

Liquefaction Hazard Assessment with Integrated Deterministic and Performance-Based Approach for Coastal Megacities: A Case Study of Karachi, Pakistan

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ABSTRACT

This research investigates liquefaction hazard assessment in Karachi, the economic hub of Pakistan. Karachi's coastal region is increasingly vulnerable to liquefaction-related hazards, which pose a significant threat to the city's seismic resilience. Rapid, unplanned urbanization and increasing earthquake risk in this densely populated area underscore the critical need for a comprehensive assessment of liquefaction hazard, which remains largely unquantified in existing local research. In this research, a comprehensive assessment of liquefaction potential is provided for the Defence Housing Authority (DHA) Phase-VIII Karachi using a comprehensive framework. The initial geotechnical characterization identifies Holocene silty sand deposits (SM) with a relative density of 38% and a shallow water table within 3 meters of the ground surface. Recent seismic hazard analysis, both probabilistic and deterministic, suggests that Karachi could experience a maximum PGA as high as 0.99g in some areas. The liquefaction potential was evaluated using advanced 3D finite element modeling in PLAXIS 3D software with the UBC3D-PLM Model, alongside an empirical simplified procedure. The simplified procedure consistently yields a Factor of Safety (FS) less than unity throughout the 9.14 m soil depth in both sand and silt layers, indicating high susceptibility of the coastal soil to liquefaction. Numerical modeling in PLAXIS 3D confirms these findings, showing a rapid increase in the pore water pressure ratio (r_u) to unity, with loss of soil strength and stiffness due to reduced effective stress. The soil is completely liquefied, with vertical settlement up to 0.125 m under a load of 100 kN/m². The research concludes that current infrastructure along the coastal belt of Karachi requires urgent geotechnical intervention, such as jet grouting or deep foundation transfer, to mitigate the catastrophic potential of seismically induced soil instability.

1. Introduction

The major global concern in densely populated coastal urban cities is the escalating threat posed by earthquakes. Because of its complex seismotectonic setting and large coastal interface, Karachi, Pakistan, is highly susceptible to earthquake-induced damage [1]. A high-intensity earthquake could cause extensive damage to the coastal megacity and principal economic hub of Pakistan, given its large population and increasing urbanization. Additionally, the city's location along the coastal belt increases liquefaction-related hazards [2]. Factors such as soil liquefaction potential and certain fundamental geotechnical weaknesses are critical in such situations, exacerbating damage caused by seismic activity [3]. Liquefaction is the phenomenon in which the granular, cohesionless, loose saturated soil loses its

strength and stiffness and can lead to catastrophic infrastructure failure even during earthquakes of moderate scale [3,4]. The stability of the structural foundations is directly undermined by the loss of soil's strength and stiffness. Despite its location near the triple junction of the Arabian, Indian, and Eurasian plates and major faults, a comprehensive evaluation of liquefaction potential in the Karachi coastal belt remains a critical gap in our understanding, impeding the implementation of more effective liquefaction mitigation strategies to reduce liquefaction risk in coastal soils along the coastal belt of Sindh. Existing liquefaction evaluation methods cannot fully account for the true extent of the earthquake hazard because of local geotechnical conditions and the uncertain seismic history of Karachi's coastal belt [1]. Karachi is home to millions of People, is Pakistan's largest city, plays a major role in the country's economy,

and has extensive critical infrastructure. Therefore, a significant earthquake can cause major damage to high-rise buildings and other critical infrastructure, including human casualties and economic losses, with far-reaching national consequences. Therefore, a comprehensive evaluation of liquefaction hazard for Karachi city is needed to support risk-informed urban planning, resilient infrastructure design, and disaster mitigation strategies.

1.1 Regional Geological and Seismotectonic Context

The geotechnical characteristics of coastal soils in Karachi make them more vulnerable to liquefaction hazard due to their specific geotechnical properties [3]. Soil data from subsurface geotechnical investigation reports indicate that most of the soil along the coastal belt of Karachi, Sindh, is poorly graded, loose, saturated sand of Holocene age with a small amount of clay fraction [5]. Mostly, Karachi's geology is composed of sedimentary rocks deposited through fluvial, deltaic, and marine processes [3]. The position of the subsurface groundwater table plays a critical role in the liquefaction susceptibility of the coastal soil in Karachi [6]. The subsurface groundwater table is very shallow, almost 3 m below the ground surface in most regions along Karachi's coastal belt [5]. The high groundwater table keeps the coastal soil saturated and increases its susceptibility to liquefaction [6]. Reclaimed land and artificial fills along Karachi's coastal belt further exacerbate liquefaction susceptibility [7]. The geological composition of reclaimed land mostly consists of unconsolidated, loose material, which is very highly susceptible to liquefaction under seismic loading [8]. Constructing high-rise buildings on such unconsolidated, loose soil increases the liquefaction hazard [9]. In view of vulnerable geotechnical and seismotectonic scenarios and subsurface groundwater conditions, liquefaction risk is considered very high in different areas of Karachi, such as the Defence Housing Authority (DHA), built on loose fine-grained, saturated sand, which is highly susceptible to liquefaction during strong earthquakes with magnitudes ranging from 6.5 to 7.5 [10]. Karachi's seismotectonic setting is complex, including the triple junction of the Indian, Arabian, and Eurasian plates and major geological faults along Karachi's coastal belt [7]. Karachi's coastal region is very close to a critical triple junction, where the Indian, Arabian, and Eurasian plates meet [11]. Karachi lies 150 km east of this triple junction, where the western and north-trending arms show convergent and transcurrent rates, making the city inherently prone to seismic activity [11]. Additionally, several major active fault lines are present in the vicinity of Karachi city, including the Ornach Nal Fault, the Allah Bund Fault, the Hub Fault, the Pab Fault, the Surjan Fault, the Jhimpir Fault, and the Rann of Kutch Fault [1]. The proximity of triple junctions and major plate boundaries could generate significant earthquake shaking in Karachi City due to the large-magnitude earthquakes along these

boundaries, even if the epicenter is some distance away, as seen in the felt shaking from the Makran and Bhuj earthquakes in 2001 and 1945, respectively [11]. Deterministic seismic hazard analysis indicates that maximum Peak ground acceleration varies across Karachi, with higher values concentrated around the Nagar Parker fault, which is considered controlling and hazardous for Karachi [1]. More recently, a series of tremors was felt in Karachi during July–October 2005, causing public concern [12]. Significant earthquakes have historically impacted the Sindh region, although documented damage within Karachi itself has been relatively limited in recent history [1]. Karachi's population has felt shaking from distant earthquake events such as the Makran earthquake (1945) and the Bhuj earthquake (2001) [11]. The historical record indicates potentially damaging earthquakes in the broader Sindh region, including a massive earthquake in 893 or 894 AD that severely affected the Indus Delta and towns near present-day Hyderabad [11]. This reminds us that significant seismic activity is expected to occur in the future. The seismic hazard assessment of Karachi classifies it from the least severe Zone-I to the most severe Zone-IV. The Pakistan Building Code 2007 places Karachi in Zone-II B, having an intermediate seismic hazard value with peak ground acceleration (PGA) in the range of 0.16–0.24g. However, some researchers argue that the seismic hazard is undervalued, citing Karachi's location near major fault lines and the potential for significant earthquakes [11]. Karachi could experience ground motions with peak ground acceleration (PGA) of around 0.25g for a 475-year return period, with a maximum peak ground acceleration (PGA) value reaching as high as 0.99g in some areas, particularly near the Nagar Parkar fault [1]. For the safe and sustainable design of Karachi's buildings, some studies recommended a peak ground acceleration (PGA) of 0.19g for a return period of 475 years and 0.33g for a return period of 2,475 years, with a shear wave velocity of 750 m/sec for foundation conditions [13]. According to deterministic seismic hazard assessments, Karachi could experience ground motions ranging from 0.25g to 0.40g under typical design-level return periods. However, a controlling source like the Nagar Parker fault presents a far more severe threat [1]. This study's numerical analysis uses a Peak Ground Acceleration (PGA) of 0.99g. It is very important to define this 0.99g intensity as an extreme, collapse-level scenario. Evaluating 0.99g is crucial for performance-based assessments of critical infrastructure where preventing collapse is paramount.

1.2 Liquefaction Susceptibility

Soil liquefaction is a phenomenon in which cohesionless, loose granular soil loses its strength and stiffness due to rapid cyclic loading during an earthquake [3]. In this phenomenon, pore pressure builds up within the soil, which ultimately reduces effective stress by reducing particle-to-particle contact [14]. The soil most susceptible to liquefaction is loose, saturated, granular

material such as sand, silt, and silty sand with low clay content. There are different factors that affect the soil's susceptibility to liquefaction, such as relative density, plasticity characteristics, degree of saturation, void ratio, and the amount of fines present in the soil [3].

1.3 Research Gap and Novelty

Previous studies along the coastal belt of Karachi, Pakistan, assessed liquefaction potential using only the SPT-based "Simplified Procedure" and calculated the factor of Safety against liquefaction triggering. The method applied is inherently constrained by its inability to predict the physical effect of soil failure, such as the magnitude of vertical settlement or the rate at which pore pressure is generated. Previous studies identified Karachi as liquefaction-prone due to the presence of loose silty sand/sandy silt, shallow groundwater table, and uncertain seismicity. However, previous studies did not integrate deterministic empirical liquefaction-triggering methods with performance-based numerical modeling in Plaxis 3D to estimate actual ground deformation, settlement, and pore pressure generation.

In this study, we integrate an empirical screening method for liquefaction evaluation with advanced numerical modeling in Plaxis 3D to provide a performance-based liquefaction hazard assessment along Karachi's coastal belt. To accurately simulate the dynamic behavior of Karachi's coastal soils, we integrated site-specific soil investigation data with advanced numerical modeling using PLAXIS 3D, enabling accurate simulation of the dynamic response of soils along the Karachi coast during an earthquake. Moreover, in this research study, we also assess the soil dynamic response in an extreme collapse-level scenario (0.99g).

The core research objective of this study is to comprehensively evaluate the liquefaction potential of Karachi's soil along the coastal belt by integrating a simple screening method with advanced numerical modeling in PLAXIS 3D, targeting Defence Housing Authority (DHA) Karachi Phase-VIII.

This is achieved through detailed soil characterization of the Karachi coast, an empirical simplified procedure for initial liquefaction screening, and advanced numerical modeling using PLAXIS 3D. Ultimately, the research study presents evidence-based methods and guidelines for liquefaction mitigation in Karachi coastal soils to effectively reduce liquefaction hazards along Karachi's coastal belt. The research findings can be especially beneficial for improving the resilience of Karachi's infrastructure and population against earthquake events. Moreover, the research findings and recommendations can greatly benefit urban planning and infrastructure development in other similarly characterized coastal cities globally.

2. Methodology

2.1 Liquefaction Evaluation Based on Soil Characterization

To evaluate the liquefaction potential of Karachi's coastal soils in depth, a detailed understanding of soil characterization is essential. Mostly, soils with relative densities between loose and medium-dense, having high saturation and without cohesion, such as sands and silty sands, are highly susceptible to liquefaction potential [15]. In addition, although gravelly soils mobilize higher strength and typically dissipate pore water pressure more quickly than sand, they can still liquefy [16]. The soil that consists of Silty sands (SM) with poor gradation (SP) is particularly susceptible to liquefaction [17]. However, soils with a well-graded composition are less susceptible to liquefaction than soils with a uniformly graded composition. For example, Mae Lao sand is highly susceptible to liquefaction due to its uniformly graded nature and low fine content [18]. Additionally, the coefficient of uniformity (CU) and mean particle size (D_{50}) are important indicators of soils' susceptibility to liquefaction; soils are considered more susceptible to liquefaction if the uniformity coefficient is below five [19]. The presence and characteristics of fines (silt and clay fractions) significantly influence liquefaction resistance. Although observations conflict and research is ongoing, [20]. When the fine content in cohesionless soils exceeds approximately 30%, the soil's liquefaction susceptibility generally decreases. Soil with fine content greater than 50% is often considered non-liquefiable or of low liquefaction potential [17]. Plasticity Index (PI) and Liquid Limit (LL) are key parameters, especially for fine-grained soils. Soil with substantial plastic fines content should be appraised using the Atterberg limits [17, 19]. Liquefaction principally occurs in soils with relative density less than 50%. Denser conditions significantly increase liquefaction resistance [16]. Liquefaction requires saturated or partially saturated soil conditions. Shallow groundwater tables (< 3 m) or a rise in the groundwater table significantly increases liquefaction susceptibility [17]. For deposits with the water table at a depth of 0 to 10 ft, the critical depth for liquefaction is often around 20 ft. Liquefaction is more likely to occur at shallow depths (up to 15 m) because of low confining pressure and younger, less dense deposits [21].

2.2 Simplified Procedure for Liquefaction Evaluation

The simplified procedure, also referred to as the total stress approach, is a widely accepted method for determining the onset of soil liquefaction [22, 23]. This method assesses the liquefaction potential by comparing the seismic demand on a soil layer to its capacity to resist liquefaction using a factor of safety. It compares the Cyclic Stress Ratio (CSR), which represents the seismic demand, with the Cyclic Resistance Ratio (CRR), which represents the soil's capacity. Fig. 1 shows the

procedure. Table 1 shows the parameters required for this procedure.

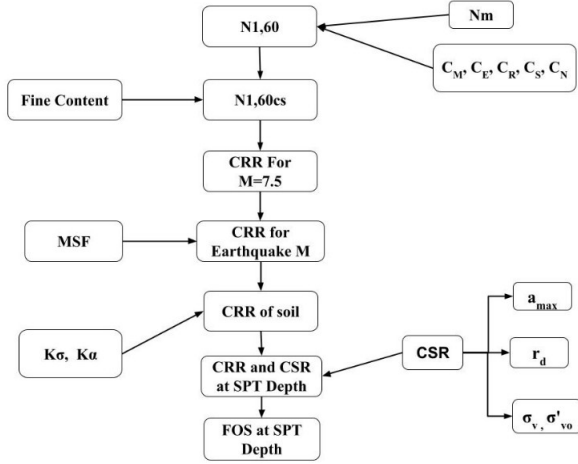


Fig. 1. The NCEER Procedure

Table 1
The Parameters Required for the NCEER Procedure

Nomenclature	Definition
$CRR_{7.5}$	Cyclic Resistance Ratio
CSR	Cyclic Stress Ratio
MSF	Magnitude Scaling Factor
K_{α}	Sloping Ground Correction
K_{σ}	Overburden Correction
τ_{avg}	Shear Stress
σ'_{vo}	Effective Vertical Overburden Stresses
a_{max}	Peak Horizontal Acceleration
σ_{vo}	Total Vertical Overburden Stresses
R_d	Stress Reduction Coefficient
z	Depth from Ground Surface (m)
N_m	Standard Penetration Resistance
$(N_1)_{60}$	SPT Blow Count for a Hammer Efficiency of 60%.
$(N_1)_{60cs}$	$(N_1)_{60}$ to an equivalent clean sand value
FC	Fines Content
C_E	Hammer Energy Ratio (ER) Correction
C_N	Factor to Normalize N_m
C_R	Rod Length Correction
C_S	Sampler Correction (with or without liners)
C_B	Borehole Diameter (Correction)

Cyclic Stress Ratio (CSR): This term represents the earthquake-induced shear stress on the soil element. The CSR is calculated using the following formula:

$$CSR = 0.65 \left(\frac{a_{max}}{g} \right) \cdot \left(\frac{\sigma_v}{\sigma'_v} \right) \cdot r_d \quad (1)$$

a_{max} = Peak horizontal ground acceleration at the surface

g = Acceleration due to gravity

σ_v = The total vertical stress

σ'_{vo} = The initial vertical effective stress

r_d = Stress reduction factor that decreases with depth.

Given are the average values of stress reduction factors

$$r_d = 1.0 - 0.00765z; \text{ for } z \leq 9.15 \text{ m}$$

$$r_d = 1.174 - 0.0267z; \text{ for } 9.15 \text{ m} < z \leq 23 \text{ m}$$

$$r_d = 0.744 - 0.008z; \text{ for } 23 \text{ m} < z \leq 30 \text{ m}$$

$$r_d = 0.50; \text{ for } z > 30 \text{ m}$$

The CSR calculation essentially estimates the average cyclic shear stress induced by the earthquake, considering the earthquake's intensity and the soil's stress state.

Cyclic Resistance Ratio (CRR): This parameter represents the soil's capacity to resist liquefaction. The CRR is primarily determined using in-situ tests such as the Standard Penetration Test (SPT) and Cone Penetration Test (CPT). The CRR is typically estimated based on SPT blow counts, corrected for various factors.

$$(N_1)_{60} = N_{SPT} \cdot C_N \cdot C_E \cdot C_B \cdot C_S \cdot C_R \quad (2)$$

The clean sand base curve for a magnitude 7.5 earthquake is used as the starting point.

$$CRR_{7.5} = \frac{1}{34 - (N_1)_{60}} + \frac{(N_1)_{60}}{135} + \frac{50}{[10(N_1)_{60} + 45]^2} - \frac{1}{200} \quad (3)$$

$(N_1)_{60}$ represents the SPT blow count normalised to 60% hammer efficiency.

Fines content correction: The CRR is adjusted based on the amount of fine-grained material in the soil. As fines content increases, liquefaction resistance can be affected.

Seed and Idriss developed equations (4) and (5) to correct $(N_1)_{60}$ to an equivalent clean-sand value, $(N_1)_{60cs}$.

$$(N_1)_{60cs} = \alpha + \beta(N_1)_{60} \quad (4)$$

Where α and β are coefficients determined from the following relationships:

$$\alpha = 0 \quad \text{for } FC \leq 5\% \quad (5a)$$

$$\alpha = \exp \left[1.76 - \left(\frac{190}{FC^2} \right) \right] \quad \text{for } 5\% < FC < 35\% \quad (5b)$$

$$\alpha = 5.0 \quad \text{for } FC \geq 35\% \quad (5c)$$

$$\beta = 1.0 \quad \text{for } FC \leq 5\% \quad (5d)$$

$$\beta = \left[0.99 + \left(\frac{FC^{1.5}}{1000} \right) \right] \quad \text{for } 5\% < FC < 35\% \quad (5e)$$

$$\beta = 1.2 \quad \text{for } FC \geq 35\% \quad (5f)$$

Magnitude Scaling Factor (MSF): The CRR is adjusted to account for earthquake magnitudes other than 7.5. The magnitude scaling factor (MSF) adjusts the CRR value based on the earthquake magnitude. Idriss established an updated set of magnitude scaling factors as given by the equation

$$MSF = \frac{10^{2.24}}{M_w^{2.56}} \quad (6)$$

where M_w represents the magnitude of earthquakes.

Overburden Correction Factor (K_σ): The CRR is adjusted to account for the effect of higher overburden pressures. Increased effective overburden pressure can enhance the soil's resistance to liquefaction. For effective overburden stress exceeding 100 kPa, the overburden correction was introduced [21].

Static Shear Stress Correction Factor (K_α): The CRR is adjusted to account for the presence of initial static shear stresses. The effect of initial static shear stress on liquefaction resistance can be considered using the K_α factor. K_α can be determined graphically as suggested by Seed et al. (2003) [24].

The CRR essentially represents the soil's ability to withstand the cyclic shear stresses induced by an earthquake.

Calculate the Factor of Safety (FS): Calculate FS as the ratio of CRR to CSR.

$$FS = \left(\frac{CRR_{7.5}}{CSR} \right) MSF(K_\sigma)(K_\alpha) \quad (7)$$

A factor of safety (FOS) less than one indicates that liquefaction likely occurs, and a factor of safety (FOS) greater than one indicates that the soil is likely to be safe against liquefaction.

2.3 Liquefaction Evaluation Based on Numerical Analysis

As a total stress analysis, the simplified procedure does not account for the development of excess pore water pressure, which is the fundamental mechanism of liquefaction. Therefore, for more critical projects, advanced methods using an effective-stress approach should be considered. The numerical approach using PLAXIS 3D offers a paradigm shift from a simplified empirical method. It shifts the engineering assessment from “if” liquefaction will occur to “what will happen when” it triggers. It can directly estimate excess pore water pressure buildup and model its real-time effect on soil strength characteristics, providing direct, quantitative predictions of ground failure and structural performance. For the performance-based design of high-rise critical infrastructure in Karachi city, this level of understanding is crucial for Karachi's seismically active regions.

2.3.1 The Typical Analysis Workflow in Plaxis 3D

To accurately simulate the behavior of Karachi's coastal soil in Plaxis 3D, numerical modeling is conducted in distinct phases. In the first phase, gravity is applied to the model to establish the initial conditions (K_0 procedure), allowing calculation of initial horizontal and vertical effective stresses and static pore water pressure under hydrostatic conditions throughout the soil mass. In the second phase, the effect of existing structures is simulated. This generates pre-earthquake stress conditions in the soil, which play a critical role because these stresses significantly affect the soil's liquefaction susceptibility. In the third phase, a selected acceleration time history is applied at the model's compliant base for dynamic analysis. The numerical model simulation

runs for the entire earthquake duration. The numerical analysis provides comprehensive insights into the liquefaction effects. The key indicator of liquefaction onset is the pore pressure ratio (r_u), with $r_u \approx 1$. Time-history plots of effective stress, excess pore pressure, and deformations such as settlement are analyzed, which reveal both total and differential settlements.

2.4 Site selection

The Defence Housing Authority (DHA) Phase-VIII in Karachi, Pakistan, was selected as a critical site for liquefaction evaluation due to a combination of its geographical location, specific soil characteristics, observed seismic activity, and ongoing urban development. Along Karachi's coastal belt, Defence Housing Authority (DHA) is a well-organized residential area.

3. Results and Discussion

3.1 Liquefaction Evaluation by Soil Characterization

The seismic liquefaction potential of the site was evaluated using a combination of Soil Characterization from field data, an empirical simplified procedure, and fully coupled dynamic-deformation numerical analyses using the finite element software PLAXIS-3D. A field investigation was carried out by drilling boreholes in DHA Phase-VIII, Karachi, and conducting standard penetration tests (SPT) at predetermined depths to get in-situ N-values. Most soil samples collected during the drilling study were disturbed and were subsequently used for laboratory classification and measurement of the necessary geotechnical parameters. To evaluate the geotechnical characterization, various tests have been performed on the soil samples. Table 2 summarizes the key geotechnical properties of the susceptible soil layers.

Table 2

Key Geotechnical Properties of Susceptible Soil Layer

Specific Gravity	2.63
SPT (N_{60}) _{avg}	28
Relative Density	38% (Medium Dense)
Sand Content	0.25 mm–0.1 mm (98.8% Fine Sand)
Fine Contents	Size = 0.1 mm–0.01 mm (1.2% Silt)
Plasticity of fines	N.P
Permeability (cm/day)	0.0034
Groundwater table	Shallow Depth from Ground surface (< 3 m)
Type of soil deposits	Silty Sand
Site Conditions	Loose Saturated Granular Soil

The soil deposits investigated in coastal Karachi predominantly consist of silty sand with the following geotechnical properties. Specific gravity of 2.63, relative density of 38% (medium dense), and a high percentage of fine sand (98.8%) with minimal silt content (1.2%). The fines are classified as non-plastic (NP),

indicating a lack of significant clay minerals that could contribute to cohesion. The coastal soil in Karachi city is highly saturated because the subsurface groundwater table is shallow (around 3 m below the ground surface) in most areas of the city. The coefficient of permeability of the coastal soil also indicates very low hydraulic conductivity, with a measured value of 0.0034 cm/day. These geotechnical factors collectively indicate that the subsurface soil profile is highly vulnerable to excess pore-water pressure buildup during earthquake events, which is the main cause of liquefaction triggering.

3.2 Liquefaction Evaluation by Empirical Simplified Procedure

The cyclic stress ratio (*CSR*), which shows the earthquake demand on the coastal soil of Karachi city, was calculated using the empirical simplified procedure based on site-specific seismic loading parameters, considering Karachi's proximity to major tectonic features such as the triple junction of the Eurasian, Arabian, and Indian plates and the nearby Bhuj, Ahmedabad, and Omara Faults. The cyclic resistance ratio (*CRR*) was estimated from the soil characterization parameters.

The simplified procedure indicates high susceptibility to liquefaction in both soil layers based on their standard penetration values. The calculated factor of safety (*FOS*) against liquefaction was consistently below 1 in both sand and silt layers, indicating a high probability of liquefaction triggering. Near the ground surface, the minimum factor of safety (*FOS*) was 0.07, which increased with depth. This indicates reduced liquefaction potential in the subsurface soil due to higher confining stress. Table 3 provides a detailed breakdown of the factor of safety (*FOS*) with depth.

3.3 Liquefaction Evaluation by Numerical Method

The soil profile consists of two layers: an upper layer of sand and a lower layer of silt, with thicknesses of 6.5 m and 2.6 m, respectively, and was modeled to assess the site response (Fig. 2). The 3D finite element model consists of 10-noded tetrahedral elements with refined meshing in the areas of interest, with dimensions of 100 m × 100 m (Fig. 3). A mesh sensitivity analysis was performed to ensure that the element size was sufficient to capture the wavelengths associated with the input ground motion. The dynamic boundaries were defined using “Compliant Base” conditions at the bottom to allow the application of the seismic motion while absorbing downward-traveling waves. Side boundaries were modeled using “Free-Field” elements, which represent the response of the semi-infinite soil medium surrounding the model and prevent spurious wave reflections. Rayleigh damping is incorporated to account for small-strain viscous damping not captured by the plastic constitutive model. The damping matrix *C* is calculated as:

$$C = \alpha M + \beta K \quad (8)$$

Where *K* and *M* represent the stiffness matrix and mass matrix, respectively. The coefficients α and β were determined based on the target damping ratio (typically 1–2%) and the predominant frequency range of the input motion. An effective-stress plasticity model, UBC3D-PLM, was specifically developed for liquefaction analysis and was used to represent the behavior of the sand and silt layers. Table 4 shows the UBC3D-PLM parameters for sand and silt. Dynamic analysis was conducted in a phased approach, beginning with a gravity-loading initial phase to establish in-situ stresses, followed by the application of foundation loads. It culminated in a dynamic phase in which an earthquake acceleration time history was applied at the model base (Fig. 4). The soil behavior during the dynamic phase was modeled as Undrained (A), which is suitable for capturing the rapid, undrained pore pressure generation during seismic loading. Then, strong earthquake motion was applied to the model with a peak ground acceleration of 0.99g for 10 seconds. Applying an extreme peak ground acceleration of 0.99g is invaluable for assessing geostucture performance under severe seismic loading (Fig. 5).

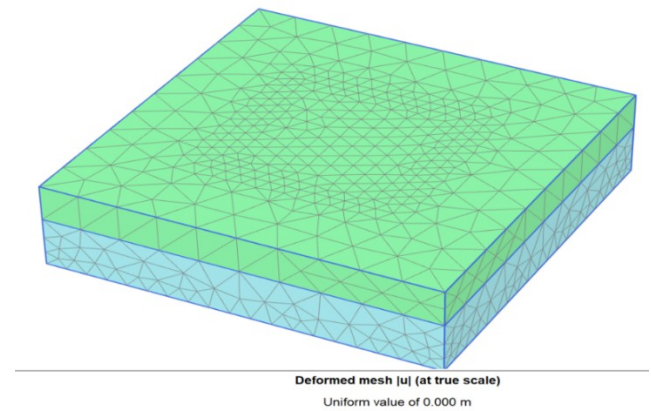


Fig. 2. 3D Finite Element Mesh and Soil Profile Model

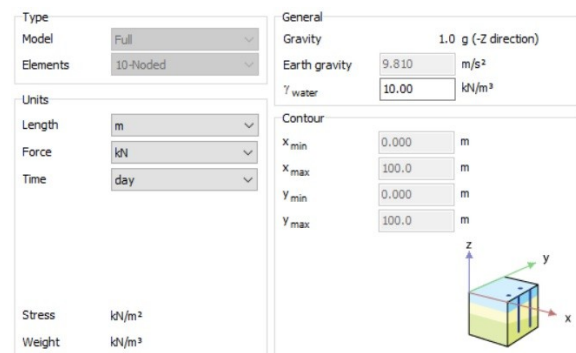


Fig. 3. General Analysis Settings and Model Contour Parameters in PLAXIS 3D

3.4 Numerical Model Outputs

Pore Pressure and Effective Stress: The pore pressure ratio in both sand and silt layers abruptly reached unity, signifying the onset of liquefaction. In Fig. 6, the time-history graph clearly indicates that the excess pore water pressure peaks at 233.04 kPa, reducing the mean effective stress from an initial value of −21.236 kPa to a near-zero residual state (Fig. 7). The PLAXIS-

Table 3

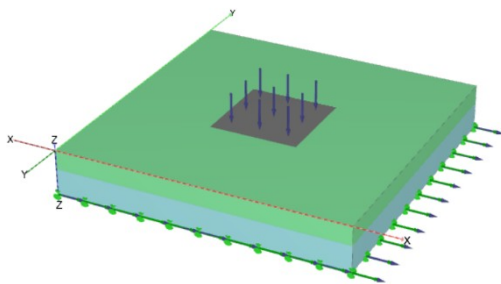
Simplified Procedure Liquefaction Summary

Soil	Depth (m)	N_{60}	γ (kN/m ³)	σ_v	σ'_v	R_d	CSR	CRR _{7.5}	FOS
Sand	0.91	8	18.6	17.03	8.06	0.99	1.349	0.105	0.07
Sand	1.82	9	18.6	34.07	16.13	0.98	1.340	0.114	0.08
Sand	2.74	22	18.6	51.10	24.19	0.97	1.330	0.252	0.18
Sand	3.65	20	18.6	68.14	32.26	0.97	1.321	0.225	0.17
Sand	4.57	23	18.6	85.17	40.32	0.96	1.311	0.266	0.20
Sand	5.48	25	18.6	102.21	48.39	0.95	1.302	0.301	0.23
Silt	6.40	28	17.6	112.97	50.18	0.95	1.377	0.379	0.27
Silt	7.31	27	17.6	129.11	57.35	0.94	1.367	0.348	0.25
Silt	8.22	31	19.6	161.39	80.66	0.93	1.206	0.568	0.47
Silt	9.14	32	19.6	179.32	89.62	0.93	1.197	0.7424	0.61

Table 4

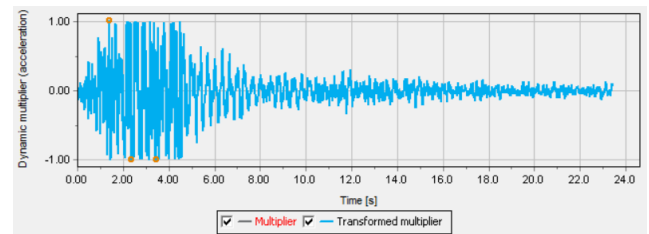
UBC3D-PLM Material Model for Sand and Silt

Parameters	Symbol	Unit	Method	Sand	Silt
Unit weight of soil	γ_{unsat}	kN/m ³	Laboratory Test	18.63	17
Saturated Unit weight of soil	γ_{sat}	kN/m ³	Laboratory Test	21.39	18
Void ratio	e_{int}	-	Mathematical Relations	0.38	0.4
Constant volume friction angle	ϕ_{cv}	(°)	Direct Shear Test	30	30
Peak friction angle	ϕ_p	(°)	$\Phi_p = \Phi_{cv} + [(N_1)_{60}] / 10$	35	33
Cohesion	C	kPa	Direct Shear Test	0	0
Elastic Shear Modulus	K_G^e	-	$K_G^e = 21.7 \times 20.0 \times [(N_1)_{60}]^{0.333}$	1156.95	1069.37
Elastic bulk modulus	K_B^e	-	$K_B^e = K_G^e \times 0.7$	809.86	748.55
Plastic Shear Modulus	K_G^p	-	$K_G^e [(N_1)_{60}]^2 \times 0.003 + 100.0$	1352.97	821.82
Elastic Shear Modulus Index	n_e	-	Curve fit (Default)	0.5	0.5
Elastic bulk modulus Index	m_e	-	Curve fit (Default)	0.5	0.5
Plastic shear modulus index	n_p	-	Curve fit (Default)	0.4	0.4
Failure Ratio	R_f	-	$R_f = 1.1 \times [(N_1)_{60}]^{-0.15}$	0.9	0.9
Atmospheric pressure	P_A	kPa	Standard value	100	100
Tension Cut-off	σ_t	kPa	Default	0	0
Densification Factor	Fac_{hard}	-	Curve fit (Default)	1	1
Corrected SPT Value	$(N_1)_{60}$	-	In-Situ Test or $(N_1)_{60} \approx [RD]^{2/15}$	19	15
Post-liquefaction factor	Fac_{post}	-	Curve fit (Default)	0.2–1	0.2–1

**Fig. 4.** 3D Model Showing the Application of Foundation and Earthquake Loads

3D numerical model confirms liquefaction potential through the observed responses.

Ground Settlement: Due to liquefaction, effective

**Fig. 5.** Earthquake Acceleration Time-History Dynamic Analysis

pore water pressure rises, reducing the effective stresses between soil particles and causing substantial vertical ground deformation that accumulates during the strong earthquake-shaking phase (Fig. 8).

The model consisted of two soil layers: a sand layer

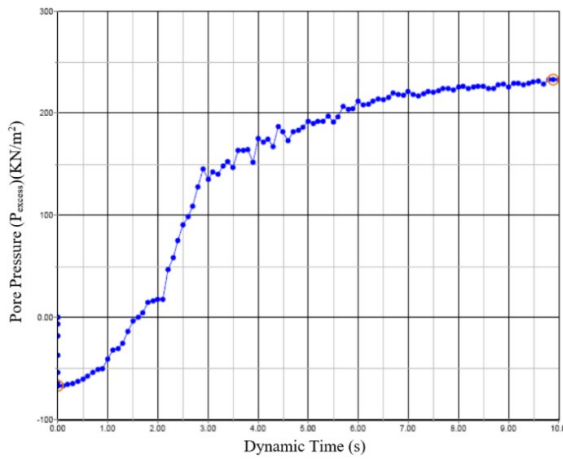


Fig. 6. Time-History Plot: Excess Pore Water Pressure vs. Dynamic Time

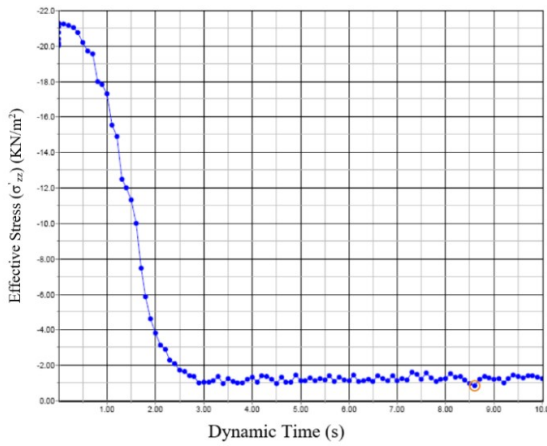


Fig. 7. Time-History Plot: Effective Stress vs. Dynamic Time

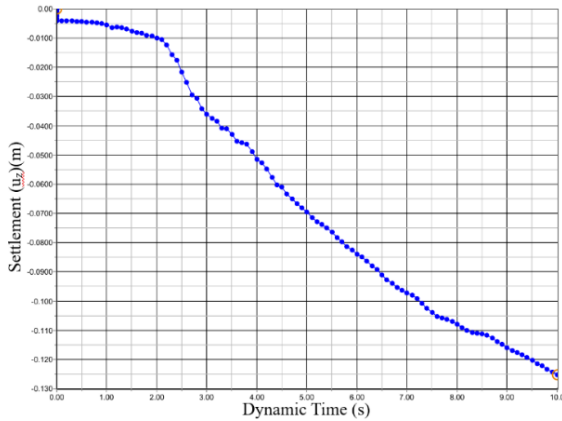


Fig. 8. Time-History Plots: Vertical Settlement vs. Dynamic Time

overlying a silt layer, and it predicted a maximum foundation settlement of 0.125 m during the simulated earthquake duration. A significant differential settlement was observed around the footing, as illustrated in Fig. 9. Table 5 summarizes these key deformation outputs.

3.5 Mechanism of Pore Water Pressure Development

The numerical modeling results of the coastal soil of Karachi in Plaxis 3D show that, in certain zones of the numerical model, the absolute values of excess pore water pressure are extremely high, reaching approximately 233.04 kPa. It is necessary to clarify that these high absolute values of excess pore water pressure are

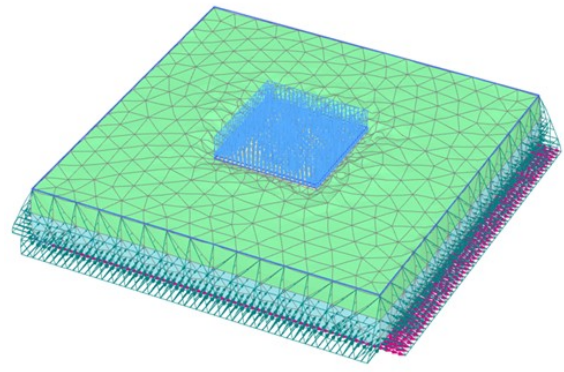


Fig. 9. Differential Settlement of Foundation

not global but localized beneath the foundation of the coastal soil of Karachi. This phenomenon is due to the results of stress redistribution, where the 100 kN/m² foundation load significantly increased the total vertical stress in the underlying subsurface soil. During the “Undrain A” dynamic analysis, the coastal soil of Karachi tries to contract, but the incompressibility of the pore water forces an increase in pressure that must overcome these higher pre-existing total stresses to achieve $r_u = 1.0$. Therefore, confirmation of liquefaction is based on the pore pressure ratio approaching 1.0, rather than the absolute magnitude of the pressure, which is a function of the superimposed load of the structure [25].

Table 5

Summary of Key PLAXIS-3D Dynamic Analysis Outputs

Parameter	Numerical Model
Methodology	Finite Element (PLAXIS-3D)
Liquefaction Potential	Confirmed ($\sigma' \approx 0$)
Depth of Liquefaction	Both layers of Sand and Silt (0–9.14 m)
Max. Excess Pore Pressure	~233.04 kPa
Effective Stress Change	Reduced to near-zero
Max. Foundation settlement during earthquake	0.125 m

3.6 Comparative Summary

Table 6 presents a comparative analysis of the results from the deterministic simplified procedure and the performance-based numerical modeling. Both approaches show strong qualitative agreement. The depth interval identified as highly susceptible by the simplified procedure ($FOS < 1.0$) aligned with PLAXIS-3D’s predicted zones where r_u approaches 1.0.

The site is highly vulnerable to liquefaction, as consistently shown by both the simplified procedure and the advanced numerical methods. Adverse geotechnical conditions, particularly a medium-dense to loose soil profile, non-plastic fine sand with a shallow water table, are the physical mechanism behind this (Table 2). The physical process is visualized by PLAXIS 3D, which illustrates how the stress from the soil is transferred

Table 6

Comparative Summary of Liquefaction Triggering Assessment

Depth (m)	Soil Type	N_{60}	FOS (Simplified)	PLAXIS-3D Outcome (r_u)	Agreement
0.91	Fine Sand	8	0.07	Liquefied ($\sigma' \approx 0$)	Yes
1.82	Fine Sand	9	0.08	Liquefied ($\sigma' \approx 0$)	Yes
2.74	Fine Sand	22	0.18	Liquefied ($\sigma' \approx 0$)	Yes
3.65	Fine Sand	20	0.17	Liquefied ($\sigma' \approx 0$)	Yes
4.57	Fine Sand	23	0.20	Liquefied ($\sigma' \approx 0$)	Yes
5.48	Fine Sand	25	0.23	Liquefied ($\sigma' \approx 0$)	Yes
6.40	Fine Sand	28	0.27	Liquefied ($\sigma' \approx 0$)	Yes
7.31	Silt	27	0.25	Liquefied ($\sigma' \approx 0$)	Yes
8.22	Silt	31	0.47	Liquefied ($\sigma' \approx 0$)	Yes
9.14	Silt	32	0.61	Liquefied ($\sigma' \approx 0$)	Yes

to the pore water during earthquake loading (Fig. 6), resulting in the loss of shear strength and stiffness of the soil (Fig. 7). The identification of hazard was validated by the strong convergence between the results of simplified procedure and PLAXIS 3D modeling on the onset of liquefaction (Table 6). The numerical model's simulation of complete strength loss ($\sigma' \approx 0$) directly matched the prediction of the simplified procedure's low factor of safety in the same zone of both sand and silt layers. In contrast to the simplified method's initial screening of liquefaction susceptibility, PLAXIS 3D offers a more comprehensive understanding. PLAXIS 3D assesses the performance of the soil-structure system, predicts time-dependent pore pressure, and, most importantly, quantifies ground settlement for engineering design. Despite its use as an effective screening tool (Table 3), the simplified procedure's main limitation is its failure to quantify the consequences of liquefaction. PLAXIS-3D bridges this gap by providing quantitative estimates of ground deformation (Table 5), which are critical for engineering design.

4. Conclusion

The liquefaction hazard along the coast of Karachi poses a significant threat to the city's urban stability and economic continuity. This research shows high susceptibility to liquefaction in the DHA region's Holocene silty sand under a design-level PGA of 0.99g. The factor of safety is well below unity in both the sand and silt layers. Empirical results show a Factor of Safety well below unity in both layers of sand and silt, which are supported by advanced PLAXIS 3D numerical simulation. Predicting a catastrophic increase in pore water pressure and vertical settlement up to 0.125 meters indicates a high likelihood of severe infrastructure damage. Effective risk mitigation requires quantitative depth, which is achieved by moving from empirical triggering assessment to performance-based numerical modeling.

5. Recommendation

Given the severe liquefaction susceptibility identified, site-specific ground improvement is a non-negotiable engineering necessity for new or existing construction

in Karachi, not a choice. Mitigation strategies must be tailored according to the soil condition, building type, and desired performance [26]. Shallow ground stabilization methods are appropriate for low-rise houses and light commercial buildings when deep foundations are uneconomical. High-viscosity grout bulbs, when injected into the loose sand layers, increase their relative density and form a reinforcing matrix [27]. Because it can be carried out with minimal disruption to the site, this technology is highly useful for the retrofitting of existing structures in metropolitan cities like the Defence Housing Authority (DHA) Karachi. High-rise buildings in Karachi apply horizontal and vertical loads on the liquefiable shallow sandy soil of the Karachi coastal belt, which cannot support them. Therefore, it is essential to transfer the structural load to a greater depth, at least more than fifteen meters. Providing vibro-stone columns in major structural developments will increase the soil's average shear modulus by providing high-permeability vertical drainage pathways. To accelerate the pore water pressure dissipation in silty sands (SM), prefabricated vertical drains (PVDs) should be used with Stone columns [26]. Moreover, for constructing structures on liquefiable soil, updated building codes must be adopted and strictly enforced, especially in liquefaction-susceptible zones. To address the liquefaction potential, the updated building codes should incorporate specific seismic design provisions for different structures, such as ground stabilization methods or pile foundations in liquefaction-susceptible areas.

6. Future Research Directions

To better evaluate liquefaction potential and mitigation strategies in Karachi City, future research could explore several important areas. To better understand subsurface conditions, particularly in Karachi's coastal belt and reclaimed land areas, more detailed and extensive subsurface soil investigations are needed. To support urban planning and risk management, Karachi needs high-resolution liquefaction hazard maps. Several earthquake scenarios and future sea-level rise must be represented in these maps. There is also a need for research in which an assessment must be done for the feasibility

and cost-effectiveness of the various ground improvement techniques for mitigating liquefaction potential. Lastly, using innovative technologies such as remote sensing and GIS to assess liquefaction susceptibility and monitor liquefaction on a large scale in Coastal Sindh can provide valuable spatial data for regional-scale liquefaction risk management and planning.

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Declarations

Conflicts of interest: On behalf of all authors, the corresponding author declares that there is no conflict of interest.

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