the end of the optimization (Pareto front).

The following criteria for selecting the final solution should be adopted:

(a) Choose solutions with a low-tension value of the central hanger, and a homogeneous tension at all the vertical hangers. This achieves a better performance in the arch, deck and in the tension of the vertical hangers.

(b) For lifting of very heavy decks ( $Ad > 1 \text{ m}^{-1}$ ), it is recommended to choose a process where all vertical hangers are acting and the lifting is produced just when the last vertical hanger is jacked. This avoids the appearance of non admissible peak stress in the arch and deck.

(c) Take a jacking tension of the hangers at least 20% lower than the maximum allowable stress (fatigue criteria). Furthermore, it is recommended a jacking tension that produces a deflection in the permanent state 25% higher than the maximum allowable deflection.

### **3.4. Network hangers.**

#### **(1) Behavior**

The application of a set of network hangers as part of the strengthening process solves the need to stiffen the system and get a truss-beam system, where the arch acts as top chord and the deck as lower chord. The network hangers are responsible to distribute the efforts from a chord to the other. In the case of the strengthening process, the behavior of these hangers varies depending on the construction phase and the influence of the vertical hanger.

Initially, the collaboration of the network hangers is passive, collaborating in the lifting process, i.e. they act when the process requires it. The forces in network hangers show a reduction of tension with the increase of the participation of the vertical hanger, due to the deformation in the arch. This provokes a compression in some of the network hangers. Normally, there are a reduced number of active network hangers after the end of the lifting stage. The

maximum tension (which is always checked to be less than the maximum tensile force in the material under permanent load to avoid fatigue and also less than the material strength) is obtained in this stage, corresponding to the hangers placed at the ends of the bridge.

Although their passive participation in the lifting process, their performance is essential to preserve admissible stress in the deck. After the lifting phase, a tensioning process of the network hangers is performed with the goal that all of them participate actively in the service state, i.e. to face vertical seismic loads (without loss of tension) and traffic loads (below of failure load). In this way, the structure performs avoiding structural changes and assists in the dynamic response of the bridge. Additionally, the involvement of these hangers reduces stress on the arch and the deck against the effects of service loads.

On the other hand, there is a number of network hangers, beyond which an increase does not generate an improved static behavior, showing an asymptotic decrease of the benefits. When the tension of the vertical hanger is low, it is recommended to increase the distance of the network hangers.

The radial arrangement is the best fit of hangers. It is favorable to the process of lifting and service, respect to rhomboid-type arrangement. This behavior is confirmed by observing the spacing between hangers. The rhomboid-type arrangement with spacing of 4 meters obtains worse results than radial arrangement with spacing between 2 and 2.6 meters (respect to the performance of the deck and the percentage used of the arch profile). In relation to the angle, it is proposed a range of  $45^\circ$  to  $65^\circ$  (similar to a new design).

## (2) Design Criteria.

- It is advisable to have areas of hangers (network and vertical) in a ratio between 0.67 and 1.
- Network hanger's arrangement.

(a) Always use passive hangers during the lifting process.

(b) When choosing a sequence of lifting process that begins at the center and continues to the ends, then it is possible to use a vertical passive hanger's arrangement.

(c) The recommended spacing of network hangers at the arch and deck is between 2 to 4 meters.

(d) It is recommended the use of a radial arrangement with individual placement of the hanger, thus increasing the efficiency.

(e) The process of tensioning of the hangers network following the lifting phase, should produce very favorable effects in front of in-service loads (truck loading and earthquake).

(f) The range of maximum and minimum axial force recommended for tensioning of the hangers is defined by  $R_n$ , the ratio between the magnitude of the force in vertical and network hangers. This is a key parameter justified by the high redundant condition of the bridge as a consequence of the vertical and network hanger arrangements (similar to a cable-stayed bridge). In this sense, when a hanger is jacked, the whole arrangement redistributes its stress producing, in some cases, a final non-tension force in some hangers. This criterion allows to control this problem. This ratio should be:  $6.5 < R_n < 10$ .

$$R_n = \frac{A_{v_i} + A_{v_{i+1}}}{A_{n_j}} \quad (\text{Equation 3})$$

Where:

$A_{v,i}$ : Axial force in the vertical hanger (i)

$A_{n,j}$ : Axial force in network hanger in the span j (part of the deck between vertical hangers (i) and (i+1))

### (3) Connections

As mentioned before, it is recommended the use of symmetrical profiles of the arch (squares).

The connection of the hangers (vertical and network) is by welding a plate at the bottom of the

profile, joined by bolts between the plate and the end of the hanger (Figure 7).

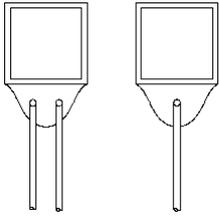


Figure 7. Connection: arch-hanger.

The connections between bars and concrete deck cannot follow the recommendations of new construction (Brunn & Schanack, 2003; Schanak, 2008); i.e. a connection by anchoring of the steel bar by welding, (Figure 8) or bars connected by its own thread in the hanger, anchored to a pitchfork, (Figure 9) (Sasek & Faler, 2006).

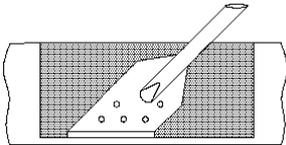


Figure 8. Connection embedded: hanger-deck.

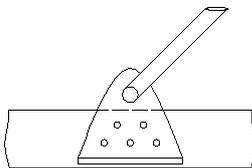


Figure 9. Pitchfork connection hanger-deck.

In the strengthening solution this is impossible, due to the requirement to drill the deck at each of the positions of the hangers, with a sufficient dimension to introduce the rods with the plates and to join the connector elements, looking for the collaboration in the transmission of forces between the old and new concretes.

The connection is also not feasible (analysis in San Luis bridge, Chile (Valenzuela, 2012)), because the width of the longitudinal beams (40 centimeters) is not enough to place a connection system and perform the physical union between the old and new concrete (because of drilling and the reinforcement, Figure 10).

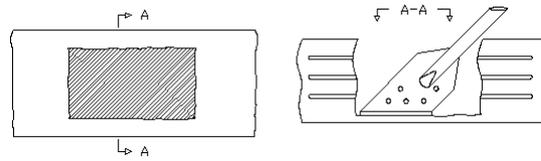


Figure10. Strengthened connection: hanger-deck.

For this reason, it is proposed a system similar to the KarolienenBridge (Unterweger, 2008), where the bar goes through the concrete element, connecting with a horizontal plate located below the longitudinal support beams. The position of the rods depends on the eccentricity between them (Figure 11). For the inclined hangers, the connection is located at each side of the girder, passing through the slab with a penetration equal to the length defined by each hanger slope. For that reason the depth of the drilling is equal to the thickness of the slab plus the projection of the slope. Then the hanger follows the longitudinal girder from the bottom of the slab reaching the bottom flange tied to the sides of the girder. Finally when arriving to the flange, it is joined to a plate (bolted), providing the support to the girder and the hanger.

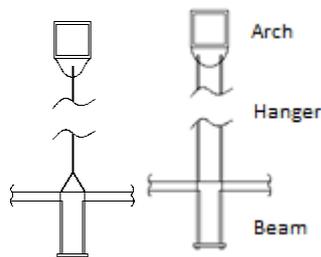


Figure11. Proposed connection: hanger-deck.

The same system is considered for tensioning the vertical hangers. However, the application of these hangers requires additional work because of their location in the area where the beam rests on the pier. For this reason, during the construction process, when un-linking the beam from the pier, it is necessary to carry out a cut in the pier in the points of support of the longitudinal beams (if the gap between pier and transverse beam is small). In this position, vertical jacks are placed (the same that are subsequently used for the replacement of bearing devices in the abutments), applying a controlled force to enable increase the gap to place the lower plate and fix the bar (Figures12 and Figure 13).

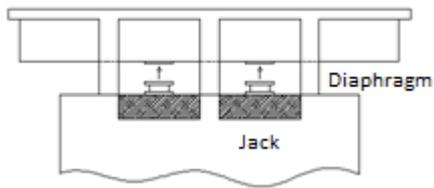


Figure 12. Lifting mechanism to placed the hangers.

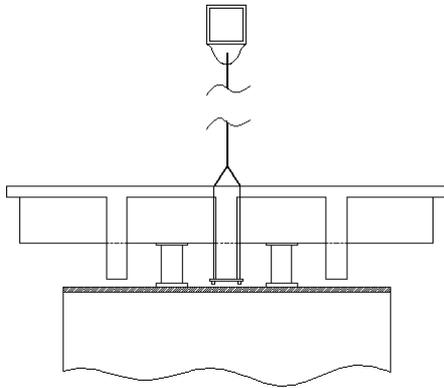


Figure 13. Final scheme: placed the hangers.

As a summary, Table 1 presents some of the design criteria previously analyzed.

Table 1.Element characteristic criteria

Elements		Strengthened bridge
<b>Arch</b>	Position	One centered (deck < 8m) Double (deck > 8m)
	Inclination	No
	Bracing	Vierendeel
	Layout	Circular
	Rise/span ratio	16-17%
	Profile	Square
	Size Profile/Length	< 1/127
<b>Network Hangers</b>	Type of hangers	2
	N° of groups	3
	Distribution	Radial - Vertical
	Distance hangers	2 to 3 meters
	Angle	55° a 65°
<b>Deck</b>	Material	Concrete reinforcement
	Type	Long. & Transv. Beams

	Increase high	Transversal beam	
	Use transversal beam	Yes	
<b>Bearings</b>	Model Support	Simple support	
	Type	Pot Bearing	
	N° devices	Original support	
<b>Connection</b>	Connection Coupling hanger	No	
	Arch-hanger	Connection plate and screw	
	Hanger-deck	Bar with inferior plate	
	Active hanger-deck	Bar with special inferior plate	
	Arch-deck		Double plate embedment + exterior prestress
			Verified pinned alternative

#### 4. Economic validation

The San Luis road bridge, Chile (Figure 14) was used to perform an economic cost comparison between the traditional and new proposed strengthening methods. . The bridge is composed of a cross-section with three continuous beams and a reinforced concrete slab (Figure 15). Diaphragms (transversal beams) are provided over the piers, abutments and at mid-span of each span. The deck is supported on three wall-piers, generating four spans (13.5 - 16.5 meters) covering a total length of 60 meters.



Figure 14. General view San Luis Bridge.

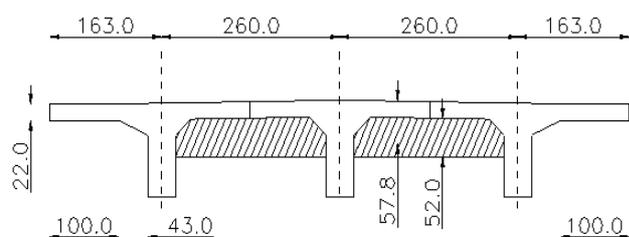


Figure 15. Transverse section (cm units).

The main damage observed (MOP, 2007), corresponds to a problem of scour of the three piers. Consequently, the foundations composed by double track railway piles present a high degree of erosion and rust. As solution to the problem, a strengthening sequence by driven steel piles at each pier was designed, despite the scarcity of vertical clearance.

The implementation of steel piles was done by a pile driver, located on an adjacent embankment, to avoid overloading the bridge. It uses tube piles ("Yoder") of 30 cms in diameter and 12 m in length, which are driven through holes at the slab to a depth of 15 meters. These piles are linked to the piers through a concrete pile cap embracing the old foundations. This process requires a diversion channel of the estuary through a parapet. These were the main activities. Other activities comprise: repair of the the concrete slab, sealing of cracks in the slab of the carriageway and sidewalk, replacement of bearing systems, expansion joints and drains, and channel improvement (construction of flood defense works) (MOP, 2007). The costs of the repair is summarized in Table 2.

Table 2. Budget of the original strengthening project

Item	Total cost [euro]
Infrastructure	148.936,40
Superstructure	104.149,40
Various repair	6.728,60
Preparation of the workspace	6.057,60
Earth moving	360,90

Drainage and protection of the deck	9.607,90
Controls and security elements	15.195,40
Additional works	31.774,40
Environmental measures	2.958,60
Works in the riverbed	54.311,40
Industrial benefit (6 %)	27.137,80
Indirect cost (13 %)	58.798,50
<b>19% VAT (Value Added Tax)</b>	<b>88.543,20</b>

The total cost of the implemented repair was 554.560 Euros.

The alternative strengthening through tied arch includes some activities similar to the original project, specifically associated with repairs to the superstructure and additional activities. On the other hand, the biggest change is associated with the reduction intervention in the infrastructure, avoiding changes in the riverbed. Thus, based on the presented criteria, the strengthening of the bridge was designed with the following solution: s a single centered arch with a square profile with dimensions 75x75x3,6 centimeters, the use of 26 network hangers and three active vertical hangers (one over each pier). Furthermore, it includes a widening and strengthening of the deck road according to the current bridge regulations in Chile. The complete design is available in Valenzuela (2010). The predicted budget for this alternative solution is presented in Table 3.

Table 3. Budget of the new strengthening project

<b>Item</b>	<b>Total cost [euro]</b>
Infrastructure	5.394,83
Superstructure	12.745,16
Various repair	4.419,60
Preparation of the workspace	3.695,88

Earth moving	183,60
Control and security elements	49.776,94
Environmental measures	2.958,61
new strengthening at superstructure	236.292,72
Industrial benefit (6 %)	18.928,00
Indirect cost (13 %)	41.010,80
<b>19% VAT (Value Added Tax)</b>	<b>71.327,20</b>

The total budget obtained corresponds to 446.734 Euros.

The proposed project is economically feasible, with a budget of 20% less than the original project, showing this new methodology as a feasible alternative to traditional strengthening.

Besides this difference on construction budgets may be already considered as relevant enough to propose the alternative strengthening solution, we should take into account additional benefits, some of them not easy to deal with in economic terms, as:

1.- The alternative solution considers an improvement (widening) of the deck, avoiding actual traffic limitations and enhancing safety to the users

2.- The elimination of the piers, will avoid future under water inspections, reducing the life-cycle cost of the bridge. Additionally, the vulnerability of the bridge to scour and seismic hazards will decrease which will translate in reduced feasibility of bridge closure and therefore in less expected user costs from a life-cycle cost perspective. By contrast, future inspection lines will be focused on the hanger system maintenance and corrosion of the metallic elements.

3.- The arch solution will completely eliminate the scour problem and therefore the necessity of any repetitive and expensive additional repair work in the future, thus decreasing the total expected maintenance and repair costs.

4.- The aesthetics of the proposed alternative is much more enhanced compared to the original bridge.

In summary, the total cost from a life-cycle cost perspective will be much lower for the proposed strengthening solution.

## **6. Conclusions.**

It has been presented and verified the feasibility of a new method of strengthening of bridges with scour problems at piers. The method attempts to eliminate the problem by lifting the deck from the original piers and support it with an upper arch, using the original deck (supplemented with an external post-tensioning) as a tied element of the arch system. This allows to eliminate the original piers.

The method requires the incorporation of steel arches, with an arrangement of vertical hangers responsible to lift the deck by jacking, and an arrangement of network hangers to modulate the stress states, providing stability in the lifting stages and improving the in-service performance to asymmetric loads.

The method is defined with a minimum of three construction phases: preparation (placement of elements), lifting (active stressing of vertical hangers) and network (tensioning of the network hangers to allow optimal in-service conditions).

The design criteria is controlled by the requirement to keep the stress state in the deck as close as possible to the original condition of the deck (a continuous beam). This controls the design of other elements and avoids possible failure of the structure. Therefore, the arch becomes the primary design parameter. Similarly, the study of vertical tensioning of the hangers (both in value of the tension force and the order of tensioning) is fundamental to obtain an efficient solution.

The design criteria derives on a set of condition to the arch and the hangers. First of all, the arch profile is higher than in the case of a new network arch. In the case of the hangers, it is

recommended a close relationship of areas between vertical and network hangers. The multi-objective optimization problem seeking to obtain the optimal tension in each hanger and order of tensioning is derived by a genetic algorithm.

The conditions at the longitudinal edges of the deck require a particular study on the joint between arch and deck.

The application of the method to the San Luis Bridge confirms that this strengthening technique is economically sustainable, durable and improves the bridge aesthetics. In the paper, the method is applied to a reinforced concrete bridge composed of longitudinal beams and upper slab. However, the methodology can be easily extended to composite (steel+concrete) bridge decks and also to slab and box-girder concrete bridges. The main limitation could be the dimensions of the cross-section and the total length of the bridge after the removal of the original piers.

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