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SEISMIC RETROFIT OF STEEL STRUCTURES

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RESUMEN

El artículo presenta una breve revisión de trabajos recientes sobre el desarrollo de soluciones para el diseño y la rehabilitación sísmica de estructuras de acero por distintos miembros de la comunidad de investigación de los Estados Unidos de América, incluyendo soluciones que se han desarrollado para la rehabilitación sísmica de puentes y edificios en la Universidad de Nueva York en Buffalo

SUMMARY

This paper provides a brief review of recent work on the development of solutions for the seismic design and retrofit of steel structures by various members of the U.S. research community, including solutions being developed at the University at Buffalo for the seismic retrofit of bridges and buildings.

INTRODUCTION

Although much efforts has been invested in recent years in the U.S. to seismically retrofit buildings having unreinforced masonry walls and reinforced concrete frames, steel buildings and bridges have also received a significant attention. In buildings having steel frames, this interest in seismic retrofit stems from the realization, following the 1994 Northridge earthquake, that the welded beam-to-column connections in moment resisting frames were likely to fail in a brittle manner, prior the development of significant inelastic response, thus negating the design intent and possibly causing safety hazards. A second impetus was provided by passage of the California Senate Bill 1953 that mandates that all health care facilities providing acute care services be retrofitted to a life-safety performance level by 2008, and a full-serviceability level by 2030. For bridges, important seismic evaluation and retrofit of major crossings have occurred since a span of the San Francisco–Oakland Bay bridge collapse the 1989 Loma Prieta earthquake. Large steel truss bridges were evaluated and in some cases retrofitted in most states where these important lifelines exist, including California, Washington, Oregon, New York, and the Mid-West States. Much of that prior work has been reported previously, and detailed overviews of these major retrofit projects would best be the domain of a textbook.

However, it is important that recent research has expanded the variety and versatility of the tools available in the structural engineer's toolbox to meet the seismic performance objectives. As such, this

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brief paper provides an overview of how this research is expanding the available options for the seismic retrofit of steel building and bridges in the U.S., by reporting on some selected recent research projects by U.S. researchers (including some work conducted by the author). Although this is can only be done superficially here due to space restrictions, many references are provided to guide the reader for further studies. It should be understood that this paper is limited to recent research known to the author and for which lead researchers kindly provided relevant information, and consequently contains unintentional omissions.

RECENT U.S. RESEARCH FOR RETROFIT OF BUILDINGS

Retrofit of beam-to-column moment connections

The damage to welded beam-to-column connections in steel moment resisting frames following the January 1994 Northridge earthquake has been well documented elsewhere (FEMA 2000). Brittle fractures originated in the beam flange groove welds and often propagated to rupture beam flanges (welds or base-metal) or column flanges. Given that this connection detail has been widely used across the U.S. in building frames to resist wind and seismic forces, the poor performance of this moment connections during the Northridge earthquake raises concerns, and seismic retrofit of connections in existing steel buildings may be desirable to avoid similar failures in future earthquakes.

A cooperative effort by the National Institute of Standards and Technology (NIST), the American Institute of Steel Construction (AISC), the University of California at San Diego, the University of Texas at Austin, and Lehigh University, examined three techniques for retrofit of existing steel moment connections, namely: (i) reduced beam section (RBS) at the bottom flange of the beam; (ii) welded haunch at the bottom flange of the beam, and; (iii) bolted brackets.

These techniques were experimentally assessed using full scale moment connection specimens (with and without a composite concrete floor slab). Pre-Northridge welded flange-bolted web connection s were built, using low-toughness weld metal at the beam flange groove welds to replicate pre-Northridge connections details and materials, then retrofitted using the above techniques. Analytical research on the same connections complemented this work, and design models and guidelines were recommended. A target plastic rotation capacity of 0.02 radian was selected. Engelhardt (personal communication, 2004) summarized the results of this program as follows:

- *"The use of a bottom flange RBS or a bottom flange bolted bracket, with the existing low toughness beam flange groove welds left in-place, provided no significant improvement in performance as compared to the pre-Northridge welded flange-bolted web connection."*
- *"The use of a bottom flange RBS, combined with the replacement of top and bottom beam flange groove welds with high toughness weld metal, provided plastic rotations on the order of 0.02 to 0.025 radian. The presence of a composite slab had little effect on the performance of this retrofit technique."*
- *"The addition of a welded bottom haunch, with the existing low toughness beam flange groove welds left in-place, resulted in significantly improved connection performance. The performance of this retrofit technique was significantly influenced by the presence or absence of a composite concrete floor slab. In the absence of a concrete floor slab, the welded bottom haunch specimens developed plastic rotations of 0.015 to 0.025 radian. When a concrete floor slab was present, the*

specimens developed plastic rotations in excess of 0.03 radian. Consequently, the composite concrete floor slab provided a significant beneficial effect for the bottom welded haunch connection."

- *"The use of bolted brackets at the top and bottom flanges provided plastic rotations in excess of 0.03 radian."*

Note that, for seismic retrofit, Uang et al. (2000) and Yu et al (2000) showed that the welded haunch scheme (Fig. 1) eliminates the need to modify the existing groove weld of the top flange. They also demonstrated that conventional beam theory cannot provide a reliable prediction of the flexural stresses in the groove welded joint, and proposed a simplified model that considers the interaction of forces and deformation compatibility between the beam and the haunch, in-turn used to develop a step-by-step design procedure.

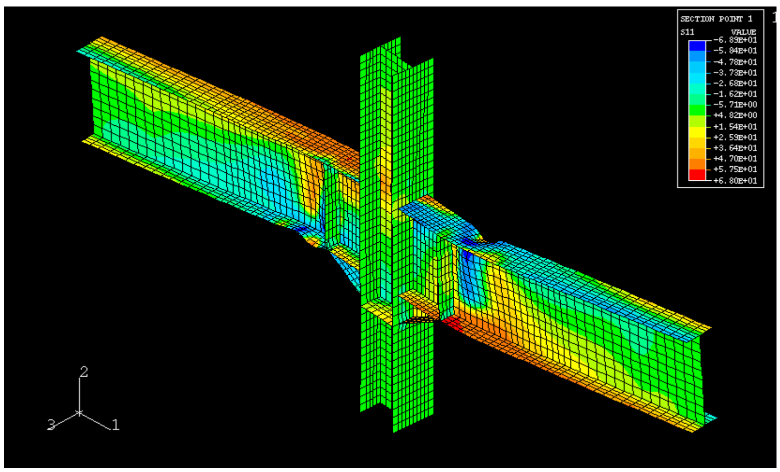


Figure 1: Model of Welded Haunched Retrofit Scheme (Chia-Ming Uang, University of California, San Diego, personal communication, 2004)

The AISC Design Guide No. 12, Modification of Existing Welded Steel Moment Frame Connections for Seismic Resistance (Gross et al 1999) was produced as a result of this research program. Civjan et al. (2000a, 2000b, 2001) also provide information on this research program.

As part of a broader research effort to improve knowledge on the behavior of moment connections recommended by FEMA-350, Hajjar et al (2003) investigated the adequacy of the design provisions for column stiffening, including both continuity and doubler plate detailing, for non-seismic and seismic design conditions, and proposed alternative column stiffener details. Component testing included monotonically loaded pull-plate experiments (Prochnow et al. 2000), cyclically loaded beam-to-column connections (Lee et al. 2002), and parametric finite element analyses (Ye et al. 2000; Webster, 2000). They concluded (Hajjar et al. 2003) that the "*AISC non-seismic design provisions for local web yielding and local flange bending are reasonable and slightly conservative in calculating the need for column stiffening*." They also provided new design equations for the limit states of local flange bending and local web yielding in fully-restrained moment connections, and presented a doubler plate details that avoids welding to the column k-line, and that performed satisfactorily under monotonic loading (Fig. 2).

In other work, Tort and Hajjar (2004) reviewed the seismic performance of rectangular concrete-filled steel tubes (RCFT) columns and beam-columns on the basis of how damage progressed through testing, both for monotonically- and cyclically-loaded specimens, for tests conducted worldwide. Deformation-based and energy-based damage functions were used to quantify this data. This database of information provides a platform from which reliability-based performance-based design provisions can be developed for RCFT.

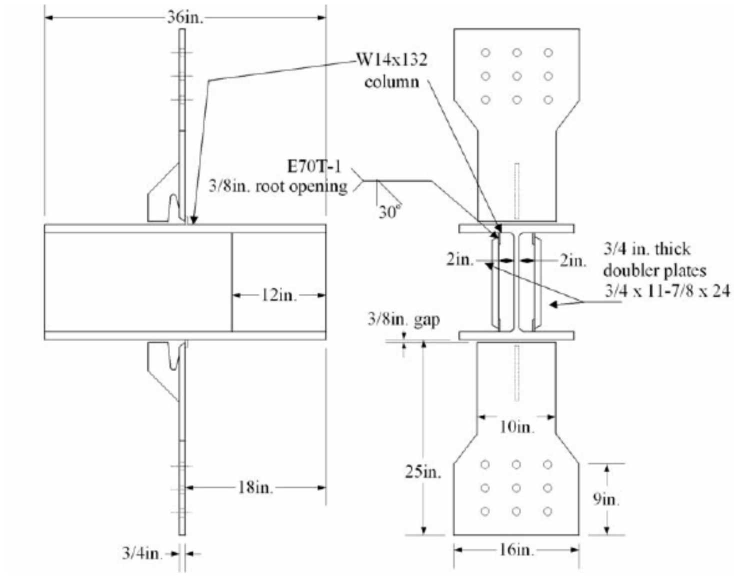


Figure 2: Detail with Doubler Plates Welded to the Column Flange away from Web
(Hajjar et al. 2003)

Frame modifications at beams' mid-span

Leelataviwat et al (1998) focused on retrofitting moment resisting frames by introducing a ductile "fuse" element in shear at mid-span of beams instead of modifying the beam-to-column connections (Fig. 3). This alternative scheme moves the plastic deformation demands away from these critical regions, while ensuring that the existing non-ductile moment frames develop a ductile mechanism. This is achieved by creating a specially braced rectangular opening in the web of each girder near the mid span. Experimental and analytical research demonstrated the effectiveness of the strategy. The proposed scheme is applicable to both upgrading of existing frames, as well as new construction.

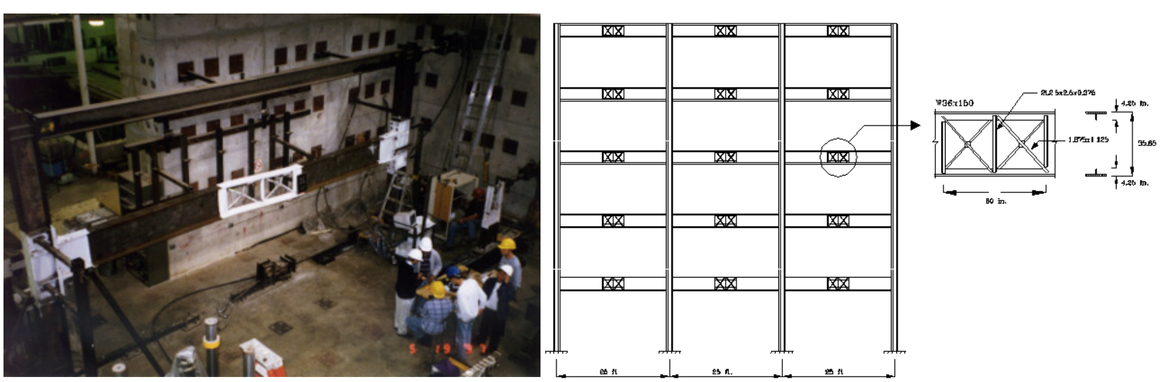


Figure 3: Frame modified with mid-span truss opening (Leelataviwat et al. 1998)

Self-centering systems

Christopoulos et al (2002a, 2002b) and Collins and Filiatrault (2003) proposed the design of self-centering structural systems as economical alternatives to special moment resisting frame used in the United States for the seismic retrofit of steel framed structures. These innovative structural systems:

- *"Incorporate the nonlinear characteristics of yielding structures and, thereby, limit the induced seismic forces and provide additional damping characteristics."*
- *"Encompass self-centering properties allowing the structural system to return to its original position after an earthquake."*
- *"Reduce or eliminate cumulative damage to the main structural elements."*

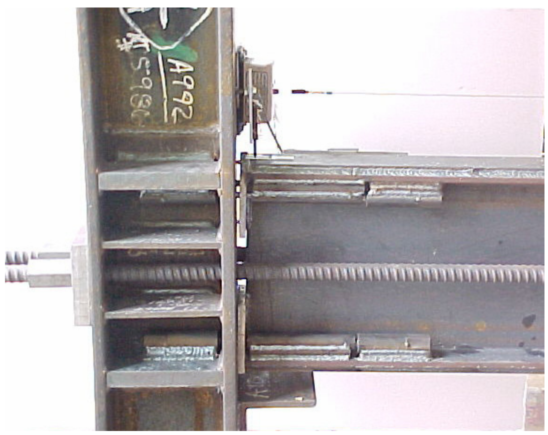


Figure 4: PTED System (Christopoulos et al 2002a)

The proposed design (Fig. 4) is a Post-Tensioned Energy Dissipating (PTED) steel frame (Christopoulos et al. 2002a, 2002b). High strength bars or tendons located at mid-depth of the beam provide the post-tension at each floor. Four symmetrically placed energy-dissipating (ED) bars threaded into couplers welded at various locations provide energy dissipation under cyclic loading. Confining steel sleeves are used to prevent the ED bars from buckling during cyclic inelastic loading. Holes in the column flanges accommodate the PT and ED bars. A corresponding hysteretic model is shown in Fig. 5.

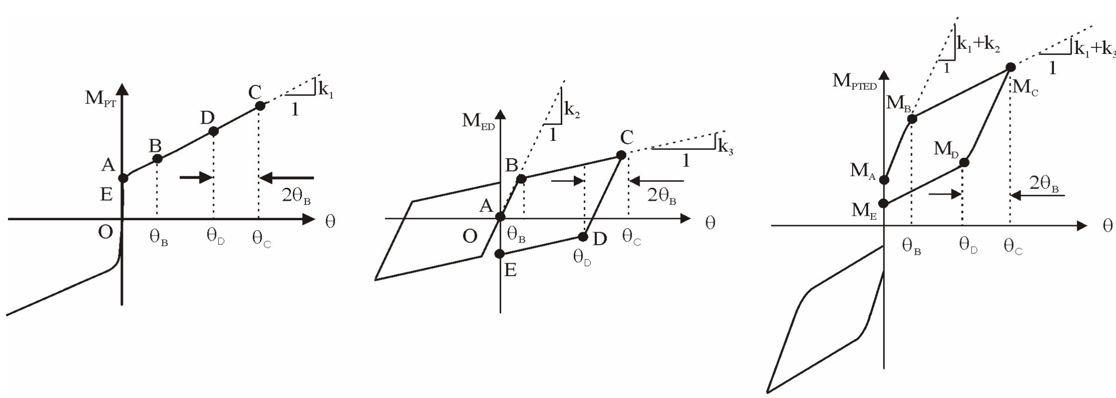


Figure 5: Hysteretic Model for PTED System (Christopoulos et al. 2002a)

Garlock et al (2004), also independently developed a post-tensioned moment connection for use in seismic resistant steel moment frames. Their particular connection relies on high strength steel strands post-tensioned after bolted top-and-seat angles are installed (Fig. 6). Inelastic deformation of (replaceable) top-and-seat angles provides energy dissipation in this case. The vertical shear is redundantly supported by both the angles and the friction between the beam and the column. The system is also redundant in that it can continue to perform satisfactorily if failure of one or more strands occurs.

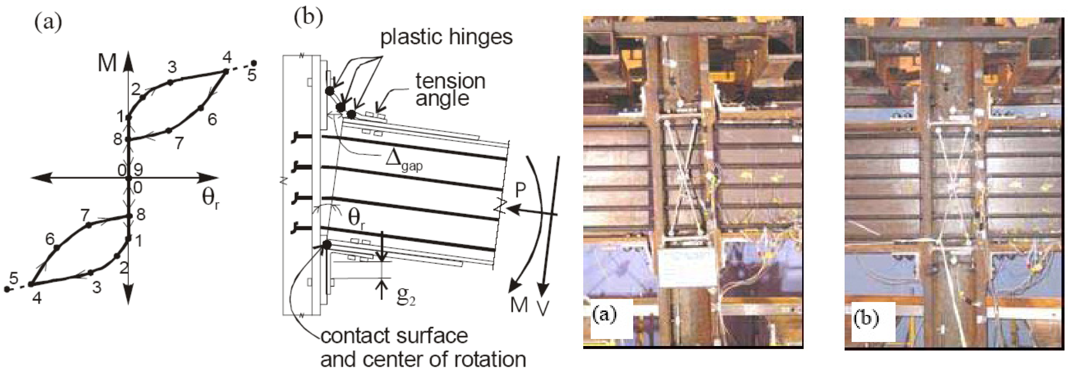


Figure 6: Post-Tension Moment Connection with Top and Bottom Seat Angles (Garlock et al. 2004)

In both studies, cyclic quasi-static testing showed that PT steel connections specimens can exhibit stable self-centering hysteretic behavior when beam local buckling and strand yielding do not occur. Garlock et al. indicated that greater strength and ductility was achieved in connections with more strands. On the basis of time-history analyses of design examples, both groups concluded that frames with PT steel connections can provide adequate seismic performance, including strength, stiffness, “ductility” and drift capacity, when subjected to earthquake loading.

Zipper frames

Leon and Yang (2003) developed a simplified design procedure for suspended zipper frames. Such frames are built from inverted-V-braced frames by adding zipper columns (i.e. vertical members connecting to the intersection points of the braces above the first floor), an elastic hat truss at the top story, and a capacity design procedure that establishes a clear hierarchy of yielding (Fig. 7). This is done to distribute uniformly along the frame height the unbalanced vertical force generated when braces buckle.

Zipper columns, first proposed by Khatib et al. (1988), tie all brace-to-beam intersection points together, and are intended to force all compression braces in a braced bay to buckle simultaneously, and thus better distribute energy dissipation over the height of the building. However, concerns exist that instability and collapse could develop once the full-height zipper mechanism forms, because of the frame’s lower lateral capacity beyond that point (Tremblay and Tirca, 2003). Moreover, a capacity design approach for zipper frames requires assumptions on what should be the desirable plastic collapse mechanism.

Leon and Yang proposed to ovecome the disadvantages of a full-height zipper mechanism by introducing a suspension system, labeled the “suspended zipper frame”. In a suspended zipper frame, the top story braces are designed to remain elastic; all other compression braces are designed to buckle. The suspended zipper struts are designed to yield in tension. Overall, lighter beams and zipper struts are obtained per this design philosophy. This also makes a capacity design procedure possible. Leon and Yang showed that their proposed design strategy results in ductile suspended zipper frames, with superior seismic performance and strength compared to ordinary zipper frames.

A distributed experiment is underway to validate the suspended zipper frame concept. In that project, shake-table tests will be conducted at the University at Buffalo to verify analytical models, frame behavior, and to establish displacement and story force response histories. These results are then used by researchers at the University of California, Berkeley, to compare with demands obtained from their own pseudo-dynamic tests, researchers at the University of Colorado will use the displacements histories to conduct test on individual and sub assemblage specimens, and researchers at Georgia Tech will use displacements and force histories as input for further quasi-static test.

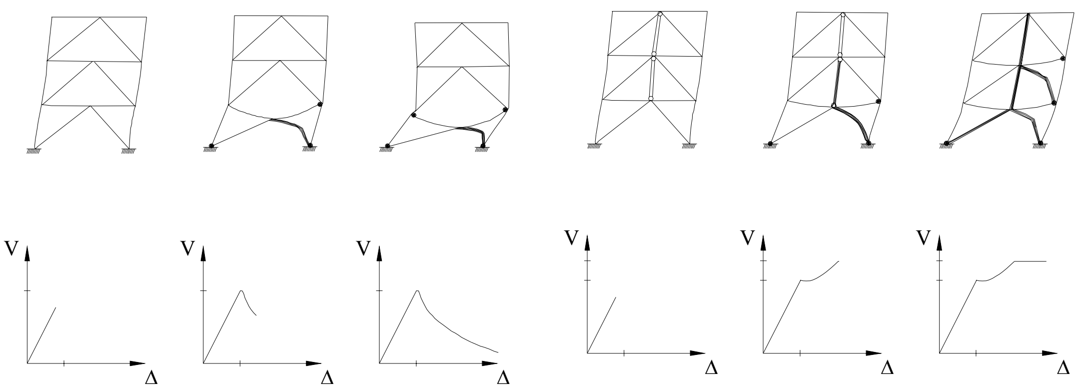


Figure 7: Push-Over Curves for Inverted-V-Braced Frames and Suspended Zipper Frames (Roberto Leon, Georgia Tech, Personal Communication, 2004)

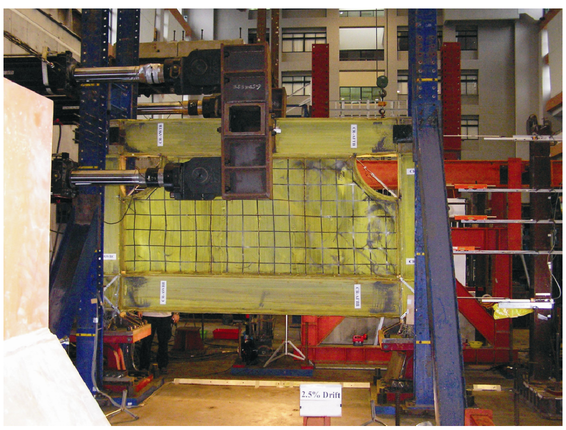
Buckling-restrained braced frames

Buckling-restrained braced (BRB) frames have received much attention in recent years in the U.S., and other authors have extensively covered the latest research and knowledge on this topic (Sabelli et al. 2003, Uang and Nakashima, 2004). Design requirements for BRB frames will be included in the 2005 Edition of the *Seismic Provisions for Structural Steel Buildings*, published by the American Institute of Steel Construction, even though at this time, most BRB systems are proprietary (as a result, testing of components and representative sub-assemblies are typically required). Many uniaxial tests of diverse types of BRBs have been conducted to date, consistently exhibiting stable hysteresis behavior (with full hysteresis loops) and excellent low-cycle fatigue life. Limited subassembly test results have showed some undesirable failure modes, typically due to buckling and cracking of gusset plates. However, it was observed in those cases that similar failures would have occurred in all types of braced frames pushed to the same displacement histories (López et al., 2002), highlighting the limited knowledge and significant need for further research on the behavior of braced frames (with their surrounding frames) in general.

Steel plate shear walls

The selection of Steel Plate Shear Walls (SPSWs) as the primary lateral force resisting system in buildings has increased in recent years as design engineers discover the benefits of this option. Its use has matured since initial designs, which did not allow for utilization of the post-buckling strength, but only elastic and shear yield plate behavior. Research conducted by Thorburn et al. (1983), Lubell et al (2000), Driver et al. (1997), Caccese et al (1993), Berman and Bruneau (2003) (among many) supported the SPSW design philosophy that reduced plate thickness by allowing the occurrence of shear buckling. After buckling, lateral load is carried in the panel via the subsequently developed diagonal tension field action. Smaller panel thicknesses also reduce forces on adjacent members, resulting in more efficient framing designs. Understanding of the seismic behavior of thin plate SPSWs has significantly improved in recent years, and design requirements for SPSWs are scheduled for implementation in the 2005 Edition of the *Seismic Provisions for Structural Steel Buildings*, published by the American Institute of Steel Construction. Yet, some obstacles still exist that may impede further widespread acceptance of this system. For example, using the yield stress for typically available steel material, the panel thickness as required by a given design situation may often be much thinner than the minimum hot rolled steel plate thickness typically available from steel mills. In such a case, using the minimum available plate thickness would result in a large difference in panel forces from that required by calculations. Attempts at alleviating this problem were recently addressed by the use of light-gauge cold-formed steel (Berman and Bruneau 2003).

Vian and Bruneau (2004) investigated the seismic performance of SPSWs designed and fabricated using low yield strength (LYS) steel panels and Reduced Beam Sections (RBS) added to the beam ends in order to force all inelastic action in the beams to those locations (Fig. 8). It was felt that this would promote increasingly efficient designs of the "anchor beams," defined as the top and bottom beams in a multistory frame, which "anchor" the tension field forces of the SPSW infill panel.



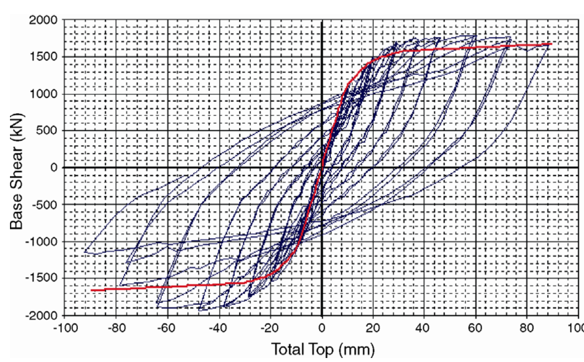


Figure 8: SPSW Specimen with Cutout Corners (right) and Typical Hysteresis Loops Solid Wall Specimen (Vian and Bruneau 2004)

SPSWs with low yield steel webs appear to be a viable option for use in resistance of lateral loads imparted during seismic excitation. The lower yield strength and thickness of the tested plates result in a reduced stiffness and earlier onset of energy dissipation by the panel as compared to conventional hot-rolled plate. The perforated panel specimen shows promise towards alleviating stiffness and over-strength concerns using conventional hot-rolled plates. This option also provides access for utilities to penetrate the system, important in a retrofit situation, in which building use is pre-determined prior to SPSW implementation. The reduced beam section details in the beams performed as designed, as shown in Fig. 9. Use of this detail may result in more economical designs for beams "anchoring" an SPSW system at the top and bottom of a multi-story frame. On-going research is focusing on developing reliable models that can capture the experimentally observed behavior, and investigating the benefits of this system on enhancing the seismic performance of nonstructural components.

Buckled Panel Following Test | RBS Yielding

Figure 9: Buckled Panel and RBS Yielding of SPW Specimen (Vian and Bruneau 2004).

Plastic rotation limits

Lee and Stojadinovic (2004) developed a cyclic yield-line plastic hinge model to estimate the rotation capacity of U.S.-type fully-restrained steel moment connections subjected to earthquakes (Fig. 10).

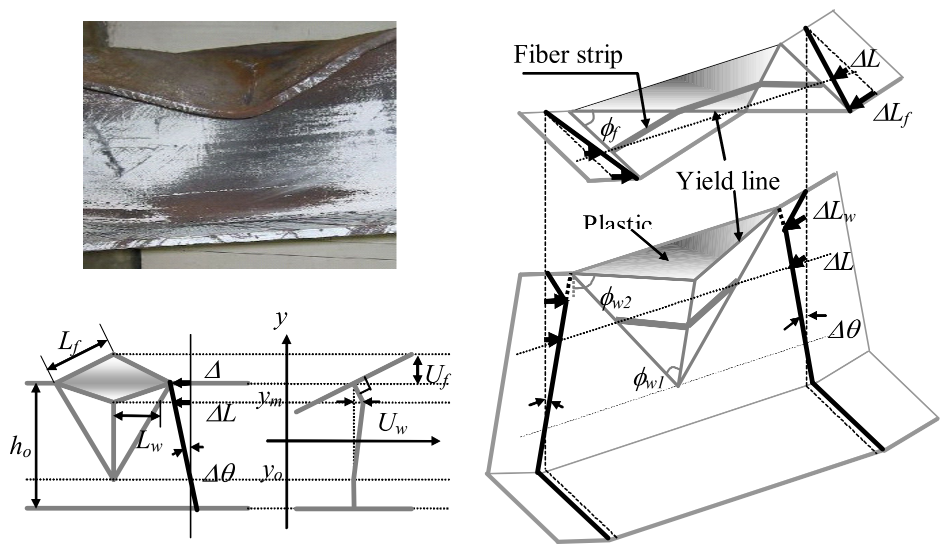


Figure 10: Yield Line Plastic Hinge Model of Local Buckling (Lee and Stojadinovic 2004)

A simplified plastic flow rule is used to model yielding at the yield lines that develop in the beam after local buckling. Stojadinovic (personal communication, 2004) indicates: "*Two limit states are considered using the cyclic yield-line plastic hinge model. First, post-peak connection strength degradation due to local and lateral-torsional buckling is used to establish a connection rotation limit similar to the limit used in FEMA-350. Second, low-cycle fatigue crack initiation model based on* *accumulative local strain concept at the critical yield line is used to predict crack initiation at the creases of the local buckles in the plastic hinge and, thus, limit plastic hinge rotation."* A proposed analytical model was also developed for estimating connection rotation capacity, calibrated using rotation capacities recorded in recent SAC Joint Venture tests. The objective of this model is to provide an analytical platform for validation of new connection types without the need for extensive proof-testing.

RECENT RESEARCH FOR RETROFIT OF BRIDGES

Shear links

Dusicka et al. (2002, 2004) investigated built-up shear links constructed using plates of different grades of steel, including high performance steels (HPS) as well as Japanese low yield point steels (LYP). These steels provided a range of nominal yield strengths from 100 MPa (14.5 ksi) to 485 MPa (70 ksi). The LYP steel in particular allowed for innovative designs of compact shear links without stiffeners.

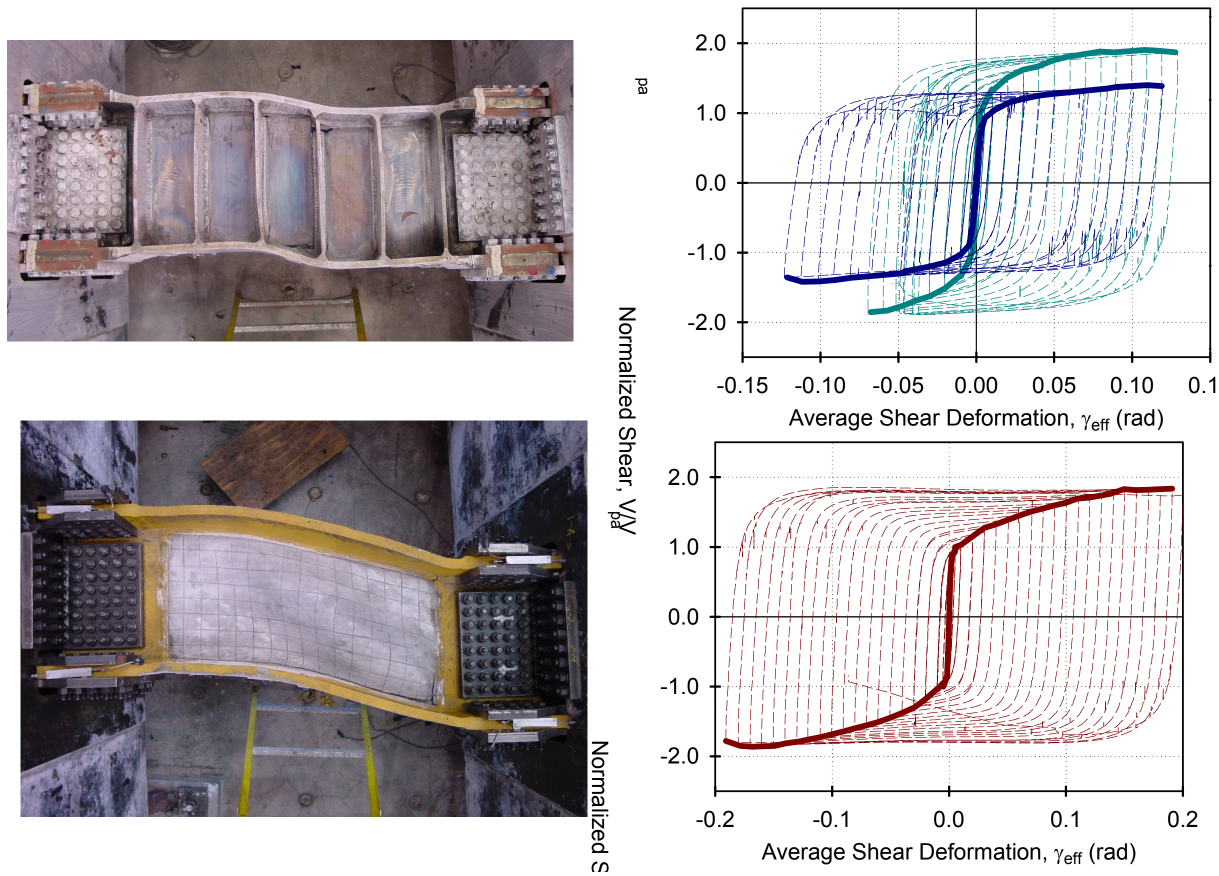


Figure 11: Deformation Near End of Test Cyclic Test Results for Shear Links with High Performance Steel (top) and Low-Yield Steel (bottom) (Dusicka et al. 2004)

Cyclic plastic deformations were applied to determine the deformation capacity, maximum resistance and ultimate failure mode of the shear link components (Fig. 11). All links tested exhibited ductile hysteretic behavior. Itani reported (personal communication, 2004): *In links where stiffeners were* *used, the failure mode initiated with cracks in the web at the stiffener connections and propagated along the heat affected zone leading to progressive tearing and failure at 0.12 rad shear rotation. Significant improvement in deformation capacity was found for the LYP links without stiffeners leading to shear deformations up to 0.20 rad. The ultimate strength was found to significantly vary among all of the specimens, the results of which directly affects capacity design of the associated structural components.*

Truss piers

Steel truss bridges are found in nearly every region of the U.S. 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. These built-up lattice type members and their connections can be the weak link in the seismic load path. Recent experimental testing (Fig. 12) of these members revealed the limited ductility that can be achieved due to global and local buckling causing significant strength and stiffness degradation (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. Analyses 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.

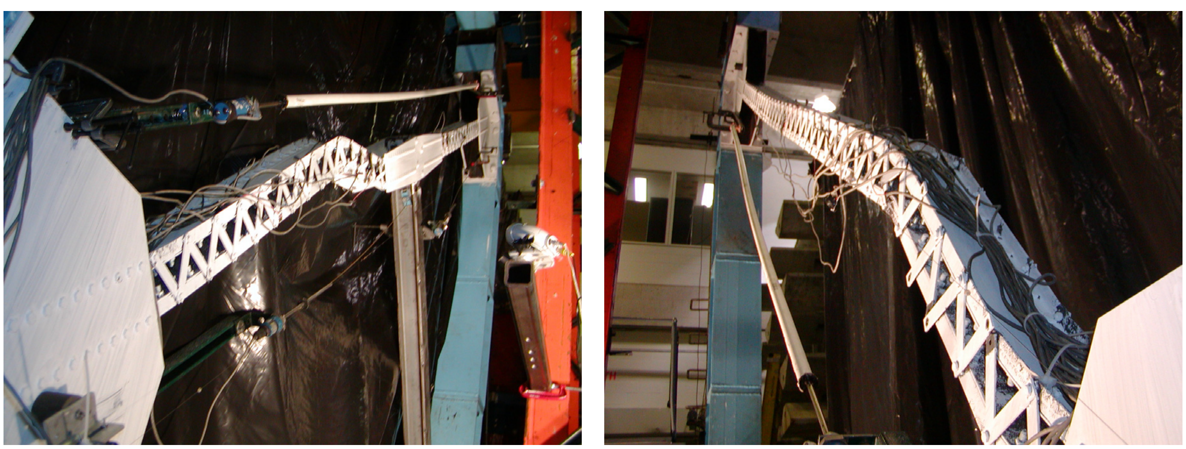


Figure 12 Global buckled shape of various specimens (Lee and Bruneau 2003)

Failure of, or releasing of, the anchorage connection allows a steel truss pier to 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 (Pollino and Bruneau, 2003). This system can also be designed to provide an inherent restoring force capability that allows 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 Fig. 13.

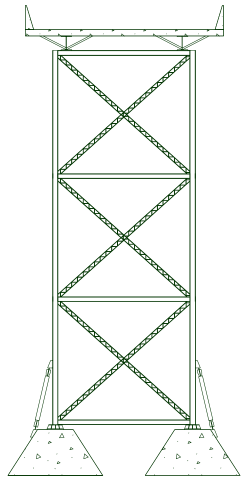


Figure 13: Sketch of retrofitted pier (Pollino and Bruneau 2003).

A controlled rocking approach to seismic resistance was implemented into 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 & Hamersley 2001) as shown in Fig. 14. Both bridges use steel yielding devices across the anchorage interface.

Figure 14. South Rangitikei Rail Bridge (left) and Lion's Gate Bridge- north approach (right).

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. 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 Fig. 15.

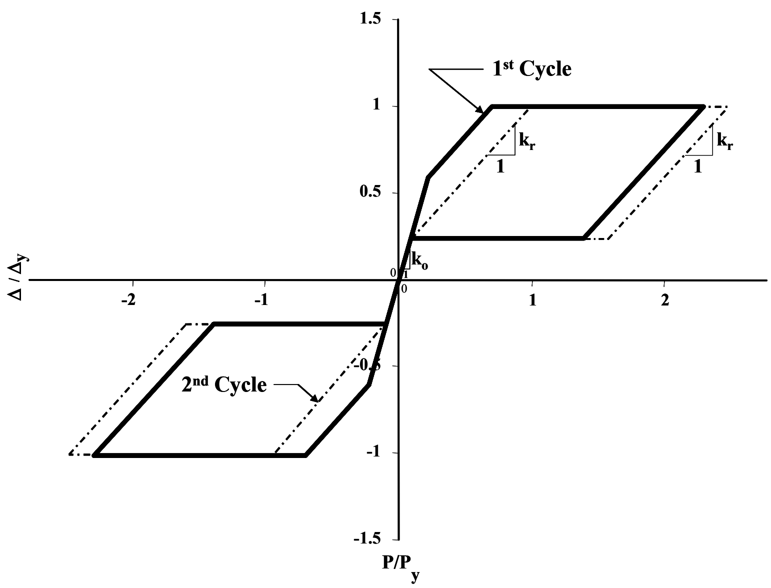


Figure 15: Hysteretic Behavior of Rocking Truss Pier (Pollino and Bruneau 2003)

A parametric study was undertaken in order to provide a preliminary understanding of system behavior. Results obtained were then used to assist in formulating a design procedure that can reliably predict the system's ultimate seismic response. In the perspective of seismic retrofit, a capacity based design procedure was also proposed to protect non-ductile elements while limiting energy dissipation to the specially detailed steel yielding devices. In a seismic retrofit perspective, a large number of constraints exist and thus a systematic design procedure that satisfy all constraints was developed. The proposed design procedure was complemented by 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 (Pollino and Bruneau 2003).

CONCLUSIONS

Much activity is underway in the U.S. on developing new methods to seismically retrofit steel buildings and bridges. This sustained research is generating the knowledge needed to ensure that the highest level of seismic performance expected of these newer systems will be achieved.

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