Seismic performance of buildings with structural and foundation rocking in centrifuge testing

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Summary
Rocking motion, established in either the superstructure in the form of a 2-point stepping mechanism (structural rocking) or resulting from rotational motion of the foundation on the soil (foundation rocking), is considered an effective, low-cost base isolation technique. This paper unifies for the first time the 2 types of rocking motion under a common experimental campaign, so that on the one hand, structural rocking can be examined under the influence of soil and on the other, foundation rocking can be examined under the influence of a linear elastic superstructure. Two building models, designed to rock above or below their foundation level so that they can reproduce structural and foundation rocking respectively, were tested side by side in a centrifuge. The models were placed on a dry sandbed and subjected to a sequence of earthquake motions. The range of rocking amplitude that is required for base isolation was quantified. Overall, it is shown that the relative density of sand does not influence structural rocking, while for foundation rocking, the change from dense to loose sand can affect the time-frequency response significantly and lead to a more predictable behaviour.

KEYWORDS
centrifuge testing, dry sand, foundation rocking, impact, Morse wavelet transforms, structural rocking

1 | INTRODUCTION

Following the aftermath of the New Zealand earthquakes (2011), it has become clear that buildings should be designed to be easily repaired after a seismic event, so that disruption is minimised. At the same time, the recent earthquake events in Chile and Mexico caused a detrimental loss of building and infrastructure stock. From a reinvesting point of view, a huge opportunity arises when reconstructing these areas. However, to reduce similar economic losses in the future, the current practice of fixed-base ductile design may not be optimal.

While current design procedures ensure sufficient ductility to ensure life safety even in extreme loading events, extended structural damage may result, favouring building demolition rather than repair. Standard seismic designs
typically do not utilise many strategies that can lead to easily repairable structures. For example, large slender structures can resist earthquake loading by utilising their rotational inertia while rocking motion is allowed on their foundation base. Alternatively, a rocking foundation can be used by mobilising both the structural rotational inertia and damping because of soil deformation below the foundation. The former (structural rocking) limits the force demand by avoiding excitation through a drastic reduction in stiffness upon uplift (ie, initiation of rocking), while the latter (foundation rocking) also takes advantage of additional damping from soil, displacing the ductility demand from the superstructure to the ground. Structural and foundation rocking have been studied extensively over the last 60 years, and although the seismic isolation principle is the same (ie, uplift reduces force demand), these approaches have never been directly compared experimentally.

This paper provides a novel, experimental, and direct comparison of the 2 types of rocking through a campaign with physical modelling by means of centrifuge testing. Two building models, one with a partial release between footings and columns and another with fixed footing-column connections, were designed to exhibit structural and foundation rocking respectively. The 2 models were placed on dry sand and tested side by side under a sequence of earthquake excitations within a centrifuge environment.

2 | OVERVIEW OF ROCKING SYSTEMS

Structural and foundation rocking can be categorised according to the type of superstructure considered and the deformability of the support conditions (Figure 1). In the simplest form of structural rocking, a rigid block is considered rocking on a rigid base. For this configuration, it is known that larger blocks are less likely to overturn than geometrically similar smaller blocks, that they should have their own rocking spectra, and that generally their rocking behaviour is chaotic.

Recognising that allowing structural rocking limits the base overturning moment that is resisted by the structure, but may result in large rigid body displacements, the rigid block type of superstructure was extended to structural systems that are equipped with post-tensioning and special damping devices to suppress large rotations during rocking. The type of damping device can vary from shear elements and bending elements to tendons. While these systems have an inherent reccentreng capability, they have typically been studied on a rigid base. The addition of elastoplastic

![Matrix of various rocking systems with respect to the superstructure properties and the deformability of their base](image-url)
elements for damping might effectively reduce displacements; however, the interaction of the structural deformations with the rocking mode becomes increasingly important.

Consideration of the flexibility of the superstructure during structural rocking provides a more accurate representation of reality, particularly for slender structures. The current understanding is that modelling of the response without explicit consideration of the impact forces developing during recentring is not sufficient, because these impacts contribute in the total excitation of the superstructure. Particularly, previous studies have shown other fundamental characteristics that emerge during the rocking response, such as the beneficial effect of slenderness and the shift in natural frequency of structural deformations. Additionally, previous studies have considered viscous dampers, soft pads, or buckling restrained braces at the rocking interface just above the foundations. These devices are used to provide energy dissipation and mitigate the impact generated at the recentring in practical applications, as an alternative to allocating energy dissipating devices in the superstructure.

Meanwhile, the rigid base assumption often adopted in design does not take into account the properties of soil explicitly, ignoring the inherent damping provided by soil deformation. The effect of soil has been addressed in research, often by using viscoelastic springs below the rocking superstructure. Generally, an apparent rocking mode with a frequency lower than the frequencies of structural deformations, or a soft system in the case of rigid body, appears. For rigid blocks, as the slenderness increases, the effect of soil flexibility and damping decreases.

Considering soil explicitly for the above categories of structures (rigid block, linear elastic superstructure with/without damping elements) results in deformation beneath the foundations. The rigid block can be reduced to a single rocking footing or visualised as a reinforced concrete wall as part of a building, or a bridge pier. Furthermore, when all of the individual block foundations are allowed to rock on soil, an admissible kinematic mechanism must be formed, and this is achieved by allowing yielding and consequently damping to develop at the ends of the beams/columns, where plastic hinges would form. While the major advantages of limiting the earthquake-induced force because of local uplift and increased damping from soil hysteresis are maintained (and at the same time, no impacts occur that excite the superstructure), soil settlements can be a barrier to practical implementation. This is particularly true for framed structures with plastic hinges at beam members, which may be more prone to differential settlements. Recently, however, it was shown that the plastic hinge mechanism might not be necessary and that energy dissipation from soil alone can be sufficient.

To summarise, structural rocking has been predominantly studied on the assumption of a rigid base or viscoelastic springs. For the design scenario that the columns of a building are detailed at their base such they can pivot about a point on their foundations, the effect of an elastoplastic soil, including the effects near the vicinity of the impact points, has not yet been explored. On the other hand, foundation rocking predominately refers to structures rocking below their foundation level. However, the rocking of discrete footings in a similar step mechanism to structural rocking has not been studied before, and in situations where a mat foundation is not appropriate, this type of system could be a useful alternative. Therefore, this paper addresses rocking above and below the foundation in a common test with a sequence of earthquake excitations on soil, to both reveal the characteristics of structural and foundation rocking in this case and to compare them (Figure 2). Hence, the terms “RA” and “RB” are used to indicate rocking above and rocking below the foundation. The main objective is to assess the acceleration and force demand that rocking systems experience during their motion as a result of strong ground shaking and of the impacts developing at the interface of the superstructure with the foundation or the interface of the foundation with the soil because of rocking.

**FIGURE 2** Two models of flexible structures rocking on discretised footings and soil, below the foundation level (RB, left) and rocking above that (RA, right)
3 | EXPERIMENTAL SETUP

3.1 | Model characteristics and instrumentation

Two building models designed to represent structural and foundation rocking were tested in an artificial gravitational environment using the Cambridge centrifuge beam with a model scale $N_g = 33 \, g$ (Figure 3). Structural rocking is expected to occur to the model rocking above its foundation level (hereafter named RA), while foundation rocking is expected to occur to the model rocking below its foundation level (hereafter named RB). The 2 models represent 3 to 4 storey buildings with shallow foundations resting on a sand bed of 7.5 m and are assumed linear elastic. The buildings were discretised with 2 slab masses and had a set of columns resisting gravity loads and a set of braces resisting lateral loads. The columns’ thickness was designed to be sufficient to carry gravity loads but to contribute very little in the lateral stiffness. As a result, the fixity of the columns with the footings did not govern the lateral stiffness, which was then tuned in conjunction with the 2 slab masses to achieve a typical period of 0.6 to 0.7 second in prototype scale. Table 1 shows the storey-lumped masses, which include the mass of slabs along with half of the columns and braces above and below the given storey. Note that an identical lumped mass for both the first (n = 1) and the second (n = 2) storeys was achieved by modifying the slab thicknesses. The geometry of the 2 models was the same; however, the mass of the footings fixed to the superstructure of the model RB cause it to have a different

![Diagram](image-url)
rocking slenderness than RA. Furthermore, the slenderness of RB is also dependent on an assumed point of rotation of its footings; assuming rotation occurs about the outer edge of the footing causes minimum slenderness, while assuming that a rotation point at the centre of the footing causes a larger slenderness (see Table 1). Finally, an FE model with beam-column and shell elements (with distributed mass), pin connections for the braces and fixed connections for the other elements, was created to approximate the first 2 lateral modal shapes during full contact. For a 2-degree of freedom system simplification of the building models, these modal shapes are not orthogonal and hence only indicative.

The static bearing pressure below each footing was approximately 80 kPa (Table 1). Care was taken to sufficiently separate the discrete footings of each building model to minimise structure-soil-structure interaction (which for adjacent buildings is known to amplify force demands, see36), by ensuring that any footing to footing (or boundary) distance is more than 2 times the width $b$ of a single footing (Figure 3). Overall, the design was chosen to ensure that the preuplift response is as similar as possible for the models and enables direct comparison between structural and foundation rocking. More information on the design properties and on a previous proof of concept centrifuge test can be found elsewhere.37

Both the response of the building models and the soil were monitored with accelerometers (see Figure 3). The accelerometers on the structure were MEMS ADXL 193 with a 400 Hz built-in filter, and the soil was monitored with embedded piezoelectric DJB A23 accelerometers. In addition, strain gauges were attached to the bracings and calibrated to measure internal forces. The signal-to-noise ratio for accelerometers and strain gauges was sufficient to produce clear raw signals (presented in this paper) without filtering at the MEMS cut-off frequency.

Finally, because this is a purely experimental investigation aimed to understand dynamic behaviour, actual model scale units are used, although the prototype scale units in acceleration, force, and frequency are also provided.

### Table 1 Characteristics of models RA and RB

| Properties                                      | RB                              | Prototype Scale | RA                              | Prototype Scale |
|-------------------------------------------------|---------------------------------|-----------------|---------------------------------|-----------------|
| First mode design period                         | 0.019 second                    | 0.66 second     | 0.02 second                     | 0.70 second     |
| Experimental first mode frequency               | 53 Hz                           | 1.6 Hz          | 50 Hz                           | 1.5 Hz          |
| Experimental second mode frequency              | 147 Hz                          | 4.5 Hz          | 136 Hz                          | 4.1 Hz          |
| Total mass of uplifting parts                    | 2.4 kg                          | 86 metric tonnes| 2.1 kg                          | 75 metric tonnes|
| Assumed 2DOF mode shapes                        | $\varphi_1^T = (1.038)$        |                 | $\varphi_1^T = (1.045)$        |                 |
|                                                  | $\varphi_2^T = (-0.391)$       |                 | $\varphi_2^T = (-0.471)$       |                 |
| Friction angle $\varphi'$                       | $33^\circ$                      |                 |                                 |                 |
| Factor of safety for vertical loading $FoS$      | $FoS = 2.3$, design approach 1/2 of EC7$^\text{35}$ |                 |                                 |                 |
| Storey lumped mass ($m_n$, $n = 1, 2$)          | $m_n = 0.83$ kg ($m_n = 30$ metric tonnes, prototype scale) |                 |                                 |                 |
| Slenderness ratio, $\tan(\alpha)$               | Internal point: 0.25            |                 | 0.30                            |                 |
| For a distributed mass configuration             | Middle point: 0.34              |                 |                                 |                 |
|                                                  | External point: 0.42            |                 |                                 |                 |

### 4 | GENERAL OVERVIEW OF TESTS

#### 4.1 | Input excitations and spectral response

The centrifuge campaign involved 4 sets of centrifuge tests. Two of these were conducted using a hydraulic servo-shaker,38 and only these are discussed here. The first set of tests involved dry, dense Hostun HN31 sand, while the second set of tests involved dry, loose sand of the same type (Table 2). For model RA, a special energy dissipating device (fuse) was designed and installed at the interface of rocking, between the top surface of the footings and the columns' free end.37 Each set of tests (1 set for dense sand and 1 set for loose sand) involved 1 flight with the fuses mounted on the RA structure (RA FUSE = ON in Table 2) and 1 flight with no fuses on the RA structure. Each flight involved the same sequence of earthquakes. No structural modifications were made to model RB between flights, so the second flight is
simply a repeat of the first. Note that the effect of the fuses on the performance of model RA was minimal, and so is not the focus of this paper, and is only peripherally discussed.

Table 2 lists the input motions, which include Kobe and Imperial Valley earthquake records as well as both single sine pulse records (nominally 30 and 50 Hz) and cyclic (30-cycle) harmonic records (nominally 30 and 50 Hz). Figure 4 shows the response spectra ($\zeta = 5\%$) for each motion as obtained from the accelerometer data near the soil free surface (sensor 8836, Figure 3). In most cases, the repeated tests on the same sand density provide nearly identical free surface soil excitations, while the corresponding tests on the different sand density are also quite similar in the frequency range of practical interest (0-100 or 0-3 Hz in prototype scale). Note that the large discrepancy in the peak value between dense and loose sand for the Imperial Valley record was found to be because of the shaker’s actual input at the base rather than any modifications from soil amplification/attenuation. Similarly, the difference between the peaks of the 50 Hz cyclic motion in dense sand was caused by specifying a different input amplitude.

4.2 Frequency content identification

The changing frequency content of the response with time was evaluated using continuous wavelet transforms.\(^3\) The Morse wavelet was selected which is controlled by the time-frequency product $\beta\gamma$. Generally, $\gamma = 3$ results in a zero-skewness wavelet about the frequency axis, while larger values of $\beta$ decrease the discretisation in the frequency domain. A value of $\gamma = 3$ was assumed, while several values of $\beta$ were evaluated in a parametric investigation. A value of $\beta = 27$ was found to provide clear results so is used throughout this paper. To mitigate boundary effects from the wavelet transformation, a reverse boundary condition was used which mirrors the signal but with a sign reversal. A weighting function $f(s) = 1/s$ was applied which is generally recommended for oscillatory signals.\(^4\)

Figure 5 shows example time-frequency maps of the storey response of models RA and RB, along with the input excitation at the base of the centrifuge box (sensor 9082, Figure 3) from the nominal 50 Hz pulse record in dense sand.

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**TABLE 2** Experimental programme with soil and input motion characteristics

| Test Set | Relative Density | RA Energy Dissipation (RA FUSE) | Test (Flight) No. | EQ-1 | EQ-2 | EQ-3 | EQ-4 | EQ-5 | EQ-6 |
|----------|------------------|---------------------------------|------------------|------|------|------|------|------|------|
| Dense sand | 96% | ON | Test-1 Kobe (1.1) | Imperial Valley (1.2) | 50 Hz pulse (1.3) | 30 Hz cyclic (1.4) | 50 Hz cyclic (1.5) | N/A |
| | | OFF | Test-2 Kobe (2.1) | Imperial Valley (2.2) | 50 Hz pulse (2.3) | 30 Hz cyclic (2.4) | 50 Hz cyclic (2.5) | N/A |
| Loose sand | 58% | ON | Test-1 Kobe (1.1) | Imperial Valley (1.2) | 50 Hz pulse (1.3) | 30 Hz cyclic (1.4) | 50 Hz cyclic (1.5) | 30 Hz |
| | | OFF | Test-2 Kobe (2.1) | Imperial Valley (2.2) | 50 Hz pulse (2.3) | 30 Hz cyclic (2.4) | 50 Hz cyclic (2.5) | pulse (2.6) |

Model scale frequencies 50 and 30 Hz correspond to prototype scale periods 0.66 and 1.10 seconds respectively.

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**FIGURE 4** Spectral response near the soil free surface for $\zeta = 5\%$
As also visualised in the linear elastic response spectra (Figure 4), the principle excitation frequency was approximately 70 Hz, higher than the nominal value of 50 Hz specified, and with appreciable higher frequency content between 100 and 400 Hz also present (Figure 5A, B). The storey accelerations of both buildings are characterised by an initial large duration cycle with clear higher frequency oscillations (Figure 5D, F, I, K), while at the same time, uplift

**FIGURE 5** Dense sand, Test-1 EQ-3: pulse excitation (A, B), vertical acceleration response of model RA (C), storeys' time-frequency response of model RA (D-G) and similarly for model RB (H-L).
occurred as evidenced by the vertical accelerometers placed at the column bases (Figure 5C, H). After some initial rocking, the structure regains full contact with the soil and the motion slowly damps out, providing a full-contact free vibration trace. At the onset of the excitation, the time-frequency maps of the storey accelerations (Figure 5E, J, G, L) reveal a high-frequency response locally in time, along with a lower frequency component that continues throughout the response. The clear peaks in the higher frequency range (>100 Hz) occur predominantly during the very brief rocking response (t = 5.65–5.67 seconds) and provide clear evidence of vibration during rocking (uplift), after which the higher frequency oscillation damps out much quicker than the lower frequency response. The peak frequency content in the higher frequency range (>100 Hz) appears to be at a slightly lower frequency for RB than for RA. Meanwhile, in the lower frequency range (<60 Hz), the peak frequency response for RA is initially at about 35 Hz during rocking, and then increases to the first mode fixed base natural frequency of approximately 50 Hz during the full contact free vibration stage. The initial rocking frequency is not fixed, but dependent on amplitude of the rocking response. For RB, the lower frequency during this initial rocking stage is barely evident in the time-frequency plots; it appears the single rocking cycle has a smaller amplitude, and thus a higher natural frequency (~50 Hz for RB compared to ~35 Hz for RA), and thus, the rocking phase is not clearly distinguishable from the full contact natural frequency in the time-frequency plots.

4.3 Observed force demand

The total lateral accelerations $\ddot{u}_{tn}$ recorded at the storey slabs ($n = 1, 2$) were used to extract the lateral external forces $F_{E_n}$ developed on the storey slabs because of both the ground motion and the subsequent rocking motion, and consequently the total external force, $F_{E,x} = \sum_{n=1}^{2} F_{E,R,n} = \sum_{n=1}^{2} m_n \ddot{u}_{tn}$. The peak value of the total external force was normalised with reference to each building’s weight and plotted for each input motion for both types of sand (Figure 6). Generally, the low-frequency excitation caused larger force demands, whereas the Imperial Valley excitation resulted in a full contact response and the smallest force demand. In addition, the added fuse had no significant effect on RA’s force demand. In all earthquakes and regardless of the type of the sand, model RA resulted in a larger external force demand than model RB. The effect of sand density on the force demand was frequency dependent. For excitations with larger high frequency content (Imperial Valley) and the 50 Hz excitations, loose sand resulted in a larger force demand than dense sand for RA. On the contrary, for excitations with a larger low-frequency content, such as Kobe and the 30 Hz cyclic and pulse motions, dense sand resulted in larger force demand than loose sand for RA. Regarding RB, a mixed trend is observed, but the difference was larger for the low-frequency excitation of 30 Hz compared to the other excitations.

4.4 Extraction of shear force demand from accelerometers

To extract the shear force demand for each storey, the superstructure damping was first approximated. Specifically, the discretisation of the models to 2DOF in full contact conditions was used. A classic damping matrix based on the modal characteristics of this configuration is shown in Equation (1). The modal damping ratios refer to the damping as obtained from full contact free flexural vibration tests on each model before the loading in the centrifuge beam. These values are only representative and were difficult to measure with accuracy. The storey damping forces $F_D$ were evaluated as

![Graph showing observed force demand normalised to each building model’s weight across all earthquakes for both types of sand](image)
\[ F_D = cu_r \Rightarrow \left\{ \begin{array}{l} F_{D,1} \\ F_{D,2} \end{array} \right\} = m \left( \sum_{m=1}^{2} \frac{2\zeta_m \omega_m}{M_m} \phi_m \phi_m^T \right) \left\{ \begin{array}{l} \dot{u}_r,1 \\ \dot{u}_r,2 \end{array} \right\} \] (1)

Therefore, the storey shear forces \( F_{S,n} \) were obtained as \( F_{S,2} = -F_{E,2} - F_{D,2}, \) \( F_{S,1} = -F_{E,1} - F_{D,1} + F_{D,2} + F_{S,2}. \) The amplitude of the damping forces was found to be very small compared to the external forces; therefore, the trend from the external forces developed because of ground shaking and rocking was also reflected in the shear forces.

### 4.5 Base isolation effect

After extracting the storey shear forces, the base isolation effect was investigated with reference to a fixed base linear elastic solution of the base shear value (Figure 7). The input considered was the excitation as recorded below each building model (accelerometers 8888 and 8838, Figure 3). The linear elastic solution to the base shear was obtained using the SRSS method and the response spectra for each earthquake, which was obtained for different values of damping corresponding to the 2 first lateral modes of the structure (as identified in the free vibration traces of the building models during centrifuge testing). The variation in response because of the damping estimate is shown in Figure 7 in the form of error bars representing 1 standard deviation away from the mean value. The largest difference to the SRSS base shear (Figure 8) was, for both building models, at the 50 Hz cyclic and pulse records; the large standard deviation occurred because of the sharp peak in the response spectra of these motions. The Kobe excitation also generally showed an appreciable reduction in base shear compared to the linear elastic solution. Regarding the Imperial Valley excitations, the linear elastic solution produced base shear values smaller than those observed in the centrifuge, suggesting that structural damping in the centrifuge testing might be underestimated in this case. In general, the isolation effect is achieved for both types of rocking when the peak rocking angle (maximum value of a doubly integrated signal obtained from the difference of the vertical accelerometers with high pass filtering) is above approximately 0.4%, which in prototype scale corresponds to 20 mm of uplift (Figure 8).

### 5 EFFECT OF ROCKING TYPE ON FORCE DEMAND

To examine the effect of the impacts generated at the interface of rocking (either above or below the foundation level), the force demand, the storey lateral raw accelerations, the vertical raw accelerations at the column ends, and the

![Figure 7](image_url) Base shear difference with the SRSS linear elastic solution
excitation below each building are closely examined in Figure 9A to H; the selected dataset is part of the low-frequency (30 Hz) excitation in dense sand (Test-2 EQ-4). The force demand is shown in total external force (Figure 9A, E) as calculated from the raw lateral storey accelerations (Figure 9B, F). In addition, a portion of the base shear force obtained by the strain gauges at the bottom storey is also plotted to examine any profile discrepancies between different instruments (i.e., MEMS accelerometers and strain gauges, Figure 9A). Because only the front side of the building models was strain gauged, the base shear of the braces was close to 50% of the actual value of the total base shear. The base shear sign convention is such that a deformed shape towards the right of the models would produce a positive shear force.

Impact can be recognized at the time when a vertical acceleration rises sharply, while the counterpart accelerometer shows an acceleration increase shortly after (Figure 9C). An example impact has been marked in the plots. Furthermore, the local maxima of the total external force have been marked in Figure 9A, E.

Regarding model RA, when the impact occurs, the external force has a zero crossing, indicating that the structure passes through the initial zero deflection state (Figure 9A). The rising part of the inertial force develops after its zero crossing with a peak value corresponding to a local maximum of the vertical acceleration immediately after the impact point (Figure 9C). Therefore, the local maxima of the force demand are related with the rocking motion as indicated with the smooth vertical accelerations, following the impacts generated at the contact points. This essentially means that, for rocking on dry dense sand, impact excites the superstructure and increases the total external force demand. This confirms previous experimental and analytical research for flexible structures rocking on a rigid base.10,11,42

Finally, note that the horizontal accelerations of the foundations of model RA followed very closely the excitation below its footings (Figure 9D), meaning that no sliding occurred.

Comparing the total external force time histories for models RA and RB, the latter experienced a smaller maximum total external force (Figure 9E). During the first half of the response, small impacts occurred at the ends of the columns, while during the second half, impact accelerations of larger amplitude developed (Figure 9G). This response suggests that following the very small duration of fixed base response initially, a weak form of foundation rocking took place, with the footings not losing much contact with the soil. Subsequently, the impacts became larger and caused more distinguishable higher mode oscillations. Note that the RB model has a lower centre of gravity because of its attached footings, so the static force required to cause uplift depends on the specific point of rotation for RB (see Table 1). From a static point of view, a high force demand for RB would mean rotation about the centre of the footing, which results in a slenderness ratio of 0.34 (264 N). However, considering the force demand caused by impact at recentring, the actual effective slenderness falls between 0.34 and 0.25 (264 to 199 N), indicating that the rotation develops about a point within the inner half of the footing. The extent of soil yielding cannot be determined from these results alone, but the results do indicate that some soil yielding appears to be likely.

Finally, comparison of the footing lateral accelerations (Figure 9D, H) provides clear evidence of uplift of the RB footings, and potential hammering during uplift. While model RA showed a lateral acceleration footing profile that
matches the soil exactly, the RB lateral footing accelerations are very different from the soil. More specifically, large lateral oscillations occurred at the left footing of RB when that footing has uplifted, i.e., after impact and subsequent rotation about the right footing.

6 | EFFECT OF SAND DENSITY ON FORCE DEMAND

The response from the low-frequency (30 Hz) excitation on the loose sand is shown in Figure 10. The response of model RA is generally very similar to the dense sand case. In contrast, model RB behaved differently in loose sand compared to dense sand. Similar to the first half of the response in dense sand (Figure 9E-H), small, sharp impacts indicate that rocking of the footings on the soil took place immediately after the initial full-contact response. However, unlike the dense sand case, rocking was suppressed significantly after a few cycles, changing the lateral acceleration profiles (Figure 10F) and reducing the total force demand (Figure 10E). The suppression of rocking is also confirmed by the lateral acceleration of the RB footing, which matches the response of the soil below nearly exactly after uplift ceases at about 6.25 seconds (Figure 10H). The above changes suggest a transition from a rocking response to a nearly full contact response with an appreciable higher mode contribution (as indicated by Figure 10F). This suggests that more soil yielding occurred for loose sand; the extent of soil yielding or rounding of the soil surface beneath the footings could not be measured directly. Visual inspection after the test showed no major differences between start and end of

FIGURE 9  Dense sand, Test-2 EQ-4: force demand of model RA from accelerometers and bottom braces (A), RA storey lateral response (B), RA column vertical response (C), soil and RA footing lateral response (D), and similarly for model RB (E-H)
the centrifuge flight. In contrast, for dense sand, the gradual increase over time of the vertical accelerations can be attributed to a densifying soil which becomes gradually stiffer. Overall, for the RB model, the soil density governed the transition from rocking with small impacts to either a full contact response with associated soil yielding (loose sand) or to rocking with progressively larger impacts (dense sand).

6.1 Evaluation of frequency content

Because the sand density can govern the type of response for foundation rocking, while having very little effect on the building performance for structural rocking, visualisation of the change in frequency content with time can provide further insights on the dynamics of the response. Wavelet transforms using the Morse wavelet with $\gamma = 3$ and $\beta = 27$ were applied for the storey lateral accelerations and the excitation as recorded at the base of the centrifuge box. The selection of the specific wavelet transform was investigated previously.

Only the time-frequency maps of the top storey accelerations are discussed and presented (Figure 11) for brevity. The time-frequency maps of the input (Figure 11A, B, G, H) are very similar in both the time and frequency domain. Higher harmonics of the dominant low-frequency input are also observed. The response of model RA was governed by the low-frequency component of the excitation, indicating that a quasi-steady-state rocking response occurred at the excitation frequency. In the higher frequency range, prominent peaks in the wavelet transform are evident during some rocking cycles. The frequency of the uplifted vibration response matches the high-frequency content of the excitation. However, theoretically, the higher mode response should be largely uncoupled from horizontal excitation input. It was not possible to confirm that, because some even higher frequency content might not have been captured because
of the 400 Hz built-in filter of the MEMS accelerometers used here. Thus, this large high-frequency response during uplift is likely caused by a combination of lateral excitation at approximately 150 Hz, combined with excitation of the uplifted vibration modes caused by impact. Overall, only minor differences can be observed between dense (Figure 11C, D) and loose sand (Figure 11I, J).
On the other hand, the quantitative change in the response profile of model RB for different sand densities (Figure 11E, K) is also reflected in the frequency content (Figure 11F, L). In dense sand, model RB exhibited foundation rocking with local high-frequency excitation during some rocking cycles, similar to the model RA response. On the contrary, in loose sand, initial foundation rocking with very weak high-frequency response was converted to a full contact response which enabled significant high-frequency excitation caused by the input excitation. The end of the main excitation then caused abrupt transition to a clear free vibration response of the first mode.

6.2 | Evaluation of moment-rotation response

The effect of sand density on the building response can also be visualised by comparing the restraining moment-rocking angle response of the 2 building models. Here, the restraining moment for model RA is the moment acting on the top surface of its footings, while for model RB, the restraining moment is presented at the soil surface. Figures 12 and 13 present the moment-rotation response for the Kobe excitation and the 30 Hz excitation respectively. Both figures exhibit a typical moment rotation response for a rocking structure, where the maximum moment is capped by the static moment that causes uplift, indicated by the solid horizontal line in the figures. However, for model RA, the static overturning moment was more significantly exceeded; this is again the result of the higher frequency excitation superposed on the rocking response. For model RB, the static moment is shown with respect to a point of rotation about the middle of the bottom surface of the footings. The restraining moment of model RB was generally within these limits, which again suggests that the effective rotation point may be slightly inward from the centre of the footing.

First, the Kobe excitation, which has a large but short duration low-frequency content, a single main cycle of rocking is induced for both RA tests and for the RB test on dense sand (Figure 12). Meanwhile, for the RB test on loose sand, a smoother rotation response is observed with an increased number of large rotation responses. This curve is representative of rotations because of soil deformations, but some uplift of the rocking foundations also occurred. Meanwhile, the 30 Hz excitation exhibits numerous cycles of large rocking amplitude. As in the previous section, for model RA, no effect of sand density is evident. Moreover, for dense sand, the cyclic response of model RB was similar to that of RA, though of smaller rocking amplitude. On the contrary, in loose sand, the cyclic response of model RB was smoother and of considerably smaller amplitude.

Overall, for small rocking amplitude, the models perform the same. In general, this means that it is impractical to distinguish actual uplift of the superstructure (model RA) or a local dynamic settlement of the footings (model RB), which again would result in an apparent rocking motion, as far as building response is concerned. For large rotations, uplift of the superstructure (including the footings for model RB) was the typical mechanism, with expected static limits being exceeded because of higher frequency excitation caused by direct excitation from the harmonics of the input, in combination with impacts and hammering action of the footings on the soil.

**FIGURE 12** Restraining moment versus rocking angle for dense (left) and loose (right) sand for the Kobe excitation. Horizontal lines indicate static overturning moment.
6.3 Overall effect of sand density

The effect of sand density on the response is further evaluated by plotting the peak force demand against the peak rocking angle for each input excitation (Figure 14). The peak values do not necessarily occur at the same instance in the time history; however, they can provide general insight into the trends forming with respect to the relative density of the sand. Fitting with piecewise lines for full contact and rocking conditions indicates that for an excitation in dense sand, a large rocking angle is associated with an increasing total force demand for both RA and RB. The reason for this correlation could be that larger rocking angles cause larger impacts and therefore larger high frequency response, or that larger rocking angles allow increased higher mode excitation by the high-frequency ground motion input. The prior hypothesis seems more plausible. Next, the slope difference in loose sand for model RA indicates that a very large peak rocking angle leads to a smaller increase of force demand for loose sand. This suggests that either the impact again causes less excitation of the high frequency uplifted vibration mode, or that the loose soil filters the high-frequency excitation input more than the dense soil. Finally, for model RB, loose sand did not allow large rotations to occur; thus, a cluster of points is created with the same force demand. This indicates that soil deformations caused energy dissipation that both decreased the rotation and high-frequency response.
Next, the local effect of sand density on the impact excitation at the rocking interface is considered more directly by extracting the acceleration as associated with an impact spike and relating that to the maximum force immediately following. For example, an impact is identified in the vertical acceleration at the base of the columns (Figure 15A, slightly before $t = 4.85$ seconds) and associated with the following local peak in the total external force time history (Figure 15B, black circle slightly after $t = 4.85$ seconds). Figure 16 plots the extracted external force versus the impact acceleration for impacting at either left or right side, for both buildings and across both types of the relative density of sand and all earthquake excitations.

**FIGURE 15** Loose sand, Test-1 EQ-5: vertical acceleration response of model RA (A) and force time history for model RA (B). Black circles are pairs of impact acceleration-force demand.

**FIGURE 16** Effect of the impact on the peak total external force, for dense (left) and loose (right) sand. Dashed lines indicate the slenderness values according to Table 2.
For dense sand, the behaviour of model RA appears again similar to that of RB. More specifically, large scattering appears for both models; however, this is even larger for RB. In general, different earthquake excitations create different clusters of points in the graph. This means that on the one hand the impact-peak force response is dependent on the excitation (frequency content, amplitude, phase difference) and at the same time can randomly vary locally. For instance, it is important to note that for all 50 Hz excitations of model RA, an asymmetric steady-state rocking response was observed, with large impact accelerations followed by small local maxima of force demand (Figure 15). This steady-state response requires further investigation, but clearly results in a system where the global rocking response governs the force demand rather than the excitation by impact, thus causing a different trend than what is observed for other excitations. Regarding model RB, the randomness appears to be more extended for the individual data clusters. This is again attributed to the difference between having a known and discrete 2-point rotation system (RA) with little uncertainty as to where the recentering occurs, as opposed to a finite area interface of rocking and increased soil deformations.

For loose sand, the behaviour of model RA appears to have a similar degree of scattering with the dense sand case. The slenderness limit of RA simply indicates the impact-induced force demand component as an addition to a static force demand for uplift. This again indicates that structural rocking is relatively unaffected by the change of relative density. On the contrary, the scattering of model RB appears reduced between dense and loose sand, and a force demand plateau forms for large impacts close to the minimum slenderness limit (0.25).

This general and local behaviour of RB is explained by recalling that its footings impose additional stress because of the building's rotation while rocking, which in conditions of loose sand can lead quickly to yielding and a stiffness reduction below the footing. The difference of preyielding soil stiffness below the footing because of change of sand density might not mean significant reduction of the impact amplitude, as this is similar for RB across both dense and loose sand, but it may prevent the development of a large impact-induced structural deformation (and hence a large force demand) by allowing a soil settlement instead. If an impact induced force demand is considered, then the actual slenderness value is smaller than 0.25, suggesting that the point of rotation in loose sand might be even further than the internal edge of the footing. In addition, when soil settlement is activated, the response does not involve large values of the peak rocking angle anymore for RB. For RA, soil yielding is essentially eliminated. Further increase of rocking rotations would not increase the soil pressure further, so would not increase soil yielding. However, increased rotations could increase impact forces, which could cause some soil yielding at impact for very large rotations.

7 | CONCLUSIONS

This paper compares the seismic behaviour of 2 building models resting on dry sand and allowed to uplift and subsequently indulge in 2 different types of rocking action. Structural rocking, defined as rocking where a building uplifts and rocks above its foundation level, was represented by a building model with no connections to its footings. On the other hand, foundation rocking, where a building is allowed to rock below its foundation level, was represented by a dynamically similar building model which had fixed column-footing connections. Sequential earthquake excitations were run with the 2 building models tested side by side in centrifuge conditions, with both low and high relative densities of dry sand considered. Evaluation of the seismic response of the 2 building models led to the following conclusions:

- The base isolation effect, an inherent benefit of rocking, was very significant for excitations with frequency content close to the fundamental natural frequency of the structures. For low-frequency excitations, there was no clear benefit of rocking compared to the fixed base linear response; both exhibited similar force demands. This was attributed to large rotations with large impacts causing additional force demand for both structural and foundation rocking. For low magnitude excitations, no significant uplift occurred.
- The weight-normalised magnitude of the maximum total base shear force developed because of the ground excitation and the intrinsic rocking mechanisms was consistently, though not extensively, larger in structural rocking for either type of sand. This finding was attributed to 2 effects. First, the foundation rocking model was effectively more slender because the effective rotation point moved away from the footing edge because of soil deformation, which decreased the static lateral force demand. Second, the structural rocking model experienced larger higher frequency vibration response, which was likely caused by increased impact excitation because of the 2-point rocking mechanism as opposed to the partial contact mechanism in foundation rocking.
For structural rocking, the sand density did not have a significant effect on the response. Limited evidence suggests only that the sand density might have a minor influence on higher frequency vibrations caused by impact. On the contrary, foundation rocking is inherently dependent on the soil conditions. This was more profound during the low-frequency excitations where loose sand ceased rocking and led to full contact response with evidence of significant soil deformations.

In general, these results demonstrate that both structural rocking and foundation rocking provide effective base isolation, and highlight some trade-offs between these systems. However, the critical effect of foundation settlements caused by foundation rocking is not addressed here because it could not be measured directly in the tests. By visual inspection, similar total and differential settlements were observed for both buildings, and these were relatively small. The wider implications of this comparison are associated with the potential uncertainty of the soil properties often encountered in practice, and the relative importance of residual settlements. Structural rocking presents a similar behaviour across different densities of sand with potentially smaller residual settlements, reducing uncertainty related to soil deformations. Foundation rocking on loose sand reduces the effects of rocking impact, while increasing soil yielding (energy dissipation) and moderately decreasing force demand.

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