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STATIC AND DYNAMIC LOADING TEST OF A RAILWAY BRIDGE

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Abstract: A case study for assessing the strength of a recently mal constructed Iraqirailway bridge was carried out. The 48.5 m long Reinforced Concrete Bridge shows a Permanent deflection of more than 3 cm at its mid span panel. Responsible Authorities feared of the safety of this bridge and asked for Structural engineering consultancy. The overall bridge elements; piers, bearings, super structure and materials strength have been verified in details, but in this paper the concentration will be focused at a novel method used to perform a loading test. The heaviest available locomotive-weighing 120 tons- was used to conduct the static and dynamic loading test. Surveying team was instructed to tabulate the levels of selected points before and after the passing of the testing locomotive and during its stoppage at certain positions. The bridge showed an acceptable performance under the actual loading of the mentioned locomotive and also it complied with the resisting requirements of the Cooper E-80 standard loading for railway bridges.
Keywords: Static, Dynamic, Loading Test, Railway Bridge, Strength Evaluation

INTRODUCTION

AD'DIWANIYA one way Railway Bridge – 300km south of Baghdad - has a total length of 48.5m and an overall width of 8m. It was constructed in June 2010 across AD'Diwaniya River. It consists of three simply supported spans, the Northern span towards Baghdad has a length of 13.8m and it is supported by seven reinforced concrete girders, the Southern span-towards AS'SAMAWA city has a similar length and supporting girders while the intermediate span has a length of 20.8m supported by nine reinforced concrete girders, as shown in Figure 1.

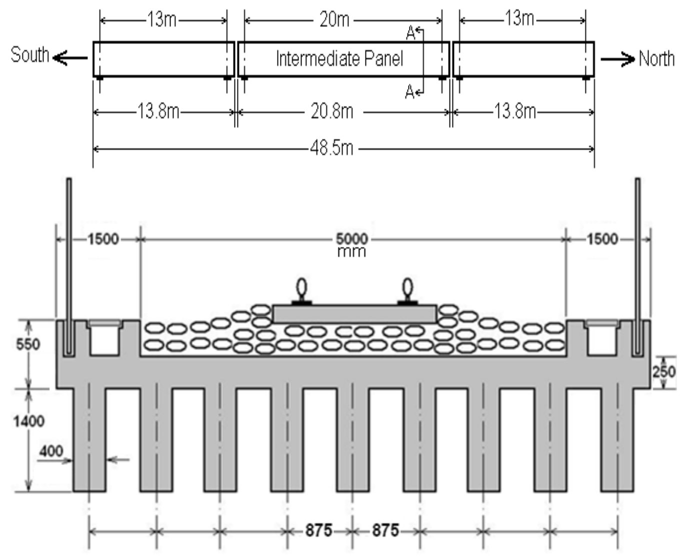

Figure 1. Bridge Profile andits intermediate panel Cross Section

All concrete girders have the same stem outside dimensions of 1400mm x 400mm, but the reinforcingsteel varies between the middle panel and the other two panels. All the girders were pre-casted on site, lifted to its final position and connected by shear connectors to a reinforced concrete - cast in-situ -deck slab to ensure the composite action.

The construction of the bridge had started in the year 2004, and then stopped for few years until it was completed in June 2010. During that period, there were some problems due to improper storage of the pre-casted girders on site.A dispute initiated regarding the strength and durability of its concrete, the extent of corrosion in its reinforcing steel which logically will be reflected on the overall structural integrity as well as its effects on the safety to serve as a major structural part of a durable railway bridge sufficient to sustain the repetitive exposure of dynamic loads.

A structural site investigation had been done to verify every part of the bridge to Figure out if it shows any signs of defects or failures. Moreover, a surveying measure for the levels and deflections at 14 selected points along the bridge profile had been recorded to check the, as built, overall geometry perfection of the bridge. An actual static and dynamic live loading tests have been done by passing the heaviest available locomotive (weighing 120 tons) at different speeds and while it was stopping at selected spots on the bridge deck slab. Again by the aid of the accompanying survey team, all the actual deflections under dynamic loading were listed. The measured deflections were compared with the allowable deflections permitted by the standard codes for such type of bridges.

ANALYSIS OF THE BRIDGE

Moment resisting check

To check the bridge initial design adequacy, the following detailed analysis according toAASHTO Specifications has been done[1].

Checking of the girders design:

$$\text{Dead load per linear foot} =$$
$$\left(\frac{10}{12}\times2.87+\frac{55}{12}\times\frac{15.7}{12}\right)\times150+\left(\frac{12}{12}\times2.87\times75\right)=1475\ \text{lb/ft}$$
$$\text{Dead load moment } M_d=\frac{1475\times65.5^2}{8}=790{,}185\ \text{ft.lb}$$

According to the American Railway Engineering Association (AREMA) Cooper E-80 train load was used to represent live loading[2] (Figure 2).

With an impact factor of $I = \frac{50}{65.5+125} = 0.26$ *and for a bridge having nine girders, the Cooper Axle loads will be multiplied by 1.26/9= 0.14 for each girder. Then the case shown in Figure 5 represents the most critical live loading for moments.*

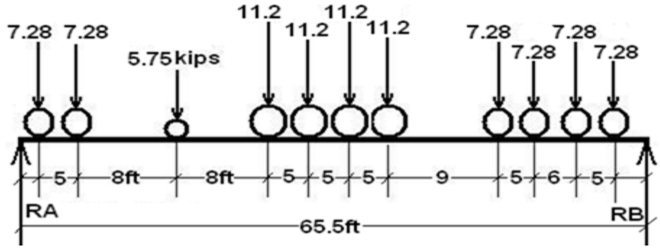

Figure 2. *Cooper E-80 Loading Position for Maximum Moment*

RB= 47.27kips

Maximum life load moment

M_{Lmax}*= 47.27x32.75 – 11.2x5 – 7.28(14+19+25+30) = 852,000 ft.lb*

According to AASHTO, Sidewalk live load of 85 psf will be applied giving an additional load of (7x85)/9 = 66 lb/ ft/ girder.

Moment due to sidewalks live loading is (66x65.5^2*)/8 = 35,454 ft.lb*

Total moment for each girder equals:

$$MT = 790{,}185 + 852{,}000 + 35{,}454 = 1{,}677{,}638 \text{ ft.lb}$$

Required area of steel is:

$$As = \frac{MT}{f_{s}\left(d-\frac{t}{2}\right)} = \frac{1{,}677{,}638\times12}{30{,}000\left(58.5-\frac{10}{2}\right)} = 12.54 \text{ in}^2$$

Required #10 bars are = 12.54/ 1.27= 9.876 ≈ 10 bars

Therefore the original design is perfect regarding the moment resistance of the girders.

Checking the Concrete Compression Limits:

By the following equation the maximum actual compression of the concrete can be calculated.

$$f_c = \frac{MT}{\left(1-\frac{h_f}{2kd}\right)bjd\times h_f} = \frac{1{,}677{,}638\times12}{\left(1-\frac{10}{2\times0.324\times58.5}\right)2.87\times12\times10\times0.89\times58.5} =$$

$$= 1525psi < 2000psi$$

(Therefore the original design is perfect regarding the maximum compressive stresses subjected to the concrete of the girders)

The maximum compression of concrete will not exceed 1525psi which is less than the maximum permitted limit of 2000psi. This result will ensure that there will be no overstress at the deck slab and it will also be useful in the process of strengthening of the bridge girders.

Shear resisting check

Maximum Dead load shear

$$V_{dmax} = \frac{1475\times65.5}{2} = 48{,}306\ lb$$

Maximum live load shear according to Cooper E-80 train loading can be calculated when the train position gives the most critical shear as shown in Figure 3.

Maximum live load shear at each of the girder supports $V_{L\,max}$*=RB*

$$= \frac{11.2\,(4.5+9.5+50.5+55.5+60.5+65.5)}{65.5}$$

$$+\frac{7.28\,(18.5+23.5+29.5+34.5)+5.75\times42.5}{65.5}$$

$$= 57.6 \text{ kips} = 57{,}600 \text{ lb}$$

Sidewalks live load shear $= \frac{66\times65.5}{2} = 2160$ *lb*

Total maximum shear at each of each girder supports is:

$$V_T = 48{,}306+57{,}600+2160 = 108{,}066 \text{ lb}$$

108,066 lb = 48 Tons, this load will be used for the design and check for each bearing pad)

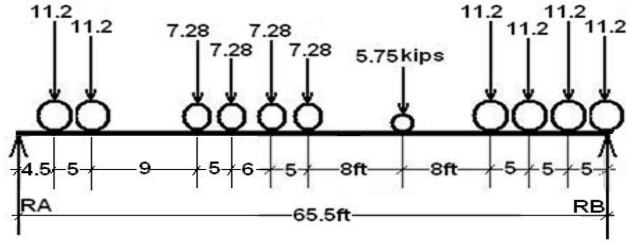

Figure 3. *Cooper E-80 Loading Position for Maximum Shear*

According to the specifications the most critical shear section is situated at a distance equals to d from support, so the calculations for shear at d (58.5 in ≈ 4.5 ft) from support will be as follows:

Maximum live load shear at d from support, according to Cooper E-80 train loading can be calculated when the train position gives the most critical shear as shown in Figure 4.

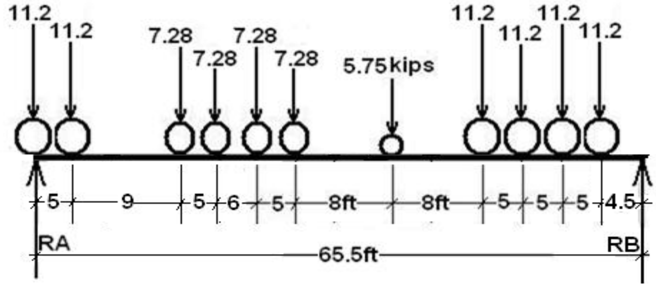

Figure 4. *Cooper E-80 Loading for Maximum Shear at d from support*

$$V_{Lmax\,@d} = \frac{11.2(5+61+56+51+46)}{65.5}$$

$$+\frac{7.28(14+19+25+30)+5.75\times38}{65.5}$$

$$= 50.5 \text{ kips} = 50{,}500 \text{ lb}$$

Total maximum shear at d from each of each girder supports is:

$$V_{Tmax@d} = 48{,}306 + 50{,}500 + 2160 = 100{,}966\ lb$$

Allowable shear stress of concrete is:

$$0.95\sqrt{f_c'} = 0.95\sqrt{5000} = 67\ psi$$

Shear stress at d from support is:

$$\frac{100966}{65\times15{,}7} = 100\ psi$$

Required spacing of #3 bars is'

$$S = \frac{0.22\times24{,}000}{(100-67)15.7} = 10.2\ in$$

{Again the original shear reinforcement design of #3@6"c/c (Φ10@150mm c/c) is accepted}

BRIDGE GIRDERS

Northern and Southern Panel Girders:

Despite of the moderate construction level of the Northern and Southern panels' girders they show no clear signs of failure.

Intermediate Panel Girders:

The Intermediate panel is supported by nine girders designated as G8, G9, G10, G11, G12, G13 G14, G15 and G16. Each girder spans 20.8 m with a stem of 1.4 m. These girders show visible mid span deflections of more than 3 cm as shown in Figure 5.

***Figure 5**. Visible mid span deflection at the right side of the intermediate panel*

Most of this visible mid span deflectionsare due to the lack of construction experience. The constructing team did not care about the construction stresses generated during the casting of the concrete of the deck slab process.This fresh concrete weighs more than 100 tons and it was applied only to the rectangular portion of the girders[3]. An expert construction contractor would not cast such long girders by using flat bottom formworks, but instead they might raise the middle of their formwork by a Cambering process[4]. The amount of cambering depends upon the span length, loading and experience to avoid this inevitable deflection. In spite of this visible deflection at the bottom face of these girders which exceed 3 cm, leveling measurements at the top of the deck slab of this panel showed no deflections. This essentially means that the construction team had increased the deck slab thickness to produce the required formation level and consequently slightly increased the dead load of the bridge. Nevertheless, this deflection shall only affect the aesthetic appearance of the bridge and may result in some minor cracks.

***Figure 6**. The Locomotive used in loading Test*

LOADING TEST

In order to evaluate the overall structural performance of the intermediate panel girders, a real full scale load testing had been performed. The Iraqi Republic Railways company kindly provided the heaviest available locomotive shown in Figure 6 which has the following properties: 120 tons of weight, 20 m of length and 6 axles.

Testing Procedure

First of all the locomotive had been stopped at several selected points to check shear and moment resisting strengths of the bridge and to measure deflections under static live loading, see Figures 7, 8, 9, 10, 11 and 12.

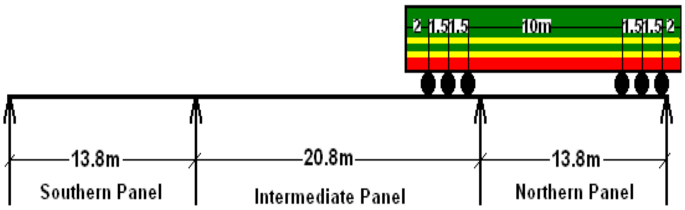

***Figure 7**. Checking Shear for Northern Panel*

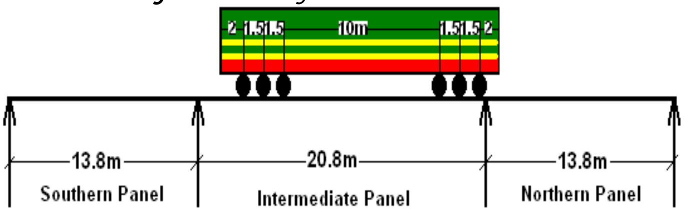

***Figure 8**. Checking Shear for Intermediate Panel*

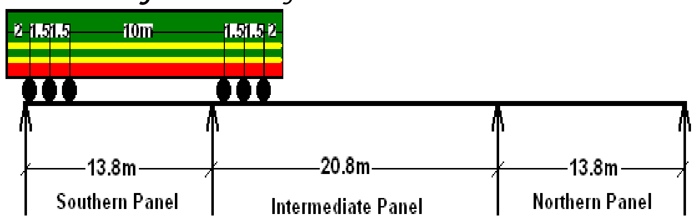

***Figure 9**. Checking Shear for Southern Panel*

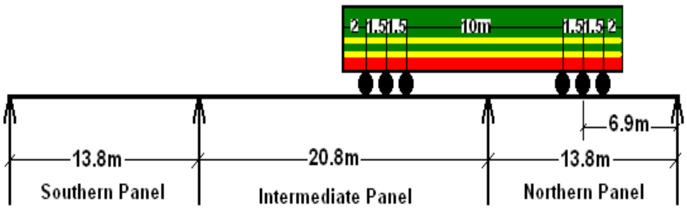

***Figure 10**. Checking Moment and Deflection for Northern Panel*

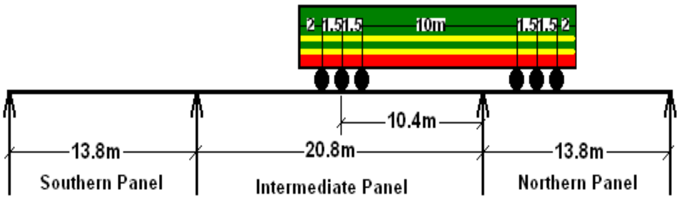

***Figure 11**. Checking Moment and Deflection for Intermediate Panel*

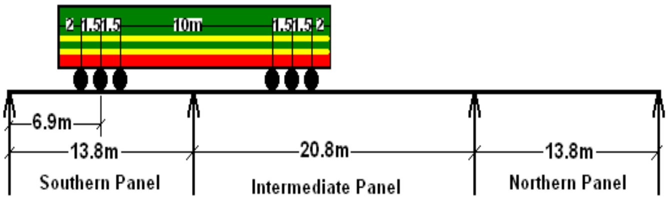

***Figure 12**. Checking Moment and Deflection for Southern Panel*

At this static live loading test there were no signs of Shear or Moment failures, and the recorded deflections were negligible.

The Second phase of the test was the Dynamic Live Loading Test. At this stage the Testing Locomotive was moving at a speed of 40 - 50 *km/h while deflections were measured. A new and easy method was adapted by perpendicularly holding the surveying ruler at mid sidewalk point of each tested panel while the level was set in a stable position outside the bridge, see Figure 13. The dynamic stage is considered more severe than the first static stage because the weight of the testing locomotive of 120 tons will be increased according to AASHTO Standards by 26% due to the impact and sudden application of the load. This will make the locomotive apply about 151 tons instead of 120 tons, which results in a single axle load of 25tons.*

***Figure 13**. Method used for measuring deflections under dynamic live loading*

Measured deflections under dynamic live loading are shown in Figure 14. Deflections of -1mm were recorded at mid spans (P3, P4, P11 and P12) of both of northern and southern panels. These deflections were less than the permitted maximum deflection by the AREMA'S live load deflection criteria which is equal to L/640;

13000/640 = 20mm

The settlements at P6 and P5 might be a normal reduction in the height of the supporting bearings under such heavy loading. While the settlements at both of P9 and P10 were little bit larger due to the torn and worn bearings which were recommended to be replaced.

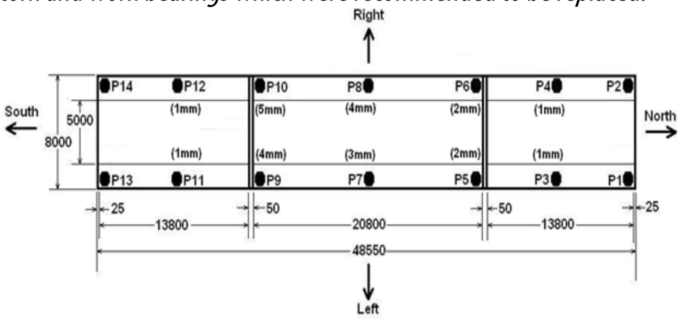

***Figure 14**. Settlement and Deflection measurements under dynamic live loading*

Deflections at P8 (-4mm) and P7 (-3mm) at mid span of the intermediate panel require the following analysis:

Figure 15 shows the position and the amount of loading that will produce the maximum moment at mid span of the intermediate panel.

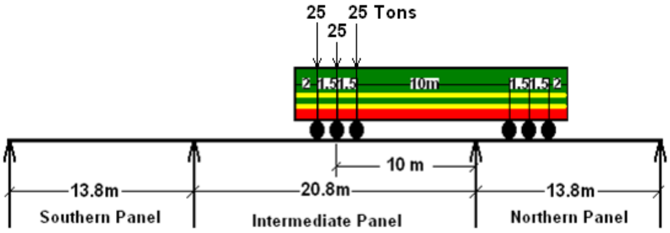

***Figure 15**. Position and amount of loading that produce maximum dynamic moment at mid span of the intermediate panel*

Maximum moment/Girder = (37.5x10-25x1.5)/9 =37 t.m

The original design was based on Cooper E-80 Loading of 300 tons for this span. The share of each girder was 300X 1.26/9= 42 tons. This load will produce a maximum moment of 115 t.m + a maximum moment due to sidewalks live loading of 5 t.m. This means that the deflection of P8 under the maximum design live loading equals:

4 x 120/37 = 13 mm

According to AREMA'S live load deflection criteria which is equal to L/640, the maximum permitted deflection for this panel under live loading is:

20000/640 = 31mm

(Therefore, this panel does comply with the permitted deflections)

Finally, although the bridge has passed the deflection live loading test, this bridge was strengthened by the use of Carbon Fiber Reinforced Polymer (CFRP) to enhance its performance and durability due to its cracked concrete.

CONCLUSIONS:

The following conclusions can be derived from this case study:

- *Loading Test of an Existing Railway Bridge can be done by applying the heaviest existing Train Locomotive.*
- *The recorded test deflections can be modified based upon the difference between the testing and the standard live loading weights.*
- *Some deflections of railway bridges can impair its atheistic appearance but still these bridges can stay in action for a long time.*

REFERENCES

[1.] Standard Specifications for Transportation Materials and Methods of Sampling and Testing, 33rd Edition and AASHTO Provisional Standards, 2013 Edition

[2.] American Railway Engineering and Maintenance-of-way Association (AREMA) Standard, Vol.2, 2013 Edition.

[3.] Alaa M. Darwish et all, "An Innovative erection technique for reducing two- third of constructional bridge deflections", European Scientific Journal ESJ, Vol.9, No.1, Macedonia, February, 2013.

[4.] Alaa M. Darwish et all, "Eliminating half of the constructional deflections of simply supported steel bridges by using temporary continuity", ACTA Technical Bulletin of Engineering, Vol. 2, No. 1, Faculty Engineering, Romania, 2013.