

Predicting the Seismic Response of Structures Employing Controlled Rocking As A Form of Base Isolation

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Abstract—A wide variety of rocking systems have been proposed to reduce damage induced by earthquakes. These systems rely on rocking as a form of base isolation, but also employ post-tensioning to limit rocking amplitude and ensure re-centering of the system. Often performance based design approaches (i.e. static pushover) are used to predict the response of these systems.

This paper employs a simple analytical model to investigate the response of post-tensioned flexible rocking structures using two different approaches. First, analytical dynamics is used to predict the full dynamic response. Second, a performance based design approach is used to predict the maximum response. The simple model allows a wide parametric study to determine the effects of relative tendon stiffness, relative structural stiffness, and rocking parameters, on the maximum response.

Comparison of the results defines limits for the relative tendon stiffness and the rocking amplitudes for which performance based design approaches are reliable. Beyond these limits, the full dynamic response must be considered. In addition, the results give insight regarding the parameters for which the benefits of rocking behavior can best be exploited.

Keywords: Rocking; Earthquake Engineering; Seismic Response; Base Isolation; Performance Based Design

I. INTRODUCTION

The purpose of the current generation of seismic design provisions is to protect life safety by preventing building collapse. These traditional methodologies were not developed to prevent damage to buildings and infrastructure or mitigate economic losses.

Developing seismic provisions and codes enabling the design of structures to achieve defined seismic performance levels is a pressing requirement in mitigating future losses from earthquake hazards. In order to achieve performance levels higher than “life-safety”, reliable retrofit and design techniques are required, enabling the isolation of structures from ground motion, reducing earthquake generated forces and, consequently, minimize response, material strains and damage [1].

Rocking systems as a form of base isolation for bridge piers and building frames has been the subject of several recent studies. The first systematic investigation of the dynamics of rocking was conducted by Housner [2], who examined the dynamic behaviour of a rigid rectangular slender block, resting on a level surface and rocking about its bottom corners under horizontal ground motion. Housner's investigation showed that tall, slender blocks are unexpectedly stable under earthquake excitation, more so than would be intuitively inferred by their stability against steady lateral loading. Subsequent studies have shown structures designed to uplift and rock on their foundations during earthquakes experience significantly reduced deformations, and consequently sustain less damage than fixed-base counterparts. Subsequent investigations [3-5] have shown a desirable base isolation effect resulting from foundation uplift in structures subjected to ground motion.

Oliveto et al. [6] built upon these findings to derive the nonlinear equations of motion for large displacements, enabling an investigation of the overturning stability of a single-degree-of-freedom flexible structure with foundation uplift, noting that a thorough understanding of the dynamic behaviour of rocking structures was still required. Acikgoz & DeJong [7, 8] further investigated the interaction of structural flexibility with base rocking for flexible rocking structures under pulse-type excitations, highlighting the effect of elasticity as dominant pulses in near field earthquake records often are responsible for the overturning collapse of rocking structures. These studies have made it feasible to develop a design methodology capable of accurately capturing the maximum response and hence enabling the design of configurations to meet specific performance requirements.

This cumulative work has shown that rocking can potentially be implemented as a form of base isolation to eliminate inelastic dynamic structural response. Further research has shown that controlled rocking enhances these effects, by limiting rocking amplitude and ensuring self-centering, virtually eliminating residual drifts and ensuring the systems remains essentially elastic [9].

An ongoing reappraisal of seismic design principals by the engineering community has resulted in a gradual realignment from force-based to performance based seismic design. Performance objectives are increasingly employed to define the level of acceptable risk [10].

Recent research has proposed applying current performance based design approaches to controlled rocking systems. Pollino & Bruneau, [11] analytically investigated a seismic retrofit technique for steel truss bridge piers, featuring a rocking-dissipating pier-foundation connection. Pollino & Bruneau's proposed system features a passive energy dissipation device (buckling restrained brace) which enhances self-centering. Performance based methods were used to predict maximum deck-level response (displacement), implementing the nonlinear-static coefficient-capacity spectrum procedure [13].

However, fundamental differences in behaviour between rigid rocking systems and linear elastic oscillators have led to questioning of the performance based design approaches proposed to predict the response of such systems, almost all of which employ linear elastic displacement response spectra for demand estimation [14]. Given the limitations of static analysis procedures (e.g. pushover analysis), the reliability of the maximum response predictions of controlled rocking structures using current methods has not been investigated sufficiently. In order to fully realize the potential of controlled rocking systems, further research is required to:

- Evaluate existing Performance Based Design (PBD) methods as a viable seismic design approach.
- Defining the limitations of Performance Based Design (PBD) using static analysis techniques, and identifying design constraints necessary to achieve optimum benefits of rocking behaviour.
- Extend recent advances in the fundamental understanding of the interaction of flexibility and rocking in free-standing flexible rocking structures to controlled (post-tensioned) flexible rocking structures.

II. MODELLING AND ANALYSIS METHODS

A. Structural Model

An idealized model of the system under consideration; a post-tensioned flexible rocking structure is shown in Figure 1. The model has a lumped mass m , positioned at height H at quiescent initial conditions and attached to a weightless, rigid foundation beam of width $2B$, by an axially rigid strut of lateral stiffness k_o and viscous damping c . The diagonal distance from the undisturbed mass to points O, O' located at the edges of the base foundation, is denoted as $R_o = \sqrt{B^2 + H^2}$ and $\alpha = \tan^{-1}(B/H)$ is the angle of slenderness. A central, unbonded post-tensioned elastic tendon of stiffness k_t passes through a duct running through the centre of the structure. The two degree of freedom system is free to uplift and may respond to horizontal excitations by rocking about pivot points O, O' in addition to translational motion of the mass, u .

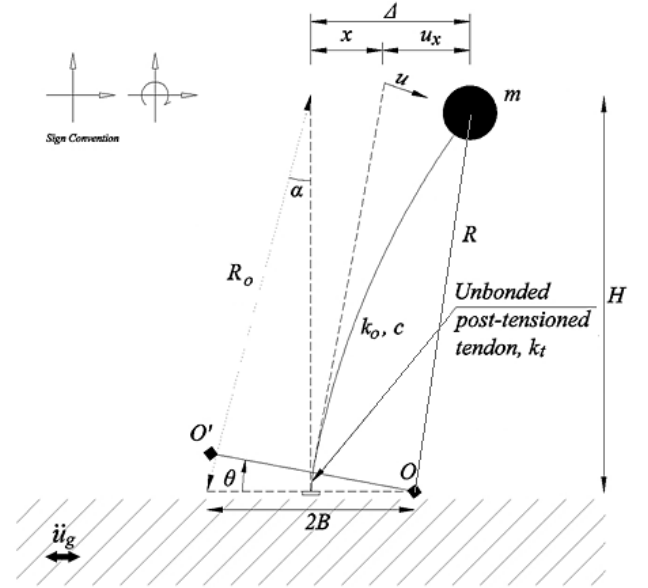


Figure 1 Schematic of idealized structural model of a post-tensioned, flexible rocking structure

Horizontal displacement $\Delta = x + u_x$, where x is the horizontal component of rocking displacement, and u_x is the horizontal component of lateral deformation u .

$$u_x = u \cos(\theta) \quad (1)$$

$$x = R \sin(\alpha) - R \sin(\alpha - \theta) \quad (2)$$

The configuration shown in Figure 1, representing the system considered in this investigation, also represents common design and retrofit solutions used in practise. For the purposes of this study, analyses were conducted assuming the system rests on rigid ground, and that the coefficient of static friction is sufficient to prevent sliding. In practise, sliding may also be prevented by using an appropriate foundation configuration.

B. Phases and Associated Equations of Dynamic Motion

Two phases define the dynamic motion of a flexible rocking structure; a phase in full contact with the ground, and an uplifted, rocking phase. The equations of motion governing these two states for a free-standing (no additional self-centering), flexible rocking structure are presented in [8].

In the current model, the equations of motion for the rocking phase account for the added post-tensioned tendon. The full contact phase is unaffected by this addition, as the tendon is not assumed to be prestressed, i.e. it is slack prior to uplift, and only exerts a resorting force in the event of rocking. The equations of motion for the rocking phase adopted from [8], expanding the model to include a post-tensioned tendon.

Full contact phase: This phase represents the response of a structure with quiescent initial conditions, such that both corners of the base are in contact with the ground. The structure's motion is identical to that of a linear elastic

oscillator and is governed by the standard differential equation for a SDOF system

$$\ddot{u} + 2\zeta\omega_n\dot{u} + \omega_n^2u = -\ddot{u}_g \quad (3)$$

Where u is the elastic translation of the mass, $\omega_n = \sqrt{k/m}$ is the angular natural frequency of the system, $\zeta = c/(2\sqrt{km})$ is the damping factor and \ddot{u}_g is the horizontal ground acceleration. The response of the structure in this phase is not affected by geometry, provided that stiffness k is an independent parameter.

Rocking phase: Once the structure uplifts, rocking and elastic deformation proceed simultaneously. This occurs once lateral displacement of the mass m reaches a critical value u_{cr} at which the overturning moments are sufficient to cause uplift. The equations of motion governing the rocking phase, which describe the elastic translation and rocking for a free rocking (unanchored) structure were derived in [7] using generalized coordinates (R, β) and extended in [8] using (u, θ) , based on a model formulated by Oliveto et al. [6]. The (u, θ) coordinates are used in this study exclusively, and equations have been modified for a SDOF system. The equations of motion for the rocking phase in terms of these parameters are (variables given in Figure 1).

$$\ddot{u} + H\ddot{\theta} + (\pm B - u)\dot{\theta}^2 + 2\zeta\omega_n\dot{u} + \omega_n^2u = -\ddot{u}_g \cos(\theta) + g \sin(\theta) \quad (4a)$$

$$(R_o^2 + u^2 \mp 2Bu)\ddot{\theta} + H\ddot{u} \mp 2\dot{\theta}\dot{u}(B \mp u) + (k_t B^2 \sin(\theta)/m) = +\ddot{u}_g(-R_o \cos(\alpha \mp \theta) + u \sin(\theta)) + g(\mp R_o \sin(\alpha \mp \theta) + u \cos(\theta)) \quad (4b)$$

Equation (4 a) describes the rocking motion of a flexible rocking structure, retrofitted with an unbonded central tendon. The first line of this equation represents the equilibrium of forces in the direction of elastic deformation u , while the second line represents the equilibrium of overturning and stabilizing moments about the rocking pivot O (or O'). The term $(k_t B^2 \sin(\theta)/m)$ accounts for the elastic action of the tendon. In Equation (4 a) the upper sign indicates rotation about the right base corner O , and the lower sign about the left foundation corner O' . Details of phase transition can be found in [7].

C. Performance Based Design Approach

This section presents the analysis procedure and design requirements for performance based seismic design of controlled rocking structures. The adopted analysis procedure is the Nonlinear Static Procedure (NSP), often referred to as the Pushover Analysis [13, 15]. Under this procedure, a model directly incorporating nonlinear response is displaced to a target displacement, and the resulting forces are measured.

This procedure has been adopted from ATC-40 [15] and modified to account for bilinear elastic behaviour characteristic of controlled rocking structures. The original document refers to inelastic material response as the source of non-linearity. However, this was modified so that the limit state is at which the system's stiffness deviates is the uplift of the structure, as opposed gradual yielding of structural members.

A mathematical model of the structure is developed, in accordance with guidelines specified in ATC-40 [15]. This model is subjected to monotonically increasing horizontal forces until a target displacement is achieved. Seismic demand can be directly expressed as elastic spectral demand. It is then possible to compare seismic demand with the structure's lateral force-displacement (pushover) capacity. This is the basis of Capacity Spectrum Method; a performance based seismic design method described in the ATC-40 document [15].

D. Dimensionless system

To present the results in an intuitive manner, for a wide range of structures, input parameters are nondimensionalized. Furthermore, in order to characterize the behaviour of the system, the absolute ratio of the rotational stiffness provided by the tendon to the rocking rotational stiffness is adopted.

Additional moment provided by the unbonded post-tensioned tendon has been shown to be

$$M_{add, tendon} = k_t B^2 \sin \theta \quad (5)$$

where k_t = tendon stiffness. Normalizing by moment of inertia of the mass and the frequency parameter

$$I = mR^2, \quad p = \sqrt{g/R} \quad (6)$$

$$\frac{M_{ad}}{p^2 I} = \frac{k_t B^2 \sin \theta}{mgR} \quad (7)$$

Assuming small angle rotation, applying the elastic moment rotation relationship

$$M = k \theta \quad (8)$$

$$\frac{M_{ad}}{p^2 I} = \frac{k_t B^2 \sin \theta}{mgR} \approx \frac{k_t B^2 \theta}{mgR} = \rho_t \theta \quad (9)$$

$$\rho_t = \frac{k_t B^2}{mgR}, \quad k_t = \frac{\rho_t mgR}{B^2} \quad (10)$$

A characterization of systems retrofitted with three ranges of rotational stiffness can be achieved using the dimensionless quantity ρ_t . Varying the values of ρ_t its value will allow for capturing these three ranges of rocking behaviour.

$$\left| \frac{M_{ad}}{M_{gr}} \right| = \left| \frac{K_{R, ad}}{K_{R, gr}} \right| = \rho_t = \begin{cases} \rightarrow 0 & \text{rocking system (negative slope)} \\ = 1 & \text{balanced system (zero slope)} \\ \gg 1 & \text{hybrid system (positive slope)} \end{cases} \quad (11)$$

Use of this parameter facilitates the determination of the effects of relative tendon stiffness on maximum response.

A flexibility/scale nondimensional parameter ρ_o will be used to determine and capture of the effects of relative structural (bending) stiffness.

$$\rho_o = \frac{k_o B^2}{mgR} = \frac{\omega_n^2 B^2}{p^2 R^2} \quad (12)$$

$$\frac{\omega_n}{p} = \sqrt{\rho_o} \frac{B}{R} \quad (13)$$

The behaviour of the system will depend on the dimensionless terms θ/α , u/u_{cr} , ρ_o , ρ_t but will also be

determined by the characteristics of the specific ground motions records to be implemented. In that sense, the response is not completely nondimensionalized. However response can be presented intuitively in terms of θ/α , u/u_{cr} and compared across the range of ground motions implemented, for a wide range of relative structural stiffness and relative tendon stiffness ρ_o , ρ_t representing a variety of behaviours which can be captured using these parameters.

III. ANALYSIS AND RESULTS

The viability of the proposed model is assessed by means of a case study. A bridge pier configuration with geometrical and material properties consistent with an existing structure is introduced in this section. A detailed example illustrates how results are obtained and compared in a parametric study.

The performance -based procedure applied in the current research, the capacity spectrum method [15], will be explained using an example. The case study is a precast concrete bridge pier configuration with geometrical and material properties consistent with an existent bridge, the South Rangitikei Rail Bridge in New Zealand, which by design employs rocking as a seismic isolation technique [16]. This configuration is modified for the purposes of this study by the addition of a central unbonded post-tensioned tendon as a self-centering device. The height of the pier $H = 76.9$ m, half base width $B = 6.73$ m, and mass $m = 1.67 \times 10^6$ kg lumped at deck level, assumed to be at height H . The pier's aspect ratio is 11.5, corresponding to a slenderness angle $\alpha = 0.09$ rad. The pier is allowed to uplift and rock on its foundation under the influence of horizontal ground motion. Sliding is assumed to be prevented by means of an appropriate foundation configuration.

A. Design Example

The bridge is assumed to be located at a class B, representing a rock site and coefficients F_a and $F_v = 1$. A site-specific design response spectrum calculated according to procedures described in the Guidelines for the Seismic Design of Highway Bridges [17], based on 5% in 50 year probabilistic data from the US geological survey [18] is used to obtain seismic demand. One second (S_1) and short period (S_s) acceleration values are 0.734g and 1.93g, respectively, corresponding to 5% damping.

Values of relative stiffness parameters ρ_o , ρ_t correspond to stiffness categories, representing proportional multiples of absolute lateral and relative stiffness respectively, according to (Eq. 10, 12). Tendon stiffness k_t is considered in multiples

28000 kN/m, which corresponds to a relative tendon stiffness of $\rho_t = 1$, and represents the tendon stiffness required to balance the pier's negative 'rocking stiffness' for small rotations, resulting in a horizontal post-uplift stiffness curve.

Thus, for ($\rho_t = 1$) $\rightarrow k_t = 28 \times 10^3$ kN/m (Eq. 10). The pier's lateral stiffness k_o is similarly considered in multiples of 28×10^3 kN/m, which corresponds to a relative initial stiffness of ($\rho_o = 1$), as defined in Eq. 12.

Horizontal deck-level displacement for selected values of ρ_o , ρ_t is shown in Figure 2. The values of ρ_o , ρ_t this examples uses are used to obtain values k_o , k_t . (Eq. 10, 12) These stiffness values determine the shape of the bilinear elastic force displacement curve to be employed in the pushover analysis.

Taking $\rho_o = 5$, $\rho_t = 100$ as an example case:

$$k_o = \frac{\rho_o m g R}{B^2} \rightarrow \rho_o = 5 \rightarrow k_o = 141 \times 10^3 \text{ kN/m}$$

$$k_t = \frac{\rho_t m g R}{B^2} \rightarrow \rho_t = 100 \rightarrow k_t = 28 \times 10^5 \text{ kN/m}$$

B. Nonlinear inelastic time history analysis

In order to verify the effectiveness of the performance-based approach employed in, a non-linear dynamic time-history analysis is performed to predict the full 'exact' dynamic response. For the analytical dynamics simulation, the absolute stiffness values are determined from relative stiffness values ρ_o , ρ_t (Eq. 10, 12). These absolute stiffness values (in units of kN/m), pier mass m and dimensions H , B constitute input parameters for the numerical solution of the equations of motion (Eq. 3, 4). The result of this simulation can be expressed in terms of the nondimensional output parameters θ/α , u/u_{cr} or directly as the total deck-level lateral displacement Δ which is evaluated according to Eq.1.

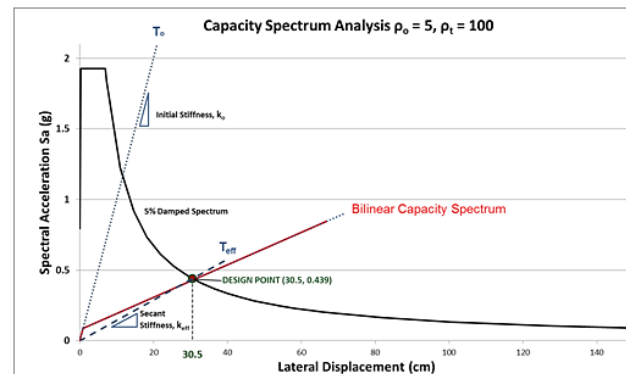


Figure 2 Example capacity spectrum analysis [15]

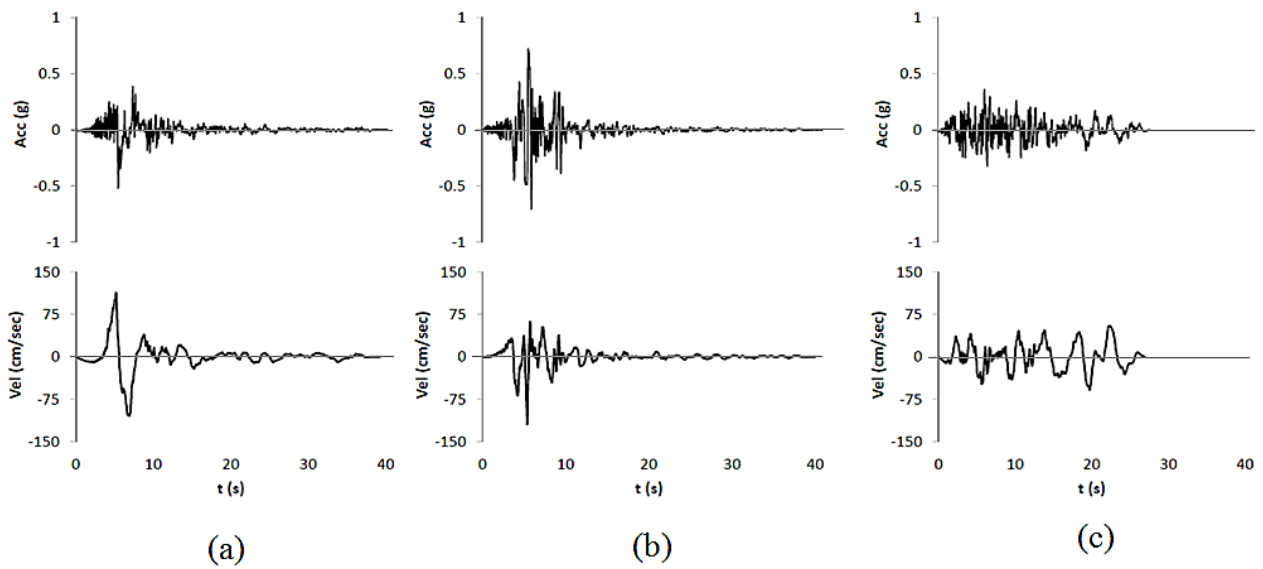


Figure 3 Pulse type ground motion records (a) 1979 Imperial Valley (El Centro Array #4) [IV79] and (b) 1994 Northridge (Newhall, Fire Station) [NR94] and non-pulse-type record (c) 1980 Victoria Mexico (Chihuahua) [VM80]. Acceleration time histories of the selected records are shown in the top row, and velocity time histories are shown on the bottom row [20]

Previous research into performance based design of controlled rocking systems—e.g. [11]—used synthetic earthquake records lacking “forward directivity effects in the near fault region” characteristics [19]. Specifically strong velocity pulses caused by fault normal, near fault pulse-type ground motion which are the primary drivers of large amplitude rocking [2, 8]. In the current research, viability of the proposed model is assessed by subjecting the bridge pier configuration to both pulse type and non-pulse type earthquake records. Time-History plots of the ground motion records used in this study are shown in Figure 3. The records were obtained from the PEER ground motion database [20].

A Comparison of results of maximum deck-level

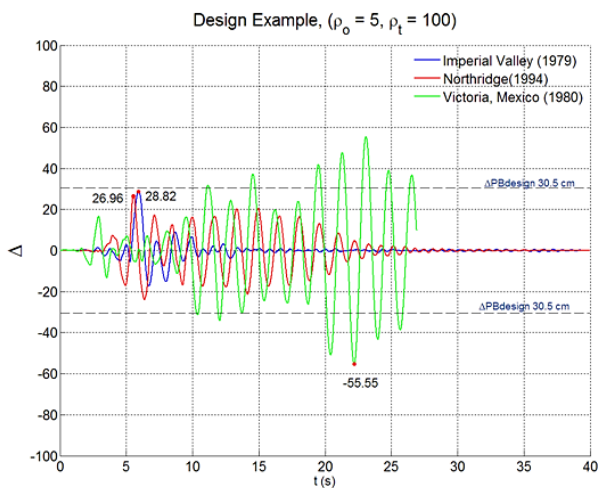


Figure 4 Lateral pier displacement Δ results of example time history analysis

displacements prediction by the performance based method and the full dynamic time-history analysis are presented as

maximum response predicted by the time-history analysis Δ_{TH} shown in Figure 4 normalized by the maximum response predicted by the performance based design procedure $\Delta_{PBdesign}$ shown in Figure 2.

C. Parametric Study

A parametric study using analytical dynamics (nonlinear time-history analysis) is conducted to determine the accuracy of maximum response values predicted by the displacement based procedure used in this study, the Capacity Spectrum Method [15]. As illustrated in the case study presented in the previous section, the primary parameters considered are relative tendon stiffness and relative structural stiffness ρ_o, ρ_t . Rocking parameters are also considered here with further recommendations for future work. A wide range of relative structural relative tendon stiffness are considered to determine the effects of these parameters on maximum response, and the limits for which performance based design approaches are reliable. The preliminary results of this parametric are shown in Figure 5.

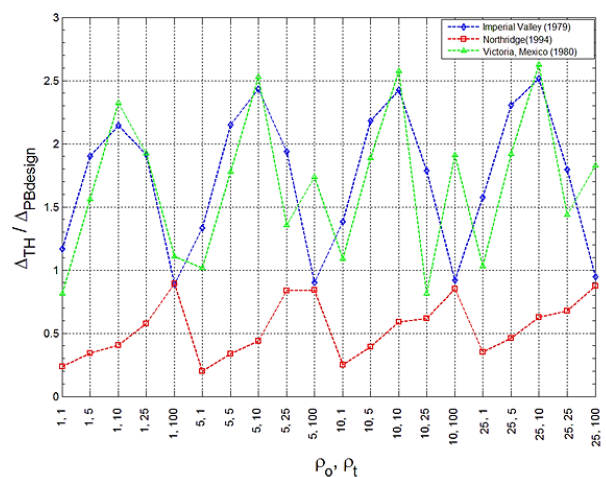


Figure 5 Non-linear time-history simulation results for deck-level displacements Δ , normalized by their respective displacement based design response value.

IV. DISCUSSION

As shown in Figure 5, the displacement based method conservatively predicts the maximum displacement for the 1994 Northridge ground motion for all twenty cases considered. Within the each relative bending stiffness category, ρ_o , the performance based approach is seen to more accurately predict maximum response for higher values of ρ_t , representing a stiffer tendon. Conversely, for the 1979 Imperial Valley ground motion, the performance based approach under-predicts response for all but four cases. These four cases, which are slightly over-predicted, pertain to the stiffest tendon, $\rho_t = 100$ for all bending stiffness categories, $\rho_o = 1, 5, 10, 100$. Prediction of maximum displacement is unconservative for all but two cases for the 1980 Victoria, Mexico ground motion.

A monotonic, upward trend in displacement based method predictions is noticeable for the 1994 Northridge ground motion, such that predictions for lower relative tendon stiffness ρ_o values is very conservative, but accuracy of prediction gradually increases with increasing tendon stiffness until a very good prediction is realized at $\rho_t = 100$, a case which represents the highest relative tendon stiffness. This pattern is similarly observed for higher bending stiffness categories $\rho_o = 5, 10$ and 25.

For the 1979 Imperial Valley ground motion, the displacement based method also yields predications of noticeable variation within structural stiffness categories. The pattern of variation is then approximately repeated across bending stiffness categories, i.e. for $\rho_o = 1, 5, 10$ and 25. However, in this case, change in prediction accuracy is not monotonous within structural stiffness categories, as is the case for the 1994 Northridge ground motion. Rather, predictions are more accurate for very stiff and very flexible tendons $\rho_t = 100, 1$ respectively. Predictions are unconservative for three intermediate relative tendon stiffness categories $\rho_t = 5, 10$ and 25, with the least conservative prediction achieved at $\rho_t = 10$.

Interestingly, the displacement based method's prediction of maximum response for the 1979 Imperial Valley ground motion is most accurate for $\rho_t = 100$, and almost identical to the prediction for another pulse-type ground motion, the 1994 Northridge earthquake record. In both cases response is slightly overpredicted. This indicates that record-to-record variability is minimal when relative rotational stiffness is at its highest value. This may imply that limiting rocking amplitude by applying a stiff tendon could limit record-to-record variability. This can be verified by comparison of results with the third ground motion record, and examining rocking amplitude time-histories.

Predictions of maximum response for the 1980 Victoria, Mexico ground motion are more chaotic, displaying less regularity distribution of normalized response than the previous two ground motion cases. Some agreement can be seen for the lowest relative rotational stiffness category $\rho_t = 1$. As for IV79, prediction discrepancy increases for stiffer tendons represented by $\rho_t = 5, 10$. At higher relative tendon stiffness there is little order to discern from the normalized displacements. Examining

response time histories amplitude time-histories is required for further interpretation.

V. CONCLUSIONS

The current study investigated the viability of applying performance based design approach to rocking structures. This was conducted by assessing the capability of an established displacement based design method to predict the response of structural systems adopting controlled rocking to limit damage due to earthquake-induced ground motion. Initial results show that for large bridge piers, long duration velocity pulses cause large rocking amplitudes the performance based method is unable to accurately predict.

The results that have been achieved so far and conclusions based on them represent initial results from a preliminary investigation. Three earthquake records were selected for investigation, and applied to a single pier configuration. The current model needs to be further developed to account for hysteric and radiation damping. A significant expansion of the number and type of earthquake ground motions, as well as the scale of structure considered, in addition to a suitable treatment of damping are required before generalize results.

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