Use of Ricker wavelet ground motions as an alternative to push-over testing

M. Loli & I. Anastasopoulos
Department of Civil Engineering, National Technical University of Athens, Greece

J.A. Knappett & M.J. Brown
Division of Civil Engineering, University of Dundee, UK

ABSTRACT: When conducting studies of seismic soil-structure interaction, it is useful to be able to define the pseudo-static 'push-over' response of the structure under static horizontal loads representing structural inertia. Normally, this would require separate centrifuge experiments with horizontal actuators attached. This paper describes an alternative procedure, using a specific type of ground motion (a Ricker wavelet) to obtain the push-over response, thereby allowing both this and the response to conventional earthquake shaking to be determined using the same (earthquake) actuator. Application of this technique to a 1:50 scale model highway bridge pier with two different shallow foundations is presented. The moment rotation ('backbone') behaviour of the footings was accurately determined to large rotations, as verified though independent 3-D non-linear finite element modelling. Ricker wavelet ground motions are therefore shown to be a useful tool for the identification of push-over system behaviour without requiring additional actuators.

1 INTRODUCTION

1.1 *Background and context*

In recent years, 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 damage (e.g. Gajan et al., 2005; Pecker, 2005; Paolucci et al., 2007; Gajan & Kutter 2008; Anastasopoulos et al., 2010; and Gelagoti et al., 2012). The key concept underpinning this design approach is that the yield moment within the foundation is lower than that which causes damage in the supported column or pier, resulting in shallow foundations which are smaller than those produced by conventional approaches (where the aim is to prevent the foundation from moving significantly). This relies on adequate characterization of the push-over response of the system (chiefly its moment-rotation behavior).

A recent collaborative study has been undertaken with collaboration between the National Technical University of Athens and the University of Dundee. This study has focused on the the use of rocking isolation for Eurocode 2/8 compliant reinforced concrete bridge structures. This has led to the further development of model concrete bridge piers for deployment in centrifuge studies (reported elsewhere). During this study, in addition to centrifuge tests using historical ground motions, it was necessary to check the moment-rotation capacity of the foundation designs. This posed a significant challenge as the timescale of the project meant that producing different centrifuge setups using horizontal actuators would not have been achieveable. It was therefore decided to investigate the possibilities of using a dynamic ground motion to produce a 'virtual push-over' of the structure-foundation model.

1.2 *Ricker wavelets*

When using a ground motion to simulate a push-over test, possible ground motions 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 modeling was conducted, as described in Section 2, to evaluate these different possibilities. The final Ricker pulse selected is shown in Figure 1, as both an acceleration (a) time history, and as spectral displacement (S_d) for the bridge system described in Section 2. It was observed that the Ricker pulse was generally able to mobilise a larger amount of the rotation capacity than the other pulse types, while also having the advantage that the waveform is continuous in acceleration, making replication of the motion easier with a mechanical system that can

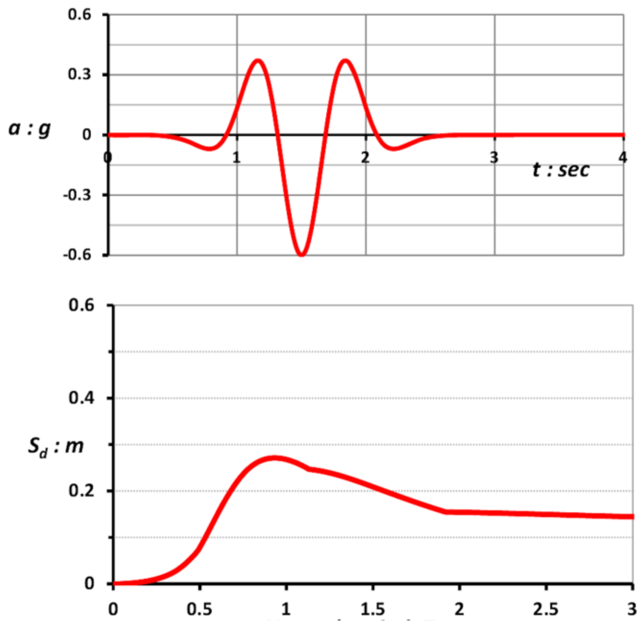


Figure 1. Ricker wavelet (1 Hz, 0.6g) and spectral displacement of highway bridge system (including effects of SSI)

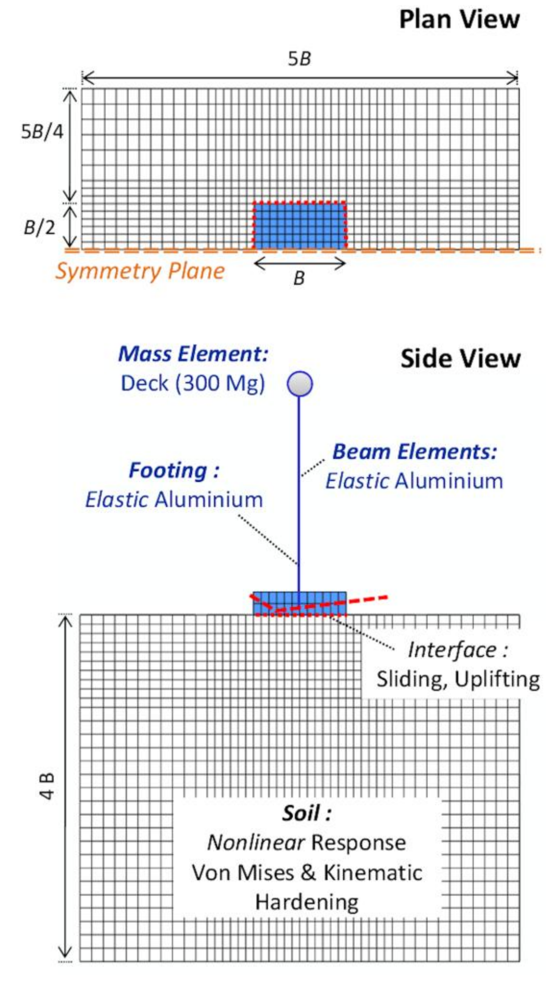
control the vibrations only within a certain band (at Dundee, this range is 40 – 400 Hz at model scale). It also has the advantage of the slip table automatically coming back to rest in its original starting position, without having a permanent displacement offset. This removes the need to re-centre the table before subsequent motions.

2 NUMERICAL MODELLING

2.1 *Model set-up*

As mentioned in the previous section, 3-D non-linear finite element modeling (FEM) was conducted using ABAQUS to investigate the behavior of the bridge structure of interest under different ground motions. These analyses also serve the function of class A predictions of the centrifuge test results which will be described in Section 3. Figure 2 displays views of the sufficiently refined FE mesh and indicates the main features of the numerical model. The geometry is that of the prototype pier, which represented a moderately tall (h = 10.75 m) highway bridge pier supported by a square (B x B) shallow foundation. The 1.5 m x 1.5 m square section pier column was simulated with 3-dimensional elastic beam elements assigned the geometric and stiffness properties of the aluminium used in the centrifuge tests E = 70 GPa and γ = 26 kN/m^3). The "deck" was modelled as a lumped mass on the top of the column. Given the relatively high position of thismass, second-order (P – δ) effects are of great importance and were therefore taken into account.

Taking advantage of symmetry upon the plane that crosses the foundation midpoint in the direction



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of shaking allowed simulation of only half of the full 3-D model, achieving greater computational efficiency.

Two models were tested; the only difference between the two refers to the foundation dimensions with the aim of comparing two different approaches to aseismic foundation design, which are summarized in Table 1. The larger footing (B = 7.5 m) follows current code provisions ensuring minimal displacements foundation interface under the design earthquake, i.e. the vertical factor of safety (FS_v) is greater than one under the expected seimic actions. The smaller alternative design (B = 4 m) promotes the newly introduced concept of foundation rocking isolation (FS_v < 1 under seismic conditions).

Table 1. Footing designs considered in this study.

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

2.2 *Soil properties*

The soil was modelled with nonlinear 8-noded hexahedral continuum elements C3D8. 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 simple 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. Despite its lack of generality and limitations, 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 3.

The same element type (C3D8), but with the assumption of linear elastic behaviour, was utilised for the footing. The soil – foundation interface was modelled using special contact elements, which allow sliding and uplifting to take place being governed by a hard-contact law and Coulomb's friction law in the normal and tangential direction respectively.

2.3 *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) with different intensities and dominant periods. For the sake of brevity, this paper presents results only for the case of excitation with a Ricker-1Hz (PGA = 0.6 g) pulse, which was selected as the most appropriate to use in the centrifuge tests.

Figure 4 presents the dynamic response of the two rotation plane in comparison to the monotonic backbone curves calculated through analysis of the same systems under horizontal push-over loading applied statically at the height of the deck. It is important to note that the calculated ultimate moment capacities are in good agreement with theoretical estimates. Furthermore, strongly nonlinear behaviour may be identified in the shape

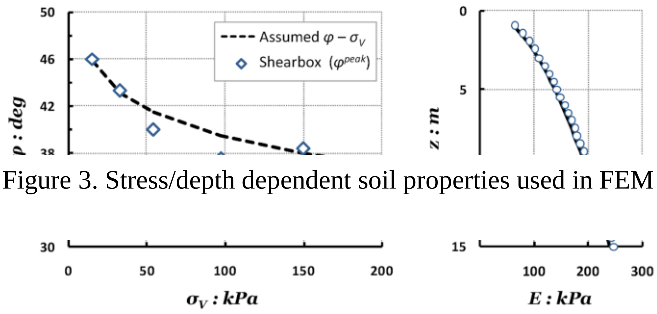


Figure 3. Stress/depth dependent soil properties used in FEM

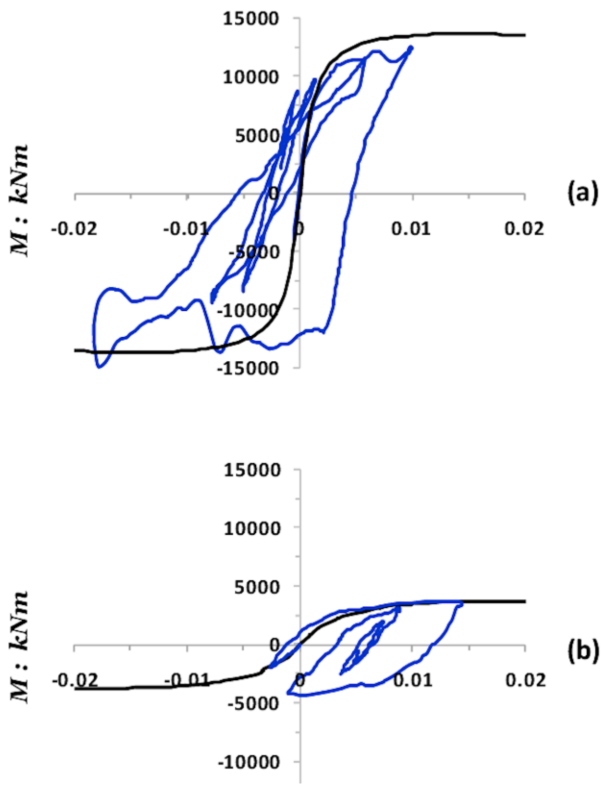


Figure 4. Foundation moment-rotation response under base excitation with a 1 Hz 0.6g Ricker pulse, compared to monotonic back-bone curve (all results from FEM)

of the single significant loop produced by the single significant pulse included in the excitation for both foundations. Excessive material nonlinearity is manifested especially in the case of the smaller foundation which leads to some considerable permanent rotation. On the other hand, the response of the larger foundation is accompanied by significant uplifting (loss of contact with the supporting soil) and hence the M–θ loop resembles the well-known characteristic S-shape. Most importantly, in both cases the dynamic loops approximate the backbone curves very satisfactorily indicating that the Ricker pulse may be used in shaking table tests to indirectly measure the ultimate lateral load foundation capacity.

3 CENTRIFUGE MODELLING

3.1 *Model setup*

Two dynamic centrifuge tests were conducted on 1:50 scale physical models of the bridge pier system, with identical super structure properties, but with different foundations (B = 7.5 m and B = 4 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,

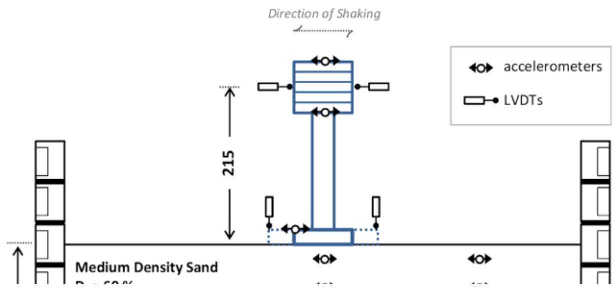


Figure 5. Centrifuge model layout (small foundation shown; large foundation indicated by dashed line). All dimensions in mm at model scale.

ϕ'_{crit} = 32°), prepared uniformly by air pluviation to $D_r \approx 60\%$. 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 accelerometers and LVDTs as shown in Figure 5. The models were loaded onto the Actidyn Q67-2 servo-hydraulic earthquake simulator (EQS) which has recently been installed on the University of Dundee beam centrifuge (see Bertalot et al. 2012 for a description of this actuator).

All subsequent results in this paper will be given at prototype scale.

3.2 *Motion replication and dynamic response*

Figure 6 shows the accelerations measured at the deck in each of the two models (along with peak values), alongside the demand motion, slip table motion (a_{table}) and free-field ground motion (a_{ff}) (top-most instrument in the right-hand column of buried accelerometers in Figure 5. It can be seen that the EQS faithfully reproduces the input motion, and that there is some free-field amplification within the soil.

Figure 7 shows the lateral drift of the deck of the bridge both as the total component, δ_{tot} (due to sliding, δ_s, flexural displacement of the pier, δ_F and rotation δ_R respectively). Due to a failure in one of the LVDTs foundation movement in the vertical direction, it was not possible to independently measure δ_R for the case of the small foundation; however, in this case the pier is expected to have experienced almost purely rotational motion so $\delta_R \approx \delta_{tot}$.

4 PUSH-OVER RESPONSE

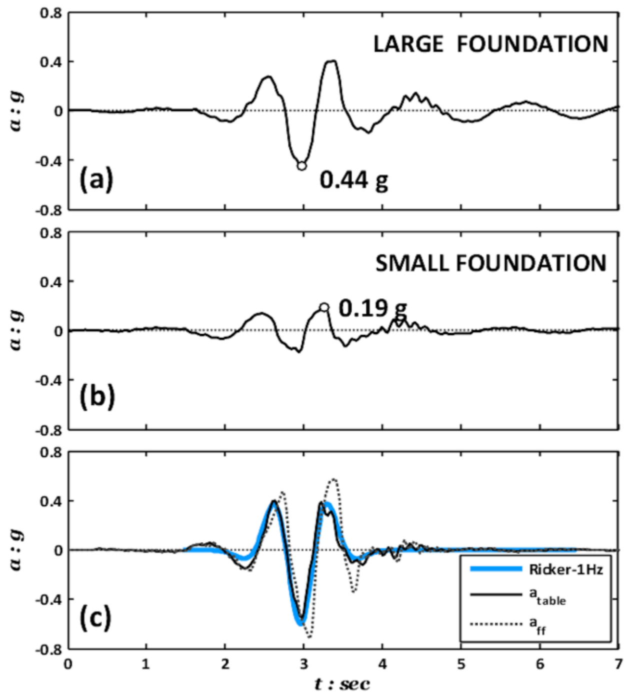
The values of δ_R were used to determine the rigid body rotation of the foundation (θ), as:

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

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 and recognising that the system is a cantilever:

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

Figure 8 shows the moment rotation loops derived for the centrifuge data, and also plots the static push-over curve determined from the FEM, for the foundation sitting on the soil, and the foundation sitting on a rigid (non-yielding) base. Considering the small foundation first, it is clear that for the case of foundations exhibiting substantial rocking, even a single Ricker pulse is sufficient to mobilise the moment capacity of the foundation well into the non-linear (large rotation) domain. The match to the



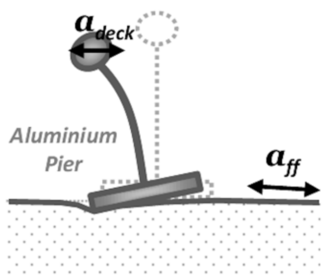


Figure 6. Acceleration time histories recorded during centrifuge tests: (a) deck acceleration with large foundation; (b) deck acceleration with small foundation; (c) ground motions.

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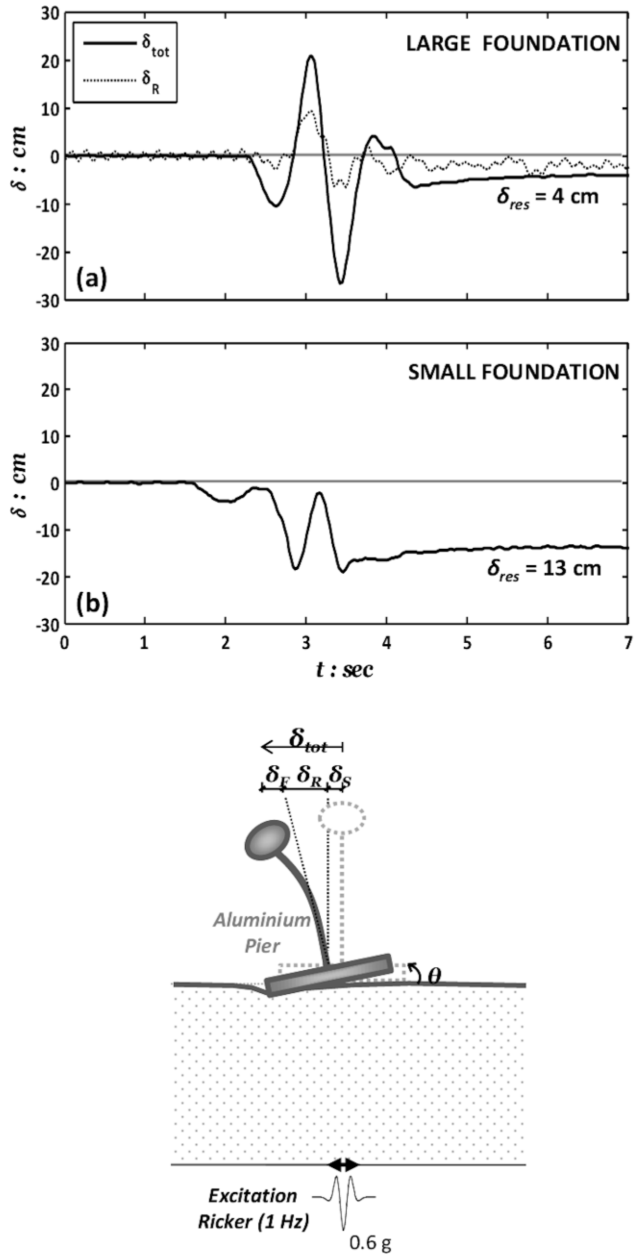

Figure 7. Deck drift time histories: (a) bridge pier on large (conventional) footing; (b) bridge pier on small (rocking) footing.

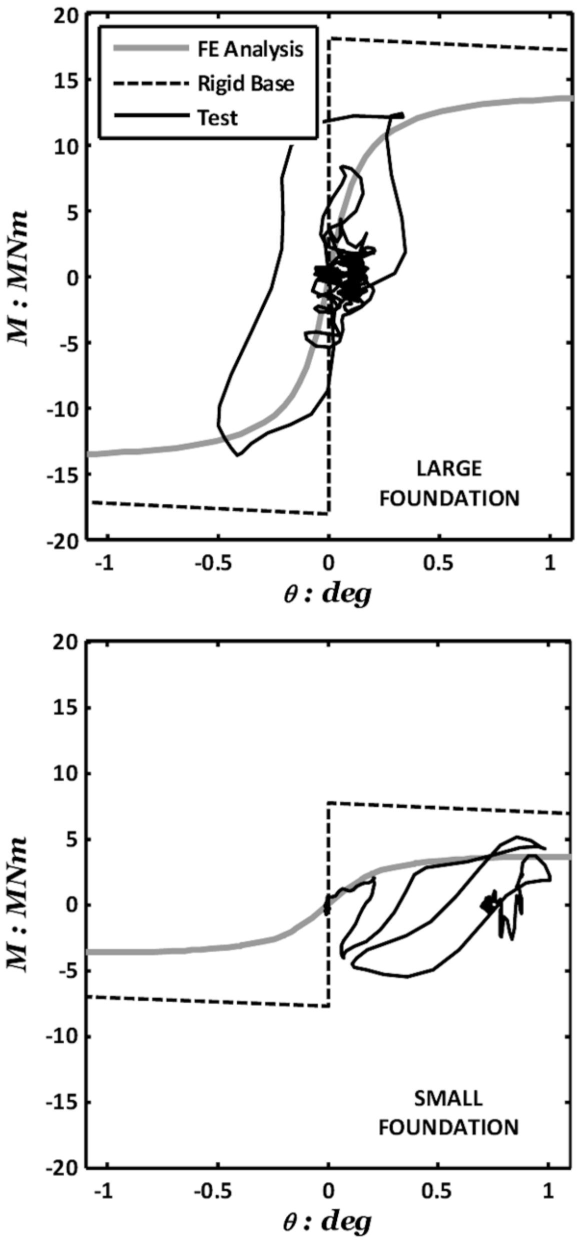

Figure 8. Foundation moment-rotation behavior from centrifuge tests, compared to monotonic push-over (backbone) curves from FEM for soil and for rocking on rigid layer.

numerical backbone curve is extremely good, and suggests that this could be determined from the centrifuge test data by fitting an envelope around the centrifuge data within the positive quadrant.

In the case of the large foundation, much smaller rotations are mobilised. However, the maximum and minimum points of the loops lie extremely close to the backbone curve. In this case, it is suggested that the centrifuge testing with the Ricker pulse is perhaps more useful for validating the push-over response determined from FEM; however this limitation could also have been drawn from Figure 4.

Although not shown here in the interests of brevity, the centrifuge models were subsequently subjected to further consecutive 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 push-over response.

5 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 push-over 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. It is expected that this method will be particularly useful in characterising system response in future centrifuge tests of seismic SSI problems, particularly given the current trend towards newer foundation designs which employ foundation rocking to seismically isolate the structure for which determination of the push-over response is extremely important.

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