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SEISMIC ASSESSMENT OF AN OVERPASS BRIDGE WITH A TORSIONAL FUNDAMENTAL MODE

Andreas J. Kappos[1*], Eleftheria D. Goutzika[2], Anastasios G. Sextos[3]

[1]Professor, Laboratory of Concrete and Masonry Structures,
Department of Civil Engineering, Aristotle University of Thessaloniki, 54124 Greece
ajkap@civil.auth.gr

[2]Graduate Student, Laboratory of Concrete and Masonry Structures,
Department of Civil Engineering, Aristotle University of Thessaloniki, 54124 Greece
elia2@civil.auth.gr

[3]Assist.Prof., Division of Structural Engineering,
Department of Civil Engineering, Aristotle University of Thessaloniki, 54124 Greece
asextos@civil.auth.gr

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Abstract. *The study focuses on the seismic response of a straight, overpass bridge with two equal spans, whose fundamental mode, unlike most bridges of this type, is purely torsional. The particular structure is supported on a two-column pier of cylindrical cross-section that is monolithically connected to the deck, while it rests on its two abutments through elastomeric bearings. The seismic assessment of the bridge was performed in the longitudinal and transverse directions using non-linear static analysis. An additional pushover analysis was carried out using the fundamental (torsional) mode loading. Parametric analyses were carried out involving consideration of foundation compliance, and various scenarios of accidental eccentricity that would trigger the torsional mode. The pushover curves derived in the longitudinal and transverse directions highlight the satisfactory performance of the bridge, even for motions twice as strong as the design earthquake. On the other hand, if one focuses on the fundamental torsional mode, the corresponding load pattern is antisymmetric and the resulting base shear zero, hence a 'standard' pushover curve cannot be drawn; an alternative pushover curve in terms of abutment shear vs. deck maximum displacement (that occurs at the abutment) is found to be a meaningful measure of the overall inelastic response of the bridge. Furthermore, assessment based on the torsional fundamental mode leads to failure of the elastomeric bearings, but the related displacements are irrelevant since they are about an order of magnitude higher than the design displacement. Clearly, proper assessment of the bridge should be based on the longitudinal and transverse loading distributions, wherein the critical pier section failure precedes the bearing failure.*

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, the development of analytical methods that would permit the quantification of the degree of global and local ductility has increasingly attracted the attention of both researches and designers. Along these lines, nonlinear static (pushover) analysis has become a popular tool for the seismic assessment of buildings [1] and bridges [2,3], 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 [3,4].

The aforementioned non-linearity expected in bridges during strong ground motions, cannot be attributed solely to yielding of reinforced concrete sections, although these are the elements that are often purposely designed to exhibit inelastic behaviour. Additionally, elastomeric bearings are designed to resist seismic displacements, even for motions stronger than the design earthquake. The response of bearings is explored with the use of linear and nonlinear force-displacement relationships, whereas boundary conditions at the abutments are modelled with the assumption of free-sliding or spring supports.

The objective of this paper is to focus on a real, already built, bridge structure with a torsional fundamental mode that has been designed to resist seismic forces through both capacity-designed elements and elastomeric bearings. The seismic performance and the torsional response of the bridge are examined using alternative modelling approaches for bearings in order to investigate the range of applicability of the standard pushover analysis to structures with a torsional fundamental mode, which, nevertheless, does not dominate the response. A brief description of the structure assessed is presented first, followed by the analysis method and the critical evaluation of the results.

2 OVERVIEW OF THE BRIDGE STUDIED

The bridge selected is a straight overpass (overcrossing) with two equal spans (instead of the commonly used three-span configuration) and total length equal to 31.5m (Fig. 1). The deck consists of a 7.7m wide reinforced concrete slab and is supported on a two-column pier of cylindrical cross-section of 1m diameter and 5m height (Figure 2). The pier is resting on a surface foundation with a stiff upper part (1.5×5×3m). The deck is monolithically connected to the twin-column pier, while it rests on its two abutments through elastomeric bearings. Displacement of the deck in both the longitudinal and transverse directions is free at the abutments, due to the lack of deck restraint in either direction. It is noted, however, that parametric analysis considering a transverse restraint at the abutment is also performed. The seat-type abutments rest also on surface foundations (footings).

The bridge was designed according to the provisions of the Greek Seismic Code (EAK-2000) [5] and the code (Circular E39/99) for seismic design of bridges [6], for seismic actions prescribed for zone I, i.e. a peak ground acceleration a_g=0.16g. The behaviour factor of the structure was estimated separately for its longitudinal and transverse directions (as prescribed by the Code, due to the different shear ratio of the pier in each direction) and was taken equal to 2.1 and 2.5, respectively, the design concept being that ductile behaviour was expected, with formation of plastic hinges in the pier columns (that were properly detailed for ductility).

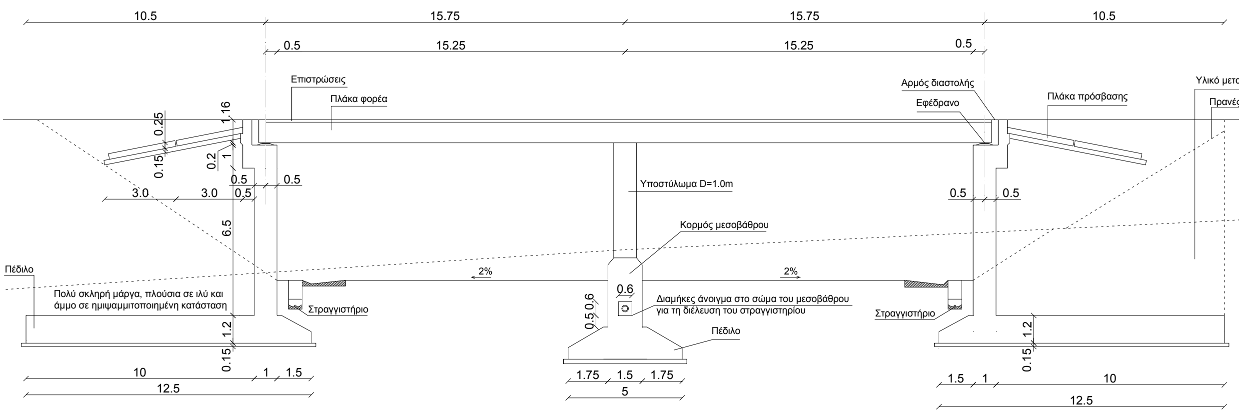

Figure 1: General view of the overpass

The concrete class used was C20/25 (characteristic compressive cylinder strength f_{ck}=20MPa) while S500 steel (characteristic yield strength f_{yk}=500MPa) reinforcement was used throughout the structure. No prestressing reinforcement was used in the deck (due to the relatively short spans).

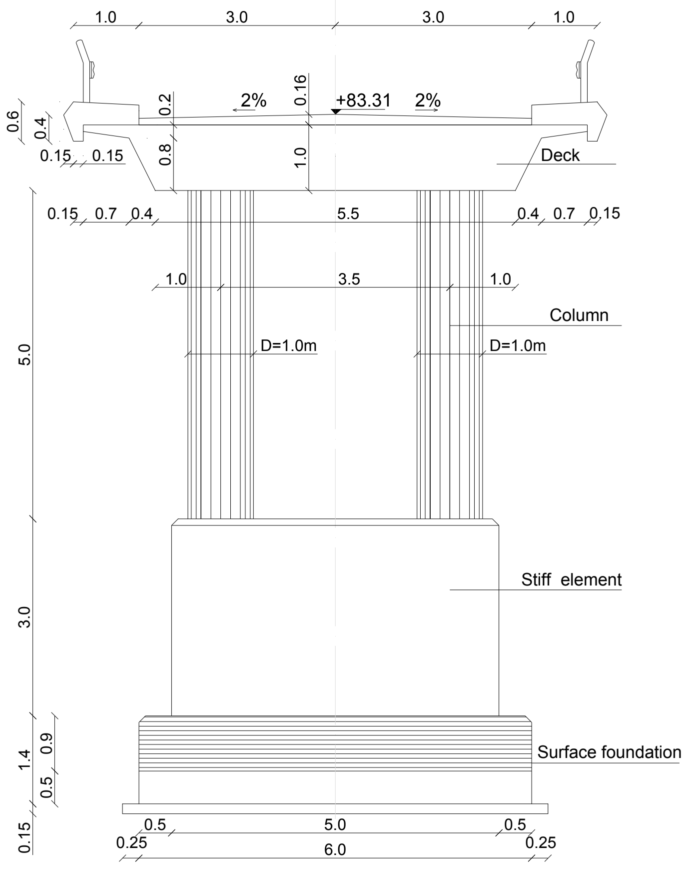

Figure 2: Overview of the foundation-pier-superstructure system cross-section.

3 FINITE ELEMENT MODELLING AND ANALYSIS OF THE STRUCTURE

3.1 Modelling aspects

The bridge was modelled using the FEM software package SAP2000 [7]. Three alternative models were developed, wherein the deck was modelled as (a) a 3D beam (stick model), (b) a grillage of 3D beams with appropriately defined stiffness, and (c) using shell elements. Models b and c were mainly intended for verifying the simpler stick model that was subsequently used for the seismic analysis and the assessment of the bridge; the specific aim was to investigate the reliability of the assumption of using a (simple) 3D beam section for the particular deck. The grillage consisting of equivalent transverse and longitudinal beams for the deck, i.e. Model b (Figure 3) was also useful for comparing the results of the present analysis with those of the original study of the bridge (carried out by a Thessaloniki engineering design firm). In both models, the monolithic connection of the deck to the pier and the pier to the stiff base was achieved using rigid elements. The detailed model using shell elements for the deck and the stiff base (Figure 4) is deemed to be able to achieve a more realistic internal force transfer between the deck, the pier and the foundation.

The stiffness of the pier members was reduced as per the E39/99 guidelines (similar to those of EC8-2 [8]). More specifically, for the members that are designed to remain elastic during the seismic event (i.e. the deck), the uncracked stiffness was used, whereas for the pier, wherein development of plastic hinging is anticipated under the design earthquake, the secant stiffness at yield equal to $EI_{eff}/EI_g=0.36$ was adopted, based on the maximum expected axial load. In order to investigate the effect of the interaction between the bridge and the supporting ground, different support conditions were investigated: (i) by assuming full fixity at the base of the pier and the abutments (Model 1); (ii) by implementing linear springs, whose static (i.e. based on the foreseen type of analysis) stiffness was evaluated using appropriate expressions for shallow foundations from the literature [9] (Model 2); (iii) by introducing transverse restraint between the deck and the abutments as a means for investigating the effect of lateral restraint (Model 3), and (iv) by directly connecting the pier and the footing without the stiff vertical element (Model 4). It is also noted that the soil springs used in Models 2 to 4 were derived for two distinct cases of soft and stiff soil. In models 3 and 4, the elastomeric bearings between the deck and the abutment were explicitly modelled using springs, the shear, axial and bending stiffness of which were estimated according to established practice [10]. In all cases, the shear modulus of soil was also reduced according to the design excitation level and the provisions of Eurocode 8 [8].

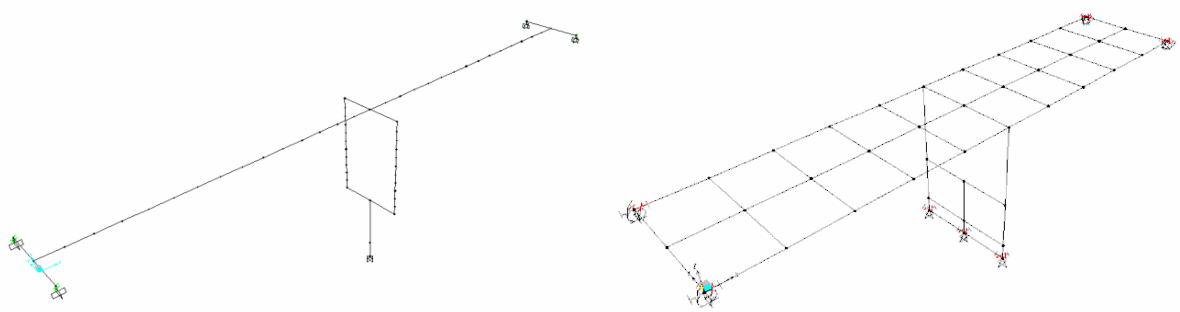

Figure 3: Stick model (a) and grillage model (b) of the bridge.

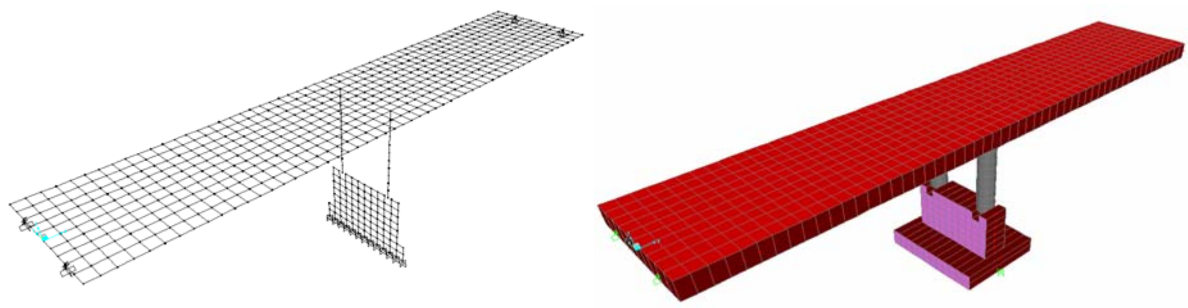

Figure 4: Finite element model (c) of the bridge using shell elements for the deck and the base

3.2 Modal and Response Spectrum Analysis of the Bridge

The dynamic characteristics calculated by the analyses of the stick and the shell element models, are found to be in acceptable agreement as seen in Table 1, where the natural periods and the relative participation factors are presented. The minor discrepancies can be attributed to the different modelling approach since the shell element model consists of more degrees of freedom compared to the stick model, thus leading to a natural period in the transverse direction (T=0.61sec) that is 8.2% higher. The differences are generally minor and in view of this, the 3D-beam deck element stick model is deemed quite reliable in terms of the (elastic) dynamic characteristics of the structure and the associated seismic force distribution. This is not the case for the grillage model, since its natural periods are significantly different (the periods computed were found equal to T_{rz}=1.47sec, T_x=1.04sec and T_y=0.46sec in contrast to T_{rz}=1.04sec, T_x=0.60sec and T_y=0.56sec derived for the reference stick model).

	Stick model		Shell element model	
Mode	**Period T (sec)**	**Modal Participation factor**	**Period T (sec)**	**Modal Participation factor**
1st (torsional)	1.04	23.2%	1.12	23.9%
2nd (longitudinal)	0.60	91.7%	0.61	91.7%
3rd (transverse)	0.56	92.2%	0.61	92.3%

Table 1: Natural Periods (sec) and modal participation factors (%) of the stick and the shell model assuming full fixity at the base of the pier.

	1st Model		2nd Model	
Mode	**Period T (sec)**	**Modal Participation factor**	**Period T (sec)**	**Modal Participation factor**
1st (torsional)	1.04	23.2%	1.06	23.2%
2nd (longitudinal)	0.60	91.7%	0.83	95.1%
3rd (transverse)	0.56	92.2%	0.83	94.6%

Table 2: Natural periods (sec) and modal participation factors (%) of stick models 1 and 2.

When the static effect of soil-structure interaction is taken into consideration, the flexibility of the structure is, as anticipated, increased (Table 2) and the natural periods of Model 2 are longer. It is noticed that the torsional mode (T_{rz}=1.04sec) was not affected by the boundary conditions of the pier, as it is dependent solely on the sensitivity to torsion of the deck. When transverse restraints are introduced at the abutments, while retaining the consideration of the soil compliance below the pier (Model 3), the longitudinal mode was not affected (T_x=0.834sec) compared to Model 2; on the contrary, both the transverse and the torsional natural periods were substantially reduced (by 83% and 96% respectively) due to the lateral restraint of the deck ends, a fact that highlights the importance of lateral restraints with regard to the overall seismic performance of a bridge. In the case of Model 4, consideration of a longer (8m) pier supported directly on the foundation, a rather more common design approach, was found to lead to periods longer by 7% for the torsional mode, 23.5% for the longitudinal mode and 26.6% for the transverse mode. It has to be noted, that in this case, the order of modes is also reversed as the transverse mode of the bridge is affected more and hence it is eventually more flexible than the longitudinal one, a fact clearly attributed to the influence of the stiff base on lateral vibration.

	2nd Model		3rd Model		4th Model	
Mode	**Period T (sec)**	**Modal Participation factor**	**Period T (sec)**	**Modal Participation factor**	**Period T (sec)**	**Modal Participation factor**
Torsional	1.06	23.2%	0.05	20.1%	1.14	24.3%
Longitudinal	0.83	95.1%	0.83	95.1%	1.09	98.5%
Transverse	0.83	94.6%	0.14	83.6%	1.13	98.6%

Table 3: Natural periods (sec) and modal participation factors (%) of stick models 2, 3 and 4 accounting for alternative analysis assumptions and design decisions for the boundary conditions of the bridge.

To study the seismic response of the bridge, a Response Spectrum analysis was also performed using the design spectrum of the Greek Seismic Code. In Table 4, the seismic displacements of the deck are presented according to the four alternative models. It is observed that, as anticipated, deck displacements increase from Model 1 to the more flexible Model 4. It is also noticeable that especially in the case of Model 3, the deck mass centre was displaced by only 0.25cm due to the transverse restraint at the abutments. In Models 1, 2 and 4 the seismic displacements correspond to the lateral edge of the deck since the presence of bearing-type deck-to-abutment connections and the subsequent torsional sensitivity of the bridge lead to higher transverse displacements at the lateral supports compared to the deck centre. Due to both the lack of lateral restraints and the presence of a taller pier, Model 4 was found to lead to higher displacements, a fact that explains why the stiff base between the pier and the foundation footing was introduced in the actual design of the bridge.

Direction	1st Model	2nd Model	3rd Model	4th Model
Longitudinal (u_X, mm)	35.6	56.3	56.3	80.0
Transverse (u_Y, mm)	32.2	56.3	2.5	83.5

Table 4: Seismic displacements (in mm) of the deck in the longitudinal and transverse direction.

4 ASSESSMENT OF THE BRIDGE PERFORMANCE

Having examined and compared the three alternative models involving different finite element discretizations of the deck, the bridge was assessed using non-linear static (pushover) analysis of the preferred (mainly for reasons of simplicity) stick model.

4.1 Inelastic modelling and analysis aspects

Pushover analysis was carried out with the aid of the F.E. program SAP2000 [7]. The inelastic behaviour of the critical cross-sections of the pier was evaluated using the fibre analysis program RCCOLA [11]. The resulting interaction surfaces (N-M_x-M_y) and moment-rotation (M-θ) curves were then imported into the critical locations of the finite element model using the built-in plastic hinges of SAP. The assessment was performed for all the alternative models (1 to 4) wherein the parameters related to foundation compliance and bearing supports were described previously. Moreover, additional parametric analyses were performed for Model 2 in order to examine various scenarios of accidental eccentricity that was expected to trigger the torsional mode.

Another interesting point, related specifically to the seismic assessment of the bridge through non-linear static analysis procedures, is the selection of an appropriate monitoring point (with respect to which pushover curves for the structure would be drawn). In bridges, unlike the case of buildings, the shape of the pushover curve inherently depends on the location of the monitoring point. The displacement of this monitoring point is used not only as a parameter of the pushover curve, but also to establish the seismic demand on the structure. For the overcrossing studied here, the location of the mass centre of the structure (the recommended monitoring point in EC8-2 [8]) was used only for assessment along the longitudinal direction; on the contrary, it was the abutment that was adopted as the monitoring point for the structure in the case of transverse excitation, since this was found to be the point of maximum displacement in this direction. This selection of monitoring point is the appropriate choice for bridges with decks unrestrained in the transverse direction, as also found in other recent studies [2, 4].

The elastic spectrum of the Greek Code was used to define the target displacements along the two principal axes. The prevailing modes in the two directions having relatively long periods ($T_{x,o}$ and $T_{y,o}$>0.5sec), the equal displacement approximation was valid in this case and simplified the estimation of target displacements.

4.2 Foundation compliance

Pushover curves for the bridge in the longitudinal and transverse directions are illustrated in Figures 5 and 6. It is observed that in the case of flexible support conditions the initial stiffness of the system is reduced, as anticipated. For the same reason, the yield and ultimate displacements are increased, while the sequence of plastic hinge formation differs from the fixed-base model due to the different internal force redistribution. The target displacement of the structure, for the design earthquake, is increased up to 35% (in the case of soft soil conditions). The available ductility of the system was found lower in the case of soft soil conditions (q_{avail}=7.5 in the longitudinal and $q_{avail.}$=8.85 in the transverse direction, respectively) compared to the assumption of stiff soil which leads to a ductility factor of 12.3 for both the longitudinal and transverse direction. In all the above cases, the high available ductility manifests a desirable response of the bridge under earthquake loading.

The ductility demand for the design earthquake, corresponding to the case where the deck displacement equals the target displacement (calculated from the design spectrum), was also estimated. It was found equal to 1.7 and 1.0, for the case of stiff and soft soil conditions, respectively; this is a clear indication of the well-known overstrength of structures designed to modern codes (the ratio V_y/V_o of yield force to design base shear, see Figures 5 and 6). The different ductilities resulting from different assumptions regarding the foundation soil stiffness (when foundation compliance is accounted for in the analysis) show that, depending on this assumption, the bridge is predicted either to enter (slightly) into the inelastic range under the design earthquake loading, or to remain essentially elastic. It is worth pointing out that even for twice the design earthquake intensity, the plastic rotation demand at the pier critical locations, is well below the available supply ($\theta_{p,avail.}$=0.031rad). As a consequence, the performance of the bridge in the longitudinal direction is deemed as very satisfactory. The same can be observed for excitation along the transverse direction, as can be inferred from the data in Figure 6.

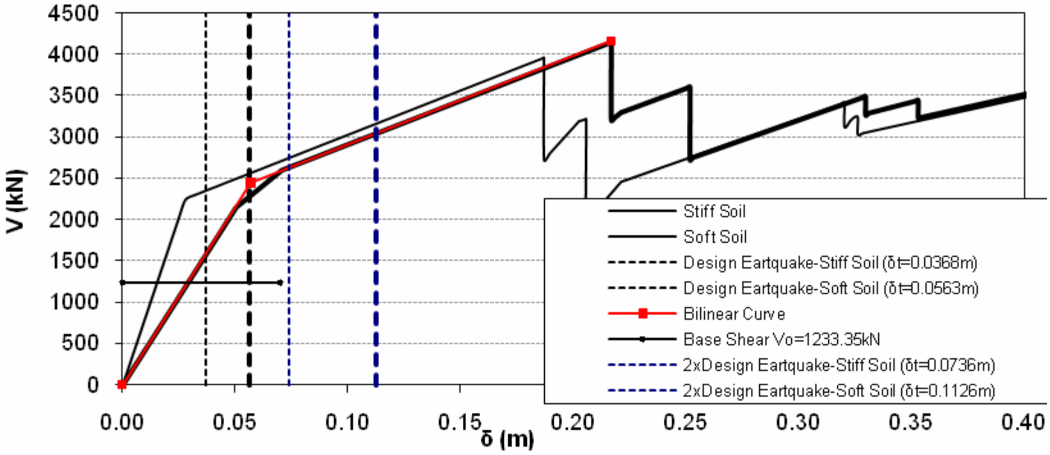

Figure 5: Pushover curve and seismic assessment of the bridge (longitudinal direction).

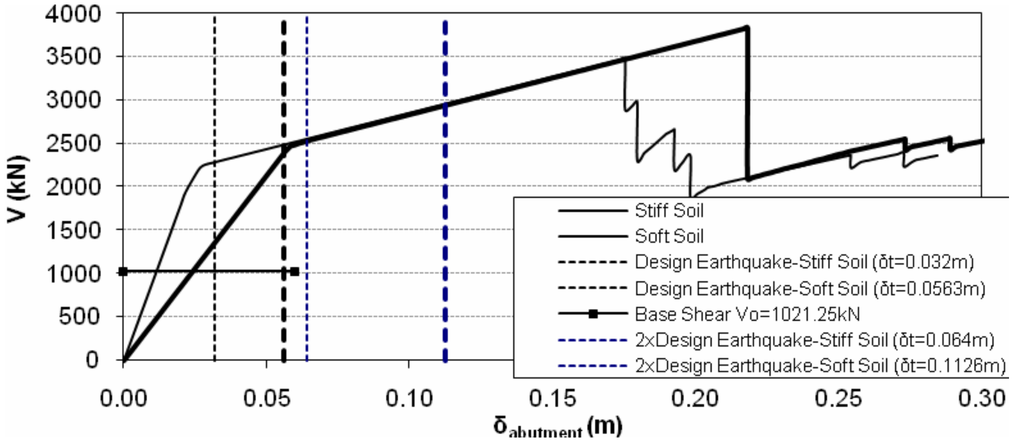

Figure 6: Pushover curve and seismic assessment of the bridge (transverse direction).

4.3 Modelling of bearings

An interesting observation from the pushover curves presented in Figures 5 and 6 is their rather unusual shape subsequent to pier failure; in particular, the strength of the system is found to increase thereafter. This apparently non-realistic situation can be simply attributed to the fact that as the bearings were essentially modelled with linear springs, failure of the pier leads to significant redistribution of internal forces, especially given the assumed linearly increasing strength and the lack of a failure mechanism for the bearings. As a result, the bearings tend to carry a higher percentage of the applied base shear that is approximately equal to 70% compared to the 30% that they carried prior to pier failure.

Clearly, such a continuous increase in strength is not realistic, nor does it lead to a conservative assessment of the bridge; of course, in a design context, one would not consider assessing the bridge for damage states beyond the failure of the pier (substantial drop in strength, as shown in Figures 5 and 6). Notwithstanding the feasibility of assessing such heavy damage states (which are, nevertheless relevant in vulnerability analysis), an effort was made to numerically simulate the non-linear force-displacement relationship of the elastomeric bearings, using information available in the literature and in the E39/99 guidelines. In the τ- γ models of Figure 7, it is seen that as the shear strain γ of the bearing increases from 1.0 to 2.0, its stiffness is increased, while for strains $\gamma>2$ a plastic condition is assumed up to the level of maximum strain $\gamma_{bu}=5$ (horizontal displacement $t_{el.}\cdot\gamma_{bu}=0.52$m in the particular case) that corresponds to complete failure of the bearing.

Based on the above, revised pushover curves for the transverse direction were drawn, shown in Figure 8 (bold lines). It is seen that the overall resistance of the structure is more rational and the residual strength of the structure remains, as anticipated, constant, subsequent to pier failure. Moreover, pier failure is observed at a displacement of 0.22m, a value that is equal to less than half the value (0.52m) that the bearings can sustain without failure. This observation implies that simply including elastomeric bearings in the analytical model (with some effective stiffness) might not be sufficient if a ‘full-range’ assessment of seismic performance is sought; in this case, the non-linear response and failure mechanism of the bearings should be properly considered.

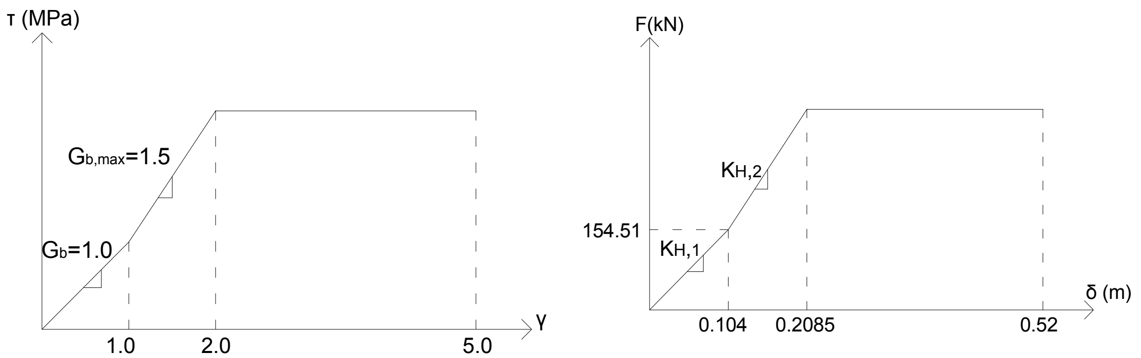

Figure 7: Nonlinear shear stress-strain constitutive law (left) and force-displacement relationship (right) for elastomeric bearings.

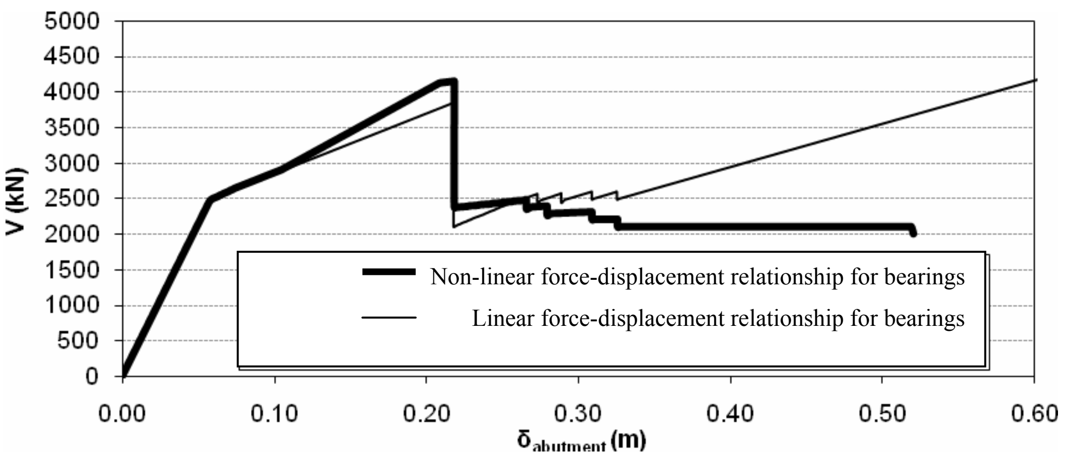

Figure 8: Pushover curve of the bridge in the transverse direction for the case of linear and nonlinear force-displacement relationship used for the elastomeric bearings.

4.4 Pushover analysis for torsional loading pattern

Another interesting issue is related to the loading pattern to be adopted for assessing the torsional mode contribution. It is noted that the modal load pattern of the torsional mode is antisymmetric (Fig. 9) and the resulting base shear is zero, hence it is simply not feasible to draw a standard pushover curve in terms of base shear versus monitoring point displacement. For such cases, use of an alternative pushover curve is proposed here (Figure 9), which is drawn in terms of abutment (rather than base) shear versus deck maximum displacement at the location of the abutment. It is noted that the target displacement estimated from the torsional load distribution (equal to δ_t=0.0174m) is substantially lower than those predicted for the translational modes, since the two-column pier connected to the rigid base behaves as a stiff frame, thus providing the bridge with a high torsional resistance, while due to the symmetry of the bridge there are no stiffness eccentricities. In the case of nonlinear modelling of the elastomeric bearings, failure of the bearings indeed precedes pier failure; nevertheless the displacement required to cause pier failure is an order of magnitude higher than the design displacement. On the other hand, the lack of deck restraint in the transverse direction leads to large displacements at the abutments (>0.52m) that cannot be sustained by the bearings.

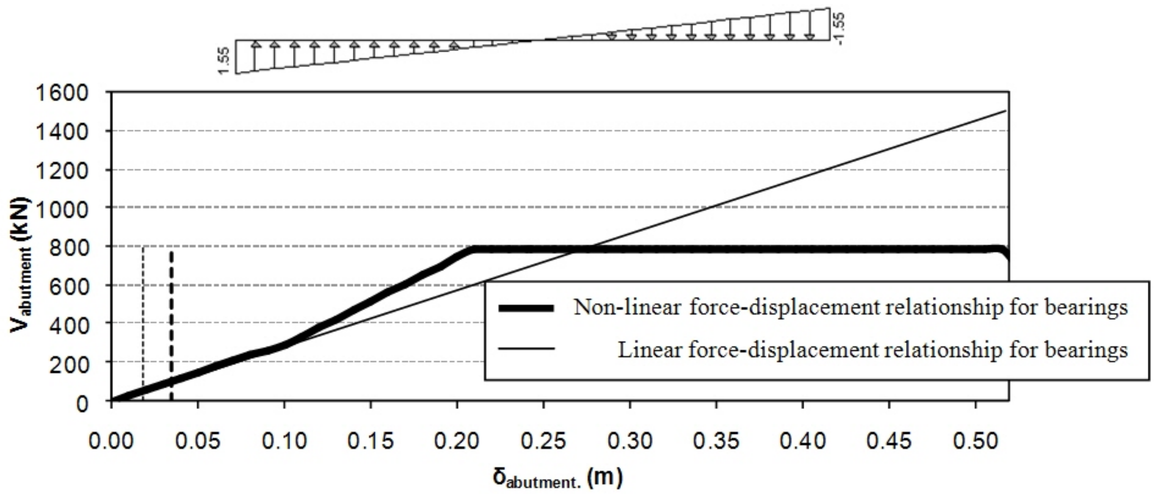

Figure 9: Abutment shear vs. deck maximum displacement according to the fundamental torsional mode.

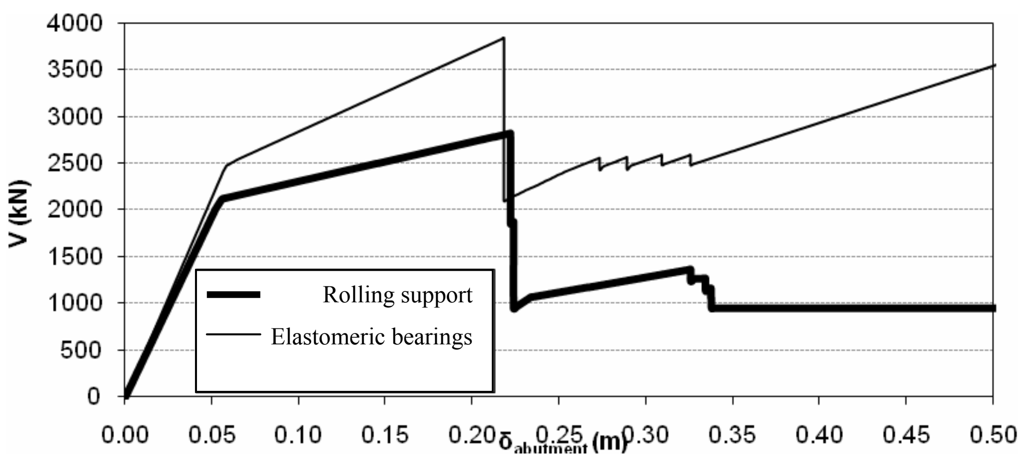

Figure 10: Pushover curve of the bridge (transverse direction) assuming roller supports and elastomeric bearings at the abutments.

An additional parametric analysis was performed with the assumption that the bridge deck is roller-supported at the abutments (e.g. use of pot bearings). In this case, the deck was considered as completely unrestrained in the longitudinal as well as the transverse direction, but the vertical displacement and the rotation were assumed restrained. Figure 10 illustrates the pushover curve of the bridge in the transverse direction where a reduction of approximately 30% of the overall strength of the bridge is observed.

4.5 Influence of the pier configuration

Having assessed the bridge for various soil conditions and different deck-to-abutment support assumptions, the bridge was subsequently assessed, by assuming that the stiff base through which the pier rests on its footing does not exist and the pier height is equal to 8m instead of 5m. As already mentioned, this 4th Model is identical to Model 2 (i.e. soil compliance is also accounted for) with the exception of the pier height. As seen in the pushover curve of the system in the transverse direction that is illustrated in Figure 11, the resistance of the system (base shear at first yield) was reduced by 30% (from 2500kN to 1800kN) while the system stiffness was also decreased by about 50% compared to the reference Model 2. The reduction of the maximum shear force was less significant (of the order of 10%) but maximum displacement was approximately 30% higher for the case of the 8m pier. On the other hand, the available ductility of the system was found equal to $q_{avail.}$=14.37 for the transverse direction, a rather high value that can be attributed to the relatively low design base shear (V_o=515.11kN) that in turn leads to a higher overstrength factor ($q_s=V_y/V_o$). The ductility of the particular system is high, but is accompanied by larger displacements which apparently were one of the reasons why the designer opted for providing the stiff base used for connecting the pier with the foundation (Fig. 2); as noted earlier, this also increased the resistance to torsion of the bridge.

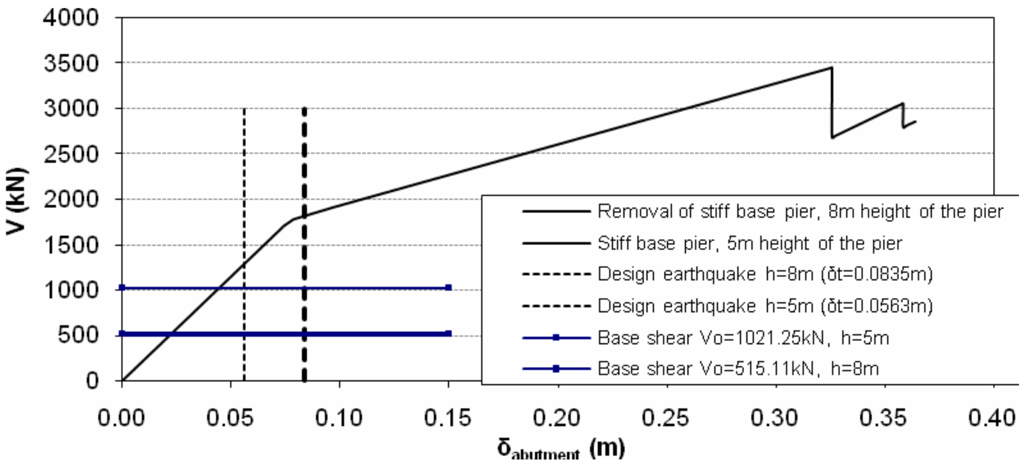

Figure 11: Pushover curve of the bridge (transverse direction) with and without the stiff base for the pier.

4.6 Torsional sensitivity

As already described in §3.2, the dynamic characteristics of the bridge are of particular interest since not only the fundamental mode is torsional (T_{rz}=1.04s), but its participation factor is only 23%, whereas the two translational modes are clearly dominant, with participation factors that exceed 95%. Therefore, in order to investigate the effect of this torsional sensitivity to the response of the bridge in the transverse direction, two scenarios of (additional) accidental eccentricity were also considered. Firstly, the bridge was directly subjected to a torsional moment $M_t=\pm F\cdot e$ (where accidental eccentricity e was taken equal to 3% of the overall bridge length) and a lateral force F following the distribution of the third mode (involving transverse displacements). Due to the application of the above torsional moment at the mass centre of the bridge, the deck exhibits an asymmetric deformation, hence the lateral displacements of the deck at the location of the abutments are different. As a second ‘scenario’, the accidental eccentricity was implicitly introduced by assuming a reduced shear modulus of one set of elastomeric bearings due to possible decay and/or accidental dislocation (which are not uncommon in practice). In both scenarios, the seismic displacements at the abutments consist of a translational (u_o) and a torsional (u_θ) component, as depicted in Figure 12.

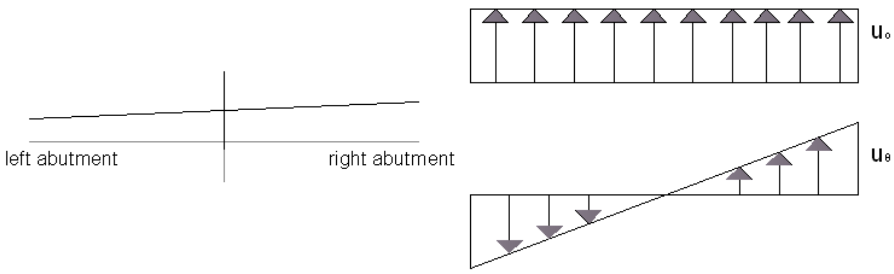

Figure 12: Deck deformation in the transverse direction and the corresponding displacement decomposition into a translational (u_o) and a torsional (u_θ) component.

Table 5 presents the transverse seismic displacements of the deck for the above two cases of accidental eccentricity. Reference Model 2 represents the case of the purely translational deformation of the deck due to the symmetric load pattern. Clearly, the accidental eccentricity, in both the above scenarios, increases the torsional component (u_θ); however, the translational component still dominates the overall response even in the case of roller supports.

	Reference Model 2	**Eccentricity due to F-M_t**	**Eccentricity due to ½ GA_{eff} at one bearing**	**Roller supports**	**Eccentricity due to F-M_t & roller supports**
u_o (mm)	56.3	59.3	57.9	59.3	7.12
u_θ (mm)	0.72	17.9	5.80	0.77	0.90

Table 5: Target displacement due to transverse excitation for the alternative models.

	Reference Model 2	**Eccentricity ½ GA_{eff} of one Elast. Bearing**	**Rolling Supports**
u_o (mm)	17.5	19.4	26.5

Table 6: Target displacements due to torsional excitation for the alternative models.

Finally, the target displacement of the deck due to the fundamental torsional mode is presented in Table 6. It is pointed out that a purely torsional excitation is not feasible; however, the seismic displacement due to a torsional load pattern was evaluated for research purposes (to get a deeper insight into the bridge’s dynamic response). It is observed that the maximum displacement occurs, as anticipated, in the model where roller supports were introduced between the deck and the abutments. This can be attributed to the fact that the sliding support of the deck at the location of the abutments significantly reduces the torsional resistance of the bridge. The reference Model 2, on the other hand, is associated with the smaller target displacement due to the relatively higher restraint provided by the elastomeric bearings. Not surprisingly, the assumption of reduced bearing stiffness leads to an intermediate level of target displacements with respect to the other two cases. It is notable, though, that in all three cases, the seismic displacement that is attributed to the torsional mode is substantially lower compared to the target displacement along the longitudinal and transverse direction primarily to the stiff frame action of the pier which provides significant torsional stiffness.

5 CONCLUSIONS

This study focused on the seismic response of a straight overcrossing with two equal spans, whose fundamental mode, unlike most bridges of this type, is purely torsional. Through an extensive parametric analysis scheme it was demonstrated that the performance of the bridge is satisfactory even for loading that corresponds to twice the level of the design earthquake. From an analysis point of view, it was shown that the reliability of the seismic assessment performed is related to a number of important assumptions regarding: (a) the consideration of the torsional nature of the fundamental mode; (b) the selection of an appropriate monitoring point for drawing the pushover curve in the transverse direction (the end, rather than the mass centre, of the deck was adopted herein), as well as of a proper measure of the applied loading when antisymmetric loading patterns are used, in a modal pushover context (the shear at the abutment was used here); (c) the accurate estimation of foundation compliance and the subsequent computation of the static stiffness to be used for the pier foundations; (d) the decision made for the boundary conditions at the location where the deck rests on the abutment, and (e) the appropriate numerical simulation of the non-linear force-displacement relationship and the pertinent failure mechanism of the elastomeric bearings used. All the above decisions were found to play a significant role in the final assessment of the bridge and as such, they have to be dully justified in each case, and sensitivity analyses should be performed wherever necessary (and feasible, if the study is carried out within a design office environment).

Another important observation made was that seismic displacements derived from the torsional excitation of the bridge demonstrate that the bridge is not particularly sensitive to torsion, despite the fact that the fundamental mode is torsional. As a result, proper assessment of this bridge should be primarily based on the longitudinal and transverse loading distributions, as it was shown that, even under torsional loading patterns, pier failure precedes bearing failure (at the abutments). Regarding the torsional sensitivity of the bridge which was highlighted by the modal analysis results, it was concluded that torsional deformation is not critical, mainly due to the high torsional resistance provided by the stiff pier and the use of elastomeric bearings (in lieu of pot bearings that are commonly used at this location) at the deck-abutment interface. It is deemed that further investigation could contribute towards the development of a more generalised non-linear static analysis procedure for short bridges with a first torsional mode.

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