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REHABILITATION OF BOX GIRDER BRIDGE

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A large pre-stress continuous box girder bridge consists of two separate parts, ten spans deck and eight spans deck. The bridge was subjected to multiple airstrikes, which caused several damages. The most serious are the complete destruction of four conductive spans in the ten spans deck, the movement of the remaining six spans longitudinally by about 1.25 m and transversely by about 0.1 m. Local damages occurred in other parts of the deck structure and minor damages in the piers and abutments. The deck structures were constructed by incremental launching method (ILM). Rehabilitation of the bridge included repair of the local damages, lifting and re-instatement the six spans, which weighs about 9,000 tons, to its original position, the constructing and launching of new box girder to substitute for missing spans, and finally connecting old and new spans effectively. This paper introduces briefly the re-instatement of the six spans, connection details between the old new structures, then assessing the efficiency of connection by analysis and load testing the connecting span.

Keywords: BS5400, Reinstating structure, Fayhaa bridge, Connection, Deflection, Bridge load test.

1 INTRODUCTION

Al Fayha'a bridge connects east and west banks of Shat Al-Arab river at Al Basrah city in Iraq. Its super structure consists of two structures separated by expansion joint at pier P(2-1). Each structure is continuous single cell pre-stressed box girder. The outside dimensions of the box are 7 m wide at the bottom, 10.4 m wide the top, and 3.55 m total height (AlKhazen 1974). The east part is ten span with total length of 430.15 m as shown in Figure 1. While the west part is eight span with total length of 330.75 m. It was constructed in the mid-seventies of the last century using incremental launching method (ILM). The bridge was built by the German company Polansky and Zolner using their post tensioning (PT) system, which consists of bonded cables made of 40 mm^2 oval wires grade ST 1420/1570 (Polensky & Zöllner 1969). The longitudinal PT are PZ A100 (33 wires) with effective force of 1,180 kN. While the transverse PT are PZ A40 (12 wires) with effective force of 393 kN. The concrete used is "Beton 35" with 28 days cylinder compressive strength of 35 MPa. The number of longitudinal cables are 26 in the top flange and 10 in the bottom flange for the bridge length.

The bridge was attacked numerous times during the gulf war. The most serious damages occurred in the east part, where the three eastern spans (S8 to S10) were completely destroyed and span (S7) was completely cut near pier P(2-8). The remaining part of the span acted as cantilever, failed at P(2-7), deflected and pulled the six spans behind it a distance of about 1.25 m. The first span dropped from the bearings and the end diaphragm leaned on pier P(2-1). The six spans are supported on piers P(2-2) to P(2-6) and temporary supports near P(2-1) and P(2-7). Damages and temporary supports are shown in Figure 2.

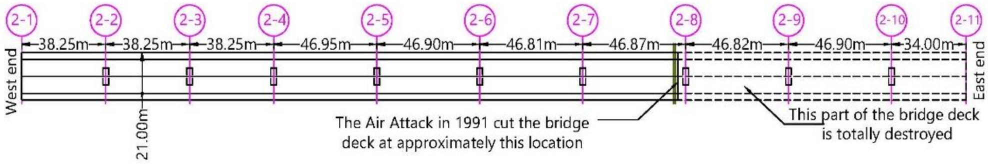

Figure 1. Plan of the bridge.

Figure 2. Damages of bridge

2 REPAIR WORK

Repair work includes the following:

- Repair of the local damages in the deck structure.
- Lift and reinstate the six spans to their original position.
- Construction replacement for the four spans and connects them with remaining six spans.

The contractor, who was entrusted the job, proposed the following:

- To construct the four new spans, (ILM) starting from east abutment is used. Double steel plate girder launching nose about 30.72 m is connected to the face of first increment using 50 mm diameter pre-stressing bars grade 1,030 MPa. Figures 3 and 4 shows the new spans during launching.

Figure 3. New spans and launching nose.

Figure 4. Inside end of new structure.

The following materials for construction are used:

- Concrete with 28 days' cylinder compressive strength of 40 MPa is used.
- Freyssinet system C&B strand cables TISS with steel grade S1640/1860 are used.
- Internal prestressing made of bonded strands in steal strip sheath.
- External pre-stressing made of individually greased and sheathed strands in external duct which is injected with cement before tensioning.
- Re-Charge of existing PT cables.
- Top pre-stressing tendons at pier P(2-7) used are 8-19T15S with total area of (22,800 mm^2), while at piers P(2-8) and P(2-9), 12-(19T15S) are used.
- The first increment is projected into span (6) by about 3.62 m, and away from clear cut end of old structure by about 2 m; cast in situ segment is used to connect two structures.

3 REINSTATING REMAINING PART OF THE BRIDGE

Two pairs of 500 ton capacity hydraulic jacks as shown in Figures 5 and 6 were provided at piers P(2-2) to P(2-6) and on one pair at temporary supports near P(2-1) and P(2-7), for vertical lifting. These jacks are supported on secondary steel chairs, which in turn supported on main steel chairs with very low friction pad separating the two chairs. Flat jacks were installed between the two chairs to cause transverse movement of the deck, after transferring the load of the six spans to the vertical jacks. After correcting the alignment of the six spans, the spans were lifted to their design level. Then the old bearings were replaced by new ones, which were designed to allow longitudinal sliding. The longitudinal movement of the six spans was achieved by using prestressing jacks for which special steel structures designed to be fixed to the two stiff diaphragms of the east and west bridges. The jacks' reaction was on the west diaphragm and the pull force acted on the east diaphragm, moving the six spans westward until their final location.

Figure 5. Reinstating original structure.

Figure 6. Reinstating original structure.

4 CONNECTION OF THE TWO STRUCTURES

Although span six did not show signs that the pre-stressing tendons loosing bond with the surrounding concrete of the box section, special steel anchors (shown in Figure 7) were used to anchor the ends of the tendons near P(2-7). The connection of the two structures is as follows (Freyssinet 2015):

- The 2 m long segment casted between the two structures. Two blisters were casted with the bottom slab, to anchor one end of two 19T15S bottom tendons, where the other ends were anchored at newly constructed blisters against the diaphragm at P(2-6).
- A special reaction frame was casted 10.82 m from P(2-7) inside the old structure against two already existing blisters on the webs of the old structure. This was used to anchor four prestressing bars near top of the box girder, then two rows of 4-19T15S tendons, one near each web. Each bar was stressed to about 1,500 kN, each of the upper two tendons was stressed to 3,700 kN, while each of the lower two tendons was stressed to 2,650 kN. The other ends of the bars and tendons were anchored at the diaphragm at P(2-7).
- For bottom external pre-stressing, two 19T15S tendons were provided to supply about 7,500 kN pre-stressing force. These tendons improved the resistances to positive moments occur in the middle region of span six and induce required stresses in the interface between the connecting segment and the old structure. To carry this out, six pre-stressing bars (3 on each on side), which were used to connect the bottom of the launching nose were, extended and anchored on the opposite face of the blisters used for the bottom pre-stressing tendons. The six bars induced a compressive force of about 9,000 kN to the bottom region of the east interface between the connecting segment and the new structure. The above system of pre stressing resulted in the following compressive stress pattern (Alani 2015):
    - At east interface, the top fiber stress is, σ_t =1.85 MPa, and the bottom fiber stress is, σ_b =5.02 MPa.
    - At west interface, σ_t = 1.83 MPa, σ_b = 4.72 MPa.
- The external pre-stressing used to connect the old and new structures cannot restore the full continuity of the original structure. Therefore, computer analysis was conducted for different rigidity conditions of the interface sections (fully rigid, completely hinge, and in between conditions were modeled). The deflections due to dead load of the deck structure at the interfaces were determined and compared with the actually measured deflections after the connection was completed and the jacks at temporary support near P(2-7) were removed. The degree of rigidity which resulted with deflections at the two interface sections closest to those actually measured was adopted and used in the analysis of the deck under the effect of live load which was used in the load test.
- Figure 8 shows the bridge after connecting both structures.

Figure 7. Anchoring old tendons.

5 LOAD TEST

5.1 Loads and Loads Arrangement

The carriageway width of the bridge is 15 m wide. Four notional lanes should be considered (BS5400 1978). Two notional lanes shall be loaded by HA-load load of 26.4 kN/m for a loaded length of 40 m in addition to 120 kN knife edge load while the rest shall be loaded with (1/3) HA load. Therefore, an estimated live load of 412 kN shall be applied. Using 12-350 kN trucks, the total load is 4,200 kN. The application of load is done through three stages, where four trucks are added in each stage. Figure 9 shows the final stage of loading.

5.2 Deflection Measurements and Evaluation

The following arrangements were taken:

- During load test, the deflection was measured at five points (mid-span, 11, 22 m to the west and east of mid-span).
- The locations of parking of the twelve trucks were marked on the road surface for exact positioning of the trucks during the load test. Then, the trucks were moved to their designated locations, at three stages and deflection measurements were taken by two surveying crews operating from platforms at piers P(2-6) and P(2-7). Readings were recorded after the deck structure was stabilized under each load increment.
- Finally, the trucks were moved outside the bridge in reversed order by three stages. The deflection measurements were taken after the deck structure was stabilized after each stage of unloading. The average values of the two sets of readings were calculated.

5.3 Results and Conclusions

The maximum average recorded deflection at mid-span is about 6 mm which represents about (1/7800) of the span length. Graphs of measured and calculated deflections at mid-span and at 11m on each side of the mid-span are plotted against loading, shown in Figure 10. In all these drawings, the graphs of measured deflections show almost a complete recovery of deflection after removal of loading with trivial residual. The comparison between the measured and theoretical deflections reflects the following:

- The recovery of deflection after removal of the applied load means that the deck structure was acting in the elastic range of the material (elastic behavior).
- The measured maximum deflections of girders are close to the theoretical values for the same loading conditions. This can be attributed to that the modeling of the deck structure reflects its actual conditions.
- In the measured deflections, East point, which is located within the new structure near pier P(2-7), shows smaller deflection than symmetrical point near pier P(2-6). The reason for this is that the end sections of the new structure has much higher rigidity than the rest of the box girder sections due to the increase of the webs thickness to accommodate the launching nose anchor bolts.
- In conclusion, the connection between old and new structures is efficient in properly transferring the loads between the two structures.

Figure 8. Bridge after connecting both structures.

Figure 9. Load test.

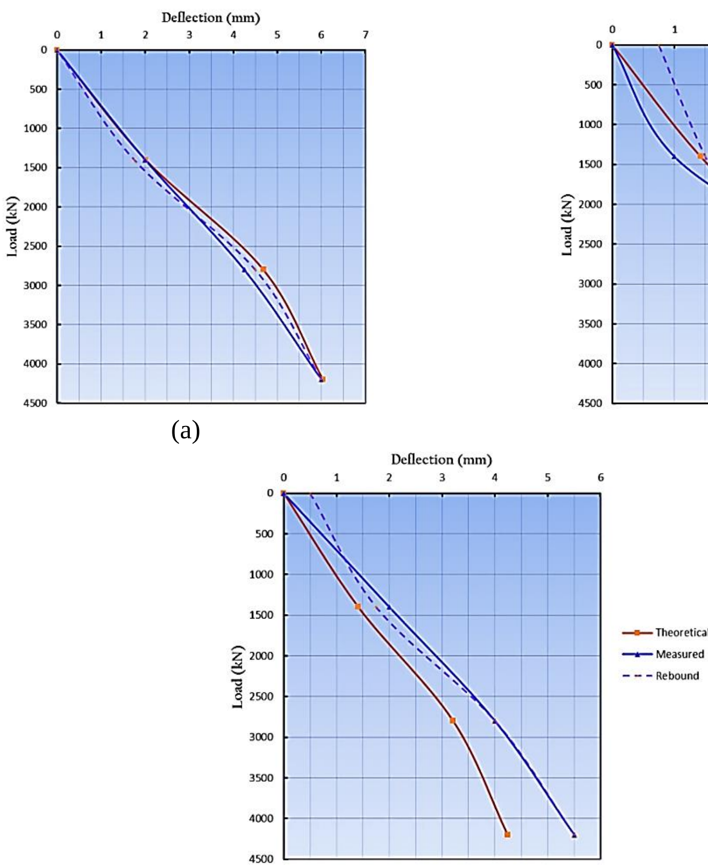

Figure 10. Load-deflection (a) at midspan; (b) 11 m west of midspan; (c) 11 m east of midspan.

References

Alani, K., *Al-Fayha`a bridge load test of span (6) Report*: An evaluation report submitted to Iraqi S.O.R.B., Alani & AlShamma Consultuncy Bureau, Iraq, 2015.

AlKhazen, Leonhardt & Andra, *Shop Drawings of Khalid Ibn Al Waleed Bridge*, AlKahzen Co., Iraq, 1974.

BS5400, Steel, Concrete and Composite Bridges: *Specification for Loads*, British Standards, P2, UK, 1978.

Freyssinet, *Al Fayha`a Bridge, Structural Analysis General Assumption*, Report No. N207EX-NCA501C, April 2015.

Polensky & Zöllner, *Prestressing System Technical Sheet*, Deutsches Institut für Bautechnik, Germany, 1969.