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IN-PLANE ROTATIONAL DEMANDS OF SKEWED BRIDGES DUE TO EARTHQUAKE INDUCED POUNDING

S. S. Catacoli[1], C.E. Ventura[2], W.D.L Finn[3] and M. Taiebat[4]

ABSTRACT

Skewed bridges are irregular structures due to the geometry of the deck and bents. Past earthquakes indicate that skewed bridges with seat type abutments exhibit greater damage than non-skewed bridges with similar seat type abutments. These bridges have become unusable due to damage at the piers and/or by unseating of the deck. The damage has been attributed to in-plane rotations caused by pounding between the skewed deck and its abutments during strong ground shaking. A comprehensive parametric study based on nonlinear dynamic analyses was performed to evaluate the effects of different skew angles and abutment design approaches on the seismic response of this type of bridges. The results demonstrated that elastic methods recommended by current seismic design provisions, and commonly used in standard practice, do not properly capture the in-plane rotations of the deck due to pounding**.** This paper advances the understanding of the seismic response of skewed bridges, and contributes to the development of displacement-based methods for these structures.

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Skewed bridges are irregular structures due to the geometry of the deck and bents. Past earthquakes indicate that skewed bridges with seat type abutments exhibit greater damage than non-skewed bridges with similar seat type abutments. These bridges have become unusable due to damage at the piers and/or by unseating of the deck. The damage has been attributed to in-plane rotations caused by pounding between the skewed deck and its abutments during strong ground shaking. A comprehensive parametric study based on nonlinear dynamic analyses was performed to evaluate the effects of different skew angles and abutment design approaches on the seismic response of this type of bridges. The results demonstrated that elastic methods recommended by current seismic design provisions, and commonly used in standard practice, do not properly capture the in-plane rotations of the deck due to pounding. This paper advances the understanding of the seismic response of skewed bridges, and contributes to the development of displacement-based methods for these structures.

Introduction

Earthquake-induced pounding occurs when the expansion gap at an abutment is closed during a seismic event leading to a deck-abutment collision. The collision generates a coupled system. The study of this system requires consideration of the Embankment-Abutment-Structure Interaction (EASI) effects. The EASI effects are particularly relevant in the case of skewed bridges with seat type abutments, as seismic damage due to past earthquakes illustrates that the superstructure tends to rotate as a result of the pounding between the deck and its abutment. The rotations increase the probability of superstructure unseating and the lateral demands of the piers.

The EASI effects depend on the soil passive pressure mobilized by pounding and the seismic design strategy of the bridge, which includes the amount of deformation expected at the

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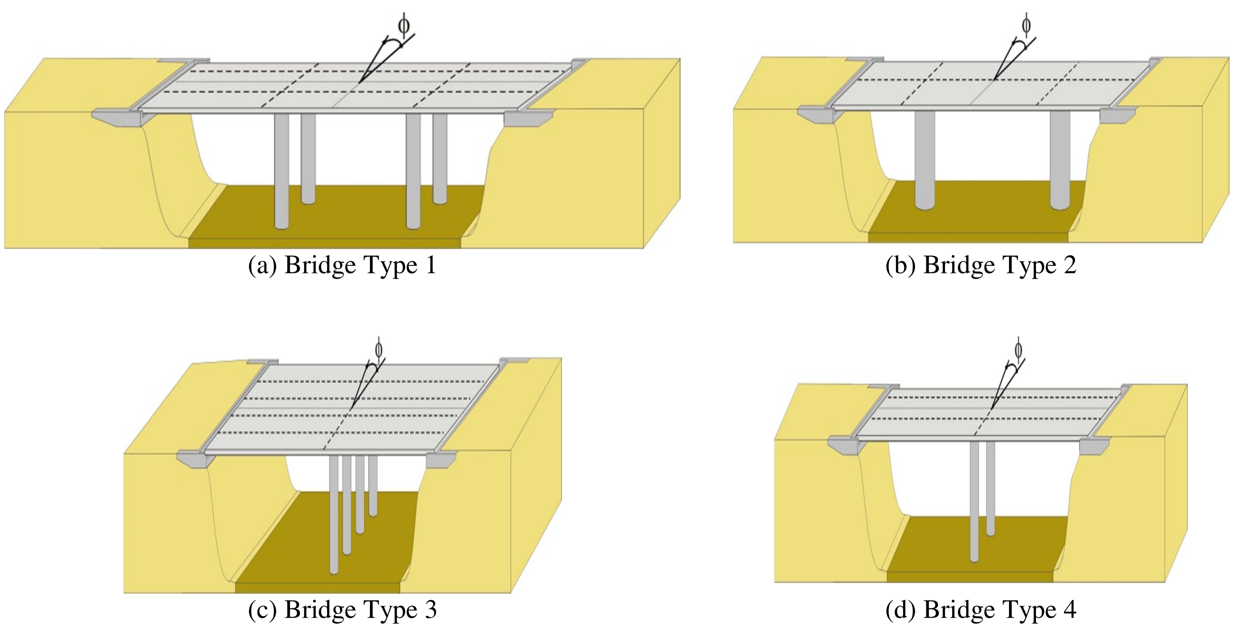
abutments, piers, and foundations. Pounding is principally a short duration mechanism, in which only compression forces are transferred once the longitudinal gap is closed. This mechanism can be properly represented using nonlinear models. Kavianijopari and Shamsabadi [1,2] progressed the study of EASI effects for skewed bridges with ductile abutments. So far, there are no studies that examine the contribution of the in-plane rotation of the deck to the total drift of the pier for different abutment design approaches.

This paper presents a parametric study of the nonlinear displacement demands of skewed bridges with different structural response at abutments and SFSI effects. The study is applied to short and medium multi-span bridges with continuous superstructure and representative of the bridge inventory in the province of British Columbia (BC), Canada.

Description of the Models

Bridge Types

Past earthquakes have predominantly damaged skewed bridges with two and three spans. The four bridge types considered in this research are selected to represent standard two and three span bridges with different cross sections, pier types and clear heights located in British Columbia. The bridges are continuous and symmetric. The superstructure is supported at the two ends on seat type abutments with two-inch (5 cm) expansion gaps. Each bridge is studied at skew angles of 15, 30, 45 and 60 degrees. The characteristics of the selected bridge configurations are summarized in Figure and Table 1.



(a) Bridge Type 1 (b) Bridge Type 2

(c) Bridge Type 3 (d) Bridge Type 4

Figure 1. Sketches of bridge types analyzed

Table 1. Summary of bridge properties

Bridge	Length (m)	Spans		Width (m)	Clearance (m)	Skew Angle (degrees)	Substructure		Superstructure Type
		No.	Lengths (m)				Type	Abutments	
Type 1	120	3	40-40-40	12	10	15-30-45-60	multi-column-frames (ϕ = 1.20 m)	seat type	Continuous – reinforced concrete I-girders
Type 2	80	3	20-40-20	12	10	15-30-45-60	Single column bent (ϕ = 1.50 m)	seat type	Continuous – reinforced concrete I-girders
Type 3	46	2	23-23	20	5	15-30-45-60	multi-column-frames (ϕ = 1.20 m)	seat type	Continuous – concrete box stringers
Type 4	46	2	23-23	12	10	15-30-45-60	multi-column-frames (ϕ = 1.20 m)	seat type	Continuous – concrete box stringers

Detailed Nonlinear Spline Models

Three dimensional spline models are developed in order to study the different bridge types at the selected skewed angles (Figure 2). In the spline models the abutment shear keys are represented by four springs in the direction of the skew, and the abutment backfill longitudinal response is represented by a set of springs perpendicular to the face of the skewed abutment. The deck and pier bent are modeled using 3D beam elements. Rigid elements are used to represent the cap beams and the abutment-caps. The models are developed using the computer program SAP 2000 [3]. A detailed description of the properties of the models is given in following sections.

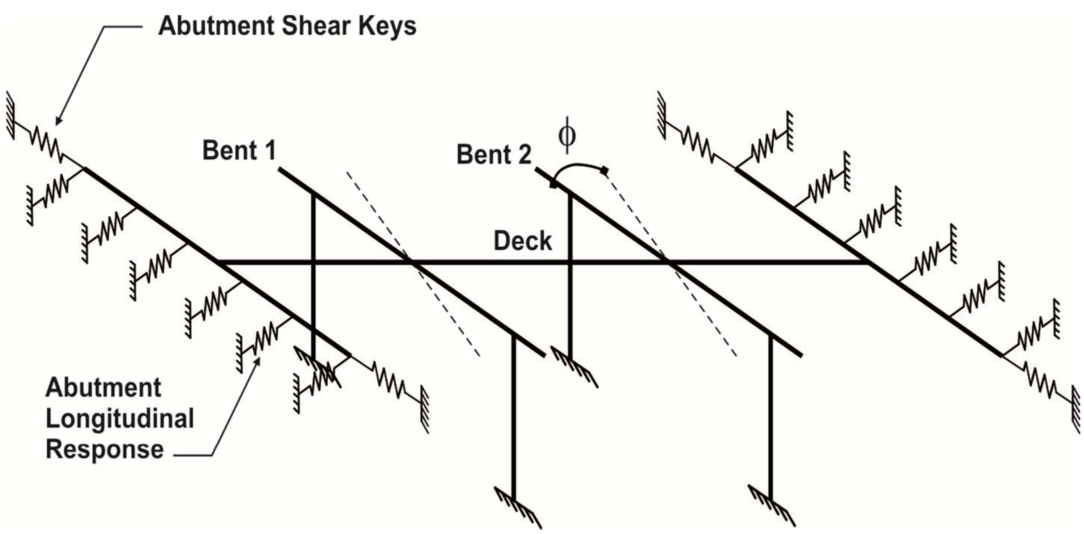


Figure 2. Spline model Bridge Type 1

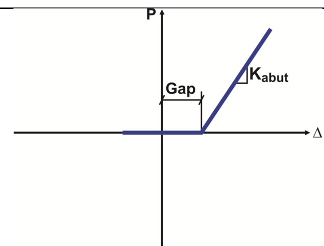
Abutment Models

Abutment Design Approaches

The level of deformation and extent of damage expected for the abutments depends on the seismic design approach of the bridge. Some jurisdictions take into consideration the contribution of the abutments to resist seismic demands. In this scenario, the expected response of the abutment backfill in the longitudinal direction might be elastic when the abutment deformations are small or inelastic when the deformations are large enough to reach the maximum capacity of the abutment backfill. In the transverse direction an elastic performance of the abutment is assumed [4].

In contrast, other jurisdictions consider only the bridge piers to resist the seismic demands and the abutments are considered only an additional source of structural redundancy. In this approach, the abutment is designed to be capacity protected in the transverse direction by making use of shear keys that act as fuses and have a brittle failure [4]. In this paper, three abutment models that combine the longitudinal deformations of the abutment-backfill with the structural response of the abutment-shear keys are used to represent the design approaches previously mentioned (Table 2).

Table 2. Summary of abutment models used to represent different design approaches

<table>
<tr><th></th><th>Transverse Direction</th><th>Longitudinal Direction</th></tr>
<tr><td>Linear Abutment with Longitudinal Gap:
In the longitudinal direction abutment-backfill only transfer compression forces once the gap is closed. In the transverse direction an elastic response of the abutment is considered.</td><td>


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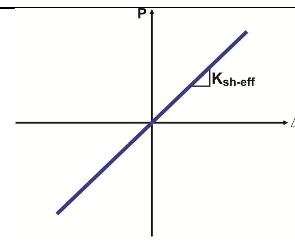
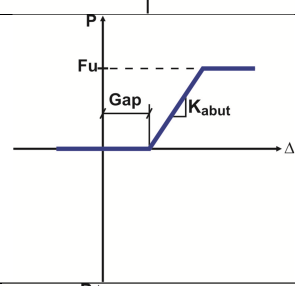


</td></tr>
<tr><td>Bilinear Abutment:
In the longitudinal direction, the backbone represents an abutment-backfill with elasto-plastic response, in which only compression forces are transferred once the gap is closed. In the transverse direction, the model represents an elastic response of the abutment.</td><td>


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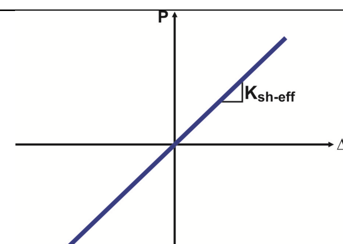
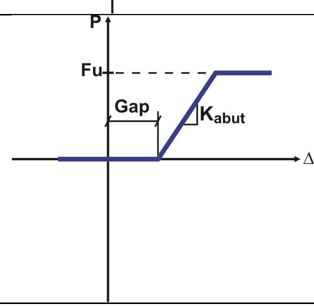


</td></tr>
<tr><td>Fusing Abutment:
In the longitudinal direction this model represents an abutment-backfill with an elasto-plastic response. In the transverse direction, the backbone represents an abutment with a lateral restraint system that only transfers forces in one direction and fuses by having a brittle failure once its maximum capacity is reached.</td><td>


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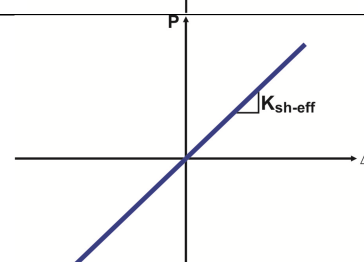

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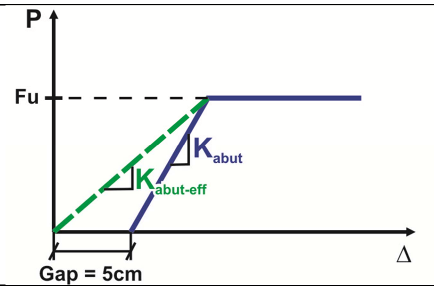
Abutment Parameters

The input parameters to define the backbone curve in the longitudinal direction for the abutment models are calculated according to the recommendations given in the Caltrans Seismic Design Criteria Version 1.6 [5]. Table 3 presents the values recommended per meter of abutment by Caltrans for seat type abutments with a backwall height of 1.7m. In the transverse direction, the input parameters used to define the abutment shear key response are based on the experiments by Silva et al. [6], the effective shear key stiffness ($K_{sh\text{-}eff}$) used in the abutment models is 18 MN/m and the maximum capacity (P_u) was 0.90 MN for each shear key. The effective shear key stiffness includes the effect of the one-inch (2.5 cm) gap between shear keys and deck.

Table 3. Input values for the longitudinal response of the abutment models

Backbone Sketch	Abutment Stiffness, K_{abut} $\frac{(MN/m)}{m}$	Effective Abutment Stiffness, $K_{abut\text{-}eff}$ $\frac{(MN/m)}{m}$	Maximum Passive Capacity, F_u $\frac{(MN)}{m}$
	29.35	6.33	0.41

Superstructure Model

The superstructure is expected to respond elastically during a ground motion. Each superstructure is represented by beam elements with equivalent section properties (Figure 2).

Bent Model

Pier bents are modeled using a beam element with effective cracked section properties ($I_{effective}$= $0.5 \times I_{gross}$). As it was the intention of this research to capture only the nonlinear effects induced by pounding, which are represented by the nonlinear abutments models considered, effective section properties were chosen over fully nonlinear hysteretic models to represent cracking of the piers. The piers are modeled rigidly connected (fixed) to the base. The cap beams are modeled by rigid elements and are assumed to be rigidly connected to the superstructure.

Selected Ground Motions

The ground motions considered correspond to crustal, subcrustal, and subduction earthquakes. The records were selected from the suite of representative ground motions recommended in a comprehensive study of the seismic hazard in south-western British Columbia (BC), developed as part of the project for the seismic retrofit of existing school buildings in BC [7].

The crustal and subcrustal records were scaled to match the Uniform Hazard Spectra (UHS) for Vancouver and the subduction records were scaled to match the UHS for Victoria, using a probability of exceedance of 2% in 50 years at a site class C. The frequency match was performed for periods from 0 to 2 seconds. The records are applied parallel and perpendicular to the direction of the skewed bents. A summary of the selected records is shown in Table 4.

Table 4. Selected ground motions

Source	Earthquake Name	Date	Station Name	Mw	Epicentral Distance (km)	PGA (g)
Crustal	Loma Prieta	18-Oct-1989	CDMG 57007 Corralitos	6.9	18.9	0.35
	Northridge	17-Jan-1994	USGS 5108 Santa Susana Ground	6.7	22.8	0.30
Subcrustal	Nisqually	28-Feb-2001	Seattle (EVA)	6.8	80.7	0.25
	El Salvador	13-Jan-2001	Unidad de Salud, Panchimalco (PA)	7.6	95.7	0.36
Subduction	Maule, Chile	27-Feb-2010	Santiago Maipu (E-W)	8.8	78.9	0.32
	Michoacan, Mexico	19-Sept-1985	La Union (UNIO)	8.1	83.9	0.41
	Tokachi-oki, Japan	25-Sept-2003	Noya (HDK107)	8.0	126.4	0.30

Displacement Demands

This section presents a discussion of the in-plane rotations due to earthquake-induced pounding and the resulting additional pier drift for different types of structural response at abutments. It is important to note that for the lateral displacements presented in this paper, the transverse direction is assumed to be in the direction of the skew and the longitudinal direction is normal to this (Figure 3). This coordinate system is adopted here as it is consistent with the predominant directions of lateral response of skewed bridges indicated by the experimental [8]. Also, it is convenient when comparing the demand and capacity of the bents, as AASHTO [4] requires a comparison with demands obtained at the azimuth of the skewed bents.

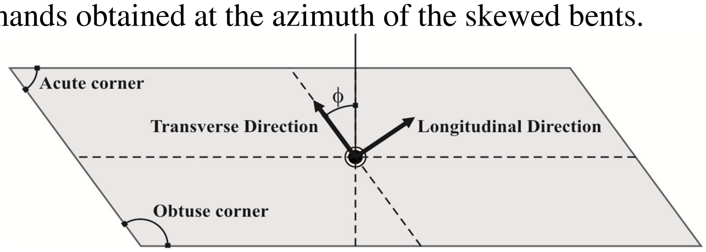


Figure 3. Directionality of the lateral response of skewed bridges

Response History of In-plane Deck Rotation

The time histories of the deck rotation at the center of mass of the bridge Type 1, skewed at 45 degrees, during the 1989 Loma Prieta earthquake are used to highlight common features observed in the in-plane rotational response. Figure 4 illustrates the strong influence of the abutment design approach in the magnitude and characteristics of the in-plane rotational response.

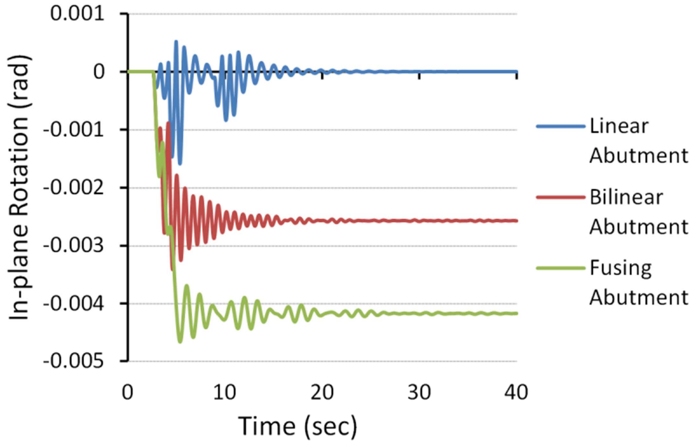


Figure 4. Response history of the in-plane rotations of the deck for different abutment design approaches (Bridge Type 1 skewed at 45 degrees – Loma Prieta Earthquake)

Linear abutments with longitudinal gap, in which the backfill remain elastic, have the lowest in-plane rotations and do not show any residual rotation (Figure 4). For bilinear abutments, in which the backfill reaches its plastic capacity, the induced in-plane rotations are larger than for linear abutments. The rotations are all negative, indicating a permanent clockwise rotation of the longitudinal axis of the deck during the ground motion. The largest in-plane rotations are obtained for fusing abutments, in which plastic deformation of the backfill along with brittle failure of the abutment shear keys are expected.

Peak In-plane Deck Rotation and Additional Pier Drift

In standard engineering practice it is important to assess the peak in-plane rotational demands of skewed bridges as they contribute to the calculation of the support length that the engineer should provide at the abutments in order to prevent superstructure unseating. These peak in-plane rotational demands also induce additional drift demands to the piers. This additional drift is calculated for the column furthest from the center of mass as illustrated in Figure 5. The additional drift is a useful measure to highlight the displacement demand missing at the pier when the in-plane deck rotation induced by earthquake pounding is ignored in the analyses.

The mean and the dispersion of the peak in-plane rotations and additional drifts obtained for Bridge Type 1 for all ground motions are presented Figure 6. The response is very sensitive to the type of abutment. It is noted that the largest demands are experienced by skewed bridges with abutment shear keys that fuse during the earthquake. This could be explained, in part, by the reduction in the lateral and rotational stiffnesses of the bridge once the shear keys fails. This type of response could also be expected for existing bridges with poor lateral restraint. The additional drift could be up to 1.8 % for skew angles of 60 degrees. The lowest demands are observed for linear abutments, for which the lateral restraint remains in place throughout the earthquake. In this case, the additional drift could be up to 0.5 %. For the bilinear abutments, the largest additional drift is average 1.4 %, and for all skew angles the response is bounded between the response of the linear and the fusing abutments.

A linear dependency in the magnitude of the peak in-plane rotation and additional drift as a function of the skewed angle is observed. This trend suggests the magnitude of the demands is proportional to the skew angle. The dispersion is in generally small, and the largest standard deviation (0.4%) was obtained for the bridge with fusing abutments skewed at 60 degrees. Similar results were observed for Bridge Type 2.

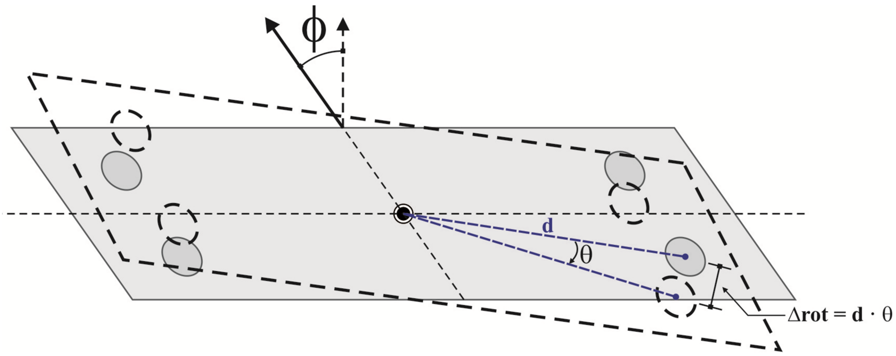


Figure 5. Illustration of additional pier displacement (Δ_{rot}) due to in-plane deck rotation

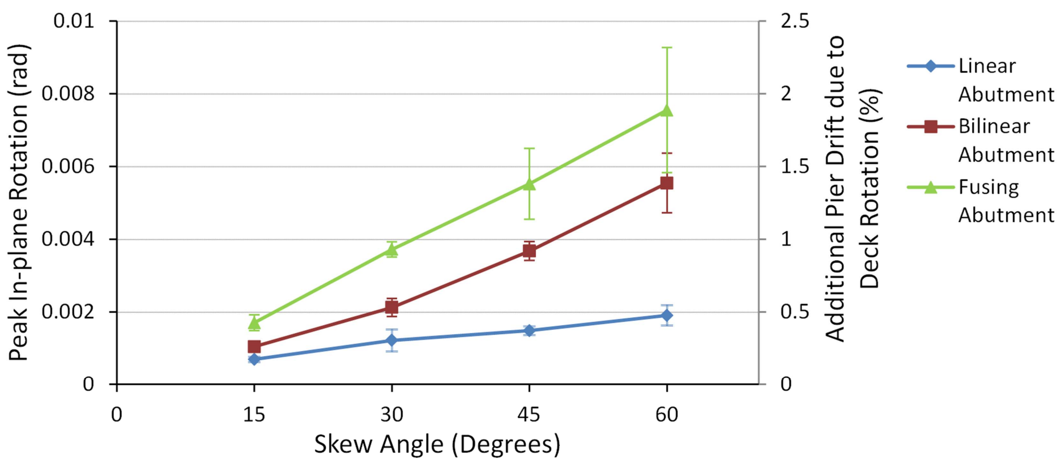


Figure 6. Bridge Type 1- Peak in-plane rotations and additional pier's drift for all ground motions

An interesting finding regarding the Bridge Type 3, is that there were no in-plane rotations of the deck for all skew angles, abutment performances and input ground motions considered. Bridge Type 3 is a wide/short (46m), two-span structure, with a 5 m high four columns bent. The reason why in-plane rotations of the deck are not observed is because the two-inch (5 cm) dimension of the longitudinal gap is not exceeded during the ground motion, and so pounding does not occur and the induced in-plane rotations are not triggered (Figure 7). The absence of in-plane rotations for Bridge Type 3 illustrates that simply because a bridge is skewed and torsionally sensitive, does not mean that it is necessarily going to be torsionally activated, the onset of in-plane rotations will depend on the size of the gap and whether or not it is closed.

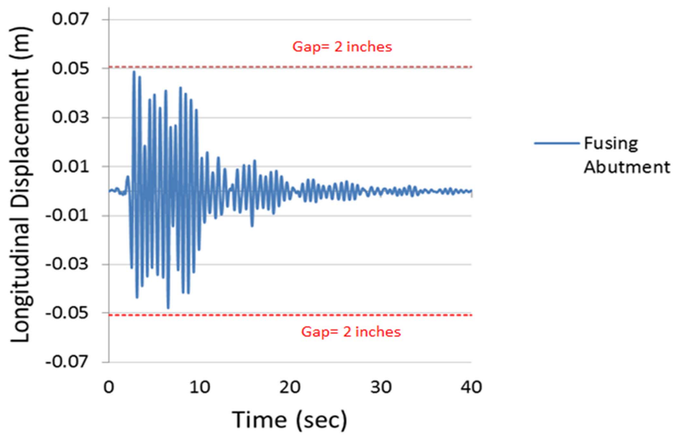


Figure 7. Longitudinal response for Bridge Type 3 with fusing abutments and skewed at 60 degrees subjected to the 1989 Loma Prieta Earthquake

In contrast, for Bridge Type 4, which is also a two-span bridge with the same length as Bridge Type 3 but with a narrower deck and two columns per bent (10 m high), in-plane rotations of the deck were observed. Since it is a two-span structure the in-plane rotations primarily increase the longitudinal drift of the pier. The pier drift increases proportionally with the skew angle. Similar to the other bridges, the lowest drift values, and dispersions are found for the linear abutments, with the largest being for the fusing abutments which can reach up to 1.25 % with standard deviation of 0.25 % for the structure skewed at 60 degrees (Figure 7).

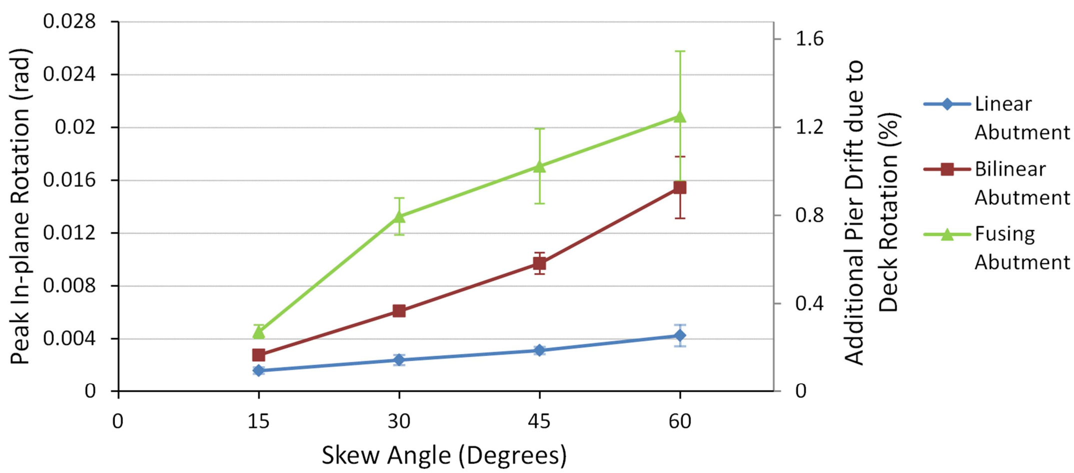


Figure 7. Bridge Type 4- peak in-plane rotations and additional pier drift for all ground motions

Conclusions

Displacement-based design provisions require an accurate assessment of the displacement demands that bridges will be subjected to during earthquake shaking. This paper demonstrated that the contribution of the in-plane deck rotation to the total drift of the pier strongly depends on the structural response expected at abutments by the approach used to design the bridge. The largest additional pier drift (up to 2%) due to the in-plane deck rotation was obtained here for bridges with abutment shear keys expected to fuse due to brittle failure during the ground motion. The lowest contribution was obtained for bridges with abutments that are expected to remain elastic throughout the earthquake. The analyses demonstrate that the additional drifts due to deck rotation increase in linear proportion with the skew angle. The results suggest that the structural response of skewed bridges in seismic regulations should be discussed separately for each type of abutment response (elastic, elasto-plastic, and fusing).

The analyses showed that in-plane rotation of the superstructure is only triggered when the longitudinal gap is exceeded. This observation implies that simply because a bridge is skewed, does mean that it is necessarily going to be torsionally activated. The analyses also showed that torsional activation depends on the size of the gap and whether or not it is closed. In summary, this paper contributes to the understanding of displacement demands on skewed bridges with seat type abutments and enhances the recommendations for performance based design of skewed bridges.

Acknowledgments

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