SAFETY ASSESSMENT OF CONCRETE RAILWAY TIED-ARCH BRIDGE IN EGYPT

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SUMMARY

An existing concrete tied-arch bridge of five simply supported spans is assessed in this study. The bridge carries a single railway track linking Etay-Elbarood city (El-Behaira Governorate, North West Cairo) to Cairo city. Visual inspection of the bridge's structural elements was carried out to define the geometrical properties and potential defects of these elements. Non-destructive testing was also conducted to determine the concrete characteristics and properties of the bridge components. Measurement data from load testing were collected to identify the structural behaviour of the bridge. Consequently, three dimensional finite element model of the bridge was performed based on test results and the model was used for structural assessment and evaluation of the bridge. The analytical model was also calibrated by using static load testing.

Keywords: *Arch Bridge, assessment, visual inspection, non-destructive testing, finite element model, load testing.*

1. INTRODUCTION

Because of their simple styles, light weight, aesthetic appearance, and ability to carry heavy loads over large spans, concrete tied-arch bridges have been of interest to bridge designers for the last five decades [1]. The use of reinforced concrete allows much greater freedom in the choice of arch curve and very large spans are possible even with heavy loads without the necessity of shaping the arch curve to the pressure curve.

In a tied-arch bridge, the thrust is carried by the arch girder, but for variable loading conditions flexural effect is produced, and shared by both arch girder and tie, depending on their respective stiffnesses [2]. In tied arch bridge, the arch is strongly compressed and internally balanced by a tension tie at deck level of a through or half-through arch [3].

The analysis of arches depends largely on how the ends of the arch are fixed. Arches are often more complicated than simple statics can determine. It is important to consider moving loads over portions of the bridge, as simply loading up the bridge with the full weight is not always the most critical case. Loading vehicles/ or trains at different locations across the span may create bending in the arch that control the design [4].

The objective of this work is to make assessment for the bridge behaviour experimentally, and analytically, and check if strengthening and/or repair is needed.

2. BRIDGE DESCRIPTION

The concrete bridge was built in 1973 for carrying a single railway track. The bridge is a through tied arch bridge with a total length of 108.55 m and width of 5.25 m. The bridge has five simply supported spans, by a movable hinge at one end and fixed one at the other. Each of the side spans 1 and 2 lie on top of two lane traffic roadways while the intermediate spans 2, 3, and 4 lie on top of the Nubaria navigational canal. Spans 1 and 2 are of identical lengths with 16.82 m for each span, while spans 2, 3, and 4 are of 22.80 m length for each. The structural system of the bridge consists of two main parallel arch girders connected at their tops by two bracing beams, tie beams at the deck level, and hangers which support the bridge floor system. The floor system of the bridge consists of the deck slab, longitudinal and cross beams. The rail track and sleepers are resting on compacted aggregate (ballasted floor) for transmitting the load to the main elements of the bridge. Fig. 1 illustrates the structural system and the layout of the bridge, and Fig. 2 illustrates the bridge cross section. The skew angle of the bridge is 34 degrees. The rise of the arch amounts to 5.95 m giving rise-to-span (f/L) of 1/2.83 and for side spans and 1/3.85 for intermediate spans.

Fig. 1. *The structural system and the layout of the bridge.*

Fig. 2. *The bridge cross section view.*

The cross sectional dimensions of the different elements of the bridge and the rail system are also illustrated in Fig. 3.

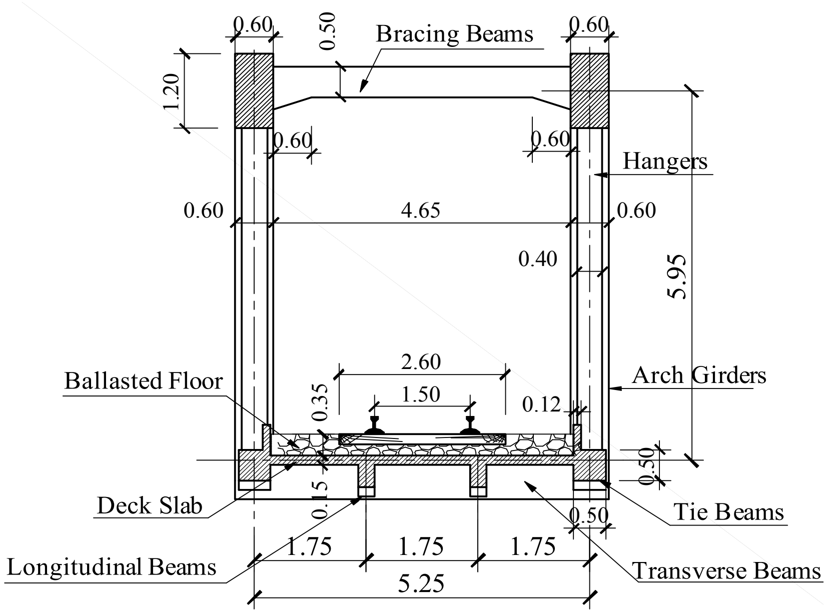

Fig. 3. *The bridge cross sectional dimensions.*

3. ASSESSMENT APPROACH

The assessment of the bridge was conducted using a three dimensional finite element model. In order to determine the defects of the geometric and material properties of different elements of the bridge that will be used for the model, visual inspection, non-destructive (in situ and laboratory) testing for the bridge elements was examined. Load testing was also conducted to calibrate the bridge model.

The bridge was investigated in detail on site for geometrical configuration of the structural elements, and for the anticipated defects. The structural parameters, defect characteristics were recorded as follows:

3.1. Measurements

The cross sections of the entire structural members of the bridge, span lengths, beam and hanger spacing, and hanger rises were measured. Also, bridge width, ballasted floor thickness, and skew angle were identified. The as-built drawings for the bridge components and dimensions were generated for comparison with the original drawing of the bridge. In general, the measured cross sections and dimensions were found to be matching those in the original bridge drawings.

3.2. Defects of arch girders

The arch girders of all spans of the bridge were found to be in an overall good condition. However, only some of the arch girders show the evidence of non-structural surface cracks. The observed cracks are usually shallow and located in the bottom or in the side surface of the arch girders as illustrated in Fig. 4. These cracks may be attributed to thermal expansion and contraction of concrete, contraction of the concrete during the curing process, or temperature gradients within massive sections of concrete [5]. These cracks generally do not affect the load-carrying capacity of a member, but may lead to higher susceptibility to other types of deterioration such as steel corrosion and thus repair is needed.

Fig. 4. *Non-structural cracks of the arch girders.*

3.3. Defects of transverse beams

On the sided traffic roadways underneath span 1 and span 5 localised damages at the soffit of reinforced concrete floor beams were recorded. These are due to the impact of overheight trucks and as a result, concrete has spalled off and the steel reinforcement was exposed to serious corrosion. Fig. 5 illustrates the collision and the wrongly repaired transverse beams of spans 1 and 5 respectively.

Fig. 5. *Damage of transverse beams of spans 1 and 5.*

3.4. Defects of hangers

As it appears in Fig. 6, forceful way for fixing the steel plates around the bottom of hangers caused the parts nearby to be crushed and an extension of vertical cracks took place. Fig. 6 illustrates the damage and cracks and the crack extension of the bottom part of the smashed hangers in both spans 1 and 5. The cracks on the parapet at transverse beam-hanger connection explain the reason of hanger crash.

Fig. 6. *Damage of hangers of spans 1 and 5.*

3.5. Non-destructive testing

The primary part of the bridge assessment is to define the strength and quality of concrete. To define the strength and quality of concrete, in site and laboratory measurements were conducted. The conducted non-destructive testing on the main structural elements of the bridge consisted of ultrasonic pulse velocity procedure, hammer rebound (Schmidt sclerometer), and drilled core testing.

3.5.1. Ultrasonic pulse velocity

Measurement of the velocity of ultrasonic pulses of longitudinal vibrations passing through concrete was used for determining the uniformity of concrete in and between members. The tests were conducted in site on 10 positions and the results showed that the concrete is fairly homogenous and lacking of voids.

3.5.2. Hammer rebound

The Schmidt rebound hammer is principally a surface hardness tester. It works on the principle that the rebound of an elastic mass depends on the hardness of the surface against which the mass impinges [6]. Empirical correlations have been established between strength properties and the rebound number. Thirty four positions on different elements of the bridge have been teseted by the hammer rebound. The result values of concrete strength were found to be greater than the specified strength of 27.5 MPa.

3.5.3. Drilled core testing

Concrete cores are usually extracted by drilling using a diamond tipped core cutter cooled with water. Eleven samples of cores were extracted from different elements of the bridge. All samples showed to be in a very good condition, no evidence of deterioration, cracks, rebar corrosion or separation were erecorded. The tested core samples also validated the specified design strength of 27.5 MPa.

3.6. Bridge modeling

A three dimensional finite element model (FEM) was performed to evaluate the bridge behaviour. The commercial software SAP2000 [7] has been employed to model and analyze the bridge previously described. Arch girders and bracing beams, vertical hangers, tie beams, bridge floor beams (both longitudinal and transverse), and end beams were idealized using frame elements. The deck slab was idealized using shell elements. The two supports of each arch girder was modelled as movable hinge at one end and fixed at the other. The rotations about the major axis of the tie beams at both ends were released so that the bending moments at both ends are vanished. In addition, the axial translations of the deck slab were released in order for the tie beams to capture the entire axial forces. Fig. 7 illustrate the isometric view of the three dimensional model of the bridge.

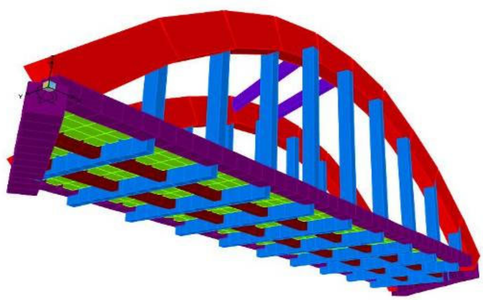

Fig. 7. *Isometric view of the three dimensional model of the bridge.*

3.7. Bridge Loading

3.7.1. Dead Loads

The dead load of the bridge includes the self weight, ballasted floor, and the rail system. The self weight was calculated automatically by the software by determining the specific weight of reinforced concrete. The specific weight of reinforcing concrete was taken 25 kN/m^3. The weight of the ballasted floor and rail system was estimated as 10 kN/m^2.

3.7.2. Moving loads

Train type "D" was considered as the moving load on the bridge in accordance with the Specification of Egyptian Railway Authority [8]. Train type "D" consists of two locomotives and two tenders followed on one side only by an unlimited number of wagons (Locomotive +Tender + Locomotive + Tender + unlimited number of wagons). The total weight of one Locomotive is 100 ton, and its length is 10.50 m, while total weight of one Tender is 80 ton, and its length is 8.40 m.

3.7.3. Impact loads (*I*)

The dynamic effect caused due to vertical oscillation and periodical shifting of the live load from one wheel to another when the locomotive is moving is known as impact load. The impact load is a product of impact factor (*I*), and the live load. The impact factor specified by the Egyptian National Railway [8] is defined from the following equation:

$$I = \frac{26}{L + 24} \tag{1}$$

where L = the effective length of the loaded member in m.

4. RESULTS AND DISCUSSIONS

The behaviour of bridge superstructure elements was evaluated under dead loads (DL) and moving load with impact (LL+*I*) based on the geometrical dimensions and material properties identified from visual inspection, non-destructive testing. The concrete strength fcu=27.5 MPa obtained from testing was used for the analytical model.

4.1. Design verification

The load carrying capacity of each member of the bridge was determined from the cross sectional areas and the area of steel bars provided in the original drawings. The load carrying capacity of each member was then checked against the ultimate load resulting from the analytical model. The load combination used for verification is as follows:

$$U = 1.4\ DL + 1.6\ [LL + I] \tag{2}$$

4.1.1. Floor slab and floor beams

The moment capacity of deck slab and floor beams of all spans in both transverse and longitudinal directions was found to be adequate for resisting the applied ultimate moment with a reasonable margin of factor of safety.

4.1.2. Arch girders, tie beams and hangers

Fig. 8 and 9 show the ultimate axial forces for the arch girders, tie beams and hangers of the intermediate spans, and ultimate moment for arch girders respectively. Tie beams and hangers are being predominantly tension members as it is demonstrated in Fig. 8, while arch girder is being predominantly a compression member with small values of bending moment. It was found that the nominal axial capacity of tie beams and hangers of all spans is much larger than ultimate axial force resulting from ultimate loads. Also, the arch girder with provided steel bars was found to be very adequate to carry the ultimate moment and axial force.

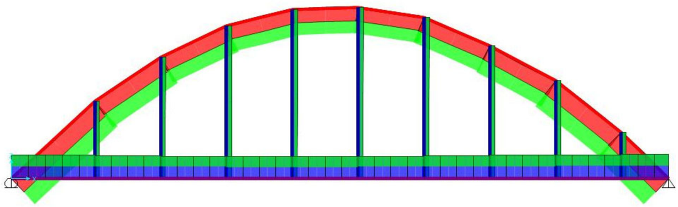

Fig. 8. *Ultimate axial forces of arch girders, ties, and hangers.*

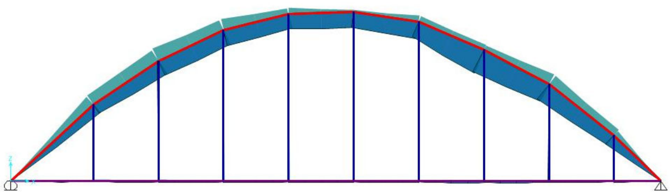

Fig. 9. *Ultimate moment of arch girder.*

4.2. Analytical deflection

The deflection of the arch bridge under moving load with impact (LL+I) was obtained from the idealized bridge as shown in Fig. 10. It is obvious that the deflected shape of the bridge is comparing very well with deflected shape of a typical tied arch bridge, since the maximum live-load deflection occurs in the vicinity of the quarter points [9]. The maximum live-load deflection was also found to be lower than the allowable live load deflection specified by AASHTO [10], which should not exceed 1/1000 of span. The maximum live-load deflection obtained from the model was found to be 4.24 mm which is much lower than 22.9 mm (span/1000).

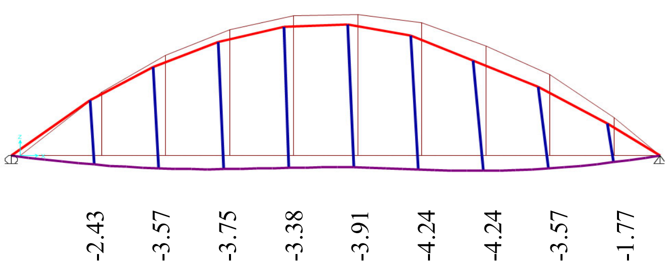

Fig. 10. *Live load deflection of arch girder.*

4.3. Load testing

The loading testing was performed under the load of locomotive of Henschel model DE 2550 of 132 tons. Strain measurements on the bridge were conducted using strain gauges at selected positions of spans 1, and 2. Deflections on the bridge were also measured at selected positions on span 5 under the same locomotive.

4.3.1. Measured stresses on members

The stresses where the strains were measured were obtained from the analytical model under the Henschel locomotive. The measured strains (ε) were then converted to stresses (σ) by the identified concrete elastic modulus (E), ($\sigma = E\,\varepsilon$). Tab. 1 and Tab. 2 showed a comparison between the measured and the calculated stresses. It is obvious that the measured stresses are generally smaller than the calculated stresses which signifying the adequacy of bridge elements to safely resist the applied loading.

Table 1. *Measured and predicted stresses for span 1.*

Point No.	Point position	Measured Stress (MPa)	Calculated Stress (MPa)
S1	Arch girder	-1.54	-1.02
S2	Tie beam	1.03	2.40
S3	Hanger	1.22	1.51

Table 2. *Measured and predicted stresses for span 2.*

Point No.	Point position	Measured Stress (MPa)	Calculated Stress (MPa)
S1	Arch girder	-0.38	-0.77
S2	Tie beam	0.10	1.50
S3	Hanger	0.85	0.80

4.3.2. Measured deflection

Deflections at measuring points of span 5 along with those obtained from the analytical model are illustrated in Tab. 3. Close agreement was reached between values at points 1 and 3, while differences have been noticed at points 2 and 3. In general, the measured deflections are lower than those obtained from the model.

Table 3. *Measured and predicted deflections for span 5.*

Point No.	Measured deflection (mm)	Calculated Deflection (mm)
1	1.00	1.03
2	0.50	1.30
3	1.00	1.30
4	0.10	1.03

5. CONCLUSIONS

The following concluding remarks can be drawn from this study:

1) The possibly neglected skew effect on the original simplified analysis of the bridge showed that conservative assumptions were done for the main resisting elements of the bridge (arch girders, hangers, and tie beams). However, for such a bridge torsional and bending moments of floor system may need great awareness as obtained by Macchi et al. [11].
2) An overall good condition of the bridge elements was recognized and the observed defects are not substantially danger. However, repair is to be done to provide safe life of the bridge.
3) The load carrying capacity of all main resisting elements of the bridge (arch girders, hangers, and tie beams) were found to exceed the design load with very large factor of safety. The load carrying capacity of floor system was found to be adequate. The measured stresses and deflections of the tested elements have found to be lower than the obtained stresses and deflections of the analytical model.

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