BI-DIRECTIONAL SEISMIC ANALYSIS AND DESIGN OF BRIDGE STEEL TRUSS PIERS ALLOWING A CONTROLLED ROCKING RESPONSE

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Abstract

4-legged bridge steel truss piers provide support for gravity, transverse, and longitudinal lateral loads of bridges. Allowing a controlled rocking response for seismic resistance of 4-legged truss piers requires the development of design equations considering ground motions in two horizontal directions and vertical excitation. First, the static kinematic and hysteretic bi-directional behavior, relevant for design, is developed analytically and combination rules for design established. The seismic response of a 4-legged pier to 3 components of ground excitation is then investigated using inelastic, dynamic time history analyses. An example is presented where some key design parameters are determined and compared to the results of dynamic analyses.

Introduction

Roadway and railway bridges supported on steel truss piers have a number of 2-legged piers primarily for support of gravity loads that also resist transverse lateral loads however do not provide any significant resistance to longitudinal lateral loads. 4-legged piers provide support for gravity, transverse, and are the primary elements for resistance of longitudinal lateral loads along with the abutments.

The controlled rocking approach to seismic resistance allows uplifting of pier legs at the foundation while displacement-based steel yielding devices (buckling-restrained braces) are implemented at the uplifting location to control the rocking response. Allowing uplift effectively increases the pier's period of vibration, partially isolating the pier. The controlled rocking system has an inherent restoring force that allows for pier self-centering following a seismic event. This approach to seismic resistance has been investigated for 2-legged (2D) piers in Pollino and Bruneau (2004).

The design of 4-legged piers must consider the bi-directional response of the controlled rocking piers along with the effects of vertical excitation. The kinematic and hysteretic behavior are developed analytically such that design rules can be developed. Capacity design principles, considering a number of dynamic effects that occur during uplifting and impact of pier legs, are applied to the existing pier and bridge deck such that they remain elastic. Maximum displacements are determined using the capacity spectrum

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method of analysis. Directional and modal combination rules are used to predict maximum developed displacements and forces.

A set of pier and device properties are then used in an example to illustrate the concepts presented and to present the results of analyses. Seven sets (x, y, and z) of synthetically generated acceleration histories are used in the analyses.

Kinematic and Hysteretic Behavior of 4-legged Pier Considering Bi-directional Response

A typical 4-legged truss pier is shown in Fig. 1. along with the defined coordinate system. Also shown is a directional vector that lies in the x-y plane at an angle α from the x-axis that will be used throughout this paper.

The cyclic hysteretic curve for a 2-legged pier was developed "step-by-step" in Pollino and Bruneau (2004). The primary difference is the use of four devices (one at the base of each leg) and these values are only valid for motion along one of the pier's primary axes, termed uni-directional response here ($\alpha = n\pi/2$ rad., n=0,1,2,...). The uni-directional hysteretic response is not path dependent beyond the 2^{nd} cycle (Pollino and Bruneau 2004). However, the bi-directional pushover curve is path dependent and therefore is only defined for the path considered.

Compatibility, equilibrium, and force-deformation relationships of a 4-legged pier are established to assist in the design of controlled rocking piers. For a seismic demand in 2 horizontal directions, it is possible for the pier to uplift and yield 3 of the devices such that it is supported vertically on one of its legs. Assuming that rotation of the pier about a vertical axis does not occur (no torsion), the top of frames 1-1 and 2-2 experience the same displacement while frames 3-3 and 4-4 experience the same displacement (see Fig. 1). The displacement of the top of frame m ($\Delta_{u,m}$) is the sum of deformations of the frame's structural members ($\Delta_{o,m}$) and rigid body rotation at the base of the frame ($\Delta_{br,m}$) (see Fig. 2) such that:

$$\Delta_{u,m} = \Delta_{o,m} + \Delta_{br,m} \quad (1)$$

where the displacement due to deformation of the frame's structural members, for a frame

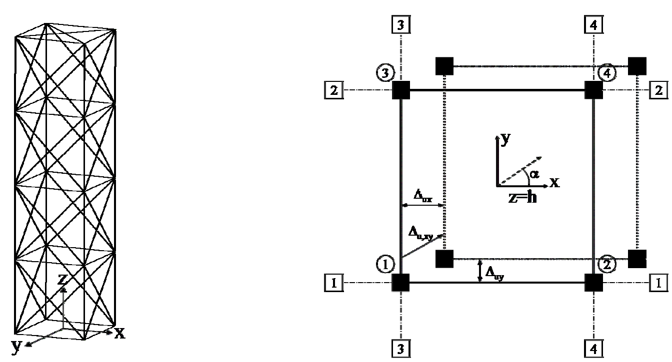


FIG. 1. TYPICAL 4-LEGGED PIER AND DEFINED COORDINATE SYSTEM

in an X-braced configuration, can be defined as:

$$\Delta_{o,m} = \frac{F_{f,m}}{k_{f,m}} = \frac{F_{f,m}}{\left(\frac{h^3}{3EI_f} + \frac{L_d^3}{2Ed^2A_d} n_p \right)^{-1}} \tag{2}$$

where $k_{f,m}$=horizontal stiffness of the top of the frame, $F_{f,m}$=horizontal shear force applied to frame m, E=modulus of elasticity of steel, I_f=moment of inertia of the frame, L_d=length of truss diagonal, A_d=cross-sectional area of truss diagonal, n_p=number of frame panels along height, h=pier height, and d=pier width. The displacement due to rigid body rotation of frame m ($\Delta_{br,Fm}$) is related to the uplifting displacement of the frame ($\Delta_{up,Fm}$), which is defined as the difference of the uplifting displacement of the two legs (i and j) of the frame such that:

$$\Delta_{br,Fm} = \Delta_{up,Fm} \cdot \frac{h}{d} = \left(\Delta_{up,Li} - \Delta_{up,Lj}\right) \cdot \frac{h}{d} \tag{3}$$

where $\Delta_{up,Li}$ and $\Delta_{up,Lj}$ are the larger and smaller uplifting displacements of the frame legs respectively, as shown in Fig. 2.

Since each pier leg acts as a member in a frame in the x- and y-direction, the uplifting displacements of the pier are dependent on the pier displacement in the x- and y-direction. For example, the uplifting displacement of pier leg 1 (see Fig. 1) can be determined by summing the uplifting displacements of frames 4 and 1 or frames 2 and 3. Using frames 4 and 1, the uplifting displacement of pier leg 1 is determined using Eqs. (1) to (3) where i=1, j=2, and m=1 such that:

$$\Delta_{up,L1} = \Delta_{up,L2} + \left(\Delta_{u,F1} - \frac{F_{F1}}{k_f} \right) \cdot \frac{d}{h} \tag{4}$$

where $\Delta_{up,L2}$ can also be determined using Eqs. (1) to (3):

$$\Delta_{up,L2} = \Delta_{up,L4} + \left(\Delta_{u,F4} - \frac{F_{F4}}{k_f} \right) \cdot \frac{d}{h} \tag{5}$$

where F_{F1} and F_{F4} is the horizontal shear applied to frames 1 and 4 respectively. If the top

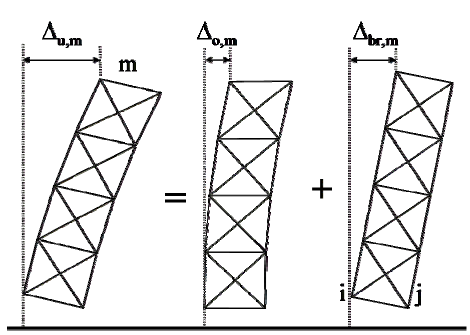


FIG. 2. KINEMATICS OF CONTROLLED ROCKING TRUSS PIER

of pier displacements are in the positive x- and y-directions and ignoring torsion then $\Delta_{u,F1}=\Delta_{u,F2}=\Delta_{u,x}$, $\Delta_{u,F3}=\Delta_{u,F4}=\Delta_{u,y}$, and $\Delta_{up,L4}=0$ and the uplifting displacement of leg 1 can be defined as:

$$\Delta_{up,L1} = \left(\Delta_{u,x} + \Delta_{u,y} - \frac{F_{F1} + F_{F4}}{k_f} \right) \cdot \frac{d}{h} \tag{6}$$

If the hysteretic path to reach $\Delta_{u,x}$ and $\Delta_{u,y}$ results in the formation of the pier's plastic mechanism, causing yield of three devices but ignoring strain hardening, the resulting free body diagram of frames 1-4 are shown in Fig. 3. Through the equilibrium of forces, the horizontal shear force to frames 1 and 3 is:

$$F_{F1,3} = \left(\frac{w_v}{8} + \frac{1}{2} A_{ub} F_{yub} \right) \cdot \frac{d}{h} \tag{7}$$

and the shear force applied to frames 2 and 4 is:

$$F_{F2,4} = \left(\frac{3w_v}{8} + \frac{3}{2} A_{ub} F_{yub} \right) \cdot \frac{d}{h} \tag{8}$$

where w_v is the vertical tributary load of the pier and $A_{ub}F_{yub}$ is the yield force of the device. Thus, considering bi-directional response ($\alpha \neq n\pi/2$ rad., n=0,1,2,...), the maximum shear force can be determined simply through equilibrium equations if the pier has deformed such that a plastic mechanism has formed. The maximum shear force in each direction is equal to the uni-directional yield force, P_y, defined in Pollino and Bruneau (2004) as:

$$P_y = \left(\frac{w_v}{2} + 2 \cdot A_{ub} F_{yub} \right) \frac{d}{h} \tag{9}$$

Assuming equal pier and buckling-restrained brace properties applied in each direction, $F_{x,max}=F_{y,max}=P_y$, then the total applied shear force and bi-directional yield force is defined as:

$$P_{y,xy} = \sqrt{F_{x,\max}^{\;2} + F_{y,\max}^{\;2}} = \sqrt{2 \cdot F_{x,\max}^{\;2}} = \sqrt{2} \cdot P_y \quad \left(\alpha \neq \frac{n\pi}{2} rad., n = 0, 1, 2, ... \right) \tag{10}$$

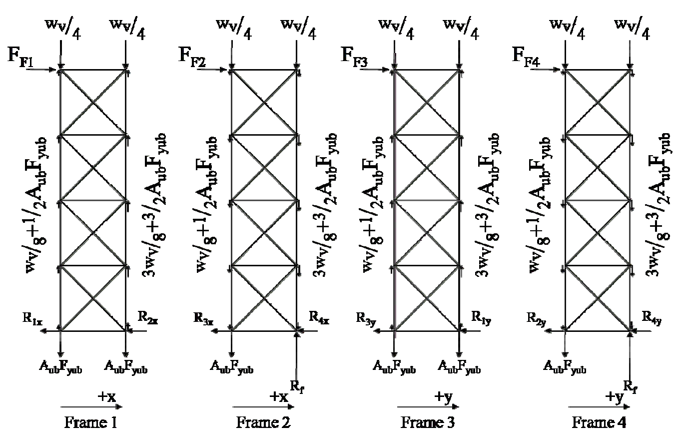


FIG. 3. FREE-BODY DIAGRAM OF FRAMES AT FORMATION OF PLASTIC MECHANISM

The uni-directional yield displacement, considering 2nd cycle properties is defined as:

$$\Delta_{y2} = \frac{\left(\frac{w_v}{2} - 2A_{ub}F_{yub}\right)\frac{d}{h}}{2\,k_f} + \frac{2F_{yub}L_{ub}}{E}\cdot\frac{h}{d} \tag{11}$$

The bi-directional yield displacement, considering a continuous linear horizontal path in the α-direction, is defined here as the vectorial displacement at the top of the pier in the x-y plane when the last device yields (using 2nd cycle properties) and using the simple geometric relationship between the x- and y-displacements with the displacement direction angle, α, the bi-directional yield displacement can be defined as:

$$\Delta_{y2,xy} = \sqrt{\Delta_{y,y}^{\;2} + \Delta_{y,x}^{\;2}} \tag{12}$$

$$\Delta_{y2,xy} = \sqrt{\Delta_{y,3}^{\;2} + \left(\frac{1}{\tan\alpha}\cdot\Delta_{y,3}\right)^2} = \Delta_{y,3}\sqrt{1 + \frac{1}{\tan^2\alpha}} \quad \left(-\frac{\pi}{4} < \alpha < \frac{\pi}{4},\ \frac{3\pi}{4} < \alpha < \frac{5\pi}{4}\right) \tag{13}$$

$$\Delta_{y2,xy} = \Delta_{y,3}\sqrt{1 + \tan^2\alpha} \qquad \left(\frac{\pi}{4} < \alpha < \frac{3\pi}{4},\ \frac{5\pi}{4} < \alpha < \frac{7\pi}{4}\right)$$

where $\Delta_{y,3}$ is the pier displacement, in the smaller displacement component direction, when the 3rd device yields using 2nd cycle hysteretic properties and is defined as:

$$\Delta_{y,3} = \frac{F_{F4}}{k_f} + 2\cdot\frac{F_{yub}\cdot L_{ub}}{E}\cdot\frac{h}{d} \tag{14}$$

and F_{F4} is defined by Eq. (8).

Design Applications

The design of a controlled rocking pier for seismic design (or retrofit) requires limiting pier displacements and ductility demands to the device while capacity design principles are applied to the pier and superstructure. More explicitly, these response quantities for design include pier drift, uplifting displacements (BRB strain), and maximum pier forces (frame shear, pier leg axial force). The controlled rocking system also has the ability to self-center using the restoring force provided by gravity if the local strength ratio, η_L, is less than unity (ignoring strain hardening) such that:

$$\eta_L = \frac{A_{ub}F_{yub}}{w_v/4} < 1.0 \tag{15}$$

To determine maximum pier displacements, the capacity spectrum analysis method (ATC/MCEER 2004) is used. This method of analysis was evaluated for 2-legged controlled rocking piers in past literature (Pollino and Bruneau 2004) and shown to predict maximum developed displacements of the pier with reasonable accuracy for design. Using this analysis method, spectral capacity and demand curves are established to determine maximum expected displacements. Spectral capacity curves are constructed for uni-directional (Eqs. (9) and (11)) and bi-directional properties (Eqs. (10) and (13)).

Pier uplifting displacements are then determined using the relationship of Eq. (6). to limit BRB strain.

The maximum forces developed during pier rocking are determined using the 100-40 directional combination rule and SRSS modal combination rule (a CQC modal combination rule could also be used) to combine the effects of bi-directional rocking of a pier subjected to 3 components of ground motion including the dynamic forces that result from impact and uplift of the pier legs. During the rocking response, the impacting and uplift of pier legs causes the excitation of vertical modes of vibration as discussed in Pollino and Bruneau (2004). It is assumed here that the dynamic amplification factors determined for 2-legged piers can be applied to the controlled rocking 4-legged pier.

Applying a 100%-40% combination rule to the two orthogonal horizontal pier displacements results in formation of the plastic mechanism (3 devices yielding) if the larger direction has a global displacement ductility of approximately 2.5 (1.0/0.4=2.5, $\mu=\Delta_{u,x}/\Delta_{y2}=2.5$). Including the dynamic forces caused during impact and uplift along with the effects of vertical excitation to Fig. 3., the free body diagram of each pier frame is shown in Fig. 4. The dynamic forces shown in Fig. 4. are shown as their maximum values and all applied in the same direction (downward). The combination rules are used to account for the fact that all of the dynamic effects are not in phase nor reach their maximum at the same time.

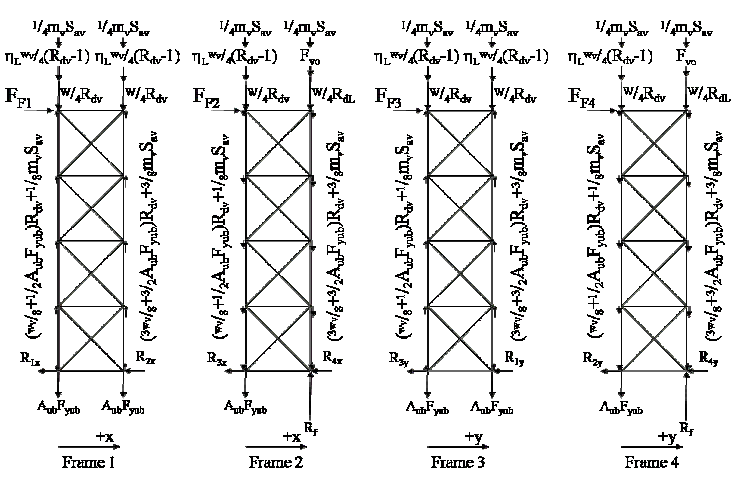


FIG. 4. FREE-BODY DIAGRAM OF FRAMES INCLUDING FORCES DUE TO DYNAMIC EFFECTS

Maximum Frame Shear

The maximum frame shear (P_{uF}) that develops, including dynamic effects, can be determined using the free-body diagram of Fig. 4. applying the appropriate combination rules such that:

$$P_{uF,100-40} = \max\begin{pmatrix} 1.0\cdot\left(P_{uF,st}\cdot R_{dv}\right) + 0.4\cdot\left(\dfrac{3m_v}{8}\cdot S_{av}\cdot\dfrac{d}{h}\right) \\ 0.4\cdot\left(P_{uF,st}\cdot R_{dv}\right) + 1.0\cdot\left(\dfrac{3m_v}{8}\cdot S_{av}\cdot\dfrac{d}{h}\right) \end{pmatrix} \qquad (16)$$

where $P_{uF,st}$=maximum frame shear considering static response and is equal to Eq. (8). S_{av} is the vertical spectral acceleration taken at the vertical period of a "fixed-base" pier, defined by T_L, and equal to:

$$T_L = 2\pi\sqrt{\frac{m_v/4}{k_L}} = 2\pi\sqrt{\frac{m_v}{4EA_L/h}} \qquad (17)$$

where A_L is the cross-sectional area of a pier leg.

Maximum Pier Leg Axial Force

The maximum developed axial force in a pier leg can be determined, assuming the pier uplifts and yields 3 devices and that the pier is completely supported vertically on one leg and considering that two pier diagonals connect to the base of the compressed pier leg and their load is applied directly into the support. Applying equilibrium at the base of the pier leg in Fig. 4. and using a 100-40 directional combination rule and an SRSS modal combination rule, the maximum pier leg force is equal to:

$$P_{uL,100-40} = \max\begin{pmatrix} 0.4\cdot F_{ve} + 1.0\cdot\sqrt{F_{vo}^2 + F_w^2 + F_{up}^2} \\ 1.0\cdot F_{ve} + 0.4\cdot\sqrt{F_{vo}^2 + F_w^2 + F_{up}^2} \end{pmatrix} \qquad (18)$$

The four terms are a result of uplifting and yielding devices along with dynamic effects that occur during pier rocking and vertical excitation as discussed previously. F_{ve} is the pier leg force resulting from vertical excitation and is equal to:

$$F_{ve} = \frac{1}{4}m_v\cdot S_{av} + \frac{3}{4}m_v\cdot S_{av}\cdot\left(1 - \frac{d}{2h}\right) \qquad (19)$$

F_{vo} results from the initial impacting of the pier leg after it has uplifted and is returning to its support:

$$F_{vo} = v_o\cdot\sqrt{\frac{m_v k_L}{4}} \qquad (20)$$

F_w is the weight tributary to a single pier leg amplified due to the sudden application of this load such that:

$$F_w = \frac{w_v}{4} \cdot R_{dL} \tag{21}$$

F_{up} is the remaining pier tributary weight and device's yield forces amplified to account for the sudden transfer of these loads to the compressed pier leg during uplift:

$$F_{up} = \left(\frac{3 \cdot w_v}{4} + 3 A_{ub} F_{yub} \right) \cdot R_{dv} \cdot \left(1 - \frac{d}{2h} \right) \tag{22}$$

The maximum impact velocity is taken as the sum of the impact velocity as a result of the pseudo-velocity of the pier from motions in the two horizontal directions (v_{ox}, v_{oy}) such that the total impact velocity, v_o, is equal to:

$$v_o = v_{o,x} + v_{o,y} = PS_{vix} \cdot \frac{d}{h} + PS_{viy} \cdot \frac{d}{h} \tag{23}$$

The pseudo-velocity is determined following methods in Pollino and Bruneau (2004). The pseudo-velocity in the y-direction is determined using a 100-40 combination rule such that Δ_{uy}=0.4Δ_{ux}.

Dynamic Analysis Example

An example dynamic response analysis is presented here to further illustrate these concepts and provide recommendations for design. A pier with aspect ratio of 4 (h/d=4), "fixed-base" lateral stiffness of 6.25kN/mm (k_f=6.25kN/mm), vertical tributary weight of 1730kN (w_v=1730kN), and effective horizontal inertial masses in each direction of w_v/g (m_x=m_y=w_v/g). The buckling-restrained brace properties are such that η_L=0.5 and L_{ub}=7315mm (F_{yub}=234MPa, Nakashima 1995). In an actual design scenario, the devices would be calibrated and pier properties (strength, stiffness) changed to satisfy a number of design constraints. This process has been detailed for 2-legged piers in Pollino and Bruneau (2004). Here, a single set of device and pier properties are considered to investigate the dynamic response and the methods for predicting bi-directional response.

First, maximum pier displacements are predicted using the capacity spectrum procedure. Two sets of spectral capacity and demand curves will be defined, one for uni-directional and the other considering bi-directional properties. The uni-directional spectral capacity curve is defined by P_y=324kN and Δ_{y2}=94.5mm. The bi-directional capacity curve is defined using Eq. (10) and (13) with $\alpha=\tan^{-1}(0.4/1.0)$ (following a 100-40 directional combination rule) such that $P_{y,xy}$=458kN and $\Delta_{y2,xy}$=289mm. The 5% damped demand spectrum is simply taken as the design spectrum defined in ATC/MCEER (2004). A site located in Northridge, CA and seismic event with 3% probability of exceedance in 75 years is considered here. The spectral acceleration values are taken from the USGS with a short period (0.2sec) spectral acceleration, S_s, of 1.95g and one-second spectral acceleration of

0.87g. The vertical spectrum is defined by shifting the characteristic period of the vertical spectrum, T_o (defined in ATC/MCEER 2004), to a shorter period range and reducing the amplitude of the horizontal spectrum. The characteristic period is reduced by a factor of 1.55 and the amplitude is reduced by a factor of 1.25. The bi-directional spectral demand curve considers the seismic demand in the two orthogonal directions. In most cases the seismic demand can be assumed equal in the two orthogonal directions and taken as the site's design spectrum. Considering a 100-40 directional combination rule, the magnitude of the design displacement vector is equal to:

$$\Delta_{u,xy} = \sqrt{\left(1.0 \cdot \Delta_{u,x}\right)^2 + \left(0.4 \cdot \Delta_{u,y}\right)^2} \tag{24}$$

If the seismic demand and uni-directional pier properties considered are identical in each direction then the predicted displacement in the x- and y-direction will be equal ($\Delta_{u,x}=\Delta_{u,y}=\Delta_u$) such that:

$$\Delta_{u,xy} = \sqrt{1.0^2 + 0.4^2} \cdot \Delta_u = 1.08 \cdot \Delta_u \tag{25}$$

Therefore, applying a 100-40 directional combination rule suggests an increase in the uni-directional displacement demand by a factor of 1.08. The spectral demand curve is then reduced for the energy dissipation that occurs due to the plastic work of the devices and defined as an equivalent amount of viscous damping. The system's equivalent viscous damping is determined using the following expression:

$$\xi_{eff} = \xi_o + \xi_{hys} = \xi_o + \left[\frac{\eta_L}{1+\eta_L} \cdot \frac{2}{\pi} \cdot \left(1 - \frac{1}{\mu}\right)\right] \tag{26}$$

where ξ_o=inherent structural damping (assumed to be 2% of critical) and ξ_{hys}=hysteretic damping provided by buckling-restrained braces during rocking response and μ=displacement ductility ratio considering 2nd cycle properties ($\mu=\Delta_{ux}/\Delta_{y2}$ for uni-directional response and $\mu=\Delta_{u,xy}/\Delta_{y2,xy}$ for bi-directional response).

For the pier and BRB properties considered here, the final spectral capacity and demand curves for uni-directional and bi-directional response are shown in Figs. 5a. and 5b. respectively. It can be seen that a displacement of 480mm is predicted considering uni-directional response. The bi-directional vectorial displacement is shown to be 500mm and thus its x-direction component following the 100-40 rule used is equal to:

$$\Delta_{u,x} = \Delta_{u,xy} \cdot \sqrt{\frac{1}{1+\tan^2\alpha}} = 500\,mm \cdot \sqrt{\frac{1}{1+\tan^2(0.38\,rad)}} = 464\,mm \tag{27}$$

Thus the predicted displacement considering bi-directional response is slightly less than that for uni-directional response.

Using the maximum developed displacement predicted considering uni-directional pier properties, the critical response quantities are determined. The maximum displacement in one of the primary directions (x or y) was found to be 480mm, thus applying the 100-40 directional combination rule, the maximum pier displacement ($\Delta_{u,xy}$) is equal to

1.08(480mm)=518.4mm. Applying Eq. (6), the maximum uplifting displacement ($\Delta_{up,L1}$), considering the seismic demand in two directions ($\Delta_{u,y}$=0.4$\Delta_{u,x}$), is equal to 164.8mm and thus a maximum buckling-restrained brace strain of:

$$\epsilon_{ub} = \frac{\Delta_{up,L1}}{L_{ub}} = \frac{164.8\,mm}{7315\,mm} = 0.0225 \quad (2.25\%) \tag{28}$$

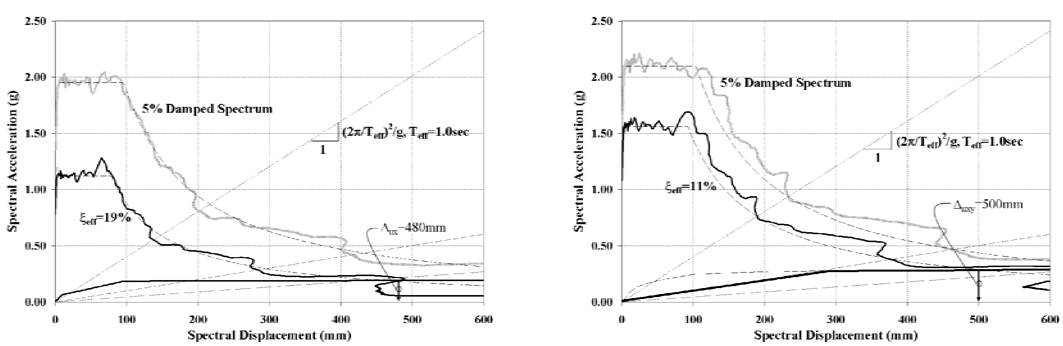
To determine the maximum force demands, the dynamic amplification factors (R_{dv} and R_{dL}) are determined using the methods presented in Pollino and Bruneau (2004) and are equal to 1.77 and 1.97 respectively. The vertical spectral acceleration at the vertical period of the pier is 1.95g (for 2% damping). The maximum frame shear, from Eq. (16), is 560.5kN. Finally, the maximum pier leg axial force is determined from Eq. (18) is equal to 4459kN.

Ground Motions

Spectra compatible ground acceleration time histories used for the dynamic analyses are generated using the Target Acceleration Spectra Compatible Time Histories (TARSCTHS) software developed by the Engineering Seismology Laboratory (ESL) at the State University of New York (SUNY) at Buffalo (http://civil.eng.buffalo.edu/users_ntwk/index.htm). Synthetic ground motions were generated by TARSCTHS matching the elastic response spectrum for the Northridge site. Seven sets of three (x, y, and z) ground motion histories are made. The average resulting x-direction spectra of the motions generated with its respective target design spectrum is shown in the spectral analysis plots of Figs. 5a. and 5b.

Results and Discussion

The results of dynamic analyses are shown in Figs. 6a.-6d. All results plots are shown with the design response quantity on the horizontal axis (single value) and the results from time history analysis on the vertical axis. The result of each time history is shown as



(A) (B)

FIG. 5. SPECTRAL ANALYSIS PLOTS (A) UNI- (B) BI-DIRECTIONAL RESPONSE

a data point along with the median, median+σ, and median-σ response. Also, a line is plotted that divides the conservative and unconservative prediction of response. The median response of all motions is found to be conservative. Pier displacements ($\Delta_{u,x}$, $\Delta_{u,y}$, and $\Delta_{u,xy}$) are shown in Fig. 6a. The uplifting displacement results shown in Fig. 6b. are approximately "as conservative" as the bi-directional displacement, $\Delta_{u,xy}$, as expected. The maximum frame shear and pier leg axial forces are shown in Figs. 6c. and 6d. respectively.

Conclusions

The bi-directional and uni-directional kinematic and hysteretic properties of controlled rocking, 4-legged steel truss piers is investigated with focus on developing design rules. Key variables for the cyclic hysteretic behavior of controlled rocking piers are defined analytically. Design rules are established to predict pier displacement, device ductility, and capacity protection of the existing pier. The design rules include the excitation of three components of ground motion and dynamic effects caused by impacting and uplifting during the rocking response. Nonlinear, dynamic time history analyses are performed to assess the design rules. Results of the analyses found the design rules to conservatively predict response with respect to the median response of all analyses run and with reasonable accuracy.

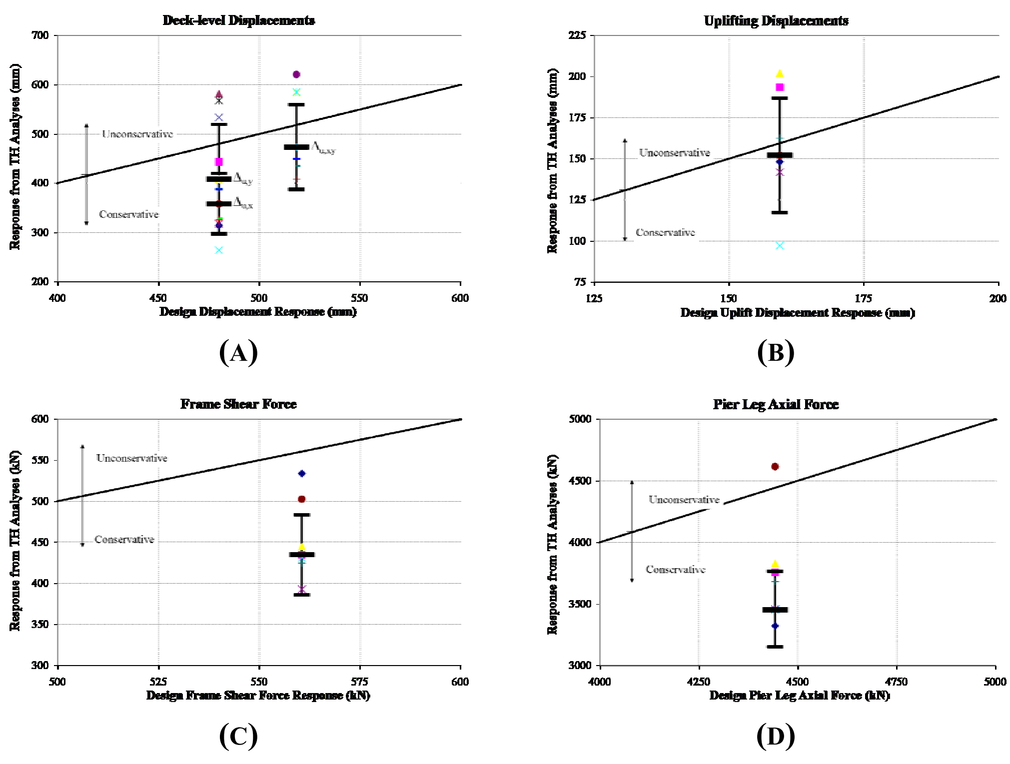


FIG. 6. RESULTS OF DYNAMIC ANALYSES. (A) MAX. PIER DISPLACEMENTS; X-, Y-, AND X-Y DIRECTIONS, (B) MAX. UPLIFTING DISPLACEMENTS, (C) MAX. FRAME SHEAR, AND (D) MAX. PIER LEG AXIAL FORCE

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