Research Article

New Structure for Strengthening Soil Embankments

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A new prestressed reinforcement device (PRD) consisting of two lateral pressure plates (LPPs) and a reinforcement bar is developed to strengthen soil embankments by improving the soil confining pressure and providing lateral constraint on embankment slopes. The reinforcement effects of PRDs were demonstrated by investigating the beneficial effects of increasing confining pressure on the soil behavior via the performance of a series of large-scale static and cyclic triaxial tests on a coarse-grained embankment soil. The results show that PRDs can effectively improve the soil shear strength, bearing capacity, ability to resist elastic and plastic deformation, critical dynamic stress, and dynamic shear modulus, and empirical methods were also developed to determine the critical dynamic stress and initial dynamic shear modulus of the embankment soil. Moreover, 3D finite element analyses (FEAs) with an LPP width of 1.2 m were performed to analyze the additional stress field in a prestressed heavy-haul railway embankment. The FEAs showed that the additional stress at a given external distance from the border of an LPP first increased to a maximum value and then gradually decreased with increasing depth; the additional stress was transferred to the zones where the subgrade tends to have higher stresses with peak stress diffusion angles of 34° (slope direction) and 27° (longitudinal direction); and a continuous effective reinforcement zone with a minimum additional stress coefficient of approximately 0.2 was likely to form at the diffusion surface of the train loads, provided that the net spacing of the LPPs was 0.7 m. The reinforcement zone above the diffusion surface of the train loads can act as a protective layer for the zones that tend to have higher stresses. Finally, the advantages and application prospects of PRDs are discussed in detail. The newly developed PRDs may provide a cost-effective alternative for strengthening soil embankments.

1. Introduction

The subgrade beneath a track structure must be constructed with high quality to guarantee the safety, stability, and service quality of the railway [1–4]. Generally, the degree of compaction, water content, and soil types of filling materials must satisfy strict requirements according to different types of design standards [5–7]. However, a number of embankments are likely to exhibit poor performance after several years of operation in spite of them having been of high quality when they were constructed. A number of researchers have reported that excessive plastic deformation (Figure 1(a)), slope instability (Figure 1(b)), and lateral expansion of embankment shoulders (Figure 1(c)) constitute a large proportion of the embankment problems observed in the field [8–11]. The 2013 Chinese railway inspection report stated that the number of slope instability and shoulder lateral expansion sections were greater than 7000 in the national railway system [12]. Even for high-speed railway embankments with better construction quality, there also have been some problems observed after approximately ten years of operation. For example, excessive differential settlements were observed at bridge approach embankments in the Beijing-Tianjin high-speed railway, excessive embankment settlements resulted in cracks of the formation layer in the Wuhan-Guangzhou high-speed railway, and embankment instability problems occurred in the Yong-Tai-Wen high-speed railway.

Geotextiles and geogrids have been widely employed in embankment construction to reduce subgrade settlement and improve embankment stability [13–17]. However, these geosynthetic materials are generally applied in new-built
embankments layer by layer, are difficult to utilize in existing embankments, and need relatively large deformation or slips along the fabric-soil interfaces to mobilize their reinforcement effects. Other methods, e.g., cement mixing columns, grouting, and removal/replacement [18–20], generally need to block the traffic and hence may cause enormous economic losses. Practically, the aforementioned embankment problems are closely related to inadequate soil confining pressure and a lack of lateral constraint on embankment slopes. To remedy these problems, a new prestressed reinforcement device (PRD) consisting of two lateral pressure plates (LPPs) and a reinforcement bar is developed. The PRDs are able to be employed in both new-built and existing embankments. Their enhancement effects were demonstrated by investigating the beneficial effects of increasing confining pressure on the soil behavior by conducting a series of large-scale static and cyclic triaxial tests on a coarse-grained embankment soil. In addition, 3D finite element analyses (FEAs) were performed to investigate the additional stress field in a prestressed heavy-haul railway embankment, including the manner in which the confining stress would be transferred to the zones where the subgrade tends to have higher stresses and a recommendation on how to decide the required spacing of the LPPs. Finally, the advantages and application prospects of the PRDs are discussed in detail.

2. Prestressed Reinforcement Device

Figure 2 is a schematic diagram of the proposed embankment reinforcement structure. For a new-built embankment, the detailed construction procedures are as follows:

1. Compact filling materials layer by layer; place the first layer of prestressed rebars (PRs) at the design height and make them perpendicular to the railway line.
2. Place the second, third, and nth layer PRs as step (1) until reaching the designed embankment height.
3. Clean up the embankment slopes and the ends of PRs.
4. Place a concrete slab (LPP) at each end of each PR.
5. Connect one end of each PR with the corresponding concrete slab.

Figure 1: Embankment problems: (a) excessive subgrade settlement; (b) embankment slope instability; (c) lateral expansion of the embankment shoulder.
(6) Connect the other end of each PR with the corresponding concrete slab and apply the designed prestressing force on the PR using a prestressing tension device.

PRs and LPPs work together to apply horizontal pressure on the embankment slopes. The structure formed by a PR and the corresponding two LPPs is named a prestressed reinforcement device (PRD), which is expected to improve the confining pressure on embankment soils. An embankment with PRD reinforcement is defined as a prestressed embankment (PE).

Existing embankments can also be strengthened by PRDs. Steps (1) and (2) of the construction methods are different from those for new-built embankments and are presented as follows:

(1) Horizontal holes with diameters of approximately 10 cm are drilled perpendicular to the railway line at the designed positions.

(2) A PR is inserted into each drilled hole from one side of an embankment and is pulled out from the other side of the embankment.

The other steps are the same as those for new-built embankments.

3. Initial Stress Field of an Unimproved Embankment

The initial stress field of an embankment has significant effects on its stability and bearing performance. The residual stresses released by compaction of soil layers are complicated [21]. Generally, the residual stress due to compaction decreases as time elapses, and the stress field of a soil embankment may be evaluated by the gravity force of filling soils. The residual stress in a soil embankment is the first principal stress, and the horizontal stresses in another two directions are the second and third principal stresses. A plane strain finite element (FE) model was utilized to investigate the stress field in a soil embankment.

The embankment analyzed in the FE model has a height of 6.0 m, a top width of 12.3 m, and a slope ratio of 1:1.5 (vertical to horizontal). The soil behavior is simulated using the Mohr–Coulomb model, and the applied soil parameters are listed in Table 1.

![Figure 2: Schematic diagram of a prestressed embankment (1 = embankment surface; 2 = prestressed rebar; 3 = concrete slab; 4 = anchor head; 5 = prestressing force; 6 = embankment slope; 7 = embankment shoulder).](image)

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
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<tbody>
<tr>
<td>$E$ (MPa)</td>
<td>150</td>
</tr>
<tr>
<td>$\mu$</td>
<td>0.3</td>
</tr>
<tr>
<td>$\gamma$ (kN/m$^3$)</td>
<td>18</td>
</tr>
<tr>
<td>$c$ (kPa)</td>
<td>10</td>
</tr>
<tr>
<td>$\phi$ (°)</td>
<td>30</td>
</tr>
</tbody>
</table>

Table 1: Soil parameters applied in FE analysis.

Note: $E$, $\mu$, $\gamma$, $c$, and $\phi$ are the elastic modulus, Poisson’s ratio, unit weight, effective cohesion, and frictional angle of the soil, respectively.

The analyzed results are shown in Figure 3, where $\sigma_z$ is the vertical stress, $\sigma_x$ is the horizontal stress, $z$ is the depth from the embankment top surface, and the horizontal coordinate represents the distance from the embankment centerline. Parameter $A$ is defined as the ratio of $\sigma_x/\gamma z$, which is applied to investigate the relationship between the vertical stress and the gravity force of the embankment soil, and parameter $B$ is defined as the lateral earth pressure coefficient (i.e., $B = \sigma_z/\sigma_z$). Figure 3 shows the stress distribution in a half embankment.

In Figure 3(a), the A lines are similar to the isosceles trapezoid shape of the embankment. The A values are approximately 1.0 within the top width of the embankment, indicating that the vertical stress in a soil embankment is mainly determined by the gravity force of the embankment soil.

The distribution of B values is more complicated. When $z$ is less than 3.0 m, the B values are less than zero, i.e., $\sigma_x$ is a tension stress. Because the tension strength of filling soils is quite small, tiny tension cracks appear near the embankment top and slope surfaces, which provide channels for water infiltration and results in the reduction of the shear resistance and stability of the embankment soil. When $z$ is greater than 3.0 m, the B values are greater than zero within the top width of the embankment but still less than the lateral earth pressure coefficient at rest ($K_0 = \mu/(1-\mu) = 0.43$), implying that the horizontal stress in a soil embankment is not sufficient to maintain a $K_0$ condition. Moreover, the B value is markedly reduced when approaching the embankment slope surface (see the solid pentagrams in Figure 3(b)), which indicates that the embankment slope is in an undesirable stress state. Hence, there may be a requirement to improve the soil confining pressure, especially for the embankment slopes or soil embankments with problems.

4. Effects of PRDs on Embankment Soil Behavior

4.1. Static and Cyclic Triaxial Test Programs. PRDs can improve the soil confining pressure and provide lateral
constraint on the embankment slopes. Here, the PRD enhancement effects were demonstrated by performing a series of large-scale static and cyclic triaxial tests on a coarse-grained soil under different confining pressures. The soil was sampled from the subgrade bed layers of the Shuo-Huang heavy-haul railway embankments located in the Hebei Province of China. The soil consisted of gravel, sand, and fines with a mass ratio of 43.2: 44.3: 12.5. The fine particles were defined as those passing a 0.075 mm sieve, and the course particles with a size greater than 2 mm but less than 60 mm exceeded 50% by mass. Figure 4 presents the particle size ($d$) distribution curve, and Table 2 lists the basic properties of the tested soil.

All of the tested samples were prepared with a diameter of 30 cm, approximately 6 times the maximum particle size ($d_{max} = 50$ mm), and a height of 60 cm. Each specimen was compacted into six layers in a steel mold, and the target dry density was achieved by controlling the thickness and the soil mass applied for each individual layer. The static shear strength properties under different testing conditions were obtained by conducting a series of large-scale static triaxial tests; the measured effective cohesions and internal friction angles are listed in Table 3. In the cyclic triaxial tests, a sine-wave cyclic loading with a frequency of 1 Hz, corresponding to a heavy-haul train speed of approximately 50 km/h in China, was applied to simulate the train load’s induced dynamic stress ($\sigma_d$) in the embankment soil. The detailed dynamic loading program of the soil specimens is illustrated in Figure 5, where $\Delta \sigma_L$ represents the static vertical stress transferred downward from the ballast, sleeper, and rail.

In the large-scale cyclic triaxial tests, the specimens were first saturated by vacuum suction. Then, an isotropic consolidation pressure was applied to the specimens until a negligible rate of volume change was observed. Then, the cyclic dynamic stress was applied with closing of the drainage valve. The detailed undrained cyclic triaxial test programs are listed in Table 4.
4.2. Static Shear Strength and Bearing Capacity. The deviator stress \((\sigma_1 - \sigma_3)\) axil strain \((\varepsilon_1)\) curves of a group of static triaxial tests \((K = 0.95, w = 9.3\%)\) are shown in Figure 6. Other static triaxial test results are similar and are not presented for simplicity. Figure 6 shows that the deviator stress increased with increasing confining pressure, implying that the static shear strength increased as well.

The static shear strength of a compacted soil follows the Mohr–Coulomb failure criterion. An increase in the confining pressure of a soil sample from \(\sigma_3\) to \(\sigma'_3\) results in an increase of the stress circle diameter at failure (Figure 7), and the corresponding vertical stress \((\sigma_1)\) at failure or the bearing capacity of the soil increases. Practically, if the confining stress has an increment of \(\Delta \sigma_3\), \(\sigma_1\) would have an increment formulated as Equation (1). The test results presented in Figure 6 yield \(\sigma_1\) of 167 kPa and 334 kPa corresponding to \(\sigma_3\) of 50 kPa and 100 kPa, respectively, which agree well with the calculated values of 156 kPa and 325 kPa using the following equation:

\[
\Delta \sigma_1 = \Delta \sigma_3 \tan^2 \left( \frac{\pi}{4} + \frac{\varphi}{2} \right).
\]  

If \(\sigma_1\) remains constant, an increase in \(\sigma_3\) leads to a decrease of the stress circle diameter and an increase of the distance from the Mohr–Coulomb failure line (as shown in Figure 8), indicating that the safety factor (SF) against shear failure increases. SF can be estimated as the ratio of shear strength \((\tau)\) to current shear stress \((\tau)\) at the potential failure surface and can be determined using Equation (2). Providing that \(\sigma_1\) remains constant, SF increases with increasing \(\sigma_3\):

\[
SF = \frac{\tau_f}{\tau} = \frac{(\sigma_1 + \sigma_3) \sin \varphi + 2c \cos \varphi}{\sigma_1 - \sigma_3}.
\]  

4.3. Dynamic Shear Strength. In the large-scale cyclic triaxial tests, a total number of 150,000 loading cycles were applied to the soil specimens unless failure occurred before the cycles were completed. Trial tests before launching the tests in Table 4 revealed that the axial accumulated plastic strain \((\varepsilon_a)\) rate of elastic stable specimens tended to be less than 0.5 mm/hour after a certain number of loading cycles (i.e., 50,000 cycles). If the axial accumulated plastic strain \((\varepsilon_a)\) rapidly accumulates to 15%, failure was considered to occur, in conjunction with the settlement limit of heavy-haul railway embankments. The detailed method for classifying the specimen patterns is presented in Table 5.
The effects of confining pressure on $\varepsilon_a$ are presented in Figure 9. It is observed that the specimen subjected to a combination of $\sigma_d = 125 \text{kPa}$ and $\sigma_3 = 15 \text{kPa}$ or $30 \text{kPa}$ failed, whereas the specimens reached an elastic stable state after approximately 2000 load cycles when $\sigma_3$ increased to 60 kPa. Andersen and Lauritzsen [22] defined the dynamic shear strength as the dynamic stress under which the soil specimen failed after a given number of loading cycles ($N_f$). Figure 10 presents the $N_f$ values of specimens tested under different confining pressures. It shows that the dynamic shear strength increases with increasing confining pressure at a given $N_f$ value. A less dynamic shear strength was obtained when $N_f$ increases providing that other test conditions are unchanged.

### 4.4 Accumulative Axial Strain and Critical Dynamic Stress

Figure 9 presents the accumulative axial plastic strain-cycle number ($N$) curves of a group of tests under different confining pressures ($\sigma_d = 125 \text{kPa}$).

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### Table 5: Method for classifying specimen patterns.

<table>
<thead>
<tr>
<th>Pattern</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>Stable</td>
<td>$\varepsilon_a$ increases slowly and does not exceed 4% after 50,000 cycles</td>
</tr>
<tr>
<td>Critical</td>
<td>$\varepsilon_a$ increases quickly or slowly and but can reach 15% after 20,000 cycles</td>
</tr>
<tr>
<td>Failed</td>
<td>$\varepsilon_a$ increases quickly and can reach 15% before 20,000 cycles</td>
</tr>
</tbody>
</table>

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### 4.4. Accumulative Axial Strain and Critical Dynamic Stress

Figure 9 presents the accumulative axial plastic strain-cycle number ($N$) curves of a group of tests under different confining pressures. It shows that increasing the confining pressure effectively slows down the development rate of permanent axial deformation. Figure 11 shows the $\varepsilon_a-N$ curves of the tested specimens. It is observed that (1) the specimens tested under the combined conditions of $\sigma_3$ of 15 kPa or 30 kPa and $\sigma_d$ of less than or equal to 100 kPa were stable; (2) the specimens experienced failure when $\sigma_d$ was greater than or equal to 150 kPa; and (3) when $\sigma_3$ was equal to 125 kPa, the axial accumulative plastic deformation continuously increased with an increasing cycle number (critical pattern), except for the specimen tested under $\sigma_3$ of 60 kPa, which was stable. Hence, a greater confining pressure could potentially reduce the development rate of accumulative axial plastic strain.
Liu and Xiao [23] presented that there is a critical dynamic stress ($\sigma_c$) below which the cumulative axial plastic strain gradually ceases and the soil specimens remain in a stable state with increasing loading cycles. In this study, the imposed dynamic stress ratio is defined as $$DSR = \frac{\sigma_d}{2\sigma_3}.$$ (3)

Figure 12 presents the distribution of DSR-$\sigma_3$ data points of the tested specimens, where their failure patterns are discriminated with different symbols. It is found that the stable specimens are mainly located in the lower-left part of the figure, whereas the specimens that experienced failure are mainly located in the top-right region. As expected, the data points corresponding to the critical-pattern specimens are located within the transition zone between the stable part and the failure part, which is defined as a critical zone in this study. Specimens located below this critical zone were stable, and the cumulative axial plastic strain gradually ceases with increasing loading cycles, while the specimens located above this zone experienced failure. Based on the test results, the DSR-$\sigma_3$ relationship of the midline of this critical zone can be formulated as $$DSR = a \cdot (\sigma_3)^b,$$ (4)

where $a$ and $b$ are constants. For the soil tested, $a$ and $b$ are equal to 41.1 and −0.88, respectively.
Dynamic stress ratio (DSR)

Initial dynamic shear modulus (\(G_0\)) (MPa)

10
15
20
25
30
35
40

0 2 4 6 8 10

20 30 40 50 60

DSR = 41.1·(\(\sigma_3\))–0.88

\[ \sigma_c = 82.2 \cdot (\sigma_3)^{0.12}. \] (5)

The \(\sigma_c - \sigma_3\) relation of the tested coarse-grained soil is plotted in Figure 12. It is shown that \(\sigma_c\) increases with increasing \(\sigma_3\).

4.5. Initial Dynamic Shear Modulus. The cyclic axial stress-strain curves of the tested samples formed a number of stress-strain loops, and the elastic stress-strain relationship was not very clear in the first few loading cycles. Therefore, the initial dynamic shear modulus \((G_0)\) in this study was determined corresponding to the fifth loading cycle.

The relationship between \(G_0\) and confining pressure obtained from the tested specimens is presented in Figure 13. It shows that \(G_0\) increases with increasing confining pressure and cyclic dynamic stress \((\sigma_d < 0.25 \text{MPa})\). Combining the effects of the confining pressure and cyclic dynamic stress on \(G_0\) an empirical formula is developed as follows:

\[ G_0 = m\sigma_3 + n\sigma_d + c, \] (6)

where \(m\), \(n\), and \(c\) are three constants; and the unit of \(c\) is MPa. For the soil tested, \(m\), \(n\), and \(c\) are equal to 142.6, 78.0, and 9.4 MPa, respectively. When using Equation (6), the unit of \(\sigma_3\) and \(\sigma_d\) should be MPa, and as a consequence, the unit of the estimated \(G_0\) is MPa. The range of \(\sigma_d\) in Equation (6) is 0.05–0.25 MPa. This range is expected to cover the cyclic dynamic stress in embankment soils in China according to field measurements; however, caution is still needed when extending Equation (6) to other test conditions that are different from those applied in this study.

Generally, the proposed PRDs can improve the confining pressure and hence is able to effectively improve the shear strength, bearing capacity, ability to resist elastic and plastic soil deformation, critical dynamic stress, and dynamic shear modulus of the embankment soil.

5. Additional Stress in a Prestressed Embankment

5.1. Finite Element Model. 3D finite element analyses (FEAs) using the ABAQUS software application (version 6.14) were performed to analyze the additional stress field in a prestressed embankment induced by a lateral pressure plate. The analyzed embankment had a height of 10.0 m, a top width of 10.0 m, a slope ratio of 1:1, and a length of 15 m. The bottom boundary was 20 m below the embankment top surface, while the left boundary was 10 m away from the embankment toe (Figure 14). The lateral pressure plate had a plan dimension of 1.2 m × 1.2 m. The soil was assumed as a weightless linear elastic medium with an elastic modulus of 150 MPa and a Poisson’s ratio of 0.27. It should be mentioned that the absolute values of the soil parameters have almost no effect on the analyzed additional stress if the embankment does not have large deformation under the applied prestress. The horizontal displacement of the left, right, front, and back boundaries was fixed, but vertical movement was allowed. At the bottom, both the vertical and horizontal displacements were fixed. In the finite element model, the lateral pressure plates and the reinforcement bar were not constructed, and the prestress applied on the embankment slope was simplified as a horizontal uniform pressure \((p = 100 \text{ kPa})\). Due to the symmetry, only half of the embankment was analyzed. The detailed mesh of the entire model is shown in Figure 14.

5.2. Additional Stress Directly beneath the Bottom of the Lateral Pressure Plate. The analysis points directly under the plate bottom were defined using characters (A–E) along the \(x\) direction and numbers (1–3) along the \(y\) direction, as shown in Figure 15(a). The additional stresses at different depths directly below the analysis points are presented in Figure 16. It
shows that the additional stress directly underneath the lower border (points A1, A2, and A3) of the loading area is smallest and that the maximum \( \sigma_z \) appears in the upper part (points D1, D2, and D3) of the loading area rather than at its centerline. The main reason is that the pressure component along the slope surface induces a compression stress in the upper part of the loading area but results in tension stress in the lower part. Moreover, except under the lower border, \( \sigma_z \) monotonically decreases as the depth increases (see lines B1-E1, B2-E2, and B3-E3 in Figure 17).

5.3. Additional Stress in Regions outside the Lateral Pressure Plate. The external regions A and B directly outside the centerlines of the loading area (Figure 15(b)) were used to investigate the effective diffusion zone of the additional stress. Figures 17(a) and 17(b) present the additional stress coefficient \( (K_z = \frac{\sigma_z}{p}) \) versus depth \( (h) \) curves at different external distances in external regions A and B (Figure 15(b)), respectively. It is observed that the maximum additional stress in the external region A is approximately 30 kPa, corresponding to a \( K_z \) value of 0.3, while the maximum additional stress in the external region B is approximately 17 kPa \( (K_z = 0.17) \), which is only 57% of that in region A. Moreover, the curves show that the additional stress first increases to a maximum value and then gradually decreases as the depth increases, and the depth of the peak point increases with the external distance \( (L_x \text{ and } L_y) \), as shown in Figure 15(b).
5.4. Additional Stress Contour and Diffusion Angles of Peak Stress. The analyzed additional stress ($\sigma_z$) contour in the $z$ direction (perpendicular to the embankment slope) is shown in Figure 18. Generally, the additional stress forms a series of stress bubbles and gradually transfers to the zones where the subgrade tends to have higher stresses. It is observed that the diffusion distance in the $z$ direction is greater than those in the $x$ and $y$ directions, indicating that the compression zone is mainly located below the bottom of the lateral pressure plate. In addition, the additional stress diffusion range beyond the upper border of the loading area is much greater than that beyond the lower border (Figure 18). The main reason is that the applied horizontal pressure consists of two components, i.e., a pressure component normal to the slope surface and a pressure component along the slope surface, the force of which readily propagates in the region beyond the upper border.

The stress contour lines in the external regions A and B are presented in Figures 19(a) and 19(b), respectively. In region A, the contour lines are asymmetric to the centerline of the plate because the pressure component along the slope surface induces compression stress in the regions beyond the upper border of the plate but results in tension stress in the regions beyond the lower border of the plate. In the $z$ direction, plotting a tangent line to each of the contour lines, the diffusion angles of peak stress ($\alpha$ and $\beta$) in external regions A and B can be evaluated as the intersection angle between the tangent line and the line connecting the tangent points.
The position coordinates of the tangent points (\((L_x, z_m)\) and \((L_y, z_m)\), where \(z_m\) is the depth where the peak \(\sigma_z\) occurred at a given external distance in regions A and B) in Figures 19(a) and 19(b) are extracted, and the \(L_x\) and \(L_y\) versus \(z_m\) relationships are presented in Figures 20(a) and 20(b), respectively. \(L_x\) and \(L_y\) increase in an approximately linear manner with increasing \(z_m\), implying that the prestress gradually propagates outward as the depth increases. The angles between the regression lines and the horizontal line in Figures 20(a) and 20(b) are approximately 34° and 27°, respectively. Consequently, the additional stress is transferred to the zones where the subgrade tends to have higher stresses (e.g., the zones directly below the rails and sleepers) with peak stress diffusion angles of 34° and 27° in external regions A and B, respectively.

5.5. Layout Spacing of the Lateral Pressure Plates. The layout spacing of the lateral pressure plates is a key issue in the design of a prestressed embankment. The lateral pressure plates diffuse the prestress to the interior of the embankment, and the additional stress subjected to multiple LPPs is superimposed, which can effectively increase the reinforcement region of the PRDs. As shown in Figure 21, the width of the LPP is 1.2 m, and the net spacing of the plates along the slope and longitudinal directions are \(a\) and \(b\), respectively. Preliminary analyses showed that the additional stress in the weak region without the action of an LPP (Figure 21) is mainly provided by the adjacent four LPPs, and the contributions of other LPPs away from this region are negligible.

Due to the lateral constraints and additional confining pressure provided by the PRDs, a PE is able to be constructed with a greater slope ratio compared with unreinforced embankments. The slope ratio of a PE is adjusted from 1:1.5 for traditional soil embankments to 1:1. A typical embankment cross section is shown in Figure 22. For the example of a heavy-haul railway embankment, generally, the length of sleepers is approximately 2.5 m; the thickness of the ballast layer is 0.3 to 0.55 m; the slope ratio of the ballast layer is approximately 1:1.75; and the embankment top width of a single heavy-haul railway line is 8.1 to 8.5 m [5, 6]. The loads subjected to the moving trains are transferred to the interior of the embankment at an angle of 45° via the end of the sleepers [5], yielding a distance of approximately 1.75 m from the train load’s diffusion surface to the slope surface, as illustrated in Figure 22. In this study, a \(K_z\) value of 0.05 is adopted as the lower limit to evaluate the effective diffusion range in the external regions of an LPP. Under the action of multiple plates, the total additional stress is superimposed from the adjacent four plates. Hence, a \(K_z\) value of 0.2 at the boundary of the zones that tend to have higher stress.
(i.e., the diffusion surface of the train loads, approximate to a depth of 1.75 m, as illustrated in Figure 22) is adopted as the lower control limit to design the plate spacing.

A series of FEAs were performed to investigate the relationship between the spacing parameters $a$ and $b$. In total, 84 cases were analyzed, as listed in Table 6; Figure 23 presents the mesh of a case with $a$ and $b$ values of 0.7 m. The analyses show that the weakest reinforcement point at different depths is located around the projection ($z$ direction) of the center point C (Figure 21) of the weak region.

The analyzed $K_z$ values at the projection of 1.75 m directly below the center point C are plotted against $b$ for different $a$ values in Figure 24. It is observed that $K_z$ decreases with increasing $a$ and $b$. The relationship between
a and b under the condition of maintaining a $K_z$ value of 0.2 can be obtained by analyzing the coordinates of the intersection points of the $K_z$ versus $b$ curves and the line $K_z = 0.2$. The analyzed $a$ versus $b$ curve is presented in Figure 25. It clearly shows that there is a negative correlation between $a$ and $b$. In addition, the values of $a$ and $b$ can be selected as 0.7 m to maintain a uniform plate layout in the slope (x) and longitudinal (y) directions, as illustrated in Figure 25. Figure 26 presents that a continuous and effective reinforcement zone with a minimum $K_z$ value of approximately 0.2 is formed at the depth of 1.75 m directly below the weak region (Figure 21), provided that the net spacing of the plates is 0.7 m. The reinforcement zone above the diffusion surface of the train loads (depth of 1.75 m) can act as a protective layer of the zones that tend to have higher stresses.

6. Advantages and Application

Prospects of PRDs

6.1. Advantages. Compared with traditional soil embankments or geotextile and geogrid reinforced embankments, PRD-reinforced embankments have the following advantages:

(1) PRDs can improve the confining pressure of the embankment soil. Increasing the confining pressure can effectively improve the shear strength, bearing capacity, ability to resist elastic and plastic soil deformation, critical dynamic stress, and dynamic shear modulus of the embankment soil.

(2) A PRD-reinforced embankment can be designed with a greater slope ratio, which would reduce the use of filling materials and construction site area.

(3) PRDs can be utilized in both new-built and existing embankments and are able to be constructed during the normal operation of a railway line.

(4) PRDs can be prefabricated in a factory, and their field assembling procedures are simple and time-saving, which is helpful for reducing construction time and engineering costs.
6.2. Application Prospect. High speeds and heavy freights are the main developing trends of railway engineering. High-speed railways have strict requirements regarding track smoothness and total and differential settlements of track structures, and hence, they require the embankments to be constructed with high strength, stiffness, and stability. For heavy-haul railways, to increase the freight transport capacity, the axle load and carriage number of the trains have become greater and greater, leading to greater dynamic loading and vibration on embankment soils and resulting in serious embankment problems. PRDs provide an effective alternative to enhance soil embankments by improving confining pressure and providing lateral constraint. The most important points are that construction of PRDs does not need to block traffic and can be performed during the normal operation of a railway line and that the engineering costs are estimated to be 20~50% less than those of other reinforcement methods.

7. Conclusions

Inadequate soil confining pressure and lack of lateral constraint on embankment slopes are important factors resulting in frequent deteriorations of an embankment (e.g., excessive plastic deformation, slope instability, and lateral expansion of embankment shoulders). A new structure, prestressed reinforcement device (PRD) consisting of two lateral pressure plates (LPPs) and a reinforcement bar, is developed to strengthen soil embankments via improving confining pressure and providing lateral constraint on embankment slopes.

The reinforcement effects of PRDs were demonstrated by investigating the beneficial effects of increasing confining pressure on the soil behavior by performing a series of static and cyclic large-scale triaxial tests on a coarse-grained embankment soil. The results show that PRDs can effectively slow down the cumulative plastic strain rate and...
improve the shear strength, bearing capacity, critical dynamic stress and dynamic shear modulus of the embankment soil. Meanwhile, simple formulas were also proposed to determine the critical dynamic stress and initial dynamic shear modulus of the soil.

In addition, 3D finite element analyses (FEAs) of a heavy-haul railway embankment with an LPP width of 1.2 m and a slope ratio of 1:1 were conducted to analyze the additional stress field in the prestressed embankment. The FEA results show that the additional stress in the external regions of an LPP first increases to a maximum value and then gradually decreases as the depth increases; the additional stress is transferred to the zones (e.g., the zones directly below the rails and sleepers) where the subgrade tends to have higher stresses with peak stress diffusion angles of 34° and 27° in the external regions along the slope and longitudinal directions, respectively; the net spacing of the LPPs along the slope direction (a) has a negative correlation with that (b) along the longitudinal direction when keeping the additional stress coefficient (Kz) constant; and a continuous and effective reinforcement zone with a net spacing of 0.7 m (i.e., the value of a and b is 0.7 m). The reinforcement zone above the diffusion surface of the train loads can act as a protective layer of the zones that tend to have higher stress.

Data Availability

The data used to support the findings of this study are available from the corresponding author upon request.

Conflicts of Interest

The authors declare that there are no conflicts of interest regarding the publication of this paper.

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