Research Article

Accumulative Plastic Deformation of the Improved Completely Weathered Granite Subgrade in High-Speed Railway

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The improvement and reuse of completely weathered granite (CWG) with poor engineering properties for backfilling embankments can help protect the environment and bring great economic benefits. The embankment of high-speed railways demands an extremely low additional subgrade settlement under long-term dynamic loads. To analyze the applicability of the improved CWG used for subgrade filling, a series of laboratory dynamic triaxial and field cyclic loading tests were carried out. On the basis of the test results, a calculation model of accumulative plastic strain considering the dynamic stress ratio and the cement content was established. Using the proposed model and the numerically calculated dynamic stress, the accumulative plastic deformation of an embankment filled with improved CWG under various driving conditions was calculated and discussed. The variation of accumulative plastic deformation of the improved CWG with loading cycles could be described by a power function. The dynamic stress level had a significant influence on the accumulative plastic strain, and the strain in the bottom layer of the subgrade bed was greatly attenuated. Accordingly, most of the accumulative plastic deformation attenuation occurred within the depth range of the subgrade bed. The final total accumulative plastic deformations of the embankment were less than 2 mm (5 mm for the limit value) after 4 million times of high-speed train loadings, proving that the improved CWG can be used for subgrade filling. The influence of the axle load on accumulative plastic deformation was more significant than that of the train speed. The proposed calculation model of the accumulative plastic strain could provide valuable reference for similar railway engineering.

1. Introduction

High-speed railways (HSRs) have been built and extensively put into service in many countries. Non-ballasted tracks with high stability, durability, and low maintenance have been widely used in HSR construction; they also have an extremely low settlement demand. Ensuring the long-term dynamic stability of the subgrade is one of the challenges in the construction of non-ballasted tracks.

The subgrade settlement includes the compression settlement of the track foundation caused by self-weight and the superstructure (i.e., the post-construction settlement) and the accumulative plastic deformation induced by the long-term action of train loads (i.e., the additional settlement). According to relevant engineering experience, under the normal operation conditions of high-speed trains, the dynamic shear strain of the subgrade mostly belongs to the small shear strain range. Accordingly, some soil structural changes are produced, resulting in the occurrence of accumulative plastic deformation. The continuously accumulating deformation may eventually exceed the allowable value with the increase in the traffic loading times. Thus, the long-term dynamic stability of the railway subgrade requires further evaluation, with the accumulative plastic deformation as an indicator. In China, the Code for Design of Railway Earth Structure (TB 10001–2016) [1] clearly stipulates that the post-construction settlement of the subgrade shall not exceed 15 mm. Furthermore, related engineering experience indicates that the additional settlement of the subgrade shall not exceed 5 mm. Once the settlement of the
subgrade exceeds the limit, the dynamic wheel load might be amplified and the track deterioration might accelerate [2, 3]. Such problems may eventually endanger the safe operation and ride comfort of high-speed trains, also resulting in high maintenance frequency and even railway operation interruption [4, 5].

Completely weathered granite (CWG) with poor engineering properties is widely distributed in the construction site of the Beijing-Guangzhou HSR in China. The amount of CWG from excavation is huge, imposing a heavy burden on the environment. The improvement and reuse of CWG for backfilling embankments can help protect the environment and bring great economic benefits. To determine the applicability of improved CWG as a subgrade filling material for HSRs, one of the important works is to ensure that the accumulative plastic deformation of the subgrade backfilled with improved CWG does not exceed the limit. According to existing research, the mechanistic-empirical method has been proven practical for calculating the accumulative plastic deformation. To ensure the accuracy of the calculation results obtained through this kind of method, a reasonable calculation model of accumulative plastic strain is indispensable.

Over the past decades, many researchers have continuously improved the calculation models of accumulative strain, and their studies mainly focused on clay and other fine-grained soils. The power model proposed by Monismith et al. [6] is widely used to describe the relationship between the accumulative strain and the number of load cycles. Li and Selig [7] introduced the soil static strength parameter into Monismith’s model, which indirectly considers the type and physical state of soil, and analyzed the settlement of soft soil under traffic load. However, the difference between the calculated result and the measured data increased with the number of load cycles. Indeed, the accumulative plastic deformation of the subgrade filler depends on many factors, such as the physical properties of soil, the number of load cycles, and the stress levels [8]. On the basis of Li and Selig’s model, some scholars proposed improved models by further considering the effect of the stress condition and/or other factors. For example, Chai and Miura’s model [9] considers the influence of the initial deviatoric stress.

Several research works focused on the accumulative plastic strain calculation model for coarse-grained soil, which is commonly used as a railway subgrade filler. Most of the results show that power law models also apply to coarse-grained soil. On the basis of their dynamic triaxial test results, Gidel et al. [10] established a power model based on load cycles and the stress level to calculate the accumulative plastic strain. Chen et al. [11] improved the model of Gidel et al. [10] by considering the initial stress state for and material properties of coarse sand. Noolu et al. [12] studied the accumulative deformation behavior of the recycled aggregate used for the bottom layer of a subgrade bed and proved that power law models could fit the experimental data well. Mei et al. [13] studied the influences of confining pressure and stress level on the deformation behavior of coarse-grained soil. Zhang et al. [14] used the power function model, which considers the effect of dynamic and confining stresses, to describe the accumulative plastic strain development of coarse granular aggregates. In addition to power law models, other models have also been documented to describe the accumulative deformation behavior of coarse-grained soil. Ling et al. [15] proposed a calculation model consisting of two simultaneous components, namely, the rapid compaction step and the relatively slow shear deformation. Accounting for the influences of octahedral normal and shear stresses, Puppala et al. [16] and Cai [17] proposed permanent strain model formulations for coarse-grained soil. However, the applicability of these models is limited by the complexity of determining the model parameters.

According to the aforementioned research on accumulative plastic strain models, the stress level is the main variable/influence factor of calculation models. Thus, the dynamic stresses under train loads are critical for obtaining the accumulative plastic deformations of the subgrade. The numerical simulation method with proper calibration and validation through field test data has been widely used to study the dynamic responses of the subgrade [18–20]. Using a 2.5 D finite element model with thin-layer elements, Bian et al. [21] found the main factors affecting the track and ground vibration when the vehicle speed is lower or higher than the critical velocity. On the basis of the three-dimensional vehicle-track-coupled dynamic model developed in [22], the track-subgrade dynamic stresses induced by high-speed trains were calculated to predict the track degradation caused by the differential subgrade settlement [23]. Using a 3D finite element model, Chen and Zhou [24] investigated the influences of train speed and line patterns on the subgrade dynamic responses for a double-line ballastless track-subgrade system. The aforementioned numerical simulation models involve the track and/or the vehicle. When the aim of the research work was track foundation, the modelling method can be appropriately simplified. Meng and Hou Yongfeng [25] established the track foundation model through a 3D finite difference method; the real dynamic stresses of the subgrade top layer of the entire train unit passing process obtained from in situ tests are imposed on the surface of the subgrade in the track foundation model. This modelling method can accurately simulate the real train loads and is convenient for application.

Although many studies on the accumulative plastic strain of coarse-grained soil have been conducted, few studies on the application of improved CWG have been reported. To analyze the accumulative deformation behavior of the HSR subgrade filled with improved CWG material, a series of laboratory and field tests were carried out. In this study, the function form of the accumulative plastic strain calculation model of the improved CWG was obtained preliminarily by fitting the data of the laboratory dynamic triaxial tests. Then, the parameters of the accumulative plastic strain model were modified on the basis of the field cyclic loading test results and numerical calculation, ensuring the reliability of the data. Furthermore, this study established the track foundation model through the FLAC 3D software, and the real dynamic stresses of the subgrade obtained from in situ tests were imposed on the surface of...
the subgrade in the track foundation model. This modelling method can accurately simulate real train loads and is convenient for application when the research work is focused on track foundation. On the basis of the modified calculation model of accumulative plastic strain and the calculated dynamic stresses calculated through the numerical simulation method, the accumulative plastic deformation of the improved CWG embankment under various real driving conditions was calculated and analyzed.

2. Calculation Model of Accumulative Plastic Strain Based on Laboratory Tests

The CWG samples were taken from the DK2102 + 240 to DK2102 + 304 region of the Qingyuan test section of the Beijing-Guangzhou HSR line. The main chemical compositions of the CWG samples with 58% quartz and 7% mica contents are SiO₂, Fe₂O₃, and Al₂O₃. The proportion of particles with a size greater than 10 mm is between 12% and 20%, and the natural moisture content is between 18% and 24%, which were verified and documented in [26]. The CWG with the above characteristics can be defined as silty sand or coarse sand, represented as C-filler or D-filler according to the Code for Design of Railway Earth Structure (TB 10001-2016), and should be treated with cement before being used as a HSR subgrade filler.

According to the Code for Soil Test of Railway Engineering (TB 10102-2004), a series of laboratory dynamic triaxial tests were carried out for the CWG, which was improved by adding 4%, 5%, 6%, and 7% cement by weight. In the tests, the confining pressures of 0, 25, 50, and 100 kPa were separately applied to the soil specimens, and the compacting factors varied from 0.85 to 0.98. For simplicity, only the test results under the following test conditions are displayed in Figure 1: confining pressure of 25 kPa, compacting factor of 0.95, cement content of 4%, and water content of 12%.

The variations of accumulative plastic strain εₚ with load times N under different dynamic stress levels displayed in Figure 1 are representative. The variations can be divided into three categories: (a) when the axial dynamic stress amplitude σₐ was relatively low (σₐ = 350 and 450 kPa), the accumulative plastic strain increased at the initial stage of loading and then tended to be stable; (b) the accumulative plastic strain increased continuously with the dynamic stress amplitude (σₐ = 650 kPa) without reaching a stable value; and (c) when the dynamic stress amplitude exceeded the limit (σₐ = 750 kPa), the accumulative plastic strain grew rapidly, and the soil failed within a relatively short time.

To obtain an applicable calculation model of accumulative plastic strain for the improved CWG, Monismith’s power model [6], Gidel’s model [10], and the semi-logarithmic model [13] were used to fit the relationship between accumulative plastic strain εₚ and cyclic loading times N. The comparison of the results shows that the power function is the most suitable. Therefore, the following equation was used as the basic fitting model in this study:

\[ εₚ = AN^B, \]  

where \( εₚ \) is the accumulative plastic strain of the improved CWG; \( N \) is the number of loading cycles; and parameters \( A \) and \( B \) depend on the type, physical state, and stress condition of soil, which have a significant influence on the accumulative plastic strain. Therefore, determining the appropriate values of these parameters is essential for the applicability of the calculation model. Parameters \( A \) and \( B \) were fitted on the basis of the multiple groups of the triaxial test data, and the results were classified and statistically analyzed according to the cement content, as shown in Table 1.

(1) Parameter \( A \) presented a plastic strain after the first cycle of repeated loading (i.e., \( N = 1 \)). Monismith [6] and Li [7] pointed out that the dynamic deviatoric stress (i.e., \( σₐ = σ₁ - σ₃ \)) had a significant influence on parameter \( A \). Furthermore, the influence of the physical state and type of soil were also non-negligible, so ultimate static strength \( σₛ \) is introduced in parameter \( A \) as the physical index. The ratio of the dynamic deviatoric stress to the static strength is taken as the dynamic stress ratio as shown in the following equation:

\[ f = \frac{σₐ}{σₛ}, \]  

where \( f \) denotes the dynamic stress ratio; \( σₐ \) is the dynamic deviatoric stress; \( σₛ \) is the ultimate static strength; and \( σₐ = 2S_u \), where \( S_u \) is the undrained shear strength, which can be calculated by shear strength parameters \( c \) and \( φ \). For the improved CWG, the parameters could be obtained from the static triaxial test.

The values of parameter \( A \) under different conditions (Table 1) show that the parameter \( A \) of all the improved CWG samples increased by nearly 2 times with the increase in dynamic stress ratio \( f \), within which the soil sample (with a cement content of 7% and a compacting factor of 0.95) increased by 3 times, from 0.112 to 0.332. Therefore, the
variation of parameter $A$ with the dynamic stress ratio is significant. On the basis of the above analysis, the relationship between parameter $A$ and dynamic stress ratio $f$ is fitted as shown in the following equation:

$$A = a \times f^b = a \times \left( \frac{\sigma_d}{\sigma_s} \right)^b,$$  \hspace{1cm} (3)

where $a$ and $b$ are the fitting parameters based on the test data.

(2) Parameter $B$ represents the variation rate of the plastic strain with the number of load cycles. Table 1 shows that parameter $B$ is greatly affected by the cement content corresponding to the soil type. Therefore, the value of parameter $B$ was obtained by the regression analysis for different types of improved CWG, namely, “average value” in Table 1.

The fitted parameter values of the various types of improved CWG under different physical conditions were obtained according to the aforementioned methods and are listed in Table 2.

By substituting equation (3) into equation (1), the accumulative plastic strain calculation model of the improved CWG under cyclic dynamic loads can be presented as follows:

$$\epsilon_p = a \times f^b \times N^B = a \times \left( \frac{\sigma_d}{\sigma_s} \right)^b \times N^B,$$  \hspace{1cm} (4)

where $\epsilon_p$ is the accumulative plastic deformation under cyclic load; $N$ is the number of cyclic loads; and $a$, $b$, and $B$ are the fitting parameters based on the dynamic triaxial test data shown in Table 2.

### 3. Modification of the Accumulative Plastic Strain Model Based on the Field Test

Considering the difference in loading and boundary conditions of the laboratory dynamic triaxial tests and the field situation, the parameters of the accumulative plastic strain model proposed through the laboratory test in Section 2 were modified on basis of the field cyclic loading test results and numerical calculation. The following activities were performed: (1) a three-dimensional finite difference numerical calculation model of the field cyclic loading test was established for obtaining the dynamic deviatoric stress of the embankment filled by improved CWG; (2) the accumulative plastic strains of the subgrade were calculated using the dynamic deviatoric stress calculation results and the calculation model proposed through laboratory test; (3) the overall accumulative plastic deformation of the embankment was calculated using the calculated accumulative plastic strains through the layer-wise summation method; and (4) the calculation model parameters were repeatedly debugged to make the calculation results of the overall accumulative plastic deformation consistent with the field cyclic loading test results.

#### 3.1. Field Cyclic Loading Test

The field cyclic loading test is one of the most direct, effective, and reliable method of studying the dynamic and deformation characteristics of railway embankments. The field cyclic loading test simulates the effect of long-term train load through applying cyclic dynamic loads on the surface of embankment backfilled with CWG, and the variation of the dynamic responses and settlement deformation of the embankment with loading cycles were measured.

The test site of the field cyclic loading test is at DK2102+260 of the Qingyuan test section, Guangdong Province, China. The test was carried out during the period when the construction of the subgrade bed had been completed and before the track laying.

#### 3.1.1. Characteristics of Backfilling

The bottom layer of the subgrade bed and the embankment body in the test site were filled by the CWG excavated in situ and improved by adding 5% and 6% cement by weight. The optimum water contents are 11.5% and 12%, respectively. The average values of the unconfined compression strength (with the compacting factor of 95% and optimum water content) 7 days after cement treatment are 520 and 690 kPa, respectively.

#### 3.1.2. Loading Equipment and Scheme

The loading system is composed of a PMS-500 hydraulic pulse fatigue testing machine, a reinforced concrete loading plate, an actuator, and a tower steel plate. The PMS-500 machine and the actuator are connected by an oil tube to provide the cyclic load. The loading plate with a size of $2.5 \times 1.5 \times 0.45$ m...
(length × width × height) is laid on the surface of the embankment for transferring the cyclic loads. The schematic of the loading system is presented in Figure 2. A large amount of heap loading was used to provide the reaction force, as shown in Figures 2 and 3.

In this test, the vertical cyclic load in the form of a half sinusoidal wave was applied on the loading plate to simulate the dynamic train load. The peak excitation force was set as $170 \pm 5 \text{kN}$ ($5 \text{kN}$ is the floating value). Given that the centroid distance of two bogies in one CRH3 car is 17.375 m and the commercial operating speed is 250–350 km/h, the corresponding frequency induced by the high-speed trains is $4.0–5.6 \text{Hz} \left( F = \frac{V}{L} \right)$. Consequently, the loading frequency was set as 4.75 Hz in this test. The total number of cyclic loads was $2 \times 10^6$.

3.1.3. Test Equipment and Instrumentation Schemes. For measuring the dynamic responses of the embankment, two types of sensors were instrumented within the embankment during the embankment construction, namely, dynamic earth pressure cells and vibration sensors. The dynamic earth pressure gauge (i.e., DYB-5, resistance strain type) is a dual-membrane bidirectional force structure. The vibration sensor (i.e., 891–II type) is a kind of multifunctional sensor suitable for low or ultra-low frequency vibration measurement. The two types of sensors were buried at 0, 0.5, 1.0, 1.9, 2.7, 3.7, and 5.2 m below the embankment surface, as shown in Figure 4. “Coinv Dasp” data acquisition instruments were adopted for data acquisition. The acquisition instruments were connected with sensors and to a personal computer (PC) by cables, as shown in Figure 5. For measuring the overall accumulative plastic deformation of the embankment, the dial indicators were installed at the four corners of the loading plate, 10 cm away from the perpendicular edges, as shown in Figures 2 and 4.

3.1.4. Test Results. Figure 6 illustrates the variation of the vibration acceleration and dynamic stress of the top layer with the load cycles. As can be seen in Figure 6, the vibration acceleration and the dynamic stress increased significantly in the first 3,000 load cycles, with respective increments of 46.91% and 43.18% compared with those in the initial loading phase. The variation of acceleration with the subsequently increasing loading cycles exhibited an oscillatory ascending trend, while the variation trend of dynamic stress was stable.

Figure 7 shows the variation of the dynamic stress and displacement along the embankment depth after 2 million load cycles. Figure 7 indicates that within the depth of 0.5–5.2 m from the embankment surface, the vertical dynamic stress and the dynamic displacement decreased linearly with the increasing depth. The attenuations of the dynamic stress and displacement within the bottom layer (i.e., 0.5–2.7 m below the surface) were 50% and 56.5%, respectively. The attenuations within the embankment body

\[ \text{Figure 2: Schematic of loading equipment.} \]

\[ \text{Figure 3: Test site of the field cyclic loading test.} \]
Figure 4: Layout of field instrumentation.

Figure 5: Dynamic response acquisition system.

Figure 6: Variation of the dynamic responses of embankment with loading cycles. (a) Acceleration. (b) Dynamic stress.
3.2. Three-Dimensional Numerical Model. The instrumented dynamic earth pressure gauge in the field cyclic loading test can only measure the vertical dynamic stress, while the calculation of accumulative plastic strain through (4) requires the dynamic deviatoric stress. Thus, a three-dimensional finite difference numerical model of the field cyclic loading test was established by the FLAC 3D calculation software to obtain the dynamic deviatoric stresses.

The geometric sizes and material parameters of the concrete loading plate and the embankment conformed to those in the actual field test. The test results in Figure 7 and the calculation results in [27] show that the dynamic stress substantially attenuated along the depth in the embankment, indicating the weak influence of the dynamic loads on the soil foundation below the embankment. Consequently, the soil foundation treated by a CFG pile net was simplified as the homogeneous material in the numerical model. The material parameters of all the structural layers are listed in Table 3.

The geometry range and boundary condition of the numerical model are shown in Figure 8. In the x-axis
direction, both sides of the model range extended 10 m from the slope foot of the embankment. In the y-axis direction, the model range also extended 10 m from the center of the loading plate, with a total of 20 m along the longitudinal direction. A free field boundary was set around the main model. The free field boundary consisted of four plane grids and four cylindrical grids, which were coupled with the side boundary of the main grids by damper elements to prevent the distortion and reflection of vibration waves. Furthermore, the range of the model along the z-axis extended 10 m below the bottom of the embankment, and a static boundary was set at the bottom of model to reduce the reflection of incident waves.

The dynamic load input of the numerical model is consistent with the half sinusoidal loading curve of the field cyclic loading test described in Section 3.1, as shown as follows:

\[ P_d = P_0 + P_d \sin(2\pi ft), \]  

where \( P_0 \) is the static axle load of the high-speed train, and \( P_0 = 140 \) kN, \( P_d \) is the dynamic load amplitude, and \( P_d = 30 \) kN, \( f \) is the loading frequency, and \( f = 4.75 \) Hz.

The calculated vertical dynamic stresses are compared with the results of the field cyclic loading test, as shown in Figure 9. Figure 9 shows that the calculated vertical dynamic stresses are very near the field measured values 0.5 and 1.0 m below the embankment surface. Although the differences between the calculation and test results at other depth locations are large, they are less than 4 kPa. Moreover, the attenuation trend of the calculation values with the increasing depth is similar to the variation curves measured in the field test. Therefore, the numerical model established in this section is reasonable, and the calculated dynamic deviatoric stresses can be applied in the subsequent calculation of the accumulative plastic deformation of the embankment.

### 3.3. Calculation of Accumulative Plastic Deformation and Modification of the Calculation Model

To calculate the overall accumulative plastic deformation of the embankment of the field cyclic loading test through the layer-wise summation method, the embankment was divided into seven layers along the depth, namely, 0–0.5, 0.5–1.0, 1–1.9, 1.9–2.7, 2.7–3.7, 3.7–4.3, and 4.3–5.2 m below the embankment surface. The calculation of the accumulative plastic deformation was conducted as follows.

1. First, the dynamic deviatoric stress \( (\sigma_d) \) of each structural layer under cyclic load was calculated by equation (6) on the basis of the triaxial stress components extracted from the numerical modelling calculation results. The calculation results are presented in Figure 9.

\[ \sigma_d = \sigma_1 - \sigma_3 = \sqrt{I_2} \]
\[ = \sqrt{\frac{1}{2} \left[ (\sigma_{zd} - \sigma_{yd})^2 + (\sigma_{xd} - \sigma_{yd})^2 + (\sigma_{zd} - \sigma_{yd})^2 + 6\tau_{yd} \right]} \]

2. Then, the accumulative plastic strain of each soil layer was calculated by equation (4) using the parameters shown in Table 2 (cement content of 6% for the bottom layer of the subgrade bed and 5% for the embankment body) and the deviatoric stress calculation results, that is, the dynamic deviatoric stress, shown in Figure 9. The calculated results are presented in Figure 10. The calculated plastic strain 5.2 m below the embankment surface is less than 0.01%. Therefore, the overall accumulative plastic deformation from the bottom of the top layer to 5.2 m below the surface was noted as the overall deformation of embankment \( S_N \).

3. Finally, the overall accumulative plastic deformation of embankment \( S_N \) was calculated according to equation (7). The calculated \( S_N \) values that vary with the loading cycles are shown in Figure 11 and referred to as “Calculated value before parameter modification.”

\[ S_N = \int_{z=0}^{z=H} e_p(z; N)dz \]

The measured results obtained from the field test are also exhibited in Figure 11 for comparison. Figure 11 shows that the calculated values before parameter modification are larger than the test results, and the variation trend with the loading cycles is also not completely consistent with the measured results. Thus, the fitted parameters from the laboratory triaxial tests (i.e., listed in Table 2) required modification. The modifications were performed as follows:
Figure 11 shows that the calculated accumulative plastic deformation before parameter modification of the first 200,000 load cycles is 0.55 mm larger than the measured value. According to equations (1) and (3), the calculated accumulative plastic deformation value in the initial loading period is mainly determined by parameter A, which is composed of parameters a and b. Thus, parameters a and b were repeatedly modified until the calculated deformation of the first 200,000 load cycles was consistent with the measured value. The final modified parameters a and b are listed in Table 4.

(2) Figure 11 shows that the measured accumulative plastic deformation was nearly constant after 1 million load cycles without increasing further with the load cycles from 1 million to 2 million. However, the calculated deformation value (before parameter modification) continued to increase after 1 million load cycles. According to equation (4), the variation trend of accumulative plastic deformation is mainly determined by the power index, that is, parameter B. Thus, parameter B was repeatedly modified until the calculated deformation was barely increasing after 1 million load cycles. The final modified parameter B is also listed in Table 4.

The accumulative plastic deformation is calculated by equation (4) using the modified parameters shown in Table 4, and the deviatoric stress calculation results are also shown in Figure 11 and referred to as “calculated value after parameter modification.” Figure 11 indicates that the values and variation law of the calculated deformation using the modified parameters show good consistency with the field test results. Therefore, the modified calculation model of accumulative plastic strain based on equation (4) and Table 4 is applicable for calculating the accumulative plastic deformation of the improved CWG filling embankment under cyclic train loads.

Figure 10: Calculated accumulative plastic strain along the depth.

Figure 11: Accumulative plastic deformation of the embankment filled with the improved CWG.
4. Accumulative Plastic Deformation of the Improved CWG Embankment under Real Driving Conditions

4.1. Simulation of Real Train Loads. To apply the numerical model established in Section 3.2 for the calculation of dynamic responses under the real train loading condition, the loading plate was removed, and the real dynamic stresses of the top layer of the subgrade bed during the entire train unit passing process obtained from in situ tests were imposed on the surface of embankment instead. The applied real dynamic stresses were measured through the instrumented sensors during the passes of CRH2-068C running at 200–350 km/h in the Beijing-Guangzhou HSR. For example, the typical vertical dynamic stress time history of the top layer during the passes of a high-speed train running at 330 km/h is presented in Figure 12.

The time history shown in Figure 12 is the test result for a certain test point, while the distribution law of the vertical dynamic stress of the subgrade bed along the cross section should be clarified. The calculated results of [27] and the laboratory test results of the full-scale model in [28] show that the distribution of the dynamic stresses of the top layer of the subgrade bed resembles a saddle along the cross section, and the maximum values appeared at the sides of the concrete base. On the basis of the distribution law of the test results in [27] (shown in the blue curve of Figure 13) and the grid size (0.5 m × 0.5 m) of the numerical model shown in Figure 8, the distribution curve was replaced by the average value of dynamic stress in the range of each grid, which could be calculated by the piecewise function described by equation (8). The distribution curve of dynamic stress as the load input was simplified to a segmented ladder as shown in Figure 13.

\[
y = \begin{cases} 
0.76F(t), & (-0.5 \leq x \leq 0.5), \\
0.88F(t), & (-1.0 \leq x < -0.5, 0.5 \leq x \leq 1.0), \\
1.00F(t), & (-1.5 \leq x < -1.0, 1.0 \leq x \leq 1.5), \\
0.90F(t), & (-2.0 \leq x < -1.5, 1.5 \leq x \leq 2.0), \\
0.50F(t), & (-2.5 \leq x < -2.0, 2.0 \leq x \leq 2.5), 
\end{cases} 
\]

where \( F(t) \) is the time history of the maximum vertical dynamic stress along the cross section measured in the field test (e.g., the time history presented in Figure 12).

The numerical model of the improved CWG embankment, which was calibrated and field validated as discussed in Section 3.2, was divided into several sections along the longitudinal direction. Then, the dynamic loads expressed as (8) were applied on the embankment surface section by section, with the time difference determined by the train speed for simulating real train loads. The calculated vertical dynamic stress time history of the top of the top layer during the passes of high-speed trains (HSTs) running at 330 km/h is presented in Figure 14. Figure 14 shows that 32 significant peak values corresponding to 8 railcar units of the HST can be clearly identified, and the calculated dynamic stress amplitudes are very near the measured values, as shown in Figure 12. Therefore, the simulation method of real train loads is reasonable and applicable.

4.2. Calculated Accumulative Plastic Deformation under Real Train Loads. According to the modelling method described in Section 4.1, the dynamic stresses obtained from in situ tests are required as input load. Given the limitation of field test data, only the dynamic stresses of the top layer of the subgrade bed during the entire train unit passing process under the three typical real driving conditions, which are HST (of axle load less than 14 tons) with a running speed of 200 km/h, HST with a running speed of 330 km/h, and track

\[\text{Dynamic stress/Maximum dynamic stress}\]

\[\text{Horizontal distance from track center (m)}\]

\[\text{Distribution law in [27]}\]

\[\text{Simplified distribution}\]

\[\begin{array}{ccc}
1.1 & 1.0 & 0.9 \\
0.8 & 0.7 & 0.6 \\
0.5 & 0.4 & 0.3 \\
0.2 & 0.1 & 0.0 \\
\end{array}\]

\| Table 4: Modified model parameters of the improved CWG based on the field cyclic loading test. \|

| Structural layer | Cement contents (%) | Field test fitting parameters |
|------------------|---------------------|-------------------------------|
| The bottom layer of subgrade bed | 6 | 0.05592 | 1.5368 | 0.0851 |
| Embankment body | 5 | 0.08148 | 1.3854 | 0.0908 |

\[\text{DrivingConditions}\]

\[\text{ImprovedCWGEmbankmentunderReal}\]

\[\text{DrivingConditions}\]
inspect vehicle (of axle load approximately 25 tons) with a running speed of 160 km/h, were imposed on the surface of embankment. The calculated dynamic deviator stress amplitudes are shown in Figure 15.

Figure 15 shows that the dynamic deviator stress increased with the train speed increase from 200 km/h to 330 km/h under a constant axle load (14t). However, the influence of train speed on the dynamic deviator stress gradually weakened with the depth increase. At 3.7 m below the embankment surface, the dynamic deviator stresses of different train speeds were nearly the same. When the axle load increased from 14t to 25t with a slight change in train speed (i.e., 200 km/h to 160 km/h), the dynamic deviator stress amplitudes increased by more than two times, and the increment was nearly unaffected by the embankment depth.

According to the calculation procedure described in Section 3.3, the accumulative plastic deformation variations with the load cycles of the embankment filled with CWG were calculated based on the calculation results of the dynamic deviator stresses in Figure 15. The modified model parameters in Table 4 were used for the calculation of the accumulative plastic strains, and the ultimate static strength ($\sigma_u$) used to calculate dynamic stress ratio $f$ was obtained based on the gravity stress of the real embankment. The calculated results are shown in Figure 16.

Figure 16(a) shows that the evolution of the total accumulative plastic deformation below the top layer of the subgrade during 4 million cyclic loading processes can be divided into four stages.

The first stage is characterized by the rapid increase in the accumulative plastic deformation corresponding to the initial 10,000 loading cycles. The deformation increments produced by the HST (with train speed of 200 km/h) and the track inspect vehicle, respectively, reached 0.705 and 3.194 mm, accounting for 59.20% and 59.59% of the final total deformations. The second stage features a gentle increase in accumulative plastic deformation corresponding to 10,000–500,000 loading cycles. The deformation increments produced by the HST (with train speed of 200 km/h) and the track inspect vehicle are 0.288 and 1.284 mm, respectively, accounting for 24.17% and 23.96% of the final total deformations. Compared with the first stage, the deformation increase rate significantly decreased. In 500,000–2,000,000 cycle stage, the deformation tended to be stable. The deformation increment was less than 0.02 mm after 2 million load cycles, that is, the fourth stage.

To evaluate the long-term dynamic stability of the embankment filled with the improved CWG, the accumulative plastic deformation under a certain number of train load cycles was analyzed. Figure 16(a) shows that after 4 million loading cycles, the final total accumulative plastic deformations under HST loading with a running speed of 200 and 330 km/h are 1.19 and 1.77 mm, respectively. The code and relevant engineering experience stipulate that the additional settlement of the non-ballasted track subgrade shall not exceed 5 mm. The accumulative plastic deformation value calculated by the modified calculation model in this study is less than 2 mm, proving that the improved CWG can be used for subgrade filling.

The increment of the final accumulative deformation is only 0.583 mm, corresponding to the train speed increase from 200 km/h to 330 km/h, and 4.158 mm corresponds to the train axle load increase from 14t (HST) to 25t (track inspect vehicle) accompanied by a slight change in train speed (i.e., 200 km/h to 160 km/h). Therefore, the train speed has a slight influence on the accumulative plastic deformation of the subgrade, while the increase in the axle load can lead to a significant increment in accumulative plastic deformation.

The final accumulative deformations within the bottom layer of the subgrade bed as shown in Figure 16(b) are 3.633 mm associated with the track inspect vehicle and 0.593 and 1.068 mm associated with HST, accounting for 67.9%, 49.8%, and 60.2% of the total accumulative plastic deformation below the top layer of the subgrade, respectively. This observation may be attributable to the significant effect of dynamic stress ratio $f$ on plastic strain. Dynamic deviatoric stress $\sigma_d$ attenuated exponentially with the increasing embankment depth, and the ultimate static strength of soil $\sigma_s$ increased due to linear increase in its gravity stress. Consequently, according to (2), dynamic stress ratio $f$ decreased rapidly with the increase in the embankment depth. Most of the accumulative plastic deformation attenuation was produced within the depth range of the subgrade bed.
Laboratory dynamic triaxial and field cyclic loading tests were performed to investigate the applicability of the improved CWG as a subgrade filling material of HSRs. On the basis of the test results and the numerical simulation model, the accumulative plastic strain calculation model was established. The characteristics of the accumulative plastic deformation under train loading were analyzed. The main works and findings are listed as follows:

1. The calculation model of accumulative plastic strain in the power function form proven to be the most suitable for the improved CWG was established by the fitting analysis based on the laboratory dynamic triaxial test data. The model considers the dynamic stress ratio and the cement content through the model parameters.

2. On the basis of the proposed calculation model and a three-dimensional finite difference numerical model of the improved CWG embankment, the accumulative plastic strains of an improved CWG embankment at the site of the field cyclic loading test were calculated. The strain values were significantly attenuated at 2.7 m below the embankment surface (i.e., the bottom of the bottom layer of subgrade bed). The proposed model was modified by debugging the model parameters through calibrating the calculated value with the measured data of the field cyclic loading test.

3. Real train loads were simulated through imposing the dynamic stresses measured from the field dynamic response tests during the passing of real train on the established embankment model. On the basis of the dynamic deviatoric stress calculation results and the modified calculation model of accumulative plastic strain, the accumulative plastic deformations of the embankment under several real train loads were calculated.

4. The total accumulative plastic deformations of the embankment increased rapidly in the initial 10,000 loading cycles and then gradually stabilized. After 4 million HST loading cycles, the final total deformations induced by the HST loading were less than 2.0 mm, which is within the additional dynamic deformation limit value of the ballastless track subgrade. Thus, the long-term dynamic stability of the improved CWG embankment could be guaranteed.

5. The dynamic deviator stress and accumulative deformation of the improved CWG embankment increased significantly with the increase in the train axle load, while the influence of train speed was relatively small.

6. The accumulative plastic deformation behavior of the improved CWG embankment was significantly affected by the dynamic stress lever. The dynamic stress ratio $f$ decreased rapidly with the increase in the embankment depth, and most of the accumulative plastic deformation attenuation was produced within the depth range of the subgrade bed.

The proposed calculation model of accumulative plastic strain is applicable for the dynamic stability evaluation of the embankment.
improved CWG subgrade and can provide valuable reference for similar railway engineering.

**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 they have no known conflicts of financial interests or personal relationships that could have appeared to influence the work reported in this study.

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**References**

[1] National Railway Administration of the People’s Republic of China, *Code for Design of Railway Earth Structure (TB 10001-2016)*, China Railway Publishing House, Beijing, China, 2016.

[2] D. Li and D. Davis, “Transition of railroad bridge approaches,” *Journal of Geotechnical and Geoenvironmental Engineering*, vol. 131, 2005.

[3] B. Indraratna, M. Babar Sajjad, T. Ngo, A. Gomes Correia, and R. Kelly, “Improved performance of ballasted tracks at transition zones: a review of experimental and modelling approaches,” *Transportation Geotechnics*, vol. 21, Article ID 100260, 2019.

[4] D. Mishra, E. Tutumluer, T. D. Stark, J. P. Hyslip, S. M. Chismer, and M. Tomas, “Investigation of differential movement at railroad bridge approaches through geotechnical instrumentation,” *Journal of Zhejiang University - Science*, vol. 13, no. 11, pp. 814–824, 2012.

[5] E. Fortunato, A. Paixão, and R. Calçada, “Railway track transition zones: Design, construction, monitoring and numerical modelling,” *International Journal of Reality Therapy*, vol. 2, no. 4, pp. 33–58, 2013.

[6] C. L. Monismith, N. Ogawa, and C. R. Freeme, “Permanent deformation characteristics of subgrade soils due to repeated loading,” *Transportation Research Record*, vol. 537, pp. 1–17, 1975.

[7] D. Li and E. T. Selig, “Cumulative plastic deformation for fine-grained subgrade soils,” *Journal of Geotechnical Engineering*, vol. 122, no. 12, pp. 1006–1013, 1996.

[8] A. Ramos, A. Gomes Correia, B. Indraratna, T. Ngo, R. Calçada, and P. A. Costa, “Mechanistic-empirical permanent deformation models: laboratory testing, modelling and ranking,” *Transportation Geotechnics*, vol. 23, Article ID 100326, 2020.

[9] J.-C. Chai and N. Miura, “Traffic-load-induced permanent deformation of road on soft subsoil,” *Journal of Geotechnical and Geoenvironmental Engineering*, vol. 128, p. 907, 2002.

[10] G. Gidel, P. Hornych, J. J. Chauvin, D. Breyssse, and A. Denis, “A new approach for investigating the permanent deformation behavior of unbound granular material using the repeated load triaxial apparatus,” *Bull Des Lab Des Ponts Chaussees*, pp. 5–21, 2001.

[11] R. Chen, J. Chen, X. Zhao, X. Bian, and Y. Chen, “Cumulative settlement of track subgrade in high-speed railway under varying water levels,” *International Journal of Reality Therapy*, vol. 2, no. 4, pp. 205–220, 2014.

[12] V. Noolu, B. M. Krishna, Y. Paluri, and S. Mogili, “Performance evaluation of recycled granular material as sustainable sub-base material,” *Materials Today Proceedings*, vol. 38, pp. 2457–2463, 2021.

[13] H. Mei, S. Satvati, and W. Leng, “Experimental study on permanent deformation characteristics of coarse-grained soil under repeated dynamic loading,” *Railway Engineering Science*, vol. 29, no. 1, pp. 94–107, 2021.

[14] F. Zhang, T. Wang, J. Bu, and Z. Yin, “Dynamic behaviors of coarse granular aggregates in high-speed railway subgrades,” *Soil Dynamics and Earthquake Engineering*, vol. 152, Article ID 107046, 2022.

[15] X. Ling, P. Li, F. Zhang, Y. Zhao, Y. Li, and L. An, “Permanent deformation characteristics of coarse grained subgrade soils under train-induced repeated load,” *Advances in Materials Science and Engineering*, vol. 2017, Article ID 6241479, 15 pages, 2017.

[16] A. J. Puppala, S. Saride, and S. Chomtid, “Experimental and modeling studies of permanent strains of subgrade soils,” *Journal of Geotechnical and Geoenvironmental Engineering*, vol. 135, no. 10, pp. 1379–1389, 2009.

[17] Y. Cai, Q. Sun, L. Guo, C. H. W. J. Juang, and J. Wang, “Permanent deformation characteristics of saturated sand under cyclic loading,” *Canadian Geotechnical Journal*, vol. 52, no. 6, pp. 795–807, 2015.

[18] A. Gomes Correia and J. Cunha, “Analysis of nonlinear soil modelling in the subgrade and rail track responses under HST,” *Transportation Geotechnics*, vol. 1, no. 4, pp. 147–156, 2014.

[19] H. Heydari-Noghabi, J. N. Varandas, M. Esmaeili, and J. A. Zakeri, “Investigating the influence of auxiliary rails on dynamic behavior of railway transition zone by a 3D train-track interaction model,” *Latin American Journal of Solids and Structures*, vol. 14, no. 11, pp. 2000–2018, 2017.

[20] M. R. Khan and S. M. Dasaka, “Quantification of ground-vibrations generated by high speed trains in ballasted railway tracks,” *Transportation Geotechnics*, vol. 20, Article ID 100245, 2019.

[21] X. C. Bian, C. Cheng, J. Q. Jiang, R. P. Chen, and Y. M. Chen, “Numerical analysis of soil vibrations due to trains moving at critical speed,” *Acta Geotech.*, vol. 11, no. 2, pp. 281–294, 2016.

[22] W. M. Zhai, K. Y. Wang, and C. B. Cai, “Fundamentals of vehicle-track coupled dynamics,” *Vehicle System Dynamics*, vol. 47, no. 11, pp. 1349–1376, 2009.

[23] Y. Guo and W. M. Zhai, “Long-term prediction of track geometry degradation in high-speed vehicle–ballastless track system due to differential subgrade settlement,” *Soil Dynamics and Earthquake Engineering*, vol. 113, pp. 1–11, 2018.

[24] J. Chen and Y. Zhou, “Dynamic behaviors of subgrade soils due to trains moving at railroad bridge approaches,” *Transportation Geotechnics*, vol. 21, Article ID 107046, 2022.

[25] F. Meng and W. T. Hou Yongfeng, “3D numerical research on bridge-subgrade transition,” *Rock and Soil Mechanics*, vol. 28, pp. 849–854, 2007.
[26] Q. He, *Study on Dynamic Characteristics and Stability of Fully Weathered Granite High-Speed Railway Subgrade*, Ph.D. Thesis, Central South University, Changsha, China, 2007.

[27] L. Dong, C. Zhao, and D. Cai, “Experimental validation of a numerical model for prediction of the dynamic response of ballastless subgrade of high-speed railways,” *China Civil Engineering Journal*, vol. 41, pp. 81–86, 2008.

[28] X. C. Bian, H. G. Jiang, C. Cheng, Y. M. Chen, R. P. Chen, and J. Q. Jiang, “Full-scale model testing on a ballastless high-speed railway under simulated train moving loads,” *Soil Dynamics and Earthquake Engineering*, vol. 66, pp. 368–384, 2014.