Laboratory tests and numerical studies of a geosynthetic reinforced pile-supported embankment

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ABSTRACT

This paper presents laboratory tests investigating geosynthetic-reinforced pile-supported embankments constructed over soft soil. A series of tests were conducted in a 4m x 4m x 0.9m pit, incorporating 16 piles and varying the thickness of the load transfer platform and geosynthetic reinforcement. A comprehensive monitoring program was implemented to track the load distribution within the granular platform and measure settlement. This research is part of the collaborative national project ASIRI+ (2019-2025), which involves around forty organizations and aims to develop design guidelines for soil reinforcement using rigid inclusions. The experimental results contribute to understanding the complex mechanisms occurring within the load transfer platform, assessing the effectiveness of the geosynthetic reinforcement. Additionally, a numerical model developed using FLAC3D software was employed to simulate the experimental tests. The model's ability to predict the settlement of the granular soil surface and the stress transmitted to the subgrade layer was validated through comparisons with experimental data. Based on this calibrated model, multiple simulations were conducted to identify the optimal geosynthetic reinforcement solution.

Keywords: geosynthetics, reinforcement, piles, embankments, load transfer platform

1 INTRODUCTION

Over the past few decades, rapid urbanization and infrastructure development have increasingly required construction on weak, highly compressible soils. To address this challenge, pile-supported embankments (PSE) have emerged as a widely adopted soil reinforcement technique since the 1990s. This system combines rigid inclusion piles installed in soft soils with a Load Transfer Platform (LTP) that effectively distributes loads to the pile heads. The LTP typically consists of granular materials like sand or gravel.

To improve the efficiency of this composite foundation, one or several layers of geosynthetics can be inserted within the granular mattress. These horizontal reinforcements improve the load transfer to the piles through the membrane effect.

Additionally, for evaluating the limit state requirements or studying the behaviour of a geosynthetic reinforced pile-supported embankment (GRPSE), experimental investigations, numerical modelling techniques are commonly used in the literature. The experimental approach comprises full-field tests or scaled model tests. Le Hello and Villard (2009) presented a series of four full-scale instrumented experiments, and the membrane effect of geosynthetics was observed. The load, which is not transferred by arching effect, is transmitted to the geotextile which is deforming in membrane. The geosynthetic sheet's displacement depends on the load applied and the stiffness of the geosynthetics sheet used. The efficiency of the geosynthetics had also been highlighted in the frame of the first French project ASIRI (Briançon and Simon, 2012). Eekelen et al. (2012) presented a series of nineteen 3D model experiments on piled embankments, and have found that the calculated Geogrid (GGR) strains using current analytical models exceed the GGR strains measured in the field and proposed a new design method inserted in the Dutch Standard CUR226 (Eekelen and Brugman, 2016)

The numerical modelling provides a helpful and powerful tool to understand the complicated behaviour of GRPSE. In this approach, the finite element method, finite difference method, and discrete element method are commonly used. Being as important as the design methods, these analysis approaches are an essential part of any geotechnical design. The three aforementioned approaches can be used separately or in combination with one another. The choice of the most relevant approach(es) depends on the needs of the designers in different application scenarios.

While many experimental and numerical studies have investigated GRPSE systems (Briançon et al., 2025; Lee et al., 2021; Lian et al., 2014; Nunez et al., 2013; Sloan, 2011; van Eekelen et al., 2012; Van Eekelen & Han, 2020), current design standards and recommendations provide guidance for LTP design, questions remain about the choice between geotextiles and geogrids, the optimal geosynthetic configuration, including number, type, and positioning within the mattress. Even if the geosynthetic reinforcement seems to improve the load transfer to the piles, its role on the settlement reduction or its part in the load transfer remain not well understood.

As part of the French national project ASIRI+ (2019-2025), which is the extension of ASIRI project (2005-2012), researchers are testing various LTP configurations to better understand their mechanisms and efficiency. A particular challenge in laboratory testing of PSE is identifying an analog material that can reliably simulate soft soil compressibility while ensuring the reproducibility of the tests.

In the present research, a laboratory test at a scale ½ was conducted using an experimental bench containing 16 rigid inclusions. The test setup consisted of three layers: a 60 cm layer of rubber granulates (Deltagom) simulating soft soil, a 25 cm layer of gravel, and a 50 cm layer of sand. 3D numerical analyses are carried out using a finite difference method, incorporated in the Fast Lagrangian analysis of continua FLAC3D.

2 EXPERIMENTAL STUDY

2.1 Test Facilities

Laboratory tests were conducted in a 4m x 4m x 0.9m experimental pit to investigate load transfer mechanisms within pile-supported embankments. The pit contained 16 rigid inclusions, each with a 15cm diameter, arranged as illustrated in **Fig.1**.

A layer of rubber granulates (Deltagom) surrounding the rigid piles was selected to simulate the soft soil behaviour without requiring a consolidation time. Above this, a 25cm gravel layer was installed to simulate the LTP, followed by a 50cm sand layer at the top to simulate the embankment. The fill stages are shown in **Table 1**.

Table 1. Fill stages and model building steps

Fill Stages	Model building steps
1	Installation of Deltagom layer with rigid inclusions (RIs), followed by displacement field initialization
2	Placement of gravel layer (load transfer platform), followed by displacement field initialization
3	Completion of sand layer placement

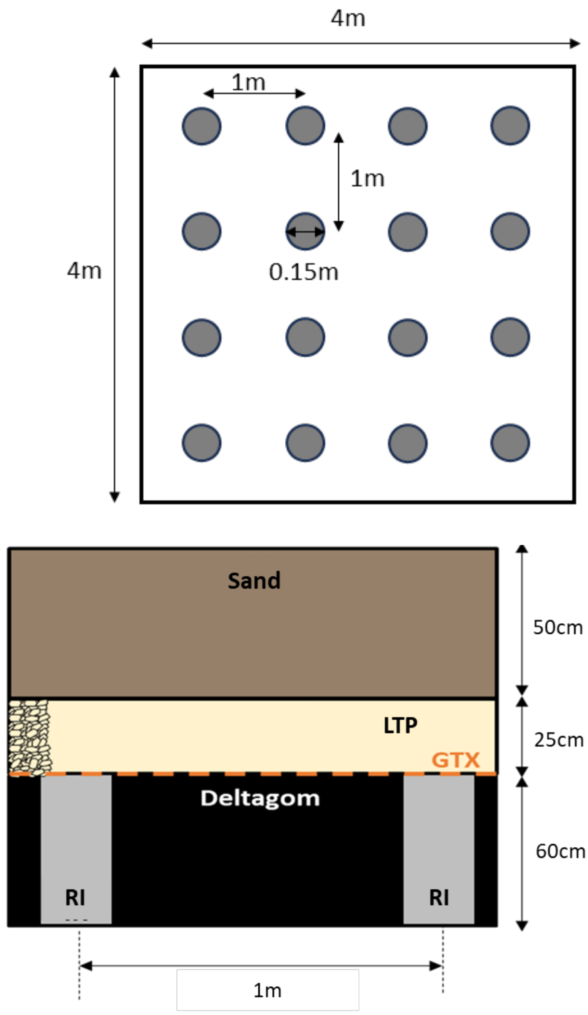


Fig. 1. Configuration of tests.

The first test, carried out without horizontal reinforcement, served as the reference test for the design of the geosynthetics. Note that each material layer is separated by a nonwoven geotextile (not designed for reinforcement) to prevent cross-contamination, as materials are reused between tests.

For the second test, two crossed monoaxial geotextiles were installed directly on the head of the rigid inclusions (**Fig.2**). The parameters of the geosynthetics used are presented in **Table 2**.

Table 2. The strength parameters of geotextile

Parameter	Standard	Specification
Stiffness $J_{(SP)}$ at ε= 2 % $J_{(ST)}$ at ε = 2 %	NF EN ISO 10319 (kN/m)	 3000 < $J_{(SP)}$ < 3500 $J_{(ST)}$ negligible

Fig. 2. Installation of the GTX.

2.2 Instrumentation

Earth pressure cells (EPC) were installed on the pile head on the central grid and on the soil at various elevations to measure the load transfer (**Fig. 3**). Settlement Sensors (SS) were installed in the soil, to follow the soft soil settlement, the deformation of the LTP, and locate the plane of equal settlement in the backfill. These sensors measure displacements through hydraulic pressure variations. Transmitters at the same level are connected in series to a reservoir mounted on a fixed support outside the test bench. The water reservoir maintains saturated sensor circuits at constant water pressure. The transmitter measures the pressure difference between its position and the reservoir.

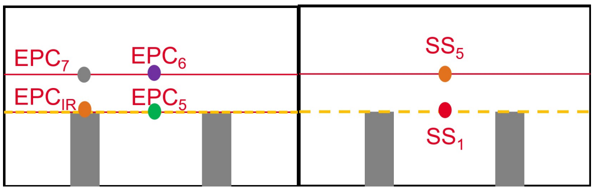


Fig. 3. Location of EPC and SS

Data was recorded using a Data Logger which can be programmed remotely. Each sensor connects to a pre-configured channel, with customizable measurement intervals. While data is stored in the system's internal memory, it can be exported to a computer. This logger was used to record static settlement values from hydraulic settlement sensors and static vertical pressures.

3 RESULTS

3.1 Load transfer

The test results reveal significant differences between reinforced and unreinforced conditions across multiple loading stages.

The incorporation of a geotextile shows no substantial influence on the characteristic patterns of the stress evolution. Nevertheless, a systematic comparison of the results remains necessary.

The first test, carried out without horizontal reinforcement, was considered the reference test for the design of the geosynthetics. After adding gravel on the Deltagom layer, stresses of 4 kPa (EPC_5) and 18 kPa (EPC_{IR}) were recorded, resulting in a 3 cm settlement (SS_5). These values are aligned with theoretical calculations based on the material weight and height.

Following the addition of 50 cm sand, stress measurements revealed varying pressures across measurement points, with EPC_5 at 4 kPa, EPC_6 at 7 kPa, EPC_7 at 25 kPa, and EPC_{IR} at 180 kPa, generating settlements of 3.8 cm (SS_1) and 3 cm (SS_5). The significant stress increase on the inclusions indicated soil arching formation and demonstrated the load transfer to the rigid inclusions.

In comparison to the unreinforced case, the geotextile-reinforced system demonstrated enhanced performance. The settlement at SS_1 significantly decreased to 1.1 cm, while stresses increased markedly across measurement points: EPC_5 rose to 7 kPa, EPC_7 to 30 kPa, and EPC_{IR} to 232 kPa. The membrane effect of the geotextile reinforcement improved the load transfer mechanism, directing more stress onto the rigid inclusions and resulting in more efficient soil arching development and better settlement control.

4 NUMERICAL ANALYSES

The finite difference software FLAC (Fast Lagrangian Analysis of Continua) 3D, developed by Itasca Consulting Group, Inc., was adopted for this numerical analysis. **Fig. 4** shows the overview of the numerical model for this case study.

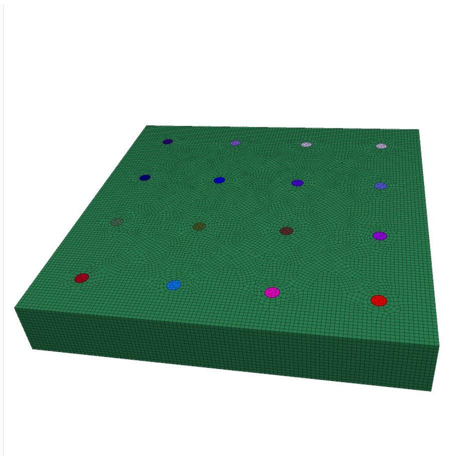


Fig. 4. Overview of the numerical model

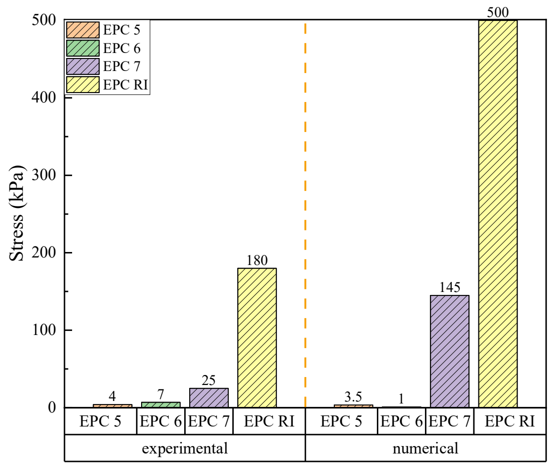
No deformations are assumed below the substratum. The bottom boundary is fixed in all three directions and the four vertical sides are blocked in their normal direction. The unit cell in this study is modeled using 20,305 zones for the soils. The initial parameters used in the numerical analysis are presented in **Table 3**. For the geotextile material, it demonstrated a stiffness of 3.5×10^6 N/m.

In the numerical modeling, different constitutive models were employed. The gravel was modeled as a linear elastic perfectly plastic material with the Mohr-Coulomb failure criterion, while the soft soil and rigid inclusions behavior was simulated using a linear elastic model.

Table 3. Input parameters for the numerical simulations

Material	Young modulus kPa	Poisson ratio	Weight kN	Cohesion kPa	Friction angle (°)
Deltagom	84	0.25	10	/	/
Gravel	50×10^3	0.3	17	5	35
Rigid inclusion	11.5×10^6	0.2	24	/	/
sand	35×10^3	0.3	19.68	6	38

The numerical model followed three calculation steps. Firstly, the soft soils and rigid inclusions are installed, then an equilibrium under self-weight is reached. In the next step, a LTP layer of 25cm thickness is placed on the pile top. One geosynthetic layer is installed in the middle of the LTP and the pile head top. Finally, the embankment is set up to 0.5 m. An illustration of the model is shown in Fig.1. Based on the initial strength parameter, the numerical results are illustrated in Fig. 5.



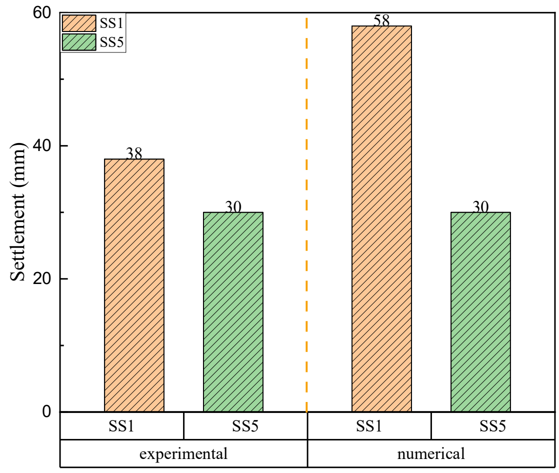


Fig. 5. Comparison of the vertical stress and vertical displacement between experiment and numerical simulation (initial simulation)

The comparison between numerical and experimental results revealed notable discrepancies, particularly in the rigid inclusion head stress, where the numerical simulation significantly overestimated the values observed in the experimental tests.

Despite multiple attempts to calibrate the strength parameters of various materials, the rigid inclusion head stress showed a minimal variation. This persistent discrepancy can be attributed to the significant stiffness contrast between the rigid inclusion and soft soil, suggesting the crucial role of the interface behavior. Subsequently, the incorporation of interface elements into the model yielded substantially improved agreement with experimental results, and other parameters used in numerical simulation are presented in Table 4.

Table 4. Adapted parameters for numerical simulation after calibration

Material	Young modulus kPa	Poisson ratio	Weight kN	Cohesion kPa	Friction angle (°)
Deltagom	Illustrated in **Fig.5**	Illustrated in **Fig.5**	10		
Gravel	50×10^3	0.3	17	5	37
Rigid inclusion	11.5×10^6	0.2	24		
sand	35×10^3	0.3	19.68	10	38
Interface	$K_n=2\times10^6$, $K_s=2\times10^6$ (kN/m)				

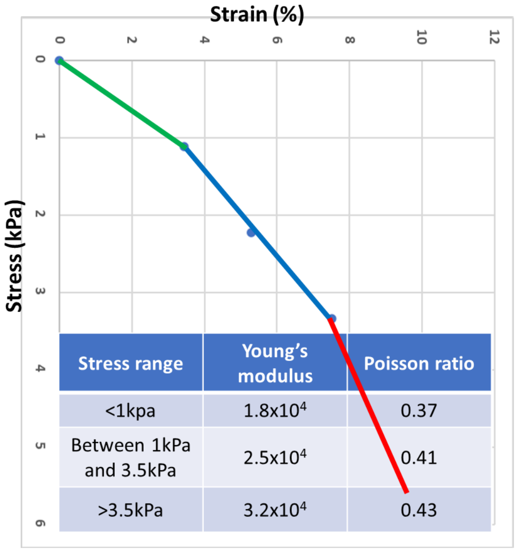
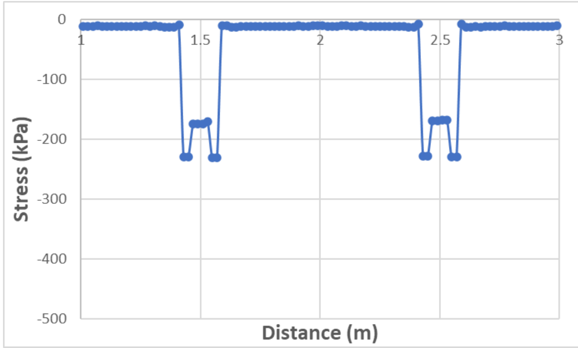


Fig. 6. Stress-strain behaviour and properties of Deltagom under different loading phases

Oedometer tests demonstrated that the stress-strain behavior of Deltagom exhibited three distinct linear segments, demarcated by stress thresholds of 1 kPa and 3.5 kPa. The compression modulus was 3.2×10^4 kPa below 1 kPa, increased to 6.4×10^4 kPa between 1-3.5 kPa, and reached 8.3×10^4 kPa beyond 3.5 kPa. To accurately characterize the stress-dependent hardening behavior of Deltagom material, a stress-dependent Young's modulus and Poisson's ratio were implemented in the numerical analysis. A unit cell model was developed to calibrate these parameters: for stresses below 1 kPa, Young's modulus of 1.8×10^4 kPa and Poisson's ratio of 0.37; for stresses between 1-3.5 kPa, Young's modulus of 2.5×10^4 kPa and Poisson's ratio of 0.37; and for stresses exceeding 3.5 kPa, Young's modulus of 3.2×10^4 kPa and Poisson's ratio of 0.43.

Similarly, the numerical simulation was conducted in three distinct stages (Stage 1, Stage 2, and Stage 3), representing the sequential construction process of the embankment. Each stage corresponds to the progressive addition of fill layers, thereby simulating the actual construction sequence. The vertical stress profile shown in **Fig. 7** demonstrates a general pattern with negative values indicating compression. The EPC_{IR} value is specifically measured at the 1.5 m position, corresponding to the first stress concentration zone. The numerical results are presented in **Fig. 8**.

(a)



(b)

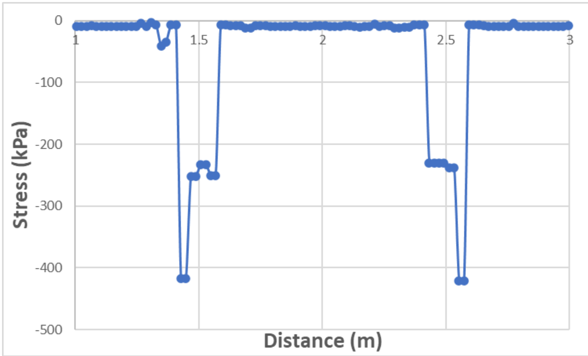
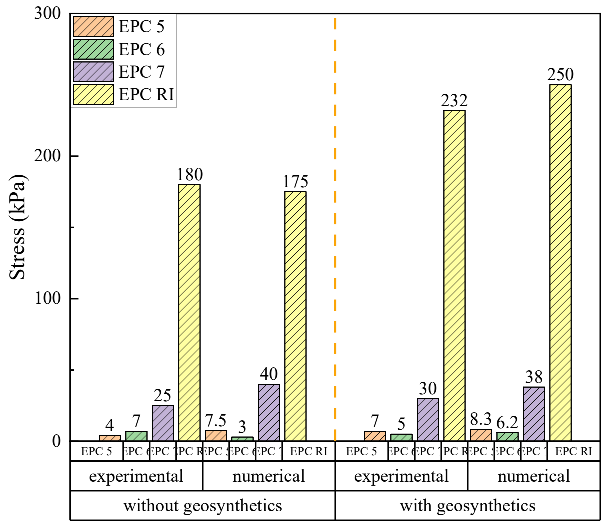


Fig. 7. Numerical analysis of the multi-stage embankment construction (a) profile of vertical stress without reinforcement (b) profile of vertical stress with reinforcement

Comparison between measured data and numerical analyses showed a strong agreement (Fig. 9). Furthermore, the difference of rigid inclusion head stress is 5.5%, which is acceptable and a reasonably good agreement can be concluded between the numerical model and experimental data. Therefore, the comparison results proved that this numerical model is reasonable and reliable for the analysis of GRPS embankments. After inserting the geotextile, the load transfer efficiency is 43%.

(a)



(b)

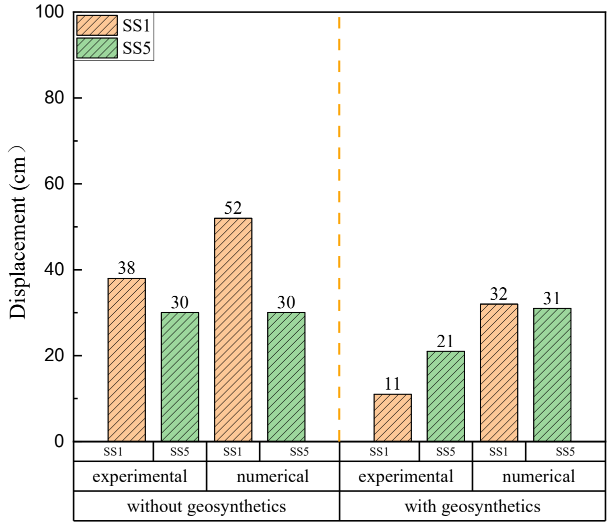


Fig. 8. Comparison of the vertical stress and vertical displacement between experiment and numerical simulation

5 CONCLUSION

Laboratory tests were conducted on a pile-supported embankment to investigate the settlement and load transfer occurring during an embankment installation. The experimental results demonstrated a significant enhancement in load transfer mechanisms with the incorporation of a geotextile reinforcement. When compared to the unreinforced system, the reinforced configuration exhibited a notable increase in stress concentration on the rigid inclusions (up to 232 kPa), while surface settlement was reduced by over 70% (from 3.8 cm to 1.1 cm). The efficiency is improved by the presence of the geosynthetic.

These experimental results were effectively validated through numerical modeling, which accurately predicted both surface settlements and stress distributions. However, the model requires a calibration, incorporating interface elements and the non-linear mechanical behavior of Deltagom, to successfully reproduce the experimental observations. This validated numerical model serves as a valuable tool for optimization studies, supporting the development of design guidelines for geosynthetic reinforcement in similar ground improvement scenarios. This integrated experimental-numerical approach provides a robust framework for understanding and designing geotextile-reinforced soil systems supported by rigid inclusions.

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