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A Mass-Gravity Scaling and Beam-/Shell-Volume Coupling Framework for Efficient Pile Installation in Centrifuge Tests

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ABSTRACT: Physical modelling, particularly through centrifuge testing, provides essential insights into geotechnical processes such as pile installation involving large deformations at realistic stress levels. However, high-fidelity numerical replication of these processes for comprehensive analysis is often computationally intensive. This paper introduces and validates a combined numerical approach designed to significantly improve the efficiency of explicit simulations for quasi-static soil-structure interactions. Explicit integration schemes are essential for large deformation analyses, yet the critical time increment is typically governed by finely meshed, non-rigid structural elements, leading to excessive simulation times. To address this, two techniques are combined: Mass-Gravity-Scaling (MGS), which increases the stable time increment by scaling density and gravity while preserving the initial geostatic stress state, and a kinematic beam-/shell-volume coupling method for the computationally efficient modelling of slender deformable bodies. Both techniques are implemented within the Coupled Eulerian-Lagrangian (CEL) framework and benchmarked against 50g centrifuge test data for jacked pile installation in sand. The combined methods achieve a computational speed-up of approximately 6 times at a moderate scaling factor (S = 10) and up to 13 times (S = 100) compared to the conventional approach, while maintaining high fidelity to the physical test results (mean percentage error remains below 7% for moderate scaling factors). The study provides a validated and practical pathway to make explicit simulations more efficient, enabling broader parametric studies and the integration of more advanced soil models within feasible runtimes.

1 INTRODUCTION

Physical modelling, especially centrifuge testing, is a cornerstone of geotechnical engineering research, providing invaluable insights into complex soil-structure interaction under realistic stress conditions at manageable model scales (Taylor, 1994; Garnier et al., 2007). These experiments are particularly valuable for large deformation problems such as pile installation, where stress history, density changes, and strong nonlinearity govern the response. However, their full interpretation and generalisation often require complementary numerical simulations to explore behaviours beyond the instrumented measurements and to conduct extensive parametric investigations.

The Coupled Eulerian-Lagrangian (CEL) method (Benson, 1992; Qiu et al., 2011) is well-suited for such simulations because it handles extreme material distortion by modelling the soil with a spatially fixed Eulerian mesh and structures with Lagrangian elements. This hybrid approach circumvents mesh distortion issues typical of purely Lagrangian Finite Element Methods (FEM) (Zienkiewicz et al., 2000), making it a preferred tool for installation-type problems involving substantial soil flow (e.g., pile penetration: Stapelfeldt et al., 2020; anchor installation: Dao et al., 2023). While CEL is especially effective for the installation phase, traditional FEM remains essential for complementary analyses such as Class-A predictions of bearing capacity (Alkateeb and Grabe, 2025a; Alkateeb and Grabe, 2026; Alkateeb, 2026; Alkateeb et al., 2026).

Despite these capabilities, computational efficiency remains a significant challenge for complex 3D simulations. In explicit schemes, the stable time increment is governed by the smallest (and often stiffest) elements in the model. This becomes particularly restrictive when modelling non-rigid penetrating bodies such as slender piles, which require fine meshing to accurately capture their deformation. These fine elements can dominate the stable time increment and lead to impractically long runtimes. In many cases, this computational demand limits the ability to perform extensive parametric studies or to incorporate advanced constitutive models necessary for precise representation of soil behaviour, thereby restricting the full potential of numerical analyses in supporting physical modelling.

To address these computational challenges, this paper investigates a combined efficiency strategy that targets both dominant sources of explicit cost. The strategy comprises the use of Mass-Gravity-Scaling (MGS), which increases the critical time increment by modifying density and gravity while preserving the initial soil stress state (Kencana et al., 2021), and the kinematic coupling of shell/beam and volume elements providing an efficient method for modelling slender deformable piles by reducing the number of degrees of freedom while maintaining an accurate structural response (Osthoff and Grabe, 2018). While both methods are individually established, their combined use within a 3D CEL framework for centrifuge-test replication and their validation against high-quality physical data require careful formulation and systematic benchmarking, especially regarding the accuracy-efficiency trade-off.

This study validates the combined MGS and kinematic coupling approach within a CEL framework against well-documented 50g centrifuge tests of pile installation in sand by Deeks and White (2006). The primary objective is to demonstrate how these methods can substantially improve computational efficiency, reducing runtimes by an order of magnitude and enabling a more practical and detailed interpretation of physical model tests without compromising accuracy.

2 METHODOLOGY

2.1 Computational challenges in explicit simulations

Explicit time integration schemes are essential for resolving large deformation problems but face significant computational challenges due to the conditional stability limit on the time step size Δt_{crit}. This critical time step is governed by the minimum element length L_e and the material's dilatational wave speed c_d, which depends on the material's Young's modulus E, density ρ and Poisson's ratio ν, as defined by:

$$\Delta t_{crit} \leq \frac{L_e}{c_d} \quad \text{with} \quad c_d = \sqrt{\frac{E(1-\nu)}{\rho(1+\nu)(1-\nu)}} \qquad (1)$$

In geotechnical analyses involving soil-structure interaction, stiff structural elements like steel piles often dictate Δt_{crit} due to their high stiffness ($E_{steel} \gg E_{soil}$) and the fine meshing required to capture deformation gradients. Discretising slender piles with standard volume elements necessitates a dense mesh, leading to prohibitively small time steps and excessive computational costs for 3D simulations. To overcome these challenges, advanced techniques such as MGS and kinematic coupling are employed to enhance computational efficiency without compromising solution accuracy, especially for problems simulating quasi-static conditions observed in centrifuge tests.

2.2 The Coupled Eulerian-Lagrangian (CEL) method

In CEL, the Eulerian domain allows material to flow through spatially fixed elements, computing material volume fractions, while a void region accommodates displaced material (Figure 1). The Lagrangian pile interacts with the Eulerian soil via a general contact algorithm based on penalty methods. This hybrid approach enables simulation of complex soil-structure interactions and extreme material distortion, making it highly suitable for processes such as pile or anchor installation (Qiu et al., 2011; Stapelfeldt et al., 2020; Bienen et al., 2021; Alkateeb and Grabe, 2025b; Dao et al., 2023, 2025).

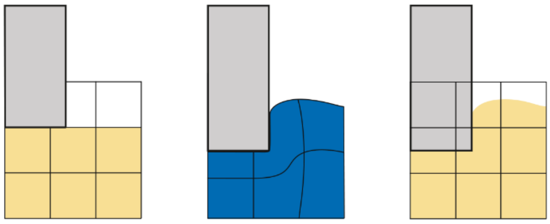

Figure 1. Schematic illustration of the Coupled Eulerian-Lagrangian (CEL) method for soil-structure interaction. The Lagrangian mesh (representing the pile) moves through the fixed Eulerian mesh (representing the soil).

2.3 Mass-Gravity-Scaling (MGS) technique

The Mass-Gravity-Scaling (MGS) technique enhances the efficiency of explicit numerical simulations for quasi-static geotechnical problems by artificially increasing the critical time increment without altering the initial geostatic stress state. Unlike traditional mass-scaling, which can introduce inaccuracies by changing the initial overburden pressure, MGS achieves this by proportionally scaling the material density ρ by a factor S while inversely scaling the gravitational acceleration g. If the scaled density is $\rho' = S{\cdot}\rho$ and the scaled gravity is $g' = g/S$, the initial vertical stress σ_v at depth z remains mathematically unchanged:

$$\sigma_v = S \cdot \rho \cdot \left(\frac{g}{S}\right) \cdot z = \rho \cdot g \cdot z \qquad (2)$$

By increasing ρ, the dilatational wave speed $c_d = \sqrt{E/\rho}$ is reduced, consequently increasing Δt_{crit} and accelerating the simulation. The increased mass, however, simultaneously amplifies the inertia of all bodies in the model. For a given penetration velocity, the higher mass produces larger inertial forces. Therefore, the applicability of MGS is contingent on the quasi-static nature of the problem: the ratio of kinetic to internal energy must remain sufficiently low to ensure that inertial effects do not distort the solution. This condition must be explicitly verified, as discussed in Section 4.3. MGS is thus particularly effective for slow, displacement-controlled quasi-static conditions, such as pile jacking during centrifuge tests, where static resistance governs the response (Kellezi and Kudsk, 2009; Kencana et al., 2021).

2.4 Kinematic coupling of shell/beam and volume elements

Kinematic coupling provides a computationally efficient approach to model slender deformable profiles, such as piles, in large deformation simulations. In the present study, beam elements are used to represent the pile; however, the same kinematic coupling concept is directly applicable to other slender structures when modelled using shell elements. This technique addresses the geometric challenge where discretising slender profiles solely with volume elements requires very fine meshing to capture bending and torsional behaviour accurately, leading to small element lengths (L_e) and restrictive critical time increments.

The core mechanical behaviour (bending and torsional stiffness) of the slender profile is defined by the shell/beam elements positioned at the profile's neutral axis (Figure 2), offering efficient structural representation with significantly fewer degrees of freedom than a full volume element discretisation.

Surrounding these structural elements, a layer of volume elements is used to represent the physical geometry of the profile and facilitate interaction with the surrounding Eulerian soil. To ensure these volume elements do not dominate the structural response or the critical time increment, they are assigned "dummy" material properties that are very soft linear-elastic parameters (e.g., Young's modulus $\approx 0.0005\ E_{\mathrm{steel}}$) and a reduced density (e.g., $\approx 0.01\ \rho_{\mathrm{steel}}$). This formulation makes them approximately 1,000 times less stiff than the shell/beam elements, allowing them to passively follow the deformation of the structural core without contributing significantly to the overall mechanical resistance. The central shell/beam elements, conversely, retain the actual material properties of the pile (e.g., steel).

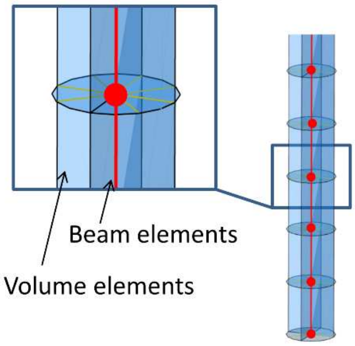

Figure 2. Schematic representation of the kinematic coupling between a central beam element and surrounding soft volume elements for a slender profile.

Kinematic constraints link the nodes of both element types, ensuring compatible deformation and load transfer between the contact layer and the structural shell. The soil-pile interaction is enforced on the outer surface of the volume elements via a penalty-based general contact formulation; therefore, the global contact forces are governed primarily by the soil constitutive response and the chosen contact law, while the soft volume elements mainly serve as a geometric/contact medium that enables soil displacement in the CEL framework. We note that introducing a compliant contact layer can slightly change the local interface compliance in a penalty formulation compared with a fully solid pile discretisation; however, the dummy material is selected to have negligible influence on the pile's structural response, and validation shows that the overall penetration resistance remains essentially unchanged. Because the volume elements are assigned a very soft linear-elastic behaviour, their influence on the mechanical behaviour of the profile is intended to be negligible and they do not control the explicit stable time increment. This approach improves computational efficiency by reducing the number of elements and allowing a coarser structural discretisation (larger characteristic element lengths L_e, which permits larger critical time increments in the explicit analysis.

3 CENTRIFUGE TESTS AND NUMERICAL MODEL SETUP

3.1 Centrifuge tests setup

To validate the accuracy and efficiency of the combined MGS and kinematic coupling techniques, the numerical simulations were benchmarked against experimental data from 50g centrifuge tests conducted by Deeks and White (2006). These tests are widely recognised in physical modelling for providing a comprehensive and reliable dataset on pile installation in sand.

The centrifuge tests were performed using the 10-metre diameter Turner beam centrifuge at Cambridge University, operating at a centrifugal acceleration of 50g. This high gravitational field enabled the replication of prototype stress levels in a scaled-down model, ensuring accurate reproduction of the soil's stress-dependent stiffness and strength. The centrifuge model container had a diameter of 850 mm and a depth of 400 mm (Figure 3), dimensions sufficient to minimise boundary effects. The tests utilised Fraction E silica sand, which has geotechnical properties similar to the well-characterised University of Western Australia (UWA) silica sand, a fine sand with subrounded to subangular grains (Chow et al., 2019).

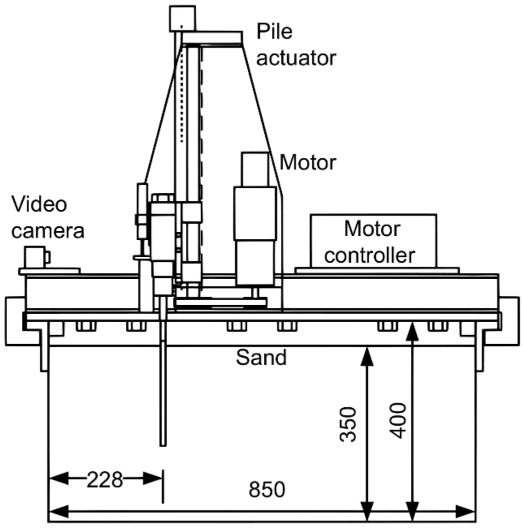

Figure 3. Cross-sectional view of the centrifuge test setup for pile installation experiments in sand (Deeks and White, 2006).

Figure 4 compares the grain size distribution of Fraction E and UWA sands, confirming their similarity. The sand was prepared through dry air pluviation to achieve a uniform density. Deeks and White (2006) report a very dense sand state with measured relative density $I_D \approx$ 90-96%. The model pile was a cylindrical, closed-ended pile with an external diameter of 12 mm and a length of 200 mm at model scale. This corresponds to a prototype pile with a diameter of 0.6 m and a length of 10 m. The pile comprised a stainless-steel friction sleeve and an aluminium alloy upper section.

The pile was installed under displacement-controlled conditions, jacked into the sand at a constant model-scale velocity of 0.4 mm/s, equivalent to a prototype rate of 2 cm/s. Instrumentation included load cells for axial forces (pile base and shaft) and displacement transducers for penetration depth. These high-fidelity measurements of penetration resistance, shaft friction, and base resistance serve as the validation benchmark.

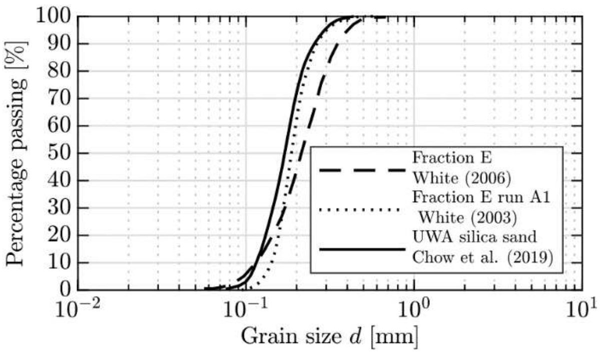

Figure 4. Comparison of grain size distribution curves for Fraction E silica sand (White, 2003; Deeks and White, 2006) and UWA silica sand (Chow et al., 2019).

3.2 Numerical model implementation

A three-dimensional CEL model was developed in Abaqus/Explicit 2023 to replicate the centrifuge tests. The numerical setup was designed to directly match the conditions of the 50g experiments.

3.2.1 Model setup

The numerical model in Figure 5 consists of Eulerian soil domain and a Lagrangian pile. The Eulerian domain was discretised using approximately 200,000 C3D8R elements with enhanced hourglass control.

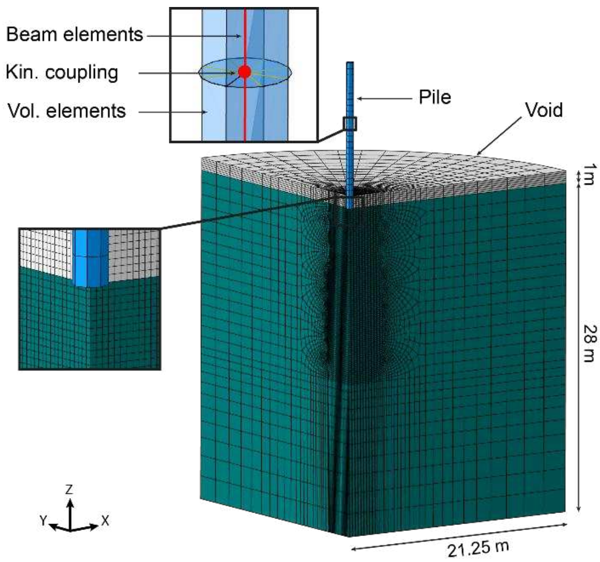

Figure 5. Finite element mesh of the quarter numerical model, showing the Eulerian soil domain and the Lagrangian pile elements.

To replicate the 50g centrifuge test conditions, the domain diameter was set to 42.5 m at prototype scale. The vertical extent included the 20 m prototype soil depth, extended by an additional 8 m to ensure numerical stability, along with a 1 m void region above the surface to accommodate soil heave. Due to symmetry, only a quarter model was employed, with X-symmetry and Y-symmetry boundary conditions applied to the respective outer surfaces. The bottom boundary was fixed in all directions, and lateral boundaries were constrained against radial displacement while allowing vertical movement. A finer mesh was used near the pile to capture high deformation gradients effectively.

For the pile, two distinct Lagrangian discretisation approaches were implemented to assess the proposed framework:

1. **Volume element pile (conventional model):** This baseline model discretised the pile using standard solid C3D8R volume elements. It provides a reference for accuracy and computational cost.
2. **Beam-/shell-volume coupled pile (proposed model):** The pile's structural response was represented by Lagrangian beam elements (B31) along the pile centreline. These beam elements were kinematically coupled (labelled as 'Kin. coupling' in Figure 5) to a surrounding layer of soft C3D8R Lagrangian volume elements (labelled as 'Vol. elements') acting as the contact interface. As described in Section 2.4, the volume elements were assigned significantly reduced stiffness and density to serve primarily as a contact interface, ensuring that the structural response was governed by the computationally efficient beam elements.

The interaction between the pile and the surrounding Eulerian soil was modelled using a general contact algorithm with penalty enforcement. A friction coefficient of $\mu = \tan(2/3\varphi)$ was applied to represent the rough interface between steel and Fraction E silica sand, consistent with the modelling assumptions used for this benchmark replication.

3.2.2 Modelling of Fraction E sand

The Fraction E silica sand was modelled using the hypoplastic constitutive model (von Wolffersdorff, 1996) extended with intergranular strain (Niemunis and Herle, 1997). This advanced constitutive law captures the non-linear, stress-dependent behaviour of sand, including dilatancy and pycnotropy (density dependence), which are the dominant mechanisms during pile installation. The material parameters for Fraction E silica sand, adopted from Pucker et al. (2013), are summarised in Tables 1 and 2.

Table 1. Hypoplastic parameters for Fraction E sand (adapted from Pucker et al., 2013).

φ_c	h_s	n	e_{d0}	e_{c0}	e_{i0}	α	β
[°]	[MPa]	[-]	[-]	[-]	[-]	[-]	[-]
30.0	1354	0.34	0.49	0.76	0.86	0.18	1.27

Table 2. Intergranular strain parameters for Fraction E sand (adapted from Pucker et al., 2013).

m_T	m_R	R	β_R	χ
[-]	[-]	[-]	[-]	[-]
3.07	5.16	10^{-4}	0.58	5.74

These parameters are considered appropriate given the similar grain size distribution and reported properties of Fraction E and UWA silica sands. For the chosen critical state friction angle $\varphi_c = 30°$, Jaky's formula ($K_0 = 1 - \sin\varphi$) yields $K_0 \approx 0.5$. This value is justified for the dense sand state of the centrifuge tests and is consistent with standard geotechnical practice for normally consolidated dense sand.

3.2.3 Modelling procedure

The numerical simulation procedure mirrors the centrifuge test phases. All simulations were performed using a quasi-static dynamic-explicit analysis, in which inertial effects are negligible compared to static resistance. This approach is appropriate for the slow, displacement-controlled pile jacking process modelled here. Initially, a geostatic step established the in-situ K_0 stress state within the Eulerian soil domain under a 50g gravitational field. Following geostatic stress initialisation, the pile installation was simulated in a dynamic-explicit step. A displacement-controlled boundary condition was applied to the top of the pile, prescribing a constant downward velocity of 2 cm/s (prototype scale), consistent with the reported test procedure.

To evaluate the efficiency gains, simulations were conducted with and without the MGS. For MGS simulations, the soil density was scaled up by a factor S (e.g., $S = 10$ or $S = 100$), and the gravitational acceleration was scaled down by the same factor (g/S), preserving the initial overburden stress according to Eq. (2). Each scaling level was tested for both the conventional and coupled pile models.

4 NUMERICAL RESULTS AND DISCUSSION

Numerical back-calculations were performed using both the conventional volume element pile model and the proposed beam-/shell-volume coupled pile model, with and without the application of the MGS.

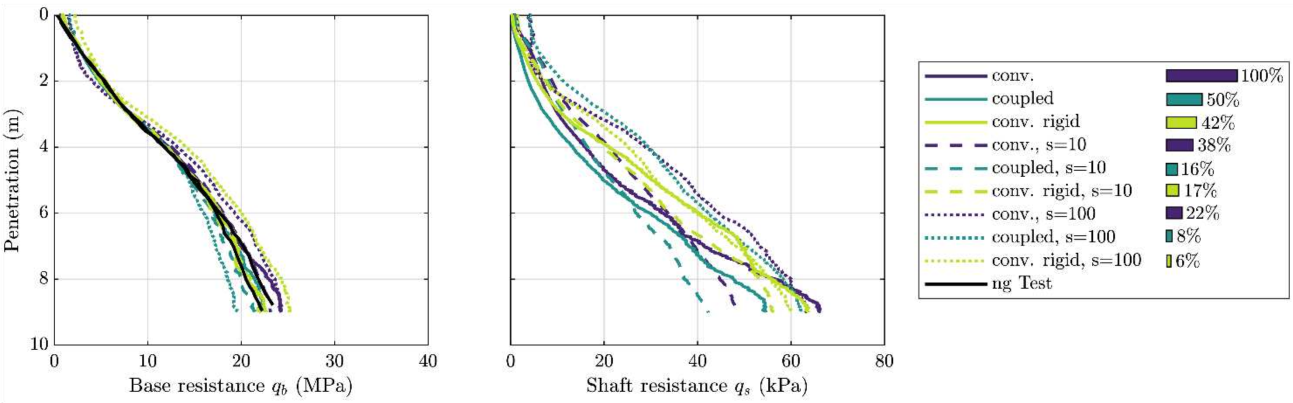

Figure 6. Comparison of simulated and experimental (a) pile base resistance q_b and (b) shaft friction q_s during pile penetration for various modelling approaches (conventional solid pile vs. coupled pile vs. rigid pile; with and without MGS at S = 10 and S = 100). The legend also reports the normalised computational time for each simulation variant.

4.1 Model validation against centrifuge tests

The numerical models were validated by comparing the predicted pile base resistance (q_b) and shaft friction (q_s) with penetration depth against the centrifuge benchmark. Figure 6 presents the comparison of simulated and experimental results.

The legend indicates the normalised computational time for each simulation variant. The results encompass nine modelling configurations: conventional solid pile, kinematically coupled pile, and rigid pile (for reference), each simulated without MGS ($S = 1$), with moderate scaling ($S = 10$), and with higher scaling ($S = 100$). Both the conventional volume element model and the coupled model, with and without MGS, generally exhibited good agreement with the centrifuge test data, effectively capturing the overall trend of increasing resistance with penetration depth.

Table 3. Mean percentage error (MPE) of simulation variants compared to centrifuge test data (Deeks and White, 2006).

Model Variant	MPE (%)
Conventional (no MGS)	3.92
Conventional, $S = 10$	4.75
Conventional, $S = 100$	14.10
Coupled (no MGS)	4.13
Coupled, $S = 10$	6.38
Coupled, $S = 100$	12.04

Table 3 quantifies the accuracy using the mean percentage error (MPE) relative to the centrifuge test data. The conventional model (without MGS) yielded an MPE of 3.92%, while the coupled model produced a comparable MPE of 4.13%. With MGS applied at a moderate factor ($S = 10$), the MPE increased marginally to 4.75% for the conventional model and to 6.38% for the coupled model. These values remain well within acceptable engineering limits for geotechnical simulations. However, higher scaling factors ($S = 100$) resulted in notably larger deviations (MPE > 12%), indicating a trade-off boundary between computational speed and physical accuracy. The results confirm that the proposed methods, particularly when utilising moderate scaling, can accurately replicate the physical centrifuge test results for pile installation.

4.2 Computational efficiency gains

Figure 6 illustrates the normalised computational time for each modelling technique. The coupled model alone significantly reduced simulation time by reducing the element count and increasing the stable time increment. The baseline (volume element pile, $S = 1$) runtime on the TUHH cluster was 7.4 hours using 8 CPU cores of an AMD EPYC 9124 node. Combining kinematic coupling with MGS produced the largest gains. The coupled model with $S = 10$ reduces runtime to approximately 16% of the baseline (≈ 6× speed-up), while $S = 100$ reduces it to 7.6% (≈ 13× speed-up). This substantial reduction enables large-scale simulations that would otherwise be computationally prohibitive.

4.3 Discussion of results

The numerical results demonstrate the effectiveness of the combined MGS and kinematic coupling techniques in enhancing the computational efficiency of quasi-static pile installation simulations while maintaining high accuracy. The coupled pile formulation reduces computational costs by assigning structural stiffness to efficient beam elements, while compliant volume elements manage the contact interface. This strategy effectively shifts the governance of the critical time increment (Δt_{crit}) from the stiff structural elements to the soil domain. Concurrently, the MGS technique further enhances efficiency by enabling larger stable time increments without altering the initial geostatic stress state.

The small difference between the conventional S = 1 and coupled S = 1 models (MPE of 3.92% vs. 4.13%) arises because the coupled model represents the pile using beam elements kinematically linked to a compliant contact layer, which slightly alters the local pile–soil interface compliance compared to a fully solid pile discretisation. Nevertheless, the overall resistance trends and MPE remain comparable, confirming that the coupling formulation does not appreciably alter the quasi-static response while substantially reducing computational cost.

The accuracy of the proposed model with a moderate scaling factor (S = 10) is comparable to that of the conventional model, with MPE values remaining below 7%. Notably, this configuration achieves computational runtimes comparable to modelling the pile as a rigid body, as the critical time increment is no longer dictated by the high stiffness of the pile elements. This indicates that the proposed techniques do not significantly compromise the accuracy of the simulation results while offering substantial computational efficiency gains that are essential for repeated or complex analyses within physical modelling studies. However, the observed increase in error with higher scaling factors (S = 100) suggests the introduction of non-negligible inertial effects from the artificially increased mass, which are not present in the quasi-static physical test.

While S = 100 produces larger deviations (MPE ≈ 12-14%), this scaling level may still be acceptable for preliminary design or parametric investigations aimed at identifying trends rather than high absolute accuracy. The rigid pile variant at S = 100 reaches 6% of the baseline (≈ 17× speed-up). These aggressive scaling factors offer valuable savings for exploratory analyses, provided that final verification employs lower scaling factors ($S \leq 10$).

Finally, it is important to emphasise the applicability limits. The MGS approach is strictly valid for quasi-static problems, such as pile jacking and slow driving, where inertial effects are negligible. For dynamic problems where inertial effects are significant (e.g., impact driving or earthquakes), the scaled mass in MGS can lead to an inaccurate dynamic response, distorting stress wave propagation and impact forces. Therefore, the application of MGS must be restricted to processes where static equilibrium is the dominant governing physics.

5 CONCLUSIONS AND OUTLOOK

This study validates a combined numerical strategy using Mass-Gravity-Scaling (MGS) and kinematic beam-/shell-volume coupling to substantially improve the computational efficiency of pile installation simulations within the Coupled Eulerian-Lagrangian (CEL) framework. The methods act complementarily: kinematic coupling provides an efficient structural representation of the slender deformable pile while maintaining a contact interface, and MGS enlarges the stable time increment of the explicit analysis by scaling density and gravity without altering the initial geostatic stress state.

When benchmarked against 50g centrifuge test data for jacked pile installation in sand (Deeks and White, 2006), the combined methodology reduced simulation time by approximately 6 times at a moderate scaling factor (S = 10) and up to 13 times at S = 100, compared to conventional volume element modelling. Mean percentage error (MPE) remained below 7% for key resistance metrics at S = 10, whereas higher scaling (S = 100) produced noticeably larger deviations, highlighting the practical need to limit scaling to preserve quasi-static response. This work provides a validated and practical tool for the detailed interpretation and numerical extension of physical model tests.

The demonstrated efficiency gains make extensive numerical investigations feasible, including broader parametric studies, the analysis of layered soils, and the use of more demanding constitutive models. The framework is equally applicable to other quasi-static penetration problems (e.g., spudcan penetration, anchor installation). Extending it to dynamic scenarios (e.g., impact pile driving) would require a fundamental reassessment of the MGS because its validity is contingent on the quasi-static assumption.

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