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VALIDATION OF 3D SEISMIC ANALYSIS FOR A SOIL-PILE-SUPERSTRUCTURE SYSTEM USING ADVANCED SOIL CONSTITUTIVE MODELS

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ABSTRACT

The simulation of the seismic response of liquefiable soils requires con-stitutive models that accurately incorporate undrained behavior in their formulations. This paper evaluates the seismic predictive capabilities of three advanced constitutive models: one based on boundary surface elasto-plasticity and two on hypoplasticity. In this context, we employ an improved hypoplastic model for undrained monotonic loading (Liao et al., 2024) combined with the intergranular strain concept (Niemunis and Herle, 1997). The modified hypoplastic model, which accounts for the hardening rate, addresses some shortcomings of the hypoplastic refer-ence model (von Wolffersdorff, 1996), improving its performance under seismic loading. To assess the practical applicability of these advanced constitutive models, a 3D finite element simulation of a soil-pile-superstructure system was conducted in ABAQUS. This system was modeled as a case study to validate the advanced models using centrifu-ge test data. The results show that the modifications to the hypoplastic model rectify its predictive capabilities in seismic analysis, leading to im-proved predictions of pore water pressure accumulation and a more accurate representation of the bending moment response in the embed-ded pile.

Keywords: soil constitutive model, seismic loading, pore water pressure, bending moment.

LITERATURE REVIEW FOR SPT AND DCPT CORRELATIONS

In soil-structure interaction, using an accurate soil model is essential for pre-dicting excess pore water pressure (EPWP) accumulation and structural response under seismic loading. Among advanced constitutive models, bounding surface plasticity and hypoplasticity are two widely studied frameworks. A well-known bounding surface model for simulating liquefiable soils is the Simple ANIsotropic SAND (SANISAND) plasticity proposed by Dafalias and Manzari (2004).

Petalas et al. (2020) extended this elastoplastic model by incorporating a fabric tensor into its formulation. Furthermore, Yang et al. (2022) proposed the memory surface and semifluidized state concepts to improve the predictive abilities of granular soils in pre- and post-liquefaction. The second framework under consideration in this study is hypoplasticity, which was developed originally by Kolymbas (1977). Since then, the model has been extended to improve its predictive capabilities for liquefiable soils (Von Wolffersdorff 1996). Niemunis and Herle (1997) proposed the Intergranular Strain (IGS) concept to extend the hypoplastic model to account for cyclic loading responses. The hypoplastic model proposed by Liao et al. (2024) represents an improvement over the Hypoplastic model, particularly in addressing the limitations of the latter with respect to monotonic undrained loading. In addition, in recent years, many researchers have proposed extended hypoplastic models to account for intergranular strain anisotropy (ISA) (Fuentes & Triantafyllidis 2015) and the semifluidized state (Liao et al 2022). The aim of this study is to investigate the predictive abilities of different advanced soil constitutive models for a boundary value application, a Soil-Pile-Superstructure Interaction (SPSI) system under seismic loading. Using the ABAQUS/Standard finite element program (Dassault Systèmes, 2020), a 3D numerical analysis is conducted to compare the simulated results with a centrifuge model test performed by Wilson (1998).

CONSTITUTIVE SOIL MODELS

In the numerical simulation, three advanced soil models were employed to predict Nevada sand behavior under seismic loading. The SANISAND

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model (Dafalias and Manzari 2004) is defined by fourteen independent parameters that govern elasticity, critical state behavior, yield surface, plastic modulus, dilatancy, and the fabric-dilatancy tensor (see Table 1). The hypoplastic model proposed by Von Wolffersdorff (1996) with the IGS concept (HP+IGS) is applied as the second model, while the third model is a new combination of a modified hypoplastic model that considers the hardening (H) effect, as presented by Liao et al. (2024), with the IGS concept (HP+IGS(H)). HP+IGS requires thirteen material parameters while the HP+IGS(H) introduces five additional input parameters to the original hypoplastic framework (Table 2). More details about the calibration of HP+IGS(H) are presented by Joneidi et al. (2025).

CENTRIFUGE MODEL TEST

The performance of these three advanced constitutive soil models in predicting the seismic response of SPSI is validated through the results of a centrifuge test conducted by Wilson (1998). The experimental setup included various dynamic excitation instruments, primarily strain gauge sensors and pore pressure sensors. In this study, the experimental results Csp3-J are used by applying the 1995 Kobe earthquake data with peak ground acceleration equal to 0.22g (Figure 1). To optimize computational efficiency, the significant duration (D5-95%) was used in the numerical analysis.

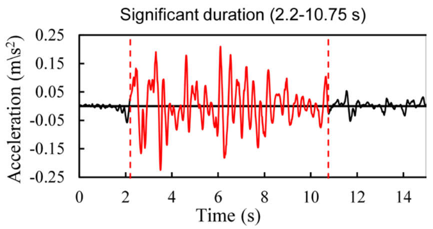

Figure 1 *Time history of the Kobe earth-quake*

Table 1 SANISAND constant parameters for Nevada Sand (from Joneidi et al.(2010))

	Index	Value [-]
Elasticity	G_0	200 [-]
	ν	0.05 [-]
Critical state	M_c	1.24 [-]
	M_e	0.71 [-]
	λ_c	0.027 [-]
	e_0	0.83 [-]
	ξ	0.45 [-]
Yield surface	m	0.02 [-]
Plastic modulus	h_0	9.70 [-]
	c_h	1.02 [-]
	n_b	2.56 [-]
Dilatancy	A_0	0.81 [-]
	n_d	1.05 [-]
Fabric-Dilatancy-tensor	z_{max}	5.00 [-]
	c_z	800 [-]

Table 2 Constant parameters applied in the HP+IGS and HP+IGS (H) models for Ne-vada Sand (from Joneidi et al.(2010))

Soil model	Index	HP+IGS	HP+IGS (H)
Von Wolffersdorff Hypoplasticity parameters	φ_c (°)	31°	31°
	h_s (MPa)	4000	4000
	n	0.30	0.30
	e_{c0}	0.887	0.887
	e_{d0}	0.511	0.511
	e_{i0}	1.15 e_{c0}	1.15 e_{c0}
	α	0.40	0.40
	β	1	1
IGS parameters	R	0.0001	0.0001
	m_R	5	5
	m_T	2	2
	β_r	0.20	0.20
	χ	3	3
Calibrated in this study	λ_1, λ_2	-	0.40, 2.5
Parameters of the modified model	e_{I0}	-	0.10
	k_I	-	12
	μ_0	-	1.30

FE MODEL AND SIMULATION PROCEDURE

The 3D FE model, shown in Figure 2, includes the soil layer dimensions, pile dimensions and depth, and element types. The aluminium pile is defined with a mass density of 2700 kg/m², Poisson's ratio of 0.33, and bending stiffness of 427 MN·m². Soil elements reach a maximum size of 1 m at depth, with finer 25 cm elements near the pile for accuracy. The pile is divided into 336 elements, with a minimum element size of 0.4 m. The 24.55 t superstructure is modeled as a lumped mass at the pile head. The analysis consists of three steps: geostatic (in situ stress), static general (pile activation), and dynamic implicit (seismic loading). Displacements at the bottom surface are fully constrained, while those on lateral boundaries are restricted perpendicular to the surface. To minimize boundary reflections, the two vertical surfaces perpendicular to the shaking direction are constrained using the Multiple Point Constraint (MPC) command to simulate laminar boundaries. The soil-pile interface is modeled with a surface-to-surface master-slave approach in both normal and tangential directions.

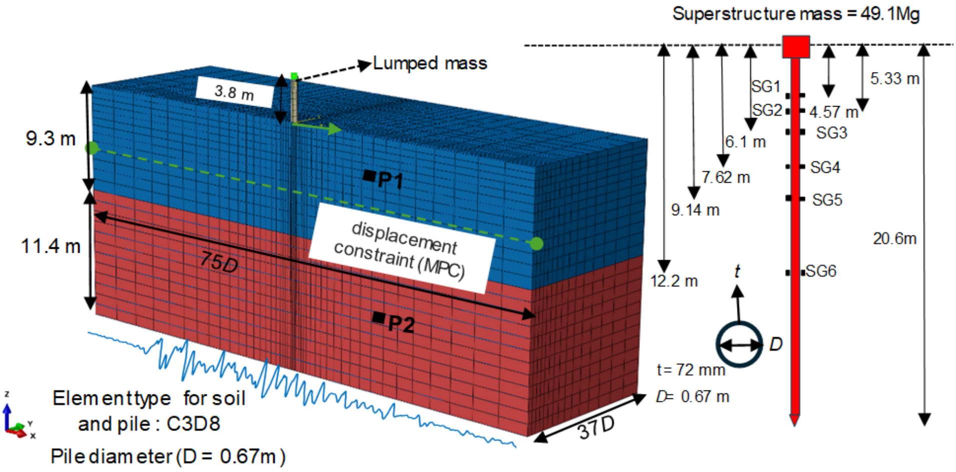

Figure 2 *3D Finite element model of SPSI and significant duration modelled in the finite element simulation*

EVALUATION OF PORE WATER PRESSURE

Figure 3 and Figure 4 show the experimental and simulated EPWP histories at different depths (P1 and P2). The earthquake-induced liquefaction potential is influenced by the initial effective vertical stress σ_v'. In Figure 3, test data show a gradual increase in EPWP to a moderate level, while SANISAND initially overpredicts but later stabilizes. HP+IGS shows strong fluctuations and a sharp initial rise, which differs from experiments and indicates liquefaction at this depth. In the dense layer (Figure 4), the centrifuge test shows a minor increase in EPWP, while SANISAND predicts a larger increase but remains below HP+IGS.

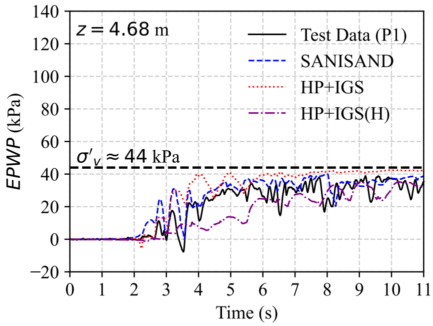

Figure 3 *Comparison of EPWP with centri-fuge test data at Mid-Loose level (P1 at z = 4.68 m)*

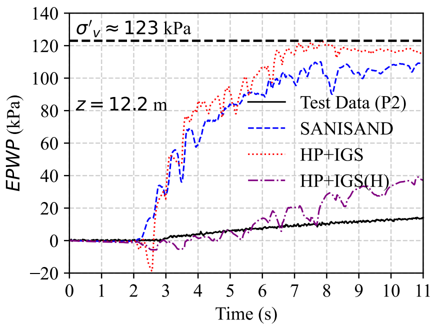

Figure 4 *Comparison of EPWP with centri-fuge test data at Mid-Dense level (P2 at z = 12.2 m)*

Compared to σ_v', HP+IGS overpredicts the liquefaction potential (Figure 3), while HP+IGS(H) provides better control, preventing excessive EPWP build-up. The improved capability of HP+IGS(H) to simulate the results of the experiments are due to modifications improving the control of mean effective stress fluctuations in cyclic undrained tests.

EVALUATION OF STRUCTURAL RESPONSE

Figure 5 shows the bending moment profile at various normalized depths during the Kobe earthquake (PGA = 0.22g), comparing centrifuge test data with SANISAND, HP+IGS, and HP+IGS(H) models. As shown in Figure 5 SANISAND and HP+IGS(H) capture the overall trend more accurately, while HP+IGS shows a larger deviation. SANISAND predicts higher bending moments in the upper region, which can be attributed to overprediction of EPWP in the soil depth. In contrast, HP+IGS overpredicts the bending moment at intermediate depths due to its rapid pore pressure generation in the early loading cycles. HP+IGS(H) shows good agreement with the experimental results, especially in controlling the moment increase at shallow depths. As it was observed in the EPWP results, HP+IGS(H) controls the accumulation of pore pressure in a more realistic way during the early loading stages, resulting in a more stable bending moment distribution. The HP+IGS model showed full liquefication, causing more stronger softening mechanisms and leading to a greater decrease in soil stiffness. It can be concluded that the lateral resistance decreases in the upper soil layers, which explains the lower bending moment in this region. Meanwhile, the stress-dilatation behavior of SANISAND maintains some resistance, delaying the onset of liquefaction and maintaining higher bending moments at shallower depths.

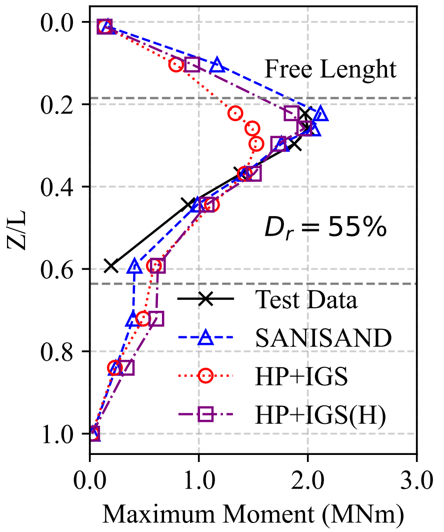

Figure 5 *Comparison of bending moment envelope with centrifuge test data*

CONCLUSION

The present study evaluates the predictive capabilities of different advanced soil constitutive models for numerical analyses as SPSI (soil-pile-superstructure interaction), under seismic loading. The finite element model has been validated with a centrifuge test previously outlined by Wilson (1998). The accumulation of pore water pressure and bending moment envelopes have been demonstrated to be the reliable indicators of the influence of soil models on the dynamic responses of SPSI. The dynamic analysis revealed the limitations of the hypoplastic model in predicting pore water pressure during the initial stage of cyclic loading. The findings showed that the HP+IGS model exhibited an overprediction of EPWP and demonstrated pronounced softening, resulting in full liquefaction and a reduction in lateral resistance. The SANISAND model, while overestimating EPWP, exhibits residual resistance due to stress-dilatancy, thereby maintaining higher bending moments. The combination of a modified hypoplastic model, as proposed by Liao et al. (2024) with the IGS concept, has been demonstrated to improve the control over EPWP and the variation in bending moments, resulting in good agreement with experimental data. It can be concluded that the modification of the hypoplastic model to enhance undrained monotonic behavior can be a suitable approach to improve the cyclic responses under seismic loading.

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