NEW TRENDS FOR SEISMIC ENGINEERING OF STEEL AND COMPOSITE STRUCTURES

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ABSTRACT

This paper summarizes ongoing research and trends in practice for both practical and innovative steel and composite steel-concrete structural systems. Selected recent work in the United States will be reviewed and trends in the design specifications will be identified, including new tests and analyses on composite steel/concrete systems aimed at improving the current U.S. design provisions; and recent research on self-centering steel frame systems with replaceable energy-dissipating fuses that provide new strategies for ensuring sustainable structures after major hazardous events.

KEYWORDS

Articulated Fuse Systems, Composite, Design, Seismic, Self-Centering Systems, Steel

INTRODUCTION

The structural engineering profession has a history of constant innovation as engineers and researchers strive to increase the safety, economy, and performance of our built environment. One manifestation of this innovation is the path new structural systems take from novel concept to introduction and eventually full integration in codes and specifications.

Steel-concrete composite structural systems have recently received expanded attention in specifications within the United States. Composite members have been included in the American Institute of Steel Construction (AISC) steel specification since the 1936 *Specification for the Design, Fabrication and Erection of Structural Steel for Buildings* (AISC 1936). However, the provisions only addressed composite beams until the 1986 *Specification for Structural Steel Buildings – Load and Resistance Factor Design* (AISC 1986), when provisions for composite columns and beam-columns were added based upon research consolidated through the Structural Stability Research Council and related venues in the 1960's through the early 1980's. The provisions for composite systems underwent only incremental changes until major revisions and restructuring occurred for the 2005 *Specification for Structural Steel Buildings* (ANSI/AISC 360-05) for non-seismic design of composite structures. Further additions were made for the 2010 *Specification* (ANSI/AISC 360-10), mainly addressing the use of noncompact or slender tubes in concrete filled steel tube (CFT) members and expanded provisions on force transfer between the steel and concrete components. Similarly, the 2010 *Seismic Provisions for Structural Steel Buildings* (ANSI/AISC 341-10) have undergone major revisions and restructuring, including integral incorporation of composite systems into the provisions for the first time. The timeline in Figure 1 highlights these changes. This expansion of provisions over the last five years was a culmination of a surge of research activities on composite systems that occurred worldwide from the late 1980's through the last several years; within the U.S., this occurred especially as part of the U.S.-Japan Cooperative Earthquake Engineering Research Program sponsored by the National Science Foundation (NSF), focusing on composite and hybrid structures, and subsequently by projects sponsored within the NSF George E. Brown, Jr. Network for Earthquake Engineering Simulation (NEES) as well as the American Institute of Steel Construction.

Self-centering structural systems with replaceable energy-dissipating fuses are an example of a structural system that has yet to be included in the AISC specifications or related specifications within the U.S., but these systems are being developed at an increasing rate worldwide and thus discussions are underway about whether to incorporate these systems, and if so how best to do so, based on recent experimental and analytical research that has been conducted. As a representative example of a related innovative structural system that has been incorporated into the building codes, Figure 1 shows the timeline of inclusion of base isolation systems in the U.S. provisions through ASCE 7, *Minimum Design Loads for Buildings and Other Structures*. First incorporated in the 1995 standard after years of research, the provisions have been extended and use of these systems has been increasing in the U.S. and other countries.

This paper summarizes some of the recent key changes in the composite construction provisions, highlights recent research in composite construction that may form the basis of improved provisions in 2016, and discusses examples of some of the innovative systems that may form a next generation of systems available to structural engineers.

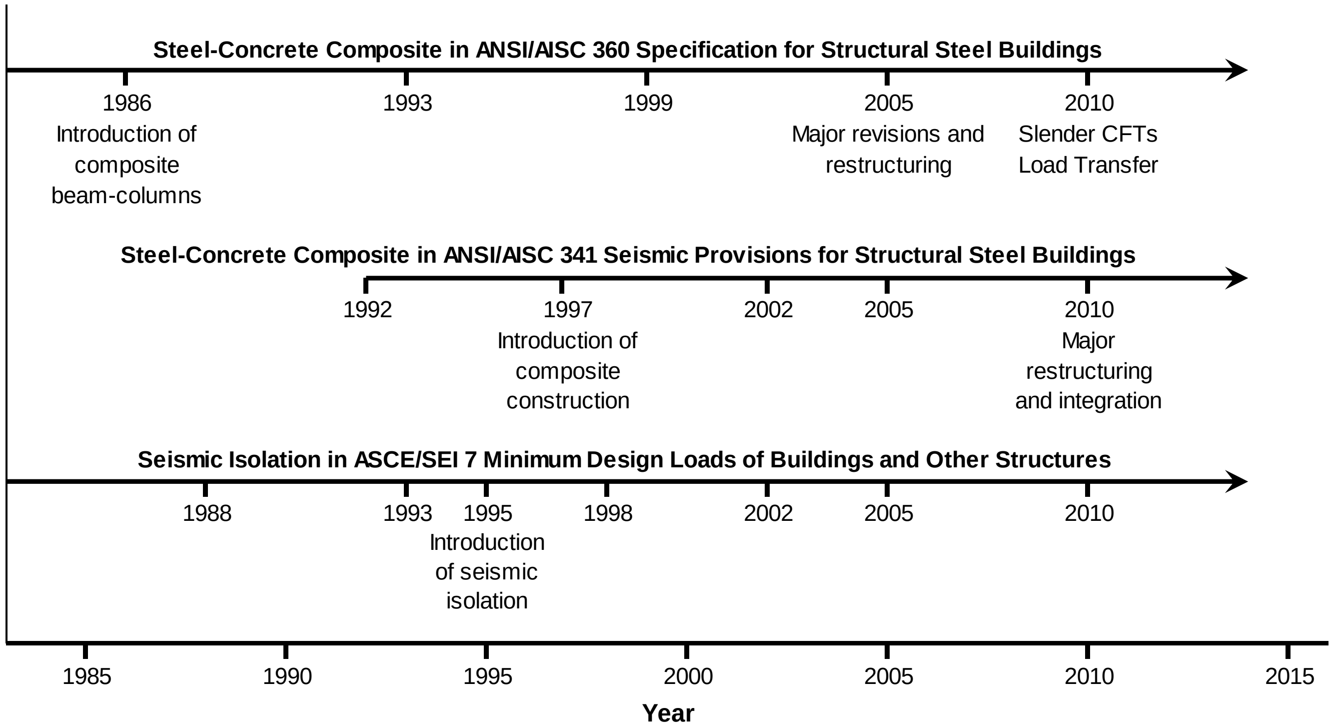


Figure 1: Timeline of Integration of Composite Columns and Base Isolation into Specifications

RECENT CHANGES IN THE STEEL-CONCRETE COMPOSITE PROVISIONS

Significant variations in behavior and progress of damage in composite members are possible due to the range of relative proportions of steel and concrete permissible in these members. Some composite beam-columns are concrete dominant and will behave more like reinforced concrete members while others are steel dominant and will behave more like structural steel members (Hajjar and Gourley 1996). Design provisions for composite members must account for this variation while simultaneously minimizing conflicts with structural steel and reinforced concrete provisions and recognizing the advantages of composite design. Numerous changes have been made to the U.S. design provisions over that past decade in light of these goals. Key changes are highlighted below.

Composite Member Strength Limit States

Prior to the 2005 *Specification for Structural Steel Buildings* (AISC 2005), the axial and flexural strengths of composite beam-columns were based on calculations that determined an equivalent steel section. This approach had limitations in that it was not applicable to columns with steel ratios below 4% and it often underestimated the contribution of the concrete, particularly for concrete-dominant composite beam-columns with low steel ratios (Griffis 2005). The beam-column strength interaction provisions in AISC (2005) are based more directly on mechanics principles. The cross section strength may now be determined using one of two methods: the plastic stress distribution method, which is applicable to most common composite column cross sections; and the more general strain-compatibility method, which is comparable to approaches often taken to compute reinforced concrete section strength. The compressive strength for axially loaded columns including length effects is then computed using a column curve that is algebraically identical to that for steel columns, and an effective stiffness, EI_{eff}, that is based on a curve fit to experimental data (Leon et al. 2007). As a result of the new methodology, the range of applicability of the provisions was extended to members with steel ratios as low as 1%.

The 2010 *Specification* provides clarified requirements for assessing compact composite members subjected to combined flexure and axial force. A number of design methods satisfy these requirements and the commentary to the specification identifies three in particular. The first method involves utilizing the interaction equations derived for structural steel members. This method is simple although conservative, as it typically underpredicts the contribution of the concrete. This is the method required for CFT members with noncompact or slender tubes, as discussed in the next section. The second method utilizes the plastic stress distribution method. The nominal strength interaction surface of the section is determined using the plastic stress distribution at several points along the interaction curve, and length effects are accounted for using a reduction factor on axial strength at all points on the interaction surface that is calculated based on the case of pure compression (Leon and Hajjar 2008). The third method is an approach presented in AISC Design Guide 6 (Griffis 1992), not presented here for brevity.

Slenderness Limits for CFT Members

The concrete core of concrete filled steel tube (CFT) members prevents inward buckling of the steel tube, delaying local buckling as compared to hollow tubes members. This was not recognized in the 1999 *Specification*, where tube slenderness limits were identical for both hollow and filled steel tubes. Provisions in the 2005 *Specification* adjusted the slenderness limits but only allowed for compact CFT members. Based on the work of Varma and Zhang (2009), new slenderness provisions are included in the 2010 *Specification*, allowing for the use of noncompact and slender tubes in CFT members. Elastic local buckling behavior of filled tubes (accounting for the difference in the shape of the local buckling mode between hollow and filled tubes) informs both the noncompact/slender limit and the critical stress in the slender range. While compact steel tubes are assumed to provide enough confinement to the concrete core so that it develops a compressive stress of $0.85f'_c$ for rectangular CFTs or $0.95f'_c$ for circular CFTs (where f'_c is the specified compressive strength of the concrete), noncompact and slender steel tubes are assumed to be unable to provide adequate confinement after the concrete core reaches a compressive stress $0.70f'_c$.

These assumptions lead to stress distributions through the section from which axial and flexural strength may be computed. As a representative example, the flexural strength provisions for RCFT members will be presented in detail. The member is classified as either compact, noncompact, or slender based on slenderness ratios (TABLE 1). For a member to be classified as compact, all of the components (i.e., both the web and flange) need to be compact, and similarly for being classified as noncompact. Once the member is classified, the nominal flexural capacity, M_n, is computed as described below and using the stress distributions shown in Figure 2. The provisions result in a typical relationship between strength and slenderness as shown in Figure 3. The resistance factor is $\phi_b = 0.90$.

TABLE 1
LIMITING WIDTH-TO-THICKNESS RATIOS OF RCFT MEMBERS SUBJECTED TO FLEXURE

Description of Element	**Width-to-thickness ratio**	**λ_p Compact/ Noncompact**	**λ_r Noncompact/ Slender**	**Maximum Permitted**
Flange of HSS	b/t	$2.26\sqrt{E/F_y}$	$3.00\sqrt{E/F_y}$	$5.00\sqrt{E/F_y}$
Web of HSS	h/t	$3.00\sqrt{E/F_y}$	$5.70\sqrt{E/F_y}$	$5.70\sqrt{E/F_y}$

For compact sections:

$$M_n = M_p \tag{1}$$

where

M_p = moment corresponding to the plastic stress distribution (Figure 2).

For noncompact sections:

$$M_n = M_p - \left(M_p - M_y\right)\left(\frac{\lambda - \lambda_p}{\lambda_r - \lambda_p}\right) \tag{2}$$

where

λ, λ_p, and λ_r = slenderness ratios determined from TABLE 1. In the case of both the web and flange are noncompact, the slenderness values are those which produce the lowest strength.

M_y = yield moment corresponding to yielding of the tension flange and first yield of the compression flange. It is calculated assuming a linear elastic stress distribution with the maximum concrete compressive stress limited to $0.7f'_c$ and the maximum steel stress limited to F_y (Figure 2).

For slender sections: M_n, is determined as the first yield moment. The compression flange stress shall be limited to the local buckling stress, F_{cr}, determined using Equation 3. The concrete stress distribution is taken as linear elastic with the maximum compressive stress limited to $0.70f'_c$ (Figure 2).

$$F_{cr} = \frac{9E_s}{\left(\frac{b}{t}\right)^2} \tag{3}$$

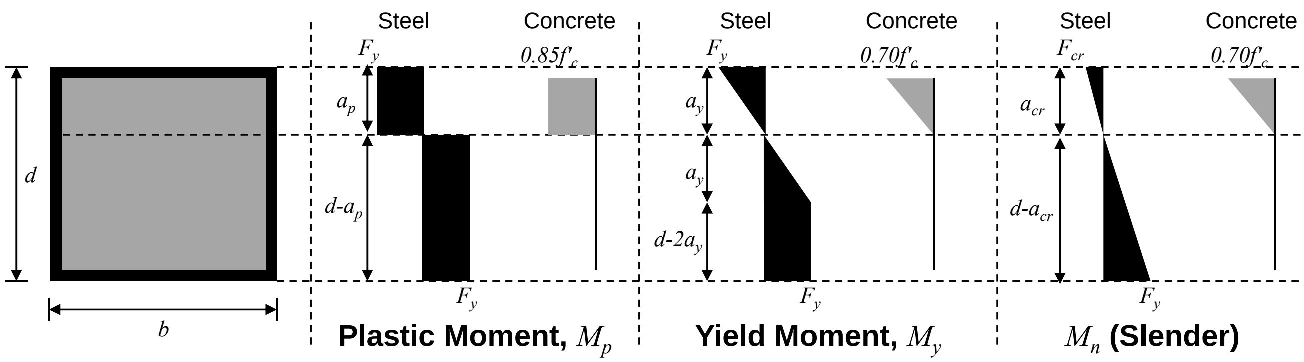


Figure 2: Stress Distribution for RCFT Subjected to Flexure (after Varma and Zhang 2009)

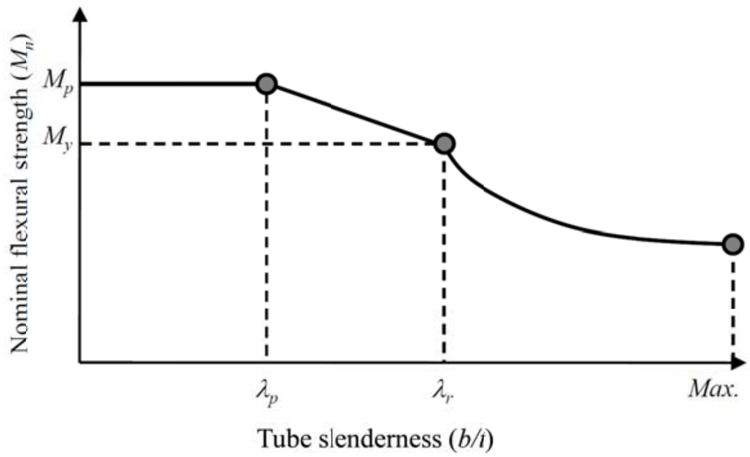


Figure 3: Nominal Flexural Strength of RCFT versus Slenderness (after Varma and Zhang 2009)

Force Transfer for Composite Members

To ensure the beneficial effects provided by steel-concrete composite members, it is important to have an understanding of the behavior of the interface between the two materials. Force transfer can be achieved through a variety of mechanisms; one of the most common is though headed steel stud anchors. Headed steel stud anchors have been predominantly used in composite beams, so much so that in previous versions of the *Specification* they were referred to as shear studs or shear connectors. Recent research by Pallarés and Hajjar (2010a, 2010b) sought to expand and enhance the provisions for headed steel stud anchors for other forms of composite construction including: composite beam-columns, boundary elements of composite wall systems, and composite connections. Prior experimental studies on steel anchors subjected to shear, tension, and combined loadings were reviewed, and various design recommendations and provisions were evaluated.

Results from this study were implemented into the 2010 *Specification* and are summarized here. The provisions directly address strength limit states for the steel shank of the anchor and for concrete breakout in shear. Limitations on the spacing and dimensions of the anchors are provided to preclude the limit states of concrete pryout and concrete breakout in tension. Specifically, to use these provisions, the aspect ratio of the steel anchor (ratio of length to width) must be greater than or equal to 5 for anchors subjected to shear or 8 for anchors subjected to tension or combined tension and shear.

The shear strength of steel headed stud anchors in composite components, Q_{nv}, is given by Equation 4. The associated resistance factor is $\phi_v = 0.65$. The engineer is expected to determine whether or not the concrete breakout is an applicable limit state. In most cases in composite construction, all nearby edges are uniformly supported in a way that prevents the possibility of concrete breakout failure. For cases where concrete breakout is applicable, anchor reinforcement is required and the nominal strength of the anchor is the minimum of the strength of the anchor reinforcement and Equation 4. Alternatively the anchorage provisions of the concrete specification (ACI 318 2008) may be used.

$$Q_{nv} = F_u A_{sa} \tag{4}$$

where

F_u = specified minimum tensile strength of the steel headed stud anchor
A_{sa} = cross-sectional area of steel headed stud anchor

The tensile strength of steel headed stud anchors in composite components, Q_{nt}, is given by Equation 5. The associated resistance factor is ϕ_v = 0.75. For cases where an anchor is located within 1.5 times its height to a free edge or 3 times its height to another anchor, anchor reinforcement is required and the nominal strength of the anchor is the minimum of the strength of the anchor reinforcement and Equation 5. Alternatively the anchorage provisions of the concrete specification (ACI 318 2008) may be used.

$$Q_{nt} = F_u A_{sa} \tag{5}$$

The interaction strength of steel headed stud anchors in composite components is assessed with Equation 6. Again, where concrete breakout in shear is applicable or edge conditions or group effects exist, anchor reinforcement should be provided or ACI 318 (2008) should be used.

$$\left[\left(\frac{Q_{rt}}{\phi_t Q_{nt}}\right)^{5/3} + \left(\frac{Q_{rv}}{\phi_v Q_{nv}}\right)^{5/3}\right] \leq 1.0 \tag{6}$$

where

Q_{rt} = required tensile strength
Q_{rt} = required shear strength

Composite Seismic Provisions

The first edition of the AISC *Seismic Specification* published in 1992 did not address the design of composite systems. Subsequent editions in 1997, 2002, and 2005 included composite systems in Part II of the specification. In the 2010 edition of the *Seismic Specification*, Part II has been eliminated and both steel and composite lateral force resisting systems have been integrated in the main provisions (Figure 1). Composite systems, for example special moment frames with CFT or steel reinforced concrete (SRC) columns, are expected to exhibit overall behavior that is similar to the corresponding structural steel system, since inelastic deformations will occur in much the same way, (i.e., flexural yielding of the girders in moment frames; brace buckling in brace frames) (Malley 2010). Recent technical changes include a requirement that cracked section properties be used in elastic analyses of composite structural systems and an increase in detail of the design requirements for composite systems such that they are consistent with structural steel systems.

RECENT RESEARCH ON STEEL-CONCRETE COMPOSITE SYSTEMS

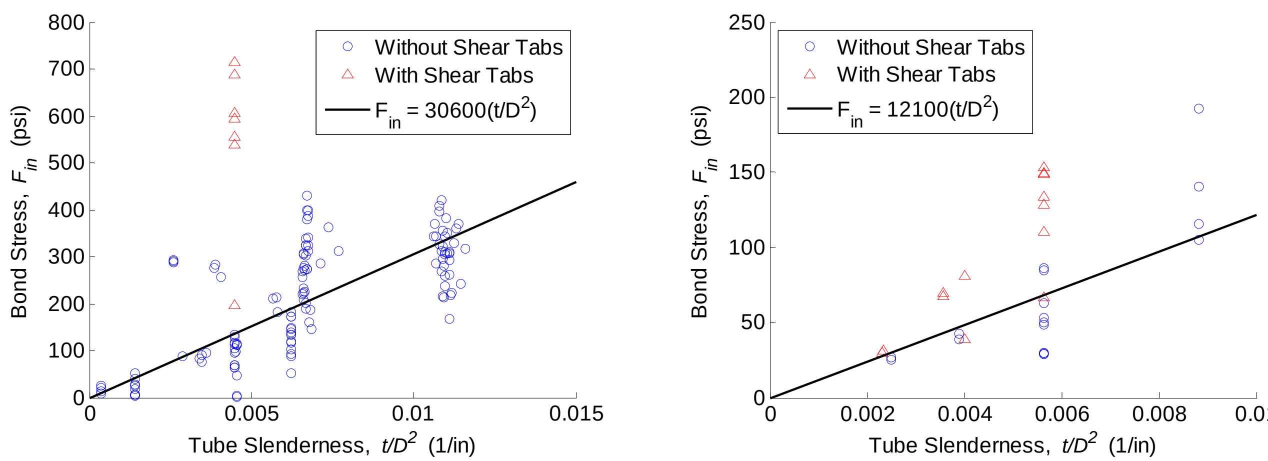
Natural Bond Behavior for CFT Members

Transfer of stress through natural bond, without the use of steel stud anchors or bearing mechanism, is often the most economical connection detail for CFT columns. Recent work by Zhang et al. (2011) has found current provisions on the natural bond strength to be conservative when compared to available experimental results and proposes new provisions which better capture observed behavior. In the 2010 *Specification*, a constant critical bond stress is assumed to act over a portion of the steel-concrete interface. Two main changes are proposed. The first is a critical bond stress that varies with tube dimensions and computed from an empirical expression fit to results of push-out tests of circular and rectangular CFT members (Figure 4). As observed in the experimental results, the proposed critical bond stresses are greater for smaller and thicker tubes and smaller for larger and thinner tubes. In addition, push-out tests where the steel tube was supported by shear tabs (as opposed to bearing on the entire tube cross-section) exhibited greater bond strength. This increase is attributed to increased normal stresses at the steel-concrete interface due to rotation of the shear tab during loading. The second change is an increase in the effective bond transfer area, determined from on an examination of experimental observations and results from specially instrumented connection tests. These changes are embodied in the recommended nominal bond strength, R_n, given in Equation 7; the associated recommended resistance factor is $\phi = 0.45$.

$$R_n = pDC_{in}F_{in} \tag{7}$$

where

p = entire perimeter of the steel-concrete interface
D = maximum width of the steel section for RCFT
= diameter of the steel section for CCFT
C_{in} = 2 if the CFT extends to one side of the point of force transfer
= 4 if the CFT extends to both sides of the point of force transfer
F_{in} = 12100 (t/D^2) for RCFT
= 30600 (t/D^2) for CCFT
F_{in} is in psi and t and D are in inches.



(a) CCFT (b) RCFT

Figure 4: Bond Stress for CFT as a Function of Tube Slenderness (after Zhang et al. 2011)

Slender CFT Beam-Column Experiments

Numerous experimental studies have been conducted on steel-concrete composite columns over the past few decades. These studies have been cataloged in databases (Aho and Leon 1997; Kim 2005; Goode 2008; Gourley et al. 2008). A review of these databases identified gaps in the experimental data, namely there have been few tests of slender members and slender sections (i.e., high D/t ratio). An experimental study was conducted to fill these gaps (Perea 2010). A series of 18 full-scale CFT beam-columns (Figure 5a, TABLE 2) were tested under three-dimensional loading.

The tubes were instrumented during concrete placement to study the effects of wet concrete on RCFT members. The hydrostatic pressure of the wet concrete causes the tubes to bulge leaving initial deformations and stresses. These were shown to cause premature local buckling of the steel tube as compared tubes that were stiffened during pouring. Further analytical studies we conducted to derive stress and deformation limits, above which stiffeners should be used during pouring or special measures should be taken during design.

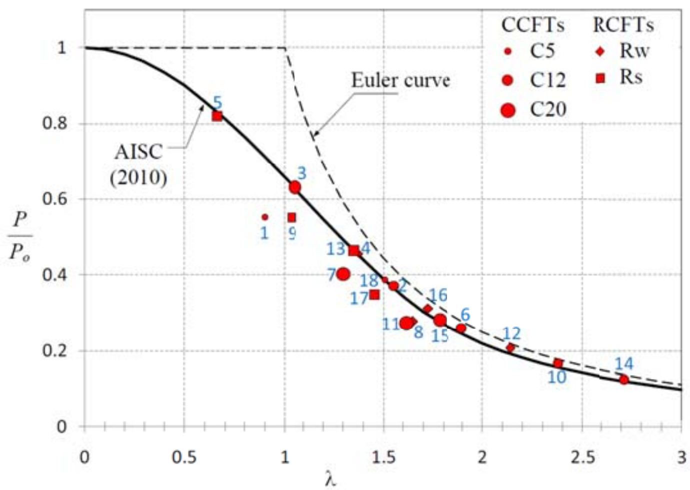
The specimens were subjected to long and varied loading histories which included concentric axial loading, non-proportional loading subjecting the specimen to cyclic axial force plus uniaxial bending, non-proportional loading subjecting the specimen to cyclic axial force plus biaxial bending, and cyclic torsional loading. Results from the first loading case, concentric axial loading, allow for a comparison to the column strength curve (Figure 5c). In this comparison, experimental data was processed to eliminate the effect of non-ideal boundary conditions; in addition, where the axial strength of the specimen exceeded that of the testing facility, the tangent plot method was employed to extrapolate the buckling load (Perea 2010). The experimental results show good agreement with the design provisions.

TABLE 2
CFT SPECIMEN TEST MATRIX (AFTER PEREA 2010)

Specimen	HSS steel							Concrete		
	L (mm) measured	D (mm) measured	t (mm) measured	D/t measured	F_y (MPa) coupon	F_u (MPa) coupon	E_s (MPa) coupon	f'_c (MPa) measured	E_c (MPa) measured	f_t (MPa) measured
1-C5-18-5	5,499	141	3.17	44.64	383	488	193,984	38.2	27,579	7.58
2-C12-18-5	5,499	324	5.91	54.84	337	446	199,162	38.4	27,579	7.58
3-C20-18-5	5,525	508	5.91	86.02	328	471	200,262	40.2	27,579	7.58
4-Rw-18-5	5,537	508x305	7.38	68.82	365	502	202,375	40.4	27,579	7.58
5-Rs-18-5	5,537	508x305	7.38	68.82	365	502	202,375	40.6	27,579	7.58
6-C12-18-12	5,499	324	5.91	54.84	337	446	199,162	90.8	41,851	11.38
7-C20-18-12	5,534	508	5.91	86.02	328	471	200,262	91.1	41,851	11.38
8-Rw-18-12	5,553	508x305	7.38	68.82	365	502	202,375	91.4	41,851	11.38
9-Rs-18-12	5,553	508x305	7.38	68.82	365	502	202,375	91.7	41,851	11.38
10-C12-26-5	7,950	324	5.91	54.84	335	470	200,210	54.5	34,474	4.14
11-C20-26-5	7,995	508	5.91	86.02	305	477	201,699	55.7	34,474	4.14
12-Rw-26-5	7,957	508x305	7.38	68.82	406	534	200,114	56.5	34,474	4.14
13-Rs-26-5	7,969	508x305	7.38	68.82	383	505	200,176	57.3	34,474	4.14
14-C12-26-12	7,950	324	5.91	54.84	383	461	191,419	80.1	39,990	5.24
15-C20-26-12	7,976	508	5.91	86.02	293	454	200,134	80.3	39,990	5.24
16-Rw-26-12	7,976	508x305	7.38	68.82	381	506	200,486	80.4	39,990	5.24
17-Rs-26-12	7,976	508x305	7.38	68.82	380	496	200,086	80.5	39,990	5.24
18-C5-26-12	7,976	141	3.17	44.64	383	488	193,984	80.7	39,990	5.24

(a) RCFT Specimen

(b) Local Buckling Observed during Testing

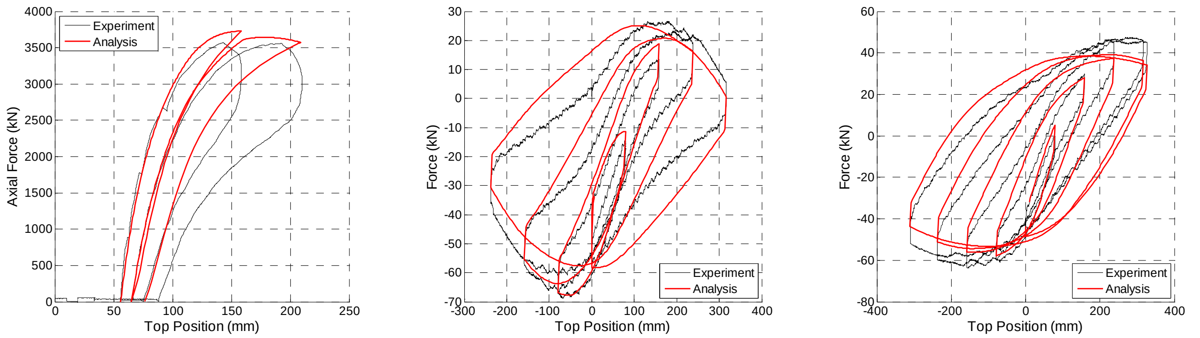


(c) Experimental Critical Load Ratios from Processed Data

Figure 5: Specimen and Results from the Slender CFT Beam-Column Experiments (after Perea 2010)

Nonlinear Modeling of Composite Members and Frames

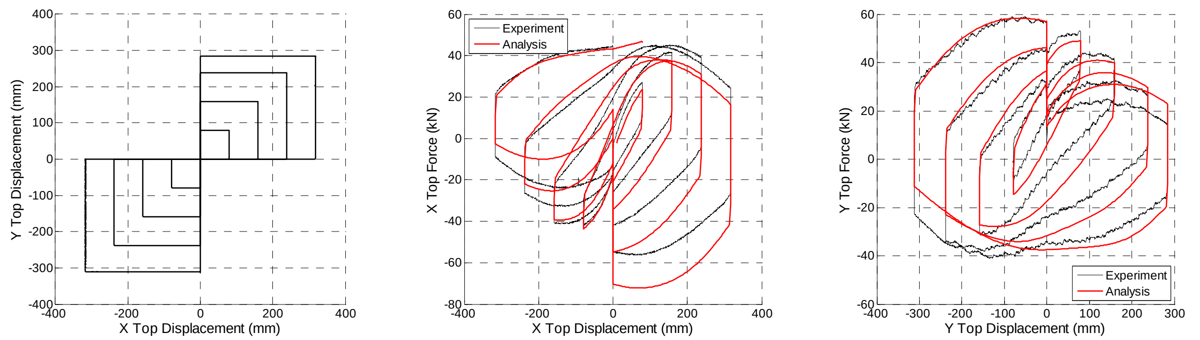
The ability to perform accurate nonlinear simulations is a key component in the assessment of the behavior of seismic force resisting systems. Building off prior work (Tort and Hajjar 2010, Denavit and Hajjar 2010), an accurate nonlinear model for composite members and frames has been developed. Key components of the model are a three-dimensional distributed plasticity mixed beam finite element and uniaxial cyclic constitutive relations for the concrete and steel materials. The model is capable of performing static and dynamic analyses of frame structures consisting of RCFT, CCFT, SRC, WF, or rectangular HSS members. The accuracy of the model is validated against a comprehensive set of results from monotonically and cyclically loaded specimens from the literature. An example of the validation is shown in Figure 6 for one of the slender beam-column tests specimens presented above. The formulation was developed and is suitable for use in large scale parametric studies to develop design recommendations for composite members and frames. Three specific studies are planned and will be carried out using this formulation. The studies aim to answer the most pressing questions related to design of composite members and frames.



(a) Load Case 1
Concentric Loading

(b) Load Case 2
Uniaxial Cyclic (P = 2,669 kN)

(c) Load Case 3
Uniaxial Cyclic (P = 1,334 kN)



(d) Load Case 4
Biaxial Cyclic (P = 2,002 kN)

(e) Load Case 4
Biaxial Cyclic (P = 2,002 kN)

(f) Load Case 4
Biaxial Cyclic (P = 2,002 kN)

Figure 6: Comparison of Experimental and Computational Results for Specimen 11-C20-26-5

Equivalent stiffness values for composite columns are used in elastic analyses to determine the fundamental frequencies of vibration of a structure, as well as seismic force and deformation demands. Such recommendations should account for the effect of material nonlinearity, most notable concrete cracking, on the average frame behavior. Recommendations will be developed through comparisons between computational results from static and dynamic analyses of composite frames and elastic analyses utilizing equivalent stiffness values.

The direct analysis method provides a more straightforward and accurate way of addressing frame in-plane stability considerations than traditional effective length factor methods (White et al. 2006). In this method, required strengths are determined with a second-order elastic analysis where members are modeled with a nominal reduced elastic stiffness and a nominal initial out-of-plumbness (the initial out-of-plumbness is often modeled using notional lateral loads). However, to date, no procedure has been established to determine appropriate reduced elastic stiffness values for composite beam-columns. Design recommendations of this type will be developed and validated against computational results from the static analyses of small sensitive benchmark frames.

Seismic performance factors are used to account for inelastic dynamic behavior in a design method which predominantly employs static elastic analysis techniques. However, the response modification factor for composite systems has been somewhat arbitrarily assigned. Using the methodology that was recently developed by the ATC-63 project (FEMA 2009), seismic performance factors will be determined for composite lateral force resisting systems. The specific structural systems of interest are composite special moment resisting frame and composite special concentrically braced frame systems.

SELF-CENTERING ARTICULATED-FUSE SYSTEMS

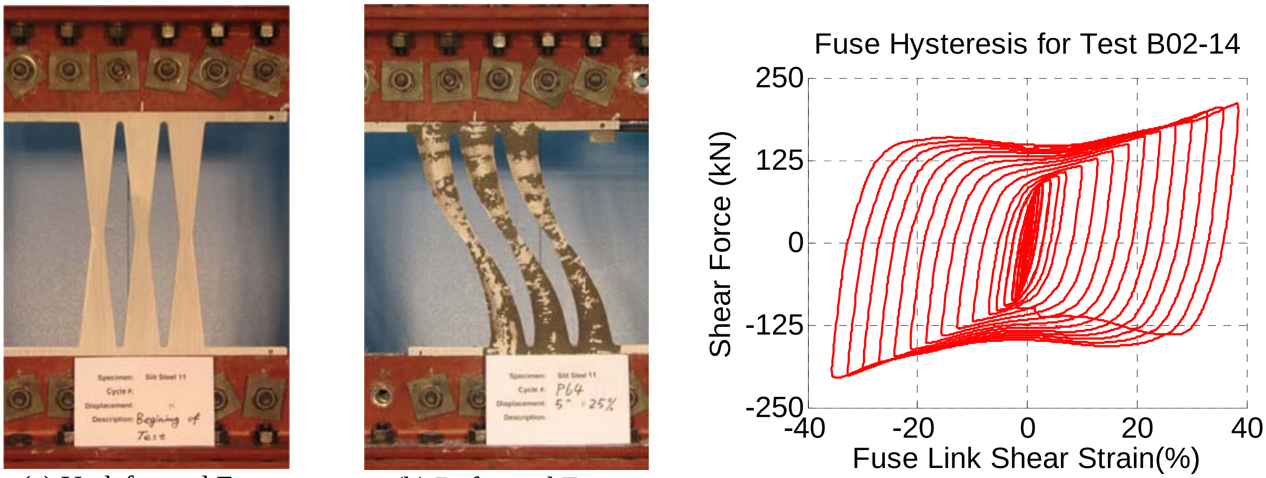
A natural outcome of the rising popularity of performance based design is that some building owners will show interest in and select structural systems that offer higher levels of performance. As the demand for high performance systems increases, opportunities arise for the development of innovative structural systems. One aspect that owners have shown an interest in is the state of the building and its reparability after a major seismic event. Traditional seismic force resisting systems will often experience inelastic action throughout the structure during a large earthquake, which results in residual drifts and distributed damage that is difficult and costly to repair. Thus, the possibility of a structure that concentrates seismic damage in replaceable elements or that does not exhibit residual drifts after an earthquake is attractive.

Three behavioral or structural features not typically present in traditional structural systems have been identified as potentially beneficial to a building seismic performance. These are: rocking, self-centering, and the use of articulated replaceable energy-dissipating structural fuses. A wide variety of research has been carried out to characterize the behavior of structural systems with combinations of these features. Recent studies have been conducted on rocking behavior in concrete structures (Holden et al. 2003; Ajrab et al. 2004; Lu 2005; Palermo et al. 2007), masonry structures (Ma et al. 2006), and steel structures (Ikenaga et al. 2006; Midorikawa et al. 2006; Pollino and Bruneau 2008). Self-centering forces have been identified in many of these studies coming from various sources, including vertical post-tensioning or column base connections designed specifically to provide self-centering forces but also the rotational stiffness of beams that frame in out-of-plane of the uplifting side of the wall. Horizontally post-tensioned moment frame systems have also been developed (Shen and Kurama 2002; Christopoulos et al. 2002; Garlock et al. 2005). For these systems, during a seismic event, the beam rotates relative to the column, opening a gap between the beam flange and the column and the post-tensioning provides a restoring force to return the beam to flush. Many elements within structural systems are capable of dissipating energy during a seismic event, though inelastic deformation, viscous forces, friction, etc.. Structural fuses, according to one definition, are replaceable elements that are designed such that all structural damage is concentrated in this element, allowing the primary structure to remain elastic (Vargas and Bruneau 2009). Such elements would clearly expedite the repair and return to service of a building following a major seismic event.

Controlled Rocking System

The controlled rocking system for steel-framed buildings combines the three features to provide a superior performance during seismic events and virtually eliminate residual drift. The system consists of steel frames that remain essentially elastic and are allowed to rock about the column bases. The specially designed column base details permit column uplift while restraining horizontal motion with bumpers or an armored foundation trough. The configuration in Figure 7b uses two side-by-side frames, although alternative configurations with single frames have also been investigated. Vertical post-tensioning strands provide self-centering forces. The strands are initially stressed to less than half of their ultimate strength, so as to permit additional elastic straining when the frames rock. Replaceable energy dissipating elements act as structural fuses that yield, absorb energy, and limit the forces imposed on the rest of the structure. In Figure 7b the fuses are configured as yielding shear elements between the two frames.

Research has been conducted (Eatherton 2010a, 2010b; Ma 2010a, 2010b) to develop this system, assess its performance, and provide design recommendations. Experimental work consisted of three phases. The first phase included component tests of the energy dissipating fuses. Fuses used in this system should have sufficient ductility and toughness so that they can dissipate energy throughout the cyclic loading expected during large earthquakes. Moreover, the fuses should be detailed to permit easy replacement in the event they become damaged. Several alternative designs were considered, but fuses that consisted of a steel plate with “butterfly” shaped links were found to provide the best performance. An example of a fuse that was tested as part of this research and its hysteretic response is shown in Figure 7.



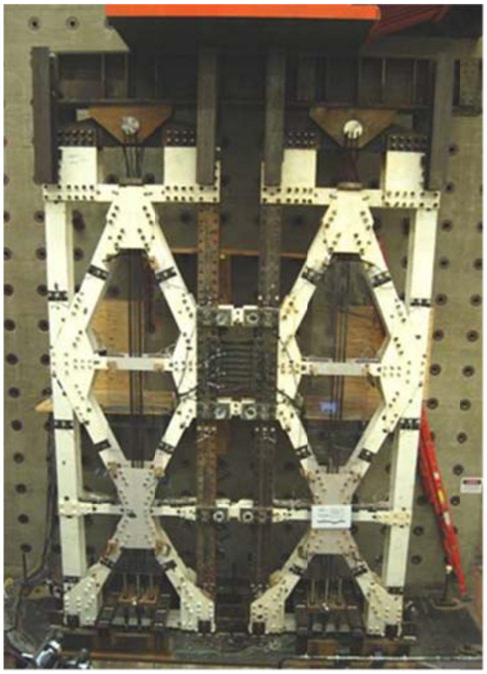
(a) Undeformed Fuse Specimen

(b) Deformed Fuse Specimen

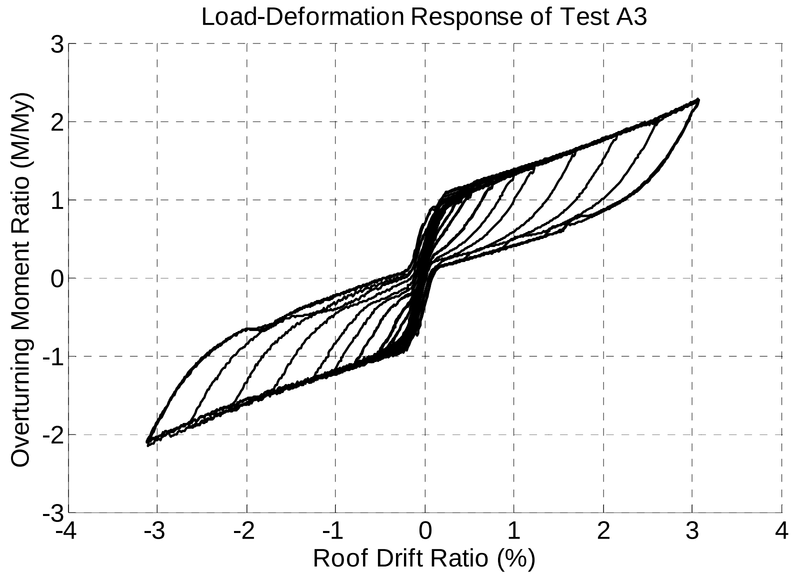
(c) Hysteretic Response

Figure 7: Fuse Specimens and Results (after Ma 2010)

The second phase included static cyclic and hybrid testing of a half-scale three-story rocking frame (Figure 8). Nine tests were conducted consisting of seven dual-frame configuration specimens, in which the two frames are linked together with fuses, and two single-frame configurations, in which there were no fuses between the two frames and instead the fuse was concentrated at the base of each frame allowing them to act independently. These test verified a number of aspects of the behavior. The connection details at the base of the columns performed well, allowing uplift and pivoting while restraining horizontal movement. The frames did not sustain any significant damage throughout testing, indicating that structural damage was successfully concentrated in the fuse elements. The fuses exhibited large deformation capacity and the ability to absorb considerable amounts of energy without fracturing. The post-tensioning strands experienced some yielding and strand fractures, but the system retained considerable load carrying capacity and the ability to self-center.



(a) Frame Specimen A3



(b) Load-Deformation Response

Figure 8: Frame Specimens and Results (after Eatherton 2010)

The third phase included shake table tests of two-thirds scale three-story specimen, using the single-frame configuration. Four sets of tests were conducted to investigate alternative fuse designs and effect of different ground motion inputs. During each test, the specimen was subjected to multiple ground motion inputs with varying degrees of intensity. The results further verified the controlled rocking system achieved its intended performance with regard to concentrating damage and virtually eliminating residual drifts after a seismic event.

CONCLUSION

This paper has reviewed key changes to the design provisions for composite construction, highlighted recent research on composite structural systems, and presented an example of a self-centering articulated-fuse system that offers enhanced performance under seismic loading. These examples demonstrate the trends of steady continual improvement to design provisions and development of new systems to satisfy the ever increasing demand for better performing structures, as well as, provide a glimpse of where the steel and composite construction may be heading in the future.

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