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Seismic Retrofit of Bridge Steel Truss Pier Anchorage Connections

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Summary

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 research investigates the dynamic characteristics of the above proposed controlled rocking/energy dissipation system with focus on design implications.

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.

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

A controlled rocking approach to seismic resistance was implemented in the design of the South Rangitikei Rail Bridge, 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 and Hamersley, 2001), shown in Figure 2. Both bridges use steel yielding devices across the anchorage interface.

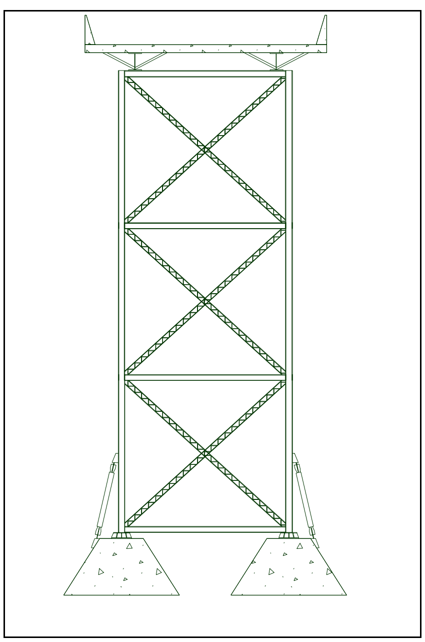


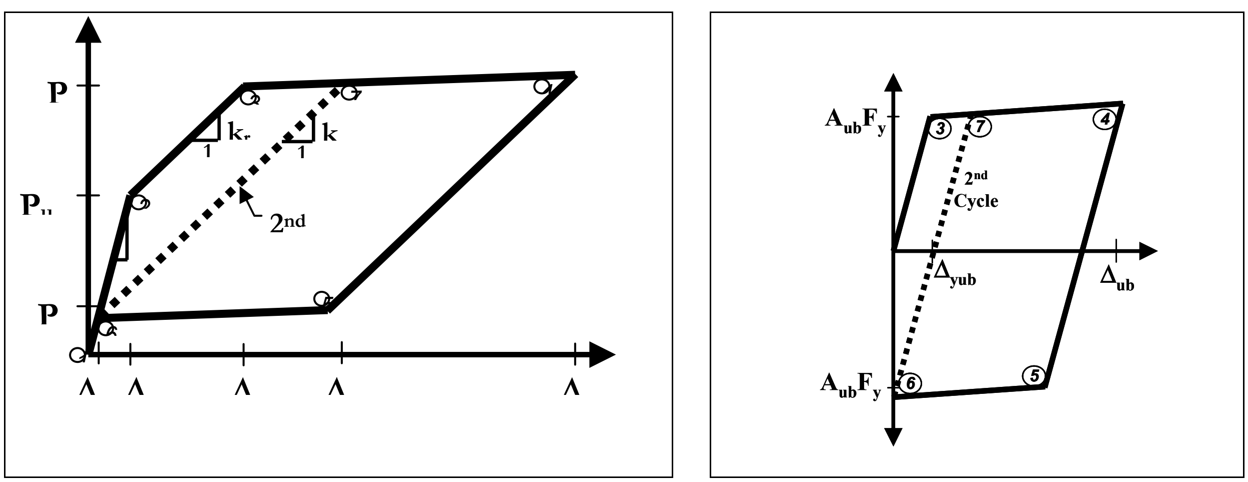
Figure 1. Sketch of retrofitted pier

Figure 2. Lions' Gate 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 ongoing and future work to validate the concept.

Controlled Rocking System for Seismic Retrofit

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 stress 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. A strength ratio, η_L, is defined here as the ratio of unbonded brace yield strength ($A_{ub}F_{yub}$) to half of the vertical weight ($W_v/2$). This is a measure of the hysteretic energy dissipation per cycle and $\eta_L<1$ provides for pier re-centering. The various steps and physical behaviors that develop through a typical half-cycle are shown qualitatively in Figure 3a and the corresponding actions of the unbonded brace during the controlled rocking response are shown in Figure 3b.



(a) (b)

Figure 3. System hysteretic behavior (half-cycle)

By symmetry, the behavior repeats itself for movement in the other direction. Transition from first to second 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.

Parametric Study

In order to provide a preliminary understanding of system behavior, a parametric study was undertaken to observe the dynamic system response and to assess the accuracy of some approximate, simplified techniques in predicting seismic response. Therefore, a number of such procedures were considered. Two of the methods of analysis considered to characterize system response are similar to the nonlinear static procedure (NSP) in FEMA 356 (FEMA 2000) while another is similar to the nonlinear static procedure for passive energy dissipation systems found in FEMA 274 (FEMA 1997). An analysis procedure similar to the latter can be found in the NCHRP 12-49 document (ATC/MCEER 2003).

The NSP requires the development of the system pushover curve, incorporating the nonlinear load-deformation characteristics of individual elements. The second cycle properties are used for determining the displacement demand due to the system's increased flexibility after the first cycle, as was described previously. Using results from the pushover analysis, a rational expression for the effective stiffness can be taken as:

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

which uses the pre- and post-uplift properties of the system. This is referred to as Method 1.

The capacity spectrum method for the design of passive energy dissipation (PED) 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. Each pier is assumed to have a single degree of freedom representing the dominant horizontal mode of vibration. 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 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 University 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.

Sample results are shown in Figure 4, 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 design methods 1 and 2 (Δ_{design}). The design methods were able to predict response with sufficient accuracy for design. 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.

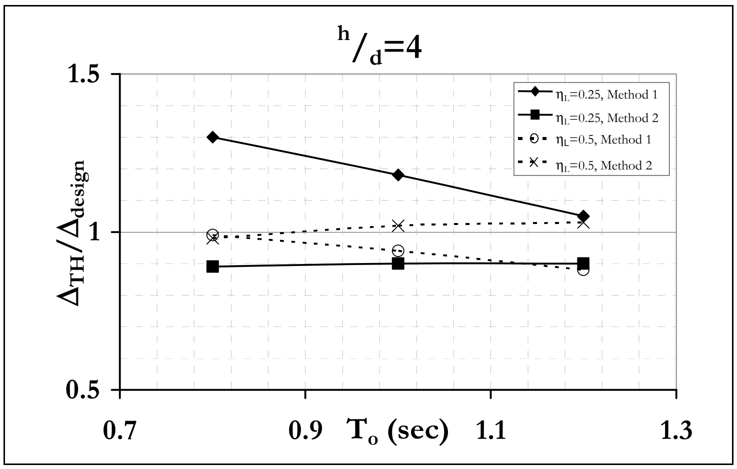


Figure 4. Sample results of parametric study

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.

Deck-level Displacement

To the writer's knowledge, there exists no solidly established rule for 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.

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 et al., 2000, is shown in Figure 5. 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.

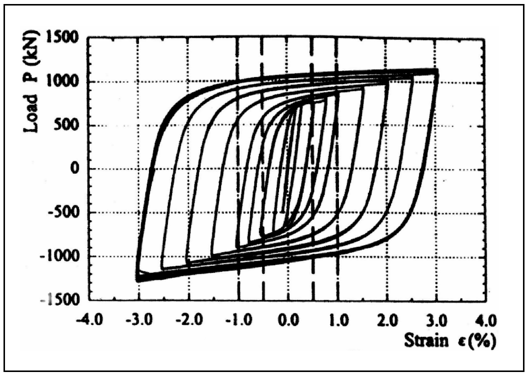


Figure 5. Experimental test results (adapted from Iwata et al., 2000)

Impact Velocity to Foundation

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

$$v_{imp} = PS_{vi}\left(\frac{d}{h}\right) \tag{2}$$

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. The base shear demand is determined by the static system yield force amplified by a factor, R_{dup}, to account for dynamic effects resulting from uplift from the foundation. A conservative estimate of the maximum axial force developed within pier legs is essential due to the pier legs being the primary gravity load resisting system of the bridge. Energy principles are used to account for the impacting of the pier legs and impulsive loading applied during rocking.

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 thereafter.

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