SECOND EUROPEAN CONFERENCE ON EARTHQUAKE ENGINEERING AND SEISMOLOGY, ISTANBUL AUG. 25-29, 2014

SENSITIVITY OF RESPONSE OF SKEWED SEAT-TYPE ABUTMENT BRIDGES TO EXTERIOR SHEARKEY MODELING

Bahareh MOBASHER[1], Roshanak OMRANI[2], Farzin ZAREIAN[3], and Ertugrul TACIROGLU[4]

ABSTRACT

This paper is aimed at quantifying the variability in predicted seismic response of bridges due to uncertainty in exterior shear key behavior. Shear keys are usually modeled as sacrificial elements; however, experiments show that the behavior of shear keys depend on their reinforcement detailing, and the construction joint between the shear key and the bridge abutment's stem wall. This paper focuses on modeling approaches for shear keys in bridges with box-girder deck and seat-type abutments. Construction of this type of bridge is common in California since 2000. An enhanced ductile shear key model, generated based on experimental results is used in this study. As for sacrificial (brittle) shear keys, a simplified analytical model is developed based on experimental evidence. The model matrix is comprehensive and comprises bridges with various abutment skew angles and a suite of forty near-fault ground motions. Three parameter lognormal distribution is employed for probabilistic seismic response assessment. The result of our investigation indicates that the numerical modeling details for exterior shear keys have significant effects on the predicted seismic response, especially for those bridges with skew-angled abutments; and as such, the accuracy and realism of these models are critical.

INTRODUCTION

Seat- and diaphragm-type abutments are the two most common abutment types for reinforced-concrete bridges in California. Seat-type abutments allow free lateral and longitudinal movements of the superstructure relative to the abutment; whereas for the diaphragm-type abutments, these movements are suppressed (SDC 1.7, 2013). Shear keys—which are crucial components in seat-type abutments—are designed to provide support in the transverse direction, transmit the lateral shear forces from small to moderate earthquakes and service loads, and fail under a strong motion event. Therefore, realistic modeling of the force-deflection characteristics of shear keys has significant influence on the predicted bridge behavior.

Observations from previous earthquakes such as Northridge 1994 show multiple incidences of external shear key and abutment failure (Fig.1). Diagonal cracks in an abutment stem wall indicate the ductile behavior of external shear keys (Priestly et al., 1994; Moehle et al., 1995).

[1] Graduate Student Research Associate, University of California, Irvine, bmobashe@uci.edu
[2] Assistant Project Scientist, University of California, Irvine, romrani@ucla.edu
[3] Associate Professor, University of California, Irvine, zareian@uci.edu
[4] Professor, University of California, Los Angeles, etacir@ucla.com

Figure 1. Damage to external shear key in an abutment in the 1994 Northridge earthquake (Source :EERI, Earthquake Spectra, Special Suppl. To Vol. 11, 1995)

Megally et al. (2001) and Bozorgzadeh et al. (2004) have conducted experiments and developed analytical models to quantify the behavior of shear keys and their failure mechanisms. This experimental program–which comprise destructive tests on 6 exterior and 7 interior typical Caltrans shear keys–was utilized to assess variation in shear key response to different loading protocols, and various detailing and reinforcement ratios. Experimental results showed that exterior shear keys may fail in a sliding shear friction (brittle shear key), or a strut-and-tie failure mechanism (ductile shear key); the failure mechanism depends on the reinforcement detailing and construction joint details at the shear key and the stem wall interface (see Fig.2). Goel and Chopra (2008) evaluated the role of the shear keys in seismic behaviour of bridges crossing fault-rupture zones. They defined the nonlinear behaviour of ductile shear keys with a simple tri-linear force deformation model based on the work of Megally et al. (2001). They concluded that upper and lower bounds for seismic demands for non-skewed bridges crossing fault-rupture zones can be obtained through analyses of two simpler nonlinear models: *i*) a bridge model with elastic shear keys, and *ii*) a bridge model without shear keys. Fell and Salveson (2013) assumed shear keys are sacrificial elements and compared the effect of two types of sacrificial shear key modeling on bridge response: *i*) the linear model suggested by California seismic provisions (SDC 1.7, 2013), and *ii*) a nonlinear model that included a gap distance and a bilinear force-deformation backbone curve. They suggested that the nonlinear shear key model is adequately accurate for bridges with flexible substructures ($T > 1$), and the linear shear key model is reasonable for short and stiff bridges. However, they did not evaluate the effects of ductile shear keys on bridge responses in their study.

Section 7.8.4 of Caltrans Seismic Design Criteria version 1.7 (SDC 1.7, 2013) provides two types of detailing for exterior shear keys: (1) isolated shear keys based on the recommendations by Megally et al. (2001), and (2) non-isolated shear keys. According to SDC 1.7 (2013), lateral stiffness of the abutment in the transverse direction, which is mostly provided by shear key, is 50% of the lateral stiffness of the adjacent bent. Both non-isolated and isolated shear keys are assumed to behave as sacrificial brittle shear keys. However, experimental studies (Megally et al., 2001) indicate that shear keys with the reinforcement detailing of non-isolated shear keys behave in a ductile manner and can result in significant damage to the stem wall of the bridge abutment. Given these observations, it is fair to conclude that the effects of variations in the nonlinear force-deformation characteristics of shear keys–ductile or brittle–on bridge demand parameters are not adequately characterized and need further investigation.

The present study focuses on two realistic nonlinear modeling approaches for exterior shear keys in bridges with box-girder deck and seat-type abutments. Force-deformation relationships of shear keys are developed here based on the experimental results by Megally et al. (2001) and Bozorgzadeh et al. (2006). In order to identify the effect of shear key modeling techniques on estimation of deck rotation, which is an important *Engineering Demand Parameter* (*EDP*) for bridges, the proposed models are incorporated into the finite element model of a typical "ordinary" two-span Caltrans bridge with a two-column bent using OpenSees (McKenna and Fenves, 2001). A suite of forty pulse-like near-faults ground motions produced by Baker (2007) is applied at 21 different incident angles to a bridge model matrix featuring 4 skew angles.

BRIDGE DESCRIPTION

A typical Caltrans ordinary bridge, La Veta avenue overcrossing bridge, (Fig. 2) with two spans and two columns bent is modeled in OpenSees (McKenna and Fenves, 2001). This bridge was built in 2001 and is located in the city of Tustin, California, at the intersection of Route 55 and La Veta Avenue (identification number 12-ORA-055-13.2-TUS). Table 1 shows the main characteristics of this bridge.

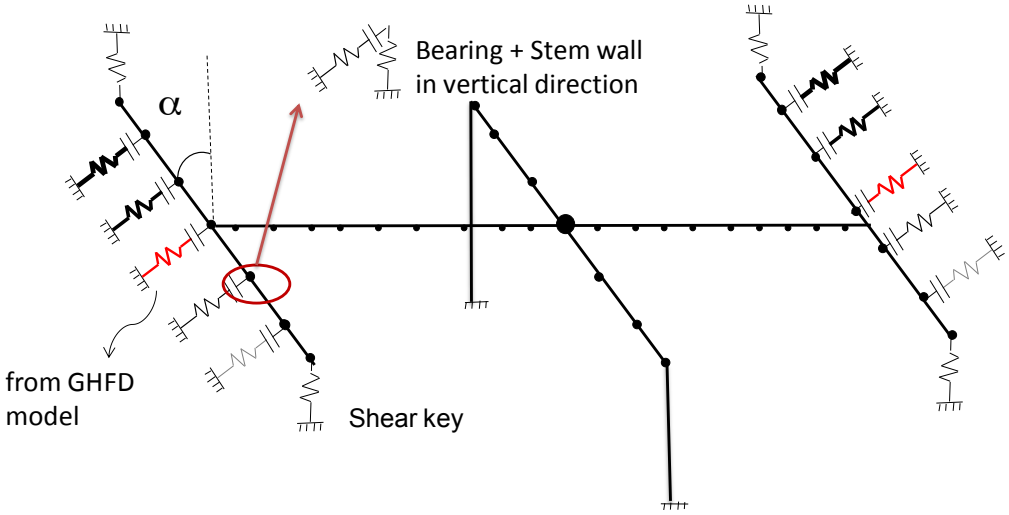

Figure 2. Bridge model considered in this study (based on Kaviani et al., 2012)

Table 1. Geometry and Properties of Bridge

Name of Bridge	La Veta Avenue Overcrossing bridge
Length	300 ft
Number of Spans	2 Span1&2:150 ft
Deck width	75.5 ft
Type of column bents	Two columns
Columns Radius	33.5 in
Column Height	22 ft
Type of Abutments	Seat Abutment

La Veta bridge has a box-girder reinforced concrete deck with seat-type abutments and 7 elastomeric bearings at each abutment. Bridge columns are modeled using lumped plasticity finite element model. In this modeling technique, nonlinear behaviour is captured along the plastic hinges, which are located at the ends of the elastic_linear zone. This model is implemented in OpenSees as `beamWithHinges` element model and updated by modified Gauss-Radau integration (Scott & Fenves, 2006). A steel model, i.e. `ReinforcingSteel`, from the materials library of OpenSees is used to include the effect of bond slip at the column base. The bridge deck is modeled using 10 elastic beam elements per each span, and the boundary condition of column bases are modeled using pinned connections. Effect of soil structure interaction is not considered in this study.

ABUTMENT MODELING

Abutments are modeled based on the abutment model developed by Aviram et al. (2008). In this approach, a rigid bar whose width is the same as the deck width is rigidly connected to the superstructure centreline. A modified version of this model is employed in the current study: the passive resistance of soil normal to the skewed backwall is modeled with five compression-only springs which are uniformly distributed along the skewed width of backwall. The constitutive behaviour of the springs is due to the generalized hyperbolic force-deformation (GHFD) relationship (Khalili-Tehrani et al, 2010). GHFD model accounts for backwall height and material properties of backfill soil in the passive resistance of abutment and its formulation is based on the Log-Spiral Hyperbolic (LSH) model of Shamsabadi et al. (2005, 2007, 2010). Stiffness and ultimate strength of the five distributed springs, which are modeled with the OpenSees `HyperbolicGapMaterial`, decrease linearly from the obtuse to the acute corner of backfill, along the skewed length of backwall as suggested by Kaviani et al. (2012). The amount of variation depends on the abutment skew. A 1" gap between the bridge deck and backwall has been incorporated in the model by shifting the backbone of the compression-only backfill springs.

Stiffness in the vertical direction is modeled with two springs in series consistent with the vertical stiffness of 7 bearings and the bridge stem wall. In the original bridge, 7 elastomeric bearings are designed for each abutment to provide a link between superstructure and substructure. However, in this research PTFE/elastomeric bearings, which are common in Caltrans bridges, are designed with the thickness of 6" (2.4" reinforcement elastomeric bearing pad) and stiffness of 800 kips/in based on the Caltrans memo for designer section 7-1(1994). PTFE sliding bearing by definition is a bearing that carries vertical load through contact stresses between each PTFE sheet, or woven fabricant, and its mating surface that permits movements by sliding of the PTFE over the mating surface (AASHTO LRFD Bridge Design Specification 5th edition, 2010). Generally, PTFE/elastomeric bearings have five components: (1) sole plate, (2) PTFE disk, (3) intermediate plate, (4) elastomeric bearing pad, and (5) masonry plate. In this study it is assumed that bearings are frictionless, and they do not provide any resistance in transverse direction.

SHEAR KEY MODELING

The shear key models developed for this study are data-driven and try to capture shear key failure mechanism. Failure in exterior shear keys can be classified in two mechanisms: (1) sliding shear friction, and (2) diagonal tension. Based on the reinforcement detailing and construction joint between the shear key and the abutment one of the two mechanisms can occur. Typically in the first mechanism, maximum transmitted shear force along the horizontal crack at the shear key interface and the stem wall can be estimated. In this model, shear key behaves as a brittle shear key. Different equations have been proposed using sliding shear model to compute shear force between two faces (Mattock, 1974; Walraven et al., 1987; Caltrans, 1993a; Megally et al., 2001). Megally et al. (2001) defined the nominal capacity of shear key based on the sliding shear friction as shown in Eq. (1).

$$V_n = \frac{\mu_f \cos\alpha + \sin\alpha}{1 - \mu_f \tan\beta} A_{vf} f_{su} \tag{1}$$

Where α is an angle of kinking of the vertical bars with respect to the vertical axis; β is an angle of inclined face of a shear key with respect to the vertical axis; μ_f is a kinematic coefficient of friction for concrete; A_{vf} is the amount of vertical reinforcement connecting the shear key to the abutment stem wall; and f_{su} is an ultimate tensile strength of the vertical reinforcement (Table.2) .

In this study brittle shear keys are defined with tri-linear back bone curve using `Hysteretis` material in OpenSees (see Fig.3). The initial stiffness of the back bone curve is the summation of shear and flexural response of a concrete cantilever with the shear key dimensions. The stiffness of the

hardening and softening part of back bone curve are assumed to be equal to 2.5% of the initial stiffness (Aviram et al., 2008). Maximum capacity of the shear key in this failure mechanism is computed based on the Eq. (1). Since the original shear key reinforcement detailing in the selected bridge makes the shear key work as the ductile shear key, the new reinforcements for sacrificial shear key are designed based on the Caltrans SDC 1.7 (2013).

Table 2. Summary of input variables for shear friction mechanism

Variables	Values
β	65°
α	37°
μ_f	0.36
A_{vf}	10 in^2

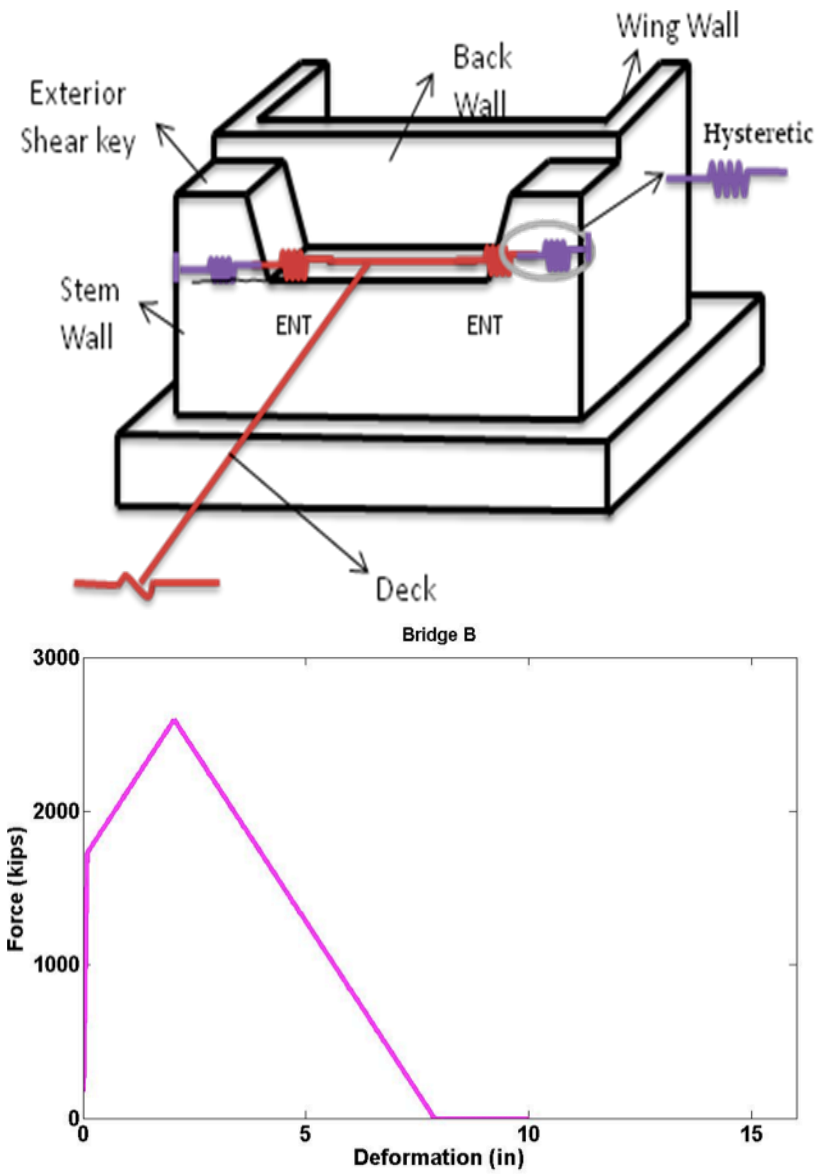

Figure 3. Shear key model based on the sliding shear friction failure mechanism (a) Brittle shear key model, (b) Force-deformation relationship for brittle shear key

Formation of cracks in the diagonal tension failure mechanism is different from sliding shear friction mechanism. After applying the lateral force, horizontal cracks initiate at the interface of the

shear key and the stem wall. These cracks propagate until they reach the first row of the vertical reinforcement of shear key. By increasing the lateral force and yielding of the shear key reinforcements, multiple inclined cracks propagate toward the bottom of the stem wall. This failure mechanism results in developing of a strut-and-tie mechanism (Megally et al., 2001; Bozorgzadeh et al., 2004) for ductile shear keys. Experimental results showed that lack of smooth construction joint and location of the vertical reinforcements connecting the shear key to the stem wall can alter the behaviour of the shear key from sacrificial to ductile shear key. According to Meally et al. (2001) shear key capacity for ductile shear keys can be obtained from the combination of two contributions as shown in Eq. (2).

$$(V_{n,s}) = V_c + V_S \tag{2}$$

Where V_c and V_s are the concrete and reinforcing steel contributions to shear resistance, respectively. In this paper, the contribution of reinforcing steel is simulated using the modified Ibarra-Medina-Krawinkler deterioration model with bilinear hysteretic response (`Bilin`) material model, and concrete contribution is modeled by a combination of elastic-perfectly plastic gap material (`EPPG`), and `Concrete02` material in serries with elastic no-tension material(`ENT`) Fig. 4 shows the shear key model and the force-deformation back bone curve of the ductile shear key for La Veta brige.

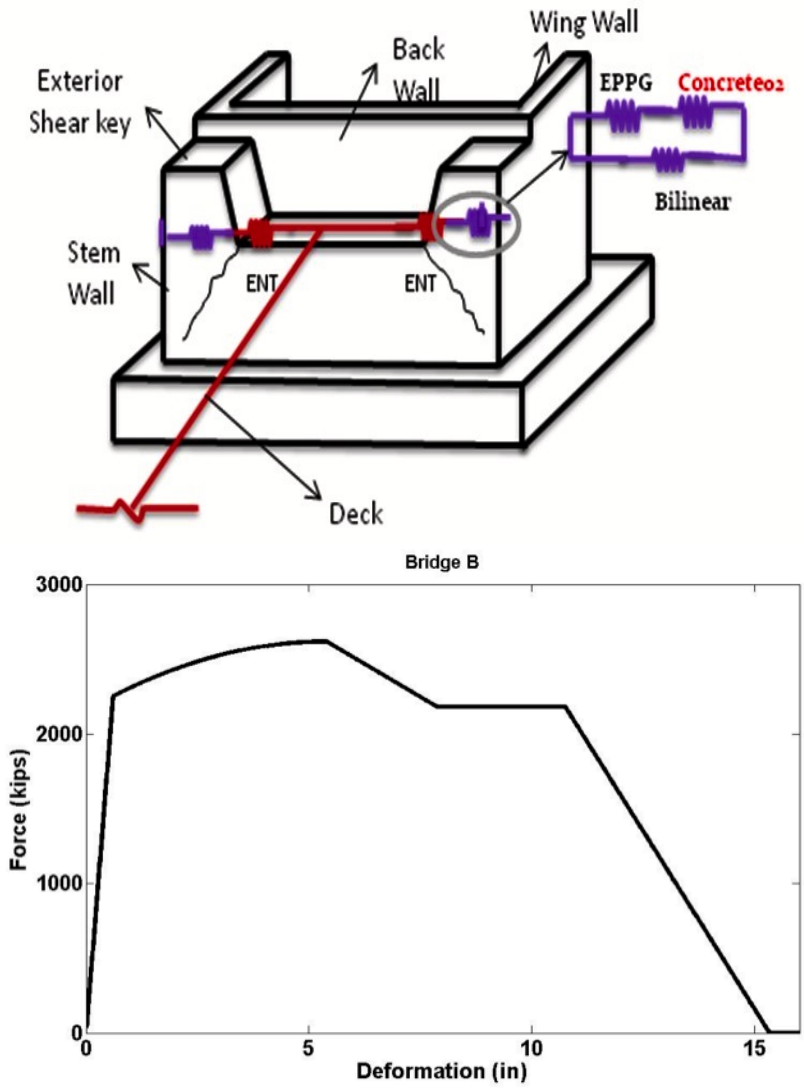

Figure 4. Shear key model based on the strut-and-tie failure mechanism (a) Ductile shear key model, (b) Force-deformation relationship for ductile shear key

BRIDGE RESPONSES FOR TWO SHEAR KEY MODELING TECHNIQUES

This investigation evaluates the seismic response of selected bridge subjected to 40 sets of pulse-like ground motions to experience near-field directivity with two horizontal components, and 21 different incident angles. These 40 sets of ground motions are selected and scaled using the procedure outlined in Baker (2007) (see Fig.5).

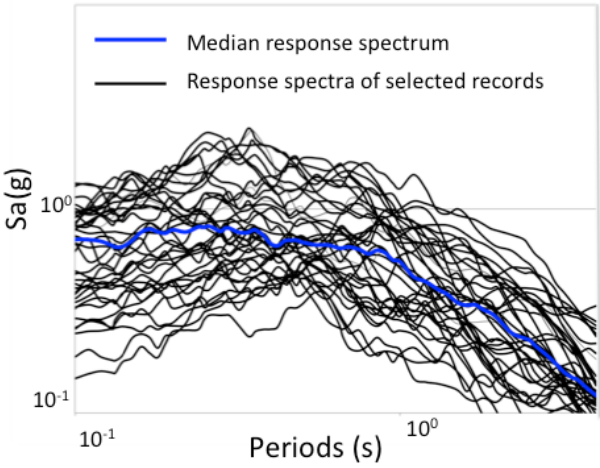

Figure 5. Geometric mean response spectra(SN/SP components)for selected pulse-like records (Shahi and Baker, 2011)

The *EDP* of interest in this study is the deck rotation. Collapse is defined as either the column drift ratio larger than 8% (Hutchinson et al., 2004), or the deck displacement relative and perpendicular to the abutment is larger than the seat width (i.e., 30").

The nonlinear response of the selected bridge with two shear key models and 4 different skew angles, a total of 8 models, is obtained using the OpenSees (McKenna and Fenves, 2001). Fig. 6 demonstrates the general trend of the bridge deck rotation versus peak ground acceleration (PGA) with four different skew angles, 21 ground motion incident angles, and two different shear key models. Generally, the bridge with the larger skew angles experiences larger deck rotation for both shear key models. The results show that the deck rotation is sensitive to the shear key modeling, a bridge with a brittle shear key has a larger deck rotation.

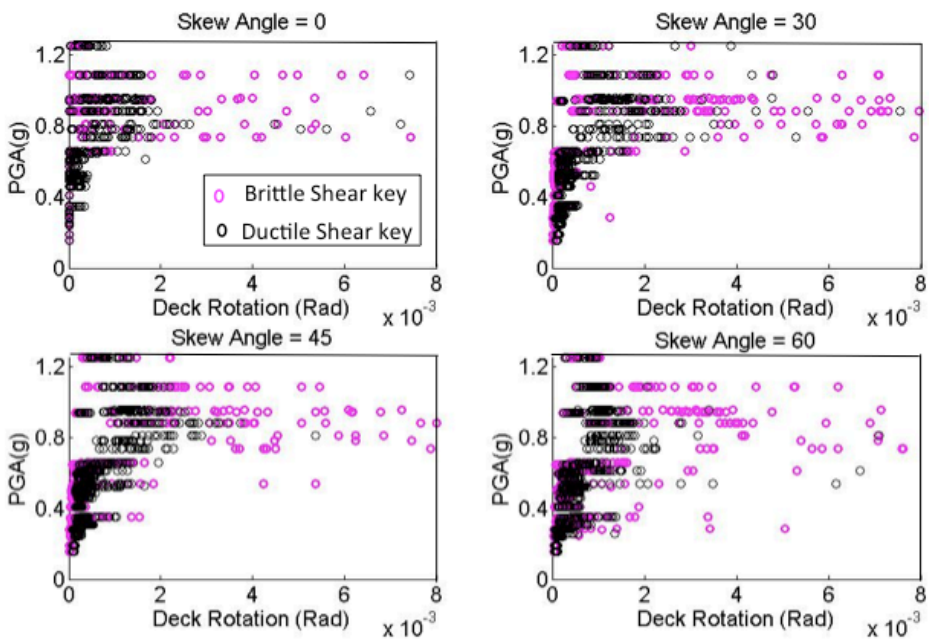

Figure 6. Deck rotation versus peak ground acceleration for selected bridge

To further investigate the effect of shear key behaviour on deck rotation we direct attention to probability distribution function of *EDP*. We assume the probability distribution function of *EDP* can be described as a lognormal distribution with three parameters: $E\widehat{D}P|NC$, $\sigma_{lnEDP|NC}$, and $P(\mathrm{C})$, that are median of *EDP* conditioned on no-collapse, dispersion of natural logarithm of *EDP* conditioned on no-collapse, and probability of collapse, respectively (see Fig.7). In order to better understand and evaluate the effects of shear key behavior on *EDP*, the median of *EDP*, $E\widehat{D}P$, values for both brittle and ductile shear keys are compared. Fig.8 shows the median deck rotations for 4 skew angles. The results illustrate that shear key modeling mostly affect bridges with larger skew angle. Additionally, the bridge with brittle shear keys illustrates larger deck rotation subjected to stronger motions with larger skew angles.

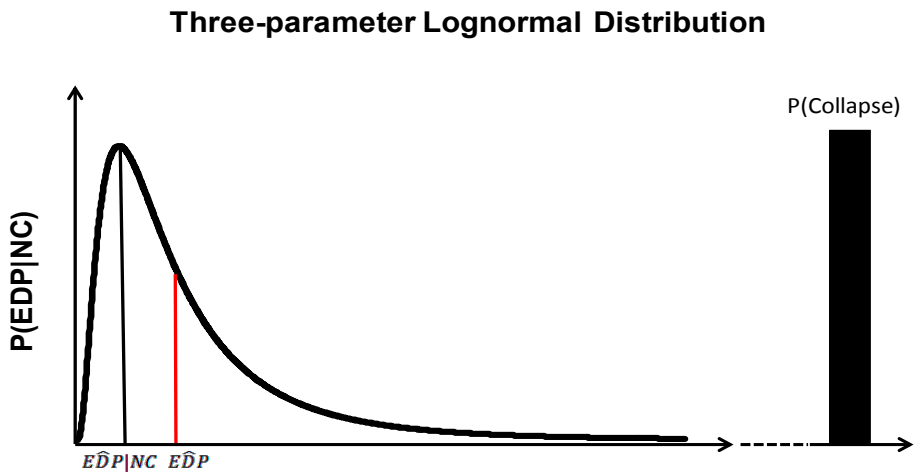

Figure 7. Three parameter lognormal distribution representing statistics of bridge deck rotation

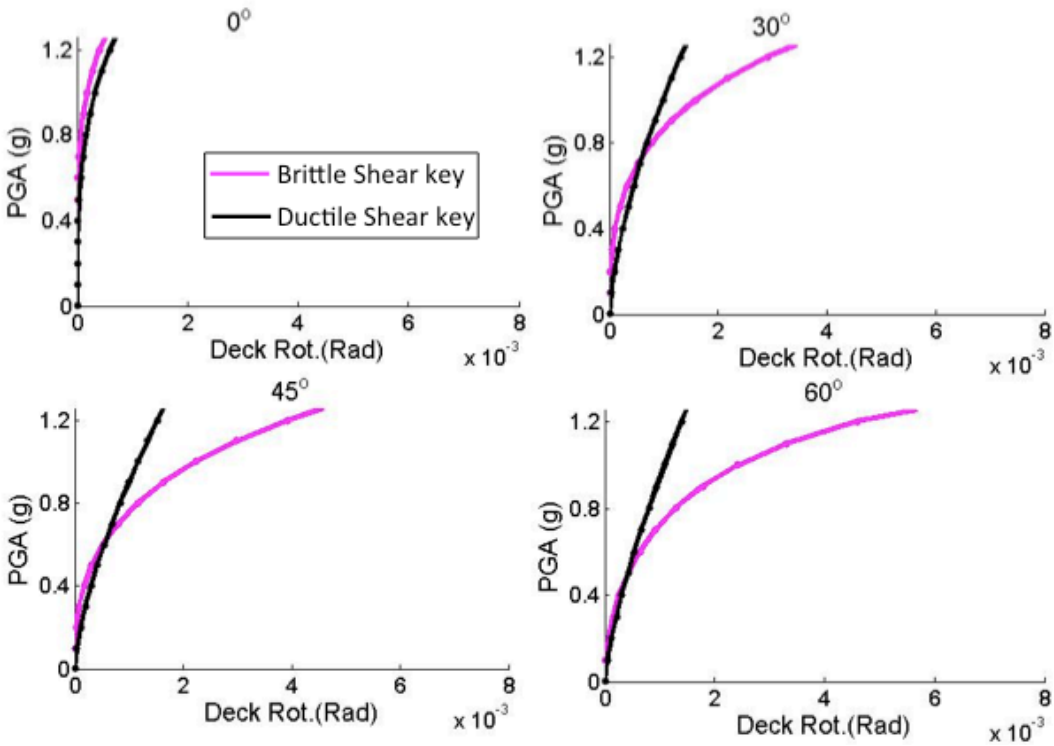

Figure 8. Median deck rotation given IM for 4 different skew angles

CONCLUSIONS

This paper describes the effect of two shear key modeling techniques (i.e., ductile and brittle) on deck rotation of skewed bridges; four different skew angles are considered in this study. A typical two-span seat type abutment bridge modeled in OpenSees is used for this purpose. The selected bridge is subjected to a set of 40 pulse-like ground motions in two horizontal directions with 21 different incident angles. The results show that the bridge responses with two different shear keys are different but comparable. These differences are more predominant in bridges with larger skew angles. In general, the selected bridge with the brittle shear keys experienced larger deck rotation.

ACKNOWLEDGEMENTS

This study is based on work supported by Caltrans under Grant 65A0391. This financial support is gratefully acknowledged. Any opinions, findings, and conclusions or recommendations expressed in this paper are those of the authors and do not necessarily reflect the views of sponsors.

REFERENCES

AASHTO (2010) AASHTO LRFD Bridge Design Specifications, American Association of State Highway and Transportation Officials, 5th edition, Washington, DC

Aviram A, Mackie K R, and Stojadinovic B (2008) Guidelines for Nonlinear Analysis of Bridge Structures in California, Technical Report 2008/3, Pacific Earthquake Engineering Research Center, University of California, Berkeley, CA

Baker J W (2007) "*Quantitative Classification of Near-Fault Ground Motions Using Wavelet Analysis,"* Bull Seismol Soc Am, 97(5): 1486-1501

Bozorgzadeh A, Megally S, Restrepo J, and Ashford S A (2006) "*Capacity evaluation of exterior sacrificial shear keys of bridge abutments,*" Bridge Engineering 555–565

Bozorgzadeh A, Megally S, and Restrepo J (2004) Seismic Response of Sacrificial Exterior Shear Keys in Bridge Abutments: Recommended Design and Construction Details, Rep. to CALTRANS, Contract No. 59A0337, Dept. of Structural Engineering, UC San Diego, San Diego

California Department of Transportation CALTRANS (2013) Seismic design criteria: Version 1.7 Sacramento California

Fell B V, and Salveson M W (2013) "*Parametric Simulation of Nonlinear Shear Key Behavior on the seismic response on Bridge Abutments,*" Bridge Engineering, January 5, doi: 10.1061/ (ASCE) BE.1943-5592.0000465

Goel R K and Chopra A K (2008) *"Role of Shear Keys in Seismic Behavior of Bridges Crossing Fault-Rupture Zones,"* Journal of Bridge Engineering 13.4 398-408

Hutchinson T C, Chai Y H, and Boulanger R W (2004) "*Inelastic Seismic Response of Extended Pile-shaft-supported Bridge Structures,*" Earthquake Spectra, 20(4): 1057-1080

Kaviania P, Zareian F, Taciroglu E (2012)*" Seismic behavior of reinforced concrete bridges with skew-angled seat-type abutments,"* Engineering Structures, 45, 137–150

Khalili-Tehrani P, Taciroglu E, Shamsabadi A (2010) *"Backbone curves for passive lateral response of walls with homogeneous backfills,"* Soil-Foundation-Structure Interaction (Orense, Chouw, and Pender, eds.) University of Auckland, New Zealand, vol. 2, pp 149-154

Mattock A H (1974) Shear transfer in concrete having reinforcement at an angle to the shear plane, Shear in Reinforced Concrete, ACI Special Publication 42, 17–42

McKenna F, Fenves G L, and Scott M H (2000) Open System for Earthquake Engineering Simulation University of California Berkeley, California

Megally S, Silva P F, and Seible F (2001) Seismic Response of Sacrificial Shear Keys in Bridge Abutments, Report No. SSRP–2001/23, University of California, San Diego,CA

Moehle J, Fenves G, Mayes R, Priestley M J N, Seible F, Uang C M, Warner S, and Aschheim M (1995) Northridge earthquake of January 17, 1994, reconnaissance report, Earthquake Spectra, Ch. 6, v.1

Priestley M J N, Seible F, and Uang C M (1994) The Northridge earthquake of January 17, 1994: Damage analysis of selected freeway bridges Structural Systems Research Rep. SSRP-94/06, Dept. Of Structural Engineering, Univ. of California San Diego, La Jolla, California

Shahi S K, and Baker J W (2011) *"Regression models for predicting the probability of near-fault earthquake*

ground motion pulses, and their period," 11th International Conference on Applications of Statistics and Probability in Civil Engineering, Zurich, Switzerland, 8p

Scott M H, and Fenves G L (2006) "*Plastic hinge integration methods for force based beam-column elements,*" Structural Engineering, ASCE, 132:2, 244-252

Shamsabadi A, Ashour M, and Norris G (2005) *"Bridge Abutment Nonlinear Force-Displacement-Capacity Prediction for Seismic Design,"* Geotechnical and Geoenvironmental Engineering, Vol. 131, No. 2, pp.1-9

Shamsabadi A, Rollins K M, and Kapuskar M (2007) "*Nonlinear soil–abutment–bridge structure interaction for seismic performance-based design*," Journal of Geotechnical and Geoenvironmental Engineering, 133(6): 707-720

Shamsabadi A, Khalili-Tehrani P, Stewart JP and Taciroglu E (2010). "*Validated simulation models for lateral response of bridge abutments with typical backfills*," ASCE Journal of Bridge Engineering, 15(3), 302-311