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The AISC 2010 Seismic Provisions for Composite Structures: Towards an Application of PBD Principles for Connection Design

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ABSTRACT: The 2010 AISC Seismic Design provisions represent a major change for composite structures for at least two reasons. First, the provisions for composite structures have been folded into a single document with those for steel; the net result, at least superficially, is that composite construction is considered equivalent to steel in quality and performance. Second, the provisions for composite members and structural systems have become more prescriptive in an attempt to ensure a minimum level of performance. An unintended consequence of this latter item is that some freedom in the introduction of innovative composite connections has been removed. In this paper, a short review of the AISC composite provisions will be given, and a case study on the development of an innovative connection will be described. A refined finite element model was developed to conduct numerical experiments on the proposed joints to obtain the global behavior of the connection and develop simplified models. Very good agreement was found between the simple and sophisticated models for strength, stiffness and energy dissipation capacity, verifying the robustness of the approach. The paper argues that careful analytical studies can replace the requirement for physical testing present in current steel codes.

1 BACKGROUND

Seismic design provisions for buildings have a long history in the USA, dating back to the Uniform Building Code of 1927. These provisions were steadily but slowly improved until the Applied Technology Council (ATC) issued a comprehensive set of seismic design provisions following the 1971 San Fernando Valley Earthquake (ATC 3-06, 1978). This effort has continued through the issuance of updated provisions by the National Earthquake Hazard Reduction Program (NEHRP). These provisions are developed through the Provisions Update Committee of the Building Seismic Safety Council (PUC/BSSC) (NIBS 2010). They were originally intended to serve as a model and resource code, but were adopted *in toto* into the 2000 version of the International Building Code, the first unified building code in the USA (IBC 2010).

Currently seismic loading provisions have been consolidated into ASCE 7-10 (ASCE 7 2010), while provisions for design of members, connections and structural systems have been remanded to material organizations. Provisions for steel structures began to be developed by the American Institute of Steel Construction (AISC) in the mid 1990s in an effort to provide a single set of consistent specifications for seismic design of metal structures. At that time PUC/BSSC has a small working group on composite structures which had produced a short draft on seismic design of a number of composite members and structural systems (NEHRP 1994). AISC decided to incorporate these provisions for composite structures as Part II of its first seismic design specification (AISC 1997). The structure of the AISC Seismic Specification remained unchanged until its last edition (AISC Seismic 2010), with Part I being a comprehensive and prescriptive set of rules for steel structures and Part II being a less complete and more performance-based document for composite members and structural systems.

Another important development in seismic design over the last 5 years has been the introduction of performance-based design (PBD) requirements for tall buildings. This development took place essentially because designers did not think that a number of the conventional provisions, particularly those requiring a backup structural system with a capacity to resist 25% of the design base shear, made sense structurally and economically for tall buildings. In the very hot property market that preceded the economic collapse of 2007, there were several dozen tall buildings being planned for high seismic cities such as San Francisco and Los Angeles. Designers for these structures proposed a series of innovative structural systems and connections that were not contemplated by the prescriptive provisions of existing codes. To attempt to provide some basic guidance as to how to demonstrate the validity of these systems, both the cities of San Francisco and Los Angeles issued guidelines on how to use performance-based design and similar concepts to justify innovative designs (LA 2005, SF 2009). This paper briefly proposes how the 2010 AISC Specification can be used, in conjunction with existing PBD guidelines, to permit the use of innovative connections without the need to perform extensive full-scale connection qualification tests

2 THE 2010 AISC SEISMIC SPECIFICATION

2.1 *Major Changes*

As usual with new versions of any specification, the AISC 2010 Seismic Provisions (AISC Seismic 2010) contains a number of new and evolutionary design requirements that reflect the latest in seismic research and practice. In addition, however, there have been two major changes. The first change, which can be considered editorial in nature, is that wherever possible, the provisions now have a consistent set of subsections for all seismic structural systems. These subsections include scope, basis for design, analysis requirements, structural system requirements, member design, and connection provisions. This new organization considerably simplifies the design process and makes possible quick comparisons between alternative structural solutions.

The second change, which is more philosophical in nature, is that the old Part I (Steel Construction) and Part II (Composite Construction) have been completely merged into a single document. For composite structures, this has had two major consequences. First, this approach makes composite design essentially equivalent to traditional steel insofar as expected performance is concerned. This is an important result because composite construction is not considered a viable option in the USA except for tall structures. It is hoped that its inclusion into a single design document with steel structures will make more designers consider composite systems for low- and mid-rise construction. The second consequence is that the old Part II provisions, which were intentionally nebulous in handling many design details, needed to be updated and rendered far more prescriptive in nature. The original NEHRP composite provisions, issued by NEHRP in 1994, were developed as enabling legislation for composite systems, and permitted the use of systems that had neither been tested nor built at the time.

The 2010 AISC main specification for steel structures (AISC 2010) provides much of the background to the seismic provisions and, as its predecessors, contains two important additional statements that are relevant to this paper. First, in its Preface it states that: "*The intention is to provide design criteria for routine use and not to provide specific criteria for infrequently encountered problems, which occur in the full range of structural design.*" Second, in its Scope, it states that: "*Where conditions are not covered by the Specification, designs are permitted to be based on tests or analysis, subject to the approval of the authority having jurisdiction. Alternative methods of analysis and design are permitted, provided such alternative methods or criteria are acceptable to the authority having jurisdiction.*" (AISC 2010).

Taken together, these two statements provide an escape route for designers wishing to circumvent prescriptive design procedures, but leave the responsibility to accept such approaches to the building officials. The PBD documents from the cities of SFO and LA cited previously aim at providing designers with some guidance on how to fulfill these requirements.

2.2 *Member and Connection Design*

General requirements for member and connection design are covered in Section D of the new AISC 2010 Specification. Insofar as member design is concerned (Section D1), the most important change in the new edition is the explicit recognition, and associated requirements, for three types of seismic member categories: conventional, moderately ductile and highly ductile. Conventional members need only comply with the requirements of the main specification (AISC 2010), while the seismic specification gives detailed requirements for moderately and highly ductile members primarily in terms of width-to-thickness ratios, bracing, and protected zone provisions.

Insofar as this paper is concerned, the most important provisions regarding connection design are given in both the general section on connections (Section D2) and on Section E3 (Special Moment Frames). The majority of the important details for this paper are given in Section E3. Connections in special moment frames are required to provide deformation capacity through yielding of beams and panel zone deformations. For steel connections, it is required that (a) the connection shall be capable of accommodating a story drift angle of at least 0.04 rad., and (b) the measured flexural resistance of the connection, determined at the column face, shall equal at least 0.80Mp of the connected beam at a story drift angle of 0.04 rad. Compliance with these requirements can come from:

1. Use of connection prequalified as per AISC 358 (2005). AISC 358 currently prequalifies 7 types of connections based on extensive testing and analysis, primarily from results of the SAC research program (FEMA 2000).
2. Testing as per Appendices K1 and K2 of the seismic specification, which require at least two full-scale connections tests under a stringent displacement history.

It is worthwhile to notice that AISC requires physical testing, and that the testing needs to be supplemented by advanced analyses.

3 PEFORMANCE BASED DESIGN (PBD) GUIDELINES

As noted earlier, there have been two initiatives aimed at promoting the use of performance-based design approaches to seismic design. They include the recommendations of (a) a committee of the Structural Engineers of Northern California, which resulted in a building ordinance for the city of San Francisco entitled *Requirements and Guidelines for the Seismic Design and Review of New Tall Buildings using Non-Prescriptive Seismic-Design Procedures* (SFO 2007), and (b) the Los Angeles Tall Building Design Council which resulted in a design guideline entitled *An Alternative Procedure for Seismic Analysis and Design of Tall Buildings Located in the Los Angeles Region* (LA 2005).

Only the LA document is truly a PBD document as it states clearly in its scope that its approach "*is based on capacity design principles followed by a series of state-of-the-art performance based design evaluations*" that aims to provide serviceable behavior under frequent events (43 year return period, or 50% probability of exceedance in 30 years) and very low probability of collapse under extreme events (2475 year return period, or 2% probability of exceedance in 50 years). The document provides analyses requirements in terms both of the model and load histories to be used. It focuses on the demand side and as a default, suggests using the member capacities established by ASC 41 (ASCE 41) to check performance. It adds an exception that "larger deformation capacities may be used only if substantiated by appropriate laboratory tests and approved by the Peer Review Panel and the Building Official."

The SFO document also utilizes the same two-level approach as the LA one, but intentionally avoids the use of PBD terminology and contains a mix of mandatory and non-mandatory provisions. It focuses on the demand side and is silent on the need for testing to justify the strength and deformation capacity of members and connections.

4 INNOVATIVE CONNECTIONS

The previous sections describe the current state of seismic design codes for composite structures in the

USA. The rest of this paper describes the how development of an innovative connection can fit within these frameworks without having to carry out extensive physical testing. The connection under consideration combines the robustness and structural efficiency of concrete-filled tube (CFT) columns with the ductility and recentering capabilities of a partially restrained (PR) connection (Fig. 1). The key innovative part of the connection is the use of a combination of mild steel and shape memory alloy (SMA) through rods as the connecting elements. The mild steel rods provide energy dissipation, while the SMA rods provide recentering capability (Hu and Leon 2010).

This type of connection is applicable in special and intermediate composite moment-resisting frames up to about 10 stories, and combines a number of the advantages associated with composite systems. For CFT columns, these advantages include the fact that the concrete prevents local buckling of the steel tube wall and the confinement action of the steel tube extends the usable strain of the concrete. CFT beam-columns and panel zones provide enhanced ductility and reduced rates of degradation under cycling at large drifts when compared to conventional steel or RC columns.

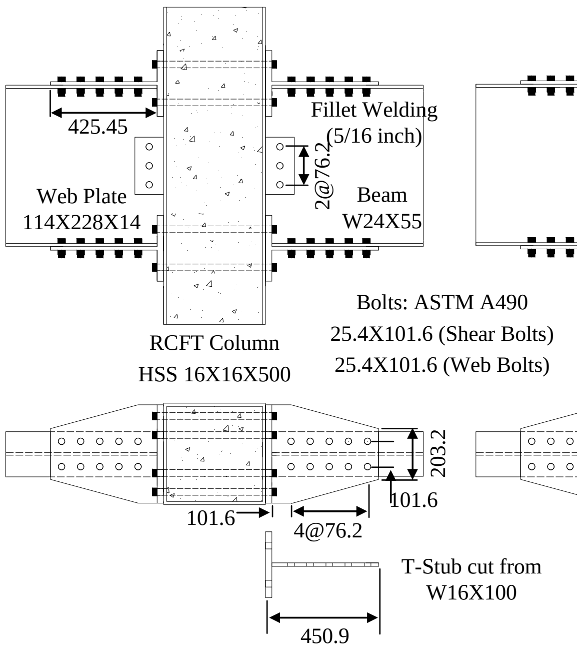


Figure 1 – PR composite connection

In order to utilize these connections in an actual structure, several obstacles need to be overcome as follows:

- The currently available PBD guidelines (see Section 3) will need to be extended to low-rise construction, as currently they are applicable only to structures over 49m in height. As the requirements for taller structures are more restrictive than those for lower ones, acceptance of this approach by progressive building officials is possible.
- The current requirement for full-scale physical testing of composite connections (see Section 2) will need to be waived and replaced by extensive advanced analytical studies. The rest of this paper looks at how this issue can be justified and the level of effort required.
- The use and scope of peer review panels need to be expanded as they need to become the main deciders, as opposed to building coded officials, of the technical merits of an innovative structural solution.

4.1 *Connection Modeling and Calibration*

The first step in justifying this approach is linked to the development of a simplified connection model whose components' cyclic behavior is well understood. In the case of the connection shown in Fig.1, these components are: (a) the steel beam; (b) the bolted T-stubs that provide the connections to the flanges; (c) the fin plate connection that provides shear transfer; (d) the mild steel and SMA rods that provide axial restraint; (e) the composite panel zone; and (f) the composite CFT column.

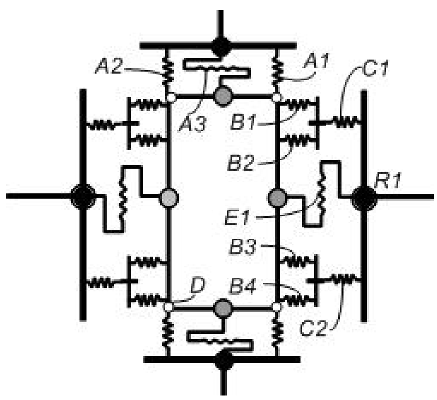


Figure 2 Simplified model of proposed connection

Fig. 2 shows a simplified generic model that is suitable to model this connection. In this model, springs C1 and C2 represent the T-stubs in tension and bearing, springs B1 through B4 represent the steel and SMA bars in tension and compression, springs D represent the joint shear deformation, and spring E the shear connection to the beam. Each of these springs can have its own backbone load-deformation curve, which can represent many forms of deformation. For example, spring C1 includes the deformations due to slippage of the bolts between the beam and T-stub, the axial deformation of the T-stub stem, the flexural and shear deformation of the T-stub flanges, and the ovalization of the bolt holes in bearing. In addition, the cyclic behavior and degradation rules for each spring can be set independently. These models can easily be implemented into a number of existing platforms; OpenSEES was used in this study (Mazzone et al. 2006).

The key question becomes whether the behavior of all of these springs can be accurately predicted. The answer, for this connection, is yes as the individual behavior of (a) T-stubs, including slip, bearing, and ovalization (Swanson 1999,), (b) composite panel zones (Wu et. al. 2007), and (c) SMA materials (Tami and Kitagawa 2002, Song et al. 2006) is well established.. In addition, behavior of both SMA in large connections (Leon et al. 2001, Ocel 2002, Penar 2005) and CFT columns member and panel (Hu et al. 2005, 2007) has been verified by a number of research efforts. Nevertheless, extensive calibration of the simplified model shown in Fig. 2 to advanced 3D FE models, such as that shown in Fig. 3, is a must. These models need to be very carefully developed to ensure proper meshing and boundary conditions. In addition, smaller detailed models of areas of stress concentrations (Fig. 4) also need to be examined to ensure that no possible failure modes are ignored. ABAQUS (2006) was the FE coded used herein.

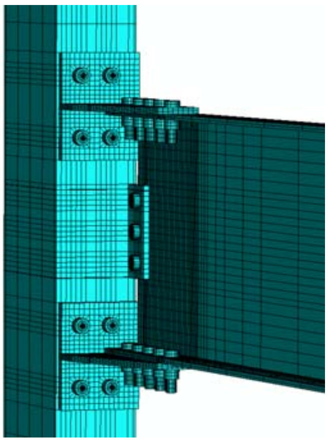


Figure 3 - 3D FE model of proposed connection

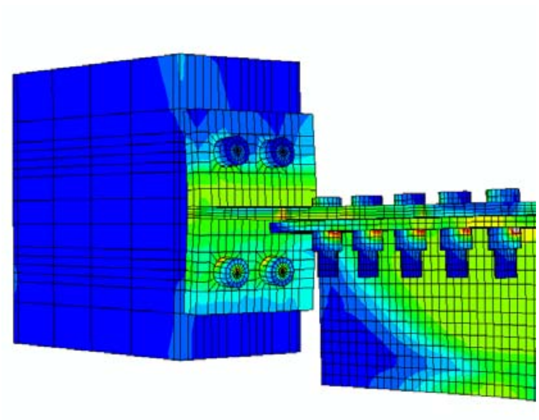


Figure 4 - Detailed FE model of critical connection region

The work requires that calibration of each individual mode of deformation be made to well-documented tests. Figs. 5 and 6 show comparisons for the axial deformations of the T-stub stem and slippage of the bolts for a T-stub specimen tested by Swanson (1999). These two deformation modes are built into spring C1 in Fig. 3. The models are capable of reproducing not only the behavior of components but also of entire connections. Fig. 7 shows a comparison between plastic rotations vs. moment for a connection tested by Swanson (1999) and a simplified model in OpenSEES. The model matches well all the test behavior both qualitatively and quantitatively.

For these connections another complicating factor is the modeling of the shape memory effect, as shown in Fig. 8. Ideally, the material shows a flag-shaped behavior as shown in Fig. 8(a), with a number of break points and trigger lines related to the heat treatment of the material. However, actual data for one quadrant shows (Fig. 8(b)) that these lines and points are not easy to characterize.

4.2 *Connection Design Issues*

The connection models described in the previous section have only been calibrated to a relatively small number of tests, so it is important that the limitations of the approach be understood. The first important limitation is that the simplified model basically does not consider interactions between failure modes. From the design standpoint this means that failure modes must be carefully separated and that a large number of yielding mechanisms must occur before any fractures or similar failures can take place. This can be accomplished by a well-thought out capacity design approach that clearly recognizes sources of over- and under-strength in the connection. For the connection in Fig. 1, this means that slippage of the bolts will occur first (clearly evident at about 1/3 of the connection strength in Fig. 7, with hole ovalization leading to larger deformations with cycling). This initial deformation mechanism should be followed by yielding of the beam, tensile yielding of the SMA and mild steel through rods, yielding of the T-stub stem in tension, flexural yielding of the T-stub flanges, and yielding of the panel zone in shear, roughly in that order. These failure modes should provide all the required ductility (on the order of 0.03 to 0.05 rad. of plastic deformation), with local buckling and local crushing of the concrete providing additional deformation capacity should plastic large rotations in excess of 0.05 rad. occurs. Under no circumstance should fracture of the through rods in tension or bolts in shear be permitted at loads less than 150% of those creating yielding mechanisms.

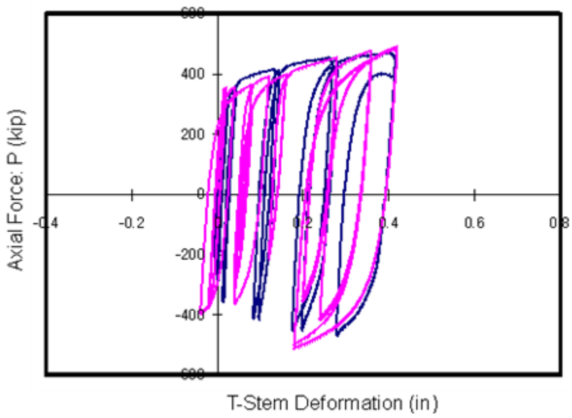


Figure 5 - Comparison of T-stub stem deformation from tests and simplified model

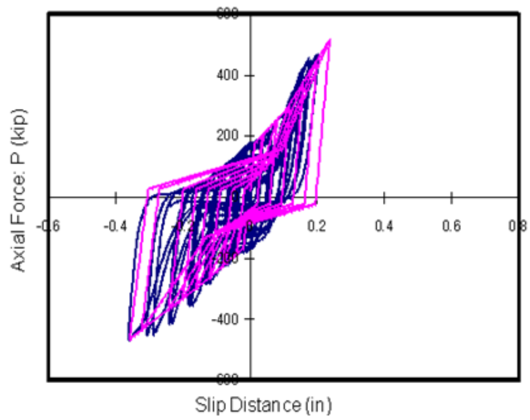


Figure 6 - Comparison of T-stub slip from tests and simplified model

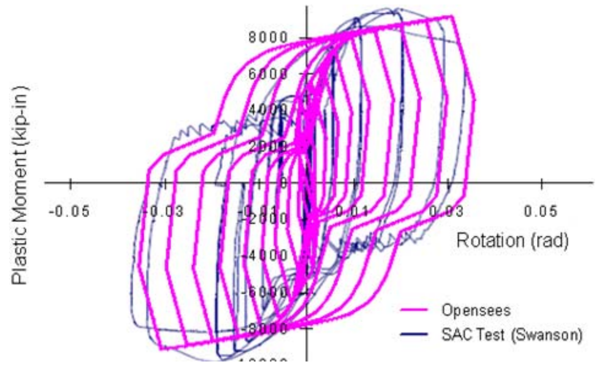
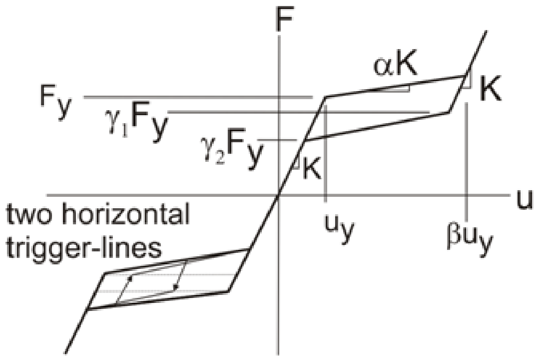


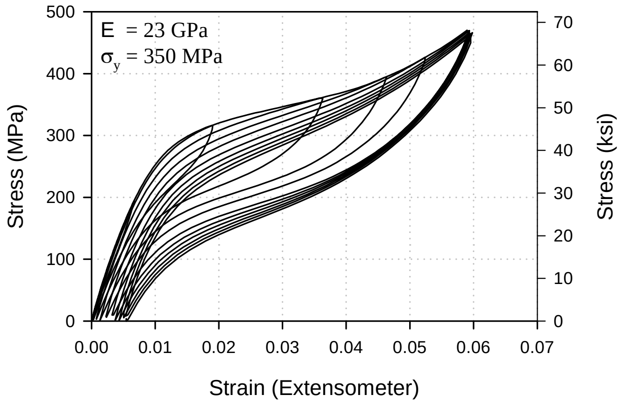
Figure 7 Comparison of measured and predicted moment vs. plastic rotations for a full-scale T-stub connection.

A second important limitation is that models such as those discussed here can only capture very limited strength and stiffness degradation. In particular, test results such as those shown in Fig. 7 indicate that, at large deformations, there is a gradual reduction of strength and stiffness due to local buckling in the beam flanges. Including this behavior in the connection model will require additional springs and the occurrence of negative stiffnesses from several degrading spring can lead to significant numerical problems for the entire spring assembly. A nice feature of this connection is that the through rods actually behave like buckling restrained braces as they are embedded in a plastic sheath with a diameter slightly larger than the bars itself, and thus their performance in compression is very similar to that in tension. This is true for the mild steel bars; there is some difference in tension and compression behavior of the SMA that is a function of the material processing. From the design standpoint degradation issues due to local buckling should be dealt with by specifying very compact sections for the beam flange and web, as well as for all other components subjected to yielding in compression. It should be noted that the large amounts of local buckling observed near the end of physical tests are more a creature of the boundary conditions from the test frame (i.e., lack of axial restraint in the beams) than a reflection of actual behavior.

A third important limitation of this approach is that it ignores the effect of the floor slab in redistributing stresses in the connection area. This issue has often been overlooked in seismic codes, but based on the work of Leon et al. (1998) and others, the latest AISC seismic specification explicitly requires that designers either account for this effect in their analysis and design or provide a small opening around in the slab around the column to limit those effects.



(a) Idealized shape memory behavior



(b) Actual behavior

Figure 8 Comparison of idealized and actual SMA behavior

An important fourth limitation is the need to arbitrarily impose some strain (or deformation) limitations on the load-deformation curves for the individual springs. Although a large number of tests have been carried out on individual components and these can serve as a basis for establishing such limits, there is little or no statistical work related to the potential for failure when several components show large deformations simultaneously. In a connection such as that under study, where a very large number of deformation mechanism are expected to con tribute to the overall connection ductility, this effect may be important.

A final problem that has not received much attention is the robustness of the analytical tools as very large deformations, and consequently very low stiffnesses, are reached. In most cases, it is impossible to ascertain the reasonableness of the results without detailed investigation of the quantitative contribution of each mechanism.

While these limitations are important, it should be recognized that they are not unique to the type of connections of interest here. In general, designers tend to dismiss them as either too difficult to handle in everyday design or as not significant to their design.

The design approach proposed here is based on establishing a prescribed series of yielding mechanisms and capacity design principles. The yielding of the through rods provides an ideal deformation mechanism, as the free length of the rods (around 40cm to 50cm in many cases) provides very large local deformations at reasonable low average strains. However, compliance with this approach is not simple as only a small number of fixed geometries are available for T-stubs cut from rolled sections. More recently, work on T-stubs made from welded plates has considerably eased that problem, and it is expected that in the very near future AISC 358 will sanction this approach.

4.3 *Frame Performance*

The last step in this design process is to evaluate the seismic performance of the overall structure through both pushover and non-linear dynamic analyses of a set of archetypical frames. This evaluation requires that comparisons be made to similarly designed, all-steel frames that comply with all the requirements present in applicable design codes. Details of these studies have been recently published (Hu and Leon 2010). Fig. 9 shows a comparison of the pushover analyses for a 6-story, 5-bay frame for a welded, all-steel special moment frame (6TSU-C7) and two frames using the connections shown in Fig. 1 (6TSU-C1 and C4). Based on initial stiffnesses, the innovative frames fall in the PR category. The welded frame shows less ductility because of weld fractures, which begin around 4% story drift. The frames with innovative connections show almost unlimited ductility and little sensitivity to P-Δ effects. Both frames show a substantial overstrength with respect to the design base shear because these frames are controlled by drift.

The frames were also subjected to the suite of SAC ground motions (SAC 2005). Fig. 10 shows typical results for the LA21 (Kobe 1995) ground motion. The welded frames show very significant residual deformations and a possible collapse at 12 sec. due to fractures in the first story connections. This conclusion is supported by the very large interstory drifts in the first floor at the maximum base shear (Fig. 11(b)). The frames with welded connection show significantly higher drifts, and likely collapses as the drifts exceed 0.1. The average and 84TH percentile interstory drifts for the frame with the innovative connections (Fig. 12), on the other hand, show large but contained deformations. A large number of frames were designed and studied using data similar to that in Figs. 10 through 12.

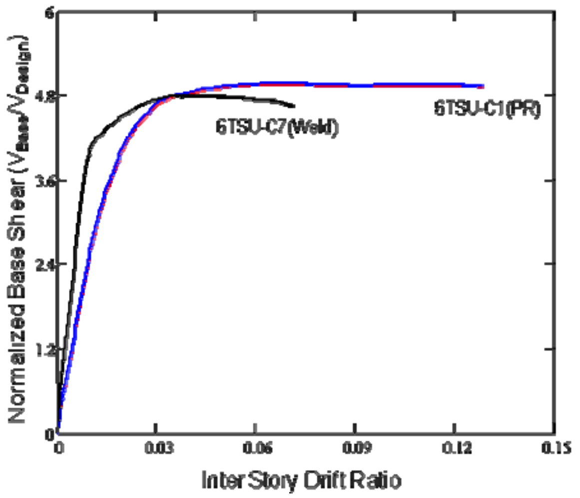
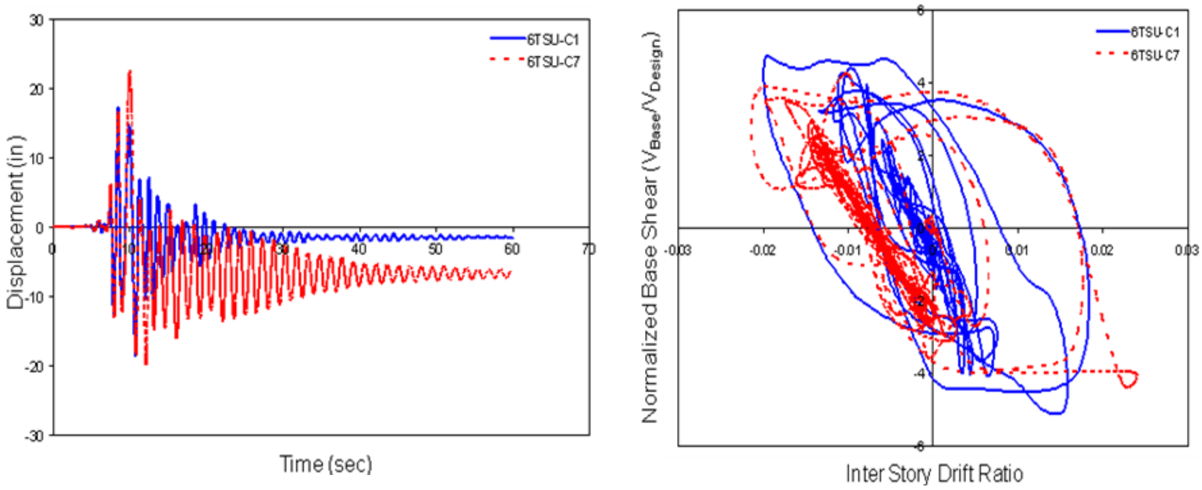


Figure 9 - Comparison of pushover behavior between frames with welded and innovative connections



(a) Roof displacement vs. time

(b) Base shear vs. average interstory drift

Figure 10 – Response of welded and innovative frames to the LA 21 ground motion

The results indicate that:

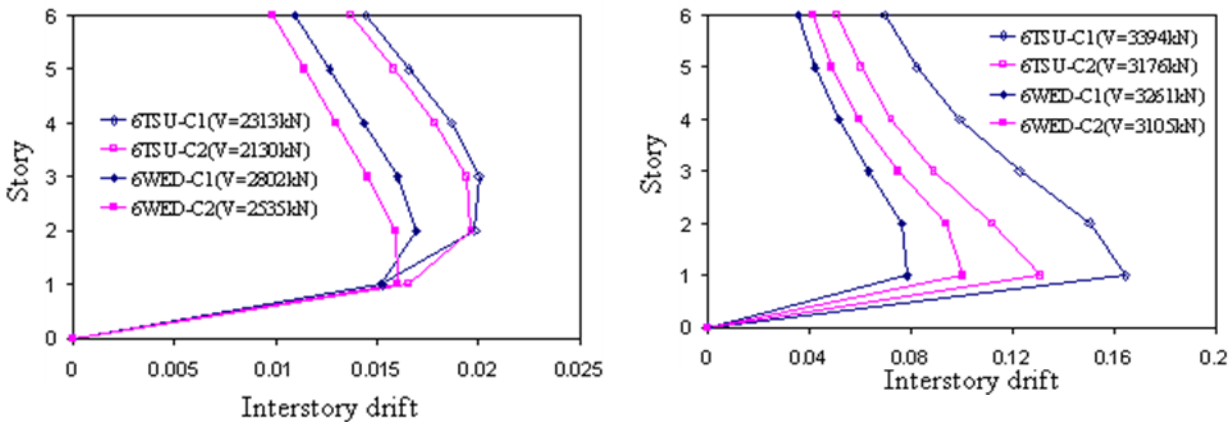
- Overall, the initial stiffness of the frames with welded connections was larger than that of those of the frames with PR connections. However, the strength of welded frames deteriorated very rapidly because the welded connections were susceptible to brittle failure and P-Δ effects. In contrast, the use of flexible tension bars in the innovative connections provided extra deformation capacity, so the composite frames with PR connections showed more gradual strength degradation. As expected, the strength of the taller frames deteriorated more rapidly than that of the shorter frame because of the large P-Δ effect due to the heavier gravity loads.

- Composite moment frames with PR connections showed smaller residual displacements than those with welded connections due to the recentering effect of the SMA bars. In addition, composite frames with PR connections showed a more gradual strength degradation. For the PR composite frames, the envelope of the cyclic curves corresponded to that of the monotonic curves when the same models were compared.

- Composite moment frames with welded connections showed the largest roof displacements. The outstanding energy dissipation properties of the PR connections resulted in lower drifts.

- Overall, maximum displacement, velocity, and pseudo-acceleration occurred at the roof. The occurrence time for each peak value subjected to the same ground motion was slightly different and trailed the occurrence of the peak ground acceleration.
- Statistical distributions of the peak interstory drifts were obtained from the dynamic analyses. The peak interstory drifts occurred at the first story level and decreased rapidly as one moved up the structure.

Both the analyses of individual connections and multi-story frames show that these innovative PR frames will display excellent performance when properly designed.



(a) At initial yield (b) At ultimate

Figure 11 – Comparison of drifts for welded and innovative frames.

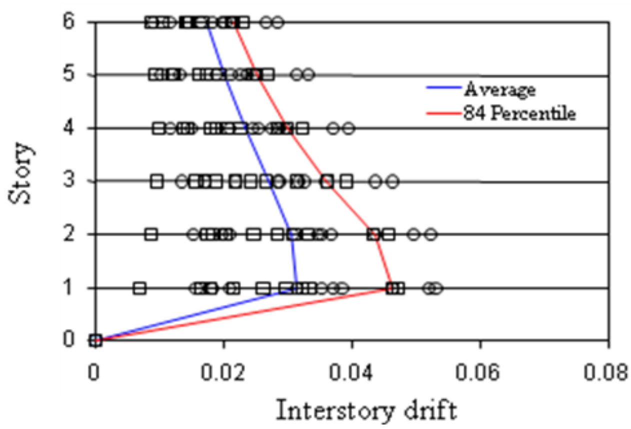


Figure 12 - Range of interstory drifts for frame subjected to a large suite of ground motions.

5 CLOSING REMARKS

This paper began with a description of some of the 2010 AISC seismic provisions for composite structures and of the LA and SFO PBD guidelines for tall buildings. It then described the detailed analyses carried out for the development of an innovative PR connection that utilizes a mix of steel and SMA rods as the main yielding mechanism. These analyses were intended to provide enough supporting evidence to convince building officials to waive the requirement for physical testing of full-scale connections present in current codes. The authors believe that careful application of the most advanced analytical tools available coupled with characterization of system response based on a large suite of ground motions provides equal or better justification than physical tests. It should be noted that physical tests have significant shortcomings at the very least with respect to the fact that (a) only two specimens are required (a statistical insignificant number); (b) only a slowly increasing set of deformation cycles are used (very different from what areal earthquake will produce); and (c) boundary conditions are highly idealized and do not correspond to actual designs (slabs and 3D effects are completely ignored in most tests) (Leon and Deierlein 1997). The authors are thus comfortable in concluding that one of the original purposes of the Network for Earthquake Engineering Simulation (NEES), the migration from pure physical testing to full-scale simulation, is well underway and succeeding.

6 ACKNOWLEDGEMENTS

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