Controlled Rocking Approach for the Seismic Resistance of Structures

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ABSTRACT

An innovative lateral force resisting system is investigated that allows steel braced frames to uplift and rock on their foundation, partially isolating the frame. Passive energy dissipation devices are implemented at the uplifting location to control the rocking response to within allowable limits. The structure's gravitational forces provide an inherent restoring force during the rocking response that can allow the system to self-center after the excitation ceases. This approach to seismic resistance is investigated here for the seismic retrofit of bridge steel truss piers.

Specially detailed, steel yielding devices (buckling-restrained braces) are used as the passive energy dissipation devices in this application. The devices likely require no maintenance and act as ductile structural "fuses" during a seismic event. The static, hysteretic behavior of a steel truss pier is developed first and key design variables identified. The dynamic characteristics of the controlled rocking/energy dissipation system are considered in formulating a capacity design procedure using simplified methods of analysis. An example is provided for the design of a single bridge pier for a specified seismic demand and set of capacity protection limits. An analytical model is developed and nonlinear, inelastic time history analysis is used to verify that the design objectives are met.

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INTRODUCTION

Recent earthquakes, such as the 1989 Loma Prieta, 1994 Northridge, and 1995 Kobe earthquake in Japan have demonstrated the need for improved methods for the design and construction of highway bridges to withstand seismic force and displacement demands. While collapse is rare, undesirable damage can leave the bridge unusable until repairs can be made. Highway bridges deemed critical in the response and recovery efforts following a major earthquake need to remain operational after an earthquake requiring the bridge to respond in a mostly elastic manner with little to no residual displacements.

Many existing steel truss bridges consist of riveted construction with built-up, lattice type members supporting a slab-on-girder bridge deck. Truss piers are typically in an x- or v-braced configuration. Steel truss bridges are found in nearly every region of the U.S.

These built-up lattice type members and their connections can be the weak link in the seismic load path. Recent experimental testing of these members revealed that they suffer global and local buckling causing significant member strength and stiffness degradation resulting in loss of pier lateral strength and major structural damage during an earthquake (Lee and Bruneau, 2003). Existing, riveted connections and deck diaphragm bracing members typically possess little to no ductility (Ritchie et. al., 1999). Another possible non-ductile failure location is the anchorage connection at the pier-to-foundation interface. Analysis of "typical" steel-concrete connections suggests it may be unable to resist even moderate seismic demands.

While strengthening these existing, vulnerable elements to resist seismic demands elastically is an option, this method can be expensive and also gives no assurance of performance beyond the elastic limit. Therefore it is desirable to have structures able to deform inelastically, limiting damage to easily replaceable, ductile structural "fuses" able to produce stable hysteretic behavior while protecting existing non-ductile elements and preventing residual deformations using a capacity-based design procedure.

Failure of or releasing the anchorage connection allows a steel truss pier to step back-and-forth or rock on its foundation, partially isolating the pier. Addition of passive energy dissipation devices at the uplifting location can control the rocking response while providing energy dissipation. An inherent restoring force is provided by gravity, allowing the rocking system to recover plastic rotational deformations at the base and possible re-centering of the pier. The device used in this application is the buckling-restrained brace. A buckling-restrained brace consists of a steel core surrounded by a restraining part, allowing the brace to reach full yield in tension and compression. Experimental testing of the braces can be found in Iwata et. al. (2000). Also, this strategy limits the retrofit effort by working at a fairly accessible location. A sketch of a retrofitted bridge pier is shown in Figure 1.

The dynamic characteristics of the above proposed rocking/energy dissipation system are investigated to develop a design procedure that can size the passive energy dissipation elements while capacity protecting the existing pier. Nonlinear time history analyses are used to assess the seismic behavior.

HYSTERETIC BEHAVIOR OF CONTROLLED ROCKING SYSTEM

The controlled rocking bridge pier system considered can be shown to develop a flag-shaped hysteresis. This is due to the combination of pure rocking response from the restoring moment

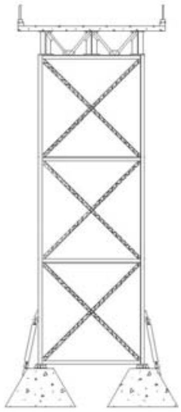

Figure 1. Retrofitted truss pier.

provided by the bridge deck weight and energy dissipation provided by yielding of the buckling-restrained braces. The key parameters for the hysteretic response of the rocking bridge pier system considered here include the fixed-base lateral stiffness of the existing steel truss pier, the aspect ratio of the pier (h/d) and the cross-sectional area (A_{ub}), effective length (L_{ub}) and yield stress of the buckling-restrained brace (F_{yub}). Also, the weight excited by horizontally imposed accelerations (W_h) and the vertical gravity weight carried by a pier (W_v) are assumed equal here and expressed as W. The various steps and physical behaviors that develop through a typical half-cycle are shown qualitatively in Figure 2 along with the corresponding actions of the buckling-restrained brace during the controlled rocking response. By symmetry, the behavior repeats itself for movement in the other direction.

Transition from 1st to 2nd cycle response occurs when the buckling-restrained braces yield in compression and the braces carry a portion of the weight after the system comes to rest upon completion of the cycle. Ignoring strain hardening in the brace and any difference in compressive and tensile yield strengths of the brace, a displacement of $2\Delta_{yub}$ within the brace will result in the brace carrying an amount of the gravity load equal to the buckling-restrained brace strength. This transfer

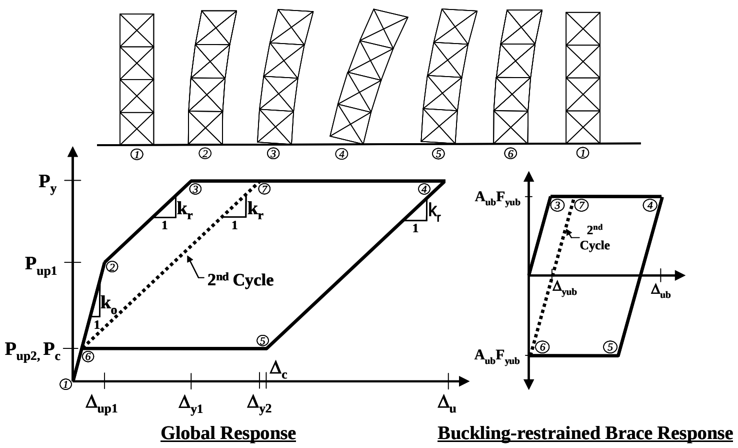

Figure 2. Hysteretic behavior of controlled rocking system.

of gravity load from the pier leg to the buckling-restrained brace reduces the base shear, P, required to initiate uplift ($P_{up2}<P_{up1}$, Figure 2) resulting in a more flexible system due to the earlier transition to the smaller lateral stiffness (k_r).

PROPOSED CAPACITY BASED DESIGN PROCEDURE

In the perspective of seismic retrofit, a capacity based design procedure is proposed here to protect non-ductile elements while dissipating energy in the specially detailed steel yielding devices. A large number of constraints exist and thus a systematic design procedure that attempts to obey all constraints is desirable.

Deck-level Displacement

To the writer's knowledge, there exists no solidly established rule of determining maximum allowable displacements for bridges. There likely exist structural elements for which deformations must be limited to prevent their damage or damage of their connections. Such deformation limits vary from bridge to bridge. Here, the deformation limits considered are those that attempt to prevent P-Δ effects from affecting the seismic behavior and a limit based on overturning stability. The smaller of these two limits is used.

A requirement shown to be adequate to prevent excessive P-Δ effects can be found in the NCHRP 12-49 (ATC/MCEER, 2004) document. This limit is:

$$\Delta_{u1} \leq 0.25 \frac{V}{W} h \tag{1}$$

where V is the lateral strength of the pier.

Another limit is set based on preventing displacement of the center of mass beyond half of the base width (d/2), with a large factor of safety since this is the point of overturning. This limit is defined by:

$$\Delta_{u2} \leq \frac{d}{2FS} \tag{2}$$

A factor of safety (FS) of 5 is recommended.

Ductility Demand on Buckling-restrained Brace

Limits on the inelastic strain demands are set in order to ensure that the brace behaves in a stable, predictable manner. These limits should be based on engineering judgment and experimental test data. A strain of 1.5% has been selected for a "maximum considered" type earthquake, as appropriate for buckling-restrained braces based on many reported experimental results including Iwata et. al. (2000). Assuming a rigid foundation, the brace elongation is equal to the uplifting displacement and related to the deck-level displacement (Δ_u) beyond yield such that this constraint is defined by:

$$\Delta_{ub} = \Delta_{uplift} = \left[\Delta_u - \frac{\left(W/2 + A_{ub} F_{yub} \right) d/h}{k_o} \right] \cdot \frac{d}{h} < 0.015 \cdot L_{ub} \tag{3}$$

Re-centering of Pier

By limiting the buckling-restrained brace yield strength to the pier leg's tributary gravity load, the plastic deformations accommodated at the base can be returned to the undeformed positions. Assuming a symmetrical two-legged pier, the limit to allow for pier re-centering is W/2. A local strength ratio, η_L, is defined here as:

$$\eta_L = \frac{A_{ub} F_{yub}}{W/2} \tag{4}$$

Thus $\eta_L<1$ allows for pier re-centering. This ratio is also a measure of the system's energy dissipation per cycle.

Forces to Existing Members and Connections

Capacity design procedures and conservative assessment of maximum force demands are needed to ensure that non-ductile elements can remain elastic and that all inelastic action occurs in the specially detailed ductile structural elements. Strength of existing bridge elements will vary from bridge-to-bridge and partial strengthening may be required in some cases.

After a pier leg uplifts from the foundation, it eventually returns to the foundation with a velocity upon impact. As the rocking motion continues, the structures weight and the device forces are transferred to the compressive side as it becomes the new axis of rotation. The hysteretic behavior, shown in Figure 2, was determined assuming static response of the controlled rocking pier. However, the dynamic response is quite different due to the excitation of vertical modes of vibration with the maximum horizontal base shear significantly exceeding the yield force, P_y.

A method is proposed here that creates an “effective” static base shear that can be used to evaluate the adequacy of the pier’s lateral bracing members followed by a method to determine the ultimate demands placed on the pier legs and foundation. For each case, dynamic amplification factors are introduced as a result of the excitation of the pier’s vertical modes of vibration during impact (R_{dL}) to and uplift (R_{dv}) from the foundation. The dynamic amplification factors depend on the impulsive nature of the transfer of loads during pier rocking and a procedure to calculate these factors is presented in Pollino and Bruneau (2004).

Effective Lateral Base Shear Demand

The effective base shear, P_u, caused by the earthquake is determined by the static yield force, P_y, amplified to account for the increased demand caused by dynamic effects as a result of uplift from the foundation and is taken here as:

$$P_u = P_y \cdot R_{dv} = \left(w/2 + A_{ub} F_{yub} \right) d/h \cdot R_{dv} < P_{u,allow} \tag{5}$$

where $P_{u,allow}$= maximum allowable shear that can be applied to the pier.

Pier Leg Demands

The demands to an impacting pier leg include a dynamic effect related to the velocity upon impact (v_o) followed by the transfer of gravitational and device forces vertically through the truss pier to the compressed leg. The total force developed in the pier leg is determined here to be:

$$P_{uL} = v_o\sqrt{\frac{mk_L}{2}} + R_{dL}\ {}^{w}\!/_{2} + \left({}^{w}\!/_{2} + A_{ub}F_{yub}\right)\cdot R_{dv}\cdot\left(1-{}^{d}\!/_{2h}\right) < P_{uL,allow} \tag{6}$$

where $P_{uL,allow}$=maximum allowable force, controlled by either the strength of the pier leg or foundation and k_L=axial stiffness of a pier leg. Other limits may need to be defined to prevent foundation settlement and/or other serviceability requirements.

Assuming that a value of A_{ub} is established during the design process to satisfy the effective lateral base shear limit (Eq. 5) or the self-centering limit (Eq. 4), the limiting impact velocity can be written as:

$$v_{o,max} = \frac{P_{uL,allow} - R_{dL}\cdot{}^{w}\!/_{2} - \left({}^{w}\!/_{2} + A_{ub}F_{yub}\right){}^{d}\!/_{h}\cdot R_{dv}\cdot\left(1-{}^{d}\!/_{2h}\right)}{\sqrt{{}^{m\cdot k_L}\!/_{2}}} \tag{7}$$

EXAMPLE

A design example is presented, to illustrate the design procedure proposed above. The seismic demand is obtained from a design spectrum specified in ATC/MCEER (2004) with one-second (S_1) and short period (S_s) spectral acceleration values of 0.5 and 1.25g respectively corresponding to 5% damping. The bridge is assumed to be located on site class B and the site coefficients, F_a and F_v are equal to 1. This leads to a characteristic spectral period, T_s, of 0.4sec, typical of a rock site.

The example uses a pier with an aspect ratio of 4 (h=29.26m, d=7.32m) with horizontal and vertical tributary weights (W_h and W_v) equal to 1730kN. The pier lateral stiffness (k_o) is taken as 12.6kN/mm and thus the "fixed-base" period of vibration is equal to 0.74sec. Also, the axial stiffness of a pier leg is taken to be 212kN/mm and the lateral strength of the pier ($P_{u,allow}$) and capacity of a pier leg ($P_{uL,allow}$) is taken as 605kN and 3980kN respectively. The buckling-restrained braces are assumed to be implemented vertically and have a steel core with a yield stress (F_{yub}) of 235MPa (LYP 235, Nakashima 1995).

The dynamic amplification factors, to account for the excitation of vertical modes during impact and uplift of the rocking response (R_{dL} and R_{dv}), are taken as 1.87 and 1.56 respectively. The methods proposed to determine the amplification factors (Pollino and Bruneau 2004) depend on many system parameters; yet these factors remain relatively constant for a single pier aspect ratio, existing pier properties, and a given seismic demand.

The buckling-restrained brace area is initially sized to satisfy the effective lateral base shear limit Eq. 5, resulting in a value of 2000mm^2. The buckling-restrained brace effective length (L_{ub}) is initially taken as 1900mm. Initial design constraints are then calculated (design iteration 0); results are shown in Table 1.

Now, the capacity spectrum method of analysis is used to predict the maximum system displacement and the uplifting displacement. As discussed in Pollino and Bruneau (2004), this method uses spectral capacity (pushover) and demand curves to predict system response. The predicted system response for each design iteration is shown graphically in Figure 3. The initial estimate of maximum system displacement is shown in the figure as Δ_o. The first iteration, based on the initial brace dimensions, results in a maximum system displacement of 158mm and an uplifting displacement of 32.9mm. The impact velocity is predicted from the pseudo-velocity of the system.

Table 1. Controlled rocking system design example.

Design Iteration	**0**	**1**	**2**		**Units**
F_{yub}	235	235	235	-	MPa
A_{ub}	2000	2000	1500	-	mm^2
L_{ub}	1900	1900	2750	-	mm
η_L	0.54	0.54	0.41	-	-
Constraints					
Δ_{u1} (Eq. 1)	914	914	914	-	mm
Δ_{u2} (Eq. 2)	732	732	732	-	mm
Δ_{uplift} (Eq. 3)	28.5	28.5	41.3	-	mm
$P_{u,allow}$ (Eq. 5)	605	605	605	-	kN
$v_{o,max}$ (Eq. 7)	118	118	156	-	mm/sec
η_L (Eq. 4)	1.0	1.0	1.0	-	-
	------------------*Design Response Values*------------------			*TH Analysis*	
Δ_u (Fig. 3)	138	158	188	187	mm
Δ_{uplift} (Eq. 3)	28.4	32.9	41.0	40.1	mm
v_o	-	137	143	141	mm/sec
P_u (Eq. 5)	-	-	487	379	kN
$P_{u,L}$ (Eq. 6)	-	-	3920	2262	kN

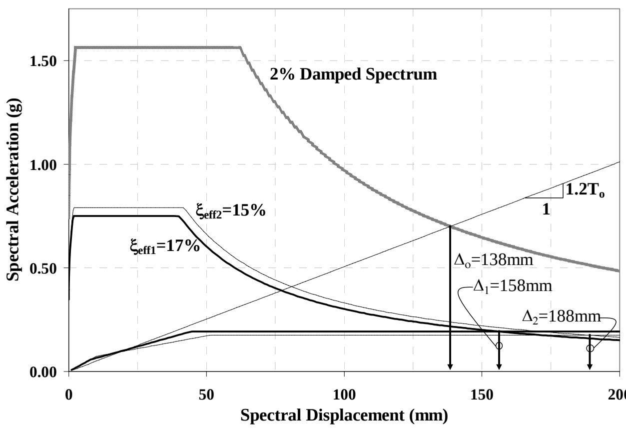

Figure 3. Example capacity spectrum design iterations.

As seen in Table 1, the values of uplifting displacement and impact velocity do not satisfy their respective constraints.

To satisfy these two constraints, L_{ub} is increased to satisfy the brace ductility demand limit and A_{ub} decreased to satisfy the pier leg demands limit. For the second iteration, the buckling-restrained brace cross-sectional area (A_{ub}) is taken as 1500mm^2 and effective length (L_{ub}) as 2750mm. The constraints are re-calculated along with each response prediction (listed in Table 1) and all design values are found to satisfy the specified design constraints.

Example Time History Analysis

To illustrate the effectiveness of the proposed design procedure, a nonlinear inelastic time history analysis is performed to calculate the "actual" response of the retrofitted system.

For the proposed controlled rocking bridge pier system considered here, the piers are excited solely in the horizontal direction. Each pier is assumed to carry an equal inertia mass both vertically and horizontally. Thus, each pier is assumed to carry an equal length of bridge deck and interaction between adjacent piers is assumed negligible. A 2-D model of each representative truss pier is used with half of the mass applied to each of the top two nodes of the truss. The pier itself is modeled with its elastic properties and all nonlinear action is modeled to occur at the foundation interface. A compression-only, "gap" element and a displacement based hysteretic element are placed in parallel across the anchorage interface, at the base of each tower leg, to model the rocking behavior. The "gap" elements represent the foundation with no tensile capacity and a large linear-elastic stiffness in compression. The hysteretic element is based on the model proposed by Wen (1976). Restraints are provided at the anchorage level that prevent movement in the horizontal direction (thus assuming that sliding is prevented) but provide no resistance to vertical movements.

A spectra compatible ground acceleration time history, used for the dynamic analysis, is generated using the Target Acceleration Spectra Compatible Time Histories (TARSCTHS) software developed by the Engineering Seismology Laboratory (ESL) at the University at Buffalo and is the implementation of the method described in Deodatis (1996). The synthetic ground motion was generated by TARSCTHS matching the design elastic response spectrum considered and had a duration of 15 sec.

Results of the TH analysis are given in Table 1, confirming reliability of the above response results and that all design constraints are satisfied. Results for selected response quantities are given in Figures 4-6 and the resulting pier hysteretic behavior is given in Figure 7. Fluctuation of the base shear shown in Figure 7, compared to the idealized flag-shaped hysteretic behavior of the controlled rocking system, is due to the excitation of vertical modes of vibration of the truss during pier uplift and is accounted for by the dynamic amplification factor, R_{dv}, as discussed previously. The deck-level displacement results (Figure 4) show the system returning to its original undeformed position at the end of the excitation due to the self-centering ability of the system.

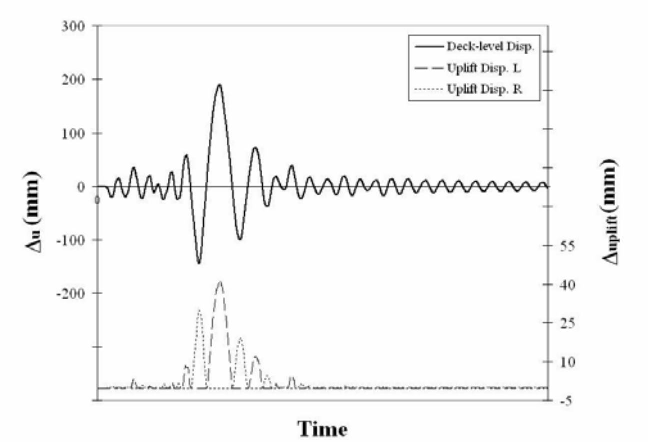

Figure 4. Deck-level and uplifting displacement results of example TH analysis.

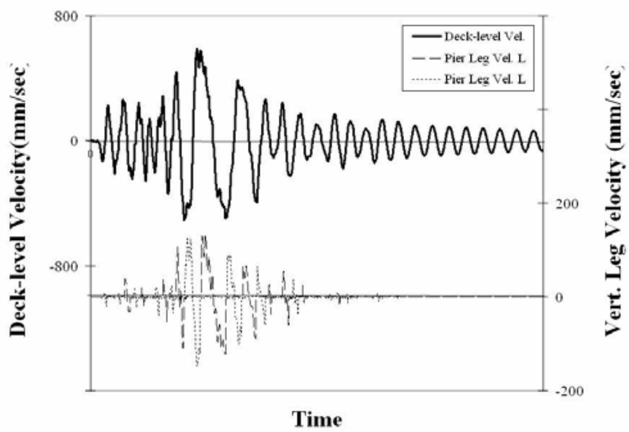

Figure 5. Deck-level and vertical leg velocity results of example TH analysis.

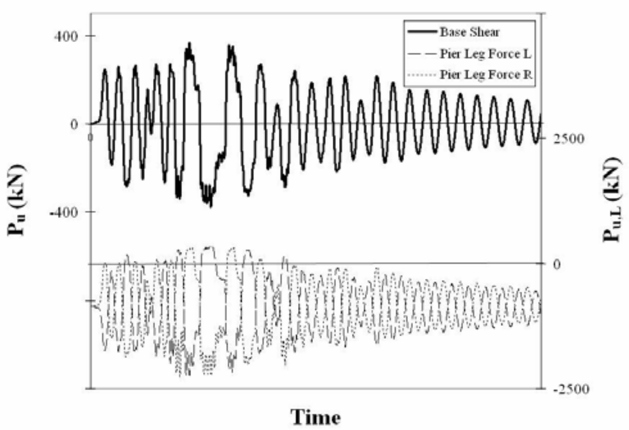

Figure 6. Base shear and leg force results of example TH analysis.

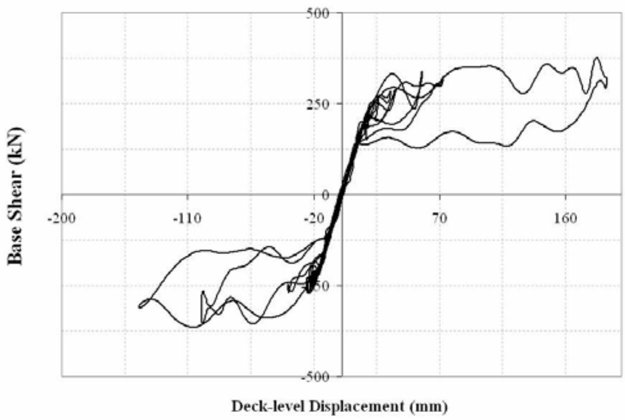

Figure 7. Pier hysteretic behavior during example TH analysis.

CONCLUSIONS

A new retrofit strategy relying on controlled rocking has been proposed to achieve ductile seismic performance of steel truss bridge piers. Buckling-restrained braces are used to provide energy dissipation to the system while limiting the base overturning moment. This retrofit strategy allows the existing pier and superstructure to remain elastic, and provide self-re-centering of the structure following earthquakes, providing a higher level of performance during earthquake motions and increasing the probability that the bridge will remain operational for response and recovery efforts following an earthquake.

A capacity-based design procedure for this seismic retrofit technique is also proposed. A set of design constraints are established to achieve ductile seismic performance. These constraints include pier drift, ductility demands on the buckling-restrained brace, self-centering, and maximum developed force limits. Methods to determine design response values (displacements, velocity, forces) are also presented.

A systematic design procedure is established and a design example is provided to show the key steps in the design procedure. A series of iterations are performed and a set of buckling-restrained brace properties selected to satisfy all design constraints. Time history analysis is then performed, using the example's pier properties and the final selected brace dimensions, to verify the response satisfied all design constraints.

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REFERENCES

ATC/MCEER (2003), NCHRP 12-49 Recommended LRFD Guidelines for the Seismic Design of Highway Bridges, Part I: Specifications, ATC/MCEER Joint Venture.

Deodatis, G. (1996). Non-stationary stochastic vector processes: seismic ground motion applications, *Prob. Eng. Mech.* 11, 149-167.

Iwata, M., Kato, T., Wada, A., (2000) "Buckling-restrained braces as hysteretic dampers" Behaviour of Steel Structures in Seismic Areas: STESSA 2000; Montreal, Canada, August 21-24, 2000, pp. 33-38.

Lee, K. and Bruneau, M., (2003) "Review of Energy Dissipation of Compression Members in Concentrically Braced Frames", Technical Report MCEER-02-0005, Multidisciplinary Center for Earthquake Engineering Research, State University of New York at Buffalo, Buffalo, NY.

Nakashima, M. (1995). "Strain-Hardening Behavior of Shear Panel Made of Low-Yield Steel. I: Test." *J. Struct. Engineering*, ASCE, 121(12), 1742-1749.

Pollino, M. and Bruneau, M. (2004). *Seismic Retrofit of Bridge Steel Truss Piers Using a Controlled Rocking Approach. Technical Report MCEER-04-0011, Multidisciplinary Center for Earthquake Engineering Research,* The State University of New York at Buffalo, Buffalo, NY.

Ritchie, P., Kauhl, N. and Kulicki, J. (1999) "Critical Seismic Issues for Existing Steel Bridges", Technical Report MCEER-99-0013, Multidisciplinary Center for Earthquake Engineering Research, State University of New York at Buffalo, Buffalo, NY.

Wen, Y. K. (1976). "Method for Random Vibration of Hysteretic Systems." *Journal of the Engineering Mechanics Division*, ASCE, Vol. 102, No. EM2.