16th World Conference on Earthquake, 16WCEE 2017

Santiago Chile, January 9th to 13th 2017

Paper N° 1695

Registration Code: S-N1461990405

EFFECT OF SOIL-FOUNDATION-STRUCTURE INTERACTION ON SEISMIC STRUCTURAL RESPONSE USING DYNAMIC CENTRIFUGE TEST

M. Lkhagvasuren[(1)], J.H.Ha[(2)], H.J.Park[(3)] , D.S.Kim[(4)]

[(1)] *Student, Department of Civil and Environmental Engineering, KAIST, eternity_l@kaist.ac.kr*
[(2)] *Student, Department of Civil and Environmental Engineering, KAIST, jgha87@kaist.ac.kr*
[(3)] *Research Professor, Department of Civil and Environmental Engineering, KAIST, heonjoon@kaist.ac.kr*
[(4)] *Professor, Department of Civil and Environmental Engineering, KAIST, dskim@kaist.ac.kr*

Abstract

Earthquake is a life-threatening and destroying natural hazard causing loss of lives and properties, leading to significant socio-economic crisis. In recent decades, an extensive research was done for the investigation and prevention of earthquake hazards, leading to the conclusion that careful consideration of soil-foundation-structure interaction is a crucial part in seismic assessment of structural systems. However, in current practice the seismic load on structure is determined from the ground motion assuming fixed base boundary conditions. In particular, majority of the previous studies were focused on the investigation of specific responses of superstructure or foundation only. Consequently, an experimental investigation assessing the behavior of foundation and superstructure and soil-foundation-structure interaction on it is still needed for better understanding of the phenomena behind it. Therefore, an experimental program was carried out to evaluate the effect of soil-foundation-structure interaction, namely the influence of foundation rocking, on seismic response of structures. The test set-up consisted of single degree of freedom structure model, shallow foundation and medium dense subsoil deposit in centrifuge container. Main test parameters were the dynamic period of the structure model and mass of foundation; in addition, the type and level of input earthquake accelerations served as secondary parameters in order to fully assess the seismic response of the system. Accelerometers, LVDT and earth pressure sensors were installed in soil, foundation and structure for capturing the seismic responses, settlements, moment-rotations and distribution of earth pressures beneath the foundation, respectively.

The superstructure response obtained from centrifuge testing were compared with fixed base case and the difference between these responses can be regarded as soil-foundation-structure interaction effect. Furthermore, the rocking of foundation is an important phenomena, dissipating energy coming from the earthquake. Accordingly, seismic response of structures sitting on lighter and heavier foundations were compared when subjected to different types of earthquakes with varying intensities. As observed from the experiment, in lower intensity earthquakes rocking is not largely influenced by its weight, however in higher intensity ones rocking is more significant in lighter foundations due to difference in ultimate moment capacity. Through LVDT, dynamic and permanent settlement of foundations were measured. Representative results are presented in this paper. To sum up, this study provides an experimental evidence on influence of rocking of shallow foundations on seismic response of structures and reduction of ductility demand of structures.

Keywords: soil-foundation-structure interaction, rocking of shallow foundation, centrifuge test

1. Introduction

The effect of soil-foundation-structure interaction on overall seismic performance of structures is well-known and several attempts to quantify its effects into the seismic design codes were done using various means of experimental, analytical and numerical tools (Deng and Kutter, 2012; Gajan et al., 2005; Gazetas et al., 2004; Mergos and Kawashima, 2005). Moreover, nonlinear soil-foundation-structure interaction has two primary effects on structural response, including the increased degree of freedom due to compliance of foundation and elongation of natural period of the structure, which can be used as a prevention mechanism against structural collapse and energy dissipation reducing the ductility demand on structure. On the other hand, if accounted carelessly, nonlinear soil-foundation-structure interaction can cause undesirable permanent settlement and rotation, which can deteriorate structure serviceability and durability. Consequently, full assessment of nonlinear soil-foundation-structure interaction (SFSI) is needed for better understanding of its effects on seismic response of structures.

This experimental study was performed to evaluate the nonlinear soil-foundation-structure interaction on seismic behavior of structure with variation of moment capacity of foundation, period of structure, type and intensity of input earthquake. Moreover, the soil-foundation-structure system was subjected to three types of earthquakes, including Ofunato, Hachinohe and Northridge; additionally to assess the effect of frequency, degradation of rocking stiffness and damping ratio, sinusoidal waves of 2Hz and 4Hz were applied.

This study gives an overall insight into the effect of nonlinear soil-structure interaction on seismic response of structures by using dynamic centrifuge facility as a tool.

2. Experimental Program

2.1 KAIST Beam centrifuge

The primary use of geotechnical centrifuge testing facility is physical modeling in which an event or behavior comparable to what might exist in prototype is replicated using small scale models (Schofield, 1980). Moreover, centrifuge modeling with shaking table equipment enables researchers to simulate real earthquake phenomena in small scale by providing an opportunity to observe and quantify effect of soil-foundation-structure interaction on seismic behavior of structures. Related scaling laws used in this study are provided in Table 1.

To make a good use of this facility, an experimental program with structure models and foundation models according to the scaling law was carried out. The test was done in beam-type centrifuge with 5m radius in Korea Advanced Institute of Science and Technology (Fig. 1). This facility has a maximum capacity of 240g-tons (Kim et al., 2012). Earthquake motion is generated by an in-flight earthquake simulator installed at the bottom of a soil container. The acceleration level of testing was 20g of centrifugal acceleration.

The soil container used in this study was the equivalent shear beam (ESB) box, which can simulate the semi-infinite field soil condition by reducing the reflection of waves at the boundary of container. The ESB soil container is installed on a horizontal shaking table attached to the centrifuge equipment.

Fig. 1 – Centrifuge testing facility of KAIST

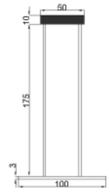
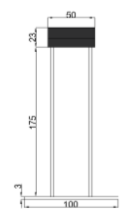
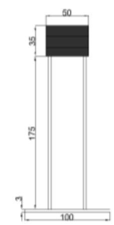
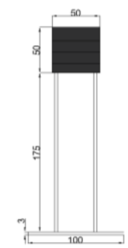
Table 1 – Scaling law for centrifuge

Parameters	Model/Prototype
Stress	1
Strain	1
Length	1/N
Acceleration	N
Mass	$1/N^3$
Time	1/N

2.2 Test set-up

The experimental specimen of this test consists of single degree of freedom structure model, shallow square mat foundation and a subsoil deposit of medium dense silica sand in centrifuge soil container. Structure models consisted of two thin steel columns with lumped square mass on top of it. Natural periods of structure models were varied by changing the weight and size of the lumped mass and were measured using impact hammer test with fixed base condition. Detailed procedures of determining related parameters for structure models are given by Kim et al. (2015). Respective properties of structure models are depicted in Table 2.

Table 2 – Structure model variation and properties

Structure models	SDOF1	SDOF2	SDOF3	SDOF4
Dimensions (mm)				
Lumped mass (kg)	0.11775	0.270825	0.412125	0.58875
Mass of plates(kg)	0.236	0.236	0.236	0.236
Effective mass, ms (kg)	0.278	0.521	0.741	1.036
Effective lateral stiffness, ks (kN/m)	60.5	60.5	60.5	60.5
Natural period (sec)	0.013	0.018	0.022	0.026
Natural period of prototype structure (sec)	0.269	0.369	0.44	0.520

Two types of shallow square mat foundations with different moment capacity were used in this experimental study, one made of aluminum and the other one made of steel, as shown in Table 3. Dry silica sand with 60% of relative density were made for this experimental study using sand raining system by adjusting height, speed and diameter of the dropping layer by layer until the thickness of the sand layer reached 580 mm. Properties of silica sand are shown in Table 4.

Several real earthquake motions including Ofunato, Hachinohe and Northridge earthquakes and sinusoidal waves with forcing frequency of 2Hz and 4Hz were applied to the base of ESB box. The intensity of the earthquake accelerations were gradually increased from low to high.

Table 3 – Foundation models

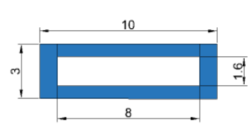
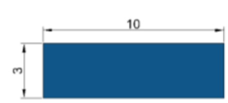
Foundation models	FND1	FND2
Material	Aluminum	Steel
Dimensions (cm)		
Mass (kg)	0.545	2.355
Density (kg/m^3)	5046	7850

Table 4 – Soil properties

Properties	**Silica sand**
Soil model properties	
Soil thickness (mm)	580
Dry density (t/m^3)	1.48
Prototype soil properties	
Centrifugal acceleration (g)	20
Soil thickness (m)	11.6
Site period (sec)	0.221

3. Discussion on test results

3.1 Fixed and flexible base motions

In geotechnical earthquake engineering, free field motion is defined as the ground motion, which is not influenced by the presence of structure or foundation. Moreover, when structure-foundation system sitting on solid rock is subjected to seismic excitations, rock motion does not deviate significantly from free-field motion due to its high stiffness. Therefore, structure foundation system can be considered to have fixed base boundary conditions. On the other hand, when the same structure foundation system is founded on compliant medium such as soil, the inability of the system to follow the free field motion will cause motion of the base of the structure to deviate from free-field motion. This process of mutual influence of soil-foundation-structure interaction is known to influence the seismic structural response. However, in most current practices, seismic load on structure is determined by the response spectrum obtained from free-field motion considering fixed base boundary conditions.

The schematic diagram showing the difference between fixed and flexible base motions are depicted in Fig. 2. In this test, the horizontal acceleration $\ddot{u}_t$ of the structure model and the horizontal acceleration $\ddot{u}_f$ of the foundation were measured accordingly. The relative acceleration of the structure was calculated as difference between total and foundation accelerations $\ddot{u}_r = \ddot{u}_t - \ddot{u}_f$, which indicated the earthquake response of the structure model affected by SFSI. For the comparison, the accelerations $\ddot{u}_{fixed}$ of the fixed base structures were calculated using the measured free-field ground motion as an input ground motion. The difference between $\ddot{u}_t$ and $\ddot{u}_{fixed}$ can be regarded as the SFSI effect.

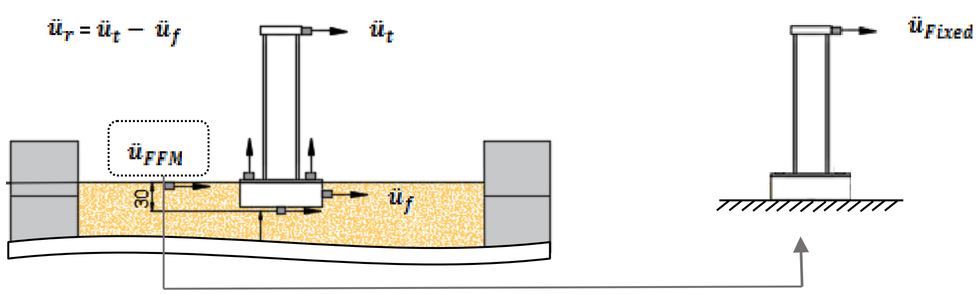


Fig. 2 – Schematic diagram for: (a) flexible and (b) fixed base motions and definitions of measured horizontal accelerations

Fig. 3 shows the representative results comparing fixed and flexible base motions for the case of SDOF2 when subjected to Ofunato earthquake of small and large intensity. According to Fig. 3 (a) and (b), it is evident that for small intensity earthquake there is no significant difference between peak accelerations of fixed and flexible base responses. However, in large intensity earthquakes, difference between acceleration responses becomes significant providing an evidence for SFSI dissipating energy coming from the earthquake and reducing the ductility demand on structures (Fig. 3 (c) – (d)).

In addition moment capacity plays an important role in determining energy dissipation through rocking of shallow foundations. For the case of FND1 with lower moment capacity, it is easier to trigger rocking motion and uplift compared with that of FND2, as it can be seen from Fig. 3, where peak acceleration is lower in case of FND1, meaning that it dissipates more energy compared with FND2.

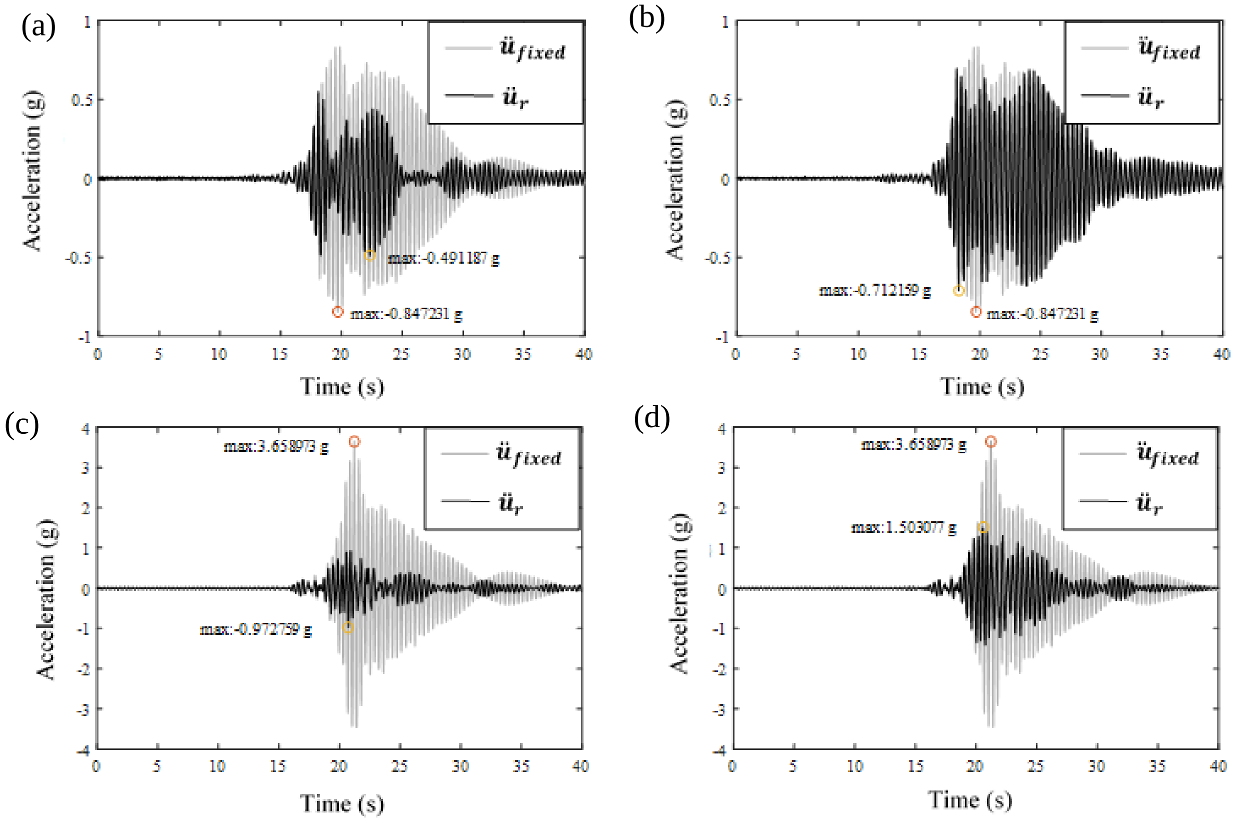


Fig. 3 – Acceleration record comparison between fixed and flexible base cases of: (a) FND1-SDOF2 small intensity (b) FND2-SDOF2 small intensity (c) FND1-SDOF2 high intensity (d) FND2-SDOF2 high intensity

3.2 Comparison between pseudo-accelerations considering SFSI effect and fixed base structure response

When soil-foundation-structure system is subjected to seismic shaking, soil and foundation experiences large inertial forces leading to rocking oscillations causing detachment of soil-foundation interface at one edge and increase of normal stresses at the other edge, which causes plastic soil yielding. Nonlinearity in soil occurs when eccentricity of axial loads is greater than L/6, where L is the length of foundation; minimum contact are required to support the axial load is known as critical contact area. According to FEMA 356 (2000), the ultimate moment capacity of rectangular foundations can be calculated using the following equation:

$$M_{ult} = \frac{V \cdot L}{2} \cdot \left(1 - \frac{q}{q_c}\right)$$

where V – vertical load, L – length of foundation, $\frac{q}{q_c}$ - critical contact area ratio.

The overturning moment M_o can be calculated using the static force equilibrium and aforementioned equation and it cannot exceed ultimate moment capacity of foundation. The maximum pseudo-acceleration $S_{a,max}$ of the structure can be determined using the ultimate moment capacity:

$$m_s \cdot \ddot{u}_t \cdot h = S_a \cdot m_s \cdot h \ \leq M_{ult} = \frac{m_t \cdot g \cdot L}{2} \left(1 - \frac{q}{q_c}\right)$$

$$S_a \leq S_{a,max} = \frac{1}{2} \cdot \frac{m_t}{m_s} \cdot \frac{L}{h} \cdot \left(1 - \frac{q}{q_c}\right) \cdot g$$

where h – height of the mass of the structure, m_t - total mass of the whole system, m_s - mass of the structure, S_a - spectral acceleration at the structure, g – gravitational acceleration. From this calculation, we can find $S_{a,max}$, which is the limiting acceleration of the structure.

Aforementioned equations were used in order to estimate seismic loads on the structure model, the pseudo-accelerations $S_{a,Test}$ of the structure and $S_{a,Fixed}$ of the fixed base structures for three types of earthquakes including Ofunato, Hachinohe and Northridge.

In Fig. 4 (a)-(d), the pseudoaccelerations $S_{a,Test}$ and the pseudoaccelerations of $S_{a,Fixed}$ are compared for four different structures with varying natural periods and two different foundations with different moment capacities. For the case of SDOF1 and SDOF2, as the peak ground acceleration increases, pseudo-acceleration $S_{a,Test}$ starts to deviate from

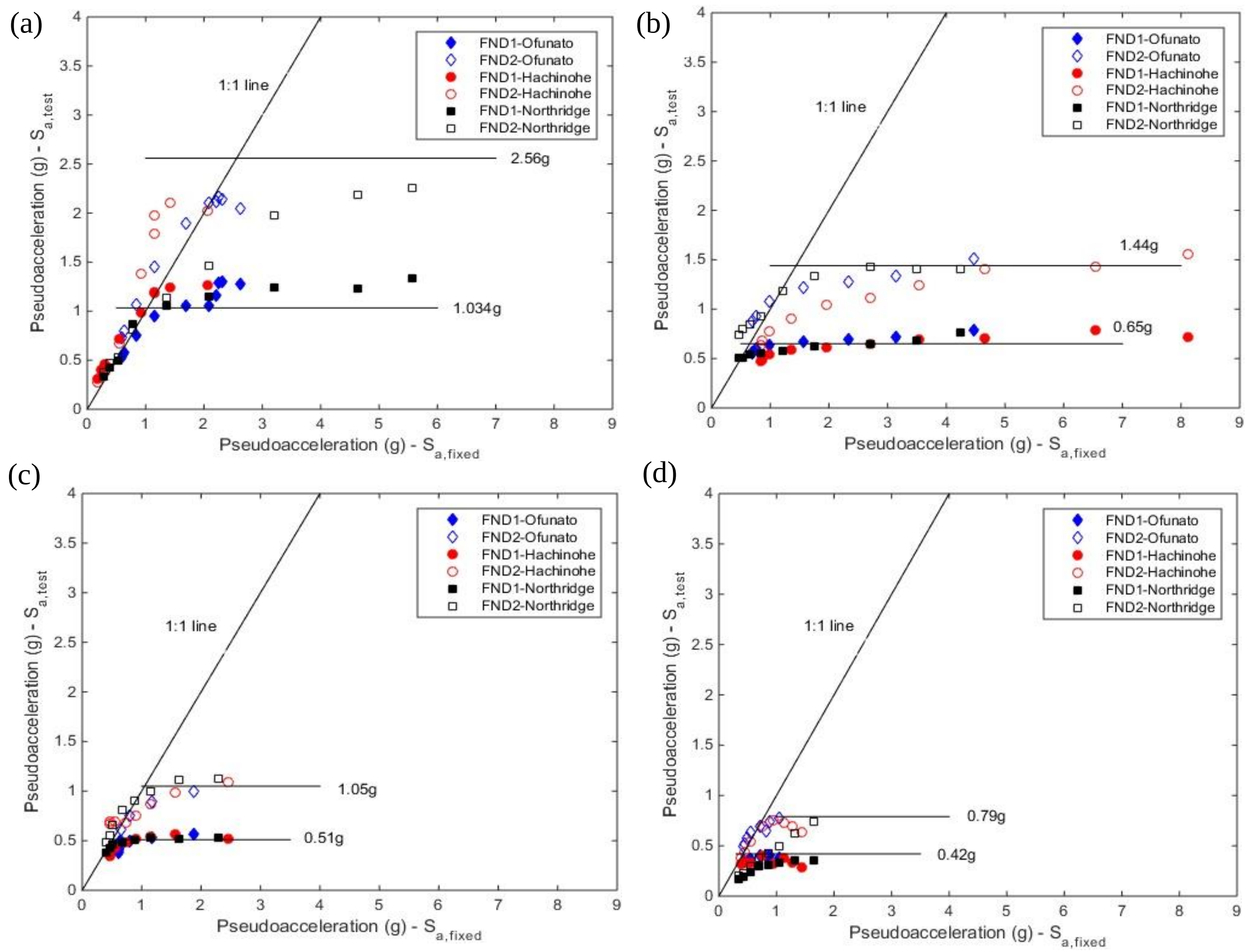


Figure 4 - Pseudoacceleration response of fixed and flexible base structures: (a) SDOF1 (T_n = 0.269 sec) (b) SDOF2 (T_n = 0.369 sec) (c) SDOF3 (T_n = 0.44 sec) (d) SDOF4 (T_n = 0.52 sec)

one-to-one line, meaning that fixed base pseudo-accelerations are significantly increasing, but pseudo-accelerations considering SFSI are converging to some certain limiting value. For the cases of SDOF3 and SDOF4, due to the difference between site period and structural natural periods, free-field motion is not amplified much, and there is no dramatic difference between pseudo-accelerations of fixed and flexible base cases.

According to Fig. 4, the fixed and flexible base pseudo-acceleration response can be divided into three regions, including region of low, medium and high intensity. In the region of low intensity, difference between fixed and flexible base peak accelerations are negligible as they fall on one-to-one line. Moreover, as the earthquake intensity increases, pseudo-accelerations for flexible base case starts to deviate from that of fixed base case. Eventually, in the region of high intensity earthquakes, there is significant deviation of flexible base pseudo-accelerations from that of fixed base case and the value converges to certain limiting value, depending on the moment capacity of the foundation.

In addition, for the case of FND1 and FND2, which have lighter and heavier foundation masses, in lower intensity region their responses are similar, however as the peak ground acceleration increases, pseudo-accelerations of FND1 starts to diverge from one-to-one line earlier compared with FND2, as it is easier to trigger vibration and rocking oscillation due to lower moment capacity.

3.3 Moment-rotation relations

In order to compare the moment and settlement of two shallow foundations with four different structures on top of it, overturning moment for each foundation was calculated the following equation:

$$\mathrm{M}_{\mathrm{overturning}} = m_s \times h_s \times a_g$$

where m_s - equivalent lumped mass of the structure, h_s - height from bottom of the foundation to the center of gravity of the lumped mass, a_g – acceleration experienced by lumped mass. Overturning moments were normalized with ultimate moments.

Representative results of moment-rotation and settlement-rotation relations when soil-foundation-structure system is subjected to Northridge earthquake of 0.53g are shown in Fig. 5 (a) – (d). According to Fig. 5, energy dissipation and uplift of foundation is the most significant for the case of SDOF1 for both FND1 and FND2 due to the closeness of structural and site periods making the earthquake motion amplified more compared with other structures. In addition, the shape of moment-rotation loop resembles an S-shape as the uplift and settlement increases for all cases of tests except FND2-SDOF3 and FND2-SDOF4. Generally, for SDOF3 and SDOF4, hysteresis loop area is relatively smaller than that of SDOF1 and SDOF2, implying that there is less energy dissipation and nonlinearity in the soil developed, soil behavior is almost in its elastic range.

Comparing FND1 and FND2 for all cases of structures, hysteresis loop area and amount of uplift are always larger for the case of FND1, showing that moment capacity plays an important role in triggering of rocking motion, consequently the dissipation of energy.

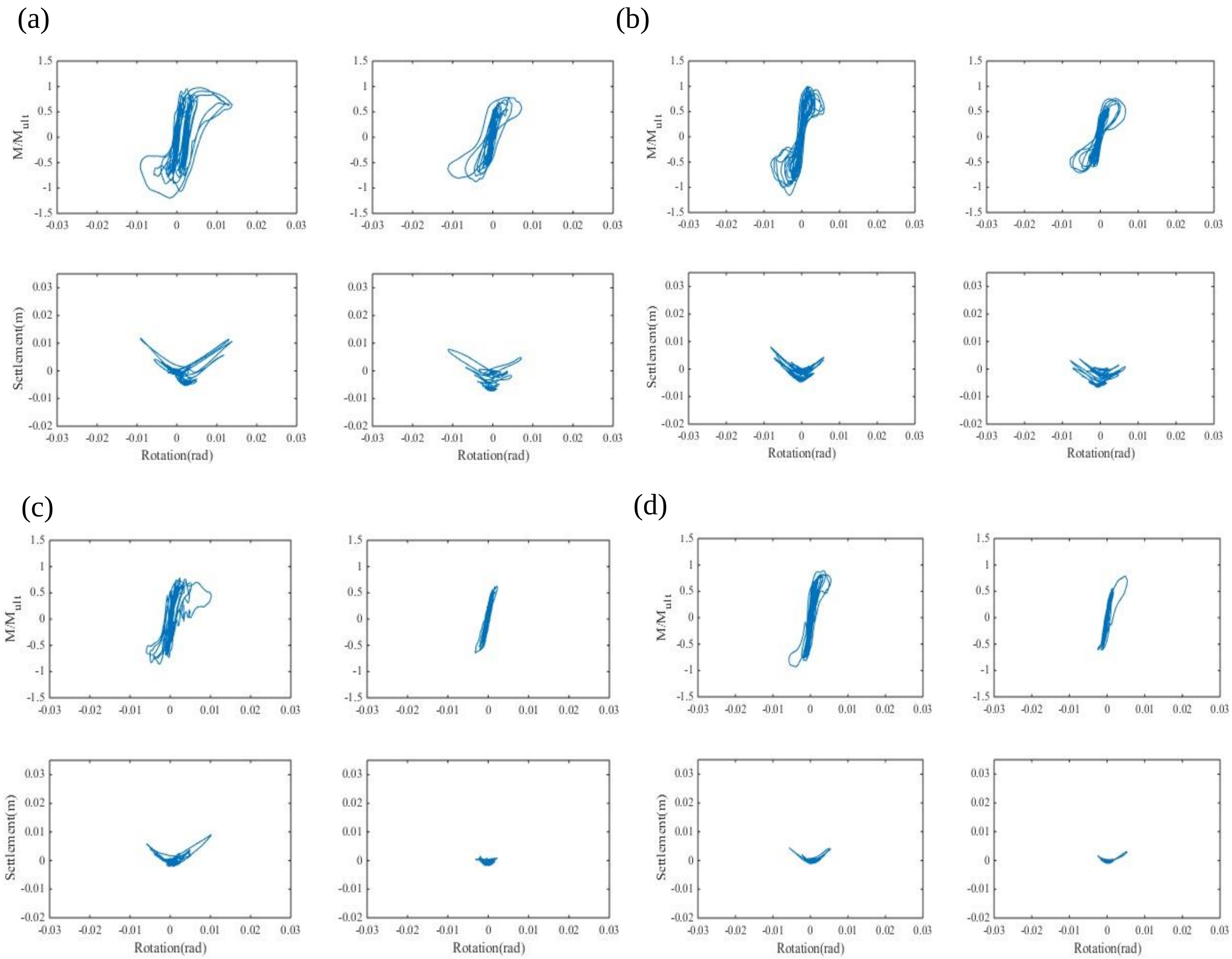
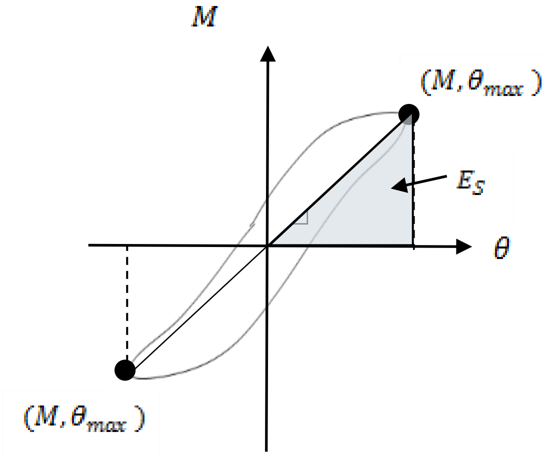


Fig. 5 – Moment-rotation and settlement rotation relations for (a) FND1-SDOF1 & FND2-SDOF1 (b) FND1-SDOF2 & FND2-SDOF2 (c) FND1-SDOF3 & FND2-SDOF3 (d) FND1-SDOF4 & FND2-SDOF4

3.4 Rocking stiffness degradation and damping of shallow foundations

According to the moment-rotation loops presented in the previous section, there is a great amount of energy dissipated through soil-foundation-structure interaction. Moreover, as the rocking oscillation occurs due to the large inertial loads developed in the structure, detachment and reattachments of foundation and soil takes place, which leads to the deformation of soil-foundation interface. Consequently, rocking stiffness of shallow foundation degrades with rotation angle increase.

Therefore, to evaluate rotational stiffness variation and damping ratio, soil-foundation-structure systems were subjected to Sine2Hz and Sine4Hz waves, respectively, in order to get regular and symmetric moment-rotation loops. The method shown in Fig. 6 was employed for the calculation of rocking stiffness and damping ratio. Firstly, moment-rotation curves were divided into separate single loops and for each loop maximum rotation angles with corresponding moments were found. Damping ratio and secant stiffness for each loop were calculated and plotted.



$$K_R = \frac{M}{\theta_{max}}$$

$$\xi = \frac{E_{loop}}{4\pi E_s}$$

Fig.6 – Definition of equivalent linear stiffness and damping ratio obtained from cyclic moment-rotation loops

Fig. 7 shows representative results of rocking stiffness and damping for SDOF3 with FND1 and FND2. As the cyclic loading intensity increases and uplift of foundation and plastic deformation of soil takes place, the foundation-soil interface area progressively decreases as the rounding of soil surface takes place. Rotational or rocking stiffness is related to these phenomena, making it gradually decrease with the increase in rotation angle. Similar trend can be observed for all four structures when sitting on two different foundations. In addition, rocking stiffness in FND2 is higher with that of FND1 as its moment capacity is higher; consequently making deformation in the soil-foundation interface less.

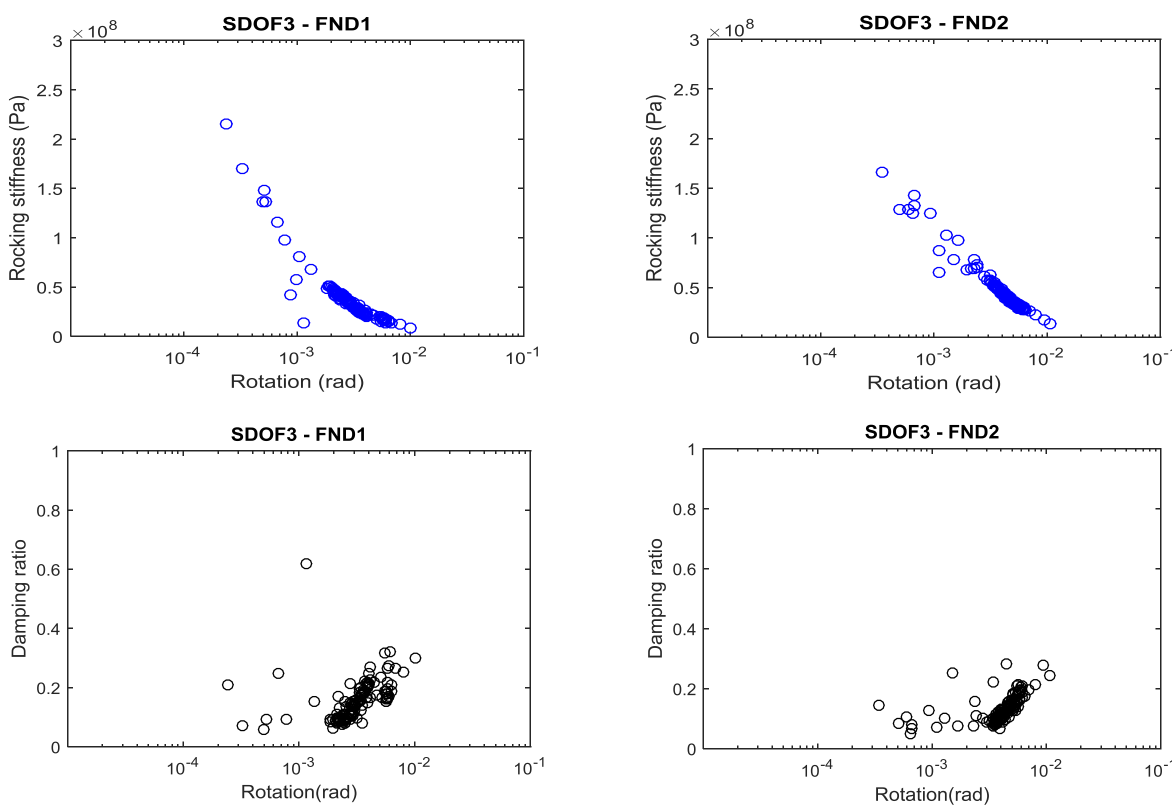


Fig.7 – Rocking stiffness and damping variation for SDOF3

4 Concluding remarks

Dynamic centrifuge testing facility is a powerful tool for the investigation of SFSI effects on seismic response of structures, as it can simulate real stress conditions in the soil. The main focus of this paper was to evaluate SFSI effect with foundation moment capacity difference with parameters such as natural period of structure, intensity and type of earthquake. From this experimental study, the following important conclusions were made:

(1) Comparison between fixed and flexible base conditions for four different structures has showed that SFSI contributes to the dissipation of energy leading to reduced ductility demand of structures especially in medium to high intensity earthquakes or seismic excitations.
(2) Estimation of pseudo-accelerations showed that as the earthquake intensity increases fixed base pseudo-accelerations increase, while flexible base pseudo-accelerations deviate from one-to-one line eventually converging to certain limiting value depending on the moment capacity of foundation.
(3) Through the analysis of moment-rotation relations and acceleration records, role of structural period on energy dissipation and amplification of motion has been evaluated. Moreover, as the structural period and site period gets closer earthquake motion tends to be amplified with more rocking oscillations and energy dissipation, consequently.
(4) Nonlinear rocking stiffness and damping of shallow foundations is of great interest characterizing nonlinear behavior of the overall system. There were observed rocking stiffness degradation and damping associated with it.

5. References

[1] Deng L, Kutter B L. Characterization of rocking shallow foundations using centrifuge model tests. Journal of Earthquake engineering and structural dynamics; 2012; 41:1043-1060.

[2] Gajan S, Kutter B, Phalen J D, Hutchinson T C, Martin G R. Journal of Soil Dynamics and Earthquake Engineering; 2005; 25: 773-783.

[3] Gazetas G, Apostolou M. Nonlinear Soil-Structure Interaction: Foundation Uplifting and Soil Yielding. Proceedings Third UJNR Workshop on Soil-Structure Interaction; 2004.

[4] Mergos P, Kawashima K. Rocking isolation of a typical bridge pier on spread foundation. Journal of Earthquake Engineering; 2005; 9:395.

[5] Schofield AN (1980): Cambridge geotechnical centrifuge operations, Geotechnique, 25:229-267.

[6] Kim DS, Kim NR, Choo YW, Cho GC (2012): A newly developed state-of-the-art geotechnical centrifuge in Korea. KSCE Journal of Civil Engineering, 17(1): 77-84.

[7] Kim D K, Lee S H, Kim D S, Choo Y W, Park H G. Rocking effect of a mat foundation on the earthquake response of structures. Journal of Geotechnical and Geoenvironmental Engineering; 2015; 141: 04014085.

[8] FEMA. (2000): Prestandard and commentary for the seismic rehabilitation of buildings. FEMA 356, Washington, DC.