CONCRETE-FILLED STEEL COLUMNS

PREDICTION OF FIRE RESISTANCE

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Introduction

The fire resistance of structural steel hollow section (SSHS) columns can be enhanced through:

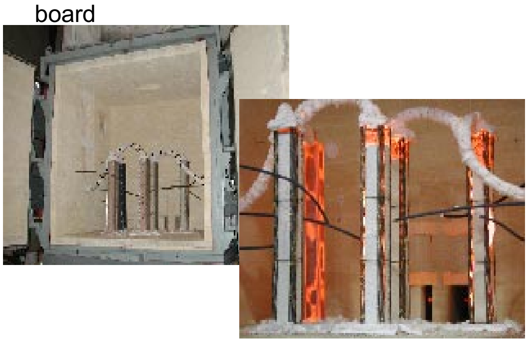
(a) encasing the section within a fire-protective coating
(b) filling with unreinforced concrete
(c) filling with fibre-reinforced concrete
(d) filling with conventional reinforced concrete

In the case of (a) the thickness of fire protection can be determined as for any steel section and reference is made to the Handbook of Fire Protection Materials for Structural Steel [1] published by Australian Institute of Steel Construction (AISC), or alternatively, using software available from the Fire Safety Design Compendium CD [2] published by OneSteel.

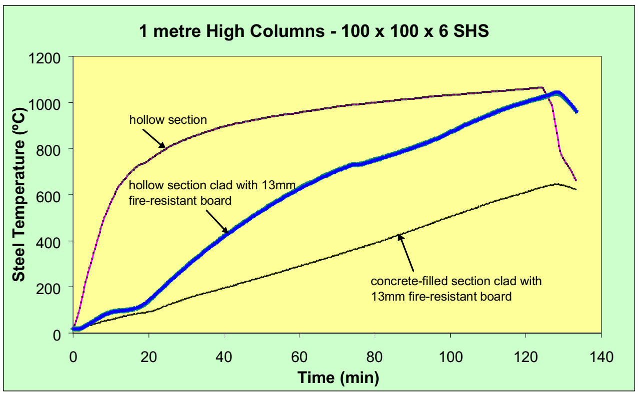
The above thicknesses do not take into account the benefits of concrete-filling. A combination of (a) and (b) will result in much lower thicknesses of fire protection as illustrated by testing conducted recently at Victoria University of Technology.

These tests were carried out on short one metre high columns of 100 x 100 x 6 SHS. The specimens were placed in a standard fire test furnace and subjected to heating under standard fire test conditions for up to 120 minutes in duration. The main specimens tested were:

- hollow section
- hollow section clad with 13mm fire-resistant board
- concrete-filled section clad with 13mm fire-resistant board



The temperatures achieved in the steel sections for the various specimens are shown below.

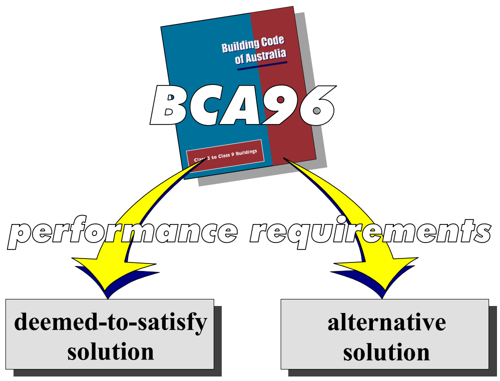


It can be seen that the presence of concrete filling greatly increases the fire-resistance of an externally protected member over that associated with an identical member that is not concrete-filled. The reduction in steel temperature due to concrete filling is only apparent for externally protected members[1].

If hollow sections are specified for architectural reasons, it is unlikely that protection of the outside of the tube will be an acceptable means of achieving the required fire-resistance level[2]. It follows that enhancing the fire resistance of a section through concrete-filling presents itself as an attractive option. The majority of hollow steel sections used in building construction are not large in cross-section and the placement of conventional reinforcement within these sections is unlikely to be cost-effective. Therefore, (b) and (c) are the preferred ways of enhancing the fire resistance of SSHS columns. This technical note provides information to allow designers to assess the fire resistance of SSHS columns when filled with either plain or fibre-reinforced concrete. In the context of this technical note the term "fire-resistance" should be taken as representing the performance of a column tested under standard fire test conditions [3].

The fire resistance thus established must be greater than or equal to the fire-resistance level (FRL) *required* for the particular situation being considered. But what fire-resistance level is required?

If the building is being assessed according to the deemed-to-satisfy provisions of the BCA [4], then the required fire-resistance levels[3] are specified. However, if aspects of the building are assessed from a fire-engineering viewpoint, then a lesser FRL may be appropriate and this can be proposed as part of an *Alternative Solution* for that particular building[4]. The *Alternative Solution* must satisfy the BCA performance requirements.



1 This phenomenon is not apparent for members protected with intumesecent coatings due to the mechanisms required to activate such coatings
2 Protection of the outside of the tube with an intumescent coating will allow the shape to remain
3 See for example, Table 3 of Specification C1.1 of the BCA
4 This will require a fire-safety engineer to undertake an assessment of the building situation being considered

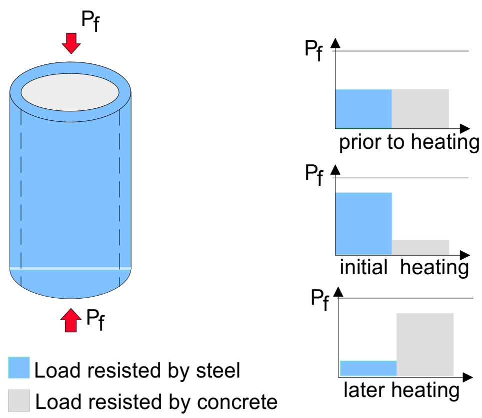
firedesignnote

Key Aspects of Behaviour

Before presenting the various approaches to assessing the fire resistance of concrete-filled SSHS columns it is helpful to appreciate some of the key aspects of the behaviour of these columns in fire.

The effect of heating an *unloaded* concrete-filled SSHS column is now considered. During the early stages of heating, the steel section will be hotter than the concrete core[5] and will therefore attempt to expand relative to the core. However if there is sufficient bond between the core and the tube, the tube will be restrained by the core and will go into compression, whilst the core will be subject to an equal tensile force. If the core has sufficient tensile strength due to its cross-sectional area (the tensile strength of concrete is relatively low) or reinforcement (if present), then the steel section will yield in compression given sufficient temperature increase. If the core does not have sufficient tensile strength to resist the force being developed within the steel section, then it will crack, relieving the tensile force within the concrete and allowing the tube to freely expand.

Under *normal temperature* conditions, compressive loading on the columns will be shared between the concrete core and the steel section in proportion to the relative stiffnesses of each part—although the load carried by the steel section will be higher if the loads during construction are carried by the steel section prior to concrete filling. A rise in temperature of the steel section will result in a further increase in compressive force within the steel section such that it may reach its yield or squash load. However, as the temperature further increases, the squash load capacity of the tube is progressively reduced and more load is transferred to the concrete core. In the limit, the concrete core will resist the entire load provided it has sufficient strength.



It is the ability of the concrete core to carry load when the steel tube cannot that is the main benefit provided by concrete filling.

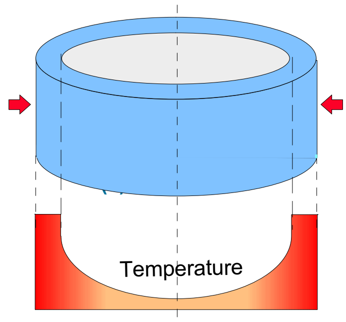
In the fire situation, it should be noted that neither the thickness of the SSHS, nor the yield stress of the section, need to be included in the assessment of the ability of the concrete to carry load .

The ability of the concrete core to resist load in the fire situation is influenced by a number of factors. These are now discussed.

i) reinforcement
The stability and strength of a concrete column is enhanced by the presence of reinforcement since this gives both flexural and axial stiffness and strength. It follows that higher levels of fire resistance will be able to be achieved if reinforcement is present. The greatest improvement in fire resistance is achieved with conventional reinforcement but testing has demonstrated that steel fibres can also provide some enhancement.

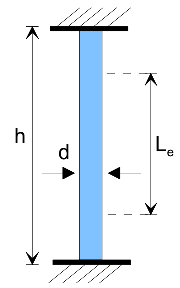
ii) cross-sectional dimensions
During later stages of heating a significant temperature gradient develops within the concrete core with the outer parts of the core being hotter than the inner parts. As the width or diameter increases, the proportion of cross-section affected by temperature rise becomes less for a given fire exposure. The capacity is also increased.

It follows that for a given load level (see (iii)), an increase in cross-sectional dimensions will give an increase in fire resistance. Alternatively, for a given fire exposure, the load able to be resisted for that period will be greater for columns of greater cross-



iii) load level
The higher the load level, the lower the fire resistance achieved. The load level is usually expressed as the ratio of applied load to ambient temperature strength.

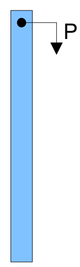
iv) slenderness
The slenderness of a column is normally expressed as a ratio (the slenderness ratio) and is taken as the effective length of the column divided by its width, diameter or other such dimension representing the minimum cross-sectional dimension of the member. As the slenderness ratio becomes larger, the fire resistance reduces, all other factors being equal.



slenderness ratio = L_e/d

It follows from the above discussion that from a fire resistance viewpoint *it will always be better to utilise a SSHS with a low wall thickness and greater cross-sectional dimension than one with smaller cross-sectional dimension and thicker walls* (but having the same area of steel).

v) eccentricity of load
The presence of eccentric loading will significantly reduce the fire resistance of an unreinforced concrete tube. This is due to the fact that if the steel tube has little flexural resistance due to its temperature all of the bending resistance must be provided by the unreinforced core - and such cores will offer little resistance. It follows that *unreinforced SSHS sections should not be used in situations subject to high eccentricity of load unless allowance is made for such loading.*



This matter is further considered later in this publication.

5 This is due to the relatively higher thermal conductivity of steel compared with concrete

Assessment of Fire Resistance

Basis

According to Specification A2.3 of the BCA, the fire resistance of an element of construction can be determined from:

- a standard fire test result—recorded and reported by a qualified organisation
- a method of calculation—provided all of the relevant parameters are considered

On this basis, this technical note presents:

(1) standard fire test results
(2) published calculated performances
(3) calculation methods

In the case of approaches (1) and (2) specific guidelines are given with respect to how these results should be applied to the particular situation being assessed. These guidelines are consistent with the requirements of Specification A2.3 of the BCA. *This technical note only gives information for concrete-filled SSHS columns in buildings where the columns are predominantly axially loaded.*

Approach 1: Use of Test Results

Standard fire tests have been conducted on concrete-filled SSHS columns at various laboratories throughout the world [5 - 8]. Each of these tests was conducted under conditions identical to those required by AS1530.4. The test results are summarised in Table 1 and cover columns constructed using concrete made from siliceous aggregates—the predominant type of aggregate used in Australia.

Table 1

Test No.	dim (mm)	f'_c (act) (MPa)	eff. length (mm)	load (kN)	time (mins)
Circular columns					
(NRC) C-01	141.3	33.1	2695	110	55
(NRC) C-02	141.3	31	2695	131	57
(NRC) C-03	168.3	32.7	2667	150	76
(NRC) C-04	168.3	32.7	3810	150	60
(NRC) C-05	168.3	35.5	2667	218	56
(NRC) C-06	168.3	35.4	2667	150	81
(CSTB) C-07	168.3	50	2520	300	56
(IBMB) C-08	168.3	41.3	4060	100	40
(NRC) C-09	219.1	31	2667	492	80
(NRC) C-10	219.1	32.3	2667	384	102
(NRC) C-11	219.1	31.9	3810	525	73
(NRC) C-12	219.1	31.9	3810	525	33
(NRC) C-13	219.1	31.7	2667	525	82
(CSTB) C-14	219.1	49.5	2520	300	102
(CSTB) C-15	219.1	48.2	2520	600	45
(CSTB) C-16	219.1	48.2	2520	600	45
(CSTB) C-17	219.1	50	2520	600	43
(CSTB) C-18	219.1	49.5	2520	900	35
(CSTB) C-19	219.1	41.3	4060	300	39
(NRC) C-20	273.1	28.6	2667	574	112
(NRC) C-21	273.1	29	2667	525	133
(NRC) C-22	273.1	27.2	2667	1000	70
(NRC) C-23	273.1	27.4	2667	525	143
(NRC) C-24	323.9	27.6	2667	699	145
(NRC) C-25	323.9	24.3	2667	1050	93
(CSTB) C-26	323.9	51.6	2520	1800	28
(NRC) C-27	355.6	23.8	2667	1050	111
(NRC) C-28	355.6	25.4	2667	1050	170
(NRC) C-29	406.4	27.6	2667	1900	71
(CTICM) C-30	406.3	30.7	2590	4500	36

Table 1 (*cont.*)

Test No.	dim (mm)	f'_c (act) (MPa)	eff. Length (mm)	load (kN)	time (mins)
Square Columns					
(CSTB) SQ 101	140	47	2520	685	24
(CSTB) SQ 102	140	51.5	2520	410	42
(CSTB) SQ 103	140	40.1	2520	190	66
(CSTB) SQ 104	140	51.6	2520	530	28
(CSTB) SQ 105	140	51.6	2520	530	24
(CTICM) SQ106	150	49	2520	376	81
(CTICM) SQ107	150	45.6	2520	286	80
(NRC) SQ-01	152.4	58.3	2667	376	66
(IBMB) SQ108	160	41.3	4060	100	68
(NRC) SQ-07	177.8	57	2520	549	80
(IBMB) SQ109	180	41.3	4060	200	42
(CTICM) SQ110	200	34.5	2520	950	36
(CTICM) SQ111	200	49	2520	740	80
(CSTB) SQ112	200	49.5	2520	1660	19
(CSTB) SQ113	200	55.8	2520	1240	39
(CSTB) SQ114	200	55.8	2520	740	88
(IBMB) SQ115	200	44.2	3570	397	22
(IBMB) SQ116	200	41.3	4060	300	52
(IBMB) SQ117	220	41.3	5800	490	16
(IBMB) SQ118	220	41.3	4060	800	15
(IBMB) SQ119	220	41.3	4060	800	34
(CTICM) SQ120	225	49	2520	1085	56
(CTICM) SQ121	225	49	2520	1520	42
(CTICM) SQ122	225	49	2520	430	165
(CTICM) SQ123	225	44.5	2520	1970	29
(CSTB) SQ124	225	40.5	2520	1000	36
(CTICM) SQ125	225	44.5	2520	1405	40
(CTICM) SQ126	225	44.5	2520	560	145
(FIRTO) SQ 127	250	57.9	2520	1950	68
(FIRTO) SQ 128	250	47.7	2520	1740	25
(NRC) SQ-17	254	58.3	2667	1096	62
(CTICM) SQ129	260	42.1	2520	1500	45
(CTICM) SQ130	260	41.5	2520	800	86
(CTICM) SQ131	260	42.1	2520	1500	49
(CTICM) SQ132	260	41.8	2520	800	114
(CTICM) SQ133	260	34	2520	800	102
(CSTB) SQ 134	260	41.5	2520	800	98
(IBMB) SQ135	260	41.5	3060	800	81
(FIRTO) SQ 136	260	41.5	2520	800	133
(BAM) SQ 137	260	41.5	2520	800	134
(IBMB) SQ138	260	41.3	4060	1000	51
(CSTB) SQ 139	265	30.2	2520	910	68
(NRC) SQ-24	304.8	58.8	2667	1130	131
(FIRTO) SQ 140	350	47.7	2520	2250	85
(FIRTO) SQ 141	350	47.7	2520	3150	39
(FIRTO) SQ 142	350	47.7	2520	4390	30
(FIRTO) SQ 143	350	48.8	2520	3950	55

Table 1 results may be used to assess the fire resistance of a member provided the following approach is adopted:

Step 1: Identify the *tested* members having the same cross-sectional outer dimensions as the member being considered in the design (hereafter called the *trial design* member)

Step 2: Choose those *tested* members that achieved the required fire resistance

Step 3: From the members chosen in Step 2 choose those that have a concrete strength equal to or less than that intended for the *trial design* member

Step 4: From the members chosen in Step 3 choose those that have an effective length greater than or equal to that of the trial design member

Step 5: From the members chosen in Step 4 choose those for which the test load is greater than or equal to that intended to be applied to the *trial design* member in the fire situation

Step 6 Check that *trial design* member can be considered to be essentially concentrically loaded (see later discussion)

The use of single test results to justify the performance of a design is implicitly accepted by the BCA since only single tests are required by AS1530.4. The results of Table 1 can therefore be directly applied. However, in cases where multiple tests have been done and where more than one fire resistance has been obtained for nominally identical members, it will be prudent for the designer to adopt a lower rather than the highest value.

Example 1:
Consider a *trial design* member: OneSteel 168.3 CHS filled with 40MPa unreinforced concrete. The load applied to member in fire is 120kN and the effective length taken as 2500mm. Minimum fire-resistance to be achieved is 60 minutes.

Step 1: From Table 1 choose (NRC) C-03 to (NRC) C-08

Step 2 Only tested members (NRC) C-03, (NRC) C-04 and (NRC) C06 (ignore thickness) are applicable

Step 3: All tested members chosen in Step 2 are OK since all have concrete strength < 40MPa

Step 4: All tested members chosen in Step 3 are OK since all have an effective length greater than 2500mm

Step 5 All tested members chosen in Step 4 are OK since all are subject to a test load of 150kN

Step 6 Check that *trial design* member can be considered to be essentially concentrically loaded (see later discussion)

It is therefore concluded that the chosen trial design member can be considered to achieve a fire-resistance of 60 minutes, assuming it to be subject to concentric loading.

Approach 2: Use of Tabulated Solutions

In the past many attempts have been made to predict the performance of concrete-filled tubes in fire [5, 9 11]. More recent approaches [8, 12-15, 15-18] have been found to give less optimistic predictions of performance and are based on modelling the member using finite element techniques where temperatures throughout the cross-section are calculated and their effect on the strength and stiffness of the concrete and steel taken into account. Despite the apparent sophistication of these methods it is still difficult to accurately predict the performance of the tested members. Some of these methods have been used to generate tables or graphs of solutions where the fire resistance is presented as a function of member cross-section, concrete strength, effective length and load level. These data have been used in this publication to determine solutions for OneSteel sections, however due to the limitations of the data, only limited solutions are available.

(a) Klinsch and Wittbecker [16]
The following predictions of performance (see Table 2) are based on the analysis conducted by Klinsch and Wittbecker [16] for concentrically loaded unreinforced circular hollow sections.

Table 2

CHS dia (mm)	f'_c (MPa)	eff. Length (mm)	load (kN)	resistance (mins)
139.7	35	2000	55	> 60
139.7	35	3000	46	> 60
165.1	35	2000	90	> 60
165.1	35	3000	64	> 60
168.3	35	2000	95	> 60
168.3	35	3000	67	> 60

These results may be used to assess whether a trial design member can achieve a fire-resistance of at least 60 minutes. The following steps should be followed:

Step 1: Identify the members in Table 2 having the same or *lesser* outer cross-sectional dimensions as the *trial design* member

Step 2: Check that the effective length of the *trial design* member falls less than or between the limiting values. Otherwise results are not applicable.

Step 3: Use linear interpolation to determine the maximum load if the effective length is between the limiting values. If the effective length is less than the lower limiting value, then the load corresponding to this value should be adopted.

Step 4: Choose the *highest* of the loads obtained from Step 3. This is the maximum load that could be applied to the column to achieve the given level of fire resistance.

Step 5: Check that the concrete strength for the trial design member is at least that given for the quoted analysis results.

Step 6: Check that trial design member can be considered to be essentially concentrically loaded (see later discussion).

Example 2:
Consider a *trial design* member: OneSteel 139.7 CHS filled with unreinforced concrete with an effective length of 2500mm.

Step 1: From Table 2 choose calculated results corresponding to 139.7 CHS sections.

Step 2 Check effective length is less than those is Table 2

Step 3: Use linear interpolation to get maximum load that can be applied in fire situation:

Max load = 55-[(55-46)x(2500-2000)/(3000-2000)]
= 50.5kN

Step 4: *50.5kN* is the maximum load that can be applied for a fire-resistance of at least 60 minutes

Step 5: Trial design member must have a concrete strength with $f'_c \geq 35$MPa.

Step 6: Check that trial design member can be considered to be essentially concentrically loaded (see later discussion).

(b) Australian Institute of Steel Construction [20]
Based on an advanced analysis undertaken by O'Meagher [8], the following solutions are given for columns having an effective length of 2000mm or less. Table 3 below gives the maximum concentric loads that may be applied to concrete-filled circular hollow sections (CHS) so that a fire-resistance of at least 60 minutes will be achieved.

Table 3

CHS diameter (mm)	f'_c (MPa)	Max fire load (kN) to achieve fire resistance of at least 60 mins
219.1	25	300
273.1	25	700
323.9	25	1200
355.6	25	1550
406.4	25	2200
457	25	2925
508	25	3750
610	25	5750
219.1	40	450
273.1	40	1050
323.9	40	2000
355.6	40	2400
406.4	40	3400
457	40	4600
508	40	5900
610	40	9100

The procedure for checking the adequacy of a *trial design member* is the same as that given in (a).

(c) Finnish Constructional Steelwork Association [19]
Based on an advanced analysis similar to that undertaken by O'Meagher, the following solutions (see Table 4) have been published for unreinforced concrete-filled square hollow sections for fire resistance periods of 30 and 60 minutes. The predictions are for both SHS and CHS sections filled with concrete having an f'_c of 40MPa.

Table 4

outer dimension (mm)	eff. length (mm)	load (kN)	resistance (mins)
Predicted Performance - SHS			
125	2000	150	> 30
125	3500	70	> 30
150	2000	310	> 30
150	3500	160	> 30
150	2000	95	> 60
150	3000	50	> 60
200	2000	760	> 30
200	3500	520	> 30
200	5000	320	> 30
200	2000	430	> 60
200	4000	200	> 60
250	2000	1430	> 30
250	4000	1000	> 30
250	6000	640	> 30
250	2000	970	> 60
250	5000	430	> 60
Predicted Performance - CHS			
139.7	2000	168	> 30
139.7	3500	76	> 30
168.3	2000	340	> 30
168.3	3500	180	> 30
219.1	2000	773	> 30
219.1	3500	512	> 30
219.1	5000	321	> 30
219.1	2000	440	> 60
219.1	3500	256	> 60
219.1	5000	155	> 60

Table 4 (*cont.*)

outer dimension (mm)	eff. length (mm)	load (kN)	resistance (mins)
Predicted Performance - CHS			
273.1	2000	1379	> 30
273.1	5000	756	> 30
273.1	7000	470	> 30
273.1	2000	970	> 60
273.1	5000	452	> 60
273.1	7000	267	> 60
323.9	2000	2130	> 30
323.9	7000	825	> 30
323.9	2000	1640	> 60
323.9	7000	500	> 60
355.6	2000	2690	> 30
355.6	7000	1333	> 30
355.6	2000	2120	> 60
355.6	7000	915	> 60
406.4	2000	3674	> 30
406.4	7000	2095	> 30
406.4	2000	3000	> 60
406.4	7000	1108	> 60
457	2000	4762	> 30
457	7000	3060	> 30
457	2000	4095	> 60
457	7000	2428	> 60
508	2000	6096	> 30
508	7000	4238	> 30
508	3000	5310	> 60
508	7000	1857	> 60

The procedure for checking the adequacy of a *trial design member* is the same as that given in (a) above.

Approach 3: Use of Simplified Calculation Method

(a) Kodur's Formula
Kodur [21] has recently proposed a simplified formula to allow the estimate of fire resistance of concentrically loaded SSHS columns. The proposed formula was developed following extensive experimental and theoretical studies of the behaviour of concrete-filled tubes in fire. A series of parametric studies were conducted using a theoretical model that accounted for the effect of elevated temperature on the mechanical properties of steel and concrete. The data generated by these parametric studies was used to derive a simplified formula to enable the prediction of fire resistance. The proposed formula and associated variables are given below:

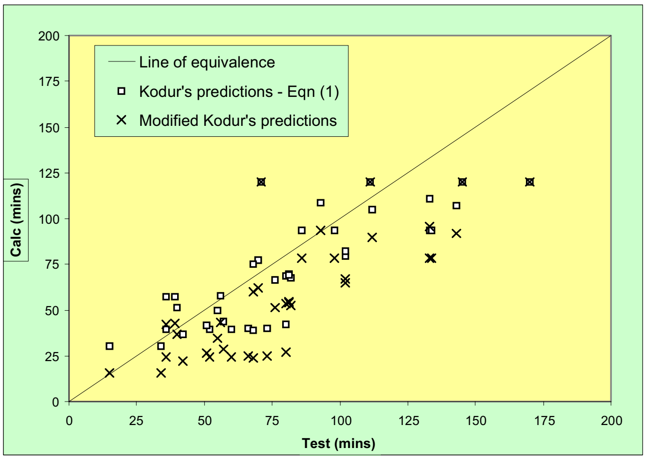
$$t = f\frac{(f'_c+20)}{(L_e-1000)}D^2\sqrt{\frac{D}{P}} \qquad \text{......Eqn (1)}$$

Table 5

Variable	Plain concrete	conventional reinforced concrete	steel fibre reinforced concrete
L_e: *effective length* (*mm*):	2000 - 4000	2000 - 4500	
f'_c: *28 day compressive strength of concrete* (*MPa*):	20 - 40	20 - 55	
D: *outside diameter or width of section (mm)*			
CHS (mm):	140 - 410	165 - 410	140 - 410
SHS (mm):	140 - 305	175 - 305	100 - 305
t: *fire resistance of column subject to standard fire test* (*mins*)	≤ 120	≤ 180	≤ 180
P: *load applied to the column in fire (kN)*			
f: *empirical factor depending on whether the section circular, square or fibre reinforced*			
CHS:	0.07	0.08 *	0.075
SHS:	0.06	0.07 *	0.065

* < 3% and 25mm cover

A comparison of the fire resistance given by the proposed equation with the results of tested members (unreinforced sections) falling within the limits of the above simple theory is shown below.



It can be seen that there is a correlation between the predicted fire resistances and those obtained from the tests. If the formula (Eqn (1)) predicted exactly the fire resistance obtained from the tests then all of the points would fall along the line of equivalence. About one half of the points fall above the line.

Also shown on this graph are the predicted fire resistances using a modified equation. The points are obtained by subtracting 15 minutes from each calculated response. The modified equation is given below:

$$t = f\,\frac{(f'_c+20)}{(L_e-1000)}\,D^2\sqrt{\frac{D}{P}}-15 \qquad \text{........Eqn (2)}$$

It is advocated that this approach be adopted when estimating the fire resistance of unreinforced column sections using this formula. The limits given in Table 5 still apply. [Note that at the limit, both equations 1 and 2 may predict a fire resistance of 120 minutes as shown for the four pairs of results above].

Example 3:
Design an unreinforced concrete-filled circular hollow section column of a building to have a fire-resistance level of 60 minutes. The column is filled with concrete having an f'_c of 40MPa. The effective length of the column is taken as 2500mm, and it may be assumed to be axially loaded. The load applied to the column in fire is 1750kN

Solution:
Design parameters: unreinforced concrete
circulare hollow section CHS
effective length L_e = 2500mm
f'_c = 40MPa
P = 1750kN
FRL = 60 minutes

From the design parameters given and rearranging Eqn (2), the outside diameter of the circular hollow section, D, can be expressed as:

$$D^{5/2} = (t+15)\sqrt{P}\,\frac{(L_e-1000)}{f(f'_c+20)}$$

From Table 5, f is taken as 0.07, and substituting all the parameters into the equation, gives:
D = 263mm

Therefore, the minimum outer diameter of the CHS require is 263mm. From the range of OneSteel product, the next section closest to and higher than 263mm is 273.1mm CHS

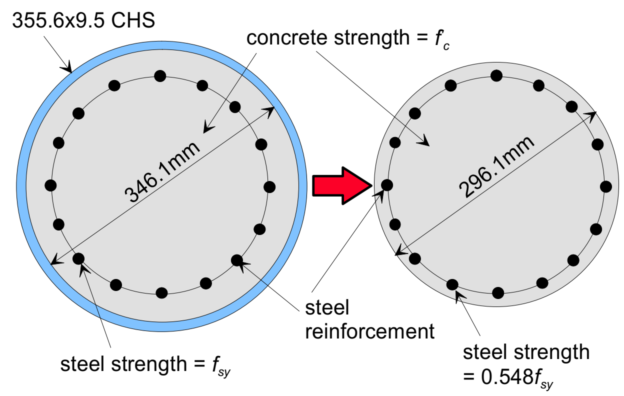
(b) Approach for Larger Diameter Columns
For circular hollow sections of larger diameter (≥ 350mm), an approximate approach can be used to determine the fire resistance. This consists of taking into account the effect of temperature rise on concrete and steel reinforcement by utilising an effective concrete cross-section and reinforcement having an effective strength. Any strength associated with the CHS section is ignored. The column is then analysed as a normal concrete column but subject to the reduced loads applicable to the fire situation (see AS1170.1[22]). The recommendations are given in Table 6 and derived from heat transfer analyses of CHS sections of varying diameters.

Table 6

FRL exposure (mins)	Diameter reduction (mm)	Steel strength reduction covers (mm)		
		30mm	40mm	50mm
30	10	1	1	1
45	20	0.83	1	1
60	31	0.63	0.83	0.99
75	41	0.47	0.67	0.84
90	50	0.35	0.55	0.72
120	66	0.16	0.35	0.52
150	82	0.02	0.20	0.37
180	96	0	0.08	0.24

Example 4:
Calculate the effective concrete cross-section and the reduction in steel strength of conventional reinforcement of a reinforced concrete-filled circular hollow section column to have a fire resistance level of 90 minutes. The cross-section in consideration is a 355.6x9.5 CHS with conventional reinforcement placed at 40mm inside of the CHS.

From Table 6, the diameter reduction of the concrete cross-section at 90 minutues FRL exposure is 50mm. The corresponding steel strength reduction of the reinforcement at 40mm cover is 0.548. The resulting cross-section with reduced strength of reinforcement are shown below.



Existing reinforced concrete-filled CHS **Effective cross-section**

The strength of the column with the effective cross-section can be assessed in accordance with AS3600[23].

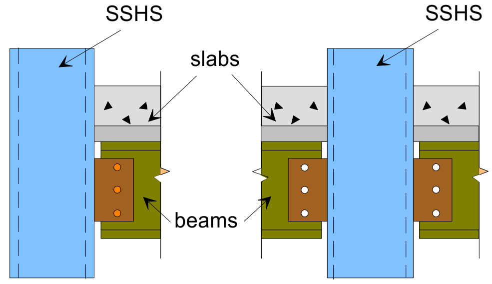
Effective Lengths

This technical note only applies to columns in braced buildings where the columns are heated over one level.

If the column is flexurally continuous at each end, then the effective length, L_e, can be taken as *0.7 x H* where H is the distance between lateral restraints (typically the floor-to-floor height). If the column is flexurally continuous at only one end and rotationally unrestrained at the other, then L_e can be taken as *0.85 x H*. Otherwise L_e should be taken as H.

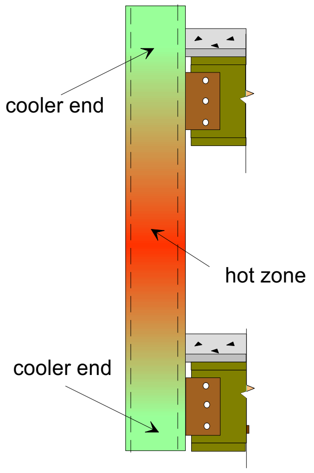
Eccentric Loading

Under ambient conditions, all steel members must be designed to resist a level of eccentric loading. This is to allow for the presence of some level of unintentional eccentricity. Should columns have beams connected to one side only they may be subject to a greater bending.



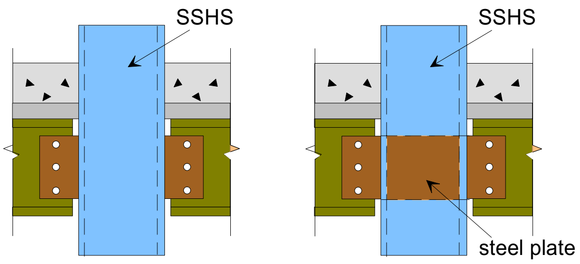
One sided **Two sided**

If an *isolated* unreinforced concrete-filled tube is subjected to eccentric loading, then the fire resistance of the member may be significantly reduced due to the fact that the column may have little bending resistance. If the column is continuous and the connections are kept adequately cool, the cooler lengths of the column will exhibit much greater flexural stiffness and therefore attract more bending moment into these regions, thus reducing the bending moments at the ends of the heated column length. It is reasonable under these circumstances to consider the column as being essentially axially loaded.



Connections

Typical connections between beams and concrete-filled SSHS members are illustrated. These include web side plates that are welded to the sides of the steel member and those where a connected plate is passed through the column.



Type (a) **Type (b)**

It is assumed that the connections and beams are kept sufficiently cool to transfer loads and provide the necessary lateral and rotational restraint to the ends of the column. This is normally achieved by fire protection of these parts. Connection Type (a) is suitable where the column is continuous since the floor loads can be transferred into the concrete core through bond between the upper cool steel section and the concrete. Such a connection could be used to transfer roof loads into a column but additional measures must be taken to transfer the loads directly into the concrete. This could be achieved by a cap plate that transfers the load in bearing or through pins through the tube into the concrete to give mechanical anchorage.

In the case of connection Type (b) where a steel plate has been slotted through the tube, the load can be transferred directly into the concrete through bearing of the steel plate on concrete.

Practical Considerations

Small holes (2 x 20mm dia) should be provided in the walls of a tube and located between 100 - 200mm from its ends and at a maximum spacing of 5m. These holes are provided to relieve steam pressure [24].

If it is intended to reinforce the concrete core with steel fibres, then it is recommended that such fibres should be 0.5mm in diameter, not longer than 38mm, and have crimped flats or hooked ends to ensure adequate pull-out resistance [25].

Conclusions

General guidelines on the fire resistance performance of SSHS columns incorporating concrete filling have been provided. Approaches 1 and 2 are tabulated solutions for unreinforced concrete-filled SSHS columns based on test results and analytical predictions, respectively. Approach 3 offers a simplified calculation method for both reinforced and unreinforced concrete-filled SSHS columns. These approaches must only be used within their prescribed limits.

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