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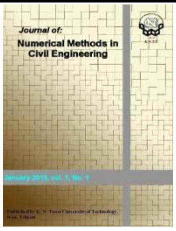

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Investigating Importance of Friction between Elastomer Bearing and Its Support on the Seismic Performance of Skew Seat-type Bridges

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Abstract:

Seat-type bridges compose a large portion of bridge inventories in different countries. There is evidence of excellent performance of these bridges in Iran during past earthquakes, that could be attributed to the slip of elastomeric bearings. This study investigates how the coefficient of friction between elastomeric bearing and its support and skew angle of the bridge could affect the performance of seat-type bridges. This assessment is done using incremental dynamic analyses on a three-span model bridge. The finite element model accounts for the slip of bearings, backfill passive resistance, and plastic deformation of columns. The results show that while the skew angle predominantly affects the required seat width in different codes, the correlation between the required seat width and coefficient of friction is much stronger. It is also established that, considering the mean response, there is no possibility of unseating even for maximum considered level ground motion. At the same time, the possibility of a loss of access due to abutment displacement is quite probable even for a design-basis earthquake. Furthermore, it is shown that the slip of the bearings significantly reduces seismic demand on the substructure. The findings show the paramount importance of modeling bearing slip in any seismic assessment of these bridge types.

1. Introduction

Skew seat-type bridges compose a large portion of bridge inventory in Iran and other countries. Typical approach in assessing the seismic performance of these types of bridges does not account for the slip of the elastomeric bearing in the event of large earthquakes. Excellent performance of these types of bridges in the past earthquakes in Manjil-Rudbar Iran 1990 earthquake [1] and Bam 2003 Iran earthquake [2] substantiates this research. There are two distinct points about the construction practice in Iran, 1) lack of any connection between elastomer and superstructure/substructure, 2) lack of any positive connection between superstructure and substructure. Accounting for the slip could eliminate the need for seismic retrofit of a large portion of the existing bridge inventory. The seismic response of skewed bridges is evident from the unseating of the bridge deck leading to collapse.

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Yashinski et al. [3] and Kawashima et al. [4] reported frequent cases of unseating and the collapse of skewed bridges in the Maule Chile 2010 earthquake. They attributed this failure to different factors, including lack of strong shear keys and diaphragms at supports and large rotation of the bridge deck.

Different researchers considered the seismic performance of skewed bridges that were constructed following California (Caltrans) practice, where there are integral columns, and seat-type bearings are only used on the abutments. Meng and Lui [5] simulated seismic performance of the Foothill Boulevard undercrossing that suffered heavy damage during the San Fernando 1971 Earthquake. They found that the flexibility of the bridge deck, boundary conditions of columns, and skewness, played a dominant role in the seismic performance of the bridge. Kaviani et al. [6] developed the fragility of two-span bridges with equal and unequal span length and skew angles from 0 to 60 degrees. Ghotbi [7] investigated the importance of accounting for soil structure interaction (SSI) in seismic assessment of skewed bridges. He found that accounting for SSI could have a decreasing or increasing effect on the different response parameters.

Omrani et al. [8] investigated the effect of epistemic uncertainty on the performance of two-span bridges with seat-type abutment and integral bent cap. They found strong coupling between the assumed model of backfill and shear keys for bridges with moderate to high skew angles. This observation is mainly due to the well-known interaction between shear keys transverse force at cut corners of abutment and backfill longitudinal force, that was reported by Kawashima et al. and Wu [9]. To reduce this coupling, Wu suggests using a larger gap in abutment expansion joints. Considering this Filipov et al. [10] recommended an increase in the design force of the retainers to avoid unseating in skew bridges. This finding shows how assumptions regarding the design force of shear keys/side retainers could affect the seismic performance of bridges.
Several researchers/institutes propose adopting frictional response of unbonded elastomeric bearings as a device to reduce seismic demand on the substructure. Kelly and Konstantinidis [11] and Konstantinidiis et al. [12] investigated the cyclic response of unbonded elastomeric bearings. They found a stable hysteretic response and no tearing in the elastomer for distortional deformation up to 200%. Extensive research in this regard resulted in IDOT's [13] three-tier seismic design strategy. This seismic design strategy includes the following lines of defense, 1) fusing action of connection force (side retainers and dowels), 2) providing adequate seat width to avoid unseating, 3) utilizing elements plastic hinging as the last resort to dissipate earthquake energy.
While the frictional response of elastomeric bearing is the main component of this design strategy, there are diverse attitudes regarding coefficient of friction (CoF) by provisions of different codes or institutes. AASHTO [14] proposes elastomeric bearing with sole plate and without masonry plate (connected only to superstructure), without any positive connection between superstructure and substructure and assuming CoF of 0.2. AASHTO allows the design of elastomeric bearings as sacrificial elements and allows slipping of the bearing in strong ground motions. Chinese Guidelines for Seismic Design of Highway Bridges [15] do not allow for the slip in the bearings and assumes a CoF of 0.15. IDOT allows for slipping of the bearings but does not specify the value of CoF that should be used in any seismic assessment.
FHWA [16] requirement for minimum seat width (Nd) is

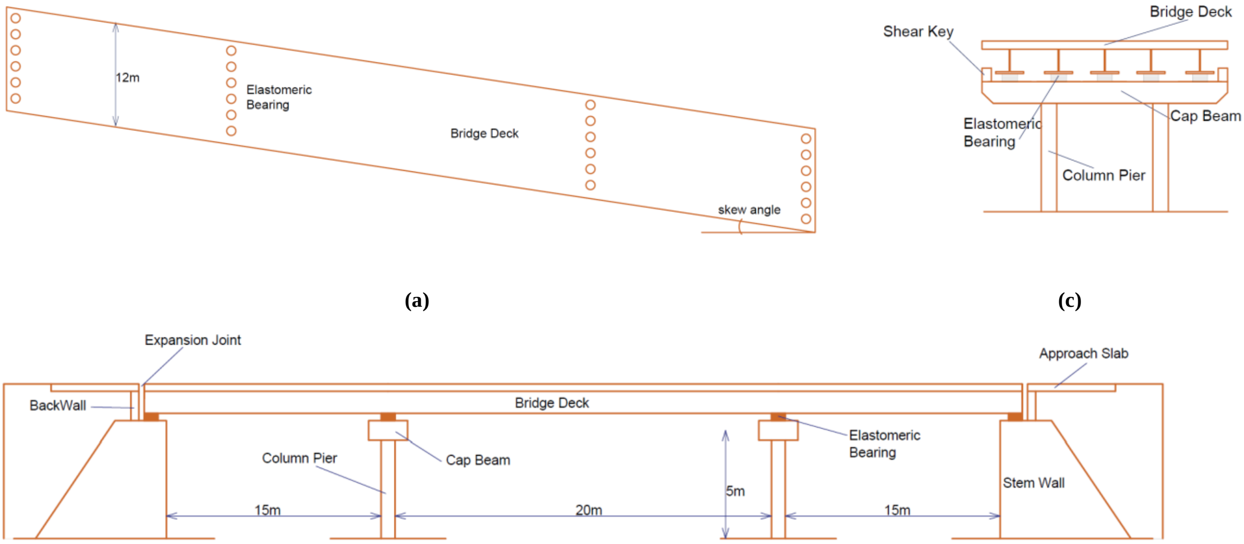
$$
\begin{aligned}
&N_d = [100 + 1.7L + 7.0H + 50P]Q \\
&P = \sqrt{[1 + 4(\min\{B/L, 3/8\})^2]H} \qquad (1) \\
&Q = (1 + 1.25S_1)/cos\alpha
\end{aligned}
$$

where L is the distance between joints, H is pier height, B is deck width, α is skew angle, and S_1 is anticipated spectral response acceleration at a period of 1 s.

The seating width requirement of AASHTO or FHWA does not account for different construction types. They use the same equation for bridges with integral bent cap and also for seat-type bridges. It is well known that integral bent cap could be effective in reducing seismic displacement demand, and for this reason, California practice is mainly based on integral bent caps. At the same time, there are a large number of bridges with seat-type bearings, and their displacement demand is anticipated to be greater than that for bridges with integral bent caps. This could imply that there is a need for improvement in the evaluation of the seating width requirement for the bridges with seat-type bearings.
Considering the seismic performance of abutments, its failures in tilting or shifting are not as critical as large longitudinal displacement of the abutment that leads to loss of access, which is highly probable in the case of seat-type bridge. FHWA [16] evaluating seismic retrofit efficiency, imposes limits on the abutment transverse and longitudinal displacement of 75 and 150 mm, respectively. These displacements are much smaller than seat width requirements and are more likely to control the seismic design. AASHTO only considering structural collapse, ignores the loss of access in its evaluation of seismic design adequacy.
This paper investigates how the performance of skew seat-type bridges could be affected by skew angle and also the coefficient of friction (CoF) between elastomeric bearings and their concrete support. The engineering demand parameters (EDPs) that are studied include required seat width to prevent unseating, the possibility of a loss of access due to large abutment displacement, and plastic rotation demand in the substructure. Incremental dynamic analysis (IDA) is used to evaluate the bridge response at different demand levels including design-basis earthquake (DBE) and maximum considered earthquake (MCE) levels. The evaluations are done for a combination of different CoF and skew angles. In the following sections, after presenting the model bridge, the development of the finite element model for the model bridge is described, then ground motions (GMs) used in the analyses are introduced. Finally, the performance of the model bridge is evaluated using IDA.

2. Case Study

The model bridge is a 50 m long three span (15/20/15 m) continuous superstructure with a width of 12 (Figure 1). It includes two bents, each of which has two circular columns. The main design parameters of the bridge are given in Table 1. The elastomeric bearings have no masonry or sole plates. No dowels are used to connect the deck to the column cap. Shear keys are located on the bent and also on the abutment. Ignoring rotation in the foundation, the model assumes a rigid foundation.



(b)

Fig. 1: The model bridge, a) plan, b) longitudinal view, c) transverse section.

Table. 1: Bridge main design parameters

Materials/Elements	Dimensions/Properties		
Concrete	f_c'=40 MPa		
Rebars	Yield stress 400 MPa, Ultimate Strength 500 MPa		
Deck	I shaped girder with 200 mm thick concrete deck and 50 mm overlaying asphalt		
Deck Height	1700 mm		
Column	Diameter: 1200 mm Height: 5000 mm	Longitudinal bars: 30T28 (rebar dia.=25 mm)	Transverse Spirals: T 14 @ 75 mm
Cap Beam	Height: 1200 mm Width: 1500 mm	$I_x = 0.216\ m^4$	$I_y = 0.337\ m^4$
Abutment Back Wall	Width: 12000 mm Height: 1700 mm Thickness: 150 mm	Longitudinal bars: T 16 @ 300mm	Transverse bars: T 16 @ 300mm
Expansion Joint Width at Abutments	50 mm		
Elastomeric Bearing	Length: 300 mm Width: 300 mm Height: 100 mm	Elastomer: 8 layers Layer thickness : 10 mm	Steel Shims: 7 layers Layer thickness: 3 mm
Fundamental Period Based on Elastic Model	0.86 sec		
Design Spectral Acceleration	0.875 g		

3. Finite Element Modeling

The finite element analyses are done employing OpenSees [17]. There are different sources of nonlinearity in the response of seat-type bridges subjected to strong ground motions, which include

- Frictional response of slipping elastomeric bearings
- Development of plastic hinges in the bent columns
- Failure of backwall and hysteretic passive resistance of the backfill

Column caps due to capacity-protected design procedure of AASHTO, essentially remain elastic. Different elements and materials in the OpenSees library are employed to model slip in elastomeric bearings, nonlinear deformation in columns, and backfill passive resistance, which are described in details in the following section.

Correct modeling of the slip of elastomeric bearings has paramount importance in the seismic assessment of seat-type bridges. To model frictional and distortional response of elastomeric bearings, zero-length *flatSliderBearing* element that models Coulomb friction is combined with *elastomericBearingBoucWen*, *axialSP*, and elastic stiffness, to model distortional/axial/rotational responses of the elastomeric bearing. In the calibration of *elastomericBearingBoucWen* parameters, default values of parameters are used, except for hysteretic shape parameter $beta for which 0.9 is used.

To model nonlinear flexural deformation in the columns, the *nonlinearBeamColumn*, a forced-based element, is deployed. This element accounts for the propagation of nonlinear deformation along the element length by evaluating the sectional response at distinct Gauss points (5 points in this study). Sectional discretization includes six radial and 24 circumferential divisions. Steel rebars are modeled using uniaxial material *ReinforcingSteel*. Concrete confinement is modeled using modified Mander model [18] and employing uniaxial material *concrete*02.

Capacity-protected column caps are modeled using *elasticBeamColumn* element with section properties given in Table 1.

Large abutment displacement could limit access to the bridge. To have a reasonable estimate of the abutment displacement, it is of prime importance to consider the interaction between abutment and backfill. In the case of a large earthquake, large longitudinal displacement of deck results in the shear failure of the back wall of the abutment and mobilization of passive resistance of backfill. In other words, nonlinear deformation includes brittle shear failure of backwall and hysteretic passive response of backfill. This could have a significant impact on the longitudinal response of the bridge, especially for short to medium-length bridges. Backwalls similar to shear keys, are considered sacrificial elements, and similarly, their seismic response is brittle. The back wall and stem wall are modeled using *elasticBeamColumn* element, and the connection of the back wall to the stem wall is modeled using the zero-length element with brittle shear failure implementing *ElasticPPGap* uniaxial material property. Backfill passive response is modeled implementing *twoNodeLink* element with a uniaxial material property of *QzSimple*1 simulating soil passive response. The strength of the backfill is evaluated using the approach of CALTRANS [19]. Figure 2 shows the typical hysteretic response of backfill.

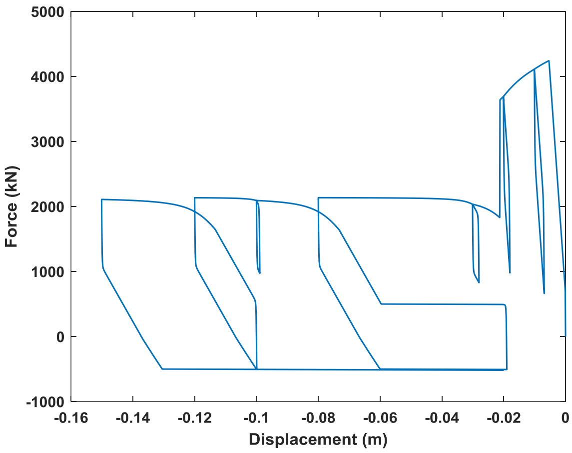


Fig. 2: Spectrum of the selected ground motion compared with target and mean spectrum

4. Ground Motions and Incremental Dynamic Analysis Procedure

Incremental dynamic analysis provides a means to evaluate the adequacy of the seismic response for different levels of ground motion intensities. To evaluate the seismic performance of the model bridge, incremental dynamic analyses are carried out employing GMs of Table 2, which include ten records on soil class D of AASHTO with a magnitude between 6.5 to 7.5. The selection of GMs is done utilizing PEER Ground Motion Database Machine and assuming epsilon of 2.0 at bridge first mode period (https://ngawest2.berkeley.edu).

Table. 2: List of ground motion records considered in incremental dynamic analyses

Earthquake	Year	Station	Record No.	Magnitude	R (km)	V_{s30} (m/sec)
Northern Calif-03	1954	Ferndale City Hall	20	6.5	26	219
San Fernando	1971	LA - Hollywood Stor FF	68	6.6	23	316
Tabas, Iran	1978	Boshrooyeh	138	7.4	24	324
Imperial Valley-06	1979	Calexico Fire Station	162	6.5	11	231
Corinth, Greece	1981	Corinth	313	6.6	10	361
Superstition Hills-02	1987	Calipatria Fire Station	720	6.5	27	205
Loma Prieta	1989	Agnews State Hospital	737	6.9	25	239
Landers	1992	Coolwater	848	7.3	20	352
Niigata, Japan	2004	NIG022	4212	6.6	18	193
Chuetsu-oki, Japan	2008	Joetsu City	4853	6.7	26	294

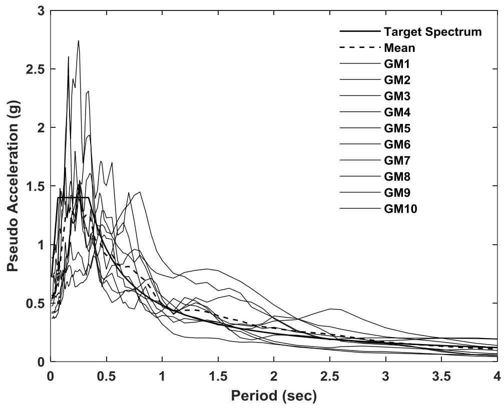


Fig. 3: Spectrum of the selected ground motion compared with target and mean spectrum

First, using the least square method, a single scale factor is applied on all of the GMs such that, the mean spectrum of the GMs fit the target spectrum that is a design spectrum with a 1000 years return period. The scaled GMs are shown in Figure 3. This set of modified GMs has a mean spectrum approximating the design spectrum (DBE earthquake). To account for different ground motion intensities including service and MCE level ground motions, a scale factor from 0.25 to 1.75 in steps of 0.125, is applied on the records. Noting that design spectrum has a 1000 years return period, for MCE level ground motions, the scale factor will be about 1.25.

5. Results

There are three main engineering demand parameters (EDPs) affecting the seismic response of the seat-type skewed bridges

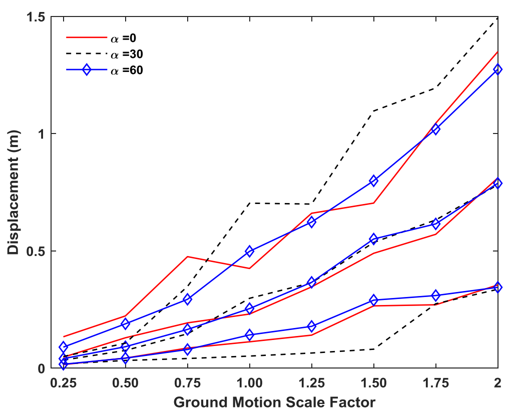
- Possibility of unseating
- Possibility of loss of access
- Adequacy of seismic detailing of the substructure in terms of rotational demand and capacities of columns

The main parameters affecting the seismic response in this study include the coefficient of friction of elastomeric bearings and skew angle of the bridge. Models considered in this study are introduced in Table 3, which includes CoF of 0.2, 0.4, and 0.8, together with skew angles of 0, 30, and 60 degrees. In the following, the effect of these parameters on the EDPs of interest will be evaluated.

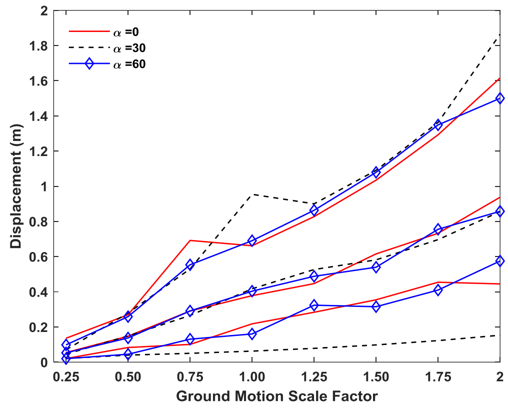
Table. 3: Models considered in the study

Notation	Coefficient of friction	Skew angle
F2S0	0.2	0
F2S30	0.2	30
F2S60	0.2	60
F4S0	0.4	0
F4S30	0.4	30
F4S60	0.4	60
F8S0	0.8	0
F8S30	0.8	30
F8S60	0.8	60

Design ground motion intensity based on the AASHTO requirement corresponds to 1000 years return period (Design-Basis Earthquake or DBE). The intensity of MCE could be approximated by 1.25 times the intensity of DBE. Figure 4 gives the evolution of the maximum deck displacement at the obtuse corner for increasing ground motion intensities in the longitudinal and transverse directions. Results are for CoF of 0.2 and different skew angles (F2S0, F2S30, and F2S60). Due to the presence of backfill passive resistance, longitudinal displacements are smaller than transverse ones. There is no apparent change in the pattern of displacement magnitude for different skew angles. The same pattern is also observed for CoF of 0.4 and 0.8.



(a)



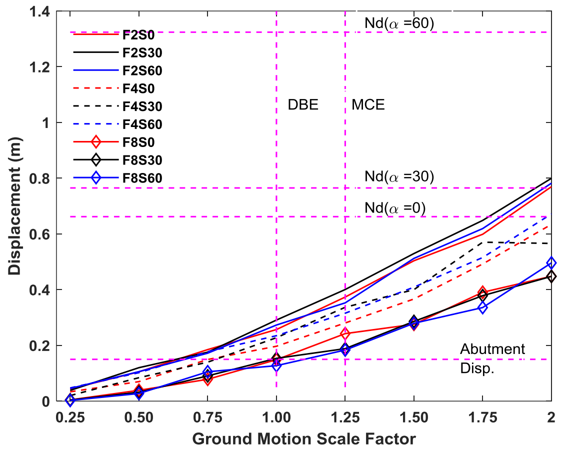
(b)

Fig. 4: Evolution of deck obtuse corner displacement with increasing intensity of GMs, including minimum, maximum and mean displacements (models F2S0, F2S30, F2S60), a) longitudinal direction, b) transverse direction

Following the approach adopted by the current performance-based design codes (e.g., ASCE 7-16), and in the view of better accuracy in the evaluation of mean response compared with its dispersion, in the following, assessment of the deck displacement is done by only considering the mean value of response parameters. Figure 5 compares the evolution of the mean displacement of the deck center rather than the maximum deck corner displacement for increasing intensities of GMs. The figure evaluates the impact of friction and skew angle on the seismic response. Also shown in the figure is the required seat width as required by FHWA (Equation 1) and also FHWA limitation on abutment displacement to avoid loss of access. The figure shows that change in the skew angle has a negligible impact on the deck displacement compared to friction. The correlation between deck displacement and CoF is much stronger than that for the skew angle.

The interesting point is that, change in the seat width requirement of FHWA has a l significant variation for different skew angles, and does not account for possible variation in the coefficient of friction. Considering Figure 5a for the different coefficients of frictions and skew angles, there is no possibility of unseating even for MCE level earthquakes. However, loss of access even for earthquake magnitudes smaller than DBE is quite probable. Noting that lower-level earthquake in FHWA to control of service condition has an intensity around 50% of DBE. It could be concluded that at least for this earthquake magnitudes, there is no possibility of a loss of access.

Another response parameter of interest is the lateral drift of the column top at the level of column caps. Figure 6 depicts the change in the mean column drift (column top displacement divided by column height) versus GMs' scale factor. Drift ratios are well below the typical capacity of ductile columns (e.g. [16] and [20]), an indication of essentially elastic substructure. Slip of bearings results in quasi-isolation of the bridge, resulting in negligible nonlinear deformation in the substructure. Energy dissipation mechanisms include friction due to the slip of elastomer bearings, and also backfill passive resistance. This complies with the observed seismic performance of these types of bridges in past earthquakes in Iran.



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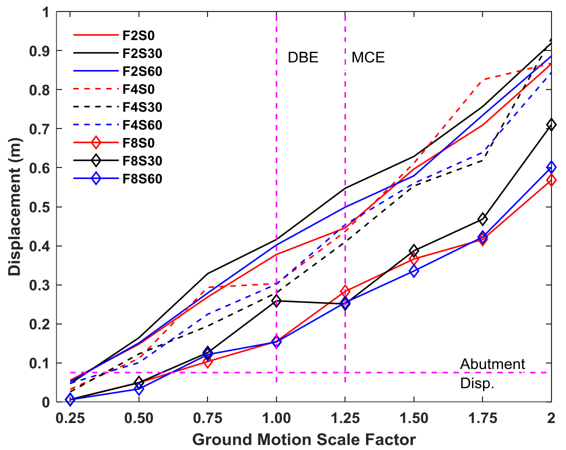

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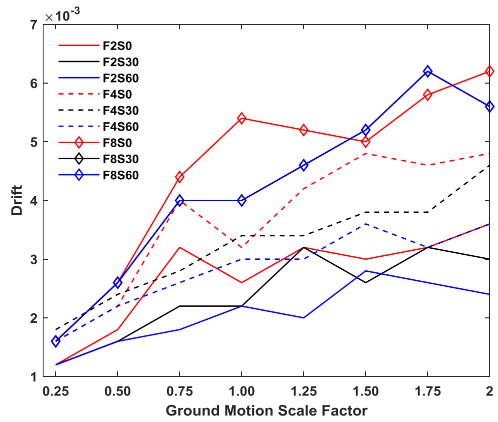
Fig. 5: Mean deck displacement evolution with ground motion intensity, a) longitudinal direction, b) transverse direction



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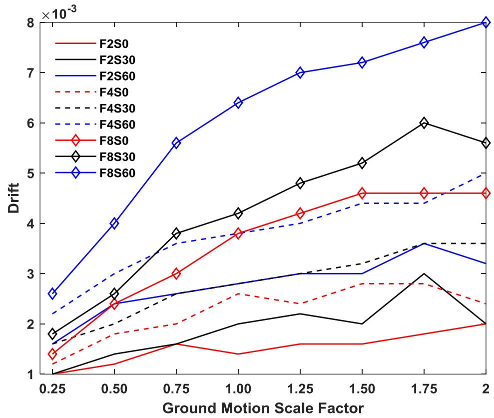

(b)

Fig. 6: Mean column drift of different models, a) longitudinal direction, b) transverse direction

6. Conclusion

This study investigates the slip of elastomer bearings and skew angle on the seismic performance of seat-type skew bridges. Engineering parameters of interest include deck and abutment displacement that could lead to unseating and loss of access, and the evolution of rotational demand in the substructure. While the codes' requirement to avoid unseating is mainly dependent on the skew angle, the results indicate that the change in the coefficient of friction between elastomeric bearing and supporting concrete is more important than the skew angle. It is also found that mean deck displacement after initiation of slip does not lead to unseating, however, the loss of access due to large abutment displacement is quite probable for hazard intensity in the level of design-basis earthquake. Furthermore, it is established that the occurrence of slip significantly reduces ductility demand in the substructure. The results show -notable- enhancement in the seismic performance of seat-type bridges when accounting for bearing slip.

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