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Deflection Measurement of Masonry Arch Bridges with Tall Piers: Case Study of Shahbazan Bridge

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ABSTRACT

A common practice for detailed assessment of masonry bridges is to use recorded deflection signature of mid-span of such structures due to predefined loading schemes. However, measuring the deflection of bridges with tall piers or those situated over deep valleys introduces certain difficulties, since common deflection-meters require a reference point relative to which the measurement is carried out. Common approaches currently proposed by researchers include indirect measurement of deflection based on acceleration or tilt measurements. However, due to small values of deflection in masonry arch bridges, such approaches may not produce the expected results. Moreover, optical methods are difficult and rather expensive to implement due to small deflection values and far reaching points. This paper proposes the application of piers as the reference point of common deflection meters. To demonstrate the applicability of the proposed approach, a case study is carried out in which a 70 years old railway masonry arch bridge is instrumented for structural health monitoring. The studied bridge is a seven span railway masonry arch bridge with tall piers of almost 40 meters and span lengths of 21.5m. It is concluded that vertical deflection signature of conventional method could be derived from inflating that of proposed method by a factor of 1.86. Therefore, the method proves to be practical for the purpose of structural health monitoring and calibration of numerical models.

1. Introduction

Aiming at increasing the throughput of the network, railway administrators seek out new solutions such as increasing the axle load or operational speed of trains to allow more trains in the network. One major obstacle in doing so is the limited capacity of existing structures in the network such as bridges. In this regard, evaluating the performance of such structures subjected to different loading schemes and operational speeds seem to be the prerequisite of increasing the axle load.

Iranian railway organization has started a project of increasing the axle load of its railway network from the current 20 tons to 25 tons. One major problem in doing so is the existence of old masonry bridges in the network such as 'Shahbazan' bridge, which has been in service for more than 70 years. The problem with evaluating the performance of

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bridge is the complexity of Masonry Bridge behavior, which has been of great debate during recent years [1-8].

A common approach in assessing such structures is employing calibrated finite element models of bridges. To do so, dynamic load tests are carried out on the bridge employing operational loading regime of the structure. Response of the bridge in terms of vertical deflection of different points of the bridge are recorded and used for calibration of finite element model [1, 2, 5]. However, the main problem for recording the vertical deflection of Shahbazan bridge is its' relatively tall piers, which are up to 40 meters. Tall piers make it rather difficult to employ common deflection meters or tilt-meters. This paper proposes a novel method which enables the application of common deflection-meters for bridges with tall piers. Moreover, deflection signature is used to calibrate the finite element model of the bridge which is developed in Abaqus engineering software.

2. Bridge Characteristics

Shahbazan is a masonry arch bridge built more than 70 years ago in north western region of Iranian railway network (Figure 1). The bridge consists of five long spans of 21.5 meters, one span of 20.68, and two spans of 10 meters, totaling a length of 193 meters. The superstructure consists of U33 rails, wooden sleepers, and rigid fasteners. Maximum allowable speed on the bridge is 60 km/h. In order to have the characteristics of the material used in building the bridge, a series of tests have been conducted. Cores from various segments of the bridge are taken to a lab and tested to determine the compressive strength of the material. Material characteristics of the bridge are presented in Table 1.

Figure 1: A view of the Shahbazan bridge

Table 1: Material characteristics of Shahbazan bridge

Section	Density	Modulus elasticity (GPa)	Poison ratio
Arch #1	2500	24	0.167
Arch #2	2500	29	0.167
Arch #3	2500	30	0.167
Arch #4	2500	30	0.167
Arch #5	2500	24	0.167
Arch #6	2500	24	0.167
Arch #7	2500	24	0.167
Arch #8	2500	24	0.167
Fill concrete	2500	10	0.167
Piers	2500	30	0.167

3. Test Instrumentation

The aim of field tests is to determine the response of Shahbazan bridge to the passage of the test train. For this purpose, vertical deflection of the third span is monitored. Deflection of arch is supposed to be recorded with a frequency and accuracy of at least 20 Hz and 100 µm, respectively. In order to record the deflection of any spot on the arches of bridge with such standards, a reference point is needed on which the deflection meter is fixed and any displacement relative to the fix point is recorded. For this purpose, a type of deflection recording sensor called 'Deflected Cantilever Displacement Transducer', or simply put 'DCDT', is used. DCDTs comes with a cable that is fixed to a reference point. Normally, the cable is fixed to a heavy weight laid beneath the spans. However, due to tall piers in Shahbazan bridge, this approach may be rather difficult to implement. Moreover, due to strong winds in the region, considerable error may be introduced in the results. Therefore, a new approach is proposed and results are compared with conventional method.

In the new approach, the cable is fixed to piers in a triangle shape, rather than to a heavy weight beneath the bridge span. The method is schematically shown in Figure 2. In order to compare the results of the proposed method with those of the conventional method, a cable is also fixed to a heavy weight (a steel sleeper is used for this purpose) laid beneath

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the bridge span. A figure of DCDT and cable fixed to a heavy weight is presented in Figure 3. DCDT sensor is capable of recording the displacement in a range of 25 mm with an accuracy of 10 µm.

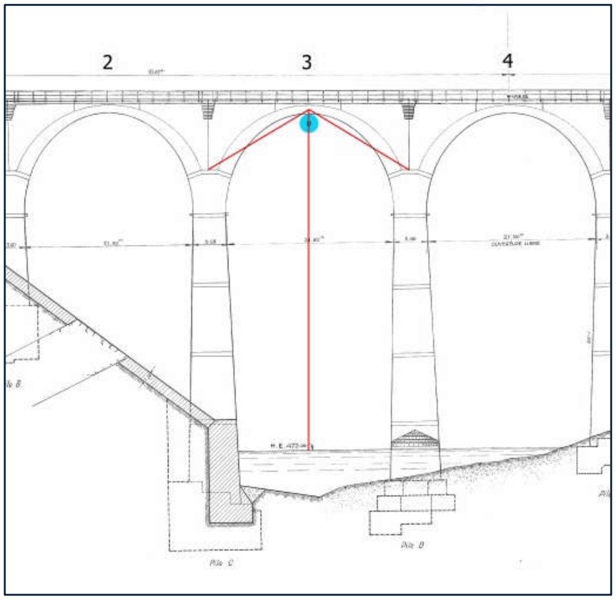

Figure 2: Triangle and conventional method of fixing DCDT's cable to a relative point

Figure 3: a) DCDT sensor mounted in the middle of the span, b) DCDT's cable fixed to the steel sleeper placed beneath the span (conventional method)

To determine the exact speed and location of test train on the bridge, a series of LVDT sensors and strain gauges are mounted on the rail, as shown in Figures 4 and 5. Data is recorded with a frequency of 2 KHz throughout the tests.

Figure 4: LVDT mounted on sleeper to determine the exact speed and location of test train axles

Figure 5: Strain gauge mounted on rail heal to determine the exact speed and location of test train axles

Three 6-axle locomotives and five 4-axle freight wagons are used to form the test train as shown in Figure 6. Three different train formations are considered throughout the tests: full train consists of 3 locomotives and 5 freight wagons (3L5W), three locomotives (3L), and a single locomotive (1L). Axle spacing and loads are presented in Figure 6, schematically. Dynamic tests are repeated for each train formation with speeds ranging from 10 to 65 km/h, totaling 37 dynamic tests.

4. Field Test Results

A calculated assessment presumes that together with the geometry, foundations and loads, all essential material properties and their status are known or can be estimated and that the load transfer

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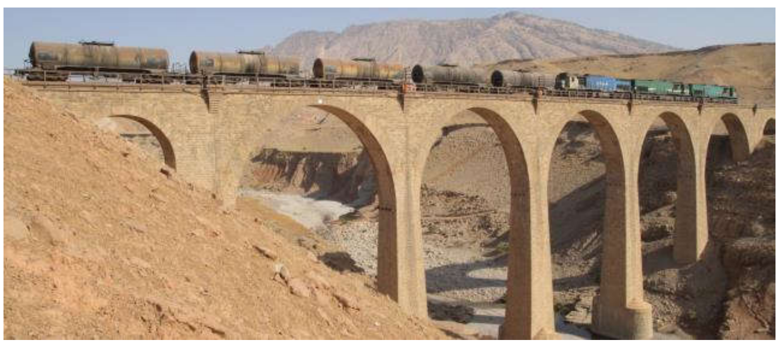
can be described realistically in mathematical terms [9]. In reality, however, it is fairly difficult to determine the exact material properties of the whole material in building masonry bridges. There are sometimes ambiguities in the structure of such bridges as well. In such cases, field tests are one useful way of determining the overall behavior of the bridge, due to applying predefined loading schemes.

Results suggest that peak vertical deflection of conventional cable setup is on average 86% higher than that of triangle cable setup. In other words, vertical deflection signature of conventional method could be derived by inflating that of proposed method by a factor of 1.86.

In conventional method, vertical deflection is measured relative to ground, which could be taken as a fixed point with almost zero vertical deflection due to train loading. However, CP1 and CP2 (connection points of cable to adjacent piers) will deflect due to train loading and therefore, vertical deflection of triangle method is less than that of conventional method. Figure 7 presents vertical deflection of the middle of the third span as derived by both cable setups. As could be seen, pattern of two signatures are almost identical. Peak vertical deflection of middle of the third span due to all train runs are determined and presented in Figure 8.



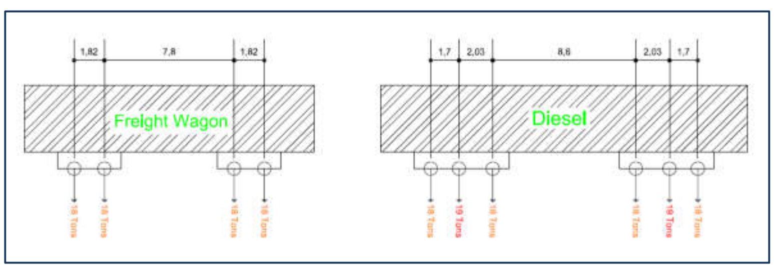


Figure 6: Schematic plan of test train axles

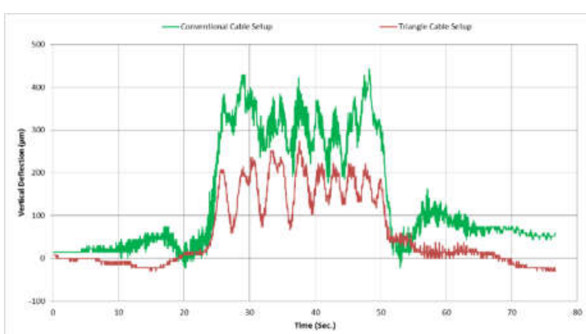


Figure 7: Vertical deflection signature of middle of the third span by two cable setups (3L5W with a speed of 16 km/h)

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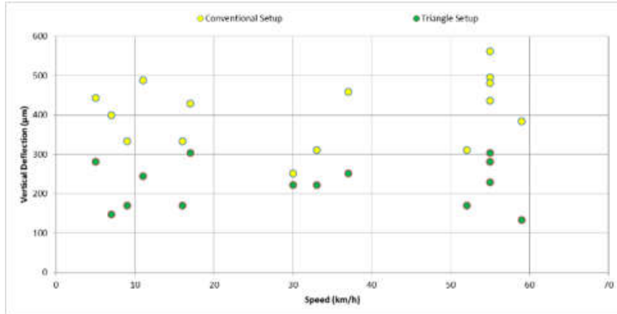

Figure 8: Peak vertical deflection values of middle of the third span for both cable setups

If CP1 and CP2 are low enough, their deflection due to train loading will be insignificant and the two methods would result in almost identical results. Doing so, however, introduces errors similar to that of conventional method (error due to wind, and difficulty of application). Yet the fact that triangle cable setup results are less than that of conventional method would not be an issue, since 3D finite elements allow for determination of vertical deflection of any point on the structure. Therefore, relative deflection of the mid-span to that of CP1 and CP2 could be determined and used to calibrate using the results of the dynamic load tests.

5. Numerical Model of Shahbazan Bridge

To study the safety of the bridge, a 3D finite element model of the bridge is developed in Abaqus engineering software, as shown in Figure 9. All geometrical and structural characteristics of the bridge are considered, which include joints and filling material. Connection between piers and foundations are modeled as fixed joints.

The finite element model of the bridge is calibrated to minimize the differences between analytically and experimentally estimated modal properties by changing some uncertain modeling parameters such as material properties and boundary conditions. Modulus of elasticity is used as the calibration parameter for each span, and will be modified to make sure that the numerical model conforms to the response of Mianeh bridge in terms of vertical deflection and natural frequencies as recorded during field tests, as shown in Figure 10.

6. Assessment of Stresses in Serviceability Limit State

According to BD 91/04 [10], the permissible compression stresses due to a loading scheme of D+1.2L in serviceability limit state shall not exceed 0.4 f_k. The BD91/04 standard also mandates that the eccentricity of the center of compression in arch ring (which is designated with letter ‘e’) shall not exceed 0.25h, in which 'h' is the overall thickness of the arch. Service loads considered in analysis are typical traffic trains of the region (including TF1 and TF2) and proposed loading scheme of ' EN 1991-2' [11] (LM71) with an axle load of 25 tons, which is presented in Figure 11.

In order to determine the compressive strength of masonry, calibrated modulus of elasticity for each span and the following equation from Euro-code standard are used [11] and the results are presented in Table 2. KE is conservatively set to 3000.

$$f_k = E/KE \qquad (1)$$

where f_k is the compressive strength of masonry, E is the modulus of elasticity.

The calibrated model is then used to assess the stresses on the bridge spans due to application of LM71 loading scheme. The corresponding results are presented in Table 3. It should, however, be noted that the results are highly dependent on modulus of elasticity of the masonry. If modulus of elasticity of the masonry in Shahbazan bridge is derived from equations proposed by UIC or Eurocode, the bridge may not sustain stresses due to LM71 loading scheme.

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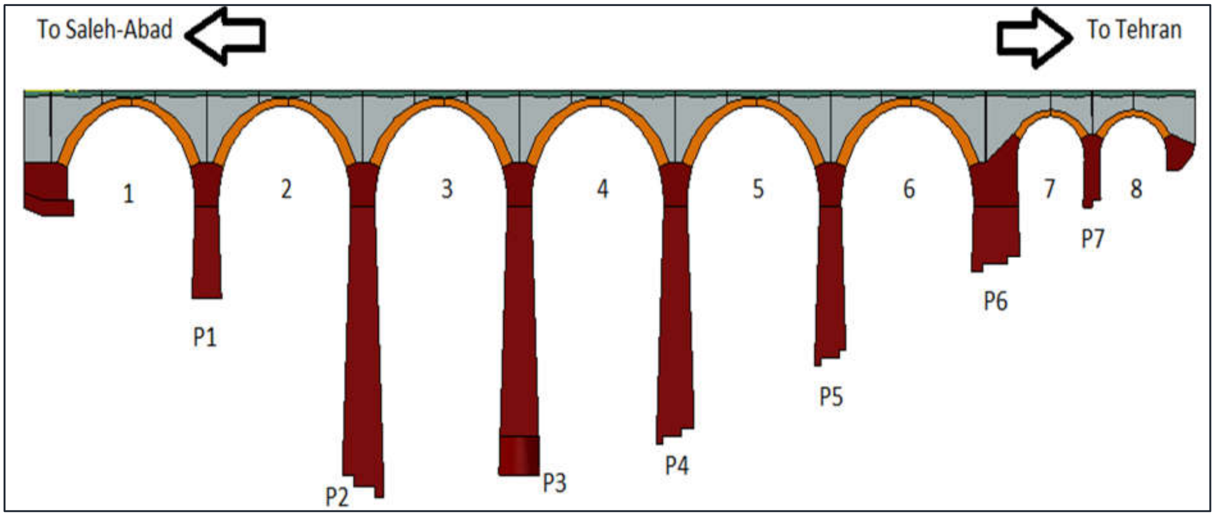

Figure 9: Numerical model of Shahbazan bridge

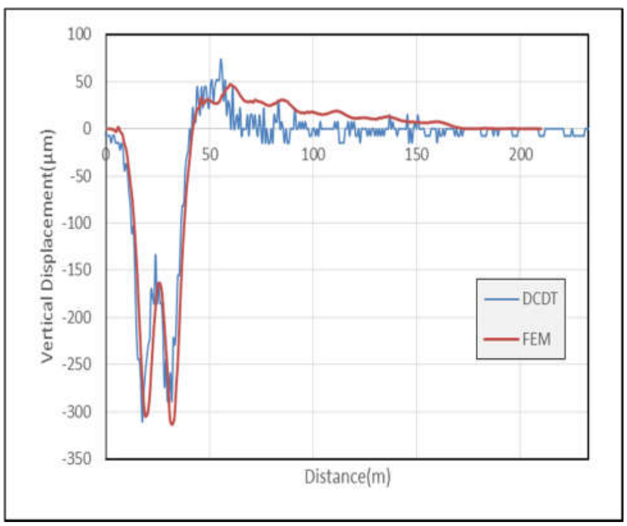

Figure 10: Calibration of numerical model using experimental deflection signature of the bridge (deflection of middle of the 1st span, for a single loco pass)

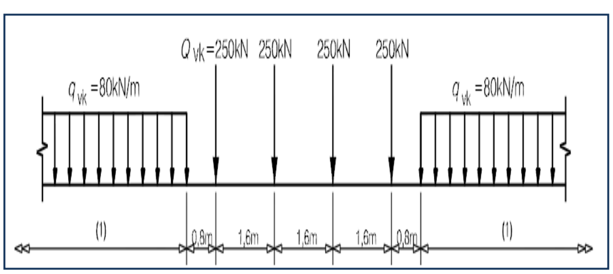

Figure 11: Calibration proposed loading scheme of EN 1991-2' ', with an axle load of 25 tons (LM71)

Table 2: Calibrated elasticity modulus, and compressive strength of modeled material

Arch ID	Modulus of elasticity	Compressive strength (MPa)	Permissible stress (MPa)
1	24	8.00	3.20
2	29	9.66	3.86
3	30	10.00	4.00
4	30	10.00	4.00
5	24	8.00	3.20
6	24	8.00	3.20
7	24	8.00	3.20
8	24	8.00	3.20

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Table 3: Stresses and the eccentricity of the center of compression in different spots of Shahbazan bridge under LM71 loading scheme

Arch ID	Position		S(Mpa)	h(mm)	e(mm)	e/h	Result
1	Mid. span	Top fiber	-1.38	1000	137.53	0.14	OK
		Bot. fiber	-0.13				
	Quarter span	Top fiber	-1.39	1200	65.65	0.05	OK
		Bot. fiber	-0.70				
2	Mid. span	Top fiber	-1.34	1000	187.33	0.19	OK
		Bot. fiber	0.08				
	Quarter span	Top fiber	-0.51	1200	14.55	0.01	OK
		Bot. fiber	-0.59				
3	Mid. span	Top fiber	-1.46	1000	165.30	0.17	OK
		Bot. fiber	-0.01				
	Quarter span	Top fiber	-0.48	1200	28.78	0.02	OK
		Bot. fiber	-0.64				
4	Mid. span	Top fiber	-1.38	1000	136.28	0.14	OK
		Bot. fiber	-0.14				
	Quarter span	Top fiber	-0.47	1200	32.97	0.03	OK
		Bot. fiber	-0.65				
5	Mid. span	Top fiber	-1.44	1000	156.78	0.16	OK
		Bot. fiber	-0.04				
	Quarter span	Top fiber	-0.48	1200	35.65	0.03	OK
		Bot. fiber	-0.69				
6	Mid. span	Top fiber	-1.14	1000	100.47	0.10	OK
		Bot. fiber	-0.28				
	Quarter span	Top fiber	-0.47	1200	17.54	0.01	OK
		Bot. fiber	-0.56				
7	Mid. span	Top fiber	-0.14	800	87.49	0.11	OK
		Bot. fiber	-0.68				
	Quarter span	Top fiber	-0.57	900	2.32	0.00	OK
		Bot. fiber	-0.59				
8	Mid. span	Top fiber	-0.10	800	99.42	0.12	OK
		Bot. fiber	-0.70				
	Quarter span	Top fiber	-0.44	900	4.97	0.01	OK
		Bot. fiber	-0.47				

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7. Conclusions

Numerical model calibration of masonry arch bridges has gained significant attention during recent years. In this method, dynamic load tests are carried out and vertical deflection of mid-spans are determined and used for calibration of numerical models by altering material characteristics and boundary conditions. Recording vertical deflection in bridges with tall piers, however, is rather difficult and wind may introduce significant error in results. To overcome such difficulties, a new approach is introduced in which deflection-meter cable is fixed to piers.

Two methods are applied in an old in-service masonry arch bridge in Iranian railway network and the results are compared. It is concluded that the pattern of vertical deflection of proposed method is almost identical to that of conventional method and the method is applicable for the purpose of structural health monitoring. However, due to vertical deflection of piers, the vertical deflection signature of the proposed method is a scaled version of that of the conventional method. Results suggest that deflection signature of conventional method could be derived from that of the proposed method by inflating it by a factor of 1.86.

The vertical deflection signatures are then used to calibrate the numerical model of the bridge which is developed in Abaqus engineering software. The calibrated model is then used to assess whether the bridge could withstand an increased axle load of 25 tons. The results suggested that all arches are safe as higher axle load is applied to the bridge and therefore, the allowable axle load of the bridge could be increased by 5 tons.

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