Summary
Reconstruction of the damaged steel bridge in the town of Albenga was necessary as a consequence of the flood of November 1994.
Constraints both due to the dimensions and of a historical-environmental nature demanded an unusual design and the realization of a tied-arch bridge spanning almost 100 metres. The aim of reuniting the divided town as soon as possible was achieved in 10 months with a steel construction.
This paper deals with the criteria that determined the choice of structural design, the static and dynamic analysis performed, the main stages of construction, erection, checks and testing of the bridge.

Introduction
The flood of November 5th 1994 irremediably damaged the old three-arched steel bridge, built at the beginning of the 20th century, which crossed the River Centa in the town of Albenga.
It was necessary to build a new bridge. The bridge span was 100 metres and the maximum allowable deck depth was 180 cm. No piers were permissible in the river-bed, thus the choice was between a cable-stayed or arch bridge. The arch bridge solution was selected because:
a. it was both less expensive and more suitable for a 100 metre span;
b. the arch of the new bridge perfectly encloses the three arches of the old bridge; moreover this allows comparison of arch bridges from the beginning and end of the century.
This approach permitted the design of a slender trussed arch with a deck depth of 140 cm, suspended by cables in the center of the carriageway.
The other pressing need was the speed of construction and the realization of an avant-garde structure, which is why steel was predicated, permitting construction of the 18 segments of the bridge in a workshop, while abutments were built on the site, and making assembly possible without troublesome supports.
This led to a significant reduction in both the time and cost of construction.

2 Bridge structure
The structure consists of an arch which carries a box-girder, all of steel, to form a tied-arch (the box girder is the tie of the arch): in this way both the foundations and the abutments are simpler and cheaper.
The bridge spans 98 metres and the arch is 21 metres high.
Figs. 1 and 2 show the bridge in its surroundings and a partial view; in fig. 4 the transversal section is shown.
The arch is made from three tubes 609.6 mm of diameter and 40 mm thick, welded together in order to form 12 metre long section, which were subsequently calendered.
The main tubes are joined to each other through smaller tubes 139.7 mm in diameter and 12.5 mm thick, to form a very slender trussed arch.
Every 5 metres, a cable supports the deck; it is fixed with a standard strand socket to the arch and by a zinc-poured socket inside the girder. Cables are constituted from full locked coil strands, 65 mm in diameter encapsulated in a polyethylene pipe. Strands are formed from 111 wires of size 3.58 to 6 mm, with a minimum ultimate stress of 1570 MPa. They are pre-stretched to 50% of the breaking load, in this way they have an ideal modulus of elasticity E=159800 MPa.
The deck is formed from a box-girder with trussed diaphragms every 5 metres which include the connections of the cables.
Linked to the diaphragms there are side corbels which support the concrete floor.
The corbels work in a strut-tie way to support eccentric loads; they are lightened with holes to increase their slenderness.
The box-girder is the torsionally stiff element of the structure: it supports torque due to eccentric loading, while vertical loads are taken by the tied-arch.
The bottom of the box-girder has a hole every 5 metres between the diaphragms; these openings give aeration to the inside parts of the box in order to prevent condensation which could compromise durability.
The box-girder is fixed on PTFE bridge bearings, two for each side.
The roadway is made from a concrete slab 25 cm thick, designed in the longitudinal direction which is the sense of tensile forces due to tie behaviour; the slab is continuous over the diaphragms and corbels which occur every 5 metres.

3 Structural analysis
The bridge is a tied-arch with suspension cables only along the central plane, thus eccentric loads are borne by the box-girder.
The structure is a composite bridge, because the slab is connected to the steel box-girder by welded-stud shear connectors.
Firstly the structure was dimensioned with approximate analysis to determine arch and deck stiffness in the vertical plane.
A torsional analysis permitted us to dimension the section and plate thickness of the box-girder, making the Bredt assumption of constant shear flow, either from a resistive or from a deformative point of view.
After that, the principal elements were dimensioned and three-dimensional finite elements analysis began, refining model with successive analysis. Inertial characteristics of the steel box-girder were assigned to central beams, while the slab was considered only after having been connected. Exterior beams have been used to insert eccentric loads but don't have their own geometrical and inertial properties, only masses. Crossframes and corbels are modeled with infinite stiffness and positioned as in reality. The arch was modeled taking account of all component steel tubes, each one with its own geometrical characteristics. Firstly a dynamic analysis was performed, to estimate natural frequencies and general flexibility. This dynamic approach and several static analyses were repeated a number of times in order to optimize stiffness and deformability of the bridge structure, especially out of vertical plane. Fig. 3 and Table 1 show the final result of the dynamic analysis, with mode shapes and natural frequencies: the fundamental mode is the vibration of the single arch out of the vertical plane with return period 1.147 sec. (0.871 Hz). Static analysis has been performed in three phases:
- phase 1: just the steel structures carrying their own weight as well as the weight of the concrete slab;
- phase 2: composite structure supporting dead loads, shrinkage and creep;
- phase 3: composite structure with live loads.

These three simple phases were in reality considerably more complicated. In fact, during construction, the first phase recorded stress paths of a 4 spans continuous beam before the arch began to work; moreover, once the concrete slab had solidified, lowering the deck onto provisional supports permitted pre-stressing of the concrete, limiting cracking in service, when the entire deck is subjected to tensile stress.

In the third phase, that of live loads, 25 different loading positions were considered in order to maximize bending moments in all structural elements, plus 25 loads for shear and another 25 for torque. A static analysis was performed for wind action, because the heterogeneity of the structure precludes regularization of vortex shedding and structural stiffness prevents aeroelastic phenomena. Torsional divergence of the deck is not possible because of its stiffness. These conclusions were obtained using various national and European codes.

3.1 Dynamic analysis
Analysis was performed on the three dimensional model using finite element techniques, in order to obtain the main eigenvalues and eigenvectors of the structure. The model included all masses and stiffness as in the real structure. Masses have been distributed on beam elements. The mode shapes and natural frequencies of the structure are visible in Fig. 3 and described in Table 1.

3.2 Seismic actions
The bridge site is not considered a seismic area according to the national code, but a seismic analysis was carried out anyway. The unusual structure called for a dynamic analysis through a modal superposition using the response spectrum method. The design spectrum used has a spectral acceleration defined as \( a/g = C \cdot R \cdot \varepsilon \cdot \beta \cdot I \) for horizontal seismic action. The parameters related to the construction requirements are:

- seismic intensity coefficient: \( C = 0.07 \)
- response coefficient (function of time period of mode of vibration considered):
  - for \( T > 0.8 \) sec: \( R = 0.862 / T^{2/3} \)
  - for \( T \leq 0.8 \) sec: \( R = 1 \)
- foundation coefficient: \( \varepsilon = 1 \)
- structure coefficient: \( \beta = 1 \)
- seismic protection coefficient: \( I = 1.4 \)

In compliance with the seismic code for bridges, analysis was carried out considering only masses for self-weight and dead loads. Analysis was limited to the first 5 mode shapes and results were combined using the square root of sum of squares method (SRSS).

3.3 Non-linear analysis
A non-linear analysis of the global three dimensional model was performed in order to control the sensitivity of the arch to second order effects. Only geometric non linear effects were taken into account. Geometric non linear analysis with finite element method consists in including the geometric stiffness matrix \([ Kg]\) in stiffness calculations and solving iteratively series of linear procedures up-dating the stiffness matrix each time. The matrix \([ Kg]\) is calculated for each element from the axial force acting on it and it is summed to the conventional...
elastic stiffness matrix \([Ke]\). Calculation of \([Ke] + [Kg]\), assemblage and system solution are repeated, checking for convergence through displacements and forces norma.

Non linear analysis showed that the structure was well dimensioned and it didn't suffer instability: the algorithm stopped after few iterations, converging to the final solution, which was nearly the same as the linear-elastic analysis.

### 3.4 Local analysis

Side corbels were initially dimensioned with a trussed model, then analysis continued using the finite element method. Shell finite elements were used for modeling the web of the corbels including holes and voids; shell elements have six degrees of freedom per node, they were necessary to couple beam elements used to simulate stiffeners and flanges. With this analysis it was possible to rigourously design stiffeners, flanged connections to the box, splice plates and bolts. The cross frame was analyzed with trussed models assuming that the loads from the deck were in the least favourable positions.

The concrete slab is continuous over cross frames and corbels; the impact of vehicles was considered on the first span and reinforcement is designed to account for two-way behaviour.

The allowable stress design of the concrete slab considered stresses coming from:
- pre-stressing due to imposed displacements;
- tensile longitudinal stress due to arch thrust;
- transversal tensile stress due to strut-and-tie model in side corbels.

The sum of these forces was checked, along with a crack control.

### 3.5 Time history analysis

The vibrations of bridges due to moving traffic are important for two reasons: firstly, the stresses are increased above those due to static-load application; secondly, excessive vibration may be noticeable or even intolerable to persons on the bridge.

The first effect is accounted for by increasing the live load of the dynamic factor given by the code as a function of bridge span. On the other hand, to be sure that structural acceleration is tolerable to users, a time history analysis is required.

The analysis considered constant moving forces, simulating the passage of a single heavy vehicle with a constant speed along the deck, assuming that the mass of the vehicle is negligible.

Based on these assumptions the time-varying force corresponding to a moving vehicle is:

\[
f(t) = F \cdot \sin(n \cdot \pi \cdot v \cdot t / L)\]

where:
- \(F\): moving force
- \(v\): constant velocity
- \(L\): bridge span

The bridge was studied with two moving loads of 200 KN each, crossing the bridge at a constant velocity of 55 km/h, corresponding to \(V_0 = 15.24 \text{ m/s}\), the speed at which the relative maximum of response is reached [2].

Analysis began at the point when the vehicle enters the bridge (\(T_0 = 0\)) and lasted for 20 seconds, considering that forced vibration is applied only while the force is on the span, \(T_1 = 6.43 \text{ sec}\), after which natural damped free vibration occurs with initial conditions corresponding to the moment when the forces leave the span.

The overall equivalent viscous damping adopted is \(\xi = 0.01\), considered the most probable for this type of composite structure. It is obtained by the sum of material damping plus structural plus bearings damping:

\[
\xi = \xi_M + \xi_S + \xi_F = 0.003 + 0.004 + 0.0024 = 0.0094
\]

Integration time-interval adopted is \(\Delta t = 0.1 \text{ sec}\), which is approximately the recommended value of one tenth of the time period of the highest mode (1.147 sec).

The Result Acceleration has a minimum of \(-6.42 \text{ cm/sec}^2\) after 6.6 seconds from the beginning of forced vibration, when the vehicle leaves the span (fig.5).

Codes of practice give an indication of human perceptibility thresholds for vertical harmonic vibrations in the frequency range 1 to 10 Hz:

<table>
<thead>
<tr>
<th>Level</th>
<th>Acceleration</th>
</tr>
</thead>
<tbody>
<tr>
<td>just perceptible</td>
<td>3.4 cm/sec²</td>
</tr>
<tr>
<td>clearly perceptible</td>
<td>10 cm/sec²</td>
</tr>
<tr>
<td>disturbing</td>
<td>55 cm/sec²</td>
</tr>
<tr>
<td>intolerable</td>
<td>&gt; 180 cm/sec²</td>
</tr>
</tbody>
</table>

It is clear that vibrations due to passage of a heavy vehicle in service use will be perceptible but far from being unpleasant.
3.6 Structural tests
Controls were carried out using the allowable stress method. Resistance and stability controls were carried out on all elements during every phase of the erection and service. In the first stages of erection, when the slab was not in place, the resistance section was the steel one alone. Maximum stresses in the flanges and the web were compared with stresses allowed for grade Fe 510 steel, of which the bridge is entirely built. Also local plate buckling was controlled for all panels. The sum of stresses was used for allowable stress design and to control local buckling of the webs of the box-girder. At this time, as well as its own weight, concentrated loads were present due to the temporary piers used for erection of the arch with their supported loads (fig. 6). Final tests provided the sum of stresses due to following three stages:
- first stage: continuous beam over temporary piers; only steel box-girder resistant, with the weight of the steel and the concrete slab;
- second stage: tied-arch with superimposed effects of imposed displacements, dead loads and shrinkage. Now the resistant section of the deck is a composite, the concrete slab being reduced with a factor \( m = 18 \) in order to account for long term effects, while arch elements have only the properties of their steel section;
- third stage: tied-arch with composite deck and live loads positioned in 75 different ways. At this stage the resistant section of deck is a composite with a concrete reduction factor \( m = 6 \) because loads are instantaneous. Superimposition of stresses was done by means of an automatic routine for each element, testing the composite section with the sum of stresses of stage 1 plus stage 2 plus those of stage 3 relative to the maximized force. In this way every element is tested three times, subjected to maximum bending, shear and torque forces, either for resistance or for stability. The same was done for forces coming from wind blowing on the loaded bridge and from seismic actions. The last test carried out on the bridge considered the loss of a stay cable, loss due to hypothetical failure or substitution of a cable. The resistance and the deflection of the bridge were tested, in order to have sufficient reliability.

4 Construction
Considering that it was necessary to build the bridge in a short time, optimization concerned design and erection, working simultaneously in the shop and at the site.

4.1 Shop production
The steel box is assembled from 8 members. Every member is made from two inclined girders and a bottom plate connected slip-critically by M27 bolts. The box-girder is completed by the 17 cross frames, which support the end fittings of the cables and the side corbels. Repetition of the elements and the conceptual simplicity of their assembly, entirely bolted, permitted rapid fabrication and site construction. The arch was entirely shop pre-assembled and welded to form 9 members. Members have coupling on the edges to recall on site the exact position of pre-assembly, corresponding to the theoretical configuration. The helical lock-coil strands were entirely shop fabricated and tested; they arrived on site rolled up as large spirals about two metres in diameter. All members fabricated in the shop were blast cleaned and sent to the protection/coating cycle. The whole protection was realized in the shop in order to shorten the construction time; field coats were limited to revisions made necessary by erection procedures.

4.2 Erection
The erection design was continuously modified in order to optimize time and to use human resources and machines as they were available; it consisted of the following stages:
- ground composition of box members, completed with cross frames and side corbels;
- positioning of box-members on the new abutments and the piers of the old bridge, used as provisional supports. All the liftings were done using a truck crane with a capability of 250 tons;
- positioning of the precast-concrete plank and reinforcement;
- casting of the central part of the deck, leaving 3 metre joints over the provisional supports: this partial slab was used for service, in order to cross the bridge with various vehicles. Joints left over piers limited stress due to shrinkage and tensile force coming from negative moments from the continuous beam;
- erection of two provisional trussed supports on the slab, to hold up the three members forming the arch;
- field assembly of the 9 members of the arch and their welding into three main parts;
- lifting of the three parts of the arch into their final position, done with two truck cranes, positioning them over the first box-girder member and over the trussed supports (fig. 6); elevation positioning was controlled instrumentally in comparison with the geometric computer model. Welding on elevation of the three pipes of every joint;
- lowering of the deck using jacks onto the old piers and removing thickeners until the theory profile of stage 1 was
reached (reduced to consider its own weight at the time);

- erection of cables and pre-loading with theoretical forces coming from the equilibrated configuration;
- lifting of the deck onto the provisional supports and removing of thickeners to allow the whole tied-arch structure to work;
- casting of last parts of the slab and grouting of the joints;
- adjusting of strand forces to their service load and geometrical control of the bridge profile. Adjustment of stay-cables was done in two stages: firstly with 60% of theoretical load of equilibrated configuration, secondly with 100%. In this way a structure equilibrated with theory stage 1 was obtained;
- finishing;
- static and dynamic testing.

5 Construction checks
A variety of checks was carried out at all stages of construction. These checks are of different types:
1. dimensional control carried out during fabrication, on plate thickness and the geometry of shop-assembled and field-assembled parts.
   An elevation check of the bridge and its geometry during erection. This was done during different load and constraint conditions;
2. inspection of materials, bolts, welds: the Italian Welding Institute was responsible for the control of all materials, directly or through certification, to control dimensions and to control welds executed either in the shop or in the field; design bolt-torque was checked by the designer.
   Italian Welding Institute checks consisted of:
   - presence in the shop to program and check the construction of the box-girder and corbels: to execute complete penetration welds, in line automatic submerged-arc welding was used; for all other joints semi-automatic continuous wire welding in gas protection was used;
   - shop presence to configure and control arch construction. Joints of tube of ø 609.6 mm and 40 mm thick were rolling welded with TIG process on the first pass, manual welding with basic covered electrode and submerged-arc for other passes. Complete-penetration welds of connecting pipes have been realized with manual welding using basic-covered electrodes; all other connections were done with semi-automatic continuous wire welding.
   Particular attention was necessary to verify alignment of the edges of the tubes of ø 609.6 mm. The tolerance of commercial tubes caused differences in edge level which required grinding to guarantee fusion of the edges (DIN code non-welded pipes admit large tolerance either in diameter or in thickness).

6 Static and dynamic testing
Ten trucks, 400 kN heavy each, were used for static testing in three different positions, in order to check forces due to maximum load on the whole structure, maximum semi-symmetric load for the arch and maximum torque for the box-girder:
1. 5 + 5 trucks located at the center of the bridge;
2. 4 + 4 trucks located only on the first half of the bridge;
3. 10 trucks located only on a side;

In order to compare theory and testing, an agreement with a test-engineer led to measurement of deflections of both the deck and the arch in the middle and at a quarter of the span, elongation of the bridge at the bearings, transversal deflections at the center of the bridge, stresses in steel in the lower flange of the box and on the lower main tube at a quarter of the span and at the fixed-end.
The axial force in a stay-cable was also measured.

In tab. 2 and 3 measured and calculated values are compared; the following comments may be made:
1. the measured deflections are in good agreement with the calculated ones;
2. residual strains were very small, thus confirming that the structure has a good elastic behaviour;
3. the semi symmetric load, the most critical for the arch, gave deflections close to those calculated;
4. the max torque load exhibited a torsional stiffness of the box-girder greater than the theoretical one, confirming that further to Bredt's assumptions for torsion, the presence of the concrete slab and its constraints contribute to rigidity;
5. concerning forces in the stay-cables, quantities measured were a little inferior to those calculated;
6. stresses measured in the flange of the box-girder were lower than calculated because of the contribution from the plate between the flanges;
7. stresses measured in the arch were lower than calculated. The reason lies in the proximity of the strain-gauges to the connections with secondary elements, where local stress flow changes the global effect in the element.

For the dynamic test a single 400 kN heavy truck was used, crossing the bridge and driving over a bump 12 cm high and wide.
Another measurement was made with the same truck braking in the center of the bridge, longitudinally and transversally.
For the last test, the arch was stressed orthogonally to its main direction, through a steel rope connected at the quarter span of the arch and pulled by a crawler crane located in the river: the rope had a sample with a calibrated rupture load of 40 kN, which, breaking, led the arch to vibrate.

During these dynamic loads structural vibrations were registered, through accelerometers positioned either on the deck or on the arch.

From the signals, Fourier transforms were carried out and, hence, power spectral acceleration allowed frequency peaks and their correlated principal mode shapes to be found. In figs. 7 and 8 the resulting spectral accelerations of the arch are shown.

Important confirmations of the structural model came from this analysis: the mode shapes measured and their periods were very similar to those calculated (tab. 4 and 5). The maximum registered vertical acceleration at mid-span, on the side, was 16 cm/sec², when the loaded truck passed over the bump with one side wheel only, an acceleration considered clearly perceptible but not disturbing. Instead a vertical acceleration of 80 cm/sec² was registered when the truck passed over the bump with all the wheels; this last acceleration is in the range of those considered unpleasant.

Clearly, accelerations due to passages without bumps are much lower and similar to those calculated (6.42 cm/sec²), which are in the range of perceptible but not disturbing accelerations.

Testing on materials, construction, static and dynamic loading allowed the test-engineer to verify the structural safety and to open the bridge.

The test-engineer decided, in agreement with the designer, to include the controls planned to guarantee serviceability in the testing certificate.

7 Materials

The materials prescribed were the following:

Main structure: steel type Fe510 C UNI 7070 for thickness ≤ 20 mm.
steel type Fe510 D UNI 7070 for thickness > 20 mm.

The type of steel used for construction of the arch was St. 52.0 in compliance with DIN 2448-1629

Angles and plates: steel type Fe510 B UNI 7070
Stud connectors: steel type St37 3k DIN 17100, f_yk = 355 N/mm²
Bolts: in compliance with UNI 5712, class 10.9 UNI 3740,
main girder: slip-critical with μ = 0.3

Welded joints: in compliance with CNR-UNI 10011/88 and IIS specifications

Diaphragms and crossing: shear working

Stay cables: locked-coil zinc coated strands ø 65 mm.

Concrete:
for piles: Rck = 25 MPa
for foundations: Rck = 30 MPa
for abutments and slab: Rck = 40 MPa

Steel bars with enhanced bond for concrete: FeB 44k

7.1 Material incidences

Bridge span between restraints: 98 metres
Width: 15.4 metres
Total steel for the deck: 350 ton
Total steel for the arch: 248 ton
Total steel incidence: 390 Kg/m²
Concrete slab 25 cm thick: 3.85 m³/mt, for a total of about 380 m³
Dead loads: 200 Kg/m², that is about 3 ton/mt
Total bridge weight: 18.6 ton/mt

REFERENCES

Fig. 1 - View of the new bridge, in the background the ancient town

Fig. 2 - Lateral view
Fig. 3 - First 4 mode shapes
Fig. 4 – Transversal section of the bridge.

Fig. 5 - Calculated vertical acceleration at mid span, on the side, during the passage of an heavy truck.
Fig. 6 - Erection: stage 8

Fig. 7 - Spectrum accelerations of the arch out of the vertical plane
Fig. 8 - Spectrum of acceleration of the arch in the vertical plane

<table>
<thead>
<tr>
<th>mode</th>
<th>frequency (Hz)</th>
<th>period (sec)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.871</td>
<td>1.147</td>
</tr>
<tr>
<td>2</td>
<td>0.997</td>
<td>1.003</td>
</tr>
<tr>
<td>3</td>
<td>1.648</td>
<td>0.607</td>
</tr>
<tr>
<td>4</td>
<td>1.856</td>
<td>0.539</td>
</tr>
<tr>
<td>5</td>
<td>1.999</td>
<td>0.500</td>
</tr>
<tr>
<td>6</td>
<td>2.006</td>
<td>0.498</td>
</tr>
</tbody>
</table>

Tab. 1 – First 6 natural frequencies of the bridge

Description of mode shapes:

mode 1: \(1^{st}\) bending mode of the arch out of the vertical plane

mode 2: \(2^{nd}\) bending mode of the whole bridge in the vertical plane

mode 3: torsional mode of the deck

mode 4: \(3^{rd}\) bending mode of the whole bridge in the vertical plane

mode 5: \(1^{st}\) bending mode of the deck out of the vertical plane, \(2^{nd}\) bending mode of the arch out of the vertical plane

mode 6: \(4^{th}\) bending mode of the whole bridge in the vertical plane
<table>
<thead>
<tr>
<th>Measure Point</th>
<th>Hemisymmetric Loading (Deflection mm)</th>
<th>Maximum Loading (Deflection mm)</th>
<th>Maximum Torque (Deflection mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Calculated</td>
<td>Measured</td>
<td>Calculated</td>
</tr>
<tr>
<td>¼ Span</td>
<td>-76.4</td>
<td>-61.0</td>
<td>-4.8</td>
</tr>
<tr>
<td>½ Span</td>
<td>-12.0</td>
<td>-36.0</td>
<td>-45.1</td>
</tr>
<tr>
<td>¾ Span</td>
<td>+52.1</td>
<td>+31.3</td>
<td>-14.5</td>
</tr>
<tr>
<td>¼ Arch</td>
<td>-67.5</td>
<td>-63.0</td>
<td></td>
</tr>
<tr>
<td>½ Arch</td>
<td>-3.7</td>
<td>-14.0</td>
<td>-26.8</td>
</tr>
<tr>
<td>¾ Arch</td>
<td>+56.0</td>
<td>+51.0</td>
<td></td>
</tr>
<tr>
<td>Mid Span Side Corbel</td>
<td>-12.0</td>
<td>-8.0</td>
<td>-49.2</td>
</tr>
</tbody>
</table>

Tab. 2 - Comparison of measured and calculated deflections

<table>
<thead>
<tr>
<th>Measure Point</th>
<th>Hemisymmetric Loading (Stresses MPa)</th>
<th>Maximum Loading (Stresses MPa)</th>
<th>Maximum Torque (Stresses MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Calculated</td>
<td>Measured</td>
<td>Calculated</td>
</tr>
<tr>
<td>Lower Flange of Box-Girder at Mid Span</td>
<td>ca. 0</td>
<td>7.6</td>
<td>50</td>
</tr>
<tr>
<td>Beginning of Main Lower Pipe, on the Top</td>
<td>-47</td>
<td>-9.6</td>
<td>-51</td>
</tr>
<tr>
<td>Beginning of Main Upper Pipe, on the Top</td>
<td>-23</td>
<td>-13.4</td>
<td>-41</td>
</tr>
<tr>
<td>Tensile Force in Central Cable (KN)</td>
<td>1107</td>
<td>1083</td>
<td>1333</td>
</tr>
</tbody>
</table>

Tab. 3 - Comparison of measured and calculated stresses ("-" = compressions)
### Tab. 4 - Comparison of measured and calculated natural frequencies of the arch

<table>
<thead>
<tr>
<th>ARCH</th>
<th>calculated</th>
<th>measured</th>
</tr>
</thead>
<tbody>
<tr>
<td>1st bending of the arch out of vertical plane</td>
<td>0.871 Hz</td>
<td>0.8 Hz</td>
</tr>
<tr>
<td>2nd bending of the arch in vertical plane</td>
<td>0.997 Hz</td>
<td>0.8 Hz</td>
</tr>
<tr>
<td>3rd bending of the arch in vertical plane</td>
<td>1.856 Hz</td>
<td>1.9 Hz</td>
</tr>
<tr>
<td>2nd bending of the arch out of vertical plane</td>
<td>1.999 Hz</td>
<td>2.1 Hz</td>
</tr>
</tbody>
</table>

### Tab. 5 - Comparison of measured and calculated natural frequencies of the deck

<table>
<thead>
<tr>
<th>DECK</th>
<th>calculated</th>
<th>measured</th>
</tr>
</thead>
<tbody>
<tr>
<td>2nd bending of the whole bridge in vertical plane</td>
<td>0.997 Hz</td>
<td>1.1 Hz</td>
</tr>
<tr>
<td>torsional mode of the deck</td>
<td>1.648 Hz</td>
<td>1.6 Hz</td>
</tr>
<tr>
<td>3rd bending of the whole bridge in vertical plane</td>
<td>1.856 Hz</td>
<td>1.9 Hz</td>
</tr>
<tr>
<td>1st bending of the deck in horizontal plane</td>
<td>1.999 Hz</td>
<td>1.8 Hz</td>
</tr>
</tbody>
</table>