Concrete-Filled Tubes Columns and Beam-Columns: A Database for the AISC 2005 and 2010 Specifications

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Abstract:

This paper describes historical efforts to synthesize available experimental data on composite columns, including both steel reinforced concrete (SRC) and concrete-filled steel tube (CFT) specimens. The latest version of this database has about 1387 circular CFTs (912 columns and 475 beam-columns), 826 rectangular CFTs (524 columns and 302 beam-columns) and 267 SRCs (119 columns and 148 beam-columns). This database was developed and analyzed to determine gaps in the available knowledge. As a result of these analyses, a large set of full-scale experiments have recently been completed that encompass both the most slender specimens tested to date and comparisons to specimens with high-strength concrete. In addition, the tests were conducted under a complex set of cyclic load histories that have permitted the development and calibration of advanced computational models.

Keywords: Composite columns, concrete-filled tubes, databases, American specifications

1 Introduction:

While there have been numerous experimental test series for composite columns in the past, in particular for CFT members, most of the documentation in the literature is limited to columns with particular characteristics. These characteristics are mainly: small cross-sections, low-slenderness, relatively simple load protocols and end-restraints, compact width-thickness ratios, and conventional strength materials. Specimens with these characteristics are the only viable option when the capabilities of the laboratory equipment are limited and the testing of full-scale specimens with complex boundary conditions is impractical or uneconomical.

The results from specimens with low slenderness but a wide range of cross-section sizes have been useful for the quantification of the cross-section strength and behavior of short members. In turn, results from slender specimens, which have mainly had small cross-sections and been cast and tested in a horizontal position, have been useful for parametric studies accounting for the length effects and effective confinement of CFT columns and beam-columns. However, extrapolation is needed from these results to members with sizes closer to those used in practice, and the conclusions from these research efforts may need to be adjusted accordingly.

This paper reviews some of the previous research that focused on the experimental tests of CFT elements. Innovations, principal contributions, and main conclusions of selected previous research studies are briefly discussed. There is no intent to provide a complete summary of all efforts; such summaries are available elsewhere (Aho, 1997; Kim, 2005; Leon *et al.*, 2005; Goode 2007; Gourley *et al.*, 2008). The objectives of this work are two-fold. First, to assess the ability of the new AISC 2005 and AISC 2010 composite beam-column design provisions to provide reasonable and conservative results for a large data set of concrete-filled steel tubes. Second, to identify gaps in the existing datasets that will serve as the guide to the development of an experimental test series and associated computational studies; these studies are described in Perea (2010) and Denavit et al. (2010).

2 Review of previous experimental studies on CFT columns and beam-columns

Table 1 summarizes chronologically the material and geometric properties of CFT specimens tested in previous experimental research studies. The information in this table was extracted from the collected databases (Leon *et al.*, 2005; Goode, 2007; Gourley *et al.*, 2008), which will be discussed in the following section.

2.1 Notable CFT test series

As seen in Table 1, Kloppel and Goder performed the first documented experimental research on CCFTs in 1957. This research, originally published in German, is cited in English by Knowles and Park (1970) and Roik and Bergmann (1989), who also described the experimental results and other details. Kloppel and Goder performed collapse load tests on hollow and concrete-filled steel tubes. Three tests were examined in detail, with stresses and strains in both the steel and concrete tabulated for incremental values of concentric load. Based on the experimental data analysis, these authors established the initial design formulas for CFT columns.

Table 1 – Summary of experimental studies in CFT columns and beam columns

Reference	*Type*	*KL (ft)*	*tube size (in) (D) or (b)x(h)*	*wall thickness (in)*	*Max D/t*	*f_c' (ksi)*
Kloppel and Goder (1957)	*CCFT*	*2.8 – 7.6*	*3¾ – 8½*	*$^{1}/_{8}$ – ½*	*55*	*3 – 4*
Salani and Sims (1964)	*CCFT*	*5*	*1 – 3*	*$^{1}/_{32}$ – $^{3}/_{32}$*	*55*	*3 – 4*
Chapman and Neogi (1966)	*CCFT*	*1.3 – 6.8*	*5 – 14*	*$^{1}/_{16}$ – $^{3}/_{8}$*	*78*	*3 – 9.6*
	RCFT	*1.3*	*4.5x4.5*	*$^{3}/_{16}$, $^{3}/_{8}$*	*25*	*4.6*
Furlong (1967)	*CCFT*	*3*	*4.5, 5, 6*	*$^{1}/_{16}$, $^{3}/_{32}$, $^{1}/_{8}$*	*98*	*3 – 6.6*
	RCFT		*4x4, 5x5*	*$^{3}/_{32}$, $^{1}/_{8}$, $^{3}/_{16}$*	*46*	
Gardner and Jacobson (1967), Gardner (1968)	*CCFT*	*0.5 – 7.5*	*3 – $6^{2}/_{3}$*	*$^{1}/_{16}$ – $^{3}/_{16}$*	*65*	*2.6 – 6.3*
Knowles and Park (1969)	*CCFT*	*0.8 – 5.7*	*3¼, 3½*	*$^{1}/_{16}$, ¼*	*60*	*5.4 – 6*
	RCFT		*3x3*	*$^{1}/_{8}$*	*21*	*5 – 6.8*
Neogi, Sen and Chapman (1969)	*CCFT*	*4.6 – 10.9*	*5 – $6^{2}/_{3}$*	*$^{1}/_{16}$ – $^{3}/_{8}$*	*78*	*2.7 – 9.7*
	RCFT	*1.3*	*4.5x4.5*	*$^{3}/_{16}$ – $^{3}/_{8}$*	*24*	*4.7*
Janss and Guiaux (1970)	*CCFT*	*1.7 – 14.3*	*$3^{3}/_{4}$ – $8^{2}/_{3}$*	*$^{1}/_{8}$ – ¼*	*36*	*4.5*
Janss (1974)	*CCFT*	*3.9 – 5.3*	*10.8 – 16*	*$^{3}/_{16}$ – $^{3}/_{8}$*	*81*	*4 – 5.5*
	RCFT	*4.3 – 4.6*	*13x13*	*$^{3}/_{16}$ – $^{7}/_{16}$*	*72*	*4 – 4.5*
Bridge (1976)	*RCFT*	*7, 10*	*6x6, 8x8*	*$^{1}/_{4}$ – $^{7}/_{16}$*	*21*	*4.5 – 5.1*
Zhong (1978)	*CCFT*	*0.8 – 16.4*	*$3^{3}/_{4}$ – $19^{1}/_{2}$*	*$^{3}/_{32}$ – ½*	*84*	*3.2 – 7.9*
Tang (1978)	*CCFT*	*1.4 – 4.9*	*4.2*	*$^{1}/_{8}$*	*35*	*5.4*
Tomii and Sakino (1979a, 1979b)	*RCFT*	*1.0*	*4x4*	*$^{3}/_{32}$, $^{1}/_{8}$, $^{3}/_{16}$*	*43*	*3 – 4.6*
SSRC Task Group 20 (1979)	*CCFT*	*3.4 – 7.6*	*$3^{3}/_{4}$ – $8^{1}/_{2}$*	*$^{5}/_{32}$ – ¼*	*53*	*3 – 4.3*
Cai (1981)	*CCFT*	*2.3 – 12.1*	*$6^{1}/_{2}$*	*$^{3}/_{16}$*	*33*	*4, 5.5, 6*
Tang et al. (1982)	*CCFT*	*0.5 – 6.5*	*3 – 11.8*	*$^{1}/_{16}$ – $^{7}/_{16}$*	*100*	*3 – 8*
Zhou (1983)	*CCFT*	*6.7 – 9.7*	*4, $5^{3}/_{16}$*	*$^{5}/_{32}$ – $^{7}/_{32}$*	*30*	*3.6, 5.4*
Zhong (1983)	*CCFT*	*1.1 – 5.3*	*4¼*	*$^{1}/_{16}$ – $^{7}/_{32}$*	*57*	*3.1, 4.5*
Cai and Jiao (1984)	*CCFT*	*0.9 – 12.1*	*$3^{3}/_{4}$ – 12.6*	*$^{3}/_{32}$ – ½*	*102*	*3.9 – 6.8*
Cai and Gu (1985)	*CCFT*	*1.1 – 18.2*	*$4^{1}/_{4}$*	*$^{1}/_{8}$*	*27*	*4.2*
Wang and Yang (1985)	*CCFT*	*0.9*	*$5^{1}/_{4}$*	*$^{3}/_{32}$ – ¼*	*55*	*2.5, 3.9*
Sakino et al. (1985)	*CCFT*	*0.7*	*4*	*$^{1}/_{32}$ – ¼*	*192*	*2.6, 5.4*
Sakino et al. (1985)	*CCFT*	*0.7*	*4*	*$^{1}/_{32}$ – ¼*	*192*	*2.6, 5.4*
Chen et al. (1988)	*CCFT*	*0.6 – 2.2*	*2 – 6½*	*$^{1}/_{8}$ – $^{3}/_{16}$*	*38*	*4.8*
Pan (1988)	*CCFT*	*8 – 11.9*	*6½*	*$^{3}/_{16}$*	*38*	*6.3*
Lin (1988)	*CCFT*	*1.6, 2.6*	*6*	*$^{1}/_{32}$ – $^{3}/_{32}$*	*214*	*3, 5*
	RCFT		*6x6, 6x8*		*284*	*3.3, 5.1*
Cederwall,Engstrom and Grauers (1991)	*RCFT*	*9.83*	*4.72x4.72*	*$^{3}/_{16}$–$^{10}/_{32}$*	*24*	*5.65-14.9*
Sakino and Hayashi (1991)	*CCFT*	*0.8, 1.2*	*7*	*$^{1}/_{8}$ – $^{3}/_{8}$*	*58*	*3.2, 6.6*
Bergmann (1994)	*RCFT*	*3.3, 13.1*	*7x7, 10¼x10¼*	*$^{5}/_{16}$*	*35*	*13.4*
Matsui et al. (1995, 1997)	*CCFT*	*2.2 – 16.3*	*6½*	*$^{3}/_{16}$*	*40*	*4.6 – 5.9*
	RCFT	*2 – 14.8*	*6x6*	*$^{3}/_{16}$*	*33*	
Shakir-Khalil (1996)	*RCFT*	*9.6 – 16.1*	*4x6*	*$^{3}/_{16}$*	*33*	*5.2 – 6*
Inai and Sakino (1996)	*RCFT*	*1.2 – 3.2*	*several square sizes*	*$^{3}/_{16}$ – $^{3}/_{8}$*	*72*	*3.7 – 13.2*
Roeder and Cameron (1999)	*CCFT*	*2.7 – 6.3*	*10¼ – 23¾*	*$^{1}/_{4}$ – $^{1}/_{2}$*	*108*	*6.4 –6.9*
Bridge and O'Shea (1997, 2000)	*CCFT*	*1.8 – 2.2*	*6½, 7½*	*$^{1}/_{32}$ – $^{1}/_{8}$*	*220*	*5.5 – 16.5*

Table 1 – Summary of experimental studies in CFT columns and beam columns (cont.)

Reference	*Type*	*KL (ft)*	*tube size (in) (D) or (b)x(h)*	*Wall thickness (in)*	*Max D/t or h/t*	*f_c' (ksi)*
Nakahara and Sakino (1998)	*RCFT*	*2*	*8x8*	$^{1}/_{8} - ^{1}/_{4}$	*63*	*17.3*
Varma (2000)	*RCFT*	*4*	*12x12*	$^{1}/_{4}, ^{3}/_{8}$	*50*	*16*
Seo Chung (2002); Seo, Tsuda and Nakamura (2002)	*RCFT*	*1.6 – 12.3*	*5x5*	$^{1}/_{8}$	*40*	*9.3 – 14*
Mursi et al. (2003)	*RCFT*	*9.9*	*4.7, 6.7, 8.7, 10.6*	$^{3}/_{16}$	*52*	*3*
Han and Yao (2003)	*CCFT*	*1 – 6.6*	*4 – 8*	$^{1}/_{8}$	*65*	*3 – 6.8*
	RCFT	*1.8 – 7.7*	*several rect. Sizes*		*134*	*2.7 – 8.5*
Lam and Williams (2004)	*RCFT*	*1.0*	*4x4*	$^{5}/_{32} - ^{3}/_{8}$	*23*	*3.6 – 11.5*
Hardika and Gardner (2004)	*RCFT*	*5.9*	*8x8*	$^{3}/_{16} - ^{3}/_{8}$	*44*	*6.4 – 14*
Han and Yao (2004)	*CCFT*	*6.6*	*8*	$^{1}/_{8}$	*65*	*6.8*
	RCFT	*2, 7.6*	*8x8*			
Ghannam et al. (2004)	*CCFT*	*7.2, 8.1*	$4^{1}/_{3} - 6.5$	$^{3}/_{32} - ^{3}/_{16}$	*58*	*1.5 – 4.8*
	RCFT	*6.6, 8.2*	*4x4,4x8, 5½x5½, 3½x6*		*48*	
Perea et al. (2010)	*CCFT*	*36, 52*	*5.56, 12.75, 20*	*0.134,* $^{1}/_{4}$	*86*	*5, 12*
	RCFT	*36, 52*	*20x12*	$^{5}/_{16}$	*67*	

One of the first comprehensive experimental research study using both CCFT and RCFT columns and beam-columns was published by Furlong (1967), who tested CFT specimens with both concentric and eccentric loads. Based on this experimental data, design equations were proposed to estimate the ultimate strength of beam-columns. In addition, the concrete-steel interaction was taken into accounted explicitly for the first time. Furlong observed that the two materials behave independently of one another at strains below 0.001, where the Poisson's ratio of concrete is lower that of steel; this difference in lateral expansion resulted in a non-contact state between these two materials at low levels of strain. As strains increased, the concrete expanded laterally at a greater rate than the steel. Above strains of 0.001, the concrete Poisson ratio began to approach that of steel and, as a consequence, the steel started providing confinement to the concrete. Simultaneously, the concrete core stabilized the steel wall of the tubes, preventing premature local buckling and allowing the tube to attain its full yield capacity. Data from these tests also show that creep had an influential effect on the specimen behavior.

Tomii and Sakino (1979a) conducted experimental research on square CFT specimens to determine moment-curvature (M-ϕ) relationships. Under a constant axial load, moment was applied to the section in uniformly increasing amounts. In addition to the very detailed experimental study, the authors proposed analytical equations to estimate the ultimate moment of a RCFT section. Analytical moment-curvature (M-ϕ) relationships were developed and compared to the experimental results. Moment-curvature (M-ϕ) plots show an increase in ductility with a decrease in D/t ratio. Specimens with D/t=24 behaved in a ductile manner, while columns with D/t=44 and high axial load had a softening branch in the M-ϕ diagram. Even though this last set of columns failed in a rather brittle manner, the tension side of the steel tube still yielded. It was suggested that the ductility increase as the D/t ratio decreased was due largely to the confinement of the concrete by the steel. In Tomii and Sakino (1979b), additional square CFT tests were reported in five series. Each series contained different material properties and h/t ratios, and the parameters evaluated were the shear span ratio (a/h) and axial load ratio (P/P_o). The tubes were annealed to remove residual stresses and the specimens were tested with double fixed boundary conditions (K=0.5). This research indicated a negligible effect due to the a/h ratio. It was also observed that for specimens with a high axial load ratio (P/P_o = 0.5), after a certain amount of decrease in the lateral strength, the hysteretic loops tended to stabilize and even showed a slight increase in lateral resistance.

3 Compiled experimental databases

3.1 Previous Databases

There have been several large-scale efforts to compile an experimental database that summarizes the principal results of composite columns tests, either as CCFT, RCFT or SRC cross-sections, and/or as columns or beam columns tests. The first effort of this type was by an SSRC committee in 1979 (SSRC Task Group 20, 1979), which reported a collection of 179 tests that included 73 tests on axially loaded CFTs, 30 tests on axially loaded SRCs, 32 test on eccentrically loaded

CFTs and 44 tests on eccentrically loaded SRCs. As a result of this database analysis, this committee proposed a design specification for composite columns which was adopted in the Chapter I of the AISC LRFD Specification (1986). Three years later, Roik and Bergmann (1989) collected experimental data from 208 tests reported at the time in the literature, and used this database for the development and calibration of the Eurocode specifications EC-4 (1992) for composite columns.

In the mid-1990s, a research team guided by Galambos gathered experimental test results on composite columns with the purpose to investigate through Monte-Carlo simulations reliability indices for CFT and SRC columns designed by the existing EC-4 (1992) Eurocode (Sulyok and Galambos 1995) and the AISC (1993) LRFD Specifications (Lundberg and Galambos 1996). This database contained data for 389 available tests that included 119 tests on CFT columns and 128 CFT beam columns, and 59 tests on SRC columns and 83 SRC beam columns. All the data compiled was for tests with monotonic loads, and the material and geometric properties as well as the experimental peak strength are reported.

In 1996, Aho and Leon (1996), collected a database for SRC, CCFT and RCFT columns and beam columns. Insofar as materials and geometric properties were concerned, there were no specific limitations on the database. In total, this research reported nine databases, six for SRC, CCFT and CFT columns and beam columns, and three for those data that had parameters that could not be compared with the rest of the data. The latter contained mainly data coming from shear critical specimens tested cyclically in Japan. These databases were used for the evaluation of the existing AISC-LRFD Specification (AISC 1993) and Eurocode (EC4 1994) design equations and their corresponding reliability indices. Additionally, the data analysis aided the authors in developing design equations, which were used as the basis of the current AISC Specification design provisions for composite columns (AISC, 2005). In 2005, Kim and Leon added new information and edited these databases, resulting again in six databases for SRC, CCFT and RCFT of columns and beam-columns. These updated databases were used in the evaluation of the current AISC Specification (AISC, 2005, 2010) and Eurocode (EC4, 1994).

Kawaguchi *et al.* (1998) compiled an experimental database for CFT beam-columns from tests conducted in Japan. The collected data included monotonic and cyclic loads and documented material and geometric properties, experimental stiffness, and strengths and deformations associated to the peak and other characteristic levels (i.e. post-peak strength dropped 5%, maximum rotation reached the 1/100 value). Based on the experimental data, these authors developed analytical models for the calculation of the flexural strength and the rotational capacity of CFT beam-columns. Four years later, Nishiyama *et al.* (2002) gathered data from tests conducted in Japan as part of the U.S.-Japan Cooperative Research Program on Composite and Hybrid Structures. This database included test results for specimens under monotonic and cyclic loads, and was divided for both CCFTs and RCFTs in (1) centrally-loaded stub-columns, (2) eccentrically-loaded stub columns, (3) beam-columns, and (4) sub-assemblages. The database also reported material and geometric properties, calculated stiffness, period of vibration and costs, among other particular information of each test. Based on this collected data, the authors developed design formulas to calculate the strength and deformation capacities of CFT elements.

Another team that has gathered experimental results and collected them in databases from 2001 has been led by Hajjar (Gourley *et al.*, 2008). Gourley and Hajjar (1993) is the first version of a synopsis for CFT beam-columns subjected to monotonic and cyclic loads. This database has been updated and refined with more data in later versions in 1995, 2001 and 2008 (Gourley *et al.,* 2008). The latest compilation in Gourley *et al.* (2008) provides a summary of the behavior and experimental work of concrete filled steel tube members, connections, and frames that are reported in detail in the literature. These published studies have been summarized with an emphasis on experimental setup and properties, analytical methods presented and key results from the work under various loading conditions. Tort et al. (2003) augmented this database by documenting the progression of damage in a large number of CFT monotoically and cyclically loaded experiments, thus identifying CFT limit states in addition to the peak strength value that is the primary focus of other databases.

More recently, Goode published online in 2006 a database for CFT columns and beam columns, with an update in 2007 (Goode 2007). His first database compilation was divided in short (L/D<4) and long elements, in columns (no moment) and beam-columns (with uniaxial or biaxial bending moment with and without a preload), and for circular, rectangular and polygonal cross-section shapes. Goode's latest updated compilation published online in 2007 (http://web.ukonline.co.uk/asccs2) is composed of 13 databases that summarize the experimental results of 1819 CFT specimens. This database has been used by Goode and Lam (2008) for the evaluation of the strength predicted by the Eurocode EC-4 (2004); the comparison between the experimental strength and the EC-4 prediction in this studies have shown good predictions in general for CFTs, except in RCFTs with concrete above 75 MPa (10.9 ksi) of strength where the EC-4 prediction has underestimated the experimental strength.

3.2 Brief description of previously tested CFT specimens with extreme characteristics

The longest CFT specimens tested and documented in the specialized literature were conducted by Cai and Gu (1985) for CFTs with an effective length of 18.2 feet and diameter of 4¼ inches, Cederwall et al. (1991) with an effective length of 9.83 ft. and a square tubes of 4.73 in., and by Shakir-Khalil and Al-Rawdan (1996) with an effective length of

16.1 feet and 4x6 inch cross-section size. Cai and Gu's results on slender columns show elastic buckling failures that were well predicted by the Euler's buckling; they also mentioned that the behavior of slender CCFTs is influenced by initial imperfections (i.e. out-of-straightness and loading eccentricity). For RCFTs, Shakir-Khalil and Al-Rawdan (1996) reported lower strength with an increase in length due to local buckling that generally took place on the wider side of the tubes.

By 2009, the specimens with the largest circular CFTs cross-section size were tested by Luksha and Nesterovich (1991) using spiral welded tubes with diameter of 40.2 inches and effective length of 10 feet. Two types of failure were observed in this study: (1) for the small diameter specimens, failure was characterized by the local buckling of the steel and the crushing of the concrete; (2) for the large diameter specimens, failure was reported as a shear failure. In turn, the previous largest square CFT cross-section was conducted by Janss (1974) with 13x13 inch cross-section size and effective length of 4.36 feet. For rectangular CFT shapes, Han and Yao (2003) tested tubes with 14.2x9.4 inchcross-section size and effective length of 4.7 feet. In contrast, the specimens with the largest width-thickness ratio documented were performed by O'Shea and Bridge (1997) for CCFTs with a *D/t* ratio equal to 221, and by Lin (1988) for RCFTs with an *h/t* ratio equal to 284. Both studies used special tubes with $^{1}/_{32}$ inches in thickness, 7½ inches diameter, and effective length of 2.2 feet for the CCFTs, and 6x8 in. and effective length of 2.6 feet for the RCFTs.

Specimens that have been filled with high strength concrete include CCFTs tested by O'Shea and Bridge (1997) using concrete with a compressive strength of 16.5 ksi, and RCFTs tested by Nakahara and Sakino (2000) and Varma (2000) with concrete strengths of 17.3 and 16 ksi, respectively.

3.3 Review of previous experimental studies on CFTs under torsional loading

Although few CFT specimens have been tested under torsional loading, in general, it is expected that these elements would perform well when loaded in torsion due to the position of the steel tube at the perimeter of the CFT cross-section. Additionally, circular CFT's torsional behavior is excellent due to the circular shape of the steel tube. Among the few studies of experimental studies of circular CFTs under torsion are those reported by Lee *et al.* (1991) and Xu *et al.* (1991). Lee *et al.* (1991) tested short CCFTs under monotonic and cyclic torsional loading with and without compression; results of this study show higher torsional resistance in the CCFTs with higher compression loads. Xu *et al.* (1991), in contrast, tested short ($L=7D$), medium ($L=13D$) and long ($L=20D$) CCFTs under torsional loading, also with and without compression; the diameter of these specimens were 3½ and 4½ inches. These authors reported non-abrupt torsional failures at rotation angles of 5, 9, and 14 degrees for the short, medium, and long columns, respectively. Contrary to Lee et al.'s results, the ultimate torsional moment resistance decreased with an increase in the axial load ratio, so the highest torsional moment was attained in the pure torsion case. The characteristic failure mechanism was a cracking of the concrete followed by a propagation of the cracks along the length of the tube in a spiral pattern. Lack of experimental data is evident for rectangular and square CFT specimens under torsional loading and thus further work in this area is needed.

3.4 Gaps in the experimental databases

One of the premises for the selection a test matrix for any research is that the specimens to be tested fill gaps with respect to the available experimental data. This motivated an analysis of the existing data to find out where the gaps were and what parameters should be accounted to fulfill this goal. To accomplish this objective, the databases compiled by Leon *et al.* (2005), Goode (2007) and Gourley *et al.* (2008) were joined and edited into a unified database. The unified database used in this research excludes duplicated data, tests where significant inconsistencies were found, or tests that correspond to specimens that do not share the characteristics of the specimens included in the database (i.e., steel-only specimens, concrete-only specimens, cross sections with a non-circular or non-rectangular geometry, etc). In addition, this database was updated with new data published up to 2009. Thus, this refined database included results for 1387 CCFTs (912 columns and 475 beam-columns), 826 RCFTs (524 columns and 302 beam-columns) and 267 SRCs (119 columns and 148 beam-columns). A primary use of this database was to check the overall validity of the AISC design provisions, which had been substantially modified in the 2005 version (AISC 2005). The changes in that specification brought composite column design in the U.S. into substantial agreement with the Eurocode insofar as nominal cross-section strength is concerned. The length effects, however, are applied in a different manner by U.S. and European specifications, so the final designs cannot be compared directly, particularly as the resistance factors are different. The following figures summarize the analysis of the unified database for composite SRC, CCFT and RCFT columns and beam-columns. Although the current experimental work focuses on CFT cross-sections only (Perea 2010), the results from SRC tests are also given for future reference and are encompassed in the ongoing computational research (Denavit et al. 2010). In Figure 1, histograms of the experimental data from both columns and beam-columns are shown with respect to the slenderness parameter (λ), the concrete strength (f_c'), the steel yield stress (F_y), the longitudinal steel ratio ($\rho_s=A_s/A$), the *D/t* ratio for CFTs, the *h/t* ratio for RFCTs, and the reinforcement steel ratio for SRCs ($\rho_{sr}=A_{sr}/A$).

A comparison of the experimental strength normalized with the analytical column curve as given by the Chapter I in the AISC 2005 Specifications (P_{exp}/P_o) are illustrated for RCFT columns (Figure 2a), CCFT columns (Figure 2b) and SRC columns (Figure 3). These figures show a large dispersion, mainly for short columns. This large dispersion is due primarily to the confinement effects, which are influenced by the depth-thickness ratio and the size of the cross-section..
Figure 4 shows the normalized experimental strength obtained for CCFT beam-columns with different slenderness parameter (λ). For reference, the continuous line represents the *P-M* interaction diagram of a steel element as described by Equations H1-1a and H1-1b in the AISC 2005 Specifications. As illustrated in these figures, the slenderness, which impact second order moments, reduce the *P-M* strength as expected.

The data was studied in numerous ways to identify significant gaps. For example, Figure 5 shows a plot of available data for encased composite columns in terms of the steel and concrete strengths and the reinforcement ratio. It is clear from this figure that there were very few tests with both high strength steel and concrete, and that most of the tests were for relatively large reinforcement ratio (>8%). This and similar plots clearly show what parameters have been explored in most of the prior experimental studies. As illustrated in these figures, most experimental data is concentrated in:

- Short and intermediate slender columns ($\lambda \leq 1.5$), a range where the inelastic buckling strength governs.
- Normal strength concrete ($f_c' \leq 6$ ksi).
- Conventional yield stress (36 ksi $\leq F_y \leq$ 50 ksi) in the steel.
- Low D/t ratios in CFT specimens (D/t ≤ 50).
- High steel ratio ρ_s ($8 \leq \rho_s \leq 16$)
- Low steel reinforcement ratio in SRC specimens ($\rho \leq$ 1%).

These observations indicated that a test series with the following characteristics was desirable:

- Slender columns (λ>1.5) to study a range where the second-order effects are larger and elastic buckling governs.
- CFTs filled with moderate strength concrete (f_c' = 5 ksi) and high strength concrete (f_c' = 12 ksi).
- Sections with the large *D/t* ratios, at the upper bound of what is available in the HSS steel market.
- A wide variety of HSS cross-sections, from tubes with small cross-section size where the slenderness effects dominate, up to tubes with the highest available cross-section size that are common in civil construction.
- Specimens with the largest length that can be handled in the tallest laboratory available.
- Specimens with high strength ($\underline{P_n}$ > 1000 kips, M_n > 6000 kip-ft at balance point, V > 600 kips).
- Use of complex multiaxial load histories to allow the development of reasonable values for the effective stiffness, interaction strength, and evolution of plasticity response of these elements.
- Inclusion of torsional load histories.

4 Conclusion

To address these needs, Perea (2010) conducted a comprehensive research program on CFTs that consisted in 18 full-scale and slender specimens loaded with different load cases and boundary conditions (Table 2). These specimens are the longest (*L*=18', 26', *KL*=36', 52') and the most slender ($1.2 < \lambda < 3$) CFT specimens tested to date. The tests were possible through use of the NEES Multi-Axial Sub-assemblage Testing (MAST) system, which consists of a stiff steel crosshead connected to four vertical actuators (each with a capacity of 330 kips and ±20 in. strokes) and two actuators in each horizontal axis (each with a capacity of 440 kips and ±16 in. strokes). The MAST system has the capability of controlling six degrees-of-freedom independently with a maximum capacity of P_z = 1320 kips of vertical force and V_x=V_y=880 kips of horizontal force. In addition, advanced computational models have been developed and calibrated as part of this work, and a set of parametric studies is underway on the seismic response of composite frames so as to evaluate the seismic performance factors and stability assessment procedures used in the U.S. for composite construction (Denavit et al. 2010). The formulation was verified against a wide range of monotonic and cyclic experiments, including short columns, beams, and proportionally and non-proportionally loaded beams columns. Several elastic and dynamic problems were also analyzed to validate the geometrically nonlinear and dynamic formulation. The studies showed that accurate results are obtained for composite members and frames subjected to a variety of loading conditions (e.g., see Figure 6).

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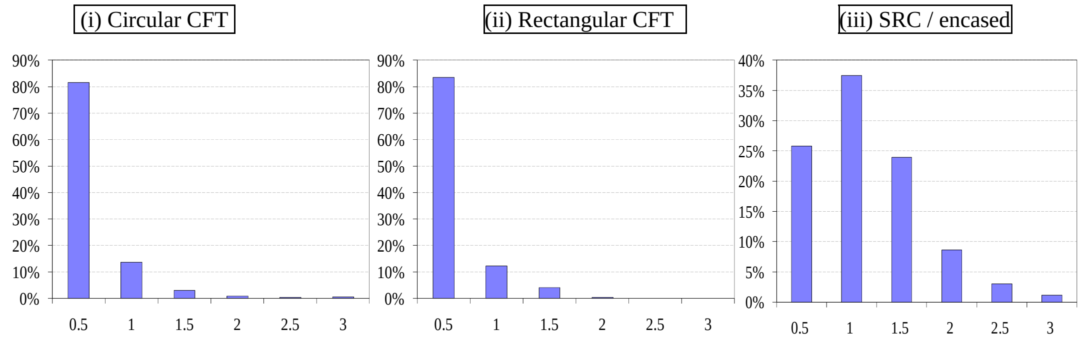
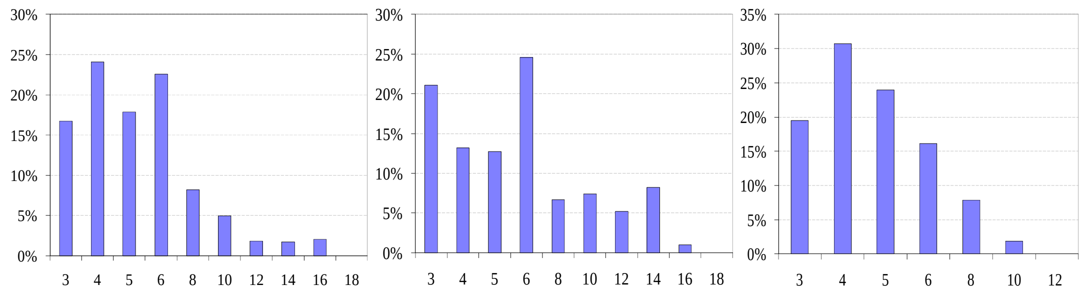
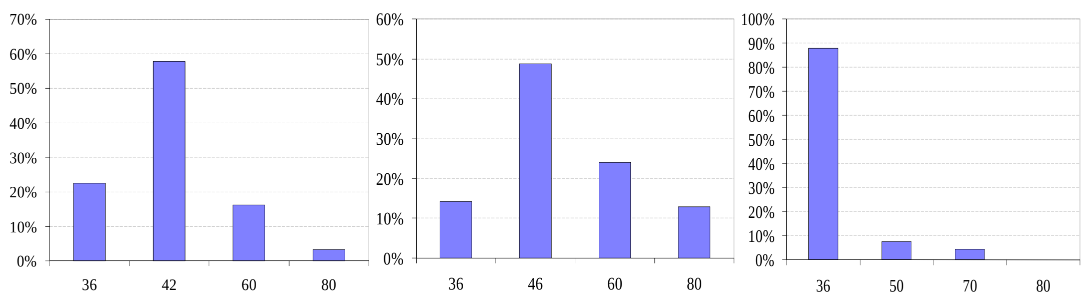


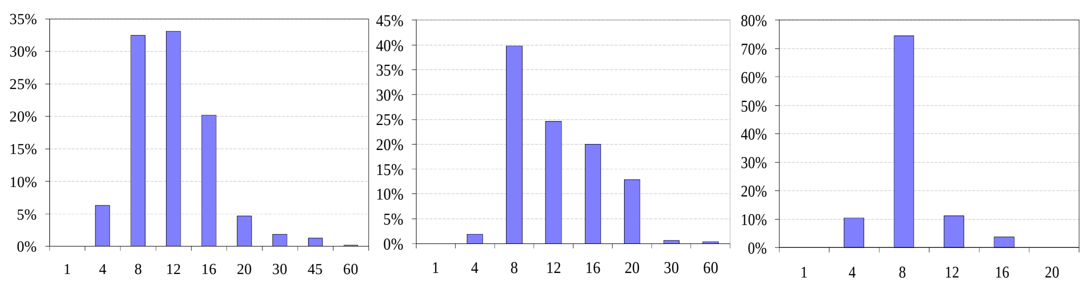
(a) Number of tests vs. the Slenderness parameter (λ)



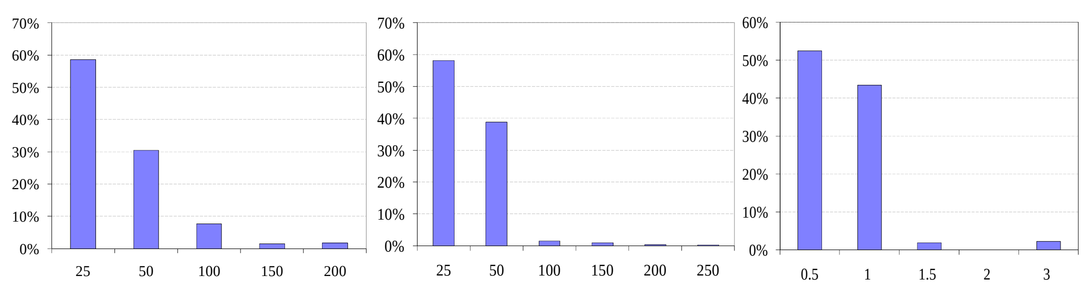
(b) Concrete strength f_c' (ksi)



(c) Yield stress F_y (ksi)

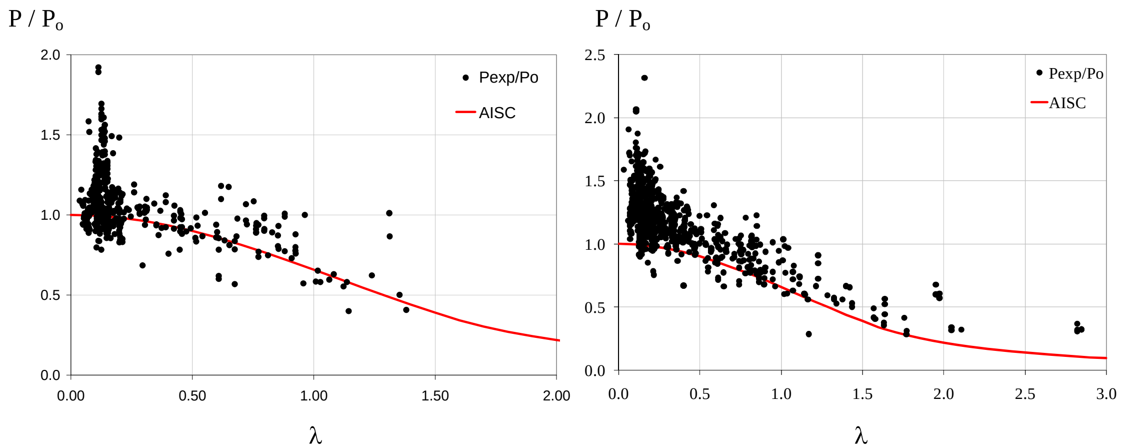


(d) Steel ratio $\rho_s = A_s/A$ (%)



(e) D/t ratio *(f) h/t ratio* *(g) Reinforcement ratio $\rho_{sr}=A_{sr}/A$ (%)*

Figure 1. Histograms obtained from the unified database



(a) RCFTs *(b) CCFTs*

Figure 2. Experimental vs. analytical column curve for CFTs

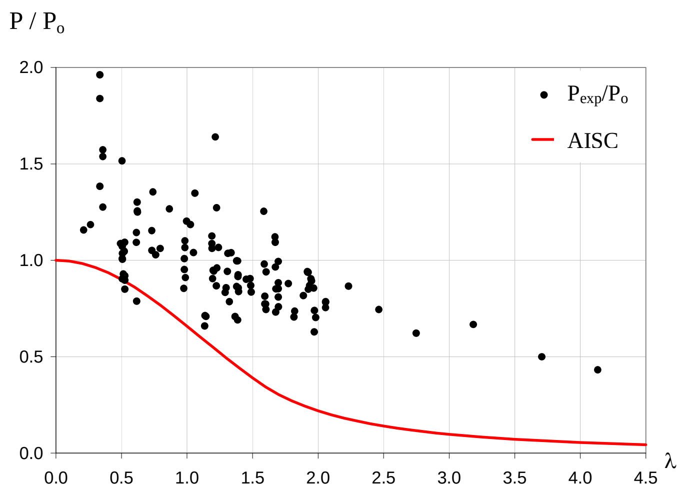
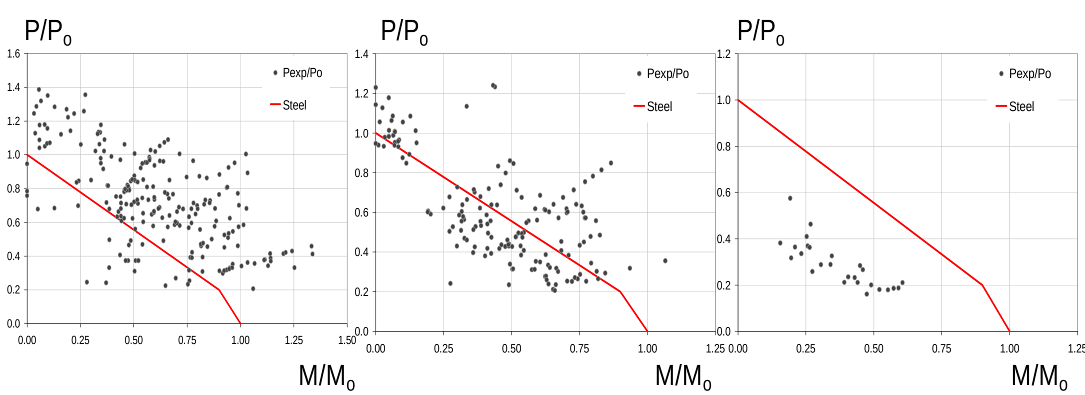


Figure 3. Experimental vs. AISC column curve for SRCs



(i) λ > 0.5 (ii) 0.5 > λ > 1.0 (iii) 1.0 < λ < 1.5

Figure 4. Normalized experimental strength obtained for CCFT beam-columns for different slenderness parameter (λ) ranges.

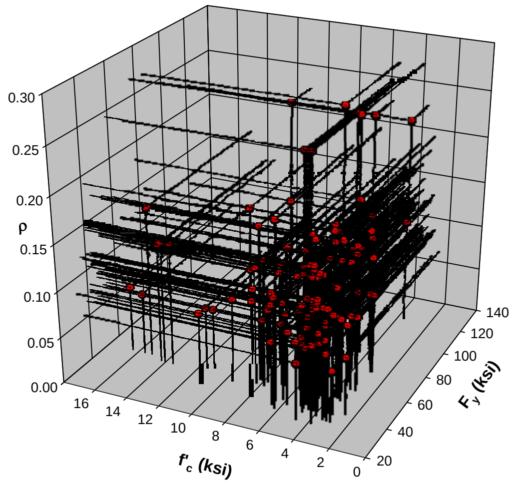


Figure 5 – Data available for circular CFTs with respect to material strengths and reinforcement ratio.

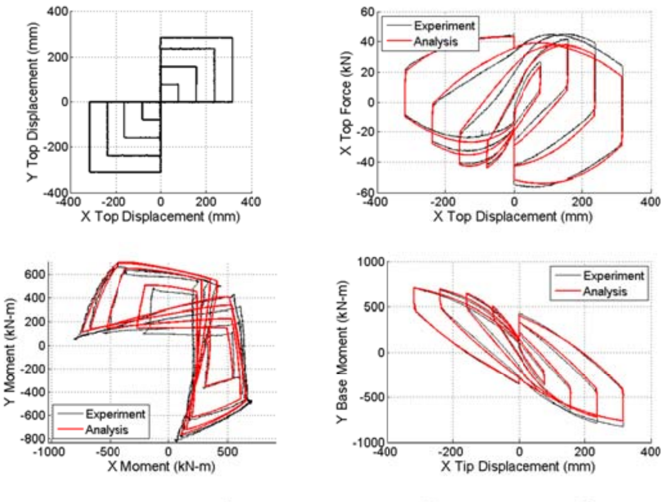


Figure 6 – Comparison of experimental and analytical results for a complex multiaxial load history

Table 2 – Test matrix of the CFT specimens with nominal values

Specimen name	L (ft)	Steel section HSS D x t	*Fy* (ksi)	*f_c'* (ksi)	*D/t*
C5-18-5	18	HSS5.563x0.134	42	5	45
C12-18-5	18	HSS12.75X0.25	42	5	55
C20-18-5	18	HSS20x0.25	42	5	86
Rw-18-5	18	HSS20x12x0.25	46	5	67
Rs-18-5	18	HSS20x12x0.25	46	5	67
C12-18-12	18	HSS12.75X0.25	42	12	55
C20-18-12	18	HSS20x0.25	42	12	86
Rw-18-12	18	HSS20x12x0.25	46	12	67
Rs-18-12	18	HSS20x12x0.25	46	12	67
C12-26-5	26	HSS12.75X0.25	42	5	55
C20-26-5	26	HSS20x0.25	42	5	86
Rw-26-5	26	HSS20x12x0.25	46	5	67
Rs-26-5	26	HSS20x12x0.25	46	5	67
C12-26-12	26	HSS12.75X0.25	42	12	55
C20-26-12	26	HSS20x0.25	42	12	86
Rw-26-12	26	HSS20x12x0.25	46	12	67
Rs-26-12	26	HSS20x12x0.25	46	12	67
C5-26-12	26	HSS5.563x0.134	42	12	45

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