SHAKING TABLE EXPERIMENTS AND SIMPLIFIED NUMERICAL SIMULATION OF A SHALLOW FOUNDATION TEST MODEL

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ABSTRACT :

The set of experiments carried out at the Public Works Research Institute, Tsukuba, Japan, is a unique opportunity to analyze the seismic behaviour of shallow foundations under seismic loading of various levels of intensity up to complete failure. The experimental set up consists of a shallow foundation test model (0.5m x 0.5m in plan) located at the surface of a laminar shear box (4m x 4m in plan and 2.1 m high), filled in with dry Toyoura sand. The box is shaked at its base with a sequence of excitations of various degree of intensity, representative of the seismic input motion used for seismic design in Japan. In this contribution the experiments are simulated using a simplified approach consisting of a single degree-of-freedom structure founded on a compliant foundation with 3 degrees-of-freedom (vertical, horizontal, rocking). The nonlinear behaviour of the soil-foundation system is allowed by numerical modelling using a nonlinear elasto-perfectly plastic macro-element, defined by a suitable yield surface and plastic flow rule. The results of the numerical analyses are very satisfactory in terms of the accurate simulations of the time history of overturning moments, driving the nonlinear behaviour of the foundation. The simulation of the accumulated settlements and rotations was much more difficult: satisfactory results were obtained by using a suitable stiffness degradation rule as a function of the total plastic rotation accumulated during shaking. These results throw light on the debated issue of the foundation ductile behaviour during strong seismic shaking and on its possible role in the evolution of performance-based design concepts including nonlinear soil-structure interaction.

KEYWORDS: shallow foundations, dynamic soil-structure interaction, shaking table tests, nonlinear macro-element

1. INTRODUCTION

In the framework of seismic design according to capacity principles, it is generally recognized that any damage to the foundation should be avoided. This means that the nonlinear capacity of the system is exploited at the superstructure level alone, typically permitting the energy dissipation at suitably selected points through formation of plastic hinges or insertion of isolation/dissipation devices. This requirement is partly supported by budget considerations, due to the time and economic costs related to the post-earthquake inspection and verification of the foundation system, but it is also justified based on the lack of well established and calibrated methods to study the post-yielding behavior of soil-foundation systems under strong seismic loads.

With the ever increasing interest towards performance based approaches for seismic design, there is now an increasing awareness of the effects of the interaction between the foundation and the superstructure and its role on the overall seismic capacity of the system (see e.g. ATC-40, 1996; Martin and Lam, 2000; Pecker 2006). This awareness is conflicting with the lack of reliable methods for predicting foundation yielding under strong earthquake shaking and for putting it into proper consideration in the framework of such approaches.

For this purpose, nonlinear dynamic finite element simulations of large numerical models, including the superstructure, the foundation and the surrounding soil, are probably not the best tool, since they require the use of sophisticated soil constitutive laws and very large computational times to perform a comprehensive set of parametric analyses. To overcome such limitations, but preserving at the same time the essential features of the dynamic soil-structure interaction problem, the macro-element concept has become more and more popular (Nova and Montrasio, 1991; Paolucci, 1997; Cremer et al., 2001; Le Pape and Sieffert, 2001). It basically

consists of modelling the soil-foundation system as a single non-linear macro-element, having 3 degrees of freedom (dof) for in-plane analyses, defined by a suitable failure surface and plastic flow rule. However, although the macro-element approach is quite promising, it has not been supported so far by adequate experimental evidence, at least for seismic applications.
Indeed, few experimental results are available on the nonlinear soil-shallow foundation interaction under dynamic seismic loads, including, among the most relevant ones, the quasi-static large-scale tests performed at the Joint Research Center at Ispra, Italy (Negro et al., 2000; Faccioli et al., 2001) and at the Public Works Research Institute (Shirato et al., 2008) at Tsukuba, Japan, and the fully dynamic tests in centrifuge (Zeng and Steedman 1998, Gajan et al., 2005) and in shaking table (Maugeri et al. 2000).
In this work we make reference to the papers by Shirato et al. (2008) and Paolucci et al. (2008) to summarize the experimental setup and main results of a comprehensive set of shaking table tests on the performance of a model shallow foundation resting at the surface of a laminar box filled with dry sand, and excited by real accelerograms of various levels of amplitude, carried out at the Public Works Research Institute, Tsukuba, Japan. The purpose of this contribution is to show that, after a suitable improvement of the method proposed by Paolucci (1997), it is possible to capture with a satisfactory agreement many of the details of the experimental response, even during the highly nonlinear phases of foundation behavior.

2. THE SHAKING TABLE EXPERIMENTS ON A MODEL SHALLOW FOUNDATION AT PWRI

We refer the reader to Shirato et al (2008) for a thorough presentation of the large scale shaking table experiment that was set up at the Public Works Research Institute (PWRI), Tsukuba, Japan, for inducing realistic seismic loads on a test model suitable for nonlinear dynamic soil-structure interaction analyses. A laminar box, having dimensions 4 m x 4 m in plan and 2.1 m high was placed on the table. Dry Toyoura sand was filled in the box and compacted in layers so that nearly homogeneous soil conditions were obtained, with relative density D_r=80%, mass density $\rho = 1.6 \cdot 10^3$ kg/m^3, and angle of internal friction $\varphi = 42.1°$, the latter one measured through drained triaxial tests.
A test model was located on the center of the fill surface (Figure 1). This model consists of three main structural components: a steel rack at the top, 5227 N heavy, a 0.5 m sided square foundation block at the bottom, and a short steel beam with I cross-section connecting the two massive blocks. The total height of the model is 0.753 m, while the height of the center of mass is 0.420 m from the base of the foundation. The total weight of the structural model is 8385 N.
A summary of the experimental sequence is given in Table 1. The sand fill was prepared twice, and the corresponding sets of tests are denoted as Case 1 and Case 2. Each set was sub-divided into several experimental phases according to the type of excitation employed. In the first phase (Case 1-1), the shaking table was excited by a long duration, sinusoidal type waveform, referred to in the following as sweep wave excitation, with nominal peak acceleration amplitude of 50 gals. However, the actual motion recorded on the shaking table demonstrated that there are differences with the excitation transmitted by the control system, understandable when the huge dimensions of the experimental setup are considered. Table 1 reports in parentheses the actual peak acceleration recorded at the shaking table. In the subsequent phase (Case 1-2), the shaking table was excited by the long-duration acceleration history (Figure 2.a) recorded at Schichiho Bridge, Hokkaido, Japan, during the 1993 Hokkaido Nansei Oki Earthquake (MW=7.8). The model was then excited by a second sweep wave (Case 1-3), after which the structural model was raised, and the disturbed soil zone in the close vicinity of the foundation was leveled and tampered. Then, the test model was relocated at the same position, and the shaking table was excited by the NS component recorded during the 1995 Kobe Earthquake (M_W=6.9) at Japan Metrological Agency (JMA, Case 1-4). The Schichiho Bridge and Kobe JMA motions are consistent with the Type I and Type II seismic actions, respectively, as defined by the Japanese technical norms for highway bridges [19], the first one characterized by relatively small amplitude and long duration ground motion, and, the second, by large amplitude and short duration. The Kobe JMA earthquake record was selected both for Case 1-4 (Figure 2.b) and Case 1-5 (Figure 2.c). Before Case 1-4 experiment, the model pier footing was uplifted, the soil surface was leveled, and the model was replaced at the original position, with an embedment depth of 50 mm (20% of the footing depth and 10% of the footing length). Before Case 1-5

experiment, the pier footing model was just moved 1 m to the west without any treatment for the soil surface, with no embedment.

The experimental set-up was rebuilt before Case 2 set of experiments, including removal and refill of sand. The foundation embedment was 10 mm (4% of the footing depth and 2% of the footing length). After the sweep wave shaking (Case 2-1), the model was then excited by the Kobe JMA record (Case 2-2, Figure 2.d), with a 0.8 scaling factor. The embedment and the scaling were planned to prevent the model from complete failure as happened in Case 1-5. The experiments ended up with a final sweep wave excitation (Case 2-3).

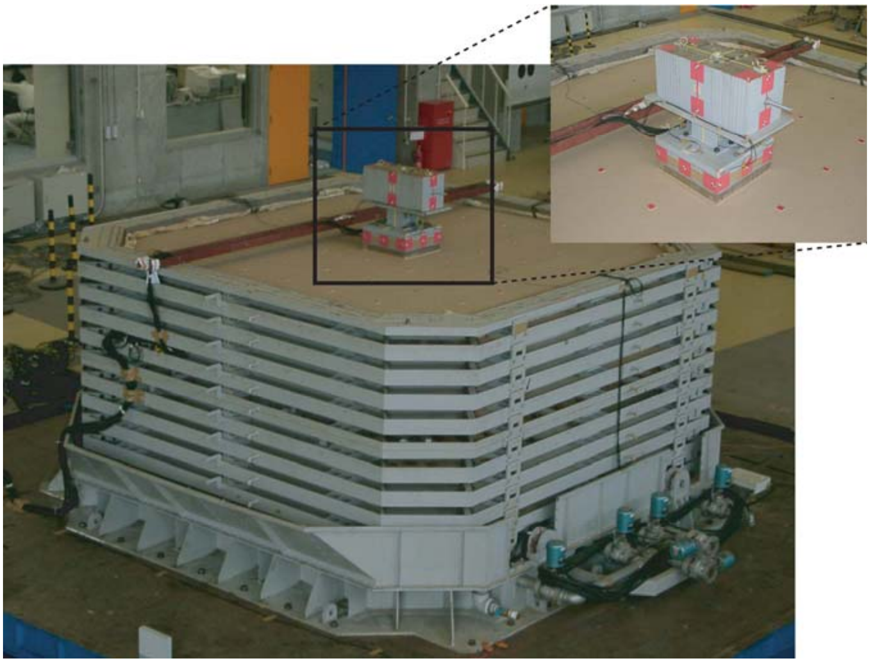

Figure 1. Picture of the experimental setup, including the test model.

Table 1. Summary of the performed experiments

Case	Input motions		Embedment depth (mm)
Case 1	Step 1	Sweep wave, 50 gal (112 gal)	0 mm
	Step 2	1993 Hokkaido Nansei Oki, Shichihou Br., Type I, 386 gal (601 gal)	
	Step 3	Sweep wave, 50 gal (110 gal) ⇒ *recompaction*	
	Step 4	1995 Kobe, JMA Kobe, Type II, 812 gal (712 gal)	50 mm
	Step 5	1995 Kobe, JMA Kobe, Type II, 812 gal (726 gal) ⇒ *removal of sand* ⇒ *preparation*	0 mm
Case 2	Step 1	Sweep wave, 50 gal (110 gal)	10 mm
	Step 2	A scaled motion of the 1995 Kobe, JMA Kobe, Type II, 650 gal (557gal)	
	Step 3	Sweep wave (116 gal)	

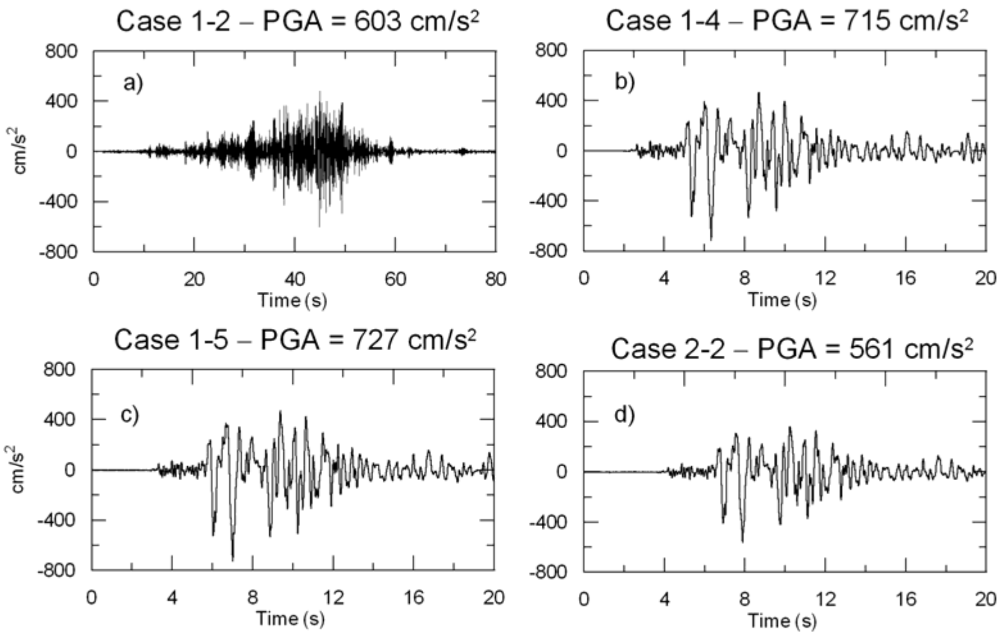

Figure 2. Earthquake records on one of the four corners of the shaking table metal plate during the different excitation phases, reported in Table 1: (a) 1993 Hokkaido Nansei Oki Earthquake Schichihou Bridge record, (b) and (c) 1995 Kobe Earthquake JMA record, (d) scaled 1995 Kobe Earthquake JMA record. Note the different time scale of Case 1-2.

3. COMPUTATIONAL PROCEDURE

The computational procedure is based on the simplified approach proposed by Paolucci (1997), consisting of a two-dimensional soil-structure interaction model having 4 dof (Figure 3), one to account for the horizontal motion of the superstructure, assumed to behave in the linear-elastic range, while the soil-foundation system is represented by a 3 dof (horizontal, rocking and vertical motion) elasto-perfectly plastic macro-element, with yielding function defined according to Nova and Montrasio (1991) and a plastic flow rule according to Cremer et al. (2002). Further details on the numerical procedure are reported in Paolucci et al. (2008).

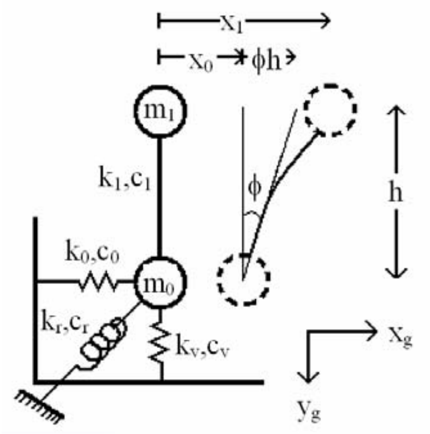

Figure 3. The 4 degrees-of-freedom model used for dynamic nonlinear soil-structure interaction analyses

A further improvement of our results was obtained by taking into account that during the strongly nonlinear phase of the excitation the instantaneous foundation-soil contact area decreases, owing to successive cycles of foundation rotations. This was implemented by a simple degradation rule for the foundation stiffness parameters

(k_0, k_v and k_r, see Figure 3), described by the following equation:

$$B' = B\left(1 - D\left(\theta_p\right)\right) \tag{3.1}$$

where B is the foundation width and D is a degradation parameter defined as follows:

$$D\left(\theta^p\right) = \frac{D_1}{1 + 1/D_2\theta_p} \tag{3.2}$$

where D_1 and D_2 are model parameters related to the ultimate D value and to the degradation speed, respectively, while θp is the cumulated plastic foundation rotation at a specified instant of time. The best agreement with observations has been obtained using D_1 = 0.75 and D_2 = 5000/rad. Details on this model and on its implementation in the numerical procedure are given in Paolucci et al. (2008).

4. NUMERICAL SIMULATION RESULTS

We have studied the response of the model to the four earthquake input motions considered in the sequence of shaking table experiments (see Table 1 and Figure 2). Referring the reader for a more detailed presentation of results, a sample of them is shown in Figure 4 in terms of the normalized overturning moment M/VB. We have omitted Case 1-5 for sake of brevity, mainly because it was run just after Case 1-4, without any intermediate preparation work of the foundation soil, so that the dynamic soil properties are not the same as in the other experiments. The horizontal displacements will also be omitted in the discussion, since they generally result in negligible permanent deformations.

All plots in Figure 4 show the normalized overturning moment observed (thick lines) and simulated (thin lines) within the 10 s long most severe phase of the excitations. Within this phase, the foundation behaviour is highly nonlinear, and frequently reaches a threshold eccentricity ratio that in our experiments turns out to be slightly higher than 0.4. For all cases, we have compared both results with and without stiffness degradation, to show that, although the simplest approach without degradation is already rather satisfactory, the improvement obtained by the stiffness degradation rule (1) and (2) leads in many cases to capture some important details of the nonlinear response, including the period elongation following the major yielding phases of the model response. Note that, while in the calculations with the non-degrading model the damping value was kept constant to the 5% value estimated from the small-amplitude vibrations, in the degrading model we raised such value to 10% (15% for Case 1-2), in order to limit the amplitude of fluctuations of the computed response, especially in terms of rotation, and improve the agreement with the observations. The higher frequency excitation for Case 1-2, and the corresponding short-duration yielding phases, can explain the higher damping ratio required to fit the observations.

When considering foundation rotation (Figure 5), the agreement is again quite satisfactory, especially if we consider the loading magnitude and the large permanent effects observed on the test model. Again, the degrading model is suitable to capture some important details of the observed time histories, except for Case 1-4 for which the numerical calculation diverges.

The main limitation of our numerical calculations is clearly the vertical settlement prediction, as shown in Figure 6: our elasto-perfectly plastic macro-element model is not presently able to simulate the accumulation of large vertical settlements throughout the excitation. However, when the magnitude of settlements and the number of loading cycles is limited as in Case 2-2, the prediction of the final vertical displacement is relatively satisfactory, although details of the time history are missed. Comparing rotations and vertical displacements in Figures 5 and 6, it seems that there is a coupling of these degrees of freedom in the nonlinear range, that is not suitably accounted for in our simplified model.

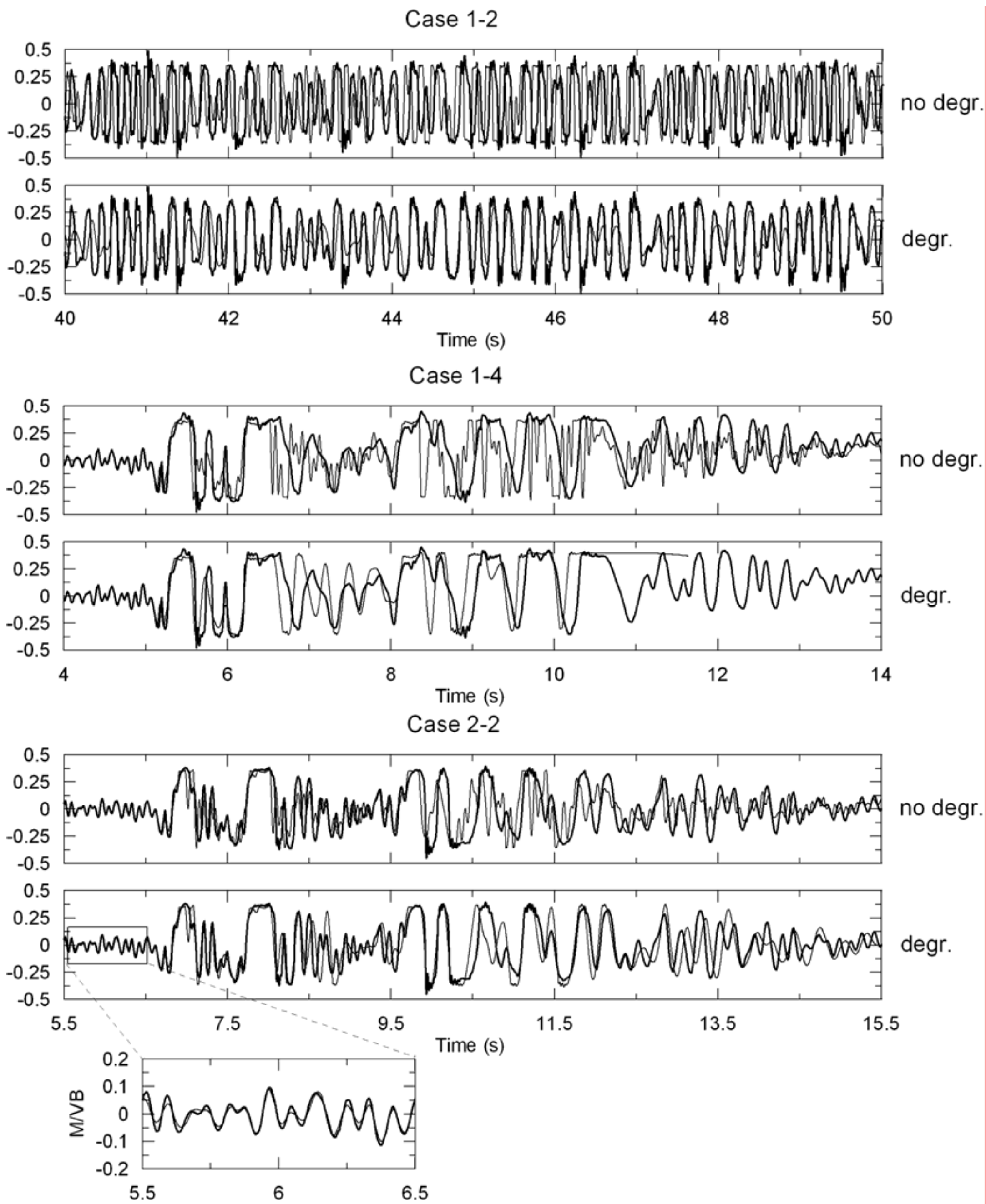

Figure 4. Time histories of the normalized overturning moment observed (thick lines) and simulated (thin lines) either with or without stiffness degradation. The small picture at the bottom shows a zoom of the observed vs. simulated comparison during the pre-yielding phase of the excitation. From Paolucci et al. (2008).

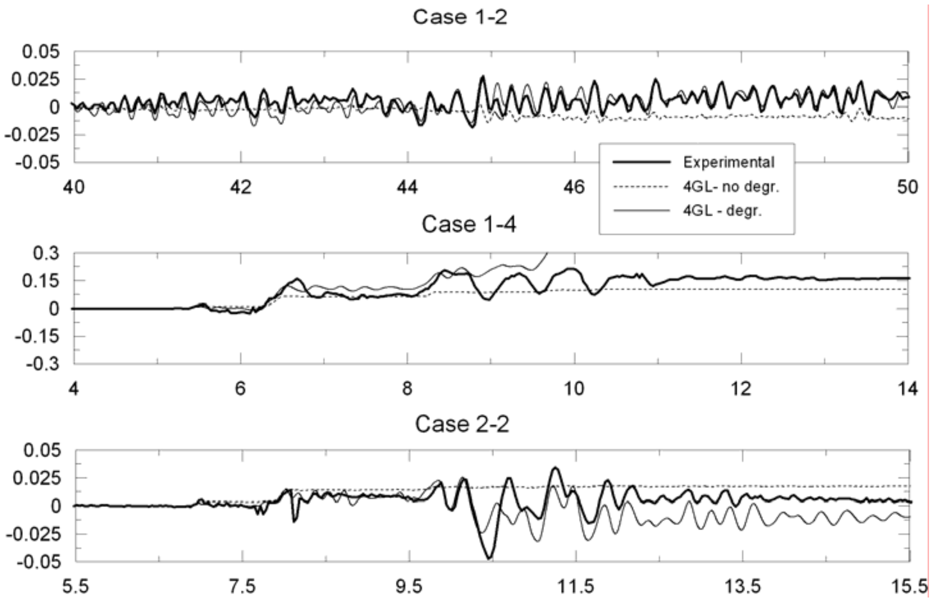

Figure 5. Same as Figure 4, in terms of foundation rotation time histories. From Paolucci et al. (2008).

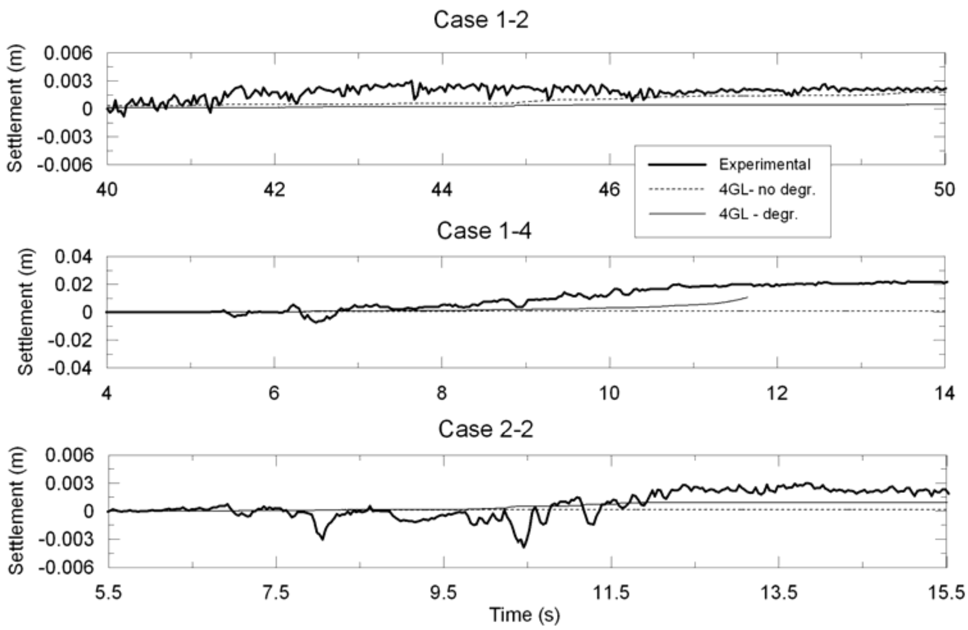

Figure 6. Same as Figure 4, in terms of foundation vertical settlement. From Paolucci et al. (2008).

5. CONCLUSIONS

Considering the very severe dynamic loading conditions on the foundation, the simplicity of the numerical model and the rapidity (just few seconds of a PC) to complete these computations, the results achieved should be considered fully satisfactory in terms of prediction of rocking behaviour. The introduction of a simple stiffness degradation rule allowed us to further improve the numerical results and to capture details of the observed time histories of overturning moment and rotation. No significant improvement of the results was obtained by considering P-Δ effects, except close to the complete overturning of the test model.
On the other hand, except for the case of short duration and relatively small magnitude excitation (Case 2-2), the

proposed approach fails to predict the observed foundation settlements when the number and frequency of loading cycles (as in Case 1-2) and/or their amplitude (as in Case 1-4) is considerable. The lack of agreement may be due either to the exceedingly simple nonlinear perfectly elasto-plastic model used in this work, or to the insufficient coupling of the vertical response of the foundation with the rocking one, during the strongest excitation phases.
Finally, we believe that, in spite of the previous drawbacks in the predictive capability of the model, its overall satisfactory performance could be hardly achieved by more sophisticated nonlinear finite element numerical codes, at the price of a much more demanding computer time.

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