



Comparison Of Insitu And Numerical Analysis Of An Embankment On Soft Clay Using Plaxis-2D

Sonia, B.^{*}, Bhanuchitra, M.² and Padmavathi, V.³

1Post Graduate Student, 2Research Scholar, 3Professor in Dept. of Civil Engineering

JNTUH University College of Engineering, Science and Technology Hyderabad, India

Abstract: The stability of an embankment during and post-construction is predominantly influenced by the magnitude of displacements occurring beneath the embankment. Numerous studies have documented the adverse effects of excessive displacements on the performance of the embankment and nearby structures. Various ground improvement techniques are employed, along with the deployment of in-situ instrumentation, to mitigate significant issues such as large displacements and potential embankment failure. This study leverages data from multiple case studies on embankments and utilizes finite element software, PLAXIS 2D, to analysed displacements. The lateral displacements and settlements were computed using PLAXIS 2D. The results, including lateral displacements, settlements, and pore water pressures derived from finite element analysis (FEA), were then systematically compared with parameters obtained through experimental, numerical, and observational methods documented in existing case histories.

Keywords: Soft clay, embankment, lateral displacement, settlement, PLAXIS 2D

1.Introduction: An embankment is a raised structure made of compacted soil or aggregates, supporting roads, railways, or acting as water barriers. Constructing embankments on soft clay poses geotechnical challenges due to low shear strength, high compressibility, and poor bearing capacity, necessitating advanced engineering solutions to ensure stability and minimize settlement. Soft ground under embankments undergoes consolidation and plastic flow, complicating the relationship between displacement and failure. Deformation behavior is crucial for stability, with displacement monitoring providing a more accurate and practical method than earth pressure or pore water pressure measurements. Key parameters for modelling embankment behavior on soft clay include bulk unit weight (γ), undrained shear strength (S_u), undrained modulus of elasticity (E_u), drained modulus of elasticity (E), Poisson's ratio (ν), and lateral earth pressure coefficient (k_0). Accurate parameter determination and selecting a suitable constitutive model, like Mohr-Coulomb or Modified Cam Clay, are essential for predicting settlements, displacements, and pore pressures. The Mohr-Coulomb model, requiring cohesion (c) and internal friction angle (ϕ), is widely used due to its simplicity and efficiency in geotechnical analyses, despite its limitations in capturing complex soil behavior. Proper parameter selection and model choice are critical for reliable numerical simulations and safe embankment designs on soft soils.

2. Case Studies

1 Case 1: Rio De Janeiro Embankment

An instrumental trial embankment was constructed by the Brazilian Highway Research Institute near Rio de Janeiro on a flat, swampy site with an 11-meter-thick clay deposit beneath sand and gravel layers. The clay's plastic limit averaged 13.2 kN/m^3 , with water content ranging from 60%–80% at the top to 50%–60% at the bottom. In fig:1 and In fig:2 The embankment measured 40 meters in width, 80 meters in length, and included 0.70-meter-high berms spaced 20 meters apart. It featured slopes of 1V:6H and 1V:2H as per Mesri and Trak's recommendations. Instrumentation included surface markers, inclinometers, piezometers, extensometers, and settlement plates. The embankment reached a height of 2.5 meters before gradual general slip began at 3.1 meters, preventing further height increases.

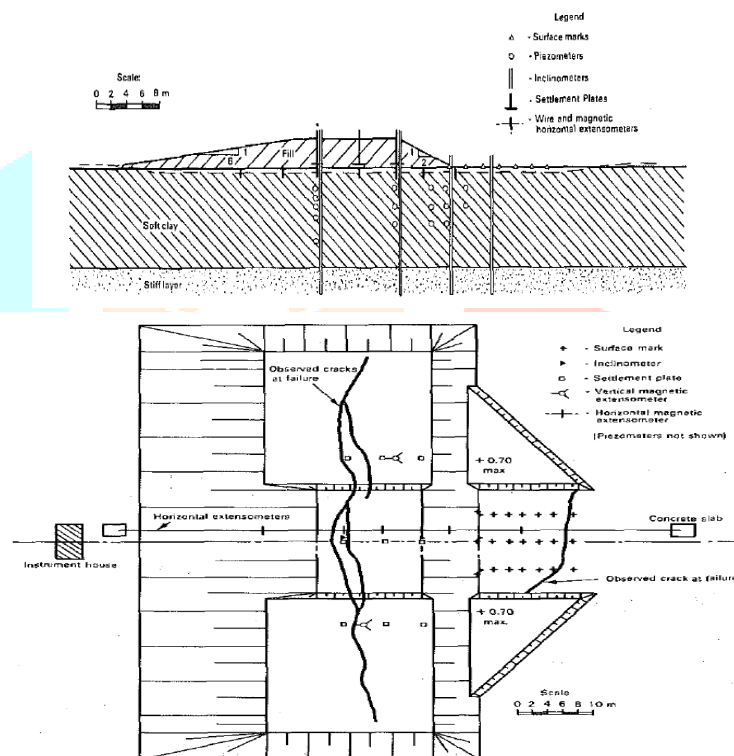


Fig:1 Plan and elevation of the embankment

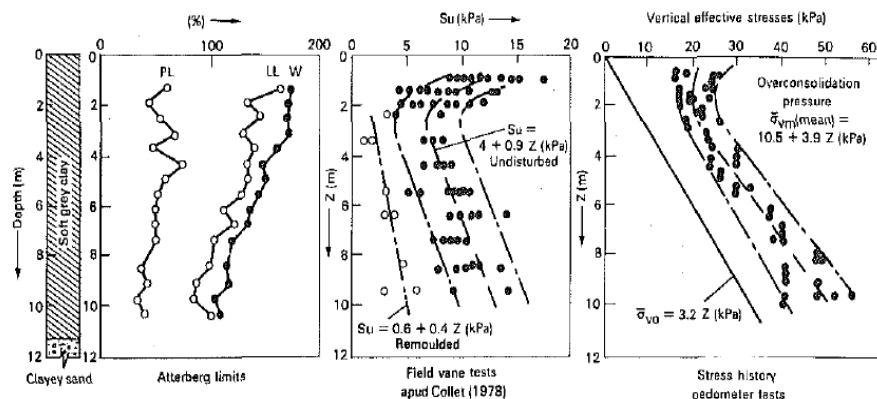


Fig:2 soil profile for Rio De Janeiro

2 Case 2: Ariake Embankment

In Saga, Japan, a test embankment was constructed on soft Ariake clay near the Ariake Sea. The embankment had base dimensions of 46.8 m by 21.8 m, with side slopes of 1:1.8 and 1:2, and top dimensions of 37.8 m by 13.8 m. Built at a rate of 0.1 m/day over 50 days, it reached a failure height of 2.5 m and a final height of 6.3 m with a width of 26.4 m. The embankment fill had an average unit weight of 19.0 kN/m³. Geotechnical data, including plastic limit, liquid limit, natural water content, void ratio, and unit weight, were collected from boreholes. Data on lateral displacements, settlements, and excess pore water pressure during construction, provided by Jinchun Chai et al. (2012), illustrated the embankment's behavior on soft ground.

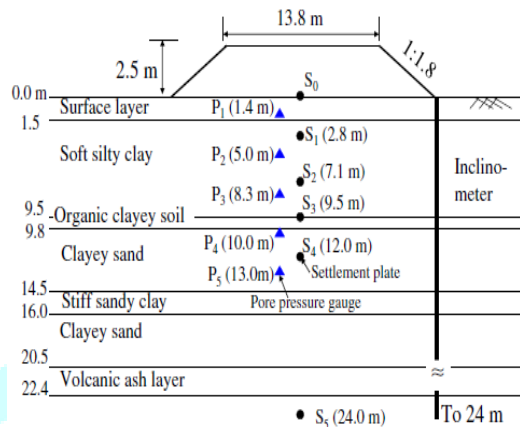


Fig:3 Plan and elevation of the embankment

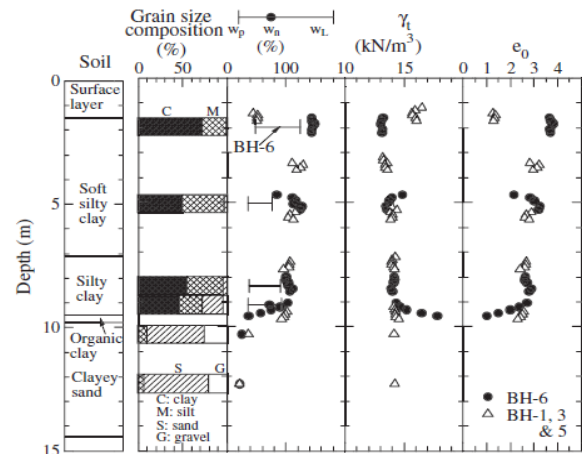


Fig:4 soil profile for Rio De Janeiro

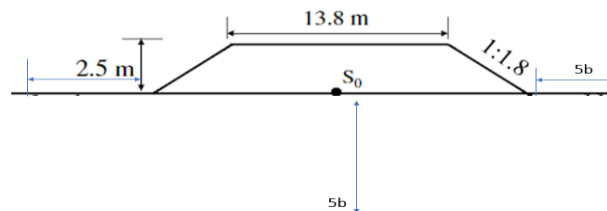


Fig:5 Elevation

Methodology:

Finite element analysis using PLAXIS 2D software was conducted under plain strain conditions to simulate real-world conditions, as shown in Figure 6. The model used the Mohr-Coulomb criterion for both the foundation and embankment soil, considering different drainage conditions (undrained and drained). The model was set up with 15-noded triangular elements, employing a medium to coarse mesh, refined in high-stress areas. Soil parameters, including bulk unit weight (γ), undrained shear strength (S_u), undrained modulus of elasticity (E_u) for soft clay, drained modulus of elasticity (E) for embankment soil, and Poisson's ratio (ν), are listed in Table 1. The deformation results for the insitu were compared with numerical analysis

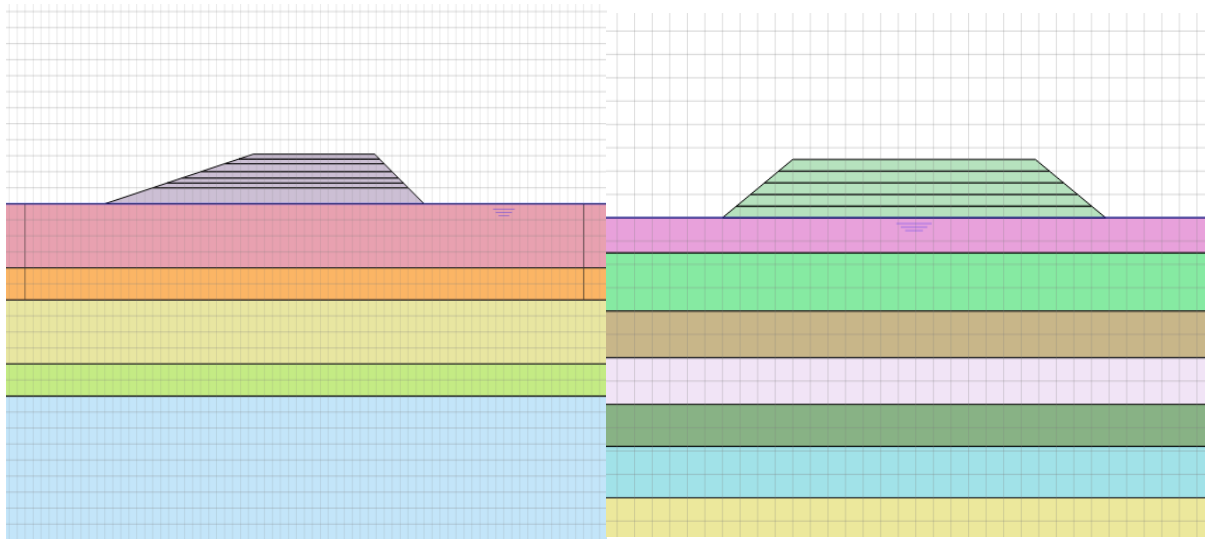


Fig 6. PLAXIS Model of Embankments

Table 1: Soil Parameters considered in FEM

Name of the embankment	Depth (m)	Unit weight of soil, γ (kN/m ³)	Undrained Young's modulus E_u (MPa)	Poisson's ratio (ν)	Undrained shear strength (kPa) S_u	Effective friction angle, ϕ (°)
Rio De	0.0-4.0	15	1.44	0.499	9	-
	4.0-6.0	15	1.53		9	-
	6.0-8.0	14.8	2.3		10	-
	8.0-10.0	14.5	2.1		14	-
	10.0-12.0	14.5	15.0	0.35	5.4	28
	12.0-200.0	15.2	50	0.30	5	36
Ariake	2.0-5.0	16.0	3.0	0.499	15	-
	5.0-7.0	15.0	4.95		19	-
	7.0-10.0	15.0	6.3		21	-
	10.0-12.0	15.0	6.3		21	-
	12.0-15.0	18.0	25.0		5	30
	15.0-20.0	18.0	20.0	0.35	20	35
	20.0-55.0	18.0	3.75	0.30	20	35

3. Results and Discussion

3.1.1 Settlements

Case 1. Rio de Janeiro

The settlements result from FEM analysis to compare the Rio de Janeiro case study with predicted FEM analysis. The analysis of embankment construction phases revealed that settlements consistently decreased in each phase when compared to the unreinforced embankment shown in Figure:7(a) The settlements were increased significantly throughout the filling stage in actual field conditions, particularly between heights of 2.5 and 2.8 m, with maximum settlements are ranging from 238 to 250 mm.

Case 2: Ariake Embankment

In fig:7(b) to compare the Ariake case study with predicted FEM analysis. The embankment construction was completed in 50 days. the embankment height was 2.5m.were constructed in FEM . the settlements were decreased in FEM analysis as compare to measured analysis was 180 mm. due to the plastic strain was concentrated largely in the very soft layer, and squeezing the soil sideways meant that there might be slight stress decrease deeper down.

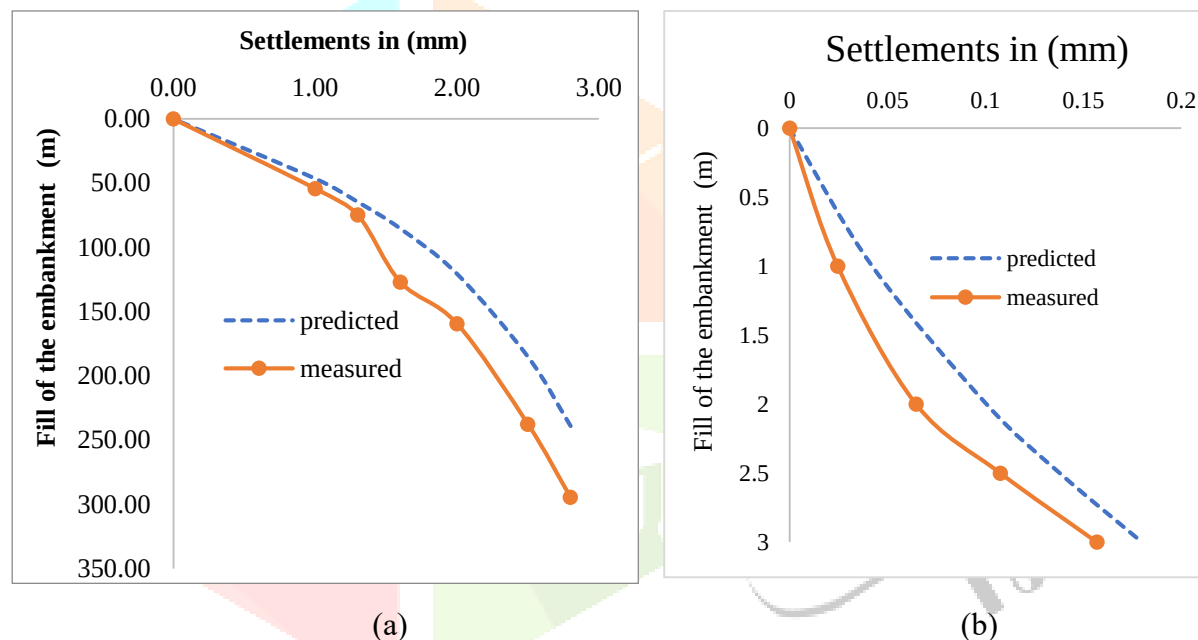


Fig.7. Settlements at the centre of the embankment

The observed settlement differences can be attributed to the consideration of undrained parameters during the staged construction of the embankment. In this context, the undrained Young's modulus is assumed to be greater than the field-drained Young's modulus, leading to an overestimation of the predicted response. This discrepancy arises because, in real-world conditions, pore water pressure gradually dissipates over time, whereas in Finite Element Method (FEM) analysis, it is assumed to have an immediate impact. As a result, the FEM analysis overlooks the gradual dissipation of pore pressure, causing the predicted behavior to appear stiffer than it actually is. Additionally, the accuracy of the Mohr-Coulomb method is highly dependent on the precise determination of key parameters such as cohesion (c), the angle of internal friction (ϕ), and the undrained Young's modulus. These parameters are inherently challenging to measure accurately, leading to potential variability and uncertainty in the analysis results. Reliable predictions require precise characterization of these parameters, as any inaccuracies can significantly affect the outcomes of geotechnical assessments.

3.1.2 Lateral Displacements

a) At the toe of the embankment and along the vertical plane at the toe

Case 1: Rio De Janeiro Embankment

Lateral displacements at the toe of the embankment show a gradual increase with the increase of fill height. The displacements begin to diverge more significantly between the cases at intermediate heights, with a notable increase during the filling phase between heights of 2.5 and 2.8 meters, resulting in displacements of 250 mm to 300 mm displacements., there was an initial increase in lateral displacements; however, these displacements decreased by 36% by the end of construction, as illustrated in Fig.8, The maximum lateral displacements are observed along the vertical profile at the toe, detailed in Fig.9. In the unreinforced scenario, the maximum lateral deformation occurred at a depth of 3.6 meters from the ground level, ensuring better stability throughout the embankment height.

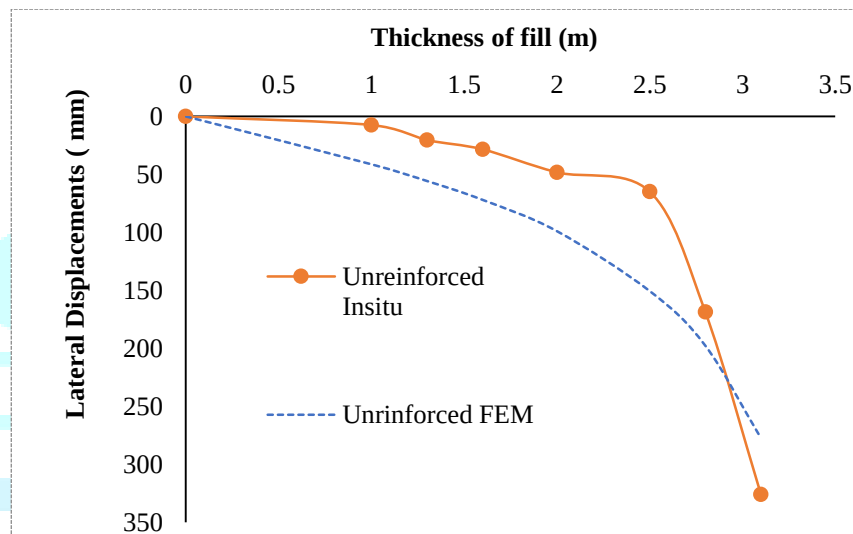


Fig.8 Lateral displacements vs thickness of fill

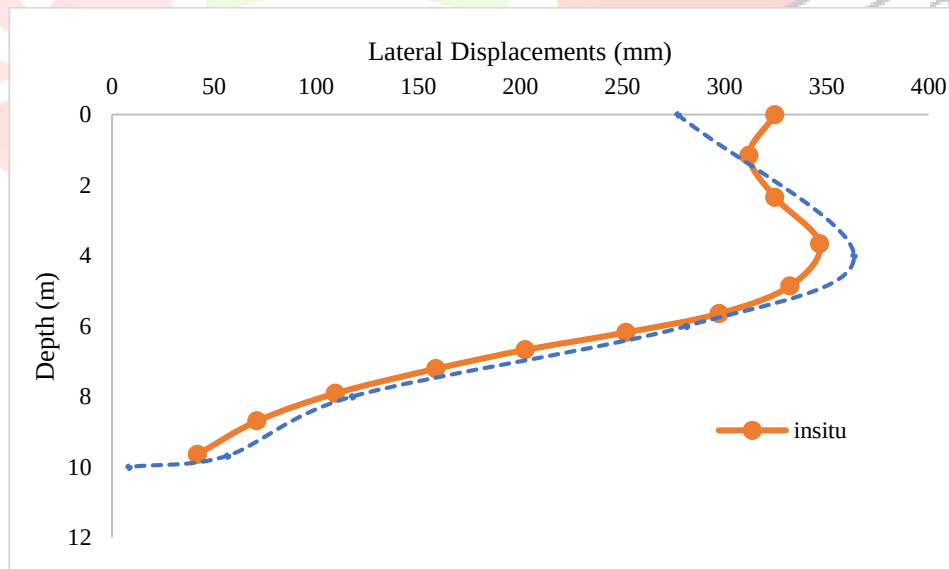


Fig. 9. Lateral displacements along the vertical plane at toe of the embankment

Case 2: Ariake Embankment

Lateral displacements at the toe of the embankment show a gradual increase with the increase of fill height. The displacements begin to diverge more significantly between the cases at intermediate heights, with a notable increase during the filling height of 2.5 meters, resulting in displacements of 40 mm displacements., there was an initial increase in lateral displacements; however, these displacements decreased by 40% by the end of construction, as illustrated in Fig.11., The maximum lateral displacements are observed along the vertical profile at the toe, detailed in Fig.11. In the unreinforced scenario, the maximum lateral deformation occurred

at a depth of 3.6 meters from the ground level, ensuring better stability throughout the embankment height. In the field, lateral displacement data for the embankment fill were not recorded in the given case study. As a result, Figure 10 presents the Finite Element Method (FEM) analysis results, which indicate a maximum predicted lateral displacement of 180 mm.

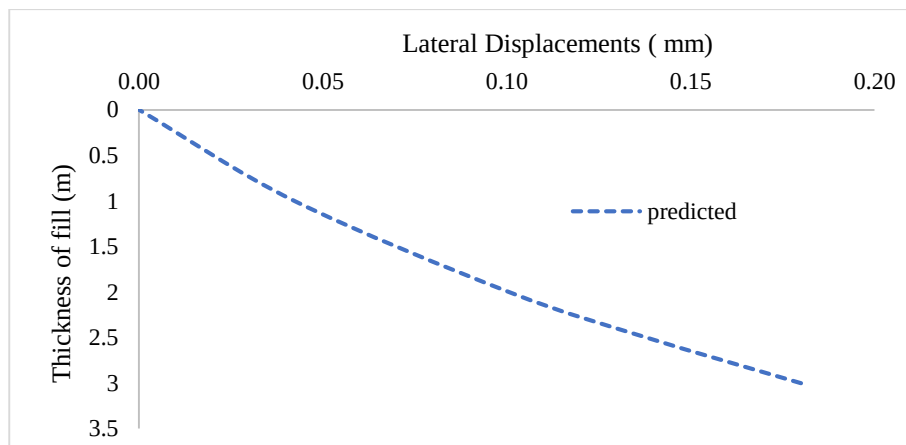


Fig.10 Lateral displacements vs thickness of fill

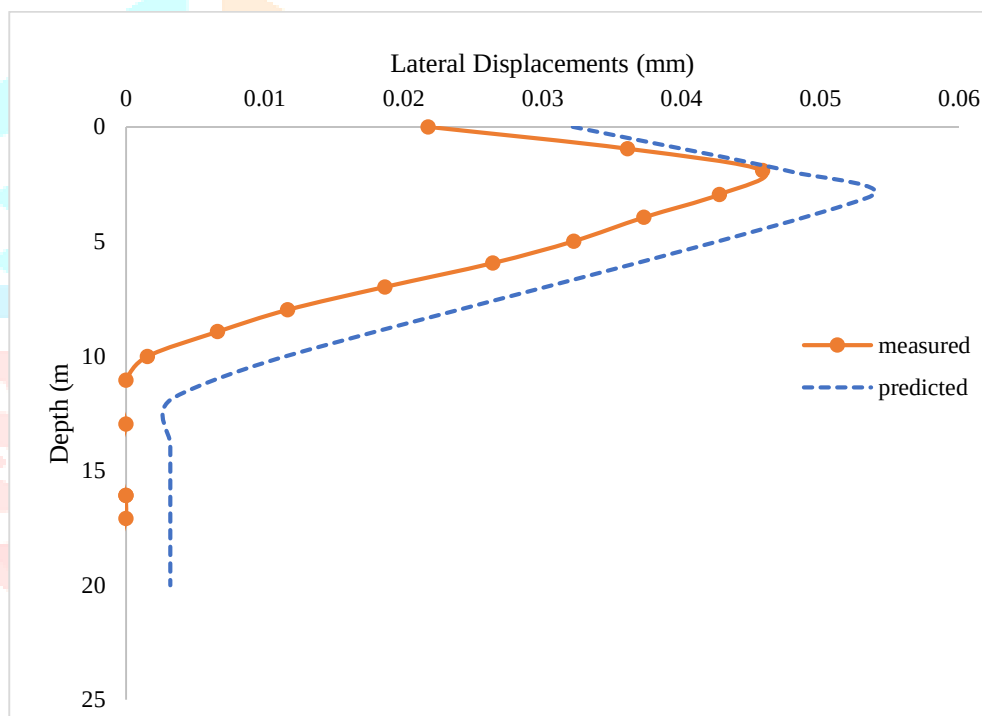


Fig. 11. Lateral displacements along the vertical plane at toe of the embankment

3.1.3 Factor of Safety:

The FEM analysis for the embankment at Rio de Janeiro and Ariake showed a factor of safety of 1.30 and 1.66 at a height of 3 m respectively, while the unreinforced embankment exhibits accelerating deformations, signalling the risk of failure

CONCLUSION

The performance of embankment case histories was evaluated and compared to the predicted values from finite element analysis (FEA).

1. Settlements were reduced by 25% and remained relatively constrained in Case 1, while in Case 2, the reduction was approximately 20% compared to the field measurements.
2. In Case 1, the maximum lateral displacement along the vertical plane at the toe showed a 10.5% increase compared to field measurements, while in Case 2, substantial displacement was observed at depths exceeding 1 meter, with an overall lateral displacement increase of approximately 20% relative to the measured values.
3. The comparison between previous predictions and the current study elucidates that the assumption of the undrained modulus of elasticity (E_u) plays a pivotal role in governing vertical and lateral displacements, thereby significantly influencing the precision of the analytical results.
4. The comparison of field-measured values with FEM analysis indicates discrepancies, leading to inaccurate results. Consequently, it is recommended to explore alternative methodologies in FEM analysis that can yield more precise outcomes, which should be addressed in future research endeavours.

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