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King Saud University

College of Engineering

Civil Engineering Department

CE 499

Design and Behavior of Concrete-Filled Elliptical Steel Stub Columns at Elevated Temperatures

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CE 499 Graduation Project - Approval

We hereby approve the report entitled:

Design and Behavior of Concrete-Filled Elliptical Steel Stub Columns at Elevated Temperatures

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Abstract

Concrete-filled steel tubes are gaining increasing prominence in a variety of engineering structures, with the principal cross-section shapes being square, rectangular and circular hollow sections. A recent addition to this range has been that of elliptical hollow sections. One of the major concerns for such columns has always been the exposure to fire and subsequent cooling for which different methods are adopted. This study investigates the effect of cooling regimes after exposure to elevated temperature on axial behavior of concrete-filled elliptical stub columns on the axial compression response of these columns. The experimental program involves the testing of twenty-four columns consisting of two column cross-sections of solid columns and one size of annular column. The cooling regimes considered in the study were annealing and water quenching. The columns were tested for two possible modes of load transfer viz. core loaded and composite loaded. The columns were designed for given loads and their behavior was studied in terms of ultimate load capacity, load-deformation pattern, mode failure, stresses in outer steel shell and axial stiffness.

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1. Background

The concrete-filled steel tube (CFST) column system has many advantages compared with the ordinary steel or the reinforced concrete system. One of the main advantages is the interaction between the steel tube and concrete: local buckling of the steel tube is delayed by the restraint of the concrete, and the strength of concrete is increased by the confining effect of the steel tube.

One of the preferred choices for designers nowadays is the concrete-filled elliptical steel columns. Though some research on these columns has been initiated but there are many crucial issues that are still intriguing especially when high strength concrete is used and the column is annular. The inside hole may provide better way of dissipation of heat of hydration especially when high strength concrete is used. The present study deals with some of these issues such as the study of the effect of heat of hydration and the influence of cooling regime after exposure to elevated temperature on the axial load carrying capacity of such composite columns. The strength of concrete used in the study is M40 grade.

In summary, the behavior of concrete-filled elliptical steel tube columns was investigated experimentally. The parameters considered in the study were: the width-to-thickness ratio of the steel tubes, tubular versus annular columns filled with normal and high-strength concrete, exposure to elevated temperatures (ambient and 600^{o}C) and the effect of cooling regime (annealing vs. water quenching)

2. Literature Review

Concrete-filled steel tubes are gaining increasing prominence in a variety of engineering structures, with the principal cross-section shapes being square, rectangular and circular hollow sections. Similar to concrete-filled tubular columns, void-filling of elliptical hollow sections (EHS) will produce increased strength, stiffness, energy absorption and fire resistance. A recent addition to this range has been that of elliptical hollow sections. The structural response of empty elliptical tubes has been examined in many studies.

Zhao and Packer [1] carried out experimental tests to study the behavior of concrete-filled elliptical hollow steel (EHS) stub columns. Both normal concrete and self-consolidating concrete (SCC) were used in the testing program. Different loading methods were investigated; e.g., loading through steel alone, loading through concrete alone and loading through the whole cross- section. An equivalent rectangular hollow sections (RHS) was proposed to derive such a limit for EHS sections in axial compression. From the results of this study it was concluded that, for the concrete-filled EHS stub columns, fully loaded across the cross-section and in axial compression, there was little confinement of the enclosed concrete at ultimate capacity of the stub column, since the steel reaches its capacity in compression. For concrete-filled EHS stub columns, where only the concrete was loaded in axial compression, it was shown that very significant concrete confinement is generated, because the steel remains elastic and can sustain considerable hoop tension.

Yang et al. [2] examined the behavior of concrete-filled elliptical hollow sections. They tested 21 specimens, with three nominal tube thicknesses and three concrete grades. The effects of steel tube thickness, concrete strength and constraining factor on elastic stiffness, ductility and ultimate strength were studied. To simulate the effects of concrete shrinkage, the inner surfaces of 6 of the 21 test specimens were coated with grease prior to casting. To investigate confinement effects, a further 6 of the 21 test specimens were loaded through the concrete core only. The results of the tests presented in this study were combined with those from previous studies, and compared with existing design provisions for square, rectangular and circular

concrete-filled tubes. On the basis of these comparisons, design recommendations for concrete-filled elliptical hollow sections were made in this study

Lam and Testo [3] presented a paper on the behavior and design of axially loaded elliptical steel hollow sections filled with normal and high strength concrete. The experimental program comprised of testing EHS specimens with three nominal wall thicknesses and the infill concrete cube strengths varied from 30 to 100 MPa. They studied the effect of steel tube thickness, concrete strength, and confinement, together with column strengths and load-axial shortening curves were evaluated. Comparisons of the tests results together with other available results from the literature were made with current design methods used for the design of composite circular steel sections in Eurocode 4 and AISC codes. It was found that existing design guidance for concrete filled circular hollow sections may be safely applied to concrete filled elliptical steel tubes.

Dai and Lam [4] studied the axial compressive behavior of short concrete-filled elliptical steel columns using the ABAQUS/Standard solver, and developed a new confined concrete stress–stain model for the concrete-filled elliptical steel hollow section. The accuracy of the simulation and the concrete stress-strain model was verified experimentally. The stub columns involved in the experimental testing consisted of 150 x 75 elliptical hollow sections (EHS) with three different wall thicknesses (4 mm, 5mmand 6.3 mm) and concrete grades C30, C60 and C100. The finite element model successfully predicted the basic axial compressive behavior of short concrete-filled composite columns with elliptical steel hollow sections observed in tests, such as the maximum axial compressive resistance and the full load versus end-shortening curve. The deformed shape of the concrete-filled steel elliptical stub column under axial compression was also captured, such as the outwards local buckling of hollow sections and concrete core lateral expansion, etc.

In this paper LIU Xi-chao and ZHA Xiao-xiong [5] studied the distribution rule of interaction between steel tube and core concrete for elliptical concrete filled steel tube (CFST) short columns under axial load which was derived by theoretical methods, and the contour of pressure distribution of core concrete was obtained using finite

element simulation. In this study, the assumption of effectively confined zone distribution of core concrete was proposed according to the two results above, the influence of cross-section shape on the confinement effect is analyzed and an unified calculation formula of practical strength for elliptical CFST short column is obtained, based on the ideology of "Unified Theory" and existing unified formula of loading capacity of circular CFST short column under axial compression. Meanwhile, tests of six elliptical CFST short columns under axial load was also carried out and the test results were in good agreement with the results of equations proposed in this study, effectively verifying the correctness of calculation formula, theoretical analysis and finite element simulation.

In a study, Espinos et al. [6] presented a non-linear three-dimensional finite element model is presented in order to study the behavior of axially loaded concrete filled elliptical hollow section (CFEHS) columns exposed to fire. The numerical model is first validated at room temperature against a series of experiments on CFEHS stub columns available in the literature and subsequently extended to study the performance of slender columns at elevated temperatures. The aim of this work is to understand and represent the behavior of axially loaded CFEHS columns in fire situations and to compare their effectiveness with that of the circular concrete filled tubular (CFT) columns. Parametric studies to explore the influence of variation in global member slenderness, load level, cross- section slenderness and section size are also presented in this study. Finally, guidance on the fire design of CFEHS columns is proposed. As expected, the fire resistance of the columns decreases with an increase in member slenderness and load level, as well as with an increase of the section factor (A/V ratio).

In this study, the behavior of elliptical CFST columns were investigated experimentally. The parameters considered in the study were: the width-to-thickness ratio of the steel tubes, tubular versus annular columns filled with normal and high-strength concrete, exposure temperatures (ambient and 600^{o}C) and the effect of cooling regime (annealing vs. water quenching).

3. Objectives

The objectives of this research are:

- ❖ To study the behavior of elliptical CFST columns, solid and annular.
- ❖ To study the effect of cooling regimes on load carrying capacity of the above composite columns when exposed to elevated temperature
- ❖ To study the effect of the mode of load transfer on elliptical CFST columns (core loaded and composite loaded).

4. Significance of the Project

The concrete filled steel tubular columns are commonly used due to their excellent structural performance, which takes advantage of the combined effect of steel and concrete working together - the steel tube provides confinement to the concrete core, resulting in increased compressive strength, while the concrete core restricts inward deformation of the steel tube thus enhancing local buckling resistance and enabling the use of thinner cross-sections.

The architects and engineers have shown great interest in elliptical tubular columns due their aesthetic appeal and reduced visual intrusion, combined with their structural advantages associated with sections of differing major and minor axis properties.

Studies on concrete filled elliptical steel tubes are in their infancy. The use of high strength concrete is so far restricted to reinforced concrete sections. It is necessary to investigate the behavior of high strength concrete when used in filling tubular members and especially the elliptical steel tubes.

The present study focuses on some of the crucial issues involved in the behavior of such composite columns such as the effects of heat of hydration, and cooling regime when exposed to elevated temperature on the load carrying capacity of such composite columns. Two sizes of outer elliptical tube have been used in the study.

The larger size tube has also been made annular. The strength of concrete proposed to be used is M40. Two common cooling methods – one involving annealing and another involving water quenching were studied.

5. Skill development and learning potential in project

The project is multifaceted having great potential for learning. The students specifically learned the following:

i) Structural design of composite columns for various load combinations
ii) Management of research projects
iii) Concrete mix design for normal as well as high strength concrete
iv) Material testing including instrumentation
v) Structural testing including instrumentation
vi) Fire endurance design of composite columns
vii) Thermal analysis of composite members
viii) Computer programming
ix) Effect of cooling regime on the behavior of composite columns

6. Composite Columns

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

The use of composite columns provides large saving in cost by increasing the useable floor area by a reduction in the required cross-section size. This is very important in

the design of tall buildings in cities where the cost of letting spaces are extremely high. These are particularly significant in the lower storey of tall buildings where stubby columns usually exist. Composite columns can provide an excellent monotonic and seismic resistance in two orthogonal directions.

6.1 Design

GIVEN:

Specified compressive strength of concrete, f_c' = 40 MPa

Specified yield strength of steel, f_y =420 MPa

Modulus of elasticity of steel, Es =200 GPa

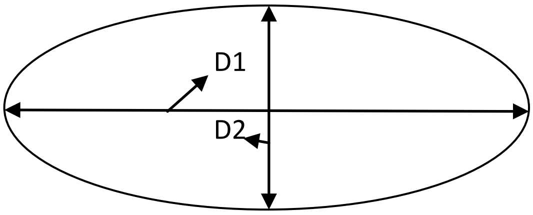
Major outside diameter = 200 mm

Minor outside diameter = 100 mm

Thickness of tube, T = 6 mm

Major inside diameter, D1 = 188 mm

Minor inside diameter, D2 = 88 mm



Calculations	**Reference**
Factored axial load, P_u = Max (1.4 D, 1.2D + 1.4L) = 1400 KN	ACI 9.2.1

Strength reduction factor, ϕ = 0.65

Area of concrete = $\frac{\pi * D1 * D2}{4}$ = 12994 mm^2

Area of steel = $\left[\frac{\pi * (2T + D1) * (2T + D2)}{4}\right] - \left[\frac{\pi * D1 * D2}{4}\right]$ = 2714 mm^2

Percentage of steel = (area of steel / total area) = (2714/12994) *100 =17.3%

Design axial strength,

$$\phi P_{n,\max} = 0.8\phi(0.85 fc' * Ac + fy * Ast) \quad = 1265 \text{ KN} \qquad \text{ACI 10.3.6.2}$$

Table.1 Details of stub columns

Item	**A1**	**A2**	**B2 (Annular)**	
			Outer	**Inner**
Outside dia. of tube (mm)	-	-	-	26.7
Major outside dia., D1 (mm)	150	200	200	-
Minor outside dia., D2 (mm)	75	100	100	-
D1/D2	2	2	2	-
Thickness of tube, T (mm)	4	6	6	2.87
Fc' (MPa)	40	40	40	
Fy (MPa)	420	420	420	
Es (GPa)	200	200	200	
Percentage of steel	**15.4**	**17.3**	**19.1**	
$\phi P_{n,\max}$ **(kN)**	**661**	**1265**	**1322**	

6.2 Fire endurance

Due to the thermal mass of concrete, composite columns always possess a higher fire resistance than corresponding steel columns. (It may be recalled that composite columns were actually developed for their inherent high fire resistance). Composite columns are usually designed in the normal or 'cool' state and then checked under fire conditions.

Additional reinforcement is sometimes required to achieve the target fire resistance. Some general rules on the structural performance of composite columns in fire are

The fire resistance of composite columns with fully concrete encased steel sections may be treated in the same way as reinforced concrete columns. The steel is insulated

by an appropriate concrete cover and light reinforcement is also required in order to maintain the integrity of the concrete cover. In such cases, two-hour fire resistance can usually be achieved with the minimum concrete cover of 40 mm.

For composite columns with partially concrete encased steel sections, the structural performance of the columns is very different in fire, as the flanges of the steel sections are exposed and less concrete acts as a 'heat shield'. In general, a fire resistance of up to one hour can be achieved if the strength of concrete is neglected in normal design. Additional reinforcement is often required to achieve more than one hour fire resistance.

For concrete filled tubular sections subjected to fire, the steel sections are exposed to direct heating while the concrete core behaves as 'heat sink'. In general, sufficient redistribution of stress occurs between the hot steel sections and the relatively cool concrete core, so that a fire resistance of one hour can usually be achieved

For longer periods of fire resistance, additional reinforcement is required, which is not provided in normal design. Steel fiber reinforcement is also effective in improving the fire resistance of a concrete filled column. It is also a practice in India to wrap the column with ferrocement to increase the fire rating.

7. Composite v/s Steel and RC columns

Steel members have the advantages of high tensile strength and ductility, while concrete members have the advantages of high compressive strength and stiffness. Composite members combine steel and concrete, resulting in a member that has the beneficial qualities of both materials. Composite columns may be classified into two principal types:

- Open sections partially or fully encased in concrete, (Fig. 1)
- Concrete-filled hollow steel sections. (Fig. 2)

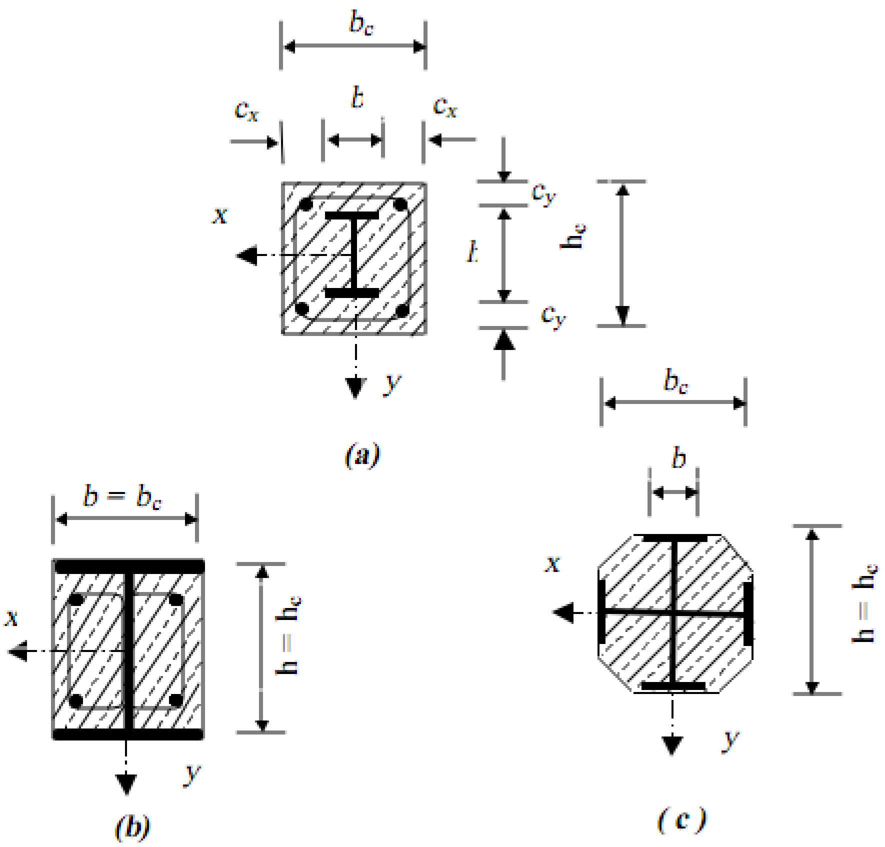


Fig. 1: Typical cross - sections of fully and partially concrete encased columns

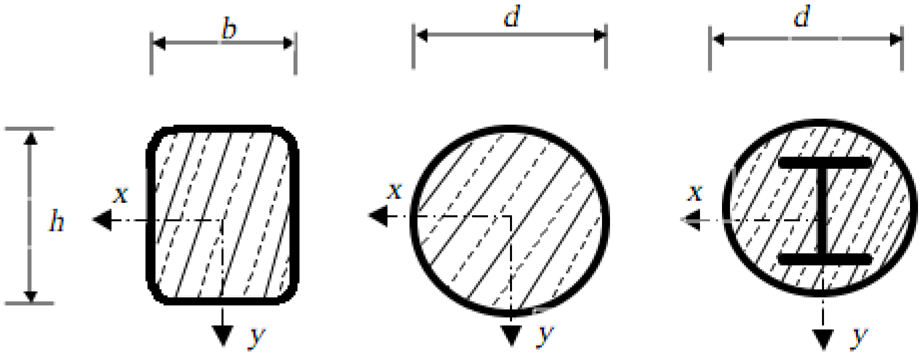


Fig. 2: Typical cross-sections of concrete filled tubular sections

Partially encased columns are based on steel I or H sections, with the void between the flanges filled with concrete. In fully encased columns the whole of the steel section is embedded within a minimum cover depth of concrete. The two main types of composite column are the steel reinforcement concrete column, which consists of a steel section encased in reinforced or unreinforced concrete, and the concrete-filled steel tube (CFT) columns, which consists of a steel tube filled with concrete. CFT columns have many advantages over SRC columns.

Concrete-filled hollow sections may be circular or rectangular. The concrete fills the section, and its compressive strength is enhanced by its confinement. This is an additional advantage for the compression resistance of the column.

Composite columns can provide considerable advantages compared to open steel columns. For example, a cross-section of slender exterior dimensions can resist high axial loads. Different cross-sections of the same exterior dimensions can carry very different loads, depending on the thickness of the steel section, the strength of the concrete and the area of reinforcement used. It is possible to keep the same column dimensions over several stories of a building, which provides both functional and architectural advantages.

In the case of concrete-filled hollow sections, the steel provides a permanent formwork to the concrete core. This allows, for example, the steel frame to be erected and the hollow column sections subsequently to be filled with pumped concrete. This leads to appreciable savings in the time and cost of erection. In addition, the confinement provided by the closed steel section allows higher strengths to be attained by the concrete. In the case of circular concrete-filled tubes the steel, in providing confinement to the concrete, develops a hoop-tension which increases the overall load-carrying capacity, although this is often ignored in design. Creep and shrinkage of concrete, are also generally neglected in the design of concrete-filled tubes, which is not the case for concrete-encased sections. On the other hand,

complete encasement of a steel section usually provides enough fire protection to satisfy the most stringent requirements without resorting to other protection systems. For partially encased sections, and for concrete-filled hollow sections, codes of practice on fire resistance require additional reinforcement.

The advantages and disadvantages of concrete-filled steel tubular columns as compared to the conventional reinforced concrete and structural steel columns are given in the following.

- The load carrying capacity of both materials viz. concrete and steel is enhanced. The strength of concrete gets enhanced due to its confinement by steel tube. Whereas, the load carrying capacity of steel tube gets increased because of the support provided by concrete thus increasing local buckling resistance of steel tube thereby enabling the use of thinner cross-sections of steel tubes.
- Accommodating high percentage of steel (>5%) in reinforced concrete columns is difficult whereas in concrete-filled steel tubular columns, the percentage of steel may be even up to 25% thus reducing the size of column considerably as compared to RC or steel columns
- The formwork is not required and no any fabrication of reinforcing bars thus considerably reducing the construction time
- Drying shrinkage and creep of the concrete are much smaller than in ordinary reinforced concrete

Disadvantages :

- Fire endurance of these columns is more than that of steel columns but less than reinforced concrete columns.
- The splicing of tubes and joints with other members requires special attention of engineers and contractors.

- Though there is considerable saving in the labor and formwork cost but the structural steel being expensive, the use of higher percentage of steel give rise to slight increase in the cost of construction.
- If same size tubes are used on a project, the transportation cost would be more. Whereas, the use of different diameters reduces the transportation cost.

8. Work Plan

The experimental plan involves the testing of concrete-filled elliptical steel stub columns. Two different sizes of elliptical columns with the ratio of major to minor diameter of 2 were used. The height of specimens was kept three times the outer major diameter. For studying the effect of mode of load transfer, the specimens were tested under axial compression for two types of loadings – load applied on concrete core and load applied on composite section.

8.1 Test Matrix

The details of the experimental program are given in Table 2. The sizes of different columns are given in Table 1 earlier.

Table 2: Test matrix

Column ID	Concrete grade (MPa)	Loading	Number of specimens			
			Ambient	Heated		Total
				Annealed	Quenched	
A1	0	-	1	1	1	3
	40	Core	1	1	1	3
		Composite	1	1	1	3
A2	0	-	1	1	1	3
	40	Core	1	1	1	3
		Composite	1	1	1	3
B2	40	Core	1	1	1	3
		Composite	1	1	1	3
					G. Total =	24

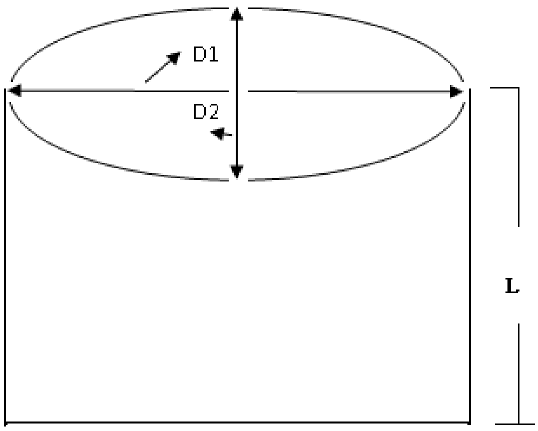


Fig. 3 Elliptical shape dimensions as defined in Table 1

8.2 *Project Requirements*

i. Materials required

- Steel pipes
- Frame for transferring specimens from furnace to cooling tank
- Concrete

Fig.4 Elliptical Steel Sections

Instruments used

- Compression testing machine
- Furnace
- Cooling tank
- CNC machine for the fabrication of specimens
- Compressometer, LVDTs, strain gauges and data logger

Fig.5 Compression testing machine

ii. *Software employed*

- Excel

8.3 Research methodology

- Experimental Program
  - Material Testing
  - Structural Testing
- Numerical Analysis

The experimental program is explained in detail in the next section (Section-9).

8.4 The project management

All team members were involved in all tasks of the project. However, the 24 specimens shown in Table 2 were followed by team members as follows:

Specimens of type A1 was the responsibility of the student: ***Waleed***

Specimens of type A2 was the responsibility of the student: ***Feras***

Specimens of type B2 (annular) was the responsibility of the student: ***Mohammed***

Also **Mohammed** was the team leader.

9. Experimental Program

9.1. Concrete Mix Design

Though the ready mixed concrete M40 has been used in the project, however, the concrete mix design was taken up for developing an understanding about the procedure involved for deciding the mix ingredients and their proportions for both normal and high strength concrete.

Normal Strength Concrete

Properties of NORMAL strength concrete:

- Its slump varies from 25.4-101.6 mm
- Density ranges from 22 Kg/m^3 to 27.5 Kg/m^3.
- It is strong in compression and weak in tension.
- Air content 1 - 2%.
- Normal concrete is not durable against severe conditions e.g. freezing and thawing.

Mix Design of NORMAL Strength Concrete:

Table 4: Mix design of Normal strength concrete

Materials (Absorption %)	Weight kg/m^3 (Trail Mix-1)	Weight kg/m^3 (Trail Mix-2)	Weight kg/m^3 (Trail Mix-3)
Cement	400.00	360.00	360.00
Fine Aggregate	780.00	1152.00	1152.00
Coarse Aggregate	1050.00	612.00	612.00
Water	240.00	216.00	180.00
Water-cement Ratio	0.60	0.60	0.50
Super Plasticizer	0.00	0.00	0.00

Fig.6 Trial Mix for Normal Concrete

Fig.7 Concrete Cylinders

After working the mixture and then cast into inside the cylinders, and after **28 days** were tested for stress and strain (Figs. 8 and 9).

Fig.8 Test in Progress

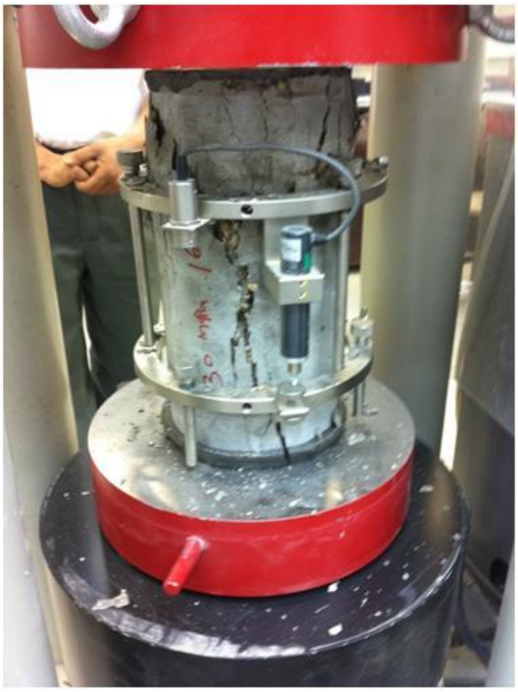
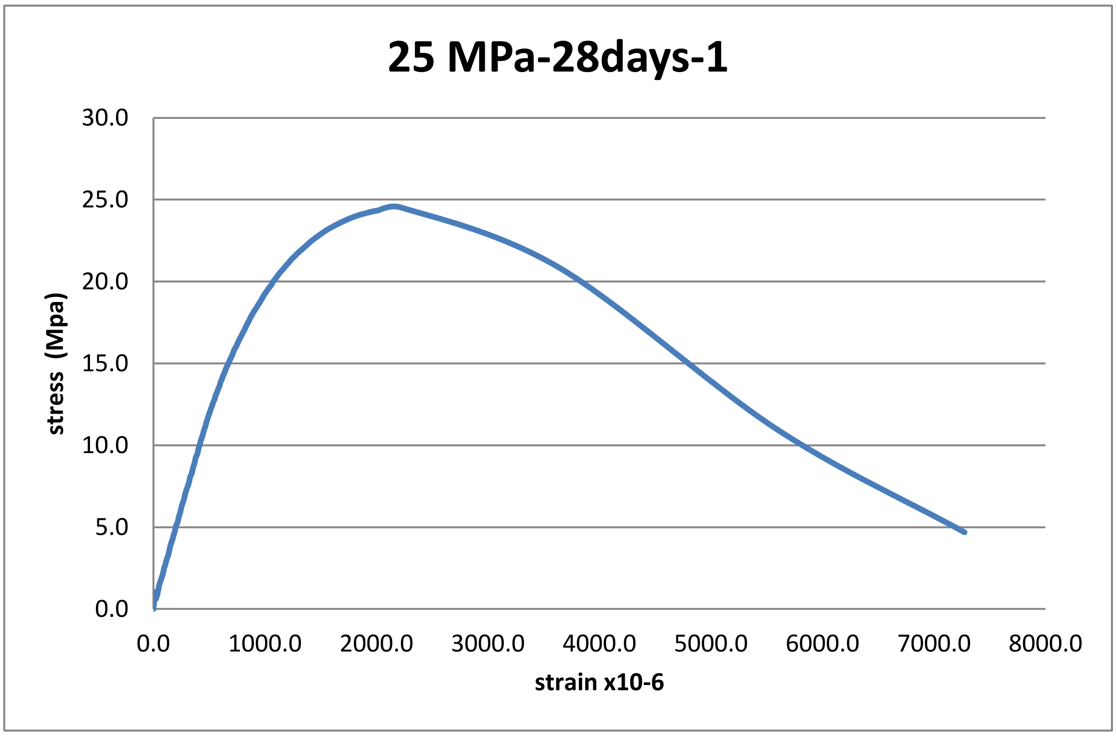
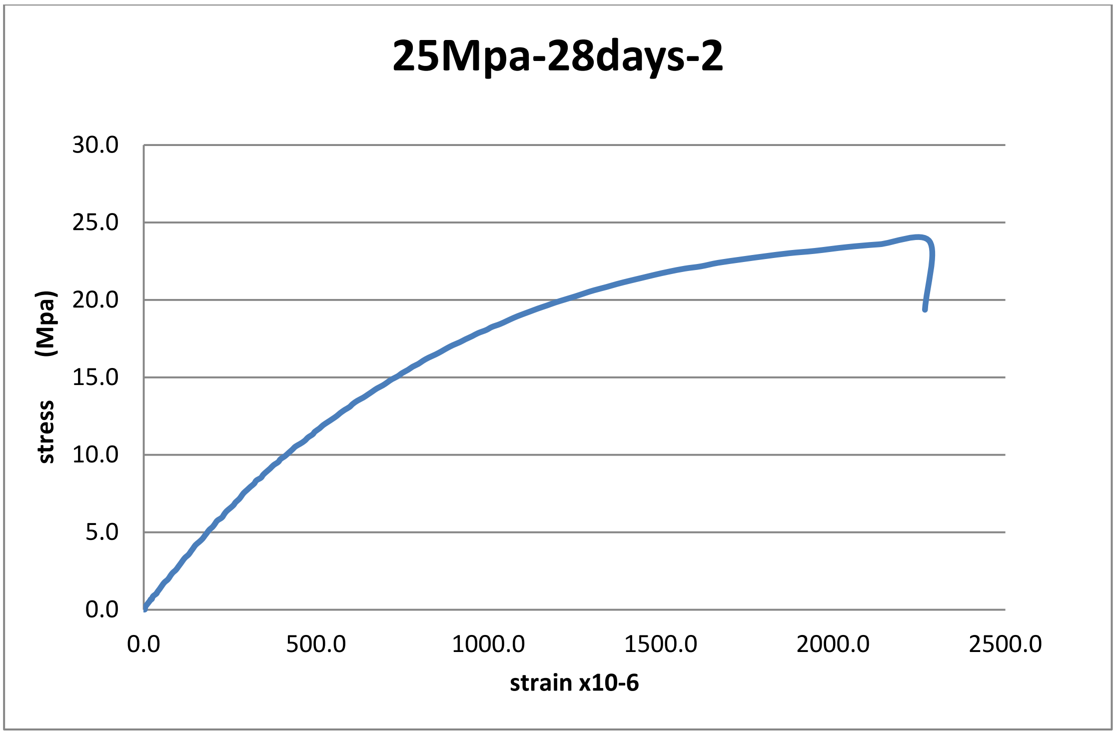


Fig. 9 Specimen Failure

And the test results for the two samples are shown in Figs. 10 and 11.




25Mpa-28days-2
30.0
25.0
20.0
15.0
10.0
5.0
0.0
stress (Mpa)
0.0
500.0
1000.0
1500.0
2000.0
2500.0
strain x10-6

High strength concrete

Properties of HIGH Strength Concrete:

- Compressive strength of high strength concrete mix is usually greater than 50 MPa.
- High strength concrete is made by lowering the water cement (W/C) ratio to 0.35 or lower.
- Often silica fume is added to prevent the formation of free calcium hydroxide crystals in the cement, which might reduce the strength at the cement aggregate bond.
- Low w/c ratios and the use of silica fume make concrete mixes significantly less workable, which is particularly likely to be a problem in high-strength concrete applications where dense rebar cages are likely to be used. To compensate for the reduced workability in the high strength concrete mix, superplasticizers are commonly added to high-strength mixtures.
- Aggregate must be selected carefully for high strength mixes, as weaker aggregates may not be strong enough to resist the loads imposed on the concrete and cause failure to start in the aggregate.

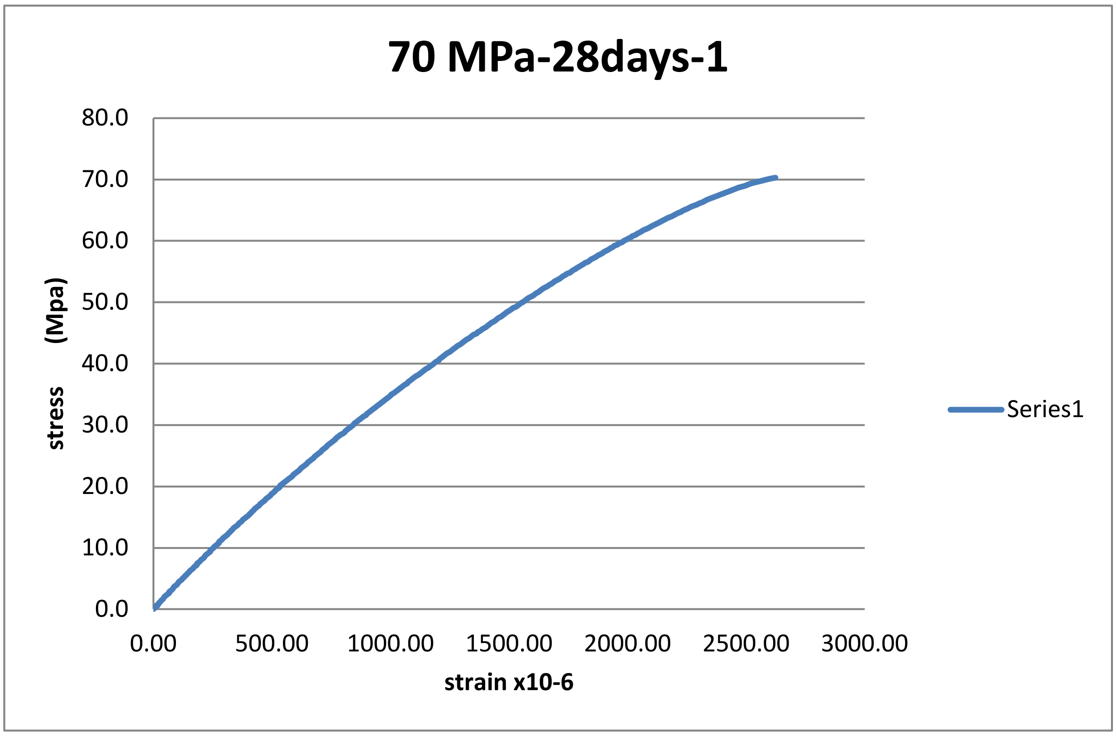
Table.5 (Mix design for **High** Strength Concrete)

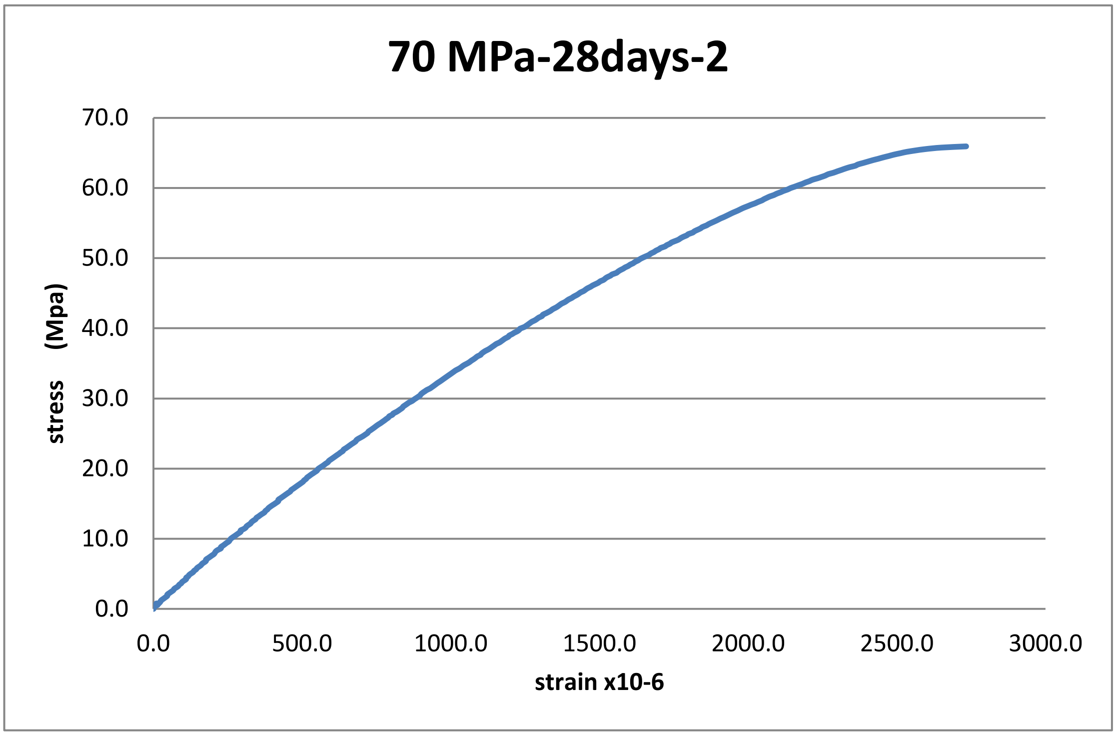
Materials (Absorption %)	Weight kg/m^3 (Trail Mix-1)	Weight kg/m^3 (Trail Mix-2)	Weight kg/m^3 (Trail Mix-3)
Cement	550.00	550.00	490.00
Fine Aggregate	715.00	715.00	785.00
Coarse Aggregate	990.00	990.00	931.00
Water	154.00	154.00	137.20
Water-cement Ratio	0.28	0.28	0.28
Super Plasticizer	15.10	13.00	13.00

Fig.10 (mix preparation of **High** Strength Concrete)

Fig. 11 Concrete Cylinders

After working the mixture and then cast into inside the cylinders, and after **28 days** were tested, and the results were different for the two samples as follows :





The primary **difference between high-strength concrete and normal-strength concrete** relates to the compressive strength that refers to the maximum resistance of a concrete sample to applied pressure. Although there is no precise point of separation between high-strength concrete and normal-strength concrete, the American Concrete Institute defines high-strength concrete as concrete with a compressive strength greater than 6000 psi (41 MPa).

Table 6 (Results of cylinders Tests within days 14, 28 days)

Test results for Concrete mix design Cylinders (Elliptical Column Project)			
Trial Mixes	**Target Strength (Mpa)**	**Testing days**	**Average Specified Cylinder Compressive Strength (MPa)**
1	25	14	18.8
	25	28	24.1
2	25	14	22.4
	25	28	24
3	25	14	T.B.C
	25	28	T.B.C
1	70	14	82
	70	28	86.2
2	70	14	67.5
	70	28	72
3	70	14	62.6
	70	28	70.1
4	70	14	T.B.C
	70	28	T.B.C

9.2 Material Properties

9.2.1 Concrete

i) Concrete Constituents:

- Ordinary Portland Cement = 400 kg/m^3
- Fine aggregate (Fineness modulus, FM = 2.770) = 753 kg/m^3

- Coarse aggregate (Maximum Size of Aggregate = ¾ inch) = 1049 kg/m^3
- Water to cement ratio, W/C = 0.41
- Admixture: FEB Master builder CRP4 = 0.6% by wt. of cement

ii) Fresh concrete

Fresh concrete is that stage of concrete in which concrete can be moulded and it is in plastic state. This is also called "Green Concrete". Another term used to describe the state of fresh concrete is **consistence**, which is the ease with which concrete will flow.

iii) Hardened concrete

The compressive strength of concrete was determined by testing cylindrical concrete specimens of 150 × 300 mm size. The stress-strain diagram obtained is shown in Fig. 12.

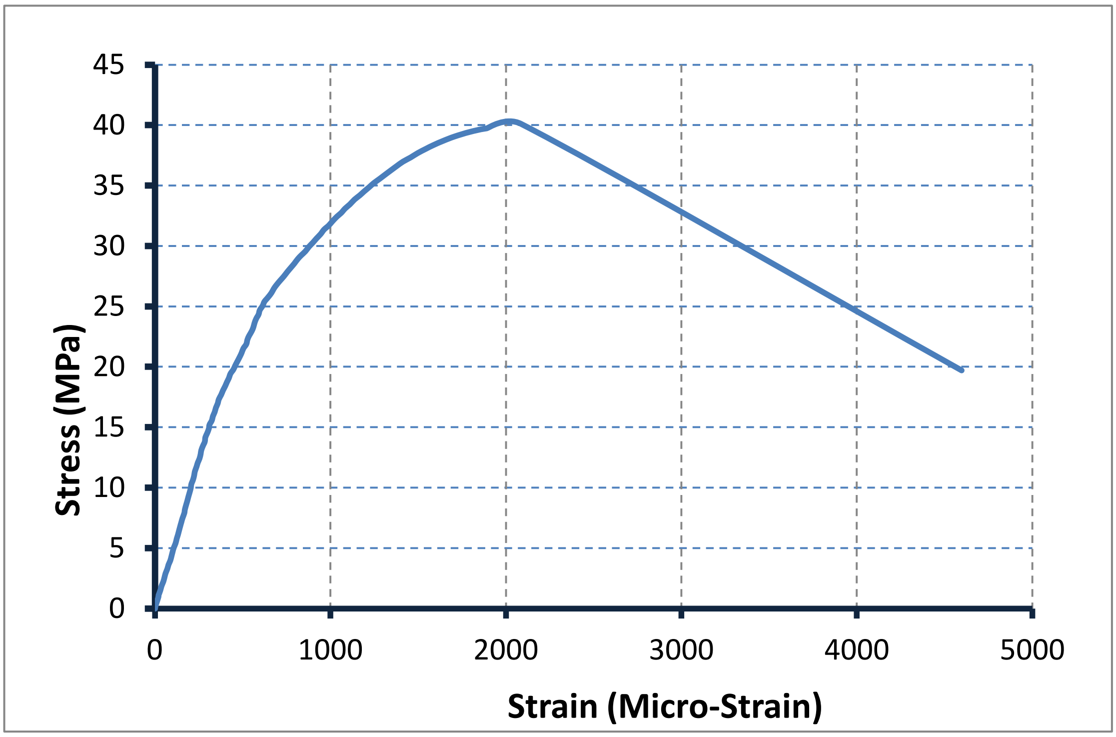


Fig 12: Stress-strain diagram of concrete used in the study

9.2.2 Structural steel

In this experiment pipes were used as a substitute for the reinforcement.

The samples of elliptical pipes were tested in axial compression. The required and experimental strength of elliptical pipe of size A1 was:

Theoretical Results : Pu = Fy .A = 420 * (π * $\left(\left(\frac{150}{2}\right) * \left(\frac{75}{2}\right)\right) - \left(\left(\frac{142}{2}\right) * \left(\frac{67}{2}\right)\right) = 573\ KN$

Laboratorial Results : Pu = 904 KN

9.3 Casting of Steel encased Concrete column specimens

Pictures showing the preparation of specimens, steel tubes, annular columns and concrete casting etc. are shown in Appendix A

9.4 Heating and Cooling of Specimens

For the purpose of heating, the specimens were exposed to 600 ^{o}C temperature for 3 hours duration. For the purpose of simulating the exposure of columns to elevated temperature during fire, the column ends were covered with 100 mm concrete slab for avoiding direct exposure to heat from inside (Fig. 13). The two types of cooling regimes considered were: annealing and water quenching. The annealing was done by leaving the specimens in oven for 24 hours after switching it off, whereas, the quenching was performed by immersing the specimens in water after taking the specimens out of the oven soon after heating. For water quenching, heated specimens were taken out from the oven using a specially fabricated trolley, as shown in Fig. 14. The process of water quenching is depicted in Fig. 15.

Fig. 13: Specimens inside the oven with concrete cover slabs

Fig. 14: Specimens being taken out of the oven on an especially prepared trolley

(a) Specimens lifted from trolley using crane

(b) Specimens being immersed in water

(c) Specimens after immersion in water

Fig. 15: Water quenching of specimens

9.5 Test Setup for Circular Steel Encased Concrete Columns

The test setup for testing the steel encased concrete columns consists of the instrumentation program and applying axial compressive load on the column in a compression testing machine.

9.5.1 Instrumentation Plan

The circular column specimens were instrumented using three LVDT’s to measure the displacement in the axial direction and three strain gauges attached in the lateral direction to measure the circumferential strains. The LVDT’s was attached to the steel, the setup as shown in Fig. 16 and the gauge length to measure the displacement was 150 mm and 200 mm respectively for the 450 mm and 600 mm column specimens. The shortening of whole height of specimens was also measured. For the measurement of lateral strains in outer steel shell, the strain gauges were attached to the steel surface at mid-height of the specimen, using an adhesive Cyanoacrylate (CN), once the surface is ground for proper adhesion.

Fig. 16 Instrumentation Plan

9.4.2 Testing Machine

The column specimens were tested under uniaxial compression using the AMSLER compression testing machine available in the structural laboratory of the King Saud University. The machine is controlled by a hydraulic setup and has two display channels which can be set to load and displacement respectively. The test was displacement controlled with a rate of loading of 0.25 mm/sec. Please see Fig. 17 below which shows the test-setup and the display for the control unit of the AMSLER testing machine.

Fig. 17

10. Results and Discussion

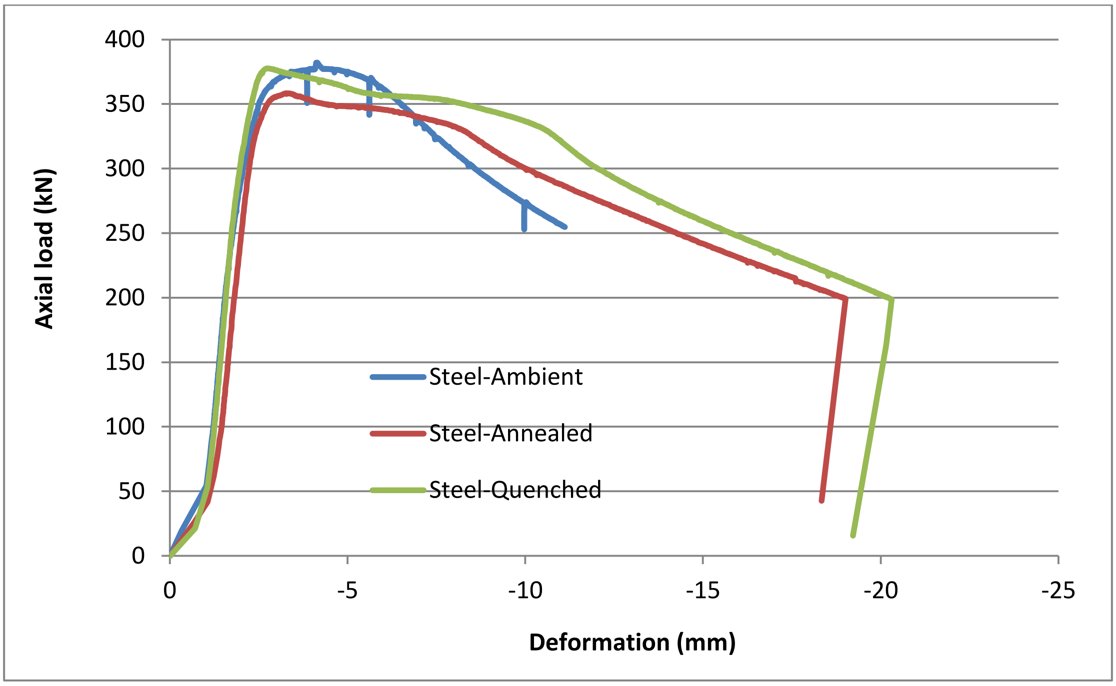
The sign convention followed in the discussion is:

The shortening and compressive strains are taken negative whereas the compressive load and compressive stress is taken as positive.

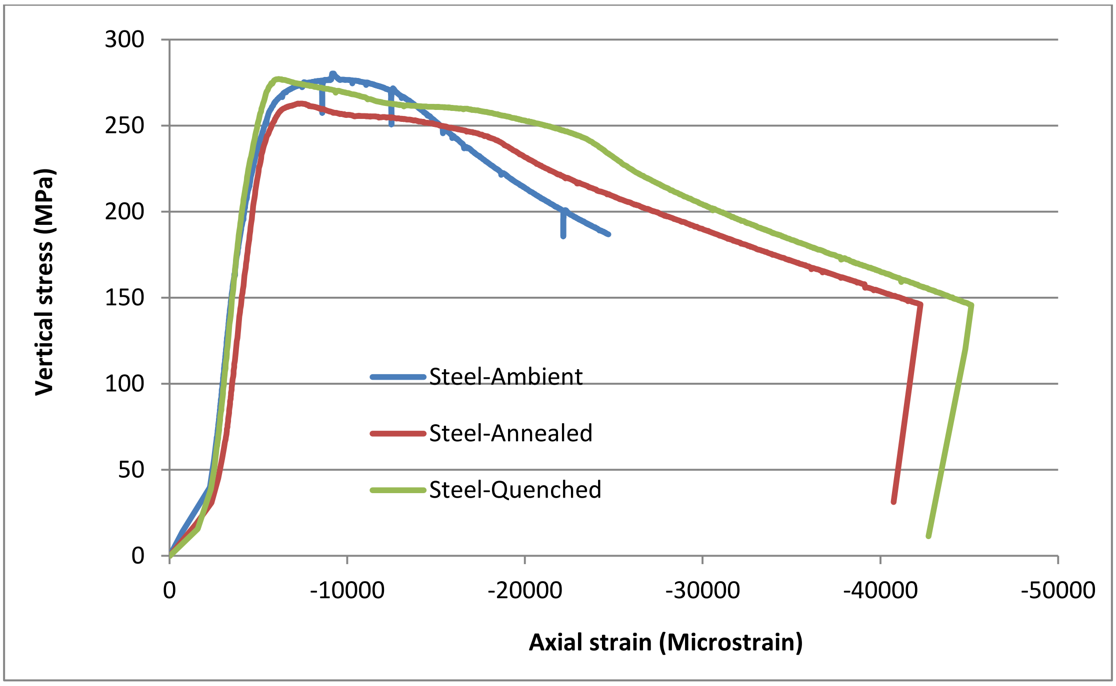
10.1 Effect of heating and cooling regimes on steel cols

Figures 10.1 and 10.2 show the load-deformation and stress-strain curves for the steel columns A1 and A2 respectively tested for three exposure conditions viz. ambient, annealed and quenched after exposure of heat. The ultimate load and the corresponding stress are given in Table 10.1. The observations from the table and figures are given in the following:

- The peak stress in steel varies from 62% to 79% of yield strength of steel and thus the yield stress could not be reached in steel columns.
- The post-heating load carrying capacity of steel got reduced by up to 13.9%. A comparison of annealed and quenched specimens shows that the annealed specimens exhibit more loss in the load carrying capacity.

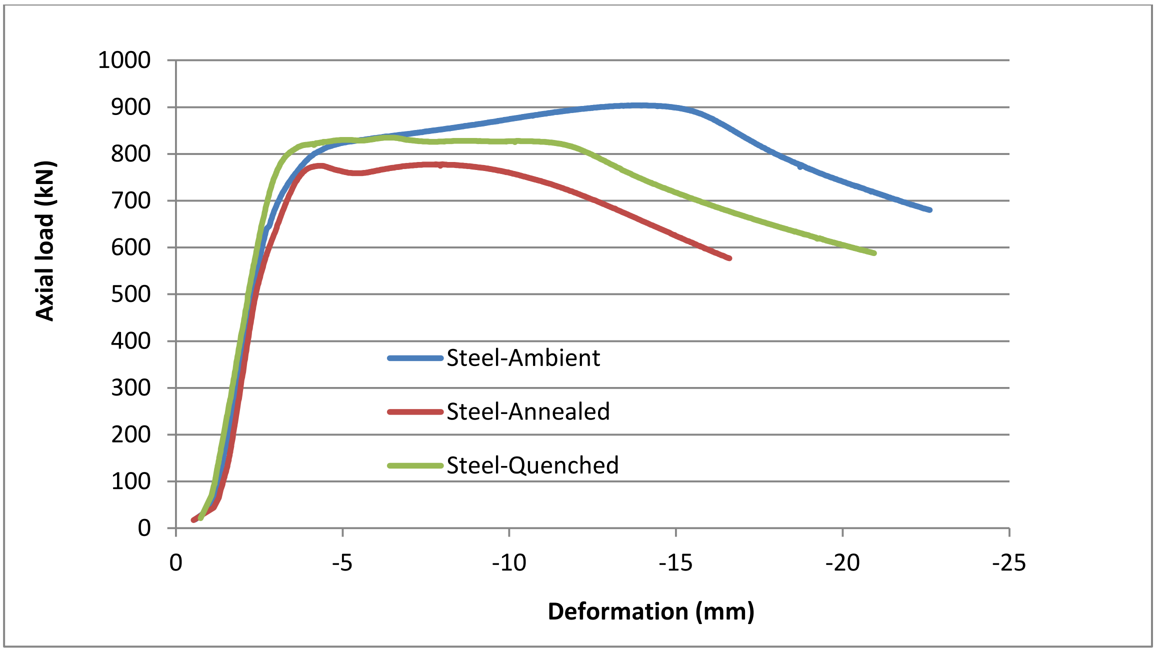


a) Load vs deformation

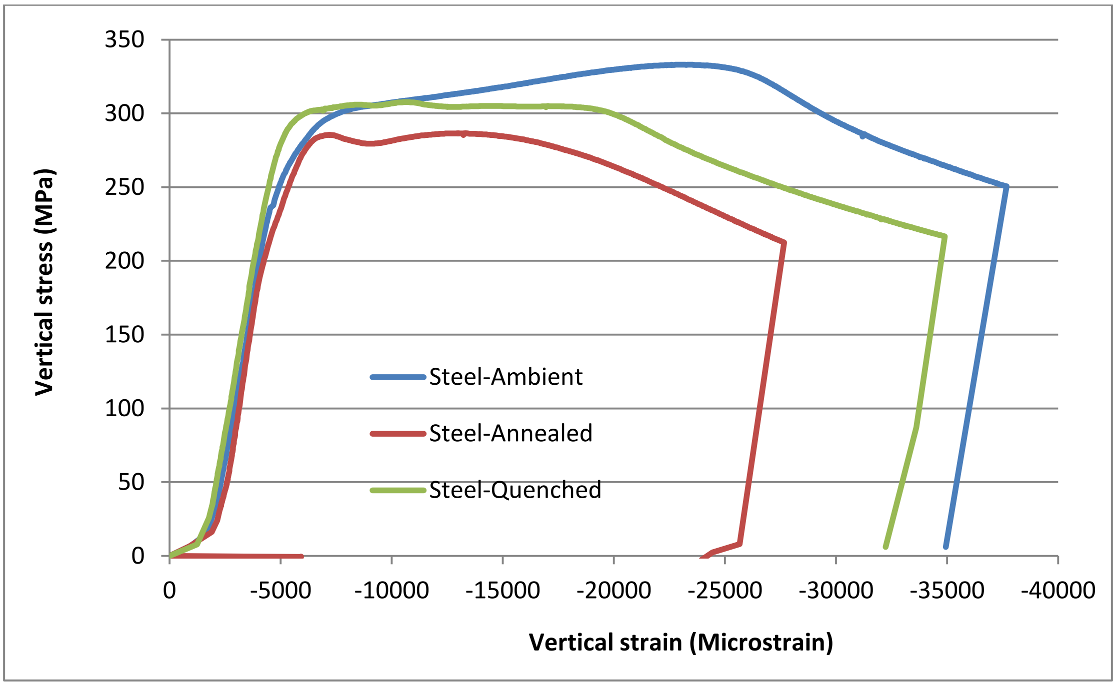


(b) Stress vs strain

Fig. 10.1: Effect of heating and cooling regimes on steel columns A1



a) Load vs deformation



(b) Stress vs strain

Fig. 10.2: Effect of heating and cooling regimes on steel columns A1

Table 10.1: Effect of heating and cooling regimes on steel columns

Column ID	Area of cross-section (mm^2)	Ambient		Annealed		Quenched	
		Load (kN)	Stress[2] (MPa)	Load[1] (kN)	Stress[2] (MPa)	Load[1] (kN)	Stress[2] (MPa)
A1	1363	382.1	280.2 (0.67)	358 (-6.2%)	262.8 (0.62)	378 (-1.15%)	277.0 (0.66)
A2	2714	903.8	333.0 (0.79)	778 (-13.9)	286.6 (0.68)	835 (-7.6%)	307.7 (0.73)

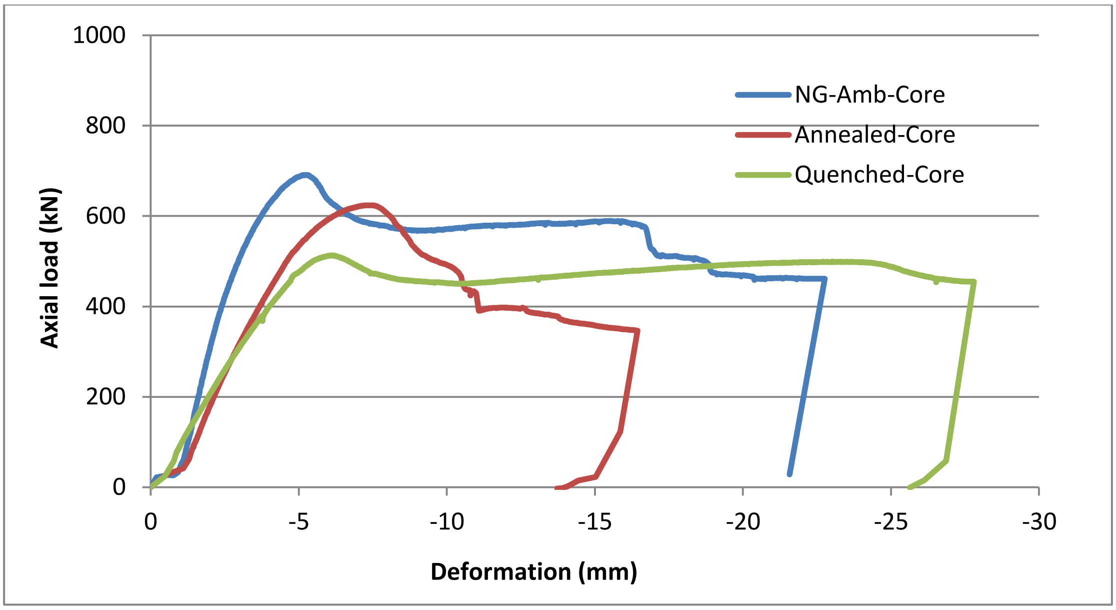
[1] Value within brackets is the percentage change with respect to ambient

[2] Value within brackets is the ratio of stress to yield stress

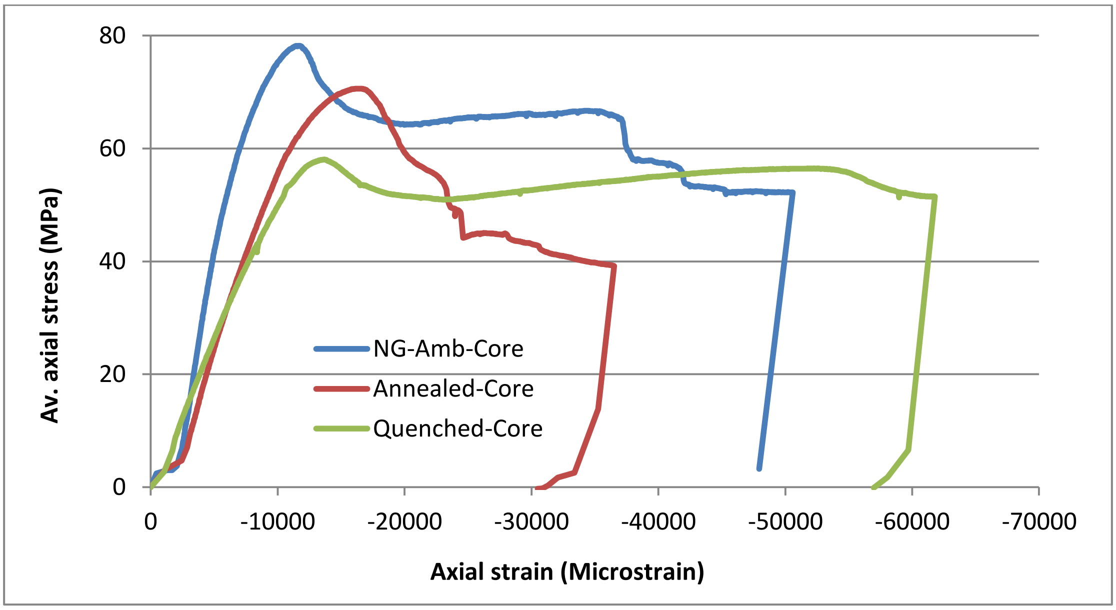
10.2 Effect of heating and cooling regimes on concrete filled columns

Figures 10.3 to 10.8 show the load-deformation and stress-strain curves for the concrete-filled steel columns A1, A2 and B2 tested under three exposure conditions viz. ambient, annealed and quenched after exposure of heat. The ultimate load and the corresponding stress are given in Table 10.2. The observations from the table and figures are given in the following:

- At ambient temperature, the ratio of peak average stress to the uniaxial compressive strength of solid CFST columns varies from 2.0 to 2.1, whereas for annular CFST columns, it varies from 2.1 to 2.3.
- The loss of load carrying capacity of quenched CFST columns is more than the annealed columns.

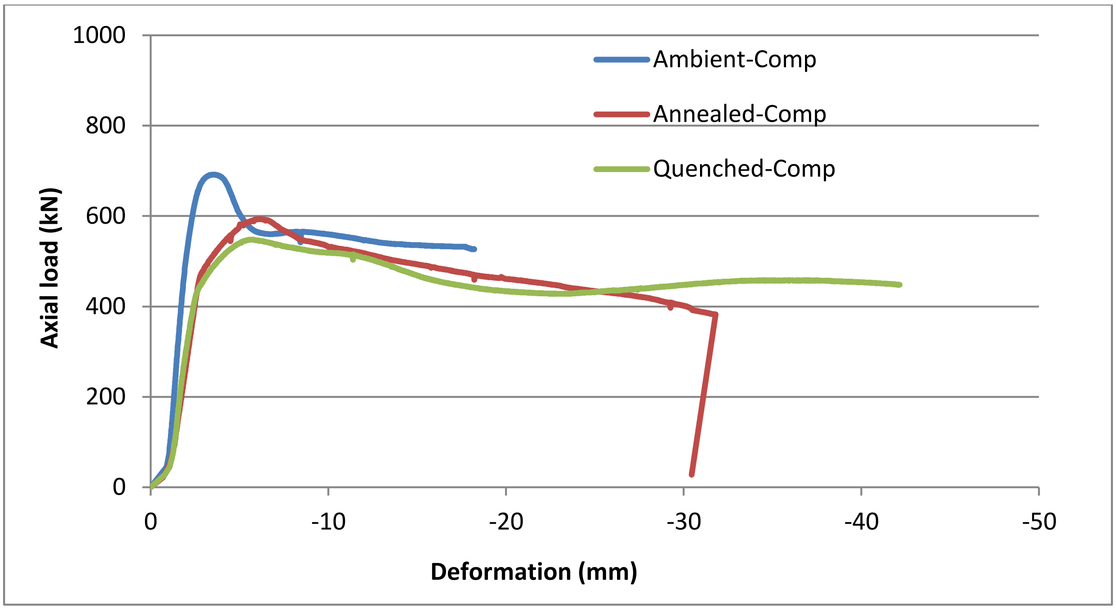


(a) Load vs deformation

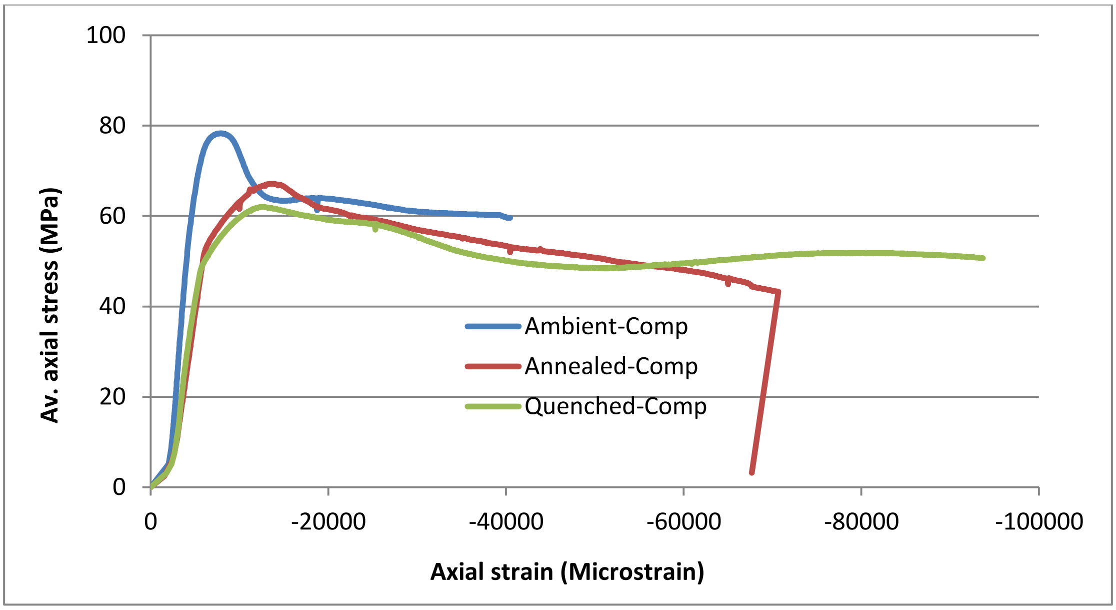


(b) Stress vs strain

Fig. 10.3: Effect of heating and cooling regimes on core loaded concrete filled columns type A1

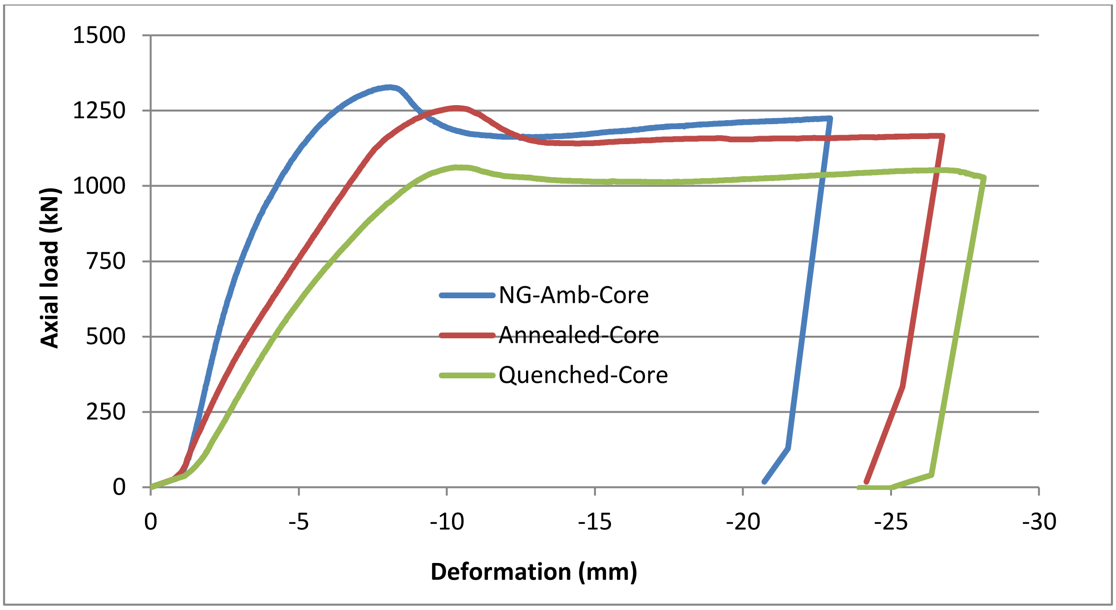


(a) Load vs deformation

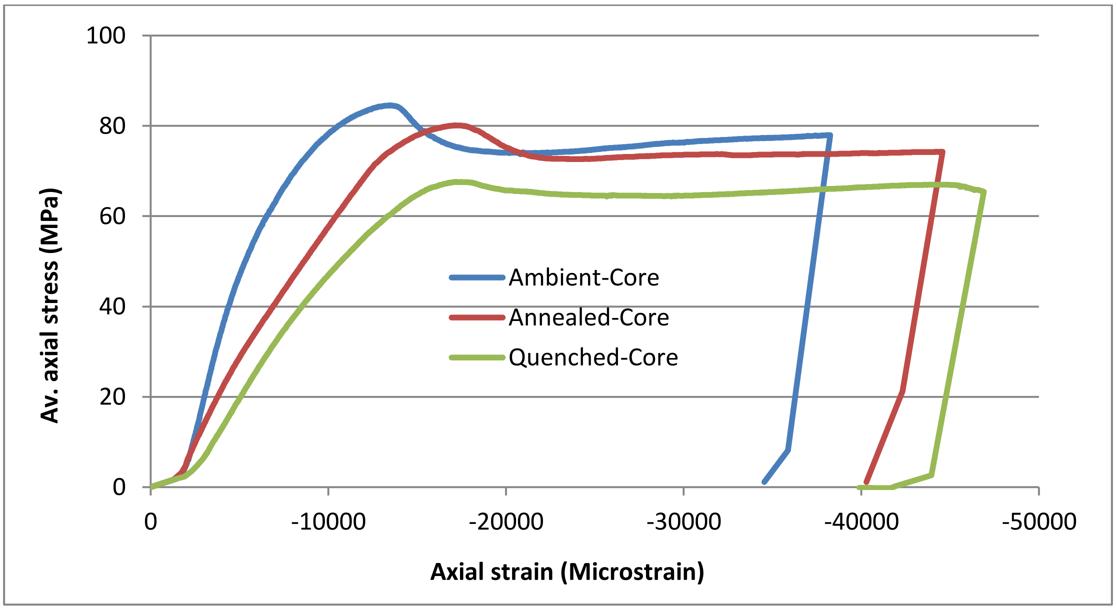


(b) Stress vs strain

Fig. 10.4: Effect of heating and cooling regimes on composite loaded concrete filled columns type A1

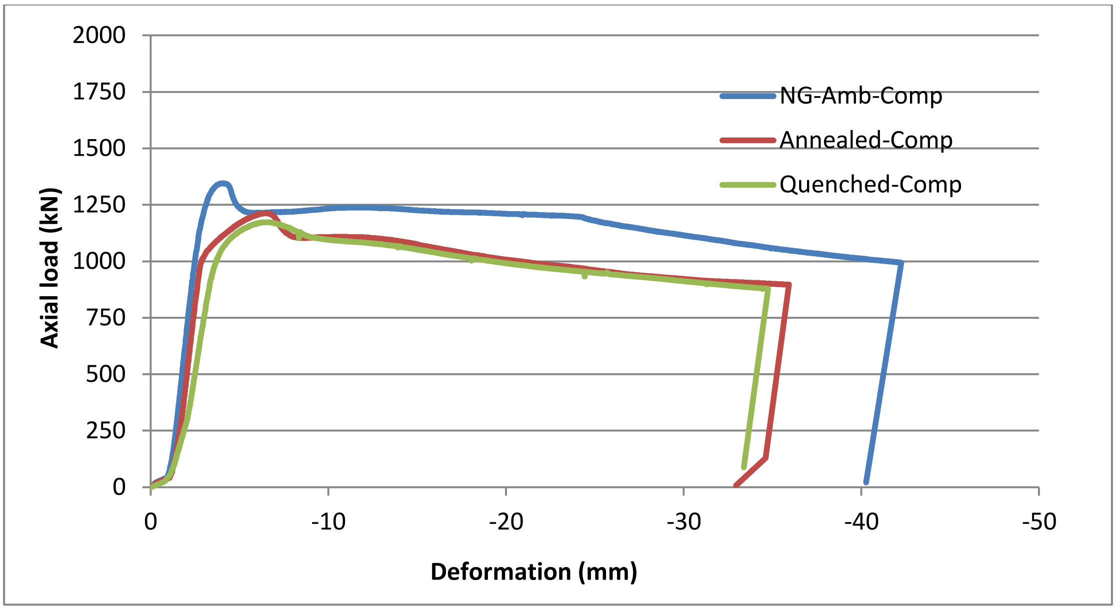


(a) Load vs deformation

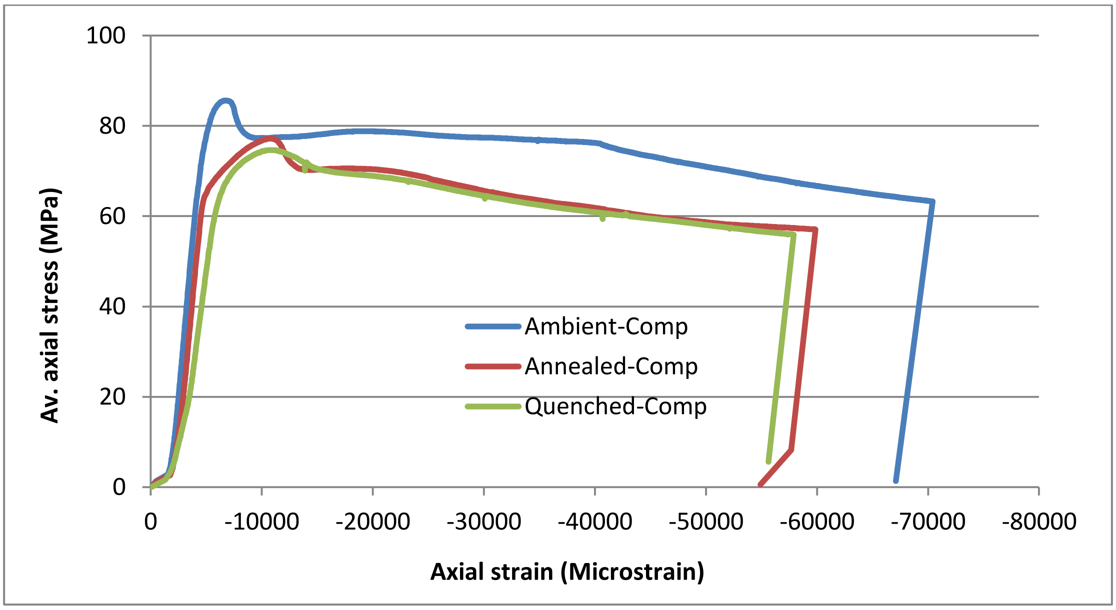


(b) Stress vs strain

Fig. 10.5: Effect of heating and cooling regimes on core loaded concrete filled columns type A2

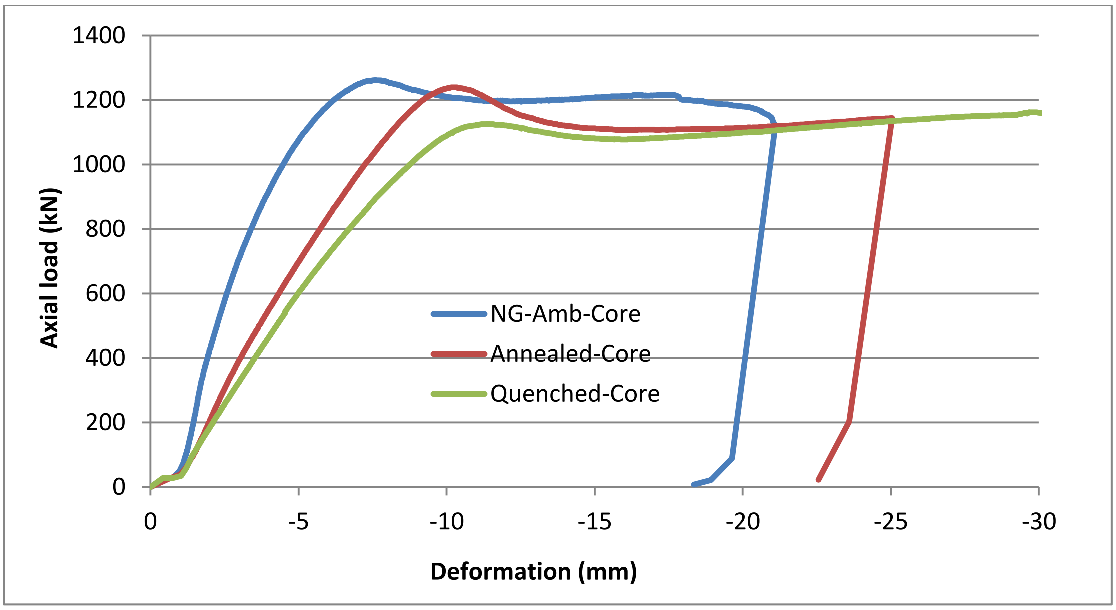


(a) Load vs deformation

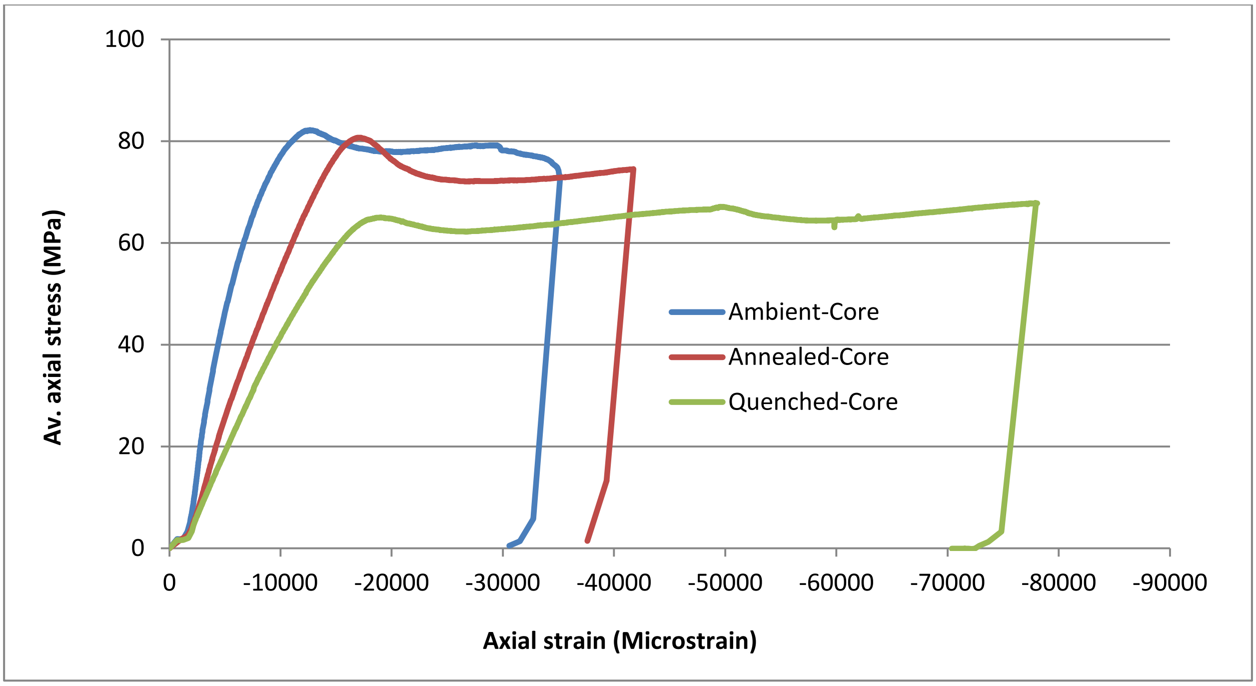


(b) Stress vs strain

Fig. 10.6: Effect of heating and cooling regimes on composite loaded concrete filled columns type A2

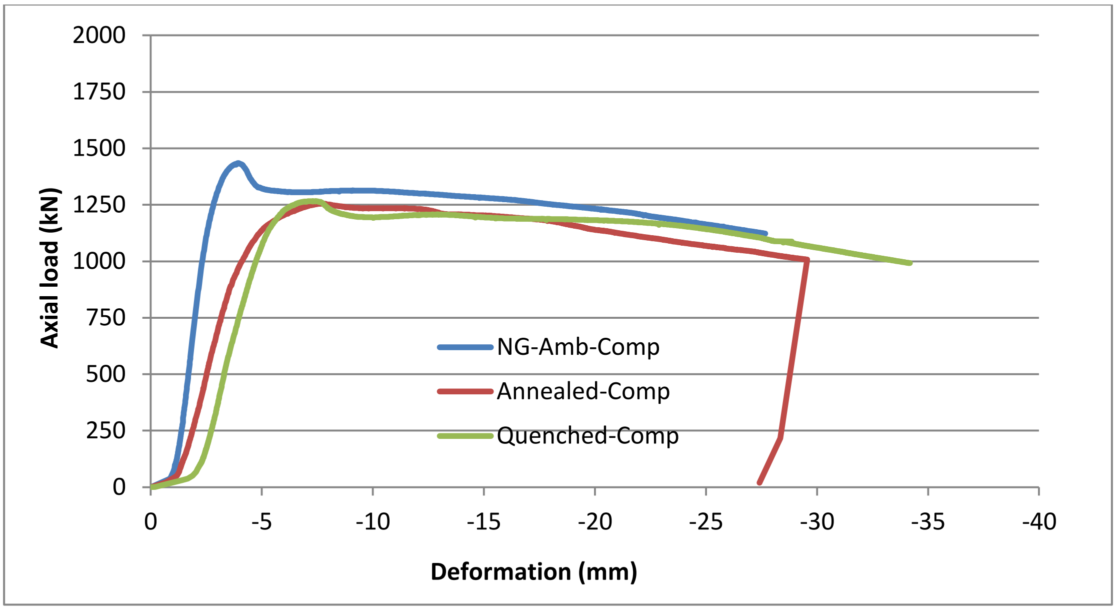


(a) Load vs deformation

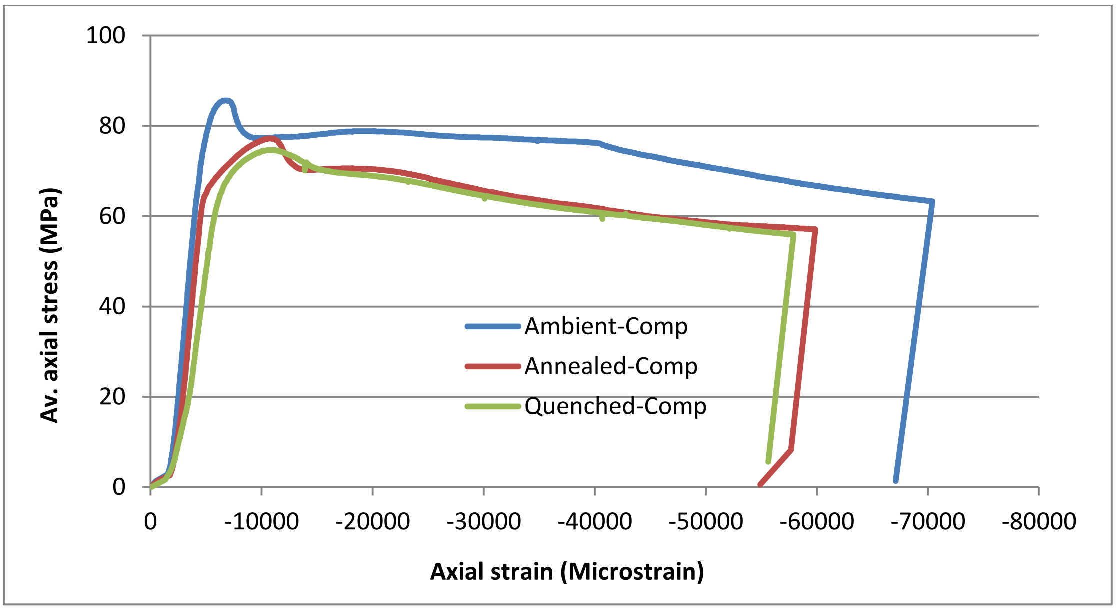


(b) Stress vs strain

Fig. 10.7: Effect of heating and cooling regimes on core loaded concrete filled columns type B2



(a) Load vs deformation



(b) Stress vs strain

Fig. 10.8: Effect of heating and cooling regimes on composite loaded concrete filled columns type B2

Table 10.2: Effect of heating and cooling regime on concrete-filled steel columns

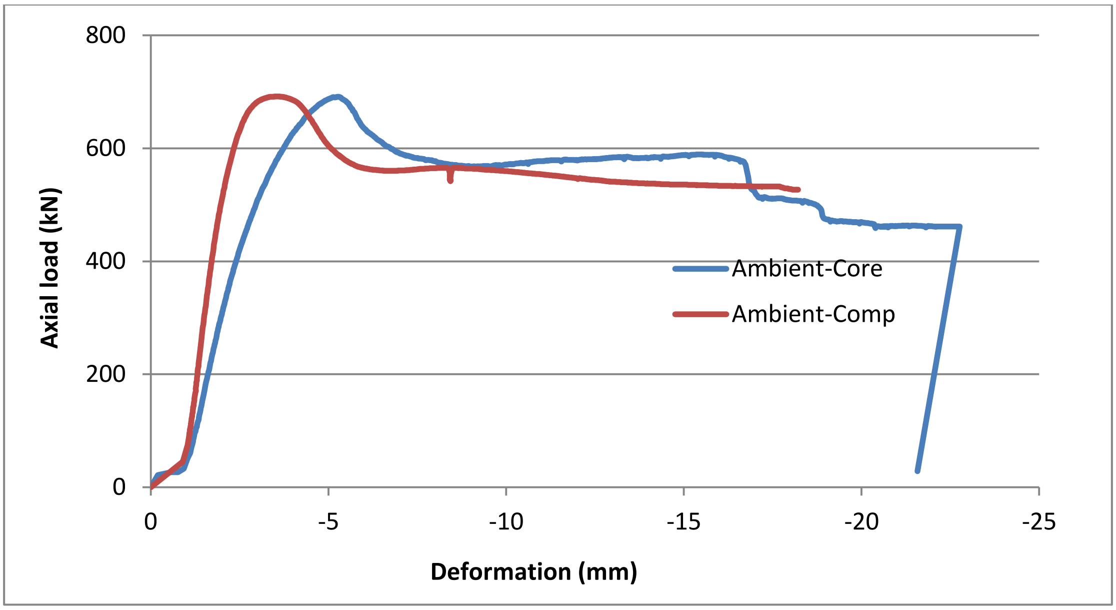
Column ID	Area of cross-section (mm^2)	Load carried by column (kN)			Peak average stress (MPa)		
		Ambient	Annealed[1]	Quenched[1]	Ambient[2]	Annealed[2]	Quenched[2]
Core Loaded							
A1	8836	691.2	624 (-9.75%)	513 (-25.8%)	78.2 (1.96)	70.6 (1.76)	58.0 (1.45)
A2	15708	1328.2	1259 (-5.22%)	1062 (-20.05%)	84.6 (2.11)	80.1 (2.00)	67.6 (1.69)
B2	15363	1262.1	1240 (-1.77%)	1175 (-6.87%)	82.2 (2.05)	80.7 (2.02)	76.5 (1.91)
Composite loaded							
A1	8836	692	593 (-14.27%)	548 (-20.8%)	78.3 (1.96)	67.1 (1.68)	62.0 (1.55)
A2	15708	1344.9	1213 (-9.83%)	1173 (-12.8%)	85.6 (2.14)	77.2 (1.93)	74.7 (1.87)
B2	15363	1434.4	1255 (-12.51%)	1266 (-11.7%)	93.4 (2.33)	81.7 (2.04)	82.4 (2.06)

[1] Value within brackets is the percentage change with respect to ambient

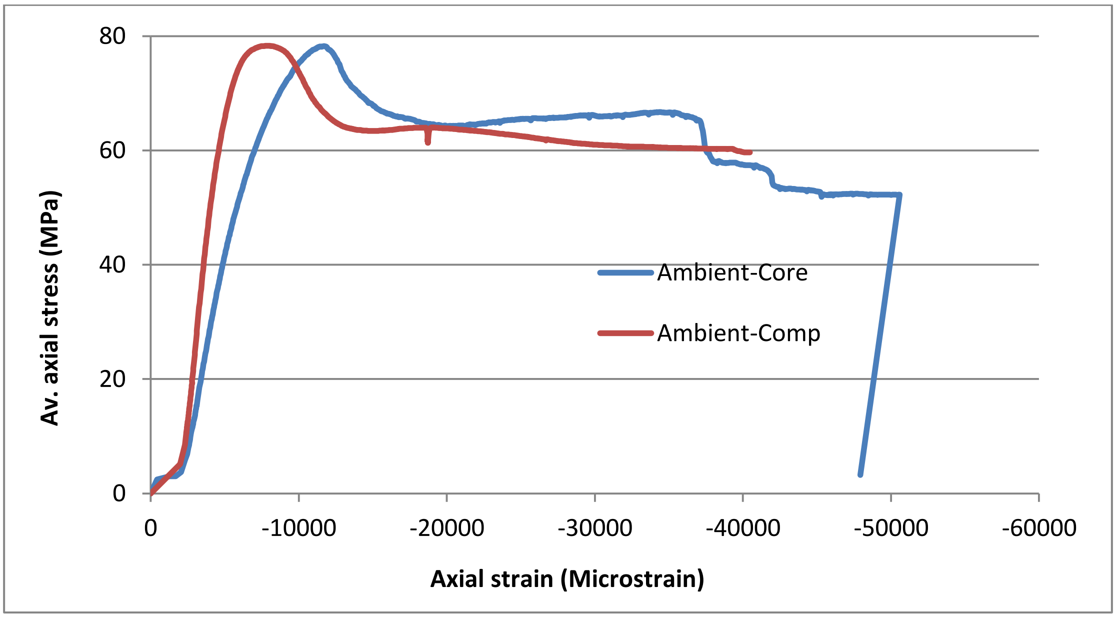
[2] Value within brackets is the ratio of average stress to compressive strength of concrete

10.3 Effect of load transfer

Figures 10.9 to 10.17 show the load-deformation and stress-strain curves for the concrete-filled steel columns A1, A2 and B2 tested under three exposure conditions viz. ambient, annealed and quenched after exposure of heat. The ultimate load and the corresponding stress are given in Table 10.3. It is observed from the table and figures that the load carrying capacity is almost same whether load is transferred through core or through composite section.

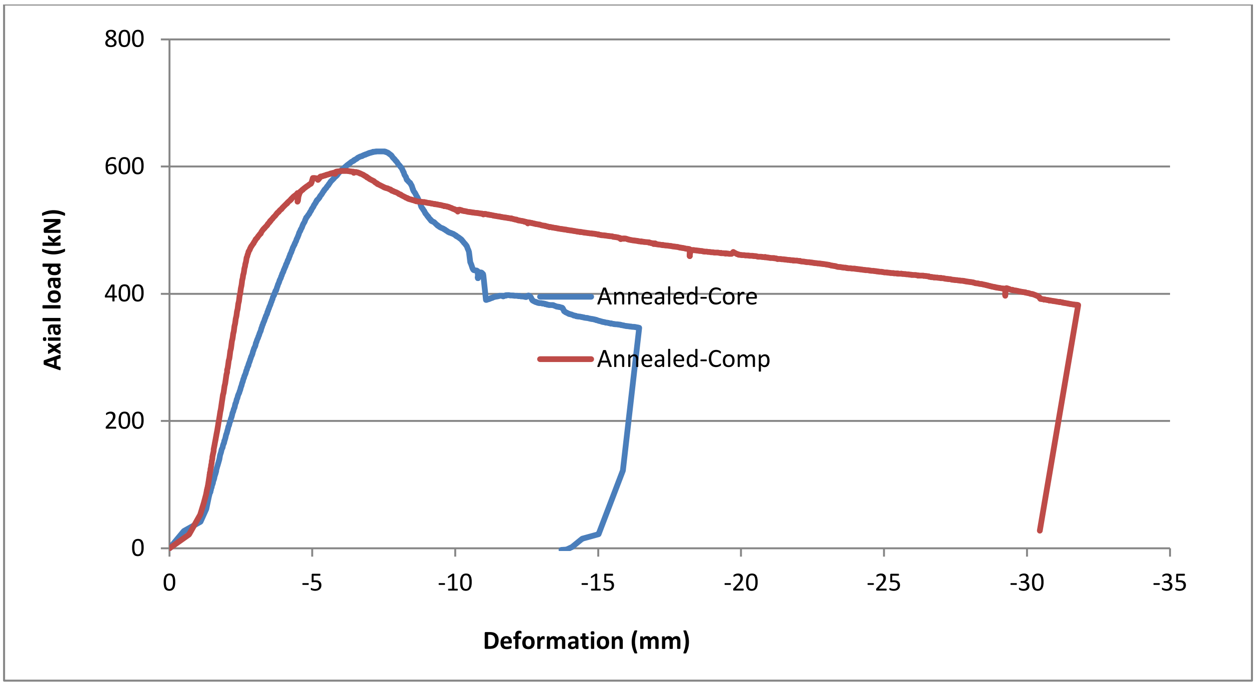


(a) Load vs deformation

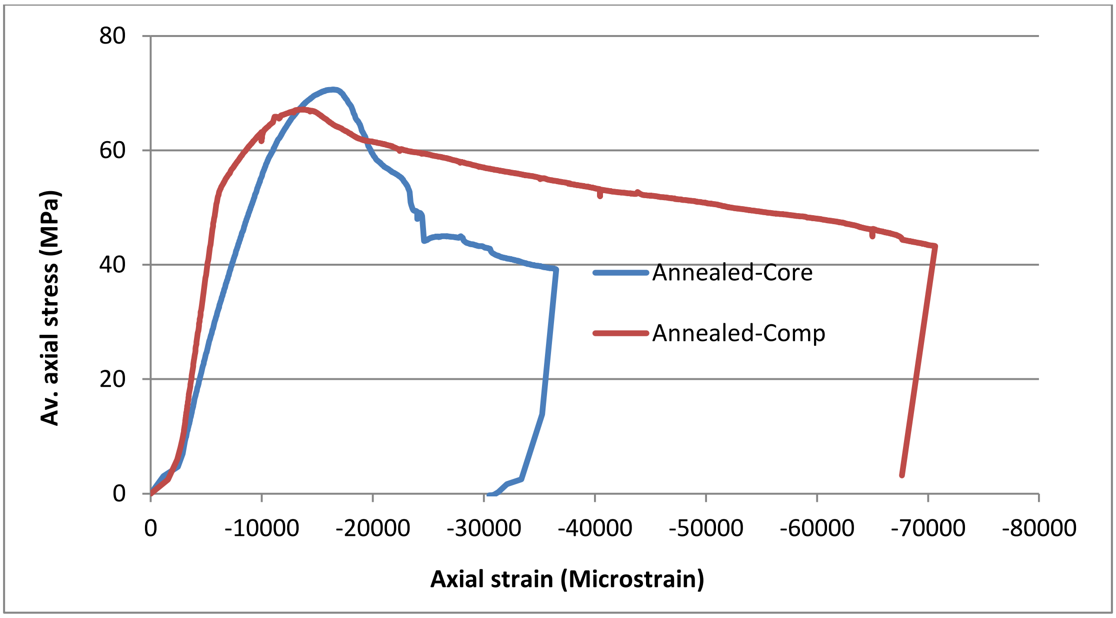


(b) Stress vs strain

Fig. 10.9: Effect of load transfer at ambient condition on CFST columns type A1

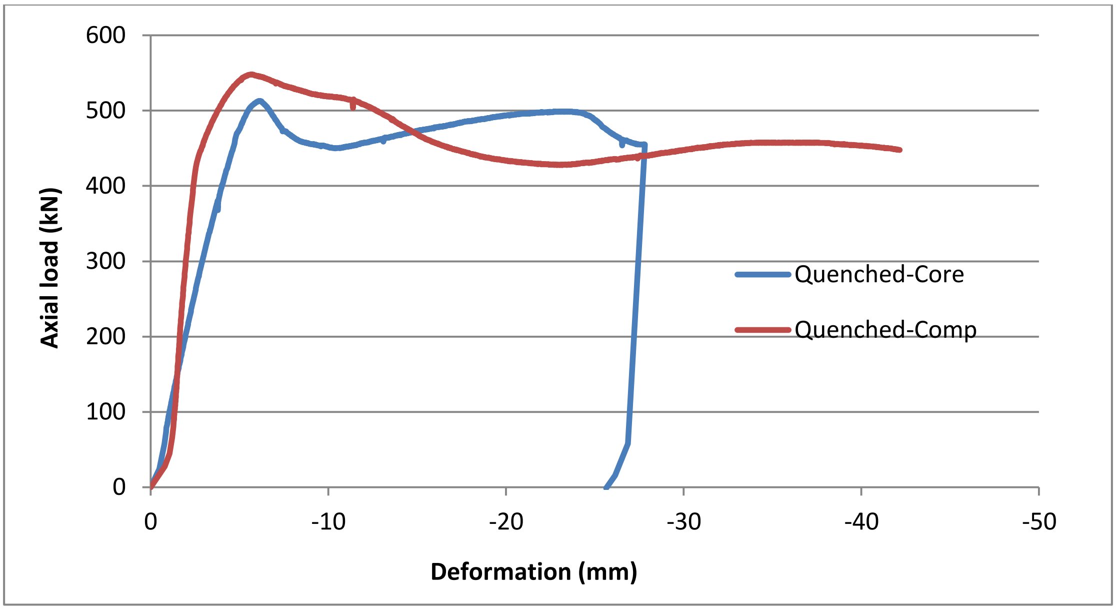


(a) Load vs deformation

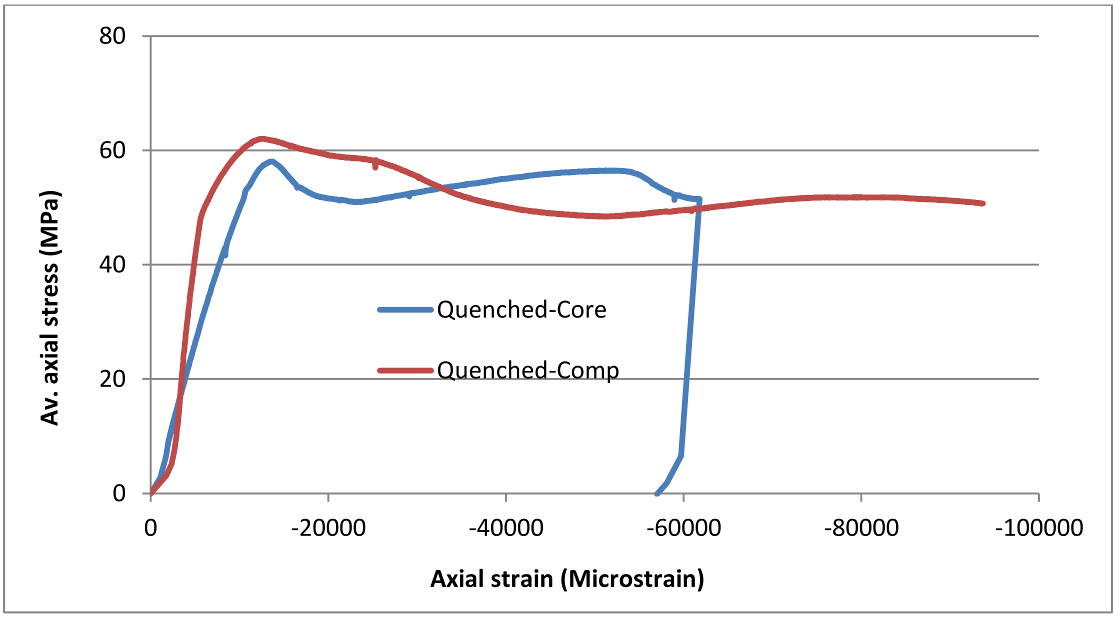


(b) Stress vs strain

Fig. 10.10: Effect of load transfer on CFST annealed columns type A1

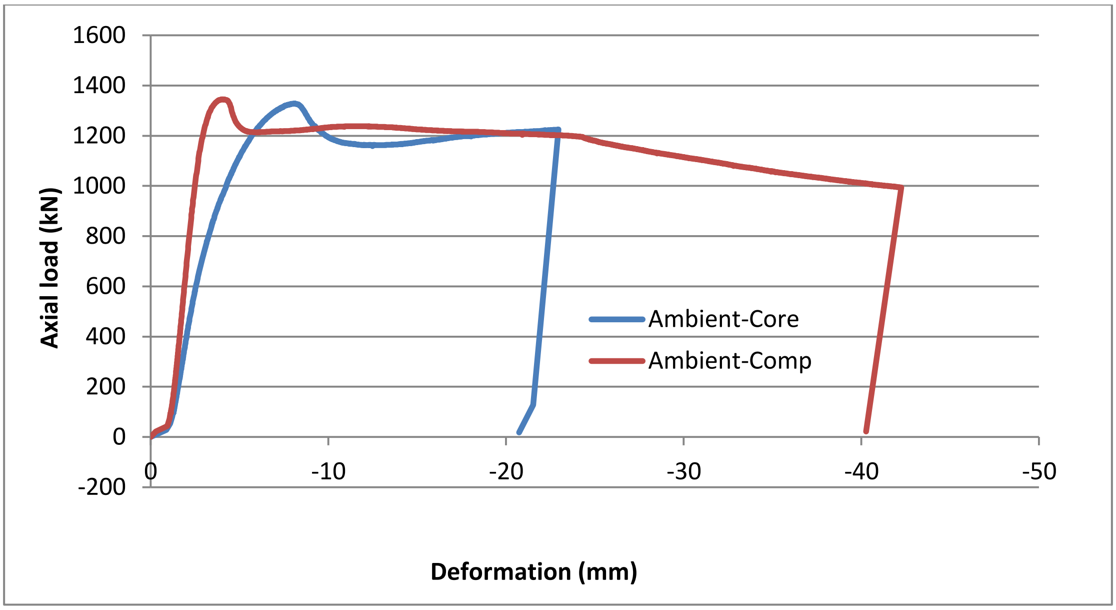


(a) Load vs deformation

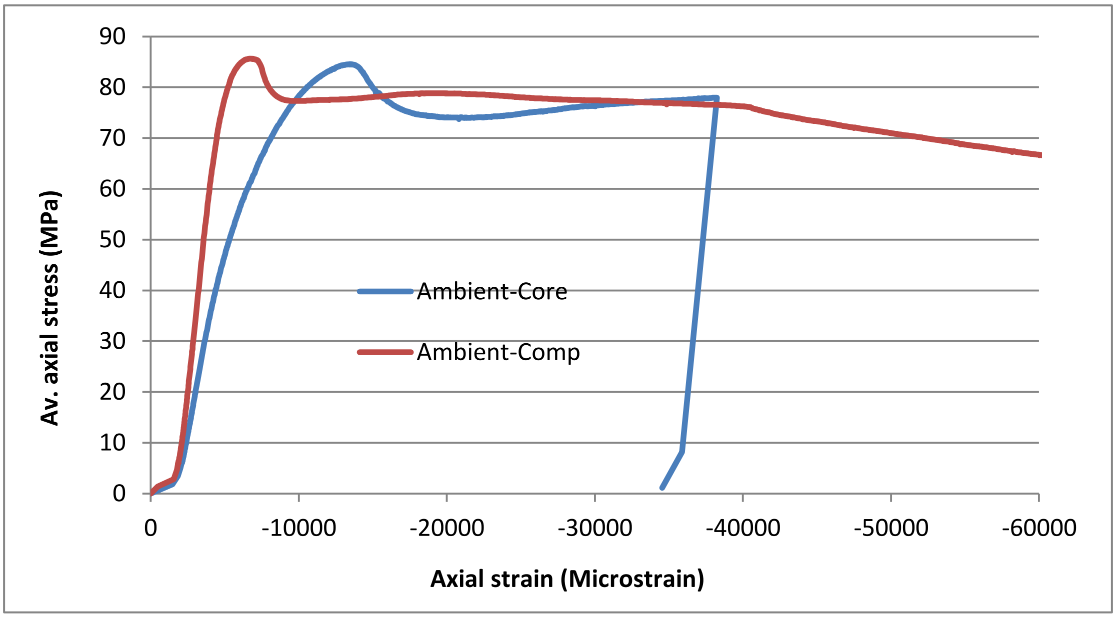


(b) Stress vs strain

Fig. 10.11: Effect of load transfer on CFST quenched columns type A1

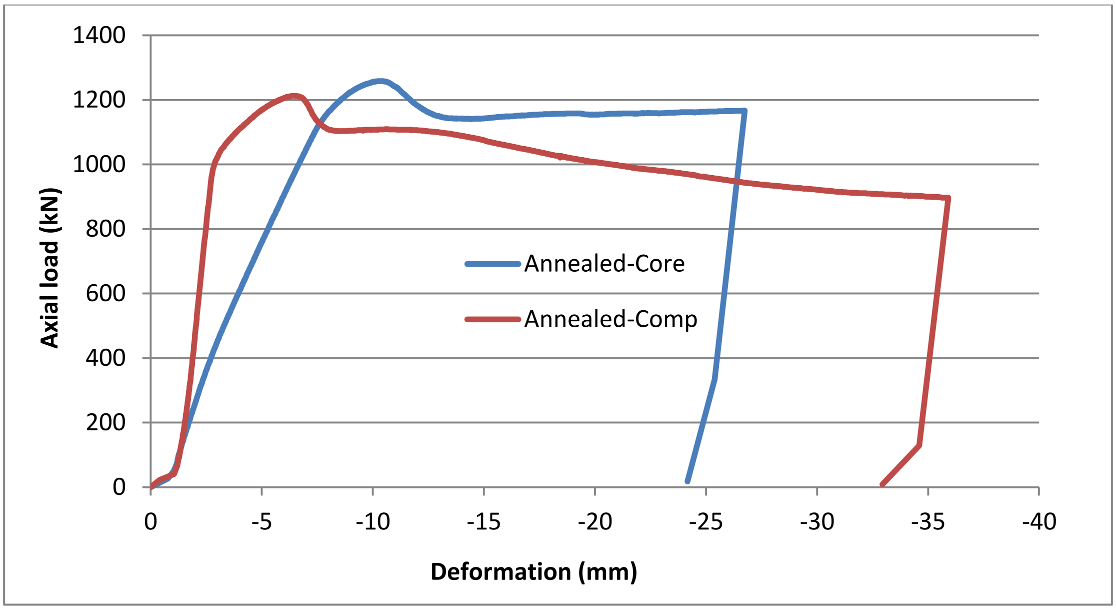


(a) Load vs deformation

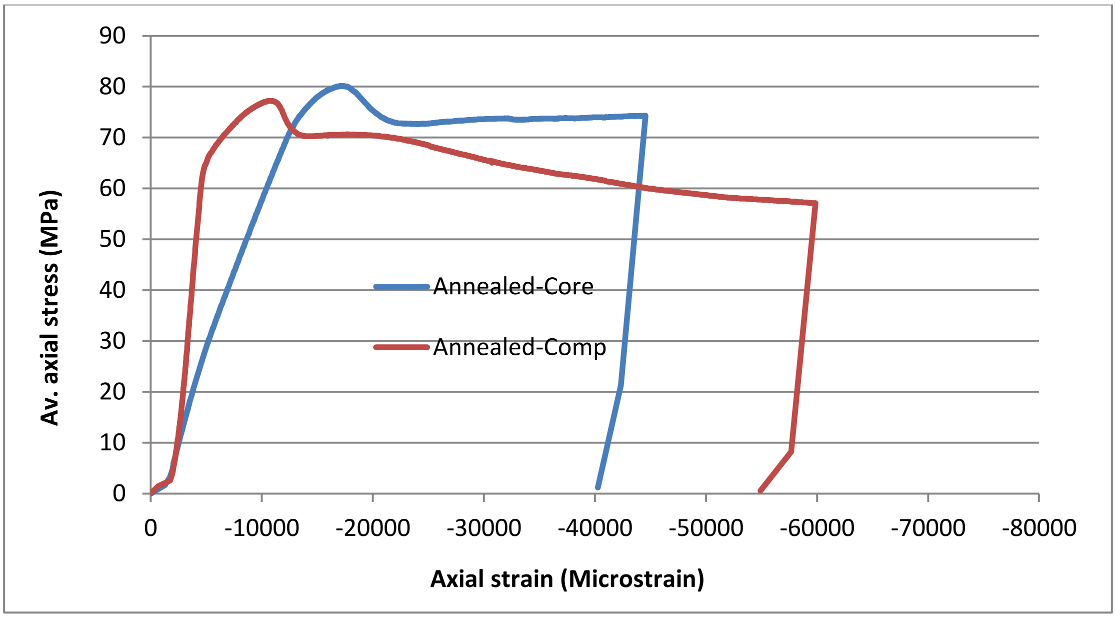


(b) Stress vs strain

Fig. 10.12: Effect of load transfer at ambient condition on CFST columns type A2

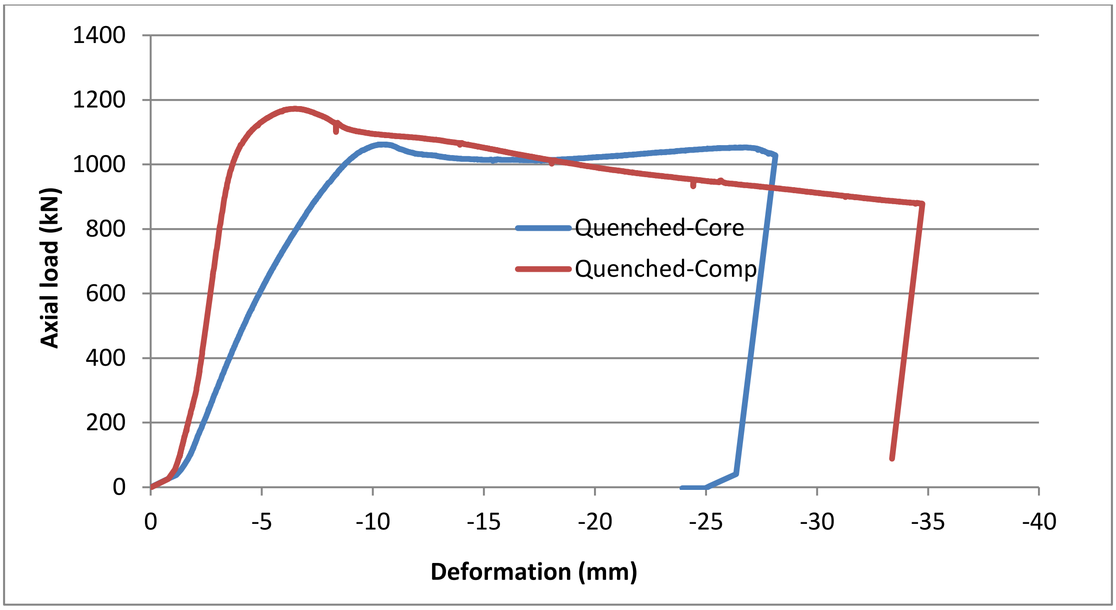


(a) Load vs deformation

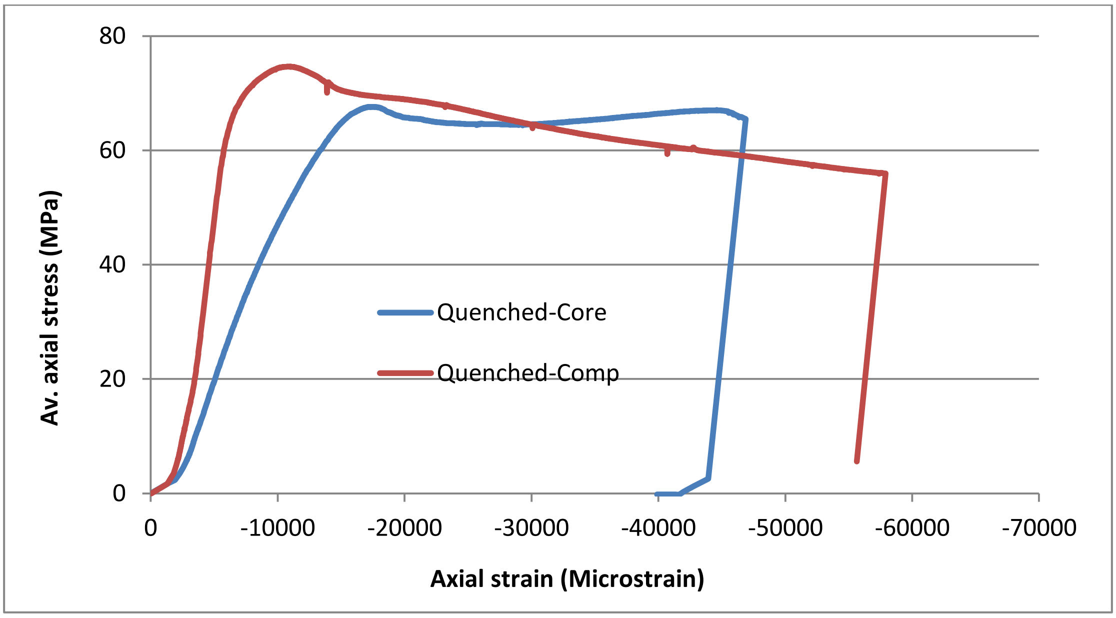


(b) Stress vs strain

Fig. 10.13: Effect of load transfer on CFST annealed columns type A2

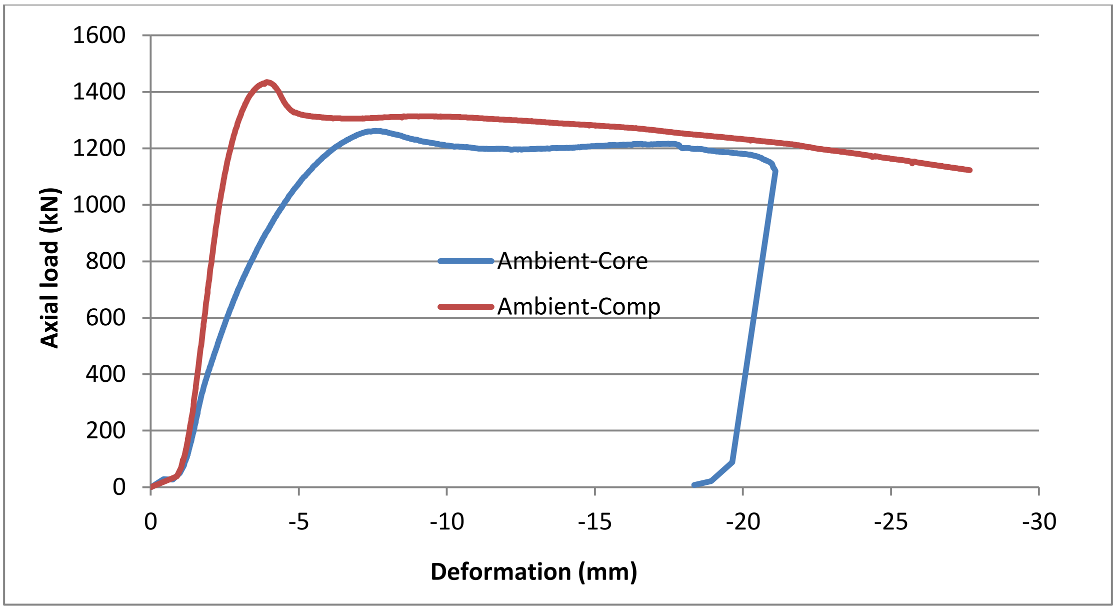


(a) Load vs deformation

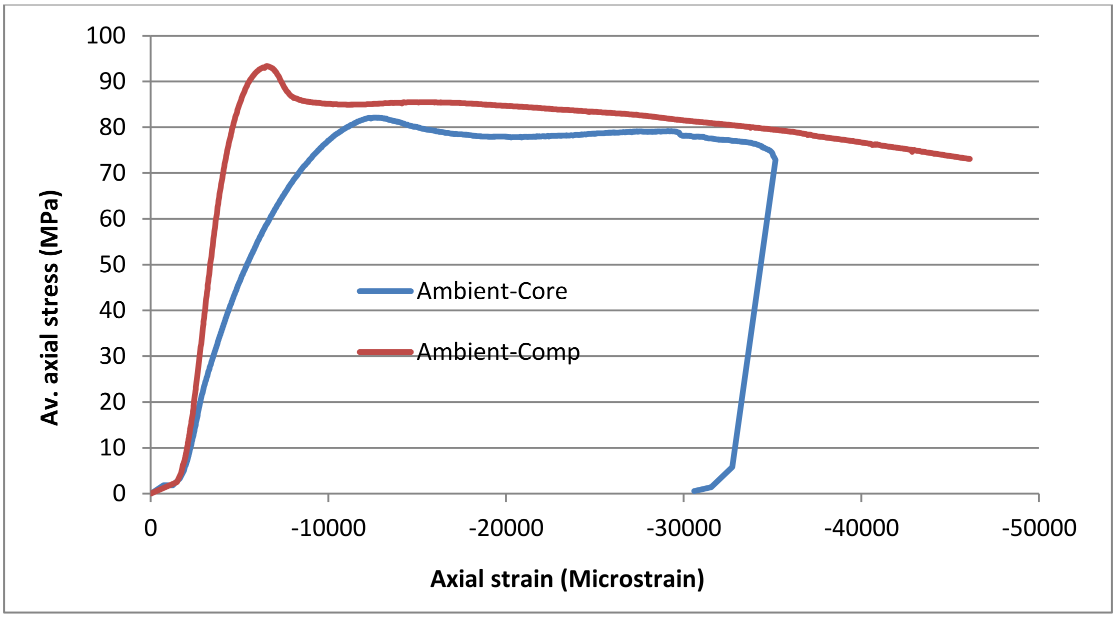


(b) Stress vs strain

Fig. 10.14: Effect of load transfer on CFST quenched columns type A2

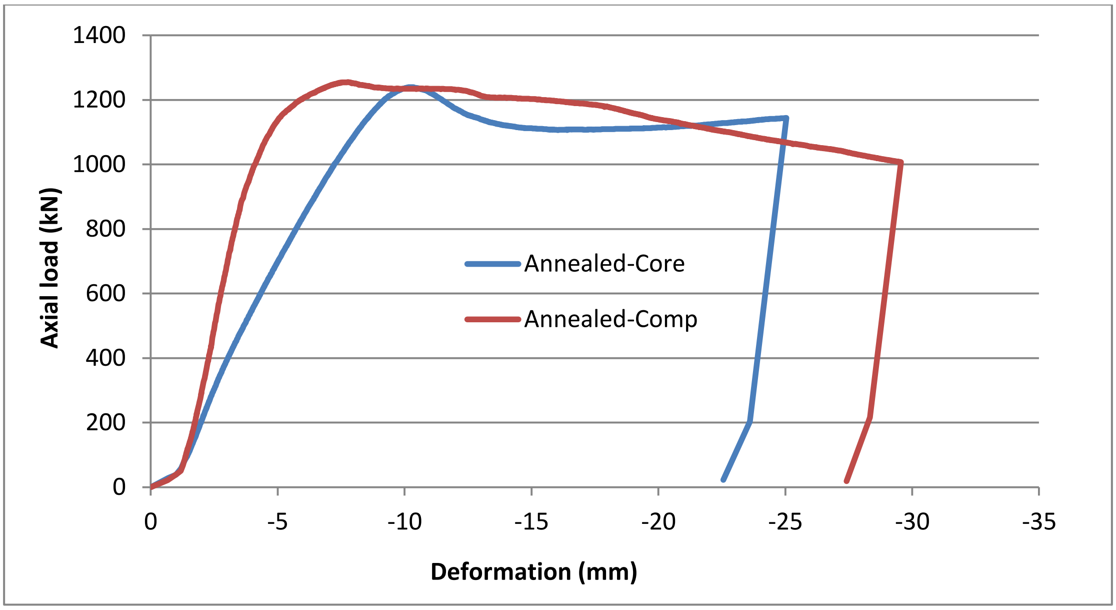


(a) Load vs deformation

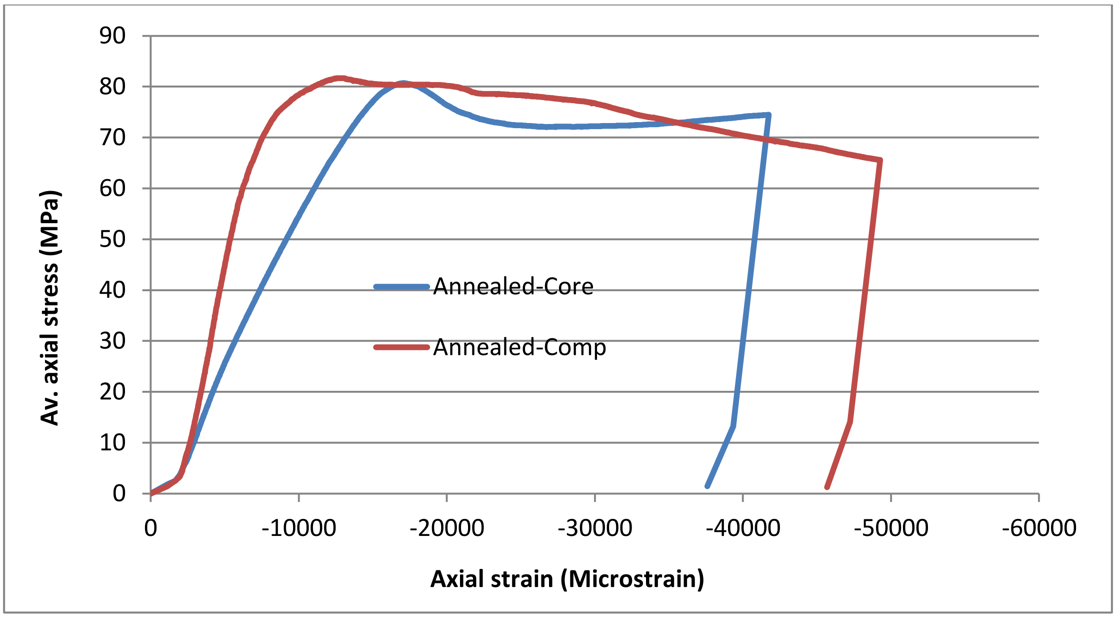


(b) Stress vs strain

Fig. 10.15: Effect of load transfer at ambient condition on CFST columns type B2

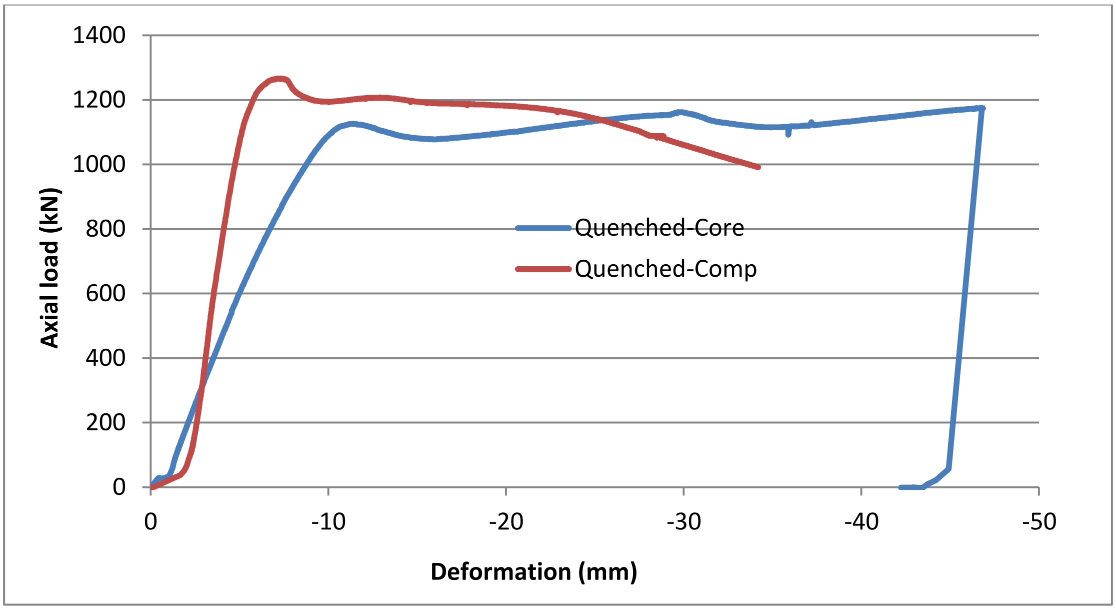


(a) Load vs deformation

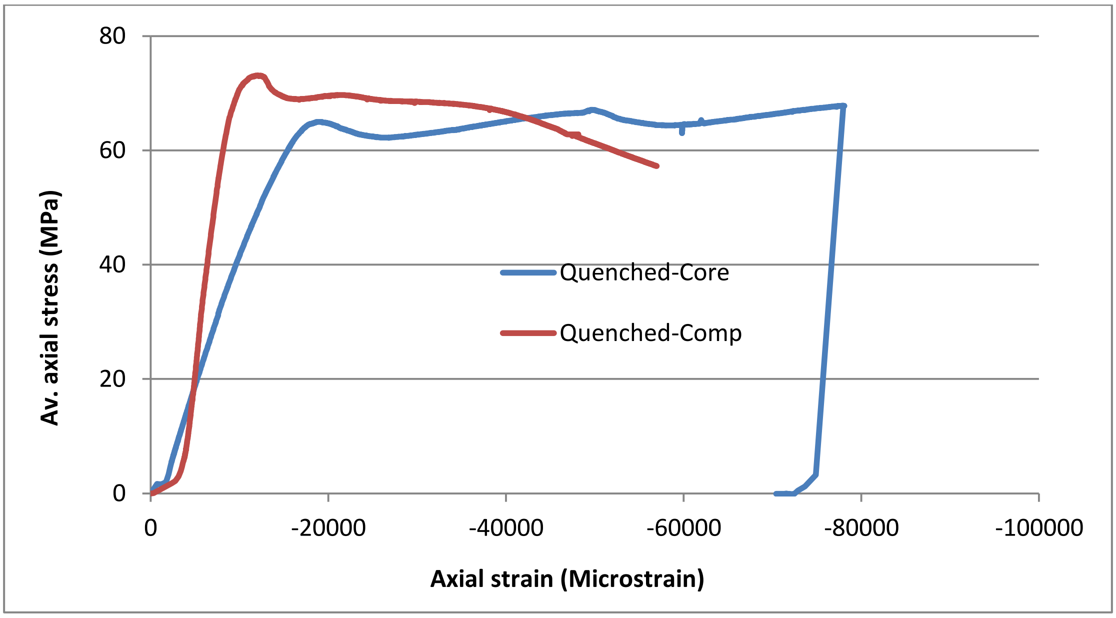


(b) Stress vs strain

Fig. 10.16: Effect of load transfer on CFST annealed columns type B2



(a) Load vs deformation



(b) Stress vs strain

Fig. 10.17: Effect of load transfer on CFST quenched columns type B2

Table 10.3: Effect of load transfer

Column ID	Load carried by column (kN)		P_{Comp}/P_{Core}
	Core Loaded, P_{Core}	Composite Loaded, P_{Comp}	
Ambient			
A1	691.2	692	1.001
A2	1328.2	1344.9	1.012
B2	1262.1	1434.4	1.13
Annealed			
A1	623.8	593.2	0.95
A2	1258.8	1212.7	0.96
B2	1239.7	1254.9	1.01
Quenched			
A1	512.9	548.1	1.06
A2	1061.8	1172.7	1.10
B2	1175.3	1266.1	1.077

10.4 Performance of annular column

A comparison of load carrying capacity of solid CFST column A2 and its annular column B2 is given in Table 10.4. It is observed from the table that the load carrying capacities of solid and annular columns are almost same for all exposure condition. Thus the solid columns may be made annular without any loss of axial load carrying capacity.

Table 10.4: Performance of annular columns

Exposure Condition	Load carried by column (kN)		P_{B2}/P_{A2}
	Solid Column (A2), P_{A2}	Annular Column (B2), P_{B2}	
Core Loaded			
Ambient	1328.2	1262.1	0.95
Annealed	1258.8	1239.7	0.98
Quenched	1061.8	1175.3	1.1
Composite Loaded			
Ambient	1345	1434	1.07
Annealed	1212.7	1254.9	1.03
Quenched	1172.7	1266.1	1.08

10.5 Confinement of concrete by steel column

Figures 10.18 to 10.20 show the variation of lateral strain in outer steel surface of CFST columns A1, A2 and B2 respectively. The strain values and corresponding stresses in outer steel shell at working load (50% of the ultimate load) are reported in Table 10.5. It is observed from the table and figures that the circumferential stress developed at the vertices of minor diameter of outer steel shell for solid columns varies from 13% to 19% of yield stress whereas for annular columns it is roughly 12% of yield stress. Whereas, the circumferential stress developed at the vertices of major diameter of outer steel shell for solid columns varies from 7% to 8% of yield stress whereas for annular columns it is roughly 10% of yield stress. Thus the stress at the extremities of major diameter is less than the stress at the extremities of minor diameter of outer steel shell.

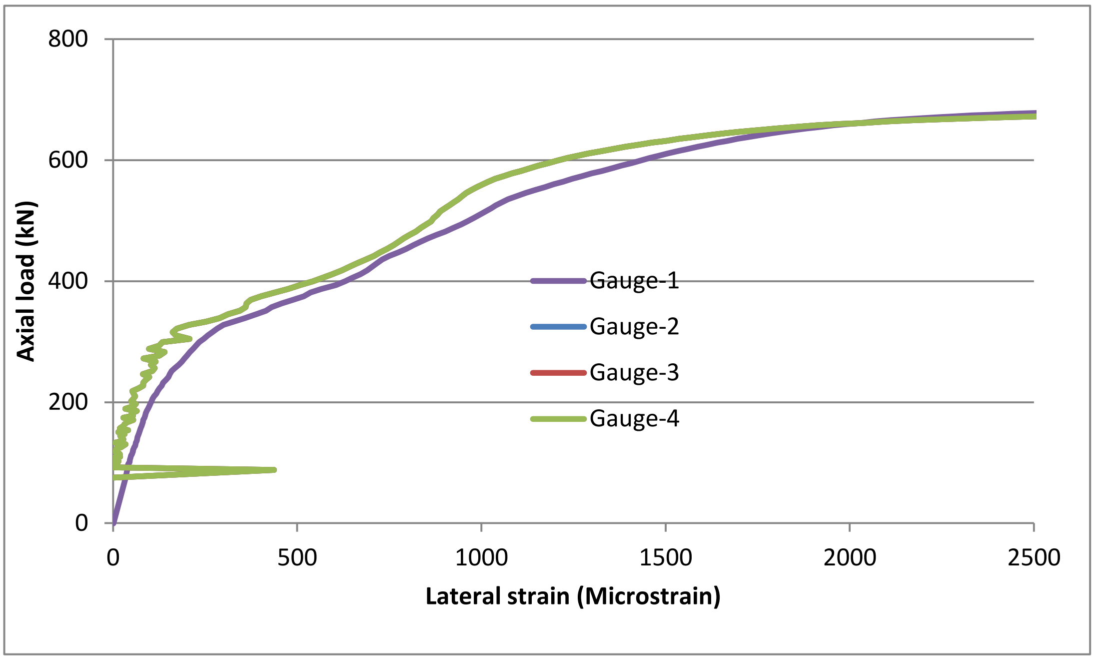


Fig. 10.18: Lateral strain in steel for CFST column type A1 at ambient temperature

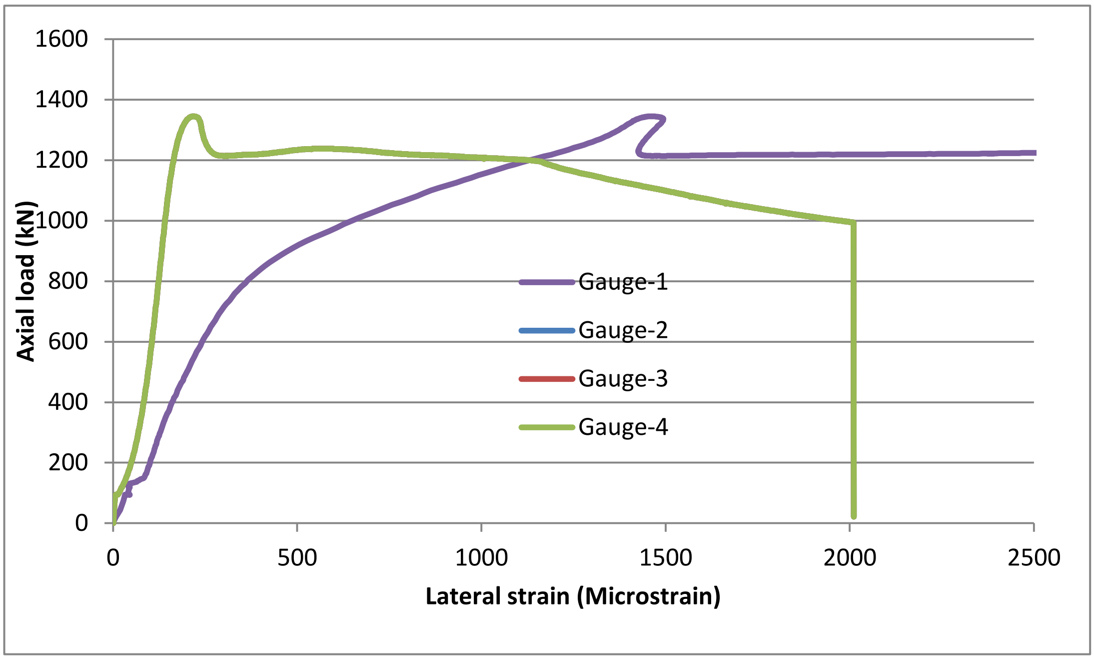


Fig. 10.19: Lateral strain in steel for CFST column type A2 at ambient temperature

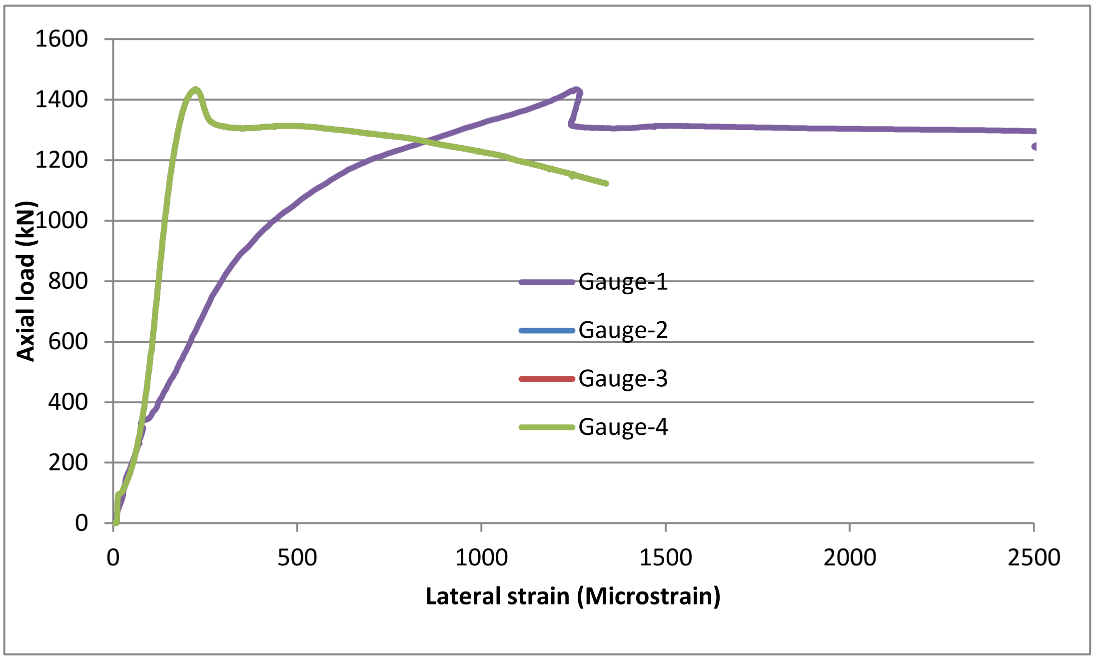


Fig. 10.20: Lateral strain in steel for CFST column type B2 at ambient temperature

Table 10.5: Lateral strain and stress in outer shell for 50% of ultimate load

	On vertices of minor diameter		On vertices of major diameter	
Column ID	Strain (Micro-strain)	Stress[1] (MPa)	Strain (Micro-strain)	Stress[1] (MPa)
A1	391.7	78.3 (0.19)	174	34.8 (0.08)
A2	278.5	55.7 (0.13)	150	30.0 (0.07)
B2	257.0	51.4 (0.12)	210	42.0 (0.10)

[1] Value within brackets is the ratio of stress to the yield stress of steel

10.6 Axial stiffness of concrete filled steel tubular columns

The axial stiffness of columns at ambient temperature are reported in Table 10.6.

Table 10.6: Axial stiffness of columns

Column ID	Axial stiffness (kN/mm)
A1	533.3
A2	713.8
B2	874.3

10.7 Mode of failure

Some of tested specimens are shown in Appendix-A. The failure of column specimens is by bulging of outer steel shell at approximately the mid height of column.

11. Conclusions

i) The yield stress could not be reached in steel columns when steel columns without concrete filling were tested.

ii) The load carrying capacity of steel post-cooling after exposure to elevated temperature got reduced by up to 13.9%.

iii) A comparison of annealed and quenched specimens shows that the annealed specimens exhibit more loss in the load carrying capacity.

iv) At ambient temperature, the ratio of peak average stress to the uniaxial compressive strength of solid CFST columns varies from 2.0 to 2.1, whereas for annular CFST columns, it varies from 2.1 to 2.3.

v) The loss of load carrying capacity of quenched CFST columns is more than the annealed columns.

vi) the load carrying capacity is almost same whether load is transferred through core or through composite section

vii) The load carrying capacities of solid and annular columns are almost same for all exposure condition. Thus the solid columns may be made annular without any loss of axial load carrying capacity.

viii) The circumferential stress developed at the vertices of minor diameter of outer steel shell for solid columns varies from 13% to 19% of yield stress whereas for annular columns it is roughly 12% of yield stress. Whereas, the circumferential stress developed at the vertices of major diameter of outer steel shell for solid columns varies from 7% to 8% of yield stress whereas for annular columns it is roughly 10% of yield stress. Thus the stress at the extremities of major diameter is less than the stress at the extremities of minor diameter of outer steel shell.

References

1) X.L. Zhao, J.A.Packer. Tests and design of concrete-filled elliptical hollow section stub columns, Thin-Walled Structures Vol. 47 (2009) Pg. 617–628.

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Appendix A

Fig. A-1 Samples before casting

Fig. A-2 Annular column

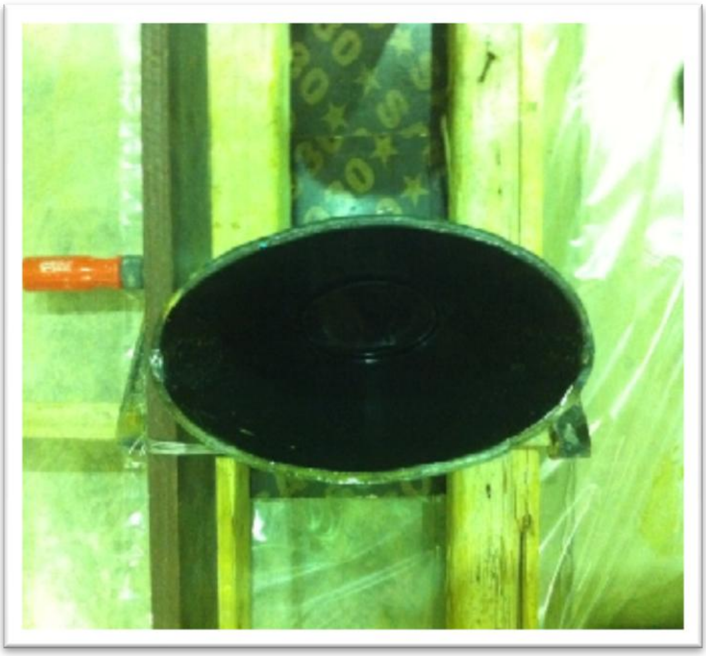

Fig. A-3 composite column

Fig. A-4 Curing

Fig. A-5 Machine

Fig. A-6 During Testing

Fig. A-7 using LVDT

Fig. A-8 Specimens after casting and curing

Fig. A-9 Oven

Fig. A-10 Quenching Tank

Fig. A-11 The Team

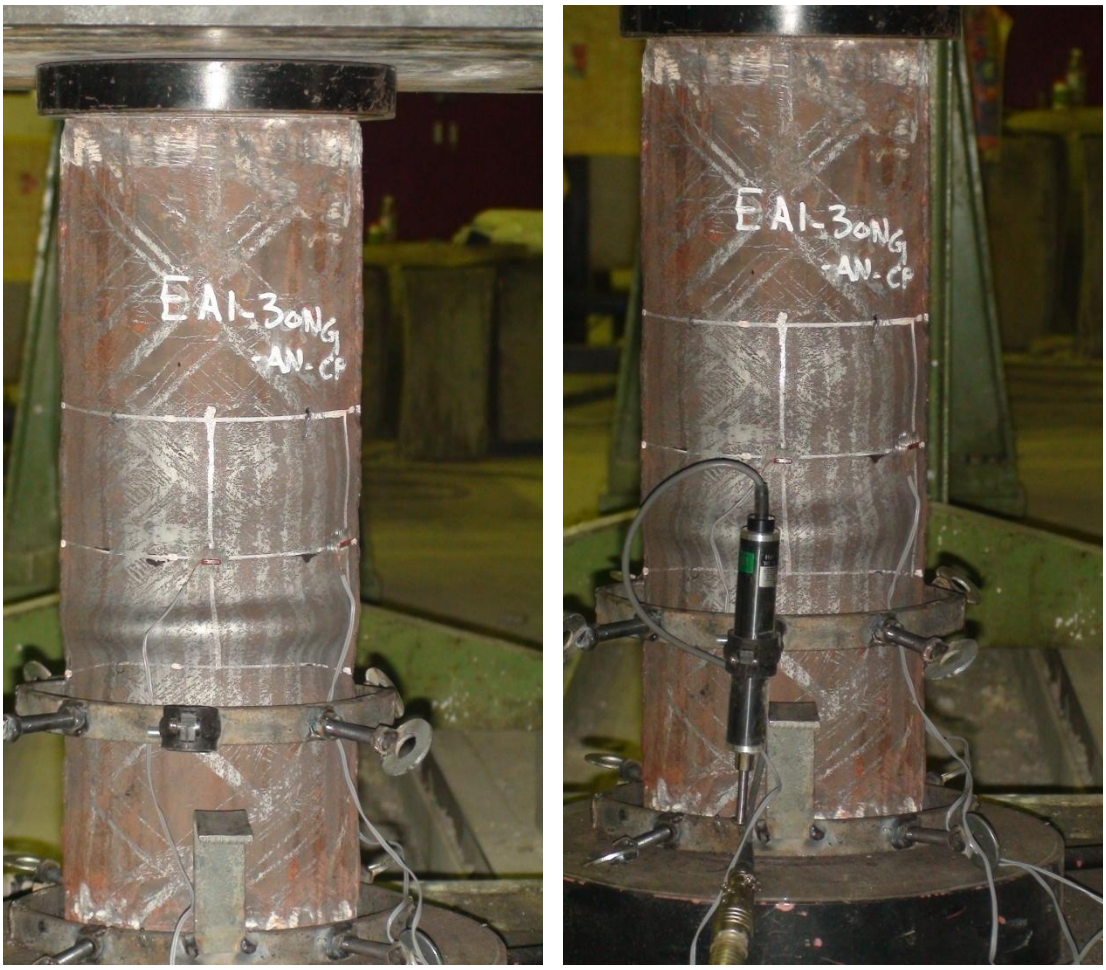

Fig A-12 Some of the tested specimens showing mode of failure