The New, Balanced Cantilever, Bridge over Acheloos River in Greece

Chrysanthos Maraveas[1], Konstantina Tasiouli[1] and Konstantinos Miamis[1]
[1]C. MARAVEAS PARTNERSHIP – Consulting Engineers, Athens, Greece.

Abstract: The paper presents the new bridge over the river Acheloos in Greece. The position of the bridge poses several construction and design challenges, mostly due to the soil characteristics at the foundation, the increased seismicity of the region and the high flow of the river. The project involves the construction of a small balanced cantilever bridge with three spans of length 47.5m – 80m – 47.5m. The main innovation in the structure is that the deck is supported exclusively on high damping rubber bearings, a technique not commonly encountered in such bridges. The selection of this structural system was a result of the poor geotechnical / foundation conditions, which differed considerably among the piers, as well as their low height. This led to the design of special deck stabilizing systems for the various construction stages. The design included application of the principles of earthquake engineering, an advanced construction schedule and a displacement monitoring procedure. Information regarding various analysis and design aspects, construction details as well as cost data are also presented.

Keywords: concrete, balanced cantilever, seismic isolation, deck stabilization.

1. Introduction

The new bridge that will be discussed in this paper is part of the Astakos - A5 motorway, which is currently under construction and spans over the river Acheloos. The river, second longer in Greece, is approximately 70m wide at the location of the bridge. It has a high flow (approximately 6m), which is continuous throughout the year. The geology of the region comprises different soil profiles. In the east riverside, limestone is mainly encountered, while deposits of loose sand and clay (with poor geotechnical characteristics) are met in the west. Furthermore, the bridge is located in a high-seismicity region and the different geotechnical conditions across the river may lead to spatial variability of the seismic action. The bridge has three spans and is 175m long. Its carriageway consists of one lane per direction and lies on a straight line in plan and elevation. The total width of the deck (including the sidewalks) is 15.50m. The structural system of the bridge and the construction method were selected to account for the topology, the geology and the seismicity of the region, as mentioned above. The box-girder deck, which will be constructed according to the balanced cantilever method, shall be supported by high damping rubber bearings (HDRBs) on wall-type short piers. This connection of the deck to the piers poses several problems regarding its stabilization during the construction stages of the cantilevers. Special temporary support (consisting of two steel frames, hydraulic jacks and special bearings designed to resist uplift forces and restrain lateral displacements) will be provided during the construction phase of the project. The analysis and design of the bridge was performed according to the Eurocodes and the estimated cost is approximately 3,700,000 €.

2. Structural characteristics of the bridge

2.1 General layout

The bridge is located in the southwest region of Greece, in a valley crossed by the river Acheloos. It is supported on two piers two abutments and has three spans (47.5m – 80m – 47.5m). The general layout is shown in Figure 1 (a)-(b). The two piers of the bridge shall be located in each riverside, because their construction in the river bed with conventional falsework was neither reasonable nor economic due to the continuous flow of the river throughout the year. The fact that the width of the river is approximately 70m at the location of the bridge allowed for the construction of a single span over the river. The construction method and the materials were also determinant factors in the selection of the structural system of the bridge. In the preliminary design, two alternatives were studied. Apart from the solution of a concrete box-girder bridge deck constructed via the balance cantilever method, the alternative of a composite deck (steel beams-concrete slab) via the incremental launching construction method was examined. The main advantage of the concrete box girder bridge against a composite deck is the durability of concrete and the low maintenance cost. For the reasons mentioned above, the first solution was selected. Furthermore, common practice (1) suggests that "the economical range of span lengths for cast-in-place cantilever construction begins at roughly 70m and extends to beyond 250m". According to the topology of the region, the average height of the piers is 5m and the length of the end spans is 47.5m. The structural system of the bridge is modified during the construction sequence, from cantilever to closed frame and finally to a continuous three span girder supported by High Damping Rubber Bearings (HDRB) on the piers and the abutments.

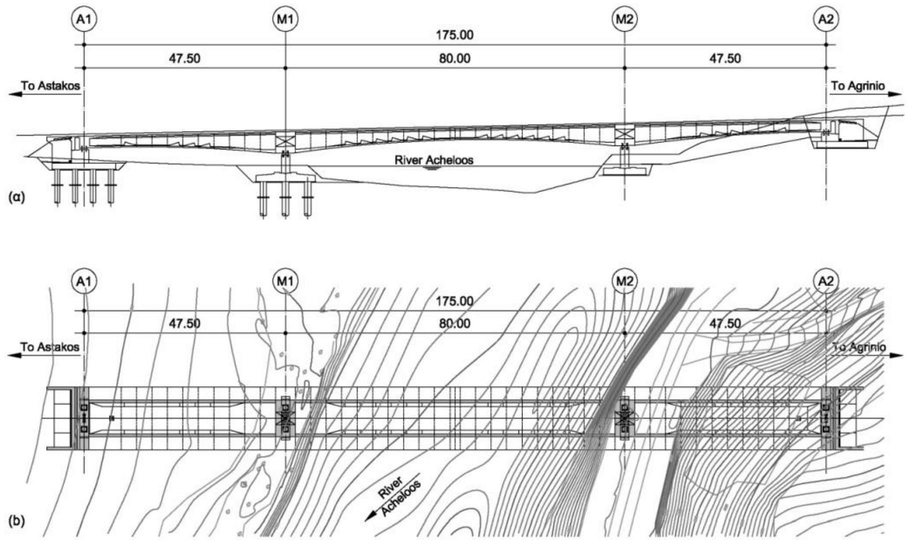


Figure 1. Elevation (a) and plan (b) of the new Acheloos Bridge.

2.2 Superstructure

The deck of the bridge is a prestressed concrete, class C45/55, box girder. Its cross section consists of a single cell box with two vertical webs. In the longitudinal direction, the depth of the girder ranges from 5.00m (sections close to the supports) to 2.75m (midspans). The bottom slab thickness varies from 0.80m to 0.30m, respectively. The cross-sectional depth variation follows a parabolic curve. The remaining dimensions of the cross section are constant along the bridge. More specifically, the web thickness is 0.60m; the top slab is 0.30m thick with 1.50m x 0.60m haunches and a total width of 15.50m. The center-to-center distance of the webs is 8.00m. The cross-sections directly above the supports are designed to form a solid region (called solid diaphragm), to ensure that the deck loads are safely transferred to the bearings or the temporary support. These diaphragms also play the role of stabilizing the cross section under torsion. The specified post tension system is VSL 6S-12 (12 strands with nominal diameter 15.7mm each) with prestress anchorage type GC. The prestressing steel quality is 1570/1770. Typical cross sections are shown in Figure 2(a)-(b).

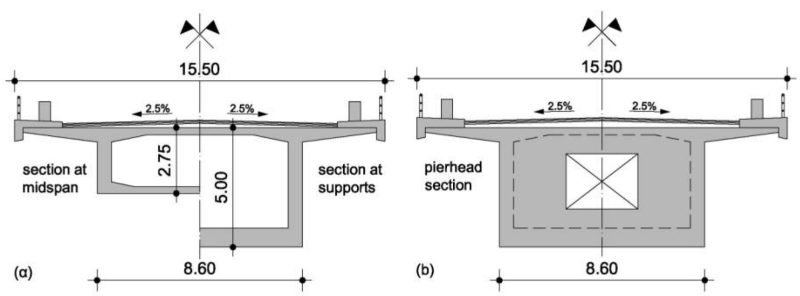


Figure 2. Cross sections of box girder.

2.3 Substructure

The two rectangular 10.20m x 2.00m reinforced concrete piers, M1 and M2, have an average height of 5m. The specified concrete class is C35/45. A thorough geotechnical investigation was carried out in order to determine the foundation conditions for the bridge. Four drillings in total (two per river side), with an average depth of 30m, were carried out along the axis of the bridge. According to the results of the investigation, the geology of the region is characterized by the existence of a stiff limestone layer with a deep slope (east riverside) and newly formed deposits of loose sand and clay with very poor soil characteristics (west riverside). Based on the collected geotechnical data, the soil is classified as type A for the east riverside and as C for the west riverside according to EN1998-1 (2). For this reason, two different foundation types were selected, depending on the soil profile encountered. According to the geotechnical study, 16 piles with a diameter of 1.20m and a length of 31m were deemed necessary to safely transfer the loads from abutment A1 to the underlying soil. These will be connected via a 15m x 15m pile head with a depth of 1.50m. It should be noted that negative skin friction was accounted for in the geotechnical calculations. The foundation of pier M1 consists of 12 piles, 37m long, with a diameter of 1.50m and a 14m x 19m pile cap with variable depth (ranging from 1.50m to 2.0m). On the east riverside, pier M2 and abutment A2 are founded on spread foundations with plan dimensions of 10m x 15m and 12m x 15m, respectively. Their depth is 1.50m. The specified concrete class of the pile heads and spread foundations is C35/45, while that of the piles is C25/30.

3. Seismic isolation

The different foundation conditions along the bridge are expected to result in spatial variability of the seismic action. The varying soil profiles (limestone on one river side and clay/sand on the other) result in different propagations of the seismic wave. Although this phenomenon is taken into account according to the regulations of EN1998-2 (3), it is preferable to isolate the superstructure from the different excitations among its supports. The reason behind this is that the short piers of the bridge are very stiff and their ability to absorb a portion of the seismic action is limited. If the deck were to be connected to the piers monolithically, the earthquake loads induced by the foundation motion would be increased and the spatial variability of the seismic action would adversely affect the structural system of the bridge. Seismic isolation of the deck with the use of two (HDRB) in every pier and abutment led to the reduction of the seismic force to both the deck and the foundation. The specified bearings are Alga HDH1200/144 type and have a viscous damping of 16%. The diameter of the rubber is 1.20m and its thickness is 144mm.

4. Construction phases

As mentioned before, the bridge is constructed according to the balanced cantilever method. Initially, the foundation, piers and abutments are cast with the use of conventional scaffolding. The bearings are placed on the piers. However, because their use does not provide stability for the deck during the construction stages of the cantilevers, temporary support systems are necessary. In general it has been stated (4) that if the deck is designed to be continuous over the piers, it is necessary to either provide temporary fixity between them during the construction phase (by means of wedges and stressed tendons) or to provide temporary supports from the piers. Due to the low height of the piers, a temporary support system fixed to their foundation was selected for this bridge. The pierhead is constructed with conventional scaffolding and is stabilized with temporary prestress tendons (type Algaslab A3F15 or equivalent) that are anchored in the foundation and the flanges of the cross section. After the construction of pierhead M1-0 (Figure 3), the temporary stabilization system of the cantilevers is placed. The length of the pierhead was selected to be 4.60m, for the purposes of assembling the travelers and facilitating the placement of the stabilizing support system. The latter (Figure 4 (a)-(b)) consists of a steel spatial frame, with HEB700 columns, stiffened HEB650 beams and diagonal members with a HEB300 cross-section. All members are joined with bolted connections. The steel frame will be assembled in situ, and will be fixed on the pilehead with mechanical anchors. Hydraulic jacks with a mechanical lock system will be placed on the steel frame to transfer compression arising from possible overturning of the deck. This system can only resist compressive forces, so the bearings and the piers were designed to carry the horizontal and uplift forces during the construction phase. The bearings have anchors and temporary lock plates that restrain horizontal displacements. Therefore, the fixity (in terms of displacements and forces) of the deck with the underlying structural system during the erection phase is provided by the bearings. The asymmetric overturning moment, analyzed in a pair of forces with a lever arm of 2.00m, is resisted both by the steel frame (compressive forces transferred via the jacks) and the piers with the bearings (uplift forces). Following the assembling of the temporary support, the balanced cantilever construction from pier M1 is initiated. The deck is divided in segments of equal length (3.65m), labeled M1-1 to M1-10 and M1-1' to M1-10' (Figure 3). For the erection initiated from pier M1 ten construction stages will take place. In each one of them, which has a duration of seven days, the construction sequence involves the prestress of the previously cast segment, the positioning of the traveler to its new location, the placement of the reinforcement and the ducts and finally the casting of concrete for the new segment. The sequence is repeated for every construction stage until the cantilever is completed. The same progressive construction sequence is repeated for the erection initiated from pier M2. Then the midspan closure segment (M1-M2) is constructed and the prestress tendons of the midspan are placed. Following that, the closures of span A1-M1 and A2-M2 are completed. The temporary support systems are disassembled and the bridge is given to service. Moreover, the required precamber was calculated for every construction stage of the bridge.

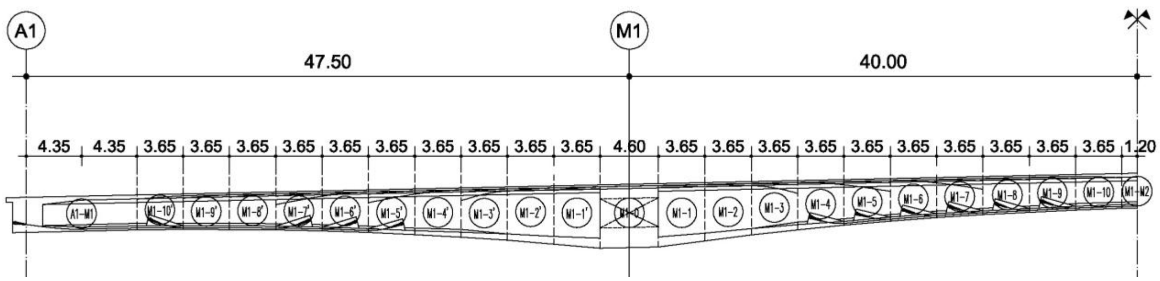


Figure 3. Typical segments of the bridge.

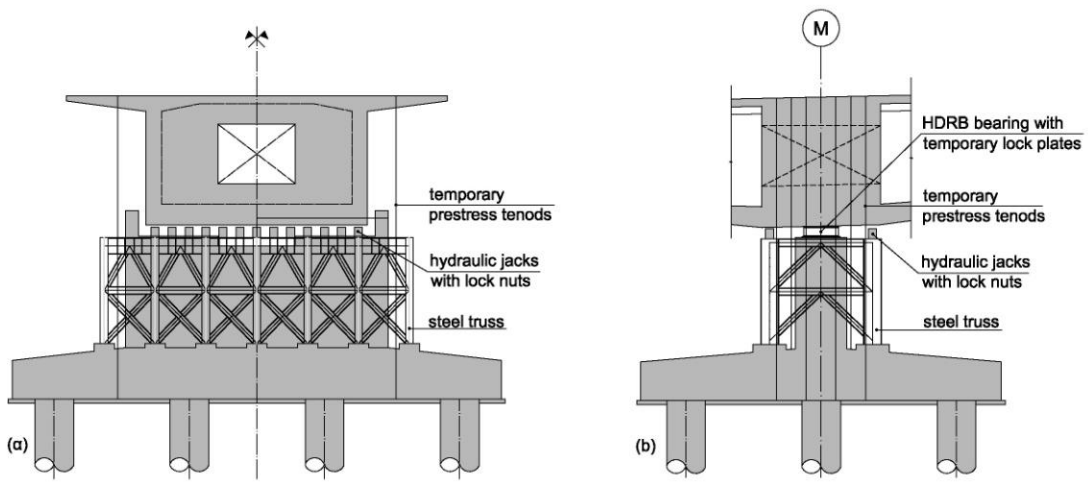


Figure 4. Temporary support systems during cantilever erection.

5. Analysis and design

5.1 Static analyses

For the design of the bridge, an exhaustive series of analyses was performed. The bridge was simulated as a spatial frame using the commercial program RM Bridge 2000 (Bentley) (5) which performs 4D time-dependent analysis throughout the construction progress. The bridge was assumed to be fixed at its supports. The design followed the regulations of the Eurocodes and the necessary checks were performed for every construction stage. The actions during execution were taken into account per EN1991-1-6 (6) and EN1992-2 (7). Their summary is given in Table 1.

Table 1. Loads during construction.

Permanent Loads	
Self weight	25.0 kN/m^3
Weight of wet concrete	26.0 kN/m^3
Traveller	700kN
Live Loads	
Construction loads	1.50 kN/m^2
Wind	0.20 kN/m^2

The initial applied prestress force in each tendon is approximately 2400kN. The losses were calculated per EN1992-1 (8). During the cantilever construction, the prestress layout consists of straight tendons (with small deviations in the anchorage zones) placed in the upper flange of the girder in order for hogging moments to be resisted. The effects of shrinkage and creep were calculated according to EN1992-1 (8) for every construction stage depending on the age of each segment. The structural adequacy checks during construction are identical with those for the completed structure. These are performed for the combinations of actions specified in EN1990 (9). More specifically, serviceability was satisfied by limiting the compressive and tensile stresses in concrete and steel, in conjunction with performing checks for the control of cracking. Regarding the ultimate limit state, the bending and shear resistance of the girder were calculated and the longitudinal reinforcement required to avoid brittle failure caused by inadequate prestress was determined. Moreover, for the verification of static equilibrium and structural adequacy during the construction stages, an accidental action caused by loss of the traveler or asymmetric cast of one segment was taken into account. This action is critical for the design of the temporary support system, because it results in the maximum overturning moment.

Following the design of the cantilever structure, analysis of the completed bridge (after the closure of the midspan and the endspans) was performed. In the working stage of the bridge, the modification in the structural system was taken into account. Due to the construction sequence of the bridge and the parabolic variation of the height of the beam, the prestress layout of the bottom flange consists of tendons with parabolic shape, which are anchored in special blisters. A typical prestress layout is shown in Figure 5.

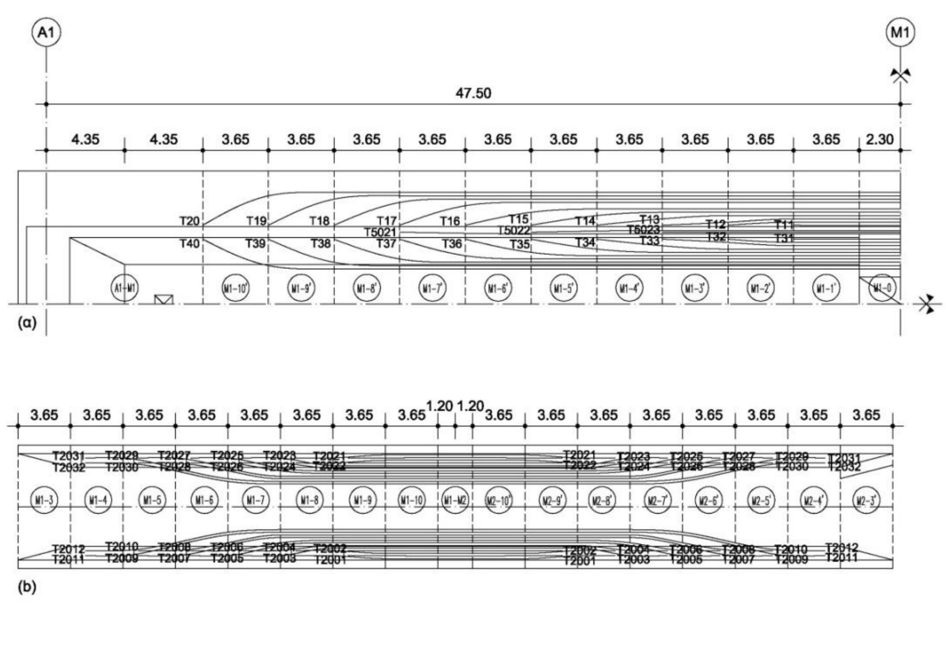


Figure 5. Prestress layout (a) top slab (b) bottom slab.

The final structural system of the bridge is a continuous girder connected to the piers with HDRBs. The bridge is designed to carry traffic loads according to load model 1, as specified in EN1991-2 (10). During the working life of the bridge, actions arising from superimposed permanent loads (asphalt layers and sidewalks etc), thermal actions per EN1991-1-5 (11) and wind actions per EN1991-1-4 (12) were taken into account in addition to the traffic loads. A relevant summary is given in Table 2.

Table 2. Loads during the working life of the bridge.

Permanent Loads	
Self weight	25.0 kN/m^3
Weight asphalt layers	24.0 kN/m^3
Live Loads	
Traffic Load	Load Model 1 (LM1)
Thermal Action	T_{max} =45 ^oC T_{min}=-15^oC (area Astakos, Greece)
Wind	V_{bo}=33km/h, terrain II

5.2 Dynamic analyses

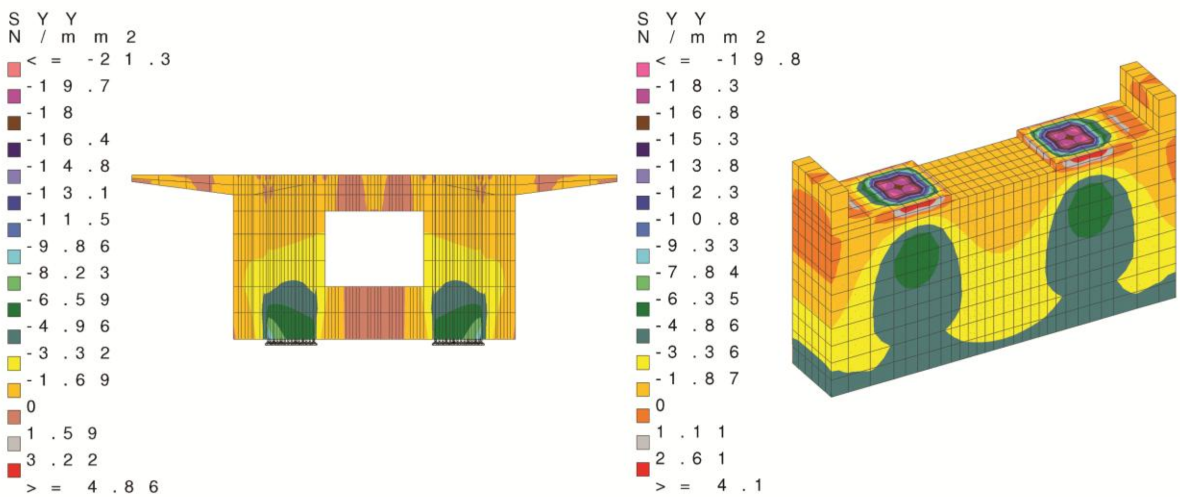
Due to the high seismicity of the region, the earthquake response of the bridge is a crucial aspect in its design. The response spectrum adopted for the analysis was selected according to the regulations of EN 1998-1 (2). The maximum design ground acceleration is a_g=0.24g and the importance class of the bridge is I. The introduced seismic action reflects an earthquake with a return period of 475 years and a probability of exceedance in 100 years equal to 0.19. During the construction stages, this seismic action is reduced via a factor of 0.50 (per EN1998-2 (3)). According to EN1998-2 (3), spatial variability of the seismic action should be taken into account when more than one ground types are encountered in the foundation regions of bridges with a continuous deck. Type 1 elastic spectrum (2) was selected for the response spectrum analysis of the bridge (which was performed for the construction phases and the completed structure). The dominant factor in the response of the bridge is the seismic isolation provided by the use of HDRBs. This modification to the structural system leads to seismic force mitigation via stiffness reduction and increase of damping. More specifically, the horizontal stiffness of the bearings is k=11000kN/m and their viscous damping is 16%. This is approximately 220% higher than the damping of the conventional monolithic structure. If the use of HDRBs was not implemented, (monolithic structure) the response of the bridge would fall within the plateau of the design response spectrum. The use of seismic isolation devices, however, increased the effective period of the structure (to T=1.90sec), and the response fell in the descending part of the spectrum (lower accelerations). Moreover, the increased damping of the system reduces the displacements of the deck. Furthermore, the ductility of the piers is very limited and, although a behavior factor could be adopted for the longitudinal direction, the design forces induced to the foundation would be those corresponding to the capacity design of the piers. This would lead to oversized foundations for the bridge.

Summarizing, multi-response spectrum analysis was performed for the final structural system of the bridge. The increased effective damping of the system was taken into account by modification of the response spectrum for the periods greater than 0.80T_{eff}, as mandated by EN1998-2 (3). All response spectrum analyses were performed for the most severe ground type (category C). Additionally, the effects of spatial variability were taken into account according to EN1998-2 (3) by imposing individual sets of differential displacement to the base of the supports and combining the most severe results with those of the response spectrum analysis according to the SRSS rule.

5.3 Additional analyses

Apart from analyzing the bridge as a spatial frame, additional sets of individual analyses were performed. These included: i) analysis of the box girder in the transverse direction, ii) 3D simulation of the solid transverse diaphragms of the bridge deck iii) 3D analysis of the piers and abutments, iv) analysis of the foundations and v) analysis of the steel support frames.

More specifically, the concrete box girder was analyzed in the transverse direction. Its cross-section was simulated as a closed frame (by the use beam elements) for one unit length. Results were used for the detailing of the box section. Analysis of the transverse diaphragms of the bridge deck was performed for the construction stages, in order to determine the stress-state in the region of the bearings and the jacks of the temporary support system. In this case, the actions causing the most onerous effect were due to accidental loss of the traveler. In the working stage of bridge, the most adverse stress-state in the concrete region close to the bearings was caused by the combination of actions resulting in the maximum vertical load. The solid diaphragm was simulated with 8-noded solid elements in the commercial analysis software STAAD Pro 8Vi (13). Based on the resulting stresses, the required amount of reinforcement was calculated. It was also verified that compressive stresses in the concrete did not exceed the specified design strength. In Figure 6, the vertical stresses for the solid transverse diaphragm of the deck and the piers are presented. The same simulation and analysis was performed for the piers and the abutments. It has to be noted that the results of these analyses where verified with the results of analytical truss analogy mathematical expressions. For the analysis/design of the foundation, four separate models with beam elements (one for the foundation system of each abutment/pier) were created, with the reactions obtained from the analysis of the spatial model being introduced as loads. The support frame was modeled as a spatial frame with beam elements. Analysis results were then used for its design.



(a) Vertical stress contours for the solid diaphragm (b) Vertical stress contours for the pier

Figure 6. Vertical stresses for the maximum vertical loads.

6. Cost data

The bridge estimated cost is 3,700,000 €. A summary of the quantities of the materials is shown in Table 3.

Table 3. Quantities of materials.

Prestress Concrete C45/55	3000 m^3
Reinforced Concrete C35/45	2200 m^3
Reinforced Concrete C25/30 (piles)	1400 m^3
Prestress Steel Y1770S	75000 kg
Reinforcing Steel	1150000 kg
Alga HDRB 1200/144	8 pieces

7. Conclusions

The new bridge presented here is part of the Astakos - A5 Motorway, which is currently under construction. It is crosses the river Acheloos, which is the second longer in Greece, and will contribute significantly to the urban development of the region. The topology and geology of the region posed several challenges regarding the selection of the structural system and the construction method. The selection of a concrete bridge in contrast to a steel one was more economic, because the durability of concrete reduces the maintenance cost. Moreover, a possible repair cost resulting from steel corrosion due to the wet environment was avoided. The cantilever construction method was deemed the most efficient for this project, as it allowed bypassing several constructional difficulties arising from the use of conventional scaffolding in a river with continuous flow. To deal with the high seismicity of the region, seismic isolation of the bridge via HDRBs was proposed. This solution led to the stabilization of the deck during cantilever construction via temporary steel frames. Due to the low height of the bridge, the temporary stabilization systems will be supported on its foundation in order to avoid structural intervention/modifications to the piers.

8. References

1. Menn, C., “Prestressed Concrete Bridges”, Birkhäuser Verlag, 1990, Berlin, Germany.
2. European Committee for Standardization, “Eurocode 8: Design of structures for earthquake resistance - Part 2: Bridges, European Standard EN 1998-2”, 2005, Brussels, Belgium.
3. European Committee for Standardization, “Eurocode 8: Design of structures for earthquake resistance - Part 1: General rules, seismic actions and rules for buildings, European Standard EN 1998-1”, 2004, Brussels, Belgium.
4. Mathivat, J., “The Cantilever Construction of Prestressed Concrete Bridges”, John Wiley & Sons Ltd., 1983, Great Britain.
5. Bentley Systems Incorporated, “RM Bridge V8i 08.10.03.02 Design Software”, 2002, California, U.S.A.
6. European Committee for Standardization, “Eurocode 1: Actions on structures - Part 1-6: General actions - Actions during execution, European Standard EN 1991-1-6”, 2005, Brussels, Belgium.
7. European Committee for Standardization, “Eurocode 2: Design of concrete structures - Part 2: Concrete bridges - Design and detailing rules, European Standard EN 1992-2”, 2005, Brussels, Belgium.
8. European Committee for Standardization, “Eurocode 2: Design of concrete structures - Part 1-1: General rules and rules for buildings, European Standard EN 1992-1-1”, 2004, Brussels, Belgium.
9. European Committee for Standardization, “Eurocode: Basis of structural design, European Standard EN 1990”, 2002, Brussels, Belgium.
10. European Committee for Standardization, “Eurocode 1: Actions on structures - Part 2: Traffic loads on bridges, European Standard EN 1991-2”, 2003, Brussels, Belgium.
11. European Committee for Standardization, “Eurocode 1: Actions on structures - Part 1-5: General actions - Thermal actions, European Standard EN 1991-1-5”, 2003, Brussels, Belgium.
12. European Committee for Standardization, “Eurocode 1: Actions on structures - Part 1-4: General actions - Wind actions, European Standard EN 1991-1-4”, 2003, Brussels, Belgium.
13. Bentley Systems Incorporated, “STAAD.Pro V8i Structural Analysis and Design Software”, 2007, California, U.S.A.