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SEISMIC EVALUATION OF AN EXISTING OVERPASS STEEL BRIDGE BY TIME HISTORY ANALYSIS FOR ITS RETROFIT DESIGN

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ABSTRACT

There are several steel bridges in the urban transportation systems worldwide, which have been designed and built several years ago, and therefore, their seismic resistance should be evaluated to decide on their possible seismic retrofit. Many of overpass bridges in Tehran metropolis fall in this category. In this study the seismic capacity of an overpass steel bridge of cantilever pier type, which is more than 35 years old, with 26 spans of 24 meters, maximum height of 6.24 meters has been evaluated. The bridge piers have rectangular cross-section with dimensions of 50 cm by 179 cm. The bridge deck girders are plate girders of I-section type with 108 cm height, web and flange thicknesses of 1.5 cm and 2 cm, respectively, and lower and upper flange widths of 40 cm and 175 cm, respectively. The upper flanges, covered by a special coating, act as the main surface of the deck for carrying the traffic loads. Girders have been connected to each other by UNP200 lateral beams. For seismic evaluation of the bridge, at first it was modeled by a powerful computer program and its modal properties were obtained. Then, several earthquake records, compatible with the bridge site soil , and peak ground acceleration of up to 0.6g were selected and by time history analysis the potential plastic hinges in the bridge piers were recognized. Cape Mendocino, Northridge, Kobe and Palm Spring are among the used earthquakes. Results show that the bridge is laterally weak, and its short piers at both ends of the bridge are susceptible to shear failure. On this basis some retrofit designs, such as using supporting cables at either sides of the bridge deck, and lessening the stiffness of short piers are now under study to be finalized in near future.

Keywords: Cantilever piers, Plate girders, Lateral beams, Plastic hinges, Short piers

1. INTRODUCTION

There are several steel bridges in the urban transportation systems worldwide, which have been designed and built several years ago, and may have insufficient seismic resistance, based on the criteria of the recently developed seismic design codes. Therefore, their seismic resistance should be evaluated to see if they need any seismic retrofit. Many of overpass steel bridges in Tehran

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metropolis fall in this category, and therefore, they need seismic evaluation. The fist works conducted on seismic evaluation of steel bridges go back to early 90s (Astaneh-Asl 1993), and has been continued up to now. Among these works just a few ones have conducted dynamic analysis (Dameron et al. 2003; Imbsen and Sarraf 2003; Usami et al. 2004; Padgett and Des Roches 2008).

In this study the seismic capacity of an overpass steel bridge of cantilever pier type, which is more than 35 years old, with 26 spans of 24 meters, maximum height of 6.24 meters has been seismically evaluated, by using time history analyses. The details of the study are presented briefly in the following sections of the paper.

2. INTRODUCING THE CONSIDERED BRIDGE

A general view of the considered cantilever pier type overpass bridge, showing almost half of the bridge, is shown in Figure 1.

Figure 1: A general view of the considered cantilever pier type overpass bridge

The transverse and longitudinal beams carrying the bridge deck, and the tapered cantilever girders, transferring the deck load to the piers are clearly seen in Figure 1. The maximum and minimum heights of piers are respectively 6.24 m and 0.75 m, and all piers have rectangular cross-section with dimensions of 50 cm by 179 cm. The bridge deck main beams are plate girders of I-section type with 108 cm height, web and flange thicknesses of 1.5 cm and 2 cm, respectively, and lower and upper flange widths of 40 cm and 175 cm, respectively. The upper flanges of the deck girders, covered by a special coating, act as the main surface of the bridge for carrying the traffic loads. Girders have been connected to each other by some lateral trusses whose upper cords are beams made of UNP200, as shown in Figure 2.

Figure 2: The lateral trusses connecting the deck girders together

Construction of the bridge dates back to four decades. Unfortunately there is no map and technical document of the bridge from that time. In 1993, the as-built drawings of the bridge were provided by a consulting firm, which have been used for developing the computer numerical model of the bridge for its seismic evaluation, as explained briefly in the next section.

3. DEVELOPING THE NUMERICAL MODEL OF THE BRIDGE

For seismic evaluation of the bridge, it was necessary to model the bridge structure by a powerful computer program, as shown in Figure 3.

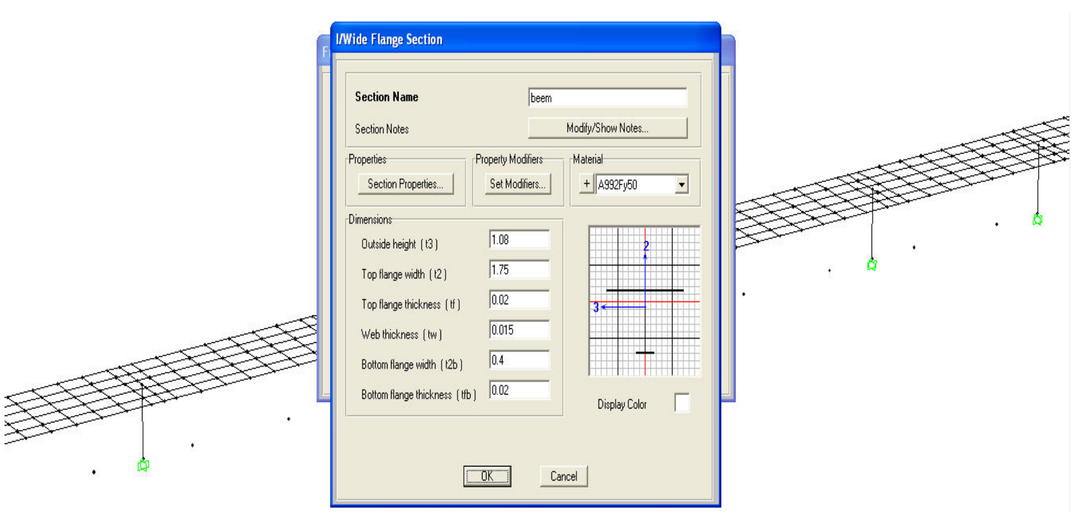

Figure 3: Developing the bridge structure’s numerical model

By using the numerical model the fundamental period of the bridge structure is obtained as 0.33 sec, which corresponds to the longitudinal motion of the bridge deck and bending of the bridge piers with respect to their weak axis. The fundamental period can be obtained by manual calculation as well. For this purpose the stiffness of all piers of the bridge in its longitudinal direction are calculated as shown in Table 1.

Table 1: The stiffness values of the bridge piers

Pier height (m)	0.70	1.90	2.85	3.80	4.79	5.50	6.00	6.24	2.69
Piers stiffness (tonf/m)	48137	53361	15811	6670	3330	2200	1694	1906	18803
Number	2	2	1	1	2	3	2	12	1

In Table 1 the stiffness values of piers with different heights and the number of piers with the same stiffness can be seen. Based on the total longitudinal stiffness of the bridge structure and its total weight the fundamental period of the bridge structures by using the code simple formula is obtained as 0.36 sec, which is close enough to the value obtained the computer numerical model.

4. SEISMIC EVALUATION OF THE BRIDGE

To evaluate the seismic behavior of the bridge structure, several three-component earthquake accelerograms, compatible with the bridge site soil, with different peak ground acceleration (PGA) values up to 0.6g, were selected. Cape Mendocino, Northridge, Kobe and Palm Spring are among the used earthquakes. Figures 4 and 5 show the time histories and the pseudo acceleration spectrum of one of the three components of Palm Spring earthquake as a sample of the used earthquakes.

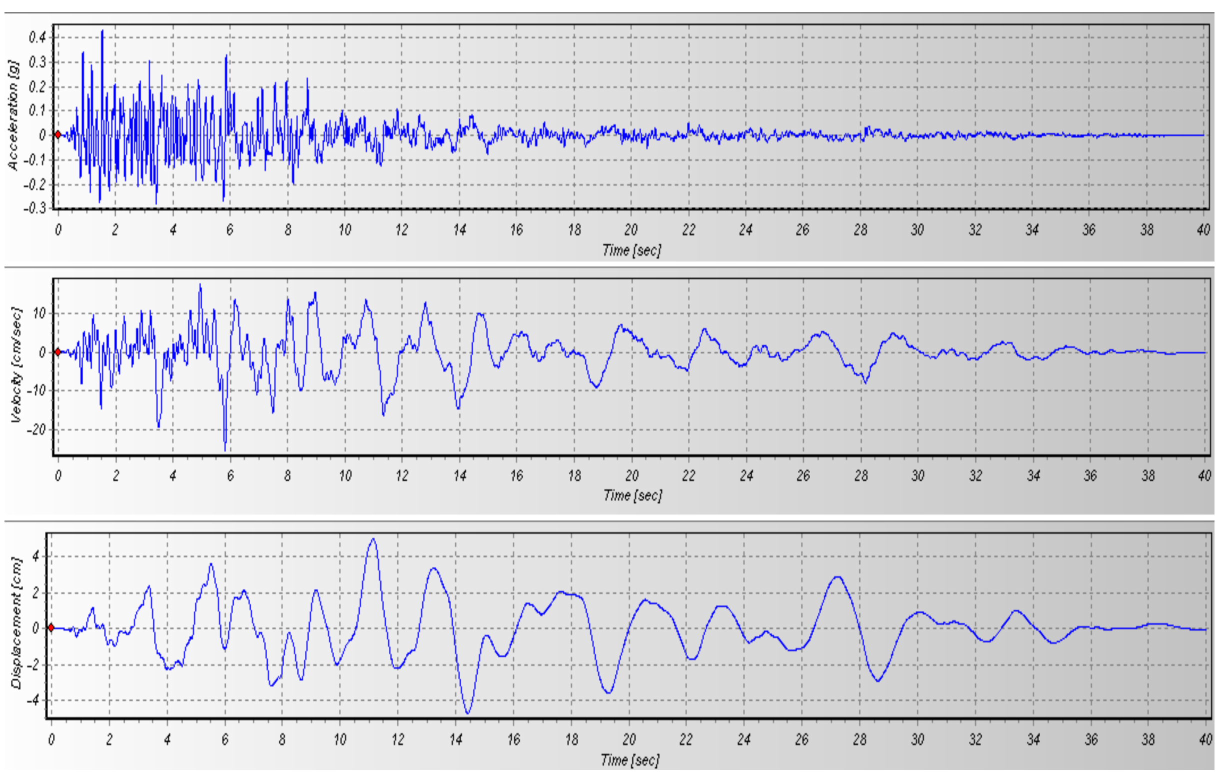

Figure 4: Acceleration, velocity, and displacement time histories one of the components of Palm Spring earthquake as a sample of the used earthquakes

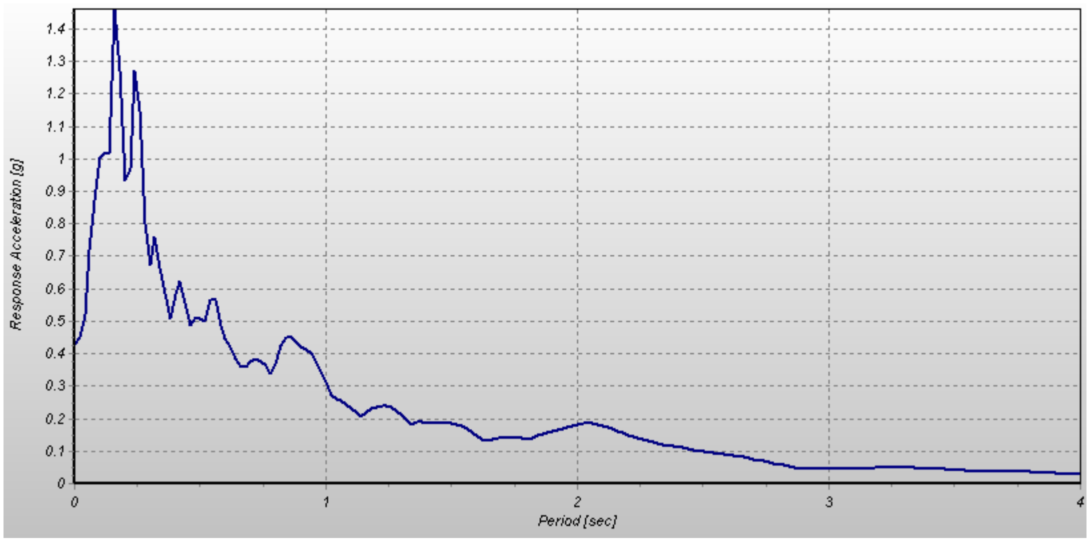

Figure 5: Pseudo acceleration spectrum of one of the components of Palm Spring earthquake as a sample of the used earthquakes

Each set of three-component accelerograms were applied to the bridge structure with various incidence angles, form 0 to 90 degrees, by 10 degrees increment for find the critical incidence degree for each case. Regarding that the stiffness and the height of piers are not the same, it is obvious that the shortest pier will experience the highest stress values. Table 2 shows the values of the maximum shear force and resulting shear stress, and related section parameters, for using in the well-known formula $\tau=V.Q/(I.t)$, in case of the shortest pier of the bridge subjected to Palm Spring record as a sample.

Table 2: Maximum shear stress values in the shortest pier of the bridge subjected to Palm Spring earthquake records

The pier section moment of Inertia ($I - m^4$)	Two times of the wall thickness of the pier box section ($t - m$)	The shear force value ($V - kgf$)	The shear stress ($\tau - kgf/cm^2$)
0.006183	0.04	1471205	1827

It is seen in Table 2 that the maximum shear stress exceeds remarkably the allowable value, which is 960 kgf/cm^2 and this means that the short piers of the bridge are weak in shear. As another main factor for checking the behavior of the structure in experiencing the plastic phase, the maximum combined stress value, based on the normal and the shear stresses in the bottom section of the bridge piers was considered. This normal stress value can be calculated by the following well-known formula, which is related to biaxial bending case.

$$\sigma = \left(\frac{M_3}{S_3} + \frac{M_2}{S_2} + \frac{P}{A}\right) \qquad (1)$$

where M_3 and M_2 are respectively the moment value around the two main axes of the pier section, P is the vertical load, S_3 and S_2, are respectively the modulus of section of the pier with respect to its main axes, and finally A is the cross-sectional area of the pier. The shear stress values are also obtained by the well-known corresponding formula, and then the combined stress values are obtained for different points in the bottom section of the pier.

A sample time history of the maximum values of normal stresses, calculation by Eq. (1), for the case of Palm Spring earthquake is shown in Figure 6.

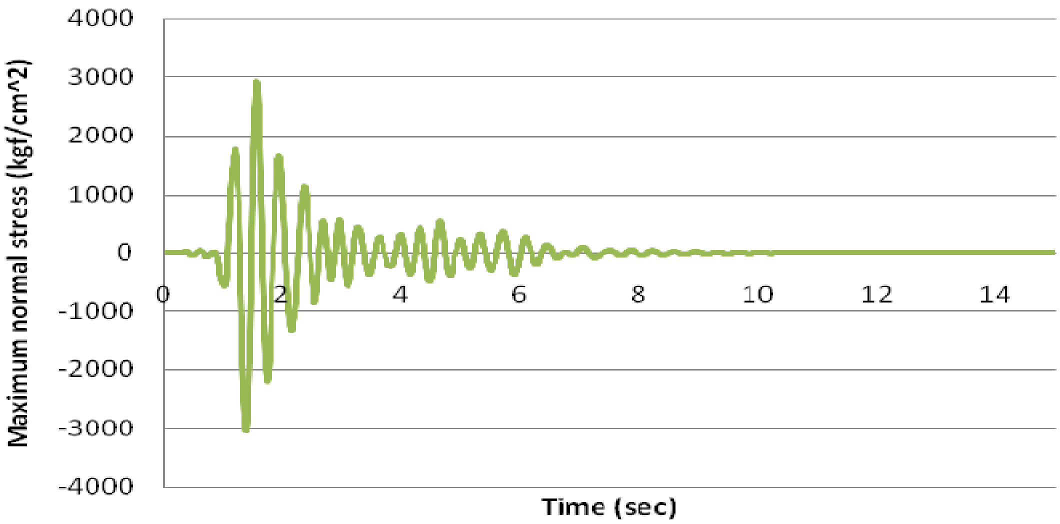

Figure 6: Sample time history of the maximum values of normal stresses at the bottom shortest pier of the bridge subjected to Palm Spring record

The maximum normal stress value in Figure 6 is 3150 kgf/cm^2 which is far beyond the yielding stress value of 2400 kgf/cm^2. This means that the bridge piers experience plastic deformations when subjected to Palm Spring earthquake records. Another important response value for evaluating the seismic behavior of the bridge is it maximum displacement response. Figure 7 shows the displacement response histories of a point at the top of the tallest pier of the bridge subjected to Palm Spring earthquake record.

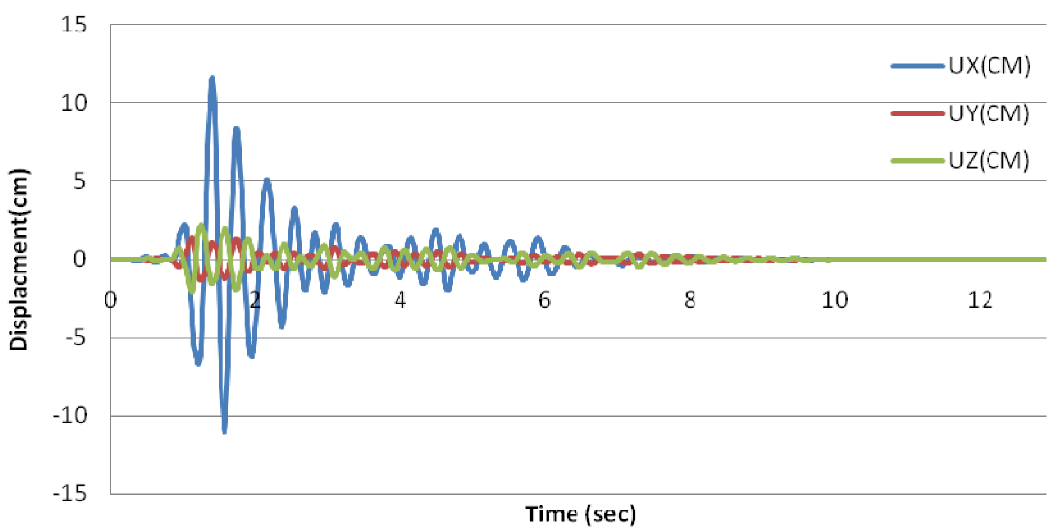

Figure 7: Displacement time histories (in three main directions: X as longitudinal, Y as lateral and Z as vertical) at the top of the middle pier of the bridge subjected to Palm Spring record

It can be seen in Figure 7 that the maximum displacement response of the bridge is in its longitudinal direction. The maximum absolute displacement value of the bridge subjected to Palm Spring earthquake, which is around 12.5 cm, is more than the provided gap between the deck and the bridge abutment.

As the last issue for seismic evaluation of the bridge, the effect of angle of incidence of the earthquake excitation was investigated. For this purpose this angle was considered to vary form 0 to 90 degrees by and increment of 10 degrees. The maximum responses were observed when the incidence angle of the main component of earthquake excitation was 0. This is due to the dominancy of the longitudinal motion of the bridge deck in it vibration modes.

5. PROPOSING THE RETROFIT DESIGN

In order to propose a suitable seismic retrofit design for the bridge, adding some element for increasing the stiffness of the bridge in its longitudinal direction was considered, based on the results of seismic analyses. In this regard using some cables, were considered as diagonal elements between piers at both sides as shown in Figure 8.

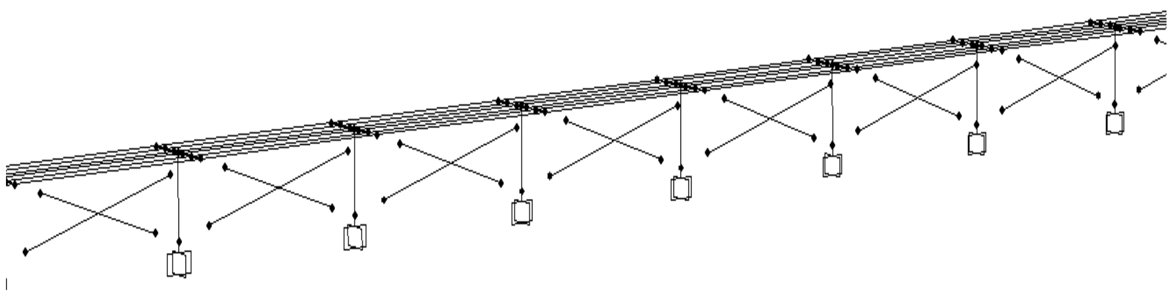

Figure 8: The proposed seismic retrofit design for the bridge

The details of the retrofit design and its evaluation can not be presented here because of lack of space, and can be found in the main report of the study (Hashemnejad 2012).

6. CONCLUSIONS

Based on the numerical results obtained by extensive time history analyses of the bridge structure, it can be concluded that:

- The bridge structure is weak in longitudinal direction to excessive motion, and therefore, collision between the deck and the abutment is very likely.
- The short piers of the bridge at its both ends are susceptible to shear failure, and if they fail during a large earthquake the excessive force transferred to other piers can result in the collapse of the bridge, even subjected to lateral forces.
- The tall piers of the bridge are also weak against the combined effect of shear, and axial forces and bending moments.

Based on the above facts, the bridge can be considered as highly vulnerable subjected to the probable earthquakes in Tehran metropolis. It is also worth mentioning that the age of the studied bridge is over 40 years, and in this regard the fatigue can be a serious problem. On this basis it is recommended that the allowable stress values are modified and used in the seismic evaluation of the bridge. Finally, it should be noted that as this bridge and several bridges, with similar structural system exist in Tehran, and all of them have crucial roles in traffic flow in this metropolis, the seismic retrofit of this bridge and its similar ones should be given priority by the authorities of the seismic retrofit programs of the county.

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