Suspended Steel Roof of the Archeological Site of the School of Aristotle

Charis J. Gantes[1], Konstantina Koulatsou[1]

[1]*Institute of Steel structures, School of Civil Engineering, National Technical University of Athens, Greece*

ABSTRACT

The structural design of the cable-suspended steel roof covering the archaeological site of the School of Aristotle in Athens, Greece is presented. The preliminary architectural proposal, which was awarded first prize in a competition organized by the Greek Ministry of Culture, provided for 65m span, arch-type main structures, each suspended by means of five suspension cables from a single pylon, stabilized by a pair of back-stay cables. Main arches were spaced at 11m and connected by means of purlins and bracing. The structural design concentrated on avoiding deviations from architectural requirements. Nevertheless, as a result of the vaulted shape of the roof, several cables were found to relax under service loads, thus the number, locations, cross-sections and prestressing of cables had to be re-evaluated.

The present paper focuses on nonlinear analyses for understanding the behavior, predicting all possible failure mechanisms, and evaluating the ultimate strength of the roof by means of commercially available finite element software. Emphasis is placed on the role of flexural buckling of the pylon and lateral-torsional buckling of the main arch beam in the bearing capacity of these two members, both having complex geometry and varying cross-section, thus requiring a novel approach extending beyond code specifications. Failure dominated by either material yielding or instability is addressed, as well as interaction of failure modes. Steps include setting up an appropriate finite element model, obtaining critical buckling modes from linearized buckling analysis (LBA), and then using a linear combination of these modes as imperfection pattern for a geometrically and material nonlinear imperfection analysis (GMNIA). Equilibrium paths accompanied by snapshots of deformation and stress distribution at characteristic points are used to evaluate the analysis results, identify the dominant failure modes and optimize the structural performance.

PRESENTATION OF CABLE-SUSPENDED STEEL ROOF

The archeological findings of the School of Aristotle are located in the center of Athens, Greece. For the protection of the archaeological site, an architectural competition was held by the Greek Minister of Culture and the preliminary architectural proposal (Figure 1) of the present suspended steel roof was awarded the first prize.

The roof steel structure (Figure 2) consists of six parallel main arches of span of 60m at a distance of 10.5m that are connected to each other with purlins and horizontal bracings. Each main arch is pinned on the ground on one edge, while the other edge is supported by a V-shaped column. The arches are suspended by prestressed cables from a 25m tall pylon, which has a slight inclination with respect to the vertical plane. The stability in and out of plane of the pylons is ensured by two back-stayed prestressed cables. On the exterior of the pylons and columns, independent steel structures for the extension of the cladding and eccentric vertical bracings are provided.

Figure 1: Suspension cables and arch geometry of main beams

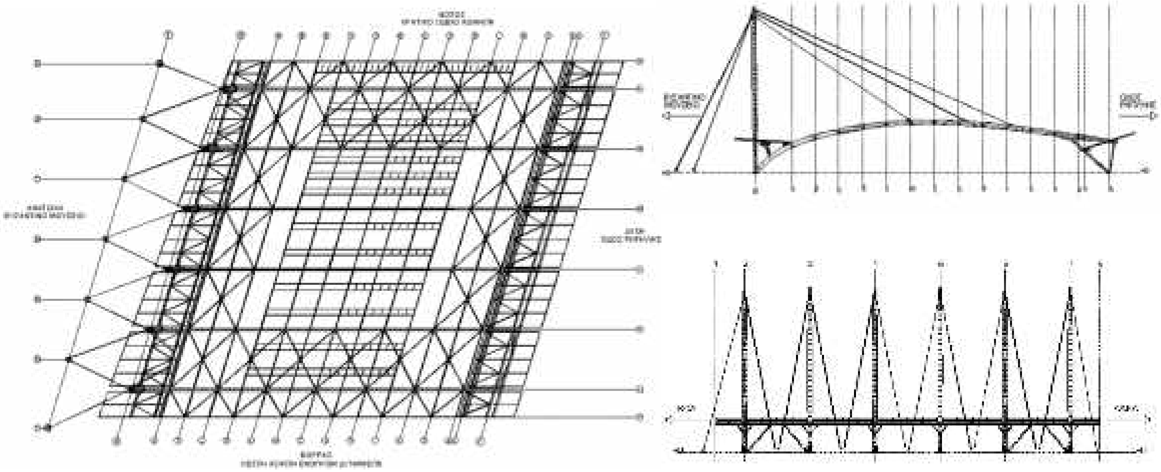


Figure 2: Plan view and sections of the suspended steel roof

The arches are suspended by prestressed cables from a pylon of 25m height which has a slightly inclination in relation to the vertical plane. The stability in and out of plane of the pylons is ensured by two back-stayed prestressed cables. Towards the sides of the pylons and columns, independed steel structures for the extension of the cladding and eccentric vertical bracings at the edge panels are placed.

Location, number and prestressing of cables

According to the preliminary architectural and structural proposal, five suspension cables should be used for each main arch. The first analysis for permanent loads and prestress indicated that several cables relaxed under service loads as a result of the shallow arch shape of the roof. Therefore, the initial cable locations had to change. In order to function properly, the suspension cables must be located in the most flexible areas of the arch, so that the deflections due to the large span can be eliminated as much as possible. As in this case, the suspension cables have an unfavorable inclination with respect to the horizontal plane, they also induce significant axial forces to the main arches. Therefore an investigation of the optimum cable locations and prestress was necessary.

The procedure to determine the optimum cable locations and prestress is a complex design problem, especially in cases of complex structural systems, as the present steel roof, for which the nonlinear analysis that are necessary due to the inherent geometric nonlinearity of cables have convergence problems in case the initial magnitude of prestress and the ratio between cables prestress are far from the final ones. In order to overcome this problem, an iterative procedure was carried out [1], in which successive analysis for simple models were initially performed, which gradually became more complex and eventually approached, with sufficient accuracy, the expected actual behavior of the structure. This resulted in adopting three cables for each main arch instead of the five that had been proposed by the preliminary architectural design. The corresponding structural model is illustrated in Figure 3.

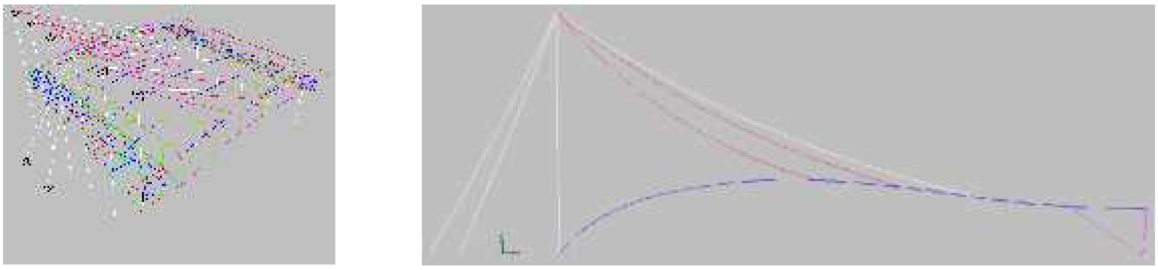


Figure 3: 3D view of entire roof and elevation of main arch

NUMERICAL EVALUATION OF MAIN MEMBERS OF THE STEEL ROOF

The prevailing approach proposed by modern codes for carrying out checks at the ultimate limit state consists of obtaining action effects by means of linear elastic analysis of the structure subjected to design loads, and comparing them for each member to resistances that account for both types of eventual nonlinearity. This is usually accomplished by multiplying the cross-section resistance, assuming full exploitation of the material capacity of the cross-section, by appropriate reduction factors, representing the influence of geometric nonlinearity. The results are quite reliable for ordinary structural systems, consisting of members with normal profile cross-sections as well as built-up cross-sections. However, these specifications can not be applied for more complex steel structures such as the structural system of the presented steel roof with non-ordinary geometry and variable cross-sections. Thus, in the present article, a systematic methodology is presented for predicting the ultimate capacity of the main members of the steel roof, by means of nonlinear numerical analysis, making use of commercially available finite element software [2-3]. The used methodology performed on the steel roof is described step by step, accompanied with equilibrium paths and deformation and stress figures, for the pylons and the main beams-arches.

Proposed numerical methodoly

The basic steps of the employed methodology are briefly repeated here:

i. *Setting-up an appropriate finite element model:*

In the first step, attention must be paid to the chosen finite element software, the type of finite elements and the numerical solution algorithm employed. The model should be able to predict all anticipated failure mechanisms. For example, modeling a column with beam elements is not appropriate for predicting failure where local or lateral buckling may be predominant. Shell or plate elements should be used instead, and the mesh should be fine enough to capture the curvature of the part of the cross-section where local buckling occurs. Moreover, the mesh density chosen is important. A rough mesh density can prevent the prediction of local buckling, however a very fine density increases considerably the required computational time.

ii. *Carrying out linearized buckling analysis:*

Critical buckling loads obtained by means of linearized buckling analyses (LBA) are, in most cases, not safe predictions of strength [4]. However, this type of analysis must always precede the subsequent, more exact, nonlinear analyses. Critical buckling loads obtained by means of linearized buckling analyses are an initial indication, and in most cases an upper bound, of actual strength. Taking also into account the fact that this is a very fast and inexpensive type of analysis, it may be very useful as a tool for evaluating alternative structural scheme solutions during preliminary design, before resorting to the much more time-consuming and expensive nonlinear analyses. Moreover, buckling modes obtained by means of linearized buckling analyses are commonly used as initial imperfections for geometrically and material nonlinear imperfection analyses (GMNIA). Such imperfections are necessary in order to trigger all possible failure mechanisms and to make sure that the critical failure mechanism is captured by the nonlinear analysis algorithm.

iii. *Carrying out non-linear analysis with imperfections:*

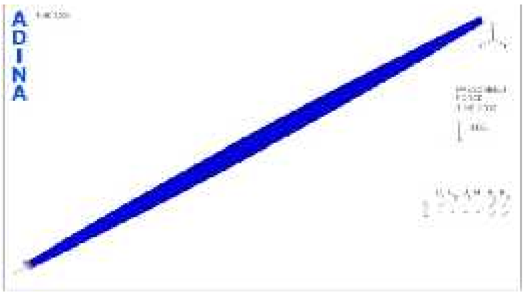
A nonlinear analysis may include large displacement effects, usually called geometric nonlinearity, or material behavior beyond the linear-elastic range, usually called material nonlinearity, or both. Geometric nonlinearity is usually critical for slender structures. Steel structures are in most cases slender, either in terms of small plate thickness with respect to width, a situation that may lead to local buckling, or in terms of long members with respect to their cross-sectional dimensions, which may lead to global buckling, of flexural, lateral or other type. Material nonlinearity is usually critical for stocky structures with low slenderness. A geometrically and material nonlinear imperfection analysis (GMNIA) is recommended for reliably understanding the behavior, predicting all possible failure mechanisms, evaluating the strength, and assessing the vulnerability of steel structures. In many cases it is useful to carry out first separate analyses accounting only for material nonlinearity (MNIA) and geometric nonlinearity (GNIA), which help us identify the prevailing failure mechanism of the structure in question. But the ultimate strength evaluation should always be based on GMNI analyses, accounting for both types of nonlinearity.

In case of non-linear analysis, attention must be paid to shape and size of the initial imperfections and to the evaluation of the results. As stated above, in step (ii), the initial imperfections have the shape of a linear combination of the critical buckling modes obtained from linearized buckling analysis. With the exception of cases with steep unstable post-buckling equilibrium paths, for example frequently encountered in thin-walled shells, the size of initial imperfections is not very important for the type of response and the values of critical response quantities obtained from the analysis. For certain types of structures pertinent codes specify recommended magnitudes of imperfections, or manufacturing and erection tolerances that can be regarded as upper bounds of imperfection magnitudes.

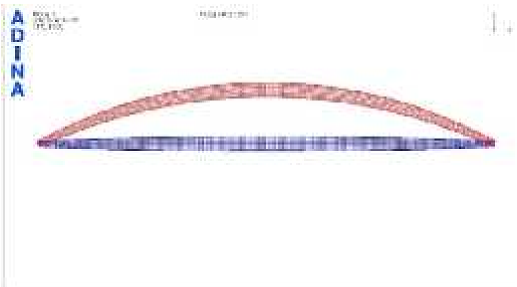
Finally, it is recommended to evaluate the results of GMNI analyses by using a plot of the equilibrium path, accompanied by characteristic snapshots of the structure's deformation and stress distribution along this path. Snapshots of the structure's deformation and stress distribution at characteristic points (for example, before and after change of slope or change of curvature, at the ultimate limit point, at the end of the curve) are very useful in appreciating the physical phenomena occurring with progressing external loading. Comparison of representative numerical results with analytical results of similar cases for which code specifications exist, is absolutely encouraged as a means for calibrating and verifying the numerical results. Numerical analysis can then, with increased reliability, be extended for other cases of geometry, cross-sections, material, boundary conditions, or loads, thus performing "numerical experiments" that may help optimize the design.

Evaluation of the pylon of the steel roof over Aristotle's Lyceum

For the design of the pylon, made of S355 steel, a compression member with hollow circular cross-section, varying over the height, and of the main arch, a member under compression and bending with varying box cross-section with protruding flanges, nonlinear numerical studies have been carried out using the finite element program ADINA [5]. In order to obtain the strength of the pylon with variable cross section against compression, the pylon has been modelled with shell finite elements (Figure 4a). A design decision was taken to avoid local buckling as a potential failure mode, therefore class 1 or 2 cross-sections were used over the height. This was confirmed by the results of the linearized buckling analysis, which showed no local buckling modes among the first ten. Thus, only the first global flexural buckling mode (Figure 4b) was used as initial imperfection.



(a) Undeformed model



(b) First buckling mode

Figure 4: Finite element model of pylon of steel roof over Aristotle's Lyceum

Then, GNI and GMNI analyses were carried out and the corresponding equilibrium paths are shown in Figure 5. It is observed that the presence of imperfections reduces significantly the elastic critical load with respect to the linear one. Failure is encountered on the ascending branch of the equilibrium path, and it is due to inelastic global flexural buckling. This is verified by the deformation and stress snapshots on three characteristic points before, on and beyond the critical point (Figure 6). The formation of plastic hinges at two locations helps identify these cross-sections as weak, so that increased thickness at these areas would be the most appropriate way for improving strength.

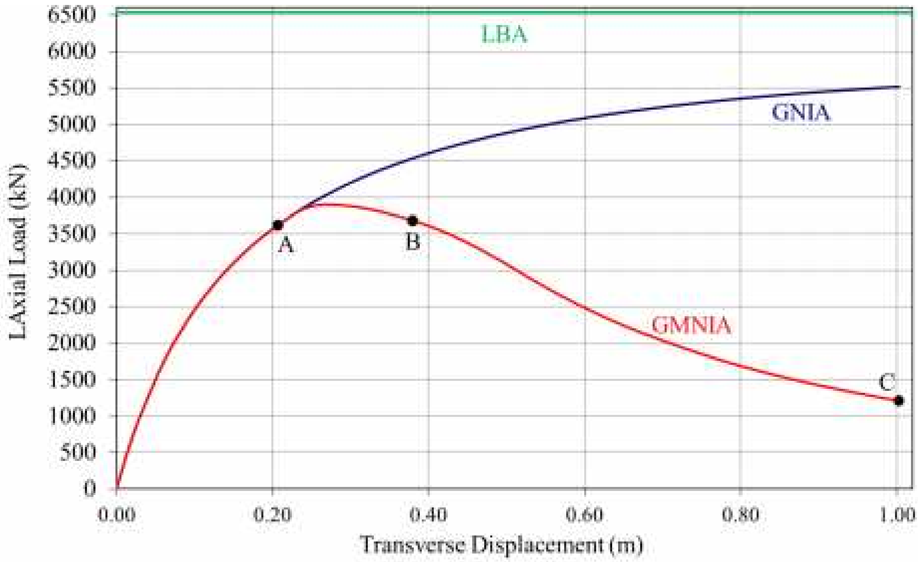
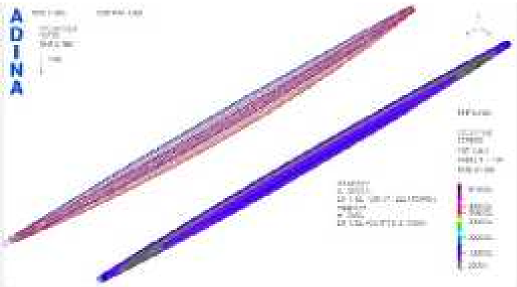
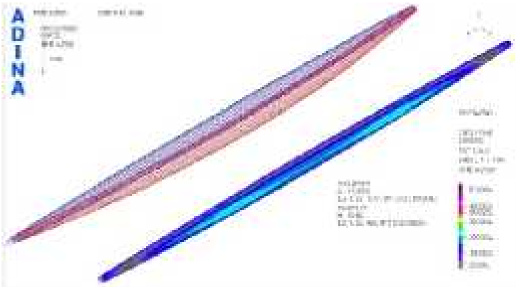


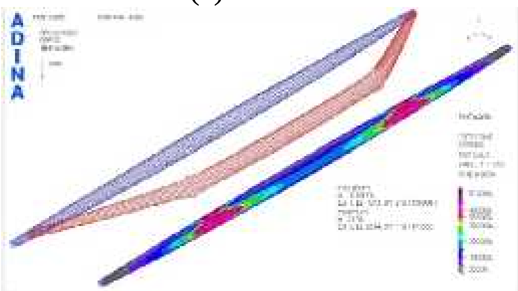
Figure 5: Equilibrium paths of pylon of steel roof over Aristotle's Lyceum



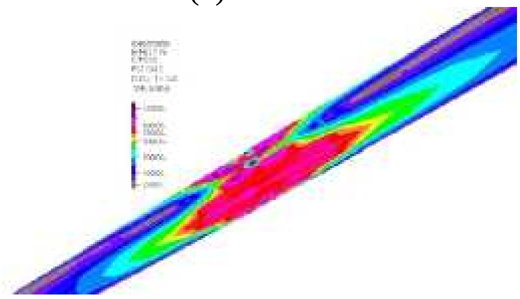
(a) Point A



(b) Point B



(c) Point C

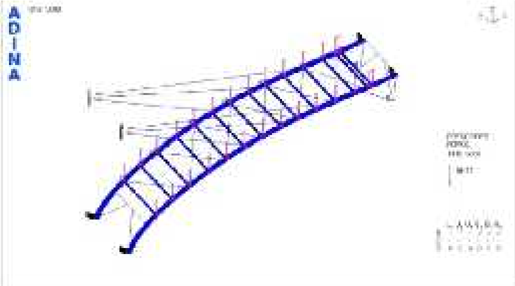


(d) Point C (detail)

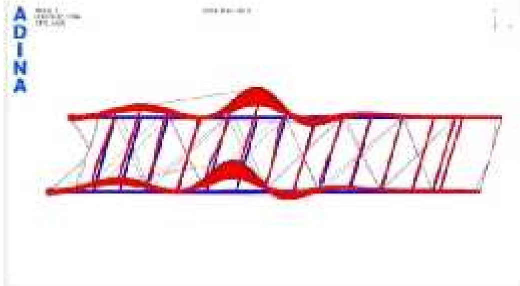
Figure 6: Deformation and von Mises stress distribution at characteristic points along the GMNIA equilibrium path of the pylon (Units: kN, m)

Evaluation of the pylon of the steel roof over Aristotle's Lyceum

In order to evaluate the behavior of the main arches, made of S355 steel, against lateral buckling, use of code provisions is of questionable validity due to the unusual shape of the arches and of their cross-section. Therefore, a finite element model consisting of two main arches and their transverse and diagonal connecting members was analyzed (Figure 7a). Main arches were modelled with shell elements, bracing members with beam elements, and suspension cables by nonlinear, tension-only, prestressed truss elements. By considering two adjacent arches, the lateral support provided to the arches by their bracing system is taken into account. Cross-sections were class 1 and 2, so that local buckling was not critical.



(a) Undeformed model



(b) First buckling mode

Figure 7 Finite element model of two adjacent main arches of steel roof over Aristotle's Lyceum

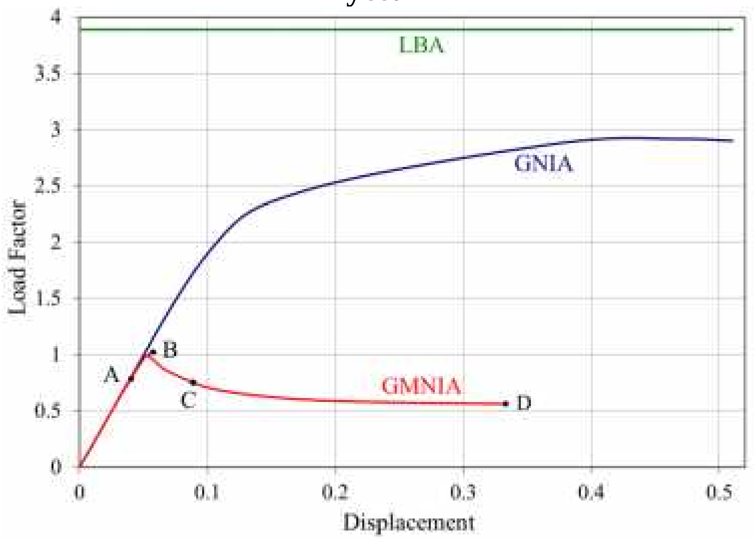
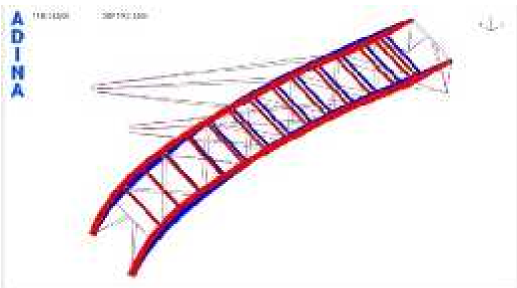
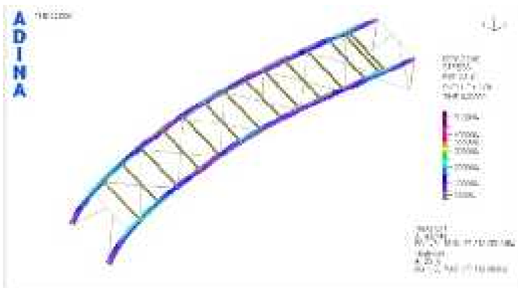


Figure 8: Equilibrium paths of main arches

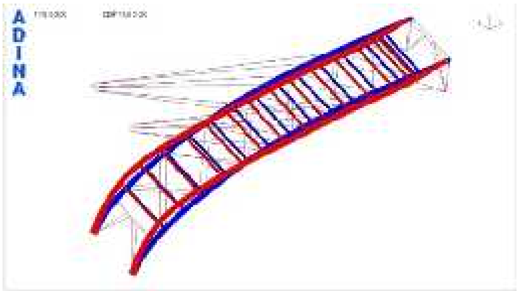
Linearized buckling analysis was first performed in order to obtain the first buckling mode and the corresponding elastic critical buckling load (Figure 7b). This mode was then used as initial imperfection for the subsequent GNI and GMNI analyses. The results are presented by the equilibrium paths (Figure 8), and the snapshots of stress and deformation at characteristic points A, B, C and D (Figures 9 and 10) along the GMNIA path. It is concluded that the box shaped cross-sections combined with the lateral support provided by the bracing system are sufficient for preventing lateral buckling, thus failure is primarily due to material yielding, with the formation of plastic hinges becoming evident from the deformation and stress pictures on the descending branch of the equilibrium path.



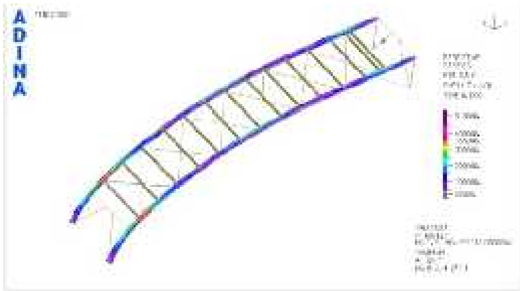
(a) Point A - deformation



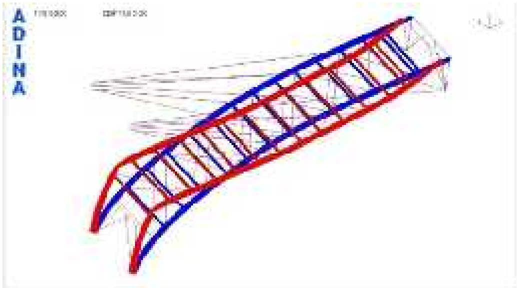
(b) Point A – stresses



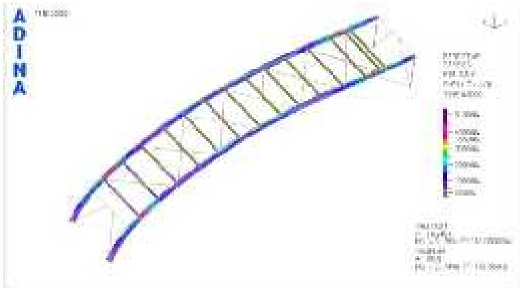
(c) Point B - deformation



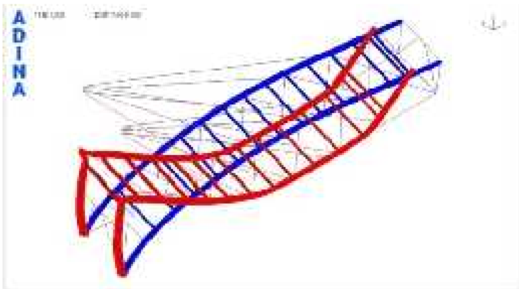
(d) Point B – stresses



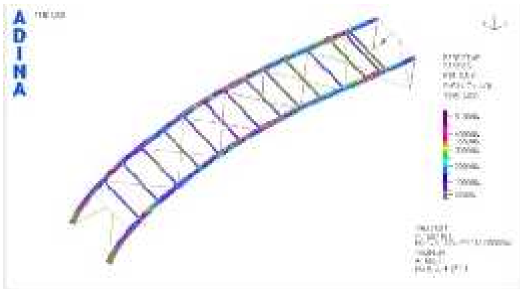
(e) Point C - deformation



(f) Point C – stresses



(g) Point D - deformation



(h) Point D – stresses

Figure 9: Deformation and von Mises stress distribution at characteristic points along the GMNIA equilibrium path of the main arches (Units: kN, m)

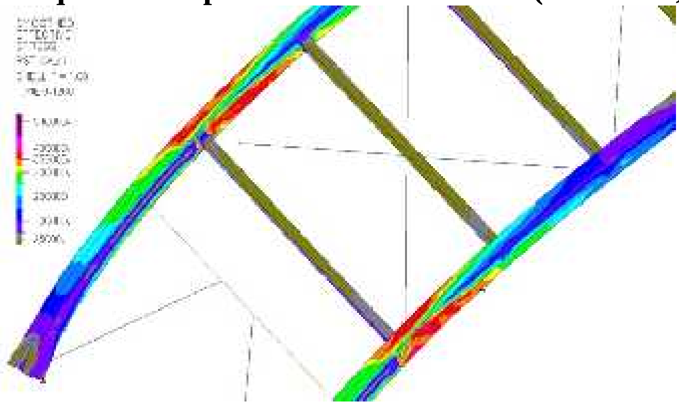


Figure 10: Von Mises stress distribution at characteristic point D along the GMNIA equilibrium path of the main arches (detail) (Units: kN, m)

conclusion

In geometrically complex structural cable systems it is difficult to achieve the desirable tensile function of cables as many of those are found to relax under the service loads. Apart from the fatigue problems that may occur due to relaxation at the edges of the cables, relaxation also creates problems to the convergence of non-linear analyses. The initial prestress magnitude must be close to the final one and the analogy between the prestress of the cables should also be relevant. Moreover, for the prediction of the failure mechanisms and strength of main members of the structure a systematic methodology has been presented by means of nonlinear numerical analysis, making use of commercially available finite element software. Via the presented examples, advanced numerical analysis methods are accessible to practicing structural engineers, by providing guidelines for using such methods for structural design, and by demonstrating some of the capabilities afforded by such methods to the structural engineering community. The procedure described in the present article and performed for the suspended steel roof of the archeological site of the School of Aristotle, can be adopted in similar complex structural cases.

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