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ANALYTICAL AND EXPERIMENTAL RESEARCH ON THE POSSIBILITIES OF ATTAINING A MONOLITHIC CONNECTION WITH EARTHQUAKE RESISTANCE, AESTHETIC AND ECONOMIC ADVANTAGES FOR THE PREFABRICATED BRIDGES

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Abstract. *Two cases are examined: The first one examines the possibility of attaining a monolithic connection between deck and piers, in order to avoid using elastomeric bearings and obtain a continuous frame ductile system. The second one examines an innovative prefabrication proposal which, instead of using conventional simply supported precast prestressed beams, uses bilateral precast prestressed cantilevers, leaving gaps of order of 10m at the spans between the cantilevers, which are filled with cast in situ concrete by using suspended formwork in a next construction phase. In this second case the wall-type abutments are connected monolithically with the ends of the deck and are utilized as seismic stoppers. The experimental survey included five test specimens, through which the performance of the monolithic connections of the deck to the piers at the support positions is examined. The two proposals were applied as alternative solutions for the superstructure of real bridges. The first one was applied on a bridge of the PATHE motorway which has 5 spans and a total length of 177.5m, while the second one was applied as alternative solution on a bridge of a motorway in Crete with a length of 400m, which was designed according to the cantilever construction method.*

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1 INTRODUCTION

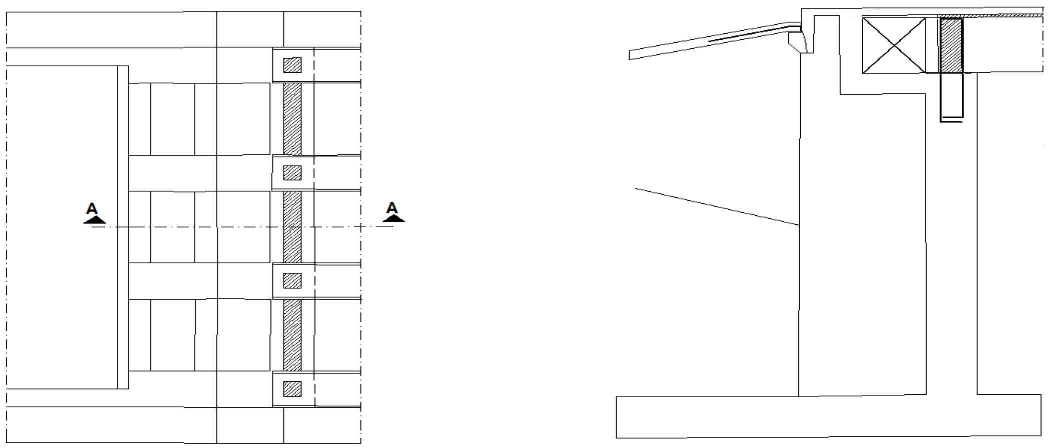
The utilization of prefabrication in the construction of bridges lies in the construction of prefabricated beams, which are used as simply supported members seated at the ends of every span, the length of which typically ranges from 30 to 40m. The construction process is known: First the construction of the piers is completed, then the bearings are placed on the pier heads, afterwards the simply supported beams are placed by means of two cranes or a gantry crane (carroponte) and finally the construction of the deck is completed with the in situ concreting of the deck slab, through which the forming of a single static system is accomplished. Multi-span simply supported bridges with a superstructure consisting of precast prestressed concrete beams or steel beams (composite deck) have flourished since 1950. During the 60s and 70s the needs for rapid expansion of the national road networks in combination with the requirements for rapid construction, led to the great increase in the number of these bridges internationally.

The above described system apart from its advantages has also drawbacks, which are related to the basic requirements, the fulfillment of which is associated with the respective compliance criteria, which in this case are economy, aesthetics, earthquake resistance and durability [1].

The bearings and the expansion joints are two types of accessories, that are absolutely necessary for the serviceability of the structural system of a prefabricated bridge, burden significantly on the one hand the construction cost and on the other the maintenance cost of these projects, having as a result that the second one can sometimes reach the level of construction cost [2]. Furthermore, the non-economicity comes from the forced recourse to the inevitably, due to the construction way, series of simply supported spans and the resignation from attaining an integral system, through which a significant relief in moments at the spans would be achieved through the utilization of the supports. The impacts, besides economy, extend to the aesthetics, as the height of the beams cross sections could be reduced due to the desired continuity of the system. However the aesthetics is extensively negatively affected by two other necessary concessions that are related to the construction process that is applied until today. The first one lies in the fact that the prefabricated beams are seated on the pier heads and not embedded over their entire height in them. The second one lies in the necessary selection of an unattractive large width for the pier heads in the order of 3m, in order to attain a safe seating for the simply supported beams, that are situated on both sides of the pier head, and leave room for the visit during the periodic inspections of critical positions in the project life time.

Regarding earthquake resistance, although it is possible to attain a seismic isolated system with the current practice, this is accomplished with the consumption of huge volume of bearings [3], which actually, due to the inevitable premature aging, should be replaced several times during the project life, a fact that harms the durability of the system and increases the long-term cost of the bridge. Regarding prefabricated bridges that have earthquake resistance requirements [4,5], the single deck slab gave to the static system the advantage of the single disc, which through connecting all spans, enabled management of the bridge superstructure as a single mass and made the problem of earthquake oscillation of the system statically defined. Consequently the use of elastomeric bearings as a means of seismic insolation was made feasible due to their ability to increase the eigenperiod of the system to any desired value, so as to obtain a drastic reduction of inertial seismic actions for the benefit of the piers. It is noted that due to the seismic isolation the value of behaviour factor is taken $q = 1$, which implies a more conservative design for the webs and the foundations of the piers.

Apart from the seismic isolated system, which is covered by special regulations [6,7], there is the ductile system, resulting for prefabricated bridges by using appropriate stoppers, placed on the pier heads (cap beams) of the piers, which serve the functional needs of the superstructure, and are activated during the seismic excitation of the system [3]. The resulting ductile system enables the drastic reduction of inertial seismic actions and also allows a drastic reduction in the volume of elastomeric bearings, whose thickness in this case is determined by serviceability criteria. In another ductile system, characterized as similar to the above described system, if suitable conditions exist (adjacent tall piers), the superstructure can be connected to two or more consecutive piers through hinge (the superstructure is wedged in stoppers) so the longitudinal inertial earthquake actions are undertaken by these piers, while at the rest of the piers the superstructure is supported on sliding bearings [8]. It is noted that there is a quasi-asymmetry regarding the functional requirements between longitudinal and transverse direction of the system, since the longitudinal direction is threatened by in-service constraint movements and therefore there is a demand for difficult constructional measures to prevent them, in contrast to the transverse which is untouched. Also there is a problem of compromising functional and seismic requirements in the longitudinal direction, which are normally conflicting, namely everything that favors the one hurts the other. In the current bibliography, one can read that the elimination of end joints is not currently feasible and certainly, while it solves ideally the seismic problem of the difficult longitudinal direction, however loads the functional problem of the same direction with onerous requirements, which are hardly manageable. One form of an immovable abutment is depicted in Figure 1 [9].



(a) (b)

Figure 1: Form of an immovable abutment with two internal struts: (a) Plan view (b) Section A-A.

However, despite these drawbacks the prefabricated bridges have two important advantages. The first one is that they enable the construction of superstructures at a great height above the ground and they give solutions when the construction of scaffold is impossible or unprofitable. The height of 10m is considered practically as such a limit. The second one is that they enable the rapid construction of the deck compared with the cast in situ construction. Therefore, it was natural for the research of the last decades to seek to eliminate the disadvantages and benefit from the advantages of these bridges. The efforts which have sought and achieved in the first phase the continuity of the deck at its supports are mentioned as main improvements [10,11,12]. In this way it is possible to achieve savings of about the half volume of the bearings [13,14,15] and significantly reduce the moments at spans and thus the required prestressing of precast beams, thanks to the activation of supports for a part of the loads (pavement, sidewalks, live loads). Regarding the earthquake resistance problem the resulting system continues to operate as a floating deck on bearings and thus has to be treated with the rules of seismic isolation.

Subsequent efforts of research seeks the monolithic connection of the deck to the piers. In this way it is achieved both the elimination of the bearings, except from the end seating positions of the deck on the abutments, and the release from the need to implement behavior factor q = 1 as the resulting system can function as a ductile one [16]. However the resulting benefits in economy, durability, aesthetics and earthquake resistance are accompanied by the introduction of a difficult problem regarding functionality. More specifically the attained monolithic connection affects dramatically the functionality as piers obstruct the contraction and expansion of the deck and are subject to functional stresses due to intense restraints. Thus the need arises for the removal or mitigation of the effects of monolithic connection and this can be treated with the use of flexible (of decreased stiffness) piers [17,18]. Also the monolithic connection of the resulting system is negatively affected at the seating positions of simply supported precast beams, as for the exerted loads on deck after the attainment of monolithic connection, the hindered by monolithic connection rotations of the beams in their seating positions should be examined experimentally regarding the occurrence of undesirable effects in the resulting continuous span.

The attainment of a real monolithic connection, and not a quasi-monolithic connection that is provided by the continuous deck slab, has been a target for many years and various proposals have been made for its realization [19]. The expected advantages can be summarized as follows: a) The additional permanent and live loads of the superstructure are carried by the continuous system in a more economical way since the supports are also activated to bear these loads and b) Elimination of the two rows of support bearings in the positions of the piers and their limitation to a single row, namely saving 50% of them [20]. Certainly the acquired static system is characterized by a series of drawbacks, such as the ''submersing'' continuity tendons (Figure 2), the impossibility of mutating the system into a ductile one and others.

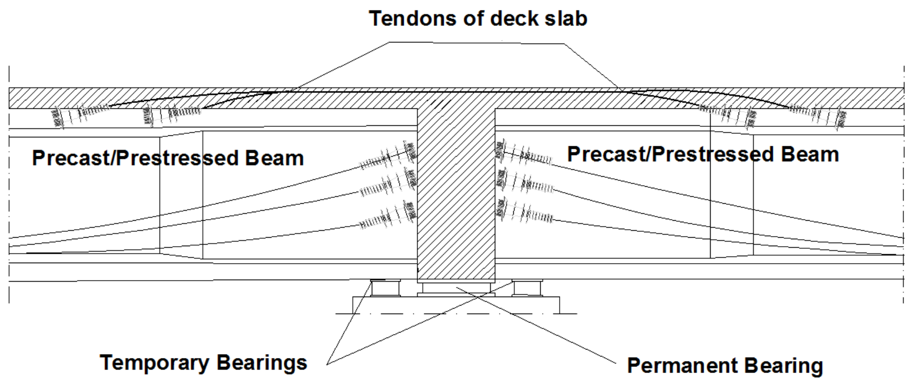


Figure 2: An indicative proposal to connect prefabricated elements in order to restore deck continuity in the positions of the piers [21].

Ensuring a monolithic connection of the deck to the piers is a challenge that is worth pursuing since it enlarges by far the advantages of a simply continuous deck. Mentioning from many only a few, the elimination of all bearings in the positions of the piers and the mutation of the system into a ductile one. Certainly the acquisition of these two main advantages that favor the earthquake resistance, the economy and the durability of the bridge, implies dealing with the functional problems that require certain measures, such as reducing the stiffness of the piers and in combination with taking measures to ensure the regularity of the system against earthquake. It is noted that the potential to improve the regularity is an important advantage, since this may result in a significant reduction of inertial seismic actions and provide a more economical solution for the piers and their foundations which are the structural parts that are designed, based on the critical seismic loads combination.

Two proposals of scalable monolithic connection at bridges, where the prefabrication is exploited, are examined in this paper. In the first case that is examined in the present paper, one proposal ensuring monolithic connection of the deck to the piers is examined experimentally and analytically, avoiding even the handicap of submersing tendons, since the support positions of the mutated seismic system are conventionally reinforced. Measures to meet emerging functional problems in the longitudinal direction of the bridge are also proposed, so that the proposal for the monolithic connection is made feasible in applications. Besides, the possibility of gaining additionally an aesthetic advantage was examined, since it is known that the type of bridges under consideration suffers from an aesthetic point of view [22].

In the second case that is examined in the present paper, a new type of management of the problem of deck shaping using precast beams is proposed. The result is closer to the desired monolithic connection and the proposed solution could be described as hybrid as the use of the cast in situ deck section is extended and is not limited to the construction of the deck slab, but further extends to the beams of the central regions of spans. Significant advantages are also achieved concerning the aesthetics and the economy and the resulting monolithic connection is clearly improved because now concerns all loads and has eliminated most disadvantages mentioned above, which are related to the attainment of monolithic connection in the positions above the piers. It is noted that the proposed utilization of prefabrication can substantially increase the length of the spans, which with the existing practice reaches up to 40m.

2 DESCRIPTION OF THE FIRST PROPOSAL

The variations involved in the proposed construction process, which results in a monolithic connection, are significant. In this case the prefabricated beams are not seated on the pier heads of the piers, but embedded in them, since they are placed to their final position at the phase when the pier heads are concreted only at the lower flange and for a thickness (height) of approximately 15-20cm, where the bottom reinforcement and the bottom part of the stirrups are incorporated, which are expected to serve as suspension reinforcement. As it is obvious, in order for the precast beams to be seated on the pier heads-cantilevers, scaffold of heavy type must be erected for the temporary support of these cantilevers, that is feasible thanks to the foundation pile cap of the pier concerned (Figure 3). For the transverse support of the necessary metal scaffold, the web of the pier is utilized. The seating depth of each prefabricated beam is around 1m. This involves the selection of the pier head width greater than 2m. The gap that is left between the fronts of the respective beams is selected in the range of 20-30cm. After the beams placement, follows as before the placement of the precast deck slabs, and then the upper reinforcement of the deck slab, of the pier head and of the deck supporting reinforcement, for which conventional reinforcement and not prestressed is selected, are placed. The embedding concreting follows as before, which is advisable to begin from the middle of every span and be directed towards the corresponding piers, whose pier heads should be concreted last, so that the necessary rotations at the seating points of the prefabricated beams have taken place before the concreting of the pier heads.

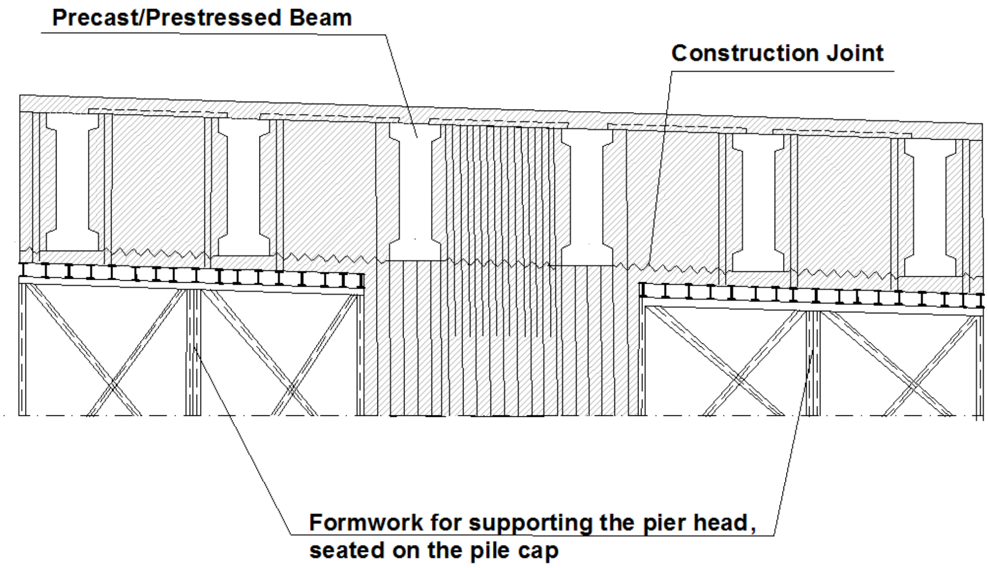


Figure 3: Metallic formwork for the support of the pier head during concreting.

It is noted that any objections: a) about the mechanical response of the system of respective beams which after the in situ concreting will be required to operate as continuous, will be treated according to Regina Probationum (Queen of Evidence), namely the experiment and b) about the possibility in a continuous system of beams that the reinforcement at the spans is prestressed, while the corresponding reinforcement at the supports is conventional, it is emphasized that this is feasible and does not create any regulatory problem. It is also recalled that the self weight of the superstructure is around 60% of all gravity loads and therefore the formed continuous system needs to carry only the remaining percentage of these loads, namely the largest percentage of loads is carried by a series of simply supported systems, namely by the prefabricated beams.

It is recommended for the proposed construction method that the length of the end spans should be 80% -90% of the length of the intermediate spans, as the beams of the end spans are seated at the abutments on elastomeric bearings and thus the benefit from the reduction of the moments due to the continuity of the system is smaller for these spans than for the intermediate spans where the beams are connected at both ends monolithically to the piers.

3 EXPERIMENTAL RESEARCH

It is obvious that the critical area of the mechanical behavior of which, the acceptance of this proposal for a monolithical system depends on, is the area where the connection of the beams of both sides takes place at the piers heads. Therefore and because each analytical estimation and evidence may not be convincing, it is imperative to conduct experimental research in order to verify the mechanical response of this area under conditions of bending stress. The question arises if the connection of the respective beams that is attempted within the pier head of the piers, behaves as originally monolithic or not.

The experimental survey included five test specimens. More specifically, the experimental program included two beams test specimens originally monolithic and three test specimens which were constructed each one from two sections of semi-beams that were joined in retrospect, under conditions that represent the construction process proposed in the frame of this paper. The examined test specimens represent the static subset of the actual deck between the symmetrical, as to support, zero-points of moments of the proposed continuous system, located on both sides of the support on the pier. It is noted that in the support position, where the support reaction of the pier exists, there is a maximum support moment. This static subset located, as it was said, between the zero-points of moments, may be modeled as a simply supported system loaded with a single force in the middle, which represents the reaction exerted on the pier.

The five specimens of the experimental research are divided into two groups. The first group included one beam test specimen originally monolithic (single beam) and two test specimens which were constructed each one from two sections of semi-beams, that were joined in retrospect (composite beams). The second group included one beam test specimen originally monolithic (single beam) and one test specimen constructed from two sections of semi-beams that were joined in retrospect (composite beam). The specimens of the first group had a cross section with a height of 40cm, while the specimens of the second group had a height of 60cm, 50% greater than the first. The reason we examined the problem in two versions, was to determine the possible influence of geometrical data of the problem on the mechanical behavior in the connection area of the respective beams. The width of the cross sections of the examined beams was constant and equal to 20cm. For the first group, the range of the joint left between the two half-beams was examined as a parameter, which in one case was 10cm and in the other case was 1cm. It is worth noting that there was a joint of 10cm in both groups, to allow comparison between the results of the two groups. The reinforcement of the beams is depicted in Figure 4 and the loading setup of specimens is depicted in Figure 5. The concrete category of the beams was C30/37 (B35). The specimens that were constructed from two semi-beams and subsequently connected include in their central area as well a section of expanded width which represents the effective section of the pier head of the pier, on which the respective beams are seated (Figure 6). This choice was made in order to examine the problem more realistically. The length of the test specimens of the first group was equal to 3m and the corresponding static span equal to 2.7m, while the length of the test specimens of the second group was equal to 4m and the corresponding static span equal to 3m.

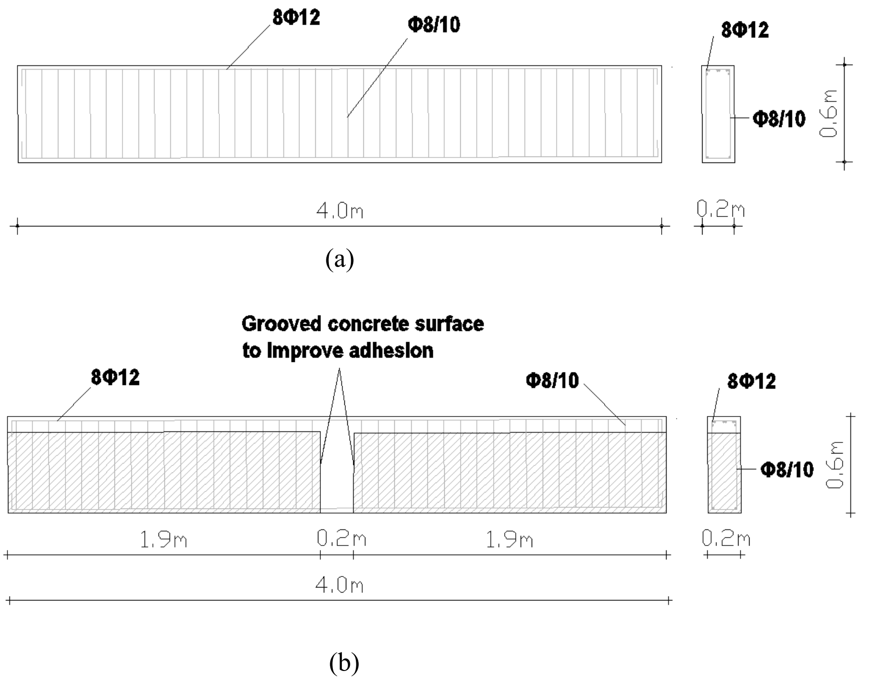
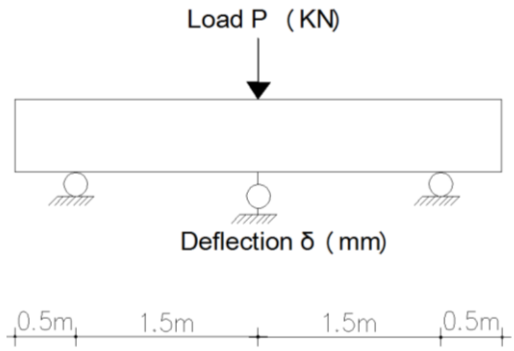


Figure 4: (a) Single beam reinforcement of the second group of specimens. (b) Composite beam reinforcement of the second group of specimens.

(a)



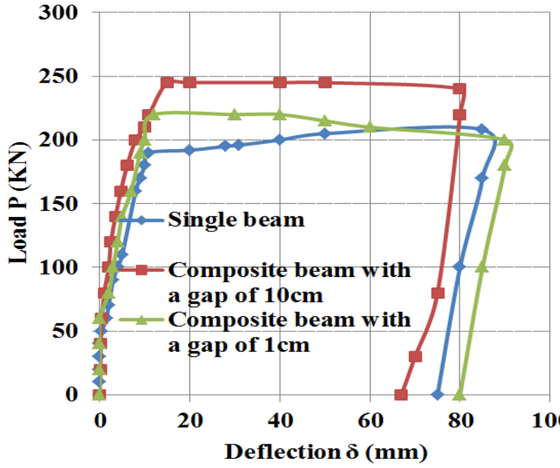
(b)

Figure 5: (a) Loading setup of specimens of the first group. (b) Loading setup of specimens of the second group.

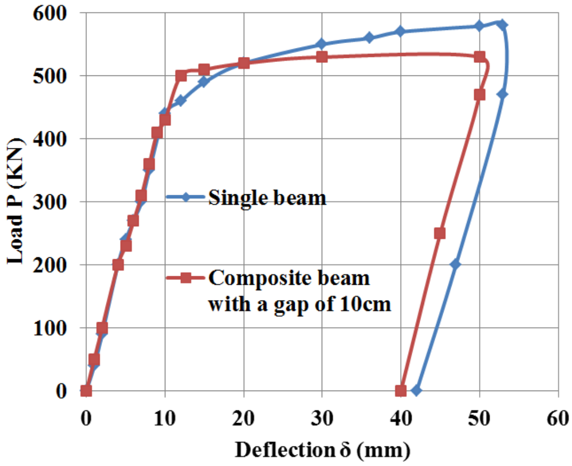
Figure 6: Composite beam with a section of expanded width.

The concreting of the two single test-beams took place at once, both lengthwise and heightwise of the section, while the concreting of the rest three was made in two phases: in the first phase the respective pairs of semi-beams were not concreted throughout their height, but 10cm of the cross section height at the upper flange were left without being concreted, so that there is a complete correspondence with the actual concreting situation of the bridge deck. The stirrups, the geometry of which is adapted to the full height of the cross sections, extend in this gap of concreting. After the hardening of the concrete of the semi-beams, they are placed in the corresponding formwork so that there is, in the middle of it, the gap of 10cm or 1cm mentioned above. The placement of bending reinforcement follows, which is continuous for the two semi-beams and same as this of the respective single test beams, which were concreted at once. Then the remaining height of 10cm and the gap of 10cm or 1cm between the semi-beams are concreted, the lower section of which gap, which is related to the bending compression zone of the single beam, is filled with non-shrinking cement mortar of superior quality.

After the maturity of concrete, the specimens were subjected to a single load applied into the midst of the beams. Simultaneously with the increasing monotonic loading, the deflections were measured in the middle of the beams, so that the corresponding load-deformation diagrams are drawn (Figure 7). At the same time the images of cracking of the specimens due to bending were recorded at different stages of loading (Figure 8). It must be noted that strong reinforcement with stirrups versus shear was placed so that there are no symptoms of shear failure (capacity design).

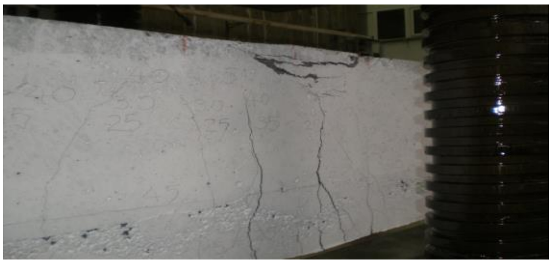


(a)



(b)

Figure7: (a) Load-deflection diagram for the first group of specimens. (b) Load-deflection diagram for the second group of specimens.


(a)


(b)

Figure 8: (a) Compression zone fracture of the single beam of the first group of specimens. (b) Compression zone fracture of the composite beam of the first group of specimens.

From the resultant load-deformation curves both for the single specimens concreted at once and secondly for the respective specimens concreted in phases, it can be clearly concluded that the mechanical behavior is the same and therefore the proposal can be considered as reliable, since the gradual attainment of a monolithic connection in the positions where the prefabricated beams are seated presents no side effects and has the same behavior as if the monolithic property came from a cast in situ case, without using prefabricated parts [23].

4 APPLICATION OF THE FIRST PROPOSAL IN A STUDY VARIATION OF A PREFABRICATED BRIDGE

The “reference” bridge, with the technical code T8, extends from CH.17+375.22 to CH. 17+552.78 of the right branch in the subsection Ag. Marina–Raches of the PATHE Motorway (Figure 9).

Figure 9: Project location (Agia Marina – Raches).

4.1 Description of the “Reference” Bridge

According to the Design of Road Works, the axis of the right branch of the motorway is in plan view on a circular arc with a radius R=3,000m at the bridge location, while the road red line is at an upper height of 20m above the natural ground, meeting with a small skew angle a valley on the northern outskirts of Stylida town plan. The superstructure is formed by pre-stressed precast concrete beams. The end spans are 34.75m long, while the three intermediate spans are 36m long. The bridge has 5 spans and has a total length of 177.5m (Figure 10).

The superstructure is formed by 6 T-beams, with a height of 2m, at axial distances of 2.5m and a concrete deck slab cast in situ over the precast beams top flanges of 25cm in thickness, having a total height of 2.25m and a total width of 14.2m (Figure 11a). The deck slab is composed by precast slabs 10cm thick, supported at the top flange of the beams, and a cast in situ slab 15cm thick, having a total thickness of 25cm and forming together with the beams, the final deck cross section. The weight of the prefabricated beams is 72.6tn, their tendons are prestressed in a prestressing bed in direct bond with concrete and in a single phase. The prestressing tendons are composed by 40 and 46, for the end and the intermediate beams respectively, straight wires with a nominal diameter of 0.6”, namely having each one a cross section of 139mm^2.

The slight curvature in plan view of the deck is formed by the cantilevers of the cast in situ sidewalks beyond the end beam, 42-47cm wide towards one side and 48-53cm towards the other side. The concreting of that area of the sidewalks, despite its limited width, presupposes the use of, resistance to bending, welded reinforcement cages in a lattice form (Gittertrager or trallici) at adjacent prefabricated slabs, so that this section is concreted supported at these cages, also presupposing the use of prefabricated parts for external surfaces.

The pier has the form of one-column pier of hollow circular cross section, having an exterior diameter of 3m and an interior diameter of 2m, namely is 0.5m thick (Figure 11b). The column heads are connected with a pier head, shaped anvil, 3.5m wide, leaving a gap of 1m between the fronts of the respective beams for the case that a carroponte mechanism is used for the placement of the beams.

The simply supported superstructures are connected over the piers with a connection slab (connection flange) in order to eliminate the intermediate joints, namely their procurement, installation and maintenance costs in combination with the improvement in comfort feeling during the traffic of vehicles. Elastomeric expansion joints of type ALGAFLEX T200 AS are installed only at the abutments, while the ability to visit the interior of the pier columns is ensured through a manhole with an opening 0.80 x 0.80m at the upper surface of the pier head. The precast beams are supported at the piers on circular elastomeric bearings of type

ALGABLOC NBC4, while similar bearings are installed between the deck and the earthquake resistant stoppers.

The concrete category of the deck and the piers is C30/37 (B35), the category of steel reinforcement is S500s and the category of the prestressing wires is 1570/1770.

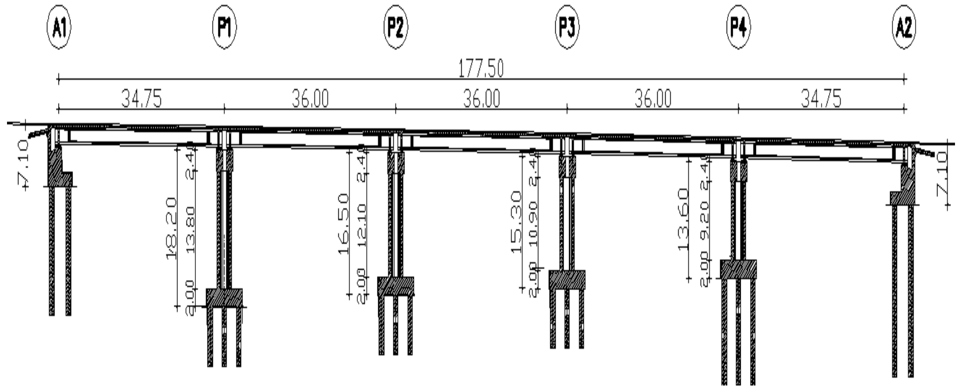
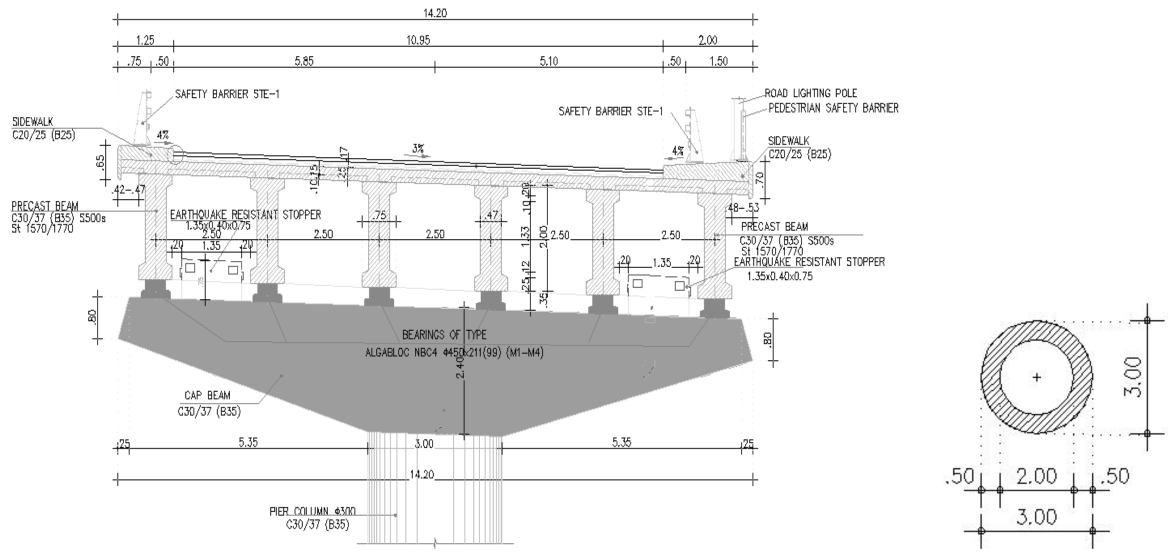


Figure 10: Longitudinal section of the bridge.



(a) (b)

Figure 11: (a) Bridge typical support section. (b) Pier cross section.

4.2 Redesigned bridge

For the redesign of the existing bridge, the proposed construction method keeps, from the conventional construction method, the prefabricated beams and slabs, but differs radically from it at the piers, where the prefabricated beams, instead of seating on elastomeric bearings, are embedded in the pier heads. The aim of this embedment is to accomplish the continuity of the static system, at least for a substantial proportion of the functional loads (additional permanent and live loads), and improve the seismic behavior of piers. According to this proposal, the deck is not monolithically connected to the abutments, but is seated on proper bearings and the seismic displacements are selectively controlled through seismic links (stoppers). The two intermediate beams, out of 6 beams of the bridge cross section, are seated directly on the piers, without needing support during their construction.

The proposed construction method is as follows:

- The precast/prestressed beams and the precast slabs are constructed on the ground.
- The piers whose stiffness must be drastically reduced, can not obviously maintain the hollow circular cross section of the piers of the reference bridge. Therefore they should have a hollow rectangular cross section, that is particularly suitable for ''play-ing'' with the stiffness (Figure 12):
  a) If the pier height is big (e.g. greater than 20m), then the hollow cross section can be maintained complete.
  b) If the pier height is medium (from 15-20m), then the hollow section is seperated into two blades and two small walls.
  c) If the pier height is small (smaller than 15m), then the two small transverse walls can be divided in half. In this case where the pier heights are about 15m, the second (b) choice has been made. The piers are constructed at first stage with cast in situ concrete and have adequate width for the seating of the prefabricated beams. The top section of the pier is constructed as a single section together with a part of the pier head.
- The precast I-beams are placed in their final position by crane and are seated on the bottom part of the pier head (about 20cm above the bottom formwork).
- The in situ concreting of the pier head follows, embedding parallely the prefabricated beams, in order to accomplish a fully monolithic connection of the precast beams of the deck to the piers. The concreting can reach up to the deck slab.
- Subsequently, the precast slabs are placed on the precast beams and the compression reinforcement of both the cast in situ deck slab and the beams is placed.

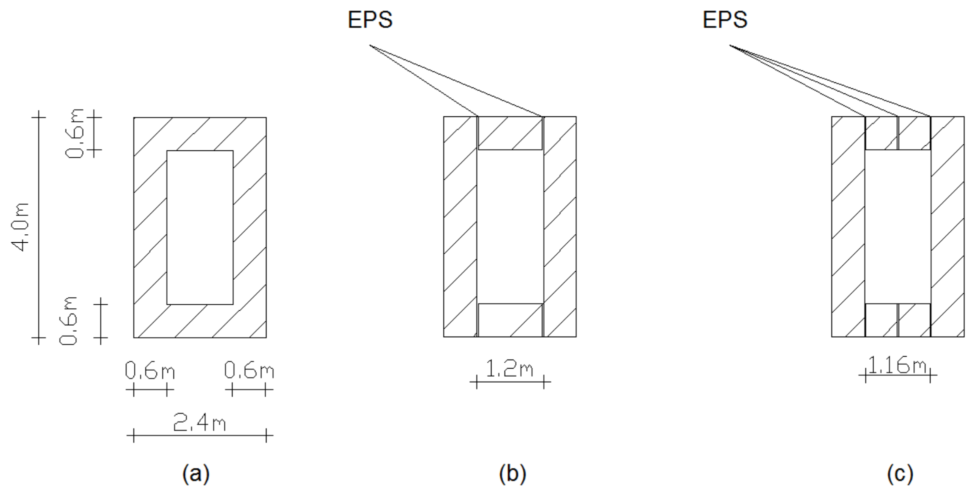


Figure 12: Cross section of the pier (a) for height greater than 20m (b) for height between 15 and 20m (c) for height smaller than 15m.

The main features of this method are the large width of the pier head and the need for using metallic formwork, that is based on the pier foundation, for the temporary support of the beams. The rigid connection of the precast beams to the pier head is ensured by placing suspension reinforcement (stirrups) on each side of the beams.

Of course, there is a need for proper reinforcement of the monolithic node, in order to avoid congestion of reinforcement. As it becomes evident from the following drawing (Figure 13a), conventional reinforcement and not prestressed is chosen at the supports. The section of the pier head is given in Figure 13b).

The program SAP 2000 v 17.1.1 [24] was used for the analysis.

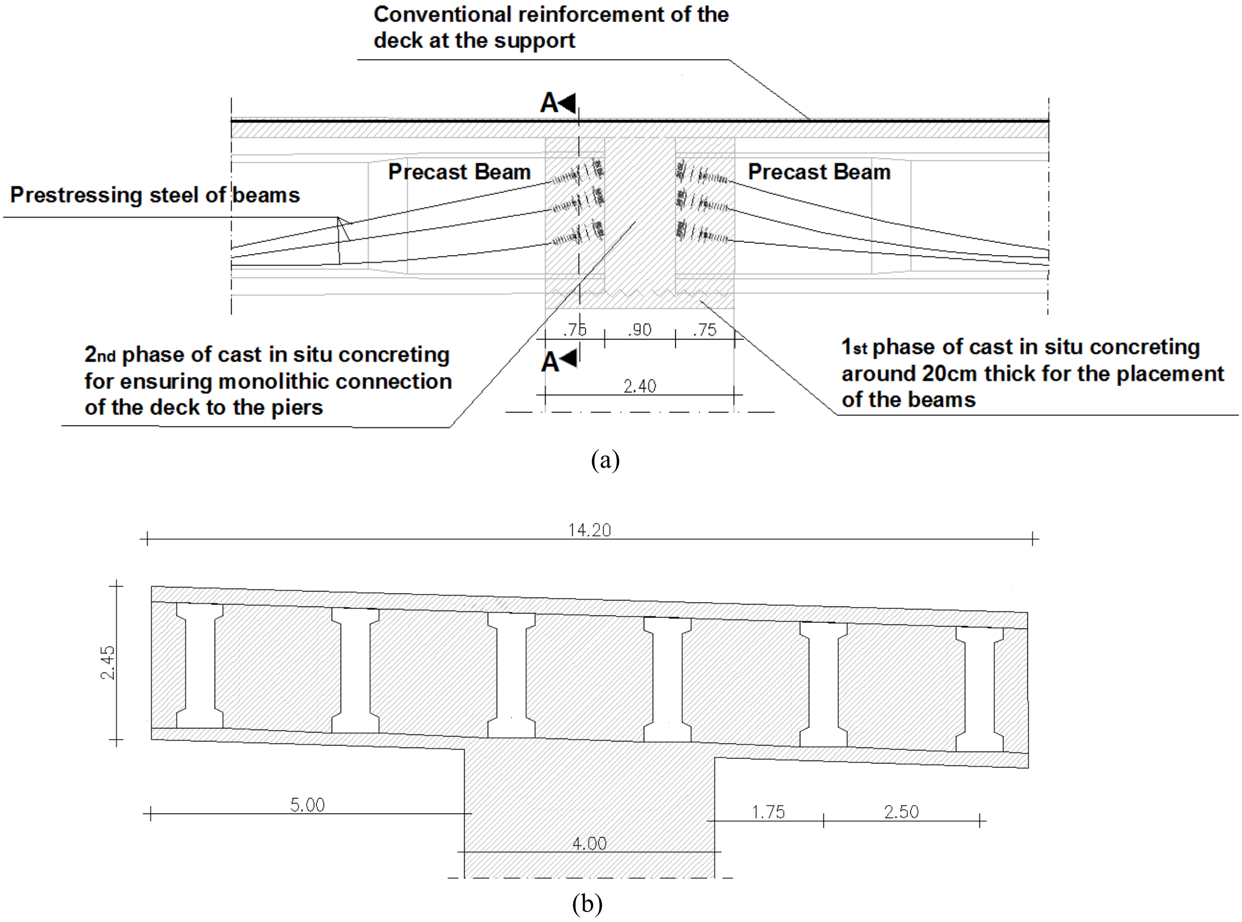


Figure 13: (a) Longitudinal section at the pier. (b) Cross section A-A at the pier.

4.3 Dimensioning of the pier head

The pier head is the structural element that transfers the loads of the superstructure to the piers. It has a compact rectangular cross section, a constant width of 2.4m and a height of 2.45m (together with the deck slab). In essence, there are two cantilevers on either side of the pier, that end up to it.

The process of determining the internal forces is as follows: the loading of the beam due to its self weight and furthermore the loads that carries one cantilever (due to symmetry) in the positions of P_1 and P_2 beams (concentrated load), that result from the analysis of the superstructure for a span, are taken into account. The concentrated loads are obviously multiplied by two, since two spans correspond to each pier head [25].

Based on what has been said above and loading one-sidedly the deck of the bridge, (putting the vehicle in the nearest position to the sidewalk), in order to have the most unfavorable situation for section loads, the corresponding loads, due to concurrent beams, are calculated. The most unfavorable – one-sided loading of the deck due to traffic loads is given in Figure 14 [26].

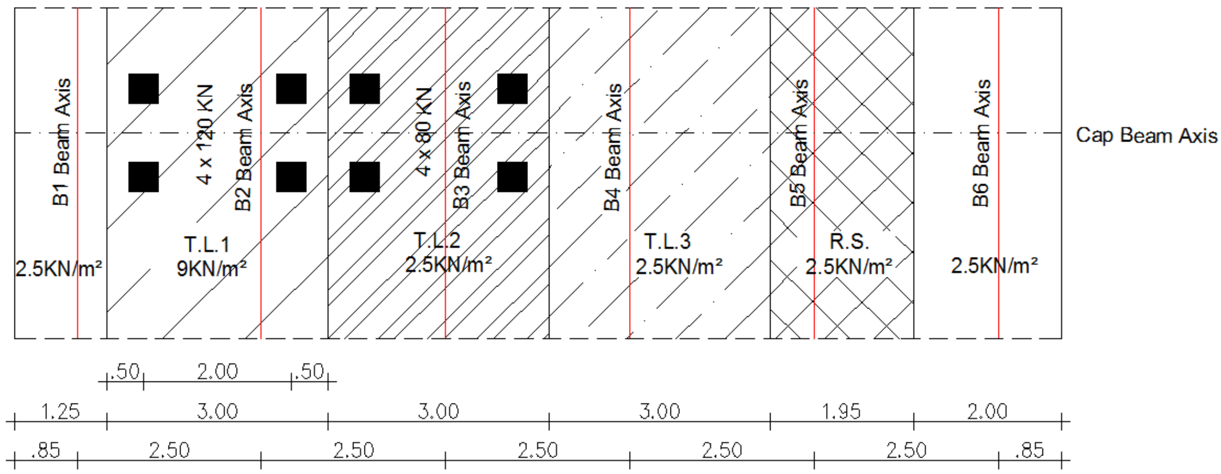


Figure 14: Characteristic position of the most unfavorable loading.

The load from the self weight of the pier head and the single loads from the beams are given in Figure 15.

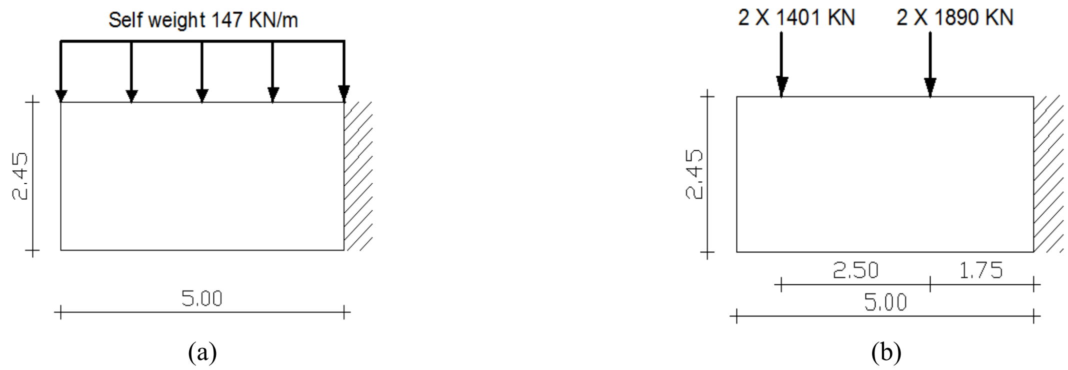


Figure 15: (a) Load from the self weight of the pier head. (b) Single loads from the beams.

The design moment is: M_{sd} = 21,010kNm for the ultimate limit state in bending.
The required reinforcement is: $A_{s,req}$= 230 cm^2.
Consequently the bending reinforcement at the pier head is 48Φ25 (2X24 Φ25/100 - $A_{s,prov}$ = 235.5 cm^2).

Simultaneously 32Φ25 (2X16 Φ25/150) are placed at the bottom flange not only as montage reinforcement, but also as reinforcement for the containment of creeping deformations of the cantilevers, since as it is known, the existence of strong compression reinforcement reduces significantly the influence of creep.

The precast beams are seated on the pier head creating an additional strain. This additional weight is imposed in the bottom flange of the pier head and is accompanied by the risk of yielding reinforcement of the bottom flange and destruction of the existing reinforcement cover. For these reasons suspension reinforcement (stirrups) must be placed at the seating positions of the precast beams, in order to ‘’transfer’’ this strain towards the upper sidewall of the beam and avoid the risks.

Then the required suspension reinforcement is calculated:

- At the seating position of beam 1: $a_{sw,req}$ = 64.4 cm^2.
- At the seating position of beam 2: $a_{sw,req}$ = 86.9 cm^2.

Finally stirrups Φ14/150 of 16 legs are placed as suspension reinforcement per cross section in two cross sections left and two right with a distance of the two positions between them 15cm → $a_{sw,prov} = 98.6\ cm^2$.

The shear check position is at a distance d from the support for the ultimate limit state in shear. The design shear at the specific point is: $V_{Ed} = 3{,}329$ kN

A) Check of the compressive diagonal:

$V_{Rd\ max} = 27{,}000kN > 3{,}329\ kN = V_{Ed}$

So it has been proved that the compressive diagonal has sufficient strength.

B) Shear reinforcements:

The required shear reinforcement is calculated: $a_{sw,req} = 29.2\ cm^2/m$. Stirrups Φ14/300 of 16 legs are placed as shear reinforcement in all areas → $a_{sw,prov} = 98.6cm^2/\ m$.

The chosen reinforcement of the pier head is depicted in Fig. 16, 17.

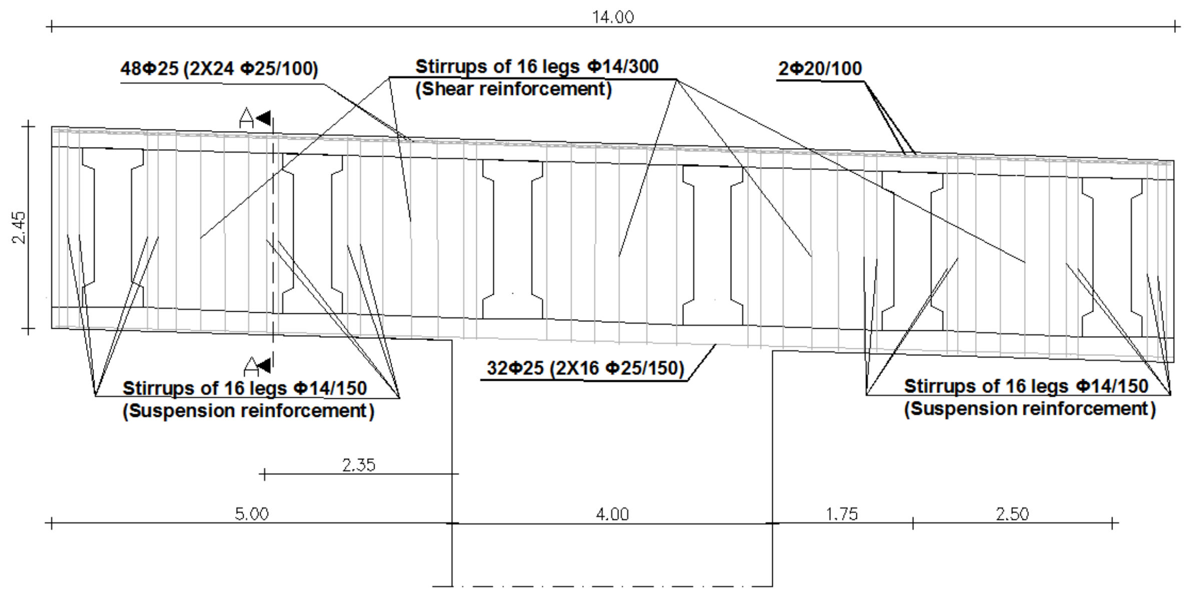
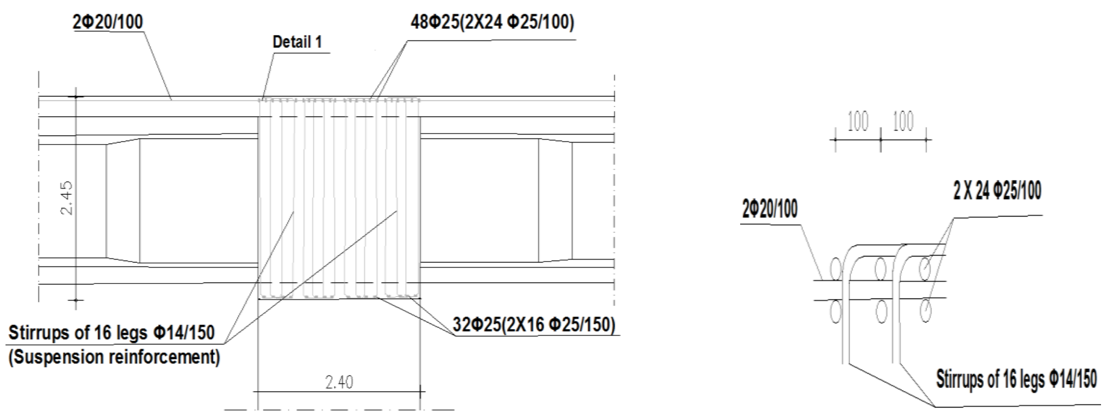


Figure 16: Pier head cross section.



(a) (b)

Figure 17: (a) Deck longitudinal section A-A. (b) Reinforcement detail at the upper flange of the pier head.

Regarding the reinforcement needs of the pier head, it is worth mentioning that in case of difficulty to meet the functional criterion relating to the allowable range of cracking, there are alternatives to improve the situation, either by reducing the cantilevers length through increasing the cross section length of the underlying pier, or even resorting to the solution of prestressing, that will lead to the decongestion of the situation. It is noted that any applied pretensioning does not have the disadvantages of ''submersing'' tendons since the anchorages are constructed ''normally'' at the fronts of the cantilevers ends.

5 CONCLUSIONS FROM THE FIRST PROPOSAL

In this paper a variation in utilizing prefabricated beams was examined, both experimentally as well as analytically, rather than seated on bearings at the pier heads, but as embedded at their end sections in the pier heads and simultaneously eliminating the bearings, which are placed only at the end supports of the bridge on the abutments. The functional problems, particularly that of the shrinkage of the deck, were dealt with by using rectangular hollow sections for the piers, seperated into two blades and two small walls, in order to drastically increase their flexural flexibility, that is absolutely required because of the great length of this kind of bridges and the lack of bearings on piers.

In this paper a PATHE bridge was utilized and was redesigned, based on the above interventions in its load bearing structure, with the main objective to examine the feasibility or not of their integration in its structural system. The main conclusions of this examination are the following:

1. The durability of the resulting system is much greater due to the elimination, but also the removing of the need for periodical replacement of the much greater volume of bearings, which are used in the conventional method.
2. The economy is greater for the proposed system because of the abolishment of bearings at piers, the continuity of the system for a large proportion of the loads and thanks to a drastic reduction of the prestressing reinforcement.
3. The aesthetics is better due to the embedment of the beams into the pier heads and the potential reduction in the height of the beams of the superstructure.
4. The safety of the resulting system not only is not smaller than that of the conventional method, but because of the frame function of the bridge, results greater.
5. The functionality which, as it is known, is negatively affected by the monolithical connection, is manageable.

6 DESCRIPTION OF THE SECOND PROPOSAL

6.1 Construction of the bilateral cantilevers of the superstructure and placement of them on the pier heads

Under the proposed construction process, instead of simply supported precast beams, integral and balanced bilateral precast cantilevers with a cross section of variable height are constructed, which are going to be seated on the pier heads with a seating length equal to the width of the cross section of the pier heads (Figure 18). They have variable sections regarding both the width and height (Figure 19).

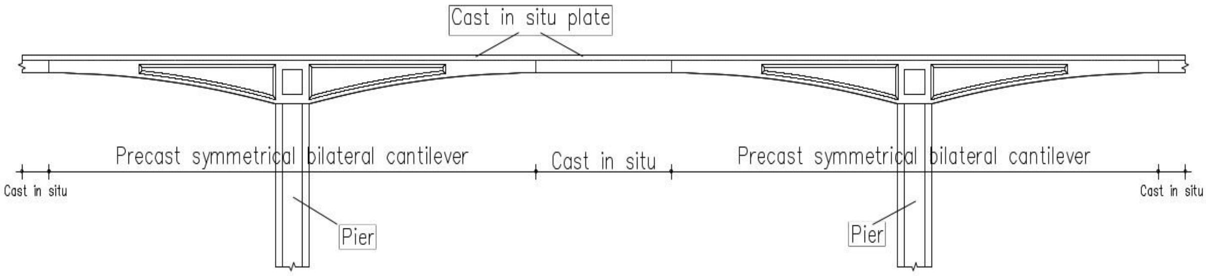


Figure 18: Longitudinal section of the bridge.

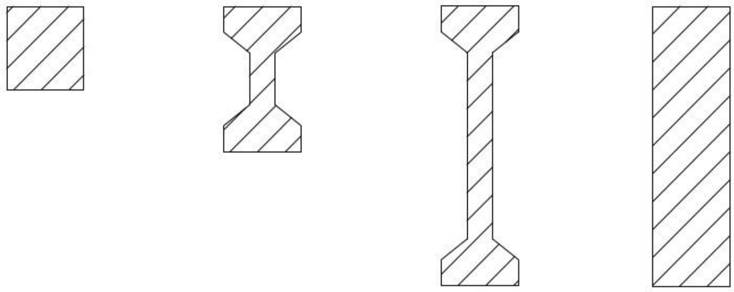


Figure 19: Typical sections of the precast bilateral cantilevers.

The number of these precast bilateral cantilevers (PBC) per span may follow the rules governing the conventional prefabrication. Namely, for a distance between each other ranging from 2.5 to 3m, there are 4 to 6 precast bilateral cantilevers. Regarding the width and the length of the bilateral cantilevers, respective selection criteria may be considered, for the width both the desired efficiency of the compression zone at the supports of the continuous system and the anchoring requirements of prestressing tendons, which are at the upper flange of these prefabricated members, and, for the length, with given the possibility of using sections of variable height and width, a considerable length of the cantilevers on both sides may be achieved, if the weight of 1,000KN, which is considered as the economic weight for lifting by two cranes, is taken as a criterion. Otherwise, a carroponte mechanism is used for their placement. E.g. for a width of 0.75m and for a height of 2.7m at the beginning of the cantilever, at the support position, and of 0.80m at the end of the cantilever, cantilever length of about 20m and a total length of the prefabricated part of the order of 40m can be achieved. This means that, if one considers that the central part of the span is of the order of 10m, there is a possibility of achieving spans with a length over 50m. This is of great importance, as it expands greatly the possibilities of prefabrication applications by exceeding the limit of 40m, that is regarded as a boundary for the length of the span in the conventional construction method. This is made feasible due to: 1. the double tee sections of the precast members (instead of the rectangular of the conventional prefabrication), 2. the variable sections (instead of the classic ones), 3. the monolithic result which creates moments at the supports (instead of the simply supported system).

After the placement of the precast members on the hollow web of the pier and the bilateral cantilevers-pier heads, follows the concreting of the deck slab which refers to the area of PBC, together with the completion of the concreting of the pier head, for which preceded, even before placing the PBC, part of its bottom flange, about 20cm thick, where the bottom compression reinforcement of its seating cantilevers and the bottom part of the stirrups, which undertake shear forces and suspend the reactions of the gravity loads transferred by PBC, are incorporated (Figure 20). Regarding the concreting process after the placement of PBC, it is noted that the placement of the necessary precast deck slabs 7 or 8cm thick, where the bottom reinforcement of the deck slab is incorporated, precedes, then follows, after obtaining a work floor, the placement of the upper reinforcement of this slab and the conventional longitudinal supporting reinforcement of the superstructure, which is calculated in a next phase in addition to prestressing tendons, and follows the concreting of the remaining thickness of the seating cantilevers, along with the deck slab up to the ends of the PBC. It should be noted that the placement of the precast deck slabs and the concreting should take place respecting strictly the symmetry of imposing loads on the sensitive against topple t-shaped static system, the from the ground temporary support of which is impracticable and costly because of the generally big height of the piers. The attainment of a monolithic behavior of the connection of the bilateral cantilevers is ensured: a) through the provision of a large central hole in the PBC in their supporting area, with which the continuity at the pier heads is achieved and through appropriate longitudinal reinforcement (with respect to the pier heads), it can be considered that the pier heads function as integral and b) thanks to the intermediate sections between the precast bilateral cantilevers, the integral deck slab and the suitable geometry of the sidewalls of the precast bilateral cantilevers, situated within the pier heads, which are easily formed during their concreting locally in their central region by means of corrugated metal sheets, whereby the relative rotation between the precast bilateral cantilevers and the effective parts of the hollow webs of the pier for the longitudinal direction of the system is prevented. Regarding the earthquake, since the effective cross-sectional width of the pier at the top is reduced because of the inactive flexural areas, which are created by the presence of the precast bilateral cantilevers, supplementary reinforcement must be placed bilaterally in the upper region of the hollow web, so that the flexural bearing capacities up and down are equal, and the zero-point of the moments is in the middle of the height, as the web of the pier behaves as restrained at both ends between the lower pile cap and the upper deck.

As regards the criterion for determining the exerted straight pretensioning at the top flange of the PBC, this is selected initially to satisfy the requirement of zero deflections of the cantilevers not only for the self weight of PBC, but also for the subsequently concreted deck slab in the position of the cantilevers. In other words, the prestressing moment should be equated with the moment of the gravity loads in this construction phase at the beginning of the cantilevers. It should however be noted that the resulting prestressing tendons may be increased in the dimensioning phase of the complete system and for its most unfavorable loading with permanent and live loads [27].

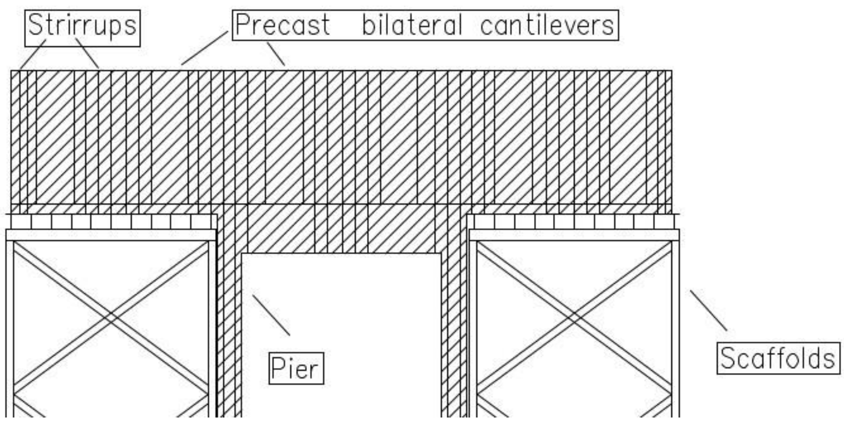


Figure 20: Construction of pier heads-cantilevers.

6.2 Construction of the abutments

The selection of the variable cross-section of the superstructure for economic reasons, in combination with corresponding aesthetic reasons, requires the construction of a new type of abutment, on which the end of the superstructure will not simply be seated but will be restrained in it, so that the intended variability of the cross-section may be extended at the end spans too. This is difficult and things become more difficult as the use of prefabrication also at the end spans of the bridge requires adjustments because conditions vary radically with respect to those of the piers. The main difference lies in the fact that the PBC can not be symmetric, as in the case of piers, therefore support of the cantilever that is on the side of the bridge would be required for their seating on the wall-type web of the abutment. This support is not considered easy to be constructed although generally in the areas of the abutments there is a small distance between the superstructure and the ground. So, three solutions for the construction of the abutments are proposed and examined, which have limitations, advantages and disadvantages.

Solution 1

If the conditions of the relief of the area permit it and the length of the end part of the superstructure is not big, the cast in situ construction of the system consisting of the abutment and the semi-cantilever of the deck is feasible, instead of the precast semi-cantilevers of the abutments. This solution is the easier one.

Solution 2

Precast non-symmetric semi-cantilevers are used, which are simply supported at cast in situ abutments. This solution does not abandon the prefabrication, but it sets two limitations: 1. The precast parts should be placed at abutments with a wall-type web. 2. A system of longitudinal bar (cable) is foreseen for each beam, at a significant distance from the wall-type abutment on the side of the embankment, so that the tilting moments of these non-symmetric parts can be undertaken. The lever arm should be about 3m.

Solution 3

This case mainly concerns bridges in areas of high seismicity. The wall-type web of the abutment, on which the end part of the superstructure is seated, can also be used as a seismic stopper. Measures to meet emerging functional problems should be taken, so that the proposal is made feasible in applications. There are two distinct cases depending on the height of the abutments. In case of abutments of great height, the wall-type web of the abutment is seperated into two blades, in order to decrease the stiffness of the abutments in the longitudinal direction. The distance of the blades must be adequate for the seating of the precast non-symmetric cantilevers. In case of short abutments, which is the usual case, the separation of the wall-type web of the abutment into two blades would not solve the functional problem. So, the wall-type web of the abutments is not embedded in the pile cap, but is seated on the pile cap on bearings which are placed not at the upper flange of the pile cap, but at the bottom of a grooving, with appropriate depth and width, so that the sides of the grooving function as bilateral stoppers in case of an earthquake and simultaneously allow through the existing gaps the functional movements of the superstructure.

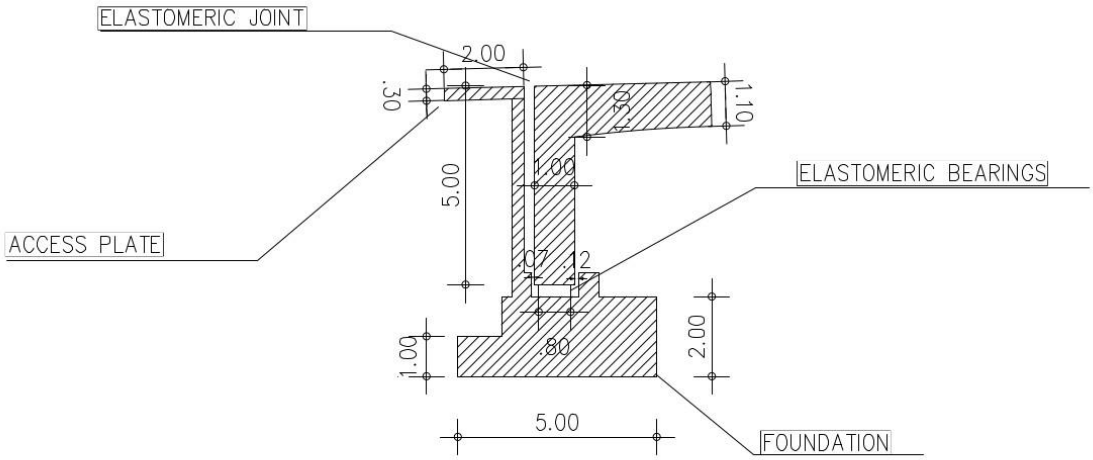
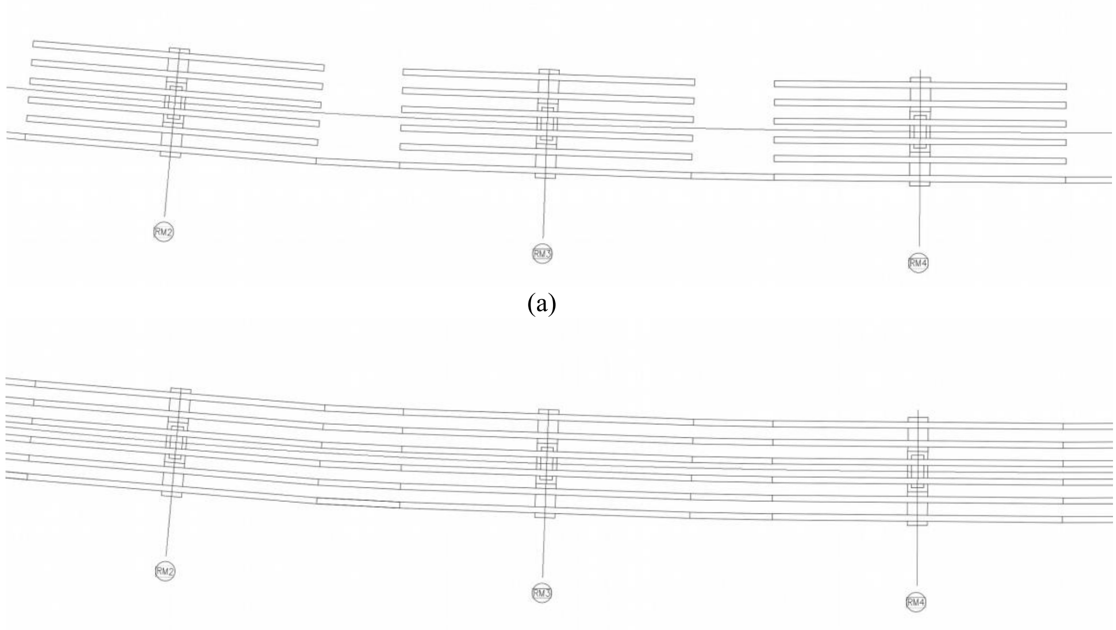


Figure 21: Wall-type web of the abutments which is seated on the pile cap on bearings, placed at the bottom of a grooving.

6.3 Construction of the central parts of the spans

The cast in situ central parts of each span are concreted with the use of suspended formworks. The formworks are suspended at the ends of the opposite cantilevers. This occurs simultaneously at every span but only along one 'line' of PBC (Figure 22a). The procedure must be very strict because of the sensitivity to non-symmetric loadings of the gradually constructed system. After the hardening of the concrete of the first 'line', the suspended formworks are removed and are placed at the next line. On this way the final system is completed (Figure 22b).



(a)

(b)

Figure 22: Consecutive construction of the central parts of the spans.

The formworks are dimensioned, for economy reasons, to undertake their self-weight and the weight of the concreted beam and not the weight of the deck slab, that will be concreted afterwards. The dimensions of a formwork, appropriate for a 10m long central span, are shown in figure 23.

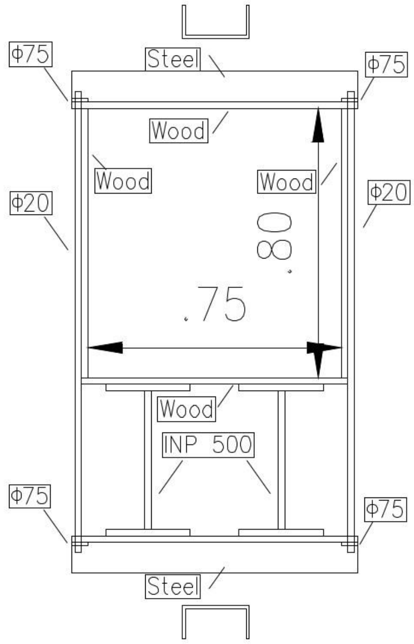


Figure 23: Typical suspended formwork.

6.4 Construction of the deck slab

After the concreting of the pier heads, in which the precast beams are embedded, and the construction of the central parts of the spans, follows the construction of the deck slab. In this point it must be noticed that the compression reinforcement of the supports has been already placed and concreted with the pier heads-cantilevers. At first, precast slabs are placed bridging the gap between two ‘lines’ of bilateral cantilevers. Then precast slabs are placed in the next ‘lines’. The procedure is the same with the one followed for the central parts of the spans and is also strict. Finally, the cast in situ part of the deck slab is concreted. The precast slabs have a thickness of 8-10cm and the whole deck slab of 25-30cm generally. The concreting of the cast in situ part of the deck slab follows the same procedure with the central parts of the spans and the precast slabs.

6.5 Summary of the construction procedure

The construction procedure consists of ten steps:

1. Construction of the webs of the piers.

2. The random construction procedure for the superstructure is not allowed.

3. Construction of the lower flange, 0.2m thick, of the bilateral cantilevers of the pier heads, which are transversal in relation to the webs of the piers, on which the precast bilateral cantilevers will be placed.

4. Placement of PBC on the pier heads and concreting of the bilateral cantilevers of the pier heads up to the upper flange of PBC. Specifically for the abutments, there are three alternative solutions: a) the preferred cast in situ solution, under limitations, b) the use of cables in order to undertake the moments of the non-symmetric cantilevers, c) the simply support choice for the superstructure, characterized by earthquake resistance and aesthetic handicap.

5. Placement of suspended formworks along one 'line' of PBC and concreting of the 10m long central parts of the spans.

6. Removal of the formworks and placement of them along the next 'line' of PBC and concreting of this 'line'and so on for all the lines of PBC.

7. Placement of the precast slabs consecutively, in the same way as above .

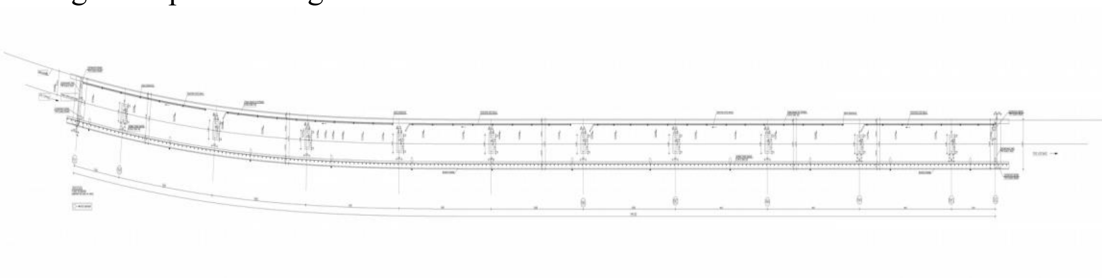
8. Placement of the upper, longitudinal and transversal reinforcement of the deck slab and of the upper longitudinal reinforcement at the supports and the bilateral cantilevers of the pier heads.

9. Concreting of the upper flange of the pier heads-cantilevers.

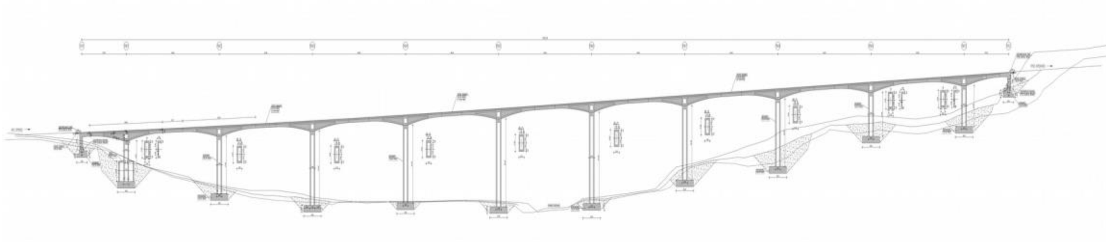
10. Concreting of the cast in situ part of the deck slab, following the procedure of consecutive lines as by PBC (4 and 5).

7 CASE STUDY FOR THE SECOND PROPOSAL

The bridge which will be used as a case study is the C4 bridge of Northern Road Axis of Crete. According to the real study, the bridge will be constructed with the cantilever method. Modifying the study according to the new proposed method, the bridge has 11 spans (the intermediate spans are 46m long and the end spans are 22m long) and a total length of 458m. The precast bilateral cantilevers have a length of 36m which leads to a 10m long cast in situ central part of each span. These cantilevers have a double tee section, a height of 2.7m at the beginning of the cantilever, at the support position, and of 0.80m at the end of the cantilever. The upper and bottom flange of the cantilevers are 0.75m wide, and their web is 0.25m wide. Six precast cantilevers are placed on each pier with a distance of 2.3m between each other. The deck slab has a thickness of 30cm. The prestressing tendons of the precast cantilevers are at the upper flange and are prestressed in a prestressing bed in direct bond with concrete. They are composed by 40 straight wires having each one a cross section of $A=1,39cm^2$. The redesigned bridge is depicted in figure 24.



(a)



(b)

Figure 24: (a) Horizontal section and (b) Longitudinal section of the redesigned bridge.

Two issues will be investigated:

1. The function of the abutments which are seated at the pile cap on bearings which are placed not at the upper flange of the pile cap but at the bottom of a grooving, with appropriate depth and width, so that the sides of the grooving function as bilateral stoppers (Figure 21).

2. The regularity of the integral bridge, in relation to the piers which have variable height and whose sections should be altered.

7.1 Abutments seated on bearings and restrained between stoppers

In functional mode the abutments can move on the bearings as there are foreseen gaps equal to the functional movements between the abutments and the stoppers (Figure 21). So, in the functional model the abutments will be simulated with their bearings. In seismic mode, where the movements will be larger, the abutments will hit to the stoppers, although they need to traverse the distance of the gap. So, in the seismic model the abutments will be simulated with hinges (simulating the stoppers) but with reduced stiffness due to the gaps. The final stiffness of the abutments at this simulation can be calculated by using the following figure 25 from EC8-2.

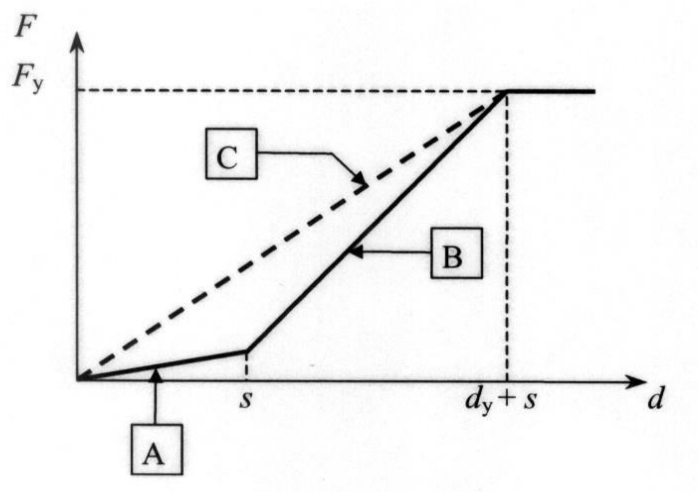


Figure 25: Stiffness of the abutments traversing a gap.

, where:
s = Slack of the link.
d_y = Yield deflection of supporting element.
A = Stiffness of the bearing.
B = Stiffness of supporting element.
C = Linear approximation of the curve.

At first, the stiffness of the abutments as sections is calculated. They have a 13.25*1.00 section and 2Φ20/80 reinforcement in the primary direction. The stiffness is calculated from the following equations:

$$J_{eff} = 0.08*J_{un} + J_{cr} \text{ (EC8)}$$
$$J_{cr} = M_y/(E_c*\varphi_y)$$
$$E_c*J_{eff} = \nu*M_{rd} / \varphi_y \text{ (EC8)}$$
$$E_c*J_{eff} = 300*M_{rd,h}*d \qquad (1)$$

The programs Fagus– 4, Section designer, KSU – RC were used for the sections analysis.

Finally, the stiffness of the abutment is J_{eff} = 40% * J_{un}, namely equal to the 40% of the stiffness of the uncracked section.

The abutments are embedded in the superstructure and simply supported on bearings. So, they function in seismic mode as inverted cantilever. The embedding type can be considered as one from the following three depicted in figure 26.

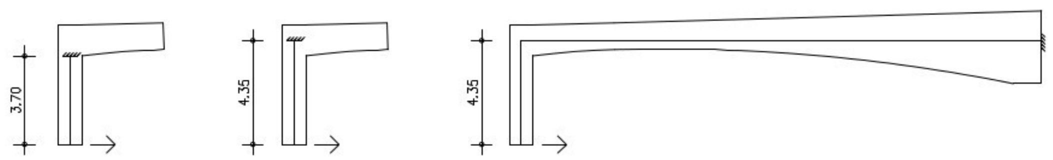


Figure 26: Types of abutments embedding.

The stiffness of the system abutment – bearing is calculated equal to 9% of the total uncracked stiffness of the section. According to the seismic analysis, the abutments do not fail and they undertake 26-30% of the total shear force of the bridge in the longitudinal direction, depending on the piers section. The abutments are also checked under functional loads for the avoidance of cracks.

7.2 Variable sections of piers changing the regularity of the bridge

The final result of this construction method is an integral bridge. So, the piers should have similar stiffness in order to avoid the failure of the short piers because of the big shear force that they will have to undertake otherwise. In this point, it must be noted that this problem does not exist in the conventional prefabrication due to the bearings at the pier heads. Furthermore, the short piers must have reduced stiffness, comparing with the tall ones, so that we have a regular or 'almost' regular bridge. In this case study the reduction of the stiffness is achieved by dividing the hollow section of the tall piers in half (case 1), or seperating the section into two blades (case 2) (Figure 27). This leads to an aesthetic result comparing with the case of piers with totally different dimensions.

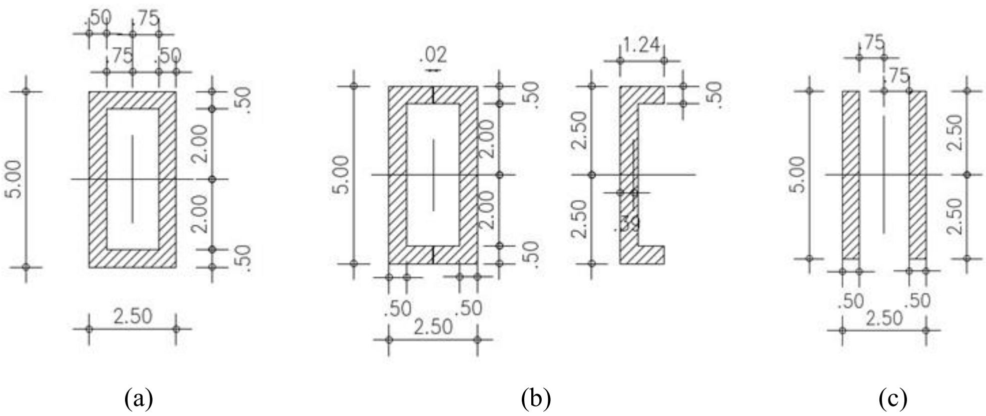


Figure 27: Sections of piers: (a) tall piers, (b), (c) short piers.

The bridge has totally 10 piers. In the following table 1 the section of each pier for the two cases is given.

		case 1	case 2
pier	height	section	section
A1	5	rectangular	rectangular
M1	12	hollow with joints	blades
M2	17	hollow	hollow
M3	22	hollow	hollow
M4	23	hollow	hollow
M5	26	hollow	hollow
M6	27	hollow	hollow
M7	22	hollow	hollow
M8	20	hollow	hollow
M9	13	hollow with joints	blades
M10	12	hollow with joints	blades
A2	5	rectangular	rectangular

Table 1: Piers sections for the two cases.

The bridge is characterized as regular if $\rho \le \rho_0 = 2$, where: $\rho = \frac{r_{\max}}{r_{\min}} \le \rho_0$, (2)

where: r_i is a dimensionless coefficient calculated for each pier: $r_i = q * \frac{M_{Ed,i}}{M_{Rd,i}}$ (3)

Furthermore, the piers which undertake shear force smaller than the 20% of the total shear force in the direction under consideration, can be excluded from the control. This last exception from the control is always true in this case study, because of the large number of the piers. So, the spirit of the code was interpreted and only the piers which undertake too small shear force comparing with the rest were excepted .

In case 1 the bridge was considered as irregular and in case 2 as regular. The reasons for this are:

1. The hollow sections with joints have moment of inertia equal to the 11,3% of the hollow section in the longitudinal direction. In the transverse direction it has the same moment of inertia with the hollow section.
2. The blade sections have moment of inertia equal to the 1,9% of the hollow sections in the longitudinal direction, and 57,7% of moment of inertia of hollow sections in the transverse direction.

8 CONCLUSIONS FROM THE SECOND PROPOSAL

A new method of prefabrication is described and applied in this paper concerning the superstructures of bridges and not only. Compared to the conventional one, it has significant advantages, that are gained after a number of difficult constructional and functional problems have been resolved. The most important conclusions from the analytical investigation of the problem can be summarized as follows:

1. The proposal has undoubted innovation features in the implementation of Prefabrication.
2. By applying the proposed method, superstructures of bridges at any height from the ground can be constructed.

3. It leads to a monolithic result and relieves Prefabrication from the till nowadays absolutely necessary use of bearings and this implies not only a reduced bridge construction cost but also a reduced maintenance cost.

4. It makes possible to increase the length of the bridge spans, which from 40m, that is considered as a limit for the conventional prefabrication, can exceed 50m, resulting in savings in the number of piers.

5. Allows the adjustment of the section height of the bridge superstructure to the existing strain induced by gravity loads, and this, besides economy, has an important impact on the aesthetics of the result as well as on its integration into the environment, taking into account that the proposal concerns generally viaduct and ravine bridges.

6. It is free from the unattractive seating of the superstructure on the upper flange of the pier heads, as the precast beams are embedded over their entire height in the pier heads.

7. Allows the application of prestressing bed for the precast cantilevers resulting in saving in anchorages, which are the most expensive part of prestressing applications.

8. It allows applications of full, limited and partial pretension, and in any case is accompanied by significant savings in prestressing steel.

9. There are three variants examined for the support of the superstructure at the abutments which are accompanied by corresponding constructional, aesthetic and earthquake resistance advantages. In particular, the earthquake resistance advantage consists in the function of the wall-type web of the abutment as a seismic stopper. This utilization of the wall-type web as a stopper has significant advantages, such as the containment of seismic displacements and the resulting relief of the piers, which undertake the inertial seismic actions.

10. The accomplished monolithic result of the system enables a better and more economical treatment of the earthquake, as it is mainly dealt with by ductility, reduces seismic displacements and hence the impact of the critical for tall bridges PΔ -effect.

11. Allows the functional problems arising from the monolithic result to be handled in a flexible way, as it exploits the division of the generally hollow sections of the pier.

9 GENERAL CONCLUSIONS

Two proposals of scalable monolithic connection at bridges, where the prefabrication is exploited, were examined in this paper. In the first case, prefabricated beams with a length equal to the length of the span are utilized, while in the second, which is the peak point of this research, prefabricated static subsets of bilateral cantilevers of variable section height are used, which are afterwards completed with cast in situ sections, concreted with suspended at the cantilevers formwork. The second proposal is far superior, regarding all the evaluation criteria. From the comparison of the two cases that were applied in two cases of bridges of high requirements, the following conclusions were reached:

1. The resulting material consumption economy in the second case is particularly striking, since gross calculations have shown that only 2/3 of these materials (concrete, conventional reinforcement and prestressing steel) are required compared to the first proposal.

2. From the second proposal, a controlled and substantial reduction in stiffness, on the one hand of the piers and on the other of the superstructure, is resulting, with beneficial effects: a) for the functionality of the system and in particular in mitigating the intensity caused by the constraints due to the expansion and contraction of the deck, and due to any differential settlements of piers and b) the resulting flexibility of the overall system increases the eigenperiod, resulting in drastic reduction of inertia seismic actions, which relieves besides piers, their foundations significantly as well.

3. The resulting aesthetics in the second case is clearly improved thanks to the curvature of the bottom flange, which reminds the traditional geometries of mountain bridges, which utilized the arch and the bow to undertake the loads at spans of great length.

4. The near to zero cost of bearings and expansion joints eliminates the additional maintenance costs, which as a rule are usually accompanied by the shutdown of traffic on the bridge, during the replacement of these accessories.

5. The use of specially shaped abutments as stoppers relieves the piers, that are the structural elements, that undertake most of the seismic strain, thus ensuring the earthquake resistance of the bridge with minimal cost.

It is concluded from the foregoing that the second proposal obviously outclasses both the first one and the corresponding, till nowadays applied, proposals for prefabricated bridges.

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REFERENCES

[1] I. Moschonas, M. Tsitotas, I.A. Tegos, Assessment criteria for static systems of bridges, *13th Greek Concrete Conference,* Rethymno, Crete, Greece, October, 1999.

[2] M. Pötzl, J. Schlaich, Robust Concrete Bridges without Bearings and Joints, *Structural Engineering International*, 4/96, 1996.

[3] CEN [European Committee for Standardization], *EN 1998-2:2005 Eurocode 8: Design of structures for earthquake resistance, Part 2: Bridges*, 2005.

[4] I.A. Tegos, S.D. Tegou, M.A. Tsitotas, A Proposal for the improvement of the earthquake resistance of multi-span precast I-Beam bridges, *Proceedings of the Istanbul Bridge Conference*, Istanbul, Turkey, 2014, ID: 137.

[5] I.A. Tegos, S.D. Tegou, A Proposal for the Improvement of the Earthquake Resistance of Multi-Span Precast I-Beam bridges, *International Conference IBSBI 2014 – Innovations on Bridges and Soil-Bridges Interaction*, 16-18 October 2014, Athens, Greece, ID: 062.

[6] Ministry of public works of Greece, *Greek seismic code – EAK 2000*, Athens, 2000 (Amended June 2003).

[7] Ministry of public works of Greece, *Guidelines for seismic design of bridges in combination with DIN-FB 102, 103, 104*, Athens, 2007.

[8] I.A. Tegos, S.D. Tegou, A proposition for the enhancement of the earthquake resistance and the cost-effectiveness of precast I-beam bridges: implementation in a precast I-beam bridge of Egnatia Odos motorway, *Chapter in A seismic design and construction in Egnatia Odos, the highway connecting Epirous through Macedonia to Thrace and the eastern border of Greece*, Hellenic Society for Earthquake Engineering, Ziti Publications, Thessaloniki, Greece, ISBN: 978-960-99529-0-3, 2010.

[9] I.A. Tegos, A proposal to improve the earthquake resistance of partially prefabricated concrete bridges of multiple spans, *Research Program funded by EPPO (Earthquake Planning and Protection Organization)*, 2002.

[10] A. Kumar, Continuity in composite concrete bridge construction, *International conference on short and medium span bridges*, Montreal, Canada, August, 1986.

[11] A. Kumar, Composite concrete bridge superstructures, Wexham Springs, *British Cement Association*, 1988, pp 46, Publications 46.505.

[12] A. Kumar, Locally separated deck slab continuity in composite bridges, *Continuous and integral bridges*, edited by B.P. Pritchard, published by E&FN Spon, 2-6 Boundary Row, London, 1994, ch. 21, pp 239-246.

[13] B.P. Pritchard, Integral in-situ crossheads for continuous precast beam deck viaducts, *International Conference on short and medium span bridges*, Toronto, Canada, 1982.

[14] B.P. Pritchard, A.J. Smith, Investigation of methods of achieving continuity in composite concrete bridge decks, *TRL Contractor /report CR 247*, Crowthorne, 1991.

[15] B.P. Pritchard, Continuity of precast pretensioned concrete beams by embedment in integral crossheads, *Continuous and integral bridges*, edited by B.P. Pritchard, published by E&FN Spon, 2-6 Boundary Row, London, 1994, ch. 20, pp 229-237.

[16] S. Tegou, Analytical and experimental research on the earthquake resistance performance of a proposed new type of abutment that can contain the seismic movements of the bridge superstructure, *Aristotle University of Thessaloniki*, Thessaloniki, Greece, 2012.

[17] V.Pilitsis, I.A.Tegos, A Proposal for Improving Regularity of Integral Concrete Bridges, *Proc. of 2nd International Conference on Innovations on Bridges and Soil-Bridge Interaction (IBSBI 2014)*, Athens, Greece, October 16-18, 2014.

[18] I.A. Tegos, V. Pilitsis, The Use of Piers of High Flexibility for the Seismic Isolation of Integral Bridges, *Proc. of 2nd International Conference on Innovations on Bridges and Soil-Bridge Interaction (IBSBI 2014)*, Athens, Greece, October 16-18, 2014.

[19] K. Katirtzoglou, S.D. Tegou, I.K. Papaefthymiou, I.A. Tegos, Influence Precast applications for the accomplishment of monolithic results in seismic resistant bridges, *2013 Van Earthquake Symposium,* 23-27 October 2013, Van, Turkey, ID:104.

[20] N. Fardis, C. Katsaras, T. Panagiotakos, Design of a seismic isolated bridge with precast beams and achieved continuity, *17th Greek Concrete Conference,* Thessaloniki, Greece, November, 2016.

[21] W. Rossner , Brücken aus Spannbeton-Fertigteilen, *Ernst & Sohn Verlag*, Berlin,1988.

[22] CEB-FIB [The International Federation for Structural Concrete], Precast concrete bridges, State-of-the-art report, *Bulletin 29*, fib Task Group 6.4, Switzerland, 2004.

[23] I.K. Papaefthymiou, M. Tzortzinakis, I.A. Tegos, A research on the possibilities of attaining a monolithic connection between superstructure and piers at prefabricated bridges, *17th Greek Concrete Conference,* Thessaloniki, Greece, November, 2016.

[24] Computers and Structures Inc, *SAP 2000 nonlinear version 17.1.1 user's reference manual*, Berkeley, California, 2013.

[25] DIN [German Institute for Standardization], *DIN – Fachbericht 102: Concrete bridges*, 2003.

[26] DIN [German Institute for Standardization], *DIN – Fachbericht 101: Actions on bridges*, 2003.

[27] I.K. Papaefthymiou, O. Markogiannaki, I.A. Tegos, An innovative proposal for prefabrication of bridges with economic, aesthetic, earthquake resistance and constructional advantages, *17th Greek Concrete Conference,* Thessaloniki, Greece, November, 2016.