Rehabilitation for shear strength of prestressed concrete girders and continuity reinforcement of bridge deck slabs along with full load test verification

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- This paper presents a case study of a highway bridge rehabilitation project in India. Shortly after opening, the bridge experienced significant damage, including shear cracks near the ends of almost all girders and flexural cracks in the deck slab.

- Through the rehabilitation efforts, external steel plates were installed on the girder webs near the supports to enhance shear strength and a reinforced concrete overlay with negative moment reinforcement was added to the deck slab to improve continuity and flexural capacity.

- This study provides practical insights into the performance and durability of bridge rehabilitation measures under real-world conditions.

Structural rehabilitation of bridges is essential to ensure safety, serviceability, and an extended operational life, particularly for structures exhibiting early signs of distress. This paper presents a study focusing on the investigation, analysis, fullload testing, and structural rehabilitation of two adjacent bridge structures along the north-south corridor of the national highway network in India. The overpass bridges sustained significant structural damage shortly after opening, leading to their immediate closure. The study shows the challenges posed by structural distress arising from discrepancies between intended and actual construction sequences, deficiencies in shear and negative continuity moment reinforcement, and their compounded effects on structural performance.

Each overpass bridge is approximately 500 m (1640 ft) long and consists of two separate structures for right- and left-hand traffic, along with an additional span for road and rail crossings. The bridge structures comprised a total of 380 precast, pretensioned concrete I-shaped girders, designed specifically for this rehabilitation project. These simple-span pretensioned girders, after being launched and positioned on temporary bearings, were made continuous for superimposed dead loads and live loads through cast-in-place diaphragms integrated with the deck slab, resulting in a connected unit of three consecutive spans of 25 m (82 ft) each. Once the deck slab hardened, these girders functioned as continuous girders for all subsequent loads.[1]

Observed distress included inclined web shear cracks near the ends of almost all 380 girders and flexural cracks in the deck slab perpendicular to the carriageway, raising concerns about the structural integrity and load-carrying capacity of the bridges. Taking the observed distress as an indication of a serious underlying problem, a detailed investigation was carried out with three aims:

- to assess the existing condition of the overpass bridge
- to identify the underlying deficiencies
- to develop an appropriate strengthening plan to restore strength and functionality

The analyses revealed that the observed damage was primarily caused by discrepancies between the proposed construction sequence during design and the sequence followed on-site. In addition, shear strength deficiencies in the prestressed concrete girders due to excessive debonding of prestressing strands and inadequate shear reinforcement, along with insufficient continuity reinforcement in the deck slab, compounded the structural issues. Review of traffic records and axle-load surveys confirmed that the bridge was not subjected to overloads during its early service life. The demands due to most common three-axle commercial vehicles weighing approximately 350 kN (78.5 kip) in two lanes and those weighing approximately 700 kN (157 kip) in one lane were lower than the demands due to one lane of design vehicle (IRC 70R wheeled vehicle) weighing approximately 1000 kN (225 kip). Thus, the distress was not due to excessive loading but rather to the identified structural deficiencies under design-level loads.

To address these deficiencies, a customized rehabilitation plan was developed without altering the overall load-resisting mechanism or affecting the available benefits of prestressing. The measures included installing external steel plates on the girder webs near the supports to enhance shear strength and adding a reinforced concrete overlay with negative moment reinforcement to the deck slab to improve continuity and flexural capacity. The effectiveness of these strengthening measures was validated through a full-scale load testing program under prescribed design loads. By documenting the challenges, solutions, and outcomes, this study provides practical insights into the performance and durability of bridge rehabilitation measures under real-world conditions. Through these findings, the paper aims to contribute to the broader understanding of these practices and guide future design and construction processes.

Brief description of overpass bridge

The overpass is constructed for two lanes of traffic, having a carriageway width of 8.5 m (27.9 ft) along with a 1.25 m (4.1 ft) wide safety curb on one side and crash barriers on both sides (**Fig. 1**). It consists of two parallel bridge structures, each dedicated to one direction of traffic, with a total length of approximately 500 m (1640 ft). The bridge is made up of a series of three continuous span units, each having a span length *L* of about 25 m (82 ft). Each span is supported by a system of five girders. The superstructure is composed of a total of 380 precast, pretensioned concrete I-shaped double-bulb girders. The cross-section detail of the prestressed concrete I-girders is shown in Fig. 1. These simple-span girders were made continuous over the three 25 m spans using a cast-in-place concrete deck slab with continuity diaphragms.

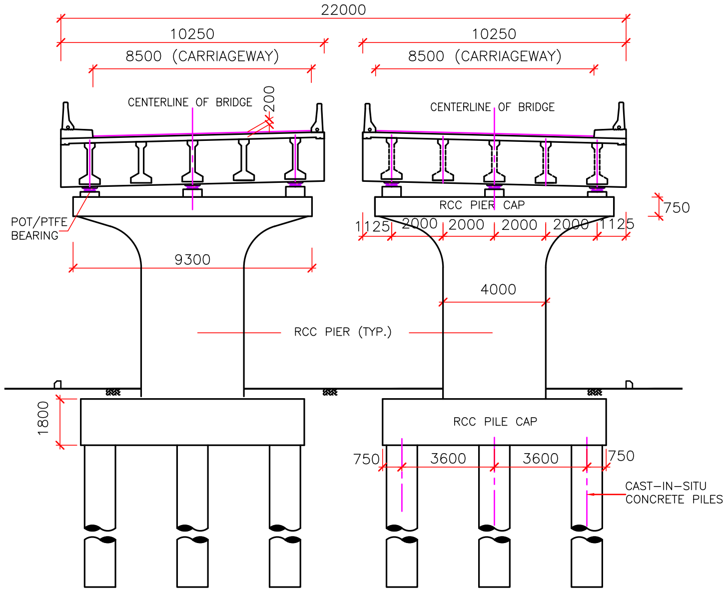
These bridge structures were designed as simple spans for dead loads of girders, deck, and diaphragms while behaving as continuous spans for live loads and superimposed dead loads. Each prestressed concrete girder was prestressed by pretensioning seven-wire, 15.2 mm (0.6 in.) diameter strand steel of Class 2 type conforming to Indian Standard (IS) 14268.[2] The ultimate tensile strength of the strand is 260.7 kN (58.6 kip); the strands were tensioned up to a force of 189 kN (42.5 kip). The cross section of the girder at midspan, the profile of prestressing cables, and the details of bonded and debonded strands are shown in **Fig. 2**.

The bridge was designed per Indian highway bridge code Indian Roads Congress (IRC) 6[3] for load cases A and B and checked for load case C as follows (**Fig. 3**):

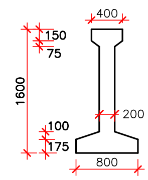
- load case A: one lane of IRC 70R tracked vehicle with two tracks each weighing 350 kN (78.7 kip)
- load case B: one lane of IRC 70R wheeled vehicle with a train of vehicles on seven axles with a total load of 1000 kN (224.8 kip)
- load case C: two lanes of IRC Class A loading, which is a train of wheeled vehicles on eight axles weighing a total load of 554 kN (124.6 kip)

Observed distress and its causes

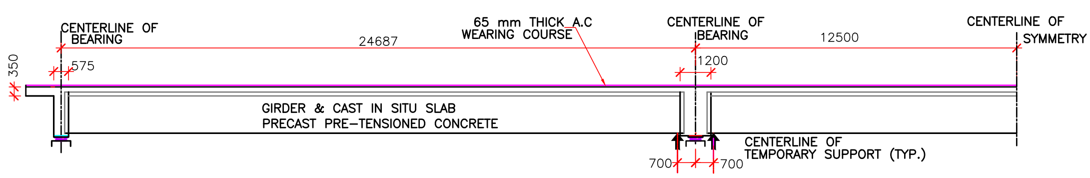
Within three months of being commissioned, the structure experienced visible distress in the form of inclined web cracks near the ends of nearly all girders and transverse cracks in the deck slab running perpendicular to the carriageway (**Fig. 4**). A close inspection of girder cracks was performed, and the maximum crack width of about 0.45 mm (0.018 in.) was measured using a crack detection microscope. Figure 4 shows a cluster of downward-inclined cracks in one of the observed spans. Following the observation of distress, the bridge was immediately closed to traffic for safety. The rehabilitation project commenced shortly thereafter. Based on the nondestructive and concrete tests, the compressive strengths of M45 grade concrete were a characteristic compressive strength of cube at 28 days of 45 MPa (6.5 ksi) and cylinder strength of 36 MPa (5.2 ksi) for the prestressed concrete girders and the M35 grade concrete for the deck slab had a compressive strength of 35 MPa (5.1 ksi). These concrete compressive strengths were considered adequate for the assumed structural design requirements, which ruled out any material deficiencies as the cause of failure. Furthermore, no visible



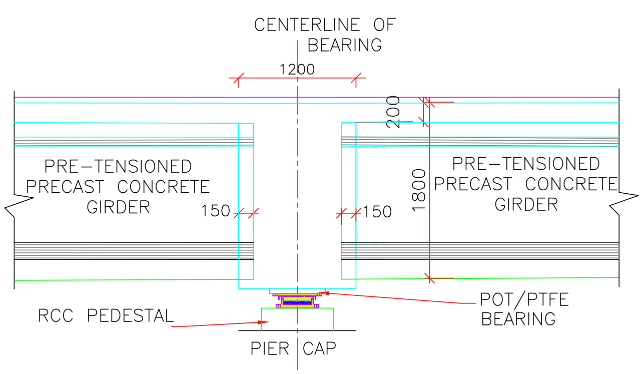
Typical cross section details



Cross section of the precast concrete girder at midspan



Longitudinal section



Details of the diaphragm at the continuous support

View of the right hand side and left hand side carriageway of the overpass structure composed of three-span modules of simple-span precast, prestressed concrete girders made continuous and an expansion joint between three-span modules over the pier

Figure 1. Details of bridge overpass. Note: All dimensions are in millimeters. A.C. = asphalt concrete; LHS = left hand side; POT = pot type bearing; PTFE = polytetrafluoroethylene bearing; RCC = reinforced cement concrete; RHS = right hand side; Typ. = typical. 1 mm = 0.0394 in.

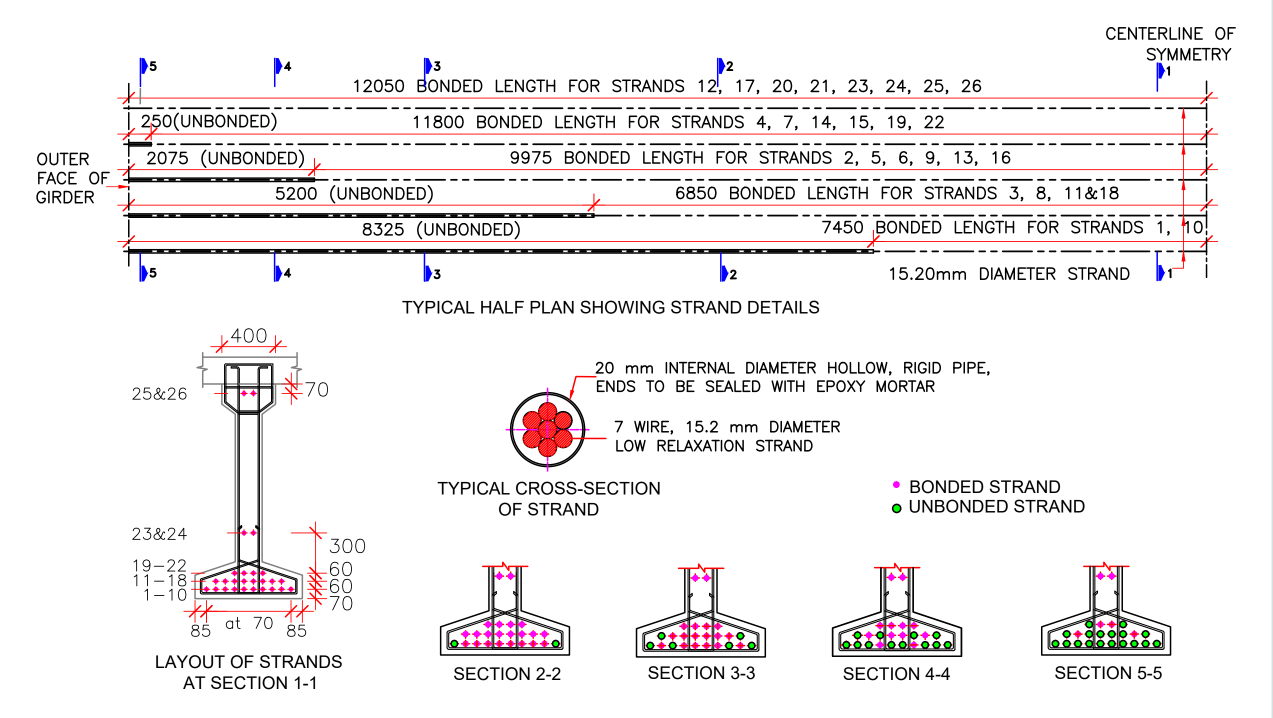


Figure 2. Profile of prestressing strands in precast concrete I-girder. Note: All dimensions are in millimeters. 1 mm = 0.0394 in.

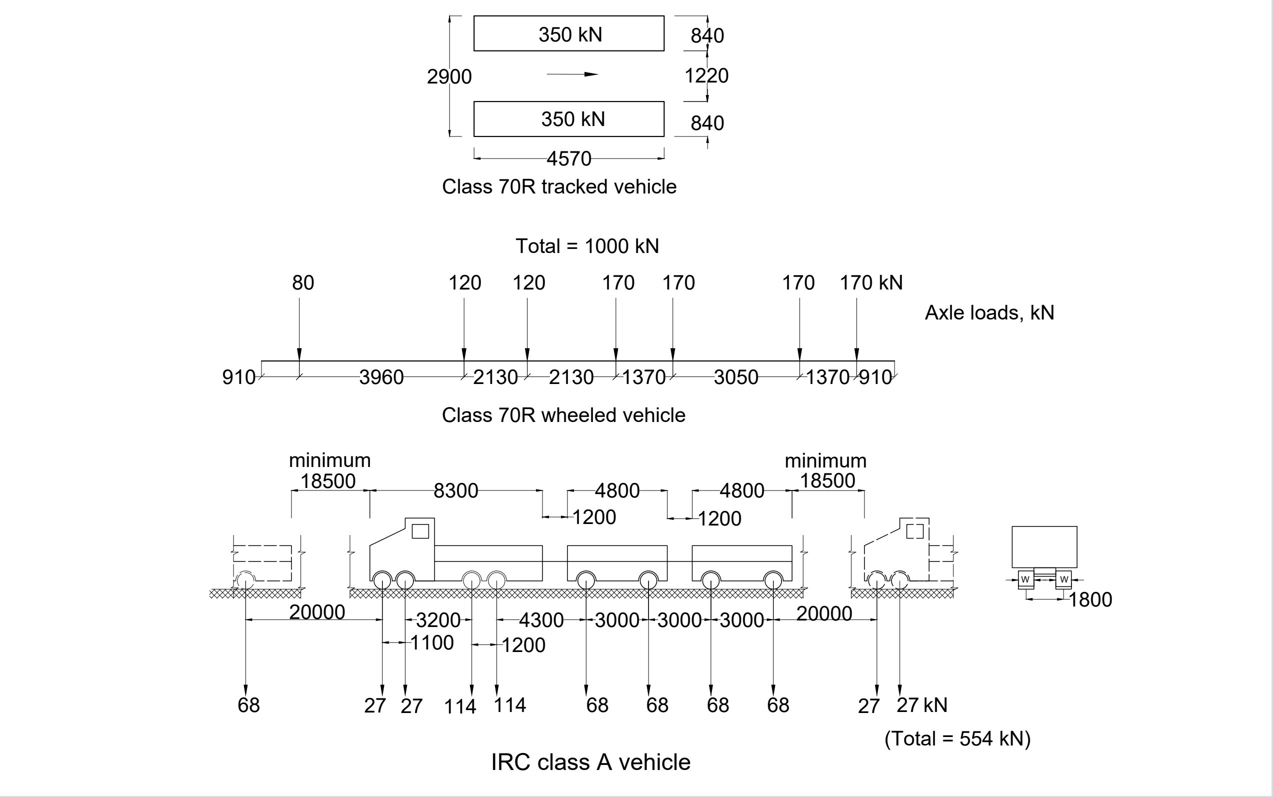


Figure 3. Vehicular loading for bridge design as per Indian Roads Congress. Note: All dimensions are in millimeters. 1 mm = 0.0394 in.; 1 kN = 0.225 kip.

Inclined cracks in the web of a prestressed concrete girder near its ends

Transverse cracks typically observed on the soffit of the reinforced concrete deck slab between girders

Figure 4. Cracks that formed within three months of conditioning. Note: Cracks in images have been enhanced for visibility.

cracks were observed in the diaphragm during site inspection, indicating that the diaphragm size and reinforcement were adequate to resist the negative moments transferred to it under the actual construction sequence.

To compute the forces and stresses resulting from various loads and their combinations, the three-span module of the bridge structure of simple-span prestressed concrete girders made continuous was modeled using bridge modeling software (**Fig. 5**).

Difference in sequence of construction

Typically, precast, prestressed concrete girders are initially designed to carry their self-weight as simply supported members. After placement, the deck slab and diaphragms are cast monolithically, providing the required reinforcement to transform the system into a composite, continuous structure. This continuity is intended to be effective primarily for superimposed dead loads and live loads.[4] Prior to composite action, however, the structure behaves as simply supported under the dead weight of the girders, deck, and diaphragms.

A major discrepancy was observed between the intended and implemented construction sequences. The design calculations assumed that the deck slab would be placed while the girders were still simply supported, with continuity established only after deck and diaphragm placement (intended sequence). In actual construction, however, the diaphragm was placed several days before the deck slab, resulting in the girders developing partial continuity through the diaphragms prior to deck placement (implemented sequence). Positive moment

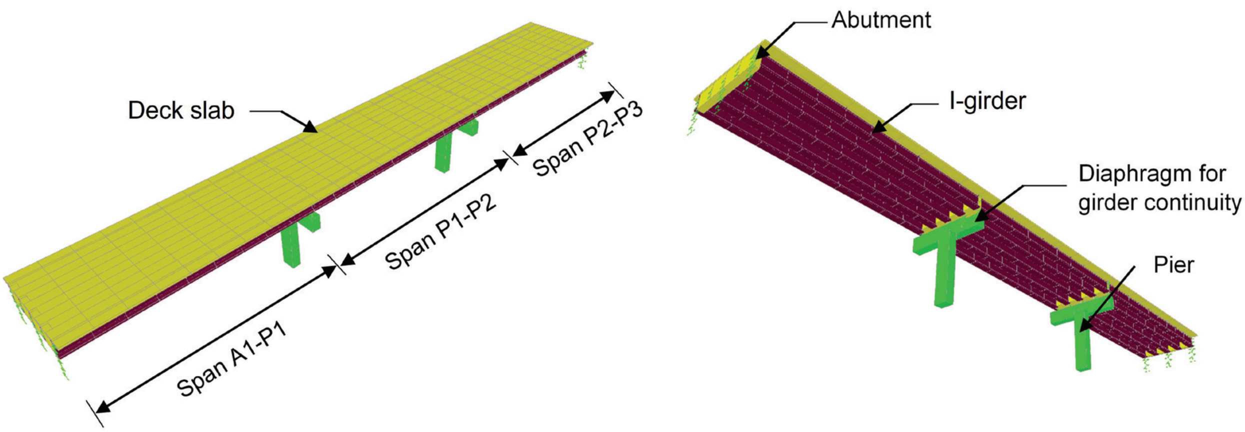


Figure 5. Finite element idealization of three-span bridge modeling software. Note: A1 = abutment number 1; P1 = pier number 1; P2 = pier number 2; P3 = pier number 3.

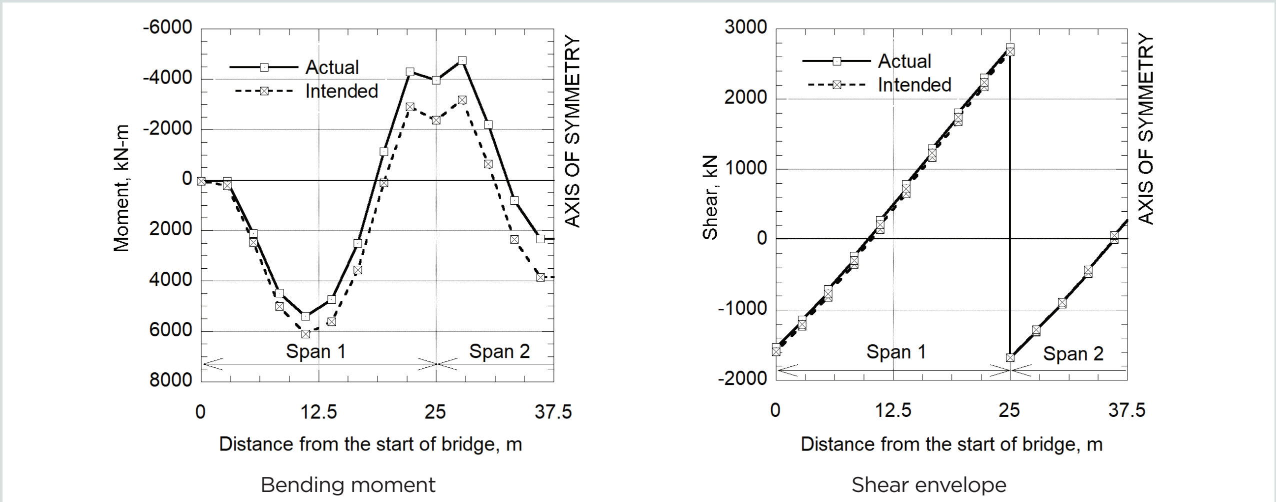


Figure 6. Comparison of bending moment and shear envelopes for intended and actual (implemented) construction sequences at the ultimate load combination considering dead load, live load, superimposed dead load, and prestress. Note: 1 m = 3.281 ft; 1 kN = 0.225 kip; 1 kN-m = 0.7376 kip-ft.

continuity between the adjacent precast concrete girder spans was achieved through a cast-in-place reinforced concrete diaphragm of 1200 mm (47.2 in.) width with the girder ends embedded 150 mm (5.9 in.) into the diaphragm (Fig. 1).

The influence of this sequence change was quantified by analyzing both sequences. For the ultimate load combination considering dead loads, live loads, superimposed dead loads, and prestress, the implemented staging led to an approximate 53% increase in hogging (negative) moments at the pier due to partial fixity and secondary restraint effects, whereas shear forces were largely unchanged (**Fig. 6**). Because stagewise analysis was not performed during the original design, these moment differences were not captured, highlighting the need for explicit stage-construction checks and diaphragm-reinforcement verification in design practice to ensure structural safety.

In Fig. 6, the implemented sequence accounts for staged construction (diaphragm placed before the deck) and time-dependent effects, resulting in higher hogging moments at the pier.

Shear strength deficiency in prestressed concrete girders

The prestressed concrete girders of the bridge structure had inadequate shear capacity due to excessive debonding of prestressing strands and underestimation of shear forces in outer girders. It should be noted that debonding of strands is typically not a preferred choice for structural integrity; however, debonding is used as an economical way to design many similar girders. The current Indian highway bridge code, IRC 112,[5] has introduced a quantitative limit on debonding through its special publication, IRC SP-71,[6] restricting it to 33%. Earlier versions of the code, however, did not provide any specific quantitative guidelines, allowing debonding as needed to balance external moment demands. This lack of restriction resulted in some bridge project designs before 2020 incorporating debonding levels exceeding 80% near girder ends to counterbalance low bending moments in these areas. Such excessive debonding contributed to issues such as inclined cracking, even at early ages of service.

In the absence of explicit national guidelines, bridge designers often relied on international standards, which also vary significantly in the limits imposed on strand debonding. For example, in the 2017 American Association of State Highway and Transportation Officials' *AASHTO LRFD Bridge Design Specifications*[7] strand debonding was limited as follows:

- 25% of all strands
- 40% of strands in a given horizontal row
- no more than 40% of debonded strands or four strands, whichever is greater, at a section
- no exterior strands in any row, citing the study by Shahawy et al.,[8] which showed that full-scale girders with 40% debonded strands lacked sufficient shear capacity

The 2024 AASHTO LRFD specifications[9] extends the limit to 45% per row and recommends bonding all strands within the horizontal limits of the web if more than 25% of the total strands are debonded in single-web flanged sections. Similarly, Shahrooz et al.[10] permits higher debonding but with specific restrictions:

- up to 60% of all strands
- up to 80% of strands in a given horizontal row other than the bottommost row, which is limited to 50%

- no more than 40% of debonded strands or four strands, whichever is greater, at a section

- all exterior strands within a row must remain bonded

The 2019 *California Department of Transportation's California Amendments to AASHTO LRFD Bridge Design Specifications*[11] also imposes an overall debonding limit of 33% and restricts debonding in any horizontal row to no more than 50%.

The prestressed concrete girders in the overpass structure do not meet these provisions in detailing of prestressing strands (Fig. 2):

- At girder section 5-5, a shear-critical zone, 69% of all strands were debonded, exceeding the limits specified by AASHTO,[7] IRC,[6] and Shahrooz et al.,[10] which restrict debonding to 25%, 33%, and 60%, respectively.

- At girder section 5-5, 100% of strands in the bottommost row were debonded, exceeding the Shahrooz et al.[10] limit of 50% and at section 4-4, 80% of strands in the bottommost row were debonded, which is the Shahrooz et al.[10] upper limit.

- At girder section 5-5 and in the bottom two rows at sections 3-3 and 4-4, all exterior strands were debonded, contrary to the prohibition in both AASHTO[7] and Shahrooz et al.[10] guidelines.

In addition, strands located within the flange along the web width were also debonded. These design oversights likely had significant implications for the shear capacity of the girders. Excessive debonding reduces the concrete's contribution to shear resistance, as confirmed by experimental findings from various researchers, including Shahawy et al.,[8] Shahrooz et al.,[10] Krishnamurthy,[12] Nagle and Kuchma,[13] and Bolduc.[14] For instance, Wesson[15] reported a 35% reduction in shear strength in test specimens with 50% debonding compared with specimens with no debonding and increasing the debonding to 75% led to a 61% reduction in shear strength.

The moment and shear envelope of each girder type (inner and two outer at either end) at ultimate load combination considering dead loads, live loads, superimposed dead loads, and prestress are shown in **Fig. 7**. It can be clearly seen that there is a significant difference in moment and shear demands between inner and outer girders. However, in the design, shear force corresponding to an inner girder was used for estimating the transverse shear reinforcement. Moreover, it was found that the provided shear reinforcement was about 20% less than that required for a typical inner girder (11.3 cm^2/m [0.53 in.^{2}/ft] instead of 13.6 cm^2/m [0.64 in.^{2}/ft]). This discrepancy becomes more severe for the outer girders, which carry about 25% more shear force than the inner girders. This observation is further confirmed by more significant distress observed in outer girders.

In addition to the previously mentioned shear deficiency, the maximum shear stress in the girder approaches the allowable limits. At a critical section, 1.3 m (4.3 ft) from the centerline of the bearing, the ultimate shear demand V_u for the right outer girder was found to be 1802 kN (405 kip), as shown in Fig. 7, and the corresponding shear stress, obtained as 5.2 MPa (0.75 ksi), marginally exceeds the allowable limit of 5 MPa (0.725 ksi) for a prestressed concrete beam of M45 grade concrete per IRC 18.[16] As previously discussed, the lack of prestress due to excessive debonding in the shear-critical region significantly reduces the shear capacity of concrete and the allowable limit

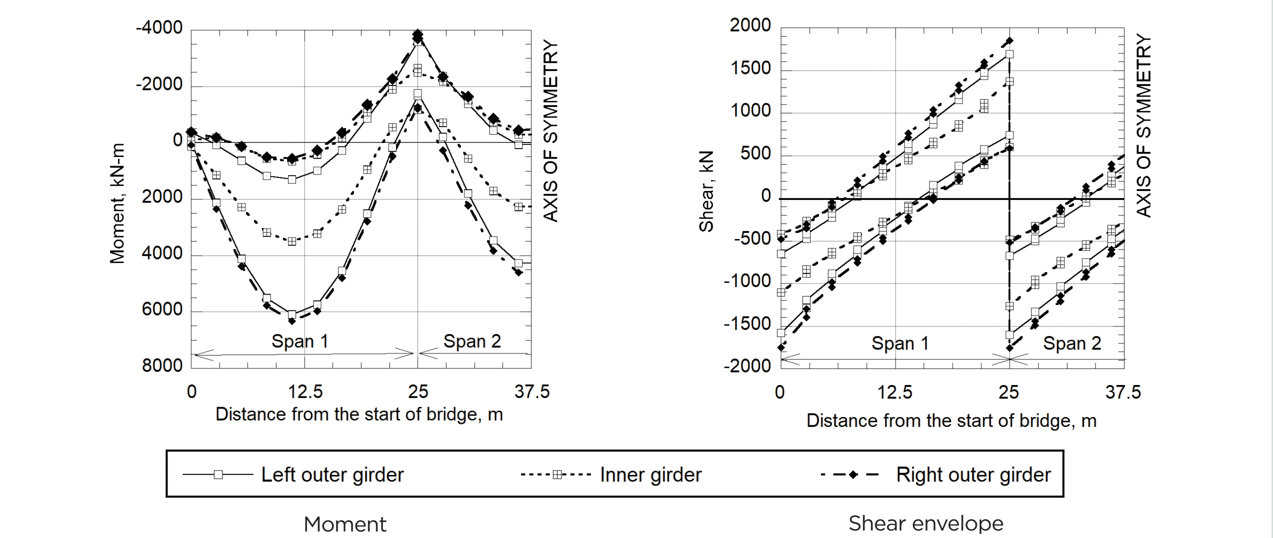


Figure 7. Moment and shear envelope of each girder type at ultimate load combination considering dead load, live load, superimposed dead load, and prestress. Note: 1 m = 3.281 ft; 1 kN = 0.225 kip; 1 kN-m = 0.7376 kip-ft.

of 5 MPa may be inapplicable at these sections. According to the 2024 AASHTO LRFD specifications,[9] the maximum allowable shear stress is 5.5 MPa (0.8 ksi) for the fully bonded case, which drastically reduces to 2 MPa (0.3 ksi) for a girder with 69% of the strands debonded. Similarly, per IS 1343:2000,[17] the allowable shear stress is 4.8 MPa (0.7 ksi) for fully bonded girders, but this value decreases to 3.4 MPa (0.5 ksi) when 69% of strands are debonded. These reductions emphasize the detrimental effect of debonding on structural performance. Further, the shear stress levels in the girder could have been reduced if the recommendation for web thickening at the ends, as provided in IRC SP-71,[18] had been implemented.

The indiscriminate debonding of prestressing strands in the shear-critical region and underestimation of shear force accompanied by inadequate shear reinforcement contributed to the shear cracking observed in the girder webs. These deficiencies in shear capacity can be compensated for either by increasing the transverse reinforcement or by thickening the web of the prestressed concrete girder in the shear-critical region.

Inadequacy of negative continuity reinforcement of deck slab

A major deficiency in the layout of negative continuity reinforcement in the deck slab was noticed. In the original structural drawings, the main deck slab reinforcement was curtailed at 3.5 m (11.5 ft) from the centerline of bearing support, a distance insufficient for most loading situations. This curtailment appears to have been underestimated for the minimum negative moments derived from the moment envelope at the continuous supports. For optimal performance, the longitudinal reinforcement resisting the negative design moments must be anchored into the deck slab concrete, which remains in compression. Therefore, this negative moment reinforcement should have been curtailed at a point at least one development length beyond the theoretical point of inflection for the negative moment envelope. In this case, with a development length of 1.0 m (3.3 ft), the effective anchorage length reduces to 2.5 m (8.2 ft), which is inadequate to resist the design moments.

For a load case of maximum negative moment at the continuous supports, the theoretical cutoff point for negative moment reinforcement is 5.6 m (18.4 ft) for outer spans (**Fig. 8**). To ensure sufficient anchorage, the bars should actually be curtailed at a point at least one development length distance (about 1.0 m [3.3 ft]) beyond the theoretical cutoff point for the negative moment. A shorter curtailment of negative continuity reinforcement at 3.5 m (11.5 ft) (effective 2.5 m [8.2 ft] anchorage) from the centerline of the bearings indicates that a part of the deck slab in tension did not have adequate reinforcement, which may have been a major factor contributing to the transverse cracks in the deck slabs.

Although the Indian code acknowledges the concept of partial continuity, it does not provide explicit provisions for designing continuity reinforcement, which may lead to potential shortcomings in addressing such critical design requirements. In the absence of well-defined domestic guidelines, designers often turn to international specifications for guidance.

Rehabilitation plans

The deficiencies in the web shear strength of the prestressed concrete girders and negative continuity reinforcement of the deck slab were the major causes of distress in the bridge. These issues are not related to the prestressing resistance mechanism and can be addressed by increasing the strength of the existing girder and deck slab, as well as enhancing the cross-sectional resistance using new materials or members integral with the existing ones. The proposed strengthening plan consisted of the following key interventions:

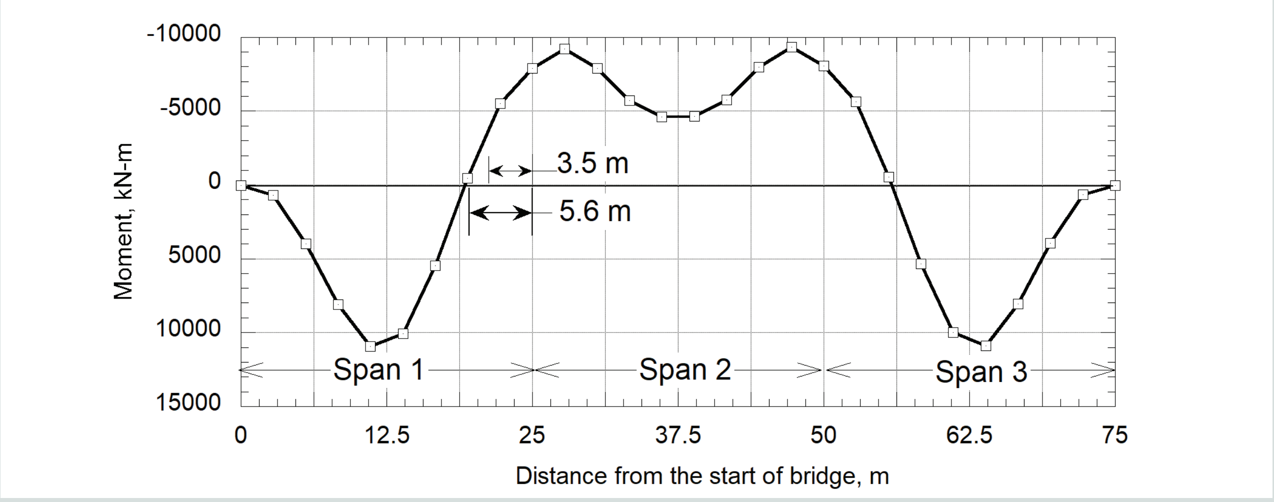


Figure 8. Bending moment diagram for entire bridge section showing the theoretical point of cutoff at 5.6 m and actual point of cutoff at 3.5 m for negative moment reinforcement. Note: 1 m = 3.281 ft; 1 kN-m = 0.7376 kip-ft.

- low-viscosity epoxy injection to seal the existing cracks, ensuring structural integrity

- the installation of externally bonded and bolted steel plates to the girder web to enhance shear capacity and mitigate existing deficiencies

- the addition of a new 100 mm (3.4 in.) thick concrete overlay to the existing reinforced concrete deck slab (after removing the existing wearing surface) to improve the flexural strength of the deck and install new continuity reinforcement of sufficient anchorage length

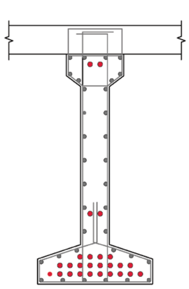
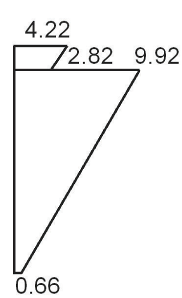
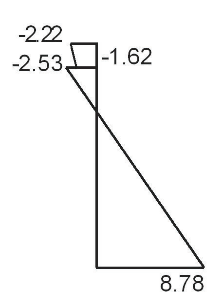
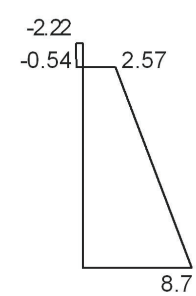
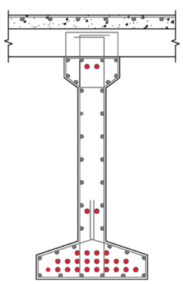
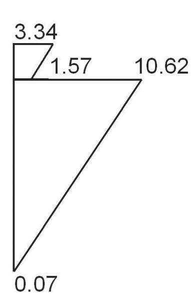
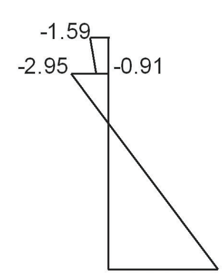
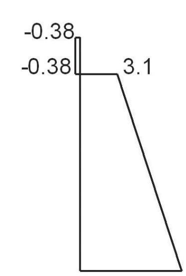
Further, the proposed strengthening plan did not significantly alter the global stress pattern, and therefore it is unlikely to reduce the beneficial action of prestressing the girders. Analyses of the bridge structure with the additional deck slab overlay were carried out, and only minor changes were noticed in the stresses after strengthening of the deck (**Table 1**). The total stresses across the existing and retrofitted section were computed at three locations along the girder for the ultimate load combination considering dead loads, live loads, superimposed dead loads, and prestress. Moreover, the compressive stress values satisfy the permissible compressive stress limit of $0.33f_{ck}$ per IRC 18[16] for both the existing and retrofitted section. The details about the design of these strengthening techniques are described in the following sections.

External steel plates on girder webs for shear strengthening

To address the shear deficiencies identified in the prestressed concrete girders, 5 mm (0.2 in.) thick external steel plates conforming to IS 2062[19] of E250 grade were attached to both sides of the girder web. The plates were bonded using a combination of epoxy adhesive and 16 mm (0.6 in.) diameter high-tensile-strength bolts to enhance shear strength while maintaining the original load-resisting mechanism of the girder. This dual anchorage approach—epoxy bonding for improved surface crack control at low stress levels (serviceability state) and bolted connections for effective load transfer at higher stress levels (ultimate capacity)—was indicated by previous experimental studies[20,21] and ensures a durable composite action.

The plate thickness and bolted connection details were designed following the procedure proposed by Adhikary and Mutsuyoshi[20] for externally bonded systems. Based on this method and considering M16 Grade 8.8 bolts in double shear, the required number and spacing of bolts were determined to ensure adequate shear transfer between the girder web and external plates. The design resulted in two vertical rows of M16 bolts placed at 500 mm (19.7 in.) center-to-center spacing along each plate, which satisfies the spacing and load-transfer requirements recommended by Adhikary and Mutsuyoshi[20] to ensure uniform shear transfer and prevent local plate debonding or buckling. Concrete and steel surfaces

Table 1. Comparison of total stresses for existing and retrofitted sections of the girder for the ultimate load combination considering dead loads, live loads, superimposed dead loads, and prestress

Section		Section 1-1 (midspan)	Section 5-5 (1.3 m from centerline of bearing)	Section 6-6 (0.2*L* from support)
Existing				
Retrofitted				

Note: All dimensions are in megapascals. Stresses are shown at the top and bottom fibers of each section, measured up to the top of the deck slab for the existing case and up to the top of the new overlay for the retrofitted case. *L* = span length = 25 m. 1 m = 3.281 ft; 1 MPa = 0.145 ksi.

were cleaned, grit-blasted, and degreased prior to adhesive application. Low-viscosity epoxy (with a shear strength of approximately 10 MPa [1.45 ksi] and bond modulus 2 to 3 GPa [290 to 435 ksi]) was applied in a 1 to 2 mm (0.04 to 0.08 in.) thick uniform layer on both contact surfaces to ensure full composite action at service loads. This configuration provides uniform shear transfer and prevents local plate buckling or premature debonding. The steel plates provided an additional shear strength of approximately 460 kN (103 kip), thereby enhancing the overall shear capacity of the prestressed concrete girders by about 30%. The general configuration and key details are shown in **Fig. 9.**

All existing cracks in girders were sealed with low-viscosity epoxy injection prior to steel jacketing (**Fig. 10**). Through holes were drilled in girder webs at predetermined locations to install threaded bolts, which were torqued in three stages to 1 kN-m (0.74 kip-ft) to achieve positive connection. Both the concrete and steel surfaces were prepared by degreasing and grit blasting to remove loose particles. A uniform epoxy adhesive layer of 1 to 2 mm (0.04 to 0.08 in.) thickness was applied before bonding. A view of the strengthened girders is shown in Fig. 10.

Deck slab overlay for negative continuity moment enhancement

The existing deck slab reinforcement was curtailed prematurely, providing only 10 mm (0.4 in.) diameter bars at 200 mm (8 in.) center-to-center spacing extending over the required 6.6 m (21.6 ft) length, resulting in available flexural resistance M_{en} of

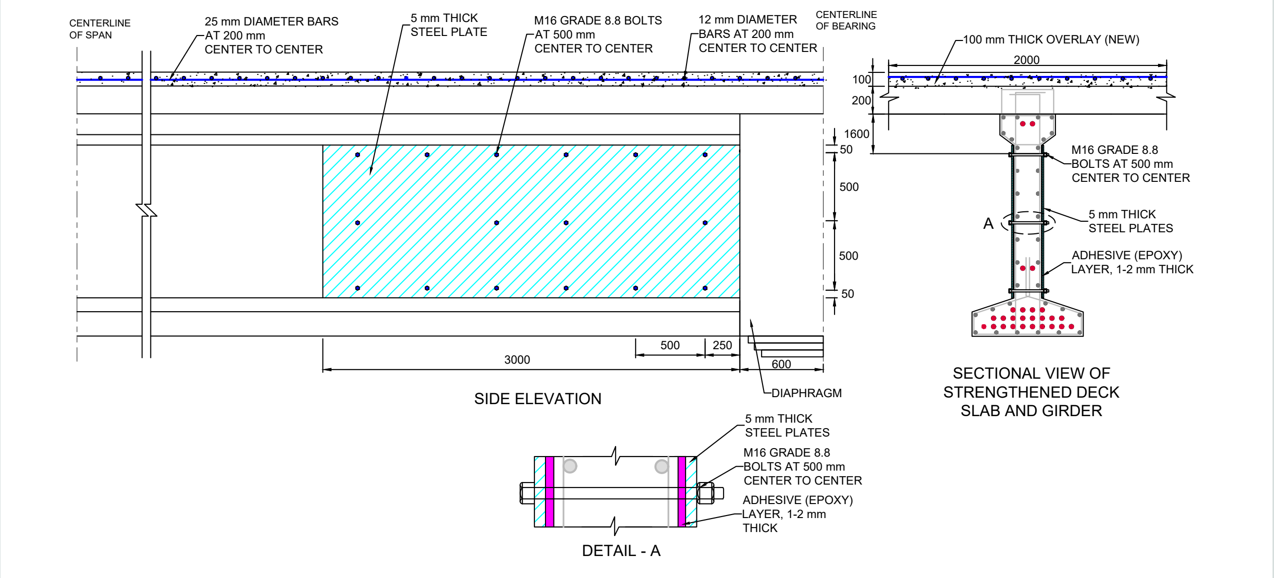


Figure 9. Details of shear strengthening of girder with 5 mm thick steel plates (E250 Grade) bonded with epoxy and anchored using M16 Grade 8.8 bolts at 500 mm spacing in two vertical rows on each side of the web, designed in accordance with Adhikary and Mutsuyoshi (2006). Note: All dimensions are in millimeters. 1 mm = 0.0394 in.; 1 MPa = 0.145 ksi.

Low-viscosity epoxy injection in existing cracks in girders

Steel plates attached to girder for shear strengthening

Scarified surface of deck after milling process

Figure 10. Details of strengthened girders.

248 kN-m (183 kip-ft). However, at the critical section of the right outer girder, the negative moment demand under service load M_n of 1695 kN-m (1250 kip-ft) far exceeded the available strength and the reinforcement corresponding to this moment must therefore extend 6.6 m from the support, which includes the inflection point from the centerline of the support plus an adequate anchorage length. To meet the flexural demand, additional reinforcement was provided in the form of 25 mm (1 in.) diameter bars at 200 mm center-to-center spacing in the 100 mm (4 in.) thick overlay. This reinforcement extended 7.5 m (24.6 ft) from the support and then curtailed with 50% of the bars being staggered (**Fig. 11**). The new overlay successfully achieved the required flexural capacity.

The existing 65 mm (2.6 in.) thick top bituminous wearing course was removed, and unsound concrete was marked using nondestructive techniques, such as sounding, and removed in patches. An additional 30 mm (1.2 in.) of sound concrete was also removed for better bonding with the new overlay. A specialized milling machine was used to ensure precise removal without disturbing the underlying concrete. The milling process left the surface scarified with grooves deeper than 3 mm (0.12 in.) at 25 mm (1 in.) spacing (Fig. 10). Past studies have shown that a concrete overlay on such a scarified surface will develop monolithic action up to the ultimate flexural capacity of the slab panel without the need for additional provision of dowels.[22]

Before placing the overlay, cracks wider than 0.2 mm (0.008 in.) were sealed with low-viscosity, freeflowing, high-strength, injectable epoxy. The reinforcement was then assembled in both longitudinal and transverse directions (along and across the bridge span) using the layout shown in Fig. 11. The surface was thoroughly cleaned, soaked to achieve a saturated surface-dry condition, and coated with high-strength epoxy to enhance the bond between the new and existing concrete. The wearing surface was laid in the usual manner after the placement of the overlay concrete.

Full load testing for effectiveness of the strengthening plan

The load test cases were devised for the single-lane loading of a Class 70R 1000 kN (225 kip)[3] wheeled vehicle as live load plus design impact load, to develop maximum shear forces and bending moments in the prestressed concrete girders of the bridge deck. Six load cases were developed:

- load case 1: maximum shear force at abutment location (A1–P1)
- load case 2: maximum hogging (negative) bending moment at support 2 (A1–P1)
- load case 3: maximum sagging (positive) bending moment between support 1 and support 2 (A1–P1 and P2–P3).
- load case 4: maximum hogging bending moment at support 3 (P2–P3)
- load case 5: maximum shear force at pier location at support 2 (P1–P2)

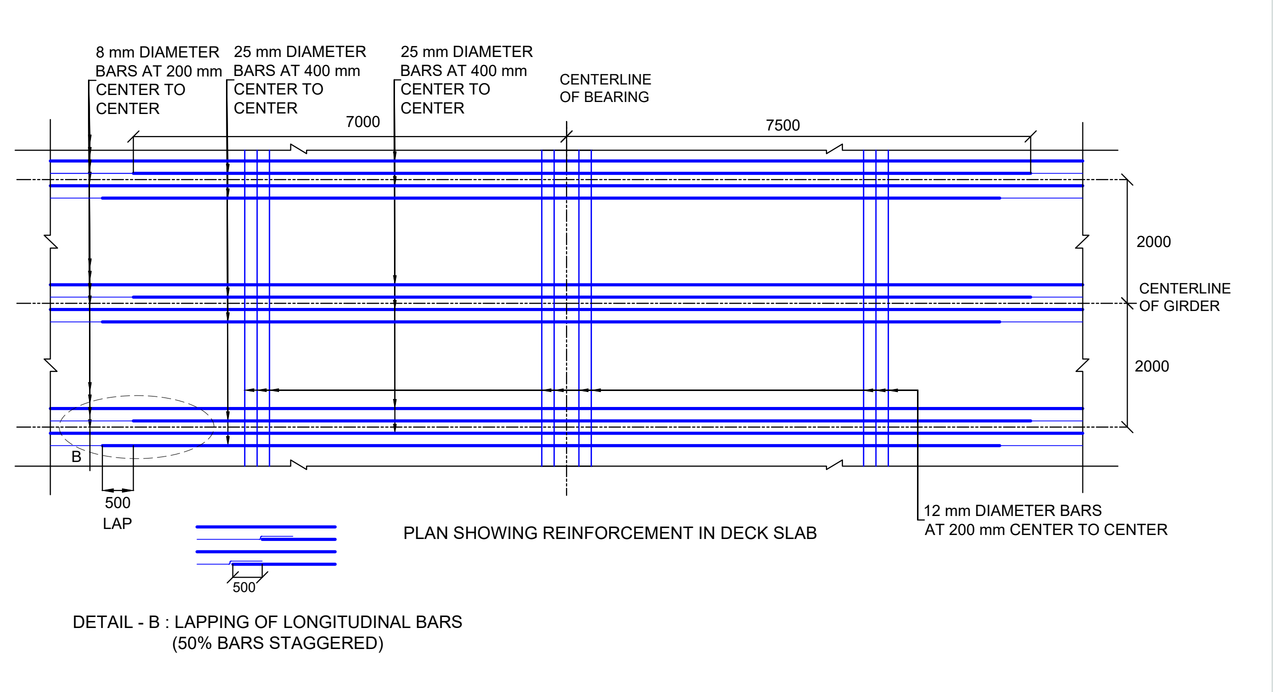


Figure 11. Reinforcement details for 100 mm thick deck slab overlay. Note: All dimensions are in millimeters. 1 mm = 0.0394 in.

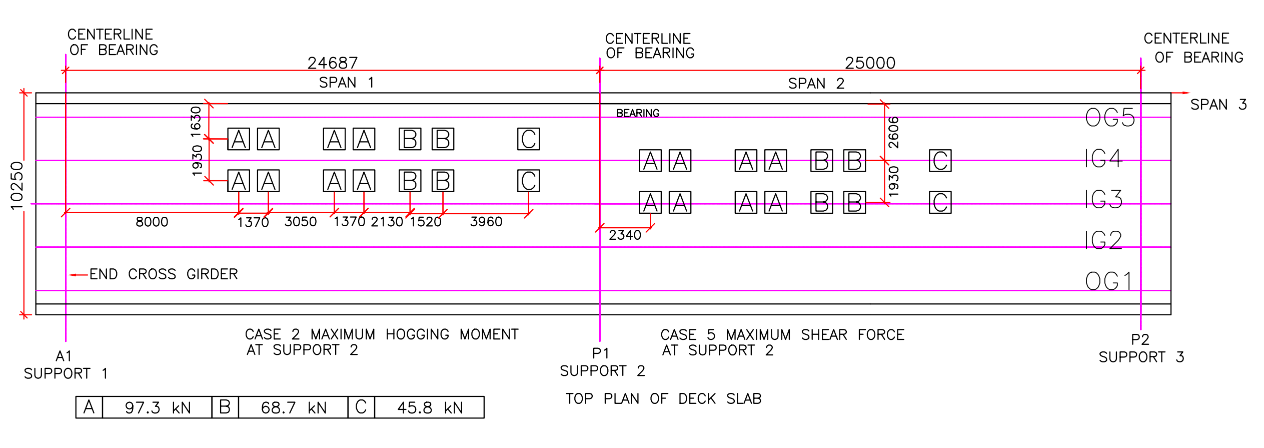


Figure 12. Positioning of the loading points for load cases 2 and 5. Note: All dimensions are in millimeters. A1 = abutment number 1; IG2 = inner girder number 2; IG3 = inner girder number 3; IG4 = inner girder number 4; OG1 = outer girder number 1; OG5 = outer girder number 5; P1 = pier number 1; P2 = pier number 2. 1 mm = 0.0394 in.; 1 kN = 0.225 kip.

- load case 6: maximum sagging bending moment between support 2 and support 3 (P1–P2)

Here, A1 denotes abutment 1, and P1, P2, and P3 denote pier 1, pier 2, and pier 3, respectively.

These load cases differ in the positioning of the loading points involving either one or two spans to create the maximum effect of internal actions considered in the design of the prestressed concrete girders at various critical sections. A schematic for positioning of the loading points for load cases 2 and 5 is shown in **Fig. 12**.

The bridge deck was loaded using electrohydraulic-operated jacks positioned at loading points designated per the loading pattern specified for each load case. The use of jacks permit-

View of the loading arrangement for the load test and electro-hydraulic jacks placed over the wheel imprints of Class 70R 1000 kN vehicle

Platform with dead weights below the deck and arrangement for measuring girder deflection using dial gauges

Arrangement for measuring girder deflection using dial gauges

Dial gauge

Electric resistance strain gauge rectangular rosette installed on steel plate

Figure 13. Typical loading arrangement and strain gauges to measure shear strains in steel plates for load cases 5 and 6. Note: 1 kN = 0.225 kip.

ted accurate application of loads during different phases of the load test, such as loading in the predetermined increments of 30%, 50%, 70%, 80%, 90%, and 100% of the total load; sustaining the full load for 24 hours, and then unloading in the same steps as the loading phase. The loads from jacks were resisted by the deadweight reaction system suspended below the deck. This system consisted of a kentledge platform with a total weight of 1150 kN (259 kip), supported by an arrangement of beams and tie rods. A typical loading arrangement is shown in **Fig. 13**.

Dial gauges of sufficient accuracy (0.01 mm [0.0004 in.]) were used to measure girder deflections at the quarter-span locations (*L*/4, *L*/2, and 3*L*/4) of each loaded girder (Fig. 13). These deflection measurements were made only for the loaded spans in each load case. For temperature corrections to these deflections, the girder deflections, along with the temperature measurements in the unloaded condition, were recorded from dawn to dusk in the days preceding the load test. The observed girder deflections with a change in the ambient temperature were used to correct the deflections observed during the load test by compensating for the thermal deflections. Calibrated thermocouples were used to monitor the temperature of the deck slab and prestressed concrete girders during the load testing.

In addition, strain gauges in the form of rectangular rosettes were installed to measure shear strains in the steel plates used for shear strengthening of the girder web (Fig. 13) during load cases 5 and 6.

The strain readings were sampled by an online, real-time, computer-controlled data-acquisition system during the load testing sequence. The positioning of various sensors used during load testing is shown in **Fig. 14**. No visible cracking or damage was noticed in the girder and deck slab during the entire testing program.

Load-deflection measurements

The load-deflection data for all six load cases were first corrected for the temperature variations as detailed in IRC SP-51.[23] Selected load-deflection plots, where the peak girder deflections were about 2 mm (0.08 in.) or more, are shown in **Fig. 15**. These plots clearly show the loading branch, the region for the sustained load, and the subsequent unloading branch. The corrected load-deflection data were then used to calculate the percentage recovery of girder deflections on the removal of load at the position of each dial gauge. According to IRC SP-51,[23] the minimum required deflection recovery for prestressed concrete members is 85% at 24 hours after removing the test load. **Table 2** shows the recovery percentages of girder deflections, with most locations achieving a recovery exceeding 93%. Several instances showed a full recovery of 100%, confirming that the acceptance criterion is met at most locations considered for all six load cases.

Strain measurements

The shear strain time history plots for strain gauge rosettes that showed maximum strain in the steel plates are shown in **Fig. 16**. Shear strain values increase as load increases, indicating that steel plates participate in resisting the applied load. The maximum shear strain values recorded during the loading phase were 137.93 and 162.86 microstrains near locations P1 and P2, respectively (Fig. 16). These strains correspond to shear stresses of 13 and 11 MPa (1.9 and 1.6 ksi) in the steel plates, indicating their participation in load sharing with the concrete web of the prestressed concrete girder.

Simulation analysis of load testing and comparison with observed data

The rehabilitated module of the overpass was modeled using bridge modeling software for numerical prediction of

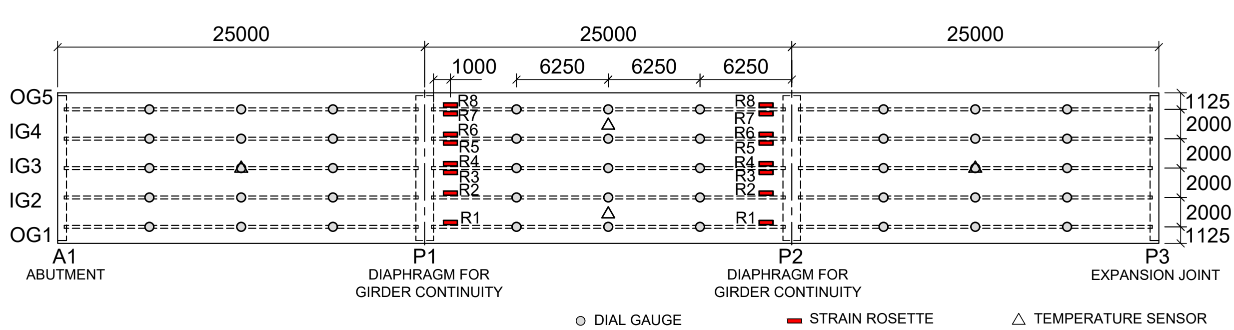


Figure 14. Schematic diagram of the positioning of various sensors installed to monitor response of the rehabilitated module. Note: All dimensions are in millimeters. A1 = abutment number 1; IG2 = inner girder number 2; IG3 = inner girder number 3; IG4 = inner girder number 4; OG1 = outer girder number 1; OG5 = outer girder number 5; P1 = pier number 1; P2 = pier number 2; P3 = pier number 3. 1 mm = 0.0394 in.

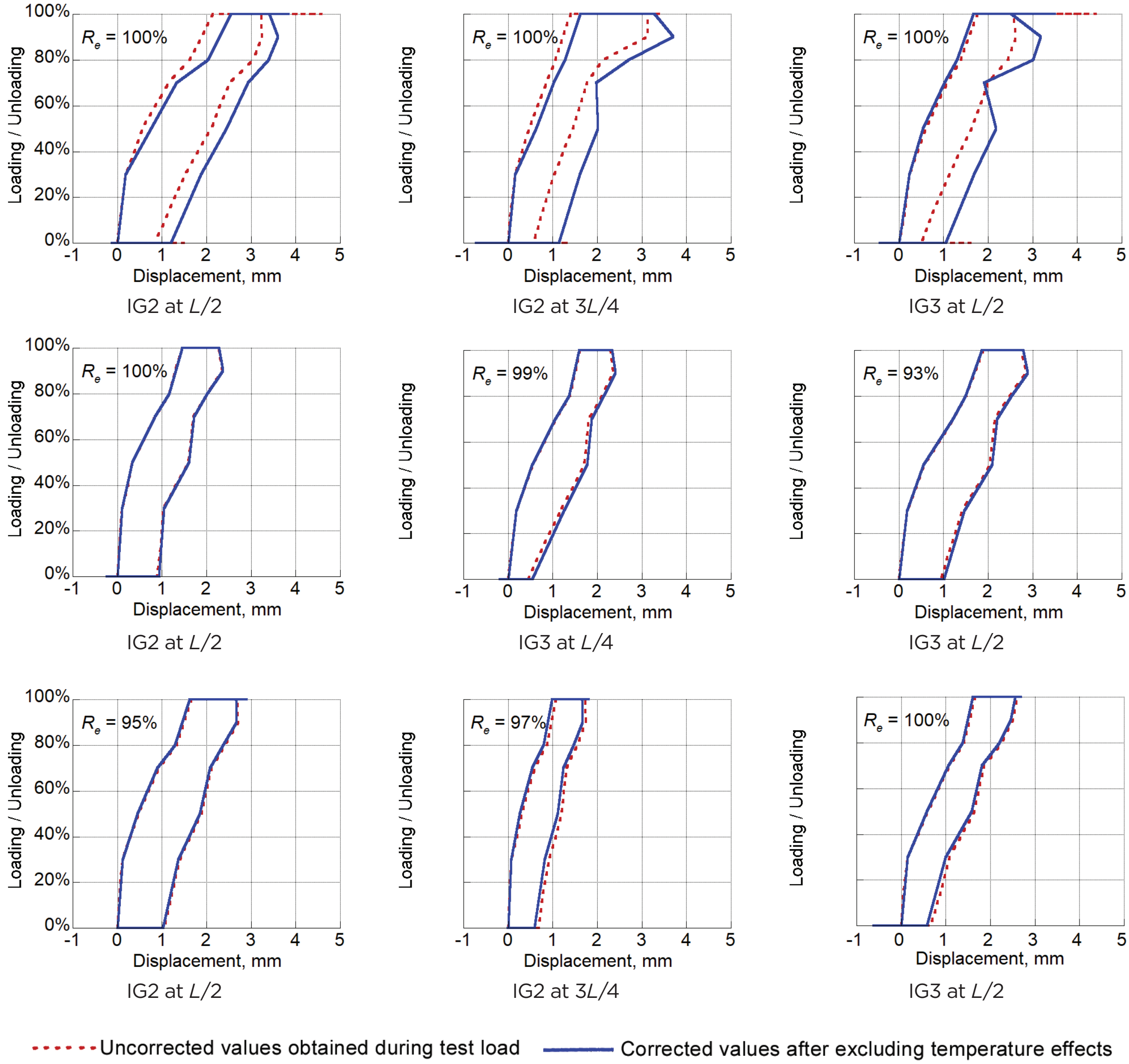


Figure 15. Selected load-deflection plots of girders for various load cases of the testing program. Note: IG2 = inner girder number 2; IG3 = inner girder number 3; L = span length = 25 m; P1 = pier number 1; P2 = pier number 2; R_e = recovery percentage. 1 mm = 0.0394 in.

response quantities such as deflection and internal forces, which were compared with the observed data for further evaluation of the structure. Girder deflections predicted by the bridge modeling software analysis are compared with the observed values in **Fig. 17**. Observed values during the testing program reasonably matched the predicted values except for some discrepancies noted for outer girders in some load cases.

Further, this numerical model was also used to predict shear forces in the prestressed concrete girders at 1 m (3.3 ft) from both ends. Shear strains measured by strain rosettes at these sections were used to determine shear forces in the steel plates. Assuming that the shear stress is uniform across the depth of the steel plate, the shear force resisted by the steel plate can be determined by multiplying the shear stress by the sectional area of the steel plate. Thus, the shear force resisted by steel plates at both ends was calculated for all five girders and compared with the total shear force in the girder at the same section obtained from the analysis. These calculations are summarized in **Table 3** for load case 6, where on average 58% of the total shear force was resisted by the steel plates, which were installed to strengthen the web of the prestressed concrete girders. These observations clearly indicate that the steel plates participate in load sharing with the concrete web of the prestressed concrete girder and consequently reduce the shear stresses in the concrete web. These results provide conclusive evidence that steel-plate strengthening of prestressed concrete girders to enhance shear strength of the girder is effective and can be relied upon to share the load with the concrete web. While the present load testing was intentionally restricted to service-load levels to avoid any

Table 2. Maximum deflection and the corresponding recovery of different girders during the load test

Load case	Position	Maximum deflection, mm	Deflection at 24 hours after placement of test load, mm	Deflection at 24 hours after removal of test load, mm	Recovery, %
1 (A1-P1)	IG2 at $L/4$	3.24	2.61	0.55	79
	IG2 at $L/2$	3.78	3.77	0.42	89
	IG3 at $L/2$	3.98	2.61	-0.28	100
2 (A1-P1)	IG2 at $L/2$	4.14	4.02	0.84	79
	IG2 at $3L/4$	3.30	2.59	0.70	73
3 (A1-P1)	IG2 at $L/2$	4.81	3.96	-0.09	100
	IG3 at $L/2$	3.79	3.15	-0.28	100
3 (P2-P3)	IG2 at $L/2$	4.37	3.74	0.00	100
	IG2 at $3L/4$	3.02	2.34	-0.31	100
	IG3 at $L/2$	3.51	2.16	-1.19	100
4 (P2-P3)	IG2 at $L/2$	3.84	3.60	-0.07	100
	IG2 at $3L/4$	3.70	3.70	-0.05	100
	IG3 at $L/2$	3.50	3.18	-0.38	100
	IG3 at $3L/4$	3.64	3.62	0.99	73
5 (P1-P2)	IG2 at $L/2$	2.36	2.36	-0.05	100
	IG3 at $L/4$	2.40	2.40	0.01	100
	IG3 at $L/2$	2.88	2.88	0.18	94
6 (P1-P2)	IG2 at $L/2$	2.90	2.67	0.13	95
	IG2 at $3L/4$	1.80	1.67	0.06	96
	IG3 at $L/2$	2.69	2.46	-0.53	100

Note: A1 = abutment number 1; IG2 = inner girder number 2; IG3 = inner girder number 3; L = span length = 25 m; P1 = pier number 1; P2 = pier number 2; P3 = pier number 3. 1 mm = 0.0394 in.

distress in the rehabilitated bridge, similar externally bonded and bolted steel-plate systems have been extensively verified in laboratory studies under ultimate loading. Barnes et al.[21] performed full-scale beam tests on reinforced concrete members strengthened with bolted and adhesively bonded steel plates and loaded them to failure. Their results showed substantial increases in ultimate shear capacity, ranging from approximately 80% to 160%, and ductile failure governed by plate yielding and diagonal shear. Adhikary and Mutsuyoshi[20] also observed improved shear strength and delayed diagonal cracking in externally plated beams tested to failure. These findings provide experimental confirmation that the adopted strengthening concept can sustain factored loads, though further research on full-scale prestressed concrete girders is recommended to verify its ultimate performance in field conditions. The rehabilitation was completed in approximately 12 months, after which the overpass bridge was reopened for traffic and has remained in service for over nine years without further distress.

Conclusion

This study documents the challenges encountered and the lessons learned from the rehabilitation of a distressed overpass bridge structure, focusing on issues stemming from excessive strand debonding, improper cutoff of negative moment reinforcement, and deviations from the intended construction sequence. The distress evident through shear cracks in pretensioned concrete girders and flexural cracking in the concrete deck was directly linked to deficiencies in shear strength and negative moment reinforcement.

A customized rehabilitation plan was implemented, and it involved the installation of externally bonded and bolted steel plates on the girder webs as well as the addition of negative moment reinforcement in a new concrete overlay on the existing reinforced concrete deck slab. To assess the effectiveness of these strengthening measures, a load test was conducted under the code-prescribed working loads. Results indicated

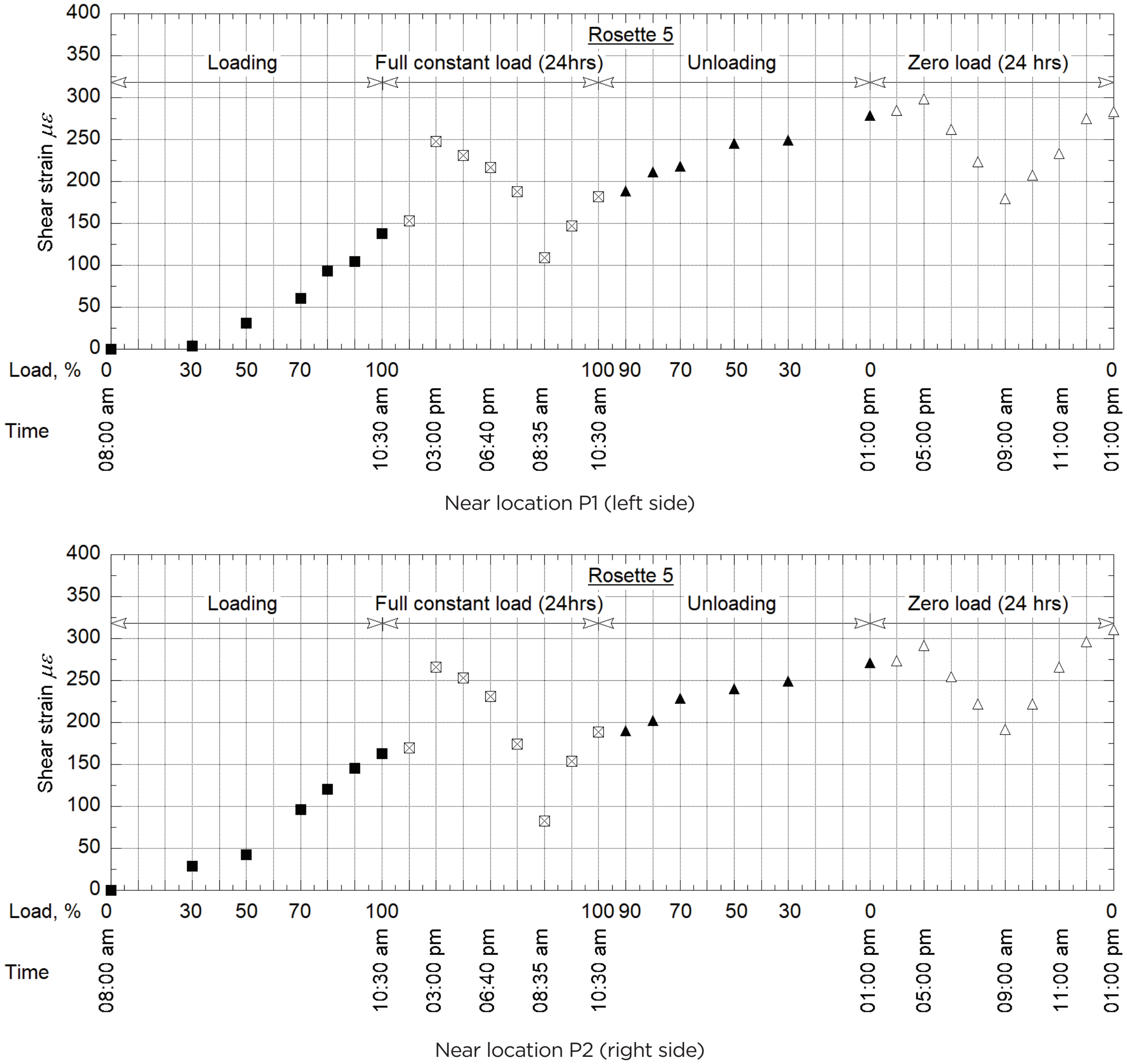


Figure 16. Shear strain time history plots. Note: P1 = pier number 1; P2 = pier number 2.

that deflection recovery at 24 hours after load removal exceeded the acceptable minimum of 85% for prestressed concrete members in most locations. Furthermore, the rehabilitated structure exhibited no signs of failure or damage during any of the six load cases in the testing program. The steel plates actively participated in the load-resisting mechanism, carrying approximately 58% of the total shear force and thereby reducing the shear demand on the concrete web, as verified through strain measurements.

These findings confirm that the rehabilitated overpass bridge module successfully passed the load-testing program, with strengthening elements effectively contributing to the load-resisting mechanism as designed. Importantly, the original load-resistance mechanism and the effects of prestressing remained intact due to the carefully designed rehabilitation approach. The study thus demonstrates that bonded-bolted steel-plate strengthening can serve as a reliable and practical solution for restoring shear capacity in distressed prestressed concrete girders without compromising their prestressing efficiency.

Beyond addressing immediate technical challenges, this study provides valuable insights for practitioners, emphasizing the importance of maintaining construction sequences, understanding the implications of excessive strand debonding, and implementing rigorous design verification in bridge rehabilitation projects.

In addition, the allowable strand debonding ratio should be carefully and conservatively selected, considering its effect on service-stage performance. Although Indian and Caltrans

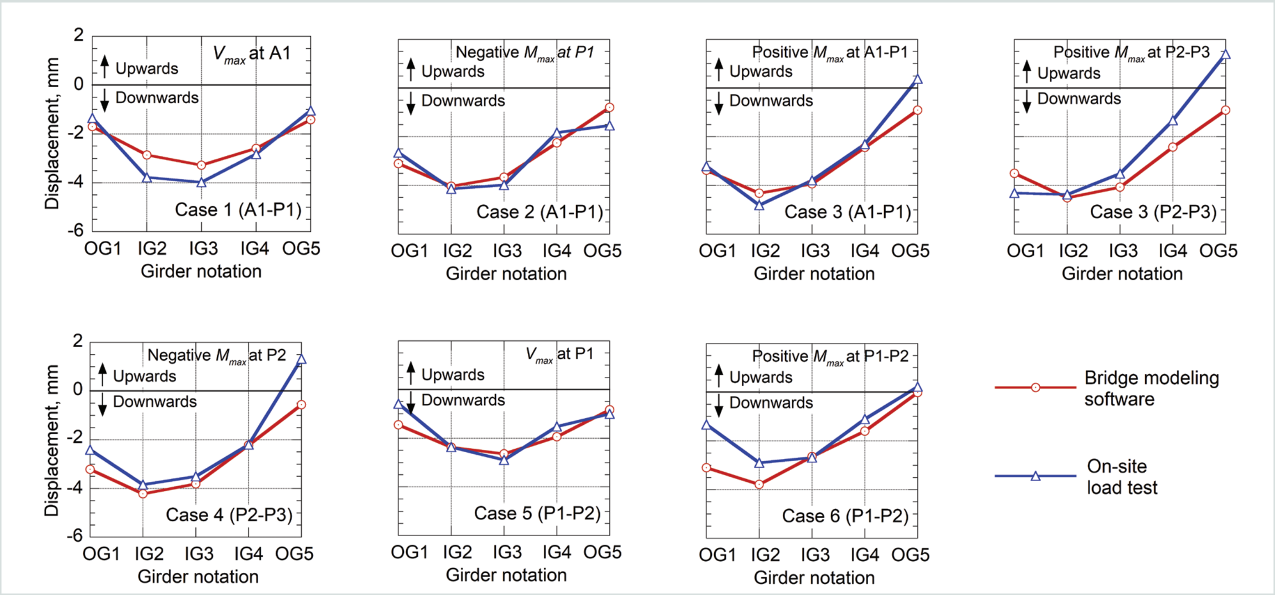


Figure 17. Comparisons of measured girder deflection profile with profiles of those predicted by bridge modeling software. Note: A1 = abutment number 1; IG2 = inner girder number 2; IG3 = inner girder number 3; IG4 = inner girder number 4; M_{max} = maximum moment; OG1 = outer girder number 1; OG5 = outer girder number 5; P1 = pier number 1; P2 = pier number 2; P3 = pier number 3; V_{max} = maximum shear force. 1 mm = 0.0394 in.

Table 3. Shear force resisted by steel plates during load case 6

Strain gauge number	Shear strain, με		Shear force, kN		Total shear force shared by plates, kN		Total shear force on girder (bridge modeling software), kN	
	Left side	Right side	Left side	Right side	Left side	Right side	Left side	Right side
R1	41.0	53.7	20.7	27.1	41.3	54.2	2.9	6.5
R2	85.6	80.4	43.2	40.5	86.3	81.1	89.3	87.3
R3	55.9	39.2	28.2	19.7	67.8	55.7	187.1	174.6
R4	78.6	71.3	39.6	35.9				
R5	162.9	137.9	82.1	69.5	105.3	77.5	170.8	150.7
R6	46.2	15.9	23.3	8.0				
R7	28.3	13.3	14.3	6.7	26.5	10.3	90.0	85.5
R8	24.4	7.2	12.3	3.6				
Total					327.3	278.8	540.1	504.6

Note: R1 = strain rosette number 1; R2 = strain rosette number 2; R3 = strain rosette number 3; R4 = strain rosette number 4; R5 = strain rosette number 5; R6 = strain rosette number 6; R7 = strain rosette number 7; R8 = strain rosette number 8. 1 kN = 0.225 kip.

bridge design specifications currently limit the overall strand debonding ratio to 33%, it is recommended that debonding detailing guidelines should align with the guidelines provided in the 2024 *AASHTO LRFD Bridge Design Specifications*[9] to ensure crack control, adequate shear transfer, and improved serviceability. At the same time, further research on full-scale prestressed concrete girders should validate the ultimate capacity and long-term performance of bonded-bolted steel-plate strengthening systems.

Acknowledgments

The authors wish to express their heartfelt gratitude to technical staff of Structural Engineering Laboratory, especially Mohit Dwivedi, for the on-site measurement and data collection and invaluable help from PNC Infratech, Agra, under the leadership of Rakesh Khanduri. They also thank Alok Raj and Anurag Singhal, former graduate students, for their help in numerical modeling and processing of field measurements.

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Notation

f_{ck} = compressive stress

L = span length

M_{en} = available flexural resistance

M_{max} = maximum moment

M_n = negative moment demand under service loads

R_e = recovery percentage

V_{max} = maximum shear force

V_u = ultimate shear demand

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Abstract

This paper presents a study, including the investigation, analysis, rehabilitation, and verification through full-load testing, of two adjacent overpass bridge structures that experienced significant structural damage shortly after entering service. The bridges comprise multiple units, each having three simple-span precast, pretensioned concrete girders made continuous through cast-in-place concrete diaphragms and deck. Structural damage included flexure cracks in the deck slab and web shear cracks in girders near supports. Investigations identified several contributing factors, including discrepancies between assumed and actual construction sequences, shear strength deficiencies in girders due to excessive strand debonding, inadequate shear reinforcement, and insufficient negative moment continuity reinforcement in the deck slab. Rehabilitation involved externally bonded and bolted steel plates on debonded girder webs near supports and additional negative moment continuity reinforcement through a new concrete overlay on the existing deck. A full-load test confirmed the effectiveness of these measures under design vehicle loads. The rehabilitated bridges successfully carried the design loads, with strengthening elements effectively contributing to the load-resisting mechanism. A numerical model developed using bridge modeling software validated observed response quantities. This study illustrates a methodical and practical approach to addressing structural deficiencies in prestressed concrete girder bridges caused by excessive strand debonding and inadequate continuity reinforcement, providing insights for enhancing long-term safety and performance.

Keywords

Bridge rehabilitation, debonding, flexural crack, full-load testing, girder, shear crack, structural performance.

Review policy

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Publishing details

This paper appears in *PCI Journal* (ISSN 0887-9672) V. 71, No. 2, March–April 2026, and can be found at https://doi.org/pcij71.2-02. *PCI Journal* is published bimonthly by the Precast/Prestressed Concrete Institute, 8770 W. Bryn Mawr Ave., Suite 1150, Chicago, IL 60631.

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