Removal of Yield Stress Limitation for Composite Tubular Columns

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1. INTRODUCTION

Composite action between different structural materials has been utilized in bridge and building construction for a number of years. Traditionally constructed of steel and concrete, composite structures take advantage of the strength characteristics of each material. For example, a composite bridge girder utilizes the properties of steel in the tension zone and the compressive strength of the concrete in the deck. Similarly, a column made of a steel tube filled with concrete maximizes the steel strength by restraining the primary failure mode, i.e., local buckling of the tube wall, while simultaneously improving the load sharing between steel and concrete. The tube also provides confinement for the concrete, preventing early spalling and effectively raising the available strength of the material. The two materials are thus mutually beneficial.

Modifying the existing strength formulas for steel columns to address the unique characteristics of composite compression members, design criteria for composite columns were developed by Furlong and others.[1,2] In essence, the procedures were based on the use of a modified yield stress, radius of gyration, and modulus of elasticity for the composite cross section, in combining the contributions of the steel and the concrete. In this fashion, composite columns were given design criteria that took advantage of the combined strength of steel and concrete.

The AISC *Load and Resistance Factor Design (LRFD) Specification*[3] for composite columns is primarily based on a 1979 Structural Stability Research Council (SSRC) paper.[2] It restricts the yield stress to 55 ksi. The limitation was introduced because of the perceived need for strain compatibility between the steel and unconfined concrete, as the latter appears in encased composite columns. Specifically, a limit strain of 0.0018 was imposed on the composite element, because this is the value at which unconfined concrete becomes unstable and begins to spall. With a modulus of elasticity of steel of 29,000 ksi, this strain corresponds to a yield stress of approximately 55 ksi.

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It is the purpose of this paper to examine the 55 ksi yield stress limitation in view of the performance characteristics of composite tubular columns. Concrete that is confined by the continuous shell provided by steel tubes are shown to be capable of sustaining substantially higher strains than 0.0018 before either material fails. The potential benefits for composite structures may be significant.

The scope of the work did not allow for the development of a new composite column strength model. However, the conservatism of the current criteria point to a significant need for further studies of this problem. In particular, a strength model that incorporates concrete confinement effects will be most useful.

2. TECHNICAL BACKGROUND

Prior to the publication of the AISC *Load and Resistance Design Specification*[3] in 1986, structural steel design had no officially sanctioned guidelines for the design of composite compression members. General provisions had been given in the reinforced concrete (RC) design code,[4] although these criteria tended to be very conservative. This was partly because of the relatively small area of reinforcement of RC columns, as compared to composite members.

Economy of construction demands that the combined strenqth and stiffness of steel and concrete be as close to optimally utilized as possible. Furlong[1] recognized that the steel column design rules might be the most suitable for achieving this for composite columns. This approach had the additional advantage that designers were familiar with the underlying criteria.

The majority of the supporting empirical data is summarized in the 1979 report of Task Group 20 of the Structural Stability Research Council.[2] The task group was established in 1973 as a standing committee of the council, to address the behavior of concrete filled tubes and concrete encased structural steel composite columns. It was postulated that steel-concrete composite compression members should behave as plain steel columns if, in the composite cross section, the strength and stiffness of the structural steel alone were several times greater than those of the structural concrete.

Much of the research on concrete-filled steel tubes and encased shapes was conducted by Furlong,[1] Kloppel and Goder,[6] Knowles and Park,[7] Salani and Jims,[8] Gardner and Jacobson,[9] Janss,[10] Loke,[11] Johnson and May,[12] and Bridge and Roderick.[13] The recommendations for concrete-filled tubes drew primarily on the 1976 Furlong paper, which summarized the work of these individuals, and proposed using the AISC allowable stress[5] column equations for composite columns.

The results of 68 tests of axially loaded concrete-filled tubes were addressed in the SSRC report. The tubular specimens ranged in diameter from 1.00 to 14.00 inches, with effective lengths between 10.0 and 91.0 inches. Actual steel yield stress varied from 32.1 to 87.8 ksi, with concrete compressive strengths from 2.60 to 9.60 ksi. A table of data comparing the test loads with the theoretical allowable loads, based on the proposed modifications to the AISC allowable stress equations, was prepared to give an indication of actual safety factors. The ratios varied from 1.28 to 3.68, with an average of 2.26 and a coefficient of variation of 20 percent (0.20).

The researchers envisioned two different approaches to composite column strength. The initial studies[8,9] led to equations that were based on the premise that the member was a concrete column reinforced with a steel tube. With this method, the strength was determined with equations similar to those developed for the American Concrete Institute (ACI).[12] Subsequent research has focused on a methodology that considered the element to be a steel column filled with concrete, thus developing predictor equations based on the structural steel design specifications.[5]

Furlong proposed that the logic of the AISC Specification could be applied to axially loaded composite columns if the influence of the concrete could be accounted for in an efficient, convenient manner. He suggested that the strength, stiffness, and representative cross-sectional parameters could be given as equivalent values of yield stress, F_y^* modulus of elasticity E^*, and radius of gyration, r^*. These parameters were defined as:

$$F_y^* = F_y + 0.85 f_c'(A_c / A_s) \tag{1}$$

$$E^* = E_s + E_c\,(A_c / A_s) \tag{2}$$

$$r^* = \sqrt{\left(\frac{E_s I_s + 0.5 E_c I_c}{E_s A_s + 0.5 E_c A_c}\right)} \tag{3}$$

In Equations 1 to 3, the symbols are: F_y = yield stress of steel shape; f_c' = compressive strength of concrete; A_c = area of concrete in the cross section; A_s = area of steel shape; E_s = modulus of elasticity of steel; E_c = modulus of elasticity of concrete; I_s = moment of inertia of the steel shape; and I_c = moment of inertia of the concrete in the cross section.

Once the parameters had been determined, they could be used in any number of steel column formulas, replacing F_y, E, and r with the respective equivalent values. The original Furlong paper suggested the following allowable stresses:

For $Kl / r \le C_c$:

$$F_a = \frac{\left[1 - \frac{1}{2}\left(\frac{Kl}{r^* C_c}\right)^2\right] F_y^*}{\frac{5}{3} + \frac{3Kl}{8 r^* C_c} - \frac{1}{8}\left(\frac{Kl}{r^* C_c}\right)^3} \tag{4}$$

For $Kl / r > C_c$:

$$F_a = \frac{12\pi^2 E^*}{23(Kl / r^*)^2} \tag{5}$$

where C_c was defined according to the AISC-ASD Specification,[13] substituting E^* and F_y^* for E and F_y, as appropriate.

Using the AISC-ASD Specification equations with the modified parameters, the data from the 68 concrete-filled tube tests gave generally higher factors of safety than for bare steel members. It is likely that this was a result of the stabilizing effect of the concrete fill. That is, the cross-sectional geometry of the tube was maintained by the concrete, delaying local buckling of the steel, and the steel at the same time prevented the concrete from spalling.

Several subsequent refinements were made to the original work of Furlong.[1] Specifically, for concrete-filled tubes, the yield stress was modified to account for the presence of reinforcing steel, and the modulus of elasticity was altered to consider the effects of creep and concrete cracking. In addition, the radius of gyration was simplified to be that of the steel section alone. As a result, the three equations for concrete filled tubes that are used in the current LRFD specification[3] are:

$$F_{my} = F_y + F_{yr}\left(\frac{A_r}{A_s}\right) + 0.85 f_c'\left(\frac{A_c}{A_s}\right) \tag{6}$$

$$\text{with } F_y,\ F_{yr} \le 50 \text{ ksi}$$

$$E_m = 29{,}000 + 0.4 E_c\left(\frac{A_c}{A_s}\right) \tag{7}$$

$$r_m = r_s \tag{8}$$

For short columns, the compressive strength is found as the sum of the compressive capacities of the each component. However, it is implied that superposition is valid only if compatible strains are maintained until the nominal squashing or crushing of steel and concrete has been reached.

It was the judgment of the committee that a strain limitation should be imposed, to prevent the concrete loads from exceeding those which would cause instability and spalling. The upper limit was defined as 0.0018 for axially loaded cross sections, a value consistent with the peak stress as defined by the ACI Code.[4] This provided an upper bound for the steel strength of $0.0018E_s$, or approximately 55 ksi.

3. COLUMN STRENGTH MODELING

Since this study is aimed only at extending the existing criteria, no new strength models are developed here. The requirements of sections E and I of the AISC LRFD Specification[3] therefore constitute the most appropriate set of formulas, particularly because they reflect ultimate limit states.

The AISC-LRFD column equations, as they apply to composite tubular columns, are given in the following. Specifically, without reinforcing steel within the tube, they are:

$$\text{Design Strength} = \phi_c P_n \tag{9}$$

where the resistance factor is $\phi_c = 0.85$, and P_n is the nominal strength:

$$P_n = A_s F_{cr} \tag{10}$$

A_s is the cross-sectional area of the steel shape, and F_{cr} is the critical stress, defined by Equations 11 and 12 for the relevant ranges of slenderness, thus:

for $\lambda_c \leq 1.5$:

$$F_{cr} = (0.5^{\lambda_c^2})\, F_{my} \tag{11}$$

for $\lambda_c > 1.5$:

$$F_{cr} = \left(\frac{0.877}{\lambda_c^2}\right) F_{my} \tag{12}$$

The representative terms in these equations are expressed by Equations 13 through 15, as follows:

$$\lambda_c = \left(\frac{kl}{r_m \pi}\right)\sqrt{\frac{F_{my}}{E_m}} \tag{13}$$

$$F_{my} = F_y + 0.85 f_c' \left(\frac{A_c}{A_s}\right) \tag{14}$$

$$E_m = 29{,}000 + 0.4 E_c \left(\frac{A_c}{A_s}\right) \tag{15}$$

The usual limitations apply to these criteria when used for composite tubes.[3]

4. COLUMN TESTING PROGRAM

4.1 Overview

Considering that the study was essentially an extension of previous investigations, it was felt that an experimental program should be established which would be consistent with the work of Furlong and others.[1] A schedule for full scale column testing was therefore prepared.

Specimen lengths were selected to provide a set of slenderness ratios consistent with previous experiments. In addition, these lengths were also identified as being within the range typically encountered in exposed micropile* construction, thus having direct applicability to another segment of the construction industry.

For analysis purposes, the actual material properties of the steel and grout materials were needed. These were obtained by the usual steel tension tests and chemical analyses, and concrete cylinder compression tests. In addition, deformations and strains were measured with linearly variable displacement transducers (LVDTs) and strain gages, respectively, on the full scale test specimens.

4.2 Selection of Materials

A primary objective of this work was to extend the existing design equations to include higher strength steel materials, specifically, steel with yield stress higher than 55 ksi. Therefore, it was decided to use steel tubes with a nominal yield stress of 80 ksi. This value represents the practical upper limit of yields for steel tubes commonly used in building and bridge construction, and is readily applicable to the AISC Specification. The tube material specification used was that of the American Petroleum Institute (API) N-80 material standard.[14]

In addition to the steel selection, it was necessary to develop a mix design for the concrete materials. It was felt that the integrity of the test procedures could best be maintained by limiting the number of construction variables. This was achieved by using a neat cement grout mixed at the site, where the water/cement ratio could be controlled. Type II cement was used in a ratio of 5:1 (gallons:bag), or 0.45 by weight.

4.3 Test Specimens

A variety of slenderness ratios were obtained through the selection of tubes with varying diameters, wall thicknesses, and lengths. Twenty-foot lengths of tube were readily available, with diameters of 5.5 and 7.0 inches, and wall thicknesses of 0.363 and 0.502 inches, respectively. Two 20-ft sections of pipe, one of each diameter, were cut into five sections each. This yielded three sections of tube which were 3′-0 long, one which was 10′-0, and a section long enough to provide tension test specimens. Table 1 shows the number and types of column test specimens that were selected.

This selection gave slenderness ratios ranging from 15.7 to 65.9. The specimens were filled with grout in the vertical position, using common tremie equipment. This assured proper filling and topping off.

* Micropiles are high-capacity piles often used for underpinning existing structures. They are made of high-strength, small diameter steel tubes, filled with cementitious grout. In some applications, micropiles are used to support structures which will be undermined by a subsequent excavation, thus exposing the piles and effectively changing them from laterally supported piles into columns.

Table 1. Column Test Specimens

Diameter	Wall Thickness	Length	Number
7.0 in.	0.502 in.	3.00 ft	2
7.0 in.	0.502 in.	10.00 ft	1
5.5 in.	0.363 in.	3.00 ft	2
5.5 in.	0.363 in.	10.00 ft	1

4.4 Test Procedure and Specimen Instrumentation

One tension test coupon was obtained from each size of the two lengths of steel pipe to determine yield stress, tensile strength, and elongation at rupture, in accordance with the provisions of ASTM A370-89.[15] In addition, two samples were analyzed for chemical content, as specified in ASTM E1097.[16] For the grout tests the six-inch diameter grout cylinders were tested at 28 days for compressive strength.

Due to the anticipated maximum load of 1,500 kips, it was arranged to have the testing performed at Fritz Engineering Laboratory at Lehigh University. Figure 1 shows one of the short composite columns installed in the laboratory's five million pound Baldwin universal testing machine.

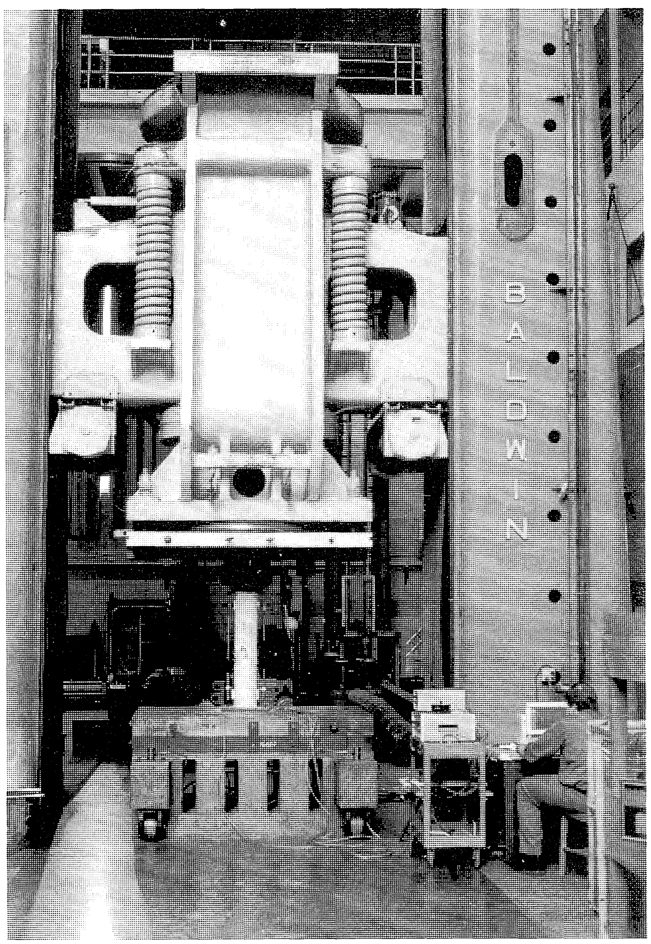

Fig. 1. Test setup for short column.

In order to minimize end effects for the longer specimens, pinned-end fixtures were specified for these specimens. These fixtures had been designed to allow free rotation of the column ends in all directions. A pinned-end fixture was also used on the top during the short-column testing. The bottom end of each short column was supported on a flat table.

Rosette stacked tee strain gages were installed at mid-height of the test specimens, to evaluate load eccentricity effects. Connected to a personal computer, the test team was able to monitor the strain gage data in real time.

Once the installation of the strain gages was completed, the column was placed in the testing machine. In order to achieve a more uniform distribution of load between the tube and grout fill, copper sheets were placed between the ends of each tube and its supports. A whitewash solution was painted on the exterior of the tube prior to loading. This enabled the test team to monitor areas of local yielding throughout the test.

While the whitewash was drying, linearly variable displacement transducers (LVDTs) were mounted on the specimens, as follows:

a. *For the 10′-0 long specimens (see Figure 2):*
Two longitudinal LVDTs were placed on opposite sides of the tube, to measure vertical displacement. In addition, two transverse LVDTs were placed at mid-height, at 90 degree directions relative to each other, to measure lateral displacements. Finally, the output from an additional LVDT was used to monitor the load cell.

b. *For the 3′-0 long specimens:*
Two longitudinal LVDTs were used to measure the vertical displacement. One LVDT monitored the load.

Each tube was loaded continuously to failure, i.e., until the tube was no longer able to sustain its maximum load. In general, LVDT and strain gage measurements were made in increments not exceeding three percent of the anticipated ultimate load, but with increasing frequency beyond initial yielding.

5. TEST RESULTS

5.1 Material Properties

The tension tests of the material in the two N-80 pipe sections confirmed the specified minimum yield stress. Thus, the 7.0-in. tube material gave a yield stress of 86.1 ksi, determined at 0.2 strain offset, and a tensile strength of 111.8 ksi. By comparison, the 5.5-in. tube material had a yield stress of 98.9 ksi, and a tensile strength of 111.6 ksi. Elongations at fracture were 29.5 percent and 25.5 percent, respectively.

Two chips obtained from the same sections of pipe were analyzed for chemical content, in accordance with ASTM

E1097. The results, which satisfy the API N-80 criteria,[14] are summarized in Table 2.

Two cylinders of the material used in filling the pipes were tested to determine the 28-day unconfined compressive strength. The results were consistent, with values of 5,660 and 5,480 psi.

5.2 Column Test Results

All of the column test specimens responded similarly throughout the loading range, first with local yielding occurring in the tubes adjacent to the end fixtures, followed by gradual bending and associated overall yielding of the specimens at maximum load.

Local buckling was not observed. This can be attributed to the restraining effect of the grout fill, which forced the steel tubes to maintain their cross-sectional geometry. Because no local buckling was permitted, the columns did not fail in the typical sudden collapse of tubes, but merely continued to yield gradually. This allowed the specimens to sustain significant loads even after the maximum load had been attained.

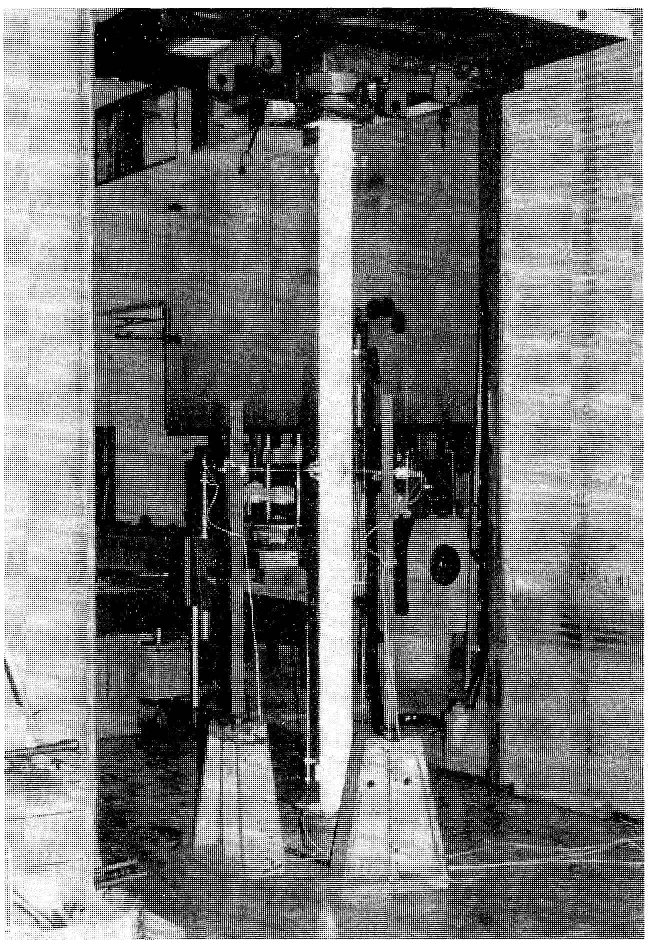

Fig. 2. Test setup for long (10 ft) column.

Table 2.
Steel Chemical Analysis

Element	5.5 in.	7.0 in.
Carbon	0.24	0.32
Manganese, %	1.04	1.50
Phosphorus, %	0.007	0.007
Sulphur, %	0.005	0.017
Silicon, %	0.23	0.031

Upon removal of the specimens from the testing machine, it was apparent that at some point during the load application, bond between the grout and the interior steel surface had been overcome. The grout column was observed to be protruding slightly (less than one sixteenth of an inch) at the top of the steel tube, as seen in Figure 3. This would have occurred during unloading of the specimen, with the grout rebounding to a greater extent than the steel. A summary of the column test results is given in Table 3.

Figures 4 and 5 show typical load-deflection diagrams for the short columns. All of the specimens exhibited a linear relationship until at least 75 percent of the maximum load had been reached. After this point, increasingly large strains were observed, as is common with stub column tests.

Figures 6 and 7 show that the longer specimens behaved linearly until the maximum load was attained. At peak loads, the effects of eccentricity, due to the bent shape of the tube, greatly reduced any further load-carrying capacity of the columns. The maximum sustainable force therefore quickly dropped off as the deformations increased.

6. EVALUATION OF TEST RESULTS

Table 4 summarizes the material and geometric properties used to calculate the nominal strength, P_n, for the test specimens under investigation. Equations 10 through 15 were used

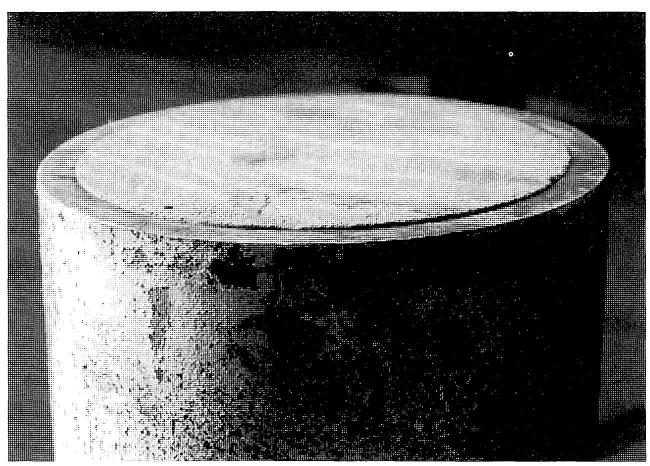

Fig. 3. Evidence of loss of bond between steel and grout.

Table 3.
Summary of Column Test Results

Tube Diameter (in.)	Tube Length (in.)	Slenderness Ratio	Maximum Experimental Loads (kips)
5.50	36.0	19.8	685
5.50	36.0	19.8	584
5.50	120.0	65.9	450
7.00	36.0	15.7	1181
7.00	36.0	15.7	1242
7.00	120.0	52.2	969

for this purpose. In order to establish a measure of the performance of the model used to formulate the equations, the test load is then divided by the nominal strength. These ratios range from 0.93 to 1.36, with an average value of 1.19, a standard deviation of 0.14, and a coefficient of variation of 12 percent.

Table 5 summarizes the results from similar tests* performed during the past several years.[2] It can be seen that the results of the current study are consistent with the historical data. Specifically, the previous tests resulted in an average test-to-design ratio of 1.38, with a standard deviation of 0.27 and a coefficient of variation of 20 percent.

Although the average ratio of test loads to design strengths is approximately 12 percent higher for the past studies, the variability of these tests, as measured by the coefficient of variation, is 42 percent higher. This is not an unexpected finding, considering the diversity of test specimens, testing equipment and testing procedures, as well as the great number of researchers involved in the studies.

7. RESISTANCE FACTOR DEVELOPMENT

The load and resistance factor design (LRFD) criteria are probabilistically based, with the means and coefficients of variation of the properties of the material(s) in the cross section, the relevant geometrical measure(s) of the member, and the correlation between theory and tests being the significant parameters for the resistance side of the design equation.[17,18] In the current study, it was decided to base the resistance factor development on the material and geometric properties of the steel tube, along with the correlation between full-scale test results and the strength predicted by the composite column model of the LRFD Specification.

The resistance factor, ϕ, is determined from the usual equation:[3,18]

* Sixteen of the original 84 tests were spiral welded tubes filled with concrete, and are therefore not included in the comparisons.

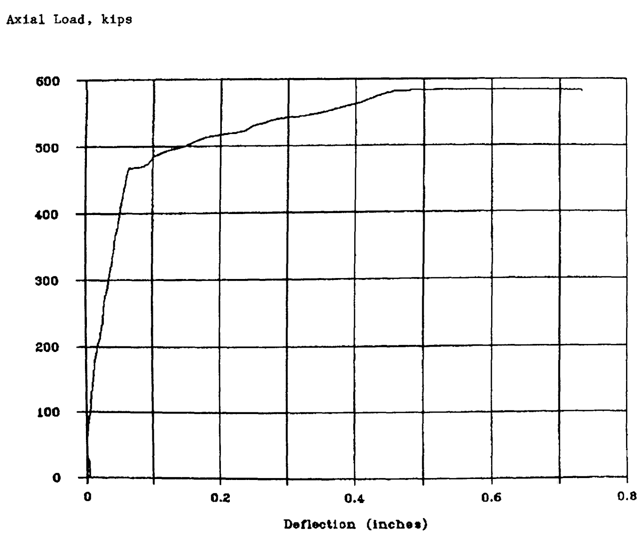

Fig. 4. Load-deflection curve for short column (D = 5.5 in.; t = 0.363 in.).

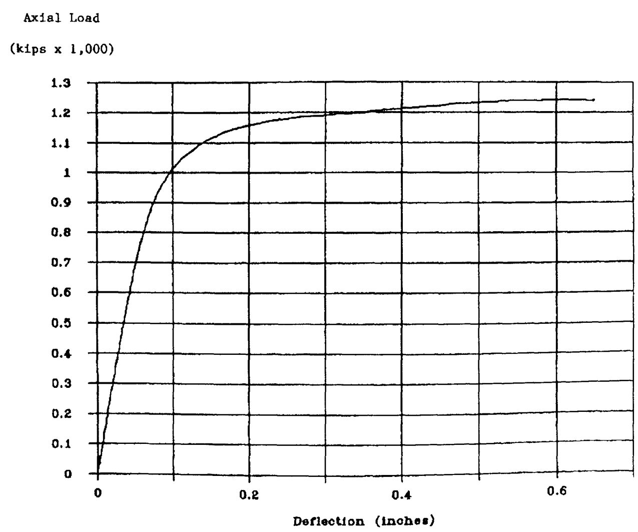

Fig. 5. Load-deflection curve for short column (D = 7.0 in.; t = 0.502 in.).

Table 4. Axially Loaded High-Strength Filled Tubes								
O.D. (in.)	A_s (in.^{2})	A_c (in.^{2})	F_y (ksi)	f_c' (ksi)	Kl (in.)	P_{test} (kips)	P_n (kips)	$\frac{P_{test}}{P_n}$
5.50	5.86	17.90	98.90	5.57	36	685	627	1.09
5.50	5.86	17.90	98.90	5.57	120	450	349	1.29
7.00	10.25	28.24	86.10	5.57	120	969	713	1.36
5.50	5.86	17.90	98.90	5.57	36	584	627	0.93
7.00	10.25	28.24	86.10	5.57	36	1181	984	1.20
7.00	10.25	28.24	86.10	5.57	36	1242	984	1.26
							Avg	1.19
							Std Dev	0.14
							COV	12%

$$\phi = (R_m / R_n) \exp(-0.55\beta V_R) \quad (16)$$

where R_m is the mean resistance, β is the reliability index, and V_R is the coefficient of variation of the resistance. Using the basic resistance equation to express the magnitude of the strength as influenced by the most important parameters, Equation 17 relates the terms in the common form:[18]

$$R = R_n MFP \quad (17)$$

where R is the actual (random) resistance, and the symbols M, F, and P are the material, fabrication, and professional factors, respectively. These reflect the influence of the variability of the material strength, the geometry of the member, and the correlation between theory and test.

As already noted, for the current study it was decided to base the material factor development on the properties of the steel tube alone. This was done partly because the contribution of the steel shape to the composite column strength usually outweighs that of the concrete. As an example, using the pure squash load ratio $(A_sF_y)/(A_cf_c')$, for the six current tests the ratio averages 5.71; for the six current plus the 68 earlier column tests the ratio averages 2.48. The weighted relative contribution of the concrete strength variability to the coefficient of variation of the fabrication factor will therefore be small.

The material factor was consequently defined by

$$M = F_{sy} / F_y \quad (18)$$

where F_{sy} is the actual static yield stress, and F_y is the specified minimum value for the steel in the tube.

Similarly, the fabrication factor for the composite tubular column was defined as

$$F = A / A_n = A / A_s \quad (19)$$

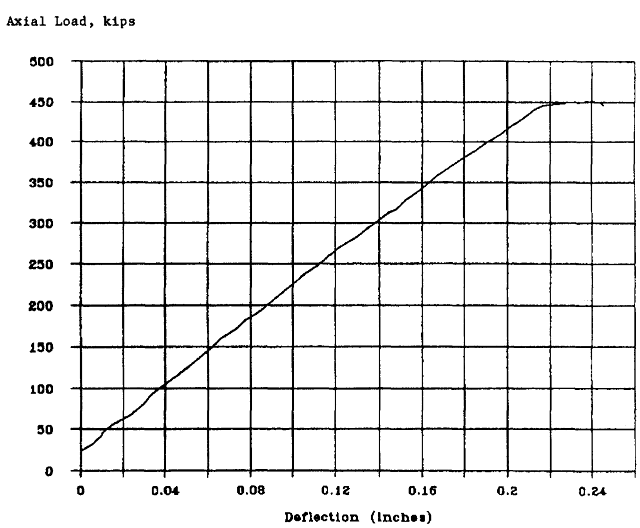

Fig. 6. Load-deflection curve for long column (D = 5.5 in.; t = 0.363 in.).

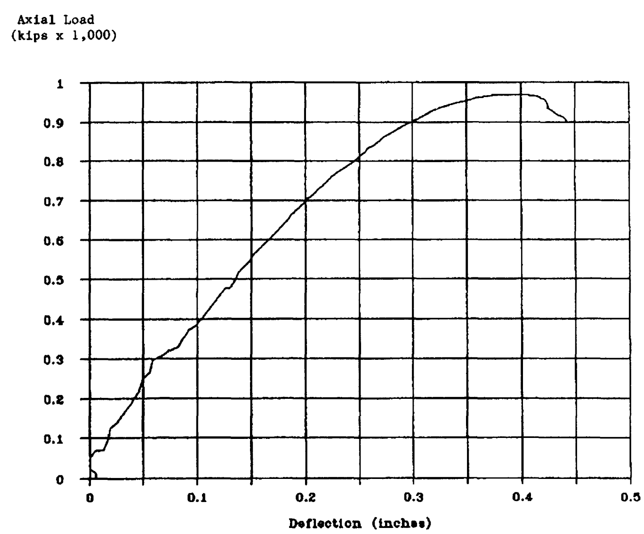

Fig. 7. Load-deflection curve for long column (D = 7.00 in.; t = 0.502 in.).

Table 5. Data from Earlier Column Tests[2]

O.D. (in.)	A_s (in.^{2})	A_c (in.^{2})	F_y (ksi)	f'_c (ksi)	Kl (in.)	P_{test} (kips)	P_n (kips)	$\frac{P_{test}}{P_n}$
3.74	5.07	5.92	39.9	2.94	33.9	229	206	1.11
					55.9	209	189	1.11
					78.0	203	166	1.23
	1.63	9.36	50.7	3.62	33.9	150	105	1.42
					55.9	131	96	1.37
					78.0	119	83	1.43
8.50	4.22	52.5	42.3	3.32	87.4	371	307	1.21
				4.32		509	347	1.47
	6.13	50.6	56.8	3.32		549	455	1.21
			50.8	4.32		645	462	1.40
3.74	1.63	9.36	49.0	3.49	80	104	80	1.30
4.76	2.14	15.6	45.2	3.06	41.3	162	131	1.23
				3.51		192	137	1.40
				3.06	91	143	110	1.29
				3.51		163	115	1.42
	3.11	14.7	49.8	3.06	41.3	227	184	1.23
				3.51		245	189	1.30
				3.06	91	180	152	1.18
				3.51		195	156	1.25
1.00	0.11	0.68	76.0	4.04	42	3.52	2.34	1.50
1.50	0.48	1.29				24.7	18.5	1.34
2.00	0.40	2.74				27.1	26.5	1.02
3.00	0.59	6.47		3.95		72.0	56.2	1.28
14.0	18.7	135	51.5	5.52	22	2576	1595	1.62
				4.76		2408	1508	1.60
	13.5	140		3.40	21.1	1671	1099	1.52
	8.07	146	40.1	3.04	21.5	791	700	1.13
5.01	0.99	18.7	53.8	9.60	19.7	289	203	1.43
			47.7			289	197	1.47
5.00	1.78	17.9	53.8		20.0	293	238	1.23
			47.7			293	228	1.29
4.00	1.49	11.1	87.8	4.95	60	184	141	1.31
						180		1.28
4.76	2.33	15.5	65.5	4.99	41.3	260	205	1.27
				4.29		246	197	1.25
				3.76		214	190	1.13
6.00	2.29	26.0	60.2	3.03	89.4	211	175	1.21
						198		1.13
3.01	0.63	6.5	52.7	3.62	60	55.0	43.2	1.27
				5.93	24	95.2	63.5	1.50
				3.76		74.2	52.2	1.42
4.50	1.72	14.2	60.0	4.20	33	165	148	1.12
						170		1.15
5.00	1.46	18.2	42.0	5.10	59	143	128	1.10
						140		1.10
						148		1.16
6.00	1.14	27.1	48.0	3.05		153	118	1.30
				3.75		163	132	1.23
				3.75		165		1.25
5.51	6.14	17.7	38.5	4.66	16	663	305	2.17
			39.0			663	308	2.15
5.53	3.25	20.8	41.9	4.74	16	410	219	1.87
			43.2			410	223	1.84
6.62	3.62	30.6		4.56	32	451	271	1.66
				6.28		502	314	1.60
						475		1.51
				3.34		392	240	1.63
3.50	2.36	7.26	58.0	5.81	68	138	127	1.09
				5.75	56	160	140	1.16
				5.65	44	161	151	1.07
				6.06	32	206	163	1.27
				5.92	20	223	169	1.32
3.25	0.55	7.74	70.0	6.00	68	50.5	52.7	0.96
				5.36	56	66.2	57.0	1.16
				5.92	44	80.0	65.8	1.22
					32	90.0	71.0	1.27
					20	110.0	74.9	1.47
					10	119.2	76.8	1.55
							Avg	1.33
							Std Dev	0.23
							COV	17%

Table 6. Statistical Properties and Resistance Factors for Composite High-Strength Steel Tubular Columns									
No. of Tests	F_y	M_m	V_M	F_m	V_F	P_m	V_p	V_R	ϕ
6	55	1.15	0.08	1.0	0.05	1.70	0.06	0.11	1.63
31	55	1.15	0.08	1.0	0.05	1.42	0.16	0.19	1.20
6	80	1.15	0.08	1.0	0.05	1.31	0.08	0.12	1.23
31	Nom.	1.15	0.08	1.0	0.05	1.24	0.11	0.15	1.12
6	Act.	1.15	0.08	1.0	0.05	1.19	0.12	0.15	1.06
31	Act.	1.15	0.08	1.0	0.05	1.22	0.12	0.15	1.09

where A is the actual cross-sectional area, set equal to that of the steel tube alone, i.e. $A = A_s$. A_n is the nominal (catalog or handbook) area of the tube.

The simplification was made since the concrete area, A_c, is governed by the inside dimension of the steel tube. Its coefficient of variation will therefore be at most equal to that of the steel, but usually much less, because of larger absolute value of A_c. Furthermore, although an elastic transformation of the concrete area into an equivalent area of steel could be made, based on the modular ratio $n = E_s / E_c$, this would have limited applicability because of the ultimate limit state modeling of the composite column strength. It was therefore decided to base the fabrication factor only on the properties of the steel tube.

The professional factor is given by the expression

$$P = P_{test} / P_n \tag{20}$$

where P_{test} is the experimental member capacity, and P_n is the nominal strength, based on the ultimate limit state model. P_n is computed on the basis of the actual material and geometric properties of the member that has been tested, such that the professional factor reflects only the reliability of the predictor equation itself. That is, any variation between test and theory is attributed to modeling inadequacy. In other words, the test is presumed to give the correct value of the strength.

The key statistical characteristics that are used in the LRFD development are the mean and the coefficient of variation. For the composite tubular columns of the current study the relevant expressions for M, F, and P are:

For M:

$$M_m = [(F_{sy} / F_y)_m]_{tube} = [(F_{sy})_m / F_y]_{tube} \tag{21a}$$

$$V_m = [V_{Fsy}]_{tube} \tag{21b}$$

and it is noted that F_y is the specified minimum yield strength of the tube steel, and therefore deterministic.

For F:

$$F_m = [(A / A_n)_m] = [A_m / A_n]_{tube} \tag{22a}$$

$$V_F = [V_A]_{tube} \tag{22b}$$

For P:

$$P_m = (P_{test} / P_n)_m \tag{23a}$$

$$V_p = V \text{ (test-to-theory ratio)} \tag{23b}$$

Combining all of these influences, the mean and coefficient of variation of the resistance of the member are the found by the common procedure.[18]

The specific aim of the current study was to examine the performance of composite tubular columns in steel grades with yield stress higher than 55 ksi, in comparison to the LRFD column strength model. The test data for such members provide the information necessary to calculate the resistance factors. As shown in Table 6, ϕ-values have been calculated for several cases, as follows:

1. *For the six tests conducted as part of the current study:*
   (a) F_y = 55 ksi
   (b) F_y = specified minimum value = 80 ksi
   (c) F_y = actual yield stress
2. *For the 25 previous tests, for which the yield stress was larger than 55 ksi, plus the six from the current study:*
   (a) F_y = 55 ksi
   (b) F_y = specified minimum value
   (c) F_y = actual yield stress

It is noted that the specified minimum yield stress for the 25 previous tests could not be found in the available literature. In lieu of this value, the actual yield stresses were used for the strength computations, in combination with the nominal yields for the six current tests.

The mean material factor, M_m, and its associated coefficient of variation are the representative values from Galambos and Ravindra[17] for high-strength steel tubes. The mean fabrication factor value of 1.0 reflects the assumption that the shape is equally likely to have areas larger and smaller than the nominal value. The coefficient of variation for fabrication reflects good control of tolerances.[18]

The resistance factor values shown in Table 6 are all larger than 1.0, and some of them significantly so. Since ϕ reflects the collective uncertainty of the member strength, as influenced by all of the relevant parameters, it is normally expected to be less than 1.0. In this case it is clear that the high ϕ-values can be attributed to the column strength model that has been used. In particular, limiting the yield stress to 55 ksi leads to very high factors. Even by using the actual specified minimum of F_y = 80 ksi for the six columns that were tested in the current project (line 3 of Table 6), the value of ϕ remains larger than 1.0.

The findings point to several necessary improvements in the criteria for composite tubular columns. First, an improved ultimate strength model is needed. This must take into account the significantly higher strength of concrete under fully confined conditions. The current limitation of F_y = 55 ksi for the structural steel is based on a perceived need for strain compatibility between steel and unconfined concrete. This is overly conservative for tubular members. However, such a model development is beyond the scope of this project.

Secondly, as a step in the right direction, but short of providing a new P_n-expression, it is important to remove the 55 ksi limitation for F_y for tubular shapes. Using the specified minimum value of F_y = 80 ksi for the six tests of the current study, along with the current P_n-equation, still leads to ϕ-values larger than 1.0, but they are more realistic numbers. Actual F_y-data should not replace the nominal values, since a benchmark magnitude has to be provided for P_n.

It is therefore recommended that the composite column criteria for tubular members be revised to allow for nominal yield stress values as high as 80 ksi. To maintain the correlation with the steel column criteria, it is further suggested that the ϕ-value of 0.85 be maintained, and that the design strength be computed as prescribed in Sections E and I of the LRFD Specification. Still conservative to a significant degree, the removal of the 55 ksi limitation will move the design strength closer to the actual level.

8. CONCLUSIONS AND RECOMMENDATIONS

The current AISC design specification[3] for composite columns restricts the maximum usable yield stress to 55 ksi. This limitation is attributed to the requirement of strain compatibility between the steel and the concrete in the composite column cross section. Whereas this is a valid provision for encased members, it is not applicable to tubular members filled with concrete. The integrity of the composite column can be maintained at yield stresses substantially hiqher than 55 ksi.

Six composite columns with yield stress in excess of 80 ksi were tested as part of this study. Five of the members sustained ultimate loads in excess of the predicted nominal strength, with one exhibiting a test load slightly less than nominal. The ratios of test load to design strength varied from 0.93 to 1.36, with an average value of 1.19, and a coefficient of variation of 12 percent. This compares well with the results of 68 previous tests of generally milder steel elements, which had test load-to-design strength ratios ranging from 0.96 to 2.17, with an average value of 1.33, and a coefficient of variation of 17 percent. In 25 of these 68 tests, steels with yield stresses in excess of 55 ksi were tested. This sub-group exhibited test-to-design strength ratios ranging from 0.96 to 1.55, with an average value of 1.23, and a coefficient of variation of 11 percent, also comparing favorably to the performance of the entire group.

It is important to note that the steel tubes maintained their cross-sectional shape, and that no distress of the concrete was evident at the ultimate load. It is therefore recommended that the 55 ksi yield stress limit be increased to 80 ksi.

Further studies need to be conducted to develop an improved composite column model for tubular members. The current approach is very conservative, and tends to penalize these high capacity members.

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10. REFERENCES

1. Furlong, Richard W., "AISC Column Logic Makes Sense for Composite Columns, Too," AISC *Engineering Journal,* Vol. 13, No. 1, First Quarter, 197-6 (pp. 1–7).
2. Task Group 20, Structural Stability Research Council, "A Specification for the Design of Steel-Concrete Composite Columns," AISC *Engineering Journal,* Vol. 16, No. 4, Fourth Quarter, 1979 (pp. 101–115).
3. American Institute of Steel Construction (AISC), *Manual of Steel Construction—Load and Resistance Factor Design,* First Edition, AISC, Chicago, Illinois, 1986.

4. Kloppel, K. Von, and Goder, W., "Investigations of the Load Carrying Capacity of Concrete-Filled Steel Tubes and Development of a Design Formula," *Der Stahlbau,* Vol. 26, No. 1, January and February, 1957 (in German).
5. Knowles, R. B. and Park, R., "Strength of Concrete-Filled Steel Tubular Columns," *Journal of the Structural Division,* ASCE, Vol. 95, No. ST12, December 1969 (pp. 2565–2587).
6. Salani, H. J., and Sims, J. R., "Behavior of Mortar-Filled Steel Tubes in Compression," *Journal of the American Concrete Institute,* Vol. 61, No. 11, October 1964.
7. Gardner, N. J., and Jacobsen, E. R., "Structural Behavior of Concrete-Filled Steel Tubes," *Journal of the American Concrete Institute,* Vol. 64, No. 7, July 1967 (pp. 404–413).
8. Janss, J., "Calculation of Ultimate Loads on Metal Columns Encased in Concrete," *Report No. MT 89,* Industrial Center of Scientific and Technical Research for Fabricated Metal, Brussels, April 1974 (in French).
9. Loke, Y. O., *The Behavior of Composite Steel-Concrete Columns,* Ph.D. Dissertation, University of Sydney, Sydney, Australia, December 1968.
10. Johnson, R. P., and May, I. M., "Tests on Restrained Composite Columns," *The Structural Engineer,* Vol. 56B, No. 2, June 1978.
11. Bridge, R. O., and Roderick, J. W., "The Behavior of Built-Up Composite Columns," *Research Report No. 306,* School of Civil Engineering, University of Sydney, Sydney, Australia, July 1977.
12. American Concrete Institute (ACI), *Building Code Requirements for Reinforced Concrete—ACI 318-89,* Chapter 10, Commentary Section R10.2.6, ACI, Detroit, Michigan, November 1989.
13. American Institute of Steel Construction (AISC), *Manual of Steel Construction—Allowable Stress Design,* Ninth Edition, AISC, Chicago, Illinois, 1989.
14. American Petroleum Institute, *Specification for Casing and Tubing—5CT (Spec. 5CT),* Second Edition, API, Washington, D.C., November 1989.
15. American Society for Testing and Materials (ASTM), *Standard Test Methods and Definitions for Mechanical Testing of Steel Products,* ASTM Standard No. A370-89, ASTM, Philadelphia, PA, 1989.
16. American Society for Testing of Materials (ASTM), *Standard Guide for Direct Current Plasma Emission Spectrometry Analysis,* ASTM Standard No. E1097-86, ASTM, Philadelphia, PA, 1986.
17. Galambos, T. V., and Ravindra, M. K., "Properties of Steel for Use in LRFD," *Journal of the Structural Division,* ASCE, Vol. 104, No. ST9, September 1978 (pp. 1459–1468).
18. Ravindra, M. K., and Galambos, T. V., "Load and Resistance Factor Design for Steel," *Journal of the Structural Division,* ASCE, Vol. 104, No. ST9, September 1978 (pp. 1337–1353).

NOMENCLATURE

A_c = Area of concrete in column cross section
A_r = Area of longitudinal bar reinforcement in column cross section
A_s = Area of steel (rolled shape or tube) in column cross section
E_s = Modulus of elasticity of steel = 29,000 ksi (constant)
E_c = Modulus of elasticity of concrete/grout/mortar
E_m = Modified modulus of elasticity for composite column cross section
F_a = Allowable compressive axial stress
F_e = Euler (elastic) buckling stress = $\pi^2 E / (KL / r)^2$
F_e' = Euler stress divided by factor of safety
F_{my} = Modified value of yield stress for composite column cross section
F_y = Specified minimum yield stress of structural steel
F_{yr} = Specified minimum yield stress of longitudinal reinforcement
I_s = Moment of inertia of steel shape
I_c = Moment of inertia of concrete/grout/mortar shape
K = Effective length factor
l = Unbraced length
P = Applied load
P_a = Allowable axial compressive load
P_n = Nominal axial compressive load
f_a = Axial compressive stress at service load
f_c' = Specified compressive strength of concrete
r_m = Radius of gyration of the steel tube in composite column