

COMPARATIVE ANALYSIS OF STATIC AND SEISMIC BEHAVIOUR IN REINFORCED EARTH WALLS WITH POLYMERIC STRIP REINFORCEMENT

A Dissertation Submitted
in partial Fulfillment of the Requirements for the Award of the
Degree of

MASTER OF TECHNOLOGY IN GEOTECHNICAL ENGINEERING BY

Aakash Kumar
(2K23/GTE/10)

Under the supervision of
Prof. Amit Kumar Shrivastava



Department of Civil Engineering
DELHI TECHNOLOGICAL UNIVERSITY
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Shahbad, Daulatpur, Main Bawana Road, Delhi-110042)

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CANDIDATE'S DECLARATION

I, **Aakash kumar** (2K23/GTE/10) student of M. Tech., Geotechnical Engineering, hereby declare that the work which is being presented in dissertation entitled “**Comparative Analysis of Static and Seismic Behaviour in Reinforced Earth Walls with Polymeric Strip Reinforcement**” in partial fulfillment of the requirements for the award of the Degree of Master of Technology, submitted in the Department of Civil Engineering, Delhi Technological University, Delhi is an authentic record of my own work carried out during the period from 2024 to 2025 under the supervision of **Prof. A.K. Shrivastava**.

The content presented in the dissertation has not been submitted for the award of any other degree of this or any other institute.

(AAKASH KUMAR)

This is to certify that the student has incorporated all the corrections suggested by the examiners in the thesis and that the statement made by the candidate is correct to the best of our knowledge.

Prof A.K Shrivastava
(Supervisor)

Place: Delhi
Date:

(Signature of Examiner)



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CERTIFICATE BY THE SUPERVISOR(s)

It is certified that **Aakash Kumar** (2K23/GTE/03) has carried out their research work presented in this report entitled “**Comparative Analysis of Static and Seismic Behaviour in Reinforced Earth Walls with Polymeric Strip Reinforcement**” for the award of **Master of Technology** from Department of Civil Engineering, Delhi Technological University, Delhi, under my supervision. The report embodies results of original work, and studies are carried out by the student himself and the contents of the report do not form the basis for the award of any other degree to the candidate or to anybody else from this or any other University/Institution.

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ABSTRACT

Reinforced earth walls have become a vital component in modern geotechnical engineering, particularly for infrastructure located in seismically active regions. Their ability to accommodate deformations and enhance slope stability makes them a preferred solution for retaining structures. This study presents a comparative analysis of the static and seismic behavior of reinforced earth walls using polymeric strip reinforcements of varying tensile strengths. The objective is to assess how different reinforcement capacities and soil conditions influence wall performance under dynamic loading. Experimental investigations were carried out to determine the properties of the backfill material, including standard Proctor compaction, California Bearing Ratio (CBR) tests, and dynamic soil characterization. These parameters were used as inputs for two-dimensional finite element modeling in PLAXIS 2D. The walls were analyzed using polymeric strips with tensile strengths of 30 kN, 40 kN, and 60 kN under varying cohesion and friction angle values. The simulations revealed that walls with lower-strength reinforcements (30 kN and 40 kN) exhibited instability under seismic loading, primarily due to inadequate tensile resistance. In contrast, the configuration with 60 kN reinforcement and higher shear strength soil ($\phi = 35.48^\circ$) demonstrated stability, effective energy dissipation, and controlled deformation, with displacement limited to ~ 65 mm and peak acceleration reaching ~ 165 m/s² at the crest. The study confirms that reinforcement strength and soil shear properties are critical to seismic performance, and highlights the necessity of proper material selection in design. The findings provide valuable insight for engineers and designers to enhance the seismic resilience of reinforced soil structures.

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LIST OF SYMBOLS, ABBREVIATIONS

E'	Young's modulus
EA	Normal stiffness
EI	Flexural rigidity
μ	Poisson's ratio
c_u	Undrained Cohesion
ϕ	Friction Angle
ψ	Dilatancy Angle
γ_{dry}	Dry Unit Weight
γ	Unit Weight
GS	Geo strap
G	Specific gravity
ν	Poisson's ratio
ρ	Internal friction angle
K_a	Active Earth pressure

CHAPTER 1

INTRODUCTION

1.1 GENERAL

Reinforced Earth (RE) technology has significantly transformed the domains of geotechnical and transportation engineering in recent decades. It provides an innovative, economical, and efficient method for constructing retaining walls, embankments, bridge abutments, and slope stabilization systems. The fundamental principle of RE walls involves enhancing the mechanical properties of soil by incorporating reinforcement materials such as metallic strips, geogrids, or polymeric strips. These reinforcements engage with the surrounding soil to mobilize tensile forces, thereby reducing deformation and improving the structural integrity and load-bearing capacity of the system. Among various reinforcement options, polymer-based strips like Paraweb are increasingly preferred due to their high tensile strength, corrosion resistance, lightweight nature, and ease of handling during construction. In the context of India's rapidly expanding infrastructure—particularly the development of national highways—polymeric reinforced earth walls are becoming a popular solution, especially in areas with weak or marginal soils. In regions prone to seismic activity, evaluating the dynamic performance of reinforced earth structures becomes critically important in addition to their static stability. Earthquakes impose transient and cyclic loads that alter the stress distribution within the soil-reinforcement system. These dynamic effects can affect the deformation behavior, failure mechanisms, and long-term performance of RE walls. Therefore, a comprehensive understanding of both static and seismic responses is essential for safe and durable design. This study focuses on a real-time project located along National Highway NH-148B, between Bhiwani and Hansi, specifically between chainage 4+300 to 4+400, where a four-lane highway is under construction. Soil samples were collected from the site and analyzed for basic index properties. California Bearing Ratio (CBR) tests were conducted with and without the inclusion of Paraweb strips to evaluate improvements in soil strength. A scaled model tank was fabricated to replicate a reinforced earth wall for dynamic testing, and

numerical analyses were performed using PLAXIS 2D software under both static and seismic loading conditions.

1.2 INTRODUCTION TO STATIC ANALYSIS

Static analysis is a fundamental method in geotechnical engineering used to assess the equilibrium and stress-deformation characteristics of soil structures under the influence of gravity and externally applied static loads. This approach assumes that the loads are applied gradually, thereby neglecting the effects of acceleration and inertia forces. For reinforced earth walls, static analysis is crucial in evaluating parameters such as bearing capacity, lateral earth pressure, wall displacement, and the tensile forces mobilized within the reinforcement under normal service conditions.

Although static analysis plays a vital role in both preliminary assessments and detailed design stages, it does not fully capture the complex responses of soil-structure systems under dynamic or seismic loading. Nonetheless, it serves as a dependable reference point for interpreting seismic analysis results and is commonly used to verify outcomes obtained from experimental investigations.

1.3 INTRODUCTION TO FINITE ELEMENT ANALYSIS

Finite Element Analysis (FEA) is a robust computational technique widely employed to model and analyze the complex behavior of soil-structure interaction systems. It works by dividing a large, continuous domain—such as soil masses or reinforced structures—into smaller, discrete elements. This discretization allows for the detailed calculation of stress-strain responses under varying boundary conditions and loading scenarios. FEA facilitates accurate predictions of parameters such as displacement, stress distribution, pore pressure changes, and potential failure mechanisms.

In geotechnical engineering, FEA has become increasingly popular due to its ability to incorporate the nonlinear characteristics of soil, including anisotropy, plasticity, and time-dependent behavior. Tools like PLAXIS 2D, which are based on the finite element method, are extensively utilized for simulating both static and seismic

performance of reinforced soil systems, offering high levels of accuracy and insight into real-world performance.

1.4 DIFFERENT WAYS TO MODEL IN PLAXIS 2D

PLAXIS 2D supports two main conditions for modelling geotechnical parts.

1.4.1 PLANE STRAIN

Plane strain analysis assumes that deformation in the out-of-plane direction is negligible or zero, making it ideal for modeling long and continuous structures such as retaining walls, embankments, and slopes. This simplification reduces the problem to two dimensions while still capturing the essential stress-strain behavior of the system. In the context of reinforced earth walls, plane strain analysis is the most commonly used mode in PLAXIS, as it effectively represents wall deformation, stress distribution, and reinforcement response with computational efficiency and reliable accuracy.

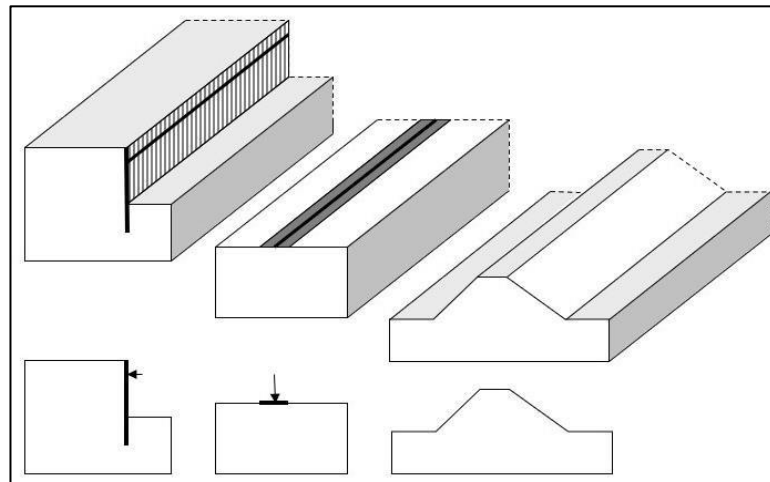


Figure 1.1 Plain Strain Modelling (Source: <https://communities.bentley.com>)

1.4.2 AXISYMMETRIC

Axisymmetric analysis is based on the assumption that the structure and loading

are symmetric around a central vertical axis, allowing a two-dimensional representation of a three-dimensional circular system. In this approach, a sector or slice of the structure—typically spanning one radian or a portion of the full circle—is modeled, with the understanding that it revolves around the axis to form the complete geometry.

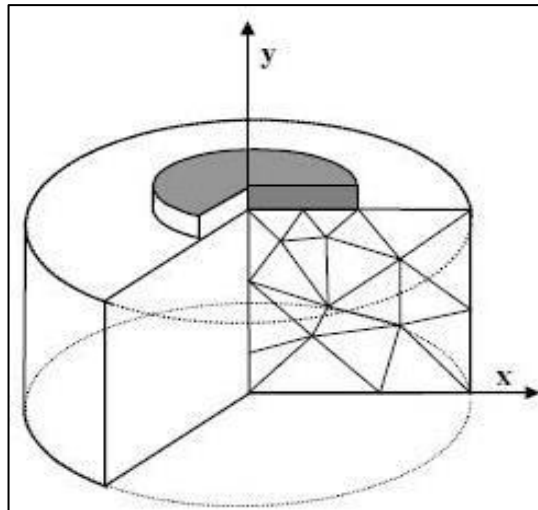


Figure 2.1 Axisymmetric Modelling (Source: <https://communities.bentley.com>)

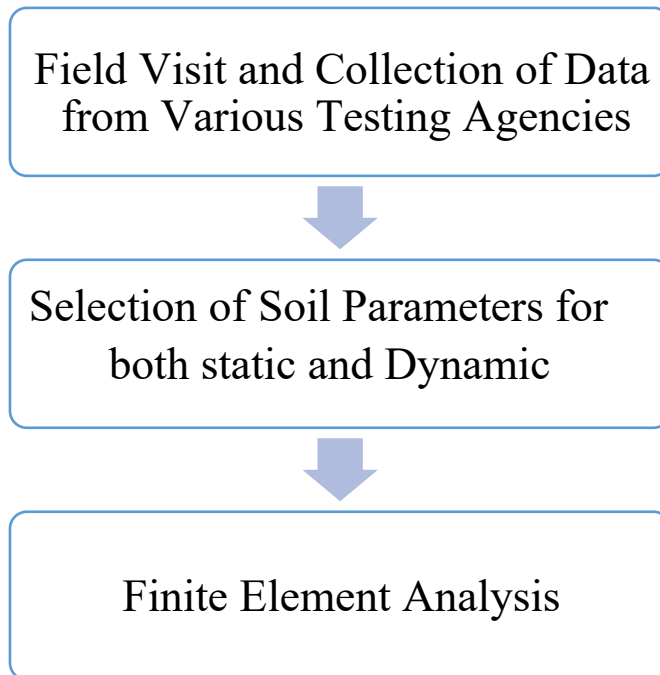
This modeling technique is particularly useful for analyzing structures such as circular footings, tunnels, and cylindrical storage tanks, where radial symmetry is present. Although axisymmetric analysis is not generally applicable to reinforced earth walls due to their linear and planar nature, it plays a critical role in evaluating geotechnical problems involving rotational or circular symmetry.

1.5 MOTIVATION OF PRESENT STUDY AND PROBLEM STATEMENT

The present study focuses on a real-world infrastructure project situated along National Highway 148B (NH-148B), between Bhiwani and Hansi, specifically from chainage 4+300 to 4+400, where a four-lane highway is currently under construction. This region lies within Seismic Zone IV, as defined by IS 1893:2016, indicating a high level of seismic vulnerability. Such conditions demand rigorous geotechnical evaluation and resilient structural design, particularly for essential earth-retaining systems like Reinforced Earth (RE) walls. Due to the seismic sensitivity of the Bhiwani region, it is

imperative that RE walls be designed to withstand earthquake-induced forces. However, conventional design practices often emphasize static stability and may overlook the dynamic soil-reinforcement interaction under seismic loading. This oversight can pose significant challenges, especially in areas where soil properties vary significantly over short distances. To bridge this gap, the current research adopts a combined experimental and numerical methodology. Soil samples were collected from the site and subjected to various laboratory tests, including index property analysis and California Bearing Ratio (CBR) tests—both with and without Paraweb polymeric reinforcement strips—to evaluate improvements in load-bearing capacity. For the dynamic study, a model tank with dimensions $65\text{ cm} \times 30\text{ cm} \times 25\text{ cm}$ was constructed to simulate the behavior of reinforced earth walls under controlled laboratory conditions. The setup was used to investigate the dynamic response and AVD (Amplitude–Velocity–Displacement) characteristics of the reinforced soil system. Simultaneously, numerical modeling was carried out using PLAXIS 2D, a finite element software widely used in geotechnical analysis. The laboratory tank model was first replicated in PLAXIS to validate the experimental results. Following this validation, a full-scale reinforced earth wall with a height of 2.3 meters was modeled and analyzed under both static and worst-case seismic scenarios, incorporating realistic seismic parameters in accordance with relevant IS codes. This integrated approach—combining physical testing with advanced numerical simulation—provides a more comprehensive understanding of RE wall performance under seismic conditions. It also establishes a validated framework that can be applied to the design and assessment of similar systems in earthquake-prone regions.

1.6 VARIOUS STAGES INVOLVED IN THE STUDY:



1.7 OBJECTIVES OF THE STUDY:

1. To determine the engineering properties of soil collected from the NH-148B project site through laboratory testing.
2. To study the dynamic response characteristics of reinforced and unreinforced soil systems using a laboratory tank model.
3. To evaluate the effect of varying tensile strengths of polymeric strip reinforcement on the seismic behavior of reinforced earth walls.
4. To perform finite element modeling of a 3.2m high reinforced earth wall in PLAXIS 2D and evaluate its static and seismic performance.
5. To assess the role of soil–reinforcement interaction in controlling wall stability and deformation under seismic loading.

1.8 STRUCTURE OF THIS DISSERTATION WORK

- Introduction
- Objectives
- Literature Review
- Methodology
- Result and Discussion
- Conclusion
- Future scope of the work
- References

CHAPTER 2

LITERATURE REVIEW

Huali et al., Johari and Maroufi (2024) [1]; geogrid reinforcement is a key method for improving the seismic performance of retaining and reinforced earth walls. It helps control lateral movement, reduces backfill pressure, and enhances stability during earthquakes. Huali et al. found that placing geogrids near the mid-height of soil walls offers a good balance of load control and deformation resistance, improving seismic response. Johari and Maroufi highlighted that geogrid effectiveness depends on soil conditions. Proper placement and tensioning are crucial to prevent failures like pullout and rupture. These studies show that well-placed geogrid reinforcement improves seismic strength by limiting deformation and preventing internal failures.

Yue et al. (2024) [2] carried out an in-depth investigation into the seismic response of gabion-reinforced retaining wall slopes using large-scale shaking table tests. The study analyzed key dynamic parameters such as peak acceleration amplification, incremental stresses, displacements, and tensile forces across different slope layers. Advanced techniques, including the Hilbert-Huang Transform, were employed to assess the slope's dynamic characteristics in both the time-frequency and energy domains. Findings revealed that seismic energy dissipation was significant, with peak acceleration amplification factors consistently below 1.9—lower than those typically observed in unreinforced slopes. The upper slope layers (particularly layers 3 to 5) showed heightened sensitivity to seismic loading, characterized by greater dynamic stress, displacement, and tensile force responses. The study highlighted the importance of focusing seismic design strategies on these vulnerable upper layers to enhance overall slope stability.

Ren et al. (2024) [3] carried out a detailed study on the seismic behaviour of tiered retaining wall systems, with a particular focus on the effect of offset distance between successive tiers. The findings indicated that tiered walls are more efficient than single-tier

systems in distributing seismic forces, thereby lowering the lateral earth pressure on individual wall segments. The study emphasized the importance of offset distance in controlling the dynamic response of the wall. It was observed that an optimal offset can significantly minimize lateral displacement and enhance the structural stability of the system under seismic loading. The researchers concluded that well-planned tier spacing contributes to better energy dissipation and reduced deformation, making tiered retaining walls a suitable and resilient design choice for seismic zones.

Fan et al. (2024) [4] explored the seismic response of tiered retaining wall systems subjected to near-field ground motions, particularly those exhibiting pulse-like characteristics. The study found that these types of seismic excitations significantly intensify deformation behaviour, especially in multi-tier configurations where relative displacement between tiers becomes more evident. The research emphasized that pulse-type ground motions, common in near-field seismic events, cause abrupt energy input into the structure, leading to considerable displacement and rotational movements within the retaining wall system. These results underscore the necessity for seismic design frameworks to incorporate frequency-sensitive and motion-specific considerations, particularly for retaining walls located near active fault lines, to ensure improved structural resilience and safety.

Yazdandoust (2024) [5] explored the seismic performance of retaining walls reinforced with steel strips, with particular emphasis on how different reinforcement lengths impact wall stability. The study employed numerical modeling to simulate earthquake-induced forces and assess the structural behavior of the system. By analyzing steel strips of various lengths, the research demonstrated that selecting optimal reinforcement dimensions can markedly improve wall stability. This improvement is attributed to enhanced interaction between the backfill material and the reinforcement during seismic activity.

Johari and Maroufi (2024) [6] examined the effects of soil heterogeneity on the seismic

behavior of retaining walls, utilizing stochastic modeling to capture the spatial variability of soil properties. Their research aimed to understand how various failure mechanisms respond to heterogeneous subsurface conditions. The results revealed that internal failure modes are considerably more affected by soil variability than external ones. This underscores the importance of accurately representing subsurface conditions in models to ensure reliable seismic performance evaluations and design outcomes.

Vadavadagi and Chawla (2023) [6] explored how varying rail axle loads affect the performance of geosynthetic-reinforced back-to-back mechanically stabilized earth (BBMSE) walls. Their research employed both scaled-down laboratory experiments and three-dimensional finite element simulations to assess the deformation behavior of the walls under different reinforcement configurations, including variations in geogrid spacing and stiffness. The study utilized coal mine overburden (OB) as an eco-friendly substitute for traditional backfill materials. The findings revealed that overlapping geogrid layouts significantly minimized wall deformation—showing up to a 58% reduction compared to connected arrangements. Moreover, the results indicated that closer reinforcement spacing and stiffer geogrids enhanced the stability of the walls. Overall, the study emphasizes the importance of optimizing reinforcement design and material selection to improve the structural and seismic performance of BBMSE walls in railway applications.

Cai et al. (2023) [6] conducted a comparative study evaluating the seismic performance of retaining walls configured as single-tier, two-tier, and three-tier systems. The primary goal was to determine which structural arrangement best balances deformation control with reinforcement efficiency under seismic loads. Their results indicated that the two-tier retaining wall system provided superior performance compared to the single-tier and three-tier configurations. Specifically, single-tier walls showed larger deformations, while three-tier walls, despite their increased complexity, did not deliver proportional

improvements in stability or reinforcement effectiveness. The two-tier design, on the other hand, demonstrated an optimal combination of enhanced stability, reduced displacement, and efficient reinforcement utilization. The study also highlighted that increasing the number of tiers beyond two could potentially compromise structural stability and diminish reinforcement efficiency. These findings underscore the critical need for meticulous design optimization, especially in seismic-prone areas where dynamic response plays a key role in ensuring structural safety.

Komak Panah and Eftekhari (2021) [3] performed a series of shaking table tests to investigate the seismic response of polymeric strip-reinforced soil (PSW) walls constructed adjacent to rock slopes. Their research analyzed the impact of reinforcement length, the nature of the slope material, and characteristics of seismic excitation. Scaled-down physical models were subjected to harmonic loading with variable amplitudes to replicate earthquake effects. The findings indicated that PSW walls built against rock slopes experienced substantially less deformation and displacement than those adjacent to soil slopes, primarily due to the rock's restraining effect on active wedge failure. Interestingly, even with a reduced reinforcement length ratio ($L/H = 0.3$) at the base of the walls on rock slopes, comparable stability to that of longer reinforcements on soil could be achieved. The study also underscored the significant roles of reinforcement length and confining pressure in influencing dynamic properties like shear modulus and damping ratio. Notably, the lowest section of the PSWs, where shorter reinforcements were used, showed the highest deformation and least stiffness, marking it as the most susceptible area during seismic loading.

Samee et al. (2021) [7] carried out an in-depth experimental investigation to evaluate the seismic behavior of back-to-back mechanically stabilized earth (BBSRW) walls reinforced with steel strips. The study included seven shaking table experiments on scaled-down models, exploring the effects of varying reinforcement overlap lengths and connection types between the opposing walls. The researchers assessed how these design

parameters influenced critical seismic responses, including wall deformation, failure patterns, reinforcement forces, and acceleration amplification. Findings revealed that interconnecting the opposing walls considerably reduced lateral movement and enhanced seismic performance, whereas unconnected walls exhibited greater deformation and increased dynamic stresses. Although shorter overlap lengths and disconnected setups resulted in lower reinforcement forces, they also caused undesirable failure modes and more pronounced wall displacement. The study concluded that careful optimization of reinforcement overlap and inter-wall connectivity is essential to achieving a balance between structural safety and cost-effectiveness in the seismic design of BBSRW systems.

Xu et al. (2021) [8] investigated the impact of ground motion frequency on the seismic performance of retaining wall systems. The study aimed to assess how variations in frequency, particularly high-frequency seismic excitations, influence the structural stability of these systems. The findings revealed that high-frequency ground motions significantly increase the risk of sliding and tilting deformations, especially in retaining walls with limited structural flexibility or inadequate reinforcement. This behaviour is primarily attributed to potential resonance effects induced by higher frequencies. The research highlighted the critical importance of considering the frequency content of ground motions during seismic design. It recommended incorporating frequency analysis into both experimental assessments and numerical modelling to ensure a more accurate evaluation of seismic behaviour and to enhance overall design reliability in earthquake-prone regions.

Safaei et al. (2021) [9] conducted a series of shaking table tests to investigate the seismic performance of multi-tiered geogrid-reinforced mechanically stabilized earth (MSE) walls featuring wrap-around facings. The study compared single-, double-, and triple-tiered wall configurations under dynamic loading, using scaled physical models developed in accordance with established design standards. These models were exposed to sinusoidal seismic waves of increasing intensity to simulate earthquake conditions. The research

focused on parameters such as face displacement, settlement of the backfill, acceleration amplification, and observed failure patterns. The results demonstrated that multi-tiered MSE walls offered significantly enhanced seismic performance compared to single-tiered systems, with lateral displacements reduced by as much as 230%. Additionally, tiered configurations delayed the initiation of plastic deformation and failure, while also reducing acceleration amplification—particularly in the upper sections of the wall. The study highlights the effectiveness of tiered wall structures with geogrid wrap-around facings for improving seismic resilience in retaining wall systems.

Xu et al. (2021) [10] explored the seismic behavior of reinforced soil retaining walls equipped with two distinct facing systems: a conventional full-height panel and an innovative hybrid facing combining modular blocks with cast-in-place concrete. Through shaking table tests on scaled-down wall models, the researchers evaluated dynamic performance indicators such as wall displacement, acceleration amplification, earth pressure distribution, and loads at reinforcement connections. The findings revealed that under lower seismic intensities (peak acceleration below 0.40g), the hybrid facing performed comparably to the full-height panel system. However, at higher seismic levels, the hybrid wall exhibited vertical discontinuities, resembling the behavior of segmental wall systems. Despite these cracks, the hybrid configuration maintained more consistent acceleration amplification and experienced less overall displacement. Additionally, the hybrid model recorded lower connection loads and a more balanced distribution of dynamic earth pressures compared to the full-panel design. The study concluded that hybrid facings can achieve similar seismic durability while offering construction advantages and potential cost savings.

Başbuğ et al. (2021) [9] conducted an extensive series of 1-g shaking table experiments to investigate the seismic performance of geosynthetic reinforced soil (GRS) walls. The study involved testing four large-scale physical models under various conditions, including changes in backfill type, reinforcement configuration and stiffness, and wall

face inclination (batter angle). Earthquake motions from the Izmit, Sakarya, and El-Centro events were applied at increasing intensity levels to simulate progressive seismic loading. The results showed that wall displacements were generally minor until ground shaking exceeded 0.4g, after which notable deformations were observed. Among the key findings, reinforcement stiffness played a more influential role than reinforcement length in limiting seismic-induced strains. Additionally, gravel backfill outperformed sand, attributed to its greater internal friction and interlocking characteristics. The study identified Arias Intensity (IA) as a more reliable indicator of wall response than peak ground acceleration (PGA), with stronger correlations to both reinforcement strain and wall movement. Based on these findings, the authors developed a preliminary analytical model to estimate reinforcement strain, incorporating IA, reinforcement characteristics, and backfill properties—offering valuable insights for tailored seismic design of GRS structures.

Li et al. (2021) [11] conducted large-scale shaking table experiments to analyze the seismic response and lateral displacement of modular-block reinforced soil retaining walls (RSRWs). Their study aimed to clarify how these walls behave dynamically and to establish benchmarks for displacement control during horizontal seismic loading. Two scaled wall models—a single-step and a two-tiered wall—were tested using both actual earthquake records and synthetic seismic inputs. The experiments revealed that lateral displacements grow as peak ground acceleration (PGA) increases, with the highest deformations occurring at the topmost modular blocks. Both wall types showed a combination of translational and rotational displacement patterns, and residual displacements were also greatest in the upper wall sections. The researchers proposed a displacement threshold of 3.8% of the wall height as an early indicator of damage after seismic events. Additionally, a four-level damage classification system was introduced, ranging from nearly intact to completely destroyed, based on observed deformation patterns. These results highlight the importance of setting clear displacement limits to improve seismic design and post-earthquake evaluation of modular-block RSRWs

Komak Panah et al. (2021) [12] investigated how polymeric-strip reinforcements influence the seismic behavior of retaining walls, emphasizing the roles of reinforcement properties, strip length, and slope geometry. Using finite element analysis, the researchers simulated seismic conditions to assess structural performance. The findings indicated that polymeric strips can significantly improve seismic resistance, particularly when paired with well-optimized geometric configurations. Additionally, the study highlighted the crucial impact of slope geometry on both stress distribution and wall displacement. The authors concluded that tailoring reinforcement strategies to specific material characteristics leads to more effective seismic performance than relying on generic design methods.

Shrivastava and Chauhan (2020) [11] carried out a numerical study to evaluate the seismic response of two-tiered Mechanically Stabilized Earth (MSE) retaining walls, comparing their performance to traditional single-tier systems. Employing finite element analysis, the researchers examined how varying reinforcement lengths and seismic intensities influenced wall stability, as measured by the factor of safety (FOS), and analyzed possible failure modes. The results demonstrated that the two-tiered wall configuration offered significantly better performance, showing up to a 47% improvement in FOS under seismic loading. The study identified ideal reinforcement lengths of $0.8H$ for static conditions and $1.1H$ for seismic scenarios. Failure mechanism evaluations revealed wedge-shaped failure zones in two-tiered systems, accompanied by lower stress concentrations than those found in single-tiered designs. The findings highlight the enhanced structural performance and seismic durability of two-tiered walls, supporting their application in seismically active areas requiring tall retaining wall solutions.

2.2 RESEARCH GAPS:

After going through a detailed research literature available on the numerical analysis of the reinforced earth walls utilizing polymeric strip reinforcement under both static and seismic conditions, the following research gaps are observed:

- Lack of comparative analysis between static and seismic behavior in polymeric strip-reinforced earth walls.
- Limited experimental-numerical validation for reinforced walls using polymeric strips in seismic regions.
- Absence of AVD (Amplitude–Velocity–Displacement) metrics in dynamic performance evaluation of reinforced soil systems.
- Scarcity of studies focused on polymeric reinforcement design in Zone IV seismic.

CHAPTER 3

STUDY LOCATION

3.1 AREA OF STUDY

The study area is located along National Highway 148B (NH-148B) between the towns of Bhiwani and Hansi in the state of Haryana, India. The specific section under investigation lies between chainage 4+300 and 4+400. Figure 3.1 shows the geographic location of the study site using Google Earth imagery. This region falls within Seismic Zone IV, indicating a moderate to high seismic risk. The coordinates of the site are approximately $28^{\circ}49'30''$ North latitude and $76^{\circ}09'50''$ East longitude. The site was selected due to ongoing highway construction activities and its suitability for assessing the seismic behavior of polymeric strip-reinforced earth walls.

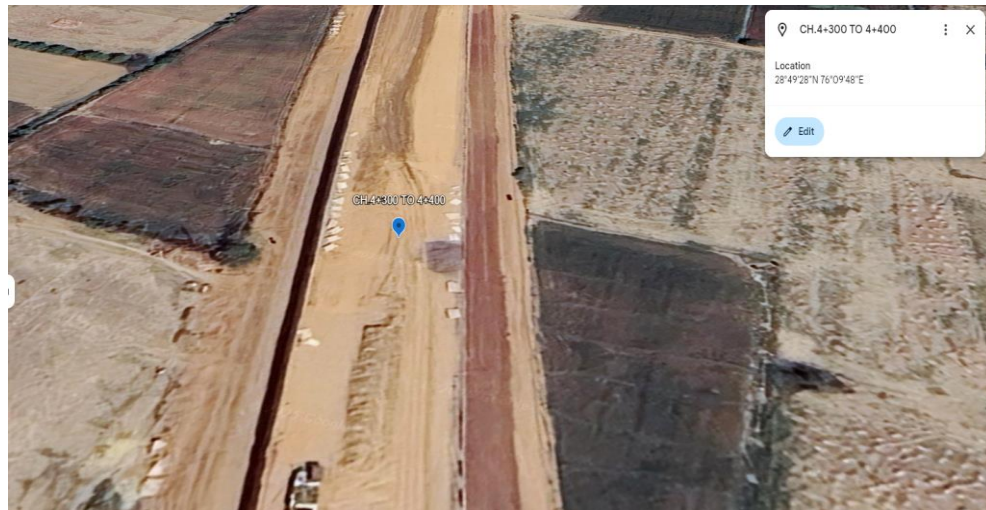


Figure 3.1 Map Showing Chainage 4+300 to 4+400 of NH-148B in Seismic Zone IV
(Latitude: $28^{\circ}49'30''$ N, Longitude: $76^{\circ}09'50''$ E)

3.2 DESCRIPTION OF THE FILED OF STUDY AND PROPERTIES OF MATERIALS

The seismic stability analysis was conducted on a reinforced earth wall with a

design height of 3.2 meters and a width of 22.30 meters. The structural response of the wall was examined under both static and seismic conditions using relevant geotechnical parameters. The geographic setting of the study site is illustrated in Figure 3.2. Three distinct soil zones were considered in the analysis: reinforced soil, retained soil, and foundation soil. Both the reinforced and retained soils have a unit weight of 20.0 kN/m^3 and a design internal angle of friction of 32.0° .



Figure 3.2 Geometry and Site condition of the Reinforced Earth wall at chainage 4+300 to 4+400.

The foundation soil was modeled as an equivalent uniform soil with an equivalent unit weight of 18.0 kN/m^3 , an internal angle of friction of 30.0° , and zero cohesion. The water table was assumed to be at the base elevation of the wall. For earth pressure estimation, the coefficient of active earth pressure (K_a) was taken as 0.3073 for both internal and external stability. The properties of the polymeric geogrid strip used for reinforcement are summarized in Table 3.1.

3.3 SEISMICITY

The study site falls within Seismic Zone IV, which is characterized by a moderate to high seismic hazard. Accordingly, seismic parameters were incorporated into the design of the reinforced earth wall to simulate earthquake loading conditions realistically. A maximum ground acceleration coefficient (A) of 0.120 was used to represent the peak horizontal ground motion at the site. Furthermore, a maximum wall acceleration coefficient (A_w) of 0.160 was applied to account for the amplified response of the wall structure during seismic events. For design calculations, an acceleration coefficient of 0.135 was adopted for both internal and external stability assessments. To account for dynamic interaction between soil and reinforcement, the seismic soil–geogrid friction coefficient (F^*) was assumed to be 80% of its static value, as per established seismic design practices.

Table 3.1 Properties of the polymeric geogrid strip used for reinforcement

S. No.	Geostrip type	Ultimate	Width of	Strength Reduction Factor, RF			Total	Stiffness, (EA), KN/m
		Tensile Strength (T_{ul}), kN	Geostrip (w), M	Creep, RF_{CR}	Installation Damage, RF_D	Durability RF_D	Reduction Factor, RF_{Total}	
				RF_{ID}				
1	GS30	30	0.09	1.43	1.05	1.21	1.817	1835
2	GS40	40	0.09	1.43	1.05	1.21	1.817	5307

CHAPTER 4

METHODOLOGY

In this chapter the various methods used for the study are explained and with the help of the flowchart the sequence of the methodology followed is shown below:

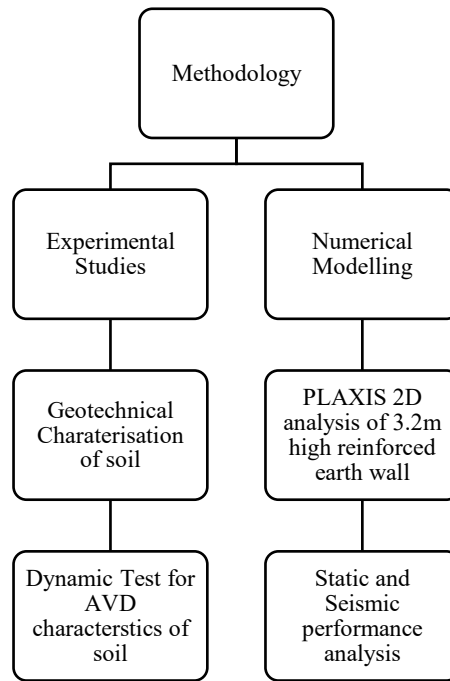


Figure 4.1 Schematic Diagram Depicting the Methodological Framework of the Study.

4.1 EXPERIMENTAL STUDIES

Several laboratory tests were conducted to determine the engineering properties of the soil. The procedures and methodologies adopted for each test are described in the following sections.

4.1.1 PARTICLE SIZE DISTRIBUTION ANALYSIS

The particle size distribution test was conducted to determine the relative proportions of different grain sizes present in the soil sample, in accordance with IS: 2720 (Part 4) – 1985. The results obtained from this test were used to plot the grain size distribution curve, which helps in the classification and prediction of soil behavior. Two standard techniques were employed for this purpose:

- Sieve analysis for particles larger than 0.075 mm.
- Hydrometer analysis for particles smaller than 0.075 mm.

Sieve analysis is a method used to determine the particle size distribution of soil, specifically for particles larger than 0.075 mm. It is commonly applied to classify coarse-grained soils such as sand and gravel but is not suitable for accurately assessing the finer fractions. The test involves a set of sieves made from woven wire mesh with square openings. A measured quantity of soil—determined based on the maximum particle size—is placed on the top sieve of a stack arranged in decreasing mesh sizes from top to bottom. The stack is then mechanically or manually shaken for a specified period to allow particle separation based on size.

In wet sieve analysis, around 200 grams of soil sample was initially soaked in water to disperse clumps and ensure separation of individual particles. The soaked soil was then thoroughly washed through a 75 μm sieve to remove fine particles such as silt and clay. The portion of soil retained on the 75 μm sieve, representing the sand fraction, was collected, oven-dried, and weighed. This dried sample was subjected to mechanical sieving using a standard set of sieves with openings of 4.75 mm, 2.00 mm, 1.18 mm, 600 μm , 425 μm , 300 μm , 150 μm , and 75 μm (see Figure 4.2). Following a specified shaking period, the mass of soil retained on each sieve was recorded. These values were used to construct a particle size distribution curve. From this data, parameters such as effective grain size (D_{10}), uniformity coefficient (C_u), and curvature coefficient (C_c) were calculated to classify the soil and assess its engineering properties.

4.1.2 DETERMINATION OF SPECIFIC GRAVITY

Specific gravity is a dimensionless parameter defined as the ratio of the density of soil solids to the density of water at a specific temperature. In this study, the specific gravity of the soil was determined using the density bottle method as per IS: 2720 (Part 3/Sec 2) – 1980 (see figure 4.3). Approximately 50 grams of oven-dried soil sample was used for the test. First, the empty density bottle was weighed and recorded as M_1 .

The bottle was then partially filled with the dry soil sample and weighed again, denoted as M_2 . Distilled water was added to the bottle containing the soil until it was completely filled, and the new weight was recorded as M_3 . Lastly, the bottle was emptied, thoroughly cleaned, refilled with only distilled water, and weighed to obtain M_4 .

The specific gravity of the soil solids was computed using the values obtained during the test and applying the following standard formula:

$$G = \frac{(M_2 - M_1)}{(M_4 - M_1) - (M_3 - M_2)} \quad (1)$$

Where;

- M_1 = Mass of empty density bottle
- M_2 = Mass of bottle + dry soil
- M_3 = Mass of bottle + soil + water
- M_4 = Mass of bottle filled with water only

4.1.3 COMPACTION TEST

Soil compaction is a process used to increase the soil's dry density by reducing the volume of air voids within the soil mass.

The efficiency of this process is assessed by evaluating the dry density achieved during compaction. The point at which the soil reaches its maximum dry density (MDD) corresponds to the optimum moisture content (OMC)—the moisture level that allows the

soil to be compacted most effectively. In this study, the Modified Proctor Test, as prescribed in IS: 2720 (Part 8) – 1983, was conducted on sandy soil to determine its compaction characteristics. The selected soil had a low fines content (less than 10%) and passed through a 20 mm IS sieve, meeting the requirements of IS: 1498–1970 for compaction testing. For each test, approximately 2.8 kg of soil was thoroughly mixed with water in varying amounts to simulate different moisture conditions. The mixture was then compacted in a 1000 cc cylindrical mould in five equal layers, with each layer receiving 25 blows from a 4.89 kg rammer dropped from a height of 450 mm. Multiple tests were performed at different moisture contents to generate a series of dry density values. These values were used to plot a compaction curve, from which the OMC and MDD were derived, providing critical input for evaluating soil suitability in engineering applications.

4.1.4 DIRECT SHEAR TEST (DST)



Figure 4.2 Arrangement of equipment for conducting the Direct Shear Test

The Direct Shear Test is a widely used laboratory method for determining the shear strength characteristics of soils. Its straightforward and rapid procedure makes it ideal for evaluating both cohesive and non-cohesive soils. Due to its reliability and simplicity, this test is frequently employed in geotechnical engineering, especially when assessing the stability of slopes, foundations, and retaining walls. This test yields two fundamental soil parameters: cohesion (c) and the angle of internal friction (ϕ). Cohesion represents the

natural adhesive forces between soil particles, while the angle of internal friction measures the soil's ability to resist sliding under shear stress. The methodology for this test follows the guidelines specified in IS: 2720 (Part 13) – 1986. To perform the test, soil samples were prepared using a cutter measuring 60 mm × 60 mm × 25 mm. Each sample was carefully trimmed and leveled to maintain uniformity before being placed in the direct shear apparatus. The test was carried out at a constant displacement rate of 1.25 mm per minute. Multiple specimens were subjected to varying normal stresses: 0.5 kg/cm², 1.0 kg/cm², and 1.5 kg/cm². The peak shear stress corresponding to each applied normal load was recorded. These results were plotted on a graph with normal stress along the X-axis and peak shear stress along the Y-axis. The cohesion value (c) was determined from the Y-intercept of the best-fit line, while the slope of the line provided the angle of internal friction (ϕ).

4.1.5 CALIFORNIA BEARING RATIO (CBR)

To evaluate the bearing capacity of the subgrade soil, a series of California Bearing Ratio (CBR) tests were performed following the procedures specified in IS: 2720 (Part 16) – 1987. The tests were conducted in an unsoaked condition to reflect field scenarios where the soil is not fully saturated. Each soil sample was compacted in a standard CBR mould with dimensions of 150 mm diameter and 175 mm height using the heavy compaction method to achieve maximum dry density. No soaking period was applied to maintain consistency with typical dry field conditions. After compaction, a surcharge weight was placed on top of the specimen to simulate the overburden pressure. A standard cylindrical plunger of 50 mm diameter was then penetrated into the soil at a constant rate of 1.25 mm/min, and the load readings corresponding to penetration depths of 2.5 mm and 5.0 mm were recorded using a proving ring. The CBR value was determined by comparing the measured load with standard values, and the higher value of the two penetrations was reported as the final CBR. Tests were carried out on both unreinforced soil and soil reinforced with polymeric strips. For the reinforced samples, three configurations were used: a single layer of polymeric strip was placed at half the specimen height (H/2); two

layers were placed at one-third intervals ($H/3$); and three layers were placed at one-fourth intervals ($H/4$). These variations were aimed at assessing the effect of reinforcement depth and distribution on the load-bearing performance of the soil.

4.1.6 DYNAMIC BEHAVIOR OF SOIL BASED ON AVD PARAMETERS

This study investigates the dynamic behavior of soil by analyzing acceleration, velocity, and displacement (AVD) under controlled dynamic loading. These parameters help evaluate how soil responds to transient or cyclic forces such as earthquakes, traffic, or machinery vibrations, and are critical for assessing energy absorption, stiffness, damping, and liquefaction potential. Experiments were conducted using a confined steel tank filled with soil compacted to a specified density and moisture content. A dynamic pulse generator simulated vibratory loads, while a digital vibration meter with accelerometers measured real-time responses in terms of acceleration (g), velocity (mm/sec), and displacement (mm peak). Two types of soil conditions were tested: unreinforced and reinforced. The reinforced soil included polymeric strips (60 kN tensile strength) placed horizontally at depths of 5 cm, 15 cm, and 25 cm within a 30 cm high tank. Accelerometers were positioned at the bottom, middle, and top of the tank for both cases to assess the response throughout the soil profile. The results provided insights into resonant frequency, damping ratio, and dynamic stiffness, which are crucial for seismic design and geotechnical engineering applications.

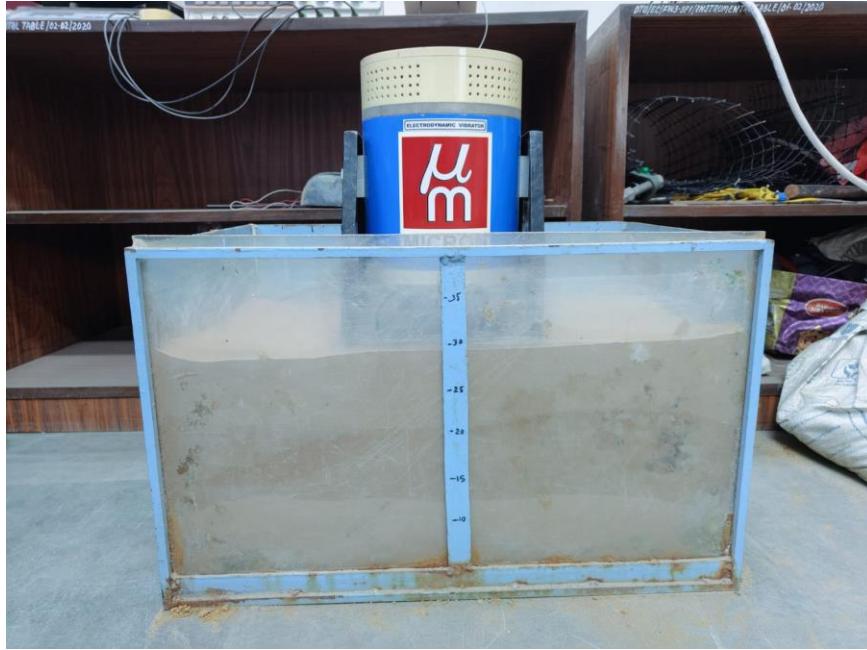


Figure 4.3 Arrangement of equipment for conducting the AVD characteristics of soil.

4.2 NUMERICAL MODELLING

This study models the dynamic behavior of a reinforced earth (RE) wall (3.2 m high) and a tank setup under axisymmetric conditions using finite element analysis. Poorly graded sand was used, modeled with the Hardening Soil model and concrete panels as facing. Simulations were conducted for both reinforced and unreinforced conditions. The Mohr-Coulomb criterion was applied to represent soil's elasto-plastic behavior. A fine mesh with fifteen-noded triangular elements was used, and appropriate boundary conditions were set. Input parameters such as Young's modulus, Poisson's ratio, cohesion, friction angle, dilatancy angle, and dry unit weight are listed.

4.2.1 EVALUATION OF AVD PARAMETERS IN TANK SETUP

To investigate the dynamic behavior of soil, a controlled tank setup was employed with dimensions 0.65 m in length, 0.35 m in height, and 0.25 m in breadth. A vibrator plate with a base length of 0.25 m was positioned centrally at the top surface of the soil to apply dynamic excitation. Two configurations were tested: one without reinforcement (Figure 4.9), and another reinforced with 60 kN polymeric strips (Figure 4.8) both using identical soil properties. The tank, modeled as rigid concrete, was constrained along all three sides to replicate real boundary conditions. The soil and reinforcement behavior were modeled using axisymmetric conditions with a 15-noded triangular finite element mesh. The material properties were considered as elastic and isotropic. Sinusoidal loading was applied at frequencies of 2 Hz, 4 Hz, 7 Hz, and 10 Hz, with a static preload of -0.04 kN/m². The input parameters for the simulation, including Drained type, Young's modulus (E), Poisson's ratio (μ), soil saturated unit weight (γ_{sat}) and dry unit weight (γ_{unsat}), are summarized in Table 4.1.

Table 4.1 Material properties of the subsoil

Parameters	Name	Soil	unit
Material model	Model	Linear Elastic	-
Drainage type	Type	Drained	-
Soil unit weight unsaturated	γ_{unsat}	20	kN/m ³
Soil unit weight Saturated	γ_{sat}	20	kN/m ³
Young's modulus (constant)	E'	$5.0 * 10^4$	kN/m ³
Poisson's ratio	ν	0.3	-
K ₀ determination	-	Automatic	-
Lateral earth pressure coefficient	K _{0,κ}	0.50	-

This numerical modeling provided a comparative understanding of the experimental findings under both reinforced and unreinforced conditions. To capture the dynamic

behavior of the system, PLAXIS 2D's dynamic module was utilized. The simulation was governed by the time-dependent equation of motion:

$$M\ddot{u} + C\dot{u} + ku = F \quad (2)$$

Where;

- M = Mass matrix (defined from soil and structural density)
- C = Damping matrix (calculated using Rayleigh damping parameters)
- K = Stiffness matrix (defined from material stiffness properties such as E', ν)
- F = Time-dependent load vector
- $\ddot{u}$ = Acceleration
- $\dot{u}$ = Velocity
- u = Displacement

In PLAXIS 2D, Rayleigh damping is introduced through user-defined button parameters α_R and β_R using the following relation:

$$C = \alpha_R M + \beta_R K \quad (3)$$

Where;

α_R = controls mass-proportional damping, dominant at lower frequencies

β_R = controls stiffness-proportional damping, dominant at higher frequencies

Dynamic loading was applied using vibratory pulse input at frequencies of 2 Hz, 4 Hz, 7 Hz, and 10 Hz to simulate realistic cyclic excitation. The characteristics of the vibratory pulse generator, including amplitude and frequency ranges, are presented in Table 4.2. To prevent wave reflection and boundary effects, viscous boundaries were applied along the vertical edges of the model. These boundaries absorbed outgoing waves and improved the accuracy of dynamic simulation within the confined tank. In the reinforced model, polymeric strips with a tensile strength of 60 kN were modeled as elastic geogrid elements. Their properties, such as stiffness, thickness, and interface strength, are summarized in Table 4.3.

Table 4.2 Material properties of the vibratory pulse generator

Parameter	Name	Value	Unit
Material Type	Type	Elastic; instropic	-
Normal Stiffness	EA	7.6×10^6	kN/m
Flexural rigidity	EI	2.4×10^4	kNm ² /m
Width (Base plate)	-	0.25	m
Weight	w	0.4	kN/m/m
Poisson's Ratio	ν	0.0	-
Rayleigh α	α	0.5712	-
Rayleigh β	β	1.447×10^{-3}	-

Table 4.3 Material properties of the Polymeric strip (GS60)

Parameter	Name	Value	Unit
Material Type	Type	Elastic	-
Normal Stiffness	EA	7458	kN/m
Flexural rigidity	EI	24×10^3	kNm ² /m
Grade of Reinforced	GS60	60	kN

These elements were embedded in the soil at specified layers to replicate the reinforcement layout used in the experimental tank setup. The numerical results, including AVD parameters, were then compared against experimental observations to assess the influence of reinforcement on the seismic response of the soil system.

4.2.2 DESIGN EQUATION OF REINFORCED EARTH WALL BY FHWA-NHI-10-024

Reinforced Earth Walls (REWs) are a type of mechanically stabilized earth (MSE) structure where soil strength is enhanced using horizontal layers of reinforcement like geogrids or metallic strips. The design of these structures under both static and seismic conditions is essential for their stability and performance. The Federal Highway Administration (FHWA), in its guideline FHWA-NHI-10-024, provides a systematic approach for designing REWs, including parameters for reinforcement interaction, earth

pressure coefficients, and seismic considerations.

4.2.2.1 PULLOUT RESISTANCE FACTOR

i. Static condition

$$F^* = C_i \cdot \tan \phi \quad (4)$$

ii. Seismic Condition

$$F^* = 0.8 C_i \cdot \tan \phi \quad (5)$$

Where;

- C_i = Coefficient of interaction for Pullout
- ϕ = Angle of internal friction

4.2.2.2 INTERFACE FRICTION ANGLE

$$\rho = \tan^{-1}(C_{ds} \tan \phi) \quad (6)$$

Where;

- ρ = Interface Friction angle
- ϕ = Angle of internal friction

4.2.2.3 ACTIVE EARTH PRESSURE COEFFICIENT

$$K_a = \tan^2 \left(45 - \frac{\phi}{2} \right) \quad (7)$$

Where;

- K_a = Active Earth pressure coefficient
- ϕ = Angle of internal friction

4.2.2.4 DYNAMIC EARTH PRESSURE COEFFICIENT (MONONOB-OKABE METHOD)

$$K_{AE} = \frac{\cos^2(\phi - \psi - 90 + \theta)}{\cos \psi \cos^2(90 - \theta) \cos(\beta + 90 - \theta + \psi) \left[1 + \left(\frac{\sin(\phi + \beta) \sin(\phi - \psi - \beta)}{\cos(\beta + 90 - \theta + \psi) \cdot \cos(\beta + 90 + \theta)} \right)^{0.5} \right]^2} \quad (8)$$

Where;

- K_{AE} = Dynamic earth pressure coefficient
- ϕ = Angle of internal friction
- Ψ = Seismic inertial angle
- Θ = wall inclination from vertical
- β = Backfill slope angle

4.2.2.5 MAXIMUM WALL ACCELERATION

$$A_m = (1.45 - A) * A \quad (9)$$

Where;

- A_m = Maximum wall acceleration
- A = Maximum Ground acceleration

4.2.2.6 PULLOUT RESISTANCE OF REINFORCEMENT

$$R_p = F^* \sigma_h C L_e \quad (10)$$

Where;

- R_p = Pullout resistance
- F^* = Pullout resistance factor
- σ_h = Horizontal stress
- C = Reinforcement effective perimeter
- L_e = Effective embedment length

4.2.3 ANALYSIS OF A 3.2 M REINFORCED EARTH WALL USING PLAXIS 2D

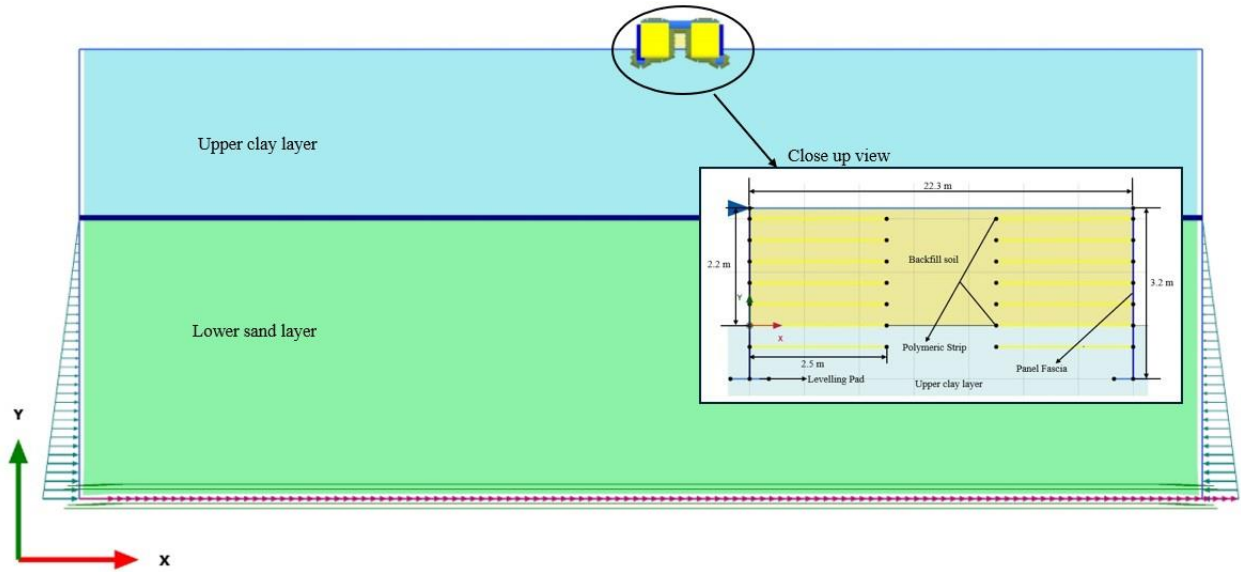


Figure 4.4 2D Numerical Model of 3.2 m High Wall in PLAXIS with Boundary Conditions

To evaluate the static and seismic performance of a polymeric strip-reinforced earth wall, a full-scale numerical model with a height of 3.2 meters was developed using PLAXIS 2D. The wall was designed with two modular facing panels, each 1.6 meters in height. The total wall height included an embedment depth of 1.0 meter below ground surface and 2.2 meters above ground level. Polymeric strip reinforcements, each with a tensile capacity of 60 kN, were placed at horizontal intervals of 0.8 meters, starting 0.4 meters from the base as shown in figure 4.10. The reinforcement length was maintained at 2.5 meters, resulting in a reinforcement-to-wall height ratio greater than 0.7, in line with common design recommendations. The soil profile consisted of a compacted backfill zone and a two-layered foundation soil. Geotechnical parameters including Young's modulus, Poisson's ratio, cohesion, internal friction angle, and dry unit weight were adopted as listed in Table 4.4 for material properties of foundation soil.

Table 4.4 Material properties of foundation soil.

Parameter	Name	Upper clay layer	Lower sand layer	Unit
Material Model	Model	HS small	HS small	-
Type of material behaviour	Type	Drained	Drained	-
Unsaturated Soil unit weight	γ_{unsat}	16	20	kN/m ³
Soil unit weight Saturated	γ_{sat}	20	20	kN/m ³
Secant stiffness in standard drained triaxial test	E_{50}^{ref}	$2.0 \cdot 10^4$	$3.0 \cdot 10^4$	kN/m ²
Tangent stiffness for primary oedometer loading	$E_{\text{oed}}^{\text{ref}}$	$2.561 \cdot 10^4$	$3.601 \cdot 10^4$	kN/m ²
Unloading/ reloading stiffness	$E_{\text{ur}}^{\text{ref}}$	$9.484 \cdot 10^4$	$1.108 \cdot 10^4$	kN/m ²
Cohesion	c'_{ref}	10	5	kN/m ²
Friction angle	φ	18	28	°
Dilatancy angle	ψ	0	0	°
Shear strain at which $G_s = 0.722G_0$	$\gamma_{0.7}$	$1.2 \cdot 10^{-4}$	$1.5 \cdot 10^{-4}$	-
Shear modulus at very small strains	G_0^{ref}	$2.7 \cdot 10^5$	$10 \cdot 10^5$	kN/m ²
Poisson's ratio	ν'_{ur}	0.2	0.2	-

When subjected to cyclic shear loading, the HS small model will show typical hysteretic behaviour. Starting from the small-strain shear stiffness, G_0^{ref} , the actual stiffness will decrease with increasing shear. Figures 4.11 and 4.12 display the Modulus reduction curves, i.e. the decay of the shear modulus with strain. The upper curve shows the secant shear modulus and the lower curve shows the tangent shear modulus. In the HS small model, the tangent shear modulus is bound by a lower limit, G_{ur} .

$$G_{\text{ur}} = \frac{E_{\text{ur}}}{2(1 + \nu_{\text{ur}})} \quad (11)$$

Where;

- $G_{\text{ur}} =$
- $E_{\text{ur}} =$

- $v'_{ur} = \text{Poisson's ratio}$

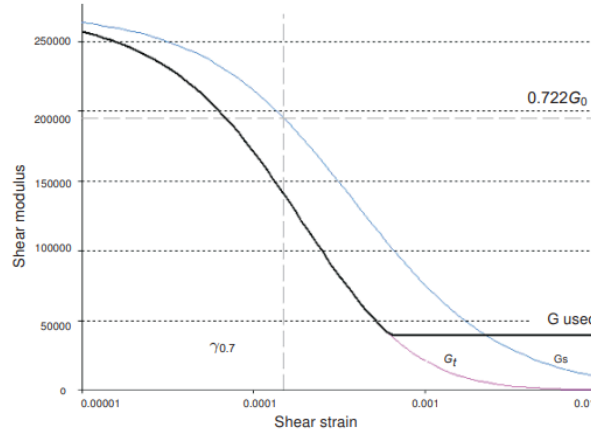


Figure 4.5 Modulus reduction curves for the upper clayey layer (Plaxis2D)

The values of G_{ur}^{ref} for the Upper clay layer and Lower sand layer and the ratio to G_0^{ref} are shown in Table 4.5. This ratio determines the maximum damping ratio that can be obtained.

Table 4.5 G_{ur} values and ratio to G_0^{ref}

Parameter	Name	Upper clay layer	Lower sand layer	Unit
	G_{ur}	39517	41167	kN/m ²
Ratio	G_0^{ref} / G_{ur}	6.75	2.5	

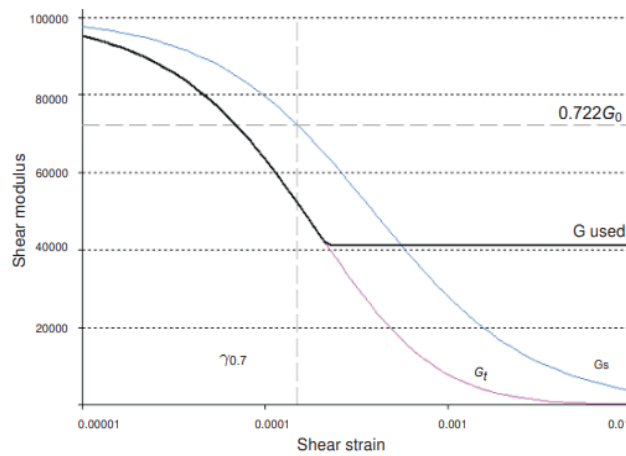


Figure 4.6 Modulus reduction curves for the lower sandy layer (Plaxis2D)

To represent realistic soil behavior in PLAXIS 2D, key parameters were assigned based on results from both laboratory tests and field investigations. Depending on the expected degree of nonlinearity and strain-dependent stiffness reduction, the material behavior was modeled using either the Mohr-Coulomb constitutive models. A drained condition was assumed throughout, reflecting scenarios where pore pressures are able to dissipate over time. Separate unit weights were assigned for unsaturated (above water table) and saturated (below water table) zones to reflect groundwater influence. A constant Young's modulus represented small-strain stiffness, while Poisson's ratio described volumetric deformation behavior. The initial horizontal stress ratio (K_0) was determined using Jaky's formula, suitable for normally consolidated soils:

$$K_0 = 1 - \sin (\phi) \quad (12)$$

Where;

- K_0 = coefficient of earth pressure at rest
- ϕ = angle of internal friction (in degrees)

PLAXIS automatically calculates K_0 when the friction angle is provided. These input parameters, summarized in Table 4.6, served as the foundation for simulating the soil response under both static and dynamic loading conditions. The facing panels were modeled as linear elastic elements with properties such as normal stiffness, unit weight, flexural rigidity, and Poisson's ratio, as summarized in Table 4.5.

Table 4.6 Material properties of the Backfill soil

Parameters	Name	Soil	unit
Material model	Model	Mohr-Coulumb	-
Drainage type	Type	Drained	-
Soil unit weight unsaturated	γ_{unsat}	17.80	kN/m ³
Soil unit weight Saturated	γ_{sat}	19.90	kN/m ³
Young's modulus (constant)	E'	$2.0 * 10^4$	kN/m ³
Poisson's ratio	ν	0.3	-
K ₀ determination	-	Automatic	-
Lateral earth pressure coefficient	$K_{0,\kappa}$	0.50	-

Table 4.6 Material properties of facing panel

Parameter	Name	Value	Unit
Material Type	Type	Elastic; instropic	-
Normal Stiffness	EA	$8.64 * 10^6$	kN/m
Flexural rigidity	EI	14580	kNm ² /m
Width (Base plate)	-	0.015	m
Weight	w	7.2	kN/m/m
Poisson's Ratio	ν	0.2	-
Rayleigh α	α	0.05	-
Rayleigh β	β	0.002	-

The levelling pad, which supports the facing panels and helps distribute loads to the foundation soil, was modeled with its corresponding mechanical properties, including unit weight, stiffness characteristics, and damping behavior. The damping effect for both the facing panel and levelling pad was captured using Rayleigh damping parameters, which were also defined in the model based on modal frequency considerations and are presented in Table 4.6. A 15-noded triangular finite element mesh was used, with localized refinement near the facing and reinforcement regions to ensure accuracy. The soil was modeled using the Mohr-Coulomb failure criterion to simulate its elasto-plastic response. Reinforcements were represented as elastic line elements with perfect bonding to the

surrounding soil. Interface elements were intentionally excluded to maintain simplicity in interaction modeling, as practiced in similar numerical studies. Boundary conditions were assigned by fully restraining vertical and horizontal displacements at the model base and applying horizontal fixity along vertical boundaries.

Table 4.7 Material properties of levelling pad

Parameter	Name	Value	Unit
Material Type	Type	Elastic; instropic	-
Normal Stiffness	EA	6.615*10 ⁶	kN/m
Flexural rigidity	EI	67581	kNm ² /m
Width	-	0.7	m
Weight	w	6.13	kN/m/m
Poisson's Ratio	ν	0.2	-
Rayleigh α	α	0.05	-
Rayleigh β	β	0.002	-

The back of the wall and the top surface were left free to deform. Both static loads (including self-weight and applied surcharge) and seismic loading were applied. For dynamic analysis, a built-in earthquake record available within the PLAXIS 2D software library was used. The selected ground motion corresponds to the 1990 Upland Earthquake, with a local magnitude of 5.4, and is provided in .SMC format directly within PLAXIS. This record was chosen for its compatibility with Seismic Zone IV design criteria, as specified in IS 1893, and its suitability for modeling near-field seismic effects. The accelerogram was applied as a horizontal base excitation at the foundation level of the model. The use of a validated, pre-integrated accelerogram ensured accurate simulation of seismic loading conditions while eliminating the need for external data processing or conversion. The seismic loading was applied in the form of a ground acceleration time history, and additional analyses were performed to evaluate the wall's sensitivity to variations in reinforcement and soil properties. When the tensile strength of the reinforcement was reduced below 60 kN, the wall remained stable under static conditions but failed under seismic loading. Similarly, reductions in soil cohesion (c) and friction angle (ϕ) led to collapse during seismic excitation, despite maintaining stability under

static loads. These results highlight the critical role of adequate reinforcement and soil strength in ensuring the seismic reliability of reinforced earth walls. The overall numerical setup enabled a detailed assessment of lateral displacement, failure patterns, and reinforcement behavior, offering valuable insights into key design parameters governing wall performance under both static and dynamic conditions.

CHAPTER 5

RESULTS AND DISCUSSIONS

5.1 EXPERIMENTAL RESULTS

5.1.1 PARTICLE SIZE DISTRIBUTION ANALYSIS

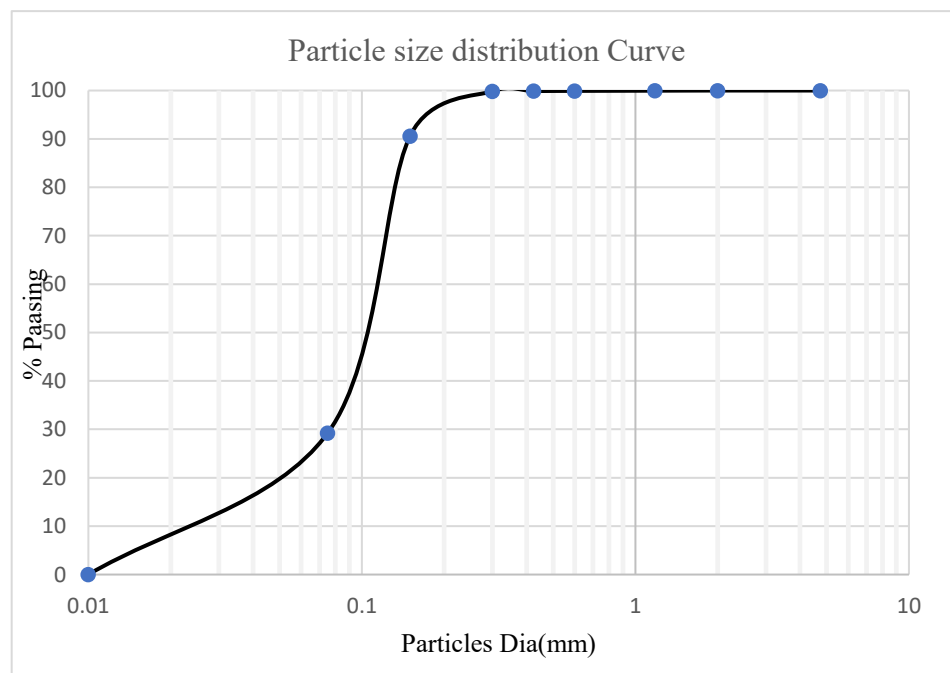


Figure 5.1 Particle size distribution curve

Table 5.1 Uniformity Coefficient and Curvature Coefficient of the soil sample

Parameters	Values
D_{10} (mm)	0.023
D_{30} (mm)	0.076
D_{60} (mm)	0.113
Uniformity Coefficient	5.004
$C_u = D_{60}/D_{10}$	
Curvature Coefficient	2.275
$C_c = D_{30}^2/(D_{10} * D_{60})$	

5.1.2 SPECIFIC GRAVITY TEST

Table 5.2 Specific Gravity of the soil sample

Parameters	Notation	Sample 1	Sample 2
Mass of empty container	M_1 (gm)	35.171	35.352
Mass of container + Dry soil sample	M_2 (gm)	41.169	41.358
Mass of container + soil sample + water	M_3 (gm)	89.480	90.225
Mass of container + water	M_4 (gm)	85.635	86.492
Specific Gravity	G	2.785	2.642

The average specific gravity of the tested soil sample = $(2.785 + 2.642) = 2.713$

5.1.3 MODIFIED COMPACTION TEST

Table 5.3 Water Content and Dry Density relation for compaction test

S. No.	Dry Density, γ_d (kN/m ³)	Water Content, w (%)
1	16.97	6.776
2	17.09	8.623
3	17.37	10.087
4	17.81	11.426
5	17.29	13.362

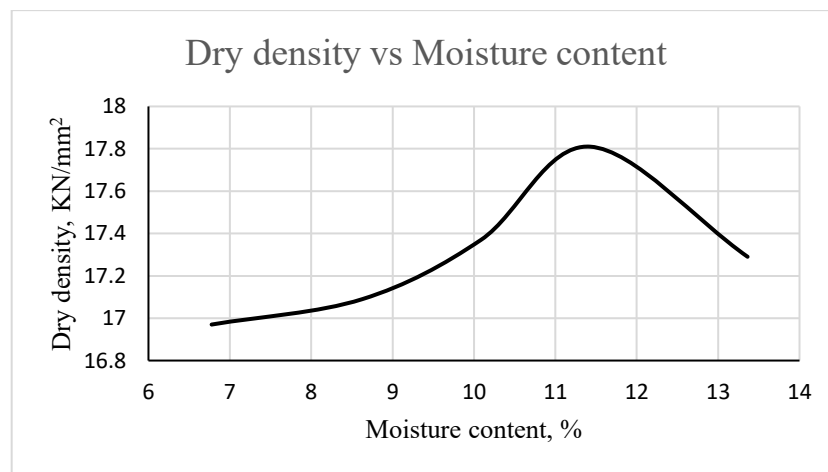


Figure 5.2 Variation of Dry density with moisture content

Table 5.4 Results of compaction test

	OMC (%)	MDD, γ_{dmax}		Corresponding bulk density, γ_b	
		(g/cm ³)	(kN/m ³)	(g/cm ³)	(kN/m ³)
Soil Sample	11.5	1.815	17.8	2.025	19.87

5.1.4 DIRECT SHEAR TEST

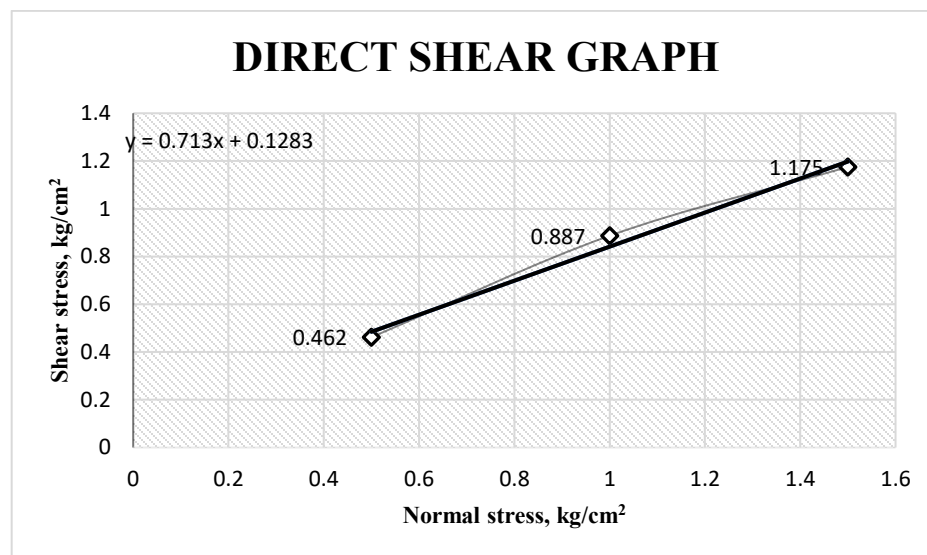


Figure 5.3 Mohr-Coulomb Failure envelope

Table 5.5 Value of cohesion (c) and internal friction angle (ϕ)

Parameter	Value
Cohesion (kg/cm ²)	0.128
Internal Friction Angle (°)	35.48

Table 5.6 Summary of test results

S.No	Soil Property	Values
1	Classification of soil	Poorly graded sand (SP)
2	Optimum Moisture Content (%)	11.5
4	Bulk Unit Weight (γ_b) (kN/m ³)	19.87
5	Dry Unit Weight (γ_d) (kN/m ³)	17.8

6	Cohesion (kg/cm ²)	0.128
7	Angle of internal friction (ϕ)	35.48
8	CBR Value (96 Hrs-Soaked Condition) (%)	16.22

5.2 AVD EXPERIMENT TEST RESULTS

The AVD experiment was conducted to analyze the dynamic behavior of both the steel tank and its numerical model developed in PLAXIS 2D. The results for acceleration, velocity, and displacement obtained from the physical tank experiment and the PLAXIS model simulations are presented below. These findings provide a comprehensive understanding of the tank's response under dynamic loading conditions.

5.2.1 DYNAMIC RESPONSE RESULTS IN TANK MODEL

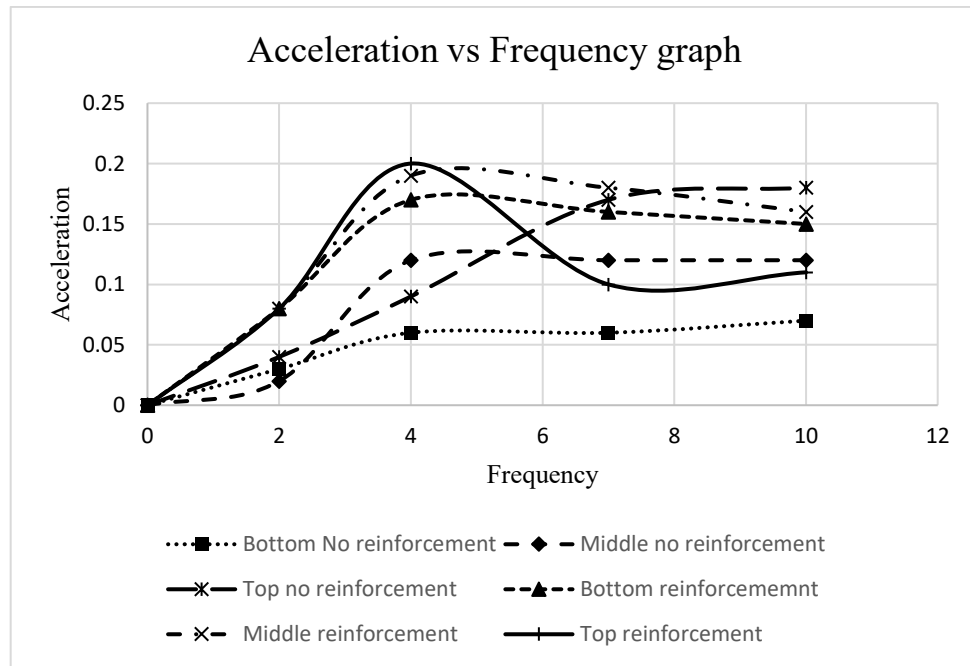


Figure 5.4 Frequency vs. Acceleration Behavior of Soil with Reinforced (60 kN Polymeric Strip) and Unreinforced.

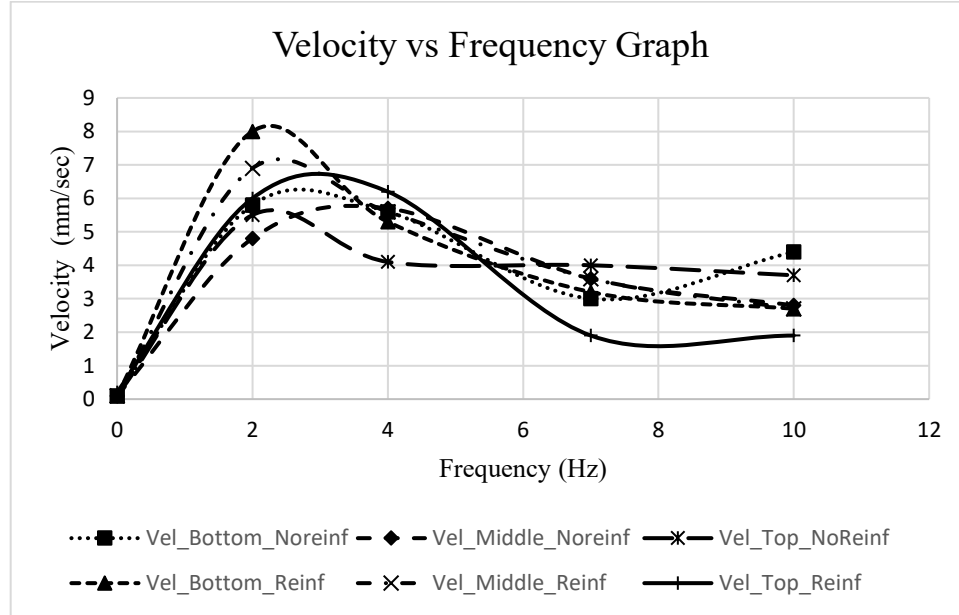


Figure 5.5 Velocity vs. Frequency Behavior of Soil with Reinforced (60 kN Polymeric Strip) and Unreinforced.

A comparative study was conducted to assess the dynamic response of soil using an experimental tank model subjected to harmonic loading at 2 Hz (15 s), 4 Hz (20 s), 7 Hz (25 s), and 10 Hz (30 s). The tests were performed under two conditions—without and with geosynthetic reinforcement (60 kN polymeric strip). In the unreinforced case, a peak acceleration of 0.18 g was observed at the top of the soil during 10 Hz excitation, with acceleration increasing from bottom to top. With reinforcement, the acceleration reduced to 0.11 g, showing a more uniform response across depths (Figure 5.5). At 2 Hz, the unreinforced model showed displacements of 0.225 mm (bottom) and 0.222 mm (top), indicating localized deformation. The reinforced model showed slightly higher displacements of 0.333 mm (bottom) and 0.261 mm (top), but these were more evenly distributed (Figure 5.6). Additionally, the velocity response of the soil under different frequencies demonstrated that reinforcement plays a crucial role in mitigating dynamic effects (Figure 5.7). These results demonstrate that geosynthetic reinforcement effectively reduces acceleration and enhances the stability of the soil under dynamic loading conditions.

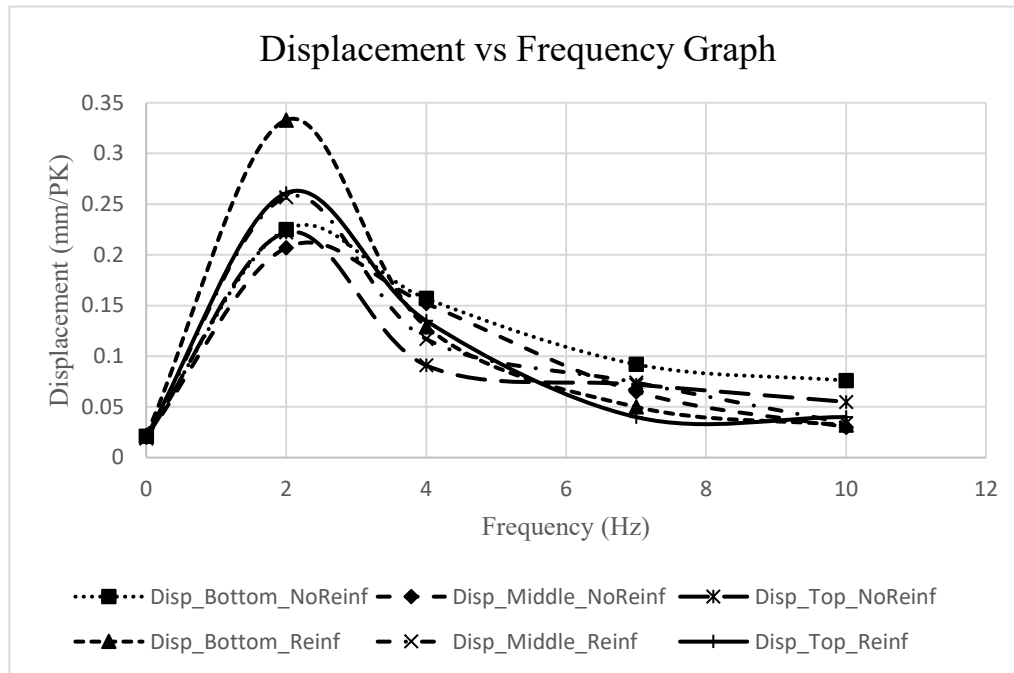


Figure 5.6 Displacement vs. Frequency Behavior of Soil with Reinforced (60 kN Polymeric Strip) and Unreinforced.

5.3 RS WALL DESIGN HEIGHT 3.20 M (ACCORDING TO FHWA-NHI-10-024)

5.3.1 INPUT DATA

- Soil Reinforcement = GEOSTRIP
- Facial Type = SEGMENTAL PANEL

Table 5.7 Wall Geometry

Parameter	Value
Design height of wall, H	3.20 m
Depth of Embedment, E	1.00 m
Exposed Height of wall above the finished level, H _e	2.20 m
Road Material thickness, T _m	0.44 m

Table 5.8 External Surcharge

Parameter	Value
Dead load from road pavement, q_1	9.68 kN/m ²
Live load, q_2	24.00 kN/m ²
strip load, S_L	25.00 kN/m ²
Width of friction slab, b (Strip Load)	2.30 m
Distance from wall edge to centre of strip load, d	1.15 m

Table 5.9 Wall Inclination

Parameter	Value
Measured from vertical, w	0 °
Measured from horizontal, θ	90 °

Table 5.10 slopes

Parameter	Value
Front slope angle, β_s	0°
Back slope angle, β	0°
Back slope rise, S	0.0m

Parameter of Backfill & Foundation soilReinforced fill, Y Retained fill, b foundation soil considered as a uniform soil, f

Y , kN/m ³	Φ , °	c , kN/m ²
20	31	N/A
20	31	N/A
18	30	0

Table 5.11 Reinforcement and Reinforced soil parameters

Parameter	value
Coefficient of interaction for pullout	0.9
coefficient of Interaction for Direct Sliding	0.9
Pullout Resistance factor- Static	0.541
Pullout Resistance factor- Seismic	0.433
Reinforced effective unit parameter, C	2.00
Scale correction factor, α	1.00
Interference friction angle for direct sliding,	28.40
Reduction factor for connection break, C_{ru}	1.00

Table 5.12 Earth pressure coefficient

Parameter	Value
Reinforced, K_{ar}	0.320
Retained, K_{ab}	0.320

Table 5.13 Dynamic Earth pressure coefficient

Parameter	Value
Dynamic earth pressure coefficient, K_{AE}	0.425
Dynamic earth pressure coefficient increment, ΔK_{AE}	0.105

Table 5.14 Seismic parameter

Parameter	Value
Seismic Zone	IV
Maximum ground acceleration Coefficient, A	0.12
Maximum wall acceleration coefficient, A_m	0.160

Table 5.15 Facia panel details

Parameter	Value	unit
Height of facial Panel, H_p	1.60	m
Average width of facial panel, T_p	2.00	m
Thickness of facial panel, W_p	0.18	m
Unit weight of facial panel, γ_p	25.00	kN/m ²

Table 5.16 Properties of the polymeric geogrid strip used for reinforcement

S. No.	Geostrip type	Ultimate Tensile Strength (T_{ul}), kN	Width of Geostrip (w), M	Strength Reduction Factor, RF			Total Reduction Factor, RF_{Total}	Stiffness, (EA), KN/m
				Creep, RF_{CR}	Installation Damage, RF_{ID}	Durability RF_D		
1	GS20	20	0.09	1.43	1.05	1.21	1.817	
2	GS30	30	0.09	1.43	1.05	1.21	1.817	1835
3	GS40	40	0.09	1.43	1.05	1.21	1.817	5307
4	GS50	50	0.09	1.43	1.03	1.21	1.782	
5	GS60	60	0.09	1.43	1.03	1.21	1.782	
6	GS75	75	0.09	1.43	1.03	1.21	1.782	
7	GS85	85	0.09	1.43	1.03	1.21	1.782	
8	GS100	100	0.09	1.43	1.03	1.21	1.782	

5.3.2 LOAD AND RESISTANCE FACTORS

A. INTERNAL STABILITY

Table 5.17 Internal stability

Parameter	Value
Load factor for vertical earth pressure	1.35
Load factor for earthquake loads	1.00
Load factor for live surcharge	1.75
Load factor for dead load surcharge	1.50

Table 5.18 Resistance Factors for Geosynthetics

Resistance factor	Static	Combined static / Seismic
Reinforcement tension (strength)	0.90	1.20
Reinforcement tension in connectors	0.90	1.20
Reinforcement pullout	0.90	1.20

B. EXTERNAL STABILITY

Table 5.19 Limits for external stability

Parameter	Value
Load factor Sliding and Eccentricity: static	1.00
for vertical Bearing capacity: Static	1.35
earth pressure Sliding and Eccentricity: combine static/ seismic	1.00
Bearing capacity: combine static/ Seismic	1.35
Load factor of active lateral earth pressure	1.50
Load factor of active lateral earth pressure during earthquake	1.50
Load factor for earthquake loads	1.00
Load factor for live load surcharge	1.75
Load factor for dead load surcharge	1.50

Table 5.20 Limits for Resistance factors for geosynthetics

Resistance factors for geosynthetics	Static	Combined static/ seismic
Sliding for reinforced soil and foundation soil	1.00	1.00
Sliding for reinforced soil and reinforcement	1.00	1.00
Bearing capacity for shallow foundation	0.65	0.65

Table 5.21 Trail for stability analysis

Parameters	Value
The minimum trail length of reinforcement (Greater of 0.7H & 3m)	3.00
Provide length of reinforcement at bottom layer	3.00
The vertical spacing between reinforcement	0.80

Table 5.22 Results summary for stability of RE wall

Parameter		Layer 1	Layer 2	Layer 3	Layer 4
Elevation (z), m		0.40	1.20	2.00	2.80
Length of Geostrip (L), m		3.00	3.00	3.50	5.20
Type of Geostrip		GS40	GS40	GS30	GS30
No. of Geostrips per m length		2.0	2.0	2.0	2.0
Sliding	(Static)	1.36	1.70	2.70	7.90
	(Seismic)	1.14	1.47	2.44	7.58
Overturing	(Static)	3.06	5.15	14.88	172.79
	(Seismic)	2.49	4.36	13.27	165.20
Eccentricity	(Static)	0.172	0.111	0.062	0.062
	(Seismic)	0.209	0.128	0.065	0.062
Strength	(Static)	1.09	1.27	1.14	1.41
	(Seismic)	1.39	1.62	1.44	1.71
Pullout	(Static)	1.40	1.16	1.09	1.24
	(Seismic)	1.37	1.13	1.03	1.02

5.3.3 SEISMIC MODELLING OF A 3.2M REINFORCED WALL IN PLAXIS 2D

5.3.3.1 CASE 1: INADEQUATE REINFORCEMENT

In the first failure case, the 3.2 m reinforced earth wall was modeled with lower-strength polymeric strips (30 kN and 20 kN), embedded at 2.5 m length and spaced at 0.4 m. The soil parameters ($c = 0.1280$ kPa, $\phi = 35.48^\circ$) were kept constant. During the free vibration phase, the model failed with the error “NaN found in stiffness matrix element,” indicative of numerical instability from excessive deformation as shown in figure 5.8 The failure is attributed to insufficient tensile strength in the upper reinforcement layers, leading to the loss of confinement and internal collapse. This behavior is consistent with seismic amplification effects, where the upper portion of the wall experiences higher dynamic demand. The reinforcement was unable to dissipate the induced energy effectively, resulting in structural failure. This case emphasizes that adequate reinforcement strength is essential to withstand seismic loads and maintain wall integrity

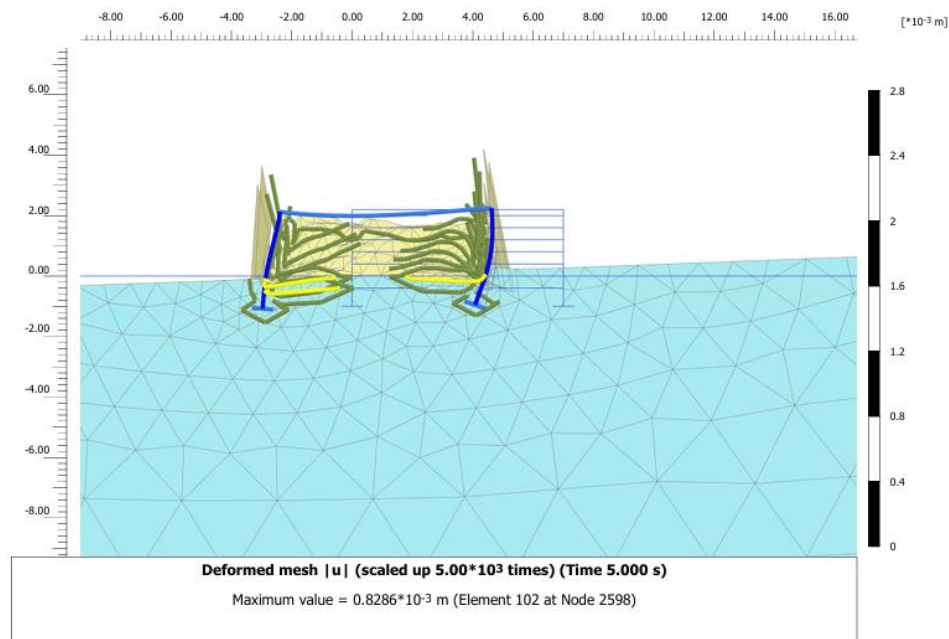


Figure 5.7 Deformed Mesh at Free Vibration Phase showing failure pattern with 30/20 kN polymeric strips.

5.3.3.2 CASE 2: LOW SHEAR STRENGTH SOIL

In the second failure scenario, the model used high-strength 60 kN polymeric strips, but the soil was assigned poor shear strength properties ($c = 10$ kPa, $\phi = 18^\circ$). The simulation failed during the construction phase, producing the error “Soil body seems to collapse”. The low friction angle led to shear failure under self-weight and construction loads as shown in figure 5.9 Despite adequate reinforcement, the weak soil lacked the capacity to provide lateral and vertical support to the system. The deformed mesh indicated large displacements and soil spreading, characteristic of bearing and internal shear failure. This case demonstrates that even robust reinforcement systems require compatible soil properties to function effectively, reinforcing the principle that soil-reinforcement interaction is a dual-responsibility system for ensuring seismic performance.

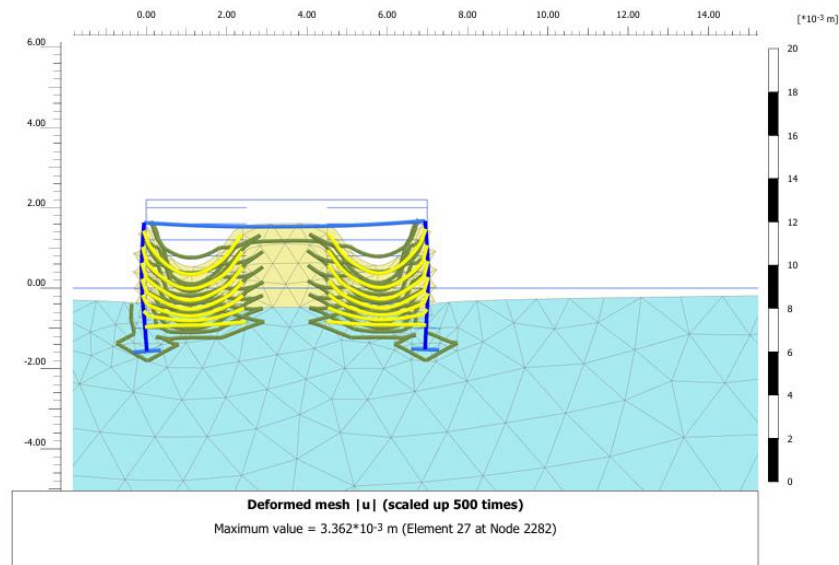


Figure 5.8 Deformed Mesh at Free Vibration Phase showing failure pattern with 30/20 kN polymeric strips.

5.3.3.3 CASE 3: ADEQUATE REINFORCEMENT AND SOIL:

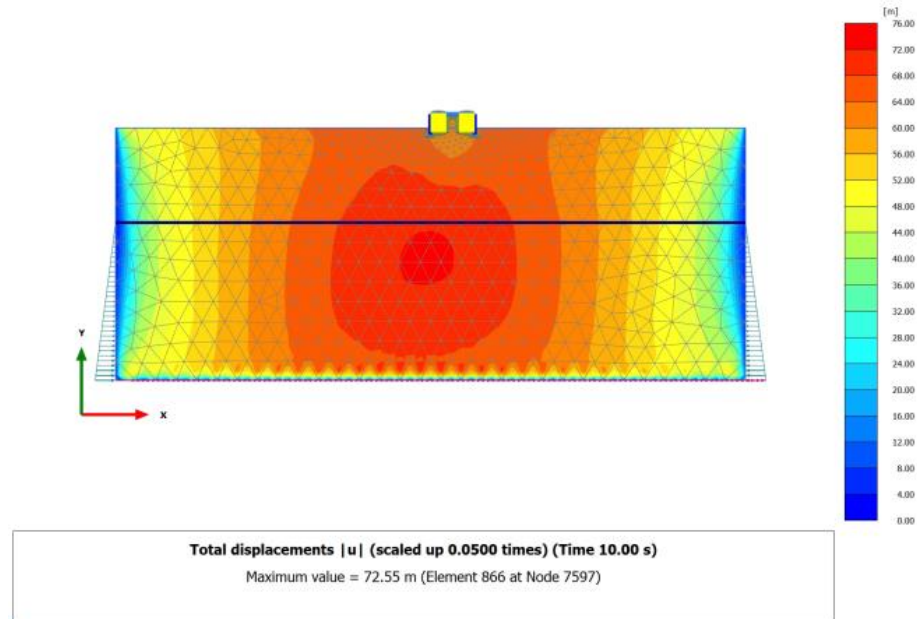


Figure 5.9 Total Displacement Contours at End of Seismic loading for Wall with 60 kN Polymeric Strips.

The seismic behavior of the 3.2 m high reinforced earth wall was thoroughly assessed using PLAXIS 2D through dynamic simulations, focusing on time history analyses of acceleration, velocity, and displacement. The model incorporated polymeric reinforcement strips with a tensile strength of 60 kN, spaced at 0.4 m vertically and extended to a length of 2.5 m, embedded in soil with cohesion of 0.1280 kPa and an internal friction angle of 35.48° . This configuration demonstrated stable performance during both construction and seismic loading phases. Time history results revealed peak acceleration values of approximately 165 m/s^2 at the wall crest (Node 1986), indicating seismic amplification, while the base (Node 4637) showed comparatively lower values, reflecting effective damping behavior as shown in figure 5.11. Velocity response peaked at around 16 m/s, with a clear rise and decay trend indicative of energy dissipation, and displacement stabilized near 65 mm, as shown in figure 5.12 remaining within serviceability limits. The total displacement contour confirmed a maximum deformation

of 72.55 m at Node 7597, distributed uniformly across the structure, emphasizing the wall's ability to maintain structural integrity under seismic loading as shown in figure 5.13. The combination of high-strength reinforcement and adequate soil parameters enabled controlled deformation, minimized differential movement, and prevented failure. These findings underscore the importance of proper reinforcement-soil interaction in seismic design, ensuring sufficient tensile capacity and shear resistance to withstand dynamic stresses effectively.

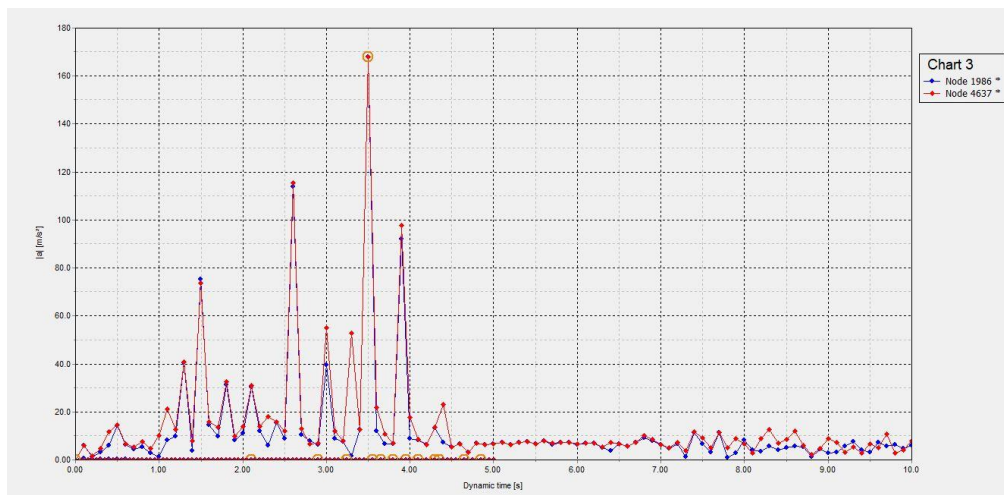


Figure 5.10 Time History Plot – Acceleration vs Dynamic time (Top and Bottom nodes of wall)

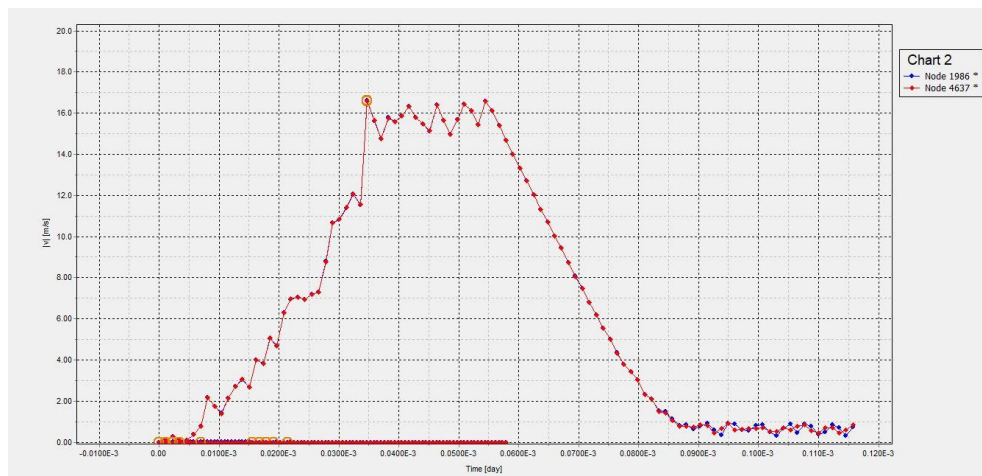


Figure 5.11 Time History Plot – Velocity vs Dynamic time (Top and Bottom nodes of wall)

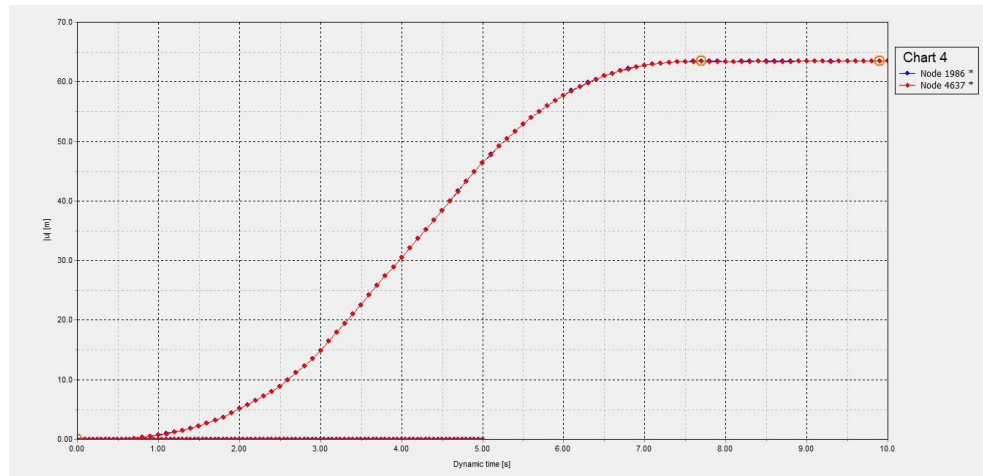


Figure 5.12 Time History Plot – Displacement vs Dynamic time (Top and Bottom nodes of wall)

Table 5.23 Comparative Summary of Seismic Performance for Reinforced Earth Wall Cases

Parameter	Case 1: Inadequate Reinforcement	Case 2: Low shear strength soil	Case 3: Adequate reinforcement and soil
Reinforcement	30/20 kN polymeric strip	60 kN polymeric strips	60 kN polymeric strips
Soil Cohesion (c)	0.1280	10.0 kPa	0.1280
Soil friction angle (ϕ)	35.48	18	35.48
Error/ outcome	NaN in stiffness matrix (Error 36)	Soil collapse during construction (Error 101)	Stable under seismic and construction loading
Max. Displacement	-	-	~65 mm (top), ~72.55 m total (Node 7597)
Acceleration	-	-	~165 m/s ²
Velocity	-	-	~16 m/s
Remarks	Failure due to inadequate tensile strength	Failed due to lower shear strength soil	Passed – Effective soil-reinforcement interaction

CHAPTER 6

CONCLUSIONS

The present study involved a comparative analysis of static and seismic behavior of a 3.2 m high reinforced earth wall using polymeric reinforcement materials of varying tensile strengths, conducted through PLAXIS 2D numerical modelling. The objective was to evaluate the stability and deformation characteristics of the wall under different reinforcement-soil configurations during dynamic loading.

- Three reinforcement configurations (30 kN, 40 kN, and 60 kN) were evaluated under different soil conditions. Walls with low-strength reinforcement and poor soil properties exhibited failure, whereas the combination of 60 kN polymeric reinforcement and stronger soil demonstrated satisfactory stability under both construction and seismic loading conditions.
- The stable configuration exhibited peak acceleration of $\sim 165 \text{ m/s}^2$ and velocity of $\sim 16 \text{ m/s}$, with permanent displacement limited to $\sim 65 \text{ mm}$ at the crest and a maximum deformation of $\sim 72.55 \text{ mm}$.
- Dynamic time-history plots indicated effective damping, uniform deformation distribution, and controlled energy dissipation throughout the wall height.
- The reinforcement-soil interaction played a critical role in performance; insufficient tensile strength or weak soil properties led to failure, while a balanced combination ensured wall stability.

This study validates the effectiveness of high-strength polymeric strips and suitable backfill soil in resisting seismic loading in reinforced earth structures. The findings can aid in optimizing geosynthetic reinforcement design and provide a framework for selecting appropriate soil-reinforcement combinations in seismic regions. The results further demonstrate the capability of PLAXIS 2D as a powerful tool for evaluating seismic behavior in reinforced soil systems.

6.2 SCOPE FOR FUTURE WORK

Future work can include using real earthquake data, 3D modeling, studying groundwater effects, advanced material models, long-term performance, and experimental validation to improve seismic design of reinforced earth walls.

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