Footbridge over Moraca River - Podgorica

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Summary:

The suspension footbridge in Podgorica, Montenegro, is 71.5 m long slender structure, resting on old pylon foundations of the previous 70 years old bridge. Concerning bridge`s slenderness and deformability, especially in the assembly stages, the SOFISTIK software calculation had been performed so as to be able to have an overview on the bridge`s behaviour in all constructing phases. After the works had been done, the test load and dynamic parameters measurements confirmed all the presumptions from the structural analysis.

1 GENERAL

Suspension footbridge over the Moraca river in Zagoric is aimed for the pedestrian traffic between Podgorica – Danilovgrad motorway and the part of Podgorica city at the left river bank (Piperska street). The nearest surrounding bridge is located 518 m downstream. The bridge is 4.04 m wide, with the free passage profile of 3.6 m; the total bridge length is 71.5 m; the bridge has convex profile grade line, in vertical curve of radius R = 300.75 m, which ensures the pleasant view of the bridge.

Figure 1: Footbridge

2 STATICAL SYSTEM

The statical system is suspension bridge, with cables anchored on steel pylons filled with concrete; the visual effect of the new bridge is very similar to the old one, which was demounted short time ago. The suspension cables are made of “Z” wires (locked coil ropes Φ40 mm). At the longitudinal distance of 2.75 m, the hangers are mounted on suspension cables; the hangers are “open spiral strands” of radius Φ13 (16) mm. The cross beams are hanged at the hangers` bottoms, over which the corrugated “holorib” steel plate is placed, as the framework for concrete deck slab 10 – 12 cm thick. Besides being the framework, the “holorib” steel plate acts also as reinforcement for the deck slab in the longitudinal direction. The longitudinal stiffnes of the bridge is achieved by two “Freyssinet” cables 6Φ15.7 mm placed through the holes in IPE cross – girders, and stressed in the anchor blocks at S4. Those cables acts due to the vertical curved position, say by “equivalent loading”, so contributing to the longitudinal system stiffness and dislocating the oscilating frequencies out of dangerous zones of 1.6 – 2.4 Hz. Also, avoiding the use of longitudinal girders, the bridge gets the pleasant, thin silouetthe of total depth of 45 cm only (L2/100), totally covered with fine shaped concrete cornice – edge beam.

The longitudinal fixed bearing is located at S2, while the movable elastomeric bearings are located at S1, S3 & S4. The lateral supporting for the deck slab is provided at all piers and abutments.

3 STRUCTURAL ANALYSIS

The bridge is analysed according to „YU Code for bridge loadings“ from 1991., the code which is very similar to DIN 1072 & DIN 1075. Due to the span longer than 10 m, the uniformly distributed load of 4,0 kN/m² was applied. It’s important to emphasize that the passengers load in Eurocode 1 – Load model no. 4, asks for the value less than adopted, as it is $2{,}0 + 120 / (L + 30) = 3{,}6$ kN/m². The wind load is taken from actual „YU Code“, with the check for „slender structures“ from the Eurocode 1 – „wind load for bridges“. The temperature difference between top and bottom deck slab face (daily change), temperature difference between steel elements (hangers & suspension cable) and deck slab, as well as temperature change in the elements axis (seasional change), was performed. Seismic check was performed after the multimodal analysis had been done, for the loads induced from IX degree of MCS scale, but, due to the mass of the bridge, the seismic load was not governing for the design. Relevant combinations of actions were also checked, but separately, with no superposition, as the calculation is performed with second order theory. The structural system elements were checked for the ultimate limit states, also the serviceability limit states were checked for deflections, rotations and inclinations.

The structural analysis is performed by SOFISTIK software, using options for geometric nonlinearity (SYST PROB TH3), and nonlinear spring elements. As the suspension cables and hangers are the elements counteracting the tension forces only, also the influence of great displacements on section forces are not neglectable, the second order theory and the nonlinarity of the system were neccessary to include in the calculations.

Special attention was given to the assembly phases in calculation, say concreting stages, because the system is higly sensibile in those phases, as at the moment there's no system elements active that would give the longitudinal stiffness to the construction (Freyssinet cables stressed to 10% only, and no deck slab hardened ...).

After the calculation of deformations of construction due to dead loads, the pre-deformation of system was performed so as to the construction should come to designed position after all dead load was added. The pre-deformation was done by assemblage of pylon's tops for calculated values (-75 mm and + 40 mm) towards river banks, and by cutting the suspension cables and hangers (in factory) to the lengths comprising the elongations that those elements should get in construction after the all dead loads were added.

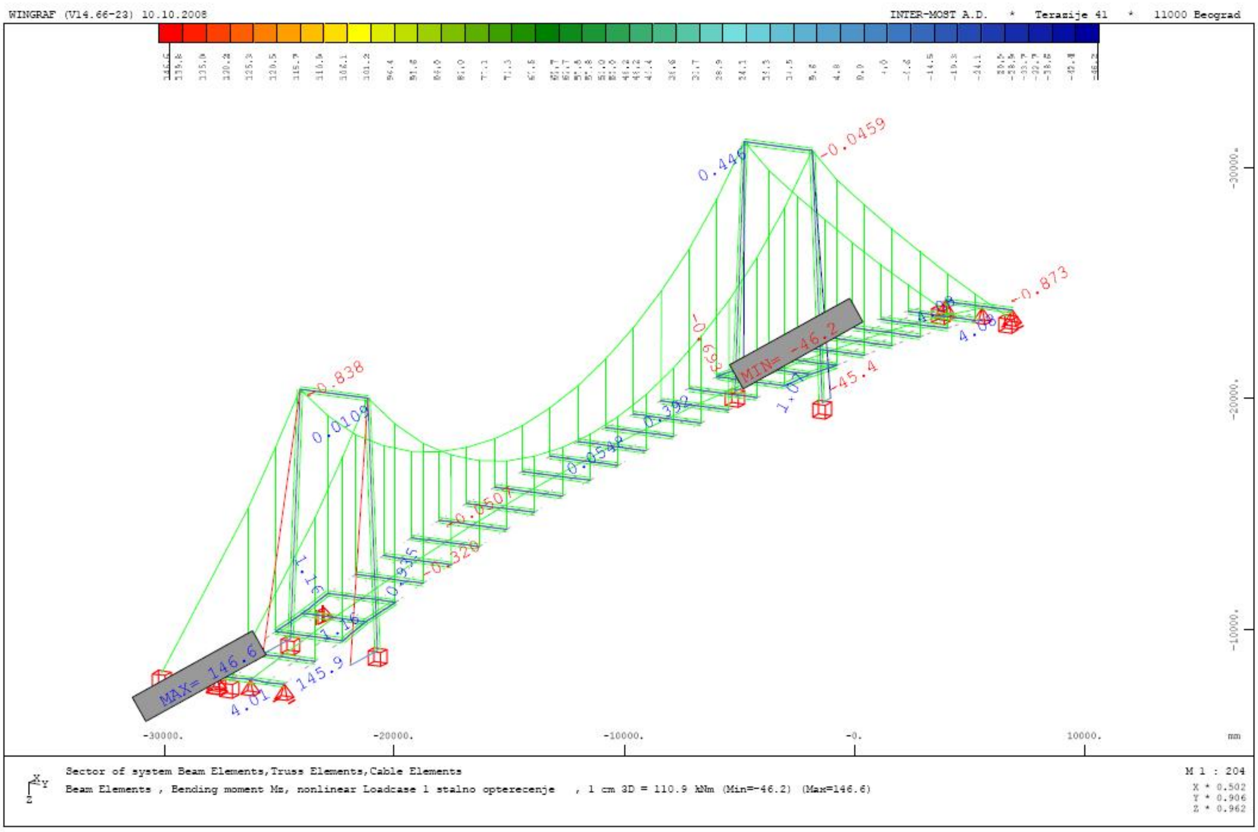

Figure 2: Pylons longitudinal bending moments due to dead load and prestressing

It’s important to emphasize that there were no supports bellow the cross girders between the pylons’ piers in the prestressing and concreting stages, so the nonlinear springs were introduced at that positions, with the iteratively calculated „GAP“ of 8,7 mm at S2, and 158,6 mm at S3 pylon. The values specified are the deflectons of the deck slab at those points between pylons due to dead load and prestressing. Owing to those nonlinear springs with the „gaps“ specified, the supports of the deck between pylons’s piers are activated for the loads that were to be applied after the prestressing – additional dead load (waterproofing, pavement, guardrails, cornices ...), wind, seismic, temperature and live load.

As the problem is nonlinear, it’s not possible to do the superposition of the different load cases’ effects, so the system was separately loaded for all load cases. Concerning the eigenvalues, the first six modes were treated, and the four of them are located bellow the dangerous range for footbridges (f < 1,6 Hz). Having in mind that the bridge has no longitudinal girders, their role in the bridge’s longitudinal stiffness very succesfully was replaced by Freyssinet prestressing cables with the initial force of 2538 kN. Those cables had stiffened the construction in the longitudinal direction, so dislocating the oscilating frequencies out of the dangerous zone.

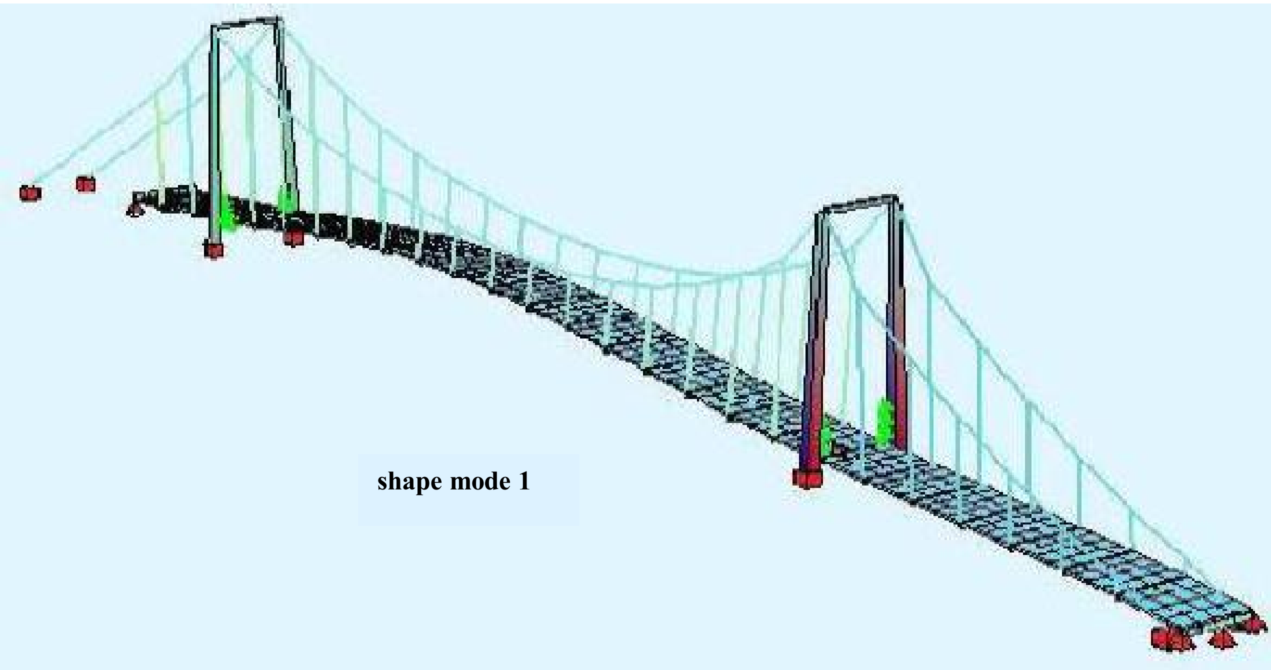

Figure 3: Shape mode 1

4 WORKS EXECUTION

The foundations of pylons rests on the sound rock, with no cracks detected, or potential sliding planes. The horizontal forces are low, so the pylons’ supports are dimensioned for axial forces and bending moments. The anchorage blocks for stiffening and suspension cables at S1 & S4 were dimensioned to withstand the forces by their mass and weight and with the help of soil wedge engaged using „I240“ steel girders embeded in holes lateraly mounted in soil.

The single pylon components, two piers 500/400 mm at the bottom, 300/300 mm at the top, 12 m long, were made on ground, by welding three sides, putting the reinforcement cage in, and closing the pier by welding the fourth side. After the piers were hoisted at the designed position, the cross beam Φ193 mm at their tops was positioned and welded.

The pylons were fastened by bolts at concrete pedestals assuming inclined positions of pylons' tops for the values – 75 mm at S2, and + 40 mm at S3. After that, the pylons were filled with concrete and the suspension cables were hanged on pylons' tops. The hangers were fastened to suspension cables and the IPE220 cross girders were hanged od hangers' bottom ends. It's important to say that the suspension cables at S4 and all hangers at their bottoms has the devices for the length correction. IT'S ALSO IMPORTANT TO EMPHASIZE THAT, AFTER THE BRIDGE WAS COMPLETED, IT WAS NOT NECCESSARY TO CORRECT THE STEEL ELEMENTS' LENGTHS, WHICH MEANS THAT THE CONSTRUCTION ACTS AS IT WAS DESIGNED AND CALCULATED IN THE STRUCTURAL ANALYSIS !

The assemblage of prestressing cables was the next operation. Those cables found their way through the holes in IPE220 ribs, and were stressed from the active anchorage at S4. To prevent loosing the vertical position of hangers during the first phase stressing operation, the provisional U80 beams were bolted to IPE220 ends, so keeping the geometry of the hanging system. Between the pylons' piers there were no designed hangers to bear the cross girder IPE220, so during the prestressing operations the provisional I240 steel girders were used, resting on IPE220 cross girders left and right from the pylons. The operation was performed as the cross girders between pylons' piers must be free to deflect in first prestressing stage, deck slab concreting stage, and second prestressing stage. After all those operations had finished and steel cantilever bearings were mounted on pylons' piers under cross girders, the provisional I240 girders were demounted.

The trapezoidal "holorib" plates were mounted over the IPE220 cross girders, having both the function of formwork for the concrete deck slab and the reinforcement of the slab in the final position. The depth of the slab is 10 – 12 cm.

Before the concreting phases had started, the part of stiffening prestressing was performed up to the level of 10% of the final force. The concreting of the deck slab was done in six stages, which had been chosen to decrease the expected deformations of the system during the operations. As the system is very deformabile in those stages, it was of great importance to finish all concreting phases (departed from each other by the 30 cm wide “plug”) as soon as possible, so avoiding the previously concreted section to be cracked (if had hardened !) by the deformations induced from next section concreting ! The whole operation of deck slab concreting, divided in six sections (total volume of concrete was 30 m^3) was finished in less than three hours. The next day the concrete “plugs” of 30 cm were completed between concrete sections. After the concrete strength had reached 25 MPa, the second stage of prestressing was performed, to the final level of designed force of 2538 kN.

The next operation was assemblage of steel cantilever supports at the inner sides of pylons’ piers – at the S2 there’s a fixed bearing, the connection between cross girder and steel support done by bolts, while at the S3 the bearing is movable – elastomeric pad bearing 150 x 100 x 28 mm. After the supports had been positioned, the provisional U80 and I240 girders were demounted.

The final operations - bridge furnishings, were: edge beams concreting, guardrails, waterproofing, pavements, cornices, decorative and functional lightings for the bridge …

5 FOUNDATIONS

At the S1 position there’s massive anchorage block, the place for anchoring both left suspension cables and prestressing stiffening cables. The anchorage block is massive, plan dimensions 6,58 x 7,00 m, depth 3,0 m. This block is the support for the wall which is the anchorage for left suspension cable. The wall is 2,0 m high, and of various depth from 185 cm to 73 cm. The prestressing cables were anchored at the back side of the block. The block is composed of reinforced and plain – ballast concrete. The cantilever walls are C30/37 concrete, the rest of reinforced concrete is C25/30. The plain – ballast concrete is C12/16. In the bottom of the excavated foundation, the lateral holes were digged, dimensions 40 x 50 x 55 cm, for the I240 steel girders serving for activating soil weight to counteract active forces.

At the S4 position there`s the unique anchorage block also, for the active prestressing cables and suspension cable. The plan dimensions are 5.30 x 7.00 m, the depth 2.50 m. Beams and slabs constituting the control room, as well as supporting beam, were made of reinforced concrete, while the rest of the block was made of plain – ballast concrete. The eventually correction of the suspension cable length is to be done from the S4 position.

The pylons` foundations were made of C25/30 reinforced concrete, plan dimensions 3.00 x 6.50 m, 1.00m depth, except the "back tooth" which depth is 1.40 m. The pylons` piers rests on reinforced concrete pedestals, 110 x 100 x 50 cm.

The anchor – blocks acts by the self – ballast, so providing the necessary stability of the system as a whole. The safety margin against overturning, for the full service load, is 1.0. The missing security (until 1.50) had been provided using I240 steel beams embedded in the soil, thus engaging the soil mass. The safety reserve was provided taking into account the soil mass only, not counting on the shear strength around the soil – ballast block. The plain concrete ballast was hanged to reinforced concrete elements by steel meshes and anchorage bars. Maximal stress at the soil is 236 kN/m^2 only.

Having experienced the excavations in the similar material, we had decided to dig vertically sided excavations, machine – done. The lateral holes for steel beams were digged manually, using drill – hammers. Extra spaces left after the excavation were filled with plain concrete C12/16.

6 LITERATURE

[1] Eurocode 3: Design of steel structures, part 1.11 – design of structures with tension elements

[2] "SOFiSTiK" documentation and tutorials