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Seismic Vulnerability assessment of concrete railway bridge using nonlinear analyses

Araliya Mosleh[*,a], Hamed Hamidi[b], Ahmad Nicknam[c], José Jara[d], Humberto Varum[e]

[a]Department of Civil Eng., Faculty of Engineering, University of Aveiro, 3810-193 Aveiro, Portugal.
[b]Department of Civil Eng., Babol Noshirvani University of Technology, Iran.
[c]Department of Civil Eng., Iran University of Science&Technology, P.O. Box 16765-163, Tehran, Iran.
[d]Department of Civil Eng., Faculty of Engineering, Universidad Michoacana de San Nicolas de Hidalgo, Morelia, Mexico.
[e]CONSTRUCT-LESE, Department of Civil Eng., Faculty of Engineering, University of Porto, 4200-465 Porto, Portugal.

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Abstract

Bridges, as an essential part of transportation system, play a special role in economy, politic and military all around the world. Serviceability of bridges is of high importance especially in emergencies, helping injured people and proportioning transportation after earthquakes. Earthquake regulation codes, normally suggest linear methods to conduct the analysis, and nonlinear analysis is barely used by practicing engineers. In this paper, we analyze a typical concrete bridge located in Iran and use both nonlinear dynamic and static procedures at two hazard levels. This study compares both methodologies, analyze the results and presents some recommendations to reduce the seismic vulnerability.

Keywords: Concrete bridges; Seismic vulnerability; Nonlinear dynamic analysis; Nonlinear static analysis; Retrofitting.

1. Introduction

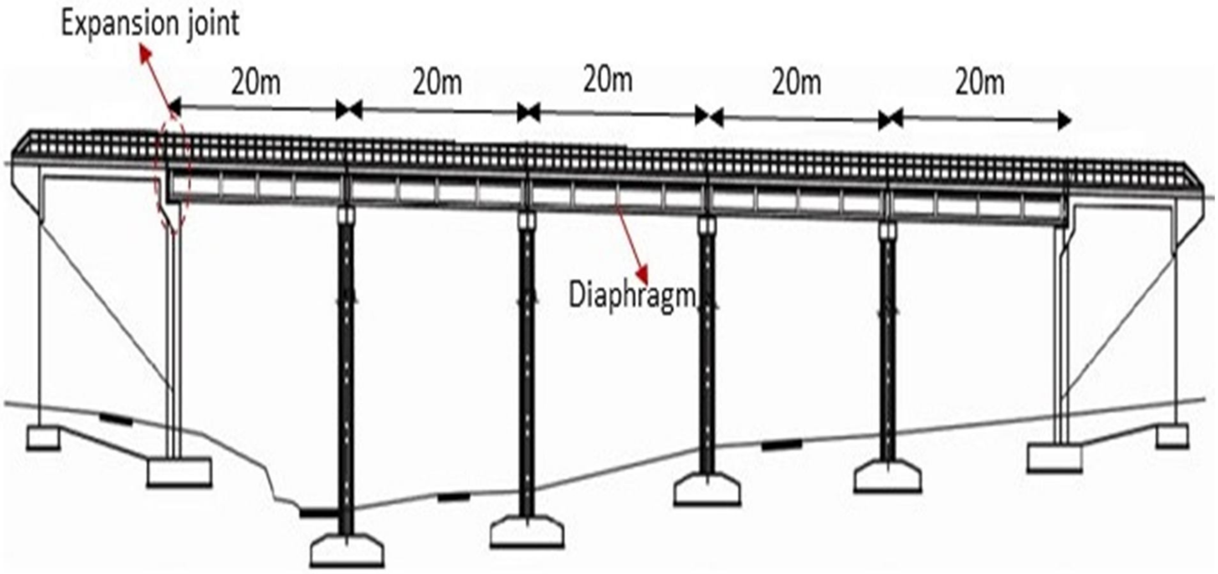
The damages produced by recent seismic events all over the world have shown that steel-concrete composite bridge structures are very sensitive to earthquake loading. Current seismic design codes for bridges allow the formation of plastic hinges at the base of the piers during severe shaking in order to reduce the seismic forces. Other structural members such as deck, bearing devices, abutments and foundations should be designed to remain elastic in order to avoid brittle failure. Up to now the most important earthquakes around the world, especially in Iran have left severe damage in bridges and other important structures [1-10]. The analysis of bridges damages during earthquakes is instructive. It is also critical to assess the level of vulnerability of the bridge due to certain ground motions in order to evaluate the seismic performance of highway bridges [11-13]. To ensure a bridge performance and reduce the amount of damage, a proper design is highly recommended. Current codes suggest recommendations and guidelines to avoid severe damages during earthquakes. In recent years, several researchers have focused on evaluating the seismic vulnerability of steel and concrete bridges. Billah and Alam [14] studied reinforced concrete (RC) bridge piers by considering incremental dynamic analysis and seismic hazard scenarios. Avsar et al. [15] evaluated the qualitative damage assessment criteria for ordinary highway bridges by considering the critical bridge components in terms of proper engineering demand parameters. Nielson and DesRoches [16] determined the vulnerability of steel and concrete girder bridges based on nonlinear analyses. Jara et al. [17] evaluated a parametric study to assess the expected demands of seismically deficient medium length highway bridges retrofitted with RC jacketing aimed at determining the best jacket parameters. Since Iran has 5090 km of railways and more than 10,000 bridges in its road network, it is important to study the vulnerability of its bridges [6]. This research, investigates the seismic behavior and vulnerability of a well-known concrete bridge in Iran, using nonlinear static procedure (push over) and nonlinear dynamic analysis.

2. Description of the bridges

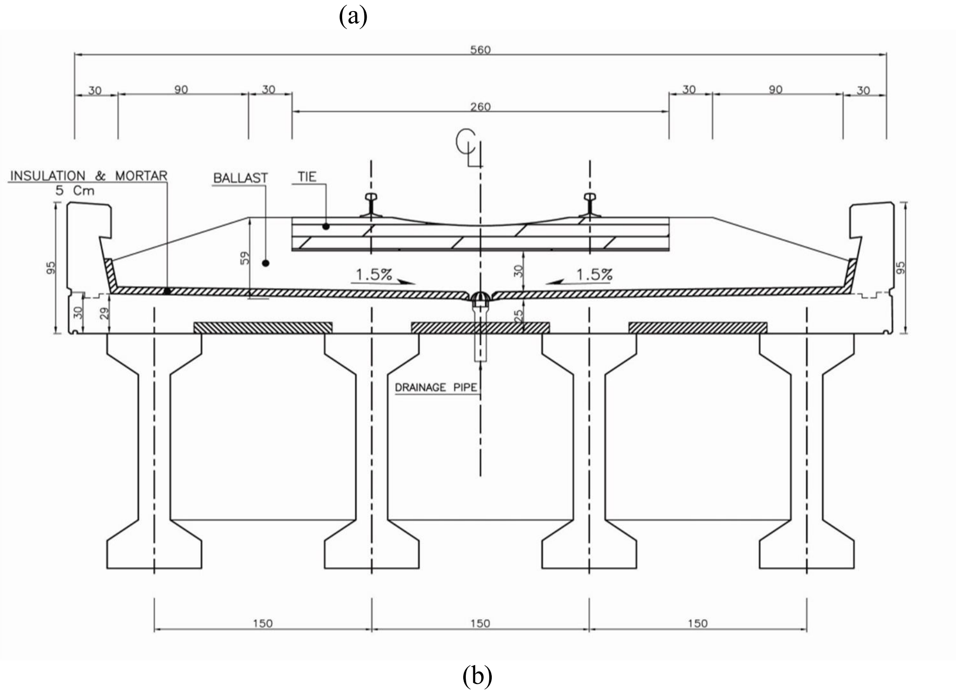
A commonly bridge typology in Iran is the Multi-Span-Simply-Supported (MSSS) system. In a MSSS bridge each span is simply supported with separation gaps between the adjacent spans and between the end spans and the abutments. To evaluate the vulnerability of this type of bridges by using nonlinear static and dynamic analyses, this study selected a real concrete bridge as a representative of common bridges located in south part of Iran with the following characteristics:

* Corresponding Author: a_mmosleh@yahoo.com

- The bridge length and width are 100m and 5.6m respectively. It is a one traffic lane bridge with five 20-m long spans.
- The bridge's superstructure has four I-shaped prefabricated concrete girders with 19.8m length and 1.7m height and 0.25-m thick slab.
- The wall piers have variable heights (12.75, 14, 16.75 and 18m). Pier walls and abutments have surface mat footings.
- Each longitudinal beam rests on a neoprene bearing with dimensions of 0.40×0.30×0.052m and the decks are located on pile-caps with dimensions of 1.2×1.9m.
- The bridge piers are rectangular cross section with dimensions 6.8×1.2m.
- The bridge has four wall piers, two of them with dimensions of 10×7.2×1.7m, and the others have dimensions of 10×8×1.7m.
- One abutment is 15 m height with section dimensions of 5.1 × 7 × 10m. The other one is 12 meter height with section dimensions of 5.1 × 6 × 10m).
- Tables (1) and (2) display material properties. Figure 1 shows a longitudinal and a side view of the bridge.



(a)



(b)

Figure 1: (a): Longitudinal view, (b): Side view of superstructure

Table (1): Concrete material properties

Component	Cement Type	F_c (MPa)	E_c (MPa)
Beam	Portland-type I	34	29559
Deck	Portland-type I	30	27366
Foundation & Abetment	Portland-type VI	30	27366

Table (2): steel material properties

E_s (MPa)	F_y (MPa)	Bar Type
210000	400	AIII

3. Analytical modeling

The analytical model of the bridge was built using SAP2000 [18]. Figure 2 shows a 3D view of the model. The numerical model of the bridge consists in a three dimensional structure created with the software SAP2000. The structural components used are:

FRAME element: Beam–column element is employed to model decks, columns and cap beams. To evaluate the nonlinear behavior of the columns, both column ends have concentric plastic hinges capable of considering P-M-M interaction curves for steel and reinforced concrete column section. This study assumes FEMA 356 recommendations [19] for modeling parameters and numerical acceptance criteria for nonlinear procedures.

Figure 2: Three dimensional view of studied bridge

LINK element: Elastomer bearings are modeled using simple connection element. The link element is employed to model the gap and impact between adjacent spans and between end-span and the approach slab. Expansion bearings are modeled as roller elements; however, as determined by Mander et al. [20] they do possess some stiffness and friction resistance.

Considering that the bridge has massive abutment, for simplicity they are modeled in both sides as rollers and bearings joints. An important element that must be considered in deck modeling is the transverse diaphragms. These beams prevent girders from individual movements and provide required torsion stiffness of the decks. If transverse diaphragms are not considered, then the dominant vibration mode is torsion-mode. Concrete beams are located on elastomeric support and connected with pile-cap. At first stage, the horizontal bearing stiffness is equal to force-displacement curve of neoprene. If the force continues to be increasing, eventually the bearing fails and the only resistant force will be the friction between pieces.The piers were modeled as equivalent-columns with plastic hinges assigned according to FEMA 356 [19]. Because of the high in-plane deck-girder stiffness, the lateral bridge stiffness is controlled by substructure elements.

4. Loading

4- 1- Gravitational loading

The dead load on the bridge depends on:

W_{DL} = beam+ slab+ concrete ramp+ diaphragm+ balustrade+ pile-cap+ asphalt =354 kg/m^2

Live load has been considered as uniform load according to the code No. 139 [21]. SAP2000 automatically determines dead load values using the specific weights of the elements. Initially, a static analysis was conducted to determine deck deflections, column axial loads, total weights of the structure and shear force distribution.

4- 2- Seismic Loading

According to the seismic code for bridge design [21], seismic lateral force can be determined as function of the bridge fundamental period and a response spectra. The lateral earthquake force in the deck can be calculated as following equation:

F=CW, C = ABI/R

where, W: weight of deck + x% live load, F: deck force applied in mass center, C: earthquake coefficient, A: design acceleration, I: bridge importance factor, B: response spectrum coefficient and R: behavior factor.

The fundamental vibration periods of the bridge in the longitudinal direction (x) and the transverse direction (y) are:

T_x=1.08 (s), T_y=0.4 (s)

B_x=2.0, B_y=1.03

C_x=0.108, C_y=0.28

For earthquake load condition, live loads can be neglected if the live load amplitude is less than half of the dead load. Otherwise, two-third of deck total load (dead and live) should be used in calculation. In this research the live load of the deck has not been considered.

Elements and components shall be evaluated for forces and deformations associated with 100% of the design forces in the X direction plus the forces and deformations associated with 30% of the design forces in the perpendicular horizontal Y direction and vice versa [19].

The minimum number of modes for modal analysis is 3-times the number of spans and it is not required to use more than 25 [22]. This study includes the first 15 bridge modes.

Elements and components of structures shall be designed or verified for $p-\Delta$ effects, defined as the combined effects of gravity loads acting in conjunction with lateral drifts due to seismic forces [19]. This research includes $p-\Delta$ effects in the analysis.

The load combinations used in the analyses are:

Gravity Upper bound: Q_G=1.1(Q_D+Q_{SI}) +0.5Q_L

Gravity Lower bound: Q_G=0.9(Q_D+Q_{SI})

and Q= Q_G±Q_E,

Where: Q_D=dead load, Q_L=live load, Q_{SL}=weight of upper structure, Q_E=seismic load [23].

5. Primary investigation and Rehabilitation objective

According to AASHTO code [11], the girder depth should be at least:

$0.07 \times L = 0.07 \times 20 = 1.4 < 1.7 \rightarrow ok$ (The girder depth is 1.7m)

According to Caltrans [24] and AASHTO codes [22] the abutment seating width is appropriate.

The mentioned bridge is considered as a very important structure and the rehabilitation objective according to AASHTO [22], FHWA [23] and CALTRANS [24] has been selected as “optimal”. It means that the life safety (LS) of residents should be provided under hazard level1, and the bridge stability should remain and prevent collapse (CP) under hazard level2.

6. Nonlinear analysis

6- 1- Hazard earthquake level

This study uses the results of a probabilistic seismic hazard analysis conducted in the area of interest for two hazard levels (HL1 and HL2) [25]. Hazard level 1 is associated with 10% exceedance probability of earthquake in 50 years (475 years return period). Hazard level 2 is determined based on 2% exceedance probability of earthquake in 50 years (2475 years return period). Figure (3) and Table (3) displays the design spectra and spectra parameters, respectively.

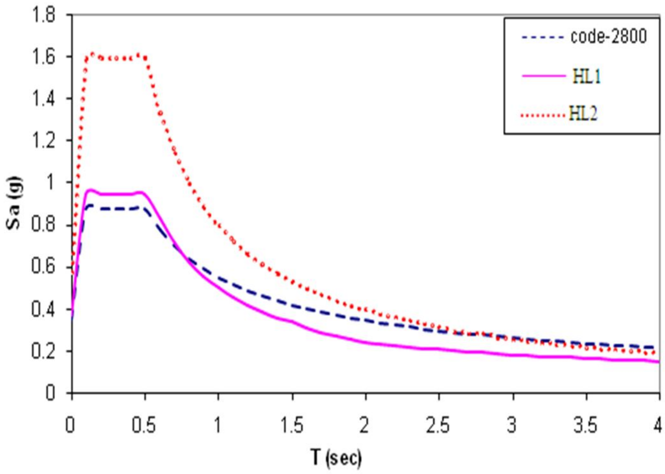


Figure 3: Design spectra versus the ones obtained via PSHA [25]

Table 3: Spectral parameters

Parameter	Ca	Cv
Hazard Level 2	0.33	0.557
Hazard Level 1	0.211	0.36

6- 2- Nonlinear static procedure (Pushover analysis)

Performance-based design philosophy is a powerful seismic performance evaluation tool, and nonlinear static procedures have increasingly employed. This procedure is now widely used in engineering practice to predict seismic demands in building structures [26]. Some new methods have been developed such as Modal Pushover Analysis (MPA) [27] and Adaptive Pushover Procedure (APP) [28]. According to FEMA 356, the mathematical based model of structures with rigid diaphragms should undergo monotonically increasing lateral forces or displacements until either a target displacement is reached or the structure collapses. The nonlinear static (pushover) analysis method (NSP), developed here use "line elements" approach, and are based on the degree of refinement in representing the plastic yielding effects. The elasto-plastic behavior is modeled with two possibilities: (1) distributed plasticity model, modeled accounting for spread-of-plasticity effects in sections and along the beam-column element and (2) plastic hinge, when inelastic behavior is concentrated at plastic hinge locations. Both local (P-δ) and global (P-Δ) nonlinear geometrical effects are considered in analysis. Pushover analysis provides the base shear force versus top displacement curve of the structure, usually called capacity curve. To evaluate whether a structure is adequate to sustain a certain level of seismic loads, its capacity has to be compared with the requirements corresponding to a scenario event (demand). The aforementioned comparison can be based on force or displacement. In pushover analyses, both the force distribution and target displacement are based on very restrictive assumptions, i.e. a time-independent displacement shape. Thus, it is in principle inaccurate for structures where higher mode effects are significant, and it may not detect the structural weaknesses that may be generated when the structure´s dynamic characteristics change after the formation of the first local plastic mechanism. One practical possibility to partly overcome the limitations imposed by pushover analysis is to assume two or three different displacements shapes (load patterns), and to envelope the results, or using the adaptive force distribution that attempt to follow more closely the time-variant distributions of inertia forces. Uniform triangular distribution has been used and it has shown that triangular distribution is more critical. The minimum number of modes included in this study corresponds to an effective mass participation of at least 90%.

6- 3- Nonlinear Dynamic Procedure

Nonlinear dynamic analysis calculates structure response based on nonlinear behavior of materials and non-geometric behavior of the structure. This method assumes that the stiffness and damping matrices, could change from one step into another, but remain fixed each time step. The bridge response under earthquake acceleration is calculated by numerical methods for each time step.

Nonlinear dynamic analysis (NDA) is the most accurate method used in structural analysis. Indeed, the main target in this method is solving the movement's dynamic equilibrium differential equation.

6- 4- Selection of Accelerograms and the Scaling Process

This research selected seven accelerograms for nonlinear dynamic analysis and average response values were determined (Table 4). The accelerograms used in nonlinear dynamic analysis should be compatible with the site. These characteristics include the PGA, frequency contents, duration of severe movements and design spectrum [29].

Spectrum scaling method has been used to achieve compatible accelerograms. This method normalizes to 1g the PGA of each accelerogram and determines the response spectrum for 5% damping. The area under this spectrum between periods of 0.2s and 1.5s and the area under site spectrum between the mentioned periods are obtained. The accelerogram is scaled with a factor obtained from the ratio of the site spectrum area to the accelerogram spectrum area. Using this procedure, the energy of accelerograms is harmonized with site design spectrum [29].

Table (4): Selected earthquakes and corresponded scale factors

Earthquake	Year	PGA (g)	Scale Factor	
			HL1	HL2
KOBE	1995	0.82	1.11	1.15
NORTHRIDGE	1994	0.51	1.13	1.17
LOMA PRIETA	1989	0.48	1.29	1.33
CAPEMENDOCINO	1992	0.39	1.3	1.34
KOCAELI, TURKEY	1999	0.38	1.41	1.45
N. PALM SPRINGS	1985	0.59	1.45	1.49
SUPERSTITI HILLS	1987	0.38	1.11	1.15

6- 5- Hinge types

The nonlinear behavior of the elements is defined based on FEMA356 [19]. PMM hinges were employed for columns and M3 hinges for beams. ATC-40 and FEMA-356 documents have developed modeling procedures, acceptance criteria and analysis procedures for pushover analysis. These documents define force-deformation criteria for hinges used in pushover analysis. As shown in Figure 4, five points labeled A, B, C, D, and E are used to define the force deflection behavior of the hinge and three points labeled IO, LS and CP are used to define the acceptance criteria for the hinge. (IO, LS and CP stand for Immediate Occupancy, Life Safety and Collapse Prevention respectively.) The values assigned to each of these points vary depending on the type of member as well as many other parameters defined in theATC-40 and FEMA-356 documents.

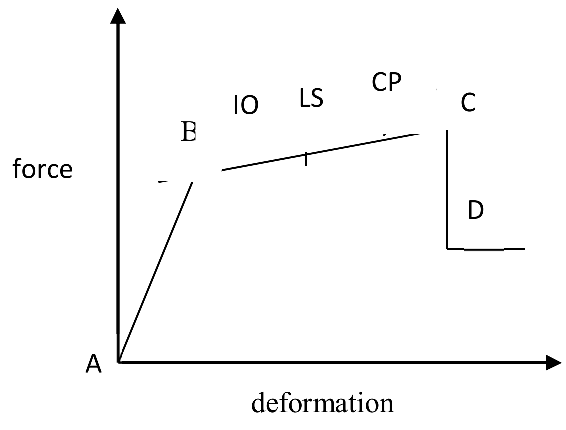
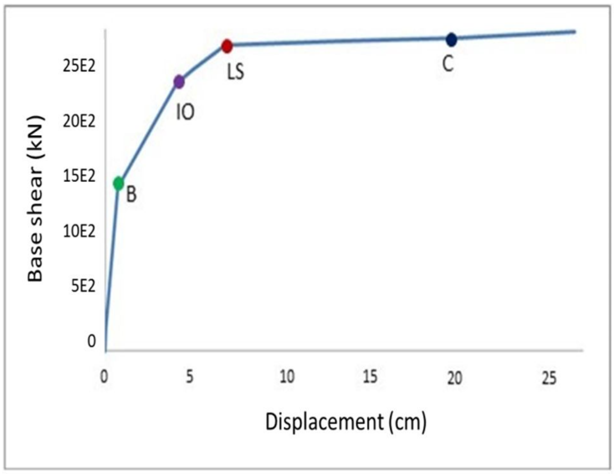


Figure 4: Force-Deformation for Pushover Hinge

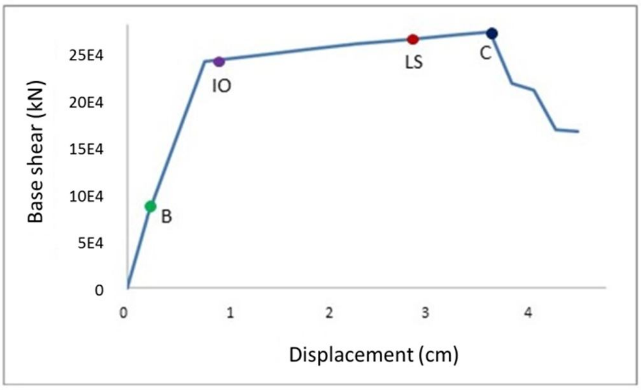
7. Results

7- 1- Nonlinear static results

The bridge was analyzed for two hazard levels (HL1 and HL2) and two horizontal directions (X and Y). Results show that the bridge is more vulnerable in the longitudinal direction. Figure 5 display hinge results and the capacity spectrum of the bridge in the longitudinal and transverse directions for HL1 hazard level, respectively. This figure shows the capacity curve in terms of top displacement versus base shear and it represents the envelope of the structural behavior under inelastic incursions. The capacity curve corresponds to a top node of a middle column in the longitudinal and transverse directions. The push-over curve for the transverse direction shows a higher initial stiffness and strength than the curve in the longitudinal direction the bridge. By applying a uniform triangle load pattern, the first incursion in the nonlinear behavior occurred at base shear 200.9 kN and 77817 kN for the longitudinal and transverse direction, respectively. At this stage, the displacements were 0.03 and 0.2 cm, respectively. In the transverse direction the bridge remains in linear region before a displacement of 0.2 cm. As mentioned before the maximum shear capacity in the transverse direction was of 172099 kN with a displacement demand of 4.51 cm. These values in the longitudinal direction were 2972 kN and 27.34 cm, respectively. These differences are understandable because the very stiff piers in the transverse direction.



(a)



(b)

Figure (5): base shear – displacement in (a): the longitudinal direction, (b): transverse direction

7- 2- Nonlinear dynamic results

The nonlinear dynamic analyses of the bridge subjected to two horizontal components of seven accelerograms using a direct integration method were conducted. Figure 6 presents the displacement time-history of the bridge deck subjected to the Loma Prieta seismic record. Table 5 shows the results of this analysis at two hazard levels. None of the elements fulfills the acceptance criteria and in many cases an instability problem is presented. The bridge is stiffer in the transverse direction and because of that, all damage scenarios are presented in the longitudinal direction. According to Table 5, Palm- Spring earthquake is the only seismic record the bridge can sustain without reaching the collapse limit state under Hazard level 2. Under HS1, the bridge would present damages without reaching collapse for Northridge, Kobe and Cape Mendocino seismic records with PGAs 0.51, 0.82 and 0.39 g, respectively. It is notable that the bridge passed the acceptance criteria when subjected to accelerograms with different PGAs. This means the earthquake energy, which is the area under the elastic response spectrum, between the boundary periods defined as acceleration spectrum intensity (ASI), is very important. The comparison of two ground motion, namely Palm spring with PGA=0.59 g and Northridge with PGA=0.51 g, shows that the first one produces less damage than the other with smaller PGA.

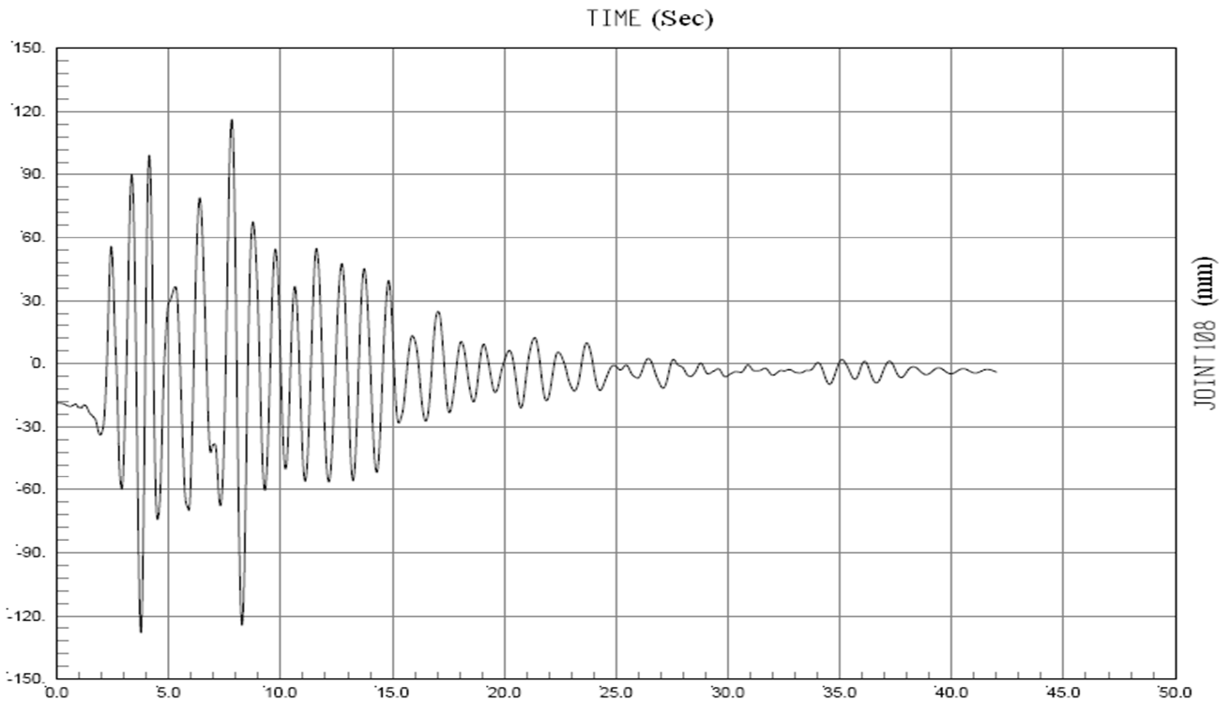


Figure (6): Displacement time-history at the control point of the bridge (Loma Prieta EQ.)

Table (5): Bridge demands in dynamic nonlinear analyses

Earthquake	Hazard Level 1	Hazard Level 2
SUPERSTITION HILLS	100[1]	INS[2]
PALM SPRINGS	100	100
NORTHRIDGE	INS	INS
LOMA PRIETA	100	INS
KOCAELI	100	INS
KOBE	INS[1]	INS
CAPE MENDOCINO	INS	INS

[2]x: percent of elements that do not satisfy acceptance criteria
[1]ins: instability occurred

8. Conclusions

This research analyzes one existing concrete bridge designed with old codes. Push-over and dynamic time history analyses in the longitudinal and transverse directions were conducted. The bridge was analyzed for two different seismic hazard levels. The conclusions of the study are summarized as follows:

1. The results of the static and the dynamic nonlinear analyses have shown the vulnerability of the bridge designed with previous codes.
2. The pushover results show that the bridge is stiffer in the transverse direction than in the longitudinal direction. The displacement capacity in the longitudinal direction reach 27.34 cm and is much longer than in the transverse direction with value equal to 4.51 cm. However, the base shear capacity in the transverse direction is substantially greater than that of the longitudinal direction with values of 172099 and 2972 kN, respectively.
3. The bridge was subjected to accelerograms in the longitudinal and transverse directions to evaluate the acceptance criteria. In the transverse direction, the bridge is stiffer and shear damage is expected. This study considers only flexural damage. Therefore, the influence of shear hinges could improve the results of this research.
4. The bridge passed the acceptance criteria (LS) under HS1 for Northridge, Kobe and Cape Mendocino with PGAs 0.51, 0.82 and 0.39 g, respectively. This result makes relevant to assess the earthquake energy between the boundary periods, defined as acceleration spectrum intensity (ASI).
5. Considering the nonlinear dynamic analysis, the bridge is highly vulnerable in the longitudinal direction.
6. Those bridges designed with old seismic codes are vulnerable to strong motions and should be candidates to be retrofitted.

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