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Case Study of Structural Health Monitoring of an Age Old Stone Masonry Arch Bridge

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ABSTRACT: Health monitoring of existing structures are significantly important, particularly to ensure efficient delivering of its functional utility with adequate safety. The early detection of damages and subsequent adoption of effective mode of repair depends on this condition assessment of the structural health. The health monitoring of age old structure is further challenging task in absence of information of its constructional aspect. Visual inspection and Non-destructive tests will be helpful to assess the condition of the structure. But the global structural performance is more important aspect for its acceptance criterion. The load deflection behaviour for loading and unloading is able to assess the structural behaviour in a global manner. The creep behaviour of the structure will further clarify from its material consideration. A case study of health monitoring of an age old stone masonry arch bridge on the national highway of India is presented in this proposed paper.

1 INTRODUCTION

The importance of condition monitoring of structure in assessing its integrity in the context of safety is immense. Particularly in case of bridge structure, which is a significantly important part of the country's infrastructures, should be periodically monitored not only for the safety aspect but also from timely adaptation of an economical and effective restoration scheme. There are several methods of condition assessment of a bridge structure and few are also stipulated in different codes. Some of the methods are suitable where all the design details are available for analytical verification. Similarly, availability of sufficient traffic data and corresponding loading arrangement are required for full scale load test. However, there are hardly any design & construction details available for a very old bridge abandoned for a long time. The cost benefit analysis also does not recommend conducting a full scale load test with detail loading arrangement and such situation offers a challenge in condition assessment technique. The improvised load test as discussed later is based on the Indian road congress specified method and considered the structural behavior as a whole, which may provide a solution to this challenging situation. This particular bridge was constructed more than 100 years back as per available information. The objective of the work is to assess the present condition of the structure. The future use of the said structure and the mode of strengthening, if required, will be based on the present condition of the structure.

1.1 Description of the structure

Lilajan Bridge is located near Dhobi, Bihar, India on the National Highway. The exact location is at 43 kilometres from Aurangabad towards Barwa-Adda, Bihar, India. This arch type bridge was made of stone masonry and situated in the old alignment of river Lilajan. The bridge was kept abandoned for a long time due the change of river course and subsequent construction of new bridge, but seems to be in good condition. The condition assessment of the bridge was required in connection with widening and strengthening of existing two lanes of National Highway utilising this old abandoned arch bridge if it seems feasible. It is a two lane stone-masonry arch bridge having three spans. The length of each span of the arch is approximately 16.2 m, having a pitch of one fifth. The thickness of the arch varies between 720 mm to 900 mm gradually reducing from support towards the crown. The details of field measurements are given in Table 1. The width of the carriage-way is about 9.10 m. The deck of the bridge is made of flexible pavement on compacted earth. The central span of the arch bridge was selected for testing for various loading condition. The improvised load test was conducted on a cloudy day of relatively low temperature variations. The temperature at the top and bottom of the deck was noted. The average temperature recorded at the top of the deck was 330 ºC while 29.50 ºC at the bottom. Therefore, the error due to temperature variation is small.

Table 1 : Detail measurements of the abandoned arch bridge.

Sec	I	1/I	X	h	h/I	y	y^2/I	x^2/I
	(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
1	0.0311	32.15	0	3.314	106.55	0.6366	13.031	0
2	0.03377	29.61	-0.81	3.283	97.22	0.6056	10.86	.427
3	0.03955	25.287	-1.62	3.215	81.30	0.5376	7.308	66.36
4	0.04267	23.44	-2.43	3.083	72.26	0.4056	3.8561	138.41
5	0.04428	22.58	-3.24	2.918	65.89	0.2406	1.3071	237.03
6	0.04595	21.76	-4.05	2.648	57.63	-0.0294	0.0188	356.92
7	0.04939	20.24	-4.86	2.358	47.74	-0.3194	2.0655	478.06
8	0.05679	17.609	-5.67	1.963	34.57	-0.7144	8.987	566.11
9	0.05875	17.022	-6.48	1.533	26.095	-1.1444	22.29	714.76
10	0.06075	16.461	-7.29	1.007	16.576	-1.6704	45.93	874.80
For half span:		226.274	-	-	605.827	-	115.65	3451.88
For whole span:		452.55	-	-	1211.65	-	231.30	6903.76

2 ANALYSIS OF THE STRUCTURE

The schematic diagram for the elastic analysis of the arch based on the field measurement is shown in Fig. 1.

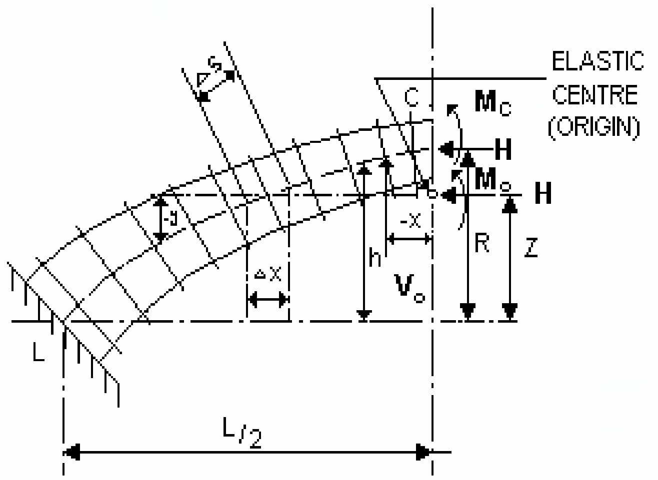

Figure 1 : Elastic analysis of the arch.

Where, H = Horizontal thrust at springing level
V_0 = Vertical reaction at fixed end
M_0 = Fixed end moment
R = Rise of the Arch

L = Span of the Arch
C = Crown of the Arch
I = Moment of Inertia

Now, with C as origin, the equation of the parabola is $y = Cx^2$.
Putting boundary condition at $x = L/2$, $y = -R$, we get $C = - 4R/L^2$
So, the equation of the arch is

$$y = (R-Z) - 4R.\ x^2/L^2 \quad (1)$$

Now, the Elastic center $Z \Rightarrow \int y\,dx = \sum (h/I) / \sum (1/I)$
$= \sum$ column 5 / $\sum$ column (Table-1)
$= 1211.65 / 452.55$
$= 2.677$ Meters.

Now, $y = h - Z$, $R = 3.314$ Meter, $y_{max} = R - Z = 0.637$ Meter.

The influenced line of bending moment at arch crown is drawn, calculating the bending moments for unit load at different section as shown below in Table 2 to Table 6.

Table 2 : Bending moments for Unit Load at Section 1.

Section	M	M/I	Mx/I	My/I
1	0	-	-	-
2	-0.81	-26.04	21.09	-15.77
3	-1.62	-40.96	66.36	-22.02
4	-2.43	-56.96	138.41	-23.10
5	-3.24	-73.16	237.04	-17.60
6	-4.05	-88.13	356.93	+2.59
7	-4.86	-98.37	478.08	+31.42
8	-5.67	-99.84	566.09	+71.33
9	-6.48	-110.30	714.74	+126.23
10	-7.29	-120.00	874.80	+200.45
For whole span Σ =		-713.76	3433.54	353.53

Table 3 : Bending moments for Unit Load at Section 3.

Section	M	M/I	Mx/I	My/I
1	0	-	-	-
2	0	-	-	-
3	0	-	-	-
4	-0.81	-18.99	138.41	-23.10
5	-1.62	-36.58	237.04	-17.60
6	-2.43	-52.88	356.93	+2.59
7	-3.24	-65.58	478.08	+31.42
8	-4.05	-71.32	566.09	+71.33
9	-4.86	-82.73	714.74	+126.23
10	-5.67	-93.33	874.80	+200.45
For whole span Σ =		-421.41	2138.39	332.313

Table 4: Bending moments for Unit Load at Section 5.

Section	M	M/I	Mx/I	My/I
6	-0.81	-17.626	71.385	0.5182
7	-1.62	-32.789	159.355	10.473
8	-2.43	-42.790	242.619	30.569
9	-3.24	-55.151	357.378	79.638
10	-4.05	-66.667	486.002	111.361
For whole span Σ =		-221.023	1316.74	232.559

Table 5 : Bending moments for Unit Load at Section 7.

Section	M	M/I	Mx/I	My/I
8	-0.81	-14.263	80.871	10.189
9	-1.62	-27.576	178.692	39.82
10	-2.43	-40.00	291.60	66.816
For whole span Σ =		-81.839	551.163	116.825

Table 6 : Bending moments for Unit Load at Section 9.

Section	M	M/I	Mx/I	My/I
10	-0.81	-13.33	97.20	22.27
For whole span Σ =		-13.33	97.20	22.27

The reactions of the fixed arch are calculated numerically as follows, where L indicates Left Support, R indicates Right Support and C indicates crown of the arch.

$$M_0 = -\frac{\text{Area of M/I diagram for 'Left' and 'Right' cantilevers LC and CR}}{\sum 1/I \times \Delta s}$$

$$= -\frac{\sum (M/I)\,\Delta s}{\sum (1/I)\,\Delta s}$$

$$V_0 = +\frac{\bar{x}\text{ times area of 'Left' and 'Right' M/I diagrams}}{\sum 1/I \times \Delta s}$$

$$= +\frac{\sum (Mx/I)\,\Delta s}{\sum (x^2/I)\,\Delta s}$$

$$H = +\frac{\bar{y}\text{ times area of 'Left' and 'Right' M/I diagrams}}{\sum (y^2/I)\,\Delta s}$$

$$= +\frac{\sum (My/I)\,\Delta s}{\sum (y^2/I)\,\Delta s}$$

Where, $\bar{x}$ = Centroidal abscissa and y = Centroidal ordinate of M/EI diagram. The Moment at crown of the Arch

$$M_c = M_0 - (R-Z)\,H$$

When the unit load is on the left half

$M_L = M_0 + ½V_0 + Z.H - a$ Where, a = horizontal distance of the unit load from L.

Similarly when the unit load at the right half

$$M_L = M_0 - ½V_0 + Z.H$$

The values of M_0 , V_0 & H for the unit load at different sections are calculated and shown in Table 7 below.

Table 7 : Values of M_0 ,V_0 & H for unit load at different sections.

Unit load at	M/I	1/I	M_0 (ton)	Mx/I	x^2/I	V_0 (ton)	My/I	y^2/I	H (ton)
1	-713.76	452.55	1.5772	3453.54	6903.76	0.50	353.53	231.30	1.5284
3	-421.41	452.55	0.9312	2138.39	6903.76	0.3097	332.313	231.30	1.4367
5	-215.02	452.55	0.4751	1316.74	6903.76	0.1907	232.559	231.30	1.005
7	-81.839	452.55	0.1808	551.163	6903.76	0.0798	116.825	231.30	0.5051
9	-13.330	452.55	0.0295	97.20	6903.76	0.0141	22.27	231.30	0.0963

The influenced line of bending moment for the fixed arch is developed and the improvised loading positions for the desired crown moment are determined accordingly

3 TEST PROCEDURE

3.1 Loading Arrangement

Two common commercial trucks having total weight 20.25 ton each were available and has been used for the test. The front axle load of each truck was weighted as 4.75 ton whereas the rear axle was weighted as 15.5 ton. The actual variation of loading, as per SP-37, IRC, for different percentages of bending moment at crown, was not available at this remote site. This problem is taken care of by adjusting the position of loading, which is determined from influence line diagram of the particular arch. The fixed arch has been analyzed numerically based on elastic approach. Both the single truck loading and double truck loading were arranged for the test. The load test has been performed for the reciprocal arrangement of the loading to assess whether the structural symmetry still remains at its present state.

The sequence of loading was as follows:

- Load case I: One truck at mid span in eccentric position towards upstream facing Dhanbad.
- Load case II: One truck at mid span in eccentric position towards downstream facing Banaras.
- Load case III: Two parallel trucks at mid span were placed eccentrically towards downstream facing Banaras.

A three-dimensional model based on finite element method of the arch bridge has been analysed using STAAD software package. The arch is made of stone masonry. The material property of the stone masonry is evaluated from sample of the stone collected at site and the crushing strength obtained at laboratory was 414.4 MPa. The compressive strength of the composite stone masonry is evaluated considering the size of the stone blocks, the skilled arrangement of the stone blocks at site and the thickness of the mortar used. The correlated value of the modulus of elasticity (E) of the stone masonry which is a function of the masonry compressive strength is assumed to be 1.5×10^4 MPa. The theoretical deflection is shown in Table 8.

Table 8 : Theoretical Crown deflection of the fixed arch.

Location of Dial gauge	Theoretical Crown Deflection (cm)		
	Load Case-1	Load Case-2	Load Case-3
Upstream edge	0.022	-0.00185	0.042
Centre Line	0.006	0.006	0.035
Downstream edge	-0.00185	0.022	0.0017

3.2 Deflection Measurement

Vertical deflections of the arch crown at three locations, two at external edges and one at the center have been measured for different loading conditions. The horizontal deflections are also measured at three same locations at each support to monitor the opening of the span under the loading condition. The deflections at all these locations are measured using dial gauges fixed at the selected spots. To distinguish easily the occurrence of cracks, if any, under the loading conditions, quarter spans from each side of the center-line was painted white, prior to the test. The occurrence of the flexural cracks, and the crack width, if any, was monitored using “Crack detection microscope”. For the arranged loading neither new cracks have been formed nor the extension of cracks have been noticed.

4 RESULT AND DISCUSSION

The test loading was applied in stages of producing approximately 0.5 M_c , 0.75 M_c and 1.0 M_c instead of 0.5 w, 0.75 w, 0.9 w & and 1.0 w as specified in the IRC code as the variation of loading arrangement was not available. The deflections were noted approximately after 5 minutes of loading or the steady reading whichever is later. The measured deflection at the crown for the arranged maximum loading is much lower (about 0.36 mm) than that of the maximum permissible value of 1.25 mm. Similarly, the measured horizontal deflection i.e. spread of piers are also very much less than 0.4 mm as specified in the IRC code. This maximum permissible deflection refers to the next heavier vehicle load presently plying over the bridge, as specified in the IRC code. Whereas, the deflections at the crown and at the springing level of the arch have measured for the single and double trucks loading, each of weight 20.25 ton as available at site. To assess the condition of the bridge structure, the permissible recovery of the crown deflection of the arch after unloading should be at least 80 % for acceptance. The average recovery at the downstream edge is about 83.3 % as shown in Table 10, which is more than the permissible limit of acceptance. However, the average recovery of the crown deflection at the upstream edge is about 57 % as shown in Table 9, which is much lower than the specified limit of 80 %. The details of the recovery of deflections are tabulated below.

Table 9 : Recovery of deflections for load case-I (Least Count = 0.01 mm).

Loading Edge	Dial Gauge Location	Deflection Reading	Recovery of Deflection	Percentage Recovery	Average Recovery
Upstream	Loading Edge	27	15	55	57 %
	Central Line	17	10	59	
	Opposite Edge	0	3	-	

Table 10 : Recovery of deflections for load case II & III (Least Count = 0.01 mm).

Loading Edge		Dial Gauge Location	Deflection Reading	Recovery of Deflection	Percentage Recovery	Average Recovery
Downstream	II	Loading Edge	24.5	24.5	100	83.3 %
		Central Line	15	10	67	
		Opposite Edge	0.5	0.5	100	
	III	Loading Edge	33	31	94	
		Central Line	36	28	78	
		Opposite Edge	10	06	60	

The comparison of the measured deflection with the theoretical deflection calculated for the idealized finite element model is given below both for the single truck and for the double trucks improvised loading arrangement.

Table 11: Comparison of theoretical and measured deflections.

Location of Dial Gauges	Crown Deflection in cm.					
	Load Case-1		Load Case-2		Load Case-3	
	Theoretical	Measured	Theoretical	Measured	Theoretical	Measured
Upstream edge	0.022	0.027	-0.00185	0.0005	0.0017	0.010
Centre Line	0.006	0.017	0.006	0.015	0.035	0.034
Downstream edge	-0.00185	-0.001	0.022	0.0245	0.042	0.033

The static deflection is the reflection of the structural behavior as a whole. The comparison of the theoretical and measured deflection is not closely matching in most of the cases due to errors involved in measurements as well as in idealizing the model. However the comparisons indicate that, the measured deflection pattern at the downstream edge is in the tune of theoretical one but the integrity of the structure is in doubt at upstream edge where the measured deflection was much more than the theoretical deflection.

5 CONCLUSIONS

The acceptance criterion for the arch bridge is governed by the following stipulations as specified in IRC, Special Publication SP 37. Where no crack is observed, the load for rating shall be the minimum of

(a) The load on rear axle causing a deflection of 1.25 mm in case of test vehicles having single rear axle.
(b) The load causing a spread of pier by 0.4 mm.
(c) The load recovery of crown deflection or spread of pier to a value of 80 %.

Keeping view to the above stipulations, the measured data and subsequent analysis, it may be inferred that the bridge is quite safe against the improvised loading. However, the weakness of the bridge, particularly in the upstream edge is significantly noticed from the overall structural behavior under the subject loading.

The measured deflections are compared with that of theoretical deflections computed using STAAD software package. The measured deflection are better matched in case of double truck loading, however the measured deflection is more than the theoretical value in case of single truck eccentric loading, which puts doubt on the structural integrity. However, these theoretical deflections are based on the idealized finite element model, which may vary from the actual one. Again the linear behavior of the bridge structure under the arranged improvised loading can not be the same for the heavier load.

The symmetry of the structure is also suffered to an extent as observed from the deflection data as well as from the recovery of deflections in the case of reciprocal loading. The negative deflection was observed at the downstream edge opposite to the loading edge in load case-I, which is in the tune of the theoretical value. However in case of load case-II, the observed deflection at the upstream edge for the reciprocal arrangement of opposite edge loading was not complying with the computed deflection. This abnormal behavior also indicates doubt regarding the overall structural integrity, particularly towards the upstream edge.

From this improvised load testing, it may be inferred that the existing abandoned stone masonry arch bridge can be well utilized and the life could be enhanced with proper strengthening measure, adopting effective rehabilitation scheme for its restoration.

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