Interaction between Theory and Practice in Civil Engineering and Construction
Edited by Komurlu, R., Gurgun, A. P., Singh, A., and Yazdani, S.

ISBN: 978-0-9960437-2-4

REHABILITATION AND LOAD TESTING OF A BRIDGE

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Al-Fatha Bridge is constructed on Tigris River at Baiji town, about 220 km north of Baghdad. Beside its main purpose of connecting the main highway west of Tigris River with the Kirkuk city, its deck structure is purposely designed to carry pipes conveying crude oil from Kirkuk oil fields to Baiji refinery and to the oil exporting network of the country. During the war on Iraq in spring 2003, the bridge was subjected to an air strike. In addition to the damages caused by the impact and explosion of the munitions, the great fire of the crude oil erupted and lasted several days caused much heavier damages in different parts of the structure. The parts of the bridge damaged beyond repair, were replaced, while the parts which were found to have sufficient structural integrity were repaired. This paper addresses the repair and testing of the most affected span exposed to fire. The efficiency of the repair work had been evaluated by carrying out a load test on the above mentioned span. The deflection at different stages of loading were recorded at specified points and compared with theoretical results. Good agreement was obtained between the theoretical and measured deflections, which prove the adequacy of the repair work. This paper introduces briefly an assessment of damages of the bridge, the rehabilitation work, analysis and load test results.

Keywords: BS5400, Grillage method, Prestressed girder, Fire, Deflection, Repair.

1 INTRODUCTION

Al-Fatha Highway Bridge consists of ten equal simply supported spans of 42m long each. Each span consists of two separate, side by side, super structures supported by common wall type piers and cellular type abutments.

The two super structures are separated by about 50mm wide gap. Each super structure contains 7.1m wide carriageway, 2.3m side walk and 0.4m wide raised median curb. The super structure is made of three 3.5m deep bulb-T precast prestressed concrete girders and 30cm thick cast in situ R.C. deck slab on top of the girders. The spacing between each two adjacent girders is 3.2m. In these spaces a number of pipes carrying crude oil pass. The bulb of girders is 0.8m wide and 0.6m high and two 0.3m high inclined sides which end with a 0.22m wide web. The top flange is 1.22m wide with 0.2m end thickness and 0.25m thickness at the junction with the web. The bulb houses four pre stressing straight tendons. Another two parabolic tendons start at 2.73m and 3.37m at ends of girders and drop to 0.59m and 0.78m above the bottom of girders at mid span. Each tendon is made of 19-ϕ10.5mm grade

1670MPa strands complying with BS5896-80. The concrete cover for the straight tendons is about 150mm. While the concrete cover for the parabolic tendons within the web is about 60mm. All the information about the deck structure mentioned above were gathered from site measurement of damaged girders, since no document was available about the bridge after most of the important government offices were vandalized by arson during the invasion. The air attacks on the bridge, besides causing heavy damages due to the impact and explosion of weapons, they ignited a great fire of the crude oil contained in the pipes passing through the deck structure.

2 ASSESSMENT OF DAMAGES

Many missions of inspection for the bridge took place. The first one was made by the General Corporation of Roads & Bridges (GCRB) engineers. Then after the rehabilitation contract was awarded, many inspection missions were made for the bridge. The reason for these multiple inspection was the difficulty of accessing the deck structure from below because the space between the girders was full of distorted steel pipes and their steel supports. See Figure 1. The obvious damages recorded immediately after the attack, were the following, noting that span numbering starts from west (Baiji) end of the bridge:

- A complete collapse of span nine.
- Large opening in the middle of the upstream carriageway of span six with heavy damage of its middle girder.
- Excessive deflection of the downstream carriageway of span eight.
- The other spans were affected to a varying degree, where the fire had caused peeling of concrete cover and the exposure of reinforcing bars. Cracking appeared in some of the main girders.
- The two abutments show only minor damages.
- Piers P1, P2 and P3 were in good conditions except for their crossheads which suffered moderate to minor damages. Piers P6, P7, P8 and P9 suffered heavy damages throughout their exposed parts above water line. While piers P4, P5 suffered moderate damages at parts above the mid height of piers.
- Most of the expansion joints were damaged.
- After thorough study of the results of inspections, it was decided to replace spans six to ten and demolishing the exposed parts of piers P6, P7, P8, and P9, then rebuild them to the original design.
- As for the rest of the bridge parts, which mainly suffered from exposure to fire, a decision was taken to restore them to their original capacity by carrying out engineering repair work. Among these, span five being closed to span six, which was subjected to the direct attack, and hence suffered more from the fire caused by the attack. See Figure 2. Thorough inspection of span five was conducted and the state of each girder was documented. In general, the elements of the deck structure did not exhibit signs of distress, like excessive deflection and/or heavy cracking.

3 REPAIR WORK

The repair work was mainly concentrated on the replacement of the affected surface concrete by good quality repair material. In addition, the cracks should be sealed with suitable filling material. Therefore, the repair scheme used for the deck structure included the following:

- The removal of all the loose materials until a layer of sound concrete was reached.
- Then a water jet was applied to the exposed areas of concrete in order to remove the dust and the remaining loose materials, thereby leaving clean and sound surface. See Figure 3.
- The next step was to fill all visible cracks with structural quality injection material, i.e., low viscosity high strength epoxy sealing system conforming to ASTM C881 Specification. Injection material was selected and applied according to the instructions of the manufacturer. This type of material can be used to seal cracks up to 50mm width.
- After completion cracks injection work, the following task was to apply the replacement for the damaged surface layers. For this task, high quality shrinkage compensated cementitious material with polymer fiber reinforcement in order to reduce cracking, and with good bonding properties with underline concrete was chosen. A suitable material was chosen and applied by pressure sprayer. See Figure 4. Because the areas which were covered were large, they were divided to small panels by narrow guiding strips made of the same material to the required thickness. Then these panels were sprayed and troweled to the accurate level of the guiding strips. See Figure 5.
- At one location, a small area of the surface concrete had to be removed deeper than thickness of concrete cover. In this area, normal concrete with polymer fiber was casted against vertical formwork to bring it to the level of adjacent clean surfaces. Then finally it was covered by spraying repair mortar with the rest of the web.

4 EVALUATION OF SPAN 5 STRENGTH

As a conservative assumption, only the four straight prestressing cables are assumed to be effective in resisting the applied moments in the girders. Calculations of the resisting moment were carried out according BS5400-P4. The calculation of maximum bending moments and shear forces due to factored loads are less than the calculated resisting capacity of the girder. From these calculations, it was concluded that additional external reinforcement will not be necessary, Alani & AlShamma Consultancy Bureau Report (2010).

5 LOAD TEST

After completing the repair work of span five, a load test was carried out to verify the adequacy of the repair work conducted on the girders and deck slab of the upstream carriageway.

5.1 Loads and Loads Arrangement

A carriageway 7.1m wide, according to BS5400-P2, consists of two notional lanes. For design purposes, either the two notional lanes are loaded with full HA load, or one lane is loaded with full HA load while the other is loaded with HB load. For an effective span for girders of 41.44m, the total span load which gives maximum deflection at mid-span was determined to be about 2000kN. To simulate the effect of this load, eight trucks were selected each weighs 250kN. See Figure 6. This load was applied in four increments; each consists of two trucks, i.e., 500kN per increment.

Figure 1. Damages of bridge.

Figure 2. Damages of span five.



Figures 3, 4, & 5. Different stages of repair work.

Figure 6. Load test (full load stage).

The deck structure was modeled as grillage using composite section properties for the members in the main span direction and rectangular elements having a thickness equal to that of the deck slab. The analysis was carried out for the same load increments and loading positions as per the load test. The theoretical deflections at three points along each of the three girders (mid-span and quarter points) were calculated.

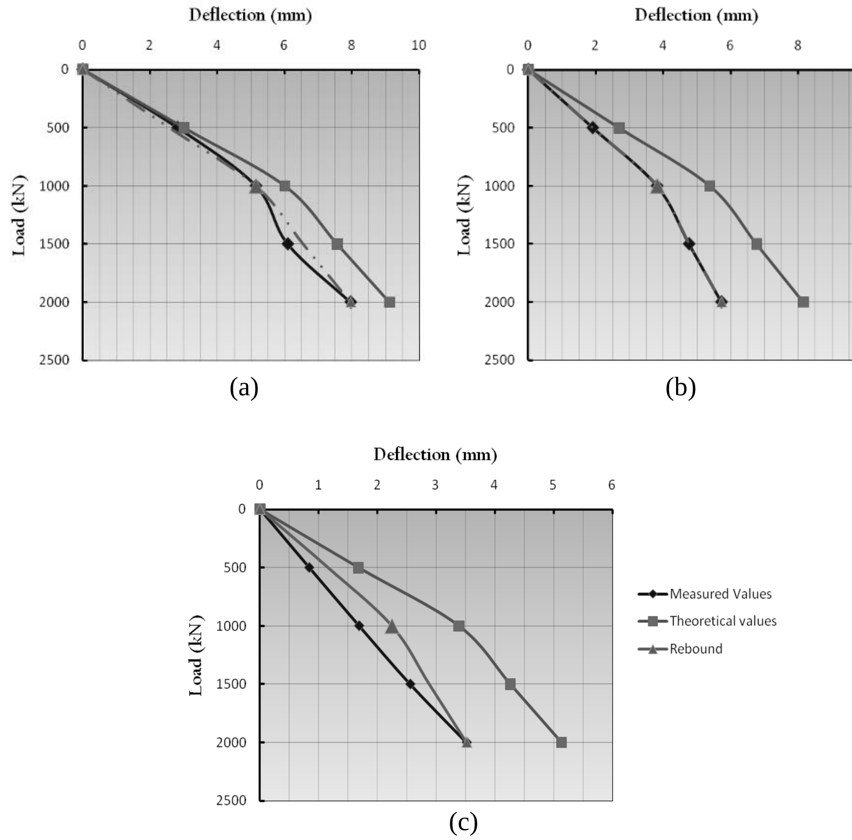

Figure 7. Load-deflection at midspan (a) girder 1. (b) girder 2. (c) girder 3.

5.2 Deflection Measurements and Evaluation

A load test is carried out on October 10th, 2010. Special platforms were designed, constructed and erected on the piers at the two ends of the span at a level of about 2m below the bottom of girders. These platforms were used by the surveying crews.
Six dial gauges were placed under the ends of the girder for measuring end settlement (elastomeric vertical deformation). Three measuring sticks were fixed to the soffit of each of the three girders (mid-span and quarter points).

The locations of the left rear tires of the eight trucks were marked on the road surface for accurate positioning of the trucks during the load test. Then, the trucks were moved to their designated locations at four stages. Readings were recorded after the deck structure was stabilized under each load increment.

After reading the deflections under the full test load, the four trucks closer to the ends of the span were ordered to leave the span, then the deflections were recorded after the structure was stabilized. Finally the last four trucks were moved out of the span, and the deflections were recorded.

6 RESULTS AND CONCLUSIONS

Graphs of measured and calculated deflections at mid-span of the three girders are plotted against loading. See Figure 7. In this Figure, the theoretically calculated deflections are higher than the measured values. The graphs of measured deflections show complete recovery of deflection after removal of loading.
The above results reflect the following:

- The complete 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 smaller than the calculated values for the same loading conditions may be attributed to that the approximation used in the modeling of the structure is on the conservative side and that the repair material properties are appreciably higher than the concrete used in the original structure, i.e., the repair material has little bit higher modulus of elasticity (E_c).

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