

Numerical simulation of the seismic response of a typical motorway bridge in the longitudinal direction: the effect of abutment stoppers

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ABSTRACT

The majority of modern motorway networks usually contain a few hundreds of bridges having a great variety of structural characteristics. It is quite common to classify them in different typologies in order to study representative cases. The most common classification schemes are based on the main structural characteristics, such as the number of spans, the type of the deck (continuous or simply-supported), and the pier-deck connection (monolithic or on bearings). Nevertheless, the presence of abutment stoppers may significantly affect their seismic performance calling for a more detailed classification. In the present work, the effect of the aforementioned factor on the seismic response of motorway bridges in the longitudinal direction is examined. For this purpose, a rigorous 3D model of a typical modern motorway bridge of the Attiki Odos motorway (Athens, Greece) is developed and subjected to seismic loading in the longitudinal direction. Based on the results of the nonlinear dynamic time history analyses, it can be concluded that the effect of abutment stoppers is quite significant, and must be taken into account. Therefore, the resistance mechanism of abutments triggered when the bridge deck collides on the stoppers is further examined. To that end, a model of a typical motorway bridge abutment is developed. The latter consists of a cantilever retaining wall of typical dimensions, while two characteristic embankment soil profiles are studied, one cohesive (clayey) and one cohesionless (sandy). Overall, it is shown that the resistance mechanism of the abutments is rather complicated and highly dependent to the soil profile. Nevertheless, it is proven to significantly affect the seismic response of the bridge, especially for earthquakes that exceed the design limits.

Keywords: overpass bridge, abutment stoppers, seismic response, nonlinear finite elements

INTRODUCTION

A great variety of bridge typologies can be found around the world, rendering the task of developing a global classification rather ambitious. The present study focuses on modern motorway bridges, such as those encountered in the Attiki Odos motorway in Athens, Greece. With a total length of 65 km, Attiki Odos is a modern motorway serving as a ring road of the greater metropolitan area of Athens. It includes a variety of critical structures, such as bridges, tunnels, retaining walls, slopes, and embankments. A total of 192 bridges can be found along the motorway. Analyzing such a number of bridges one-by-one would require quite some computational effort. Hence, there is a need for classification of the motorway bridges in representative classes.

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Several classification schemes are available in the literature, such as the one proposed by Anastasopoulos et al. (2015). Nevertheless, to the best of the authors' knowledge, the available schemes don't take into account the presence of stoppers at the abutments. However, most modern motorway bridges are equipped with abutment stoppers, either having a substantial clearance, and therefore being activated after a measurable relative displacement, or being practically in contact with the deck being inactive under service loads, but preventing seismic displacement at the abutments. Although the effect of such retention devices is recognized, there are only few studies dealing with the contribution of the abutments and of the corresponding embankments (e.g., Wilson & Tan, 1990; Zhang & Makris, 2002a; 2002b; Kotsoglou & Pantazopoulou, 2007; Tegou et al., 2010). On the other hand, a substantial amount of experimental work has been conducted for different types of abutments and embankment soils, including monotonic (Duncan and Mokwa, 2001; Wilson & Elgamal, 2009), cyclic (Thurston, 1986a; Maroney et al., 1994; Gadre & Dobry, 1998; Rollins & Cole, 2006; Heiner et al., 2008), and seismic loading (Gadre & Dobry, 1998). In the present work, the effect of abutment stoppers to the seismic response of modern motorway bridges is numerically investigated, considering the longitudinal direction of seismic loading.

NUMERICAL ANALYSIS METHODOLOGY

Most modern motorway bridges are equipped with abutment stoppers having a clearance (δ_c) to the abutment wall. In the case that the deck drift δ does not exceed the available clearance, its only restraint comes from the shear resistance of the bearings. Nevertheless, as schematically illustrated in Fig. 1, after the available clearance is consumed, the stoppers are engaged impeding the movement of the deck, thus undertaking a substantial portion of the inertia forces. This way, the stoppers are actively limiting the deck drift δ below a specific level, aiming to diminish the probability of unseating and to ensure the structural integrity of the bridge under seismic motions exceeding the design limits. Moreover, this limitation of δ by the stoppers protects the bearings from excessive seismic deformations.

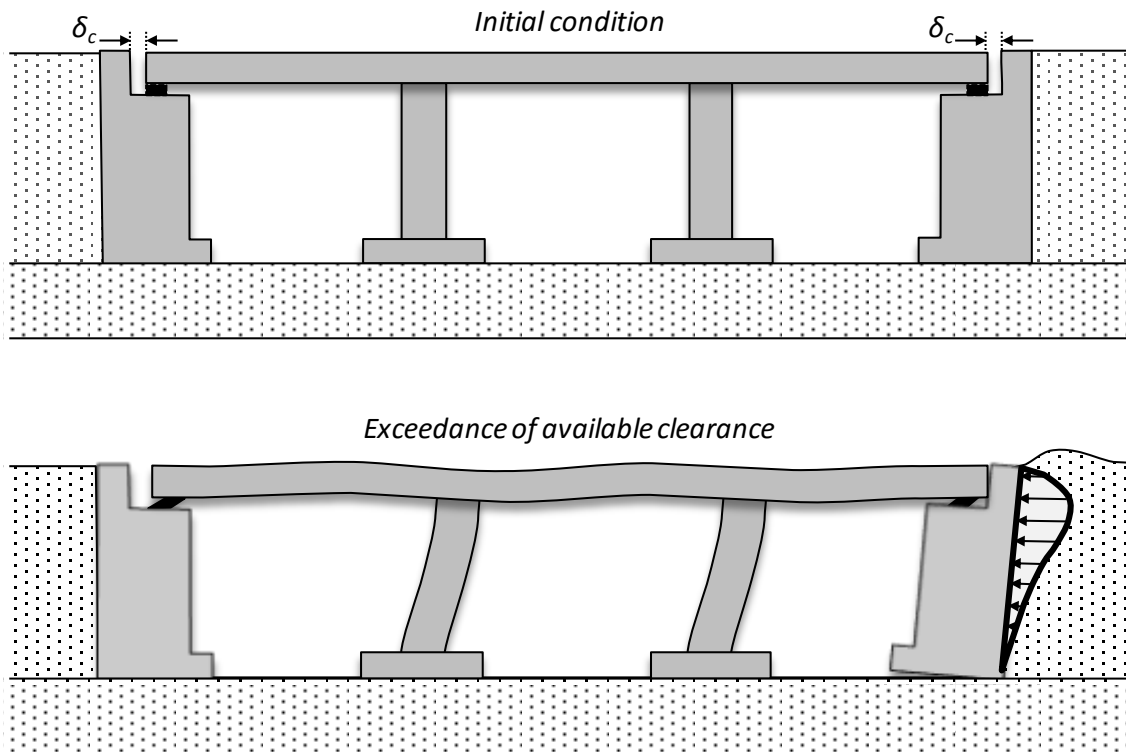


Figure 1. Sketch of the role of abutments on longitudinal bridge response, illustrating the resistance mechanism after the available stopper clearance is exceeded.

To examine the aforementioned resistance mechanism of abutments, a typical overpass bridge (A01-TE20) of the Attiki Odos Motorway, is selected as an illustrative example. Besides its simplicity, the selected bridge system is representative for about 30% of the bridges of the specific motorway, and is also considered quite common for metropolitan motorways in general. As shown in Fig. 2a, the selected system is a symmetric 3-span bridge with a continuous pre-stressed concrete box-girder deck, supported on two reinforced concrete (RC) cylindrical piers of diameter $d = 2$ m and height $h = 8.8$ m. The piers are monolithically connected to the deck, which is supported by 4 elastomeric bearings at each abutment. Each bearing is 0.3 m x 0.5 m (longitudinal x transverse) in plan and has an elastomer height $t = 63$ mm. The piers are founded on $B = 8$ m square footings, while the abutments consist of cantilever-type retaining walls of 9 m height and 1.5 m thickness. The latter are connected to two side walls of 0.6 m thickness and founded on a rectangular 7 m x 10.4 m rectangular footing.

The performance of the bridge is analyzed employing the FE code ABAQUS. For this purpose, a rigorous 3D model of the bridge is developed, taking account of the foundations, the abutments, and the soil (Fig. 2b), based on the work of Anastasopoulos et al. (2015). The deck and the piers are modeled with elastic and inelastic beam elements, respectively. The reinforcement of the $d = 2$ m RC piers has been computed according to the provisions of the Greek Code for Reinforced Concrete (ΕΚΩΣ, 2000). The inelastic behavior of the piers is simulated with a nonlinear model, calibrated against the results of RC section analysis using the USC_RC software (2001). Linear elastic springs and dashpots are used to model the compression ($K_{c,b}$) and shear stiffness ($K_{s,b}$) and damping ($C_{c,b}$, $C_{s,b}$) of the bearings:

$$K_{c,b} = \frac{E_c A}{t n} \quad (1)$$

$$K_{s,b} = \frac{G A}{t n} \quad (2)$$

$$C_{c,b} = \frac{2 K_{c,b} \xi}{\omega} \quad (3)$$

$$C_{s,b} = \frac{2 K_{s,b} \xi}{\omega} \quad (4)$$

where E_c : the compression modulus of the elastomer; A : the plan area of the bearing; t : the thickness of the individual elastomer layers; n : the number of individual elastomer layers; G : the shear modulus of the elastomer; ξ : the damping coefficient of the bearing; and ω : the angular frequency of reference (assumed to be equal to the dominant mode of the bridge).

The bridge–foundation–abutment–soil system (Fig. 2b), the footings and the abutments are modeled with elastic hexahedral continuum elements, assuming the properties of RC ($E = 30$ GPa). An idealized 20 m deep substratum of homogeneous stiff clay is considered, having undrained shear strength $S_u = 150$ kPa. In addition to examine the effect of embankment soil to the overall resistance of the abutment, two idealized soil profiles are considered, representing cohesive and cohesionless embankment material. In the first case, a homogeneous stiff clay layer of undrained shear strength $S_u = 150$ kPa is considered. In the latter case, a homogeneous medium-dense sand layer is examined, having a friction angle $\phi = 35^\circ$. The soil is also modeled with hexahedral continuum elements. Nonlinear foundation and embankment soil behavior is modeled with a kinematic hardening model, with a Von Mises failure criterion and associated flow rule. The evolution law of the model consists of a nonlinear kinematic hardening component, which describes the translation of the yield surface in the stress space, and an isotropic hardening component, which defines the size of the yield surface as a function of plastic deformation (Gerolymos & Gazetas, 2005). Calibration of model parameters requires knowledge of: (a) the soil strength (expressed through undrained shear strength S_u for clay or friction angle ϕ for sand); (b) the small-strain stiffness (expressed through G_o or V_s); and (c) the stiffness degradation (G – γ and ξ – γ curves). More details on the model can be found in Anastasopoulos et al. (2011).

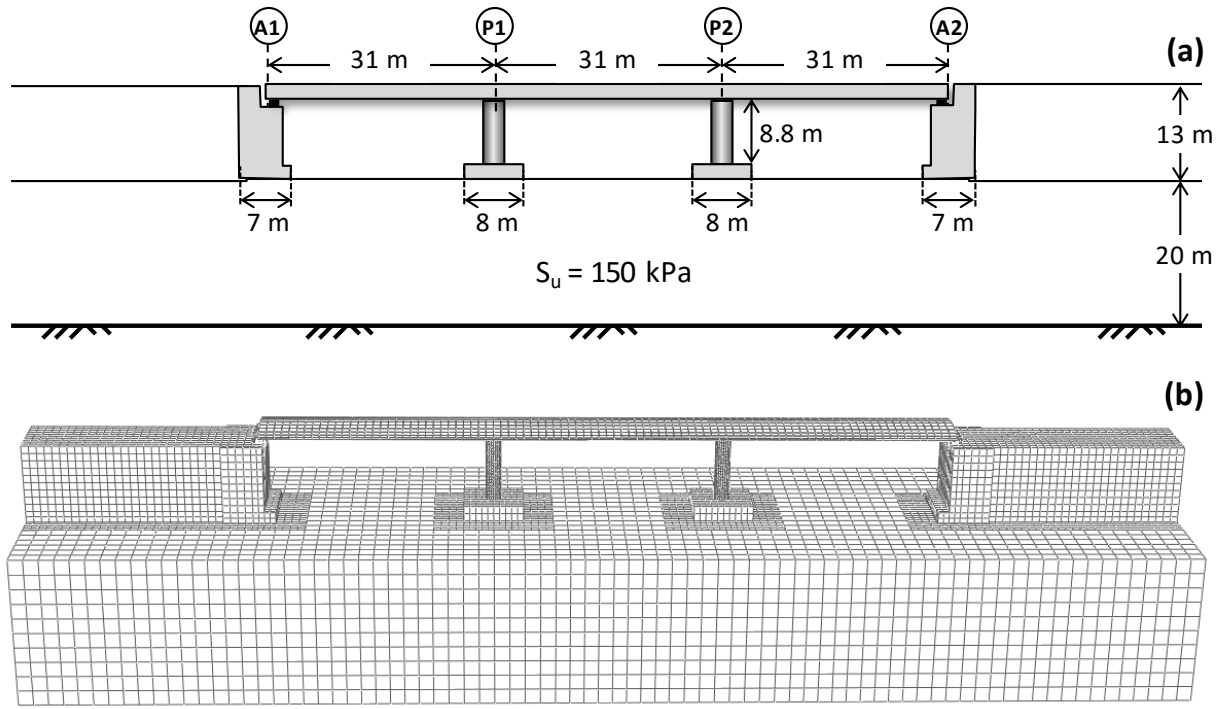


Figure 2. Typical overpass bridge A01-TE20 of the Attiki Odos motorway: (a) longitudinal section and key attributes; and (b) rigorous 3D model, including the foundations, the abutments, and the subsoil.

Appropriate “free-field” boundaries are used at the lateral boundaries of the model, while dashpots are installed at its base to simulate the half-space underneath the 20 m of the soil that is included in the 3D model. A tensionless interface is introduced between soil and foundation elements in order to model uplifting and sliding, assuming a friction coefficient $\mu = 0.7$. More details on the model can be found in Anastasopoulos et al. (2011; 2015). For the interfaces between the retaining wall and the embankment a maximum shear capacity of $0.5 \cdot S_u$ is considered for the cohesive embankment (Powrie, 2013), while a friction coefficient $\mu = \tan \delta = \tan(0.76\phi)$ is considered for the cohesionless one, according to the minimum values proposed by Potyondy (1961) for sand-to-concrete interface. A reinforced soil embankment is considered, which is quite common in such motorway bridges (due to space limitations). The latter is modeled in a simple manner, by installing appropriate kinematic constraints in the transverse direction. Finally, special “gap” elements are introduced between the deck and the abutments in order to model the effect of the stoppers. An initial clearance δ_c is assigned, with the gap elements being activated only in compression and only after δ_c is consumed. More specifically, when δ_c is consumed the “gap” elements close and the deck starts pushing the wall against the embankment triggering thus its resistance.

RESULTS

The seismic response of the A01-TE20 bridge in the longitudinal direction, with and without stoppers, is examined. The comparison between the two systems is conducted for both cohesive and cohesionless embankments in terms of slow-cyclic pushover and dynamic nonlinear time-history analyses. The relevant results are summarized in Fig. 3 for the clayey embankment and in Fig. 4 for the sandy one. An initial clearance $\delta_c = 0.08$ m is assigned, which is about the average value of available clearances in the bridges of the Attiki Odos motorway. For the dynamic analyses a series of 4 strong to very strong earthquake records are used as seismic excitation (Fig. 3b; 4b), aiming to induce a large enough deck drift to exceed the available clearance and the stoppers to be engaged. For such strong seismic shaking, the comparison confirms that the effect of stoppers can be quite beneficial, for both embankment soil profiles. More specific, as depicted in Fig. 3a and 4a, at the first cycle of loading and till the moment that the deck collides on the stoppers at the abutment, the $F-\delta$ response of the two systems is identical, as expected. After the collision, however, the resistance of the abutment system (wall-embankment soil) is mobilized, increasing significantly (almost 3 times for the clayey embankment and 2 times for the sandy one) the total resistance of the bridge. A similar behaviour is also observed during the consequent cycles of unloading and reloading. The aforementioned response is further

pronounced considering the results of the dynamic analyses (Fig. 3c-f; 4c-f). For all four seismic excitations the maximum observed deck drift δ is significantly reduced for the analyses with stoppers, confirming their beneficial impact. Indicatively for the notorious Takatori-000 record from the 1995 Kobe earthquake and for the cohesive embankment, the resulting deck drift δ in the case with stoppers is half the one without stoppers (from almost 50 cm to 25 cm). A maximum drift of 25 cm is still considered a quite serious damage to the bridge, but at least it has been significantly reduced protecting the bridge from total collapse. In addition, the bridge deck without stoppers experiences not one but three cycles of more than 40 cm drift, resulting in its almost certain collapse. On the contrary, in the case of stoppers the maximum experienced deck drift does not exceed 25 cm for all three important cycles. It is also evident that the system becomes stiffer. Furthermore, an additional beneficial impact of stoppers specifically in dynamic loading is the energy dissipation due to the plastification of the nonlinear embankment soil when the deck collides on the abutments. Considering the relevant results for the case of sandy embankment (Fig. 4c-f), the favourable impact of stoppers is less pronounced, but still important. Therefore, the aforementioned conclusions are similar for both embankment soil profiles. The observed behaviour is also consistent with previous research on the subject (e.g., El-Gamal & Siddharthan, 1998; Faraji et al., 2001; Chapman et al., 2005; Wood et al., 2007; Wood, 2009), confirming the need to account for abutments stoppers when analyzing the seismic response of bridges in the longitudinal direction.

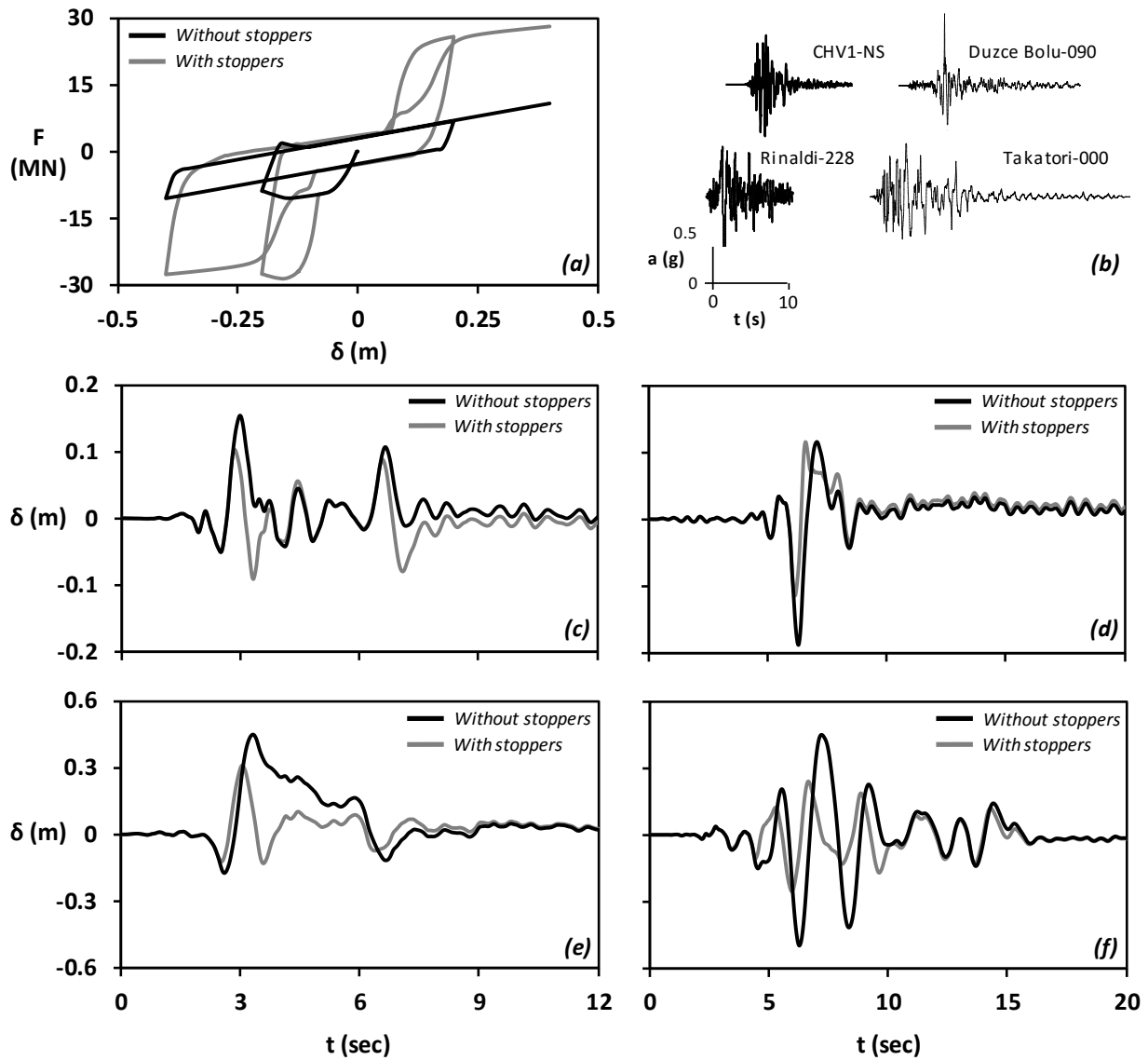


Figure 3. Comparison of the performance of the A01-TE20 bridge in the longitudinal direction for clayey embankment, with and without stoppers, in terms of: (a) slow-cyclic pushover (F – δ) response; (c–f) time histories of deck drift δ using four different seismic records (b) as seismic excitation for: (c) CHV1-NS (Kefalonia, 2015); (d) Duzce Bolu-090; (e) Rinaldi-228; and (f) Takatori-000 (Kobe, 1995) records.

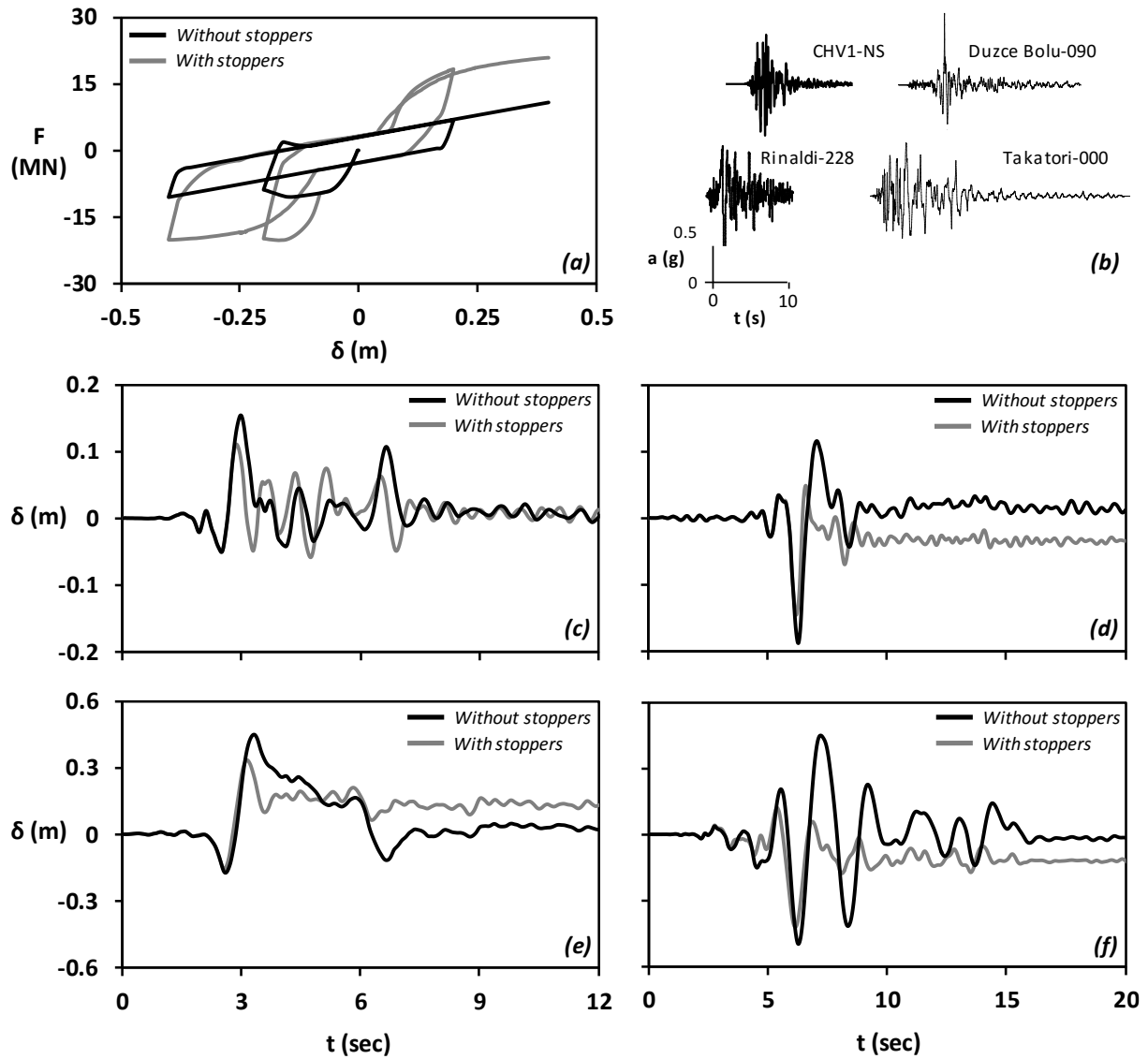


Figure 4. Comparison of the performance of the A01-TE20 bridge in the longitudinal direction for sandy embankment, with and without stoppers, in terms of: (a) slow-cyclic pushover (F - δ) response; (c-f) time histories of deck drift δ using four different seismic records (b) as seismic excitation for: (c) CHV1-NS (Kefalonia, 2015); (d) Duzce Bolu-090; (e) Rinaldi-228; and (f) Takatori-000 (Kobe, 1995) records.

Based on the previous results, the next step is to further examine the resistance mechanism of the abutment system (cantilever wall-embankment). To that end, a cohesive (clayey) embankment and a cohesionless (sandy) one are considered in order to investigate the effect of different soil types. In addition, due to the high computational effort needed for the analysis of the rigorous 3D model, an equivalent 2D model of the abutment is developed (Fig. 5). The latter is used to investigate its response when loaded at the top (deck level), which corresponds to the loading of the abutment when the deck collides on the stoppers during seismic loading. Figure 5 shows the comparison of the response between the rigorous 3D model and the simpler 2D one subjected to passive-type loading at the top, for both cohesive (Fig. 5a) and cohesionless (Fig. 5b) soil. The comparison is performed in terms of deformed mesh with superimposed plastic strain contours (top row), and distribution with depth of horizontal stresses (bottom row). The developed failure mechanism for the cohesive soil is also illustrated (top row). Overall, it can be concluded that the effect of the out-of-plane width of the wall, as well as of the side walls to the total mobilized horizontal resistance of the abutment is not that important. Therefore, the simpler 2D model can be considered as a good approximation.

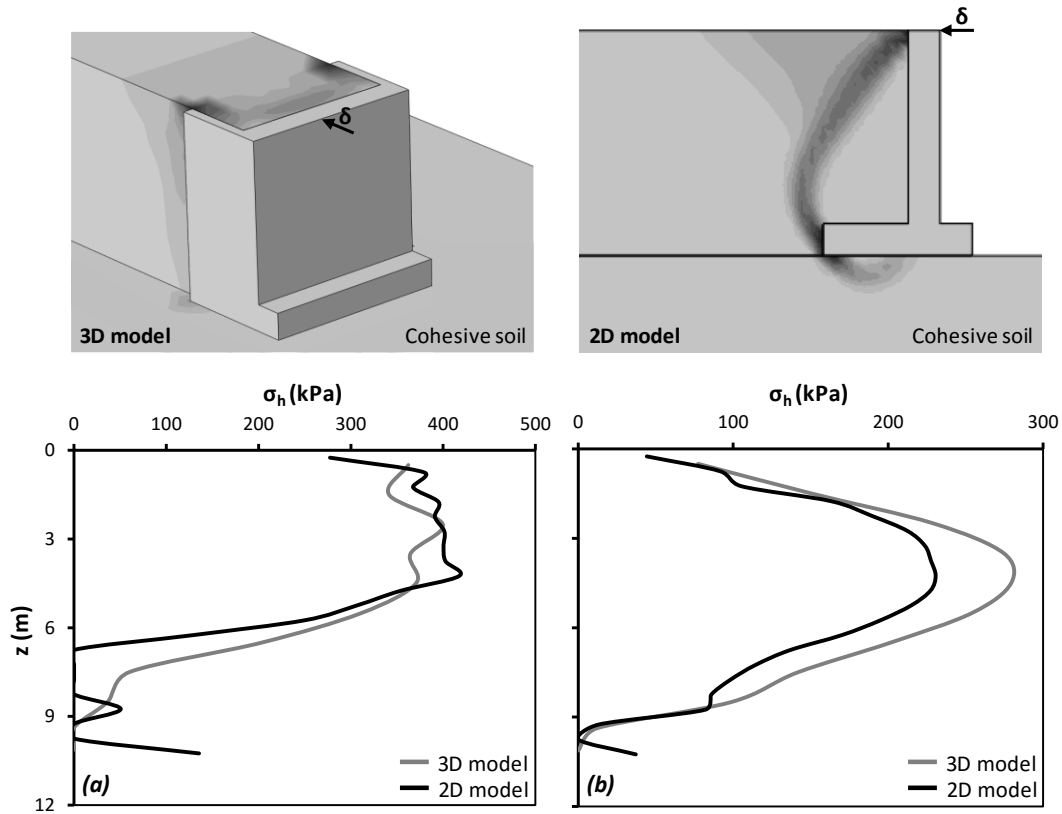


Figure 5. Passive-type loading at the top of the abutment: comparison of the 3D model to the simpler 2D model in terms of deformed mesh with superimposed plastic strain contours (top), illustrating the developing failure mechanism for cohesive soil, and distribution with depth of horizontal stresses (bottom) for: (a) cohesive; and (b) cohesionless soil.

The 2D model is subsequently used to compare the behavior of the abutment cantilever wall subjected to passive-type loading at the top and to traditional passive conditions with rotational restraint (Fig. 6). The comparison is again performed in terms of deformed mesh with superimposed plastic strain contours (top row), and distribution with depth of horizontal stresses (bottom row) for cohesive (Fig. 6a) and cohesionless (Fig. 6b) soil. The developed failure mechanism for the cohesive soil is also illustrated (top row). A first conclusion is that the failure mechanisms are quite different for both soil profiles. For the traditional passive conditions with rotational restraint, the developed failure wedge starts from the base of the wall and is transmitted to the soil surface at an angle of about 45° to the horizontal level for the cohesive soil and of about 35° for the cohesionless. The latter compare well to the theoretical solutions. For the cohesionless profile the inclination is a bit steeper than the theoretical value of 27.5° ($45 - \phi/2$), which is attributed to the use of a small cohesion $c = 2$ kPa for numerical stability, the fact that this is not a gravity wall but a cantilever one and that there is an interface at the base of the wall, between the cohesionless embankment and the cohesive soil base.

On the contrary, the passive-type loading of the cantilever wall at its top without rotational restraint, mobilizes a different failure mechanism which results in much different total resistance. In the particular case for the clayey embankment the failure wedge is much shallower and the passive resistance is fully mobilized only at about half of the depth (Fig. 6a). For the sandy embankment the failure wedge is quite different and a rotational-type failure mechanism is mobilized (Fig. 6b). Reasonably enough, the finally developed failure mechanism at an embankment depends not only on the type of loading and the soil profile, but equally significantly on the type and dimensions of the wall and the soil underneath. Additionally, the vertical loading of the wall (expressed through the vertical factor of safety) also affects the resistance, as it changes the response of the wall from uplift to rotational – type failure of the soil underneath. The impact of the aforementioned factors should be thoroughly examined in order to gain more insights about this complex behavior. The latter is beyond the scope of the present paper.

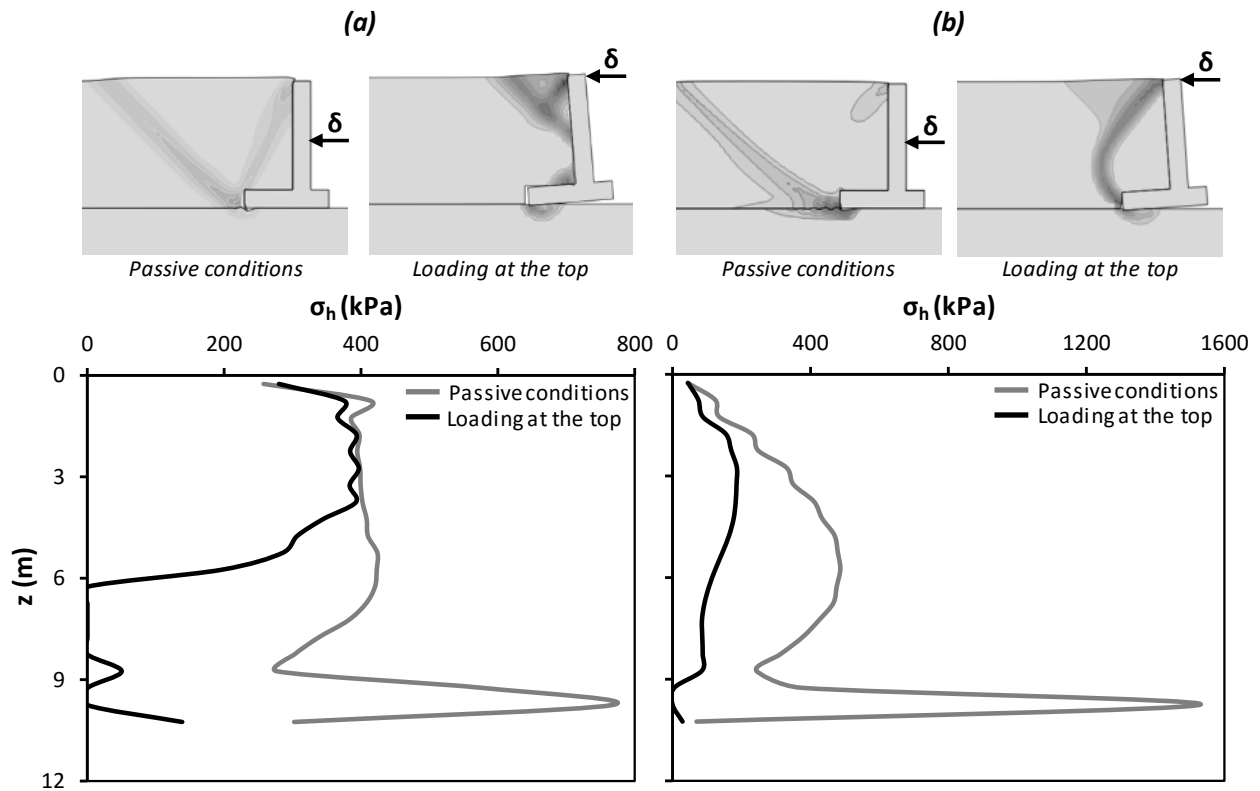


Figure 6. 2D analysis of the abutment subjected to passive-type loading at the top and to traditional passive conditions (with rotational restraint). Deformed FE mesh with plastic strain contours (top), and distribution with depth of horizontal stresses (bottom) for: (a) cohesive; and (b) cohesionless soil.

CONCLUSIONS

The motorway networks worldwide usually contain a few hundreds of bridges with different structural characteristics. It is common practice to classify them in different typologies in order to study representative cases. Several classification schemes are available in the literature, such as the one proposed by Anastasopoulos et al. (2015). Nevertheless, the presence of stoppers at the abutments is rarely accounted for. However, most modern motorway bridges are equipped with abutment stoppers, either having a substantial clearance, or being practically in contact with the deck. The presence of the latter may significantly affect their seismic performance, as schematically illustrated in Fig. 1.

In the present study, the effect of the abutment stoppers on the seismic response of such bridges in the longitudinal direction is examined. To that end, a rigorous 3D model of a typical modern motorway bridge (A01-TE20) of the Attiki Odos motorway is developed (Fig. 2). Two different embankment soil profiles are considered: a cohesive one, and a cohesionless one. The rigorous 3D model is used to perform two different sets of analyses: one without considering the abutment stoppers and one that takes into account their presence. The comparison is shown for both a clayey and a sandy embankment in terms of slow-cyclic pushover $F-\delta$ response (Fig. 3a; 4a) and time histories of deck drift δ (Fig. 3c-f; 4c-f) using four different strong to very strong seismic records (Fig. 3b; 4b) as seismic excitation. According to the results the impact of abutment stoppers is quite beneficial reducing significantly the maximum deck drifts and thus protecting it from total collapse for both soil profiles. In the case of sandy embankment their beneficial impact is less pronounced. The observed behavior is consistent with previous research on the subject. Therefore, it is concluded that the presence of abutment stoppers is important, calling for a more detailed classification.

Subsequently, the resistance mechanism of the abutment system (cantilever wall–embankment), triggered when the bridge deck collides on the stoppers is further examined. To that end and due to the high computational effort necessary for the analysis of the rigorous 3D model, a simpler 2D model of the abutment is developed (Fig. 5). The comparison between the response of the two models subjected to passive-type

loading at the top (without rotational constraint) is shown for both the cohesive embankment (Fig. 5a) and the cohesionless one (Fig. 5b). The comparison is performed in terms of deformed mesh with superimposed plastic strain contours (top row), and distribution with depth of horizontal stresses (bottom row). The developed failure mechanism for both soil profiles is also illustrated (top row). Overall, it can be concluded that the effect of the out-of-plane width of the wall, as well as of the side walls to the total mobilized horizontal resistance of the abutment is not that important. Therefore, the simpler 2D model can be considered as a good approximation.

Finally, using the 2D model of the abutment, its behavior subjected to passive-type loading at the top is compared to the relevant one simulating traditional passive conditions with rotational restraint (Fig. 6). The comparison is again performed in terms of deformed mesh with superimposed plastic strain contours (top row), and distribution with depth of horizontal stresses (bottom row) for cohesive (Fig. 6a) and cohesionless (Fig. 6b) soil. The main conclusion is that the failure mechanisms, and hence the total resistance of the abutment, are quite different for both soil profiles. The latter depends not only on the type of loading and the embankment soil profile, but equally significantly on the type and dimensions of the wall, the foundation soil underneath, the vertical loading of the wall (expressed through the vertical factor of safety). The impact of the aforementioned factors should be thoroughly examined in order to gain more insights about this complex behavior.

ACKNOWLEDGEMENT

The financial support for this paper has been provided by the research project “SYNERGY 2011” (Development of Earthquake Rapid Response System for Metropolitan Motorways) of GGET–EYDE–ETAK, implemented under the “EPAN II Competitiveness & Entrepreneurship”, co-funded by the European Social Fund (ESF) and national resources.

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