

Effect of a Soil Buffer Layer on the Seismic Response of Rocking-Based Isolation Systems in Soft Soil

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Abstract: Rocking isolation systems founded on soft soils commonly exhibit settlement-dominated responses, leading to accumulated plastic strains and degradation of recentering capacity. Although ground improvement elements reduce total settlement, direct footing-improvement element induces localized stress concentrations that may hinder stable rocking behavior. This study investigates the influence of introducing an intermediate soil layer between the shallow foundation and the improvement elements on seismic response characteristics. A three-dimensional nonlinear finite difference model is developed to simulate a $3\text{ m} \times 3\text{ m}$ shallow footing supported by discrete pile groups and diaphragm wall systems. A granular sand layer, defined as a soil buffer, is inserted between the footing and the improvement elements. Buffer thicknesses of 0, 60 cm (B/5), and 100 cm (B/3) are evaluated through parametric analysis. The results indicate that increasing buffer thickness reduces peak rotational amplitude; in the pile-supported configuration, rotation decreases from 0.121 rad (no buffer) to 0.083 rad for a 100 cm buffer. While the presence of the buffer increases residual settlement relative to direct-contact systems, increasing it from 2.2 cm to 5.5 cm in JG-PR, settlement remains significantly lower than the unimproved condition (13.6 cm). Among the examined cases, a thickness of 60 cm (B/5) provides the most favorable balance between rotation control and recentering, with the diaphragm wall system (JG-PR-SB60) achieving a recentering ratio of 0.987. The findings demonstrate that the buffer layer modifies the contact stress transfer mechanism, promoting distributed shear deformation within the interface zone and reducing localized bearing concentration at improvement elements.

Keywords: Soil layer, Rocking isolation, Soft soil, Granular soil, Recentering ratio.

I. INTRODUCTION

Conventional seismic design philosophy generally enforces elastic behavior at the foundation level, intentionally shifting inelastic demand to the superstructure. In this

framework, plastic hinging is expected to develop at predetermined structural locations, most commonly at the base of columns, where damage is concentrated and repair may be costly. Although this strategy provides predictability in structural performance, it often results in significant structural damage even under moderate-to-strong seismic excitations. An alternative performance-based approach permits controlled nonlinear behavior within the soil–foundation system. Rather than constraining the foundation against uplift, rocking isolation intentionally allows footing rotation and partial detachment from the supporting soil. In this mechanism, energy dissipation occurs through cyclic soil yielding and geometric nonlinearity at the soil–foundation interface, thereby reducing ductility demand in the structural elements.

The fundamental principle of rocking isolation relies on mobilizing moment capacity through progressive contact area reduction during uplift. As the footing rocks, compressive stresses concentrate over a limited zone while the opposite edge detaches. This cyclic redistribution of stresses generates hysteretic damping within the supporting soil, which substantially can reduce transmitted accelerations and bending demands in the superstructure. Experimental and analytical investigations have demonstrated the effectiveness of this concept. Early centrifuge and numerical studies by Gajan & Kutter (2008) and Anastasopoulos et al. (2010) showed that rocking foundations can significantly decrease column curvature ductility demand while maintaining global stability. These findings established rocking as a viable foundation-level energy dissipation mechanism. Despite these advantages, the application of rocking systems on soft or poorly consolidated soils introduces critical challenges. Soft soils exhibit reduced bearing capacity and increased compressibility, which may lead to settlement-dominated responses rather than stable rocking. Under such conditions, excessive residual settlement and permanent rotation can accumulate, undermining post-earthquake serviceability.

Subsequent research has explored the correlation between rotation amplitude and permanent settlement. Deng et al. (2012) and Hakhmaneshi & Kutter (2016) observed that although rocking isolation protects structural components, repeated cyclic

loading may induce irreversible soil deformation, leading to permanent tilt. This trade-off between structural protection and foundation serviceability remains a central concern in practical implementation. To address these limitations, various ground improvement techniques have been proposed. Reinforcement systems such as geogrids, geocells, and soil–cement stabilization aim to enhance bearing capacity and stiffness, thereby limiting excessive settlement while preserving controlled rocking behavior. These methods improve the load–deformation characteristics of the supporting soil but may alter the intended nonlinear interaction mechanism.

Allmond & Kutter (2014) introduced the concept of unattached piles, in which piles provide vertical resistance without rigidly connecting to the foundation. This configuration mitigates tensile demand in piles and maintains rotational freedom, reducing the risk of structural damage within the foundation system. Building upon this concept, (Ko et al., 2019) identified the so-called “soil-rounding” effect, whereby inserting a deformable soil layer between the footing and piles allows the contact surface to evolve into a rounded stress distribution during rocking. This mechanism limits abrupt stress concentration at pile heads and modifies the ultimate moment capacity through progressive mobilization of bearing resistance.

High-resolution visualization techniques, Particle Image Velocimetry (PIV), have been used to study the subsurface deformation mechanisms, identify three distinct failure modes like soil densification, wedge deformation and scoop deformation (Moradi et al., 2023, 2024). It has been reported that increasing embedment depth suppresses the outward wedge mechanism responsible for settlement accumulation. The transition toward a scoop-dominated response yielded a 69% reduction in settlement while preserving substantial energy dissipation through extended sliding surfaces. The displacement fields plotted in these experiments confirm that the physical interaction at the soil–foundation interface governs the threshold between sinking-dominant and rocking-dominant behavior. Introducing unattached or semi-fixed interfaces offers a means of balancing dissipation against permanent deformation. Amiri et al. (2025) proposed a semi-fixed pile-to-cap connection that permits controlled vertical uplift without complete foundation separation; in long pile groups, this configuration reduced residual settlements by up to 30%. Parametric studies by Bin et al. (2024) on the OpenSees platform revealed that declining soil stiffness produces a sharp increase in bending moments at the foundation–column joint, dictating the importance of subgrade stiffening for managing energy distribution across the system.

Soil composition and the parameters of the seismic input also exert considerable influence on rocking isolation efficiency, and the interplay between these factors warrants closer examination than it has so far received (Irani et al., 2021, 2023; Nikvand & Bonab, 2024). It is reported that rocking foundations on non-plastic silts follow behavioral trends broadly consistent with sand-based results, yet silty soils display a markedly heightened sensitivity to input motion frequency. Sadjadi et al. (2023) investigating pulse-type excitations, showed that stiff rocking systems undergo stronger oscillations and greater residual settlement under near-fault motions, and identified Cumulative Absolute Velocity (CAV) as a more reliable intensity measure for predicting permanent deformations than conventional

metrics. Xiong et al. (2024) extended the problem to spatially inclined incident waves, finding that oblique wave incidence enlarges both torsional and rocking responses. Soil nonlinearity amplifies low-frequency seismic energy during such events—an effect that planar wave assumptions fail to capture.

Work on self-centering rocking piers by Rassoulpour et al. (2025) indicates that while self-centering mechanisms effectively reduce residual drifts, their capacity to control settlement depends strongly on the degree of soil–structure interaction and the frequency content of the ground motion. These investigations confirm that a yielding granular interface can transform the ground into an active element of structural protection, assuring structure’s post-earthquake serviceability.

Although these advancements highlight the importance of interface conditions, the mechanical role of an intermediate soil layer has not been systematically quantified for different improvement configurations. In particular, the influence of such a buffer on rotational amplitude, recentering capacity, and residual settlement remains insufficiently understood when discrete piles or diaphragm wall systems are employed (Ko et al., 2019). The interface layer may serve multiple functions: redistribution of contact stresses, modulation of shear strain localization, and alteration of the stiffness contrast between footing and improvement elements. However, excessive compliance could increase settlement, whereas insufficient compliance may reintroduce stress concentration. Determining an optimal balance therefore requires rigorous numerical evaluation (Irani et al., 2023). Despite the existing body of research, a clear gap remains in systematically quantifying how interface compliance influences the coupled response of rotation, settlement, and recentering across different improvement configurations (Irani et al., 2024). Previous studies have primarily focused on either ground improvement or rocking behavior in isolation, without explicitly addressing their combined interaction through a controllable intermediate layer. This limitation motivates the present study, which aims to bridge this gap by providing a unified numerical assessment of interface-modified rocking systems.

Accordingly, this study investigates the seismic response of a square rocking footing founded on soft soil and supported by jet grout improvement elements, including discrete pile groups and diaphragm walls. A three-dimensional nonlinear finite difference framework is employed to examine the effect of introducing a soft granular sand buffer layer of varying thickness between the footing and the improvement elements. The objective is to quantify its influence on rotation amplitude, residual settlement, and recentering performance, and to clarify the underlying stress transfer mechanisms governing the soil–structure interaction. The necessity of this research arises from the lack of a systematic evaluation of how interface compliance modifies the balance between energy dissipation and serviceability in rocking systems constructed on improved soft ground. While previous studies have demonstrated the benefits of rocking foundations and ground improvement independently, their combined interaction through an intermediate soil layer has not been rigorously characterized. The innovation of this study lies in explicitly modeling the buffer layer as a controllable design parameter and comparatively assessing its performance across distinct improvement configurations.

II. NUMERICAL MODELING PRINCIPLES

This section presents the fundamental numerical framework adopted to simulate the seismic response of the rocking foundation system. The governing equations, constitutive relationships, contact formulations, and numerical stability criteria are outlined to establish the theoretical basis of the modeling approach. Particular emphasis is placed on capturing nonlinear soil behavior and interface mechanics, which are critical to the accurate representation of rocking-induced response. The seismic response of the rocking foundation system is investigated using a three-dimensional nonlinear finite difference framework. The numerical formulation is implemented in FLAC3D V6.0 (Itasca, 2020), which solves the dynamic equilibrium equations in an explicit time-marching scheme. This approach is particularly suitable for problems involving large deformation, nonlinear material behavior, and complex soil–structure interaction (SSI), all of which are inherent to rocking systems subjected to seismic excitation. The governing dynamic equilibrium equation for a deformable continuum in incremental form can be expressed as:

$$\nabla \cdot \sigma + \rho b = \rho \ddot{u} \quad (1)$$

where σ is the Cauchy stress tensor, ρ is the mass density, b is the body force vector, and $\ddot{u}$ denotes the acceleration vector. The left-hand side represents internal and external forces, while the right-hand side accounts for inertial effects. In the explicit finite difference scheme, accelerations are obtained directly from force imbalance at each time step, and velocities and displacements are updated accordingly. To ensure numerical stability in the explicit formulation, the time step must satisfy the Courant–Friedrichs–Lewy (CFL) condition:

$$\Delta t \leq \frac{l_{\min}}{c} \quad (2)$$

$$c = \sqrt{\frac{K + \frac{4G}{3}}{\rho}} \quad (3)$$

where $l_{\min}$ is the smallest element dimension, c is the wave propagation velocity in the medium, K is the bulk modulus and G is the shear modulus. This criterion governs the maximum allowable time increment and directly influences computational cost and stability. The soil behavior is represented using the Mohr–Coulomb elastoplastic constitutive model. The shear yield function is defined as:

$$f_s = \tau - c - \sigma_n \tan \phi \quad (4)$$

where τ is the shear stress, c is cohesion, σ_n is the normal stress on the failure plane, and ϕ is the internal friction angle. Yielding occurs when $f_s = 0$. This model captures the primary features of granular soil response, including frictional resistance and shear-induced plastic flow. The elastic response prior to yielding follows Hooke's law for isotropic materials:

$$\sigma_{ij} = D_{ijkl} \cdot \epsilon_{kl} \quad (5)$$

where D_{ijkl} is the fourth-order elastic stiffness tensor, and ϵ_{kl} is the strain tensor. For isotropic conditions, the elastic constants

are defined by Young's modulus (E) and Poisson's ratio (ν), with the G given by $G = E/(2(1 + \nu))$. These relations govern the pre-yield stiffness of both the foundation soil and the buffer layer.

Rocking behavior is characterized by partial uplift and rotation of the footing. The mobilized moment capacity of a rigid square footing subjected to vertical load V can be expressed in simplified form as $M = V \cdot e$ where e is the eccentricity of the resultant vertical load from the footing center. Uplift initiates when $e \geq B/6$, with B denoting footing width. This geometric nonlinearity is explicitly captured in the numerical model through contact separation and stress redistribution. Energy dissipation during rocking is quantified through hysteretic damping, computed as the area enclosed by the moment–rotation loop:

$$E_d = \oint M \, d\theta \quad (6)$$

where M is the rocking moment and θ is the rotation angle. Larger loop areas correspond to increased energy dissipation through soil yielding. The recentering ratio R_r is defined as:

$$R_r = 1 - \frac{\theta_r}{\theta_{\max}} \quad (7)$$

where θ_r is residual rotation and $\theta_{\max}$ is peak rotation. This parameter provides a quantitative measure of post-earthquake stability. Prior to presenting the adopted parameters, it is essential to clarify that the material properties are selected to represent typical soft granular deposits improved by jet grout elements. The parameters reflect stress-dependent stiffness and moderate frictional strength, consistent with field-scale behavior. The structural components are modeled as linear elastic materials with high stiffness to ensure that nonlinear response is confined to the soil domain. The adopted properties are summarized in Table 1.

The soft soil layer exhibits relatively low stiffness and strength, facilitating noticeable plastic deformation under cyclic loading. The buffer sand possesses slightly higher friction angle but moderate stiffness to enable stress redistribution without excessive stress concentration. Jet grout elements are assigned significantly higher stiffness to simulate rigid improvement inclusions. Structural elements are modeled elastically to isolate soil-controlled nonlinear response. Contact interaction between the footing and the underlying soil or buffer layer is governed by interface elements that allow separation and sliding. The normal and shear contact stresses satisfy:

$$\sigma_n \geq 0, \quad \tau \leq \sigma_n \tan \delta \quad (8)$$

where δ is the interface friction angle. This formulation permits uplift when compressive stress vanishes and controls shear transfer during rocking cycles. Boundary conditions are selected to minimize wave reflection and artificial confinement. Lateral boundaries are treated with viscous dashpots based on Lysmer–Kuhlemeyer absorbing boundaries, defined as:

$$c_n = \rho c_p A \quad \text{and} \quad c_s = \rho c_s A \quad (9)$$

where c_n and c_s are dashpot coefficients in normal and shear directions, c_p and c_s denote compression and shear wave

velocities, and A is the boundary area. These boundaries effectively absorb outgoing waves during dynamic excitation. Prior to dynamic excitation, the initial stress state of the soil domain is established through gravity loading under static equilibrium conditions. The in-situ vertical effective stress is computed as:

$$\sigma'_v = \gamma z \quad (10)$$

where γ is the unit weight of soil and z is depth. The initial horizontal stress is defined using the at-rest earth pressure coefficient ($K_0 = 1 - \sin \phi$). So,

$$\sigma'_h = K_0 \sigma'_v \quad (11)$$

This initialization ensures realistic stress-dependent stiffness distribution prior to seismic loading. Material damping is incorporated using Rayleigh damping in the small-strain elastic range to control high-frequency numerical oscillations. The Rayleigh damping matrix is expressed as:

$$C = \alpha M + \beta K \quad (12)$$

where M and K are the mass and stiffness matrices, and α and β are mass- and stiffness-proportional damping coefficients. The damping ratio ξ at a circular frequency ω is defined as:

$$\xi = \frac{\left(\frac{\alpha}{\omega} + \beta \omega \right)}{2} \quad (13)$$

The coefficients are calibrated to provide approximately 5% damping within the dominant frequency range of the input motion. Mesh discretization is selected based on wave propagation requirements. The maximum element size Δl satisfies:

$$\Delta l \leq \frac{\lambda_{\min}}{10} \quad (14)$$

where $\lambda_{\min}$ is the minimum wavelength associated with the highest significant frequency component of the input motion. Refinement is concentrated near the soil–foundation interface to accurately capture stress gradients and uplift behavior. Sensitivity analyses confirm that further refinement produces negligible variation in rotation amplitude (<3%). Plastic correction in the Mohr–Coulomb model follows a return-mapping algorithm to ensure stress states remain on or within the yield surface. The incremental stress update is decomposed into elastic predictor and plastic corrector steps, maintaining numerical stability during cyclic loading with large strain localization.

Settlement is quantified as the permanent vertical displacement at the footing centroid after motion ceases as $S_r = \omega(t_{\text{end}})$. Rotation is computed from differential vertical displacement between opposite footing edges ($\theta = \frac{\Delta \omega}{B}$). These

definitions ensure consistent comparison across all parametric cases. To provide generalized interpretation, response quantities are also expressed in dimensionless form. The normalized rotation is defined as:

Table 1 Material properties used in the numerical models

Materials	γ (kg/m ³)	E (MPa)	ν	c (kPa)	ϕ (°)
Soft soil	1800	25	0.35	5	28
Buffer sand	1700	40	0.30	2	34
Jet grout elements	2000	1500	0.25	-	-
Footing (steel equivalent)	7850	200000	0.30	-	-

$$\theta^* = \frac{\theta}{\theta_{\text{ult}}} \quad (15)$$

$$S^* = \frac{S_r}{B} \quad (16)$$

where S^* is the normalized residual settlement, defined as the ratio of permanent vertical displacement to the B , and provides a dimensionless measure of post-seismic serviceability; θ^* is the normalized rotation, expressed as the ratio of peak rotation θ to the ultimate rotation capacity θ_{ult} (i.e., $e=B/2$), and represents the mobilized rotational demand relative to the system's theoretical limit; S_r is the residual (permanent) vertical settlement measured at the centroid of the footing after the cessation of dynamic excitation.

III. MATERIALS AND METHODS

This section describes the modeling strategy adopted to implement the numerical framework introduced previously. The focus is on ensuring methodological consistency across all simulated cases while isolating the influence of key variables, namely improvement configuration and buffer thickness. The adopted procedures are designed to enable reproducible simulations and direct comparison of response metrics under identical loading conditions. The numerical investigation was conducted through a structured and reproducible modeling workflow to ensure internal consistency across all configurations and to isolate the mechanical contribution of the soil buffer layer. All simulations were performed at prototype scale using three-dimensional nonlinear finite difference analysis. The methodology integrates geometric modeling, constitutive assignment, interface definition, static initialization, dynamic excitation, parametric configuration, and systematic response extraction. The objective was to maintain identical modeling assumptions across all cases, varying only the improvement layout and buffer thickness so that performance differences can be attributed exclusively to these parameters.

A. Geometric Configuration

The soil domain was defined as a 20 m × 15 m × 9 m three-dimensional block. These dimensions were selected following preliminary boundary-sensitivity checks to ensure that wave reflections and boundary confinement effects do not influence the rocking response. The footing was positioned at the center of the domain to maintain symmetric boundary distance in both horizontal directions. A square shallow footing with width $B=3$ m and thickness of 1 m was modeled at ground surface with zero embedment. The absence of embedment ensures that uplift, contact migration, and rotational response are governed strictly by rocking mechanics rather than passive confinement along

footing sides. Two improvement layouts were considered beneath the footing:

- JG-4C: Four discrete jet grout piles located at the footing corners.
- JG-PR: Two orthogonal pairs of diaphragm walls forming a closed-grid improvement system.

The element length of all jet grout elements was fixed at 3 m in every improved model to eliminate geometric bias in comparison. Figure 1 shows the overall numerical configuration considered in this study. The spatial arrangement of the footing and the improvement elements within the soil domain is illustrated. The figure provides a general view of the modeling setup adopted for the simulations.

B. Soil Buffer Layer Definition

The primary variable of the investigation is the intermediate granular soil layer inserted between the footing base and the improvement elements. This layer is referred to as the soil buffer (SB). Three thickness conditions were examined as (i) SB0: 0 cm (direct footing–improvement contact), (ii) SB60: 60 cm (B/5), and (iii) SB100: 100 cm (B/3). The SB0 case represents rigid stress transfer from footing to improvement elements, while SB60 and SB100 introduce increasing compliance and potential shear redistribution within the interface zone. Figure 2 presents the different soil buffer configurations examined in the analysis. The relative position of the buffer layer between the footing and the improvement elements is indicated. The figure serves to distinguish the SB0, SB60, and SB100 conditions used in the parametric study. The soft soil deposit and the soil buffer layer were modeled using the Mohr–Coulomb elastoplastic model to capture frictional yielding, shear strain localization, and irreversible settlement accumulation. Jet grout elements were modeled as linear elastic inclusions with high stiffness to simulate cemented soil behavior. The footing and superstructure were assigned linear elastic steel-equivalent properties with sufficiently large modulus to behave effectively as rigid bodies. Before presenting the adopted properties, three points are clarified:

- First, soft soil parameters represent loose granular deposits with moderate friction and low stiffness typical of poor seismic ground conditions.
- Second, the buffer layer was assigned slightly higher friction angle but moderate stiffness to allow shear redistribution without brittle stress concentration.
- Third, jet grout stiffness was selected to be significantly larger than surrounding soil to represent effective ground improvement.

The contrast in stiffness between jet grout elements and surrounding soil ensures that load transfer mechanisms differ between SB0 and buffer cases.

C. Mesh Strategy and Interface Modeling

The domain was discretized using a three-dimensional finite difference grid. Mesh refinement was concentrated directly beneath the footing and within the soil buffer layer where stress gradients and shear localization are expected to be highest. Away from the foundation zone, a coarser grid was adopted to reduce computational effort. Interface elements were assigned between

the footing and the soil/buffer layer, and between the buffer layer and the jet grout elements. These interfaces allow compression-only normal stress transfer and frictional sliding in shear. Separation (uplift) occurs when normal stress reduces to zero, enabling realistic rocking and contact area migration. Prior to dynamic loading, the soil domain was brought to static equilibrium under gravity loading. Vertical effective stresses were generated according to soil unit weight, and horizontal stresses were initialized using the at-rest earth pressure coefficient. Convergence was verified by monitoring displacement stabilization and force imbalance. This stage ensures that all dynamic responses are measured relative to a mechanically consistent in-situ stress field.

D. Dynamic Loading and Boundary Conditions

Dynamic excitation was applied at the base of the model in the form of an artificial harmonic acceleration time history with a peak ground acceleration (PGA) of 0.5g. The input motion was imposed as a prescribed acceleration boundary condition to ensure uniform upward propagation of seismic waves through the soil column. A gradual amplitude ramp was incorporated at the beginning of the record to prevent sudden force imbalance and high-frequency numerical noise, thereby allowing stable mobilization of the rocking mechanism. The frequency content of the excitation was selected to fall within the range capable of activating significant rotational demand in a footing of width $B=3$ m resting on soft soil, ensuring that uplift initiation and cyclic contact migration could fully develop.

To reduce artificial wave reflection from the model boundaries, viscous absorbing (Lysmer-type) dashpots were assigned along the lateral faces of the domain. These absorbing boundaries simulate radiation damping by allowing outgoing shear and compression waves to dissipate energy as they propagate toward the model limits. This treatment approximates semi-infinite soil conditions and prevents amplification or distortion of the dynamic response due to boundary confinement. The base boundary was configured to transmit the prescribed motion while maintaining compatibility with the absorbing side conditions. Figure 3 presents the adopted acceleration time history used in the simulations. The waveform defines the excitation applied uniformly to all analyzed configurations, ensuring that observed differences in response are exclusively related to improvement layout and buffer thickness rather than input variability. With the loading protocol and boundary conditions established, the study proceeds to the definition of the parametric case matrix described in the following section.

E. Parametric Case Matrix and Response Metrics

The parametric framework of this study was established to isolate the mechanical influence of the soil buffer layer while maintaining full consistency in all other modeling parameters. The simulation matrix was constructed by combining the two improvement configurations, such as JG-4C (four-corner jet grout piles) and JG-PR (diaphragm wall grid system); with the three buffer conditions SB0, SB60, and SB100. In addition, an unimproved Soft Soil model was included to provide a reference condition representing rocking behavior without any ground enhancement.

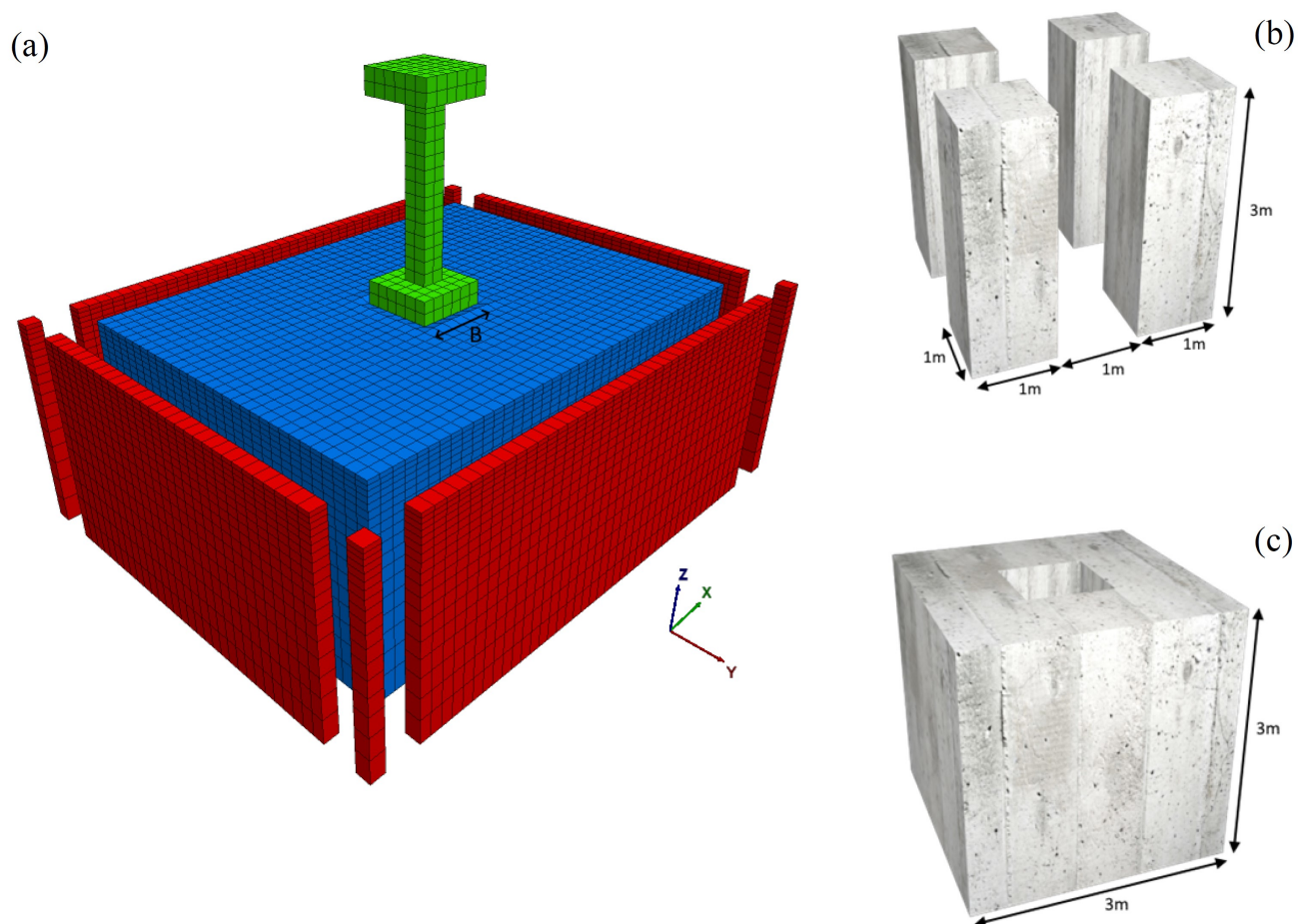


Fig. 1 3D model: (a) soil domain and square footing ($B = 3$ m); (b,c) jet grout layouts (JG-4C, JG-PR)

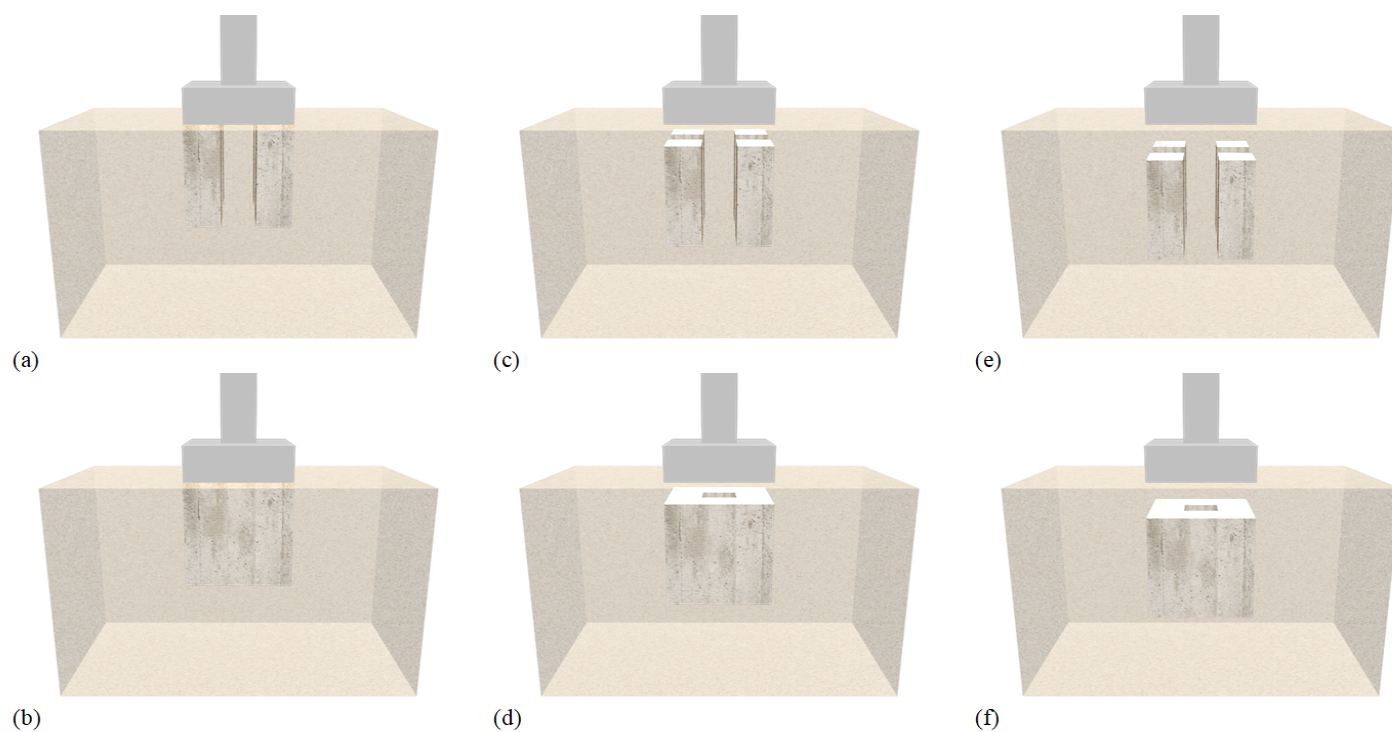


Fig. 2 Cross-sections of footing configurations: (a,b) SB0, (c,d) SB60, (e,f) SB100

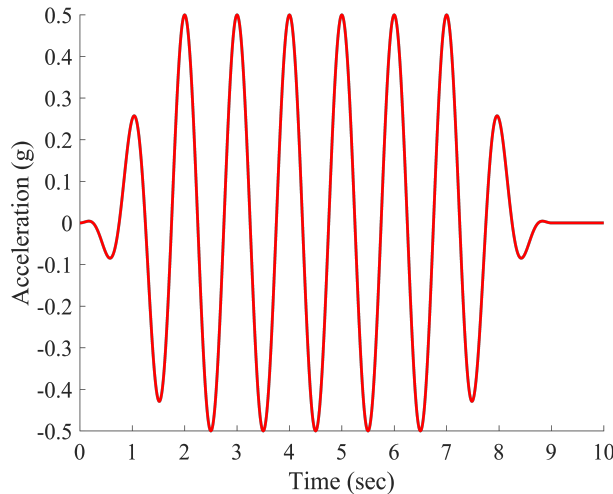


Fig. 3 Artificial harmonic acceleration (PGA = 0.5g) for dynamic loading

Table 2 Parametric case configurations

Case ID	Improvement Type	Buffer Condition
Soft Soil	None	-
JG-4C-SB0	4 corner piles	0 cm
JG-4C-SB60	4 corner piles	60 cm
JG-4C-SB100	4 corner piles	100 cm
JG-PR-SB0	Diaphragm walls	0 cm
JG-PR-SB60	Diaphragm walls	60 cm
JG-PR-SB100	Diaphragm walls	100 cm

Several constraints were imposed to preserve comparability across cases. First, the Soft Soil configuration serves as the baseline reference against which all improved systems are evaluated, allowing direct quantification of improvement efficiency in terms of rotation control and settlement reduction. Second, the embedment depth and geometric dimensions of the jet grout elements were kept identical in all improved models to eliminate geometric bias. Third, the applied seismic excitation, material properties, mesh density, and boundary conditions were held constant throughout the study. Consequently, the only variables influencing the response are the improvement layout (JG-4C versus JG-PR) and the buffer thickness (SB0, SB60, SB100). This structured approach enables clear identification of how interface compliance modifies stress transfer, rotation amplitude, and recentering capacity for different improvement mechanisms. The complete set of analyzed configurations is summarized in Table 2.

For each case listed in Table 2, response quantities were extracted using a consistent post-processing procedure to enable direct cross-comparison. Rotation (θ) was calculated from differential vertical displacement between opposite footing edges divided by B . Residual S_r was defined as the permanent vertical displacement at the footing centroid after cessation of shaking. Normalized S^* and θ^* were computed to enable dimensionless comparison. Moment–rotation hysteresis loops were extracted to quantify energy dissipation, and recentering performance was evaluated using residual-to-peak rotation ratios. The normalization configurations are illustrated in Table 3.

IV. RESULTS AND DISCUSSION

This section presents a comparative evaluation of the seismic response across all analyzed configurations, with particular emphasis on identifying the role of interface compliance in modifying system behavior. The discussion is structured to first establish the baseline response of the unimproved soil condition, followed by a systematic assessment of improved cases under varying buffer thicknesses. Key response parameters, including rotation amplitude, settlement, recentering ratio, and energy dissipation, are interpreted in relation to the underlying soil–structure interaction mechanisms. To evaluate the seismic performance of the rocking foundation system, the analysis focuses on two fundamental response relationships: the moment–rotation (M – θ) behavior and the settlement–rotation (s – θ) response. These relationships directly capture the rotational demand, load-transfer mechanism, energy dissipation, and post-seismic serviceability of the system.

In addition to peak response quantities, particular attention is given to rotation amplitude, residual settlement, recentering ratio, and hysteretic energy dissipation. By maintaining identical excitation and modeling conditions across all configurations, the observed differences in response can be attributed exclusively to the improvement layout and the soil buffer thickness. The response of the reference Soft Soil model establishes the baseline behavior of a rocking footing resting directly on unimproved soft ground. As illustrated in Figure 4, the M – θ relationship exhibits characteristic S-shaped hysteresis loops associated with progressive contact migration and plastic yielding beneath the compressed footing edge. However, the corresponding s – θ response reveals a settlement-dominated mechanism, with cumulative downward displacement during cyclic rocking. The model records a residual settlement of 13.6 cm, rotation amplitude of 0.093 rad, and a maximum rocking moment of 1163 kNm. The relatively low recentering ratio (0.5704) confirms that, although rocking is activated, the soft soil lacks sufficient stiffness and confinement to prevent significant permanent deformation (Table 4).

The introduction of jet grout improvement elements substantially alters the settlement–rotation coupling. As shown in Figure 5, all improved configurations demonstrate a pronounced reduction in permanent settlement compared to the Soft Soil case. In particular, the direct-contact configurations (SB0) exhibit the lowest residual settlements, with values of 0.023 m for JG-4C-SB0 and 0.022 m for JG-PR-SB0. This behavior is attributed to direct vertical load transfer from the footing to the stiff jet grout inclusions, thereby limiting compressive strain development in the surrounding soft soil. When a soil buffer layer is introduced (SB60 and SB100), residual settlement increases relative to SB0 cases but remains significantly lower than that of the Soft Soil model. The increase is mechanically consistent: the buffer layer introduces compliance at the interface, permitting distributed shear deformation before load transfer reaches the rigid inclusions. Although this mechanism slightly increases vertical displacement, it simultaneously reduces stress concentration and promotes more stable rocking behavior. The settlement trend therefore reflects a controlled trade-off between stiffness and rotational stability.

Table 3 Normalized residual settlement and rotation for different cases

Case ID	Normalized rotation	Normalized residual settlement
Soft Soil	0.563	0.045
JG-4C-SB0	0.733	0.008
JG-4C-SB60	0.624	0.026
JG-4C-SB100	0.503	0.037
JG-PR-SB0	0.690	0.007
JG-PR-SB60	0.642	0.018
JG-PR-SB100	0.618	0.032

Table 4 Summary of four performance parameters of the models

Case ID	Recentering Ratio	Residual Settlement (m)	Energy Dissipation (kJ)	Rotational Amplitude (rad)
Soft Soil	0.570	0.136	345.5	0.093
JG-4C-SB0	0.672	0.023	523.1	0.121
JG-4C-SB60	0.884	0.077	394.2	0.103
JG-4C-SB100	0.846	0.111	338.6	0.083
JG-PR-SB0	0.984	0.022	750.2	0.114
JG-PR-SB60	0.987	0.055	632.1	0.106
JG-PR-SB100	0.939	0.096	496.5	0.102

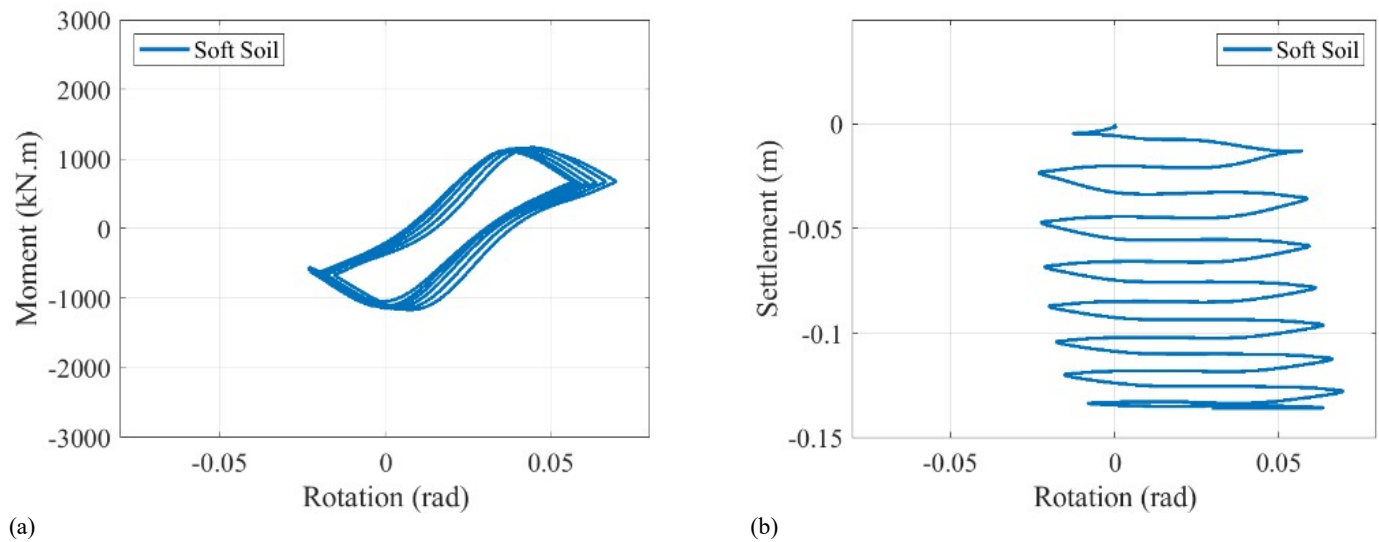


Fig. 4 (a) Moment-rotation ($M-\theta$) and (b) settlement-rotation ($s-\theta$) responses of the soft soil reference model

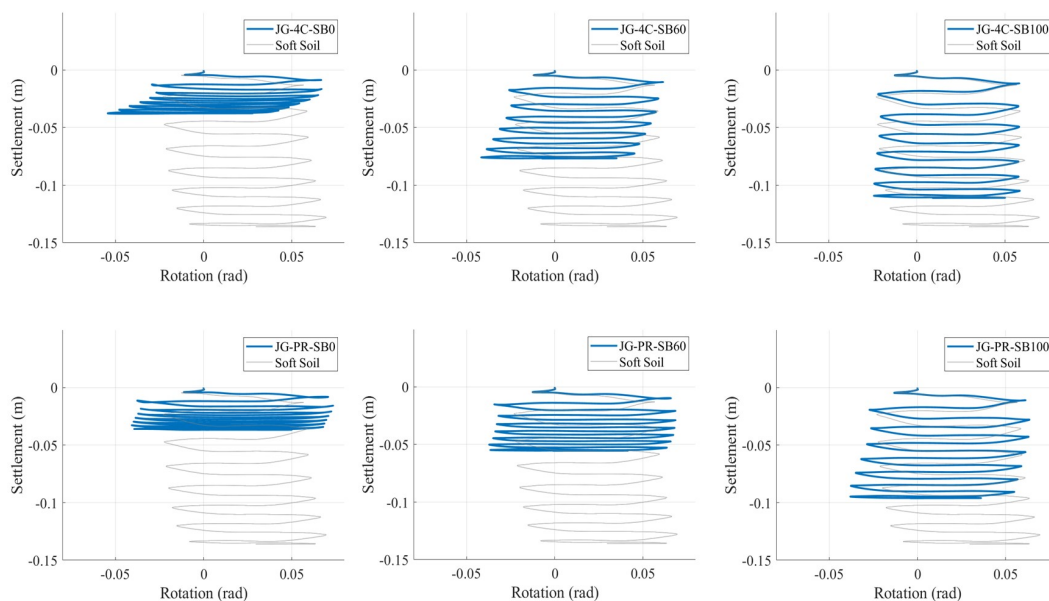


Fig. 5 Settlement-rotation ($s-\theta$) responses for all improved configurations

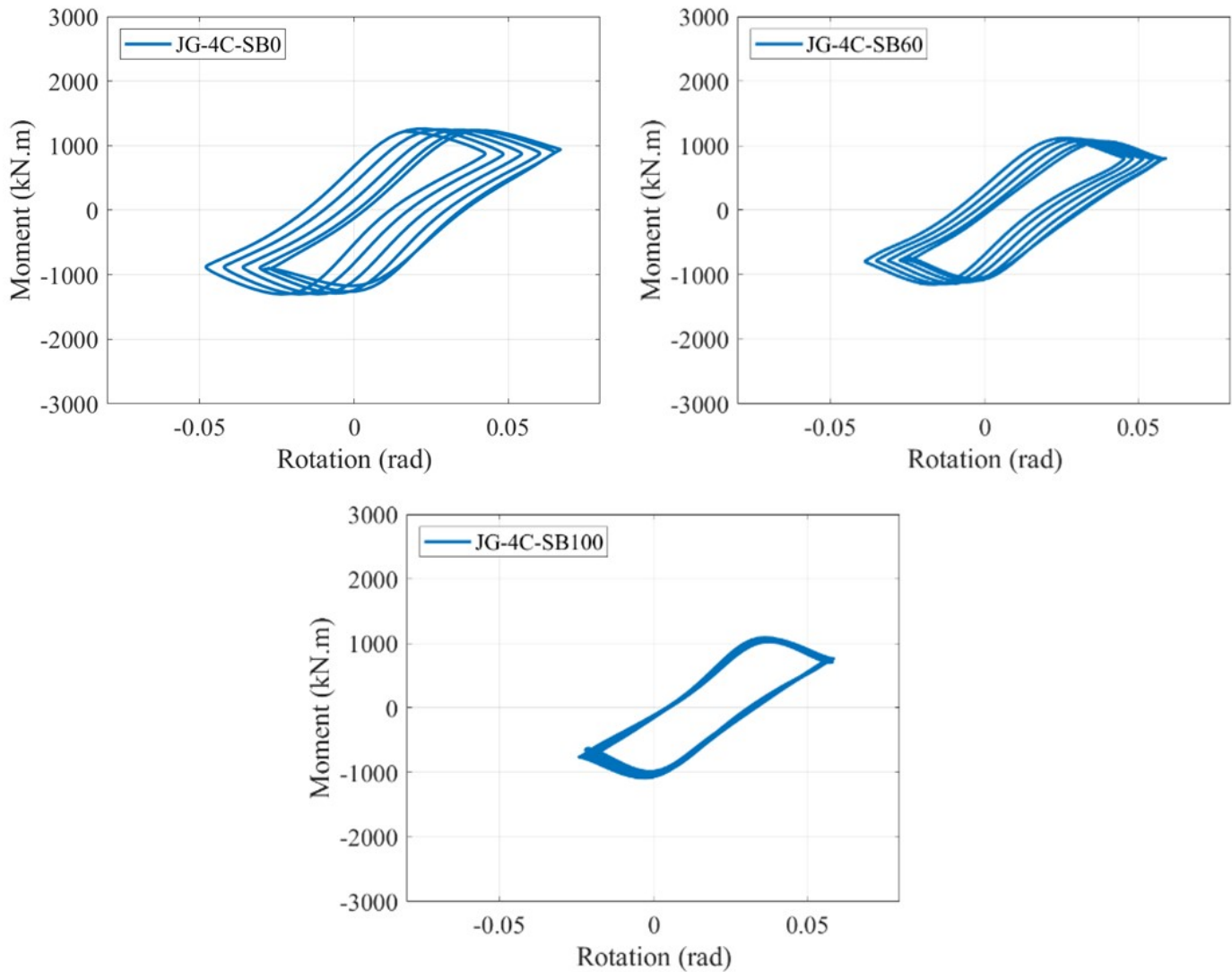


Fig. 6 Moment–rotation ($M-\theta$) hysteresis loops for improved configurations

This observed trade-off highlights the importance of interface deformability in regulating the balance between vertical load transfer efficiency and rotational flexibility. While increased stiffness enhances bearing capacity and limits settlement, it also constrains deformation pathways, leading to more abrupt stress redistribution during rocking. Conversely, introducing controlled compliance allows gradual stress transfer and reduces localized failure, thereby stabilizing the rotational response.

Peak rotation alone can be misleading because it depends strongly on excitation characteristics. For this reason, rotation amplitude (difference between maximum positive and negative rotation) is adopted as the principal rotational demand parameter. The $M-\theta$ responses of all improved cases are presented in Figure 6. Interestingly, the SB0 configurations exhibit higher rotation amplitudes than the Soft Soil model. Rotation amplitude increases to 0.121 rad for JG-4C-SB0 and 0.114 rad for JG-PR-SB0. This increase can be explained mechanically: by increasing vertical stiffness and reducing settlement, the system mobilizes greater rocking moment capacity, allowing larger rotational excursions before significant stress concentration occurs. The slightly higher amplitude in JG-4C-SB0 compared with JG-PR-

SB0 is attributed to the discrete nature of the corner piles, which provide fewer continuous resistance paths than the diaphragm wall grid, resulting in reduced rotational restraint.

A distinct trend emerges when the soil buffer layer is introduced. Increasing buffer thickness reduces rotation amplitude. For the JG-4C configuration, amplitude decreases from 0.121 rad (SB0) to 0.0828 rad (SB100). A similar trend is observed for JG-PR models. Figure 7 summarizes the amplitude variation across all cases. The reduction in amplitude is accompanied by improved recentering ratios, indicating enhanced post-seismic stability. The underlying mechanism is associated with the “soil-rounding” effect at the footing–buffer interface. In SB0 cases, direct impact between the footing and rigid jet grout elements generates localized stress concentration and abrupt contact transitions. With the introduction of a granular buffer layer, contact stresses are redistributed through distributed shear strains within the interposed soil, leading to smoother stress gradients and reduced overturning tendency. Among the examined thicknesses, SB60 (B/5) provides the most favorable recentering performance.

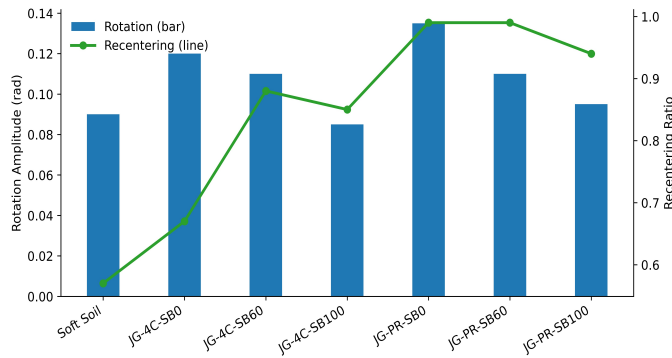


Fig. 7 Comparison of rotation amplitudes and recentering ratios for all cases

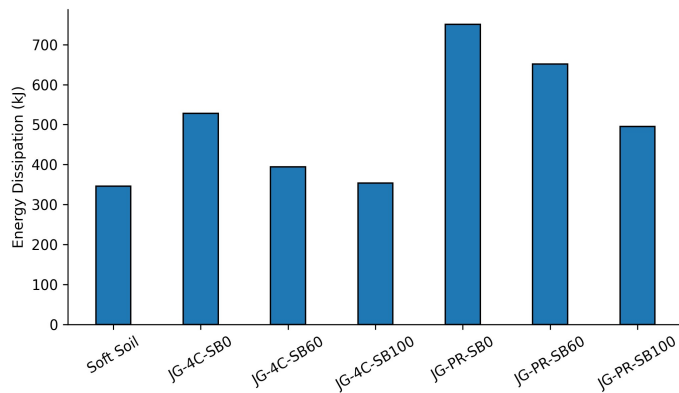


Fig. 8 Comparison of the amount of dissipated energy for all cases

The JG-PR-SB60 model achieves a recentering ratio of 0.987, indicating near-complete recovery of vertical alignment even under $\text{PGA} = 0.5g$ excitation. Neither SB0 nor SB100 configurations exceed this performance level, suggesting that excessive compliance (SB100) reduces energy dissipation efficiency, whereas zero compliance (SB0) increases stress concentration.

Energy dissipation capacity is quantified by the area enclosed within the $M-\theta$ hysteresis loops. Figure 8 illustrates the variation of dissipated energy for representative configurations. The SB0 cases demonstrate the highest energy dissipation; for example, JG-PR-SB0 dissipates 750.19 kJ compared to 496.54 kJ for JG-PR-SB100. The larger loop area in SB0 models results from intense localized plastic yielding and higher stress concentration at the contact interface. As buffer thickness increases, hysteresis loops become narrower. This reflects the redistribution of stresses within the granular buffer layer, which reduces localized stress concentration and shifts the deformation mechanism toward distributed shear strain. Although this mechanism lowers total energy dissipation, it enhances recentering efficiency and reduces peak rotational demand. So, an inherent balance exists between energy dissipation and post-seismic alignment stability.

Table 4 summarizes the principal response metrics for all analyzed configurations, including residual settlement, rotation amplitude, recentering ratio, and energy dissipation. This comparative overview highlights the trade-offs between settlement control, rotational stability, and energy dissipation across the parametric matrix.

V. CONCLUSION

This study employed three-dimensional nonlinear finite difference analysis in FLAC3D to investigate the seismic behavior of a square rocking foundation incorporating an intermediate soft granular soil buffer layer placed above jet grout improvement elements. The results demonstrate that interface compliance plays a decisive role in governing rotational demand, recentering efficiency, settlement accumulation, and energy dissipation under seismic loading. The analyses reveal that increasing ground stiffness through direct footing–improvement contact (SB0) effectively reduces settlement but may amplify rotational amplitude. In contrast, introducing a soil buffer layer decreases rotation amplitude as buffer thickness increases. For example, increasing the buffer thickness to 100 cm (B/3) reduced the rotational amplitude of the JG-4C model from 0.121 rad to 0.083 rad, indicating a significant mitigation of overturning tendency. This trend confirms that controlled interface compliance can counteract the amplification of rotational demand associated with rigid ground improvement. The improvement in rotational stability is accompanied by enhanced recentering performance. The highest recentering ratio (0.987) was achieved in the JG-PR-SB60 configuration, corresponding to a buffer thickness of B/5. This performance is attributed to the “soil-rounding” mechanism, whereby the interposed granular layer redistributes contact stresses through distributed shear deformation rather than localized stress concentration at rigid inclusion heads. Among the examined cases, the B/5 thickness provides the most balanced behavior in terms of rotational control and post-seismic alignment. Although the soil buffer does not outperform SB0 configurations in minimizing residual settlement, it substantially reduces settlement compared with the unimproved Soft Soil case. However, this improvement is accompanied by a moderate reduction in hysteretic energy dissipation relative to direct-contact systems. These findings indicate that rocking isolation on improved soft ground requires a balance between settlement control, rotational stability, and energy dissipation, with the buffer layer serving as a tunable design parameter to optimize overall seismic performance.

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DATA AVAILABILITY STATEMENT

The data supporting the findings of this study are available from the corresponding author upon reasonable request. No publicly archived dataset was generated.

ETHICS APPROVAL

This study did not involve human participants, personal data, or animal subjects. No specific ethical approval was required.

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AUTHORS' CONTRIBUTIONS

Ali Ghaffarnejhad Parto conducted the primary data analysis and contributed to data collection, preprocessing, and interpretation, and drafted the initial

manuscript, provided supervision, conceptual guidance, critical revision of the manuscript, overall project administration, and final approval of the manuscript for publication. All authors reviewed and approved the final version of the manuscript.

CONFLICT OF INTEREST

The authors have not disclosed any competing interests.

USE OF AI

AI tools were not used in data generation, analysis, results, or decision-making.

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REFERENCES

- Allmond J.D., Kutter B.L. (2014). Design considerations for rocking foundations on unattached piles. *Journal of Geotechnical and Geoenvironmental Engineering*, 140(10), 04014058. [https://doi.org/10.1061/\(ASCE\)GT.1943-5606.0001162](https://doi.org/10.1061/(ASCE)GT.1943-5606.0001162).
- Amiri A., Hosseini S.M.M.M., Soroush A. (2025). Experimental study of influencing pile to cap connections on the rocking behaviors of piled foundations. *Scientific Reports*, 15(1), 14937. <https://doi.org/10.1038/s41598-025-98940-x>.
- Anastasopoulos I., Gazetas G., Loli M., Apostolou M., Gerolymos N. (2010). Soil failure can be used for seismic protection of structures. *Bulletin of Earthquake Engineering*, 8, 309-326. <https://doi.org/10.1007/s10518-009-9145-2>.
- Bin J., Huang Z., Wu J., Pan P., Jing L. (2024). Influence of rocking shallow foundation parameters and analysis of seismic response characteristics. *Buildings*, 14(12), 3788. <https://doi.org/10.3390/buildings14123788>.
- Deng L., Kutter B.L., Kunnath S.K. (2012). Centrifuge modeling of bridge systems designed for rocking foundations. *Journal of Geotechnical and Geoenvironmental Engineering*, 138(3), 335-344. [https://doi.org/10.1061/\(ASCE\)GT.1943-5606.0000605](https://doi.org/10.1061/(ASCE)GT.1943-5606.0000605).
- Gajan S., Kutter B.L. (2008). Capacity, settlement, and energy dissipation of shallow footings subjected to rocking. *Journal of Geotechnical and Geoenvironmental Engineering*, 134(8), 1129-1141. [https://doi.org/10.1061/\(ASCE\)1090-0241\(2008\)134:8\(1129\)](https://doi.org/10.1061/(ASCE)1090-0241(2008)134:8(1129)).
- Hakhamaneshi M., Kutter B.L. (2016). Effect of footing shape and embedment on the settlement, recentering, and energy dissipation of shallow footings subjected to rocking. *Journal of Geotechnical and Geoenvironmental Engineering*, 142(12), 04016070. [https://doi.org/10.1061/\(ASCE\)GT.1943-5606.0001564](https://doi.org/10.1061/(ASCE)GT.1943-5606.0001564).
- Irani A., Bonab M.H., Assadi-Langroudi A., Maleki Tabrizi E. (2024). Pore-pressure-dependent performance of rocking foundations. *Soil Dynamics and Earthquake Engineering*, 183, 108772. <https://doi.org/10.1016/j.soildyn.2024.108772>.
- Irani A., Bonab M.H., Sarand F.B., Katebi H. (2023). Overall improvement of seismic resilience by rocking foundation and trade-off implications. *International Journal of Geosynthetics and Ground Engineering*, 9(4), 40. <https://doi.org/10.1007/s40891-023-00454-x>.
- Irani A.E., Bonab M.H., Sarand F.B., Katebi H. (2021). Behavioral evaluation of soil-based rocking foundations against seismic loading: A review study. *Geotechnical Geology*, 17(1), 455-463.
- Irani A.E., Bonab M.H., Sarand F.B., Katebi H. (2023). Investigation on the behavior of bridge piers considering rocking isolation constructed on non-plastic silts and sands using 1g shaking table tests. *Periodica Polytechnica Civil Engineering*, 67(3), 835-845. <https://doi.org/10.3311/PPci.21713>.
- Itasca (2020). *Fast Lagrangian Analysis of Continua in 3 Dimensions: User's guide (Version 6.0)*. Itasca Consulting Group, Minneapolis.
- Ko K.W., Ha J.G., Park H.J., Kim D.S. (2019). Centrifuge modeling of improved design for rocking foundation using short piles. *Journal of Geotechnical and Geoenvironmental Engineering*, 145(8), 04019031. [https://doi.org/10.1061/\(ASCE\)GT.1943-5606.0002064](https://doi.org/10.1061/(ASCE)GT.1943-5606.0002064).
- Moradi M., Hosseini S.M.M.M., Khezri A. (2023). Investigation of rocking mechanism of shallow foundations on sand via PIV technique. *Earthquake Engineering & Structural Dynamics*, 52(6), 1762-1784. <https://doi.org/10.1002/eqe.3842>.
- Moradi M., Khezri A., Hosseini S.M.M.M., Park H., Lee D. (2024). Experimental investigation of embedment depth effects on the rocking behavior of foundations. *Geosciences*, 14(12), 351. <https://doi.org/10.3390/geosciences14120351>.
- Nikvand V., Bonab M.H. (2024). Seismic response analysis of tunnels using a coupled polygonal finite element method. *Civil and Geoengineering Letters*, 1(1), e100009.
- Rassoulpour S., Shiravand M.R., Safi M. (2025). Effect of soil-structure interaction on seismic behavior of self-centering rocking piers supported on shallow foundations. *Soil Dynamics and Earthquake Engineering*, 194, 109339. <https://doi.org/10.1016/j.soildyn.2025.109339>.
- Sadjadi M., Fadaee M., Ghannad M.A., Jahankhah H. (2023). Seismic performance of deformable rocking soil-structure systems subjected to pulse-type excitations. *Journal of Earthquake Engineering*, 27(12), 3290-3318. <https://doi.org/10.1080/13632469.2022.2134232>.
- Xiong G., Wang F., Zhou F. (2024). Torsional and rocking response analysis of foundations subjected to spatially inclined incident SV waves considering soil nonlinearity. *Soil Dynamics and Earthquake Engineering*, 184, 108868. <https://doi.org/10.1016/j.soildyn.2024.108868>.