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

Michael Pollino[1] and Michel Bruneau, Ph.D., P.Eng.[2]

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 of controlled rocking 4-legged piers, relevant for design, is developed analytically and some of the design rules are established. The seismic response of a 4-legged pier to 3 components of ground excitation is then investigated using inelastic, dynamic time history analyses.

Introduction

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

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

[1]Graduate Research Assistant, University at Buffalo, 212 Ketter Hall, Buffalo, NY 14260, E-mail: mpollino@eng.buffalo.edu
[2]Director, Multidisciplinary Center for Earthquake Engineering Research, and Professor, Dept. of Civil, Structural and Environmental Engineering. University at Buffalo, 212 Ketter Hall, Buffalo, NY 14260, E-mail: bruneau@buffalo.edu

developed such that simple 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 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 compare the results of nonlinear, time history analyses and the design rules. Seven sets (x, y, and z) of synthetically generated acceleration histories are used in the time history 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 a defined coordinate system. Also shown is a directional vector that lies in the x-y plane at an angle 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" and shown to not be path dependent beyond the 2nd cycle in Pollino and Bruneau (2004). The primary difference between the hysteretic behavior of controlled rocking 2-legged piers and the uni-directional response (=n /2 rad., n=0,1,2,...) of 4-legged piers is the use of four energy dissipating devices (one at the base of each leg). However, when a 4-legged pier is free to move in the entire x-y plane, the hysteretic curve is path dependent and therefore is only defined for the path considered. Thus, the kinematic and hysteretic behavior of 4-legged piers, considering bi-directional response, is investigated to provide recommendations for design to account for the uncertainty in the hysteretic path.

Compatibility, equilibrium, and force-deformation relationships for a 4-legged pier are developed below to assist in the design of controlled rocking piers. For earthquake excitation in 2 horizontal directions, it is possible for the pier to uplift such that it is supported vertically on one of its legs. In that case, 3 of the energy dissipating devices located at the base of the pier could yield depending on the magnitude of the respective uplifting displacement. Assuming that rotation of the pier about a vertical axis does not occur (i.e. neglecting torsional response), the top of parallel frames undergo the same horizontal deformations. In other words, using the notation illustrated in Fig. 1, the top of frames 1-1 and 2-2 experience the same horizontal displacements while frames 3-3 and

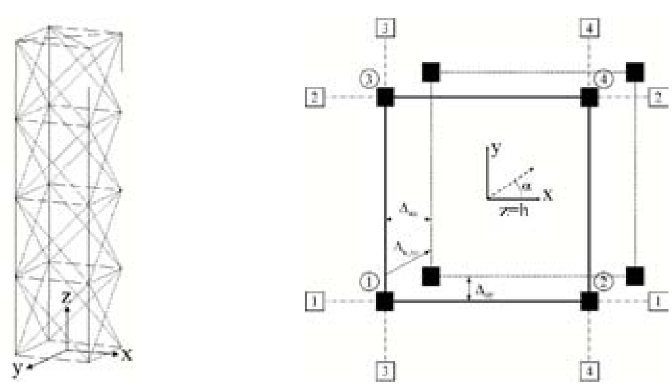


Figure 1. Typical 4-legged pier and defined coordinate system

4-4 experience the same displacements. The displacement of the top of frame m ($_{u,m}$) is the sum of deformations due to the flexibility of the frame's structural members ($_{o,m}$) and rigid body rotation at the base of the frame ($_{br,m}$) (see Fig. 2) such that:

$$\Delta_{u,m} = \Delta_{o,m} + \Delta_{br,m} \tag{1}$$

The displacement due to rigid body rotation of frame m ($_{br,Fm}$) is related to the uplifting displacement of the frame ($_{up,Fm}$), which is defined as the difference of the uplifting displacement of the two legs (i and j) of the frame such that:

$$\Delta_{br,Fm} = \Delta_{up,Fm} \cdot \frac{h}{d} = \left(\Delta_{up,Li} - \Delta_{up,Lj}\right) \cdot \frac{h}{d} \tag{2}$$

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

Since each pier leg "belongs" to two frames (one in each of the x- and y-direction), the uplifting displacement of any given pier leg is dependent on the pier lateral displacement in the x- and y-direction. For example, considering a global displacement in the +x- and +y-directions, the uplifting displacement of pier leg 1 (see Fig. 1 for pier leg numbers) can be determined by summing the uplifting displacements of frames 4 and 1 or frames 2 and 3 (where frame "x" here is defined as the frame located along line x-x in Fig. 1). If the top of pier displacements are in the positive x- and y-directions, and ignoring torsion as indicated earlier, then $_{u,F1}=$ $_{u,F2}=$ $_{u,x}$, $_{u,F3}=$ $_{u,F4}=$ $_{u,y}$, and $_{up,L4}=0$ and the uplifting displacement of leg 1 is given by:

$$\Delta_{up,L1} = \left(\Delta_{u,x} + \Delta_{u,y} - \frac{F_{F1} + F_{F4}}{k_f} \right) \cdot \frac{d}{h} \tag{3}$$

If the hysteretic path to reach $_{u,x}$ and $_{u,y}$ results in the formation of the pier's plastic mechanism, defined as any pier displacement resulting in simultaneous yielding of three of the energy dissipating devices, having elastic-perfectly plastic behavior (i.e. neglecting strain hardening), then simply through the equilibrium of forces, the horizontal shear force to frames 1 and 3 is:

$$F_{F1,3} = \left(\frac{w_v}{8} + \frac{1}{2} A_{ub} F_{yub} \right) \cdot \frac{d}{h} \tag{4}$$

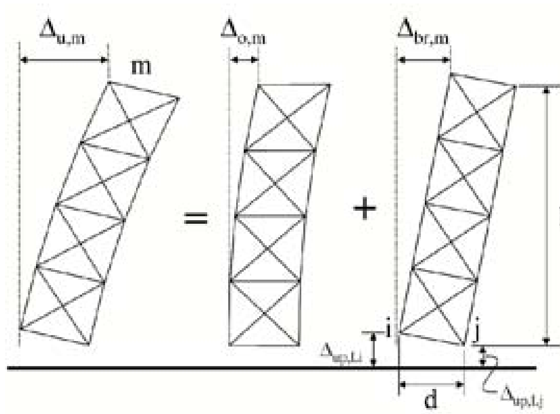


Figure 2. Kinematics of controlled rocking truss pier

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

$$F_{F2,4} = \left(\frac{3w_v}{8} + \frac{3}{2} A_{ub} F_{yub} \right) \cdot \frac{d}{h} \tag{5}$$

where w_v is the vertical tributary load of the pier and $A_{ub}F_{yub}$ is the yield force of the device.

Assuming identical frame stiffness and energy dissipating devices applied in each direction, the uni-directional yield force, P_y, is defined in Pollino and Bruneau (2004) as:

$$P_y = \left(\frac{w_v}{2} + 2 \cdot A_{ub} F_{yub} \right) \frac{d}{h} \qquad \left(\alpha = \frac{n\pi}{2} rad., n = 0, 1, 2, ... \right) \tag{6}$$

Considering bi-directional response ($\neq$ n /2 rad., n=0,1,2,...) and bi-directional yielding (3 devices yield), the maximum shear force in each direction (F_{F1}+F_{F2}, F_{F3}+F_{F4}), is also equal to the uni-directional yield force, thus $F_{x,max}$=$F_{y,max}$=P_y, and the total applied horizontal shear force and bi-directional yield force is defined as:

$$P_{y,xy} = \sqrt{F_{x,\max}^{\;2} + F_{y,\max}^{\;2}} = \sqrt{2 \cdot F_{x,\max}^{\;2}} = \sqrt{2} \cdot P_y \qquad \left(\alpha \neq \frac{n\pi}{2} rad., n = 0, 1, 2, ... \right) \tag{7}$$

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

$$\Delta_{y2} = \frac{\left(\frac{w_v}{2} - 2A_{ub}F_{yub} \right) \frac{d}{h}}{2\,k_f} + \frac{2F_{yub}L_{ub}}{E} \cdot \frac{h}{d} \tag{8}$$

The bi-directional yield displacement, considering a continuous linear horizontal path in the -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, , the bi-directional yield displacement can be defined as:

$$\Delta_{y2,xy} = \sqrt{\Delta_{y,y}^{\;2} + \Delta_{y,x}^{\;2}} = \Delta_{y,3} \sqrt{1 + \frac{1}{\tan^2\alpha}} \qquad \left(-\frac{\pi}{4} < \alpha < \frac{\pi}{4}, \; \frac{3\pi}{4} < \alpha < \frac{5\pi}{4} \right) \tag{9}$$

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

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

$$\Delta_{y,3} = \frac{F_{F4}}{k_f} + 2 \cdot \frac{F_{yub} \cdot L_{ub}}{E} \cdot \frac{h}{d} \tag{10}$$

and F_{F4} is defined by Eq. 5.

Design Applications

The design of a controlled rocking pier for seismic design (or retrofit) requires limiting pier displacements and ductility demands to the energy dissipating devices, while capacity design principles are applied to limit the maximum forces transferred to the pier and superstructure. More explicitly, the response quantities that must be controlled as part of the design process include: pier drift, uplifting displacements (to limit BRB strain), and maximum pier forces. The limits of these response quantities for achieving the desired seismic performance will vary from bridge-to-bridge but must be met. The controlled rocking system also has the ability to self-center using the restoring force provided by gravity if the local strength ratio, $_L$, is less than unity (ignoring strain hardening) such that:

$$\eta_L = \frac{A_{ub} F_{yub}}{w_v / 4} < 1.0 \tag{11}$$

Displacements

Maximum pier displacements must be limited for bridge serviceability requirements and to maintain pier stability. To determine maximum pier displacements (either uni-directional or bi-directional), the capacity spectrum analysis method (ATC/MCEER 2004) is used. Using this analysis method, spectral capacity and demand curves are established to determine maximum expected displacements. Spectral capacity curves are defined using an idealized, bi-linear curve using Eqs. 6-9. The reduced spectral demand curves are obtained in this case considering an amount of viscous damping given by:

$$\xi_{eff} = \xi_o + \xi_{hys} = \xi_o + \left[\frac{\eta_L}{1 + \eta_L} \cdot \frac{2}{\pi} \cdot \left(1 - \frac{1}{\mu}\right) \right] \tag{12}$$

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

Uplifting Displacements

Pier leg uplifting displacements are determined using the relationship of Eq. 3. to limit BRB strain. For an axially yielding device, such as the BRB, the axial strain of the device is related to the uplifting displacement of the leg which the device is attached to (assuming it is implemented vertically) through the simple relationship:

$$\epsilon_{ub} = \frac{\Delta_{uplift}}{L_{ub}} \tag{13}$$

Maximum Pier Forces

Maximum pier forces are limited such that pier members remain elastic (capacity protection) and all inelastic action occurs at the base of the pier within the energy dissipation devices. For 4-legged piers, the maximum developed forces considers bi-directional pier response using a 100-40 directional combination rule and SRSS modal combination rule 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. It can be shown that 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, $= {}_{u,x}/ {}_{y2}$=2.5). The maximum frame shear (P_{uF}) that develops, including dynamic effects and directional combination of the horizontal and vertical excitation effects, is determined as:

$$P_{uF,100-40} = \max\left(\begin{array}{l} 1.0\cdot\left(P_{uF,st}\cdot R_{dv}\right) + 0.4\cdot\left(\dfrac{3m_v}{8}\cdot S_{av}\cdot\dfrac{d}{h}\right) \\ 0.4\cdot\left(P_{uF,st}\cdot R_{dv}\right) + 1.0\cdot\left(\dfrac{3m_v}{8}\cdot S_{av}\cdot\dfrac{d}{h}\right)\end{array}\right) \tag{14}$$

where $P_{uF,st}$=maximum frame shear considering bi-directional, static response and is equal to Eq. 5. R_{dv} is the dynamic amplification factor applied to the vertical loads impulsively transferred during uplift. S_{av} is the vertical spectral acceleration taken at the vertical period of a "fixed-base" pier, defined by T_L, and equal to:

$$T_L = 2\pi\sqrt{\frac{m_v}{4k_L}} = 2\pi\sqrt{\frac{m_v}{4EA_L/h}} \tag{15}$$

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

Dynamic Analysis Example

A set of seven nonlinear, time history analyses are presented here for an example pier and seismic demand to further illustrate these concepts and provide recommendations for design. The expressions for predicting response quantities for design discussed in the previous section are compared with the results of the analyses. Design considering uni-directional and bi-directional response will be investigated and implications on design noted.

Pier and Energy Dissipating Device Properties

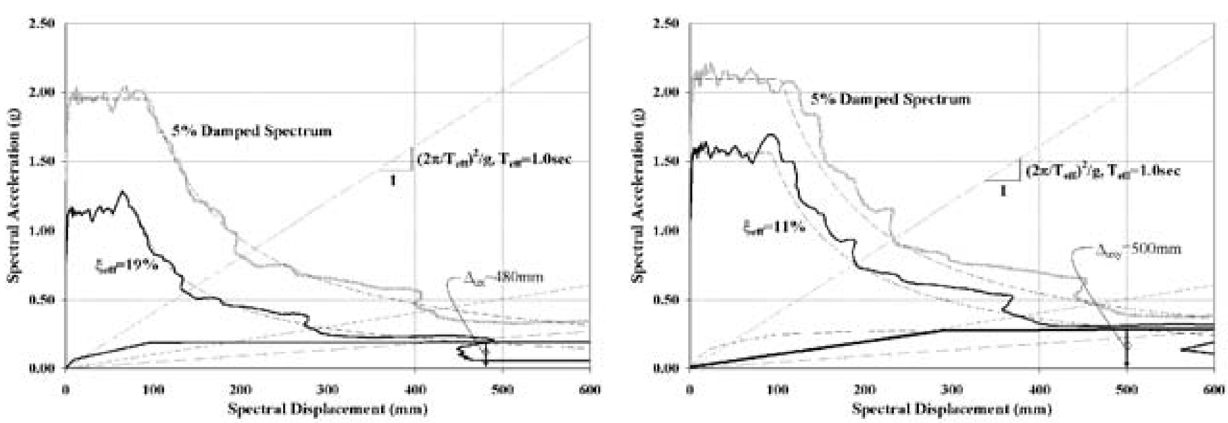
A pier with aspect ratio (h/d)of 4, "fixed-base" lateral stiffness (k_f) of 6.25kN/mm, vertical tributary weight (w_v) of 1730kN, and effective horizontal inertial masses in each direction of w_v/g ($m_x=m_y=w_v/g$). The buckling-restrained brace properties are such that ${}_L$=0.5 and L_{ub}=7315mm (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).

Seismic Hazard and Ground Motions

A site located in Northridge, CA and seismic event with 3% probability of exceedance in 75 years is considered here with a seismic design spectrum as defined in ATC/MCEER (2004). 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 for 5% damping. The vertical spectrum is defined by shifting the characteristic period of the horizontal spectrum, T_o (by a factor of 1.55), to a shorter period range and reducing its amplitude (by a factor of 1.25). Seven sets of three (x, y, and z) ground motion histories are made (14 matching horizontal spectrum and 7 matching the vertical). Each ground motion is synthetically 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). The average resulting spectra of the horizontal motions generated with its respective target design spectrum is shown in the spectral analysis plots of Figs. 3a. and 3b.

Simplified Analysis Procedure

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. Idealized, bi-linear spectral capacity curves are constructed for uni-directional (Eqs. 6 and 8) and bi-directional properties (Eqs. 7 and 9, with $=\tan^{-1}(0.4/1.0)$ following the 100-40 directional combination rule). The uni-directional spectral capacity curve is defined by P_y=324kN and $_{y2}$=94.5mm and bi-directional capacity curve is defined by $P_{y,xy}$=458kN and $_{y2,xy}$=289mm. The uni-directional spectral demand curve is simply taken as the design spectrum defined in ATC/MCEER (2004) then reduced for the system's energy dissipation espressed as an amount of viscous damping per Eq. 12. The bi-directional spectral demand curve considers the seismic demand in the two orthogonal directions since its corresponding capacity curve uses hysteretic properties of bi-directional response. The bi-directional demand curve is also reduced per Eq. 12.



(a) (b)

Figure 3. Spectral analysis plots (a) uni-directional and (b) bi-directional

The final spectral capacity and demand curves for uni-directional and bi-directional response are shown in Figs. 3a. and 3b. 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 464mm.

Thus the predicted displacement considering bi-directional response is slightly less than that for uni-directional response. The increase in system strength of bi-directional hysteretic properties from uni-directional properties ($P_{y,xy}=\sqrt{2}\,P_y$) likely leads to the prediction of smaller displacements. Therefore it will be conservative to use the uni-directional properties for the prediction of pier displacements.

Prediction of Design Response Quantities

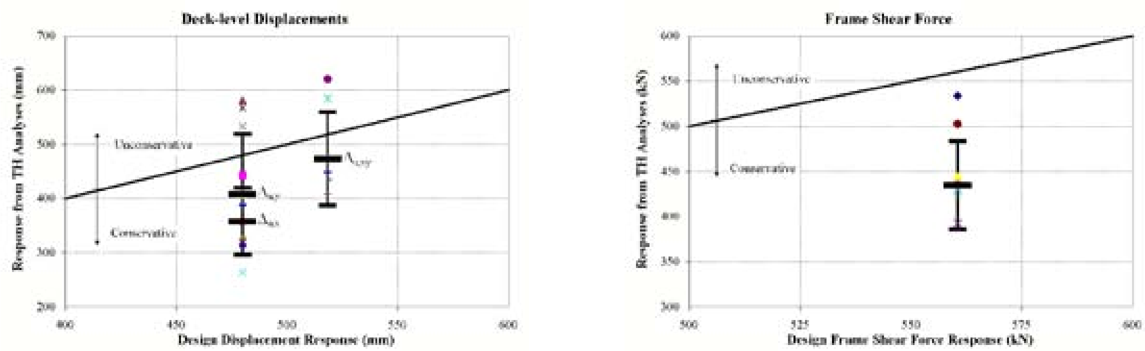
Using the maximum developed uni-directional displacement of 480mm, the critical response quantities are determined. Applying the 100-40 directional combination rule, the maximum pier displacement ($_{u,xy}$) is equal to1.08(480mm)=518.4mm. Applying Eq. 3, the maximum uplifting displacement ($_{up,L1}$), considering the seismic demand in two directions ($_{u,y}$=0.4 $_{u,x}$), is equal to 164.8mm, which corresponds to a maximum buckling-restrained brace strain of 2.25% (Eq. 13). 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. 14, is 560.5kN.

Results and Discussion of Analyses

Results of the seven time history analyses are shown in Fig. 4. Pier displacements ($_{u,x}$, $_{u,y}$, and $_{u,xy}$) are shown in Fig. 4a and the maximum frame shear is shown in Fig. 4b. All plots show the predicted design response quantity on the horizontal axis (single value) and the results from the seven time history analysis on the vertical axis. The result of each time history analysis is shown as a data point along with the median, median+ , and median- response. Also, a line is plotted in each figure dividing conservative and unconservative prediction of response. The median response of all analyses is found to be conservative.

Conclusions

The bi-directional and uni-directional kinematic and hysteretic properties of controlled rocking, 4-legged steel truss piers has been investigated. Some key variables for the cyclic hysteretic behavior of controlled rocking piers have been identified as pier displacements, device strain, and maximum frame shear and analytical expressions have been developed for their calculation by hand (design rules). The design rules account for three components of excitation and dynamic effects caused by impacting and uplifting during the rocking response. Nonlinear, dynamic time history analyses are performed to assess the effectiveness of these analytical expressions used for design. Results from nonlinear time history analyses found the expressions to conservatively predict response with respect to the median response of all analyses run and with reasonable accuracy.



(a) (b)

Figure 4. Results of dynamic analyses (a) max. pier displacements; x-, y-, and xy-directions and (b) max. frame shear

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