



INTRODUCTION TO A NEW DESIGN PROCEDURE FOR ORDINARY BRIDGES WITH ROCKING FOUNDATIONS

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ABSTRACT

It is proposed that rocking foundations for bridges can provide improved performance and reduce cost when compared to bridges with deep foundations and columns that require high ductility capacity to accommodate seismic demands. Rocking foundations can reduce demands on columns and provide a mechanism of re-centering to control the accumulation of drift. An outline of a new proposed design procedure for bridges with rocking foundations is presented. The procedure requires that the foundations be designed as an integral, energy dissipating part of the soil-foundation-column-deck-abutment system. Allowing the foundations to rock induces the possibility that settlements will accumulate; practical procedures for estimation of settlements that might occur during shaking are therefore needed and a few approaches are proposed in this paper.

Introduction

Traditionally, structural bridge engineers have preferred to design bridges that have foundation and deck systems that remain essentially elastic during seismic loading. This is accomplished by making the foundations and deck significantly stronger than the columns of the bridge. In this way, the large majority of yielding, ductility, and energy dissipation demand are placed on the bridge columns. Alternatives to relying mainly on the columns for absorption of displacement demand is to introduce rocking foundations at the base of the columns or through use of other innovative energy dissipation and base isolation systems. When compared to rocking foundations, yielding concrete columns have the advantages that the strength and stiffness of the columns can be determined by codified procedures involving the column diameter, the amount and type of reinforcement and concrete strength. Furthermore, damage to columns could be easily inspected. Too much reliance on reinforced concrete columns for energy dissipation may, however, also have disadvantages compared to rocking foundations.

Rocking shallow foundations have been shown to be capable of providing high ductility and energy dissipation (Gajan & Kutter 2008). Rocking foundations could work as an alternative to plastic columns as an energy dissipater. If the footing is designed to rock before the column yields, the column can be isolated from excessive ductility demand. As the footing rocks, a gap opens up as the unloaded side of the foundation lifts up off the ground. After the earthquake motions stops,

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the gap closure results in “re-centering” of the bridge deck. Imagine tilting your refrigerator a few degrees and letting go. As long as the tilt is not too large, the refrigerator ends up sitting nearly level. Economically, the rocking footings are more advantageous because they are smaller than the footing designed not to rock and are cheaper than deep foundations.

One of the reasons that rocking foundations have not been widely adopted as a desirable and cost-effective foundation for bridges is that methods for assessing the behavior and performance of rocking foundations have not been adequately verified and codified for acceptance by the profession. On the path to broadening the acceptance of foundation rocking in the seismic bridge design, this paper introduces an outline of factors that need to be considered in such a design process. The scope of this paper is focused on ordinary 2-span bridges such as that picture in Fig. 1, although the concepts may be extended to a wide range of bridge types. Fig. 2(a) depicts the moment vs. rotation behavior of a typical rocking shallow foundation subjected to slow cyclic loading (Deng & Kutter 2009). The hysteresis indicates a good energy dissipater, a non-degrading rocking moment capacity and the re-centering property.



Figure 1. An ordinary bridge with double-column bents and abutments.

Conventional Design of Bridges with Strong Foundations and Ductile Columns

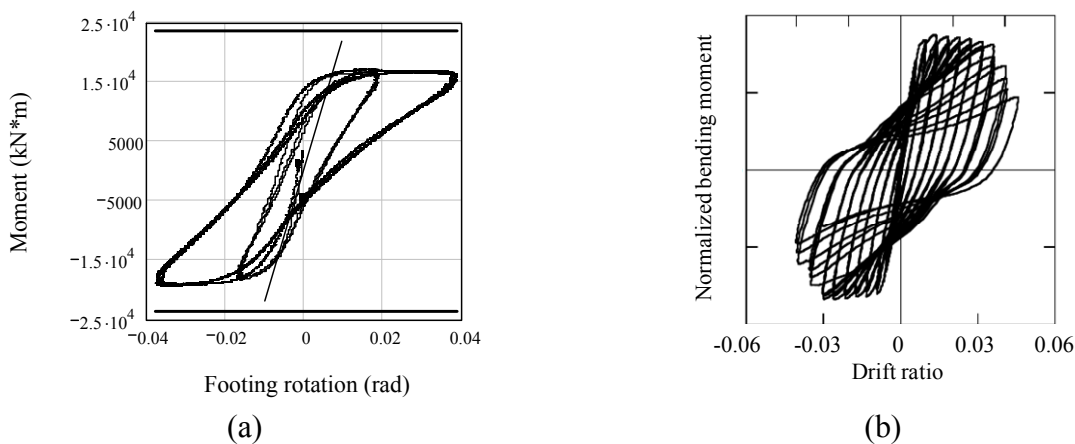


Figure 2. Typical moment vs. drift ratio of (a) concrete column and (b) rocking footing

The conventional procedure for the seismic bridge design (Caltrans 2006) requires the foundations to be strong and stiff relative to the bridge columns. As the bridge deck is also designed to remain elastic, the columns are the weak links in the system. The columns must have sufficient strength to control drifts to acceptable levels and in addition, they must provide sufficient ductility to prevent collapse after many cycles into the nonlinear range. In such a system, the columns provide most of the energy dissipation in the system. A large amount of research has been performed to quantify the ductility capacity of bridge columns as well as to

develop new concepts for design of ductile columns (e.g., Fig. 2(b) shows a typical response of a concrete column in terms of bending moment vs. drift ratio (Kawashima 2009)).

Fig. 3 shows a simplified flowchart summarizing key features of the Caltrans seismic design procedure for bridges with elastic footings. The procedure starts with the specification of site conditions, ground motion hazard, and design ground motion spectra. Traditionally, the goal of the seismic design is to prevent collapse in the maximum credible earthquake (MCE). It is presumed that the serviceability of a bridge designed to this standard will be satisfactory after a more frequent but less intense “functional evaluation earthquake”.

The determination of the geometry and loading conditions of a bridge (e.g., number of lanes, span, skew, traffic and wind loading) is assumed to be decided prior to the seismic design. Bridges may be supported by multi-column bents (Fig. 4(a)) or a single-column bent (Fig. 4(b)). Lateral loads may be caused by earthquake, wind or vehicle inertia forces. The bent geometry and column fixity conditions must consider the eccentric vertical load due to vehicular traffic. If the bridge uses single-column-bent(s), then columns must have a moment connection at the bent cap beam and a moment connection at the bottom connection to the foundation. If the bents are supported by more than one column Fig. 4(a), the system can withstand eccentric traffic loads even if there is a “pinned” connection at one end of each column as indicated in Fig 4(a). The pinned connection is able to transmit axial and shear loads, but negligible moment. 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.

The columns next are designed to withstand dead loads and live loads.

The natural period of the bridge is determined, often by assuming the foundations are rigid and assuming that the lateral transverse lateral loads taken by the abutments are negligible. Seismic displacement demand is estimated to be equal to the spectral displacement at the first mode period of the system based on the MCE. Some procedures require accounting for the elongation of the period caused by yielding of the column, perhaps using “equivalent linear” (the secant stiffness instead of the stiffness before yield). The displacement demand is used to size the abutment seat width to ensure that the deck does not fall off of its supports. It also determines the ductility demand on the columns.

After the columns are designed, the foundations are

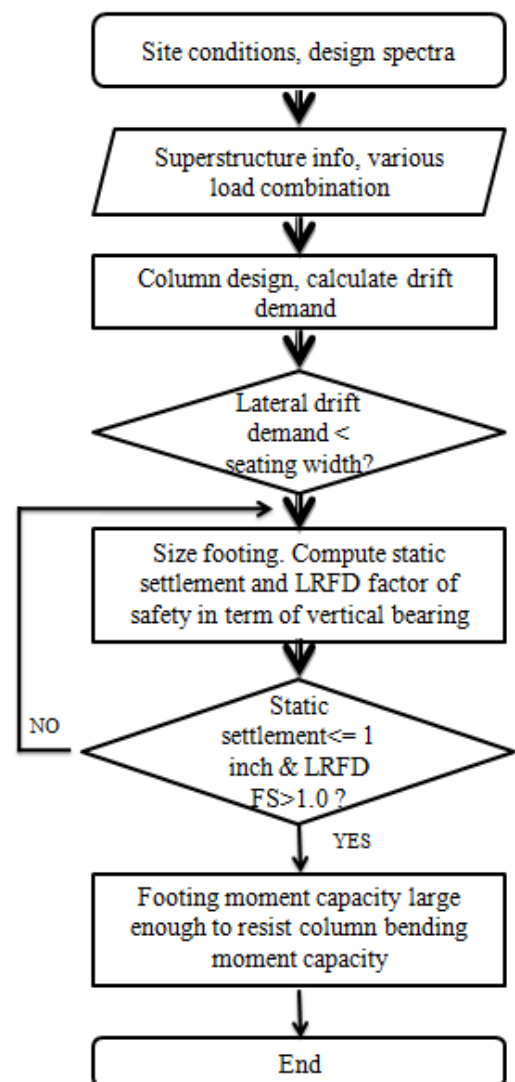


Figure 3. Simplified Conventional bridge design.

checked for settlement under dead and live loads. The maximum combined vertical settlements are required to be less than 1 or 2 inches, depending on the type and size of the bridge. Settlements are typically calculated by conventional foundation design methods. Rocking moment capacity of the foundation is checked. Typically, the foundations are design to be about 20% stronger than the columns to ensure that they remain essentially elastic when the column yields.

Introduction to Procedure for Design of Bridges with Rocking Foundations

The goal of this procedure is to design a system of rocking foundations to protect the columns from damage. To minimize ductility demand at the base of the columns, it is required that the moment capacity of the rocking mechanism is less than the moment capacity of the column. To eliminate ductility demand at the top of the column, a pin connection should be introduced between the bent cap beam and the top of the column as indicated in Fig. 4(a), ductility demands due to bending at the top of the column are also eliminated. In addition, it may also be possible to devise a rocking foundation for a single column bent (Fig. 4(b)). However, due to the need for a moment connection at the top of the column, a rocking foundation for a single column bent would result in rotation of the bridge deck, and torsional loads on the deck which complicate the design. Hence this possibility is not addressed in this paper, and a fixed connection is desirable for a single-column-bent bridge with either rocking or fixed footings (Fig. 4(b) and 4(c)).

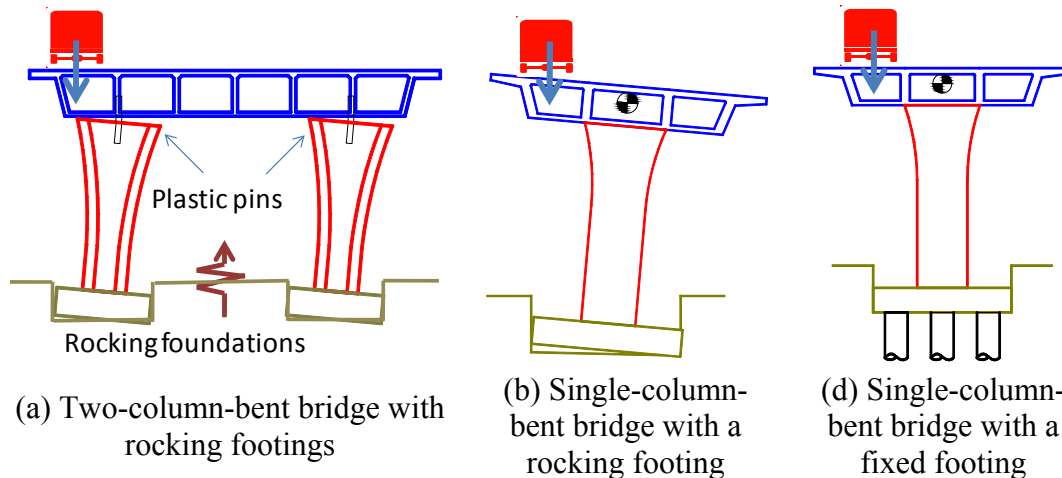


Figure 4. Illustrative concepts of the rocking footings and top plastic pins.

A free body diagram of the rocking system considered is sketched in Fig. 5. This figure also defines some notation such as the distance from the base of the footing to the base of the deck as H_c and the footing dimension as L_f . Fig 6 shows a flow chart for the proposed design procedure, which will be further described in the remainder of this Section.

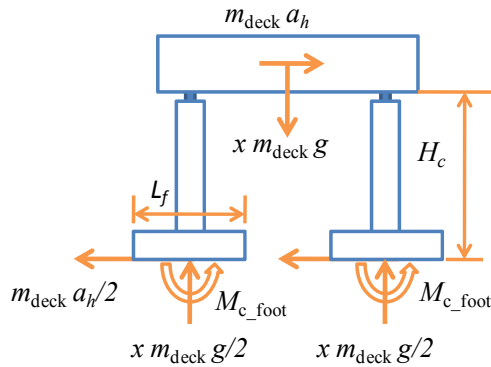


Figure 5. Free body diagram of the rocking-footing bridge.

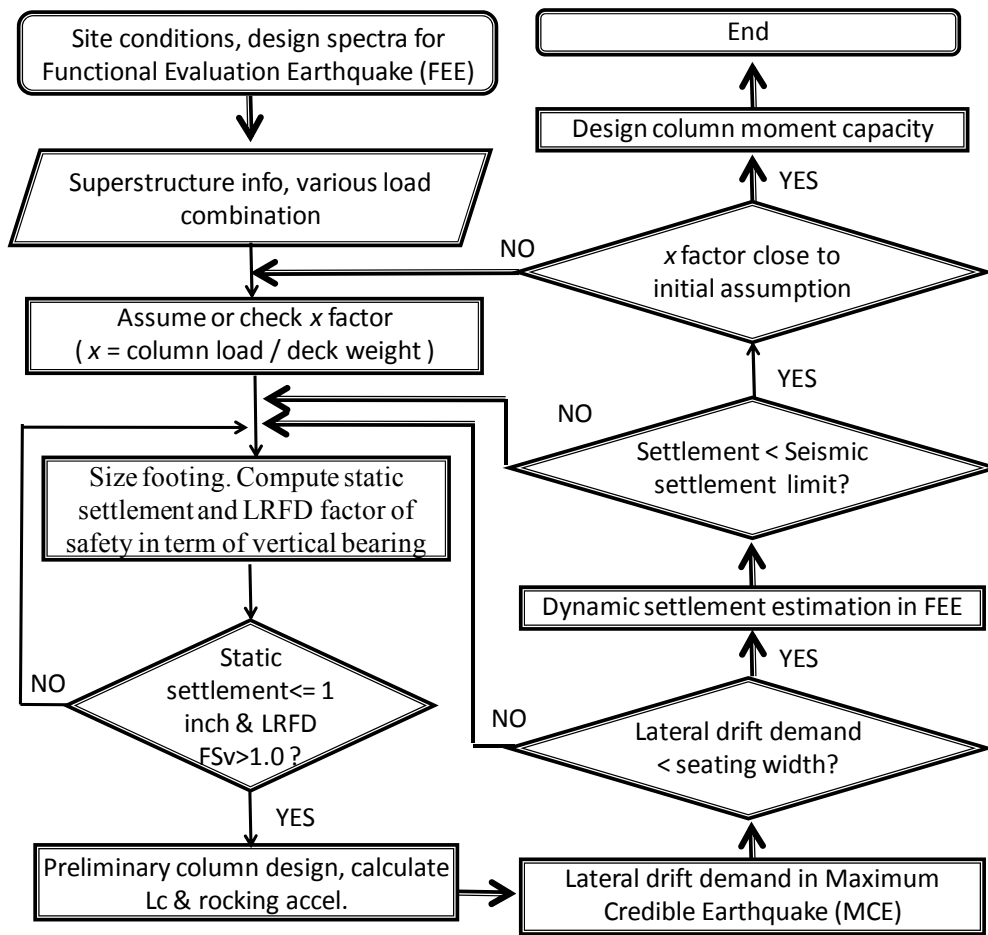


Figure 6. Flowchart for proposed design procedure allowing rocking footings.

Characterization of Site, Bridge Geometry, and Seismic Hazard

The superstructure geometry (length, skew, etc.) and loading requirements are the same as for the traditional procedure. Likewise, the quantification of site conditions, earthquake hazard and design spectra are very similar to those for conventional design procedures. However, as this

is a relatively new design concept, it is suggested that bridges with rocking foundations be designed to avoid collapse (preserve life safety) during an MCE, and in addition, to preserve serviceability during an intermediate level earthquake. In particular, the rocking of a foundation may lead to settlement of the foundation. The possibility that rocking-induced settlement could interrupt serviceability of the bridge in a Functional Evaluation Earthquake (FEE, Housner 1994), smaller than the MCE should be considered. Two hypothetical design spectra for FEE and MCE are depicted in Fig. 7.

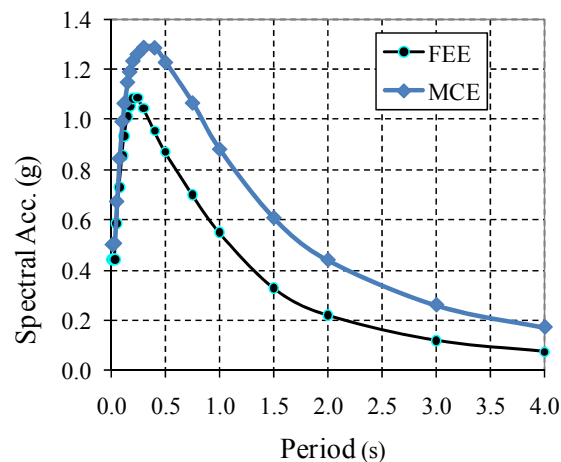


Figure 7. Design acceleration response spectra chosen for example calculations.

Load Distribution Factor (x-factor)

The third box in the flow chart shown in Fig. 6 requires determination of the “x-factor”. The x-factor is defined as the fraction of the total bridge deck weight that is supported by a bent. For a two-span two-column single-bent bridge system, for example, it may be assumed that half the vertical load is supported by the abutments and the other half is supported by the mid-span bent. Hence $x = 0.5$ should be a reasonable estimation for this typical bridge system.

If the abutments settle significantly, the vertical load on the footings and hence the rocking moment capacity of the footings would increase. If the rocking moment capacity increases above the moment capacity of the column, then ductility demand will be shifted from the rocking mechanism to a hinge at the base of the column. Settlement of the footing associated (perhaps as a result of rocking) would have the opposite effect, decreasing the axial load on the footings and decreasing the rocking moment capacity. The potential change in axial load on the footings depends on the relative settlement between the footings and the abutments as well as the bending stiffness of the bridge deck. The effect of this relative movement on the rocking moment capacity and the associated bending stresses in the bridge deck needs to be considered in the design, so the x-factor will need to be checked after the seismic settlements are assessed.

It is important to note that the proposed procedure also assumes that all of the horizontal seismic loads are taken by the columns; i.e., the bridge is assumed to be supported on frictionless bearings at the abutments. The x-factor is important because it determines the ratio of the horizontal tributary mass and the vertical tributary mass loads applied to the column and rocking foundation. The abutments resist some of the weight of the deck, but if the deck is supported on bearings at the abutments, the abutments may not support any of the lateral loads. The moment

capacity of a rocking foundation is nearly proportional to the vertical load on the foundation. Hence, if the x-factor is larger than expected, then the moment capacity of the rocking foundations will be larger than expected; this could lead to overstrength of the foundation moment capacity and could potentially lead to reduction in rotation demand on the rocking mechanism and increasing ductility demand on the column.

Determination of Footing Dimensions L_f and L_c

The footings dimensions are first designed based on settlement considerations and bearing capacity using the loads determined above. Notice that load and resistance factors may apply to the bearing capacity calculation. H_c is the column height (from the pin connection to the footing), L_f is the footing length, and L_c is called the “critical contact length” as defined in a prior section.

$$L_c = \frac{V}{q_n \cdot B_f} \quad (1)$$

where q_n is the unfactored conventional bearing capacity of a spread footing with the contact area of L_c by B_f . Iterative calculation is required to determine L_c because a change of L_c affects the shape factor L_c / B_f , which in turn affects q_n which has a secondary effect on L_c .

Preliminary Column Sizing

The column is designed to resist the full moment capacity of the footing with an overstrength factor $(LF)_{cf}$ applied to the footing capacity and an understrength factor $(RF)_{cf}$ applied to the column moment capacity as indicated in Eq. 2. Appropriate values for the load factor and resistance factor in this equation should depend on the anticipated uncertainty in the values. At present the RF and LF for bending capacities have not yet been determined, but the concept should be clarified.

$$(RF)_{cf} M_{c_column} \geq (LF)_{cf} M_{c_foot} = (LF)_{cf} \frac{V \cdot L_f}{2} \cdot \left(1 - \frac{L_c}{L}\right) \quad (2)$$

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. If we look back to the free body diagram in Fig. 5, the rocking acceleration could be obtained from Eq. (3).

$$\frac{a_h}{g} = \frac{x \cdot L_f}{2 \cdot H_c} \left(1 - \frac{L_c}{L_f}\right) \quad (3)$$

Typically values of a_h may range between 0.1 and 0.3 depending on the x factor, column height and footing dimensions. Values near this upper limit may be appropriate to limit drift in very large earthquakes, while values near the lower limit may be satisfactory for lower intensity events. The initial rocking stiffness could be obtained from Eq. (4) (FEMA 2000)

$$K_{rock} := \frac{G_0 \cdot B_f^3}{1 - \nu} \cdot \left(0.4 \cdot \frac{L_f}{B_f} + 0.1 \right) \cdot \left[1 + 2.5 \cdot \frac{t_f}{B_f} \cdot \left[1 + 2 \cdot \frac{t_f}{B_f} \cdot \left(\frac{t_f}{D} \right)^{-0.2} \cdot \sqrt{\frac{B_f}{L_f}} \right] \right] \quad (4)$$

However, this equation does not account for the reduction in stiffness that occurs after rocking is initiated.

Collapse Prevention in the MCE

The authors believe that the most probable mechanism of collapse of an ordinary 2-span bridge is considered to be unseating of the deck. Toppling due to P-Δ effects should also be evaluated. To assure that collapse will not occur, the lateral displacement demand during the Maximum Considered Earthquake (MCE) must be evaluated. Comparison of the displacement demand to the seat width, and comparison of the P-Δ moment to the degraded moment capacity of the system should enable evaluation of the ability of the system to prevent collapse

Serviceability in the 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. Caltrans, however, 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 needs to be finalized.

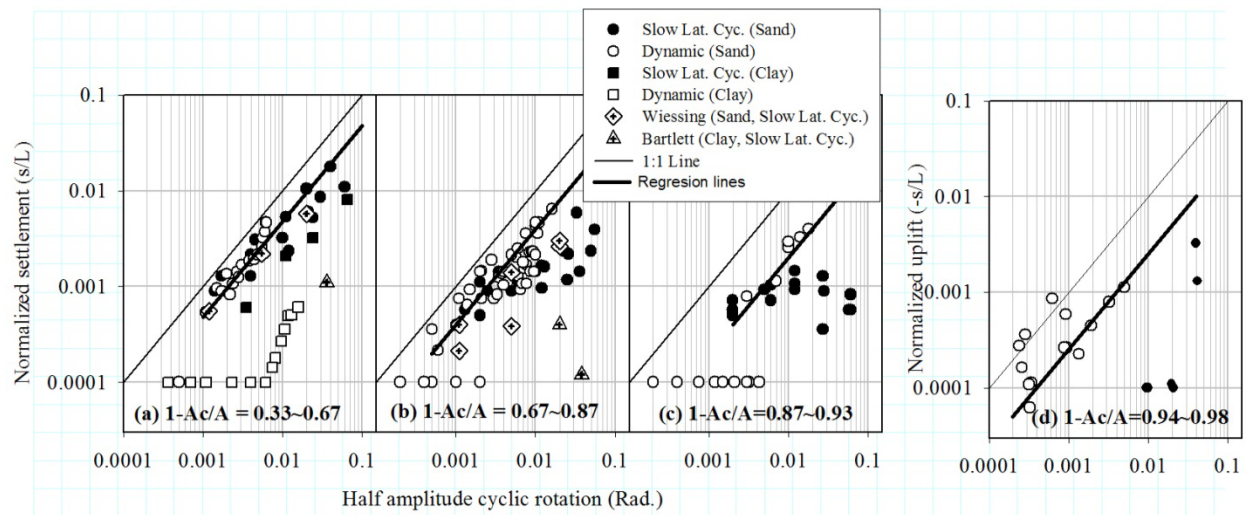


Figure 8. Normalized settlement vs. half amplitude cyclic rotation of a rocking footing (Gajan & Kutter, 2008).

Gajan and Kutter (2008) summarized settlements due to rocking observed in many experiments in the centrifuge and laboratory model tests as shown in Fig. 8. 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. In fact, it was often observed that the footings actually tended to suffer permanent uplift instead of settlement for $1-L_c/L_f > 0.94$. Methods for determination of L_c and L_f were discussed earlier.

It is suggested that the magnitude of settlement may be estimated by determining the magnitude of the cyclic rotation demand and the number of cycles of shaking. The cyclic rotation demand may be estimated as the displacement demand divided by the column height.

Displacement Demand

In order to evaluate the settlement during the FEE and the potential for collapse in the MCE, it is necessary to estimate the displacement demand in the corresponding earthquakes. Three methods are being considered for evaluation of the displacement demand. These methods are described in more detail by Deng et al. (2010)

Spectral Method

This is perhaps the simplest method for evaluation of the displacement demand. The stiffness of the system and fundamental period of the system should be determined. It needs to be established whether it is more appropriate to use the elastic stiffness or the equivalent linear secant stiffness for the natural period of the system. Once this period is determined, one may simply look up the displacement demand on a graph of the spectral displacement as a function of period for the design spectra (MCE or FEE).

Nonlinear SDOF Analysis

If the elastic natural period is known and the rocking acceleration is determined using Equation (3), then a simple nonlinear analysis can be done for an equivalent single degree of freedom system with the same natural period and yield acceleration equal to the rocking acceleration. This would be a time-history analysis with the design ground motion used for the input motion.

Finite Element Analysis

More advanced numerical methods are available. For example, Gajan et al. (2010) describe more accurate methods for modeling the moment-rotation-settlement-sliding behavior of rocking foundations using models that are available in OpenSees. In these more sophisticated analyses, the bridge, nonlinear column, and abutment restraints can be included in the simulations.

Conclusions

This paper outlines proposed design procedures for bridges with rocking foundations. These procedures need to be formalized before rocking foundations can be adopted in routine practice as the ductile mechanism for accommodating seismic demand.

In order to use rocking foundations as a primary energy dissipation mechanism, we suggest that the soil-foundation-column-deck-abutment system needs to be designed as a system. The rocking mechanism, for which a hinge forms in the soil at the toe of the footing, has some advantages over the bending mechanism where a plastic hinge forms in the reinforced concrete column. For one, the rocking foundations have a re-centering tendency. (The gap under the footing that forms during rocking closes back up after shaking.) For another, they are smaller and more economical. One concern about rocking foundations is that the methods for assessment

of permanent deformations have not yet been formalized. Methods for evaluating the settlement caused by rocking are suggested in this paper. The allowable differential settlement due seismic loading with regard to serviceability also needs to be decided. One factor that complicates the analysis of bridge systems with rocking foundations is that differential settlement also affects the axial load on the footing, and the moment capacity of the footing is sensitive to the axial load. Work is ongoing to address these and other issues.

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