NUMERICAL MODELING OF CONCRETE COMPOSITE STEEL TUBES

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Abstract

In modern days, building design emphasizes on enhancing flexibility of the floor space by reducing the cross section of column size. Enhancing compressive strength of concrete on a smaller column gives more strength and more usable floor space. The use of high strength concrete in smaller column, endeavor to brittle failure. This would be prevented by reducing space of stirrup for higher ductility. However, this causes the stirrup to form a natural plane of separation between the confined concrete core and the unconfined concrete cover, the risk of a premature spalling of the concrete cover increases. This is one important reason why it is advantageous to use Composite Columns consisting of Concrete-Filled Steel Tubes (CFST) instead of traditional Reinforced Concrete Columns. This study aims at developing a suitable constitutive model addressing the behavior of Concrete Filled Steel Tubular column on the compressive response under axial loads. Ultimate load carrying capacities obtained by the Authors using experiments have been compared with the Numerical model values. Three-dimensional nonlinear finite element models developed to study the force transfer between steel tube and concrete core. The nonlinear finite element program ABAQUS 6.12-1 is used. The interaction between steel tube and concrete core is the discussing issue for understanding the behavior of Concrete-Filled Steel Tube Columns (CFST). The numerical results validated with experimental data extracted from previous researchers (International & National) in the field including few experiments by the Authors in terms of Ultimate loads and deformation modes. Modeling related problems such as the definition of boundary conditions, imperfections, concrete-steel interaction, material representation and others are investigated using a comprehensive parametric study. The numerical results are validated through comparison with experimental data in terms of ultimate loading and deformation modes.

A comparison of ultimate failure loads from nonlinear finite element program ABAQUS 6.12-1 with the predicted failure load from Eurocode Part-4 (EC4) (British Standards Institutions), ACI-318 (2005) (American Concrete Institute), for axially loaded columns will be carried out in this research study

From the study it is concluded that, developed Numerical model fits nearer to perfection and depicts the behavior well with 5-10% error. Also, behavior when only Steel tube is loaded has been depicted with loading only Concrete-infill and loading on both infill and Steel tube simultaneously and the combination gave higher ultimate loads. The numerical results validated well with the previous researchers too.

Keywords: *Composite, Confinement, Capacity, Stiffness, Ductility*

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1. INTRODUCTION

At present in Civil Engineering, structures have been redesigned using emerging technology with concrete-filled steel tubular (CFST) structure. A composite steel-concrete structure consists of steel tube and concrete core inside it. The steel tube acts as a permanent formwork and has regular cross-sections: circular, rectangle and square.

Steel sections with concrete infill are being increasingly used as structural members, as filling the steel section with concrete increases both its strength and ductility without increasing the section size. Since steel confines the concrete, the use of formwork can be discarded. However, concrete confinement depends on many factors such as column diameter, the thickness of steel tube, the concrete strength and the yield stress of the steel tube.

It is observed that in long columns, general buckling and in short columns, crushing of concrete. The performance of CFST under sustained loads is different from ordinary reinforced concrete columns. In RCC columns, concrete experiences contraction as it sets during its early age.

Little success has been achieved so far in developing an accurate model due to the complexity in modeling the concrete confinement. The ABAQUS 6.12-1 program was used for the modeling. The unconfined uniaxial stress–strain curve for concrete provided in the ABAQUS [1] material library is used. Hu et al. [6] developed a nonlinear finite element model using the ABAQUS [1] program to simulate the behavior of concrete-filled steel tube columns. The concrete confinement was achieved by matching the numerical results by trial and error via parametric study.

The main objective of this study is to develop an accurate finite element model to simulate the behavior of concrete filled steel tube columns. The finite element program ABAQUS 6.12-1 is used in the analysis.

2. FINITE ELEMENT MODELING

In order to accurately simulate the actual behavior of concrete filled steel tube columns, the main components of these columns have to be modeled properly.

2.1. Modeling and Meshing

The element library of finite element software ABAQUS 6.12-1 is used to select the type of element. Solid elements were found to be more efficient in modeling the steel tube and the concrete Fig.2, as well as it clearly defined boundaries of their elements. Three-dimensional eight-node solid element (C3D8) illustrated in Fig.1 is used in this study.

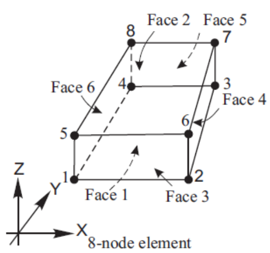

Fig.1 Eight-node solid elements (C3D8)

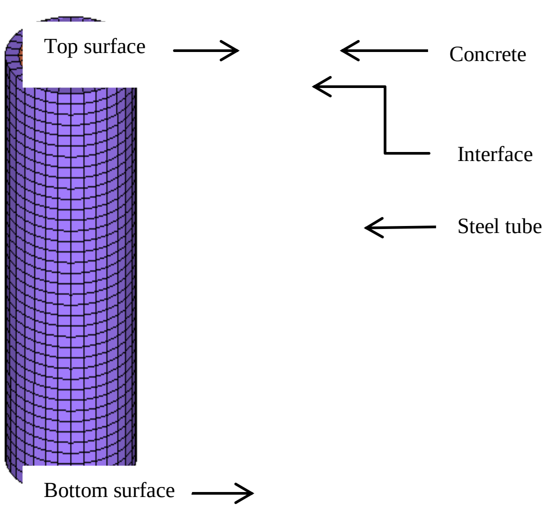

Fig.2 Finite element mesh of concrete filled steel tube column

2.2. Material Model of Concrete

To define the concrete behavior in the FE model, a stress-strain diagram for the confined concrete should be established first. The equivalent stress–strain diagram for confined concrete under compressive loading, as shown in Fig. 3 is used in the proposed FE model. This approach is similar to the one adopted by Ellobody et al [3]

The material properties shown in Fig. 3, is used in defining the nonlinear compressive behavior of the concrete infill are defined as follows:

f_c = Unconfined concrete cylinder compressive strength.

ε_{cc} = 0.003, as recommended by the ACI Specification [2].

f_{cc} = Confined concrete compressive strength.

$= f_c + k_1 f_l$ Proposed by Mander et al. [9]

ε_{cc} = corresponding confined strain to f_{cc}

$= \varepsilon_c \left(1 + k_2 \frac{f_l}{f_c}\right)$ Proposed by Mander et al. [9]

f_l = lateral confining pressure from the steel tube section.

$= \frac{2\,\sigma_\theta\, t}{D}$

$\sigma_\theta = 0.1 f_y$ Equations proposed by Mander, et al. [9].

The factors (k_1) and (k_2) are taken as 4.1 and 20.5, respectively, as given by Richart et al. [10]

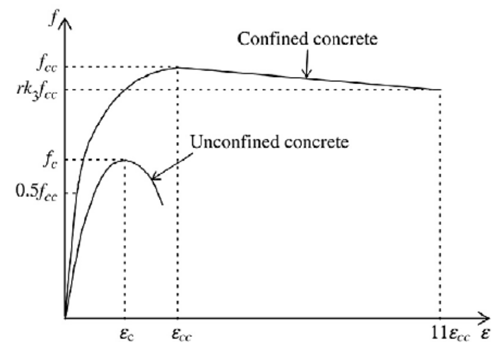

Fig.3 Equivalent stress–strain curves of unconfined and confined concrete

The first part is the initially assumed elastic range to the proportional limit stress. The value of the proportional limit stress is taken as $0.5(f_{cc})$ as given by Hu et al. [6]. The Young's modulus of confined concrete (E_{cc}) is calculated using the ACI code [2]:

$$E_{cc} = 4700\sqrt{f_{cc}} \text{ MPa}$$

The Poisson's ratio (μ_{cc}) of confined concrete is assumed as 0.2.

The second part of the curve is the nonlinear portion starting from the proportional limit stress $0.5(f_{cc})$ and going to the confined concrete strength (f_{cc}). This part of the curve can be determined from following equations, proposed by Saenz [11].

$$f = \frac{E_{cc}\,\varepsilon}{1 + (R + R_E - 2)\left(\frac{\varepsilon}{\varepsilon_{cc}}\right) - (2R - 1)\left(\frac{\varepsilon}{\varepsilon_{cc}}\right)^2 + R\left(\frac{\varepsilon}{\varepsilon_{cc}}\right)^3}$$

$$R_E = \frac{E_{cc}\,\varepsilon_{cc}}{f_{cc}}$$

$$R = \frac{R_E(R_\sigma - 1)}{(R_\sigma - 1)^2} - \frac{1}{R_\varepsilon}$$

The constants R_σ and R_ε are equal to 4 as recommended by Hu and Schnobrich [7].

The third part of the confined concrete stress–strain curve is the descending part from the confined concrete strength *(f_{cc})* to a value lower than or equal to $rk_3\ f_{cc}$ with the corresponding strain of $11\varepsilon_{cc}$. Equations proposed by Hu et al. [6].

$$k_3 = \begin{cases} 1 & 21.7 \le D/t \le 40 \\ 0.00003391\left(\frac{D}{t}\right)^2 - 0.010085\left(\frac{D}{t}\right) + 1.3491 & 40 \le D/t \le 150 \end{cases}$$

The approximate value of r can be calculated from empirical equations given by Ellobody et al. [3]

$$r = \begin{cases} 1 & f_c \le 30MPa \\ \frac{(f_c - 30)(0.5+1)}{(100-30)} & 30 \le f_c \le 100MPa \\ 0.5 & 100MPa \le f_c \end{cases} \qquad (10)$$

There are several material definition algorithms provided by ABAQUS 6.12-1 for the nonlinear behavior of concrete materials. The concrete is modeled by Drucker–Prager yield criteria model available in the ABAQUS [1] material library. Two parameters (Drucker Prager and Drucker Prager Hardening) are used to define the yield stage of confined concrete. The linear Drucker–Prager model is used with associated flow and the isotropic rule. The material angle of friction (β) and the ratio of flow stress in triaxial tension to that in compression *(K)* are taken as 20° and 0.8, respectively, as recommended by Hu et al. [6].

2.3. Material Model of Steel Tube

An elastic-plastic model with the von Mises yield criterion is used to describe the constitutive behavior of steel tube. The complete stress-strain relation obtained from uniaxial tension tests has been used in steel material model. Material properties of steel, such as the Young's modulus and Poisson's ratio are taken as E_s=210000 MPa and μ =0.3, respectively.

2.4. Concrete – Steel Tube Interface

The contact between the steel tube and the concrete is modeled by interface elements. The interface elements consist of two matching contact faces of inner steel tube and outer concrete elements. The friction between the two faces is maintained as long as the surfaces remain in contact. The coefficient of friction between the two faces is taken as 0.20 in the analysis. However, the two contact elements are not allowed to penetrate each other.

2.5. Load Application

The uniform compressive loading was applied to top of column to the steel section (Fig.4), concrete section (Fig.5) and to the entire section (Fig.6). The behavior of column when only Steel tube is loaded has been depicted with loading only Concrete section and loading on both concrete and steel tube section simultaneously and the combination gave higher ultimate loads.

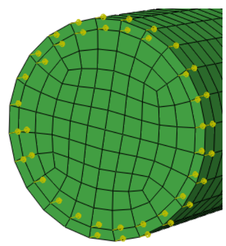

Fig.4 Loading steel section

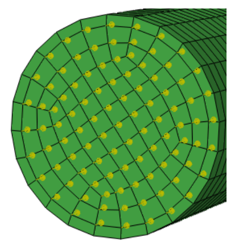

Fig.5 Loading concrete section

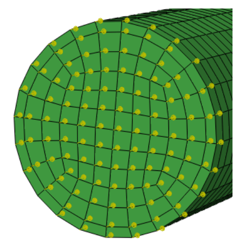

Fig.6 Loading entire Section

2.6. Boundary Conditions

Boundary conditions were enforce with displacement $\delta_x = \delta_y = \delta_z = 0$ on the bottom surface. The top surface of the column is fixed with $\delta_x = \delta_y = 0$ allowing displacement to take place in z direction.

2.7. Solution Procedure

The calculation involves one step of a static buckling analysis. Due to high nonlinearities at local and global levels, accompanying the traced inelastic, unstable and collapse behavior, Riks analysis was chosen as the solution [1]. The Riks method is based on the concept of arc length as a measure of the solution progress in load-displacement configuration space. The increments are established automatically by the program. The user specifies only initial, minimum and maximum increments. The magnitude of an increment depends on the number of iterations and attempts, needed in the previous increment.

For nonlinear problems, as the one described here, the solutions are depended on the magnitude of increments. Although usually they follow initially the same equilibrium path, very often the solution near the critical point experiences problems with convergence and premature termination. For all cases considered here the fast and proper solution was obtained for the default set of analysis parameters and a limited number of points defining inelastic material response. Smaller initial increments or more detailed stress-strain curves led to long calculations and premature termination.

3. VERIFICATION OF FINITE ELEMENT MODEL

3.1. Experimental Results

To verify the finite element model, a comparison between the experimental results and finite element results is carried out. The ultimate loads obtained from the tests (P_{Test}) and finite element analyses (P_{FE}) have been investigated. Table: 1 shows a comparison of the ultimate loads of the concrete filled steel tube (CFST) columns obtained experimentally and numerically using the finite element model. It can be seen that good agreement has been achieved between the two sets of results for most of the columns. A maximum difference of 5-10% was observed between experimental and numerical results for column specimens.

4. ANALYTICAL STUDY

4.1. American Concrete Institute: Building code requirements for Structural Concrete

The ACI [2] use the formula for calculating the squash load. Code doesn't consider the effect due to concrete confinement. The squash load for circular columns is determined by

$$P_u = 0.95\, A_c\, f_c + A_s\, f_s$$

A modification for ACI equations is proposed by Giakoumelis and Lam [5]. A coefficient is proposed for the ACI equation to take into account the effect of concrete confinement on the axial load capacity of concrete filled circular steel tube, a revised equation was proposed as follows:

$$P_u = 1.3\, A_c\, f_c + A_s\, f_s$$

The capacities given by the ACI code are too conservative whereas those calculated by using new equations are more realistic, especially for circular columns.

Table-1: Comparison between tests, finite element, EC4, ACI

Grade of Concrete	*D (mm)*	*t (mm)*	*L/D*	*D/t*	P_{Test} *(KN)*	P_{FE} *(KN)*	P_{Test} / P_{FE}	P_{EC4} *(KN)*	P_{ACI} *(KN)*	P_{Lam} *(KN)*
M20	33.40	1.65	5.988	20.24	72.30	71.58	1.010	74.48	64.54	69.52
	42.20	1.65	4.739	25.58	93.20	98.62	0.945	105.62	87.74	96.06
	48.30	1.65	4.141	29.27	110.4	117.29	0.941	128.73	105.18	116.31
M25	33.40	1.65	5.988	20.24	78.30	73.71	1.062	77.07	67.92	74.15
	42.20	1.65	4.739	25.58	99.20	102.87	0.964	110.07	93.39	103.79
	48.30	1.65	4.141	29.27	118.3	124.50	0.950	134.65	112.74	126.65
M30	33.40	1.65	5.988	20.24	84.30	75.42	1.178	79.65	71.30	78.77
	42.20	1.65	4.739	25.58	102.0	103.96	0.981	114.39	99.03	111.51
	48.30	1.65	4.141	29.27	128.3	131.77	0.974	140.58	120.29	136.99

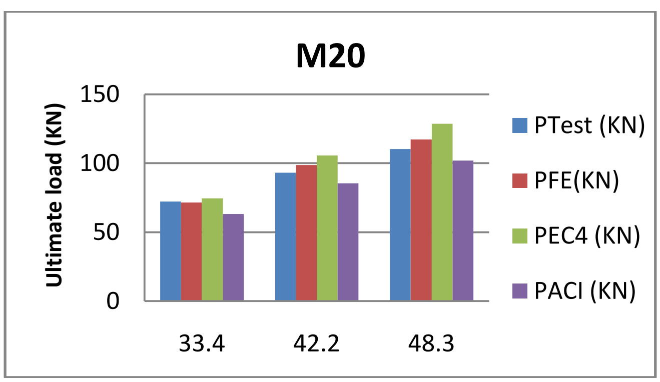

Chart-1: Grade of Concrete M20

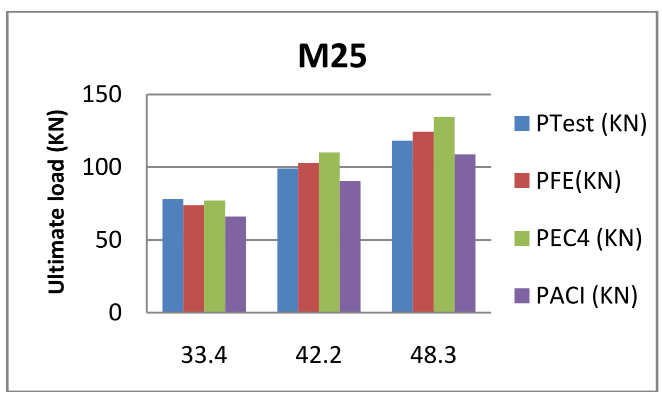

Chart-2: Grade of Concrete M25

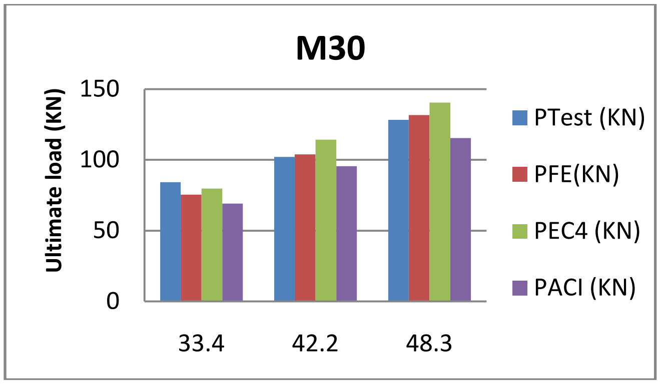

Chart-3: Grade of Concrete M30

4.2. Eurocode 4

EC4 [4] is the most recently completed international standard in composite construction. EC4 covers concrete filled steel section sections with or without reinforcement. EC4 considers confinement effects for circular sections when relative slenderness (λ) has value less than 0.5. It is the only code that treats the effects of long-term loading separately. The ultimate axial force of a square column is

$$P_u = A_c f_c + A_s f_s$$

Where: A_s and A_c are the area of steel and concrete respectively, and f_s and f_c are the strength of steel and concrete respectively.

For circular columns, confinement effects have to be incorporated if the relative slenderness λ is less than 0.5, where λ is defined as

$$\lambda = \sqrt{\frac{A_s f_s + 0.85 A_c f_c}{N_{cr}}}$$

$$N_{cr} = \frac{(E_s I_s + E_c I_c)\pi^2}{l^2}$$

$$\eta_1 = 4.9 - 18.5\lambda + 17\lambda^2 + 1$$

$$\eta_2 = 0.25(3 + 2\lambda)$$

$$P_u = A_s f_s \eta_2 + A_c f_c \left(1 + \eta_1 \frac{t f_s}{d f_c}\right)$$

5. COMPARATIVE STUDY

5.1. General

The ultimate section capacity of different CFST sections based on their *D/t* ratio and *L/D* ratio were found according to Eurocode 4 [4], ACI [2] & Giakoumelis and Lam [5]. The theoretical and numerical capacity of CFST sections developed using the above codes denote that increase in *D/t* ratio enhances the capacity which is due to increased confinement pressure and decrease in *L/D* ratio also enhances the capacity of the section which is due to the slenderness effect.

5.2. Slenderness Ratio of the Column

The length to diameter ratio (*L/D*) represents the slenderness of the column. The failure modes of concrete-filled columns are characterized by yielding of steel followed by crushing of concrete. The strength increase will occur only for columns of smaller slenderness ratio (or *L/D* ratio). Columns with greater slenderness ratio fail by overall buckling. Hence it can be observed from the analytical results that the decrease in *L/D* ratio increases the section capacity of the CFST column.

5.3. Diameter to Thickness Ratio

The increase in *D/t* ratio may be either due to the increase in diameter or due to the decrease in thickness of the section. Hence it is analyzed by keeping the thickness constant and varying the diameter. The increase in *D/t* ratio with increased diameter for a constant thickness represents the improvement in cross section of the steel tube and hence produces greater section capacity.

5.4. Grade of Concrete

The strength of concrete core decides stiffness of CFST columns. Stiffness increases with increase in concrete strength but columns fail due to crushing of concrete exhibiting brittle behavior when filled with high strength concrete. But it is a fact that increase in concrete core strength increases the strength of filled columns to a larger extent, no matter of either *D/t* ratio or *L/D* ratio.

5.5. Performance Index (PI)

In the LRFD code [8], a column is classified as composite if it has a structural steel area to the cross sectional area ratio of more than 0.04 otherwise it is treated as a concrete column. In Eurocode 4 [4], the steel contribution ratio in a composite column section, which is defined as the ratio of the steel section strength to the composite section strength, must be greater than 0.2. To evaluate the section performance of composite columns, a performance index is proposed here as

$$PI = \frac{\Delta_{ultimate}}{\Delta_{yielding}}$$

The performance indexes (PI) so determined are listed in Table 2. The performance indexes decreases with increase in *D/t* (ref. Chart-6). The reasons are very similar to the causes of the strength index changing with increase of *D/t.*

5.6. Ductility Index (DI)

One of the parameters used to quantify section ductility is the ductility index.

$$DI = \frac{\varepsilon_{95\%}}{\varepsilon_{ue}}$$

ε_{ue} is the strain at the ultimate load, and $\varepsilon_{95\%}$ is the strain when the load falls to 95% of the ultimate load. The ductility indexes *(DI)* so determined are listed in Table 2. The ductility index decreases with increase in *D/t* (ref. Chart-5). The reasons are very similar to the causes of the strength index changing with increase of *D/t*.

5.7. Strength Index (SI)

A strength index is defined to quantify the section strength:

$$SI = P_{Test} / P_{ACI}$$

$$\text{Where } P_{ACI} = A_s f_s + 0.95 A_c f_c$$

The section capacity as per ACI [2]

The strength indexes *(SI)* so determined are listed in Table 2. The strength index decreases with increase of *D/t* (ref. Chart-8, 9). The reasons are that, the constraining factor for the specimen decreases with increase of *D/t*, i.e. the composite action of steel tube and core concrete becomes smaller (ref. Chart-7).

$$\xi = \left(\frac{As}{A_c}\right)\left(\frac{f_s}{f_c}\right)$$

6. CONCLUSIONS

1. Increase in *D/t* ratio enhances section capacity (*Pu*) of column, as the confinement pressure increases only when diameter increases.
2. Increase in slenderness ratio (or *L/D* ratio) reduces section capacity (*Pu*) of the column.
3. The strength of concrete core greatly influences the section capacity CFST columns. Sections filled with high strength concrete exhibits higher section capacity (*Pu*).
4. Increase in concrete core strength increases section capacity (*Pu*) of column to a large extent, no matter of either *L/D* or *D/t* ratio.
5. Stiffness increases with increase in concrete strength but columns fails due to crushing of concrete which shows brittle failure behavior when filled with higher grade of concrete.
6. Increase in grade of concrete depicts the confinement factor i.e. the composite action of steel tube and core concrete becomes smaller.
7. Varying the diameter & grades of concrete, increase in confinement factor will not affect ductility of column.
8. Comparison with Finite Element model results obtained from ABAQUS 6.12-1 and experimental results for columns with different grades of concrete with different geometric dimensions (length, diameter, and thickness) results in predicting the column behavior.
9. Results from parametric study showed that column design rules specified in EC4 [4], ACI [2] & Giakoumelis and Lam [5] are in well agreement with experimental results but EC4 is closer to obtained results.
10. Strength increase will occur in CFST columns only in the smaller slenderness ratio (*L/D*).

Table-2: Stiffness index, performance index, ductility index, confinement factor, strength index

Grade of Concrete	*D (mm)*	*t (mm)*	*L/D*	*D/t*	*Stiffness Index (N/mm)*	*Ductility Index*	*Performance Index*	*Confinement Factor*	*Strength Index (EC4)*	*Strength Index (Abaqus)*
M20	33.40	1.65	5.988	20.24	73.909	6.1527	10.5522	2.891	0.971	1.010
	42.20	1.65	4.739	25.58	56.524	5.2930	9.9483	2.211	0.882	0.945
	48.30	1.65	4.141	29.27	43.797	3.1068	5.2683	1.901	0.858	0.941
M25	33.40	1.65	5.988	20.24	85.717	6.7669	12.8798	2.313	1.051	1.062
	42.20	1.65	4.739	25.58	80.434	5.2790	11.3537	1.769	0.902	0.964
	48.30	1.65	4.141	29.27	65.147	3.2404	9.1260	1.520	0.806	0.950
M30	33.40	1.65	5.988	20.24	103.795	6.1079	9.3610	1.927	1.088	1.118
	42.20	1.65	4.739	25.58	99.900	5.3329	8.5613	1.474	0.892	0.981
	48.30	1.65	4.141	29.27	89.424	3.9065	7.7178	1.267	0.756	0.974

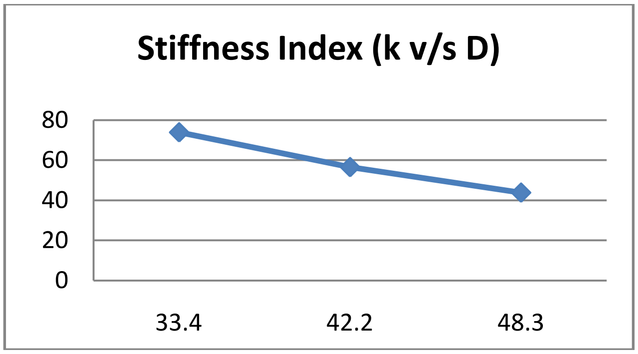

Chart-4: Variation of Stiffness index.

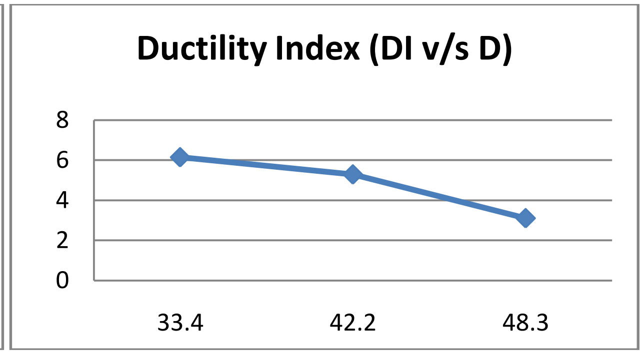

Chart-5: Variation of Ductility index.

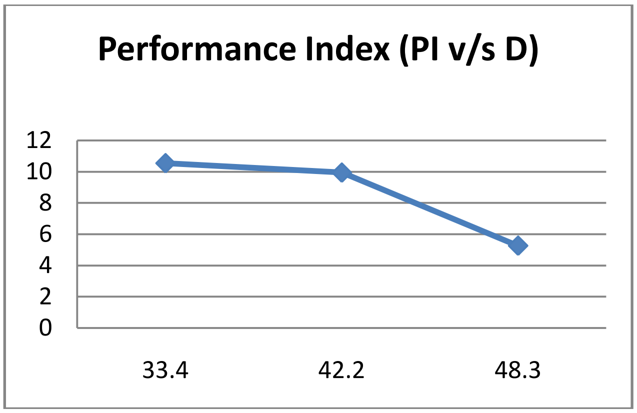

Chart-6: Variation of Performance index

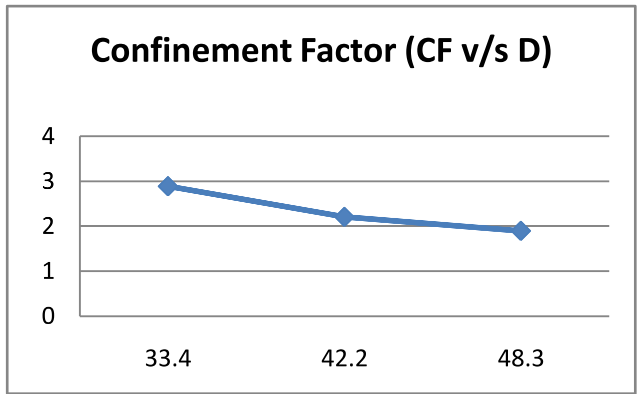

Chart-7: Variation of Confinement factor

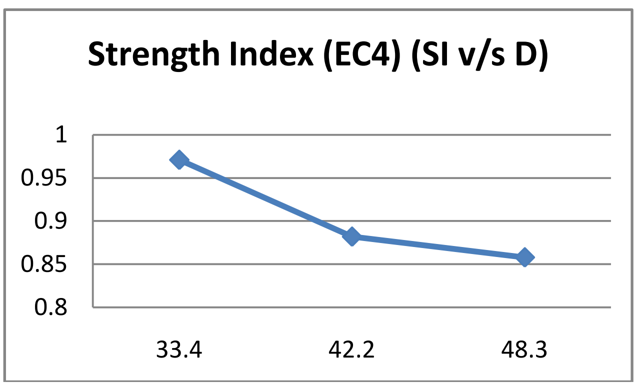

Chart-8: Variation of Strength index EC4

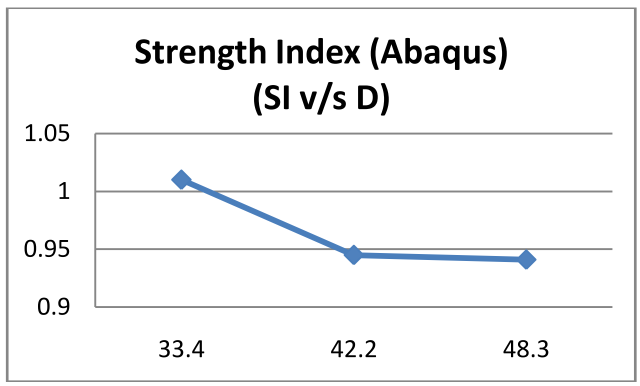

Chart-9: Variation of Strength index (Abaqus).

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BIOGRAPHIES

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