Use of Ricker motions as an alternative to pushover testing

Loli, Marianna; Knappett, Jonathan; Anastasopoulos, Ioannis; Brown, Michael

Published in:
International Journal of Physical Modelling in Geotechnics

DOI:
10.1680/ijpmg.14.00009

Publication date:
2015

Document Version
Peer reviewed version

International Journal of Physical Modelling in Geotechnics

Use of Ricker motions as an alternative to pushover testing

--Manuscript Draft--

Manuscript Number:	IJPMG-D-14-00009R1
Full Title:	Use of Ricker motions as an alternative to pushover testing
Article Type:	2014 ICPMG Prize Winner
Corresponding Author:	IOANNIS ANASTASOPOULOS, PHD University of Dundee Dundee, UNITED KINGDOM
Corresponding Author Secondary Information:	
Corresponding Author's Institution:	University of Dundee
Corresponding Author's Secondary Institution:	
First Author:	MARIANNA LOLI, MSc
First Author Secondary Information:	
Order of Authors:	MARIANNA LOLI, MSc
	JONATHAN A KNAPPETT, PhD
	IOANNIS ANASTASOPOULOS, PHD
	MICHAEL J BROWN, PHD
Order of Authors Secondary Information:	
Abstract:	When undertaking centrifuge studies on seismic soil-structure interaction, it is useful to be able to define the pseudo-static "pushover" response of the structure. Normally, this requires separate centrifuge experiments with horizontal actuators. This paper describes an alternative procedure, using Ricker ground motions to obtain the pushover response, thereby allowing both this and the response to seismic shaking to be determined using a centrifuge-mounted shaker. The paper presents an application of this technique to a 1:50 scale model bridge pier with two different shallow foundations, as part of a study on seismic protection using rocking isolation. The moment rotation ("backbone") behaviour of the footings was accurately determined in the centrifuge to large rotations, as verified though independent 3D dynamic nonlinear finite element modelling. Ricker wavelet ground motions are therefore shown to be a useful tool for the identification of pushover response without requiring additional actuators. Furthermore, a simplified analytical methodology is developed allowing prediction of the maximum foundation rotation induced by a specific Ricker pulse. This methodology may be useful in predicting the characteristics (frequency and acceleration magnitude) of the Ricker pulse required to describe the pushover response of any (practically) rigid oscillator supported on shallow foundations.

Use of Ricker motions as an alternative to pushover testing

by

M. Loli[1], J.A. Knappett[2], I. Anastasopoulos[2], M.J. Brown[2]

[1] National Technical University of Athens, Greece ;[2] University of Dundee, UK

ABSTRACT

When undertaking centrifuge studies on seismic soil–structure interaction, it is useful to be able to define the pseudo-static "pushover" response of the structure. Normally, this requires separate centrifuge experiments with horizontal actuators. This paper describes an alternative procedure, using Ricker ground motions to obtain the pushover response, thereby allowing both this and the response to seismic shaking to be determined using a centrifuge–mounted shaker. The paper presents an application of this technique to a 1:50 scale model bridge pier with two different shallow foundations, as part of a study on seismic protection using rocking isolation. The moment rotation ("backbone") behaviour of the footings was accurately determined in the centrifuge to large rotations, as verified though independent 3D dynamic nonlinear finite element modelling. Ricker wavelet ground motions are therefore shown to be a useful tool for the identification of pushover response without requiring additional actuators. Furthermore, a simplified analytical methodology is developed allowing prediction of the maximum foundation rotation induced by a specific Ricker pulse. This methodology may be useful in predicting the characteristics (frequency and acceleration magnitude) of the Ricker pulse required to describe the pushover response of any (practically) rigid oscillator supported on shallow foundations.

NOTATION LIST

The following symbols are used in this paper:

a	Acceleration
a_{deck}	Deck acceleration
a_{E}	Acceleration at the model base (Excitation)
a_{FF}	Free field acceleration
B	Foundation width
d	Displacement
D_r	Relative density of the soil
E	Young's Modulus
f_E	Excitation frequency
FS_E	Factor of safety in seismic loading
FS_V	Factor of safety in vertical loading
H	Pier height
M	Moment load
M_{deck}	Deck mass
N	Vertical load
PGA	Peak ground acceleration
Q	Horizontal load
Sa	Spectral acceleration
S_d	Spectral displacement
t	Time
T	Period
z	depth
γ	Unit weight
δ	Horizontal displacement
δ_{F}	Horizontal displacement due to column bending
δ_{R}	Horizontal displacement due to foundation rotation
δ_{res}	Residual horizontal displacement
δ_{s}	Horizontal displacement due to foundation sliding
δ_{tot}	Total horizontal displacement
ϑ	Rotation
ϑ_c	Critical rotation causing overturning on a rigid base
ϑ_{uplift}	Rotation causing onset of uplifting
σ_v	Total vertical stress

φ	Soil friction angle
φ'_{peak}	Peak friction angle
φ'_{peak}	Peak friction angle

INTRODUCTION

The understanding of seismic soil–structure interaction (SSI) has been developed to the point where it is possible to utilize the ductile characteristics of foundation rocking to protect structures from more catastrophic brittle forms of failure (e.g. Gajan et al., 2005; Pecker, 2005; Paolucci et al., 2007; Gajan&Kutter 2008; Anastasopoulos et al., 2010; Gelagoti et al., 2012). The key concept underpinning this design approach is that the moment capacity of the foundation is lower than that which causes damage to the supported column or pier, resulting in shallow foundations which are smaller than those produced by conventional design approaches (aiming to prevent inelastic foundation response). This relies on adequate characterization of the moment–rotation pushover response of the system.

A recent collaborative study has been undertaken between the National Technical University of Athens and the University of Dundee. This study, reported in Loli et al. (2014), focussed on the use of rocking isolation to seismically protect Eurocode 2/8 (2005/2004, respectively) compliant reinforced concrete bridge structures. **Figure 1**showsthe conceptual prototype problem, where a 10.75 m tall bridge pier, carrying the dead load of the deck (300 Mg), is founded on a shallow, 10 m thick, layer of medium density sand (relative density, D_r = 60%) with a square (B x B) footing. This was structurally designed to resist a ground motion of 0.2g to Eurocode 8 design principles, as outlined in Loli et al. (2014). Two models were tested, the only difference between them being the foundation dimensions, with the aim of comparing two different approaches to aseismic foundation design as summarized in **Table 1**. The larger footing (B = 7.5 m) followed current code provisions ensuring minimal displacements of the soil–foundation interface under the design earthquake, i.e. the factor of safety against seismic loading (FS_E) is greater than one. The alternative design (B = 4 m) promoted the newly introduced concept of foundation rocking isolation with $FS_E < 1$.

During this study, the model bridge piers were realistically modelled using new scale-model reinforced concrete ("model-RC") developed at the University of Dundee and described in Knappett et al. (2010), Knappett et al. (2011) and Al-Defae & Knappett (2014a; 2014b). The structural design and validation of the properties of the model piers is described in Loli et al. (2014). In addition to testing the response of these damageable structures under historical ground motions, it was necessary to check the moment–rotation response of the foundation designs, to ensure that the structural moment capacity fell between the moment capacities of the two foundations which were associated with $FS_E < 1$. The yield surfaces for the two foundations in N–Q–M space (N = vertical load; Q = horizontal load; M = moment) for these foundations on the test soil (dry sand, properties given below) were estimated after Butterfield & Gottardi (1994) and are shown in **Figure 2**. Two yield surfaces are shown in each case with the outer representing peak friction angle conditions (φ'_{peak}) and the inner based upon the critical state friction angle for the soil (φ'_{crit}). It can be seen that the combination of loads acting on the structure lies between the yield surfaces implying that the small foundation will isolate the structure through rocking, while the larger one will not.

Determining the capacity of the footings posed a significant challenge as the timescale of the project meant that producing different centrifuge setups using horizontal actuators would not have been achievable. Therefore, it was decided to investigate the possibility of using a carefully selected dynamic ground motion to produce a “virtual pushover” of the structure–foundation model. This latter set of tests was conducted on a geometrically identical pier model, but with the model RC piers replaced with elastic columns, made of aluminium, to supress concrete failure and focus on foundation response. Initially it was intended to match these to the initial linear elastic bending stiffness of the model-RC columns (= 10.76 GNm^2 at prototype scale); however, preliminary numerical modelling (described later) indicated that using a stiffer structure, approximately 2.7 times stiffer in bending, would suppress flexural oscillations and ensure that the dynamic response of the pier be dominated by foundation rocking. Although this lead to unrealistic modelling of the bridge pier stiffness characteristics, promoting foundation rocking in this way was essential in facilitating the approximation of the foundation moment capacity and moment–rotation backbone curve through shaking which was the main objective of the herein presented work.

RICKER WAVELETS

When using a ground motion to simulate a pushover test, possible excitation alternatives include a step-type motion or various types of pulse input, including single sine pulses, “fling” pulses and Ricker wavelets. All of these are capable of producing a peak spectral displacement of significant magnitude to induce substantial rocking, provided that the dynamic characteristics of the structural system are tuned to have a suitably long natural period. Preliminary numerical modelling was conducted, as described in the following section, to evaluate these different possibilities. Thanks to their single characteristic pulse, Ricker wavelets were found to be most efficient in accurately approximating the monotonic static response. It was observed that they are generally able to mobilise a larger amount of the rotation capacity than the other pulse types, while also having the advantage that the earthquake actuator slip table automatically comes back to rest in its original starting position, without having a permanent displacement offset. This removes the need to re-centre the table before subsequent motions.

A total of 15 different Ricker wavelets were considered involving three different dominant frequencies, f_E = 2 Hz, 1 Hz, and 0.5 Hz, scaled to peak accelerations of 0.2 g, 0.4 g, 0.6 g, 0.8 g and 1.0 g. **Figure 3** shows the acceleration (α) time histories of the Ricker wavelets used in the numerical analysis and the respective elastic displacement spectra (S_d) for PGA = 1.0 g.

NUMERICAL MODELLING

Finite Element Discretisation

3D dynamic nonlinear finite element (FE) modelling was conducted using ABAQUS to investigate the behaviour of the bridge structure and underlying soil under different ground motions for simulating pushover. These analyses also serve the function of predicting the centrifuge test results which will be described later in the paper. **Figure 4** displays the sufficiently refined FE mesh and indicates the main features of the numerical model. The geometry is that of the centrifuge model at prototype scale. The deck and the footing were simulated using 8-noded hexahedral continuum elements, attributed the elastic properties of steel and aluminium respectively. The same element type, incorporating nonlinear material response, was used to model the sand layer. The 1.5 m × 1.5 m square section pier was simulated with 3D elastic beam elements assigned the geometric and elastic stiffness properties of the aluminium section used in the centrifuge tests (E = 70 GPa, γ = 26 kN/m^3).

Given the relatively high position of the deck mass, second-order (P–δ) effects are of great importance and were therefore taken into account. The lateral boundaries of the model are free to move horizontally and were assigned suitable stiffness properties so as to realistically reproduce the response of the equivalent shear beam (ESB) flexible wall container used in the centrifuge and described in Bertalot (2012). Taking advantage of symmetry upon the plane that crosses the foundation midpoint in the direction of shaking allowed simulation of only half of the full 3D model, achieving greater computational efficiency.

Soil Properties

The nonlinear behaviour of medium density silica sand (relative density D_r = 60%, unit weight γ = 15.5 kN/m^3), which was used in the experiments, was simulated using a simplified kinematic hardening model with Von Mises failure criterion and associated flow rule, modified appropriately so as to reproduce the pressure-dependent behaviour of sands as well as that of clays. Details of this model can be found in Anastasopoulos et al.(2011). Despite its lack of generality, the model has been shown to capture satisfactorily the nonlinear response of a shallow foundation upon compliant soil (Anastasopoulos et al., 2011). Moreover, in an attempt to provide a more realistic representation of the pressure-dependent sand behaviour, a user subroutine was encoded to provide variation of strength and stiffness properties with depth according to the $\varphi - \sigma_v$ and E–z relationships shown in **Figure 5**. The data in the figure is based on basic characterisation testing for the sand used in the centrifuge model tests, which may be found in Al-Defae et al. (2013). The soil–foundation interface was modelled using contact elements, which allow sliding and uplifting to take place whilst being governed by a hard-contact law and Coulomb's friction law in the normal and tangential directions, respectively.

Response under Ricker Excitation

A series of dynamic analyses were conducted in the time domain, wherein the model base was excited by a variety of idealized pulses (namely, sine, fling and Ricker pulses). Their intensity characteristics, such us peak acceleration and frequency, were parametrically varied in order to determine the pulse most appropriate to use in the centrifuge tests. Yet, this selection of excitation time histories was limited by a requirement for maximum displacement lower than 0.25 m, which is the capacity of the shaking table in prototype scale. **Figures 6a–6b** show acceleration and displacement time histories of four of the pulses used in the numerical study. It may be observed that in all cases the input displacement does not exceed the limit of 0.25 m. Given this restriction, the Ricker pulse appears to have two significant advantages: it ensures greater spectral response over a wide range of periods (**Figure 6e**); and it gives zero permanent displacement facilitating the simulation of the excitation time history with a shaking table.

Figure 7 shows the numerically computed dynamic response of the two foundations in the moment–rotation plane under excitation with 1 Hz and 0.5 Hz Ricker pulses (for two different PGA magnitudes). This is compared to the monotonic backbone curves calculated through numerical analysis of the same systems under horizontal pushover loading applied statically at the centre-of-mass of the deck. It is important to note that the calculated ultimate moment capacities are in good agreement with theoretical estimates (refer to **Figure 2**).

Strongly nonlinear behaviour may be identified in the shape of the single significant loop produced during each excitation pulse, this being presumably more pronounced with increasing excitation acceleration and/or dominant period. Having substantially greater displacement spectral ordinates over the entire range of periods (see **Figure 3**), the 0.5 Hz Ricker pulses naturally cause both foundations to respond well within the nonlinear regime, pushing them to much larger rotation amplitudes in comparison to shaking with the 1Hz Ricker pulse. Excessive material nonlinearity is manifested especially in the case of the smaller foundation leading to some considerable permanent rotation for both PGA cases shown. On the other hand, the response of the larger foundation is mainly associated with uplifting (loss of contact with the supporting soil), rather than soil yielding, and hence the M–ϑ loop resembles the well-known characteristic S–shape. Most importantly, in both cases the dynamic loops approximated the backbone curves satisfactorily indicating that Ricker pulses, especially those having substantially large dominant period, may be used in centrifuge tests to indirectly measure the ultimate lateral load foundation capacity.

CENTRIFUGE MODELLING

Model Set-up

Two dynamic centrifuge tests were conducted on 1:50 scale physical models of the bridge pier system, with identical super-structural properties, but with different foundations (B = 7.5 m and 4.0 m). In each case, the structures were placed on dry fine Congleton silica sand (HST95, γ_{max} = 1758 kg/m^3, γ_{min} = 1459 kg/m^3, D_{60} = 0.14 mm, D_{10} = 0.10 mm, critical state friction angle ϕ'_{crit} = 32^o), prepared uniformly by air pluviation to a relative density, $D_r \approx$ 60%. The deposit of sand was 200 mm deep (i.e., 10 m at prototype scale) and was prepared within the equivalent shear beam (ESB) container described by Bertalot (2012) to minimize dynamic boundary effects. Instrumentation consisted of 13 type ADXL78 MEMS accelerometers (± 70-g range) and Linear Variable Differential Transformers (LVDTs) as shown in **Figure 8**. The models were loaded onto the Actidyn Q67-2 servo-hydraulic earthquake simulator (EQS: see Bertalot et al., 2012 and Brennan et al., 2014 for a detailed description). Due to the limitations in displacement capacity of the EQS, it was not possible to reproduce the desired 0.5 Hz Ricker pulses and therefore the 1 Hz Ricker wavelet with PGA = 0.6 g was used as excitation in both tests. All subsequent results in this paper will be given at prototype scale at 50-g.

Motion Replication and Dynamic Response

Figure 9shows the accelerations measured at the centre of mass of the deck in each of the two models – these were determined as the average of the instruments at the top and bottom of the deck mass as shown in **Figure 8**. Also plotted are the demand motion, slip table motion and free field ground motion (top-most instrument in the right-hand column of buried accelerometers in **Figure 8**). It can be seen that the EQS faithfully reproduced the input motion, and that there was some free-field amplification within the soil.

Figure 10shows the lateral drift of the deck of the bridge, the total component δ_{tot} (due to sliding, δ_s, flexural displacement of the pier, δ_F and rotation δ_R); δ_{res} represents the residual value of δ_{tot} (i.e. the final unrecovered displacement). Due to a failure in one of the LVDTs recording the vertical foundation movement, it was not possible to independently measure δ_R for the case of the small foundation. However, geometric and physical properties of the pier standing on the small foundation (slenderness, relatively low factor of safety in vertical loading, and significantly lower foundation rotational stiffness in comparison to the large foundation) suggest that rocking motion would sufficiently dominate the other two possible modes of response (i.e. sliding would be significant for a less slender oscillator and flexural bending was intentionally suppressed here by the significantly high column stiffness) so as to assume $\delta_R \approx \delta_{tot}$. This dominant role of rocking oscillations, especially in the case of the small foundation, was confirmed by the results of the numerical analysis.

PUSHOVER RESPONSE

The values of δ_R were used with pier height h to determine the rigid body foundation rotation:

$$\theta = \sin^{-1}\left(\frac{\delta_R}{h}\right) \quad (1)$$

The moment at the bottom of the pier (which is the same as the moment input to the foundation) was determined using the accelerometer data at the deck (a_{deck}), recognising that the system is a cantilever:

$$M = m_{deck} a_{deck} h \quad (2)$$

In Equation (2), m_{deck} represents the mass at the top of the pier. **Figure 11**shows the moment rotation loops derived for the centrifuge data, and also plots the static pushover curve determined from the FEM. Considering the small foundation first, it is clear that for the case of foundations exhibiting substantial rocking, even a single Ricker pulse was sufficient to mobilise the moment capacity well into the non-linear (large rotation) domain. The match to the numerical backbone curve is very satisfactory, and suggests that this could be determined from the centrifuge test data by fitting an envelope around the centrifuge data within the positive quadrant (**Figure 11**).

In the case of the large foundation, much lower rotations were mobilised. This is due to the fact that the large foundation has significantly greater stiffness and capacity, therefore leading the pier to respond primarily through swaying and secondarily through rocking (as indicated by **Figure 10a**, where δ_R is a smaller proportion of the total deck drift δ_{tot}) as opposed to the small foundation pier, which responds primarily through rocking. Nevertheless, the maximum and minimum points of the loops agree well with the backbone curve. In this case, it is suggested that centrifuge testing with a single Ricker pulse was perhaps more useful for validating the pushover response determined from FE modelling. However, it is noted, and will be further elaborated in the following section, that this pulse would be much more efficient in describing the M–ϑ behaviour of the large foundation as well, had the pier column been stiffer or rigid enough to supress swaying in favour of rocking.

The centrifuge models were subsequently subjected to further consecutive and identical Ricker pulses which demonstrated that the foundations could be pushed further into the large rotation range to provide a more complete determination/validation of the pushover response, as shown in **Figure 12** (note change in scales compared to **Figure 11**). In the case of the large foundations (**Figure 12a**), the point of peak moment and rotation in the positive quadrant (shown by a circular marker) tracks laterally to the right along the backbone curve, confirming that the foundation is moving into the elasto-plastic plateau, while for the small foundation the successive pulses capture the descending branch of the backbone curve, though the moment capacity appears to be slightly higher than that predicted from the FEM. Although not tested here, a smaller magnitude pulse (or pulses) could first have been used with both foundations to determine a point (or multiple points) on the initial elastic section of the backbone curve.

The results shown in **Figure 12** suggest that the use of multiple sequential Ricker pulses allows the virtual pushover to be conducted to a desired amount of rotation. The maximum moment points can then be joined together to provide a good estimate of the backbone curve (or, preferentially, to validate an independent calculation, e.g. by FEM).

SIMPLIFIED ANALYTICAL METHODOLOGY

Following validation of the new procedure, it was considered desirable to develop a simple analytical methodology to allow estimation of the characteristics of the Ricker pulse (f_E, PGA) required to describe the monotonic pushover response of similar single–degree–of–freedom equivalent oscillator systems having height to centre-of-mass = H and contact width with the soil = B for use in experimental modelling without the need to employ preliminary numerical analysis. To do so, it was necessary to make the simplification of considering the pier as rigid enough to respond predominantly through rocking and minimize flexural deformation of the column. This would be a valid assumption in the case of a relatively slender pier ($H/B > 1.5$) supported on a shallow foundation designed according to the principle of rocking isolation (see Gelagoti et al., 2012; and Loli et al., 2014). In a few words, this refers to an under-designed foundation (with $FS_E < 1$) and moment capacity lower than the capacity of the supported column section.

A series of further numerical analyses were conducted, including all the aforementioned Ricker pulses (**Figure 3**) for the same numerical model, with the only difference being the stiffness of the column, which was 100 times larger than in the centrifuge tests, so as to be close to rigid. Results are summarized in **Figure 13** for both foundation sizes. For PGA > $B/(2H)$, i.e. when uplifting is expected, the maximum rotation experienced by the foundations was found to have a very strong correlation with the spectral displacement of the free field motion at large periods (here a period of 5 sec was taken as reference). More specifically, the following relationship may be deduced:

$$\sin\left(\max(\theta)\right) = S_d^{T=5\text{sec}} / \mathrm{H}, \text{ for } \theta_{uplift} > \theta > \theta_c \quad (3)$$

Knowing the desired maximum foundation rotation, one may use Equation (3) to determine the spectral displacement required and hence the characteristics of a suitable pulse. It should be noted that this methodology is valid in the large displacement domain, where the foundation response id well off the linear regime and uplifting or soil yielding takes place. This is to say, the desired maximum rotation may be a fraction of the critical rotation causing overturning on a rigid base, ϑ_c = atan($B/(2H)$), and presumably greater than the rotation causing onset of uplifting (ϑ_{uplift}). The latter may be approximated making use of the Winkler foundation model as described by Apostolou et al. (2007).

SUMMARY AND CONCLUSIONS

In this paper it has been demonstrated that a Ricker wavelet type ground motion can be used in a centrifuge earthquake simulator to determine or validate the pushover response of shallow foundation systems, without requiring additional actuator set-ups. This approach was found to provide useful information on the foundation response for cases when either small or large amounts of rocking are expected. A structure representing a typical bridge pier tested with two different sizes of shallow foundation was considered, with 3D nonlinear dynamic FE modelling used initially to demonstrate the concept and size the Ricker pulses. Centrifuge testing was then conducted which demonstrated that the shape of the 'backbone' moment-rotation curve for the foundations could be determined be enveloping the moment rotation response from an appropriately-sized Ricker pulse, and that this matched the prediction from the FE modelling. It was further shown that the subsequent application of additional pulses could extend the curve to larger rotations. A simple expression was also developed, based on the centrifuge test data and further numerical parametric study that can be used in determining the properties of the Ricker pulse that will produce a desired amount of rotation for systems with different aspect ratios (H/B). It is expected that the use of Ricker pulses will be particularly useful in characterising system response in future centrifuge tests of seismic soil–structure interaction problems, particularly given the current trend towards novel foundation designs which employ foundation rocking to seismically isolate the structure for which determination of the pushover response is extremely important.

LIMITATIONS

The presented methodology has been developed on the basis of centrifuge testing and numerical analysis of single-degree-of-freedom structures supported on shallow footings and its implementation naturally refers to such structures. The method relies on the seismically induced rocking vibration of the structure and therefore its effectiveness depends on the ability of the structure to respond predominantly through rocking as opposed to sliding or flexural vibrations. This requirement is related to the geometry (slenderness) and the rigidity of the structure. In particular, the method was shown to provide accurate results for slender structures (H/B > 1.5) with foundations designed in accordance with the newly introduced concept of rocking isolation.

ACKNOWLEDGEMENT

The first author wishes to acknowledge financial support from the project FORENSEIS, financed through the ARISTEIA programme of the Ministry of Education of Greece, and the Coordinator of this project, Professor George Gazetas, whose mentoring and ideas are greatly appreciated. The authors would also like to thank Mark Truswell for his assistance with the centrifuge tests and model fabrication.

REFERENCES

Apostolou M, Gazetas G and Garini E (2007) Seismic response of slender rigid structures with foundation uplifting – *Soil Dynamics and Earthquake Engineering* **27**: 642–654. DOI: 10.1016/j.soildyn.2006.12.002

Al-Defae AH, Caucis K and Knappett JA (2013) Aftershocks and the whole-life seismic performance of granular slopes – *Géotechnique* **63**(14): 1230–1244. DOI: 10.1680/geot.12.P.149

Al-Defae AH and Knappett JA (2014a) Stiffness matching of model reinforced concrete for centrifuge modelling of soil-structure interaction. In *Proc. 8th International Conference on Physical Modelling in Geotechnics, ICPMG '14*, Perth, Australia, 14-17 January: 1067-1072.

Al-Defae AH and Knappett JA (2014b) Centrifuge modelling of the seismic performance of pile-reinforced slopes − *Journal of Geotechnical and Geoenvironmental Engineering*, ASCE **140**(6): 04014014. (DOI: 10.1061/(ASCE)GT.1943-5606.0001105)

Anastasopoulos I, Gazetas G, Loli M, Apostolou M and Gerolymos N (2010) Soil failure can be used for seismic protection of structures − *Bulletin of Earthquake Engineering* **8**(2): 309 – 326. DOI: 10.1007/s10518-009-9145-2

Anastasopoulos I, Gelagoti F, Kourkoulis R and Gazetas G (2011) Simplified Constitutive model for Simulation of Cyclic Response of Shallow Foundations: Validation against Laboratory Tests − *Journal of Geotechnical and Geoenvironmental Engineering*, ASCE **137**(12): 1168-1154. DOI: 10.1061/(ASCE)GT.1943-5606.0000534

Bertalot D (2012) *Behaviour of shallow foundations on layered soil deposits containing loose saturated sands during earthquakes*. PhD Thesis, University of Dundee, UK.

Bertalot D, Brennan AJ, Knappett JA, Muir Wood D and Villalobos FA (2012) Use of centrifuge modelling to improve lessons learned from earthquake case histories. In *Proc. 2nd European conference on Physical Modelling in Geotechnics, Eurofuge 2012*, 23-24 April, Delft, the Netherlands.

Brennan AJ, Knappett JA, Bertalot D, Loli M, Anastasopoulos I, and Brown MJ (2014) Dynamic centrifuge modelling facilities at the University of Dundee and their application to studying seismic case histories. In *Proc. 8th International Conference on Physical Modelling in Geotechnics, ICPMG '14*, Perth, Australia, 14-17 January: 227-233.

Butterfield R, Gottardi G (1994) A complete three dimensional failure envelope for shallow footings on sand − *Geotechnique* **44**(1): 181–184.

CEN. Eurocode 2: "Design of concrete structures — Part 2: Concrete bridges. Design and detailing rules." EN 1992-2:2005. Brussels, 2005.

CEN. Eurocode 8: "Design of structures for earthquake resistance — Part 1: General rules, seismic actions and rules for buildings." EN 1998-1:2004. Brussels, 2004.

Gajan S, Kutter BL, Phalen JD, Hutchinson TC and Martin G (2005) Centrifuge modeling of load-deformation behavior of rocking shallow foundations − *Soil Dynamics and Earthquake Engineering* **25**(7–10): 773–783. DOI: 10.1016/j.soildyn.2004.11.019

Gajan S and Kutter BL (2008) Capacity, settlement, and energy dissipation of shallow footings subjected to rocking − *Journal of Geotechnical and Geoenvironmental Engineering*, ASCE, **134**(8): 1129–1141. DOI: 10.1061/(ASCE)1090-0241(2008)134:8(1129)

Gelagoti F, Kourkoulis R, Anastasopoulos I and Gazetas G (2012) Rocking Isolation of Frame Structures Founded on Separate Footings − *Earthquake Engineering & Structural Dynamics* **41**(7): 1177-1197. DOI: 10.1002/eqe.1182.

Knappett JA, Reid C, Skeffington K, O'Reilly K and Gilhooley P (2010) Modelling precast concrete piling for use in the geotechnical centrifuge. In *Proc. 7th International Conference on Physical Modelling in Geotechnics, ICPMG '10*, Zürich: 141-146.

Knappett JA, Reid C, Kinmond S and O'Reilly K (2011) Small-scale modelling of reinforced concrete structural elements for use in a geotechnical centrifuge − *Journal of Structural Engineering*, ASCE **137**(7): 1263-1271. DOI: 10.1061/(ASCE)ST.1943-541X.0000371.

Loli M, Knappett JA, Brown MJ, Anastasopoulos I and Gazetas G (2014) Centrifuge modeling of rocking–isolated inelastic RC bridge piers − *Earthquake Engineering & Structural Dynamics* **43**(15): 2341-2359. DOI: 10.1002/eqe.2451.

Paolucci R, Shirato M and Yilmaz MT (2007) Seismic behavior of shallow foundations: shaking table experiments vs numerical modeling – *Earthquake Engineering and Structural Dynamics* **37**(4): 577–595. DOI: 10.1002/eqe.773.

Pecker A (2005) Design and Construction of the Foundations of the Rion Antirion Bridge. In *Proceedings of the 1st Greece–Japan Workshop on Seismic Design, Observation, Retrofit of Foundations*, Athens, 119 - 130.

FIGURE CAPTIONS

Figure 1. Schematic of the problem considered: bridge pier on a shallowfoundation considering two different sizes/designs.

Figure 2. Foundation yield surfaces along with structural capacity shown for comparison.

Figure 3. Acceleration time histories and elastic (ξ = 5%) displacement response spectra (shown only for PGA = 1 g) forthe Ricker wavelets: (a) f_E = 2 Hz; (b) f_E = 1 Hz; and (c) f_E= 0.5 Hz.

Figure 4. Details of the 3D FE simulation of the centrifuge model.

Figure 5. Stress and depth dependent soil properties used in the FE model.

Figure 6. Acceleration and displacement time histories of the idealized pulses used as bedrock excitations: (a) Sine 1 Hz, 0.2 g; (b) Sine 1.7 Hz, 0.44 g; (c) Fling 1 Hz, 0.5 g; (d) Ricker 0.7 Hz, 0.6 g; (e) acceleration response spectra of the bedrock excitation pulses (ξ = 5%).

Figure 7. Numerically computed foundation moment–rotation loops compared to monotonic pushover response for: the large foundation model with (a) 1Hz and (b) 0.5 Hz Ricker wavelets; and the small foundation model with (c) 1Hz and (d) 0.5 Hz Ricker wavelets for two different amplitudes (PGA = 0.6 g and 1.0 g).

Figure 8. Experimental set-up for centrifuge testing (small foundation shown; large foundation indicated by dashed line, all dimensions in millimetres).

Figure 9. Accelerations recorded during centrifuge tests: (a) deck acceleration, large foundation; (b) deck acceleration, small foundation; (c) demand, input and free field motions.

Figure 10. Deck drift time histories recorded during centrifuge tests for table excitation with Ricker 1Hz PGA = 0.6 g: (a) bridge pier on large foundation; and (b) bridge pier on small foundation.

Figure 11. Foundation moment-rotation behaviour determined from centrifuge tests, compared to monotonic pushover (“back-bone”) curves from FE model: (a) large foundation,and (b) small foundation.

Figure 12. Foundation moment-rotation loops recorded for a series of four successive, practically identical, Ricker pulses (f_E = 1Hz, PGA = 0.6 g), compared with monotonic FE predictions: (a) large foundation, and (b) small foundation.

Figure 13. Summary of the numerical and experimental results: maximum drift experienced at the deck due to foundation rotation$\delta_{R,max}$= $H\sin(\vartheta_{max})$, with respect to the large period spectral displacement $S_{d(T = 5\ sec)}$of the free field excitation.

TABLE CAPTIONS

Table 1. Footing designs considered in this study (all values at prototype scale).

REVIEWER 1

Reviewer's Comment	Author Response
The authors present a new technique to account for pushover response in dynamic soil-structure interaction problems. The paper is well written and should be accepted in its present form.	The authors would like to thank the reviewer for her/his kind words and approval for publication.

REVIEWER 2

Reviewer's Comment	Author Response
The paper presents a technique that can be used to avoid the need of having an in-flight actuator to perform the pushover procedures for SSI centrifuge experiments. While the new technique seems to be potentially promising, some details are still missing. For example, there should be a section discussing the limitations of the technique. What if the mass is distributed rather than concentrated? Also, how would the method work for a yielding pier or column? Some other comments are mentioned below. For the purpose of revision, the page numbers mentioned below are based on assigning number 1 to the title and abstract page. (Please number the pages in the revised paper)	The authors would like to thank the reviewer for the thorough review of our manuscript and her/his comments which significantly contributed in improving the quality of this paper. Following the reviewer's suggestion, a section has been added in the paper discussing the limitations of the proposed simplified methodology. More specifically, it is clearly stated that the latter has been developed on the basis of centrifuge testing and numerical analysis of single-degree-of-freedom structures supported on shallow footings and its implementation naturally refers to such structures. Moreover, the method relies on the seismically induced rocking vibration of the structure and therefore its effectiveness depends on the ability of the structure to respond predominantly through rocking as opposed to sliding or flexural vibrations. This requirement is related to the geometry (slenderness) and the rigidity of the structure. In particular, the method was shown to provide accurate results for slender structures (H/B > 1.5) and practically rigid oscillators or structures supported on soft soil.

In page 4 "preliminary numerical modelling (described later) suggested that the amount of pushover could be maximised by using a solid aluminium pier, approximately 2.7 times stiffer in bending." Please elaborate.	Following the reviewer's suggestion, the following details were added in the text: "…however, preliminary numerical modelling (described later) suggested that using a stiffer structure, approximately 2.7 times stiffer in bending, would suppress flexural oscillations and ensure that the dynamic response of the pier be dominated by foundation rocking. Although this lead to unrealistic modelling of a bridge pier, promoting foundation rocking in this way was essential in facilitating the approximation of the foundation moment capacity and moment–rotation backbone curve through shaking which was the main objective of the herein presented work."
In page 6 "Von Mises failure criterion and associated flow rule, modified appropriately so as to reproduce the pressure-dependent behaviour of sands as well as that of clays. Details of this model can be found in Anastasopoulos et al. (2011)." Why did not the authors just use Mohr Coulomb failure criteria instead of adopting a modified Von Mises criterion, which is typically used for clay?	The Mohr Coulomb model is suitable for simulating static problems on sandy soils. Yet, it is known to dramatically over-predict damping and therefore raising concerns when used in dynamic problems. Therefore, in this study, we made use of the modified Von Mises model developed by Anastasopoulos et al., 2011 which has been extensively validated against physical modelling tests and shown to reliably reproduce the dynamic/cyclic response of shallow foundations over both sand and clay.
In page 7, "This confirms that over a wide range of periods, the Ricker pulse provides a greater spectral response than the other types of motion." How can you reach this conclusion from comparing base motion of different amplitudes!? May be if you fix the amplitude of the base motion, the conclusion is not going hold true.	The reviewer's question is reasonable as the concept of motion selection was not explained in sufficient detail. The motions were selected with respect to a fixed maximum displacement due to the shaking table displacement capacity. The text was revised to describe this concept more clearly: "A series of dynamic analyses were conducted in the time domain, wherein the model base was excited by a variety of idealized pulses (namely, sine, fling and Ricker pulses). Their intensity characteristics, such us peak acceleration and frequency, were parametrically varied in order to determine the pulse most appropriate to use in the centrifuge tests. Yet, this selection of

	excitation time histories was limited by a requirement for maximum displacement lower than 0.25 m, which is the capacity of the shaking table in prototype scale. **Figures 6a−6b** show acceleration and displacement time histories of four of the pulses used in the numerical study. It may be observed that in all cases the input displacement does not exceed the limit of 0.25 m. Given this restriction, the Ricker pulse appears to have two significant advantages: it ensures greater spectral response over a wide range of periods (**Figure 6e**); and it gives zero permanent displacement facilitating the simulation of the excitation time history with a shaking table".
In page 9 "in this case the pier was expected to have experienced almost purely rotational motion so <delta>R ?<delta>tot" What is the basis of this conclusion?? Previous publication??	Geometric and physical properties of the pier standing on the small foundation (slenderness, relatively low factor of safety in vertical loading, and significantly lower foundation rotational stiffness in comparison to the large foundation) suggest that rocking motion would sufficiently dominate the other two possible modes of response: sliding (would be significant for a less slender oscillator), and flexural bending (intentionally suppressed here by the significantly high column stiffness). This dominant role of the rocking mode of response, especially in the case of the small foundation, was confirmed by the results of the numerical analysis. The text was revised accordingly.

In page 11 "To do so, it was necessary to make the simplification of considering the pier as rigid enough to respond predominantly through rocking and minimize flexural deformation of the column. This would be a valid assumption in the case of slender structures" How are Slender Structures rigid and responding predominantly through rocking?	We thank the reviewer for pointing out this inaccuracy in the text. We have revised the text to be more clear: "This would be a valid assumption in the case of a relatively slender pier (*H/B* > 1.5) supported on a shallow foundation designed according to the principle of rocking isolation (see Gelagoti et al., 2012; and Loli et al., 2014). In a few words, this refers to an under-designed foundation (with FS_E < 1) and moment capacity lower than the capacity of the supported column section".
In page 12 "Knowing the desired maximum foundation rotation, one may use Equation (3) to determine the spectral displacement required and hence the characteristics of a suitable pulse." How about the frequency of the base motion and natural frequency of the structure? Do they have an influence?	The frequency of the base motion is incorporated in the calculations through the displacement response spectra of the excitation (S_d). The elastic natural frequency of the structure does not play any role because we refer to the large displacement domain, where uplifting takes place and the response is strongly nonlinear. In this case, while the foundation rotates and uplifting becomes more important, the equivalent dominant period of the system has been found to increase dramatically. Therefore, the presented methodology uses spectral ordinates at large periods (T = 5 sec was found to give quite satisfactory estimates). The text was revised to clearly state this.

Figure
Click here to download high resolution image

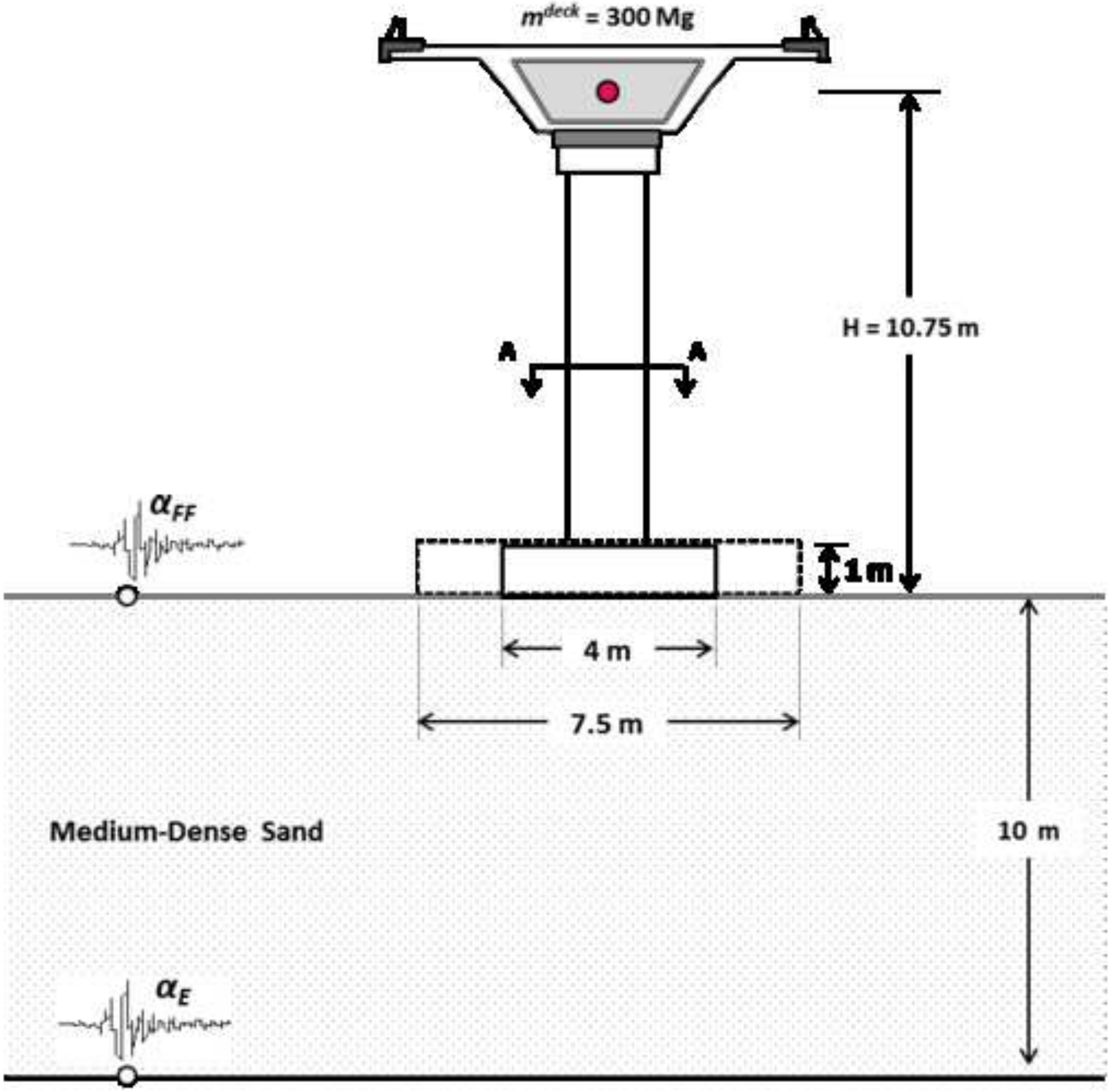

Figure 1. Schematic of the problem considered: bridge pier on a shallow foundation considering two different sizes/designs.

Figure
Click here to download high resolution image

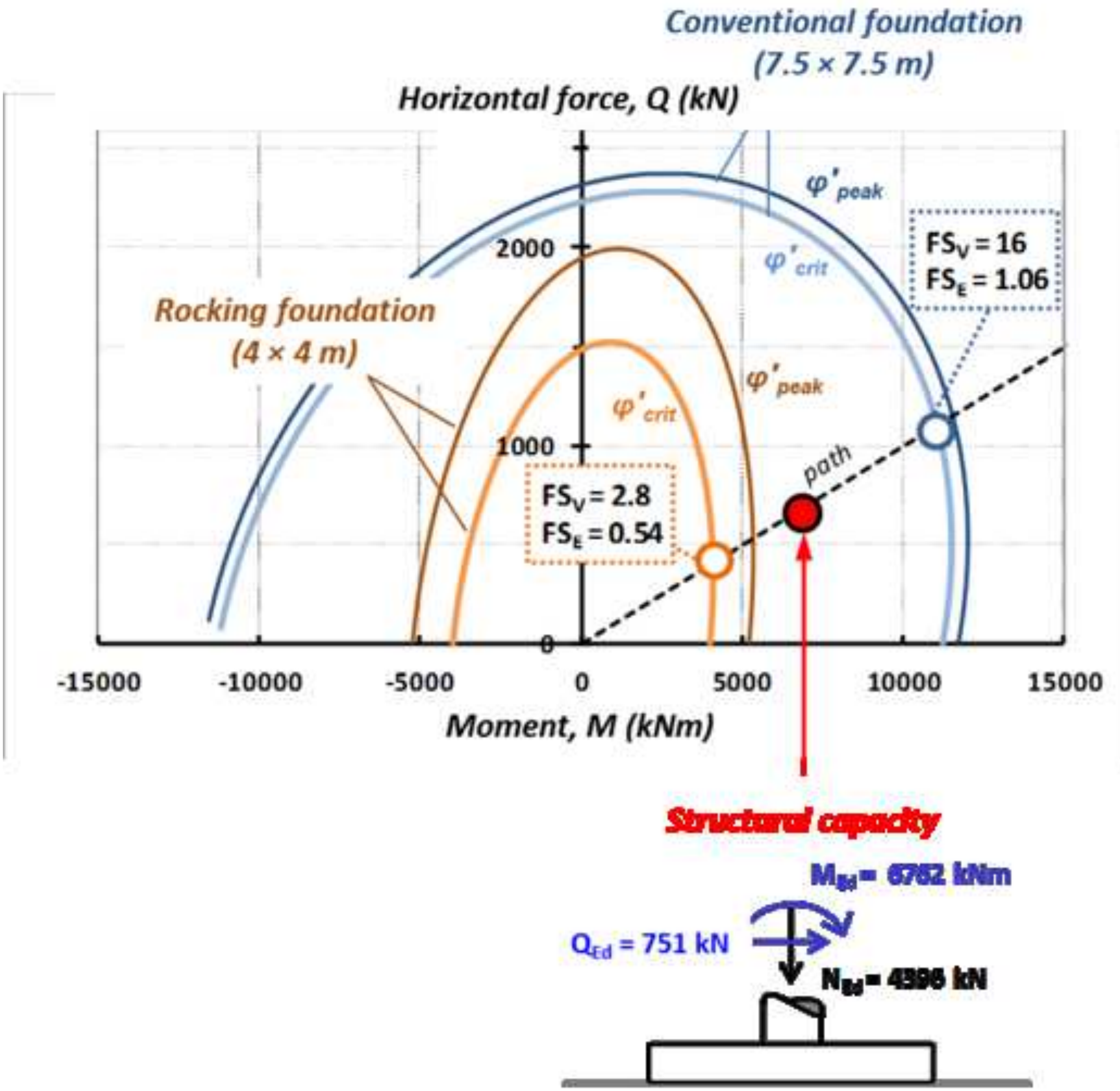

Figure 2. Foundation yield surfaces along with structural capacity shown for comparison.

Figure
Click here to download high resolution image

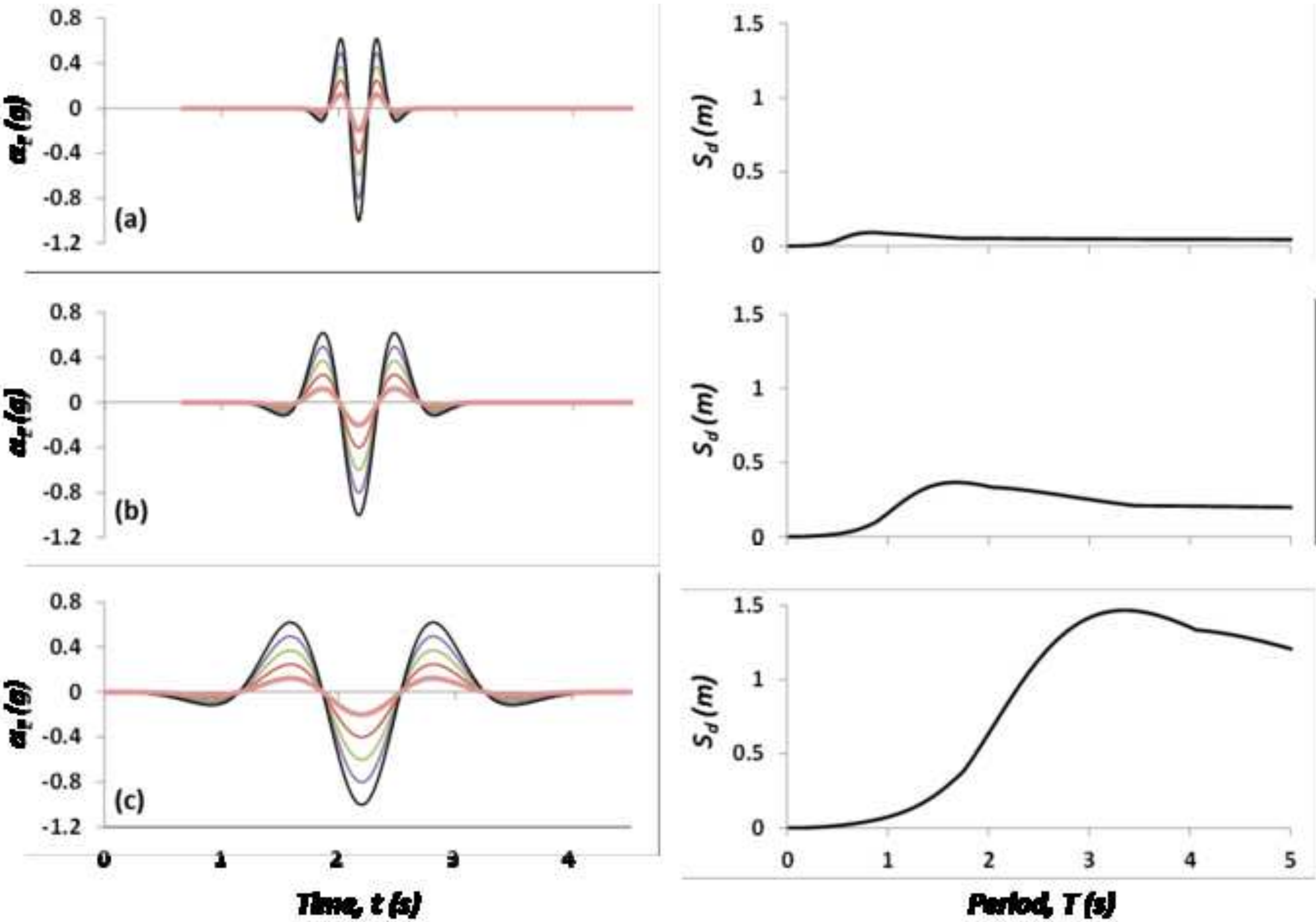

Figure 3. Acceleration time histories and elastic (ξ = 5%) displacement response spectra (shown only for PGA = 1 g) for the Ricker wavelets: (a) f_E = 2 Hz; (b) f_E = 1 Hz; and (c) f_E = 0.5 Hz.

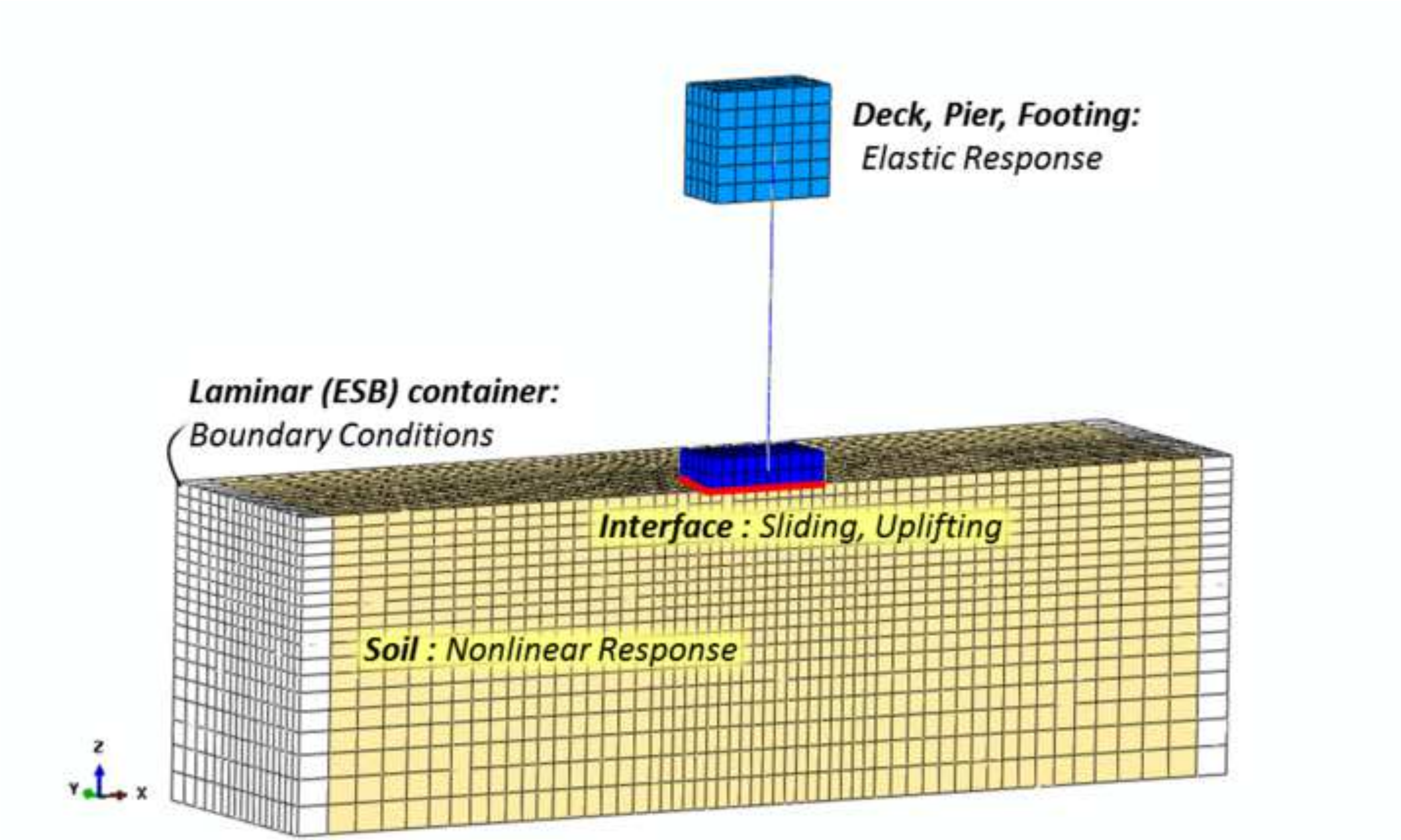

Figure 4. Details of the 3D FE simulation of the centrifuge model.

Figure
Click here to download high resolution image

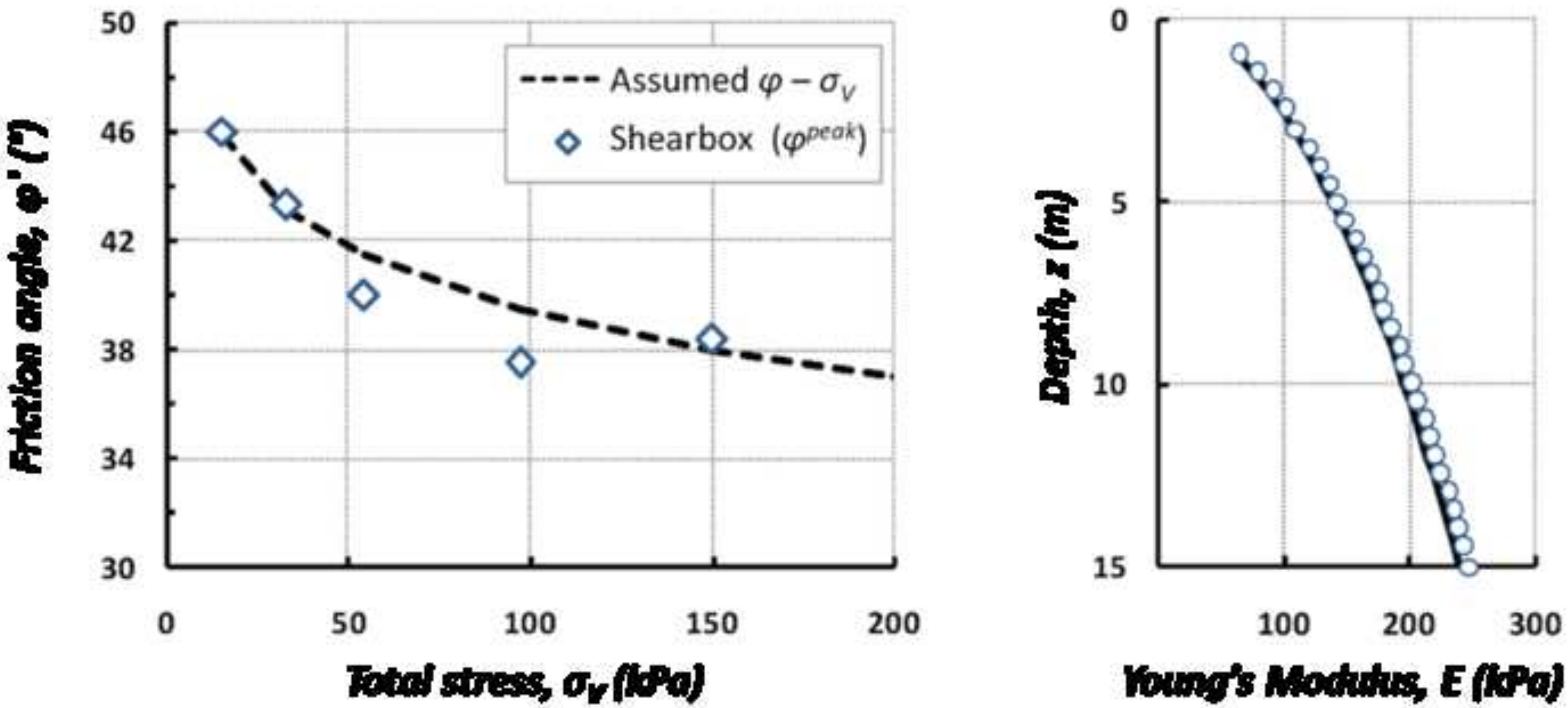

Figure 5. Stress and depth dependent soil properties used in the FE model.

Figure

Click here to download high resolution image

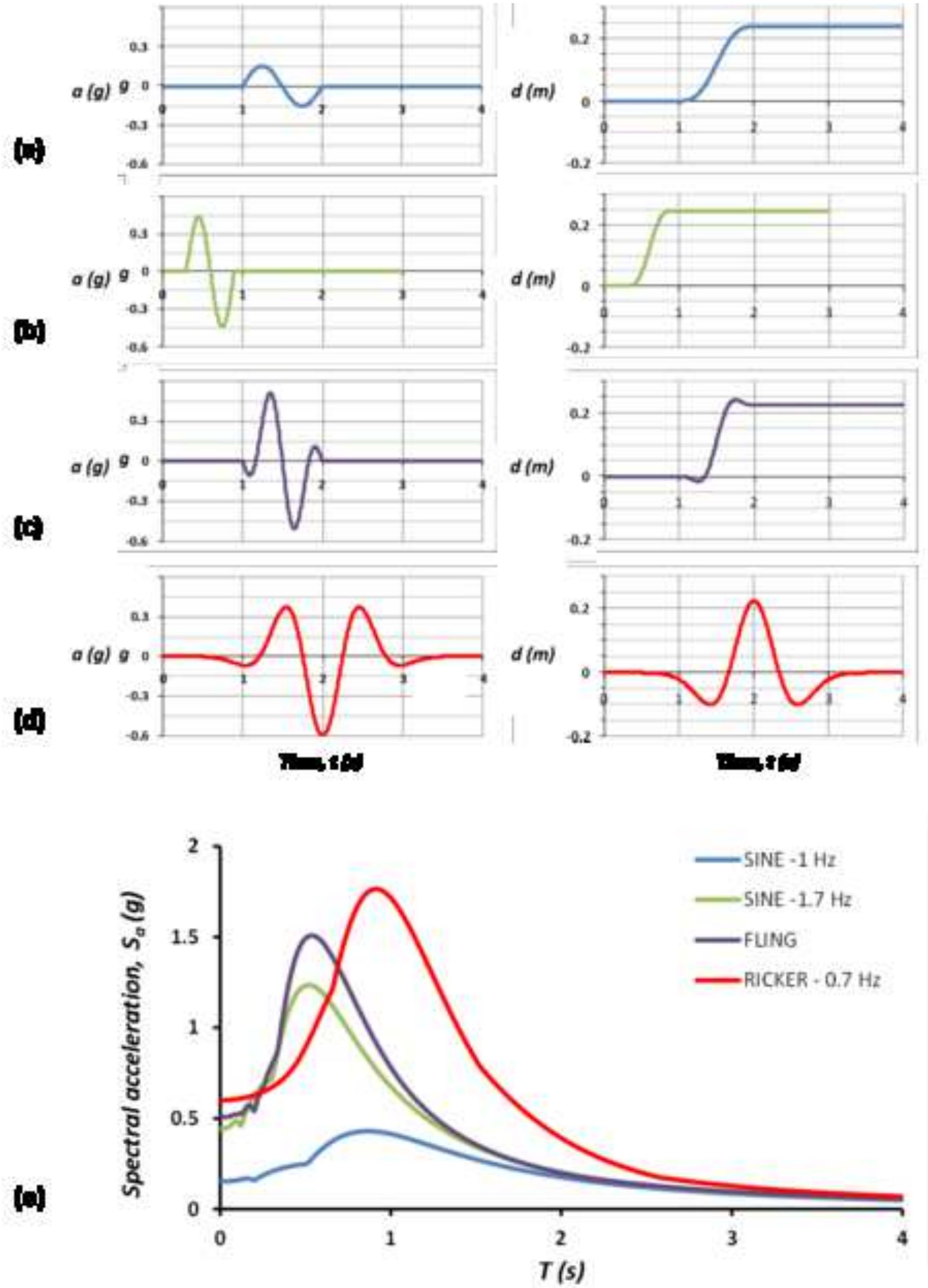

Figure 6. Acceleration and displacement time histories of the idealized pulses used as bedrock excitations: (a) Sine 1 Hz, 0.2 g; (b) Sine 1.7 Hz, 0.44 g; (c) Fling 1 Hz, 0.5 g; (d) Ricker 0.7 Hz, 0.6 g; (e) acceleration response spectra of the bedrock excitation pulses (ξ = 5%).

Figure
Click here to download high resolution image

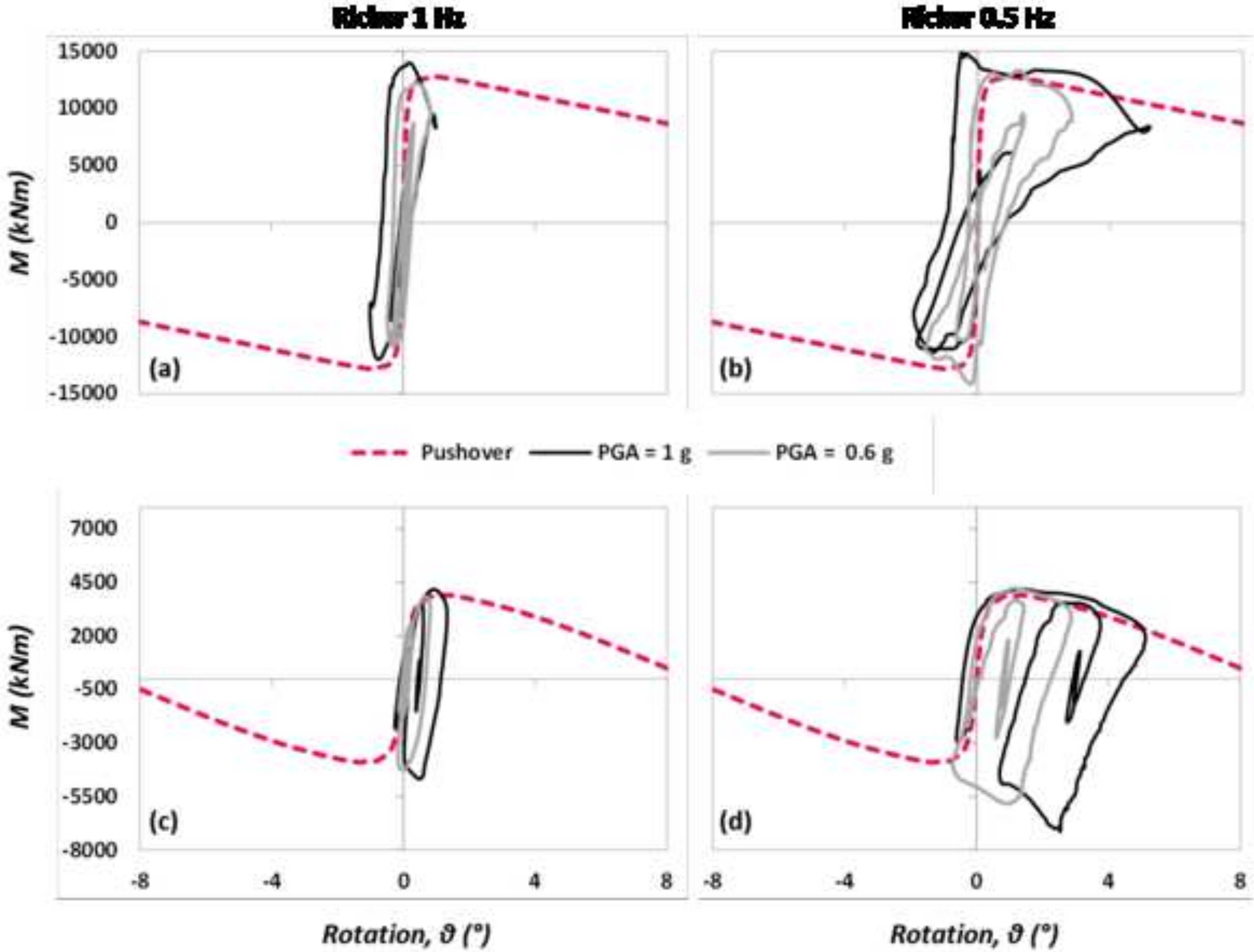

Figure 7. Numerically computed foundation moment–rotation loops compared to monotonic pushover response for: the large foundation model with (a) 1 Hz and (b) 0.5 Hz Ricker wavelets; and the small foundation model with (c) 1Hz and (d) 0.5 Hz Ricker wavelets for two different amplitudes (PGA = 0.6 g and 1.0 g).

Figure
Click here to download high resolution image

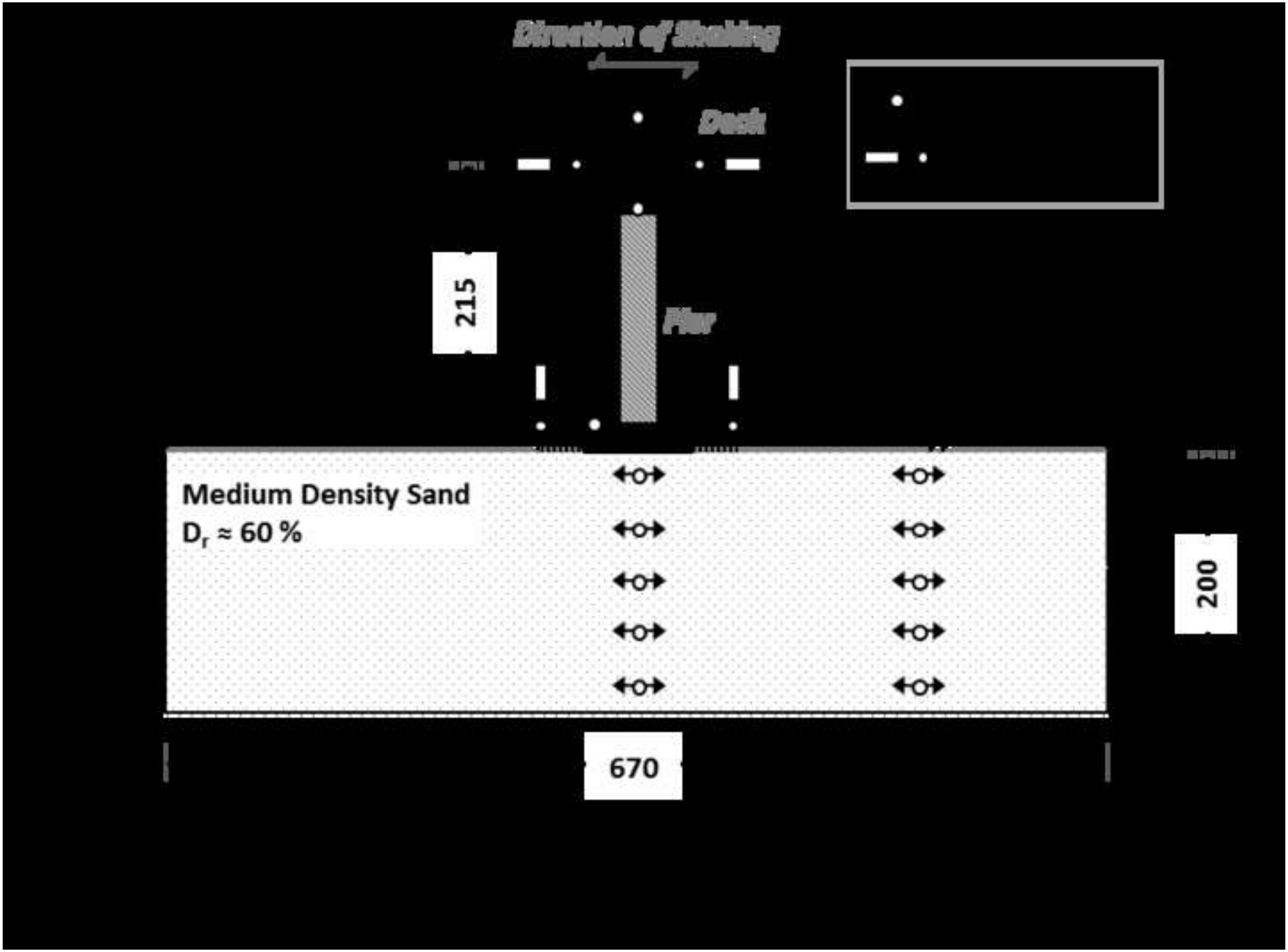

Figure
Click here to download high resolution image

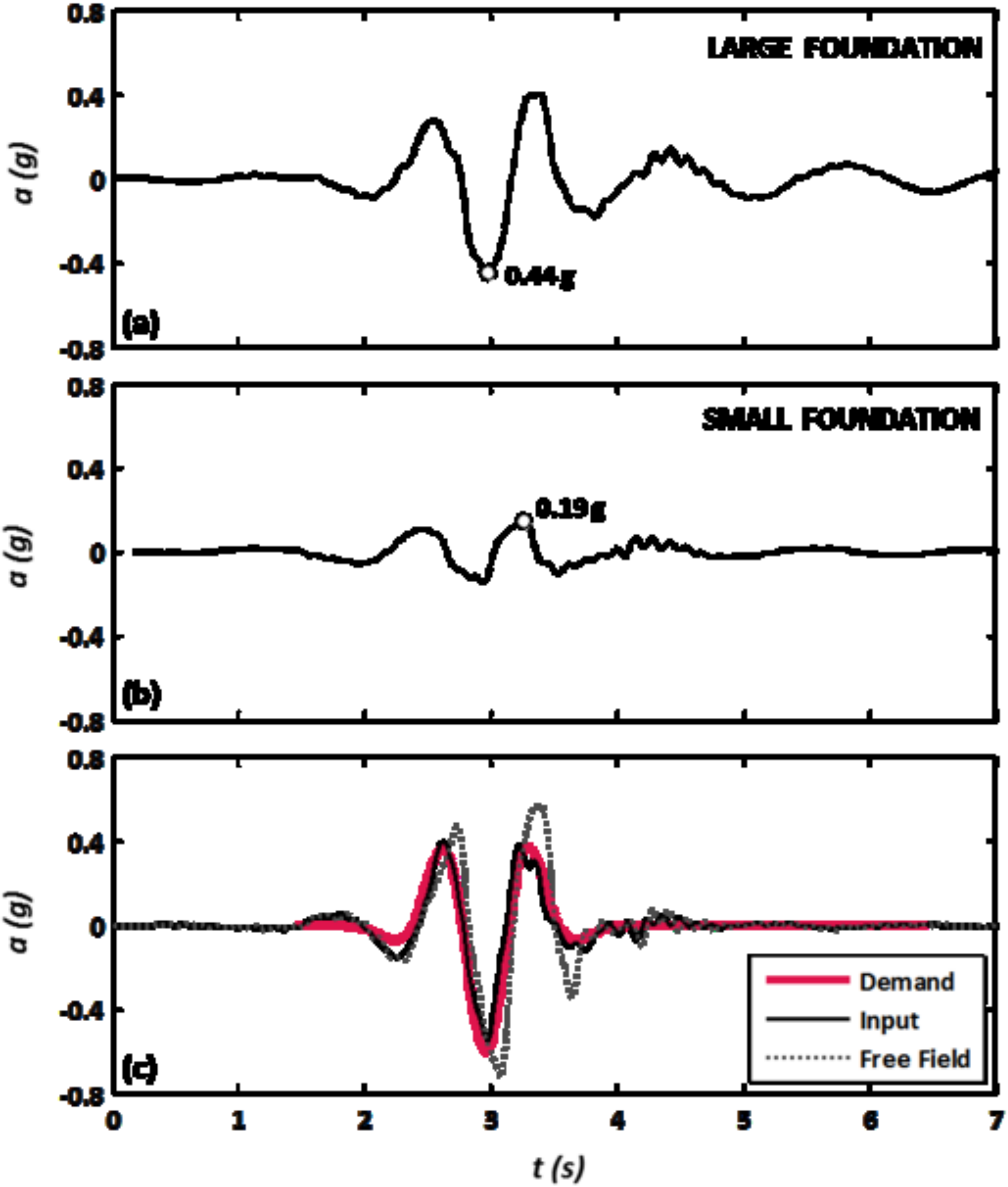

Figure 9. Accelerations recorded during centrifuge tests: (a) deck acceleration, large foundation; (b) deck acceleration, small foundation; (c) demand, input and free field motions.

Figure
Click here to download high resolution image

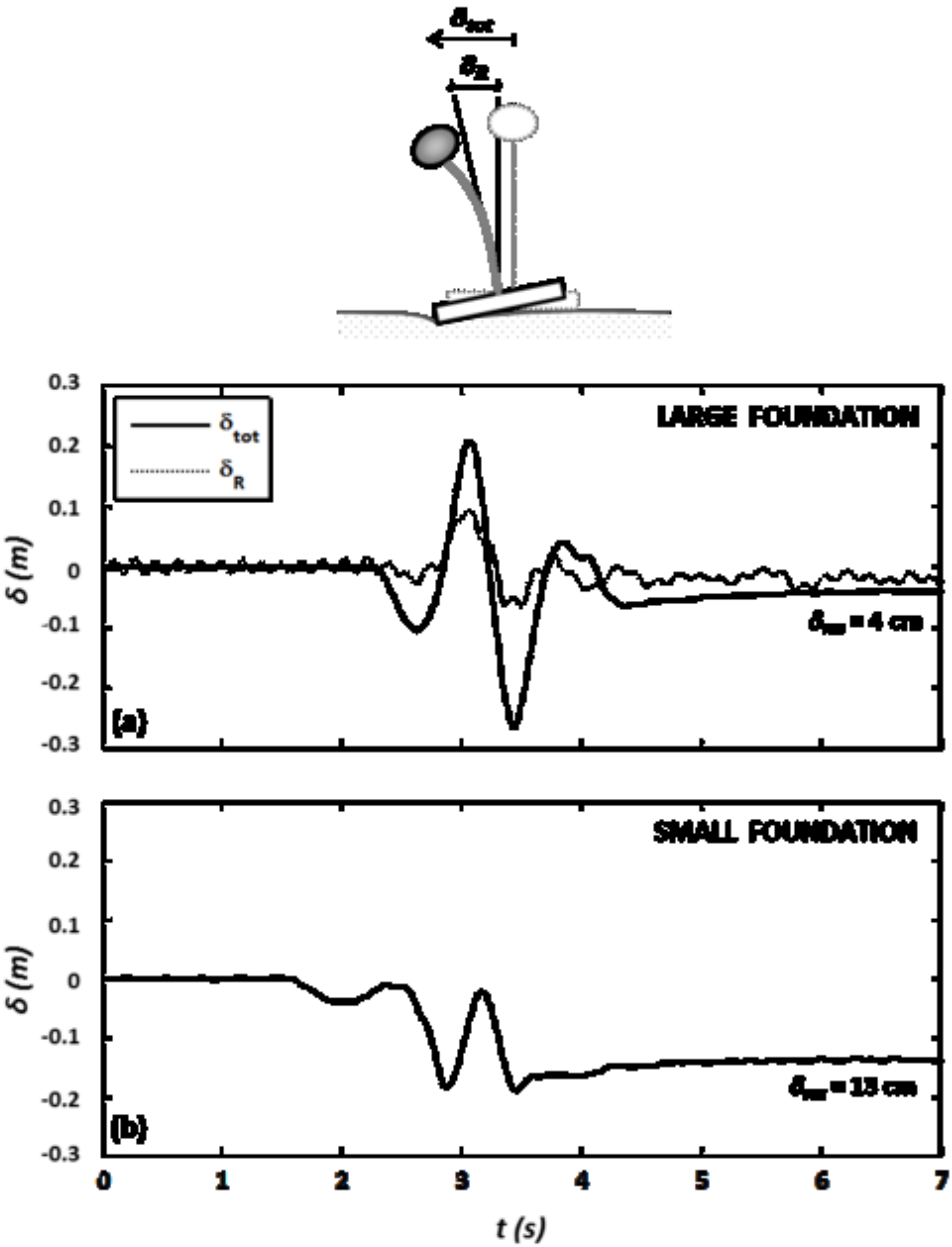

Figure 10. Deck drift time histories recorded during centrifuge tests for table excitation with Ricker 1Hz PGA = 0.6 g: (a) bridge pier on large foundation; and (b) bridge pier on small foundation.

Figure
Click here to download high resolution image

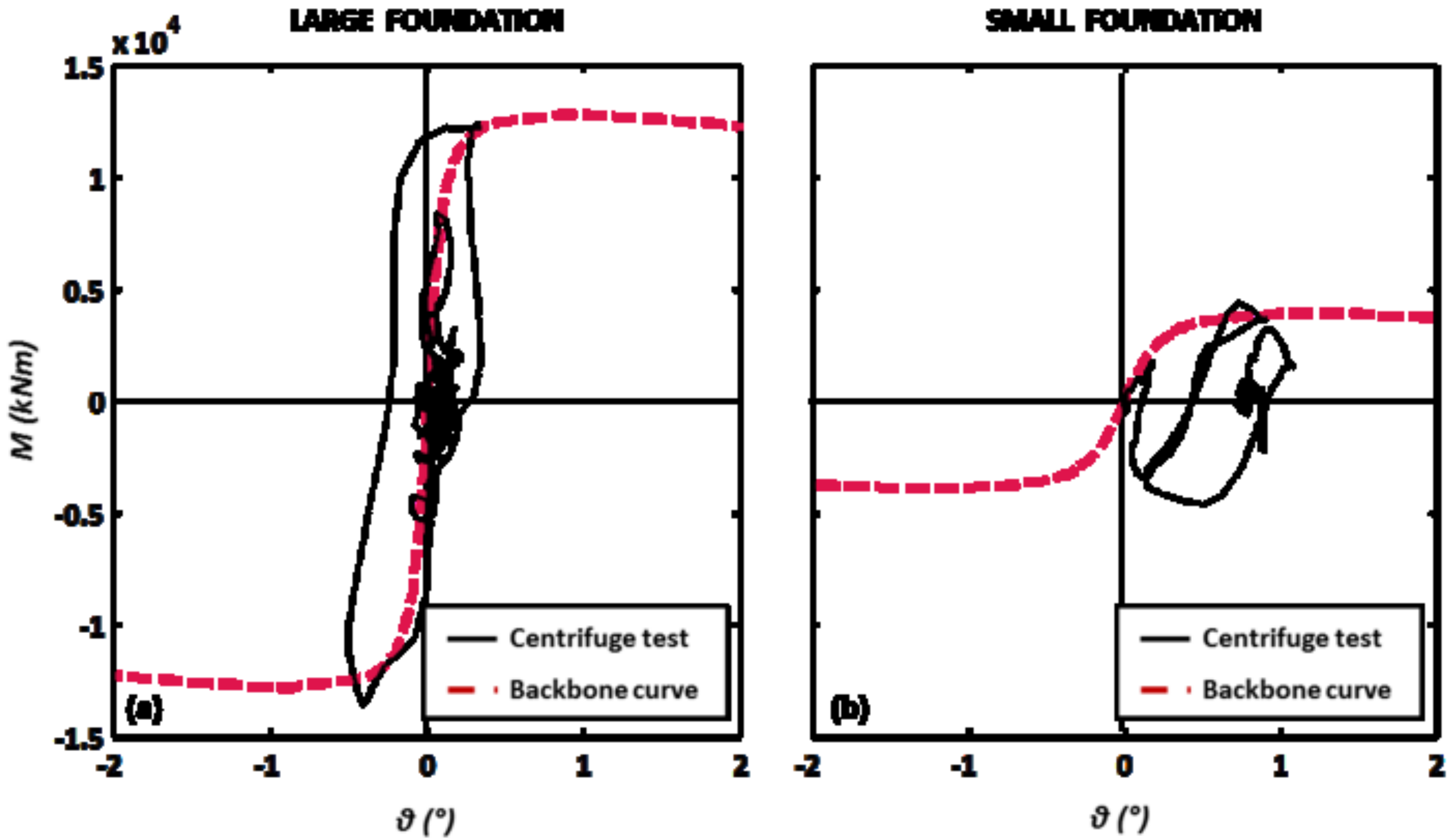

Figure 11. Foundation moment-rotation behaviour determined from centrifuge tests, compared to monotonic pushover ("back-bone") curves from FE model: (a) large foundation, and (b) small foundation.

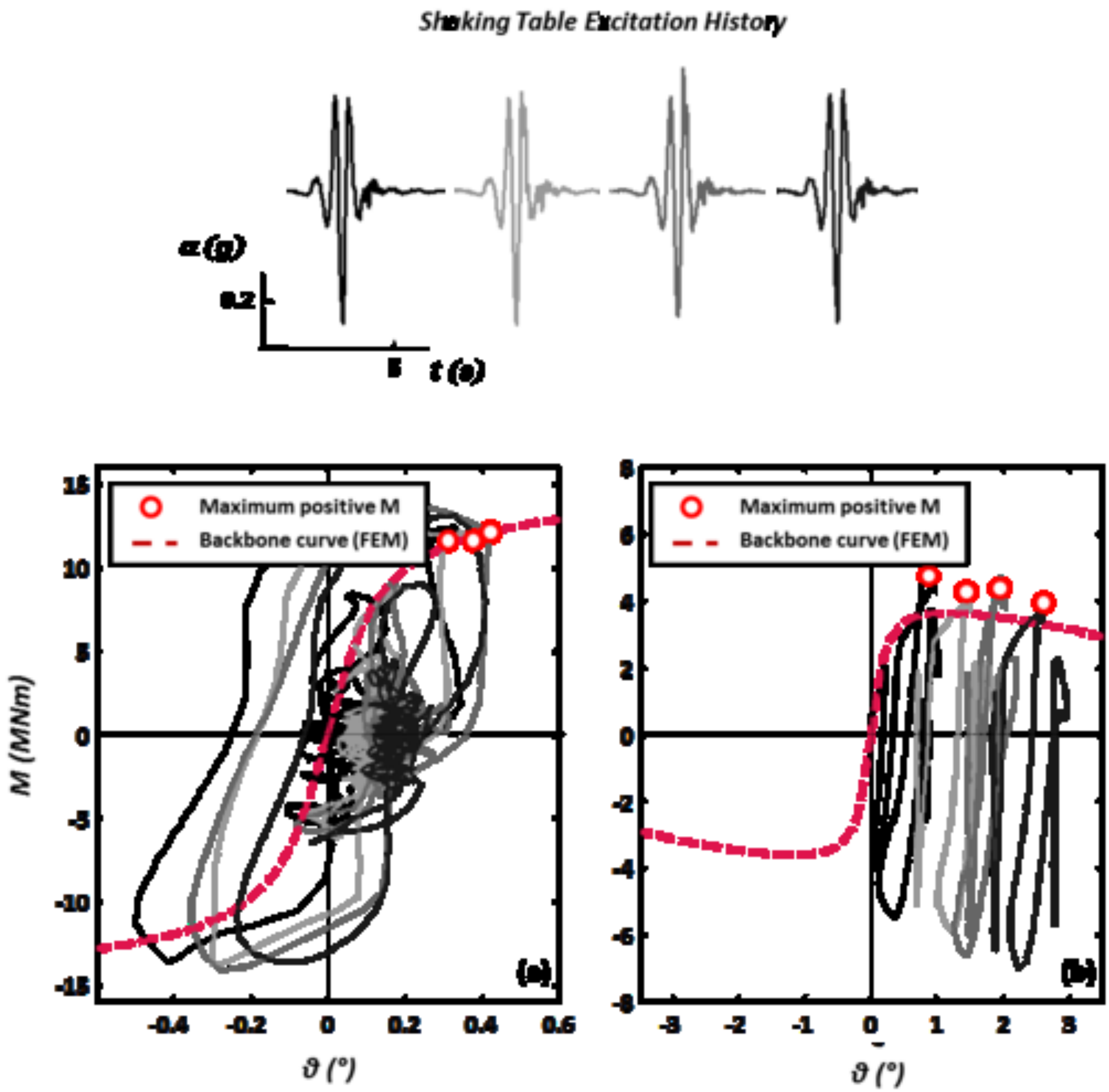

Figure 12. Foundation moment-rotation loops recorded for a series of four successive, practically identical, Ricker pulses (f_E = 1Hz, PGA = 0.6 g), compared with monotonic FE predictions: (a) large foundation, and (b) small foundation.

Figure
Click here to download high resolution image

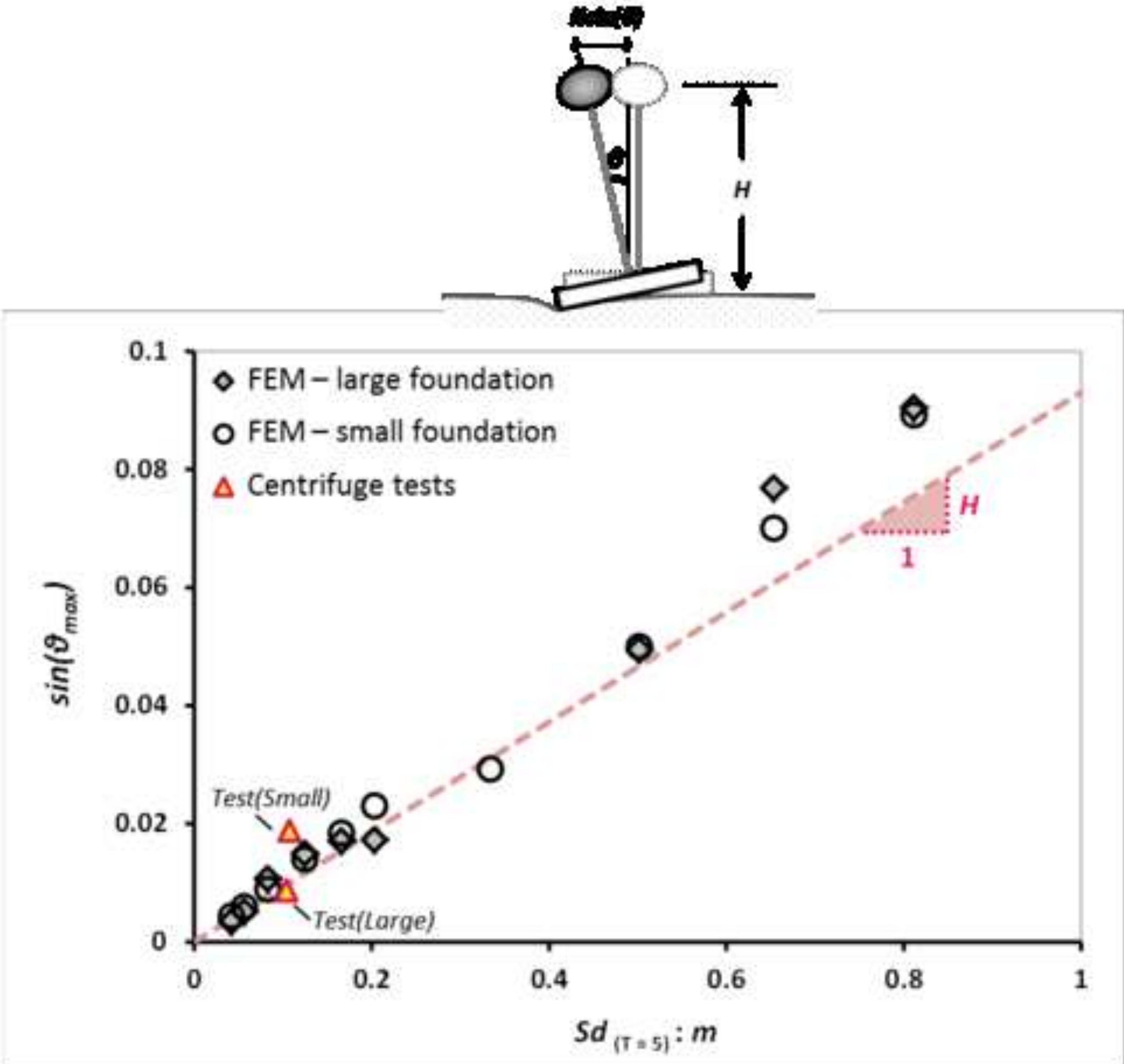

Figure 13. Summary of the numerical and experimental results: maximum drift experienced at the deck due to foundation rotation $\delta_{R,max} = H\sin(\vartheta_{max})$, with respect to the large period spectral displacement $S_{d(T=5sec)}$ of the free field excitation.

Table 1. Footing designs considered in this study (all values at prototype scale).

Property	Large footing	Small footing
Breadth (m)	7.5	4.0
Vertical load (MN)	4.9	4.0
Design shear load (MN)	1.0	0.7
Design moment (MNm)	10.6	7.6
FS_v (static)	18	3.5
FS_v (seismic)	1.7	0.6