Tied concrete arch bridge strengthening

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Summary

This work presents the concept design for the strengthening of a reinforced concrete tied arch bridge from the 50s. The bridge comprises four 28 m spans, supported by four arch-shaped girders with tension ties, also of reinforced concrete, connecting their bases. The girders supports on piers and heavy abutments consist of concrete hinges. The structure, originally designed for the 1943 TB-24 live load, had already undergone jacketing in the 80s. Among other pathologies discussed in this work, this jacketing was heavily deteriorated. As strengthening, a re-jacketing with tensioning of the ties was proposed, allowing the structure to work compressed and reducing vibrations and fatigue. Corbels with elastomeric bearings were constructed, after which the existing concrete hinges were diamond wire sawed. Once the horizontal deformations of the decks were released, the already jacketed structure was effectively prestressed without part of the prestressing being absorbed by the abutments.

1 INTRODUCTION

In June 2020, during an inspection of the bridge over the Rio Vermelho on the southbound lane of BR-163/MT, a severe level of deterioration was observed in several structural elements, with concrete plates manually detaching and reinforcement bars completely loose from the structure. The level of vibration of structural elements with the passage of heavy trucks was also concerning. Due to the severe pathologies found, traffic on the bridge had to be restricted to one lane in the center of the deck and later fully closed until strengthening work began. Figure 1 shows a photo taken during the inspection.

Fig. 1 Bridge photos taken during 2020 inspection.

According to the IBGE library [1], the structure in question was built in 1953 for a live load of 24 tf, Class I of NB6/43 [2] (Brazilian road live load standard of 1943). According to Folha Regional MT [3], the bridge was strengthened in the 1980s, likely to adapt its capacity to the TB36 live load (NB6/60). This reinforcement, performed by jacketing several elements, is evident in the structure, with different-colored concrete and ribbed reinforcement bars differing from the original smooth ones. In the photo presented in Figure 2, taken during the inspection, one can see that the elements originally had slimmer sections.

The jacketing applied has already detached from the original concrete in several areas, leaving the reinforcement bars exposed and heavily corroded, as seen in Figure 2. Additionally, surface repair marks from around 2015 for previously apparent pathologies can be identified. Some of these repairs already show crack propagation to the surface.

In elements where the jacketing has completely detached or is loose from the original structure, it can be said that the capacity aligns with the original design criteria, and there is an additional dead load from the inoperative jacketing. The original design's live load (Class I of NB6/43) is much lower than the current TB450 of NBR7188:2024 [4] (current Brazilian road live load standard). Combined with over seven decades of deterioration, the bridge's safety level was likely below the desired level.

The following sections present some aspects of the bridge strengthening project to comply with current standards criteria.

Fig. 2 Photos of existing jacketing deterioration conditions.

2 STRUTURAL SYSTEM

The bridge consists of four isostatic spans supported by girders composed of arches and ties. The girders are likely supported by reinforced concrete hinges (Freyssinet type) at the central piers and massive abutments. This type of support allows rotations but prevents displacements. In other words, the girders are not isostatic, as their bases are horizontally restrained to each other. Figure 3 presents an illustration of the bridge elevation.

When a girder undergoes bending, the base of the arch requires a vertical support reaction and a horizontal reaction that is approximately twice as large, due to the inclination of the element. In terms of deformations, it is as if the arch deforms, attempting to move its feet apart but is prevented by the support. The vertical reaction has a simple mechanism and is provided by the piers and abutment walls. The horizontal reaction is more complex and can be generated by the tension ties of the girders themselves or by adjacent spans transferring the load to the abutment (since all supports are fixed). The proportion of the horizontal load absorbed by the tie of the flexed girder versus the portion transferred through adjacent spans to the abutments depends on their relative stiffness. Figure 4 illustrates these mechanisms.

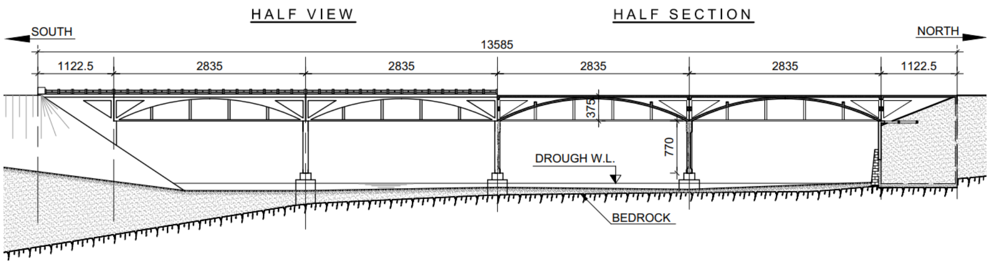

Fig. 3 Original bridge structure illustration. Half view on the left and half section on the right. (dimensions in cm).

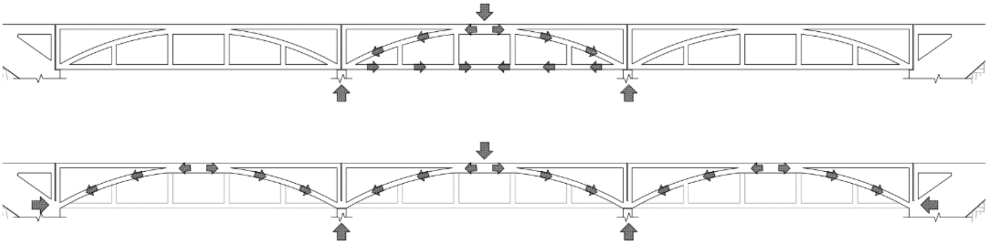

Fig. 4 Upper image: horizontal component of arch support reaction absorbed by tie. Bottom image: horizontal components of arch support reaction absorbed by abutments through other spans.

Numerical analyses were conducted to evaluate the horizontal restriction behavior at the base of the arches (Figure 5). When considering the arch supports completely fixed horizontally (by the abutments), deflections of about 16 mm occur for permanent loads. However, if we ignore the horizontal movement restriction imposed by the support and allow the tie to absorb all the load, the deflection increases to 45 mm, and the horizontal deformation of the support reaches 10 mm. Since the abutments are large concrete boxes filled with soil and backfill on the outer face, it is expected that their horizontal displacement stiffness is much greater than the axial stiffness of the four ties. Thus, the original structural scheme of the bridge is highly dependent on the horizontal stiffness of the abutments; small displacements in the supports generate significant deflections and stresses in the superstructure.

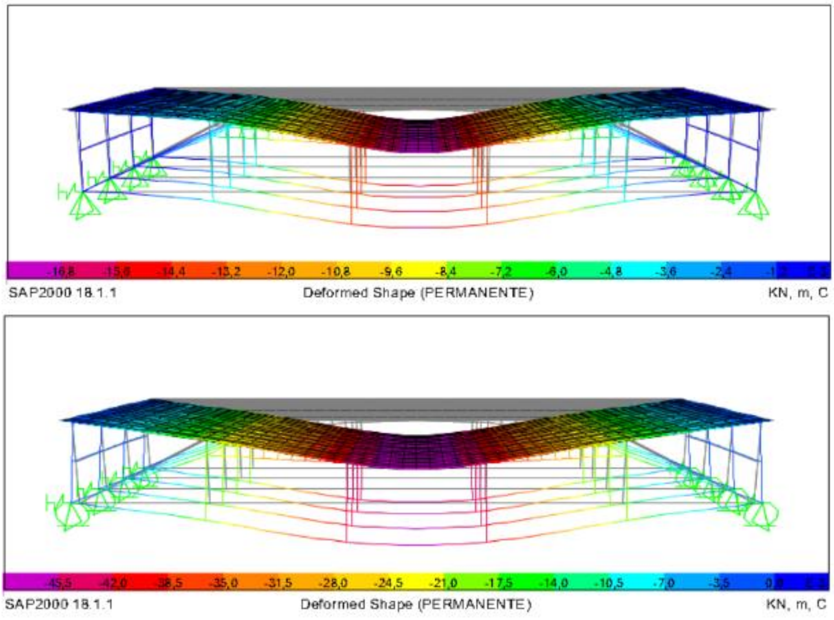

Fig. 5 Structural model deformation of permanent loads. Upper image with free lateral bearing displacements. Bottom image with fixed lateral bearing displacements.

3 DEFORMAÇÕES PERAMENTES

Three topographic surveys were conducted on the bridge in July and August 2020 to monitor the permanent deflections of the superstructure. The situation found is illustrated in Figure 6 by the average deflections obtained in the three surveys at the left and right edges of the slab (without pavement). A significant difference in deformations is observed, especially in the extreme spans, with 3.0 cm in span 4 and 15.0 cm in span 1.

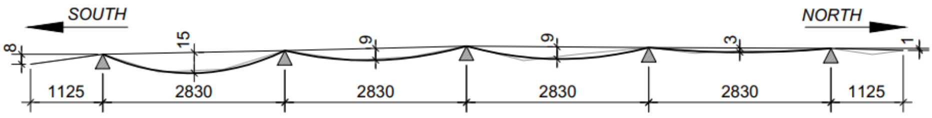

Fig. 6 Deck elevation survey to inquire permanent deformations (dimensions in cm).

Considering the sensitivity of deflections to any horizontal displacement of the abutments, a probable explanation for the deformation configuration found is the displacement of the south abutment (E1) towards the approach embankment.

The numerical models developed indicate that the magnitude of the instantaneous deflections for permanent load in the spans varies from 1.5 cm to 4.5 cm, depending on whether the abutment is laterally immovable or fully flexible. If we roughly consider that concrete creep has amplified these deformations by a factor of three, the deflections would range from 4.5 cm to 13.5 cm. In the case of span 1, the deformation approaches the condition without horizontal displacement restrictions, indicating that the support was 'pulled,' causing the arch and the tie to operate under potentially more severe conditions than originally designed.

4 STRENGTHENING APPROACH

4.1 Superstructure

The original structure was designed for a significantly lower live load than that prescribed by the current standard, which aligns with the actual highway traffic. Additionally, 67 years of service have reduced the load-bearing capacity of the reinforcement due to corrosion and fatigue, making superstructure strengthening inevitable.

The existing jacketing, likely added to adapt the bridge to TB36, is visibly degraded, with detached regions and exposed, corroded reinforcement bars. Ultrasonic tests identified voids in the jacketed elements in various areas, indicating that in other undetached regions, the contact between the original concrete and the jacketing has already been lost. It is not feasible to apply another jacket over the existing one, as force transfer via shear cannot be ensured between three different layers, and excessive section enlargement would occur. These reasons lead to the conclusion that the existing jacketing must be demolished to execute the new bridge strengthening.

Upon analyzing the large deformations observed and considering the structural scheme of the bridge, where the girder bases are horizontally restrained to each other, it was concluded that the superstructure of spans 1, 2, and 3 might be experiencing excessive stresses due to some settlement of the southern abutment. This condition makes a passive jacketing ineffective, as it does not improve the bridge's performance under service conditions. An active jacketing, with girder prestressing, appears to be more suitable, as it could relieve the excessive stresses likely present and also recover part of the deformations. However, the current bridge support system prevents any prestressing, as the forces would be transferred to the abutments, which are much stiffer than the girder ties.

To enable effective prestressing, flexible or sliding bearings were introduced, and existing reinforced concrete hinges were cut. This approach ensured that prestressing forces acted efficiently on the girders, reducing service stresses and recovering part of the deformations. Additionally, new corbels were created at the tops of the piers and abutments to accommodate the new support devices.

Figures 7 to 9 illustrates the proposed design for superstructure strengthening.

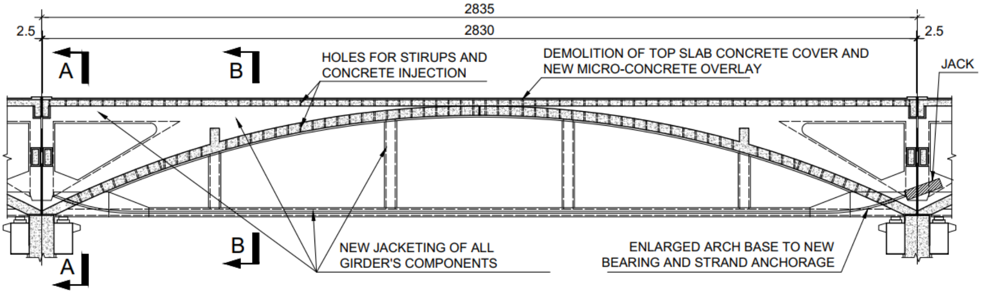

Fig. 7 Longitudinal section of strengthened girder (dimensions in cm).

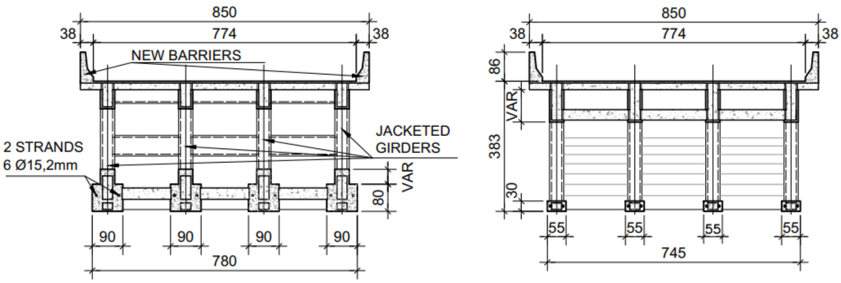

Fig. 8 Section A-A e section B-B (dimensions in cm).

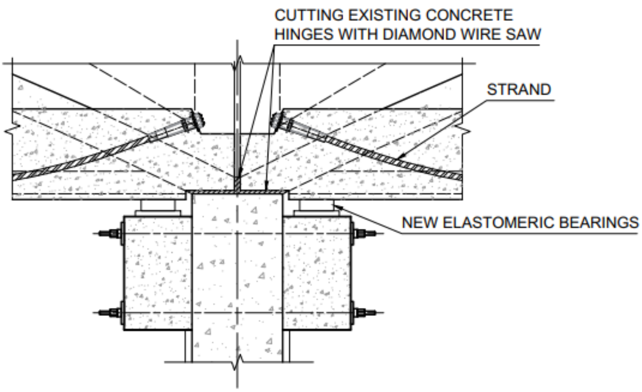

Fig. 9 Superstructure hinge cutting and new elastomeric bearing installation.

4.2 Foundations

The foundations of the central piers are composed of drilled shafts seated on sandstone. Underwater inspections indicated that these shafts lack peripheral reinforcement and are suffering erosion from the water current, particularly at pier P1 (south), where approximately half of the concrete section may have been lost. This foundation condition led to the bridge's closure and an emergency recommendation for repairs to the most critical shafts.

It was not possible to determine the exact depth of the drilled shafts or the material they rest on. Therefore, their safety for the superstructure's new loads and traffic conditions under TB450 cannot be evaluated. As these elements are made of plain concrete, any load increase beyond their capacity could lead to sudden failure. Given the unknown safety factor and the risk of abrupt collapse, new foundation elements were constructed to increase resistance.

The adopted approach was to assume a current safety factor of 1.0 (since no failure has occurred) and design additional foundations to bring this factor up to 2.0 in the ultimate limit state. The new foundation elements were designed to take on 50% of the current vertical load and absorb all horizontal loads and moments, ensuring adequate safety while optimizing materials.

Additionally, the new piles enabled the repositioning and modification of the support devices, allowing the new foundations to absorb the new eccentricities and horizontal loads that were no longer fully absorbed by the abutments.

The chosen construction method involved root piles drilled into the rock, positioned around the existing shafts. To address the water column, permanent steel casings were used.

At the abutments, new foundation elements were designed to support the newly introduced transverse beams, preventing excessive cantilever action that would generate significant bending moments on the walls. This reduced both vertical and horizontal support loads, improving safety margins.

Figure 10 illustrates the proposed design for foundation strengthening.

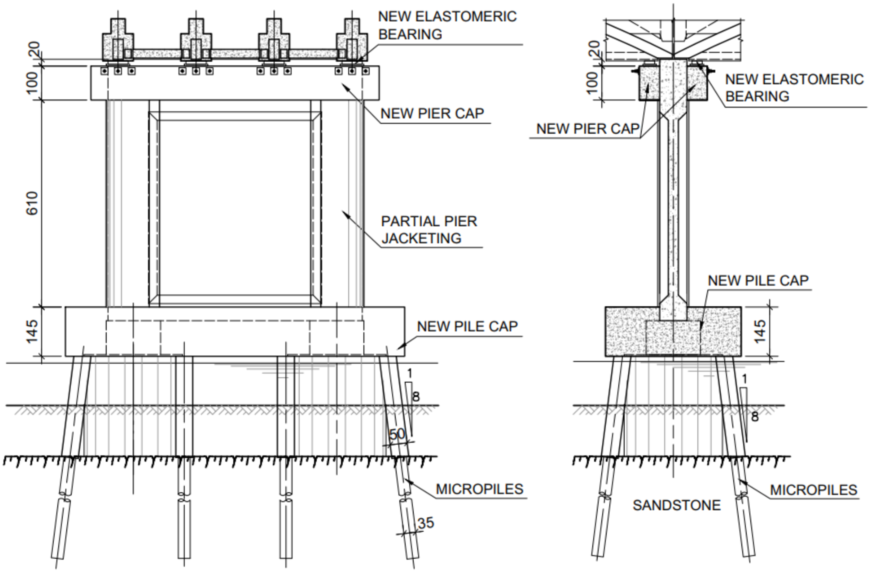

Fig. 10 Intermediate supports strengthening.

5 CONCLUSIONS

Some key points on bridge reinforcement design concepts can be drawn from this work:

- Bridge superstructures with concrete hinges are highly sensitive to support displacements. Additionally, they prevent reinforcements that involve longitudinal prestressing of the deck. When dealing with old bridges with this type of support device, a solution is to create support bases for laminated elastomeric bearings. Once the bearings are installed, the concrete hinges can be cut using diamond wire saws. This ensures that deformations imposed on the infrastructure generate minimal reactions in the superstructure and vice versa.
- Jacketing with new reinforcement bars is a well-established concrete structure strengthening technique. However, the surface treatment between the old and new concrete is crucial to ensure that the reinforcement is effective rather than merely adding dead weight to the original structure.

- In cases of widening or increasing the live load capacity of bridges with unknown foundations, a rational and conservative criterion is to assume that the foundation is operating with a safety factor of 1.0 (it cannot be lower than this, as the structure is still standing). From this assumption, complementary foundation reinforcement is designed to achieve a total safety factor appropriate for the new loads. In practice, this leads to considering approximately 50% of the operational capacity of the existing foundations. This is an unquestionable premise from a safety standpoint.

The strengthening work on the bridge over Rio Vermelho was completed in August 2023. Figure 11 shows a photo of the bridge after the reinforcement.

Fig. 11 Bridge photo after new strengthening.

Acknowledgements

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References

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