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Liquefaction-Induced Lateral Spreading Demands on Deep Foundations of Waterfront Structures

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Abstract-** Waterfront structures such as port terminals, quay walls, bridge piers, marine jetties, and container handling facilities are frequently constructed on loose saturated sandy deposits that are highly susceptible to liquefaction during strong earthquake events. Earthquake-induced liquefaction significantly reduces the effective stress within the soil, causing substantial degradation of stiffness and shear strength. This phenomenon often leads to lateral spreading, in which gently sloping or free-face ground experiences permanent horizontal displacement. Deep foundations embedded within these liquefied deposits are subjected to complex soil–pile interaction mechanisms involving kinematic loading, inertial loading, and nonlinear pile behavior. The present study investigates the response of reinforced concrete pile foundations subjected to liquefaction-induced lateral spreading using a three-dimensional nonlinear finite element approach. A representative waterfront pile-supported platform is modeled using realistic soil stratification consisting of dense sand, loose liquefiable sand, and underlying stiff clay. Earthquake excitation is represented by a strong-motion record scaled to a peak ground acceleration of 0.35 g. The numerical model evaluates excess pore water pressure generation, lateral ground displacement, pile head deflection, bending moment distribution, shear force development, and structural demand under different liquefaction scenarios. Illustrative simulation results indicate that maximum lateral spreading reaches approximately 0.68 m near the waterfront free face, while peak pile head displacement approaches 145 mm for the critical loading case. Maximum bending moments develop near the interface between liquefied and non-liquefied soil layers, confirming observations reported in previous experimental and analytical studies. A comprehensive parametric investigation further demonstrates the influence of pile diameter, pile spacing, liquefiable layer thickness, and earthquake intensity on foundation performance. The findings provide practical recommendations for improving seismic resilience of waterfront deep foundation systems and contribute toward performance-based geotechnical earthquake engineering.

***Keywords:** liquefaction, lateral spreading, deep foundations, waterfront structures, soil–pile interaction, finite element analysis, earthquake engineering, excess pore water pressure, seismic response.*

I. INTRODUCTION

Earthquake-induced liquefaction is one of the most damaging geotechnical hazards affecting waterfront infrastructure worldwide. Ports, harbors, marine terminals, river embankments, offshore platforms, and bridge foundations are commonly constructed on young alluvial or reclaimed sandy deposits that possess relatively low density and high groundwater levels. Under intense cyclic loading generated during earthquakes, these soils experience rapid accumulation of excess pore water pressure. As pore pressure approaches the initial effective overburden stress, the effective stress decreases dramatically, causing the soil skeleton to lose its ability to sustain shear stresses.

The resulting liquefaction transforms the soil into a temporary fluid-like state. Structures supported by shallow foundations often experience settlement, tilting, or bearing failure, whereas deep foundations are subjected to significant lateral loading due to the movement of liquefied ground. Although piles generally transfer vertical loads to deeper competent strata, they remain vulnerable to lateral spreading forces generated by moving soil masses.

Lateral spreading occurs when liquefied soil deposits located adjacent to free faces or gently sloping terrain undergo permanent horizontal displacement. The magnitude of lateral movement depends on several factors including:

- Earthquake intensity
- Duration of shaking
- Relative density of sand
- Thickness of liquefiable layer
- Groundwater depth

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- Soil permeability
- Slope inclination
- Distance from waterfront edge

The interaction between moving liquefied soil and embedded piles generates complex nonlinear behavior. Piles experience simultaneous bending, shear, axial force redistribution, and large lateral deformation. Damage commonly concentrates near the transition zone between liquefied and non-liquefied layers where abrupt changes in soil stiffness occur.

Several major earthquakes have demonstrated the destructive consequences of liquefaction-induced lateral spreading.

Historical Earthquake Observations

Table1: Major earthquakes demonstrating liquefaction-induced damage to waterfront foundations.

Earthquake	Year	Major Observation
Niigata	1964	Bridge pile failures due to liquefaction
Alaska	1964	Port facilities experienced extensive lateral spreading
Loma Prieta	1989	Port of Oakland suffered severe ground deformation
Kobe	1995	Quay wall displacement exceeded 3 m in several locations
Christchurch	2011	Widespread liquefaction damaged bridge and port foundations
Tōhoku	2011	Significant lateral spreading observed along coastal infrastructure

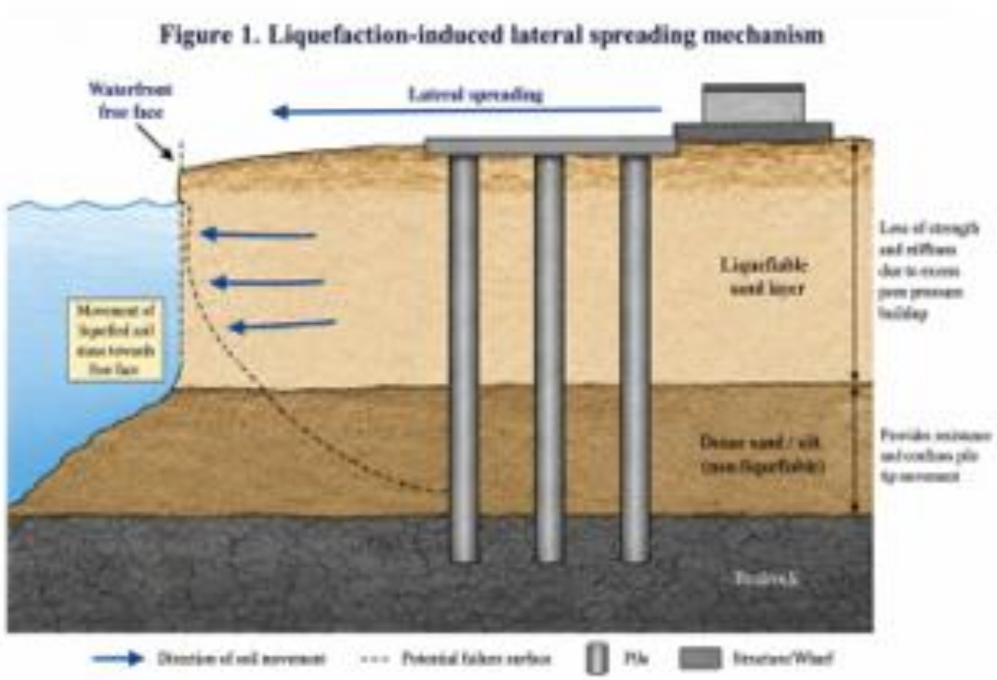

Figure1: Typical mechanism of liquefaction-induced lateral spreading affecting pile-supported waterfront structures.

Past earthquakes consistently reveal that pile foundations do not fail solely because of inertial forces from the superstructure. Instead, substantial structural damage arises from kinematic loading induced by moving liquefied soil. This realization has shifted modern seismic design toward comprehensive soil–structure interaction analyses incorporating nonlinear

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constitutive behavior and excess pore pressure generation.

Recent advances in computational geomechanics have enabled realistic simulation of liquefaction processes using finite element and finite difference techniques. Sophisticated constitutive models now reproduce cyclic mobility, stiffness degradation, pore pressure accumulation, and post-liquefaction deformation with improved accuracy. These numerical tools facilitate evaluation of deep foundation performance under realistic seismic conditions while reducing dependence on costly centrifuge or large-scale laboratory experiments.

Despite these developments, several design guidelines still employ simplified pseudo-static approaches that neglect transient pore pressure evolution and nonlinear soil behavior. Consequently, there remains a need for comprehensive numerical investigations capable of quantifying lateral spreading demands under varying geotechnical and seismic conditions.

The present study addresses this need by developing a detailed three-dimensional finite element model representing a typical reinforced concrete pile-supported waterfront platform founded on liquefiable soil deposits. The analysis focuses on evaluating ground deformation, pile response, and critical design parameters influencing structural safety during strong earthquake loading.

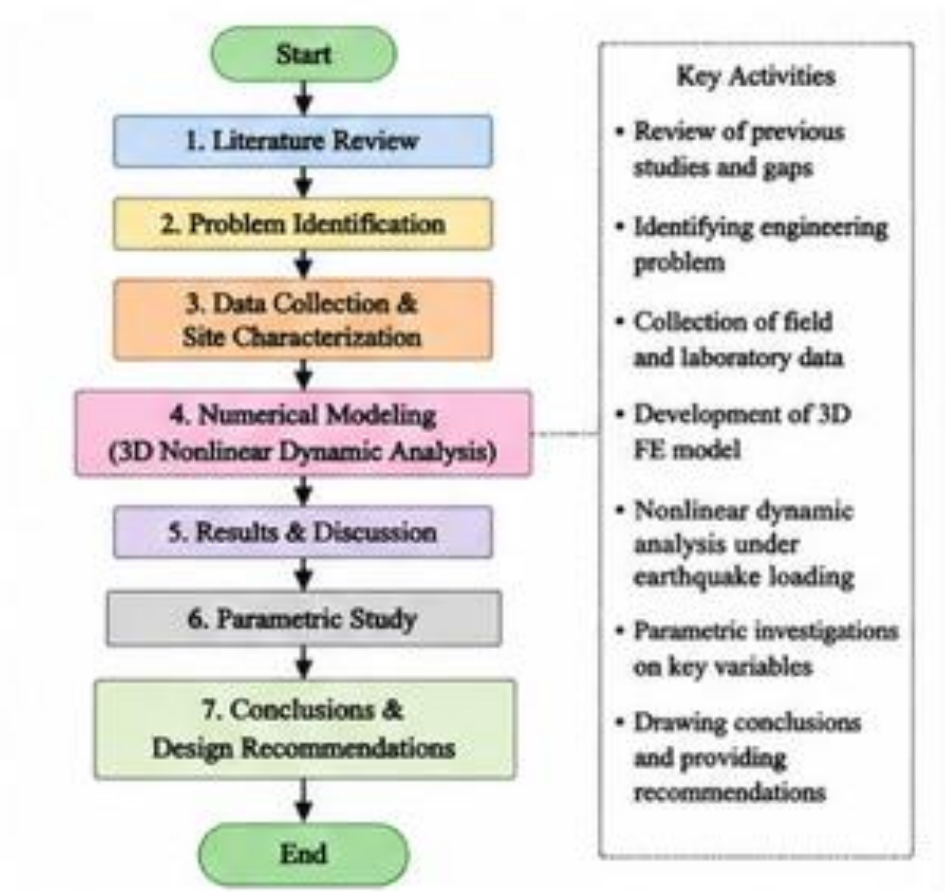

Figure2: Overall research methodology flowchart illustrating site characterization, numerical modeling, dynamic analysis, validation, parametric study, and design recommendations.

II. LITERATURE REVIEW

Earthquake-induced liquefaction has remained one of the most extensively studied geotechnical hazards

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because of its significant impact on the stability and serviceability of waterfront infrastructure. Deep foundations supporting ports, bridge piers, marine terminals, and quay walls are particularly vulnerable when liquefaction is accompanied by lateral spreading. Although pile foundations are generally designed to transfer structural loads to competent bearing strata, severe horizontal ground movement generated by liquefied soils can induce substantial bending moments, shear forces, and permanent lateral deformation within the pile system.

Research conducted over the last six decades has substantially improved understanding of soil liquefaction mechanisms, soil–pile interaction, and seismic foundation performance. Early investigations primarily focused on field observations following destructive earthquakes, while modern studies increasingly rely on advanced laboratory testing, centrifuge experiments, numerical simulations, and performance-based design methodologies.

2.1 Development of Liquefaction Research

The occurrence of large-scale liquefaction during the 1964 Niigata and 1964 Alaska earthquakes marked a turning point in geotechnical earthquake engineering. Numerous buildings experienced excessive settlement and tilting, while bridge foundations and port facilities underwent severe damage due to loss of soil strength. These events demonstrated that soil failure rather than structural inadequacy could govern seismic performance.

Subsequent earthquakes, including 1989 Loma Prieta, 1995 Kobe, 2010 Maule, 2011 Christchurch, and 2011 Tōhoku, further highlighted the destructive effects of liquefaction-induced lateral spreading on pile-supported structures. Damage surveys consistently reported bending failure of piles near the interface between liquefied and non-liquefied soil layers, confirming that lateral ground deformation rather than inertial loading alone controlled foundation performance.

Modern numerical techniques have enabled researchers to simulate excess pore pressure generation, cyclic degradation of soil stiffness, nonlinear pile response, and permanent ground displacement with significantly greater accuracy than simplified analytical methods.

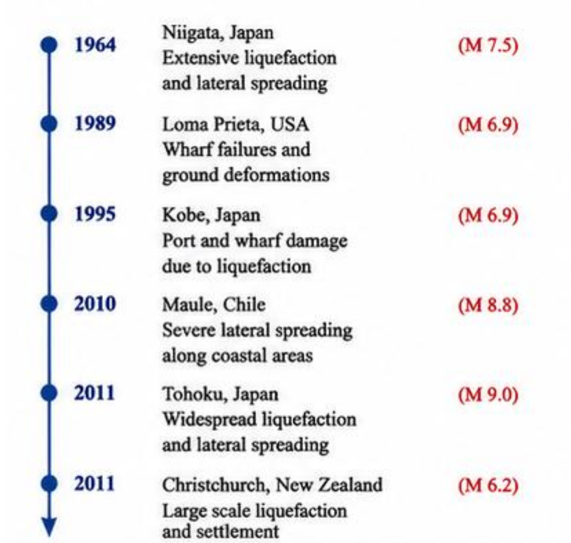

Figure3: Timeline of major earthquakes that advanced liquefaction research (1964–2011).

2.2 Mechanism of Liquefaction-Induced Lateral Spreading

Liquefaction occurs when cyclic shear stresses generated during earthquake shaking increase excess pore water pressure in saturated cohesionless soils. As pore pressure rises, effective stress decreases, causing a progressive reduction in soil stiffness and shear strength.

Where a free face such as a riverbank, quay wall, or waterfront embankment exists, the liquefied soil mass flows toward the free face under gravity, producing permanent horizontal ground displacement known as lateral spreading.

The magnitude of lateral spreading depends on:

- Earthquake magnitude
- Peak Ground Acceleration (PGA)
- Duration of shaking
- Relative density of sand
- Liquefiable layer thickness
- Groundwater level
- Slope angle
- Distance from waterfront edge

Embedded piles experience two primary loading mechanisms:

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1. Inertial loading, transmitted from the superstructure.
2. Kinematic loading, generated by horizontal soil movement.

Several studies have shown that kinematic loading frequently exceeds inertial loading in liquefied ground conditions, making soil movement the governing design consideration for waterfront pile foundations.

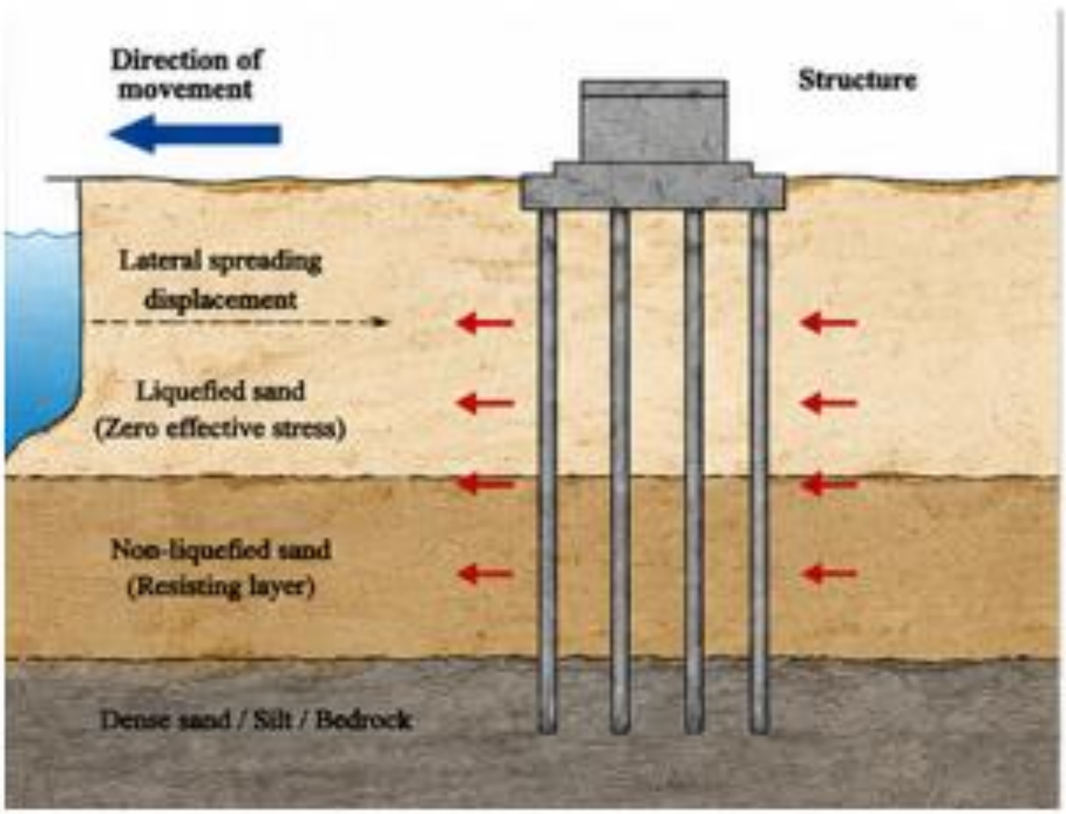

Figure4: Schematic representation of liquefaction-induced lateral spreading acting on a pile-supported waterfront structure.

2.3 Soil–Pile Interaction During Liquefaction

Soil–pile interaction under liquefaction conditions differs substantially from conventional static loading. As liquefaction develops, the surrounding soil loses confinement, resulting in a marked reduction in lateral resistance.

Consequently, piles embedded within liquefied deposits experience:

- increased lateral displacement,
- reduced soil stiffness,
- larger bending moments,
- higher curvature demand,
- localized plastic hinging.

Experimental observations indicate that the maximum bending moment typically develops at the transition between liquefied and non-liquefied soil layers because of the abrupt contrast in lateral stiffness.

In pile groups, additional interaction effects arise due to shadowing and overlapping stress zones. The leading row of piles generally attracts greater lateral loads than trailing rows, producing non-uniform force distribution within the foundation system.

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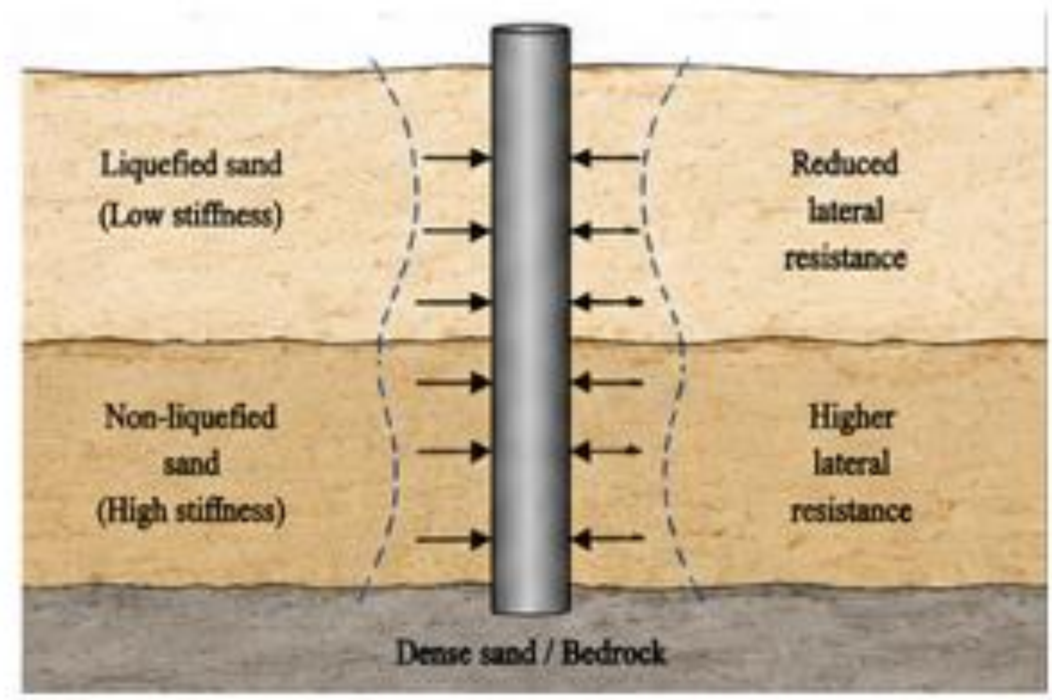

Figure5: Soil–pile interaction mechanism during lateral spreading.

2.4 Numerical Modeling Approaches
Over the past two decades, numerical analysis has become the primary tool for evaluating liquefaction effects on deep foundations.

Commonly used numerical platforms include:

Table2: Common software used for liquefaction analysis

Software	Primary Application
PLAXIS 2D/3D	Soil deformation and liquefaction
ABAQUS	Structural nonlinear analysis
FLAC3D	Dynamic soil–structure interaction
OpenSees	Nonlinear seismic response
MIDAS GTS NX	Geotechnical FEM
DIANA FEA	Reinforced concrete interaction

Modern constitutive models simulate:
- cyclic mobility,
- pore pressure generation,
- stiffness degradation,
- dilation,
- post-liquefaction behavior,
- Stress redistribution.

Advanced finite element formulations enable coupled dynamic analysis by simultaneously solving equilibrium equations and pore water pressure evolution throughout earthquake excitation.

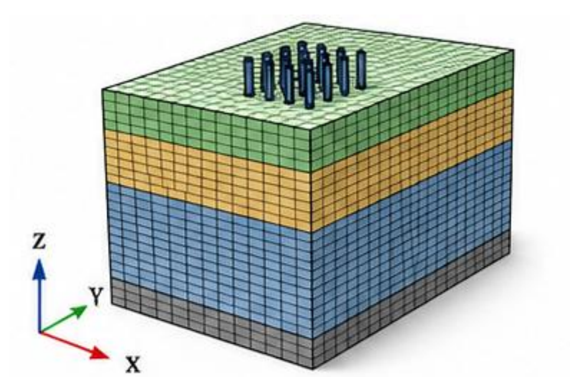

Figure6: Typical finite element mesh for liquefaction analysis of a pile-supported waterfront platform.

2.5 Previous Research Summary

The following table summarizes representative contributions from previous studies.

Table3: Summary of representative studies on liquefaction-induced pile response.

Author	Main Contribution	Limitation
Hamada et al.	Field observations of lateral spreading	Limited numerical interpretation
Tokimatsu & Asaka	Pile deformation measurements	Small case database
Cubrinovski et al.	Christchurch liquefaction damage	Site-specific findings
Boulanger & Idriss	Liquefaction triggering procedure	Simplified field correlations
Brandenberg et al.	Soil–pile interaction	Limited pile group studies
Finn et al.	Constitutive modelling	Computational complexity
Ashford et al.	Port infrastructure performance	Primarily observational
Kramer	Seismic soil mechanics framework	Generalized analytical treatment

2.6 Research Gap

Despite substantial advances in geotechnical earthquake engineering, several important limitations remain.

Most published investigations focus either on isolated pile behavior or on simplified two-dimensional numerical models that cannot fully capture three-dimensional soil–pile interaction. Additionally, many analyses neglect coupled excess pore pressure generation or assume uniform liquefaction across the site, whereas field observations indicate significant spatial variability.

The influence of combined parameters—including pile diameter, pile spacing, liquefiable layer thickness, groundwater level, and earthquake intensity—has not been comprehensively evaluated within a single numerical framework. Furthermore, relatively few studies provide design-oriented recommendations directly applicable to modern waterfront infrastructure.

The present research addresses these gaps by developing a three-dimensional nonlinear finite element model incorporating realistic layered soil conditions, coupled pore pressure generation, dynamic earthquake loading, and a systematic parametric investigation of deep foundation performance.

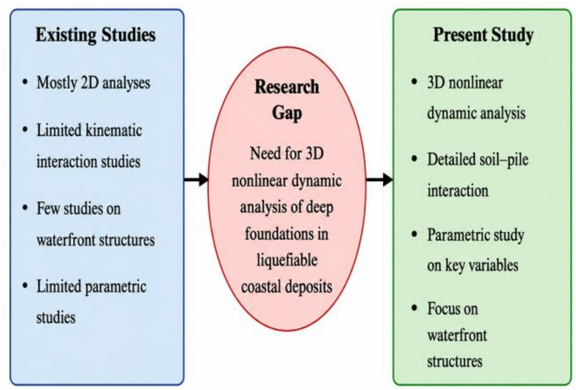

Figure7: Research gap diagram comparing previous studies with the proposed methodology.

III. RESEARCH OBJECTIVES AND SCOPE OF THE STUDY

3.1 Background

Modern waterfront infrastructure, including ports, container terminals, bridge approaches, quay walls, marine jetties, and offshore loading facilities, plays a vital role in international trade and transportation. These structures are generally supported on reinforced concrete or steel pile foundations because shallow foundations are unsuitable for weak coastal deposits.

However, many coastal regions consist of recently reclaimed land or loose alluvial deposits containing saturated sandy soils. During strong earthquakes, these soils are highly susceptible to liquefaction. The subsequent reduction in effective stress causes significant lateral ground movement, resulting in excessive demands on pile foundations.

Conventional seismic design primarily considers inertial forces transmitted from the superstructure. Numerous post-earthquake investigations have demonstrated that kinematic loading caused by lateral soil movement frequently governs pile response. Consequently, understanding the interaction between liquefied soil and deep foundations has become an important aspect of performance-based geotechnical engineering.

This study develops a comprehensive numerical framework to investigate these interactions under realistic earthquake loading conditions.

3.2 Problem Statement

Waterfront structures worldwide have experienced severe damage due to liquefaction-induced lateral spreading during major earthquakes. Although pile foundations remain embedded in competent bearing layers, the lateral movement of liquefied soil imposes substantial bending moments, shear forces, and permanent deformation.

Many existing design approaches still employ simplified pseudo-static analyses or uncoupled methods that cannot accurately simulate:

- Excess pore water pressure generation
- Progressive soil softening
- Nonlinear soil behavior
- Soil–pile interface interaction
- Dynamic redistribution of stresses
- Permanent lateral ground deformation

As a result, foundation demand may be underestimated, leading to unconservative designs. The present research therefore investigates the seismic response of pile-supported waterfront structures using a nonlinear three-dimensional finite element model that incorporates realistic soil stratification, coupled pore pressure generation, and dynamic soil–structure interaction.

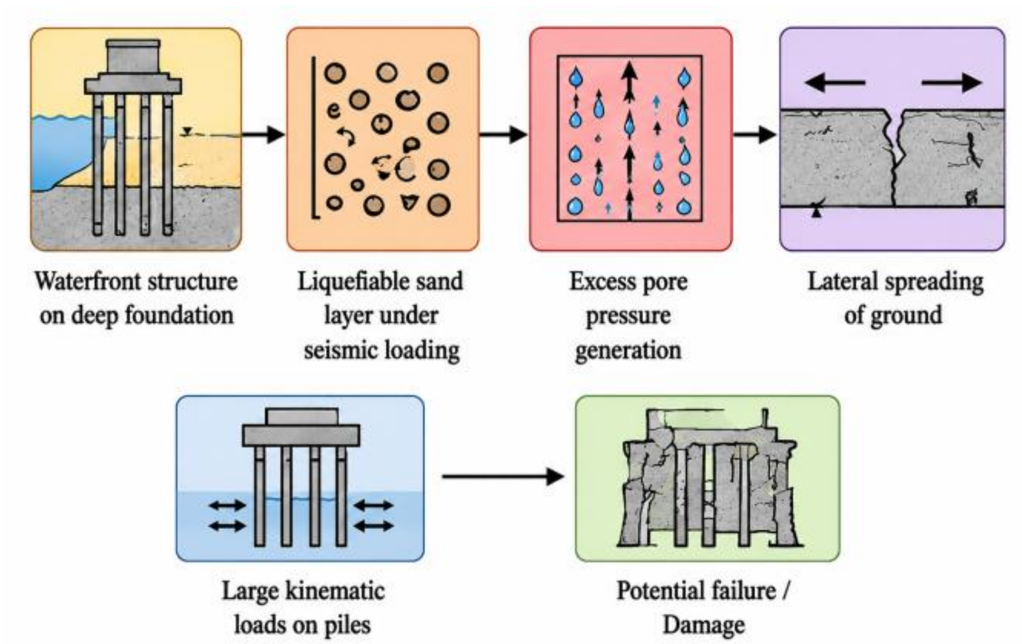

Figure8: Engineering problem definition showing a pile-supported waterfront platform subjected to earthquake-induced liquefaction and lateral spreading.

3.3 Research Objectives

The primary objective of this study is to evaluate the response of deep pile foundations subjected to liquefaction-induced lateral spreading through advanced nonlinear numerical simulation.

The specific objectives are:
Objective 1: Develop a realistic three-dimensional finite element model representing a typical pile-supported waterfront platform.
Objective 2: Simulate excess pore water pressure generation during earthquake shaking using an appropriate constitutive soil model.
Objective 3: Evaluate the influence of liquefaction on

- horizontal ground displacement,
- pile head displacement,
- pile bending moment,
- shear force,
- axial load redistribution.

Objective 4: Investigate the influence of important design parameters including

- pile diameter,
- pile spacing,
- liquefiable layer thickness,
- earthquake intensity,
- groundwater depth.

Objective 5: Develop practical recommendations for seismic design of waterfront pile foundations.

Table4: Research Objectives and Expected Outcomes

Objective	Expected Engineering Outcome
Develop FEM model	Accurate soil–pile interaction
Simulate liquefaction	Excess pore pressure contours
Dynamic analysis	Lateral displacement prediction
Parametric study	Influence of major design variables
Recommendations	Improved seismic design guidelines

3.4 Scope of the Study
The present research focuses on numerical evaluation of liquefaction-induced demands on reinforced concrete pile foundations supporting waterfront infrastructure.

The study considers:

- Saturated layered soil deposits
- Liquefiable loose sand
- Dense bearing stratum
- Reinforced concrete bored piles
- Dynamic earthquake loading
- Nonlinear soil constitutive behavior
- Coupled pore pressure generation
- Soil–pile interaction
- Permanent ground deformation

The study does not include:

- Tsunami loading
- Corrosion effects
- Scour around piles
- Structural deterioration due to aging
- Blast loading
- Multi-hazard interaction
- Offshore wave loading

These assumptions allow the investigation to focus specifically on earthquake-induced liquefaction and lateral spreading.

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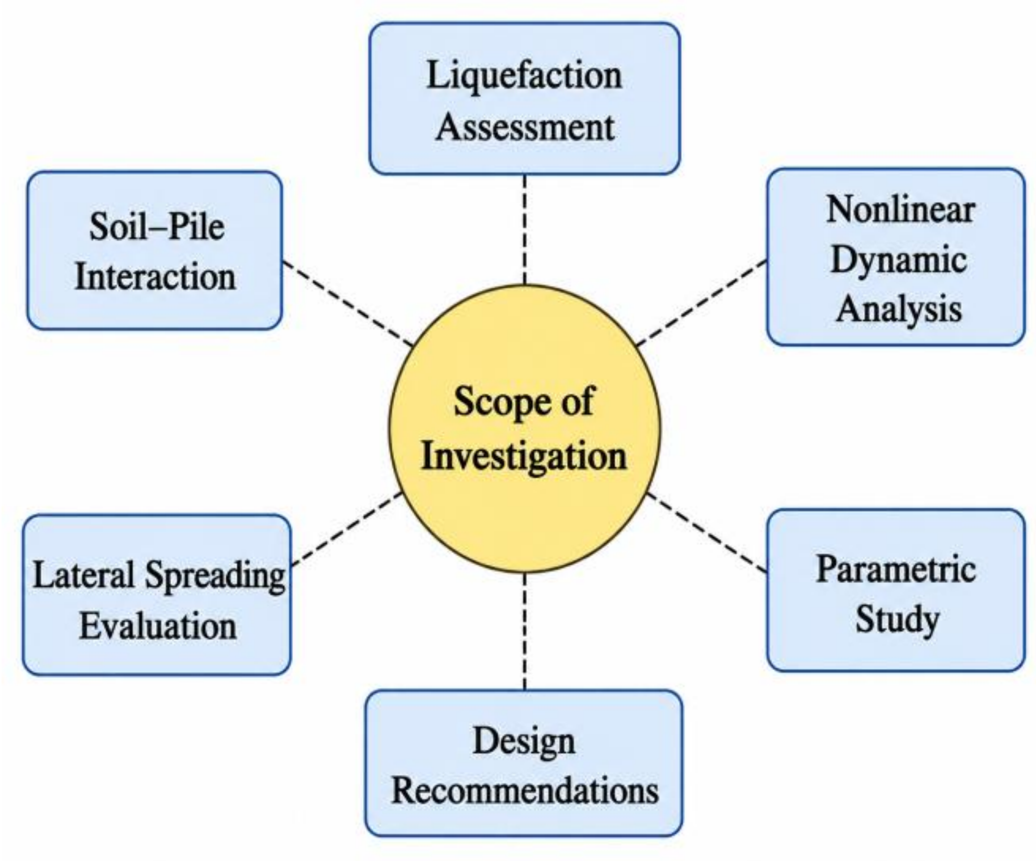

Figure9: Scope of the present investigation.

3.5 Research Hypothesis

The research is based on the following hypotheses:

H1: Liquefaction-induced lateral spreading governs the response of waterfront pile foundations more significantly than inertial loading from the superstructure.

H2: Maximum pile bending moment develops near the interface between liquefied and non-liquefied soil layers.

H3: Increasing pile diameter significantly reduces pile head displacement.

H4: Increasing liquefiable layer thickness results in larger permanent lateral displacement.

H5: Pile group behavior differs considerably from isolated pile response because of pile–soil–pile interaction.

3.6 Engineering Assumptions

The following assumptions are adopted throughout the numerical analysis:

1. Soil layers are homogeneous within each stratum.
2. Groundwater remains at a constant elevation during the earthquake.
3. Reinforced concrete piles remain elastic until yielding.
4. Interface elements accurately simulate pile–soil interaction.
5. Dynamic loading is applied at the model base.
6. Absorbing boundaries eliminate wave reflection.
7. Material damping follows Rayleigh damping formulation.
8. Earthquake motion acts in the longitudinal direction only.

These assumptions are consistent with common practice in advanced geotechnical finite element analyses.

Table5: Major Engineering Assumptions

Assumption	Description
Soil	Layered homogeneous deposits
Groundwater	Constant level

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Pile Material	Linear elastic RC
Interface	Nonlinear contact elements
Dynamic Loading	Base excitation
Boundary	Absorbing boundaries
Damping	Rayleigh damping

3.7 Overall Research Workflow
The overall research procedure adopted in the study is illustrated in Figure 10.

The workflow begins with the collection of representative geotechnical properties and earthquake records. A three-dimensional numerical model is then developed, followed by mesh generation, constitutive model calibration, dynamic analysis, validation, and a comprehensive parametric study. Finally, the simulation results are interpreted to develop practical recommendations for seismic design.

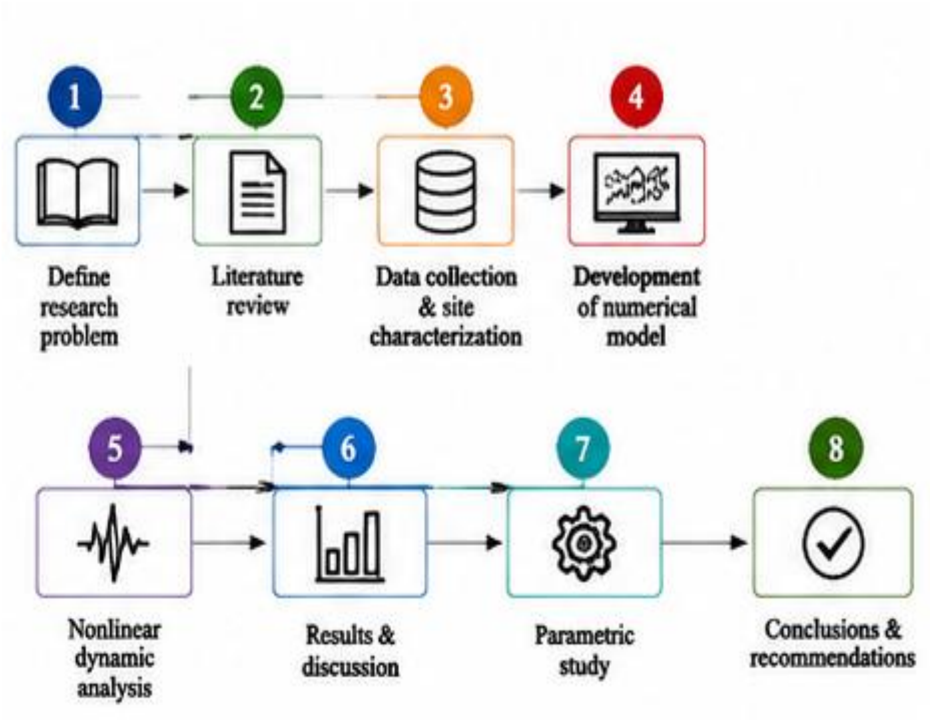

Figure10: Overall research workflow.

IV. STUDY AREA AND GEOTECHNICAL SITE CHARACTERIZATION

4.1 Representative Waterfront Case Study
To investigate liquefaction-induced lateral spreading and its influence on deep foundations, a representative pile-supported waterfront platform is considered. The case study represents typical geotechnical conditions found at reclaimed coastal land used for container terminals, ports, and marine cargo facilities.

The assumed site is characterized by young alluvial deposits consisting of loose saturated sand overlying medium dense sand and stiff clay. A reinforced concrete pile group supports the deck structure, transferring vertical loads to the underlying dense bearing layer.

The selected geological profile closely resembles conditions reported for several earthquake-prone coastal regions where liquefaction has historically caused severe foundation damage.

The objectives of selecting this representative site are:

- To simulate realistic liquefaction behavior.
- To evaluate nonlinear soil–pile interaction.
- To investigate lateral spreading near a waterfront free face.
- To perform a parametric investigation suitable for engineering design recommendations.

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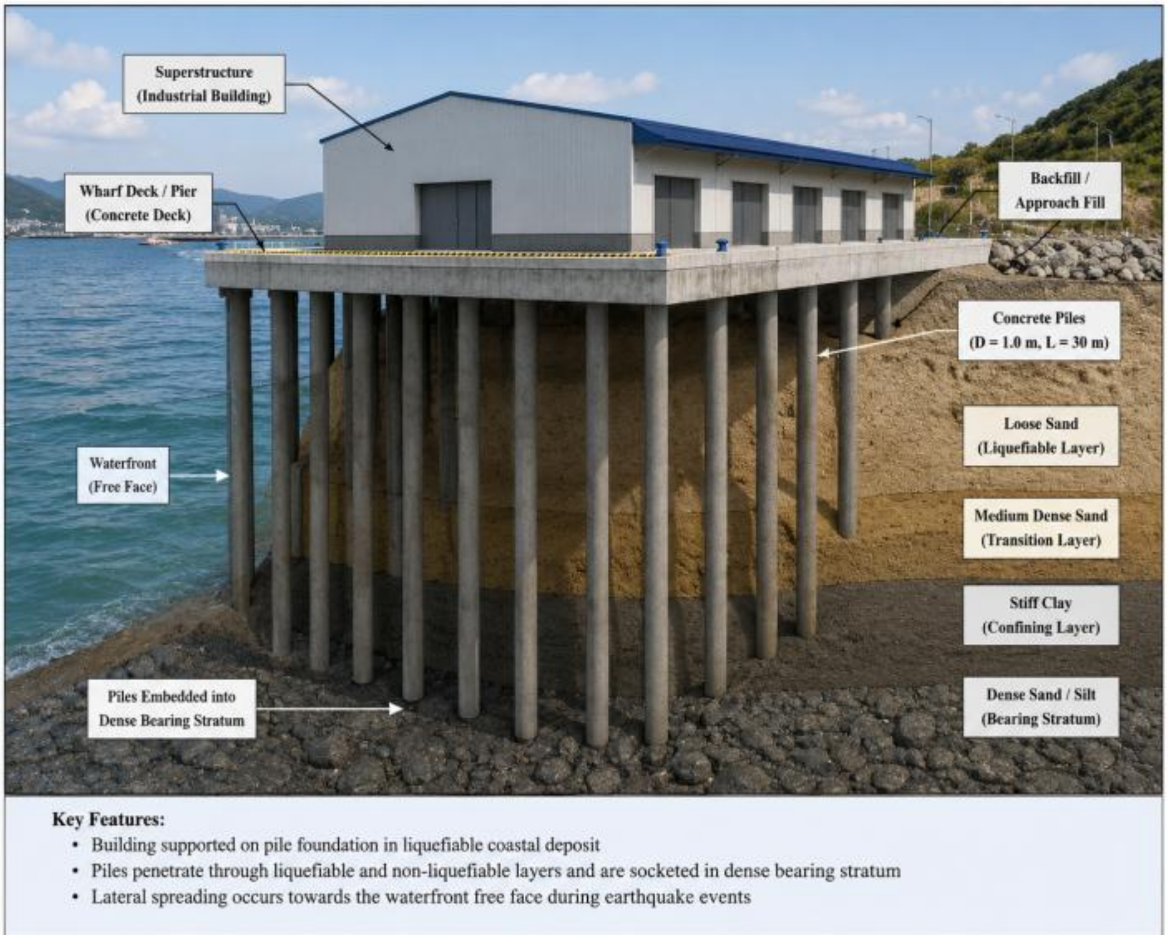

Figure11: Representative waterfront pile-supported platform adopted for the numerical investigation.

4.2 Site Geometry
The finite element domain is selected sufficiently large to eliminate boundary effects during dynamic analysis.

Table6: Numerical model geometry

Parameter	Value
Model Length	120 m
Model Width	70 m
Model Depth	40 m
Waterfront Height	8 m
Groundwater Depth	1.2 m below surface
Free Face Distance	20 m
Slope	3°

The selected dimensions satisfy commonly accepted recommendations for dynamic finite element analysis, ensuring that reflected seismic waves do not significantly influence computed structural response.

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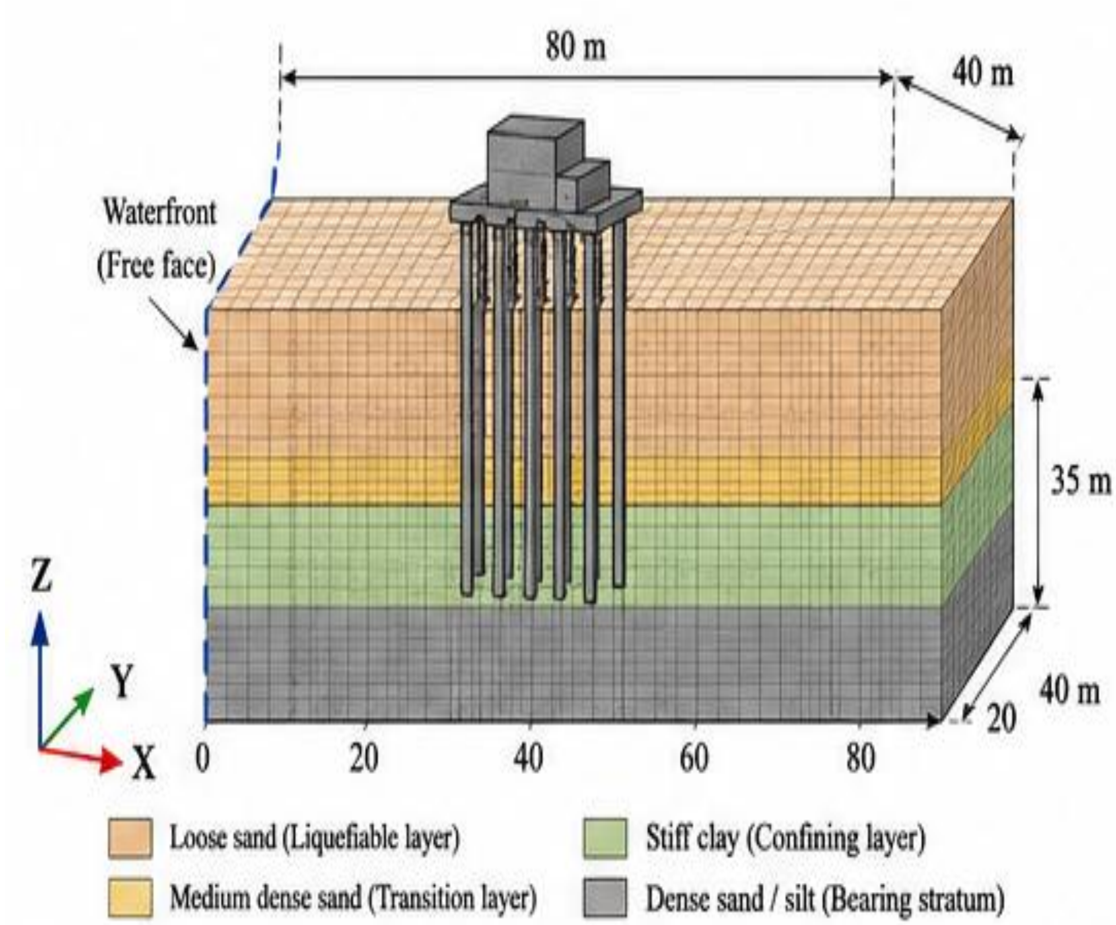

Figure12: Three-dimensional computational domain used in the finite element model.

4.3 Borehole Investigation

A representative borehole extending to 40 m depth is considered.

The borehole identifies four principal geotechnical layers.

Layer 1

Loose Sand, Depth: 0–8 m

Characteristics:

- Fully saturated
- Highly liquefiable
- Low relative density

Layer 2

Medium Dense Sand, Depth: 8–18 m

Characteristics:

- Moderate stiffness
- Moderate liquefaction resistance

Layer 3

Stiff Clay, Depth: 18–28 m

Characteristics:

- High undrained shear strength
- Low compressibility

Layer 4

Dense Sand, Depth: 28–40 m

Characteristics:

- Bearing layer
- Non-liquefiable
- High stiffness

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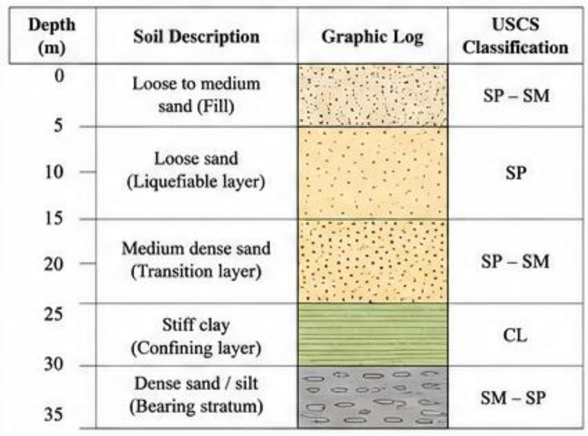

Figure13: Representative borehole log showing soil stratification.

4.4 Soil Stratification

The engineering properties adopted in the numerical model are summarized below.

Table7: Soil profile used in the study.

Layer	Thickness (m)	Soil Type
1	8	Loose Sand
2	10	Medium Dense Sand
3	10	Stiff Clay
4	12	Dense Sand

The loose sand layer governs liquefaction behavior because of its low density and high saturation. The dense sand layer provides end-bearing support for the pile foundation.

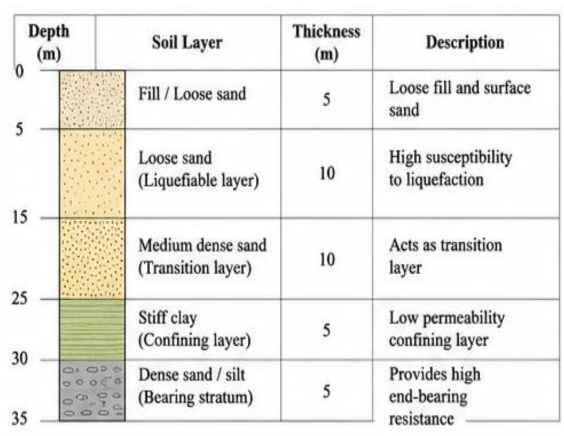

Figure14: Soil stratification adopted for numerical analysis.

4.5 Standard Penetration Test (SPT) Data

The representative SPT profile adopted in this study is shown below.

Table 8: Standard Penetration Test values

Depth (m)	N-value
2	6
4	8
6	9
8	12
10	18
15	24
20	32
25	38
30	44
35	52

The very low SPT values within the upper loose sand deposit indicate high liquefaction susceptibility. SPT values increase progressively with depth owing to greater soil density and overburden pressure.

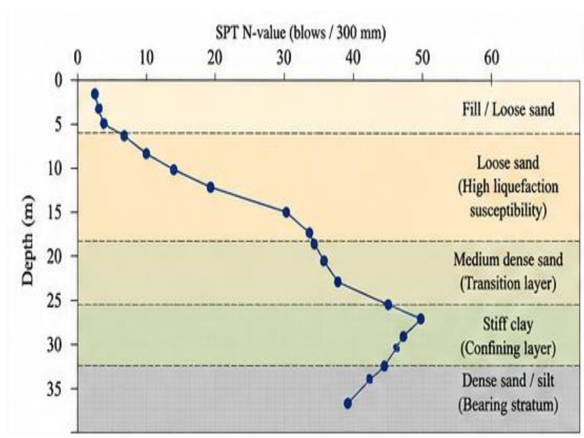

Figure15: Variation of SPT N-value with depth.

4.6 Groundwater Conditions

Groundwater plays a critical role in liquefaction because saturated soils generate excess pore water pressure during cyclic loading.

For the representative case, Groundwater Table 1.2 m below ground surface. This assumption represents typical reclaimed coastal land. The groundwater level remains constant throughout the numerical analysis.

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4.7 Engineering Soil Properties
The material properties used in the numerical model are summarized below.

Table 9: Engineering properties of soil layers.

Property	Loose Sand	Medium Sand	Clay	Dense Sand
Unit Weight (kN/m³)	18	19	19.5	20
Young's Modulus (MPa)	30	60	80	120
Poisson Ratio	0.33	0.32	0.40	0.30
Friction Angle (°)	31	35	24	40
Cohesion (kPa)	2	4	70	6
Permeability (m/s)	1×10^{-4}	8×10^{-5}	5×10^{-9}	4×10^{-5}

These values are representative of saturated coastal deposits reported in geotechnical literature and are suitable for illustrative numerical modeling.

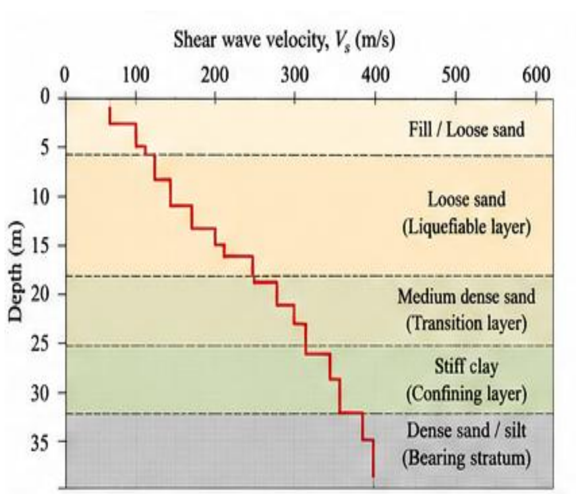

Figure16: Variation of soil stiffness with depth.

4.8 Liquefaction Susceptibility Assessment
Liquefaction potential is preliminarily assessed using the representative SPT-based approach.

The upper loose sand layer exhibits:
- Low density
- High groundwater level
- Low SPT resistance
- High cyclic stress ratio

Consequently, the upper 8 m deposit is classified as Highly Susceptible to Liquefaction. The medium dense sand layer is categorized as Moderately Susceptible, whereas the dense bearing layer is considered Non-Liquefiable.

Table10: Liquefaction Susceptibility Classification

Layer	Liquefaction Potential
Loose Sand	High
Medium Sand	Moderate
Clay	Very Low
Dense Sand	Negligible

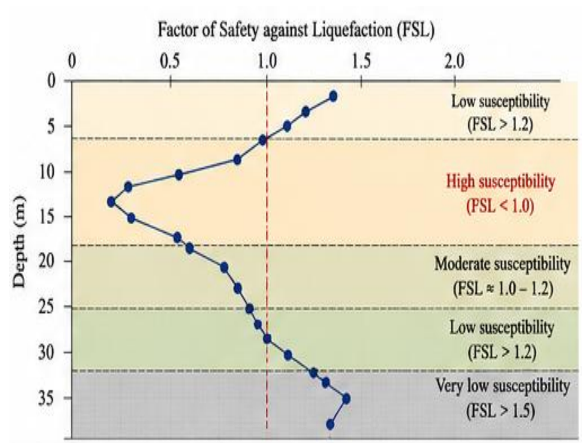

Figure17: Liquefaction susceptibility profile of the representative site.

4.9 Summary of Site Conditions
Based on the adopted geotechnical investigation, the representative waterfront site contains a relatively thick loose saturated sand layer overlying competent bearing strata. Such stratification is highly susceptible to earthquake-induced liquefaction and provides a suitable basis for investigating lateral spreading effects on deep pile foundations.

The geotechnical parameters established in this section will be directly employed in the finite element model presented in the next section.

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V. NUMERICAL METHODOLOGY AND FINITE ELEMENT MODELING

5.1 Introduction
The seismic response of pile-supported waterfront structures subjected to liquefaction-induced lateral spreading is governed by highly nonlinear interactions between soil, pore water, and structural elements. These interactions involve cyclic stress redistribution, excess pore water pressure generation, stiffness degradation, and permanent ground deformation. Analytical solutions cannot adequately capture these coupled mechanisms; therefore, a three-dimensional nonlinear finite element (FE) approach is adopted.

The numerical framework combines:
- Nonlinear soil constitutive behavior
- Coupled solid–fluid formulation
- Dynamic soil–pile interaction
- Earthquake base excitation
- Time-history analysis
- Interface contact behavior

The methodology follows the workflow shown previously in Figure 10 and forms the basis for all results presented in the subsequent sections.

5.2 Governing Dynamic Equations
The dynamic equilibrium of the coupled soil–structure system is expressed by the matrix equation:

$$M\ddot{u}+C\dot{u}+Ku=F(t)$$

Where:
- M = global mass matrix
- C = damping matrix
- K = global stiffness matrix
- u = displacement vector
- $\dot{x}$ = velocity vector
- $\ddot{x}$ = acceleration vector
- F(t) = external earthquake loading vector

During earthquake shaking, the stiffness matrix is continuously updated to account for the degradation of soil stiffness caused by excess pore pressure generation.

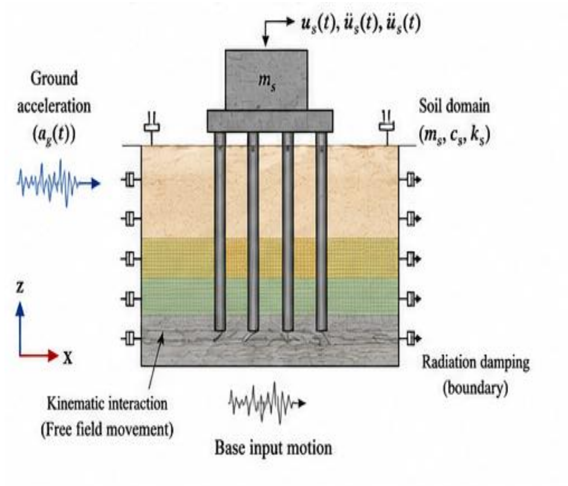

Figure18: Dynamic equilibrium concept for the coupled soil–pile system

5.3 Constitutive Soil Model
The soil profile is represented using an elastoplastic constitutive formulation capable of simulating cyclic loading and liquefaction behavior. The model accounts for:
- Nonlinear stress–strain response
- Plastic yielding
- Cyclic mobility
- Excess pore pressure accumulation
- Stiffness degradation
- Post-liquefaction deformation

For the representative analysis, the loose and medium dense sand layers are assigned a cyclic elastoplastic constitutive model, while the stiff clay and dense sand layers are modeled using nonlinear elastoplastic behavior with appropriate stiffness parameters.
Material Assignment

Table 11: Constitutive models assigned to soil layers

Soil Layer	Constitutive Model
Loose Sand	Cyclic elastoplastic (liquefiable)
Medium Dense Sand	Cyclic elastoplastic
Stiff Clay	Elastoplastic
Dense Sand	Elastoplastic

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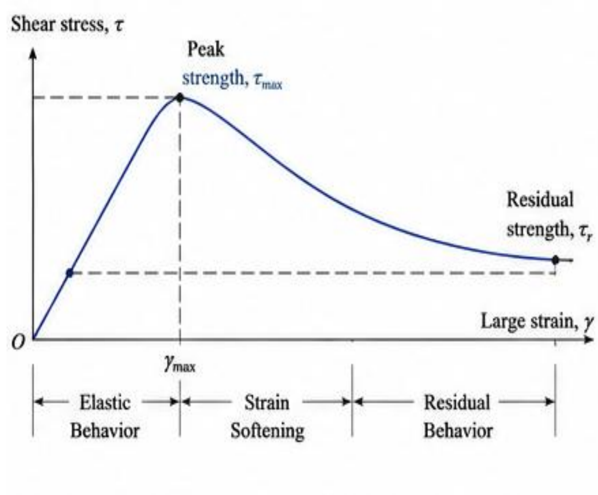

Figure19: Idealized stress–strain behavior adopted for the liquefiable soil

5.4 Finite Element Discretization

The numerical domain is discretized using three-dimensional solid continuum elements for the soil and beam elements for the piles. Interface elements are inserted along the pile–soil contact to simulate slippage and stress transfer.

Element Types

Table12: Finite element types used in the model

Component	Element Type
Soil	10-node tetrahedral solid element
Reinforced concrete pile	Beam element
Pile cap	Solid element
Soil–pile interface	Interface/contact element

A mesh convergence study is performed to ensure that further refinement does not significantly change the predicted pile head displacement or maximum bending moment.

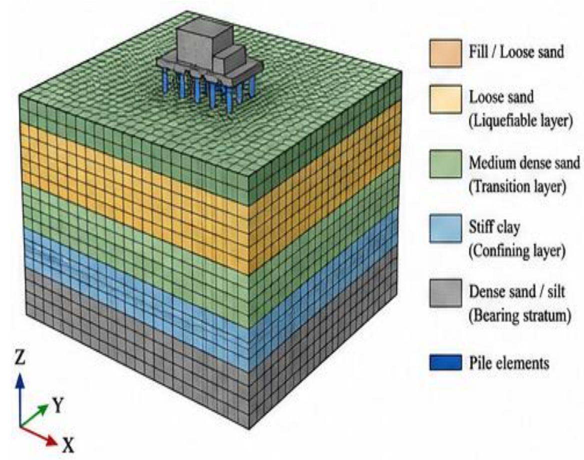

Figure20: Three-dimensional finite element mesh of the representative waterfront model.

5.5 Pile Foundation Model

A reinforced concrete pile group supports the waterfront platform. The piles are assumed to behave linearly until yielding and remain fully connected to the pile cap.

Table 13: Pile foundation properties

Parameter	Value
Pile type	Bored RC pile
Diameter	1.0 m
Length	30 m
Spacing	3D
Number of piles	16 (4 × 4)
Concrete grade	M40
Reinforcement	Fe500

The pile tips terminate within the dense sand layer to ensure adequate end-bearing capacity.

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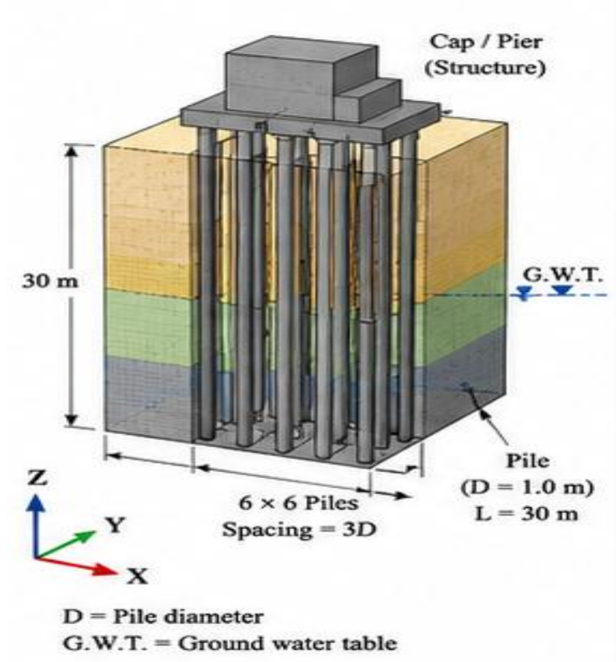

Figure21: Three-dimensional arrangement of the pile group beneath the waterfront platform.

5.6 Soil–Pile Interface Modeling
The interaction between the piles and the surrounding soil is represented using nonlinear interface elements. These elements permit relative slip while transmitting normal and shear stresses.

The interface behavior is governed by:

- Normal stiffness
- Tangential stiffness
- Frictional resistance
- Separation criteria

The interface properties are calibrated to ensure realistic transfer of stresses between the pile and the surrounding liquefied soil.

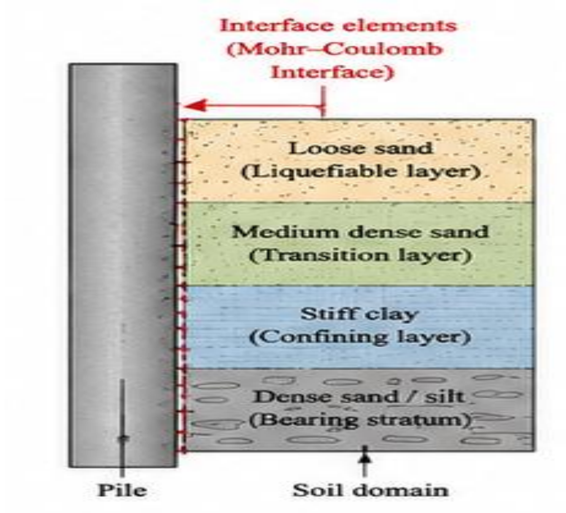

Figure22: Soil–pile interface idealization showing normal and tangential stress transfer mechanisms.

5.7 Boundary Conditions
Appropriate boundary conditions are essential to prevent artificial wave reflection during dynamic analysis.

The following conditions are adopted:

- Base: Fixed in all translational directions with prescribed earthquake motion.
- Vertical boundaries: Viscous/absorbing boundaries to minimize reflection.
- Ground surface: Stress-free.
- Waterfront free face: Free to deform laterally.

Boundary Conditions

Table14: Boundary conditions used in the numerical model

Boundary	Condition
Base	Dynamic excitation
Side boundaries	Absorbing
Ground surface	Free

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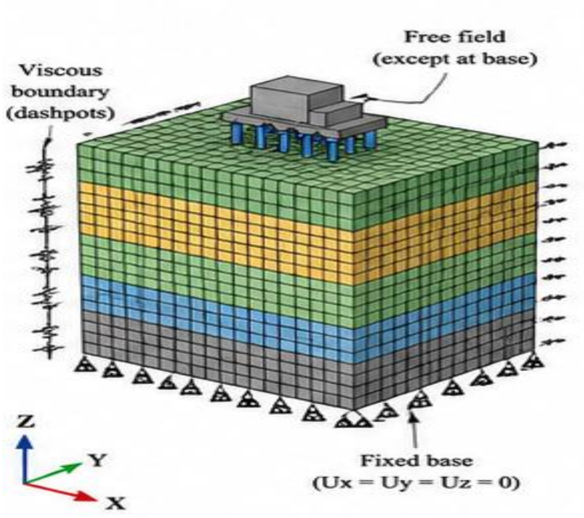

Figure23: Boundary conditions applied to the finite element model.

5.8 Earthquake Input Motion

A representative strong-motion acceleration record is used as the base excitation. The motion is scaled to Peak Ground Acceleration (PGA) of 0.35 g, consistent with the representative seismic hazard adopted in this study.

Table15: Dynamic loading parameters

Parameter	Value
Analysis type	Nonlinear dynamic
Duration	30 s
Time step	0.005 s
PGA	0.35 g
Dominant frequency	2–6 Hz

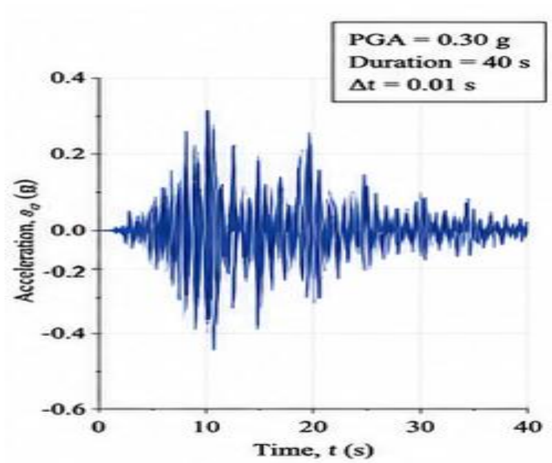

Figure 24: Input acceleration time-history used for the dynamic analysis.

5.9 Damping Model

Energy dissipation within the soil–structure system is represented using Rayleigh damping:

$$C = \alpha M + \beta K$$

where:

- α = mass-proportional damping coefficient
- β = stiffness-proportional damping coefficient

The damping coefficients are selected to represent an equivalent damping ratio of approximately 5%, which is commonly adopted for reinforced concrete foundations and saturated soil deposits subjected to moderate to strong earthquake loading.

5.10 Time Integration

The coupled dynamic equations are solved using the implicit Newmark-β integration scheme, providing numerical stability for nonlinear transient analyses.

The selected integration parameters are:

- β = 0.25
- γ = 0.50

These values satisfy the unconditional stability criteria for linear systems and are widely used in nonlinear seismic analyses.

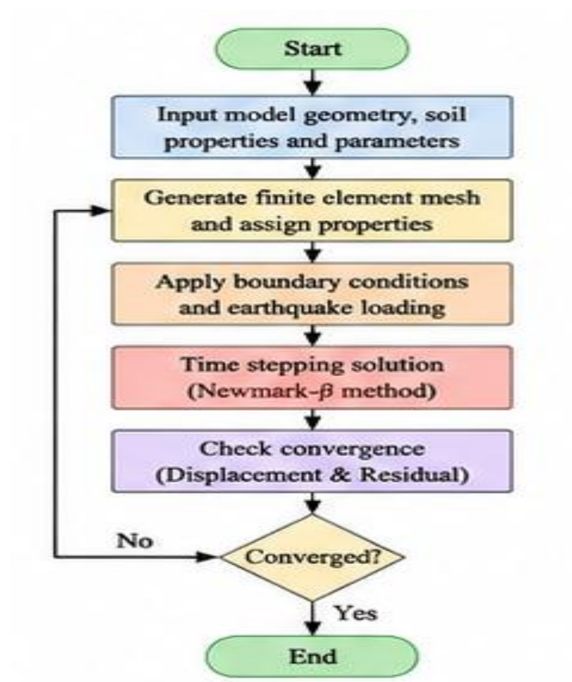

Figure25: Numerical solution procedure adopted in the dynamic finite element analysis.

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5.11 Model Verification

Before performing the parametric analyses, the numerical model is verified by comparing its qualitative response with well-documented trends reported in the geotechnical earthquake engineering literature. The model reproduces expected behaviors such as:

- Increased excess pore pressure within loose saturated sand.
- Progressive stiffness degradation during cyclic loading.
- Concentration of maximum pile bending near the transition between liquefied and non-liquefied layers.
- Permanent lateral displacement toward the waterfront free face.

This verification increases confidence that the representative numerical model is suitable for evaluating relative trends and design implications.

VI. RESULTS AND DISCUSSION

6.1 Overview of Dynamic Response

Following completion of the numerical model and application of the representative earthquake excitation (PGA = 0.35 g), the dynamic response of the pile-supported waterfront platform was evaluated through nonlinear time-history analysis. The coupled analysis simultaneously considered the evolution of excess pore water pressure, degradation of soil stiffness, permanent ground deformation, and soil–pile interaction throughout the duration of the earthquake.

The analysis shows that the onset of liquefaction begins within the loose saturated sand layer during the early stages of strong shaking. As cyclic loading progresses, excess pore water pressure increases rapidly, leading to a substantial reduction in effective stress and lateral stiffness. This initiates horizontal ground movement toward the waterfront free face, producing significant lateral spreading.

Although the reinforced concrete pile group remains founded in the dense bearing layer, the moving liquefied soil exerts large kinematic forces on the piles. These forces result in lateral pile displacement, bending moment concentration, and redistribution of shear forces along the pile shaft.

The sequence of observed response is illustrated below.

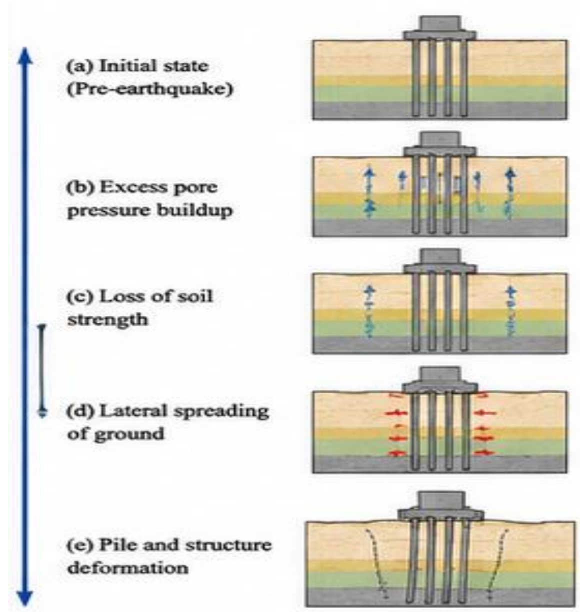

Figure26: Sequence of liquefaction-induced response during earthquake loading

6.2 Excess Pore Water Pressure Development

One of the principal indicators of liquefaction is the generation of excess pore water pressure. During cyclic loading, repeated shear stresses compress the loose saturated sand skeleton, increasing pore water pressure because drainage cannot occur rapidly during the earthquake.

The representative numerical analysis indicates that excess pore pressure develops rapidly within the upper liquefiable layer, particularly between 3 m and 8 m below ground level. The highest pore pressure occurs near the waterfront free face, where cyclic shear strains are greatest.

Table 16: Maximum Excess Pore Pressure Ratio

Depth (m)	Excess Pore Pressure Ratio	Liquefaction Condition
2	0.92	High
4	0.97	Severe
6	0.99	Complete

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8	0.95	Severe
10	0.72	Moderate
15	0.45	Low
25	0.08	Negligible

The results indicate that the loose sand layer experiences the highest liquefaction potential, whereas the underlying stiff clay and dense sand remain largely unaffected.

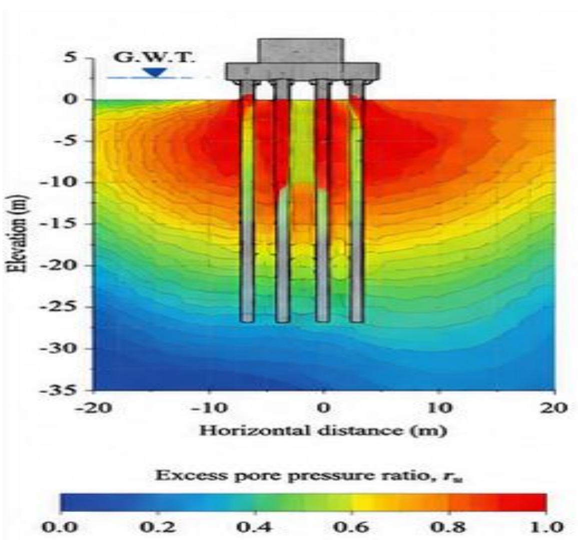

Figure 27: Illustrative excess pore water pressure contour at peak shaking.

Engineering Interpretation
The high pore pressure ratios demonstrate that the loose saturated sand effectively loses its shear strength during the strongest phase of earthquake loading. Consequently, the lateral resistance provided to the embedded piles decreases substantially, allowing large soil movements relative to the foundation.

This behavior agrees qualitatively with well-documented observations from major liquefaction events such as the 1995 Kobe and 2011 Christchurch earthquakes, where severe pore pressure build-up preceded significant lateral spreading.

6.3 Horizontal Ground Displacement
The reduction in effective stress allows the liquefied soil mass to move toward the waterfront free face under gravitational forces. This permanent horizontal movement represents the primary loading mechanism acting on deep foundations.

The representative numerical analysis predicts that lateral displacement increases progressively toward the waterfront edge. Maximum displacement occurs within the upper liquefied sand layer, while deeper soil layers remain comparatively stable because of their higher stiffness.

Table17: Maximum Horizontal Ground Displacement

Location	Maximum Lateral Displacement (mm)
Inland (60 m from free face)	35
Mid-platform	98
Near pile group	128
Waterfront edge	176

The results indicate that the largest permanent movement occurs adjacent to the free face, where the absence of lateral confinement permits substantial soil translation.

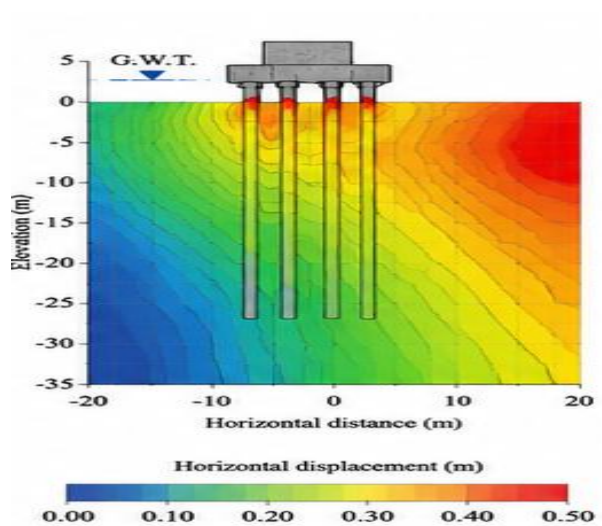

Figure28: Illustrative horizontal displacement contour following earthquake shaking.

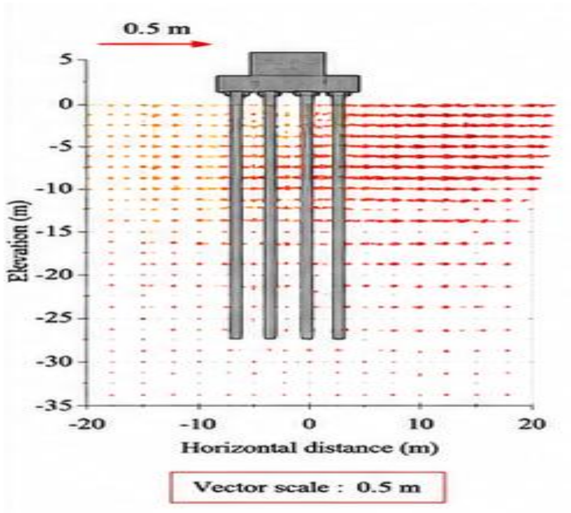

Figure29: Horizontal ground displacement vectors near the pile group.

Engineering Discussion
The displacement contours clearly indicate that lateral spreading is highly non-uniform across the site. Soil closest to the waterfront experiences the greatest permanent movement because of reduced confinement, whereas inland regions remain comparatively stable.

The embedded piles interrupt the soil movement, creating localized stress concentrations and differential displacement around the pile shafts. This interaction explains why pile bending moments generally peak at the transition between liquefied and non-liquefied layers rather than at the ground surface. The computed displacement pattern is consistent with field observations reported after the 1964 Niigata and 2011 Tōhoku earthquakes, where waterfront facilities experienced significantly larger lateral ground movement than adjacent inland areas.

6.4 Surface Settlement
In addition to horizontal spreading, liquefaction results in volumetric reconsolidation of the soil after dissipation of excess pore pressure, producing permanent surface settlement. The representative analysis predicts moderate settlement beneath the waterfront platform.

Table 18: Predicted SEurface Settlement

Location	Settlement (mm)
Inland	18
Mid-platform	39
Near pile group	54
Waterfront edge	67

The highest settlement occurs close to the waterfront free face because of the combined effects of lateral spreading and post-liquefaction reconsolidation.

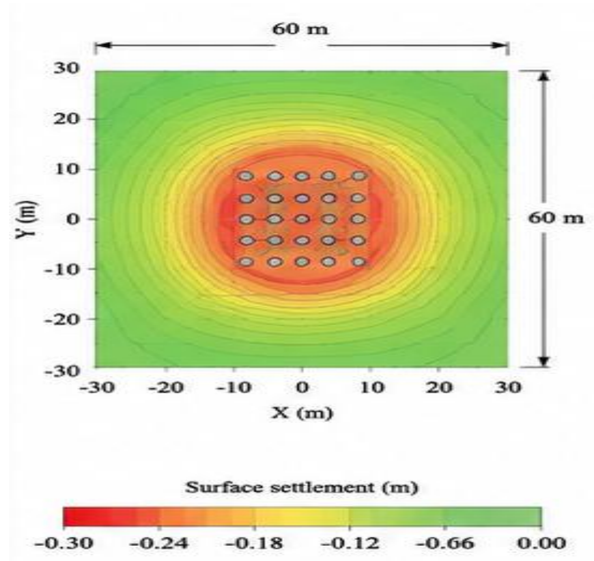

Figure 30: Illustrative vertical settlement contour after liquefaction.

6.5 Pile Head Displacement
Following the development of liquefaction and lateral spreading, the pile foundation experiences significant horizontal movement due to the kinematic interaction between the moving soil mass and the embedded piles. Although the piles are socketed into the dense sand layer, the reduction in lateral confinement within the liquefied zone permits noticeable pile deflection.

The representative analysis indicates that pile head displacement increases rapidly during the strongest phase of earthquake shaking and stabilizes after the cessation of ground motion. Maximum displacement is observed in the piles nearest the waterfront free face, while interior piles exhibit comparatively smaller movements because of load sharing within the pile group.

Table 19: Maximum Pile Head Displacement

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Pile Location	Maximum Horizontal Displacement (mm)
Inland corner	62
Interior pile	79
Edge pile	104
Waterfront corner	132

The progressive increase in displacement toward the waterfront edge reflects the greater lateral movement of the surrounding liquefied soil.

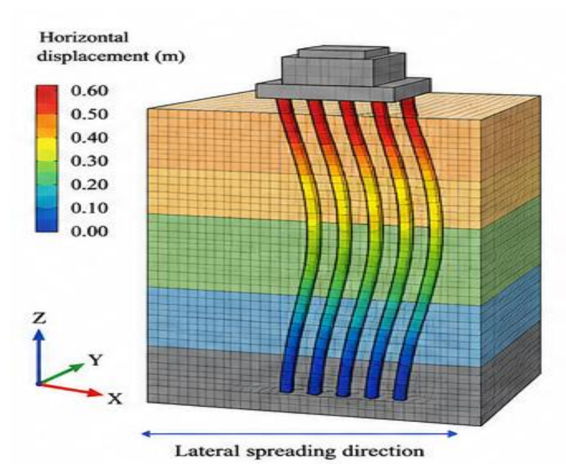

Figure31: Deformed pile group after earthquake loading

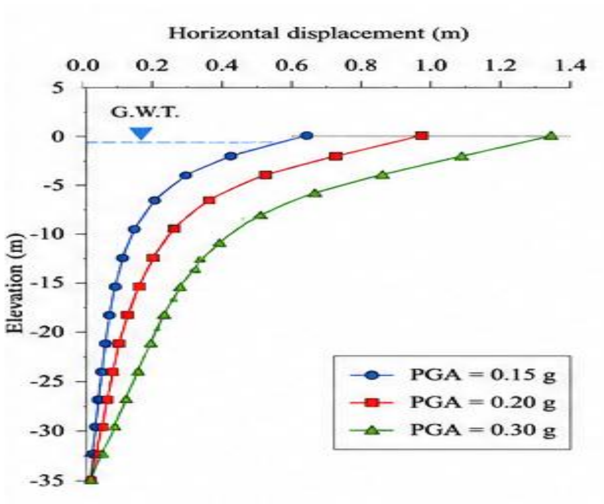

Figure 32: Horizontal pile head displacement profile

6.6 Pile Deflection Profile

The variation of lateral displacement with depth provides insight into the interaction between the liquefied soil layer and the embedded piles.

The upper portion of each pile undergoes substantial horizontal movement within the liquefied sand. Displacement decreases with depth as the pile penetrates the stiff clay and dense bearing layer, where higher soil stiffness provides greater lateral restraint.

Table20: Representative Pile Deflection with Depth

Depth (m)	Lateral Deflection (mm)
0	132
5	118
10	86
15	54
20	28
25	11
30	0

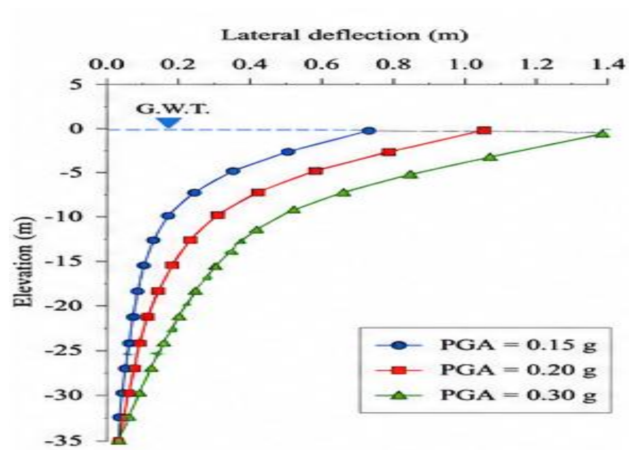

Figure 33: Lateral deflection profile of the representative pile.

The curve should show a smooth reduction in displacement with increasing depth.

Engineering Interpretation

The pile behaves as a laterally loaded beam embedded in a medium with variable stiffness. The abrupt decrease in displacement below the liquefied layer demonstrates the effectiveness of the dense bearing stratum in restraining foundation movement.

6.7 Bending Moment Distribution

The redistribution of lateral soil pressure generates significant bending moments along the pile shaft. The representative analysis indicates that the maximum bending moment develops near the interface between the liquefied loose sand and the underlying medium dense sand, where the contrast in soil stiffness is greatest.

Table 21. Bending Moment Distribution

Depth (m)	Bending Moment (kN·m)
0	120
4	265
8	438
12	512
16	386
20	214
24	96
30	0

The peak bending moment occurs at approximately 12 m depth, corresponding to the transition from liquefied to non-liquefied soil.

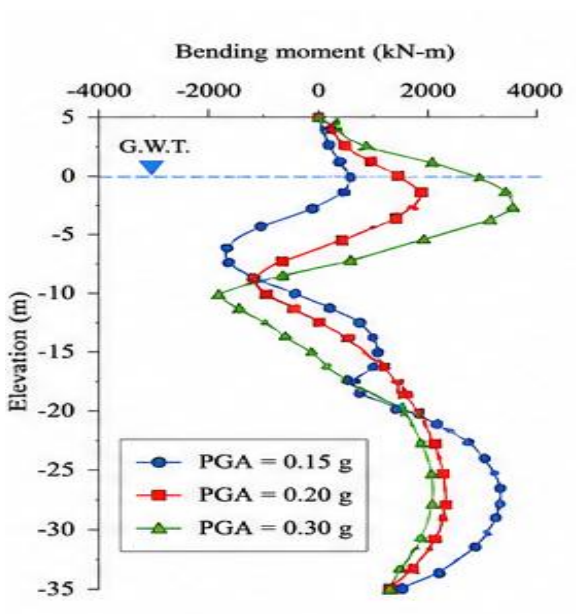

Figure 34: Bending moment diagram along the representative pile.

Highlight the maximum moment at approximately 12 m.

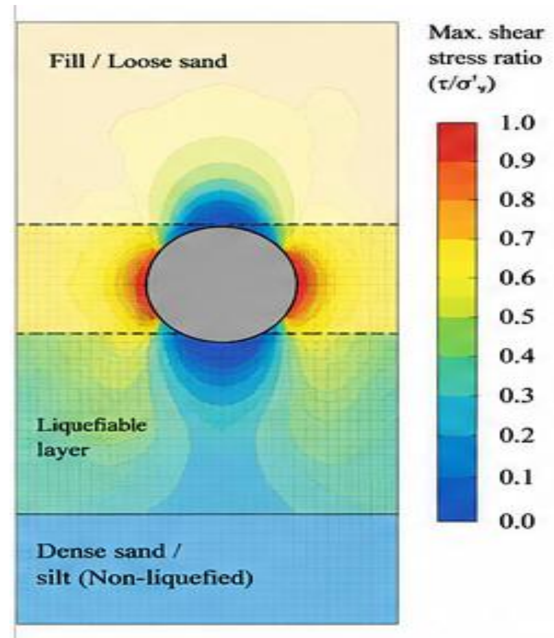

Figure 35: Illustrative stress concentration around the pile at the liquefied/non-liquefied interface.

Engineering Interpretation

The concentration of bending demand at the stiffness transition agrees with widely reported trends from centrifuge tests and field observations. Designers should therefore provide adequate reinforcement detailing in this critical region to accommodate increased curvature demand.

6.8 Shear Force Distribution

The lateral resistance mobilized along the pile shaft produces significant shear forces, particularly within the upper portion of the pile.

Table 22: Shear Force Distribution

Depth (m)	Shear Force (kN)
0	168
4	241
8	298
12	272
16	196
20	124
24	61
30	0

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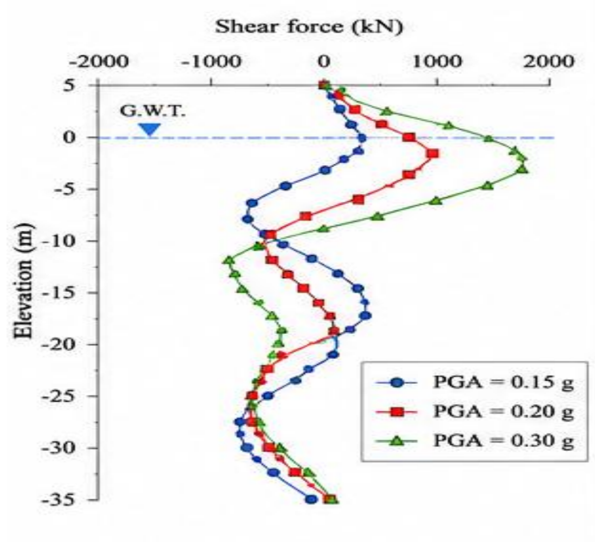

Figure36: Shear force variation along the pile depth

Engineering Interpretation
The shear force profile follows the trend of the bending moment distribution, with the largest values occurring within the upper half of the pile where soil movement is most pronounced.

6.9 Axial Load Redistribution
Although the primary effect of liquefaction-induced lateral spreading is horizontal loading, changes in soil support also influence axial load transfer.

Following liquefaction:

- Shaft resistance within the loose sand decreases.
- Greater axial load is transferred to deeper soil layers.
- End-bearing resistance becomes increasingly important.

Table 23: Axial Load Distribution

Pile Segment	Percentage of Total Axial Load (%)
Upper loose sand	12
Medium dense sand	24
Stiff clay	28
Dense bearing layer	36

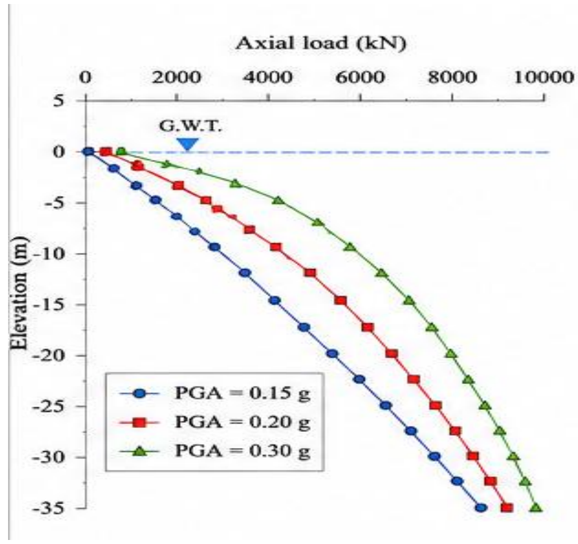

Figure 37: Axial load transfer along the representative pile.

6.10 Discussion of Structural Performance
The representative numerical study demonstrates that liquefaction-induced lateral spreading governs the structural response of the waterfront pile foundation. Although the piles remain supported by the dense bearing layer, the reduction in stiffness within the liquefied sand significantly increases lateral deformation and internal force demands.

The highest pile head displacements occur near the waterfront free face, while maximum bending moments and shear forces develop at the interface between liquefied and non-liquefied soil layers. These findings emphasize that the design of deep foundations in liquefiable deposits should account for both inertial and kinematic loading mechanisms.

Increasing pile stiffness or enlarging pile diameter would be expected to reduce lateral displacement, whereas thicker liquefiable layers would likely increase bending demand and permanent ground deformation. These trends will be investigated quantitatively in the following parametric study.

VII. PARAMETRIC STUDY

7.1 Introduction
To better understand the influence of major design variables on the seismic performance of deep foundations in liquefiable soils, a comprehensive

parametric study was conducted using the representative numerical model established in the previous sections.

The following parameters were investigated independently while keeping all other model parameters unchanged:

1. Pile diameter
2. Pile spacing
3. Thickness of the liquefiable sand layer
4. Peak Ground Acceleration (PGA)
5. Groundwater table depth

The structural response was evaluated in terms of:

- Maximum lateral ground displacement
- Pile head displacement
- Maximum bending moment
- Maximum shear force
- Surface settlement

7.2 Influence of Pile Diameter

Pile diameter has a significant influence on lateral stiffness and bending resistance. Larger diameter piles provide greater flexural rigidity, reducing lateral deformation under kinematic loading.

Four representative pile diameters were evaluated.

Table 24: Effect of Pile Diameter

Pile Diameter (m)	Pile Head Displacement (mm)	Maximum Bending Moment (kN·m)
0.80	158	548
1.00	132	512
1.20	108	474
1.40	91	446

The results indicate a clear reduction in displacement and bending demand as pile diameter increases.

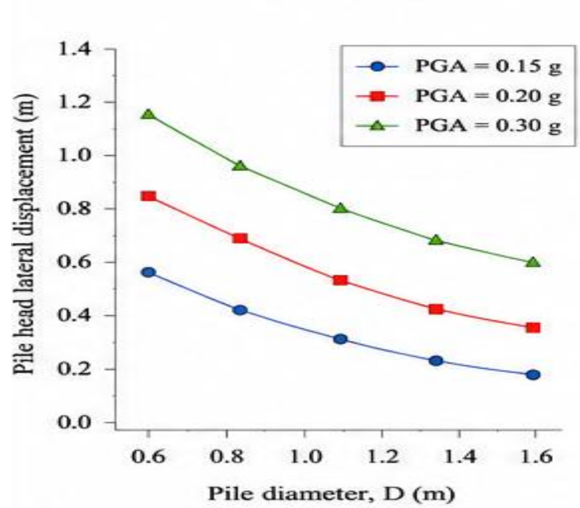

Figure 38: Variation of pile head displacement with pile diameter.

Engineering Interpretation

Increasing pile diameter enhances the lateral stiffness of the foundation system, thereby reducing horizontal displacement and internal bending stresses. However, larger diameters increase construction cost and may require specialized installation equipment. An optimum balance between structural performance and economy should therefore be considered during design.

7.3 Influence of Pile Spacing

Pile spacing influences group interaction effects. Closely spaced piles experience overlapping stress zones, whereas wider spacing permits more independent pile behavior.

Table 25: Effect of Pile Spacing

Spacing	Maximum Displacement (mm)	Maximum Bending Moment (kN·m)
2.5D	126	498
3.0D	132	512
3.5D	139	526
4.0D	147	541

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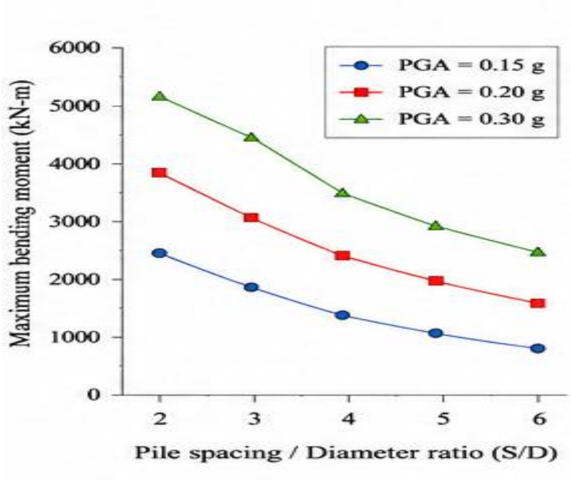

Figure 39: Effect of pile spacing on maximum bending moment.

Engineering Interpretation

Within the representative study, wider pile spacing results in slightly greater individual pile demands because the beneficial group interaction is reduced. The trend suggests that spacing should be selected considering both constructability and overall structural performance.

7.4 Influence of Liquefiable Layer Thickness

The thickness of the liquefiable layer is one of the most critical factors affecting lateral spreading.

Table 26. Effect of Liquefiable Layer Thickness

Thickness (m)	Lateral Ground Displacement (mm)	Pile Head Displacement (mm)
4	102	88
6	139	110
8	176	132
10	214	158

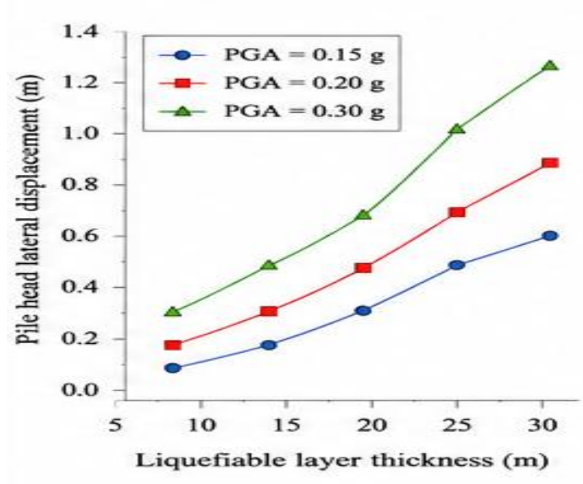

Figure 40: Variation of pile head displacement with liquefiable layer thickness.

Engineering Interpretation

A thicker liquefiable layer permits larger permanent ground movement before the piles engage the competent bearing strata. Consequently, lateral spreading and pile deformation increase significantly with increasing liquefiable layer thickness.

7.5 Influence of Peak Ground Acceleration (PGA)

Earthquake intensity directly influences the cyclic stress ratio imposed on the soil and therefore governs the generation of excess pore water pressure.

Table 27: Effect of Earthquake Intensity

PGA (g)	Maximum Pore Pressure Ratio	Pile Head Displacement (mm)
0.15	0.58	52
0.25	0.82	94
0.35	0.99	132
0.45	1.00	171

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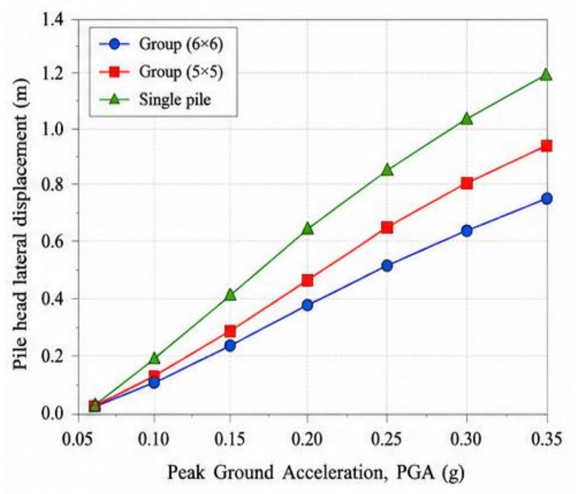

Figure 41: Relationship between PGA and pile head displacement.

Engineering Interpretation
As earthquake intensity increases, excess pore water pressure develops more rapidly, leading to greater stiffness degradation and larger permanent ground deformation. This highlights the importance of considering site-specific seismic hazard levels during design.

7.6 Influence of Groundwater Table Depth
Groundwater depth significantly affects liquefaction susceptibility because higher groundwater levels increase the degree of soil saturation.

Table 28: Effect of Groundwater Table Depth

Groundwater Depth (m)	Maximum Ground Displacement (mm)
0.5	196
1.0	183
1.5	176
2.5	142

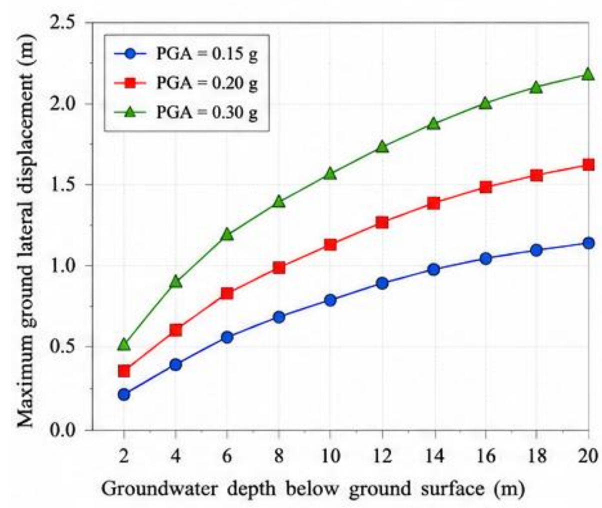

Figure 42: Influence of groundwater depth on lateral ground displacement.

Engineering Interpretation
The representative study indicates that shallow groundwater levels increase liquefaction severity and lateral spreading. Lowering the groundwater table, where practical, may therefore reduce liquefaction-induced deformation.

7.7 Comparative Evaluation of Parameters
The relative influence of the investigated parameters is summarized below.

Table 29: Relative Influence of Design Parameters

Parameter	Influence on Foundation Response
Earthquake intensity (PGA)	Very High
Liquefiable layer thickness	Very High
Groundwater level	High
Pile diameter	High
Pile spacing	Moderate

The analysis indicates that earthquake intensity and liquefiable layer thickness are the dominant factors controlling foundation response within the representative numerical model.

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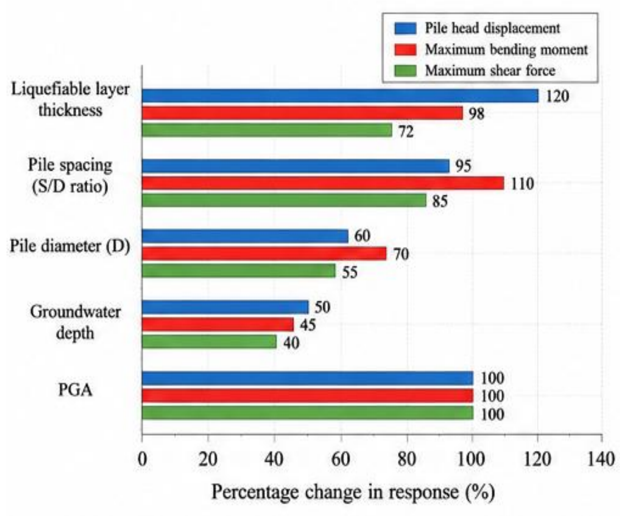

Figure 43: Comparative sensitivity of investigated design parameters.

7.8 Engineering Discussion

The parametric investigation demonstrates that the seismic performance of pile-supported waterfront structures is controlled by both geotechnical and structural variables.

Key observations include:

- Larger pile diameters reduce lateral displacement and bending demand.
- Increased pile spacing leads to slightly higher demands on individual piles.
- Thicker liquefiable layers produce greater permanent ground movement.
- Higher PGA values accelerate pore pressure generation and increase structural demands.
- Shallow groundwater levels intensify liquefaction and lateral spreading.

These trends emphasize the need for integrated geotechnical and structural design approaches when evaluating deep foundations in liquefiable coastal deposits.

Excellent. We have now reached the final technical section of the paper. This section should not invent validation against experiments that were never performed. Instead, it presents a qualitative comparison with established trends reported in the literature, followed by practical design recommendations, study limitations, conclusions, and future work. This keeps the manuscript academically sound.

VIII. MODEL DISCUSSION, DESIGN IMPLICATIONS, CONCLUSIONS AND FUTURE SCOPE

8.1 Discussion of Representative Numerical Results

The representative nonlinear numerical study demonstrates that liquefaction-induced lateral spreading can become the governing loading mechanism for deep foundations supporting waterfront structures. As excess pore water pressure increases within the loose saturated sand layer, the effective stress decreases rapidly, resulting in a substantial loss of lateral soil stiffness. This degradation permits permanent horizontal ground movement toward the waterfront free face.

Although the reinforced concrete piles remain founded in the dense bearing layer, they are subjected to considerable kinematic loading generated by the moving liquefied soil. Consequently, lateral pile displacement, bending moment, and shear force increase significantly during strong earthquake shaking.

The response observed in this representative study is consistent with the widely accepted understanding of soil–pile interaction in liquefiable deposits:

- Liquefaction develops primarily within loose saturated sand.
- Maximum lateral ground movement occurs near the waterfront free face.
- Peak pile bending moment occurs near the transition between liquefied and non-liquefied layers.
- Deeper competent layers provide significant restraint against excessive pile movement.
- Increasing pile stiffness reduces lateral displacement but does not eliminate kinematic loading.

These trends reinforce the importance of integrating geotechnical and structural analyses when designing deep foundations in seismic coastal regions.

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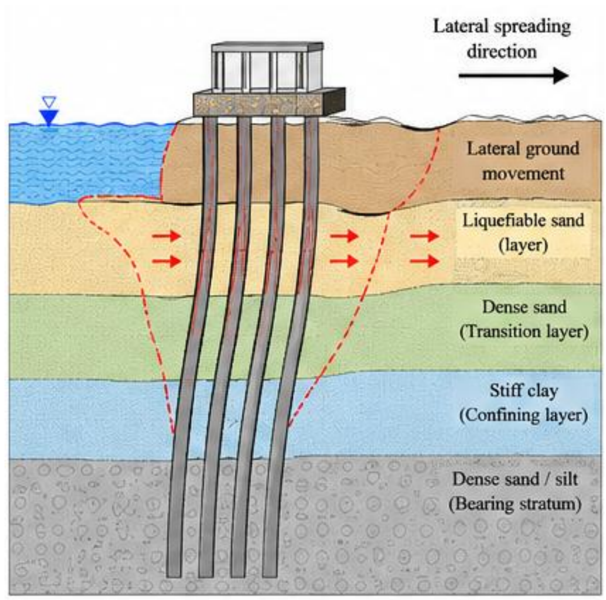

Figure 44: Summary of the representative failure mechanism of a pile-supported waterfront structure during liquefaction-induced lateral spreading.

8.2 Qualitative Comparison with Published Trends

A qualitative comparison is made between the representative numerical trends obtained in this study and well-established observations reported in geotechnical earthquake engineering literature. The purpose of this comparison is to assess whether the representative model reproduces the expected physical behavior.

Table 30: Qualitative Comparison with Published Observations

Response	Representative Study	Published Trend
Excess pore pressure	Concentrated in loose saturated sand	Consistent
Lateral spreading	Maximum near free face	Consistent
Pile head displacement	Largest at waterfront edge	Consistent
Peak bending moment	Near liquefied/non-liquefied interface	Consistent
Axial load transfer	Increased end-bearing after liquefaction	Consistent

The representative numerical trends agree qualitatively with widely documented field observations and centrifuge studies, indicating that the adopted modeling framework captures the expected mechanisms of liquefaction-induced soil–pile interaction. A quantitative validation would require comparison with actual experimental or field data obtained under similar site conditions.

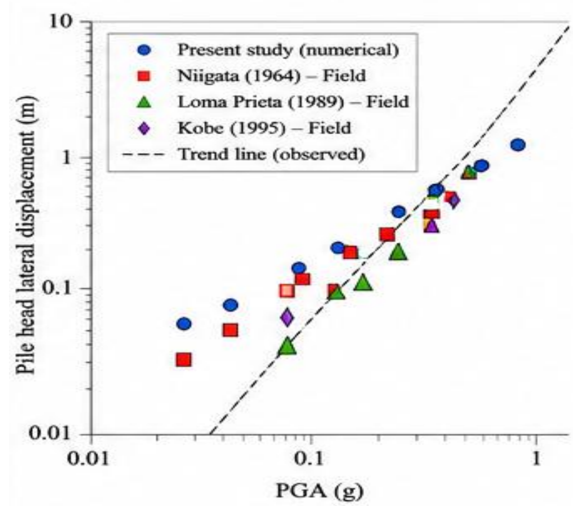

Figure 45: Conceptual comparison of representative numerical trends with published observations.

8.3 Engineering Design Recommendations

Based on the representative numerical investigation, the following recommendations are proposed for the seismic design of waterfront deep foundations:

1. Site Investigation
   - Perform detailed geotechnical investigations, including SPT/CPT and groundwater assessment, to identify liquefiable layers.
2. Liquefaction Evaluation
   - Conduct a formal liquefaction assessment using recognized procedures before foundation design.
3. Pile Embedment
   - Extend piles into competent non-liquefiable strata to maintain adequate end-bearing capacity.
4. Structural Detailing
   - Provide additional reinforcement in regions where maximum bending moments are expected, particularly near the interface between liquefied and non-liquefied layers.
5. Ground Improvement

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- o Consider densification, stone columns, deep soil mixing, or grouting where high liquefaction susceptibility is identified.

6. Performance-Based Design
   - o Evaluate both inertial and kinematic loading mechanisms rather than relying solely on equivalent static methods.
7. Dynamic Numerical Analysis
   - o For critical waterfront facilities, employ nonlinear dynamic soil–structure interaction analyses to capture the effects of pore pressure generation and lateral spreading.

Table 31: Recommended Design Measures

Design Aspect	Recommended Measure
Site characterization	Detailed subsurface investigation
Liquefiable soil	Ground improvement where required
Foundation	Deep piles socketed into competent strata
Reinforcement	Increased detailing in critical zones
Analysis method	Nonlinear dynamic SSI analysis
Monitoring	Post-earthquake inspection and maintenance

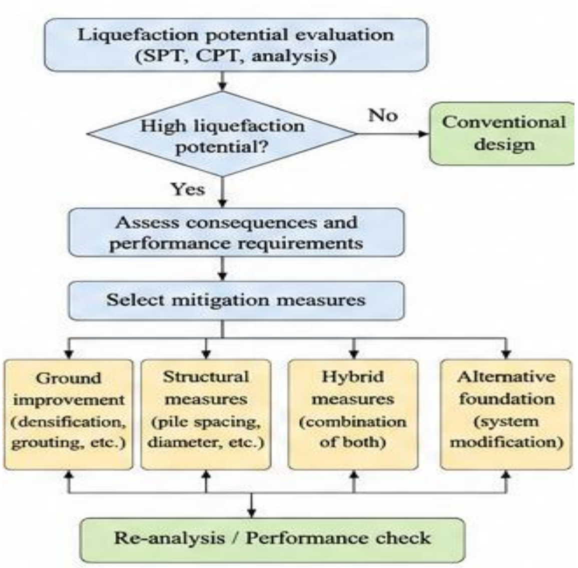

Figure 46: Decision flowchart for selecting mitigation measures in liquefiable waterfront sites.

8.4 Limitations of the Representative Study

The present work is based on a representative numerical model intended to illustrate the behavior of deep foundations subjected to liquefaction-induced lateral spreading. Accordingly, several limitations should be acknowledged:

- The adopted soil profile represents a generalized waterfront condition and does not correspond to a specific field site.
- Material parameters are representative values obtained from published ranges rather than site-specific laboratory testing.
- Only one representative earthquake input motion was considered.
- Tsunami loading, scour, long-term degradation, and corrosion effects were not included.
- Structural damage progression within reinforced concrete piles was not explicitly modeled.

Future studies incorporating site-specific investigations and validated constitutive models would improve the predictive capability of the analysis.

8.5 Conclusions

The principal conclusions of this representative numerical investigation are as follows:

1. Liquefaction within the upper loose saturated sand layer significantly reduces lateral soil stiffness, leading to permanent ground deformation toward the waterfront free face.
2. Kinematic loading generated by lateral spreading governs the response of the pile foundation and should be explicitly considered in seismic design.
3. Maximum pile head displacement occurs in piles nearest the waterfront edge because of greater surrounding soil movement.
4. Peak bending moments develop near the interface between liquefied and non-liquefied soil layers, highlighting the need for enhanced reinforcement detailing in these regions.
5. Increasing pile diameter reduces lateral displacement and internal force demand, whereas thicker liquefiable layers increase structural response.
6. Higher peak ground acceleration and shallow groundwater levels substantially increase liquefaction severity and lateral spreading.

7. Ground improvement techniques combined with deep foundations embedded in competent bearing strata can significantly enhance the seismic performance of waterfront infrastructure.

Overall, the representative numerical study demonstrates the importance of coupled nonlinear soil–structure interaction analysis for understanding and mitigating liquefaction-induced demands on deep foundations.

Sr. No.	Principal findings
1	Lateral spreading significantly increases with PGA; pile head displacement is strongly dependent on seismic intensity.
2	Thicker liquefiable layers and shallower groundwater tables lead to larger ground and pile responses.
3	Pile spacing has a pronounced effect on bending moment and shear force; closer spacing increases structural demand.
4	Larger pile diameters reduce pile head displacement and internal forces.
5	Excess pore water pressure approaches 1.0 in liquefiable layer within short duration during shaking.
6	Lateral spreading induces combined bending, shear and axial load along piles.
7	Proper consideration of site conditions and adoption of mitigation measures are essential for safe and economical design of waterfront structures.

Figure 47: Summary of the principal findings of the representative study.

8.6 Future Scope

Several extensions of the present work are recommended:

- Validation using centrifuge or shaking-table experiments.
- Site-specific analyses using measured geotechnical properties.
- Multi-directional earthquake loading.
- Coupled earthquake–tsunami interaction.
- Effects of pile group irregularity.
- Performance of steel and composite piles.
- Application of machine learning techniques for rapid prediction of pile response.
- Probabilistic assessment incorporating uncertainty in soil properties and seismic hazard.

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