Post-disaster construction and bearing replacement of Bahaddarhat flyover

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ABSTRACT: A 1.37 km long four-lane flyover has been commissioned in October 2013 at Bahaddarhat in Chittagong city. Structural system of the flyover consists of precast post-tensioned concrete I-girders acting compositely with cast-in-situ reinforced concrete deck slab. On two occasions during construction one or more girders accidentally fell to the ground below. The latter accident caused fourteen fatalities. Post-accident investigation revealed serious flaws not only in construction, but also in design, supervision and project administration. Following an assessment of condition of the completed works, a systematic approach was adopted to repair and renovate the faulty works and complete the remaining works. The faulty elastomeric bearings were replaced after commissioning of the flyover. Recommendation for simple modification to end diaphragm design to aid bearing replacement is put forward. It is demonstrated that careful attention to details and a scientific and systematic approach can ensure safe construction and an acceptable and adequate flyover.

1 INTRODUCTION

1.1 *Background*

Bahaddarhat intersection, a congested road junction, is the only major gateway to Chittagong city from south-eastern parts of the country including the coveted tourist destination Cox's Bazaar. Roads from other important destinations such as Kaptai and Bandarban also connect here. Traffic at the intersection consists of a large volume of heterogeneous mix of vehicles such as bus, minibus, trailer, truck, microbus, car, motor bike, and various forms of non-motorized vehicles. A large number of pedestrians also pass through the intersection. Area surrounding the intersection is of a generally residential character with commercial establishments lining both sides of the connecting roads. Traffic volume at the intersection far exceeded its capacity and congestions were common, especially during peak hours. To overcome the problem by augmenting the existing road capacity and to provide an easy access to city from the south, Chittagong Development Authority undertook construction of a flyover at the intersection. A project office was instituted to implement the task, a consultant was appointed to design and supervise the works, and a contractor was engaged to execute the construction.

As construction work of the flyover was progressing, a tragic accident occurred in the evening of 24th November 2012. Three girders being constructed overhead on the pier heads fell to the ground on a gathering of people. Several human lives were lost in the event besides causing severe injuries to scores of others. This accident was not the first in that construction. Previously on 29th June 2012 in a similar incident a girder fell off its overhead supports; but fortunately there was no fatality. Such successive accidents raised concern among the client and the Government regarding safety measures practiced during construction and also the competence of persons executing the works. Besides this aspect, concern was also raised regarding quality of the construction work, materials used and constructed elements. An expert technical committee was formed by the Ministry of Housing and Public Works to investigate quality aspects of the construction. The committee in their report (MHPW, unpubl.) pointed out many deficiencies in the construction, including among other things faulty construction methodology, poor safety measures, inadequate supervision and mismanaged project execution.

1.2 *Post-Disaster Rehabilitation and Construction*

After the second fatal accident on 24th November 2012, construction of the flyover was suspended. It was decided to assess condition of construction of the flyover, check adequacy of various constructed elements, review design of the flyover, devise remedy for any deficiency found, and work out a technologically sound and safe construction methodology. Special Works Organization of Bangladesh Army was entrusted with supervising the works to completion, who in turn commissioned the services of a local consulting house with proven excellence in flyover projects. The construction was resumed under their combined supervision only after the desired assessment had been made and aspects of methodology and management of construction had been revised. Inferior elements of construction were identified and marked for replacement or renovation as appropriate. Construction of the flyover was completed and the facility opened to traffic in October 2013 without any other mishap.

This paper gives an account of condition of constructed portions of the flyover immediately after the fatal second accident, and measures taken for remedying the defects and continuing the construction. Technique adopted for replacement of elastomeric bearings of the completed and commissioned flyover without disrupting traffic is also described.

2 AN OVERVIEW OF BAHADDARHAT FLYOVER

An aerial view of the flyover taken immediately after the accident is shown in Figure 1, where the supports and spans are identified for reference. The extent of construction of the superstructure up to the accident is discernible in the figure. The broken girders in span S13 which fell in the pond (pumped out to search for bodies) can also be seen.

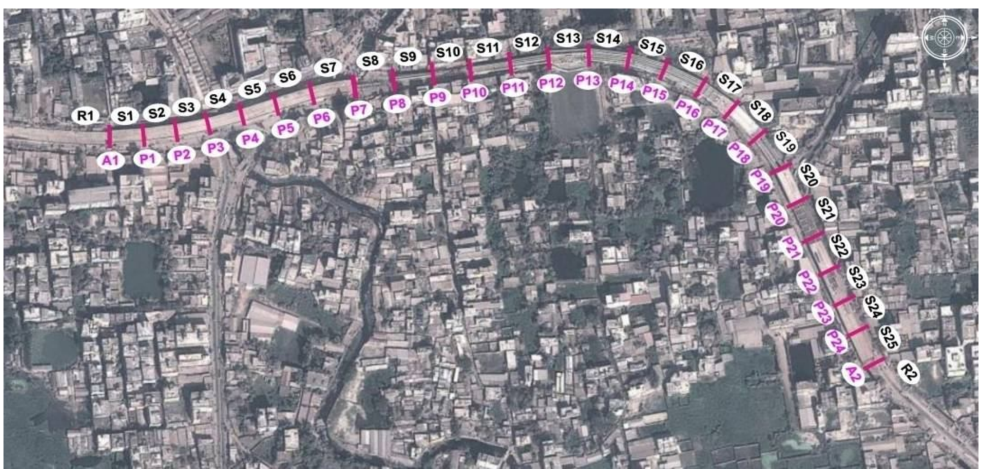


Figure 1. Satellite view of Bahaddarhat flyover after the accident

The flyover alignment follows existing ground road layout which is essentially a curved path. The curved alignment is formed by connecting a series of straight segments. The 1001m long elevated structure comprises seven 35m long and eighteen 42m long simply supported spans. The elevated structure is linked to the ground road at both ends by two anchored earth ramps, R1 and R2, of lengths 164.95m (160m in original design) and 201.25m (160m in original design), making up a total flyover length of 1367.2m. The structural system chosen for the simply supported spans is precast post-tensioned concrete I-girders acting compositely with cast in situ deck slab. The deck section consists of a 200mm thick slab resting on six 2m deep I-girders for 35m spans and seven 2.4m deep I-girders for 42m spans. In the original design, the deck slab is overlain by a 50mm thick concrete wearing course. The deck is flanked by reinforced concrete parapets topped by pipe rails and divided by a reinforced concrete median to create a two-lane roadway for each direction.

The flyover deck is supported on single circular RC piers 2.5m in diameter and up to 7.29m in height with cantilever pier heads to support the girders. The anchored earth ramp is retained by 6.275m high counter fort abutments that also bear the girders of terminal spans. The piers and abutments bear on pile foundations consisting of 1m diameter bored cast in situ piles having lengths between 34m and 46m.

3 CONDITION ASSESSMENT OF THE FLYOVER AFTER ACCIDENT

Immediately after the fatal accident of November 2012, an assessment of extent and condition of the completed works was undertaken.

3.1 *Extent of Construction Completed up to Second Accident*

When the second fatal accident took place, all the foundation and substructure works, including piles, pile caps, piers, abutments and pier heads, were complete. Bearing plinths on pier heads were in place ready for receiving the girders. Of a total of 168 post tensioned girders, concrete was cast for 133 girders. However, prestressing tendons in some of these girders had yet not been stressed, and in yet others they had been stressed but the ducts not grouted. A total of 12 deck slabs out of 25 had been cast either fully or partially. Concrete wearing course and edge beams of expansion joints had also been laid where the deck slab was completed. Some concrete parapets had been cast on the two sides of completed deck slabs. A low height median had been designed and reinforcement for this projected from the deck slabs. It was necessary to check all these completed works for adequacy in design, material, dimension, position, line, level, finishing and appearance.

Besides, scaffolding for some unfinished girder works were in place. These also needed checking for adequacy, especially as this element of construction was related to the operation that triggered the accident.

Some Nehemiah panels of anchored earth walls of ramp R1 were in place. A portion of the anchored earth block and retained fill in this ramp had also been laid. This needed to be checked for adequacy of design, ground capacity and quality of materials and compaction.

3.2 *Condition Assessment*

Objectives of the condition assessment were among others: to make an assessment of condition of already constructed elements, identify deficient elements and classify them according to severity of inadequacy and difficulty of reconstruction, assess extent of damage to remaining parts due to the incidence of collapse, review design of flyover geometry and structure and suggest implementable remedies, review and rectify construction methodology to ensure safe construction, and suggest any functional restriction that may be imposed due to non-rectifiable deficiencies.

A thorough investigation of completed works was undertaken by physical examination supplemented by design drawings and quality check reports. Information about adopted construction methodology was gathered and scaffolding works were evaluated. The following were principal findings of condition assessment.

3.2.1 *Flyover geometry*

Considerations for good flyover geometry include: smooth horizontal alignment and vertical profile satisfying standard specifications, proper deck width and its widening at bends, and camber and super elevation as required including transition between these. Design drawings of the flyover were found to be deficient in many of the required geometric parameters. The flyover lacks both a smooth horizontal alignment fitted with proper circular and transition curves and a smooth vertical profile featuring proper crest and sag curves.

Thorough topographic measurements revealed that position of many of the pier centers do not match with the coordinates shown in the drawings, the deviation being up to 354 mm. It was also found that many pier heads were not oriented in the direction of normal to the centre line.

There were serious deficiencies in the approved drawings regarding cross slopes of the decks. Of the 26 supports, equal cross falls (-2.5% or -2%) with a central crown were shown for 20 locations, the remaining 6 locations had a super elevation of either 2% or 2.5%. It is not clear why two different cross falls were chosen for straight sections in the same roadway. Cross slopes were indicated only at the support locations with no information for intermediate locations. For transition spans where cross fall or super elevation is different on adjoining supports, information on deck level variation within the span is missing. Some of these transition spans had already been cast. Field measurements indicated that desired cross slope and super elevation were not maintained in the constructed slabs. The variations in cross slope included deviation from design value by more than 1%, reverse super elevation, and deviation from design level by more than 25mm. Little could be done to correct these errors in many of the constructed deck slabs.

Designed carriageway width for two lanes in each direction is 6.5 m. No provision for widening of decks for sections on curves was kept, although geometrical calculations warrant such widening for some of the curves.

As per working drawings, flyover ramps at both ends start off with a 3% linear vertical slope. The flyover is horizontal from P5 to P20. Two 5% parabolic curves from A1 to P5 and from P20 to A2 join the linear slopes with the central horizontal portion. The profile misses the transition between two different gradients at a location. Necessary sag curves to ensure smooth riding are missing both for transition from ground level to linear slope and between linear slope and parabola. Fitting a geometrically proper vertical curve was almost impossible as some of the deck slabs were already in place. Insertion of transition curves resulted in a longer ramp, especially for R2 as the actual ground level there was found to be much lower than shown in the drawing.

As per available drawings, the seven girders in 42 m spans and six girders in 35 m spans were to be placed parallel to each other. For spans in straight sections where the pier heads are parallel, all girders in a span are to be of the same length. For spans on curves, adjacent pier heads would be nonparallel requiring each girder in the span to be of a different length. No shop drawing to the level of detail required was apparently prepared, resulting in constructed girders of randomly different lengths. Sometimes the girders were either too short or too long for the location.

Bearing plinths are required to be concentric with the bearing pad and designated bearing point on girder soffit to produce a uniform distribution of bearing force. All bearing plinths had been constructed maintaining a common distance and orientation without consideration of flyover alignment, pier head orientation and girder length, alignment or orientation.

Deck slabs that had been cast had uniform cantilever lengths beyond the exterior girders, resulting in decks with parallel edges that meet at an angle. The resulting zigzag alignment of the deck would be a hindrance to safe and smooth driving. The two terminal slabs at the abutments had been cast substantially longer than fits the space necessitating retrofitting of abutment seats to accommodate the back walls.

3.2.2 *Structural aspects*

Physical condition of constructed elements of the flyover left a lot to be desired. There were evidences of general deficiencies and errors in measurement due to lack of proper shop drawings, absence of strict and informed supervision control, lack of strict quality check, and poor formwork, centering and workmanship etc. A few of the major deficiencies are mentioned here.

A total of 242 elastomeric bearing pads out of the required 336 had already been installed. Many of these pads exhibited various distress signs, such as lateral or reverse longitudinal sway due to eccentric application of load, bulging, crack, etc. Signs of faulty and careless installation were aplenty. Figure 2 shows two such cases of improper installation and a failed bearing pad.

a) Bamboo packing under bearing pad   b) Bearing pad floating on sand packing   c) Failed bearing pad

Figure 2. Samples of poor installation and quality of bearing pads

Besides faulty installation, material quality of the bearing pads fell short of required standards. According to technical committee report of the ministry (MHPW, unpubl.), tests conducted at BUET on bearing specimens collected from the accident site revealed that all the specimens failed to meet requirements of tensile strength, compression set, peel strength and heat persistence. Peel strength of the specimens was reported as zero against a specified minimum of 7 kN/m. The result was subsequently corroborated by physical tests on other 'approved' specimens lying in site store.

In absence of shop drawings, the girder formwork was prepared randomly without regard to actual length and configuration required for the location. For instance, the same length of formwork had been used for the middle I-section and adjustments for girder length made by adjusting length of a rectangular end block. This resulted in possibility of spread of bursting forces beyond the end block to the girder stem and also the possibility of critical shear section falling outside the end block. In one girder the end block length was measured as 900 mm in place of specified 1800 mm. The practice also resulted in improper overhang lengths beyond centre of bearing, being either substantially more or somewhat less than the specified 400 mm. In some girders the effective end block length had been reduced further and in some others the full area of elastomeric bearing pad was not in contact with girder soffit resulting in high bearing stresses.

Methodology of construction of prestressed concrete girders had been concreting on overhead casting beds between pier heads, tensioning of cables on casting site, grouting of cable ducts and then lifting and shifting to their bearings. General appearance of the girders indicated poor formwork and workmanship, finished concrete faces not being neat and of uniform texture. Some of the girders were found to be bent horizontally indicating unbalanced tension applied to tendons due to faulty stressing sequence and cable profile. An example of poor workmanship of girder is shown in Figure 3.

Figure 3. Bulging of girder concrete due to poor formwork

The girders were constructed and shifted to position individually, but they must have been laterally supported in order to avoid danger of tilting. In fact it is such tilting that caused the three girders to fall on the bazaar underneath in the fateful evening of the second accident. The practice appeared to be virtually non-existent.

Diaphragms contribute to the integrity of PC girder and deck slab system and distribute concentrated vehicle loads among the girders. It appeared from the quality of construction of the diaphragms that no regard for their importance crossed the minds of the builders. During lifting and shifting of PC girders, most of the continuity bars embedded in the girder for lapping with diaphragm bars have been cut off for convenience of attaching lifting/shifting collars. For some diaphragms, bars had already been fabricated but casting not done yet. In these reinforcement fabrications it was found that adequate lapping of bars had not been provided, some with only 25 to 50 mm nominally welded lapping. In some cases position of the diaphragms were found to be misaligned with the embedded bars kept in the PC girder, missing them completely. Some such embedded bars are visible outside the diaphragm. Already 32% of the diaphragms had been cast, most likely ignoring the above issues. Photographs of some typical diaphragms are presented in Figure 4.

Out of the total 25 deck slabs 12 had already been cast with inappropriate line and level, rendering it difficult to correct the profile for the remaining decks. The difference of levels of the constructed deck from the design level was as high as 87 mm with some of them being constructed in a cross slope that is opposite of the design. As per design a 50 mm thick concrete wearing course had been laid over the RC deck. The surface finish of the wearing course was extremely poor and unsuitable as a riding surface. The wavy pattern of a finished deck is shown in Figure 5. There is no alternative but to lay an asphalt wearing course on top of the existing surface to provide a suitable riding surface. The four-lane deck of this urban flyover has been provided with a width of 6.5m for two lanes in each direction. This being inadequate as it is, the efficiency has been further reduced by not providing required deck widening on curves. Besides, the designed median barrier is a mere 300 mm high concrete strip. Such low median is unsuitable for an urban flyover; headlight glare of opposite vehicles will cause driving hazard; it may be crossed by errant vehicles causing serious head on collisions. It may also be crossed by jay walking pedestrians. Reinforcement kept in deck slab for the median barrier can be seen in Figure 6.

a) Continuity bars shaved off b) Inadequate lap of diaphragm bars c) Misaligned diaphragm

Figure 4. Faulty construction of diaphragms

Figure 5. Poor riding surface of deck

Figure 6. Rebars for low height median

Inter-span expansion joints have been designed as 25mm gaps with a 20mm wide rubber strip insert. Steel edge beams that appeared to be makeshift contraptions made from rolled angle sections without adequate anchorage to deck concrete were found to have been installed. No rubber strip was yet installed. Clearly the expansion joint design is quite inadequate and unbefitting of a modern flyover. The poorly anchored edge beams are likely to soon be dislodged under vehicular impact creating a bumpy ride condition. The loosely inserted rubber strip will be extruded by wheel impact and deck expansion; they will also provide no seal causing rain water to percolate in the bearing area underneath, causing the bearings to deteriorate fast. Figure 7 shows an installed expansion joint and an inadequately anchored makeshift edge beam ready for deck concreting.

a) Installed expansion joint

b) Makeshift edge beam with inadequate anchorage

Figure 7. Condition of expansion joint

Ramps of the flyover were specified to be anchored earth embankments of a particular brand, although such practice is not allowed by Government procurement rules. The ramp design consists of compacted sand fill contained within and retained by so called 'Nehemiah' anchored earth blocks on both sides. Although the earth blocks were to be anchored by galvanized steel rod inserts, it was found that steel rods used were rusted indicating that they were not galvanized, merely painted silver gray. Grain size and compaction of the fill material were also found to fall short of specifications.

4 REVIEW AND REVISION OF DESIGN

Although all foundation and substructure work and substantial part of superstructure work have been completed, a review of all elements of the structure was conducted to determine any implementable corrective measure or any functional limitation of the flyover.

No calculation of design pile capacity could be found; pile capacity shown on the drawings was apparently based on result of three test piles. Soil investigation report used during design contained bore logs showing SPT value only for the bearing stratum on which the pile had been terminated, with no information regarding the class and resistance of soil in the upper strata. No account for skin friction can therefore be taken in capacity calculation, and calculated capacity of piles falls substantially below the reported test capacity. Seismic design provision requires pier stirrups to be continued in the pile cap, but these had been terminated some distance above the cap jeopardizing seismic performance. Pier heads are balanced cantilever beams springing from pier shaft and so need only top reinforcement to carry flexure. However, seismic design provision requires that bars amounting to half the top main bars be placed at bottom of such elements. But amount of steel at bottom of pier head was found to be only 11% of top steel. All the above deficiencies are beyond the scope of any correction at this stage of construction.

Bearing plinth reinforcement did not include confinement stirrups. This inadequacy would severely limit performance as highly stressed compression blocks, especially for the higher plinths. Plinth top detail showed raised edges to form a receptacle for the elastomeric bearings. This would create a trough of water in which the bearing is seated, especially as the expansion joint above is not sealed; this is likely to harm its properties within a short period of time. The desirable detail should have been such that water is directed away from the bearing. The inadequacy has been corrected by grinding off the raised edge to facilitate drainage.

Specified 28-day cylinder strength of girder concrete was 35 MPa with a minimum strength of 30 MPa at transfer of prestress. A design check revealed that compressive stress at bottom of mid-span section due to jacking force of 1653 kN specified in drawings exceeded the code stipulated $0.55\sqrt{f_{ci}'}$. Tendon arrangement was found to be unsymmetrical for a substantial length of girder. This together with any possible unsymmetrical sequence of tensioning may have caused the constructed girders to be bent horizontally. These deficiencies were corrected for the girders yet to be constructed with expected good result.

The deck slab, despite being a continuous one way slab spanning between girders perpendicularly to traffic, has been reinforced such that amount of top main steel is only half that at bottom. This allocation of reinforcement defies logic, as negative moment is always larger than positive moment for such continuous spans.

Expansion joint design is faulty on two accounts: the joint gap is not sealed thereby allowing deck water to seep into the bearing area underneath, and the joint edge beams are not adequately anchored to deck concrete. As riding surface of the constructed concrete wearing course was unacceptable, it was decided to lay a 30 mm asphalt wearing course on top. This would raise level of deck surface necessitating raising the expansion joints as well. It was decided therefore to discard all installed expansion joints and replace them with properly manufactured strip seal joints.

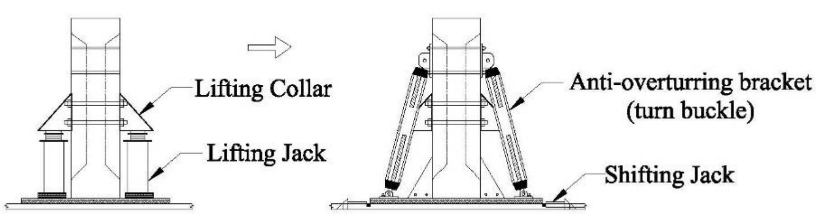
Median strip was originally designed as a 300mm high concrete barrier with a base width of 300mm and a top width of 250mm. The median design was revised to render it safer, but the base width had to remain fixed as otherwise the already restricted roadway width will be further reduced. Keeping the same side slope as in original design and making the top width 150 mm, a height of 950mm was achieved from which 30mm would be lost to asphalt wearing course. Although a non-standard shape, the new median design was made structurally adequate for the anticipated vehicle impact load.

Despite minor detailing deficiencies design of Nehemiah earth blocks was found to be generally adequate. However, the design appeared to have been based on tentative soil properties which have no reflection of actual soil properties found in the exploration report. Soil exploration conducted recently confirmed this fact and also revealed that the bearing ground condition is not adequate to carry loads of the embankment. A very soft highly plastic silty clay layer having a high settlement potential exists at a depth of 7.5 m to 16.5 m, a fact overlooked in the original design. Ground improvement should have been an option, but non-implementable at the present stage. The ramp together with adjacent ground road will settle and cause damage to the flexible pavement above. In fact such settlement started soon after commissioning of the flyover and is progressing.

5 REVISION OF CONSTRUCTION METHODOLOGY

Construction methodology for the deck superstructure of this flyover involved casting, tensioning and grouting of girders on overhead casting beds. Subsequently they were lifted and shifted by jacks to their designated locations on bearing pads. Throughout the lifting and shifting procedure the girders would be laterally unsupported and thus highly unstable. Sometimes the girders were rested temporarily at some intermediate location adding to the number of such unsafe operations. Although highly risky, this is a common method of construction of such decks in this country, especially as adequate room for casting at ground level and then lifting and positioning by cranes is often not available. It is to be noted that it is this shifting operation that caused tipping of three girders that fell to ground in the second fatal accident. Safety concern during shifting of girders and also while resting individually at a location prior to stabilization by diaphragms demanded some modification of the procedure to improve safety.

Preparing the girders in pairs or threes, tying them together with diaphragms and then shifting them together were considered, but the methodology would involve substantial additional time for completion. Instead a simple stabilization arrangement by brackets was devised and the girders shifted individually. The arrangement is shown in Figure 8. It consists of a pair of anti-overturning brackets fitted at each end of a girder attached to the travelling wooden sleeper used for shifting. The anti-overturning bracket was fabricated with a purpose made heavy duty long turnbuckle attached by pin ends to girder and wooden sleeper. The arrangement proved to be effective as any tendency of tilting of the girder during horizontal travel was prevented by the brackets.



Lifting arrangement Shifting arrangement

Figure 8. Anti-overturning brackets for girder shifting

Besides during shifting, the girders are unstable even while standing at a location until several of them are tied together by diaphragms. To improve stability of individual girders while standing alone, a similar concept was utilized and a stabilizing arrangement implemented. The arrangement is shown in Figure 9. It consists of a pair of supporting posts attached by bolts to the girder at each end.

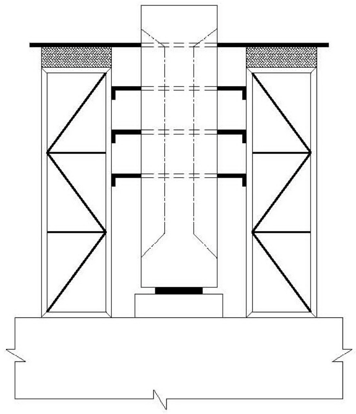


Figure 9. Stabilizing arrangement for girder standing alone

6 REPLACEMENT OF BEARINGS

A total of 242 elastomeric bearings were installed prior to the accident, girders placed over them and deck slab cast. Many of these bearings exhibited signs of distress due to inadequate material quality and faulty installation. Bearings that had been approved for use were recovered from site store. Their quality was found to be so poor that layers of rubber could easily be peeled off from the rusted steel laminas by hand. These were discarded and new bearing pads used for the remaining girders. Prompted by evidence of poor quality of the bearings in site store, it was decided to replace all earlier installed bearings, although some of them did not so far show much distress. However, because of the urgency of opening the flyover to traffic, the operation had to be scheduled after commissioning when traffic would be travelling on the deck. In the planned operation one end of a deck with six or seven girders would be raised by synchronized jacking of all the girders. Upward movement of the deck would be some 20mm or just sufficient to allow the existing bearings to be taken out and new bearings installed. This ensured that the operation could be carried out while traffic is on the deck.

Apparently the necessity of replacement of bearings was completely ignored in the design, although it is common practice elsewhere in the world. Ideally the jacks should be placed on pier heads under the girders in front of the bearing plinths. However, this means providing extra width of pier heads which may not be economical. The jacks can alternatively be placed under the end diaphragms between the girders at predetermined locations. In this case the end diaphragms in each span have to be adequately designed as they will have to bear half the dead and live load of a span. Standard plans for prestressed concrete girder and RC slab composite bridges issued by Ministry of Surface Transport of Government of India (IRC 1992) provide such strengthened end diaphragms with clearly etched jack location points for lifting up the superstructure to replace bearings. However, no distinction between end and intermediate diaphragms has been made in design of Bahaddarhat flyover. Besides, the way starter bars in the girders had been shaved off at location of the end diaphragms, these cannot be trusted to carry much vertical load.

Space available on pier head in front of bearing plinths was inadequate for accommodating the required capacity jacks; the concrete there was also mostly outside the reinforcement cage and hence susceptible to spalling under load from jack. To tackle the problem, a bracket was fabricated from heavy steel sections to provide full or partial seating for jacks under the girders beyond the pier head top. The bracket system consisted of pairs of heavy channel sections placed vertically against the pier head on either side. Channels on the two sides of pier head were pulled against each other by HT alloy steel threaded bars thereby putting gripping pressure on the pier head concrete. Jacks were positioned on a cross head fitted to top of the channels, the jacking reaction being realized by mobilizing friction between vertical channels and pier head concrete. An alternative simpler bracket arrangement was used where part of the jack could sit on pier head concrete. The bracket consisted of a horizontal arrangement of channels placed at top of pier head that formed a local extension of pier head in front of bearing plinth. As part of the jack reaction will be transferred to pier head concrete, a pre-compression was applied by screw jack to top of pier head. This would reduce chance of spalling of pier head concrete under the jack. In both the arrangements it was necessary to jack up both spans on the two sides of a pier head so that no unbalanced forces occur that may tilt the brackets. This was done even when bearing replacement was required under one span only. A similar arrangement was also devised for replacing the bearings on top of the abutments.

Bearing replacement operation started some two months after opening of the flyover to traffic. All 242 bearings were replaced successfully with the traffic plying on the deck above.

7 DISCUSSION AND CONCLUSIONS

Heavy constructions such as flyovers are new in this country. Flyovers are not only more visible than river bridges, they are also more difficult to construct in view of the limited work space available in a congested urban setting. Safety of both workers and of the public is of paramount importance. The paper has described the condition of works completed prior to the second fatal accident in November 2012. Deficiencies in construction and also in design were aplenty in this project.

One of the most serious deficiencies in Bahaddarhat flyover has been the geometrical design. Geometrical deficiencies exist in alignment, profile, width and cross slope, all yielding an inefficient and to some extent hazardous driving condition. Structural design also lacked accuracy and vision in some aspects. Quality of construction reflected poor workmanship and skill as well as lack of proper and informed supervision of control. Required site practices such as preparation and approval of adequately detailed shop drawings were totally absent.

Some of the deficiencies could be corrected such as by replacing the installed bearings and expansion joints, correcting tendon profile, laying asphalt wearing course etc, but many others had to be compromised because of the difficulty of implementation at the final stage of construction. These included compromising on geometrical design and structural detailing for the foundation and substructure. Ramp settlement was anticipated but steps to eliminate or reduce the settlement by ground improvement or structure extension could not be implemented.

In conclusion, flyover design and construction should be entrusted to consultants and contractors with proven experience of success in this particular type of project. In the design side, need for bearing replacement should be recognized and provision kept for future such replacement work. Geometrical design should be given topmost priority, especially when horizontal curves are involved. In the construction side, the culture of preparation and submission of acceptable shop drawings by the contractor should be inculcated and preferably made mandatory in the contract document.

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