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STRENGTHENING AND MAINTENANCE MEASURES OF A BAILEY-TYPE ROAD BRIDGE

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ABSTRACT: In this paper, a survey based on experimental data accompanied by a detailed study on the assessment of the carrying capacity of an old Bailey-type steel truss road-bridge that is still in service is presented. This task is achieved through field measurements under static and dynamic loads as well as experimental results based on the properties of the steel material. An analytical model has been employed to assess the carrying capacity of the bridge under seismic loads and wind loads according to the provisions of current regulations. Based on the experimental and analytical results presented herein, strengthening and maintenance measures have been proposed in order to ensure strength and carrying capacity of the bridge under the current requirements. Also, an estimation of the remaining life of the bridge against fatigue is presented taking into account the proposed strengthening measures.

KEYWORDS: Bridge dynamics, maintenance, strengthening measures

1 INTRODUCTION

Designed and developed during the 2^{nd} W.War, the Bailey-type steel truss bridge played a leading role in as much the various geomorphology fields of battle as in the reconstruction of the Hellenic road network thereafter.

For the re-establishment of the road network in the region of Thermon-Aetoloakarnania in 1954 after the destruction with explosives of the arched stone Bania bridge in 1944, a new simply supported single-span steel truss bridge of Bailey-type has been constructed with the method of promotion with span length 54,87m.

During the 70's, the increase of traffic loads in the Western sector of National Road Network and the periodical redirection of traffic circulation on the particular road axis (Antirion – Bania bridge – Agrinion – Epirus) in order to cover transport and travel needs has led the local authorities to build an intermediate pier-support in order to strengthen the structure.

The challenge of the present evaluation resulted after the obvious surface deteriorations of the bridge due to corrosion of the steel elements of the bridge as well as the deck. The problems which resulted at the phase of in-situ inspection are lack of constructional and maintenance data regarding the intermediate pier-support.

Photo 1. The Bania bridge up to 1986

Photo 2. The Bania bridge in its present form

The measured dimensions of cross-sections of the truss members were certified by a similar study [1]. The particular structures have been designed to accommodate passage of military vehicles for temporary use under controlled conditions. The extensive and non-verified use of such bridges due to the passage of heavy vehicles increases the danger of failure due to fatigue.

2 BRIDGE DESCRIPTION

The steel truss bridge of Bailey-type was named after the team head British engineer Sir Donald Bailey. It constitutes from elements assembled with bolts and can cover either single or multiple spans.

Each frame has length of 3,048m with cross-section as shown in Fig. 1 and it has four holes at its edges where connection with neighboring frames is done with bolts. The Bania bridge in Thermon - Aetoloakarnania is characterized as a triple-double truss-frame since it is constitutes from two complex truss beams with three lines of frames at width and two lines at height each (Fig. 2). The total span length is 54,87m covered by 108 such frames.

In order to support the deck and to distribute the traffic loads to the main truss beams, cross girders made from IPN profile have been used as shown in Fig. 3.

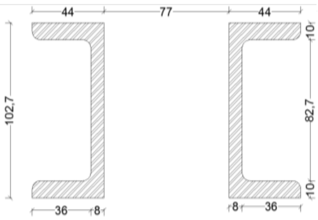

Figure 1. Top and bottom cross section in mm

Figure 2. Cross section of the main beam in mm

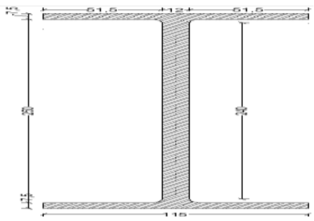

Figure 3. Cross girder cross section in mm

The transfer of loads from the deck to the main beams is through the cross girders. The deck is wooden with galvanized sheets of thickness 5mm. The weight per frame is 5.880kgr, which results to a total weight of the bridge 18 x 5.880 kgr = 105840 kgr or 1058,4 kN, and total mass of 107,8 kNsec^{2}m^{-1} considered in the dynamic analysis. The inertia properties of the main truss beams resulting from EC-3 is I_{eff} = 0,08895 m^4 according to eq(1).

$$I_{eff} = 0.5 \cdot A_f \cdot h_0^2 \tag{1}$$

where A_f is the area of cross-section of each flange and it is 0,31 m^2, while when calculated theoretically and with the use of software it is I_{eff} = 0,0833m4. In the analysis the most unfavorable value I_{eff} =0,0833m^4 has been used.

3 STRUCTURAL STEEL PROPERTIES

Specimens have not been taken for this particular study due to practical and economical reasons since valuable results on the material with the same year of production and similar use before placement in local Pakistan are reported in a similar work [1]. The steel quality BS 968 is met in various forms at this time period of production in combination with the development of metal industry and it is used in the frames and the cross girders while for the rest elements it is used steel BS 15 [2].

Table 1. Properties for structural steel BS 968

Property	Material	Units	Value
E: elasticity modulus	Steel BS 968	GPa	206,8
f_y: yield stress	Steel BS 968	MPa	344
f_u: failure stress	Steel BS 968	MPa	540

The steel used in this bridge has presented deterioration and damage in various members as well as extensive corrosion.

Photo 3. Corrosion of steel

Photo 4. Keystone damage

4 IN-SITU MEASUREMENTS AND QUALITY CONTROL

The maximum deflection of a simply supported beam under uniform load is given by:

$$w_{max} = \frac{5}{384} \cdot \frac{p \cdot \ell^4}{EI} \tag{2}$$

where ℓ is the length of the beam =54,864 m

p is the uniform load =1058,4kN/54,864m=19,29 kN/m

EI: is the bending stiffness=206,8 GPa*0,0833m^4.

and hence

$$w_{max} = \frac{5}{384} \cdot \frac{19{,}29 \cdot 54{,}864^4}{206{,}8 \cdot 0{,}0833} = 0{,}132\,m = 132\,mm \tag{3}$$

On site measurements of the maximum deflection due to bending with the use of geodetic instruments resulted 34mm. This means that, after erection of the intermediate support, the bridge has been elevated and then was left to rest on the bearing, so that the initial deflection was decreased by 98mm. Taking into account the effect of initial slippage e_o.

$$e_o = \frac{0{,}5 \cdot L \cdot \delta_o}{h \cdot \cos\alpha} \tag{4}$$

it has been calculated that the maximum bending deflection was finally decreased by $w_M + e_o$=123 mm.

Photo 5. Intermediate support detail

Considering the simply supported beam as a single degree-of-freedom oscillator the first (fundamental) eigenfrequency results from eq (5)

$$f_1 = \frac{1}{2\pi}\left(\frac{\pi}{L}\right)^2\sqrt{\frac{EI}{m}} \quad (5)$$

where m=(1058,4kN / 9,81 m/sec^2) / 54,864m = 1,967 kNm^{-2}sec^2. The computed value is f_1 = 1,545 sec^{-1}.

It was not possible to setup instrumented dynamic measurements on site. For this reason and in order to validate the reliability of the model in which beam elements have been used, instrumented measurements have been performed on a similar simply supported bridge with length 18,29 m regarding free oscillation as well as the Frequency Response Function. From these measurements, a time history response has been obtained which was then applied on the FE model via the Sap2000 FE software. The correlation of the results of the real structure and the FE model was excellent.

5 NUMERICAL RESULTS

In order to validate the analytical results, a numerical model has been developed in which the same loading was applied so that the improved behavior of the bridge after placement of the intermediate support can be evaluated. Besides the classical static and dynamic analyses, resulting the basic load combinations ULS and SLS, analyses with one moving load of class group 12 passing through both models as well as three moving loads of class group 12 according to the current code provisions have been performed [3,4].

Photo 6. The triple-double Bania bridge

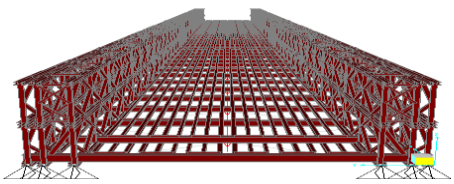

Photo 7. Finite element model of the bridge

Numerical analyses have also been performed in two bridge models, one simply supported and a second with two spans. Imperfections or damages in the analysis of the single span bridge were not taken into account since the main goal was not to evaluate the structural behavior of the bridge before 1986 for which no information was available. All structural damages were assumed to occur after the placement of the intermediate support. Extensive corrosion of the structure is obvious in all members and its extent is not possible to be assessed by visual inspection. In the evaluation of the results, the effect of the corrosion was based on reliable experimental measurements [5].

From the analyses of the two models, the fundamental eigenfrequency and eigenvector are determined, i.e., $T_1 = 0{,}664$ sec ($f_1 = 1{,}51$ sec^{-1}) for the case of single span simply supported bridge with theoretical value 0,647 sec (2,5% error) while for the two span bridge with intemediate support it is T_1=0,248 sec.

The fundamental eigenshapes resulted from the analysis of the two models are shown in Figs 4 and 5, respectively.

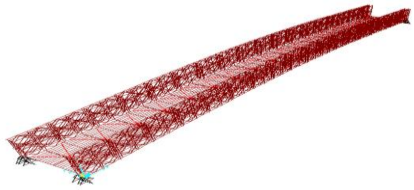

Figure 4. Fundamental eigenshape of 1-span

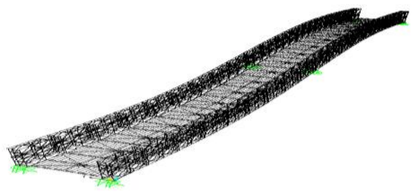

Figure 5. Fundamental eigenshape of 2-span

The effectiveness of intermediate support placement becomes more clear by observing the following diagrams.

Figure 6. Dynamic response for 1 vehicle pass

Figure 7. Dynamic response for 3 vehicles pass

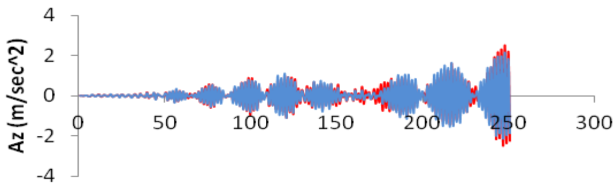

Figure 8. Acceleration diagram for 3 vehicles pass

From the load combinations including the effect of wind loading, it resulted no high stress values capable to cause failure or serviceability issues.

For the control of strength against fatigue in this particular type of structure, the main points of interest [6] are the connection points of the vertical stands to the bottom flange of the frames as well as the holes to which the horizontal stiffening members are connected. The evaluation of fatigue strength for this particular bridge is imperative since the policy of equitable use refers to short time operation and under continuous supervision [7].

The fatigue detail for the vertical stands and for the holes to which the horizontal stiffening members are connected is determined to be 100.

The number of vehicles that are passing annually through the particular bridge classifies the structure into traffic category 4 that is 0,05*106, while the life duration from the erection time is 60 years. The check against fatigue is done according to the criteria given by eqs (6) and (7) and it resulted that for equitable use as well as overloading conditions, the bridge does not suffer any problems due to fatigue

$$\gamma_{Ff} \cdot \Delta\sigma_{E_2} \leq \frac{\Delta\sigma_c}{\gamma_{Mf}} = \frac{90}{1{,}15} = 78{,}26 \quad \text{where } \gamma_{Ff}=1{,}0 \text{ and } \gamma_{Mf}=1{,}15 \tag{6}$$

$$\gamma_{Ff} \cdot \Delta\tau_{E_2} \leq \frac{\Delta\tau_c}{\gamma_{Mf}} = \frac{75}{1{,}15} = 65{,}22 \quad \text{where } \gamma_{Ff}=1{,}0 \text{ and } \gamma_{Mf}=1{,}15 \tag{7}$$

$$\Delta\sigma_{E_2} = \lambda \cdot \varphi_1 \cdot \Delta\sigma_p \quad \text{where } \varphi_1=1 \text{ (dynamic factor for road bridges)} \tag{8}$$

$$\Delta\tau_{E_2} = \lambda \cdot \varphi_2 \cdot \Delta\tau_p \quad \text{where } \varphi_2=1 \text{ (dynamic factor for road bridges)} \tag{9}$$

6 MODELING CONSIDERATIONS

However, the ideal conditions described in the finite element model and the current regulation are not possible to take into account all the imperfections and structural damages that may influence the operation of the bridge structure. This requires also experimental verification for numerous cases checked against normal use. Evaluating the full structure including the foundation and abutments as well as the superstructure, damages were detected at the bearing positions of the bridge since it rests on stone abutments where a concrete plate has been placed to facilitate the support of the bridge. In both abutments, damage has been detected in the keystone of the arches most likely due to heavy vehicles, which when entering and leaving the bridge cause impact phenomena. During the inspection, it was detected that a small number of frames has suffered damages due to the collision of vehicles on the vertical stands as well as buckling.

A full raw of bolts is missing at the connection of to the first line of frames. The cross-girders fasteners are relaxed because of cyclic loading. Oxidation of the bridge structure is extensive and deep in all structural elements. During the effort of surface scratching with a metal brush it was not possible to detect clean steel. Because the big height of the frame and the connection detail between secondary beams and the deck, the vertical stiffeners have not been inspected in detail but only visually with the aid of photographs. Also, the intermediate support bearings do not fully conform to the instructions of the manufacturer handbook [3] since the item required to achieve pinned support conditions is missing. All the above findings render enough difficulty to create a reliable model.

7 CONCLUSIONS – STRENGTHENING MEASURES

Summarizing the above, one concludes that the intermediate support affects positively the development of dynamic phenomena as well as the fatigue strength of the members. An increase of the bearing capacity of the structure is also achieved according to the manufacturer handbook by transforming the single span bridge to a two-span continuous bridge after placement of the intermediate support. The vertical displacements and accelerations at L/4 are decreased in all load cases. The danger of fatigue failure has been completely eliminated based on the numerical results. However, the existing cracks at welded connections should be under constant monitoring. Given that the structure cannot be strengthened with the addition of new frames, it is alternatively proposed to maintain the existing frame. At first, the keystones of the abutment arches must be restored with repair mortars to maintain their form as well as their historical character. Replacement or full repair of damaged existing frames or structural members must follow. The whole deck must be temporarily removed in order to replace damaged horizontal stiffeners. The steel member surfaces must be cleaned with sandblast and anti-corrosion paint. All damaged connections must be restored. A final step is to build bumpers just before the entry at both sides of the bridge in order to decrease vehicle speeding that influences the dynamic behavior of institution. Other solutions such as the placement of tuned mass dampers or a steel-concrete composite deck were excluded as non-favorable alternatives.

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