The effect of In-span Hinges on the Seismic Behavior of Multiple-Frame Reinforced Concrete Box Girder Bridge

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Abstract Multi-span reinforced concrete box-girder bridges comprise a major part of the highway bridge in several states in the United States such as California. Approximately half of these bridges have at least one in-span hinge and is known as multiple-frame bridges. This paper aims at studying the effect of in-span hinges on the bridge seismic behavior. A four-span concrete box-girder bridge with unequal height piers is considered in this study, and is modeled once without any in-span hinges and once with in-span hinges considered at two location along the bridge deck (at 20% and 80% of the second bay length). Three-dimensional model of the bridge is developed using OpenSees and two different records with different intensities (strong and weak) are used to perform nonlinear time-history analysis. The results indicate that the most critical state is the one without any in-span hinges under strong earthquake excitation. However, under the less intense earthquake, larger seismic demands are observed when the in-span hinge is placed in the second span at 20% of the span length. Overall, it is found that the in-span hinge causes reduction in the seismic vulnerability of the main components. A suitable location for in-span hinge is in the second span at 80% of the bay length, where the whole of the structure approximately divides into two separate equal frames.

1 Introduction

Based on the analysis of a database of Californian bridges assembled by state engineers, multi-span reinforced concrete box-girder (MSCBG) bridges was found to account for about 37% of the state bridge inventory, which is comprised of single and multiple frame bridges. MSCBG bridges have a high torsional strength and allow the designers to increase the span length. In some of the longer bridges due to the thermal issues and also to allow the frames to have an independent seismic vibration, designers have to implement an in-span hinge (expansion joint). It is worth noting that 15% of California box-girder bridges have at least one in-span hinge [1]. In this case, bridges are typically referred to as multiple-frame concrete box girder bridges because of the independent performance of the bridge frames.

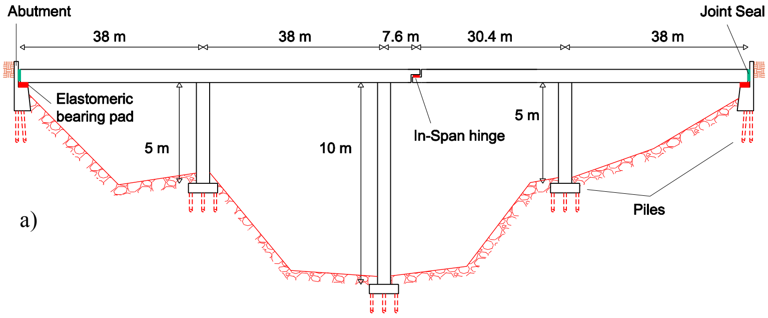
Following the earthquake in Northridge (1994), five of the California's bridges collapsed completely and at least 200 more were severely damaged [2]. Hence, in the last three decades, the California Department of transportation (Caltrans) has been working on improving the seismic design requirements of highway bridges. Among them, some of the reinforced concrete bridges have not been adequately addressed by seismic engineers. The multiple-frame concrete box girder bridges belong to this category. One of the main factors that distinguish this class of bridges from others is related to the in-span hinge. Thus, the frequency of this class of bridges, damages caused by the past earthquakes, and the lack of adequate studies on the seismic behavior of this bridge type are the main reasons to choose this type of bridge in this study. Based on the experiment done by Wang J, Carr A, Cooke N et al [3], relative joint displacements of these bridge classes include a dynamic component and a pseudo-static component. The dynamic component is because of the inertia effects arising from the variation between the vibrations of the two adjacent frames parted by the expansion joint. The pseudo-static component is due to the time delay between the vibrations of the adjacent frames. In addition, as described by Priestley M, Seible F, Calvi G, [2] the frames vibrations are affected by the stiffness and the yield strengths of the frames.

In this paper, in order to study the effect of the in-span hinge on the bridge seismic behavior, a four-span concrete box Girder Bridge with unequal height piers is considered. Several cases are considered for the bridge configuration with and without in-span hinges. In the case of including the in-span hinge, two different locations along the bridge deck (at 20% and 80% of the second bay length) are considered.

2 Bridge Modeling

The evolution of robust analytical modeling over the past years currently leads to reliable and more realistic prediction of the seismic behavior of bridges. A three-dimensional (3D) analytical finite element is developed using the OpenSees platform [4], which considers both geometrical and material nonlinearities. The OpenSees model is used in this study to conduct nonlinear time history analysis to evaluate the bridge seismic behavior. Note that choosing the geometry and configuration of the prototype bridge for this study along with all parameters used for modeling of the bridge are based on an extensive review of bridge plans in California, which is also consistent with the national bridge inventory database [5]. A general view of the prototype bridge used in this study is shown in Fig. 1.

a)



b)

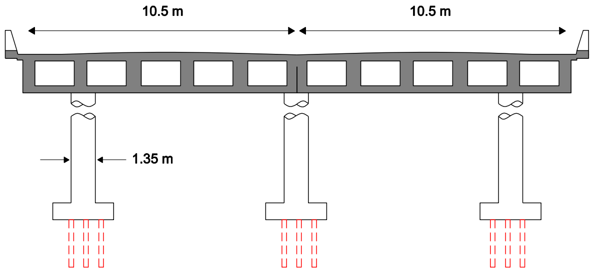


Fig. 1 A general view of the multi-frame concrete box-girder bridge in: a) longitudinal direction, and b) cross-section.

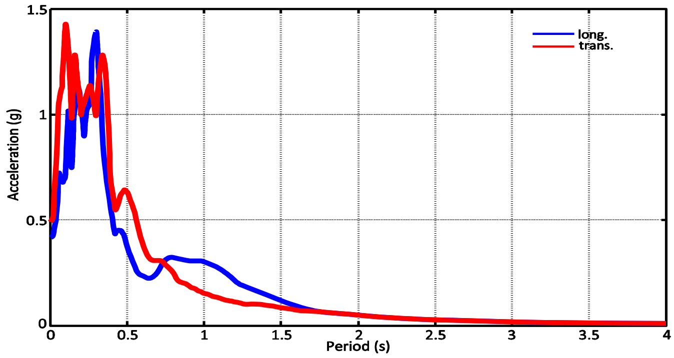
The bridge is irregular in height so that the middle frame height is 10 m, while the other two frames are 5 m in height (Fig. 1a). For modeling of the columns, nonlinear beam column elements with fiber-defined cross sections are considered. In the fiber sections each of the confined and unconfined concrete, and also the reinforcing steel, is defined in the precise coordinates of the cross sections. In this four-span bridge, columns with circular cross section have a diameter of 1.67 m. The reinforced concrete behavior is modeled using Concrete07, which is one of the available material models for concrete modeling in OpenSees. This material is provided based on the proposed model of Chang and Mander [6]. In comparison with other available concrete material models in OpenSees (e.g. Concrete01 or 03), Cocnrete07 exhibits higher initial stiffness, less softening after the peak tensile force, and it eliminates the issue of the sudden drop in the tensile concrete capacity. The reinforcing steel is modelled using the Reinforcing Steel material in OpenSees. In this material model, the fatigue and buckling behavior of steel during loading is included, which is ignored in other available material models (e.g. Steel01 or 02). In this study, the steel yield strength is 382 MPa, and the concrete compressive strength is 33.2 MPa. The percent of the longitudinal and transverse reinforcement of the columns are 2% and 0.5%, respectively. The soil-structure interaction is accounted for with a set of springs, which is designated as substructure method [7]. In order to describe the behavior of the foundation at the column base, a series of translational and rotational springs are typically considered. The effective stiffness of the translation springs in this study is assumed 97.65 MN/m but the rotational stiffness is neglected considering the pinned connection nature at the base of multi column bents. Due to the integration between the substructure and superstructure, rigid links are used to connect the column top to the deck elements. The abutment backwall has a height of 1.67 m and width of 21.53 m. Nonlinear springs are used to model the behavior of the abutments in both the transverse and longitudinal directions. In order to capture the response of the abutment backfill soil in passive performance (when the wall is moving toward the soil), the hyperbolic model recommended by Shamsabadi and Yan [8] is employed and added to the pile contributions. However, piles alone account for the active performance (when the wall is moving away from the soil). Also, the transverse resistance is assumed to be provided solely by the pile.

Linear elastic beam column element with lumped mass representing the longitudinal deck elements are connected to rigid deck elements in the transverse direction [9, 10]. The separate part of the superstructure at the expansion joint and also the endpoints of the deck beside the seat type abutment are connected by elastomeric bearings which are modelled by elastic-perfectly plastic springs [9,10]. These bearings typically transfer horizontal forces by friction and their behavior is characterized by sliding which in turn depends on the initial stiffness. The response of the bearing pad is captured using the Steel01 material provided by OpenSees. The bearing dimensions is 6 x 36 x 36 cm, a shear modulus of 1.78 MPa is defined for the bearing pads, the value of 0.2 is assigned to the friction coefficient, and the capacity of the elastomeric bearing is 95 KN. Due to the potential pounding at in-span hinge and also at the seat type abutments, impact elements are considered. A bilinear model developed by Muthukumar [11] is used for pounding between the two adjacent frames and also between the deck and the abutments. The stiffness parameters, yield displacement, and maximum deformation are consistent with those proposed by Nielson [12]. For shear keys, a multi-linear models based on the work done by Megally S, Silva P, Seible F [13] is considered, and they are located in the seat type abutments to limit movement along the transverse direction. A capacity of 600 KN is assigned to the shear keys. It should be noted that zero length elements describing the response of elastomeric bearing pads and pounding are connected in parallel and are connected to the transverse rigid deck elements. In the abutment, these are then connected in series with the soil-pile springs.

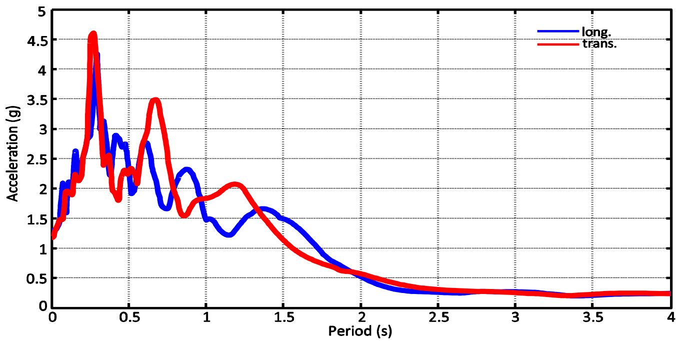
3 Ground Motion Records

As previously mentioned, nonlinear time-history analysis of the given prototype bridge is conducted to investigate the dynamic response of the bridge in different in-span hinge cases. Two orthogonal horizontal pairs of ground motions are considered. The response spectra of each of the two selected ground motions in the transverse and longitudinal directions are shown in Fig. 2. The first record represents a near fault earthquake in California, which is selected from the suite of 20 ground motions pertinent to Los Angles developed for SAC project database

[14]. The second one is a typical far fault record from the PEER Strong Motion Database [15]. These two records have peak ground acceleration of 1.29g and 0.41g in the longitudinal direction, and 1.18g and 0.47g in the transverse direction, respectively.



(a)



(b)

Fig. 2 Response spectrum of the a) near-fault and b) far-field ground motions selected for this study in transverse and longitudinal horizontal directions.

4 Modal and Nonlinear Time-History Analysis

The modal analysis for all the three cases (without in-span hinge, with in-span hinge at 20%, and at 80% of the second bay length) is conducted using OpenSees and the periods of the bridge for the first three modes are listed in Table 1. It is observed from the table that the in-span hinge increases the structural vibrations periods, i.e. the lowest fundamental period of the bridge is in the case without in-span hinge.

Nonlinear time history analysis is conducted using the two ground motions previously mentioned. Selected analysis results and bridge component response are presented and discussed in this section. The deck displacement history in the longitudinal direction, where the in-span hinge effect can be highlighted, is shown for the near-fault and far-field earthquake cases in Figs. 3 and 4, respectively. According to the figures, the in-span hinge at 80% of the second bay length leads to a 25% reduction in the deck displacement along the longitudinal direction in both earthquake scenarios when compared to the other cases including the deck without any in-span hinge or with an in-span hinge at 20% of the second bay length. Fig. 5 shows the deck displacement history in the transverse direction but only for the near-fault earthquake case for brevity. The figure shows that the deck with an in-span hinge at 80% of the second bay length experiences about 48% less displacement along the transverse direction than the deck with an in-span hinge at 20% of the second bay length.

In addition, the force-displacement relationships of the backfill soil spring and the bridge shear key are shown in Figs. 6 and 7, respectively. It can be seen from Fig. 6 that the in-span hinge at 80% of the second bay length causes almost 34% reduction of seismic demand of the backfill soil displacement compared to the case without any in-span hinge. Fig. 7 illustrates a similar observation for the shear key response as the effect of the in-span hinge on the seismic response of the backfill soil. One last response quantity that is reported here in this paper is the lateral force-displacement relationship of the abutment pile, which is shown in Fig. 8. The figure indicates that in the case of having one in-span hinge at 80% of the second bay length approximately 35% less seismic demand can be expected for the piles in comparison with the case without any in-span hinge. From the previous discussion and seismic response of the different bridge components presented in Figs. 3 through 8 in case of near-fault and far-field earthquakes, it is noted that the in-span hinge located at the 80% of the second bay length results in the minimum seismic demands for approximately all the components.

Table 1. Vibration periods of different OpenSees bridge models.

Vibration mode / In-span hinge location	First mode (sec)	Second mode (sec)	Third mode (sec)
no in-span hinge	0.60	0.40	0.30
at 20% second bay span	0.74	0.65	0.48
at 80% second bay span	0.62	0.61	0.55

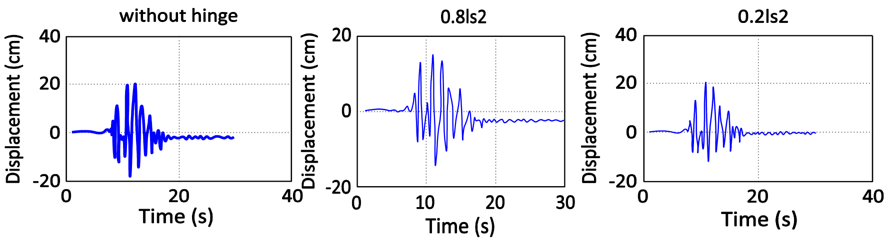


Fig. 3 The deck displacement along longitudinal under the near-fault earthquake.

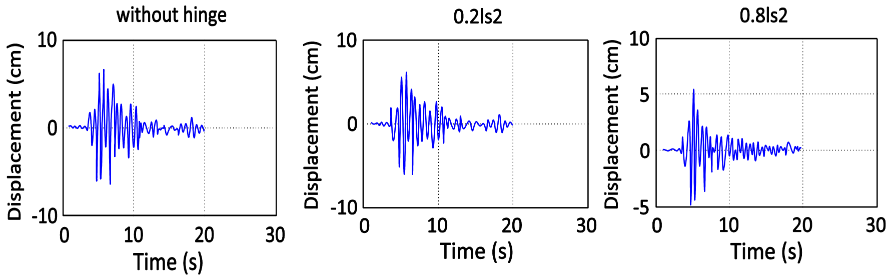


Fig. 4 The deck displacement along longitudinal under the far-field earthquake.

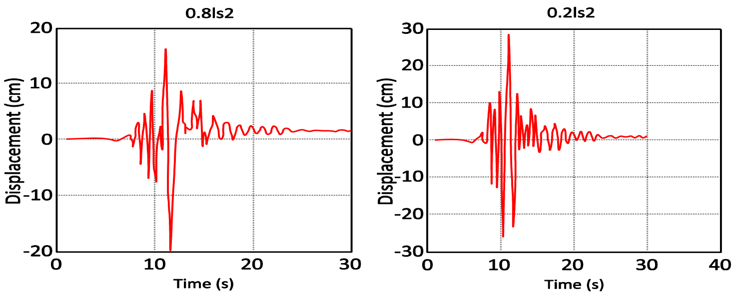


Fig. 5 The deck displacement along transverse under the near-fault earthquake recorded at the in-span hinge location.

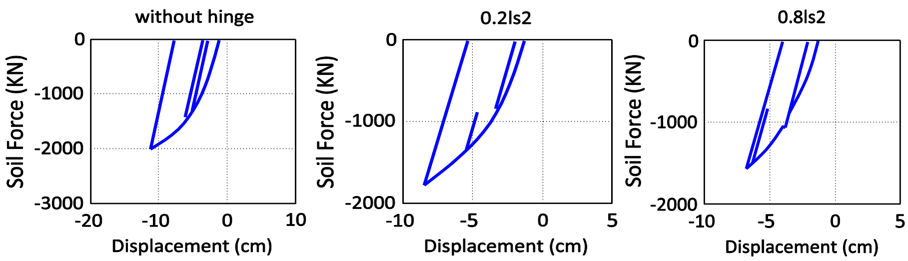


Fig. 6 The backfill soil response under the near-fault earthquake.

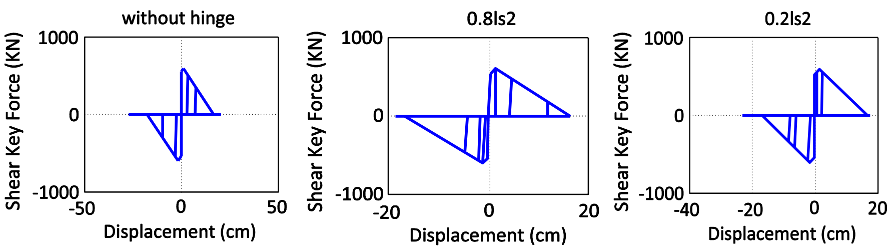


Fig. 7 The shear key response under the near-fault earthquake.

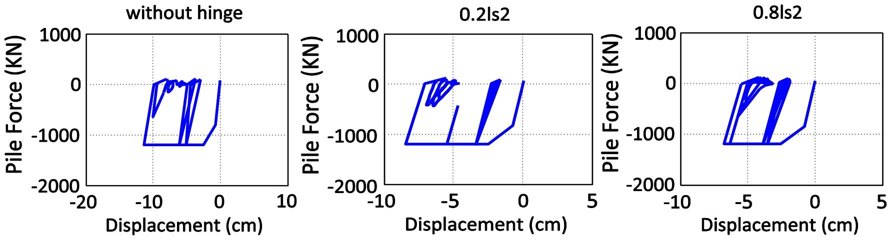


Fig. 8 The abutment pile response under the near-fault earthquake.

5 Summary and Conclusions

In this paper a 3D analytical model for a four-span multi-frame concrete box girder bridge with unequal height piers, which is common in California, has been developed using OpenSees platform. In order to study the effect of the in-span hinge on the bridge seismic behavior, nonlinear time history analysis has been conducted for different bridge cases. The bridge is modeled without any in-span hinges and with in-span hinges, which in turns is considered at two locations along the bridge deck (at 20% and 80% of the second bay length). Two pairs of orthogonal horizontal earthquake records from previous California events with strong (near-fault) and weak (far-field) intensities have been utilized in the analysis. The seismic response of all different bridge components, e.g. columns, deck displacement, bearings, shear keys, backfill soil, pile, and foundation have been compared in different cases.

According to the results, it can be seen that the bridge seismic behavior is favorably affected by the in-span hinge. The in-span hinge in general reduces the demands and hence, reduce the seismic vulnerability of the different bridge components particularly under severe earthquakes. This is attributed to the fact that the structure's mass under the earthquake acceleration is split between the two independently-acting frames. Thus, less demands and lower displacements can be expected. As a result, the in-span hinge should be considered especially for long bridges. To avoid phase difference in the dynamic behavior of two adjacent bridge frames, the best location for the in-span hinge in case of a four-span bridge is recommended at 80% of the second bay length where the bridge is almost evenly divided into two frames with similar lengths.

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