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SEISMIC ASSESSMENT OF AN OVERPASS BRIDGE ACCOUNTING FOR NON-LINEAR MATERIAL AND SOIL RESPONSE AND VARYING BOUNDARY CONDITIONS

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Abstract. *Seismic assessment of bridges using the popular 'pushover' (i.e. nonlinear static) analysis technique often ignores the effect of some sources of nonlinearity such as those associated with the foundation soil and the boundary conditions, that may significantly modify the overall performance of the structure and the estimated pushover curves. In this context, the seismic response of a typical overpass constructed along the 670km Egnatia highway in northern Greece is assessed herein using lumped plasticity models to account for the inelastic behaviour of the critical cross-sections of piers and piles, and non-linear springs to consider foundation-soil compliance. The results of the analysis show a markedly different seismic behaviour when the abutment – soil system is included in the analysis, rather than simply considering a pinned support (in the transverse direction) as usually done in previous studies. Furthermore, for stronger excitations, it is seen that as inelastic mechanisms (of piers, piles, pile caps, and soil) are introduced and different non-linear components (i.e. joint /gap closure) are activated, the assumptions made on the foundation and soil compliance play an increasingly important role that can potentially modify the anticipated failure hierarchy, as well as the ensuing pushover curves in both directions of the bridge.*

1 INTRODUCTION

It is now well-established that elastic analysis of structures subjected to seismic actions, typically in the form of response spectrum analysis, cannot always predict the hierarchy of the failure mechanisms, while it is not able to quantify the energy absorption and force redistribution that result from the gradual plastic hinge development within the structure. For this reason, during the last 15 years or so, the development of analytical methods that would permit the quantification of the degree of global and/or local ductility (that depends on the level of earthquake excitation) has increasingly attracted the attention of both researchers and designers. Along these lines, nonlinear static (pushover) analysis has become a popular tool for the seismic assessment of buildings [1] and bridges [2], despite the fact that its main advantage of lower computational cost, compared to nonlinear dynamic time-history analysis, is counter-balanced by its inherent restriction to structures wherein the fundamental mode dominates the response. A recent contributor to this popularity is the extension of the push-over approach to consider higher mode effects for both buildings [3], [4] and bridges [5].

The aforementioned non-linearity expected in bridges during strong ground motions, cannot be attributed solely to yielding of reinforced concrete (R/C) sections, although these are the elements that are often purposely designed to exhibit inelastic behaviour. On the contrary, both additional material non-linearity mechanisms (of the foundation and/or backfill soil) and geometrical non-linearity mechanisms (activation of control components such as bearings, 'stoppers', or seismic joints) can also play a significant role in the overall system response. Moreover, despite the existence of specific guidelines in the US [6], [7], [8], [9] and in Europe [10] for the *design* of piled foundations and abutments and the decisions related to the above components, only minor guidance is provided for the *modelling* of the problem from a numerical point of view, or for the consideration and assessment (even statically) of the soil-foundation-pier-deck [10,11], and soil-abutment-deck system [12,13,14], interaction that are associated with high level of uncertainty. As a result, in design, the abutment capacity and stiffness, as well as the effects of soil nonlinearity, are most commonly completely ignored, mainly for reasons of convenience.

The scope of this paper, therefore, is to focus on a real, already built, bridge structure that has been designed to resist seismic forces through both capacity-designed elements and control components (bearings, 'stoppers' and seismic joints), in order to assess its performance in the light of a refined finite element modelling of the abutments and the pier foundation subsystems.

It is noted that the dynamic interaction of the complete system is a far more complex and multiparametric problem, especially in terms of the coupling between the geometry of the abutment and the backfill/embankment, the dynamic soil properties at high strains (as expressed by the G-γ-d curves), and the dynamic characteristics of the structure. Nevertheless, it is deemed that this issue, although studied in depth in the literature [15], [16] is too complex and (inevitably) frequency-dependent, to permit drawing conclusions of general validity, regarding the relative contribution of the aforementioned material and geometrical non-linearity sources. As a result, the specific assessment presented here is performed using the non-linear static (pushover) method. A brief description of the structure assessed is presented first, followed by the analysis overview and critical evaluation of the results.

2 OVERVIEW OF THE BRIDGE STUDIED

The particular bridge studied is an overpass (overcrossing) along the Egnatia highway. It is a three-span, symmetric bridge (span lengths are 19, 32 and 19 m respectively) curved in elevation (max camber of 8%), that intersects the highway axis at an angle of 75.3°. The deck is 11m wide and 1.60m high. The prestressed deck has a hollow T-beam-like section (see Fig. 1) and is supported on two circular piers of 1.70m diameter and 8.50m height which are monolithically connected to the superstructure and the foundation. At the abutments (which have a 10.50×1.20m wall section of 5.0m height), the deck is connected through two pot bearings that permit sliding along the two principal bridge axes and a sliding joint separates the deck from the backwall (Figure 2). Seismic forces are also resisted by the activation of stoppers (in the transverse direction) which are constructed at the seating of the abutments. The foundation on the other hand is deep, due to the soft clay formations characterizing the overall area. The pier foundation consists of a 2×2 pile group of 28.0 to 32.0m long piles, connected with a 1.60×5.0×5.0m pile cap, while the abutments are supported on a 1×4 pile row 27 to 35.0m long at 2.80m axial spacing, all piles having equal diameter of 1.0m.

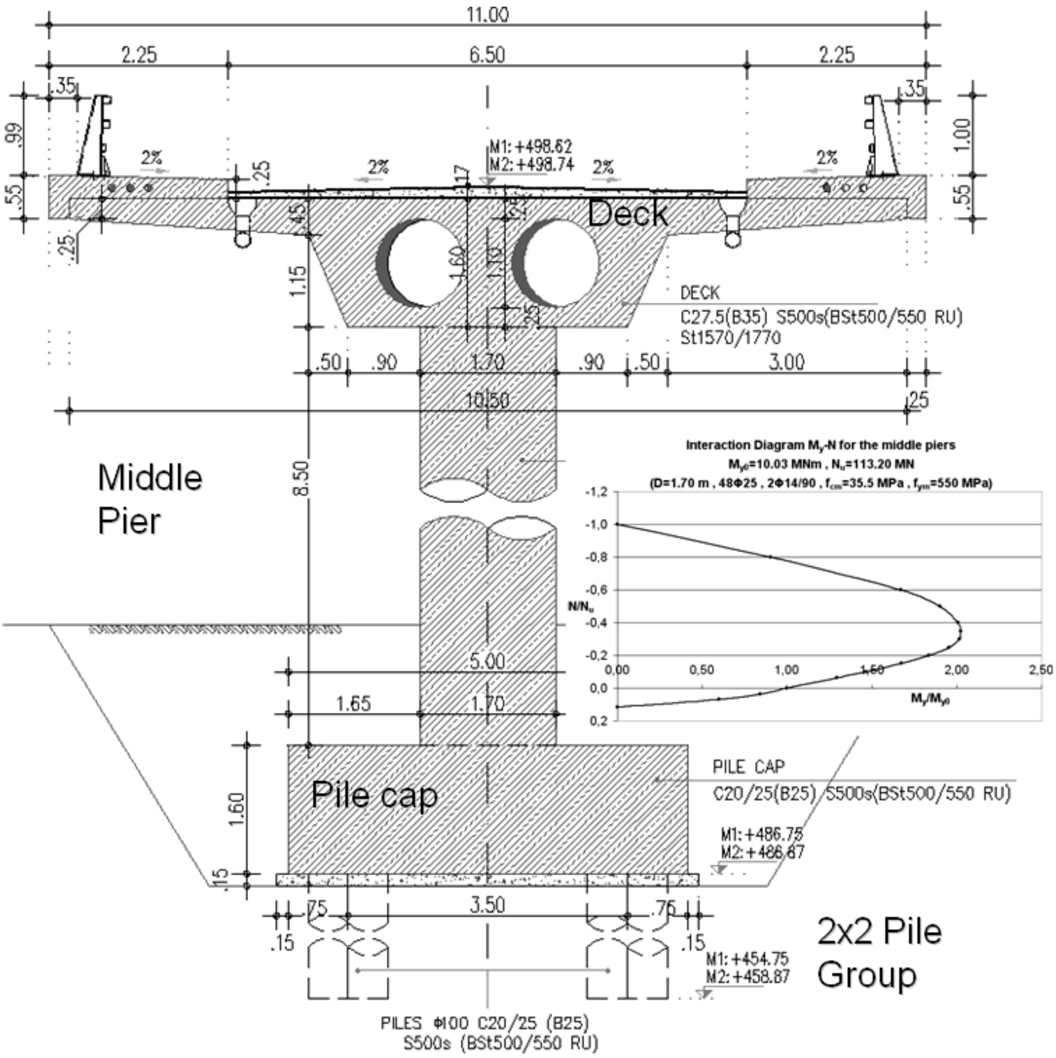

Figure 1: Overview of the foundation-pier-superstructure system cross-section (Piers M1 and M2)

The bridge was designed for normal loads according to the German Norms (i.e. DIN 1055, 1045, 1072, 1075, 1054, 4227, 4085, 4014) while the seismic design was carried out according to the Greek standards EAK 2000 [17] and E39/99 [18], the first being the Greek Seismic Code for the design of structures (General and Buildings) and the latter the Code for the Seismic Design of Bridges. The bridge site is located in the Seismic Risk Zone I which is equivalent to a peak ground acceleration of a_g = 0.16g, while the vertical peak ground acceleration is taken as 0.7 times the horizontal one. The behaviour factors of the system were also adopted according to the E39/99 document and are: q_x=2.50, q_y=3.50, q_z=1.00 for the response in the three principal directions, respectively.

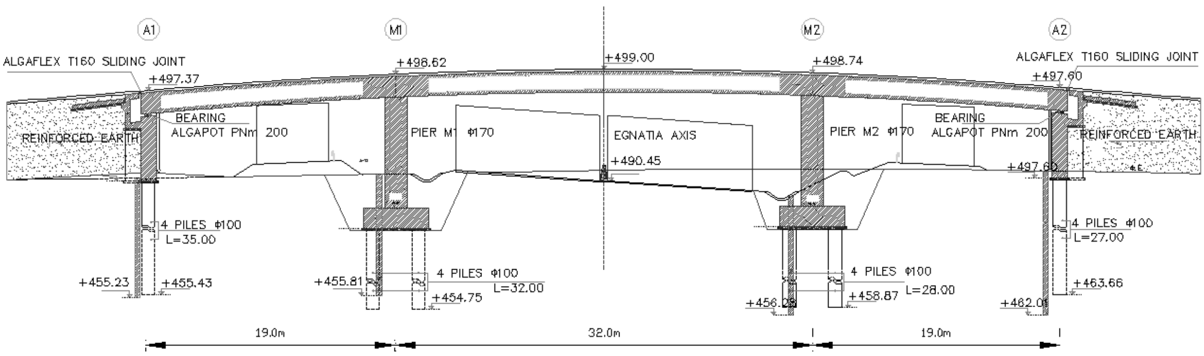

Figure 2: Longitudinal cross-section of the bridge

3 FINITE ELEMENT MODELLING AND ANALYSIS OF THE STRUCTURE

3.1 Modelling aspects

The bridge was modelled using the SAP2000 program [19]. The program is capable of performing a three-dimensional structural analysis using response spectrum, non-linear static (pushover), and time history analysis. A three-dimensional model was created with linear Frame Elements to reflect the geometry, boundary conditions, and material behaviour of the bridge studied. The structure is adequately discretised to account for the bridge parabolic elevation shape and a continuous mass approach was used instead of lumped masses, utilizing the program features for section design and distributed mass calculation (it is noted that the total mass is equal to 1900.4t). The section stiffness was reduced as per the E39/99 guidelines. In particular, for the prestressed members that are designed to remain elastic during the seismic event (i.e. the deck), the uncracked stiffness is used, whereas for the piers, wherein development of plastic hinging is expected under the design earthquake, the secant stiffness at yield is adopted, based on the maximum expected axial load. For the two piers M1 and M2, the reduced stiffness EI_{eff}/EI_g =0.29÷0.37 was calculated using two alternative fibre element, cross-section analysis programs, FAGUS [20] and RCCOLA [21].

In order to investigate the effect of modelling assumptions on the bridge response, four different finite element models were developed: in Model 1, the middle pier stiffness is taken uncracked and the pier supports are considered as completely fixed (apparently corresponding to the maximum stress and minimum displacements case). In Model 2, the pier stiffness is reduced according to the lower bound coefficient (0.29) described above, whereas Model 3 additionally accounts for the (linear elastic) pier foundation-soil system compliance. Finally, Model 4 is used to investigate the effect of adopting the upper bound of stiffness reduction coefficient (0.37), plus the pier foundation flexibility. The static pile-soil interaction is accounted by attaching linear (at this stage) Winkler springs along the pile length. The corresponding stiffness is derived from the first branch stiffness of the P-y curve (i.e. lateral soil resistance vs. deflection relationship) proposed by Matlock [22] after appropriate linearization. It is also noted that Model 3 was used for the seismic design of the middle piers, while seismic displacements of the deck, the (subsequent) design of the seismic joints and the design of the bearings at the abutments, are based on the analysis of Model 4 (Figure 3).

3.2 Modal and Response Spectrum Analysis of the system

The dynamic characteristics of the four finite element models studied in this parametric study are summarized in Table 1. It is notable that although pier cracking and foundation flexibility lead to an increase of the fundamental period (in the longitudinal direction) by a factor of almost 2, the same conditions do not affect considerably the vibration along the transverse direction (mode 2). In order to study the particular difference in more detail, a Response Spectrum analysis is performed using the design spectrum of the Greek Seismic code. It is observed that, as anticipated, the deck displacements increase from Model 1 to the more flexible Model 4, but to a significantly different degree when assessed along the longitudinal or the transverse axis. In particular, in the longitudinal direction the displacements of the deck are 8.8cm for Model 4 compared to 2.6cm for Model 1 (i.e. increase of 237%) whereas, in the transverse direction, the corresponding displacements increase from 2.7cm to only 3.3 (i.e. 19% increase).

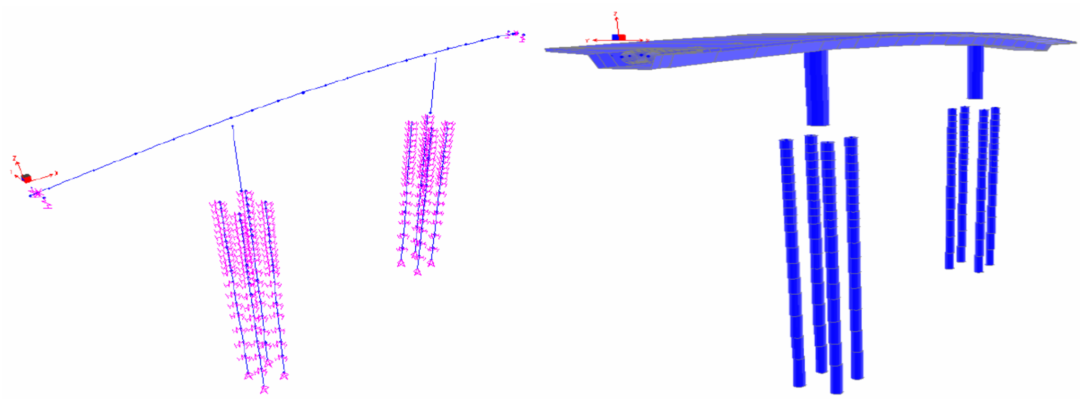

Figure 3: Finite element discretization of Model 4 of the bridge

FE Model ID#	Fundamental Period T (sec)	Ratio T_{Modeli}/T_{Model1}	Fundamental Mode shape	modal participation factor
1	0.512	1.00	Translational (longitudinal)	98.9% along x-x
2	0.708	1.38	Translational (longitudinal)	99.7% along x-x
3	0.959	1.87	Translational (longitudinal)	98.9% along x-x
4	1.012	1.97	Translational (longitudinal)	98.8% along x-x
FE Model ID#	**2^{nd} mode of vibration Period (sec)**		**2^{nd} Mode shape**	**modal participation factor**
1	0.463	1.00	Translational (transverse)	81.7% along y-y
2	0.496	1.07	Translational (transverse)	81.7% along y-y
3	0.501	1.08	Translational (transverse)	80.1% along y-y
4	0.505	1.09	Translational (transverse)	80.0% along y-y

Table 1: Dynamic characteristics of the bridge for the 4 alternative FE models

From the preliminary (modal and response spectrum) analysis results therefore, it is concluded that there is a strong indication that the participation of the middle piers in resisting seismic forces, is significantly lower for the case of seismic excitation along the transverse direction. In other words, the system stiffness in the transverse direction is mainly controlled by the abutment stiffness and this explains the reason why the parametric modification of the middle piers stiffness (both in terms of section cracking and their foundation compliance) has such a small effect on the overall system flexibility.

As a result it was deemed necessary and interesting, before proceeding to the more refined non-linear static analysis of the bridge and identify the relative effect of the various (material and geometrical) non-linear mechanisms, to study in more depth the linear and non-linear behaviour of the foundation-abutment-backfill soil system, as described in the following section.

3.2 Non-linear Static analysis of the abutment-backfill system

3.2.1 Soil non-linear behaviour

The most commonly adopted engineering method for calculating the non-linear pseudo-static interaction between piles and the surrounding soil is bi-linear Winkler models, in which the soil reaction to pile movement is represented by independent unidirectional translational spring elements distributed along the pile shaft to account for the soil response in the elastic and inelastic range. Along these lines, the force-dependent stiffness of the foundation-soil system was employed by using the P-y curves proposed in [22]:

$$\frac{p}{p_u} = 0.5\left(\frac{y}{y_{50}}\right)^n \tag{1}$$

where p is the soil resistance, y is the deflection corresponding to p, P_u is the ultimate soil resistance defined according to standard relations given by API [23], n is a constant relating soil resistance to pier-pile deflection and y_{50} is the corrected deflection at one-half the ultimate soil reaction determined from laboratory tests. The above (obviously, depth-dependent) relationship is introduced into the Finite Element model as a tri-linear force displacement curve, with the assumption that y_u is taken as 15 times the y_{50} which in turn equals to 0.05 for the particular soft soil. It is noted that the first branch stiffness of the P-y curve adopted is of paramount importance for both the modal and response spectrum analysis presented above, but also for the non-linear static (pushover) analysis conducted herein since it essentially controls the loading pattern.

3.2.2 Abutment stiffness

Various approaches for modelling abutments have been proposed in the literature (a summary of which can be found in [24]); however, abutment compliance is only rarely accounted for in developing the pushover curves of bridges. As mentioned previously therefore, a more refined modelling of the abutment-induced stiffness was deemed necessary in order to obtain a realistic estimate of the non-linear response of the bridge as a whole and especially of the base shear distribution among the structural elements. For this purpose, the foundation-abutment-backfill soil was studied separately and the appropriate pushover curves (i.e. seismic force vs. monitoring point displacement) were derived for the case of excitation along the two principal axes of the abutment (apparently coinciding with those of the bridge).

The transverse seismic force is assumed at the level of the stopper, while in the longitudinal direction, the displacement is taken at the top of the wall. It is noted that the abutment is indeed expected to resist seismic forces in the case of strong excitations that result to the joint closure. The abutment wall is modelled with 2D shell elements, while the piles with linear frame elements supported on the (depth-dependent) non-linear springs as described above and is depicted in Figure 4. For investigation purposes, the analysis is performed both for the (actual) soft soil conditions and for the case of a significantly stiffer supporting soil. The resulting four alternative finite element models of the abutment-foundation-soil system involve: a1) an abutment supported on soft soil conditions, where friction springs are used along the piles together and the appropriate vertical stiffness is introduced with the use of a (compression-only) spring at the tip of the piles, and a2) a similar FE system of practically infinite vertical stiffness (the tip displacements are restrained). Two additional models are also examined, with the same boundary conditions as a1 and a2 but for the case of stiff soil (models b1 and b2).

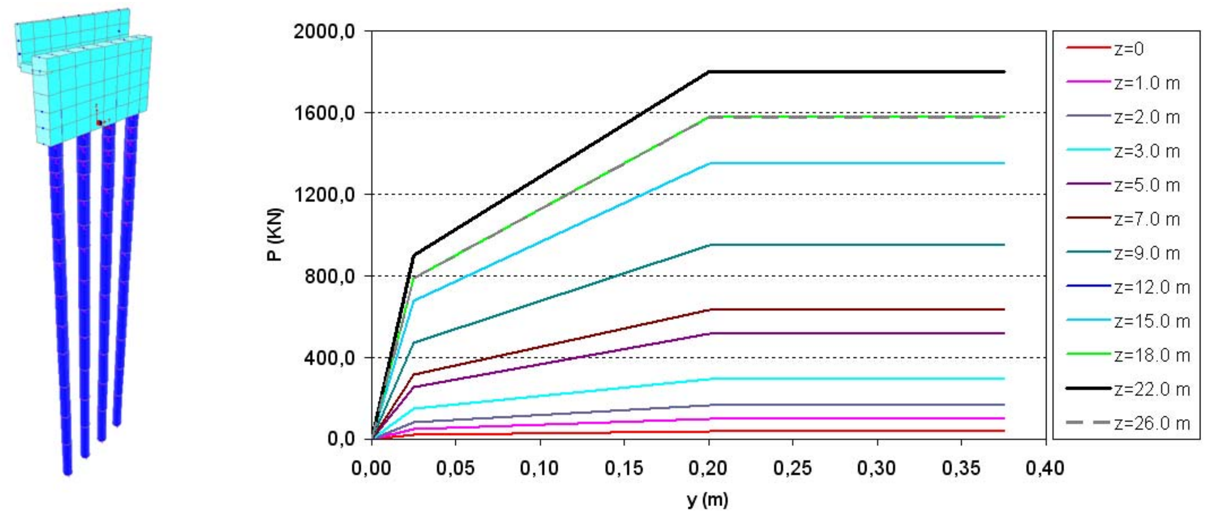

Figure 4: Depth-dependent multi-linear load-deflection curves used along the abutment piles to account for soil compliance (values for models a1 and a2 – soft soil conditions)

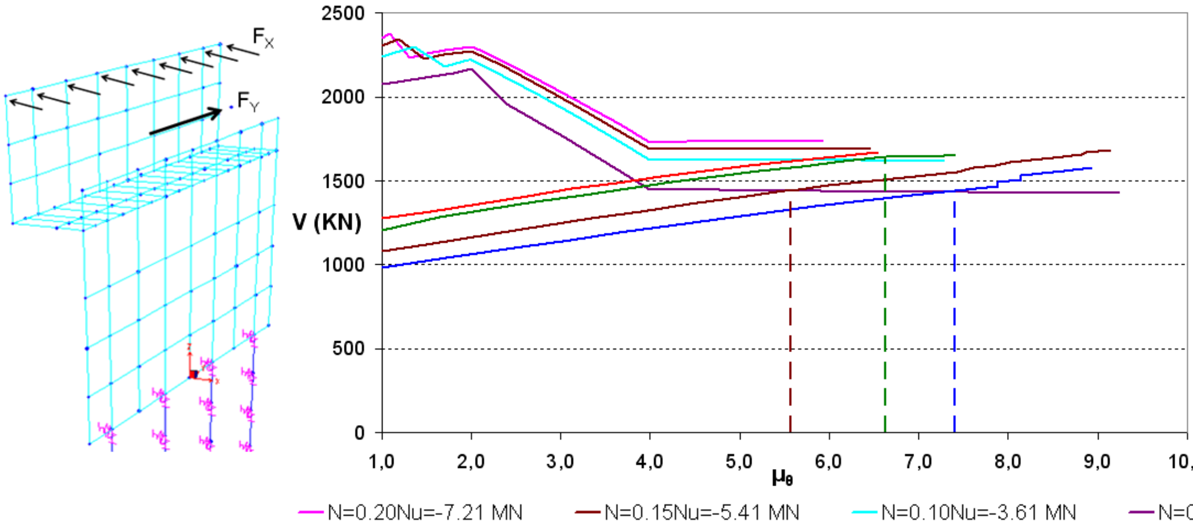

Figure 5: FE Modelling (left) and Shear Strength vs. Rotational Ductility (V_n-μ_θ) versus Shear Force vs. Rotational Ductility (V_{sd}-μ_θ) diagram (right) for soft soil conditions

Apart from soil flexibility, pile non-linearity is also considered for all models in terms of potential plastic hinge development and shear failure. The corresponding Shear strength vs. Rotational Ductility (V_n-μ_θ) and Shear Force vs. Rotational Ductility (V_{sd}-μ_θ) diagrams are illustrated in Figure 5 where it is seen that for the case of soft soil conditions, three piles fail in shear for large values of plastic rotation. Accordingly, four piles suffer shear failure right after their yield moment is exceeded. The fact that the foundation piles are able to resist the imposed bending moments and shear forces up to their flexural and shear capacity level affects the overall system stiffness. The resulting final pushover curve in the transverse direction of the complete abutment-foundation-soil system is illustrated in Figure 6. As anticipated, for the case of soft soil conditions, the overall system stiffness is significantly lower. The pushover curve derived is used to control the transverse abutment stiffness of the final (most refined) finite element model of the complete bridge.

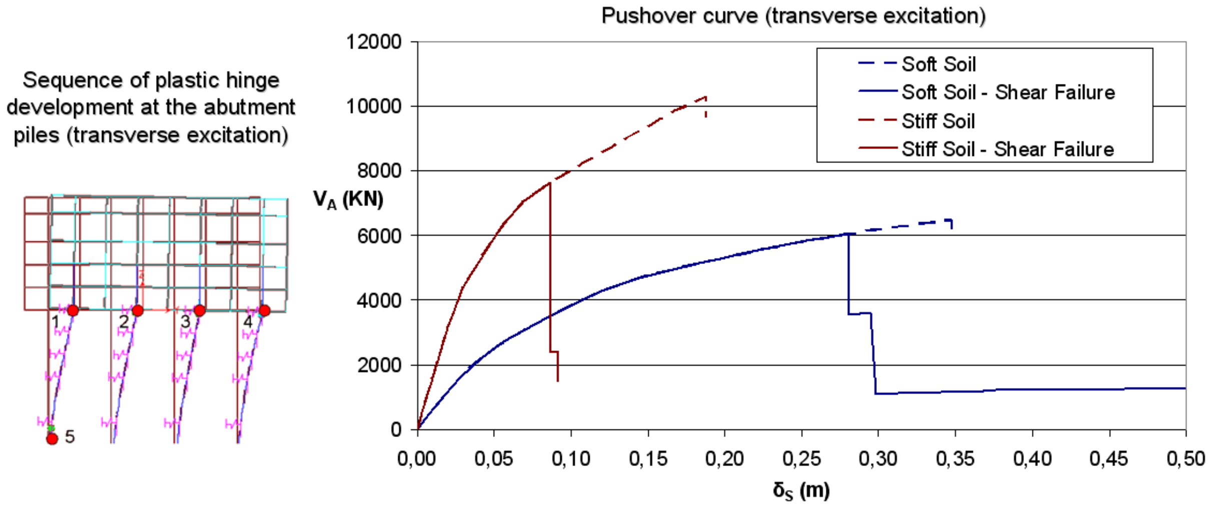

Figure 6: Sequence of plastic hinge development (left) and non-linear response (in terms of pushover curve) of the abutment-foundation-soil system (right)

It is also noted that in the longitudinal direction, the flexibility of the backfill (attributed to the presence of reinforced soil at the back of the abutment as seen in Figure 2) is also considered through an additional spring which, apparently, is independent of the overall foundation soil conditions. As a result, the stiffness of the backfill-abutment-foundation-soil system in the longitudinal direction is derived as a series of two systems (i.e. backfill and abutment-foundation-soil), hence, the non-linear force-displacement curve adopted for the bridge edge supports is essentially the summation of the two discrete component stiffness. The assessment of the entire bridge under the prescribed seismic loads is discussed below.

4 ASSESSMENT OF THE BRIDGE PERFORMANCE

4.1 Longitudinal direction

Having modelled all sources of material non-linearity (i.e. the effect of the backfill compliance, foundation soil yielding, pier and pile plastic hinge development and pile head failure in shear), as well as geometric non-linearity (i.e. gap closure, stopper activation and bearing sliding) two non-linear static analyses are performed along the two principal axes of the bridge. The pushover curve of the overall system for the longitudinal direction is illustrated in Figure 7. It is observed that in the case of soft soil conditions (as is also the actual case), three plastic hinges develop (plastic rotations correspond to 2÷10% of the ultimate rotation θ_{pu}) at the middle piers M1 and M2 before the system reaches the critical displacement δ=12cm where the joint closes. At a displacement δ=14cm the backfill soil yields and the system stiffness is considerably reduced until the ultimate displacement δ=22cm, where the bridge abutments are considered unstable due to unrecoverable damage as the reinforced soil (which is not laterally restrained by wing walls) cannot be supported any more. At this ultimate stage, four plastic hinges have developed at the two piers, exhausting the 35% and 49%, respectively, of the available plastic rotations.

The same picture is observed for the case of stiff soil. Clearly, the difference of the overall bridge system stiffness is evident only for displacements that do not exceed δ=12cm, that is, prior to the gap closure. For larger displacements the stiffness is controlled by the abutment and backfill behaviour and is very similar to the previous case.

4.2 Transverse direction

It is interesting to notice in Figure 8 that in the transverse direction, the non-linear mechanism is indeed different, compared to the longitudinal excitation case. In particular, an abrupt stiffness reduction is observed for displacement equal to δ=31cm for the case of soft soil and δ=10cm for the case of stiff soil, due to premature shear failure at the head of the abutment piles. At this stage, plastic hinges (with plastic rotation equal to 40÷42% of the ultimate plastic rotation θ_{pu}) have developed at the base of piers M1 and M2, but only for the case of soft foundation soil conditions. Once the abutment contribution to the system stiffness is eliminated due to damage to the piles supporting it, seismic forces are resisted mainly by the middle piers until bridge failure. It is notable that the particular design concept is not very commonly adopted, as the middle piers are typically designed to primarily attract earthquake loading. Therefore, it is necessary to evaluate the performance of the structure in the light of the (code-prescribed) anticipated level of seismic forces.

4.3 Overall assessment

The target displacements of the bridge under study for the two directions, the two alternative soil conditions and the two earthquake levels (i.e. design earthquake and twice the design earthquake) are also depicted in Figures 7 and 8 (the complete calculation process can be found elsewhere [25]). It is observed that at least for the design earthquake, the bridge performance is very good, as no major damage is expected for both directions, independently of the foundation soil type. This fact can be attributed up to a certain degree to the material overstrength, but most importantly, to the rather conservative design approach adopted (i.e. gross-section stiffness was assumed for the middle piers).

For twice the design earthquake, in the longitudinal direction, the joint is expected to close. Consequently, the overall bridge system stiffness in the longitudinal direction is significantly increased due to the activation of the backfill-abutment-foundation-soil subsystem. In the transverse direction, on the other hand, although damage is indeed minor for the case of soft foundation soil even for displacements corresponding to twice the level of the design earthquake, the abutment piles are expected to suffer significant damage due to shear failure at their head when the supporting soil is stiff. This situation is apparently detrimental because the abutments can no longer resist even their own earth pressures, hence the bridge stability is jeopardized and the high ductility of the middle piers is never utilized.

It has to be noted herein, that the system displacement that corresponds to twice the design earthquake intensity is obviously different from the double of the design displacement, and its definition is not straightforward for the particular bridge since the estimated pushover curves are not bi-linear. Therefore, a more accurate estimation of the bridge response under forces significantly higher than tose associated with the design earthquake intensity, should involve inelastic dynamic (time-history) analyses. Such analyses could also verify whether the *dynamic* soil-foundation-abutment-superstructure interaction, compared to the purely static approach adopted herein, essentially modifies the observed plastic hinge sequence and failure mechanisms.

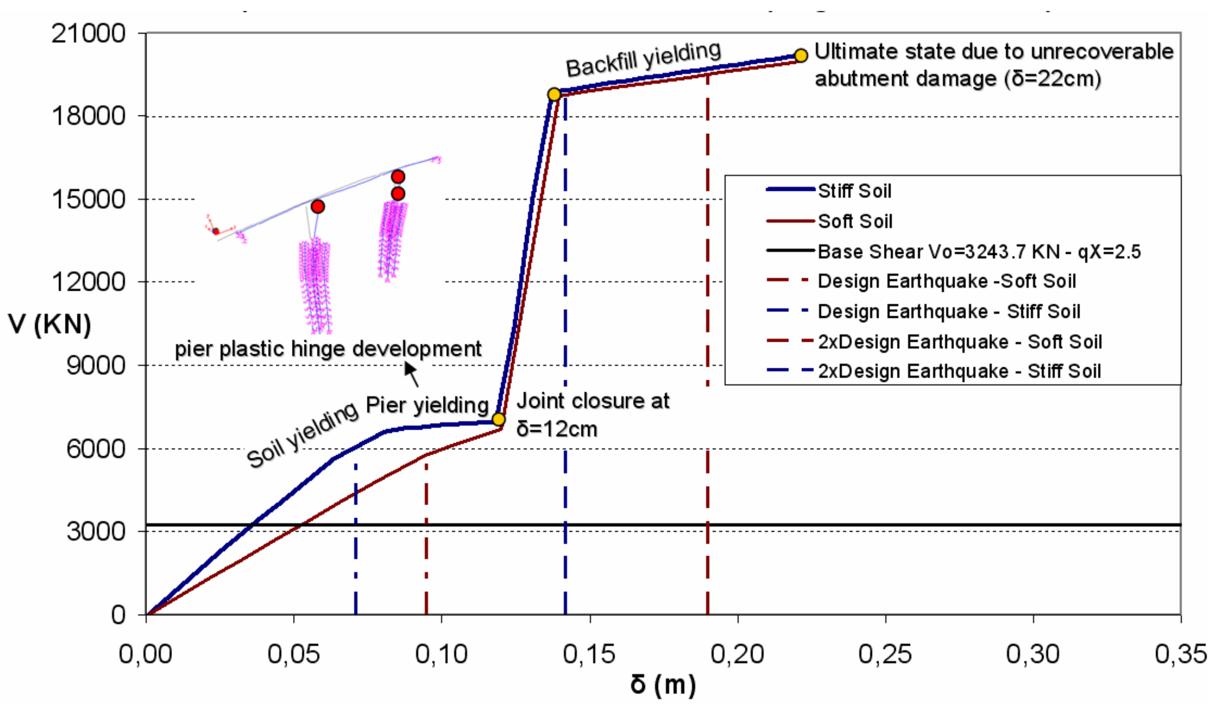

Figure 7: Pushover curve and seismic assessment of the overall system (longitudinal direction)

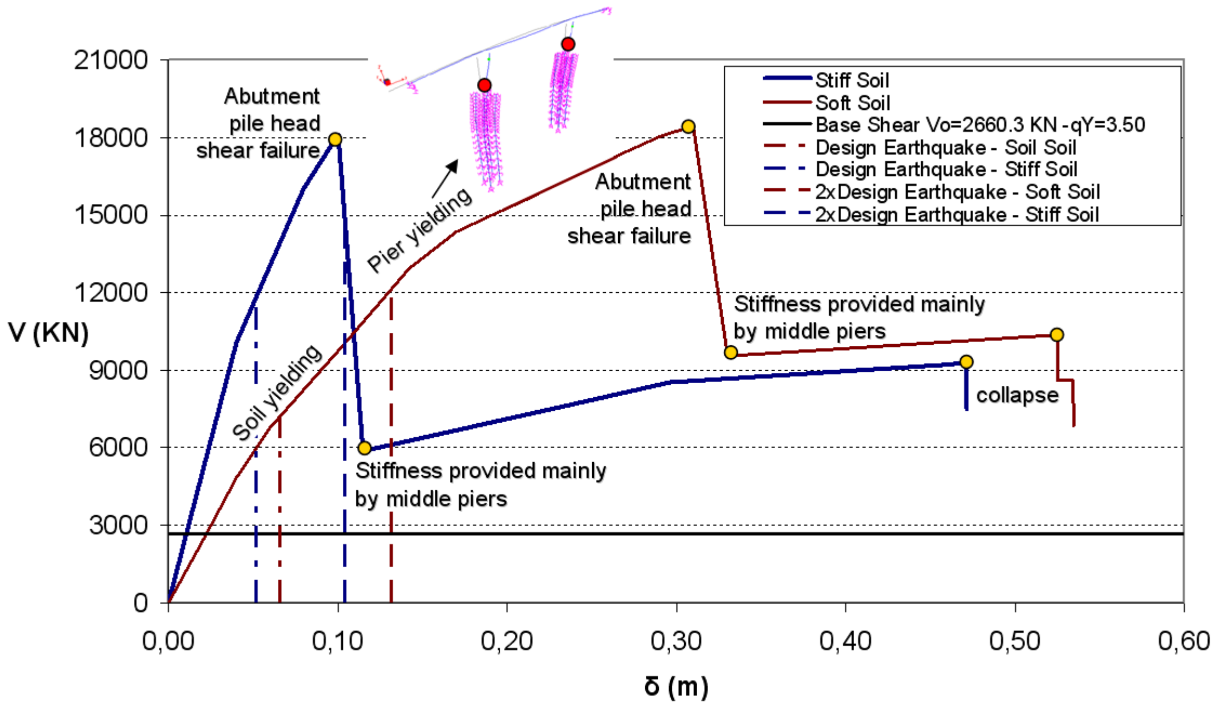

Figure 8: Pushover curve and seismic assessment of the overall system (transverse direction)

5. CONCLUSIONS

This paper deals with the analysis, design and assessment of an overpass bridge along the Egnatia Highway in Greece. The performance of the structure is assessed through non-linear static (pushover) analysis accounting for various sources of material non-linearity (backfill compliance, foundation soil yielding, pier and pile plastic hinge development, and pile failure in shear), as well as geometric non-linearity (i.e. gap closure, stopper activation and bearing sliding). Then, its inelastic behaviour under seismic loads is examined and finally a seismic assessment of the bridge is performed.

The results show a very good behaviour of the bridge for the design earthquake. For twice the displacement of the design earthquake, closure of the end joint is expected and the backfill-abutment-foundation-soil system is activated, a fact that drastically increases the resistance of the entire structure, while the safety factor against collapse remains high. It is only in the transverse direction, and for the case of stiff soil, that premature shear failure at the head of the abutment foundation piles is expected to raise stability issues as the abutments are no longer able to withstand the permanent backfill pressure.

Based on the aforementioned observations, it is concluded that the abutment stiffness can be a critical parameter in certain cases (such as the bridge studied), hence refined modelling approaches are necessary in order to reveal the potential mechanisms triggered. It is also notable that the shape of the pushover curve calculated when gap closure is modelled in bridge analysis is different from the familiar bilinear one. Further investigation, especially towards the identification of the dynamic soil-abutment-embankment interaction effect is deemed necessary.

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