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MODEL CALIBRATION AND NONLINEAR SITE RESPONSE ANALYSIS FOR MEDIUM COMPACTED SATURATED VOLCANIC SAND

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Abstract

Site response analyses on saturated sand from Iceland have been performed with nonlinear (NL) and equivalent-linear (EL) methods. An advanced constitutive model was calibrated to simulate the nonlinear (NL) behavior of the sand by means of a monotonic Direct Simple Shear (DSS) test conducted in this study. The DSS test was conducted on dry sand under a constant volume conditions. In such a procedure, the change in vertical stress during shearing is assumed to be equal to the excess pore water pressure. In this study, the computed equivalent blow count ($N_{1,60}$) is limited to the top four meters, and a relative density of 60% has been considered. An average shear wave velocity (V_s) profile has been measured through the Multichannel Analysis of Surface Waves (MASW) technique. The calibrated model is employed in the nonlinear site response analysis, and the results are compared with those of the equivalent linear method. The EL and NL analyses in DEEPSOIL and PLAXIS, respectively, predicted liquefaction at the site and aligned with surface evidence and prior researches.

Keywords: Liquefaction, Site response analysis, DSS, Non-linear analysis, Constitutive model, Equivalent-linear analysis, Python

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1 INTRODUCTION

The seismic activity in Iceland is concentrated in two complex fracture zones, namely the South Iceland Seismic Zone (SISZ) and the Tjörnes Fracture Zone (TFZ), which are associated with the Mid-Atlantic plate boundary. Over the past three centuries, about 30 earthquakes with a magnitude greater than six have been recorded in these regions [1]. In the SISZ, destructive earthquakes often occur in sequences.

Iceland has the highest seismic hazard in Northern Europe, comparable to that in Southern Europe. The two main seismic zones in the country have a Peak Ground Acceleration (PGA) of 0.5g with a mean return period of 475 years. Since 2000, three destructive earthquakes have struck the largest agricultural region in South Iceland, with earthquakes with moment magnitudes (M_w) of 6.5 and 6.4 occurring in 2000 and one M_w 6.3 earthquake in 2008. These events caused significant damage to the area's buildings, farms, power plants, bridges, and other infrastructure. However, no serious injuries or fatalities occurred due to these events [2].

The soil in Iceland is mainly composed of Holocene soils that are typically normally consolidated and have basaltic origins. Additionally, postglacial sediments are present in a large portion of the island and are considered geologically young and rapidly built up. This results in local soils typically consisting of coarse silty particles, including coarser grains, often loosely compacted [3, 4].

Loose, partially, or fully saturated soils may lose strength substantially during earthquake shaking or sudden changes in stress conditions, a phenomenon referred to as liquefaction. The effects of liquefaction can vary, including sudden or gradual lateral deformations, ground oscillations, vertical settlements, and the development of sand boils, depending on soil and site characteristics [5]. Structures resting on or buried in liquefied soil are at risk of severe damage. Hence, assessing soil site liquefaction hazards is crucial in seismically active regions.

The study aims to evaluate liquefaction at the Arnarbaeli site by the Ölfus river in south Iceland by comparing site response analysis results from DEEPSOIL's equivalent linear (EL) method and PLAXIS' nonlinear (NL) simulation. It also seeks to calibrate the NL model by minimizing differences between numerical and experimental shear strength values. Hence, the study determines if both methods accurately predicted liquefaction and if the results aligned with surface evidence and previous studies.

2 SOIL CHARACTERISTICS AT THE SITE

Less than 10 km from the epicenter of the M_w 6.3 Ölfus earthquake in 2008, liquefaction was observed at the Arnarbaeli site, which is situated on the banks of the estuary of the Ölfus river and consists of a thick layer of silty sand. The earthquake had a PGA of 0.88g, affecting approximately 5000 low-rise residential buildings [6]. The occurrence of liquefaction can be attributed to the site's soil characteristics, combined with the strong shaking caused by the earthquake. The event highlights the potential hazards in SISZ of earthquake-induced liquefaction in areas with similar soil conditions and seismic activity. The most notable surface indications of liquefaction, including ground settlements and sand boil, were observed at the Arnarbaeli site. Arnarbaeli is comprised of volcanic sand deposits that are relatively uniform. Table 1 and 2 summarize the tested sand's geotechnical characteristics, the soil layers' shear wave velocity (V_s), and the equivalent blow count ($N_{1,60}$) for down to 100 m depth. This information was obtained from various sources [7–10]. The SPT equivalent values up to a depth of 4 m were determined by Green et al. [7] using DCP field test results. However, for the depth range between 4 and 100 m, this study extrapolated the $N_{1,60}$ values.

Table 1 Physical properties of the tested sand [7]. Soil properties represented in the table include specific gravity (G_s), minimum and maximum dry unit weight ($\gamma_{d,min}$, $\gamma_{d,max}$), saturated unit weight (γ_{sat}), and uniformity coefficient (C_u).

Soil	Classification symbol	G_s [-]	$\gamma_{d,min}$ [g/cm^3]	$\gamma_{d,max}$ [g/cm^3]	γ_{sat} [g/cm^3]	C_u [-]	*Fines content* [%]
Arnarbaeli sand	SW-SM	2.84	11.6	16.4	18.2	9	7

Table 2 Soil layers properties, [7–10].

Layer no	Thickness [m]	V_s [m/s]	$N_{1,60}$ [-]
1	2	75	3.0
2	3	115	10.6
3	5	230	32.1
4	10	330	94.8
5	20	420	321.1
6	30	480	1177.4
7	30	530	3470.6

3 UNDRAINED MONOTONIC DSS TEST

This study's sample was taken from a shallow depth (0.5 meters) at the Arnarbaeli site. The results of the monotonic direct simple shear (DSS) test are presented in Figure 1, which includes the shear stress (τ) versus shear strain (γ) on the left and the pore-water pressure (PWP) buildup against γ, on the right side of the picture. The figure provides essential information on the behavior of the sand under constant volume conditions. The DSS test was performed under a vertical stress level of 100 kPa and a relative density of (D_r) 60%.

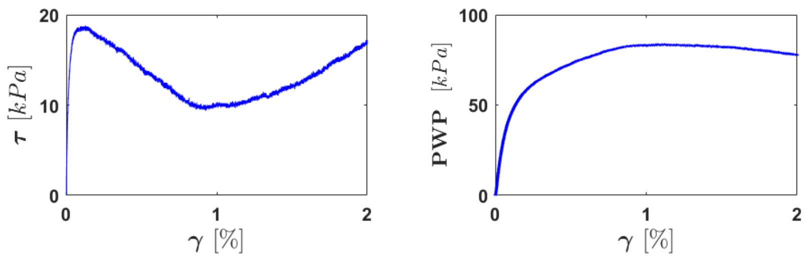

Figure 1 Shear stress and PWP against shear strain. Monotonic direct simple shear test performed under constant volume conditions.

Jaky's equation was used to determine the coefficient of lateral earth pressure at rest (K_0) [11]. Table 3 summarizes the test specifications of the monotonic DSS test conducted in this study.

Table 3 Summary of monotonic DSS test.

Test	Loading type	Void ratio	K_0	D_r [%]	Vertical pressure [kPa]	Friction angle [degree]
DSS - 60 - 100	Monotonic	1.08	0.425	60	100	31.3

4 SITE RESPONSE ANALYSIS

In evaluating the soil effects on ground motion, three main approaches are typically used: the attenuation relationship approach, the soil coefficient approach, and the site response analysis [12]. However, the first two approaches are limited in their applicability since they oversimplify the actual soil distribution. Site response analysis offers more detailed results, such as the variation of seismic waves in terms of amplitude, duration, and frequency at different depths of the soil deposit.

In this study, the liquefaction potential of the Arnarbaeli site was evaluated using one-dimensional site response analysis with two different methods and software. The DEEPSOIL software was used for the EL approximation, while PLAXIS was used for the NL approach.

4.1 Equivalent linear analysis

The EL model iteratively selects shear modulus and damping ratio based on soil parameters, Table 1 and 2. The model uses the Pressure-Dependent Modified Kondner-Zelasko (MKZ) soil model and a Non-Masing Re/Unloading hysteretic loading and unloading formulation. The Darendeli reference curve and reduction factor [13, 14] have been utilized to calibrate the tested sand. Figure 2 illustrates the shear modulus reduction, material damping, and shear strength curves based on Darendeli's empirical formulas [13]. The 'Current' term reflects the behavior of a soil layer used in the analysis, adjusted with the 'Fit Curve' to match field measurements and observations. The 'Fit Curve' adjusts the soil layer's properties to align with the field measurements, while the 'Reference Curve' serves as a basis for defining the properties of the layered soil profile.

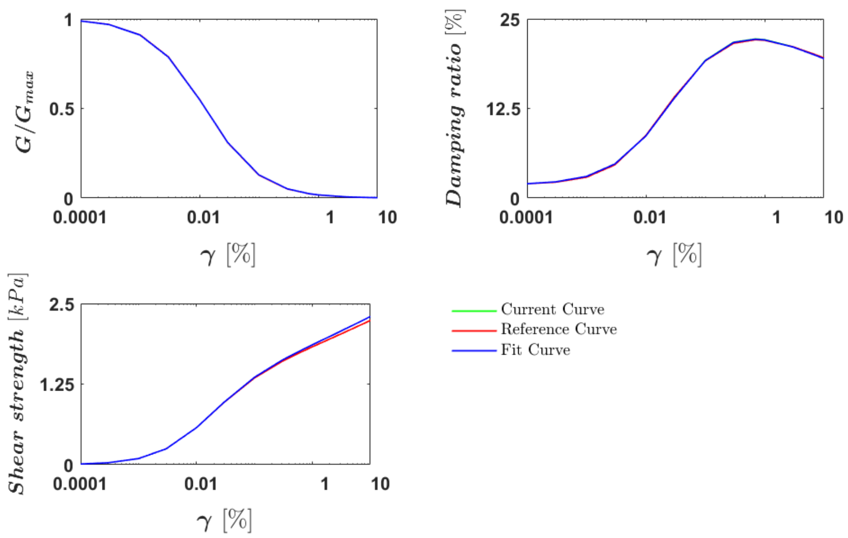

Figure 2 Plot generated for the first soil layer illustrating the iteration procedure towards a strain compatible normalized shear modulus (G/G_{max}), material damping, and shear strength curves based on DEEPSOIL [14].

Green et al. [7] determined the SPT equivalent values for a depth down to 4 m using DCP field test results. In this study, $N_{1,60}$ values were extrapolated to a depth of 100 m. The cyclic resistance ratio (*CRR*) for a M_w 6.3 event over the same depths was then estimated using the correlation proposed by Youd et al. [15]. The cyclic stress ratio (*CSR*), which represents the cyclic loading level on the soil caused by an earthquake, was determined using EL analysis.

4.2 Calibration of UBC3D model

The UBC3D-PLM model is an advanced method for simulating liquefaction behavior in dynamic situations [16]. This model utilizes two yield surfaces for a smooth transition near the mobilized friction angle and employs the Mohr-Coulomb yield condition with a hardening law similar to the Hardening Soil model. The input parameters for this model are based on commonly used tests such as drained triaxial tests or standard penetration tests. The UBC3D-PLM model is beneficial for undrained dynamic calculations where the evolution of excess pore pressures in sandy soils needs to be accurately modeled. The parameters of the model [16] and their calibrated values are summarized in Table 4.

The manual [16] suggests a set of equations for determining the model parameters based on the normalized N_{SPT}. In this study, the suggested equations for obtaining the initial values of the soil model's parameters were found to be unsuitable due to several limitations. The lack of field data and other restrictions associated with the suggested equations posed barriers to this approach. Therefore, to ensure accurate modeling, a calibration process was undertaken. In this calibration, a wide range of values for each variable of the model was considered. By taking these steps, it was possible to develop a more accurate and reliable model for the soil.

The parameters in the list (Table 4) are calibrated through trial and error with a script that runs the PLAXIS DSS simulator using Python. This iterative process continues until the difference between the numerically computed and experimentally obtained shear strength is less than 10%.

Table 4 Parameters of the UBC3D-PLM model with their corresponding calibrated values.

Function	Symbol	Definition	Calibrated value	Unit
Stiffness Parameters	k^{*e}_B	Elastic bulk modulus factor	1150	[-]
	k^{*e}_G	Elastic shear modulus factor	350	[-]
	k^{*p}_G	Plastic shear modulus factor	150	[-]
	me	Rate of stress-dependency of elastic bulk modulus	0.35	[-]
	ne	Rate of stress-dependency of elastic bulk modulus	0.45	[-]
	np	Rate of stress-dependency of plastic shear modulus	0.45	[-]
	p_{ref}	Reference pressure	101.32*	[kN/m^2]
Strength Parameters	φ_{cv}	Constant volume friction angle	31.3*	[°]
	φ_p	Peak friction angle	32.5*	[°]
	C	Cohesion	0*	[kN/m^2]
Advanced Parameters	R_f	Failure ratio	0.9	[-]
	$N_{1,60}$	Corrected SPT value	12	[-]
	f_{dens}	Densification factor	1.0*	[-]
	f_{Epost}	Post-liquefaction factor	1.0*	[-]

* These parameters were kept constant during the calibration process.

The UBC3D-PLM model is capable of reproducing the buildup of PWP. In this study, the excess pore pressure ratio (R_u) was used to differentiate between liquefied and non-liquefied soil layers. R_u represents the normalized pore water pressure to initial effective vertical stress. A zone is considered to be liquefied if it has a maximum R_u greater than 0.7. A R_u value of 1.0 indicates that the corresponding layer is in a completely liquefied state [12].

4.3 Performance of the model

The UBC3D-PLM model is used to perform a numerical reproduction of an undrained monotonic DSS test. The Soil Test Tool from the finite element program PLAXIS is employed for the numerical simulation of the test. Figure 3 illustrates the performance of the UBC3D-PLM model.

5 RESULTS AND DISCUSSION

In this study, site response analyses were used to predict liquefaction occurrence at the Arnarbaeli site. The EL site response analysis in DEEPSOIL and the NL simulation in PLAXIS were compared. The NL model was calibrated using a trial and error iterative code in Python to reduce the differences between the shear strength of the soil in both the numerical (UBC3D-PLM constitutive soil model) and experimental (DSS) test. The differences were kept below 10%. The trends of pore water generation and shear stress over the DSS test are similar, as shown in Figure 3. While the study attempted to keep the simulated and experimental shear strength values close, comparing pore water generation between the two methods demonstrates a better match.

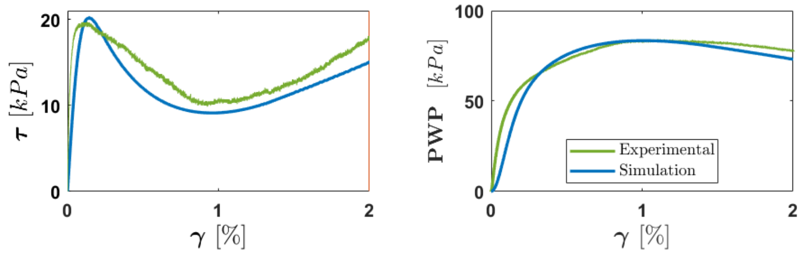

Figure 3 Comparison between experimental results and the UBC3D-PLM model's simulation of a monotonic DSS test with D_r = 60% and vertical pressure of 100 kPa by PLAXIS SoilTest simulator tool; (left) shear stress and (right) pore water pressure generation.

Figure 4 shows a liquefaction resistance curve ($N_{1,60}$ - CRR) scaled to the M_w 6.3 Ölfus earthquake.

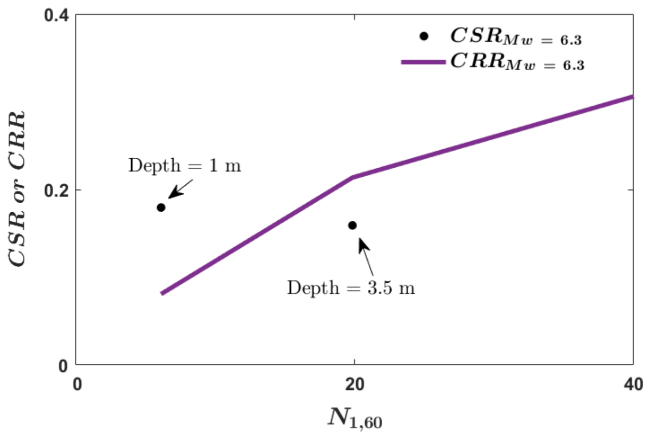

Figure 4 Graph displaying liquefaction evaluation results for the Arnarbaeli soil obtained through EL site response analysis.

Figure 4, 5, and 6 show that the first soil layer (with a thickness of 2 m) underwent complete liquefaction, while the NL analysis showed liquefaction also occurring in deeper layers. In other words, the second and third layers were also liquefied, as indicated by R_u values greater than or equal to 0.9 (Figure 6).

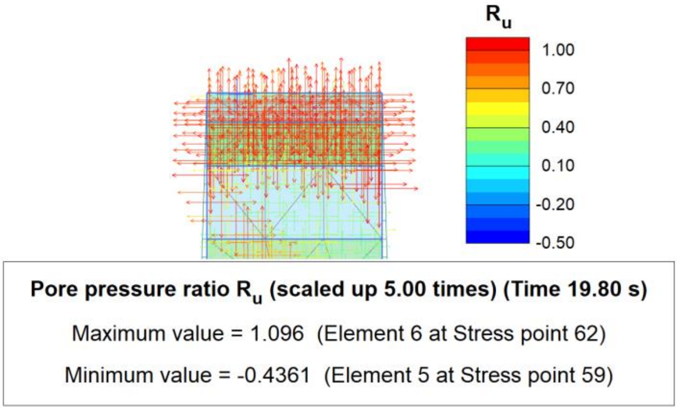

Figure 5 R_u distribution obtained with PLAXIS for the top 3 soil layers. The red arrows represent R_u values greater than 0.8.

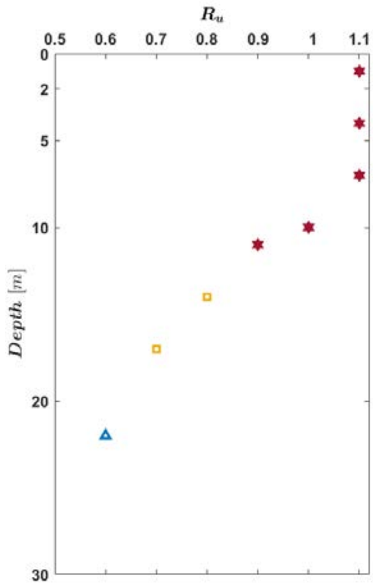

Figure 6 PLAXIS-derived R_u values plotted against depth for the top five layers. Red pentagrams indicate R_u values greater than or equal to 0.9, orange diamonds represent R_u values between 0.7 and 0.8, and blue triangles represent non-liquefied layers with $R_u < 0.7$.

6 SUMMARY AND CONCLUSIONS

In conclusion, this study successfully predicted liquefaction occurrence at the Arnarbaeli site. Both the EL site response analysis in DEEPSOIL and the NL simulation in PLAXIS predicted liquefaction and were consistent with surface evidence of liquefaction at the site and in previous studies. The differences in shear strength between the numerical and experimental tests were kept below 10% through Python calibration of the NL model. The advanced constitutive model provided more detailed results than the equivalent linear analysis and *CSR* vs. $N_{1,60}$ graph. The NL analysis indicated liquefaction of the top three soil layers down to a depth of 10 meters. In contrast, the EL analysis showed liquefaction occurring only in the first layer.

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