REVISTA MATÉRIA V.27 N.02
ISSN 1517-7076 articles e13188, 2022

Utilization of industrial waste materials in concrete-filled steel tubular columns

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ABSTRACT

The idea of this particular study is to utilize industrial waste materials in an effective manner so as to give strength to steel tubular columns. Composite construction using steel - concrete has emerged as one of the fastest methods of construction in India. The inherent advantage of the steel-concrete composite section lies in that the two principal elements - the steel and the concrete are normally used in a manner so that full potential of both may be realized and the best utilization of their respective properties can be made. In this paper, an attempt was made with steel tubular columns in-filled with plain concrete, partial replacement of fine aggregate by fly ash & quarry dust and coarse aggregate by rubber, slag from the steel industry, granite and construction & demolition (C&D) debris concrete. The column specimens are to be tested under axial compression to investigate the effects of industrial waste materials. The effects of the steel tube and the strength of concrete are examined. 24 specimens were tested with the strength of concrete as 20 MPa and D/t ratio 25.40. The columns were 114.3 mm in diameter and 4.5 mm in thickness are 300, 600 & 900 mm in length. Strength characteristics and failure modes are to be discussed. The test results are to be compared with the values predicted by Eurocode 4, Australian Standards and American Codes and new theoretical models will be suggested for the design. From the test results, it was observed that the load-carrying capacity of steel tubular columns in-filled with various industrial waste materials concrete is greater than the conventional concrete. Hence this research would give a solution for effective utilization of industrial waste materials such as rubber, granite, C&D debris, steel slag, fly ash and quarry dust in concrete.

Keywords: Rubber, Steel Slag, Fly Ash, Quarry Dust, Steel Tubular Sections.

1. INTRODUCTION

Industrial waste materials are a common problem in modern living. Waste minimization is increasingly seen as an ecologically sustainable strategy for alleviating the need to dispose of waste materials, which is often costly, time and space consuming, and can also have significant detrimental impacts on the natural environment. Within India, the government is concerned with developing policies and programmes to bring about successful outcomes to waste minimization. This is seen as being essential to reduce the total amount of waste materials going into landfill, especially in the urban areas where land is very scarce. Recycled materials usually produce cheaper end products for the consumers, hence further justifying their use from an economical viewpoint. Waste materials aggregate concrete can utilize demolition material from concrete and masonry constructions. Though several studies have been made on the reuse of concrete waste, only limited studies have been made with respect to the use of demolition brick masonry as aggregate. Most of the waste materials produced by demolishing structures are disposed of by dumping them as landfill or for reclaiming land. But with the demand for land increasing day by day, the locations, capacity and width of the land that can receive waste materials are becoming limited. Add to it the cost of transportation, which makes disposal a major problem. Hence, reuse of demolition waste appears to be an effective solution and the most appropriate and large-scale use would be to use it as aggregates to produce concrete for new construction.

The technique of composite construction has assumed great importance due to some of its inherent advantages in comparison to the cast-in-situ construction concrete. A major application of this technique in housing is the construction of composite beams, columns and slabs made of cement concrete with rolled steel sections. But in India, many constructions have not used these types of steel tubular composite sections due

Corresponding Author: Erode Krishnasamy Mohanraj
Received on: 10/04/2020
Accepted on: 17/02/2022

10.1590/S1517-707620220002.1388

to lack of awareness. Steel members have the advantages of high tensile strength and ductility, while concrete members have the advantages of high compressive strength and stiffness. Composite members combine steel and concrete, resulting in a member that has the beneficial qualities of both materials. The two main types of the composite column are the steel-reinforcement concrete column (Figure 1), which consists of a steel section encased in reinforced or un-reinforced concrete, and the concrete-filled steel tubular (CFST) columns (Figure 2), which consists of a steel tube filled with concrete. CFST columns have many advantages over steel-reinforcement concrete columns. The major benefits of concrete-filled steel tubular columns are:

- Steel column acts as permanent and integral formwork
- The steel column provides external reinforcement, and
- The steel column supports several levels of construction before concrete being pumped.

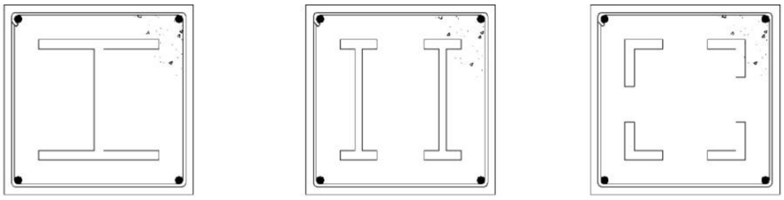


Figure 1: Concrete Encased Composite Columns

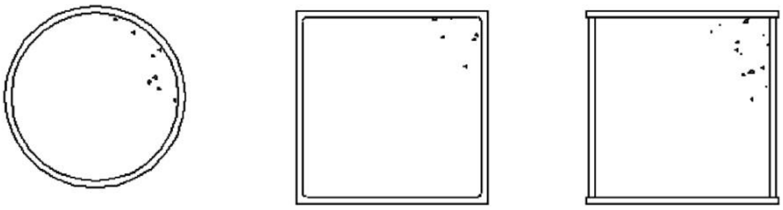


Figure 2: Concrete Filled Composite Columns

Although CFST columns are suitable for all tall buildings in high seismic regions, their use has been limited due to a lack of information about the true strength and the inelastic behaviour of CFST members. Due to the traditional separation between structural steel and reinforced concrete design, the procedure for the designing CFST column using the American Concrete Institute's (ACI) code is quite different from the Load and Resistance Factor Design (LRFD) method suggested by the American Institute of Steel Construction's (AISC).

1.1 Concrete Filled Steel Tubular Sections

Circular tubular columns have an advantage over sections when used in compression members, for a given cross-sectional area, they have a large uniform flexural stiffness in all directions. Filling the tube with concrete will increase the ultimate strength of the member without a significant increase in cost. The main effect of concrete is that it delays the local buckling of the tube wall and the concrete itself, in the restrained state, can sustain higher stresses and strains that when is unrestrained.

The uses of CFSTs provide a large saving in cost by increasing the floor area by a reduction in the required cross-section size. This is very important in the design of tall buildings in cities where the cost of letting spaces are extremely high. These are particularly significant in the lower storey of tall buildings where short columns usually exist. CFSTs can provide excellent monotonic and seismic resistance in two orthogonal directions. Using multiple bays of composite CFST framing in each primary direction of a low-to-medium rise building provides seismic redundancy while taking full advantages of the two-way framing capabilities of CFSTs [1]. Experimental research on CFST columns has been ongoing worldwide for many decades, with significant contribution having been made particularly by researchers in Australia, Europe and Asia. The vast majority of these experiments have been on moderate scale specimens (less than 200 mm in diameter) using normal and high-strength concrete and there is no much research on the study of CFSTs infilled with concrete using waste materials.

1.2 Review of Literature

Mandal et al [2] reported in his paper elsewhere have shown that the strength of recycled aggregate concrete is comparatively lower than that of a similar mix of conventional concrete. However, with the use of fly-ash, it may be possible to produce recycled aggregate concrete with an improvement in strength. The results of this investigation also show that drying shrinkage strain, permeability and water absorption of the recycled aggregate concrete is more compared to conventional concrete. However, the quality of recycled aggregate concrete is found to be improved considerably with the addition of fly-ash. This, in turn, improves the durability of recycled aggregate against sulphate and acid attack. Therefore, the result of that study provides strong support for the feasibility of using recycled aggregates instead of natural aggregates for the production of concrete. He suggested that more research studies on recycled aggregate concrete are necessary for the practical application of recycled aggregate concrete.

Ramamurthy and Gumaste [3] studied the properties of recycled aggregate concrete and he reported that recycled aggregates possess relatively lower bulk density and higher water absorption as compared to those of fresh granite aggregates. The compressive strength of recycled aggregate concrete is relatively lower and the variation depends on the strength of original (demolished) concrete from which the aggregates have been obtained. This reduction is mainly caused by the bond characteristics of recycled aggregate and the fresh mortar of the recycled concrete.

Sahu et al [4] concluded in his paper that if 40 % sand is replaced by quarry dust in concrete, it will not only reduce the cost of concrete but at the same time will save a large quantity of natural sand and will also reduce the pollution created due to the disposal of this stone dust on valuable fertile land. There has been inadequate utilization of large quantities of crushed stone as alternative material left out after crushing of rock to obtain coarse aggregate/ballast for concrete. Crushed stone dust does not satisfy the standard specification of fine aggregate in cement mortar and concrete. Efforts have been made to replace river sand by rock dust. Nagaraj et.al [5] has studied the effect of rock dust and pebble as aggregate in cement and concrete. They found that crushed stone dust could be used to replace the natural sand in concrete. Shukla et al [6] investigated the behaviour of concrete made by partial/full replacement of river sand by crushed sand dust as fine aggregate and reported that 40 % can be replaced by crushed stone dust without affecting the strength of concrete.

A series of tests had been carried out by O’Shea and Bridge [7] on the behaviour of circular thin-walled steel tubes. The tubes had diameter to thickness (D/t) ratio ranging between 55 and 200. The tests included; bare steel tubes, tubes with un-bonded concrete with only the steel section loaded, tubes with concrete infilled with the steel and concrete loaded simultaneously and tubes with the concrete infill loaded alone. The test strengths were compared to strength models in design standards and specification. The results from the tests showed that the concrete infill for the thin-walled circular steel tubes has little effects on the local buckling strength of the steel tubes.

However, O’Shea and Bridge [8] found that concrete infill can improve the local buckling strength for rectangular and square sections. Increased strength due to confinement of high-strength concrete can be obtained if only the concrete is loaded and the steel is not bonded to the concrete. For steel tubes with a D/t ratio greater than 55 and filled with 110-120 MPa high-strength concrete, the steel tubes provide insignificant confinement to the concrete when both the steel and concrete are loaded simultaneously. Therefore, they considered that the strength of these sections can be estimated using Eurocode4 (EC4) with confinement ignored. The influence of local buckling on the behaviour of short circular thin-walled CFSTs has been examined by O'Shea and Bridge. Two possible failure modes of the steel tube had been identified, local buckling and yield failure. These were found to be independent of the diameter to wall thickness ratio. Instead, the bond between the steel and concrete infill determined the failure mode. A design method has been suggested based on the recommendations in EC4 [9].

Kilpatrick et al [10, 11] examined the applicability of the Eurocode 4 for the design of CFSTs which use high-strength concrete and compare 146 columns from six different investigations with EC4. The concrete strength of columns ranged from 23 to 103 MPa. The mean ratio of measured/predicted column strength was 1.10 with a standard deviation of 0.13. The EC4 safely predicted the failure load in 73% of the column analyzed. For axially loaded thin-walled steel tubes, local buckling of the steel tube does not occur if there is sufficient bond between the steel and concrete. For concrete strength up to 80 MPa, EC4 can be used with no reduction for local buckling. For concrete strength in excess 80 MPa, EC4 can still be used but with no enhancement of the internal concrete confinement and no reduction in the steel strength from local buckling and biaxial effects from confinement. Thin-walled circular axial compression and moment can be designed using the EC4 with no reduction for local buckling.

Lin-Hai Han. et.al. [27] presents a life-cycle based finite element analysis modelling for concrete-filled steel tubular (CFST) stub columns. The effects of initial imperfections, preloads during the construction process, long-term sustained loads and corrosion are considered in the modelling. To reveal the life-cycle time-dependent behaviour of CFST stub columns, influences of the above factors on the full-range load-displacement relations and the load-transfer mechanism are discussed in regard of five typical working conditions. Considering the effects of these factors, definition of the ultimate state of CFST stub columns under typical working conditions are discussed in this paper. Parametric studies are carried out to calibrate the simplified design models for CFST stub columns at different loading cases. A database with information of 1096 CFST stub column specimens are established. Reliability analysis proves that the proposed simplified methods meet the reliability requirements in accordance with current design standards for CFST members under the five discussed cases. Partial factor for axial compressive strength of the composite section is suggested at the end of this paper.

Lin-Hai Han [28] developed a mechanics model that can predict the behaviour of concrete-filled hollow structural section (HSS) beams. A form of unified theory, where a confinement factor (ξ) was introduced to describe the composite action between the steel tube and filled concrete, is used in the analysis. A series of concrete-filled square and rectangular tube beam tests were carried out. The main parameters varied in the tests were the depth-to-width ratio (β) from 1 to 2, and tube depth to wall thickness ratio from 20 to 50. The load vs. lateral deflection relationship was established for concrete-filled HSS beams both experimentally and theoretically. The predicted curves of load vs. mid-span deflection are in good agreement with the presented test results. Formulas which should be suitable for incorporation into building codes are developed for calculating the moment capacity of concrete-filled HSS beams. Comparisons are made with predicted beam capacities and flexural stiffness using the existing codes, such as AIJ-1997, BS5400-1979, EC4-1994, and LRFD-AISC-1999.

Jianzhuang Xiao. et.al [29] presents the results of axial compression tests on recycled aggregate concrete (RAC) confined by steel tubes and RAC confined by glass fiber reinforced plastic (GFRP) tubes. The objective of this study is to evaluate the mechanical properties of confined RAC under axial compressive loading. The main parameters in the tests are the recycled coarse aggregate replacement percentage and the tube material. Research findings indicate that both the strength and deformation of RAC are obviously improved. The mechanical properties of RAC confined by steel tubes are better than those of RAC confined by GFRP tubes when all parameters are kept the same. It is also found that the peak stress of RAC confined by steel tubes decreases while the corresponding strain increases when the replacement percentage of RCA is increased, which are similar to those of RAC confined by GFRP tubes. Expressions for the peak stress and the stress–strain relationship of the confined RAC are then obtained based on the test data. Finally, the finite element method (FEM) is applied to study the effect of the outer tube thickness and the core RAC strength on the stress–strain relationship of confined RAC.

2. MATERIALS AND METHODS

2.1 Materials

A total of twenty-four specimens of Circular (designated C) sections were tested for this study. All specimens were tested with the strength of concrete as 20 MPa and a D/t ratio 25.40. The columns were 114.3 mm in diameter and 300, 600 & 900 mm in length. The column specimens were classified into eight different groups filled with plain concrete (designated PC), partial replacement of fine aggregate by 10% fly-ash (designated FA) and 40% quarry dust (designated QD) and coarse aggregates by 25% rubber (designated RC), 25% steel slag (designated SS), 25% granite (designated GR) and 25% C&D debris (designated CD).

The rest of the column specimens were tested as hollow sections for comparison (designated HC).

2.2 Specimen Properties

All the specimen properties are given in Table 1. All the specimens were fabricated from circular hollow steel tube and filled with seven types of concrete. The average values of yield strength and ultimate tensile strength for the steel tube were found to be 260 and 320 MPa respectively. In the present experimental work, the parameters of the test specimens are the size of the specimen, the strength of concrete and L/D ratio of columns. To prevent the steel hollow column section from local buckling, ACI [12] required the width-to-thickness (B/t) ratio of the steel hollow section not greater than the following limit:

$$for\ 114.3\ mm\ dia, the\ B/t\ is\ 25.40\ <\ \sqrt{(3E_S/f_y)}\ =\ 48.63 \quad (1)$$

Table 1: Specimen properties

Reference Columns	Outer Dia D (mm)	Thickness t (mm)	D/t	Length L (mm)	L/D	Steel Strength f_y (MPa)	Concrete Cube Strength f_{cu} (MPa)
C1-HC	114.3	4.5	25.40	300	2.62	260	NA
C2-PC	114.3	4.5	25.40	300	2.62	260	25.03
C3-FA	114.3	4.5	25.40	300	2.62	260	22.75
C4-QD	114.3	4.5	25.40	300	2.62	260	24.47
C5-GR	114.3	4.5	25.40	300	2.62	260	23.26
C6-CD	114.3	4.5	25.40	300	2.62	260	28.14
C7-RC	114.3	4.5	25.40	300	2.62	260	20.15
C8-SS	114.3	4.5	25.40	300	2.62	260	21.48
C9-HC	114.3	4.5	25.40	600	5.25	260	NA
C10-PC	114.3	4.5	25.40	600	5.25	260	25.03
C11-FA	114.3	4.5	25.40	600	5.25	260	22.75
C12-QD	114.3	4.5	25.40	600	5.25	260	24.47
C13-GR	114.3	4.5	25.40	600	5.25	260	23.26
C14-CD	114.3	4.5	25.40	600	5.25	260	28.14
C15-RC	114.3	4.5	25.40	600	5.25	260	20.15
C16-SS	114.3	4.5	25.40	600	5.25	260	21.48
C17-HC	114.3	4.5	25.40	900	7.87	260	NA
C18-PC	114.3	4.5	25.40	900	7.87	260	25.03
C19-FA	114.3	4.5	25.40	900	7.87	260	22.75
C20-QD	114.3	4.5	25.40	900	7.87	260	24.47
C21-GR	114.3	4.5	25.40	900	7.87	260	23.26
C22-CD	114.3	4.5	25.40	900	7.87	260	28.14
C23-RC	114.3	4.5	25.40	900	7.87	260	20.15
C24-SS	114.3	4.5	25.40	900	7.87	260	21.48

2.3 Concrete Properties

The concrete mix was obtained using the following dosages: 3.75 kN/m^3 of Portland cement, 5.23 kN/m^3 of sand, 11.62 kN/m^3 of coarse aggregate with maximum size 12 mm, and 0.192 m^3 of water. Fly-ash (waste from the thermal plant - Mettur Thermal Plant, TN), quarry dust (waste from crusher), slag (waste from steel industries - Agni Steels, TN) granite (waste pieces from granite industries) and C&D debris (construction and demolition debris) by weight basis and rubber (waste pieces from the tire) by volume basis are taken. To characterize the mechanical behaviour of concrete, three cubes, three prismatic and three cylindrical specimens were prepared from each concrete and tested. The mean values of the strength related properties of concrete at the age of 28 days are summarized in Table 2. During the preparation of the test specimens, concrete was cast in layers and light tamping of the steel tube using a wooden hammer was performed for better compaction.

Table 2: Concrete properties

Type of Concrete	f_{ck} (MPa)*	f_{cr} (MPa)*	f_{ct} (MPa)*
Conventional concrete	25.03	3.06	2.26
Partial replacement of fine aggregate by fly-ash 10 %	22.75	3.19	2.44
Partial replacement of fine aggregate by quarry dust 40 %	24.47	3.17	2.53
Partial replacement of coarse aggregate by granite 25 %	23.76	3.07	2.35
Partial replacement of coarse aggregate by C&D debris 25%	28.14	3.50	2.82
Partial replacement of coarse aggregate by rubber 25%	20.15	2.81	1.31
Partial replacement of coarse aggregate by steel slag 25%	21.48	3.37	2.31

* average of three cubes, prisms and cylinders respectively

2.4 Test Methods

All the tests were carried out in an Electronic Universal Testing Machine of a capacity 1000 kN. The columns were hinged at both ends and an axial compressive load applied as shown in Figure 3. A pre-load of about 5 kN was applied to hold the specimen upright. Dial gauges were used to measure the lateral and longitudinal deformations of the columns. The load was applied in small increments of 20 kN. At each load increment, the deflection at the centre was recorded. All specimens were loaded up to failure.

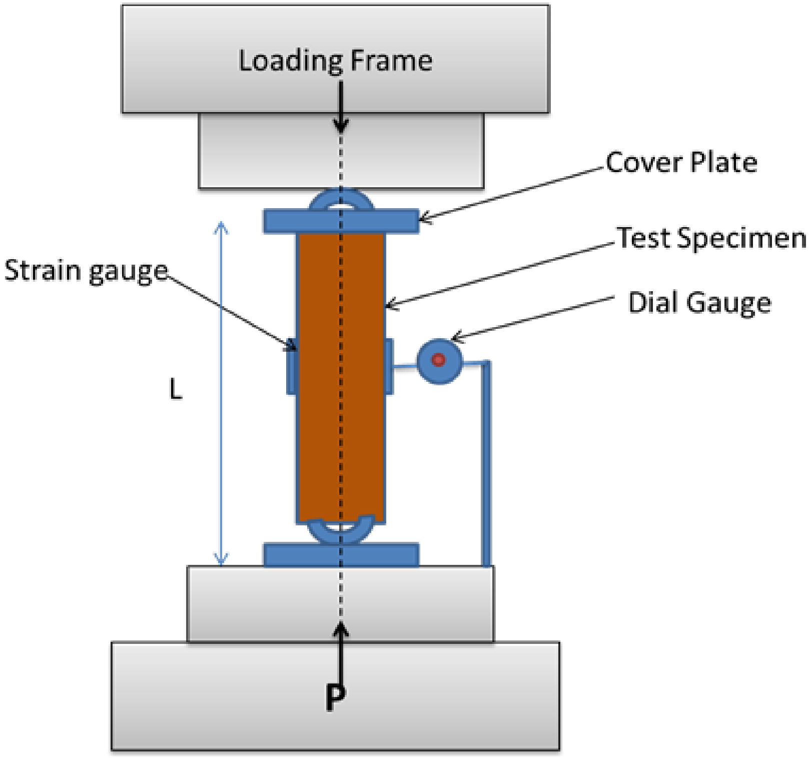


Figure 3: Test set up of concrete-filled steel tubular column

3. TEST RESULTS AND DISCUSSIONS

3.1 Comparison with Eurocode 4 part 1-1 (EC4), ACI 318-95 (ACI) and Australian Standards AS 3600 & AS 4100 (AS)

EC4 part1-1 [9] is the most recently completed international standard in composite construction. EC4 part 1-1 covers concrete-encased and partially encased steel sections and concrete-filled sections with or without reinforcement. EC4 part 1-1 uses limit state concepts to achieve the aims of serviceability and safety by applying a partial safety factor to load and material properties. EC4 part 1-1 is the only code that treats the effects of long-term loading separately. The ultimate axial force of a column is

$$N_{EC4} = A_S f_y \eta 2 + A_c f_{cc} \left(1 + \eta 1 \left(t f_y / D f_{cy}\right)\right) \tag{2}$$

The ACI [12] and Australian Standards [13] use the same formula for calculating the squash load. Neither code takes into consideration the concrete confinement. The limiting thickness of steel tube to prevent local buckling is based on achieving yield stress in a hollow steel tube under monotonic axial loading which is not a requirement for an in-filled composite column. The squash load is determined by

$$N_{ACI/AS} = 0.85\, A_c\, f_{cc} + A_s\, f_y \tag{3}$$

Detailed comparisons of the load-carrying capacity of composite columns are presented in Table 3. For the first set of specimens having a small L/D ratio (2.62) is the increase in the value of N_{test} ranges from 3 to 15 %. Whereas in the case of the second set of specimens with L/D ratio of 5.25 the N_{test} values increase ranges from 9 to 21 % and the third set of specimens with large L/D ratio (7.87) the N_{test} values increases ranges from 13 to 26 %. Hence the strength of infill concrete and L/D ratio influences the critical load-carrying capacity.

Table 3: Comparison of load-carrying capacity with existing code

Reference Columns	N_{test} (kN)	N_{Ec4} (kN)	$N_{ACI/AS}$ (kN)	N_{test}/N_{Ec4}	$N_{test}/N_{ACI/AS}$	Modified $N_{ACI/AS}$ (kN)	N_{test} / Modified $N_{ACI/AS}$
C1-HC	462.85	NA	NA	NA	NA	NA	NA
C2-PC	885.95	787.51	551.80	1.125	1.605	848.14	1.045
C3-FA	815.45	772.76	538.29	1.055	1.515	827.38	0.986
C4-QD	900.45	783.88	548.48	1.148	1.642	843.03	1.068
C5-GR	916.55	776.06	514.31	1.181	1.782	832.03	1.102
C6-CD	925.65	807.64	570.21	1.146	1.653	876.44	1.056
C7-RC	798.65	755.95	522.90	1.056	1.527	803.72	0.994
C8-SS	900.85	795.45	522.58	1.128	1.723	812.22	1.109
C9-HC	460.50	NA	NA	NA	NA	NA	NA
C10-PC	810.55	689.79	551.80	1.175	1.468	809.51	1.001
C11-FA	795.65	675.60	538.29	1.177	1.478	789.70	1.008
C12-QD	860.45	686.30	548.48	1.253	1.568	804.64	1.069
C13-GR	878.65	678.77	514.31	1.294	1.708	794.13	1.106
C14-CD	885.95	709.18	570.21	1.249	1.554	836.53	1.059
C15-RC	728.95	659.45	522.90	1.105	1.394	767.11	0.950
C16-SS	880.50	701.50	522.58	1.255	1.685	796.47	1.105
C17-HC	403.20	NA	NA	NA	NA	NA	NA
C18-PC	788.80	620.69	551.80	1.271	1.429	728.94	1.082
C19-FA	765.85	606.49	538.29	1.262	1.423	711.11	1.077
C20-QD	800.55	617.20	548.48	1.297	1.460	724.56	1.105
C21-GR	827.50	609.66	514.31	1.357	1.609	715.09	1.157
C22-CD	855.65	640.13	570.21	1.337	1.501	753.27	1.136
C23-RC	678.50	590.34	522.90	1.149	1.298	690.77	0.982
C24-SS	794.58	600.40	522.58	1.323	1.520	701.51	1.133

The N_{EC4}, and N_{test} loads of various infill concrete materials is presented in Figure 4. It is observed that the EC4 equation gives a underestimates the load-carrying capacity of concrete-filled composite columns varying from 5 to 35 % as compared to experimental results. But a comparison with ACI/AS codal equation shows that the equation underestimates the critical load carrying capacity of columns varying up to 30 to 78%. This observation was also made by Georgios Giakoumelis and Dennis Lam [14] hence they proposed a modified equation as

$$N_{ACI/AS} = 1.3\, A_c\, f_{cc} + A_s\, f_y \tag{4}$$

Figure 5 shows the comparison of test results with ACI/AS and the modified ACI/AS equation.

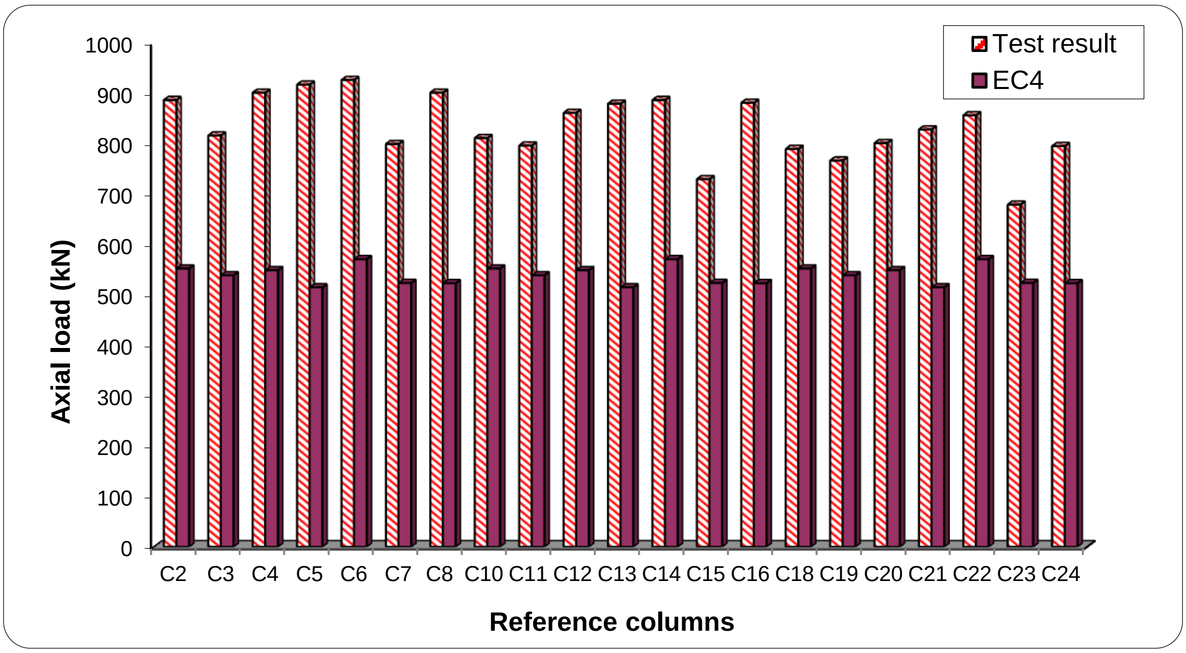


Figure 4: Comparison of test result with EC4

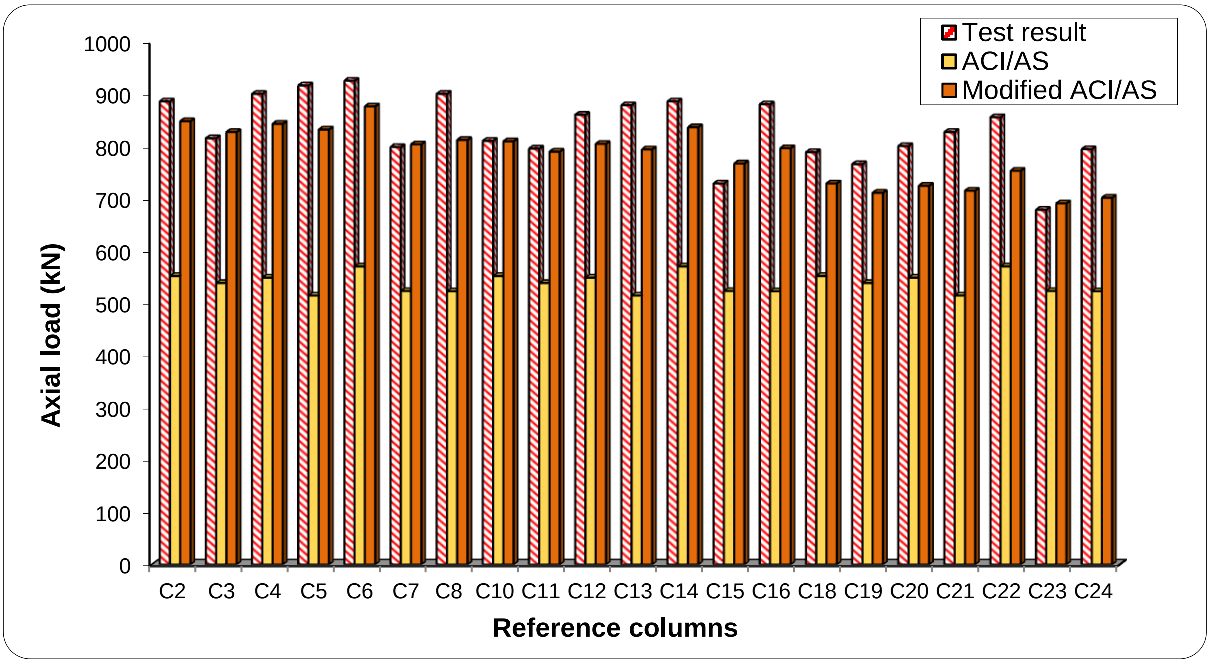


Figure 5: Comparison of test result with ACI/AS and modified ACI/AS

3.2 Results and Discussions

The tests were conducted on 24 specimens with different L/D ratio of 2.62≈2.5, 5.25≈5.0 & 7.87≈8.0 and also with infilling of plain concrete and partial replacement of fine aggregate by fly-ash & quarry dust and coarse aggregate by granite, rubber, slag and C&D debris. The test results were given in Figure 4 and Figure 5.

Figure 4 and Figure 5 compares the relationship between the compressive strength of concrete to the strength of the column predicted by EC4 part 1-1, ACI/AS, modified ACI/AS [14] and experimental test results. From the Figure 4 and Table 3 it was observed that EC4 and ACI/AS underestimate the strength of column but modified ACI/AS is well correlating with experimental results (L/D=5.25) and hence equation (4) is applicable for steel tubular section in-filled with concrete. Figure 6 shows failure shapes of the tested circular columns. From the figure 6, it can be observed that when slenderness value is low i.e L/D < 4, columns are mainly fails due to crushing. For intermediate slenderness such as 4 < L/D > 8 columns are fails by combination of crushing and buckling. For high slenderness i.e L/D > 8, columns are fails by buckling. Figure 7. Shows the axial load versus deflection curve for the different length of the columns.

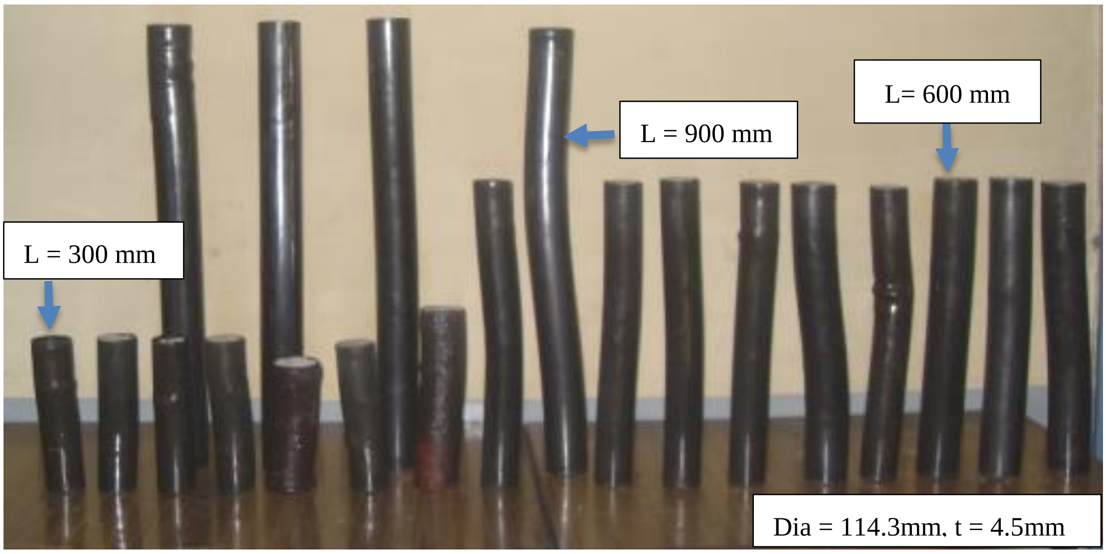
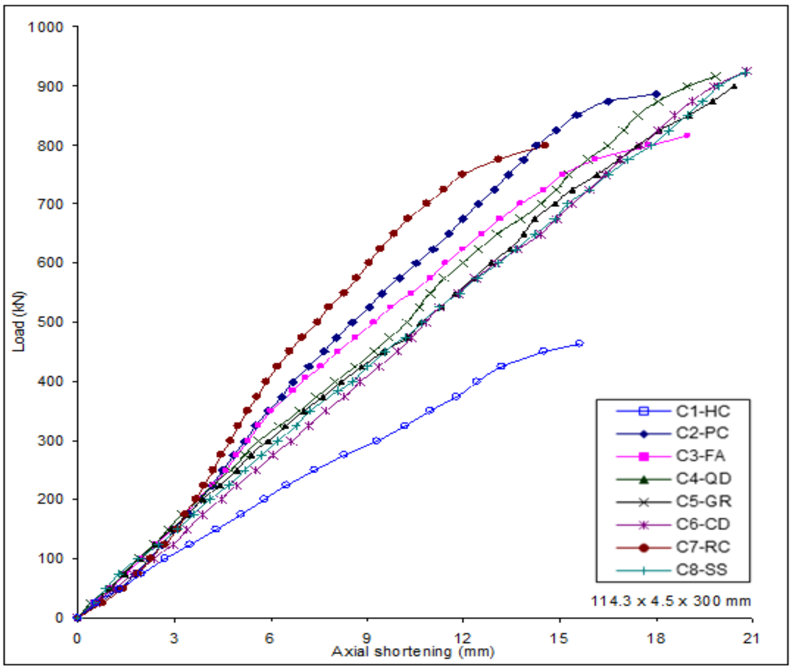


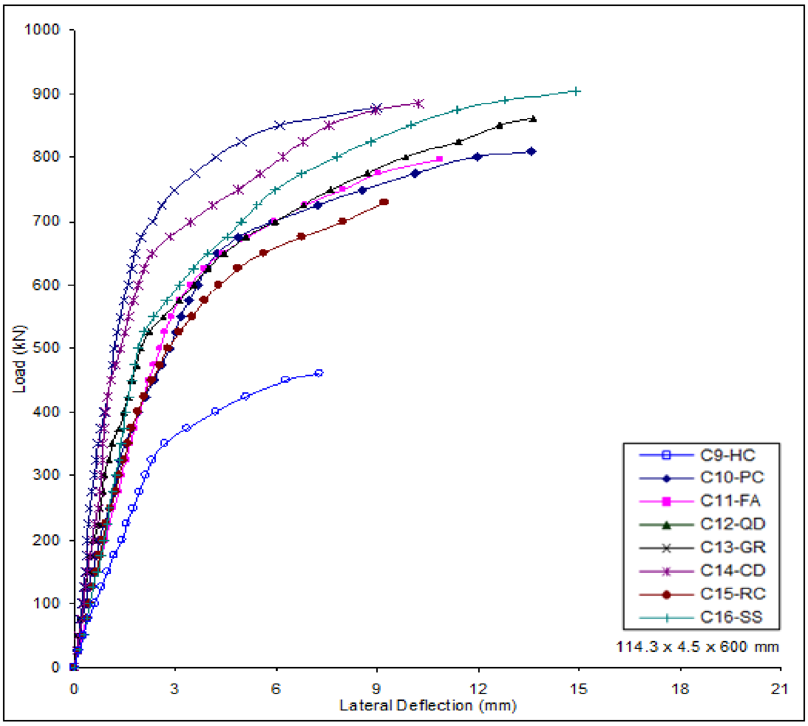
Figure 6: Failure of circular specimens

Also, it was noticed that all the codal provisions underestimating the strength of the column about 30-78 %. It is found that, when L/D ratio reduces, the predicted strength also reduces. In EC4, the difference between predicted and actual strength is 5-35 % only because the slenderness effect has been considered. But in ACI/AS, the difference is up to 78 % because there is no consideration for slenderness effect or L/D ratio. Hence this equation may hold good for L/D > 10 some factor should be multiplied with the existing ACI/AS equation to predict the exact strength.

(a)


Load (kN)
0
100
200
300
400
500
600
700
800
900
1000
0
3
6
9
12
15
18
21
Axial shortening (mm)
C1-HC
C2-PC
C3-FA
C4-QD
C5-GR
C6-CD
C7-RC
C8-SS
114.3 x 4.5 x 300 mm

(b)


Load (kN)
0
100
200
300
400
500
600
700
800
900
1000
0
3
6
9
12
15
18
21
Lateral Deflection (mm)
C9-HC
C10-PC
C11-FA
C12-QD
C13-GR
C14-CD
C15-RC
C16-SS
114.3 x 4.5 x 600 mm

(c)

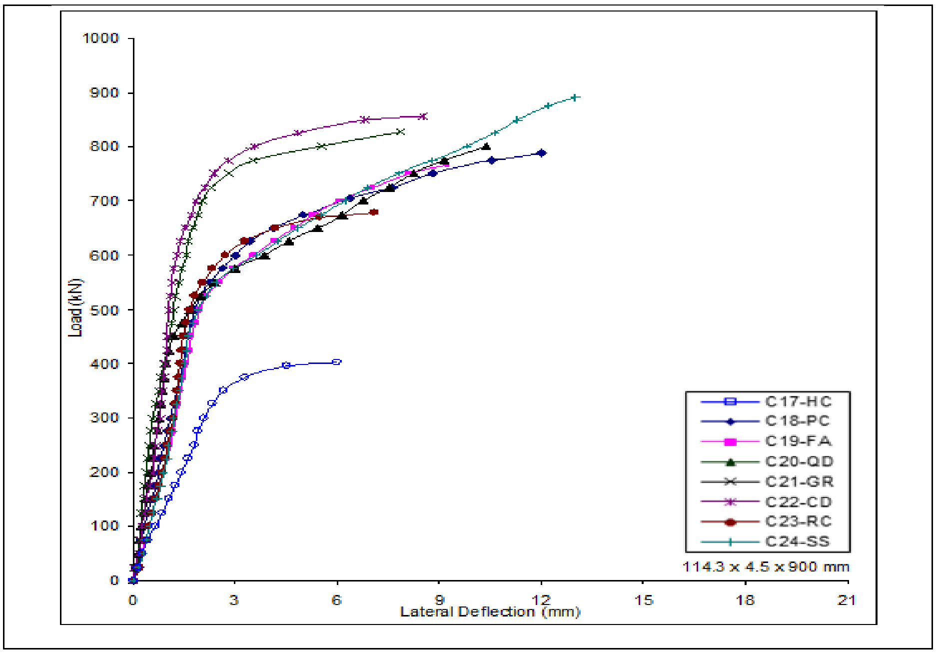


Figure 7: Load Vs Deflection curve for different length of the column (a) L-300, (b) L-600 and (c) L-900

In this study, from experimental results, a factor **k** is suggested for different L/D and λ values. Table 4. Shows the k values for differrent L/D and λ values. Now the equation is slightly modified by multiplying a factor **'k'.** The proposed equation for column is

$$N_{ACI/AS} = \boldsymbol{k}\,[0.85\,A_c\,f_{cc} + A_s\,f_y] \tag{5}$$

Table 4: Values of k for different L/D and λ ratios.

L/D ratio	1	2	3	4	5	6	7	8	9	10	11	12
λ	0.045	0.087	0.137	0.185	0.230	0.260	0.320	0.347	0.388	0.447	0.498	0.542
k factor	1.9	1.8	1.7	1.6	1.5	1.4	1.3	1.2	1.1	1.0	0.9	0.8

To validate the proposed equation three columns of different dimensions have been tested and compared with predicted results and the results are tabulated in Table 5. From Table 5 it was found that the proposed equation gives almost the same strength obtained by experimental result.

Table 5: Comparison of test results with the proposed equation

Outer Dia	**Thick**	**L/D**	**Test result**	**ACI/AS**	**Proposed eqn. (1)**
88.9 mm	4.0 mm	10.1	385.95 kN	356.90 kN	393.67 kN
88.9 mm	4.0 mm	6.75	450.65 kN	356.90 kN	452.92 kN
76.1 mm	3.2 mm	3.94	417.30 kN	255.48 kN	408.77 kN

4. CONCLUSIONS

The results obtained from the tests on composite columns presented in this paper allow the following conclusions to be drawn.

- The predicted axial strengths using EC4 part 1-1 are lower than the results obtained from experiments ranging from 5 to 35 %.

- The ACI/AS also underestimates the strengths and the variation is 30 to 78 %.
- ACI/AS equation gives better results for long columns of L/D > 10.
- For L/D < 12, the modified equation is proposed with the multiplying factor 'k', k values are suggested for different L/D ratio varying from 1 to 12.
- The verification experiment indicates that the proposed equation more accurately predicts the strength of concrete-filled tubular columns.
- The strength of steel tubular columns in-filled with concrete is about 58 to 112 % of hollow columns, and the strength depends on the compressive strength of the in-filled materials.
- The strength of CFSTs with partial replacement of fine and coarse aggregate by waste materials is almost the same as that of plain concrete.
- The strength of partial replacement of quarry dust as fine aggregate and granite, C&D debris and slag as coarse aggregate in CFST columns is more than that of plain concrete.
- The excellent prediction was achieved for C03, C06, C07, C10, C11, C12, C14, C19 & C23 CFST columns, with N_{test} /modified $N_{ACI/AS}$ ratio around unity.
-

5. ACKNOWLEDGMENTS

The authors thank All India Council for Technical Education (AICTE), New Delhi and Department of Science and Technology (DST), New Delhi for financial support also, Principal and Management of Nandha Engineering College (Autonomous) for constant encouragement.

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Appendix

D - outside diameter of column

t - wall thickness of steel tube

N_{EC4} - ultimate axial load of composite column

$N_{ACI/AS}$ - ultimate squash load

As - area of steel tube

Ac - area of concrete

f_{cc} - characteristics cube compressive strength of concrete

f_{cy} - cylinder compressive strength of concrete (0.8 times of f_{cc})

η_1 - co-efficient of confinement for concrete

η_2 - co-efficient of confinement for steel

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