Localised resource for UK

NCCI: Design of reinforced concrete filled, hot finished structural steel hollow sections in fire

The UK National Annex to BS EN 1994-1-2 recommends that informative Annex H (Simple calculation model for concrete filled hollow sections exposed to fire all around the column according to the standard temperature-time curve) should not be used. This NCCI document provides alternative guidance.

March, 2014

In association with TATA STEEL

Design Guide for Concrete Filled Hot Finished Structural Hollow Section (SHS) Columns

Y C Wang

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Forward

The Design guide for SHS concrete filled columns was first published by The Steel Construction Institute in 2002, collated and edited at Corus Tubes, Corby from original contributions by Dr Stephen Hicks and Mr Gerald Newman of SCI and supplementary text from Mr Mike Edwards and Mr Andrew Orton of Corus. It has been widely used in practice.

This new guidance document has been updated to be consistent with the requirements of Eurocode 4: Design of composite steel and concrete structures — Part 1-1: General rules and rules for buildings (BS EN 1994-1-1[12]) and Eurocode 4: Design of composite steel and concrete structures —Part 1-2: General rules-Structural fire design (BS EN 1994-1-2[13]), including the provisions of each standard's UK National Annex. It should be noted that the UK National Annex to BS EN 1994-1-2[13] prohibits the use of Annex H in determining the capacity of a composite column subject to compression and bending at elevated temperature and makes reference to non-contradictory complementary information (NCCI), in lieu of this. This design guide constitutes the relevant NCCI information.

This revised edition also includes the latest advice on fabrication, erection and constructability and updated worked examples.

This edition of the publication was prepared for Tata Steel Europe by Professor Yong Wang of The University of Manchester. The author would like to thank Mr John Dowling and Mr Paul Watson of Tata Steel Europe, Mr Rob Halpin of Byrne Bros, Mr James Scarisbrick and Mr Iain Sproat of Ramboll, Mr. Mike Banfi and Dr. Angus Law of Arup, and Mr. David Brown of SCI for their comments and advice on the publication, which are gratefully acknowledged.

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Summary

A concrete filled structural hollow section provides architects and engineers with an aesthetically pleasing, robust and inherently fire resistant column. This publication contains design information for these columns for both the ambient temperature and fire conditions. The information is based on the European code for composite construction for the ambient temperature condition, BS EN 1994-1-1[12], the main difference between ambient temperature and fire designs being the modifications of mechanical properties at elevated temperatures for the fire conditions. Also included are building and bridge case studies illustrating the use of concrete filled columns and practical guidance on concrete filling and connection design.

This design guide forms the basis of design software, FireSoft.

This guide is applicable to hot finished structural sections of fine grain steel (BS EN 10210[4,5]) and does not apply to cold formed sections (BS EN 10219). This guide is applicable to simple construction only.

This guide supersedes the Guide of the same title published by Corus Tubes in 2002.

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1 Introduction

1.1 General

Structural Hollow Sections are the most efficient of all structural steel sections in resisting compression. Tata Steel Celsius® 355NH hot finished structural hollow sections have a high strength to weight ratio and produce slender attractive lines that make them a natural choice for building structures. These hollow sections can achieve a constant external dimension for all weights of a given size, which enables them to achieve standardisation of architectural and structural details throughout the full height of the building if required.

By filling hollow sections with concrete and reinforcement a composite section (concrete filled tube or CFT) is produced (see Figure 1-1), which will increase the section's room temperature load carrying capacity, whilst retaining all the advantageous features of the basic unfilled section. Alternatively, for the same original load capacity, the presence of concrete and reinforcement permits smaller composite sections to be used. Where this happens, the reduction in section size also gives advantages in subsequent construction processes, including a reduced surface area for painting and lower lifting weights for cranes.

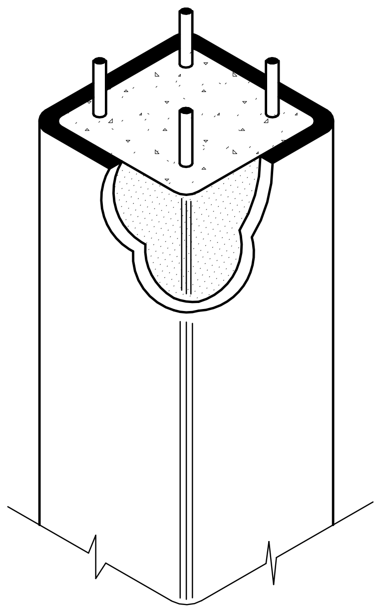


Figure 1-1 Concrete filled square hollow section

The type of CFT column chosen will depend on the design specification, but the construction process anticipated and the equipment available will also have an important influence. With larger columns, steel reinforcing bars may be easily fitted within the column and this has the added advantage that no external fire protection is then usually necessary. However, in many cases an unreinforced, fire protected, column may still provide a more convenient and more economic solution even where it results in larger column plan dimensions.

In terms of fire resistance, concrete filled hollow sections can be divided into those that are externally protected against fire and those that have no such protection. A further division can be made by differentiating between those that are filled with plain concrete or reinforced concrete.

In general, hollow sections with external fire protection will not need internal reinforcement to achieve desired fire ratings. There may be rare situations where internal reinforcement and external protection are used together. This usually happens in situations where the columns are carrying high loads and it is necessary to keep column sizes manageable.

Where the section is filled with concrete without reinforcement, the external fire protection thickness may be significantly reduced compared to that required for the same section when unfilled. This is possible because of the heat sink effect of the concrete, which effectively reduces the section factor of the composite column. These reductions have been shown to be particularly substantial where intumescent coatings are the preferred form of external fire protection. This is usually the case when using hollow sections.

Concrete filled structural hollow sections that are not externally protected against fire mainly use a reinforced concrete core to resist the fire limit state design loads. In general, such sections will need to contain reinforcement in order to sustain the required fire limit state design loads for fire resistance periods of 60 minutes or more.

In cases where smaller columns are used, it may not be possible to include internal reinforcement and therefore externally applied fire protection must be specified to achieve fire ratings of 60 minutes and above.

In general small columns, having plan dimensions less than 300 mm say, would be designed without steel reinforcing bars because of the relatively small effect of the reinforcement and the difficulty of fitting in the bars and of placing concrete with them present. This type of column will usually require external fire protection, such as intumescent coatings, to achieve the required level of fire protection.

Cost comparisons

Comparisons have shown that, to establish the cost competitiveness of a hollow section column, it is necessary to take into account both the cost of the supplied and erected steel section together with the cost of any fire protection. Such comparisons have shown that concrete filled hollow sections can offer a competitive first choice solution to structural columns in multi-storey construction.

Construction

Concrete or grout filling of structural hollow sections requires no special equipment and the filling operation may be integrated into other concreting operations. The enhancement in the overall efficiency obtained by filling a steel structural hollow section with concrete allows the designer a wider choice of sections. Filled hollow section columns combine the advantages of economy in the use of materials with the construction advantages of the use of steelwork. Columns, whether externally fire-protected or not, will usually arrive to site as fully finished elements with make-ups only at column splice joints, if any. Concrete filling of the hollow section columns can take place on or off site. If filling takes place on site, then the steel column and its connections should be designed to carry all construction loads so that the operation of filling the columns can be taken off the critical path. In larger buildings, the best economy is obtained by planning for the simultaneous working of different trades at different levels or plan positions. The filling of hollow sections could occur simultaneously with other concrete works at that level, e.g. the concrete pour of a concrete of composite floor slab or screed. The hollow section columns may be filled from the top with a self-compacting or other type of mix; alternatively, they can be filled from the bottom, through a gate-valve, with a pumpable concrete.

1.2 Advantages of composite hollow sections

In the initial construction period

The steel section eliminates the need for formwork. Concrete placement and compaction in many cases are unhindered by internal reinforcement.

Erection schedules are not dependent on concrete curing time.

During finishing

The concrete filling is protected against mechanical damage. Additional external fire protection is usually not necessary if using internal reinforcement (Figure 1-2). Slender columns reduce the application time and cost of applied finishes.

Completed building

- Greater useable floor area.
- Reduced maintenance.
- Aesthetically pleasing.

Figure 1-2 Tubular column with reinforced concrete infill. This will not require external fire protection

Many of these advantages were recognised early in the history of the use of iron and steel hollow sections in construction, indeed the first known patent relating to the concrete filling of circular hollow sections dates from 1898. The wider use of the composite concrete hollow section did not really begin until the mid 20th Century following the results of structural investigation and the availability of a comprehensive range of structural hollow sections.

The material included in this manual makes the design of structural hollow section columns filled with concrete simple and rapid when standard sections are used in conjunction with concrete of common grades.

1.3 Applications of concrete filled columns

1.3.1 FLEET PLACE HOUSE, LONDON (Figure 1-3)

The site of Fleet Place, off Holborn Viaduct in the City of London was comprehensively redeveloped at the same time as the adjacent Thameslink station. This eight-storey high office block on the site was built using concrete filled external CHS columns on each longitudinal face of the building and has clear spans on the inside. So that they could be supported off existing pilecaps, the CHS columns were kinked at first floor level, except at the entrance leading to the Thameslink station behind the building, where the columns were kinked at second and third floor levels.

Figure 1-3 Fleet Place House, London

Technical details

Construction details

The columns have a constant external diameter of 323.9 mm but, within this serial size, vary from 323.9*30CHS at the first floor, 323.9*16CHS between second and fifth floors and 323.9*12.5CHS between sixth and seven floors. Tata Steel Celsius® 355 was used.

The fire rating for internal elements was generally two hours but the requirement for the external columns was only 35 minutes in view of their position outside the cladding line. No external fire protection was given to the columns, a 45 minute rating being achieved by concrete filling alone. The concrete infill used was to Grade 40 or Grade 60 depending on the vertical load.

Project data

Client: Heron Property Corporation

Architect: Skidmore, Owings & Merrill

Structural Engineer: Waterman Partnership

1.3.2 MONTEVETRO APARTMENT BLOCK, LONDON (Figure 1-4)

Steel composite tubular columns have been used on the facade of this high specification apartment block, which looks out across the River Thames. The composite columns were chosen because of their small plan area and their slender shape which minimised the obstruction to the views out from the apartments.

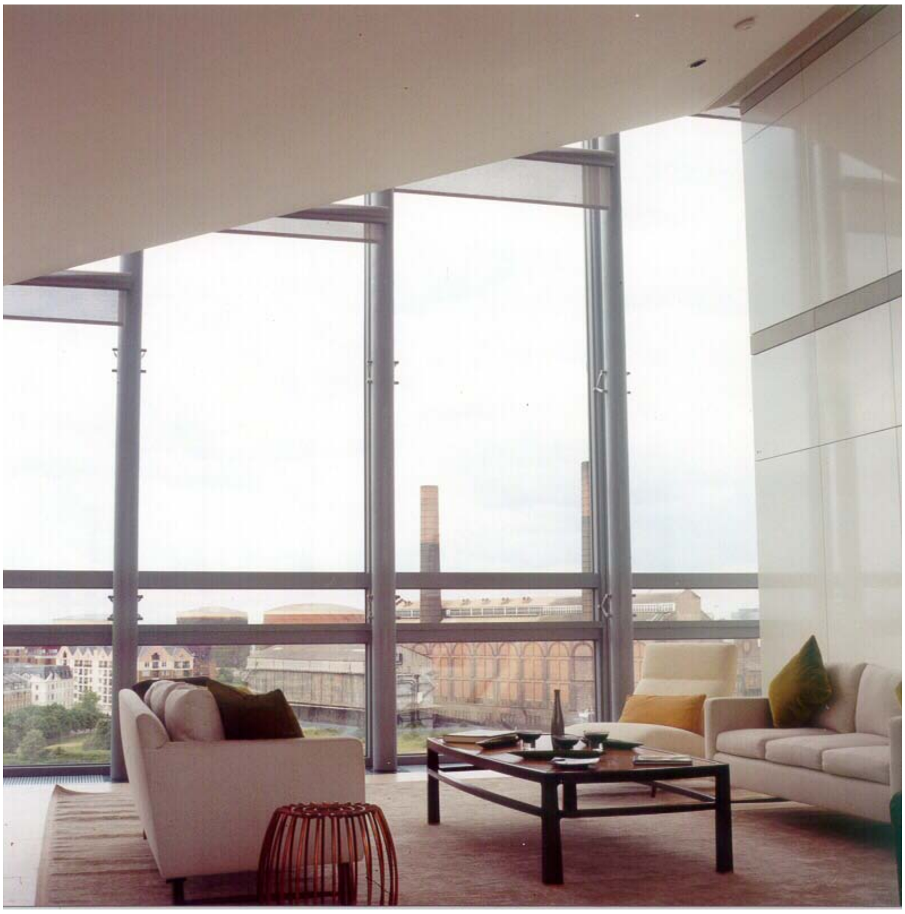


Figure 1-4 Montevetro Apartment Block, London

Technical details

Construction details

CHS columns are used and, in the typical case, are only 244.5 mm in diameter. Within this serial size the columns vary from 244.5*16CHS to 244.5*20CHS; for the most heavily loaded areas, the columns were increased to a maximum size of 355.6*16CHS.

The steel CHS columns are used to support concrete flat slab floors. At the highest point the apartment block has twenty storeys and there are heavy axial loads in the columns in these areas. Tata Steel Celsius® 355 was used.

The fire rating required for the columns varied between one and two hours and was achieved by the combined effect of the concrete infill and a thin film intumescent coating. No reinforcement was used inside the CHS columns.

Project data

Client: Taylor Woodrow Capital Developments

Architect: Richard Rogers Partnership

Structural Engineer: Waterman Partnership

1.3.3 PECKHAM LIBRARY, LONDON (Figure 1-5)

This library building is of striking appearance. It is supported at the front by concrete-filled CHS columns, angled to form an irregular facade and enclosing a covered space, which is an extension of a new public square. There are 7 supporting columns, 18 metres long, which are angled out of the vertical, the columns supporting steel tubular trusses at the fourth floor and a steel roof above. At ground floor level, the columns have a height of 18 metres, giving a height to width ratio of 37. The angling of the columns helps to provide additional stiffness against lateral loads and limits the development of bending moment in the columns.

Figure 1-5 Peckham Library, London

Technical details

Construction details

At ground floor level, the column line consists of a 323.9*20CHS column at each end with 323.9*16CHS columns in-between. At first floor level, these section sizes reduce to 323.9*6.3CHS members. All columns were designed without external protection, utilising an infilled concrete core within the tubes to satisfy the requirement for a one-hour fire rating. The concrete infill is reinforced with 8 T12 longitudinal bars with T6 links at 175 centres. Tata Steel Celsius® 355 was used.

Project data

Client: London Borough of Southwark

Architect: Alsop Architects

Structural Engineer: Adams, Kara, Taylor

1.3.4 QUEENSBERRY HOUSE, LONDON (Figure 1-6)

The steel columns used to support the floors of this six-storey office and commercial buildings are CHS tubular columns. The columns use a tube-in-tube system in which one CHS section is placed inside a larger one with all the voids grouted after erection of the floor structure.

Figure 1-6 Queensberry House, London

Technical details

Construction details

There is an atrium in the middle of the building, which is 6 metres wide, with 12 metre clear spans each side of the atrium in the transverse direction. In a typical case the column consists of a 457*10CHS outer tube and a 323.9*6.3CHS inner tube and supports two floor beams on each side. All columns were delivered to site in two three-storey lengths and were joined by means of an *in situ* concrete joint in the inner tube and by bolting and welding on the outer tube. After erection, the columns only needed to be made good at the joint and given a final finish coat of paint.

No external fire protection was necessary, the internal column and grout infill having sufficient load capacity by itself in the fire limit state. At room temperatures, the full section capacity of the tube-in-tube column is utilised. Tata Steel Celsius® 355 was used.

Project data

Client: General Accident and Capital & City

Architect: RHWL

Structural Engineer: Buro Happold

1.3.5 SEG PLAZA, SHENZHEN, CHINA (Figure 1-7)

Concrete filled sections are widely used in prestigious high rise building and bridge constructions. At the time of its construction, the SEG Plaza in Shenzhen, China was the tallest building in the world using concrete-filled steel tubular columns. It has 76 storeys with a four level basement, each basement floor having an area of 9653m^2. The main structure is 291.6m high with an additional roof feature giving a total height of 361m.

(a) (b)

Figure 1-7 SEG Plaza, Shenzhen, China (photo courtesy of Prof. L H Han, Tsinghua University, China)

Technical details

Construction details

The concrete filled tubes were used as the external columns of the frame as well inside as concrete shear walls (Figure 1-8). The diameter of the columns used in the building ranges from 900mm to 1600mm. The steel tubes were brought to the site in lengths of three storeys and concrete was poured from the top of the column. The critical design loads for the SEG Plaza building were seismic and wind loads. Therefore, rigid connections (Figure 1-9) between the steel beams and the concrete filled tubes were used. Figure 1-9 shows a typical connection example.

Figure 1-8 SEG Plaza under construction (photo courtesy of Prof. L H Han, Tsinghua University, China)

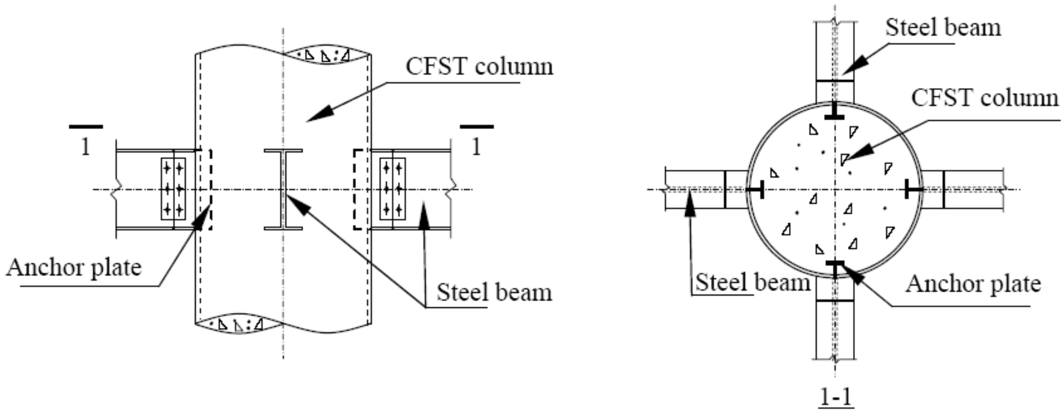


Figure 1-9 Typical connection used in SEG Plaza, Shenzhen, China

The required standard fire resistance period for this building is 3 hours. By using concrete infill, the fire protection thickness was less than 20% of that necessary to protect steel tubes without concrete infill. A lightweight spray material was used as fire protection.

1.3.6 MOORGATE EXCHANGE, LONDON (Figure 1-10)

Moorgate Exchange (Figure 1-10) is a 12-storey office building on the site of an old telephone exchange in the heart of The City of London.

Figure 1-10: Moorgate Exchange, London (during construction)

Technical details

All the internal columns are exposed circular hollow section with 508 external diameters around the cores and 457 external diameters around the façade

As all the columns are visible, a fine painted finish on the columns was an architectural requirement. In order to remove the need for intumescent coating on the columns, it was decided to fill them with reinforced concrete to achieve the 90 minute fire rating. This method of fire protection also reduced the possibility of damage to the exposed coating. The fine finishes (Figure 1-11) highlighted the superior surface quality of Celsius® 355NH hot finished structural hollow section, which was achieved by high pressure de-scaling during the manufacturing process.

Figure 1-11 Exposed concrete filled sections

‘FireSoft’, developed by the University of Manchester for Tata Steel Europe, was used to design the columns, with load transfer from beams to columns generally achieved through fin plates, in order to simplify fabrication, with fire protection applied in the ceiling zone, a requirement for load transfer. For some columns, adequate shear transfer could not be achieved and the fin plates were fed through slots in the CHS and externally fillet welded in order to achieve the shear transfer to the concrete.

In order to provide maximum flexibility during construction, the rebar cage was installed inside the CHS in the fabrication shop and the steel frame erected before concrete was pumped into the columns from a hole in the base (Figure 1-12). Filling the columns on site gave significant advantages to the contractor and took filling off the critical path. The only programme requirement was that the floor under the base of the column was cast prior to filling the columns.

Figure 1-12: Concrete pumped into the column from base

Project Data:

Main Client: Telex Sàrl
Client monitoring architect: Pringle Brandon Perkins+Will
Architect: HKR
Main contractor: Skanska
Structural engineer: Ramboll
Project manager: GVA Second London Wall
Steelwork contractor: Severfield-Rowen Structures
Steel tonnage: 2,900 tonne

1.3.7 ARCH BRIDGES (Figure 1-13)

Concrete filled tubes have been used in a large number of arch bridges in China. An important advantage of using concrete filled tube in arch bridge construction is the reduced construction cost. Since the weight of a hollow steel tube is comparatively low, erection of the steel tube can often be performed with relatively simple construction technology. The frequently used methods include cantilever launching and either horizontal or vertical "swing", whereby each half-arch is rotated into position. The steel tubular structure is itself stable so that there is no need for formwork nor for a temporary bridge structure during concrete casting.

Figure 1-13 A concrete filled tubular arch bridge under construction (photo courtesy of Prof. L H Han, Tsinghua University, China)

2 AMBIENT TEMPERATURE DESIGN

2.1 General

In general, a composite column must be designed for the ultimate limit state. For structural adequacy, the internal forces and moments resulting from the most unfavourable load combination should not exceed the design resistances of the composite cross-sections. While local buckling of the steel sections may be eliminated, the reduction in the compression resistance of the composite column due to overall buckling should be taken into account, together with the effects of residual stresses and initial imperfections. Moreover, the second order effects in slender columns, as well as the effect of creep and shrinkage of concrete under long-term loading, must be considered, if they are significant. The reduction of flexural stiffness due to cracking of the concrete in the tension area should also be considered. These are provided for either explicitly, or empirically, in BS EN 1994-1-1 [12].

2.2 Material properties

2.2.1 Structural steel

Hot-finished steel

Tata Steel Celsius®355 is the TATA Steel brand name for hot-finished fine grained structural hollow section, Celsius®355NH being produced to the European Standard, BS EN 10210[4,5]. S355NH. Celsius®355 sections are produced by the electric weld process and the NH denotation signifies that they have a Charpy impact minimum average energy value of 40J at –20°C and are delivered in the normalised condition, This is essential to ensure good weldability/formability and adequate ductility for the most demanding applications. Celsius®355NH sections are produced to the technical delivery requirements of BS EN 10210-1[4] with dimensions and tolerances to BS EN 10210-2[5]; however, Celsius®355NH sections have an improved corner profile of 2*T* maximum compared to the maximum *3T* allowable in the standard.

Nominal values of the yield stress f_y, and the ultimate tensile stress f_u, for structural steel are presented in Table 2-1 below.

Table 2-1 Mechanical properties of Celsius Sections

Steel grade		Minimum yield strength R_{eH} MPa			Tensile strength R_m MPa at specified thickness ≤ 65 mm
		Specified thickness (mm)			
Steel name	Steel number	≤16	>16 ≤40	>40 ≤65	
S355NH	1.0539	355	345	335	470

Reproduced from Table B.3 BS EN 10210-1[4]

Other design values for the steel sections are given as follows:

E_a = Modulus of elasticity = 210 000 N/mm²

G_a	=	Shear modulus	=	$\dfrac{E_a}{2(1+\nu_a)}$
ν_a	=	Poisson's ratio	=	0.3
ρ_a	=	Density	=	7850 kg/m³

The material, section properties and assumed design imperfections, used in both this design guide and in FireSoft software, are based on hot finished hollow sections; the use of cold-formed sections may give the same mechanical properties but they will have a reduced design capability. For the differences between hot finished and cold formed hollow sections, see Tata Steel Europe Tubes guidance[15].

2.2.2 Structural concrete

Normal strength concrete

Concrete strengths are based on the characteristic cylinder strengths f_{ck} measured at 28 days in accordance with Table 3.1, of BS EN 1992-1-1[8]. The different strength classes, and the associated cube strengths, given by this Eurocode are presented in Table 2-2 below. Classification grades of concrete, such as C20/25, refer to the cylinder/cube strength at the specified age.

For normal weight concrete, the mean tensile strength f_{ctm} and the secant modulus of elasticity E_{cm}, for short-term loading are also given in Table 2-2. The effect of creep and shrinkage of concrete may be significant under long-term loading in some cases. As will be discussed in Section 2.6.2, provision is given within BS EN 1994-1-1[12] to reduce the secant modulus of elasticity, depending on the proportion of permanent load acting on the column.

The density of structural concrete is assumed to be 2400 kg/m³ for plain, unreinforced, concrete and 2500 kg/m³ for reinforced concrete.

Table 2-2 Characteristic compressive strength f_{ck} (cylinders), mean tensile strength f_{ctm} and secant modulus of elasticity E_{cm} for structural concrete

Strength class of concrete	C20/25	C25/30	C30/37	C35/45	C40/50	C45/55	C50/60
f_{ck} (N/mm²)	20	25	30	35	40	45	50
f_{ctm} (N/mm²)	2.2	2.6	2.9	3.2	3.5	3.8	4.1
E_{cm} (N/mm²)	30000	31000	33000	34000	35000	36000	37000

High strength concrete

In BS EN 1994-1-1[12], the highest concrete grade allowed in concrete filled steel tubular columns is C50/60. However, it is possible to use higher strength concrete (up to f_{ck}=100N/mm^2) to improve the compressive strength and fire resistance of concrete filled sections. Two types of high strength concrete (HSC) may be considered: plain HSC and steel fibre reinforced HSC. At elevated temperatures, because plain HSC suffers much greater reductions in strength, modulus of elasticity as well as ductility compared with normal strength concrete (NSC), the improvement in fire resistance from filling steel sections with plain HSC over that from NSC, is small. Therefore, it is not recommended to use plain HSC filling to increase fire resistance of CFT columns. In contrast, since steel fibre reinforced HSC possesses similar retentions in strength and modulus of elasticity as well as similar ductility as NSC at elevated temperatures, the use of steel fibre reinforced HSC instead of NSC can give considerable improvement in fire resistance of CFT columns; however, because there is no established design standard that gives information on strength and modulus of elasticity of steel fibre reinforced HSC at elevated temperatures, this design guide does not cover steel fibre reinforced HSC.

2.2.3 Steel reinforcement bars

In the UK, steel bars for reinforcement of concrete should conform to BS 4449+A2[2] or BS EN 10080[3]. Both standards state a characteristic yield strength of 500 N/mm^2. Reinforcement used should have ductility class of either 500B or 500C[8].

2.3 Partial safety factors

National authorities are free to select appropriate values for partial safety factors for loads and materials, and substitute them for 'boxed' values in the Eurocodes. The boxed values and the UK National Annex (NA) values are (Table 2-3):

Table 2-3 Partial safety factors

Loads:	**BS EN 1994-1-1[12] 'boxed' values**	**UK NA**
Imposed (variable) load, γ_Q	1.50	**1.50**
Dead (permanent) load, γ_G	1.35	**1.35**
Materials:		
Steel, γ_M	1.10	**1.0**
Concrete, γ_c	1.50	**1.50**
Reinforcement, γ_s	1.15	**1.15**

2.4 Basis of design method

This guide is limited to the design of isolated straight members under compression or combined compression and bending. Therefore, it is assumed that any global second-order effect has been considered when obtaining the design bending moments at the ends of the member. For members in braced non-sway frames with simple construction, the end bending moments are generated by the unbalanced reaction forces from the connected beams.

Two methods of design are given within BS EN 1994-1-1[12]: general design method and simplified design method.

2.4.1 General design method

The general design method can be used for any composite column. It should take account of second-order effects including residual stresses, geometrical imperfections, geometrical and material non-linearity, local instability, cracking of concrete, creep and shrinkage of concrete and yielding of structural steel and of reinforcement.

In order to allow for these design considerations, it is necessary to use sophisticated computer software and the design effort is considerable. Thus, the general design method is not preferred for use in practical design, and is outside the scope of this publication.

This design guide presents the simplified design method.

2.4.2 Simplified design method

Section 2.5 lists the limits of applicability of the simplified methods, as given in BS EN 1994-1-1[12]. When using the simplified design method, it is necessary to distinguish between whether the composite column is under compression only, or under combined compression and bending. In both cases, the method in section 2.7 can be used, in which the effect of member imperfection is converted into a design bending moment within the member. However, for columns under compression only, it is much quicker using the column buckling resistance method, as presented in Section 2.6.3.

In all cases, it is necessary to calculate the plastic (squash) resistance (section 2.6.1, including the effects of local buckling) and flexural stiffness (section 2.6.2) of the composite cross-section.

2.5 Restrictions on the simplified design method

The application of the simplified design method is subject to various restrictions, as follows:

(a) It is only applicable to simple construction.

(b) The column is doubly-symmetrical and is of uniform cross-section over the height of the column.

(c) The steel contribution ratio δ must satisfy the following conditions:

$$0.2 \leq \delta \leq 0.9$$

If δ is less than 0.2, the column may be designed according to BS EN 1992-1-1[8]. If δ is larger than 0.9, the concrete is ignored in the calculations, and the column is designed as a bare steel section according to BS EN 1993-1-1[10].

(d) The maximum non-dimensional slenderness ratio of the composite column $\bar{\lambda}$ is limited to 2.0.

(e) The maximum amount of longitudinal reinforcement that can be considered in the analysis is 6% of the concrete area.

2.6 Compression resistance

2.6.1 Squash (plastic) resistance, $N_{pl,Rd}$

The plastic resistance, to compression, of a composite cross-section represents the maximum compression that can be applied to a short composite column. It is important to recognize that stocky concrete filled circular hollow (CHS) sections exhibit enhanced resistance due to the triaxial containment effects. Concrete filled rectangular and elliptical sections (RHS and EHS) do not achieve such enhancement.

Local buckling of steel hollow sections

Before the plastic resistance of the concrete filled hollow section is calculated, the designer should ensure that local buckling of the steel does not occur. To prevent premature local buckling, the section should not be classified as "slender" in compression by checking the following width to thickness ratio:

For concrete filled rectangular or square hollow sections (RHS/SHS) $\quad \dfrac{h}{t} \leq 52\varepsilon$

For concrete filled circular hollow sections (CHS) $\frac{d}{t} \leq 90\varepsilon^2$

where:

t is the wall thickness of the steel hollow section in mm.

h is the larger outer dimension of the rectangular hollow section in mm.

d is the outer diameter of the circular hollow section in mm.

$\varepsilon = \sqrt{\frac{235}{f_y}}$

f_y is the nominal yield strength of the steel section in N/mm².

For concrete filled elliptical hollow sections (EHS), the same equation above for CHS sections can be used, provided the diameter "d" of a CHS is replaced by an effective diameter "d_e" for an EHS, given by: $d_e = 2a[1 + f(a/b - 1)]$, in which $f = 1 - 2.3(t/2a)^{0.6}$ where "2a" and "2b" are respectively the overall longer and shorter dimensions of the EHS and "t" is the EHS thickness. For Tata Steel Celsius® 355 EHS, a/b=2, giving $d_e = 2a(1 + f)$.

Concrete filled elliptical and rectangular hollow sections (EHS and RHS)

The plastic resistance of a concrete filled elliptical or rectangular hollow section (i.e., the so-called "squash load") is given by the sum of the resistances of the components as follows:

$$N_{pl,Rd} = \frac{A_a f_y}{\gamma_M} + \frac{A_s f_{sk}}{\gamma_s} + \frac{A_c f_{ck}}{\gamma_c} \qquad \textbf{2-1}$$

where:

A_a is the area of the steel section.

A_s is the area of the reinforcement.

A_c is the area of the concrete.

f_y is the yield strength of the steel section.

f_{sk} is the characteristic yield strength of the steel reinforcement bars.

f_{ck} is the characteristic compressive strength (cylinder) of the concrete.

To aid calculations of A_a and A_c for an elliptical hollow section (EHS), the cross-sectional area of a solid elliptic with overall dimensions of "2a" and "2b" is πab.

For ease of expression, $\frac{f_y}{\gamma_M}$, $\frac{f_{sk}}{\gamma_s}$ and $\frac{f_{ck}}{\gamma_c}$ are presented as design strengths of the respective materials in the remainder of this guide as: f_{yd}, f_{sd} and f_{cd} respectively. As a result of this simplification, the above equation for the plastic resistance of the composite column, can be rewritten in the following compact form:

$$N_{pl,Rd} = A_a f_{yd} + A_s f_{sd} + A_c f_{cd} \qquad \textbf{2-2}$$

Concrete filled circular hollow sections (CHS)

For composite columns with concrete filled circular hollow sections, the increased resistance of concrete due to the confining effect of the circular hollow section may be included. This restraint to transverse strain in a three dimensional confinement results in increased concrete resistance. At the same time, circular tensile stresses in the circular hollow section also arise, which reduce its axial resistance.

In general, the resistance of a concrete filled circular hollow section to compression may increase by up to 15% under simple axial loads when the effect of tri-axial confinement is considered. However, this effect on the resistance enhancement of concrete depends also on the slenderness of the composite columns and is significant only in stocky columns. For composite columns with a non-dimensional slenderness of $\bar{\lambda} > 0.5$ (where $\bar{\lambda}$ is defined in Section 2.6.3), this effect should be neglected and the plastic resistance assessed as for rectangular hollow sections.

In addition, further linear interpolation is necessary to take account of any effective load eccentricities. However, the eccentricity, *e* of the applied load may not exceed the value *d*/10, where *d* is the outer diameter of the circular hollow section.

The eccentricity, *e* is defined as follows:

$$e = \frac{M_{Ed}}{N_{Ed}}$$

where:

M_{Ed} is the maximum <u>design</u> moment (second order effects are ignored).

N_{Ed} is the <u>design</u> applied load.

The plastic resistance of a concrete filled circular hollow section may be obtained as follows:

$$N_{pl,Rd} = A_a \eta_a f_{yd} + A_s f_{sd} + A_c f_{cd}\left[1 + \eta_c \frac{t}{d}\frac{f_y}{f_{ck}}\right] \qquad \textbf{2-3}$$

where:

t is the wall thickness of the steel hollow section in mm.

$$\left.\begin{aligned} \eta_c &= \eta_{c0}\left(1-\frac{10e}{d}\right) \\ \eta_a &= \eta_{a0} + (1-\eta_{a0})\frac{10e}{d} \end{aligned}\right\} \text{ for } 0< e \le d/10$$

$$\left.\begin{aligned} \eta_c &= 0 \\ \eta_a &= 1.0 \end{aligned}\right\} \text{ for } e > d/10$$

The basic values η_{c0} and η_{a0} depend on the non-dimensional slenderness ratio $\bar{\lambda}$ (to be defined in section 2.6.3), and are defined as follows:

$$\eta_{c0} = 4.9 - 18.5\bar{\lambda} + 17\bar{\lambda}^2 \qquad \text{but } \eta_{c0} \ge 0$$

$$\eta_{a0} = 0.25\left(3 + 2\bar{\lambda}\right) \qquad \text{but } \eta_{a0} \le 1.0$$

If the eccentricity e exceeds the value $d/10$, or if the non-dimensional slenderness ratio $\bar{\lambda}$ exceeds the value 0.5, then $\eta_{c0} = 0$ and $\eta_{a0} = 1.0$. Table 2-4 gives the basic values η_{10} and η_{a0} for different values of $\bar{\lambda}$.

Table 2-4: Basic values of η_{c0} and η_{a0} to allow for the effect of triaxial confinement in concrete filled circular hollow sections

Non-dimensional slenderness ratio $\bar{\lambda}$	**0**	**0.1**	**0.2**	**0.3**	**0.4**	**≥ 0.5**
η_{c0}	4.90	3.22	1.88	0.88	0.22	0
η_{a0}	0.75	0.80	0.85	0.90	0.95	1.00

2.6.2 Effective flexural stiffness

Short-term loading

The effective flexural stiffness of the composite column $(EI)_{eff}$ is obtained from adding up the flexural stiffness of the individual components of the cross-section:

$$(EI)_{eff} = E_a I_a + E_s I_s + 0.6 E_{cm} I_c \qquad \textbf{2-4}$$

where:

I_a, I_s and I_c are the second moment of area, about the appropriate axis of bending, for the steel section, the reinforcement and the concrete (assumed uncracked) respectively. To aid calculations for an elliptical section, the second moment of area about the longer axis of a solid elliptical with overall longer and shorter dimensions of “2a” and “2b” is $\frac{\pi ab^3}{4}$.

E_a and E_s are the elastic moduli for the structural steel and the reinforcement respectively.

$0.6E_{cm}I_c$ is the effective stiffness of the concrete component (the 0.6 =0.8/1.35) factor is an empirical multiplier, which has been determined from a calibration exercise, to give good agreement with test results.

E_{cm} is the secant modulus of elasticity for structural concrete; see Table 2-2.

Long-term loading

For composite columns under long-term loading, the creep and shrinkage of concrete will cause a reduction in the effective elastic stiffness of the composite column, thereby reducing the buckling resistance. For exposed concrete, the modulus of elasticity of concrete E_{cm} should be reduced to the value $E_{c,eff}$ given below:

$$E_{c,eff} = E_{cm} \frac{1}{1 + \left(N_{G,Ed} / N_{Ed}\right)\varphi(\infty, t_0)} \qquad \textbf{2-5}$$

Where

$\varphi(\infty, t_0)$ is the creep coefficient, determined according to Figure 3.1 of BS EN 1992-1-1[8] (reproduced as Figure 2-1 below), in which t_0 is the time (in days) of first loading on concrete.

N_{Ed} is the total design axial load.

$N_{G,Ed}$ is the part of the design axial load permanently acting on the column.

The concrete in a filled section is sealed and there is no loss of moisture. Therefore, the effect of creep and shrinkage of the concrete in a concrete filled section is less than that of exposed concrete. A method to take advantage of this beneficial effect is to set the value of h_0 to be the maximum when using Figure 2-1 (Figure 3.1 of BS EN 1992-1-1[8]).

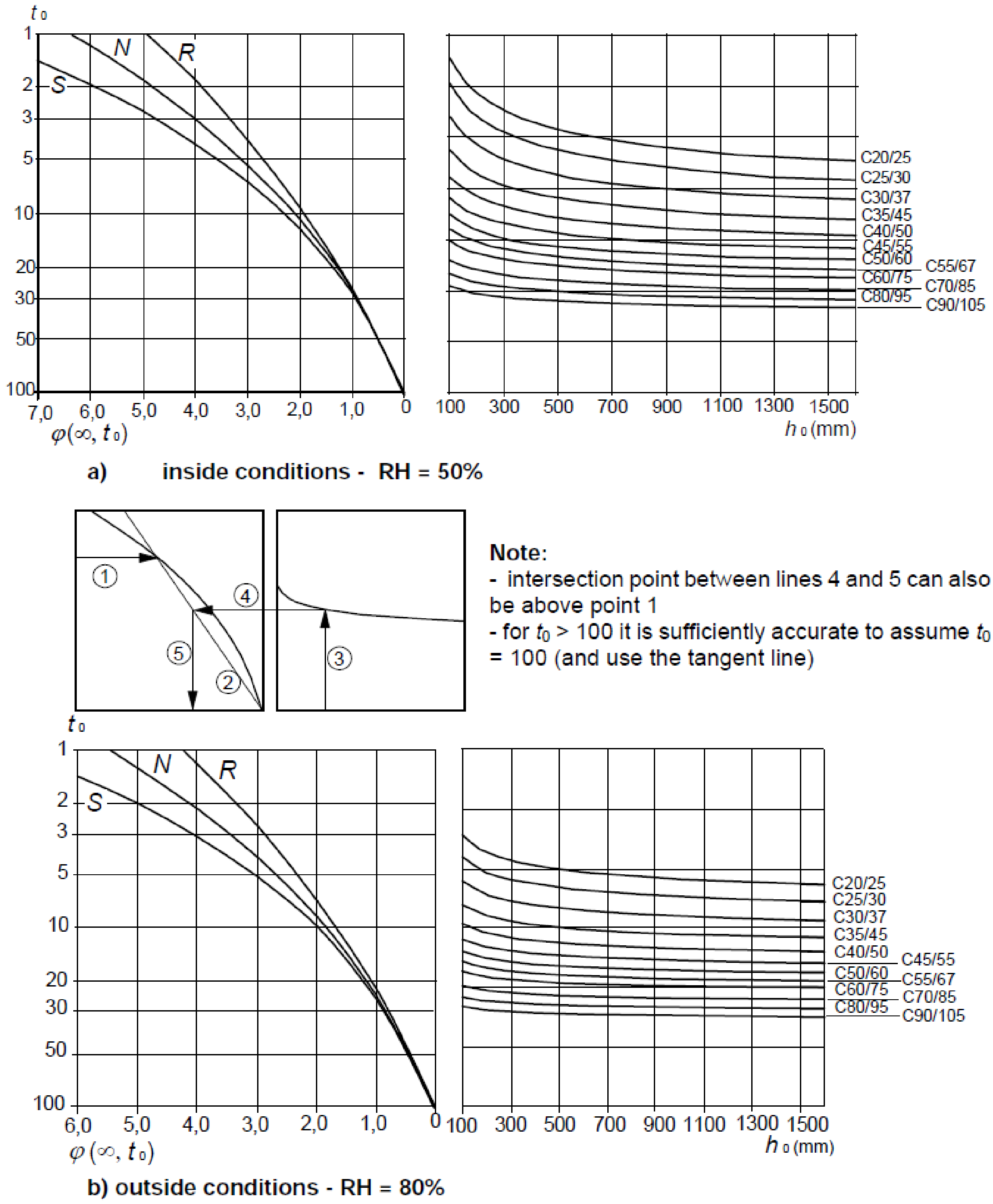


Figure 2-1 Method for determining the creep coefficient $\varphi(\infty, t_0)$ for concrete

2.6.3 Column buckling resistance under compression

The plastic resistance to compression of a composite cross-section $N_{\mathrm{pl,Rd}}$ represents the maximum load that can be applied to a short column. However, for slender columns, with low elastic critical load, overall buckling considerations may be more significant.

The resistance of a composite column in axial compression (buckling load) is obtained from:

$$N_{Rd} = \chi . N_{pl,Rd} \qquad \textbf{2-6}$$

where:

χ is the reduction coefficient for buckling, and is dependant on the non-dimensional slenderness ratio $\bar{\lambda}$ (defined in equation 2-8).

The reduction factor may be determined from:

$$\chi = \frac{1}{\phi + \left[\phi^2 - \bar{\lambda}^2\right]^{0.5}} \qquad \text{but } \chi \le 1.0 \qquad \textbf{2-7}$$

where:

$$\phi = 0.5\left[1 + \alpha\left(\bar{\lambda} - 0.2\right) + \bar{\lambda}^2\right]$$

α is an imperfection parameter depending on the buckling curve considered (section 2.6.4).

The non-dimensional slenderness ratio is given by:

$$\bar{\lambda} = \sqrt{\frac{N_{pl,R}}{N_{cr}}} \qquad \textbf{2-8}$$

where:

$N_{pl,R}$ is the plastic resistance of the composite cross-section to compression, according to equation 2.1, with $\gamma_M = \gamma_s = \gamma_c = 1.0$.

N_{cr} is the Euler buckling load, given by:

$$N_{cr} = \frac{\pi^2 (EI)_{eff}}{l^2} \qquad \textbf{2-9}$$

in which:

$(EI)_{eff}$ is the effective elastic flexural stiffness of the composite column (see Section 2.6.2).

l is the buckling length of the column.

The buckling length l of an isolated non-sway composite column in simple construction may conservatively be taken as equal to its system length L.

2.6.4 Relevant Buckling Curves and Imperfection Factors

The relevant buckling curve is selected according to the following guidance:

- For reinforcement ratio not more than 3%: curve “a”, α=0.21;
- For reinforcement ratio greater than 3%, for tube in tube arrangement (Figure 2-2 (b)) and for open section in tube arrangement (Figure 2-2 (c)): curve “b”, α=0.34;

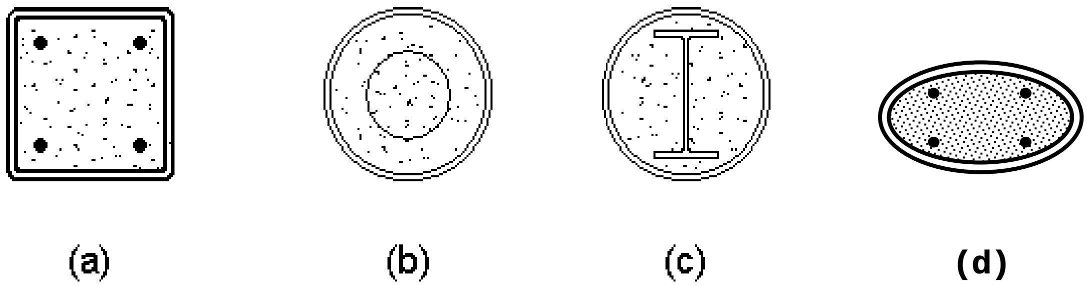


Figure 2-2 Typical column cross-sections

2.7 Combined compression and bending

In BS EN 1994-1-1[12], the resistance of a composite column subjected to combined compression and bending is determined from the bending moment – axial force interaction curve of the cross-section of the column. The bending moments for this design check should include both the externally applied bending moments and the bending moments due to geometrical imperfection of the column (second-order effects). The second-order effects within the column length should be accounted for as explained in section 2.7.3.

2.7.1 Interaction curve of composite cross-section

The bending moment and compressive axial load interaction curve of a composite cross-section can be obtained by considering several possible positions of the plastic neutral axis within the cross-section, and determining the internal forces and moments from the resulting plastic stress blocks. For the simplified method given within BS EN 1994-1-1[12], sufficient accuracy in estimating the effects of combined compression and bending may be found by constructing the interaction curve, shown in Figure 2-3, from 4 or 5 points.

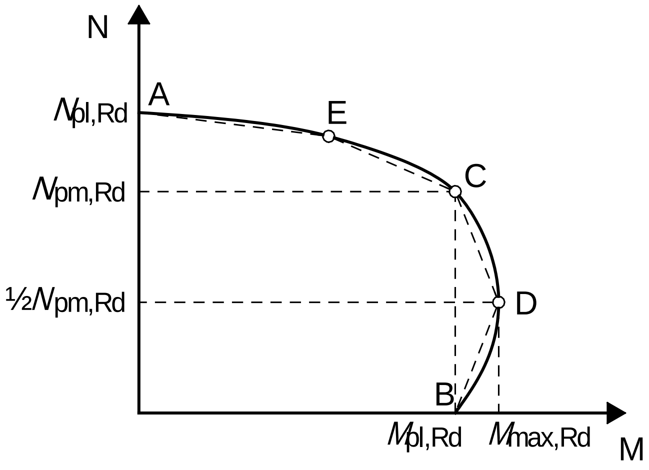


Figure 2-3 Composite cross-section interaction curve with linear approximation

When constructing the interaction curve, the bending moment is taken about the centre of the cross-section. Figure 2-4 shows the distribution of stresses for calculating the key points indicated in Figure 2-3. The values of the key points in Figure 2-3 are as follows:

- Point A corresponds to pure compression in the cross-section so the plastic neutral axis is at an edge of the cross-section. For point A:

$$N_A = N_{pl,Rd}$$
$$M_A = 0$$

- Point B corresponds to pure bending in the cross-section:

$$N_B = 0$$
$$M_B = M_{pl,Rd}$$

- At point C, the plastic neutral axis is symmetrical to that of point B (pure bending) with regard to the centre of the cross-section. The axial compression and moment resistance of the composite column are given as:

$$N_C = A_c f_{cd}$$
$$M_C = M_{pl,Rd}$$

- At point D, the plastic neutral axis coincides with the centroid of the cross-section, and the resulting axial force is half of the value at point C, i.e.:

$$N_D = N_c / 2$$
$$M_D = M_{max,Rd}$$

- Point E is approximately mid-way between points A and C, and is often required for highly non-linear interaction curves, in order to achieve a better approximation. For concrete filled structural hollow sections, the use of point E will yield a more economical design; however, much more calculation effort is required. Thus, to retain simplicity, point E tends not to be used.

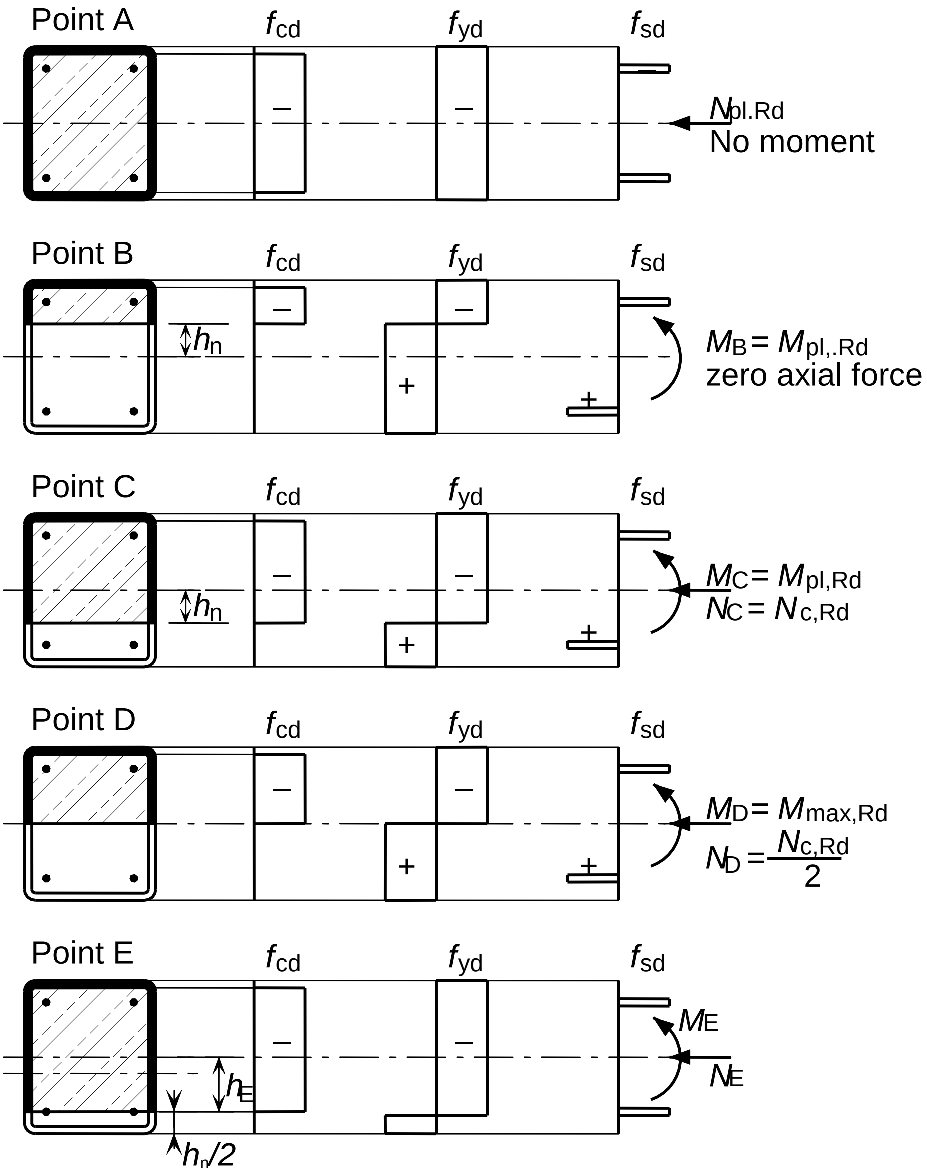


Figure 2-4 Stress distributions for the points on the interaction curve for concrete filled hollow sections, according to BS EN 1994-1-1[12]

Interested readers may wish to consult reference[19] for proof of the axial load and bending moment values for points C and D.

APPENDIX A describes how to calculate $M_{pl,Rd}$ and $M_{max,Rd}$.

2.7.2 Design checks

All the following conditions should be satisfied (Figure 2-5c):

$$\frac{M_{\mathrm{y},E\mathrm{d}}}{\mu_{d\,\mathrm{y}} M_{\mathrm{pl,y,Rd}}} \leq \alpha_{M,y} \tag{2-10}$$

$$\frac{M_{\mathrm{z},E\mathrm{d}}}{\mu_{d\,\mathrm{z}} M_{\mathrm{pl,z,Rd}}} \leq \alpha_{M,z} \tag{2-11}$$

and $$\frac{M_{\mathrm{y},E\mathrm{d}}}{\mu_{d\,\mathrm{y}} M_{\mathrm{pl,y,Rd}}} + \frac{M_{\mathrm{z},E\mathrm{d}}}{\mu_{d\,\mathrm{z}} M_{\mathrm{pl,z,Rd}}} \leq 1.0 \tag{2-12}$$

where:

y,z are the axes of the composite cross-section;

$M_{E\mathrm{d}}$ is the design bending moment, including the bending moment due to geometrical imperfection in the column, and which may be factored to allow for second-order effects, if necessary. This is discussed in section 2.7.3.

μ_{d} is the moment resistance ratio obtained from the interaction curve, as shown in Figure 2-5(a) and Figure 2-5(b).

$M_{\mathrm{pl,Rd}}$ is the plastic moment resistance of the composite cross section.

α_M is a calibration coefficient, taken as 0.9 for steel grades S235 to S355 inclusive, and 0.8 for steel grades S420 and S460.

It is necessary to consider the effect of geometric imperfections only in the critical plane of the column buckling. Therefore, the moment resistance ratio, μ, in the other plane is evaluated without the consideration of imperfections. In general, it will usually be obvious which of the axes is more likely to fail and the imperfections need to be considered for this direction only. If it is not obvious which plane is more critical, checks should be made on both planes.

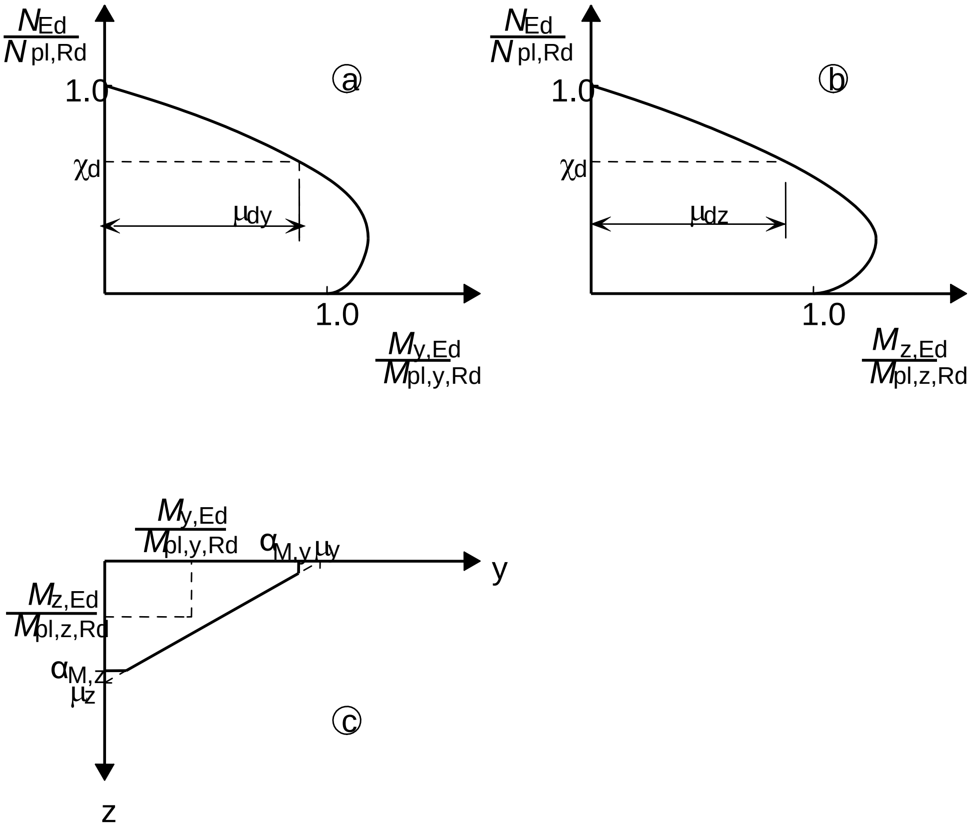


Figure 2-5 Verification for combined compression and bi-axial bending

2.7.3 Design bending moments due to second-order effects

When a column is under combined compression and bending, the design bending moment comprises of three parts as shown in Figure 2-6: the externally applied bending moments (Figure 2-6b), the 1st order bending moment arising from member imperfections (Figure 2-6c) and the bending moment from second-order (P-δ) effect (Figure 2-6d).

The 1st order bending moment from member imperfection is calculated by multiplying the design axial compression load by the member imperfection. The member imperfection is related to the buckling curve (section 2.6.4), being L/300 and L/200 for buckling curve "a" and "b" respectively, where L is the column length. Member imperfection is applied only in the plane of buckling.

The design bending moment is the higher of the maximum end bending moment (Figure 2-6b) and the bending moment within the column length. To obtain the maximum bending moment within the column length, the second-order effect should be applied to all sources of the 1st order bending moments by multiplying the maximum 1st order bending moment by a factor k given by:

$$k = \frac{\beta}{1 - \frac{N_{Ed}}{N_{cr,eff}}} \qquad \textbf{2-13}$$

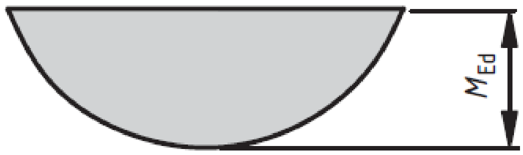
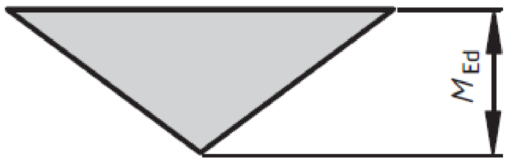
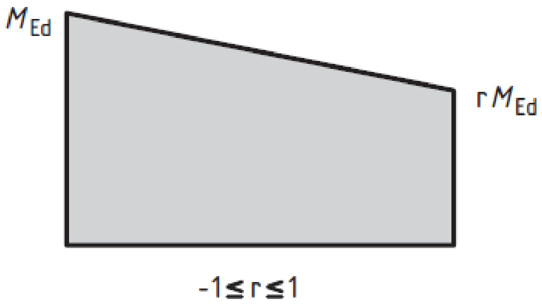
where:

N_{Ed} is the total design axial compression load;

$N_{cr,eff}$ is the elastic critical load of the composite column based on the column length, L;

β is the equivalent moment factor, given in Table 2-5.

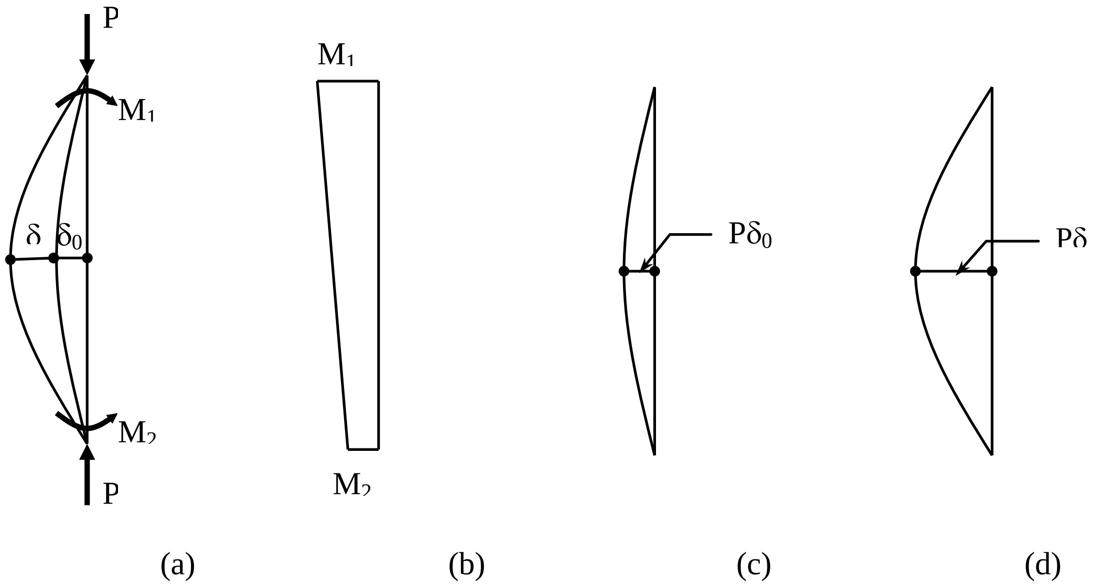
Table 2-5 Factors β for the determination of moments to second order theory (from BS EN 1994-1-1[12])

Moment distribution	Moment factors β	Comment
 	First-order bending moments from member imperfection of lateral load: $\beta = 1.0$	M_{Ed} is the maximum bending moment within the column length ignoring second-order effects
	End moments: $\beta = 0.66 + 0.44\gamma$ but $\beta \geq 0.44$	M_{Ed} and γM_{Ed} are the end moments from first-order or second-order global analysis

The elastic critical load $N_{cr,eff}$ is calculated by using the effective flexural stiffness of the cross-section:

$$(EI)_{eff,II} = 0.9\left(E_a I_a + E_s I_s + 0.5 E_{cm} I_c\right) \qquad \textbf{2-14}$$

The value of E_{cm} should be replaced by that of $E_{c,eff}$ (Eqn. 2.5) if the effect of long-term loading is considered.



(a): Column loads and deflections

(b): Primary bending moment distribution

(c): Bending moment distribution from initial deflections

(d): Bending moment distribution from displacements

Figure 2-6 Bending moments in a column

2.7.4 Other design considerations

Curved columns

Curved concrete filled tubular structures can be used in bridge construction. The SCI publication P281[17] gives comprehensive design guidance on design of curved steel structures. The design recommendations may be extended to concrete filled tubular structures. In the SCI design guide on curved steel structures, the steel design strength is reduced to take into account out-of-plane bending of the flanges of open sections or plates of the tube sections. However, in concrete filled tubular structures, since the steel tube is in contact with the concrete infill, the effect of out-of-plane bending of the steel tube may be neglected.

Seismic performance

This design guide gives information for designing concrete filled tubular columns under static loading and fire. Seismic performance of CFT columns is outside the scope of this design guide. Nevertheless, it is worth pointing out that when a CFT column is under cyclic loading, the circumferential deformation and ovalization of the steel tube are prevented by the concrete infill and the concrete core is confined by the steel tube, the results of which are high stiffness, high strength, high ductility and high energy absorption under cyclic loading. The hysteresis curves of CFT columns are similar to those of steel columns without local failure. Therefore, CFT columns are particularly suited to application in seismic areas.

2.8 Longitudinal and transverse shear

In general, the applied internal forces and moments from a member connected to the ends of a composite column are distributed between the steel section and the concrete. BS EN 1994-1-1[12] requires that adequate provision should be made for the distribution of these internal forces and moments.

For concrete filled steel structural hollow sections, the shear resistance between the steel section and the concrete is achieved by both chemical bond and friction at the interface. In these circumstances, the design shear resistance, developing at the interface between the concrete and the inner wall of the steel section, is limited to: 0.40 N/mm² for a square or rectangular hollow section (RHS); and 0.55 N/mm² for a circular hollow section (CHS).

Similarly, the design transverse shear forces may be assumed to act on the steel section alone or to be shared between the steel section and the concrete. For the latter case, the shear force to be resisted by the concrete must be assessed in accordance with BS EN 1992-1-1[8], whereas the shear force to be resisted by the steel section may be checked according to von Mises yield criterion. However, it is simpler in design to assume that the whole of the transverse shear force acts on the steel alone. Figure 2-7 indicates the reduction in the design strength of the shear area (web) that will occur within a steel section subjected to transverse shear stress.

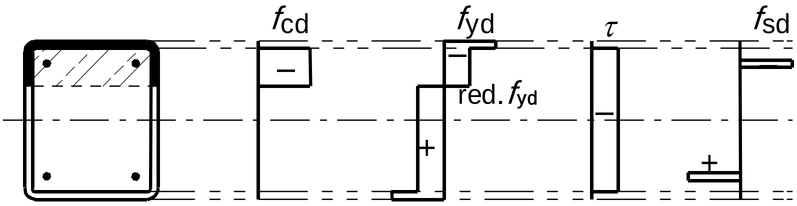


Figure 2-7 Reduction of design strength of steel within shear area in the presence of transverse shear stress

For design purposes, any reduction in the design steel strength in the shear area of the steel section may be transformed into a reduction in steel thickness. For a steel section under major axis bending, the effective wall thickness of the 'web' $t_{w,d}$ in the presence of transverse shear may be evaluated as follows:

$$t_{w,d} = t_w \left[1 - \left(\frac{2V_{a,Sd}}{V_{pl,a,Rd}} - 1 \right)^2 \right] \quad \textbf{2-15}$$

where:

$V_{a,Sd}$ is the design shear force to be resisted by the cross-section

$V_{pl,a,Rd}$ is the plastic resistance of the steel cross-section in shear = $A_v \dfrac{f_{yd}}{\sqrt{3}}$

A_v is the shear area of the steel section.

For rectangular hollow sections of uniform thickness:

Load parallel to depth, h, $A_v = Ah/(b+h)$

Load parallel to breadth, b, $A_v = Ab/(b+h)$

For circular hollow sections of uniform thickness, $A_v = 2A/\pi$

For elliptical hollow sections of uniform thickness t:

Load parallel to the longer axis, 2a, $A_v = (2a-t)t$

Load parallel to the shorter axis, 2b, $A_v = (2b-t)t$

However, no reduction in the web thickness is necessary when:

$$V_{a,Sd} < 0.5V_{pl,a,Rd} \quad \textbf{2-16}$$

Using the effective wall thickness of the 'web' $t_{w,d}$ of the steel hollow section, the moment resistance of the composite cross-section may be evaluated using the same set of expressions given within section 2.7.1: without any modification.

2.9 Load introduction from column top

Where a load is applied to a composite column from the top of the column, it must be ensured that it is distributed between the individual components of the cross-section in proportion to their design resistances. For composite columns using SHS, according to BS EN 1994-1-1[12], this may be achieved as follows:

(i) No mechanical shear connection needs to be provided for load introduction through a cap plate, at the top of a column, if the full interface between the concrete section and endplate is permanently in compression after due consideration of the effects of creep and shrinkage.

(ii) If the cross-section of a cap-plate is only partially loaded (see Figure 2-8), loads may be distributed with a ratio of 1:2.5, over the thickness of the end plate. The concrete stresses should be limited then in the area of the effective load introduction area for concrete filled hollow sections according to Figure 2-8 and Figure 2-9 below.

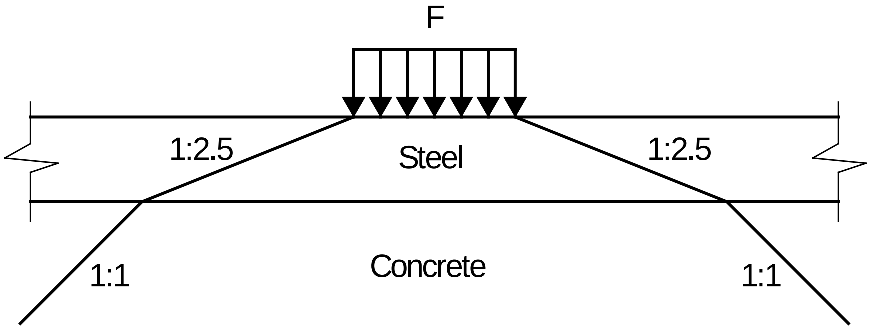


Figure 2-8 Load dispersion through a locally loaded cap plate

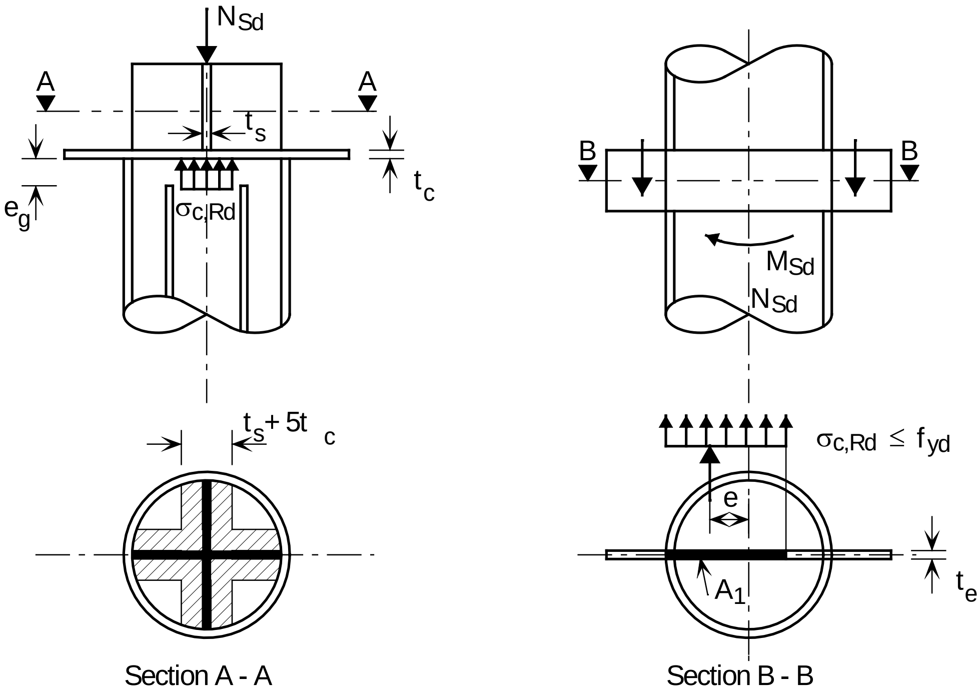


Figure 2-9 Load introduction into a concrete core through a gusset plate

(iii) When a concrete filled circular or rectangular square hollow section is only partially loaded by plate stiffeners at a cap column divider plate position (column section type A-A in Figure 2-9), or from a gusset plate through the profile at an intermediate column length position (section type B-B in Figure 2-9), the local design resistance strength of concrete $\sigma_{c,Rd}$ under the gusset plate or stiffener, resulting from the sectional forces of the concrete section, should be determined by:

$$\sigma_{\mathrm{c,Rd}} = f_{\mathrm{cd}} \left(1 + \eta_{\mathrm{cl}} \frac{t}{a} \frac{f_{\mathrm{y}}}{f_{\mathrm{ck}}} \right) \sqrt{\frac{A_{\mathrm{c}}}{A_{\mathrm{l}}}} \leq \frac{A_{\mathrm{c}} f_{\mathrm{cd}}}{A_{\mathrm{l}}} \leq f_{yd} \qquad \textbf{2-17}$$

where:

f_{cd} and f_{ck} are the design strength of the steel and the characteristic strength of the concrete respectively.

t is the wall thickness of the steel tube.

a is the diameter of the tube or the width of the rectangular section.

A_{c} is the cross-sectional area of the concrete.

A_{l} is the loaded area under the gusset plate according to Figure 2-9.

η_{cL} is 4.9 for circular steel tubes; and 3.5 for rectangular sections.

The ratio $A_{\mathrm{c}} / A_{\mathrm{l}}$ in the equation above should not exceed 20.

(iv) For concrete filled circular hollow sections, longitudinal reinforcement may be fully taken into account when assessing cross-sectional design parameters, even where the reinforcement is not welded to the end plates or in direct contact with the endplates, provided that the gap e_{g} between the reinforcement and the end plate does not exceed 30 mm (see also column section type A-A in Figure 2-9).

2.10 Load introduction at intermediate level

When loads are introduced at an intermediate position on an SHS length, the introduction length should be assumed not to exceed 2*d*, where *d* is the minimum transverse dimension in the case of concrete filled rectangular/elliptic hollow sections or the outside diameter of the column for circular hollow sections.

The design shear strength at the interface between the steel and concrete is 0.40 N/mm² for RHS/EHS or is 0.55 N/mm² for CHS. These design bond strength values were lower bound to test results using test specimens that had already experienced shrinkage before testing. The shear forces should be determined from the change of sectional forces of the steel or reinforced concrete section within the introduction length, where the sectional forces should be determined by plastic theory. At a beam connection position, it is necessary to check that:

For an RHS column: $(1-\delta)V_{Sd} / A_s$ < 0.40 N/mm² with $A_s = 2db$

For an EHS column: $(1-\delta)V_{Sd} / A_s$ < 0.40 N/mm² with $A_s = 2.422d^2$

For a CHS column: $(1-\delta)V_{Sd} / A_s$ < 0.55 N/mm² with $A_s = 2\pi d^2 / 4$

where:

V_{Sd} is the design shear load to be transferred to the column by a beam connection.

δ is the steel contribution ratio.

A_s is the usable shear area/connection at the concrete interface.

b is the breadth of RHS face at a shear connector.

d is the minimum dimension of an RHS or EHS or diameter of a CHS.

If load introduction would give rise to excessive interface shear stresses, then a through gusset plate (Figure 4-1) should be provided in the load introduction area, to enable the additional load to be introduced into the concrete core.

2.11 Design examples

Three examples are given below to illustrate how to use the design guide:

(1) to check resistance of a concrete filled RHS column in composite action;

(2) to check load introduction at intermediate level for the column in (1); and

(3) to calculate the cross-section capacities of a short concrete filled CHS column.

2.11.1 Design checks for a concrete filled RHS column (Figure 2-10)

Design data

*Tata Steel Celsius® 355NH rectangular cross-section: RHS 300*200*10mm*

Steel grade: S355

Concrete grade: C50/60

Steel reinforcement: 4H20, see Figure 2-10 *for arrangement;* f_s*=500N/mm*^{2}

Axial load: 2500kN

Eccentricity:

About the major axis (y-y): 50mm at both ends, double curvature bending

About the minor axis (z-z): 25mm at the top,0 at the bottom

Length L=4m about both the major and minor axes

Partial material safety factors: γ_M*=1.0,* γ_c*=1.5,* γ_s*=1.15*

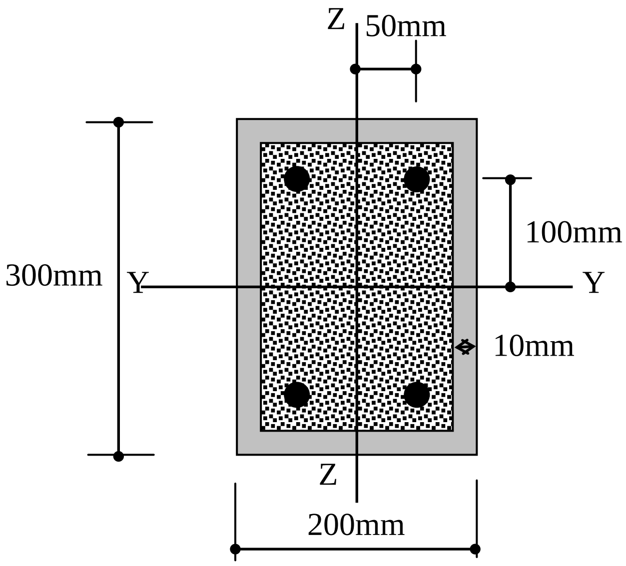


Figure 2-10 ***Dimension used in the example***

Calculation results

- *Material properties:*

f_{yd}*=355/1.0=355 N/mm*^{2}*,* f_{cd}*=50/1.5=33.33 N/mm*^{2}*,* f_{sd}*=500/1.15=434.78N/mm*^{2}

E_a*=*E_s*=210000 N/mm*^{2}*,* E_{cm}*=37000N/mm*^{2}

To consider creep and shrinkage (according to Figure 2-1):

Assume t_0*=30 days and inside conditions,* $\varphi(\infty, t_0)$*=1.25.*

Assume $N_{G,Ed}/N_{Ed}$*=0.5,*

$E_{c,eff} = 37000 * \frac{1}{1+0.5\times1.25}$ =22769N/mm^2 [Eqn. 6.41]

All clauses are according to BS EN 1994-1-1[12] unless stated.

- *Cross-sectional properties*

A_s*=1256.6 mm*^{2}, A_c*=49143.3mm*^{2}, A_a*=9600mm*^{2} *(neglecting the corner rounding off)*

$I_{s,y}$*=1256cm*^{4}, $I_{c,y}$*=31672cm*^{4}, $I_{a,y}$*=12072cm*^{4} *(neglecting the corner rounding off)*

$I_{s,z}$*=314cm*^{4}, $I_{c,z}$*=13294cm*^{4}, $I_{a,z}$*=6392cm*^{4} *(neglecting the corner rounding off)*

Check reinforcement ratio: $0<\rho_s=1256.6/49143.3 = 2.6\% < 6\%$ [Cl. 6.7.3.1.(3)]

- *Slenderness ratio*

$N_{pl,Rk}$*=1255.6*0.5+49143.3*0.05+9600*0.355=6493 kN* [Cl. 6.7.3.3(2)]

$(EI)_{e,y}$*=1256*2.1+0.6*31672*0.228+12072*2.10=32321 kN.m*^{2} [Eqn. 6.40]

$$N_{cr,y}=\frac{\pi^2 (EI)_{e,y}}{L^2}=\frac{\pi^2 * 32321}{4^2}=19936\ kN$$

$$\bar{\lambda}_y = \sqrt{6493/19936} = 0.57 < 2$$ [Eqn. 6.39]

$(EI)_{e,z}$*=314*2.1+0.6*13294*0.228+6392*2.10=15901 kN.m*^{2}

$$N_{cr,z}=\frac{\pi^2 (EI)_{e,z}}{L^2}=\frac{\pi^2 * 15901}{4^2}=9808\ kN$$

$$\bar{\lambda}_z = \sqrt{6493/9808} = 0.814 < 2$$

- *Squash load*

$N_{pl,Rd}$*=1255.6*0.43478+49143.3*0.03333+9600*0.355=5591.9 kN* [Eqn. 6.30]

Check steel contribution factor:

$0.2<\delta=9600*0.355/5591.9=0.61<0.9$ [Eqn. 6.38]

- *Bending moment capacity:*

$W_{ps,y}$*=125.66cm*^{3}, $W_{pc,y}$*=3402.34cm*^{3}, $W_{pa,y}$*=972cm*^{3} *(neglecting the corner rounding off)*

Assume $A_{sn,y}=0$*; equation A.6 gives* $h_{n,y}$*=41.95mm < 100mm, so assumption is correct.*

$W_{psn,y}=0$, $W_{pcn,y}$*=317.76cm*^{3} *(equation A.8 of this guide),* $W_{pan,y}$*=35.2 cm*^{3} *(equation A.7 of this guide)*

Equation A.1 gives $M_{pl,y,Rd}$*=438.6 kN.m*

Similarly:

$W_{ps,z}$=62.83 cm^3, $W_{pc,z}$=2205.17cm^3, $W_{pa,z}$=732cm^3 *(neglecting the corner rounding off).*

$h_{n,z}$=35.84mm.

$W_{psn,z}$=0, $W_{pcn,z}$=359.66cm^3 *(equation A.8)*, $W_{pan,z}$=25.69 cm^3 *(equation A.7 of this guide)*

$M_{pl,z,Rd}$= 308.8 kN.m

- *Key points on the interaction curve about the y-y (major) axis*

$N_{A,Rd}/N_{pl,Rd}$=1.0, $M_{A,y,Rd}$=0.0

$N_{B,Rd}$=0.0, $M_{B,y,Rd}$=1.0

$N_{C,y,Rd}$=49143.4*0.03333=1638kN, $N_{C,y,Rd}/N_{pl,Rd}$ =0.293, $M_{C,y,Rd}$=1.0

$N_{D,y,Rd}$=1638/2=819kN, $N_{D,y,Rd}/N_{pl,Rd}$ =0.146

$M_{D,y,Rd}$=972*0.355+0.5*3402.34*0.03333+125.66*0.43478=456.4 kN.m *(equation A.10 of this guide)*

$M_{D,y,Rd}/M_{pl,y,Rd}$=1.04

Figure 2-11(a) shows the non-dimensional interaction curve of the cross-section about the y-y (major) axis.

Similarly, the values of the key points on the non-dimensional interaction curve of the cross-section about the z-z (minor) axis (Figure 2-11(b)) are:

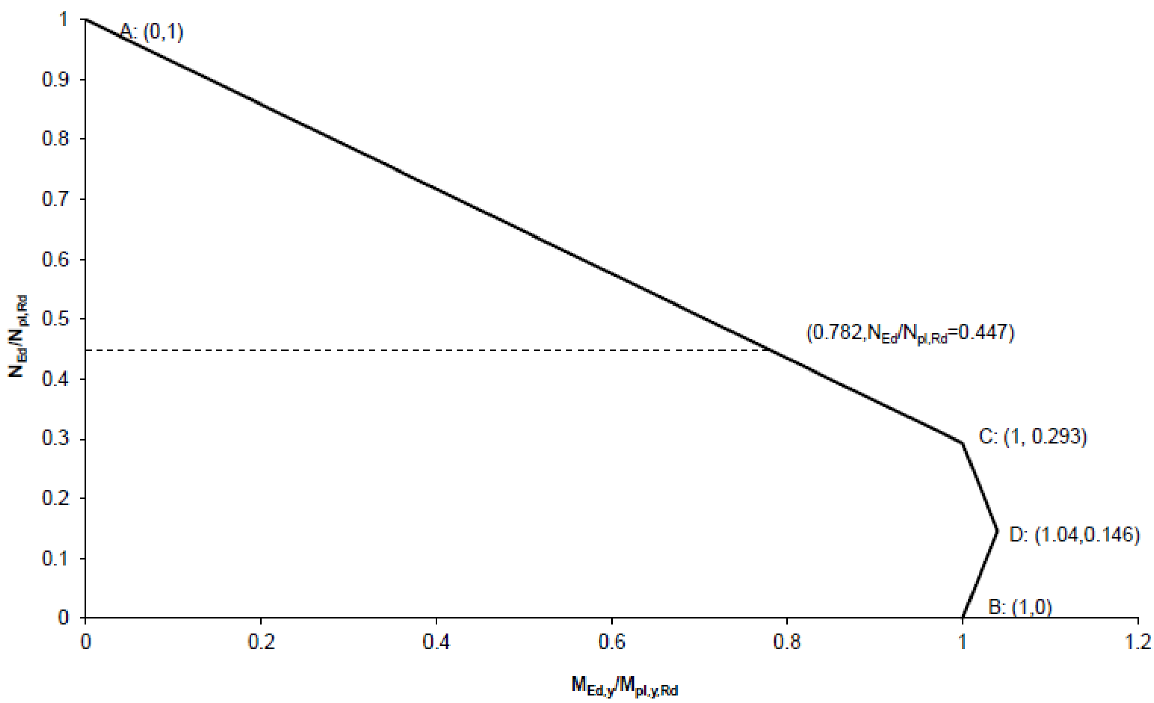
$M_{D,z,Rd}$=732*0.355+0.5*2205.17*0.03333+62.83*0.43478=323.9 kN.m

$N_{A,Rd}/N_{pl,Rd}$=1.0, $M_{A,z,Rd}/M_{pl,z,Rd}$=0.0

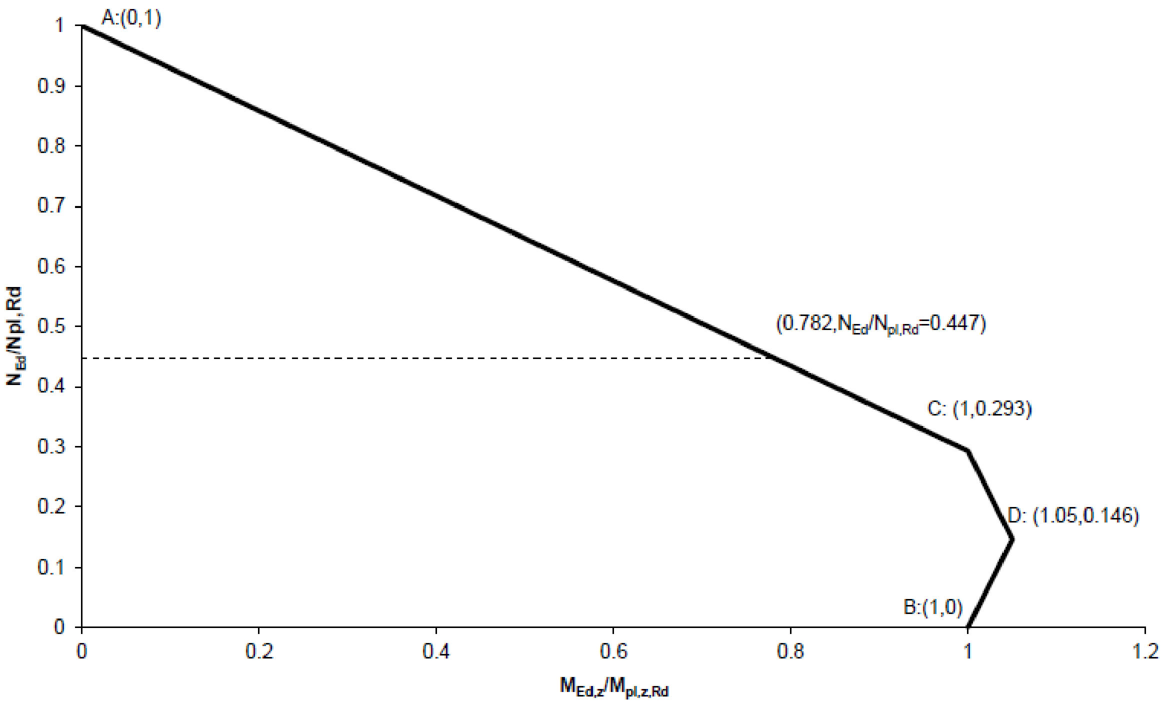
$N_{B,Rd}/N_{pl,Rd}$=0.0, $M_{B,z,Rd}/M_{pl,z,Rd}$ =1.0

$N_{C,Rd}/N_{pl,Rd}$=0.293, $M_{C,z,Rd}/M_{pl,z,Rd}$ =1.0

$N_{D,Rd}/N_{pl,Rd}$=0.146, $M_{D,z,Rd}/M_{pl,z,Rd}$ =1.05



(a) Interaction curve about y-y (major) axis



(b) Interaction curve about z-z (minor) axis

Figure 2-11 Axial load – bending moment interaction curves

- *Check column resistance*

Design axial load: N_{Sd}*=2500kN,* $N_{Ed}/N_{pl,Rd}$*=0.447*

From Figure 2-11(a): $\mu_{dy}=0.782$

From Figure 2-11(b): $\mu_{dz}=0.782$

To determine the design bending moment:

Assume buckling about the y-y (major) axis as critical

- *Bending moment about the y-y (major) axis:*

*Maximum bending moment at column end: 2500*0.05=125kN.m*

Within the column height:

For end bending moments: $\gamma=-1$, $\beta_y=0.66-0.44 = 0.22 < 0.44$, so $\beta_y=0.44$ [Table 6.4]

From equation 2-14:

$EI_{eff,y,II}=0.9*(1256*2.1+0.5*31672*0.228+12072*2.10)=28439\ kN.m^2$

$$N_{cr,eff,y}=\frac{\pi^2(EI)_{eff,y,II}}{L^2}=\frac{\pi^2*28439}{4^2}=17543kN$$

$$k_y=\frac{0.44}{1-2500/17543}=0.513\ \text{[Eqn. 6.43]}$$

For bending moment due to member imperfection:

$\rho_s=2.6\%<3\%$, *buckling curve "a", member imperfection = L/300 = 4000/300=13.3mm* [Table 6.5]

$$\beta_y=1.0,\ \text{so}\ k_y=\frac{1.0}{1-2500/17543}=1.166$$

*So the design moment is 125*0.513+1.166*2500*13.3/1000 = 102.9 kN.m*

This is less than the maximum end moment of 125kN.m, so

$M_{Ed,y}=125kN.m$

- *Bending moment about the z-z (minor) axis:*

Member imperfection is not considered.

From equation 2-14:

$(EI)_{eff,z,II}=0.9*(314*2.1+0.5*13294*0.228+6392*2.10)=14038\ kN.m^2$

$$N_{cr,z}=\frac{\pi^2(EI)_{eff,z,II}}{L^2}=\frac{\pi^2*14038}{4^2}=8660\ kN$$

β_z=0.66+0 = 0.66

$$k_z = \frac{0.66}{1-2500/8660} = 0.93 < 1$$

*So $M_{Ed,z}$=2500*0.025=62.5 kN.m*

Check column capacity:

$$\frac{125}{0.782*438.6} = 0.364 < 0.9, \quad \frac{62.5}{0.782*308.8} = 0.259 \le 0.9, \quad 0.364+0.259=0.623<1$$

So the column has sufficient resistance for buckling about the “y-y” axis.

Assume buckling about the z-z (minor) axis as critical

- *Bending moment about the y-y (major) axis:*

Member imperfection is not considered.

*Maximum bending moment at column end: 2500*0.05=125kN.m*

Within the column height: 0.513<1

So $M_{Ed,y}$=125kN.m

- *Bending moment about the z-z (minor) axis:*

*The maximum end bending moment: 2500*0.025=62.5 kN.m*

Within the column height:

For bending moment due to member imperfection:

βz=1.0, so $k_z = \frac{1.0}{1-2500/8660} = 1.406$

*So the design moment is 62.5*0.93+1.406*2500*13.3/1000 = 104.9 kN.m*

Check column capacity:

$$\frac{125}{0.782*438.6} = 0.364 < 0.9, \quad \frac{104.9}{0.782*308.8} = 0.434 \le 0.9, \quad 0.364+0.434=0.798<1$$

So the column is ok.

2.11.2 Check load introduction at intermediate level

For the column in the above example, a shear force of 300 kN is introduced by a fin plate connection to the narrow face. Check load introduction.

Calculation results

Load to be introduced to concrete = 300(1-0.61)=117*

*Introduction length d=2*200mm=400mm*

*Bond strength = 0.4*400*200/1000=32kN<117kN, provide through plate connection.* [Table 6.6]

2.11.3 Cross-section property calculations for a short concrete filled CHS column

For circular columns, it is possible to take into account the effect of confinement of the concrete core by the outer steel tube. This effect is only allowed for in design for relatively stocky columns, with non dimensional slenderness $\bar{\lambda}$ not exceeding 0.5. A deliberately short column is used in the following design example to demonstrate how the confinement effect is dealt with.

<u>Design data</u>

CHS 323.9x16.0

Steel grade: S355

Column effective length: 1.0m

Concrete grade: C20/25

Reinforcement: 8H20, with axis distance (to centre of rebar) 30mm, $f_{sk}=500N/mm^2$

Load eccentricity: 20mm

<u>Calculation results</u>

Material properties of the cross-section

$f_{yd}=f_{yk}=355N/mm^2$

f_{cd}=20/1.5=13.33N/mm^2

f_{sd}=500/1.15=434.78N/mm^2

$E_{c,eff}$=30000/(1+4.9)=5084.7N/mm2 (Assuming $N_{G,Ed}/N_{Ed}$=1, S-type concrete, loading age minimum time when using Figure 2-1)

Geometrical properties of the cross-section

$A_s=8*\pi*10^2=2513.3mm^2$

$A_c=\pi*(323.9/2-16)^2-2513.3=64407mm^2$

$A_a=\pi*(323.9/2)^2-\pi*(323.9/2-16)^2=15477mm^2$

For I_s:

Radius of rebar position = 323.9/2-16-30=115.95mm, area per rebar=2513.3/8=314.2mm^2

4H20@distance 115.95cos(45)=82mm to centre line

2H20@distance 119.95mm to centre line

2H20 on centre line

$I_s=4*314.2*(82)^2+2*314.2*119.95^2=1749.2cm^4$

$I_c==\pi*(32.39/2-1.6)^4/4-1749.2=33888cm^4$

$I_a=\pi*(32.39/2)^4/4-\pi*(32.39/2-1.6)^4/4=18390cm^4$

$EI_{eff}=E_aI_a+E_sI_s+0.6E_{c,eff}I_c=(210000*18390+210000*1749.2+0.6*5084.7*33888)/10000$

$=43326kN.m^2$

$N_{cr}=\pi^2(EI_{eff})/L_e^2=427610kN$

$N_{pl,rk}=A_af_y+A_sf_s+A_cf_c=(15477*355+2513.3*500+64407*20)/1000=8039kN$

$\bar{\lambda}=\sqrt{8039/427610}$ =0.137

AS $\bar{\lambda}$ *is less than 0.5, take into account the confinement effect of circular hollow section on concrete infill.*

Plastic compression resistance

e/d=20/323.9=0.062<0.1

$\eta_{c0} = 4.9 - 18.5\overline{\lambda} + 17\overline{\lambda}^2$ *=4.9-18.5*0.137+17*0.137*^{2}*=2.7*

$\eta_{a0} = 0.25\left(3 + 2\overline{\lambda}\right)$*=0.25*(3+2*0.137)=0.818*

$\eta_c = \eta_{c0}\left(1 - \frac{10e}{d}\right)$ *=2.7*(1-10*0.062)=1.026*

$\eta_a = \eta_{a0} + \left(1 - \eta_{a0}\right)\frac{10e}{d}$ *=0.818+(1-0.818)*10*0.062=0.931*

$$N_{\mathrm{pl,Rd}} = A_{\mathrm{a}}\eta_a f_{\mathrm{yd}} + A_{\mathrm{s}} f_{\mathrm{sd}} + A_{\mathrm{c}} f_{\mathrm{cd}}\left[1 + \eta_c \frac{t}{d}\frac{f_{\mathrm{y}}}{f_{\mathrm{ck}}}\right] \qquad [Eqn.6.33]$$

*=(15477*0.931*355+2513.3*434.78+64407*13.33*(1+1.025*16/323.9*355/20))/1000*

=7838kN

Use equation A.6 of this guide, (A_{sn}*=2H20=2*314.2mm*^{2}*)*

$$h_{\mathrm{n}} = \frac{A_{\mathrm{c}} f_{\mathrm{cd}} - A_{\mathrm{sn}}\left(2 f_{\mathrm{sd}} - f_{\mathrm{cd}}\right)}{2*2a f_{\mathrm{cd}} + 4t\left(2 f_{\mathrm{yd}} - f_{\mathrm{cd}}\right)} = \frac{64407*13.33 - 2*314.2*\left(2*434.78 - 13.33\right)}{2*323.9*13.33 + 4*16*\left(2*355 - 13.33\right)} = 6.02mm$$

Bending moment capacity

W_{ps}*=(2*314.2*115.95+4*314.2*82)/1000=175.9cm*^{3}

$$W_{pa} = \frac{32.39^3}{6} - \frac{\left(32.39 - 2*1.6\right)^3}{6} = 1518.2cm^3$$

$$W_{pc} = \frac{\left(32.39 - 2*1.6\right)^3}{6} - 175.9 = 3939.4cm^3$$

W_{psn}*=0*

W_{pan}*=2*16*(6.02)*^{2}*/1000=1.2cm*^{3} [Eqn. A-7 of this guide]

W_{pcn}*=(323.9-2*16)*(6.02)*^{2}*/1000=10.6cm*^{3} [Eqn. A-8 of this guide]

Point A

N_A*=7838kN,* M_A*=0*

Point B

$N_B=0$

$M_B=0.355*(1518.2-1.2)+0.43478*(175.9-0)+0.5*0.01333*(3939.4-10.6) = 641.2kN.m$

Point C

$M_C=M_B=641.2kN.m$

$N_C=A_c f_{cd}=64407*0.01333=858.5kN$

Point D

$N_D=0.5N_C=429.3kN$

$M_D= M_B=0.355*1518.2+0.43478*175.9+0.5*0.01333*3939.4 = 641.7 kN.m$

3 FIRE DESIGN

3.1 General

The presence of load bearing concrete within a hollow steel column has a beneficial effect on the fire resistance of the steel section. Where hollow columns are plain concrete filled, they will usually also be fire protected in the conventional way using externally applied protection but, in most cases, significant periods of fire resistance can be obtained without the need for external protection if the concrete is reinforced. Guidance on both methods of achieving fire resistance is given in this section; however, the emphasis is on the use of unprotected sections. Nevertheless, applying external protection has the practical advantage of removing the need to use reinforcement bars to obtain longer periods of fire resistance.

Note that in any particular case, reducing the applied loads will increase the fire resistance of a column.

Extensive experimental and theoretical investigations on the fire performance of concrete filled columns, without applied protection, have been carried out in Europe and the UK with the support of: the European Coal and Steel Community (ECSC); the International Committee for the Development and Study of Tubular Structures (CIDECT); and various national governments. These studies have led to the development of the design rules that are now included in the design codes of many countries in addition to BS EN 1994-1-2[13] and BS5950-8[14].

The design guidance presented in this publication is based on the Eurocode methodology rather than BS 5950-8[14].

Generally, it is not convenient to carry out the fire design calculations by hand. Design software FireSoft, developed at the University of Manchester for Tata Steel Europe, has been written to enable quick design. The software is available from the Tata Steel Europe. FireSoft is based on the method presented below.

3.1.1 Behaviour in fire

Under ambient conditions the steel and concrete material in a concrete filled hollow section move together, and hence longitudinal steel and concrete strains are equal. Accordingly, the stress in each material is proportional to the ratio of the elastic moduli of the two materials.

On heating, the steel will try to expand more rapidly than the concrete, and will therefore begin to resist a greater proportion of the applied load. However, at the same time, the steel yield stress and modulus of elasticity will begin to reduce and, eventually, the steel will begin to shed load into the concrete.

Heat from the steel shell will be transferred to the outer layers of the concrete core, causing their temperature to rise. However, concrete is not a good conductor of heat, and the rate of heat flow through the core will be slow.

As the temperature of the outer layers increases, the concrete strength itself will begin to fall as the heat degrades the concrete. The degradation includes the driving off of water, which is present both as free moisture and from the hydrated constituents of the mix. This produces a marked plateau in the concrete’s temperature-time profile as a considerable amount of heat is absorbed in converting this moisture to steam.

It is imperative that venting is provided in the steel shell to allow any steam to escape. BS EN 1994-1-2[13] recommends that the sections should contain one vent hole, with a minimum diameter of 20 mm, at the top and bottom of each storey. The longitudinal spacing of these holes should never exceed 5 m. The vent holes should preferably be within the ceiling zone, and care should be taken to ensure that these vent holes are not cast within the slab.

The steel shell restrains the concrete and prevents direct flame impingement, both preventing progressive spalling and reducing the rate of degradation of the core.

Failure of the column will occur when the combined strength of the steel and concrete has reduced to the level of the applied load.

Parameters affecting fire performance

The most significant design parameters affecting the performance of concrete filled columns in fire are detailed below.

Material strength

From solely a fire resistance perspective, a more efficient column is obtained by using a higher strength concrete with a thinner-walled steel section. This gives more advantageous (higher) ratio of concrete load capacity to overall column strength.

Column size

As the external size of a column increases, the cross-sectional area of the concrete core will increase at a faster rate than that of the steel. Accordingly, the concrete core of a larger sized column will support a greater proportion of the total load than a smaller one.

External protection (if applied)

Externally applied protection will reduce the rate of heating and will therefore increase the fire resistance.

Applied load

The lower the level of the applied load, the lower the stresses produced, and the longer the period of fire stability of the column.

Effective length

For short columns, failure will occur when the combined strength of the materials reduces to a level that is less than the applied load (i.e., a crushing failure). As the column length increases, failure will become progressively more related to instability considerations. BS EN 1994-1-2[13] allows a reduction in the effective length of columns in fire for pure axial load only (see Section 3.3.1 for further details).

Bending moments and eccentricity

There are three sources of primary bending moment in a column, as a result of: (a) eccentricity of the axial load; (b) lateral loading within the column height and (c) bending moment transfer from the connected beams in continuous frames (however, continuous frames are beyond the scope of this document).

Reinforcement

The presence of reinforcement in the concrete will improve the flexural and axial properties of the core and so improve the fire resistance, particularly where buckling stability and/or bending moments are major factors.

The Eurocodes use the parameter ‘axis distance’ and not concrete cover to describe the position of the reinforcement within the concrete core. Axis distance is measured from the centre of the bar to the inside of the steel tube, as shown in Figure 3-1.

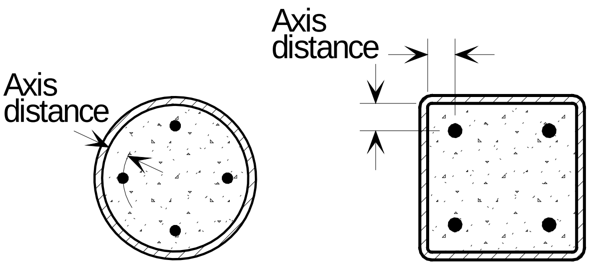


Figure 3-1 Illustration of axis distance for CHS and SHS sections

3.2 General design method

When designing a structure for the fire limit stage, the following three general steps should be followed:

3.2.1 Assessment of fire behaviour

For structural fire safety design, the fire temperature–time relationship should be determined. Either the standard fire exposure or a parametric fire curve may be used. The standard fire exposure is based on a fixed time-temperature relationship described in fire test standards. This time-temperature relationship gives increasing temperature with time so the standard fire resistance rating is used to limit the fire exposure time. The parametric fire temperature-time relationship is intended to describe the entire period of a fire, including burn-out. To obtain a parametric fire temperature – time relationship, the necessary design input data are fire load, opening in the fire compartment and construction materials for the fire compartment. Information is given in BS EN 1991-1-2[7]. Parametric time-temperature curves should only be used by people with appropriate experience.

3.2.2 Calculation of temperatures in the structure

Having determined the appropriate fire exposure condition, the structural temperatures are calculated by performing a heat transfer analysis. For concrete filled tubular construction, it is generally necessary to use a numerical heat transfer program to perform this task. Assuming uniform temperature distribution along the column height, the essence of this heat transfer calculation is to solve a 2-D transient heat conduction equation under the combined convective and radiant heat transfer boundary condition. The necessary input data are thermal properties (thermal conductivity, specific heat and density of steel, concrete and the fire protection material, if any) and those relating to the thermal boundary condition. The thermal properties of steel and concrete are given in Appendix B, along with their mechanical properties at elevated temperatures.

In BS EN 1991-1-2[7], the thermal boundary condition is given as:

Rate of heat input to the external surface of a composite column

$$\text{(W/m}^2\text{)} = \alpha\left(\theta_g - \theta_s\right) + \phi\varepsilon_r\sigma\left[\left(\theta_g + 273\right)^4 - \left(\theta_s + 273\right)^4\right] \quad \textbf{3-1}$$

where:

θ_s is the column surface temperature (ºC)

θ_g is the fire gas temperature (ºC)

$\varepsilon_r = \varepsilon_m \varepsilon_f$ is the resultant emissivity between the column surface and the fire gas

ε_m is the resultant emissivity between the fire and the steel surface

σ is the Stefan Boltzmann constant $= 5.67 \times 10^{-8}$ W/m² ºK^4

α is the fire gas convection heat transfer coefficient = 25 W/m^2 ºK

ϕ is the orientation (configuration or view) factor = 1.0

The resultant emissivity can be approximated to $\varepsilon_r = \varepsilon_m \varepsilon_f$, where ε_m is the emissivity of the steel surface and ε_f is the emissivity of the fire (0.8) A value of ε_m =0.625 is given in BS EN 1993-1-2[11] and 0.7 in BS EN 1994-1-2[13]. ε_m =0.625 has been used in calculating the fire resistance of a large number of fire tests on unprotected concrete filled tubular columns and is found to give acceptable results. Therefore, ε_m =0.625 may be used in design calculations and $\varepsilon_r = \varepsilon_m \varepsilon_f$ =0.5[20].

3.2.3 Evaluation of load bearing capacity of the structure

In Annex H of BS EN 1994-1-2[13], a method is proposed to calculate the compressive strength of concrete filled tubular columns at elevated temperatures and a simple equation is also given to account for the effect of eccentricity of the axial load. The Annex H method follows an old method[16] of calculating composite column axial resistance at ambient temperature. For fire resistance calculations, this method is particularly difficult to implement as it would involve many iterations of calculation, with each round of iteration having to use the detailed material non-linear stress-strain relationships of steel and concrete at different elevated temperatures for each of the many blocks into which the composite cross-section is divided. In addition, in Annex H of BS EN 1994-1-2[13], the effect of small eccentricities is taken into account by introducing two modification factors, one as a function of the reinforcement ratio and one as a function of eccentricity & column dimensions. The CIDECT research project (15Q) [18] has assessed the Annex H calculation method and found that the Annex H method can be grossly inaccurate.

The UK's National Annex does not permit the Eurocode method in Annex H to be used. As an alternative method to enable using concrete filled tubular sections, the column design method for the fire limit state should be consistent with that at ambient temperature, which is given in Chapter 2 of this guide, provided the reductions in strength and stiffness of the steel and concrete at elevated temperatures are taken into consideration. This is the basis of design software FireSoft. In the quality assurance document for FireSoft, the calculation results are compared with a large number of standard fire resistance test results for concrete filled tubular sections under axial load or combined axial load and bending, and the FireSoft calculation results are close to the test results and on the conservative side.

3.2.4 Distinction between columns under compression only and columns under combined compression and bending

For ambient temperature design, distinction is made between columns under compression only (section 2.6.3) and columns under combined compression and bending (section 2.7). The same distinction can be made for fire design. If the column is under combined compression and bending, FireSoft should be used. If the column is under compression only, the simplified method in Section 3.3 can be used instead. This may apply to the case of bottom storey columns where the moment from the out of balance beam reactions is negligible.

3.3 Special considerations at elevated temperatures

Due to differences in behaviour of a composite column at the fire limit state and at ambient temperature, the following modifications to the ambient temperature design method are necessary.

3.3.1 Column effective length in fire

As part of a frame, a column is rotationally restrained by the surrounding structure. In an enclosure fire situation, the column in the fire compartment is rotationally restrained by the surrounding beams and slabs in the fire compartment and the cool columns outside the fire compartment. Although the fire heated beams and slabs will provide some rotation restraint to the column, this restraint is unreliable and difficult to quantify and should be discarded.

However, since the flexural bending stiffness of the heated column will be very low at the fire limit state, the relative rotational restraint stiffness of the cool columns outside the fire compartment to the heated column can be very high, making the rotation restraint provided by the cool columns to the heated column approach that of rotationally fixed. Therefore, in non-sway frames, BS EN 1994-1-2[13] states that the effective length of the heated column should be taken as 0.5 times the system length if the heated column is restrained by cool columns at both ends; and should be 0.7 times the system length if the heated column is restrained by a cool column at only one end. .

However, the UK National Annex to BS EN 1994-1-2[13] recommends replacing the values of 0.5 and 0.7 by values of 0.7 and 0.85 respectively.

Figure 3-2 illustrates the UK National Annex recommendations.

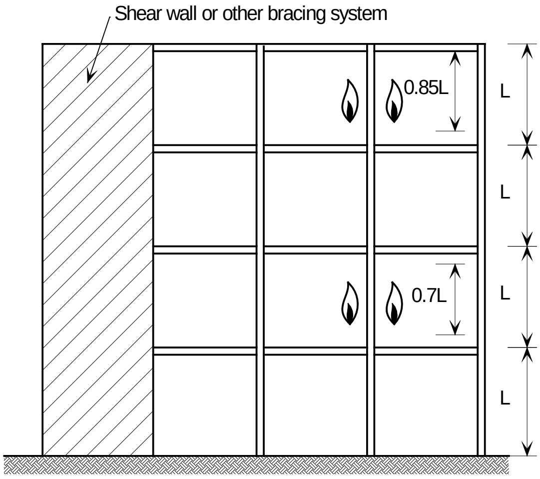


Figure 3-2 Effective length of columns in fire conditions (N.B., the values of 0.7 and 0.85 are taken from the UK National Annex)

It is important to recognise that the reduced column effective length is a result of the additional rotational restraint provided by the surrounding cool columns. Therefore, the cool columns should (a) be continuous from the heated columns; (b) be outside the fire enclosure.

3.3.2 Column buckling curve

The column buckling curves are used to account for different effects of initial imperfections (residual stress, initial deflections) in different types of columns. In the fire situation, such effects can be overwhelmed by "imperfections" generated in the fire, e.g. non-uniform temperature distributions and sensitivity of mechanical properties of materials at elevated temperatures. Therefore, a uniform column buckling curve is recommended for different types of columns for the fire limit state design. This is column buckling curve ‘c’ and the imperfection factor α, is 0.49.

It should be pointed out that the column effective length and column buckling curve are used only if the column is under axial compression.

For columns under combined compression and bending, the second-order effects are explicitly accounted for by including the bending moments due to imperfections. Because buckling curve “c” is used, the member imperfection for fire design is L/150.

3.3.3 Cross-sectional properties

It is generally not feasible to carry out fire design calculations by hand and use of the accompanying software, FireSoft, is advised.

It is necessary to divide the cross-section into a number of elements. The structural section properties are calculated by summing the properties of all the elements, with due account being paid to the effect of the temperature on each element.

Squash (plastic) resistance, Nfi,pl,Rd

The squash resistance of the composite column is calculated from the following summation:

$$N_{\mathrm{fi,pl,Rd}} = \sum_j \frac{A_{\mathrm{a},\theta}\, f_{\mathrm{a,max},\theta}}{\gamma_{\mathrm{M,fi,a}}} + \sum_k \frac{A_{\mathrm{s},\theta}\, f_{\mathrm{s,max},\theta}}{\gamma_{\mathrm{M,fi,s}}} + \sum_m \frac{A_{\mathrm{c},\theta}\, f_{\mathrm{c},\theta}}{\gamma_{\mathrm{M,fi,c}}} \qquad \textbf{3-2}$$

where:

$A_{\mathrm{a},\theta}$ is the area of an element of the steel section.
$A_{\mathrm{s},\theta}$ is the area of an element of the reinforcement.
$A_{\mathrm{c},\theta}$ is the area of an element of the concrete.
$f_{\mathrm{a,max},\theta}$ is the strength of the steel section element at temperature θ (see Appendix B).
$f_{\mathrm{s,max},\theta}$ is the strength of a steel reinforcement bar at temperature θ (see Appendix B).
$f_{\mathrm{c},\theta}$ is the strength of the concrete element at temperature θ (see Appendix B).
$\gamma_{\mathrm{M,fi,i}}$ is the partial material safety factor for the element.

The UK National Annex recommends the following partial material safety factors:

Steel, $\gamma_{\mathrm{M,fi,a}}$	1.0
Concrete, $\gamma_{\mathrm{M,fi,c}}$	1.0
Reinforcement, $\gamma_{\mathrm{M,fi,s}}$	1.0

Effective flexural stiffness

Under fire conditions, thermal stresses will be present in a composite column. Because the stress-strain relationships of steel and concrete at elevated temperatures are non-linear, the effects of thermal stresses on the flexural stiffness of a composite cross-section should be considered. The stress-strain relationship of concrete at elevated temperatures is similar to that at ambient temperature. Therefore, the secant modulus that is used in equation 2.4 for ambient temperature design may continue to be used for fire design. In addition, the calibration factor of 1.35 in equation 2.4 for ambient temperature design may be dropped for fire design. For the steel tube and reinforcement (if any), the modulus in fire design may be lower than the initial tangent modulus of elasticity. Therefore, the effective flexural stiffness of the composite column $(EI)_{\mathrm{fi,eff}}$ can be determined from:

$$(EI)_{\mathrm{fi,eff}} = \sum_{\mathrm{j}} \varphi_{a,\theta} E_{\mathrm{a},\theta} I_{\mathrm{a},\theta} + \sum_{\mathrm{k}} \varphi_{s,\theta} E_{\mathrm{s},\theta} I_{\mathrm{s},\theta} + \sum_{\mathrm{m}} \varphi_{c,\varphi} E_{\mathrm{c}m,\theta} I_{\mathrm{c},\theta} \qquad \textbf{3-3}$$

where:

$I_{\mathrm{a},\theta}$, $I_{\mathrm{s},\theta}$ and $I_{\mathrm{c},\theta}$	are the second moment of area of each element about the relevant axis of composite cross-section.
$E_{\mathrm{a},\theta}$, $E_{\mathrm{s},\theta}$ and $E_{\mathrm{c}m,\theta}$	are respectively the initial modulus, initial modulus and secant modulus of the stress-strain relationship for the steel tube, reinforcement and concrete at its temperature θ.
$\varphi_{\mathrm{a},\theta}$, $\varphi_{\mathrm{s},\theta}$ and $\varphi_{\mathrm{c},\theta}$(=0.8)	are reduction factors accounting for the effect of thermal stresses.

BS EN 1994-1-2[13] does not give values for the reduction factors for concrete filled tubes. However, the values for partially encased composite sections in Table G.7 of BS EN 1994-1-2[13] may be used. For design under standard fire exposure, these values are:

Fire rating = R30 and R120: $\varphi_{\mathrm{a},\theta}=\varphi_{\mathrm{s},\theta}=1.0$.

Fire rating = R60: $\varphi_{\mathrm{a},\theta}=\varphi_{\mathrm{s},\theta}=0.9$.

Fire rating = R90: $\varphi_{\mathrm{a},\theta}=\varphi_{\mathrm{s},\theta}=0.8$.

Interaction curve of the composite cross-section at elevated temperatures

Due to non-uniform distribution of temperatures in the cross-section, it is no longer possible to derive analytical equations to evaluate the bending moment – axial load interaction curve of the composite cross-section. Instead, the bending moment and axial load capacities are calculated by summing up the contributions of all the blocks of the cross-section, which are the same as those used in heat transfer analysis.

Effective flexural stiffness for the determination of internal forces

At ambient temperature, the effective flexural stiffness for the determination of the internal forces is given in equation 2.14. BS EN 1994-1-2[13] does not have an equivalent equation for fire design. It is recommended to use the same equation for fire design, but replacing the ambient temperature modulus of elasticity values by those at the elevated temperatures. Furthermore, the coefficient "0.5" in equation 2.14 should be replaced by 0.4 (=0.5*0.8) as allowance for the effects of thermal stresses ($\varphi_{c,\theta}$=0.8).

3.4 Protected columns

Reinforced CFT rarely require external fire protection. Unreinforced concrete-filled structural hollow section columns with a design capacity based on the room temperature properties of the full cross-section may be protected against fire with externally applied insulating materials. This is usually necessary for fire resistance periods of 60 minutes and above.

Fire protection thicknesses are based on critical steel temperatures, i.e. the temperature at which the column will fail for a given load (utilization) level. Default critical temperatures are given in the Yellow Book [1]. For situations where the column is not fully loaded, and the utilization factor can be found, alternative (higher) critical temperatures may be calculated. This can have the effect of significantly reducing fire protection thicknesses.

Based on the research by the author of this guide published in The Structural Engineer[20], the following simplified method may be adopted to determine the required fire protection thickness:

1. Determine the ambient temperature resistance of the CFT column according to BS EN 1994-1-1[12].

2. Calculate the utilization factor in fire, defined as:

$$\mu_{fi} = \frac{\text{Load at fire limit state}}{\text{Resistance at } 20^{o}\text{C}} \qquad \textbf{3-4}$$

3. Using the utilization factor, assess the critical temperature for the steel section using the default critical temperature table (Table NA.1) for compression members in the UK National Annex to BS EN 1993-1-2[11].

4. Consult the fire protection manufacturer to obtain a suitable fire protection thickness to ensure that the steel in the CFT section does not exceed the critical temperature, as calculated above for the required fire resistance period.

The load at the fire limit state is calculated using partial safety factors for permanent load of 1.0. The partial safety factors for variable load are taken from BS EN 1990[6] Table A1.1 using guidance in Clause NA.2.7 of BS EN 1991-1-2[7].

3.5 Protection to the connection when using unprotected columns

If the CFT column is unprotected but the connection is protected (see Figure 1-11), the impact of additional heat conduction from the unprotected CFT column to the connection should be taken into consideration. This is usually addressed by extending the fire protection to the connection around the perimeter of the CFT and then extending it down the tube beneath the connection. This is a process called coatback.

Finite element heat transfer simulations (see Appendix C for detailed results) have been carried out to examine the maximum additional temperature rises in the connection for varying a number of typical CFT column dimensions and coatback distances (extension of the fire protection below the lowest point of the connection component).

The results indicate that when the extension (coatback) distance beneath the connection is at least 150mm, there is little heat transfer from the unprotected CFT column to the connection zone so that the maximum additional temperature rise in the connection is negligible. If the coatback distance is 50mm, the additional heat conducted from the unprotected CFT column to the connection zone increases the connection temperature by a maximum value of about 100^{o}C.

Based on the results of these finite element simulations, the following advice is provided for unprotected, reinforced tubes with minimum outside diameter of 323mm and minimum thickness of 12.5mm:

- Where the coatback distance is a minimum of 150mm from the lowest connection component, the critical temperature for specifying the fire protection thickness should be the same as the connecting steel beam.
- If the coatback distance is less than 150mm, subject to a minimum of 50mm, the critical temperature for specifying the fire protection thickness to the CFT column should be 100^{o}C lower than that of the connecting steel beam.

3.6 Column splice

To preserve the appearance of the exposed steel tubular section, the column splice is usually embedded in the composite floor. If this is the case, the splice can be taken as being fully protected by the surrounding concrete. No additional fire protection is necessary.

If the splice is not in the composite floor and is subject to significant bending moment, care must be taken to ensure that the bolts in tension maintain their strength relative to the steel tube section. Because bolts lose a higher proportion of their strength than the steel tube section at high temperatures, it may be necessary to use a few more bolts than required only to transfer the bending moment. The advice of a competent structural fire engineer should be sought.

4 Fabrication and Connections

4.1 Simple Connections

Simple connections are normally assumed to give vertical support but to provide only limited restraint against rotation: these connections are assumed to be able to rotate without damage. A selection of some common examples is shown in Figure 4-1.

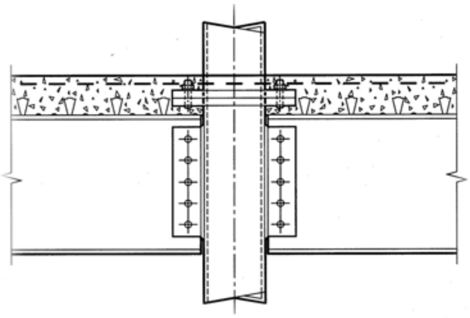
The most common arrangements for beam-to-column joints make use of bolted connections via attachments welded to the faces of the hollow section column. By far the most common connection of this type is the fin plate connection, using a flat plate welded to each column face. If the shear load to be transferred from the beam to the column is high (thus necessitating the introduction of mechanical shear connectors inside the tubular section if using fin plate connection), through-plate connection would be preferable to simplify fabrication. For RHS columns, an alternative is the web cleat connection, using single angle sections, or T-sections, welded to the column face. The use of double angles is a further option and provides greater capacity than is available using a single angle. A further option for CHS or RHS columns is the use of the reverse channel connection. In this connection, the legs of a channel (reverse channel) are welded to the column face and the web of the channel is connected to the beam via an endplate. The channel can be cut from a hot-rolled channel section. However, hot-rolled channel sections have thinner webs than flanges (legs). Because the web of the channel often governs design of the connection, it is preferable to increase the web thickness. Therefore, for improved performance, the reverse channel can be made from a hot-rolled rectangular/square hollow section which has the same thickness. Furthermore, the rounded geometry of the tubular profile gives the connection much better deformation capacity than a hot-rolled channel section. Although compared to fin plate/through-plate connections, this type of connection incurs some additional fabrication cost. It has good ductility, which may be advantageous when improved robustness of the structure is desired.

Under a shear force in simple construction, the reverse channel web/end plate components are designed following the same rules as for endplate connection design and the reverse channel leg/weld components are designed as for fin plate connection.

Simple steel connections using flexible end plates or double angle cleats, which are bolted direct to the column, are also possible with RHS columns. These joints use either expanding bolt types, such as Hollobolt, or fully threaded bolts in tapped holes produced by the Flowdrill system.

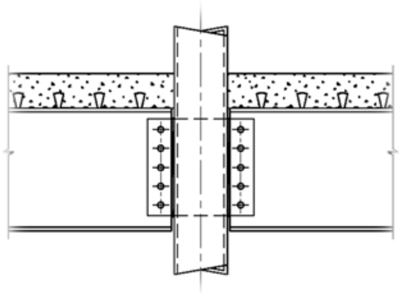
Fin plate/Shear tab connection

Section through composite deck floor with steel beam connected to RHS or CHS column by a finplate; a very economic joint; a seating cleat may be used to help erection, with removal afterwards if required.



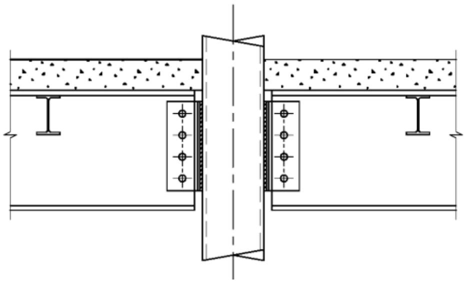
Through-plate connection

Section through floor with through-plate passed through slots and welded to each face of RHS or CHS column; the through plate connection allows significant axial forces and bending moments to be transferred from the beam, if this is required.



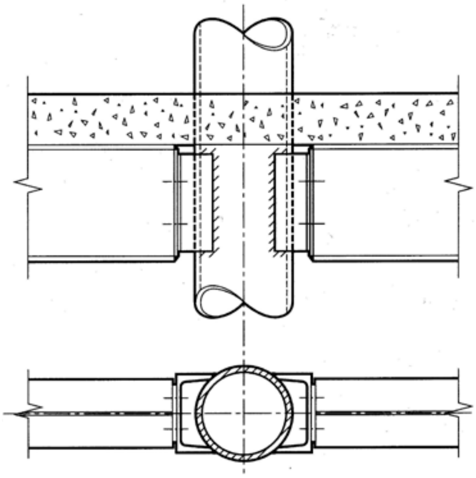
Web cleat connection

Section through floor with secondary and primary beams which are connected to RHS columns by a T-section web cleat; this connection can have greater stiffness and robustness than the equivalent fin plate connection



Reverse channel connection

Section through floor with steel beams connected to CHS columns by a channel fixing; a standard flexible end plate is normal at the beam end, which then develops only nominal moments. A similar detail is used for RHS columns.



Hollobolt or Flowdrill connection

Section though floor with steel beam connected to face of RHS column by Hollobolts or fully threaded bolts in Flowdrill holes; only nominal moments develop with flush or partial depth flexible end plates; thicker plates can develop moment capacities between 10 to 15 per cent of beam capacity.

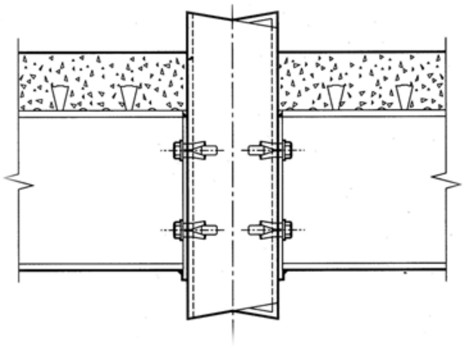


Figure 4-1 Beam-to-column joints – simple connections

Where large moments are to be transferred, it may be necessary to stop and start the columns (Figure 4-2). A load bearing stiffener can be used along with the web of the beam to transfer the axial loads from the storeys above to the column; this ensures that no moment from the beam over is transferred to the column below.

When filling concrete from the bottom, it is necessary to have a hole in the cap plate of the column below to ensure full compaction of concrete underneath the cap plate. The bearing capacity of the concrete in the column below should be checked (section 2.9).

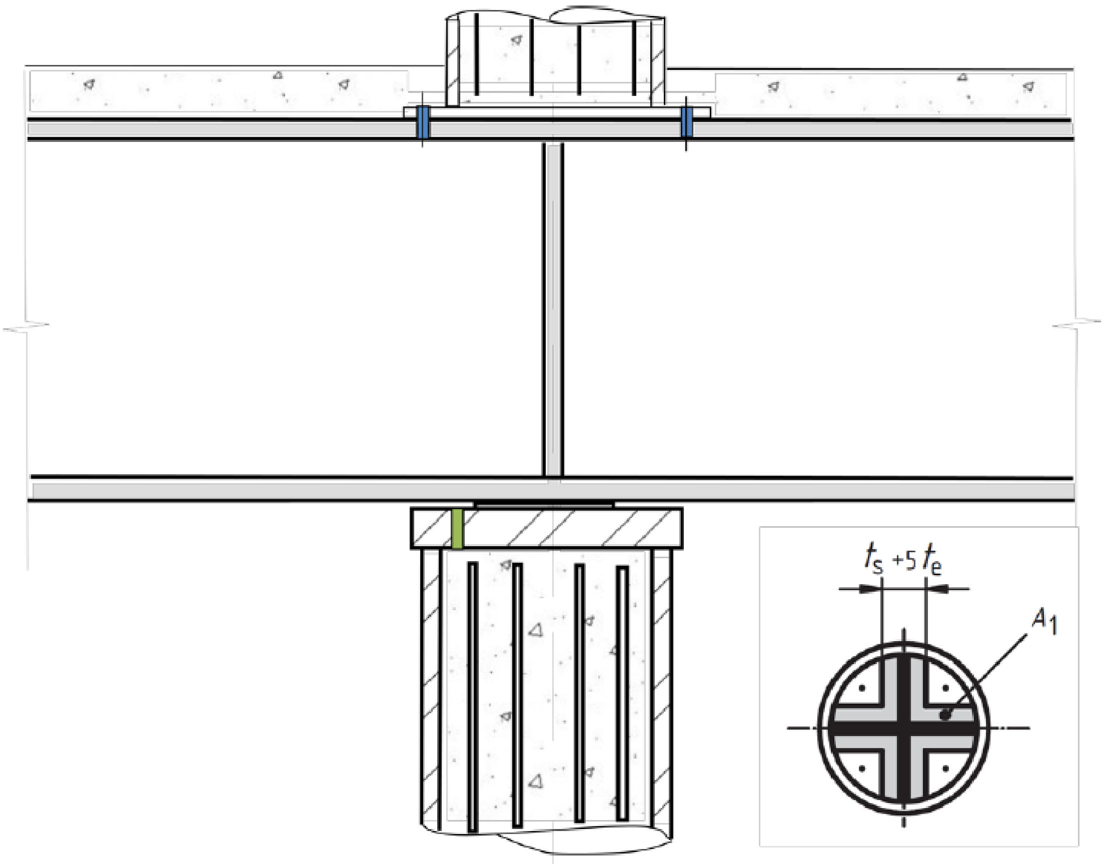


Figure 4-2 Beams over column for complete rigid connection

4.2 Load introduction in fire

Connections generally need to be either fire protected or shielded by other elements such as floor slabs.

Questions sometimes arise as to how the loads are transferred into a concrete filled column from an incoming beam, when the outer steel tube is exposed to fire below the connection. If bolts or studs protrude into the concrete core, then some degree of direct load transfer can be envisaged. However, it will often be the case that the beam-to-column connection takes the form of a fin plate, resulting in no obvious direct load path into the concrete core. In this case, the load transfer is via the cold part of the steel tube above the connection. The beam reaction is therefore resisted by tension in the tube, which is transferred into the concrete core by a combination of shear, bond and direct axial load via the column cap.

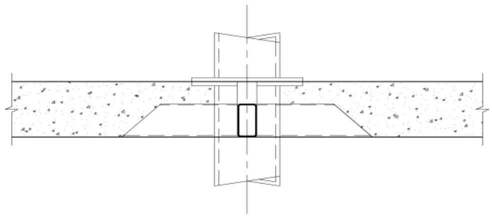
4.3 Shearhead connections to slabs

The use of composite or precast concrete floor slabs, supported on steel beams, can lead to economical designs. In each of these cases, the fixing of the steel beams to the concrete filled hollow sections can be achieved by using the connections shown in Figure 4-3.

In situ concrete beams and floors may also be used with concrete filled hollow sections. Figure 4-3 illustrates some shearhead connection details to provide shear and moment transfer between reinforced concrete flat slabs and composite columns. For calculating punching shear resistance of the flat slab, the dimensions of the column may be enlarged by the horizontal lengths of the shearhead arms.

Centre stub shearhead

Section through concrete flat slab floor with column tubular stubs welded in cross form slotted through cut-outs in the tube and welded to the RHS or CHS columns.



Grid shearhead

Section through concrete flat slab floor with grid welded to faces of RHS columns; the connection is designed for transfer of tensile and compressive forces from the slab to the shearhead elements.

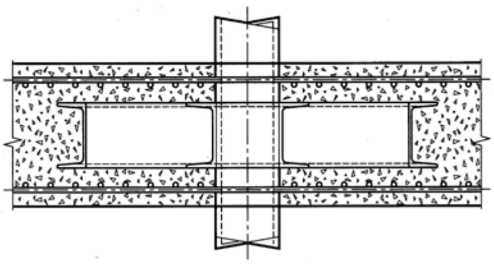


Figure 4-3 Shearhead connections to in situ concrete slabs

4.4 Base plate connections

With the exception of the need to provide a drain/vent holes in the wall of the hollow section, no specialised details are required for base plate connections. A typical example of a base plate for a concrete filled column using SHS is shown in Figure 4-4.

Baseplate with loose bolts

Section through base with cast-in bolts in boxes; bolts are loosened soon after concrete has set; underside of base and bolt boxes are grouted after baseplate is packed up and column is plumb.

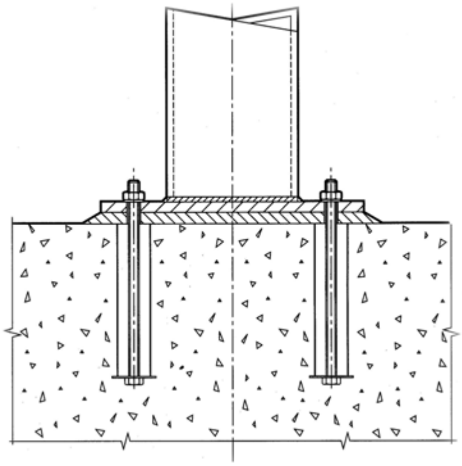


Figure 4-4 Base plate connections

5 CONSTRUCTION OF CONCRETE FILLED TUBES

5.1 Introduction

5.1.1 Construction advantages of CFT columns

Many different ways of filling steel tubes with concrete or grout mixtures are available, either on-site or off-site depending on the particular site and contract conditions. Where only small lengths of tube are being handled at any one time, off-site filling is a possibility and this may have benefits in reducing the number of site operations and speeding up the building process. However, in general, filling of columns is done on-site, and this is the only practical method for long lengths of tube. In either case, filling of columns may be done rapidly and in parallel with other site operations so that, in general, this operation does not add any additional time to the overall building programme.

Structural design of concrete filled columns is usually done on the basis that all or some part of the construction loads, and coexisting dead and wind loads, may be taken by the unfilled bare steel columns, so that filling of the columns may be done at convenient intervals. In a typical multi-storey building, construction of the bare steel tubes may be allowed to advance 7 or 8 storeys before filling is needed; the bare steel columns support the concreted floors and allow simultaneous working of different trades at each level. In this way the CFT column combines all the normal advantages of steelwork in the construction phase, rapid erection, with the economic advantages of composite columns in service, that is economic load-bearing capacity in compression.

Another very important advantage of the CFT column, as compared to a reinforced concrete column, is the elimination of formwork and the consequent time saving in the construction phase.

5.2 Concrete specification and placement

5.2.1 Concrete specification

The properties achieved by the concrete in its fluid state at placement and in its final solid state depend on the binders, aggregates and admixtures used but particularly on the water content. The principal properties monitored are the strength and shrinkage in the final state and the workability, cohesion and ease of pumping in the fluid state.

Any reasonable strength concrete mix may be used to fill a steel tubular column assuming it is fluid, so that it can be pumped up or poured in from the top of the column and vibrated in position. Normally, to achieve a fluid pumpable mix, it is necessary to add a superplasticiser or use a grout and achieve a minimum slump of 160-210 mm (S4). High strength concrete (HSC) is often an attractive option as it reduces the floor area occupied by the columns and this generally requires the use of a water reducing agent as well as a superplasticiser. However, as explained in section 2.2.2, for fire design, using HSC has little advantage over using normal strength concrete.

Portland Cement is the principle binder in the finished concrete filled tubular column and can attain cylinder compressive strengths of up to about 80 Mpa by itself. However additional binders may be added, such as fly ash, ground granulated blast furnace slag and condensed silica fume to improve properties and reduce costs. The addition of fly ash reduces the water content, and therefore increases the strength so that cement content may be reduced, as well as giving a faster gain in strength at 28 days and better chemical resistance. The use of ground granulated blast furnace slag has similar effects to fly ash. Another expensive but very effective additive is silica fume. The use of silica fume can increase the strength of a simple Portland Cement concrete mix by about 20% and is particularly applicable for a high strength mix, above say a cylinder compressive strength of 80 Mpa, where a simple Portland Cement mix is near its limit. It produces a very cohesive and pumpable mix but does need the addition of a superplasticiser to reduce the water content that would otherwise be required.

5.2.2 Workability, cohesion and ease of pumping

The ideal infill mix is workable, cohesive and easy to pump. These properties have to be achieved by trial and error with the selected materials. Workability can be assessed by use of slump and flow table spread tests and these might be useful in arriving at a suitable mix but are only a guide as to the suitability of the mix. For a normal strength mix, a workability test is a guide as to the water content of the mix, and therefore its strength, as well as a measure of the fluidity of the mix. For a high strength mix the water content has to be strictly controlled and workability is then controlled by the dosage of admixtures, particularly the superplasticisers and water reducing agents used.

Table 5-1 presents a sample mix for Grade C50/60 concrete ($1m^3$).

Table 5-1 A sample mix for Grade C50/60 concrete

Component	Quantity (per m^3)
Coarse Aggregate	1020 KG (maximum size 10mm)
Sand	690 KG
Cement	336 KG
Water/cement ratio	0.369
PFA	144 KG
Super Plasticiser	3.12 Litre

5.2.3 Methods of placement

Concrete or grout infill to the tubular column may be placed inside the column either from the top of the column or by pumping from the bottom (Figure 5-1). For filling from the top, a mobile pump, with an extension boom reaching to the top of the column being filled, may be used to place concrete directly, using a pipe taken to the bottom of the empty tube. This limits segregation caused by dropping of the concrete. For small columns, a tremie pipe also helps to control segregation. Another method is to use funnels at the top of the column, concrete being brought in by skip. Care must be taken to avoid large drops into the column from the top, although if a good cohesive mix has been obtained this may still possible if the bottom of the column does not contain an excessive amount of coarse aggregate, which would lead to ‘honeycombing’, or voids between aggregate particles.

Figure 5-1 Cut-off valve for pumping concrete (photo courtesy of Arup)

Vibration of the concrete is necessary, either by clamped on external vibrators or by internal vibrators (if possible). It has been reported that some external vibrators cause standing waves leading to poor compaction, although this has not been the general experience.

For larger contracts, the concrete or grout infill is much more conveniently placed by pumping from the bottom, usually through a gate valve fitted to the bottom of the column. Figure 5-1 shows a typical example. As well as being an extremely convenient method that avoids disruption to other site operations, pumping from the bottom avoids the need for vibration and essentially eliminates the presence of voids in the fill material. For this method grouts or fluid concrete mixes need to be specified.

It is also possible to bring tubular columns onto site in two or three storey height lengths that have been pre-filled off site. However this then requires grouting of gaps at the column splice positions and careful control to prevent voids in the infill material.

Whatever placement method is chosen it is important that the previously cast surface is clean but otherwise no scabbling of the surface or special treatment is generally required. Provision of standby equipment in case of breakdowns or delay is essential, especially the provision of standby pumps, when a pumping method is selected.

5.2.4 Concrete placed by filling from top

The concrete for columns filled from the top is best compacted by an internal (or poker) vibrator: this type should be used whenever possible. The vibrator should be lowered to the bottom of the column and switched on immediately before any concrete is placed. The concrete should be placed slowly around the vibrator, which must be left running continuously, and raised as the concrete level rises, such that its position is just below the surface of the concrete; its position may be judged by raising and lowering it, and noting the change in sound as it emerges from the concrete.

Placing the concrete requires careful consideration because of the cost of the plant required to handle relatively small quantities. The majority of concrete used on site is ready-mixed, and this is most economical if it can be used in 6 m^3 loads, which need to be placed within half an hour if waiting charges are to be avoided. A crane and skip could handle concrete at this rate into the large columns. A mobile pump with a boom can place the concrete continuously as it is being compacted and, provided that the poker diameter is at least 50 mm, this rate of placing could be achieved in medium sized columns. However, concrete in smaller columns should be placed at a proportionally slower rate, in order to achieve satisfactory compaction. Mortar pumps with 50 mm diameter rubber hoses could be used, provided that the maximum aggregate size does not exceed 10 mm. Generally, each time the maximum size of aggregate is halved, the cement content needs to be increased by 20%.

5.2.5 Concrete fill placed by pumping from bottom (Figure 1-12)

Concrete fills or grout that are placed by pumping from the bottom are the preferred methods on large contracts. Depending on the design conditions, and in particular the method used to achieve fire resistance, maximum benefit from the use of CFT columns is generally achieved by use of a high strength concrete with cylinder strengths above 50 MPa. Pumping of fluid concrete mixes up to 30 metres in height is achievable by most pumps in general use and pumping heights in excess of 250 metres are achievable with special equipment. The pipeline pressure at the pump to get flow up a medium sized tube has been measured as being approximately 60% greater than the static pressure of the fluid concrete and is no more than that required to fill the column from the top. The presence of reinforcement, diaphragms or any other kind of obstruction within the column will require slight increases in the pumping pressures. Pumping rates in excess of $50m^3$/hour are achievable.

Inlet holes should be at least 100mm in diameter. It should be checked that holes can be hidden in the floor finishes to avoid expensive making good. Ideally holes can be left uncovered (Figure 1-12). It is worthwhile to check the squash load of the steel effective section to remove the need to patch the column with welded cover plate.

Air vents at the top of the column should be placed in such a way that full compaction under capping plates is achieved. In multi-storey simple construction it may be convenient to leave holes up to 100 mm diameter in the upper cap plate to enable concrete through to the inlet hole on the storey above (Figure 5-2). The inlet hole of the storey above can be used as an inspection hole to confirm the correct level has been achieved.

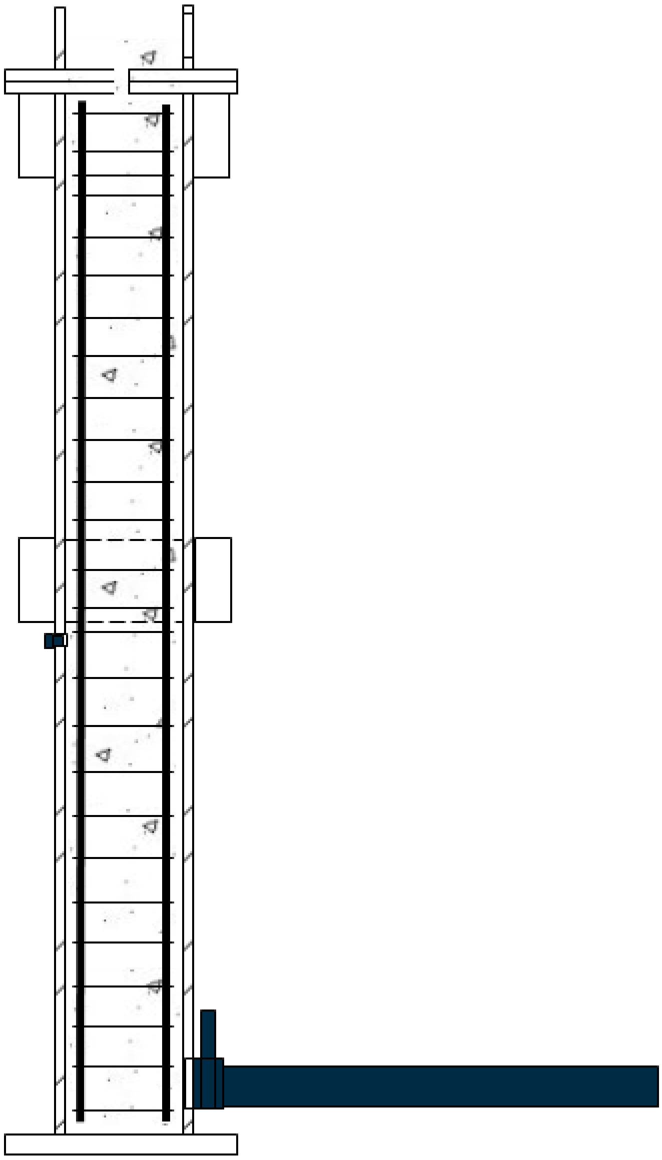


Figure 5-2 Hole in cap plate to allow concrete through

5.2.6 Reinforcement

It is preferable to insert reinforcement into the steel tubes in the fabrication shop. Prewelded cages up to 15m in length can be bought from stockists by the steelwork fabricator and then slotted into the tubes with rollers and spacers on the longitudinal reinforcement. This requires coordination between the fabricator and the reinforcement provider (likely the concrete sub-contractor). It is important to check the reinforcement for quality assurance control prior to welding of the cap plates. Erection can then proceed on site, as with a traditional steel frame and using similar column splice lengths. Care should be taken to ensure that the longitudinal bars do not obstruct the inlet holes when bottom filling the tubes (Figure 5-3).

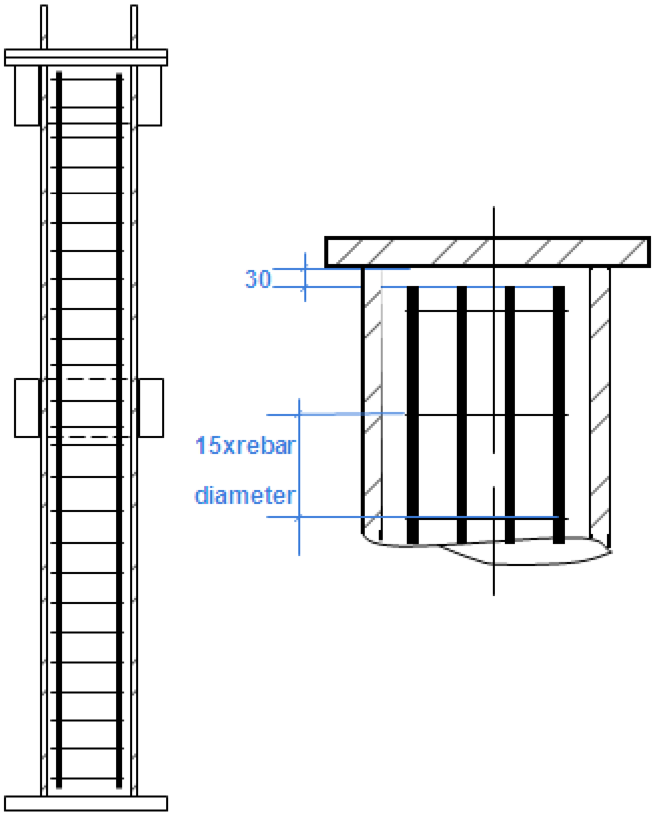


Figure 5-3 Placement of reinforcement cage

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APPENDIX A Bending moment resistance of concrete filled cross-section

A.1 Point B

The plastic moment resistance of a concrete filled hollow section may be evaluated as follows:

$$M_{\mathrm{pl,Rd}} = f_{\mathrm{yd}}(W_{\mathrm{pa}} - W_{\mathrm{pan}}) + 0.5 f_{\mathrm{cd}}(W_{\mathrm{pc}} - W_{\mathrm{pcn}}) + f_{\mathrm{sd}}(W_{\mathrm{ps}} - W_{\mathrm{psn}}) \qquad \text{A.1}$$

where:

f_{yd}, f_{sd}, f_{cd} are $\frac{f_{\mathrm{y}}}{\gamma_M}$, $\frac{f_{\mathrm{sk}}}{\gamma_{\mathrm{s}}}$ and $\frac{f_{\mathrm{ck}}}{\gamma_{\mathrm{c}}}$ respectively

W_{pa}, W_{pc}, W_{ps} are the plastic section moduli for the steel section, the concrete of the composite cross-section (assumed to be uncracked) and the reinforcement respectively.

W_{pan}, W_{pcn}, W_{psn} are the plastic section moduli of the corresponding components within the region of $2h_{\mathrm{n}}$ from the centre-line of the composite cross-section.

The values of the relevant parameters in the above equation for concrete filled hollow sections are:

A.1.1 Rectangular hollow sections

$$W_{\mathrm{pc}} = \frac{(b-2t)(h-2t)^2}{4} - \frac{2}{3}r^3 - r^2(4-\pi)\left[\frac{h}{2} - t - r\right] - W_{\mathrm{ps}} \qquad \text{A.2}$$

where: r is the internal radius of the corners to the hollow section

A.1.2 Circular hollow sections

$$W_{\mathrm{pc}} = \frac{(d-2t)^3}{6} - W_{\mathrm{ps}} \qquad \text{A.3}$$

A.1.3 Elliptical hollow sections

To aid calculations for elliptical hollow sections, the plastic modulus of a solid portion (shaded part in Figure A.1) of an elliptic, with dimensions shown in Figure A.1, about the dotted central axis of the elliptic, is:

$$W_p = \frac{2ab^2}{3}\left[1-\left(\frac{h_n}{b}\right)^2\right]^{\frac{3}{2}} \qquad \text{A.4}$$

Therefore, for an elliptical hollow section,

$$W_{pc} = \frac{4(a-t)(b-t)^2}{3} - W_{ps} \qquad \text{A.5}$$

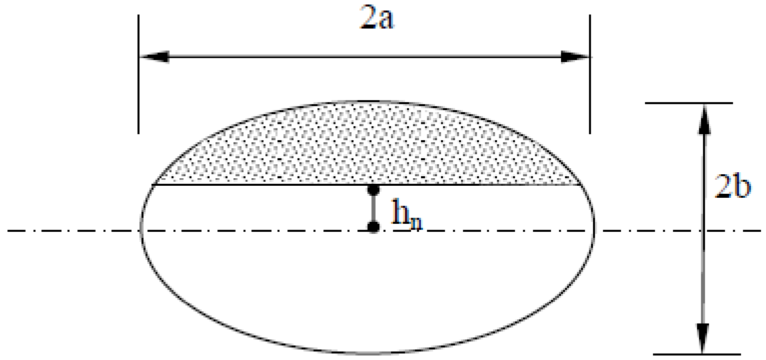


Figure A.1: Dimensions of elliptic

In general, for all three types of section:

$$h_{\mathrm{n}} = \frac{A_{\mathrm{c}} f_{\mathrm{cd}} - A_{\mathrm{sn}}(2 f_{\mathrm{sd}} - f_{\mathrm{cd}})}{2 * 2a f_{\mathrm{cd}} + 4t(2 f_{\mathrm{yd}} - f_{\mathrm{cd}})} \qquad \text{A.6}$$

where: A_{sn} is the area of reinforcing bars within the region of $2h_{\mathrm{n}}$ from the centre–line of the composite cross-section.

For rectangular hollow sections, it can be explicitly stated that:

$$W_{\mathrm{pan}} = 2t.h_{\mathrm{n}}^2 \qquad \text{A.7}$$

$$W_{\mathrm{pcn}} = (2a-2t)h_{\mathrm{n}}^2 - W_{\mathrm{psn}} \qquad \text{A.8}$$

Equation A.4 may be used to obtain exact equations for circular and elliptic sections. However, the resulting equations will be complex. As an alternative, equations A.7 and A.8 can be applied to circular and elliptical sections with a high accuracy by substituting diameter d (for CHS) or dimension 2a (for EHS) for breadth b.

A.2 Point E

For the calculation of the resistances at the additional point E, $N_{E,Rd}$ and $M_{E,Rd}$ (see above), the neutral axis is located between h_n and the border of the section, so that $h_E = 0.5h_n + 0.25h$. Using rectangular hollow sections, the axial resistance of the column for this case is:

$$N_{E,Rd} = b(h_E - h_n) f_{cd} + 2t(h_E - h_n)(2 f_{yd} - f_{cd}) + A_{sE}(2 f_{sd} - f_{cd}) + N_{pm,Rd} \qquad \text{A.9}$$

where A_{sE} is the sum of the areas of reinforcement lying in the additional compression region between h_E and h_n.

The magnitude of $M_{E,Rd}$ is calculated from the above equations but with h_E substituted for h_n in the values of W_{pan} and W_{pcn}.

Again, the above equations can be applied to circular and elliptical sections by substituting diameter d (for CHS) or dimension 2a (for EHS) for breadth b but may become highly over-conservative. In such circumstances it may be preferable to simply apply a linear interpolation between points A and C.

A.2 Point D

The plastic neutral axis coincides with the centre of the composite cross-section, therefore:

$$W_{pcn} = W_{pan} = W_{psn} = 0$$
$$M_{max,Rd} = f_{yd}W_{pa} + 0.5 f_{cd}W_{pc} + f_{sd}W_{ps} \qquad \text{A.10}$$

APPENDIX B Thermal and mechanical properties of steel and concrete

The design software requires the following input data for the material properties for steel and concrete:

- Thermal properties

For both steel and concrete, the required thermal properties are thermal conductivity (λ), specific heat (c_p) and density (ρ).

- Mechanical properties

For both steel and steel reinforcement, the required mechanical properties are the retention factors for the yield strength (k_y) and Young's modulus of elasticity (k_E) as functions of time.

For concrete, the required mechanical properties are the retention factors for the peak stress (k_c) and the concrete strain (ε_{cu}) at the peak stress, both as functions of the concrete temperature. Using these two factors, the retention factor for the secant modulus of elasticity ($k_{Ec,T}$) of concrete at temperature θ can be calculated as:

$$k_{Ec,T} = k_c \frac{\varepsilon_{cu,20}}{\varepsilon_{cu,\theta}}$$

where $\varepsilon_{cu,\theta}$ and $\varepsilon_{cu,20}$ are the concrete strain at peak stress at temperature θ and at ambient temperature respectively.

The following data give the thermal and mechanical properties of different types of steel and concrete as functions of temperature.

B.1 Steel (BS EN 1993-1-2[11], BS EN 1994-1-2[13])

B.1.1 Thermal properties

The specific heat of steel c_a [in J/kg·K] can be determined by:

$$c_a = 425 + 7.73\times10^{-1}\theta_a - 1.69\times10^{-3}\theta_a^{\;2} + 2.22\times10^{-6}\theta_a^{\;3}, \text{ for } 20°\text{C} \le \theta_a < 600°\text{C}$$

$$c_a = 666 + 13002/(738-\theta_a), \quad \text{for } 600°\text{C} \le \theta_a < 735°\text{C}$$

$$c_a = 545 + 17820/(\theta_a - 731),$$ for 735°C ≤ θ_a < 900°C

$$c_a = 650,$$ for 900°C ≤ θ_a ≤ 1200°C

The thermal conductivity of steel λ_a [in W/m·K] can be determined by:

$$\lambda_a = 54 - 3.33 \times 10^{-2}\theta_a,$$ for 20°C ≤ θ_a < 800°C

$$\lambda_a = 27.3,$$ for 800°C ≤ θ_a < 1200°C

The density of steel is 7850 kg/m^3.

B.1.2 Mechanical properties

Table B.1 gives the retention factors for the effective yield strength and modulus of elasticity of steel at elevated temperatures.

Table B.1 Retention factors for yield strength and modulus of elasticity of steel at elevated temperatures

Steel temperature θ_a (°C)	Reduction factor for effective yield strength $k_{y,\theta} = f_{y,\theta}/f_y$	Reduction factor for the slope of the linear elastic range $k_{E,\theta} = E_{a,\theta}/E_a$
20	1.000	1.000
100	1.000	1.000
200	1.000	0.900
300	1.000	0.800
400	1.000	0.700
500	0.780	0.600
600	0.470	0.310
700	0.230	0.130
800	0.110	0.090
900	0.060	0.0675
1000	0.040	0.0450
1100	0.020	0.0225
1200	0.000	0.0000

B.2 Normal strength concrete: normal weight concrete

The concrete strength class ranges from C12/15 to C50/60.

The density of the normal weight concrete may be taken as 2400 kg/m^3.

B.2.1 Thermal properties

The specific heat of dry concrete $c_{c,\theta}$ [in J/kg·K] (i.e. moisture content by weight u = 0%) can be determined by:

$c_c = 900 + 80\ (\theta_c / 120) - 4(\theta_c / 120)^2$ J/kgK for $20°C \leq \theta_c \leq 1200°C$

Where the moisture content u is not considered explicitly in analysis, the specific heat of concrete may be modelled by peak value between 100°C and 200°C as given below:

$$c_c^* = \begin{cases} 1470 & \text{for } u = 1.5\% \\ 2020 & \text{for } u = 3.0\% \\ 5600 & \text{for } u = 10.0\% \end{cases}$$

Figure B.1 shows how the peak value may be implemented.

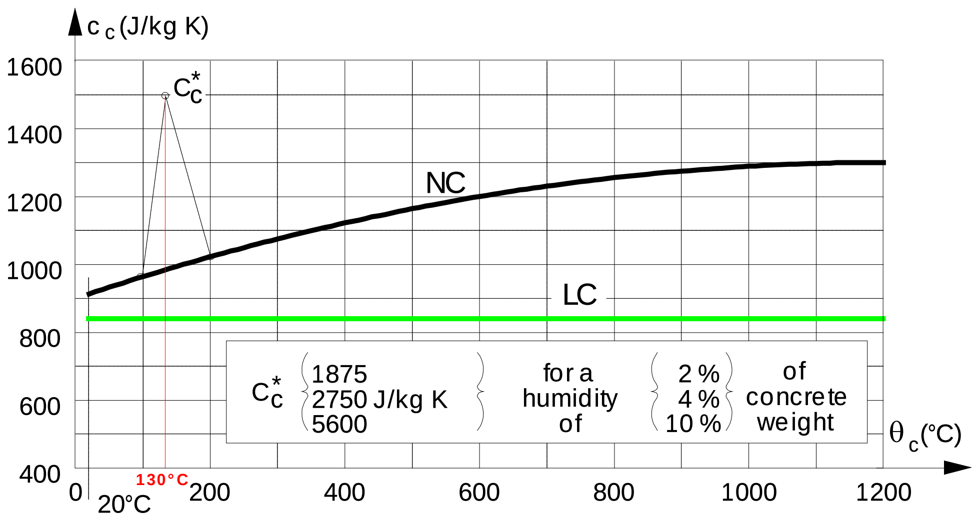


Figure B.1 Specific heat of concrete as a function of temperature

The value of u = 10.0% may occur for hollow sections filled with concrete. For other moisture contents, linear interpolation between the above given values is acceptable.

The above thermal properties are for siliceous aggregates. For calcareous aggregates, the same properties may be used, which is normally on the safe side. If more precise information is needed, reference is made to section A.1.3, Appendix 1, of BS EN 1992-1-2[9].

The thermal conductivity of concrete λ_c [in W/m·K], for 20°C ≤ θ_c ≤ 1200°C, can be determined as follows:

$$\lambda_c = 2 - 0.24\ (\theta_c / 120) + 0.012\ (\theta_c / 120)^2 \ \text{W/mK}$$

B.2.2 Mechanical properties

Table B.2 gives the values for the retention factor of peak stress and for the concrete strain at peak stress at different elevated temperatures.

Table B.2: Values for the retention factor for peak stress and for strain at peak stress of normal weight concrete (NC) and light weight concrete (LC) at elevated temperatures.

Concrete Temperature θ_c [°C]	$k_{c,\theta} = f_{c,\theta} / f_{c,20°C}$		$\varepsilon_{cu,\theta} \times 10^3$
	NC	LC	NC
20	1	1	2,5
100	0,95	1	3,5
200	0,90	1	4,5
300	0,85	1	6,0
400	0,75	0,88	7,5
500	0,60	0,76	9,5
600	0,45	0,64	12,5
700	0,30	0,52	14,0
800	0,15	0,40	14,5
900	0,08	0,28	15,0
1000	0,04	0,16	15,0
1100	0,01	0,04	15,0
1200	0	0	15,0

B.3 Normal strength concrete: lightweight concrete

B.3.1 Thermal properties

The density of unreinforced lightweight is in the range of 1600 to 2000 kg/m^3.

The specific heat c_c of light weight concrete may be considered to be independent of the concrete temperature:

c_c = 840 J/kgK

The thermal conductivity λ_c of light weight concrete may be determined from the following:

$\lambda_c = 1{,}0 - (\theta_c / 1600)$ W/mK for $20°C \leq \theta_c \leq 800°C$

$\lambda_c = 0{,}5$ W/mK for $\theta_c > 800°C$

B.3.2 Mechanical properties

See Table B.2.

B.4 Cold worked reinforcing steel

The thermal properties of cold worked reinforcing steel may be taken as the same as those of structural steel.

Table B.3 gives the values of the retention factors for the effective yield strength and modulus of elasticity of cold worked reinforcing steel at elevated temperatures.

Table B.3: Values for the retention factors for the effective yield strength and modulus of elasticity of cold worked reinforcing steel at elevated temperatures

Steel Temperature θ_S [°C]	$k_{y,\theta}$	$k_{E,\theta}$
20	1,00	1,00
100	1,00	1,00
200	1,00	0,87
300	1,00	0,72
400	0,94	0,56
500	0,67	0,40
600	0,40	0,24
700	0,12	0,08
800	0,11	0,06
900	0,08	0,05
1000	0,05	0,03
1100	0,03	0,02
1200	0,00	0,00

APPENDIX C Results of finite element heat transfer analysis

Assumptions

Column length=2 m

Model exposed to standard fire up to 90 minutes (5400 sec)

Stefan-Boltzmann constant=5.669e-8 W/m^{2}K

Absolute zero=-273.15°C

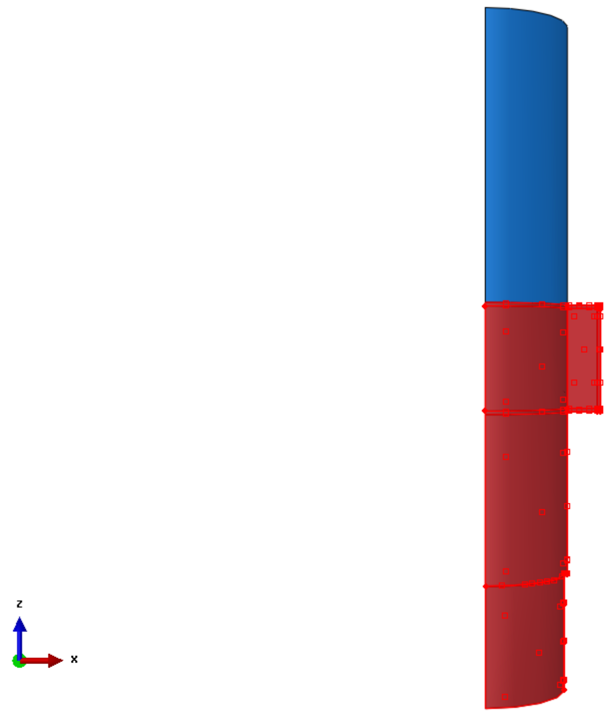
Emissivity on exposed side=0.3

Convection heat transfer coefficient=25 W/m^{2}K

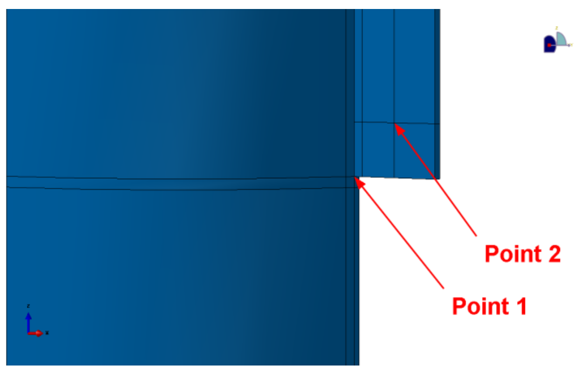
Element size: Through the section=0.005

Through the length=0.01

Element type: DC3D8 (An 8-node linear heat transfer brick)



Exposure surface (red colour/dotted surface)



Interested points (1 and 2)

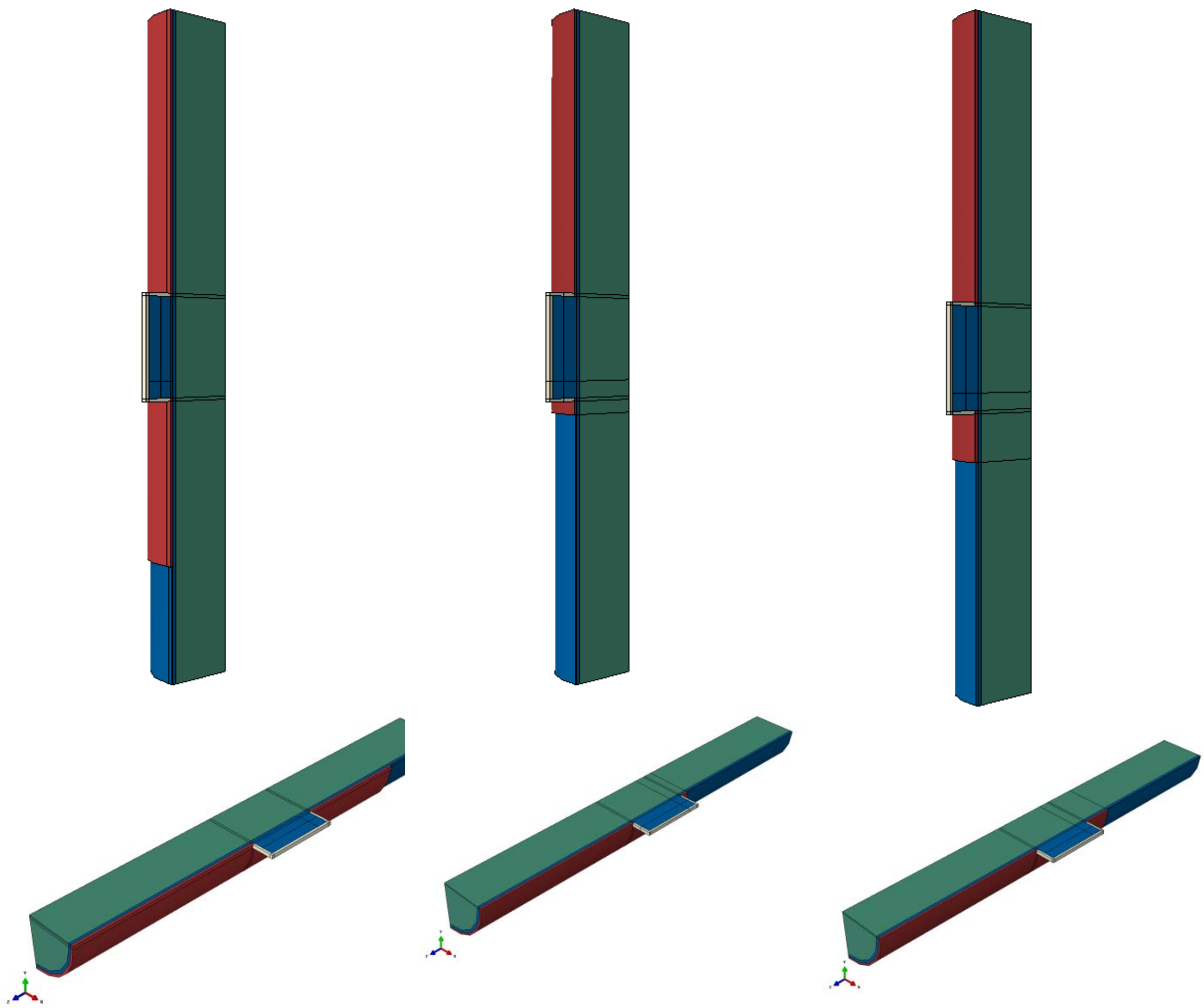
Note:
The thermal conductivity of fire protection was assumed to give temperatures on point 1 and 2 around 510°C when fire protection was applied on the whole model.

Cases 1 to 3

CHS: 457x12.5 mm

Fin plate: 300x100x12.5 mm

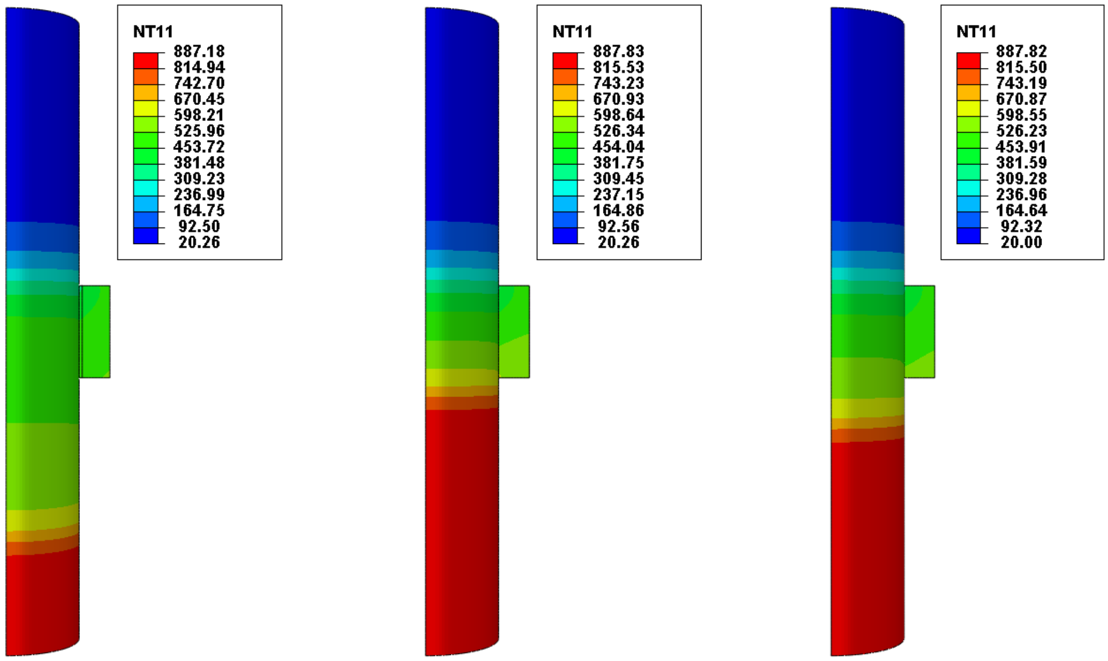
Models



CASE 1 (coat back =500 mm) **CASE 2 (coat back = 50 mm)** **CASE 3 (coat back = 150 mm)**

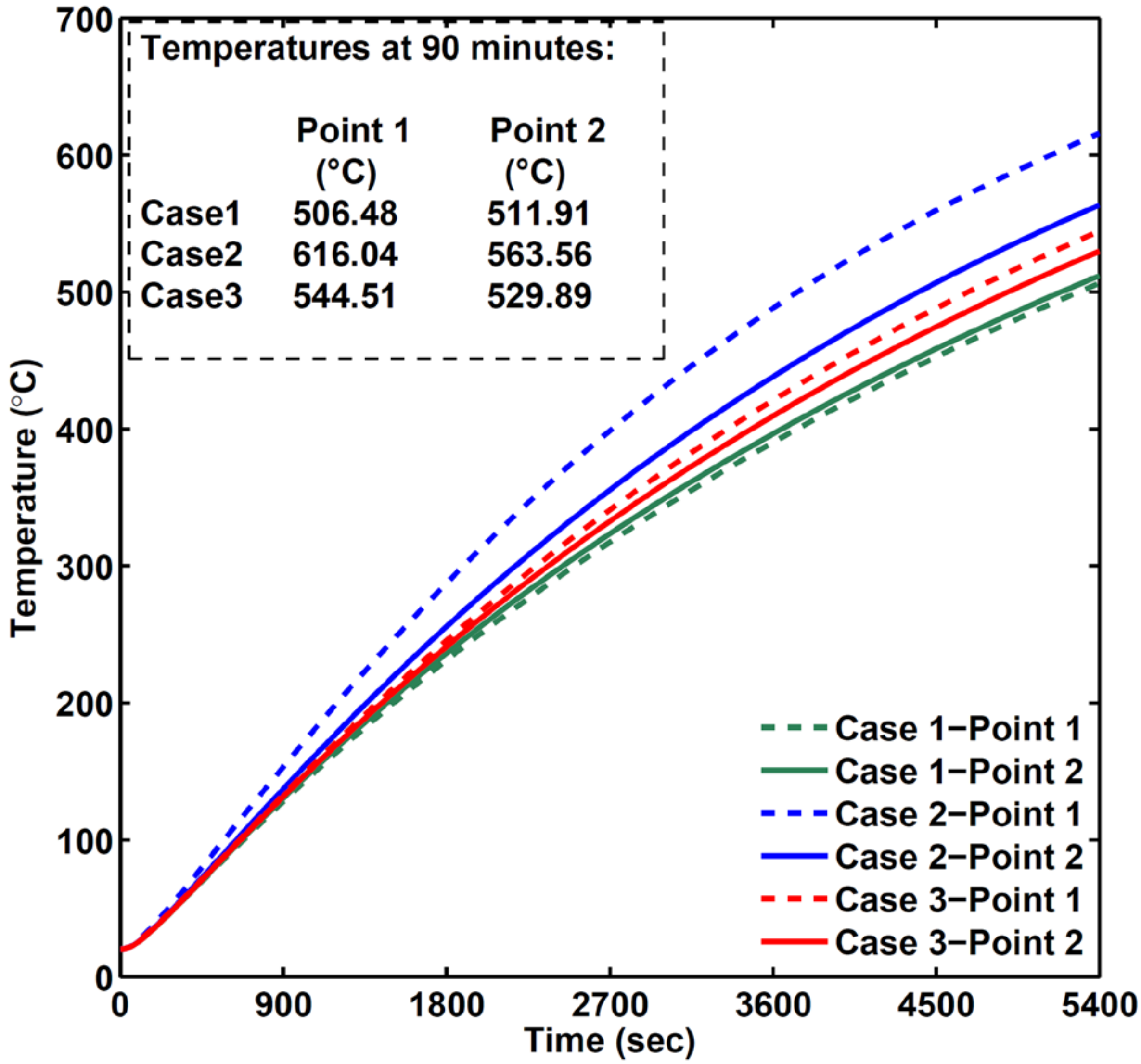
Labels:
Red: Fire protection on column (t=10 mm, kp=0.31)
Grey: Fire protection on fin plate (t=10 mm, kp=0.055)
Green: Concrete (based on BS EN 1994-1-2[13])
Blue: Steel (based on BS EN 1993-1-2[11])

Results



CASE 1 (coat back=500 mm) **CASE 2 (coat back =50 mm)** **CASE 3 (coat back=150 mm)**

Temperature distribution on steel tube



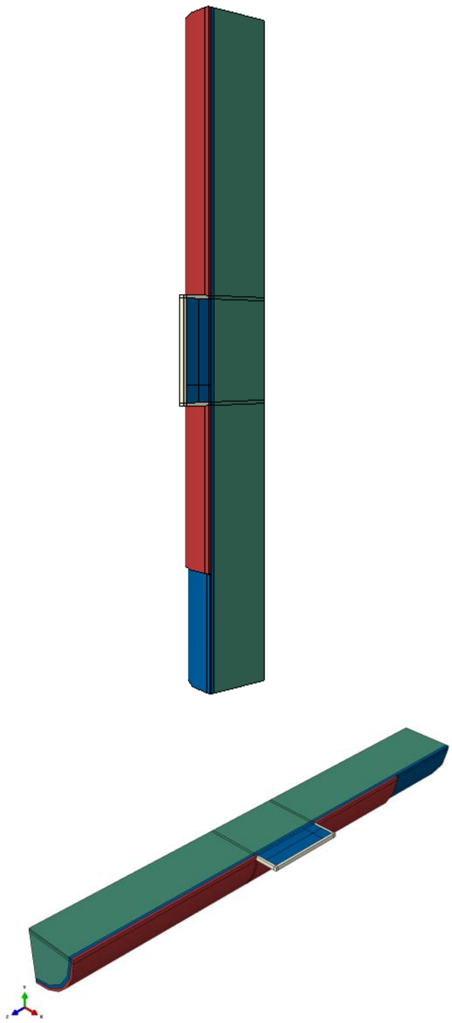
Temperature – time curves

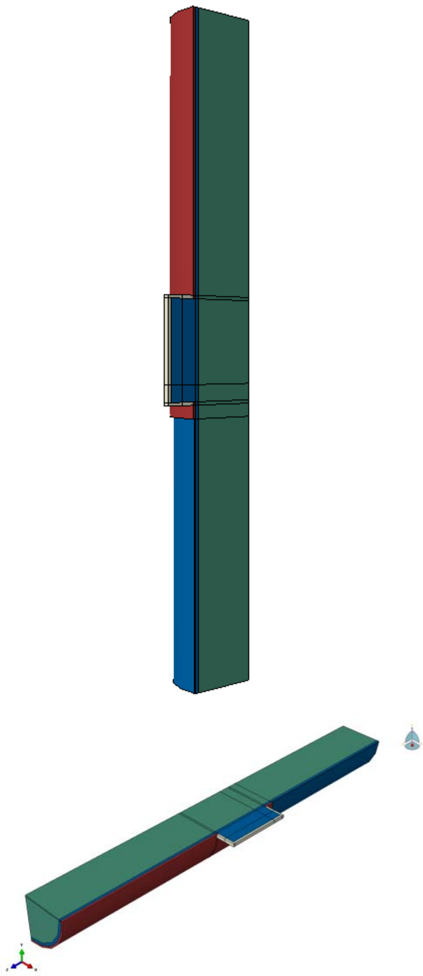
Cases 4 and 5

CHS: 457x16 mm

Fin plate: 300x100x12.5 mm

Models

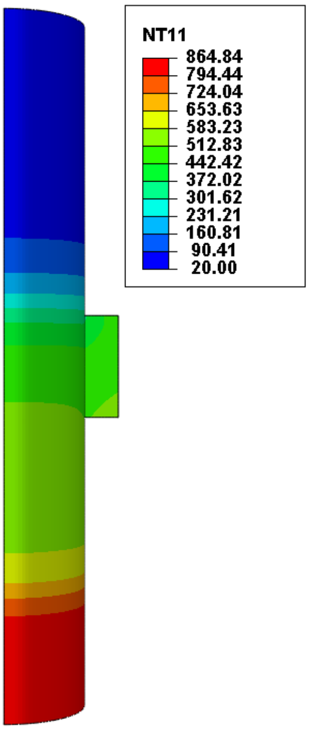


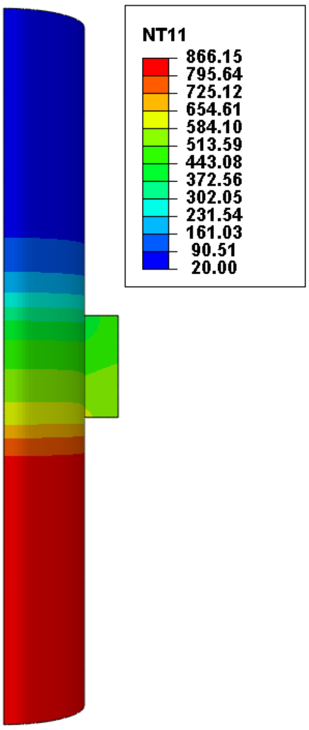


CASE 4 (coat back=500 mm) **CASE 5 (coat back=50 mm)**

Labels:
Red: Fire protection on column (t=10 mm, kp=0.35)
Grey: Fire protection on fin plate (t=10 mm, kp=0.05)
Green: Concrete (based on BS EN 1994-1-2[13])
Blue: Steel (based on BS EN 1993-1-2[11])

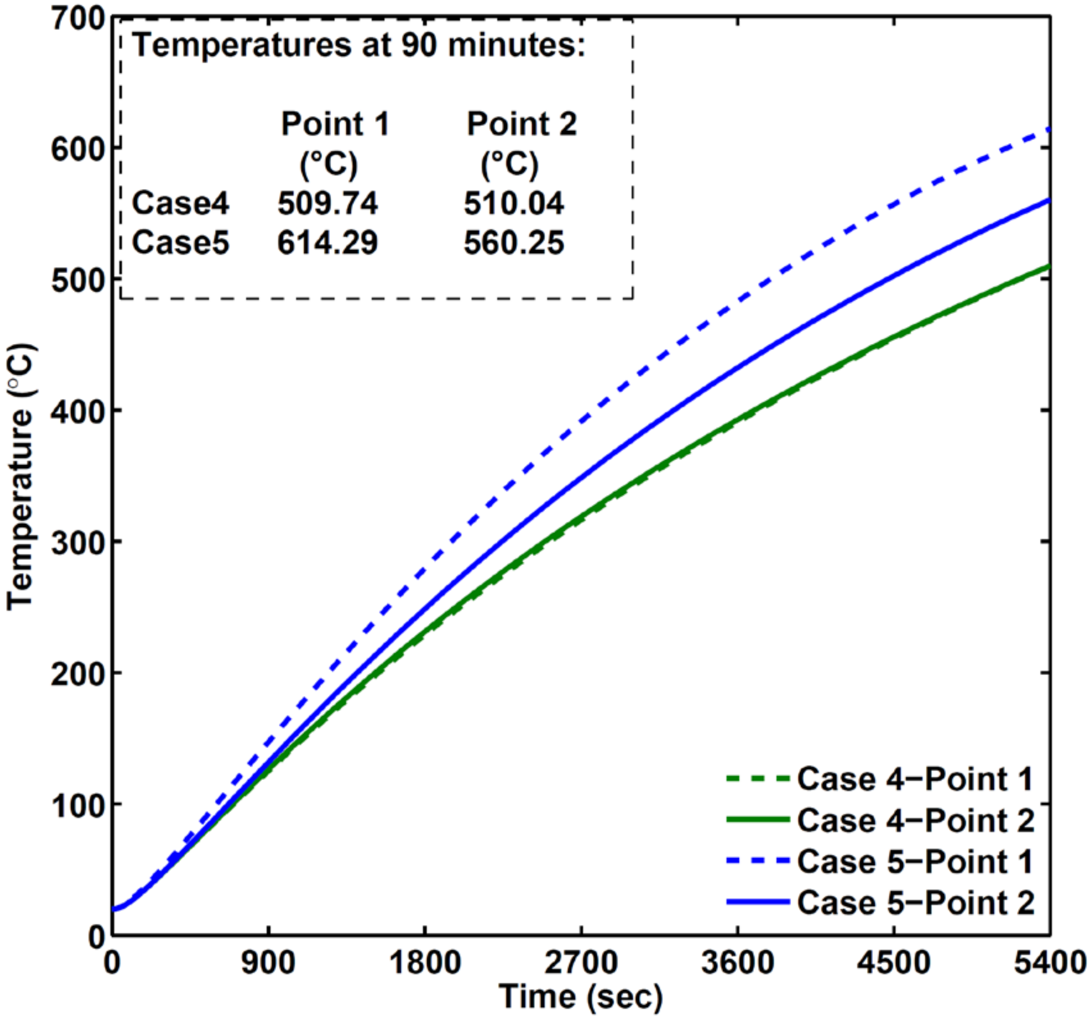
Results





CASE 4 (coat back =500 mm) **CASE 5 (coat back=50 mm)**

Temperature distribution on steel tube



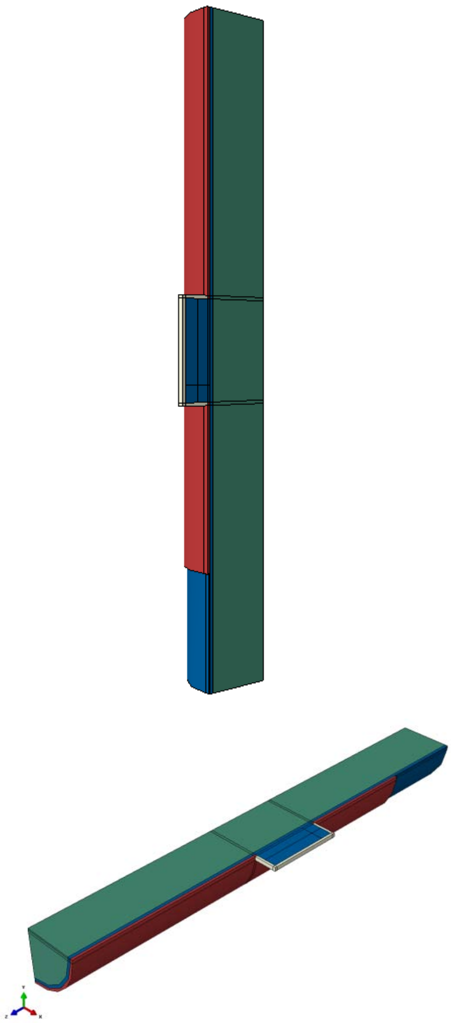
Temperature – time curves

Cases 6 and 7

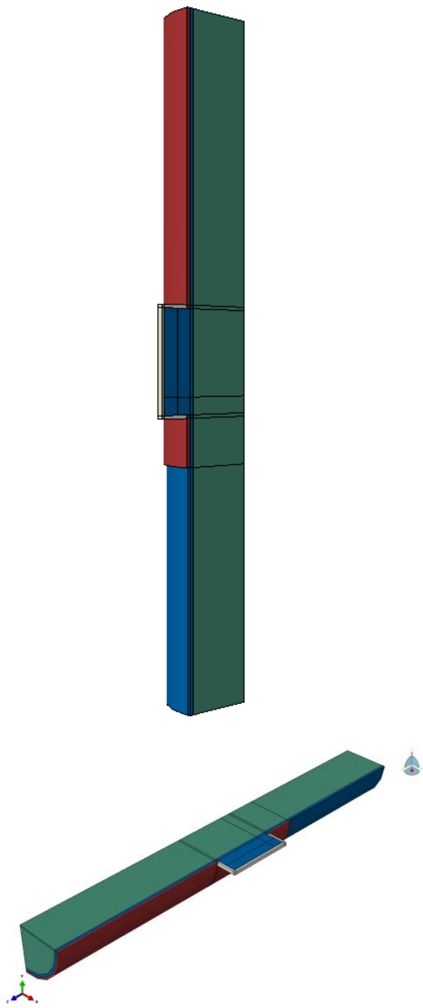
CHS: 323.9x12.5 mm

Fin plate: 300x100x12.5 mm

Models



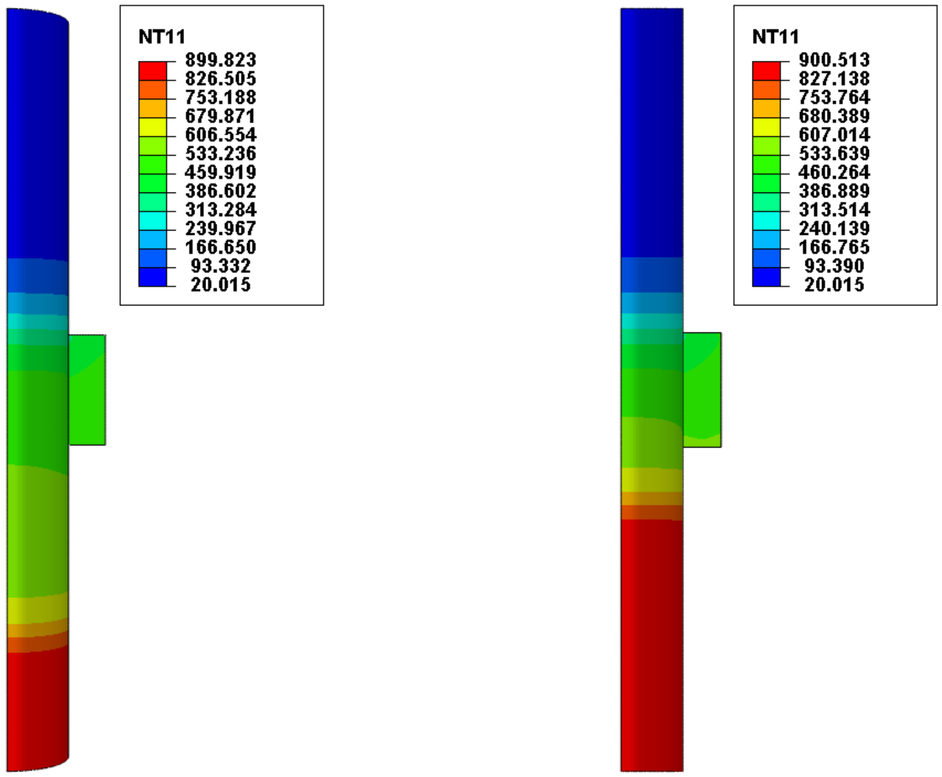
CASE 6 (coat back =500 mm)



CASE 7 (coat back=150 mm)

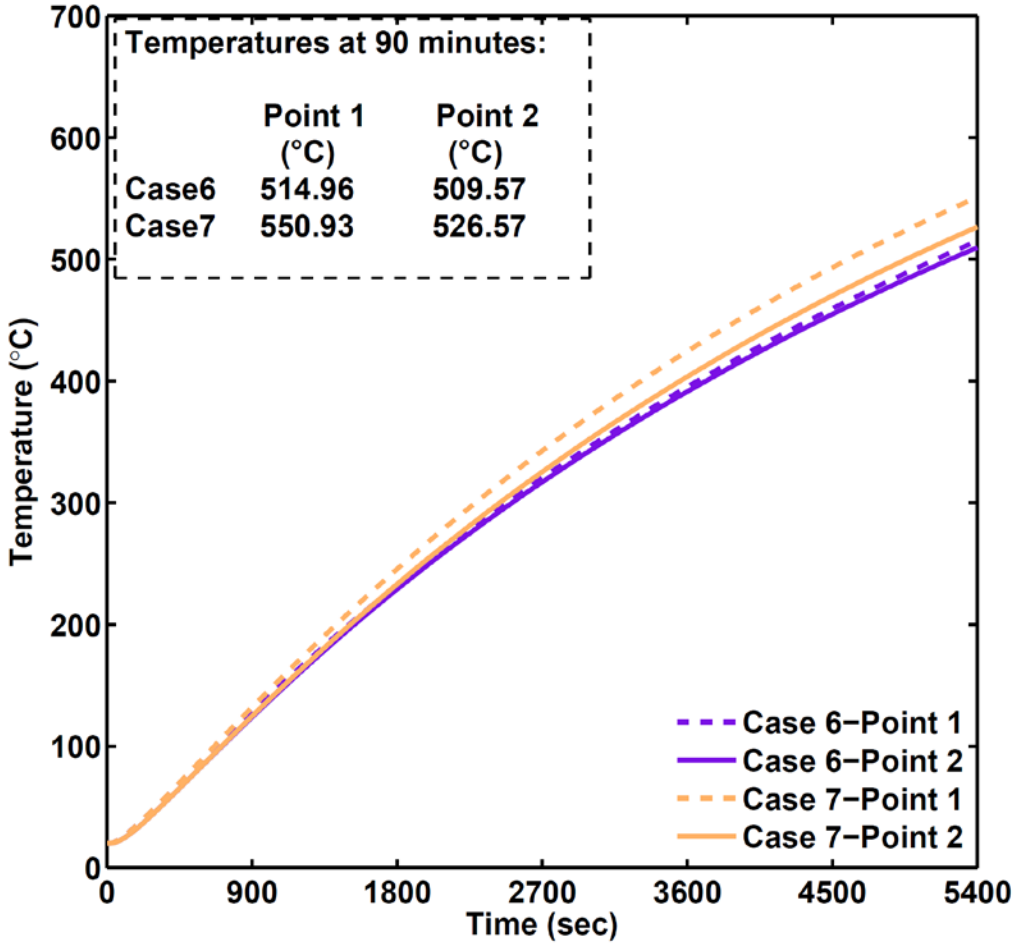
Labels:
Red: Fire protection on column (t=10 mm, kp=0.3)
Grey: Fire protection on fin plate (t=10 mm, kp=0.045)
Green: Concrete (based on BS EN 1994-1-2[13])
Blue: Steel (based on BS EN 1993-1-2[11])

Results



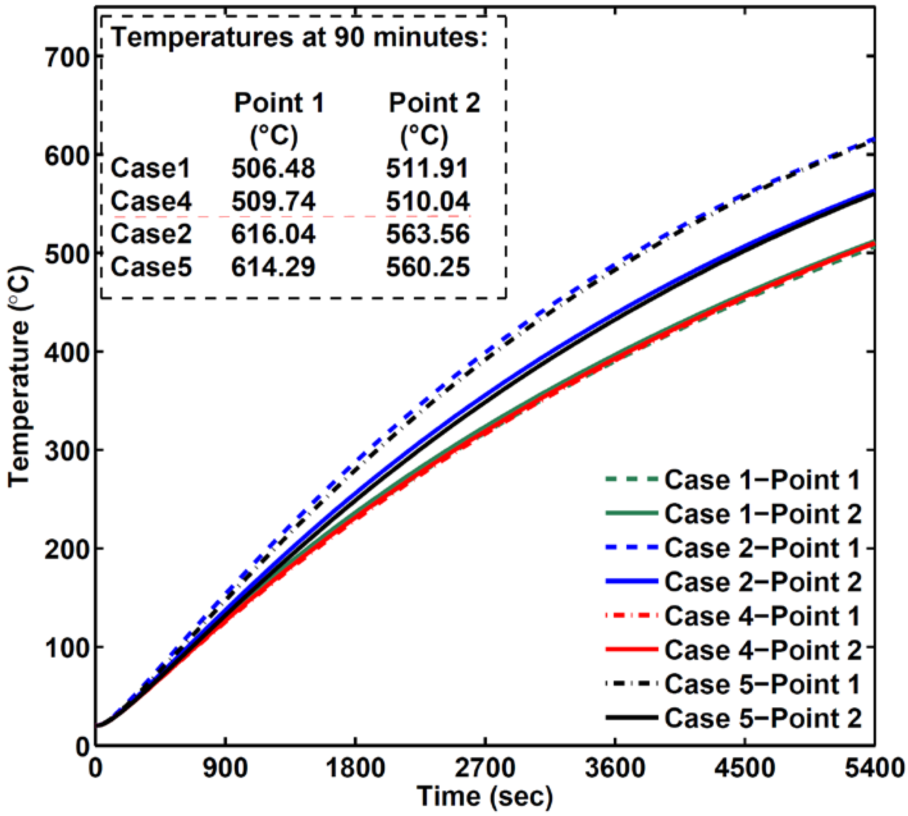
CASE 6 (coat back=500 mm) **CASE 7 (coat back =150 mm)**

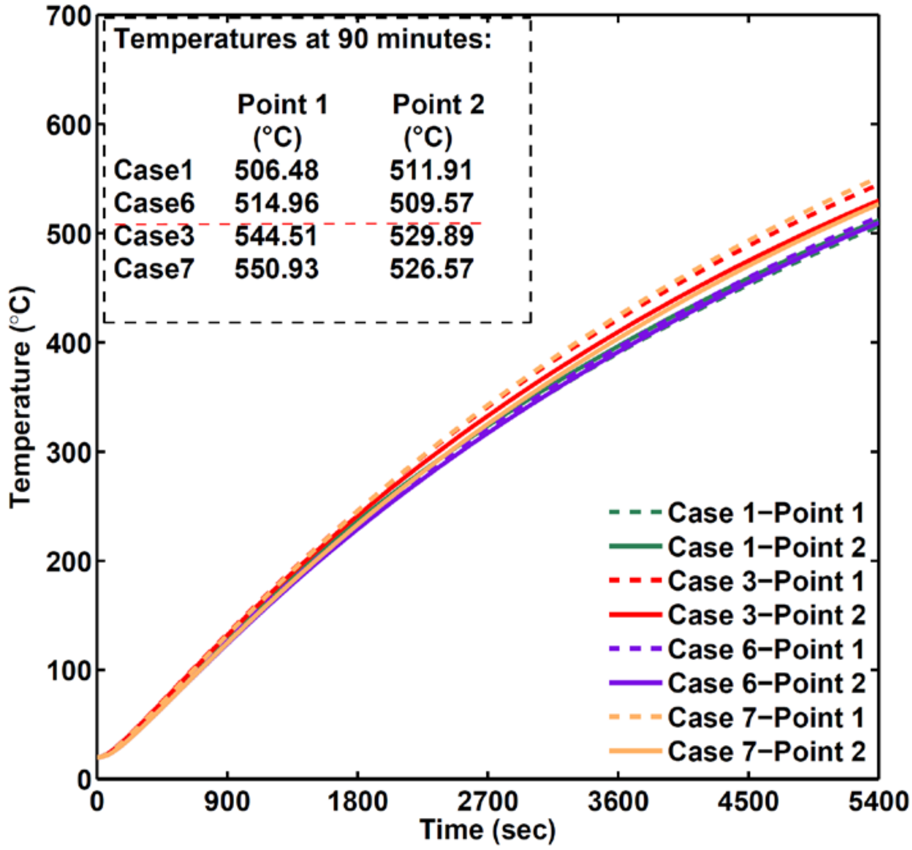
Temperature distribution on steel tube

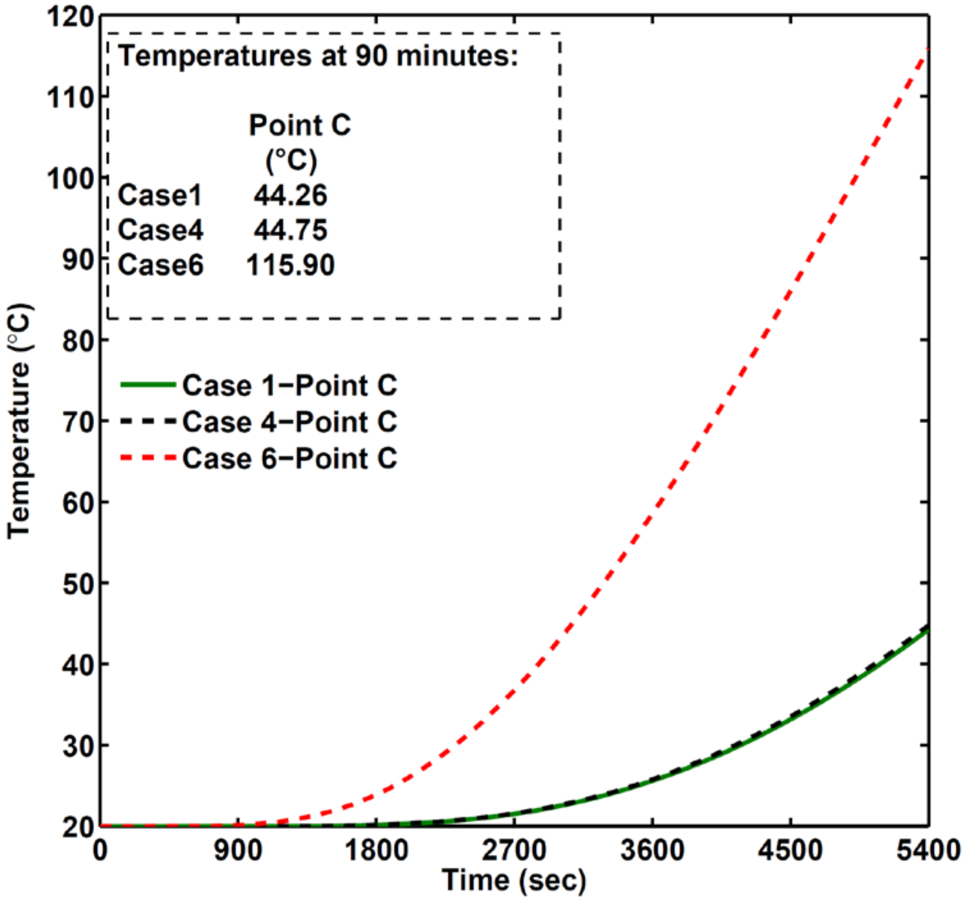


Temperature – time curves

Comparison of results







Temperatures at 90 minutes:
Point C
(°C)
Case1 44.26
Case4 44.75
Case6 115.90
Case 1−Point C
Case 4−Point C
Case 6−Point C
Temperature (°C)
Time (sec)
120
110
100
90
80
70
60
50
40
30
20
0
900
1800
2700
3600
4500
5400