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Seismic performance assessment of composite concrete-steel deck truss bridges

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Abstract. Bridges are of significant structures during the occurrence of natural disasters in terms of two aspects: structural resilience and continue providing lifeline service after the event. Analyzing the data regarding the vulnerability of bridges in Iran over the past three decades indicate that a considerable number of the bridges do not possess sufficient quality to continue providing service and withstand the impending earthquake in their lifetime. The possible reason could be attributed to the fact that today's development in seismology and seismic design codes were not available a few years ago. This paper studies the seismic behavior of composite concrete-steel deck truss bridges. The case study is conducted on Aly Dar Bridge with concrete-pier-based steel deck truss system which is one of the important arterial bridges in Khash-Iranshahr route in Sistan and Baluchestan province of Iran. The bridge is simulated using SAP2000 software. The mechanical properties of bridge material and its existing condition are obtained through site investigation and in-situ observations. This study involves linear and nonlinear static analyses based on AASHTO bridge design recommendations and also linear and nonlinear dynamics analyses. The results indicate that in order to comply with the requirements prescribed for the displacement of the bridge deck, increasing the deck seating length is necessary. Moreover, as regards the distribution of plastic hinges, the columns at the junction of foundation should be strengthened to prevent the formation of plastic joints in the substructure.

Key words. Steel deck truss bridge, retrofit, nonlinear static analysis, dynamic time history analysis.

1. Introduction

Bridges are key structures since any damage inflicted to them incurs life and property losses during and after earthquakes. Considering the damage sustained by structures, particularly the arteries, during the past earthquakes, a variety of methods have been increasingly developed during recent decades aimed at seismic rehabilitation and retrofit of the structures. Among the engineering structures,

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bridges have considerable importance. Casualties and economic losses after a violent earthquake will be decreased if bridges are not damaged and are not out of operation. Therefore, it is necessary to identify plausible damages inflicted to bridges and their causes for a safe design practice. Until the publication of AASHTO codes in 1983, inertial forces induced by the earthquake were not taken into account in the seismic design practice of bridges. Although the requirements of seismic retrofit were recognized after the San Fernando earthquake in 1971, but it was Loma–Prieta earthquake in 1989 that urged the Caltrans to begin extensive studies in order to strengthen highway bridges and especially steel deck bridges [1]. Until 1995, seismic design of new bridges and retrofit of existing bridges were performed based on the design philosophy of life safety performance through control of deck collapse on the support. But after the Northridge (1994) and Kobe (1995) earthquakes, it was found out that the design philosophy of keeping the structure at life safety performance level is not sufficient and retrofitting measures for the life safety performance level are not adequate, and collapse and failure philosophy must be avoided as much as possible [2].

Nearly over the last century the steel truss has been often used for the construction of highway bridges. Until the end of the twentieth century, truss bridges were the preferred structure for long-span bridges. Over 80 percent of bridges with spans longer than the AASHTO limit of 150 m are truss bridges. Most of these structures were designed without the current level of knowledge about ground motions, magnitude of seismic forces, and complex structural responses. Among the various types of steel bridges, deck-truss bridges are particularly vulnerable to earthquake induced inertial forces. In these bridges, the structural system (sway frames/bracing) should remain in the elastic region to be capable of transferring the deck forces safely to the supporting substructure (piers and abutments) through bearings [1, 2].

Skagit River Bridge located in Mount Vernon, Washington, as the last example of steel truss bridge collapse, was built in 1955 and was collapsed in 2013. The bridge collapsed due to the fact that its four spans were built independent of each other and there was no extra structural element in the design to secure the structure versus destruction of one span. By erosion of three parts of a truss, internal forces of truss elements were increased and indirectly caused destruction of the compressive elements of truss's upper chord and in the end, led to the immediate collapse of one span. This event could be avoided by objective assessment of the bridge and adding or replacing some parts based on new seismic assessment models [1].

Machacek and Cudejko [3] studied the distribution of longitudinal shear along an interface between steel and concrete in composite truss bridges. Their findings indicate that nonlinear distribution of longitudinal shear is highly dependent on rigidity of shear connections and densification of shear connections at truss nodes. Mechanical behavior of composite joints in a truss cable-stayed bridge was investigated based on experimental tests and finite element method. Their results reveal that the maximum strain of the steel plate and concrete cord remain in the elastic region for up to 1.7 times the design load, which demonstrates the considerable safety margin of such composites. Liu et al. [4] studied the performance of joints in double deck truss bridge and carried out fatigue tests under repeated loading on three different composite joints, namely headed studs, concrete dowels, and perforated plates. The experimental results indicated that by increasing the applied load the deflection increased linearly and the stiffness gradually decreased. Examining fatigue details involving reinforcing bars, welding seams, and shear connectors revealed that the major factor affecting the fatigue resistance of joints is stress concentration of welding seams at intersections between gusset plate and diagonal truss members. Alampalli and Kunin [5] investigated the behavior of the rehabilitated truss bridge. The results of load testing conducted on the bridge demonstrated that no composite action exists between the deck and the superstructure and localized bending effects play a major role in the strain distribution of FRP decks. Yin et al. [6] studied the performance of connections in steel-concrete composite truss bridges. They conducted experimental tests on eight specimens and performed finite element analyses to evaluate the mechanical behaviour of connections. The results of experimental tests and FE analyses indicated that decreasing the average strain of the concrete chord along with increasing the strain of perfobond-rib (PBL) shear connectors improves the load transfer capacity of these type of connections. Kareemi et al. [7] carried out experimental tests on composite steel-concrete truss beams made of prefabricated steel trusses encased in concrete. The observed internal resisting mechanism was that of a steel truss that interacts with an inclined concrete strut. This concrete strut carries the load that cannot be carried anymore by the yielded steel bars and transfers the shear forces to the adjacent triangles, leading to an increase in the overall shear capacity of the composite beam. Sarraf and Bruneau [8] proposed an innovative retrofit strategy to enhance the seismic performance of deck-truss bridges. Their alternative was based on replacing the existing non-ductile and lower-end bracing with ductile panels. These energy dissipating devices can yield and protect both superstructure and substructure. Their findings indicated that the ductile retrofit devices exhibit a perfect hysteretic behaviour and prevent damage in other structural members of the bridge by dissipating the seismic energy. Li et al. [9] proposed a simple analytical solution to predicting deflection of a hybrid FRP-aluminum modular space truss bridge. According to the findings of their study, the deflection caused by the bridge deck and vertical web members was small, and the deformations of the lower chord members and diagonal web members contributed more to the overall deflection of the bridge.

Borchers implemented two methods of robustness and redundancy in the bridge design. In this study various types of trusses including Warren, K, and the Pratt truss were analyzed. The 2D bridge was modeled in SAP2000 using linear static analysis. Value of forces of the robust and redundant bridges and the original ones were compared. The results showed that robustness and redundancy are effective for bridges with long and short spans, respectively. Also, it was concluded that the cost of designing a bridge to be resilient is a marginal percentage of the overall cost of constructing the bridge [10].

Peckan et al. [11] investigated the seismic retrofit strategies for steel deck-truss bridges. They presented an economical retrofit method protecting both superstructure and substructure of steel deck-truss bridges by embedding fuse elements and a type of energy dissipation device in the end-sway frames. They concluded that using fuse-bars with high capacity reduces the intensity of forces transmitted to the supports. On the other hand, the use of fuses and dampers in the piers decreases the demand for retrofitting the existing steel bearings, and the axial forces of the columns.

From the results, it can be demonstrated that diagonal members at the mid-span and also hangers and posts should have adequate strength to withstand lateral loading In this paper seismic performance of steel truss deck bridges are studied by performing linear and nonlinear static and dynamic analyses. A comparison is made between the bridge responses obtained by the aforementioned analysis methods. For this study Aly Dar bridge located in the south-east of Iran is modeled. The bridge specifications are described in the next session.

2. Qualitative evaluation of steel bridge deck truss

Generally, the improvement of an existing bridge design requires preparing a systematic study containing the investigation of bridge parts and plausible failure modes, gathering information and investigating the site, collecting the records of previous earthquakes and identifying important faults at the surrounding area, and so forth. Aly Dar bridge (Fig. 1), constructed in Khash-Iranshahr route in 1972–73, is 150 m long and 10.5 m wide (Fig. 2) and is a two-way bridge. It is spanning a valley and is constructed over a seasonal river (with the water depth of 1.5 m) with five 30 meter spans and is the largest bridge on Khash-Iranshahr route. Khash-Iranshahr path is the only available south-to-north path in Sistan and Bluchestan province and has great importance in the transportation network. In other words, this line is considered as the only vital artery in the area.

Fig. 1. Aly Dar bridge

Cross section of this bridge includes two 3.5 m traffic lane and 75 cm curb, 75 cm pavement and 35 cm guardrail on both sides, (see Fig. 3)

Aly Dar Bridge contains a truss with composite material of concrete and steel. It is located on a type III soil (175<Vs<375 m/s) in a region with high seismic risk. Based on the available technical reports, the soil at the site comprise from pure clay to silty sand along with cobble with a zero liquefaction potential. General studies of the bridge resulting from site investigation and its evaluation are summarized in Table 1, and also the accuracy of material properties is determined in the modeling.

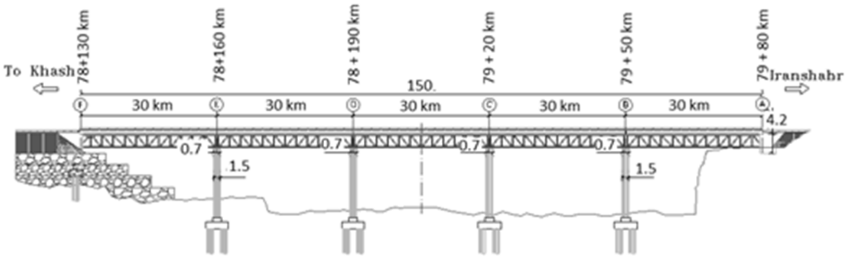

Fig. 2. Longitudinal profile of Aly Dar bridge (dimensions in meters)

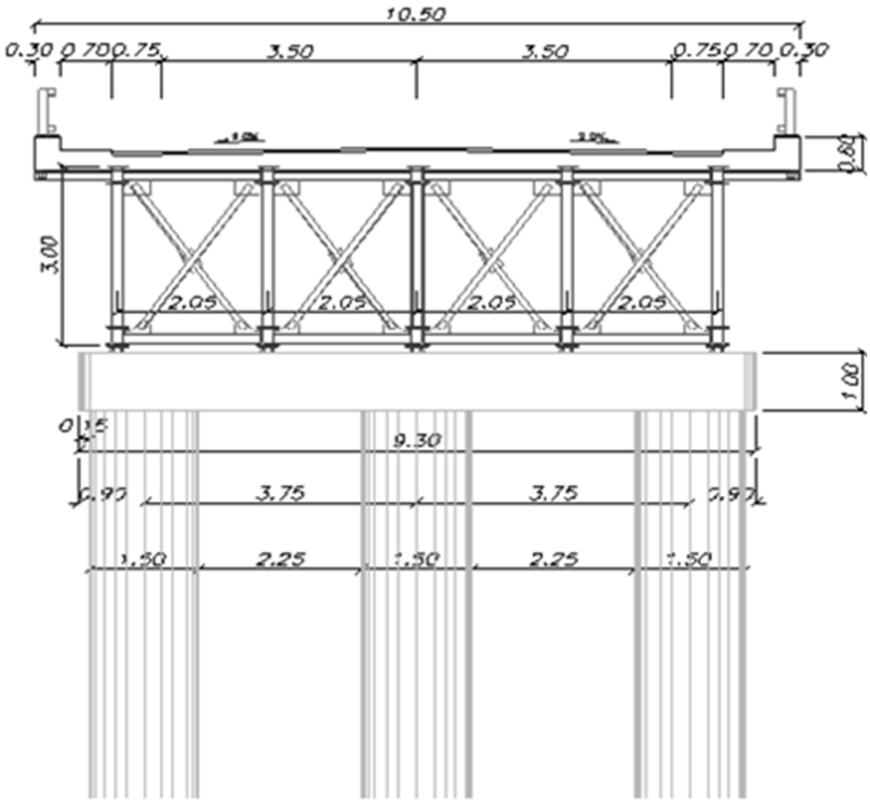

Fig. 3. View of bridge cross section (dimensions in meters)

3. Structural model and material properties

The general method used in this study aims to investigate and analyze the modeling of the bridge by finite element method based on SAP2000 software. Concrete deck is simulated using the shell four-joint element with six degrees of freedom. Steel truss, concrete columns and concrete bent caps are also modeled by frame element.

Moreover, deck connections and the columns are modeled using the link element. Hinge elements are defined using non-linear characteristics of concrete material. The characteristics of the concrete and steel materials are presented in Tables 2 and 3.

Table 1. The result of objective visit and its evaluation

The studied elements	Specifications	Material
Foundations	Without non-uniform settlements, crack and twist	Concrete
South abutment	Dimensions: 8×9 m, without non-uniform settlement and twist-honeycombed in some points superficially	Concrete
North abutment	Dimension: 8×5.8 m, without non-uniform settlement and twist-honeycombed in some points superficially	Concrete
Columns	Height 30 m and diameter 150 cm, without non-uniform settlement and twist	Concrete
Bent caps	Dimensions 100×170×930 cm, honeycombed and cracked superficially, no considerable shear crack and rise is observed	Concrete
Slabs	Dimensions 150×5.10 m, no rise, crack and breading is observed	Concrete
Girders	Length of 50 m and height of 3 m, steel truss- rolled- without rust, crippling and deformation, acceptable quality for joints	Steel
Bracings	Length of 3×5.2 m, coaxial and cross-having deformation from inappropriate implementation way-inappropriate joint quality	Steel
Expansion	Length of 5.10 m with opening of 7 cm on the columns and abutments-seam edge with single angle profile	Steel
Seat	Length of 40×40 cm and height of 25 cm, constant type without the possibility of freely movement in longitudinal and transverse directions- having an appropriate situation of joint and welding	Steel

Table 2. Characteristics of the utilized material in the bridge

Material	Density (kg/m^3)	Minimum compressive strength on standard sample (kg/m^2)	Concrete class
Concrete	2500	280	C-28
Material	Density (kg/m^3)	Flowing strength (kg/m^2)	Final strength (kg/m^2)
Steel ST-37	7850	2400	3700

4. Loading and analysis of bridge

The dead load, live load, and earthquake load are defined according to AASHTO HB-17 (2002) [12–14]. The dead load is assumed 220 kg/m^2 and the live load is equivalent to the truck type H and HS. The seismic characteristics of the structure are as follows: PGA is equal to 0.3 g. Site soil is of type III and the behavior factor $R = 3$ is used.

Table 3. Characteristics of the bridge sections

Elements of bridge	Characterization
Columns (bent)	Circle with 1.5 m diameter-42 longitudinal bar with 2.5 cm diameter transverse bar of 1.27 cm diameter with 15 cm distances
Bent cap	Rectangular with 100×170 cm with two rows of bar and the number of 10 with 3.2 cm diameter transverse bar of 1.27 cm diameter with 15 cm distances
Slabs	Concrete slab of 30 cm diameter
Girders	2IPE300
Braces	2UNP 160×50
Expansion joints	L 100×10

In this paper, four methods of linear and nonlinear static and dynamic analyses are used in order to study the structural behavior. Linear analyses are performed according to AASHTO HB-17 recommendations (2002). Definition of nonlinear hinges is based on FEMA-356 guidelines.

5. Seismic behavior of structure

5.1. Nonlinear static analysis (Pushover analysis)

Pushover analysis has been very popular during recent years for performance-based earthquake resistant design practice. Also, it should be noted that pushover analysis corresponds to a monotonically increasing lateral load which does not take into account strength deterioration due to cumulative damage. In the present study, Aly Dar Bridge is evaluated using nonlinear static method, for assessing the seismic behavior. Four different levels of seismic performance are defined in evaluating the seismic performance of the structure. These four performance levels from high to low levels include Operational Level (OL), Immediate Occupancy (IO), Life Safety (LS), and Collapse Prevention (CP), respectively. Estimating the demands at low performance levels (Life Safety and Collapse Prevention levels) requires the consideration of inelastic behavior of the structure. Force-deformation diagram for the concrete plastic hinges is depicted in Fig. 6.

Target displacements are calculated using equation

$$\Delta = \frac{0.25 F_{\mathrm{v}} S_{\mathrm{I}} T_{\mathrm{eff}}}{B} \tag{1}$$

For longitudinal and transverse directions the target displacement is calculated

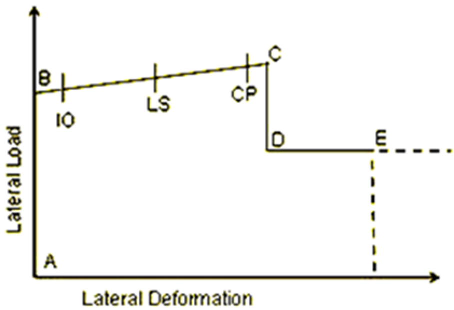

Fig. 4. Non-linear characteristics of concrete section (plastic hinges) in the bridge based on FEMA356

as follows

$$\Delta_{\text{longitudinal}} = 0.16\,\text{m},$$

$$\Delta_{\text{transverse}} = 0.65\,\text{m}.$$

In the above equations, Δ means the displacement, F_{v} is the vertical force, B denotes the damping ratio of the structure, S_{I} stands for the sectional area and T_{eff} represents the effective fundamental period.

Figures 7 and 8 show the plastic hinge formation due to traverse lateral and longitudinal lateral load.

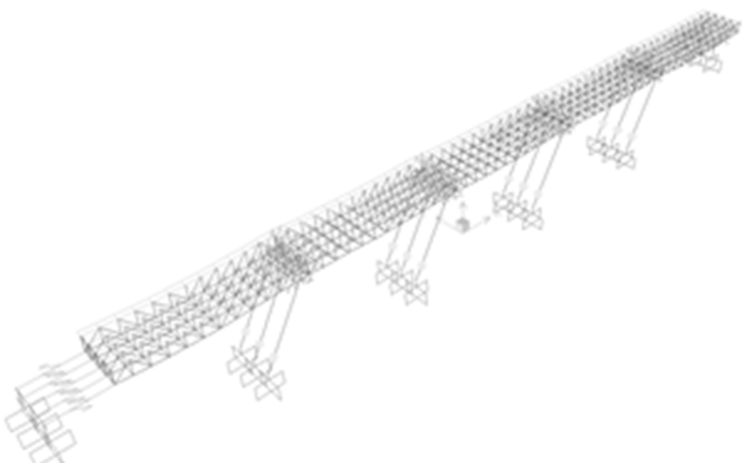

Fig. 5. Plastic hinge formation in target displacement due to longitudinal lateral load

According to the distribution of plastic hinges formation (Fig. 7) in the longitudinal directional, the plastic hinges (C (failure) level) are located in the bottom end of the columns. In contrast, in the transverse, the damages are occurred in the

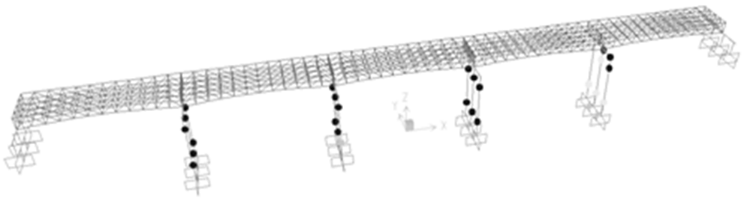

Fig. 6. Plastic hinge formation in target displacement due to transverse lateral load

columns connecting to the foundation and bent cap (Fig. 8).

5.2. Results of nonlinear dynamic analysis

To calculate nonlinear time history response of the bridge, the present paper considers a 3D model subjected to an ensemble of earthquake records and simulated in SAP2000. The maximum forces and dispacement of the elements are calculated. In this paper, three records of ground motion which are compatible with the soil type of the bridge site are used: Tabas (1979), El Centro (1940) and Northridge (1994). Table 4 presents the characteristics of the selected ground motions.

Table 4. Characteristics of strong ground motions

Row	Earthquake	Year	Depth (km)	Magnitude	PGA
1	El Centro	1940	15	7.1 (Ms)	0.31 g
2	Tabas	1979	10	7.8 (Ms)	0.81 g
3	Northridge	1994	9	6.7 (Mw)	0.55 g

By employing the comparative coefficients in records, the nonlinear time history response of the bridge is discussed. Figures 9 and 10 depict the plastic hinge formation due to the above-mentioned earthquakes. In addition, the displacements of the bridge deck are represented in Tables 5 and 6.

According to the distribution of plastic hinges formation (Figs. 9–10) the plastic hinges (E (damage) level) are located in the bottom end of the columns and connections to the foundation and bent cap.

5.3. Linear dynamic analysis results

According to the linear dynamic analyses, the displacement of the deck subjected to the longitudinal earthquake is equal to 17 cm (Tables 7 and 8). This value is smaller than the allowable seat length which is 91.8 cm, see eq. (2), based on the AASHTOO recommendations. In addition, the displacement of the deck in the transverse direction of the bridge under the longitudinal earthquake is 111 cm which

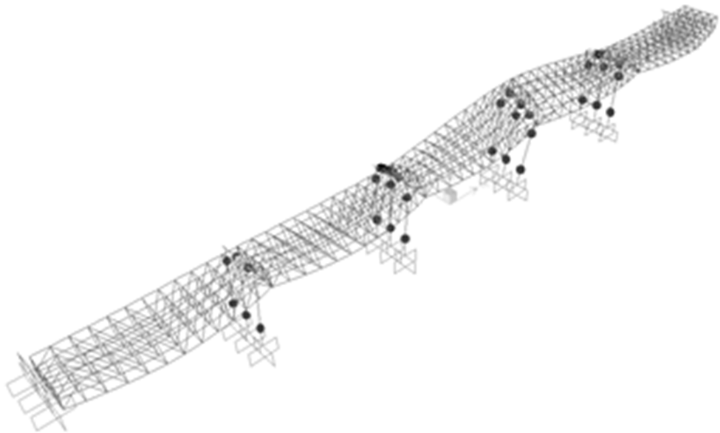

Fig. 7. Plastic hinges formation on El Centro earthquake

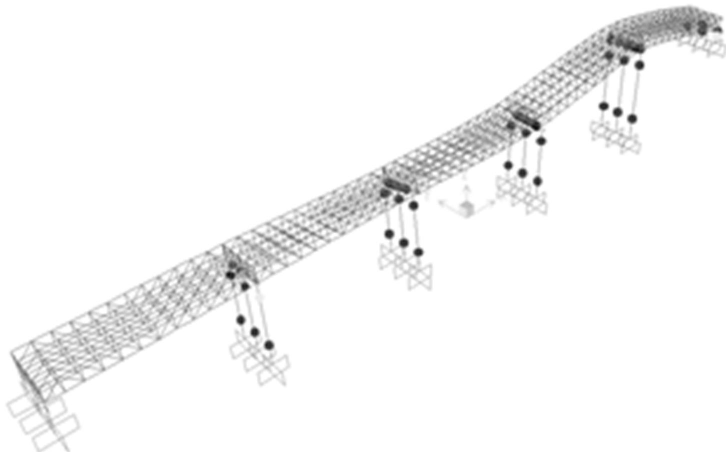

Fig. 8. Plastic hinges formation on Tabas earthquake

is smaller than the allowable seat length N according to AASHTO guidelines

$$N = 600 + 5L + 10H = 600 + 150 + 168 = 918 \; mm\,, \tag{2}$$

where L denotes the length and H is the height.

Table 5. Displacement on the pier due to nonlinear dynamic analysis

Output Case	Step Type	$U_1(x)$ (cm)	$U_2(y)$ (cm)
ELCENTRO	Max	20.0	80.6
ELCENTRO	Min	–18.5	–87.9
TABAS	Max	23.6	99.3
TABAS	Min	–23.7	–94.8
NORTHRIGE	Max	22.3	106.2
NORTHRIGE	Min	–21.6	–34.4

Table 6. Base reaction due to nonlinear dynamic analysis

Output Case	Step Type	Global F_X (Tonf)	Global F_Y (Tonf)
ELCENTRO	Max	3156	1114
ELCENTRO	Min	–2778	–1983
TABAS	Max	2732	1387
TABAS	Min	–2341	–1050
NORTHRIGE	Max	3637	1120
NORTHRIGE	Min	–2793	–811

Table 7. Displacement of deck in the linear dynamic analysis

Output Case	U_1 (cm)	U_2 (cm)
epy ($r = 1$)	0	111
epx ($r = 1$)	17	0

Table 8. Displacement and rotation on bent cap

Joint	No.	Output case	U_1 (cm)	U_2 (cm)	U_3 (cm)	R_1 (rad)	R_2 (rad)	R_3 (rad)
Node on bent cap	584	epy ($r = 1$)	–0.051	110.587	0.129	–7.013	0.000	–0.035
	584	epx ($r = 1$)	17.136	0.080	0.048	–0.005	0.001	0.000
Node on support	2069	epy ($r = 1$)	–0.026	3.583	0.129	–0.001	0.000	0.000
	2069	epx ($r = 1$)	15.202	0.001	0.007	0.000	0.016	0.000

5.4. Comparison of the results

In order to make a conclusion based on the results obtained from different adopted methods, the comparison results for the dynamic and static analyses are briefly presented in Table 9 and Figs. 11 and 12. Also, the base shear values are presented in Table 10.

Table 9. Comparative deck displacement based on different method of analysis

Output Case	Step Type	$U_1(x)$ (cm)	$U_2(y)$ (cm)
epy ($r = 1$)		0	111
epx ($r = 1$)		17	0
ELCENTRO	Max	20	80.6
ELCENTRO	Min	–18.5	–87.9
TABAS	Max	23.6	99.3
TABAS	Min	–23.7	–94.8
NORTHRIGE	Max	22.3	106.2
NORTHRIGE	Min	–21.6	–34.4

As can be seen in Table 9 and Figs. 11 and 12, according to the mode shape, the displacements in longitudinal direction of bridge are very smaller than the in the transverse direction.

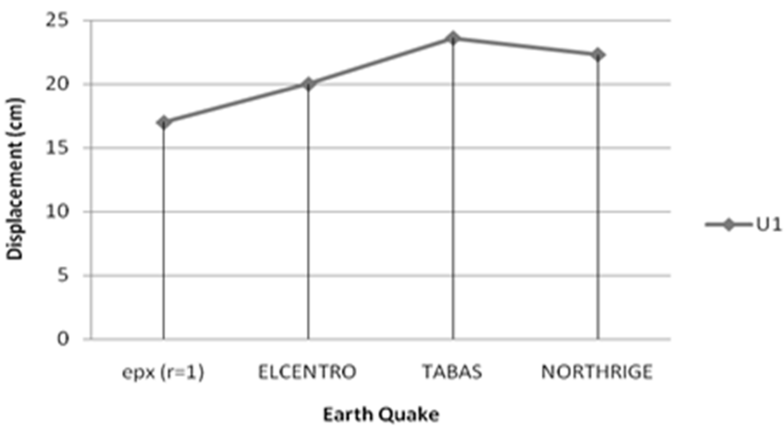

Fig. 9. Displacement of bridge in longitudinal direction

6. Conclusion

After the qualitative study of composite concrete-steel deck truss bridge, this paper deals with the seismic behavior of the studied bridge based on AASHTO guidelines. The seismic behavior of the bridge has been evaluated using linear and nonlinear static analyses as well as linear and nonlinear dynamic analyses.

The qualitative evaluation of the results indicates that surface cracks in some areas of the concrete have been formed and the cover of concrete is damaged. Some repairs are needed in the surface of the areas such as columns and abutments. It should be noted that due to poor maintenance bracing connections were not in good condition. As a result, they need retrofitting. Other parts of the bridge were healthy.

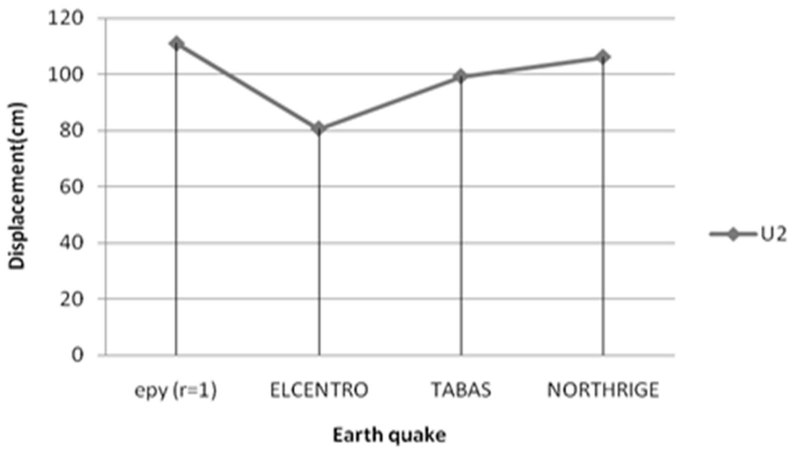

Fig. 10. Displacement of bridge in transverse direction

Table 10. The comparison of base shear in different analyses (base reactions on nonlinear time history analyze)

Output Case	Step Type	Global F_X (Tonf)	Global F_Y (Tonf)
ELCENTRO	Max	3156	1114
ELCENTRO	Min	–2778	–1983
TABAS	Max	2732	1387
TABAS	Min	–2341	–1050
NORTHRIGE	Max	3637	1120
NORTHRIGE	Min	–2793	–811
epy (r=1)	LinStatic	0	–1143
epx (r=1)	LinStatic	–2948	0

1. The results of superstructure analysis indicate that the bottom chords and truss diagonal elements near the truss supports need retrofitting.

2. The substructure analysis represents that in addition to the weakness observed in some column and bent cap hinges, as one of the most crucial weaknesses; this bridge needs the increase of the seat length of deck in order to provide the requirements of deck displacement. Furthermore, regarding the plastic hinge distribution, it can be demonstrated that in order to prevent the formation of plastic hinges in substructure, the columns need retrofitting and strengthening at their connections

to the foundation.

3. From another perspective, the result of comparing nonlinear dynamic and static analyses emphasizes on the reality that in dynamic analysis, the bridges especially the columns are most vulnerable to plastic hinge formation.

4. In comparison of relative displacement values and base shear values, the average value of nonlinear dynamic analyses is more than the corresponding value in nonlinear static analyses.

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