



Journal of Highway and Transportation Research and Development

Home page: www.sciopen.com/journal/2095-6215



Design modification, stability verification, and engineering evaluation of geosynthetic reinforced soil integral bridges (GRS-IBs)

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Abstract: Geosynthetic reinforced soil integral bridges (GRS-IBs) combine a jointless structural system with reinforced backfill mechanically connected to the abutment. The K57+125 highway overpass in Quanzhou, China, is a GRS-IB, modified from a jointed bridge. This study investigates its design modification, stability verification, and economic performance. After conversion from the original jointed bridge to the GRS integral system, the main-girder prestressing arrangement was redesigned, reducing the number of tendon groups from five to four. Internal stability verification showed that the maximum reinforcement tensile force was 13.84 kN and the minimum pullout safety factor was 96.05, while the reduced ultimate bearing capacity and the reduced dead-load bearing capacity corresponding to 1% vertical strain both exceeded the applied vertical pressure of 97.59 kPa. The sliding and overturning safety factors were 2.18 and 5.92, respectively. Compared with the original bridge scheme, concrete and prestressing-strand consumption decreased by approximately 36% and 12.76%, respectively, while reinforcing-steel consumption increased by approximately 18%. The estimated initial construction cost was nearly 10% lower. These results demonstrate the structural feasibility and economic potential of the adopted GRS integral bridge scheme.

Keywords: Bridge engineering; GRS-IB; design optimization; stability verification; tie bars; lower construction cost

1 Introduction

Jointless bridges eliminate expansion joints and thereby reduce deterioration and maintenance associated with these components [1-2]. Integral bridges achieve structural continuity through the monolithic connection between the superstructure and abutments; however, this continuity transfers thermally induced longitudinal movements directly to the abutments and surrounding backfill [2-3]. Repeated thermal movements can generate cyclic soil-structure interaction, progressive lateral earth pressure, and accumulated backfill deformation or approach settlement [4-5]. Such progressive accumulation is also associated with the ratcheting behavior of granular materials under repeated loading [6]. Consequently, abutment-backfill interaction remains a key issue governing the long-term behavior of integral bridges [3-5].

Geosynthetic reinforced soil (GRS) technology has been increasingly investigated and applied in bridge abutments because the compacted backfill and closely spaced reinforcement act as a composite system [7]. Experimental studies have shown that reinforcement spacing, stiffness, and facing conditions can significantly influence the deformation, reinforcement strain, and load-transfer behavior of GRS bridge abutments [8-9]. The connection between the reinforcement and facing is also critical to structural performance. Inadequate connections may lead to progressive deformation, facing separation, and loss of structural integrity [10], whereas connecting the reinforcement firmly to the facing can reduce backfill

settlement and adverse earth-pressure development under cyclic horizontal movement [11]. Therefore, effective interaction between the reinforced backfill and the abutment system is particularly important for integral bridge applications subjected to repeated longitudinal movements.

A GRS integral bridge system was proposed by Tatsuoka et al. [12], in which geosynthetic reinforcement is firmly connected to full-height rigid facings and the girder is integrated with the facings without bearings. Subsequent research and engineering applications demonstrated the feasibility of this structural system and emphasized the importance of the connection between the reinforced backfill and facing [13]. More recent physical model tests have further shown that an appropriately configured reinforcement-facing connection can reduce surface settlement and improve the response of GRS integral abutments under cyclic thermal loading [14]. These studies provide a basis for improving the coordinated response between structural and geotechnical components in GRS integral bridge systems.

Although GRS technology has been introduced into integral bridge engineering in China, engineering applications of GRS integral bridges with tie-bar connections remain relatively limited. The K57+125 highway overpass on the Zhenghe-Yongding Expressway provides an engineering case for examining the design and construction of this bridge system. The detailed construction technologies of this project, including GRS construction, tie-bar installation, and construction of the girder-abutment-approach slab

Received: 7 August 2026; Revised: 14 September 2026; Accepted: 20 September 2026

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DOI: 10.26599/HTRD.2026.9480108

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system, were previously reported by Huang and Ouyang [15]. Building on that previous work, the present study summarizes only the construction features necessary to describe the engineering application and focuses primarily on the design modification and engineering evaluation of the bridge. Specifically, the prestressing arrangement of the main girder is examined following conversion from the original jointed bridge to the GRS integral system, the internal and external stability of the GRS abutment is verified, and changes in construction quantities and initial construction cost are evaluated. The results provide additional engineering evidence for the design and application of GRS integral bridges.

2 Engineering background

The K57+125 highway overpass in Quanzhou, Fujian Province, was originally designed as a single-span simply supported T-girder bridge equipped with expansion joints at both abutments. It was later redesigned as a GRS integral bridge with tie bars, as illustrated in Fig. 1. The bridge has a total length of 68.7 m, a span length of 41.6 m, and a width of 19.0 m. The prestressed concrete T-girders are 40.0 m long and 2.5 m deep. The starting and ending abutments have total heights of 10.65 m and 11.75 m, respectively. The abutment stem is 1.5 m thick, and the foundation consists of a single row of 1.8 m diameter concrete piles.

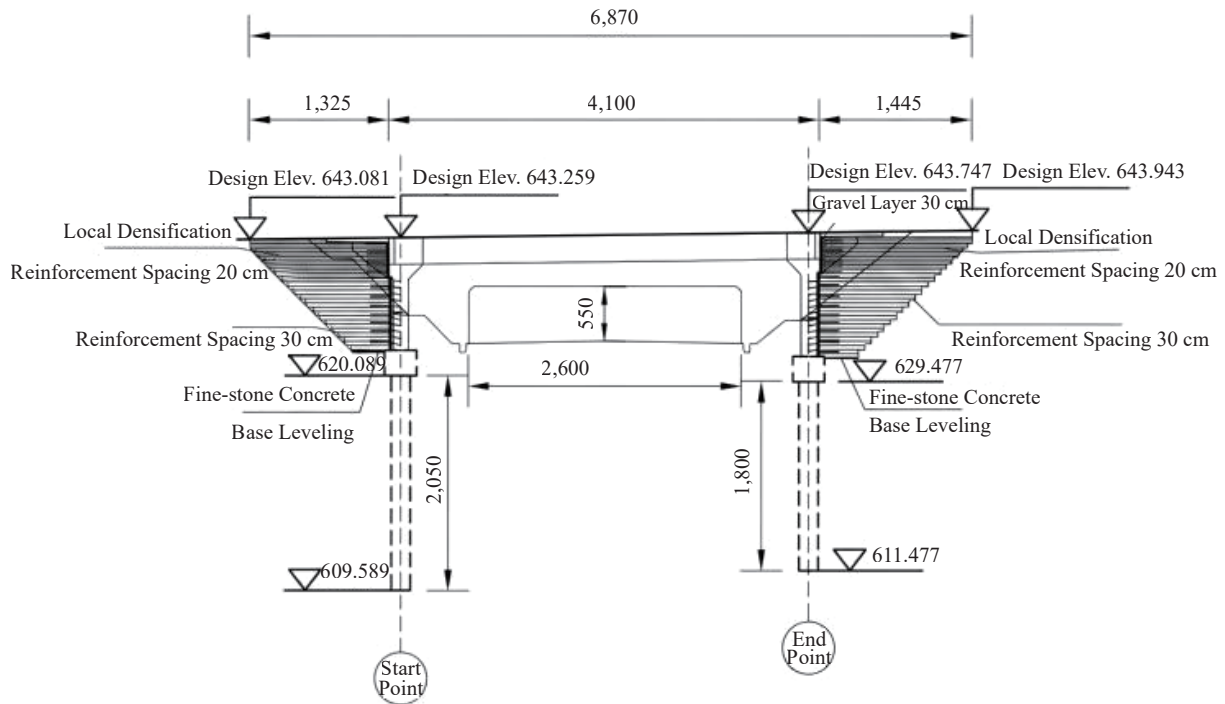


Fig. 1 General layout of the highway overpass (unit: cm).

The backfill behind the abutments consists of GRS with woven geotextile reinforcement. The reinforcement spacing is 0.2 m near the abutment top and 0.3 m in the middle and lower zones. The GRS mass is connected to the abutment stem by 16 mm diameter stainless-steel tie bars. These structural parameters are subsequently used in the stability verification of the GRS abutment.

Detailed construction configurations and procedures of the bridge have been reported previously [15]; therefore, only the key construction features relevant to the subsequent design and stability evaluations are summarized in the following section.

3 Key construction technologies

3.1 Construction of GRS behind the abutment

Before construction of the reinforced soil mass, a 20 cm thick C30 fine-aggregate concrete leveling layer was placed beneath the GRS to provide a level and stable construction base. L-shaped galvanized steel-mesh panels, as shown in Fig. 2, and gravel bags were used as temporary facing and support elements during construction.

Polypropylene woven geotextile was placed with its principal strength direction perpendicular to the abutment. The reinforcement was maintained flat and taut during placement and wrapped around the gravel bags with a uniform wrap-around length of 2.0 m, as shown in Fig. 3. Well-graded crushed stone or gravel was then

placed and compacted in layers using light compaction equipment, with a required degree of compaction of not less than 95%.



Fig. 2 L-shaped steel-mesh panels used as temporary facing.

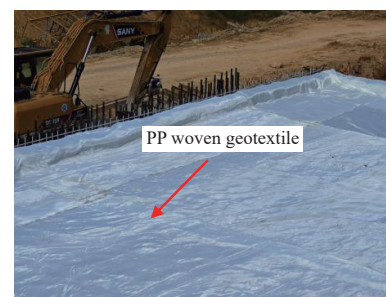


Fig. 3 Wrap-around arrangement of the geotextile reinforcement.

The reinforced soil mass was connected to the abutment by 16 mm diameter 316 stainless-steel tie bars installed at vertical intervals of 0.6 m. The tie bars were connected to angle steels to provide a mechanical connection between the reinforced soil and

the abutment. The reinforcement, facing system, fill, and tie bars were constructed progressively until the specified elevations were reached. Representative details of the tie-bar connection are shown in Fig. 4.



Fig. 4 Construction of tie bars. (a) Side view; (b) Top view.

3.2 Construction of the girder-abutment-approach slab system

Because the girder is rigidly connected to the abutment, construction of the girder-abutment-approach slab system requires coordinated sequencing between the superstructure, abutment, and GRS backfill. The principal construction stages are summarized in Fig. 5.

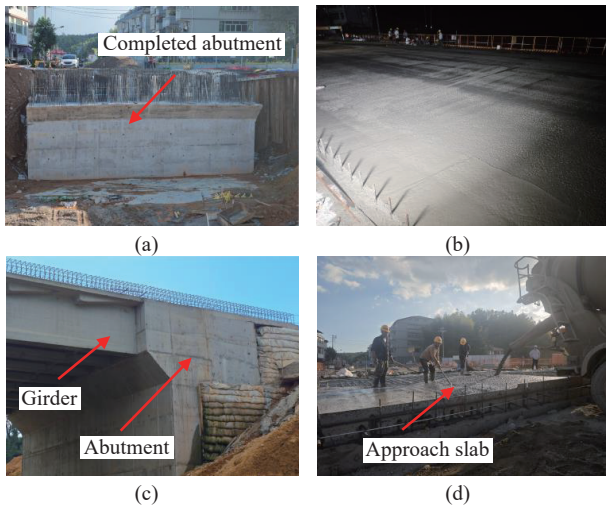


Fig. 5 Construction sequence of the girder-abutment-approach slab system. (a) Completed abutment; (b) Deck-slab casting; (c) Girder-abutment integral connection; (d) Approach slab construction.

4 Design modification and engineering evaluation

4.1 Design modification and verification of the GRS integral bridge

The original bridge was designed as a simply supported prestressed concrete T-girder bridge. After conversion to the GRS integral bridge, the main girders were rigidly connected to the flexible abutments, resulting in a change in the structural system. Accordingly, the prestressing tendon arrangement of the main girder was redesigned. As shown in Fig. 6, the number of prestressing tendon groups was reduced from five (N1-N5) in the original jointed bridge to four (N1-N4) in the GRS integral bridge, accompanied by corresponding adjustments to the longitudinal tendon profiles and their locations at the support and midspan sections.

4.2 Internal stability verification of the GRS abutment

Due to space limitations, the starting-side GRS abutment was selected for internal stability verification. According to the project

design calculations, the verification included the tensile strength of the reinforcement, pullout resistance of the reinforcement, ultimate bearing capacity of the reinforced soil mass, and vertical deformation under service conditions. The effects of vehicle loads and other surcharge loads were incorporated through the additional stresses acting at the reinforcement layers.

(1) Tensile Strength Verification of the Reinforcement

Polypropylene woven geotextile was used as the reinforcement material in this project. The horizontal tensile force T_i carried by the reinforcement at the i th layer per unit wall length was calculated using Eq. (1):

$$T_i = \left[(\sigma_{vi} + \sum \Delta\sigma_{vi}) K_i + \Delta\sigma_{hi} \right] s_{vi} / A_r \quad (1)$$

where, σ_{vi} is the vertical stress due to the self-weight at the reinforcement layer (kPa); $\sum \Delta\sigma_{vi}$ is the additional vertical stress caused by surcharge loads (kPa); K_i is the earth pressure coefficient; $\Delta\sigma_{hi}$ is the additional horizontal stress (kPa); s_{vi} is the vertical spacing of the reinforcement (m); and A_r is the reinforcement coverage ratio m^{-1} , taken as 1.

The calculated maximum tensile force in the reinforcement was 13.84 kN, whereas the allowable tensile strength of the reinforcement used in the project was 48 kN. Therefore, the tensile strength requirement of the reinforcement was satisfied.

(2) Pullout Resistance Verification of the Reinforcement

The resistance of the reinforcement against pullout was evaluated using the pullout safety factor F_s , as expressed in Eq. (2):

$$F_s = T_{pi} / T_i \quad (2)$$

The pullout resistance T_{pi} of the reinforcement at the i th layer was calculated using Eq. (3):

$$T_{pi} = 2 \sum \sigma_{vi} B L_{ei} f \quad (3)$$

where, T_{pi} is the pullout resistance of the reinforcement at the i th layer (kN); T_i is the horizontal tensile force in the reinforcement at the i th layer (kN); L_{ei} is the effective reinforcement length beyond the failure surface (m); $\sum \sigma_{vi}$ is the vertical stress acting at the i th reinforcement layer (kPa), including the self-weight of the reinforced soil, the overlying road fill, and other dead loads; B is the reinforcement width (m), taken as 1 m; and f is the friction coefficient between the reinforcement and soil, taken as 0.67.

The minimum calculated pullout safety factor was 96.05, which was greater than the required value of 1.5. Therefore, the pullout resistance requirement of the reinforcement was satisfied.

(3) Ultimate bearing capacity verification of the reinforced soil mass

The ultimate bearing capacity $q_{n,an}$ of the reinforced soil mass was calculated using the design method adopted for this project, as expressed in Eq. (4):

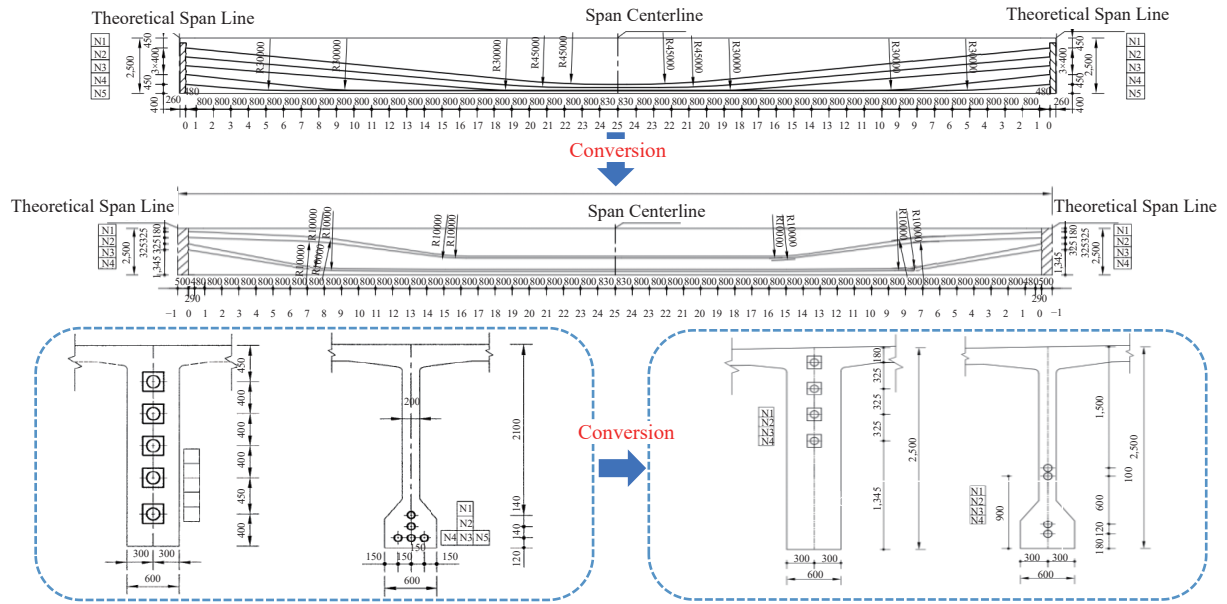


Fig. 6 Comparison of prestressing tendon arrangements of the main girder before and after bridge conversion.

$$q_{n,an} = \left[0.7 \frac{sv}{6d_{max}} \left(\frac{T_f}{S_v} \right) \right] K_p = 1,155.28 \text{ kPa} \quad (4)$$

where, S_v is the reinforcement spacing (m); d_{max} is the maximum particle size of the fill (m), taken as 0.0254 m; T_f is the ultimate tensile strength of the reinforcement (kN/m), taken as 120 kN/m; and K_p is the Rankine passive earth pressure coefficient.

The calculated ultimate bearing capacity of the reinforced soil mass was 1,155.28 kPa. After applying a resistance reduction factor of $\Phi=0.45$, the reduced bearing capacity was 519.87 kPa. The design vertical pressure acting on the reinforced soil mass was $V_f=97.59$ kPa. Since 519.87 kPa > 97.59 kPa, the ultimate bearing capacity requirement of the reinforced soil mass was satisfied.

(4) Vertical deformation verification under service conditions

In addition to the ultimate bearing capacity, the vertical deformation of the reinforced soil mass under service conditions was evaluated. The dead-load bearing capacity corresponding to a vertical strain of 1%, $q_{DL,allow@ \varepsilon=1\%}$, was calculated using Eq. (5):

$$q_{DL,allow@ \varepsilon=1\%} = 0.2 \left[0.7 \frac{sv}{6d_{max}} \left(\frac{T_f}{S_v} \right) \right] K_p = 231.06 \text{ kPa} \quad (5)$$

The reduced dead-load bearing capacity of the reinforced soil mass corresponding to a vertical strain of 1% was 103.97 kPa, which was greater than the vertical load acting on the abutment, $V_f=97.59$ kPa, thereby satisfying the specification requirements.

4.3 Checking for external stability of the GRS abutment

The external stability of the same starting-side GRS abutment was subsequently checked. In the calculations, the GRS mass was treated as a gravity-type structure, and sliding stability, overturning stability, and foundation bearing capacity were evaluated.

(1) Sliding stability verification

The active earth pressure behind the abutment was calculated using Rankine earth pressure theory. The Rankine active earth pressure coefficient K_a was taken as 0.17, and the equivalent surcharge height accounting for the vehicle load and pavement was 3.95 m. The earth pressure was calculated as follows:

The earth pressure intensity at the top of the reinforced soil mass, q_1 , was calculated using Eq. (6):

$$q_1 = \gamma h_q b K_a = 19 \times 3.95 \times 19 \times 0.17 = 244.88 \text{ kN/m} \quad (6)$$

The earth pressure intensity at the bottom of the reinforced soil mass, q_2 , was calculated using Eq. (7):

$$q_2 = \gamma(h_q + h) b K_a = 19 \times (3.95 + 10.3) \times 19 \times 0.17 = 882.84 \text{ kN/m} \quad (7)$$

Accordingly, the total active earth pressure E_a was calculated using Eq. (8):

$$E_a = (q_1 + q_2) / 2 \times h = (244.88 + 882.84) / 2 \times 10.3 = 5,807.72 \text{ kN} \quad (8)$$

Neglecting the passive earth pressure in front of the abutment, the sliding stability safety factor K_c was calculated in accordance with Appendix H of the *Specifications for Design of Highway Subgrades* using Eq. (9):

$$K_c = \mu \frac{G + E_y}{E_x} = 2.18 > 1.3 \quad (9)$$

where, μ is the friction coefficient between the abutment base and the foundation soil, taken as 0.5 because the foundation consists of completely weathered crystal-tuff classified as soft rock; G is the total vertical force (kN), including the self-weight of the reinforced soil mass and the vertical load transferred from the superstructure, equal to 13 793.49 kN; E_y is the vertical component of the active earth pressure (kN); and E_x is the horizontal component of the active earth pressure (kN).

Therefore, the sliding stability of the abutment satisfied the design requirement.

(2) Overturning stability verification

The overturning stability safety factor K_θ was calculated using Eq. (10):

$$K_\theta = \frac{\sum M_b}{\sum M'_b} = \frac{GZ_G + E_y(Z_x + 1.5) + F_N(h - h_b/2)}{E_x Z_y} \quad (10)$$

where, $\sum M_b$ is the sum of resisting moments (kN·m); $\sum M'_b$ is the sum of overturning moments (kN·m); Z_G is the distance from the resultant vertical force acting on the abutment to the front edge of the abutment base (m); Z_x is the horizontal distance from the resultant earth-pressure force to the base (m); F_N is the axial force exerted by the main girder on the abutment (kN), taken as 8,617.6 kN under dead load; h_b is the main-girder depth (m), equal to 2.5 m; and Z_y is the vertical distance from the resultant earth-pressure force to the base (m). The remaining symbols are as defined above.

The calculated overturning stability safety factor was $K_\theta = 5.92 > 1.5$, indicating that the abutment satisfied the overturning stability requirement.

(3) Foundation bearing capacity verification

According to Section 5.2 of the *Specifications for Design of*

Foundation of Highway Bridges and Culverts, the foundation bearing capacity of the GRS abutment was evaluated using Eqs. (11) and (12):

$$\sigma_{\max} = \frac{\sum N}{A} + \frac{M}{W} \leq k[\sigma] \quad (11)$$

$$\sigma_{\min} = \frac{\sum N}{A} - \frac{M}{W} > 0 \quad (12)$$

where, $\sigma_{\max}$ and $\sigma_{\min}$ are the maximum and minimum stresses at the foundation base (kPa), respectively; $\sum N$ is the resultant vertical force acting on the foundation base (kN); A is the base area of the GRS abutment (m^2); M is the moment about the center of the foundation base ($\text{kN}\cdot\text{m}$); W is the section modulus of the foundation base (m^3); k is the allowable bearing-capacity enhancement factor of the foundation soil, taken as 1.2; $[\sigma]$ is the allowable bearing capacity of the foundation soil (kPa), taken as 350 kPa.

The calculated maximum and minimum stresses at the foundation base were 407.98 kPa and 130.2 kPa, respectively. Since $\sigma_{\max} < 1.2 \times 350 = 420$ kPa and $\sigma_{\min} > 0$, the foundation bearing capacity satisfied the design requirement.

4.4 Comparison of construction quantities and costs

The conversion from the original jointed bridge to the GRS integral bridge significantly changed the construction quantities. The use of flexible integral abutments and a single row of piles reduced the material quantities required for the abutments, pile caps, and pile foundations. According to the project quantity estimate, the total concrete consumption was reduced by approximately 36%, whereas the reinforcing steel consumption increased by approximately 18% because of the increased reinforcement in the integral abutments. In addition, the quantity of prestressing strands was reduced from 17,929.1 kg to 15,642 kg, corresponding to a reduction of 12.76%, while the expansion joints were eliminated and PP woven geotextiles were additionally required. Overall, the estimated initial construction cost of the GRS integral bridge was nearly 10% lower than that of the original jointed bridge.

5 Conclusions

Based on the design modification, construction practice, stability verification, and quantity and cost evaluation of the K57+125 GRS integral bridge, the following conclusions can be drawn:

(1) The adopted construction scheme integrates a concrete leveling layer, temporary steel-mesh and gravel-bag facing, controlled placement and compaction of the geosynthetic-reinforced fill, and tie-bar connection between the reinforced soil mass and the abutment. In the girder-abutment-approach slab system, coordinated sequencing between the superstructure, abutment, and GRS backfill was adopted to accommodate the requirements of integral construction.

(2) Conversion from the original simply supported jointed bridge to the GRS integral system required redesign of the main-girder prestressing arrangement. The number of prestressing tendon groups was reduced from five (N1-N5) to four (N1-N4), with corresponding adjustments to the tendon profiles and locations.

(3) The internal and external stability verifications confirmed that the GRS abutment satisfied the adopted design requirements. The maximum reinforcement tensile force was 13.84 kN, lower than the allowable value of 48 kN, and the minimum pullout safety factor was 96.05. The reduced ultimate bearing capacity and the reduced dead-load bearing capacity corresponding to 1% vertical strain were 519.87 kPa and 103.97 kPa, respectively, both exceeding the applied vertical pressure of 97.59 kPa. The sliding and overturning safety factors were 2.18 and 5.92, respectively, while the maximum and minimum foundation stresses were 407.98 kPa and 130.2 kPa, satisfying the adopted design requirements.

(4) Compared with the original jointed bridge, the GRS integral bridge reduced concrete consumption by approximately 36% and prestressing strand consumption by 12.76%, although reinforcing-steel consumption increased by approximately 18%. The estimated initial construction cost was nearly 10% lower, indicating an economic advantage for the adopted bridge scheme.

Acknowledgements

None.

Funding

This study was supported by the University-industry Cooperation Project in Fujian Province (Grant No. 2024Y4002) and 2023 Fujian Provincial Transportation Science and Technology Demonstration Project (Grant No. SF20230202). The authors are grateful for their support.

Declaration of competing interest

The authors have no competing interests to declare that are relevant to the content of this article.

Author contribution statement

Dongqing Hong: Data curation, Project administration, Visualization. Fuyun Huang: Conceptualization, Software, Methodology, Funding acquisition. Jinrong Shi: Data curation, Writing - review & editing. Suying Ouyang: Data curation, Writing - original draft.

Data availability statement

Data will be made available on request.

Use of AI statement

During the preparation of this research, we used DeepSeek V3.2 and Doubao 1.72.3_win to improve grammar and readability. After using this tool, we reviewed and edited the content as needed and take full responsibility for content of the publication.

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