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THE DESIGN OF A NETWORK ARCH BRIDGE CROSSING OVER ARACHOS RIVER IN ARTA, GREECE

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THE DESIGN OF A NETWORK ARCH BRIDGE CROSSING OVER ARACHTHOS RIVER IN ARTA, GREECE.

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ABSTRACT: This paper describes the structural design of a network Arch bridge located in Arta over Arachthos River. A network arch is defined as an arch bridge with inclined hangers where some hangers cross other hangers at least twice.

The development of the road Network and the peripheral road in the Area of Arta-Greece set the opportunity to design an innovative road Arch bridge that is intended to be a landmark for the city and to contribute to the quality of a new leisure area.

KEY WORDS: Bridge; Network Arch; Hanger

*Figure 1.*Architectural visualization of the Bridge.

1 INTRODUCTION

Optimal hanger arrangement in arch bridges not only lead to minimum values of the axial forces and force variations in the hangers and minimum values of bending moment and moment variation in the arch, but also it allows to use small cross sections and low weight with aesthetical and structural advantages.

In literature there are more bridges with fan and vertical hangers arrangement than network arch bridges; fan arrangement is generally chosen for aesthetical reasons even if other solutions show better structural behavior. Some network arch solutions with aesthetical advantages and very good structural behaviour have been designed by Tveit (1987, 2001). Brunn and Schanack (2003) proposed a new hanger arrangement for railway bridges with concrete decks. In our design we adopted these methods and we optimized the angle of hangers by solving multiple models.

The bridge is 160m long with spans 20m-120m-20m. The Central span is a network Arch Bridge.(Fig 2.). The central span crosses the river Arachthos which has constant flow during the winter and the summer.

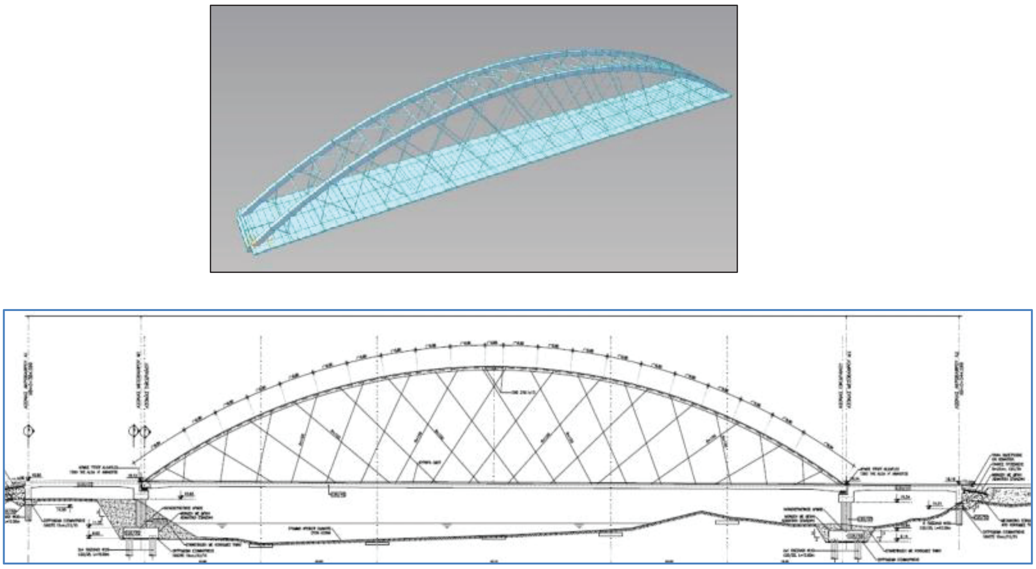

Figure 2. Structural Model using MIDAS Civil.

2 DESIGN

2.1 Hanger Arrangement and Arch.

Since bending moments in arches depend on the configuration of the line of thrust and they ought to be reduced in arch bridges, it is necessary to align the line of thrust with the center line of the arches. To have the best distribution of efforts, the upper hanger nodes should be placed equidistantly (shown with the distance d on the figure 3) and the hangers should cross the arch with the same angle (represented by α). This angle is actually the angle between the hanger and the line starting in the middle of two hangers to the center line of the arch (the dotted line in Fig.3).

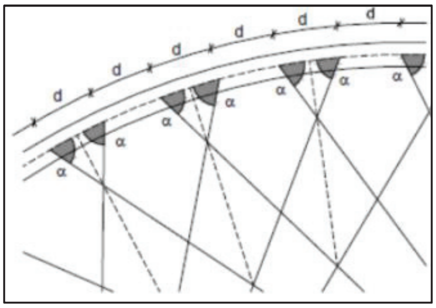

Figure 3. The hangers cross symmetrically the radii with same angle.

Following the above literature recommendations Arta bridge has 18.0m rise of the arch keeping a value close to optimal 15% of the span as Tveit advised. Larger arch rises decrease internal forces but respecting aesthetics it should not exceed 17% of the span. Also by making test with the hanger inclination and by literature it was decided to be 35^0. Also by increasing the number of hangers the bridge behavior doesn't significantly change. There were used 30 rods hangers in each side with 100mm diameter and structural steel material S460 ML. The upper hanger connections are spaced 4.10m along the arch length.
The arches have a constant box cross section with external dimensions of H/W = 0.71 m / 0.55 m. (Fig 4.). The webs and flanges consist of 50 mm thick steel plates, respectively, and are made of structural steel S 350 ML. The cross section has a parallelogram shape with the web plates parallel to the arch planes; the flanges are horizontal. The arches are laterally supported by a wind bracing formed by rhombuses made of circular hollow steel sections (S 235) with an external diameter of 219.1 mm and a thickness of 10 mm.

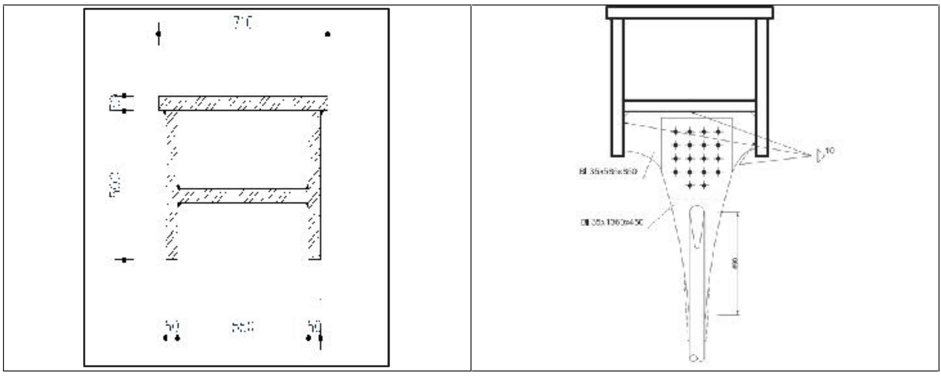

Figure 4 .Arch cross section and connection with Rod member.

Each set of hangers is shifted half the diameter of the hangers out of the arch plane. This allows them to cross without deflections. The eccentricity causes torsional moments in the arch profiles, which are partially taken by the wind bracing. The direction of the eccentricity changes from each hanger connection to the next, so the torsional moments counteract each other. In the bridge that we designed no relaxation occurred in the hangers.

2.2 Deck Cross section.

The tie of the bridge consists of a solid concrete slab spanning 10.65m between the hangers. The prestressing in longitudinal direction mainly counteracts thrust of the arches. The depth of the slab is 0.7m in middle span and 1.0m at supports.

When the distance between the arches is less than 18 m, the deck should be made of concrete and prestressed. This gives a slender structure, less noise and saves materials.(Fig 5.)

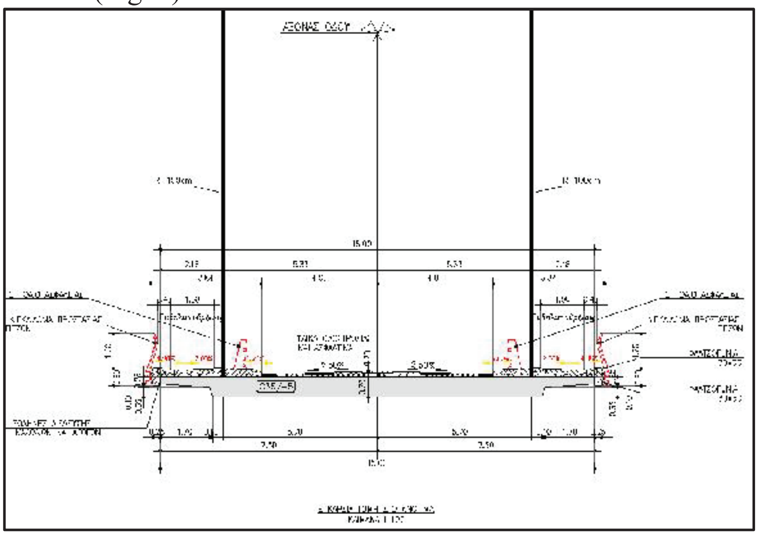

Figure 5.Cross section of the Deck.

Certainly, the increased dead load increases the bending moment. But the higher effective depth and the increased lever arm of the tendon counteracts the negative effect of the higher dead load. Therefore the required additional depth will be moderate and the compression reinforcement can be made redundant.

Approximate calculations showed that a thickness of about 70 cm at the slab's mid-span would be enough to eliminate compression reinforcement. Besides, a thicker tie improves the torsionaly rigidity and stiffness of the deck.
A thinner deck could be achieved by applying transverse prestressing in the length of the bridge. Although this could be the optimal solution it would lead to increased cost and design time.
The deck is made of C35/45-XC3 concrete (according to EN 1992-1-1 and EN 206-1 and is longitudinally prestressed by twelve 22-strand prestressing tendons.
The Design of the bridge adopted the Eurocode2-2 for the deck design. Midas Civil has compliance with the new codes and that was of great help.(Fig. 6.)

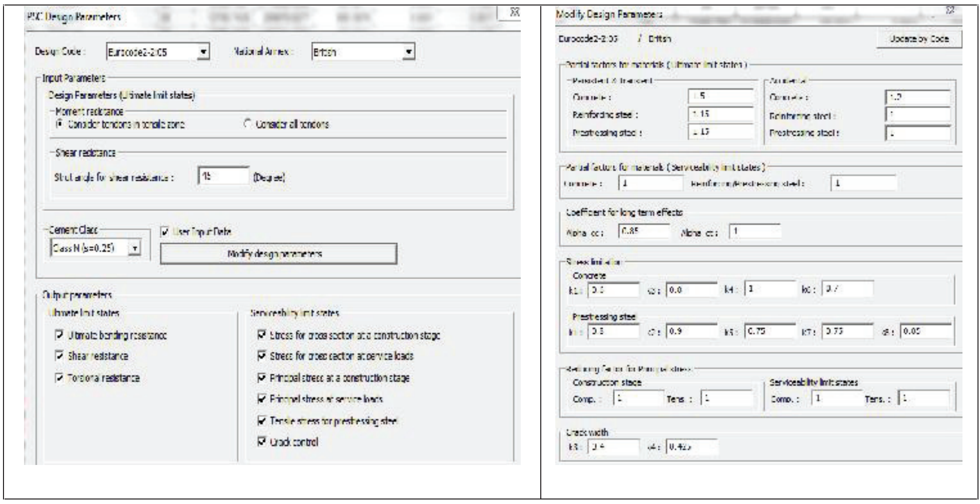

Figure 6. Midas Civil and Eurocodes Design.

3 FEM-CALCLULATION

3.1 General.

For the FEM-calculations a single structural analysis software package was used. Most of the investigations were performed with MIDAS CIVIL-KOREA.

Several models were created in order to perform the necessary check. (ULS, SLS, Dynamic, Buckling, etc).
Mainly two models were created. One by simulating the deck as beam elements, and one by using plate elements.
For the model with plate elements their nodes were aligned to the bottom plane of the tie. In that way it was possible to shape the bridge deck like the real cross-sections by applying different thickness to the plane elements. The cantilevers were connected by couplings to the nodes of the bridge deck elements providing fixed connection to the rigid body at the reference nodes. (Fig. 7)

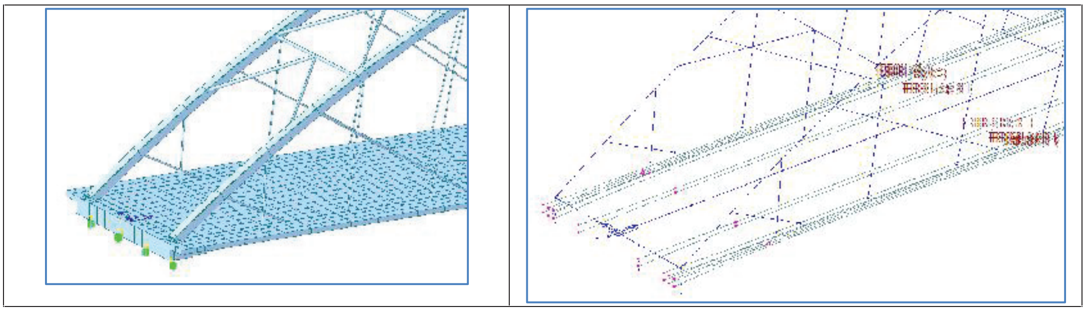

Figure 7. 3D plate elements model and beam element with prestressing tendons.

The arches were modeled using beam elements with a length of about 0.5 meters. The truss members of the wind bracing were also beam elements with truss properties. They originate mainly in the torsional moments in the arch due to the eccentric connection of the hangers and can be ignored for the assessment.

The hangers were modeled using cable elements that only sustain tension in case of non-linear analysis. This has to be considered when calculating influence lines. Since analysis is carried out in linear fashion, hangers will take compression forces, instead of relax. This leads to increased internal forces and is therefore on the safe side. The cable elements were connected eccentrically to the arch. At their intersections the horizontal deflection perpendicular to the arch plane was coupled. In that way it was possible to calculate deflections and mode shapes of the hanger web.

3.2 Buckling analysis of the Arch.

The arches receive mainly axial compression forces and are therefore in danger of collapse due to buckling. Additionally, there are in-plane bending moments My due to the hanger forces and out-of-plane bending moments Mz and torsional moments Mt due to horizontal forces (like wind) on hangers and arches. Additionally the eccentricity of the hanger connections causes torsional bending. The arches were verified using second order analysis to prove the buckling resistance.

For this purpose it is required to apply the initial bow imperfection specified in EN 1993-1-1: 5.3.2 on the arch. The relevant buckling curve is the first mode shape for each axis of the arch profile.

The mode shapes were determined by the dynamic analysis of MIDAS CIVIL(Fig 8.)

The stability verification of the arch is performed according to the following steps:

Step 1: Determining decisive buckling mode

Step 2: Calculating imperfections

Step 3: Implementing imperfections in the MIDAS model

Step 4: Running a geometrically nonlinear analysis

Step 5: Verification of the results

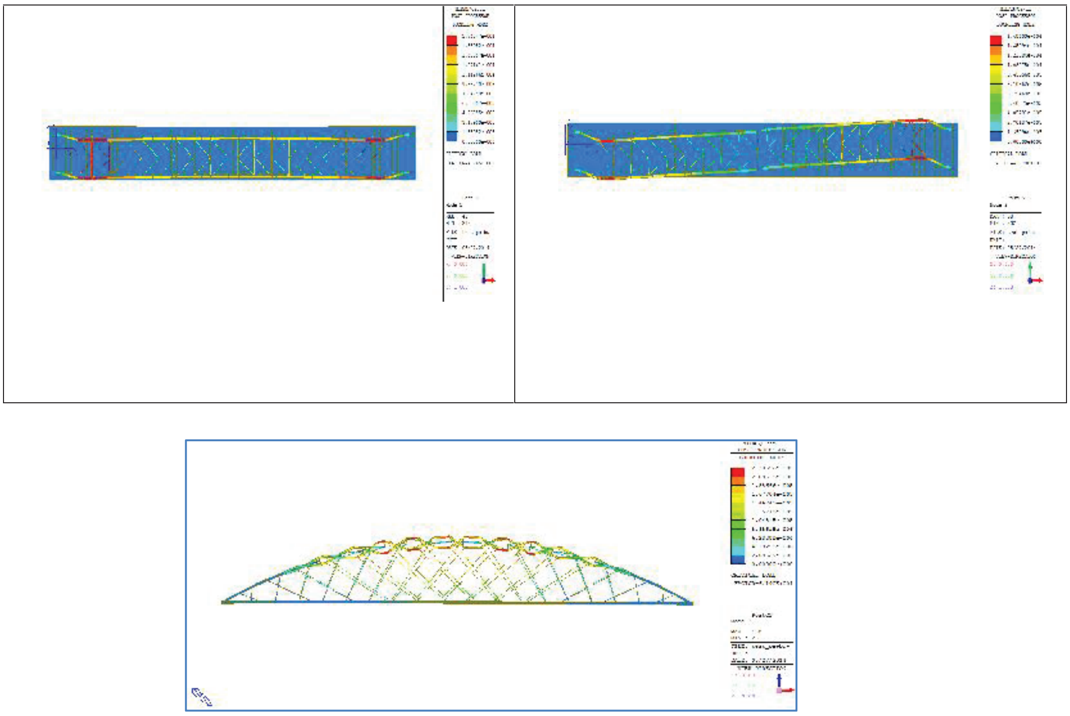

Figure 8. First three Buckling mode shapes.

4 FATIGUE INVESTIGATION.

Bridges are subjected to dynamic loading, which makes the consideration of the fatigue behavior necessary. This is especially important for hangers and hanger connections, since they receive larger force variations than other bridge members. Subjected also to horizontal loading, hangers and their connections are therefore significantly prone to fatigue failure. In our bridge two fatigue assessments were made.

- Fatigue assessment based on nominal stress ranges.
- Fatigue assessment based on geometric stress ranges.

The second method of assessment is necessary because the hanger connection details are more complex than the test specimen with which the detail categories and fatigue strength curves, such as in the Eurocode 3, were created. If the geometry and the loading differ significantly from the listed detail categories, the nominal stress is not meaningful, and its application would lead to wrong results. Therefore, local stress concentrations at geometric discontinuities were investigated.

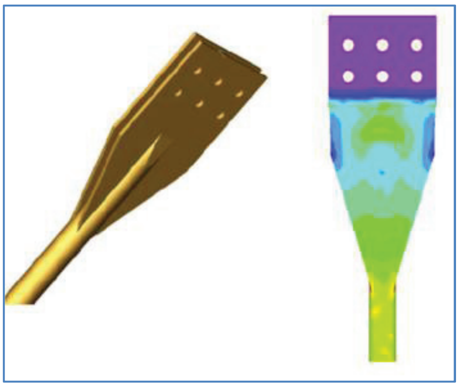

Figure 9.Fatigue assessment based on geometric stress ranges.

5 CONCLUSION

In this work the design of Arachthos bridge in Arta-Greece was discussed. The reduction of cost, resulting from the use of network arch bridges is of great interest. The structural members of network arches are mainly subjected to axial forces. Generally, structures with this characteristic are considered as efficient.
The arch root calls for special attention while designing it. The stress range due to live load is likely to exceed the allowed limits, because of the skew weld between the arch and the end plate which takes nominal stresses and shear stresses from the large axial force in the arch. One possible solution to improve this detail is enlarging the flanges of the arch profile and transferring the forces partially to the horizontal plate above the bearings. The minimum distance of the prestressing strand anchorages and the end cross girder require an enlargement of the concrete tie at the arch root.
Also care must be taken in the buckling calculation analysis and the fatigue of the steel components of the bridge.

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