NONLINEAR SEISMIC SOIL-ABUTMENT-STRUCTURE INTERACTION ANALYSIS OF SKEWED BRIDGES

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Abstract

This paper investigates the impact of near-field ground motions on skew bridges. Three-dimensional models of single-, two- and three-span box girder bridges with seat-type abutments, single- and two-column bents and various skew angles are presented. Abutment-soil interaction was modeled by nonlinear normal springs skewed to the principal bridge axis. Abutment shear keys were modeled using nonlinear springs in the skewed transverse direction. Nonlinear time-history analyses were performed using seven sets of ground motions with two lateral components incorporating near-fault effects. The analyses shows that the superstructure undergoes significant rotations about the vertical axis that result in permanent lateral deck offset at the abutments.

Introduction

Dynamic nonlinear soil-abutment-structure interaction (SASI) is very complex and plays a significant role in the global response of skewed bridges to earthquake excitation. Earthquake records with near-source ground motion characteristics (e.g., the 1994 Northridge, California, the 1995 Kobe, Japan, and the 1999 Izmit and Duzce, Turkey earthquakes) have increased the awareness of the importance of nonlinear seismic analyses on of bridge structures employing SASI. Skewed bridges tend to rotate during a seismic event. The superstructure rotation can cause excessive transverse movement and unseating of the superstructure and pounding to the abutment backwall.

Global seismic behavior of skewed bridges is affected by a number of factors, including bridge skew angle, deck width, deck flexibility, number of spans, number of columns per bent, column ductility, soil-abutment-superstructure interaction, abutment shear keys, soil-bent foundation-structure interaction, abutment bearing pads, and characteristics of the seismic source. This study focuses on nonlinear time-history analyses of a typical straight, single-span, two-span and three-span skewed-concrete box girder bridges subjected to seven sets of two-components ground motions with high velocity pulses.

Traditional bridge design practice evaluates dynamic performance of skewed bridges using stick models with lumped springs to represent the abutment structure and foundation. However, when a bridge has skewed abutments as shown in Figure 1, the lateral bridge response is affected by transverse loading due to the coupling nature of the two horizontal directions. Therefore, a three-dimensional model of the bridge structure and at least two-component loading must be considered. Bridge elements considered include the nonlinear abutment-backfill, bridge deck, bent cap and bent columns.

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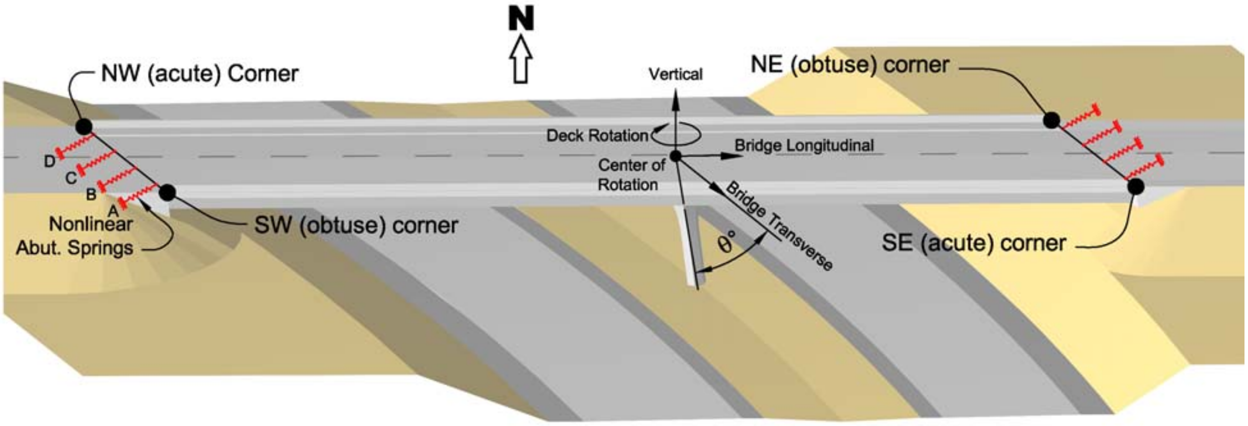


Figure 1. Bridge Structure Configuration

Bridge Models

Three-dimensional finite-element models of various bridge structures were developed using the computer program SAP2000 (2006). Shell elements with superstructure properties were used to model the bridge deck and frame element with moment curvature and cracked sectional properties were used to model bridge columns as shown in Figure 2. A total of five typical bridge configurations as shown in Figure 3 were modeled:

- Bridge No. 1 - A typical single-span bridge,
- Bridge No. 2 - A two-span with a single-column center bent,
- Bridge No. 3 - A two-span bridge with a dual-column at the center bent,
- Bridge No. 4 - A three-span bridge with single-column bents, and
- Bridge No. 5 - A three-span bridge on two bents supported by two columns per bent based.

These bridges were based on actual bridge plans from Meloland Road Bridge in Imperial County, and LaVeta Avenue Bridge and Redhill Avenue Bridge over I-405 in Orange County.

At the bent columns, plastic hinges were used at the top and pinned connections at the base of the column. A pinned column-footing connection was used to simplify the models and to allow the analysis to focus on the seismic response of the skewed bridges without the complexity of the bridge foundation. The longitudinal abutment model consisted of (1) a gapping element between the bridge deck and the abutment backfill, and (2) a nonlinear spring. The gapping element was to simulate 1-inch expansion gap typical for seat type abutments of bridges in California. The nonlinear spring represents the near-field load-deformation behavior of the abutment-embankment. Abutment transverse shear key behavior was simulated using a multi-linear plasticity model that addresses shear key resistance as a function of relative displacement between bridge deck and abutment (Shamsabadi and Kapuskar, 2006). The model included the 1-inch expansion gap between shear key and deck, and the limiting passive capacity of the embankment soil.

The shell models were then converted to spline deck models. The seismic response of the bridge structures using shell models and spline models were found to be comparable. The simpler spline models were used for the purpose of the parameter study below because they were much less computationally intensive than the shell models. All models were developed with zero skew angles and modified for skew angles of 25, 45 and 60 degrees. The five example bridge models shown in Figure 3 are configured for 45-degree skew.

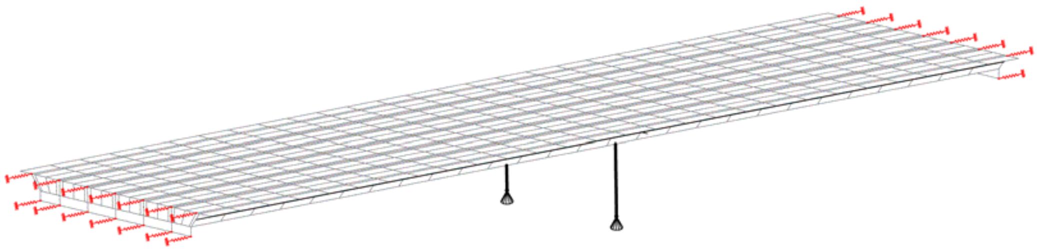


Figure 2. Typical Bridge Model Using Shell Elements

<u>Input Ground Motions</u>

A total of seven ground acceleration time histories with two horizontal components incorporating near-field characteristics were selected for input ground motions. The time histories were engineered to be spectrum-compatible with Caltrans standard spectra Soil Type D for a Magnitude 8 seismic event with a peak ground acceleration of 0.7g per Caltrans Seismic Design Criteria (2006). The records with peak ground accelerations, velocities and displacements of the time histories are listed in Table 1. Vertical components were ignored.

Table 1. Input Ground Motion Characteristics

Motion No. & Earthquake Record	Bridge Longitudinal Direction				Bridge Transverse Direction			
	Peak Acc. (g)	Peak Vel. (in/sec)	Peak Disp. (in)	Time of Vel.Pulse (sec)	Peak Acc. (g)	Peak Vel. (in/sec)	Peak Disp. (in)	Time of Vel.Puls e (sec)
Developed Spectrum-Compatible Motions								
1-1979 El Centro	0.7	56	38	8.3	0.7	51	40	8.8
2-1994 Northridge, Sylmar	0.7	68	21	7.9	0.7	47	30	10.6
3-1992 Landers, Lucerne	0.7	53	23	12.6	0.7	43	23	11.0
4-1994 Northridge, Rinaldi	0.7	79	23	4.0	0.7	52	25	7.3
5-1989 Loma Prieta, Los Gatos	0.7	51	36	9.9	0.7	51	27	9.9
6-1995 Kobe, Takatori	0.7	44	32	7.7	0.7	43	28	10.5
7-1976 Turkey, Erzincan	0.7	58	36	4.9	0.7	39	39	6.0

The two components were applied in the longitudinal and transverse directions of the bridge. All input motions exhibit high-velocity pulses; the components with the largest velocity pulses were applied in the longitudinal direction. The peak accelerations, velocities and displacements including the time of the velocity pulses for both components are listed in Table1. The largest velocity pulses occurred between 4.9 to 12.9 seconds. Figure 4 shows velocity time histories of all seven input motions applied in the longitudinal bridge direction.

Results of the Analysis

Figures 5 to 9 show response time histories of deck rotation about the vertical axis due to the seven motions for each of the five bridge models, respectively. Each figure shows deck rotation for the 25, 45 and 60-degree skew configurations. The time histories for the nonskewed configurations are not shown due to space constraints but findings are discussed below. From these analyses, the following observations are made:

- The bridge dynamic behavior is dependent on the characteristics of the input ground motions. The magnitude of permanent rotation varies among all seven input motions, particularly for Bridges No. 1 to 3, and all bridge skew configurations.
- The bridges experienced significant amounts of rotation about the vertical axis during seismic ground shaking when the abutments are skewed, whereas the nonskewed bridge showed little or no rotation.
- The decks experienced significant amount of rotation which builds up during initial peak cycles shortly after the velocity pulses occurred (about 3 to 13 seconds). None of the decks returned to their original position. Bridge No. 1 (single-span without column) and No. 2 (with one single-column bent) experienced the largest magnitudes of rotation.
- The bridge decks then rotated back in a reverse direction by a small amount. The amount of this rotation was largest for Bridges No. 1 and 2. Bridges No. 3 to 5, which consist of more rigid bridge structure systems, showed little or no significant reverse rotation.
- Subsequent deck rotations were small for Bridges No. 1 and 2. The response is observed to undergo more and more oscillation for the more rigid Bridges No. 3 to 5 (with multiple-column bents and more spans).

Figure 10 is a three-dimensional histogram depicting the spread of peak deck rotations due to the seven input ground motions for all five bridge models and three skew configurations. Figure 11 shows the respective spread of permanent residual rotations. Figure 12 summarizes the average residual deck rotations developed from the seven response time histories of Figures 5 to 9. In the three histograms, the response of Bridge No. 2 due to Motion 7 was eliminated because it resulted in an extreme response (see Figures 6b and c).

Figures 10 to 12 do not show a clear trend between the magnitude of deck rotation and skew angle among all seven input motions. For some motions, the rotation (lateral deck displacement) increased with skew angle while for others it decreased. Unexpectedly, the largest rotation can occur at lower skew angles.

It can also be seen from Figures 10 to 12 that deck rotation decreased as the rigidity of the bridge structure increases from Bridge No. 1 to 5. The magnitudes of rotation were similar among the three skew angles for most of the input motions, although rotations of the “softer” structures of Bridges No.1 to 3 tended to increase with increasing skew. For example, Bridge No. 2 (with center column) and Bridge No. 1 (without column) have the same bridge lengths and the center of mass and the center of rigidity of the bridge system (located at the top of the bent as shown in Figure 2b); however, Bridge No. 2 experienced slightly lower rotations for all three skew angles compared to Bridge No. 1 due to the additional column shear and torsional resistance in the bridge system. Figures 10 and 11 also show that the bridges with less rigidity had a wider range of rotations among the seven input motions, while the multi-column and/or multi-span bridges much were less affected by the input motion used.

The authors have shown that when the deck is pushed into a skewed abutment during ground shaking, the skewed abutment backwall generates asymmetric passive soil resistances between the acute and obtuse corners of the wall that cause the ends of the deck to “bounce off” the wall in the bridge transverse direction, resulting in deck rotation (Shamsabadi and Kapuskar, 2006). The width and capacity of this passive wedge depends on abutment (embankment) width, skew angle and ground motion characteristics.

Conclusions

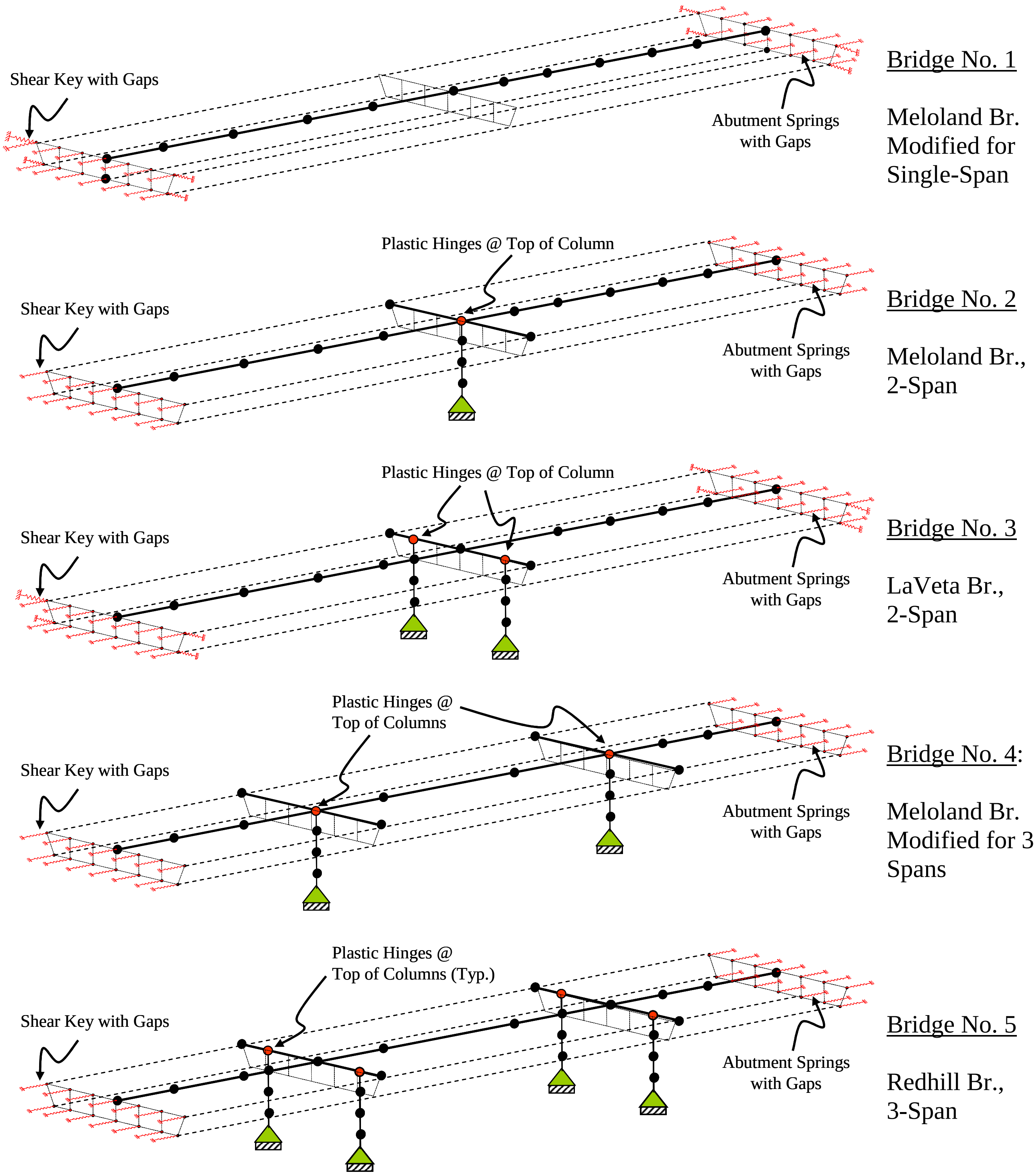
Three-dimensional models of a two-span box girder bridge were developed incorporating 0, 25, 45 and 60-degree skew. Nonlinear time history analyses using seven two-component lateral input ground motions were performed to evaluate the seismic response of the abutments and the superstructure as a function of skew angle. All motions exhibited high-velocity pulses characteristic of near-field effects.

From the results, it was found that the bridge superstructure undergoes significant rotations about the vertical axis during seismic ground shaking and is permanently displaced laterally from the original location by the end of shaking. The nonskewed bridge deck did not experience significant rotation or permanent transverse displacement despite the large velocity pulses in the input motions. For the skewed bridges, the deck rotation built up during the initial peak cycles of shaking of all input ground motions. Once this large rotation had occurred, the deck did not return to its original position for any of the three skewed configurations. Depending on the intensity of the velocity pulses, the observed deck may cause the deck to become unseated at the abutments. The deck rotation occurs in skewed bridges due to the development of an non-uniform passive soil wedge behind the abutment wall that results is asymmetric soil reactions between the acute and obtuse corner of the wall (Shamsabadi and Kapuskar, 2006).

No clear trend was observed between the magnitude of deck rotation and the skew angle. It was observed that deck rotation decreased as the rigidity of the bridge structure increased (e.g., as the number of columns per bent and/or the number of spans increased).

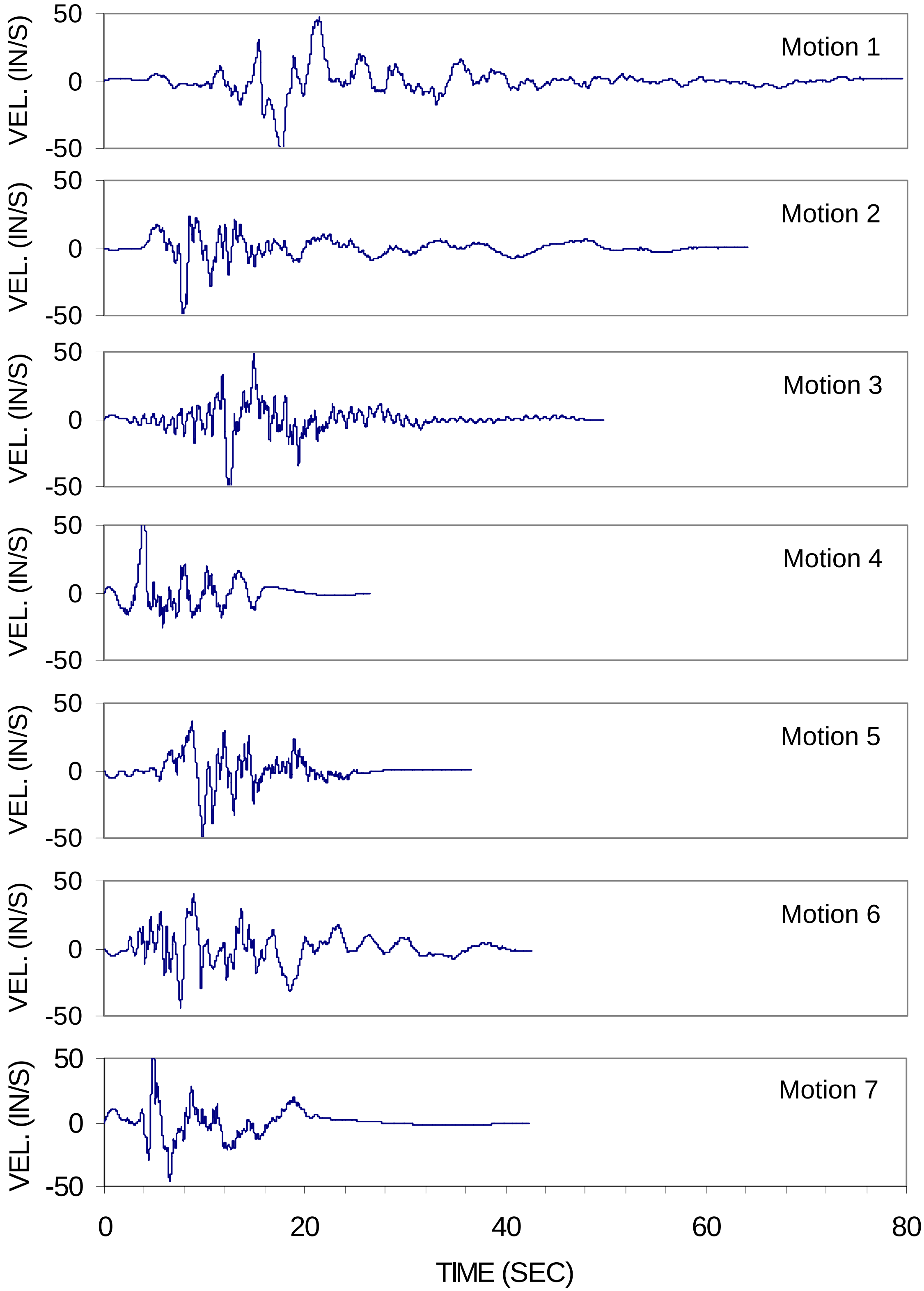
Furthermore, the analyses showed that the bridge dynamic behavior is influenced by the characteristics of the input ground motions. The bridges with less rigidity showed a wider range of response as a function of input motions used, and the multi-column and/or multi-span bridges much were less affected. This strongly suggests that single- or two-span bridges with significant skew should be analyzed using motions with two or three components and several earthquake records should be considered to capture the behavior of the structure details.

In the bridge models the abutment shear keys and the abutment backfill were simulated using a nonlinear link element. The results showed that shear keys can control and complicate overall bridge response for any skew angle. The shear keys could potentially fail when subjected to strong ground shaking, which could allow the bridge to rotate and cause the deck to become unseated at the east abutment due to lateral movement. It is therefore important that bridges with significant skew be analyzed using three-dimensional models to capture the behavior of the structure details.



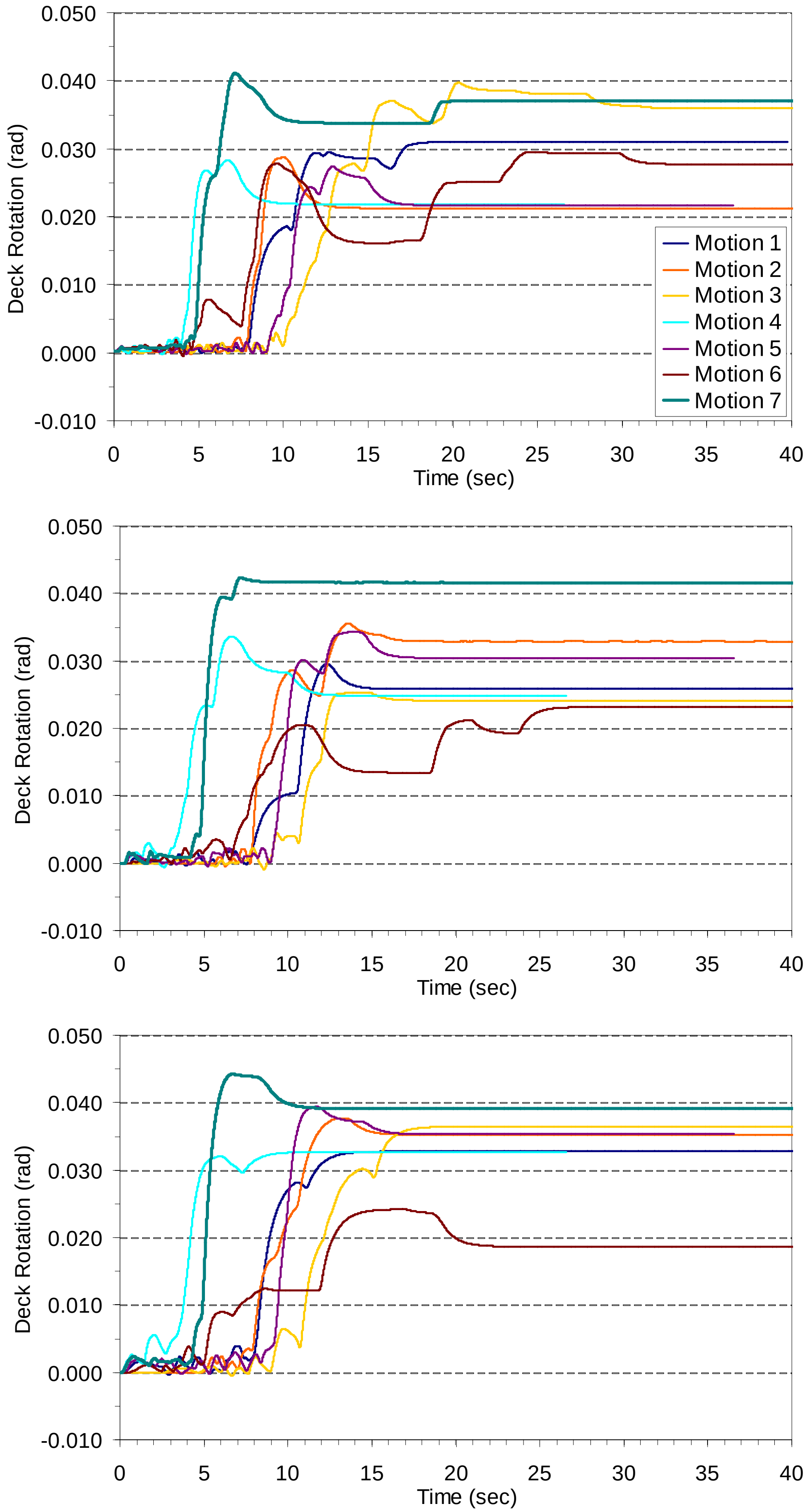
Note: All models shown at 45-degree skew configuration.

Figure 3. Bridge Structural Models Using Spline Elements



Note: Velocity time histories of first horizontal component shown.

Figure 4. Input Ground Motions

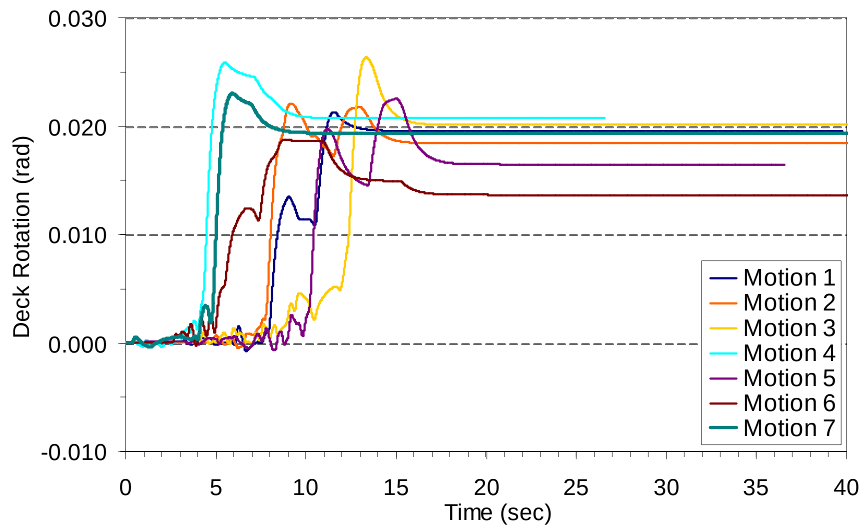


(a) 25° Skew

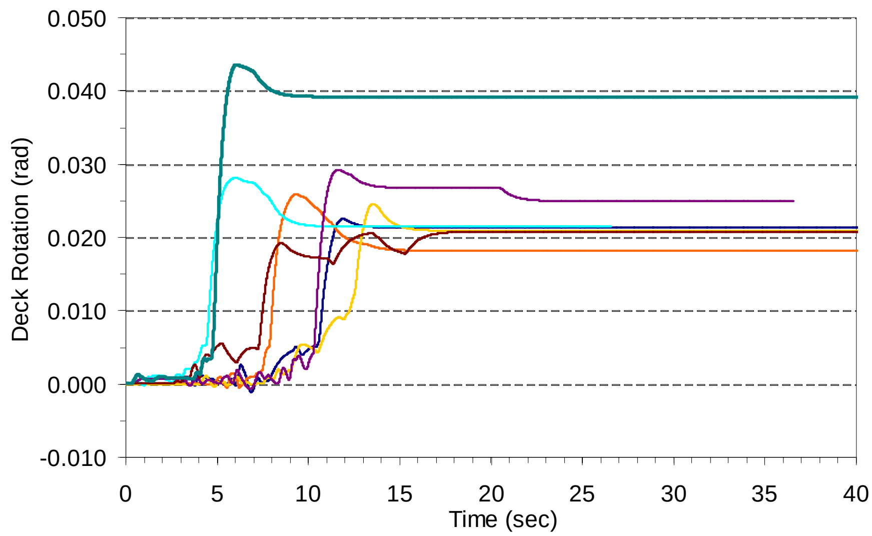
(b) 45° Skew

(c) 60° Skew

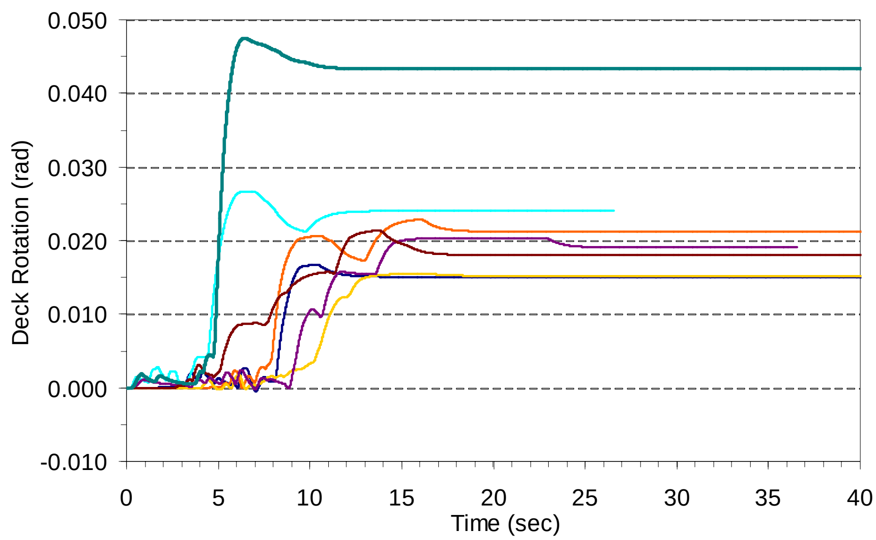
Figure 5. Time Histories of Deck Rotation for Bridge No. 1



(a) 25^o Skew



(b) 45^o Skew



(c) 60^o Skew

Figure 6. Time Histories of Deck Rotation for Bridge No. 2

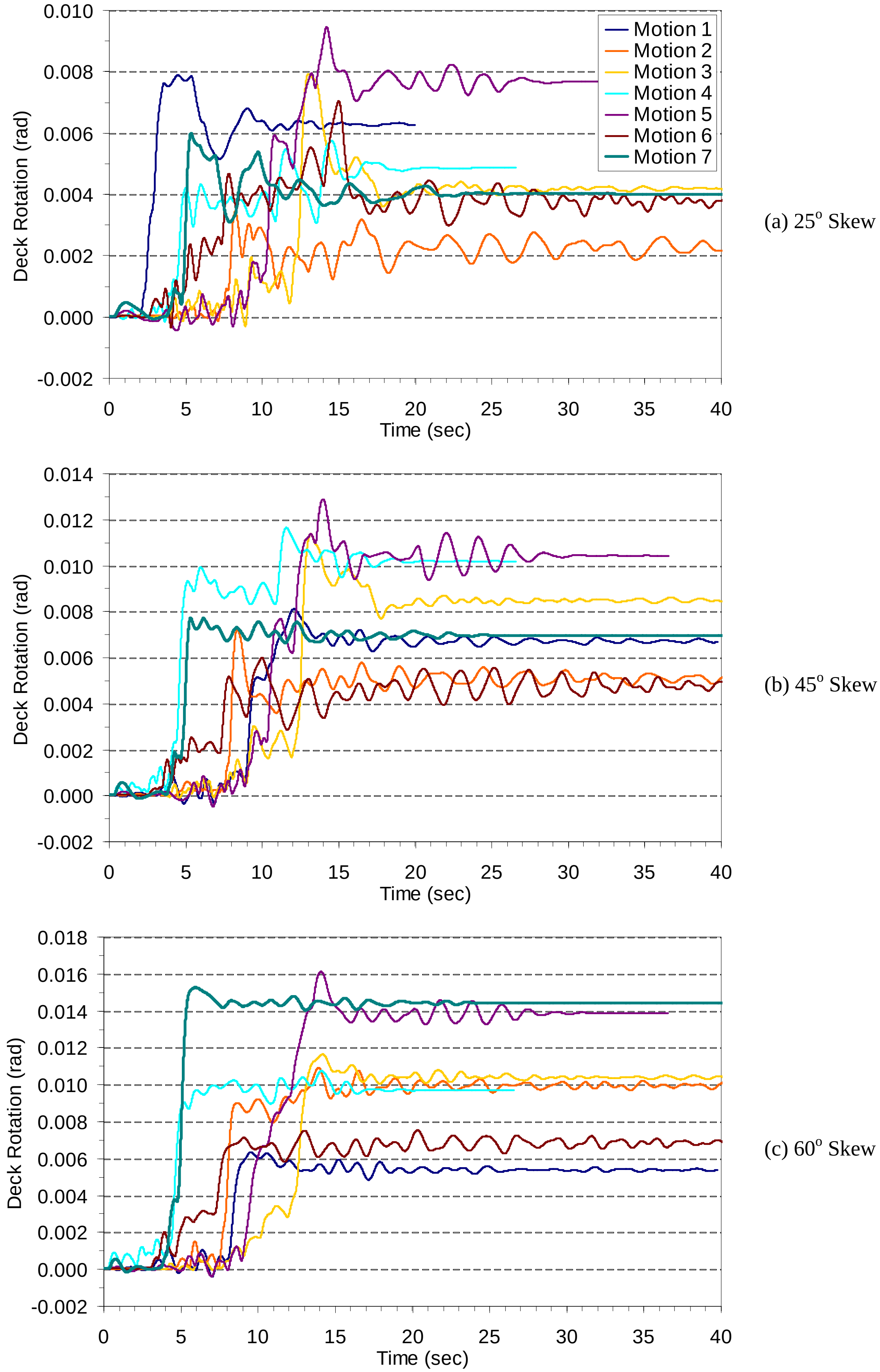
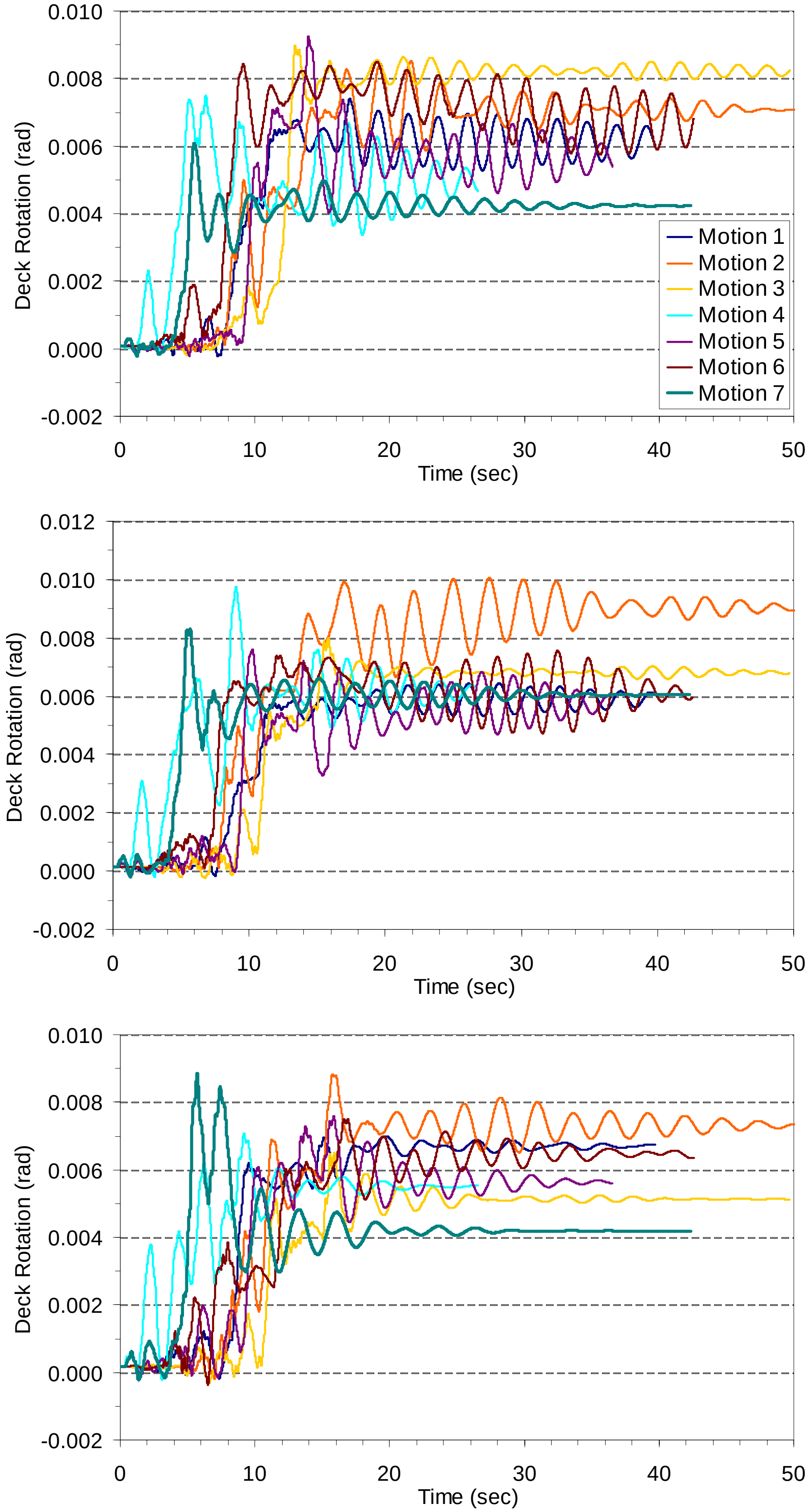


Figure 7. Time Histories of Deck Rotation for Bridge No. 3

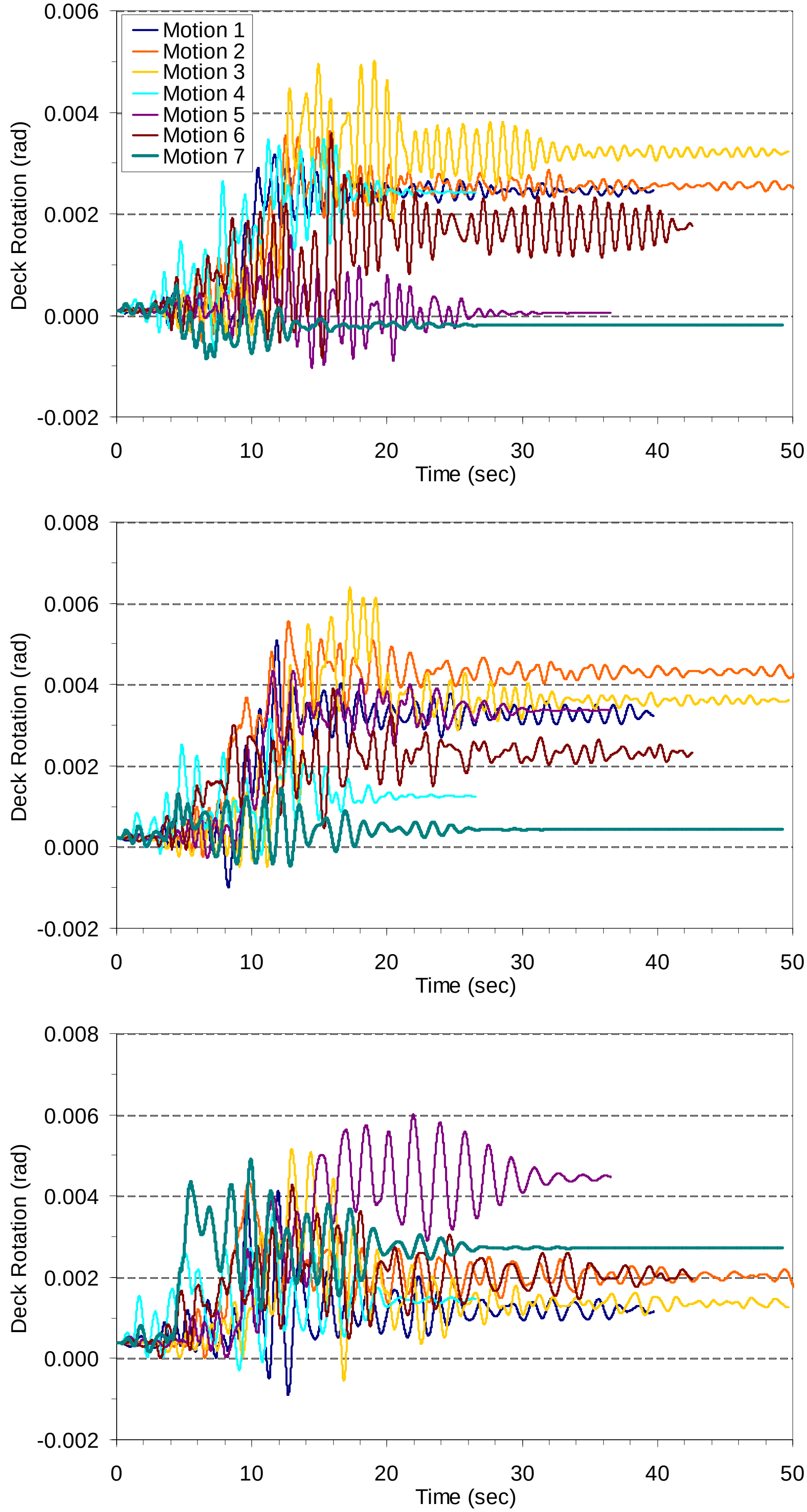


(a) 25^{o} Skew

(b) 45^{o} Skew

(c) 60^{o} Skew

Figure 8. Time Histories of Deck Rotation for Bridge No. 4



(a) 25^{o} Skew

(b) 45^{o} Skew

(c) 60^{o} Skew

Figure 9. Time Histories of Deck Rotation for Bridge No. 5

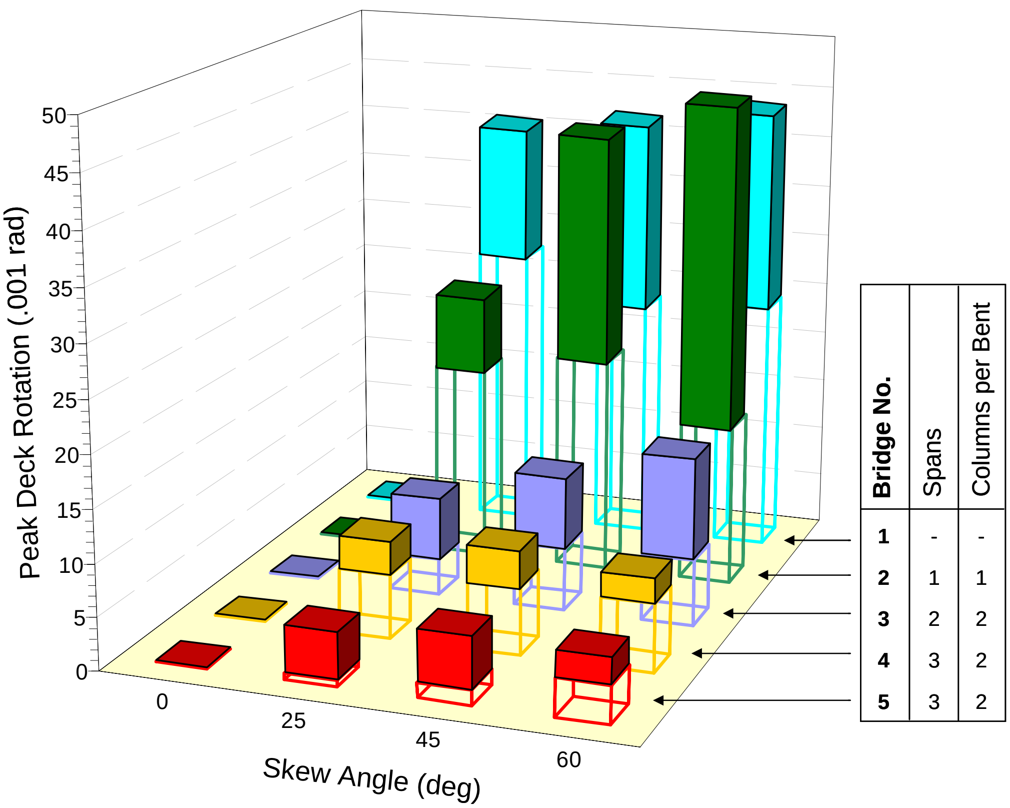


Figure 10. Range of Peak Deck Rotations

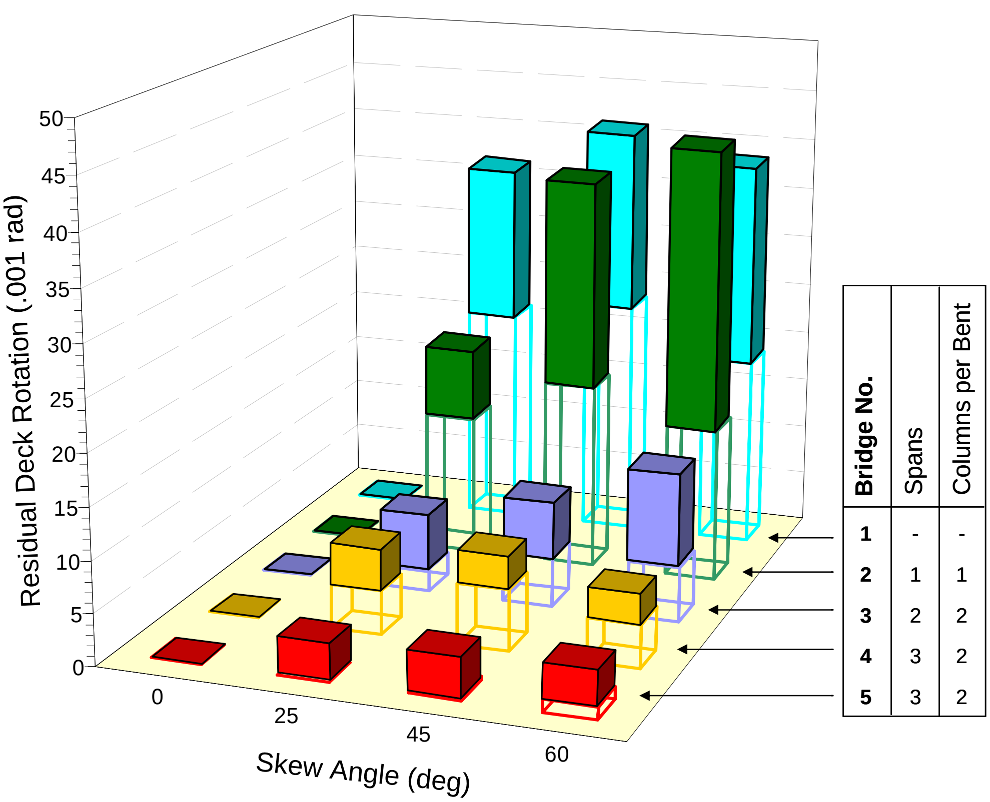


Figure 11. Range of Residual Deck Rotations

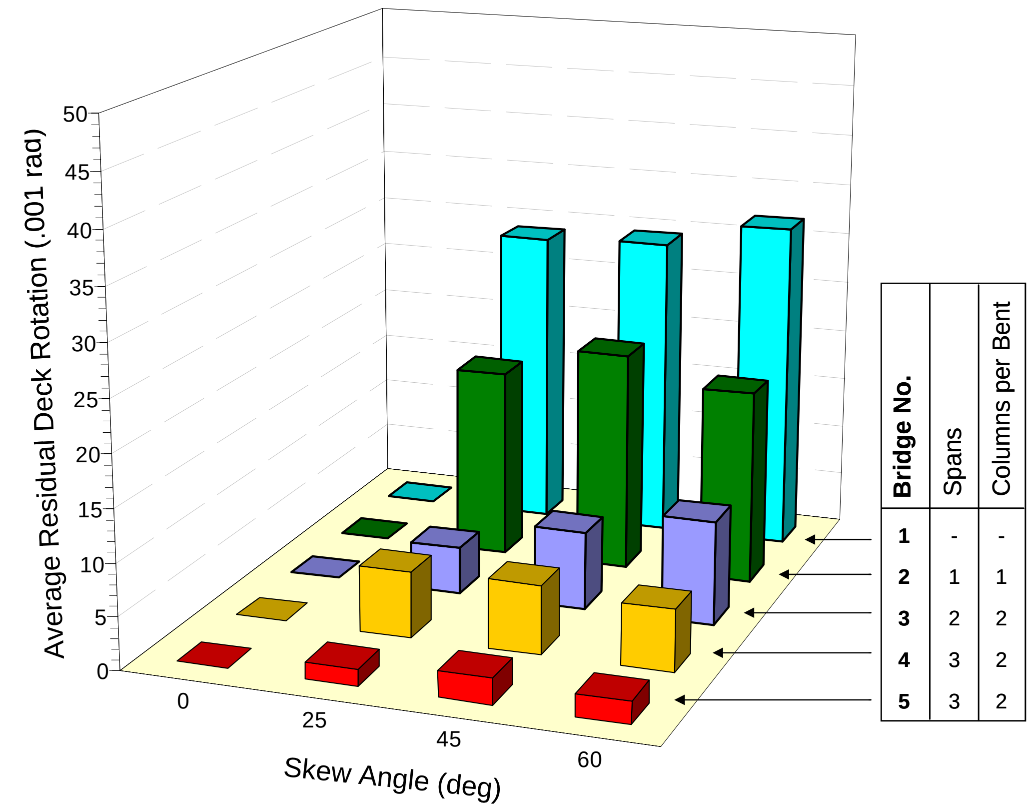


Figure 12. Average Residual Deck Rotations

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