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External post-tensioning of pier heads and bearing replacement of Hatirjheel bridge #4

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ABSTRACT: Newly constructed Bridge #4 in Hatirjheel project at Dhaka developed extensive cracks in both of the pier heads. The two-lane bridge comprises three 30m long simple spans each carried by two triangular reinforced concrete arches at exterior edges of the roadway bearing on Y-shaped pier heads and abutments. Inadequately designed tie members of pier heads developed profuse tension cracking soon after opening to traffic. A scheme for increasing tensile capacity of these members by external post-tensioning has been implemented as an immediate yet permanent retrofitting measure. Elastomeric bearings deteriorated and expansion joint sealing was non-functional, both due to deficient design. A scheme for replacement of bearings by lifting entire decks and installation of neoprene compression seals in expansion joints has been devised. It is demonstrated that monitoring with timely action for remedying any distress can not only avert a disaster but also give new lease of life to a bridge.

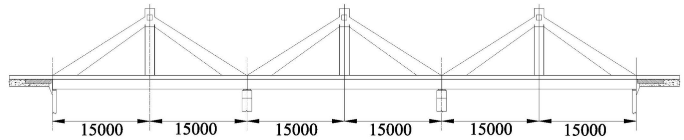
1 INTRODUCTION

Development at Hatirjheel of a low lying urban wasteland into a pleasant clear lake with its multifunctional characteristics has been viewed by many as an urban wonder. Hatirjheel perimeter is defined by a one-way circuitous road that serves to provide an important east-west link in the city’s transportation system. Four bridges provide links between roads on two banks of the lake. These bridges as the other structures in the developed landscape have been designed to be architecturally significant, each one of them being of a unique yet visually compatible appearance.

Bridge #4 near eastern end of the lake is a three-span triangular tied arch shape supported on two abutments and two Y-shaped piers. Tie members at top of the Y-piers developed extensive cracks soon after commissioning. Bearings of the same bridge exhibited signs of failure. Expansion joints were virtually open with non-existent or inadequate sealing causing the bearing area underneath to remain wet and dirty. This paper describes the retrofitting measure implemented to arrest and close the pier head cracks by applying external post-tensioning. It also presents the scheme for replacement of bearings and sealing of expansion joints with neoprene compression seals. The paper demonstrates the importance of monitoring and timely remedial measure to save an otherwise doomed bridge and emphasizes the need for incorporation of measure in design for future maintenance of a bridge.

2 AN OVERVIEW OF HATIRJHEEL BRIDGE #4

The bridge (Fig. 1) consists of three simply supported cast-in-situ reinforced concrete spans supported on two abutments and two piers built on pile foundations. The superstructure in each span is essentially a pair of triangular tied arches with central steel hangers. Bottom tie members of the arches are also the main girders that carry the deck load. The deck load in each span is transferred to the two longitudinal girders by 14 cross girders. The cross girders divide the deck slab into one-way panels ultimately transferring the deck load to the main girders. The main longitudinal girders transfer the deck load as well as load of the triangular arch to the pier or abutment top at four corners in each span. The piers are Y-shaped structures with a tie member at top. Deck load is transferred to top of the Y at ends of the tie member, subjecting the tie member to pure tension.



a) Photograph of the bridge b) Bridge elevation

Figure 1. Hatirjheel Bridge #4

3 CRACKS IN PIER HEAD AND REMEDY BY RETROFITTING

3.1 *Description of the Cracks*

Soon after commissioning of the bridge in early 2013, cracks appeared in top tie members of both pier heads (Fig. 2). Cracks on the faces were vertical turning around to form complete rings around the member. Regular monitoring of the cracks showed that both their number and width increased rapidly. It was realized that the members have failed in tension and were on the verge of severing altogether. In such an event, upper arms of the Y of pier head would act as free cantilevers, causing them to suffer excessive deformation and most possibly collapse. The cracks were many, at least 8 or 9 per member by the time remedial measure was undertaken. Widths of the cracks were also significant some exceeding 2 mm.

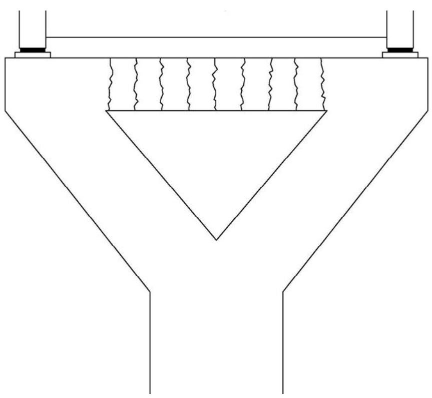
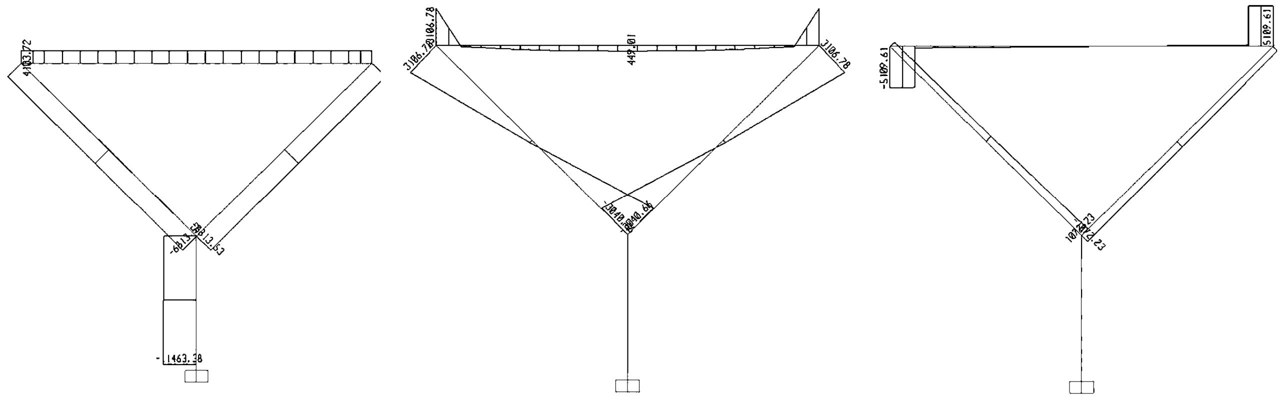


Figure 2. Cracks in pier head

3.2 *Cause of the Cracks*

Dead load of each span of the bridge was 800 ton being carried at only four locations on supports, producing a dead load bearing reaction of 200 ton. Live load reaction at each bearing was calculated as 50 ton. Each pier head carries two spans on four elastomeric bearings, a pair at each end of tie member at top of the arms of Y. The concentrated force thus applied at each such location on pier head was (200+50) ×2=500 ton. An analysis of the pier for this gravity load revealed that the tie member was subjected to a uniform tension in excess of 4104kN. Axial force, shear force and bending moment diagrams of the pier under combined dead and live loads are presented in Figure 3.

Structural drawings of the bridge show that tie members have a 1500 × 1200 mm rectangular cross section reinforced with 12-ø25 and 6-ø20 bars providing a total steel area A_s of 7776 mm^2. Ignoring concrete contribution in resisting tension and also the negligible flexural stress due to self weight, the stress developed in steel reinforcement for dead load and full design live load comes to (4104 × 1000/7776) = 528 MPa. The corresponding stress due to dead load alone is 422 MPa. Design yield stress of the steel used being 413 MPa, the reinforcement has possibly yielded, or at best has had excessive deformation if full design live load has not yet been applied. The possibility of the tie member being severed causing a total collapse of the substructure together with the superstructure was a reality.



Axial force diagram | Bending moment diagram | Shear force diagram

Figure 3. Pier analysis results (kN, m)

3.3 *Design and Execution of Retrofitting*

Since the tie member has already failed for all practical purposes and was on the verge of severing, it was necessary to reverse the failure by application of forces counteracting the structural action. Any attempt to repair and seal the cracks would be futile. As strength of the member was inadequate due to deficient design, retrofitting to make it adequate was warranted. Passive retrofitting by adding reinforcement such as bonded CFRP strips or wrapping would add to strength but not reverse the distress that had already taken place. Strengthening the member by external post-tensioning was the logical solution.

As the member is permanently subjected to pure tension with negligible associated flexure, post-tensioning need be applied only in the axial horizontal direction. This can be achieved by tensioning straight high strength bars on exterior vertical faces of the member pressing together ends of the member through anchor blocks. Using threaded bars would be more convenient to anchor than prestressing strands. It was decided to apply axial compressive force to the tie member to overcome most of the design tension 4104 kN substantially relieving the reinforcing steel which might have already yielded.

The required force would be applied by tensioning six 40 mm diameter fully threaded high tensile alloy steel bars conforming to BS4486:1987 having an ultimate strength of 1030 MPa. To avoid excessive asymmetry of compressive stress in the member during stressing, the bars were to be stressed sequentially in pairs. The applied jacking force would be subject to prestress losses due to elastic deformation of concrete, creep of concrete, anchorage slip and relaxation of steel. As the bars would be stressed sequentially, there would be a progressive loss of prestress due to elastic deformation of concrete for bars stressed earlier in the sequence. As post-tensioning would be applied to mature concrete and as the compressive stress in concrete due to this force would only be about 2.5 MPa, creep in concrete would be much lower than if prestress was applied earlier in its life. Anchorage loss due to dirt or angularity between bearing faces of plate, washer and nut, and tolerance between bar thread and nut, would also be small considering the total elongation of bars some 9m long. Relaxation of steel would be taken as 3.5% for these alloy steel bars as specified in BS4486:1987 and bar manufacturer's recommendation. Calculated total loss stood at about 8.5% of initial force.

The retrofitting arrangement is shown in Figure 4. The bars would be stressed and locked by two anchor blocks bearing against vertical end faces of the tie member. The anchor blocks consist of a 50 mm thick bearing plate stiffened by a system of 25 mm thick plates all having a yield stress of 275 MPa. A finite element analysis of the anchor block yielded a maximum stress of 136 MPa, giving a factor of safety more than 2.

As the first step in tensioning operation, the end blocks together with the threaded bars had to be supported by friction against ends of the tie member. An initial stabilizing force amounting to about 4% of the total tension was applied for this purpose. The remaining force was applied in three stages, each time picking up a pair of bars. The stressing sequence is presented in Table 1. In each stage each pair of bars was stressed a little more than the next pair to compensate for immediate losses such as elastic deformation of concrete and anchorage slip. Applied force in each bar was less than 58% of its ultimate strength. The net force in all the bars after accounting for all losses was 4093.5 kN, or 99.7% of the design tension in tie member.

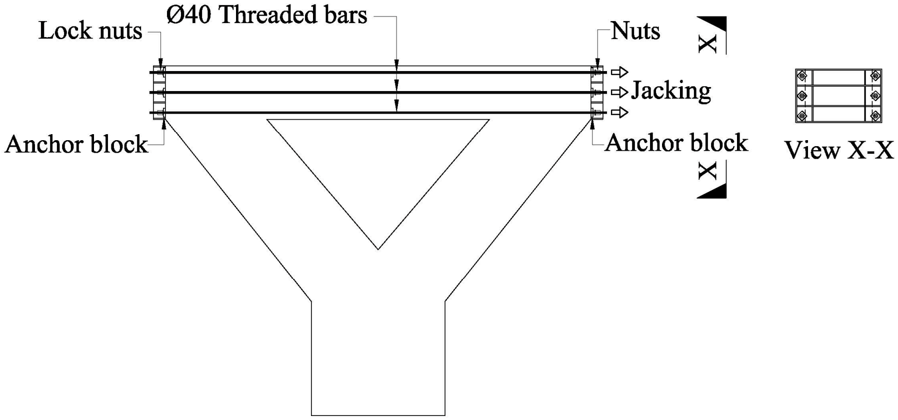
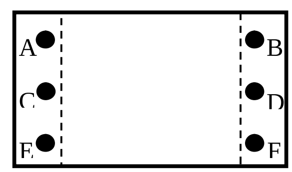


Figure 4. Retrofitting arrangement

Table 1. Sequence of stressing alloy steel bars.

Stage	Bar desig-nation	Applied force (N)	% of total force	Designation	Applied force in each pair (N)	% of Total	Force per bar (N)	Expected total elongation* (mm)
	A-F	60148.15	4.04	A-F	1494054.28	33.396%	747027.14	33.4119
Initial	B-E	59844.53	4.02	B-E	1491232.80	33.333%	745616.40	33.3488
	C-D	59540.92	4.00	C-D	1488473.29	33.271%	744236.64	33.2871
	A-F	476327.38	31.97	Total	4473760.37	100.000%		
1	B-E	476327.38	31.97	Calculated total loss =8.5%			* E = 170,000MPa	
	C-D	476327.38	31.97	Net applied force =4093.5kN			L = 9550mm	
	A-F	481389.16	32.31					
2	B-E	481262.91	32.30					
	C-D	481186.62	32.29					
	A-F	476189.58	31.96					
3	B-E	473797.97	31.80					
	C-D	471418.37	31.64					



Note: Forces measured by pressure gauge readings to be confirmed by bar elongation measurements.

Jacking force was applied by locking one end of the bars by nuts and tensioning from the other end by cylindrical jacks. As two bars would be tensioned synchronously, a single hydraulic pump was used with two manifolds connected to the jacks. Applied force in the bar was confirmed by comparing pressure gauge readings with bar elongation measurements. After each stage of tensioning the bars were locked by tightening nuts at the jacking end. After full tension was applied, the cracks were all but closed. The retrofitted members are to be kept under surveillance. If required at any time in the future, top up tension can be applied easily for which provision has been kept. A photograph of the completed job is shown in Figure 5. The retrofitting arrangement can be covered with metal or concrete if architecturally desirable.

Figure 5. Retrofitted pier head

4 BEARING REPLACEMENT

4.1 *Physical Condition of the Bearings*

When work of retrofitting was being performed, it was observed that elastomeric bearings under the girders, both on the piers and on the abutments, exhibited signs of failure in the form of splitting and bulging (Fig. 6). The bearing area was found to be wet and dirty due to seepage of deck water through expansion joints. As situation was the same for all the bearings, it was considered necessary to review design and quality of the bearings and correct the problem.

Figure 6. Condition of a typical bearing

4.2 *Review and Correction of Bearing Design*

The elastomeric bearings were 550×400×91 thick with five layers of 3mm thick steel laminas. When checked by performing calculation in accordance with AASHTO LRFD Bridge Design Specifications (6^{th} ed, 2012), the design was found inadequate to carry the required bearing reaction of 250 ton. A revised design with 550 × 530 × 91 thick bearing having five 3 mm steel laminas would satisfy all requirements of AASHTO Method B. The new bearings have been manufactured to strict quality checks and accepted after satisfactory performance in all relevant tests at independent laboratory.

4.3 *Design of Bearing Replacement Scheme*

With traffic closed, lifting one end of the deck to replace the bearings would require at least 200 ton jacking force under the end of each main girder. Bearing plinths are so constructed that there is only 150 mm space available on pier heads for locating jacks under the girders, 50 mm of this being cover concrete beyond the reinforcement cage leaving only 100mm for seating the jacks. On the abutment side, there is no room at all for placing jacks under the girders as bearing plinths there are constructed flush with abutment face (Fig. 7). An alternative and more convenient location for placing the jacks would be under the end cross girders. But they were found to be inadequate for this loading condition; an analysis showed that only about 80 ton of jacking force can be applied on these members, the remaining 120 ton must be carried by jacks placed directly under the girders. The lifting scheme was to employ five 100 ton capacity jacks at each of the two corners of a span on pier head, three being placed directly under the main girder and two under the cross girder. Thus a total of ten jacks operated synchronously by a single pump and a system of manifolds were used to lift one end of a span. For the abutments however, placing jacks in front of the bearing plinth was not possible. A bracket bolted to the abutment was designed to act as a platform for jacks to be placed directly under the girders (Fig. 8). Figure 9 shows replaced bearings under girders of two adjacent spans on a pier head.

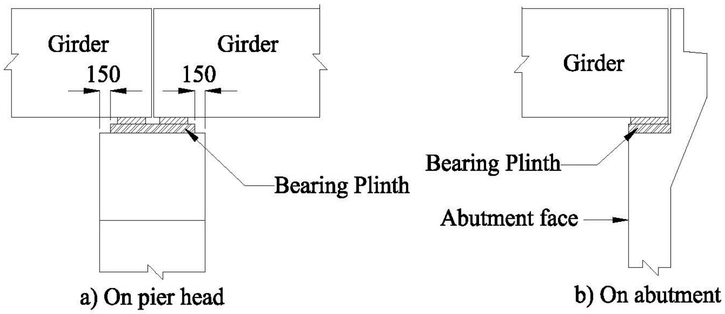


Figure 7. Inadequate space in front of bearing plinths

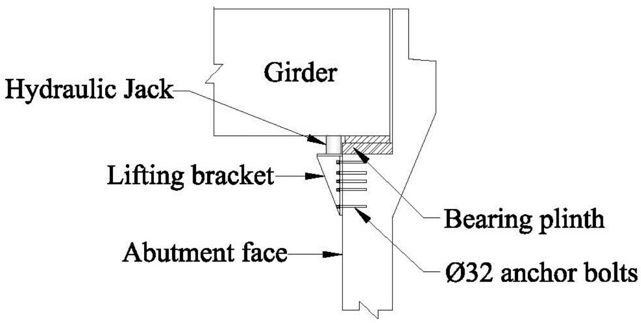


Figure 8. Lifting bracket for abutment front

Figure 9. Photograph of replaced bearings

5 EXPANSION JOINT SEAL

Expansion joints have been provided between the simply supported spans as well as between a span and the abutment back wall. The joint edge is formed by a 100 × 100 × 15mm MS angles anchored in deck concrete by 16mm diameter bars at 500 mm intervals. The 30 mm gap is shown in drawings to be sealed by folded aluminum sheet of unspecified thickness topped by sand-bitumen filler. Apparently, the arrangement was either not implemented or the sealing proved to be ineffective, as water easily percolated through the joints to the bearing area below. No joint sealant except dirt and debris was found on inspection of the leaking joints. It was decided to clean the gaps and install factory manufactured neoprene compression seal of cellular cross section.

6 DISCUSSION AND CONCLUSIONS

The bridge's distress demonstrates how small lapses in design and detailing can lead to major problem in performance and safety of the structure. It also demonstrates lack of appreciation and foresight by not recognizing the need for bridge maintenance such as replacement of bearings.

Bridges are the most vulnerable man-made structure in a communication system. Firstly, their failure can often be sudden and disastrous compared to other stretches of road. They are also a structure where the applied loading is at the mercy of road users beyond control of the designer. This is especially so in Bangladesh where little control is exercised on load limits of vehicles. Bridges should therefore be regularly inspected especially from the underside and their performance monitored. The distress in Hatirjheel Bridge #4 occurred in pier head surrounded by lake water away from the eye. Many bridge supports remain permanently out of sight rarely ever inspected by any maintenance crew. The present problem came to notice only because vigilance was exercised in monitoring the structure. Quick action in appreciating the problem and its cause, and implementing subsequent remedial measure saved the bridge.

Elastomeric bearings are widely used in Bangladesh and sadly their quality is hardly checked. These bearings like all other types of bearing need maintenance and eventual replacement. Provision should be kept in bridge design for convenient lifting of decks to replace the bearings. Adequate room may be kept on pier head or abutment top in front of the bearing plinths for locating lifting jacks. If the resulting width of pier head or abutment top becomes prohibitive, the alternative would be to design end cross girders adequately to take up jacking loads for the lifting operation. Standard drawings of prestressed concrete girder bridges issued by Ministry of Surface Transport of Government of India (IRC 1992) have specially designed end cross girders with jacking locations etched on the member to be used for deck lifting by jacks. Provision of jacking location with etched marking on the structure should be a required design criterion for all bridges.

During the course of replacing the bearings it was found to be extremely difficult to dislodge the old bearings from under the girders. The girders were obviously cast directly on top of the bearings thereby causing the concrete to bond with the rubber. A better practice would be to place an aluminum foil on top of the rubber bearing to separate the cast-in-situ concrete.

In short, maintenance including bearing replacement needs should be considered in the design stage and adequate provision kept for easier and more economic maintenance jobs. Bridge owners should ensure compliance with these requirements.

REFERENCE

IRC 1992. Standard plans for highway bridges, Prestressed concrete beams and R.C.C. slab type superstructure. Indian Roads Congress, for Ministry of Surface Transport (Roads Wing), Govt. of India, New Delhi.