Seismic retrofit of bridge steel truss pier anchorage connections

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ABSTRACT: In assessments of the seismic adequacy of existing steel bridges, the anchorage of steel truss piers to their foundations often has insufficient strength to resist seismic demands. Many other non-ductile failure locations may also exist along the seismic load path that cannot provide adequate seismic performance. Although 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. 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. Specially detailed, hysteretic energy dissipating elements (unbonded braces) act as ductile structural "fuses" in this application. An inherent re-centering capability is also possible. This paper investigates the dynamic characteristics of the above proposed controlled rocking/energy dissipation system with focus on design implications. Non-linear response history analyses presented here demonstrate the effectiveness and potential benefits of the proposed retrofit solution.

1 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.

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

Figure 1. Typical steel truss bridge.

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. This system also provides an inherent restoring force capability allowing for automatic re-centering of the tower, leaving the bridge with no residual displacements after an earthquake. 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.

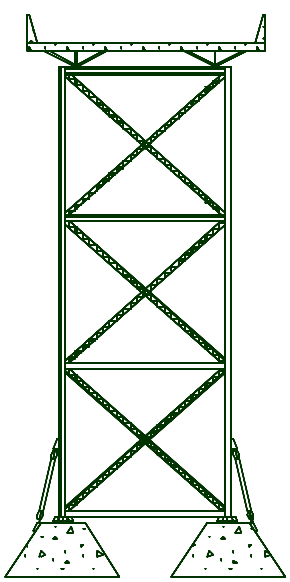

Figure 2. Sketch of retrofitted pier.

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. Both bridges use steel yielding devices across the anchorage interface.

Figure 3. South Rangitikei Rail Bridge.

This paper presents results from research on the dynamic characteristics of the above proposed rocking/energy dissipation system. Nonlinear time history analyses are used to assess the seismic behavior of the bridges retrofitted per this strategy. Observations on the resulting response, along with capacity design concepts and other constraints, needed to protect all other elements, are used to formulate a design procedure for the proposed controlled rocking retrofit strategy. This procedure is briefly outlined, including an overview of on-going work to validate the concept.

Figure 4. Lion's Gate Bridge- north approach.

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 are shown qualitatively in Figure 5 with the corresponding actions of the unbonded brace during the controlled rocking response are shown in Figure 6.

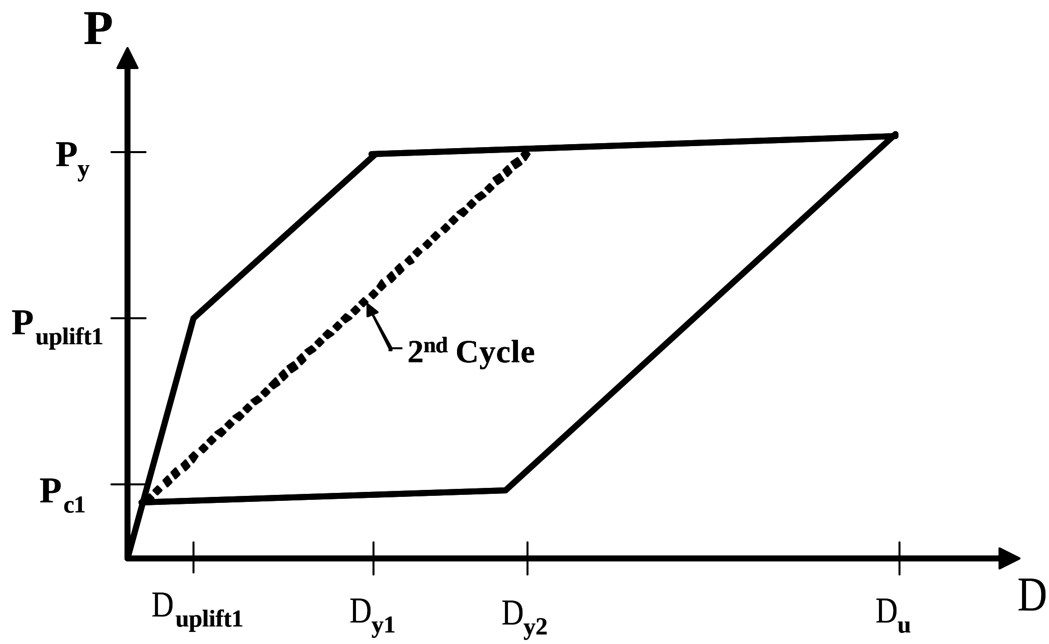

Figure 5. Half-cycle of hysteretic behavior.

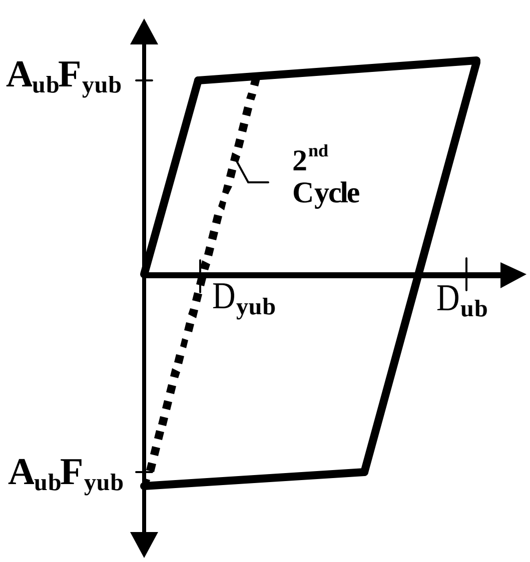

Figure 6. Unbonded brace response.

By symmetry, the behavior repeats itself for movement in the other direction. 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. Hysteretic behavior in the 1st and subsequent cycles, for a given magnitude of inelastic deformation in the unbonded braces, is shown on a single plot in Figure 7.

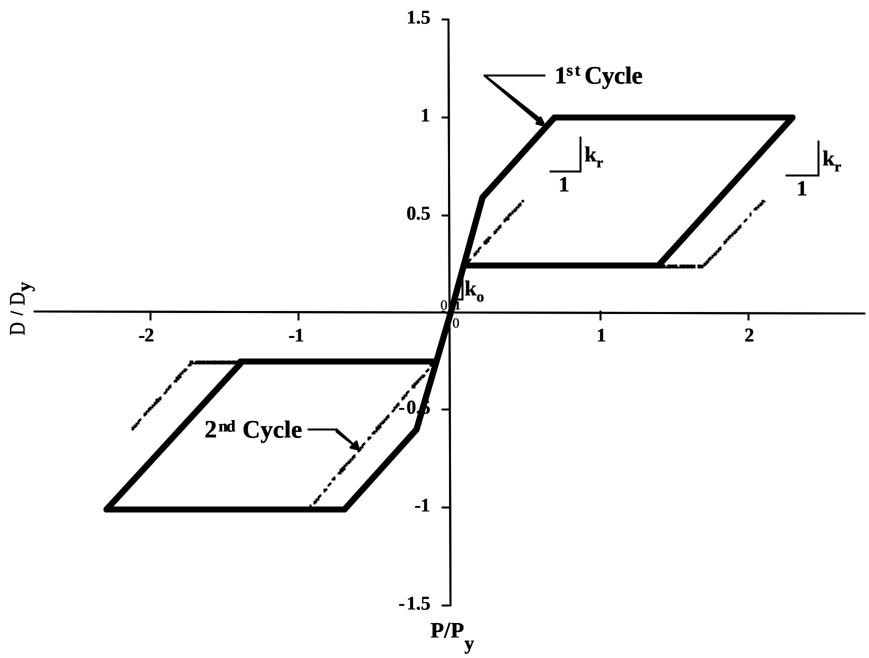

Figure 7. Cyclic hysteretic response.

3 PARAMETRIC STUDY

A parametric study was undertaken in order to provide a preliminary understanding of system behavior. A total of 27 cases were considered. Results obtained are then used to assist in formulating a design procedure (presented in Section 4) that can reliably predict the system's ultimate seismic response. For the purpose of this parametric study, the only constraint imposed was to limit strain on the unbonded braces to an arbitrarily selected value of 1.5%.

3.1 *Discussion of parameters*

A range of parameters assumed representative of steel truss bridge piers were established to investigate the horizontal displacement response of the self-centering system having a flag-shaped hysteretic behavior as shown above. Some tower properties used in this study are given in Table 1. Other parameters varied include the seismic demand characterized by the 1-second spectral acceleration (S_1) varied from 0.25g to 0.75g and the method of analysis used (discussed in Section 3.2).

Table 1. Pier properties used in study.

Aspect Ratio	"Fixed-base" Stiffness (k_o)	"Fixed-base" Period (T_o)
4	12.0 kN/mm	0.76 sec
3	18.4 kN/mm	0.61 sec
2	31.0 kN/mm	0.47 sec

3.2 *Simplified methods of analysis*

One of the objectives of this parametric study was to assess the accuracy of some approximate, simplified techniques in predicting seismic response. Therefore, a number of such procedures were considered. 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. The nonlinear static procedure begins by developing the pushover curve incorporating the nonlinear load-deformation characteristics of individual elements. The load profile for the rocking bridge pier system is taken as a single horizontal load applied at the level of the bridge deck. The 2nd cycle properties are used for determining the displacement demand due to the system's increased flexibility after the first cycle as was seen in the previous section.

A conservative estimate of the effective stiffness can be taken as the rocking stiffness (k_r), as shown in Figure 7. This characterization is referred to as Method 1. A rational expression for the effective stiffness can also be taken as:

$$k_{eff} = k_o\left(\frac{\Delta_{up2}}{\Delta_{y2}}\right) + k_r\left(\frac{\Delta_{y2} - \Delta_{up2}}{\Delta_{y2}}\right) \tag{1}$$

This characterization of the effective stiffness is similar to that in FEMA 356 (FEMA 2000) for systems that experience progressive yielding and do not have a definite yield point and is referred to as Method 2.

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 as shown in Figure 8. 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. Each pier is assumed to have a single degree of freedom representing the dominant horizontal mode of vibration. This is referred to as Method 3.

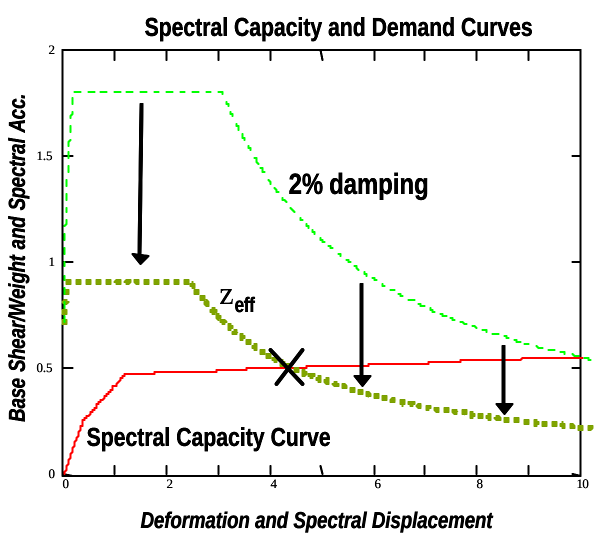

Figure 8. Capacity spectrum plot.

3.3 *Time history analysis*

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 provide 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.

3.4 *Results of parametric study*

The simplified methods of analysis, given the sole constraint of reaching 1.5% strain in the unbonded brace, were able to reliably predict ultimate displacement response for the representative bridge piers. Partial results are presented in Figure 9 below for a seismic demand of S_1=0.75g, thus 9 of the cases considered. These results show the proposed simplified analysis methods are able to predict system response in a conservative manner. While this parametric study validated the proposed concept, additional bounds must be defined to allow the formulation of a design procedure and to ensure that response may be reliably predicted for a more complete range of possible solutions. Part of this work in progress is described in the following section (this work will be completed by the time of the conference, where full results will be presented).

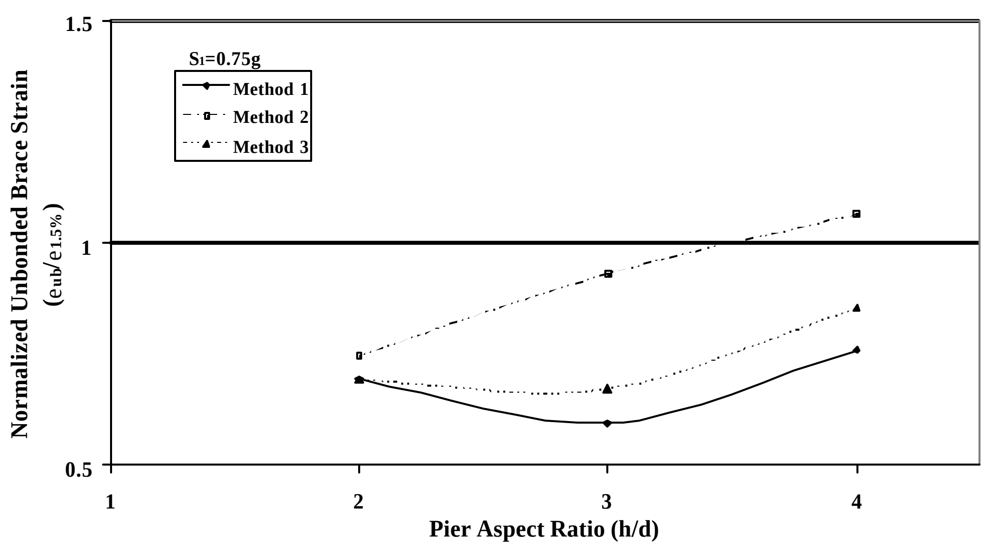

Figure 9. Observed strain demands normalized by target strain of 1.5%.

4 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 in which the boundaries of compliance and non-compliance of the design constraints are plotted with respect to two key design parameters. The two design parameters used are the length and cross-sectional area of the unbonded brace, L_{ub} and A_{ub} respectively.

4.1 *Deck-level displacement*

To the writer's knowledge, there exists no solidly established rule of determining maximum allowable displacements for bridges. Although there are no non-structural components in bridge structures that would warrant the specification of limited drifts to prevent damage, 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:

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

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:

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

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

4.2 *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 on the ultimate inelastic cyclic response of the brace. A strain of 1.5% has been arbitrarily selected here as appropriate for unbounded braces, based on reported experimental results. Therefore this constraint can be established in terms of brace elongation by:

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

4.3 *Impact velocity to foundation*

After a tower 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 other loads are transferred to this foundation as it becomes the new axis of rotation.

A method for determining the demand placed on a general foundation element is proposed based on the conservation of energy. An approach similar to that of Housner (1963) for rigid, rocking blocks is used to determine the reduction in kinetic energy caused by impacting to the foundation.

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. Finally, the velocity constraint can be defined as:

$$v_{max} \leq v_d \tag{6}$$

4.4 *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.

A method is proposed here that creates an "effective", static shear that can be used to evaluate the adequacy of the pier's seismic load path. The base shear demand is determined by the static system yield force amplified by a factor, R_d, to account for the dynamic response. Thus, the ultimate base shear demand can be expressed as:

$$P_u = \left(\frac{W}{2} + A_{ub}F_{yub}\right)\frac{d}{h}R_d \tag{7}$$

Modification of the key design parameters, A_{ub} and L_{ub}, to limit the base shear to an acceptable level or strengthening of the non-ductile elements along the lateral load path can satisfy this constraint.

A sample graphical, capacity based design plot is shown in Figure 9.

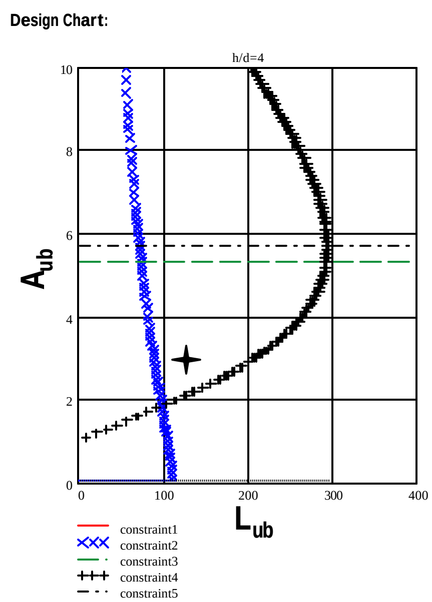

Figure 10. Sample design plot.

An example set of time history analysis results obtained following the above design procedure are presented in Figures 11 to 14. Results are normalized by their respective allowable values determined from the design constraints presented in Section 4.1. The values obtained are less than 1.0, indicating that the resulting system response complied with the design intent. More cases are being investigated at the time of this writing, to ensure reliability of the proposed design procedure.

5 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. Preliminary results suggest that the proposed retrofit strategy using the capacity design procedure can predict response such that desired performance is achieved. Further research is needed to verify results of this analytical study. Dynamic experimental testing of rocking steel truss piers with passive energy dissipation devices implemented at the anchorage location is expected in the near future.

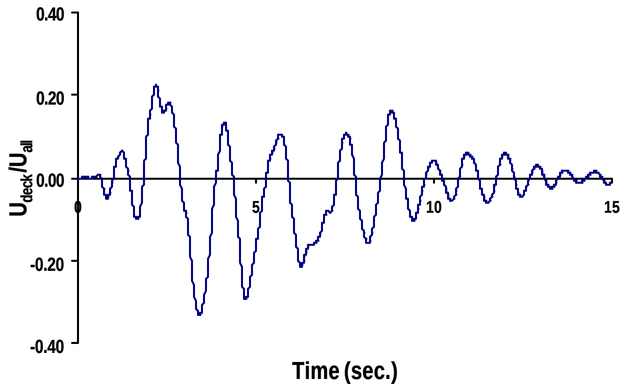

Figure 11. Deck-level displacement.

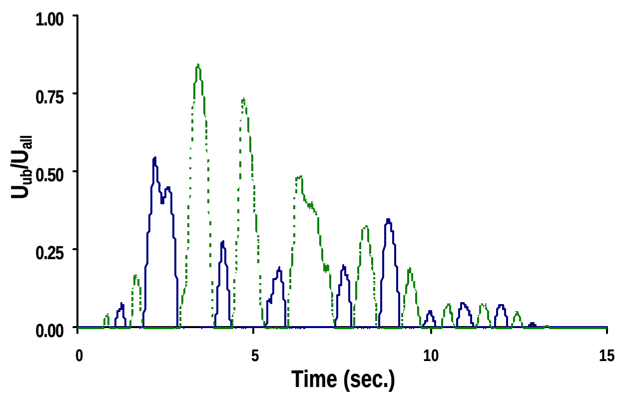

Figure 12. Uplift at foundation.

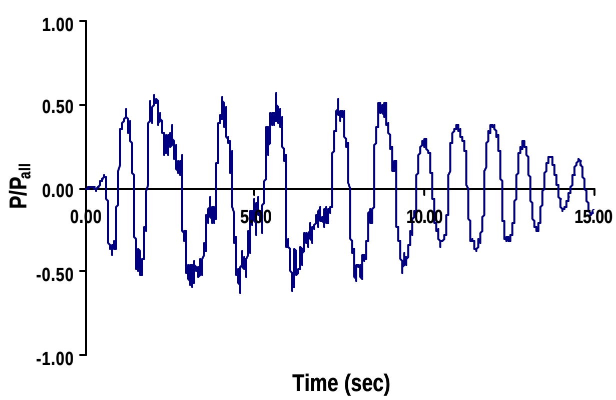

Figure 13. Horizontal base shear.

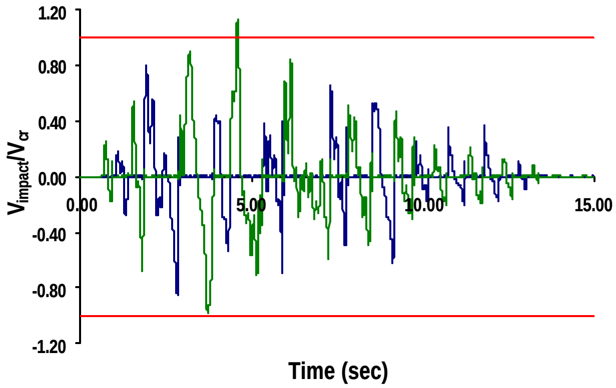

Figure 14. Critical impact velocity.

6 REFERENCES

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

Dowdell, D. and Hamersley, B. (2001) "Lions' Gate Bridge North Approach: Seismic Retrofit", Behaviour Steel Structures in Seismic Areas: Proceedings of the Third International Conference: STESSA 2000; Montreal, Canada, August 21-24, 2000, pp.319-326

FEMA (1997), FEMA 274 NEHRP Recommended Provisions for Seismic Regulations for New Buildings and Other Structures, Part-2-Commentary, Building Seis mic Safety Council for the Federal Emergency Management Agency, Washington, D.C.

FEMA (2000), FEMA 356 NEHRP Recommended Provisions for Seismic Regulations for New Buildings and Other Structures, Building Seismic Safety Council for the Federal Emergency Management Agency, Washington, D.C.

Housner, G.W. (1963). "The Behavior of Inverted Pendulum Structures During Earthquake," Bulletin of the Seismological Society of America, Vol. 53, No.2, pp. 403-417.

Iwata, M., Kato, T., Wada, A., (2000) "Buckling-restrained braces as hysteretic dampers" Behaviour of Steel Structures in Seismic Areas, STESSA 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.

Priestley, M.J.N., Seible, F. and Calvi, G.M. (1996). Seismic Design and Retrofit of Bridges, John Wiley & Sons, New York.

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.