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Ultimate load capacity of circular hollow sections filled with higher strength concrete

Hamid Reza Kavoossi
University of Wollongong

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ULTIMATE LOAD CAPACITY OF CIRCULAR HOLLOW SECTIONS FILLED WITH HIGHER STRENGTH CONCRETE

A thesis submitted in fulfilment of the requirements
for the award of the degree of

Doctor of Philosophy

from

UNIVERSITY OF WOLLONGONG

by

HAMID REZA KAVOSSI, BE, MSC

DEPARTMENT OF CIVIL AND MINING ENGINEERING

1993
DECLARATION

This is to certify that the work presented in this thesis was carried out by the author in the Department of Civil and Mining Engineering, University of Wollongong, and has not been submitted to any other university or institute for a degree except where specifically indicated.

Hamid Reza Kavoossi
1993
ACKNOWLEDGEMENT

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Special Acknowledgment is made to his wife for her encouragement and patient company.

Finally the author wishes to thank his parents for their forbearance and understanding during such a long period of time in which he placed this study above his responsibility to them.
ABSTRACT

An experimental and theoretical study concerning the ultimate load behaviour of the circular hollow sections filled with higher strength concrete has been carried out. This study involves the structural characteristics of the composite sections under axial concentric and the eccentric loads.

A total number of 63 experimental tests have been carried out for the different loading situations and structure forms. This study has been divided into the three main parts. The first part concerns the behaviour of the hollow sections, and the second part is a study on the ultimate load behaviour of the circular hollow sections filled with higher strength concrete under concentric loads. In the last part the structural behaviour of the composite sections under eccentric loads has been studied.

The steel grade of the circular hollow sections can be categorised as a high strength steel with the yield stress in the range of 400-450 MPa. The compressive capacity of the concrete is within the range of 60-109 MPa. The maximum outside diameter of the steel tube used is 168.3 mm and the minimum is 114.3 mm. The wall thicknesses for the maximum diameter tube are 4.8 mm and 9.53 mm. Although these dimensions are not commonly used for structural purposes but covering the whole range of possible dimensions increase the possible options for a perfect design. An epoxy coated tubes of 4.8 mm wall thickness have also been considered. The wall thicknesses for the minimum outside diameter are 6.3 mm and 4.8 mm. For the justification of the results obtained in Chapter three, another set of tubes with an outside diameter of 62 mm and wall thickness 2 mm and 2.5 mm, and a set of four steel tubes with outside diameter equal to 114.3 mm and wall thickness equal to 6.0 mm have been studied.

In each part of the thesis the results obtained by the tests have been compared with a number of test results by the other authors, and the available criteria for these studies have been investigated to obtain a better solution for estimation of the load carrying capacities.

In addition, the experimental specimens have been modelled for a non-linear finite element analysis to investigate the intensity of stresses in the different directions. Furthermore, the interaction of the concrete core and the steel tube have been modelled by the utilisation of gap elements to perform a real analytical model for the composite sections.
The results of this study show that some of the available methods, and even codes of practice, have a non-realistic estimation for the axial load capacity of the composite sections. The studies on the ultimate load behaviour of the bare tube show that the relationship between the ultimate strain and the slenderness of the tube can be presented by a linear equation which is in agreement with previous studies.

The results of the finite element analysis conclude to an approximate method for determining the axial load-shortening curve of the composite section which is in good agreement with the finite element results and the results obtained by the experiments.
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NOTATION

(-)' = \frac{\partial}{\partial x}(-)

(-)' = \frac{\partial}{\partial \phi}(-)

4H = Length of one complete cycle of folding.

A_g = Total cross section area of the steel tube and the concrete core.

AE and Av = Apparent modulus of elasticity and Poisson’s ratio, respectively.

A_f_y and A_e_y = Yield stress and strain according to the apparent values for
Modulus of elasticity and Poisson’s ratio, respectively.

A_s = Cross section of the steel tube.

C_E and C_v = Classic modulus of elasticity and Poisson’s ratio, respectively.

C_f_y and C_e_y = Yield stress and strain according to the classic values for
Modulus of elasticity and Poisson’s ratio, respectively.

d = outer diameter of steel tube.

d_e_p = Incremental plastic strain vector.

D_e_p = Plastic stress-strain matrix.

\delta_{ij} = The simplest second order symmetric tensor (Kronecker delta)
or substitution tensor

d\lambda = A non negative constant which may vary throughout the
loading history.

E = Modulus of elasticity for steel.

E_b = Bending energy.
\( \varepsilon_0 \) = Circumferential strain.

\( E_t \) = Equivalent modulus of elasticity for the composite section, Eq. 7.4.8.3.

\( E_m \) = Membrane energy.

\( \varepsilon_r \) = Radial strain.

\( F \) = Von-Mises yield criterion.

\( f_{ul} \) and \( e_{ul} \) = Ultimate stress and strain in steel tube, respectively.

\( f_y \) = Yield stress of the steel tube.

\( f_y \) = yield stress of steel.

\( F(F) \) = The vector of grid point forces due to forces generated as functions of displacement, temperature and stress history.

\( H \) = Half of the length of a folding element.

\( J_2 \) = Second invariant of the stress deviator tensor.

\( l \) = length of steel tube(l).

\( \lambda \) = relative slenderness.

\( m \) = Eccentricity parameter (as shown in Fig. 3.3.1), Where the eccentricity parameter can be measured from the test results.

\( m \) = The number of the complete waves around the circumference of the circular hollow sections (integer).

\( M_u \) = Bending moment corresponding to the axial load along the composite column.

\( M_0 \) = Fully plastic bending moment per unit length = \( \frac{1}{4} \cdot \sigma_0 \cdot t^2 \)

\( MS \) = Micro strain
\( v \) = Poisson's ratio of steel.

\( n \) = The number of the half waves along the length of the cylinder (integer).

\( P \) = Perimeter of the steel tube.

\( P \) = Uniform load per unite length of the circumference of cylinder.

\( P_u \) = Ultimate axial load of the composite column.

\( P_{au} \) = Maximum load of the composite section.

\( P_c \) = Load on composite section.

\( P_m \) = The optimum value of the mean crushing force.

\( P_{sq} \) = Squash load of the steel tube.

\( P_{ul} \) = Ultimate load in steel tube.

\( \{P\} \) = Vector of applied external load.

\( \{Q\} \) = Force due to constraints.

\( r \) = Outer radius of steel tube.

\( \sigma_l \) = Longitudinal stress.

\( S_{ij} \) = Stress deviator tensor.

\( \sigma_o \) = Yield stress of the steel.

\( t \) = Wall thickness of steel tube.

\( T,C \) = Yield strength in uniaxial tension and compression.

\( \lambda_s \) = Normalised slenderness.

\( w \) = Density of concrete.

\( \sigma_c \) = Stress in steel tube.
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CHAPTER ONE

INTRODUCTION

1.1 INTRODUCTION

Based on the experimental approaches on a number of construction methods for the columns in high rise structures, it is found that the concrete filled steel tube is a very constructable and economical solution for multi-storey building Ref. [54]. A major factor is the ability to reduce the costs and the on-site duration of construction.

From the viewpoint of the structural characteristics, the most important reason for the economy of this kind of column, is the confinement of the concrete by the tube, which further boosts the capacity of the concrete. This is not possible with any other composite column configuration Ref. [54]. On the other hand, a substantial proportion of the axial load is taken on the high strength concrete, which is the most economical way to resist the compressive force.

In addition, the constructibility of the unreinforced filled tube is its principal advantage. The absence of reinforcement has extensive benefits in reducing cranage, handling, and site labour, as well as greatly simplifying fabrication and assembly of the columns. Moreover, the stability of the steel tube will be improved against the local buckling.

According to the advantages of this system, there is an emerging trend all over the world towards the use of concrete filled hollow sections for the design of high rise structures. For instance, many structures by this method have been constructed in Australia Refs. [44, 54, 55], China Ref. [45, 46, 47], United States of America [56, 42], and many other places [38, 39, 43, 48, 57]. In addition, the ductility of this system make the composite section as a most suitable structural system in seismic areas.

The conventional method for evaluation of the ultimate load capacity of a composite section under the general form of loads relies on a series of experimental factors to consider the changes in the structural characteristics of the steel tube and the concrete core, due to the confinement effects. Furthermore, the ultimate load capacity has to be determined by considering the combination of the axial load and the bending moment.
1.2 SCOPE OF RESEARCH

According to the previous theoretical studies and the experimental research that has been outlined in Chapter 2, the current research concerns the following areas:

(1) investigations on the mix proportions of the higher strength concrete from the viewpoint of workability and practical strength.

(2) an extensive study on the structural behaviour of the bare circular hollow sections to determine the necessary characteristics of the high strength steel tube (electric resistance weld, ERW).

(3) parallel studies on the effect of the heating in the process of the epoxy coating of the high strength steel circular hollow sections.

(4) theoretical investigations for determining the stress from the measured strain in a steel tube under axial compression.

(5) study on the plastic mechanism of a bare high strength circular hollow section under axial compression.

(6) an extensive study on the structural behaviour of the higher strength concrete (70 < f'c < 100 MPa), confined in a circular hollow section.

(7) experimental, comparative, and, statistical studies on the ultimate load carrying capacity of a circular hollow section filled with higher strength concrete.

(8) experimental studies for investigation of the load carrying capacity of a circular hollow section filled with higher strength concrete when the load is applied on the steel tube only.

(9) theoretical and experimental investigation on the ultimate load capacity of a composite circular hollow section filled with higher strength concrete when the axial load is applied eccentrically.

(10) non-linear finite element analysis of the bare tubes and composite sections with use of the scalar elements (Gap element) to model the interaction of the concrete core and the steel tube.
1.4 OUTLINE OF THESIS

There are nine chapters in the thesis presented in order to furnish the scope of the research.

Chapter 2, introduces previous work on the confinement of concrete and the stress-strain relationships for different specifications of the concrete. In addition, many of the available studies about the behaviour of confined concrete in a steel tube and the load carrying capacity of the composite sections are presented.

The theoretical and experimental studies about the structural characteristics of the bare high strength steel tubes are discussed in Chapter 3. The main concern in this study is to establish the necessary relationships and method for use in the studies on the composite sections.

In this chapter, the methods for determining the stresses from measured strains have been studied and a computer program for practical use in this thesis have been developed. Furthermore, a new method for computing the average values of measured strains from the tests have been discussed.

The experimental program in this chapter has been justified with a new series of the steel tube with different dimensions to get a good relationship between the ultimate strain in the bare steel tube and the slenderness defined as Eq. 1.4.1:

$$\lambda_s = \sqrt{\frac{f_y}{f_c}} = 1.286 \sqrt{\frac{(d/2)/t}{fy/E}}$$

(1.4.1)

where, $t =$ wall thickness, $d =$ outer diameter, $f_y =$ the yield stress for steel, $E =$ modulus of elasticity, $f_c =$ longitudinal stress in the steel tube at squash load

Chapter 4 deals with the structural characteristics of confined concrete in a steel tube. The importance of this study is to establish a realistic estimation of the enhancement of the compressive strength of the confined concrete, as well as the beneficial effects in the ductility of the composite sections. A considerable number of experimental and theoretical equations for this purpose have been compared with the experimental results in this study. Moreover, the best available equations in this field have been modified for improving the accuracy of the results.

The studies on the structural behaviour of a circular hollow section filled with higher strength concrete are presented in Chapter 5. The load in this case applies to both, the steel tube and the concrete core. The recommended relationships by some of the codes
of practice have been studied in this chapter and a practical equation for evaluation of
the load carrying capacity of a composite section, based of the results in this study and
some other experimental results in the literature, have been suggested.

An experimental study about the behaviour of the composite section when the axial
load is applied on the steel tube only is given in Chapter six.

Chapter seven represents a detailed study on the behaviour of composite sections under
eccentric loads. In this study the strain and stress history of the composite section
during the loading process have been investigated. The conventional method for the
design of a composite section under axial eccentric load has been studied, and the
necessary modifications are introduced to calculate a more accurate result for the axial
load carrying capacity.

Chapter eight discusses a series of non-linear finite element analysis on the structural
behaviour of the bare steel tube and the circular hollow section filled with concrete.

Chapter nine presents conclusions of the research work and recommendations for
further work.
CHAPTER TWO

LITERATURE REVIEW

2.1 HISTORICAL BACKGROUND ABOUT CONFINED CONCRETE

Axial load on confined concrete is commonly used to investigate the effects of lateral stresses on the longitudinal behaviour. In the research results by Kitada, Yoshida and Nakai Ref. [20], the stress path of the encased concrete at the midheight of the specimen, that is indirectly calculated by using the measured strains of the steel tube, is compared with the failure criteria proposed by Ottosen Ref. [21] and Cai Ref. [22]. The experimental results show that the stress path of radial and axial stresses proceeds along the failure criterion of Cai rather than that of Ottosen. The general form of the Cai failure criterion can be shown by Eq. 2.1.1.

\[ f_c^* = f_c [1 + 1.5 \sqrt{\frac{\sigma_r}{f_c}} + 2 \frac{\sigma_r}{f_c}] \]  

(2.1.1)

where:

- \( f_c \) = Ultimate compressive strength of unconfined concrete.
- \( f_c^* \) = Ultimate compressive strength of confined concrete.
- \( \sigma_r \) = Radial stress on concrete.

As the failure criterion proposed by Cai can be expressed by a simple equation and coincides well with the stress path obtained from the experiments by Kitada et al., it seems to be applicable and useful to evaluate the behaviour of composite columns in axial concentric compression.

Early investigations showed that the strength and the corresponding longitudinal strain at the ultimate stage of concrete confined by an active hydrostatic fluid pressure can be presented by the following simple relationships Refs. [34, 35, and, 36]:

\[ f_c^* = f_c + K_1 \sigma_r \]  

(2.1.2)

\[ \varepsilon_c^* = \varepsilon_c (1 + K_2 \frac{\sigma_r}{f_c}) \]  

(2.1.3)

Where \( \varepsilon_c^* \) = the maximum concrete strain, under the lateral fluid pressure \( \sigma_r \); \( \varepsilon_c \) = unconfined concrete strain; and \( K_1 \) and \( K_2 \) = coefficients that are functions of the
concrete mix and the lateral pressure. Richart et al. Refs. [34, and, 36] found the average values of the coefficients for the tests they conducted to be $K_1 = 4.1$ and $K_2 = 5.6$. Also, Balmer Ref. [35] found from his tests that $K_1$ varied between 4.5 and 7.0 with an average value of 5.6, the higher values occurring at the lower lateral pressures. Richart et al. Refs. [34, and, 36] also found that the strength of concrete with active confinement from lateral fluid pressure was approximately the same as the concrete with passive confinement pressure from closely spaced circular spirals causing an equivalent lateral pressure.

The stress strain curve of confined concrete can also be predicted for various forms of confinement. Confinement of concrete by transverse reinforcement in reinforced concrete columns is approximately in the same situation as composite circular hollow sections. One of the most practical models in this field is the modified Kent and Park model that is a function of transverse reinforcement and concrete specifications. The relationship for the stress enhancement factor can be presented as the following equation:

$$K = 1 + \frac{\rho f_{yh}}{f_c}$$  \hspace{1cm} (2.1.4)

where:

$\rho = \text{Ratio of the volume of transverse reinforcement to the volume of concrete core measured from the outside of the hoop.}$

$f_{yh} = \text{Yield strength of transverse reinforcement.}$

The modified model assumes that for the confined concrete core, the maximum stress reached is $Kf_c$, and the strain corresponding to the maximum stress is 0.002K. The detailed form of this model for the stress-strain behaviour of concrete, according to Fig. 2.1.1, can be shown as follows:

Region AB ($\varepsilon_c < 0.002K$)

$$f_c = Kf_c\left[\frac{2\varepsilon_c}{0.002K} - \left(\frac{\varepsilon_c}{0.002K}\right)^2\right]$$  \hspace{1cm} (2.1.5)

Region BC ($\varepsilon_c > 0.002K$)

$$f_c = Kf_c[1 - Z_m(\varepsilon_c - 0.002K)]$$  \hspace{1cm} (2.1.6)

but not less than 0.2$Kf_c$, in which
\[ Z_m = \frac{0.5}{3 + \frac{0.29f_c}{145f'_c - 1000} + \frac{3\rho s}{4}\sqrt{\frac{h''}{sh}} - 0.002K} \]  

and:

\[ Z_m = \frac{\tan \theta_m}{Kf'_c} \]

and in which \( f_c \) is in megapascals; \( K \) is as given in Eq. 2.1.4; \( h'' \) = width of the concrete core measured to the outside of the peripheral hoop; and \( sh \) = centre to centre spacing of hoop sets Ref. [23].

According to the test results, carried out at the University of Canterbury on reinforced concrete columns, the modified stress-strain curve by Kent and Park takes the increase in peak stress into account accurately, but the increase in strain at peak stress was not as high as was actually observed in the tests Ref. [24].

Moreover, experimental results by Scott, Park and Priestly showed that the Modified Kent and Park stress-strain relationship for compressed concrete confined by hoop reinforcement gave good agreement with the stress strain curves measured for the concrete core in the concentric load tests conducted at a low strain rate. For high strain rate tests, good agreement with the measured curves was obtained from the relations by applying a multiplying factor of 1.25 to the peak stress, the strain at the peak stress, and the slope of the falling branch Ref. [29].
Based on the assumption that the effectively confined concrete area is less than the core area and is determined by the distribution of longitudinal steel, the resulting tie configuration and the spacing of the ties, an analytical model has been proposed by Sheikh, Shamim and Uzumeri Ref. [28]. The governing equations for a square section with uniform distribution of longitudinal steel bars are given as:

\[
f_c^* = Kf_c
\]

\[
K = 1 + \frac{(h')^2}{P_{occ}} \left[ (1 - \frac{nc^2}{0.55(h')^2}) (1 - \frac{sh}{2h}) \right] \sqrt{\rho_s f_y h}
\]

Where \( A_{co} \) = area of core measured from centre to centre of the perimeter tie; \( A_s \) = area of longitudinal steel; \( h' \) = core size measured from centre to centre or perimeter tie in inches; \( C \) = distance between laterally supported longitudinal bars of \( 4h'/n \) in inches; \( f_c \) = Strength of unconfined concrete in the column = \( Kp f_c \); \( f_s \) = stress in the lateral steel in psi; \( f_c \) = cylinder strength of concrete in psi; \( Kp \) = ratio of unconfined concrete strength in the column to \( f_c \); \( n \) = number of arcs containing concrete that is not effectively confined, also equal to the number of laterally supported longitudinal bars; \( P_{occ} = Kp f_c (A_{co} A_s) \), unconfined strength of concrete core in kips; \( sh \) = tie spacing in inches; \( \rho_s \) = ratio of the volume of tie steel to the volume of concrete core; and \( e_c \) = strain corresponding to the maximum stress in the unconfined concrete.

As the strength of concrete in the specimens, on which the model was based, was in the vicinity of 28 MPa (4000 psi), Eq. 2.1.9 produced results that compared well with the test results. A more general equation, suitable for variable concrete strength, was later suggested (Sheikh and Yeh Ref. [59, 58]) as follows:

\[
e_c^* = 0.0022K
\]

Eq. 2.1.14 also makes it possible to use a standard second degree parabolic equation to represent ascending parts of the curves for both the confined and unconfined initial tangent modulus of elasticity.

Mander et al. Ref. [30] have proposed a unified stress-strain approach for confined concrete applicable to both circular and rectangular shaped transverse reinforcement. The stress-strain model is illustrated in Fig. 2.1.2. For a slow strain rate and monotonic loading, the longitudinal compressive concrete stress \( f_{con} \) is given by Eq. 2.1.15.

\[
f_{con} = \frac{f_c^* x r}{r - 1 + x^r}
\]
\[ x = \frac{\varepsilon_{\text{con}}}{\varepsilon_c^*} \]  

(2.1.16)

Where \( \varepsilon_{\text{con}} \) = longitudinal compressive concrete strain and \( f_{\text{con}} \) = longitudinal compressive stress of concrete.

\[ \varepsilon_c^* = \varepsilon_c [1 + 5(\frac{\varepsilon_c^*}{f_c} - 1)] \]  

(2.1.17)

\[ r = \frac{E_c}{E_c - E_{\text{sec}}} \]  

(2.1.18)

where:

\[ E_c = 5,000 \sqrt{f_c} \]  

(2.1.19)

is the tangent modulus of elasticity of the concrete in MPa and

\[ E_{\text{sec}} = \frac{f_c^*}{\varepsilon_c^*} \]  

(2.1.20)
To determine the confined concrete compressive strength $f_c^*$, a constitutive model involving a specified ultimate strength surface for ultimate compressive stresses was used by Mander et al.. The "Five-Parameter" multi axial failure surface described by William and Warnke Ref. [60] was adopted, as it provides excellent agreement with triaxial test data. The calculated ultimate strength surface based on the triaxial tests of Shickert and Winkler Ref. [61] was utilised. When the confined concrete core is placed in triaxial compression with equal effective lateral confining stresses $\sigma_1$ from spirals, circular hoop or a steel circular hollow section Ref. [30], it can be shown that the confined strength given is:

$$ f_c^* = f_c(-1.254 + 2.254 \sqrt{1 + \frac{7.94 \sigma_1}{f_c} - \frac{2 \sigma_1}{f_c}}) $$

(2.1.21)

Another analytical model available in the literature for the confinement of concrete by rectangular ties, was proposed by Fafitis and Shah Ref. [25]. To define the stress-strain behaviour of confined concrete, a set of equations to predict the ascending and descending curve of the confined concrete was proposed.

$$ f_{con} = f_c^* \left[1 - \left(1 - \frac{\varepsilon_{con}}{\varepsilon_c^*}\right)^A\right] $$

(2.1.22)

for the ascending branch, and

$$ f_{con} = f_c^* \exp[-K(\varepsilon_{con} - \varepsilon_c^*)^{1.15}] $$

(2.1.23)

for the descending branch where $f_{con}$ and $\varepsilon_{con}$ are stress and strain for the confined concrete, $f_c^*$ and $\varepsilon_c^*$ are stress and strain at the peak of the stress-strain curve, and $A$ is a parameter that controls the ascending part and is given by:

$$ A = \frac{E_c \varepsilon_c^*}{f_c^*} $$

(2.1.24)

where $E_c$ = Tangent modulus of elasticity, given by relevant ACI formula Ref. [26] as follows:

$$ E_c = 33w^{1.5} \sqrt{f_c} $$

where, $w$ = density of concrete in lb/ft³, and $f_c$ is the compressive strength of plain concrete expressed psi. Other parameters that are used in Eqs. 2.1.22, and 23 are:
\[ K = 0.17 f_c \exp(-0.01 \sigma_t) \]  
\[ \sigma_t = \frac{2 A_s f_y}{sh \cdot d} \]

where, \( \sigma_t \) is called confinement index, \( d \) = diameter of the core, \( sh \) is the vertical spacing, \( f_y \) = yield stress of the confining reinforcement, and, \( A_s \) is the cross sectional area of the spirals or hoops.

To evaluate Eqs. 2.1.22, and, 23 the peak stress \( f_c^* \) and the corresponding strain \( \varepsilon_c^* \) must be known. The expressions for these two parameters where determined from the statistical analysis of experimental data on 76 by 152 mm concrete cylinders Ref. [27].

\[ f_c^* = f_c^* + (1.15 + \frac{3048}{f_c}) \sigma_t \]  
\[ \varepsilon_c^* = 1.027E-7 f_c^* + 0.0296 \frac{\sigma_t}{f_c} + 0.00195 \]

Comparison of the results with concrete of compressive strength 27.6 MPa (ordinary concrete) and 62 MPa (high strength concrete) indicates that the drop in load after the first peak is steeper for high strength concrete, as discussed by Fafitis and Shah, Ref. [28]. The proposed model satisfactorily predicted ultimate loads, moment, curvature, and rotations of the round and square columns subjected to cyclic loading.

The confinement index, \( \sigma_t \), which is also called the lateral pressure at the peak load can also be expressed by Eq. 2.1.29.

\[ \sigma_t = \frac{2 A_s f_y}{d} \left( \frac{1}{sh} \cdot \frac{1}{1.25d} \right) \]

where \( A_s \) and \( sh \) are the area and the spacing of spiral reinforcement, \( d \) = diameter of the confined concrete core, and \( f_y \) = yield strength of the spiral steel. Eq. 2.1.26 is an approximate form of Eq. 2.1.29.

According to the proposed calibration method by Chen and Mau Ref. [31], for the nonlinear nature of the concrete stress-strain relationship under a triaxial stress state, various equivalent confining stresses \( \sigma_t \) in combination with other factors, such as the spiral strength index \( f_y/f_c \) and the spiral spacing index \( s/d_s \), have been tried. It was found that the combination of \( \sigma_t \) with \( f_y/f_c \) gives a good correlation with the peak
stress, and predictions using the following formula have a standard deviation of 0.15 when compared to the combined 72 cases of the tests and numerical solutions.

\[
f_c^* = 1 + 0.28 \sqrt{\frac{f_y}{\sigma_r}}
\]

(2.1.30)

where:

\[f_c^* = \text{Peak average stress for confined concrete.}\]

Eq. 2.1.30, is the same as the previous relationships in this field, but, the confinement of lateral stresses (K1 in Eq. 2.1.30) is specified according to the test results and concrete constitutive law. This model has been formulated for concrete with a compressive strength \(f'_c \leq 41 \text{ MPa (6000 psi)}\).

With previous relationships for ordinary concrete, the required equations for the prediction of the stress-strain curve for high strength concrete in reinforced concrete columns have been suggested by Young, Nour, and Nawy Ref. [32]. The proposed equations were determined from the experimental results in their tests using a linear regression method. The peak stress \(f_c^*\) is equal to \(K f'_c\), where \(K\) and \(f'_c\) = the effective confinement and the concrete cylinder strength, respectively. The expression for \(K\) is in the form

\[
K = 1 + 0.0091(1 - \frac{l - 0.245sh}{h})(\rho + \frac{nd'' - \rho_s}{8 sh d'} \sqrt{f'_c} f_{yh})
\]

(2.1.31)

where \(sh\) = the centre to centre spacing of the lateral ties in inches; \(h\) = length of one side of the rectangular ties in inches; \(n\) = number of longitudinal steel bars; \(d'\) = nominal diameter of lateral ties in inches; \(r_s\) = volumetric ratio of lateral reinforcement; \(r\) = volumetric ratio of longitudinal reinforcement; and \(f_{yh}\) = yielding stress of the lateral steel in psi.

A similar relationship was originally suggested by Sargin Ref. [62], and was modified later by Valenas et al. Ref. [63]. An expression similar to that of Sargin was developed to predict the peak strain \(\varepsilon_c^*\):

\[
\varepsilon_c^* = 0.00265 + \frac{0.0035(1 - 0.734 sh)(\rho_s f_{yh})^2}{\sqrt{f'_c}}
\]

(2.1.32)
Other parameters which are necessary for prediction of the stress-strain curve, are the inflection of the stress/strain curve on the descending branch \((f_i, \varepsilon_i)\), and the stress/strain at \((f_{2i}, \varepsilon_{2i})\) an arbitrarily selected point on the descending branch where \(\varepsilon_{2i}\) is equal to \((2\varepsilon_i - \varepsilon_0)\). The expressions are in the following form:

\[ f_i = f_c^* \left( \frac{0.25(\varepsilon_c^*)}{f_c^*} + 0.4 \right) \quad (2.1.33) \]

\[ \varepsilon_i = K \left[ 1.4 \left( \frac{\varepsilon_c^*}{K} \right) + 0.0003 \right] \quad (2.1.34) \]

\[ f_{2i} = f_c^* \left[ 0.025 \left( \frac{f_c^*}{1000} \right) - 0.065 \right] \geq 0.3 f_c^* \quad (2.1.35) \]
### 2.2 HISTORICAL BACKGROUND ABOUT CIRCULAR HOLLOW SECTIONS FILLED WITH CONCRETE

Serious studies on the structural behaviour of stocky concrete filled steel tubes returns to the decade of 50's. First existing report in this field is the test by Armco Ref. [37] which indicated that the ultimate strength of concrete filled steel tube piles was dependent on a lateral restrain factor $K_p$. The ultimate load capacity, $P$ was expressed by the equation:

$$ P = A_s f_y + A_c f_c + K_p A_c $$

(2.2.1)

where:

- $A_s$ = Cross sectional area of steel
- $f_y$ = Yield stress of steel
- $A_c$ = Cross sectional area of concrete
- $f_c$ = Compressive strength of concrete

$K_p$ is a factor which depends on the dimensions and material properties of the concrete and steel tube. Later in 1964, a study on the elastic behaviour of mortar filled steel tubes was performed by Salani and Sims Ref. [38] to investigate the variation of modulus of elasticity and Poisson's ratio by change in the applied axial load on both, the steel tube and mortar. The results showed that in elastic range, the modulus of elasticity decreased when the axial load was increased. In 1967, theoretical investigations by Gardner and Jacobsen Ref. [39] revealed that for low strains, the value of Poisson's ratio for concrete is in the range of 0.15 to 0.25 but for large strains the value can even rise to approximately 0.6. At the initial stages of loading of the concrete filled steel tubes, Poisson's ratio of the concrete is lower than the steel tube, and thus the steel tube does not restrain the concrete core. The initial circumferential steel hoop stresses are compressive and the concrete is under a lateral tension. As the load is increased the lateral deformations of the concrete gets enough magnitude to change the tensile stresses between the steel tube and the concrete core to a compressive stress. Considering this deformation mechanism of the circular hollow sections filled with concrete under axial compression, the stress on the concrete at the ultimate stage of loading can be expressed as follows:

$$ s_c = f_c + K_1 s_r $$

(2.2.2)

where $K_1$ is an empirical factor, and $s_r$ = radial stress on concrete.
According to the proposed method in this paper, the ultimate load of the composite stub columns can be represented by Eq. 2.2.3.

\[ P = A_c \sigma_c + A_c K_1 \left( \frac{t}{r_c} \right) s_t + A_s s_{sl} \] (2.2.3)

where \( t \) = wall thickness of steel tube, \( r_c \) = radius of concrete core, \( s_t \) = circumferential stress on steel tube, and, \( s_{sl} \) = longitudinal stress in steel tube. The proposed equation which was originated from previous investigations, was in fair agreement with the results of the tests. In addition, they found that the recommended equations by codes of practice such as ACI and NBC for computing the allowable axial load of composite columns are very conservative. In some cases the actual factor of safety against the collapse of short lengths columns exceeds 4.5.

The experimental results by Furlong Ref. [40], that concerns the available design equations for short and long composite columns, have shown that the calculation of strength and external measurements of strain suggest that the steel and concrete sustain load somewhat independently of one another. Thus, there appeared to be little benefit to the concrete from any confinement activity of the surrounding steel. Steel walls resisted increasing amounts of stress without local buckling until nominal yield strains were observed, indicating that the concrete core stabilised the thin walled steel encasement.

The high cost of seamless steel tubing limits the use of this type of the tube in construction of composite columns. To overcome this cost disadvantage the use of spiral welded steel tube is investigated by Gardner Ref. [41]. Spiral welded steel tube costs only 40 to 50 percent of the cost of equivalent seamless tubing. However, as the material is not cold worked the yield stress is somewhat lower. Several spiral welded pipe columns were made and tested to check on the applicability of conventional design methods to this type of columns. The test results obtained by Gardner have shown that the computed allowable load by the recommended equations in NBC and ACI are conservative and the calculated ultimate load by the Eq. 2.2.4 has a margin of safety in the range of 1.2 to 1.5. It means that the beneficial effects of the confinement of the concrete is not as small as is concluded by Furlong.

\[ P = A_c \sigma_c + A_c f_y \] (2.2.4)

In another study by Furlong Ref. [42], for the design of steel-encased concrete beam columns the results of the tests on short lengths of concrete-filled steel tubes showed that adhesive bond between the steel wall and concrete core was too weak to prohibit separation or sliding at relatively low stress levels. Therefore, the effect of confinement
on the concrete core in a steel tube, under axial compression is not significant at the early stages of loading.

Knowles and Park Ref. [43] believed that the research on concrete filled steel tubular columns has often been restricted to short specimens, and this has led to the methods for calculating the ultimate loads for columns which may seriously overestimate the load carrying capacity. Accordingly, research on the concrete filled steel tubular columns has to be dealt with the behaviour of both short and long columns.

Based on the numerous research results on long column tests, there has been little evidence of an increase in concrete strength due to confinement. However, the behaviour of an axially loaded steel tube filled with concrete will vary according to the method in which the ends of the member are loaded. It was found that the average longitudinal strain at which the volume begins to increase was 2,000 micro strain (0.002) with a standard deviation of 260 micro strain, and that this increase in volume occurred at a load which was an average of 0.954 of the maximum load with a standard deviation of 0.0434. The increase of volume of the concrete was large and rapid when this stage was reached. Results of their experiments showed that in general a steel tube filled with concrete as a long composite column, and loaded axially, behaved as would be expected from classical theories of mechanics.

The sudden increase of volume at a certain value of longitudinal compressive strain, causes an internal pressure inside the steel tube which in turn causes the steel tubes to exert a confining stress on the concrete and thus increase the longitudinal compressive stress that the concrete can carry. However, it was shown that the steel tube filled with concrete failed by column buckling in long columns before the longitudinal compressive strain was sufficient to cause the concrete to begin to increase in volume in most of the cases. But evidence of an increase in concrete strength was obtained for the short circular hollow sections filled with concrete. It was found that the stage of increase in strength was difficult to estimate because the end effects affected the strain readings at the midheight of the specimens.

Although there was a considerable variation in the unconfined compressive strength of the concrete cores of the column tested, with the lowest strength at the top and the highest at the bottom, all the circular hollow sections filled with concrete failed at midheight, and the variation in concrete strength does not appear to have affected the failure mechanism. Experimental investigations by Bridge Ref. [44] have revealed that the concrete filled steel tubes have the ability to continue to carry a substantial proportion of their maximum loads for further deformation beyond that at maximum
load. This ductility, tenacity or toughness of these columns has been attributed to the concrete which stabilises the walls of the tube thus preventing local buckling failures. Bridge believes that full advantage of the tubular form of composite construction will not result until more is known about beam column connections. In spite of this and the fact that the behaviour of the isolated pin-ended column may not truly represent the behaviour of continuous columns in the building frame, an understanding of the former is necessary if methods of analysis, based on sound assumptions, are to be established from which rational design procedures may eventually be determined.

Results of theoretical and experimental investigations of Cai Ref. [22], showed that the ultimate load of short circular hollow sections, filled with concrete can be represented by Eqs. 2.2.5,6.

\[
P_u = A_c f_c (1 + \sqrt{\phi} + 1.1 \phi)
\]  

(2.2.5)

or

\[
P_u = A_c f_c (1 + 2\phi)
\]  

(2.2.6)

where:

\[
\phi = \frac{A_s f_y}{A_c f_c}
\]

The results of the Eqs. 2.2.5,6 are in a good agreement with the experimental results by the other researchers in this field Refs. [42, 22, 45-47]. The ratio of \(P_u\) from the tests to computed \(P_u\) by Eqs. 2.2.5,6 for 322 specimens ranges from 0.69 to 1.53, the arithmetic mean is 1.076 and the standard deviation 0.164. In other words, the increase in ultimate load capacity of a short filled circular hollow section under axial compression is equal to the axial ultimate load of the bare tube (Eq. 2.2.6). According to Cai’s investigations, under eccentric loading, both the longitudinal and hoop strain vary across the section of the column, and no theory is available for the inelastic deformation and strength enhancement of concrete under a non-uniform triaxial strain system.

Furthermore, the behaviour of concentric loaded composite column is significantly affected by imperfections such as initial curvature of the column, unavoidable eccentricities in the application of the load, imperfect end conditions and nonhomogeneity of the materials.
Cai also believes, to meet the needs of column design, that it is legitimate to establish a short empirical formula on the basis of adequate experimental data rather than to drive a tedious theoretical equation. The proposed formula was as follows:

\[ P_o = P_u \phi_o \]  
(2.2.7)

where:
\( P_u \) = Ultimate strength of concentric loaded short column computed by Eqs. 2.2.5 or 2.2.6.
\( \phi_o \) = Global strength reduction factor due to slenderness ratio and eccentricity ratio.
\( P_o \) = Ultimate load capacity of a circular hollow section filled with concrete, under eccentric loading.

Results of Eq. 2.2.7 are in a good agreement with the available experimental results in the literature Refs. [42, 22, 45-47].

In a separate investigation on the ultimate load behaviour of filled circular hollow sections by Kitada, Yoshida and Nakai Ref. [20], the results of experimental investigations has shown good agreement with the proposed equations by Cai. Moreover, they have concluded that in the tests where both the steel tube and encased concrete are loaded simultaneously, a composite effect can not be expected until the encased concrete is crushed due to the differences of Poisson's ratio between steel and concrete.

In one of the cases where the encased concrete is subjected to axial compression alone, the encased concrete is subject to a the triaxial stress state due to the restraint of the radial displacement by the steel tube. Consequently, not only the circumferential stress, but also axial compressive stress are induced to the middle part of the steel tube by friction between the steel tube and the encased concrete. The magnitude of the axial compressive stress of the steel tube greatly depends upon the friction between the steel tube and concrete surface. This behaviour has also been observed in the research on confined concrete which is to be presented in Chapter 4.

Kitada et al. have also concluded that the composite column loses its stiffness against the axial displacement substantially after the yielding of the steel tube. Thereafter, they can still carry the slightly additional compressive load until the axial displacement becomes considerably large.
The experimental results by Lin Ref. [48] have shown that for the concrete filled circular hollow sections under axial compression, the longitudinal strain, corresponding to the maximum stress in the confined concrete, remains constant regardless of the concrete unconfined strength, the shape of the section, and the thickness of the steel tube. The only effective parameter, is the length of the specimen, a shorter specimen yielded with a higher value of longitudinal strain.
CHAPTER THREE

EXPERIMENTAL AND THEORETICAL STUDIES ON THE ULTIMATE LOAD CAPACITY OF CHS

3.1 INTRODUCTION

The theoretical plastic axial load (squash load) for a circular cylindrical shell can be simply represented by Eq. 3.1.1, without local wall buckling.

\[ P_{sq} = A_s \cdot f_y \]  \hspace{1cm} (3.1.1)

where:

\[ A_s = \text{Cross section of the steel tube.} \]

\[ f_y = \text{Yield stress of the steel tube.} \]

\[ P_{sq} = \text{Squash load of the steel tube.} \]

Experimental results however are not always in agreement with Eq. 3.1.1 as some cases show a greater capacity than the squash load. The reasons that have widely been accepted by many researchers are the manufacturing process, such as strain hardening, material properties, geometrical imperfections, the history of stresses (residual stresses) and the influence of local buckling on section strength.

The ultimate load capacity is a function of wall thickness \((t)\), outer diameter \((d)\) and the length \((l)\) of the CHS plus material properties for the steel, the yield stress \((f_y)\), modulus of elasticity \((E)\), and Poisson's ratio \((v)\). The relationship between ultimate load and the mechanical and material properties makes use of non-dimensional parameters such as normalised relative shell buckling wall slenderness Ref. [1]:

\[ \lambda_s = \sqrt{f_y/\sigma_c} \equiv 1.286 \sqrt{(d/2)/t} \cdot (f_y/E) \]  \hspace{1cm} (3.1.2)

where \(\sigma_c = \text{local buckling stress, and the normalised ultimate axial load:} \)

\[ \overline{\sigma_u} = \frac{P_u}{P_{sq}} \]  \hspace{1cm} (3.1.3)
All available relationships for evaluation of the ultimate load of circular hollow sections are mainly based on the stability criteria such as the buckling modes in the longitudinal and the circumferential directions. According to mathematical solutions, the stability of circular hollow sections under axial compression can be investigated by some festoon curves with appropriate parameters as shown in the Fig. 3.2.2 for two of the specimens in this study Ref. [2].

The final relationship for these curves can be represented by Eq. 3.1.4.

\[ q_2 = f(l, n, m, k) \]  \hspace{1cm} (3.1.4)

where:

- \( q_2 \) = Normalised axial load on the steel tube.
- \( m \) = The number of the complete waves around the circumference of the circular hollow sections (integer).
- \( n \) = The number of the half waves along the length of the cylinder (integer).
- \( l \) = Length of the tube.

\[ k = -\frac{t^2}{12r^2} \]

and:

\[ q_2 = \frac{P}{D} = (1 - v^2) \frac{P}{E_t t} \]

\[ P = \text{Uniform load per unit length of the circumference of cylinder.} \]

\[ D = \frac{E_t}{1 - v^2} \]

where \( t \) is the wall thickness and \( r \) is the outside radius of the steel tube.

The local buckling load of the cylinder can be determined by finding the minimum \( q_2 \) for a different number of half waves along the length of the cylinder (\( n \)).

The theoretical solution for a general form of support condition becomes very difficult, but for a short circular cylindrical shell as a test specimen, Fig. 3.1.1a, the solution can be represented as an exponential function for the general solution of the differential equations. The lateral distortion of the cylinder of different stages of loading will be in a form of trigonometric and exponential functions as shown in Fig. 3.1.1b.
Fig. 3.1.1a A circular hollow section under axial compression.

Evidently this solution does not consider the non-linear behaviour of the material, and other methods may be used for this purpose. As a result, the theoretical threshold of instability in circular cylindrical shells can be estimated by the above method and compared with the experimental results Ref. [2].

The elastic-plastic behaviour of circular cylindrical shells may be investigated by theoretical and numerical methods. The theoretical approach shows a poor agreement with the experimental results, and always a margin of safety separates the design curves Ref. [1]. Consequently, the numerical method for any specific dimensions may have better results.

Fig. 3.1.1b Lateral deformation of circular hollow section under different intensities of axial compression Ref. [2].

The numerical method is basically the non-linear finite element method. This study has two main branches, material non-linearity and geometrical non-linearity. Different
methods for material non-linearity have been proposed, but the principal theory in these methods are the associated flow rule of Prandtle-Reuss and the Von-Mises yield criterion. The method is then a numerical process to reduce the errors. Some of these methods may be classified in Table 3.1.1. The last two methods are described in detail in Ref.[3].

Table 3.1.1 Some of the methods in non-linear finite element analysis.

<table>
<thead>
<tr>
<th>T.S.M</th>
<th>Tangent Stiffness Method</th>
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</thead>
<tbody>
<tr>
<td>I.S.M</td>
<td>Initial Stress Method</td>
</tr>
<tr>
<td>I.I.S.M</td>
<td>Improved Initial Stress Method</td>
</tr>
<tr>
<td>H.M</td>
<td>Hybrid Method</td>
</tr>
<tr>
<td>M</td>
<td>Modified Load Increment Method</td>
</tr>
</tbody>
</table>

Among the available software for non-linear finite elements the program, MSC/NASTRAN, is a relatively world wide software with good capability in theory and practice. The fundamental method used to obtain a static non-linear solution is to minimise the error vector of unbalanced forces acting at all grid points component \( \delta \), given by Equation 3.1.5 Ref. [4].

\[
\delta = \{P\} + \{Q\} - \{F\} \tag{3.1.5}
\]

where:

\( \{P\} = \) Vector of applied external load.

\( \{Q\} = \) force due to constrains.

\( \{F\} = \) The vector of grid point forces due to forces generated by element motion and stress. The terms are functions of displacement, temperature and stress history.

This is an iterative method where convergence is based on the load steps and the stress-strain relationship of the material.

In geometrically non-linear problems, large displacements of structural joints affects the total behaviour of the structure. MSC/NASTRAN provides static solutions for both
large displacements and material non-linearity. This program has been used for non-linear finite element analysis throughout this section.

Experimentally, seventeen axial compression tests for steel stub columns have been carried out. Dimensions for the specimens are 168.3 mm for outside diameter with the wall thickness of 4.8 and 9.53 mm. The other dimension for the steel tube is 114.3 mm for outside diameter and wall thickness equal to 6.3, and 6.0 mm. Moreover, to justify the results obtained, another specimen with the outside diameter equal to 62.0 mm and wall thickness of 2.5 mm has also been used. The steel type for all of the tube is relatively the same, with the nominal ultimate strength of 350 MPa. Furthermore, one more case of cylindrical hollow section that was coated with epoxy for better environmental protection has also been carried out in this study.

3.2 ANALYSIS OF ELASTIC STABILITY IN CIRCULAR HOLLOW SECTIONS

Considering a circular hollow section with a general form of loading, as shown in Fig. 3.2.1b, where the loads are defined as follows:

1) A uniform normal load on its wall, \( p_r = -p \).

2) An axial compression, applied at the edges, the force per unit of circumference being \( q \).

3) A shear load applied at the same edges as to produce a torque in the cylinder. The shearing forces is \( T \).

The normal pressure \( p \) produces the hoop force \( N_\phi I = p r \), The axial load produces the longitudinal force \( N_x I = -q \), and the shear load \( T \) produces shearing force, \( N_\phi x I = N_\phi x I = -T \). This is a membrane stress system, and is uniform all over the shell Fig. 3.2.1a.

The stability of circular cylindrical shells under general form of loading in elastic equilibrium can be discussed in a conventional form with differential equations but with some changes in the case of axial loading.
Fig. 3.2.1a The system of membrane stresses on a shell element.

Considering a circular cylindrical shell with a general form of loading, as shown in Fig. 3.2.1b, and use of an oblique coordinate system and skew forces for an engraved unit vectors Fig. 3.2.1c on cylindrical shell, the normal and shear forces on the sides of the element can be written as follows Ref. [2].
Fig. 3.2.1c Undeformed and deformed element with engraved unit vectors.

\[
\begin{align*}
\overline{N}_x (1+\varepsilon_x) & \cdot r \cdot d\phi \\
\overline{N}_{x\phi} (1+\varepsilon_{\phi}) & \cdot r \cdot d\phi \\
\overline{N}_\phi (1+\varepsilon_{\phi}) & \cdot dx \\
\overline{N}_{\phi x} (1+\varepsilon_x) & \cdot dx
\end{align*}
\] (3.2.1a...d)

where:

\[
\begin{align*}
N_{\phi 1} &= -p \cdot r \\
N_{x 1} &= -q \\
N_{x\phi 1} &= -T
\end{align*}
\]  

\[
\begin{align*}
\overline{N}_\phi &= N_{\phi 1} + N_\phi = -p \cdot r + N_\phi \\
\overline{N}_x &= N_{x 1} + N_x = -q + N_x \\
\overline{N}_{x\phi} &= N_{x\phi 1} + N_{x\phi} = -T + N_{x\phi} \\
\overline{N}_{\phi x} &= N_{\phi x 1} + N_{\phi x} = -T + N_{\phi x}
\end{align*}
\] (3.2.2a...d)

The N...'s are (without subscript 1) additional stress resultants from imposed additional deformation described by the displacements u, v, w.

Mathematical procedures for determining the equilibrium equations have been discussed in Ref. [2]. The method involves a relatively complicated mathematical procedure to derive the equilibrium equations in the three directions of u, v, and, w, that are the deformation vectors, in the direction of the global coordinate system. where u represents the axial deformation, v, and, w are in the tangential and radial directions, respectively. The three equation of equilibrium are given as Eqs. 3.2.3 a,b,c.
Chapter Three Experimental and Theoretical Studies on the Ultimate Load Capacity of CHS

\[
\frac{1+v}{2} u'' + v' + \frac{1-v}{2} v'' + w' + k(\frac{1-v}{2} u'' - w'' + \frac{1+v}{2} w'')
\]

\[-q_1(u'' - w') - q_2 u'' - 2q_3 u'' = 0
\]

\[
\frac{1+v}{2} u'' + v' + \frac{1-v}{2} v'' + w' + k(\frac{3}{2} (1-v) v'' - \frac{3-v}{2} w'')
\]

\[-q_1(v'' - w') - q_2 v'' - 2q_3(v'' - w') = 0
\]

\[
\nu, u + v + w + k(\frac{1-v}{2} u'' - u'' - \frac{3-v}{2} v'' + w'' + 2w'''' + w'' + 2w'' + w) + q_1 (u' - v' + w') + q_2 w'' - 2q_3 (v' - w'') = 0
\]

\[(3.2.3a,b,c)\]

where, \(\frac{\partial}{\partial x}(-) = (-)'\) and \(\frac{\partial}{\partial \phi}(-) = (-)'\).

The solution of these equations for a cylinder under axial compression was introduced in Eq. 3.2.4. The detailed form of the solution is:

\[
q_2 = ((1-v^2)\lambda^4 + k[(\lambda^2 + m^2)^4 - 2(\lambda^6 + 3\lambda^4 m^2 + 4(2-v)\lambda^4 m^2 + 2\lambda^2 (1-v) m^4)] [\lambda(\lambda^2 + m^2) + \lambda^2 m^2]^{1/2}
\]

\[(3.2.4)\]

The solution of Eq. 3.2.4 depends on the values of \(m\) (the number of the complete waves around the circumference of the circular hollow section) and \(n\) (the number of half waves along the length of the circular hollow section) plus the geometric and material specifications. To ease the solution process, Eq. 3.2.5 can be used for the definition of the parameter "\(\lambda\)" which is more convenient to use in this part.

\[
\lambda = \frac{\pi}{n \cdot r}
\]

\[(3.2.5)\]

As an example, the minimum buckling load for tubes B1 (I, II) or D1 (I, II) is about 200. The value of \(n\) can be covered by the Eq. 3.2.5 for the different values of \(\frac{1}{n \cdot r}\). For the value of \(m\) the range of 1 to 10 will cover all of the expected deformation patterns of the steel tube (Ref. 2), therefore, the minimum value of \(q2\) for different values of \(m\) will be one of the points in the final festoon curve (Ref. 2).

To avoid of tedious manual computation a computer program was developed for the whole process of this step. Detailed list of the program is in Appendix A. Furthermore, the complete flow chart of the program is also given in F. 3.2.1.
Refer to the Fig 10 (pp. 427, Ref. 2) the outcome of the program is a festoon curve that has more than one minimum. The parameter that has been used in Fig. 10 (Ref. 2) for the horizontal axis of the coordinate system is \((-\frac{1}{n.r})\) that according to the Eq. 3.5.2 has an indirect dependence to \(\lambda\).

As an example, the minimum buckling load for tubes B1 (I, II) or D1 (I, II) is about 200 for \((q_2 + 4)\). It means for a load equal to \(P = 21.1 \text{ kN/mm}\) (the force per unit of circumference), equivalent to \(\sigma = 4,395 \text{ (MPa)}\), the tube will be in an unstable position. Obviously, the steel yields before this stress and, therefore, the elastic solution is no longer valid.

In view of plastic behaviour of such a tube, according to the deformed generator of the cylinder, the bending stress will pass the yield limit eventually, Fig. 3.1.1b. As soon as this happens, the largest bulge of the cylinder will tend to be squeezed flat. Except for this, the elastic theory is still applicable and if the test is continued, the next fold will grow until it also starts to yield and is squeezed flat. This form of deformation is called an "elephant foot failure" (see details in Ref. [2])
The common characteristics of the curves in Fig. 3.2.2 is the linear behaviour at the beginning of the curve that is the characteristic of a plate under axial compression, festoon curves in the middle for local buckling of the tube with successive minimums that is resulted from the different values of $n$ and $m$, and linear part at the end for overall buckling of a tubular column.

![Fig. 3.2.2 Festoon curves for tubes with $d=168.3$ and $t=4.8,9.53$ (mm).](image)

![Fig. 3.2.3 Theoretical buckling load for a circular hollow section, under axial load only.](image)

On the other hand, the aforementioned method can be easily solved for a number of steel tube dimensions, to get a more practical solution for evaluation of $q_2$. In this regard, the Eq. 3.2.4 was solved for the range of $(d/t)$, from 0.0 to 100. As shown in
Fig. 3.2.3, the obtained coordinate for \((d/t)\) and \(q_2\) can be readily defined by an exponential equation. Regarding the proposed simplified method in Ref. [2], this solution is more practical and can simply be used in the studies of circular hollow sections.

3.3 ANALYSIS OF PLASTIC STABILITY IN CIRCULAR HOLLOW SECTION

In most engineering structures it is possible to investigate the yield behaviour by a plastic mechanism. The concept of a plastic mechanism may be applied to a simple tensile, compressive or flexural member or even a complex structure, for instance, a space frame or slabs with different materials. This method must be accompanied by a realistic prediction of the load capacity of the structure, perhaps derived from a number of collapse mechanisms. The use of the plastic mechanism in the study of circular hollow sections under axial compression is not a new concept. According to one of the latest investigations in this field Ref. [6], the plastic solution of the plastic mechanism can be restricted by two models for axially loaded CHS. Basically, these two methods are not different. Two assumptions for the folding elements are considered so that each of these two cases are specially suitable for a specific range of dimensions.

The first method, which mostly describes the mechanism in thin tubes, is based on a folding element with a transition zone. This is a representative element of the crushed zone from which the entire deformed tube can be assembled by translation, rotation and mirror reflection.

Regardless of the solution method, that uses a geometrical solution for the internal and external energy, the result furnishes an equation for the determination of the mean crushing force that is defined by the global energy balance Ref. [6].

\[
P_m \cdot 4H = E_b + E_m
\]  
(3.3.1)

where:

\(E_b\) = Bending energy.  
\(E_m\) = Membrane energy.  
\(H\) = Half of the length of a folding element.  
\(4H\) = Length of one complete cycle of folding.  
\(P_m\) = The optimum value of the mean crushing force.  
The optimum value of mean crushing force is given by:

\[
P_m = 4 \pi^3 \cdot \frac{3}{2} \cdot \int_{\frac{2\pi}{t}}^{\frac{2\pi}{t}} = 22.27 \cdot \frac{2\pi}{t}
\]  
(3.3.2)

where:
\[ M_0 = \text{Fully plastic bending moment per unit length} = \frac{1}{4} \cdot \sigma_0 \cdot t^2 \]
\[ \sigma_0 = \text{Yield stress of the steel}. \]

The maximum axial load can be represented by the following equation.
\[
\frac{P}{P_m} = 1 + \frac{\sqrt{1-m^2}}{\pi m} + \frac{\sqrt{1-m^2}}{m} \tag{3.3.3}
\]

where:
\[ m = \text{Eccentricity parameter (as shown in Fig. 3.3.1), Where the eccentricity parameter can be measured from the test results.} \]

![Fig. 3.3.1 A transition zone for linear folding elements.](image)

The second method relies on a different folding element which is shown in Fig. 3.3.2. This new plastic mechanism for a tube, under axial compression, represents a thick walled tube undergoing local folding after yielding point, and has a curved shape. Accordingly, the Eq. 3.3.2 can be reformed to the Eq. 3.3.4.
\[
\frac{P_m}{M_0} = 31.74 \sqrt{\frac{2t}{r}} \tag{3.3.4}
\]

And the instantaneous crushing force, normalised with respect to the mean crushing force is as follows:
\[
\frac{P}{P_m} = f(\alpha) \tag{3.3.5}
\]

(refer to the Ref. [6] for more details of Eq. 3.3.5)
According to the reported experimental results, the maximum value of the peak loads in a CHS has a descending rate throughout a number of loading cycles. As the edges of the tube are completely constrained at the first cycle of loading, the eccentricity is at its maximum value. Therefore, the tubes under axial compression show a maximum load capacity at the beginning of the loading. Rationally, the load capacity of the tube will approach to a limiting value of the eccentricity after a number of squeezed cycles.

The following study concerns the load capacity at the first stage of loading (first peak load), therefore, the continuation of the loading process in the case of a perfect folding mechanism is not of interest in this research. Moreover, the eccentricity of the folding elements cannot be specifically determined by this analytical method Ref. [6]. However, the main conclusion from this part of the analysis is that the mean crushing force does not depend on the eccentricity parameter m. Furthermore, the mathematical relationship for the load-axial shortening of the steel circular hollow section can be calibrated by a number of test results for any specific dimension.

![Diagram of the transition zone with two curved folding elements.](image)

Fig. 3.3.2 The transition zone with two curved folding elements.

To illustrate the theoretical results for the first folding element (straight-element model) model, Fig. 3.3.3 representing two complete cycles of a load-deflection curve for three different values of m. This graph is normalised by the mean crushing force $P_m$ and the length of a complete cycle of loading $\delta_{eff}$. A similar diagram can be drawn for the second folding element (curved-element model).
Chapter Three Experimental and Theoretical Studies on the Ultimate Load Capacity of CHS

Fig. 3.3.3 A load displacement diagram of a straight-element model for three values of the eccentricity.

Fig. 3.3.4 Experimental static axial load versus axial shortening for three complete periods

Although the theory of the straight folding elements for studying the ultimate load behaviour of a circular hollow section uses a simplified plastic mechanism to obtain the above mentioned relationships, nevertheless the experimental results are in a good agreement with the theory. Regardless of the plastic deformation at the ultimate load for a specimen with outside diameter equal to 62.5 (mm) and wall thickness equal to 2.5 (mm), Fig. 3.3.4, the loading cycle and the difference between periodical ultimate loads show good agreement with the assumed model Ref. [6].
3.4 THEORETICAL METHOD FOR DETERMINATION OF STRESSES FROM MEASURED PLANAR STRAINS IN ELASTIC AND ELASTIC-PLASTIC RANGE

The response of material is more complicated in the elastic-plastic range than the elastic range and often anisotropy is a significant parameter. Accordingly, the solution for the determination of stresses from measured strains in the elastic-plastic range relies on an iterative method.

The principal concepts in the iterative numerical method are the yield criterion, the associated flow rule, and the iterative process. Two methods are introduced at this time. The first method is proposed by Komatsu and Kitada Ref. [3], and the second one is discussed by Sharp Ref. [8]. The methods are basically similar and the main concepts rely on the previous work by Theocaris Ref. [9], Gallagher et al. Ref. [10] and, others.

For a discussion on the determination of stresses from measured strains, it is necessary to use the most suitable yield criterion. Amongst the numerous proposed criteria for yielding loci of ductile materials, the methods of Tresca and Von-Mises are in better agreement with experiments than those by other methods, and also are in a simple form. In addition, that these two criteria are also in a fair agreement with each other. Moreover, according to the nature of the problem and necessity of simplicity for the equations, one of the methods may be preferred for use. For instance, the explicit form of the Von-Mises yield criterion, and better compatibility (Taylor and Quinney Ref. [11]) make it more suitable for the following studies. However, in the following sections, these two criteria are discussed in detail.

3.4.1 Tresca Yield Criterion

The Tresca yield criterion of maximum shear theory assumes that yielding will occur when the maximum shear stress reaches the value of maximum shear stress occurring under simple tension. The maximum shear stress is equal to half the difference between the maximum and minimum principal stresses. Therefore, since \( \sigma_1 = \sigma_2 = 0 \), under simple tension the maximum shear stress at yield is \( \sigma_1/2 \). Accordingly, yield will occur when any one of six following conditions is reached:
\[ \sigma_1 - \sigma_2 = \pm \sigma_0 \]
\[ \sigma_2 - \sigma_3 = \pm \sigma_0 \]
\[ \sigma_3 - \sigma_1 = \pm \sigma_0 \] (3.4.1.1a,b,c)

These equations represent a cylindrical shape with a hexagonal cross-section, along the normal axis to the \( \pi \)-plane \((\sigma_1 + \sigma_2 + \sigma_3 = 0)\) from the centre of coordinates in a \( \sigma_1, \sigma_2, \sigma_3 \) coordinate system. The cross-section of this cylinder with the plane of \((\sigma_1 - \sigma_2)\) is shown in Fig. 3.4.1.1.

3.4.2 Distortion Energy Theory, Or Von-Mises Yield Criterion

Similar to the theory of maximum strain energy, this theory assumes that if the distortion energy equals to the distortion energy at yield in simple tension, then yielding will begin. Therefore, after some calculation for determining the energy, the results can be shown in Equation (3.4.2.1).

\[ \frac{1}{2}[\{(\sigma_1 - \sigma_2)^2 + (\sigma_2 - \sigma_3)^2 + (\sigma_3 - \sigma_1)^2\}]= \sigma_0^2 \] (3.4.2.1)

This equation again represents a circular cylinder along the normal to the \( \pi \)-plane and the Von-Mises circle (cross-section of cylinder) circumscribes the Tresca hexagon.

The yield criterion of Von-Mises represents an ellipse in the biaxial plane which is shown in Fig. 3.4.2.2. The mathematical equation for the biaxial case is represented by Equation (3.4.2.2) for principal stresses.

\[ \sigma_1^2 + \sigma_2^2 - \sigma_1 \sigma_2 = \sigma_0^2 \] (3.4.2.2)
When considering the difference in compression and tension yield capacity of a material, a modified Von-Mises criterion is also suggested in the following equation Ref. [12].

\[
\begin{align*}
(\sigma_1 - \sigma_2)^2 + (\sigma_2 - \sigma_3)^2 + (\sigma_3 - \sigma_1)^2 + 2(C-T)(\sigma_1 + \sigma_2 + \sigma_3) &= 2CT
\end{align*}
\]

where, \( T, C = \) Yield strength in uniaxial tension and compression respectively.

According to Raghava et al. Ref. [12], this criterion was originally proposed by Schleicher in 1926, and was suggested independently by Stassi-D'Alia in 1967 Ref. [13]. The Von-Mises yield criterion was also modified for anisotropic behaviour of materials by Hill Ref. [14] and Theocaris Ref. [15]. Anisotropic behaviour of steel has no significant effect on the ultimate load capacity of CHS, therefore it will no longer be considered in this study Ref. [1].

### 3.4.3 Stress-Strain Relationship

Whereas the strains are linearly related to the stresses by Hooke's law in the elastic range, it is not linear in the elastic-plastic and plastic ranges of materials. Furthermore, a more complicated distinction between elastic and plastic stress-strain relationships is the uniqueness of stresses for every strain in the elastic range, but not in the plastic range. Generally, the stresses are not uniquely determined by the strains and depend on the whole history of loading, in other words how the stress state is reached Ref. [16].

To find a method for determining the stresses from measured strains it is required to find a relationship between plastic strains and stresses. In this view the equations of Prandtle-Reuss would be the basis of the further action. Reuss assumed that the plastic strain increment is, at any instance of loading, proportional to the instantaneous stress deviation i.e;
where $S_{ij}$ is the stress deviator tensor and $d\lambda$ is a non negative constant which may vary throughout the loading history. In these equations the total strain increments are assumed to be equal to the plastic strain increments, the elastic strain being ignored. Thus these equation can only be applied to problems of large plastic flow and cannot be used in the elastoplastic range. Moreover, these equations state that the increments of plastic strain depends on the current values of the deviatoric stress state. It is also implies that the principal axes of stress and plastic strain increment tensors coincide. The Eq.3.4.3.1 is also defined in principal directions as follows:

\[
\frac{d\varepsilon^p}{S_1} = \frac{d\varepsilon^p}{S_2} = \frac{d\varepsilon^p}{S_3} = d\lambda
\]  

or

\[
d\varepsilon^p_k = S_k d\lambda \quad k=1,2,3
\]  

or

\[
\frac{d\varepsilon^p_1 - d\varepsilon^p_2}{S_1 - S_2} = \frac{d\varepsilon^p_2 - d\varepsilon^p_3}{S_2 - S_3} = \frac{d\varepsilon^p_3 - d\varepsilon^p_1}{S_3 - S_1} = d\lambda
\]  

By use of Equation (3.4.3.1) and the yield criterion of Von-Mises, $d\lambda$ can be readily determined.

\[
d\lambda = \frac{3}{2 \sigma_e} \frac{d\varepsilon^p}{m}
\]  

It is convenient to define an equivalent or effective stress and an equivalent or effective plastic strain increment as:

\[
\sigma_e = \left(\frac{3}{2} S_{ij} S_{ij}\right)^{\frac{1}{2}} = \sqrt{\frac{3}{2} \left( S_1^2 + S_2^2 + S_3^2 \right)} = \frac{3}{\sqrt{2}} \tau_{oct} = \sqrt{3T_2}
\]

\[
d\varepsilon_p = \frac{\sqrt{3}}{3} \left[ \left( d\varepsilon^p_{xy} - d\varepsilon^p_{yx} \right)^2 + \left( d\varepsilon^p_{yz} - d\varepsilon^p_{zy} \right)^2 + \left( d\varepsilon^p_{zx} - d\varepsilon^p_{xz} \right)^2 + 6(d\varepsilon^p_{xy})^2 + 6(d\varepsilon^p_{yz})^2 + 6(d\varepsilon^p_{zx})^2 \right]
\]
\[ \tau_{oct} = \frac{1}{3} \left\{ \left( \sigma_1 - \sigma_2 \right)^2 + \left( \sigma_2 - \sigma_3 \right)^2 + \left( \sigma_3 - \sigma_1 \right)^2 \right\}^{1/2} \]  
\hspace{1cm} (3.4.3.6_{a,b,c})

and \( J_2 \) is the second invariant of the stress deviator tensor. Considering Eq. 3.4.3.7 for the stress deviator tensor:

\[ S_{ij} = \sigma_{ij} - \sigma_m \delta_{ij} \]  
\hspace{1cm} (3.4.3.7)

where:

\[ \delta_{ij} = \text{The simplest second order symmetric tensor (Kronecker delta) or substitution tensor, defined by:} \]

\[ \delta_{ij} = 0 \text{ IF } i \neq j \]
\[ \delta_{ij} = 1 \text{ IF } i = j \]
\[ \delta_{ij} = \begin{bmatrix} 1 & 0 \\ 0 & 1 \end{bmatrix} \]

The stress-strain relations may be determined as follows:

\[ \frac{\Delta \varepsilon_x}{\sigma} = \frac{\Delta \varepsilon_y}{\sigma} = \frac{\Delta \varepsilon_z}{\sigma} = \frac{\Delta \varepsilon_{xy}}{2 \tau_{xy}} = \frac{\Delta \varepsilon_{yz}}{2 \tau_{yz}} = \frac{\Delta \varepsilon_{zx}}{2 \tau_{zx}} = \frac{3}{2} \frac{\Delta \varepsilon_{ij}}{\sigma} S_{ij} \]  
\hspace{1cm} (3.4.3.8a,...,g)

Accordingly, the increments of stresses from increments of strains can be shown as:

\[ \Delta \sigma_{ij} = 2 \mu \Delta \varepsilon_{ij} + \lambda \delta_{ij} \Delta \varepsilon_{kk} - F_{ij} S_{mn} \Delta \varepsilon_{mn} \]
As is mentioned in Ref. [8], the parameters are:

\[
F = \frac{18\mu^2}{(2H' + 6\mu)\sigma_e^2}
\]  
(3.4.3.9)

and

\[
\mu = \frac{E}{2(1+\nu)}
\]

\[
\lambda = \frac{Ev}{[(1+\nu)(1-2\nu)]}
\]

\[H' = \text{The slope of stress versus plastic strain curve in uniaxial tension.}\]

In experimental procedures, strains are usually measured on the surface of structural members, therefore, one of the stresses (the stress normal to the surface \(\sigma_3 = 0\)) is zero. In the case of axial compression on circular hollow sections, the stress in the radial direction is approximately zero. If one measures two of the three strains, then three equations are needed to be determined for the three unknowns Ref. [8]. Subsequently, Eq. 3.4.3.9 reduce to:

\[
de_3 = \frac{(FS_1S_3 - \lambda)de_1 + (FS_2S_3 - \lambda)de_2}{\lambda + 2\mu - FS_3^2}
\]

\[
d\sigma_1 = (\lambda + 2\mu - FS_1^2)de_1 + (\lambda - FS_1S_2)de_2 + (\lambda - FS_1S_3)de_3
\]

\[
d\sigma_2 = (\lambda - FS_1S_2)de_1 + (\lambda + 2\mu - FS_2^2)de_2 + (\lambda - FS_2S_3)de_3
\]  
(3.4.3.10_a,b,c)

The relationship of the stresses and strains in the elastic-plastic range may also be written in matrix notation. Considering the following presentation for the plastic flow rule:

\[
de_p = \frac{\partial F}{\partial \sigma} d\lambda
\]  
(3.4.3.11)

where:

\[
\sigma = \begin{bmatrix} \sigma_{ij} \end{bmatrix}
\]

\(F = \text{Von-Mises yield criterion.}\)

\(de_p = \text{Incremental plastic strain vector.}\)
The non-negative scalar $d\lambda$ becomes [3]:

$$d\lambda = \left[H' + \left(\frac{\partial F}{\partial \sigma}\right)^T D_e \left(\frac{\partial F}{\partial \sigma}\right)\right]^{-1} \left(\frac{\partial F}{\partial \sigma}\right)^T D_e \left(\frac{\partial F}{\partial \sigma}\right) d\varepsilon$$

(3.4.3.12)

where:

$$D_e = \frac{E}{1 - \nu^2} \begin{bmatrix} 1 & \nu & 0 \\ \nu & 1 & 0 \\ 0 & 0 & \frac{1 - \nu}{2} \end{bmatrix}$$

(3.4.3.13)

$d\varepsilon$ = Incremental strain vector.

If a structure is in an elastoplastic state, the incremental strain vector $d\varepsilon$ at any point may be considered as the sum of an incremental elastic strain vector $d\varepsilon_e$ and an incremental plastic strain vector $d\varepsilon_p$ as follows:

$$d\varepsilon = d\varepsilon_e + d\varepsilon_p$$

(3.4.3.14)

For the incremental elastic strain $d\varepsilon_e$:

$$d\varepsilon_e = D_e^1 \cdot d\sigma$$

(3.4.3.15)

Substituting Eq. 3.4.3.11, and Eq. 3.4.3.14 yields:

$$d\sigma = D_e (d\varepsilon - \frac{\partial F}{\partial \sigma} d\lambda)$$

(3.4.3.16)

Considering $d\varepsilon_p = d\lambda$, and substituting Eq. 3.4.3.12 into Eq. 3.4.3.16, the incremental stress vector is then given by:

$$d\sigma = D_{ep} d\varepsilon$$

(3.4.3.17)

Where $D_{ep}$ is a plastic stress-strain matrix.

$$D_{ep} = D_e - D_p$$

(3.4.3.18)
and:

$$D_p = \frac{D_e \begin{bmatrix} \frac{\partial F}{\partial \sigma} \\ \frac{\partial F}{\partial \sigma}^T \end{bmatrix} D_e}{H' + \begin{bmatrix} \frac{\partial F}{\partial \sigma}^T \\ \frac{\partial F}{\partial \sigma} \end{bmatrix} D_e \begin{bmatrix} \frac{\partial F}{\partial \sigma} \\ \frac{\partial F}{\partial \sigma}^T \end{bmatrix}}$$

(3.4.3.19)

More details can be found in Ref. [3,10].

For a tube with isotropic hardening material under axial compression, the circumferential stress along the height of the tube is zero except for the end supports. Therefore, for a point far enough from the supports the stresses are in longitudinal direction only. On the other hand the behaviour of steel in a tube is affected by the manufacturing process and use of the results of a tensile coupon test will not reveal all the characteristics of a steel tube. Considering the characteristics in compression for the stress-strain curves of the steel tube the results of these tests can be used for computing the value of $H'$. In both of the above mentioned methods (Eqs. 3.4.3.10a,b,c and Eq. 3.4.3.19), it is necessary to obtain stresses as a function of plastic strain in order to compute $H'$ for uniaxial test results. This can be achieved by subtracting the elastic strain $\sigma/E$, from the total strain. With use the test results, the aim is to get a smooth fit through the data that will give a reasonable representation of the plastic behaviour of the material. In this view a Ramberg-Osgood relationship, where $K$ and $N$ describe the plastic strain, is used (Ref. [8]). The values of $K$ and $N$ can be computed according to results obtained from the uniaxial compression tests for each specimen (Eq. 3.4.3.11). These values are computed by (SAS) software and are as follows (Table 3.4.1):

Table 3.4.1 Constant values in Ramberg-Osgood equation for each type of test specimen.

<table>
<thead>
<tr>
<th>Test No.</th>
<th>K</th>
<th>N</th>
</tr>
</thead>
<tbody>
<tr>
<td>A1- I,II</td>
<td>959.9</td>
<td>.114</td>
</tr>
<tr>
<td>B1- I,II</td>
<td>591.3</td>
<td>.064</td>
</tr>
<tr>
<td>C1- I,II</td>
<td>1277.8</td>
<td>.055</td>
</tr>
<tr>
<td>D1- I,II</td>
<td>607.8</td>
<td>.048</td>
</tr>
</tbody>
</table>
\[ \varepsilon = \frac{\sigma}{E} + \left(\frac{\sigma}{K}\right)^{1/N} \]  \hspace{1cm} (3.4.3.11)

Although both of the above mentioned methods (Eqs. 3.4.3.10a,b,c and Eq. 3.4.3.19) for computing the stresses from measured strains are suitable to use for the following study, the first method was used, because of its simplicity for programming. To determine the stresses for each test a computer program was developed, the listing of which is given in Appendix C.

3.5 TEST ARRANGEMENTS AND PROCEDURES

3.5.1 Experimental Concepts and Specifications

As higher strength steel for structural purposes may be more economic than mild steel, it is sometimes preferred in use as a principal material. In this view, most available structural circular hollow sections are made up of relatively high strength steel. Regarding the purpose of the use, the manufacturing process may be different. As an example, for a member with high internal pressure, a seamless tube would be more suitable than one fabricated by the electrical welding resistance method.

This study concerns on the mechanical behaviour of short cylinders (stocky columns) under a pure axial compression. The dimensions are chosen to cover a relative wall slenderness from d/t=18 to 35, with two specimens for minimum slenderness and two for maximum slenderness. The two specimens for maximum slenderness are only different in their manufacturing method. The specimens D1-I,II are epoxy coated so as to provide extra environmental protection. Therefore, these tubes are able to perform in corrosive environments.

The specimens of minimum slenderness are A1-I,II and C1-I,II; the dimensions for wall thicknesses and diameters are different but the manufacturing method and the material specifications are similar. Structural differences between these two specimens, perhaps, may be expected due to residual stresses during the process of manufacturing. Full details of the specimens are presented in the Table 3.5.1.
The length of specimens must be long enough to avoid the effects of the ends. The specimens are loaded by the supports of the testing machine at both ends. The undistorted height of the specimens is determined according to a general numerical analysis by a non-linear finite element method. According to the observed rate for slope of the stress-strain curve for steel and the recommended curves in Ref. [5], so material properties are introduced to the finite element program.

Table 3.5.1 Specification of the test specimen.

<table>
<thead>
<tr>
<th>No.</th>
<th>Specimen</th>
<th>Outside Diameter d(mm)</th>
<th>Wall Thickness t(mm)</th>
<th>Length l(mm)</th>
<th>d/t</th>
<th>Grade</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>A1-I</td>
<td>114.3</td>
<td>6.30</td>
<td>500</td>
<td>18.14</td>
<td>C350</td>
</tr>
<tr>
<td>2</td>
<td>A1-II</td>
<td>114.3</td>
<td>6.30</td>
<td>500</td>
<td>18.14</td>
<td>C350</td>
</tr>
<tr>
<td>3</td>
<td>B1-I</td>
<td>168.3</td>
<td>4.80</td>
<td>500</td>
<td>35.06</td>
<td>C350</td>
</tr>
<tr>
<td>4</td>
<td>B1-II</td>
<td>168.3</td>
<td>4.80</td>
<td>500</td>
<td>35.06</td>
<td>C350</td>
</tr>
<tr>
<td>5</td>
<td>C1-I</td>
<td>168.3</td>
<td>9.53</td>
<td>500</td>
<td>17.66</td>
<td>C350</td>
</tr>
<tr>
<td>6</td>
<td>C1-II</td>
<td>168.3</td>
<td>9.53</td>
<td>500</td>
<td>17.66</td>
<td>C350</td>
</tr>
<tr>
<td>7</td>
<td>D1-I</td>
<td>168.3</td>
<td>4.80</td>
<td>500</td>
<td>35.06</td>
<td>C350</td>
</tr>
<tr>
<td>8</td>
<td>D1-II</td>
<td>168.3</td>
<td>4.80</td>
<td>500</td>
<td>35.06</td>
<td>C350</td>
</tr>
</tbody>
</table>

~ = Epoxy coated tube.

Fig. 3.5.1.1 Lateral deformation in three load steps.
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Results of lateral deflection for a tube with \( d=168.3 \) and \( t=4.8 \) (mm) show that even at complete yielding there is a non-distorted region at the middle of the model. In this region, lateral deflection has an equal magnitude throughout the non-distorted height of the specimen. The lateral deflection for three steps of loading are shown in Fig. 3.5.1.1 these three steps of loading are in the elastic region, elastic-plastic region and finally the strain hardening region. The load steps (L. S.) at the top of the Fig. 3.5.1.1 denote the axial deformation applied to the analytical model.

3.5.2 Testing Method and Instrumentation

Considering the grade of steel, the expected capacity for the heaviest specimens (C1-I,II) is about 170 tons for proof stress and more than 250 tons for complete yielding. The strain hardening of material and the geometry of the section have a considerable effect on the yielding capacity. Accordingly, a hydraulic testing machine with a maximum capacity of 5000 (kN) for axial force and maximum clearance of 1100 (mm) has been considered for these series of tests. The axial force and longitudinal deflection are measured with transducers, and the load deflection curves are drawn with a pen plotter.

![Two strain gages with perpendicular directions.](image)

Fig. 3.5.2.1 Position of strain the gauges on the specimens.

For obtaining the exact material properties, such as modulus of elasticity and Poisson's ratio, step by step information for stresses and strains is required. In this view the specimens A1-II, B1-II, C1-II,D1-II are tested with high elongation strain gauges spaced at 90° along the tube circumference. The number of strain gauges used for each specimen is three double strain gauges, one in the longitudinal direction and the other in
the transverse direction. Each pair is the circumferential direction, as shown in Fig. 3.5.2.1. The strain gauges are located at the midheight of the specimens.

Reading the strain gauges and also applied loads is a time consuming process, and, more importantly, may effect the total behaviour of the specimens. Therefore, for reading the strain gauges a computerised data acquisition system was used. A diagram of the testing instrumentation is shown in Fig. 3.5.2.2.

![Instrumentation diagram and test layout.](image)

**3.5.3 Improvement of the Test Results**

The results of experimental tests are divided into two principal groups. In the first group, for each of the tests, one plot represents the load-deflection curve. This load deflection curve can be simply changed to the stress-strain and even normalised with respect to the yield stress or any other stress basis. In this method a scanner is used to transfer the original graph to a computer file. The change of scales in both axes to achieve the desired curve can be carried out by appropriate software. In the process of scanning, each individual point directly transfers from scanner to the computer and no distortion or change in position occurs. The accuracy of this process is 300 points in a square inch. Therefore, there is no effect such as scatter of shape that normally happens in an ordinary copying machine. The amendment of the scanned graphs consists of some minor corrections for probable spots in the paper and to straighten some non-uniformity in thickness of lines with a small correction for better appearance.

In order to obtain a better sense for different behaviour of the axially compressed tubes, the graphs must be combined with each other in a proper manner. The combination of graphs is also accomplished by computer. During the process of combination, the
orientation of the graphs are not changed and the starting points of the load deflection curves are accurately matched.

In addition to the graphs for load-deflection curves, there are also results for stress-strains in specimens which have strain gauges. The results from strain gauges readings should not be different for one specimen in different longitudinal or circumferential positions of the strain gauges, but the results do not show the equality of strains for each position of strain gauges. As is shown in Fig. 3.5.2.1 the strain gauges are located in circumference of the tubes separated by an angular distance of 90°. In each location of strain gauges two strain gauges are installed with perpendicular directions.

Although all tubes are accurately squared before testing in compression, nevertheless, small tolerances in dimensions are inevitable that may make considerable differences in the results of strains for different locations. The main reason for the differences of strains in the circumferential direction is the effect of non-uniform touch of the testing machine support with the ends of the specimens. Considering an exaggerated sketch which is given in Fig. 3.5.3.1, if the bottom end accurately sit on the bottom support, the top end of the specimen may not touch the top support in an appropriate form. Most of the inaccuracy will be compensated by the rotation of the top support (Fig. 3.5.2.2), but, as the results show, it cannot be completely removed.

![Fig. 3.5.3.1 Sketch for the general position of the test specimen.](image)

Hence, non-uniform strain will be developed in the circumferential direction that should be considered for computation of average strain in the test results.

The actual distribution of strains in the circumferential direction of the tubes depends on the position of contact between the edges of the tubes and the support faces. To
simplify the issue, a linear distribution of strain from a maximum value to a minimum value in circumferential direction may be considered. Maximum value is in the Point A (Fig. 3.5.3.1), and the minimum value is in the point B. Fig. 3.5.3.2 shows the circumference of the tube in a developed position.

Fig. 3.5.3.2 The distribution of strain along the circumference of the cross section.

The strains obtained from the test results are in a set of three strains in the longitudinal direction and three strain in the transverse direction. Considering the middle strain gauge as STRM, the right strain gauge as STRR and the left strain gauge as STRM. The left slope (SLR) and the right slope (SLL) can be shown as Fig. 3.5.3.3.

where:

\[ SLL = \frac{STRM - STRL}{P/4} \]  \hspace{1cm} (3.5.3.1)

\[ SLR = \frac{STRM - STRR}{P/4} \]  \hspace{1cm} (3.5.3.2)

P = Perimeter of the steel tube.

The possible numerical relationship between the two introduced slopes can readily be written as follows:

a) \( SLR > 0 \) AND \( SLL \leq 0 \)
b) \( SLR > 0 \) AND \( SLL > 0 \)
c) \( SLR \leq 0 \) AND \( SLL \leq 0 \)
d) SLR < 0 AND SLL > 0

The next parameter which is important is the absolute value of the right and left slope. In this regard the following equations can be written for the different situations of the absolute slopes.

1) ABS(SLR) > ABS(SLL)
2) ABS(SLR) < ABS(SLL)
3) ABS(SLR) = ABS(SLL)
4) ABS(SLR) = 0 OR ABS(SLL) = 0

Considering the previous proportion of the slopes in Eqs. 3.5.3.3-6 and Eqs. 3.5.3.7-10, the total cases of the positions of strains, would be sixteen independent cases.

As an example, case (c) represents an ascending slope at the left hand side of the middle point (or zero slop) and an ascending slope at the right hand side of the middle point. Considering the second set of cases, for instance, case (2), if the absolute value of the right slope is less than the slope of the left hand side, the geometric representation of this case can be given as Fig. 3.5.3.4.

From the geometry consideration, the steeper slope is the slope of the strain distribution throughout the circumference of the steel tube and the magnitude of the maximum strain can be computed by intersecting the line with the slope of SLL and a line with the slope of SLR from the point STRR.
According to this method for computing the average strain in a tube under compressive axial force, a computer program was developed to compute the average strain from the measured strains during the tests. The list of the computer program which is in Fortran language, is given in Appendix B.

To justify the proposed averaging method, four separate tests with use of four set of strain gages in each test was carried out. The method of testing was exactly the same as the main specimens in this study, and the specimens were similar to the tests A1(I,II) and C1(I,II). The position of strain gauges are shown in Fig. 3.5.3.5. Two series of data for computing according to the proposed average method was considered (series A and B). Moreover, the total results for strain gauges in a series of four data for longitudinal strain was also considered.

The results of all of the tests are more or less similar, therefore, one of the specimens (A1(I,II)) will be discussed in detail. The obtained average for two series of data (A: N, W, and, S strain gauges) and, (B: S, E, and, N strain gauges), and the total average from four strain gages (N, S, W, and, E) are drawn versus the measured stresses in Fig. 3.5.3.6. The two stress-strain curves, computed by the proposed average method are in a good agreement with the direct average from the series of four data. Therefore, the method is acceptable for use in the further studies.
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Average stress-strain curve from 4 strain gauges.
Average stress-strain curve from 3 strain gauges (A).
Average stress-strain curve from 3 strain gauges (B).

Fig. 3.5.3.6 Stress-strain curve for specimen A1 III by two average values, A, B, and direct average.

For a more accurate comparison of the results, the summarised tabulated averages of four strain gauges are shown in Table 3.5.3.1. These results show that the average for series A and B are exactly the same, therefore, the position of the strain results for any set of strain gauges (A or B) has no effect in the averaging method.

Table 3.5.3.1 Strains in longitudinal direction and average results for the specimen A1 III.

<table>
<thead>
<tr>
<th>Steps</th>
<th>E. Vert.</th>
<th>W. Vert.</th>
<th>N. Vert.</th>
<th>S. Vert.</th>
<th>Average (4)</th>
<th>Average (A)</th>
<th>Average (B)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>186</td>
<td>12</td>
<td>82</td>
<td>105</td>
<td>96.25</td>
<td>93.5</td>
<td>93.5</td>
</tr>
<tr>
<td>2</td>
<td>306</td>
<td>75</td>
<td>190</td>
<td>220</td>
<td>197.75</td>
<td>205</td>
<td>205</td>
</tr>
<tr>
<td>3</td>
<td>415</td>
<td>175</td>
<td>299</td>
<td>337</td>
<td>306.5</td>
<td>318</td>
<td>318</td>
</tr>
<tr>
<td>4</td>
<td>519</td>
<td>276</td>
<td>407</td>
<td>453</td>
<td>413.75</td>
<td>430</td>
<td>430</td>
</tr>
<tr>
<td>...</td>
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<td>...</td>
<td>...</td>
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<td>...</td>
<td>...</td>
<td>...</td>
</tr>
<tr>
<td>12</td>
<td>1618</td>
<td>1376</td>
<td>1623</td>
<td>1703</td>
<td>1580</td>
<td>1663</td>
<td>1663</td>
</tr>
<tr>
<td>14</td>
<td>2336</td>
<td>2119</td>
<td>2439</td>
<td>2496</td>
<td>2347.5</td>
<td>2467.5</td>
<td>2467.5</td>
</tr>
</tbody>
</table>

where:
E. Vert = Longitudinal strain in the strain gauges, located at the eastern side of the specimen. Similar definition for W, N, and S.
Average (4) = Average value from the results of four strain gauges.
Average (A) or (B) = average from the strain gauges in series (A) or (B).
3.5.4 Evaluation of the Stress-Strain Results

The secondary effects on the results obtained may cause some non-realistic or perhaps incorrect conclusions and have to be recognised. Furthermore, the possible errors should also be evaluated and eliminated from the raw results. The most probable errors can be listed as follows:

- Non-uniform contact between the tube ends and supports of the testing machine.
- Error in recorded values for strain gauges and load readings.
- Dimensions of the cross sections.
- Bulge form behaviour of a compressed tube specially for thick walled tubes, due to friction between the stub and machine platens.

Referring to the 3.5.3, the possible inaccuracy in squareness of the tubes can be overcome by finding the average of strains throughout the height of the specimens. This average of the strains represents the characteristic behaviour of a tube under axial compression. The stress-strain curve for the three different positions of the strain gauges and the average curve for one of the specimens (B1-II) are drawn in Fig. 3.5.4.1. It is clearly recognisable from the Fig. 3.5.4.1 that the position of average strain is always constant at the elastic and elastic-plastic stages of loading.

![Stress versus strain curve for different positions of strain gauges and average values of strains.](image-url)
In addition, the effect of thickness variation throughout the circumference of the section is also corrected in this method. The variation of thickness throughout the circumference causes non-uniform distribution of stresses which is similar to the non-squareness of the section and can be superposed to it. Fortunately, this is not a significant problem for thin walled tubes, but for thick-walled tubes, the manufacturing process is such that some variation in wall thickness throughout the circumference is created, that may reach a maximum value in the seam area. However, the variation is linear and compatible with non-squareness of the sections. Fig. 3.5.4.2 shows the cross section of a thick walled tube with the nominal diameter of 168.3 mm and wall thickness of 4.8 mm

3.6 TEST RESULTS AND DISCUSSION

3.6.1 Load-Deflection Behaviour

The compression tests for all eight bare tube specimens have been carried out by the same method. The procedure is mainly based on a strain control method the quasi-static state for all tests. The strain rate has been constant throughout the tests for all specimens (0.0020-0.0025 ((mm/mm)/min)). The load-deflection curves for four specimens are shown in Fig. 3.6.1.1. As mentioned in Table 3.2.1, for the total number of the tests in the experimental program, there are four pairs of specimens, and the dimensions are the same for each pair. The test results show a similar behaviour for
each pair of specimens, therefore, only one of the curves for each pair are shown in Fig. 3.6.1.1.

The ductile behaviour of thick walled tubes (d/t = 35) is considerably better than that of thin walled tubes (d/t = 18) that the reason lies in the plastic mechanism for thick walled tubes. In thick walled tubes the plastic region in the locally buckled ends is wider than that for the thin walled tubes. Consequently, the folding elements as discussed in 3.3 tend to be folding curved elements. In this case, the coefficient for computing the maximum axial load capacity, Eq. 3.3.4, is larger than the coefficient of Eq. 3.3.2 which represents the straight folding elements. Moreover, regarding the form of the plastic mechanism, the plastic region in the tube's shell is longer for thick walled sections. The differences between plastic mechanisms may also be recognised in the photo 3.6.1.1 which shows the deformed specimens after testing. Specimen C1-I longitudinally deformed more than the rest of the specimens, but the buckled end was not folded like the rest of the specimens.

![Fig. 3.6.1.1 Load deflection curves for steel CHS stub column tests.](image-url)
The results of these tests are shown in Tables 3.6.1.1\textsubscript{a,b,c}. The table is prepared so that to be comparable with the available results in this field and contains complete geometrical specifications, yield and ultimate parameters.

Table 3.6.1.1\textsubscript{a,b,c} Geometrical specifications and the test results.

(a)

<table>
<thead>
<tr>
<th>Test No.</th>
<th>Geometrical specification</th>
<th>Classic structural parameters</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>d (mm)</td>
<td>t (mm)</td>
</tr>
<tr>
<td>A1(I,II)</td>
<td>114.3</td>
<td>6.3</td>
</tr>
<tr>
<td>B2(I,II)</td>
<td>168.3</td>
<td>4.8</td>
</tr>
<tr>
<td>C4(I,II)</td>
<td>168.3</td>
<td>9.53</td>
</tr>
<tr>
<td>D3(I,II)</td>
<td>168.3</td>
<td>4.8</td>
</tr>
</tbody>
</table>

(b)

<table>
<thead>
<tr>
<th>Test No.</th>
<th>Apparent structural parameters</th>
<th>Structural parameters from Load-axial shortening curves</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>AE (GPa)</td>
<td>Av</td>
</tr>
<tr>
<td>A1(I,II)</td>
<td>296.4</td>
<td>.288</td>
</tr>
<tr>
<td>B1(I,II)</td>
<td>230.3</td>
<td>.139</td>
</tr>
<tr>
<td>C1(I,II)</td>
<td>302.8</td>
<td>.319</td>
</tr>
<tr>
<td>D1(I,II)</td>
<td>221.3</td>
<td>.222</td>
</tr>
</tbody>
</table>
Table 3.6.1.1

<table>
<thead>
<tr>
<th>Test No.</th>
<th>Relative slenderness ( \lambda )</th>
<th>Relative ultimate stress ( \sigma_u )</th>
<th>Relative ultimate strain ( \varepsilon_u )</th>
<th>Relative yield stress ( \sigma_y )</th>
<th>Relative yield strain ( \varepsilon_y )</th>
<th>( \lambda_A )</th>
<th>( \lambda_C )</th>
</tr>
</thead>
<tbody>
<tr>
<td>A1(I,II)</td>
<td>.140 .173</td>
<td>1.25 1.22</td>
<td>24.81 16.32</td>
<td>2.36 2.25</td>
<td>.154 .190</td>
<td>.190</td>
<td></td>
</tr>
<tr>
<td>B1(I,II)</td>
<td>.224 .241</td>
<td>1.15 1.14</td>
<td>8.69 7.48</td>
<td>1.45 1.87</td>
<td>.246 .265</td>
<td>.265</td>
<td></td>
</tr>
<tr>
<td>C1(I,II)</td>
<td>.146 .183</td>
<td>1.23 1.18</td>
<td>22.65 14.37</td>
<td>1.39 1.83</td>
<td>.160 .201</td>
<td>.201</td>
<td></td>
</tr>
<tr>
<td>D1(I,II)</td>
<td>.243 .256</td>
<td>1.08 1.08</td>
<td>6.21 5.58</td>
<td>2.34 1.98</td>
<td>.267 .281</td>
<td>.281</td>
<td></td>
</tr>
</tbody>
</table>

Where

- \( MS = \) Micro strain
- \( CE \) and \( Cv \) = Classic modulus of elasticity and Poisson’s ratio, respectively.
- \( C_{fy} \) and \( C_{ey} \) = Yield stress and strain according to the classic values for Modulus of elasticity and Poisson’s ratio, respectively.
- \( AE \) and \( Av \) = Apparent modulus of elasticity and Poisson’s ratio, respectively.
- \( A_{fy} \) and \( A_{ey} \) = Yield stress and strain according to the apparent values for Modulus of elasticity and Poisson’s ratio, respectively.
- \( f_{ul} \) and \( e_{ul} \) = Ultimate stress and strain in steel tube, respectively.
- \( P_{ul} \) = Ultimate load in steel tube.

and other parameters are defined as follows:

The yield stresses and strains are obtained from the 0.2\( \varepsilon \) offset proof stress (the intersection point of stress-strain curve and line of \( \sigma = 0.002 + E.\varepsilon \)) and are written in Tables 3.6.1.1a, b. Non-dimensional parameters are used to compare the results with each other and available results in references, the parameter \( \lambda \) (relative slenderness) is introduced; it has the geometrical specifications of the tube, as well as the material properties. This parameter is define as Eq. 3.1.2, but, the constant factor has not been considered, Eq. 3.6.1.1.

Accurate values of the classical constants, \( E \) and \( v \), may not always be accurately obtained in a compression test on a tube. The reason, as discussed in 3.5.3, is the accuracy of these kind of tests and more importantly, the change of structural characteristics during the manufacturing process, the existence of the residual stresses and the geometric specifications. With this view, two values are specified for each introduced parameter, one according to the classical values of the modulus of elasticity and Poisson's ratio of steel, and the other one is based on the observed values (apparent).

Other non-dimensional parameters that are required in the next sections are relative ultimate and yielding stresses and strains. These parameters are specified according to the proposed evaluation method in Ref. [1], and are given in Eqs. 3.6.1.1 and 3.6.1.2a,b.
\[ \lambda = \sqrt{\frac{d}{t} \cdot \frac{f_y}{E}} \]

\[ \bar{\sigma}_u = \frac{\sigma_u}{f_y} \]
\[ \bar{\varepsilon}_u = \frac{\varepsilon_u}{f_y} \]

(3.6.1.1)

(3.6.1.2a,b)

where:

\( \sigma_u \) and \( \varepsilon_u \) are ultimate stress and strain, respectively.

More importantly, the results of these tests have to be compared with the recommended criteria in codes of practice and existing results. Accordingly, the design buckling curve in DIN 18800 and the recommended limit in Eurocode 3 were used Appendix [J].

Fig. 3.6.1.2 shows the position of current test results and reported results in Ref. [1] on a \( \lambda - \bar{\sigma}_u \) coordinate system. The limits from the codes of practice are also drawn in this figure. As it can be seen from the figure, the current test results furnish conservative results when compare with both of the codes, but the results from Ref. [1] do not show good agreement. It seems the reason should be suspected in preparing the specimens. As it is explained on the report (Ref. [1]), the specimens are machined to obtain different wall thickness and diameter from one source tube. This process may have an effect on the distribution of residual stresses and more importantly, the material properties are different throughout the wall thickness of the tube. This is a considerable parameter specially in thick walled tubes Ref. [1]. Moreover, the problems of axial load capacity of very thick walled tubes are completely different with ordinary dimensions of tubes. In a very thick walled tube like specimens No. 8, 5, 3, and 2, in Ref. [1], the plastic mechanism at ultimate load is totally different with available proposed plastic mechanisms. It seems there is an overall plastic mechanism that with the existing dimensions (A1(I,II) and C1(I,II)), the effect of supports are considerably high and perhaps longer specimens have to be chosen. Except from the above mentioned tests the rest of the results show a fair agreement with the code limits.

To find an experimental explanation for the ultimate load capacity of the CHS, it appears that the current results can be formalised in a linear manner. In this view a linear approximation method were applied to the results (apparent values for \( E \) and \( \nu \)), to obtain a linear equation between relative slenderness \( (\lambda_s) \) and ultimate load \( (P_u) \) as shown in Eq. 3.6.1.3. The apparent results can be fitted in a better accuracy than the classical values. Moreover, with omission of the test No. D1(I,II), which represents the
ultra tube with special coating for environmental protection, the result will have more accuracy in line fitting. Eq. 3.6.1.3 will be valid for the range of $\lambda$ from $\lambda=0.15$ up to $\lambda=0.27$ and can be used in further studies.

$$\frac{\sigma_u}{f_y} = 1.447 - 1.687 \sqrt{\frac{d}{t} \cdot \frac{f_y}{E_{App}}}$$

(3.6.1.3)

On the other hand, the interrelation between the $[\varepsilon_u - \lambda]$ has the best relationship and should be the best method to explain the behaviour of CHS columns. Fig. 3.6.1.3 shows the scatter of the test results in a $[\varepsilon_u - \lambda]$ coordinate system. Both of the apparent and classical values which are used to obtain the parameters of $\varepsilon_y$ and $\lambda$ are fitted with two lines in a good accuracy. Further justification for this results is performed in 3.7.

Accordingly, two equations can be proposed for evaluation of the ultimate strain at the ultimate load of an axially compressed tube within the range of $\lambda$, from $\lambda=0.15$ to $\lambda=0.27$ for both apparent and classical values for $E$ and $v$ Eqs. 3.6.1.4a,b.

$$\varepsilon_u = \frac{f_y}{E_{App}} (49.3 - 148.2 \sqrt{\frac{d}{t} \cdot \frac{f_y}{E_{App}}})$$

$$\varepsilon_u = \frac{f_y}{E_{Cls}} (37.8 - 104.1 \sqrt{\frac{d}{t} \cdot \frac{f_y}{E_{Cls}}})$$

(3.6.1.4a,b)

Fig. 3.6.1.2 The scatter of the current test results and results in Ref. [1].
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3.6.2 Stress and Strain Relationship

The relationship between stresses and strains and biaxial stresses are investigated in this section. As in the previous section, both classical and apparent values for modulus of elasticity and Poisson's ratio are considered in this study. The principal effort is to evaluate the capability of the CHS's for composite purposes and to find out the best criteria for use in this evaluation. Furthermore, the elastic and plastic behaviour of these structural members are studied in more detail.

The amounts of longitudinal and circumferential strains are obtained according to the average method which was explained in 3.5.3. The stresses are obtained from the measured strains as explained in 3.4. This section is according to the results from the specimens with three double strain gauges.

3.6.2.1 Modulus of Elasticity and Poisson's Ratio

The elastic constants such as modulus of elasticity and Poisson's ratio are not generally constant parameters throughout the whole history of stress. The value of modulus of elasticity and Poisson's ratio are mainly based on the material parameters in principal directions, geometry of the specimens, history of the stresses, actual distribution of stresses in principal directions, and the linear part of the stress-strain curve. It has widely been accepted to use of the early stages of stress-strain curves for obtaining the

Fig. 3.6.1.3 Scatter of the test results and curve fit for apparent and classical values of $E$ and $v$.

![Graph showing scatter of test results and curve fit for apparent and classical values of $E$ and $v$.]
elastic parameters. In this study, the elastic parameters are determined from the relatively linear parts of the stress-strain curve of the circular hollow sections under axial compression. Based on the values of $E$ obtained for the specimens, the stresses at the different stages of loading can be computed (Appendix [C]).

![Stress versus Strain at the early stages of loading.](image)

Fig. 3.6.2.1.1 Stress versus Strain at the early stages of loading.

Fig. 3.6.2.1.1 contains the results from four test specimens and the equation of each line is presented in Table 3.6.2.1.1. The correlation factor ($r$), for each line represents the accuracy of the line of best fit with the test results. Therefore, the specified modulus of elasticity is the best representative value of $E$ for these specimens.

<table>
<thead>
<tr>
<th>Test No.</th>
<th>$E$ (GPa)</th>
<th>Correlation ($r^2$)</th>
</tr>
</thead>
<tbody>
<tr>
<td>A1 II</td>
<td>296.4</td>
<td>.998</td>
</tr>
<tr>
<td>B1 II</td>
<td>230.3</td>
<td>.999</td>
</tr>
<tr>
<td>C1 II</td>
<td>302.8</td>
<td>.998</td>
</tr>
<tr>
<td>D1 II</td>
<td>221.3</td>
<td>.99</td>
</tr>
</tbody>
</table>

Table 3.6.2.1.1 The slope of the fitted line and the correlation factor in Fig. 3.6.2.1.1.

The other elastic parameter is Poisson's ratio which can be determined in the same manner as modulus of elasticity, Fig. 3.6.2.1.2. The correlation of these results is not as good as that of the modulus of elasticity. But, these results are quite acceptable to be considered as the apparent Poisson's ratio for the tubes. Results are given in table 3.6.2.1.2.
Fig. 3.6.2.1.2 Vertical stress versus vertical strain in elastic range.

Table 3.6.2.1.2 The slope of the fitted line and the correlation factor in Fig. 3.6.2.1.2.

<table>
<thead>
<tr>
<th>Test No.</th>
<th>( v )</th>
<th>Correlation ( (r^2) )</th>
</tr>
</thead>
<tbody>
<tr>
<td>A1 II</td>
<td>.288</td>
<td>.998</td>
</tr>
<tr>
<td>B1 II</td>
<td>.139</td>
<td>.999</td>
</tr>
<tr>
<td>C1 II</td>
<td>.319</td>
<td>.998</td>
</tr>
<tr>
<td>D1 II</td>
<td>.222</td>
<td>.99</td>
</tr>
</tbody>
</table>

The important points in these results is the close values of Poisson's ratio for specimens A1 II and C1 II (\( d/t = 18.1 \) and 17.7, respectively), and the difference of Poisson's ratio for the specimens B1-II and D1-II. For the first two specimen (A1-II, and C1-II) the \( d/t \) ratio is approximately the same, and, for the specimens (B1-II and D1-II) that the specimen D1 II is epoxy coated, the \( d/t \) ratio is also the same (35.1 for both). The Poisson's ratios less than \( v = 0.29 \) (steel Poisson's ratio), represents that the steel tube is under compressive stresses in circumferential direction, that means the tube can not freely expand. Theoretically, for a tube under pure axial compression the circumferential stress must be zero, regardless of the effects from the boundary conditions at the supports.

According to the results for Poisson's ratio, the process of epoxy coating of the steel tubes, increases the apparent Poisson's ratio. Furthermore, The Poisson's ratio in thick walled tubes (B1 II and C1 II) is larger than the other tubes. More details of apparent elastic parameters are given in Chapter 4.
3.6.2.2 Stress Strain Curves for the Tests with Strain Gauges

Three type of stresses are introduced in this section. The first one is the stresses which are inferred from the testing machine. The second and third one are stresses which are obtained from measured strains, with the use of apparent and classical values for E and v.

According to the above mentioned stresses and measured strains, three sets of stress-strain curves for each, of the four tests are shown in Figs. 3.6.2.2.1a,...,d.

The whole curves are drawn to illustrate the complete behaviour in the elastic and plastic ranges. As could be predicted from the Table. 3.6.2.1.1, the differences between stresses are more considerable in the test Nos. A1 II and C1 II. But the stress-strain curve approximately follows the stress-strain curve from direct measurements in the curves for test Nos. B1 II and D1 II. It means the effect of dimensions on the specimen Nos. B1 II and D1 II. are much lower than the specimen Nos. A1 II and C1 II.

The common behaviour in all of the curves in Figs. 3.6.2.2.1a,...,d is the drop of stresses at the last point in curves of stresses from measured strains. The magnitude of this drop is different in all of the curves, and it appears that the dimensions of the specimens with smaller values of (d/t), make a more accurate result for the stress-strain curve.

According to the locus of biaxial stresses in the Von-Mises yield criterion, the decrease of major stress after yielding is inevitable. But, the phenomenon of strain hardening would reduce the rate of this drop for major principal stresses.

There is also a considerable differences between the computed stresses from the measured strains at the early stages of inelastic behaviour, from the stresses directly obtained by measurements. This difference shows the effect of the manufacturing process and the geometry on thick walled tubes. As a result, the dimensional effect and the effects of the manufacturing process can be summarised as follows:
For thick walled tubes with \(((d/t) \leq 18)\), the effect from residual stresses and deformation patterns due to the dimensions is considerably high on obtaining stresses from measured strains. Based on the manufacturing process in tube maker factories, heat treatment may not be performed in conventional products, therefore, plastic deformation of steel plates will produce residual stresses in steel tubes. Moreover,
plastic deformation of an under compression steel tube has a direct relationship with the outside diameter and wall thickness that discussed in 3.3. Conversely, for relatively thin walled tubes with \((d/t) \geq 35\), this effect is more effective at the final stages of loading and the rest of the stresses are in a good agreement with the direct measurements of the stresses.

3.6.2.3 Biaxial Stresses on the Surface of the Specimens

The stresses in the longitudinal and circumferential directions are measured according to the method which was explained in Part 3.4. The strain gauges are attached to the outer surface of the specimens, therefore, the computed stresses furnish the amount of stresses just on the surface of the specimens.

According to non-linear finite element analysis for the specimen No. B1 II and the theoretical equations for a tube under axial compression, the intensity of the circumferential stresses must be zero while the specimen is under axial compression. Regardless of the small circumferential stresses in elastic range, all of the specimens confirm the analysis results, but in the plastic range, the circumferential stresses increase rapidly after yield point is reached in axial compression. As the sign of stresses shows, all of the circumferential stresses are in tension. Biaxial normalised diagrams are shown in Figs. 3.6.2.3.1a, b for all of the specimens.

On of the important points which arises here is the absence of circumferential stresses, for all of the specimens in the elastic range. This indicates that the expansion of the tubes under axial compression can be accurately predicted by axial strains and the values of Poisson's ratio.

Sudden changes at the last stages in the plastic region in all of the biaxial graphs for stresses is due to the local buckling of the tubes at the supports. The local buckling at the supports, decreases the increasing rate of the transverse stresses in circumferential direction. The ellipse which are drawn in Figs. 3.6.2.3.1a, b are the Von-Mises criterion Ref. [3].
Chapter Three Experimental and Theoretical Studies on the Ultimate Load Capacity of CHS

- Fig. 3.6.2.3.1a Normalised stresses (apparent values of E and v).

- Fig. 3.6.2.3.1b Normalised stresses (classic values of E and v).
3.7 FURTHER JUSTIFICATIONS FOR THE RELATIONSHIP OF \([\varepsilon_u - \lambda]\)

To justify the results obtained in 3.6.1, another set of tests was carried out. The principal concepts for this new set of tests were repetition of some of the tests in previous set of experimental program and a new set of two dimensions for increasing the number of points in the concluded graphs in Part 3.6.1. The detailed specifications of this new set of experiments are given in Table 3.7.1.

<table>
<thead>
<tr>
<th>No.</th>
<th>Specimen</th>
<th>Outside Diameter d (mm)</th>
<th>Wall Thickness t (mm)</th>
<th>Length l (mm)</th>
<th>d/t</th>
<th>Grade</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>A1-III</td>
<td>114.3</td>
<td>6.00</td>
<td>500</td>
<td>19.05</td>
<td>C350</td>
</tr>
<tr>
<td>2</td>
<td>A1-IV</td>
<td>114.3</td>
<td>6.00</td>
<td>500</td>
<td>19.05</td>
<td>C350</td>
</tr>
<tr>
<td>3</td>
<td>C1-III</td>
<td>168.3</td>
<td>9.53</td>
<td>500</td>
<td>35.06</td>
<td>C350</td>
</tr>
<tr>
<td>4</td>
<td>C1-IV</td>
<td>168.3</td>
<td>9.53</td>
<td>500</td>
<td>35.06</td>
<td>C350</td>
</tr>
<tr>
<td>5</td>
<td>E1-I</td>
<td>60.5</td>
<td>2.50</td>
<td>500</td>
<td>24.20</td>
<td>C350</td>
</tr>
<tr>
<td>6</td>
<td>F1-I</td>
<td>60.5</td>
<td>2.50</td>
<td>500</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

The testing procedure was entirely the same as the previous set of experimental program and the only difference was in the number of strain gauges that were attached to the outer surface of the specimen Nos. A1 (III,IV) and C1(III,IV). In this new set the number of places for measuring the strains during the test were increased to 4. With this regard the strain gauges were attached at an angular distances equal to 90° at the midheight of the specimens.

To determine the required specifications for further discussions, the load-axial shortening curves for each specimens are given in Figs. 3.7.1 and 2. According to these curves the value of ultimate strain can be readily determined. Same as previous discussion about \([\varepsilon_u - \lambda]\), the scatter of the results can be illustrated as Fig. 3.7.3. Considering the fitted line for the results according to the classic values of E and v, in Part 3.6.2, the fitted line in this series of results are in a good agreement with previous results. Eq. 3.7.1 is representing the fitted line for all of the results including the obtained results in Part 3.6.2.
Fig. 3.7.1 Axial load-shortening curves for specimen Nos. A1 (III,IV) and F1 (III,IV).

Fig. 3.7.2 Axial load-shortening curves for specimen Nos. E1 I and F1 II.
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Results according to the classic values of $E$ and $v$

$$\varepsilon_u = -126.668\lambda_s + 37.922 \quad r^2 = 0.963$$

Fig. 3.7.3 Scatter of the test results and curve fit for classical values of $E$ and $v$.

$$\varepsilon_u = \frac{f_y}{E_{ClS}} \left( 37.9 - 115.2 \sqrt{\frac{d}{t}} \frac{f_y}{E_{ClS}} \right)$$

Consequently, the ultimate strain in a stub column test can be determined according to the classic values of modulus of elasticity and Poisson’s ratio plus the yield strength of steel and the geometrical specifications of steel circular hollow section for a limited range of $\lambda$.

The use of Eq. 3.7.1 is for determining the ultimate strain in a steel tube for carrying the ultimate load. Beyond the ultimate strain, the load carrying capacity of the steel tube in longitudinal direction, has a sudden drop until the start of minor load capacity that is shown in Fig. 3.7.2.

On the other hand, Eq. 3.7.1 for a specific range of steel tube with different dimensions can be readily calibrated to get necessary specifications for the plastic analysis and design of structures with circular hollow sections. Also, further studies may be conducted toward more accurate scalar factors for Eq. 3.7.1 in the studies about the steel tubes.
3.8 SUMMARY

The experimental results for the ultimate load capacity of axially compressed short steel circular hollow sections do not completely coincide with the theoretical results. The main reason for the large scatter of the test results in the range of small relative slenderness values must be suspected in material properties and manufacturing process rather than in the buckling phenomena.

However, a series of seventeen experimental tests with different dimensions of the circular hollow section ($d/t = 17, 25, 35$) and one type of tube which is epoxy coated has been carried out. The epoxy coated tube is a special tube for hazardous environmental situations such as off-shore, marine, or, buried structures. This tube has been pre-heated up to 250° C and then is covered with a suitable epoxy with thickness of 500 microns. The heating process is not longer than 10 minutes but the structural effects have to be studied for possible benefits or damage that may occur during this process. In this study the geometrical buckling theory in the elastic and the elastic-plastic range under axial compression, non-linear finite element analysis, and the results of stub column tests are discussed.

Results of these tests show that the relationship between ultimate strain and a proposed relative slenderness is expressible by a linear equation. Moreover, correlation of the results shows that the accuracy is considerably high. On the other hand, the results of experiments have proven that the epoxy coated tube is not suitable for composite purposes, but inversely, the un-coated section has ideal properties for composite purposes for the particular tube dimensions.

Elastic properties of the steel tubes are expressed by two sets of apparent and classic values. The apparent values are elastic parameters that cover not only the material properties but also include the effects of manufacturing process. Therefore, it can be another characteristic of steel tubes that have a great effect on the elastic and plastic analysis and design of such a section. The classic values are values obtained from conventional test procedures on coupon test specimen. In view of material properties it is a useful parameter, but in case of a very accurate analysis for structural design purposes necessary amendments have to be applied.
4.1 INTRODUCTION

One of the most important points in the design of steel-concrete structures is to remove or minimise the brittle behaviour of concrete. In view of the stability analysis of structures, in the post-yielding range, the sudden decrease in load bearing capacity of beams, columns and joints, may cause severe damage in the case of unexpected live loads. However, code designed structures are capable of surviving severe earthquakes or other form of loading providing they are able to absorb and dissipate the energy by ductile behaviour in the post-elastic range.

Numerous conventional solutions are used to enhance the ductile behaviour of concrete. The most practical way in beams, slabs and columns is to apply the allowable limit of reinforcement percentage and to use mild steel to provide enough elongation. Another way that is usually applicable in seismic areas, for beams and columns, is to encase the concrete at the most probable areas for hinges by additional shear reinforcement. The last method is part of compulsory requirements for designing a safe structure in areas with a high risk of earthquake.

An important method that has attracted the keen attention of numerous researchers and structural designers, is the full confinement of concrete with a suitable hollow section. Regardless of its constructional advantages, the increase in ultimate load capacity and change of plastic behaviour of concrete can be considerably enhanced. The increase of the ultimate load capacity can be more than two times the normal compressive capacity of concrete Ref. [22], and, according to the experimental results in available literature, the plastic range can also be extended to even more than 10% of the length of the specimens. On the other hand, there is a mutual interaction between the hollow section and the concrete. The elastic and plastic load capacity of hollow sections can also be improved by this method.

Considering the behaviour of high strength concrete, the problem is even more important than for ordinary concrete. As is shown in Appendix [H], high strength
concrete has a brittle pattern of failure in an uniaxial compression test. This behaviour is totally unacceptable for structural purpose (because of sudden decrease in load carrying capacity). For instance, the recommended limits in various codes of practice, such as ACI-ASCE, 352 Ref. [19], limits the joint-shear stress to $\gamma \sqrt{f'_c (\text{psi})}$, where the factor $\gamma$ is a function of type of joint and loading condition. The value of this expression should not exceed 41 MPa Ref. [19]. However, the use of high strength concrete in high rise structures as a new form of concrete that has great advantages in comparison with ordinary concrete is inevitable. Therefore, necessary provisions should be considered for eliminating the brittle behaviour of high strength concrete.

The following studies concern higher strength concrete fully confined in circular hollow sections. The effects of the tube’s strength on the elastic and plastic behaviour for different dimensions of the steel tubes and the concrete has been investigated. A total number of eight experimental tests has been carried out; the dimensions and structural specifications are shown in Table 4.1.1.

<table>
<thead>
<tr>
<th>Tube No.</th>
<th>D (mm)</th>
<th>t (mm)</th>
<th>L (mm)</th>
<th>$f_y$ (MPa)</th>
<th>$f_c$ (MPa)</th>
<th>As (mm²)</th>
<th>Ac (mm²)</th>
</tr>
</thead>
<tbody>
<tr>
<td>A2(I)*</td>
<td>114.3</td>
<td>6.30</td>
<td>500.</td>
<td>389.2</td>
<td>69.6</td>
<td>1651</td>
<td>8609</td>
</tr>
<tr>
<td>B2(I)*</td>
<td>168.3</td>
<td>4.80</td>
<td>500.</td>
<td>397.4</td>
<td>61.3</td>
<td>2465</td>
<td>19780</td>
</tr>
<tr>
<td>C2(I)*</td>
<td>168.3</td>
<td>9.53</td>
<td>500.</td>
<td>441.1</td>
<td>71.6</td>
<td>4753</td>
<td>17492</td>
</tr>
<tr>
<td>D2(I)*</td>
<td>168.3</td>
<td>4.80</td>
<td>500.</td>
<td>450.0</td>
<td>71.4</td>
<td>2465</td>
<td>19780</td>
</tr>
<tr>
<td>A2(II)‡</td>
<td>114.3</td>
<td>6.30</td>
<td>500.</td>
<td>389.2</td>
<td>69.6</td>
<td>1651</td>
<td>8609</td>
</tr>
<tr>
<td>B2(II)‡</td>
<td>168.3</td>
<td>4.80</td>
<td>500.</td>
<td>397.4</td>
<td>61.3</td>
<td>2465</td>
<td>19780</td>
</tr>
<tr>
<td>C2(II)‡</td>
<td>168.3</td>
<td>9.53</td>
<td>500.</td>
<td>441.1</td>
<td>71.6</td>
<td>4753</td>
<td>17492</td>
</tr>
<tr>
<td>D2(II)‡</td>
<td>168.3</td>
<td>4.80</td>
<td>500.</td>
<td>450.0</td>
<td>71.4</td>
<td>2465</td>
<td>19780</td>
</tr>
</tbody>
</table>

where:

* = Load-axial shortening control test.
‡ = Load-axial shortening control, and, strain control test.

4.2 RESEARCH SIGNIFICANCE

Although the confinement of concrete in different forms of surrounding structures have been studied for years, nevertheless, considering the different behaviour of concrete it is necessary to obtain the required parameters according to the actual behaviour of the concrete. More importantly, the direct studies about the confinement effects on the higher strength concrete (Ref. [19] and similar studies in references) are not enough to be used as a reliable basis for the structural behaviour of this kind of concrete.
Furthermore, the stability of the concrete (formation of major cracks) is also an important issue in the structural design. The effectiveness of confinement by the steel tubes on the brittle behaviour of the higher strength concrete have to be investigated for determining the stable or unstable plastic mechanism in the composite columns. Consequently, following study was carried out to investigate the influence of the steel tube on the structural characteristics of the higher concrete.

4.3 EXPERIMENTAL PROGRAM

The structural behaviour of confined concrete in circular hollow sections (CHS) subjected to axial compression depends principally on the stiffness of the circumferential steel tube, the strain rate, the concrete strength and the longitudinal stresses in the steel tube. The effects of the longitudinal stresses in the steel tube has not specifically been investigated for the higher strength concrete. The concrete compressive strength was kept more or less constant.

This research program is concerned with the effect of biaxial stresses in the steel tube on the strength and mechanical properties of concrete. The strain rate was constant for all of the specimens at 0.002-0.0025 ((mm/mm)/min.) to model a quasi static test method.

The experimental program was carried out in two phases. The first phase used the specimens with the equipment necessary for measuring the strains at the midheight of the specimens. Phase two consisted of same number and dimensions of specimens as phase one, but the specimens were prepared for measuring the axial load-deflection behaviour only.

Detailed specification of the concrete and the steel tubes are discussed in Appendix [H], and Chapter 3, respectively. Based on the obtained results in those sections, the material properties of steel can be categorised as a high strength steel and for concrete, as a higher strength concrete.

4.4 TEST SET-UP AND MEASUREMENTS

The experimental set-up is shown in Figs. 4.4.1a,b. Test specimens were placed between one spherical support at the top and an immovable support at the bottom. The axial load was directly transferred to the concrete core by two loading discs. The
loading discs were accurately machined from a thick steel plate for each specimen with 20 mm thickness.

Fig. 4.4.1a Test set-up (schematic).

Fig. 4.4.1b Position of strain gages.

The testing machine that was used for these tests was a standard compression test machine with a maximum capacity of 5000 (kN) for axial compression. For readings the strains from strain gages in the first phase of the experimental program a data acquisition system was used. With this system, the output results were directly recorded at the different intensity of stresses. Moreover, the curves of axial load-shortening were also drawn with separate instruments. For this purpose, two LVDTs, one for vertical load and one for longitudinal shortening was used. The calibration of all instruments was carried out before and after the tests. The calibration control for the testing machine and the LVDT for axial deformation are explained in Appendices D and E, respectively.

4.5 IMPROVEMENT AND EVALUATION OF THE TEST RESULTS

The need for improvement of the test results, and the evaluation of the observed behaviour, is the same as that described in 3.5.3. A similar method was used for
correcting the results from strain gauges, and also the original graphs were corrected according to the explained procedure in section 3.5.3.

An additional point in this section is the effect of the loading discs on the output of axial deformation. Considering the classical value for modulus of elasticity and dimensions of $d=168.3$, $t=4.8$ mm, the maximum displacement for both of the loading discs is about 0.036 mm which is not effective for the graphs of load-axial deformation.

4.6 TEST RESULTS AND DISCUSSION

4.6.1 Axial Load-Shortening Behaviour

As was discussed in section 4.3, all of the experiments were based on a quasi static loading procedure. Accordingly, the loading rate for all of the eight specimens were 0.002-0.0025 ((mm/mm)/min.). The specimens were centralised by the guide lines on the platens of the testing machine. The axial load-shortening curves for four specimens are shown in Fig. 4.6.1.1. As it was mentioned in Table 4.1.1, for the total number of the tests in the experimental program, there were four pairs of specimens that the dimensions were the same for each pair. The test results have such similar behaviour in a pair, therefore, only one of the curves in each pair are shown in Fig. 4.6.1.1.

The most important point in all of the load deflection curves is the plastic behaviour even beyond 3% in the longitudinal direction that seems it can be extended even up to more than 5% in the longitudinal direction. This shows the enhancement of the concrete ductile behaviour when confined in a steel tube.

According to the shape of the curves, it can be expressed that the stronger tubes have less drop in sustainable axial load after the ultimate point. Reverse to the behaviour of hollow sections, the ultimate strain is more or less equal, and shows that the concrete predominantly controls the whole behaviour of the composite structure. Nevertheless, the ultimate stress for each test is different and is greater than the ultimate stress for unconfined concrete. The axial load-shortening curve for specimen No. A2(I,II) shows a lower modulus of elasticity than the rest of the specimens. The cause appears to be the low slenderness of this section in relation to the other specimens, which caused some bending deformation in addition to the axial deformation. The reason for the bending deformation is in the inevitable eccentricity after formation of cracks in the concrete core.
The ductile behaviour of the composite section is brought about by the ductile behaviour of the steel tube. The steel tube at the ultimate stage of loading ($\varepsilon_y > 0.005$), strongly holds the concrete but its lateral expansion provides a finite space to accommodate the crushed concrete. The crushed concrete behaves as an incompressible fluid that is completely confined by the steel tube. Therefore, the longitudinal force on the concrete converts to mainly circumferential and partly longitudinal stresses in the steel tube, and also the compressive stresses in the confined concrete, Fig. 4.6.1.1. This behaviour is not always the case for high or higher strength concrete in composite sections. The failure pattern of high or higher strength concrete is usually accompanied by major cracks in the concrete mass. This behaviour changes the expected form of the failure pattern for confined concrete, but the general behaviour does not change significantly. As observed from the Fig. 4.6.1.1, for test No. B2(I,II) and No. D2(I,II), because of the weaker circumferential strength of the stub tubes, the drop of the sustainable load is greater than that for the other specimens. It means the major cracks change the distribution of the stresses, so that, the load concentrates on a small area of the steel tube and consequently the ultimate load drops. The deformed shape of the specimens after the tests confirms the above mentioned discussion, Photo 4.6.1.2. The concrete core is also shown in Photo 4.6.1.2. The crushed area in the concrete confirms the illustrated model in Fig. 4.6.1.2.

Table 4.6.1.1 shows the specifications of the loads for the composite column tests that are required for further discussion. Regarding the determined values of yield stresses for the steel tubes with different dimensions (Chapter 3), and the ultimate strength obtained from the standard tests ($F'_c$), parameters such as the concrete share of the axial load, the increase of the compressive capacity of concrete due to confinement, and other structural parameters, can be determined.

Table 4.6.1.1 Specifications of compressive capacity of confined and plain concrete, and CHS respectively.

<table>
<thead>
<tr>
<th>Specimen No.</th>
<th>Ultimate load of confined concrete ($F_c$) (kN)</th>
<th>$f_c$ (MPa)</th>
<th>$F'_c$ (kN)</th>
<th>Ultimate load of CHS ($F_S$) (kN)</th>
<th>$F_c/F'_c$</th>
<th>($F_c-F_S)/F'_c$</th>
</tr>
</thead>
<tbody>
<tr>
<td>A2(I,II)</td>
<td>1770</td>
<td>69.6</td>
<td>599.2</td>
<td>1042</td>
<td>2.95</td>
<td>1.21</td>
</tr>
<tr>
<td>B2(I,II)</td>
<td>3390</td>
<td>61.3</td>
<td>1212.5</td>
<td>1130</td>
<td>2.80</td>
<td>1.86</td>
</tr>
<tr>
<td>C2(I,II)</td>
<td>4655</td>
<td>71.6</td>
<td>1252.4</td>
<td>2575</td>
<td>3.72</td>
<td>1.66</td>
</tr>
<tr>
<td>D2(I,II)</td>
<td>3455</td>
<td>71.4</td>
<td>1412.3</td>
<td>1200</td>
<td>2.45</td>
<td>1.60</td>
</tr>
</tbody>
</table>

For presenting the analytical curves for the stress-strain behaviour of the concrete, under axial and lateral stresses, records from the strains at the surface of the steel tube is the best representation for axial deformation. As shown in Fig. 4.6.1.2, the total axial shortening of the confined concrete, consist of two translations at the both ends and
structural deformation at the middle of the specimens. Overall deformation for representing the material behaviour of confined concrete cannot be used in this form of confinement (stocky columns). The reason is that the same plastic mechanism occurs in the confined concrete for different heights. The height of the penetrating cones for a specimen with different height seems to be equal, and, the state of the concrete in between is also the same. Therefore, the overall deformation of concrete does not represent the material behaviour of the concrete, but is a good representation of structural behaviour.

Fig. 4.6.1.1 Axial load -shortening curve for confined concrete.

Fig. 4.6.1.2 Modelling of undeformed and deformed confined concrete.
Photo 4.6.1.1 Deformed specimen after tests. From left to right; D2-I, C2-I, B2-I, A2-I.
According to the results obtained for longitudinal and circumferential strains for the outer surface of the steel tube, the steel tube is also under the axial load even at the early stages of loading. Furthermore, because of the difference in Poisson's ratio for steel and concrete, before complete formation of cracks in concrete, the steel tube separates from the concrete core (see Chapter 8), therefore, considering the superposition of the maximum axial load capacities of the steel tube and the concrete core is a reasonable basis for distribution of the axial load between the steel and concrete at the yield point.
Regarding the axial load capacity of circular hollow sections, the maximum compressive capacity of confined concrete can be calculated by subtracting the recorded maximum load for CHS from the maximum axial load of confined concrete. Results in Table 4.6.1.1 show that the increase in maximum load capacity of the concrete is between 1.3-1.9 times that of the unconfined concrete strength. According to the proposed relationships by Cai, Eq. 4.2.1, the radial stresses on the concrete core must be about 10% of the compressive capacity of unconfined concrete in order to achieve the above mentioned results. On the other hand, the observed results from the compression tests on confined concrete, without subtracting the axial load capacity of the steel tube, show that the compressive capacity of the confined concrete, from a structural view, increases to 2.5-3.7 times that of the unconfined concrete.

Regarding the circumferential yield state of the steel tube, and the following analytical model, the radial stresses can be computed by Eq. 4.6.1.1.

\[
f_{y,t} = \frac{d_c}{2} \sigma_r
\]

(4.6.1.1)

where:

\[ \sigma_r = \text{Radial stress (internal pressure) in the steel tube.} \]

For instance, the radial stresses at the ultimate stage for specimen No. B2(I-II) is approximately 6% of the steel yield strength which is high. But, this is the case of concrete, confined by a circular hollow section under a condition of loading on concrete core only. Circumferential yield and the confining pressure can be accurately computed by the Eq. 4.6.1.1. It means, that the confinement pressure on the concrete must be larger than the results of the proposed equation by Cai; detailed discussion is given in section 4.6.3.
4.6.2 Apparent Modulus of Elasticity and Poisson's Ratio

The curves of computed longitudinal stresses from measured strains by the method that was explained in Chapter 3, and the longitudinal strains are illustrated in Fig. 4.6.2.1. The figure represents the elastic range of stress-strain curve for all of the specimens. The elastic behaviour of each steel tube is represented by a linear approximation. For each test, the results are in agreement with the previous results for Circular Hollow Sections. Moreover, because of more recorded points for longitudinal and transverse strains, the computed stresses from measured strain are more accurate. Therefore, the estimated line for elastic behaviour of the steel tubes are more accurate indeed.

The results are again showing an unexpected modulus of elasticity for thick walled tube and a reasonable and acceptable magnitude for thin walled tubes.

The reason for this abnormality of modulus of elasticity lies in the opposite directions of the longitudinal strains at the midheight of the thick walled specimens, due to bulging and axial deformation. For an element on the midheight of the steel tube, two major strain directions can be recognised: one for longitudinal shortening and the other one for the bulge effect, Fig. 4.6.2.2.

The true longitudinal strain is $\varepsilon_L$, occurring at the middle of the wall, but the recorded strain from the test (strain gauges) is $\varepsilon_L - \varepsilon_B$. It means that the measured strains are lower than the real strains and eventually, the stress-strain curves show a larger modulus of elasticity. The results for thinner tubes are more realistic, therefore the
length of the specimens for thin walled tube is long enough to simulate a uniform confinement on the concrete core. But, considering the deformation pattern of the thick walled tubes, Specimen Nos. A2II and C2II, the confinement pressure on the concrete is not uniform throughout the height of the tube, and is clearly recognisable from the Photos 4.6.1.1-2 and Fig. 4.6.1.2.

\[ \varepsilon_b = \frac{E_A}{E_s} \varepsilon_1 = \frac{283500 - 210000}{283500} \varepsilon_1 = 0.259 \varepsilon_1 \]  

Eq. 4.6.2.1 can be used for specimen Nos. A2(I-II) and C2(I-II) that are thick enough to show the bulge effect due to the axial compression on the concrete only. As a result, the lateral expansion of the thin walled CHS in a composite column, when the concrete core is under axial load, is more uniform than that for the thick walled tubes.

Another important result in the case of the stub column tests is the deformation pattern of thick walled tubes under axial compression. As shown by Eq. 4.6.2.1, bulging deformation makes an overall strain in the steel tube in the opposite (tensile) direction of longitudinal compressive strain in outer surface of the steel tube. This deformation decreases the confinement pressure on the concrete core especially at the middle of specimen. Therefore, the confinement is less effective than for the thin walled tube.
On the other hand, the lateral deflection at the elastic range can be presented by an apparent Poisson’s ratio or in other words, the proportion of the circumferential strain to the longitudinal strain. The representative line for each set of the data from the experimental results, are shown in Fig. 4.6.2.5. At the early stages of elastic range, the circumferential deformation of the tubes is relatively the same for all of the specimens, that should be more than the classical values.

![Graph showing longitudinal strain versus circumferential strain](image)

Fig. 4.6.2.5 Longitudinal strain versus circumferential strain.

### 4.6.3 Stress-Strain Curve for Confined Concrete

According to the results obtained from the tests, the total load on the composite columns that was applied on the concrete, and the resultant load on the concrete core and steel tube can be computed by the following equation.

\[
F_{Con} = F_{Appl} - F_{St} \quad (4.6.3.1)
\]

where:

- \(F_{Con}\) = load on concrete.
- \(F_{Appl}\) = total applied load on the specimen.
- \(F_{St}\) = load carried by steel tube.
But regarding the differences between the recorded stresses during the test and the computed stresses from the measured strains that was described in Chapter 3 for steel tube (Fst), the stress-strain curve obtained for the steel tube and concrete core must be corrected. The main concept in the correction method is to compute the difference between the measured stress from recorded strain and the observed stress at any specific point (Hollow section tests). For this purpose, a computer program was developed to do the correction process for each test (Appendix F) and the flow chart is shown in F.4.6.3.1. The corrected stress-strain curve for the concrete is accompanied for comparison, Figs. 4.6.3.1,...4. In these figures, (A) denotes the stresses that are computed according to the apparent elastic parameter (E, v), and, (C) denotes the stresses that are computed according to the classic values of elastic parameters.

In all of the cases, the corrected stress-strain curves for the concrete show larger stresses than the uncorrected curves. The correction for specimens No. A2(I-II) and No. C2(I-II) is considerably higher than the rest of tests. The reason was discussed in section 4.6.2 and lies in the overall bulging deformation in the thick walled tubes. Moreover, the yielding plateau for all of the curves indicate a slight increasing slope that can be considered as the strain hardening behaviour of the encased concrete. At the final stage of the stress-strain curve, the sudden increase of stresses is caused by the longitudinal yield of the steel tube that imposes more stress on the concrete core.

Fig. 4.6.3.1 Corrected stress-strain curve for concrete for test A2II, and, the stress-strain curve for concrete before correction.
Reading data from a recorded file of stress-strain curve where two sets of stresses, one from direct measurement (P/A) and the other one computed from measured strain are considered.

Reading data of stress (computed from measured strain)-strain curve, to be corrected for computed stress measured strain.

Computing the amount of correction for any specific strain according to the results in hollow section tests and correction results in step (2).

Output for the results of corrected stresses for any specific strain.

F 4.6.3.1 Flow chart of the program for correction the stresses from measured strain according to the results obtained for hollow section tests.
Fig. 4.6.3.2 Corrected stress-strain curve for concrete for test B2H, and, the stress-strain curve for concrete before correction.

Fig. 4.6.3.3 Corrected stress-strain curve for concrete for test C2H, and, the stress-strain curve for concrete before correction.
Chapter Four The mechanical behaviour of confined concrete in steel tubes

4.6.4 Confinement Pressure on Concrete

The most important part of this study is the confinement pressure on the concrete core during the axial compression tests. Considering Eq. 4.6.1.1, for computing the radial pressure on the concrete core, the graph of radial stress versus longitudinal stress on the concrete can be used, Fig. 4.6.4.1. These Figures represent the lateral stresses on the concrete core, according to the apparent and classic values for modulus of elasticity and Poisson’s ratio. Moreover, the graphs are accompanied by the best approximation curves which are obtained according to conventional statistical methods.

It is clearly recognisable from all of the results that the increase of confinement pressure on the ultimate load capacity of the concrete core is limited to a specific value which depends on the unconfined compressive capacity of concrete and the dimensions of the steel tube, as well as the material properties. At first glance, the limiting effect can be considered when the steel tube is circumferentially yielded. Regarding the radial stress on the concrete core due to the circumferential yielding of the steel tube for each test, and the relevant curve in Fig. 4.6.4.1, the maximum effect of the confinement can be computed. Results of this computation using the apparent values of modulus of elasticity and Poisson’s ratio are shown in Table. 4.6.4.1.
Considering each representative equation for the tests, the relationship between the longitudinal stresses in the confined concrete and the lateral stresses may also be affected by other parameters such as size effects, support conditions and the length of the specimen. The experimental results of this study show that the enhancement of concrete compressive capacity is not as high as was predicted by available analytical equations, such as Cai (Eq. 2.1.1).

![Graph showing axial stress versus radial stress for different specimens.](image)

Fig 4.6.4.1 Results for axial stress versus radial stress on concrete mass for confined concrete.

To find the accuracy of the proposed equations for the evaluation of strength factor (K) of confined concrete in literature, a number of equations which are mainly designed for reinforced concrete are used, Table 4.6.4.2. For using these equations, the only parameter that is different in composite columns is the method for evaluation of the radial stress on concrete core. To estimate the radial stress at the ultimate stage of
loading, it is assumed that the steel tube is circumferentially yielded. Therefore, the radial stress can be computed by the Eq. 4.6.1.1.

Table 4.6.4.1 Strength ratio of the specimens.

<table>
<thead>
<tr>
<th>Test Nos.</th>
<th>fy (MPa)</th>
<th>RSTy (MPa)</th>
<th>CSTS (A) (MPa)</th>
<th>f'c (MPa)</th>
<th>K</th>
<th>Kr</th>
</tr>
</thead>
<tbody>
<tr>
<td>A2II</td>
<td>389</td>
<td>45.4</td>
<td>128</td>
<td>69.6</td>
<td>1.84</td>
<td>2.95</td>
</tr>
<tr>
<td>B2II</td>
<td>397</td>
<td>23.3</td>
<td>145</td>
<td>61.3</td>
<td>2.37</td>
<td>2.80</td>
</tr>
<tr>
<td>C2II</td>
<td>441</td>
<td>53.0</td>
<td>180</td>
<td>71.6</td>
<td>2.51</td>
<td>3.72</td>
</tr>
<tr>
<td>D2II</td>
<td>450</td>
<td>26.4</td>
<td>148</td>
<td>71.4</td>
<td>2.07</td>
<td>2.45</td>
</tr>
</tbody>
</table>

where:

\[
RSTy = \text{radial stress for circumferentially yielded steel tube. (Yield strength for steel in compression and tension are considered to be equal).}
\]

\[
K = \text{Proportion of confined to unconfined concrete strength (strength ratio) without reduction of compressive capacity of steel tube.}
\]

\[
Kr = \text{Proportion of confined to unconfined concrete strength (strength ratio) with reduction of compressive capacity of steel tube.}
\]

\[
\text{CSTS} = \text{concrete strength at the highest confinement rate according to the corrected curves in Fig. 4.6.3.1 to 4 for apparent elastic characteristic of the steel tube.}
\]

The best analytical results which are in good agreement with the experimental results are the proposed equations by Cai, and, Schickert and Winkler. The general form of these two equations are the same but the numerical coefficients are not equal. The main form of the equations is as follows:

\[
f'_c = f_c \left[ A + B \sqrt{\frac{\sigma_f}{f'_c} - D \frac{\sigma_f}{f_c}} \right] \quad (4.6.4.1)
\]

This is a practical equation for determining the ultimate compressive capacity of differently confined concrete. However, enough statistical data must be available for determination of the factors in Eq. 4.6.4.1. As a general method, for a specific concrete and confinement method, Eq. 4.6.4.1 can be calibrated for use in structural design of composite columns.

It is important to mention that the equations proposed by Cai, and, Schickert and Winkler predict the compressive strength of the confined concrete, regardless of the transferred axial stresses to the steel tube. It means that these equation overestimate the ultimate compressive capacity of confined concrete. Based on the results for axial strain in steel tubes during the tests, the axial stress in the concrete core transfers to the steel tube in a very short distance. Therefore, the steel tube is also in a state of
compressive stress during the test and may even yield longitudinally. As a result, the proposed equations by Cai and Schickert et al. may be corrected as Eq. 4.6.4.2 and 3.

Table 4.6.4.2 Strength ratio, \( K = \frac{f^c}{f'c} \), for confined concrete.

<table>
<thead>
<tr>
<th>Source of the results</th>
<th>Specimen Nos.</th>
<th>Remarks</th>
<th>Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td>Results in this study (Table 4.6.4.1)</td>
<td>1.84</td>
<td>2.37</td>
<td>2.51</td>
</tr>
<tr>
<td>Results in this study (Table 4.6.1.1)</td>
<td>2.95</td>
<td>2.80</td>
<td>3.72</td>
</tr>
<tr>
<td>Cai (1987)</td>
<td>3.51</td>
<td>2.68</td>
<td>3.77</td>
</tr>
<tr>
<td>Modified model of Cai</td>
<td>2.04</td>
<td>1.87</td>
<td>2.07</td>
</tr>
<tr>
<td>Early investigations (1927) by Richart</td>
<td>4.1</td>
<td>4.1</td>
<td>4.1</td>
</tr>
<tr>
<td>Balmer (1949)</td>
<td>4.5-7.0</td>
<td>4.5-7.0</td>
<td>4.5-7.0</td>
</tr>
<tr>
<td>Modified Kent and Park (1982)</td>
<td>2.47</td>
<td>1.81</td>
<td>2.67</td>
</tr>
<tr>
<td>Schickert and Winkler (1977)</td>
<td>3.04</td>
<td>2.50</td>
<td>3.18</td>
</tr>
<tr>
<td>Modified model of Schickert and Winkler</td>
<td>1.57</td>
<td>1.69</td>
<td>1.51</td>
</tr>
<tr>
<td>Fafitis and Shah (1985)</td>
<td>1.95</td>
<td>1.57</td>
<td>2.06</td>
</tr>
<tr>
<td>Chen and Mau (1990)</td>
<td>5.46</td>
<td>4.44</td>
<td>6.04</td>
</tr>
<tr>
<td>Yong, Malaka and Nawy (1988)</td>
<td>1.42</td>
<td>1.22</td>
<td>1.49</td>
</tr>
</tbody>
</table>

\[
\frac{f^c}{f_c} = 1 + 1.5\sqrt{\frac{\sigma_t}{f_c}} + 2\frac{\sigma_t}{f_c} - \rho_{sl}\frac{f_y}{f_c} \tag{4.6.4.2}
\]

\[
\frac{f_c}{f_c} = -1.254 + 2.254\sqrt{1 + \frac{7.94\sigma_t}{f_c}} - 2\frac{\sigma_t}{f_c} - \rho_{sl}\frac{f_y}{f_c} \tag{4.6.4.3}
\]

where:

\( \rho_{sl} \) = Proportion of the volume of longitudinal steel to the volume of concrete.

Results of Eqs. 4.6.4.2 and 4.6.4.3 are shown in Table 4.6.4.2. It can be concluded that Eq. 4.6.4.2 furnishes better results than Eq. 4.6.4.3, and both are in a good agreement.
with the experimental results. As the experimental results show, for an equal $f_y$ the strength ratio increases when the $D/t$ ratio increases. But the analytical results in most of the introduced methods show a decrease in strength ratio. The reason for this behaviour may be determined by the effects of dimensions which are not considered in the analytical methods. On the other hand, there is an increase in the strength factor for equal $D/t$ ratio when $f_y$ increases. Both the analytical and experimental results are more or less in agreement for lower $D/t$ ratios, but again the experimental results are not in agreement with the analytical results for larger $D/t$ ratios. This phenomenon needs further investigation. In fact statistical equations for structural characteristics of concrete cannot precisely predict the actual behaviour of a specific concrete. Therefore, results of the two equations, Eqs. 4.6.3.2 and 3, will be used for further investigation of composite columns in this study.

### 4.6.5 Stress-Strain Curve for the Steel Tubes

Stresses at the midheight of the steel tubes were computed according to the measured strains by the method that was explained in Chapter 3. For computing the stresses from measured strains, two sets of data for elastic specification of the steel tube were used, one was based on the apparent observations for elastic characteristics of the steel tube, and the other one was according to the classic values. As discussed in 4.6.2-3, based on the results obtained in Chapter 3, the computed stresses from the measured strains in the steel tube were different from the results observed from the tests. Accordingly, the evaluated stresses for the steel tubes in the following studies were corrected. As explained before, the main concept in the correction method is to compute the amount of correction at any specific strain throughout the loading history of the hollow sections. Then, the correction of stresses at different values for strain can be computed for filled sections, according to the required correction that was obtained from the previous stage (hollow section). The resulting curves for the stress-strain behaviour for the filled sections in the elastic and the plastic regions, before and after correction, are illustrated in Figs. 4.6.5.1,2,3,4.

A common feature for all of these curves is the sudden drop at the ultimate stage of loading. This is a quite explainable behaviour, based on the Von-Mises yield criterion for planar stresses. In the case of high minor stresses, the sudden drop at the yield plateau must occur because the steel should follow the yield criterion, which is an oblique ellipse in the plane of the principal stresses. On the other hand, the drop of steel stress in the longitudinal direction (major axis) for thick walled tubes is not as
high as for the thin walled tubes. This is the reason for a greater drop in the load deflection curve of confined concrete, at the point where yielding starts, Fig. 4.6.1.1. As a result, the drop of load in the load-shortening curve of the confined concrete is caused by the sudden drop in longitudinal stresses in the steel tube and furthermore, the drop is more considerable for thin walled tubes.

![Stress-strain curve for test No. A2II.](image_url)
Stresses are calculated according to the Apparent values of $E$ and $v$

Stresses are calculated according to the Classic values of $E$ and $v$

Corrected stresses ...

Fig. 4.6.5.2 and 3 Stress-strain curves for test Nos. B2 II (up) C2 II (down).
The yield behaviour of the steel tube is a very important point in the design of composite columns. The steel tube under axial load and internal pressure due to the concrete expansion in composite columns has a considerably short yield plateau, and does not show strain hardening. Therefore, the circumferential stresses in the steel tube may not always be an advantage for a circular hollow section filled with concrete, as a composite column.

4.6.6 Biaxial Stresses in the Steel Tubes

Based on the results obtained according to the computed stresses from measured strains, the curve of two major (longitudinal) and minor (circumferential) stresses can be illustrated as in Figs. 4.6.6.1 to 8. Once again, results are presented according to the apparent and classical values of elastic properties for each tube. The principal purpose of these curves is to show the proportional values of stresses in the two principal directions.
Considering the Figs. 4.6.6.3, 4, 7, 8 which represent the principal stresses in the full range, the yield behaviour of the steel tube in the longitudinal direction is quite considerable. According to the yielding criterion of Von-Mises, it is supposed that the steel tube, in the case of high circumferential stresses, must yield at lower longitudinal stresses than the uniaxial yield stress. But, as the results for the tests Nos. B2II and D2II show, the yield stress at the ultimate load of the tube is approximately equal to the uniaxial yield stress even at about 40% of yielding stress in the circumferential direction. This is an advantage in this kind of steel tube (high strength steel) that makes it more suitable for composite behaviour.

Fig. 4.6.6.1, 2 Normalised longitudinal stress versus normalised circumferential stress according to the apparent (A) and classic (C) values of E and v.

Fig. 4.6.6.3, 4 Normalised longitudinal stress versus normalised circumferential stress according to the apparent (A) and classic (C) values of E and v.
Fig. 4.6.6.5, 6 Normalised longitudinal stress versus normalised circumferential stress according to the apparent (A) and classic (C) values of E and v.

Fig. 4.6.6.7, 8 Normalised longitudinal stress versus normalised circumferential stress according to the apparent (A) and classic (C) values of E and v.

All of the curves are in fair agreement with the Von-Mises yield criterion, except the curves for thick walled tubes. As mentioned in Sec. 4.6.2 the computed stresses from the measured strains for the thick walled tubes are required to be amended for the effects of bulging deformation. This is done for the longitudinal direction, but, to get an accurate representation of strains for compressed steel tube filled with concrete or bare, it is necessary to test longer specimens. The longer specimens are influenced less by the supports conditions. This phenomenon needs further investigation.
4.7 DUCTILITY OF CONFINED CONCRETE IN THE STEEL TUBE

Considering the results of the load-axial shortening curves for the confined concrete, the specimens showed a considerable ductile behaviour. Ductility has an important role in the stability of structural frames, especially for the elements that are under compressive force systems. Therefore, a realistic criterion for evaluation of ductile behaviour of the elements such as composite hollow section columns is required for the necessary computation of the stability of structures.

According to the conventional methods, to evaluate the stability of a structural frames, the plastic ultimate load capacity of a structural element has to be estimated for different load situations. Although the exact value of the ultimate load for a specific dimension of a composite column can be determined by experimental tests, approximate values are suitable for the actual stability calculation methods. In this regard, according to the load-axial shortening curves obtained for the specimens, the plastic plateau of all of the specimens displayed a relatively horizontal line. But this horizontal line is in a lower position than the ultimate load capacity of the composite section (confined concrete). The drop of ultimate load capacity of the specimens depends on the dimensions of the steel tube and the material properties of both steel and concrete. Results of this study show that at about 3.5% axial deflection the specimens were in the plastic plateau, and therefore representing the ultimate position of the sections.

On the other hand the drop of ultimate load capacity of the sections with a thick walled steel tube is not as high as the drop for the thin walled steel tubes. All of the specimens had a slight strain hardening capacity. In other words, the plastic plateau has a slight ascending slope. Hence, the existing load at the longitudinal strain of 0.035, divided by the maximum load capacity of the composite section can be a reasonable measure of the ductility of the confined concrete. Moreover, this relationship may be changed for different axial strains according to the deformability of the structure. For instance, the ductility factor for an ideally elastic-plastic material would be equal to 1.0.

\[
D_\varepsilon = \frac{P_\varepsilon}{P_u}
\]  

(4.7.1)

where \(D_\varepsilon\) = ductility factor, \(P_\varepsilon\) = at any specific strain and \(P_u\) = ultimate axial load.

Based on the Eq. 4.7.1 for a strain of 3.5% in the longitudinal direction, the ductility factors for specimen Nos. A2(I,II) and C2(I,II), are 0.97 and 0.94 respectively, and for
specimen Nos. B2(I,II) and D2(I,II), which are thin walled tubes, the ductility factors are 0.82 and 0.77 respectively. As the ductility factors show, the composite sections with thick walled tubes furnish a more ductile behaviour than the composite sections with thin walled tubes.
Chapter Four The mechanical behaviour of confined concrete in steel tubes

SUMMARY

Intensive investigations about the effects of confinement on concrete have been carried out during the past decades. The main effort in these studies was to reduce or eliminate the brittle behaviour of concrete in structural elements, such as columns, beams, and, joints. Results of these studies revealed the beneficial effects of confinement on the ultimate load capacity and post yielding behaviour of the concrete.

The utilisation of confined concrete is in the compressive members such as reinforced concrete columns or composite hollow section columns. This study is concerned with the mechanical behaviour of confined higher strength concrete in circular hollow sections to establish the realistic relationships for evaluation of the compressive capacity of the confined higher strength concrete in a steel tube.

In the test reported herein, axial load is directly applied to the encased concrete with two loading discs; one on the top of the specimen and the other one at the bottom. Analytical and experimental results show that the transfer of load from the concrete to the steel tube occurs in a very short distance from the loading surface of the concrete, so that, the steel tube longitudinally yields. The yield plateau for longitudinal response of the steel tube is considerably shorter than for uniaxial loaded steel, and strain hardening was not observed. On the other hand, the steel tube approaches yield in the circumferential direction, that is completely an expected response in this case. More importantly, the behaviour of concrete at the post-yielding stage is different for all of the specimens. In low confinement cases, the yielding forms with the major cracks that change the shape of steel tube in an unsymmetrical shape, whereas, for high circumferential confinement this phenomenon was not observed.

Results of this study show that concrete can successfully be utilised as a very ductile material to furnish all necessary requirements in composite circular hollow sections under axial load.
CHAPTER FIVE

ULTIMATE AXIAL LOAD CAPACITY OF STOCKY STEEL CHS FILLED WITH HIGHER STRENGTH CONCRETE
(LOAD ON STEEL AND CONCRETE)

5.1 INTRODUCTION

Research on the structural behaviour of composite columns in form of hollow sections filled with concrete or mortar returns to about 60 years ago. The first purpose of this method was to protect the hollow sections from internal corrosion. Further investigations in this field manifested a considerable increase in the ultimate axial load capacity of these composite sections, as well as protection from internal corrosion. Later on, the other subjects which attracted keen attentions of investigators in field of structural engineering, were the effect of circumferential stresses from steel tube to the concrete core with different type of steel tube and concrete.

These studies had two principal points, the first point was the necessity of investigation on the different types of the steel tubes, and the second point was to investigate the structural behaviour of the concrete with different compressive strengths. The manufacturing methods for the steel tubes consist of different types of deformation procedures and welding of the seams as well as different material properties for steel. Likewise, structural characteristics of the concrete are also different, from low compressive capacity to a very high strength concrete.

The purpose of this study is to investigate the structural behaviour of steel circular hollow sections filled with higher strength concrete to form a composite column. Moreover, the effects of the heating process in the secondary operations on the steel tubes, for instance the epoxy coating or other similar methods for covering the outside of the tubes against the environmental hazards, are also investigated. To achieve the aims of this study the structural behaviour of the composite columns under axial compression has been modelled by the stub column tests.

To cover the maximum range of usual dimensions in structural engineering construction, the proportion of the outside diameter to the wall thickness equivalent to 18 and 35 has been considered. The maximum outside diameter is 168.3 mm and the
minimum is 114.3 mm. The wall thicknesses for the maximum outside diameter are 4.8 mm and 9.53 mm, and for the minimum outside diameter the wall thickness is 6.3 mm. The nominal yield strength of the steel tube is 350 MPa and the compressive capacity of the higher strength concrete is within the range of 60-80 MPa.

5.2 RESEARCH SIGNIFICANCE

Contemporary trends in structural design have tended toward the adoption of ultimate load predictions as the basis for design. The ultimate strength of circular hollow sections filled with concrete has been conveniently described by different codes of practice. But the generality of the recommended equations to include the wide range of material properties for steel tubes and concrete furnish considerable differences between the computed strength of composite columns with the actual strengths. Consequently, there exists a need for design rules that will consider the different behaviour of the materials. For instance, the mechanical behaviour of higher strength concrete is entirely different with the ordinary strength concrete. The higher strength concrete is more brittle than ordinary concrete.

On the other hand, restrictions for use of higher strength concrete in the filled circular hollow sections has not been specified as accurately as the recommended specifications for reinforced concrete. In the case of thin walled tubes, the brittle behaviour of the higher strength concrete and formation of major cracks in the concrete mass cannot be prevented by the steel tube. This behaviour of circular hollow sections filled with higher strength concrete is a very unpredictable behaviour and in some cases it may cause brittle failure in structural members.

This experimental research program, is concentrated on the prediction of ultimate load capacity of CHS filled with higher strength concrete and the structural behaviour of these elements in elastic and plastic stages of loading. Moreover, to achieve the most realistic model of these composite columns, two methods of loading, load on both steel tube and the concrete core, and load on steel tube only is considered. The discussion concerning the specimens with load on steel tube only will be presented in Chapter 6.

5.3 TEST PROCEDURE AND MATERIAL PROPERTIES

The test procedure is to make a number of stub columns which are axially loaded to failure. To insure that the experimental results are consistent, two stub columns of each type were made. Test columns were made from three different sizes of commercial
electrical resistance welded seam tubes (ERW), which are suitable for structural members with high internal pressure as well as axial stresses. To investigate the effect of heating in the process of epoxy coating of this kind of steel tube, a set of epoxy coated specimens has also been considered for this experiments.

The testing method for this set of experiment is entirely the same as previous investigations in Chapters 3, and 4. The geometrical specifications and material properties of the experimental program are given in Table 5.3.1.

<table>
<thead>
<tr>
<th>No.</th>
<th>Specimen</th>
<th>Outside Diameter d (mm)</th>
<th>Wall Thickness t (mm)</th>
<th>Length l (mm)</th>
<th>d/t</th>
<th>f'c (MPa)</th>
<th>Steel Grade</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>A4-I</td>
<td>114.3</td>
<td>6.30</td>
<td>500</td>
<td>18.14</td>
<td>69.6</td>
<td>C350</td>
</tr>
<tr>
<td>2</td>
<td>A4-II</td>
<td>114.3</td>
<td>6.30</td>
<td>500</td>
<td>18.14</td>
<td>69.6</td>
<td>C350</td>
</tr>
<tr>
<td>3</td>
<td>B4-I</td>
<td>168.3</td>
<td>4.80</td>
<td>500</td>
<td>35.06</td>
<td>61.3</td>
<td>C350</td>
</tr>
<tr>
<td>4</td>
<td>B4-II</td>
<td>168.3</td>
<td>4.80</td>
<td>500</td>
<td>35.06</td>
<td>61.3</td>
<td>C350</td>
</tr>
<tr>
<td>5</td>
<td>C4-I</td>
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<td>4.80</td>
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<td>71.6</td>
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<td>71.6</td>
<td>C350</td>
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<tr>
<td>7</td>
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<td>500</td>
<td>35.06</td>
<td>71.4</td>
<td>C350-</td>
</tr>
<tr>
<td>8</td>
<td>D4-II</td>
<td>168.3</td>
<td>4.80</td>
<td>500</td>
<td>35.06</td>
<td>71.4</td>
<td>C350-</td>
</tr>
</tbody>
</table>

where:

\( \sim \) = Epoxy coated tube.

\( f'c \) = Unconfined compressive strength of concrete.

### 5.4 TEST RESULTS AND DISCUSSION

#### 5.4.1 Axial Load-Shortening Behaviour

The ultimate load behaviour of a composite circular hollow section, regardless of the joints effects can be modelled by the stub column tests. Theoretically, this model represents the behaviour of the composite column, between two horizontal surfaces with equal axial deformation in each surface.

Although the horizontal friction between the top and the bottom ends of the specimen with the surface of the testing machine will make some differences between the specimen and real composite column, nevertheless, considering a specimen with enough height, will reduce the constraint effects of the supports.

The curves of axial load-shortening behaviour for all of the specimens are given in Fig. 
5.4.1.1. One of the important points in these curves is the drop of the load of the specimens after the peak load. Two main effective parameters can be recognised from the curves. First parameter is the effect of the wall thickness, and the second parameter is the dimensions of the specimens. For thin walled tubes the drop of axial load after the peak load is much greater than that of the thick walled tubes. On the other hand, the specimen Nos. A4 (I,II), that have the smallest diameter, have a lower drop in load in comparison with the specimen Nos. C4 (I,II).

![Axial load-shortening curve.](image)

The other important difference between the composite thin walled tubes and thick walled tubes is the formation of major cracks in the confined concrete. As is shown in Photo 5.4.1.1, formation of major cracks in the concrete core produces a few slip surfaces that make an irregular deformation pattern for the steel tube. This irregularity of lateral deformation for thin walled tubes is more considerable than for the thick walled tubes. In addition, the irregular lateral deformation of the specimens is a direct effect of the failure pattern of concrete. The failure pattern of the unconfined higher
strength concrete is an explosive failure with major crack pattern development. Although in the case of a confined higher strength concrete the failure pattern is not explosive, nevertheless, most of the cases are accompanied by major cracks.

Considering the behaviour of higher strength concrete in the composite circular hollow sections to achieve an efficient use of material, it is necessary to use internal reinforcement for preventing the formation of major cracks in the concrete core. By such a treatment, the cracks will be distributed in the concrete mass. Moreover, the drop of the ultimate load will be reduced.

To compare the results of this study with the existing results in the literature, the ultimate load capacity of the specimens and the minimum ductility factor as defined in 4.7 is shown in Table 5.4.1.1.

<table>
<thead>
<tr>
<th>Specimen</th>
<th>$P_u$ (kN)</th>
<th>$D_{\min}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>A4-I</td>
<td>1750</td>
<td>0.94</td>
</tr>
<tr>
<td>A4-II</td>
<td>1745</td>
<td>0.94</td>
</tr>
<tr>
<td>B4-I</td>
<td>3190</td>
<td>0.69</td>
</tr>
<tr>
<td>B4-II</td>
<td>3190</td>
<td>0.76</td>
</tr>
<tr>
<td>C4-I</td>
<td>4510</td>
<td>0.92</td>
</tr>
<tr>
<td>C4-II</td>
<td>4600</td>
<td>0.90</td>
</tr>
<tr>
<td>D4-I</td>
<td>3400</td>
<td>0.70</td>
</tr>
<tr>
<td>D4-II</td>
<td>3395</td>
<td>0.72</td>
</tr>
</tbody>
</table>

Considering the post peak load behaviour of the composite circular sections when the axial load is directly applied to the concrete core, (Chapter 4), the differences of residual axial load for all of those specimens with the specimens in this Chapter is negligible. Hence, in both of the cases, the residual load of the composite section can be achieved regardless of the loading situation, Fig. 5.4.1.2.
Chapter 5 ultimate axial load capacity of stocky steel CHS filled with higher strength concrete...

5.4.2 Apparent Modulus of Elasticity and Poisson’s Ratio

As discussed in Chapter 4, the elastic parameters for the steel tube can be determined from the early stages of stress-strain history of the specimens. Considering the results obtained in Fig. 5.4.2.1, the modulus of elasticity is approximately the same as the results given in Chapters 3 and 4. Once again the apparent modulus of elasticity for thick walled tubes are higher than that of the thin walled tubes. This difference shows that the steel tube in the composite section behaves as a hollow section and the interaction between steel tube and concrete core (in the elastic range) is negligible.

Therefore, the evaluation of the maximum allowable load in the range of the service load can be computed by Eq. 5.4.2.1. As the equation shows, the steel tube and the concrete core has been considered as a separate structural member.

\[
P_u = f_{st} A_{st} + f_c A_c \tag{5.4.2.1}
\]

where:
$P_U$ = Allowable load on composite circular hollow section.
$f_{st} = $ Allowable stress on steel tube.
$f_c = $ Allowable stress on concrete core.
$A_{st} = $ Cross sectional area of steel tube.

$A_C = $ Cross sectional area of concrete core.

\[
\sigma = 0.294\varepsilon + 0.321 \quad r^2 = 1.000
\]
\[
\sigma = 0.227\varepsilon + 1.103 \quad r^2 = 1.000
\]
\[
\sigma = 0.309\varepsilon + 1.543 \quad r^2 = 1.000
\]
\[
\sigma = 0.220\varepsilon - 0.247 \quad r^2 = 1.000
\]

Fig. 5.4.2.1 Stress strain curve in elastic range for all of the specimens.

Likewise, the lateral deflection at the elastic range can be presented by the apparent Poisson’s ratio. The fitted line for each specimen is given in Fig. 5.4.2.2. Same as the apparent Poisson’s ratio in Chapter 4, the values of the Poisson’s ratio for all of the specimens are more or less equal. This behaviour shows a similar pattern to displacements for all of the specimens regardless of the differences in dimensions. In addition, the values of Poisson’s ratio for all of the specimens are close to the classic value of the Poisson’s ratio (0.27 - 0.29), that once again illustrates the independent behaviour of the steel tube and concrete core.
Considering the results in Chapter 4, the Poisson’s ratio in the elastic range is much higher than the results in this Chapter. Therefore, the design of structural joints should consider special joints where the loads from the girders or slabs are directly transferred to the concrete core. An example of such a joint is represented in Appendix C.

### 5.4.3 Stress-Strain Curve for Confined Concrete

The stress-strain curves of concrete cores in the composite columns are obtained according to the method that was explained in 4.6.3. For comparing the effects of corrections on the concrete stresses the corrected curves and raw curves for each of the specimens are given in Figs. 5.4.3.1 to 4.

The common point in all of the stress-strain curves is the considerable increase of the concrete ultimate strength that is more or less similar to the results in Chapter 4. This behaviour ensures the increase of concrete strength (at the ultimate stage) even in the case of simultaneous load on the steel tube and concrete core.

Although the method for computing the stresses in the concrete core is accurate enough, nevertheless, the irregular lateral deformation of the specimens due to the major cracks in the concrete core affects the measured strains. Therefore, the few last points in the
stress-strain curves of the concrete core are inaccurate. To evaluate the ultimate compressive capacity of the concrete in this study the corresponding stress at the 0.5% of longitudinal strain has been considered. This is an approximate method to evaluate the proof stress of the materials where no well defined yield plateau exists. Results for ultimate stress in concrete in core are shown in Table 5.4.3.1.

Table 5.4.3.1 Compressive strength of concrete core according to the stress-strain curves by apparent and classic values of elastic parameters.

<table>
<thead>
<tr>
<th>Specimen No.</th>
<th>$f'_c(A)$ (MPa)</th>
<th>$f'_c(C)$ (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>A4 II</td>
<td>93.8</td>
<td>93.4</td>
</tr>
<tr>
<td>B4 II</td>
<td>110.7</td>
<td>110.6</td>
</tr>
<tr>
<td>C4 II</td>
<td>108.9</td>
<td>111.5</td>
</tr>
<tr>
<td>D4 II</td>
<td>105.2</td>
<td>105.2</td>
</tr>
</tbody>
</table>

where:

$f'_c(A)$ = Compressive capacity of concrete by apparent values for elastic parameters.

$f'_c(C)$ = Compressive capacity of concrete by classic values for elastic parameters.

The following abbreviations have been used in Figs. 5.4.3.1 to 4:

RC A = Raw stress-strain curve of concrete by apparent elastic parameters.

RC C = Raw stress-strain curve of concrete by classic elastic parameters.

CC A = Corrected stress-strain curve of concrete by apparent elastic parameters.

CC C = Corrected stress-strain curve of concrete by classic elastic parameters.
Fig. 5.4.3.1 Corrected and raw stress-strain curve for concrete, for test No. A4 II.

Fig. 5.4.3.2 Corrected and raw stress-strain curve for concrete, for test No. B4 II.
Chapter 5 ultimate axial load capacity of stocky steel CHS filled with higher strength concrete...

Fig. 5.4.3.3 Corrected and raw stress-strain curve for concrete, for test No. C4 II.

Fig. 5.4.3.4 Corrected and raw stress-strain curve for concrete, for test No. D4 II.
5.4.4 Confinement Pressure on Concrete

The lateral stresses on the concrete core according to the Eq. 4.6.1.1 for all of the specimens are illustrated if Fig. 5.4.4.1. These figures are representing the radial stresses on the concrete core, according to the apparent and classic values for modulus of elasticity and Poisson's ratio.

Fig. 5.4.4.1 Results for axial stress versus radial stress on concrete core.
As in the previous study on the behaviour of confined concrete in the steel tubes, the increase of confinement pressure on the ultimate load capacity of concrete core is limited to a specific value which depends on the unconfined compressive capacity of the concrete and the dimensions of the steel tube as well as material properties.

To determine the maximum confinement pressure according to the tests results, the radial stress at the ultimate compressive stress of the concrete core has to be computed. Considering the obtained results in Table 5.4.3.1 The ultimate radial stresses can be readily obtained from the data charts of Fig. 5.4.4.1. The results for the ultimate radial stress are given in table 5.4.4.1.

Table 5.4.4.1 Ultimate radial stress of concrete core according to the data charts of Fig. 5.4.4.1.

<table>
<thead>
<tr>
<th>Specimen No.</th>
<th>$\sigma_{r}(A)$ (MPa)</th>
<th>$\sigma_{r}(C)$ (MPa)</th>
<th>RSTY (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>A4 II</td>
<td>16.4</td>
<td>14.8</td>
<td>45.4</td>
</tr>
<tr>
<td>B4 II</td>
<td>6.7</td>
<td>6.1</td>
<td>23.3</td>
</tr>
<tr>
<td>C4 II</td>
<td>13.7</td>
<td>23.4</td>
<td>53.0</td>
</tr>
<tr>
<td>D4 II</td>
<td>3.6</td>
<td>3.2</td>
<td>26.4</td>
</tr>
</tbody>
</table>

where:

$\sigma_{r}(A)$ = Radial compressive stress according to the apparent values of elastic parameters.

$\sigma_{r}(C)$ = Radial compressive stress according to the classic values of elastic parameters.

RSTY = Radial stress for circumferentially yielded steel tube (Yield stress for steel in compression and tension are considered to be equal)

To investigate the relationship between the ultimate strength of confined concrete and the actual radial stress, the Eqs by Cai, Schickert and Winkler was used. The observed compressive strength in Table 5.4.3.1 and the corresponding radial stress in Table 5.4.4.1 have been considered at this stage. The computed results for all of the specimens are given in Table 5.4.4.2.

Table 5.4.4.2 Ultimate radial stress of concrete core according to the data charts of Fig. 5.4.4.1.

<table>
<thead>
<tr>
<th>Specimen No.</th>
<th>$f'_c$ (MPa)</th>
<th>Observed $f'_c$ (MPa)</th>
<th>Observed $\sigma_r$ (MPa)</th>
<th>$f'_c$ by Cai [Eq. 2.1.1] (MPa)</th>
<th>$f'_c$ by Schickert, Winkler [Eq. 2.1.21] (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>A4 II</td>
<td>69.6</td>
<td>93.8</td>
<td>16.4</td>
<td>153.1</td>
<td>145.7</td>
</tr>
<tr>
<td>B4 II</td>
<td>61.3</td>
<td>110.7</td>
<td>6.7</td>
<td>105.1</td>
<td>98.6</td>
</tr>
<tr>
<td>C4 II</td>
<td>71.6</td>
<td>108.9</td>
<td>13.7</td>
<td>146.0</td>
<td>139.0</td>
</tr>
<tr>
<td>D4 II</td>
<td>71.4</td>
<td>105.2</td>
<td>3.6</td>
<td>102.6</td>
<td>93.7</td>
</tr>
</tbody>
</table>
Results of the analytical equations by Cai, Schickert and Winkler are in a fair agreement with the results obtained in this study. In fact, considering the secondary effects on the test results of unconfined compressive strength of the higher strength concrete and the effects of the dimensions, the results are acceptable as a basis for design purposes.

5.4.5 Stress-Strain Behaviour of the Steel Tube

The stress-strain curves for the steel tubes are computed according to the discussed method in Chapters 3, and 4 and are shown in Figs. 5.4.5.1 to 4.

Converse to the stress-strain curves of confined concrete in Chapter 4, the drop of the longitudinal stresses in the steel tube for all of the cases are not observed in this case. The main reason for this situation is the lower circumferential stresses in steel tube for the case of load on both, the steel tube and the concrete core. But, this behaviour of steel tube has not significant effect on the total behaviour of the composite column. As the curves of load-axial shortening show for both of the cases of load on concrete only, and load on both the steel tube and the concrete, the drop of load after the peak point is higher for the second case.

In addition, for all of the cases, the longitudinal stresses in the steel tubes approach the maximum compressive stress, regardless of the dimensions of the steel tube. The changes in the maximum stress in the steel tubes from the corrected curves compared with the computed apparent yield stresses are given in Table 5.4.5.1.

<table>
<thead>
<tr>
<th>Specimen No.</th>
<th>( f_y (A) ) (MPa)</th>
<th>( f_y^*(A) ) (MPa)</th>
<th>( f_y (A) / f_y^*(A) )</th>
</tr>
</thead>
<tbody>
<tr>
<td>A4 II</td>
<td>389.0</td>
<td>353.0</td>
<td>0.91</td>
</tr>
<tr>
<td>B4 II</td>
<td>397.0</td>
<td>359.0</td>
<td>0.91</td>
</tr>
<tr>
<td>C4 II</td>
<td>441.0</td>
<td>441.0</td>
<td>1.00</td>
</tr>
<tr>
<td>D4 II</td>
<td>449.0</td>
<td>425.0</td>
<td>0.95</td>
</tr>
</tbody>
</table>

where:

\( f_y (A) \) = Yield stress for the steel tube by apparent values for elastic parameters.

\( f_y^*(A) \) = Yield stress for the steel tube for the filled section and load on concrete and steel according to the classic values for elastic parameters.

As a result, in all of the cases the maximum yield stress for the steel tube is not achievable in the case of the composite circular hollow section.
Fig. 5.4.5.1 Stress-strain curve of steel tube drawn for the apparent and classic values of $E$ and $\nu$ for test No. A4 II.

Fig. 5.4.5.2 Stress-strain curve of steel tube drawn for the apparent and classic values of $E$ and $\nu$ for test No. B4 II.
Fig. 5.4.5.3 Stress-strain curve of steel tube drawn for the apparent and classic values of $E$ and $v$ for test No. C4 II.

Fig. 5.4.5.4 Stress-strain curve of steel tube drawn for the apparent and classic values of $E$ and $v$ for test No. D4 II.
5.4.6 Biaxial Stresses in the Steel Tubes

The intensity of the circumferential stresses in the steel tube are necessary for computing the radial stresses on the concrete core. In addition, the biaxial stresses of the steel tube will give the clear ideas of the failure pattern for the steel tube. Accordingly the diagrams of the biaxial stresses with the same method as was discussed in Chapters 3, and 4, for the apparent values of the elastic parameters are presented in Fig. 5.4.6.1. To simplify the figure, all of the curves are drawn in the same chart and a quarter of the Von-Mises yield criterion with a bold line is given in this figure.

Fig. 5.4.6.1 Normalised longitudinal stress versus normalised circumferential stresses according to the apparent values of elastic parameters.

Regardless of the results for the specimen No. C4 II, the other results show a very low circumferential stress before longitudinal yield of the steel tube. Once again the low value of the circumferential stresses denote that the steel tube does not apply radial stress to the concrete core, before complete yield in longitudinal direction.

The circumferential stress after the yield point of the steel has a significant increasing rate. This phenomenon shows that the Poisson's ration of concrete has also a considerable increase, therefore, the confinement pressure on the concrete core is much more higher after longitudinal yielding of the steel tube.
5.4.7 Poisson’s Ratio in the Complete Loading History

The curves of longitudinal stresses versus Poisson’s ratios which are the proportion of the circumferential strain to the longitudinal strain are given in Fig. 5.4.7.1. The common point for all of the curves is the sudden increase of Poisson’s ratio when the steel tube yield longitudinally.

Fig. 5.4.7.1 Poisson’s ratio for the whole load history of the specimens.

This behaviour is due to the formation of cracks in the concrete mass that increases the
volume of concrete. Considering the behaviour of the specimens B4-II and D4-II, the expansion of the concrete core is more effective than the other specimens.

On the other hand the two thick walled specimens A4-II, and, C4-II, have the same pattern of behaviour. But, the value of Poisson’s ratio in the specimen C4-II is higher than that of the A4-II. This difference shows that the proportion of dimensions for the composite column is also effective on the rate of confinement. In fact, the reason of this behaviour is mainly due to the effects of the loading edges and the period of the longitudinal deformation in the steel tube. Hence, a longer specimen for the thick walled tube may have different behaviour.

5.5 ANALYTICAL EQUATIONS FOR COMPUTING THE ULTIMATE LOAD OF THE CIRCULAR HOLLOW SECTIONS FILLED WITH CONCRETE.

Early investigations on the axial load behaviour of the CHS filled with concrete Ref. [42], showed that the recommended equations for computing the Permissible load, has a large margin of safety, when compared with the ultimate load capacity. On the other hand, the equations like Eq. 2.2.4, under-estimate the ultimate load capacity of the sections. With this regard, the correct estimation of the ultimate load capacity of these kind of composite sections should be based on the beneficial effects of confinement on the concrete core and the lesser strength for the steel tube due to the circumferential stresses.

Most of the available equations for evaluation of the ultimate load capacity of filled circular hollow sections only consider the beneficial effects of confinement on the concrete core and disregard the decrease of steel longitudinal strength. But, the recommended relationships for the evaluation of ultimate load capacity of these sections in CEB Ref. [18], seems to cover all effective parameters. General form of the equation is given in Eq 5.5.1.

\[
P_u = A_c\left\{\left[f'_c + \eta_1\left(t/d\right)f_y\right]/\gamma_{mc}\right\} + A_s\left[\eta_2 f_y/\gamma_{ms}\right]
\]

(5.5.1)

where:

\[\eta_1, \eta_2 = \text{Constant values listed in table 5.5.1.}\]
\[\gamma_{mc}, \gamma_{ms} = \text{Material partial safety factors of concrete and structural steel, that in this study are assumed to be equal 1.0.}\]
Chapter 5 ultimate axial load capacity of stocky steel CHS filled with higher strength concrete...

Table 5.5.1 The values of factors $\eta_1$, and $\eta_2$.

<table>
<thead>
<tr>
<th>$l/d$</th>
<th>$\eta_1$</th>
<th>$\eta_2$</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>9.78</td>
<td>0.76</td>
</tr>
<tr>
<td>5</td>
<td>6.60</td>
<td>0.80</td>
</tr>
<tr>
<td>10</td>
<td>3.94</td>
<td>0.85</td>
</tr>
<tr>
<td>15</td>
<td>1.86</td>
<td>0.90</td>
</tr>
<tr>
<td>20</td>
<td>0.49</td>
<td>0.95</td>
</tr>
<tr>
<td>25</td>
<td>0.00</td>
<td>1.00</td>
</tr>
</tbody>
</table>

On the other hand, the proposed equations for computing the ultimate axial load capacity proposed by the Draft for Development of Eurocode 4 [Ref. 64] is exactly the same as the Eq. 5.5.1 but considers a lower effect of confinement than the Eq. 5.5.1. The difference in this new proposal is just in determining the parameter $\eta_1$ that magnifies the concrete axial load capacity. The detailed form of the proposed equation in Draft Eurocode 4 is as follows:

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

(5.5.2)

$$
\eta_{20} = 0.25 (3 + 2\lambda)
$$

(5.5.3)

These values can also be readily obtained from the Table 5.5.2

Table 5.5.2 values of parameters $\eta_{10}$ and $\eta_{20}$ for $\lambda$.

| $\lambda$ | 0.0 | 0.1 | 0.2 | 0.3 | 0.4 | $\geq 0.5$
<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>$\eta_{10}$</td>
<td>4.9</td>
<td>3.22</td>
<td>1.88</td>
<td>0.88</td>
<td>0.22</td>
<td>0.0</td>
</tr>
<tr>
<td>$\eta_{20}$</td>
<td>0.75</td>
<td>0.8</td>
<td>0.85</td>
<td>0.9</td>
<td>0.95</td>
<td>1.0</td>
</tr>
</tbody>
</table>

The values of $\eta_1$ and $\eta_2$ for $0 < e < d/10$ where “e” is the eccentricity of axial load, are as follows:

$$
\eta_1 = \eta_{10} (1 - 10e/d)
$$

(5.5.4)

$$
\eta_2 = \eta_{20} + (1 - \eta_{20}) (10e/d)
$$

(5.5.5)
For \( e > d/10 \), \( \eta_1 = 0.0 \) and \( \eta_2 = 1.0 \).

The value of the effective stiffness of the composite section has to be determined as follows:

\[
(EI)_c = E_a I_a + 0.8 E_c I_c + E_s I_s
\]

where:

\( I_a, I_c \) and \( I_s \) are the second moments of area for the considered bending plane of the structural steel, the concrete (assumed to be uncracked), and the reinforcement, respectively.

\( E_a \) and \( E_s \) are the modulus of elasticity for the structural steel and the reinforcement.

The next step is the determination of critical axial load, \( N_{cr} \), by Eq. 5.5.7.

\[
N_{cr} = \frac{\pi^2 (EI)_c}{l^2}
\]  

(5.5.7)

And finally the value of \( \lambda \) can be computed as follows:

\[
\lambda = \sqrt{\frac{N_{pl,R}}{N_{cr}}}
\]  

(5.5.8)

where \( N_{pl,R} \) is the value of \( P_u \) according to 5.5.1 including the partial factors of safety are equal to 1.0, the value of \( \eta_2 \) is taken as 1.0, and the value of \( \eta_1 \) is equal to 0.0.

Considering the geometrical and material specifications of the test specimens, the necessary parameters for computing the ultimate axial load capacity are as follows:

Table 5.5.3 Specifications of the tests specimen for computing the ultimate load capacity by Draft Eurocode 4.

<table>
<thead>
<tr>
<th>Specimen No.</th>
<th>Specifications</th>
</tr>
</thead>
<tbody>
<tr>
<td>( f_y ) (MPa)</td>
<td>( f'c ) (MPa)</td>
</tr>
<tr>
<td>A4 II</td>
<td>389.2</td>
</tr>
<tr>
<td>B4 II</td>
<td>397.4</td>
</tr>
<tr>
<td>C4 II</td>
<td>441.4</td>
</tr>
<tr>
<td>D4 II</td>
<td>450.0</td>
</tr>
</tbody>
</table>
To compare the experimental results for the ultimate load capacity of the specimens in this study with the available equations for the evaluation of the ultimate load capacity, the estimated values by Eqs. 5.5.1 and 5.2.4 to 6, plus experimental results are given in Table 5.5.4.

Considering the estimations obtained for the ultimate load capacity of the sections, none of the proposed equations are have a safe evaluation for the ultimate load capacity, especially for the thick walled tubes. As was discussed in Chapter 4, these equations are based on the complete circumferential yield of the steel tubes.

Table 5.5.4 Comparative results on the ultimate load capacity of CHS composite stub column tests.

<table>
<thead>
<tr>
<th>Specimen No.</th>
<th>Ultimate load capacity $P_u$ (kN)</th>
<th>Eq. 5.5.1</th>
<th>Eq. 2.2.4</th>
<th>Eq. 2.2.6</th>
<th>Eq. 2.2.5</th>
<th>Draft Eurocode 4</th>
<th>Test results</th>
</tr>
</thead>
<tbody>
<tr>
<td>A4 II</td>
<td>2378.0</td>
<td>1366.0</td>
<td>2161.0</td>
<td>2120.0</td>
<td>1483.0</td>
<td>1750.0</td>
<td></td>
</tr>
<tr>
<td>B4 II</td>
<td>3747.0</td>
<td>2191.0</td>
<td>3170.0</td>
<td>3378.0</td>
<td>2608.0</td>
<td>3190.0</td>
<td></td>
</tr>
<tr>
<td>C4 II</td>
<td>6342.0</td>
<td>3349.0</td>
<td>5446.0</td>
<td>5179.0</td>
<td>3525.0</td>
<td>4510.0</td>
<td></td>
</tr>
<tr>
<td>D4 II</td>
<td>4279.0</td>
<td>2519.0</td>
<td>3626.0</td>
<td>3881.0</td>
<td>2965.0</td>
<td>3400.0</td>
<td></td>
</tr>
</tbody>
</table>

The results obtained for the radial stresses on the concrete core show that the value of circumferential stresses in the steel tube is always less than the yielding strength at the ultimate stage of loading (at the approximate value of $\varepsilon_l = 0.5\%$), where $\varepsilon_l$ is the longitudinal strain. Therefore, the expected beneficial effects of confinement is definitely less than the considered values in Eq. 5.5.1, and, 2.2.5-6.

On the other hand, the reduction factor for the steel compressive strength in Eq. 5.5.1, is also based on the circumferential yield of the steel tube and has to be modified. Obviously, the modifications for the aforementioned relationships have to be based on a large number of tests results.

The ultimate axial load capacity obtained according to the proposed method in Draft Eurocode 4 [Ref. 64] gives more reliable results than other of the methods. The factor of safety seems to be in an acceptable range, and the confinement effect on the concrete strength is less significant than the previously proposed method by CEB [Ref. 18].
5.6 PROPOSED EQUATION FOR COMPUTING THE ULTIMATE LOAD CAPACITY OF SHORT CIRCULAR HOLLOW SECTIONS FILLED WITH CONCRETE

To simplify the calculation method for the evaluation of ultimate load capacity of a circular hollow section filled with concrete the total relationship according to the Eq. 2.2.4 is reduced to two main parameters of \( \frac{P_u}{A_c f'_c} \), and \( \frac{A_s f_y}{A_c f'_c} \). Based on the available test results in the literature, the relationship between these two parameters is more or less linear. The results of 46 experimental tests for stub columns and the best linear relationship is given in Fig. 5.6.1. Therefore, the ultimate load capacity can be readily obtained by a simple equation which is the best approximation for these results.

Based on the coordinate system in Fig. 5.5.1, the corresponding equation is as follows:

\[
\frac{P_u}{A_c f'_c} = 1.437 \frac{A_s f_y}{A_c f'_c} + 1.065
\]  

(5.6.1)

or:

\[
P_u = 0.9A_s f_y + A_c [1.065f'_c + 2.148(\frac{1}{d})f_y]
\]

(5.6.2)

Comparing the Eq. 5.6.2 with the similar equation (Eq. 5.5.1) the increasing factor for the concrete compressive strength is considerably lower than the recommended value in Eq. 5.5.1 for the dimensions in this study (short column). This is the main reason for over estimation of this equation which is a part of the recommendations about
composite columns in the CEB (European Committee).

Computed results according to the Eq. 5.6.1 or 2 are given in Table 5.6.1. The obtained ultimate load capacities are in a good agreement with the test results.

<table>
<thead>
<tr>
<th>Specimen No.</th>
<th>Ultimate load capacity $P_u$ (kN)</th>
<th>Test results (kN)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Eq. 5.6.1or 2</td>
<td></td>
</tr>
<tr>
<td>A4 II</td>
<td>1750.0</td>
<td>1750.0</td>
</tr>
<tr>
<td>B4 II</td>
<td>2697.0</td>
<td>3190.0</td>
</tr>
<tr>
<td>C4 II</td>
<td>4347.0</td>
<td>4510.0</td>
</tr>
<tr>
<td>D4 II</td>
<td>3010.0</td>
<td>3400.0</td>
</tr>
</tbody>
</table>
5.7 SUMMARY

An experimental study on the behaviour of short circular hollow section filled with higher strength concrete has been carried out. The dimensions of the test specimens are the same as the studies on the behaviour of confined concrete in Chapter 4.

Results of this study show that the proposed equation by Cai has a good agreement with the results of the experiments in this study. On the other hand the recommended equation by CEB does not accurately predict the compressive capacity of the composite sections. In addition, the suggested equation for the evaluation of the compressive capacity of a composite section has a good agreement with the experimental results in the literature and results in this study.

Furthermore, the results obtained from the confining stresses on the concrete show that the effect of the confinement of the concrete core, before the longitudinal failure of the steel tube, is small. Hence, the analytical model (Fig. 4.6.1.3), which is the basis for most of the empirical equations for computing the maximum axial load, is not generally correct.
CHAPTER SIX

ULTIMATE AXIAL LOAD CAPACITY OF STOCKY STEEL CHS FILLED WITH HIGHER STRENGTH CONCRETE

(LOAD ON STEEL TUBE)

6.1 INTRODUCTION

Conventional joints in structural frames connecting the beams to columns are designed so that the vertical loads are directly applied to the outer surface of the columns. In the case of circular hollow sections filled with concrete, the steel tube has to be able to transfer the loads from the beams to the whole composite column. Regardless of the local behaviour of the steel tube at the ultimate stage, the transfer of axial load from the steel tube to the concrete core is also an important issue that is investigated in this part.

To study the behaviour of the composite sections in the real situation of loading for the composite columns, the same procedure as in previous studies was considered for the experimental program. The differences in this part are principally the loading method and the supports conditions. In this study, the specimens were made as before, but the length of the concrete core was shorter than the length of the steel tube. The vertical load was directly applied to the steel tube at the upper end, so that the concrete core was not in touch with the loading surface.

The dimensions and material properties for this new set of experiments have the same specifications of the specimens in Chapter 5.

6.2 TEST PROCEDURE AND MATERIAL PROPERTIES

As in previous studies in Chapters 4 and 5, the test procedure is to make a number of stub columns which are axially loaded to failure. To insure that the experimental results were consistent, two stub columns of each type were made. Test columns were made from three different sizes of commercial electrical resistance welded seam tubes (ERW). To investigate the effect of heating in the process of epoxy coating of this kind of steel tubes, a set of two epoxy coated specimens has also been considered for these experiments.
The testing method for this set of experiments is entirely the same as previous investigations in Chapters 3 and 4. The geometrical specifications and material properties of the experimental program are given in Table 6.2.1. The experimental set-up is shown in Fig. 6.2.1. A gap of 20 mm was considered to simulate the case of the applied load on steel tube only. This gap is long enough for the study of elastic behaviour of composite section without direct load on concrete core, but short enough.

Table 6.2.1 Specifications of the test specimens.

<table>
<thead>
<tr>
<th>No.</th>
<th>Specimen</th>
<th>Outside Diameter d(mm)</th>
<th>Wall Thickness t(mm)</th>
<th>Length l(mm)</th>
<th>d/t</th>
<th>$f'_c$ (MPa)</th>
<th>Steel Grade</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>A3-I</td>
<td>114.3</td>
<td>6.30</td>
<td>500</td>
<td>18.14</td>
<td>69.6</td>
<td>C350</td>
</tr>
<tr>
<td>2</td>
<td>A3-II</td>
<td>114.3</td>
<td>6.30</td>
<td>500</td>
<td>18.14</td>
<td>69.6</td>
<td>C350</td>
</tr>
<tr>
<td>3</td>
<td>B3-I</td>
<td>168.3</td>
<td>4.80</td>
<td>500</td>
<td>35.06</td>
<td>68.9</td>
<td>C350</td>
</tr>
<tr>
<td>4</td>
<td>B3-II</td>
<td>168.3</td>
<td>4.80</td>
<td>500</td>
<td>35.06</td>
<td>68.9</td>
<td>C350</td>
</tr>
<tr>
<td>5</td>
<td>C3-I</td>
<td>168.3</td>
<td>9.53</td>
<td>500</td>
<td>17.66</td>
<td>68.9</td>
<td>C350</td>
</tr>
<tr>
<td>6</td>
<td>C3-II</td>
<td>168.3</td>
<td>9.53</td>
<td>500</td>
<td>17.66</td>
<td>68.9</td>
<td>C350</td>
</tr>
<tr>
<td>7</td>
<td>D3-I</td>
<td>168.3</td>
<td>4.80</td>
<td>500</td>
<td>35.06</td>
<td>67.1</td>
<td>C350-</td>
</tr>
<tr>
<td>8</td>
<td>D3-II</td>
<td>168.3</td>
<td>4.80</td>
<td>500</td>
<td>35.06</td>
<td>67.1</td>
<td>C350-</td>
</tr>
</tbody>
</table>

where:

$\sim$ = Epoxy coated tube.

$f'_c$ = Unconfined compressive strength of concrete.

6.3 TEST RESULTS AND DISCUSSION

6.3.1 Load Axial Shortening Behaviour

The curves of axial load-shortening behaviour for all of the specimens are given in Fig. 6.3.1.1. As was expected, each specimen has two yielding plateaus: the first one represents the behaviour of the steel tube, and the second denotes the ultimate load.
behaviour of the composite section. One of the important points in these curves is the drop of ultimate load of the specimens at the first peak. The thin walled tubes have a considerable drop past the ultimate load, whereas the thick walled specimens do not show a softening behaviour. This behaviour depends entirely on the local plastic buckling phenomenon that is more likely to happen for the thin walled specimens.

On the other hand after the complete contact for the loading surface with the top end of the concrete core, the section behaves completely as a composite column. But a difference with the previous studies of the composite section, which is common for all of the new set of specimens, is the sudden decrease of the load that was accompanied by a crushing noise. The position of this behaviour, in the direction of axial shortening axis, is more or less constant for all of the specimens, and is shown by a dashed line CN in Fig. 6.3.1.1. In addition, the approximate starting point of the composite behaviour of the specimens has also a constant position. The position of this behaviour is shown by dashed line CB.

The sudden decrease of the peak load in the composite sections is due to the crushing behaviour of the concrete core. In the top part of the specimens, the steel tube is completely separated from the concrete core, therefore, the concrete core behaves as an unconfined concrete.

The other important behaviour for these specimens is the softening characteristics which are more or less common for all of the tests. The drop of load after the second peak is not as high as that for the specimens where the load was applied on the concrete core or the concrete core and steel tube. It denotes that the ductility factor (Chapter 4, 4.7) is higher in this case. To represent a rough idea for the ductile behaviour of the specimens, Table 6.3.1 is given for the ductility of composite sections for the different forms of loading.

As a result, the drop of ultimate load capacity for the composite sections is due to the local plastic buckling of the steel tube. But converse to the bare tubes, the local buckling for the composite sections is not a periodic behaviour and the ultimate load approaches to a limiting value.
Table 6.3.1 Comparative results for the ductility of the specimens in different loading conditions.

<table>
<thead>
<tr>
<th>Loading method</th>
<th>Ductility factor $D_{\text{min}}$</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>A3- (I,II)</td>
</tr>
<tr>
<td>Load on concrete core</td>
<td>0.97</td>
</tr>
<tr>
<td>Load on steel tube and concrete core</td>
<td>0.94</td>
</tr>
<tr>
<td>load on steel tube</td>
<td>1.00</td>
</tr>
</tbody>
</table>

As in previous studies, the other important difference between the composite thin walled and thick walled tubes in this Chapter is the formation of major cracks in the confined concrete. As is shown in Photo 6.3.1.1, formation of major cracks in the concrete core produces a few slip surfaces that make an irregular deformation pattern for the steel tube. This irregularity of lateral deformation for the thin walled tubes is more significant than for the thick walled tubes.

Photo 6.3.1.1 Specimens after tests. From left to right; D3-I, C3-I, B3-I, A3-I.
6.3.2 Apparent Modulus of Elasticity and Poisson’s Ratio

As discussed in previous studies the elastic parameters for the steel tube can be determined from the early stages of the stress-strain history of the specimens. Considering the results obtained in Fig. 6.3.2.1, the modulus of elasticity is approximately the same as the results obtained in Chapters 3, 4, and 5. Once again the modulus of elasticity for thick walled tubes is higher than that of the thin walled tubes. Also in this case, the difference shows that the steel tube in the composite section behaves as an independent hollow section, and that the interaction between the steel tube and the concrete core is negligible. It shows that there is no tensile strength between the internal face of steel tube and the external face of the concrete core.
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\[ \sigma = 0.298 \epsilon_1 - 0.840 \quad r^2 = 1.000 \]

\[ \sigma = 0.228 \epsilon_1 + 0.112 \quad r^2 = 1.000 \]

\[ \sigma = 0.311 \epsilon_1 + 0.923 \quad r^2 = 1.000 \]

\[ \sigma = 0.218 \epsilon_1 + 0.364 \quad r^2 = 1.000 \]

Fig. 6.3.2.1 Stress strain curve in elastic range for all of the specimens.

\[ \epsilon_h = -0.250 \epsilon_1 - 1.598 \quad r^2 = 1.000 \]

\[ \epsilon_h = -0.260 \epsilon_1 - 10.562 \quad r^2 = 0.99 \]

\[ \epsilon_h = -0.263 \epsilon_1 + 2.824 \quad r^2 = 0.999 \]

\[ \epsilon_h = -0.280 \epsilon_1 + 7.067 \quad r^2 = 0.997 \]

Fig 6.3.2.2 Longitudinal strain versus circumferential strain.

Considering the results obtained in Chapter 4, the Poisson's ratio in the elastic range is much more higher than the results in this study. The values obtained for the Poisson's
ratio of steel tubes are more or less equal to each other and close to the classic value of the Poisson's ratio for steel (0.27 - 0.29).

### 6.3.3 Stress-Strain Curve for Confined Concrete

The stress-strain curves of concrete cores in the composite columns are obtained according to the method that was explained in 4.6.3. For comparing the effects of corrections on the concrete stresses the corrected curves and raw curves for each of the specimens are given in Figs. 6.3.3.1 to 4.

The common point in all of the stress-strain curves is the very low intensity of stresses in the concrete core, before the loading surface touches the top of the concrete. Converse to the loading case that was discussed in Chapter 4 (vertical load on the concrete only), the transfer of the vertical load to the concrete core will not happen in this case. Once again, the reason is in the lateral expansion of the steel tube and weak tensile strength between the internal face of the steel tube and external surface of the concrete core.

Therefore, the other disadvantage of the load on the steel tube, that was mentioned in the previous studies for conventional joints, is the complete isolation of the steel tube and concrete core at the range of service loads. Furthermore, the behaviour of the composite sections also depends on the length of the specimens and the amount of gap. As it is shown in Fig. 6.3.1.1 the amount of gap depends on material and geometric characteristics of the steel tube. For a gap equal to 20 mm the specimens B3 and D3 have local buckling after first peak load but it is not occurred for the rest of specimens. Fig. 6.3.1.1 also shows that the amount of gap has no effect on the first and second peak load. In the case of complete constraint at the loading edges, the lateral expansion of the steel tube has a periodic form, therefore, the behaviour of longer columns is different from shorter columns. The experimental studies in Refs. [10, 11, 12], in Chapter 4, show that the longer columns have a lower compressive capacity.

In Figs. 6.3.3.1-4, the abbreviated legends are as follow:

- **RC A** = Raw stress-strain curve of concrete by apparent elastic parameters.
- **RC C** = Raw stress-strain curve of concrete by classic elastic parameters.
- **CC A** = Corrected stress-strain curve of concrete by apparent elastic parameters.
- **CC C** = Corrected stress-strain curve of concrete by classic elastic parameters.
Fig. 6.3.3.1 Corrected and raw stress-strain curves for concrete, for test No. A4 II.

Fig. 6.3.3.2 Corrected and raw stress-strain curves for concrete, for test No. B3 II.
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Fig. 6.3.3.3 Corrected and raw stress-strain curves for concrete, for test No. C3 II.

Fig. 6.3.3.4 Corrected and raw stress-strain curves for concrete, for test No. D3 II.
6.3.4 Stress-Strain Behaviour Of The Steel Tube

The stress-strain curves for the steel tubes are computed according to the method explained in Chapters 3 and 4 and are shown in Figs. 6.3.4.1-4.

Once again, the drop of the longitudinal stresses in the steel tube for all of the cases is not observed in this case, the main reason being that the circumferential stresses are lower in the steel tube.

A comparison of the maximum stresses in the steel tubes from the corrected curves with the computed apparent yield stresses is given in Table 6.3.4.1. The apparent yield stress is the stress at the strain equal to 0.5%. These values are obtained from the stress-strain curves, according to the apparent values of the elastic parameters.

Table 6.3.4.1 The proportion of the compressive yield stresses in the steel tubes.

<table>
<thead>
<tr>
<th>Specimen No.</th>
<th>(f_y(A)) (MPa)</th>
<th>(f^*y(A)) (MPa)</th>
<th>(f^*y(A)/f_y(A))</th>
</tr>
</thead>
<tbody>
<tr>
<td>A4 II</td>
<td>389.0</td>
<td>409.9</td>
<td>1.05</td>
</tr>
<tr>
<td>B4 II</td>
<td>397.0</td>
<td>454.3</td>
<td>1.14</td>
</tr>
<tr>
<td>C4 II</td>
<td>441.0</td>
<td>459.9</td>
<td>1.04</td>
</tr>
<tr>
<td>D4 II</td>
<td>449.0</td>
<td>460.5</td>
<td>1.03</td>
</tr>
</tbody>
</table>

where:

\(f_y(A)\) = Yield stress for the steel tube by apparent values for elastic parameters.

\(f^*y(A)\) = Yield stress for the steel tube for the filled section and load on steel tube according to the classic values for elastic parameters.

As a result, in the case of loading on the steel tube only, the steel tube will be more stable than a bare tube, therefore, the achievable yield stress for the steel tube has a slight increase.
Fig. 6.3.4.1 Stress-strain curves of steel tube drawn for the apparent and classic values of E and v for test No. A3 II.

Fig. 6.3.4.2 Stress-strain curves of steel tube drawn for the apparent and classic values of E and v for test No. B3 II.
Fig. 6.3.4.3 Stress-strain curves of steel tube drawn for the apparent and classic values of $E$ and $v$ for test No. C3 II.

Fig. 6.3.4.4 Stress-strain curves of steel tube drawn for the apparent and classic values of $E$ and $v$ for test No. D3 II.
6.3.5 Biaxial Stresses in the Steel Tubes

The intensity of the circumferential stresses in the steel tube are necessary for computing the radial stresses on the concrete core. In addition, the biaxial stresses of the steel tube will give a clear idea of the failure pattern for the steel tube. Accordingly the diagrams of the biaxial stresses, using the same method as was discussed in Chapters 3, 4, and 5, for the apparent values of the elastic parameters, are presented in Fig. 6.3.5.1. As for the previous studies, the curves for the thin walled sections B3-II and C3-II follow the Von Mises yield criterion, but for the thick walled specimens B3-II and C3-II, due to the dimension effects, the results for the stresses are not accurate.

![Fig 6.3.5.1 Biaxial stresses in the steel tubes.](image)

The other point in these curves is the composite behaviour of the specimens when the concrete core is also under the vertical load. The ultimate load capacity of the section in this case is approximately the same as the results observed for the cases of load on the concrete core and load on the steel tube.

6.3.6 Poisson's Ratio in the Complete Loading History

The curves of longitudinal stresses versus Poisson’s ratios which are the proportion of the circumferential strain to the longitudinal strain are given in Fig. 6.3.6.1. The
common point for all of the curves is the sudden increase of Poisson’s ratio when the steel tube is in a state of longitudinal yield.

This behaviour is due to the formation of cracks in the concrete mass that increase the volume of concrete. Considering the behaviour of the specimens B3-II and D3-II (thin walled tubes), the expansion of the concrete core is more effective than for the other specimens.

Fig. 6.3.6.1 Poisson’s ratio for the whole load history of the specimens.
Another important point in the behaviour of the steel tubes, before the ultimate load capacity is reached, is the value of Poisson's ratio. For all of the cases, in the range of $f_{st} = 100$ to 300 MPa, the value of Poisson's ratio is between 0.25 to 0.30, that is larger than the concrete Poisson's ratio. Hence, the composite behaviour for the sections in this case will not be expected.

As was the case of load on both the steel tube and the concrete core, the two thick walled specimens A3-II, and, C3-II, have more or less the same behaviour. But, the value of Poisson's ratio in the specimen C3-II is higher than that of the A3-II. Once again, this difference shows that the proportion of dimensions for the composite column has an effect on the confinement.
6.4 SUMMARY

The stresses in composite sections when the axial load is applied to the steel tube only is studied in this Chapter. The specifications of the experimental tests are the same as those used for the previous experiments in Chapter 5.

Based on the results in this Chapter, the axial load does not transfer to the concrete core, when the concrete is not connected to the loading support. In other words, the compressive stress on the concrete core is near to zero for all of the cases. The obtained values for the elastic parameters are more or less equal to the values obtained from the bare tubes. The behaviour indicates that the concrete core does not carry any load before the loading support reaches it.
CHAPTER SEVEN

ULTIMATE LOAD BEHAVIOUR OF COMPOSITE STEEL CHS UNDER ECCENTRIC LOADING

7.1 INTRODUCTION

In the case of stocky columns, the actual load combination for the columns in a structural frame is in the form of biaxial moments and axial loads. To evaluate the load capacity for such columns, it is necessary to investigate their behaviour in an experimental as well as a theoretical program.

As a part of the above program, the ultimate load behaviour of the composite steel circular hollow sections filled with higher strength concrete has been studied. The important aspects of this study were to investigate the behaviour of higher strength concrete in the real situation of a composite column, and the effects of the brittleness of the higher strength concrete. Moreover, the confinement effects, which had been ignored in some of the available equations for the evaluation of the ultimate load capacity, have also been investigated in this study.

The experimental program consisted of 30 tests for the dimensions (114.3-4.8), (168.3-4.8) and (168.3-9.53) mm. To study the effects of the length of specimens, the dimension (168.3-4.8) mm were tested for the two different lengths of 750 and 950 mm. For the rest of the specimens, the length was considered to be equal 950 mm.

To investigate the combination of the biaxial moments and axial loads, the vertical load was applied to the specimen with an eccentricity in form of a knife edge. Similar to the experiment for the stub column tests, for each set of experiments two tests were carried out for each eccentricity to make sure that the results obtained were accurate and confirm each other. Because of the necessity of the evaluation of stresses in the crushing area of the columns, the strains were measured with high elongation strain gauges for one specimen in each pair of tests.

The theoretical investigations dealt with the study of the available proposed model for computing the ultimate load capacity of the steel circular hollow sections filled with concrete, and the recommended methods in case of biaxial moments and axial loads. In the above study the necessary amendments regarding the effects of confinement for the
range of the specimens under investigation has been proposed and compared with the actual experimental tests, and also with some of the results available in the literature. In addition, the load deflection history of the composite sections has been studied by the axial load-shortening curves obtained from the tests, and the general behaviour of these structural members has been discussed.

7.2 RESEARCH SIGNIFICANCE

Considering the significance of the utilisation of different material properties in the composite sections, the study on the different material behaviour in conventional structural systems is doubtlessly an important issue. In the case of high strength concrete, which is a key component in an efficient design for high rise structures, the brittleness effects of high strength concrete has to be investigated in structural members such as composite columns. As discussed in Chapter 4, because of the failure pattern of the high strength concrete, the utilisation of this structural material has been restricted to a certain extent to avoid the brittle failure of the whole structure. One of the aims in this study is to investigate how important is the brittleness behaviour of higher strength concrete in composite sections.

On the other hand, most of the recommended equations used for computing the ultimate load capacity of composite sections are based on the conventional assumptions for the reinforced concrete structures, and the beneficial effects of the confinement of the concrete core are not considered. In this study the confinement effects of the concrete core have been discussed by computing the values of the circumferential stresses in the steel tube.

7.3 TEST PROCEDURE AND MATERIAL PROPERTIES

The test procedure was simply to make a number of long columns which were eccentrically loaded to failure. To ensure that the experimental results are consistent, two long columns of each type were made. Test columns were made from three different sizes of commercial electrical resistance welded seam tubes (ERW), which are suitable for structural members with high internal pressure as well as axial stresses.

The testing method for this set of experiments is the same as used in the previous investigations in Chapters 3, 4, 5, and, 6. The geometrical specifications and material properties of the experimental program are given in Table 7.3.1.
The concrete used for this series of tests had higher slump than the previous tests for the stub column tests. The slump of the concrete was about 90 mm at the beginning of concreting. The specimens were casted vertically in three stages and the concrete was vibrated thoroughly in each stage.

For applying the eccentric load to the columns, special support system was designed. To minimise the frictional force due to the rotation of the support system the axial load was applied by a hardened steel knife edge to the appropriate slot in the supporting plate Fig. 7.3.1. The top and bottom wedges were supported by a steel frame to prevent any movement during the tests.

Fig. 7.3.1 Sketch of the tests set up for the eccentric loading tests.
Table 7.3.1 Specifications of the test specimens.

<table>
<thead>
<tr>
<th>No.</th>
<th>Specimen</th>
<th>Outside Diameter d (mm)</th>
<th>Wall Thickness t (mm)</th>
<th>Length l (mm)</th>
<th>d/t</th>
<th>Eccentricity (mm)</th>
<th>f'c (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>E1-I</td>
<td>114.3</td>
<td>4.80</td>
<td>950</td>
<td>18.14</td>
<td>0.0</td>
<td>95.2</td>
</tr>
<tr>
<td>2</td>
<td>E1-II~</td>
<td>114.3</td>
<td>4.80</td>
<td>950</td>
<td>18.14</td>
<td>0.0</td>
<td>95.2</td>
</tr>
<tr>
<td>3</td>
<td>E2-I</td>
<td>114.3</td>
<td>4.80</td>
<td>950</td>
<td>18.14</td>
<td>20.0</td>
<td>84.3</td>
</tr>
<tr>
<td>4</td>
<td>E2-II~</td>
<td>114.3</td>
<td>4.80</td>
<td>950</td>
<td>18.14</td>
<td>20.0</td>
<td>84.8</td>
</tr>
<tr>
<td>5</td>
<td>E3-I</td>
<td>114.3</td>
<td>4.80</td>
<td>950</td>
<td>18.14</td>
<td>40.0</td>
<td>84.8</td>
</tr>
<tr>
<td>6</td>
<td>E3-II~</td>
<td>114.3</td>
<td>4.80</td>
<td>950</td>
<td>18.14</td>
<td>40.0</td>
<td>95.2</td>
</tr>
<tr>
<td>7</td>
<td>E4-I</td>
<td>114.3</td>
<td>4.80</td>
<td>950</td>
<td>18.14</td>
<td>60.0</td>
<td>84.3</td>
</tr>
<tr>
<td>8</td>
<td>E4-II~</td>
<td>114.3</td>
<td>4.80</td>
<td>950</td>
<td>18.14</td>
<td>60.0</td>
<td>84.3</td>
</tr>
<tr>
<td>9</td>
<td>E5-I</td>
<td>168.3</td>
<td>4.80</td>
<td>950</td>
<td>35.06</td>
<td>0.0</td>
<td>109.0</td>
</tr>
<tr>
<td>10</td>
<td>E5-II~</td>
<td>168.3</td>
<td>4.80</td>
<td>950</td>
<td>35.06</td>
<td>0.0</td>
<td>109.0</td>
</tr>
<tr>
<td>11</td>
<td>E6-I</td>
<td>168.3</td>
<td>4.80</td>
<td>950</td>
<td>35.06</td>
<td>20.0</td>
<td>109.0</td>
</tr>
<tr>
<td>12</td>
<td>E6-II~</td>
<td>168.3</td>
<td>4.80</td>
<td>950</td>
<td>35.06</td>
<td>20.0</td>
<td>109.0</td>
</tr>
<tr>
<td>13</td>
<td>E7-I</td>
<td>168.3</td>
<td>4.80</td>
<td>950</td>
<td>35.06</td>
<td>40.0</td>
<td>109.0</td>
</tr>
<tr>
<td>14</td>
<td>E7-II~</td>
<td>168.3</td>
<td>4.80</td>
<td>950</td>
<td>35.06</td>
<td>40.0</td>
<td>109.0</td>
</tr>
<tr>
<td>15</td>
<td>E8-I</td>
<td>168.3</td>
<td>4.80</td>
<td>950</td>
<td>35.06</td>
<td>60.0</td>
<td>109.0</td>
</tr>
<tr>
<td>16</td>
<td>E8-II~</td>
<td>168.3</td>
<td>4.80</td>
<td>950</td>
<td>35.06</td>
<td>60.0</td>
<td>109.0</td>
</tr>
<tr>
<td>17</td>
<td>E9-I</td>
<td>168.3</td>
<td>4.80</td>
<td>750</td>
<td>35.06</td>
<td>0.0</td>
<td>109.0</td>
</tr>
<tr>
<td>18</td>
<td>E9-II~</td>
<td>168.3</td>
<td>4.80</td>
<td>750</td>
<td>35.06</td>
<td>0.0</td>
<td>97.4</td>
</tr>
<tr>
<td>19</td>
<td>E10-I</td>
<td>168.3</td>
<td>4.80</td>
<td>750</td>
<td>35.06</td>
<td>20.0</td>
<td>109.0</td>
</tr>
<tr>
<td>20</td>
<td>E10-II~</td>
<td>168.3</td>
<td>4.80</td>
<td>750</td>
<td>35.06</td>
<td>20.0</td>
<td>78.6</td>
</tr>
<tr>
<td>21</td>
<td>E11-I</td>
<td>168.3</td>
<td>4.80</td>
<td>750</td>
<td>35.06</td>
<td>40.0</td>
<td>78.6</td>
</tr>
<tr>
<td>22</td>
<td>E11-II~</td>
<td>168.3</td>
<td>4.80</td>
<td>750</td>
<td>35.06</td>
<td>40.0</td>
<td>109.0</td>
</tr>
<tr>
<td>23</td>
<td>E12-I</td>
<td>168.3</td>
<td>4.80</td>
<td>750</td>
<td>35.06</td>
<td>60.0</td>
<td>84.8</td>
</tr>
<tr>
<td>24</td>
<td>E12-II~</td>
<td>168.3</td>
<td>4.80</td>
<td>750</td>
<td>35.06</td>
<td>60.0</td>
<td>97.4</td>
</tr>
<tr>
<td>25</td>
<td>E13-I</td>
<td>168.3</td>
<td>9.53</td>
<td>950</td>
<td>17.66</td>
<td>20.0</td>
<td>109.0</td>
</tr>
<tr>
<td>26</td>
<td>E13-II~</td>
<td>168.3</td>
<td>9.53</td>
<td>950</td>
<td>17.66</td>
<td>20.0</td>
<td>109.0</td>
</tr>
<tr>
<td>27</td>
<td>E14-I</td>
<td>168.3</td>
<td>9.53</td>
<td>950</td>
<td>17.66</td>
<td>40.0</td>
<td>109.0</td>
</tr>
<tr>
<td>28</td>
<td>E14-II~</td>
<td>168.3</td>
<td>9.53</td>
<td>950</td>
<td>17.66</td>
<td>40.0</td>
<td>109.0</td>
</tr>
<tr>
<td>29</td>
<td>E15-I</td>
<td>168.3</td>
<td>9.53</td>
<td>950</td>
<td>17.66</td>
<td>60.0</td>
<td>109.0</td>
</tr>
<tr>
<td>30</td>
<td>E15-II~</td>
<td>168.3</td>
<td>9.53</td>
<td>950</td>
<td>17.66</td>
<td>60.0</td>
<td>109.0</td>
</tr>
</tbody>
</table>

where:

\( f'c \) = Unconfined compressive strength of concrete.

~ = Tests with strain gauges.

7.4 TEST RESULTS AND DISCUSSION

7.4.1 Load Axial Shortening Behaviour for Specimens E1(I, II) to E4(I, II)

The curves of axial load-shortening behaviour for the specimens E1(I, II) to E4(I, II) are given in Figs. 7.4.1.1 and 2. One of the important characteristics of these curves is the drop of load of the specimens after the peak load. Two different characteristics may be recognised from the axial load-shortening curves. For the specimens with larger
eccentricity, the ultimate load capacity is significantly low. The drop off rate of the load capacity for the specimens with larger eccentricity is not as high as the specimens with smaller eccentricity, in other words, the ductility factor is higher for the former specimens.

A review of the graphs of axial load-shortening for these specimens indicates that the axial load-shortening curves converge to a certain value of the axial load which in this case, is about 400 kN. This means, regardless of the change in the eccentricity, at the post yielding stage of the axial load-shortening behaviour of the specimens, the final load carrying capacity is constant or it tends to be a plastic plateau.
The post yielding behaviour of these specimens shows that the plastic mechanism in the short composite columns does not form a plastic hinge. Throughout the loading process of the short composite columns, the plastic zone of the column (at the middle of the column) symmetrically expands along the height of the column. This behaviour of a composite column is mainly due to the presence of concrete inside the steel tube, and this is one of the advantages of filled circular hollow sections.

To summarise the structural behaviour, the test results are given in Table 7.4.1.1. As the results in this table indicate, the values of the ultimate loads and the ultimate axial shortenings are more consistent for the columns with lower eccentricities. The reason for this behaviour is the formation of plastic regions at different positions of composite columns. For the specimens with high eccentricity, the plastic regions are formed at the loading ends of the specimen as well as at the midheight. The formation of two plastic regions in the composite column with high eccentricities affects the total eccentricity at the midheight of the specimen, and thereby results in different values for the ultimate load capacity.

The increase of axial load develops a lateral displacement that is maximum at the midheight of the column. Theoretically, at the maximum axial load the first plastic region starts the middle of column, but, in some cases, because of the concentration of stresses at the supports specially for high eccentricities, formation of local plastic regions at the supports is possible. Therefore, in the study of the plastic behaviour of composite columns the axial load capacity depends on the location of the plastic regions in the test specimens.

Table 7.4.1.1 Ultimate load and axial shortening for specimens E1(I, II), to E4(I, II).

<table>
<thead>
<tr>
<th>Specimen No.</th>
<th>Ultimate axial load Pu (kN)</th>
<th>deflection at Ultimate load (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>E1 I</td>
<td>1209.</td>
<td>0.55</td>
</tr>
<tr>
<td>E1 II</td>
<td>1247.</td>
<td>0.56</td>
</tr>
<tr>
<td>E2 I</td>
<td>799.</td>
<td>0.75</td>
</tr>
<tr>
<td>E2 II</td>
<td>805.</td>
<td>0.76</td>
</tr>
<tr>
<td>E3 I</td>
<td>535.</td>
<td>0.97</td>
</tr>
<tr>
<td>E3 II</td>
<td>601.</td>
<td>0.80</td>
</tr>
<tr>
<td>E4 I</td>
<td>553.</td>
<td>0.71</td>
</tr>
<tr>
<td>E4 II</td>
<td>417.</td>
<td>1.40</td>
</tr>
</tbody>
</table>

Photos 7.4.1.1 and 2 show the specimens after the tests.
Photo 7.4.1.1. Specimens E1-I to E4-I after the tests. From left to right; E4-I, E3-I, E2-I, E1-I.
Photo 7.4.1.2. Specimens E1-II to E4-II after the tests. From left to right; E4-II, E3-II, E2-II, E1-II.
7.4.2 Load Axial Shortening Behaviour for Specimens E5(I, II) to E8(I, II)

The curves of axial load-shortening behaviour for the specimens E5(I, II) to E8(I, II) are given in Figs. 7.4.2.1 and 2. The structural characteristics of these specimens are more or less the same as for the tests of the E1(I, II) to E4(I, II), but the behaviour of the concrete is slightly different. In this set, the concrete shows a brittle behaviour that means the effect of confinement in this dimension is lower.

As shown in Figs. 7.4.2.1 and 2, in the vicinity of the ultimate load the formation of major cracks cause a series of sudden drops of the load which in some cases changes the whole axial load-shortening curve.

The reason for this behaviour seems to be the high compressive strain at the middle of the specimen in the compressive area that produces local buckling in the steel tube. The local buckling introduces a considerable gap between the steel tube and concrete core, with the effect that the concrete will no longer have a confinement pressure. In this situation, the failure pattern of the concrete will be of a highly unstable form.

The results for the ultimate load capacity of this set of specimens is more consistent than the previous set. The reason for this behaviour is the low concentration of stresses at the support of the specimens. In the previous set, the eccentricities of 40 and 60 mm were close to the edge of the cross section, and thereby the concentration of stresses at the supports changed the ultimate load behaviour of the test specimens.

Fig. 7.4.2.1 Axial load-shortening for specimens E5 I, to E8 I.
The results for the axial load capacity and the axial shortening at the ultimate load are summarised in Table 7.4.2.1.

Table 7.4.2.1 Ultimate load and axial shortening for specimens E5(I, II), to E8(I, II).

<table>
<thead>
<tr>
<th>Specimen No.</th>
<th>Ultimate axial load Pu (kN)</th>
<th>deflection at Ultimate load (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>E5 I</td>
<td>2848.</td>
<td>0.64</td>
</tr>
<tr>
<td>E5 II</td>
<td>2850.</td>
<td>0.66</td>
</tr>
<tr>
<td>E6 I</td>
<td>2015.</td>
<td>0.63</td>
</tr>
<tr>
<td>E6 II</td>
<td>2055.</td>
<td>0.69</td>
</tr>
<tr>
<td>E7 I</td>
<td>1485.</td>
<td>0.69</td>
</tr>
<tr>
<td>E7 II</td>
<td>1516.</td>
<td>0.69</td>
</tr>
<tr>
<td>E8 I</td>
<td>1161.</td>
<td>0.81</td>
</tr>
<tr>
<td>E8 II</td>
<td>1180.</td>
<td>0.74</td>
</tr>
</tbody>
</table>

Photos 7.4.2.1 and 2 show the specimens after the tests.
Photo 7.4.2.1 Specimens E5-I to E8-I after the tests. From left to right E8-I, E7-I, E6-I, E5-I.
Photo 7.4.2.2 Specimens E5-II to E8-II after the tests. From left to right E8-II, E7-II, E6-II, E5-II.
7.4.3 Load Axial Shortening Behaviour for Specimens E9(I, II) to E12(I, II)

The curves of axial load-shortening behaviour for the specimens E9(I, II) to E12(I, II) are given in Figs. 7.4.3.1 and 2. The structural behaviour of this set is the same as those of the set of E5(I, II) to E8(I, II). The results of this set of tests as shown in Table 7.4.3.1 indicate that the values of the axial shortenings at the ultimate load are different from those of the tests E9(I, II) to E12(I, II).
Table 7.4.3.1 Ultimate load and axial shortening for specimens E9(I, II), to E12(I, II).

<table>
<thead>
<tr>
<th>Specimen No.</th>
<th>Ultimate axial load Pu (kN)</th>
<th>deflection at Ultimate load (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>E9 I</td>
<td>2695.</td>
<td>0.81®</td>
</tr>
<tr>
<td>E9 II</td>
<td>2706.</td>
<td>0.99</td>
</tr>
<tr>
<td>E10 I</td>
<td>2037.</td>
<td>0.73®</td>
</tr>
<tr>
<td>E10 II</td>
<td>2055.</td>
<td>0.84</td>
</tr>
<tr>
<td>E11 I</td>
<td>1540.</td>
<td>1.17</td>
</tr>
<tr>
<td>E11 II</td>
<td>1538.</td>
<td>0.66®</td>
</tr>
<tr>
<td>E12 I</td>
<td>1164.</td>
<td>1.12</td>
</tr>
<tr>
<td>E12 II</td>
<td>1125.</td>
<td>1.22</td>
</tr>
</tbody>
</table>

®: Sudden failure of concrete.

Photos 7.4.3.1 and 2 show the specimens after the tests.
Photo 7.4.3.1 Specimens E9-I to E12-I after the tests. From left to right E12-I, E11-I, E10-I, E9-I.

Photo 7.4.3.2 Specimens E9-II to E12-II after the tests. From left to right E9-II, E10-II, E11-II, E12-II.
7.4.4 Load Axial Shortening Behaviour for Specimens E13(I, II) to E15(I, II)

The curves of axial load-shortening behaviour for the specimens E13(I, II) to E15(I, II) are given in Figs. 7.4.4.1 and 2. In this set of experiments, because of the high axial load capacity associated with the eccentricity of 0.0 which was out of the capacity of the supporting system, the tests were carried out up to the minimum eccentricity of 20.0 mm. In this case, the concrete had a considerably better ductile behaviour than that for previous specimens.

Fig. 7.4.4.1 Axial load-shortening for specimens E13 I, to E15 I.

Fig. 7.4.4.2 Axial load-shortening for specimens E13 II, to E15 II.
The results for the ultimate load capacity and the ultimate axial shortening are summarised in Table 7.4.4.1, and the deformed specimens after the tests are shown in Photos 7.4.4.1 and 2.

Table 7.4.4.1 Ultimate load and axial shortening for specimens E9(I, II), to E12(I, II).

<table>
<thead>
<tr>
<th>Specimen No.</th>
<th>Ultimate axial load Pu (kN)</th>
<th>deflection at Ultimate load (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>E13 I</td>
<td>2914</td>
<td>0.84</td>
</tr>
<tr>
<td>E13 II</td>
<td>2830</td>
<td>0.96</td>
</tr>
<tr>
<td>E14 I</td>
<td>2250</td>
<td>1.01</td>
</tr>
<tr>
<td>E14 II</td>
<td>2274</td>
<td>1.04</td>
</tr>
<tr>
<td>E15 I</td>
<td>1782</td>
<td>1.39</td>
</tr>
<tr>
<td>E15 II</td>
<td>1795</td>
<td>1.24</td>
</tr>
</tbody>
</table>
Photo 7.4.4.1 Specimens E13 I to E15 I after the tests. From left to right E15-I, E14-I, E13-I.
Photo 7.4.4.2 Specimens E13 II to E15 II after the tests. From left to right E13-II, E14-II, E15-II.
7.4.5 Distribution of Strains in Steel Tube

To study the distribution of strains at the different stages of loading as well as at different eccentricities, the summarised distributions of strains for all of the specimens are shown in Figs. 7.4.5.1a,b,c,d to 7.4.5.4a,b,c,d. The vertical axis for all of the figures represents the value of strain in the longitudinal or the circumferential direction, and the horizontal axis shows the position of the recorded strain. In all of the cases, the strains are measured in three different points along the circumference at the midheight of the specimens.

From the position of the neutral axis in all of the cases, it is evident that the movement of the depth of the neutral axis for the specimens with lower eccentricity is much higher than those with higher eccentricity. This behaviour shows the effectiveness of the moment in the distribution of strain for the columns with low eccentricity. Moreover, the position of the neutral axis in the cases of low eccentricities is well outside the cross section, indicating that the section has mainly yielded due to compressive stresses.

In the case of low eccentricity, the ultimate load carrying capacity of the columns has less ductility than the specimens with higher eccentricity. In other words, the movement of the neutral axis to its limiting value (section under pure moment) is much faster.

A direct relationship between the confinement effect and the value of the circumferential strains (tensile with positive sign) indicates that the increase of compressive strength of concrete in the compressive region is higher for the specimens with lower eccentricity. Moreover, the confined region of the concrete is larger in the specimens with lower eccentricity. In other words, the confinement effects for a concentrically loaded specimens has its highest value, but, for a specimen under pure moment the confinement effects reduce to a minimum value.

In addition, the results of this part show that the section which are plane before bending remain plane after bending. According to the results obtained in tests Nos E5 II to E8 II and E9 II to E12 II which have different heights, the ultimate load capacities are more or less equal. This behaviour shows that the difference of 20.0 mm in the length of specimens is not effective on the ultimate load capacity.

But, for the shorter specimens, the value of strains at the first two eccentricities (0.0 and 20.0 mm) are higher than the strains for longer specimens. This reflects the greater rigidity of the shorter specimens.
Fig. 7.4.5.1a Distribution of longitudinal (left) and transverse (right) strain versus cross-section for the specimens E1 II.

Fig. 7.4.5.1b Distribution of longitudinal (left) and transverse (right) strain versus cross-section for the specimens E2 II.

Fig. 7.4.5.1c Distribution of longitudinal (left) and transverse (right) strain versus cross-section for the specimens E3 II.
Fig. 7.4.5.1d Distribution of longitudinal (left) and transverse (right) strain versus cross-section for the specimens E4 II.

The values of axial loads in each particular distribution of strain are given in Table 7.4.5.1. These values are normalised according to the ultimate load capacity of each individual case. Similar tables are presented for each set of tests.

Table 7.4.5.1 Normalised axial load for the distribution of strains.

<table>
<thead>
<tr>
<th>Specimen No.</th>
<th>Ultimate load (kN)</th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>E1 II</td>
<td>1247.</td>
<td>0.04</td>
<td>0.20</td>
<td>0.40</td>
<td>0.60</td>
<td>0.80</td>
<td>0.99</td>
</tr>
<tr>
<td>E2 II</td>
<td>799.</td>
<td>0.06</td>
<td>0.19</td>
<td>0.38</td>
<td>0.56</td>
<td>0.75</td>
<td>0.88</td>
</tr>
<tr>
<td>E3 II</td>
<td>601.</td>
<td>0.08</td>
<td>0.25</td>
<td>0.42</td>
<td>0.58</td>
<td>0.67</td>
<td>0.83</td>
</tr>
<tr>
<td>E4 II</td>
<td>417.</td>
<td>0.12</td>
<td>0.24</td>
<td>0.36</td>
<td>0.48</td>
<td>0.60</td>
<td>0.72</td>
</tr>
</tbody>
</table>

Fig. 7.4.5.2a Distribution of longitudinal (left) and transverse (right) strain versus cross-section for the specimens E5 II.
Fig. 7.4.5.2b Distribution of longitudinal (left) and transverse (right) strain versus cross-section for the specimens E6 II.

Fig. 7.4.5.2c Distribution of longitudinal (left) and transverse (right) strain versus cross-section for the specimens E7 II.

Fig. 7.4.5.2d Distribution of longitudinal (left) and transverse (right) strain versus cross-section for the specimens E8 II.
### Table 7.4.5.2 Normalised axial load for the distribution of strains.

<table>
<thead>
<tr>
<th>Specimen No.</th>
<th>Ultimate load (kN)</th>
<th>Specimen No.</th>
<th>Ultimate load (kN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>E5 II</td>
<td>2850</td>
<td>E6 II</td>
<td>2055</td>
</tr>
<tr>
<td>E7 II</td>
<td>1516</td>
<td>E8 II</td>
<td>1180</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Specimen No.</th>
<th>Ultimate load (kN)</th>
<th>Specimen No.</th>
<th>Ultimate load (kN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>E5 II</td>
<td>2850</td>
<td>E6 II</td>
<td>2055</td>
</tr>
<tr>
<td>E7 II</td>
<td>1516</td>
<td>E8 II</td>
<td>1180</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Specimen No.</th>
<th>Ultimate load (kN)</th>
<th>Specimen No.</th>
<th>Ultimate load (kN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>E5 II</td>
<td>2850</td>
<td>E6 II</td>
<td>2055</td>
</tr>
<tr>
<td>E7 II</td>
<td>1516</td>
<td>E8 II</td>
<td>1180</td>
</tr>
</tbody>
</table>

![Fig. 7.4.5.3a Distribution of longitudinal (left) and transverse (right) strain versus cross-section for the specimens E9 II.](image)

![Fig. 7.4.5.3b Distribution of longitudinal (left) and transverse (right) strain versus cross-section for the specimens E10 II.](image)
Fig. 7.4.5.3c Distribution of longitudinal (left) and transverse (right) strain versus cross-section for the specimens E11 II.

Fig. 7.4.5.3d Distribution of longitudinal (left) and transverse (right) strain versus cross-section for the specimens E12 II.

Table 7.4.5.3 Normalised axial load for the distribution of strains.

<table>
<thead>
<tr>
<th>Specimen No.</th>
<th>Ultimate load (kN)</th>
<th>Normalised load 0.02</th>
<th>Normalised load 0.20</th>
<th>Normalised load 0.41</th>
<th>Normalised load 0.59</th>
<th>Normalised load 0.81</th>
<th>Normalised load 1.00</th>
</tr>
</thead>
<tbody>
<tr>
<td>E9 II</td>
<td>2706.</td>
<td>0.02</td>
<td>0.20</td>
<td>0.41</td>
<td>0.59</td>
<td>0.81</td>
<td>1.00</td>
</tr>
<tr>
<td>E10 II</td>
<td>2055.</td>
<td>0.02</td>
<td>0.20</td>
<td>0.39</td>
<td>0.59</td>
<td>0.79</td>
<td>1.00</td>
</tr>
<tr>
<td>E11 II</td>
<td>1538.</td>
<td>0.03</td>
<td>0.20</td>
<td>0.39</td>
<td>0.59</td>
<td>0.81</td>
<td>0.91</td>
</tr>
<tr>
<td>E12 II</td>
<td>1125.</td>
<td>0.04</td>
<td>0.22</td>
<td>0.40</td>
<td>0.63</td>
<td>0.80</td>
<td>0.96</td>
</tr>
</tbody>
</table>
Fig. 7.4.5.4a Distribution of longitudinal (left) and transverse (right) strain versus cross-section for the specimens E13 II.

Fig. 7.4.5.4b Distribution of longitudinal (left) and transverse (right) strain versus cross-section for the specimens E14 II.

Fig. 7.4.5.4c Distribution of longitudinal (left) and transverse (right) strain versus cross-section for the specimens E15 II.
Table 7.4.5.4 Normalised axial load for the distribution of strains.

<table>
<thead>
<tr>
<th>Specimen No.</th>
<th>Ultimate load (kN)</th>
<th>0.02</th>
<th>0.19</th>
<th>0.41</th>
<th>0.60</th>
<th>0.80</th>
<th>1.00</th>
</tr>
</thead>
<tbody>
<tr>
<td>E13 II</td>
<td>2830.</td>
<td>0.02</td>
<td>0.19</td>
<td>0.41</td>
<td>0.60</td>
<td>0.80</td>
<td>1.00</td>
</tr>
<tr>
<td>E14 II</td>
<td>2274.</td>
<td>0.02</td>
<td>0.20</td>
<td>0.40</td>
<td>0.59</td>
<td>0.79</td>
<td>1.00</td>
</tr>
<tr>
<td>E15 II</td>
<td>1795.</td>
<td>0.03</td>
<td>0.20</td>
<td>0.39</td>
<td>0.62</td>
<td>0.81</td>
<td>0.92</td>
</tr>
</tbody>
</table>

7.4.6 Distribution of Stresses in Steel Tube

The values of stresses are computed according the method explained in Chapter 3. To summarise the results obtained in this part, the distribution of stresses are shown at the eccentricity of 40.0 mm for all of the specimens. The rest of the results are more or less the same. Distribution of stresses for the specimens E3 II, E7 II, E11 II, and E14 II are shown in Figs. 7.4.6.1 to 7.4.6.4, respectively.

The common aspect for all of the specimens is the low intensity of radial stresses even in the compressive face of the steel tube. This phenomenon shows that the expected confinement from the steel tube on the concrete mass is not as high as a specimen under concentric axial load with the same longitudinal stress. The reason for this behaviour is the high longitudinal strain that completely changes the direction of stresses in circumferential direction. In other words, the circumferential stress at the midheight of the specimens is entirely in the compressive state instead of tensile.

In addition, although the distribution of strains across the cross section of the specimens is linear, the distribution of stresses in the steel tube is not linear. This phenomenon represents the effects of the shear stresses in the steel tube which in a concentrically loaded specimen is negligible. Consequently, in an accurate solution for the evaluation of ultimate load carrying capacity of an eccentrically loaded circular hollow section filled with concrete, the value of longitudinal stresses can be computed according to the real distribution of stresses in steel tube.

A practical outcome for this part is that the confinement effects on the concrete core in the case of a composite circular hollow section under the combination of axial load and bending moment are not large. Therefore, the use of the unconfined compressive capacity of concrete for computing the ultimate load carrying capacity seems to be reasonable procedure.

Figs. 7.4.6.1 to 7.4.6.4 in this part are accompanied by Table 7.4.6.1 to show the value of axial load in each case.
Fig. 7.4.6.1 Distribution of longitudinal (left) and transverse (right) stress versus cross-section for the specimens E3 II.

Fig. 7.4.6.2 Distribution of longitudinal (left) and transverse (right) stress versus cross-section for the specimens E7 II.

Fig. 7.4.6.3 Distribution of longitudinal (left) and transverse (right) stress versus cross-section for the specimens E11 II.
Fig. 7.4.6.4 Distribution of longitudinal (left) and transverse (right) stress versus cross-section for the specimens E14 II.

Table 7.4.6.1 Normalised axial load for the distribution of strains.

<table>
<thead>
<tr>
<th>Specimen No.</th>
<th>Ultimate load (kN)</th>
<th>0.08</th>
<th>0.25</th>
<th>0.42</th>
<th>0.58</th>
<th>0.67</th>
<th>0.83</th>
</tr>
</thead>
<tbody>
<tr>
<td>E3 II</td>
<td>601.</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>E7 II</td>
<td>1516.</td>
<td>0.03</td>
<td>0.20</td>
<td>0.40</td>
<td>0.59</td>
<td>0.79</td>
<td>0.98</td>
</tr>
<tr>
<td>E11 II</td>
<td>1538.</td>
<td>0.03</td>
<td>0.20</td>
<td>0.39</td>
<td>0.59</td>
<td>0.81</td>
<td>0.91</td>
</tr>
<tr>
<td>E14 II</td>
<td>2274.</td>
<td>0.02</td>
<td>0.20</td>
<td>0.40</td>
<td>0.59</td>
<td>0.79</td>
<td>1.00</td>
</tr>
</tbody>
</table>

7.4.7 Practical Equations for Computing the Ultimate Load Capacity

Although the theoretical models and necessary assumptions are exactly the same in all of the available equations the results in some cases are different. The proposed equations by Cai for evaluation of the ultimate load capacity of composite sections, which rely on the geometric properties and the specifications of load, give good agreement with the experimental results of about 316 tests [Ref. 22]. On the other hand, some of the recommended equations in this field do not directly consider the length of the composite column in the computation processes, and therefore they probably are more conservative and in some cases are more difficult to use (Ref. [22]).

However, regardless of the slenderness effects on the ultimate load capacity of the composite columns, the load carrying capacity of these sections depend on the deformation behaviour. The experimental results in this study have shown that the plastic mechanism of the composite sections is always at the middle of the specimens with hinged joints at both ends. In addition, regarding the overall deformation of the columns under eccentric loads, the actual eccentricity at different positions in the column depend on the value of the deformation plus the initial eccentricity on the
Therefore, consideration needs to be given to evaluate the additional eccentricity due to the deformation pattern at the ultimate stage of loading.

Evidently, the additional eccentricity will be different for the different column lengths and the longer column (not slender) would have larger additional eccentricity. This situation has been considered empirically in the proposed equation by Cai, but in the other available equations the additional eccentricity that reduces the ultimate load carrying capacity of the columns has not been considered.

The initial eccentricity also has an effect on the value of the additional eccentricity. Based on the experimental results in this study, the ultimate deflection of the specimens for the different values of initial eccentricity is considerably different, which shows the importance of this parameter on the ultimate load carrying capacity of the section. As a result, in the case of a hinge supports for both ends of a column, two parameters of length and the initial eccentricity have to be considered for the evaluation of the axial load capacity.

More generally, the deformation pattern of the column (in case of the rigid or semi rigid supports) is important for the estimation of ultimate load capacity. With this regard, realistic evaluation of the additional eccentricity must be accompanied with the equation of the ultimate load carrying capacity.

7.4.8 Plastic Mechanism in a Composite Column with Hinges at Both Supports

The conventional method in the plastic analysis of structures is to assume a plastic mechanism for the collapsed member. The mechanism must be as realistic as possible for the final deformation of the member, and more importantly the deformation energy must be in its lowest magnitude. The most probable form for the deformation of a composite column under eccentric load is shown in Fig. 7.4.8.1. The main assumption in this mechanism is the elastic behaviour for all of the column length except the plastic hinge at the middle of the column.

According to the equilibrium equation for half of the column, the ultimate moment in the column can be presented by the Eq. 7.4.8.1. On the other hand, the total deformation energy in the column due to the axial load and bending moment can be presented by the Eq. 7.4.8.2

\[ M_u = P_u (e_i \cos \theta + e_x) \]  

(7.4.8.1)
Fig. 7.4.8.1 Plastic mechanism in the composite column.

\[ P_u \left( \frac{P_u}{2E_tA_g} - 1 \right) - M_u \theta = P_{au} \left( e_i \sin \theta + \frac{1}{2} - \frac{1}{2} - \frac{P_u}{2E_tA_g} \cos \theta \right) \]  

(7.4.8.2)

where:

- \( P_u \) = Axial load along the composite column.
- \( P_{au} \) = Maximum load of the composite section.
- \( M_u \) = Bending moment corresponding to the axial load along the composite column.
- \( E_t \) = Equivalent modulus of elasticity for the composite section, Eq. 7.4.8.3.
- \( A_g \) = Total cross section area of the steel tube and the concrete core.
- \( e_i \) and \( e_x \) = Initial and additional eccentricities respectively.

\[ E_t = \frac{E_cA_c + E_sA_s}{A_c + A_s} \]  

(7.4.8.3)

Considering the following parameters:

\[ P_u = \frac{P_{au}}{\cos \theta} \]  

(7.4.8.4)

\[ K_{au} = \frac{1}{2E_tA_g} \]  

(7.4.8.5)
The relationship between the ultimate load and the additional eccentricity can be shown by the Eq. 7.4.8.6.

\[
\frac{P_{au}}{\cos \theta} K_{au} + M_{u} \theta = P_{au} (e_{j} \sin \theta + \frac{1}{2} (1 - \cos \theta) + P_{au} K_{au}) \tag{7.4.8.6}
\]

where:

\[
\theta = \cos^{-1}\left(\frac{e_{x}}{\frac{1}{2} P_{au} K_{au}}\right) \tag{7.4.8.7}
\]

Eq. 7.4.8.7 is obtained according to the geometrical position of the specimen. The Eq. 7.4.8.6 represents a general plastic mechanism that cannot be used before the determination of the axial load carrying capacity of the section. The presented mechanism shows that the actual eccentricity for a composite section under ultimate load is generally larger than the initial eccentricity. The value of the additional eccentricity is an extensive iteration calculation and depends on the elastic characteristics of the steel tube and the concrete core as well as on the plastic relationship of the axial load and bending moment. Therefore, any approximate relationships for evaluation of the additional eccentricity may have to be based on the experimental results of the suitable dimensions of the composite column. More details are given in 7.4.10.

7.4.9 Axial Load and Bending Moment at the Ultimate Stage of Loading

To compute the ultimate load carrying capacity of a composite column, different combinations of axial load and bending moment have to be considered. In this regard, the following assumptions have to be considered for the simplicity of mathematical formulation.

- There exists complete interaction between steel and concrete up to collapse.
- Sections which are plane before bending remain plane after bending.
- Concrete in tension has zero strength.
- Ideal elastic-plastic stress-strain curve for structural steel plus strain hardening which starts at the 10\(\varepsilon_{y}\) (yielding strain) with the slope of \(E_{s}/30\) may be used.
- Geometrical and structural imperfections of material including residual stresses are assumed to be negligible.
The proposed relationships for the stress-strain behaviour of confined concrete as discussed in Chapter 4, rely on the experimental data. In all of the models, it is accepted that the confined concrete has a strain softening behaviour after the peak stress and that the slope of the softening curve depends on the confinement stress. On the other hand, the experimental results in this study showed that the stress in the vicinity of the peak load has a slight change in comparison with the stresses at large strain. Therefore, in each individual case, the stress-strain curve of concrete may be defined according to the structural characteristics.

As the results for the longitudinal and transverse stresses for the eccentrically loaded composite columns in 7.4.6 show the circumferential stress in the steel tube in the compressive region of the composite section is very low and the confinement stress on the concrete core is low. Accordingly, the increase of compressive strength for the concrete core may not be considered. But, the concrete core at the post failure range is not as free as an unconfined concrete and the steel tube prevents the significant volume increase of the concrete. It can be assumed that the concrete behaves as a plastic material up to two times of failure strain and beyond that it has a softening behaviour Fig. 7.4.9.1.

The real behaviour of concrete is somewhere between the dashed line (general behaviour of confined concrete) and the solid line which is the lowest possible position of the stress-strain curve of the concrete. Hence, the adopted behaviour for the concrete core is the safe assumption (extreme boundary) for design purposes.

Fig. 7.4.9.1 General elastic-plastic behaviour of concrete core.

The slope of the softening part of the stress-strain curve is always out of the limits of the plastic analysis for the composite sections. Nevertheless, for the post yielding studies on these sections, it has to be determined according to the actual behaviour of concrete at a low confinement pressure. For the further studies in this Chapter, it is
assumed that the slope of the softening part of the stress-strain curve for the concrete is $E_c/40$.

Distribution of stresses and strains for the cross section of the composite column, which shows a partially plastic cross section, are given in Fig. 7.4.9.1. It is assumed that the composite section yields when the maximum strain in the compressive region is equal to the $\epsilon_c$ that is given in Eq. 7.4.9.1.

$$\epsilon_c = 2(f'_c / E_s)$$  \hspace{1cm} (7.4.9.1)

![Fig. 7.4.9.2 Distribution of strain and stress across the cross section of the composite column.](image)

At the strain of $\epsilon_c$, the concrete has a compressive stress equal to $f'_c$ which is the unconfined compressive strength of concrete. Fig. 7.4.9.2 (b) shows the exact distribution of the stresses in the concrete core, and Fig. 7.4.9.2 (c) is the modelled behaviour of concrete. The distribution of stresses in the steel tube are given in Fig. 7.4.9.2 (d).

![Fig. 7.4.9.3 Distribution of strain and stress across the cross section of the composite column for the post yielding stage.](image)

The same method can be applied for studies of the post yielding behaviour of the composite sections. In this case, the values of the strains in the tensile and the
compressive regions of the section are higher than that of the beginning of the plastic behaviour. The distribution of stresses and strain can be shown by Fig 7.4.9.3.

To compute the values of the axial load and bending moment at different positions of the depth of neural axial, the stress-strain distribution according to the Fig. 7.4.9.3 is considered. In this regard, the whole cross section of the columns was divided to a number of horizontal strips. The number of these strips depend on the required accuracy and the dimensions of the outside diameter of the column. In this study, considering the maximum outside diameter to be 168.3 mm, the number of strips was considered as equal to 100.

![Graphs showing normalized axial load versus normalized bending moment](image)

Typical curves for the ultimate axial load and bending moment combinations:

(a): For the specimens E1 to E4 (I, II)
(b): For the specimens E5 to E12 (I, II)
(c): For the specimens E13 to E15 (I, II)

![Graphs showing normalized axial load versus normalized bending moment](image)

Fig. 7.4.9.4. Normalised relationships of the axial load and the bending moment in the composite columns in this study.

where:

\[
\text{Normalised axial load} = \frac{\text{Axial load at any specific point}}{\text{Squash load of the composite column}}.
\]
Chapter 7 Ultimate load behaviour of composite steel CHS under eccentric loading

Normalised bending moment = Bending moment at any specific point divided by the ultimate flexural capacity of the composite column.

To illustrate the aforementioned method in real cases, the values obtained from the tests can be used. For each test, the value of the axial ultimate load has been measured so that the corresponding ultimate moment can be determined according to the design curves in Fig. 7.4.9.4. The corresponding ultimate moments for all of the specimens are computed in Table 7.4.10.1.

7.4.10 Additional Eccentricity for the Specimens

The value of the total eccentricity for each of the tests in this study can be computed by the proportion of axial load capacity and the corresponding value of the bending moment. The bending moment can be determined according to the method discussed in 7.4.9. The values of the axial load capacity and the corresponding bending moment plus the total eccentricity are given in Table 7.4.10.1.

As the result show, the actual failure of the columns happen in a larger value of the eccentricity which is due to the lateral deformation of the column. Fig. 7.4.10.1 shows the test results for the specimen E11 I. The expected value for the axial load capacity and bending moment is the point of the intersection for the relationship of the load and moment with a constant value of eccentricity and the plastic relationship for the axial load and bending moment. But the experimental results show that the ultimate position happens at the point A. This behaviour shows the effect of the additional eccentricity on the ultimate load capacity of the short composite columns.

Fig. 7.4.10.1 Expected values of the ultimate load capacity of the section and the values obtained by the test for specimen No. E11 I.
Except for the proposed equation of the load carrying capacity of the composite sections by Cai, the other recommended equations do not consider the additional eccentricity that is effective even in short columns. However, the proposed equations by Cai is not in good agreement with results of the tests in this study. More importantly, in all of the cases, the predicted results by Cai are not safe especially for the low eccentricity cases (last column in Table 7.4.10.1).

**Table 7.4.10.1 Structural characteristics of the test specimens.**

<table>
<thead>
<tr>
<th>No.</th>
<th>Specimen</th>
<th>Eccentricity (mm)</th>
<th>Ultimate axial load Pu (kN)</th>
<th>Ultimate moment Mu (kN.mm)</th>
<th>Total eccentricity (mm)</th>
<th>Additional eccentricity (mm)</th>
<th>Pu by Cai (kN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>E1-I</td>
<td>0.0</td>
<td>1209.</td>
<td>15338.2</td>
<td>12.69</td>
<td>12.69</td>
<td>1629.5</td>
</tr>
<tr>
<td>2</td>
<td>E1-II-</td>
<td>0.0</td>
<td>1247.</td>
<td>14426.1</td>
<td>11.57</td>
<td>11.57</td>
<td>1629.5</td>
</tr>
<tr>
<td>3</td>
<td>E2-I</td>
<td>20.0</td>
<td>799.</td>
<td>24469.7</td>
<td>30.63</td>
<td>10.63</td>
<td>912.8</td>
</tr>
<tr>
<td>4</td>
<td>E2-II-</td>
<td>20.0</td>
<td>805.</td>
<td>24349.6</td>
<td>30.25</td>
<td>10.25</td>
<td>912.8</td>
</tr>
<tr>
<td>5</td>
<td>E3-I</td>
<td>40.0</td>
<td>535.</td>
<td>29336.9</td>
<td>54.84</td>
<td>14.84</td>
<td>645.5</td>
</tr>
<tr>
<td>6</td>
<td>E3-II-</td>
<td>40.0</td>
<td>601.</td>
<td>28280.7</td>
<td>47.06</td>
<td>7.06</td>
<td>675.1</td>
</tr>
<tr>
<td>7</td>
<td>E4-I</td>
<td>60.0</td>
<td>553.</td>
<td>29085.6</td>
<td>52.60</td>
<td>-7.40</td>
<td>499.3</td>
</tr>
<tr>
<td>8</td>
<td>E4-II-</td>
<td>60.0</td>
<td>417.</td>
<td>30275.2</td>
<td>72.60</td>
<td>12.60</td>
<td>499.3</td>
</tr>
<tr>
<td>9</td>
<td>E5-I</td>
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<td>2848.</td>
<td>29676.5</td>
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<td>10.42</td>
<td>3519.6</td>
</tr>
<tr>
<td>10</td>
<td>E5-II-</td>
<td>0.0</td>
<td>2850.</td>
<td>29611.2</td>
<td>10.39</td>
<td>10.39</td>
<td>3519.6</td>
</tr>
<tr>
<td>11</td>
<td>E6-I</td>
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<td>2015.</td>
<td>56268.4</td>
<td>27.92</td>
<td>7.92</td>
<td>2400.4</td>
</tr>
<tr>
<td>12</td>
<td>E6-II-</td>
<td>20.0</td>
<td>2055.</td>
<td>55068.9</td>
<td>26.80</td>
<td>6.80</td>
<td>2400.4</td>
</tr>
<tr>
<td>13</td>
<td>E7-I</td>
<td>40.0</td>
<td>1485.</td>
<td>70277.8</td>
<td>47.33</td>
<td>7.33</td>
<td>1821.2</td>
</tr>
<tr>
<td>14</td>
<td>E7-II-</td>
<td>40.0</td>
<td>1516.</td>
<td>69541.0</td>
<td>45.87</td>
<td>5.87</td>
<td>1821.2</td>
</tr>
<tr>
<td>15</td>
<td>E8-I</td>
<td>60.0</td>
<td>1161.</td>
<td>77379.2</td>
<td>66.65</td>
<td>6.65</td>
<td>1467.2</td>
</tr>
<tr>
<td>16</td>
<td>E8-II-</td>
<td>60.0</td>
<td>1180.</td>
<td>77064.6</td>
<td>65.31</td>
<td>5.31</td>
<td>1467.2</td>
</tr>
<tr>
<td>17</td>
<td>E9-I</td>
<td>0.0</td>
<td>2695.</td>
<td>34663.5</td>
<td>12.86</td>
<td>12.86</td>
<td>3806.2</td>
</tr>
<tr>
<td>18</td>
<td>E9-II-</td>
<td>0.0</td>
<td>2706.</td>
<td>34305.6</td>
<td>12.68</td>
<td>12.68</td>
<td>3596.2</td>
</tr>
<tr>
<td>19</td>
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<td>55610.9</td>
<td>27.30</td>
<td>7.30</td>
<td>2595.8</td>
</tr>
<tr>
<td>20</td>
<td>E10-II-</td>
<td>20.0</td>
<td>2055.</td>
<td>55068.9</td>
<td>26.80</td>
<td>6.80</td>
<td>2405.5</td>
</tr>
<tr>
<td>21</td>
<td>E11-I</td>
<td>40.0</td>
<td>1540.</td>
<td>68964.4</td>
<td>44.78</td>
<td>4.78</td>
<td>1825.1</td>
</tr>
<tr>
<td>22</td>
<td>E11-II-</td>
<td>40.0</td>
<td>1538.</td>
<td>69012.5</td>
<td>44.87</td>
<td>4.87</td>
<td>1969.5</td>
</tr>
<tr>
<td>23</td>
<td>E12-I</td>
<td>60.0</td>
<td>1164.</td>
<td>77329.7</td>
<td>66.43</td>
<td>6.43</td>
<td>1521.5</td>
</tr>
<tr>
<td>24</td>
<td>E12-II-</td>
<td>60.0</td>
<td>1125.</td>
<td>77895.4</td>
<td>69.24</td>
<td>9.24</td>
<td>1499.1</td>
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<td>25</td>
<td>E13-I</td>
<td>20.0</td>
<td>2914.</td>
<td>61702.1</td>
<td>21.17</td>
<td>1.17</td>
<td>3525.2</td>
</tr>
<tr>
<td>26</td>
<td>E13-II-</td>
<td>20.0</td>
<td>2830.</td>
<td>64752.2</td>
<td>22.88</td>
<td>2.88</td>
<td>3525.2</td>
</tr>
<tr>
<td>27</td>
<td>E14-I</td>
<td>40.0</td>
<td>2250.</td>
<td>84998.1</td>
<td>37.78</td>
<td>-2.22</td>
<td>2647.6</td>
</tr>
<tr>
<td>28</td>
<td>E14-II-</td>
<td>40.0</td>
<td>2274.</td>
<td>84200.6</td>
<td>37.03</td>
<td>-2.97</td>
<td>2647.6</td>
</tr>
<tr>
<td>29</td>
<td>E15-I</td>
<td>60.0</td>
<td>1782.</td>
<td>99930.0</td>
<td>56.08</td>
<td>-3.92</td>
<td>2119.8</td>
</tr>
<tr>
<td>30</td>
<td>E15-II-</td>
<td>60.0</td>
<td>1795.</td>
<td>99527.8</td>
<td>55.45</td>
<td>-4.55</td>
<td>2119.8</td>
</tr>
</tbody>
</table>

**7.5 ALTERNATIVE APPROACH TO EVALUATION OF ULTIMATE LOAD CAPACITY OF THE COMPOSITE COLUMN**

Although the simplicity of the equations for evaluation of the ultimate load capacity is an important issue, the accuracy of the estimated load capacity cannot be disregarded.
According to the curves of ultimate load versus bending moment (Fig. 7.4.9.4) the plastic behaviour of the column may not be estimated by a series of extreme behaviours for the column. In other words, specific characteristics of the composite column furnish unique ultimate axial load-bending moment curves. Moreover, considering the simplicity of the numerical analysis of the section by a suitable computer program (as used in 7.4.9), there is no reason to simplify the relevant equations. To carry out this procedure, it is necessary to have one assumption. Considering the results in Table 7.4.10.1, the additional eccentricity may be estimated by Eq. 7.5.1.

$$e_x = K + L e_i$$  \hspace{1cm} (7.5.1)

The rest of the process for determining the ultimate load capacity of the section is completely analytical. The results are fairly accurate within a reasonable margin of safety that depends on the values of K and L. In addition, Eq. 7.5.1 may be calibrated against a number of test results for any special geometric and structural specifications of the steel tube and concrete core.

The values of K and L can be computed according to the test results. But the results are not general enough to be used as an accurate basis for the computation process. Approximate values of $K = 10.0$ and $L = 0.03$ are considered. Based on these values, the ultimate axial load for the specimens in this study can be determined by finding the intersection point of the load-moment line due to the computed total eccentricity and the ultimate load-moment which is computed by the plastic analysis of the composite section. A computer program was developed to calculate the axial load capacity of the section according to a given value of initial eccentricity.

The values of the computed axial load and the obtained values by the test results are given in Table 7.5.1. As the results show, the computed values for the axial load capacity are in a good agreement with test results. More importantly, none of the results has a greater value than the corresponding experimental result, hence the method has a margin of safety which even in the worst cases (results for thick walled tubes) does not exceed from 15% of the actual load capacity of the composite column.

Results for thick walled tubes are not as accurate as the results for thin walled tubes. The main reason for this behaviour is the lower value of the additional eccentricity in this case. Nevertheless, comparing the results by the conventional method Ref. [18] which is always more than the actual compressive capacity of the column, and the proposed results by Cai, the computed values for the axial load capacity are more realistic.
Table 7.5.1 Structural characteristics of the test specimens.

<table>
<thead>
<tr>
<th>No.</th>
<th>Specimen</th>
<th>Eccentricity (mm)</th>
<th>Ultimate axial load Pu (kN)</th>
<th>Pu by proposed method (kN)</th>
<th>Pu by Cai (kN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>E1-I</td>
<td>0.0</td>
<td>1209.</td>
<td>1303.</td>
<td>1629.5</td>
</tr>
<tr>
<td>2</td>
<td>E1-II~</td>
<td>0.0</td>
<td>1247.</td>
<td>1303.</td>
<td>1629.5</td>
</tr>
<tr>
<td>3</td>
<td>E2-I</td>
<td>20.0</td>
<td>799.</td>
<td>803.</td>
<td>912.8</td>
</tr>
<tr>
<td>4</td>
<td>E2-II~</td>
<td>20.0</td>
<td>805.</td>
<td>803.</td>
<td>912.8</td>
</tr>
<tr>
<td>5</td>
<td>E3-I</td>
<td>40.0</td>
<td>535.</td>
<td>569.</td>
<td>645.5</td>
</tr>
<tr>
<td>6</td>
<td>E3-II~</td>
<td>40.0</td>
<td>601.</td>
<td>569.</td>
<td>675.1</td>
</tr>
<tr>
<td>7</td>
<td>E4-I</td>
<td>60.0</td>
<td>553.</td>
<td>414.</td>
<td>499.3</td>
</tr>
<tr>
<td>8</td>
<td>E4-II~</td>
<td>60.0</td>
<td>417.</td>
<td>414.</td>
<td>499.3</td>
</tr>
<tr>
<td>9</td>
<td>E5-I</td>
<td>0.0</td>
<td>2848.</td>
<td>2879.</td>
<td>3519.6</td>
</tr>
<tr>
<td>10</td>
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<td>2850.</td>
<td>2879.</td>
<td>3519.6</td>
</tr>
<tr>
<td>11</td>
<td>E6-I</td>
<td>20.0</td>
<td>2015.</td>
<td>1933.</td>
<td>2400.4</td>
</tr>
<tr>
<td>12</td>
<td>E6-II~</td>
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<td>2055.</td>
<td>1933.</td>
<td>2400.4</td>
</tr>
<tr>
<td>13</td>
<td>E7-I</td>
<td>40.0</td>
<td>1485.</td>
<td>1398.</td>
<td>1821.2</td>
</tr>
<tr>
<td>14</td>
<td>E7-II~</td>
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<td>1516.</td>
<td>1398.</td>
<td>1821.2</td>
</tr>
<tr>
<td>15</td>
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<td>1161.</td>
<td>1082.</td>
<td>1467.2</td>
</tr>
<tr>
<td>16</td>
<td>E8-II~</td>
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<td>1180.</td>
<td>1082.</td>
<td>1467.2</td>
</tr>
<tr>
<td>17</td>
<td>E9-I</td>
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<td>2695.</td>
<td>2879.</td>
<td>3806.2</td>
</tr>
<tr>
<td>18</td>
<td>E9-II~</td>
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<td>2706.</td>
<td>2879.</td>
<td>3596.2</td>
</tr>
<tr>
<td>19</td>
<td>E10-I</td>
<td>20.0</td>
<td>2037.</td>
<td>1933.</td>
<td>2595.8</td>
</tr>
<tr>
<td>20</td>
<td>E10-II~</td>
<td>20.0</td>
<td>2055.</td>
<td>1933.</td>
<td>2405.5</td>
</tr>
<tr>
<td>21</td>
<td>E11-I</td>
<td>40.0</td>
<td>1540.</td>
<td>1398.</td>
<td>1825.1</td>
</tr>
<tr>
<td>22</td>
<td>E11-II~</td>
<td>40.0</td>
<td>1538.</td>
<td>1398.</td>
<td>1969.5</td>
</tr>
<tr>
<td>23</td>
<td>E12-I</td>
<td>60.0</td>
<td>1164.</td>
<td>1082.</td>
<td>1521.5</td>
</tr>
<tr>
<td>24</td>
<td>E12-II~</td>
<td>60.0</td>
<td>1125.</td>
<td>1082.</td>
<td>1499.1</td>
</tr>
<tr>
<td>25</td>
<td>E13-I</td>
<td>20.0</td>
<td>2914.</td>
<td>2497.</td>
<td>3525.2</td>
</tr>
<tr>
<td>26</td>
<td>E13-II~</td>
<td>20.0</td>
<td>2830.</td>
<td>2497.</td>
<td>3525.2</td>
</tr>
<tr>
<td>27</td>
<td>E14-I</td>
<td>40.0</td>
<td>2250.</td>
<td>1878.</td>
<td>2647.6</td>
</tr>
<tr>
<td>28</td>
<td>E14-II~</td>
<td>40.0</td>
<td>2274.</td>
<td>1878.</td>
<td>2647.6</td>
</tr>
<tr>
<td>29</td>
<td>E15-I</td>
<td>60.0</td>
<td>1782.</td>
<td>1499.</td>
<td>2119.8</td>
</tr>
<tr>
<td>30</td>
<td>E15-II~</td>
<td>60.0</td>
<td>1795.</td>
<td>1499.</td>
<td>2119.8</td>
</tr>
</tbody>
</table>

Considering a suitable factor of safety for the material properties of the composite column (steel and concrete), the results can be used as a safe and accurate evaluation of the axial load capacity of short composite columns.

Regardless of additional eccentricity and other parameters such as imperfection of steel tube and slenderness of the whole column, the basis of the proposed method for computing the ultimate load capacity of a composite section (CHS) is the same as proposed in draft for development of Eurocode 4. In this code, the moment capacity obtained from interaction curve for compression and uniaxial bending compared with the actual moment in the composite section (details in 4.8.3.11 to 4.8.3.13 Eurocode 4) [Ref. 64].
7.6 COMPARATIVE STUDY ON DETERMINATION OF ULTIMATE MOMENT CAPACITY OF THE COMPOSITE SECTIONS

Although the basis for computing the ultimate moment capacity of a composite section by all of the available methods is more or less the same the results obtained are not completely in agreement with each other. One of the conventional methods which can be used with an appropriate graph to estimate the position of the neutral axis is the method that has been discussed in Ref [57]. The concept of this method is based on the equilibrium equation of the cross section and yielding state for both the steel tube and concrete core. The same concept have been used by the CEB Ref. [18] but the method for evaluation of the neutral axis is different.

As an example, one of the specimens for this study may be used. The results for the ultimate moment capacity of the section are given in Table 7.6.1. The dimensions of the example is 168.3 mm for outside diameter and 4.8 mm for wall thickness. The compressive capacity of concrete is 84.3 MPa and the yield stress of the steel is 400 MPa.

<table>
<thead>
<tr>
<th>Dimensions</th>
<th>Ultimate moment by Johnson and Buckby[57] (kN.mm)</th>
<th>Ultimate moment by CEB [18] (kN.mm)</th>
<th>Ultimate moment by analytical method in this study (kN.mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>168.3-4.8</td>
<td>61,325,000.</td>
<td>49,440,000.</td>
<td>65,848,000.</td>
</tr>
</tbody>
</table>

Result from the CEB is entirely different from the results by the other two methods. To find out the differences caused by the use of other dimensions, the calculations were repeated for a series of dimensions. The outside diameter varies from 100 mm to 260 mm with the wall thickness from 3 mm to 7.9 mm. The results for the ultimate load capacity by these three method are shown in Figs. 7.6.1 and 2.

The important results shown by this numerical study is that the proposed method by CEB is fairly in agreement with the method by Johnson and Buckby for the large outside diameters. But at the low outside diameters and the minimum range of the wall thicknesses, results by the CEB method show a poor accuracy. As mentioned before, the basic concept for both of the methods by CEB and Johnson and Buckby is the same, and the differences show that the mathematical solution by CEB which considered an equivalent wall thickness for the concrete core (tc) does not have the same accuracy for a range of different dimensions of composite columns.
Fig. 7.6.1 comparative results for the computation methods of the ultimate moment capacity of a composite column (outside diameter from 100 to 160 mm).

Fig. 7.6.2 comparative results for the computation methods of the ultimate moment capacity of a composite column (outside diameter from 180 to 240 mm).
where:

the outside diameter for the composite sections are given in Table 7.6.2. The wall thickness for each outside diameter varies from the 3.0 to 7.9 mm.

Table 7.6.2 The outside diameter for the different sets in Figs. 7.6.1 and 2.

<table>
<thead>
<tr>
<th>Graph set</th>
<th>Outside diameter D (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>100</td>
</tr>
<tr>
<td>B</td>
<td>120</td>
</tr>
<tr>
<td>C</td>
<td>140</td>
</tr>
<tr>
<td>D</td>
<td>160</td>
</tr>
<tr>
<td>E</td>
<td>180</td>
</tr>
<tr>
<td>F</td>
<td>200</td>
</tr>
<tr>
<td>G</td>
<td>220</td>
</tr>
<tr>
<td>H</td>
<td>240</td>
</tr>
</tbody>
</table>

The method introduced in this study, which is based on the distribution of stresses according to the values of strains in the cross section of the composite column, is in a good agreement with the method by Johnson and Buckby at the low outside diameters. Whereas, with an increase in the outside diameter, the results are not in good agreement. This comparison shows that assuming a constant equivalent plastic stress for the steel tube and the concrete core for the circular section may furnish inaccuracy for some geometric specifications. As an example, considering the results in Fig. 7.6.2, the value of the ultimate moment capacity of the section is estimated to be about 20% less than the value which has been obtained by the exact analysis.

The problem in this case is that the computation of the ultimate moment capacity in a conservative way that results in a value less than the actual moment capacity of the cross section is not a safe method for the design of a composite CHS column. The design curves of the axial load capacity and bending moment indicate that a lesser value of the ultimate moment capacity will increase the values of the normalised moments that eventually shifts the design curve into the right hand side in the coordinate system. Therefore, based on practical design procedures, the computed axial load carrying capacity of the section will be more than the real value given in Fig. 7.6.3. Hence, consideration needs to be given to design of composite sections by accurate methods to avoid any over-estimation of the load carrying capacity.
Fig. 7.6.3 Effect of underestimation of ultimate moment capacity on the value obtained of axial load carrying capacity.
7.7 SUMMARY

The ultimate load behaviour of composite circular hollow section under axial eccentric load have been studied in this chapter. In the experimental part of this study, 30 tests for different dimensions of the steel tube and different values of the eccentricity have been carried out.

The method which is suggested for the evaluation of the axial ultimate load capacity of composite section under the eccentric load is in a good agreement with the results obtained in this study. Furthermore, it is shown that the recommended equations by CEB for evaluation of the plastic moment in a composite section does not give accurate results in some ranges of dimensions.

On the other hand, the proposed method in Draft Eurocode 4, that relies on an analytical method for each composite section, is a realistic code for computing the load capacity characteristics of composite sections (7.5). But, it has to be mentioned, the effect of confinement for both of the cases, pure axial compression and compression combined with uniaxial or biaxial bending are considered to be equal in Draft Eurocode 4; it is not a safe assumption in general. It seems that necessary amendments for the combination of axial load and uniaxial or biaxial bending have yet to be included in this code.
8.1 NON-LINEAR FINITE ELEMENT ANALYSIS FOR BARE STEEL CHS

8.1.1 General Concepts

To investigate the ultimate load behaviour of a bare steel CHS under pure axial load in the elastic and plastic range, a non-linear finite element analysis was carried out. As was discussed in Part 3.1, the solution method was based on an iterative procedure to minimise the error vector of unbalanced forces acting at all grid points in the structural model. For obtaining an accurate history of forces, stresses, and strain behaviours of the model, the applied load must be separated into appropriate loading steps to magnify the changes in the structural behaviour of the model. Therefore, loading was considered in a manner to obtain more information in the elastoplastic and plastic stages.

In addition, the model was considered according to the axisymmetric form of the real specimen and loadings, and as such a quarter of a tube was modelled for a non-linear finite element analysis.

8.1.2 Material Properties of Steel Tube and Detailed Specifications and Finite Element Meshing of the Model

The stress-strain behaviour of the steel was modelled by a simple model. In this model the stress-strain behaviour of steel in the elastic range was considered as a straight line and the plastic behaviour was represented by a horizontal line. The value of yield plateau length was chosen as $e_{sh}/e_y=10$, and the strain hardening modulus was taken as $E_{sh} = E/30$, where $E$ is the modulus of elasticity equal to 200 GPa, and, $e_y$ and $e_{sh}$ are yield and strain hardening strains, respectively. The stress strain curve is shown in Fig. 8.1.2.1.

The model considered for this study has the same geometric specifications as the specimen No. B1 (I,II). As mentioned in 8.1.1, because of the symmetric behaviour of
the complete model, half of the height of a quarter of the specimen with appropriate constraints was modelled for analytical studies. The elements used for the finite element meshing were brick elements with eight nodes. The number of elements was selected in such a way that the ratio of the element dimensions were less than 2, both in the longitudinal and the circumferential directions.

Two different finite element mesh gradings were considered to control the accuracy of the modelling. For the first solution 200 brick elements were used, and 600 elements for the second solution.

8.1.3 Analytical Results of the Bare Steel Model

Results from the solutions from the two mesh gradings show the same values for the ultimate axial load and the ultimate axial strain. Moreover, the analytical results are in good agreement with results obtained from the tests.

The only difference for the two analytical results was the duration required for the solution. For the first case that had 200 brick elements, the solution stopped due to the bad geometry of the elements at the axial deformation of \( d = -0.95 \) (mm), whereas the solution for the second model that had 600 brick elements stopped at the axial deformation of \( d = -1.744 \). Therefore, an extension of the curves of load-axial shortening can be obtained by the use of a finer mesh model. The load axial shortening curves for both the analytical cases are shown in Figs. 8.1.2.2 and 3.
The second model displayed a second half wave of lateral deformations at the loaded end of the model, as it has higher axial deformations. Figs. 8.1.2.4 and 5, represent the deformed model at different axial deformations. The geometrical positions of the first local lateral deformation for both the models are more or less the same exhibiting the accuracy of the results to be the same for both the models.
Chapter Eight Finite element analysis

8.2 GENERAL CONCEPTS AND ANALYTICAL METHODS FOR CIRCULAR HOLLOW SECTIONS FILLED WITH CONCRETE

This section deals with a finite element model used to analyse a particular type of composite circular hollow section under axial load applied on the concrete only. The principal aim of this study is to investigate the stress distribution in the concrete core
and the surrounding steel tube. In addition, the effect of the confinement on the concrete core in the elastic range for both concrete and steel is investigated by the use of gap elements at the common surface of these materials.

The analysis method was based on the non-linear behaviour of both the steel and concrete. The non-linear stress-strain curves for both concrete and steel were introduced as material properties of the elements to the finite element program. The solution method, described in Chapter 3, is based on an iterative procedure.

To model the interaction of the outer surface of the concrete to the inner surface of the steel tube, gap elements have been used. The structural properties of the gap elements were so introduced that the tensile and shear stresses between the common surface of the steel and concrete had a negligible value. On the other hand, the gap elements were strong enough to transfer any compressive stresses from the steel tube to the concrete core.

![The gap element](image)

The gap element is shown in Fig. 8.2.1.1. Each gap element has its own coordinate system that must be specified according to the expected movement and desired loads that would be transferred from one mass to the other. With this view, two axial stiffness were defined for the gap elements. An axial stiffness for the closed gap \((U_a - U_b > U_0)\) \(K_A\), and an axial stiffness for the open gap \((U_a - U_b < U_0)\) \(K_B\), where, \(U_a\) and \(U_b\) are normal displacements for grid A and grid B respectively; \(U_0\) is the initial gap opening (real > 0, if it is used) and XYZ are the axes of the coordinate system that is a unique system for each gap element. The gap elements are also able to transfer the shear forces between the two masses. To specify the capability of the shear strength, the transverse stiffness \(K_T\) was also defined for the closed gap elements in the two directions, Y and Z. Obviously, an open gap will not possess a transverse (shear)
stiffness. The whole system represents the condition that the tensile strength between the steel and the concrete is approximately zero, and when the concrete surface slips the shear strength is also negligible.

Fig. 8.2.1.2 Force-deflection curve for non-linear analysis.

Figs. 8.2.1.2 and 8.2.1.3 show the force-deflection relationship and shear force respectively of a gap element for the non-linear analysis of the model. $MU_1$ and $MU_2$ are the coefficients of static friction to specify the maximum shear force for the gap element, and $\Delta v$ and $\Delta w$ are the transverse deformations on a plane perpendicular to the direction of the gap element.

Another important concept for solving a steel-concrete structure is the possibility of change in Poisson's ratio during the finite element analysis. The real stress-strain
history for concrete is that the Poisson's ratio has a variable value. Although the change of the Poisson's ratio in the elastic range has an insignificant effect on the structural behaviour of the whole model, in the elastic-plastic stage, and more importantly in plastic range, the effect of Poisson's ratio is significant [34].

8.2.2 Material Properties For Steel And Concrete

The results of the analytical method for the evaluation of the structural behaviour of the model rely on realistic input data for the mechanical behaviour of the elements. As was mentioned in 8.2.1, the mechanical characteristic of concrete as a brittle material cannot easily be introduced to the available finite element software. Moreover, existing mathematical formulas for representing the mechanical behaviour of concrete are based on the statistical information of a specific concrete that is not necessarily applicable for other concrete specifications. To simplify the solution and, more importantly, for providing a general descriptive analysis for confined concrete in a steel tube, the general behaviour of concrete has been considered for each model. Accordingly, it has been considered that the concrete at the ultimate stage of stress behaves as a ductile material and the stress-strain curve is more or less horizontal (as was observed in the experiments). On the other hand, the increase in the compressive strength of the concrete due to confinement, which is a material property, was considered from the results obtained by the experiments.

Numerous curves have been proposed for the stress-strain curve of confined concrete (Chapter 2). In all of these proposals, the ascending part of the curve can be represented by a straight line, without significant effect on the accuracy of the model. In addition, the slope of this line is the modulus of elasticity of concrete which is an
invariable parameter of concrete in the elastic range Refs. [23, 24, 25, 27, 29, 30]. Therefore, the stress-strain curve of concrete can be modelled by a simple model as illustrated in Fig. 8.2.2.1. The tensile strength of concrete was considered to be zero. The stress-strain behaviour of the steel in the elastic range was considered as shown in Fig. 8.1.2.1.

As stated before an important elastic parameter for the finite element analysis of the models is Poisson's ratio. The results of experimental studies show that the Poisson's ratio for all of the specimens is relatively the same but larger than the unconfined Poisson's ratio of concrete which was measured for all of the concrete batches. Hence, the results of experimental studies for confined concrete were applied to the analytical model.

**8.2.3 Modelling the Specimens for Finite Element Analysis**

The general geometry and finite element meshing considerations in modelling an experimental specimen is to use symmetry and a minimum number of elements. The models in this study are axisymmetric composite steel circular hollow sections under axisymmetric loading. Hence, the axisymmetric solutions can be used for the analysis of the model. The available software for this purpose cannot be used for the axisymmetric problems with non-linear material properties, and as such a quarter of the specimen was considered for the analytical model.

![Fig. 8.2.3.1 Geometry and finite element meshing of the analytical model.](image)

The elements used for the models were brick and penta elements. Fig. 8.2.3.1 is the sketch of the geometry and finite element meshing of the models.
8.2.4 Loading Steps for the Analytical Model

For creating an analytical model without the effects of the supports conditions, the longitudinal displacements on the loading faces of the concrete core have to be equal. This form of loading follows the testing method in which loads were applied by the longitudinal deformation of the testing machine’s support. The magnitude of the axial deformation of the concrete core was determined so as to impose a major strain (in longitudinal direction of the model) of about 70% of the ultimate longitudinal strain as the first step of loading. In the second step of loading, the imposed strain was about the ultimate longitudinal strain, which had been determined at the experimental stages of this study. Each step was divided into twenty sub-steps to increase the accuracy of the computations, and to provide more information about the behaviour of the model.

8.2.5 Longitudinal Compressive Stresses in Concrete Core and Steel Tube

The analytical models which were used in this study represent the tested specimens. To study the general behaviour of the confined concrete in a steel tube, the specifications of the specimen No. Al (AII) were used. Results of the stresses are discussed in detail in order to obtain a clear idea of the stresses in the confined concrete and steel tube simultaneously. The rest of the specimens (Bl, Cl, and, Dl) have also been solved, but the results are summarised in appropriate tables to give a clear sense of the effect of confinement in the range of the dimensions in this study.

Considering the general data input in 8.2.1 to 8.2.4 and the results obtained for the steel and concrete, the detailed specifications of the analytical model are given in Table 8.2.5.1. The height for all of the analytical models was taken as 250 mm. It was assumed that the modulus of elasticity for the confined concrete is equal to the modulus of elasticity of the unconfined concrete. This assumption is in agreement with most of the proposed models for stress-strain behaviour of the concrete Refs. [23, 24, 25, 27, 29, 30].

<table>
<thead>
<tr>
<th>Model No.</th>
<th>Material properties for concrete</th>
<th>Material properties for steel tube</th>
<th>Load steps (Applied strain)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>f’c (MPa)</td>
<td>Ec (GPa)</td>
<td>Vc (Con.)</td>
</tr>
<tr>
<td>Al</td>
<td>71.0</td>
<td>40.0</td>
<td>0.45</td>
</tr>
<tr>
<td>Bl</td>
<td>61.0</td>
<td>40.0</td>
<td>0.45</td>
</tr>
<tr>
<td>Cl</td>
<td>72.0</td>
<td>40.0</td>
<td>0.45</td>
</tr>
</tbody>
</table>
where $K$ is the strength ratio of concrete as was defined in Chapter 4 (Table 4.6.4.1).

To study the distribution of axial stresses in the concrete core and steel tube, the complete results of the longitudinal stresses for model AI in all load steps is illustrated in Fig. 8.2.5.1.

![Graph](image)

**Fig. 8.2.5.1** The axial compressive stresses on the concrete core and steel tube.

The results that are presented in Fig. 8.2.5.1 show that the compressive stress on the concrete core in the different radial distances from the centerline is uniform. Stresses in the concrete core in regions C, D and E, as explained in 8.2.6 show different region of the concrete core by its radial distance.

### 8.2.6 Tangential Stresses In Concrete Core And Steel Tube

To evaluate the effect of the steel tube for confinement of the concrete core, the tensile stresses in the circumferential direction of steel tube have to be considered. It is also necessary to determine the intensity of the tangential stresses in the concrete core to evaluate the confinement effect from the transverse direction of the main confinement pressure from the steel tube. Fig. 8.2.6.1 represents the tangential stresses for the concrete core. The position of the results in Fig. 8.2.6.1 are as follows:

- (B): From $R = 33.46$ to $50.85$ mm
- (C): From $R = 19.65$ to $33.46$ “
- (D): From $R = 8.7$ to $19.65$ “
- (E): From $R = 0.0$ to $8.7$ “
The circumferential stresses in the steel tube are given in Fig. 8.2.6.3. In comparing the intensity of stresses in the steel tube and the concrete core, the stresses in the concrete are combined with the stress curve of the steel tube. As the results show, (Figs. 8.2.6.1 and 2), the concrete starts to yield at about -8.5 MPa (compressive stress), whereas the steel tube yields at 65.0 MPa (tensile stress) in the circumferential direction. Results for the steel tube are more or less in agreement with the experimental results at the early stages of yielding in the steel tube (4.6.6), where the Poisson’s ratio of concrete is less than 0.5.
Beyond the maximum circumferential stresses in the steel tube at a longitudinal strain of the model equal to 0.0022 (or 0.22%), the stresses do not increase, which is not in agreement with the experimental results (4.6.6).

![Graph of circumferential stress versus longitudinal strain](image)

**Fig. 8.2.6.3** Stress versus longitudinal strain in the model for steel tube and concrete core.

The reason for the constant circumferential stress in the steel tube lies in the uniform expansion of the steel tube at the middle of the specimen (axisymmetric structure and loading). The constant circumferential stress in the steel tube represents the theoretical limit of the confining pressure on the concrete core in the cross section of a composite column when it is removed from the effects of the loading supports.

### 8.2.7 Radial Stresses in Concrete Core and Steel Tube

This section deals with the intensity of the confinement stresses on the concrete core. Considering the theoretical results which are illustrated in Fig. 8.2.7.1, the confinement stress on the concrete core in the uniform part of the model is low. This result shows that even with the assumed Poisson's ratio equal to 0.45, that produces complete contact for the concrete core and the steel tube, the confinement of concrete is significantly low.

The values of the radial stresses in the steel tube and the concrete core tend to be lower than the stresses in other directions and they depend on the values of the Poisson's ratio in the steel tube and the concrete core.
Fig. 8.2.7.1 Confinement stress versus longitudinal strain in the model in concrete core.

If the values of Poisson's ratio for both the materials are equal, the radial stresses in the steel tube and the concrete core will be equal to zero. To summarise the result, the relationships for the Poisson's ratio in the composite section may be shown as follows:

\[ \nu_S = \nu_C \quad \text{Radial stress on steel and concrete core} = 0 \]
\[ \nu_S < \nu_C \quad \text{Radial stress on steel and concrete core} < 0 \text{ (compressive)} \]
\[ \nu_S > \nu_C \quad \text{Radial stress on steel and concrete core} > 0 \text{ (tensile)} \]

where

\[ \nu_S = \text{Poisson's ratio in the steel tube, and, } \nu_S = \text{Poisson's ratio in the concrete core.} \]

Considering the third case, which is the real situation for the concrete core and the steel tube in a circular hollow section filled with concrete, in an axially loaded column the concrete will behave independently from the steel tube. Although at the plastic stage of the axially loaded column, the Poisson's ratio of the concrete increases to a higher value and always is larger than the Poisson's ratio of the steel tube, nevertheless, the confining stress will be independent of the concrete compressive strength. Once again, this situation is an ideal condition where the effect of the loading supports is considered to be negligible.
8.2.8 Lateral Deformation of the Analytical Model A2I(A2II)

The deformation pattern of the composite section for the model A2I(A2II) is given in Fig. 8.2.8.1. The results represent a uniform expansion for the model at a few elements from the loaded end in the elastic range. But in the plastic range, the influence of the loaded end is much more significant and uniformity of the lateral expansion occurs in the five bottom elements (theoretically at the middle of the elements).

![Graph showing lateral deformation](image)

Fig. 8.2.8.1 Lateral (radial) deformation of the model A2I(A2II) at the end of each load step.

The theoretical failure pattern of the composite section, when the load is applied on the concrete only, proves that even in such cases the cross section of the specimen (model) after a short distance from the loading face is behaving as a composite section.

8.2.9 Load-Axial Shortening Curves for the Model A2I(A2II)

The results of the finite element analysis are compared with the experimental results, by the curve of axial load versus longitudinal shortening (Fig 8.2.9.1). As mentioned earlier in Chapter 4, the strength ratio (K) for the concrete core in this particular
example was considered to be equal 1.84. This value was determined according to the confining stresses on the concrete core in Sec 4.6.4 (Table 4.6.4.1). The axial load-shortening curve of the analytical model shows that the axial load capacity of the confined concrete is higher than that obtained experimentally. As a result the value obtained for the increase of the compressive strength of the concrete core is lower than the values resulted by the experimental tests (Table 4.6.4.1).

![Graph](image_url)

Fig. 8.2.9.1 Analytical curve for compressive load versus axial shortening.

To determine an accurate value of the strength enhancement for the concrete core, the compressive strength of the steel tube was considered to be equal to the $A_s f_y$. In this case the enhancement of the compressive capacity of the concrete will decrease to the value of $K = 1.43$. Considering this new value, the axial load-shortening curve of the specimen can be determined according to the curve obtained for $K=1.84$. More details and the comparison between the analytical curve and the experimental curve are given in 8.2.11.

According to the material properties introduced for the analytical model, the materials are completely elastic-plastic, and even the concrete does not exhibit strain-softening behaviour. Therefore, the results also show an elastic-plastic behaviour for the model.

### 8.2.10 Analytical Results for the Other Models

Although the material properties of the elements (concrete, steel, gap elements) are not the same, the general behaviour of the models is more or less similar. The structural characteristics of the models, as defined in 8.2.1 to 8.2.9 can be tabulated as follows.
The first parameters are the three principal stresses in the steel tube and the concrete core, and the other parameters are the mean stress in the concrete core and steel tube, and the Von Mises stress in steel tube. The results are given in Table 8.2.10.1.

<table>
<thead>
<tr>
<th>Model No.</th>
<th>Principal stress A</th>
<th>Principal stress B</th>
<th>Principal stress C</th>
<th>Mean stress</th>
<th>Von Mises Stress in steel</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Concrete (MPa)</td>
<td>Concrete (MPa)</td>
<td>Concrete (kN)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>A2(II)</td>
<td>-5.0</td>
<td>-7.2</td>
<td>-125.0</td>
<td>40.7</td>
<td>471†</td>
</tr>
<tr>
<td>B2(I1I)</td>
<td>-1.3</td>
<td>-1.5</td>
<td>-86.7</td>
<td>28.0</td>
<td>408©</td>
</tr>
<tr>
<td>C2(I1I)</td>
<td>-1.7</td>
<td>-3.2</td>
<td>-127.0</td>
<td>39.7</td>
<td>465‡</td>
</tr>
<tr>
<td></td>
<td>steel (MPa)</td>
<td>steel (MPa)</td>
<td>steel (kN)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>A2(II)</td>
<td>41.0</td>
<td>-5.0</td>
<td>-430</td>
<td>125</td>
<td></td>
</tr>
<tr>
<td>B2(I1I)</td>
<td>54.0</td>
<td>-4.0</td>
<td>-374</td>
<td>104</td>
<td></td>
</tr>
<tr>
<td>C2(I1I)</td>
<td>48.0</td>
<td>-9.5</td>
<td>-419</td>
<td>120</td>
<td></td>
</tr>
</tbody>
</table>

where:

© = At the load step 1.675 mm (Total longitudinal strain is equal to 0.197%).

‡ = At the load step 2.35 mm (Total longitudinal strain is equal to 0.318%).

8.2.11 Approximate Method for Determining the Axial Load-Shortening Behaviour of the Composite Section

As explained in 8.2.7, the structural and the load characteristics of the composite sections can be categorised as an axisymmetric problem. Therefore, the longitudinal stresses of the concrete and the steel tube may be considered for the analytical solution of this case. Accordingly, Eq. 8.2.11.1 can be used in all of the stages of load history in the composite section.

\[
P_C = f_{st}A_s + f_cA_c \tag{8.2.11.1}
\]

where:

\[A_s\text{, and } A_c\text{ = Cross sectional area of the steel tube and the concrete core, respectively.}\]

\[f_{st}\text{, and } f_c\text{ = Longitudinal stress of the steel tube and the concrete core, respectively.}\]

\[P_C\text{ = Axial load of the composite section.}\]

In this case, even the strain softening of the concrete can be considered for the approximate analysis of the composite section. Considering the stress-strain behaviour
of the steel tube and the concrete core as shown in Fig. 8.2.11.1, the axial load-shortening of the composite section can be estimated in five steps, as Eqs. 8.2.11.2.

![Stress-strain curve of the confined concrete and steel tube.](image)

Fig 8.2.11.1 Stress-strain curve of the confined concrete and steel tube.

1. $\delta < 1.\varepsilon_c$ \hspace{1cm} $P_c = f_{st}.A_s + f_c.A_c$
2. If $\varepsilon_c < \varepsilon_y$ then: $1.\varepsilon_c < \delta < 1.\varepsilon_y$ \hspace{1cm} $P_c = f_{st}.A_s + f_c.A_c$
   If $\varepsilon_c > \varepsilon_y$ then: $1.\varepsilon_c < \delta < 1.\varepsilon_y$ \hspace{1cm} $P_c = f_{y}.A_s + f_c.A_c$
3. $1.\varepsilon_y < \delta < 1.K_a.\varepsilon_c$ \hspace{1cm} $P_c = f_{y}.A_s + f_c.A_c$
4. $1.K_a.\varepsilon_c < \delta < 1.\varepsilon_{sh}$ \hspace{1cm} $P_c = f_y.A_s + A_c.\delta/l.K_b.E_c$
5. $1.\varepsilon_{sh} < \delta$ \hspace{1cm} $P_c = A_s \left( f_y.+(\delta/l).E_{sh}\right) + A_c.\delta/l.K_b.E_c$  \hspace{1cm} (8.2.11.2)

where:
- $K_a$ and $K_b$ are shown in Fig. 8.2.11.1.
- $\delta = \text{Axial shortening.}$
- $P_c = \text{Axial load.}$
- $l = \text{Height of the specimen.}$

Accordingly, the load-shortening curve of the analytical model can be represented as in Fig. 8.2.11.2. Considering the structural specifications of the specimen A2, the curve of the axial load-shortening can be obtained by Eq. 8.2.11.1. The structural specifications of the specimen A2 (I,II) are give in Table 8.2.11.1.
Based on the structural specifications in Table 8.2.11.1, the curve of axial load-shortening by approximate method is presented in Fig. 8.2.11.3. To compare the result obtained by this method and the experimental and finite element results, the curve of axial load-shortening by the later two methods are also presented in the same figure.

The approximate method and the finite element results are in a good agreement with the experimental ones in the ranges indicated. By determining the required parameters in the approximate method, the axial load-shortening curve of the confined concrete in a steel tube can be determined. The advantage of this method lies in the simplicity and
accuracy of the result that can be easily utilised in similar situations. Moreover, considering the limitations of finite element analysis for solving the structures with a softening material (MSC/NASTRAN), the approximate method is a good alternative for solving composite sections.

8.3 ANALYTICAL MODEL OF A COMPOSITE CIRCULAR HOLLOW SECTION FILLED WITH CONCRETE (LOAD ON BOTH, THE STEEL TUBE AND THE CONCRETE CORE)

8.3.1 General Characteristics

To study a realistic model that represents a composite section under axial load, an analytical model which is not affected by the support conditions has been considered. In this new model, the height is 30 mm, the outside diameter is 168.3 mm and the wall thickness is 4.8 mm. The generated mesh in this new model is shown in Fig. 8.3.1, which is repeated three times to cover the whole length of the model.

The material properties for the concrete core and the steel tube have been considered as in the previous analysis. The properties of the gap elements were determined to prevent the singularity in the global stiffness matrix during the non-linear solution. In addition, the tensile stiffness of the gap elements was considered to furnish a very small tensile capacity between the steel tube and the concrete core (the structural characteristics of the gap element have been determined by a trial and error method to avoid the singular situation of the global stiffness matrix throughout the non-linear solution).

To consider a model free from the support conditions, all of the grid points are released in the X-Y plane (the plane which is perpendicular to the longitudinal direction). The model in this study is an axisymmetric model with an axisymmetric load, therefore, a quarter of the model has been considered for the finite element analysis.

To study the effects of the Poisson’s ratio for concrete, two values of Poisson’s ratio were considered for this study. The first value is Poisson’s ratio which has been obtained from the tests on the concrete specimens (0.23), and the second value is an imaginary value (0.45) that is more or less in agreement with the results obtained from the composite section with load on the steel tube and the concrete core.
8.3.2 Analytical Results

The analytical results in the case of $v_c = 0.23$ indicate that the concrete core was isolated from the steel tube in both, the elastic and the plastic range. Under these conditions the concrete core has no confining stresses in the radial direction. In addition, according to the axisymmetric situation, the tangential stresses are also insignificant. As a result, the steel tube and the concrete core act independently, therefore, the approximate method that was discussed in 8.2.11 is applicable in this case.

In the reality, the propagation of the cracks in the concrete core produces a material with a high value of Poisson's ratio. To consider the expansion of the concrete core due to the cracks, a new model with the Poisson's ratio equal to 0.45 was solved. In this case the isolation of the concrete core and the steel tube does not happen. The important problem in this new model is to check the confining stresses in the concrete core, and to check how significant the confinement on the concrete is.

The analytical results in this case are similar to those in the previous studies in 8.2.6 and 8.2.7. Hence, in an ideal condition for a composite section, the confining stresses due to the steel tube is not significant. Consequently, use of the unconfined compressive capacity for computing the ultimate load capacity of a composite circular hollow section under the concentric loads is recommended.
Fig. 8.3.2 Finite element meshing and the geometry of the analytical model after loadings for $v_c = 0.23$. 
8.4 SUMMARY

A series of non-linear finite element analyses for the circular hollow sections and circular hollow sections filled with concrete have been carried out in this chapter. The gap elements have been used for modelling the interaction of the concrete core and the steel tube.

The results of axial load shortening behaviour obtained by these analyses are in a good agreement with the experimental results. In addition, an approximate method has been suggested for estimation of the axial load-shortening behaviour of the composite sections. The results by the approximate method are also in a good agreement with those obtained from the experiments.

The analytical results show that under ideal conditions for a composite circular hollow section filled with concrete which is not affected by the support conditions, the effects of the radial and tangential stresses in the concrete core are negligible. Whereas, the experimental results, because of the constraints at the supports and formation of cracks in the concrete core after the peak load, are not in agreement with results of radial and tangential stresses in the concrete core obtained by the finite element analyses.
CHAPTER NINE

CONCLUSIONS AND RECOMMENDATIONS FOR FURTHER STUDIES

9.1 CONCLUSIONS

Based on the experimental and theoretical results outlined in previous chapters, the following conclusions are made.

9.1.1 Experimental and Theoretical Studies on the Ultimate Load Capacity of CHS

The test results for the bare steel tube found herein furnish conservative results when compared with both of the codes (DIN 18800 [7], and, ECCS [18]), but do not show good agreement with the results from Ref. [1]. It seems the reason for the disagreement of the results lies in the preparation of the specimens. As it was explained in the report (Ref. [1]), the specimens were machined to obtain different wall thicknesses and diameters from one source tube. This process may have an effect on the distribution of residual stresses and, more importantly, the material properties may be different throughout the wall thickness of the tube. This is a significant parameter especially in thick walled tubes, Ref. [1].

According to the reported experimental results and the test results in this study to justify the theory of folding elements at the plastic stage, the maximum value of the peak loads in a CHS has a descending rate throughout a number of folding cycles. As the edges of the tube are completely constrained at the first cycle of loading, the eccentricity is at its maximum value. Therefore, the tubes under axial compression show the highest load capacity at the beginning of the loading. Rationally, the load capacity of the tube will approach to a limiting value of the eccentricity after a number of folding cycles.

For determining the average strain from three strain gauges in a bare steel tube an averaging method was proposed. By use of this method the average of strains can be computed accurately. In addition, all geometric imperfections that produce unbalanced behaviour for longitudinal and transverse strains, in a compressed steel tube can be corrected by this method.
The interrelation between the normalised ultimate strain and the normalised slenderness, Ref [1], displays the best relationship and might be the best method to explain the ultimate load behaviour of bare CHS columns. The results of this study show that both of the apparent and classical values, which were used to obtain the parameters of normalised ultimate strain ($\bar{\varepsilon}_y$) and normalised slenderness ($\bar{\lambda}$), are fitted with two lines to a good accuracy.

Based on the method used for computing the stresses from measured strains for thick walled tubes with $d/t \leq 18$, the effect of residual stresses and the deformation patterns due to the dimensions is significant for obtaining stresses from measured strains. Conversely, for relatively thin walled tubes with $d/t \geq 35$, this effect is more significant at the final stages of loading, and the rest of the stresses are in a good agreement with the direct measurements of the stresses.

9.1.2 The Mechanical Behaviour of Confined Concrete in Steel Tubes

Based on the shape of the stress-strain curves for the confined concrete in the steel tubes, the thick walled tubes with $d/t \leq 18$ have less drop in sustainable axial load after the ultimate point than the thin walled tubes with $d/t \geq 35$. Based on the Eq. 4.7.1 for a strain of 0.035 in the longitudinal direction, the ductility factors for the thick walled specimens, are within the range of 0.94-0.97, and for the thin walled specimens the ductility factors are 0.77 to 0.82. Moreover, in contrast to the behaviour of bare hollow sections, the peak longitudinal shortening at peak strength of the composite section is more or less constant (plastic behaviour), and shows that the concrete predominantly controls the whole behaviour of the composite column.

Another important result in the case of stub column tests is the deformation pattern of thick walled tubes under axial compression. As it was shown by Eq. 4.6.1.2, overall bulging deformation produces an overall longitudinal strain in the steel tube opposite to that of the longitudinal compressive strain in the outer surface of the steel tube. This deformation decreases the confinement pressure on the concrete mass. Therefore, the confinement is less effective than that for the thin walled tube.

The effect of the increase of confinement pressure for both thick and thin walled tubes on the ultimate load capacity of the concrete core is limited to a specific value (Fig. 4.6.4.1). This value depends on the unconfined compressive capacity of the concrete and the dimensions of the steel tube, as well as the material properties of the steel tube, such as elastic parameters. However, according to results obtained for the
compressive strength of the concrete in stub column tests, the compressive capacity of confined concrete in steel tube reached to about 1.8 times of the unconfined compressive strength.

To estimate the ultimate load capacity of concrete confined in the steel tubes, Eqs. 4.6.4.3, 4 can be used. The difference of these equations from the available equations is the subtraction of the strength of the steel tube from the general form of the equation for compressive capacity, Eq. 4.6.4.2. The experimental results of this study show that the enhancement of concrete compressive capacity is not as high as was suggested by analytical equations, such as that presented by Cai (Eq. 4.2.1).

The drop of load capacity of the specimens after reaching the peak load depends on the dimensions of the steel tube and the material properties of both steel and concrete. The results of this study show that at about 0.035 axial strain in the longitudinal direction the load capacity of the specimens has reached a plastic plateau, and therefore represents the residual load of the sections.

9.1.3 Ultimate Axial Load Capacity of Stocky Steel CHS filled with Higher Strength Concrete, Load on Steel and Concrete, and Load on Steel Tube only.

Regarding the behaviour of higher strength concrete (f'_c > 40 MPa) in the composite circular hollow sections, to achieve an efficient use of material, it seems to be necessary to use internal reinforcement for preventing the formation of major cracks in the concrete core. Therefore, the cracks will be distributed in the concrete mass so that the drop of the post peak load capacity will be reduced. The effect of major crack formation for the thin walled specimens with d/t > 35 is more significant than the specimens with d/t ≤ 18.

The result of this study indicates that the higher strength concrete can be used successfully in the composite thick walled steel circular hollow sections with d/t ≤ 18. Considering the post peak load behaviour of the composite column the higher strength concrete behaves as a ductile material.

Converse to the stress-strain curves of confined concrete in Chapter 4 (load on concrete core only), the drop of the longitudinal stresses in the steel tube for all of the cases in Chapter 5 (load on steel tube and concrete core) is not observed in this case. The principal reason for this situation is the lower circumferential stresses in the steel tube for the case of load on both the steel tube and the concrete core.
Based on the experimental results, proposed method for computing the compressive strength in Draft Eurocode 4 has a reasonable margin of safety, that is resulted from the lower confinement effect on the concrete core than the proposed equation in Ref. 33.

9.1.4 Ultimate Load Behaviour of Composite Steel CHS under Eccentric Loading

Considering the estimations obtained analytically for the ultimate load capacity of the eccentrically loaded columns, the proposed equations by Cai do not provide a safe evaluation for the ultimate load capacity for all of the specimens, especially for the thick walled tubes.

In the case of small eccentricities, the drop in post-peak load carrying capacity of the columns is greater than that for the specimens with higher eccentricity. In other words, the movement of the neutral axis to its limiting value (section under pure moment), is much faster.

The proposed method for evaluation of the axial load capacity of the composite short columns in Chapter 7 shows good agreement with the test results. By use of a suitable set of safety factors for the material properties, the method can be safely used to estimate the ultimate load capacity for stocky columns ($l/r \leq 11$) for design purposes.

9.1.5 Finite Element Analysis

The results of the finite element analyses show that under ideal conditions for a composite section, the effects of the radial and tangential stresses in the concrete core are negligible. However, the experimental results, because of the constraints at the supports, are not in agreement with results obtained by the finite element analysis. Consequently, use of the unconfined compressive capacity of concrete for computing the ultimate load capacity of a composite circular hollow section under the concentric loads is recommended.

The approximate method that is proposed for the axial load-shortening behaviour of concentrically loaded composite column is in a good agreement with the experimental results in the elastic and plastic ranges. The advantage of this method lies in the simplicity and accuracy of the result that can be easily utilised in the similar situations. Moreover, considering the limitations of finite element analysis for solving
the structures with a softening material (MSC/NASTRAN), the approximate method is a good alternative for solving composite sections.

9.2 RECOMMENDATIONS FOR FURTHER WORK

To improve the structural characteristics of the circular hollow sections filled with concrete the axial stress in the steel tube has to be limited to a certain value. In other words, it is more beneficial to change the direction of the maximum stress in the steel tube for the circumferential direction instead of longitudinal direction. In the conventional form of the beam-column joints the axial load is directly applied to the steel tube, therefore, based on the axisymmetric situation of the structure and the loads, the steel tube in the circumferential direction has a very low influence on the concrete core.

The conventional beam-column joint can be modified by utilising a new form of beam-column joint that is shown in Appendix I. In this proposal, the axial load from the floor system can be directly transferred to the concrete core by a separate composite steel tubular section with a suitable wall thickness where the outside diameter of this section is almost equal to the internal diameter of the main column. The steel tube in the main column will then be more effective in the circumferential direction so that the concrete is confined to a greater degree. Moreover, in this new system pre-cast columns can be readily used in structural frames.

Another important problem for the composite steel circular hollow section filled with high strength concrete is the formation of major cracks in the concrete core in the case of low confining stresses. A solution for this case is the use of reinforcement in these kinds of composite sections. Regarding the differences of the composite sections from the conventional reinforced concrete structures, the requirements for the minimum and maximum percentage of the reinforcement and the other conditions in a concrete structure have to be investigated.

The design of the structural joints in composite sections is also an important problem in the design of this kind of column. The available recommendations are based on a large factor of safety, and in most of the cases the designs are based on the experience and engineering judgement of the designer. With this regard, to improve the safety and the economy of this kind of composite section, detailed specifications for the design of the structural joints is a first priority.
One of the most important problems in the structural behaviour of composite circular hollow sections is the ductility of these kind of columns and joints in seismic areas. This case has been studied extensively for conventional reinforced structures, but few reports are addressed to the composite column problem. Obviously, for the stability analysis of composite frames in seismic areas the loading and unloading behaviour of the composite structures needs to be considered as a key part of the required data.

The limitation on the compressive capacity of the concrete is also an important problem. According to most of the codes of practice for the design of reinforced concrete structures, the limitation of the compressive capacity prevents the brittle failure of the structural frame. Unfortunately, this problem has not been investigated widely in the case of composite steel tubes to date. More results of such a study will be useful where high strength concrete is used in the core.
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APPENDICES

APPENDIX A: PROGRAM NO. ONE: "COMPUTATION OF FESTOON CURVE FOR ULTIMATE ELASTIC BUCKLING LOAD OF STEEL TUBES"

```plaintext
DIMENSION Q(11,300),QL(300)
PRINT *, 'OUTSIDE DIAMETER/2='
READ *, A
PRINT *, 'WALL THICKNESS='
READ *, T
PRINT *, 'MODULES OF ELASTICITY='
READ *, E
PRINT *, 'POISSON S RATIO='
READ *, V
PARAMETER (PI=3.14159265)
XK=(T**2)/(12*(A**2))

2000 XM=XM+1
IF (XM.EQ.11) GOTO 10
DO 100 B=1,280
IF (B.LE.100) XL=(PI/B)*100
IF ((B.GT.100).AND.(B.LE.190)) XL=PI/(1+(B-100)/10)
IF (B.GT.190) XL=PI/(B-180)
QA=2*(V*(XL**6)+3*(XL**4)*(XM**2)+(4-V)*(XL**2)*(XM**4) + XM**6)
QB=2*(2-V)*(XL**2)*(XM**2)+(XM**4)
QC=(1-V**2)*(XL**4)+XK*(((XL**2)+(XM**2))**4-QA+QB)
QD=(XL**2)*(((XL**2)+(XM**2))**2)+(XL**2)*(XM**2)
Q(XM,B)=(QC/QD)* 10000
100 CONTINUE
GOTO 2000
10 DO 110 I=0,280
R=1
IF (Q(R,I).LT.Q(R+1,I)) THEN
MIN=Q(R,I)
ELSE
MIN=Q(R+1,I)
ENDIF
20 R=R+1
IF (R.GT.9) GOTO 30
IF (MIN.GT.Q(R+1,I)) THEN
MIN=Q(R+1,I)
ELSE
GOTO 20
ENDIF
GOTO 20
ENDIF
30 QL(I)=MIN
PRINT *, 'QL(',I,')=',QL(I)
110 CONTINUE
OPEN (UNIT=3,FILE='FESTOON1',STATUS='NEW')
OPEN (UNIT=4,FILE='FESTOON2',STATUS='NEW')
WRITE (3,*) 'NO. L/(N*A)  Q2  ' DO 120 A=0,280
IF (A.LE.100) THEN
XX=A/100
```

APPENDIX B: PROGRAM NO. TWO "COMPUTATION OF EQUIVALENT STRAIN IN LONGITUDINAL AND TRANSVERSAL DIRECTIONS".

C

THIS PROGRAM COMPUTES THE EQUIVALENT STRAINS.
DIMENSION S(100),V(100,3),H(100,3),VV(100),HH(100),K(100)
PRINT *, 'NUMBER OF DATA SET=
READ *,N
OPEN (2, FILE='NODF1', STATUS='OLD')
DO 1 I=1,N
READ (2,90) K(I),V(I,1),V(I,2),V(I,3),H(I,1),H(I,2),H(I,3),S(I)
1 CONTINUE
PRINT *, 'NO. V1 V2 V3 H1 H2 H3 VERT.LOAD'
DO 2 I=1,N
PRINT 90,K(I),V(I,1),V(I,2),V(I,3),H(I,1),H(I,2),H(I,3),S(I)
2 CONTINUE
PARAMETER (PI=3.14159265)
PRINT *, 'THE OUTSIDE DIAMETER OF CHS(MM)='
READ *,DDD
P=DDD*PI
DO 3 I=1,N
STRR=V(I,1)
STRM=V(I,2)
STRL=V(I,3)
IF ((STRM.EQ.0).OR.(STRR.EQ.0).OR.(STRL.EQ.0)) THEN
STRA=(STRL+STRM+STRR)/2
ELSE
GOTO 131
ENDIF
GOTO 130
131 CALL CONS(STRR,STRM,STRL,STRA,P)
130 VV(I)=STRA
STRR=H(I,1)
STRM=H(I,2)
STRL=H(I,3)
CALL CONS(STRR,STRM,STRL,STRA,P)
HH(I)=STRA
C
3 CONTINUE
PRINT *,"WOULD YOU LIKE TO SAVE THE RESULTS 0=N, 1=Y ?"
READ *,Z
IF (Z.EQ.0) GOTO 20
OPEN (3,FILE=NODF2,STATUS='NEW')
PRINT *,NO. VSTN HSTN LOAD(TON)
DO 41=1,N
WRITE (3,91) I,VV(I),HH(I),S(I)
4 CONTINUE
90 FORMAT(3X,I3,6(F7.0,1X),F9.2)
91 FORMAT(I3,F10.1,F10.1,F15.5)
20 END
SUBROUTINE CONS(STRR,STRM,STRL,STRA,P)
SLR=(STRM-STRR)/(P/4)
SLL=(STRM-STRL)/(P/4)
IF (ABS(SLR).GT.ABS(SLL)) SL=ABS(SLR)
IF (ABS(SLR).LT.ABS(SLL)) SL=ABS(SLL)
IF (ABS(SLR).EQ.ABS(SLL)) SL=ABS(SLR)
R=((P/2)*SL)/2
IF ((SLR.GT.0).AND.(SLL.LE.0)) THEN
   GOTO 10
ELSE
   GOTO 11
ENDIF
10 IF (ABS(SLR).EQ.ABS(SLL)) STRA=STRL-R
   STRA1=(((P/2)-(STRM-STRL)/SL)/2)*SL+STRL-R
   IF (ABS(SLL).GT.ABS(SLR)) STRA=STRA1
   STRA2=(((P/2)-(STRM-STRM)/SL)/2)*SL+STRM-R
   IF (ABS(SLR).EQ.ABS(SLL)) STRA=STRA2
   IF (SLL.EQ.(0.0)) STRA=STRM+((P/8)*SL)-R
   GOTO 12
11 IF ((SLR.GE.(0.0)).AND.(SLL.GT.(0.0))) THEN
   GOTO 13
ELSE
   GOTO 14
ENDIF
13 STRA3=(((P/4)-(STRM-STRL)/SL)/2)*SL+STRM-R
   IF (ABS(SLR).GT.ABS(SLL)) STRA=STRA3
   IF (ABS(SLL).EQ.ABS(SLR)) STRA=STRM-R
   STRA4=(((P/4)-(STRM-STRR)/SL)/2)*SL+STRM-R
   IF (ABS(SLL).GT.ABS(SLR)) STRA=STRA4
   IF (SLL.EQ.(0.0)) STRA=STRM+((P/8)*SL)-R
   GOTO 12
14 IF ((SLL.GE.(0.0)).AND.(SLR.LT.(0.0))) THEN
   GOTO 15
ELSE
   GOTO 16
ENDIF
15 RR=((P/2)-(STRR-STRL)/SL)/2)*SL
   STRA5=(((P/4)-(STRR-STRM)/SL)/2)*SL+STRM-R
   IF (ABS(SLL).GT.ABS(SLR)) STRA=STRA5
   IF (ABS(SLL).EQ.ABS(SLR)) STRA=STRR-R
   IF (ABS(SLR).GT.ABS(SLL)) STRA=RR+STRR-R
Appendices

APPENDIX C: PROGRAM NO. THREE: "COMPUTATION OF STRESSES FROM MEASURED STRAINS"

C

THIS PROGRAM COMPUTES THE STRESSES FROM MEASURED STRAINS.
DIMENSION V(100),H(100),D(100),VV(100,10),SV(100)
DIMENSION SH(IOO)
DIMENSION SS(100,10),S(100),K(100),XV(100)
PARAMETER (PI=3.14159265)
PRINT *,'NUMBER OF DATA SET='
READ *,N
OPEN (2,FILE=,NODF2^STATUS=,OLD,)
PRINT *,'NO. VER STRN HOR STRN VER STRS'
DO 11=1,N
READ  (2,90) K(I),V(T),H(I),S(I)
PRINT 86,Ka),V(T),Ha),S(D
V(T)=V(I)/(lE+6)
H(I)=H(I)/(lE+6)
1 CONTINUE
PRINT *,'OUTSIDE DIAMETER (MM)='
READ *,DMR
PRINT *,'WALL THICKNESS (MM)='
READ *,TKS
PRINT *,IS THE TUBE AN IMPROVED SEAM WELDED TUBE Y=1 N=0'
READ *,T
CSA=((DMR**2)-((DMR-TKS*2)**2))*PI/4.0
DO 10 I=1,N
S(I)=(S(I)*10000.)/CSA
10 CONTINUE
IF ((DMR.EQ.168.3).AND.(TKS.EQ.4.8).AND.(T.EQ.0)) THEN
C Data for elastic properties of CHS (168.3-4.8, and , coated)
UN=.139 E=230260
C Data for Ramberg Osgood Values _"_,
ONN=.0637499 OKK=591.3079723
PRINT *,*168.3-4.8 ORDINARY TUBE (BLACK)'
PAUSE
GOTO 1001
ENDIF
1001 IF ((DMR.EQ.168.3).AND.(TKS.EQ.4.8).AND.(T.EQ.1)) THEN
E=221330
END
UN=.222
OKK=607.8248411
ONN=.0479074
PRINT *,'168.3-4.8 ULTRA PRESSURE TUBE (RED)'
PAUSE
ELSE
GOTO 1011
ENDIF
1011 IF  (DMR.EQ.168.3).AND.(TKS.EQ.9.53).AND.(T.EQ.0)) THEN
E=302800
UN=.319
OKK=1277.766508
ONN=.054893
PRINT *,'168.3-9.53 ORDINARY TUBE (BLACK)'
PAUSE
ELSE
GOTO 1021
ENDIF
1021 IF  ((DMR.EQ.114.3).AND.(TKS.EQ.6.3).AND.(T.EQ.0)) THEN
E=296400
UN=.28
OKK=959.8732711
ONN=.1143268
PRINT *,'14.3-6.3 ORDINARY TUBE (BLACK)'
PAUSE
ELSE
PRINT *,'CHEK THE DIMENSIONS AGAIN!'
PAUSE
ENDIF
SMU=E/(2*(1+UN))
SLAN=(E*UN)/((1+UN)*(1-2*UN))
DO 888 J=1,N
XV(J)=V(J)
888 CONTINUE
DO 3 I=1,N
D(I)=V(I)-Sa)/E
IF (D(I).LT.(.0020)) GOTO 3
IF (D(I).EQ.(.0020)) GOTO 101
104 L=L+1
W(I,L)=(V(T)+V(I-l))/2.0
SS(I,L)=(S(T)+S(I-l))/2.0
D(T)=Wa,L)-SS(I,L)/E
IF (pa).GE.(.001990)).AND.(D(I).LT.(.00201)) GOTO 103
IF (D(I).GT.(.0020)) GOTO 100
V(I-1)=W(I,L)
Sa-D=SS(I,L)
GOTO 104
100 V(I)=VV(I,L)
Sa)=ssa,L)
GOTO 104
103 SY=SS(I,L)
VY=VV(I,L)
GOTO 105
101 SY=S(I)
VY=V(I)
GOTO 105
3 CONTINUE
Do 505 \( J = 1, N \)
\( V(J) = -1.0 \times X V(J) \)
\( S(J) = -1.0 \times S(J) \)
CONTINUE
Do 15 \( I = 1, N \)
IF (S(I).GE.(.-55*SY)) GOTO 106
IF (S(I).LT.(.-55*SY)) GOTO 108
SV(I) = \( (E/(1-UN**2))*(V(I)+UN*H(I)) \)
SH(I) = \( (E/(1-UN**2))*(H(I)+UN*V(I)) \)
GOTO 15
S1 = 2*SV(I-1)-SH(I-1)
S2 = 2*SH(I-1)-SV(I-1)
S3 = -(SV(I-1)+SH(I-1))
SVPR = SV(I-1)
HPM = \( 1/(1/OKK)*(1/ONN)*((SVPR/OKK)**((1/ONN)-1)) \)
DSNV = \( V(I+1)-V(I) \)
DSNH = \( H(I+1)-H(I) \)
SE = \( 1.5*(S1*S1+S2*S2+S3*S3)**.5 \)
F = \( (18*(SMU**2))/((2*HPM+6*SMU)*(SE**2)) \)
DSND = \( (F*S1*S3-SLAN)*DSNV+((F*S2*S3-SLAN)*DSNH \)
DSND = \( DSV+(SLAN-F*S1*S3)*DSND \)
DSH = \( DSH+(SLAN-F*S2*S3)*DSND \)
SV(I) = SV(I-1)+DSV
SH(I) = SH(I-1)+DSH
15 CONTINUE
PRINT *, 'NO. VER. STRN HOR. STRN STRN --> VER. STRS HOR. + STRS YIELD. STRN'
DO 999 1 = 1, N
V(I) = XV(I)*(1E+6)
H(I) = H(I)*(1E+6)
WRITE (*,87) I, V(I), H(I), S(I), SV(I), SH(I), SY
999 CONTINUE
OPEN (UNIT=3, FILE='NORF, STATUS='NEW')
WRITE (3,*) 'NO. VER. STRN HOR. STRN VER*STRS --> VER. STRS HOR. + STRS YIELD. STRS'
WRITE (3,*) 'NO. VER. STRN HOR. STRN VER*STRS --> VER. STRS HOR.'
DO 21 1 = 1, N
WRITE (3,87) I, V(I), H(I), S(I), SV(I), SH(I), SY
21 CONTINUE
86 FORMAT(1X,1I3,1X,3(F8.1,1X))
87 FORMAT(1X,1I3,1X,3(F8.1,1X),4X,2(F8.2,2X),1X,F10.4)
90 FORMAT(I3,F10.1,F10.1,F15.5)
500 END

APPENDIX D CALIBRATION OF THE TESTING MACHINE FOR COMPRESSION LOAD

To achieve the most accurate results from the tests, it is necessary to check the calibration of the testing machine. The usual method for calibration of the testing machines is to use standard electronic or hydraulic load cells. In this regard, a standard
hydraulic load cell was used to check the calibration of the testing machine. Results of the readings for the load cell and the dial gauge of the testing machine are shown in Table D.1. As it is shown in the Table C.1 the dial gauge of the testing machine was accurate for the experimental program and the small differences are not significant.

<table>
<thead>
<tr>
<th>Load Step</th>
<th>Capacity=1.0 (MN)</th>
<th>Capacity=2.5 (MN)</th>
<th>Capacity=5.0 (MN)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Load (kN)</td>
<td>Load (kN)</td>
<td>Load (kN)</td>
</tr>
<tr>
<td>1</td>
<td>100</td>
<td>96</td>
<td>92</td>
</tr>
<tr>
<td>2</td>
<td>204</td>
<td>192</td>
<td>193</td>
</tr>
<tr>
<td>3</td>
<td>304</td>
<td>292</td>
<td>299</td>
</tr>
<tr>
<td>4</td>
<td>405</td>
<td>400</td>
<td>400</td>
</tr>
<tr>
<td>5</td>
<td>506</td>
<td>500</td>
<td>503</td>
</tr>
<tr>
<td>6</td>
<td>608</td>
<td>605</td>
<td>603</td>
</tr>
<tr>
<td>7</td>
<td>709</td>
<td>703</td>
<td>704</td>
</tr>
<tr>
<td>8</td>
<td>812</td>
<td>803</td>
<td>810</td>
</tr>
<tr>
<td>9</td>
<td>916</td>
<td>905</td>
<td>911</td>
</tr>
</tbody>
</table>

where M and L represent the readings from the testing machine and the load cell, respectively.

**APPENDIX E CALIBRATION OF THE MEASURING TOOLS FOR AXIAL SHORTENING AND STRAIN RATE OF THE TESTING MACHINE**

For measuring the axial shortening of the specimens during the tests, an LVDT with the maximum voltage of 10.0 volts was used. The input voltage of the LVDT was supplied by a DC electric adaptor in the range of 1 to 10 volts. To change the output voltage in the LVDT to the equivalent displacement an HP computer was used. For calibrating the computer, it was necessary to find out the relationship between the output voltage of LVDT and the axial displacement. In this regard, a special form of micrometer was used and the result of the calibration is shown in Fig. E.1.

The input voltage from the electricity supply to the LVDT was +9.73 volts. For the same input voltage to the LVDT, the equation in Fig. E.1 can be used for measuring the axial displacement of the specimen.

Another important parameter for testing the specimens is the strain rate adopted when using the testing machine. Although the machine which was used for the testing program is a standard testing machine, the readings have to be justified by other measuring tools to make sure that the system does not make any systematic errors for the results of the experiments. To draw the axial movement of the supports of the
testing machine versus time an LVDT and an HP computer was used. The output voltage of the LVDT was transferred to the computer and was combined with time to draw the axial movement of the testing machine versus the time. The results of the readings for the different rates on the testing machine is given by Table E.1. The measurements of the longitudinal displacements was performed according to the Fig E.1 and the time was measured by computer.

![Fig. E.1 Output voltage of LVDT versus longitudinal displacement.](image)

**Equation for changing the voltage to the axial displacement:**
\[ \Delta L = -14.242V + 26.357 \]

<table>
<thead>
<tr>
<th>Data No.</th>
<th>Strain rate on the testing machine</th>
<th>Starting Voltage ((V_0)) (V)</th>
<th>Finishing voltage ((V_{60})) (V)</th>
<th>Strain rate by computer ((\text{mm/Sec}))</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0%</td>
<td>+1.145</td>
<td>+1.168</td>
<td>0.0054</td>
</tr>
<tr>
<td>2</td>
<td>1%</td>
<td>+1.145</td>
<td>+1.194</td>
<td>0.0120</td>
</tr>
<tr>
<td>3</td>
<td>2%</td>
<td>+1.145</td>
<td>+1.213</td>
<td>0.0160</td>
</tr>
<tr>
<td>4</td>
<td>3%</td>
<td>+1.146</td>
<td>+1.234</td>
<td>0.0200</td>
</tr>
<tr>
<td>5</td>
<td>4%</td>
<td>+1.147</td>
<td>+1.259</td>
<td>0.0267</td>
</tr>
<tr>
<td>6</td>
<td>5%</td>
<td>+1.148</td>
<td>+1.284</td>
<td>0.0320</td>
</tr>
<tr>
<td>7</td>
<td>10%</td>
<td>+1.153</td>
<td>+1.420</td>
<td>0.0635</td>
</tr>
<tr>
<td>8</td>
<td>15%</td>
<td>+1.160</td>
<td>+1.613</td>
<td>0.1078</td>
</tr>
<tr>
<td>9</td>
<td>20%</td>
<td>+1.168</td>
<td>+1.763</td>
<td>0.1417</td>
</tr>
<tr>
<td>10</td>
<td>25%</td>
<td>+1.163</td>
<td>+1.819</td>
<td>0.1562</td>
</tr>
</tbody>
</table>
APPENDIX F: THE LIST OF THE FORTRAN PROGRAM FOR CORRECTION OF LONGITUDINAL STRESSES ACCORDING TO THE RESULTS OF HOLLOW SECTION TESTS.

```
DIMENSION S(100), SMA(IOO), SMC(100), EQV(IOO), CORA(IOO), CORC(IOO)
PRINT *, 'NUMBER OF DATA SET FOR CORRECTION CURVE'
READ *, N
OPEN(1,FILE='CORT1.DAT',STATUS='OLD')
DO 1 I=1,N
READ (1,*) S(I), SMA(I), SMC(I)
1 CONTINUE
PRINT *, 'NUMBER OF DATA SET FOR UNDER CORRECTION SET='
READ *, M
OPEN(2,FILE='T1EQV.TXT',STATUS='OLD')
DO 2 I=1,M
READ (2,*) EQV(I)
2 CONTINUE
DO 3 I=1,M
DO 4 J=1,N
IF ((EQV(I).LE.S(J+1)).AND.(EQV(I).GT.S(J))) THEN
CORC(I) = ((SMC(J+1)-SMC(J))/(S(J+1)-S(J)))*(EQV(I)-S(J-1))
CORA(I) = ((SMA(J+1)-SMA(J))/(S(J+1)-S(J)))*(EQV(I)-S(J-1))
GOTO 3
ELSE
GOTO 4
ENDIF
4 CONTINUE
3 CONTINUE
OPEN(3,FILE='T1EQV',STATUS='NEW')
DO 5 I=1,M
WRITE (3,*) EQV(I), CORA(I), CORC(I)
5 CONTINUE
```

APPENDIX G A MULTIPURPOSE FORTRAN PROGRAM FOR COMPUTING THE ULTIMATE LOAD AND BENDING MOMENT IN COMPOSITE COLUMNS AND DESIGN.

```
DIMENSION XI(105), YI(105), XC(105), YC(105), XS(105), STR(251)
DIMENSION FCC(101,251), FS(105,251), MTOTNA(251), MTOTNC(251)
DIMENSION XMC(105,251), XMS(105,251), FTOT(251), CMTOT(251), XBB(251)
DIMENSION FTOTN(251), XMTOTN(251), XXX(20,70), XMULT(20,70), DPU(255)
WRITE(*,*) 'INPUT THE FILENAME FOR COMPARING CURVES'
READ(*,10) FNAME
10 FORMAT(A11)
WRITE(*,*) 'INPUT FILE NAME FOR THE DESIGN CURVES'
```
READ(*,10)GNAME
OPEN(2,FILE=FNAME,STATUS='UNKNOWN')
OPEN(3,FILE=GNAME,STATUS='UNKNOWN')
OPEN(5,FILE='DATA',STATUS='OLD')
READ(5,*) R,T,FC,EC,FY,ES,XMAXSTR
READ(5,*) N,USTRN
  CONA=EC/FC
  CONB=2*CONA
  ESOFT=EC/40
  STL=FY/ES
  STLB=STL*10.0
  EHAD=ES/40
  TTTT=T
  WRITE(*) 'ITERATION FOR THE NUMBER OF WALL THICKNESS='
  READ(*,*) NUM1
  WRITE(*) 'ITERATION FOR THE NUMBER OF OUTSIDE DIAMETER='
  READ(*,*) NUM2
  WRITE(*) 'INITIAL ECCENTRICITY (MM) ='
  READ(*,*) EI
  ETOT=EI+(10.0+EI*0.03)
  DO 20000 NN=1,NUM2
    DO 10000 KK=1,NUM1
      REALMU
      AC=((R-T)**2.0)*3.141592654
      AS=((R**2.0)*3.141592654)-AC
      PULT=(AC*FC)+(AS*FY)
      TX=0.5*(2*R-2*T)*(1-(1-((0.48*FC)/FY))**0.5)
      TET=COS(AS/(TX*(2*R-2*T-TX)+2*T*(2*R-T)))
      TETA=(ACOS((1-(2*(T+TX)/(2*R)))**3.0))*TET
      ECCCC=3*(2*TETA-(SIN(TETA))**2.0)
      ECC=(2*(2*R-2*T)**2.0)*((SIN(TETA))**3.0))/ECCCC
      ESSS=((2*R)**2.0-((2*R-2*T)**2.0))*3*TETA
      EU=ECCC+(2*TETA/3.1415926543)*(ESSS-ECCC)
      MU=AS*FY*EU
      PAM=(0.4*FC)/FY
      TOLER=5.0
      30005 DO 30000 11=1,200
        XXI=(II/800.0)*3.141592654
        RSIDE=PAM*((2*R-2*T)**2.0)*(3.141592654-SIN(2*XXI)-2*XXI)
        XLSIDE=16*(2*R-T)*T*XXI
        IF ((ABS(RSIDE-XLSIDE)).LE.TOLER) THEN
          GOTO 30012
        ELSE
          GOTO 30000
        ENDIF
      30000 CONTINUE

TOLER=TOLER+5.0
IF (TOLER.GT.500) GOTO 30010
GOTO 30005

30008 DO 30002 II=ID,IU
XXI=(II/8000.0)*3.141592654
RSIDE=PAM*((2*R-2*T)**2.0)*(3.141592654-SIN(2*XXI)-2*XXI)
XLSIDE=16*(2*R-T)*T*XXI
IF ((ABS(RSIDE-XLSIDE)).LE.(TOLER/10)) THEN
SMOD=(T**3.0)*(((2*R/T)-1)**2.0))
AL=II/4000
SMOM=(T*((2*R-T)**2.0)*((AL)*SIN(AL)+COS(AL)-1))
SMOMR=(((COS(AL)**3.0)/3.0)-.25*(SIN(AL))*3.1416-SIN(2*AL)-2*AL)
SMOMR=SMOMR*(0.25*PAM*((2*R-2*T)**3.0))
SMOM=(100/SMOD)*(SMOM+SMOMR)
UMC=FY*SMOD*(1+(SMOM/100))
GOTO 30010
ELSE
GOTO 30002
ENDIF

30002 CONTINUE
TOLER=TOLER+5.0
IF ((TOLER/10).GT.500) GOTO 30010
GOTO 30008

3010 DO 100 I=1,N/2
IF (((R-T)**2.0)-(R-((R*2)/N)*I)**2.0).GT.0.) GO TO 1500
XI(I)=((R**2.0)-(R-((R*2)/N)*I)**2.0)**0.5
YI(I)=0.0
GOTO 100

1500 XI(I)=((R**2.0)-(R-((R*2)/N)*I)**2.0)**0.5
YI(I)=(((R-T)**2.0)-(R-((R*2)/N)*I)**2.0)**0.5

100 CONTINUE
DO 200 I=1,N/2
J=(N/2)+I
XI(J)=XI((N/2)-I)
YI(J)=YI((N/2)-I)

200 CONTINUE
DO 300 I=1,N
XC(I)=(YI(I)*2.0)*((R*2.0)/N)
YC(I)=R-(((R*2)/N)*I)
XS(I)=2.0*(XI(I)-YI(I))*((R*2.0)/N)

300 CONTINUE
XNMIN=(USTRN/(XMAXSTR+USTRN))*R*2
DO 400 J=1,250
XBAR=XNMIN+J*1.0
XBB(J)=XBAR
DO 500 I=1,N
STR(I)=(((R*2)/N)*I)*(USTRN/XBB(J))-USTRN
RSTR=STR(I)
IF (RSTR.GE.(0.0)) GOTO 800
IF (RSTR.LE.(-CONB)) GOTO 600
IF ((RSTR.LE.CONA).AND.(RSTR.GT.CONB)) GOTO 700
CONSTS=RSTR*EC
GOTO 900

600 CONSTS=-FC+(RSTR+CONB)*ESOFT
GOTO 900

700 CONSTS=-FC
GOTO 900
800  CONSTS=0.0
900  IF (RSTR.LE.(-STLB)) GOTO 1000
    IF ((RSTR.GT.(-STLB)).AND.(RSTR.LE.(-STLA))) GOTO 1100
    IF ((RSTR.GT.(-STLA)).AND.(RSTR.LE.STLB)) GOTO 1200
    IF ((RSTR.GE.STLA).AND.(RSTR.LE.STLB)) GOTO 1300
STLSTS=FY+(RSTR-STLB)*EHAD
GOTO 1400
1000  STLSTS=FY+(RSTR+STLB)*EHAD
GOTO 1400
1100  STLSTS=FY
GOTO 1400
1200  STLSTS=RSTR*ES
GOTO 1400
1300  STLSTS=FY
1400  FCC(I,J)=XC(I)*CONSTS
    FS(I,J)=XS(I)*STLSTS
    XMC(I,J)=FCC(I,J)*YC(I)
    XMS(I,J)=FS(I,J)*YC(I)
500  CONTINUE
DO 2000 K=1,N
    XFTOT=XFTOT+FCC(K,J)+FS(K,J)
    XXMTOT=XXMTOT+XMC(K,J)+XMS(K,J)
2000  CONTINUE
FTOT(J)=XFTOT
    XMTOT(J)=XXMTOT
    XFTOT=0.0
    XXMTOT=0.0
400  CONTINUE
DO 5500 J=1,250
    IF (FTOT(J).GT.(0.0)) GOTO 5500
    IF ((XXK.GT.(0.0)).OR.(XXK.LT.(0.0))) GOTO 5500
    XXK=(XMTOT(J)-XMTOT(J-1))*(FTOT(J-1)/(FTOT(J)-FTOT(J-1)))
    XMULT(NN,KK)=XMTOT(J-1)-XXK
    XXX(NN,KK)=XXX(J)
GOTO 5555
5500  CONTINUE
5555  XXK=0.0
    WRITE(2,80) XMULT(NN,KK),MU,UMC,2*R/T,XXX(NN,KK),T
    WRITE(*,80)XMULT(NN,KK),MU,UMC,2*R/T,XXX(NN,KK),T
DO 3000 I=1,N
    AAC=AAC+XC(I)
    AAS=AAS+XS(I)
3000  CONTINUE
    NUMBT=(NN-1)*NUM2+KK
WRITE(*,*) AC,AS,AAC,AAS,NUMBT
AAC=0.0
AAS=0.0
    XMULTL=XMULT(NN,KK)
    IF (T.GE.(T+KK*.1)) GOTO 10000
    T=T+0.1
DO 5505 J=1,250
    DPU(J)=ABS(ETOT+(XMTOT(J)/FTOT(J)))
5505  CONTINUE
DO 5506 J=1,249
    IF (J.GT.1) GOTO 5507
    IF (DPU(J).LE.DPU(J+1)) THEN
        XMINO=DPU(J)
APPENDIX H  MIX PROPORTIONS AND GENERAL CHARACTERISTICS OF HIGHER STRENGTH CONCRETE

H.1 Introduction

The use of higher strength concrete has increasingly attracted the attention of designers and researchers for a more efficient use of material, and to avoid large dimensions for structural members. Particularly, in some cases such as high rise buildings and light structures, the use of higher strength concrete is a necessary tool for reducing the sizes and, more importantly, the weight of the structure. But the most important factor is that the brittle behaviour of higher strength concrete restricts its in conventional design procedures, therefore other concepts for existing structural systems must be introduced. According to the numerous research reports concerning the effects of confinement on concrete (references are mentioned in chapter four) the ductility of higher strength concrete can be increased by interacting with other structural materials. For instance,
stirrups in a reinforced concrete column or beam or a hollow section filled with concrete, will furnish an increased structural performance.

The effects of confinement on concrete has a direct dependence to the equivalent stiffness of the surrounding material. Consequently, for each individual case of confinement and specification for concrete, a specific equation is required to define the behaviour of concrete.

In this study, according to the dimensions of the tests and the necessary requirements for concrete, the design procedure and references which are used to achieve the desired concrete performance are explained in detail. For this purpose, according to the mix design method by "ACI Method- American Concrete Institute" a trial mix was designed, and the proportion of material was changed to get the best possible compressive strength and required workability. The percentage of material for the best proportion, consisting of one coarse aggregate (<=10 mm), sand and Portland cement, were constant and, changes were mainly applied to the percentage of water. With regard to the necessity of a low percentage of water, a water reducer or superplasticiser was used. For making a denser and stronger concrete, condensed silica fume, with a percentage recommended by manufacturer has also been used.

The final concrete mix adopted had a compressive strength of about 70-80 MPa with a suitable workability for the purpose of use in composite columns with circular hollow section tubes (CHS).

**H.2 Material Specifications and Basic Proportions**

The materials considered for the higher strength concrete are one kind of coarse aggregate, river sand, Portland cement (type one), water, water reducer (superplasticiser) and silica fume. These materials are conventionally the principal parts of a higher strength concrete, and the mix proportion must be designed in order to achieve all required specifications. For the base of a suitable mix, the following proportions are fixed according to reported experiments in higher strength concrete and the necessary requirements.

\[
\frac{S}{S + CA} \equiv 0.32-0.35, \quad \frac{W}{C} \leq 0.30, \quad SP=1\%C, \quad SF = 10\%C
\]

(H.2.1)

where:

- S=Sand.
- CA=Coarse aggregate.
- W=Water.
- C=Cement.
- SP=Superplastisizer.
SF=Silica fume.

The basic proportions may have some slight differences in the final mixes. The method for changing the proportions to get a better compressive capacity is mainly based on the water to cement ratio, which is the most significant parameter.

### H.2.1 Coarse Aggregate and Sand

The coarse aggregate for this concrete is crushed basalt with a maximum dimension less than 10 mm. This aggregate is commercially produced in the Illawarra region for the purposes of structural concrete. The regional experience and laboratory tests for the quality of this material has proven its suitability for concrete. The other component of the concrete is river sand that is suitable for high quality concrete.

### H.2.2 Superplasticizer

The superplasticiser that usually used for reducing the percentage of water is commercially called RHEOBUILD [2]. The main objective of the incorporation of the superplasticiser is to lower the water-cement ratio to a minimum value. Certain problems related to the pumping of concrete in structural components such as a tube, can be easily overcome. Moreover, use of less water, reduces the shrinkage characteristics of the concrete, and more importantly, because of decrease in void spaces in the concrete mass, the compressive capacity increases. Although different superplasticisers perform the same function in concrete, they are based on different chemicals. Some are synthetic while others are based on natural products.

The general mechanisms in superplasticisers are basically surface active agents and their hydrocarbonate tail is adsorbed on the cement grain and negative charges are produced in water. The electrostatic sheath so formed reduces inter-particle attraction, and therefore, this efficient dispersion reduces water retention and more water is available in the system Ref. [2].

According to the recommended percentage by the manufacturer, the cement-superplasticiser can be in the range of 1% to 1.5%; the greater the percentage the greater the fluidity. On the whole, the required slump is the effective parameter to measure the considered percentage of superplasticiser. The percentage of superplasticiser can also be considered from the available codes of practice, for instance “Specifications for superplasticising admixture”, according to the B.S, 5057, part 3, 1985.
H.2.3 Silica Fume

Condensed silica fume (CFS) is a by-product of the smelting process used to produce silicon metal and ferrosilicon alloys. Other names for CSF that can be found in the literature are; microsilica, ferrosilicon dust, arc-furnace silica, silica flue dust and amorphous silica.

Recently, significant attention has been invested in condensed silica fume. The advantage of including condensed silica fume over conventional Portland cement concrete is mainly in its higher strength and lower permeability. Under particular conditions, it can attain a strength of approximately 96 MPa, about 2 to 3 times the strength of Portland cement concrete. The nature of the hydration products of CSF, and its influence on cement hydration, are not entirely understood at present, but, results of more than 500 publications in several languages make enough reliability for using of CSF in structural concrete Ref[ ].

In the current study, CSF has been added to the concrete for increasing the compressive strength. The percentage of CSF according to the different studies in this field can be between 5% to 10% of cement by weight.

H.3 Concrete Mixes

The mix proportion design, according to the ACI method, is based on proportion by weight. The first trial mix designed as follows (masses are in “kg”):

\[
\begin{align*}
C &= 550 \\
W &= 118 \\
S &= 578 \\
CA &= 1155 \\
SP &= 5.5 \\
SF &= 55
\end{align*}
\]

This is the basis for further investigations to obtain the final mix design.

Regarding the required changes to allow for the moisture of aggregates the first mix of .04 cubic meters had the following weights.

\[
\begin{align*}
C &= 22 \\
W &= 3.2 \\
S &= 22 \\
CA &= 45 \\
SP &= 0.22 \\
SF &= 2.2
\end{align*}
\]

The results of this mix had a very poor workability that made it unsuitable for composite CHS columns. Adjustment with cement and water changed the workability that was considered on the next mixes. The compressive capacity was then very high (72, 65 MPa at 7 days) which was ideal for this study.

Final proportions for the higher strength concrete after some adjustments on the cement and water were as follows for a batch of 0.025 cubic meters.
C=15
\[ w = 4.725 \times 12.77 \left( \frac{1 + A}{1.02} \right) - 26.5 \left( \frac{1 + B}{1.01} \right) \]
\[ S = \frac{100}{1.02} \left( 1 + A \right) 12.77 \]
\[ CA = \frac{100}{1.01} \left( 1 + B \right) 26.50 \]
\[ SF = 1.5 \]
\[ SP = 0.15 \]

where:
A = Moisture of sand (% by weight).
B = Moisture of coarse aggregates (% by weight).

This mix had a suitable workability for this project, which was used for all of the tests. Because of the sensitivity of this kind of concrete to the quality of materials used some changes in compressive capacity were expected.

The concrete for each series of tests was made separately and the specification for each batch is shown in the Table H.3.1.

<table>
<thead>
<tr>
<th>Mix specification</th>
<th>Compressive capacity “MPa” (7 days)</th>
<th>Compressive capacity “MPa” (28 days)</th>
<th>Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td>First mix</td>
<td>65, 72</td>
<td>56, 73 *</td>
<td>-Explosive failure.</td>
</tr>
<tr>
<td>Adjusted mix</td>
<td>27, 27, 23</td>
<td>61, 61, 57</td>
<td>-Explosive failure.</td>
</tr>
<tr>
<td>Final mix</td>
<td>---</td>
<td>63, 71, 70, 72, 73</td>
<td>-Explosive failure.</td>
</tr>
</tbody>
</table>

* : Specimens failed by longitudinally splitting, therefore, results do not show the true compressive strength.

For better comparison of the improvement of the proportions in the concrete mix, Table H.3.2 shows the accurate weight of each component.

<table>
<thead>
<tr>
<th>Mix specifications</th>
<th>Batch size “m³”</th>
<th>S</th>
<th>CA</th>
<th>C</th>
<th>W</th>
<th>SP</th>
<th>SF</th>
</tr>
</thead>
<tbody>
<tr>
<td>First mix</td>
<td>0.025</td>
<td>13.70</td>
<td>28.30</td>
<td>13.80</td>
<td>3.85</td>
<td>0.14</td>
<td>1.4</td>
</tr>
<tr>
<td>Adjusted Mix</td>
<td>0.025</td>
<td>13.40</td>
<td>28.30</td>
<td>13.80</td>
<td>4.20</td>
<td>0.21</td>
<td>1.4</td>
</tr>
<tr>
<td>Final Mix</td>
<td>0.025</td>
<td>12.70</td>
<td>26.50</td>
<td>15.00</td>
<td>4.70</td>
<td>0.15</td>
<td>1.5</td>
</tr>
</tbody>
</table>
H.4 Mechanical Behaviour of Final Mix in Elastic Range

The mechanical properties of primary concrete in this study are mainly Poisson's ratio and the modulus of elasticity. These parameters are measured according to the Australian Standard 1012, part 17-1976 [5]. The method which is described in this Standard is based on the measured compressive strength of concrete. It is required to apply a test load equal to 40% of the average compressive strength of moulded cylinders or cores tested in accordance with AS1012, Part 9 or AS 1012, Part 14, respectively.

The relationship for computing the modulus of elasticity is presented in Eq. H.4.1.

\[
E = \frac{\sigma_1 - \sigma_2}{\varepsilon_2 - 0.00005}
\]

(H.4.1)

where:

- \( E \) = Modulus of elasticity.
- \( \varepsilon_2 \) = Longitudinal strain at the test load.
- \( \sigma_2 \) = Longitudinal stress at the test load.
- \( \sigma_1 \) = Longitudinal stress when the deformation is such that the specimen is subjected to a longitudinal strain of 50 micro strain.
- Test load = 40% of average compressive strength.
- Load rate = 1.25 mm/min.

The relationship for computing the Poisson's ratio has the same concept with the modulus of elasticity and is computed as follows:

\[
\nu = \frac{\varepsilon_4 - \varepsilon_3}{\varepsilon_1 - 0.00005}
\]

where:

- \( \nu \) = Poisson's ratio.
- \( \varepsilon_4 \) = Average transverse strain at midheight of the specimen produced by the stress at test load.
- \( \varepsilon_3 \) = Average transverse strain at midheight of the specimen by stress at an average longitudinal strain of 50 micro-strain.
- \( \varepsilon_1 \) = Corresponding average, longitudinal strain produced by stress at test load.

Based on this method, the modulus of elasticity and Poisson's ratio for the final specimen are shown in Table H.4.1.
Table H.4.1 Test results for Poisson’s ratio and modulus of elasticity for final mix.

<table>
<thead>
<tr>
<th>Test No.</th>
<th>$\varepsilon_1$ Micro strain</th>
<th>$\varepsilon_3$ Micro strain</th>
<th>$\varepsilon_4$ Micro strain</th>
<th>$\sigma_1$ MPa</th>
<th>$\sigma_2$ MPa</th>
<th>$\nu$</th>
<th>$E$ GPa</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>804</td>
<td>22</td>
<td>188</td>
<td>19.06</td>
<td>247</td>
<td>0.22</td>
<td>39.2</td>
</tr>
<tr>
<td>2</td>
<td>805</td>
<td>13</td>
<td>183</td>
<td>10.10</td>
<td>250</td>
<td>0.22</td>
<td>40.4</td>
</tr>
</tbody>
</table>

H.5 Plastic Behaviour Of Higher Strength Concrete

As mentioned in Table H.3.1, higher strength concrete has a very brittle behaviour; in other words the failure mechanism in this kind of concrete is an explosive or sudden failure pattern which is resulted from the stiffness of the testing machine. In a stiff machine for compression test, the failure pattern is not in an explosive or sudden pattern but this kind of machines are not usually used. However, for an unconfined concrete, the plastic behaviour is insignificant and can always be disregarded. Fig. H.5.1, shows the three load-deflection curves for three higher strength concrete tests which were tested at 7 days.

![Load deflection curves for three specimens of adjusted mix at 7 days.](image-url)
APPENDIX I  PROPOSED JOINT IN COMPOSITE COLUMNS.

Fig. I.1 Typical beam-column joint in a composite frame.
The construction procedure can be briefly explained as follows:

1. The joint part consisting of the internal steel tube plus four, three, or even two welded shear plates with holes for connection of girders installs to the lower column. In this installation, a four piece ring (bottom) construction is used to provide a suitable space between main column and the joint part.

2. Connection of girders to the shear plates with bolts (in Fig. 1.1 with three bolts).

3. Installation of upper main column with another four piece ring construction to make suitable space as explained in step 1.

4. Installation of internal reinforcement, concreting and curing.

5. Removal of ring constructions and proceed for the next storey.

APPENDIX J  EUROCODE 3 AND DIN 18800 FOR TUBULAR MEMBERS.

The draft codes Eurocode 3/1/ and DIN 18800 part 1/2 give for tubular members with class 2 cross-section (no plastic design) the limit:

\[ \text{limit } (d/t)_2 = 70.2 / f_y \]

with \( f_y \) in (MPa)

The same two draft codes give for class 1 cross-sections (plastic design) the limit:

\[ \text{limit } (d/t)_1 = 50.2 / f_y \]