CONSTRUCTION PROCEDURES FOR LOW COST ARCH BRIDGES IN BOLIVIA

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SUMMARY

As a developing country, Bolivia has limited economical source and technology to build bridges with specific requirements. By the need of fast developing of traffic infrastructure, Bolivian designers, construction companies and administration of the central and local governments prefer to use standardized solutions. On despite of that, trying to solve specific problems and improving the aesthetics of the structures, few arch bridges were constructed in the country. The main challenge, in arch bridge projects in Bolivia was to keep the price tightly competitive with other solutions, considering; the stage of construction, the use of local materials, the equipment available and so on. On the projects presented in this papers, the key point was the methodology used for the construction.

Keywords: *Bolivia, low cost, value engineering project.*

1. INTRODUCTION

The paper will shortly review two of the most representative arch bridges built in Bolivia. The work will focus on the main details of this bridges.

Table1. *Summary of the arch bridges in Bolivia exposed in this paper.*

Bridge Name	Main Span	Year of Competition	Type	Construction Method
Vaqueria	96	2007	Half thought arch	Prefabricated arch segments
Bicentenario	3x50	2011	Bowstring inclined hangers	Prefabricated arch segments

2. VAQUERIA BRIDGE

The first project that is included in the work is Vaqueria Bridge, located near the community of Tacopaya, in Cochabamba, a region at the middle of Bolivia specifically at Coordinates of 761070m east and 8025400m, and an altitude of 3225m above the sea level. The half though arch bridge was concluded in 2007.

The original project was tendered by the Bolivian state through the local Government of the department of Cochabamba, the construction was made by Bartos & CIA SA which requested the technical support for the design company - Vega Consultores-, a Bolivian design company. The originally project was developed on a Prestressed concrete beam and Reinforced concrete slab, a typical system in Bolivia, generally composed by 3 spans of 30m each one, that was located a few meters above the riverbed. The abutments and piers are supported on shallow foundations, and complemented with a retaining wall across the river in order to control the sediments.

During the conceptual design, two conditions were important parameters; the continuous growing up of the riverbed and the necessity of construction of big roads access on the riverbed, reasons that changed completely the initial project and the consequent move of the alignment of the road. The solution proposed was move the bridge to the upstream in order to get advantage of the zig-zag alignment of the road and to connect the bridge with the next existing road, the problem was that this new alignment had a high slope, requiring an increase of bridge deck level in 12 meters and was also required 14 meters more in length. After analysing several alternatives, a feasible solution was a half though arch, which reduced significantly the volumes of construction, thanks to the advantage of the shale rock is near the surface at both ends of the arch, which is favourable from the point of view of transmission of the horizontal forces. The engineering challenge with an arch solution in general is to develop a construction method of the arch that did not increase significantly the cost of the project, keeping it competitive with the initial project.

The new design consisted in an arch of 96m span, the longest in Bolivia, with a total length of the bridge of 114.30 meters. The design was made, as a Value Engineering Change Proposal, by prefabricating 60 meters of the central frame of the arch which included: part the arch and the longitudinal and transversal beams. The main features of the bridge are the following:

- Design code — AASHTO LRFD 2004
- Total length — 114.30 m
- Arch span — 96 m
- Prefabricated arch length — 60 m
- Strength of arch and deck concrete: — 35 MPa
- Strength of block-foundation concrete: — 21 MPa
- Reinforcement steel — Grade 60 – 420 MPa yield strength
- Post-tensioning tendons — Grade 270 – 1860 MPa ultimate strength
- Arch Geometry: — Circular, 73.04 m of radius
- Arch length span to raise: — 5.4
- Number of hangers — 6 each side
- Type of hangers: — Regular PT 12.7 mm strands
- anchored on regular external PT anchorages grouted with cement and expansion additive.
- Number and width of lanes — 2 lanes / 3.65m each
- Distance between arch planes — 8.70 m
- Depth of the deck — 0.95 m

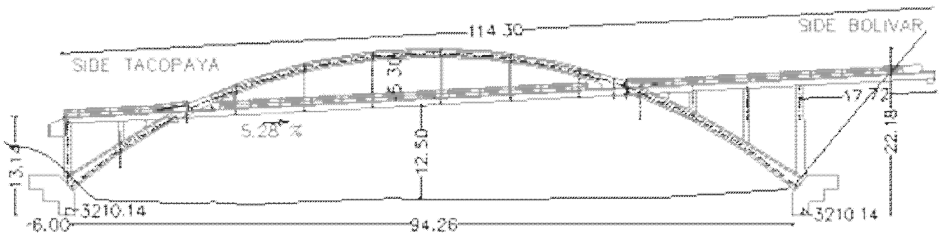

Fig. 1. *Longitudinal view of the Vaqueria Bridge.*

The substructure was defined by; two reinforced concrete stepped blocks casted against shale rock, two for each arch side. Transversally, those blocks are connected with vertical reinforcement concrete walls, filled with compacted soil and cyclopean concrete.

The main superstructure longitudinally is composed of:

- Two planes lateral arches of 96 meters of span, composed by an "I" section of variable height from 2.0 meters to 1.20 meters.
- The deck is composed, two longitudinal beams with rectangular shape with 0.95 centimetres of depth that are part of the sidewalk. These longitudinal beams are connected with the steel hangers to the arch at the central frame and with the columns at the lateral frames. Transversally, both longitudinal beams are connected with post-tensioned concrete beams at the central frame and reinforced concrete beams at the lateral frames.
- The slab is made by reinforced concrete 0.25 meters depth, which was divided in two parts, one precast 0.15 meters half slab and the top subsection "cast in-place" give a monolithically connection between them.

The construction sequence of the bridge consisted in the following steps:

- Reinforcement placing and casting of foundation blocks
- Casting of lean concrete for the temporary supports
- Reinforcement placement and casting, in parallel of lateral frames, central frame and half depth precast slabs.
- Sequential Tensioning of vertical hangers
- Uplift of central frame
- Reinforcement and pouring of joint between lateral and central arch.
- Post-tensioning of longitudinal tendons.
- Placement of precast half depth slabs
- Pouring of the upper in-situ half depth slab
- Installation of prefabricated railings

To make possible the lifting of the central frame, the structure was provided with 8 reinforced concrete corbels, one at each corner of the central frame and the other 4 at the end of the lateral frames. Each pair was aligned in order to allow a straight connection between the central frame and the lateral frame through the lifting tendons. Such alignment was slightly deviated because the central span was cast horizontally, and needed to rotate on its position in order to get the specified slope. The lifting tendons were composed of 12 strands of 12.7 millimeters. These four uplifting tendons were designed to support the total weight of the central frame, plus 30% of impact, without overpassing the 70% of ultimate strength of the strands. The uplift tendons were tensioned by four regular hydraulic jacks of 200 tons, supported above the lateral frames. The weight of the central frame was 420 tons that was lifted in steps of max 20centimetros, according to the range of the jack, this procedure took around 4 days. After the central frame reached its final position, a wet joint was needed to be materialized in order to get a continues structure. The basic concept of the design was the modification of the structural conditions from the lifting stage to the service stage, by the redistribution and change of direction of forces once the temporary supports were released. At that moment, the vertical forces of the temporary supports should have been transferred into compression forces along the complete arch, in this sense, the main aspect that has been taken into account was to guarantee a solid and proper fill of spaces between parts and give continuity to the reinforcement. In order to achieve a proper confinement in the joint, transverse reinforcement was left inside both parts of the joint to be bent and closed after the overlapping of splices..

However, additionally to the mechanical and geometric continuity of the structural arch, the Prestressed Tendons (PT) for the lifting were left inside the structure, beside this, the longitudinal tendons of the lateral spans were extended beyond the joint inside the central frame arch, this introduce additional compression stresses in the joint.

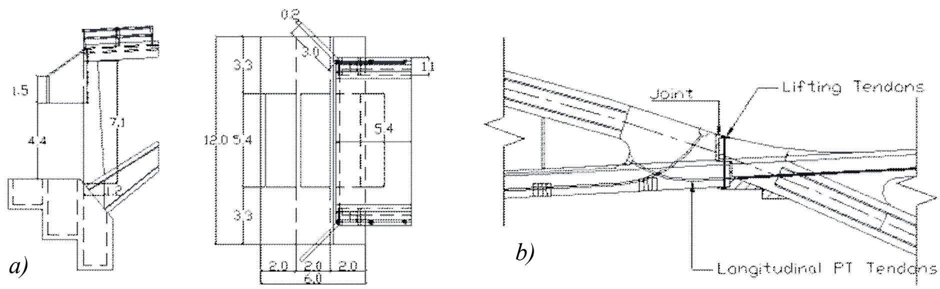

Fig. 2. *a) Detail of the foundations of the arch and the deck;*
b) Detail of the joint of the arch by construction sequence.

The total cost of the project in the year 2007 was around 577 thousand American dollars that was 10% larger than the original project, slightly below the legal allowed limit according the contractual requirements. The main item volumes are the following:

- Concrete for foundations: 371.74 m^3
- Steel for foundations 23.11 tons
- Concrete for arch and superstructure 648.19 m^3
- Steel for superstructure 79.36 tons
- PT steel 3.36 tons
- Hangers 71.78 m

a) *Bridge at lifting stage*

b) *Bridge at final stage*

Fig. 3. *a) Bridge during the lifting; b) Bridge in service.*

Some issues were found during the erection of the bridge that are worth to be mentioned:

- Since the PT hydraulic jacks that were used for the uplifting were different, the velocity of the piston was also different, so at the beginning of the first try, before the operations the hydraulic jacks were correctly synchronized because at the beginning of the operation one of the uplifting hydraulic jacks, took more load than the other jacks, generating a small twist of the central frame and consequently an unbalanced load condition that damaged one of the concrete supports. So it demanded a double-check in the control of lifting for the second try, which means a more accurate control in displacements of the central frame on each lifting point and the definition of the maximum difference in height between each pair of lifting points in the transverse direction.
- The damage on the lifting concrete corbel required a slightly modification the hole for the lifting tendon. Which left one rebar exposed, that at certain time as the slope of the central frame was changing, one of the uplifting tendon became in touch with the exposed rebar wearing some strands until breaking of some wires. The main problem in such design was the lack of an emergency additional anchorage which may allow the replacement of tendon.
- It was detected that the repetitive load application and the fact that the distribution of loads between strand was changing in the multiple steps in tensioning, caused a differentiated effect on wedges, having found some of them broken, which needed to be replaced but were not properly checked during the lifting.
- Instead of sand jacks or other special device, which may allow a gradual release of forces in the temporary columns, this activity was performed through manual chopping of concrete, so the release was not as smooth as expected.

The method applied for lifting of the central frame, resulted satisfactory with small difficulties at the beginning and at the end of the procedure, as mentioned above, but in order to improve the procedure used in the Vaqueria's bridge construction, as well as its safety conditions during its uplifting, it would have been recommended to include some special details in the lifting component of the project:

- To include spare anchorages that allow the installation of an alternative lifting tendon in case of emergency.

- To use special wedges and anchorages, proved for repetitive loads and revise periodically the state of wedges and forces on each strand, in order to guarantee a uniform distribution of forces.
- To use special devices at the support points which allow a smooth release of loads.

3. BICENTENARIO BRIDGE

The second bridge included in this paper is called the Bicentenario Bridge, located in the city of Tarija, situated at the south part of Bolivia near the border with Argentina, more accurately at UTM coordinates 320190m East, 7617190m South at zone 20. The total length of this bridge of is 150 meters, divided in 3 arches of 50 meters, each the bridge was concluded in 2011.

The original project was tendered by the Bolivian state through the Local Government of Tarija city, and constructed by the “Asociacion Accidental El Peregrino” which also developed the design of the project. The main issue that demanded a Value Engineering Change Proposal was the update of soil studies that recommended the use of deep foundations instead of the shallow foundation included in the base line design. In order to keep the feasibility of the project, some additional requirements about optimizing the construction costs and time were two important parameters on the conceptual design.

The arch and the deck were constructed by precast segments supported in four concentrated temporary steel supports which held the weight of the prefabricated arch segments and longitudinal beams. At its time the longitudinal beams supported the transverse beams through corbels.

The main characteristics of the bridge are the following:

- Design code: AASHTO LRFD 2007
- Total length 150 m
- Arch span 3x50 m
- Concrete strength of PT beams 35 MPa
- Concrete strength of arch and deck: 28 MPa
- Concrete strength of piles and pile cap: 21 MPa
- Reinforcement steel Grade 60 – 420 MPa yield strength
- Post-tensioning tendons Grade 270 – 1860 MPa ultimate strength
- Arch Geometry: Parabolic geometry of the arch
- Arch length span to raise : 4.76
- Number of hangers 14 each side with an “X” arrangement
- Type of hangers: galvanized, greased and sheathed
- PT strands anchored on regular external PT anchor blocks, grouted at the ends with cement and expansion additive.
- Number and width of lanes 3, 3.6 m each
- Depth of the deck 1.12 m

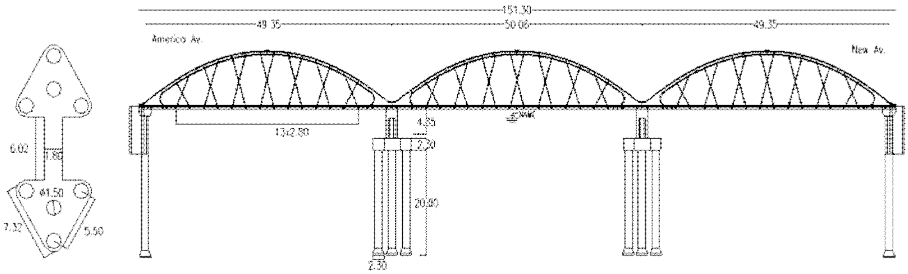

Fig. 4. *Plan view of the pile head (left) Longitudinal view of the arches (right).*

The substructure was defined by reinforced concrete piles of 1.50 m of diameter in a double arrow shape pile cap on each pier.

Longitudinally the superstructure is composed by:

- Three parabolic arch spans of 50 meters each, arranged in two lateral planes with a distance between them of 11.4 meters, the section of the arches is a box section of 1.10 meters width and 0.80 meters height. The box section is solid in the neighbourhood of the hangers.
- Two longitudinal beams in the deck with a rectangular cross section of 0.40 meters by 0.75 meters are part of the sidewalk. These longitudinal beams are connected with the PT hangers arranged in X shape. The X arrangement of the hangers aimed to an aesthetics condition more than a structural improvement in the structural performance, therefore, although an important improvement was detected, but in the authors perception the longitudinal behaviour of the arch is not the same as a network arch bridge.
- Transversally, the longitudinal beams are connected with precast post-tensioned concrete beams.
- The slab is made in reinforced concrete of 0.30 meters depth, which was divided in two parts, one precast 0.15 meters half slab and the top half "cast in-place" give a monolithic connection.
- The construction sequence of the bridge consisted in the following stages:
- Precast of the bridge in segmented elements: Arch, longitudinal beams, transversal beams.
- Drilling of the soil, reinforcement placement and pouring of the concrete for the piles
- Construction of the pile caps and piers.
- Assemble of the first frame and casting of arch joints
- Tensioning of longitudinal tendons and hangers.
- Assemble and tensioning of the second frame
- Assemble and tensioning of the third frame
- Placement of precast slabs

- Reinforcement placement and casting of the top half slab
- Adjustment of strand stress
- Installation of railings.
- The cost of the project was 2.39 million American dollars, in the year 2011.

a) Bridge at assembling stage. *b) Bridge at final stage.*

Fig. 5. *a) Bridge during the lifting; b) Bridge in service.*

Relevant aspects of the project:

The bridge has been designed integral with the substructure, so some special actions were included in the staged construction process in order to relieve some stresses, some initial shortening effects before the closure of joints and elastic shortening due to longitudinal post-tensioning were reduced by the use of temporary sliding plates, the initial shrinkage, which has a fast growth at the first ages of the concrete, was reduced through the hardening of concrete of the segments that were casted 8 months before they were installed; a similar effect was expected on creep due to the fact the creep effect is lower when the concrete elements are more mature. The casting of joints between super and substructure was planned at night when the super structure was contracted by the low temperature. The pouring of cast in place part of the slab was planned after the connection between the superstructure and substructure, in order to transfer a portion of arch horizontal forces into the substructure pushing it outwards to compensate “partially” the shortening effects. The remaining shortening effects are expected to be absorbed by structural configuration of the abutments with one row of piles and soft soil conditions around it, so special reinforcement was applied in the top half of the piles to absorb this stresses. The deck expansion and contraction effects due to the temperature and the elastic elongation due to the horizontal components of the forces were designed to act against the backfill, and also getting advantage of the geometric properties of the arch, in which the elongations would modify the geometry of the arch pulling up the deck, inducing a modified geometry that would reduce the strut behaviour of the longitudinal beams in the deck. Of course, the effect of each concept above mentioned was not so evident in terms of magnitude of demands force, so all those effects were introduced only in the staged construction model through an “in-house software” which has been developed for analysis of cable stayed bridges [1] and takes into account the geometric nonlinear staged construction and the geometric nonlinear analysis by a geometric stiffness array for bar elements and cable stay elements.

Another issue in the project was the determination of tensioning forces in hangers, in order to get a uniform distribution among them and individual strands. Since the structural system of the arch is very flexible during the tensioning of hangers and the installation of the hangers are sensitive to the deformation of the arch by the short lengths, a calculation method that allows the tuning of hangers with the available equipment and the tensioning in sequence was developed. The method considers the use of N+1 independent 2D linear elastic models, being N the number of tensioning steps. Each model is composed with all the structural elements that are acting at the specific tensioning stage, it is possible to determine the effect on all the structural components that are already installed at the specific stage. On each model a unitary strain value is applied on the group of hangers that are being tensioned simultaneously in the same stage, in the case of the "Bicentenario Bridge" they were two hangers in each sequence because they were available two hydraulic jacks. On each model, the resulting forces on each hanger is recorded for both, the passive hangers and the active hangers, the displacement of a representative point, for instance the central span, is recorded as well. Of course compression values are expected in tendons near to the active hangers, but these should be respected, as they will become reducing forces.

With this information it is possible to check the evolution of forces through the different stages, pointing out that 0 is the values applied to hangers that are not installed yet at a specific stage:

$$f_{11}\,k_1+f_{12}k_2+\ldots+f_{1j}k_j\;+\ldots\;f_{1N}k_N = F \tag{1}$$

$$0 \quad + \quad 0 \quad +\ldots+f_{ij}k_j+\ldots\;f_{2N}k_N = F \tag{2}$$

$$0 \quad + \quad 0 \quad + \; 0+\ldots\ldots+f_{NN}k_N = F \tag{3}$$

f_{ij}: Force at the hanger i in the structural model or tensioning stage j.

k_j: Coefficient to be applied on structural model j

F: the final force, equal for all the hangers

Since F is unknown, to solve the system, it is important to include an additional equation, so this is where the last model is used, considering a fictitious condition in which only the concrete elements are activated as dead loads, as if the temporary supports were released before the installation of any hanger. We can have in this case the hypothetical displacement that need to be reversed, thus the lacking equation is as follows:

$$d_1{*}k_1+\ldots d_j{*}k_j+\ldots+d_N{*}k_N = D \tag{4}$$

Being d_j: The displacement obtained in the model j due to unitary shortening

D: the sum of the hypothetical displacement of the characteristic point with maybe plus a camber value defined apriori. The tensioning forces will be determined as fii*ki.

These values obtained with such model resulted accurate enough and were introduced on the staged construction model to verify if these forces were not over demanding the structure, and after that, they were applied with two multi strand hydraulic jacks, controlled by pressure and deflection of the arch with values obtained from the staged model. It is worth mention that the tensioning process was performed before the slab was cast, thus the forces on hangers were small, so the control in deflections of the arch resulted were more reliable.

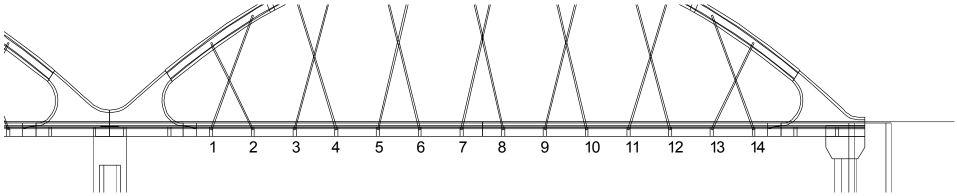

Fig. 6. *Name of the cables for the sequence of tensioning.*

The procedure of tensioning with multi strand jack is fast but does not guarantee a uniform distribution of tension among the strands, so it was necessary to include a final tuning on strands that was applied after the slab was cast. To do that, since the stresses on cables had changed due to the slab, a measure of forces was statistically performed in order to confirm the theoretical values, with those real values a force by strand was defined, slightly above the calculated ones to ensure that all the strands are approximately at the same stress.

Table 1. *Resume of the tensioning sequence.*

Sequence of prestressed	Cables		Force [kN]	Stress [kg/cm^2]	Deformation [cm]
1	4	11	206	53	S/D
2	7	8	276	71	25
3	4	11	213	55	37
4	2	13	97	25	5
5	3	12	117	30	6
6	6	9	73	20	4
7	5	10	68	20	3
8	1	14	67	20	2

Additionally to those measures, the lower anchorage was provided with a double anchor plate and the strands were left long enough to allow the be coupled in case that a re-tensioning were necessary in the future.

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