



Performance of Railway Embankments on Soft Ground Improved with Partially Encased Granular Columns

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Abstract

Studies on the effectiveness of partial encasement of granular columns in improving the performance of railway embankments on layered soft ground are limited. To address this gap, a finite element analysis (FEA) was performed using an axisymmetric unit cell model to evaluate the influence of a single granular column on the surrounding soft soil. A fully coupled flow-deformation analysis was adopted to capture both consolidation and irrecoverable settlements, as well as temporal changes in excess pore water pressure during the construction phase of an embankment. Partial encasement lengths of 20%, 40%, 60%, and 80% were analysed and compared with ordinary and fully encased columns under staged construction and train loading conditions. After the sequential embankment construction, equivalent dynamic train loads of 25 and 35 tonnes have been applied at speeds ranging from 40 to 160 km/h to evaluate the response of the improved soft ground. Further, a moving load analysis was conducted using the axle load and geometric specifications of the Alfa Pendular train to assess the undrained response of the improved soft ground. Embankment settlement and stresses were predicted at the base of the embankment, including at the top of the granular column and surrounding soft soil, and the results are presented. These outcomes provide valuable insights for practitioners, enabling informed decision-making for the design of embankments with partially encased granular columns in soft ground.

Keywords Granular columns · Partial encasement · Railway embankment · Soft ground · Finite elements

Introduction

The load-sharing behaviour of the granular column–soft soil composite is primarily governed by the passive radial resistance from the surrounding soft soil. The concept of providing encapsulation to improve the radial confinement of granular columns was initially proposed by Van Impe [1]. Previous studies have reported improvements in load-bearing capacity and vertical settlement control by providing

the full depth of encasement to granular columns through experimental investigations [2–7] and numerical simulations [8–11]. However, from an energy and cost perspective, installing geotextile encasements along the full length of the granular columns presents a notable challenge in the actual field conditions. In the meantime, it has been observed that significant bulging occurs at shallow depths [12] in the granular columns, suggesting that the full length of encasement may not always be necessary. In this perspective, experimental investigations [4, 13] and numerical simulations [14] have been conducted to analyse the behaviour of partial encasement to the granular columns; however, these studies often assume a homogeneous soil medium, which does not fully replicate the actual field conditions. Due to the high installation and instrumentation costs, limited studies have evaluated the performance of embankments constructed over granular column-reinforced soft ground with full [12] and partial encasements [15] through large-scale field experiments. On the other hand, numerical simulations offer an alternative to investigate the behaviour of a railway embankment constructed on soft ground reinforced

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with partially encased granular columns. However, such numerical studies are very limited in the existing literature, focusing on the staged construction of an embankment over a layered soft soil profile.

Previous numerical studies have investigated the behaviour of partially encased granular columns in homogeneous soil profiles, employing an elastoplastic analysis for the embankment layer and a consolidation analysis for the rest periods during the sequential construction of the embankment [14, 16]. This approach fails to capture temporal changes in pore water pressure during construction stage itself, thereby affecting the accuracy of predicted settlements and the estimation of the overall project completion time. The present study employs a fully coupled flow-deformation analysis procedure to address these limitations, which simultaneously accounts for the deformations and temporal changes in pore water pressures during the embankment construction phase. It provides a more realistic simulation of field behaviour. Further, limited studies in the existing literature have reported the behaviour of railway embankments resting on improved soft ground subjected to train loads after the final rest period.

To address these research gaps, this paper evaluates the performance of a railway embankment constructed on a layered soft soil profile at Ballina, New South Wales, Australia, reinforced with partially encased granular columns throughout the sequential layers of embankment construction and during train loading stages. This study comprises the following components: (a) a summary on soil stratification at Ballina, New South Wales, along with the construction of the existing trial embankment at the NTF site, (b) the development of an axisymmetric finite element (FE) model and its validation with field observations, (c) an investigation into the effect of different encasement lengths of granular columns on the behaviour of the railway embankment compared to uncased and fully encased granular columns during the sequential construction stages, (d) an assessment of the impact of train loads on the performance of railway embankment resting on improved soft ground after the post-construction stage using the equivalent dynamic load approach, and (e) an evaluation of the response of the railway embankment under train-induced cyclic loading conditions.

Field Testing and Finite Element Modelling

The Australian Research Council (ARC), in collaboration with the Centre of Excellence in Geotechnical Science and Engineering (CGSE), established the National Field-Testing Facility (NTFF) at the city of Ballina, located on the eastern coast of Australia, with the global coordinates of 28.84°S

latitude and 153.56°E longitude. On this NTF site, more than 15 boreholes have been drilled to collect high-quality soil samples for studying the mechanical behaviour of soil layers, especially estuarine soft soil deposits, which commonly serve as foundation soil for transportation infrastructure along the eastern and southern coastal regions of Australia. The subsurface soil profile at the NTF site (as shown in Fig. 1) comprises a surface layer (0–0.2 m) of very soft, organic material derived from sugarcane residues, underlain by a stiff clayey sandy silt extending to 1.5 m. From 1.5 to 10.5 m, the profile consists of Ballina estuarine clay, a dark, high-plasticity silty clay with very soft consistency near the surface. A transitional zone of fine-grained grey sand having increased fines extends from 10.5 to 14 m, followed by a Holocene sand layer (14–18 m) that is medium to dense and fine-grained. Beneath this, a Pleistocene stiff silty clay stratum extends to approximately 24 m depth, underlain by bedrock composed of argillite meta-sediments. At a location near Flathead Lane within the NTF site, a 3 m high trial embankment was constructed to investigate the performance of granular column-reinforced soft ground. The reinforced area was $18 \times 8 \text{ m}^2$, with 50 granular columns arranged in a square pattern (five rows of ten columns) installed through the wet vibro-replacement technique. The columns, embedded through 9 m of soft estuarine clay and into the transition zone, had final diameters of 800 mm, 1000 mm, and 1200 mm with 2 m spacing between the columns. Before embankment construction, the surface layer of alluvial soil deposits was excavated, and a working platform with a gravel bed and a sand blanket was prepared. Settlement plates were installed above selected columns to monitor the vertical displacements throughout the sequential construction of embankment layers. The average settlements recorded at the base of the embankment at the NTF site, as reported in the existing literature [17], were used to validate the adopted numerical modelling approach in this study.

The present study investigates the behaviour of a railway embankment constructed on a soft soil profile through FE simulations, focusing on the influence of partial encasement of granular columns in railway engineering applications. Figure 1 presents the typical cross-sectional profile of the railway embankment reinforced with granular columns on a soft soil profile at the NTF site, while Fig. 2 illustrates the various levels of partial encasement considered in the study. Details of the FE modelling approach are provided in the following subsection.

Finite Element Modelling

A two-dimensional (2D) finite element analysis (FEA) is performed using the *PLAXIS 2D* program [18] through an

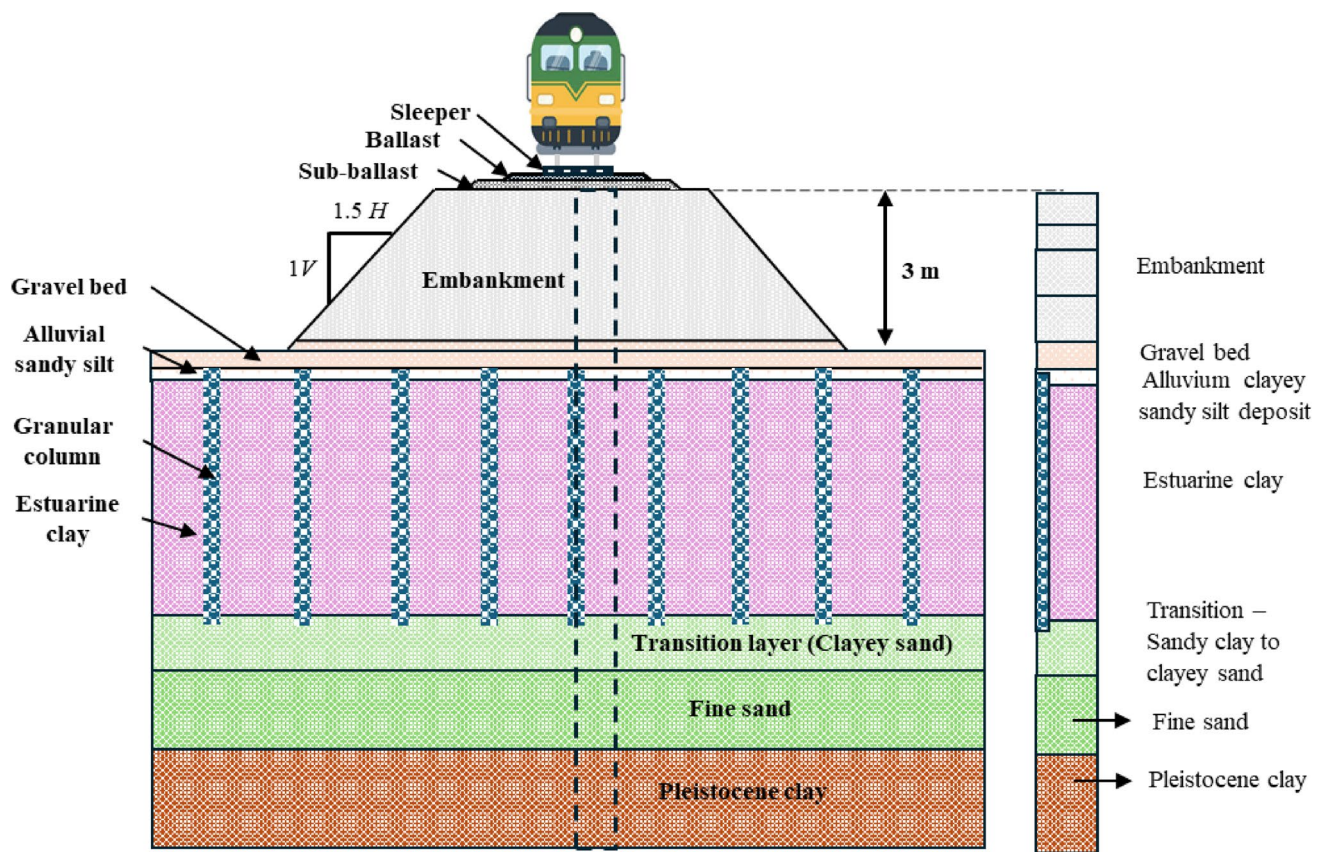
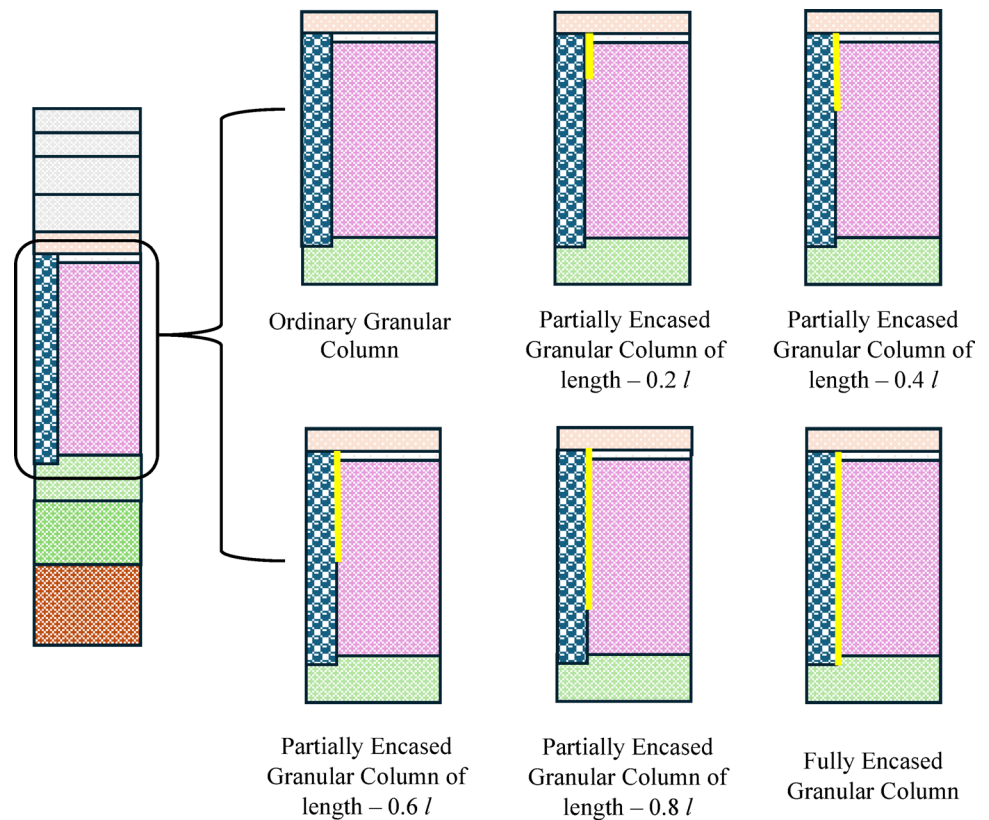


Fig. 1 Typical cross-sectional profile of the railway embankment resting on the soft soil profile at NTF site, Ballina, New South Wales

Fig. 2 Different percentages of partial encasement adopted to granular columns in the study

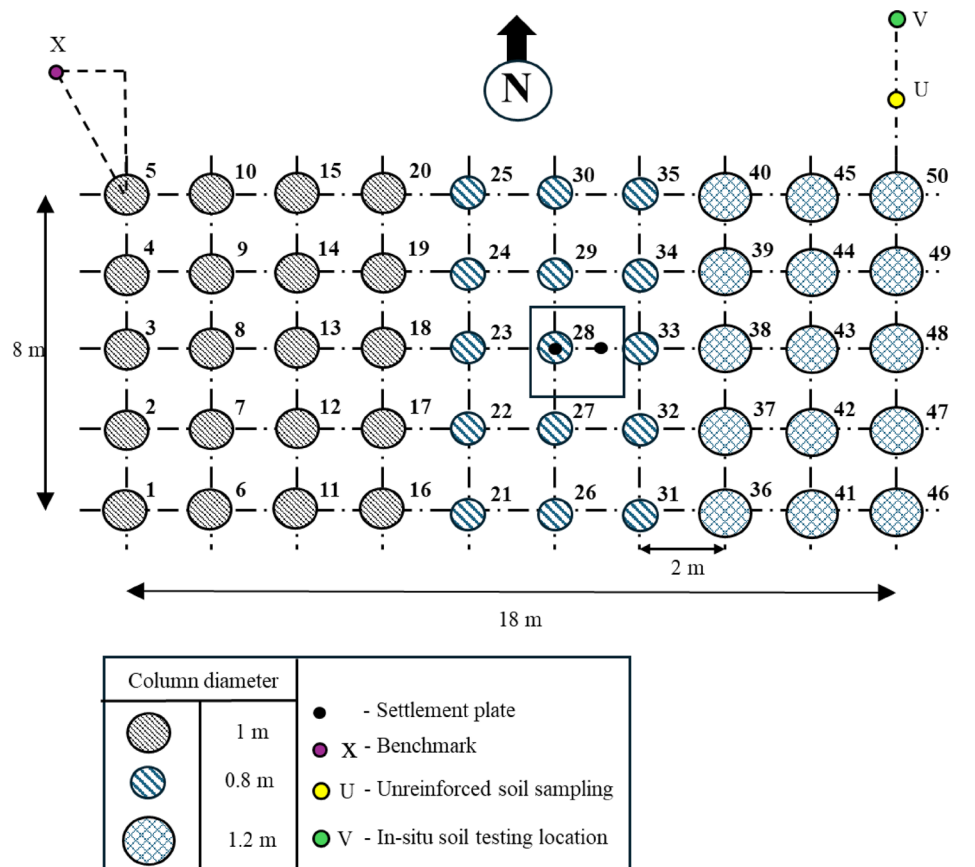


axisymmetric unit cell modelling approach. This approach idealizes a single granular column and its surrounding soil as an equivalent circular area, derived from a square pattern of column arrangement, to realistically replicate the field conditions, as illustrated in Fig. 3. The equivalent radius (R_e) is calculated using:

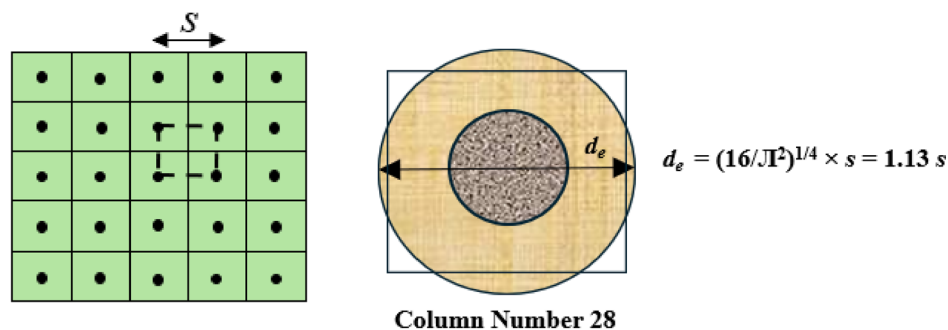
$$R_e = 0.564 \times S. \quad (1)$$

where S denotes the centre-to-centre spacing between adjacent columns [19]. The present study considers granular columns with a diameter of 800 mm and a length of 10.5 m, installed at a spacing of 2 m at the NTF site, resulting in a spacing-to-diameter (S/d) ratio of 2.5 and a length to diameter (L/D) ratio of 13.125. These columns are modelled with partial and full geotextile encasements, with an axial stiffness of 2000 kN/m, as illustrated in Fig. 2. The FE domain is discretised using 15-noded second-order triangular elements with 12 Gaussian integration points, and the

Fig. 3 (a) Layout of installed 50 granular columns at the NTF site [17] and (b) Axisymmetric unit cell representing a single granular column and surrounding tributary soil area



(a)



(b)

updated mesh analysis procedure is enabled to account for large deformation effects.

In field applications, seamless woven geotextiles are used to provide uniform lateral confinement and to prevent the formation of weaker zones [12]. In the present study, the encasement is modelled using geogrid elements with axial stiffness of 2000 kN/m at a strain level of 5% and zero bending stiffness. The thickness and ultimate tensile strength of the geotextile are taken as 4 mm and 350 kN/m, respectively, based on the manufacturer specifications given by hygeosynthetics (<https://hygeosynthetics.com>), and are consistent with the previous studies [16, 20, 21]. It is important to note that, in this FE analysis, the mechanical response of encasement is governed primarily by its axial stiffness, while the other reported material properties do not directly influence the geogrid element response. For the 15-noded second-order triangular elements, geogrid elements are defined with five nodes and five stress points, and axial forces are computed at the Newton-Cotes stress points. Interface elements are introduced to simulate the thin shearing zone at the contact between the encasement material, granular column, and surrounding soft soil. For triangular elements, the corresponding interface elements are defined by five pairs of nodal points, each pair interacting with the five nodes and five stress points of each soil element. The roughness of this interaction is modelled using the strength reduction factor (R_{inter}) as 0.67, consistent with the values used in the previous studies [16, 21–23]. The position of nodes and stress points for the soil elements, geotextile

encasement, and interface elements is shown in Fig. 4. A robust triangulation algorithm generates the finite element mesh for the defined geometry. Mesh density is controlled using a global meshing parameter (l_e), which represents the target element size. This parameter is determined based on a relevant element size factor (r_e), and the geometric boundaries (x_{min} , x_{max} , y_{min} , y_{max}), as.

$$l_e = r_e \times 0.06 \times \sqrt{(x_{max} - x_{min})^2 + (y_{max} - y_{min})^2} \quad (2)$$

Appropriate boundary conditions are applied to the FE model to simulate the axisymmetric problem domain, as in Fig. 5. The vertical boundaries are constrained to prevent horizontal movement, while vertical movement is allowed. The bottom boundary is fully restrained, restricting both vertical and horizontal movements. Specifically, the lateral boundaries are treated as shear-free, with no radial displacement, and the bottom boundary is fixed to prevent vertical movement. A closed consolidation boundary is applied along three sides of the model – bottom, left, and right – to represent ‘no flow’ conditions through these boundaries, while drainage is permitted only through the upper surface of the model. The closed boundary at the bottom of the model simulates the presence of an impervious layer, thereby preventing the dissipation and generation of pore pressure. The left boundary represents the axis of symmetry, across which no flow occurs, and the right boundary corresponds to the outer edge of a unit cell within a group of

Fig. 4 Distribution of nodes and stress points in (a) 15-noded triangular soil element, (b) interface element connected to the soil element, and (c) geogrid element

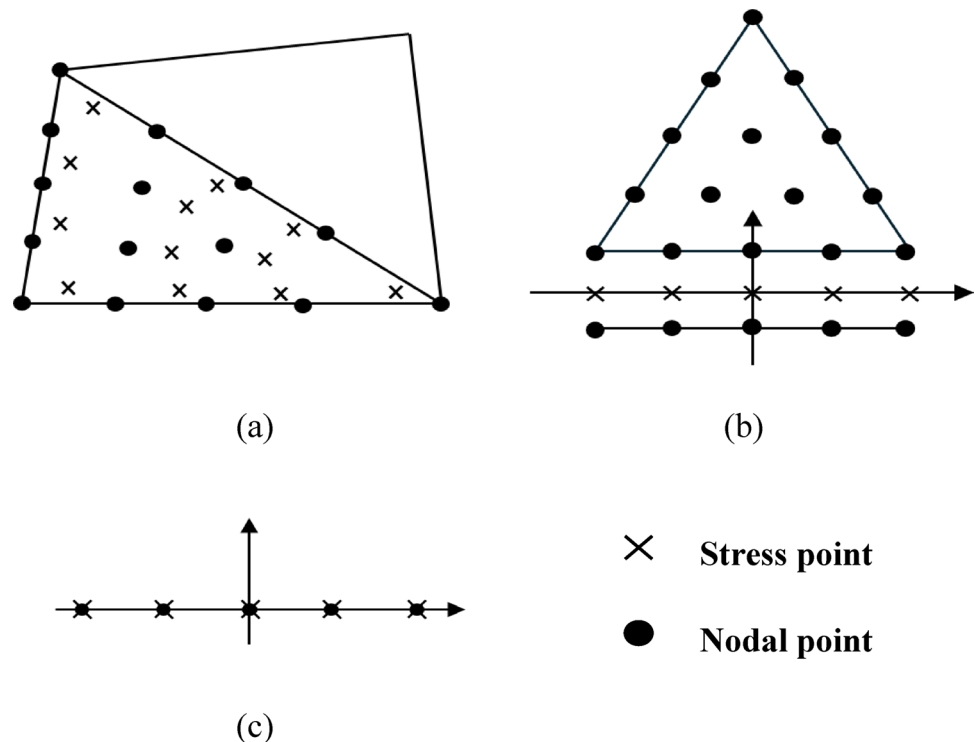
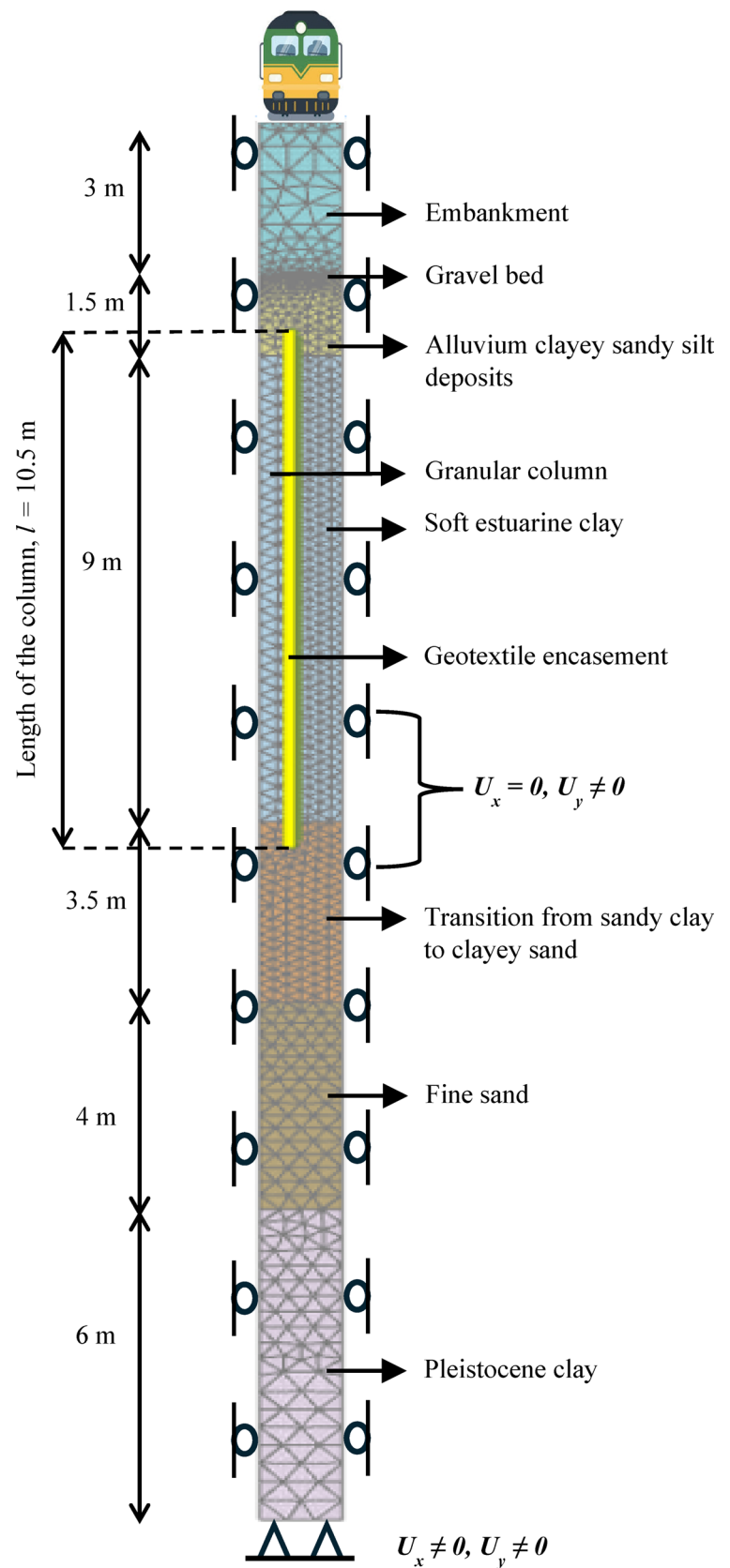


Fig. 5 Mesh discretisation of the axisymmetric FE unit cell model representing the railway embankment on the soft soil profile at the NFTF site



granular columns, across which no dissipation is assumed. The selection of appropriate constitutive models and material properties for the embankment fill, soil layers, granular column, and encasement layer is discussed in the following section.

Constitutive Models

The selection of constitutive models is a critical component of FE analysis, as it significantly influences the accuracy of the mechanical response of granular column-soft soil composites under sequential embankment construction and train loading stages. A wide range of constitutive models – from linearly elastic to advanced nonlinear formulations – are available in the literature. Advanced soil models typically require a detailed set of input parameters, often obtained through extensive laboratory testing. However, such comprehensive data is rarely available for field applications. In the present study, which focuses on the layered soft soil profile at the NFTF site in Ballina, the adoption of advanced soil constitutive models is constrained by the limited availability of reliable soil parameters from the existing literature [24–27]. Therefore, simplified yet representative constitutive models are selected to reflect site-specific conditions while maintaining the computational costs.

This study employs the Mohr-Coulomb constitutive model to simulate the behaviour of various components, in line with previous studies that have adopted it for embankment fill [28], gravel bed, alluvial clayey sandy silt deposits, the transition zone (clay to sand), and Holocene sand [29, 30]. As a first-order approximation, this model combines the elastic behaviour, governed by Hooke's law of isotropic elasticity, with perfectly plastic behaviour defined by the Mohr-Coulomb failure criterion within a non-associated plasticity framework. The assumption of constant stiffness enables faster computations, making it suitable for geotechnical engineering problems. The Mohr-Coulomb model is suitable for simulating sequential embankment construction and analysing equivalent dynamic loading conditions. However, it has limitations in capturing stress-dependent stiffness variations under realistic moving train loads. Despite this, it remains widely adopted for its applicability, computational efficiency, and conservative predictions. In the present study, the sequential construction of a railway embankment over a layered soft soil profile is simulated, followed by the application of a moving train load at the top of the embankment to evaluate the response of the granular column reinforced soft soil deposit. Given the limited availability of a detailed set of material parameters and the need for general applicability to the field conditions, the Mohr-Coulomb model is adopted to simulate the embankment fill.

Locally available gravels near the NTF site were used to construct the granular columns, and the Hardening Soil (HS) model was adopted in this study to capture the behaviour of granular columns. This elastoplastic hyperbolic model accounts for shear and compression hardening, enabling the capture of stress-dependent stiffness and the irreversible compaction behaviour of soils under primary loading. Despite its advantages, the HS model is unsuitable for highly compressible soft soils. Its limitations become evident when the ratio of tangent stiffness to secant stiffness under one-dimensional loading ($E_{oed}^{ref}/E_{50}^{ref}$) falls below 0.5. For the Ballina estuarine clay, this ratio is approximately 0.333, suggesting that the HS model would not realistically represent the behaviour of soft soil under such conditions. As a result, the soft soil (SS) model is adopted for the Ballina estuarine clay, as it is more appropriate for soft clay deposits. The SS model effectively captures the initial compressibility, stress-dependent stiffness, and the unloading-reloading behaviour of soft clays. An undrained analysis is carried out by incorporating the effective strength parameters for the soil layers, while specifying the drainage condition as Undrained A for all materials [18]. This approach enables the generation of excess pore water pressures during staged construction, and subsequent consolidation analysis can be performed following the undrained calculation to simulate the rest period and the associated dissipation of pore pressures. The groundwater table is assumed to be located 1 m below ground level, as in the case at the NTF site. A detailed summary of the soil constitutive models and the corresponding material properties used in the study is given in Table 1. The detailed steps involved in the FE simulation procedure and the validation of the adopted modelling approach with the field data are presented in the following section.

Simulation Procedure and Validation

The FE simulation incorporates staged construction of embankment layers, including the appropriate rest periods, followed by the application of train loading. The construction sequence for each embankment layer is illustrated in Fig. 6, and the adopted analysis procedure is summarised in Table 2. The simulation begins by establishing the initial geostatic stress state using the K_0 procedure and activating the granular column with a plastic analysis procedure. Subsequently, the gravel fill and sand bed layers are placed at the base of the embankment in two days ($t=0$ to 2) and analysed using a coupled flow-deformation analysis. A three-day rest period ($t=3$ to 5 days) is then introduced and simulated using a consolidation analysis procedure, allowing for the dissipation of excess pore water pressure and the prediction

Table 1 Constitutive models and their corresponding material properties adopted in the present study

Layer	Embankment	Gravel bed	Alluvial clayey silt ⁴	Estuarine clay ⁵	Transition – Sandy clay to clayey sand ⁶	Fine sand ⁷	Pleistocene clay ⁸	Granular column ⁹
Constitutive model	MC ¹	MC ¹	MC ¹	SS ²	MC ¹	MC ¹	SS ²	HS ³
Elastic modulus, E (kPa)	40,000	41,000	4000	-	4000	6000	-	-
Poisson's ratio, ν	0.3	0.3	0.35	0.4	0.3	0.25	0.35	0.3
Unit weight, γ (kN/m ³)	21.1	22.76	16	15.21	16	18	14.8	15.5
Cohesion, c' (kPa)	5	1	5	2	5	0	15	0
Friction angle, ϕ' (°)	38	42	30	30	30	35	28	52.5
Dilation angle, ψ (°)	6	5	-	-	-	-	-	6.6
Secant modulus, E_{50}^{ref} (kPa)	-	-	-	-	-	-	-	24,800
Tangent modulus, E_{oed}^{ref} (kPa)	-	-	-	-	-	-	-	21,220
Stiffness modulus, E_{ur}^{ref} (kPa)	-	-	-	-	-	-	-	74,300
Compression index, C_c	-	-	-	1.151	-	-	0.28	-
Swelling index, C_s	-	-	-	0.061	-	-	0.014	-
Coefficient of horizontal permeability, k_h (m/day)	0.1206 [#]	0.1206 [#]	1.7×10^{-4}	2.86×10^{-3}	2.592×10^{-4}	4.32×10^{-4}	0.288×10^{-4}	$10^{\#}$
Coefficient of vertical permeability, k_v (m/day)	0.1206 [#]	0.1206 [#]	3.44×10^{-4}	5.72×10^{-4}	6.48×10^{-5}	1.08×10^{-4}	0.3×10^{-4}	$10^{\#}$

Note: ¹ Mohr-Coulomb (MC) model; ² Soft Soil (SS) model; ³ Hardening Soil (HS) model; ^{4,6,7} data from [53]; ⁵ data from [26, 30]; ⁸ data from [17, 27]; ⁹ data from [54]; [#] assumed values from past experience

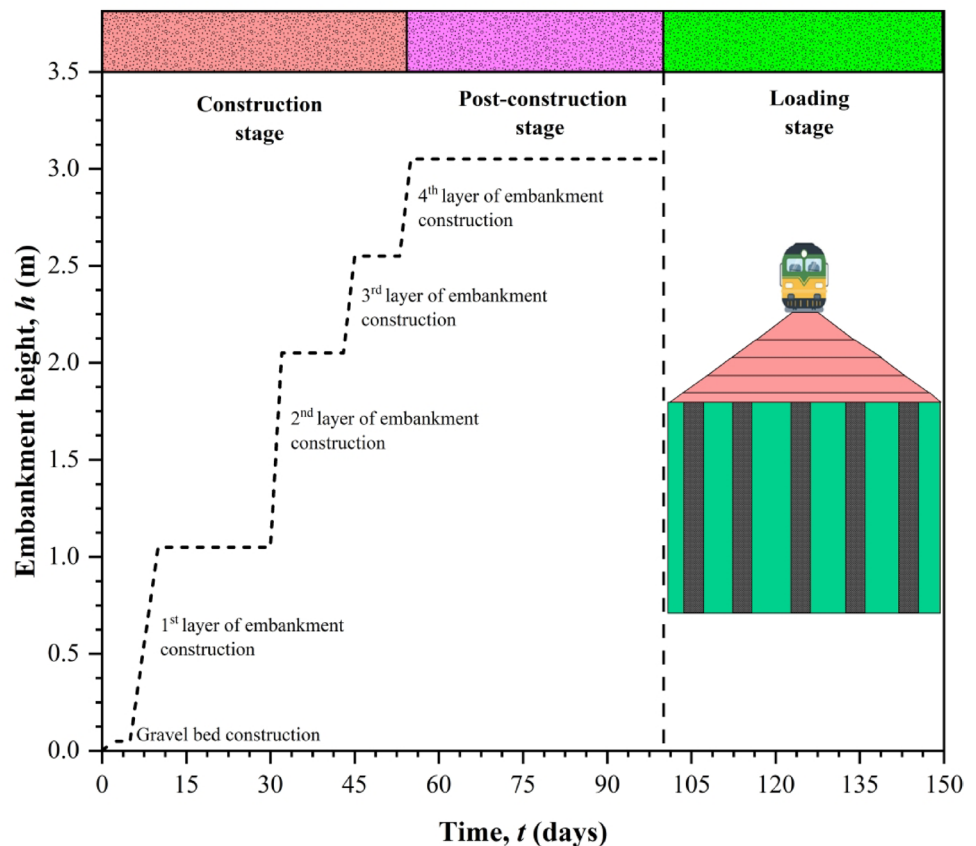
Fig. 6 Sequential layer of embankment construction followed by the operational loading condition

Table 2 Simulation phases of the embankment and analysis procedure adopted in the present study

Construction phase	Time			Analysis procedure
	Start day	End day	Duration (days)	
Geostatic stress state	-	-	-	K_0 procedure
Installation of a granular column	-	-	-	Plastic
Gravel-fill and sand bed	0	2	2	Coupled flow-deformation
Rest period	3	5	3	Consolidation
First 1 m of embankment height	6	10	5	Coupled flow-deformation
Rest period	11	30	20	Consolidation
Next 1 m of embankment height	31	32	2	Coupled flow-deformation
Rest period	33	43	11	Consolidation
Next 0.5 m of embankment height	44	45	2	Coupled flow-deformation
Rest period	46	53	8	Consolidation
Next 0.5 m of embankment height	54	55	2	Coupled flow-deformation
Final rest period	56	100	45	Consolidation
Simulation of train load at the top of the embankment	-	-	-	Plastic / Dynamic with consolidation

of consolidation settlements. The first 1 m embankment lift is constructed in five days ($t=6$ to 10) and analysed using the coupled flow-deformation analysis, followed by a 20-day consolidation phase ($t=11$ to 30). The second lift, another 1 m in height, is constructed in two days ($t=31$ to 32) and followed by an 11-day rest period ($t=33$ to 43). The third lift, comprising 0.5 m of embankment material, is placed in two days ($t=44$ to 45), followed by an 8-day consolidation phase ($t=46$ to 53). The final 0.5 m lift is added in two days ($t=54$ to 55), with a subsequent final rest period of 45 days ($t=56$ to 100). At the end of these construction stages, the stress state of the improved soft ground is established. Train loading is then applied at the top of the embankment to evaluate the response of the granular column-reinforced soft soil composite following the sequential construction of the embankment.

Validation using a case study data

In the present study, the FE domain is discretised using a robust triangulation method, and a mesh sensitivity analysis is performed to assess its effect on the predicted settlements and effective vertical stresses. The results indicate that increasing the mesh density improves the accuracy of predictions; however, the variation between 1053 and 1713 elements is less than 1%, demonstrating that the chosen range of elements has a minimal impact on the overall results.

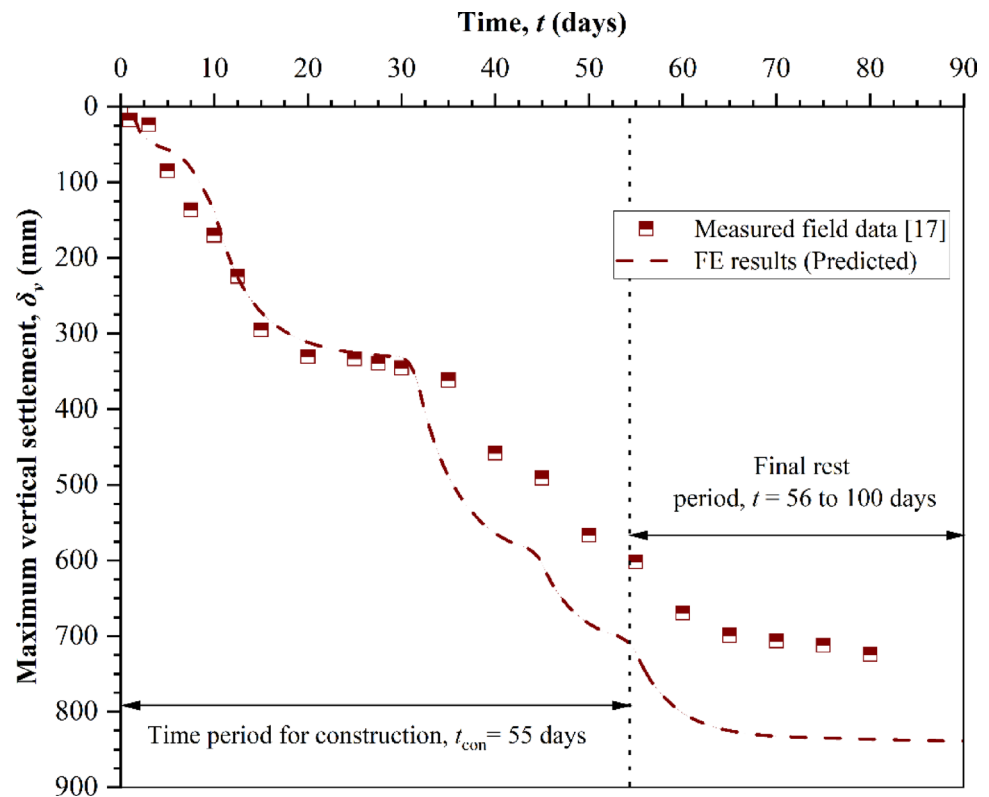
Therefore, a mesh size ranging from 1050 to 1800 elements, with an element size factor of 0.333, is adopted. Local mesh refinements are applied in regions of high stress gradients to balance accuracy and computational efficiency. The discretised mesh for the railway embankment resting on soft ground reinforced with encased granular columns is shown in Fig. 5. The FE results are compared with the observed field data of the NTF site [17], as shown in Fig. 7. It is observed that up to approximately 30 days, the FE predicted settlements closely match the field measurements. Beyond 30 days, during the completion of embankment construction and final rest period – the predicted settlements show noticeable deviation from the field data. This deviation is attributed to: (a) uncertainties in material properties, (b) the inability of the Mohr-Coulomb model (used for soil layers other than the Estuarine and Pleistocene clays) to capture stress-dependent stiffness behaviour, and (c) the limitations of the axisymmetric unit cell modelling approach, which cannot fully represent settlements induced by global lateral deformation of the embankment. Nevertheless, the average predicted settlement at the top of the granular column and the surrounding soil, the results show a reasonable agreement with the field data of the trial embankment at the NTF site in Ballina, New South Wales, confirming the suitability of the adopted modelling procedure for this study.

Validity of the Adopted Unit Cell Modelling Approach

The present study utilises a 2D axisymmetric FE analysis for the following reasons:

- (i) The study primarily focuses on evaluating the efficacy of granular column inclusion with partial encasements in improving the soft ground and predicting the behaviour of the granular column-soft soil composite under operational loading conditions. It involves modelling a single granular column and its tributary soil area, based on the column arrangement and spacing, allowing for a detailed assessment of the behaviour of each component within the composite system, compared to a multi-column configuration.
- (ii) The study also involves the sequential construction of embankment layers followed by train loading stages, which are computationally intensive. Adopting a 2D axisymmetric FE model significantly reduces computational time while maintaining the accuracy of the simulation.
- (iii) Modelling encasement in a plane strain model is not straightforward, as it typically requires increasing

Fig. 7 Comparison between the predicted settlements from the present study and the observed field data [17]



the shear strength parameters of granular columns to replicate the effect of encasement indirectly. However, this approach does not accurately represent the actual behaviour. In contrast, the 2D axisymmetric model enables the direct simulation of the geotextile encasement around granular columns [31, 32].

- (iv) A comparison between the 2D axisymmetric and three-dimensional (3D) unit cell FE models in the previous studies [33–35] shows that the variation in final vertical settlement, stress concentration ratio, and lateral displacement of the granular column is minimum (3 to 5%). This further confirms the suitability of the 2D axisymmetric FE model for the present study, as consistently adopted in the previous studies [17, 21, 36]. The results of the 2D axisymmetric FE analyses, conducted throughout the construction, post-construction, and operational loading stages, are discussed in detail in the following section.

Results and Discussion

This section presents the results of the FE simulations and discusses their significance in addressing the research gap identified in this study. Six cases are analysed, including an uncased granular column, a granular column partially

encased along its length by 20%, 40%, 60%, and 80% of its length (l), and a fully encased granular column-soft soil composite. These cases evaluate the effectiveness of partial encasement of a granular column installed in the layered soft soil profile at the NFTF site in Ballina, New South Wales. The results are organised into two main parts: the sequential layer of embankment construction and the operational train loading conditions. Key parameters evaluated during the construction and post-construction stages include embankment settlement, effective vertical stresses, development and dissipation of excess pore water pressures, and lateral bulging of the granular column. During operational loading conditions, the performance of the constructed embankment on soft ground reinforced with granular columns is assessed based on settlement response, effective vertical stress, and its mobilisation on the granular column, as detailed in the following sections.

Construction and Post-construction Stages

The influence of partial encasement of granular columns on controlling vertical settlement, effective vertical stress distribution, the development and dissipation of pore water pressures, and the nature of lateral bulging is assessed for the railway embankment constructed on soft ground reinforced with granular columns during sequential construction. The vertical settlements are computed at a selected nodal point at the base of the embankment for different cases, including

the unreinforced condition, and partial encasement of the granular column. The results, presented in Fig. 8, depict the final settlements at the end of the construction stage ($t=0$ to 55 days) and at the end of the final rest period ($t=56$ to 100 days), which are 724.53 mm and 846.38 mm, respectively, for the unreinforced case. Similarly, for the encasements of 20%, 40%, 60%, and 80% of the granular column length (l), the settlements respectively are 613.64 mm and 716.19 mm, 504.61 mm and 584.87 mm, 417.68 mm and 482.88 mm, 344.76 mm and 394.62 mm, and for the full encasement the settlements are 320.76 mm and 366.37 mm, respectively. Compared to the unreinforced case, the corresponding reductions in settlements are 15.38%, 30.89%, 42.95%, 53.37%, and 56.71%, respectively. Settlement contours are developed based on the computed values at the base of the embankment with respect to the embankment height during the sequential construction of embankment layers and are shown in Fig. 9. It is seen that the encasement of up to 60% of column length significantly reduces the embankment settlement compared to the case of an ordinary granular column. Beyond 60% of the encasement, the additional reduction in settlement during construction stage is marginal, thereby advocating a limited effectiveness of longer encasements.

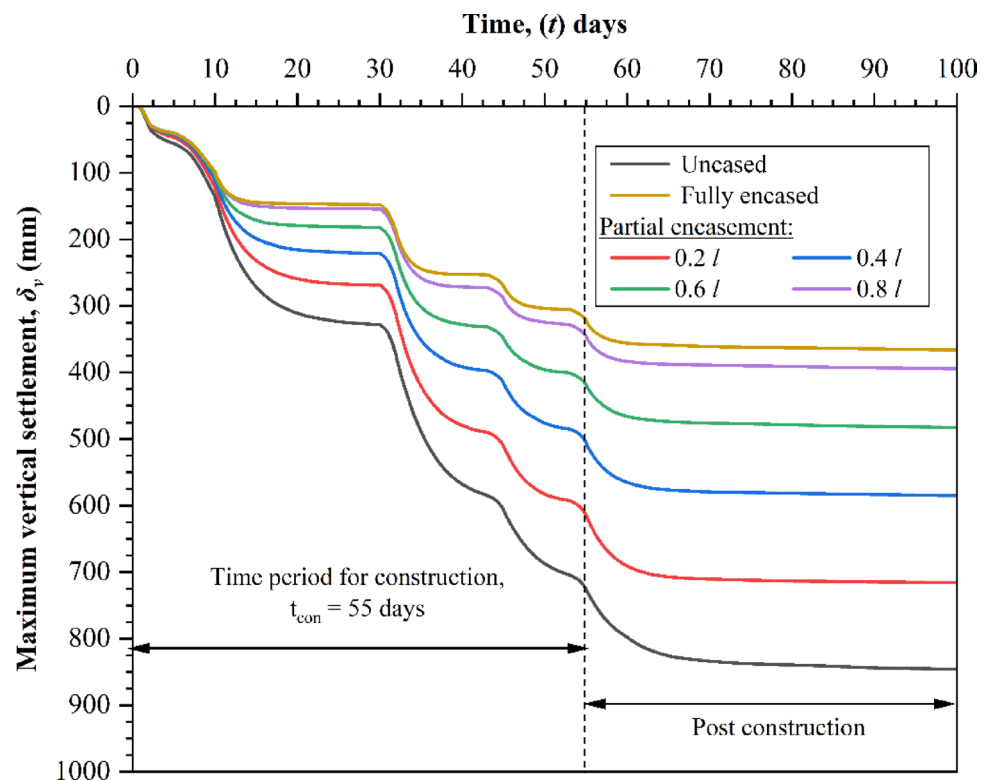
Effective vertical stresses are computed at the top of the granular column and the surrounding soft soil for various cases, including uncased, partially encased, and fully encased granular columns. The results depicted in Fig. 10

present the effective vertical stresses predicted throughout the construction stage and the final rest period. The effective vertical stresses at the top of the granular column and the surrounding soft soil are 218.46 kPa and 63.44 kPa for the uncased case; 386.42 kPa and 40.53 kPa for the 20% of encasement length; 395.67 kPa and 38.46 kPa for the 40% of encasement length; 396.846 kPa and 35.64 kPa for the 60% of encasement length; 408.6 kPa and 34.11 kPa for the 80% of encasement length; and 411.7 kPa and 33.39 kPa for the fully encased granular columns, respectively. The variations in vertical stresses at the top of the granular column and the surrounding soft soil at the base of the embankment are attributed to the soil arching effect [37], which arises from the difference in stiffness between the two components of the granular column-soft soil composite. Although the soil arching is inherently a 3D phenomenon, it is quantified in the 2D axisymmetric analysis through the stress concentration ratio, SCR (η), defined as the ratio of the effective vertical stress at the top of the granular column (σ_{gc}) to the surrounding soft soil (σ_{ss}), expressed as.

$$\eta = \frac{\sigma_{gc}}{\sigma_{ss}} \quad (3)$$

The corresponding SCR values for the uncased, partially encased (20%, 40%, 60%, and 80% of the length of the granular column), and fully encased granular column are 3.44, 9.53, 10.29, 11.13, 11.98, and 12.33, respectively.

Fig. 8 Variation of vertical settlement at the base of the railway embankment for different encasement lengths of granular column



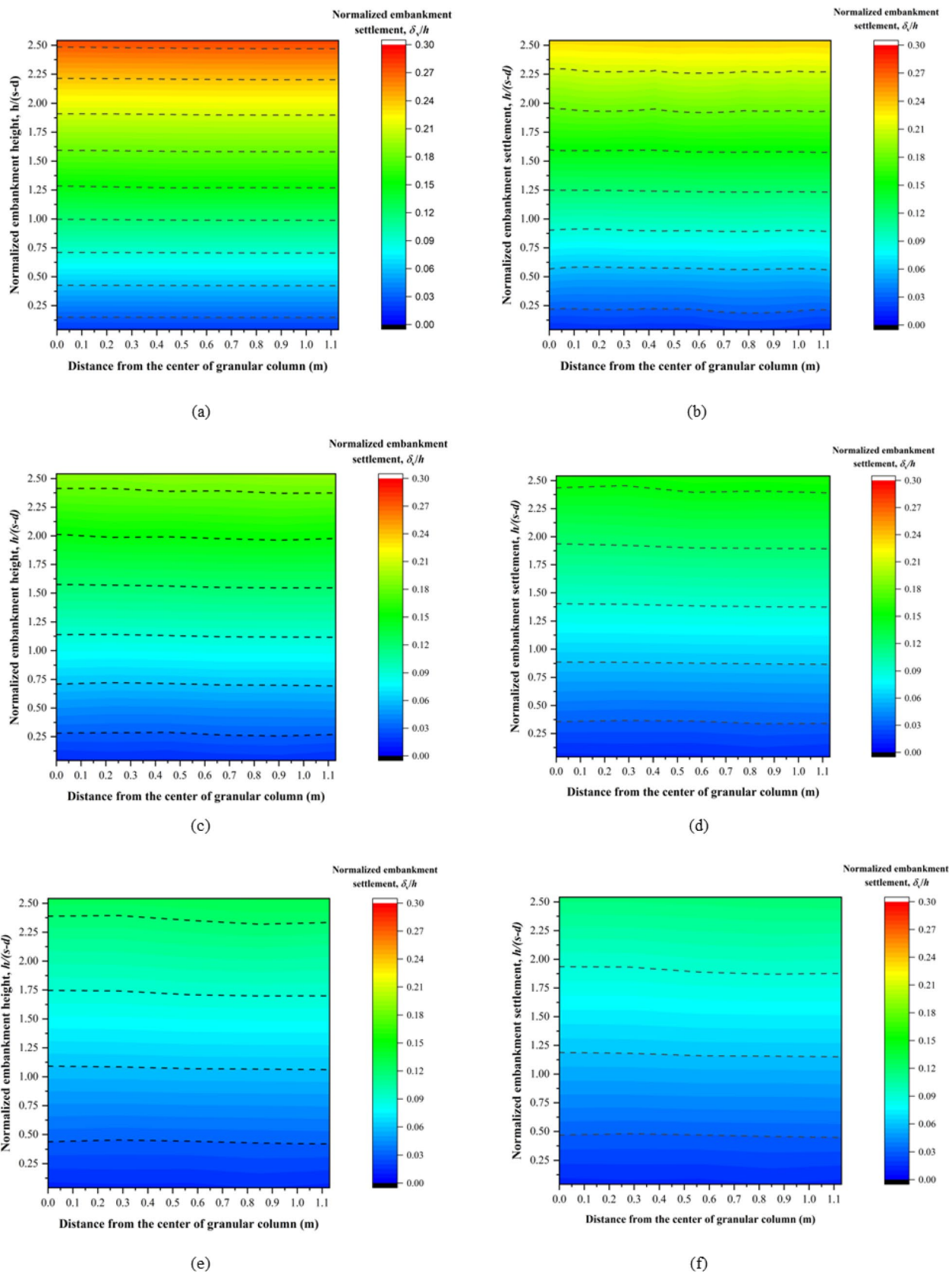
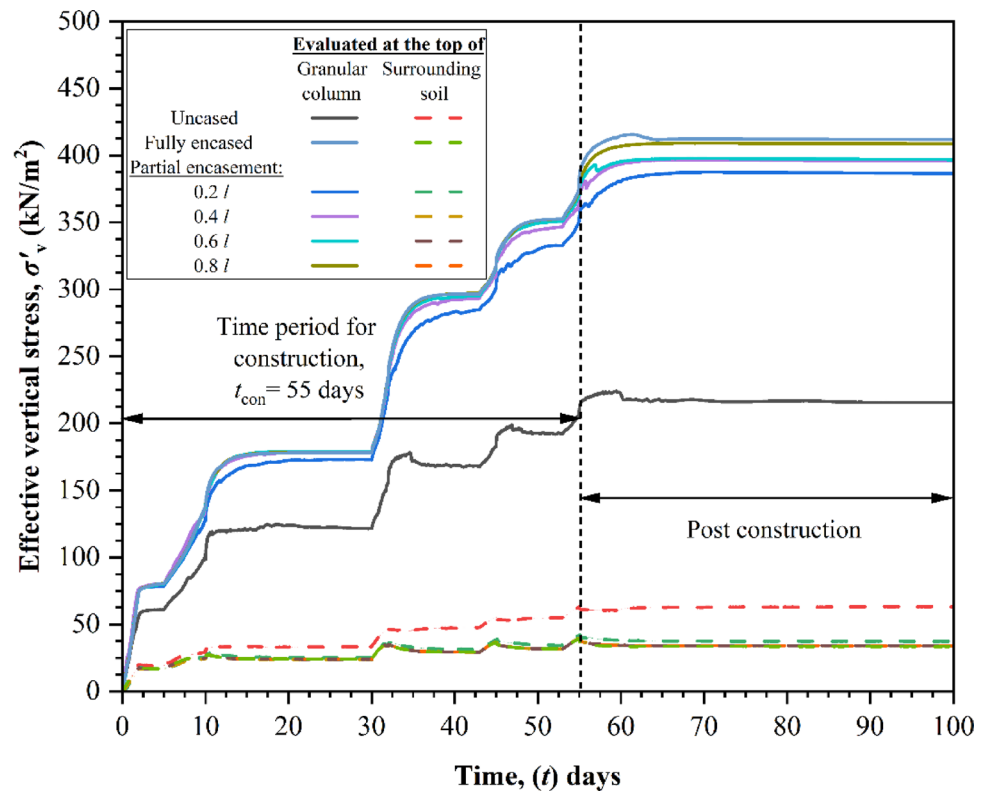


Fig. 9 Variation of vertical settlement at the base of embankment for different cases: (a) ordinary granular column, (b–e) partially encased columns of 20%, 40%, 60%, and 80% of the total length (l), and (f) fully encased granular column

Fig. 10 Variation of vertical stress at the top of the granular column and surrounding soft soil at the base of the railway embankment for different encasement lengths of the granular column



Experimental and field on uncased granular column – soft soil composites have reported SCR values typically ranging from 1.5 to 4 [13, 17], indicating that the predicted values of 3.44 for the uncased column falls within the reported range. Furthermore, previous studies have shown that fully encased columns can achieve SCR values of up to about 12 [12], which is in good agreement with the present findings. Stress contours are developed based on the computed values at the base of the embankment with respect to embankment height during the sequential construction and are shown in Fig. 11. The results indicate that the encasement lengths between 20% and 40% effectively improve the stiffness of granular columns, enabling the granular columns to carry higher embankment loads during both the construction and final rest period stages. These findings suggest that encasing the portion of the granular columns where lateral bulging is more pronounced increases the surrounding confining pressure, thereby improving the load-bearing capacity of the granular columns. However, to effectively control vertical settlement, higher encasement lengths – typically 60% are required, as they provide additional confinement over a greater depth of the granular column and contribute to effective load transfer from the embankment during sequential layers of embankment construction. Figure 12 compares the settlement reduction and stress concentration ratios for the various encasement lengths considered in this study, providing insight into the optimal encasement length required to

achieve the desired level of performance improvement during both the construction and final rest periods.

The load bearing mechanism of the granular column – soft soil composite system is governed by the interaction between the lateral column deformation and consolidation of the surrounding soil. When a vertical load is applied, the granular column tends to expand laterally, and the corresponding lateral stress triggers the development of excess pore water pressure in surrounding soft soil. During the construction and subsequent rest period, dissipates excess pore water pressure, which increases its stiffness and enhances lateral confinement, thereby improving the load bearing capacity of the composite. Figure 13 presents the development and dissipation of pore water pressures at the mid-depth of the estuarine clay for various cases: uncased, partially, and fully encased granular columns. Pore pressures are developed during the construction of embankment layers and dissipated during the corresponding rest periods. The maximum pore pressure observed at the end of the second embankment lift is 15.78 kPa (uncased), 15.36 kPa (20% of encasement), 14.69 kPa (40% of encasement), 12.84 kPa (60% of encasement), 12.48 kPa (80% of encasement), and 12.35 kPa for the fully encased granular column. Compared to the uncased column, the dissipation of pore water pressure becomes progressively smoother as the encasement length increases. Fully encased columns demonstrate the most efficient drainage behaviour within the granular column-soft soil composite system. This improvement is attributed to

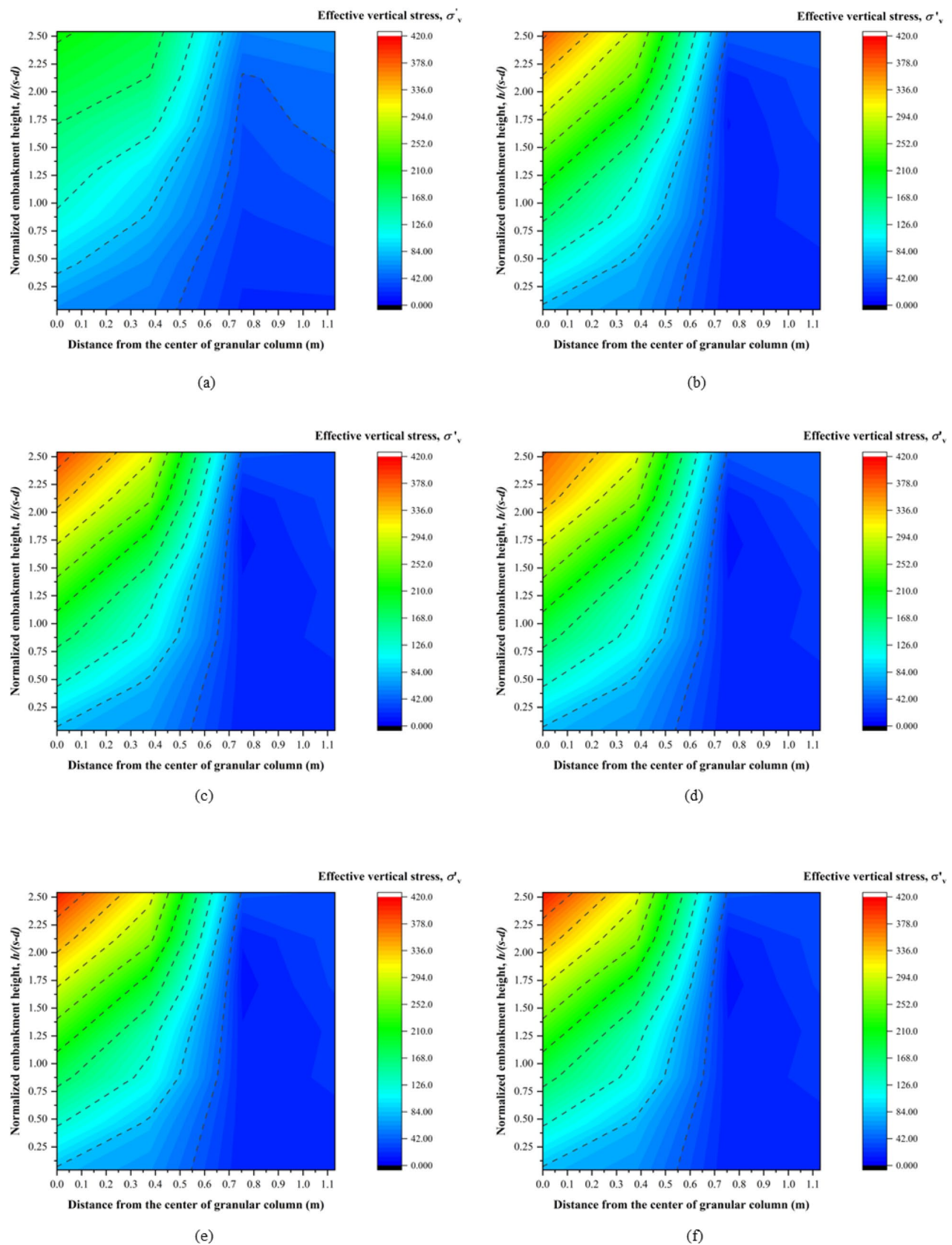


Fig. 11 Variation of effective vertical stress at the base of embankment for different cases: (a) ordinary granular column, (b–e) partially encased columns of 20%, 40%, 60%, and 80% of the total length, and (f) fully encased granular column

Fig. 12 Comparison of settlement reduction and stress concentration ratios for different cases of partial encasements of the granular column during the construction stage

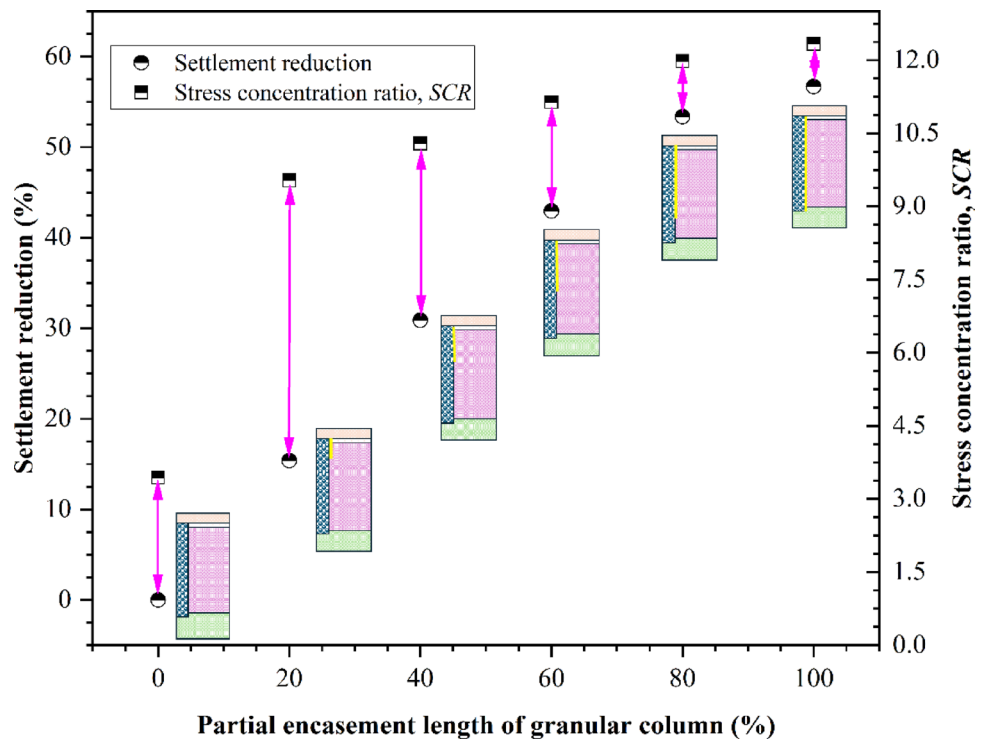
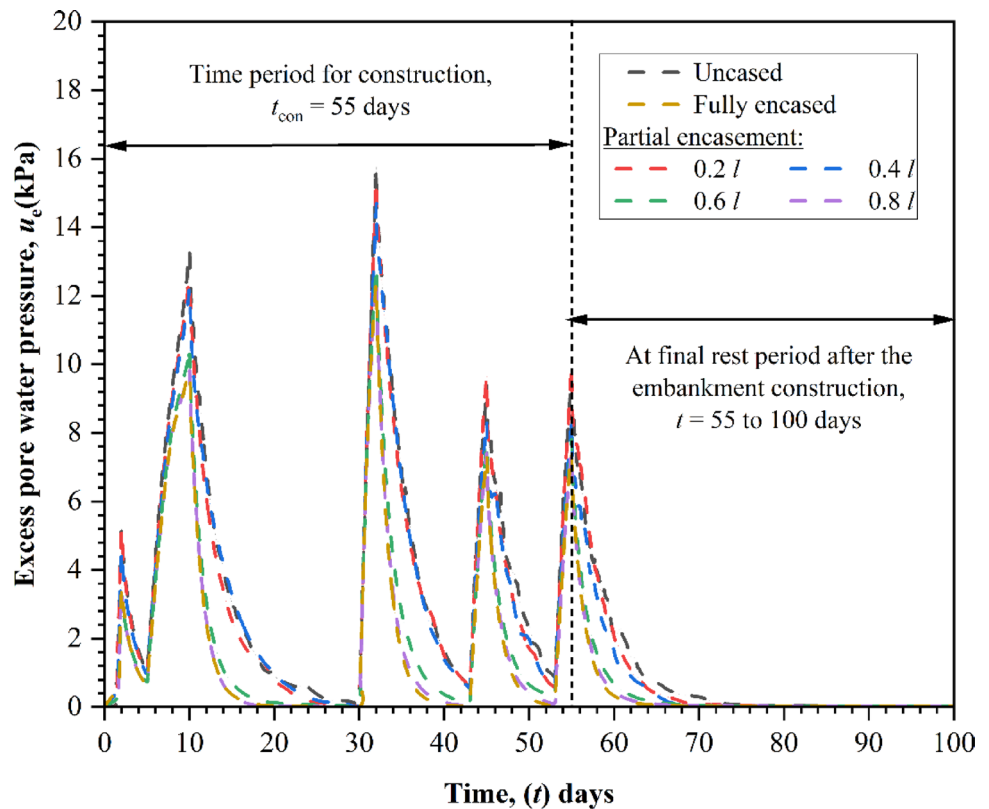


Fig. 13 Development and dissipation of excess pore water pressure in the soil beneath the embankment for different encasement lengths of granular column



the increased stiffness and lateral confinement provided by the longer encasement, which fully covers the bulging zone of the column. As a result, the excess pore water pressure generated in the surrounding soil is slightly lower compared

to the uncased and partially encased columns. However, the rate and extent of pore water pressure dissipation are primarily governed by the permeability nature of the column material, and therefore, the overall dissipation behaviour

and residual pore water pressure remains same across all the cases. The results of this study indicate that encasing approximately 60% of the column length facilitates quicker pore pressure dissipation compared to an uncased column. In fully encased columns, the reduced lateral deformation leads to slightly lower consolidation of the surrounding soft soil during the construction stage, which may result in comparatively higher settlements during subsequent train loading stages. These findings from the study on the development and dissipation of pore water pressure are helpful in determining the construction timeline of the embankment, which directly influences project cost and time management. Previous studies [14, 16] employed a plastic analysis approach in FE calculations, which does not account for the dissipation of pore water pressures during the construction of embankment layers. As a result, these studies tend to overestimate the time required for pore pressure dissipation during the construction phase of the embankment. To address these limitations, the present study employed a fully coupled flow deformation analysis procedure to simulate both the development and dissipation of pore water pressures during the construction stage itself. This approach provides more practical insights for practitioners in planning the timeframes for constructing railway embankments over soft ground reinforced with granular columns.

Figure 14 (a to e) depicts the lateral displacements of partially encased granular columns with varying encasement lengths, as predicted at the end of each construction interval after the placement of successive embankment layers. For columns with 20% encasement length, a significant lateral deformation occurs at the base due to inadequate confinement during the final stage of embankment construction. In contrast, the granular columns with encasement lengths greater than 40% exhibit a significant reduction in lateral displacement, indicating that the increased confining stresses effectively restrain lateral movement and improve the stability of the column. Multiple lateral displacement patterns observed due to the development of shear bands within the granular column are shown in Fig. 14(f). These shear bands are inclined at approximately 57° , typically originating from the top and bottom boundaries of the model, where the lateral displacements are restricted. This observation aligns with the study by Vermeer [38], which states that shear bands in the geomaterials tend to form in the direction of the minor principal stress, with the inclination angle:

$$\theta = 45^\circ + 0.25(\varphi + \psi) \quad (4)$$

where φ is the friction angle, and ψ is the dilation angle. The observed lateral displacement in the column mobilises the shear stresses within the granular column, resulting in a

reduction in the horizontal stresses and a further decrease in the vertical-to-horizontal stress ratio. This stress redistribution drives the column material towards an active failure state, forming multiple shear bands. Notably, due to the layered soil profile and the granular column resting on a fine sand layer, the width of the shear band varies with depth and generally decreases beyond half of the column length. These findings are consistent with previously reported results in homogeneous soil conditions [21, 39, 40], thereby supporting the reliability of the present FE simulations. Furthermore, this study provides new insights into the behaviour of granular columns in layered soil profiles, particularly under varying degrees of partial encasement during the staged embankment construction. After simulating the sequential stages of embankment construction outlined in Table 2, an improved ground stress state is established at the base of the embankment. The following section presents a discussion of the behaviour of railway embankment under train loading conditions, an aspect which is not studied in greater detail in the existing literature.

Operational Train Loading Stages

With the completion of the simulation of embankment construction, the actual stress state of the improved soft ground is established, and the train loading is applied at the top of the FE domain to evaluate the influence of partial encasement of the granular column on the performance of the railway embankment during the operational phase. This section presents a detailed investigation into how the varying lengths of partial encasement influence the deformation behaviour and stress distribution within the embankment. The FE simulations are performed under both equivalent dynamic loading and actual train loading conditions to comprehensively assess the role of partial encasement in enhancing the performance of railway embankments constructed on soft soil deposits.

Equivalent Dynamic Loading Approach

The dynamic behaviour of moving train loads is primarily influenced by axle load, spacing, and speed. A realistic simulation of these dynamic effects in FE models poses considerable challenges and often demands high computational costs. The equivalent dynamic loading approach is widely employed in practice to simplify the computations. In this approach, the static axle load is multiplied by a Dynamic Amplification Factor (DAF) to account for the additional dynamic forces induced by the train movement and is expressed as

$$F_d = F_s \times \text{DAF} \quad (5)$$

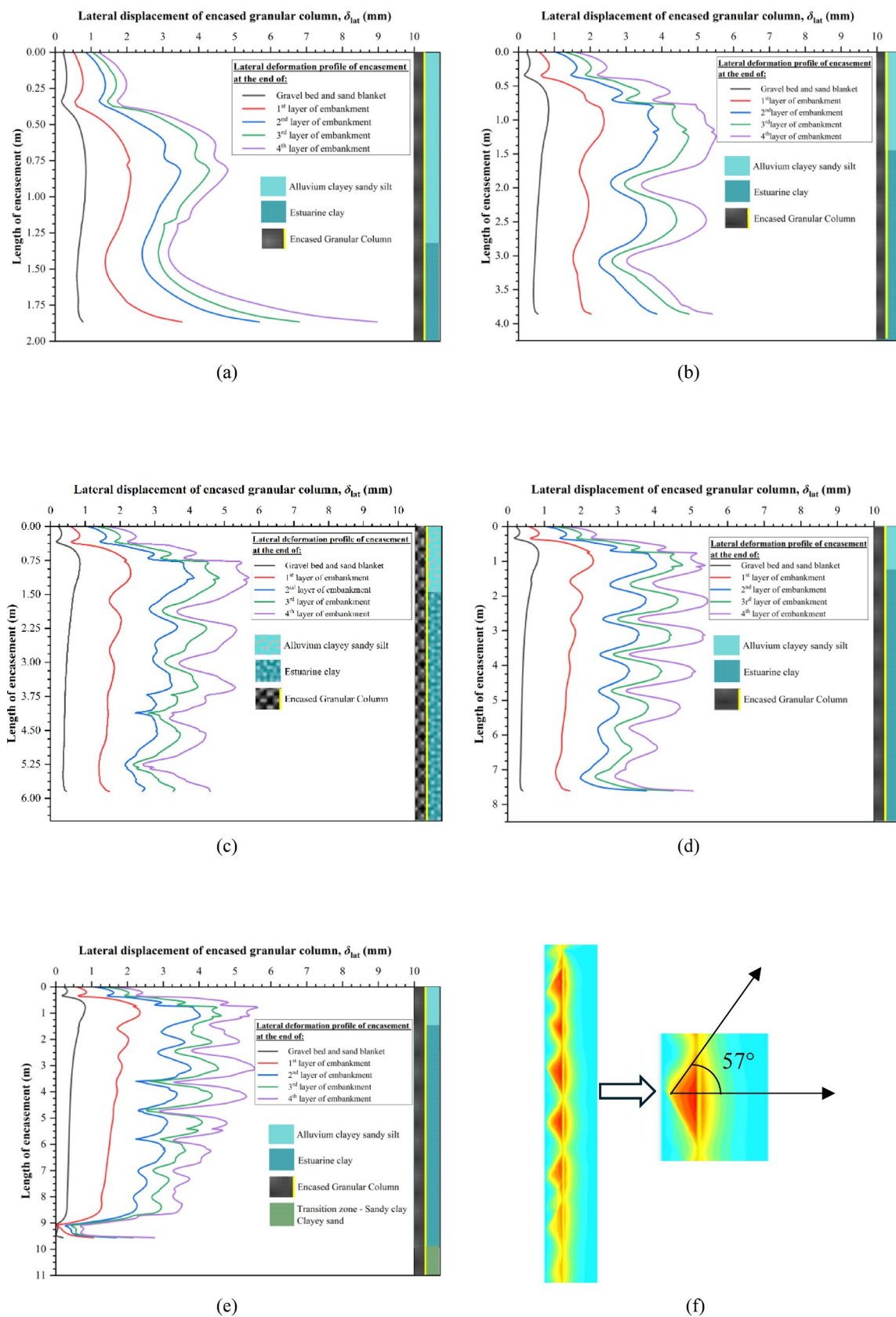
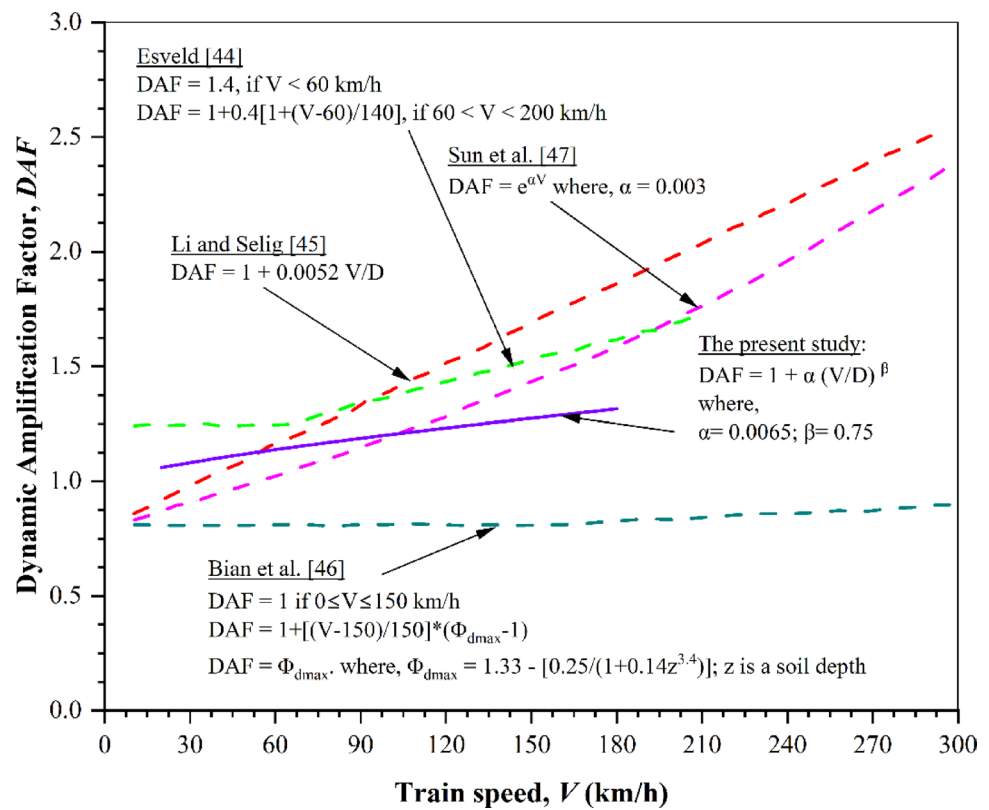


Fig. 14 Lateral displacement and shear band formation in partially encased granular column for different encasement lengths under staged embankment construction

Fig. 15 Comparison of DAF values calculated from the existing methods and field-based data [43]



where F_d denotes the design (equivalent dynamic) wheel load (kN), F_s represents the static wheel load (kN), and DAF is the dynamic amplification factor. The empirical methods proposed by Eisenmann [41] and the Office of Research and Experiments (ORE) of the International Union of Railways [42] are widely used to estimate DAF values. While these approaches are simple and effective, they are generalized and do not account for site-specific conditions, such as the ground profile or track configuration. To overcome these limitations, the present study has adopted the DAF values derived from field investigations conducted in Singleton, New South Wales, Australia [43]. The study in the Singleton soil profile comprises three distinct track formations – soft alluvial deposit, hard rock, and concrete bridge deck – and evaluated the DAF values for all these formations based on the measured ballast contact stresses beneath the sleeper as a function of train speed. For the present study, the DAF values corresponding to the first case (soft alluvial deposit) are used, as this soil profile closely resembles the surface layer of the Ballina soil profile at the NTF site. Figure 15 compares the DAF values obtained from field measurements with those derived from the conventional empirical approaches [44, 45] and previous studies [46, 47]. Train speeds ranging from 40 to 160 km/h are considered in this study under the equivalent dynamic loading approach. The corresponding magnitude of vertical stresses applied at the

Table 3 Summary of the equivalent train-induced vertical stresses on the top of the embankment

Stress calculation method	Train speed, V (km/h)	Dynamic amplification factor (DAF)	Vertical stress on the top of the embankment, σ_t (kPa)	
			Passenger train – 25 T of axle load	Freight train – 35 T of axle load
Trapezoidal approximation (2:1 method)	40	1.105	100.33	141.23
	60	1.143	103.8	145.3
	80	1.178	106.9	153.96
	100	1.212	109.78	157.97
	120	1.243	112.84	161.82
	140	1.274	115.56	165.53
	160	1.303	118.26	167.01

top of the FE domain for the axle loads of 25 T and 35 T are summarised in Table 3.

Embankment settlements are obtained at the selected nodal points, located at the top of the granular column and surrounding soil, under applied train loads, after the staged construction sequence, including the final rest period, has ended. Figures 16 and 17 present the settlement responses for axle loads of 25 T and 35 T, respectively, at different train speeds. The analysis considers various granular column configurations: uncased, partially encased (20%, 40%, 60%, and 80% of the column length), and a fully encased column. Across all configurations, the results indicate that

Fig. 16 Variation of vertical settlements at the base of the embankment subjected to a 25 T axle load under varying train speeds

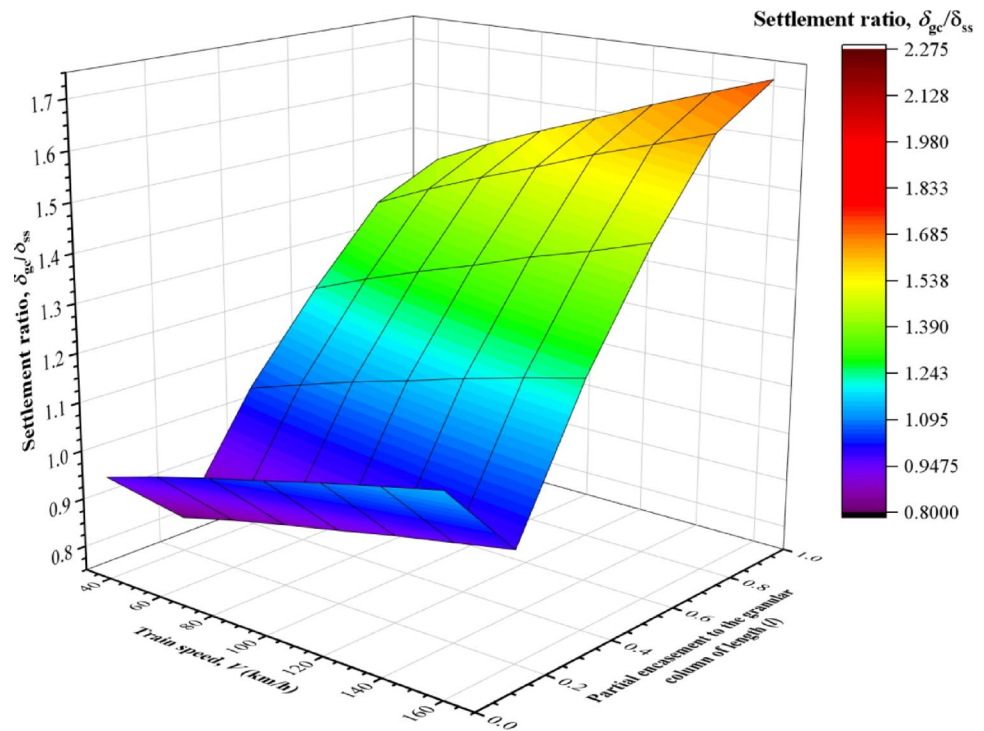
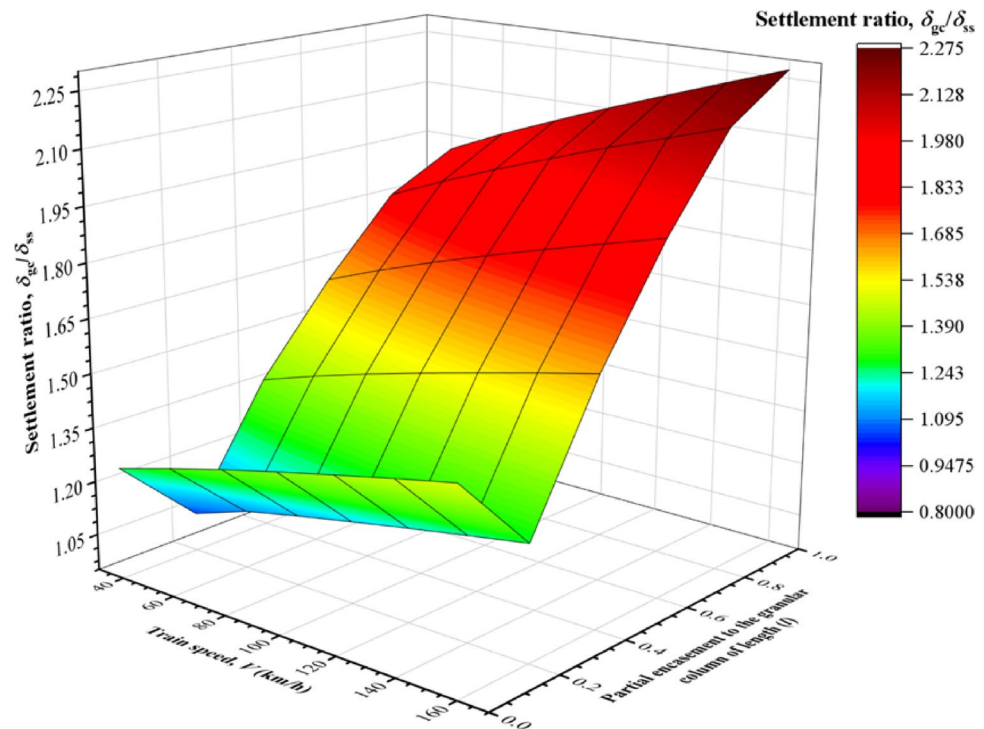


Fig. 17 Variation of vertical settlements at the base of the embankment subjected to a 35 T axle load under varying train speeds



settlement increases consistently with train speed. Notably, two significant findings can be noted from this present study: (i) a granular column with 20% of encasement length exhibited lower settlement compared to the uncased column and contrary to the conventional findings, and (ii) a granular column with larger encasements (40%, 60%, 80%, and a fully encased granular column) resulted in higher

settlements than the uncased granular column. The primary reason for these two observations is related to the degree of consolidation achieved during the staged construction of the embankment. In the case of an ordinary granular column, due to the absence of encasement, higher lateral deformations occur, resulting in a higher degree of consolidation and increased settlements during the construction stage itself.

The larger encasements (40%, 60%, 80% of column length, and fully encasement) will fully cover the bulging zone and restrict the lateral deformations, thereby transferring the embankment stresses more effectively to the granular column through soil arching. The limited degree of consolidation due to encasement results in lower settlements

Table 4 Summary of computed settlements under various train speeds for the partial length of encasements to the granular column

Partial encasement	Train speed (km/h)	25 T Axle load		35 T Axle load	
		Δ_{gc}/H_e	Δ_{ss}/H_e	Δ_{gc}/H_e	Δ_{ss}/H_e
Ordinary granular column	40	0.0095	0.01	0.0124	0.0131
	60	0.0098	0.0104	0.0129	0.0136
	80	0.0101	0.0107	0.0133	0.014
	100	0.0104	0.0110	0.0137	0.0144
	120	0.0107	0.0113	0.0140	0.0148
	140	0.0110	0.0116	0.0144	0.0151
20% of the encasement length	160	0.0112	0.0118	0.0147	0.0155
	40	0.0081	0.0085	0.0106	0.0108
	60	0.0084	0.0088	0.0109	0.0115
	80	0.0086	0.0091	0.0113	0.0119
	100	0.0089	0.0093	0.0116	0.0122
	120	0.0091	0.0096	0.0119	0.0125
40% of the encasement length	140	0.0093	0.0098	0.0122	0.0128
	160	0.0096	0.0101	0.0125	0.0131
	40	0.0104	0.0108	0.0137	0.0142
	60	0.0107	0.0112	0.0141	0.0147
	80	0.0111	0.0116	0.0146	0.0152
	100	0.0114	0.0119	0.0150	0.0156
60% of the encasement length	120	0.0117	0.0123	0.0154	0.0160
	140	0.0120	0.0126	0.0158	0.0164
	160	0.0123	0.0129	0.0162	0.0168
	40	0.0121	0.0126	0.0161	0.0166
	60	0.0126	0.0131	0.0166	0.0172
	80	0.0130	0.0135	0.0172	0.0178
80% of the encasement length	100	0.0134	0.0139	0.0177	0.0183
	120	0.0138	0.0143	0.0182	0.0188
	140	0.0141	0.0146	0.0189	0.0194
	160	0.0145	0.0151	0.0191	0.0198
	40	0.0137	0.0141	0.0181	0.0187
	60	0.0142	0.0147	0.0188	0.0194
Fully encased granular column	80	0.0147	0.0151	0.0194	0.02
	100	0.0151	0.0156	0.021	0.0206
	120	0.0155	0.0161	0.0205	0.0212
	140	0.0159	0.0164	0.0211	0.0217
	160	0.0163	0.0168	0.0216	0.0223
	40	0.0144	0.0148	0.0191	0.0197
	60	0.0149	0.0154	0.0197	0.0204
	80	0.0153	0.0159	0.0205	0.021
	100	0.0158	0.0163	0.0211	0.0217
	120	0.0162	0.0168	0.0216	0.0223
	140	0.0167	0.0172	0.0221	0.0228
	160	0.0171	0.0177	0.0227	0.0234

Note: Δ_{gc} – Settlement predicted at top of granular column; Δ_{ss} – Settlement predicted at top of surrounding soil; H_e – Height of the embankment

during the sequential construction stage; however, larger settlements occur during the train loading stages compared to those of an ordinary granular column. This clearly indicates that accounting for the degree of consolidation achieved for the different cases of partial encasement during the construction stage becomes critically important when the embankment is subjected to its operational train loading conditions. Additionally, it can be observed that for a 20% encasement length, the settlements during staged construction are lower than those of the soft soil deposit reinforced with an ordinary granular column. Although the encasement length partially covers the bulging zone, it is sufficient to mobilise the embankment stresses to the column; however, it lacks the capacity to control settlement during staged construction. This effect is attributed to the lower settlement of the embankment during the train loading stage compared to that of the ordinary granular column. A detailed summary of the computed settlements is provided in Table 4 and will serve as a valuable reference for practitioners involved in the design of railway embankments on soft ground.

Effective vertical stresses at the base of the embankment, i.e. on the top of the granular column and the surrounding soil, are obtained to quantify the mobilisation of soil arching and the stress transfer to the granular columns under applied train loads. Figures 18 and 19 illustrate the variation in computed stresses for axle loads of 25 T and 35 T, respectively, at different train speeds and encasement conditions of the granular column. For partially encased granular columns, the SCR values decrease with increasing train speed. Notably, over 20% of the encasement length, the SCR values are significantly higher than those of an ordinary granular column. However, for the larger encasement lengths (40%, 60%, 80% of the column length, and a fully encased column), a slight increase in SCR is observed compared to 20% of partial encasement. This observation suggests that providing an encasement in the upper portion of the bulging zone of the granular column is sufficient to improve the load-carrying capacity of the column; however, to effectively control the vertical settlement, a deeper length of encasement is essential. A detailed comparison of settlement ratios and stress concentration ratios (SCRs) is provided in Table 5 and will serve as a valuable reference for practitioners to determine the optimum encasement length for columns in the design of railway embankments on granular column-reinforced soft ground.

Performance of Embankment Under Moving Train Loads

The dynamic response of railway embankments to moving train loads is influenced by several factors, including axle load, train speed, axle spacing, wheel-rail interaction, track discontinuities, and the reinforced subgrade. The equivalent

Fig. 18 Variation of stress concentration ratio at the base of the embankment subjected to a 25 T axle load under varying train speeds

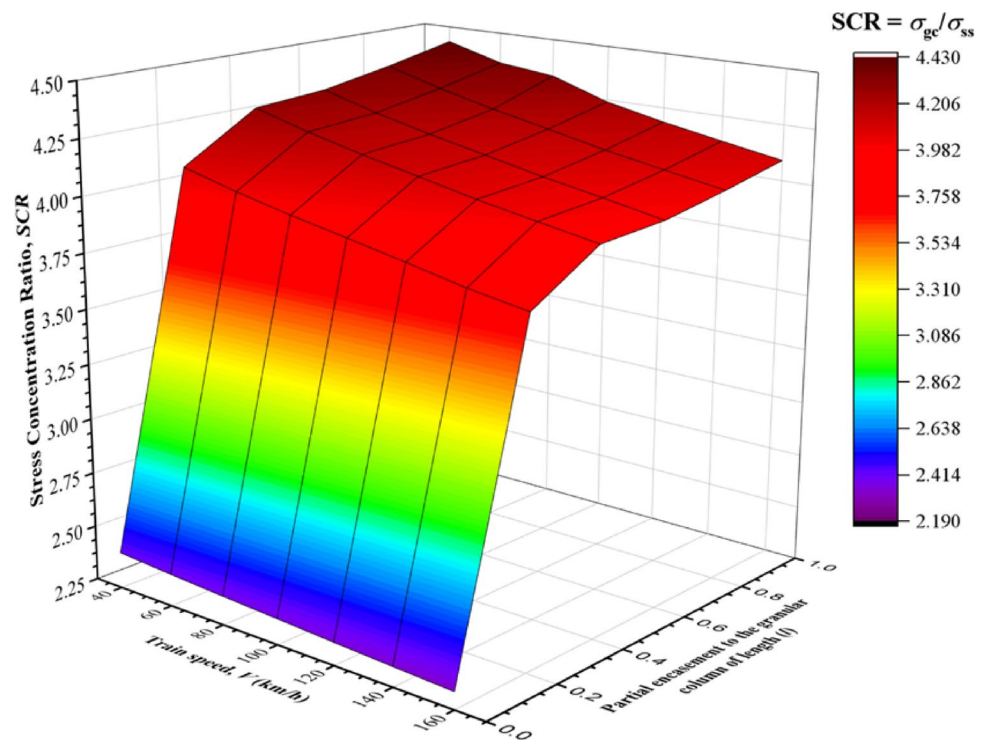
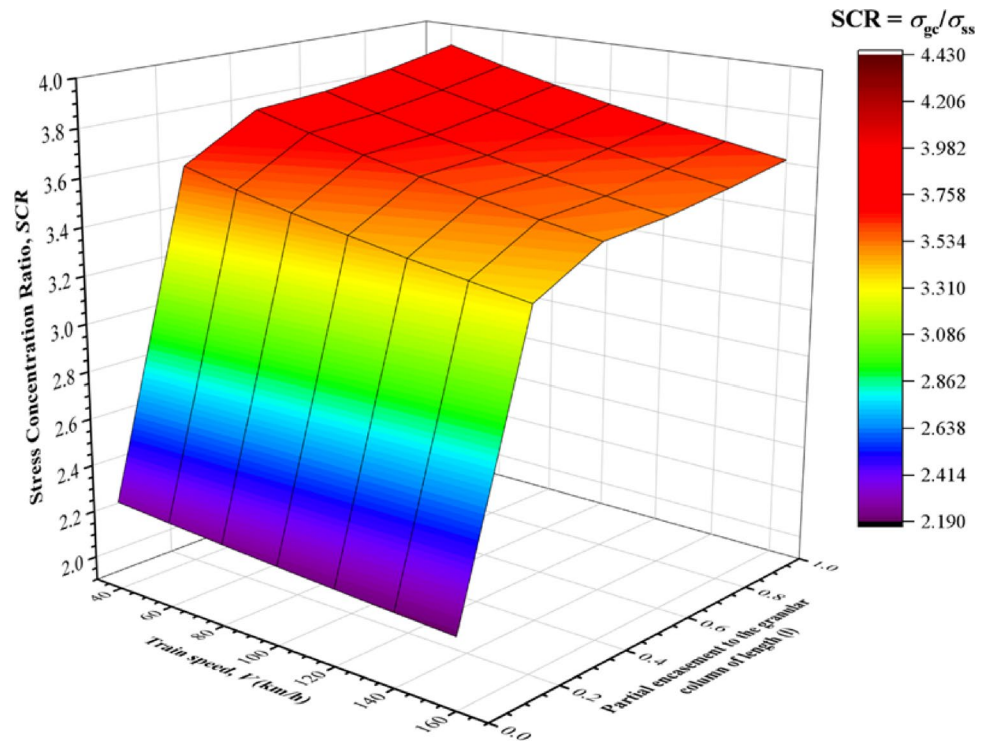


Fig. 19 Variation of stress concentration ratio at the base of the embankment subjected to a 35 T axle load under varying train speeds



dynamic loading approach is widely adopted in engineering practice for predicting embankment settlement and subgrade vertical stresses under moving train loads, due to its practical applicability and computational efficiency, as discussed in the preceding section. This method is straightforward and easy to implement; however, it tends to be overly

conservative in ensuring the safe design of embankments, resulting in increased construction costs. At the same time, FE simulations of realistic moving train loads on embankments are time-consuming and costly. However, these simulations provide a more realistic response of the embankments constructed over a soft soil deposit. This study employs the

Table 5 Comparison of embankment settlement and stress concentration ratios under various train speeds for different lengths of encasements to the granular column

Partial encasement	Train speed (km/h)	25 T Axle load		35 T Axle load	
		Δ_{avg}/H_e	SCR	Δ_{avg}/H_e	SCR
Ordinary granular column	40	0.0097	2.4146	0.0128	2.2763
	60	0.0101	2.3967	0.0132	2.2593
	80	0.0104	2.3813	0.0136	2.2434
	100	0.0107	2.3674	0.0141	2.2288
	120	0.011	2.3528	0.0144	2.2157
	140	0.0113	2.3399	0.0148	2.2035
	160	0.0116	2.3276	0.0152	2.1927
20% of the encasement length	40	0.0083	4.0802	0.0107	3.6047
	60	0.0086	4.0165	0.0112	3.5491
	80	0.0089	3.9617	0.0116	3.5003
	100	0.0091	3.9135	0.0119	3.4569
	120	0.0094	3.8633	0.0122	3.4161
	140	0.0096	3.8107	0.0125	3.3791
	160	0.0098	3.7704	0.0128	3.3453
40% of the encasement length	40	0.0106	4.2804	0.0139	3.7787
	60	0.011	4.2117	0.0144	3.7184
	80	0.0113	4.1529	0.0149	3.6658
	100	0.0117	4.1006	0.0153	3.6186
	120	0.012	4.0472	0.0157	3.5757
	140	0.0123	4.0014	0.0161	3.5356
	160	0.0126	3.9576	0.0165	3.4982
60% of the encasement length	40	0.0124	4.3041	0.0163	3.8014
	60	0.0128	4.2354	0.0169	3.7411
	80	0.0132	4.1769	0.0175	3.6881
	100	0.0136	4.1246	0.0181	3.6405
	120	0.014	4.0716	0.0185	3.5977
	140	0.0144	4.0257	0.0191	3.5574
	160	0.0148	3.976	0.0194	3.5203
80% of the encasement length	40	0.0139	4.3606	0.0184	3.8466
	60	0.0144	4.2891	0.0191	3.7866
	80	0.0149	4.2294	0.0197	3.7337
	100	0.0153	4.176	0.0203	3.6857
	120	0.0158	4.1213	0.0209	3.6442
	140	0.0162	4.0749	0.0214	3.6004
	160	0.0166	4.0293	0.0219	3.5626
Fully encased granular column	40	0.0146	4.4274	0.0194	3.9055
	60	0.0151	4.3561	0.0201	3.8422
	80	0.0156	4.3273	0.0207	3.7878
	100	0.016	4.2404	0.0213	3.7382
	120	0.0165	4.1845	0.0219	3.6926
	140	0.0169	4.1367	0.0225	3.6515
	160	0.0174	4.0907	0.0231	3.6122

Note: Δ_{avg} – Average settlement predicted at top of granular column and surrounding soil;
 H_e – Height of the embankment;
SCR – Stress concentration ratio

Alfa Pendular train configuration and loading to simulate the dynamic response of a railway embankment constructed over granular column-reinforced soft ground at the NTF site, Ballina. The selection of this train configuration and loading is based on its relevance and extensive use in recent finite element studies [48, 49]. Notably, Paixão et al. [50] conducted full-scale field tests at the embankment-bridge transition zone in the Portuguese South Main Line (global coordinates: 38°23.7780 N, 8°35.6700 W) with the passage of the Alfa Pendular train, where the dynamic wheel loads

have been measured at the site through field instrumentation. Hence, the present study utilises the same train configuration, including axle loads and spacing characteristics, to generate the time–stress history at varying operating speeds ranging from 40 to 160 km/h. Figure 20 represents the Alfa Pendular train configuration and the corresponding time histories of vertical stresses induced at the top of the embankment surface. The damping properties of the embankment fill, gravel bed, and granular column reinforced soil layers in the FE model are characterised using Rayleigh damping

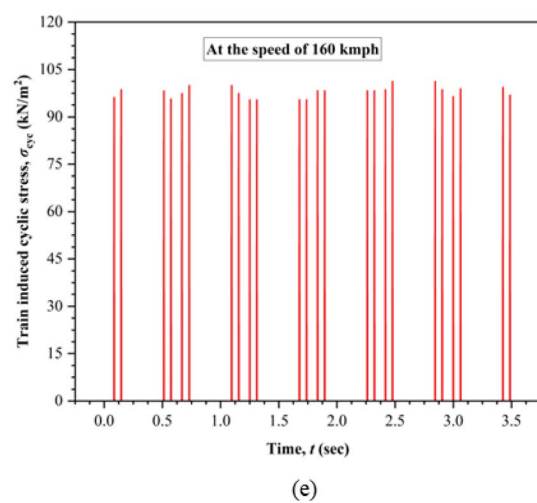
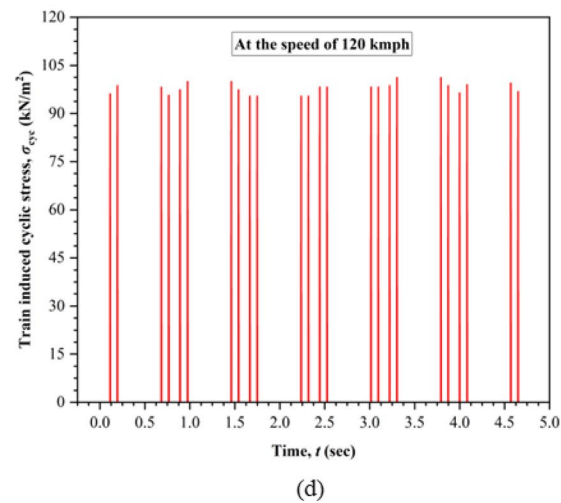
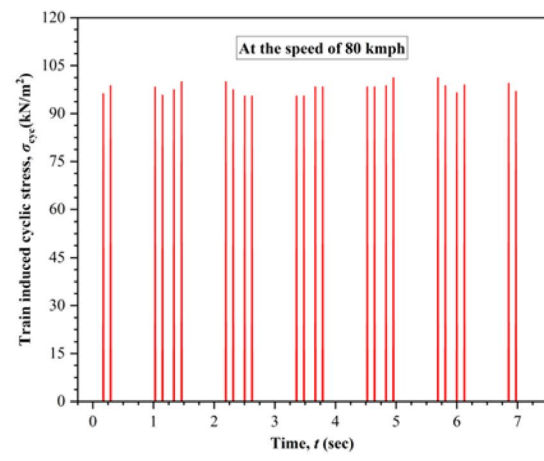
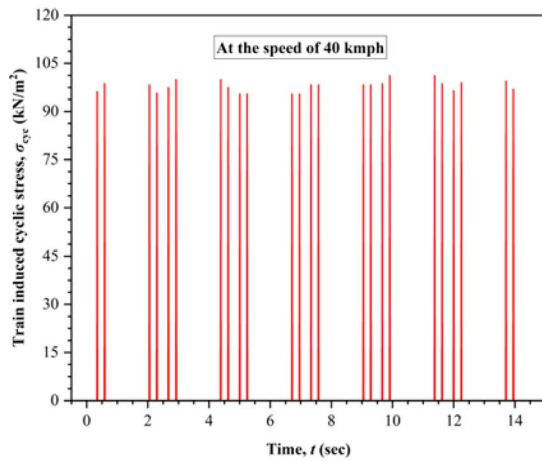
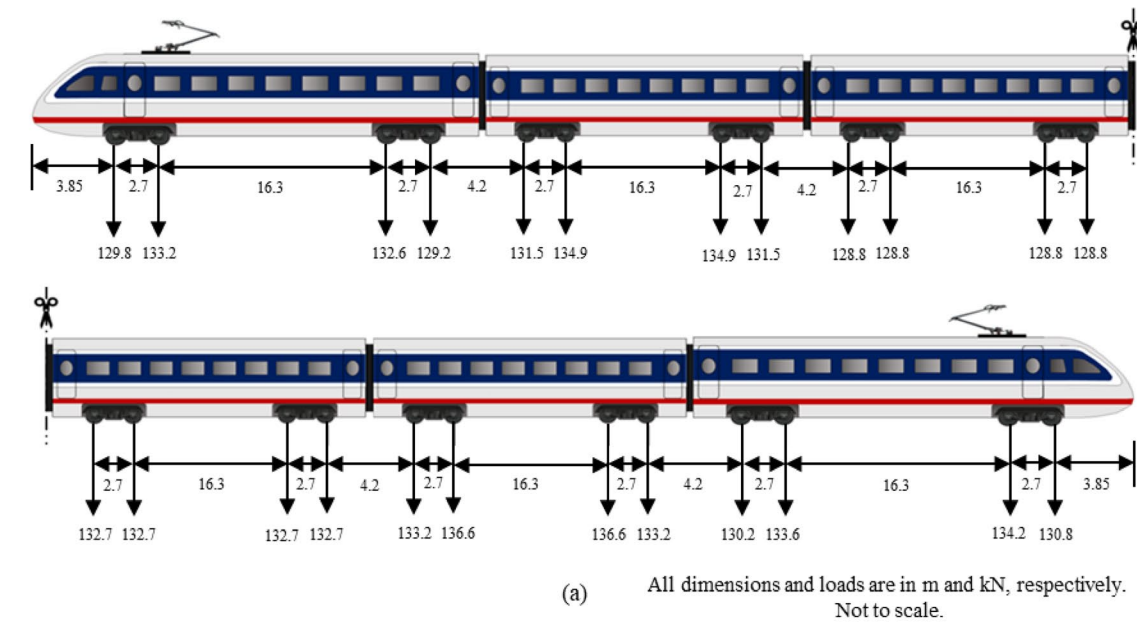


Fig. 20 (a) Configuration of the Alfa Pendular train with axle loads and spacing, and (b-e) Train-induced stress - time history corresponds to speeds of 40, 80, 120, and 160 km/h, respectively

Table 6 Summary of the damping ratios and Rayleigh damping coefficients used in the FE simulations

Soil layer	Damp- ing ratio (%)	Mass-pro- portional (α) coefficient	Stiffness- proportional (β) coefficient
Embankment fill	5	2.513	0.000637
Gravel bed	3	1.508	0.000382
Alluvial clayey sandy silt deposits	6	3.016	0.000764
Estuarine clay	7	3.519	0.000891
Transition sandy clay to clayey sand	4	2.011	0.000509
Fine sand	4.5	2.262	0.000573
Pleistocene clay	5.5	2.765	0.0007
Granular column	2.5	1.257	0.000318

coefficients, defined by the mass-proportional (α) and stiffness-proportional (β) coefficients, as [51].

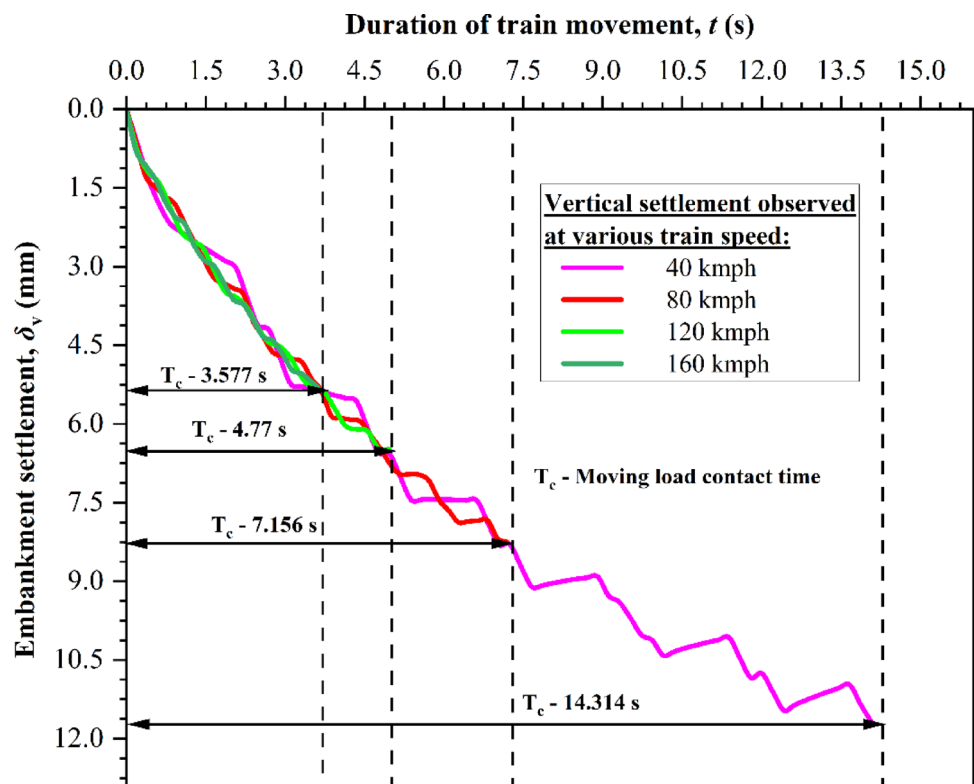
$$\alpha = \frac{2D_R f_1 f_2}{f_1 + f_2} \quad (6)$$

$$\beta = \frac{2D_R}{f_1 + f_2} \quad (7)$$

where D_R is the damping ratio, and f_1 and f_2 are the two selected natural frequencies (Hz). A train speed range of 40 to 160 km/h is considered in this study, along with an axle spacing of 2.7 m. The frequencies f_1 and f_2 are adopted

as 5 Hz and 20 Hz, respectively. The damping ratios for the soil layers are estimated using the shear wave velocity profile obtained from the seismic dilatometer and bender element tests conducted by [27], in combination with the empirical relationship between shear modulus and damping ratio given by [52]. In the 2D axisymmetric dynamic FE analysis, the left boundary is set as symmetric, and the top of the FE domain is where the train-induced loadings are applied. Absorbent (viscous) boundary conditions are applied at the bottom and right boundaries of the problem domain (Fig. 5) to prevent wave reflections. The damping ratios and the Rayleigh damping coefficients used in the FE simulations are summarised in Table 6.

The average vertical settlement at the base of the embankment during the train passage at the speeds of 40, 80, 120, and 160 km/h is shown in Fig. 21. The settlements at the end of the train passages for these speeds are 11.68 mm, 8.25 mm, 6.48 mm, and 5.37 mm, respectively. It is noted that there is an apparent reduction in the settlement with increasing train speed. This trend can be attributed to the following reasons: (a) Reduced contact time – the duration of load application at the embankment surface decreases significantly with increasing train speed. This reduced contact time results in a shorter period for stress transfer from the axle to the substructure. At lower speeds, the applied load remains on a given section of the embankment for a longer period, allowing more stresses to be fully transmitted into the soft subsoil, thereby causing higher settlement.

Fig. 21 Vertical settlement at the base of the embankment during moving train load conditions for the ordinary granular column reinforced soft soil

Conversely, at higher speeds, the loading duration is too short for the soil to fully respond to the applied stress, resulting in reduced deformation. (b) Inadequate time for pore water pressure development and dissipation in soft saturated soils – the vertical settlement of the embankment under loading is influenced not only by the immediate elastic compression but also by the development of excess pore water pressure, which contributes to the time-dependent consolidation settlement. The prolonged duration at lower speeds allows for more dissipation of excess pore water pressure during loading conditions, resulting in higher settlement. At higher speeds, the short duration limits both the generation and dissipation of pore water pressure. This results in an undrained response, and the soil behaves more like a stiffer material under rapid loading conditions, thereby reducing the overall settlement.

Figures 22 and 23 depict the variation of effective vertical stress at the top of the granular column and the surrounding soft soil, respectively, during the passage of the train at speeds of 40, 80, 120, and 160 km/h. The effective vertical stresses predicted at the top of the ordinary granular column at the end of the train passage for these speeds are 218.89 kPa, 218.52 kPa, 217.64 kPa, and 216.33 kPa, respectively. The corresponding values on the surrounding soft soil are 61.957 kPa, 61.794 kPa, 61.72 kPa, and 61.49 kPa. These results indicate that variations in train speed and the associated changes in contact time have a marginal influence on the mobilisation of vertical stress within the ordinary granular column and the surrounding soil. This

is primarily because the granular column dissipates excess pore water pressure rapidly due to its higher permeability and stiffness, thereby maintaining relatively stable, adequate stress levels under dynamic loading conditions. Furthermore, the quick application and removal of train loads at higher speeds reduces the duration of stress redistribution. At the same time, lower train speeds and longer contact time allow pore pressure dissipation and stress transfer, leading to slightly higher effective stress values. The presence of stress peaks at lower speeds can be attributed to within the extended contact period, which promotes greater consolidation in undrained response during the loading time frame. Despite these localised increases of effective vertical stresses, the overall trend suggests that the granular column system exhibits a relatively consistent stress response across the range of speeds considered in this study.

Limitations

The following are the limitations of this study, which can be addressed in the future:

- An axisymmetric unit cell model was adopted in this study to simulate the behaviour of a single granular column and its tributary area. This study provides valuable insights into settlement, stress concentration, and temporal changes in pore water pressure. This approach is

Fig. 22 Variation of effective vertical stress at the base of the embankment on the top of the ordinary granular column during moving train load conditions

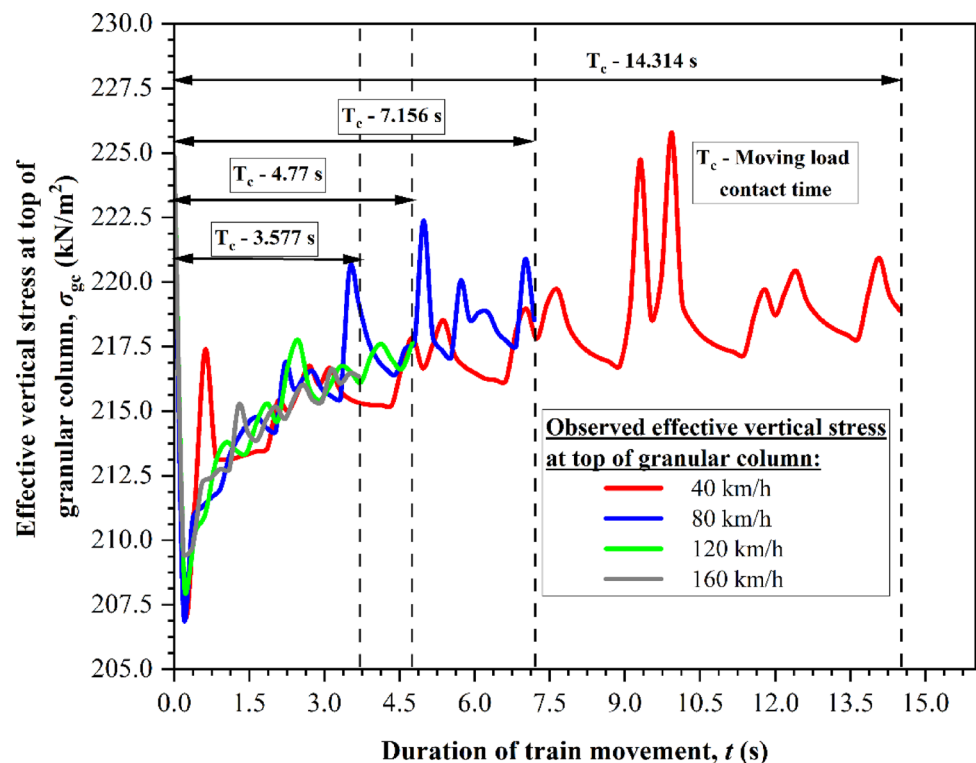
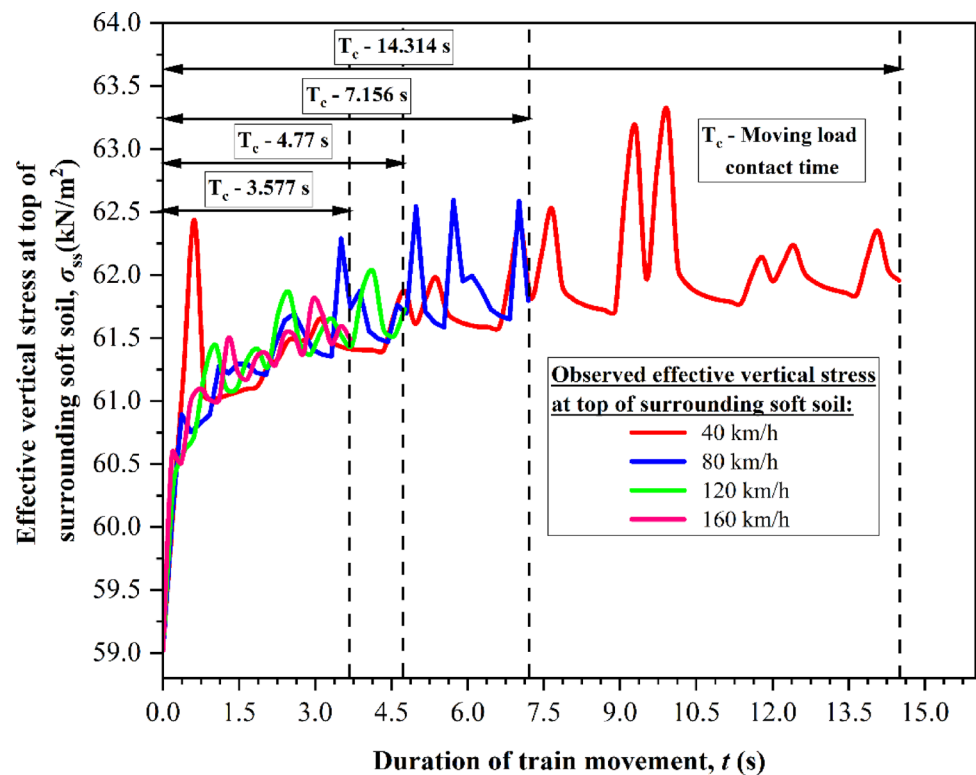


Fig. 23 Variation of effective vertical stress at the base of the embankment on the top of the surrounding soft soil during moving train load conditions



well-suited for modelling interior columns; however, it does not fully capture edge effects or the complex interactions among multiple columns in actual embankments. Therefore, the findings of this study are primarily applicable to interior column conditions, and further investigation is required to evaluate the response of full scale embankments considering lateral boundary and group interaction effects.

- The statistical variation of material properties along the depth of the estuarine clay has not been considered in this study; instead, the representative soft soil model parameters have been used in the present FE analyses and are adopted from the previous studies [26, 30].
- In this study, the Mohr-Coulomb (MC) soil model, which is a linearly elastic-perfectly plastic flow rule, is used for modelling the behaviour of soil layers other than the estuarine clay and the Pleistocene clay. Although this simplification is due to the limited availability of parameters, the use of the Mohr-Coulomb model provides a practical and useful reference for practitioners.
- Further investigations into the time-dependent, fully consolidated state would be necessary for a complete long-term performance prediction.
- The scope of the present study is limited to the analysis of the performance of an embankment resting on soft ground reinforced with partially encased granular columns, and a detailed cost-benefit analysis for specific sections has not been performed as part of the current

investigation. However, this aspect is more important for practicing engineers, and it can be addressed in future studies.

Conclusions

This paper presents a numerical investigation based on FE analysis to evaluate the influence of partial encasement of granular columns on improving the performance of a railway embankment constructed over a layered soft soil profile in Ballina, New South Wales, Australia. Based on the simulation results, the following conclusions are drawn:

- A fully coupled flow deformation analysis is adopted in the present FE simulations to model the sequential construction of the embankment. This approach offers two key benefits: it accounts for consolidation and irrecoverable settlements and accurately captures the dissipation of excess pore water pressure during the construction stage. These findings and know-how are critical in determining the appropriate construction schedule for railway embankments on soft ground reinforced with granular columns.
- Increasing the geotextile encasement length significantly reduces the embankment settlement during the

construction stage and the corresponding rest periods due to an increase in confinement to the granular column, resulting in a reduction of lateral deformations. However, longer encasements exhibited slightly increased settlements during the operational loading stage due to the limited consolidation achieved during sequential construction, which restricts lateral deformation of the column and causes more deformation under train loading. Notably, the granular column with 20% encasement length exhibited lower settlements than the ordinary granular column, while the encasement lengths of 40 to 100% resulted in higher settlements under train loading.

- Encasement lengths between 20 to 40% significantly enhance the column stiffness, enabling them to better support embankment loads by providing the necessary confinement in the bulging zone. However, longer encasement lengths, approximately 60%, are required to effectively control vertical settlements, providing better confinement and load transfer during staged construction. Under train loading condition, the stress concentration ratio (SCR) decreases with increasing train speed. Notably, a 20% encasement length significantly increases the SCR compared to the ordinary granular column, while further increases in encasement length result in only minor improvements. An encasement length of 60% is found to be optimal during the construction stage, and it continues to perform effectively under train loading conditions; however, fully encased columns provide superior performance, particularly during train loading stages.
- The partial encasement effectively dissipates excess pore water pressure during staged construction. Encasement lengths exceeding 60% improve the lateral stability of the columns due to the increased confinement provided by the surrounding soft soil, and they also offer better drainage, thereby enhancing the consolidation and overall performance of the embankment.
- Higher train speeds reduce the contact time limits, thereby limiting the dissipation of excess pore water pressure, leading to a stiffer soil response and lesser embankment settlement. The effective vertical stresses within the granular column and surrounding soft soil remained relatively stable for different speeds. These results highlight the effectiveness of granular column reinforcement in improving the performance of railway embankments during operational stages.

The present study provides a general understanding of the influence of deep soft estuarine clay deposits on the behaviour of a railway embankment. These results are particularly pertinent for regions where deep estuarine clay deposits are

encountered, such as the eastern coastal corridors of Australia. The findings from this study are applicable to other soft soil profiles globally, provided that appropriate site-specific investigation data and relevant modelling techniques are used.

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Author contributions A.R.L.K wrote the main manuscript text and A.R.L.K prepared figures and tables. S.N and G.R.D. edited the manuscript. All authors reviewed the manuscript.

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Data Availability No datasets were generated or analysed during the current study.

Declarations

Conflict of interest The authors declare that they have no known competing financial interests or personal relationships that could have appeared to influence the study reported in this paper.

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