Rocking of Inelastic Frame on Two-Layered Inelastic Soil

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ABSTRACT: The paper studies the response of a simple rocking-isolated 1-bay 2-storey frame on two-layered soil profile consisting of a stiff surface layer overlying a weak homogeneous soil stratum. Analyses were conducted employing the finite element code ABAQUS and involved monotonic and cyclic push-over tests and dynamic time-history analyses. It was shown that the existence of even a shallow surface layer of depth equal to the footing width enhances the seismic performance of the frame-foundation system by reducing the residual settlements and limiting the extent of damage in the structural members.

1 INTRODUCTION

According to the present capacity design principles, the foundation is designed so as to behave elastically even under extreme earthquake shaking. This is typically achieved by imposing conservative factors of safety against all possible "failure" modes such as mobilization of bearing-capacity, uplifting and/or sliding on the supporting ground. Nevertheless, a growing population of researchers suggest the need to relax some of the aforementioned criteria and allow for a design that accounts for, or even promotes inelastic action at the foundation level. Following this reasoning, the idea of "rocking isolation" [Mergos and Kawashima 2005] has recently been proposed as an alternative seismic design philosophy in which soil failure is not only not prohibited but is rather used as a "fuse": the foundation is deliberately *"under-designed"* to promote rocking, thus limiting the inertia forces transmitted onto the superstructure. The potential effectiveness of such a design scheme has been explored analytically by, among others [Anastasopoulos et al., 2010] and experimentally [Anastasopoulos, 2010; Drosos et al., 2011] for an idealized *RC* bridge pier, and for idealized 2-storey *RC* frame structures [Gelagoti et al., 2011a; 2011b].

For the particular frame examined by Gelagoti (2010) which was founded on clay with undrained shear strength S_u = 150 kPa, it was testified that the new design method was beneficial in terms of settlement and floor drift especially in case of an earthquake far exceeding the limits set by the current design codes. The new design method was proven to be particularly effective when a safety factor against vertical loads of greater than $FS_v \approx 5$ is ensured: foundation rocking prevails against soil yielding thus reducing the residual settlement and rotation of the footing. Still, however, for lower FS_v values, collapse may be avoided –although at the cost of increased distortion.

Despite this quite encouraging outcome, question still remains as to the generalization potential of the results to less idealized cases, such as practical applications where the exact soil properties cannot be a priori guaranteed thus jeopardizing the applicability of rocking isolation. In an effort to overcome this obstacle, this paper investigates the potential of *shallow* soil improvement, a concept commonly applicable in geotechnical engineering as a means to increase soil strength and reduce settlements. The adequacy of shallow only mitigation stems from the nature of foundation rocking which mobilizes only a shallow stress bulb within the soil layer.

2 METHODOLOGY AND NUMERICAL MODELING

A rather extreme scenario is considered hereafter in order to examine the adequacy of shallow soil improvement. It is tactically assumed that the frame investigated by *Gelagoti et al* (2010) is founded on soil of undrained shear strength S_u = 50 kPa yielding a mere $FS_V \approx 2.6$ instead of $FS_V \approx 5$ (when S_u =150 kPa) which was found to be necessary in order to promote efficient uplifting. The effect of applying shallow soil improvement on the low FS_v profile is subsequently examined by parametrically varying its depth d (expressed as a ratio of the foundation width B). The following sections compare the behavior of the frame under various loading scenarios, considering the FS_v = 5 condition as the target scenario. In case of two-layered profiles, the improved layer's strength has been considered equal to S_u = 150kPa, while the underlying one was maintained at S_u = 50kPa (Figure 1).

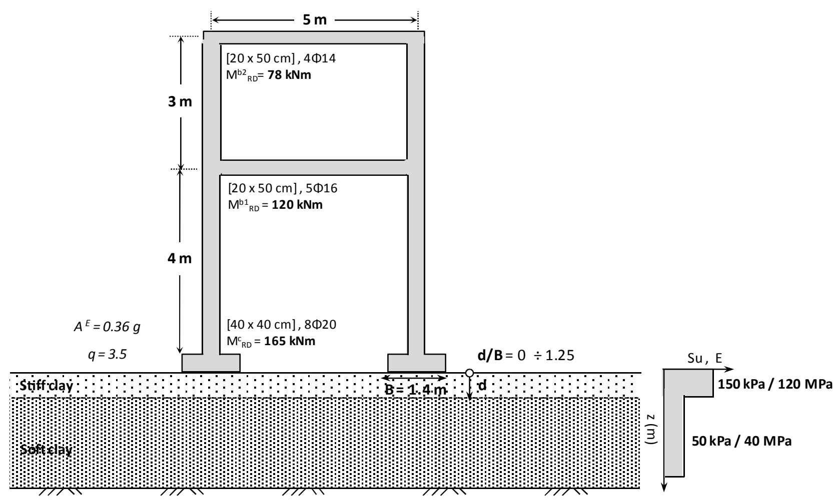

Figure 1. Geometry and member properties of the problem analyzed

Analyses have been performed utilizing the FE code ABAQUS (Figure 2). Soil is modeled with nonlinear quadrilateral continuum elements, assuming plane strain conditions. The soil foundation interface is modeled using special interface elements which allow both detachment and sliding. The seismic excitation (i.e acceleration time history) is applied at the base of the model. Free field boundaries are used at the two lateral boundaries of the model. Nonlinear soil behavior is modeled through a simple kinematic hardening model with Von Mises failure criterion, and associated flow rule. The model capability to effectively capture the rocking response of foundations has been validated against centrifuge model tests by Anastasopoulos et al. (2011). Non linear 2-D beam elements have been used for the modeling of RC beams and columns of the frame. The reinforced concrete constitutive model was the same as the one used for the soil, after proper adaptation, to simulate the non-linear moment–curvature response of the superstructure reinforced concrete members. In order to effectively capture the RC sections behavior, the model parameters are calibrated against moment-curvature relationship computed through section analysis in the X-tract 2000 software in accordance with the details of Vintzilaiou et al. (2007).

3 EFFECT OF SOIL IMPROVEMENT ON THE FOOTING'S BEARING CAPACITY

A series of initial vertical monotonic push-down tests were performed in order to calculate the safety factor against vertical load for all the subsequent analyses, including those on homogeneous soil as well the layered profiles to be discussed in the following sections. As explained previously, foundation rocking (and hence rocking-isolation) may materialize through the reduction of foundation dimensions. However, even once foundation rocking is ensured, the latter will respond to strong ground shaking either trough uplifting from the supporting soil when the factor of safety against vertical load FS_v is large, or by sinking due to excessive soil yielding in case of lower FS_V values. This may result in large residual displacements possibly unacceptable for the design. Evidently, ensuring an adequately large FS_V in order to promote uplifting, presumes that soil properties are known; a rather overoptimistic assumption in engineering practice. Shallow soil improvement provides a reasonable means of overcoming the ambivalence that such uncertainties cause to design by ensuring well known soil properties within the top soil layer. Based on the reasoning of the previous section, it is rational to expect that although the improved properties of the top layer are not adequate to increase the global factor of safety against vertical loads, they will definitely assist the desired uplifting response of the shallow foundation.

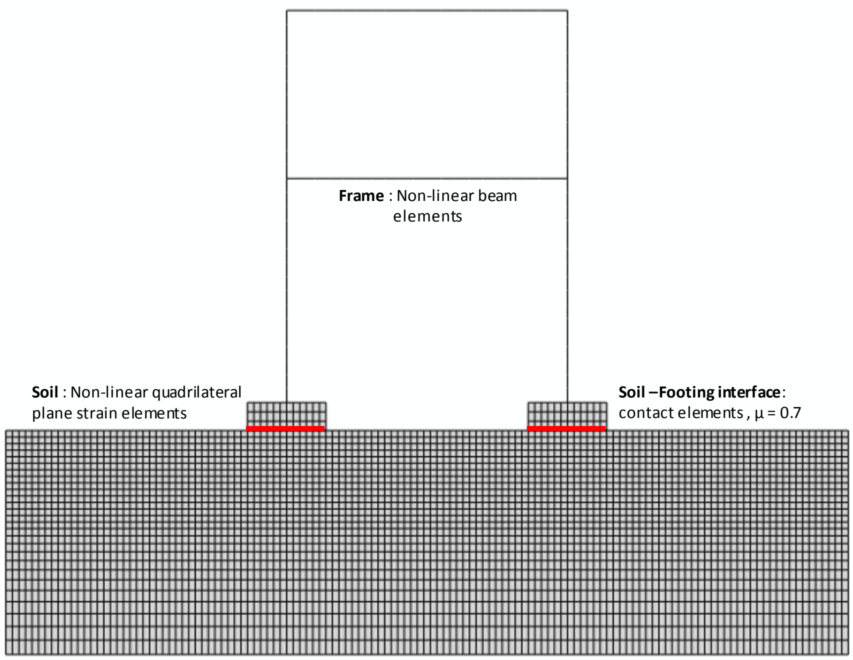

Figure 2. Finite element model

Figure 3 shows the corresponding safety factors versus the thickness of the improved surface layer for the soil profiles discussed previously. As expected, increasing the depth of the improved layer results in increased foundation capacity. It seems that a significant increase in safety factor for vertical loads takes place even when the thickness of the improved layer is relatively small. The rate of increase of FS_v is initially high, but gradually decreases for larger values of the d/B ratio. This drop in increase rate can be explained by the fact that the stress bulb produced by vertical loading is enclosed within the improved layer (for $d / B > 1$) and thus further increase of the depth of the improved layer has no influence on the failure load.

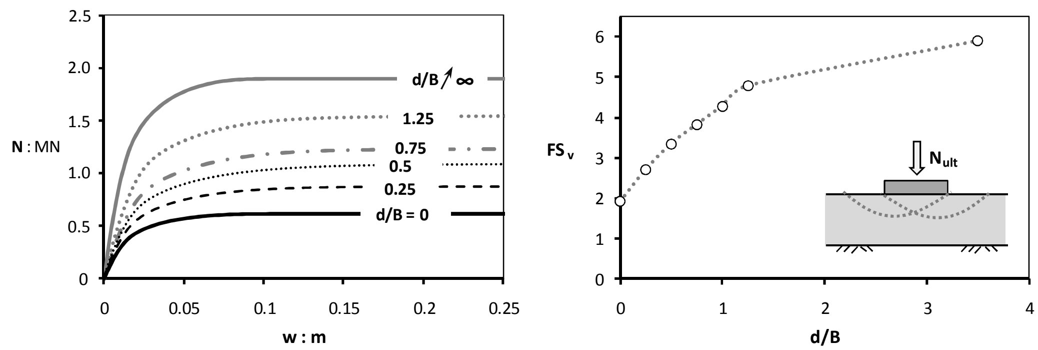

Figure 3. Evolution of Foundation Bearing Capacity and Safety factor against vertical loads with increasing depth of mitigation zone

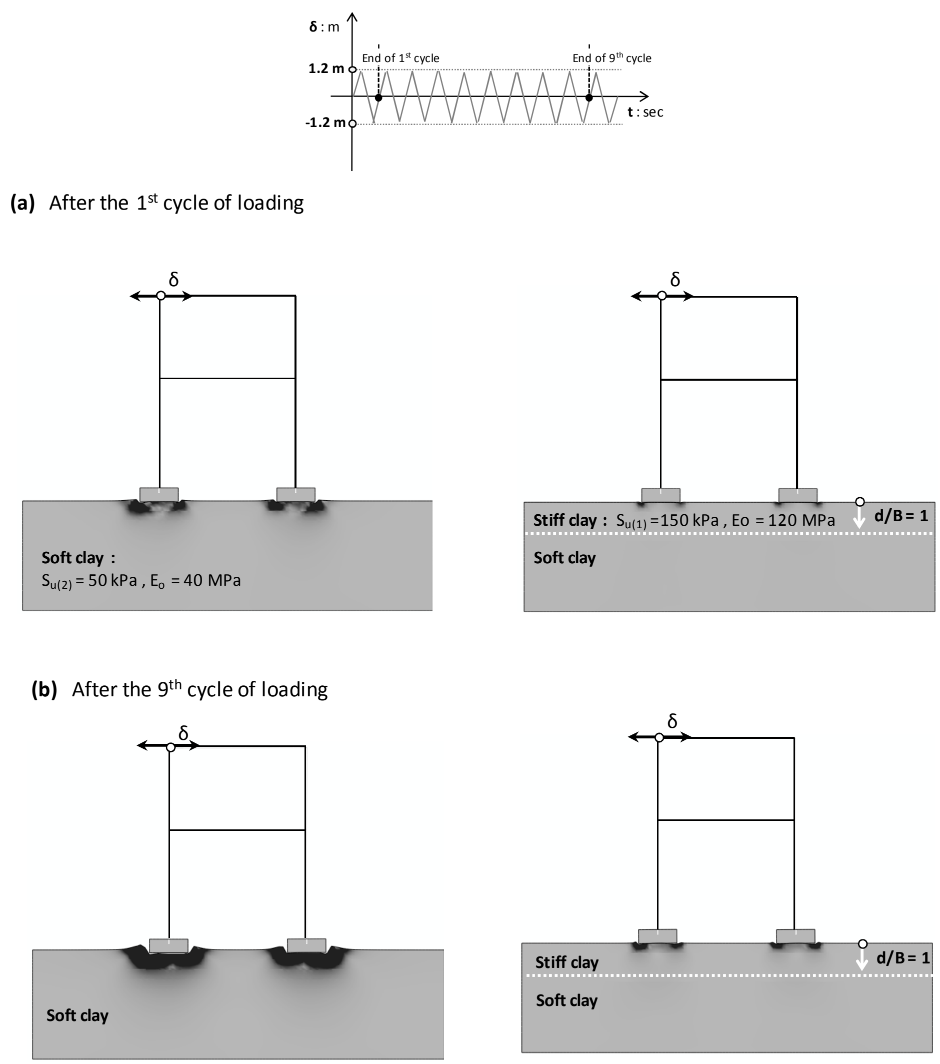

Figure 4. Frame subjected to slow cyclic horizontal loading: comparison of the distribution of plastic deformations produced after (a) the first and (b) the ninth cycle of loading

4 FRAME SUBJECTED TO LATERAL LOADING

4.1 *Response to monotonic and Cyclic Loading*

Initially, the models have been subjected to slow cyclic displacement-controlled push–over loading in the horizontal direction. Displacement is imposed on the upper left node of the frame, and consists of 10 cycles of amplitude δ = 1.2 m. This value corresponds to 75% δ_u , where δ_u is the toppling displacement of the particular frame.

Figure 4 compares the response after the 1st and after the 9th cycle in terms of contours of produced plastic strains for the two examined systems: (a) homogeneous soil with S_u = 50 kPa and (b) two-layered with a surface layer of thickness d / B = 0.5 and undrained shear strength S_{u1} = 150 kPa. Apparently, the existence of the improved zone drastically reduces the plastification underneath the footings (Fig. 4a) while it limits the rate of settlement accumulation. Even after the 9th cycle of loading, plastification is restricted within the mitigation zone without penetrating the underlying weak soil stratum.

4.2 *Response to moderately strong seismic shaking*

The aim of these analyses was to determine the response of the system under different seismic excitations and, through this procedure, estimate the adequate soil improvement depth. Initially, the frame was subjected to relatively moderate seismic excitations (i.e. within its design limits). In this case, interest is mainly focused in serviceability after the end of the earthquake. Therefore, parameters such as the irrecoverable deformation of the foundation are expected to be crucial in assessing design effectiveness. The response of the frame on improved soil (of depth d / B = 0.5 and 1.0) is compared to its response when founded on:

(a) the unimproved homogeneous soil profile of S_u = 50 kPa (FS_V = 3) and
(b) the "target" case of a competent profile of S_u = 150 kPa ($FS_v \approx 5$).

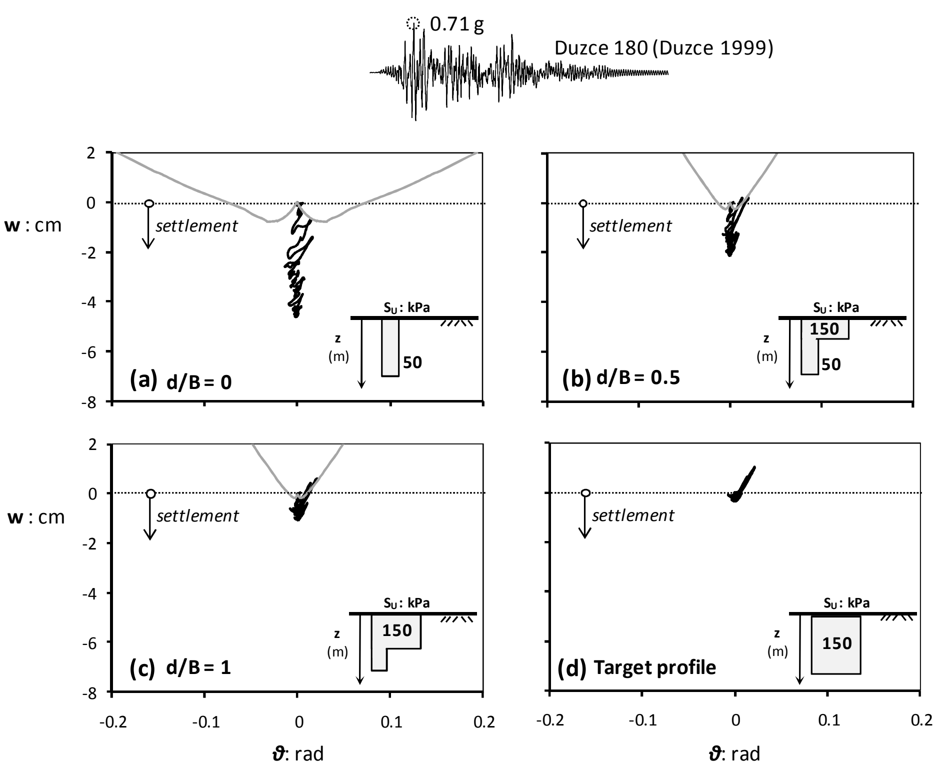

Figure 5. Frame excited by the Duzce 1999 record: comparison of vertical displacement versus rotation (w-θ) loops for the case of (a) homogeneous S_u = 50 kPa , (b) two layered profile

d/B= 0.5 , (c) two layered profile d/B= 1 ; and (d) homogeneous S_u= 150 kPa .

Figure 5 compares the evolution of settlements as a function of rotation angle of the left footing for the four systems examined, when the model is subjected to the Duzce180 record (Duzce, Turkey 1999 earthquake). Indeed, the response of the $FS_V \approx 3$ footing deviates substantially from the target $FS_V \approx 5$ response: the footing accumulates settlement *w* during each strong motion cycle, reaching a peak value of 4.5 cm instead of a mere 0.5 cm in the high FS_v case. Such a high unanticipated settlement under the design earthquake definitely questions the serviceability of the frame and should be avoided. Quite encouragingly, it is seen (Figs. 5b and c) that the use of an improved layer of depth only d/B = 0.5 significantly reduces the settlements, yet not approaching the minimal settlement developed in the target homogeneous profile. The desired behavior is better captured when the improved crust's depth increases to d / B = 1 (Figure 6c), which practically creates the necessary conditions to ensure a rather efficient uplifting response of the foundation.

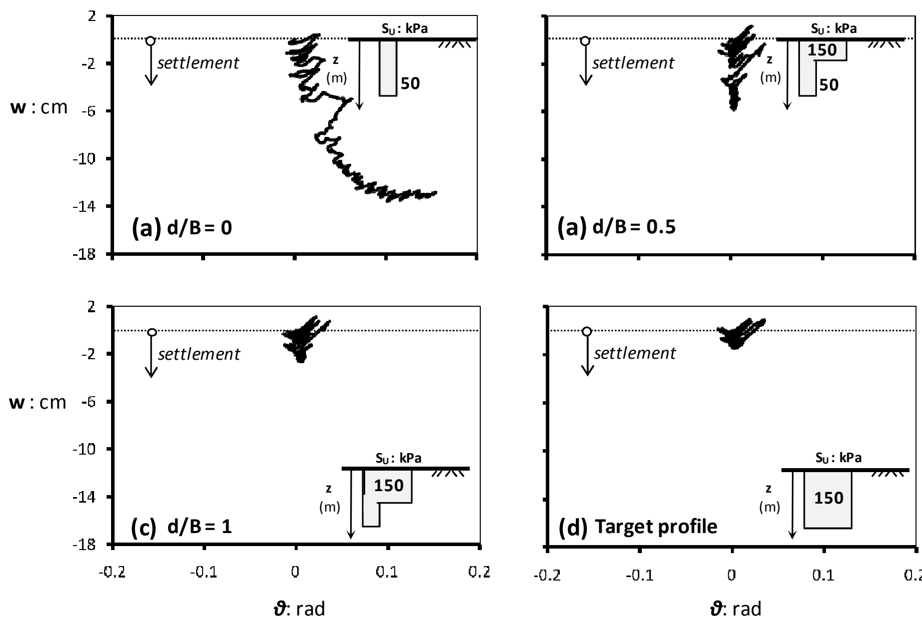

Figure 6. Frame excited by the Tabas 1981 record: comparison of vertical displacement versus rotation (w-θ) loops for the case of (a) homogeneous S_u= 50 kPa , (b) two layered profile d/B= 0.5 , (c) two layered profile d/B= 1 ; and (d) homogeneous S_u= 150 kPa .

4.3 *Response to very strong seismic shaking*

The effectiveness of shallow mitigation becomes palpably more impressive in case of the frame subjected to the Tabas (Tabas, Iran 1981) which overly exceeds the structure's design spectrum. The record is characterized by a multitude of strong motion cycles while its PGA exceeds 0.81 g. The evolution of settlements as a function of the rotation angle when the frame is founded on improved soil is illustrated in Figures 7b and c. In case of the weak (FS_v = 3) profile, the under-designed footings of the frame accumulate severe differential settlement (reflected in the developed rotation) which gradually causes the frame to practically collapse. Apparently, the sequence of many strong motion cycles produces significant plastification extending to large soil depths which, in turn, brings about irrecoverable foundation (and structural) distortion. The beneficial effect of using an improved surface layer with depth ratio just d / B = 0.5 in preventing the collapse of the building becomes obvious: it limits extent of soil yielding and aborts the development of permanent rotation which is responsible for the distortion of the superstructure (Figure 7b). The behavior is further improved when the improvement depth is d/B

= 1. The foundation response tends to imitate that of the target ($FS_V \approx 5$) profile. Although the rocking–induced residual settlement of the foundation is higher than in the homogeneous S_u=150 kPa profile (3cm instead of 2cm), it is considered as a relatively fair price to pay.

5 SUMMARY & CONCLUSIONS

The dynamic response of the system has been simulated employing nonlinear dynamic time history analysis. A quite comprehensive database of 20 recorded time-histories was used as input to assess the seismic performance of the systems under different earthquake scenarios. The selected records incorporate the effect of a wide range of strong-motion parameters such as *PGA*, *PGV*, *SA*, *SV*, frequency content, number of strong motion cycles, duration.

Figure 7 displays comparative collective results of the settlement for the left footing. Obviously, the use of a surface layer of depth only d / B = 0.5, significantly reduces the residual settlement for all seismic excitations although the target behavior of the homogeneous S_u = 150 kPa case is not perfectly imitated. Further increase of the soil improvement depth to d / B = 1 further reduces the residual settlements while the foundation behavior resembles that achieved in case of the target profile.

It is concluded that the use of a shallow improved soil layer of depth d/B = 1 is able to reduce the risk of settlement associated with uncertainties in the proper estimation of soil properties. The use of the improved surface layer has a favorable effect for the majority of the examined seismic records limiting settlement and damage in structural members.

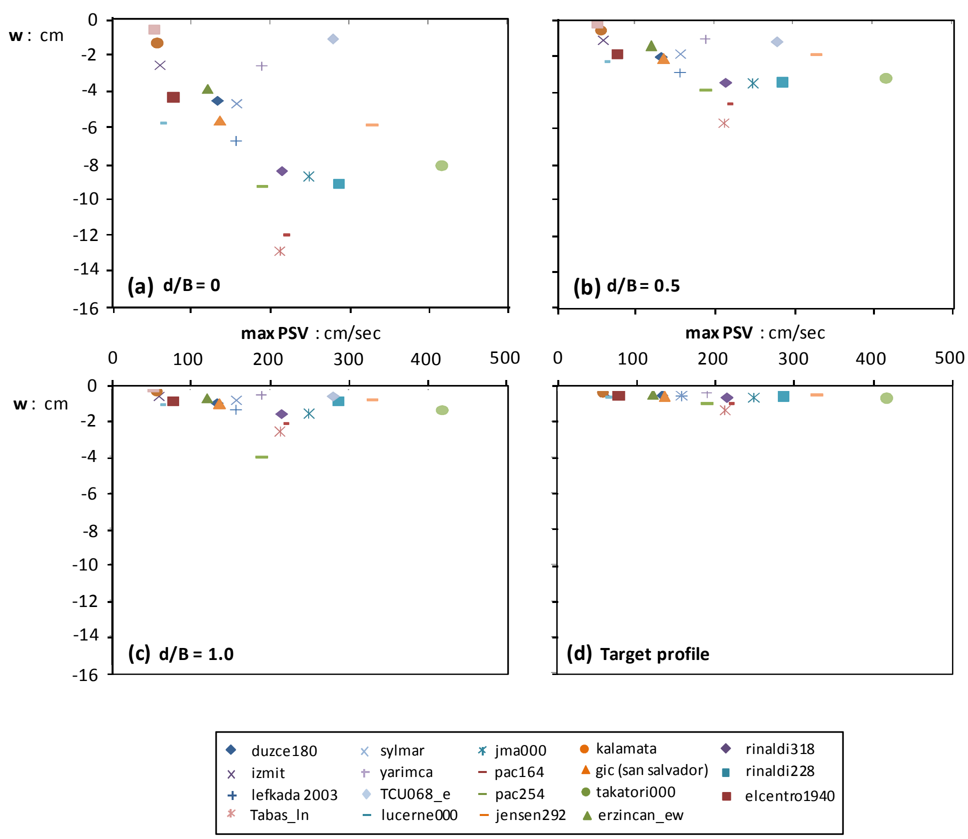

Figure 7. Conclusive results. Comparison of the residual settlement for all investigated earthquake scenarios for all the examined scenarios: (a) homogeneous S_u = 50 kPa , (b) two layered profile d/B= 0.5 , (c) two layered profile d/B= 1 ; and (d) homogeneous S_u = 150 kPa .

6 ACKNOWLEDGEMENT

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REFERENCES

Anastasopoulos I. (2010), “Beyond conventional capacity design : towards a new design philosophy”, In : *Soil–Foundation–Structure Interaction*, Orense R.P., Chouw N., Pender M.J. (editors), CRC Press, Taylor & Francis Group : New York.

Anastasopoulos I., Gazetas G., Loli M., Apostolou M, Gerolymos N. (2010) “Soil failure can be used for seismic protection of structures”, Bull. Earthquake Eng. Vol.8, pp. 309-326

Anastasopoulos I., Gelagoti F., Kourkoulis R., G. Gazetas, (2011) “Simplified Constitutive Model for Simulation of Cyclic Response of Shallow Foundations : Validation against Laboratory Tests”, Journal of Geotechnical and Geoenvironmetal Engineering, ASCE (in print)

Drosos V., Georgarakos T., Loli M., Anastasopoulos I., Zarzouras O., and Gazetas G. (2011), “Soil–Foundation–Structure Interaction with Mobilization of Bearing Capacity : An Experimental Study on Sand”, *Journal of Geotechnical and Geoenvironmental Engineering,* ASCE (*submitted for possible publication*).

Gelagoti F. (2010), “Metaplastic Response and Collapse of frame-Foundation Systemss, and the Concept of Rocking Isolation” Doctoral Thesis, NTUA.

Gelagoti F., Kourkoulis R., Anastasopoulos I., and Gazetas G. (2011a) “Rocking Isolation of Frame Structures Founded on Isolated Footings”, Earthquake Engineering and Structural Dynamics (in print)

Gelagoti F., Kourkoulis R., Anastasopoulos I., and Gazetas G. (2011b), “Rocking‒isolated Frame Structures : Margins of Safety against Toppling Collapse and Simplified Design Approach”, Soil Dynamics and Earthquake Engineering (in print)

Mergos, P. E., Kawashima, K. (2005). "Rocking isolation of a typical bridge pier on spread foundation." *Journal of Earthquake Engineering*, 9(2): pp 395-414.

Vintzilaiou E.,Tassios T.P., Chronopoulos M. (2007) “Experimental validation of seismic code provisions for RC columns”, Engineering Structures, 29, pp1153-1164