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The Fire Resistance of Web-Infilled Steel Columns

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FOREWORD

The research leading to this report was carried out by Mr G M Newman of SCI, assisted by Mr D Proe of Broken Hill Proprietary Company, Australia. The fire tests were performed at the Loss Prevention Council and were set up and monitored by British Steel Technical (Swinden Laboratories). The work was funded by British Steel (General Steels).

SCI Technical Reports

Technical Reports are intended for the rapid dissemination of research results as and when they become available. They provide an opportunity for interested members to comment and offer constructive criticisms, so that a refined design guide can be produced eventually, after taking into consideration the comments received.

Please forward your comments to Mr G M Newman or Dr R M Lawson, The Steel Construction Institute, Silwood Park, Ascot, Berkshire, SL5 7QN.

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SUMMARY

A simple method of achieving 60 minutes fire resistance of steel columns by partial concrete encasement is presented. The concrete is placed adjacent to the webs to form a square section, leaving the outer faces of the flanges exposed. The columns should be designed as 'columns in simple construction' according to BS 5950: Part 1.

Four fire tests on columns were performed. The columns were subject to loads of 0.35 to 0.55 times the 'cold' capacity of the steel section and achieved fire resistances of 58 to 72 minutes respectively. Shot-fired shear connectors were used to develop 'composite' action in fire. In two of the tests this behaviour was enhanced by welding of stiffeners at the top and bottom of the column.

A design method is proposed, together with design tables. The method is based on Eurocode 4: Part 1.2 (formerly Part 10), and uses calculated temperature distributions, which are confirmed by the tests. Comparison between the tests and predicted performance is presented. A method of including the effect of additional major and minor axes moments is included.

1. INTRODUCTION

Concrete has been traditionally used to provide fire protection to structural steel. However, it is an expensive form of protection and its use has diminished in recent years. In modern buildings lightweight proprietary materials tend to be used. Beams are normally sprayed and columns are almost invariably box encased. Providing fire protection adds an appreciable amount to the cost of a steel frame and also adds to the length and complexity of the construction programme.

SCI have been working with British Steel to reduce the cost of fire protection and to reduce the disruption caused by applying fire protection on site. One of the aims of this research is to eliminate the need for a specialist fire protection contractor and instead to build the fire resistance into the structure by making use of concrete floors, masonry walls and take advantage of the structural continuity that exists in all steel frames.

This research programme is concerned with "web-infilled columns". These are columns made from a standard Universal Column section with the areas formed by the web and flanges filled with concrete. The outer surfaces of the flanges are left exposed. A schematic view of a web-infilled column and its associated beams is shown in Figure 1.

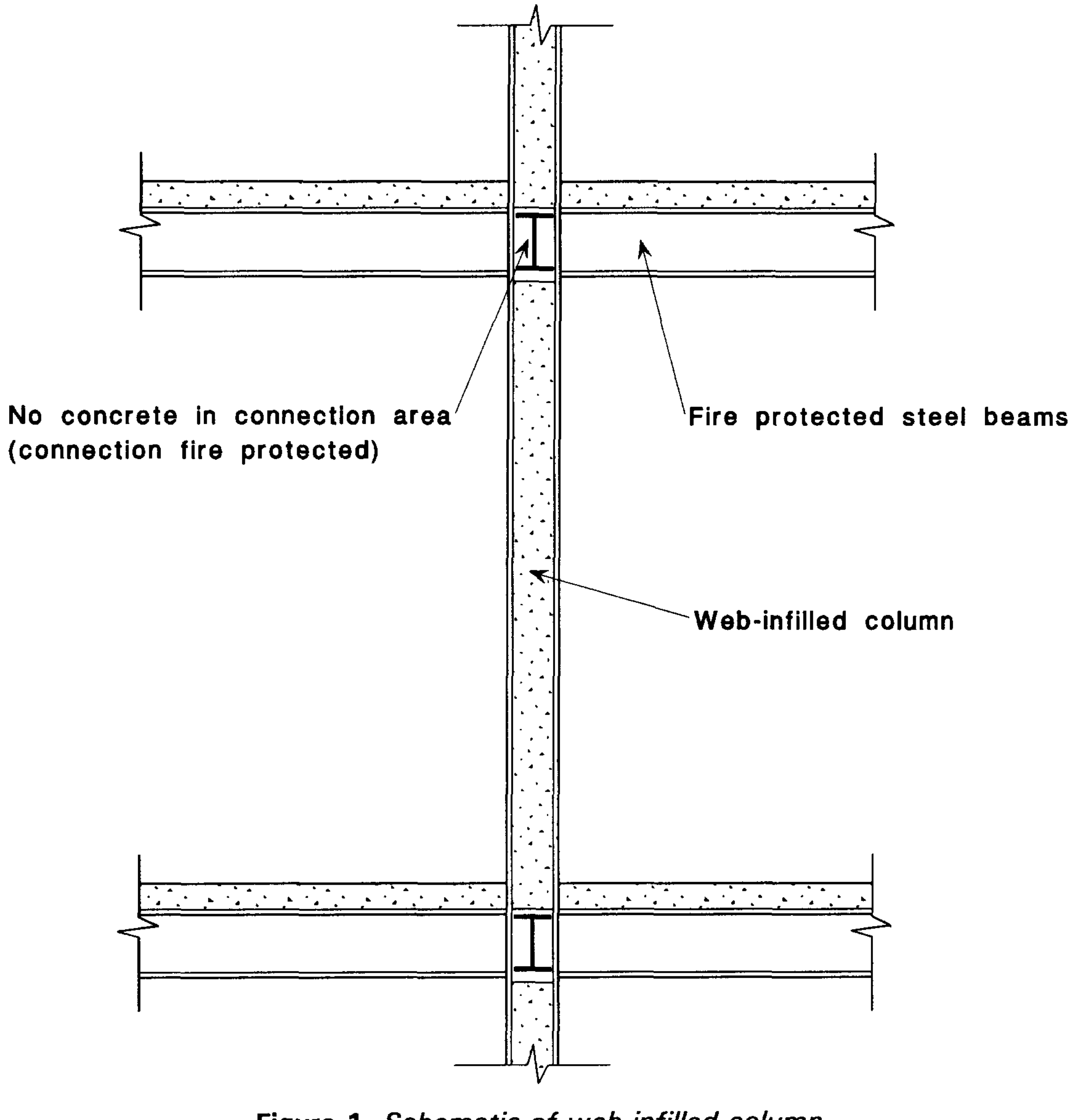

Figure 1 *Schematic of web-infilled column*

To ensure that the column acts, at least in fire, as a composite section, shear connectors are placed on the steel web. These connectors may be either shot-fired or welded. For on site use, shot-fired connectors are advantageous because heavy equipment is not required and there is no lower limit to steel thickness as for welded shear connectors. In this research Hilti HVB80 shot fired connectors were used. They were attached to the web of the steel section at regular spacings of 300 or 500 mm. In addition to the shear connectors in some of the tests a web stiffener was provided at the top of the column to contain the concrete.

By using web-infilled columns, which can be formed on or off-site, the need for expensive fire protection to the columns can be eliminated. The surface finish of the column will probably not be acceptable in prestige offices but may well be acceptable in low cost offices, institutional buildings and industrial buildings. Columns of this type are intended for buildings requiring 60 minutes fire resistance. It is unlikely that, because of the heat entering the exposed section, 90 minutes fire resistance could be economically achieved.

2. EXISTING METHODS OF FIRE PROTECTION USING CONCRETE AND BLOCKS

2.1 Concrete encased columns

For many years building regulations have recognised the beneficial effects of concrete encasement on the fire resistance of columns. Fire resistance of up to 4 hours can be obtained. However, the amount of concrete cover (to the outside of the steel section) is substantial and to control spalling of the concrete in fire, a light reinforcing mesh is required at the mid-depth of the concrete cover.

Encasing columns in this traditional manner is very expensive and increases both the size and weight of the columns considerably. A complete four sided shuttering system is required and the reinforcement has to be held in place by spacers. This is more typical of reinforced concrete practice and slows down the speed of construction.

All the fire resistance tests on encased columns were carried out in the 1950's. The bulk of the tests were designed to achieve fire resistance of more than 2 hours. The data is of little use to this study.

2.2 "Arbed" columns

Arbed, the Luxembourg steel maker, has developed a method of achieving high load capacities and long periods of fire resistance using composite columns. Details of the method are given in a recent ECCS Technical Note(1). A typical column section is shown in Figure 2. Arbed columns utilise reinforcing bars, welded stirrups and welded shear connectors. They are ideally suited for heavily loaded columns that require periods of fire resistance greater than 60 minutes.

The external appearance of an Arbed column would be identical to that of a web-filled column.

2.3 Concrete filled hollow sections

Filling a steel hollow section with concrete increases the fire resistance of the section (see Figure 3). The concrete acts as a heat sink and as an alternative load path when the steel begins to fail and load is redistributed. Design details are given in Reference(1) and also in the British Steel design manual(2). The behaviour of concrete filled structural hollow sections is affected by the loss of strength of the steel section. The load capacity achieved in fire can be increased with additional reinforcing bars.

2.4 Blocked-in columns

These are columns in which the web area is filled with concrete blocks. The blocks act as a shield to the inner faces of the section and as a heat sink. They are simply mortared into place and are not assumed to carry any load. A design guide for 30 minutes fire resistance has been published by BRE(3). The system has been used in applications where the painted, exposed steel, together with well placed blocks, formed an attractive feature. A blocked-in column is shown in Figure 4.

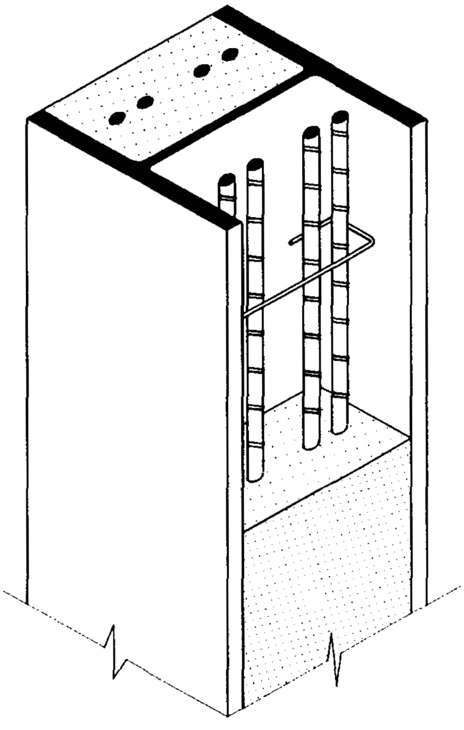

Figure 2 *Arbed column*

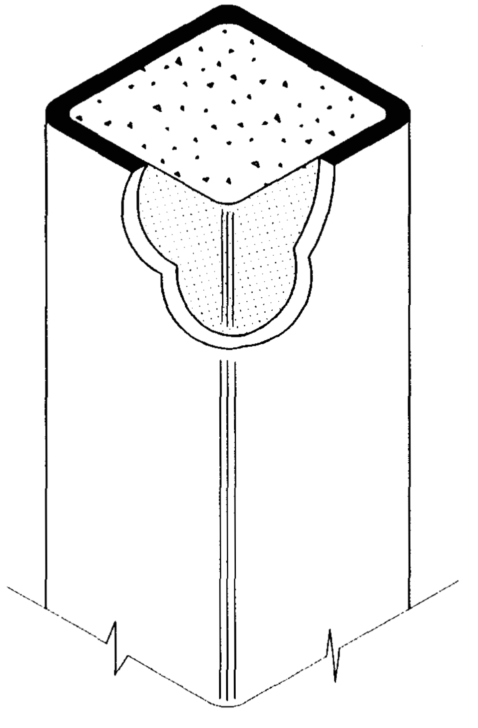

Figure 3 *Concrete filled hollow section*

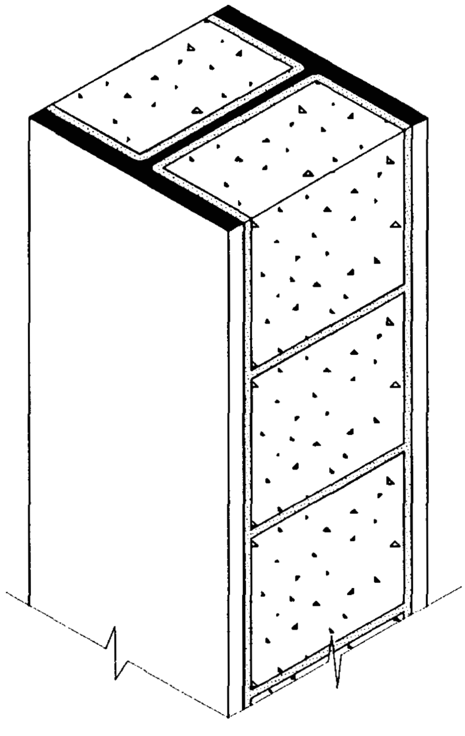

Figure 4 *Blocked-in column*

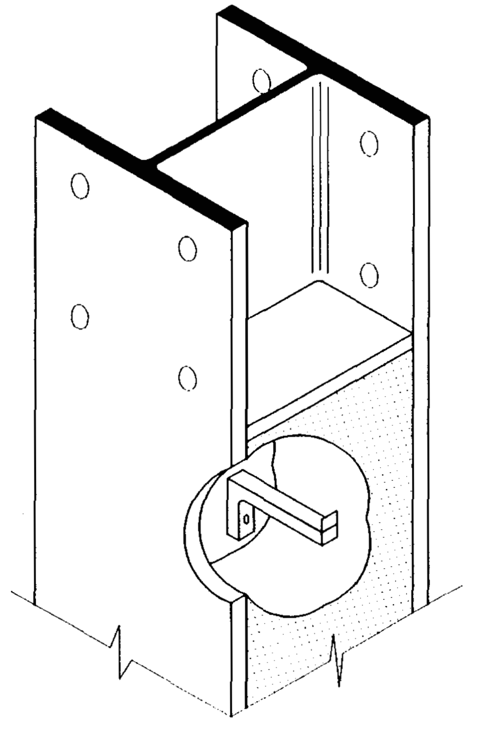

Figure 5 *Web-infilled column*

3. WEB-INFILLED COLUMNS

The idea of a web-infilled column, as shown in Figure 5, came from a desire to combine the simplicity of a blocked-in column with the superior performance of an "Arbed" column. The aim was to produce a cost-effective minimum-sized column with 60 minutes fire resistance. From the work carried out on blocked-in columns it was apparent that to obtain a 60 minute fire resistance the concrete would have to contribute to the load bearing capacity.

In the Arbed column the reinforced concrete infill provides both additional strength in the cold state and enhanced fire resistance. However, the combination of shear connectors and reinforcement adds appreciably to the cost. In the web-infilled column the unreinforced concrete is assumed *not* to contribute to the cold strength but is assumed to carry axial load in fire.

An additional feature of the proposed design is the fact that the webs are not filled right up to the top of the column, thus allowing the connection between the beams and column to be simply made. The columns could be brought to site ready filled, or alternatively, where that was impractical, the columns could be filled on site using minimal shuttering. In fire, load would be transferred from the steel column to the composite, web-infilled, column. The unconcreted part of the column and connections would be protected by the same system used to protect the steel beams.

The load transfer between the steel column and the concrete infill is achieved in two ways. Initially it was felt that shear connectors placed on the web could be used for this purpose but their performance in the first tests showed that their use alone was not adequate, so in later tests the shear connectors were supplemented by web stiffeners at the top of the column. These stiffeners have the added benefit of acting as a shutter for the concrete.

The tests were performed using Hilti shot-fired connectors attached to the column web but in practice any connector could be used provided that the web thickness is adequate for a firm attachment of the shear connectors.

4. TEST PROGRAMME

The test programme was in two phases with two tests in each phase. Phase 1 tests took place in late 1989 and Phase 2 tests about a year later. The main difference between the phases was the inclusion of the top web stiffener in Phase 2.

4.1 Phase 1

The test programme was designed so that each of the two tests would achieve a 1 hour fire resistance. The choice of section sizes was made based on the available test data on blocked-in columns and a knowledge of the commonly used sections. The test loading was calculated assuming a probable temperature distribution throughout the section.

The load in the first test was chosen to represent 90% of the maximum load that the column could reasonably be assumed to carry in fire. This corresponded to a load ratio of 0.5 based on the load carrying capacity of the steel section alone at normal temperatures. The load in the second test was decided upon following the first test.

Each test employed Hilti shot-fired shear connectors. These were placed at 300 mm centres in the first test and 500 mm centres in the second. It was felt that the larger spacing would probably be adequate for both tests but because the precise mechanism of load transfer to the concrete is not clear a cautious approach was adopted for the first test.

As noted above, in order to simulate conditions in an actual building the concrete filling was discontinued 300 mm from the top of the column. The unfilled area was fire protected with mineral wool blanket. The load in the column would therefore be initially applied to the steel section rather than the concrete. This was therefore considered to be a reasonable representation of actual building situations without effective continuity of concrete.

The column test arrangement is illustrated in Figure 6.

Thermocouples were built into each test specimen to record steel and concrete temperatures. Sufficient thermocouples were included to enable the temperature profile of the cross-section to be obtained.

4.1.1 Test 1

The test details were:

Section	254×254×73 Universal Column
Steel	Grade 43
Shear connectors	Hilti HVB80 at 300 mm centres
Test load	1129 kN

The test load was calculated to give a load ratio in fire of 0.5 based on the steel sections designed to BS 5950: Part 1[4].

4.1.2 Test 2

The test details were:

Section	254×254×73 Universal Column
Steel	Grade 43
Shear connectors	Hilti HVB80 at 500 mm centres
Test load	790 kN

The test load was calculated to give a load ratio in fire of 0.35.

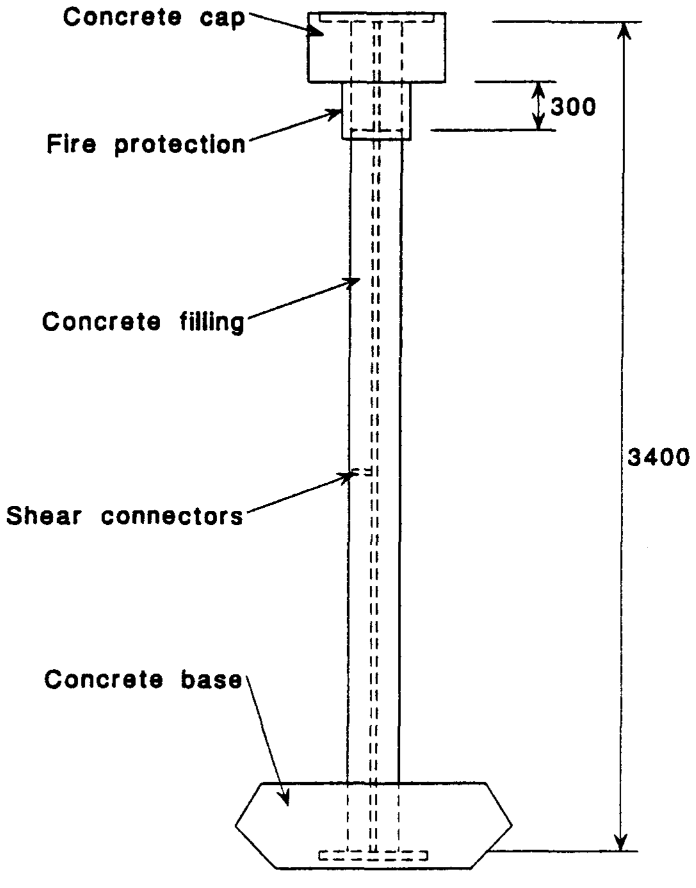

Figure 6 *Test arrangement*

4.2 Phase 2

Following lessons learned in Phase 1, the Phase 2 tests incorporated a top web stiffener. The stiffener was made from 15 mm plate with 6 mm fillet welds on both sides.

4.2.1 Test 3

The test details were:

Section	203×203×60 Universal Column
Steel	Grade 50
Shear connectors	Hilti HVB80 at 500 mm centres
Test load	976 kN

The test load was calculated to give a nominal load ratio in fire of 0.45.

4.2.2 Test 4

The test details were:

Section	254×254×73 Universal Column
Steel	Grade 43
Shear connectors	Hilti HVB80 at 500 mm centres
Test load	1244 kN

The test load was calculated to give a nominal load ratio in fire of 0.55.

5. OBSERVATIONS

5.1 Test 1

The column performed well and achieved a fire resistance of 58 minutes. The test was discontinued because the rate of deflection was excessive and total collapse was imminent. An examination of the specimen immediately after the test showed that a local failure of the column flange had occurred at the top of the column where the concrete filling commenced. No spalling of the concrete had taken place and very little lateral deflection of the column was observed. This suggested that the column remained stable at mid-height.

5.2 Test 2

The column achieved a fire resistance of 71 minutes. In all other respects it performed similarly to Test 1.

5.3 Test 3

The column achieved a fire resistance of 69 minutes. The test was discontinued because the rate of deflection was excessive and total collapse was imminent. An examination of the specimen immediately after the test showed that failure had occurred due to overall buckling about the weak axis. Unlike the behaviour in Tests 1 and 2, no local buckling had occurred, indicating that the web stiffener had performed satisfactorily. No spalling of the concrete had taken place.

5.4 Test 4

The column achieved a fire resistance of 72 minutes. The behaviour appeared to be very similar to Test 3.

The deflections (longitudinal extension) measured in each test are plotted in Figure 7.

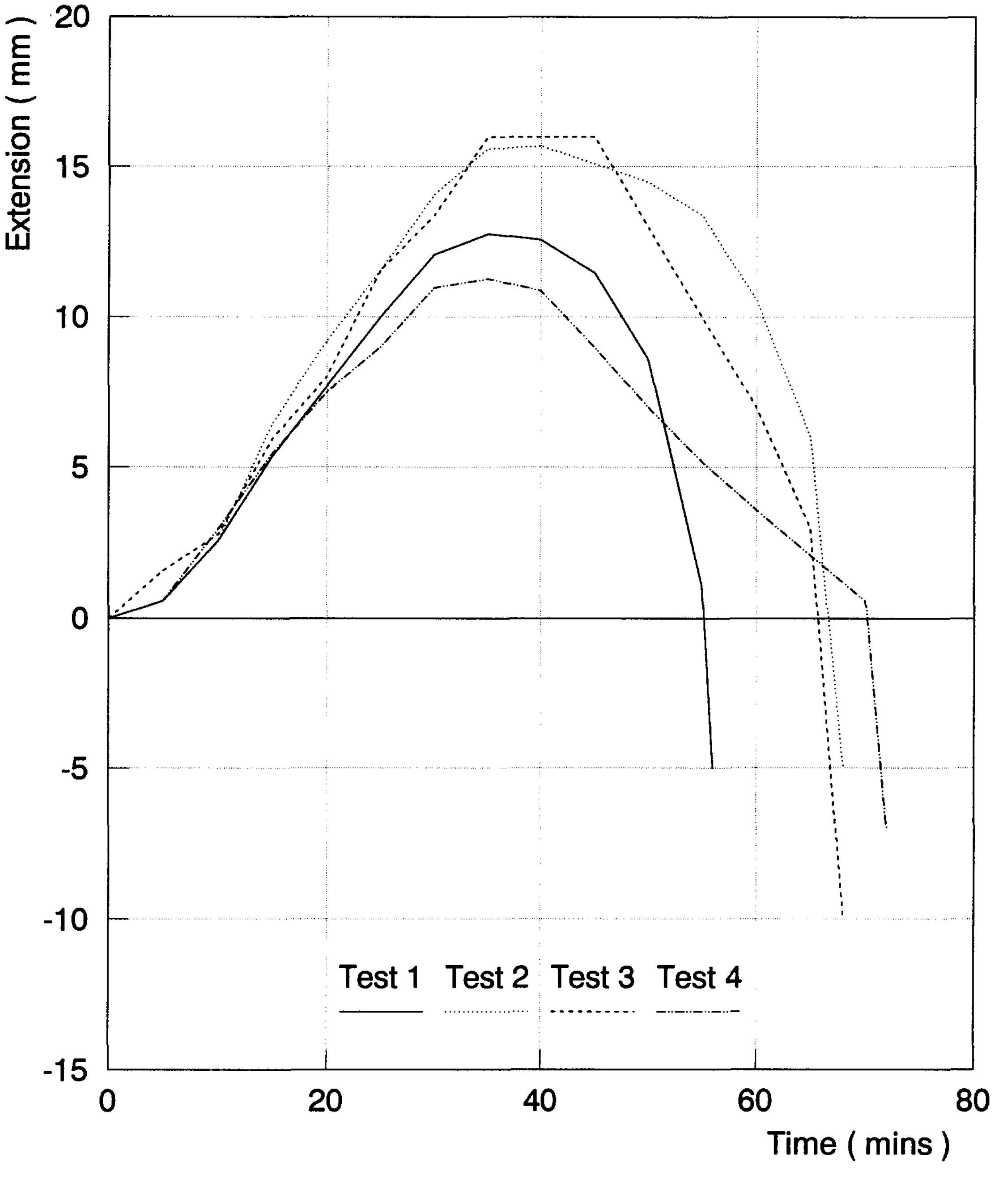

Figure 7 *Variation of extension with time for all tests*

6. DISCUSSION OF TEST RESULTS

In the first phase of tests it was clear that the columns failed locally because load could not be transferred from the steel into the concrete over a short distance at the top of the column. Incorporating the web stiffener in the second phase tests prevented this problem and failure was by overall buckling about the weak axis. In all the tests there was no sign of concrete spalling, although in the latter tests at failure the concrete cracked on the tension side and partially separated from the steel.

In all tests the shear connectors performed well with no failures. In the latter tests the web stiffeners and welds performed well with no signs of distress.

The longitudinal extension with time in all the tests is very interesting (Figure 7). Compared with non-composite columns the rate of shortening following the initial expansion is slow. This is illustrated diagrammatically in Figure 8. In both cases the expansion phase is similar. As the column heats up it starts to expand. Loss of strength then begins to cause greater compression strains in the steel and the rate of expansion reduces until yielding and buckling start to dominate and overall contraction of the column takes place. The rate at which contraction occurs is much greater in non-composite columns than in the web-infilled columns. This improved performance is caused by the load transfer from the steel to the concrete which increasingly takes place as the steel section 'fails'. Additionally, the composite column is much stiffer than the non-composite column.

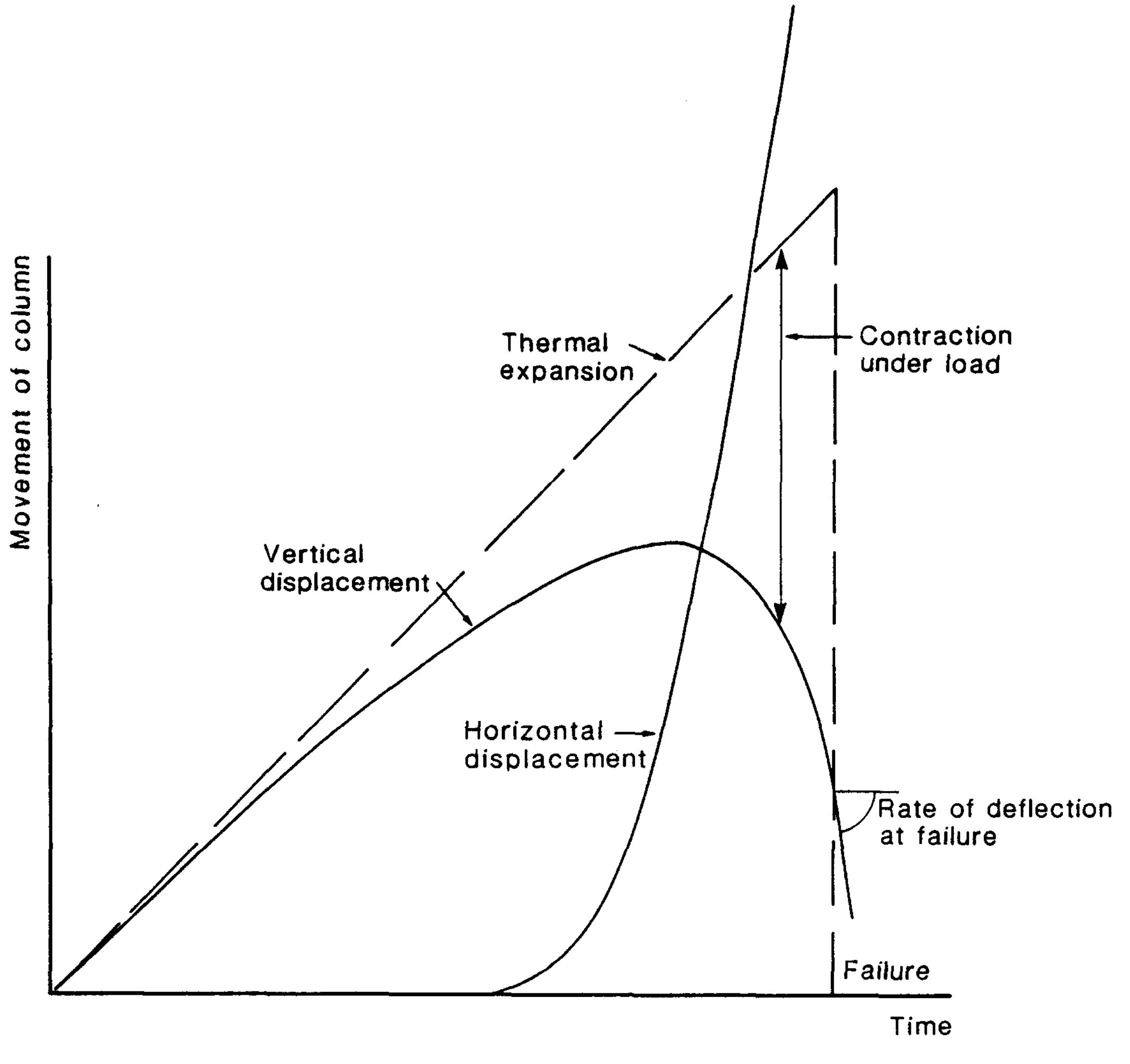

Figure 8 *Typical behaviour of steel column under axial load in a fire test*

7. ANALYSIS OF TEST RESULTS

7.1 Thermal analysis

SCI's thermal analysis program TFIRE was used to model the temperature distribution in the tests and subsequently to predict the distribution in a range of sections. Comparisons between the predicted and average measured temperatures for the two column sizes tested are shown in Figures 9 and 10.

7.2 Structural analysis

7.2.1 Buckling

The main structural analysis has been carried out following the method recommended in EC4: Part 1.2[5]. Additional analysis has been carried out using CFIRE, a non-linear analysis program developed by SCI. These analyses are described in 7.2.4. For detailed calculations this refers to ECCS - Technical Committee 3 - Technical Note 55[1].

The method is based on adapting the Eurocode strut formulae to operate at high temperatures and is now summarised, as follows.

$$N_{CR} = k\,N_p \tag{1}$$

$$k = \frac{1 + \alpha\,(\lambda - 0.2) + \lambda^2}{2\,\lambda^2} - \frac{1}{2\,\lambda^2}\left[(1 + \alpha(\lambda - 0.2) + \lambda^2)^2 - 4\,\lambda^2\right]^{½} \tag{2}$$

$$\alpha = 0.49$$

$$\lambda = \sqrt{\frac{N_p}{N_E}} \tag{3}$$

$$N_E = \frac{\pi^2\,E\,I}{L^2} \tag{4}$$

where:

N_{CR} = Buckling resistance of column

N_p = Squash load of column

k = Reduction factor on column resistance due to column slenderness

α = Imperfection factor in column

λ = Non dimensional slenderness of column

N_E = Euler buckling load.

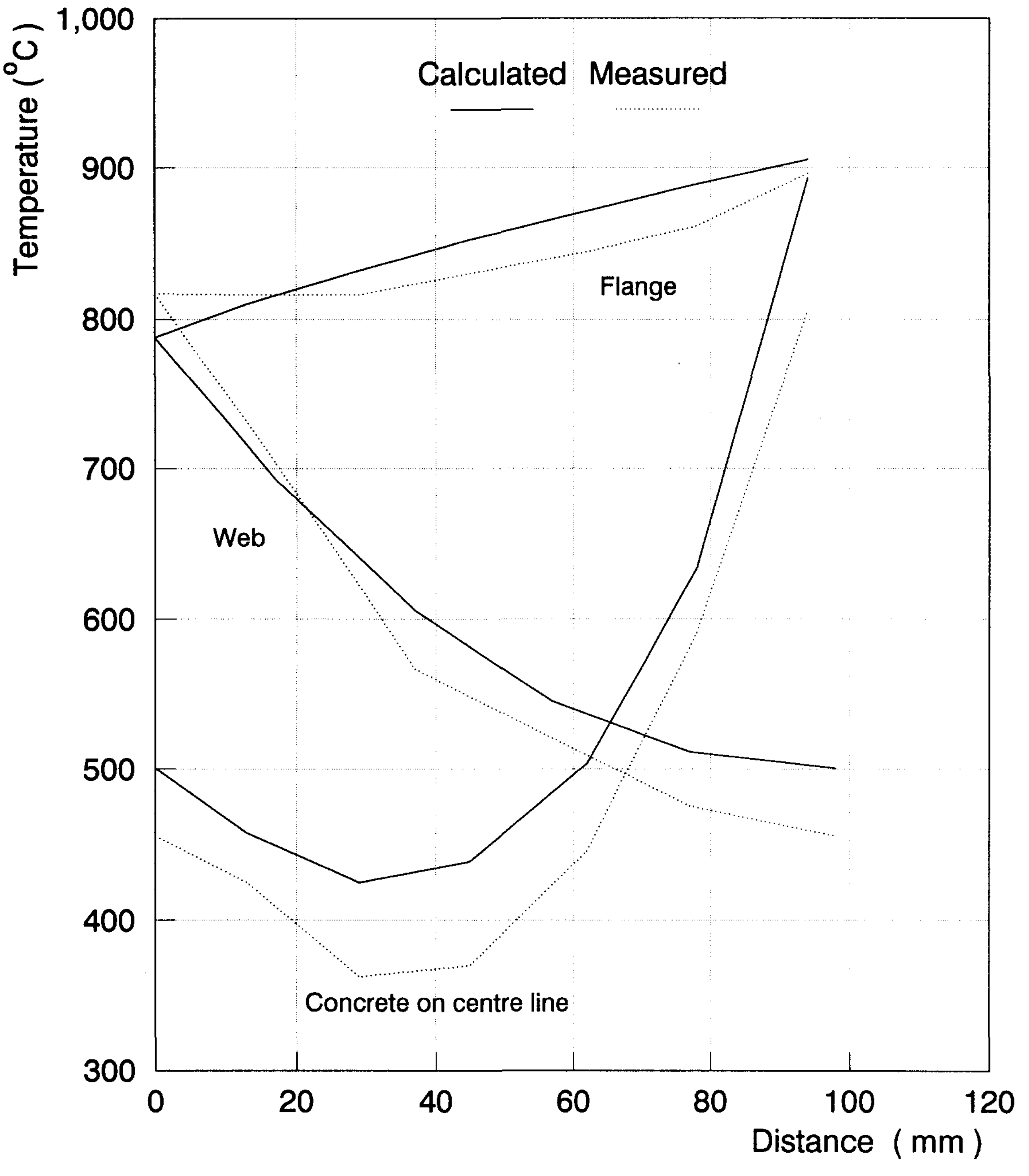

Figure 9 *Comparison between predicted and measured temperatures for Test 3*

Note on distance measurement:

Flange	The distance is from the flange/web junction
Web	The distance is from the flange/web junction
Concrete	The distance is from the web on the centre line

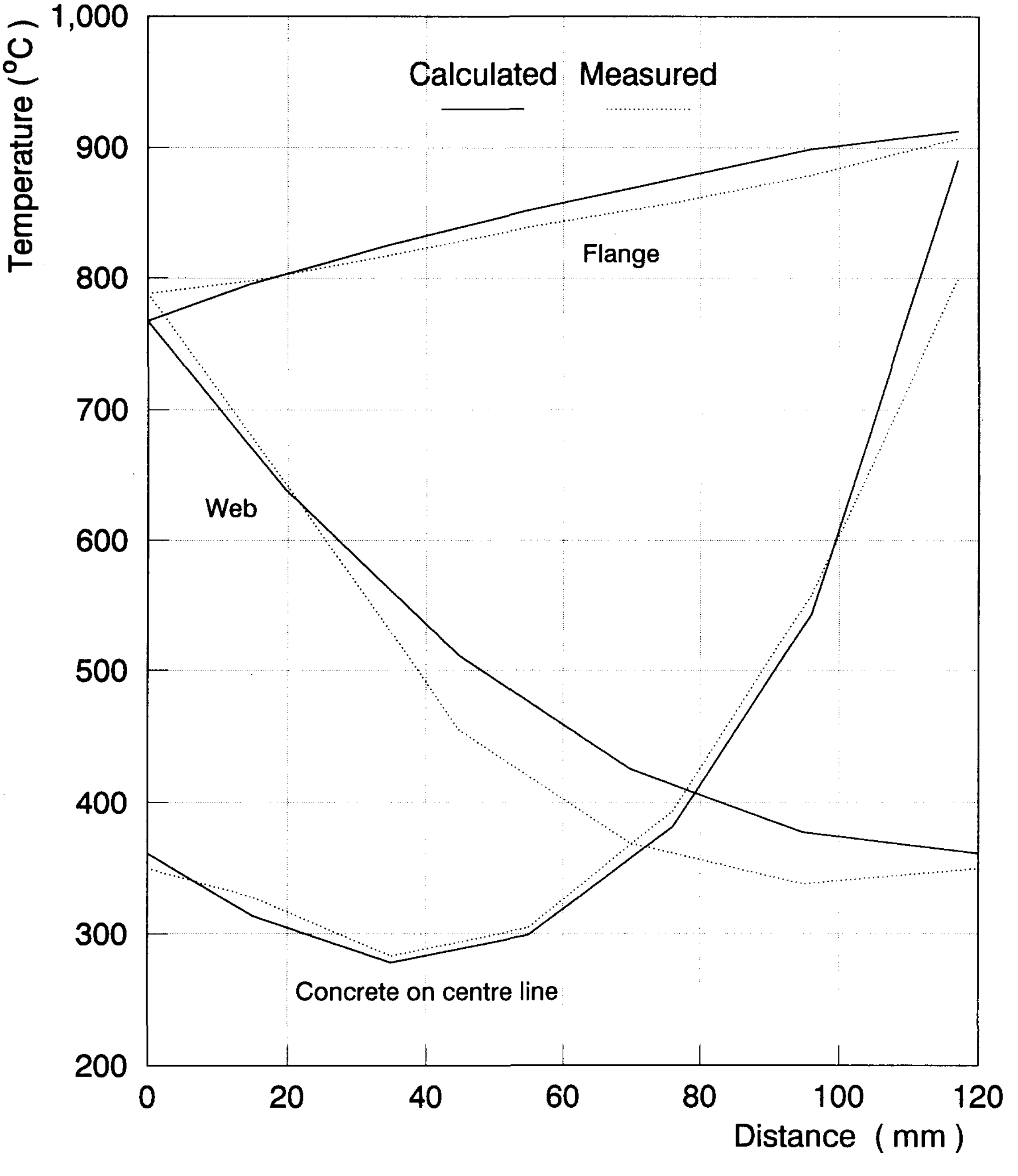

Figure 10 *Comparison between predicted and measured temperatures for Test 4*

Note on distance measurement:

Flange	The distance is from the flange/web junction
Web	The distance is from the flange/web junction
Concrete	The distance is from the web on the centre line

The Euler buckling load and the squash load are calculated at the temperature distribution across the section. This entails obtaining the temperature, elastic modulus and yield strength of each part of the section and then integrating across the section. Although the process could be carried out by hand, SCI used a grid of 42 elements across the section and developed a computer program to carry out the process.

The calculated test load and predicted capacities for Tests 3 and 4 are shown in Table 1. The earlier tests failed because of local buckling and no attempt has been made to model this phenomenum.

Table 1 *Comparison of test load and predicted capacity*

Section	Test Load (kN)	Prediction (kN)
203×203×60	976	817
254×254×73	1244	1288

The predicted capacity of the smaller section is 16% below the test load whilst that of the larger section is 4% above the test load. Both predictions are based on the same effective length for the column and it is probable that this assumption is the major cause of the discrepancy in results. It is likely that the more slender column experienced greater end restraint in the test. The amount of end restraint is a function of the relative stiffnesses of the column and support offered by the furnace. The characteristics of the furnace loading and restraining structure are not known so the effect of them cannot be calculated. Other possible reasons are inaccurate mounting of the columns and fabrication tolerances. Both these effects would have more effect on the slender column.

7.2.2 Bending

In a fire resistance test (in the UK) the ends of the column are virtually encastré and it is not possible to apply predetermined moments or vary the amount of restraint. In studying the test performance, the effects of applied moments have therefore not been included. A theoretical analysis of the effects of moments has been carried out and is described in Section 8.

7.2.3 Shear transfer

Examination of the temperatures recorded close to the web stiffeners in Tests 3 and 4 shows that the stiffeners and welds were below 300°C at failure. They can therefore be assumed to be at full strength. The plates were welded both sides with 6 mm fillet welds. In Table 2, comparison has been made between the weld capacity and the likely load carried by the concrete.

Table 2 *Comparison of weld capacity and load in concrete*

Test	Test Load (kN)	Weld Capacity (kN)	Load in Concrete (kN)
3	976	1008	517
4	1244	1258	800

The load in the concrete is based on an estimate of the ratio of concrete squash load to steel squash load.

Other analyses have been carried out on columns carrying their estimated maximum design loads (Section 9). These have also shown that the web stiffener can transfer all the load into the concrete. It is concluded that the shear connectors only carry nominal loads and that their main role is to retain the concrete and to prevent separation.

7.2.4 CFIRE analyses

CFIRE is a general analysis program for columns in fire which uses the principle that plane cross-sections remain plane and at any cross-section a linear strain distribution is found which is compatible with the axial load, the moment and the thermal strains. From this linear strain the local curvature can be found and hence the displaced shape of the complete column. Only a two dimensional analysis is carried out. The displaced shape is used to calculate the moments at each section. "P-delta" effects are therefore included. The program has been used to model two of the fire tests and to investigate the effects of moment.

In Figure 11 the CFIRE predictions of axial displacement and horizontal displacement of Tests 3 and 4 are shown together with the measured axial displacements. Horizontal displacements were not measured in the tests. It can be seen in Figure 11 that the correlation between measured and computed displacements is not particularly good, although the program was able to predict failure of Test 4 quite well but underpredicted Test 3. It appears from this comparison and other comparisons carried out by SCI that CFIRE predicts the performance of stocky columns reasonably well but is conservative for more slender columns. This is probably due to variations in concrete properties compared with the "codified" properties given in EC4[5]. It is felt by SCI that CFIRE is more useful for analysing the effects of moments on columns because of the ability to input any combination of applied moments and axial load to investigate their interaction.

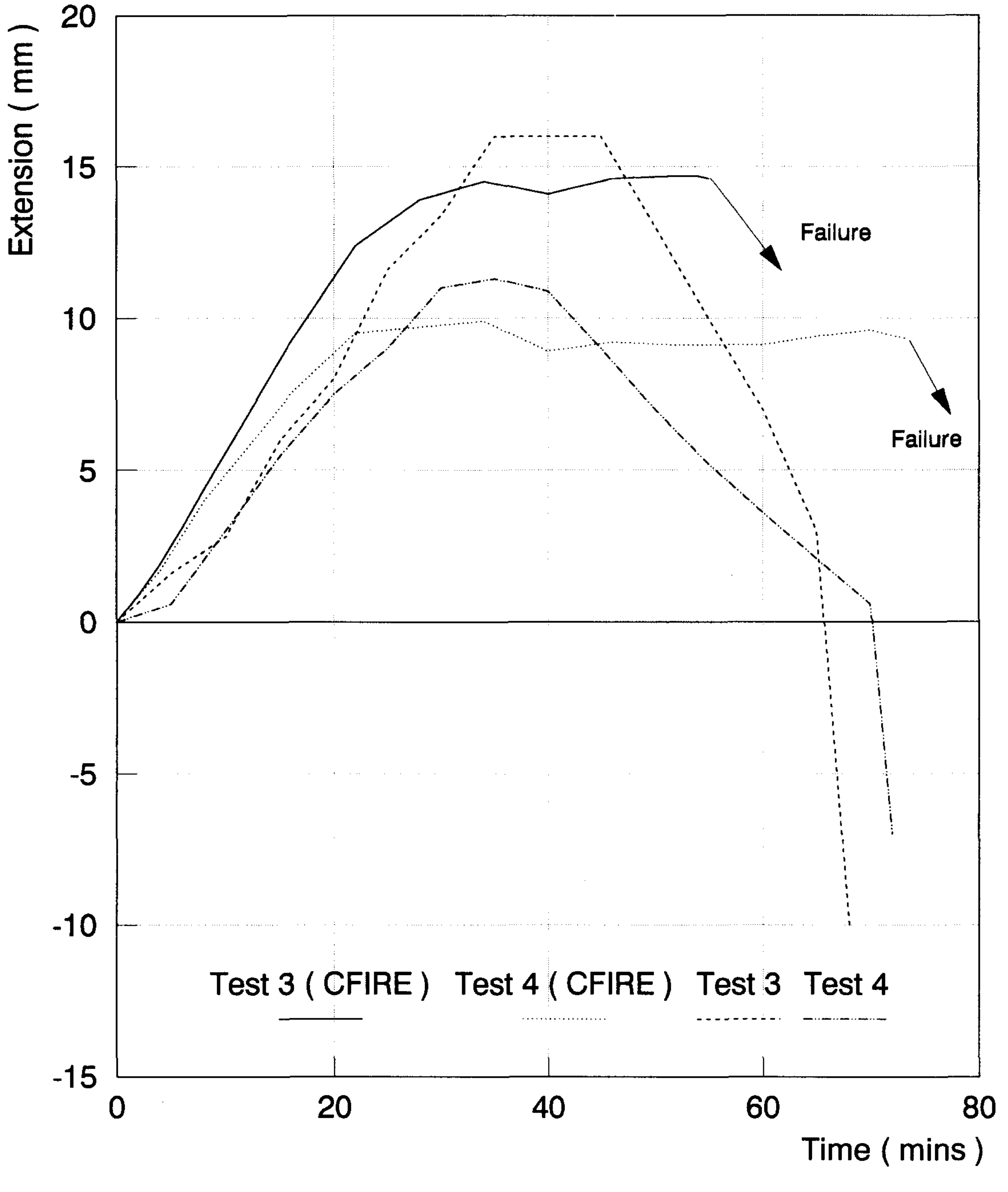

Figure 11 *Comparison of extension with time for Tests 3 and 4 with computed values*

8. THE EFFECT OF MOMENTS

In real buildings columns always have applied moments which are largely resisted by the column flanges. Because the flanges reach temperatures in excess of 800°C, it may not be safe to assume that the moment capacity of the composite section reduces at the same rate as the axial load capacity. This assumption would be reasonable when considering non-composite columns. Using CFIRE the combination of various proportions of axial load and moment were analysed. The shape of the bending moment diagram along the length of the column was also varied. The analysis was carried out mainly on a 254×254×73 Universal Column of grade 43 steel and the conclusions were confirmed by carrying out a limited number of analyses on other sections. In carrying out the analyses a small temperature gradient at either end of the column was assumed such that the ends of the column were at 80% of the temperature of the central portion. This influence reduced linearly over the end 12% of the column height. In addition to the CFIRE analyses the moment capacity of the composite section was calculated.

8.1 Force-Moment interaction

A 254×254×73 Universal Column, grade 43, with an effective length in fire of 2100 mm was analysed. The load was found for which, as a pinned strut, buckling about the weak axis, a fire resistance of 60 minutes would result. Three linear distributions of bending moment were considered.

a) Equal and opposite end moments.
b) One end moment zero.
c) One moment equal to minus 50% of the other end moment.

These are illustrated in Figure 12. It was felt that in a real structure, with the fire confined to one storey, distribution c) would be the worst reasonable design case reflecting the case of fire in one compartment. Numerous analyses were carried out to find combinations of axial load and moment which gave 60 minutes fire resistance. The results of the analyses are shown in Figure 13. The process was repeated for buckling about the strong axis (see Figure 14).

In both Figures it can be seen that the shape of the bending moment diagram is very important. With equal and opposite moments (case a) the best performance is obtained and with one end moment zero (case b) the worst performance is obtained. On first examination the performance about the strong axis appears to be appreciably better than that about the weak axis but the reduction in moment compared with the original moment capacity of the steel section is much greater about the strong axis.

About the weak axis:	
Moment capacity (cold)	83.9 kNm
Moment capacity (hot)	56 kNm
Ratio (hot / cold)	67%
About the strong axis:	
Moment capacity (cold)	271.7 kNm
Moment capacity (hot)	93 kNm
Ratio (hot / cold)	34%

The axial load capacity in fire is approximately 60% of that under cold conditions. It can be seen that the loss of strength about the weak axis is approximately equal to the loss of axial capacity. However, the loss of bending capacity about the strong axis exceeds these two due to the greater loss of strength of the flanges. It is important that this effect is reflected in any design procedure.

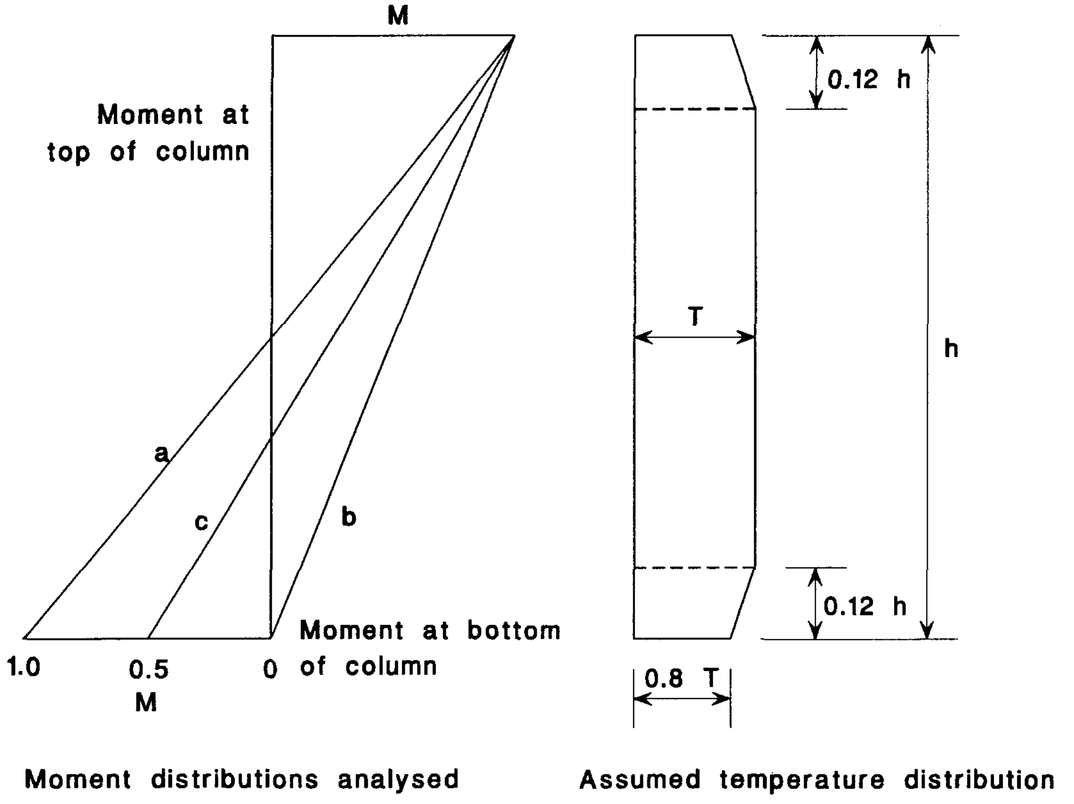

Figure 12 *Bending moment and temperature distributions assumed*

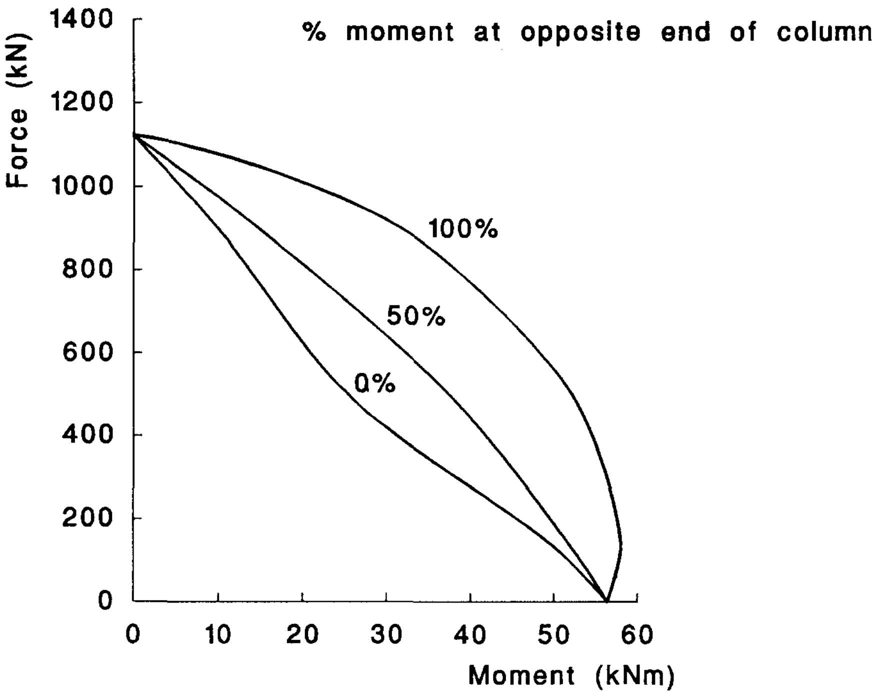

Figure 13 *Force - Moment interaction diagram for 254 × 254 × 73 column bending about YY axis*

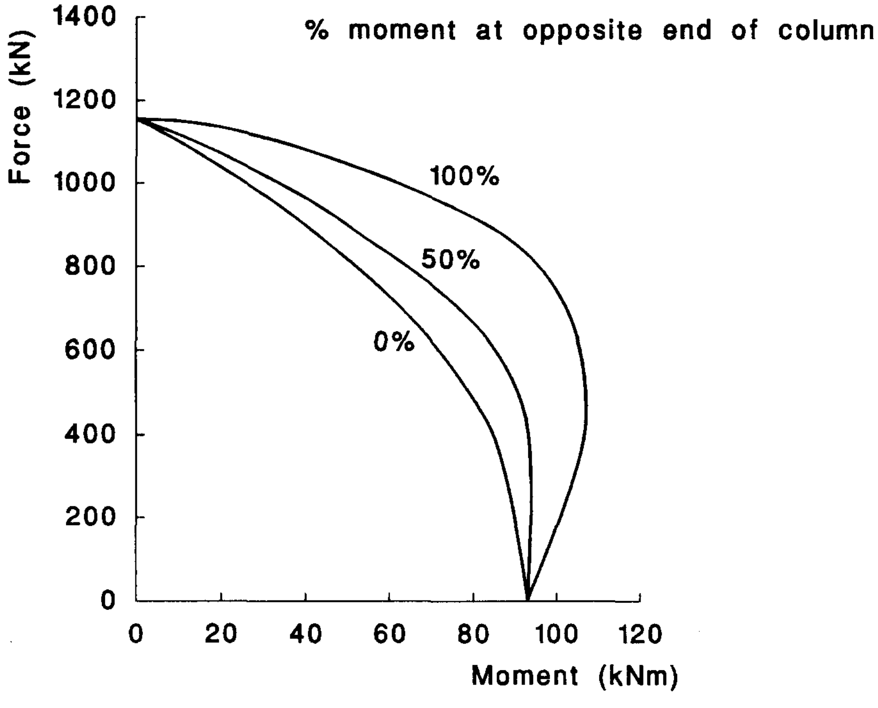

Figure 14 *Force - Moment interaction diagram for 254 × 254 × 73 column bending about XX axis*

The moment capacity obtained using the column analysis program, CFIRE, is compared with the moment capacity calculated using the method of BS 5950: Part 8(6) taking account of the reduced strength of the stress blocks. This is presented in Table 3.

Table 3 *Comparison of moment capacity calculated using CFIRE and plastic theory*

		Moment Capacity (kNm)		
Section	Axis	CFIRE moment capacity	Moment capacity method	Moment capacity (reduced temperature)
254×254×73 Grade 43	XX	93	71	100*
	YY	56	41	56*
254×254×132 Grade 43	XX	230	151	220*
	YY	100	72	105*

(* calculated assuming a general 10% reduction to the cross-section temperatures)

It is important to recognize that CFIRE leads to higher moment capacities than the stress block method of BS 5950: Part 8. This is because of the assumed temperature variation along the column. The shape of the bending moment diagram is such that the extremities of the column are relatively cool and the bending capacities exceed the applied moment. The stress block approach of BS 5950: Part 8 was found to give good correlation when the temperatures throughout the cross-section were taken as 90% of the column mid-height values (i.e. a 10% reduction). This leads to an increase in moment capacity of about 20%.

Comparing the different grades of steel shows that about the weak axis grade 50 columns perform worse than grade 43 columns. The load ratio for axial load for grade 50 is only about 80% of the grade 43 value and a similar, but smaller, loss of strength occurs in minor axis bending with the load ratio for grade 50 being only 88% of that of grade 43. For major axis bending, both grades of steel perform similarly indicating that the steel flanges, although extremely hot, are the dominant influence rather than the concrete in-fill (Table 4).

Table 4 *Influence of steel grade on bending capacity of column*

	Moment capacity - grade 43 (kNm)			Moment capacity - grade 50 (kNm)		
Axis	Cold	Fire	Ratio	Cold	Fire	Ratio
YY axis	83.9	56	0.67	108.2	64	0.59
XX axis	271.9	100	0.37	351.0	126	0.36

Based on this analysis it is proposed that a linear form of force - moment interaction is used for design purposes with the axial capacity being based on the EC4 method and the moment capacity based on the plastic moment capacity using modified temperatures. This is likely to be conservative for moments about the major (XX) axis where CFIRE indicates a convex form of interaction.

The linear interaction may be presented as:

$$\frac{P}{P_f} + \frac{M_y}{M_{fy}} + \frac{M_x}{M_{fx}} \leq 1.0 \qquad (5)$$

Where

P	=	Axial load
P_f	=	Compressive capacity
M_x	=	Applied moment about XX axis
M_{fx}	=	Bending resistance about XX axis
M_y	=	Applied moment about YY axis
M_{fy}	=	Bending resistance about YY axis

All these are calculated at the *fire limit state*.

9. DESIGN METHOD

Using the thermal and structural models, design tables have been prepared for 60 minutes fire resistance for both grade 43 (Table 5) and grade 50 (Table 6) Universal Columns. It is not considered feasible to carry out the calculation of either the axial or moment capacities by hand so no simple hand methods are proposed. The design tables give

a) Compressive capacity in the absence of moments
b) Moment capacity in the absence of axial load for bending about the major, XX, and minor, YY, axes.

The linear force - moment interaction presented in equation (5) should be used.

In preparing the design tables a number of assumptions have been made. These are in line with the recommendations of EC4: Part 1.2[5], and are as follows:

Concrete strength

The design strength of concrete is based on the cylinder strength with the appropriate reductions for temperature. No material factors are used, as proposed in Eurocode 4: Part 1.2. A cylinder strength of 33 N/mm^2 has been assumed which corresponds to a cube strength of 40 N/mm^2.

Effective length

In the design tables the load capacity in fire is given for a range of effective column lengths. The column length in the table is the effective length that the designer assumes for normal design. For the fire design, the effective length is assumed to be reduced by a factor of 0.7 due to additional restraints that are present in fire. This additional factor is built into the program. Both EC3: Part 1.2 and EC4: Part 1.2 recognise that in fire the effective length is effectively reduced because the column ends will be colder than the middle, and upper and lower storeys may be assumed to be unaffected by the fire. Both these codes recommend a 0.7 modification factor to the effective length.

Compressive capacity

The compressive capacity of the web-infilled columns has been calculated using the method given in EC4: Part 1.2 and ECCS Technical Note 55[1]. These methods result in an increase in load ratio with increasing effective length. It was considered that in view of the limited scope of the test programme the small increases in load ratio would be ignored and the design tables are therefore based upon the load ratio for a normal design effective length of not more than 3 metres.

Moment capacity

The moment capacity for each section has been calculated using plastic theory. The temperatures for this calculation are based on the computed temperatures but with a general 10% reduction to the cross-section temperatures. This reflects the fact that the ends of a column are colder than the critical central portion, and is in-line with the CFIRE analyses. The approach is possibly conservative for bending about the major, XX, axis but it is felt that until better analyses or tests can be carried out a degree of conservatism is warranted.

Bending moment diagram

The method is based on a linearly reducing moment from a maximum at one end of the storey height to almost zero at the other end. In practice this will always be conservative because some end fixity will always exist and it is likely that this fixity will effectively increase in fire conditions.

Applicability

The design method is suitable for columns in simple construction designed for normal conditions in accordance with clause 4.7.7 of BS 5950: Part 1. It should not be used for columns in frames with moment connections.

Fire resistance

Design tables have been prepared for 60 minutes. Web-infilled columns were briefly examined to see if they could achieve 90 minutes fire resistance but the load capacities were found to be very low and the longest fire test was only 72 minutes. Therefore, no recommendations are given for 90 minutes fire resistance.

Load transfer

15 mm web stiffeners, of the same grade as the column, with 6 mm fillet welds on both sides must be provided at the top of the column to allow load to be transferred from the steel into the concrete. Similar stiffeners may be required at the bottom of the column or alternatively the floor slab may be used for compression transfer to the concrete if the column is cast in situ.

Shear connectors, either Hilti or welded, are required principally to retain the concrete and prevent spalling. They should be provided at no more than 500 mm centres and positioned in a staggered pattern either side of the centre line.

Table 5 *Compressive capacity (kN) and moment capacity for grade 43 web-infilled columns for 60 minutes fire resistance*

Section size	Moment capacity (kNm)		Load ratio	Compressive capacity (kN) (at effective lengths used for normal design)				
	M_{fx}	M_{fy}		2500 mm	3000 mm	3500 mm	4000 mm	4500 mm
203×203×46	40.3	22.9	0.57	744	682	616	551	488
203×203×52	47.0	25.4	0.54	803	738	669	600	533
203×203×60	56.7	29.0	0.52	882	811	735	659	586
203×203×71	69.9	34.6	0.49	974	899	820	740	662
203×203×86	92.2	43.6	0.48	1153	1064	971	876	784
254×254×73	100.2	56.0	0.67	1499	1415	1326	1233	1137
254×254×89	126.6	66.5	0.63	1671	1579	1482	1381	1276
254×254×107	163.7	80.9	0.60	1914	1812	1703	1590	1473
254×254×132	219.7	104.5	0.57	2258	2139	2015	1884	1748
254×254×167	307.1	141.3	0.56	2800	2656	2505	2347	2182
305×305×97	183.8	105.1	0.78	2411	2307	2198	2084	1964
305×305×118	229.0	122.6	0.73	2655	2544	2428	2307	2180
305×305×137	281.4	142.7	0.70	2967	2843	2714	2578	2437
305×305×158	342.2	167.0	0.67	3282	3147	3006	2859	2706
305×305×198	475.8	220.4	0.64	3941	3782	3616	3443	3262
305×305×240	636.2	284.6	0.64	4785	4595	4398	4191	3976
305×305×283	792.4	346.3	0.65	5570	5320	5067	4810	4548

M_{fx} = Moment capacity (kNm) in fire conditions for bending about x-x axis

M_{fy} = Moment capacity (kNm) in fire conditions for bending about y-y axis

Load ratio = Compressive capacity (kN) in fire divided by 'cold' compressive capacity, in the absence of end moments

Table 6 *Compressive capacity (kN) and moment capacity for grade 50 web-infilled columns for 60 minutes fire resistance*

Section size	Moment capacity (kNm)		Load ratio	Compressive capacity (kN) (at effective lengths used for normal design)				
	M_{fx}	M_{fy}		2500 mm	3000 mm	3500 mm	4000 mm	4500 mm
203×203×46	50.9	26.0	0.51	833	750	663	578	501
203×203×52	59.4	29.2	0.49	912	824	732	641	557
203×203×60	71.8	33.6	0.47	998	902	801	702	610
203×203×71	89.6	41.1	0.45	1130	1027	919	810	709
203×203×86	118.2	52.7	0.44	1336	1214	1086	957	837
254×254×73	126.7	63.6	0.59	1670	1564	1449	1326	1202
254×254×89	161.7	77.2	0.56	1894	1776	1648	1512	1373
254×254×107	209.4	95.7	0.54	2199	2065	1921	1766	1608
254×254×132	281.8	126.1	0.52	2629	2474	2305	2126	1940
254×254×167	394.9	173.4	0.52	3320	3129	2922	2700	2471
305×305×97	232.5	118.5	0.68	2671	2544	2408	2261	2105
305×305×118	292.4	141.2	0.64	2982	2845	2698	2541	2374
305×305×137	359.9	167.0	0.62	3367	3212	3047	2869	2681
305×305×158	438.4	198.2	0.60	3765	3596	3415	3220	3014
305×305×198	611.1	266.9	0.59	4655	4450	4231	3995	3745
305×305×240	818.8	349.7	0.59	5653	5409	5148	4868	4571
305×305×283	1045.1	437.8	0.60	6644	6322	5988	5642	5285

M_{fx} = Moment capacity (kNm) in fire conditions for bending about x-x axis

M_{fy} = Moment capacity (kNm) in fire conditions for bending about y-y axis

Load ratio = Compressive capacity (kN) in fire divided by 'cold' compressive capacity, in the absence of end moments

10. CONCLUSIONS AND RECOMMENDATIONS

Based on four fire resistance tests on web-infilled columns a design procedure has been developed. Compressive capacities for a range of column sizes and effective lengths have been calculated and are presented in tables. For Universal Columns in the range 203×203 up to 305×305 a reasonable proportion of the 'cold' capacity can be carried and 60 minutes fire resistance achieved. Expressed in terms of load ratio, values in the range 0.44 to 0.78 are achieved based on the axial resistance of the *steel* section to BS 5950: Part 1. In a typical building, load ratios are rarely greater than 0.5 for columns.

In the second phase of the tests a web stiffener was incorporated at the top of the column to assist in load transfer from steel to concrete and to act as a shutter for the wet concrete. This proved to be very successful in transferring load and the local buckling failures of the first tests were not repeated. An analysis of the worst case for shear transfer in the design tables has demonstrated that all the load transfer to the concrete can be via the web stiffener. This means that the shear connectors are only required to carry nominal loads and to assist in preventing bursting of the unreinforced concrete. It is therefore recommended that a 15 mm thick web stiffener with 6 mm fillet welds on both sides is used together with shear connectors at not more than 500 mm centres. Either Hilti or welded shear connectors may be used.

The effect of bending moments has been analysed. It was found that the loss of moment capacity about the weak axis is very similar to the loss in compressive capacity but the loss about the strong axis is much greater. This is because for normal design the flanges are dominant but in fire the exposed flanges are extremely hot and lose strength at a greater rate than the rest of the section. A force - moment interaction is proposed which is suitable for columns in simple construction where moments are relatively small. The method is not suitable for columns in moment resisting frames.

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