

Experimental Study of the Controlled Rocking Response of Steel Braced Frames

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ABSTRACT

Controlled rocking of steel braced frames has been proposed for the seismic retrofit of structures. The controlled rocking approach allows a frame to uplift from its support at the base-of-column to foundation interface while displacement-based steel yielding devices are implemented at the base location to control the response. Past studies have focused on analytical investigations of the response of such a system along with design applications for controlled rocking bridge piers. This paper discusses shake table testing of a 4-legged, steel braced pier representative of a 1/5 length scale highway bridge pier. The model is subjected to a series of seismic excitations about one of its primary orthogonal axes, using horizontal and vertical base motions, and rotated 45deg. to investigate pier behavior that would be expected from bi-directional horizontal base inputs. Ground motions from the 1940 El Centro and 1994 Northridge earthquakes are used along with a synthetically generated motion. The steel yielding devices used during testing are triangular plates that yield in flexure as the pier uplifts and rocks at its base. Results of the testing focus on the overall behavior of the controlled rocking pier including global hysteretic response, uplifting displacements, and pier forces.

INTRODUCTION

The developments of seismic protective systems that provide nonlinear elastic behaviour and prevent damage to a structure's primary members have recently received increased interest. This is in part due to a growing appreciation for the ability of such systems to efficiently withstand seismic demands elastically (without damage) or directing damage to easily replaceable structural "fuses". These types of systems can often also be designed to provide a self-centering ability. This behaviour has been achieved through post-tensioning of structural members [Mander and Cheng 1997; Christopoulos et. al. 2002; Garlock et. al. 2005]. However it may also be achieved by simply allowing structures to uplift from their foundation while preventing sliding, creating a rocking response. The seismic behaviour of conventional structural systems, such as special moment-resisting frames, concentrically braced frames, and eccentrically braced frames [AISC 2005], can be satisfactory from a life safety point of view [FEMA 2003], however damage to members can leave the structure unusable until costly repairs can be made.

The study of rocking structures is not new. Housner [1963] first investigated the free and forced vibration response of rigid rocking blocks, Meek [1975] introduced aspects of structural flexibility to the seismic response of single-degree-of-freedom (SDOF) rocking structures, and Psycharis [1982] followed with an analytical study of the dynamic behavior of simplified multi-degree-of-freedom (MDOF) structures supported on flexible foundations free to uplift. Others have also conducted experimental tests on rocking structural systems including Priestley et. al. [1978], Kelley and Tsztoo [1977], and Midorikawa et. al. [2003]. Yet, investigation of applying and implementing these concepts into the design and retrofit of structures is needed.

In order to complement the system otherwise free to uplift, Pollino and Bruneau [2007] have proposed a controlled rocking approach for seismic resistance in which passive energy dissipation devices are added at the uplifting location to control the rocking response of steel braced frames. The devices considered were displacement-based steel yielding elements that could be calibrated to achieve a desired level of response. The cyclic hysteretic behavior including P-Δ effects and a capacity-based design method were developed. The design method used a number of design constraints including limiting maximum displacements, impact velocity, and maximum dynamic forces such that the structure could remain elastic and self-center following excitation. Simplified methods of analysis were used to predict system response. Parametric studies, that used nonlinear time history analyses, were performed to evaluate the design and simplified analysis methods.

In order to further investigate the dynamic rocking response of controlled rocking steel braced frames, an experimental study was conducted using a 5 degree-of-freedom (5DOF) shake table in the Structural Engineering and Earthquake Simulation Laboratory (SEESL) at the University at Buffalo (UB). A 1/5 length scale specimen, representative of a 4-legged steel braced highway bridge pier was used during testing. The model was subjected to a series of seismic excitations about one of its primary axes (θ=0deg.), using horizontal and vertical base motions, and rotated 45deg. (θ=45deg.) to investigate behavior that would be expected from bi-directional horizontal base inputs. Triangular steel plates that yield in flexure were used as the passive energy dissipation devices during testing. Properties of the experimental specimen, set-up, instrumentation, seismic input, and some results of testing are discussed in this paper.

DISCUSSION OF PROTOTYPE

Prototype properties are based on a brief review of drawings of existing bridges supported on steel truss piers. The prototype bridge pier supports an idealized 2-lane highway bridge deck between the bridge's abutments. The pier is assumed to have a tributary inertial mass in the longitudinal and transverse directions equal to its vertical mass. In general, the steel bracing diagonals tend to have a constant cross-section in each pier panel and pier legs are continuous over its height. Connections of beam members to pier legs typically were such that they could be considered pin-connected however moment resisting connection details were observed in some cases. Connection of the bridge deck to pier varies considerably depending on the type of bridge bearing used or whether bearings were used at all. Relevant prototype pier properties are given in Table 1. The vertical and vertical "shearing" periods are relevant due to the activation of these modes of vibration during rocking response. This behavior is discussed and its effects quantified in Pollino and Bruneau (2004).

SIMILITUDE SCALING AND ARTIFICIAL MASS SIMULATION

It was desired to have the largest scaled model reasonably possible given the available resources in the laboratory (table size, capacity; vertical clearance). Model scale was ultimately controlled by the vertical distance from the shake table to a workable crane clearance height (Figure 1). This led to a length scale factor of approximately 5 based on the prototype height of 30.3m.

The scale factors, λ, are defined as the ratio of the prototype quantity to the model quantity such that:

$$\lambda = \frac{Q_{\text{prototype}}}{Q_{\text{model}}} \tag{1}$$

Following an artificial mass simulation scaling law, the important quantity scale factors required and actually provided by the model are given in Table 2. The acceleration scale factor, λ_a, is equal to unity due to the prototype and model being in the same gravitation field. Since the model and prototype are made of the same material (steel), the elastic modulus scale factor, λ_E, is also equal to one. Based on the scale factors in Table 2, required model properties following this scaling law, deemed most important for testing, are shown in Table 1 along with the model properties actually provided.

It was discovered that an existing slender steel specimen was available in the laboratory that has been used in past testing and provided the relevant required model properties reasonably well. However some modifications were made to the existing structure to meet some of the similitude requirements as well as for strength purposes. The primary similitude requirements targeted were the "fixed-base" lateral and vertical periods of vibration of the model (T_{om} and T_{Lm}), the shearing mode period of vibration (T_{vm}), and the applied and restoring forces of the model which are controlled primarily by the added mass.

Although an added mass of 69.4kN/g is required by similitude, steel plates totaling 75.2KN/g were used since they were readily available in the laboratory. Connection of the mass to the pier was made through 16-9.5mm fully-tensioned, high-strength threaded rods through the 2-90mm thick steel mass plates, a double concave hardened steel bearing, mild-steel connection plate, and 2-19.1mm plate washers as shown in Figure 2(a). The connection was designed to transfer shear force in the horizontal plane and vertical forces and moments between the mass and pier. Eigenvalue and bi-directional pushover analysis were used to assess the impact of this connection to represent the boundary conditions in actual bridges. No significant differences were observed for the connection considered here.

Quantity	Prototype	Model: Required	Model: Provided
Pier Height, h	30.3m	6.06m	6.09m
Pier Width, d	7.58m	1.516m	1.518m
Pier Aspect Ratio, h/d	4.0	4.0	4.0
Inertial Mass, m_x	1730kN/g	69.2kN/g	75.6kN/g
Gravitational Weight, W_z	1730kN	69kN	75.6kN
Material Elastic Modulus, E	200MPa	200MPa	200MPa
Pier Lateral Stiffness, k_{ox}	12.6kN/mm	2.52kN/mm	2.34kN/mm
Lateral Period, T_{ox}	0.74sec	0.33sec	0.36sec
Vertical Period, T_L	0.128sec	0.057sec	0.038sec
Vertical “Shearing” Period, T_v	0.080sec	0.036sec	0.028sec

TABLE 1
PROTOTYPE PIER PROPERTIES AND MODEL PROPERTIES (REQUIRED BY SIMILITUDE AND PROVIDED)

Scaling Quantity	Dimensional Scale Requirements	Required Scale Factor
Geometric Length, L	$\lambda_L = ?$	5.00
Gravitational Acceleration, g	$\lambda_g = \lambda_E/(\lambda_L\,\lambda_\rho) = 1.0$	1.00
Acceleration, a	$\lambda_a = \lambda_E/(\lambda_L\,\lambda_\rho)$	1.00
Time, t	$\lambda_t = \sqrt{\lambda_L / \lambda_a}$	2.24
Frequency, ω	$\lambda_\omega = \left(\sqrt{\lambda_E/\lambda_\rho}\right)/\lambda_L$	0.447
Elastic Modulus, E	$\lambda_E = ?$	1.0
Force, F	$\lambda_F = \lambda_E\,\lambda_L^2$	25.0
Stiffness, k	$\lambda_k = \lambda_F / \lambda_L$	5.00
Stress, σ	$\lambda_\sigma = \lambda_E$	1.00
Strain, ε	$\lambda_\varepsilon = 1.0$	1.00
Mass Density, ρ	$\lambda_\rho = \lambda_E / (\lambda_L\,\lambda_A)$	0.200
Mass, m	$\lambda_m = \lambda_\rho\,\lambda_L^3$	25.0

TABLE 2
SIMILITUDE SCALE FACTORS FOR ARTIFICIAL MASS SIMULATION

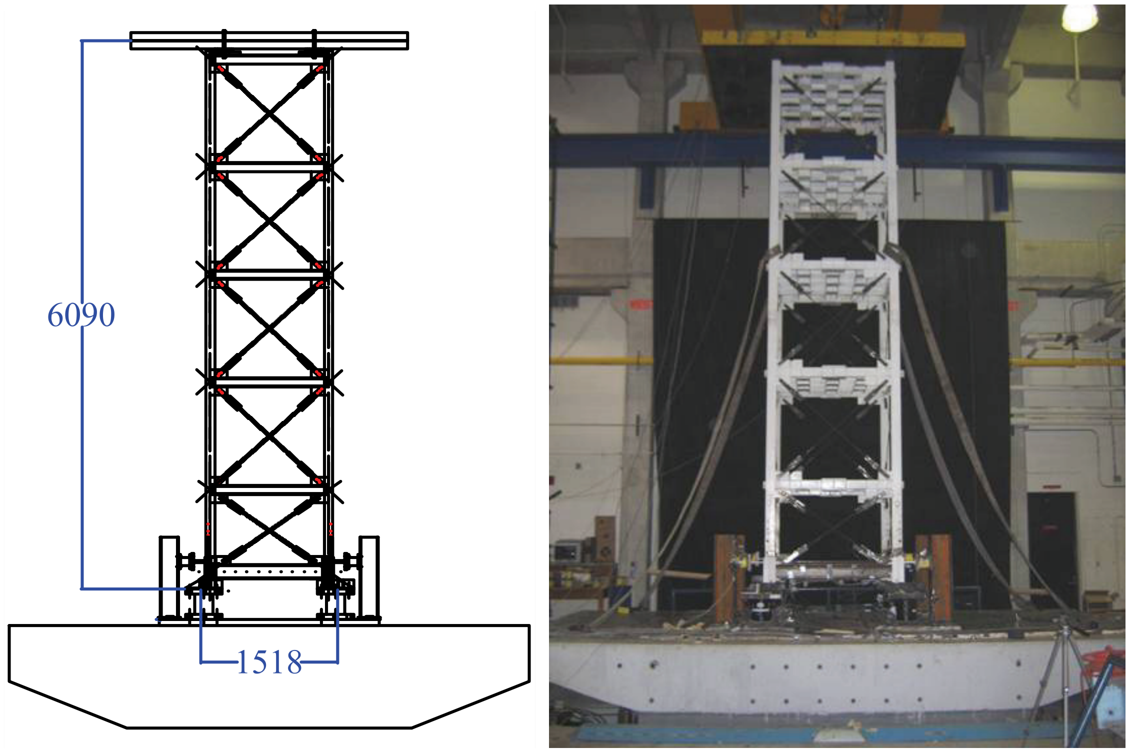


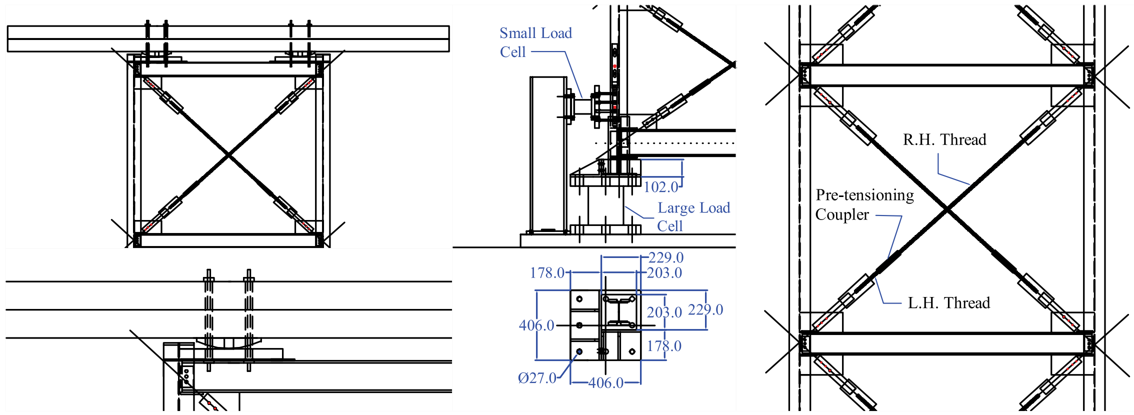
FIGURE 1
EXPERIMENTAL PIER SPECIMEN

SPECIMEN PROPERTIES

To meet similitude and strength requirements, modifications were made to the existing pier specimen. Design of the specimen's structural elements was done using the design methods in Pollino and Bruneau [2006] that considered bi-directional yielding and dynamic amplification of pier forces as a result of impacting and uplift from the structure's supports.

Modifications were made at the base of the specimen to create the boundary conditions to allow the rocking response. Angles surrounded the base plate of each pier leg, as seen in Figure 2(b), that were designed to transfer the horizontal shear forces in bearing however no resistance to vertical uplift was provided through this connection except for friction that may occur along an angle's leg as the pier leg uplifts. The angles were bolted to the top of large load cells that were attached to the shake table. Modifications made to the base of the pier specimen included attaching new base plates, adding column flange cover plates, and a base perimeter beam (collector beam).

The design of diagonal bracing members required special consideration as they were sized to meet similitude and strength requirements. The cross-sectional area of the diagonal members was sized such that the fixed-based period and vertical periods of the specimen were close to that required by similitude (see Table 1). Considering an X-braced configuration, resulted in circular threaded rod diagonal bracing members with a 9.5mm diameter. Such members have essentially no buckling capacity thus could not be



(A) (B) (C)

FIGURE 2
MODIFICATIONS TO EXISTING PIER SPECIMEN (A) MASS CONNECTION, (B) BASE CONNECTION, AND (C) DIAGONAL BRACING MEMBERS

relied on in compression. These members would have likely undergone elastic buckling during testing creating a tension-only bracing system. The prototype was not designed as a tension-only system and it was undesirable to have the dynamic effects, generally associated with such a system, participate in the response. Therefore, all diagonal bracing members were pre-tensioned to a prescribed axial force level such that all members would have a net tension force during testing and not develop compressive forces. The pre-tensioning was achieved by using right and left-handed threaded rod for the bracing members and connecting them with a reverse threaded hex coupler as seen in Figure 2(c). A strain gauge was attached on one face of the hex coupler to measure strain and determine pre-tensioning force during installation and to measure force in the bracing member during testing.

Steel yielding devices with bi-linear hysteretic behavior were designed with connections to only provide a vertical force to the base of the pier legs. The important design quantities for the devices are the plastic device force, elastic stiffness, and maximum allowable vertical displacement. Different steel yielding devices were considered for experimental testing including buckling-restrained braces (AISC 2005) and shear panel devices (Zahrai and Bruneau 1999). However, scaling both the braces and shear panels resulted in devices that were not easily and reliably manufactured or fabricated. The number of design parameters for TADAS devices resulted in dimensions of a device that could be fabricated at this scale. The plastic shear force of the device can be shown to equal:

$$V_p = \frac{Nt^2 bF_y}{4L}\left(=\eta_L \frac{w}{4}\right) \quad (2)$$

where N=number of plates, t=plate thickness, b=plate width at fixed support of device, F_y=yield stress of steel (50ksi, ASTM A572 Gr. 50), and L=length of plates from fixed support to point of loading. Devices were designed with local strength ratios (η_L) of 1.0, 0.67, and 0.33 and to undergo a rotation ($\gamma=\Delta_{up}/L_{TADAS}$) of 0.15rad. during testing. A sketch of a device for each strength ratio is shown in Figure 3.The end connection to the pier leg used ball bearings to ensure that only the vertical shear force would be transferred between the device and pier leg.

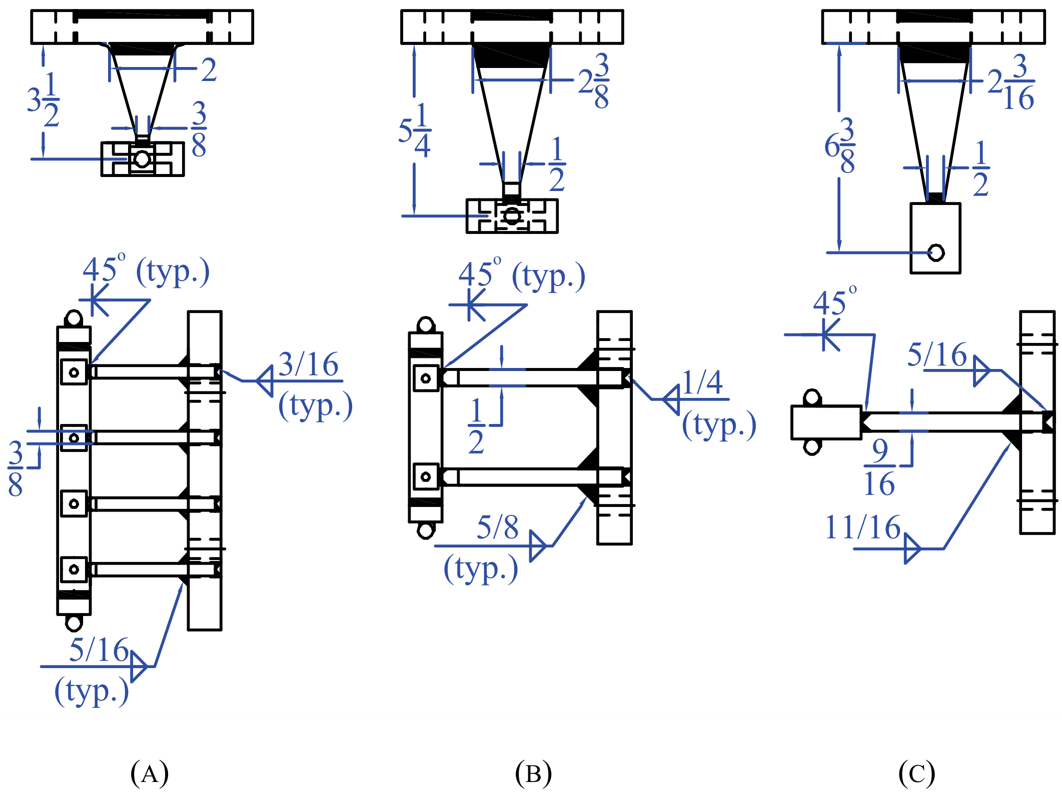


FIGURE 3
TADAS DEVICES (A) η_L =1.0, (B) η_L =0.67, AND (C) η_L=0.33

EXPERIMENTAL FACILITIES AND INSTRUMENTATION

The experimental testing was performed on the 5-DOF shake table in the Structural Engineering and Earthquake Simulation Laboratory at the University at Buffalo. The table has a nominal capacity of 20-tons with an acceleration of 1.15g and 2.30 g in the horizontal and vertical directions respectively. The table is driven by 2 horizontal and 4 vertical hydraulic actuators that are programmable with feedback control to simultaneously control displacement, velocity, and acceleration.

The instrumentation used included accelerometers, string potentiometers, 8 strain gauge based load cells, and strain gauges that were attached within the specimen. A Krypton K600 high performance dynamic mobile coordinate measurement machine that uses 3 cameras and LEDs is used to measure displacements near the base of the structure. The instrumentation layout for the experiments is shown in Figure 4.

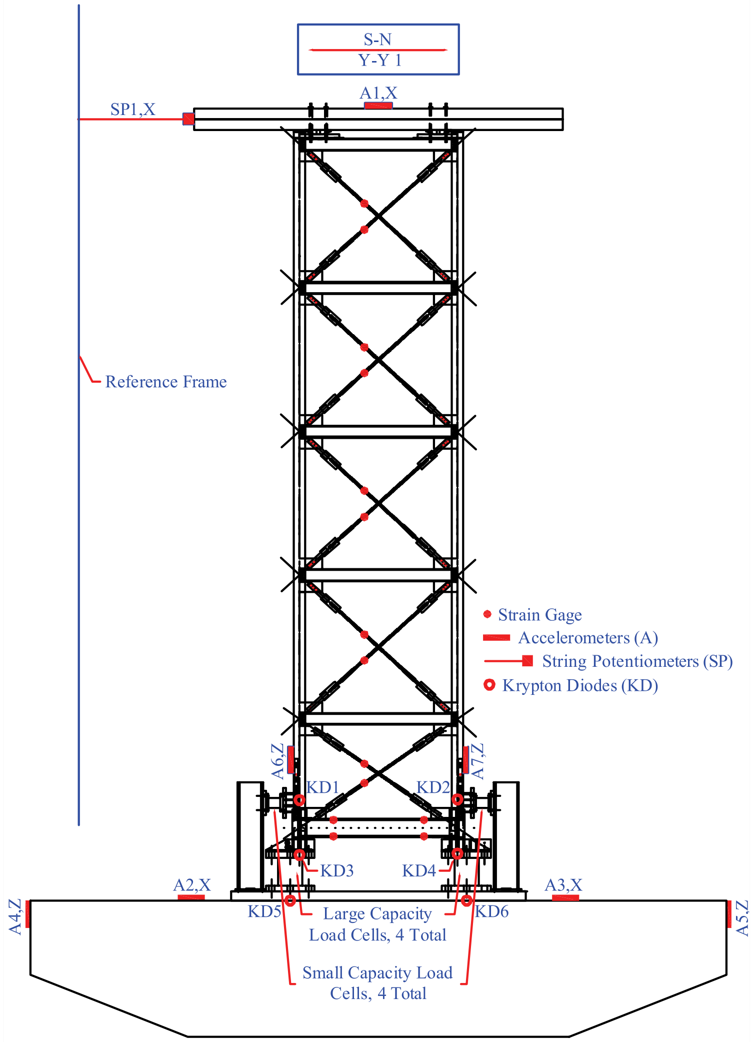


FIGURE 4
INSTRUMENTATION LAYOUT

TABLE INPUT

The input excitation to the shake table included banded white noise excitation and three seismic ground motion histories. The banded white noise tests were performed to identify elastic dynamic characteristics of the model. The white noise excitation had frequency content in the range of 0-50Hz and had a PGA of approximately 0.05g. The seismic input included ground motions from the 1940 El Centro earthquake (Array #9), 1994 Northridge earthquake (Newhall), and a synthetically generated record. The pseudo-acceleration response spectrums from these three motions, in model scale are shown in Figure 5. Under similitude scaling laws, the acceleration of the record is scaled by a factor, λ_a, which is equal to one and the time of the record scaled by the factor, λ_t, which is equal to 2.4 in these tests.

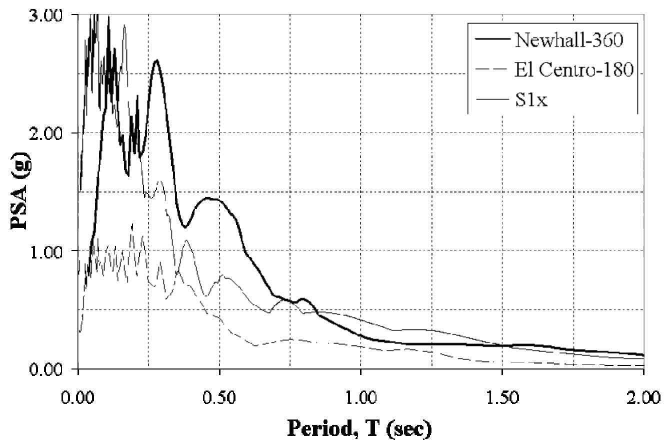


FIGURE 5
TARGET PSEUDO-SPECTRAL ACCELERATION FOR 3 SEISMIC INPUTS

EXPERIMENTAL TESTING RESULTS

A sample set of results are presented here for the set of devices with η_L=0.67, θ=0deg. and subjected to the synthetic record amplitude scaled by 1.5. The pier relative displacement was calculated using the string potentiometers at the top of the pier (total displacement) and the Krypton diodes on the table (table displacement and rotation). The maximum relative displacement was observed to be approximately 100mm however no residual displacement was present at the end of the test. The global hysteretic response of the pier shows the "flag-shaped" behavior and significant fluctuation in the base shear due to the excitation of the vertical "shearing" mode of vibration. Pier leg axial force histories, recorded from large capacity load cells, are shown for the two legs on the South and North end of the specimen. The TADAS hysteretic behavior was recorded from small capacity load cells and relative vertical displacements between Krypton diodes. The device rotation is calculated as the relative vertical deformation across the device divided by the device length. No damage was visually observed within the pier specimen following testing (only inelastic behavior in the ductile "fuses"). Also, white noise testing revealed no change in the specimen's dynamic properties from test to test.

CONCLUSIONS

An innovative approach for the seismic resistance of steel braced frame structures that allows uplift and rocking of braced frame structures at the foundation support has been investigated here through dynamic shake table testing. A 1/5 length scale model of a steel braced highway bridge pier was constructed and tested under strong seismic shaking. Experimental results of the controlled rocking model bridge pier demonstrated stable, elastic behavior of the pier while all damage was forced into the easily replaceable ductile structural "fuses". The simplified methods of analysis used for design was shown to provide conservative estimates of response with reasonable accuracy.

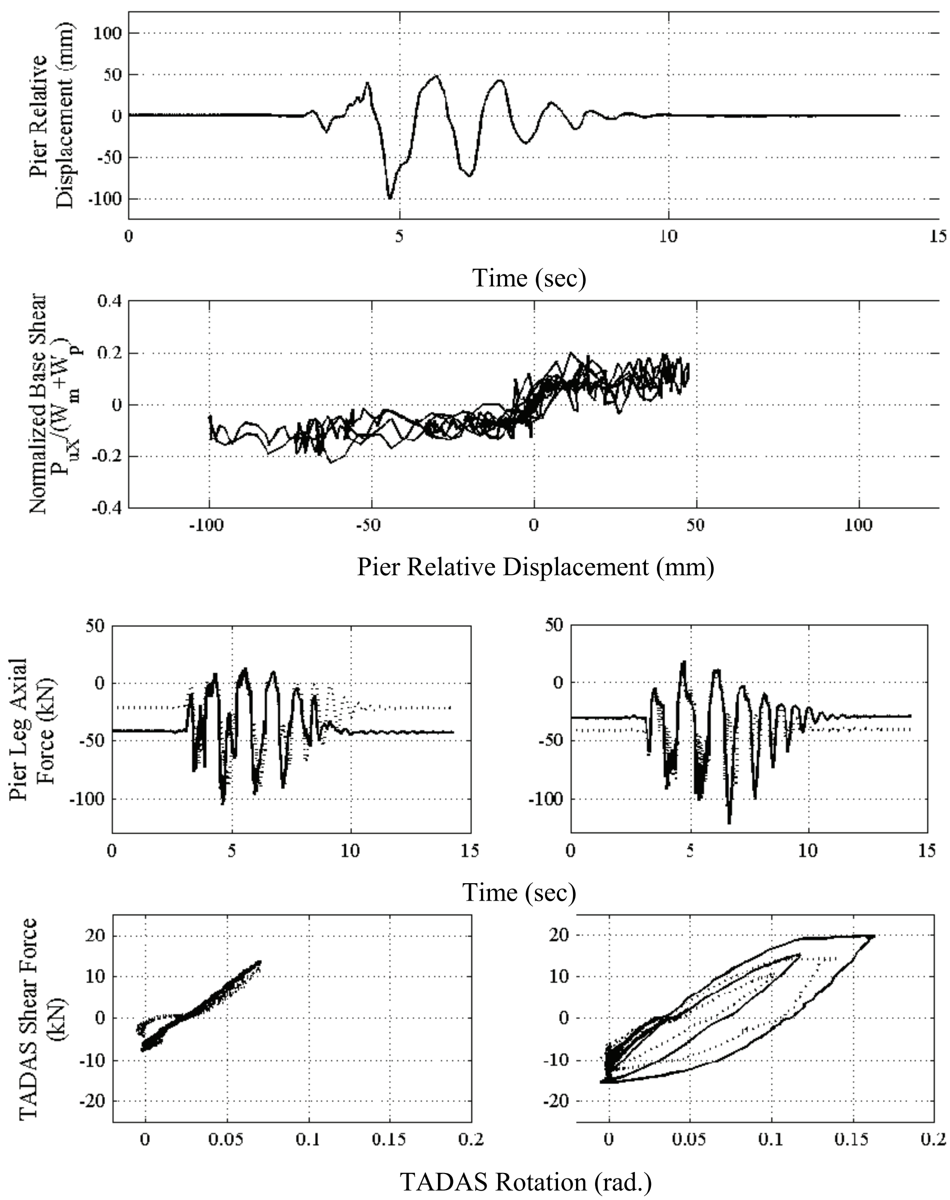


FIGURE 6
EXPERIMENTAL TESTING RESULTS (θ=0, η_L=0.67, SYN-150%)

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