SEISMIC UPGRADING OF TWO EXISTING BRIDGES: FROM CONCEPTUAL DESIGN TO CONSTRUCTION

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ABSTRACT

Upgrading of older bridges in order to satisfy the current standards and specifications has become a necessity for modern motorways in developed countries. Two specific cases of existing bridges that were retrofitted are examined in this paper, namely river Selinountas bridge and Aigio overpass bridges, both on Olympia Odos motorway, Greece. The conceptual design of the bridges is discussed indicating the main reasons for which retrofit was required and the engineering solutions that were utilized to achieve the design goals. The design of the retrofit solutions was based exclusively on the Eurocode standards (EN 1990 to EN 1998). Design aspects and construction details are presented for the finally constructed solutions. Specific constructions issues addressed during the construction are discussed in detail.

Keywords: Bridge; Retrofit; External Prestress; Fiber Reinforced Polymers

1. INTRODUCTION

The topic of this paper is the retrofit of four existing bridges of the EKPPT Motorway (Elefsina-Korinthos-Patras-Pyrgos-Tsakona) of Olympia Odos.

- Existing motorway bridge B289 of Selinountas river at Ch. 85 + 674.
- Existing overpass bridges A297, A296 and A294 located respectively at Ch. 89 + 293.50, Ch. 88 + 812.43 and Ch. 88 + 083.97.

The existing bridges were designed in 1967 and were constructed between 1968 and 1969. The retrofit design of the bridges was performed by DENCO engineering company (DENCO Engineering Consultants P.C. and DENCO Structural Engineering P.C.). The retrofit works began in 2014 and the upgraded bridges were released in April 2017.

2. RETROFIT OF RIVER SELINOUNTAS BRIDGE

2.1 Existing Structure

The existing bridge of the Korinthos-Patras segment of Olympia Odos motorway at the location of Selinountas river (Figure 1) is straight with 145m total length and consists of three simply supported spans 47.0m+48.2m+47.0m. The bridge consists of two independent branches with 12.4m width each, where each one facilitates one direction of the motorway. The deck of each branch consists of 5 cast in-situ prestressed girders with 2.65m depth that are connected together with 0.20m cast in-situ concrete slab and are supported on elastomeric bearings. The piers and abutments are wall-shaped elements with 15m height that are founded on footings at large depth, approximately 10m below the river bed. The general condition of the bridge reinforced concrete was very good.

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The design drawings and calculation reports of the original design are available but do not fully correspond to the constructed bridge. In particular the skewness angle of the constructed bridge is much larger than the one specified in the original design (39° vs 20°) and for this reason 5 prestressed girders per branch have been constructed instead of 4 in the original design. Moreover, the applied prestress was much different (32 prestressed bars Ф26 Y1050 instead of 9 tendons 12T0.5" Y1570 in the original design).

2.2 Necessity for Retrofit

The necessity for upgrading of the existing structure resulted in order to satisfy the following two main requirements based on the specifications of Olympia Odos motorway:

1. The typical cross-section of the motorway cannot be implemented inside the width of the existing bridge. In particular, an extension of the width by about 1.4m is required for each branch of the bridge. The available prestress and the available reinforcement of the deck are not sufficient for the increased loads resulting from the above extension.

2. The bridge is located in a high seismicity area near Aigio. Taking into account the requirements of modern anti-seismic standards and the proximity of the bridge to active faults, the design peak ground acceleration for bridge is specified equal to 0.36g. The existing structure does not have the required reinforcements at the base of the piers or the required foundation to satisfy the above requirement. In addition, the existing ~5cm thick elastomeric bearings cannot undertake the resulting seismic movements.

It was mandatory for the motorway to remain fully operational during construction. The retrofit works were planned for execution on one branch while the total motorway traffic was diverted to the other branch. The construction scheme was not prohibitive since road pavement reconstruction works were planned with a similar scheme for the whole length of the motorway at the same time.

Figure 1. Side view of river Selinountas bridge prior to the retrofit works (top) and after retrofit works (bottom)
2.3 Side Extension of the Bridge Deck

The side cantilevers of the deck are extended by 1.40m in order to realize the new typical cross-section of the motorway as shown in Figure 2. The extension was realized by additional C30/37 reinforced concrete elements (item (a) in Figure 2). In order to realize the required increased thickness of 50 cm at the root of the cantilever, additional SC30/37 shotcrete strengthening was made at the bottom of the deck plate (items (b) in Figure 2). The shear connection at the interface between existing and new concrete was achieved in the lower part with Φ12 dowel bars. The lower transverse reinforcement penetrates the web of the existing girder through holes. The upper transverse reinforcement is anchored by bonding. For this purpose, an 8cm thick layer of concrete was removed with hydro-blasting from the upper surface of the existing slab for a width of 2.66m. At the exposed area the upper reinforcement of the cantilever was anchored inside a high strength repair mortar layer (item (c) in Figure 2). The forces are transferred in the existing deck slab by the increased bonding. Finally, a standard SG-9 safety barrier was constructed based on the requirements for vehicle securing of the motorway (item (d) in Figure 2).

![Figure 2. Extension of the deck cantilevers by 1.40m](image)

2.4 Application of External Prestress

Based on field investigations that included magnetic measurements and revealing of the tendon anchorage, 32 pre-stretched Φ26 diameter indented bars of Y1050 grade steel were found in the girders of the deck. This prestress is sufficient for the motorway loads only for the internal girders. Conversely, due to the extension of the deck, the prestress is not adequate for the external girders.

The additional prestress required for the external girders was realized by external tendons of type EX-66Φ7 made of high-strength steel Y1570. Two symmetric external tendons were provided on each exterior beam, one for each face of the girder. The external tendons have trapezoidal layout with two steel deflection elements located approximately at L/4 and 3L/4 (item (b) in Figure 3). At these locations transverse cross-beams exist on the concrete deck that help in the distribution of the concentrated force at the tendon deviation. The steel deflector elements are made of welded S355 grade steel plates and are illustrated in detail in Figure 4. The stability of the deflector element is ensured by two transversely prestressed bars with Φ26.5 diameter made of Y1030 steel. The prestressed bars compensate for the moment introduced by the deflection of the force of the external tendons. All exposed metal plates of the deflector are galvanized for protection against corrosion.

The anchorage of the external tendons at the ends of the girders is carried out with reinforced concrete
anchoring blocks made of C40/50 fiber reinforced concrete (item (a) in Figure 3). The anchorage blocks are frictionally connected to the existing beam. The required friction for the safe transfer of forces is achieved by 8 transverse prestressed bars with Φ47 diameter made of Y1050 steel (Figure 5). Horizontal strips of Fiber Reinforced Polymers (FRPs) were placed on the girder surface in order to anchor 25% of the prestressing force at the end of the girder (Figure 6, left). For this purpose, the back face of the girder was suitably curved. Quartz sand was applied on the FRPs to increase the friction at the location where the anchorage block is constructed. The position of the prestressed bars and the general geometry of the anchorage block are fully determined by the locations where drilling of holes could be performed without affecting the existing internal tendons. The above locations were determined by on-site magnetic measurements (Figure 6 right).

The tensioning process of the two symmetrical tendons of each girder was performed in increments of 10 steps, with each step alternating between the two tendons. In this way, the deviator element remains stable since the moments of the deviating forces are introduced symmetrically.

Figure 3. Additional external prestress for the exterior girders. Top: tendon geometry in plan view and longitudinal view. Bottom: view of the end part of the external tendon where the anchorage block (a) and the deviator (b) are visible

Figure 4. Close-up of the external prestress deviator at L/4 and 3L/4 of the girder. The stability is achieved by two transversely prestressed bars (Φ26.5, Grade Y1030).
Figure 5. Close-up of external prestress anchorage block at the ends of the girder. The stability of the two adjacent symmetric anchorage blocks is achieved by friction with the existing girder and 8 transversely prestressed bars (Φ46, Grade Y1050) as shown in the insert drawing.

Figure 6. Formation of the existing girder surface before construction of the anchorage block (left). The location of holes for the transverse prestress bars have been determined by on-site magnetic measurements (right).

2.5 Strengthening of deck with Fiber Reinforced Polymers (FRPs)

The application of Fiber Reinforced Polymers (FRPs) aims to increase the strength of the deck at the locations where this is required (Figure 7). Specifically, the following strengthening was required:

a) increase of the bending strength at the lower flange of the external girders by application of 5 layers of composite carbon fibers with 1mm thickness and 600mm width for each layer.

b) increase of the shear strength at the ends of all girders by application of 2 layers of U-shaped composite carbon fibers that embrace the lower flange of the girders. The distance between the strips is variable depending on the shear demand. At the locations where the external prestress anchorage blocks are constructed the shear strengthening was realized with an equivalent configuration of 23 composite carbon strips with 20 mm by 3 mm cross-sectional area applied at each girder face, which are inserted into appropriate vertical grooves on the girder surface (Figure 6, left).

c) Application of horizontal composite carbon fiber strips at the location of the external prestress anchorage blocks in order to anchor 25% of the external prestress force at the end of the girder (Figure 6, left). Strips with 1m total width were placed on either side of the transverse bars. Composite carbon fibers with 1 mm layer thickness were used. The tensile strength of the
composite material is 986MPa whereas the tensile strength of the carbon fibers is 3100MPa.

Figure 7. Application of Fiber Reinforced Polymers (FRPs) for the bending and shear strengthening of the deck

2.6 Seismic Isolation with Friction Pendulum Bearings (FPS)

The large foundation depth of the bridge piers, approximately 10m below the riverbed, makes prohibitive any intervention concerning the increase the pier strength at their base and their foundation. For this reason, it is preferable instead of increasing the strength to reduce the seismic demand by providing a seismic isolation system. In particular, 60 triple friction pendulum (FPS) bearings were used, replacing the same number of existing bearings at the ends of each girder. The triple friction pendulum bearings consist of a teflon coated sliding element which slides in contact with stainless steel curved surfaces. The response of the friction pendulum bearings during seismic excitation corresponds to a low friction sliding on a curved surface with large radius. The nonlinear hysteresis behavior leads to an elongation of the effective period of the structure and additional energy dissipation due to the friction. The combination of the above factors leads to a significant reduction of the seismic demand on the substructure. The increased seismic displacements corresponding to the elongated natural period are limited by the increased damping.

The equivalent properties of the seismic isolation are: average friction coefficient $\mu = 0.09$, radius of curvature $R = 2.92m$, effective period $T_{eff} = 2.30s$, effective damping $\xi_{eff} = 34\%$, required displacement capacity +/- 320mm. In this design, the variability of the nominal friction value was considered equal to +/- 25%. Additional variability of the effective coefficient of friction due to time-dependent phenomena, such as ageing, temperature, and contamination were estimated according to EN1998-2, Appendices J and JJ. The corresponding overall modification factor for time-dependent phenomena was $\lambda_{max} = 1.30$. The total variability of the coefficient of friction was taken into account with two analyses, as specified in EN1998-2 Section 7: a) lower bound analysis with effective coefficient of friction $\mu = 0.09\cdot(1 - 0.25) = 0.0675$ and b) upper bound analysis with effective coefficient of friction $\mu = 0.09\cdot(1 + 0.25)\cdot1.30 = 0.146$. In general, upper bound analysis leads to maximum seismic forces in
The structure, and lower bound analysis leads to maximum displacements in seismic isolators.

The seismic isolation bearings were connected with the existing substructure and superstructure with special high bond resin. This connection is sufficient to transfer the horizontal forces as they are limited by the low value of the friction coefficient. The thickness of the seismic isolators is about 25cm, while the thickness of the existing elastomeric bearings is about 5cm. Therefore, a significant lifting of the deck was required by about 20 cm for the placement of the final bearings. The uplift of the deck by 20cm required a modification of the final motorway level for a significant length before and after the bridge. This was not prohibitive since the wearing surface of the existing motorway pavement was reconstructed entirely according to the new specifications of the project.

The flexibility introduced by the seismic isolation system leads to the requirement to connect the independent branches so as to avoid collision. The three independent spans were connected with 30cm thick continuity plates and the two independent branches were connected with 0.5m wide strips made of C30/37 reinforced concrete. In this manner the total of the superstructure behaves as a single rigid body above the isolation system. The existing expansion joints at the deck ends are replaced with large movement expansion joints. The upstand wall of the abutments is replaced with a new structure that permits a displacement gap of 0.45m that is adequate for the movement of the isolated deck.

2.7 Reduction of the earth pressure behind the abutments by lightweight EPS embankment

The principal demand on the abutment and its foundation results from the seismic earth pressure which is not affected by seismic isolation. In addition, the great depth of the foundation, about 10m below the riverbed, makes prohibitive any intervention at the base of the abutment. For this reason, instead of increasing the strength, it is preferable to reduce the earth pressure by replacing a part of the embankment with lightweight Expanded Polystyrene (EPS) material (Figure 8). The expanded polystyrene has a very low weight, in the order of 30 kgr/m$^3$ and produces negligible horizontal pressure. At the same time EPS has high strength to vertical traffic loads. Part of the embankment behind the abutments with 8m length and 5m depth was replaced by expanded polystyrene of type EPS150 with a nominal resistance at 10% strain equal to $\sigma_{10\%} = 150$ kPa according to EN14933. The EPS pieces have size 2m x 1m x 1m and are layered alternately in order to avoid joints in the same position. Above the EPS material a reinforced concrete slab with 30cm thickness from C20/25 concrete is constructed in order to protect the EPS and distribute evenly the concentrated wheel loads.

![Figure 8. Replacement of embankment behind the abutments with lightweight EPS material](image)

3. RETROFIT OF OVERPASS BRIDGES

3.1 Existing Structures

The three similar existing overpasses are located in the wider area of Aigio region. Their design and
construction dates back to 1969 to 1971 and since then they present an important landmark for the region. The existing National Road in this area is part of the road alignment of the new Motorway Road of Olympia Odos. Consequently, their preservation and structural upgrading in addition to a technically and economically preferable solution in contrast to the solution of the demolition and construction of new bridges in the partially operational phase of the existing National Road further satisfies the need to preserve the structural heritage of the area.

The overpasses consist of a special static system with supports on V-shaped columns, which is their dominant visual characteristic. The main deck structure consists of three span prestressed circular-voided slab concrete. The total width of the deck is 9.65m and the slab thickness varies for the three overpasses from 1.00m (A294) to 1.12m (A297). Their central span ranges from 28.0m (A294) to 34.0m (A297). The deck rests at each support position on a pair of inclined columns which end up every four of them on a common footing with inclined bearing surface (Figure 9). The cross-section of the columns is in the form of a parallelogram with fixed shape for the columns supporting the ends of the deck and variable width in the transverse direction for the columns supporting the central span. In addition, the main part of the deck is complemented by two simply supported spans of approximately 4.50m in length, consisting of a solid 0.40m thick slab supported on the main deck and on the corresponding abutments. Figure 9 shows a longitudinal section of the overpass A296, Figure 10 (left) a general view of the overpass A296, while Figure 10 (right) a view of pier M1 of overpass A296.

The above described geometric configuration of the pier columns functions in the longitudinal direction as a lift system. In the transverse direction, the columns in pairs are connected with a cross-beam inside the depth of the deck slab forming 4 separate one-span frames.

The design drawings and calculation reports of the original designs are partially available and are in very good agreement with the constructed structures as was revealed by an extensive survey and test program. The condition of the structures is in general terms good considering their age (~45 years) and the high seismicity of the region.

3.2 Necessity for Retrofit

Three main reasons that led to the necessity of upgrading the three structures were:
1. The high seismicity of the region which dictate design peak ground acceleration of 0.31g (overpass A294) increased to 0.36g due to proximity to active faults (overpasses A296 and A297). The existing structures were originally designed for response acceleration factor of ε=0.08 which implies many structural inadequacies as well as soil inadequacy. Specifically, the existing bearings and expansion joints are inadequate for the resulting displacement demands, the cross-beams on the deck show flexural and shear inadequacy, the base of the columns present shear inadequacy, the footings have inadequate thickness and also the foundation soil is incapable to resist the resulting actions.

2. The structures were designed for traffic load of class 30tn, while the project requirements specify a higher class of 60/30tn. That led, in some cases, to inadequacies of the shear reinforcement of the deck slab.

3. One last non-technical reason but a fairly important one is the preservation of landmark established by the three bridges while crossing the Aigio area over the last half century.

3.3 Replacement of Existing End Spans and Abutments

The retrofit solution includes the construction of new abutments capable of undertaking considerable seismic loads in the transverse direction. For this purpose, they should be founded on piles. New end spans are constructed in the position of the demolished existing spans to transfer the forces from the retained main part of the deck to the new abutments. The new spans are monolithically connected to the main deck while they transfer forces to the abutment in the transverse direction through vertically positioned elastomeric bearings. In the longitudinal direction no forces are transferred since the connection to the abutment is supplemented by a pair of pot bearings with sliding capability in both horizontal directions that undertake only vertical loads. Figure 11 shows an elevation view of the abutment.

3.4 Stiffening and Strengthening of Cross-Beams

At the positions of the deck cross-beams connecting pairs of the inclined columns of the piers, flexural strengthening is achieved by adding longitudinal reinforcing bars which are incorporated into the bottom side of the existing cross-beams with 0.20m thick shotcrete and shear strengthening is achieved by incorporating vertical reinforcing bars connected to the concrete with resin through the thickness of the cross-beam. Figure 12 shows a drawing of the interventions at the end cross-beam (towards the end of the deck). Figure 13 shows the hydro-blasting of the upper part of a cross-beam. The works were performed with caution to keep intact the existing reinforcements and the prestressing tendons ducts. Figure 14 shows an underside view of a retrofitted cross-beam. Finally, Figure 15 shows on a cross-beam the holes through which the vertical reinforcements for the shear strengthening will pass.

Figure 11. Configuration of new abutments and location of bearings: (a) Elastomeric bearings positioned vertically. (b) Pot bearings with sliding capability that undertake vertical loads
Figure 12. Interventions at the end cross-beam: (a) Flexural strengthening with addition of longitudinal reinforcing bars. (b) Shotcrete 0.20m thick. (c) Shear strengthening with addition of vertical link legs.

Figure 13. Hydro-blasting of concrete of the upper part of a cross-beams. (the grouting pipes of the ducts of the tendons are distinguished).

Figure 14. View of the shotcrete at the bottom of the cross-beam (underside view). The locations of sampling cores taken for the concrete strength verification are distinguished.

Figure 15. Opening of vertical holes through the cross-beams for the placement of reinforcing bars for their shear strengthening (their protection is distinguished).
3.5 Strengthening of Deck / Columns / Footings

Shear strengthening of the mid span of the deck slab is implemented by addition of vertical shear links connected with the concrete with resin.

The columns piers towards the center of the bridge, which are of variable width, are strengthened concerning their shear capacity by wrapping them with two layers of fiber reinforced polymer (FRP). The strengthening concerns a part of their length from their base up to a certain height, while the extension of the application of the fabrics with a single layer up to the top of the columns was made for durability reasons.

The footings were required to be strengthen by increasing their thickness with additional reinforced concrete. The monolithic connection to the existing concrete was provide through reinforcements anchored with resin. Figure 16 shows the status of the works after placing the anchors and before concreting the additional footing thickness that will also integrate the connection with the column base.

3.6 Strengthening of Foundation Soil

Finally, strengthening of the foundation soil against the risk of sliding along circular failure surface was required by constructing in front of the footings (towards the motorway) of a pile-wall with sufficient depth to prevent the above risk. Figure 17 shows a drawing with the proposed pile-wall, while Figure 18 shows the constructed pile-cap and the strengthened footing.

Figure 16. Preparation of footings for concreting of their additional thickness (connection reinforcement between existing and new concrete is shown)

Figure 17. Drawing showing the footing thickening and the pile-wall in front of the existing footing: (a) Piles. (b) Pile-cap of the pile-wall. (c) Thickening of the footing
4. CONCLUSIONS

For the first time in Greece retrofit of major bridges has been designed and constructed in order to fulfil the increased service and seismic demands of modern motorways. The optimization of the intervention methods in order to achieve the specifications of the project while the motorway remained fully operational has been a challenge for the designer of the project. The documentation of the interventions was made by applying, for the first time on bridges, of the provisions of EN 1998 – Part 3 and of the guidelines of KAN.EPE.

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6. REFERENCES

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