Effectiveness of In-soil Seismic Isolation taking into account of Soil-Structure Interaction

Efficacité d' Isolement sismique dans le Sol tenant compte de l'interaction du Sol avec la Structure

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ABSTRACT: In the present study an innovative seismic isolation method is proposed that introduces a sliding surface within the foundation soil.The sliding surface comprises of two synthetic liner layers at contact with each other creating an interface of small friction that enfolds the foundation soil. The effectiveness of the isolation system is explored as a function of the earthquake intensity accounting for soil-structure-interaction phenomena. It is shown that the proposed system serves as a fuse mechanism within the soil and substantially reduces the acceleration transmitted onto the structure. The isolated structure may be subjected to increased differential lateral displacement, due to sliding at the isolation interface – something that has to be considered in the design.

RÉSUMÉ : Dans cette étude une méthode innovante d'isolement sismique est proposé, composé d'une surface de glissement dans le sol. La surface de glissement se compose de deux couches de revêtement synthétique, caractérisé d'une résistance réduite. L'efficacité du système proposé est explorée en fonction de l'intensité sismique. Il est démontré que le système proposé acte comme un mécanisme fusible dans le sol, réduisant considérablement l'accélération transmise sur la structure. La structure isolée peut être soumis à un déplacement différentiel latérale, en raison de glissement à l'interface d'isolation – quelque chose que doit être pris en compte dans le désign.

KEYWORDS: soil-structure interaction ; in-soil isolation ; synthetic liner

1 INTRODUCTION

In the present study an isolation method is proposed that introduces a sliding surface within the foundation soil. The sliding surface comprises two layers of a smooth synthetic liner in contact with each other. Yegian et al. (2004a) were the first to propose the application of synthetic liners just below the foundation with the intention to introduce, an interface of small friction coefficient upon witch the structure would slide as a rigid block. To determine the properties of the interface, shaking table tests were conducted, concluding that the static and dynamic friction coefficient is of the order of 0.10 and 0.07, respectively. The same researchers (Yegian et al., 2004b) investigated the idea of such a sliding surface within the foundation soil, forming an isolated soil prism of ellipsoidal shape.

Based on this idea, Georgarakos & Gazetas (2006) parametrically investigated the effect of the sliding surface geometry on the seismic response. Several geometries were investigated, ranging from cyndrical, to basin-shaped, trapezoidal, and trapezoidal with wedges. The latter was found to be the optimum solution, providing the restoring force of the cylindrical surface, while being significantly easier to construct. The systems functionality is based on the ability of the isolated soil to slide on the synthetic liner, while the two wedges offer the necessary restoring force through their weight. The response of this system can be seen as mechanically analogous to a mass sliding on a horizontal surface, being restrained by two springs that work only when compressed.

The investigated system is schematically illustrated in Figure 1. The geometry of the isolation system is trapezoidal, with isolated wedges on the two sides. The synthetic liners are placed at a depth H = 2 m under the surface. The slope of the excavation trench is assumed equal to 1:1 – a realistic assumption for relatively competent soi. The isolated embankment comprises a dense gravel layer. The latter is modeled with a nonlinear constitutive model, with a Mohr-Coulomb failure criterion and non-associative flow rule. A rather large Young's modulus E = 500 MPa is assumed, while the friction and dilation angles are equal to $\varphi = 48^o$ and $\psi = 15^o$, respectively. The two wedges are filled with pumice, a lightweight material of density $\rho = 1$ Mg/m^3 and relatively small stiffness $E = 10$ MPa, in order to impose the minimum possible resistance to the sliding motion of the embankment.

The superstructure, an idealized bridge pier (for simplicity), is placed on top of the isolated embankment. The bridge pier is designed according to EC8, assuming a design acceleration $a_{gr} = 0.24$g and behavior factor $q = 2$. Having an elastic natural period $T = 0.48$ sec, the design spectral acceleration is equal to SA = 0.3g. In order to undertake the resulting design bending moment $M_D = 43$ MNm, a longitudinal reinforcement of 100Φ32 is required, combined with transversal reinforcement of Φ32/8cm.

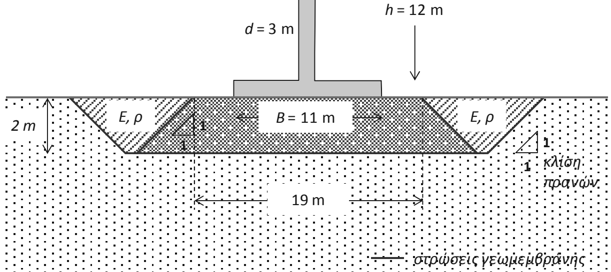

Figure 1. Schematic illustration of the in-soil isolation system under consideration.

2 NUMERICAL METHODOLOGY

The problem is analyzed employing the finite element code ABAQUS. The geometry and the key aspects of the model used in the analyses are presented in Figure 2. Assuming plain strain conditions, a representative "slice" of the soil–foundation–structure system is examined, taking account of material (soil and superstructure) and geometric (footing uplift, sliding, and P–δ effects) nonlinearities.

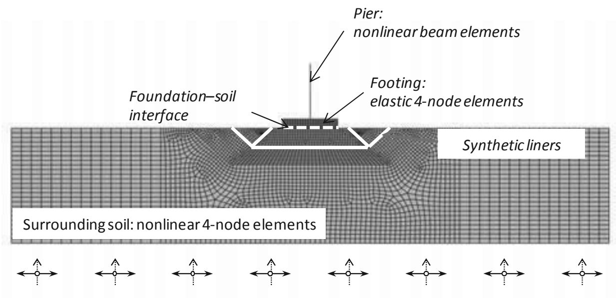

Figure 2. The finite element model used in the analyses: plain strain conditions are assumed, considering material (soil and superstructure) and geometrical (sliding, P-Δ phenomena) nonlinearities .

The soil is modeled with 4-noded continuum elements. The soil behavior is modeled through a nonlinear constitutive model with Von Mises failure criterion, nonlinear kinematic hardening and associated plastic flow rule. The footing is modeled with elastic 4-noded continuum elements with E = 30 GPa. Beam elements are used for the pier, with their nonlinear behavior being modeled with a kinematic hardening model (Gerolymos et al., 2005), similar to that of the soil. Model parameters are calibrated against moment–curvature relations of the reinforced concrete pier, computed through section analysis utilizing the XTRACT software (Imbsen & Assoc., 2004). The deck is represented by a mass element, and the contact between the different parts of the model (footing, embankment, wedges, surrounding soil) is modeled with a special interface that allows realistic simulation of possible sliding and detachment.

3 DYNAMIC RESPONSE OF THE ISOLATION SYSTEM

Initially, the in-soil isolation system is subjected to idealized Ricker pulses of characteristic frequency f = 2 Hz and gradually increasing maximum acceleration (0.1g to 0.5g). Both the fully SSI problem as well as the free-field problem (i.e., ignoring the presence of the superstructure) are analyzed.

In Figure 3 the response of the isolation system is presented in terms of maximum acceleration at the top of the isolated embankment with respect to the maximum acceleration at the surface of the non isolated free-field (PGA), both in and without the presence of the pier. Evidently, the effectiveness of the in-soil isolation system depends on the presence of the superstructure. Maximum acceleration at the top of the isolated embankment does not exceed 0.2 g without the superstructure on top. On the other hand, the presence of the pier leads to an increase in the acceleration, which in this case ranges between 0.28 g and 0.33 g.

In Figure 4 the deformed mesh with superimposed displacement contours, showing the deformation of the system when in the presence of the pier and without it. The deformation scale factor applied is deliberately large, in order to highlight the difference between the two cases examined. Observe the aforementioned increase in the acceleration that passes through the isolation layer, which is due to its deformation by the vertical pressures which are imposed by the weight of the pier. As a result, the isolated embankment is forced to slide on a curved surface, rather than a horizontal one. Consequently, the acceleration that is required for slippage is increased substantially, reducing the effectiveness of the isolation system.

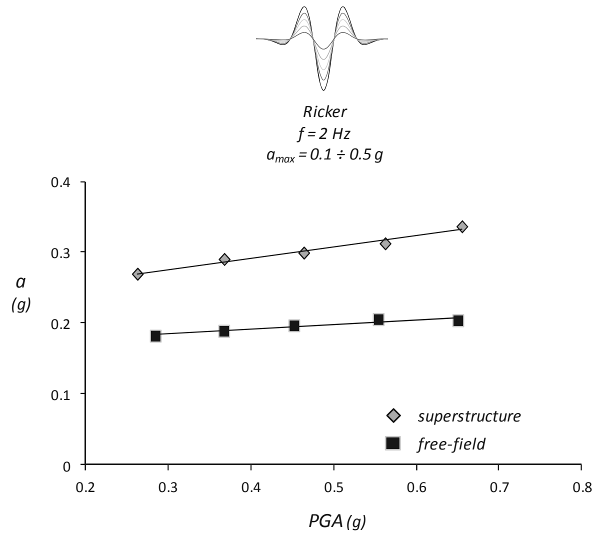

Figure 3. Maximum acceleration at the top of the isolated embankment with respect to the maximum acceleration at the surface of the non isolated free-field (PGA), with and without the presence of the pier. The bedrock excitation is an idealized Ricker wavelet of characteristic frequency f = 2 Hz, and gradually increasing maximum acceleration (from 0.1g to 0.5g).

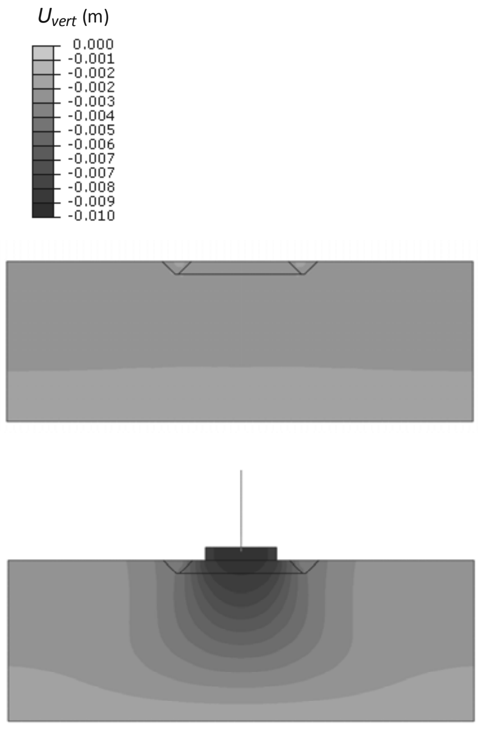

Figure 4. Deformed mesh with superimposed vertical displacement contours considering the superstructure on top of the isolated embankment and without it. (deformation scale factor = 100).

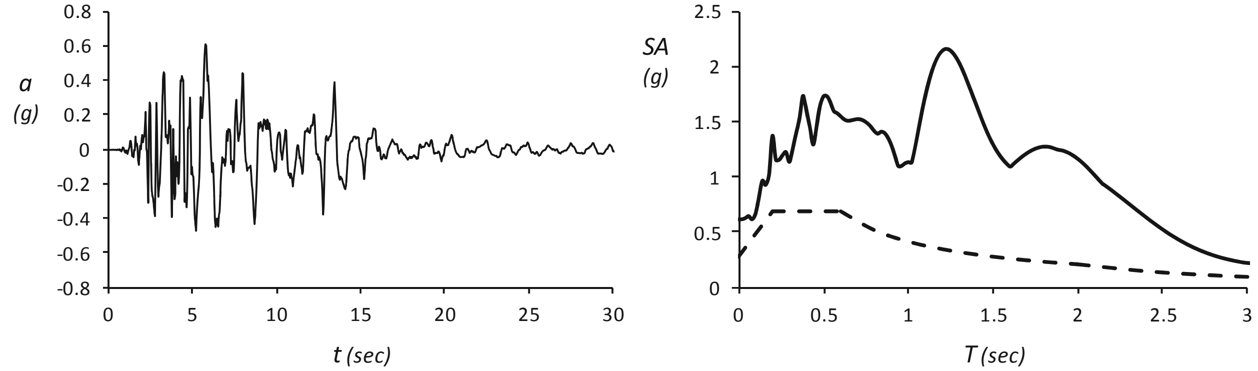

Figure 5. The Takatori record from the Kobe earthquake (Japan 1995) and its elastic response spectrum compared to the pier design spectrum.

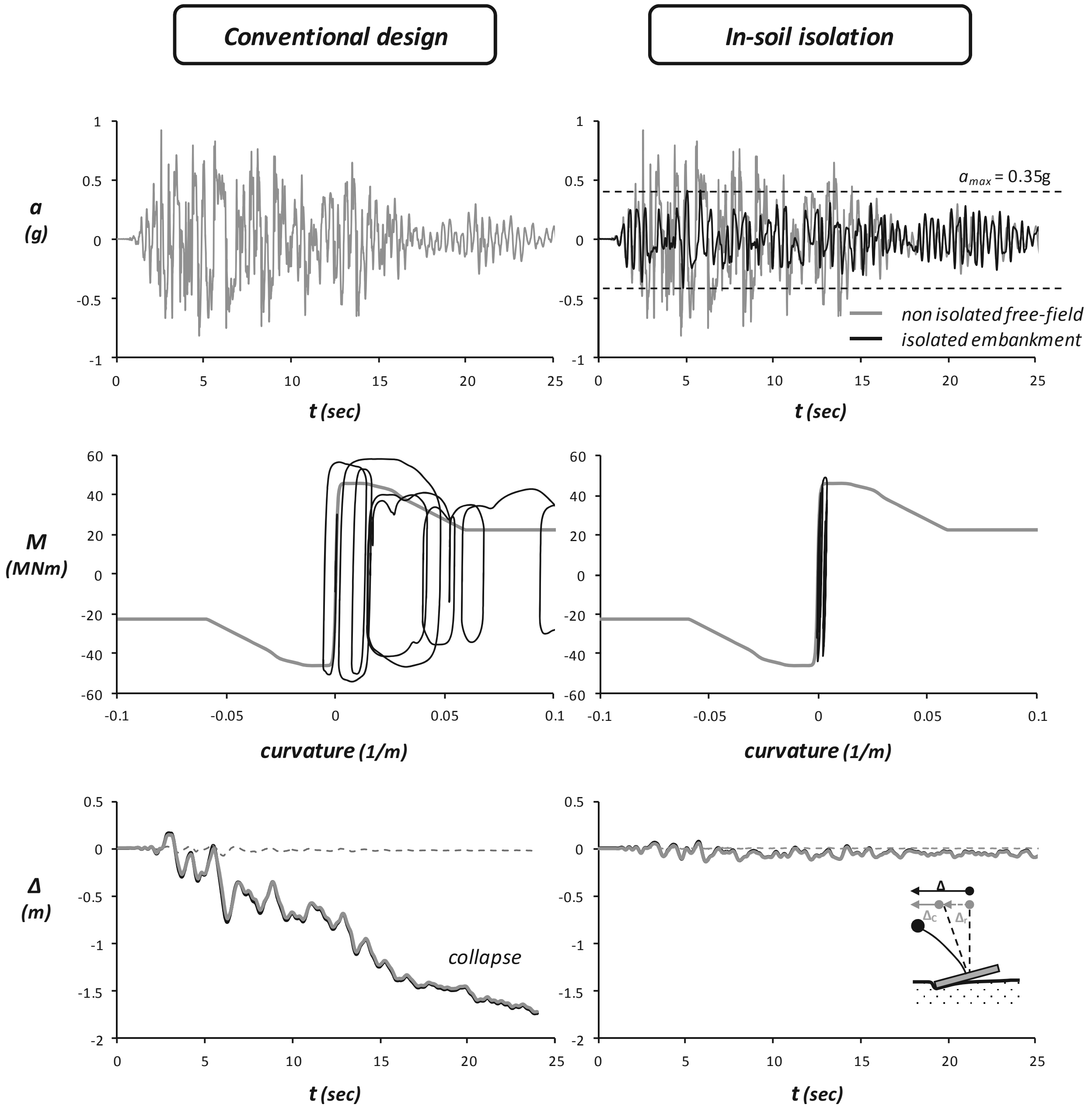

Figure 6. Comparison of the two alternatives: conventionally designed pier response versus pier response with application of the in-soil isolation system (a) Acceleration time histories at the base of the pier. (b) Moment – curvature response at the pier base and (c) time histories of deck drift Δ.

4 EFFECTIVENESS OF IN-SOIL SEISMIC ISOLATION SYSTEM SUBJECTED TO REAL RECORDS

The model is subjected to a seismic scenario significantly exceeding the design. The Takatori record (Kobe, Japan 1995) is used as seismic excitation. As seen in Figure 5, the Takatori record is a quite adverse case seismic event: the maximum recorded acceleration was 0.61g, while their spectral values substantially exceed the design accelerations of the pier throughout the entire period range.

In Figure 6 a comparison between the response of the isolated pier using the in-soil isolation system and the response of the conventionally designed pier subjected to the Takatori record is presented. Figure 6a compares the acceleration time histories at the base of the pier for each of the two alternatives. Notice that without the proposed seismic isolation, the pier is subjected to a maximum acceleration of almost 1 g. On the other hand, the favorable effect of the application of the in-soil isolation system becomes apparent, since in that case the pier is subjected to maximum acceleration of only 0.35 g at its base. This decrease in the maximum acceleration may not be adequate to reduce the required reinforcement of the pier, yet it proves to be salutary for the survival of the pier.

As depicted in Figure 6b, where the bending moment–curvature response at the base of the pier is presented, plastic hinging quickly forms at the base of the pier, leading to intense accumulation of curvature, than in turn causes the pier to exhaust its ductility capacity and ultimately to collapse. In stark contrast, the seismically isolated pier may reach the moment capacity, yet there is no significant inelastic response, indicating that the pier remains almost intact after the end of the excitation. Finally, in Figure 6c the time histories of deck drift *Δ* are presented. The conventionally designed pier accumulates horizontal offset towards the one direction and ultimately collapses. On the other hand, the pier founded on the in-soil seismic isolation system survives this extremely strong seismic scenario with maximum drift during the excitation Δ = 0.1 m, and consequently with limited if any damage. In summary, the in-soil seismic isolation system proves to be an effective measure of fuse mechanism, in case of an extreme seismic loading, preventing pier collapse.

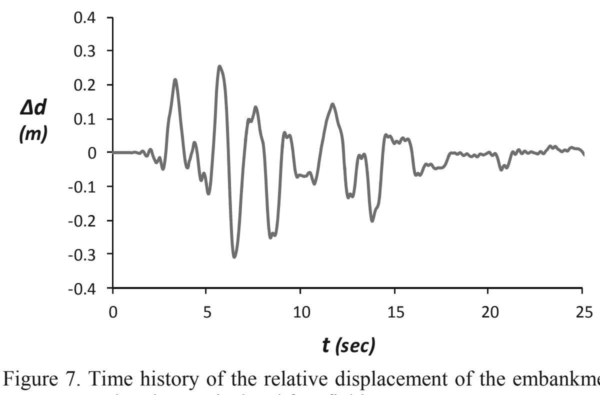

Figure 7. Time history of the relative displacement of the embankment top compared to the non isolated free field.

The beneficial function of the in-soil isolation system comes, however, with a drawback. The system is designed to impose a cut-off at the acceleration that is transmitted to the superstructure, materialized through embankment sliding. This means that excessive slip displacement may occur at the synthetic liner layer that translates to significant relative displacement of the structure compared to the non isolated free-field soil surface. This may be of importance, especially for long structures, such as bridges, where the superstructure is founded on several supports that cannot be isolated at the exactly the same manner. In Figure 7, the time history of the relative diplacement of the empbankment surface compared to that of free field is presented. During this admittedly excessively strong seismic shaking, the embankment is subjected to a significant relative dispalcment compared to the non isolated free-field, with a maximum diplacement *Δd* = 0.3 m. Although such a diffrential displacemnet may be tolerable, it has to be carefully taken into account during design.

5 CONCLUSIONS

The main conclusions of this study can be summarized as follows:

- The application of the in-soil isolation system proves to have a rather beneficial effect on the seismic perfromance of the superstructure (at least for the idelaized bridge pier examined herein). Although the decrease of the maximum acceleration that is transmitted to the superstructure is not adequate to allow the design of the pier for reduced seismic loads, it proves to quite effective in ensuring its survivability.
- The effectiveness of the isolation system depends on the presence of the superstructure. The sliding surface is curved due to the pier imposed additional stresses, demanding in this case from the isolated embankment to slide on an inclined surface rather than a horizontal one. As a result, the acceleration needed for the slip displacement to occur increases, rendering the isolation system less effective
- Since this isolation system relies on slip displacement at the base of the isolated embankment to impose a cut-off at the transmitting onto thesuperstructure accelerations, significant relative to the non isolated free field should be expected and taken into account during design.

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