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ESTIMATION OF DISPLACEMENT DEMAND FOR SEISMIC DESIGN OF BRIDGES WITH ROCKING SHALLOW FOUNDATIONS

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ABSTRACT

Rocking foundations provide many desirable characteristics for a yielding dynamic system; they are economical, they provide a ductile (energy dissipating) moment limiting hinge, and they have a natural re-centering tendency. In order to take advantage of these characteristics in routine practice, simplified design procedures for bridge systems with rocking foundations are required. Two performance targets for deterministic seismic bridge design are: collapse should not occur during the "Maximum Credible Earthquake", and, the serviceability of the bridge should not be compromised during smaller events that are expected during the lifetime of the structure (Functional Evaluation Earthquakes). The crux of the design procedure is the estimation of maximum and permanent displacements with rocking foundations. This paper describes a few candidate procedures, including (1) finite element analysis of a soil-foundation-column-deck-abutment system, (2) modeling the deck-column-foundation as a nonlinear single degree of freedom system, and (3) spectral displacement at an appropriate period of the system.

INTRODUCTION

Through physical model tests and numerical analyses, researchers are now able to reasonably quantify the dynamic behavior of a rocking foundation on competent soil (Gajan et al. 2010) The rocking moment capacity, moment-rotation, and settlement-rotation relationships are reasonably well understood. Rocking foundations provide many desirable characteristics for a yielding dynamic system; they are economical, they provide a ductile (energy dissipating) moment limiting hinge, and they have a natural re-centering tendency. However, the rocking foundation concept has not been fully adopted in the latest seismic design criteria of California.

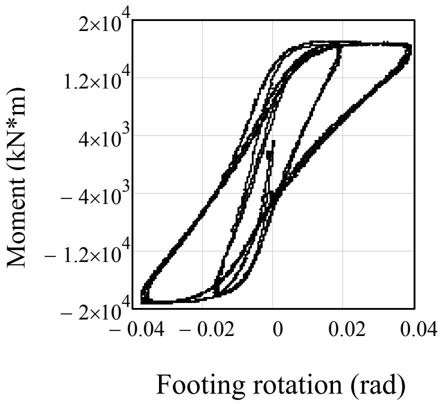
Traditional seismic design of bridges relies primarily on the ability of the column to adequately support the bridge deck loads and in addition, to absorb dynamic displacement demands through plastic hinging in the columns. The foundations are designed to be stronger than the columns in order to force the failure mechanism to be in the columns instead of the foundations. Fig. 1 demonstrates that bridges with strong foundations and yielding columns can potentially fail when the demand on the columns exceed their capacity.

Fig. 1. The collapse of the Hanshin Expressway during the 1995 Kobe earthquake.

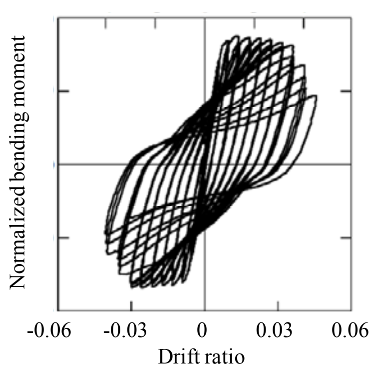
Recent research suggests that a rocking foundation may have the following advantages, particularly, when compared with a nonlinear column.

1) A rocking footing on soil is potentially an effective mechanism of energy dissipation. For example, Fig. 2(a) shows a typical moment vs. rotation curve of a rocking footing subjected to many cycles of loading during a centrifuge model test (Deng et al. 2009a). The large hysteresis loop indicates significant ability to dissipate seismic energy without

significant loss of capacity. Figure 2(b) shows the measured behavior of a reinforced concrete (RC) column. The column moment capacity is seen to degrade under large cyclic deformations.



(a)



(b)

Fig. 2. (a) Non-degrading moment capacity of a rocking foundation under cyclic loading; (b) typical hysteretic loops for cyclically loaded RC columns (Kawashima 2009).

2) A rocking footing that experiences uplift has a tendency to re-center itself as the gap under a footing closes during unloading; this helps to minimize the accumulation of the deck drift.

3) Rocking footings reduce ductility demands on columns.

4) A rocking footing that has suffered residual rotation or settlement during an extreme earthquake may be straightened or elevated by grouting under the footing.

One reason for the lack of acceptance of the concept is that detailed design procedures that account for important issues associated with rocking have not yet been established and accepted. This paper is an attempt to further develop practical design procedures for bridge systems with rocking foundations.

ROCKING MOMENT CAPACITY OF A FOOTING

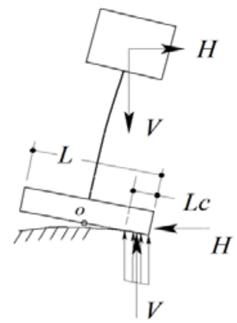


Fig. 3. Critical contact length and rocking moment capacity.

In contrast to the fact that the bearing capacity of a footing is highly uncertain, the moment capacity of a rocking footing can be determined with fairly high accuracy. A rocking footing would ultimately sit on a reduced area of soil, at which point the vertical load is counterbalanced by the bearing capacity on the critical contact area as shown in Fig. 3. This point defines the rocking moment capacity as expressed Eq. (1):

$$M_{c\_foot} = \frac{V \cdot L_f}{2} \cdot \left(1 - \frac{L_c}{L_f}\right) \qquad (1)$$

where V is the vertical load on footing; L_f is the footing length in shaking direction; L_c is critical contact length required to support the vertical load. The ratio L_f/L_c is approximately equal to the traditional factor of safety against bearing failure. Since the size of a spread footing for a bridge is typically determined by settlement considerations as opposed to bearing capacity, it often turns out that the foundations for bridges have a large excess bearing capacity; L_f/L_c values on the order of 10 to 50 are not uncommon. If L_f / L_c = 10 to 50, Eq. (1) shows that the term in parentheses typically varies between about 0.9 and 0.98, and the moment capacity typically varies between 90 and 98% of $V *L_f/2$. Thus, the soil properties are only expected to affect the moment capacity by about 8%. Uncertainties in vertical loads or embedment of the footing as well as friction on the sides of the footings are likely to be a greater contributor to the uncertainty in the moment capacity.

Figure 4 shows results from several centrifuge model tests on sand and clay. The moment is normalized by $V*L/2$. Many shallow foundations for bridges would have L_f/L_c greater than 12, thus expected settlements will be small, and the hysteresis loops become more and more “flag-shaped” (wide at the top and narrow near the origin.

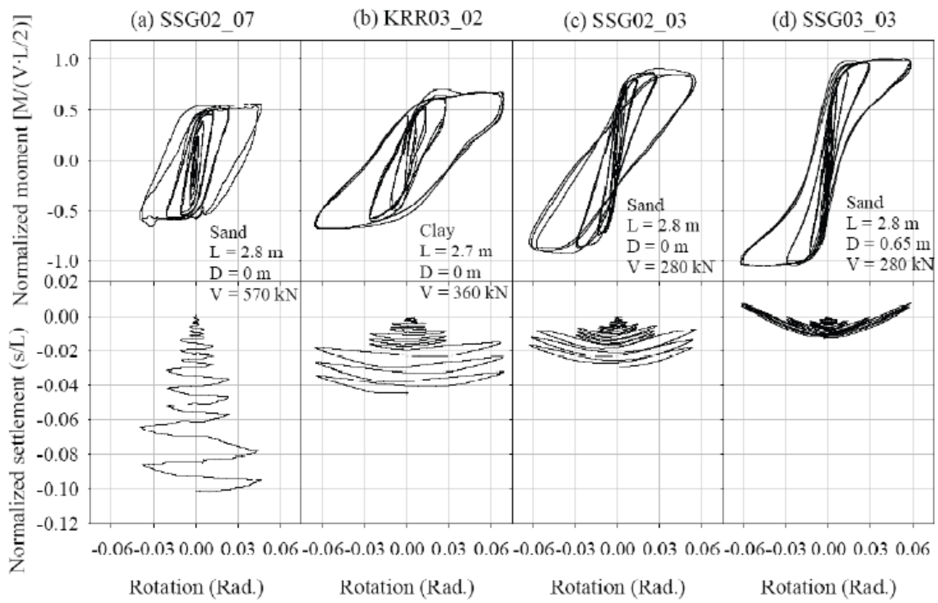


Fig. 4. Effects of L_f / L_c on the moment-rotation-settlement behavior of shallow foundations for cases where L_f / L_c equal (a) 2.2; (b) 3.0; (c)3.8; and (d) 12.3 (Gajan & Kutter 2008).

It should be clarified that the rocking moment capacity differs from the structural moment capacity of a footing. The latter capacity refers to the structural strength of a footing plate, commonly analyzed as if it was a cantilever beam.

NEW DESIGN PROCEDURE FOR BRIDGES WITH ROCKING FOUNDATIONS

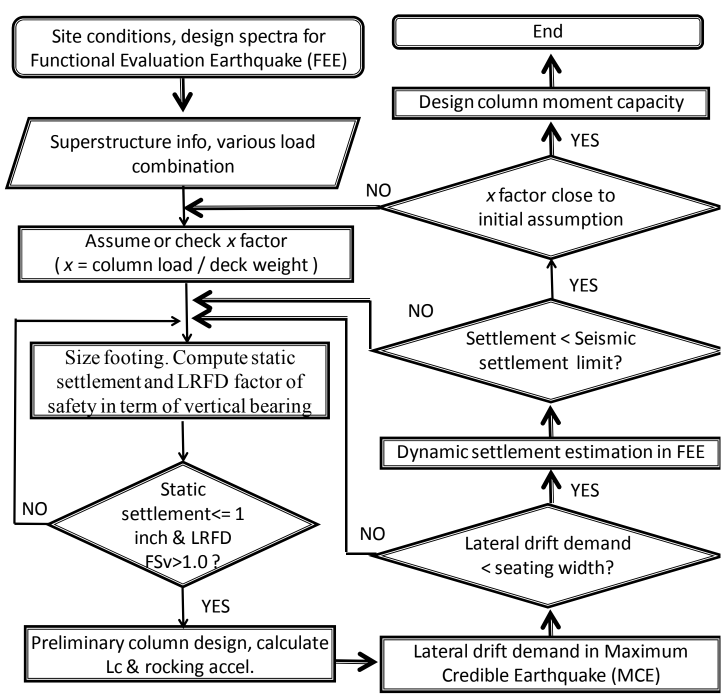


Fig. 5. Flow chart for the new design procedure.

Figure 5 shows flow chart of the proposed design procedure; an overview of this procedure is presented by Deng et al (2010). Fig. 6 illustrated an ordinary two-span bridge with spread shallow foundations and two-column bent that could potentially be designed with the new procedure. Many aspects of the design process such as characterization of soils and earthquake hazards, determination of the bridge dimensions and evaluation of deformations under dead and live loads are similar to traditional processes and are not described in this paper. The portions of the process that differ from tradition are described in more detail below.

<u>Load distribution between columns and abutment</u>

As indicated by the third box of the flow chart of Fig. 5, determination of the tributary loads on individual footings in each bent, including consideration of loads shared by the abutments is explicitly evaluated in the proposed design procedure. This load sharing is quantified by an "*x* factor" defined in Fig. 7. The *x* factor is the fraction of the deck weight that is taken by the footings in the bent under consideration. Other bents and the abutments must support the remainder of the weight. For a two-span bridge system, for example, it might be assumed that half the vertical load is taken by the abutments and the other half is equally shared by the columns supporting the mid-span bent. Hence, for such a system, $x = 0.5$ should be a reasonable estimation.

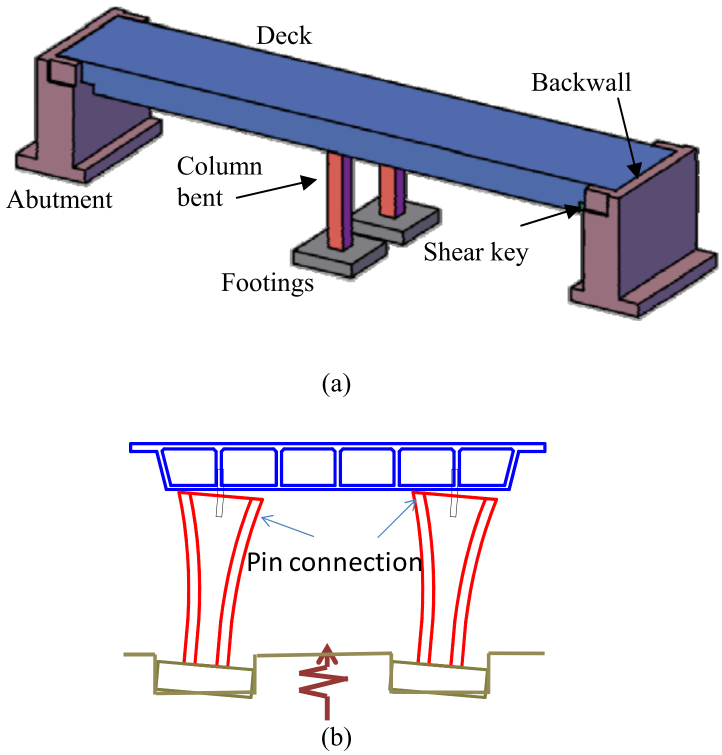


Fig. 6. Schematic of ordinary bridge system and detail showing a hypothetical in connection between the bridge deck and columns (a) 3-D view (b) vertical view.

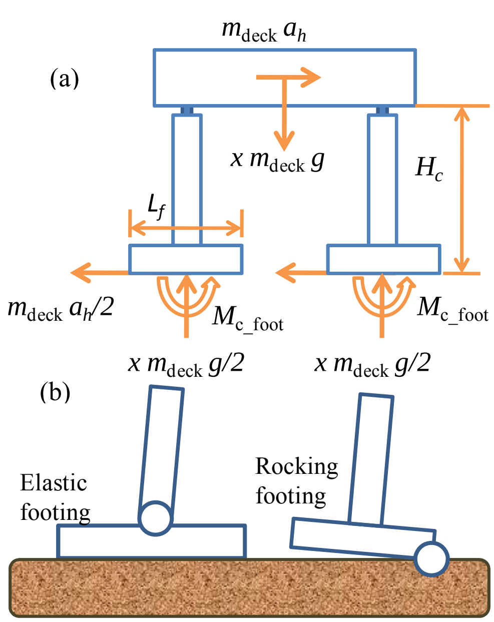


Fig. 7. (a) Free body diagram of loads on a bridge bent with pins at the column-deck connection. (b) Simplified systems with plastic hinging at the base of the column and hinging due to local yielding of the soil near the toe of a rocking footing.

The load sharing will be affected by relative settlement of the abutment and footing, and, as is apparent from Eq. 1, the rocking moment capacity of the footing is sensitive to the vertical load on the footing. Therefore explicit evaluation of the x-factor is a significant step in the proposed design procedure.

Protection of the columns

A premise of this procedure is that it is desired to make the foundations rock to protect the columns from damage. By making a pin connection between the bent cap beam and the top of the columns (Fig. 6(b)), it is possible to limit the moments that are applied to the top of the columns. The column is assumed to be fixed to a footing that can rock when a specified moment is obtained at the soil-footing interface. If the rocking moment capacity is smaller than the column moment capacity, a hinge will form in the soil beneath a loaded edge of the footing instead of in the column as indicated in Fig. 7(b). Hence the system shown in Fig. 7(a) may displace laterally without damage to the column and still take advantage of the ductility, energy dissipation, and re-centering characteristics of the rocking foundation.

Placing a pin connection at the top of the columns has additional ramifications that need to be considered. For example, it is not usually practical to use a pin connection at the top of a single-column bent because the pin would not provide torsional resistance of the deck necessary to resist large truck loads in the outer lanes of the bridge. If rocking foundations are to be used with single column bents, a moment connection would be required at the top of the column and then it must be understood that rotation of the footing will result in torsional rotation of deck. The effects of deck torsion can be accounted for in a finite element analysis of a soil-footing-column-deck-abutment system (e.g., the FE model sketched in Fig. 8). As the FE system analysis may not be practical for routine design, and to simplify the problem, the presently proposed procedure will assume that the system uses multi-column bents and pin connections at the top of the columns. An effective "pin" connection at the top end of a cast-in-place reinforced concrete column may be obtained in a manner similar to the construction of shear keys at the base of a column wherein a certain amount of longitudinal reinforcement is extended across a gap between the column and cap beam. This paper limits focus to the design of ordinary standard bridges with pin connections at the top of the columns, seat-type abutments with negligible resistance due to shear keys and abutment backwalls. This paper does not consider asymmetry due to curvature or skew of the deck.

After the deterministic design procedure is developed and accepted, it would be desirable to recast the procedure in a performance based design framework that accounts for lifecycle costs (including costs associated with construction, loss of functionality, and repair) and integration of probabilistic hazards from all sources. However, the present paper is restricted to a deterministic design of the system based on maintaining life safety (by prevention of collapse) in the Maximum Credible Earthquake (MCE) and preservation of serviceability in a Functional Evaluation Earthquake (FEE) (Housner et al.1994).

Prediction of Displacements

The key step in the proposed procedures for assessing collapse or functionality lies in the estimation of the maximum and permanent deformations. The mode of collapse most likely to be critical is the loss of seating – the bridge deck falling off the vertical bearing supports. This can be avoided by making the seat widths greater than the anticipated relative displacements in the MCE. Another mode of collapse is that the rocking foundations will tip over which again can be avoided in design by making sure that the displacements are small enough to ensure that P-Δ moments do not become greater than the moment capacity of the footing. The natural self re-centering introduced by the rocking mechanism tends to naturally discourage the accumulation of drift during repeated loading.

The assessment of functionality following an FEE is based upon the assessment of the magnitude of permanent deformations due to settlement and lateral drift. It is suggested for functionality that relative deformations should be less than 0.004 times the bridge span (AASHTO 2008). For both MCE safety and FEE serviceability evaluations, methods for predicting drift and settlement are at the crux of the problem. The remainder of this paper discusses different methods for assessing these deformations.

Rocking acceleration and initial rocking stiffness

Rocking acceleration is defined as the horizontal acceleration of the deck to mobilize the rocking moment capacity of the footing, when the footing rests on the critical contact area. The rocking acceleration is analogous to the yield acceleration concept defined by Newmark (1965). From the free body diagram in Fig. 7(a), the rocking acceleration, a_h, for a two-column bent bridge could be obtained from Eq. (2).

$$\frac{a_h}{g} = \frac{x \cdot L_f}{2 \cdot H_c} \cdot \left(1 - \frac{L_c}{L_f}\right) \tag{2}$$

For standard bridges, typical values of a_h may range between 0.1 and 0.3 depending on the x factor, column height and footing dimensions.

FEMA (2000) provides the following expression for the initial rocking stiffness:

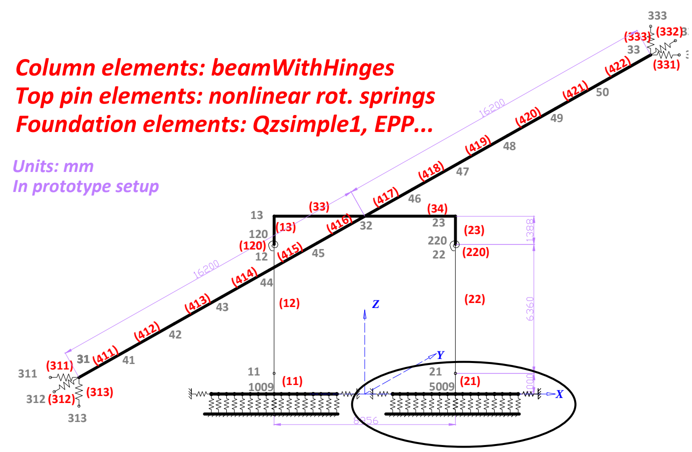
$$K_{rock} = \frac{G_0 \cdot B_f^3}{1 - \upsilon} \cdot \left(0.4 \cdot \frac{L_f}{B_f} + 0.1\right) \cdot \left[1 + 2.5 \cdot \frac{t_f}{B_f} \cdot \left(1 + 2 \cdot \frac{t_f}{B_f} \cdot \left(\frac{t_f}{D}\right)^{-0.2} \cdot \sqrt{\frac{B_f}{L_f}}\right)\right] \tag{3}$$

Centrifuge model test results suggest that the stiffness from Eq. (3) may overestimate observed stiffness.

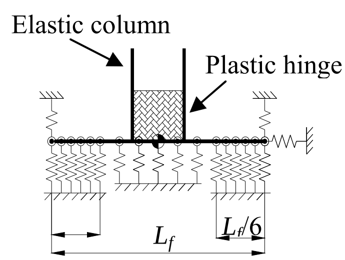
METHODS OF ESTIMATING LATERAL DISPLACEMENT DEMAND

This section provides three approaches for estimating the lateral displacement demand of the bridge superstructure subjected to a maximum credible earthquake. Included are the nonlinear dynamic FE method using OpenSEES, nonlinear SDOF analysis using BiSpec, and a relatively simple method based on elastic response spectra.

FE model of soil-foundation-column-deck-abutment system



(a) Model configuration



(b) Detailed BNWF model

Fig. 8. Three-dimensional model with Beam-on-Nonlinear-Winkler foundation: (a) model configuration; (b) detailed BNWF model.

Figure 8 shows the 3-D model of a soil-foundation-column-deck-abutment system built using elements available in OpenSEES (PEER 2008). The superstructure (deck) is simulated by elastic beam elements; columns are elastic beams with finite nonlinear hinges (Scott & Fenves, 2006). Beam-on-Nonlinear-Winkler Foundation (BNWF) elements support the base of the footing and represent the flexibility and yielding of the soil. Parameters for these elements, described by Gajan et al. (2010), are selected to model the rotational stiffness, the rocking moment capacity and the gapping and uplift beneath the unloaded portion of the foundation, the yielding under the loaded edge of the footing during rocking, and the accumulation of settlement. Following the provision of FEMA 2000, the footing base is divided into the end and central zones with springs at the end-zones (Lf/6 width) mainly providing rocking stiffness while the central zone provides mainly vertical loading stiffness. Harden & Hutchinson (2009) and Gajan et al. (2010) provide guidelines for parameter selections based on the calibrations against prior centrifuge and large-scale shallow foundation tests. Details on the configuration and attributes of the specific BNWF model are outlined in Deng et al. (2009b). The finite element model has been calibrated with the data from centrifuge modeling tests.

To illustrate the type of results that can be obtained from the analyses, example calculations were carried out for a representative bridge with two-column bent and rocking footing. The numerical model of the bridge is based on a real two-span two-column-bent prototype bridge in southern California. The footing size (L_f) is 5.04 m square, and column height (H_c) is 6.77 m. The bridge deck weighs 1169 metric tons. The natural period of the bridge system is estimated to be 0.94 sec considering the initial stiffness of the rocking footing using Eq. (3) and column stiffness. However, the finite element model indicates the first-mode natural period of 1.57 sec due to a softened initial stiffness of the rocking footing.

Two different earthquake input motions were selected for the example calculations. These motions (TCU071-E and TCU088-N) were recorded in the 1999 Chi-chi earthquake. The acceleration time series of these motions are shown in Fig. 9 and the acceleration response spectra are presented in Fig. 10. The two motions were scaled to make the spectral acceleration at a period of 0.94 sec to match the selected Caltrans design spectrum. These input motions were applied as uniform excitation to all of the fixed ends of the springs indicated in Fig 8.

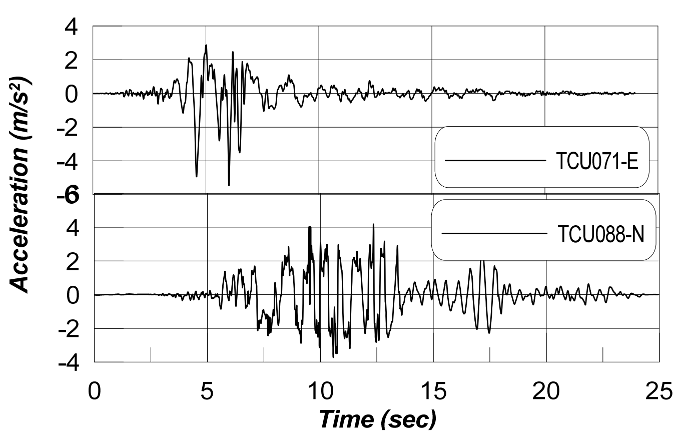


Fig. 9. Time histories of selected earthquake motions for sample calculations.

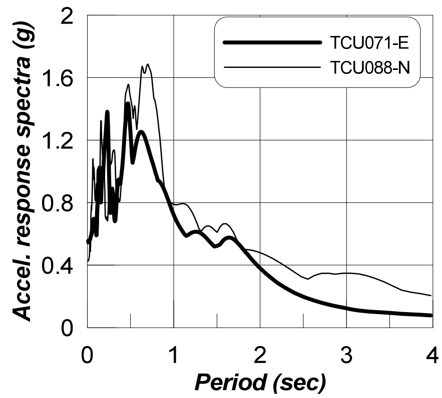


Fig. 10. Acceleration response spectra of selected motions.

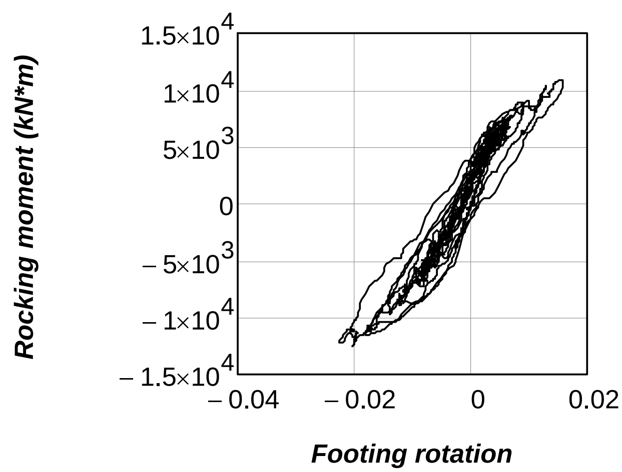


Fig. 11. Rocking moment vs. footing rotation curve of the rocking footing subjected to the TCU088-N motion.

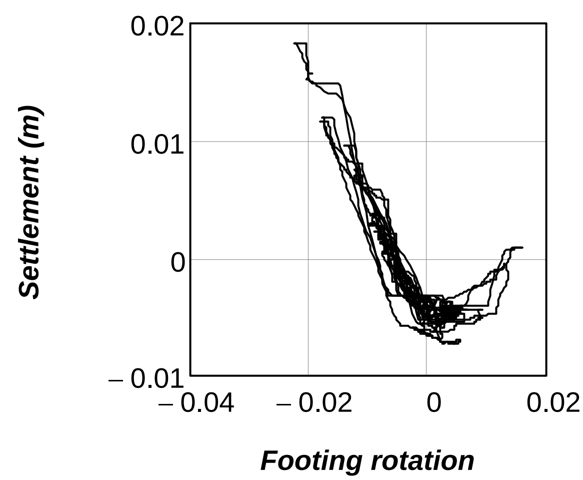
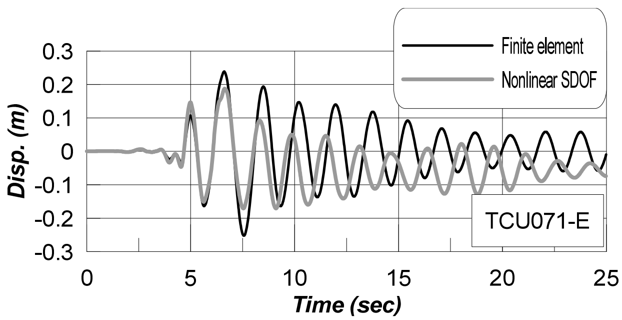


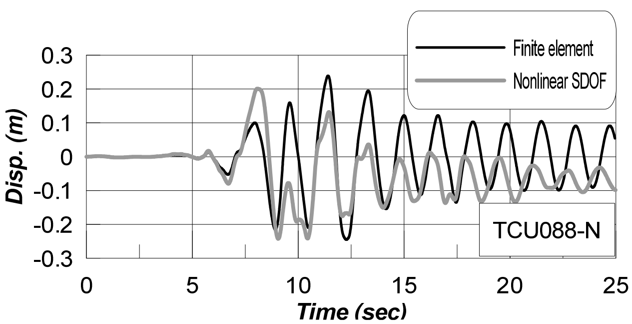
Fig. 12. Settlement vs. footing rotation curve of the rocking footing subjected to the TCU088-N motion.

Figure 11 shows the hysteretic loops of rocking moment on the footing vs. footing rotation for the TCU088-N motion. The loops exhibit the banana-shaped pattern similar to the slow cyclic tests shown in Fig. 4. The rocking moment capacity from finite element modeling is very close to the theoretical value (M_{c_foot}= 1.155*10^4 kN*m) from Eq. 1. Figure 12 shows the computed settlement-rotation relationship for the footing. The figure reasonably describes the uplift and settlement of the center of the footing together with the footing rotation in each cycle. It also indicates a negligible cumulative settlement.

The time histories of the relative displacement of the deck calculated in the finite element analysis are depicted in Fig. 13. It is observed that the bridge deck rocks back to the initial positions with little permanent drift. The maximum drifts of the deck are about 250 mm for both earthquakes. For comparison, the figure also includes the results from the nonlinear SDOF analysis using BiSpec that is described in the following sub-section.



(a)



(b)

Fig. 13. Displacement time histories of the deck using FEM compared to the nonlinear SDOF analysis (for elongated period T=1.57 sec) with the input motions (a) TCU071-E (b) TCU088-N.

<u>Nonlinear SDOF Analysis</u>

BiSpec (Version 1.62) was used to estimate seismic demands by modeling the system as an elasto-plastic (bilinear) system. BiSpec allows users to select from a variety of hysteretic models and set elastic natural period, viscous damping ratio, and post-yield stiffness, and the force at yield. For the simulations presented in Figs. 13 and 14, two system natural periods were used. The natural period (based on initial stiffness) was set to 0.94 sec. Another set of analyses was conducted assuming the elastic stiffness reduced by a factor, providing a natural period of 1.57 sec (actually this period is taken from the finite element model). For all BiSpec calculations presented here, the viscous damping was set to 5%, and post yield stiffness was set to zero. The yield strength of the bilinear spring was set to produce a yield acceleration of 0.17g, the value obtained from Eq. (2) with x = 0.5.

The same input motions (TCU071-E and TCU088-N) were used for the BiSpec analyses as were used for the finite element analysis. The results from the BiSpec calculations are compared to FE Model calculations in Fig. 8. Despite the simplicity of the BiSpec model, it is able to reasonably capture the cyclic displacement demand predicted by the FE Model. The BiSpec hysteresis model does not account for the re-centering properties of a rocking system, so the residual drift of the BiSpec model is larger than the drift predicted by the OpenSEES model.

For assessment of the unseating problem the maximum drift of the deck may be compared to the bearing seat width. Thus the maximum displacement of the deck is an important quantity. A plot of spectral displacement for a nonlinear SDOF system is one way to estimate the drift. For T = 1.57 sec, the predicted maximum drift output from BiSpec was 0.18 m (7.3 inch) and 0.24 m (9.3 inch) for the two input motions respectively. For T = 0.93 s, the predicted maximum drift output from BiSpec was 0.10 m (4.1 inch) and 0.15 m (5.8 inch) for the two input motions. These predictions were less than that of the OpenSEES simulations, but they may be close enough for some design purposes.

From Fig. 10, the spectral content of the motions are quite similar for periods smaller than about 2 sec. The input motion spectra are quite different at longer periods (e.g. T=3 sec); the TCU088-N motion is about 3 times larger than the other motion. This difference at long periods is accentuated in a plot of spectral displacement (Fig. 15).

Figure 14 shows the hysteresis curves predicted by the BiSpec model with the elongated period (T=1.57 sec) and a_h=0.17 g). The figure illustrates the bilinear constitutive behavior of the SDOF system, the yield acceleration, and maximum displacement which becomes the demand to the system. The SDOF system of shorter period (not shown in the figure) indicates a smaller maximum displacement as expected; the softer system produces larger elastic displacement and smaller plastic displacements. It should be mentioned that the relations between residual drift and elastic stiffness or rocking acceleration display some chaotic behavior. Sometimes small changes in one parameter can lead to a large change in residual drift. Figure 15 shows a spectrum of the maximum relative displacement of an elastic, perfectly plastic SDOF system as a function of elastic natural period. Note that each point in Fig. 15 represents the maximum absolute displacement computed by dynamic analysis of the single degree of freedom system. For all of the nonlinear calculations, the rocking acceleration was held constant at 0.17g. As the systems become more flexible (long period), the systems become elastic and the elasto-plastic spectra converge to the elastic spectra.

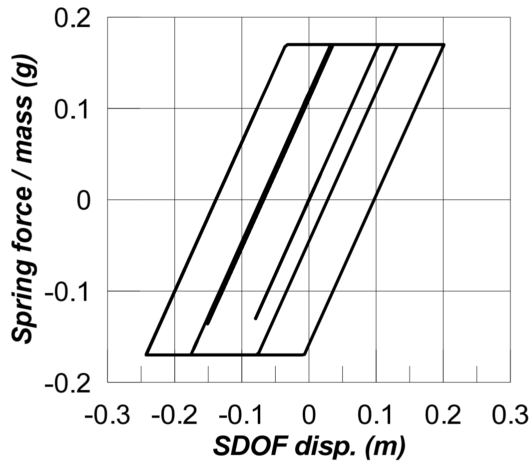


Fig. 14. Acceleration vs. displacement loops for the SDOF model with period T=1.57 sec subjected to scaled TCU088-N motion.

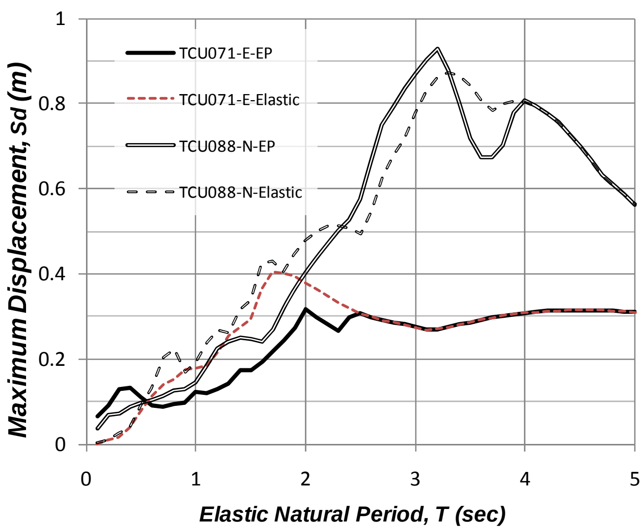


Fig. 15. Spectral displacement as a function of frequency for two ground motions.

<u>Spectral method</u>

Figure 15 compares the spectral displacements for linear systems to those for nonlinear systems. It can be seen that the elastic displacement spectrum quite reasonably tracks the elasto-plastic displacement spectrum except at very short periods. This general tendency has been called the “equal displacement rule”. In structural engineering, it is commonly suggested that the equal displacement rule applies reasonably well for periods greater than about 0.5 to 1 sec. (The equal displacement rule does not, however, apply for systems with nonsymmetrical resistance; therefore, caution should be exercised if it is used for curved and skewed bridges.)

If the equal displacement rule is accepted, then the method of estimating the seismic demand can be based on existing elastic design spectra. This eliminates the need to select specific ground motions.

The design response spectra for the MCE and FEE motions, selected from the Caltrans (2006) Seismic Design Criteria 1.4, are shown in Figs. 16 and 17. In these studies, the selected MCE spectra corresponds to M_w=8.0±0.25 and PGA=0.5 g, whereas the FEE uses the spectra for an event with M_w=6.5±0.25 and PGA=0.4 g.

Assuming linear elastic column and footing stiffness, the elastic period of the bridge system is 0.94 sec. At this period the spectral acceleration for MCE may be read directly (0.88 g) and multiplied by $(T/2\pi)^2$ to produce an estimate of the displacement demand of 0.19 m (7.6 inch).

If the degraded stiffness (T = 1.57 sec) is used, the resulting acceleration demand would be 0.58 g and displacement demand would be 0.35 m (13.8 inch). It is seen that the displacement demand from the spectral approach is comparable to the other approaches.

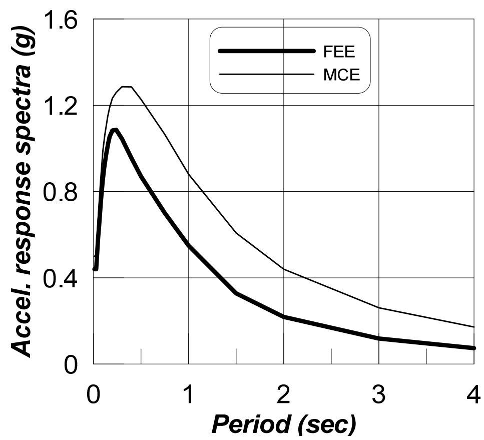


Fig. 16. Design acceleration response spectra for MCE and FEE (selected from spectra database, Caltrans 2006)

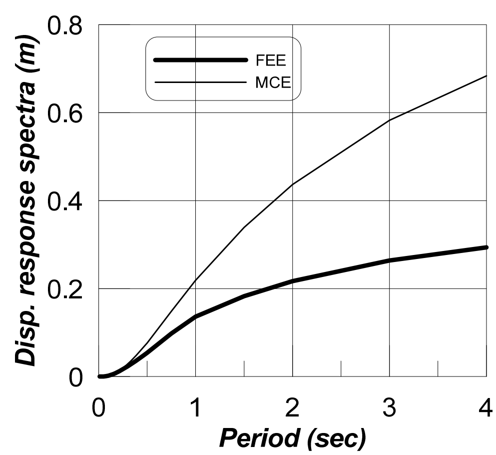


Fig. 17. Design displacement response spectra for MCE and FEE (Caltrans 2006)

Table 1 summarizes all the lateral displacement demands using the three approaches presented in this paper. Obviously, as the period elongates, the displacement demand on the superstructure (bridge deck in this study) becomes larger.

Table 1. Displacement demands from the various procedures for MCE.

Methods	Input motions	Elastic period (sec)	Displacement Demand (m)
3-D Finite element	TCU071-E	1.57	0.24
3-D Finite element	TCU088-N	1.57	0.25
Nonlinear SDOF	TCU071-E	0.94	0.10
Nonlinear SDOF	TCU088-N	0.94	0.15
Nonlinear SDOF	TCU071-E	1.57	0.18
Nonlinear SDOF	TCU088-N	1.57	0.24
Spectral method	Caltrans ARS	0.94	0.18
Spectral method	Caltrans ARS	1.57	0.35

SETTLEMENT DEMAND IN FEE

Gajan and Kutter (2008) and other researchers have shown that foundation rocking can lead to permanent settlements, even if the potential for collapse is negligible. Hence it is recommended that the magnitudes of settlements be assessed for moderate levels of shaking that are likely to occur during the lifetime of the bridge structure. In other words the structure should remain in service during the "functional evaluation earthquake". It is a matter of debate as to what level of permanent deformation is allowable if a bridge is to remain functional. AASHTO (2007) suggests that the magnitude of settlement allowed is about 0.4% of the span; for a 25 m span, this amounts to an allowable settlement of about 100 mm. Presently, however, Caltrans has a stricter requirement that due to dead loads and live loads, settlements should not exceed 25 to 50 mm. The allowable settlement that can preserve acceptable serviceability following the FEE needs to be finalized.

One of the candidate methods to estimate the magnitude of the settlements is the FE analysis of the system with the BNWF model of the foundation soil. The BNWF model has been calibrated (Gajan et al. 2010) to produce reasonable predictions of settlement. To use the nonlinear FE model analysis method, it would be necessary to select a number of ground motion time histories that represent the FEE; the settlement would then be a direct output of the FE model.

A new "simplified" procedure for estimating settlement based on elastic spectra is in the process of being refined and reviewed. One of the versions of the simplified procedure involves four steps:

(1) Calculate the lateral displacement demand using the same procedures explained in the prior section but with the design acceleration spectra for the FEE.

(2) Estimate the magnitude of the rocking footing rotations as being equal to the displacement divided by the deck height.

(3) Estimate the equivalent number of cycles of rocking.

(4) Use the settlement per cycle vs. amplitude of rotation charts presented by Gajan & Kutter (2008), shown in Fig. 18, to estimate the settlement demand during FEE.

$$s = c \cdot L_f \cdot \sum_{i=1}^{N} \theta_i^{rock} \qquad (4)$$

Values for the coefficient, c, deduced from Fig. 18 are listed in Table 2.

Gajan and Kutter (2008) summarized settlements due to rocking observed in many experiments in the centrifuge and laboratory model tests as shown in Fig. 18. This figure suggests that the amount of settlement is proportional to the rotation of the footing and that it decreases as the ratio of 1-A_c/A = 1-L_c/L_f increases. As indicated in the right most panel of Fig. 18, residual uplift was observed to occur for foundations with a 1-A_c/A greater than 0.94. For values less than 0.94, settlement was observed.

The 1-L_c/L_f for the sample bridge is 0.937, which locates in the third panel of Fig. 18. Therefore a settlement coefficient 0.2 is assumed to the settlement equation. The FEE motions used in FE analysis and nonlinear SDOF method are scaled in the way such that the spectral acceleration at 0.94 sec (which is the elastic natural period of the bridge) of these motions are the same as the design spectra at 0.94 sec of FEE shown in Fig. 16.

The method for determining the equivalent magnitude and number of cycles needs to be finalized, when the spectral displacement method is utilized. For the present example, we assume 3 cycles with the maximum total displacement demand. We expect this method to be conservative; the footing rotation that causes settlement will be less than the total drift divided by column height because some of the drift is caused by column flexibility and elastic rotation of the footing.

Except for the model of 1.57 sec period using the nonlinear SDOF method, the estimated settlements for this example is less than about 100 mm, which is likely to be less than the AASHTO serviceability limit of 0.004 times the span.

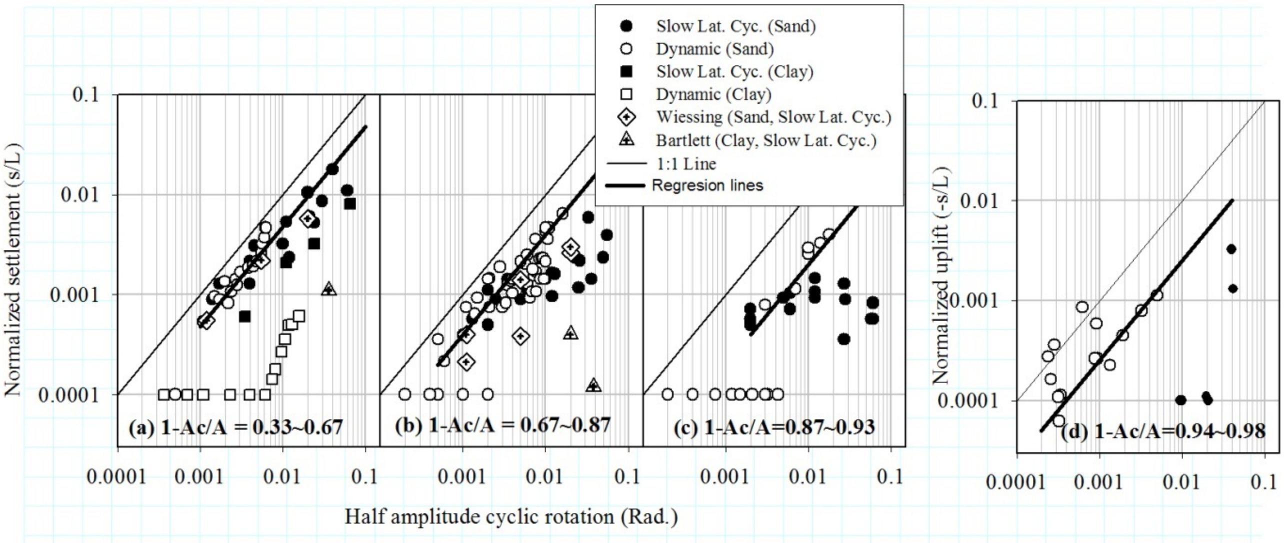


Fig. 18. Normalized settlement vs. half amplitude cyclic rotation of a rocking footing (redrawn from Gajan & Kutter, 2008)

Table 2. Empirical equations for the normalized permanent settlement

Figure	1-A_c/A	Coefficient *c* factor as in Eq. (4)
18-(a)	0.33~0.67	0.5
18-(b)	0.67~0.87	0.4
18-(c)	0.87~0.93	0.2
18-(d)	0.94~0.98	-0.25 (uplift)

Table 3. Settlement demands at various earthquake levels

Methods	Input motions	Elastic period (sec)	Settlement (cm)
Nonlinear SDOF	TCU071-E	0.94	5.1
Nonlinear SDOF	TCU088-N	0.94	5.7
Spectral method	Caltrans ARS	0.94	5.8
3-D Finite element	TCU071-E	1.57	6.7
3-D Finite element	TCU088-N	1.57	6.5
Nonlinear SDOF	TCU071-E	1.57	14.3
Nonlinear SDOF	TCU088-N	1.57	11.5
Spectral method	Caltrans ARS	1.57	7.6

SUMMARY AND CONCLUSIONS

This paper describes methods for assessment of lateral displacement demands on bridges with rocking foundations. Three methods are presented.

The first is based on nonlinear dynamic analysis of a 3-D FE model of the soil-footing-column-deck-abutment system. The analysis used nonlinear columns, and Beam on Nonlinear Winkler Foundation elements to model the rocking foundations. This method is capable of predicting the displacement demand on the deck during the Maximum Credible Earthquake (MCE) that is needed to assess the potential for collapses. It also has the capability to predict settlement in the Functional Evaluation Earthquake so that performance can be checked against serviceability limits. . The nonlinear dynamic analyses require consideration of a number of candidate ground motion time histories. For this paper, only two different ground motions were considered. For application in design, it is expected that a larger number of time histories would need to be considered.

The second method is based on nonlinear dynamic analysis of single degree of freedom oscillators with elastic perfectly plastic “springs”. The elastic stiffness of the SDOF system is set to match the natural period of the system and the yield point is selected to match such that the yield acceleration is equal to the rocking acceleration. The program BiSpec was used to carry out these analyses. The results were shown to provide a reasonable approximation of the cyclic displacement demand predicted by the FE model. However, the hysteresis model used did not properly model the re-centering properties of rocking foundations and hence the permanent drift may not be overestimated using this method.

In addition, a simplified method, justified by the “equal displacement rule”, is to estimate the displacement demand of the deck directly using design spectra specified by the Caltrans Seismic Design Criteria.

As it is known that rocking foundations can also result in foundation settlement, methods for assessing foundations settlements were also presented. Empirical relationships between the settlement per cycle of rotation and the amplitude of the rotation presented by Gajan & Kutter (2008) were used for this purpose.

A bridge system with multi-column bents, rocking foundations supporting each column, and pin joints at the column-deck connections is a good candidate bent configuration for a seismically resistant bridge system. For this configuration, a simple equation for estimating the acceleration required to cause rocking is proposed.

The routine application of rocking foundations in practice is presently hampered by a lack of understanding of the mechanics of rocking foundations and the absence of an accepted practical design method. This paper is intended to report a step forward toward establishment of accepted practical design procedures. But additional work still needs to be done, especially in finalizing simplified design procedures for estimation of settlement associated with rocking.

Rocking foundations are less expensive than pile foundations, and they have some performance benefits over rigid foundations. Rocking foundations can act as effective energy dissipation components of the bridge system; they have re-centering properties and can act as base isolators to protect the columns from excessive ductility demands.

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