Effects of Ground Motion Characteristics on Seismic Response of Rocking-Foundation Bridges

Lijun Deng¹, Bruce L. Kutter², M. ASCE, Sashi K. Kunnath³, F. ASCE

¹Ph.D. candidate, Dept. Civil Environ. Engr., University of California, 1 Shields Ave., Davis, CA 95616; PH (530) 297-1913; email: ljdeng@ucdavis.edu
²Professor, Dept. Civil Environ. Engr., University of California, 1 Shields Ave., Davis, CA 95616; PH (530) 752-8099; email: blkutter@ucdavis.edu
³Professor, Dept. Civil Environ. Engr., University of California, 1 Shields Ave., Davis, CA 95616; PH (530) 754-6428; email: skkunnath@ucdavis.edu

ABSTRACT: Past studies have revealed that the different characteristics of near-fault pulse-like and far-field broadband motions have varying effects on the response of structures. In this paper, a suite of pulse-like and broadband motions are utilized to investigate the response of rocking-foundation and hinging column bridges including large deformation effects. Incremental Dynamic Analyses were carried out on simplified models of which the following parameters were varied: the rocking strength, the column height, and the fundamental period. Results show that, in a probabilistic sense, rocking systems are more susceptible to tip-over in pulse-like motions than in broadband motions. The re-centering effect of rocking systems makes them relatively immune to damage in motions that consist of many small inelastic pulses. Elastic-perfectly plastic hinging systems have less re-centering capability and hence they are more susceptible to accumulated deformation and eventual collapse in many small cycles that might occur in long duration broadband motions.

INTRODUCTION

If rocking shallow foundations are allowed to exhibit inelastic rotations when subject to the inertia moment demand during earthquakes, they can dissipate seismic energy through soil yielding and radiation damping and reduce the force demand on substructure components. Characteristics of rocking foundations were studied by many researchers through experimental modeling (e.g., Gajan et al. 2005, Deng et al. 2012a) or numerical analyses (e.g., Raychowdhury and Hutchinson 2010). These preceding studies characterized the behavior of rocking foundations and suggested that structures designed with rocking foundations generally have better performance than conventional fixed-based spread foundations; however, these studies did not attempt to differentiate the effects of ground motions. It is known that near-fault ground motions differ from far-field ground motions in several aspects and the characteristics of ground motions could affect the response of structures dramatically.
(Kalkan and Kunnath 2006, Brown and Saiidi 2011). Since numerous bridges in California are located in the near-fault region due to the wide distribution of crustal faults, special care may be taken for rocking structures located in near-fault zones before the adoption of the rocking foundations in current design guidelines.

Deng et al. (2012b) conducted a parametric study of rocking-foundation and hinging-column bridges that were primarily subject to pulse-like motions. The present paper continues to use the numerical model of rocking-foundation bridges with “single-degree-of-freedom” mass and a nonlinear column that was developed by Deng et al. (2012b); by exercising the pulse-like and broadband motions in nonlinear dynamic analyses the paper attempts to investigate the effects of ground motion characteristics on system behavior. The effects of ground motion characteristics on the stability of rocking bridges are presented and discussed.

GROUND MOTIONS

Caltrans (2010) states that a structure of interest within 10 miles (or 16 km) of a fault should be classified in the near-fault range, where near-fault ground motions are prevailing rather than far-field broadband motions. According to the definitions in Baker (2007), pulse-like motions often contain a short-duration velocity pulse that occurs early in the velocity time history and have large amplitude. Whether a record is pulse-like can also be quantitatively evaluated by the Pulse Indicator, an empirical parameter that was defined in Agresti (2002) and used in Baker (2007). Near-fault motions can be divided into two groups: fling-step and forward-directivity motions (Kalkan and Kunnath 2006). Fling-step motions have permanent displacements that should be considered separately and superimposed on the dynamic displacement (Somerville 2002); forward directivity at a particular site is expected when the fault rupture propagates toward that site at a speed approximately equal to the shear wave velocity. On the other hand, far-field broadband motions commonly have a wide band of frequency content and numerous velocity cycles of small to moderate amplitude.

A suite of 40 pulse-like ground motions recorded at soil sites and selected by Baker et al. (2011) were used for the nonlinear Incremental Dynamic Analysis (Vamvatsikos and Cornell 2002). The strike-normal components of these motions were chosen as the input because this direction commonly has stronger pulse amplitude. Baker et al. (2011) showed that the algorithm for selecting pulse-like motions could reliably find ground motions that include the forward directivity velocity pulses, but is not sensitive to fling-step pulses. Hence, the 40 selected pulse-like motions do not necessarily include fling-step pulses. In addition, a suite of 40 far-field broadband motions (Baker et al. 2011) recorded at soil sites from earthquakes of Magnitude =7 and Distance $\approx$ 10 km were also used for comparison. The acceleration response spectra of near-fault and broadband motions are shown in Figure 2. Details of these near-fault and the broadband motions can be obtained in Baker et al. (2011).

To facilitate the Incremental Dynamic Analysis, the intensity of the ground motions was scaled by factors ranging from 0.2 to 12.0, so various bridge performance levels
ranging from no damage to collapse would be observed. Each ground motion was excited with 12~14 increasing amplification factors.

FIG. 1. (a) acceleration and (b) velocity time histories of a typical near-fault motion (forward directivity) recorded at Sylmar Hospital station in 1994 Northridge earthquake; (c) acceleration and (d) velocity time histories of an ordinary far-field broadband motion in 1999 Chi-Chi earthquake.

FIG. 2. Acceleration response spectra of the input motions: (a) 40 pulse-like motions and (b) 40 broadband motions at soil site recorded in earthquakes of Magnitude=7 and Distance ≈ 10 km.

NUMERICAL MODEL

This study considers a typical highway overpass bridge consisting of a deck supported on a single column and a shallow foundation. This type of bridge can be simplified approximately as a single-degree-of-freedom type system in the transverse direction as illustrated in Figure 3a. The model did, however, include the mass and rotational inertia of the footing so it is actually a multi degree of freedom system, but with one dominant mode. A 2-D model was developed in OpenSees (2011) platform. More details of the model are provided by Deng (2012b).

The soil-footing interaction was simulated by decoupled beam-on-nonlinear-Winkler-foundation (BNWF) springs. The nonlinear column hinge was simulated by an elastic-perfectly-plastic rotational spring at the bottom of the column as shown in Figure 3b; the cyclic degradation of actual reinforced-concrete columns is not
considered for simplicity. The footing was supported on a series of gapping bilinear spring elements which can characterize the re-centering mechanism and non-degrading moment capacity of a rocking foundation. The spring elements are uniformly spaced along the footing base. One horizontal soil spring was also attached to the center of the footing element in the model, as depicted in Figure 3b. For the present exercise, a simplistic model is used because it is believed that the mechanics of this simple model are easier to understand, there are fewer parameters to calibrate, and it is adequate for a general comparison of the performance under different types of ground motion.

FIG. 3. (a) side view of a typical ordinary bridge in transverse direction; (b) numerical model developed in OpenSees. The soil-footing contact elements are elastic-perfectly-plastic springs with tension gap; the column hinge element is elastic-perfectly-plastic rotational spring. (Deng et al. 2012b).

**Moment capacity of a rocking foundation.** The rocking moment capacity \( M_{c,foot} \) has been studied by many researchers (e.g., Gajan et al. 2005, Deng et al. 2012a) and has the general definition as in Equation 1,

\[
M_{c,foot} = \frac{Q \cdot (1 + r_m) \cdot L_f \cdot \left( \frac{A_c}{A} - \frac{A}{A} \right)}{2}
\]  

(1)

where \( A_c/A \approx 1/FS_v \) is approximately the inverse of the factor safety \( FS_v \) for vertical bearing and \( r_m = (W_{foot} / Q) \) is the ratio of the footing weight \( W_{foot} \) to the axial force on the column \( Q \). In this study, typical values \( A_c/A = 0.25 \) and \( r_m = 0.2 \) are held constant for each the simulation. \( M_{c,foot} \) can be normalized by \( Q \cdot H_c \) to yield the base shear coefficient of rocking foundations \( (C_r) \), as in Equation 2,

\[
C_r = \frac{L_f \cdot \left(1 - \frac{A_c}{A} \right) \cdot (1 + r_m)}{2 \cdot H_c}
\]  

(2)

The coefficient \( C_r \) is conceptually similar to the base shear coefficient for a conventional hinging column, \( C_y \), which is defined as the ratio of yield moment of a
bilinear column hinge to \( Q^* H_c \). If \( C_y > C_r \), the rocking will be activated prior to column yielding and we would simulate a rocking-foundation system. On the other hand, if \( C_y < C_r \), a hinging column system is modeled.

The strength of the vertical soil springs and the footing width were adjusted so that a certain \( C_r \) value could be achieved. The strength of the column hinge was also varied to obtain a certain \( C_y \) value. The \( C_y \) and \( C_r \) values in this study are listed in Table 1.

**Summary of parameters.** Two column heights were considered for this study and the fundamental system periods \( (T_1) \) are 0.3 s and 0.5 s for short bridges \( (H_c = 3.0 \text{ m}) \) and 0.5 to 1.0 s for tall bridges \( (H_c = 10.0 \text{ m}) \). These periods are common for practical ordinary bridges of similar column heights. The ratio of the rocking stiffness \( (K_R) \) to rotational column stiffness \( (K_{0C}) \), one of the parameters that may affect the determination of the fundamental system periods, was investigated by a few case studies. The \( K_{0C} / K_R \) ratios and other parameters in this study are summarized in Table 1. Deng et al. (2012b) discusses the details of the protocols of the parametrical selection. Note that results from selected parametric studies are presented in this paper.

**Table 1. Values of parameters in this paper.**

<table>
<thead>
<tr>
<th>( H_c ) (m)</th>
<th>Period, ( T_1 ) (s)</th>
<th>( C_y )</th>
<th>( C_r )</th>
<th>( K_{0C} / K_R )</th>
<th>( L_f ) (m)</th>
<th>Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td>Short:</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>3</td>
<td>0.3</td>
<td>0.3</td>
<td>0.4</td>
<td>1.0</td>
<td>2.64</td>
<td>Hinging-column</td>
</tr>
<tr>
<td></td>
<td>0.3</td>
<td>0.4</td>
<td>0.3</td>
<td>2.0</td>
<td>1.92</td>
<td>Rocking-foundation</td>
</tr>
<tr>
<td></td>
<td>0.5</td>
<td>0.3</td>
<td>0.4</td>
<td>1.0</td>
<td>2.64</td>
<td>Hinging-column</td>
</tr>
<tr>
<td></td>
<td>0.5</td>
<td>0.4</td>
<td>0.3</td>
<td>2.0</td>
<td>1.92</td>
<td>Rocking-foundation</td>
</tr>
<tr>
<td>Tall:</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>10</td>
<td>0.5</td>
<td>0.3</td>
<td>0.4</td>
<td>1/4</td>
<td>8.82</td>
<td>Hinging-column</td>
</tr>
<tr>
<td></td>
<td>0.5</td>
<td>0.4</td>
<td>0.3</td>
<td>1/1.5</td>
<td>6.17</td>
<td>Rocking-foundation</td>
</tr>
<tr>
<td></td>
<td>1.0</td>
<td>0.3</td>
<td>0.4</td>
<td>1/4</td>
<td>8.82</td>
<td>Hinging-column</td>
</tr>
<tr>
<td></td>
<td>1.0</td>
<td>0.4</td>
<td>0.3</td>
<td>1/1.5</td>
<td>6.17</td>
<td>Rocking-foundation</td>
</tr>
</tbody>
</table>

**FIG. 4.** (a) normalized moment vs. rotation hysteresis of the rocking foundation subject to cyclic loads; (b) normalized shear vs. rotation of the rocking foundation subject to monotonic loads. The rocking foundation has \( C_\theta = 0.3 \).
Cyclic and monotonic behavior of rocking foundation. Figure 4a shows the normalized moment vs. rotation hysteresis of the rocking foundation. In the analysis, both the footing and the column hinge have negligible post-yield stiffness; past experiments on rocking foundations are consistent with the observation of zero post-yield stiffness. The normalized lateral shear force vs. rotation curves in Figure 4b shows the shear force degradation due to the increasing P-Δ moment as the monotonic footing rotation increases. As the monotonic load increases, the rocking systems eventually reach approximately 30% rotation when the shear force decreases to zero and the rocking system becomes unstable under the P-Δ moment. The rocking foundation has $C_r=0.3$; the associated column remains elastic as the strength of the column hinge is greater than that of the rocking foundation.

RESULTS

This section describes three case studies to investigate the system response to broadband and pulse-like motions. It reveals the failure mechanisms of rocking foundations that are subject to pulse-like or broadband motions and compares the performance of rocking and hinging systems in term of the maximum deck drift.

FIG. 5. (left) Case 1: a rocking foundation at the verge of failure; (right) Case 2: a rocking foundation that collapsed eventually. Both cases were subject to pulse-like motions.

Case studies. The first case study involves a tall-column rocking system ($C_r = 0.3$, $C_y = 0.4$, $T_1=0.5$ s, and $H_c=10$ m) that undergoes a pulse-like ground motion with an amplification factor of 7.0. Figures 5(a) ~ (c) show the time histories of the deck lateral drift, and the velocity time history and the acceleration response spectra of the input motion, respectively. As indicated in Figure 5(b), this motion contains three large velocity pulses with approximately 3.1 s pulse period. The first-mode spectral acceleration $S_a(T_1 = 0.5$ s) was 6.874 g, which applied a very high ductility demand.
on the system. The maximum lateral drift of the deck of the system, as shown in Figure 5(a), was 3 m (or 30% rotation), which reached the instability limit of the rocking system.

However, the intensity of ground motion decreases immediately after the deck reached the maximum 3 m, as indicated by the reduced value of velocity at about time = 4 s, so the rocking system started to re-center back to the original position and avoided overturning. The residual deck drift was 2.2 m as velocity pulses kept shaking the system. If the velocity pulse of the input motion had kept going up or remained constant, the rocking system could have collapsed due to the excessive deck lateral drift.

The second case study, in which the same rocking system collapsed because of a pulse-like motion, is shown in Figures 5(d) ~ (f). The ground motion has $S_a(T_1) = 2.08$ g, which is less than that of Case 1. As indicated in Figure 5(e), the pulse period of the single velocity pulse period of this motion is 5.1 s and the peak-to-peak amplitude of the single pulse is about 10 m/s. Both the duration and amplitude of the single pulse are sufficient to mobilize the failure of the rocking system. Figure 5 indicates that the duration and amplitude of the biggest single pulse play important roles in the stability of rocking systems.

The typical calculated behavior for a broadband motion is shown in Figure 6, which depicts the deck drift time history, velocity time history and acceleration response spectrum of the input broadband motion. Amplified by a large factor, the actual input motion appears somewhat similar to a pulse-like motion because of the large velocity pulses and duration. The deck drift time history shows a few small to intermediate cycles for the initial 45 s, when the input cycles were not very strong; the catastrophic damage to the system was caused by the largest velocity pulse which in this case had a pulse period of approximately 4 s and peak-to-peak amplitude 10 m/s. The deck drift passed the instability limit, and collapse ensued. This case study suggests that the collapse of rocking foundation system is caused by a single, strong velocity cycle, not by the cumulative deck drift. Because the re-centering ability of the rocking foundation brings it back toward center after every pulse, collapse is caused by the biggest pulse. For elastic-perfectly-plastic hinging systems with little re-centering ability, on the other hand, permanent deformations may gradually accumulate during many smaller cycles until $P-\Delta$ takes over and the system collapses.
Performance comparison in a probabilistic sense. Using the methodology of Incremental Dynamic Analysis (IDA), the responses of the systems were obtained and recorded for each ground motion with multiple amplification factors. Herein the spectral acceleration at the fundamental system period, \( S_a(T_1) \), is recognized as the intensity measure of input motions. Figure 7(a) shows IDA curves of \( S_a(T_1) \) vs. maximum lateral drift of the deck (\( \Delta_{\text{max}} \)) of a typical rocking system. The rocking system has \( H_c = 10 \) m and \( T_1 = 0.5 \) s and is subject to 40 pulse-like ground motions. The maximum possible deck drift are about 10 m that is achieved when the 10 m tall column collapses and lays horizontal.

Using the probabilistic quantification method described in Deng et al. (2012b), the distribution of \( \Delta_{\text{max}} \) for a given level of the intensity measure (i.e., \( S_a(T_1) \)) and the median \( \Delta_{\text{max}} \) are obtained, as shown in Figure 7(b). Likewise, the relationship between \( S_a(T_1) \) and median \( \Delta_{\text{max}} \) of all other systems subject to either pulse-like or broadband motions can be obtained for the purpose of comparison.

FIG. 7. (a) \( S_a(T_1) \) vs. \( \Delta_{\text{max}} \) curves as amplification factors of input motions increase in nonlinear dynamic analysis; (b) probabilistic quantification of the distribution of \( \Delta_{\text{max}} \) for a given level of \( S_a(T_1) \).

Figure 8 compares the median \( \Delta_{\text{max}} \) for 10 m columns and 3 m columns for rocking and hinging systems subject to pulse-like and broadband motions. With respect to the rocking systems, in the region of large deck drifts (especially approaching collapse), the median \( \Delta_{\text{max}} \) is greater for pulse-like than for broadband motions; in other words, pulse-like motions are more damaging to a rocking system than broadband motions. Unlike pulse like motions, broadband motions tend to have numerous small cycles, which have a cumulative effect on elastic-perfectly-plastic hinging systems, but little cumulative effect on re-centering rocking systems. A sufficiently large rotation to cause collapse of a re-centering system is apparently more likely for a ground motion that has a single large pulse than a ground motion with numerous small cycles, although the two ground motions may appear to have similar \( S_a(T_1) \) values. This finding has been demonstrated in the case studies in the preceding subsection.

For hinging-column systems, the relationship between \( S_a(T_1) \) and median \( \Delta_{\text{max}} \) was remarkably unaffected by the change from pulse-like to broadband motions. The median \( \Delta_{\text{max}} \) curves representing the results from pulse-like and broadband motions
are almost identical. The observation may be resulted from the assumption of elastic-perfectly plastic (EPP) behavior of the column hinge. The column hinge does not have re-centering capability, and thus cumulative displacement of the EPP hinge could be caused equally by either the few strong pulses of pulse-like motions or many small cycles of broadband motions.

FIG. 8. Comparison of median maximum drift of deck of rocking-footing and hinging-column systems subject to pulse-like and broadband motion suite: (a) \( H_c = 10.0 \) m, \( T_1 = 1.0 \) s; (b) \( H_c = 3.0 \) m, \( T_1 = 0.5 \) s. Legend notation: PL = pulse-like motions; BB = broadband motions.

CONCLUSIONS

The present study conducted parametric analyses of rocking foundation bridges subject to a suite of 40 pulse-like and 40 broadband motions and investigated the effects of ground motion characteristics on the system response characterized by the maximum lateral deck drift. The concept of Incremental Dynamic Analysis was applied in the nonlinear time history simulations. Preliminary results show that,

1. Rocking foundation systems are less likely to collapse than hinging column systems if both systems have the same period and if \( C_y \) of the hinging system is equal to \( C_y \) of the rocking system.
2. In a probabilistic sense, rocking systems are more susceptible to collapse under pulse-like motions than broadband motions. The median \( S_a(T_1) \) of broadband motions required to tip over rocking systems is higher than that of pulse-like motions.
3. Case histories demonstrate that the collapse of a rocking system is commonly caused by velocity pulses of sufficient duration and amplitude that leads to large relative displacement of the deck.
4. Conventional hinging column bridges are not much affected by the change of motions. The column was assumed to be elastic-perfectly-plastic; hence degrading effects which could lead to softening and collapse were not considered.

ACKNOWLEDGEMENTS

The authors sincerely appreciate the financial support from NSF NEES grant #0936503 and the PEER Transportation Research Program under the project Last
Hurdles for Rocking Foundations for Bridges. California Department of Transportation funded the initial development of the numerical model under the grant number 59A0575.

REFERENCES

California Department of Transportation. (2010). *Seismic Design Criteria*, Ver. 1.6, Sacramento, CA.