Paper No. xxxx

SIMSG ISSMGE

SECOND INTERNATIONAL CONFERENCE ON PERFORMANCE-BASED DESIGN IN EARTHQUAKE GEOTECHNICAL ENGINEERING

May 28-30, 2012 - TAORMINA (ITALY)

EFFECTIVENESS OF SHALLOW SOIL IMPROVEMENT ON THE PERFORMANCE OF ROCKING-ISOLATED BRIDGE PIERS : MONOTONIC AND CYCLIC PUSHOVER TESTING

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ABSTRACT

An alternative seismic design philosophy, in which soil failure is used as a "fuse" for the superstructure has recently been proposed, in the form of "rocking isolation". Within this context, foundation rocking may be desirable as a means of bounding the inertia forces transmitted onto the superstructure, but incorporates the peril of unacceptable settlements in case of a low static factor of safety FS_v . Hence, to ensure that rocking is materialized through uplifting rather than sinking, an adequately large FS_v is required. Although this is feasible in theory, soil properties are not well-known in engineering practice. However, since rocking-induced soil yielding is only mobilized within a shallow layer underneath the footing, "shallow soil improvement" is considered as an alternative approach to release the design from the jeopardy of unforeseen inadequate FS_v. For this purpose, this paper studies the metaplastic rocking response of SDOF structures, with emphasis on the effectiveness of shallow soil improvement stretching to various depths below the foundation. A series of reduced-scale monotonic and slow-cyclic pushover tests are conducted on relatively slender SDOF systems lying on a square surface foundation. It is shown that shallow soil improvement may, indeed, be quite effective provided that its depth is equal to the width of the foundation.

Keywords: rocking isolation; physical modeling; nonlinear soil response; soil improvement

INTRODUCTION

Current seismic design principles aim at *guiding* failure to superstructure elements, prohibiting mobilization of foundation bearing-capacity, uplifting and/or sliding (e.g., Priestley et al., 1996). However, recent research findings suggest that strongly nonlinear *soil–foundation response may be beneficial*, and should be considered in design (e.g. Paolucci, 1997; Pecker, 1998; 2003; Gazetas et al., 2003; Gajan et al., 2005; Kawashima et al., 2007). Soil nonlinearity may act as a "fuse", dissipating seismic energy and reducing the ductility demand exerted on superstructure elements (Anastasopoulos et al., 2010; Gelagoti et. al., 2011a). Such findings have been corroborated experimentally, through centrifuge, large scale, and reduced-scale physical model tests (Negro et al., 2000; Faccioli et al., 2001; Kutter et al., 2003; Gajan et al, 2005; Gajan & Kutter, 2008; 2009; Paolucci et al., 2008; Shirato et al., 2008).

Based on such findings, an alternative seismic design philosophy has recently been introduced, termed "rocking isolation" by Mergos & Kawashima (2005), according to which soil failure is used as a "fuse" for the superstructure (Anastasopoulos, 2010). In contrast to conventional capacity design, the foundation is intentionally "*under-designed*" in order to limiting the inertia forces transmitted onto the superstructure

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(**Figure 1**). The potential of such design philosophy has been investigated through numerical (Anastasopoulos et al., 2010a) and experimental simulation (Anastasopoulos, 2010; Drosos et al., 2011) for a typical bridge pier, and numerically (so far) for low-rise buildings (Gelagoti et al., 2011a; 2011b). It has been shown that rocking isolation may increase substantially the safety margins against collapse, with increased settlement and foundation rotation constituting the price to pay. When the safety factor against static (vertical) loads FS_v is relatively large, foundation response is uplifting–dominated, and seismic settlement can be tolerable. In marked contrast, the response becomes sinking–dominated in case of lower FS_V, leading to excessive soil yielding and accumulation of settlement. Hence, an adequately large FS_V is required to promote uplifting-dominated response and maintain settlement within acceptable limits.

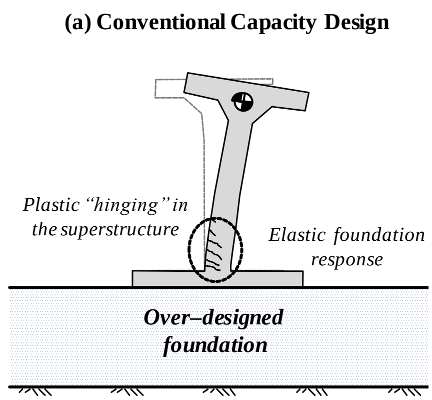

Figure 1. (a) Conventional capacity design, compared to (b) rocking isolation.

However, FS_V is a function of soil properties, which are not always accurately known. Aiming to overcome such obstacles, this paper explores the effectiveness of *shallow* soil improvement in the context of rocking–isolation. The idea stems from the very nature of foundation rocking, which tends to mobilize only a very shallow stress bulb within the soil. As shown in **Figure 2**, which plots the distribution of vertical stresses for two single degree of freedom systems (SDOF) subjected to combined *M-Q* loading (as computed through nonlinear finite element analysis), for a relatively large FS_v = 10 the soil below a shallow depth $z/B > 0.5$ is not affected by the rocking-generated vertical stresses. Even for FS_v = 12, in which case the rocking mechanism is unavoidably deeper, the vertical stresses are substantially affected at depth $z/B \leq 1$.

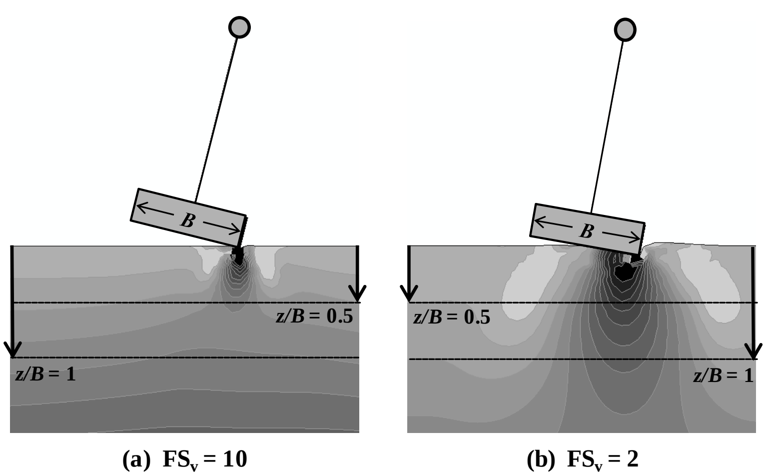

Figure 2. Contours of vertical stresses of a SDOF system subjected to combined M-Q loading, as computed through numerical analysis, for : (a) FSv = 10, and (b) FSv = 2.

This paper presents the results of reduced-scale monotonic and slow-cyclic pushover tests, conducted in the Laboratory of Soil mechanics of the National Technical University of Athens (NTUA), aiming to investigate the effectiveness of shallow soil improvement. In the context of rocking isolation, the investigated SDOF models are tested well beyond their moment capacity, into their post-peak metaplastic regime.

PROBLEM STATEMENT AND EXPERIMENTAL SETUP

A relatively slender SDOF system of h/B = 3 founded on a surface square foundation of width B is studied, considered representative of a bridge pier (B = 6 m in prototype scale) or a column of the of a low-rise building (B = 1.5 m in prototype scale). The investigated soil-structure configurations are illustrated in **Figure 3**. Focusing on foundation performance, no attempt was made to model the stiffness of the superstructure, which is assumed (relatively) rigid and elastic. The results presented herein refer to a relatively lightly loaded structure (characterized by a relatively large FS_v), the performance of which is uplifting-dominated under ideal soil conditions. Three different soil profiles were investigated: (a) D_r = 93% dense sand, which is representative of ideal soil conditions ; (b) D_r = 45% loose sand, representing poor soil conditions ; and (c) shallow soil improvement by means of a dense sand "crust", of varying depth z/B = 0.25 to 1.

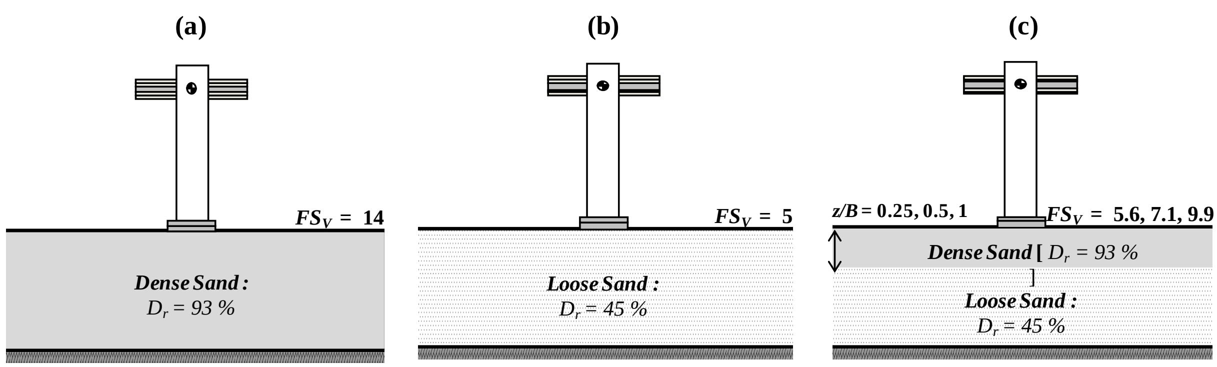

Figure 3. Sketch of the tested soil-structure systems: (a) dense sand, representing ideal soil conditions ; (b) loose sand, representing poor soil conditions ; and (c) shallow soil improvement with a "crust" of dense sand, of varying depth z/B = 0.25 to 1.

Soil Modeling

The soil specimens were prepared using dry Longstone sand. As documented in Anastasopoulos et al. (2010b), the utilized material is an industrially produced fine and uniform quartz sand, having D_{50} = 0.15 mm characterized by a uniformity coefficient C_u = 1.42. The sand was pluviated in a rigid soil container utilizing an in-house sand raining system, which is capable of producing specimens of controllable relative density D_r, achieving excellent repeatability. The system has been thoroughly calibrated, as described in detail in Anastasopoulos et al. (2010b). To avoid possible scaling-related misinterpretations, a series of vertical push-down tests were conducted to measure the bearing capacity of the B = 15 cm square foundation used in the tests (assuming a 1:40 scale in the case of the bridge pier, or 1:10 for the low-rise building), for all soil profiles examined. As summarized in **Table 1**, based on these tests the mass of the model was adjusted to produce the desired FS_v = 14 for the ideal case of dense sand.

Table 1. Measured bearing capacity of the *B* = 15 cm square foundation, for the investigated homogeneous, and two-layered soil profiles.

	Configuration	N_{ult} (kN)	N_{ult} (kN)
Ideal Soil Conditions	Dense sand	4.83	14.1
Poor Soil Conditions	Loose sand	1.70	5.0
Soil Improvement	z/B = 0.25	1.95	5.6
	z/B = 0.5	2.45	7.1
	z/B = 1.0	3.40	9.9

Modeling of the SDOF System

As illustrated in **Figure 4**, the foundation–structure model consists of a square footing of width B = 15 cm, rigidly connected to a pair of rigid steel columns, supporting the superstructure mass. The latter consists of steel plates, installed above and below a rigid aluminum slab, positioned at height h = 45 cm (so that h/B = 3). The mass of the model is adjusted so as to achieve the desired FS_v = 14 for ideal soil conditions. The physical model was installed inside a rigid soil container, on an adequately deep (at least with respect to rocking) sand layer. The installation of the SDOF system on top of the soil layer was performed using four mechanical jacks, enabling accurate positioning and minimum soil disturbance.

Loading and Instrumentation

A screw-jack actuator was utilized to apply the desired horizontal displacement at the center of mass of the SDOF system (**Figure 4**). The actuator was rigidly attached to a reaction wall (left), with its free end (right) connected to the foundation-structure model using a vertical slider and a hinged connection in series. This way, the system is allowed to freely settle, slide, and rotate during application of the horizontal displacement. The applied load was measured through a load-cell, inserted between the slider and the hinged connection. Displacements were measured through wire and laser transducers, as sketched in **Figure 4**.

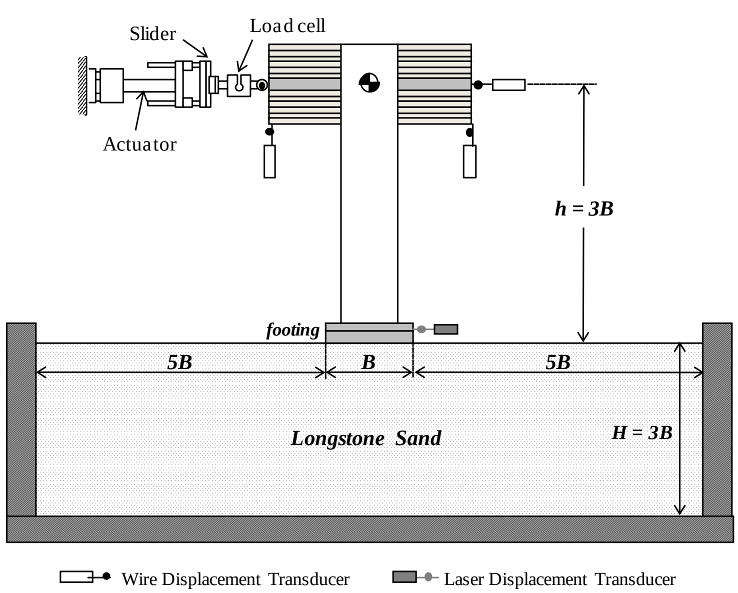

Figure 4. Experimental setup and instrumentation.

The model configurations of **Figure 3** were subjected to *monotonic* and *slow-cyclic* pushover loading. In the latter case, two different cyclic loading protocols were implemented. As shown in **Figure 5**, Type 1 load protocol consists of 14 cycles of increasing displacement δ/δ_R (where δ_R = 7.5 cm is the toppling displacement of the equivalent rigid block) ranging from 0.025 to 0.55. Type 2 consists of 31 cycles, divided into five packets of increasing amplitude: (i) 10 cycles of δ/δ_R = 0.05, (ii) 10 cycles of δ/δ_R = 0.10, (iii) 5 cycles of δ/δ_R = 0.20, (iv) 3 cycles of δ/δ_R = 0.30, and (v) 3 cycles of δ/δ_R = 0.55.

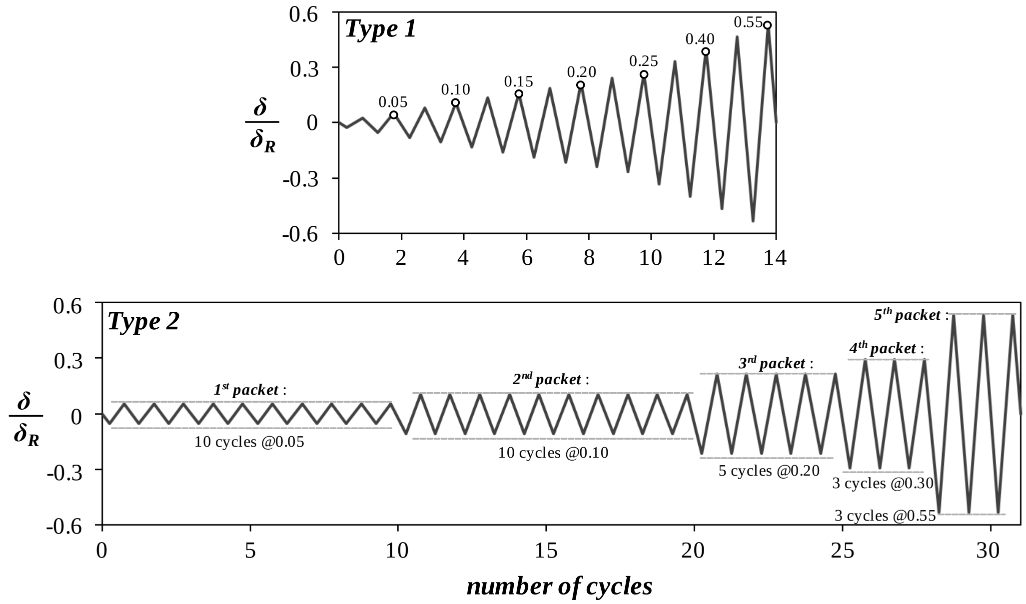

Figure 5. Displacement protocols used for cyclic loading tests, normalized to the toppling displacement δ_R of the equivalent rigid block.

THE EFFECT OF SOIL CONDITIONS ON ROCKING RESPONSE

A first set of experiments was conducted to investigate the effect of soil conditions on the rocking response of the investigated *SDOF* systems. For this purpose, the foundation-structure system was subjected to monotonic and slow-cyclic pushover loading, founded on dense D_r = 93% sand, representing ideal soil conditions, and loose D_r = 45% sand representing poor soil conditions.

The performance of the SDOF system subjected to cyclic pushover loading is summarized in **Figure 6**, in terms of moment-rotation and settlement-rotation response. Following Gajan & Kutter (2008), moment and rotation are normalized to the overturning moment $M_R = mgB/2$ = 0.026 kNm and to the toppling rotation $\theta_R = B/2h$ = 0.167 rad; the settlement w is normalized to the footing width B = 0.15 m. At this point, it should be noted that the descending branch in the moment–roatation plot is due to P–δ effects. As it would be expected, the normalized moment capacity and the overturning rotation are substantially higher for ideal soil conditions (dense sand), compared to poor soil conditions (loose sand).

The considerable difference in FS_v (14 for dense sand, as opposed to 5 for loose sand) is also reflected on the measured moment-rotation loops. While for FS_v = 5 the loops demonstrate an oval shape, with FS_v = 14 they become more *S-shaped*. The FS_V = 14 results compare qualitatively well with experiments on similar, but larger (B = 0.5 m), square footings performed at PWRI (Fukui et al., 2005). As expected, the PWRI test loops on $h/B \approx 3$ specimens are more *S-shaped* than those produced here since those tests have been conducted at an impressive $FS_{V,H} \approx 30$, where foundation uplifting was prominent. On the other hand, for the case of $FS_{V,L} \approx 14$ uplifting was naturally more evident in our tests which were conducted on more slender specimens. The most important difference lies in the settlement that is accumulated during cyclic loading. The settlement increases substantially with the reduction of FS_v : subjected to the Type 1 loading protocol, the heavily-loaded *System B* on medium sand (FS_v = 2.6) accumulates almost two times larger settlement compared to the same system on dense sand. Interestingly, even for a remarkably large FS_v = 14 (dense sand) a limited, yet non-negligible, rocking-induced settlement is observed. Nevertheless, this observation is in accord with UC Davis centrifuge model test results (Gajan & Kutter, 2008).

It becomes evident that foundation rocking may be desirable (to limit the inertia forces transmitted onto the superstructure), but unacceptable settlements may take place in case of low FS_v. Therefore, it is crucial to ensure that FS_v is adequately large, so that the response is uplifting–dominated. However, the inherent uncertainties in the exact estimation of soil properties, pose a difficulty in the assessment of FS_v,, practically limiting the applicability of rocking isolation. Still though, since rocking-induced soil yielding is only mobilized within a small depth underneath the footing, *"shallow soil improvement"* (i.e., replacement of a shallow soil layer with soil of known, better, properties) may be quite effective. If successful, this method could release the design from the jeopardy of unforeseen inadequate safety factor (due to an over-estimation of soil properties).

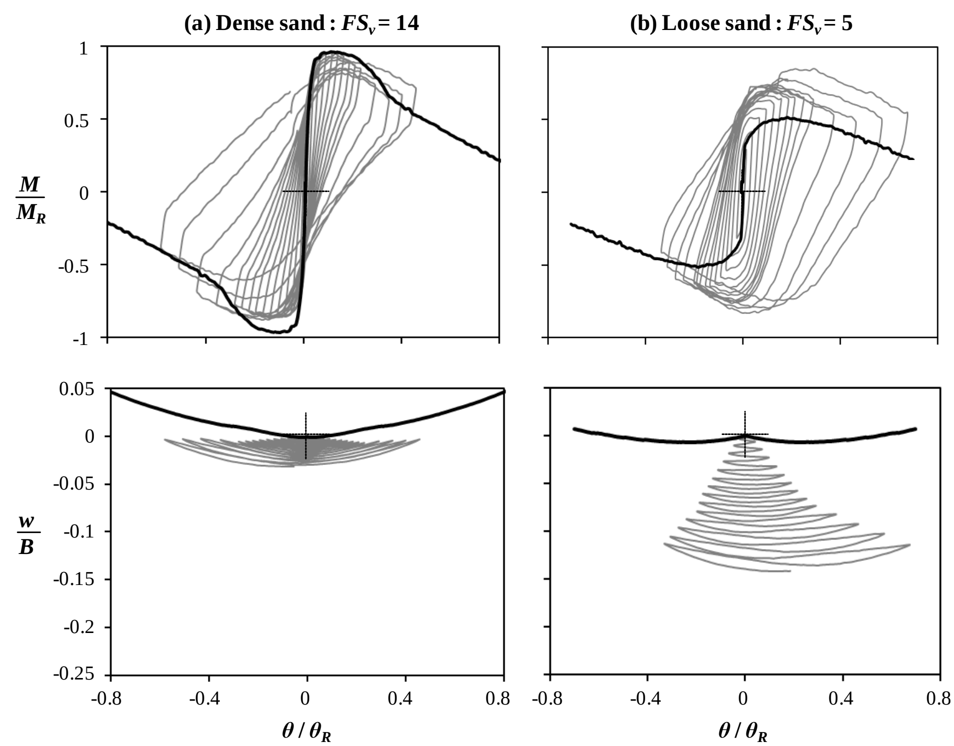

Figure 6. The effect of soil conditions: moment–rotation and settlement–rotation response of the SDOF system on dense and loose sand, subjected to cyclic loading (Type 1 protocol). Moment and rotation are normalized to the overturning moment M_R and the toppling rotation θ_R of the equivalent rigid block ; settlement is normalized to the foundation width B. The solid black lines correspond to the results of monotonic loading.

EFFECTIVENESS OF SHALLOW SOIL IMPROVEMENT

As previously discussed, the depth *z/B* of the shallow soil crust was parametrically varied from 0.25 to 1. With the shallower *z/B* = 0.25 dense sand crust proven to be ineffective, results are presented for *z/B* = 0.5 and 1. In all cases examined, the performance in dense sand is considered as the ideal case.

Monotonic Loading

The effectiveness of shallow soil improvement is summarized in **Figure 7**. In terms of monotonic moment–rotation response (**Figure 7a**), as it would be expected the moment capacity of the foundation is augmented with the increase of the depth *z/B* of soil improvement. A similar trend is observed for the toppling rotation, which also increases with *z/B*. The increase of the thickness of the dense sand crust reduces the extent of soil plastification, which tends to be restricted within the layer of increased strength. As a result, at large rotations the behavior of the *z/B* = 1 alternative is almost identical to the upper bound case of ideal soil conditions (i.e., dense sand).

As shown in **Figure 7b**, the effectiveness of shallow soil improvement becomes more evident in terms of settlement-rotation response. The rotation range where the response is sinking-dominated is represented by the grey shaded areas. Evidently, with the increase of *z/B*, uplifting is promoted for a wider rotation range. While in loose sand the footing settles up to θ/θ_R = 0.54, for *z/B* = 0.5 the sinking threshold drops to θ/θ_R = 0.24, and to 0.04 for *z/B* = 1, becoming almost identical with the ideal soil case of dense sand.

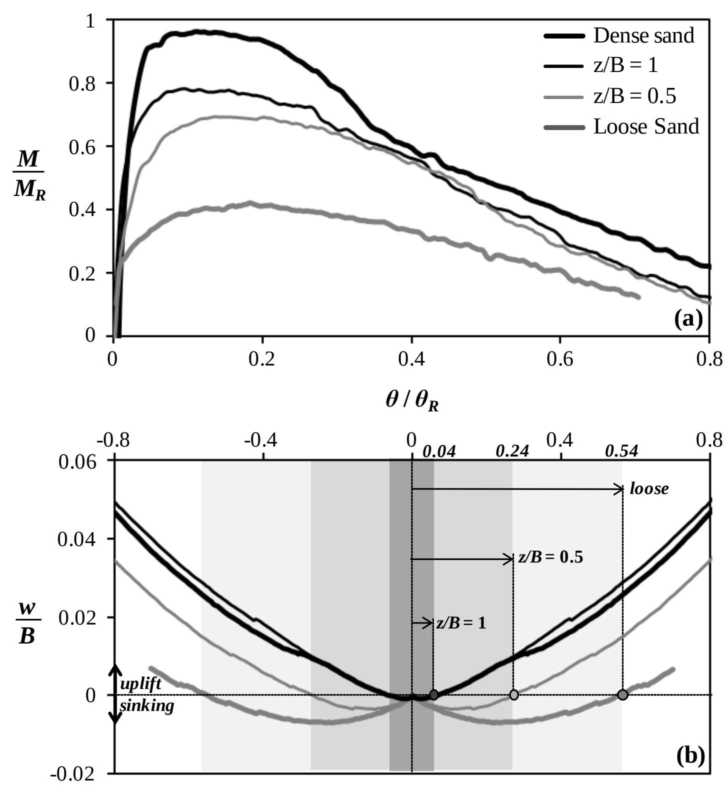

Figure 7. Effectiveness of shallow soil improvement for the SDOF system subjected to monotonic loading. Comparative assessment in terms of : (a) normalized moment–rotation, and (b) settlement–rotation response (highlighting sinking-dominated response for each case).

Moving to larger rotations, where the response is dominated by uplifting, the performance of the models on shallow soil improvement is practically identical to the ideal case of dense sand. Initially (for small θ/θ_R), the foundation is in full contact with the supporting soil, generating a deeper stress bulb, and hence being affected by the underlying loose sand layer. With uplifting initiation, the effective width of the foundation is drastically decreased, leading to a reduction of the depth of the stress bulb. As a result, the rocking-induced stresses become "confined" at smaller depth, enhancing the effectiveness of the dense sand crust.

Cyclic Loading – Type 1 Protocol

Figures 8 and **9** summarize the results of cyclic pushover tests, applying Type 1 load protocol. In terms of moment-rotation response (**Figure 8**), it is evident that as the depth *z/B* of soil improvement increases, the loops tend to transform from oval-shaped (as in loose sand) to S-shaped, resembling the ideal case of dense sand. Interestingly, all systems tend to display similar moment capacity, revealing a substantial increase of overstrength with the decrease of FS_v (see also Panagiotidou et al., 2012).

The effectiveness in terms of settlement-rotation response is summarized in **Figure 9**. The differences are now quite striking. Even for the "shallow" z/B = 0.5 soil improvement, the performance is notably superior to that on loose sand. Although the accumulated settlement is still larger than what is observed in the ideal case of dense sand, the effectiveness of the shallow dense sand crust is undeniable. A deeper z/B = 1 soil improvement is even more effective, leading to practically the same settlement with dense sand.

Hence, it can be argued that a z/B = 1 dense sand crust is enough to achieve practically the same performance with the ideal case of dense sand. A shallower improvement may also be considered as effective, depending on the desired performance and design requirements.

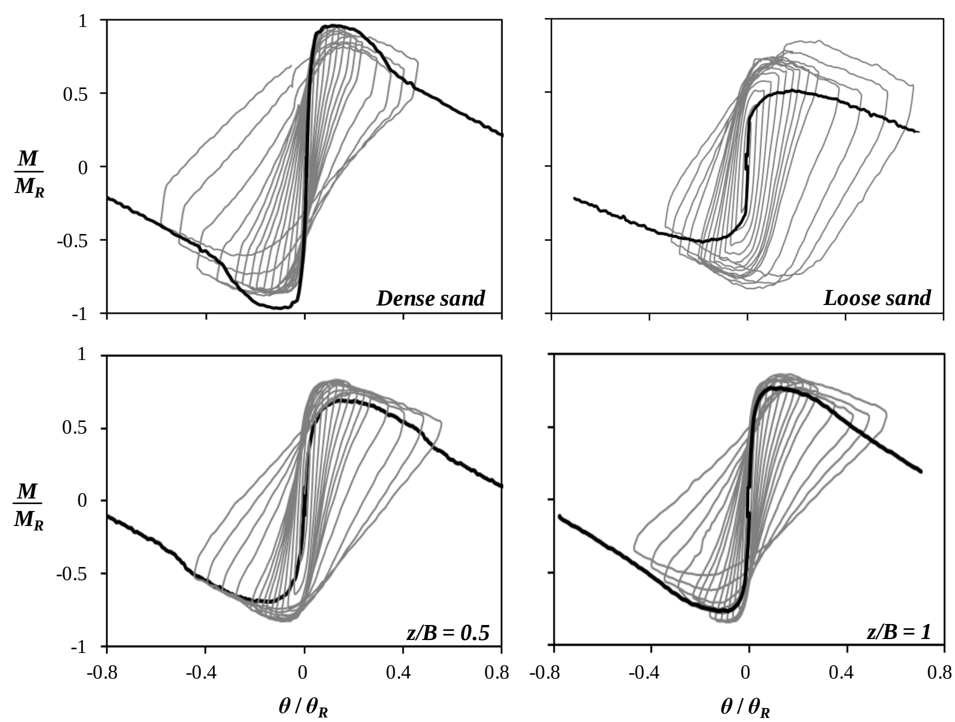

Figure 8. Effectiveness of shallow soil improvement for the SDOF system subjected to cyclic loading, Type 1 protocol. Comparative assessment in terms of moment–rotation response.

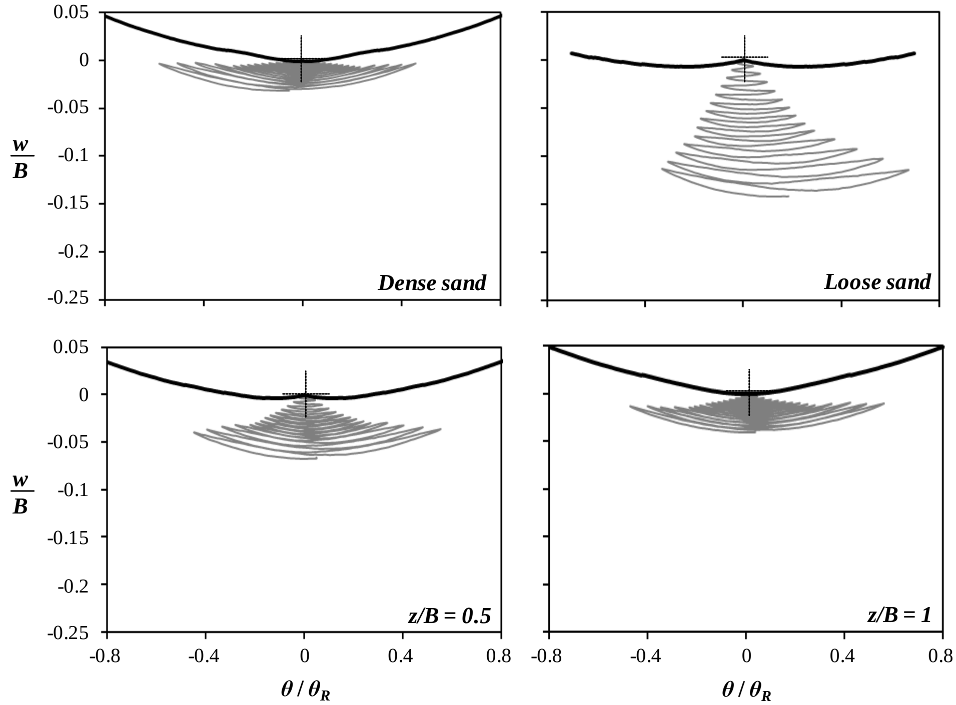

Figure 9. Effectiveness of shallow soil improvement for the SDOF system subjected to cyclic loading, Type 1 protocol. Comparative assessment in terms of settlement–rotation response.

Cyclic Loading – Type 2 Protocol

Type 2 loading protocol is utilized to assess the effectiveness of shallow soil improvement for multiple loading cycles. **Figure 10** depicts the evolution of normalized settlement *w/B* with respect to the normalized imposed rotation θ/θ_R. Although the ultimate displacement amplitude (and hence, rotation) is the same with that of Type 1 loading protocol, in this case strong amplitude cycles are preceded by a multitude of loading cycles of considerably lower imposed displacement (load packets 1 through 4). Hence, Type 2 loading protocol is expected to contain the dual effect of increasing the accumulated settlement and affecting the soil density underneath the footing.

Indeed, the effectiveness of soil improvement is quite impressive, even for this admittedly severe loading protocol. Even with a shallow *z/B* = 0.5 improvement, the accumulated settlement (after 31 loading cycles) is reduced by a factor of almost 3 compared to the case of loose sand. While for the models on loose sand, settlement is accumulated for all levels of imposed rotation, the models on dense sand and improved soil tend to accumulate settlement only during the initial smaller-amplitude cycles. Further increase of the thickness of the dense sand crust to *z/B* = 1 does not seem to reduce the accumulated settlement as remarkably. The observed behavior is in accord with the previously discussed monotonic settlement-rotation response.

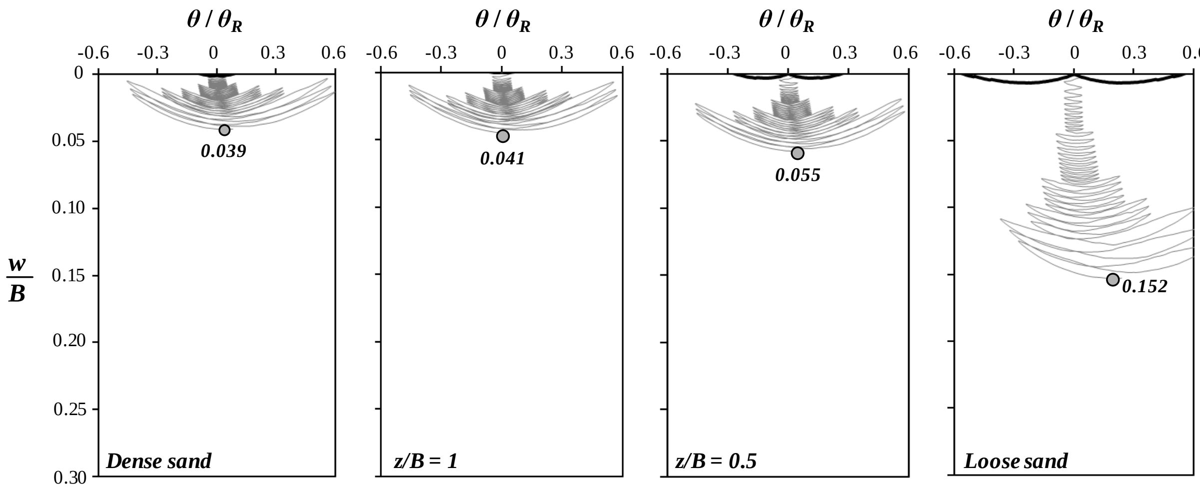

Figure 10. Effectiveness of shallow soil improvement for the SDOF system subjected to cyclic loading, Type 2 protocol. Comparative assessment in terms of settlement–rotation response..

SUMMARY AND CONCLUSIONS

The present paper investigated the rocking response of *SDOF* systems, and the effectiveness of shallow soil improvement stretching to various depths beneath the foundation. For this purpose, a series of reduced-scale monotonic and slow-cyclic pushover tests were conducted at the Laboratory of Soil Mechanics of the National Technical University of Athens. A relatively slender *SDOF* system was studied, lying on a square foundation of width *B.* The investigated foundation-structure system was first tested on ideal and poor soil conditions (dense and loose sand, respectively), to demonstrate the necessity for soil improvement. Then, the effectiveness of shallow soil improvement was studied by investigating the performance of dense sand crusts of varying depth z/B = 0.25 to 1.

At least for the cases examined herein, shallow soil improvement has been proven to be quite effective. In the case of a relatively lightly-loaded system (having FS_v = 14 in ideal soil conditions), such as the one investigated herein, a z/B = 1 dense sand crust is enough to achieve practically the same performance with the ideal case of dense sand. A shallower z/B = 0.5 soil improvement may also be considered effective, depending on design requirements. Hence, it may be conservatively concluded that an improvement layer of depth equal to the foundation width offers a safe solution for practical applications.

AKNOWLEDGEMENTS

The financial support for this paper has been provided under the research project “DARE”, which is funded through the European Research Council’s (ERC) “IDEAS” Programme, in Support of Frontier Research–Advanced Grant, under contract/number ERC–2–9–AdG228254–DARE to Professor G. Gazetas.

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