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Ultimate Axial load on Concrete filled stainless steel tubular (CFSST) short columns under Monotonic loading-a critical review

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ABSTRACT
The State of the art of concrete filled steel tubular columns is presented in this paper. Experimental data has been collected and compiled in a comprehensive format listing parameters involved in the study. Areas of further research are presented and results of ongoing experimental and numerical investigations are also shown.

Introduction

Concrete filled steel tubes (CFST) are gaining increasing usage in modern construction practice throughout the world, particularly in Australia and the Far East. This increase in use is largely due to the structural and economical advantages offered by concrete filled tubes over open and empty sections, as well as their aesthetic appeal. From a structural viewpoint, hollow sections exhibit high tensional and compressive resistance about all axes when compared with the open sections. Additionally, the exposed surface area of a closed section is approximately two-thirds that of a similar sized open section, thus demanding lower painting and fireproofing costs [1].

The use of stainless steel columns filled with concrete is relatively new and innovative, and not only provides the advantages outlined above but also brings the durability associated with stainless steel.

Composite columns comprise a combination of concrete and steel and utilize the most favorable properties of the constituent materials. Use of composite columns can result in significant savings in column size, which ultimately can lead to considerable economic savings. This reduction in column size provides is particularly beneficial where floor space is at a premium, such as in car parks and office blocks.

The term ‘composite column’ refers to a compression member in which the steel and Concrete elements act compositely. The role of the concrete core in a composite column is not only to resist compressive forces but also to reduce the potential for buckling of the steel member. The steel tube reinforces the concrete to resisting tensile forces, bending moments and shear forces, and offers confinement to the concrete.

Composite columns can buckle in local or overall modes, but this investigation is focused on the cross-section resistance of short composite columns, where only local buckling effects were exhibited.

Stainless steel possesses natural corrosion resistance; significantly, this means that, appropriately specified, the surface can be exposed without the need for any protective coatings. The principal disincentive for the utilization of stainless steel for structural elements is the high initial material cost, but, considered on a whole-life basis, cost comparisons with other metallic materials become more favorable [2].

Concrete infilling on stainless steel tubes maintains the durable and aesthetic exposed surface, but will lead to reduced column sizes and material thickness, both of which have clear economic incentives.

A number of advantages of CFTs were cited in the paper's introductory section:

1. The load-carrying capacity of the member is increased without increasing the member Size ;
2. The tube provides ideally-placed reinforcement;
3. Thinner steel tubes may be used since the concrete core forces all local buckling modes outward, delaying the onset of local buckling;
4. The tube prevents concrete spelling which increases member ductility;
5. The tube prevents concrete spelling which increases member ductility;
6. The tube serves as the concrete formwork in construction; and CFTs will have a higher fir resistance and require less fireproofing than hollow tubes because the concrete has a larger thermal capacity than air.
7. The tube serves as the concrete formwork in construction; and CFTs will have a higher fire resistance and require less fireproofing than hollow tubes because the concrete has a larger thermal capacity than air.
8. A higher critical buckling load due to the stiffening effect of the concrete;

Types of CFST Colomns:

There are two types of composite columns, those with steel tube with steel section encased in concrete and those with steel tube in-filled with concrete are commonly used in buildings. Basic forms of cross-sections representative of composite columns are indicated in Fig. 1.

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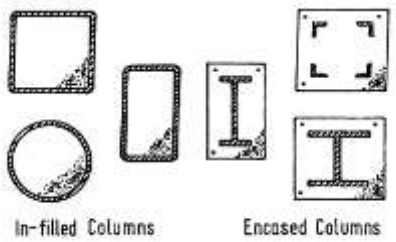

Figure 1: In-filled and encased sections

Concrete-encased steel composite columns have become the preferred form for many seismic-resistant structures. Under severe flexural overload, concrete encasement cracks resulting in reduction of stiffness but the steel core provides shear capacity and ductile resistance to subsequent cycles of overload. Concrete-filled steel tubular columns have been used for earthquake-resistant structures, bridge piers subject to impact from traffic, columns to support storage tanks, decks of railways, columns in high-rise buildings and as piles. Concrete-filled steel tubes require additional fire-resistant insulation if fire protection of the structure is necessary. Because of the increased use of composite columns, a great deal of theoretical and experimental work has been carried out,

Constitutive Relationships for Concrete Filled Steel Tubes (CFST) based upon Mechanics by Equilibrating forces on a semicircular Steel tube: [Tomii,etal.,1973;Ichinohe,et al.,1991], [Kent and Park, 1971], [Cederwall,1988], [cai1987;Ichinohe,et.al,.1991].

Concrete Constitutive Relationships:

The stress-strain behavior of concrete is a function of its uniaxial behavior, the shape of the cross-section, the relative ratio of length to diameter or depth(L/D) and the relative ratio of concrete to steel strength, commonly expressed as the diameter to thickness ratio(D/t). the latter two factors affect the degree of concrete confinement provided by the steel. The constitutive relationships for circular CFST section should be able to model both an increase in concrete strength & an increase in concrete ductility due to the confining action of the steel [Tomii,etal.,1973;Ichinohe,et al.,1991].

Circular CFST section for concrete:

The additional strength of concrete under confinement was first recognized in reinforced concrete members. Concrete confined by steel hoops develops a greater strength than unconfined concrete [Kent and Park, 1971]. Correspondingly, the strength enhancement due to confinement of the concrete in circular CFST is considered in the triaxial state of stress, the confined strength, σ_{cc} is defined in the following expression:

$$\sigma_{cc}=\sigma_c+K\sigma_c \qquad (1)$$

Where

σ_{cl} is the lateral pressure exerted by the steel tube on the concrete core K is the triaxial factor that has a value of 4 to 6 in low strength domain and a value of 3 to 4 for the high strength domain [Cederwall,1988]

The lateral pressure σ_{cl} can be derived from mechanics by equilibrating forces on a semicircular steel tube as show in figure

The circumferential tensile force in the tube is $2t\sigma_{sh}$, where σ_{sh} is the hoop or circumferential stress in the steel tube .The equilibrating force exerted by the lateral pressure of the concrete, assuming a thin-walled tube, Ie. D>>t; is $D\sigma_{cl}$, the resulting expression for the lateral pressure is:

$$\sigma_{cl}=[2t/D]\sigma_{sh} \qquad (2)$$

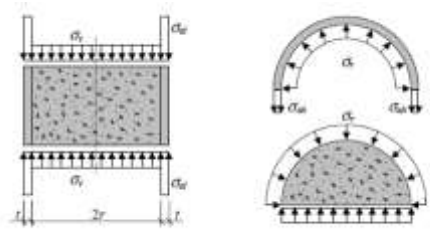

Figure.2: Lateral pressure & hoop stress on steel tube

Steel Constitutive Relationships

The steel in a concrete filled steel tube may be in a state of either uniaxial stress or biaxial stress. The effect of confinement in CFST subjected to compressive loads will result in different behavior for circular tubes. The hoop stress induced in circular & rectangular tubes. The hoop stress induced in circular tubes by expanding concrete results in a loss of longitudinal capacity which must be modeled in the compressive stress-strain curve of the steel. Any interaction that may occur between the steel & concrete in the tension region is neglected due to the concrete in tension offering little resistance. Therefore steel in tension acts independently of the concrete, & is modeled as if it were a hallow tube.

Circular CFST section for Steel:

The concrete in a circular CFST is fully confined; therefore both strength & ductility are enhanced. The steel stress-strain relationship is modeled accordingly by approximately including in the formulation the biaxial stresses induced by the lateral pressure of the confined concrete. When the steel reaches yield stress at which the large plastic strains occur, part of the load resisted by the steel is transferred to the concrete core .according to van Mises yield criteria, the longitudinal stress in the steel decreases with increasing lateral or hoop tensile stress. Therefore, the strength of the concrete core is continuously enhanced.

The enhancement is also caused by the dramatic lateral expansion of concrete due to the development of cracks at the late stage of loading. Finally, the failure of the column occurs when the resultant compressive force carried by the steel tube & the concrete core reaches the ultimate value. The total axial load N_0 carried by a concrete-filled tubular column with a vertical stress σ_{sv} in the steel tube & a vertical stress σ_{cv} in the concrete core can be approximately as:

$$N=A_C\sigma_{CV} + A_S\sigma_{SV} \qquad (a)$$

Where

A_c is the area of concrete

$A_c=\pi D_C^2/4$

A_s is the area of steel

$A_c=\pi tD_c$

Substituting in to Eqn. (1) than

$$\sigma_{cl} = \frac{2t}{D_c}\sigma_{sh} = \frac{A_s}{2A_c}\sigma_{sh} \qquad (b)$$

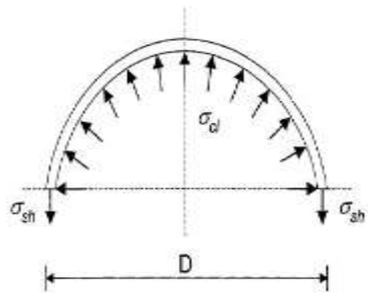

Figure.3: Definition of concrete and steel stress

From the above figure defines the relationship between steel & concrete lateral pressure. Substituting Eqn.(2) and (1) into Eqn.(a).

$N_0 = A_C\sigma_C + A_S\ (\sigma_{SV}+0.5K\sigma_{sh})$ (c)

The maximum value of N_0 can be achieved when

$$\frac{\partial N}{\partial \sigma sv} = \frac{1+k}{2}\frac{\partial \sigma sh}{\partial \sigma sv} = 0$$

$$\frac{\partial \sigma sh}{\partial \sigma sv} = -\frac{2}{k} \qquad \text{(d)}$$

The state of stress in the tube at the yield point maybe analysed by considering the van yield criterion [cai1987; Ichinohe,et.al,.1991].the von Mises-yield criterion is expressed by the following equation:

$$\sigma_{sv}^2 + \sigma_{sh}^2 - \sigma_{sv}\sigma_{sh} = \sigma_y^2$$

Differentiating Eqn. (3) with respect to σ_{sv}

$$\sigma_{sv}\left[2 - \frac{\partial\sigma_{sh}}{\partial sv}\right] - \sigma_{sh}\left[1 - 2\frac{\partial\sigma_{sh}}{\partial sv}\right] = 0$$

substitution eqn.(4) in to Eqn.(5) & rearranging it,

2(k+1) σsv-(4+k) σsh (e)

Substitution Eqn. (d) in to Eqn.(a) can be rearranged to solve for the ratio of the longitudinal steel stress to the yield stress in terms of triaxial factor "K",

$\sigma_{sv}=\gamma_v\sigma_y$

Where

$$\gamma v = \frac{k+4}{\sqrt{3(k+1)^2+9}}$$

If a triaxial factor, of 4 is assumed, Eqn. (b)produces a reduction value of γ_v equal to0.873.therefore,the longitudinal stress that causes yielding in the steel tube is reducedto0.873σ_y due to lateral confinement effect value of "K". The corresponding yield strain is also reduced by the same factor. Once the compressive stress reaches the reduced yield strength, the steel enters the perfectly plastic region, remaining at the reduced stress as show in figure

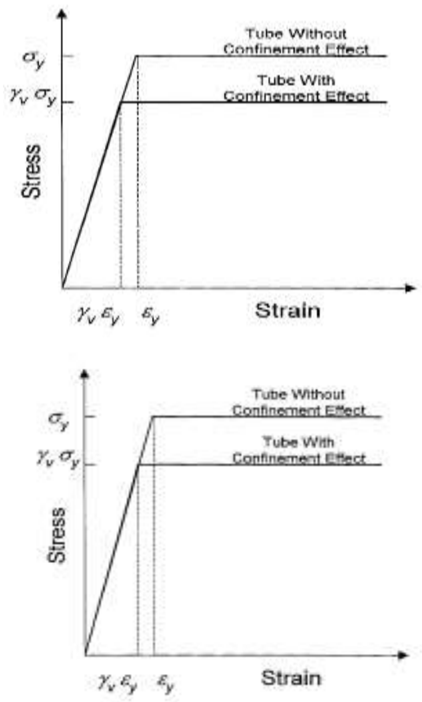

Figure.4: Steel stress-strain curve with (or) without confinement effect

The hoop stress ratio (γ_h) may be computed by using the reduction factor for the compressive stress. This value is defined as the circumferential stress in steel tube to the yield stress of the steel tube. To solve for the ratio of the steel hoop stress to yield stress in terms of the longitudinal steel stress, substituting Eqn. (c) in to Eqn.(d),than

$\sigma_{sh} = \gamma_h\sigma_y$

Where

$$\gamma_1 h = 0.5(\sqrt{(4 - 3\gamma_1 v^{\uparrow}2\ )} - \gamma_1 v \qquad \text{(f)}$$

When γ_v equal to 0.873, Eqn.(e) produces a reduction value of γ_h equal to 0.22. In past research, the hoop stress, which arises from the outward pressure of the concrete, has been assigned values ranging from 0.2σ_y to 0.5 σ_y [KnowlesandPark, 1970;Ichinohe,etal.,1991;Zhang,etal.,1991]. Substituting Eqn. (e) into Eqn. (b), the lateral pressure exerted by the steel tube on the concrete core at the ultimate load may be shown as follows:

$$\sigma_{cl} = \frac{2t}{D_c}\gamma_h\sigma_y$$

The stress-strain constitutive formulation for steel in tension in circular CFST is identical to the described in above section.

Ultimate Axial Compressive Strength

The nominal axial load carried by circular CFST column can be adopted from Eqn.

$$N_0 = A_c(\sigma_{sv} + 0.5k\sigma_{sh})$$

Expressed as follow:

$$N_0 = A_s\gamma_v f_{yd} + A_c f_{cd}\left(1 + 2k\gamma_h\frac{t}{D}\frac{f_{yd}}{f_{cd}}\right)$$

Where γ_v is the vertical steel reduction ratio, and γ_h is the hoop steel reduction ratio, derived in the previous section:

$$\gamma_v = \frac{k+4}{\sqrt{3(k+1)^2+9}}$$

$$\sqrt{4 - 3\gamma_v^2} - \gamma_v$$

Where "K" is triaxial factor that has a value of 4.0 for normal strength. For high strength concrete, Cederwall [Cederwall,1988] recommended a lower value ofK should be used. In this research a K value of 4 is adopted as suggested by Setunge [Setunge, etal., 1992] for normal strength concrete.

Eurocode 4 also provides a formula to predict the axial load for CFST column including the effect of confinement:

$$N_0 = A_s\eta_2 f_{yd} + A_c\left(f_{cd} + f_{cd}\frac{t}{D}f_{yd}\right)$$

Where

For $0 \leq e \leq 0.1D$

$$\eta_1 1 = \eta_1 10\ (1 - \square 10 \square_1 e/D\ )$$

$$\eta_2 = \eta_{20} + (1 - \eta_{20})\frac{10_e}{D}$$

And

$$\eta_{10} = 4.9 - 18.5\bar{\lambda} + 17\bar{\lambda}^2 \geq 0$$

$$\eta_{20} = 0.25\left(3 + 2\bar{\lambda}\right) \leq 1.0$$

For short column, the relative slenderness, λ is close to zero, and above co-efficient becomes η_1=4.9 & η_1=0.75 comparing Eqn.( ) & Eqn.( ), an equivalent group of co-efficients is proposed,

$$\sqrt{4 - 3\gamma_v^2} - \gamma_v$$

$$\eta_{2o} = \gamma_v = \frac{k+4}{\sqrt{3(k+1)^2+9}}$$

By trail & error, for a K of 6.9, the proposed co-efficent becomes η_1=4.83 & η_2=0.78this also implies that an implicit K value of 6.9 is Used when the Eurocode 4 procedure is followed.

An empirical expression ok K Can be proposed by best-fitting the test data from three different sources[Gardner and Jacobson,1967; Kloppel and Goder,1957;Knowles and Park,1970]

For $\overline{\lambda} \leq 0.2$

$$k = 6.9 + 6.3\overline{\lambda}$$

For 0.2<$\overline{\lambda} \leq 0.5$

$$k = 34\overline{\lambda}^2 - 26\overline{\lambda} + 12$$

Figure 6 and 7 shown the proposed co-efficients η_1 and η_2 in term of λ when the proposed K value is used. They are compared with Eurocode4 recommendations.

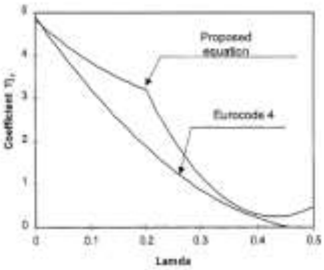

Figure.5: Comparison with Eurocode 4 and proposed method for η_1

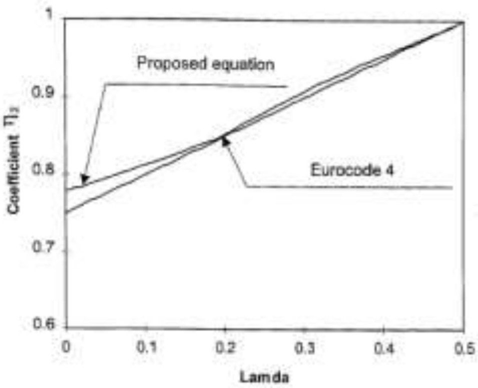

Figure.6: Comparison with Eurocode 4 and proposed method for η_2

Research on Concrete Filled Tubular Columns:

This describes a review of the behavior of concrete-filled steel tubular (CFST) members subject to axial load.

The discussion is divided in two parts.

In the first part the discussion is focused on the major theoretical & experimental researches performed throughout the world over the past several decades on CFST.

The second part summarizes the design rules for the analyses of steel concrete composite columns provided in different codes of practice are also discuses.

1. Gardner, N. J. and Jacobson, E. R., 1967

Introduction:

The authors investigated axially loaded CFT compression members both experimentally and theoretically. The results were compared to ACI (American Concrete Institute 318-63) and NBC (National Building Code of Canada, 1965), as well as tests performed by Klöppel and Goder (1957). They attempted to predict the ultimate load of short CFTs.

Experimental Study, Results, and Discussion:

At least two columns of each different tube size were tested and for each long column size a corresponding stub column was tested. Both the long columns and the stub columns first yielded in the longitudinal direction. Some interaction existed between the materials, as the CFT had a greater ultimate strength than the sum of the individual steel and concrete components.

Short Axially Loaded CFTs

A very detailed discussion of short column behavior was presented. The authors described the stress state of the CFT as load was applied. Initially, Poisson's ratio for steel (0.283) exceeded that of the concrete (0.15-0.25) and no confinement existed. As the load increased, the lateral strains in the concrete “caught up” to the strains in the steel, i.e., the Poisson's ratio of the concrete reached and then exceeded that of the steel. At this point, the tube began to restrain the concrete core and the hoop stress in the tube became tensile.

At failure, the steel augmented the concrete strength as expressed by the equation

$$\sigma_c = f_c^l + k.\sigma_r$$

Where **k** is an experimentally determined empirical factor with a value of about 4 and **σr** is the radial pressure on the concrete. The authors presented the following formula to represent the total load on the column:

$$N = A_c.f_c^l + \frac{K}{2}.A_S.\sigma_{sh} + A_S.\sigma_{sl}$$

The first term represents the concrete strength; the second, the strength contribution of the confined concrete due to the hoop stress in the tube; and the third term represents the steel strength under longitudinal compression. The bounds of this formula may be obtained by setting the hoop stress equal to yield (lower bound) or setting the longitudinal stress equal to yield (upper bound). In reality, though, the hoop stress will probably not reach half of the yield stress (Knowles, discussion). Assuming compatibility, the axial stress in the steel may be determined if the strain in the concrete at crushing is known.

2. Tomii, M. et al., 1973 (no tests)

Introduction:

This paper presented a very thorough review of the knowledge of CFT behavior to date. Topics of discussion included axially loaded short and long columns.

Tammi(1991) classified longitudinal strain relations for CFST columns in to three types. they are

1. Strain hardening type,
2. Elastic-plastic type,
3. Degrading type.
4. Strain hardening or Elasto-plastic type was observed in most circular columns and degrading types were generally observed in square columns.

***Theoretical Discussion*:**

***Short Axially Loaded Columns*:**

The shape of the CFT (i.e., circular or rectangular) will affect the axial behavior of the CFT. The relationship between axial load and longitudinal strain illustrates the marked decrease in capacity in the inelastic region for rectangular tubes. The decrease is not, however, evident for circular tubes due to the greater degree of confinement. Also, the thinner the tube and the higher the concrete strength, the steeper the descending branches of the axial load longitudinal strain curve. Most importantly, though, the CFT displays a greater ductility and strength than the sum of the individual ductilities and strengths of the constituent materials due to the interaction between the steel and the concrete.

The authors described short CFT column behavior as occurring in two phases.

The first phase is the elastic phase, in which the Poisson's ratio of concrete (0.16 to 0.25)is lower than that for steel (0.3). Provided the bond between the concrete and steel does not break down, the concrete's smaller lateral expansion will pull the steel inward, inducing compressive hoop stresses in the tube and tensile lateral stresses in the concrete. The behavior in this region is similar to the sum of the individual behaviors of the constituent materials and the authors suggested that modulus of elasticity of the CFT may be derived from uni-axial stress conditions.

In the second phase of loading, the longitudinal strain reaches a point at which the concrete undergoes volumetric expansion and its lateral expansion exceeds the lateral expansion of the steel. This occurs at a longitudinal strain of approximately 0.002. At this point in the loading, the concrete will be tri-axially confined in compression and tensile hoop stresses will exist in the steel tube. The authors suggested that both the concrete's strength and ductility will increase for both circular and rectangular CFTs due to the confinement.

The augmented concrete strength was calculated for circular tubes as follows:

$$f_{cc} = f_c^1 + 4.1\sigma_r$$

Where **σr** is the lateral pressure exerted on the concrete. This value is calculated by:

$$\sigma_r = \sigma_{sh} \cdot \left(\frac{t}{D/_{2} - t}\right)$$

where $\boldsymbol{\sigma_{sh}}$ is the hoop stress in the steel tube. Based on failure theory, for an existing hoop stress in the inelastic range, the longitudinal stress, ***σsl***, will be less than ***fy***. The reduction in the yield strength of the steel tube in the presence of a hoop stress is, however, offset by the increase in the strength of the concrete. The modified calculation of the ultimate axial strength accounting for these effects is:

$$N_0 = f_{cc}A_c + \sigma_{sl}A_s$$

Assuming no lateral interaction occurs, the ultimate axial load may be conservatively calculated by:

$$N_0 = f_c^1 A_c + f_y A_s$$

3. Bode, H., 1976 (no tests):

This paper presented a brief overview of analytical methods for calculating the ultimate capacity of short and long columns, and columns subjected to combined axial load and bending moment. He enumerated a number of advantages of filling hollow steel tubes with concrete.

Theoretical Discussion: Short Axially Loaded Columns:

Concrete in axially loaded short circular columns act independently until the concrete has reached a longitudinal strain of approximately 0.02. At this point, the concrete expands more than the steel tube. The steel is subjected to circumferential tensile stresses and the concrete is compressed triaxially resulting in a much larger strength than concrete loaded only in the axial direction. The increase in the capacity of the short column (the author defines a short circular column as a column with *L/D* ≤5) was given by the following formula originally published by Sen, (1972)

$$N_0 = 0.75A_s f_y + A_c\left[f_c^1 + \frac{3.8tf_y}{D - 2t}\right]$$

The carrying capacity of a short circular column is also increased if the concrete alone is loaded. To prevent the steel from buckling locally prior to the steel reaching the yield strength, the ACI Building Code 318-71 specifies a minimum ratio of the diameter to thickness of the steel tube:

$$\frac{D}{t} \leq \sqrt{\frac{8E_s}{f_y}}$$

4. Cai, S.-H., 1988 (c, bc, pb; m)

Introduction:

This article highlighted the results of seven phases of tests conducted and documented in earlier papers written in Chinese. Therefore, this paper gave only limited details regarding the nature of the experiments. However, a significant number of tests were performed, lending credence to the conclusions the author draws.

He also presented detailed descriptions of failure mechanisms for short and long CFT columns and derives formulas for the ultimate strength of these columns.

Experimental Study, Discussion, and Results:

The seven phases of tests conducted encompassed a wide range of loading techniques including the following: concentrically loaded short and long columns.

Short Column Behavior: The behavior of the short columns depended on the *D/t* ratio. For thin-walled tubes (*D/t* ≥ 19), the load-strain curve was linear until the first yield lines were observed in the steel. The ultimate load was reached when the slope of the load-strain curve became zero, at which point the entire steel cross-section had yielded and the concrete reached its ultimate compressive strength. For very short, thick walled tubes (*D/t* = 10), the steel confined the concrete as described in previous work by many authors. As the concrete expanded outward against the tube wall, hoop stresses were induced in the steel, effecting a decrease in the amount of longitudinal capacity of the tube and a transfer of axial load to the concrete. The confinement of the concrete allowed the section to realize a net gain in strength. Both ranges of *D/t* underwent failure when the resultant compressive force carried by the two materials reached ultimate.

Theoretical Discussion:

The author derived a formula for predicting the ultimate strength of short columns. The basic assumptions were that at the limit stage, the concrete was in a state of triaxial compression and the steel was in a biaxial state of stress, the steel conformed to the Von Mises yield criterion, and the concrete failure criterion was an empirical formula developed by the author to fit experimental data. After some algebra, the ultimate load may be expressed by the following formula:

$$N_0 = A_c f_c^1 \left(1 + \sqrt{\phi} + 1.1\phi\right)$$

Where the confinement ratio

$$\phi = \frac{A_s \cdot f_y}{A_c \cdot f_c^1}$$

5.Kitada, T. et al., 1987 (c; m)

Introduction:

The elasto-plastic behavior of short circular CFT columns was studied experimentally and compared to the values obtained from analytical methods proposed by other authors. The effect of the method of loading (i.e., load both materials, load the steel tube only, or load the concrete core only) was examined in detail as well as the effect of bond on the ductility and strength of the section. The study focused on a detailed examination of the triaxial stresses in the concrete and the biaxial stresses in the steel for the different loading methods and for the bonded and unbonded specimens.

Experimental Study, Discussion, and Results:

The test specimens were very short columns with variable parameters including the steel yield strength, the concrete compression strength, and the relative ratio of the two strengths. The specimens were loaded by three methods: 1) load both the concrete and the steel simultaneously such that the longitudinal strains in both materials are equal, 2) load the concrete core alone without applying axial compression on the tube, and 3) load the steel tube alone. The stresses in each specimen were analyzed using the Mises yield criterion which is defined by the following equation:

$$\left(\frac{\sigma_{sh}}{f_y}\right)^2 - \left(\frac{\sigma_{sh}}{f_y}\right).\left(\frac{\sigma_{sc}}{f_y}\right) + \left(\frac{\sigma_{sc}}{f_y}\right)^2 = 1.0$$

where σsh and σsc are the longitudinal and hoop stress, respectively, in the steel tube. For the different loading conditions, the authors plotted the stress path of the steel, i.e., the hoop stress versus the longitudinal stress, at different locations on the tube. Because the Poisson's ratio of the concrete is less than the corresponding ratio for steel in the elastic range, the concrete and steel do not interact and act independently of one another. At the crushing strain of the concrete, the lateral expansion of the concrete exceeds that of the steel and hoop stresses are induced in the steel tube, and corresponding triaxial stresses are induced in the concrete core. A failure curve proposed by Cai that was restated in this paper accurately described the relationship between the confining stress on the concrete and the axial compressive stress. This empirical equation is given by:

$$\sigma_c = f_c^l\left(1 + 1.5\sqrt{\frac{\sigma_r}{f_c^l}} + 2\frac{\sigma_r}{f_c^l}\right)$$

Where σc is the ultimate axial compressive stress of encased concrete subjected to a uniform, radial compressive stress, σr. The failure of the section under this loading condition occurred at the ends of the specimen. Soon after yielding, the steel buckled locally and the concrete in the region of local buckling crushed.

The load-deformation behavior of the tubes revealed that the axial capacity as computed by summing the individual strengths of the component materials largely underestimated the actual capacity of the section. The relationship between load and deformation is linear until the steel yields, at which point the concrete begins to undergo confinement and the stiffness of the member decreases substantially. For axial displacement beyond the yield point, the capacity of the section increases gradually, exhibiting good ductility. The method of loading did not affect the ultimate capacity of the section, except in the case of loading the steel alone in which case the concrete did not contribute any strength. Loading both materials simultaneously produced a slightly larger stiffness. The authors suggested that the calculation for the ultimate load of a short column proposed by Cai matched their results with sufficient accuracy. The equation is as follows:

$$P_u = A_c f'_c\left(1 + 2\frac{f_y A_s}{f'_c A_c}\right)$$

Review of Design Codes:

Eurocode 4(1994)

In this section the design recommendation in Euro-code 4 on concrete filled steel tubes are discussed in detail.

Limitations:

The Eurocode 4places the limitations of concrete strengths which should not exceed 50Mpa & the diameter to wall thickness should be limited by

D/t<=52ε for rectangular sections

D/t<=90ε^2 for circular sections

Where D is the greater overall dimension of the section parallel to a principal axis, t is the thickness of the wall of a concrete-filled hallow section, where $\varepsilon = \sqrt{\frac{235}{fy}}$ & fy is the yield strength of the steel.

Axial compression:

The Eurocode 4 code provides a formula to predict the axial load for CTST columns including the effect of confinement:

$$N_0 = A_s\eta_2\frac{f_y}{\gamma_c} + A_c\frac{f_c^l}{\gamma_c}\left(1 + \eta_1\frac{t}{D}\frac{f_y}{f_c^l}\right)$$

Where γ_c & γ_s are the partial safety factors & the value of **η1** & **η2**are given as **0<=e<=0.1D**

$$\eta_1 1 = \eta_1 10\ (1 - \square 10 \square_1 e/D\ )$$

$$\eta_2 = \eta_{20} + (1 - \eta_{20})\frac{10_e}{D}$$

And the value of η_{10} & η_{20} are given as

$$\eta_{10} = 4.9 - 18.5\bar{\lambda} + 17\bar{\lambda}^2 \geq 0$$

$$\eta_{20} = 0.25\left(3 + 2\bar{\lambda}\right) \leq 1.0$$

The relative slenderness of $\bar{\lambda}$ is given as

$$\bar{\lambda} = \sqrt{\frac{N_0}{N_{cr}}}$$

ACI 318

The ACI rules for composited columns follow the same procedures as for reinforced concrete columns. The ACI code provides the method to evaluate the strength of the cross section & the effects of slenderness & it requires a minimum eccentricity of axial load for column design.

Limitations:

In the ACI design method the yield stress of structural steel used in calculation the strength of composite columns should not excess 414 MPa, & it does not permit column design given for high strength conctete. The secant modulus of elasticity of concrete is calculated from the formula:

$$E_c = 0.043\rho_c^{1.5}\sqrt{f_c^l}$$

Where is the density of concrete in kg/m^3 & E_c & f_c' are expressed in Mpa,

This formula is not adjusted for high strength concrete, and overestimates the modulus of elasticity for concrete above 40MPa.

Axial compression:

The axial resistance N_n is limited to 85% of the theoretical squash load N_0 for concrete-filled steel tubes in ACI-318 is given as

$N_n = 0.8\varphi N_0$

Where φ is the capacity reduction factor,a value of 0.75 for concrete-filled steel tube & the value of N_0 is given

$N_0 = A_S f_s + 0.85 A_c f_c'$

Where A_c=area of concrete in cross section

A_s=area of structural steel in cross section
f_c'= strength of concrete from standard cylinder tests
f_y=yield strength of structural steel.

Work under progress by the Authors:

Experimental work is also being carried out by the authors as a part of Research study at Ghousia College of Engineering, R & D center, regarding Experimental study on the behavior of CFSST columns for ultimate load carrying capacity of axially short columns. Taguchi method has been adopted for economy. Two theoretical Equations were developed by comparing Euro code04, ASC318 and AS 3600 for the prediction of the ultimate axial load strength of concrete filled stainless steel short columns .The results from prediction were compared with the experimental data. Validation to the experimental results was made. The detailed experimental data and material properties employed in this study are based on previous experimental data by the researchers (10-15). About 45 samples data was taken for formulating a mathematical model to correlate the obtained experimental values and Neural Network model is being generate. This research is under progress and it is observed that there is good correlation

Conclusions:

Considerable progress has been made during the last two decades in the investigation of steel–concrete composite columns and use of self-compacting concrete, and information available is summarized in this paper. Fundamental knowledge on composite construction system such as ultimate strength has already been obtained by the research carried out so far.

Based on work done by following conclusions are listed below,

1. Circular hallow section have many advantage as structural members due to the fact that the properties are same for all direction,
2. Circular CFST how a greater increases in strength and a greater enhancement of ductility than rectangular CFST,
3. It is noted that the short column shows a linear behavior up to yield load and after showed a non-linear behavior, A sudden drop in the load carrying capacity is found with Large deformation,
4. Concrete filled steel tubular columnshave relatively high stiffness with plain cement concrete ,

References

[1] Hajjar JF. Composite steel and concrete structural systems for seismic engineering. J Construct Steel Res 2002;59(58):703–23.

[2] O'Shea M, Bridge R. Circular thin-walled tubes with high strength concrete infill. Composite construction in steel and concrete II. Irsee(Germany): ASCE; 1996, p. 780–93.

[3] O'Shea M, Bridge R. The Design for local buckling of concrete filled **steel** tubes. In: Composite Construction—Conventional and Innovate, Innsbruck, Austria; 1997, p. 319–24.

[4] Eurocode 4 DD ENV 1994-1-1, Design of composite steel and concrete structures. Part 1.1, General Rules and Rules for Buildings (with UK National Application Document). London: British Standards Institution; 1994.

[5] Commentary to the Third Edition of the Design Manual for Structural Stainless Steel, downloadable from www.steelstainless.org/designmanual.

[6] Gardner L. The use of stainless steel in structures. Progress in Structural Engineering and Materials 2005;7(2):45–55.

[7] ACI 318. Building code requirements for structural concrete. Detroit: American Concrete Institute; 1995.

[8] EN 1994-1-1, Eurocode 4: Design of composite steel and concrete structures, Part 1-1: General rules and rules for buildings. CEN, 2004.

[9] Sakino K, Tomii M, Watanabe K. Sustaining load capacity of plain concrete stub columns by circular steel tubes. In: Conference on concrete filled steel tubular construction; 1998, p. 112–18.

[10] Gardner L. The use of stainless steel instructures. Progress in Structural Engineering and Materials 2005;7(2):45–55.

[11]GardnerL.Thecontinuousstrengthmethod.ProceedingsoftheInstitutionofCivilEngineers-StructuresandBuildings [submitted for publication].

[12] Packer JA, Henderson JE. Hollow structural section connections and trusses—Adesign guide. Toronto, Canada: Canadian Institute of Steel Construction (CISC); 2003.

[13] Gardner L, Cruise RB, Sok CP, Krishnan K, Ministro dos Santos J. Life cyclecosting of metallic structures. Proceedings of the Institution of Civil Engineers— Engineering Sustainability 2007;60(4):166–77.