Seismic Response of Skewed RC Box-Girder Bridges

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ABSTRACT :

Seismic vulnerability of highway bridges remains an important problem and has received increased attention as a consequence of unprecedented damage observed during several major earthquakes. A significant number of research studies have examined the performance of skewed highway bridges under service and seismic loads. Nonetheless, there are no detailed guidelines addressing the performance of skewed highway bridges. Several parameters affect the response of skewed highway bridges under both service and seismic loads which makes their behavior complex. Therefore, there is a need for more research to study the effect of skew angle and the other relating factors on the performance of highway bridges. This paper examines the seismic performance of a three-span continuous concrete box girder bridge with skew angles from 0 to 60 degrees, analytically. The bridge was modeled using finite element (FE) and simplified beam-stick (BS) using SAP2000. Different types of analysis were considered on both models such as: nonlinear static pushover and linear and nonlinear time history analyses. A comparison was conducted between FE and BS, different skew angles, abutment support conditions, and time history and pushover analysis. It is shown that BS model has the capability to capture the coupling due to skew and the significant modes for moderate skew angles. Boundary conditions and pushover load profile are determined to have a major effect on pushover analysis. Pushover analysis may be used to predict the maximum deformation and hinge formation adequately.

KEYWORDS: Highway bridge, seismic response, skew, analysis, modeling

1. INTRODUCTION

Many advances have been made in developing design codes and guidelines for static and dynamic analyses of regular or straight highway bridges. However, there remains significant uncertainty with regard to the structural system response of skewed highway bridges as it is reflected by the lack of detailed procedures in current guidelines. In fact, as evidence by past seismic events (i.e. 1994 Northridge – Gavin Canyon Undercrossing and 1971 San Fernando – Foothill Boulevard Undercrossing), skewed highway bridges are particularly vulnerable to severe damage due to seismic loads. Even though a number of studies have been conducted over the last three decades to investigate the response characteristics of skewed highway bridges under static and dynamic loading, research findings have not been sufficiently comprehensive to address global system characteristics. Due to the fact that the current seismic design guidelines do not provide explicit procedures, a significantly large number of bridges are at risk with consequential threat to loss of function, life safety, and economy. Many of the existing bridges may be prone to earthquake induced damage and may require substantial retrofit measures to achieve desired seismic performance and post-earthquake serviceability. Researchers and practicing design engineers need to fully understand the overall system response characteristics of skewed highway bridges for the proper detailing of system components.

It is generally agreed that bridges with skew angles greater than 30 degree exhibit complex response characteristics under seismic loads. Several studies have investigated the effects of skew angle on the response of highway bridges (i.e. Maleki, 2002; Bjornsson et al., 1997; Saiidi and Orie, 1992; Maragakis, 1984). Saiidi and Orie (1991) noted the skew effects and suggested that simplified models and methods of analysis would result in sufficiently accurate predictions of seismic response for bridges with skew angles less than 15 degrees. On the other hand, Maleki (2002) concluded that slab-on-girder bridges with skew angles up to 30 degrees and spans up to 65 feet have comparable response characteristics to straight bridges, and therefore, simplified modeling techniques such as rigid deck modeling can be used in many instances. Bjornsson et al. (1997) conducted an extensive parametric study of two-span skew bridges modeled with rigid deck assumption.

2. BENCHMARK BRIDGE

To facilitate the objective of this study, a highway bridge was chosen from Federal Highway Administration's (FHWA) Seismic Design of Bridges Series (Design Example No.4). This bridge can not be considered as seismically isolated. The bridge is a continuous three-span box-girder bridge with 97.536 m (320 ft) total length, spans of 30.48, 36.576, and 30.48 m (100, 120, and 100 ft), and 30^o skew angle (**Figure 1**; FHWA, 1996). The superstructure is a cast-in-place concrete box-girder with two interior webs and has a width-to-span ratio (W/L) of 0.43 for the end spans and 0.358 for the middle span. The intermediate bents have cap beam integral with the box-girder and two reinforced concrete circular columns. Reinforced concrete columns of the bents are 1.219 m (4 ft) in diameter supported on spread footings. In the longitudinal direction, limited movement of superstructure is facilitated by the gap between the superstructure and the abutment as the abutment type was seat-type. In the transverse direction, interior shear keys prevent the movement. This bridge was designed to be built in the USA in a zone with an acceleration coefficient of 0.3g following 1995 AASHTO guidelines. Seismic performance of highway bridges is expected to be affected by combination of various parameters such as: skew angle, boundary conditions, superstructure flexibility, stiffness and mass eccentricity, in-span hinges and restrainers, width-to-span ratio, and direction of strong motion components with respect to orientation of bridge bent and abutments. The presented study investigates the effect of some of these parameters summarizes as: skew angle, abutment gap, shear keys, and direction of earthquake motion.

3. MODELING of BRIDGES

To facilitate a comparative parametric study of the seismic response of skewed highway bridges, a number of detailed three-dimensional (3D) finite element (FE) as well as beam-stick (BS) models were developed (**Figure 2**) using SAP2000 (2005). In all of the models, the superstructure was assumed to be linear-elastic, and all of the nonlinearity was assumed to take place in the substructure elements including bents, internal shear-keys, bearings, and abutment gap (gap opening and closing). The benchmark bridge was altered to produce models with different skew angles, but with the same overall dimensions. Skew angles of 0, 30, 45, and 60 degree were of interest.

For FE models, finite element mesh was used to model deck, soffit, girders, and diaphragms. On the other hand, bent columns and footing were modeled using 3-D frame elements. For the nonlinear analyses, nonlinearity is assumed to take place in the form of localized plastic hinges at the top and bottom of columns. The behavior of nonlinear [uncoupled] axial and moment hinges is characterized by the axial force-displacement and moment-rotation relationship, respectively. Footing-soil interaction was modeled using linear translational and rotational springs at base of the footings (**Figure 2**). Nonlinear link elements were used to model bearings, abutment gap, and shear keys.

The bearings were designed based on a shear modulus of elasticity (G) of 150 psi, and bearing dimensions of 5 in height and 256 in2 of area that supports the bridge. The initial stiffness (k_o) is calculated to be 7.68 kip/in, and the yield force (F_y) is 38.4 kips. It is assumed that the yielding occurs when the lateral deformation of the bearing pad equals its height.

Gap link elements were used to model the abutment gap which its stiffness activates only when the gap closes. On the other hand, the multi-linear plastic link elements were used to simulate the response of internal shear keys. Capacities of internal shear keys were estimated based on comprehensive study done by Megally et al. (2002). Three shear keys, one at center of each cell, were aligned along the skew angle of the bridge (**Figure 3**).

For simplified beam stick models (BS), the same procedure of modeling was followed. Nonetheless, superstructure was represented by a single beam element having the equivalent properties of the entire deck (**Table 1**). Also, the bent cap was modeled using a 3D frame element with a high inertia to facilitate the force distribution to the columns. Additional mass was assigned at abutments, mid-spans, and bent caps to account for the additional weight of the diaphragms. It should be noted that the abutments are modeled by collapsing the individual bearings, shear-key, and gap element from the FE model (**Figure 3**).

4. ANALYSIS of SKEW HIGHWAY BRIDGES

As mentioned earlier, the main objective of the study is to examine the effect of various modeling assumptions on the seismic response of skewed highway bridges.

Table 2 presents the analytical matrix of the study. The analysis includes modal, pushover, and linear and nonlinear time history analyses. Also, a comparison was conducted between the simplified beam-stick (BS) and finite elements (FE) models to assess the accuracy of BS models to capture response of models with different skews. Another comparison was done between results of nonlinear pushover analyses versus linear and nonlinear time history analyses. It should be noted that analyses were conducted for two extreme cases of boundary conditions - Case I with shear keys and Case II with no shear key. Both pushover analysis and time history analyses results are discussed in more detail in what follows.

4.1.Modal Analysis

Modal analyses were conducted for each skew and for two abutment support conditions (Case I and Case II) to determine the vibration modes of the finite element (FE) and beam-stick (BS) models discussed previously. It is noted that structural dynamic characteristics of bridges are expected to be captured more accurately by the FE models. Nonetheless, based on the comparisons between the FE and BS models, conclusions can be drawn with respect to the level of accuracy of approximations due to simplified BS models in general.

The percent difference of the predicted vibration periods with the FE and BS models was calculated. For Case I, the principle longitudinal, transverse, vertical, and coupled vibration periods predicted by both models are in good agreement with 13% relative difference. The largest percent difference is approximately 6% if 60^o skew bridge is excluded. The mode shapes starts to deviate and the largest percent of difference becomes 33% for larger skew angles. For Case II, the principle longitudinal, transverse, vertical, and coupled vibration periods predicted by both models are generally in good agreement. The largest percent difference is approximately 11% if 60^o skew bridge is excluded. Similarly, the mode shapes starts to deviate and the largest percent difference approaches to approximately 35%.

4.2.Time History Analysis

Linear and nonlinear time history analysis were conducted on finite element (FE) models with different skew angles using seven ground motions in order to investigate nonlinear response; effect of skew angle on seismic performance, and effect of direction of strong component of ground motion with respect to longitudinal and transverse directions of the bridge models. Also, time history analysis results were used to measure the accuracy of pushover analysis. It should be noted that the entire force-deformation hysteresis of shear keys could not be modeled explicitly in SAP2000, thereafter shear key links were replaced by support restraints at the same locations (Case I). However, no restraints were applied for Case II to represent two extreme conditions. Time history analyses were conducted for two extreme cases (Cases I and II) and different direction of components of ground motions.

4.2.1Selection of Ground Motions

Design acceleration response spectrum (ARS) is assumed to be that of CALTRANS with M_W of 6.5, PGA of 0.3g, and soil type B. Four ground motions were selected from the PEER strong motion data base (http://peer.berkeley.edu/smcat) among the historically recorded motions with M_W of 6.0 or larger and epicenterd distance of 20 km or less. These four motions were Kobe, Landers, Sylmar, and Rinaldi. All of the four ground motions were scaled to have PGA of 0.3g. The remaining three ground motions were artificial ground motions generated using SIMQKE (1999) to match the target 5%-damped design ARS. A comparison of scaled ground motions along with artificially generated motions and CALTRANS ARS was conducted. Average of selected seven motions matches the demand curve especially within 0.4 to 0.7 seconds which is of major interest as periods of models fall within this range. All four ground motions consisted of two components with different intensities, therefore two analysis cases were considered; the two cases are designated as ST and SL. For the ST, the stronger component was applied in transverse direction while the opposite was true for the SL case. On the other hand, SIMQKE motions were applied equally in both directions.

4.3.Pushover Analysis

Nonlinear static pushover analyses are performed on BS and FE models developed with various skew angles under consideration (0, 30, 45, and 60 degrees). The objectives were to study: 1) the accuracy of BS models versus FE models in capturing the overall nonlinear behavior of the bridges through pushover analysis, 2) the applicability of pushover analyses for relatively large skew angles, and 3) the effect of the pushover load profile.

Pushover analysis is anticipated to be sensitive to the orientation of shear keys and the pushover load profile. Therefore, three pushover load profiles are considered. These profiles are first dominant transverse mode (mode 1), combination of transverse and longitudinal (Mode 1+2), and uniform load. Pushover capacities in terms of load and displacement are reported for all of the examined parameters as well as hinge formation sequence in columns (C1, C2, C3, and C4), which are shown in **Figure 2**a.

4.4.Comparison between Time History and Pushover Analysis

In order to investigate the accuracy of pushover analysis in predicting the overall seismic response of skewed highway bridges, individual maximum responses from nonlinear time history analyses are plotted with the pushover curves. The time history and pushover data is plotted for the finite element (FE) models only (**Figure 4** and **Figure 5**). The maximum resultant base shear and maximum displacement at the center of the midspan (control node) in the transverse direction are plotted for each of the seven ground motions. Two abutment support conditions are considered Case I (shear key/restrained) and Case II (without shear key/ not restrained) with two cases of ground motions. The first case, ST, applies the stronger component in the transverse direction of the bridge while the second case, SL, is the opposite. It should be noted that

additional time history analysis was done using Kobe and Sylmar motions scaled to 0.5g for Case I and ST case and compared against pushover curves.
For Case I, The time history results compare well for the 0 degree skew. Both ST and SL cases follow the Mode 1+2 and Uniform Load pushover curves very well; although, the base shear capacity is slightly over predicted by the Uniform Load pushover curve. The time history results tend to cluster around the first hinge formation on the pushover curves. The first transverse mode pushover curve predicts the closest response for 0.5 g ground motions. For the 30 degree skew, all the predicted pushover curves compare well to the time history results for the ST and SL cases; although, the Mode 1+2 pushover curve underpredicts the response from the Kobe and Sylmar motions for the ST case (**Figure 4** and **Figure 5**). The Kobe, Landers, and Sylmar motions are closer to the fully yielded mechanism predicted by the pushover curves while the Rinaldi and SIMQKE motions are closer to the first hinge formation. Once again, the first transverse mode profile gives the better prediction for ground motions beyond the design level, but the Uniform Load also gives satisfactory results. For the 45 degree skew and ST case, the three SIMQKE and Landers motions fall below the pushover curves while the Kobe, Rinaldi, and Sylmar motions suggest that the pushover curves predicts smaller maximum base shear, except for the first transverse mode profile. The time history responses are clustered around the fully yielded mechanism for Mode 1+2 and the Uniform Load. For 0.5 g ground motions, the first transverse mode pushover curve predicts accurately the maximum response from the time history analyses. For the SL case, pushover curves over predicts the maximum response due to the three SIMQKE motions. In both Mode 1+2 and Uniform Load, the maximum responses from the time history analyses are clustered around the fully yielded mechanism. For the 60 degree skew shown in **Figure 5**, the time history results compare well with the Mode 1+2 pushover curve for both cases. The first transverse mode predicts a larger capacity and the 0.5 g ground motion results do not compare well with any of the pushover curves; although, the results are conservative with respect to the first transverse mode pushover curve. For the ST and SL cases, the maximum responses are clustered in between the partial and fully yielded mechanisms.
For Case II, For the ST and SL cases, the first transverse mode and Uniform Load pushover curves underpredict the response from all seven ground motions for the 0 degree skew. The Mode 1+2 pushover curve compares well with the time history results. All the motions cluster near the fully yielded mechanism for both cases. For the 30 degree skew, the pushover curves and time history results compare well for the ST and SL cases; although, the first transverse mode and Uniform Load pushover curves slightly underpredict the time history responses, especially for the SL case. Once again, the motions are clustered around the fully yielded mechanism. For the 45 degree skew, the seven ground motions are more scattered, especially the Kobe and Landers motions. The scatter for the ST case follows the first transverse mode and Uniform Load pushover curves exactly. The Kobe and Landers motions cluster around the fully yielded mechanism while the maximum responses due to the rest of the motions fall near the partially yielded system. For the SL case, all three pushover curves slightly underpredict the time history responses. For the 60 degree skew, the three pushover curves once again slightly underpredict each ground motion response for the ST and SL cases. All of the ground motions are clustered near the fully yielded mechanism for both cases.

5. CONCLUSIONS

As mentioned earlier, behavior of skewed highway bridges is complex and modeling assumptions affect the predicted seismic performance. In this study, various parameters were studied such as: skew angle, shear key effect, direction of components of ground motions, as well as the adequacy of pushover for dynamic analysis of skew bridges. The following conclusions are drawn:
(1) Predicted modal properties with the FE and BS models were comparable; the BS model was successful in capturing the modal coupling due to the skew and the significant modes needed for further analysis. Nonetheless, FE models should be considered when dealing with very large skew angles (> 30 degrees) in order to capture the higher mode effects.
(2) Boundary conditions (hence modeling assumptions) have a significant effect on pushover analyses and

choice of pushover load profile.
(3) For Case I and II, the Uniform Load profile was the most consistent in predicting similar sequences of hinge formation between the FE and BS models. Ultimately, the BS model, when compared to the FE model, accurately captured the overall nonlinear behavior of the bridge when using the Uniform Load profile.
(4) Bridges with larger skew angles (> 30 degrees) experienced larger deformations, which in turn, resulted in larger ductility demands; however, forces in the substructure elements remained relatively unaffected with exception to the torsional response of the columns (C1 and C4) that are on the diagonal with respect to the acute corners of the bridges.
(5) Time history analyses suggested that the shear keys had marginal effect in reducing the torsional response with increasing skew angles greater than 30 degrees.
(6) The direction of the two horizontal components of the strong motions relative to the longitudinal and transverse directions did not have any significant effect on the overall response.
(7) Maximum forces at the abutments of skewed bridges were unevenly distributed which would potentially lead to progressive failure of support elements.
(8) For larger skew angles, the results from the nonlinear time history analyses agree with the observed yield mechanism in the columns from the pushover analyses.

6. TABLES

Table 1 SECTION PROPERTIES

	Superstructure	**Bent Cap Beam**	**Bent Column**
Area (ft^2)	72.74	27	12.57
Ix - Torsion (ft^4)	1177	100000	25.13
Iy (ft^4)	401	100000	12.57
Iz (ft^4)	9697	100000	12.57
Density (lb/ft^3)	182	150	150

Table 2 ANALYTICAL MATRIX

<table>
<tr><th>Analysis Type</th><th>Analysis Cases</th><th colspan="2">Parameters</th></tr>
<tr><td rowspan="2">Modal Analysis</td><td rowspan="2">Modal</td><td>With Shear Keys (Case I)</td><td>Without Shear Keys (Case II)</td></tr>
<tr><td>Skews: 0, 30, 45, 60</td><td>Skews: 0, 30, 45, 60</td></tr>
<tr><td rowspan="3">Pushover Analysis</td><td>Transverse Mode</td><td>With Shear Keys (Case I)</td><td>Without Shear Keys (Case II)</td></tr>
<tr><td>Multimode</td><td rowspan="2">Skews: 0, 30, 45, 60</td><td rowspan="2">Skews: 0, 30, 45, 60</td></tr>
<tr><td>Uniform Load</td></tr>
<tr><td rowspan="6">Time History Analysis

Nonlinear and Linear</td><td>Kobe</td><td rowspan="2">Restrained (Case I)</td><td rowspan="2">Unrestrained (Case II)</td></tr>
<tr><td>Landers</td></tr>
<tr><td>Sylmar</td><td rowspan="3">Strong Motion in Transverse Direction / Weak Motion in Longitudinal Direction (ST)</td><td rowspan="3">Strong Motion in Longitudinal Direction / Weak Motion in Transverse Direction (SL)</td></tr>
<tr><td>Rinaldi</td></tr>
<tr><td>Simqke 1</td></tr>
<tr><td>Simqke 2</td><td>Skews: 0, 30, 45, 60</td><td>Skews: 0, 30, 45, 60</td></tr>
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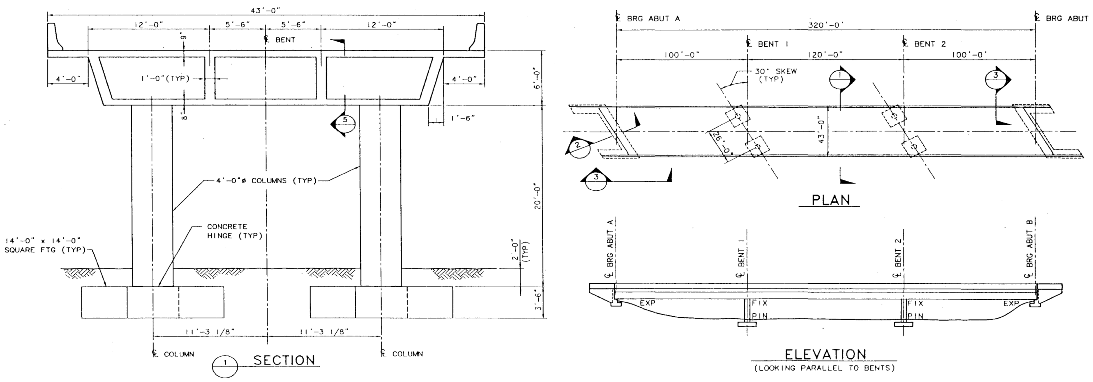
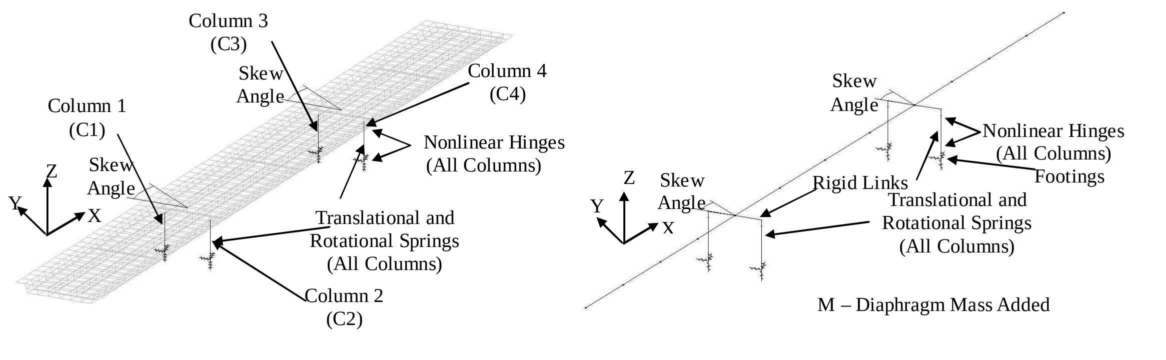
7. ILLUSTRATIONS, DIAGRAMS AND PHOTOGRAPHS



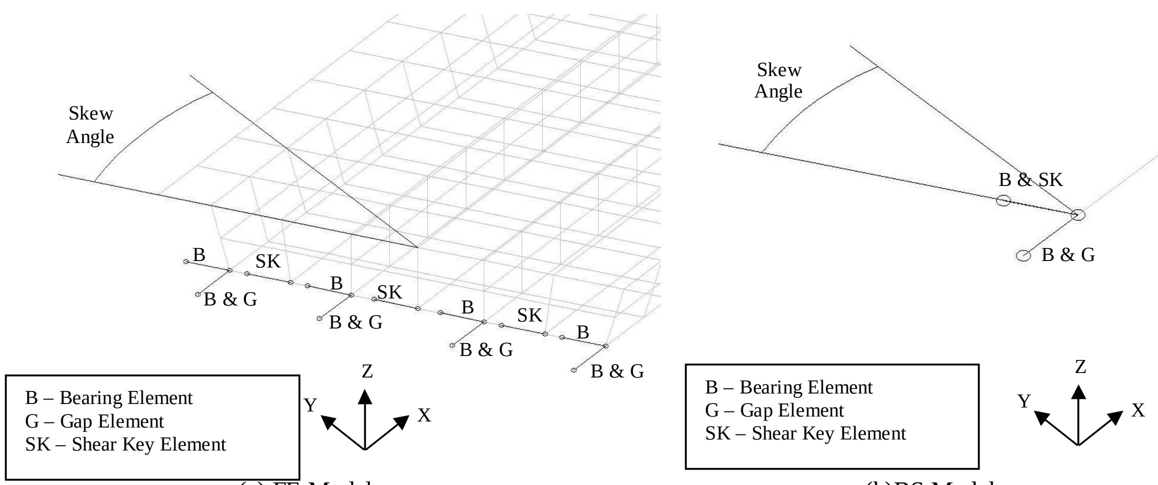
(a)Bent Elevation (b) Plan and Elevation Views

Figure 1 Benchmark Bridge Geometry



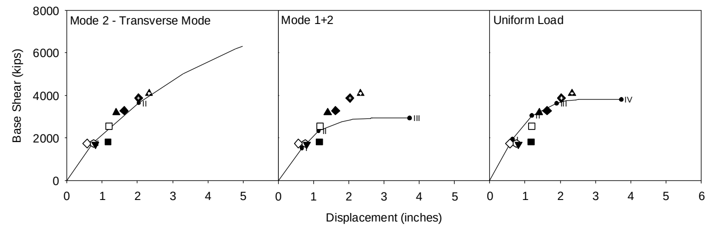
(a) Finite Element (FE) Model (b) Beam-Stick (BS) Model

Figure 2 Models Details



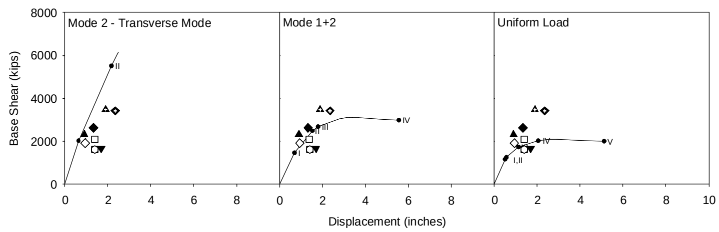
(a) FE Model (b)BS Model

Figure 3 Abutment Support Details of Bridge Models



30 Degree Skew Stages		
	Finite Element	
	Stage	Description
Mode 2	I	Yield at top of C1 and C2
	II	Yield at top of C3 and C4 and at bottom of C1, C2, C3, C4
Mode 1+2	I	Yield at top of C2
	II	Yield at top of C1, C3, C4 and at bottom of C1, C2, C3, C4
	III	Failure at bottom of C2 and C3
Uniform Load	I	Yield at bottom of C3
	II	Yield at bottom of C1, C2, C4 and at top of C2, C3, C4
	III	Yield at top of C1
	IV	Failure at top of C3

Figure 4 30 Degree Skew Time History and Pushover Comparison for ST Case and Case I



60 Degree Skew Stages		
	Finite Element	
	Stage	Description
Mode 1	I	Yield at bottom of C1, C2, C3, C4
	II	Yield at top of C1, C2, C3, C4
Mode 1+2	I	Yield at bottom of C3
	II	Yield at bottom of C1, C2, C4 and at top of C2 and C3
	III	Yield at top of C1 and C4
	IV	Failure at top of C3
Uniform Load	I	Yield at bottom of C3 and C4
	II	Yield at bottom of C1 and C2
	III	Yield at top of C1, C3, C4
	IV	Yield at top of C2
	V	Failure at top of C3 and C4

Figure 5 60 Degree Skew Time History and Pushover Comparison for ST Case and Case I

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