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Effective stress-based probabilistic nonlinear ground response analysis of Newtown suburb, Kolkata, India

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Surface topography and geology influence the response of structures to seismic excitations, particularly for shallow crustal earthquakes. Comprehensive ground response analysis (GRA) helps characterize the seismic hazard at any site by quantifying the effect of local soil conditions on altering the bedrock motion. Nonlinear ground response analysis (NLGRA) effectively models the complex stress-strain conditions induced due to seismic loading, compared to linear and equivalent linear approaches. Particularly, when assessing the effect of liquefaction on the site's surface response, the effective stress-based NLGRA is required to capture the variation in peak ground acceleration (PGA) from the liquefied layer to the ground surface. The performance of NLGRA depends on the input parameters used in the analysis, and variations in soil properties in spatial directions must be properly accounted for to evaluate the realistic response of the ground. In the present study, a two-step procedure is employed to assess the nonlinear ground response of the Newtown suburb in Kolkata, India. Initially, a one-dimensional total stress-based nonlinear ground response analysis (TSNLRA) is performed using the DEEPSOIL program, considering uncertainties in soil density, shear wave velocity, shear strength, and dynamic properties within a probabilistic framework. For lower discrepancy and faster convergence, the uncertainties in the input parameters are modelled using the quasi-Monte Carlo simulation (QMCS) technique. The results of TSNLRA are used to determine the zone within the subsoil profile that yields responses sensitive to variations in soil parameters. Furthermore, a sensitivity analysis using the Sobol sensitivity indices is conducted to identify the most influential parameter governing the surface PGA. Based on the observations from the TSNLRA, an effective stress-based nonlinear finite element analysis (NLFEA) is performed using PM4Sand, by considering uncertainties in the most influential parameter within the sensitive zone established from the TSNLRA. The variations of PGA across the profiles of the study area are captured and realistic due to the consideration of uncertainties in the input parameters. The response characteristics obtained from the probabilistic GRA will enable practicing engineers to establish the probability-based response spectra for seismic response analysis of structures and to make risk-informed decisions in seismically active regions.

Keywords: Nonlinear ground response, liquefaction, uncertainties, quasi-Monte Carlo simulation, PM4Sand, PGA

1 INTRODUCTION

Seismic-induced liquefaction can significantly affect saturated, loose to medium-dense, cohesionless soils, resulting in a reduction of shear strength and stiffness of the soil due to the buildup of excess porewater pressure (EPP) during cyclic loading. The loss in shear strength can result in various geotechnical hazards, including large settlements, lateral spreading, slope instability, and failures of foundations, embankments, and lifeline systems. Accurate prediction of liquefaction triggering and its consequences requires approaches that can model the nonlinear behavior of the soil and the generation and dissipation of pore pressure during shaking. Effective stress-based nonlinear ground response analysis (ESNLRA) can be advantageously used by integrating nonlinear constitutive models with coupled stress-strain and pore pressure responses. This enables the simulation of realistic stiffness degradation, cyclic mobility, and deformations under earthquake loading.

Deterministic or equivalent-linear analyses are insufficient for large-scale hazard studies, such as seismic microzonation, which aims to divide a region into zones of varying seismic hazard, considering both amplification and potential ground failure. These methods cannot capture the combined effects of aleatory variability in earthquake source characteristics (such as magnitude, rupture distance, and spectral content) and epistemic uncertainty in subsurface conditions (including shear wave velocity, density, plasticity, and cyclic resistance). Probabilistic nonlinear ground response analysis (PNLRA) addresses these limitations by systematically sampling soil parameters within their plausible ranges and subjecting site models to a range of hazard-consistent ground motions representing all possible earthquake scenarios. The outputs, including median and percentile-based surface response spectra, probabilistic site amplification factors, and liquefaction probability curves, are statistically robust and directly applicable for risk-informed urban planning, performance-based seismic design, and the development of targeted liquefaction mitigation strategies. Combining the effective stress models with the probabilistic frameworks provides a comprehensive approach for seismic microzonation in regions susceptible to liquefaction.

However, instead of directly performing the computationally expensive probabilistic ESNLRA for the entire soil profile, carrying out an initial screening using TSNLRA is advantageous. This preliminary step enables the evaluation of variability in response characteristics across multiple realizations of the input uncertainties. Therefore, in the current study, a hybrid approach is adopted. The interpretations from the TSNLRA are used to perform the ESNLRA. The depths exhibiting higher scatter in the responses from TSNLRA are identified as critical zones, mainly influenced by soil behavior and seismic loading. Subsequently, a probabilistic ESNLRA is performed for these depths using an advanced constitutive model such as PM4Sand. This approach reduces the computational effort and improves the accuracy of the liquefaction analysis.

2 STUDY AREA AND QUANTIFICATION OF PARAMETER UNCERTAINTY

The study area, Newtown, located in the vicinity to the northeast of Kolkata city, is a major tech hub and a growing commercial and residential landscape. Newtown is situated on the Eocene Hinge Zone (EHZ) of the Bengal Basin, which makes it vulnerable to seismic risk in the event of earthquakes. Furthermore, the presence of shallow Holocene deposits, comprising alluvial soils with a predominant composition of silts and silty sands, makes the city prone to seismic wave amplification and susceptible to liquefaction (Nath et al. 2014). Due to the rapid urbanization and land use practices, the study area has been extensively characterized using multiple field tests, including standard penetration tests (SPTs), cone penetration tests (CPTs), and Dilatometer Marchetti Tests (DMTs). In the present study, data from 64 boreholes (Figure 1) spanning an area of 35 km² have been used to characterize the study area. The subsoil profile primarily comprises surficial silt underlain by thick deposits of silty sands with consistency varying from loose to very dense at a depth of 28 m- 30 m. Based on the variation in SPT resistance values (N) along the depth, the idealized soil profile for use in one-dimensional (1-D) GRA is obtained, as shown in Figure 2.

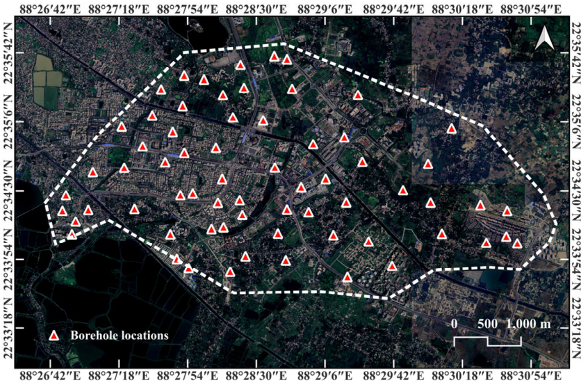

Figure 1 Locations of the boreholes used for subsurface characterization of the study area.

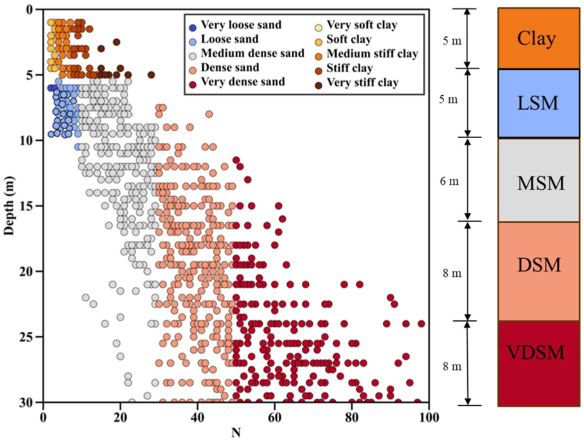

Figure 2 Variation in the SPT N values along the depth and the idealized profile used in 1D GRA.

The borehole data is analyzed to obtain the statistical parameters (mean and standard deviation) and the corresponding probability density functions (pdf) of the input parameters, such as SPT-N, unit weight (γ), plasticity index (PI), angle of internal friction (ϕ), shear strength (S_u), and groundwater table (GWT). Due to the non-availability of the measured shear wave velocity (V_s) values, N-based empirical correlations (Ohsaki and Iwasaki 1973; Imai and Tonouchi 1982; Iyisan 1996; Hasancebi and Ulusay 2007; Dikmen 2009) are used for determining the V_s. All input parameters are modeled as normal random variables, as obtained from the pdf. The uncertainty in N values is assumed to reflect the randomness in V_s. The input parameter uncertainty is propagated through a quasi-Monte Carlo simulation (QMCS) using low-discrepancy Sobol sampling (Liu and Cheng 2018). The random realizations of the soil properties were sampled from the defined pdf, and TSNLRA is performed initially. The statistical parameters of the input variables obtained from the borehole data are presented in Table 1.

Table 1 Statistical parameters of the subsoil layers used in QMCS

Property	Min	Max	Mean	CoV
Clay				
c_u (kPa)	12	83	31.2	0.570
PI (%)	8	52.0	21.23	0.530
γ (kN/m^3)	15.54	20.72	18.08	0.080
V_s (m/s)	110	193	138.6	0.163
LSM				
ϕ (°)	20	30	28	0.100
γ (kN/m^3)	14.47	21.29	17.92	0.067
V_s (m/s)	125	206	173.7	0.137
MSM				
ϕ (°)	22	32	30	0.100
γ (kN/m^3)	14.55	21.34	17.98	0.116
V_s (m/s)	166	316	231.6	0.124
DSM				
ϕ (°)	25	36	34	0.090
γ (kN/m^3)	14.52	21.32	17.96	0.122
V_s (m/s)	195	370	302.3	0.140
VDSM				
ϕ (°)	31	39	36	0.041
γ (kN/m^3)	14.52	21.32	17.96	0.122
V_s (m/s)	217	453	364.4	0.165
Depth of GWT (m)	1.0	10.5	4.9	0.461

LSM: loose silty sand; *MSM*: Medium dense silty sand; DSM: dense silty sand; VDSM: very dense silty sand

3 TOTAL STRESS BASED NONLINEAR GROUND RESPONSE ANALYSIS

The TSNLRA is performed in DEEPSOIL using the GQ/H constitutive model (Groholski et al. 2016). Normalized shear modulus reduction and damping ratio curves were generated using the empirical correlations of Zhang et al. (2005) as functions of plasticity index. The generated curves were implemented in DEEPSOIL as model input. Further, a shear strength correction, as adopted by Mandal et al. (2025a, b), was applied to the backbone curve to ensure that the shear strength calculated by the model (implied shear strength) is equal to the measured or estimated soil shear strength (target shear strength). The analysis is performed using eleven IS 1893 Part 1 (2016) spectrum-compatible ground motions (GM1-GM11) with peak ground accelerations (PGAs) ranging from 0.16 to 0.24g (Figure 3). The procedure outlined by Mandal et al. (2025b) has been adopted to obtain spectrum-compatible ground motions. The variation of PGA along the depth across the simulations for GM3 and the mean PGA for all eleven ground motions are depicted in Figure 3.

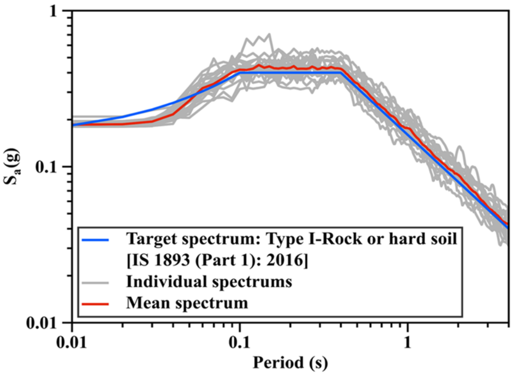

Figure 3 IS 1893 Part I: (2016) spectrum compatible ground motions.

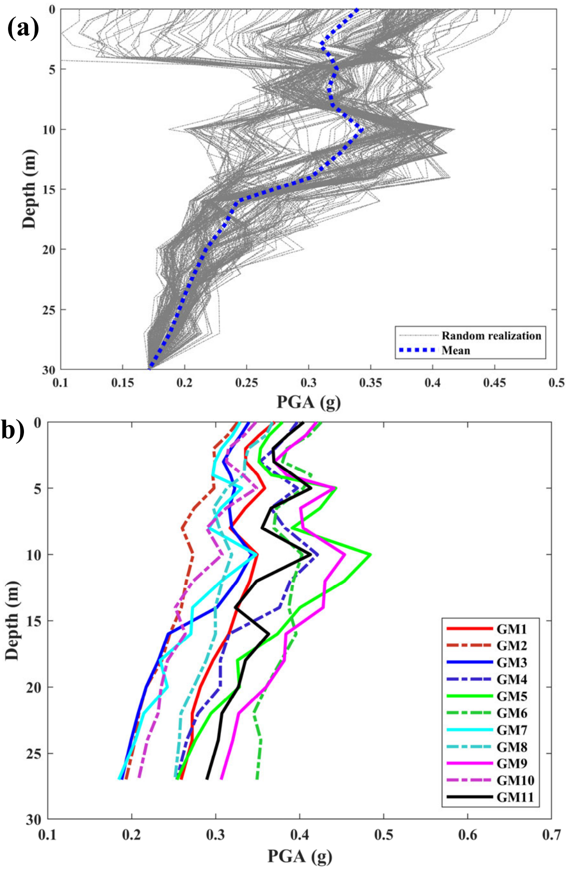

Figure 4 (a) Variation of PGA along depth across simulations for one ground motion (GM3), and (b) variation of mean PGA for the eleven ground motions

As indicated in Figure 4a, a large scatter in the PGA values is observed between the depths 5 m and 15 m, highlighting the influence of variability in the soil properties of this region on the corresponding PGA and eventually surface PGA. However, below 15 m, the scatter is narrow, and the PGA values converge, indicating that the lower influence of variability in soil properties is evident below 15 m. Similar observations are noticed across all the ground motions (Figure 4b), and therefore, the zone between the depths 5 and 15 m is found to be the most sensitive and influential layer in the profile.

Furthermore, a sensitivity analysis was conducted to identify the soil parameter whose variability has the most significant effect on the response characteristics. The relative influence of input parameters, such as γ, GWT, V_{s30}, and S_u, on the predicted surface PGA in the five soil layers (s1-s5) is quantified using Sobol sensitivity indices. The Sobol method separates the output variance into contributions from individual parameters and their interactions, allowing for an assessment of first-order or main effect and total effect sensitivities. The results of the sensitivity analysis are presented in Figure 5.

The first-order Sobol sensitivity analysis yielded a higher value for V_{s30}, signifying that V_{s30} is the most influential parameter on the surface PGA. Parameters like γ and S_u of the VDSM layer are perceived to have a moderate influence, while s1-s4 have no significant direct effect on the surface PGA. The total order Sobol index also indicated V_{s30} as the influential parameter. This signifies the interaction effects of V_{s30} with other parameters in governing the surface PGA. However, no substantial variations in the first and total order indices are observed for the other parameters, suggesting the limited interaction effects, and that the variation in PGA is primarily due to the direct effect of the parameters. Based on the former observations, it can be concluded that V_{s30} (subsequently referred to as N) is the parameter that influences the surface PGA. Therefore, the ESNLRA is performed by considering the uncertainty in the SPT-N value within the sensitive zone established from TSNLRA. The other input parameters are assumed to be equal to their respective mean values obtained from the bore log data. Although the GWT has only a minor influence on the surface PGA, understanding the influence of GWT location on the porewater pressure, particularly for liquefaction analysis, the GWT, besides SPT-N, is modelled as a random variable in the ESNLRA.

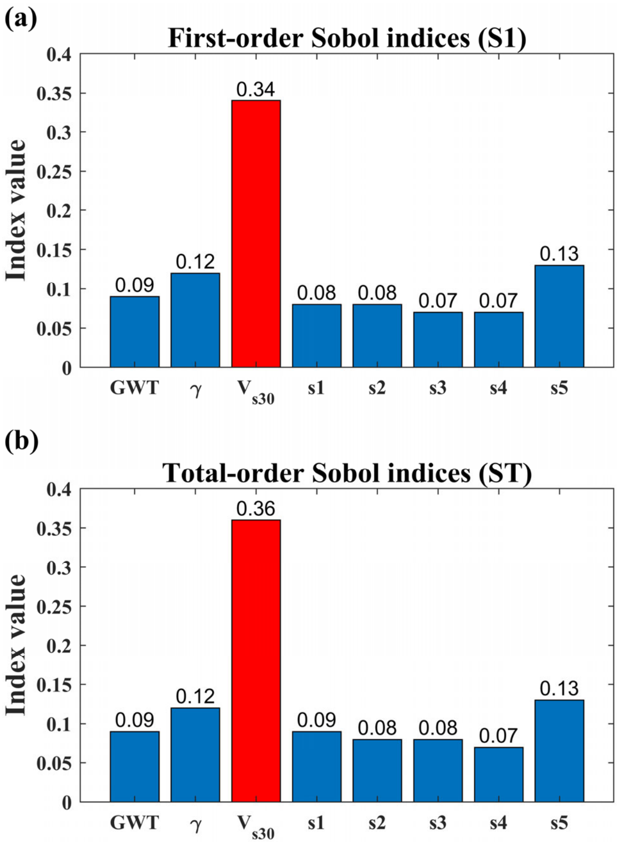

Figure 5 Results of Sobol sensitivity analysis: (a) First-order Sobol indices, and (b) total-order Sobol indices.

4 EFFECTIVE STRESS NONLINEAR GROUND RESPONSE ANALYSIS

Based on the results of the TSNLRA, the soil layer between 5 and 20 m in depth is identified as the zone exhibiting the largest variability and sensitivity in seismic response. Furthermore, the sensitivity analysis reveals that the surface response is significantly influenced by V_s (N) compared to other input parameters. Therefore, to capture the EPP generation and dissipation in the sensitive zone, an ESNLRA is performed considering uncertainty in SPT-N values. One-dimensional (1D) GRA for the idealized profile, consisting of five layers (surficial silt and four sand layers), was carried out using an advanced constitutive model implemented in FEM within PLAXIS 2D. The surficial silt and the sand layers are modelled using the Mohr-Coulomb and PM4Sand models, respectively.

4.1 *PM4Sand constitutive model*

PM4Sand is a stress-ratio-controlled, critical-state-based plasticity model developed by Boulanger and Ziotopoulou (2017) for simulating the stress-strain and pore pressure responses of fully saturated granular soils under monotonic or cyclic stress conditions. It is formulated within the framework of bounding surface plasticity and effectively captures the liquefaction-associated phenomenon, such as cyclic mobility, dilation, and post-liquefaction strain accumulation. The model consists of three primary parameters, namely apparent relative density (D_{ro}), shear modulus coefficient (G_o), and contraction rate parameter (h_{po}), as well as ten secondary parameters. The model facilitates calibration against cyclic direct simple shear test (CDSS) data, cyclic triaxial test data, or cyclic data from in-situ test-based weighting curves, enabling the practical application of the model.

4.2 *Calibration of the constitutive model*

The model parameters are calibrated using the soil element test facility provided in PLAXIS. The cyclic data determined from the liquefaction triggering relationship by Boulanger and Idriss (2016) is used with the CDSS setup in the PLAXIS Soil test facility for calibrating the model. The corrected SPT resistance $(N_1)_{60}$ of the critical layer is used to estimate the cyclic resistance ratio (CRR) corresponding to an earthquake magnitude (M_w) of 7.5. The $CRR_{7.5}$ is adjusted to the required overburden and magnitude conditions to obtain the target CRR. The equivalent number of cycles (N_c) of uniform loading required to produce liquefaction of soil in the laboratory under the considered earthquake scenario is calculated using the correlation between magnitude scaling factor (MSF) and N_C proposed by Boulanger and Idriss (2016) as

$$MSF = \left(\frac{N_{M_w=7.5}}{N_c}\right)^c \tag{1}$$

where $N_{M_w=7.5}$ is 15 and parameter c is 0.337 for sand-like soils. The primary parameters of the PM4Sand model (D_{ro} and G_o) are calculated using Equations 2 and 3.

$$D_{ro} = \sqrt{\frac{(N_1)_{60}}{C_d}} \tag{2}$$

$$G_0 = 167\sqrt{(N_1)_{60} + 2.5} \tag{3}$$

where C_d is a constant and is 46 for clean sands (Idriss and Boulanger, 2008). All secondary parameters were assigned the default values recommended by the model developers, and the h_{po} was iteratively adjusted to achieve the required liquefaction triggering criterion (LTC). In the present study, the single amplitude shear strain exceeding 3% (Boulanger and Ziotopoulou, 2017) is considered as the LRC for model calibration. The LRC obtained from the calibration is compared against the target CRR-N_c curve, and the calibrated parameters are adopted for the site-specific ESNLRA. Since the probabilistic ESNLRA involves multiple realizations of the input parameters of the critical layer, the model is recalibrated in each simulation using the corresponding set of input parameters.

4.3 *FE modelling and probabilistic GRA*

The subsoil strata in the study area are modelled as a 1D column with horizontal soil layers extending vertically up to 30 m. The model is discretized using 15-node triangular elements. The width of the column and size of the elements are determined based on the criteria proposed by Kuhlemeyer and Lysmer (1973). The lateral boundaries are modelled as tied degrees of freedom to ensure the zero lateral strain condition, while the base of the soil column is modelled as a compliant base to avoid wave reflections. The input ground motion is applied at the base of the model.

Probabilistic GRA is performed using QMCS by sampling from the pdf of the SPT-N values obtained from the borehole data. In total, 1000 realizations are sampled, and the ESNLRA is performed for each simulation. The key ground responses, such as surface PGA, EPP ratio (ru), and lateral displacement of the soil (Δ_{lat}), are evaluated for 11 spectrum-compatible ground motions.

5 SENSITIVITY ANALYSIS

The cyclic response of the soils modelled using the PM4Sand model is substantially influenced by the constitutive model parameters: D_{ro}, G_o, and h_{po}. To establish the influence of these parameters on the surface response characteristics, such as PGA, a Sobol sensitivity analysis is performed considering the model parameters of all the soil layers. The results of the first-order and total-order sensitivity indices are shown in Figure 6.

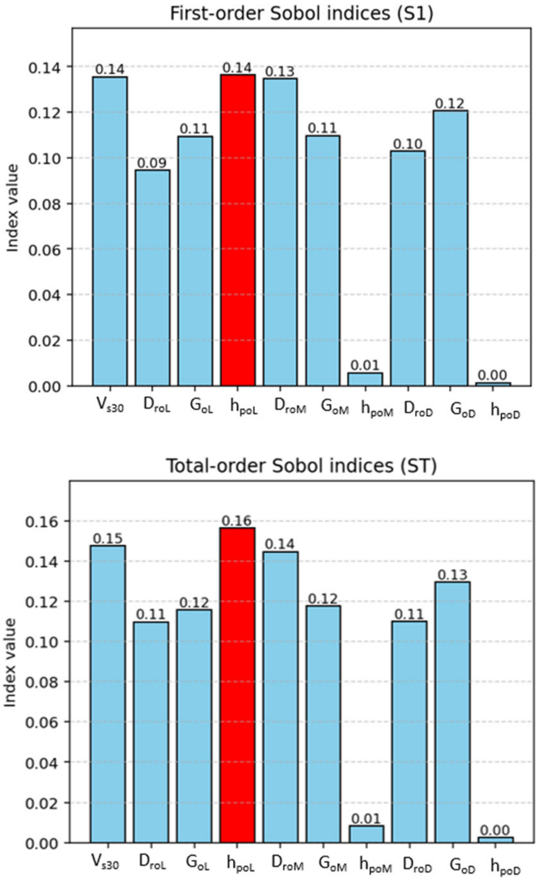

Figure 6 Results of the Sobol sensitivity analysis illustrating the relative influence of the PM4Sand model parameters of each soil layer on the surface PGA.

In Figure 6, the subscripts L, M, and D indicate the corresponding model parameters of LSM, MSM, and DSM layers, respectively. From the figure, it can be observed that the variability in surface PGA is influenced by both the overall stiffness of the soil deposit and the layer-specific constitutive behavior of the soil. V_{s30} exhibits larger sensitivity indices, a finding consistent with the results of the sensitivity analysis on TSNLRA. This highlights the importance of the stiffness of the entire soil profile in influencing the PGA characteristics of waves propagating through the soil. The stiffness and porewater pressure-related parameters of LSM show a strong influence on the surface PGA, indicating that the nonlinear and contractive behavior of the LSM layer significantly alters the surface response characteristics. However, the density and stiffness parameters of the MSM substantially contribute to the variation in PGA; however, the hpo of the MSM has relatively lower influence compared to that of LSM. Similarly, the stiffness and density parameters of the DSM show a higher contribution to the variation of PGA, while the pore pressure parameter has no contribution to the variation of PGA due to the predominant dilative behavior of dense sands. The results obtained from this study are consistent with the results of the global sensitivity analysis performed by Chou and Chen (2024). Overall, the results indicate that the surface PGA is primarily controlled by the stiffness of the subsoil profile and the nonlinear behavior of the LSM layer, while the pore pressure effects of the deeper dense sand layers have a minimal influence on the surface PGA.

6 RESULTS AND DISCUSSION

The results of the probabilistic GRA in terms of PGA, cumulative depth of liquefaction (Z_{liq}), r_u and Δ_{lat}, along the depth of the profile, corresponding to one ground motion with bedrock PGA of 0.171g, are presented in Figure 7.

Figure 7(a) presents the variation in r_u across the simulations. As indicated in the figure, the EPP buildup is negligible in the upper 5 m due to the presence of a non-liquefiable surficial silt layer. However, between depths of 5–15 m, a drastic increase in r_u is observed, with a mean value exceeding 0.8 and approaching 1.0 for some realizations, indicating potential liquefaction triggering at this depth. The larger variation in the r_u in the zone can be attributed to the sensitivity of r_u to the variability of the input parameters, such as D_r. However, for depths below 15 m, a gradual decrease in r_u is observed, indicating partial drainage or increased stiffness in deeper layers, resulting in reduced liquefaction potential.

From Figure 7(b), it can be observed that the initiation of liquefaction predominantly occurs within the depth range of 5-15 m, where the r_u is greater than 0.8. The Z_{liq} of most realizations varies between 50 and 250 cm within this depth, where loose to medium-dense sands prevail. Below 10 m, a decrease in Z_{liq} value is noticed and is approximately equal to zero, indicating that no or minor liquefaction has occurred at this depth. This Z_{liq} pattern strongly correlates with the r_u distribution from Figure 7(a), indicating that the zone between 5 and 10 m is susceptible to liquefaction.

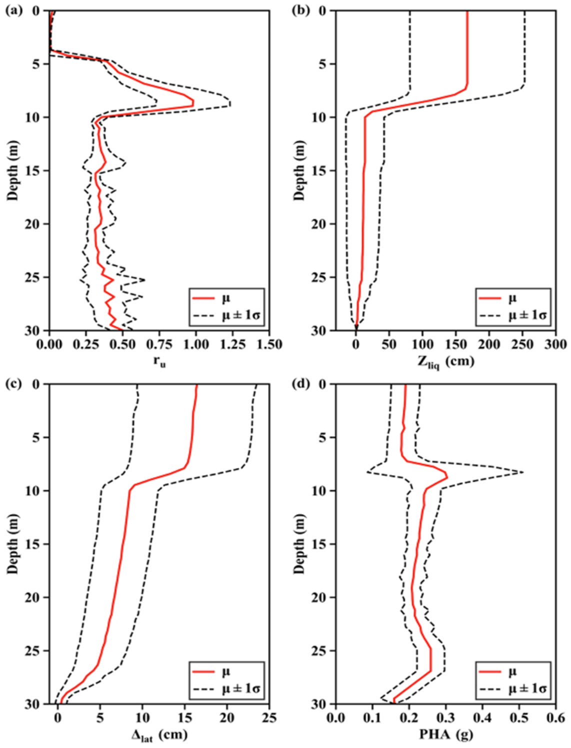

Figure 7 Variation of response parameters along the depth (a) excess porewater pressure ratio (r_u), (b) cumulative depth of liquefaction (Z_{liq}), (c) lateral displacement (Δ_{lat}), and (d) Peak horizontal or peak ground acceleration (PHA).

The depth-wise variation of lateral displacement in Figure 7c reveals that the largest horizontal deformations occurred predominantly within the 5–15 m depth interval, coinciding with the liquefiable sand layer identified in the pore pressure and cumulative liquefaction depth analyses. Mean lateral displacements in this zone are about 20–25 cm, with several realizations exhibiting values exceeding 30 cm. This indicates the potential for significant lateral movement during seismic loading. Above 5 m depth, the lateral displacements are observed due to the potential cyclic softening of the surficial silt and the liquefaction of the underlying silty sand.

Figure 7(d) shows the variation of PHA within the soil column along the depth. The variation indicates an amplification of PGA within the very dense sand layer, with no further amplification noticed up to a depth of 10 m. However, an amplification followed by a sharp attenuation is observed within the 5–10 m liquefiable layer due to the significant loss in shear strength under cyclic loading. This attenuation is less pronounced in the crustal and deeper layers, where soils maintain higher stiffness and resist substantial reductions in acceleration. The scatter within the liquefiable zone is considerable and reflects the dependency of the wave propagation on the degree of EPP generation and cyclic degradation. The difference in PHA between the shallow and deeper layers indicates variation in the energy dissipation patterns. The combined results from Figures 7(a)-7(d) indicate that the soil between depths of 5-10 m is the critical zone within the soil profile susceptible to liquefaction.

Furthermore, the pdf of Δ_{lat}, Z_{liq}, and PGA at the ground surface, along with the probability of exceedance of surface PGA, are presented in Figures 8 and 9, respectively.

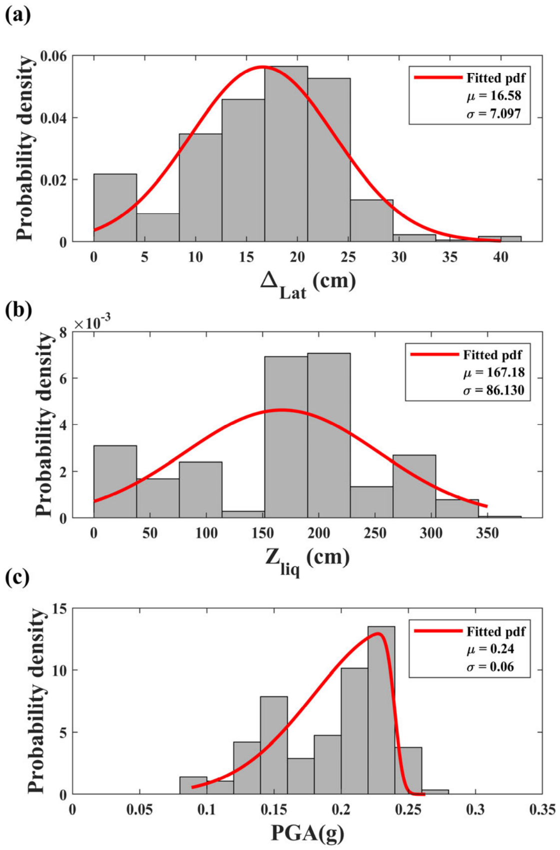

Figure 8 Probability density functions (pdf) for (a) Lateral displacement (Δ_{lat}), (b) total thickness of the liquefied layers (Z_{liq}), and (c) peak ground acceleration (PGA).

The pdf of lateral displacement indicates that the mean (μ) value of Δ_{lat} is 16.58 cm with a standard deviation (σ) of 7.1 cm. The μ of Z_{liq} is 1.67 m with a σ of 0.86 m. This information helps evaluate the required embedment depth for deep foundations or assess pile vulnerability to negative skin friction and unsupported length. A thicker liquefied zone increases the likelihood of lateral spreading and reduces the effective confining stress acting on structural elements, which can critically affect load-bearing capacity. The mean value of surface PGA obtained from the probabilistic ESNLRA for the study area, for an input bedrock motion of 0.171g, is 0.24 ± 0.06 g. The exceedance probability curve indicates that the probability of PGA exceeding 0.2 g is slightly below 30% (Figure 9). Sites with significant nonlinear response tend to attenuate the acceleration, which in turn reduces peak acceleration at the surface but may still result in large strains and pore pressure buildup at depth.

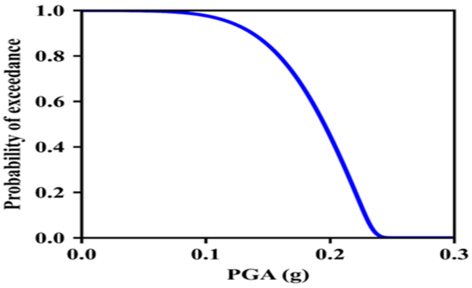

Figure 9 Hazard curve for peak ground acceleration (PGA) at surface level.

The analysis of the results from the ESNLRA for the study area indicates that the sand deposits at a depth of 5-10 m are highly susceptible to liquefaction. The results obtained from the study are realistic and definitely comparable to the findings reported in the literature from ground response and liquefaction studies (Das et al., 2025; Mandal et al., 2025a; Mistry et al., 2025; Sett et al., 2023).

6 CONCLUSIONS

Based on the results of the probabilistic TSNLRA and ESNLRA, the following conclusions are drawn:

- The hybrid approach of the combined initial TSNLRA with ESNLRA proved to be an efficient approach for identifying and evaluating the critical zones within the soil profile influencing the surface response.
- The sensitivity analysis indicated that the shear wave velocity (V_s) of the subsoil strata and the nonlinear behavior of the loose silty sand layer are important parameters significantly influencing the surface PGA. The pore pressure-related model parameters of the deep dense sand layers have a negligible influence on the variation in surface PGA.
- The surface PGA obtained from the TSNLRA is relatively higher than that obtained by a fully coupled ESNLRA, as the attenuation effects of the liquefiable soils are not captured by the TSNLRA approach, thereby yielding a higher PGA.
- The probabilistic ESNLRA indicated that the soil zone within the depths of 5 and 10 m is highly susceptible to liquefaction.
- The mean value of the r_u, Z_{liq}, and Δ_{lat} in the liquefaction susceptible zone is 0.9, 1.5 m, and 14 cm, respectively.
- The expected value of surface PGA at the study area for an input bedrock motion of 0.171g from ESNLRA is 0.24±0.06g.

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