Technical Note

Seismic Assessment of Six-Meter Spans Plain Concrete Arch Bridge

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ABSTRACT

There are a large number of old arch bridges in Iran that have been serving as railway bridges for more than seventy years. Field load testing of an old railway bridge in km-24 of Tehran-Qom railway has revealed important characteristics of the bridge and has proven that there is still a large capacity under service load. It is known that most of these bridges are not designed for earthquake and their seismic vulnerability is uncertain. This fact necessitates the investigation of earthquake resistance of these kinds of bridges. In this paper, an attempt has been made to assess seismic performance of the bridge. The bridge is a plain concrete arch structure with five six-meter spans, which has built 70 years ago. The results of dynamic load test have used to calibrate a finite element model of the bridge in which a plain strain analysis has been carried out. Vertical displacement of the arch is used as the calibration criterion. To investigate the seismic performance of the bridge, a nonlinear static analysis (pushover analysis) method is applied. Choosing several points as a control ones, the choice of control points in the pushover analysis of masonry arch bridges and its influences on seismic evaluation is investigated. The capacity curve of structure and damage levels is drawn and the creation of hinges due to the lateral loading are studied. Finally, the capacity curves and the nonlinear demand spectrum are drawn and the performance of the structure is determined. The results show that these kinds of structures are still strong under seismic exciting.

Keywords:
Plain concrete arch bridge; Seismic assessment; Finite element model; Nonlinear static analysis

1. Introduction

There are a large number of old arch bridges in Iran that have been serving as railway bridges for more than seventy years. Although these structures have well served under service load (vertical load), they have not designed for seismic load (horizontal load). Therefore, in order to know whether the structure needs rehabilitation or not, seismic assessment of the structures are the important procedure to discover the performance levels of the structures. Due to the complex behavior of the arch bridges under horizontal and vertical load, field test is an unavoidable part of assessment procedure by which an accurate model for an appropriate assessment of the structures can be done.

There are several methods to investigate arch bridges, but the most accurate and important one is the finite element method. Finite element analysis of masonry arch bridges was first done by Towler and Sawko [1]. Following them, other researchers studied the behavior of arch bridges, most of which are for vertical load [2-4]. Even though there are many researches in the field of seismic assessment of bridges [5-6], there are a few comprehensive researches for seismic assessment of masonry arch bridges. Pela et al [7] tested two bridges and used three dimensional finite element model and nonlinear static analysis; they determined the performance levels and capacity of the arch bridges. Besides, in their study, the choice of control node was investigated, where the response in the center of mass was the critical one.

In the last two decades, several procedures have been developed in order to predict the performance levels of these structures; linear static analysis, linear dynamic analysis, nonlinear static analysis, and nonlinear dynamic analysis. In principle, nonlinear dynamic analysis is the most accurate seismic evaluation tool; however, it is difficult, time-consuming, and it cannot be considered as a common practice now. Furthermore, the simple linear static method is based on the assumption of linear elastic structural behavior and it does not provide information beyond the linear behavior.

Based on the difficulty of nonlinear time history analysis and limitations of linear analysis, recently, a great deal of effort goes to implement the nonlinear static analysis in the field of displacement base design to determine the performance levels of the structures against various levels of earthquakes. Therefore, the most rational analysis method for practical applications is the inelastic static (pushover) procedure, which takes into account the nonlinear behavior of the structure.

A case study that is a plain concrete arch bridge is tested and studied in detail. The bridge was subjected to both static and dynamic tests. The tests have revealed important characteristics of the bridge such as preliminary stiffness, limit linear behavior, and the crack patterns [8]. In this paper, using the results of field tests, an accurate finite element model of the bridge is created and it is subjected to dynamic load. Then, the calibrated model is seismically assessed. Finally, according to the obtained results, the performance levels of the structure are determined.

2. Description of the Bridge

The bridge is located at kilometer 24 of Tehran-Qom railway and consists of five identical 6 m arches. The structure suffers cracking, at the crowns of five spans, and environmental damage. The crack width, at the soffit, varies between 1 and 3 cm and extends throughout the entire section. There are also visible cracks at the springing of all spans. Reinforcement has not been used and all structural parts including the arch, spandrel wall, abutment, wing wall, pier and foundation are mass concrete constructions. Thin concrete has been used as fill material to level out the surface above the arch. A profile of Bridge Km- 24 is presented in Figure (1). Geometrical characteristics of the bridge are presented in Table (1). To determine quality of concrete, cylindrical cores were taken from different parts, and the results are presented in Table (2).

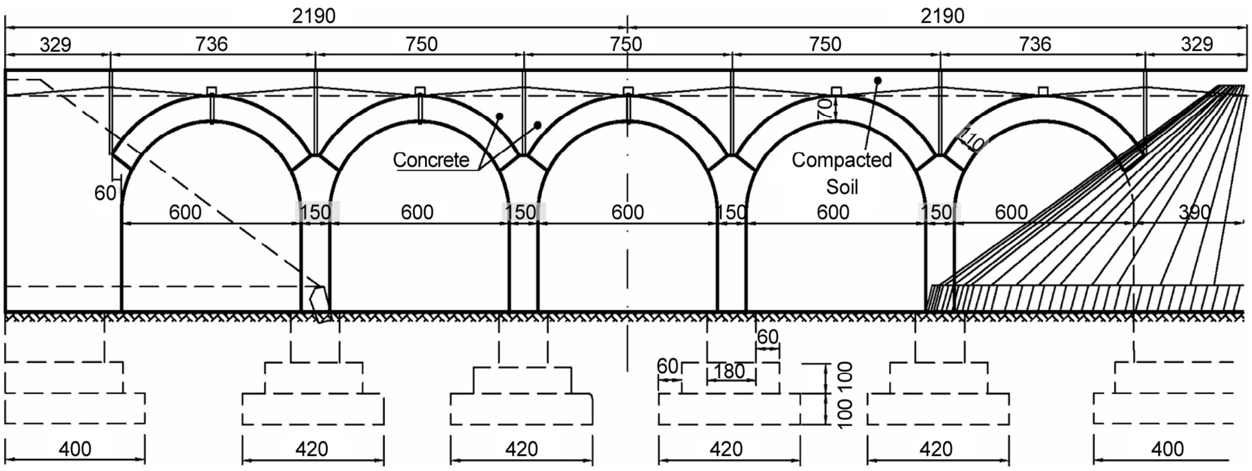

Figure 1. Geometric characteristics of Akbar-Abad Bridge (units are in centimeter).

Table 1. Characteristics of the arch bridge.

Bridge	No. of Spans	Span Length (m)	Shape of Arch	Thickness of Crown (mm)	Thickness of Arch Ends (mm)	Arch Width (m)	Bridge Height (m)	Thickness of Spandrel Walls (mm)
Km-24	5	6	Half Circle	700	1100	3.90	8	1000

Table 2. Properties of concrete based on experimental tests on cylindrical cores.

Item	Compressive Strength (MPa)	Modulus of Elasticity (GPa)	Depth of Carbonation (mm)	Density of Concrete (kg/m^3)
Concrete Fill	7.6	10.9	203	2217
Arch	39.4	24.9	0	2290
Pier	31.9	36.5	49	2250

3. Results of Dynamic Test

In the dynamic test, a 1200 kN 6-axle locomotive passed over the bridge at speeds of 10, 20, 40, 50, 60, 70, and 80 kilometer per hour and variation of vertical deflection for a speed of 80 km per hour recorded at the crown of the bridge, Figure (2). The curve includes two relatively large peaks under front and rear bogies of the locomotive. Immediately after the load has passed the span, a sharp decrease of amplitude and a relatively smooth response is observed. Fast fluctuation, under dynamic force, and rapid decrease of amplitude, immediately after passing of the locomotive, indicates the existence of a relatively large energy absorption capacity and lack of any resonance effects [8].

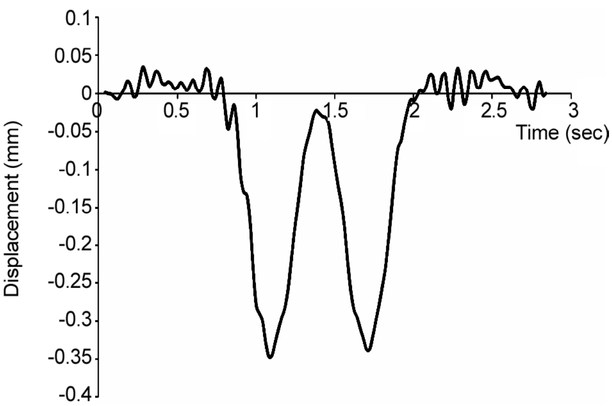

Figure 2. Variation of vertical deflection, at the crown, for a speed of 80 km/h.

Figure (3) shows the variation of vertical acceleration of Akbar-Abad Bridge (or Km-24), at the crown, under moving load. It may be observed that, when the load reaches the next span, the amplitude decreases sharply and fluctuations disappear very soon. Such characteristic was observed in all dynamic tests and indicated a convenient performance under moving locomotive. This may be described, mainly, by relatively large mass and high stiffness of the structure.

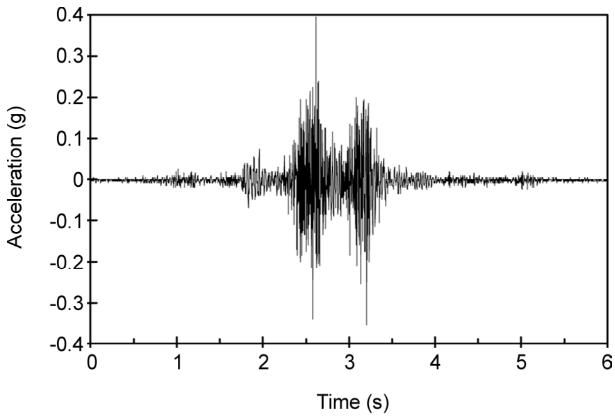

Figure 3. Variation of vertical accelerations, at the crown, for a speed of 80 km/h.

4. Numerical Model of the Bridge

Several factors can affect the behavior of the model; boundary conditions, material properties, fatigue, location and size of the cracks and etc. In this research, different parts of the bridge including arches, piers, foundation, wing wall, spandrel walls and soil are modeled in detail to represent the actual behavior of the bridge. Regarding the dominant two-dimensional behaviors of these types of structures [9], the bridge has been modeled as a plane strain finite element model, and PLANE82 and PLANE42 elements have been used in ANSYS software. The elements are shown in Figure (4). In addition, numerical characteristics of the elements are presented in Table (3).

Boundary conditions and finite element model of the bridge are shown in Figure (5). The soil beneath the bridge is modeled up to 10 meter depth. As we mentioned, there are some initial cracks in the crown, and these cracks can change the behavior of the bridge. For this purpose, the cracks and structural joints are modeled as void spaces, Figures (6) and (7). In this research, using a bilinear Drucker-Prager criterion, the nonlinearity of material has been applied. Based on the fact that concrete compressive strength is obtained from core drilled from the

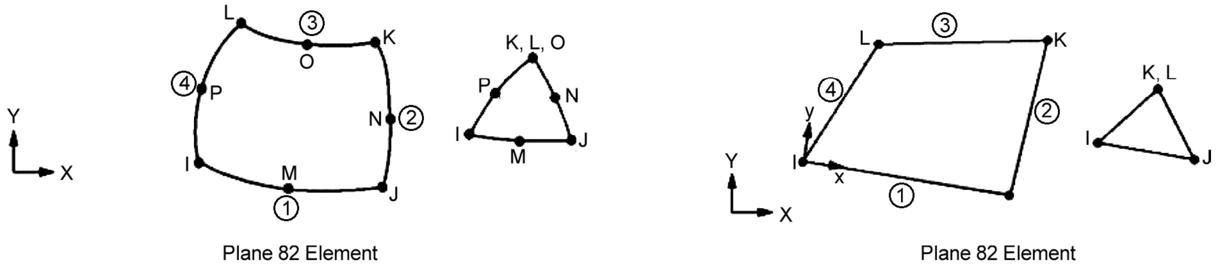

Figure 4. Element shape.

Table 3. Characteristics of the elements used in Arch Bridge model.

Item	Element Type	Number of Nodes	Element Size (m)
Arch	PLANE82	8	0.2
Pier	PLANE82	8	0.35
Concrete Fill	PLANE82	6	0.3
Soil	PLANE42	4	0.5

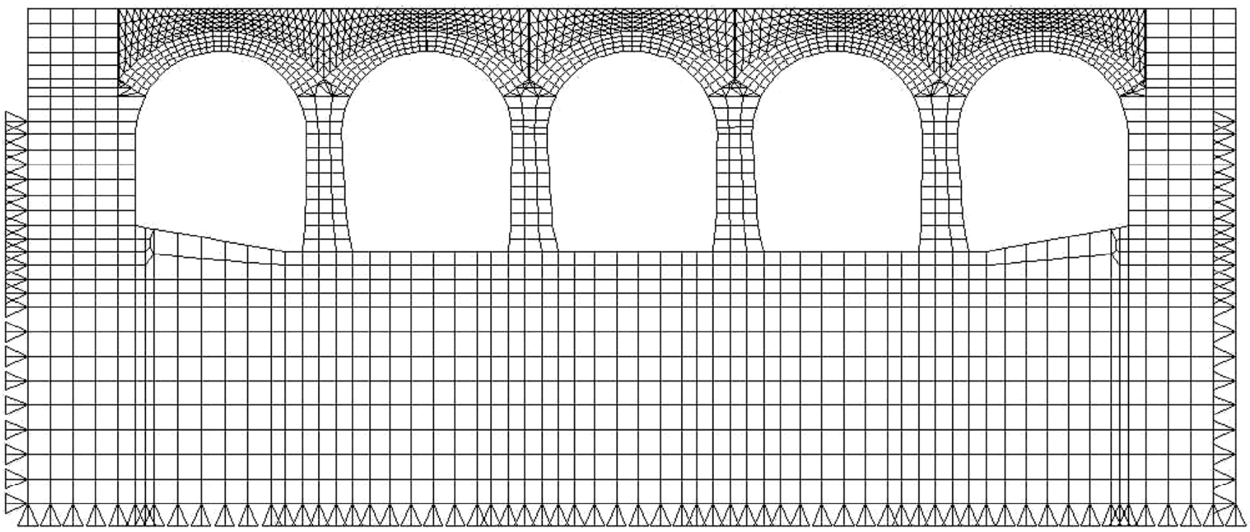

Figure 5. Finite element model and boundary condition.

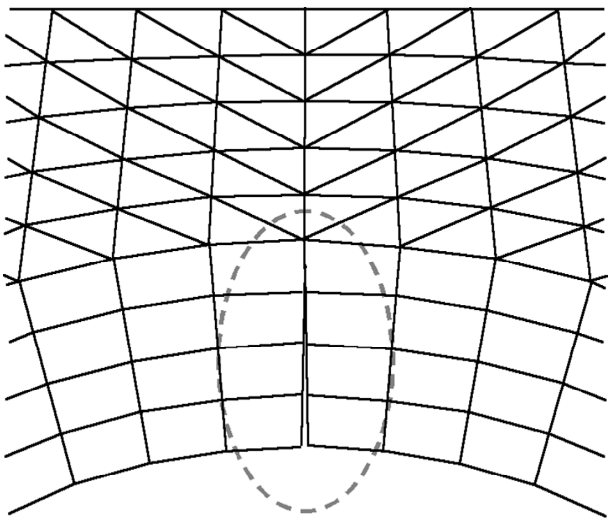

Figure 6. Modeling initial crack in the arch.

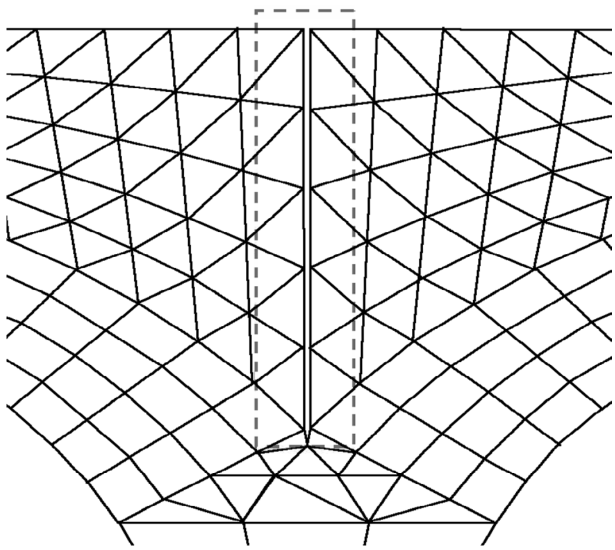

Figure 7. Modeling structural joint.

tests, the criterion parameters can be achieved directly and the mechanical properties of the soil selected as an uncertain parameter [10-11]. In order to calibrate the model, two analyses are carried out. First, the modal analysis has been performed. In this stage, to determine the first five frequencies of the model, uncertain parameters like mechanical characteristics of the soil beneath the bridge and boundary conditions are updated. In the second analysis, the time history analysis has been carried out.

For the purpose of dynamic analysis, the locomotive is considered as a moving load and the axels are concentrated load passed over the bridge at speed of 80 km/h in 0.1 sec time steps (10 Hz), which was totally done in 20 load steps. Figure (8) illustrates the locomotive axels as a moving load [8].

A plane strain model has been used to obtain secondary results. By using properties as achieved from tests, the modal analysis is performed. The natural frequencies of the model that are determined from modal analysis (first analysis) are shown in Table (4). It is necessary to mention that in this stage of analysis, undetermined parameter such as boundary conditions and mechanical properties of the soil beneath the bridge are selected as variable calibration parameters. By calibrating the mentioned parameters, the model can represent the actual behavior of the bridge. In Figure (9), the displacement correlated to the first mode of vibration is shown. As the model and the bridge frequencies are almost identical, at next step a 1200 KN wagon is crossed above the structure as a moving load. The result of field test and the model are shown in Figure (10). The final results after the calibration process are presented in Table (5).

Since the results of first five modes are similar,

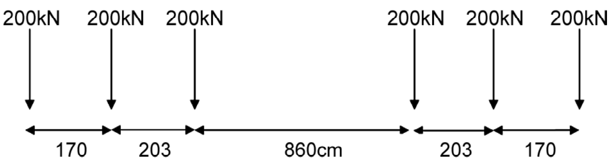

Figure 8. Simulation of Locomotive loads.

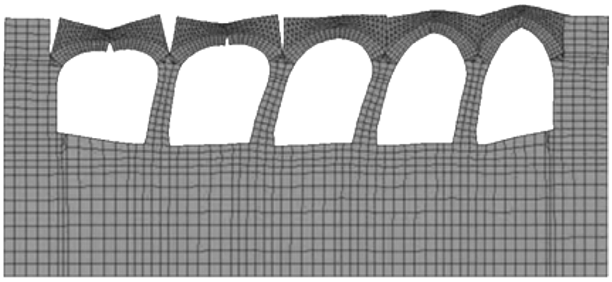

Figure 9. Displacement correlated to the first mode.

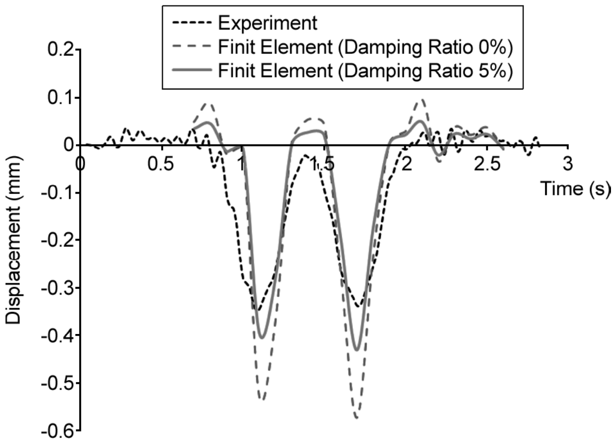

Figure 10. Simulation of the dynamic response of bridge Km-24, at the crown, for a speed of 80 km/h.

Table 4. The first five natural frequency of the bridge from test and FE model (Hz).

	First Mode	Second Mode	Third Mode	Forth Mode	Fifth Mode
Locomotive Test	14.6	21.5	26.4	31.3	35.2
Finite Element Model	14.8	20.7	22.2	27.2	28.4

Table 5. Final properties of materials after calibration.

Item	Density (kg/m3)	Modulus of Elasticity (Gpa)	Passion Coefficient	Cohesion Coefficient (Mpa)	Friction Angle (Degree)
Concrete Fill	2217	10.9	0.2883	1.915	36.5
Arch	2290	24.9	0.1676	6.591	53
Pier	2250	36.5	0.1808	5.617	51.2
Soil	2000	5	0.0.33	1	30

we can presume that the secondary dynamic characteristics of model and the bridge are equal, and this model can be used appropriately for seismic assessment. It is necessary to mention that, for the sake of simplification, damping ratio is neglected. As it is obvious neglecting the damping effect leads to response overestimation.

5. Seismic Assessment of the Bridge

The purpose of the seismic assessment is to determine the performance level of the bridge, by which the decision can be made whether the bridge needs rehabilitation or it should remain untouched. In order to evaluate the behavior of the bridge under the earthquake conditions, nonlinear analysis is necessary and the location of plastic hinges should be determined. In performance-based design method, the structure is designed according to the different performance levels. One of the most important steps in design based on performance is to estimate the nonlinear response of the structure. There are two ways to do it; nonlinear time-history analysis and nonlinear static analysis. The latter does not have the difficulty of the former, and by using the demand spectrum in the nonlinear static analysis as a demand curve, the seismic response of the structure can be suitably predicted. For this reason, displacement-based method tools such as the pushover analysis have been proven to be appropriate in the seismic evaluation of existing structures. In this research, the performance of the bridge in the horizontal direction is studied. In addition, the seismic load is replaced with an equivalent static load. Based on the fact that seismic evaluation of the masonry arch bridge is an unknown matter for researchers, there is no unique procedure to address how the bridge should be loaded. Research done by Pela et al [7] is the only comprehensive work in which the way of loading is studied. In their article, three points were suggested as control node and the results were compared. Based on the previous background, in this article, two types of loading were applied. First of all, the finite element model has been calibrated with the tests result, then after final characteristics of the structure were determined, the pushover analysis has been done in two steps. At the first step, the structure has been analyzed under the gravity loads. In the next step, the bridge has been subjected to longitudinal lateral loading. It is necessary to mention, because of the high rigidity of the bridge in the transverse lateral direction that observed, the bridge has been loaded in longitudinal lateral direction. Therefore, three-dimension model is not necessary.

In order to assess the bridge, two earthquake levels were considered; the first level for 10% occurrence probability and 475 years return period, and the second level earthquake with 2% occurrence probability and 2475 years return period [12].

6. The Capacity Curves

In order to achieve the capacity curve of the bridge, two types of loading were applied to the model and the results are compared. In the first type, a horizontal acceleration is applied to the entire structure and by selecting the center of mass of the bridge as a control node; acceleration-displacement diagram has been drawn. The next type load applied as load proportion to the mass and first mode modal displacement [13]. Thus, as to obtain the base shear-displacement diagram, three points are selected as control ones namely crown, center of mass, and a virtual point. The virtual point is obtained via the energy method. The energy of equivalent one-degree-of-freedom system is gained from the original multi degree of freedom system. In this way, the displacement energy does not belong to any specific point but it is rather a virtual point [14]. The results of the analysis are shown in Figure (11). As it is obvious, the capacity curve for different control nodes in the second type loading is almost identical. Besides, in Figures (12) and (13), the formation of plastic hinges are shown.

As it is shown in the figures, the mechanisms of failure are identical for different loading types. In the left arch, the formation of more than three plastic hinges caused local instability of the bridge.

7. Performance Levels

This bridge is located at kilometer 24 of Tehran-Qom railway. According to the reference [12], it is in a very high risk seismic zone and the maximum acceleration associated with 475 years return period earthquake is 0.35 g and for second seismic class is 0.5 g. In addition, two ground types were considered; type 2 and type 3. To obtain the inelastic spectrum, the method proposed by Fajfar and Eeri has been used [15-16]. In their research, they used ductility factor (μ_u) and reduction factor (R_μ) to convert the elastic spectrum to the inelastic one. The ratio between the maximum displacement and the linear displacement in bilinear capacity curve is defined as ductility factor. So as to estimate a bilinear curve from the capacity curve, the area under both curves must be equal and linear limit should be 60% of

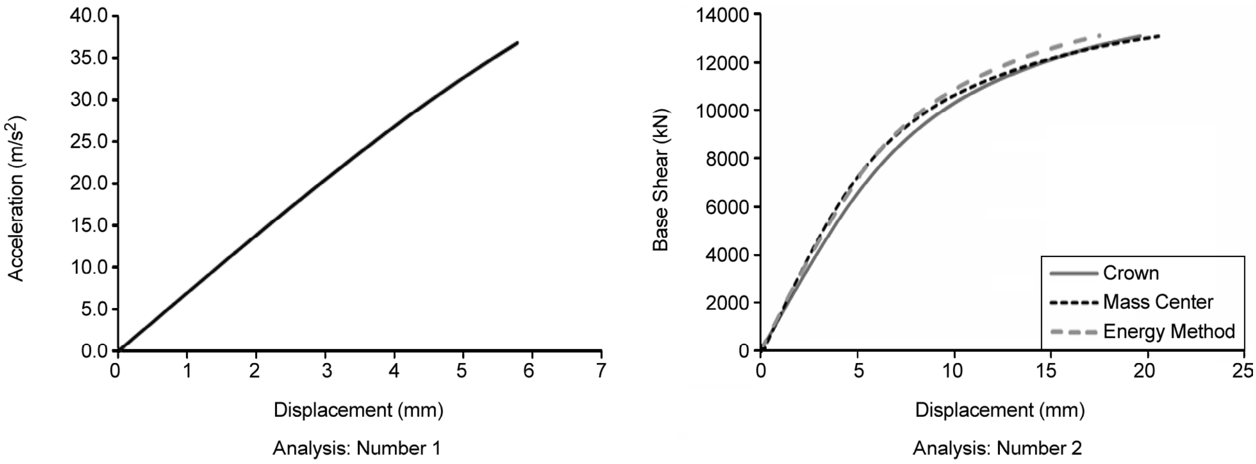

Figure 11. Shear- displacement and acceleration-displacement.

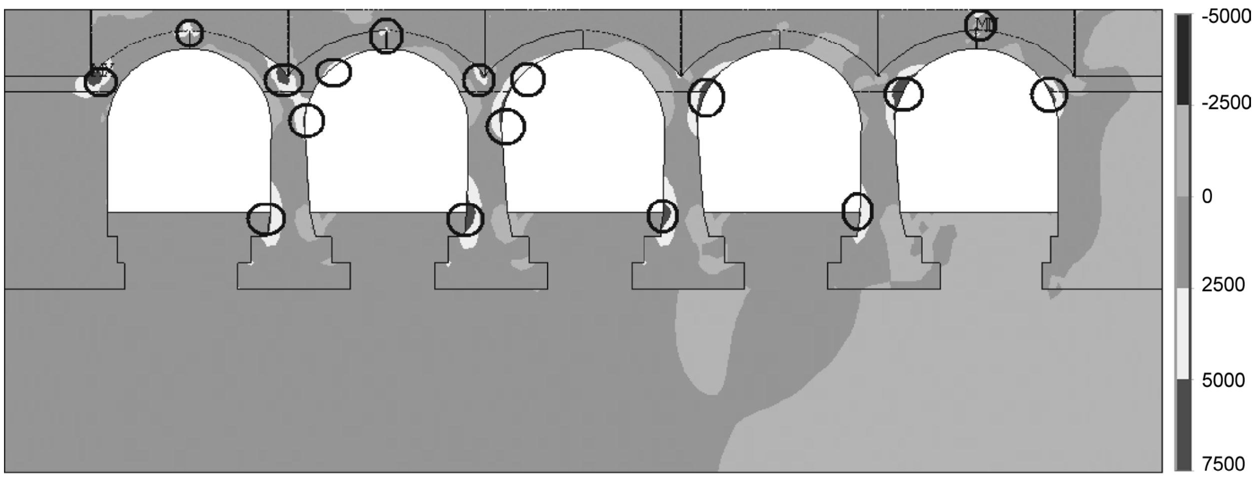

Figure 12. Failure mechanism of the bridge in first analysis.

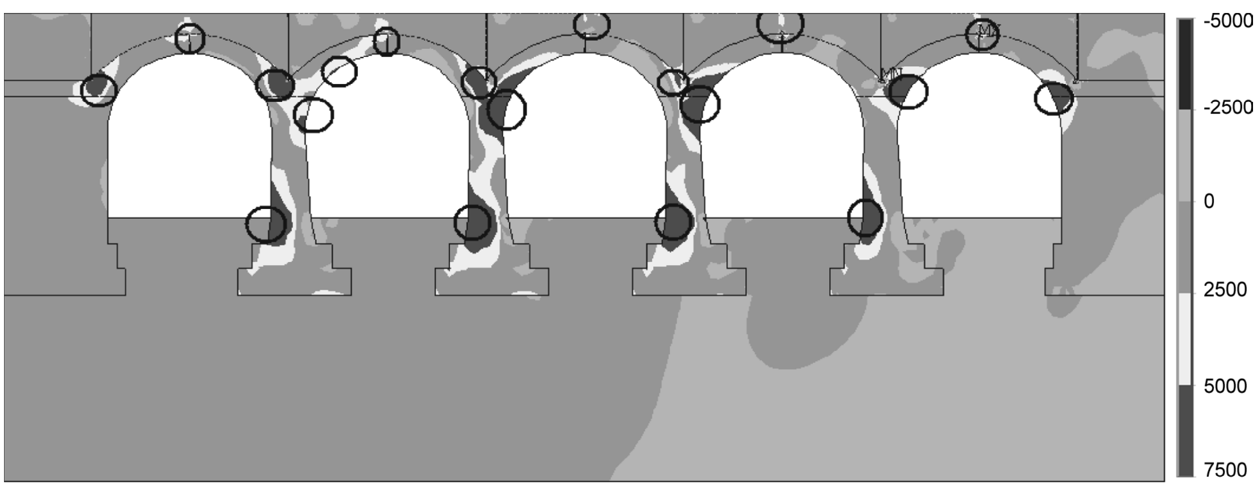

Figure 13. Failure mechanism of the bridge in second analysis.

the base shears [12]. The reduction factor depends on soil period (T_c).

$$R_\mu = (\mu_u - 1)\frac{T}{T_c} + 1; T \le T_c$$
$$R_\mu = \mu_u; T \ge T_c \tag{1}$$

The ductility factors an inelastic demand spectra versus capacity curve are shown in Table (6) and Figure (14).

Table 6. Ductility factors of different analysis.

Item	μ_u
Analysis 1	1.83
Analysis 2	4

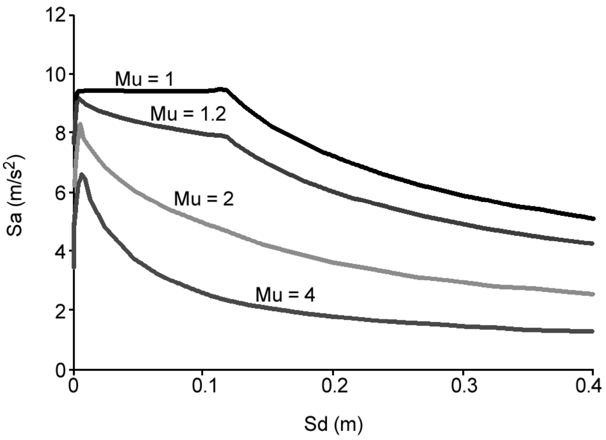

Figure 14. The effect of ductility factor (μ_u) on the inelastic spectrum.

The capacity of the structure is described by the curve of the base shear force versus displacement at a selected control point. Therefore, a multi-degree-of-freedom behavior of the structure is converted into the response of an equivalent nonlinear one-degree-of-freedom system via Eq. (2), permitting a direct comparison with the seismic demand in terms of the response spectrum.

$$\Gamma = \frac{\sum_{i=1}^{n} m_{mi}\varphi_{mi}}{\sum_{i=1}^{n} m_{mi}\varphi^2{}_{mi}} \tag{2}$$

where m_{mi} and φ_{mi} are mass and displacement of i^{th} node in m^{th} mode.

With regard to the ductility factor of the structure, the elastic seismic demand reported in the codes [12] were converted into inelastic spectrum with constant ductility. According to the bridge ductility, the in elastic demand spectrum is drawn and the point of intersection with the structure capacity curve is named the performance point. The performance of the structure for both loading types is shown in Figure (15). The safety factor is defined as the ratio between the maximum displacement and the displacement of the performance point. If the safety factor is less than unity, failure will occur and the higher the safety factor the higher the safety level

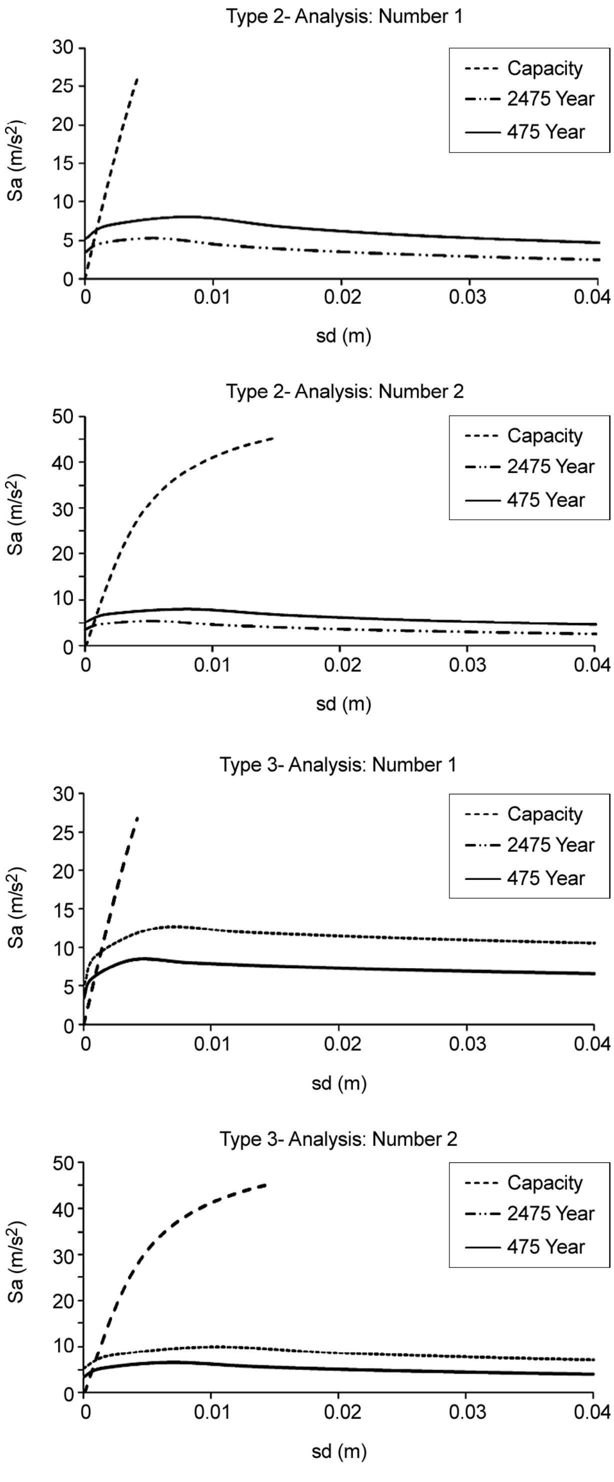

Figure 15. Performance of the bridge in different level.

of the structure, Table (7). As it is obvious from the figures, in the case of uniform acceleration load, the overall performance of the structure is worse and it is the critical case. However, even in this situation, the structure does not undergo serious damages. These results are analogous to the observation of field test where the high intensity static vertical load just caused an infinitesimal deformation. This can be due to the high strength concrete of some part of the bridge.

This kind of bridge are scattered in the railway network and the obtained results depend on the ground type, seismic zone, and the ductility factor of the bridge. In Figure (16), the performance of these bridges are shown for above-mentioned factors and different seismic zones.

Table 7. Safety factor in different condition.

	2-1[1]	3-1	2-2	2-3
475 Year	6	4.41	21.3	21.3
2475 Year	4.4	3	16.5	17.3

1. The first and the second number stand for ground type and analysis number respectively

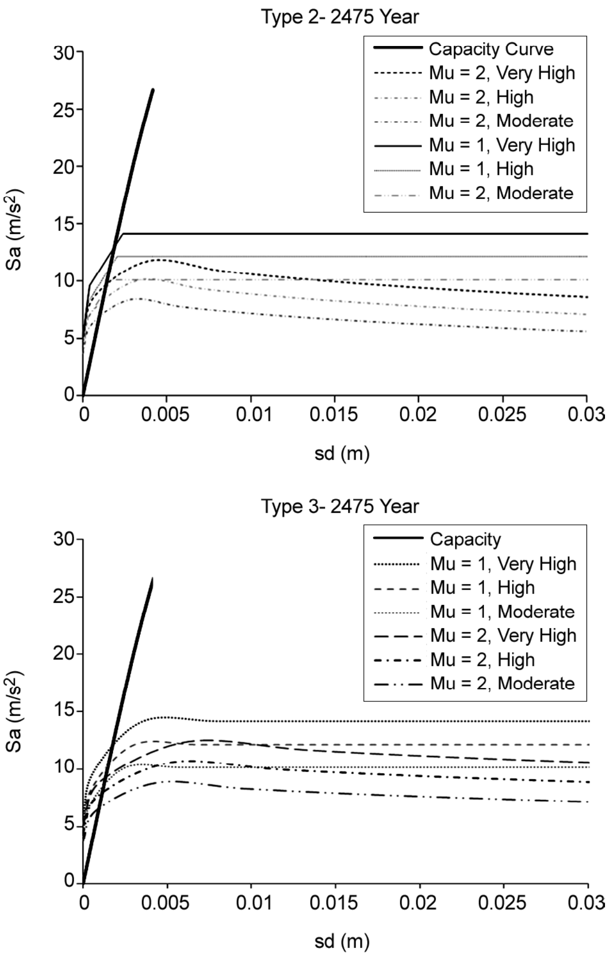

Figure 16. Performance level of the bridge.

8. Conclusion

In this paper, using a field test of plain concrete arch bridge results, a finite element model of the bridge was calibrated, and then applying horizontal acceleration and force proportional with the mass and modal displacement of the first mode of the bridge, the pushover analysis has been carried out and the location of plastic hinges was determined. In this analysis, three points were investigated as a control point and the corresponding capacity curves were drawn. Based on the results, all capacity curves are not identical. Besides, to locate the performance point using the N2 method, a linear demand spectrum was transferred to nonlinear spectrum. Generally, we can say the seismic behavior of these types of structures depends on two factors: a) parameters that affect the capacity of structures like material stiffness and spans length; and b) parameters that affect the earthquake spectrum like ductility factor, ground type and seismic hazard of the region. Based on bridge performance, it can be said that even in the strong ground motion, the bridge will not reach its capacity. Similarly, despite all the cracks and defects, as the extreme field loading of the bridge shows that the bridge is overdesigned. On the other hand, the observation and field test have shown that the structure behave like a rigid structure that confirm the results.

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