Controlled Rocking System for Seismic Retrofit of Steel Truss Bridge Piers

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ABSTRACT

In assessments of the seismic adequacy of existing steel bridges, the steel-to-concrete anchorage connections typically found at the base of steel truss piers can be potentially vulnerable, having little to no ductility and inadequate strength to resist seismic demands. Many other non-ductile failure locations may also exist along the seismic load path. Failure would result in unacceptable performance, especially for bridges deemed critical for response and recovery efforts following an earthquake.

While strengthening is an option, this approach may only transfer damage to another location. An alternative solution could be to release the anchorage connection, allowing development of a rocking bridge pier system that can partially isolate the structure. An improvement on this approach, and the retrofit solution proposed here, allows this rocking mechanism to develop, but complements it by adding passive energy dissipation devices across the anchorage interface to control the rocking response. These specially detailed hysteretic energy dissipating elements (unbonded braces) act as easily replaceable, ductile structural "fuses". This system also provides an inherent restoring force capability that allows for automatic re-centering of the tower, leaving the bridge with no residual displacements after an earthquake.

A similar controlled rocking approach to seismic resistance was implemented into the design of the South Rangitikei Rail Bridge, located in Mangaweka, New Zealand (Priestley et. al. 1996) and was used as a seismic retrofit technique in the Lions' Gate Bridge located in Vancouver, British Colombia (Dowdell & Hamersley 2001). Both bridges use steel yielding devices across the anchorage interface. The Lions' Gate Bridge retrofit utilized time history analysis to size the device and assess the adequacy of existing elements.

This paper investigates the dynamic characteristics of the above proposed controlled rocking/energy dissipation system in order to formulate a capacity design procedure that can reliably predict response using simplified methods of analysis. Design constraints are established that attempt to satisfy performance objectives. Nonlinear time history analyses are used to assess the seismic behavior of the bridges retrofitted per this strategy.

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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. A typical steel truss bridge with this type of construction is shown in Figure 1.

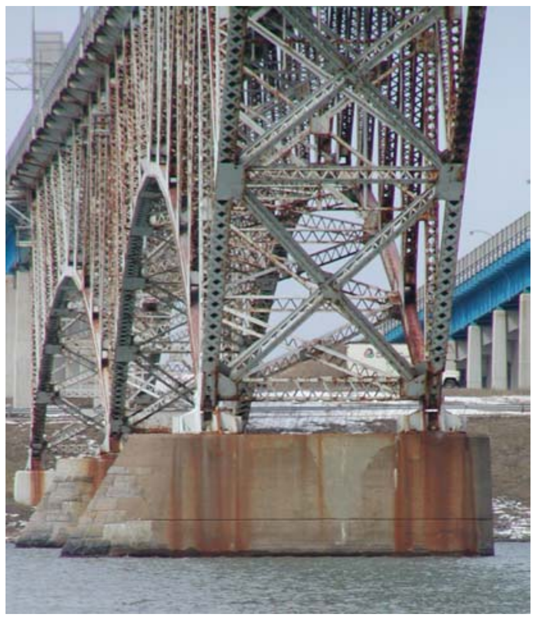

Figure 1. Typical Steel Truss Pier

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 & 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 unbonded brace. An unbonded 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 & Kato (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 2.

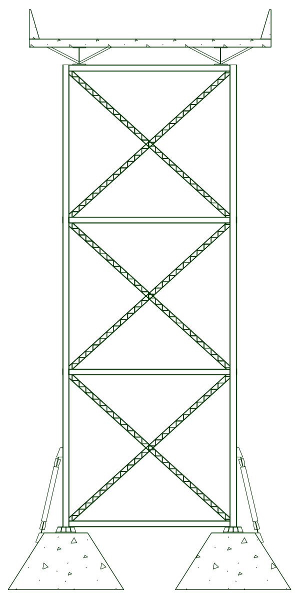

Figure 2. Sketch of Retrofitted Bridge Pier

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. Simple, existing methods of analysis are evaluated for predicting response of this type of system and nonlinear time history analyses are used to assess the seismic behavior.

CONTROLLED ROCKING SYSTEM FOR SEISMIC RETROFIT

Existing Rocking Bridges

A controlled rocking approach to seismic resistance was implemented into the design of the South Rangitikei Rail Bridge (shown in Figure 3), Mangaweka, New Zealand in the early 1970's (Priestley et. al. 1996) and was later used as a seismic retrofit technique in the Lions' Gate Bridge located in Vancouver, British Colombia (Dowdell & Hamersley 2001) as shown in Figure 4. The South

Rangitikei Rail Bridge used torsion yielding, mild steel bars as the energy dissipation device implemented at the anchorage interface while the Lion's Gate Bridge used a triangular shaped flexural yielding device at the anchorage interface.

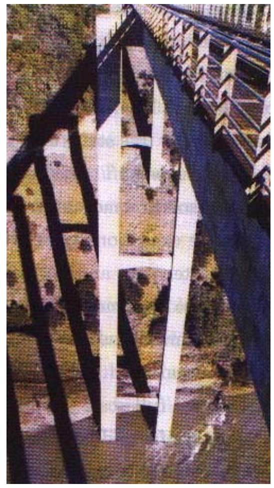

Figure 3. South Rangitikei Rail Bridge (courtesy of Ian Buckle)

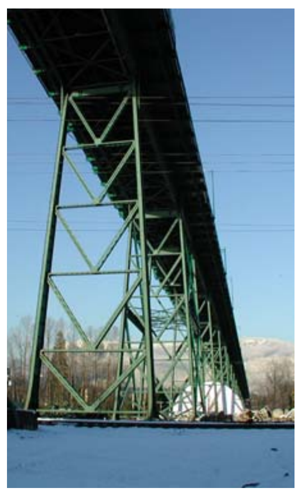

Figure 4. Lions' Gate Bridge, North Approach (courtesy of Bruce Hamersley)

Hysteretic Behavior

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 provided by the bridge deck weight and energy dissipation provided by yielding of the unbonded 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 (k_o), the aspect ratio of the pier (h/d) and the cross-sectional area (A_{ub}), effective length (L_{ub}) and yield strength of the unbonded 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 of the controlled rocking response, along with the corresponding actions of the unbonded brace are shown in Figure 3. By symmetry, the behavior repeats itself for movement in the other direction.

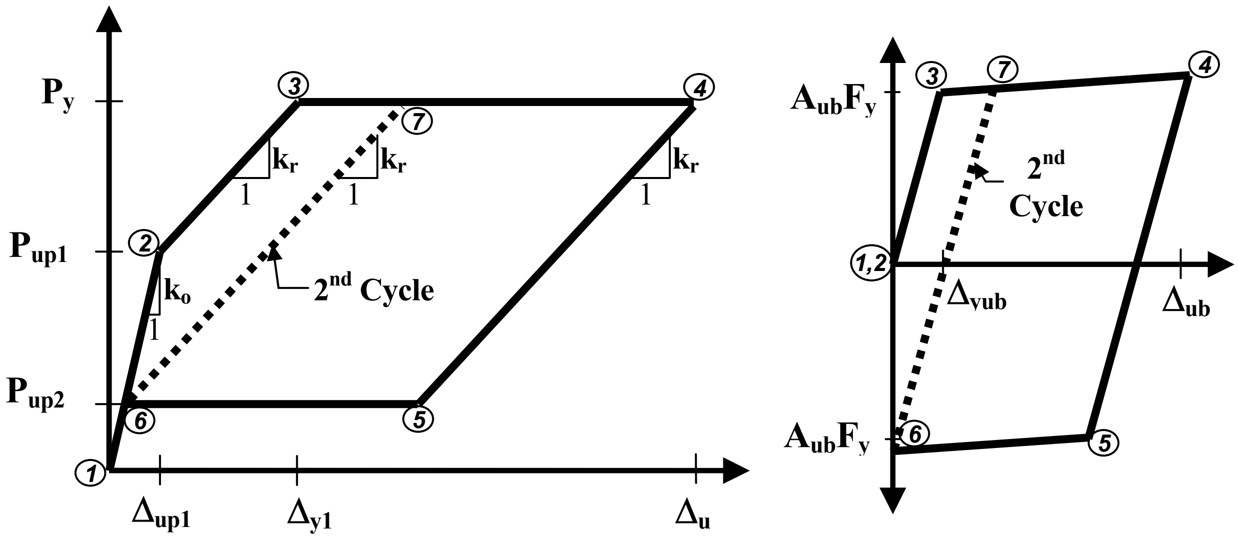

Figure 3. (a) Global Hysteretic Behavior, (b) Response of Unbonded Brace

Transition from 1st to 2nd cycle response occurs when the unbonded 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 unbonded brace strength. This transfer of gravity load from the pier leg to the unbonded brace reduces the base shear, P, required to initiate uplift ($P_{up2}<P_{up1}$, Figure 3.(a)) resulting in a more flexible system due to the earlier transition to the smaller lateral stiffness k_r.

Re-centering of Pier

By limiting the unbonded 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:

$$\eta_L = \frac{A_{ub}F_{yub}}{W/2} \tag{1}$$

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

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 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. The proposed design procedure uses a graphical approach to size the two key design parameters, the effective length and cross-sectional area of the unbonded brace, L_{ub} and A_{ub} respectively.

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 2003) document. This limit is:

$$\Delta_G \le 0.25 \frac{V}{W} h \tag{2}$$

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:

$$\Delta_G \le \frac{d}{2FS} \tag{3}$$

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

Ductility Demand on Unbonded 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. Experimental test data of the inelastic cyclic response of an unbonded brace, adapted from Iwata & Kato, is shown in Figure 4. A strain of 1.5% has been selected for a "maximum considered" type earthquake, as appropriate for unbonded braces based on many reported experimental results. Therefore this constraint can be established in terms of brace elongation by:

$$\Delta_{ub} \le 0.015 L_{ub} \tag{4}$$

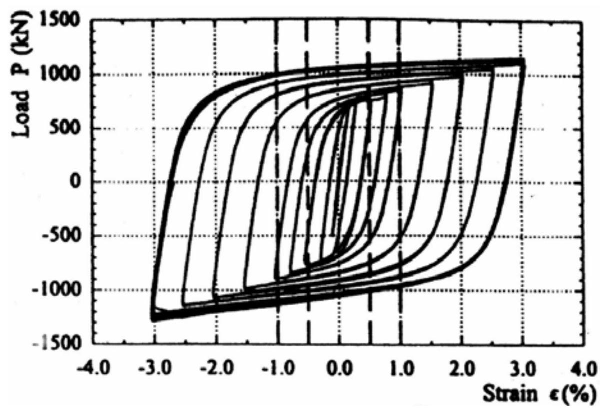

Figure 4. Experimental Test Results (adapted from Iwata and Kato 2000)

Forces to Existing Members and Connections

Capacity design procedures are used to conservatively predict the maximum force demand such that the non-ductile elements can remain elastic, forcing all inelastic action to the specially detailed, ductile structural elements.

After a tower leg uplifts from the foundation it eventually returns to the foundation with a velocity upon impact. Assuming that the maximum velocity of the bridge deck to be equal to the inelastic pseudo spectral velocity and the maximum to occur the moment before impact, the impact velocity can be taken for design purposes as:

$$v_d = PS_{vi}\left(\frac{d}{h}\right) \tag{5}$$

where the inelastic pseudo-spectral velocity, PS_{vi}, can be determined using a ductility reduction strategy.

As the rocking motion continues the structures weight and other loads are transferred to the compressive side as it becomes the new axis of rotation. The hysteretic behavior shown in Figure 3 was determined assuming static response of the controlled rocking pier. However, analysis has revealed that the dynamic response is quite different with the maximum horizontal base shear significantly exceeding the yield force, P_y. Vertical modes of vibration of the controlled rocking system are excited even when subjected solely to horizontal excitation. A sample response of the dynamic hysteresis is shown in Figure 5.

Dynamic effects occur as a result of impact to and uplift from the foundation. Two amplification factors are assigned to the static forces such that members can be capacity protected. The demands placed on a pier leg and a general foundation element, resulting from the impact velocity and dynamic loads, are determined using the conservation of energy. Modification of the key design parameters, A_{ub} and L_{ub}, to limit forces to an acceptable level or strengthening of the non-ductile elements along the lateral load path can satisfy this constraint.

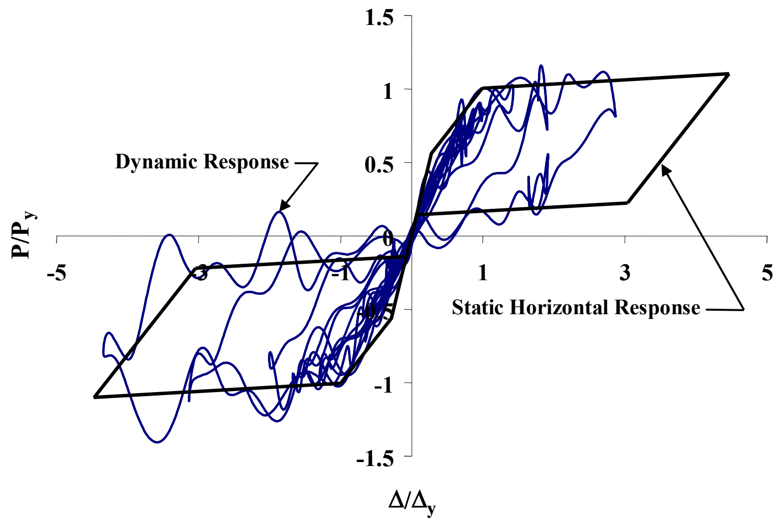

Figure 5. Dynamic Hysteretic Behavior

With appropriate constraints assigned for a particular bridge, the key design parameters (A_{ub} and L_{ub}) can be selected using an iterative analysis and design procedure or a graphical approach, similar to that proposed by Sarraf and Bruneau (1998) could be used to determine the range of A_{ub} and L_{ub} which satisfy the constraints. The procedure determines boundaries of compliance and non-compliance of the design constraints with respect to the key design parameters. A sample graphical design plot is shown in Figure 6.

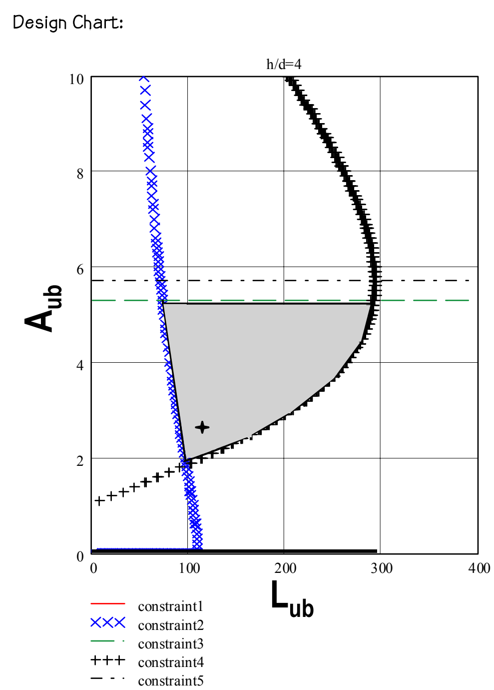

Figure 6. Graphical Design Plot

Simplified Methods of Analysis

To determine the seismically imposed demands to a self-centering, flag-shaped hysteretic system, a number of simple methods of analysis were considered to verify their accuracy in predicting response for design purposes. A first method of analysis considered to characterize system response is similar to the nonlinear static procedure (NSP) in FEMA 356 (FEMA 2000) while a second is similar to the nonlinear static procedure for passive energy dissipation systems found in FEMA 274 (FEMA 1997). An analysis procedure similar to the second one can be found in the NCHRP 12-49 document (ATC/MCEER 2003).

The NSP uses the unbonded brace's stiffness properties to determine the retrofitted effective system stiffness and then calculates a displacement demand using a 2% damped spectrum with some rational coefficients. A conservative estimate of the effective stiffness can be taken as the rocking stiffness (k_r), as shown in Figure 3. A rational expression for the effective stiffness can also be taken as:

$$k_{eff} = k_o\left(\frac{\Delta_{up2}}{\Delta_{y2}}\right) + k_r\left(\frac{\Delta_{y2} - \Delta_{up2}}{\Delta_{y2}}\right) \tag{6}$$

This characterization of the effective stiffness is similar to that in FEMA 356 for systems that experience progressive yielding and do not have a definite yield point and is referred to as Method 1.

The capacity spectrum method for the design of passive energy dissipation systems uses spectral capacity and demand curves to represent the response in a graphical format. The added energy dissipation from the unbonded braces is converted to equivalent viscous damping thus reducing the seismic demand curve from the 2% damped spectrum. This is referred to as Method 2.

Time history analysis is used to verify the adequacy of the simplified methods of analysis and to observe dynamic behavior. Analytical models were developed of the representative piers subjected to a horizontal excitation applied in a primary orthogonal direction. Each pier is assumed to carry an equal mass both vertically and horizontally. The pier itself is modeled with its elastic properties and all nonlinear action occurs at the foundation interface. "Gap" and hysteretic elements are placed in parallel across the anchorage interface to model the rocking mechanism. The hysteretic element is based on the model proposed by Wen (1976). Braces are aligned vertically in the analytical model however they may be implemented inclined to the pier. Restraints are provided at the anchorage level that prevent movement in the horizontal direction but pro-vide no resistance to vertical movements. Inherent structural damping is approximated by assigning 2% equivalent viscous damping to each mode. 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 is used to generate synthetic ground motions attempting to match elastic response spectra defined by the NCHRP 12-49 (ATC/MCEER 2003) spectrum. These motions are applied to the analytical model.

With this type of system (flag-shaped hysteretic), it was shown (including results not presented here) that Method 2 will be more reliable for all possible designs. Method 1 uses a design philosophy that was initially established for elasto-plastic systems however it appears to work reasonably well for systems with η_L>0.6. Sample results are shown in Figure 7, for an aspect ratio of 4 and strength ratios, η_L, of 0.25 and 0.5. The deck-level displacement from time history analysis (Δ_{TH}) is normalized by the displacement predicted from analysis methods 1 and 2 (Δ_{design}).

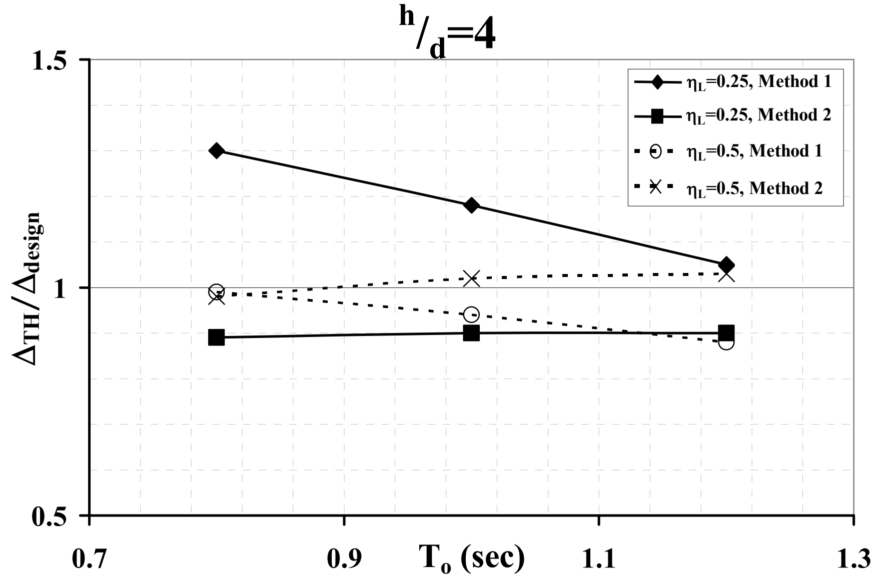

Figure 7. Normalized Displacement Results

Example Time History Results

An example set of time history analysis results obtained following the above design procedure are presented in Figures 8 to 11. Results are normalized by their respective allowable values determined from the design constraints presented in Section 4.1. It can be seen that the displacement of the bridge deck (Figure 8) oscillates about the undeformed position as it comes to rest due to the self-centering ability of the system. The values obtained are less than 1.0, indicating that the resulting system response complied with the design intent.

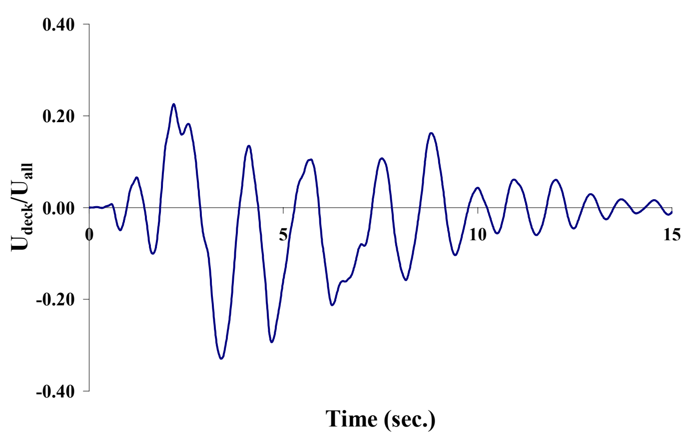

Figure 8. Deck-level Displacement

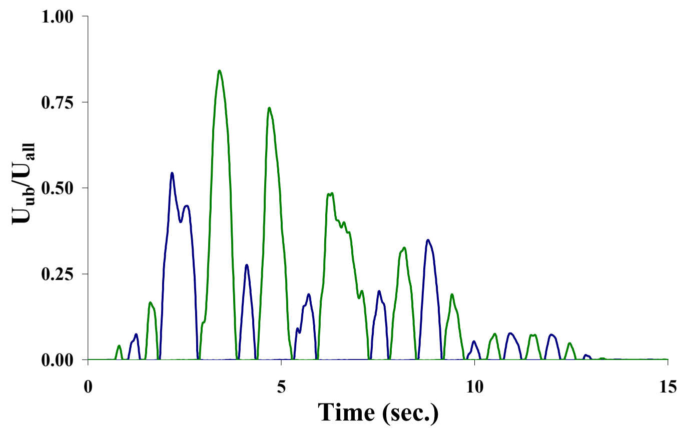

Figure 9. Uplift Displacements

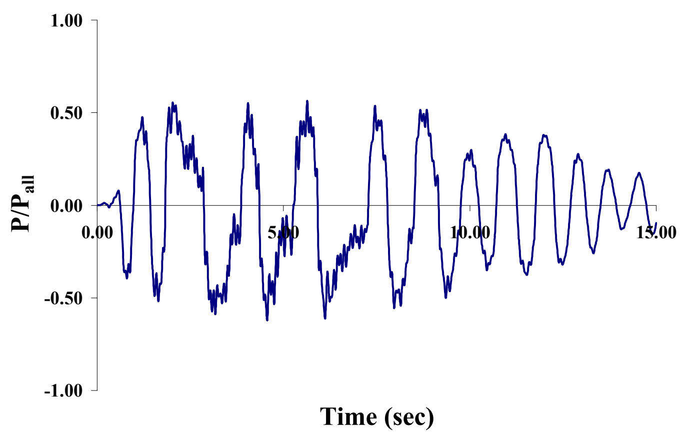

Figure 10. Base Shear

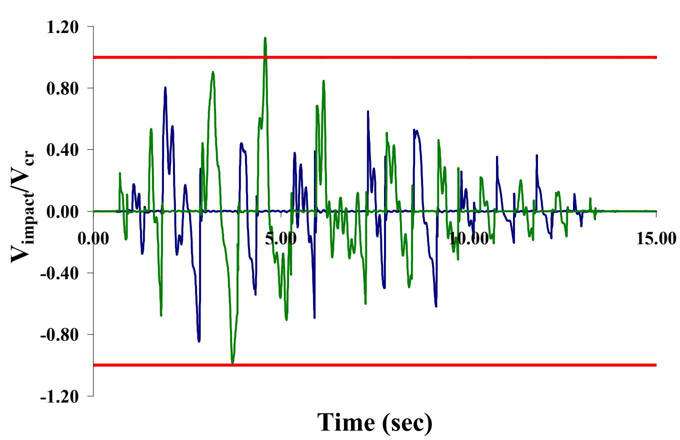

Figure 11. Deck-level Velocity

CONCLUSIONS

A new retrofit strategy relying on controlled rocking has been proposed to achieve ductile seismic performance of steel truss bridge piers. Unbonded 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-recentering 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. Results suggest that the proposed retrofit strategy using the capacity design procedure can predict response such that desired performance is achieved. Further analytical research is needed to investigate response of the rocking system subjected to bi-directional and vertical excitation, refine the existing design procedure and develop details for the implementation of the system. Dynamic experimental testing of rocking steel truss piers with passive energy dissipation devices implemented at the anchorage location is expected in the near future.

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