

Article

Trapdoor Model Tests on Multi-Layer Geosynthetic Reinforcement for Subsidence Mitigation

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Abstract

Localized subsidence remediation in narrow working areas is often constrained by limited reinforcement width and anchorage length. This study investigates the load-transfer and deformation behaviour of multi-layer geosynthetic reinforcement under such conditions through 1:5 scaled trapdoor tests and PFC2D simulations. A three-layer geotextile system was tested by varying anchorage length and interlayer spacing, and a soil arching load-transfer ratio, η , was introduced to quantify the contribution of soil arching. The results show that increasing anchorage length from $0.5B$ to B increased η from 74.82% to 86.51%, whereas a further increase to $1.5B$ raised η only slightly to 89.02%. Over the same range, surface settlement and the midspan deflection of the lowest reinforcement decreased by 26.84% and 16.46% from $0.5B$ to B , but only by a further 1.55% and 2.53% from B to $1.5B$. At $L = B$, η reached 61.89%, 86.51%, and 78.75% for $s = 0.3B$, $0.2B$, and $0.1B$, respectively, indicating that the intermediate spacing was most favourable for soil arching load-transfer. The DEM results further showed diminishing returns with increasing layer number, with the three-layer configuration achieving about 95% of the five-layer performance. Overall, high and stable load-transfer performance was obtained at $L/B = 1.0$ – 1.25 and $s/B = 0.15$ – 0.2 .

Keywords: trapdoor test; multi-layer geosynthetic reinforcement; subsidence remediation; anchorage length; layer spacing; soil arching; load-transfer ratio; discrete element method



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1. Introduction

Karst regions are commonly characterized by developed underground cavities and complex stratigraphic conditions. Under construction disturbance, sudden ground collapse can be readily triggered, posing a serious threat to the serviceability of roads, railways, and urban infrastructure. For such engineering problems, effective control of collapse-induced surface settlement and the associated load redistribution remains a key challenge in geotechnical engineering, especially in narrow corridors or under spatially constrained construction conditions [1–5].

Geosynthetic reinforced cushions have been widely used for weak ground improvement and collapse remediation because of their construction convenience, adaptability, and cost effectiveness [6–10]. By placing reinforcement within the fill, a load-transfer mechanism jointly governed by soil arching and membrane action can develop, enabling the

overburden load above the subsided area to be transferred to the adjacent stable areas, thereby reducing stress concentration and settlement at critical locations. Previous studies have mainly focused on single-layer reinforced systems and have systematically clarified, through model tests, numerical simulations, and theoretical analyses, the effects of reinforcement stiffness, anchorage configuration, and span or void width on soil arching development and deformation control [11–14]. However, in karst-remediation scenarios with restricted construction width, a single reinforcement layer often cannot provide sufficient effective anchorage length, making a multi-layer arrangement a more practical option. Compared with single-layer systems, multi-layer reinforcement can provide more load-transfer paths and distributed restraint within a limited space. Its behaviour, however, is not a simple superposition of single-layer effects, and the interlayer load-transfer mechanism and applicable parameter ranges still require further clarification [15,16].

In recent years, the discrete element method has advanced from describing the macroscopic settlement and stress response of reinforced embankments to identifying the evolution of soil arching, slip at the soil–reinforcement interface, and the reorganization of mesoscopic force chains. Wang et al. [17] showed through DEM analysis that reinforcement can suppress the upward propagation of vertical slip surfaces within the embankment and delay soil arching degradation, thereby improving load-transfer efficiency and reducing differential deformation. Delli Carpini et al. [18] further found, for reinforced embankments under local voiding conditions, that an approximately rigid trapezoidal soil body may form above the subsided area, while the anchorage zone exhibits a pronounced frictional sliding mechanism, indicating that the working state of the anchorage section directly affects the load-transfer path. For multi-layer systems, the double-layer DEM study by Sun et al. [19] showed that a multi-layer arrangement can make downward load-transfer more uniform, reduce peak tensile force near the reinforcement edges, and alter the relative contributions of soil arching and membrane action. Consistent with this, multi-layer model tests by Zhang et al. [20] indicated that the overall behaviour of multi-layer systems differs from the typical membrane-dominated response of single-layer reinforcement and is better described as a beam-like response arising from the combined action of reinforcement and fill. These studies provide an important basis for understanding multi-layer geosynthetic reinforcement. Nevertheless, most existing work has focused on pile-supported embankments or conventional support conditions, and research on multi-layer geosynthetic reinforcement under localized subsidence, restricted construction width, and limited anchorage remains insufficient. In particular, targeted combined experimental and DEM studies are still lacking on how anchorage length and interlayer spacing jointly influence the stress state of the lowest reinforcement layer, the proportion of load carried by soil arching, and the evolution of mesoscopic force chains.

In addition, previous studies have shown that the macroscopic mechanical response of granular geomaterials and related composite systems is often highly sensitive to particle-scale characteristics, loading path, and soil–structure interaction conditions, as reflected in studies on engineering muck-based geopolymers, cyclic response of calcareous sand, and lateral pile–soil interaction in coral sand [21–23]. Although these studies do not address the same problem considered here, they collectively indicate that material characteristics, boundary conditions, and structural interaction should all be taken into account when analyzing the load-transfer and deformation mechanisms of multi-layer geosynthetic reinforcement under spatially constrained conditions.

When construction width is restricted and a single reinforcement layer cannot provide sufficient anchorage length, a multi-layer arrangement becomes an evident engineering necessity. However, its interlayer interaction mechanism and applicable parameter ranges remain unclear. Current design methods still involve considerable uncertainty in

the selection of reasonable anchorage length, the optimal range of interlayer spacing, the marginal benefit of increasing layer number, and the load-sharing proportion between soil arching and membrane action, and a unified and explicit design basis is still lacking. To address these issues, this study combines 1:5 scaled trapdoor model tests and discrete element simulations based on a karst-collapse remediation project in Guilin and introduces the soil arching load-transfer ratio, η , to systematically investigate the mechanical and deformation responses of a three-layer reinforced system under different anchorage lengths and interlayer spacings. The aim is to clarify the load-redistribution behaviour and mesoscopic load-transfer mechanisms of multi-layer geosynthetic reinforcement under spatially constrained conditions, and thereby provide experimental and numerical evidence for parameter selection and mechanism interpretation.

2. Materials and Methods

2.1. Physical Model Tests

2.1.1. Similitude and Test Apparatus

In order to identify the key factors controlling karst-collapse mitigation using multi-layer geotextiles, scaled model tests were performed. The tests involved the simulation of a highway collapse in Gongcheng County, Guilin. The representative prototype dimensions were a collapse span of 1.5 m and a backfill height of 3.0 m. In accordance with the principles of similarity, the geometric scale factor was designated as $C_l = 5$, corresponding to a model-to-prototype ratio of 1:5 [24]. Here, the similarity ratio is defined as $C_x = x_p/x_m$. The tests were conducted under a 1 g field ($C_g = 1$), and the same sand was used so that $C_\rho = 1$; therefore, from $\sigma = \rho gh$, the stress similarity is $C_\sigma = C_\rho \cdot C_g \cdot C_l = 1 \cdot 1 \cdot 5 = 5$. Since cohesion c has the same unit as stress in the Mohr–Coulomb criterion, C_c is taken as C_σ . The choice of $C_l = 5$ was mainly constrained by the model box size while keeping the key geometry and sufficiently wide stable areas for instrumentation and boundary treatment. The scaling relationships for gravity, acceleration, cohesion, and other physical quantities are outlined in Table 1. The similarity constants in Table 1 were derived following the Buckingham π theorem, in which the governing variables of a 1 g plane-strain trapdoor problem are reduced to dimensionless groups to ensure consistent scaling between the model and the prototype.

Table 1. Model test similarity constants.

Physical Quantity	Dimension	Similarity Constant
Accelerations, a	LT^{-2}	$C_g = 1$
Densities, ρ	ML^{-3}	$C_\rho = 1$
Cohesion, c	$ML^{-1}T^{-2}$	$C_c = 5$
Poisson ratio, ν	1	$C_\nu = 1$
Internal friction angle, ϕ	1	$C_\phi = 1$
Force, F	MLT^{-2}	$C_F = 125$
Stresses, σ	$ML^{-1}T^{-2}$	$C_\sigma = 5$
Modulus of elasticity, E	$ML^{-1}T^{-2}$	$C_E = 5$
Area, A	L^2	$C_A = 25$
Length, l	L	$C_l = 5$
Strain, ε	1	$C_\varepsilon = 1$

The 1:5 scale adopted in this study is not intended to provide a one-to-one prediction of absolute prototype responses under 1 g conditions. Instead, it is used to reproduce the dominant load-carrying behaviour and deformation mechanisms of a multi-layer geosynthetic reinforcement during localized subsidence, while keeping the key governing parameters consistent. For the aspects targeted in this work, the controlling variables were either

preserved in the model or regulated in dimensionless form. Consequently, although the scaled model cannot be strictly equivalent to the field prototype, it can still capture the essential trends of soil arching development, load-transfer from the subsided area to the adjacent stable areas, and force redistribution in the lowest reinforcement layer under constrained construction width.

The choice of the geometric scale factor $C_l = 5$ was jointly dictated by the model box dimensions, boundary treatment, and instrumentation requirements. At this scale, the subsided area, the two stable areas, and the anchorage ranges can all be fully accommodated within the model, ensuring that the primary load-transfer paths and deformation patterns can develop adequately.

The model box comprised a channel-steel frame that was welded to form a rigid structure (Figure 1). In the longitudinal direction, the two sidewalls were composed of an 8 mm thick steel plate and a 25 mm thick tempered-glass panel, respectively. The internal dimensions of the box were 1.5 m $\times$ 0.6 m $\times$ 1.5 m (length $\times$ width $\times$ height). The void width was designated as $B = 0.3$ m, and the width of each adjacent stable area was set at 0.6 m. A movable base plate was driven by an electric screw actuator to provide slow, well-controlled vertical displacement. The trapdoor plate was affixed to the lifting mechanism by bolts.

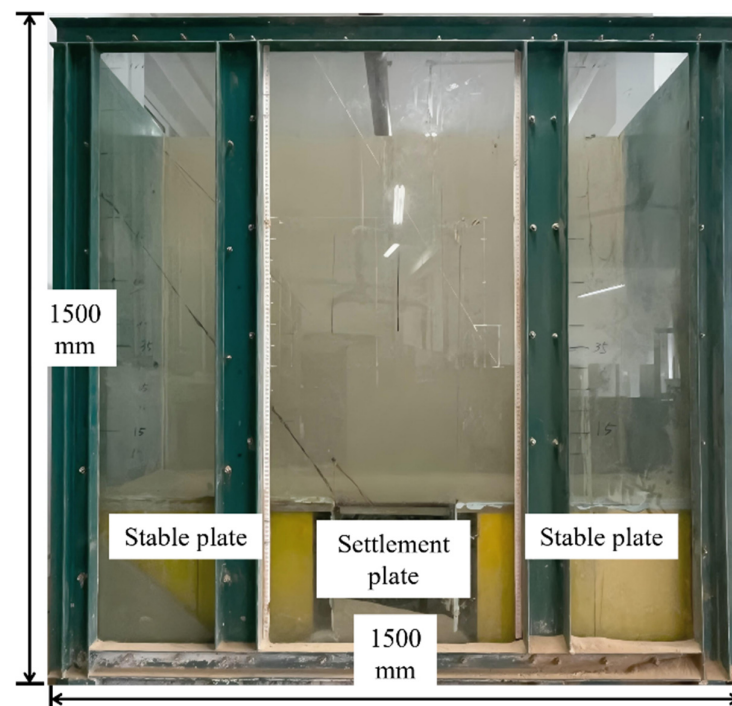


Figure 1. Photograph of the model test box.

The vertical stress was monitored using commercially available strain-gauge earth pressure cells; the central displacement of the reinforcement was monitored using MEMS (Micro-Electro-Mechanical System) tilt sensors (Shenzhen Weite Intelligent Technology Co., Ltd., Shenzhen, China); and the surface displacement of the fill was monitored using displacement transducers (Beijing Jinghaiquan Sensor Technology Co., Ltd., Beijing, China). The MEMS sensors can be embedded within the soil mass to record acceleration signals, which are processed using dynamic integration and Kalman filtering [25] to convert acceleration into displacement, thereby obtaining internal soil displacement. The earth pressure cells, displacement transducers, and MEMS sensors were connected to the same strain-measurement system and linked to a computer for real-time acquisition. The data were recorded using dedicated acquisition and analysis software at a sampling frequency

of 1 Hz. The instrumentation layout is demonstrated in Figure 2. All instruments were checked against the manufacturer's calibration parameters prior to testing, and their nominal accuracies are summarized in Table 2. For the displacements obtained from the MEMS sensors, the uncertainty arises not only from the sensor accuracy itself but also from the dynamic integration and filtering procedures used to convert acceleration to displacement. Therefore, this study focuses on relative comparisons among test cases rather than on minor differences in the absolute displacement at an individual measurement point.

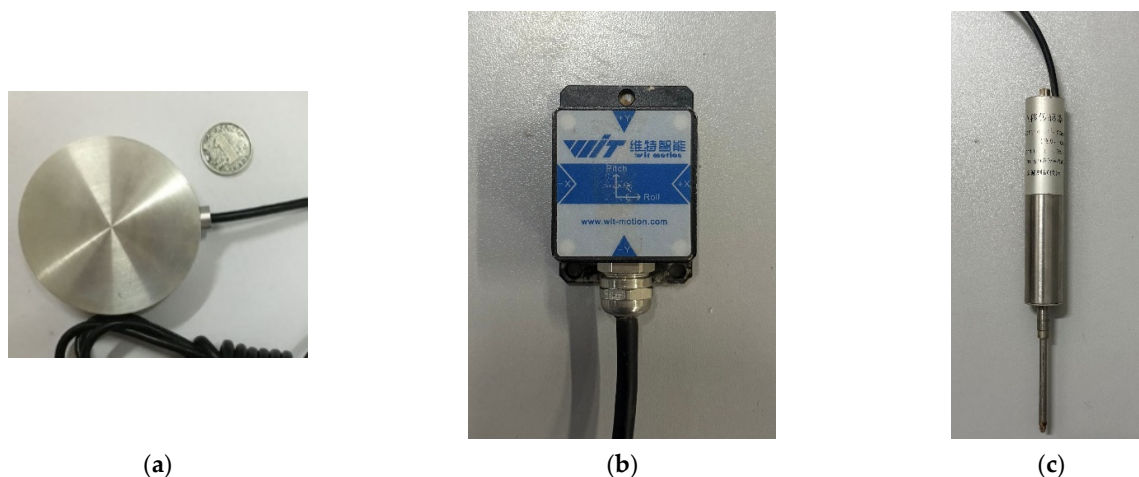


Figure 2. Instrumentation for the model tests: (a) Earth pressure cell, (b) MEMS tilt sensor, and (c) displacement transducer. The Chinese text visible on the MEMS tilt sensor is the manufacturer's logo.

Table 2. Instrumentation and nominal specifications for the model tests.

Device	Range	Specification/Accuracy
Strain-gauge earth pressure cells	0–200 kPa	$\Phi 16 \times 5$ mm; $\leq 0.5\%$ F.S.; 120% F.S.; 350 Ω ; full bridge
Displacement transducers	0–50 mm	Sensitivity: 0.001 mm
MEMS tilt sensors	± 16 g	Acceleration error: 0.01 g

2.1.2. Test Materials and Specimen Preparation

The material utilized for the fill was Lijiang sand, which was collected from Guilin. Following a natural drying process, the sand was subjected to oven drying. Subsequently, the sand was subjected to a sieving process, employing a 2 mm sieve to refine the material. The sieve analysis yielded the following results: the effective particle size was $d_{10} = 0.08$ mm, the median particle size was $d_{30} = 0.13$ mm, and $d_{60} = 0.22$ mm; the coefficient of uniformity was $C_u = 2.75$; and the coefficient of curvature was $C_c = 0.96$. The particle-size distribution curve is illustrated in Figure 3. The key properties of the test sand are outlined in Table 3.

The cohesion in Table 3 represents a small apparent cohesion obtained as the intercept of the Mohr–Coulomb fit from direct shear tests under low normal stresses and slight moisture, rather than true cementation; compared with the reference overburden stress $\sigma_0 = \gamma H$, its influence is secondary, and the behaviour is governed mainly by ϕ . With $C_c = C_\sigma = 5$, this corresponds to an equivalent prototype intercept of about 5 kPa, which is reasonable for compacted granular fills with a small fraction of fines in karst road embankments.

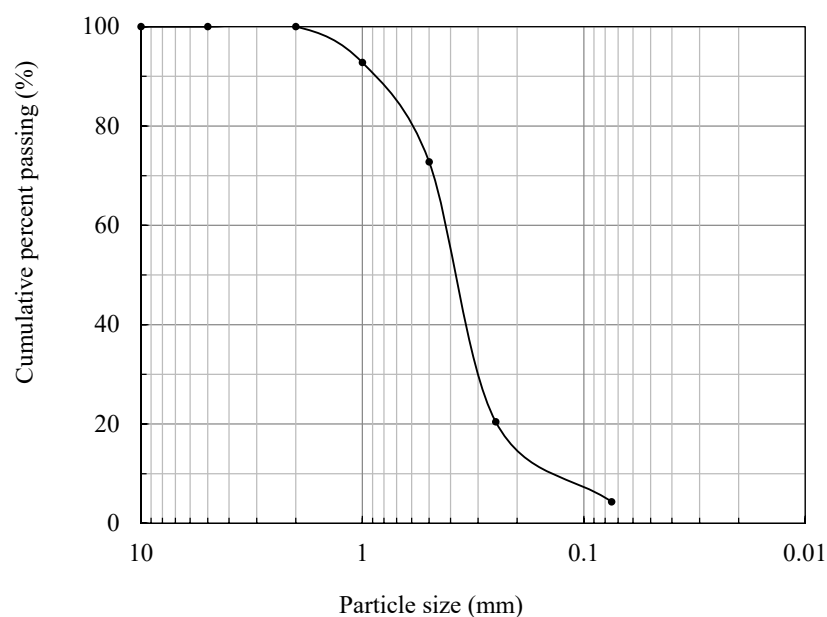


Figure 3. Particle-size distribution (PSD) curve of the test sand.

Table 3. Performance indexes of test sand.

Physical Index	Parameters
Densities (g/cm ³)	1.65
Cohesion (kPa)	1.00
Angle of internal friction (°)	33.00
Moisture content (%)	1.04

The test sand was selected to represent the granular backfill used in the field remediation in terms of soil type and shear strength, and the measured friction angle ($\varphi = 33^\circ$) and unit weight/density level are comparable to those of compacted embankment fills; therefore, the similarity of density and frictional strength was taken as $C_\rho = 1$ and $C_\varphi = 1$. In this context, the small cohesion reported in Table 3 is treated as an apparent intercept from direct-shear fitting under low normal stresses, rather than true cementation, and is used only for consistency with the stress scaling.

The selection of a woven filament geotextile (Shanxi Yige Engineering Materials Co., Ltd., Shanxi, China) as the reinforcement material is illustrated in Figure 4. The total lengths of geotextile utilized in the experimental trials were 600, 900, and 1200 mm, which corresponded to anchorage lengths of 150, 300, and 450 mm on each side, respectively. The width of the geotextile material was 600 mm. Further technical indices are provided in Table 4.

It should be noted that reinforcement similarity is governed not only by tensile strength but also by tensile stiffness. Owing to limitations in material availability and testing conditions, the model geotextile satisfied the requirements of this study in terms of geometric dimensions, installation method, and relative tensile grade, but its tensile stiffness could not be matched in a strictly rigorous sense. Accordingly, the present tests were intended primarily to identify the load-transfer mechanisms and parameter sensitivity of the multi-layer system under different anchorage lengths and layer spacings, rather than to provide a strict quantitative prediction of prototype responses. If the relative stiffness of the model reinforcement was higher than the target similitude requirement, settlement and reinforcement deflection may have been underestimated, and the contribution of soil arching may have been correspondingly overestimated; the opposite tendency may occur if the relative stiffness was lower. Nevertheless, because the same reinforcement

was used in all test cases, the comparative assessment of parameter effects retains good internal consistency.



Figure 4. Geotextile.

Table 4. Geotextile performance indexes.

	Mass per Unit Area (g/m^2)	Tensile Strength (kN/m)		Ultimate Elongation (%)		Thickness (mm)
		Warp Direction ($\geq$)	Weft Direction ($\geq$)	Warp Direction	Weft Direction	
Geotextile	200	50	35–50	35	30	2

2.1.3. Test Programme and Parameter Settings

The void width was set to $B = 300$ mm, and the fill height to $H = 600$ mm. Three geotextile layers were installed for comparison, as shown in Figure 5. Earth pressure cells were placed in the stable area within the load-transfer influence extent, taken as $0.75H$ [26]. In the present physical model, each stable area was 600 mm wide, whereas the adopted influence extent was 450 mm, leaving a residual lateral buffer of 150 mm between the outer limit of the influence extent and the sidewall. The present study investigates the effects of anchorage length of $0.5B$, B , and $1.5B$, corresponding to 150, 300, and 450 mm, and layer spacing of $B/12$, $B/6$, and $B/3$, corresponding to 30, 60, and 90 mm, on load-transfer, load sharing, and surface settlement. The test matrix is listed in Table 5.

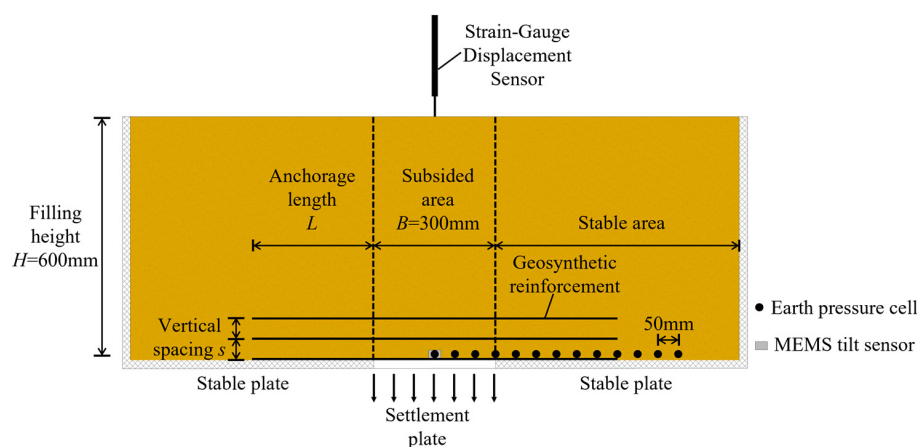


Figure 5. Layout of the model test box.

Table 5. Summary of test cases.

Case	Number of Layers	Anchorage Length (mm)	Layer Spacing (mm)
Z1	3	150	60
Z2	3	300	60
Z3	3	450	60
Z4	3	300	30
Z5	3	300	90

2.1.4. Instrumentation Layout and Test Procedure

Except for the tempered-glass observation side, the other inner walls of the model box were lined with two layers of 0.3 mm commercially available polytetrafluoroethylene film, and a thin layer of commercially available silicone grease was uniformly applied on the contact surfaces of the films to further reduce sidewall friction and boundary restraint. The combined use of PTFE film and silicone grease was intended to minimize interference from the model-box boundaries with lateral soil deformation and load redistribution [27]. The backfill was prepared by layered placement. The soil was placed in six lifts, each 100 mm thick, to a total fill height of 600 mm. Before placing each lift, the required soil mass was calculated from the target bulk density, after which the material was weighed and placed in a controlled amount. Each layer was then levelled and compacted using an electric plate compactor. To ensure consistent initial conditions among layers, all lifts were compacted to the same target bulk density of 1.65 g/cm^3 . Based on the internal dimensions of the model box, the volume of each lift was $1.5 \text{ m} \times 0.6 \text{ m} \times 0.1 \text{ m} = 0.09 \text{ m}^3$, corresponding to a target soil mass of approximately 148.5 kg. When the volume occupied by embedded sensors and local obstructions was taken into account, the target mass of each lift was corrected using the actual filling volume. After compaction of each lift, the layer thickness was checked with a steel ruler, and the bulk density was back-calculated from the measured soil mass and the actual volume to verify compaction uniformity. The deviation between the measured and target bulk densities of each layer was controlled within $\pm 0.25\%$, and the interlayer variation ranged from 1.648 to 1.654 g/cm^3 , indicating good repeatability and consistency in specimen preparation. The measured thickness, soil mass, and back-calculated bulk density of each layer are summarized in Table 6.

Table 6. Measured thickness, soil mass, and back-calculated bulk density of each layer.

Layer No.	Measured Thickness/mm	Measured Soil Mass/kg	Back-Calculated Bulk Density/ g/cm^3
1	101	150.32	1.654
2	100	148.56	1.651
3	99	147.32	1.653
4	101	149.79	1.648
5	100	148.43	1.649
6	100	148.52	1.650

After filling was completed, the model was left undisturbed for 60 min to reduce transient disturbances caused by placement and compaction. The displacement gauges were then installed, and the data acquisition system was started. During testing, the trapdoor plate was driven downward at a constant rate of 2 mm/min and stopped when the cumulative settlement reached 30 mm. The earth pressure cells, displacement gauges, and MEMS sensors were connected to the same acquisition system, which synchronously recorded vertical stress, surface displacement, and internal displacement responses at a frequency of 1 Hz. Each test condition was independently repeated twice. The stress, displacement, and load-transfer ratio reported in this paper are the average values of the two tests.

The stress and displacement profiles were obtained from discrete sensor readings along the model and plotted as fitted curves for clarity. Error bars are not shown because the profiles represent pointwise measurements and curve fitting, while the comparisons in this study focus on the overall trend and peak location rather than small point-to-point fluctuations.

2.2. DEM Numerical Model

2.2.1. Model Setup, Boundary Conditions and Loading Scheme

To ensure consistency with the scaled laboratory trapdoor tests, a two-dimensional discrete-element model was developed in PFC2D 5.0, as shown in Figure 6. The geometric scale relationship among the physical model, the DEM model, and the prototype was 1:5:5. The soil mass was represented by an assembly of circular particles; the test box boundaries and the trapdoor were defined using rigid facet walls; and the multi-layer reinforcement was discretized as bonded particle chains using the Linear-bond contact model.

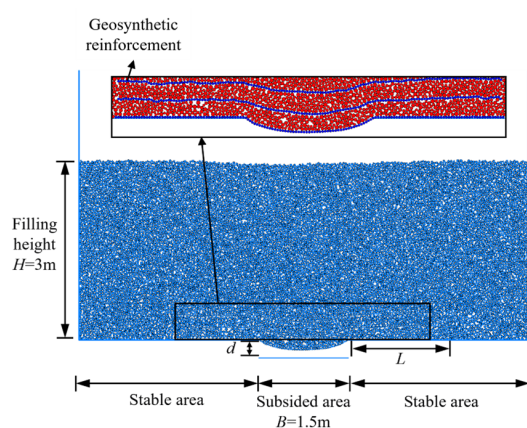


Figure 6. DEM model.

The DEM soil was represented by circular particles generated at an initial porosity of 0.18, and a coarse-graining strategy with a particle radius range of 0.03–0.05 m was adopted to improve computational efficiency. Soil contacts followed the linear model, with different contact parameters assigned during the deposition and analysis stages. The geotextile layers were modelled as bonded particle chains using the Linear-bond model. The adopted DEM micro-parameters are summarized in Table 7, and further details of the parameter calibration and coarse-graining strategy are provided in Section 2.2.2.

Table 7. Summary of DEM parameters used in PFC2D.

Category	Parameter	Symbol/Model	Value
Particle generation	Initial porosity	n_0	0.18
	Particle radius	r	0.03–0.05 m
Soil deposition stage	Contact model	Linear	-
	Normal stiffness	k_n	1.0×10^{12} N/m
	Shear stiffness	k_s	1.0×10^{12} N/m
	Friction coefficient	μ	0.10
	Local damping coefficient	α	0.10
Soil analysis stage	Contact model	Linear	-
	Normal stiffness	k_n	1.0×10^7 N/m
	Shear stiffness	k_s	1.0×10^7 N/m
	Inter-particle friction coefficient	μ_{pp}	0.65

Table 7. Cont.

Category	Parameter	Symbol/Model	Value
Boundary parameter	Particle–wall friction coefficient	μ_{pw}	0.45
Loading control parameter	Local damping coefficient	α	0.70
Reinforcement parameter	Contact model	Linear-cbond	-
	Normal stiffness	k_n	5.0×10^7 N/m
	Shear stiffness	k_s	5.0×10^7 N/m
	Friction coefficient	μ	0.68
	Bond tensile strength	$F_{cb,ten}$	5.0×10^7 N
	Bond shear strength	$F_{cb,shear}$	5.0×10^8 N

(1) Geometry and boundary conditions

The model width was set to $W = 7.5$ m and the soil height to $H = 3.0$ m. An additional space was reserved above the soil to allow particle generation and gravitational deposition. Rigid facet walls were used to form the boundaries. The bottom boundary consisted of three wall segments, with the central segment acting as the movable trapdoor of void width $B = 1.5$ m, while the two side base plates were fixed to represent the support conditions of the stable areas. The lateral boundaries were located at $x = 0$ and $x = 7.5$ m. No rigid loading plate was applied at the top; a free-surface condition was maintained so that surface settlement could develop naturally, consistent with the experimental observation approach.

(2) Discretization and placement of reinforcement layers

The three reinforcement layers were represented by three horizontal particle chains. Their installation ranges corresponded to the effective anchorage ranges adopted in the physical tests. The embedment elevations of each layer were determined according to the test layout and the similarity relationship, ensuring that the relative interlayer positions matched those in the experiments.

(3) Initial equilibrium and quasi-static control

After particle generation at the target porosity, the assembly was brought to static equilibrium under gravity. Local damping and an unbalanced-force ratio convergence criterion were employed to ensure the system reached a stable state prior to loading, thereby allowing the subsequent subsidence process to evolve in a quasi-static manner.

(4) Loading scheme: Displacement-controlled trapdoor movement

Subsidence was induced by driving the trapdoor downward under displacement control at a constant velocity. The state corresponding to a trapdoor displacement of $d = 150$ mm was recorded for model validation and subsequent analysis.

(5) Monitoring variables and outputs

Vertical stress was obtained as σ_{yy} using measurement circles, with monitoring points arranged along the stable area to capture subsidence-induced stress transfer. Surface settlement was calculated from the vertical displacements of particles near the ground surface. Reinforcement response was characterized by tracking the evolution of internal force and stress at representative locations along the particle chains.

2.2.2. DEM Parameter Calibration and Coarse-Graining Strategy

Because the PFC2D model domain is relatively large, modelling the sand using its true particle size would result in an excessive number of particles and prohibitively high computational cost. A coarse-graining strategy was therefore adopted in the DEM model,

with particle radii ranging from 0.03 to 0.05 m. This particle size range does not correspond to the actual grain size of the sand used in the physical tests; instead, it was selected to reproduce the macroscopic mechanical response of the system while maintaining acceptable computational efficiency.

Accordingly, the DEM parameters were not determined on the basis of a one-to-one correspondence between numerical particles and real sand grains, but through calibration against macroscopic response. Specifically, the baseline case with $L = B$ and $s = 0.2B$ was used for calibration, and the micro-parameters were adjusted so that the numerical model could reasonably reproduce the final ground-surface settlement, the midspan deflection of the lowest reinforcement layer, and the vertical stress distribution above the lowest reinforcement layer observed in the physical tests, including both the peak location and the peak magnitude. The calibration results show overall agreement between the DEM predictions and the test data, indicating that the adopted coarse-graining strategy is adequate for the purposes of identifying load-transfer mechanisms and conducting parametric sensitivity analyses in this study.

It should be noted that the particles in the DEM model should be regarded as numerical units representing the overall mechanical behaviour of the soil mass, rather than as real sand grains in a physical sense. The present DEM model is therefore more suitable for analyzing stress redistribution, force-chain evolution, and parameter-dependent behaviour in multi-layer geosynthetic reinforcement than for directly reproducing the true particle-scale fabric of real sand.

2.2.3. Case Matrix for Parametric Study

To extend the range of investigated conditions beyond the limited laboratory programme, a total of 25 additional DEM simulations were conducted in PFC2D by systematically varying the anchorage length (L) and the interlayer spacing (s) of the three-layer reinforcement system. For generalization and comparison, both parameters were normalized by the void width B , i.e., the anchorage-length ratio L/B and the layer-spacing ratio s/B . Each parameter combination was assigned a unique case ID (N1–N25); the L/B – s/B case matrix is given in Table 8. The baseline condition corresponding to $L/B = 1.0$ and $s/B = 0.20$ is denoted as N13.

Table 8. Matrix of DEM cases in PFC2D.

$s/B \backslash L/B$	0.1	0.15	0.2	0.25	0.3
0.5	N1	N2	N3	N4	N5
0.75	N6	N7	N8	N9	N10
1	N11	N12	N13	N14	N15
1.25	N16	N17	N18	N19	N20
1.5	N21	N22	N23	N24	N25

2.3. Soil Arching Load-Transfer Ratio: Definition and Calculation

The existing studies on load transfer in reinforced embankments have mainly focused on the vertical stress distribution acting on the reinforcement. While this approach captures the general pattern of load redistribution and the location of stress peaks, it does not directly quantify how much of the initial overburden load above the subsided area is transferred to the adjacent stable areas by soil arching. This issue is particularly important in multi-layer geosynthetic-reinforced systems, because the effects of anchorage length, layer spacing, and layer number are ultimately reflected in the proportion of load carried by soil arching.

Although the load-transfer ratio is commonly used in piled embankments to describe the proportion of load carried by rigid piles, the present problem is fundamentally different because no rigid support exists beneath the subsided area. Instead, the overburden load is redistributed toward the stable areas through the combined action of soil arching and the membrane effect of the reinforcement. On this basis, the soil arching load-transfer ratio, η , is redefined here for a purely reinforced system under local subsidence conditions. In this study, η denotes the proportion of the initial load above the subsided area carried and transferred by soil arching, whereas $1 - \eta$ represents the complementary part carried by membrane action. This index is introduced to quantify the contribution of soil arching to the overall load-transfer process and to support mechanism interpretation and comparison among different parameter cases, rather than to serve as a direct design index as in piled embankments [28–30].

For derivation, the problem is idealized as a unit-thickness system under plane-strain conditions, and the subsidence process is treated as quasi-static with inertial effects neglected. The fill is assumed to be homogeneous, with a unit weight of γ , and the initial vertical stress before subsidence is taken as $\sigma_0 = \gamma H$. Owing to symmetry about the centre-line of the subsided area, the base is divided into a subsided area of width B and stable areas on both sides, and the load is integrated over one half of the system.

The main load redistribution in the stable areas is assumed to occur within an influence range of $l_{SM} = 0.75H$, beyond which the vertical stress approximately returns to σ_0 . This value was selected with reference to previous studies and is consistent with published test results showing that the vertical stress curve in the stable areas gradually flattens with increasing distance from the subsided area. Let the vertical stress distributions in the subsided and stable areas be $\sigma_c(x)$ and $\sigma_s(x)$, respectively. As shown in Figure 7, integration over the corresponding regions gives the load reduction in the subsided area and the load increment in the stable areas. Although the choice of $0.75H$ affects the far-field integration range and thus the absolute value of η , it does not alter the overall trends among different parameter cases. Based on this idealization, the proposed index is termed the soil arching load-transfer ratio; for brevity, it is hereafter referred to as the load-transfer ratio. The soil arching load-transfer ratio, η , is defined as the ratio between the load change and the initial load in the subsided area, Q_c , as expressed in Equation (1).

$$\eta = \frac{|\Delta Q_c| + |\Delta Q_s|}{2Q_c} \quad (1)$$

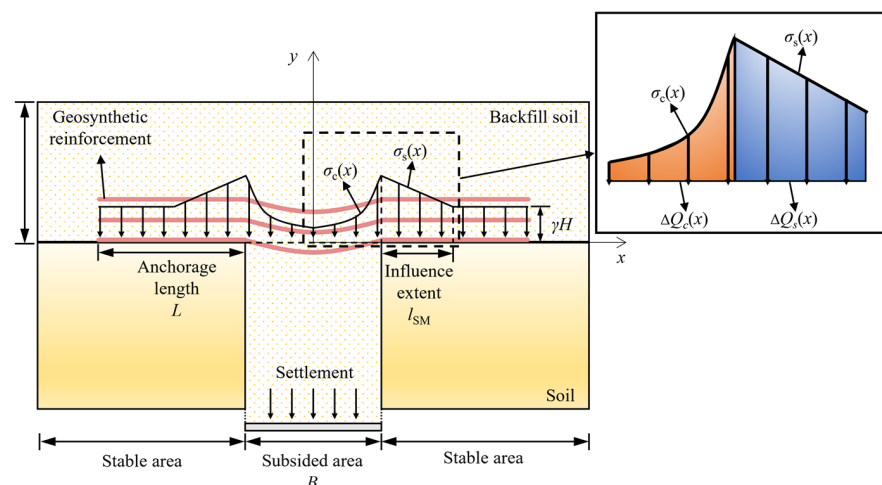


Figure 7. Schematic for calculating the soil arching load-transfer ratio.

It can thus be concluded that the load reduction in the subsided area is represented by the quantity ΔQ_c , whilst the load increase in the stable area is represented by the quantity ΔQ_s . In the fully voided stage, the load in the subsided area is approximately fully transferred to the stable area; therefore, the membrane load-transfer ratio is $1 - \eta$.

Subsequent to the aforementioned assumptions, the following relations are obtained:

$$|\Delta Q_c| = \frac{\gamma HB}{2} - \int_0^{B/2} \sigma_c(x) dx \quad (2)$$

$$|\Delta Q_s| = \int_{B/2}^{B/2+l_{SM}} \sigma_s(x) dx - \gamma H l_{SM} \quad (3)$$

$$Q_c = \frac{\gamma HB}{2} \quad (4)$$

where γ is the unit weight of the fill; l_{SM} is the influence range of load-transfer in the stable area; B is the void width; and $\sigma_c(x)$ and $\sigma_s(x)$ are the stress-distribution functions in the subsided and stable areas, respectively.

The resulting load-transfer ratio is:

$$\eta = \frac{\gamma H(B - 2l_{SM}) - 2 \int_0^{B/2} \sigma_c(x) dx + 2 \int_{B/2}^{B/2+l_{SM}} \sigma_s(x) dx}{2\gamma HB} \quad (5)$$

3. Results

3.1. DEM Model Validation Against the Baseline Tests

To verify the reliability of the developed DEM model, a baseline case consistent with the scaled trapdoor tests was selected for validation, namely, a three-layer reinforced system with an anchorage length of L equal to B and an interlayer spacing of s equal to $0.2B$. The numerical results were converted to the experimental scale according to the similitude relationships and were then compared with the measurements in terms of the final backfill surface settlement, the midspan deflection of the lowest reinforcement layer, and the longitudinal distribution of vertical stress along the lowest reinforcement, as shown in Figure 8. The DEM predictions capture both the overall pattern and the peak location of the vertical stress distribution and provide close estimates of settlement and reinforcement deformation. To quantify the agreement, the mean absolute error and the normalized RMSE of the vertical-stress profile were calculated using the experimental data as the reference at the same monitoring locations. The analysis gives MAE = 0.71 kPa and RMSE = 1.15 kPa, corresponding to NRMSE = 6.32% when normalized by the experimental peak stress of 18.1 kPa. This confirms that the DEM captures the global stress redistribution and the peak characteristics, despite local pointwise discrepancies.

3.2. Experimental Results

3.2.1. Effect of Anchorage Length on Vertical Stress Redistribution and Ratio η

As shown in Figure 9, the vertical stress carried by the lowest geotextile after subsidence exhibits a similar profile for all anchorage lengths. In each case, stress increases from the stable area toward the subsided area, reaches a maximum near the centre, and then decreases toward the opposite side. Across the subsided area, the stress distribution is close to an inverted-parabolic profile, with higher values near the centre and lower values toward both edges, consistent with Villard et al. [31]. Although the literature adopts different assumptions for the load shape on reinforcement, the present data favour the inverted-parabolic form [32]. The curves in Figure 9 are constructed from discrete sensor readings rather than continuous measurements, and the main sources of uncertainty are sensor resolution and specimen-to-specimen variability during preparation. Accordingly,

the discussion focuses on robust trends and peak locations, rather than minor point-to-point differences. The vertical stress distributions reported later in the experimental section are presented and interpreted in the same manner.

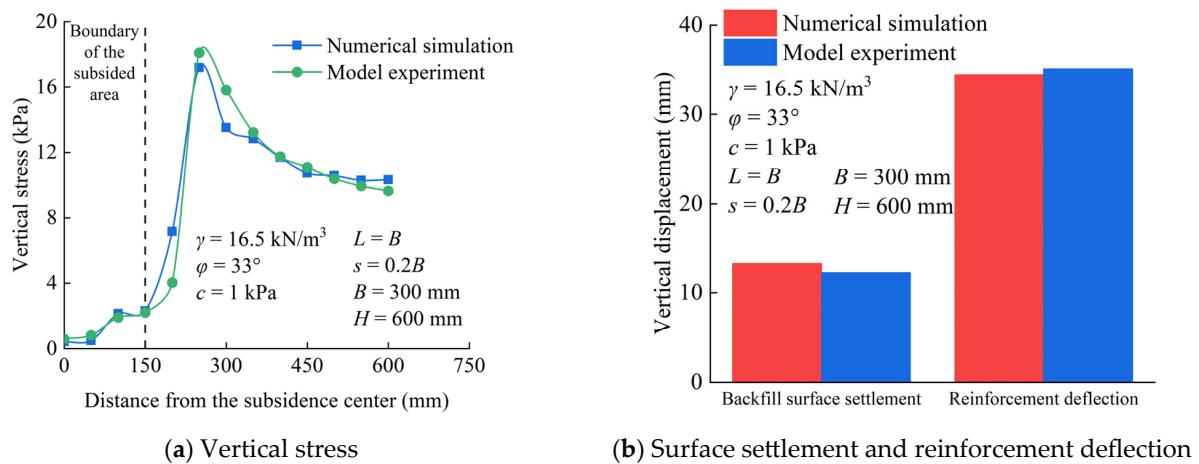


Figure 8. DEM validation against the baseline tests.

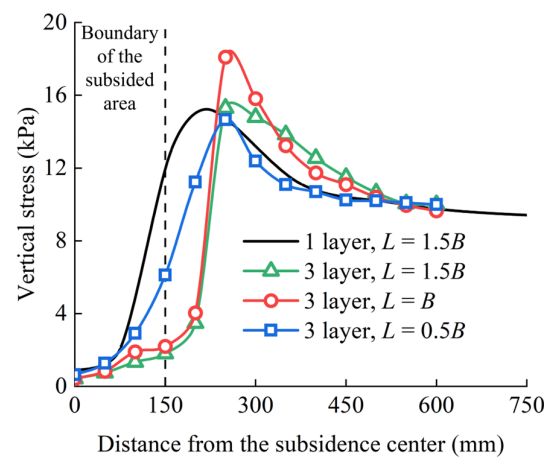


Figure 9. Vertical stress distribution for different anchorage lengths.

Compared with the single-layer reference case with L equal to $1.5B$, the three-layer system reduces vertical stress within the subsided area. At the same time, stresses near the void edge in the stable area remain comparable or slightly higher, indicating a stronger contribution from soil arching. This result suggests that multi-layer reinforcement can achieve comparable or improved load transfer with a shorter anchorage length, which is beneficial when the construction width is limited.

The anchorage-length effect is most evident within the subsided area. Increasing L from $0.5B$ to B substantially reduces vertical stress, with reductions at monitoring points reaching up to 64.03 percent. Extending L from B to $1.5B$ produces a smaller additional reduction, averaging 21.09 percent, indicating diminishing returns. An anchorage length close to B is therefore sufficient to alleviate stress concentration near the subsidence edge. Mechanistically, increasing L enhances tensile mobilization and interface shear resistance, stabilizing the coupled soil arching–membrane load path. Once L approaches B , further extension mainly spreads the transferred load within the stable area and provides limited additional benefit.

In the stable area, the peak stress occurs near x about 250 mm for all cases, but its magnitude varies with anchorage length. The peak is highest for L equal to B at 18.1 kPa,

followed by L equal to $1.5B$ at 15.3 kPa, and lowest for L equal to $0.5B$ at 13.66 kPa. With L equal to $0.5B$, tensile mobilization is limited, and load transfer is weaker, resulting in a lower peak, whereas L equal to B promotes a more concentrated near-field transfer. Extending L to $1.5B$ smooths the stress distribution, indicating more uniform load sharing across the stable area and reduced local stress concentration.

Overall, the response can be described by two regimes. From $0.5B$ to B , the behaviour is anchorage-controlled, and increasing L markedly improves tensile mobilization and strengthens soil arching. From B to $1.5B$, the behaviour becomes arching controlled, and further extension mainly redistributes the transferred load rather than increasing the peak load-transfer capacity.

As shown in Figure 10, three-layer reinforcement provides a clear improvement in load-transfer ratio. The single-layer case with L equal to $1.5B$ reaches 62.33%, whereas the three-layer system reaches 74.82% even with L equal to $0.5B$, an increase of 12.49 percentage points despite the shorter anchorage. Increasing L to B raises η to 86.51%, while a further increase to $1.5B$ produces only a small change to 89.02%. These results indicate that three layers can deliver equivalent or higher arching-dominated load transfer with reduced anchorage length.

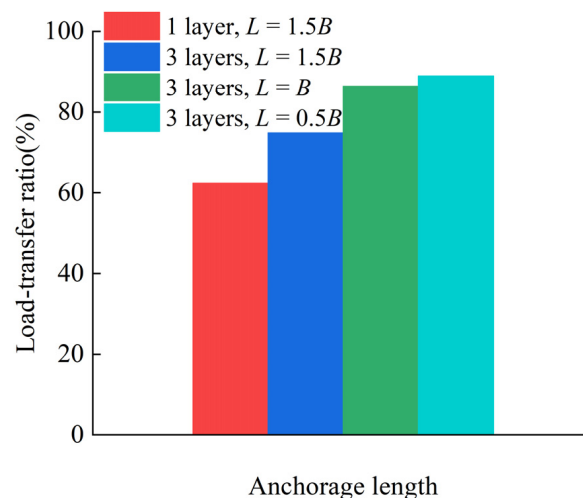


Figure 10. Load-transfer ratio under different anchorage lengths.

For the three-layer cases with s equal to $0.2B$, η increases from 74.82% at L equal to $0.5B$ to 86.51% at L equal to B , a gain of 11.69 percentage points. Extending L from B to $1.5B$ adds only 2.51 percentage points, confirming diminishing returns. Because η represents the fraction of load carried by soil arching, an increase in η indicates a shift from membrane-dominated transfer to arching-dominated transfer. The strong improvement from $0.5B$ to B suggests that soil arching becomes fully mobilized and begins to govern the transfer path within this range. Once η reaches the level observed at L close to B , the arching contribution approaches saturation under the present conditions, so extending anchorage to $1.5B$ provides only marginal additional benefit. Therefore, L close to B is a practical anchorage length that captures most of the achievable improvement in η .

3.2.2. Effect of Layer Spacing on Vertical Stress Redistribution and Ratio η

As illustrated in Figure 11, the vertical stress distribution on the lowest reinforcement layer is shown for three-layer spacings under the same three-layer configuration. Regardless of spacing, the curve retains the same overall pattern: stress remains low near the subsidence centre, rises rapidly toward the void boundary, reaches a clear peak just inside the stable area, and then decays toward a nearly uniform far-field level. Spacing, there-

fore, controls the magnitude and location of the stress concentration rather than the shape of the distribution.

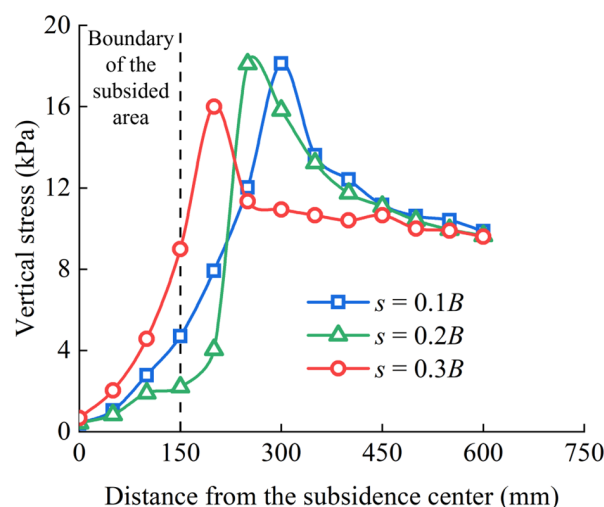


Figure 11. Vertical stress distribution for different layer spacing.

Within the subsided area where x is less than or equal to 150 mm, the stress carried by the lowest layer increases markedly with increasing spacing. At x equal to 150 mm, the stress is 8.99 kPa for s equal to $0.3B$, compared with 4.70 kPa for s equal to $0.1B$ and 2.20 kPa for s equal to $0.2B$. The same ordering is observed closer to the centre, indicating that larger spacing increases the load carried by the lowest layer over the subsided area.

In the stable area, all cases show a peak just outside the void, but the peak position shifts with spacing. The maximum occurs near x equal to 200 mm for s equal to $0.3B$, near x equal to 250 mm for s equal to $0.2B$, and near x equal to 300 mm for s equal to $0.1B$. Beyond the peak, the curves progressively converge, and differences become negligible beyond about x equal to 450 to 600 mm. Taken together, the results indicate that spacing governs where the lowest layer attracts the transferred load along the horizontal direction. Larger spacing shifts the stress concentration closer to the void boundary, whereas smaller spacing moves it farther into the stable area.

As illustrated in Figure 12, the load-transfer ratio η is compared for three-layer spacings at L equal to B . η increases from 61.89% at s equal to $0.3B$ to 86.51% at s equal to $0.2B$, then decreases to 78.75% at s equal to $0.1B$. This non-monotonic response indicates an optimum at the intermediate spacing in the present tests.

Interpreted through the definition of η , layer spacing mainly controls the relative contribution of soil arching and membrane action to load transfer. With larger spacing, the layers act more independently, which weakens the development of soil arching across the reinforced zone. A larger share of the load is then transferred through reinforcement deformation, leading to a lower η . Reducing spacing to s equal to $0.2B$ promotes stronger interaction between layers, so soil arching becomes the dominant transfer path and η reaches its maximum. When spacing is very small, the system behaves more like a laminated package, and membrane action attracts a larger share of the transferred load. As a result, the soil arching share decreases and η drops, even though overall deformation control may still improve. In this way, η complements the vertical stress results by quantifying how much of the transferred load is carried by soil arching rather than by reinforcement deformation.

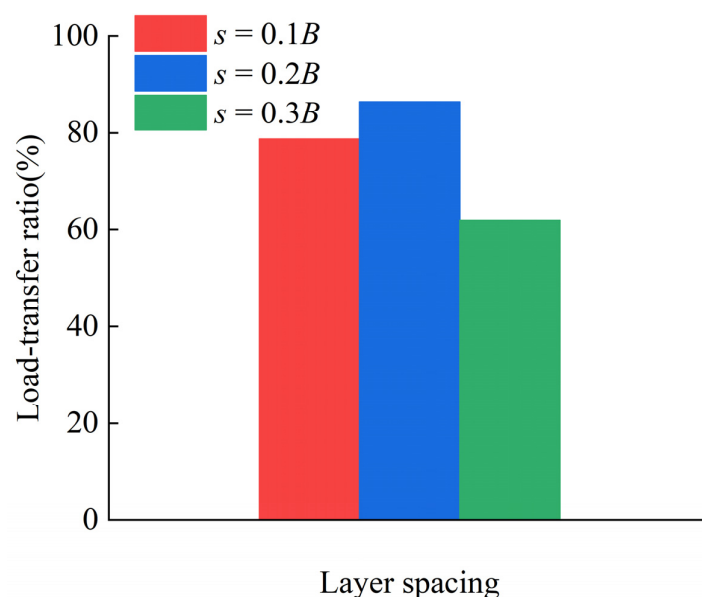


Figure 12. Load-transfer ratio under different interlayer spacings.

3.2.3. Displacement Responses

As demonstrated in Figure 13, the backfill surface settlement and the deflection of the lowest reinforcement are compared between the single-layer reference case and the three-layer system under different anchorage lengths. Relative to the single-layer case, the three-layer configuration reduces both deformation measures at all anchorage lengths. When the anchorage length is $0.5B$, the surface settlement decreases by 10.68%, and the reinforcement deflection decreases by 3.31%. When the anchorage length is increased to B , the reductions become much more pronounced, with the surface settlement decreasing by 34.65% and the reinforcement deflection decreasing by 19.23%. A comparable level is achieved at $1.5B$, where the surface settlement decreases by 35.66%, and the reinforcement deflection decreases by 21.27%. These results indicate that the three-layer system can achieve equal or better deformation mitigation with a shorter anchorage length, which is particularly beneficial when construction width limits the available anchorage distance.

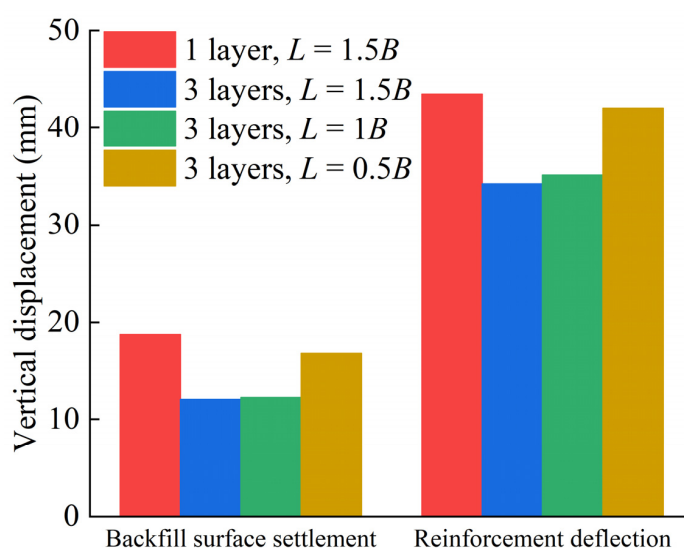


Figure 13. Vertical displacement for different anchorage lengths.

Within the three-layer system, increasing anchorage length further improves deformation control, but the improvement is strongly stage-dependent. Increasing the an-

chorage length from $0.5B$ to B reduces the surface settlement by 26.84% and reduces the reinforcement deflection by 16.46%. By contrast, extending the anchorage length from B to $1.5B$ yields only minor additional improvements, reducing the surface settlement by 1.55% and reducing the reinforcement deflection by 2.53%. This suggests that most of the deformation-control benefit is achieved once the anchorage length reaches approximately B under the present conditions, whereas further extension provides only limited refinement.

From a deformation-response perspective, the superior performance of the three-layer configuration is associated with the distributed restraint provided by multiple reinforcement planes, which limits vertical deformation accumulation through the backfill and reduces the deformation demand on the lowest layer. Increasing anchorage length further enhances this restraint by improving end confinement and enlarging the effective engagement zone of the reinforced system, thereby reducing both reinforcement sagging and surface settlement.

As shown in Figure 14, under the same anchorage condition, reducing layer spacing improves deformation control in the three-layer system. When spacing decreases from s equal to $0.3B$ to s equal to $0.2B$, backfill surface settlement decreases by 28.92%, and the deflection of the lowest reinforcement decreases by 18.64%. Further reducing spacing to s equal to $0.1B$ produces additional decreases of 7.76% in surface settlement and 6.89% in reinforcement deflection. Overall, reducing spacing from s equal to $0.3B$ to s equal to $0.1B$ decreases surface settlement by 34.44% and reinforcement deflection by 24.25%. The improvement is consistent, but the incremental gain becomes smaller when spacing is reduced below about $0.2B$.

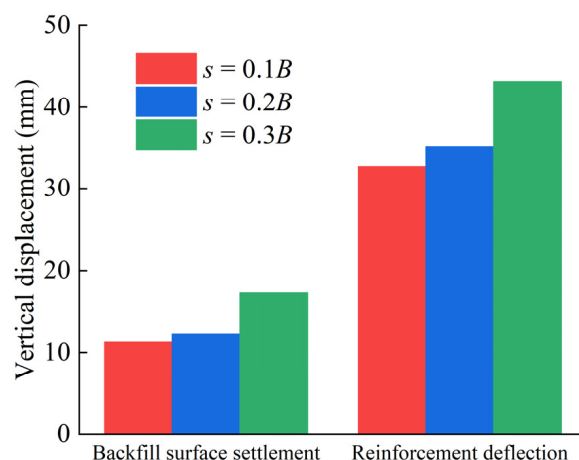


Figure 14. Vertical displacement for different layer spacing.

This spacing dependence can be explained in terms of deformation compatibility. Smaller spacing reduces the thickness of unreinforced soil between adjacent layers, which limits vertical strain accumulation and differential movement within each sublayer. With reinforcement planes placed closer together, vertical deformation is interrupted more frequently through the depth, thereby suppressing surface settlement. At the same time, the lowest reinforcement experiences a lower curvature demand because the overlying soil mass is kinematically constrained by nearby layers, which reduces its deflection.

3.3. Extended DEM Results

3.3.1. Effect of Reinforcement Layer Number on Ratio η

As shown in Figure 15, the load-transfer ratio, η , increases as the number of reinforcement layers increases from 1 to 5; however, the rate of improvement gradually de-

creases, indicating pronounced diminishing marginal returns. Specifically, η increases by 19.49 percentage points when the number of layers increases from 1 to 2, and by a further 8.69 percentage points from 2 to 3 layers. In contrast, the additional gains beyond three layers are limited, with increases of only 2.84 and 1.86 percentage points for the 3-to-4 and 4-to-5 layer transitions, respectively.

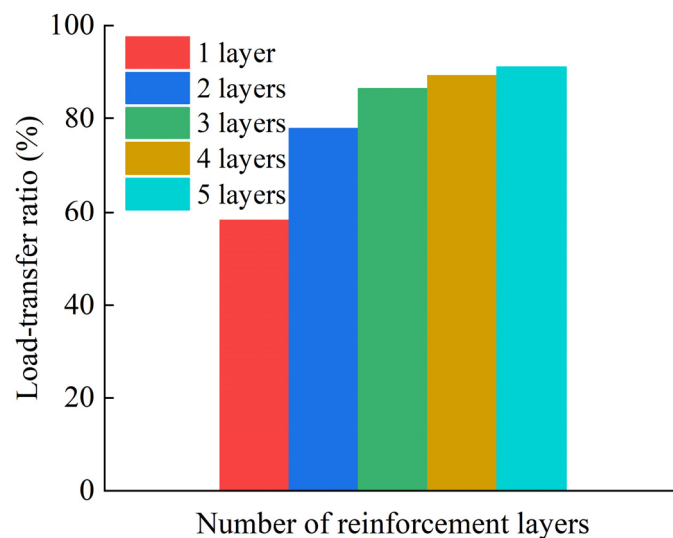


Figure 15. Load-transfer ratio η versus reinforcement layer number (DEM).

The three-layer configuration already achieves a relatively high ratio level, reaching approximately 95% of the upper-bound performance represented by the five-layer case, whereas further increasing to four or five layers provides only marginal benefit. Meanwhile, additional layers inevitably increase material consumption and construction complexity and may elevate the risk of cumulative installation errors. Therefore, a three-layer reinforcement system was adopted as the representative scheme for the subsequent laboratory model tests, as it offers a clear improvement over one- and two-layer systems while maintaining a reasonable balance between reinforcement effectiveness and practical feasibility.

3.3.2. Coupled Effects of Anchorage Length and Layer Spacing on Ratio η

Figure 16 shows how the soil arching load-transfer ratio η varies with normalized anchorage length L over B and normalized layer spacing s over B in the multi-layer reinforced system. For all spacings, η increases as L over B increases, but the improvement rate progressively decreases. The separation between the s over B curves indicates that layer spacing controls the overall level of η and modifies the benefit gained from increasing L over B .

Using s over B equal to 0.2 as an example, η increases from 72.63% at L over B equal to 0.5 to 89.56% at L over B equal to 1.5. Similar trends are observed for other spacings. For s over B equal to 0.1, η increases from 62.5% to 81.6%. For s over B equal to 0.3, η increases from 54.2% to 72.1%. In most cases, η increases rapidly as L over B rises from 0.5 to 1.0, then the gain becomes small as L over B increases beyond about 1.0 to 1.25. For example, at s over B equal to 0.15, η increases only from 91.5% to 92.7% as L over B rises from 1.25 to 1.5. This indicates that extending anchorage beyond L over B of about 1.0 to 1.25 provides limited additional improvement in η .

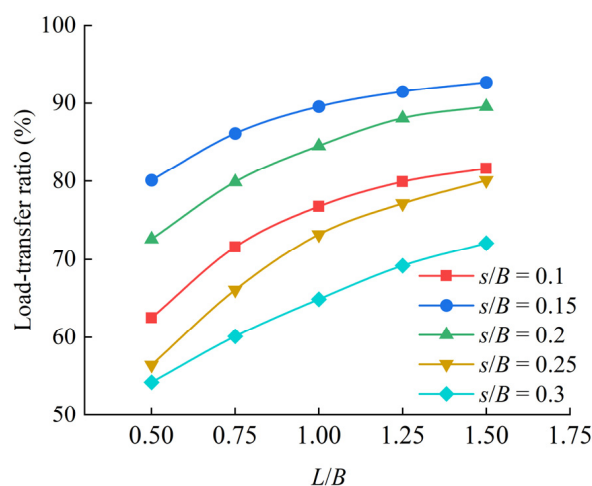


Figure 16. Load-transfer ratio η versus L/B for different layer spacings s/B (DEM).

At a fixed L over B , η varies non-monotonically with s over B . At L over B equal to 1.0, η is 76.75% for s over B equal to 0.1, 89.56% for s over B equal to 0.15, 84.51% for s over B equal to 0.2, 73.2% for s over B equal to 0.25, and 64.89% for s over B equal to 0.3. Among the tested values, s over B equal to 0.15 consistently gives the highest ratio across L over B , ranging from 80.1% to 92.7%. Larger spacings from s over B equal to 0.25 to that equal to 0.3 perform poorly overall. Very small spacing at s over B equal to 0.1 performs better than the largest spacings, but it remains clearly below the combinations with s over B equal to 0.15 to 0.2.

The slope of the curves highlights the coupling between spacing and anchorage length. With favourable spacing at s over B equal to 0.15 to 0.2, η reaches a high level even at short anchorage and then approaches a plateau as L over B increases. In contrast, with unfavourable spacing such as s over B equal to 0.3, η remains low even when L over B reaches 1.5, showing that increasing anchorage length cannot offset an inappropriate layer spacing. In practice, spacing sets the baseline ratio, and anchorage length controls the attainable improvement, so the two parameters should be selected together.

Overall, within the examined range, a moderate spacing near s over B equal to 0.15 to 0.2 combined with anchorage length near L over B equal to 1.0 to 1.25 yields high and stable η . When spacing is too large, η remains low and becomes less sensitive to further increases in L over B .

4. Discussion

4.1. Mechanism Interpretation Using Contact Force-Chain Networks

As shown in Figure 17, the contact-force network highlights clear differences between the single-layer and three-layer reinforcement cases. With single-layer reinforcement, strong contact forces concentrate mainly near the trapdoor edges and around the midspan of the lowest reinforcement. Above the subsided area, many force chains remain scattered rather than forming a stable transfer path, indicating that the overburden load is not reliably redirected to the adjacent stable areas. Strong forces also cluster locally near the boundaries, meaning that load transfer relies on a small number of contacts. Once these local contacts change, the dominant force chains can break, shift, or re-route rapidly, which promotes localized settlement and larger reinforcement deflection. Overall, the single-layer system provides only limited improvement in load redistribution.

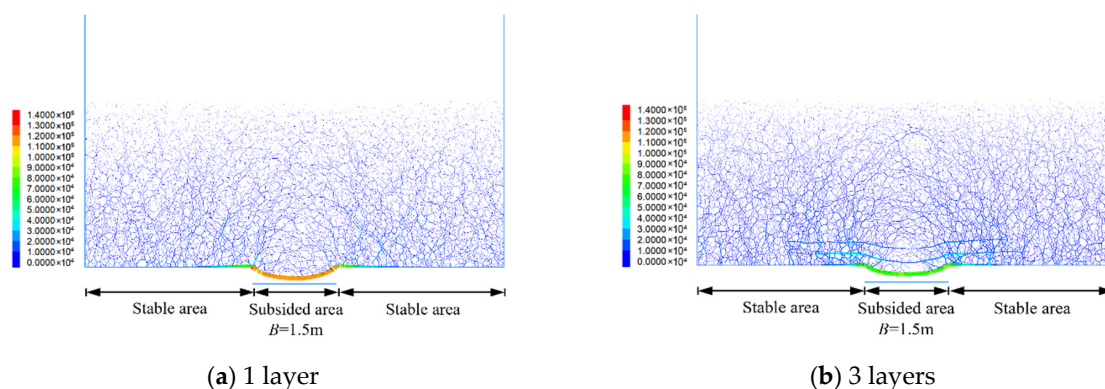


Figure 17. Contact-force chains for the 1-layer and 3-layer cases.

By comparison, the three-layer system forms a more continuous, arch-like force path above the subsided area, and the force chains extend further into both stable areas, reflecting more effective load transfer. The multi-layer arrangement offers more uniform confinement, so strong forces are distributed across the layers instead of being concentrated around a single layer. This sharing reduces local force peaks and stabilizes the load-transfer region near the boundaries. As a result, a stable load-transfer structure is more readily developed and sustained, which accounts for the higher load-transfer ratio and improved deformation control of the multi-layer system.

4.2. Limitations and Applicability

This geometric assessment is also supported by the measured stress distributions. In the layer-spacing tests, the differences among the vertical-stress curves became small beyond approximately $x = 450\text{--}600$ mm in the stable area, indicating that the dominant load-transfer response was largely contained within the adopted influence range and had decayed before reaching the far boundary. To evaluate the effect of this assumption on the calculated load-transfer ratio, the baseline PFC case was recalculated using $l_{SM} = 0.5H$, $0.75H$, and $1.0H$, which gave η values of 80.35%, 86.51%, and 86.87%, respectively. These results show that the absolute value of η varies slightly with the assumed integration range, but the increase becomes negligible once l_{SM} exceeds $0.75H$. This indicates that the choice of $l_{SM} = 0.75H$ has only a limited influence on the mechanistic interpretation and parameter comparisons presented in this study.

Hence, the selected model width is considered sufficient to capture the main load-transfer mechanism and the relative effects of anchorage length and layer spacing. Nevertheless, because the remaining buffer width is only $0.25H$, the boundary effect cannot be regarded as strictly negligible, especially for local peak values near the stable area side of the void. Accordingly, the present results are more suitable for mechanism interpretation and comparison of dimensionless trends, such as the effects of L/B , s/B , and the load-transfer ratio η , than for direct absolute prediction of field stresses, settlements, or reinforcement forces under unbounded ground conditions. Future work should further examine convergence by varying the stable area width and sidewall friction in DEM and by quantifying the resulting changes in peak stress, settlement, and η .

5. Conclusions

Based on 1:5 scaled trapdoor tests and validated PFC2D simulations, this study investigated the load-transfer and deformation behaviour of multi-layer geosynthetic reinforcement for post-collapse remediation under localized subsidence and restricted construction width. The main conclusions are as follows.

1. During subsidence, load was redistributed from the subsided area to the adjacent stable areas, and the vertical stress carried by the lowest reinforcement in the subsided area exhibited an inverted parabolic profile after stabilization. In the three-layer system, the load-transfer ratio η increased from 74.82% at $L = 0.5B$ to 86.51% at $L = B$ and only slightly to 89.02% at $L = 1.5B$. Over the same range, surface settlement and the midspan deflection of the lowest reinforcement decreased by 26.84% and 16.46% from $0.5B$ to B , but only by a further 1.55% and 2.53% from B to $1.5B$. This indicates that an anchorage length of about one void width is generally sufficient to mobilize most of the improvement.
2. Layer spacing affected load transfer and deformation differently. At $L = B$, η increased from 61.89% at $s = 0.3B$ to 86.51% at $s = 0.2B$, and then decreased to 78.75% at $s = 0.1B$, showing that $s = 0.2B$ was the most favourable for soil arching load transfer. In contrast, reducing spacing from $0.3B$ to $0.1B$ continuously reduced surface settlement by 34.44% and reinforcement deflection by 24.25%, indicating that smaller spacing is more beneficial for deformation control.
3. Increasing the number of reinforcement layers showed clear diminishing returns. η increased by 19.49 percentage points from 1 to 2 layers and by 8.69 percentage points from 2 to 3 layers, whereas the gains were only 2.84 and 1.86 percentage points for the transitions from 3 to 4 and from 4 to 5 layers, respectively. Three layers achieved about 95% of the five-layer performance. The extended DEM results further showed that layer spacing controls the baseline level of η , whereas anchorage length governs the attainable increase. Overall, high and stable load-transfer performance was obtained at $L/B = 1.0\text{--}1.25$ and $s/B = 0.15\text{--}0.2$.

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Abbreviations

The following abbreviations are used in this manuscript:

DEM	Discrete element method
MAE	Mean absolute error
MEMS	Micro-Electro-Mechanical System
NRMSE	Normalized Root Mean Square Error
PFC2D	Particle Flow Code 2D
PSD	Particle-size distribution
PTFE	Polytetrafluoroethylene
RMSE	Root Mean Square Error

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