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SENSITIVITY OF A STEEL ARCH ROAD BRIDGE TO IMPOSED FOUNDATION DISPLACEMENTS AND ROTATIONS

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ABSTRACT

It is common practice to use deep foundation (piles) for bridges in areas prone to seismic liquefaction, transferring loads to deeper, non-liquefiable soil layers. Nevertheless, recent studies suggest that the existence of a surface "crust" of non-liquefiable soil with sufficient thickness and shear strength may mitigate the consequences of liquefaction in the subsoil, so that the use of shallow foundations becomes permissible. Moreover, reduced inertia forces act on the superstructure, as the part of the subsoil that will be intentionally allowed to liquefy, will lose its shear resistance and may consequently act as seismic isolation. However, shallow foundations are admittedly more sensitive to differential settlements, which are likely to create additional actions to the bridge superstructure. This issue is investigated in the present paper for the case of a two-span steel arch road bridge. The effect of large displacements and rotations, induced at the level of the pier foundation, on the structural efficiency of the superstructure is investigated by means of nonlinear analyses. The results show that the bridge under consideration can sustain large displacements at the base of the pier without significant damage. However, the superstructure is quite vulnerable to imposed rotations of the foundation.

1 INTRODUCTION

For many engineering works, like road or railway bridges, which are to be constructed in river bank areas or in areas close to the shore, it is unavoidable that they are founded in soil layers, which are liquefiable in case of a strong earthquake. Nowadays such problems are dealt with quite conservative solutions, such as pile foundation and selection of structurally determinate systems for the superstructure. However, even in those cases, it is still possible, that the deformations which will occur in the liquefiable layer during an earthquake will be high enough to cause extreme loading to the piles and the superstructure. For this reason improvements are applied to the liquefiable soil layer in order to reduce bending moments in the piles.

An alternative design philosophy, which is based on the application of shallow foundation on liquefiable soils with a surface crust of non-liquefiable soil, is investigated at the National Technical University of Athens within the research program "Innovative design of bridge piers on liquefiable soils with the use of natural seismic isolation". This idea is based on previous research regarding the seismic response of shallow foundations on liquefiable soil [1]-[4], which has shown that the existence of a surface crust with sufficient thickness and shear strength can mitigate the consequences of liquefaction. In the proposed methodology an artificial crust of non-liquefiable soil is created over the underlying liquefiable soil using one of the available methods for soil improvement. The new design concept has the additional advantage of using the liquefied soil as a natural system of seismic isolation. In this way, reduction of the overall cost is achieved, through the construction of a more cost effective foundation, compared with the conventional pile foundation, and through the reduction of the inertia forces acting on the superstructure.

However, there may also be some potentially undesirable effects associated with this new design concept, since shallow foundations are prone to increased settlements and rotations, which may affect the actions induced to the superstructure. Especially for shallow foundations on liquefied soil, the induced displacements/rotations may be significant during or after a strong earthquake. Therefore, the applicability of the proposed design philosophy depends on the capacity of the superstructure to sustain large deformations of the foundation.

The objective of this paper is to present an investigation of the behaviour of a road steel arch bridge consisting of two simply supported spans, during the development of extreme displacements/rotations at the foundation of the pier, conducting non-linear analyses using the FE-program SAP2000 [5].

2 CASE STUDY CONSIDERED

The bridge under investigation is a two simply supported spanned steel arch road bridge, situated over a riverbank. The deck of each span has a width of 15.00m and it consists of two 42.00m long main steel beams with HEB900 cross-section, transverse steel beams having also HEB900 cross-section, arranged at distances of 2.625m and a 20cm thick concrete slab, connected with the transverse beams with steel shear connectors. Each span is suspended from two arches of CHS 750/20 profile. The height of the arches arises at 10.00m. The struts have a CHS168.3/6.3 cross section, while the horizontal and diagonal steel members, connecting the two arches, are made of profiles CHS 244.5/8 and CHS 139.7/6.3, respectively. The pier is assumed to be seated on liquefiable soil. It consists of three circular reinforced concrete columns, 8.50m tall, having a circular cross section of 1.50m diameter. Its footing has plan dimensions 14.75m x 5.40m and thickness of 2.00m. Longitudinal, transverse and vertical springs are taken into account with values K_x=430000kN/m, K_y=165000kN/m and K_z=2400000kN/m, respectively, representing the stiffness of typical abutments seated on firm and stable soil of category B, according to EC8 [6]. The two decks are connected with the pier and the abutments with two elastomeric bearings anchored at the ends of the main beams. The bearings have dimensions 600mm x 700mm / 10 sheets of 15 mm and were modelled with equivalent elastic springs. Expansion joints are formed on the deck above the pier and the abutments. The numerical model of the bridge is shown in Figure 1. All steel elements are made of S355 structural steel, while concrete grade C20/25 is used for the piers and the foundation elements and C35/45 for the composite deck. The reinforcement steel is B500C.

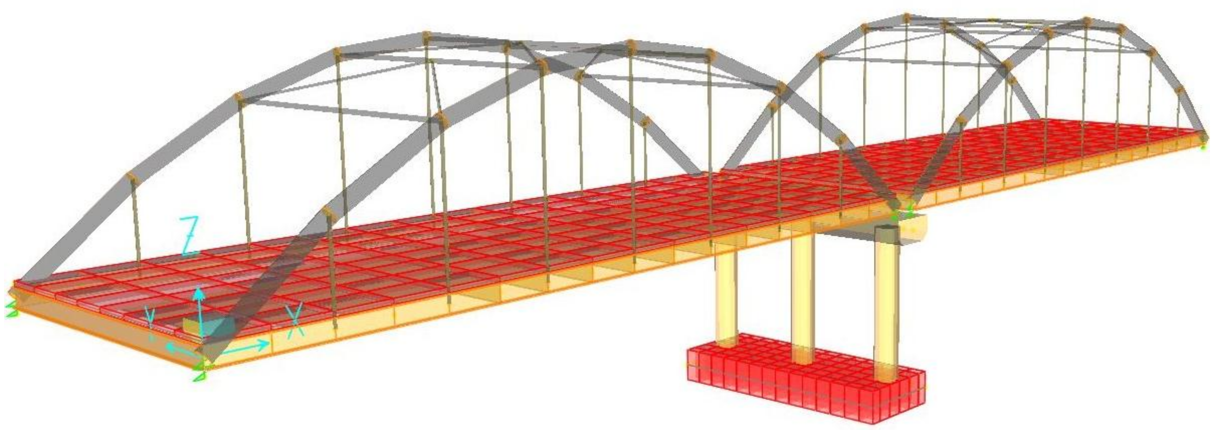
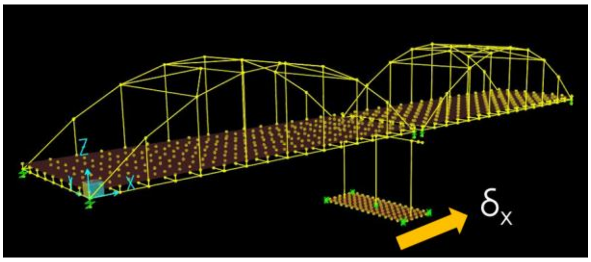


Figure 1. Numerical model of the bridge

3 METHOD OF ANALYSIS

The research goal was to investigate the soil-structure interaction for the case of extreme deformations occurring at the lever of the pier foundation, accounting for soil liquefaction during a strong earthquake, to estimate the sensitivity of the structure to such phenomena and to identify possible failure mechanisms. In the present investigation it was assumed that liquefaction occurs only in the region of the pier and not at the abutments. Non-linear analyses were performed for this reason, during which large displacements/rotations were imposed at the pier foundation, while only the self-weight of the structure was considered. For each case, the behaviour of all members of the bridge was recorded throughout the evolution of the phenomenon, in order to identify which members reach their ultimate state, which of them are the most endangered by such phenomena, in which cases failures occurred and, finally, which of the failures could actually threaten the stability of the whole structure. Non-linear material was considered only for the sections at the top and the bottom of the columns of the pier, where plastic hinges were expected, while all other members were assumed to behave elastically, even after exceeding their ultimate capacity.

Since every deformation that may develop at the footing can be analyzed in components along the six primary degrees of freedom (three displacements and three rotations), the cases that were considered included horizontal displacements and rotations, as shown in Figure 2. Imposed displacement with respect to the vertical axis z was not taken into account, because the bridge, being composed of two simply supported spans, can sustain large vertical displacements without any serious problem at the superstructure. In each case, continuously increasing displacement/rotation was applied only on one degree of freedom at the centre of the footing, until the structure collapsed, while all other degrees of freedom were considered fixed.



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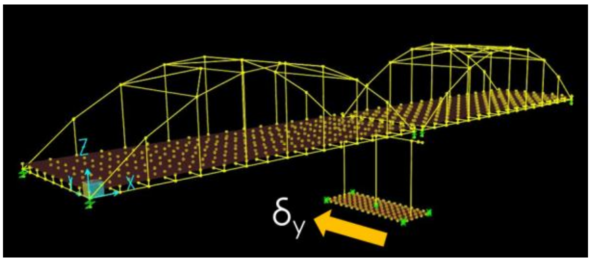

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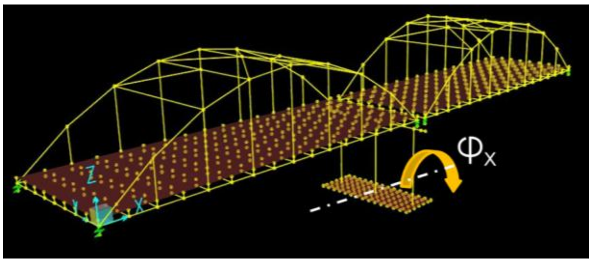

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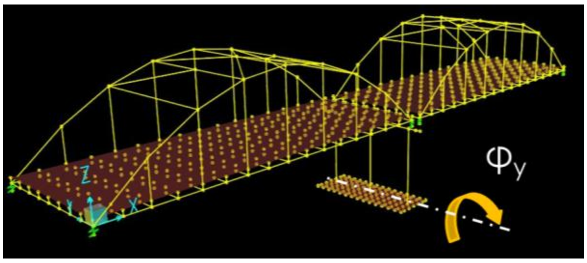

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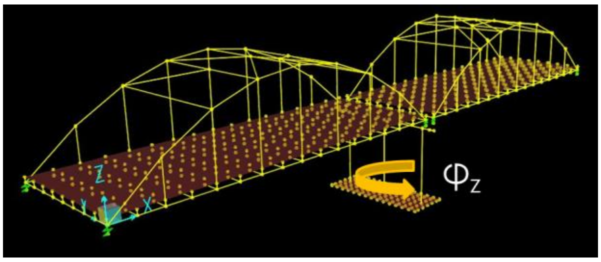

(e)

Figure 2. Imposed displacements/rotations at the pier foundation considered in the analyses: (a) displacement along the longitudinal direction (δ_x), (b) displacement along the lateral direction (δ_y), (c) rotation around the longitudinal axis (φ_x), (d) rotation around the lateral axis (φ_y) and (e) rotation around the vertical axis (φ_z)

4 RESULTS OF THE ANALYSES

For each case and for each level of loading the overall behaviour of the bridge was investigated. Emphasis was given on the failure of the important/vulnerable members of the structure, including the columns of the pier, the bearings, the expansion joint at the pier, all steel members of the arch and the deck. To this end, corresponding damage indices were calculated at each step of the analysis and plotted in common diagrams for each type of deformation, so that a comparison between the response of the different members could be achieved. These damage indices take values from 0 (corresponding to no damage) to 1 (accounting for failure). For each group of members, the largest damage index (DI) among all members is considered as the group DI. In Table 1 the definition of the DI is given for each group of members.

Table 1. Damage Indices considered

Member	Formula	Definition
Pier	$DI = \frac{\theta_m - \theta_r}{\theta_u - \theta_r}$ where: θ_m is the rotation of the cross-section at the step under consideration θ_r is the yield rotation θ_u is the ultimate rotation	The damage index is calculated in terms of plastic rotation ($\theta_m - \theta_r$) at the plastic hinge (bottom of column) over the ultimate plastic rotation ($\theta_u - \theta_r$), according to [7], assuming $dE_a = 0$ for one cycle of loading.
Bearings	DI = γ/2.50 for γ < 2.50 DI = 1 for γ ≥ 2.50	The DI is calculated in terms of the shear strain of the bearing, γ, according to EC 8-2 [10] and considering an overstrength factor 1.25
Expansion joint	$DI = \frac{\Delta l}{l}$	The DI is calculated in terms of the axial strain of the joint, where Δl is the closing of the expansion joint and l is its initial gap.
Steel Members		DI is specified as their exploitation factor, depending on the internal forces developed, according to EC 3 and EC 4 ([9]-[12]).

4.1 Imposed displacement along the longitudinal direction (δ_x)

The damage indices of the important member groups versus the imposed horizontal displacement δ_x is shown in Figure 3. It is noted that the most vulnerable members were the three columns of the pier, the failure of which occurred for a displacement of 0.65m. Large deformations evolved also at the bearings, but their DI remained smaller than 0.60 up to the failure of the pier, which does not imply serious damage to the bearings. It is also important to note that the steel superstructure was not affected by the longitudinal displacement of the pier foundation, as it is composed of simply supported spans.

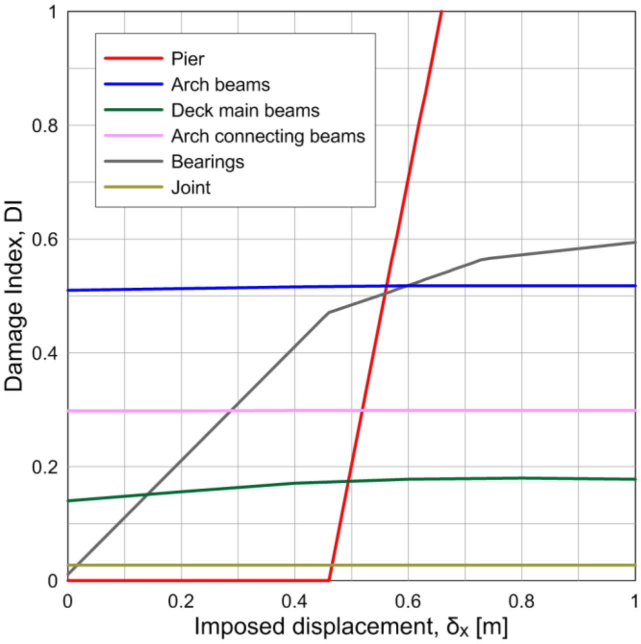

Figure 3. DI of the important member groups versus the imposed displacement δ_x

4.2 Imposed displacement along the lateral direction (δ_y)

In this case, a maximum displacement of 5.00m was applied to the model. It was shown that throughout the analysis none of the crucial members of the structure (pier - steel superstructure) were affected. This was because the two spans and the pier moved as rigid bodies and the entire impact of the imposed deformation was transferred to the bearings of the pier and the gap. The only part of the structure that was seriously affected was the expansion joint between the two spans, which reached DI=1 at about δ_y=0.93m (Figure 4). The bearings were also affected but the maximum DI was about 0.65 at δ_y=5.00m, without experiencing any kind of failure.

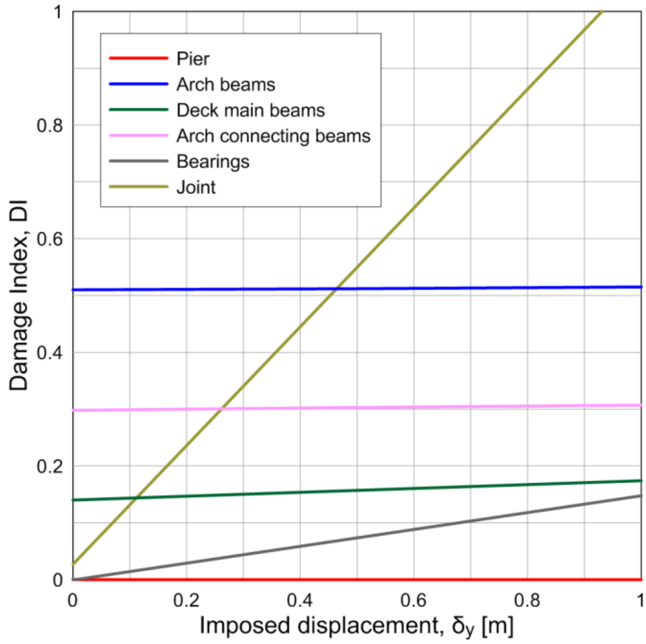

Figure 4. DI of the important member groups versus the imposed displacement δ_y

It is worth mentioning that the structure exhibits low sensitivity for imposed deformations in the lateral direction. The only part that reached the ultimate state was the expansion joint, which is replaceable and by no means can cause the collapse of the whole structural system.

4.3 Imposed rotation around the longitudinal axis (φ_x)

The analysis for imposed rotations of the footing around the longitudinal axis of the structure led to interesting conclusions. As shown in Figure 5, the first failures occurred to the steel superstructure, caused by the warping of the composite deck, which led most steel members to failure for quite small imposed rotation, about φ_x = 0.05rad. The pier reached failure at much larger rotation, about φ_x = 0.437rad, which was caused by the interaction of large bending moments developed at the base of the columns and large axial forces. It is important to mention that, since the footing has a length of 14.75m, rotation of φ_x = 0.05rad corresponds to differential settlements between the sides of the footing approximately equal to 0.75m.

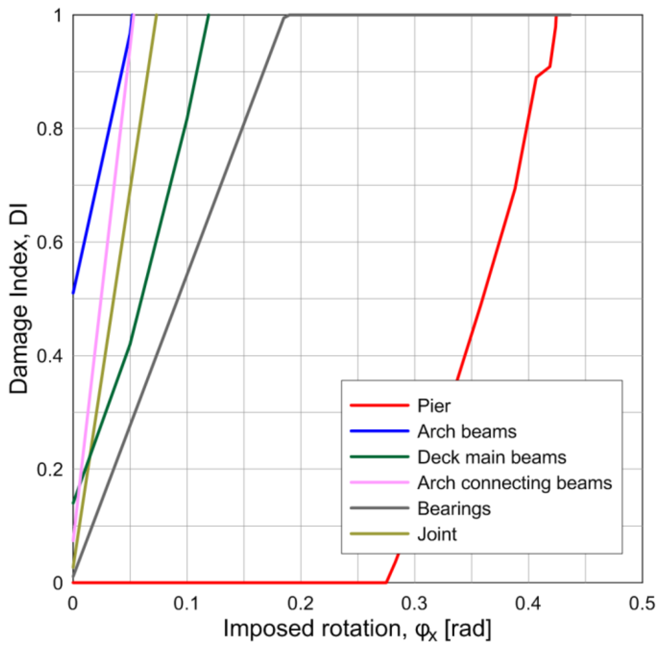

Figure 5. DI of the important member groups versus the imposed rotation φ_x

4.4 Imposed rotation around the lateral axis (φ_y)

The results of this analysis were also very interesting, as all imposed deformation is absorbed by the columns of the pier, while the other members receive almost no extra loading because of the deformation of the footing. As a result, the maximum allowable rotation around the lateral axis of the structure induced at the footing is only related to the design and the limits of the column cross-sections. As shown in Figure 6, the columns of the pier reached failure for φ_y = 0.06rad. The width of the footing is 5.40m. This means that a rotation φ_y = 0.06rad corresponds to a differential settlement of about 0.32m.

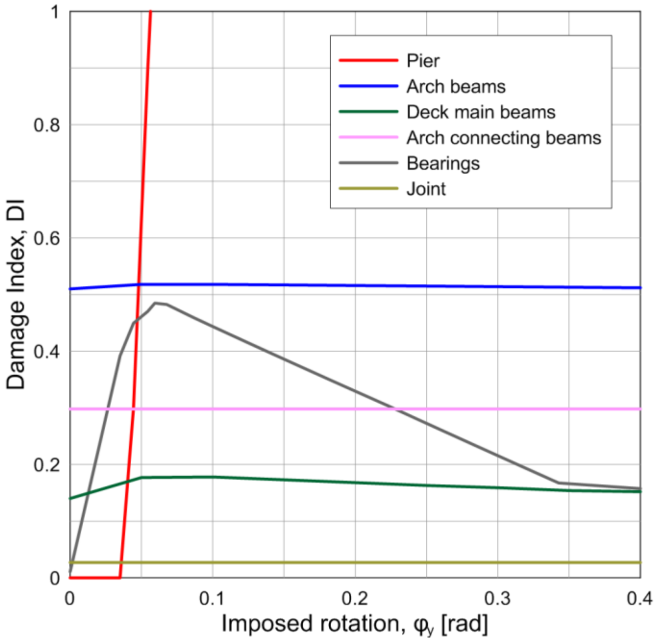

Figure 6. DI of the important member groups versus the imposed rotation φ_y

4.5 Imposed rotation around the vertical axis (φ_z)

In this case, the most vulnerable parts were the side-columns of the pier, which reached the level of ultimate state for φ_z=0.10rad (Figure 7). The middle column, on the other hand, remained almost uninfluenced. The bearings reached a value of DI=0.70 but did not fail. The superstructure was slightly influenced by this kind of deformation.

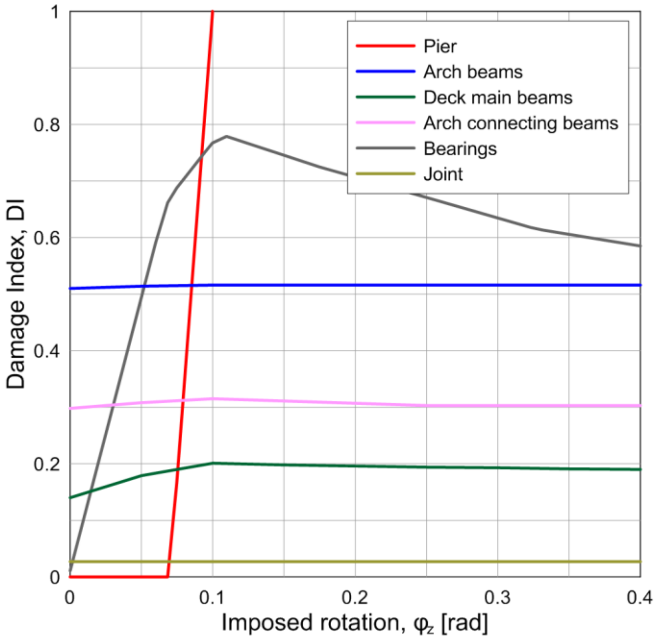

Figure 7. DI of the important member groups versus the imposed rotation φ_z

4.6 Summarized results

In Table 2 the maximum imposed displacement/rotation of each analysis up to the first failure of any member is shown. Comparing the diagram illustrating the results of each analysis it is important to note that the bridge showed the highest sensitivity for imposed rotations around the longitudinal and the lateral axis, whereas for the other three degrees of freedom its tolerance was quite higher.

Table 2. Imposed displacement/rotation at first failure

Imposed Deformation	Maximum Value - Failure	First Member Failed
δ_x	0.65m	Pier column
δ_y	1.00m	Expansion joint
φ_x	0.05rad (differential settlement = 0.75m)	Arch beam
φ_y	0.06rad (differential settlement = 0.32m)	Pier column
φ_z	0.10rad	Pier column

5 CONCLUSIONS

In this paper the behaviour of a road steel arch bridge consisting of two simply supported spans was investigated for extreme displacements/rotations imposed at the foundation of the pier. The purpose of this research was to assess potential undesirable effects from the construction of shallow foundation on liquefiable soil, which are prone to large displacements/rotations, during strong earthquakes.

Five different cases were considered in order to investigate the response of the arch bridge, subjected to imposed deformations at the centre of the pier footing. In most cases, the pier was the most vulnerable component of the bridge. It was shown that failure occurred first to the columns of the pier for displacements in the longitudinal direction, rotations around the lateral and vertical axes. Imposed rotations around the longitudinal axis were the most adverse case, since almost all members reached failure for relatively small values of imposed deformation. Finally, the analysis for imposed displacements along the lateral axis showed that a bridge, like the one assumed here, exhibits no sensitivity to such deformations.

6 ACKOWLEDGEMENTS

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