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The Performance Assessment of a Multi-Span, Box Girder Reinforced Concrete Bridge with and without Seismic Isolation

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Abstract

This paper investigates earthquake performance of a multi-span, box girder R/C bridge with and without isolation. Bridges and viaducts are important structures since they must be functional immediately and remain stable after an earthquake. For this reason, seismic isolations can be used to decrease the effects of earthquake loads on bridges. Seismic isolation on a bridge elongates the period of structure, thus the inertia and the earthquake forces are decreased without a significant change in the stiffness of the structure. Among various types of seismic isolators used in engineering practice, lead rubber bearing (LRB) was used in this study. Performance assessment was applied to the un-isolated and isolated bridge to make some comparisons, and the benefits of isolation were discussed. Linear elastic method, nonlinear static method (pushover analysis) and nonlinear time history analysis, which are defined in Turkish Earthquake Code (TEC) 2007, were used for performance assessment.

Keywords: earthquake, bridge, performance assessment, seismic isolation, damage.

1 Introduction

Basic concept of earthquake resistant design is that structures must remain stable after a destructive earthquake. Seismic energy produced by destructive earthquakes causes large displacement and deformations in structures. Structural damage occurs because of hysteretic energy and inelastic deformations [1]. However, some structures must remain stable and also suffer limited damage in order to manage aid organization and help care-seeking people immediately after a destructive earthquake, such as bridges. Seismic isolators are helpful in decreasing the effects of earthquake loads on structures, especially on bridges. It can be used for dissipating the seismic energy and elongating the period of a bridge to decrease the earthquake

loads and inertia force without a significant change in the stiffness of the structure. A bridge is typically isolated immediately below the superstructure and the purpose of the isolation is to protect the elements below the isolators by reducing the inertia loads transmitted from the superstructure [2].There is no imposed upper limit on damping provided by the isolation system since it is not aimed to reduce floor accelerations which is common for buildings.
In this study, a multi-span and box girder R/C bridge was investigated. It was designed as both un-isolated and isolated. Lead Rubber Bearing (LRB) was used as isolator. Performance assessment was applied to the bridges, and the responses of the un-isolated and isolated bridges were compared. The necessity of seismic isolation was investigated by performance assessment.

Performance assessment was conducted based on ***linear elastic method, nonlinear static method (pushover analysis)*** and ***nonlinear time history analysis*** which are defined in Turkish Earthquake Code (TEC) 2007 [3]. According to linear elastic method, effect/capacity ratio (r) of columns is the damage measure. For a section, **r** is determined by dividing the moment calculated under seismic load by taking **R** = 1 to over moment capacity. [3]. Pushover analyses were performed for both transverse and longitudinal directions. Lateral load was distributed proportional to fundamental mode and mass. Three ground motion records were used for nonlinear time history analyses. Ground motion records were scaled to the design spectrum to provide the earthquake with a 2% probability of exceedance in 50 years. Both bridges (isolated and un-isolated) were modelled by using SAP2000 [4]. The deformation and strength capacities of plastic hinges were determined by using XTRACT software program [5]. The seismic isolation was designed according to AASHTO [6].

2 Properties of the Bridge

2.1 Geometric and Material Properties

The bridge has six 30.10 m long spans, so total length of the bridge is 180.60 m. The width of the deck is 30.90 m and the height of the piers is 8.60 m. The main girder is box shaped and the section of the piers is solid polygon section. The longitudinal reinforcements of the columns are 62Φ26 and the horizontal reinforcement is Φ12/15 cm. The characteristic compressive strength of concrete is 40 MPa (C40) and the characteristic yield strength of reinforcement is 420 MPa (S420). Schematic elevation, 3D view of the bridge, and transverse section of the bridge and cross-section of columns were given in Figure 1-3, respectively.

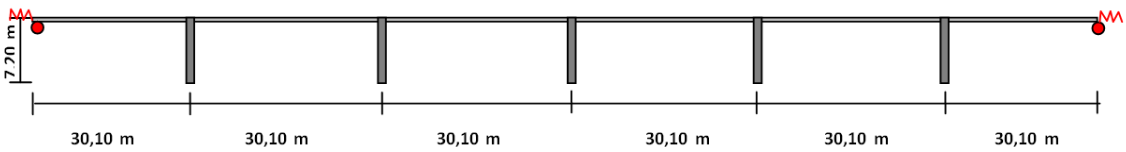


Figure 1: Schematic elevation of the bridge

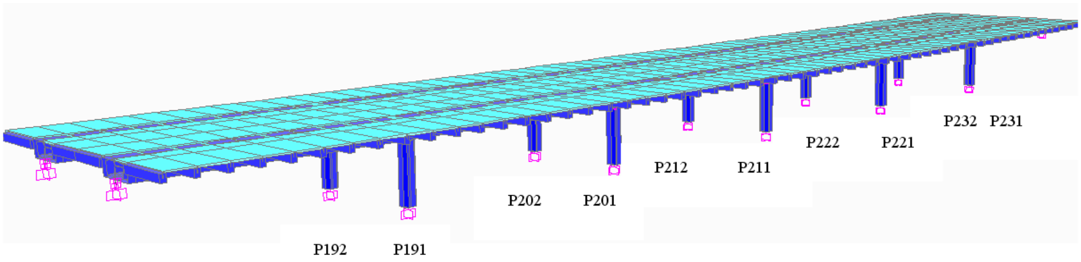


Figure 2: 3D view of the bridge

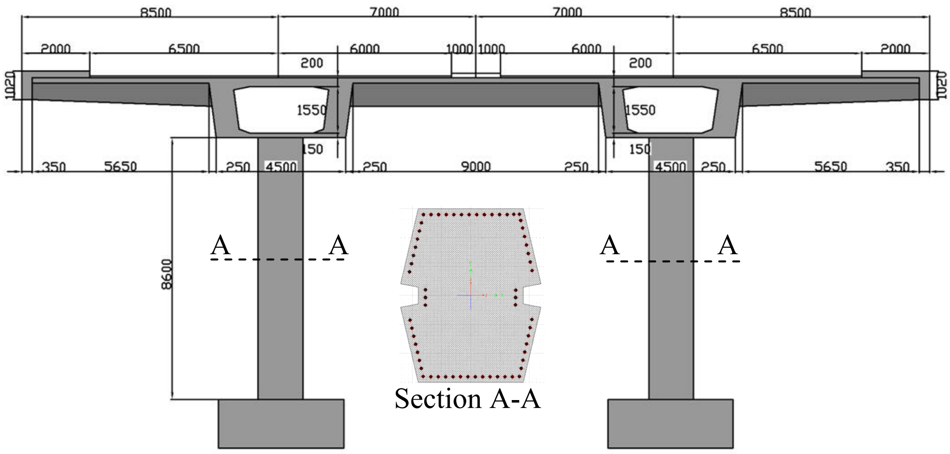
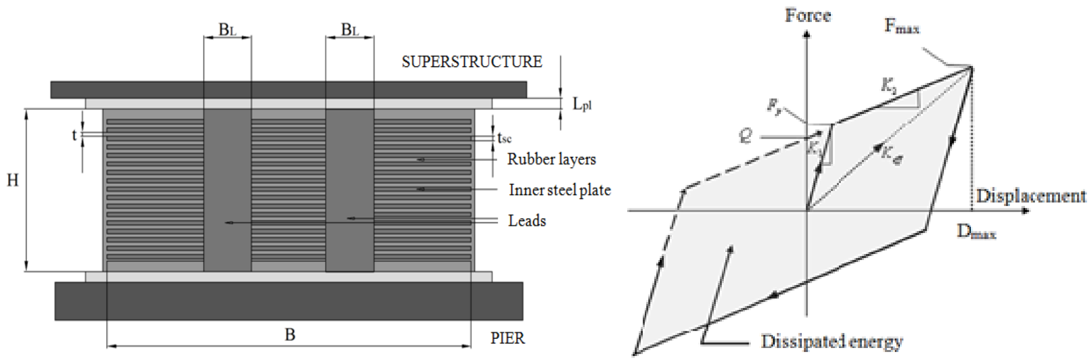


Figure 3: Transverse section of the bridge and cross-section of columns

2.2 Seismic Isolation

The elastomeric LRB generally consists of rubber and steel layers between two fixing steel plates which are located at the top and bottom of the bearing. Elastomeric material provides isolation with lateral deformation, inner and outer steel plates provide the vertical load capacity of bearing, while the lead core provides the energy dissipation or damping component and stiffness [7, 8, 9].
B is the width of bearing (970 mm), B_L is the diameter of lead core (125 mm), L_{pl} is the height of outer steel plate (50 mm), t_{sc} is the height of inner steel plate (10 mm), t is the height of rubber layer (10 mm), H is the height of bearing (335 mm). Cross-section of LRB and geometric parameters of the LRB are shown in Figure 5 (a).
The LRB was designed according to AASHTO [6]. Hysteretic model of LRB is bilinear [10] . The model and its parameter are shown in Figure 5 (b). K_1 is the elastic stiffness, K_2 is the post-yield stiffness, K_{eff} is the effective stiffness, F_y is the yield force, Q is the zero-displacement force-intercept, D_{max} is the maximum displacement and F_{max} is the maximum design force of the isolator.



(a) (b)

Figure 5: (a) Lead Rubber Bearing (LRB), (b) Bilinear model of LRB

3 Modelling

The bridges were modelled using the software program SAP2000 Nonlinear [4]. Superstructure and piers were modelled as frame elements and piers assumed as fixed at base. The connection of superstructure and piers of the bridge was modelled as fixed and isolated. So, two bridges were used for the study, isolated and un-isolated. The isolation bearings were modelled as nonlinear link elements. Plastic hinges were assigned at the bottom and top ends of the piers to model the nonlinear properties of the piers. It is recommended in AASHTO (1999) [6] that the considered number of modes is three times the number of spans for modal analysis and the upper limit is 25. In this study, 18 modes are considered since the modelled bridge has 6 spans.

4 Analysis

Performance assessments of the un-isolated and isolated bridges were conducted in the study. The effect of isolation on the seismic response of the bridge was investigated and importance of using seismic isolation was discussed. There are three methods for performance assessment in TEC 2007. These are ***linear elastic method (LEM), nonlinear static method (NSM)*** and ***nonlinear time history analysis (NTA)***. Performance assessment is applied according to section damages. Damage is defined according to internal forces and deformations for linear elastic method and nonlinear methods, respectively. Target performance level of bridges is life safety for an earthquake with a 2% probability of exeedance in 50 years [3].

4.1 LEM

According to **LEM**, effect/capacity ratio (r) of columns is the damage measure. For a section, **r** is determined by dividing the moment calculated under seismic load by

taking **R** = 1 to over moment capacity. **R** is strength reduction factor. While calculating the effect / capacity ratio, the direction of the applied earthquake shall be taken into consideration. Over moment capacity of section is the difference between bending moment capacity of the section and moment effect calculated on the section under gravity loads [3].
Limit values for damage measures (r_s) are defined according to internal forces for linear elastic method, and these values are defined based on different axial loads and design shear forces.

4.2 Nonlinear Methods

Performance assessment was conducted by using plastic deformation of sections. The damage measure is ***strain*** and performance level considered in this study has an assumed limit value of damage measure [3]. Earthquake with a 2% probability of exeedance in 50 years was considered in the present study.
The damage of structural elements after an earthquake was determined by using nonlinear deformations defined by plastic hinges. *Concentrated plasticity* was assumed for plastic hinges. Plastic hinge length is given as,

$$L_p = 0.08H + 0.022 f_y d_b \geq 0.044 f_y d_b \tag{1}$$

in which L_p is the plastic hinge length, H is the height of column, f_y is the characteristic yield strength of reinforcement and d_b is the diameter of reinforcement.

4.2.1 NSM

Seismic shear force and top displacement relation for horizontal loads is determined and that relation is plotted as a curve which is called "*pushover curve*". The bridge deck cannot be considered rigid against displacement in the transverse direction [12]. Thus, it is important for the pushover analysis selecting the control node where displacements are monitored. Top of the P211 column, given in Figure 4, was considered as the control node. Lateral load was applied to the top ends of piers proportional to the values calculated by multiplying fundamental mode amplitude and mass. Load pattern is independent of probable hinges occurring during on pushover analysis process.
Pushover curve is transformed to "*Capacity Diagram*" and "*Standard Spectrum*" which is T against S(T) is transformed to "*Demand Spectrum*" which is S_d against S_{ae}. The intersection of capacity diagram and demand spectrum is the displacement demand of earthquake [3]. S_{ae} is elastic spectral acceleration and S_d is spectral displacement. The steps of estimating inelastic displacement demand with pushover analysis are depicted in Figure 6. The bridge was pushed until the inelastic displacement demand was reached and rotation and strain values of hinges were derived. Performance levels were determined according to strain limits given in TEC 2007. In Figure 6, V_b is seismic shear force, u is top displacement, Φ_{N1} is amplitude of first vibration mode of top, Γ_1 is participation factor of first vibration mode, M_1^*

is effective mass of first vibration mode, S_{ae} is elastic spectral acceleration, S_{de} is elastic spectral displacement, S_{di} is inelastic spectral displacement and ω_1 is angular frequency of first vibration mode.

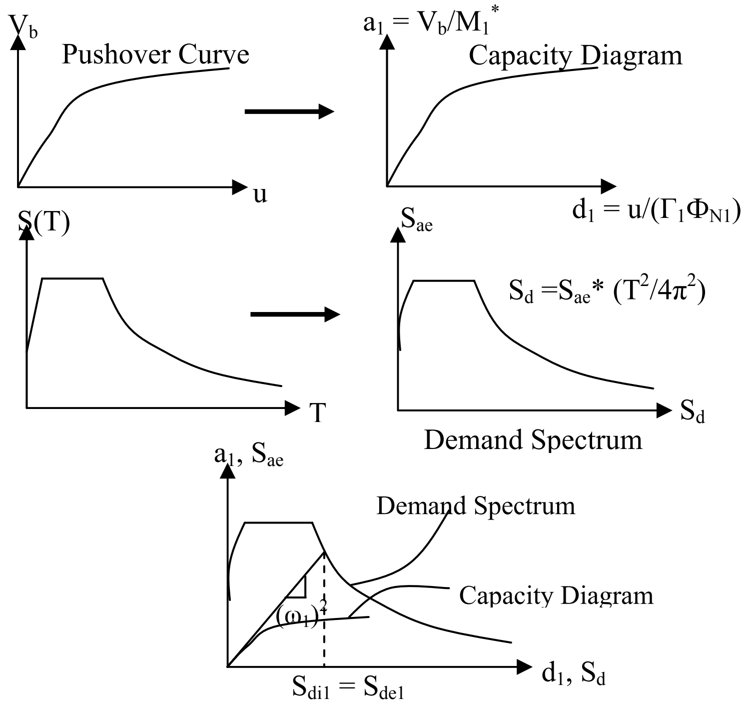


Figure 6: Steps of inelastic displacement demand for pushover analysis

4.2.2 NTA

Three ground motion records with magnitudes 6.61 and 7.51 were used for time history analysis [13]. All the records were scaled to standard spectrum given in TEC 2007 to provide a 2% probability of exeedance in 50 years. The location of the recording stations corresponds to the site class B according to USGS classification [14]. There are different limitations on the fault distance defined in literature to describe the near fault effect. However, in this study near fault effect is neglected since minimum considered fault distance is 55.3 km for ground motion records. The characteristics of ground motion records were given in Table 1.

Event	Station	Mag	Rrup(km)	Soil Class
San Fernando	Buena Vista - Taft	6.61	112.5	B
Kocaeli, Turkey	Eregli	7.51	142.3	B
Kocaeli, Turkey	Maslak	7.51	55.3	B

Table 1: Characteristics of ground motion records

5 Results

The periods of isolated and un-isolated bridges are given in Table 2. For un-isolated bridge, first mode is in transverse direction, second mode is torsional mode and third mode is in longitudinal direction. On the other hand, first mode is in transverse direction, second mode is in longitudinal direction and third mode is torsional mode for isolated bridge.

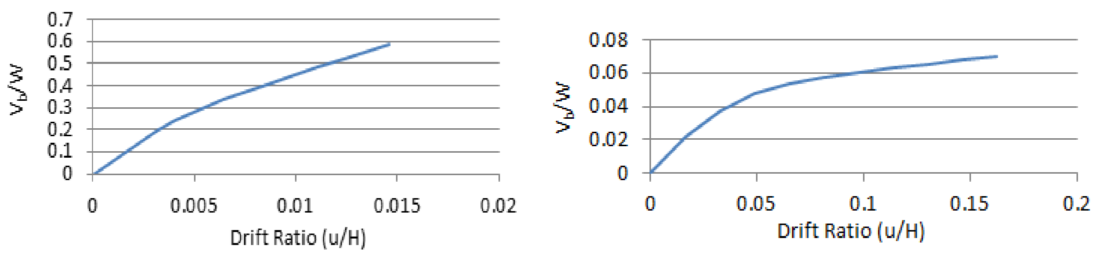
Mode number	Un-isolated Bridge	Isolated Bridge
	T (sec)	T (sec)
1	0.799	1.477
2	0.363	1.385
3	0.345	1.108

Table 2: Period of the un-isolated and isolated bridges

The performance level of the isolated bridge is immediate occupancy in transverse and longitudinal directions for all performance assessment methods. Also, earthquake response of the bridge in longitudinal direction is considerably small and damage of the columns is minimum. Thus, pushover curves and displacement time histories are given for transverse direction.

In transverse direction, the performance level of the un-isolated bridge is collapse prevention and it does not meet the requirements of life safety performance level. In longitudinal direction, the performance level of the un-isolated bridge is immediate occupancy for ***LEM.***

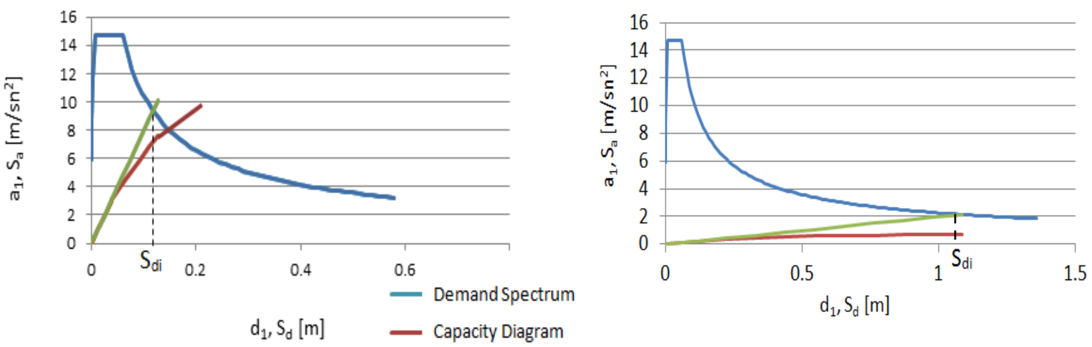
Pushover curves of isolated and un-isolated bridge, which were determined by using nonlinear static analysis, were given in Figure 7, where W is the weight of the bridge. The intersection of capacity diagram and demand spectrum for determining the inelastic displacement demand is depicted in Figure 8. The inelastic displacement demand (S_{di}) of the un-isolated bridge is 0.15 m in transverse direction and 0.04 m in longitudinal direction. S_{di} of the deck of isolated bridge is 0.112 m and S_{di} of top of the column is 0.033 in transverse direction.



(a) (b)

Figure 7: (a) Pushover curve of un-isolated bridge in transverse direction, (b) Pushover curve of isolated bridge in transverse direction

In both of directions, pushover analyses were performed until inelastic displacement demands were reached and the corresponding rotations and strains were estimated. Performance levels were estimated according to the strain limit values given in TEC 2007. In transverse direction, the performance level of the un-isolated bridge is collapse prevention, and in longitudinal direction the performance level of the un-isolated bridge is immediate occupancy for ***NSM***.



(a) (b)

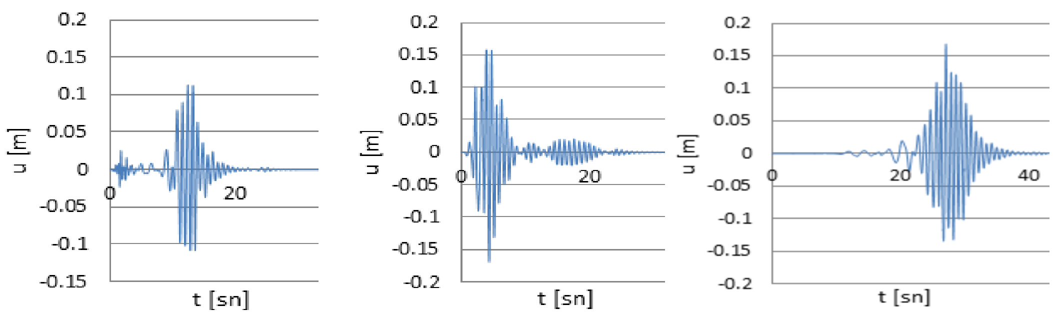
Figure 8: Assessment of inelastic displacement demand (a) un-isolated bridge, (b) isolated bridge

Displacement demand time histories of the un-isolated bridge are given in Figure 9. Inelastic displacement demand of the un-isolated bridge is 0.113 m for San Fernando Earthquake, 0.168 m for Kocaeli Earthquake Maslak Station and 0.169 m for Kocaeli Earthquake Eregli Station. According to TEC 2007, if only three earthquakes are used for time history analysis, maximum demand is used for performance assessment. Thus, inelastic displacement demand of the un-isolated bridge is 0.169 m based on time history analysis.

Displacement time histories are not given for the pier of isolated bridge since seismic isolation considerably decreases the inelastic displacement of the bridge because of dissipating large seismic energy and decreasing earthquake loads. Performance level of the un-isolated bridge is life safety in transverse direction and immediate occupancy in longitudinal direction for ***NTA***.

The purpose of the isolation is to protect the elements below the isolators by reducing the inertia loads transmitted from the superstructure [2]. Superstructures of isolated bridges can undergo large displacements since isolation allows it. However, isolation must be capable to undergo a large displacement. Horizontal displacement capacity of the LRB isolation used in this study is 0.68 m. The LRB is adequate to maintain the stability of the bridge based on LEM and NTA, however it is not adequate based on NSM.

The performance levels of the bridges for all the performance assessment methods were given in Table 3.



(a) (b) (c)

Figure 9: Displacement demand time histories of P211 column of the un-isolated bridge in transverse direction (a) San Fernando 1971 Earthquake (b) Kocaeli 1999 Earthquake Eregli Station (c) Kocaeli 1999 Earthquake Maslak Station

	LEM	**NSM**	**NTA**
Un-isolated	Collapse Prevention	Collapse Prevention	Life Safety
Isolated	Immediate Occupancy	Immediate Occupancy	Immediate Occupancy

Table 3: Performance levels of the bridges in transverse direction

6 Conclusions

- ***In transverse direction:*** Performance level of the *un-isolated* bridge is collapse prevention for LEM and NSM, life safety for NTA. Performance level of the *isolated* bridge is immediate occupancy
- Performance level of the isolated and un-isolated bridge is immediate occupancy for all performance assessment methods ***in longitudinal direction***.
- The period of first mode of the un-isolated bridge is 0.80 sec and the period of first mode of the isolated bridge is 1.47 sec. The period can be elongated and earthquake loads transmitted from superstructure can be reduced by using isolation.
- Although severe damage was determined for piers of the un-isolated bridge, the damage of piers of the isolated bridge is slight. Isolation is very important to reduce the damage and it is beneficial to use isolation for meeting the requirements of target performance level. Seismic isolation can reduce the inertia forces of the piers of the bridge by allowing the superstructure to displace. However, horizontal displacement capability of the LRB should meet the displacement demand of the earthquake.

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