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**STEEL RTD**

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# STAINLESS STEEL IN FIRE (SSIF)

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# ABSTRACT

The relatively sparse body of existing data on the behaviour of structural stainless steel at high temperatures suggests that stainless steel performs very well between 600ºC and 800ºC due to its strength and stiffness retention characteristics. This report summarises the findings of a 3½ year European research project which studied the behaviour of a range of structural stainless steel solutions subject to fire loading. The project included tests on materials, members and connections, numerical analysis and development of design guidance aligned to the Eurocodes. It aimed to identify structural solutions which give a specified period of fire resistance without any fire protection applied to the surface of the steel.

The temperature development in a range of load-bearing and separating elements concepts designed to suppress temperature rise was studied. From a programme of tests and numerical analysis on RHS with slender (Class 4) cross-sections, more economic design guidance was derived. Long fire resistance periods were exhibited in fire tests on concrete-filled stainless steel RHS and hybrid stainless-carbon steel composite floor beams.

Strength and stiffness retention characteristics for two austenitic grades not previously studied were developed through a programme of transient state tests. The behaviour of external stainless steel columns and stainless steel columns in open car parks subject to realistic fire loads was studied numerically. Tests on welded and bolted connections in fire enabled design guidance to be derived. An online design facility for predicting the fire resistance of cold formed stainless steel sections was developed.

# FINAL SUMMARY

## 1. Objectives

The objective of this project is to develop more comprehensive and economic guidance on the design of stainless steel structural members and connections when exposed to fire, including specific products meeting the requirements for 30 and 60 minutes fire resistance without fire protection. The objectives were met through a series of test programmes which were subsequently modelled numerically to calibrate numerical tools for developing design guidance aligned to the Eurocodes.

## 2. WP1: Fire resistant structures and products

Limiting the temperature rise enables the load-bearing capacity of a member to be retained for a longer period. In this work package, the temperature development in a range of concepts designed to suppress temperature rise was studied. Using finite element analysis, the EN 1363-1 standard fire curve was applied for 60 minutes to a range of systems including:

- nested tubes (with the annulus between the sections either empty, filled with mineral wool or filled with concrete),
- a corner column section partially protected by concrete walls,
- a column exposed to fire from one side,
- two profiles side by side filled with mineral wool.

Unloaded fire tests on the most promising concepts made from grade 1.4301 stainless steel were then carried out (four on the load-bearing concept and four on separating structures). Numerical models of the tests were developed and parametric studies were carried out to develop an understanding of the parameters which affect the temperature rise in these concepts. The load-bearing systems successfully suppressed the temperature rise, however, the construction practicalities of these systems needs further consideration. Simple design guidance is needed for calculating the buckling resistance of columns taking into account non-uniform temperature distribution due to the protection offered by concrete walls to corner columns.

For wall elements of 120 mm thickness, 60 minutes fire resistance can be obtained. It was shown that the sandwich panel floor construction with a 120 mm depth could demonstrate 60 minutes fire resistance provided the mineral wool is effectively placed in the voids.

The superior behaviour of stainless steel members in fire compared to carbon steel members in the temperature range 600°C to 800°C was quantified.

## 3. WP2: Composite members in fire

Seven fire tests were carried out on loaded RHS columns filled with concrete (reinforced and unreinforced) designed to achieve a fire rating of 30 and 60 minutes made from grade 1.4401 stainless steel. The tests were modelled numerically and subsequently parametric studies were carried out in order to develop design rules for composite columns. The proposed design methods are consistent with the general flow charts in EN 1994-1-2 used to check the fire resistance of composite members but include some specific characteristics to account for the distinctive behaviour of stainless steel.

To compare the performance of stainless and carbon steel composite columns, a numerical study was carried out on different RHS column cross-sections filled with unreinforced concrete. It is clear that carbon steel columns buckle at a lower load than stainless steel columns of identical size and length.

Two fire tests were carried out on hybrid stainless-carbon steel composite beams from grade 1.4401 with the stainless steel lower flange exposed and the carbon steel section unexposed. The specimens were 5 m in length and designed to achieve a fire rating of 30 and 60 minutes. The tests were modelled numerically and subsequently parametric studies were carried out in order to develop design rules for composite beams. The proposed design method is based on simple plastic moment theory, requiring the calculation of the neutral axis and corresponding moment resistance by taking into account the temperature distribution through the cross-section and the corresponding reduction in material strength.

To compare the performance of stainless and carbon steel composite beams, a numerical study was carried out on different beam cross-sections. For the same fire rating, the bending moment resistance of carbon steel beams is always lower than the beam with the exposed lower flange from stainless steel.

## 4. WP3: Class 4 cross sections in fire

A programme of six fire tests on loaded RHS columns with slender (Class 4) cross-sections was performed. The length of the columns was 0.9 m. Numerical models were calibrated against test results and then parametric studies carried out to develop more economic design guidance than is currently in existing guidance. The proposed model uses the room temperature buckling curve with the global, local and limiting slendernesses all being related to the temperature-dependent ratio of strength to stiffness. The analysis of 3.1 m long pinned columns in a standard fire shows that it is possible for unprotected Class 4 stainless steel columns to achieve 30 minutes fire resistance if the load level does not exceed 0.3.

## 5. WP4: Properties at elevated temperatures

Strength retention curves for two grades of stainless not previously studied were derived through a programme of transient state tests. The grades studied were the stabilised austenitic grade 1.4541 and STR18, a low nickel, high manganese and nitrogen austenitic steel with high strength. Using the test results, strength and stiffness parameters were derived for use with the numerical model in EN 1993-1-2.

## 6. WP5: Bolts and welds at elevated temperatures

Steady state (isothermal) tests were carried out on butt welded joints in grades 1.4318 and 1.4571 austenitic stainless steel. The strength retention factors for the butt welded joints for both the stainless steel grades were compared to factors for the base material given in the *Design Manual for Structural Stainless Steel*. It was concluded from the test results that the design strength of a full penetration butt weld, for temperatures up to 1000°C, could be taken as equal to the strength of the base material for grades 1.4318 and 1.4571 in the annealed condition.

Over forty isothermal tests from room temperature up to 900°C were performed on bolt assemblies in tension and shear; two grades of bolt were tested, A2-70 and A4-80. The tests showed that stainless steel bolts act better than carbon steel bolts at high temperatures beyond 400 to 450°C. Grade A4-80 bolts performed slightly better than grade A2-70. Based on the test results, strength retention factors were derived for stainless steel bolts.

## 7. WP6: Parametric fire design

The behaviour of external stainless steel columns and stainless steel columns in open car parks subject to realistic fire loads was studied numerically. The temperature distribution in stainless steel columns located outside a building on fire was studied and the performance was compared to equivalent columns from carbon steel grade S235. For the scenarios studied with a load level of 0.3, carbon steel columns failed after less than 30 minutes fire exposure, whereas the stainless steel columns remained stable throughout the whole fire duration. A simplified design approach was developed for external stainless steel columns.

The behaviour of stainless steel columns in open car parks of steel and concrete composite construction was studied using a fire safety engineering procedure developed in France and validated against experimental results. Numerical investigations enabled the maximum load level for unprotected stainless steel hollow columns to be determined.

## 8. WP7: Design aids and software

Rather than having a discrete set of strength retention curves for each grade of stainless steel, a preliminary set of generic strength retention curves was developed.

A less conservative approach for determining the fire resistance of stainless steel structural members was developed and published in the Third Edition of the *Design Manual for Structural Stainless Steel*.

Online software for predicting the fire resistant design of cold formed stainless steel structural members was developed (www.steel-stainless.org/software).

## 9. Conclusions

This project has investigated the performance in fire of a number of different stainless steel structural systems. Valuable fire test data have been generated. The design procedures developed now need to be tested out by practicing engineers before being submitted to the CEN technical committees responsible for preparing amendments and revisions to the Eurocodes.

This Summary Final report and the individual Work Package Reports can be downloaded at www.steel-stainless.org/fire.

# 1 INTRODUCTION

Stainless steel has many desirable characteristics which can be exploited in a wide range of construction applications. It is corrosion-resistant and long-lasting, making thinner and more durable structures possible. It presents architects with many possibilities of shape, colour and form, whilst at the same time being tough, hygienic, adaptable and recyclable. In recognition of the many desirable properties of stainless steel, a series of research projects to generate design guidance have been carried out over the last 20 years. Stainless steel structural members are designed in a similar way to carbon steel members. However, stainless steel exhibits different stress-strain behaviour to carbon steel, and this affects the design procedures for calculating buckling resistance and deflections. As a result of these research projects, European design guidance for structural stainless steel has been developed, for example in Eurocode 3, Part 1.4 (EN 1993-1-4)[1] and in the European *Design Manual for Structural Stainless Steel* (Third Edition)[2].

All metals lose strength and stiffness when heated, though there is considerable variation in the rate of the degradation of mechanical properties between different metals. Austenitic stainless steels exhibit better strength retention than carbon steels above about 550ºC and better stiffness retention at all temperatures (Figure 1.1 and Figure 1.2). The main reason for this is the difference in crystal structure of the two metals. The atoms in an austenitic microstructure are more closely packed than in carbon steels, which have a ferritic microstructure. Austenitic stainless steels have a relatively high level of alloying elements compared to carbon steels. Alloying additions tend to lower the diffusion rates of atoms within the crystal lattice at a given temperature which slows down the softening, recrystallisation and creep deformation mechanisms which control strength and plasticity at elevated temperatures. Additionally, carbon steels undergo transformation from ferrite to leanly alloyed austenite on heating. The austenitic steels, in contrast, do not undergo a structure change in the range of temperatures relevant to fire resistant design.

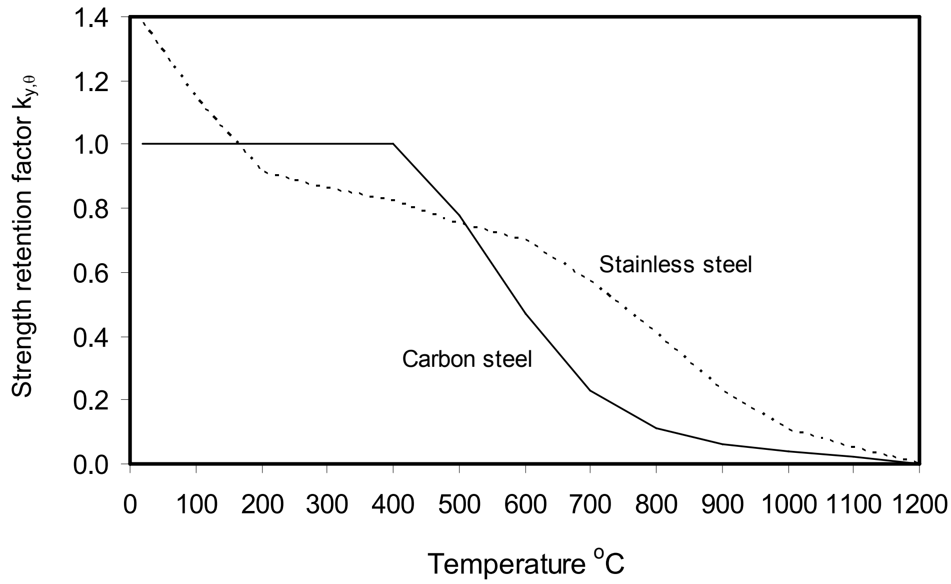



**Figure 1.1** *Comparison of stainless steel and carbon steel strength retention factors*

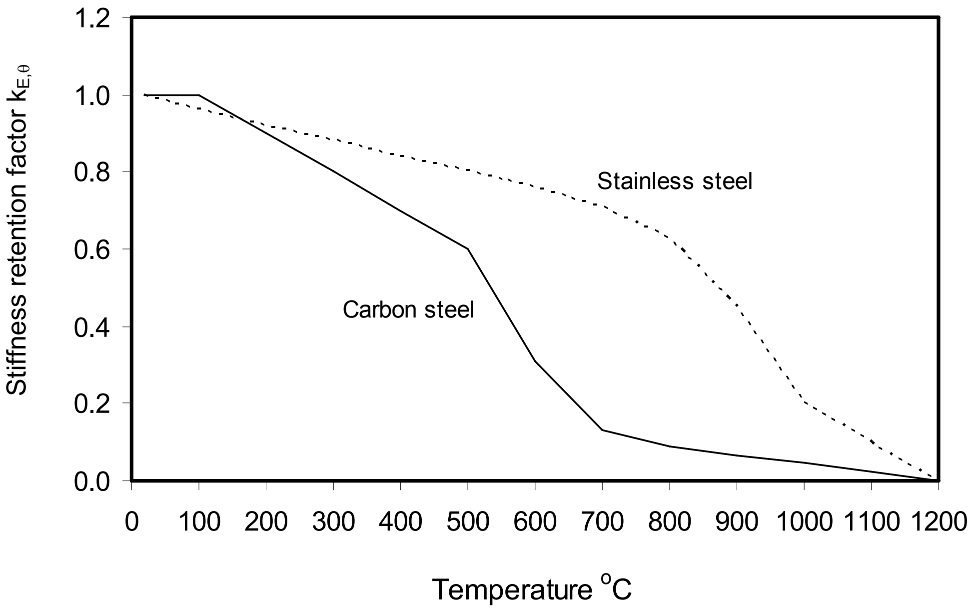



**Figure 1.2** *Comparison of stainless steel and carbon steel stiffness retention factors.*

As a result of the superior strength and stiffness retention, stainless steel columns and beams generally retain their load-bearing capacity for a longer time than equivalent carbon steel columns. Based on the results of work carried out under the ECSC funded project *Development of the use of stainless steel in construction*[3] guidance on fire resistant design is included in an informative annex in EN 1993-1-2, the Eurocode dealing with structural fire design of steel structures[4]. The studies into fire resistant design carried out under this project were fairly limited (for example welded, open sections or hollow sections filled with concrete were not studied and the guidance for Class 4 cross-sections was very conservative). The thermal and material properties at elevated temperatures for five grades of stainless steel are given in EN 1993-1-2: three austenitic grades (1.4301, 1.4401/4, 1.4571), one duplex (1.4462) and one ferritic (1.4003).

A more recent ECSC project *Structural design of cold worked austenitic stainless steels*[5] included a Work Package studying the behaviour of cold worked stainless steel members in fire and the results are included in the Third Edition of the *Design Manual for Structural Stainless Steel*.

The question whether stainless steel members can be used in buildings in load-bearing applications without fire protection is critical because aesthetic considerations are often the reason for specifying stainless steels in building structures. Eliminating the fire protection of structures will result in lower construction costs, a shorter construction period, more effective interior space utilisation, a better working environment and more aesthetic building design. Furthermore, the life-cycle costs of unprotected stainless steel structures are low. The increasing use of fire safety engineering presents good opportunities for unprotected structures based on materials with improved mechanical characteristics at high temperature.

Economic considerations mean it would be unlikely that stainless steel would be chosen solely because of its superior fire resistance. However, for specifiers considering stainless steel because of its aesthetic and durability properties, the additional benefit of providing fire resistance for a significant period whilst unprotected, might sway the balance in the favour of stainless steel. In applications where good corrosion resistance coupled with good fire resistance are required, stainless steel offers an excellent solution.

# 2 PROJECT OBJECTIVES

The objective of this project is to develop more comprehensive and economic guidance on the design of stainless steel structural members and connections when exposed to fire, including specific products meeting the requirements for 30 and 60 minutes fire resistance without fire protection.

The technical objectives are:

- To generate structural solutions where it is possible to use stainless steel structural members in buildings without fire protection, both considering the 'standard' fire and lower, more realistic fire loads.
- To generate test results on commonly used grades of stainless steel in structures; this will include tests on material, members and connections.
- To develop numerical models based on standardised methods and validated against the test results in order to generate additional data upon which a basis of design for a range of grades and types of members and connections can be established.

The commercial objectives are:

- To develop a methodology in the form of fire resistant design rules suitable for incorporation into standards that enable stainless steel members and connections to be designed cost effectively and safely in structures.
- To ensure that the deliverables of the project are in a format that is readily disseminated and used in the EU by incorporating them into European Standards. This will be achieved by the direct involvement of many of the key members of CEN committees in the project. This will maximise the likelihood of acceptance and incorporation of the rules in the standards within the necessary timescales.

# 3 WP1: FIRE RESISTANT STRUCTURES AND PRODUCTS

Detailed descriptions of the activities carried out under this work package are given in the relevant Final Work Package Reports listed in Section 11.

## 3.1 Objectives

There is a large difference between the price of carbon and stainless steel. This work package aims at the identification of structural solutions where stainless steel shows distinctive advantages over carbon steel. The main objective is to develop new stainless steel products without passive or active fire protection that can achieve 30 or 60 minutes fire resistance in a standard fire or in a natural fire. The new products will include fire-separating members and load-bearing structures.

## 3.2 Experimental work

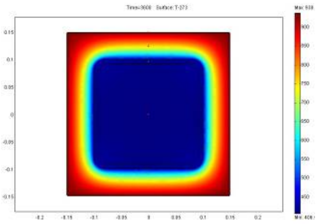
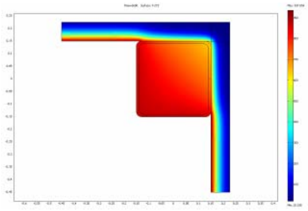
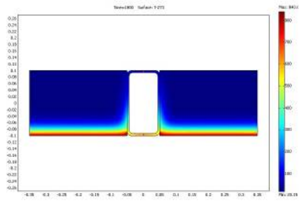
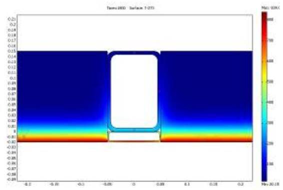
### 3.2.1 Load-bearing structures

Taking into account the demands of ease of maintenance, corrosion resistance and aesthetic appearance, various concepts for stainless steel load-bearing concepts were developed. Finite element thermal analyses were carried out on ten load-bearing cross-sections to predict the temperature development after 60 minutes exposure to the EN 1363-1[6] (ISO 834-1) standard fire curve. The heat transfer was assumed to happen through radiation and convection. The thermal properties for stainless steel were taken from EN 1993-1-2[4] and for concrete from EN 1992-1-2[7]. The exact thermal properties for the mineral wool were not available; upper and lower bounds relating to mineral wool with densities of 30 and 140 kg/m$^3$ were used. From the results of the thermal analyses, four test configurations were identified (Table 3.1). Figure 3.1 shows the predicted temperature rise for the nested column concept.

The steel columns were heated in a model furnace specially built to test loaded columns and beams. The test furnace was designed to simulate conditions to which a member might be exposed during a fire. It comprised a furnace chamber located within a steel framework. The interior of the furnace chamber was 1500 mm wide, 1300 mm high and 1500 mm deep. The interior faces of the chamber were lined with fire resistant bricks. Four oil burners were arranged on the two walls inside the furnace (two burners in each wall). The specimens were unloaded in the fire tests.

In all cases the temperatures measured by furnace thermocouples were averaged automatically and the average used for controlling the furnace temperature. Temperature readings were taken at each thermocouple at intervals of 10 seconds. Observations were made of the general behaviour of the specimen during the course of the tests and photographs and video film were taken. Figure 3.2 shows two test specimens.

**Table 3.1** *Load-bearing fire test specimens with predicted temperature distributions*

| Test | | Material | Profiles |
|---|---|---|---|
| 1- Nested tubes with fire protection (injected mineral wool) |  | EN 1.4301 and mineral wool 30 kg/$m^3$ | RHS 300 x 300 x 10 & RHS 200 x 200 x 8 |
| 2- Column section in corner, Siporex[1)] wall |  | EN 1.4301 | RHS 300 x 300 x 10 Siporex[1)] wall (150 mm thick) |
| 3- Column exposed to fire from one side |  | EN 1.4301 | RHS 200 x 100 x 6 |
| 4- Column of two parts exposed to fire from one side |  | EN 1.4301 | RHS 150 x 100 x 6 & RHS 20 x 100 x 2 |

[1)] Siporex is lightweight autoclaved aerated concrete which is completely cured, inert and stable form of calcium silicate hydrate. It is a structural material, approximately one quarter the weight of conventional concrete, composed of minute calls which give the material light weight and high thermal insulation properties. It is available as blocks and pre-cast reinforced units, i.e. floors, roofs, walls and lintels.

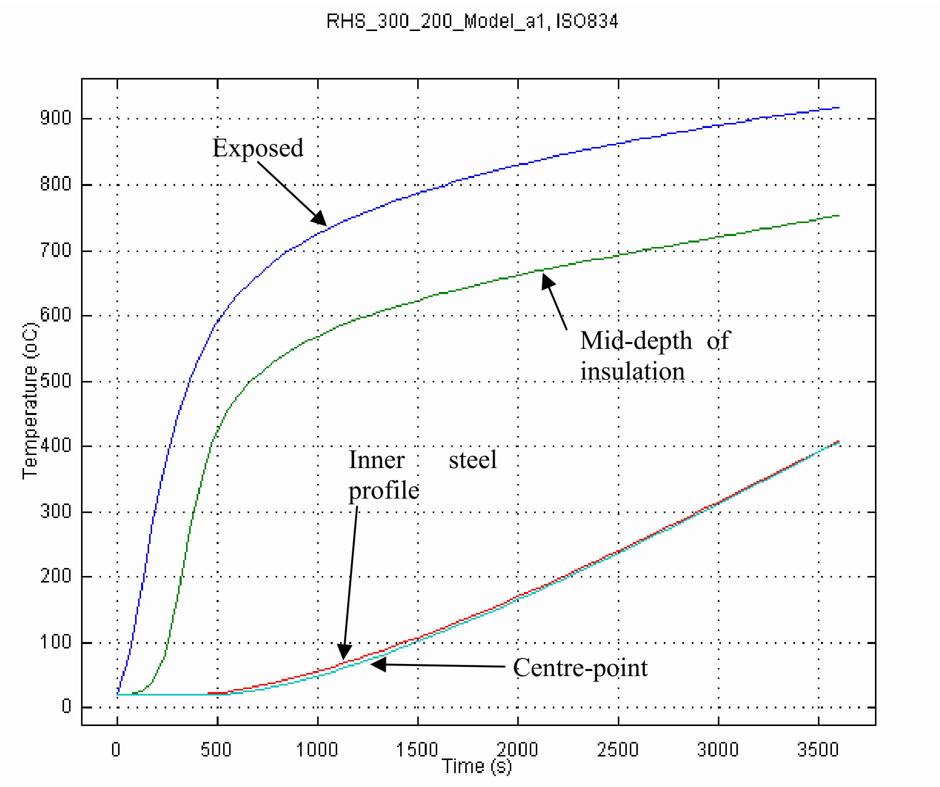



**Figure 3.1** *Predicted temperature rise for the nested column concept*

**Figure 3.2** *Load-bearing test specimens:*
*Left: Nested tube prior to testing, Right: Corner column during test*

### *Nested tubes*

The temperature at mid height of the furnace averaged 973°C after 60 minutes in the standard fire. In the middle of the outer tube (RHS 300x300x10), the mean value of measured temperatures was 925°C and in the middle of the inner tube (RHS 200x200x8), the mean value of measured temperatures was 414 °C. This means that the inner tube had about 60% of its capacity left according to EN 1993-1-2.

### *Column section in the corner*

The temperature differences between the exposed and unexposed sides were remarkable. The maximum temperature in the exposed corner (mid height of the column) was 878°C and in the unexposed corner (mid height of the column) was 466°C after 60 minutes standard fire. To utilise the low temperatures, the connection between the column and wall should be ensured.

### *Column exposed to fire from one side*

The temperature differences between the exposed and unexposed sides were remarkable. The maximum temperature in the exposed side (mid height of the column) was 806°C and in the unexposed side (mid height of the column) 299°C after 60 minutes standard fire.

### *Column of two parts exposed to fire from one side*

The temperature differences between the exposed and unexposed sides were remarkable. The maximum temperature in the exposed side (mid height of the column) was 871°C, between the two parts (measured from RHS 100x150x6) 583°C and in the unexposed side (mid height of the column) 95°C after 60 minutes exposure to the standard fire.

## 3.2.2 Separating structures

As with the load-bearing structures, finite element thermal analyses were carried out on eleven cross-sections to predict the temperature development after 60 minutes exposure to the EN 1363-1[6] (ISO 834-1) standard fire curve. Based on the Eurocode failure criteria, a separating structure fails when the temperature on the unexposed side rises to an average of 140°C or a maximum of 180°C. From the results of the thermal analyses, four test configurations were identified (Table 3.2).

**Table 3.2** *Separating structures fire test specimens*

| Test | Material | Furnace | Profiles |
|---|---|---|---|
| 1- Floor structure, corrugated core sandwich panel with fire protection (mineral wool) | EN 1.4301 & mineral wool | Cubic furnace | The dimensions of the specimen about 1.25 m x 1.25 m |
| 2- Wall structure, Z-profiles, with fire protection (mineral wool) | EN 1.4301 & mineral wool | Cubic furnace | The dimensions of the specimen about 1.25 m x 1.25 m |
| 3- Wall structure, Z-profiles, with fire protection (mineral wool) | EN 1.4301 & mineral wool | Cubic furnace | The dimensions of the specimen about 1.25 m x 1.25 m |
| 4- Floor structure, corrugated core sandwich panels, load-bearing structure | EN 1.4301 | Horizontal furnace | The max. dimensions about 5 x 3 m |

The wall elements consisted of 2 mm thick stainless steel top and bottom plates, connected by welding to the flanges of stainless steel Z profiles 60x15x1.5 which were spaced at 300 mm centres (Figure 3.3). The void between the Z profiles and the plates were filled by blowing mineral wool with an approximate density of 75 kg/m$^3$. The total thickness of the wall elements were 64 mm.

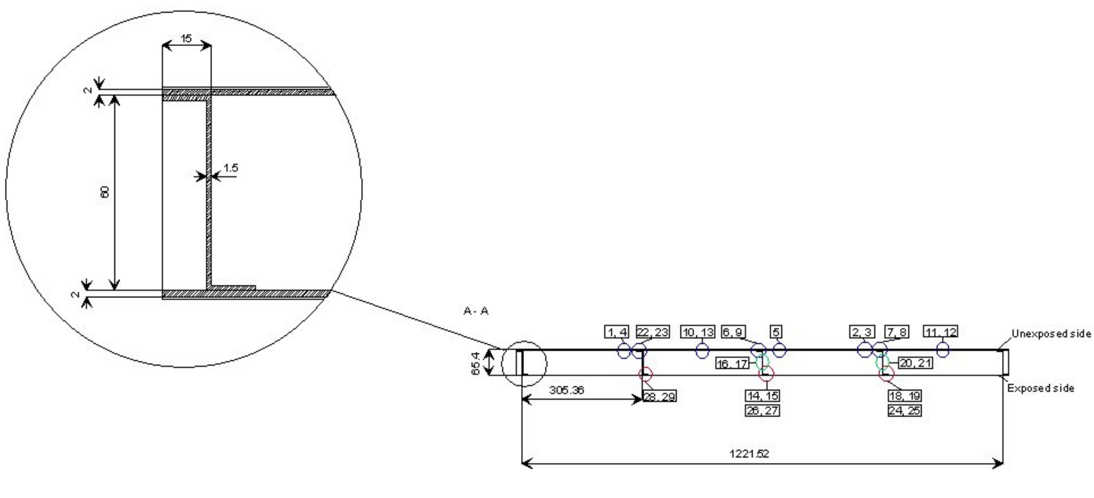



**Figure 3.3** *Wall structure test specimen: geometry and position of temperature measuring points*

Due to problems in the preparation of the first wall test specimen, only one of the three interior Z profiles was welded onto the top and bottom plates. The other two were only welded onto the top plate (on the unexposed side). This meant that there was a small air gap between the exposed side plate and the flanges of two of the interior Z profiles, and probably between the plate and the mineral wool. In the second wall test specimen, the top flange of the properly welded Z-profile.

The floor structures were corrugated core sandwich panels with mineral wool fire protection, designed by the Finnish Company Kennotech. The panels were laser welded, with stainless steel plates as cover plates (1.5mm on exposed side, 3 mm on unexposed side) and 2 mm thick V profiles forming the web (Figure 3.4). The total thickness of the floor elements were 124.5 mm. The voids in the core were filled with blowing mineral wool with an estimated density of 75 kg/m$^3$. However, a calculation based on the actual weight of the test specimen and the volume of the insulation material showed that the actual density was approximately 115 kg/m$^3$.

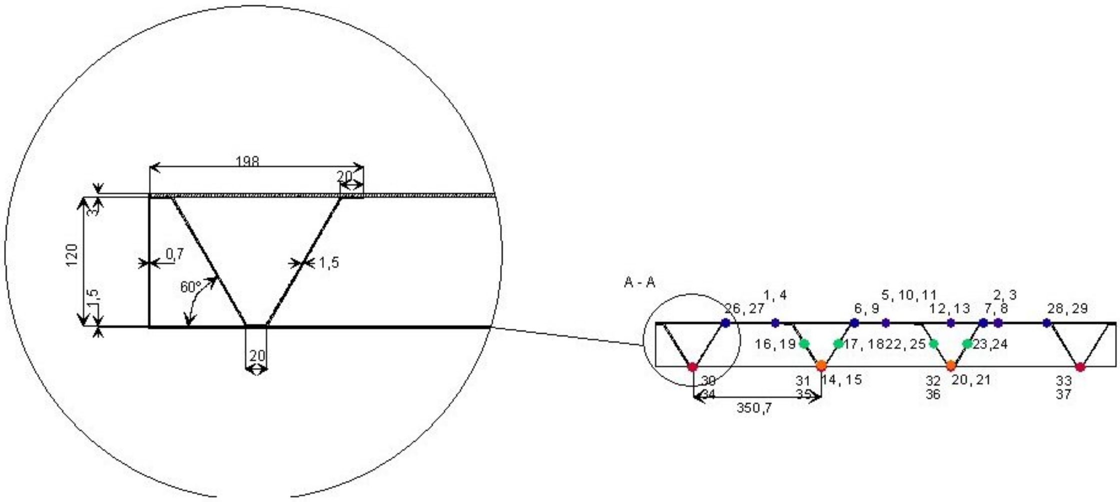



**Figure 3.4** *Floor structure test specimen: geometry and position of temperature measuring points*

The first three tests were small-scale unloaded tests in a cubic furnace and the fourth test was full-scale and loaded in a horizontal furnace (Figure 3.5). The elements were installed onto the top opening of the furnace so that their bottom surface was exposed to heating and the top surface was open to the testing hall.

**Figure 3.5** *Large scale loaded fire test on floor structure*

The wall elements did not pass the Eurocode criteria for 60 minutes fire exposure. The temperature rise measured in the small-scale unloaded floor test specimen was less than the limits specified and the system thus achieved a 60 minute fire resistance. The loaded floor test specimen similarly was expected to have a fire resistance of at least 60 minutes. However, there were voids in the core which had not been properly filled with mineral wool which led to very high temperature rises in part of the floor and so the test had to be terminated after 47 minutes.

## 3.3 Numerical studies

### 3.3.1 Load-bearing structures

***Nested tubes (thermal analysis)***

The temperature development in the columns was modelled using the material properties given in EN 1993-1-2 and compared with the temperature development using revised properties proposed by Gardner and Ng[8] and also carbon steel properties. Gardner and Ng proposed an emissivity of 0.2 for stainless compared to 0.4 given in EN 1993-1-2, and a heat transfer coefficient of 35 W/m$^2$K compared to 25 W/m$^2$K in EN 1993-1-2. These values were derived by analysing tests from 3 different laboratories in Finland, France and the UK. It is therefore thought that they are reliable parameters for describing behaviour in fire tests, but perhaps not for describing behaviour in real fires where soot is likely to build up on the surface after a short time. The issue of what should be used in design was debated by the project partners since there are a number of conservatisms already built into the design process (e.g. standard fire curve); adopting the more conservative value of 0.4 for emissivity may lead to unnecessarily uneconomic design for stainless steel.

EN 1993-1-2 gives a value of 0.7 for the emissivity of carbon steel and 25 W/m$^2$K for the heat transfer coefficient.

Numerical analyses were carried out with a two-dimensional ABAQUS model. The insulation material was characterised by a temperature gradient and the outer and inner tubes showed approximately uniform cross-section temperatures. The cross-sections were exposed to the EN 1363-1 standard fire and the temperatures were measured at the corners of the tubes, as shown in Figure 3.6. The temperature difference obtained with the different sets of material properties is small, but it is observed that the use of the Gardner and Ng values introduces an improvement in the fire resistance of stainless steel. Subsequent analyses were carried out using the material properties proposed by Gardner and Ng.

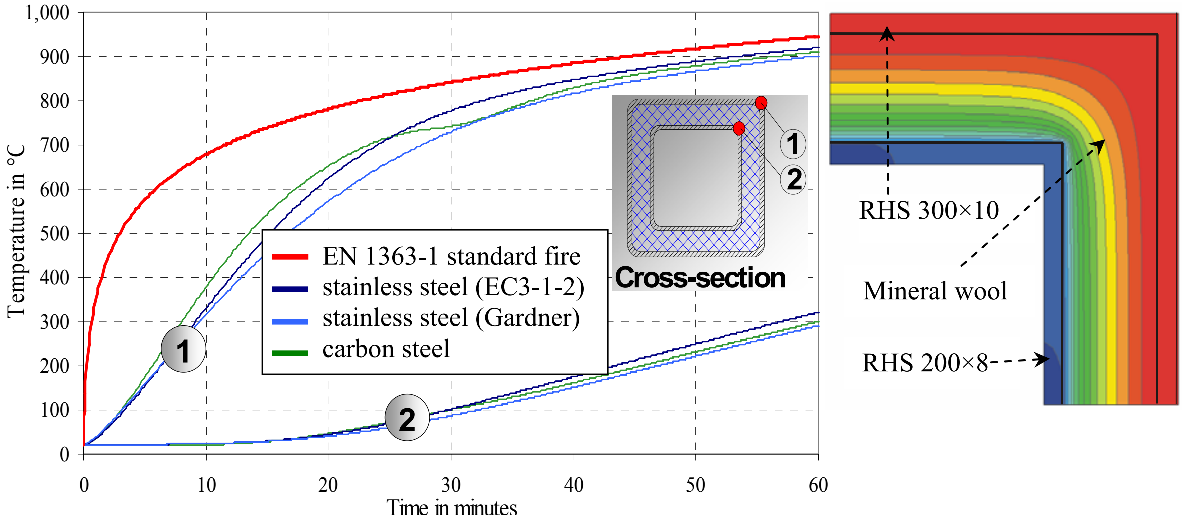



**Figure 3.6** *Cross-sectional temperatures of nested tubes for varying material properties*

Figure 3.7 and Figure 3.8 compare the measured temperatures with the predicted temperatures in the outer and inner RHS respectively. There is a substantial variation in the measured temperatures around the tube due to non-uniform heating effects. In both tubes the predicted temperatures are lower than those recorded during the test, although the difference is greater in the inner tube due to the uncertainties connected with modelling of the mineral wool.

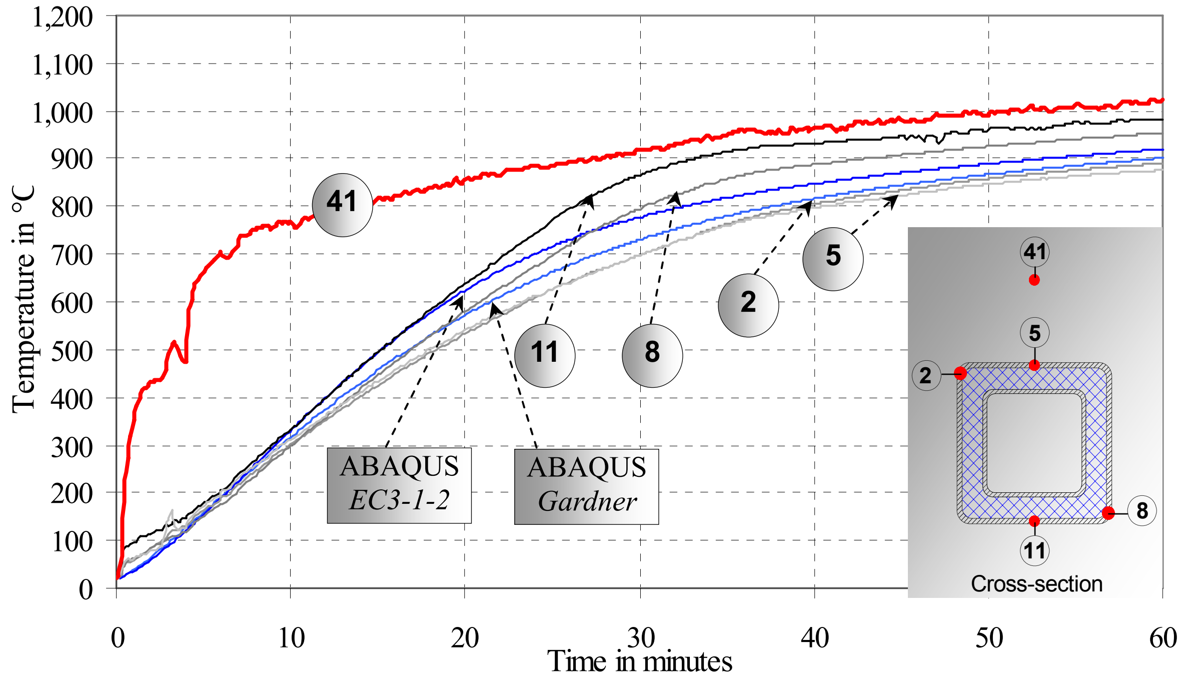



**Figure 3.7** *Comparison between test data and numerical results for outer tube*

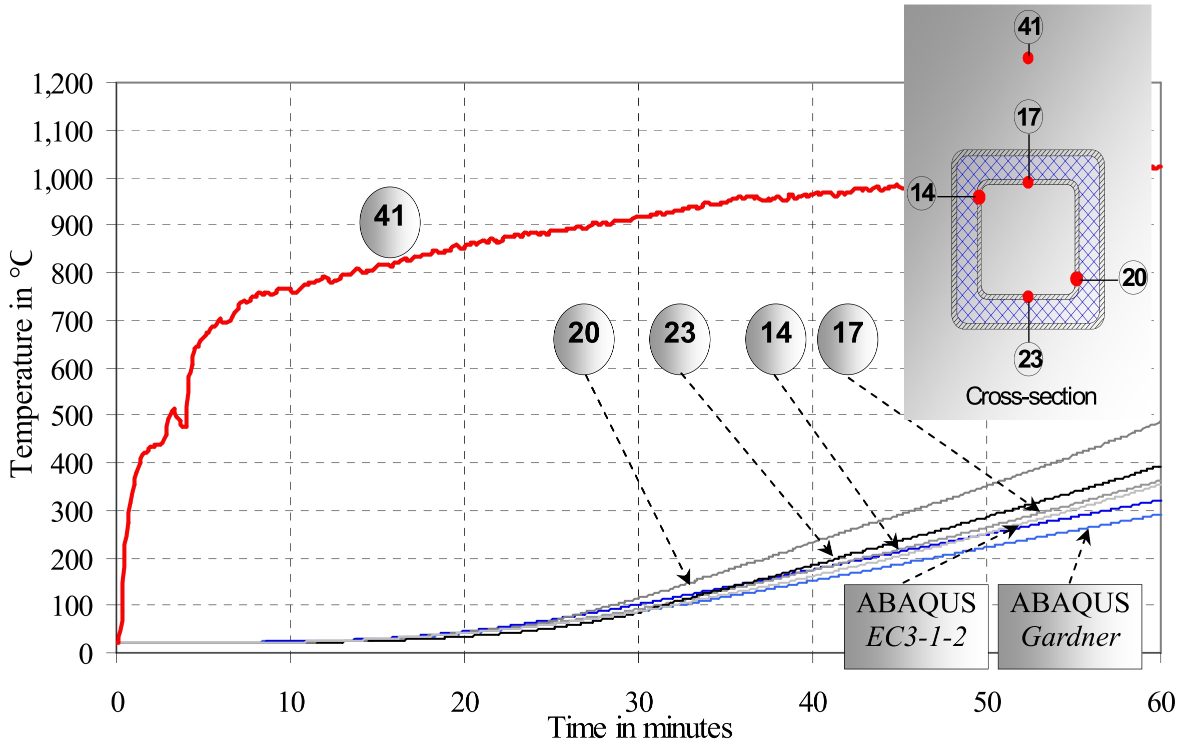



**Figure 3.8** *Comparison between test data and numerical results for inner tube*

### *Comparison of stainless steel and carbon steel column behaviour in fire*

The structural performance of a stainless steel RHS column (200x200x8) from grade 1.4301 was compared to the performance of an identical carbon steel column from grade S235 at different temperatures using finite element analysis (Figure 3.9). An imperfection factor of L/300 was assumed. The ultimate load-bearing capacity was calculated by incrementally increasing the applied load. For a cross-sectional temperature of 400°C, the stainless and carbon steel columns showed similar load-bearing capacity. At 600°C, the stainless steel columns exhibited much higher load-bearing capacity than the carbon steel columns; the ratio of ultimate loads of the stainless to the carbon steel column is about 2.0. The explanation for this is that the superior stiffness retention of stainless steel prevents early global instability. This improves the flexural buckling behaviour of the column leading to smaller lateral deflections and reducing second order effects. For temperatures between 600°C and 800°C, the ratio of stainless to carbon steel ultimate load rises significantly, clearly demonstrating that stainless steel columns show superior load-bearing behaviour to carbon steel columns in this temperature range.

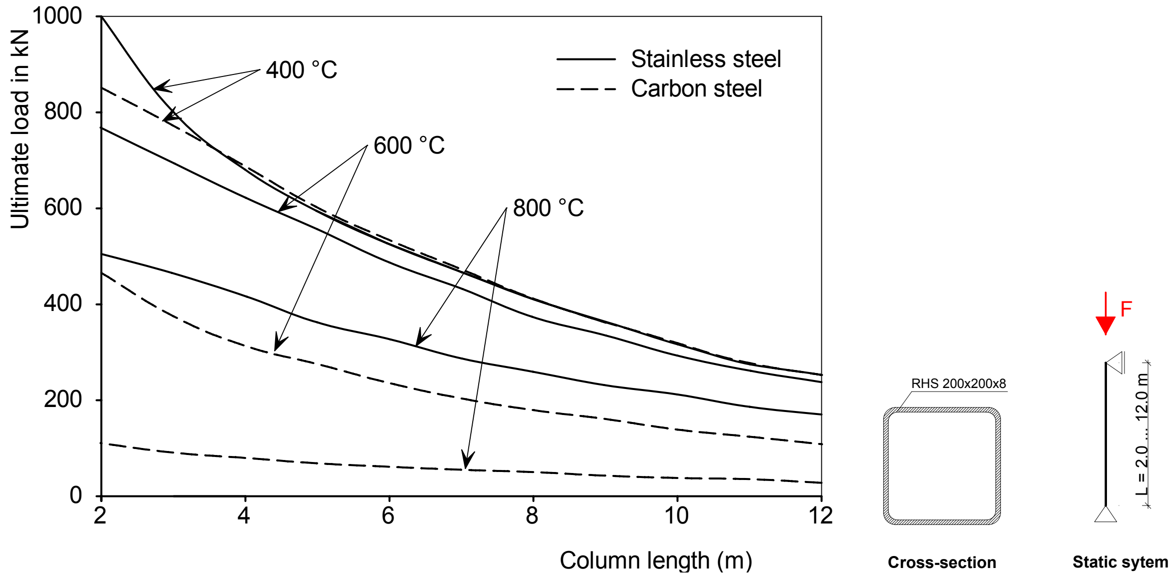



**Figure 3.9** *Ultimate loads for varying column length and cross-sectional temperatures*

In order to compare the maximum load levels in an identical carbon steel and stainless steel column, a further thermal analysis was carried out on a single RHS 200x200x8. (Note that the load level is the ratio of the buckling resistance at the fire ultimate state to the buckling resistance at room temperature.) After 30 minutes in the standard fire, the stainless steel RHS reached 698°C and the carbon steel RHS 740°C. After 60 minutes in the standard fire, the stainless steel RHS reached 848°C and the carbon steel RHS 896°C. A load-bearing analysis was then carried out for the heated stainless steel and carbon steel columns, assuming a column length of 3.5 m. The maximum load level for the stainless steel column was 84% compared to 22% for the carbon steel column.

### *Column in Siporex wall (thermal analysis)*

A two-dimensional analysis of the column was carried out. Heat transfer included both radiation and convection. Cavity radiation within the hollow section was neglected. Figure 3.10 shows the heated Siporex cross-section with a stainless steel column after 30 and 60 minutes exposure to the EN 1363-1 standard fire. The Siporex wall is clearly efficient in isolating the integrated column. After 60 minutes the whole cross section was heated, but temperature differences of up to 600°C were observed between the exposed and the unexposed sides.

The results obtained from the numerical analysis were compared with the test results. For the 3 points where the temperature was measured, the comparison was not conclusive. At measuring point 1 (exposed side) the analysis gave conservative values for the temperature. At measuring point 2 (mid cross-section) the 3 thermocouples recorded different temperatures, and the analysis coincided with one of them. At measuring point 3 (unexposed side) the recorded temperatures diverged slightly and the numerical simulation predicted slower heating.

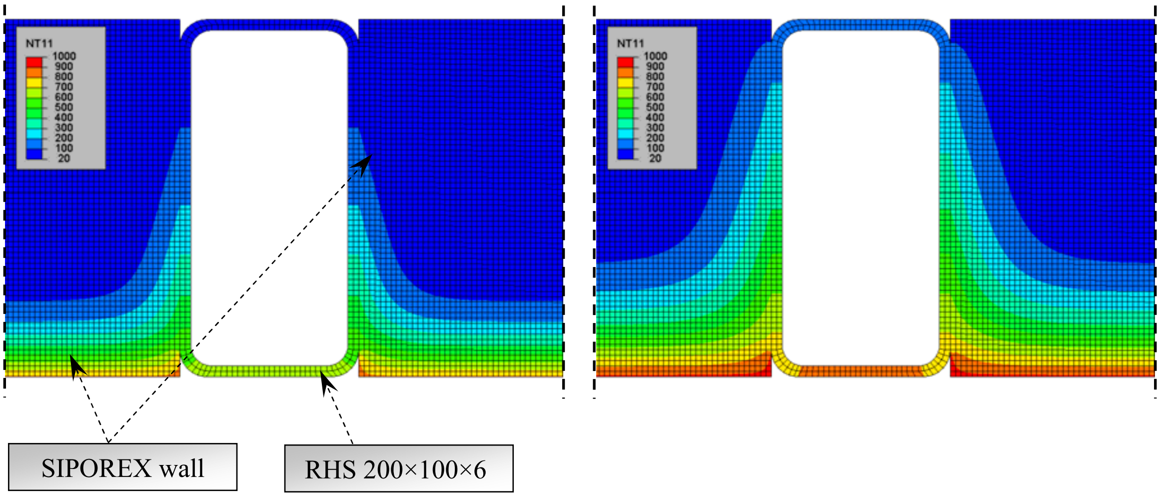



**Figure 3.10** *Heated cross-section after 30 minutes (left) and 60 minutes (right) exposure to EN 1363-1 standard fire*

### *Column in Siporex wall (mechanical analysis)*

The cross-sectional temperatures obtained from the thermal analysis were transferred to the three-dimensional mechanical model. Analyses were carried out to establish the load-bearing behaviour of stainless steel structures at elevated temperatures.

Two sets of analysis were run on a 3 m column, as shown in Figure 3.11. The hinged end conditions, with the head of the column not restrained axially, resemble columns in one-storey buildings and the fixed end conditions resemble columns in multi-storey buildings in which only one storey is exposed to fire and each storey is separated from the others by appropriate fire protection.

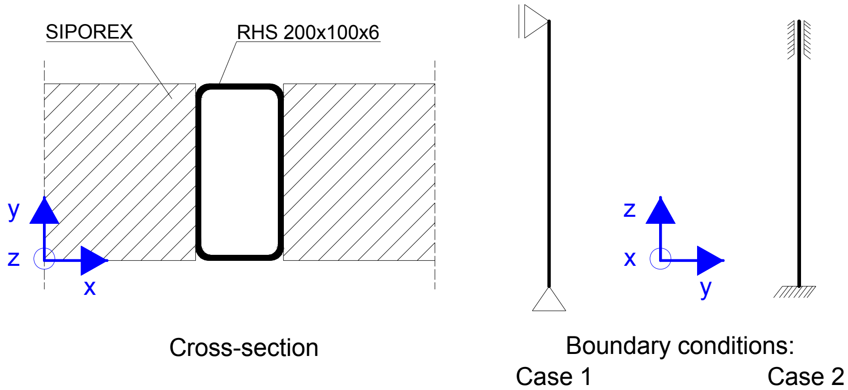



**Figure 3.11** *Cross-section and sets of boundary conditions for column in Siporex wall*

The analysis was carried out in two steps:

1. Buckling analysis: linear elastic eigenmode simulations to take the effects of local and global imperfections into consideration. The resulting deformed shape was used in the load-displacement analysis
2. Load-displacement non-linear analysis: a vertical load of 359kN (load ratio of 50% at room temperature) was applied with a 20 mm eccentricity at the top of the stainless steel column. The column was heated for 60 minutes according to the temperature amplitudes obtained from the thermal analysis. If the column still had load-bearing capacity after having been heated up, then the load was increased until failure occurred.

For case 1 (pinned ends), the stainless steel column failed by flexural buckling. At 350°C the displacement observed was about 40 mm. As the temperature rose beyond 350°C, the displacement increased very rapidly. The column failed after approximately 14 minutes. For S235 carbon steel, the column achieved 30 minutes fire resistance, with a maximum deflection of 10 mm, in contrast with 520 mm in the stainless steel column. The different failure modes resulted from the different stress-strain characteristics of carbon and stainless steel. The strains measured for both columns were comparatively low - the ultimate strain for the stainless steel column was approximately 1 %, whereas the carbon steel column failed with an ultimate strain exceeding 3 %. Figure 3.12 compares the stress-strain relationships of stainless and carbon steel at elevated temperatures and strains less than 5 %. For small strains and temperatures less than 500°C, carbon steel tends to be stiffer and stronger than stainless steel. The stainless steel column protected by the Siporex wall only reached comparatively low temperatures but the stiffness of the column was reduced which led to large deformations and additional second order effects. In contrast to this, the carbon steel column failed by local buckling of the fire-exposed side due to the sharply reduced yield strength at temperatures around 700°C.

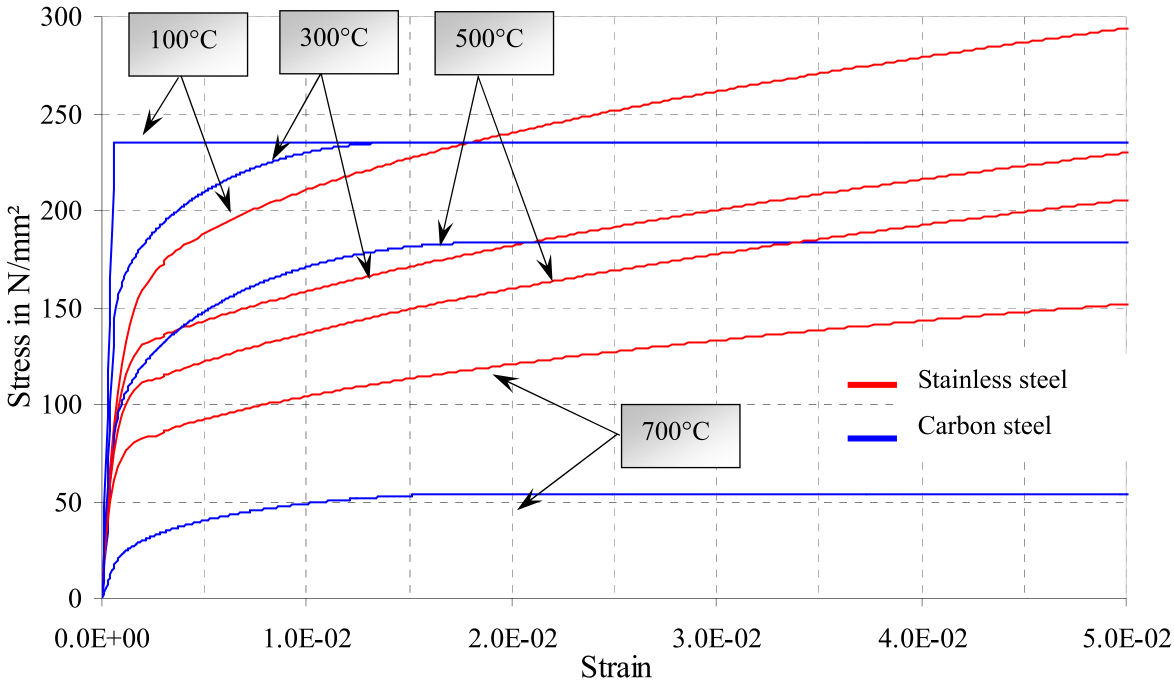



**Figure 3.12** *Comparison of stress-strain relationship at elevated temperatures*

For case 2 (fixed ends with large displacements prevented) both columns failed by local buckling. The stainless steel column reached an ultimate load of 368 kN after 60 minutes of exposure to the standard fire (corresponding to a 51% load level), whereas the carbon steel column failed after 33 minutes.

### 3.3.2 Separating structures

#### *Wall element (thermal analysis)*

Thermal models were developed of the tests. The estimated density of the blowing mineral wool material was 75 kg/m$^3$, but the actual density calculated on the basis of the floor test specimen was about 115 kg/m$^3$. Material data for mineral wool slabs with densities 30 kg/m$^3$ and 140 kg/m$^3$ were available, so it was assumed that by modelling the structures twice by using each of these mineral wool slab materials, upper and lower bounds for the unexposed side temperatures could be obtained. This proved to be a correct assumption on the basis of the modelling reported herein. Furthermore, it was noted that on the unexposed side, the heat should be assumed to be transferred by convection and radiation. A suitable value for the convection heat transfer coefficient was found to be 10 W/m$^2$K in this case and the emissivity of stainless steel was taken as equal to 0.4 on all stainless steel surfaces subject to radiation. The convection heat transfer coefficient on the exposed side was taken as equal to 25 W/m$^2$K.

Although a very good estimate of the maximum temperatures can be obtained for the unexposed side, the temperatures midway between the profiles are overestimated due to the inaccuracy of the mineral wool data and the idealisation of full continuity between surfaces, which leads to a higher temperature from the numerical analysis.

In order to satisfy the insulation criteria given in EN 1363-1 (average temperature not greater than 140°C and a temperature increase with respect to the initial of no more than 180°C) parametric studies were carried out in order to determine the depth of the wall which could achieve 60 minutes fire resistance. The parameter varied was the height of the Z profile and thickness of the insulation from 60 mm to 80, 100 and 120 mm. Figure 3.13 shows the results for the unexposed face of the flange of the Z-section. For each wall thickness, two curves are shown corresponding to different insulation densities giving upper and lower bounds for the temperatures reached at the unexposed face. Assuming that the actual temperature rise is the average of the temperature increases obtained for the two mineral wool densities, it was concluded that only the 120 mm thick wall was sufficient to satisfy the criteria. The 100 mm thick wall might be sufficient provided the mineral wool density approximated to 140 kg/m$^3$.

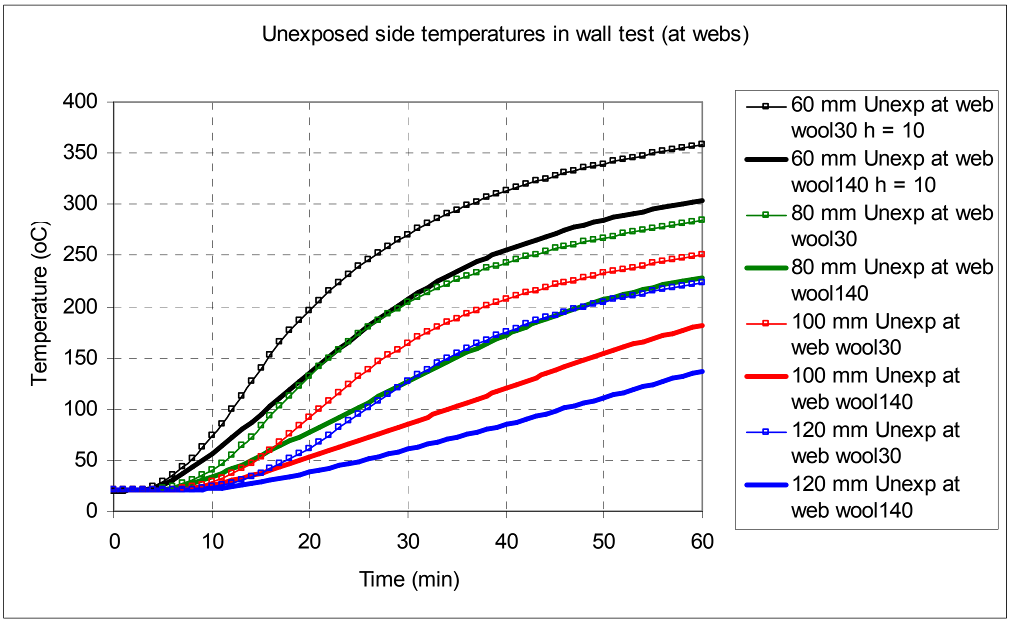
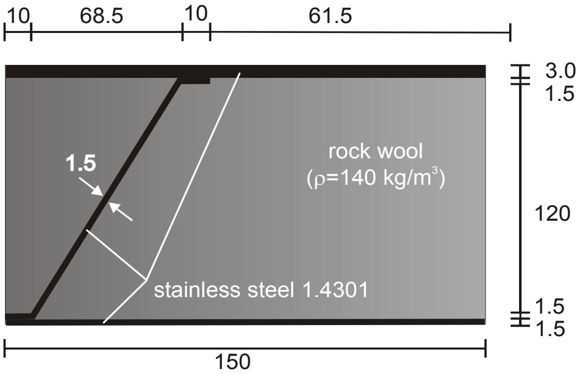



**Figure 3.13** *Calculated temperatures at the unexposed face of the flange of the Z-profile with heights 60 mm (black), 80 mm (green), 100 mm (red) and 120 mm (blue) and different insulation densities.*

### *Floor element*

In order to determine the load displacement curve of the floor element subject to the standard fire curve, a thermal analysis was first carried out. The thermal analysis was performed as a two-dimensional finite element analysis using ABAQUS. Direct heat transfer was assumed between stainless steel and the insulation material. The thermal action was applied according to the standard temperature-time curve. The coefficient of heat transfer due to convection was applied according to EN 1991-1-2[9]. The surface emissivity of the member was taken as 0.2 (fire exposed side). To reduce the size of the model, only half the rib was modelled. The geometry, mesh and boundary conditions are illustrated in Figure 3.14



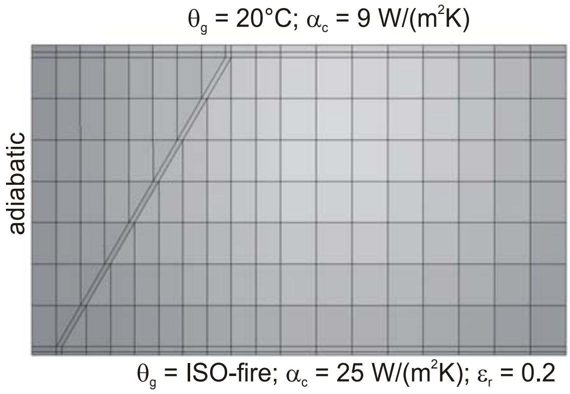
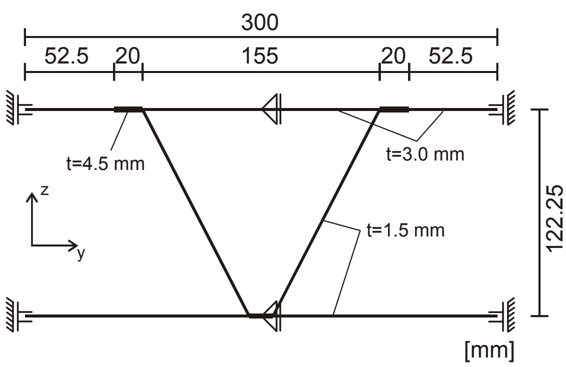




**Figure 3.14** *Thermal model for floor element*

The thermal analyses were carried out using two insulation materials 30 kg/m$^3$ and 140 kg/m$^3$. The temperature development in the member is very sensitive to the material properties of the insulation material. A comparison between the numerical analysis and the small scale test shows that the FE simulations give conservative results. A comparison with the large scale test is difficult to perform and would have to be considered with caution, due to the problems that arose during the test.

The Eurocode insulation criteria for separating members was satisfied for 60 minutes in the small-scale test and in the numerical calculations where a mineral wool density of 140 kg/m$^3$ was assumed.

For the mechanical model of the floor element, only a small part of the element was modelled to reduce the model size and calculation time. The load-carrying in the transverse direction was assumed to be negligible. A small cantilever arm was modelled as well to adapt the correct testing conditions of the large scale test. The edges of the upper and lower sheets were continuously restrained against bending around the x-axis and in the middle of the two sheets the rib was fixed against horizontal displacement in the y-direction. At the support all nodes of the web were fixed in the z-direction. At midspan, all nodes of the cross-section were restrained for bending around the y-axis and fixed against horizontal displacement in the x-direction. This is shown in Figure 3.15 The corner radius of the steel plates was neglected. Thus the upper and lower flanges are continuous plates, where the thickness in the overlapping welding zones was taken as the sum of the thickness of the clinging plates.



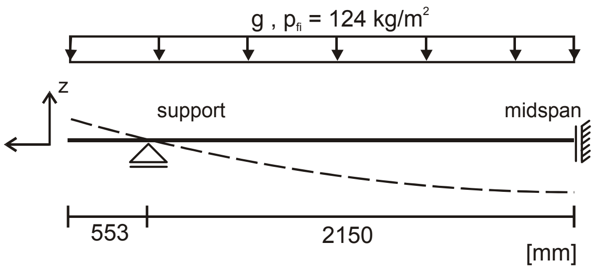




**Figure 3.15** *Mechanical model for floor element. Continuous boundary conditions*
*Left: along the edges, Right: at the ends of the rib*

A load-bearing calculation at ambient temperature was carried out to analyse the load-bearing behaviour under increasing live loads and to verify the load ratio. The output of this analysis showed that the load-bearing capacity determined by simplified calculation methods agreed well with the numerical simulation.

A static analysis was performed taking into account the temperature variation and the geometrical nonlinearities. The imperfections were simulated by superposing the FE-model onto the scaled buckling mode shape. The slab panel with the imperfect geometry was subjected to the temperatures determined in the small scale fire test and Figure 3.16 shows the predicted displacements against the measured displacements in the large-scale fire test. The deflection values obtained from ABAQUS were much higher than in the fire test. Although very large deflections were observed, these remained approximately constant during the last 15 minutes, and no failure occurred.

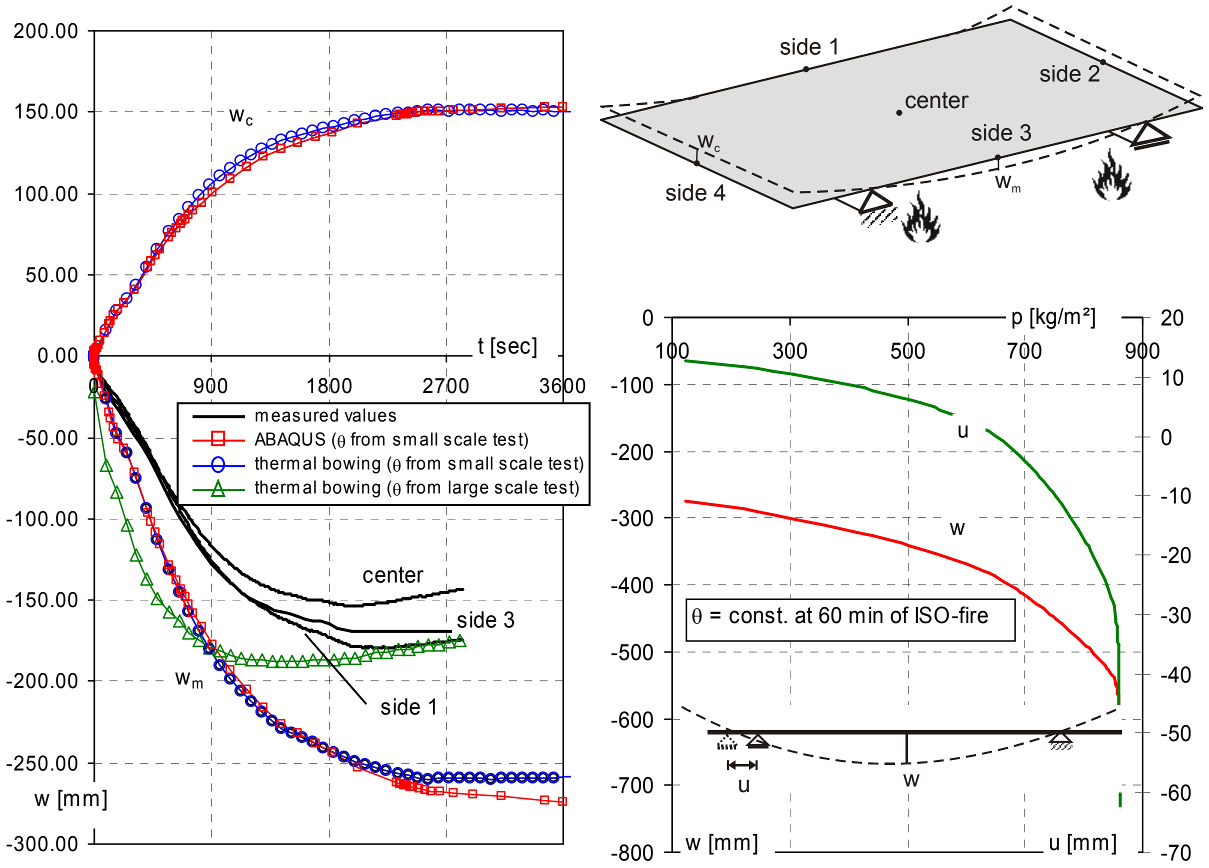



**Figure 3.16** *Left: Vertical displacement due to heating of the member against time*
*Right: Vertical displacement due to load increasing against variable loads*

The total deflection after 60 minutes fire exposure was governed by thermal bowing and not by the effects of the mechanical loads and the loss in bending stiffness due to high steel temperatures. A load-bearing calculation was also performed, following 60 minutes fire exposure with constant temperatures and increasing live loads; the results are shown on the right side of Figure 3.16. An ultimate live load of $p$=750-800 kg/m$^2$ could be estimated from Figure 3.16 due to rapidly increasing deflections which correlates with a load a load ratio of 0.35 in the fire situation.

## 3.4 Conclusions

Eight fire tests were carried out, four on load-bearing concepts and four on separating structures, and the temperature development was measured in each test. Numerical models of the test were developed and parametric studies were carried out to develop an understanding of the parameters which affect the temperature rise in these concepts.

The load-bearing systems successfully suppressed the temperature rise, however, the construction practicalities of these systems needs further consideration. There is currently no design guidance in EN 1993-1-2 for calculating the buckling capacity of a column with non-uniform temperature distribution over the cross-section. Designers would have to use finite element analysis in order to take advantage of part of the cross-section being at a significantly lower temperature. Until simple design guidance for this has been developed, it is unlikely that concepts such as the corner column will be widely adopted in practice.

A parametric study quantified the superior behaviour of the stainless steel columns in fire compared to that of carbon steel columns in the temperature range 600°C to 800°C.

Regarding the separating elements, 60 minutes fire resistance can be obtained for wall elements of 120 mm thickness. It was also shown by tests and numerical modelling that the sandwich panel floor construction with a 120 mm depth could attain the 60 minutes fire resistance period provided the mineral wool is effectively placed in the voids.

# 4 WP2: COMPOSITE MEMBERS IN FIRE

Detailed descriptions of the activities carried out under this work package are given in the relevant Final Work Package Report listed in Section 11.

## 4.1 Objectives

The objective of this work package was to develop design guidance for composite members in fire by a programme of fire tests on concrete filled columns and floor beams with concrete fire protection.

## 4.2 Experimental work

### 4.2.1 Composite columns

Seven columns were tested, each consisting of square hollow sections (SHS) filled with concrete which was reinforced in some tests and unreinforced in others. The details of the columns are given in Table 4.1. The columns were grade 1.4401 stainless steel.

**Table 4.1** *Structural details of composite columns with hollow steel sections*

| Column | Cross-section | Stainless Steel grade | Rebar | | Loading | | Fire resist. | Length |
|---|---|---|---|---|---|---|---|---|
| | b×e (mm) | | diameter | Cover[1)] (mm) | Load (KN) | eccentricity | (min) | (mm) |
| **n°1** | 150×8 | EN 1.4401 | none | - | 400 | 5 mm | 30 | 4000 |
| **n°2** | 200×8 | EN 1.4401 | none | - | 240 | 0.25×b [2)] | 60 | 4000 |
| **n°3** | 200×8 | EN 1.4401 | 4Φ14 | 30 | 630 | 5 mm | 30 | 4000 |
| **n°4** | 200×8 | EN 1.4401 | 4Φ14 | 30 | 240 | 0.25×b [2)] | 60 | 4000 |
| **n°5** | 300×8 | EN 1.4401 | none | - | 750 | 0.5×b [2)] | 30 | 4000 |
| **n°6** | 300×8 | EN1.4401 | 4Φ22 | 30 | 1000 | 0.125×b [2)] | 60 | 4000 |
| **n°7** | 300×8 | EN 1.4401 | 4Φ22 | 30 | 800 | 0.25×b [2)] | 60 | 4000 |

[1)] Distance between the axis of longitudinal reinforcements and the border of concrete core
[2)] External side of hollow steel section

Columns were subjected to a compressive load and exposed to controlled heating following the EN 1363-1 standard fire curve. The specimens were pinned at both ends; they were free to rotate about one direction but were restrained against rotation about the perpendicular direction. Figure 4.1 shows the arrangement of the columns in the furnace.

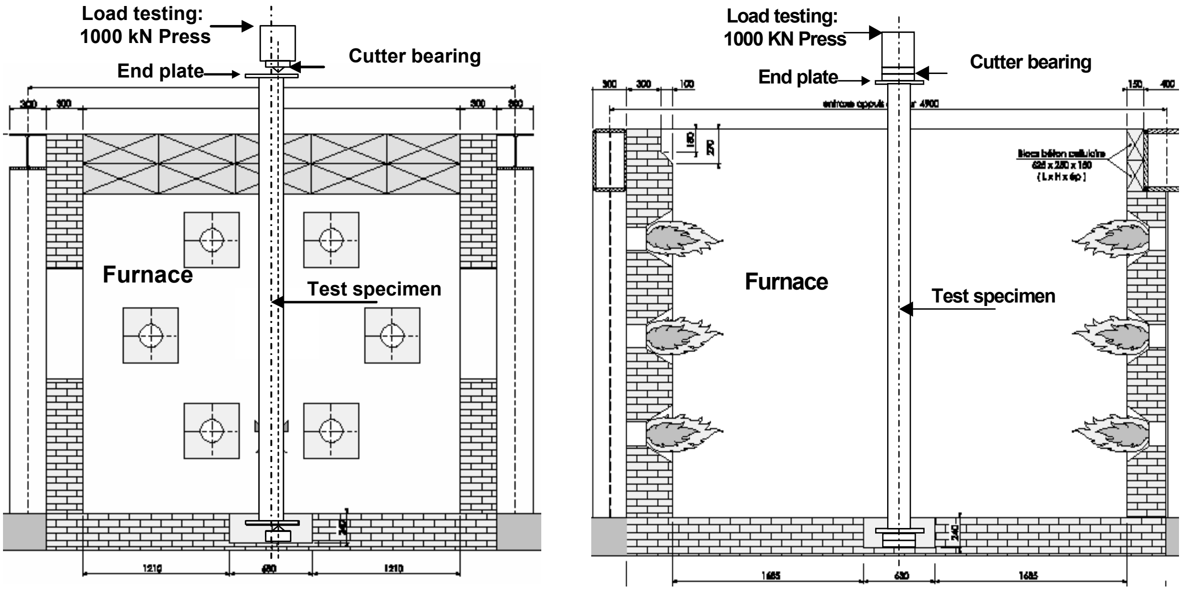



**Figure 4.1** *Test arrangement for column fire tests*

The furnace temperature was measured with 12 thermometers at 100 mm from the specimen. Thermocouples were also installed on the hollow section. Axial deformations of the columns were determined by measuring the displacement of the top of the columns using transducers. Failure time was determined by measuring the time when the specimen could not bear the applied load any more.

Table 4.2 gives failure times, temperatures and failure modes of all the specimens and Figure 4.2 shows two columns after testing. The failure times were, in general, above the expected fire ratings. (The initial design of the columns had been carried out using nominal values of mechanical properties.) The maximum deflection was found at the bottom or mid height of the columns. Local buckling occurred in the larger cross sections (200×8 and 300×8). Specimens from the tested members were taken to obtain the yield and ultimate strengths of the stainless and carbon steels and the compressive strength of the concrete. These were higher than the assumed values, whereas the yield strength of the reinforcement bars was lower than assumed.

**Table 4.2** *Measured failure time of composite columns*

| Column | Load ratio[1)] | Failure time (min) | Temperature in the hollow section at failure (°C) | Failure mode |
|---|---|---|---|---|
| **N°1** | 0.42 | 42 | 775 | Flexural buckling |
| **N°2** | 0.22 | 59.5 | 850 | Flexural buckling |
| **N°3** | 0.31 | 56 | 835 | Flexural buckling + local buckling |
| **N°4** | 0.20 | 71 | 910 | Flexural buckling + local buckling |
| **N°5** | 0.46 | 38 | 700 | Flexural buckling + local buckling |
| **N°6** | 0.29 | 70.5 | 890 | Flexural buckling + local buckling |
| **N°7** | 0.29 | 62 | 850 | Flexural buckling + local buckling |

1) The ratio of the test load to the buckling resistance of the column at room temperature, calculated using the numerical model, taking into account the load eccentricity

**Figure 4.2** *View of composite column after test: Test 2 (left) and Test 5 (right)*

### 4.2.2 Composite beams

Two simply supported hybrid I section beams were tested (stainless steel lower flange, carbon steel web and top flange). The details are shown in Figure 4.3. The stainless steel was grade 1.4401 and the length of the beams was 4.9 m. The predicted fire rating of the beams was estimated at 60 minutes. The load was applied at least 15 minutes before starting the heating process and was maintained until failure. The applied heating followed the standard fire according to EN 1363-1.

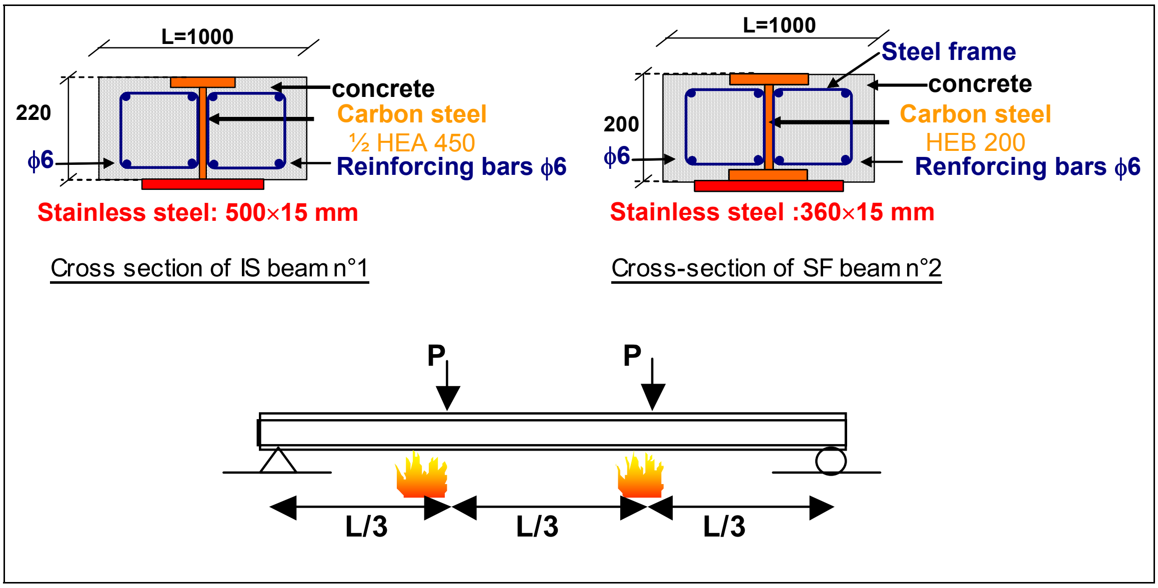



**Figure 4.3** *Structural details of beam test specimens*

The temperature history of the beams was recorded by thermocouples located at several points over the cross section. Furnace temperatures were also recorded using thermometers. The central deflection was measured with two linear displacement transducers. Material properties of the stainless steel used in the test specimens were obtained from three tensile material tests on offcuts from the steel plate used to fabricate the beams. The actual yield strength of the stainless steel plate was higher than assumed in the preliminary design of the beams.

Table 4.3 shows failure times, fire durations and load ratios of the beam tests. Failure was taken as the moment when the deflection exceeded the limit of L/20. The measured failure times were higher than expected. Figure 4.4 shows the integrated composite beam after the fire test.

**Table 4.3** *Measured failure times of composite beams*

| Beams | Load ratio[1)] | Fire duration (min) | Failure time (min) |
|---|---|---|---|
| **N°1** | 0.43 | 90 | 79 |
| **N°2** | 0.65 | 86 | 76 |

1) The ratio of the maximum moment applied during the test to the moment resistance at room temperature calculated using the numerical model

**Figure 4.4** *Integrated composite beam (no.1) after the fire test*

# 4.3 Numerical studies

## 4.3.1 Calibration of numerical model

The mechanical analysis was carried out using the program SISMEF. Temperature distributions were introduced either from a 2D heat transfer analysis or from test data. The following assumptions were made:

- The columns had pinned ends and a constant compressive load was applied during the test.
- The beams were simply supported and the contribution of the concrete slab on the mechanical fire resistance of the beams was neglected.
- Thermal and mechanical material properties were taken from EN 1992-1-2 for the concrete and reinforcement bars and from EN 1993-1-2 for the stainless steel.
- Residual stresses were neglected
- Uniform temperature distribution along the column height and the beam length, except for the top of the column which was outside the furnace.

- Full interaction (no slip) and no interaction (slip allowed) between the steel section and the concrete core were considered.

Assuming the thermal parameters recommended in EN 1993-1-2 for stainless steel ($\varepsilon_m$ = 0.4 and $h_c$ = 25 W/m$^2$K) the temperature rise predicted for the columns were in good agreement with the measured values. Overall, the calculated temperatures were conservative.

For the composite beams, again using an emissivity of 0.4 for the stainless steel, the numerical results remain on the safe side with higher predicted temperatures than measured during the test. The use of a value of 0.2 for the emissivity led to an appropriate prediction during the first 60 minutes of fire exposure but became unsafe after that period of time. This could be due to a change of the surface properties of the stainless steel during the fire exposure.

There was reasonable agreement between the measured and calculated displacements for the composite columns. At the beginning of the fire, the steel carried most of the load. As the steel was directly exposed to the fire and more sensitive to high temperatures, it collapsed suddenly due to local buckling and the entire load was then carried by the concrete core. The concrete core finally failed by buckling. If the core was not reinforced, the vertical displacement increased approximately linearly and reached a maximum just before failure occurred, when it reduces very rapidly as the column buckled.

The effects of differential thermal elongation (stresses) and interaction between the steel and concrete were studied. Reasonable agreement between the measured and calculated displacements was achieved when slip was taken into account, i.e. no mechanical interaction between the steel SHS and concrete core.

There was also a good correlation between the predicted and the measured displacements for the composite beams. The results of the numerical simulation were more accurate during the first stage of the test whilst some differences were observed at the end of the test.

### 4.3.2 Parametric studies for composite columns

Parametric studies of the behaviour of composite columns in fire were carried out in order to develop design guidelines. The temperature distribution was first calculated with a 2D heat transfer analysis and then a mechanical analysis was carried out to evaluate the ultimate buckling load using the temperature distribution as an input. The parameters studied are given in Table 4.4.

**Table 4.4** *Parameters studied for composite column numerical analysis*

| | |
|---|---|
| **Cross-sections** | 5 SHS from 150 to 500 mm, 4 and 8 mm thick each |
| **Steel grades** | 1.4301; 1.4401 and 1.4571 $f_{0,2p}$ = 240MPa $f_u$ = 2,04$f_{0,2p}$ |
| **Fire duration** | R30 and R60 |
| **Concrete** | Class C30 |
| **Reinforcing steel** | 0%; 1%; 2%; 3% and 5% in S500 steel. Concrete cover of 30 mm |
| **Column length** | reduced slenderness ratios at room temperature of $\lambda$ = 0.2; 0.3; 0.4; 0.5; 0.8; 1.0; 1.2; 1.5 and 2.0 |
| **Eccentricity** | 0; 0.125b; 0.25b and 0.5b |

The parametric studies showed that slip has no significant influence on the failure time of composite columns provided the column is filled with reinforced concrete, although taking it into account enables closer predictions of the displacements in the earlier stages of heating.

### 4.3.3 Design method for composite columns

This method is based on the method given in EN 1994-1-2, clause 4.3.5.1(1)[10].

For a given temperature distribution within a cross-sections, the load-bearing capacity of a composite column $N_{\mathrm{fi,Rd}}$ can be determined from an appropriate column buckling curve which relates the load-bearing capacity with the elastic critical load, $N_{\mathrm{fi,pl,Rd}}$, as follows:

$$N_{\mathrm{fi,Rd}} = \chi\left(\bar{\lambda}_\theta\right) N_{\mathrm{fi,pl,Rd}} \tag{4.1}$$

where:

χ is the reduction factor depending on the relative slenderness $\bar{\lambda}_\theta$ given by

$$\chi = \frac{1}{\varphi + \sqrt{\varphi^2 - \bar{\lambda}_\theta^2}} \quad \text{with} \quad \varphi = \frac{1}{2}\left(1 + \alpha\left(\bar{\lambda}_\theta - \bar{\lambda}_0\right) + \bar{\lambda}_\theta^2\right)$$

and α=0.76, $\bar{\lambda}_\theta$=0.2 (buckling curve d)

$$N_{\mathrm{fi,pl,Rd}} = \sum_j \frac{\left(A_{\mathrm{a,j}} f_{\mathrm{ay,\theta j}}\right)}{\gamma_{\mathrm{M,fi,a}}} + \varphi_{\mathrm{c,\theta}}\left(\sum_k \frac{A_{\mathrm{s,k}} f_{\mathrm{sy,\theta k}}}{\gamma_{\mathrm{M,fi,s}}} + \sum_m \frac{A_{\mathrm{c,m}} f_{\mathrm{c,\theta m}}}{\gamma_{\mathrm{M,fi,c}}}\right) \tag{4.2}$$

Where the subscripts a, s and c indicate the stainless steel hollow section, the steel reinforcing bars and the concrete, respectively.

$A_{\mathrm{a,i}}$, $A_{\mathrm{s,i}}$ and $A_{\mathrm{c,i}}$ are the areas of elements i of the cross section

$f_{\mathrm{ay,\theta}}$, $f_{\mathrm{sy,\theta}}$ and $f_{\mathrm{cy,\theta}}$ are the characteristic strengths at elevated temperature of the steel hollow section, reinforcing bars and concrete respectively.

$\varphi_{\mathrm{c,\theta}}$ is a reduction coefficient taking into account the differential effects of thermal stresses.

The relative slenderness of the column in the fire situation is give by

$$\bar{\lambda}_\theta = \sqrt{\frac{N_{\mathrm{fi,pl,R}}}{N_{\mathrm{fi,cr}}}} \tag{4.3}$$

Where

$N_{\mathrm{fi,pl,R}} = N_{\mathrm{fi,pl,Rd}}$ with $\gamma_{\mathrm{M,fi,i}} = 1.0$

$$N_{\mathrm{fi,cr}} = \frac{\pi^2 (EI)_{\mathrm{fi,eff}}}{l_\theta^{\,2}}$$

$l_\theta$ is the buckling length of the column in fire situation

The effective flexural stiffness of the section is calculated as follows:

$$(EI)_{\mathrm{fi,eff}} = \varphi_{\mathrm{a,\theta}} \sum \left(E_{\mathrm{a,\theta j}} I_{\mathrm{a,j}}\right) + \varphi_{\mathrm{c,\theta}}\left[\sum \left(E_{\mathrm{c,\theta m}} I_{\mathrm{c,m}}\right) + \sum \left(E_{\mathrm{s,\theta k}} I_{\mathrm{s,k}}\right)\right]$$

$E_{\mathrm{i,\theta}}$ is the characteristic modulus of material *i* at temperature θ.

For steel $E_{\mathrm{a,\theta}} = E$

For concrete: $$E_{\mathrm{c,\theta}} = \frac{3}{2} E_{\mathrm{c,sec,\theta}} = \frac{3}{2} \frac{f_{\mathrm{c,\theta}}}{\varepsilon_{\mathrm{cu,\theta}}} \tag{4.4}$$

$I_{\mathrm{i}}$ is the second moment of area of material i about the principal axes (y-y or z-z) of the composite cross section.

$\varphi_{a,\theta}$ and $\varphi_{c,\theta}$ are reduction coefficients due to the differential effects of thermal stresses.

$\varphi_{a,\theta}$ depends on the fire rating only and $\varphi_{c,\theta}$ is defined by six parameters depending on the cross-section size, column buckling length, reinforcing ratio and fire rating. $\varphi_{c,\theta}$ ranges from 0 to 1; when it is 0 the fire resistance of the column is provided by the hollow section only, and when it is 1, the column acts as a composite element with significant interaction between steel and concrete.

Appendix A gives the reduction coefficients $\varphi_{a,\theta}$ and $\varphi_{c,\theta}$ and the Final Work Package Report gives a complete set of expressions for the buckling resistance under eccentric loading.

Comparisons between the proposed design method and the numerical model show that in the majority of cases, the proposed design method is conservative. A comparison between the proposed design method and fire test results show that the difference does not exceed 15%. Moreover, the unsafe results correspond to columns which failed after 60 minutes, the maximum fire rating for which the proposed design method is valid. For the tests in the range of application of the proposed method, all the predictions were conservative.

### 4.3.4 Parametric studies for composite beams

2D thermal analyses were carried out to establish simplified temperature distributions for exposure to the standard fire curve for 120 minutes for eight integrated floor beams (IF beam no. 1 in Figure 4.3) and seven slim floor beams (SF beam no. 2 in Figure 4.3). Figure 4.5 shows the results for the IF beams. In all cases there is a large temperature gradient in the cross section due to the encasement of concrete. After 30 minutes, the carbon steel remains below 400°C (full strength). After 60 minutes, up to 25% of the depth of the web is above 400°C. After 120 minutes, about 50% of the carbon steel section is above 400°C. The unexposed side remains at a temperature lower than 100°C after 120 minutes of fire exposure. This means that the insulation criterion is always satisfied with this type of structural member.

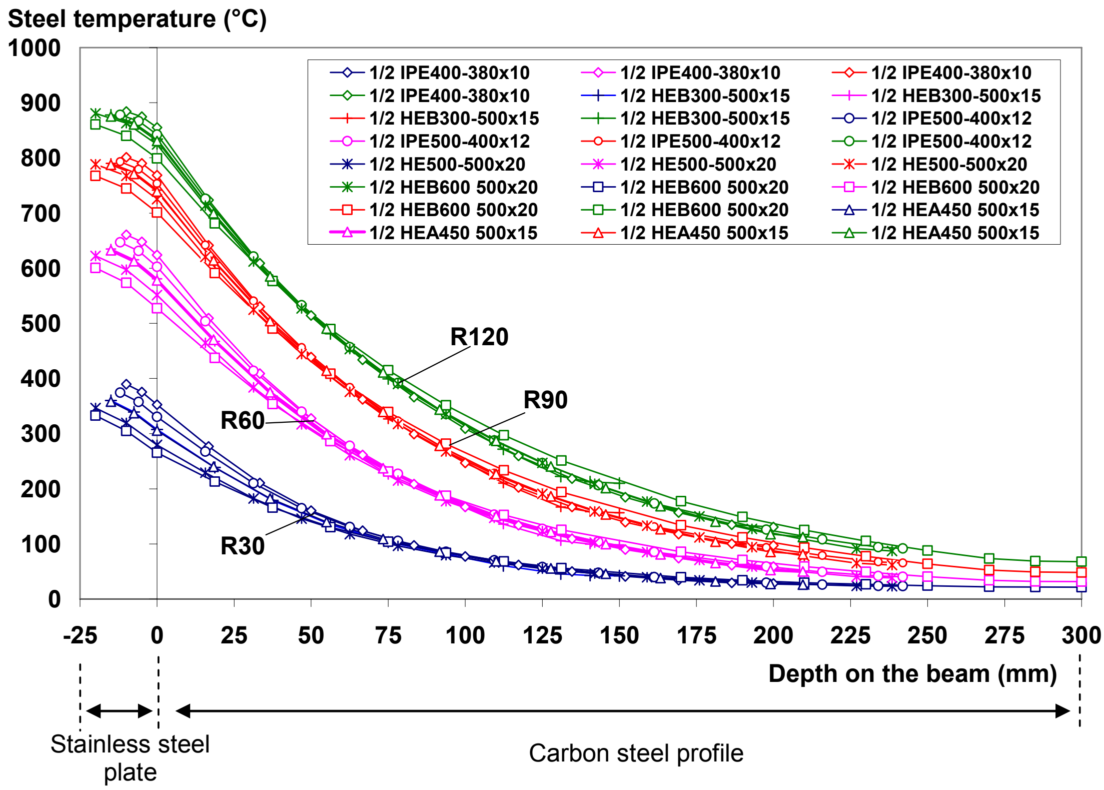



**Figure 4.5** *Temperature distribution along the depth of IF beams from 30 to 120 minutes of standard fire exposure*

### 4.3.5 Design method for composite beams

This method is based on the simple plastic moment theory. It requires the calculation of the neutral axis and corresponding moment resistance taking into account temperature distribution through the cross-section and corresponding reduced material strength.

The following simplifying assumptions have been made:

- The concrete does not contribute to the load-bearing capacity at elevated temperatures and thus may be ignored.
- Failure of the beam occurs when maximum mechanical strain exceeds 2% on the exposed stainless steel plate

The design moment resistance $M_{\mathrm{fi,t,Rd}}$ may be determined from:

$$M_{\mathrm{fi,t,Rd}} = \sum_{i=1}^{n} A_{\mathrm{i}} z_{\mathrm{i}} k_{\mathrm{y,\theta,i}} \left( \frac{f_{\mathrm{y,i}}}{\gamma_{\mathrm{M,fi}}} \right)$$

Where:

$z_{\mathrm{i}}$ is the distance from the plastic neutral axis to the centroid of the elemental area $A_{\mathrm{i}}$.

For the calculation of the design value of the moment resistance, the cross-section of the beam is divided into various components, namely:

- The stainless steel plate
- The lower flange of the carbon steel profile (when used)
- The web of the steel profile
- The upper flange of the carbon steel profile

For each component, Table 4.5 and Table 4.6 give simple rules which define temperatures and corresponding reduced characteristic strength as a function of the fire rating R30, R60, R90 and R120. Figure 4.6 shows a diagram of temperature and stress distributions over the depth of the beam.

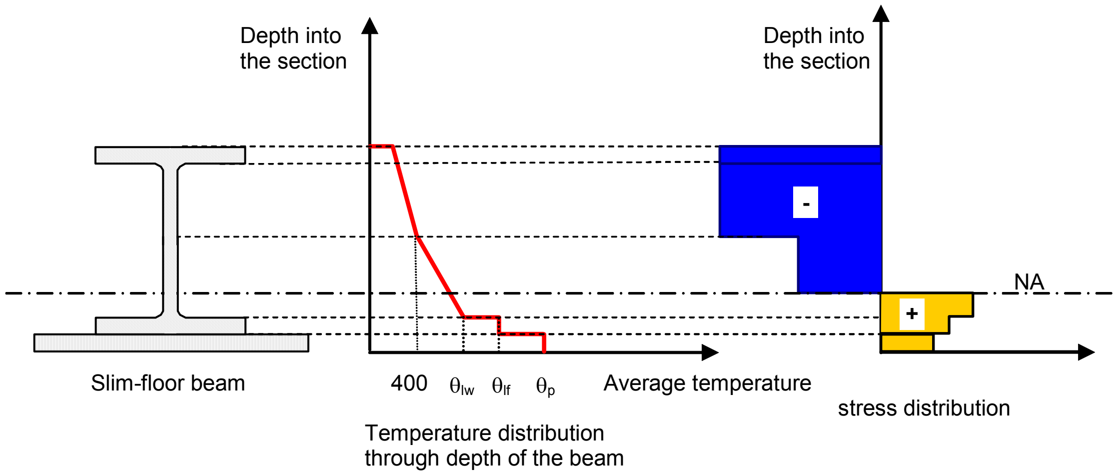



**Figure 4.6** *Temperature and stress distributions over the depth of beam*

The height $h_{\mathrm{l}}$ of the lower part of the web is given by:

$$h_l = -\sqrt{\frac{2\alpha t}{\beta}} \ln\left( \frac{380}{\theta_{lw} - 20} \right)$$

where:

$t$ is the time (s)

$\beta$ = 12.25

$\alpha$ = $\lambda_a/\rho_a C_a$ ($\lambda_a$=45 W/mK, $C_a$ = 600 J/KgK, $\rho_a$ = 7850 Kg/m$^3$ )

$\theta_{lw}$ = is the mean temperatures at the bottom edge of the web = $\kappa_3\theta_p$ in which $\theta_p$ is the temperature of the stainless steel plate and $\kappa_3$ is a reduction factor given in Table 4.6.

**Table 4.5** *Values of parameter $\theta_o$, a and b*

| Part i of the beam | Area $A_i$ | Temperature $\theta_i$ [1)] | Characteristic strength $f_{y,\theta i}$ |
|---|---|---|---|
| Stainless steel plate | Full area<br>$A_p=e_p\times b_p$ | Uniform temperature distribution<br>For IF-beam: $\theta_p = \theta_o - \kappa_1\times e_p$<br>For SF-beam: $\theta_p = \theta_o - \kappa_1\times(e_p+e_f)$ | $k_{2,\theta p}\times f_{sy,20}$ |
| Lower flange | Full area<br>$A_{lf}=e_f\times b_f$ | Uniform temperature distribution<br>$\theta_{lf} = \kappa_2\times\theta_p$ | $k_{y,\theta lf}\times f_{ay,20°C}$ |
| Lower part of the web | $A_{wl}= e_w\times h_l$ | Changes linearly from $\theta_{lw}$ to 20°C | $f_{ay,20°C}\times(1+ k_{y,\theta lw})/2$ |
| Upper part of the web | $A_{ul}= e_w\times(h_w-h_l)$ | Lower than 400°C | $f_{ya,20°C}$ |
| Upper flange | Full area<br>$A_{uf}=e_f\times b_f$ | Lower than 400°C | $f_{ya,20°C}$ |

1) $\theta_o$ , $\kappa_1$ and $\kappa_2$ are empirical coefficients depending on the fire rating only

**Table 4.6** *Values of parameter $\theta_o$, $\kappa_1$, $\kappa_2$ and $\kappa_3$*

| Fire rating | $\theta_o$ | | $\kappa_1$ | | $\kappa_2$ | $\kappa_3$ | |
|---|---|---|---|---|---|---|---|
| | IF beam | SF-beam | IF beam | SF beam | | IF beam | SF beam |
| 30 | 570 | 500 | 7 | 3 | 0.75 | - | - |
| 60 | 830 | 775 | 6 | 3 | 0.85 | 0.77 | 0.76 |
| 90 | 920 | 930 | 3 | 3 | 0.90 | 0.83 | 0.81 |
| 120 | 980 | 1025 | 2 | *3* | 0.95 | 0.87 | 0.84 |

The results obtained with the proposed design method were compared to the results predicted by numerical analysis. For the numerical predictions, the beam was firstly subjected to the EN 1363-1 standard fire curve for 60, 90 and 120 minutes under the effects of neighbouring vertical loads. Then the temperature distribution was kept constant and a vertical load P was applied, which increased gradually until the beams failed. The failure point of the beam was taken when the maximum mechanical strain in the stainless steel plate exceeded 2%, corresponding to a maximum deflection of L/15 to L/10. In general good agreement was achieved with the proposed design method and the numerical model differing by no more than 10%. For a load ratio smaller than 0.7, a fire rating of R60 (i.e. 60 minutes) is easily achievable. An integrated beam can achieve R90 and a slim floor beam can achieve R120 when the load ratio is lower than 0.5 without any applied fire protection.

### 4.3.6 Comparison between stainless steel and carbon steel

To compare the performance of stainless and carbon steel composite columns, a numerical study was carried out on three different RHS column cross-sections, each of length 3 m filled with unreinforced concrete. The results are given in Table 4.7. It is clear that carbon steel columns buckle at a lower load than stainless steel columns of identical size and length. For a given fire rating, the maximum load level of stainless steel columns increases with increasing cross-section size. This is mainly due to the lower temperature rise of the large cross-section in comparison to the smaller cross-section.

**Table 4.7** *Comparison of maximum load level for concrete filled RHS columns*

| Column | Fire rating (minutes) | Maximum load level[1)] | |
|---|---|---|---|
| | | Stainless steel column (grade 1.4401) | Carbon steel column (grade S235) |
| 150x150x8 | 30 | 0.36 | 0.15 |
| | 60 | 0.16 | 0.04 |
| 200x200x8 | 30 | 0.36 | 0.15 |
| | 60 | 0.16 | 0.06 |
| 300x300x8 | 30 | 0.65 | 0.47 |
| | 60 | 0.29 | 0.15 |

[1)] The load level is the ratio of the buckling resistance at the fire ultimate state to the buckling resistance at room temperature

To compare the performance of stainless and carbon steel composite beams, a numerical study was carried out on three different beam cross-sections. Figure 4.7 shows the comparison in the development of temperature for beams with exposed carbon steel and stainless steel lower plates. The results are given in Table 4.8. For the same fire rating, the bending moment resistance of carbon steel beams is always lower than the beam with the exposed lower flange from stainless steel. 120 minutes fire resistance can easily be achieved by the ½ HEA 450 beam with the exposed stainless steel plate providing the load level is lower than 0.33. In contrast to this, the carbon steel beam only achieved a fire resistance of 60 minutes with a load level of 0.27.

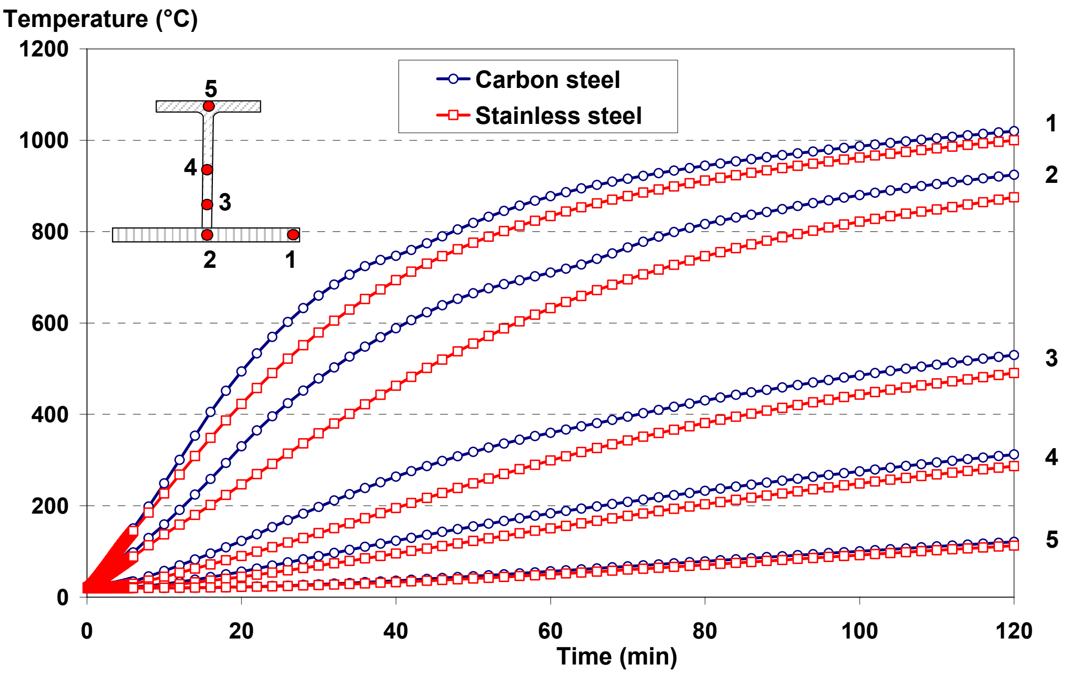
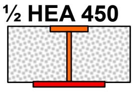
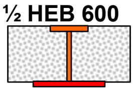
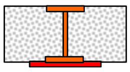



**Figure 4.7** *Comparison between temperature rise of IF beam with exposed stainless steel and carbon steel plates.*

**Table 4.8** *Comparison of maximum load level for beams with exposed carbon steel and stainless steel plates*

| Beam | Fire rating | Maximum load level[1)] | |
|---|---|---|---|
| | | Stainless steel lower plate (grade 1.4401) | Carbon steel lower plate (grade S235) |
| **½ HEA 450**  **Steel plate: 500×15 mm** | R60 | 0.72 | 0.27 |
| | R90 | 0.46 | 0.17 |
| | R120 | 0.33 | 0.15 |
| **½ HEB 600**  **Steel plate: 500×20 mm** | R60 | 0.73 | 0.31 |
| | R90 | 0.48 | 0.22 |
| | R120 | 0.36 | 0.13 |
| **HEB 280**  **Steel plate: 480×20 mm** | R60 | 0.92 | 0.55 |
| | R90 | 0.77 | 0.28 |
| | R120 | 0.58 | 0.22 |

1) The load level is the ratio of the moment resistance at the fire ultimate state to the moment resistance at room temperature

## 4.4 Conclusions

Fire tests were carried out on RHS columns filled with concrete (reinforced and unreinforced) designed to achieve a fire rating of 30 and 60 minutes made from grade 1.4401 stainless steel. The tests were modelled numerically and subsequently parametric studies were carried out in order to develop design rules for composite columns. The proposed design methods are consistent with the general flow charts in EN 1994-1-2[10] used to check the fire resistance of composite members but include some specific characteristics to account for the distinctive behaviour of stainless steel.

To compare the performance of stainless and carbon steel composite columns, a numerical study was carried out on different RHS column cross-sections filled with unreinforced concrete. It is clear that carbon steel columns buckle at a lower load than stainless steel columns of identical size and length.

Two fire tests were carried out on hybrid stainless-carbon steel composite beams with the stainless steel lower flange exposed and the carbon steel section unexposed. The specimens were 5 m in length and designed to achieve a fire rating of 30 and 60 minutes. The tests were modelled numerically and subsequently parametric studies were carried out in order to develop design rules for composite beams. The proposed design method is based on simple plastic moment theory, requiring the calculation of the neutral axis and corresponding moment resistance by taking into account the temperature distribution through the cross-section and the corresponding reduction in material strength.

To compare the performance of stainless and carbon steel composite beams, a numerical study was carried out on different beam cross-sections. For the same fire rating, the bending moment resistance of carbon steel beams is always lower than the beam with the exposed lower flange from stainless steel.

# 5 WP3: CLASS 4 CROSS SECTIONS IN FIRE

Detailed descriptions of the activities carried out under this work package are given in the relevant Final Work Package Reports listed in Section 11.

## 5.1 Objectives

The objective of this work package was to develop simple design rules for Class 4 stainless steel box columns in fire.

## 5.2 Experimental work

### *Tests at room temperature*

Four cold rolled stainless steel stub columns, length 900 mm and $\bar{\lambda} < 0.1$, with Class 4 cross-sections were tested at room temperature. The material used in the columns was grade 1.4301. The material properties were determined from tensile coupon tests on material taken from the flat faces of the columns carried out in accordance with EN 10002-1[11]. Table 5.1 gives the results of the tensile tests.

Measurements were made of the imperfections in the longitudinal direction and flatness. Four strain-gauges were used to measure stresses at mid-column.

The column tests were performed twice for each cross section and the test arrangement is shown in Figure 5.1. The load equipment was the same as that used for the tests at elevated temperatures. The loss of load-bearing capacity occurred very suddenly as a result of local buckling failure in the middle of the column. Test results are summarised in Table 5.2.

**Table 5.1** *Summary of tensile tests at room temperature*

| Cross-section | Specimen | Yield strength $R_{p0.2}$ (MPa) | | Tensile strength $R_m$ (MPa) | | Elongation (%) | |
|---|---|---|---|---|---|---|---|
| | | m | s | m | s | m | s |
| **RHS 150 × 150 × 3** | Flange face | 397 | 16.26 | 666 | 5.66 | 49.8 | 0.35 |
| **RHS 150 × 150 × 3** | Web face | 329 | 14.85 | 641 | 2.83 | 53.5 | 0 |
| **RHS 200 × 200 × 5** | Flange face | 341 | 52 | 629 | 13.43 | 56.3 | 3.18 |
| **RHS 200 × 200 × 5** | Web face | 286 | 2.12 | 616 | 2.12 | 58.0 | 0.71 |
| m=mean value, s = standard deviation | | | | | | | |

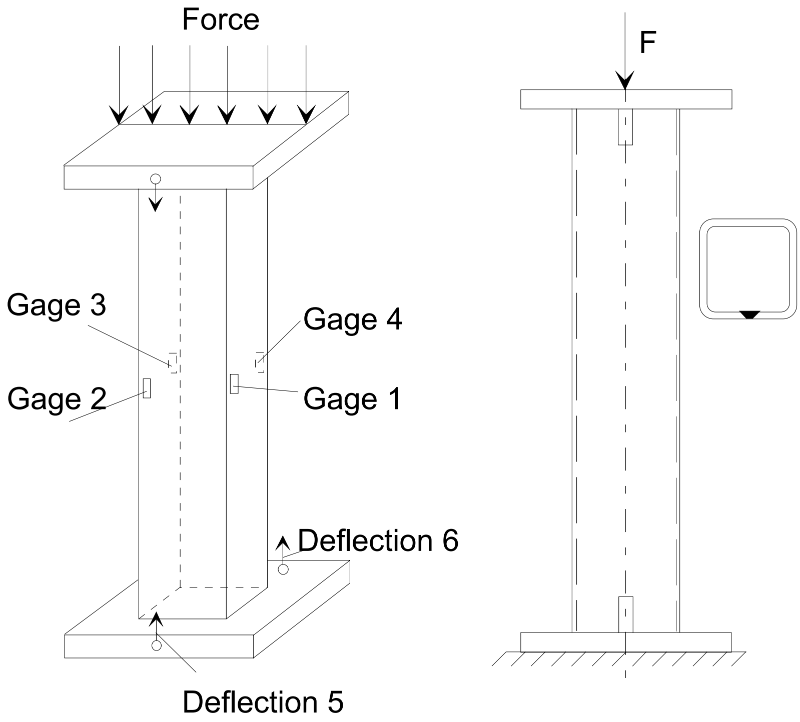



**Figure 5.1** *Test arrangement*

**Table 5.2** *Test results at room temperature*

| Profile | Failure load (kN) |
|---|---|
| RHS 200 × 200 × 5 | 1129 |
| RHS 200 × 200 × 5 | 1118 |
| RHS 150 × 150 × 3 | 398 |
| RHS 150 × 150 × 3 | 393 |

### *Tests at elevated temperatures*

Six unprotected columns were loaded concentrically at elevated temperatures. The columns were RHS with the same cross-sectional dimensions (200x200x5 and 150x150x3) and length as the room temperature tests. The test set-up was also equivalent. The columns were fixed at both ends.

The steel columns were heated in a model furnace consisting of a furnace chamber located within a steel framework. The chamber was 1500 mm wide, 1300 mm high and 1500 mm deep. It was lined with fire resistant bricks and it had four oil burners inside, two in each of the two walls. The transient state test procedure was applied, meaning that the axial load was kept constant and the furnace temperature was raised in a controlled way, at the rate of 10°C/min. The columns were tested at three different load levels.

The load was applied with a hydraulic jack of 2MN capacity located above the furnace chamber (Figure 5.2). Axial deformation of the specimen was determined by measuring the displacement of the top of the water–cooled steel unit, using transducers. The load was controlled and measured using pressure transducers.

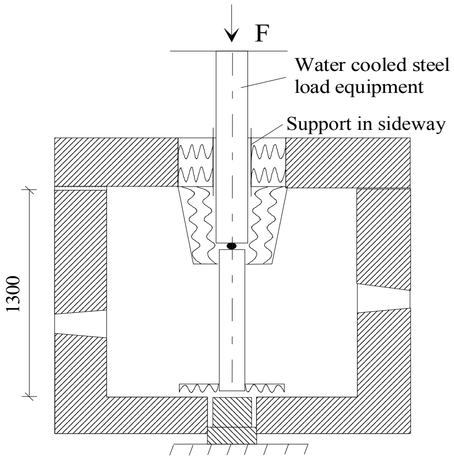



**Figure 5.2** *Test arrangement and photograph of furnace tests*

The temperature of each column was measured at 3 cross-sections using 12 thermocouples. The furnace gas temperature 100 mm away from the columns was also measured at 3 cross-sections with 12 thermocouples. The average of these thermocouples was used to control the temperature of the furnace. Both temperatures and deformations were recorded every 10 seconds.

As the specimens were short, global flexural buckling did not occur; they lost their load-bearing capacity the moment a local buckle appeared. The end temperature was the maximum temperature at the level (upper, middle or lower) where a buckle appeared. Table 5.3 gives the test results.

**Table 5.3** *Results from tests at elevated temperatures*

| Cross-section | Failure load (kN) | Load level | Failure temperature °C |
|---|---|---|---|
| 150x150x3 | 203 | 0.51 | 676 |
| 150x150x3 | 165 | 0.42 | 720 |
| 150x150x3 | 248 | 0.63 | 588 |
| 200x200x5 | 694 | 0.62 | 609 |
| 200x200x5 | 567 | 0.50 | 685 |
| 200x200x5 | 463 | 0.41 | 764 |

**Figure 5.3** *Tests specimens after fire tests*
*Left: RHS 150x150x3 Right: RHS 200x200x5*

# 5.3 Numerical studies

## 5.3.1 Calibration of numerical model

It is well established that the mechanical properties of stainless steel are strongly influenced by the level of cold-work the material has undergone. This results in significantly higher 0.2% proof strength in the corner regions compared to the flat faces. Ashraf et al[12] have proposed a formula, Eq. (5.1), to predict the strength of cold-formed corner regions $\sigma_{0.2,c}$. This equation is independent of the production route, and it can be used both for roll-formed and press-braked columns. Prediction is based on 0.2 % proof strength of the virgin sheet, $\sigma_{0.2,v}$, inner corner radius of the cross-section $r_i$ and cross-section thickness, $t$.

$$\sigma_{0.2,c} = \frac{1.881\sigma_{0.2,\mathrm{v}}}{\left(\frac{r_i}{t}\right)^{0.194}} \tag{5.1}$$

Gardner and Nethercot[13] have found that extending the corner properties to $2t$ beyond the curved portions of the cross section give the best agreement with test results.

Zhao & Blanguernon[14] have defined reduction factors for cold-worked material and concluded that for temperatures below 700°C the use of the reduction factors for the annealed material lead to conservative results. For instance, at 600°C the 0.2 % proof strength of 1.4571 C850 is more than 20% greater than the annealed material. These large differences for cold-worked material indicate that cold-forming affects the material properties at elevated temperatures.

Ala-Outinen[15] tested both virgin sheet and corner material from cold rolled square hollow sections made of EN 1.4301. It was concluded that the cold-formed material performs better at elevated temperatures than the annealed material. Table 5.4 compares the strength reduction factors derived by Ala-Outinen with the values for the annealed material given in EN 1993-1-2. A comparison of experimental results and results of FE predictions with different corner properties is presented in Table 5.5. For the FE-model, the reduction factors from EN 1993-1-2 were used for the material in the flat faces and the reduction factors derived from tests by Ala-Outinen were used for the corner regions.

**Table 5.4** *Comparison of strength reduction factors for grade 1.4301*

| Temperature °C | $k_{0.2,p,\theta}$ enhanced corner properties Ref [15] | $k_{0.2,p,\theta}$ annealed material EN 1993-1-2 |
|---|---|---|
| 20 | 1.00 | 1.00 |
| 100 | 0.91 | 0.82 |
| 200 | 0.88 | 0.68 |
| 300 | 0.83 | 0.64 |
| 400 | 0.80 | 0.60 |
| 500 | 0.70 | 0.54 |
| 600 | 0.64 | 0.49 |
| 700 | 0.42 | 0.40 |
| 800 | 0.28 | 0.27 |
| 900 | 0.10 | 0.18 |

**Table 5.5** *Comparison between test and FEA failure temperatures with different assumptions for material properties in the corner regions for 200x200x5*

| | |
|---|---|
| **Test** | 685°C |
| **FEA: No strength enhancement in corners** | 520°C |
| **FEA: Enhanced corner properties, reduction factors for annealed material in EN1993-1-2** | 625°C |
| **FEA: Enhanced corner properties, reduction factors for cold formed material according to Ref [15]** | 645°C |

Both global and local imperfections were considered and their influence was evaluated through a sensitivity analysis. The outcome of the sensitivity analysis was that the global imperfections are negligible and the local imperfections have little influence on the compression resistance. Therefore the former are neglected and for the latter, the measured values were used in the model. No residual stresses were introduced in the modelling of the column as it has been shown previously that they have little influence on the overall behaviour of a stub column [13].

A comparison between the numerical predictions and the test results shows that the numerical analysis is quite accurate and the failure mode obtained (local buckling) is consistent with the experiments (Table 5.6).

**Table 5.6** *Comparison of measured and predicted failure temperatures*

| **Cross-section** | **Temperature °C** | | **$Temp_{FE}$/ $Temp_{test}$** |
|---|---|---|---|
| | **Test** | **FE model** | |
| 150 × 150 × 3 | 676 | 716 | 1.06 |
| 150 × 150 × 3 | 720 | 758 | 1.05 |
| 150 × 150 × 3 | 588 | 593 | 1.01 |
| 200 × 200 × 5 | 609 | 482 | 0.79 |
| 200 × 200 × 5 | 685 | 645 | 0.94 |
| 200 × 200 × 5 | 764 | 732 | 0.96 |

A thermal dependent static stress analysis was carried out in ABAQUS by defining a temperature-time curve with data obtained from the tests and applying it to the field containing the mesh that defines the column. The temperature-time curve was a linear approximation of the temperatures measured with the twelve thermocouples.

### 5.3.2 Parametric studies

A parametric study was performed to investigate the behaviour of thin walled stainless steel box columns. The applied load levels, as well as the global and local slenderness were varied and the results were compared to the predicted strengths according to the EN 1993-1-2 design model. To investigate a possible practical application of Class 4 stainless steel columns, the parametric study was extended to include the length L=3.1 m for all cross-sections and load levels.

The validated FE-model was used for the parametric study. However, due to the greater slendernesses simulated than in the experiments, the global imperfections had to be taken into account. The local imperfections were taken as b/200 and the global imperfection were taken as L/1000 in accordance with the allowed tolerances in prEN1090-2[16]. With nominal material properties (including the corner properties) and cross-sectional dimensions, the failure loads from the FE-simulations at room temperature were compared to the ultimate loads calculated in accordance with EN 1993-1-4 and good agreement was obtained.

The end constraints were pinned for all columns, both at ambient temperature and at elevated temperature. It was assumed that the temperature distribution was uniform across and along the column. The failure loads from the FE-simulations at ambient temperature were used to calculate the appropriate loads for each load level, cross-section and slenderness used in the simulations at elevated temperatures.

The results from the parametric study were compared to the design model in EN 1993-1-2 in Table 5.7.

**Table 5.7** *Results from FE compared to predicted failure loads according to EN 1993-1-2 (Load level = 30% of ultimate load at the ambient temperature)*

| **Cross-section** | $\frac{\text{Failure load}_{FE}}{\text{Failure load}_{EN1993-1-2}}$ | | |
|---|---|---|---|
| | $\bar{\lambda} = 1.2$ | $\bar{\lambda} = 0.5$ | $\bar{\lambda} = 0.8$ |
| 200x200x4 | 0.76 | 0.74 | 0.73 |
| 200x200x5 | 0.81 | 0.82 | 0.79 |
| 300x300x5 | 0.71 | 0.69 | 0.67 |

It is clear that the design model according to the EN 1993-1-2 predicts the failure load at elevated temperature with varying results depending on the cross-section slenderness. Greater local slenderness leads to more conservative results. This is a result of the Eurocode method neglecting the more favourable relationship between strength and stiffness at elevated temperatures for local buckling.

The time to failure of columns of length 3.1 m were studied assuming exposure to the standard fire curve. The indication is that it is possible for unprotected stainless steel columns of this length to achieve a fire resistance Class R30.

**Table 5.8** *Predicted failure temperature and time (Load level = 30% of ultimate load at the ambient temperature)*

| **Cross-section** | **Failure temperature °C** | **Failure time (mins)** |
|---|---|---|
| 200x200x4 | 810 | 28.1 |
| 200x200x5 | 790 | 27.0 |
| 300x300x5 | 816 | 30.5 |

### 5.3.3 Development of design guidance

The intention of this new design model proposed for elevated temperatures is that it is valid even for room temperature. Therefore the buckling curve with imperfection factor, $\alpha$, and the limiting slenderness, $\bar{\lambda}_0$, are taken as 0.49 and 0.4 respectively as it is given in EN 1993-1-4. The results from the parametric study clearly indicated the importance of taking the temperature dependent relationship between strength and stiffness into account for local buckling as well as for global buckling.

The proposed design model is given below. The basic form of the buckling curve is given in eq (5.2) and (5.3). As well as the local and global slenderness being temperature dependent - eq (5.4), (5.5) and (5.6) - the limiting slenderness also depends on the strength–stiffness ratio at the temperature of interest eq (5.7).

$$\chi_{\mathrm{fi}} = \frac{1}{\varphi_\theta + \sqrt{\varphi_\theta^{\,2} - \bar{\lambda}_\theta^{\,2}}} \quad \text{but } \chi_{\mathrm{fi}} \leq 1{,}0 \tag{5.2}$$

$$\varphi_\theta = 0.5\left[1 + \alpha\left(\bar{\lambda}_\theta - \bar{\lambda}_{0,\theta}\right)\right] + \bar{\lambda}_\theta^{\,2} \tag{5.3}$$

$$\bar{\lambda}_{\mathrm{p},\theta} = \frac{\bar{b}/t}{28{,}4\varepsilon_\theta\sqrt{k_\sigma}} \tag{5.4}$$

$$\varepsilon_\theta = \varepsilon\left[\frac{k_{\mathrm{E},\theta}}{k_{0.2\mathrm{p},\theta}}\right]^{0,5} \tag{5.5}$$

$$\bar{\lambda}_\theta = \bar{\lambda}\left[\frac{k_{0.2\mathrm{p},\,\theta}}{k_{\mathrm{E},\theta}}\right]^{0,5} \quad \text{is the modified non-dimensional slenderness at temperature } \theta \tag{5.6}$$

$$\bar{\lambda}_{0,\theta} = \bar{\lambda}_0\left[\frac{k_{0.2\mathrm{p},\,\theta}}{k_{\mathrm{E},\theta}}\right]^{0,5} \quad \text{is the modified limiting non-dimensional slenderness} \tag{5.7}$$

Where

$\alpha = 0.49$, for hollow sections according to EN 1993-1-4,

| | |
|---|---|
| $\bar{\lambda}$ | is the non-dimensional slenderness |
| $\bar{\lambda}_0$ | is the limiting non-dimensional slenderness where the reduction of the strength starts due to the slenderness |
| $\bar{b}$ | is the relevant width |
| $t$ | is the relevant thickness |
| $k_\sigma$ | is the buckling factor |
| $\varepsilon$ | is the material factor |
| $k_{\mathrm{E},\theta}$ | is the reduction factor for Young's modulus |
| $k_{0.2\mathrm{p},\theta}$ | is the reduction factor for 0.2 proof stress |

Figure 5.4 shows the results for the proposed revised design model compared to FE analysis for columns with a 50% load ratio. Overall, the design method gives a mean value for the ratio of the failure load predicted by the design method to that predicted by FE of 1.01 with a coefficient of variation of 0.08. This represents a significant improvement when compared to the values predicted using the EN 1993-1-2 method.

A summary of mean values of Design model/FEA and coefficients of variation (COV) are presented in Table 5.9 below. Equivalent mean values for the current design method in EN 1993-1-2 are also shown.

**Table 5.9** *Mean values and coefficients of variation for different design models for all Class 4 cross-sections included in the parametric study.*

| Load level | 30 % | | 40 % | | 50 % | | All load levels | |
|---|---|---|---|---|---|---|---|---|
| | Mean | COV | Mean | COV | Mean | COV | Mean | COV |
| EN 1993-1-2 | 0.76 | 0.10 | 0.74 | 0.09 | 0.73 | 0.08 | 0.74 | 0.10 |
| Design Manual[2] | 0.97 | 0.15 | 0.95 | 0.14 | 0.94 | 0.14 | 0.96 | 0.17 |
| Proposed new method | 1.01 | 0.08 | 0.99 | 0.08 | 0.98 | 0.11 | 0.99 | 0.12 |

It is clear that the proposed design model gives improved predictions of the failure loads.

The results of a variety of further simulations suggest that the proposed design model can be used for different stainless steel grades.

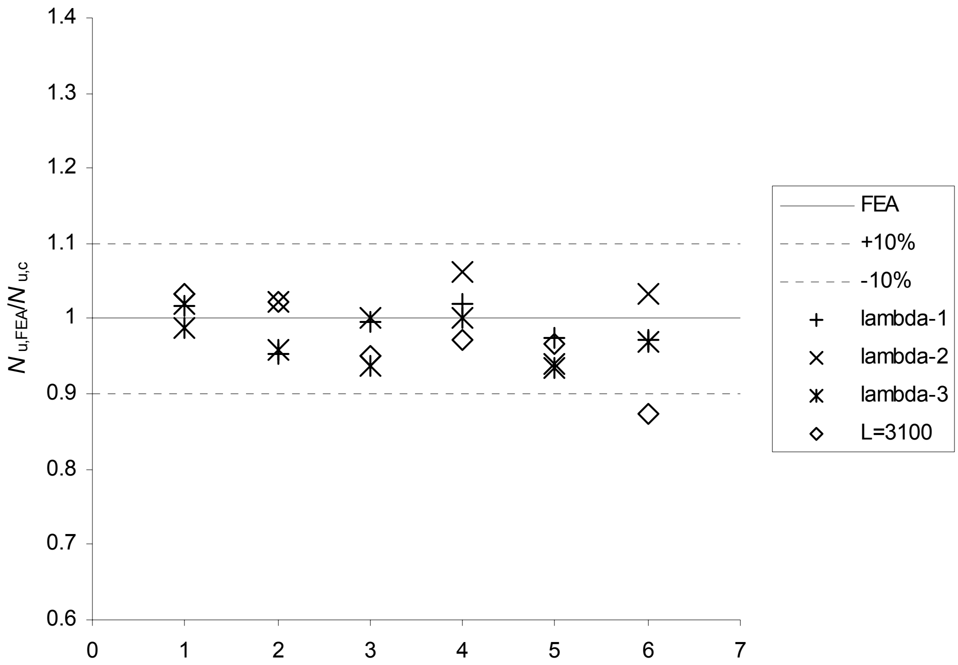



**Figure 5.4** *Comparison of the proposed design model and FEA at elevated temperature, 50 % load level*

## 5.4 Conclusions

A programme of tests on RHS with slender (Class 4) cross-sections was performed. Numerical models were calibrated against test results and then parametric studies carried out to develop more economic design guidance than is currently in existing guidance. The proposed model uses the room temperature buckling curve with the global, local and limiting slendernesses all being related to the temperature-dependent ratio of strength and stiffness. The analysis of 3.1 m long pinned columns in a standard fire shows that it is possible for unprotected Class 4 stainless steel columns to achieve 30 minutes fire resistance if the load level is low.

# 6 WP4: PROPERTIES AT ELEVATED TEMPERATURES

Detailed descriptions of the activities carried out under this work package are given in the relevant Final Work Package Report listed in Section 11.

## 6.1 Objectives

The main objective of this work package is to define the parameters for the constitutive model in Eurocode 3-1-2 for two grades of stainless steel which have not been modelled before.

## 6.2 Experimental work

Transient state (i.e. anisothermal) tests were performed on two austenitic grades:

- STR 18: A low nickel, manganese grade, delivered by Thyssen Krupp AST,
- EN 1.4541: A chromium nickel grade stabilised with titanium, delivered by Outokumpu Stainless Oy.

Both the grades were delivered in the form of 1.5 mm thick sheets in the annealed condition. The casting chemical composition of STR 18 is given in Table 6.1. The chemical composition of grade EN 1.4541 is given in the product standard EN 10088-1[17].

**Table 6.1** *Casting chemical composition of grade STR 18*

| % by mass | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| C | Si | Mn | P | S | N | Cr | Cu | Mo | Nb | Ni | Ti |
| 0.04 | 0.23 | 11.05 | 0.026 | 0.002 | 0.27 | 17.85 | 1.87 | 0.13 | 0.01 | 3.95 | 0.01 |

Transient state tests simulate the real conditions of a structure subject to fire. The specimen is positioned in the furnace and subjected to a constant load (expressed as a percentage of $R_{0.2p}$) while the temperature is raised linearly at a rate of 10°C/min from room temperature up to 1000°C. The conditions of the test are represented in Figure 6.1 and Table 6.3 summarises the experimental test programme carried out on both of grades.

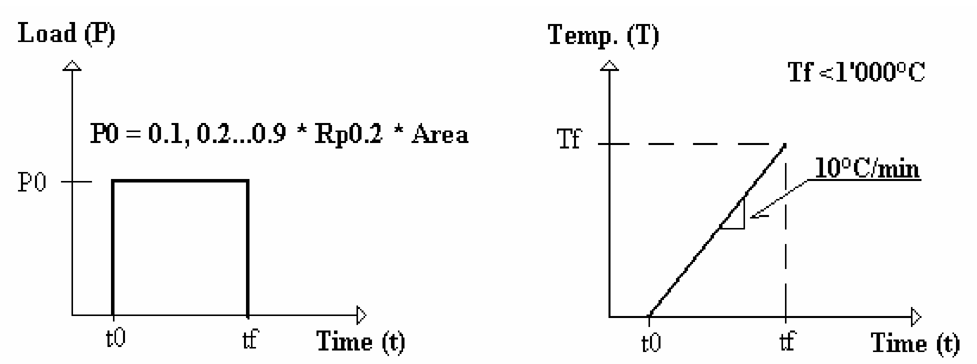



**Figure 6.1** *Transient state test*

For the transient state test, the sheets were machined to obtain specimens in the longitudinal direction. Figure 6.2 shows a sketch of the specimen for this test. Specimens to perform standard tensile tests were machined both in the longitudinal and transverse direction.

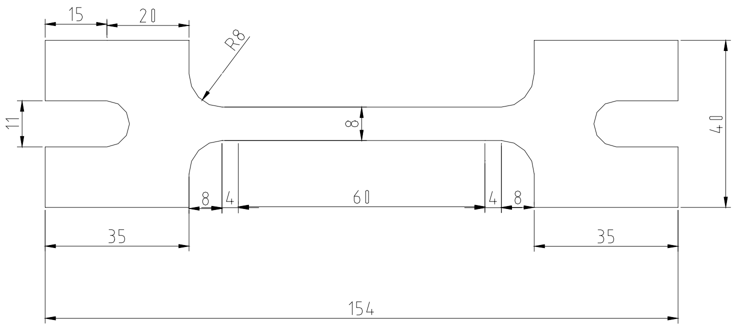



**Figure 6.2** *Transient state tests specimen*

Standard tensile tests were performed to evaluate the stress level to be applied during the transient state tests. The experimental results are summarised in Table 6.2.

**Table 6.2** *Tensile tests results at room temperature*

| Material | Test no | Longitudinal | | | | Transverse | | | |
|---|---|---|---|---|---|---|---|---|---|
| | | $R_{0.2p}$ [MPa] | $R_m$ [MPa] | A(%) | Z(%) | $R_{0.2p}$ [MPa] | $R_m$ [MPa] | A(%) | Z(%) |
| STR 18 | 1 | 392 | 759 | 71 | 55 | 385 | 744 | 67 | 51 |
| | 2 | 382 | 752 | 73 | 56 | 384 | 743 | 66 | 51 |
| | 3 | 382 | 748 | 73 | 56 | 386 | 739 | 66 | 51 |
| | ***Mean*** | ***385*** | ***753*** | ***72*** | ***56*** | ***385*** | ***742*** | ***66*** | ***51*** |
| EN 1.4541 | 1 | 236 | 658 | 79 | 57 | 265 | 663 | 80 | 59 |
| | 2 | 248 | 668 | 79 | 55 | 258 | 657 | 81 | 56 |
| | 3 | 244 | 656 | 78 | 55 | 261 | 657 | 83 | 61 |
| | ***Mean*** | ***243*** | ***661*** | ***79*** | ***56*** | ***261*** | ***659*** | ***81*** | ***59*** |

**Table 6.3** *Test programme for transient state tests*

| Material | Direction of the specimen with respect to the rolling direction | Temperature curve | Load type | Load level (% of $R_{0.2p}$) | Number of tests | Output |
|---|---|---|---|---|---|---|
| STR 18 | Longitudinal | Room temperature | Tensile | - | 3 | Stress-strain curves |
| | Transverse | Room temperature | Tensile | - | 3 | |
| | Longitudinal | Linear (10°C/min) up to failure or 1000°C | Axial and Constant | 1% | 1 | Strain-Temperature curves |
| | | | | 10% | 2 | |
| | | | | 20% | 1 | |
| | | | | 30% | 1 | |
| | | | | 40% | 2 | |
| | | | | 50% | 1 | |
| | | | | 60% | 1 | |
| | | | | 70% | 2 | |
| | | | | 80% | 1 | |
| | | | | 90% | 1 | |
| EN 1.4541 | Longitudinal | Room temperature | Tensile | - | 3 | Stress-strain curves |
| | Transverse | Room temperature | Tensile | - | 3 | |
| | Longitudinal | Linear (10°C/min) up to failure or 1000°C | Axial and Constant | 1% | 1 | Strain-Temperature curves |
| | | | | 20% | 1 | |
| | | | | 30% | 2 | |
| | | | | 50% | 1 | |
| | | | | 60% | 1 | |
| | | | | 70% | 1 | |
| | | | | 80% | 1 | |

Stress-strain curves of transient state test results at different temperatures show that there is a shift in the curves for different temperatures along the x-axis, which reveals the presence of parasite strains induced by the testing machine adjustments (see Figure 6.3). These strains can be estimated by performing a transient state test at a very low stress level (1% of $R_{0.2p}$).

Figure 6.4 shows the strain-temperature plot of the results after subtracting the parasite strains for stainless steel EN 1.4541. It is observed that the stress-strain curve still does not match the axes origin.

A further estimate of parasite strains was necessary in order to obtain a better match of the stress-strain curve with the axes' origin. This was due to difficulties in determining Young's modulus at high temperatures. It was observed that the values obtained initially were lower than those given in EN 1993-1-2. This was due to the time necessary to set up the test, which allowed material relaxation and consequently introduced creep phenomena. It is generally understood that it is very difficult to achieve a good prediction of Young's modulus at elevated temperatures in transient state tests; isothermal tests are the preferred way of measuring Young's modulus.

The difficulties encountered in the determination of the parasite strains and evaluation of elastic modulus suggest that there is a need for developing a well defined procedure for the execution of transient state tests and for the subsequent analysis of experimental data for the evaluation of retention parameters.

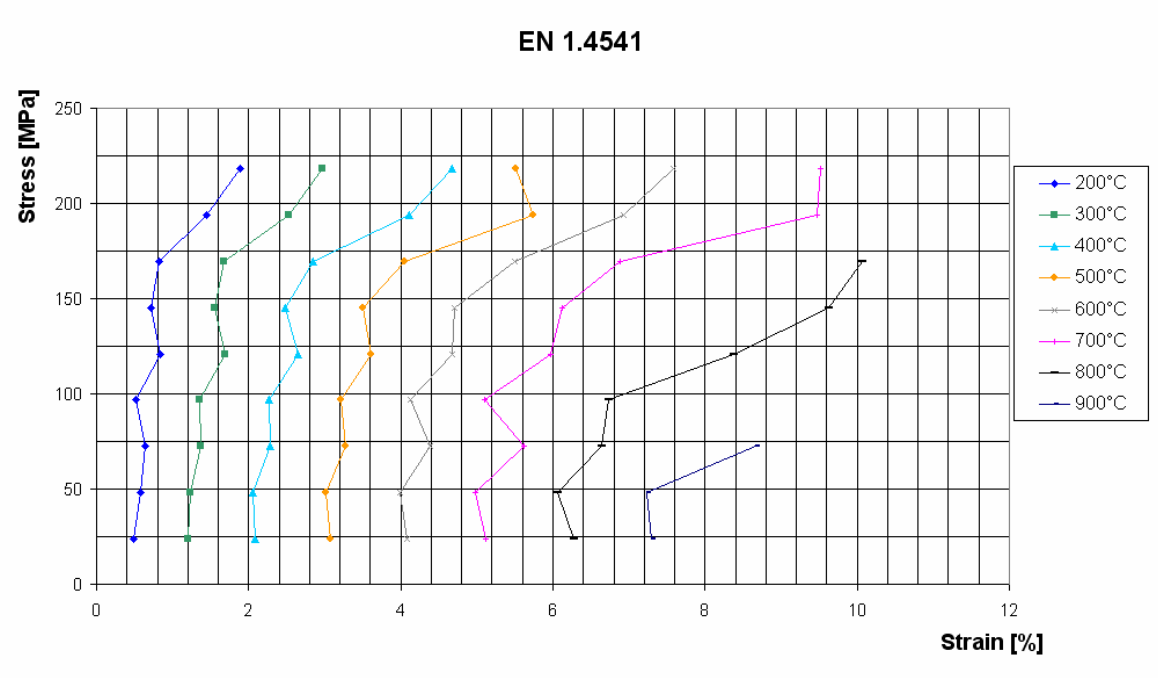



**Figure 6.3** *EN 1.4541 steel stress-strain curves including parasite strains*

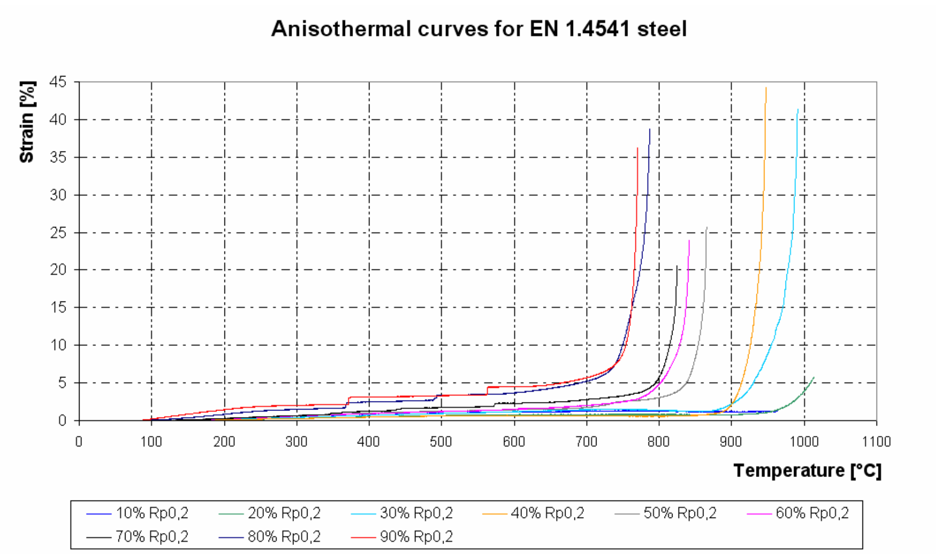



**Figure 6.4** *Strain-temperature curve from transient state tests on EN 1.4541 stainless steel (parasite strains have been subtracted)*

The data collected through the tests were used to evaluate stress-strain curves at a given temperature by extracting the corresponding value of the strain from the stress-temperature curve at a given stress level (i.e. % of $R_{0.2p}$). These experimental stress-strain curves can be fitted to a numerical model, in order to obtain the material constitutive law representing the general stainless steel behaviour at elevated temperatures. Equations from EN 1993-1-2 for stainless steel material behaviour at elevated temperatures have been found to fit well the experimental data.

The following procedure used to evaluate the retention parameters:

1. $f_{u,\theta}$ (and consequently the retention parameter $k_{u,\theta}$) and $\varepsilon_{u,\theta}$ , are taken from isothermal tensile tests (steady state tests).
2. $f_{0.2p,\theta}$ and $E_{a,\theta}$ (and consequently the retention parameters $k_{0.2p,\theta}$ and $k_{E,\theta}$) are evaluated from stress-strain curves obtained from transient state tensile tests, with $\varepsilon_{c,\theta} = \dfrac{f_{0.2p,\theta}}{E_{a,\theta}}$
3. The parameter $k_{2\%,\theta}$ is calculated as $k_{2\%,\theta} = \dfrac{f_{2\%,\theta} - f_{0.2p,\theta}}{f_{u,\theta} - f_{0.2p,\theta}}$ where: $f_{2\%,\theta}$ is graphically determined from the numerical model of the material experimental curves. $k_{2\%,\theta}$ is needed for the calculation of the yield strength with the simple calculation method, but it has no influence in the definition of material model.
4. $k_{Ect,\theta} = \dfrac{E_{ct,\theta}}{E_a}$ where $E_{ct,\theta}$ is adjusted to fit the material model with respect to the experimental data.
5. Before calculating the necessary parameters from the experimental results, parasite strains are subtracted from the test measurements.

In Figure 6.5 and Figure 6.6, experimental stress-strain curves (shown in red) and material model (shown in blue) are compared and the material retention factors for the two grades tested are reported in Table 6.4.

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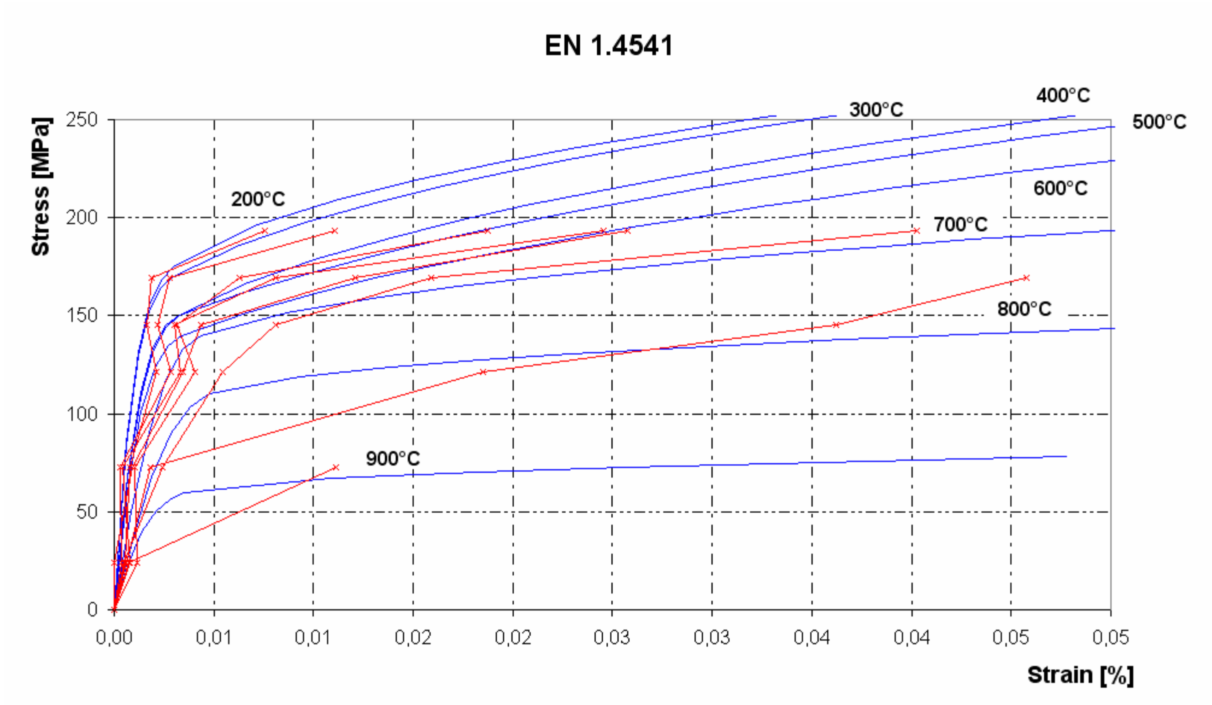



**Figure 6.5** *EN 1.4541 stress-strain curves: experimental(red) and material model (blue)*

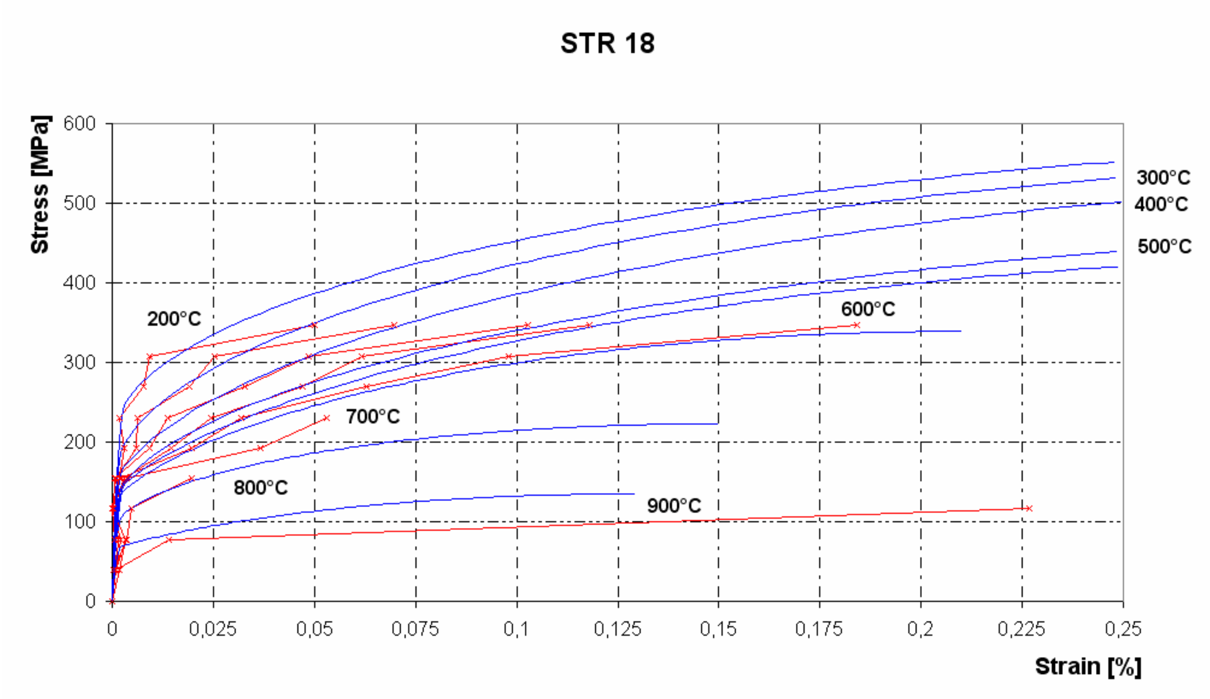



**Figure 6.6** *STR 18 stress-strain curves: experimental (red) and material model (blue)*

**Table 6.4** *Material reduction factors*

| $\theta_a$ | $k_{E,\theta} = \frac{E_{a,\theta}}{E_a}$ | $k_{0.2p,\theta} = \frac{f_{0.2p,\theta}}{f_y}$ | $k_{u,\theta} = \frac{f_{u,\theta}}{f_u}$ | $k_{2\%,\theta}$ | $k_{Ect,\theta} = \frac{E_{ct,\theta}}{E_a}$ | $\varepsilon_{u,\theta}$ |
|---|---|---|---|---|---|---|
| **EN 1.4541** | | | | | | |
| 20 | 1 | 1 | 1 | 0.14 | 0.04 | 0.63 |
| 200 | 0.92 | 0.63 | 0.73 | 0.21 | 0.04 | 0.49 |
| 300 | 0.88 | 0.61 | 0.70 | 0.22 | 0.03 | 0.4 |
| 400 | 0.60 | 0.54 | 0.70 | 0.21 | 0.03 | 0.42 |
| 500 | 0.60 | 0.54 | 0.68 | 0.19 | 0.02 | 0.4 |
| 600 | 0.50 | 0.50 | 0.62 | 0.19 | 0.02 | 0.39 |
| 700 | 0.30 | 0.50 | 0.48 | 0.19 | 0.02 | 0.52 |
| 800 | 0.20 | 0.40 | 0.34 | 0.21 | 0.02 | 0.55 |
| 900 | 0.20 | 0.22 | 0.20 | 0.18 | 0.02 | 0.89 |
| **STR 18** | | | | | | |
| 20 | 1 | 1 | 1 | 0.18 | 0.04 | 0.47 |
| 200 | 0.92 | 0.65 | 0.77 | 0.22 | 0.04 | 0.40 |
| 300 | 0.88 | 0.52 | 0.74 | 0.22 | 0.04 | 0.38 |
| 400 | 0.84 | 0.44 | 0.72 | 0.19 | 0.03 | 0.42 |
| 500 | 0.80 | 0.39 | 0.63 | 0.19 | 0.03 | 0.43 |
| 600 | 0.50 | 0.39 | 0.58 | 0.18 | 0.02 | 0.35 |
| 700 | 0.71 | 0.36 | 0.45 | 0.21 | 0.02 | 0.21 |
| 800 | 0.63 | 0.29 | 0.30 | 0.36 | 0.02 | 0.15 |
| 900 | 0.45 | 0.18 | 0.18 | 0.32 | 0.01 | 0.12 |

$\theta_a$ is the steel temperature
$k_{E,\theta}$ is the reduction factor for the slope of the linear elastic range
$k_{0.2p,\theta}$ is the reduction factor for proof strength
$k_{u,\theta}$ is the reduction factor for tensile strength
$k_{2\%,\theta}$ is the factor for determination of the yield strength $f_{y,\theta}$
$k_{Ect,\theta}$ is the reduction factor for the slope of the linear elastic range
$\varepsilon_{u,\theta}$ is the ultimate strain

# 6.3 Conclusions

Transient state tests were successfully carried out on two grades of stainless steel not previously tested before. Using the test results, strength and stiffness parameters were derived for use with the numerical model in EN 1993-1-2.

# 7 WP5: BOLTS AND WELDS AT ELEVATED TEMPERATURES

Detailed descriptions of the activities carried out under this work package are given in the relevant Final Work Package Report listed in Section 11.

## 7.1 Objectives

The objective of this work package is to study the behaviour of bolted and welded connections in stainless steel in fire.

## 7.2 Experimental work

### 7.2.1 Welded connections

Steady state (isothermal) tests were carried out on butt welded joints in two grades of stainless steel, grades 1.4318 and 1.4571 austenitic stainless steel.

The steady state tensile tests were carried out using the SWICK Z250/SW5A material testing machine according to EN 10002-5[11]. Metal Active Gas (MAG) welding was used with the following wire electrodes:

For 1.4318 steel: AVESTA 308L/MVR

For 1.4571 steel: AVESTA 318/8kNb

Welds were laid parallel to the rolling direction and test coupons were cut from a 6 mm thick stainless steel sheet transverse to the level of the base material. All welded seams were grinded to the level of the surface of the base material.

Tests were carried out at temperature intervals of 100°C up to 600°C and at intervals of 50°C from 600°C to 1100°C. The tests were performed twice and if a significant difference was measured between the two results, a third test was performed. Tests at room temperature were also carried out to determine the mechanical properties.

Figure 7.2 and Figure 7.2 show the fracture points for the butt welded joints in both grades. Welding causes a heating and cooling cycle in the area surrounding the welded joint. Critical areas in the welded connection are the HAZ and weld metal. At normal temperature, the joint area is supposed to be the most critical for failure, but at elevated temperature the joint behaviour might be different because the material is heat-treated all over[18]. In this study the fractures were mostly locating in the weld due to reason that the welds were ground to the level of the surface of the base material. Nevertheless, typically in the case of unground welds, a fracture of the joint is located in the HAZ or in the base material.

The strength retention factors for the butt welded joints for both annealed stainless steel grades were compared to factors for the base material given in the *Design Manual for Structural Stainless Steel*[2]. The results are shown in Figure 7.3 and Figure 7.4 and it is clear from these graphs that the strengths of butt welded joints at elevated temperatures in both grades were at the same level or even better as base material studied earlier projects. It can be concluded from the test results that that the design strength of a full penetration butt weld, for temperatures up to 1000°C, can be taken as equal to the strength of the base material for grades 1.4318 and 1.4571 in the annealed condition. A similar conclusion was drawn from the work by Ala Outinen[18].

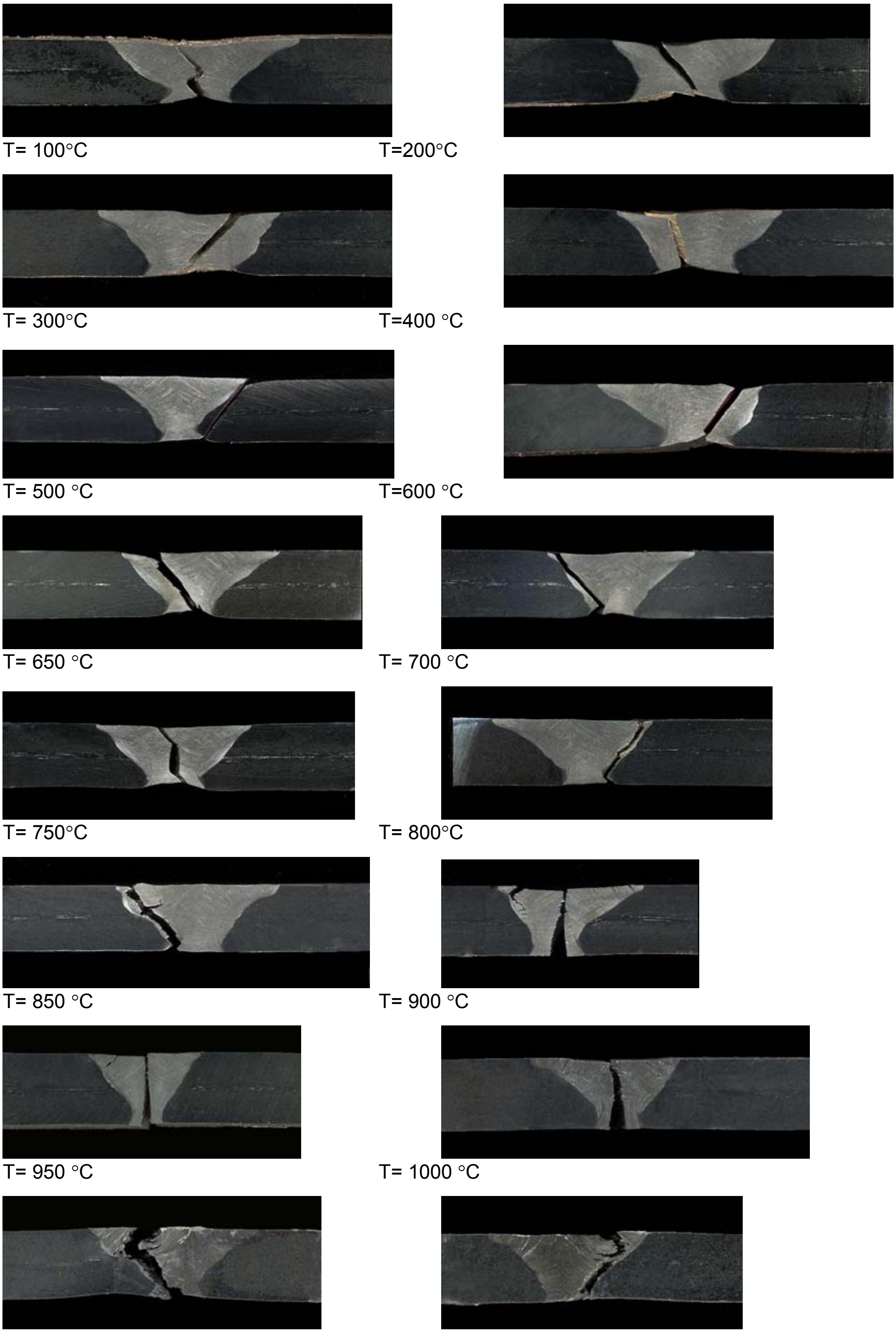
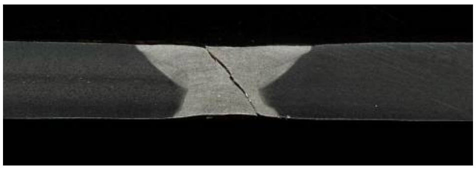
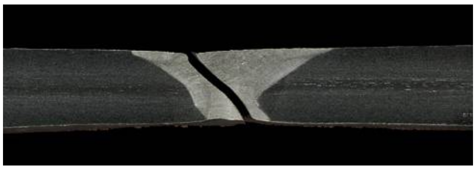
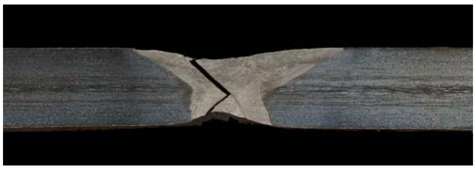


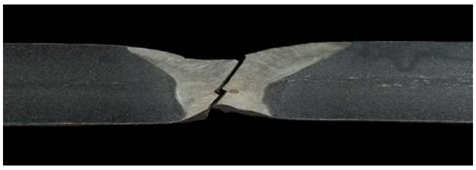
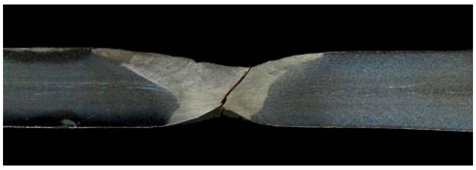
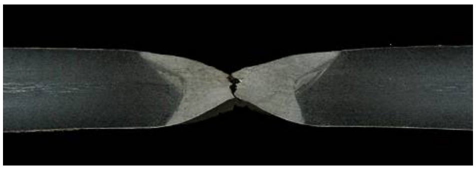

**Figure 7.1** *Fracture points of test samples in different temperatures for butt welds (grade 1.4318)*


T= 100 °C

T=200 °C

T=300 °C

T=400 °C

T=500 °C

T=600 °C

T=650 °C

T=700 °C

T=750 °C

T=800 °C

T=850 °C

T=900 °C

T=950 °C

**Figure 7.2** *Fracture points of test samples in different temperatures for butt welds (grade 1.4571)*

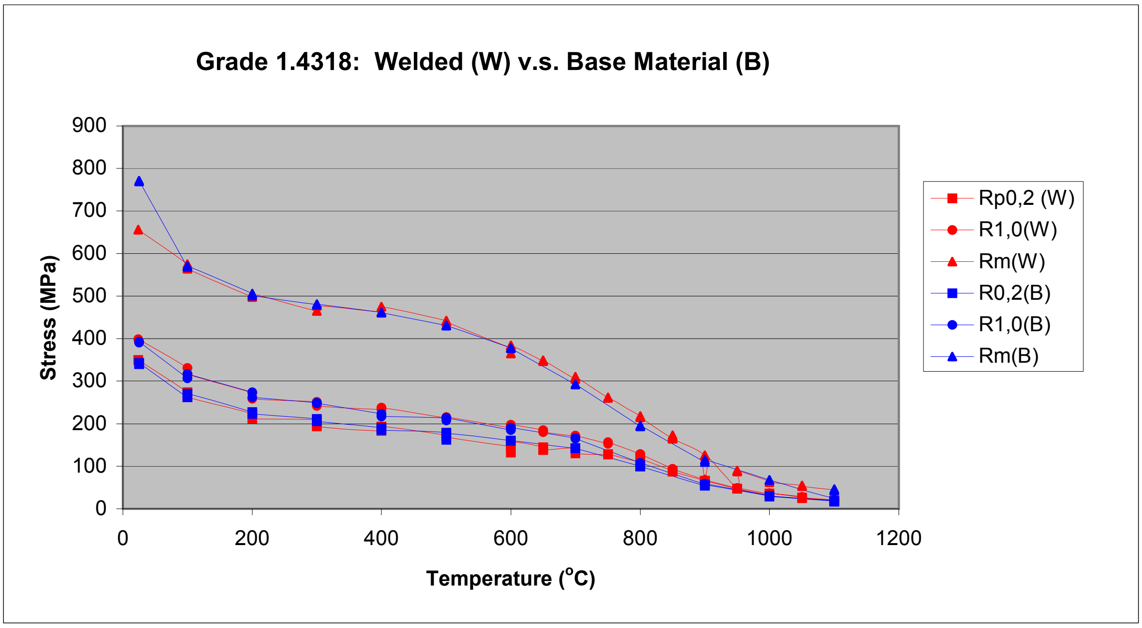



**Figure 7.3** *Tensile test results on weld materials for grade 1.4318*

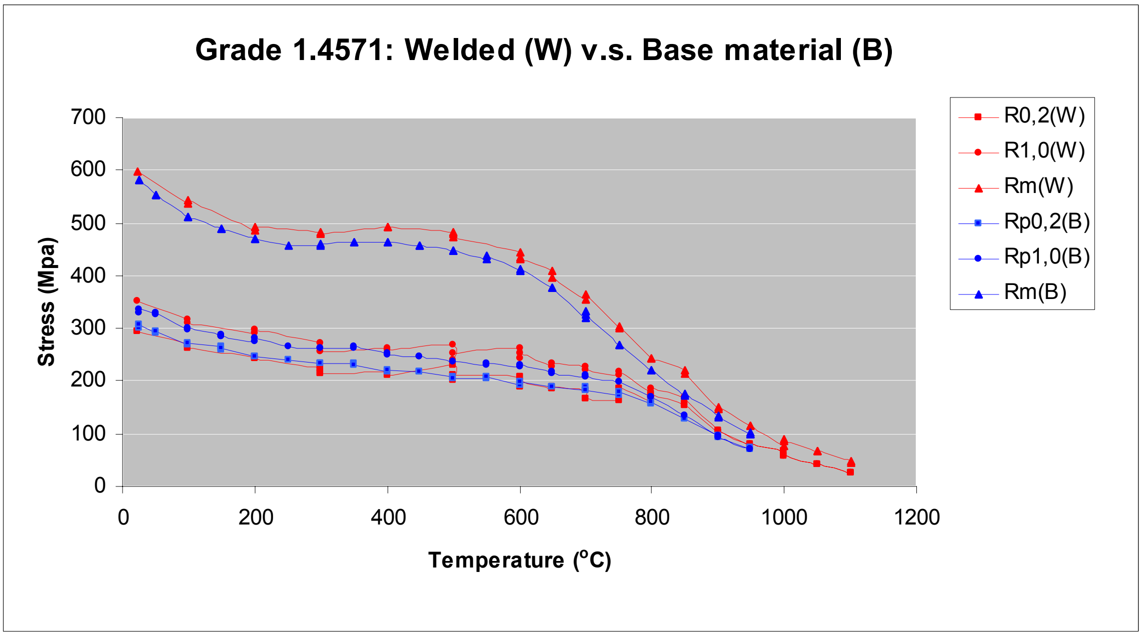



**Figure 7.4** *Tension test results on weld materials for grade 1.4571*

## 7.2.2 Bolted connections

A total of 41 stainless steel bolt assemblies machined by Ferriere di Stabio from stainless steel bars produced by Cogne Acciai Speciali were tested. Washers were not present. Austenitic bolts grade A2-70 (nominal tensile strength $f_{ub,nom}$ = 700 N/mm²) and A4-80 (nominal tensile strength $f_{ub,nom}$ = 800 N/mm²) in accordance with EN ISO 3506[19] were tested. The bolts were produced by cold forging and roll threading. The bolts were hexagonal head M12×50 half threaded in accordance with DIN 931. The nuts were also hexagonal.

Isothermal tests at 7 different temperatures from room temperature up to 900°C were performed loading the bolt assemblies in tension (21 tests) and shear (20 tests). A detailed test programme is given in Table 7.1 and the test arrangement is shown in Figure 7.5. The room temperature tensile test for the A4-80 bolt set was repeated because in the first test, failure occurred by thread stripping; it was decided to validate the result by checking if this mode of failure noticeably decreased bolt resistance. In the second test, failure occurred in the thread and the load was very similar to the first one. The testing procedure was based on the work carried out by Kirby[20] on carbon steel bolts at elevated temperature which led to the definition of the strength retention factors for carbon steel bolts in Annex D of EN 1993-1-2.

**Table 7.1** *Test programme*

| Material grade strength level | Load direction | Test temperature | No. of tests | Material grade strength level | Load direction | Test temperature | No. of tests |
|---|---|---|---|---|---|---|---|
| A2 70 | shear | RT | 1 | A4 80 | shear | RT | 1 |
| | | 200 °C | 1 | | | 200 °C | 1 |
| | | 300 °C | 2 | | | 300 °C | 2 |
| | | 400 °C | 1 | | | 400 °C | 1 |
| | | 500 °C | 1 | | | 500 °C | 1 |
| | | 600 °C | 2 | | | 600 °C | 2 |
| | | 800 °C | 1 | | | 800 °C | 1 |
| | | 900 °C | 1 | | | 900 °C | 1 |
| | tensile | RT | 1 | | tensile | RT | 2 |
| | | 200 °C | 1 | | | 200 °C | 1 |
| | | 300 °C | 2 | | | 300 °C | 2 |
| | | 400 °C | 1 | | | 400 °C | 1 |
| | | 500 °C | 1 | | | 500 °C | 1 |
| | | 600 °C | 2 | | | 600 °C | 2 |
| | | 800 °C | 1 | | | 800 °C | 1 |
| | | 900 °C | 1 | | | 900 °C | 1 |
| | | | **20** | | | | **21** |

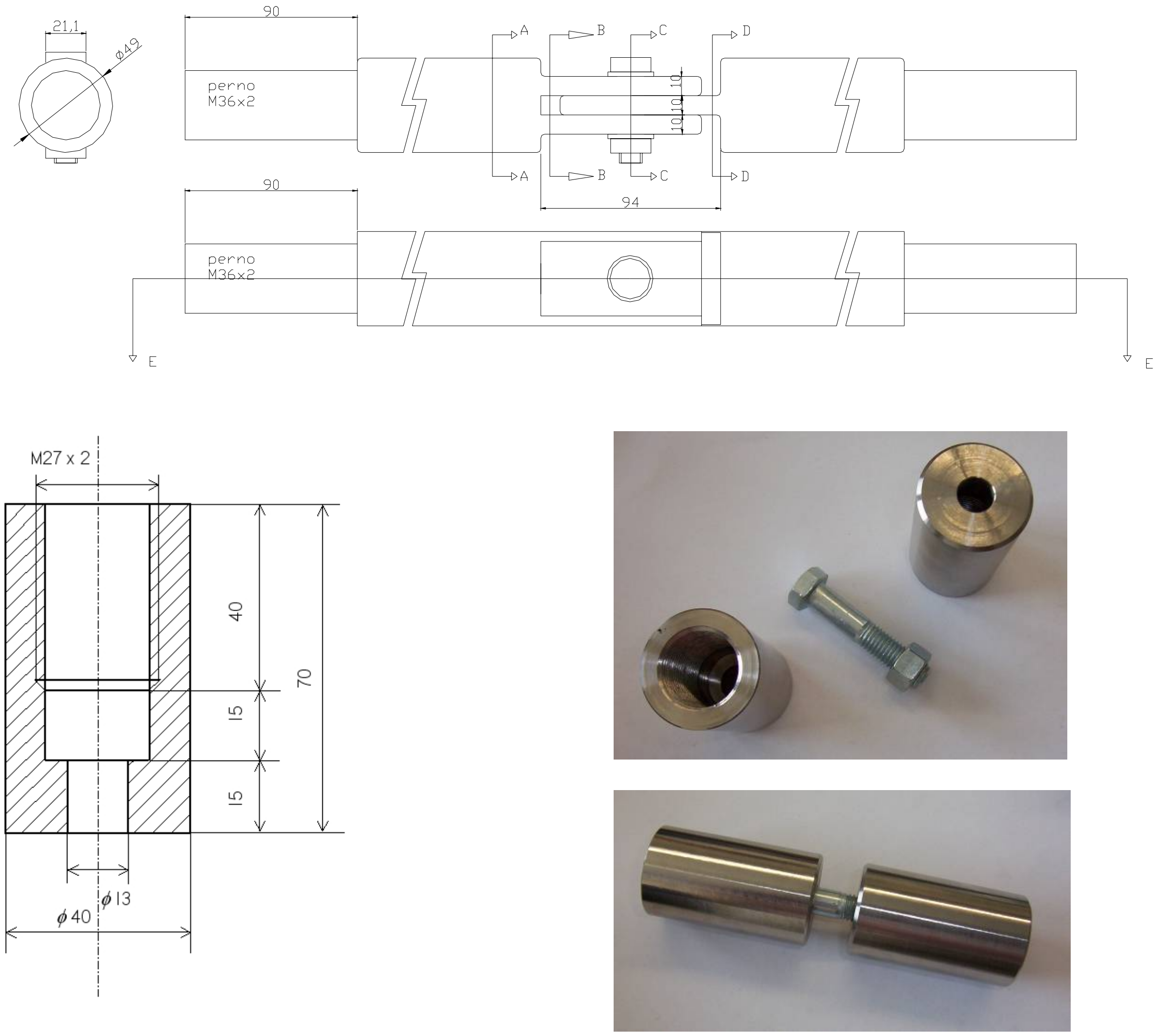



**Figure 7.5** *Connection design for bolt tests at elevated temperature*
*Above: shear test below: tensile test*
*(Dimensions are in mm)*

On the basis of EN 1993-1-8[21], the single bolt connections shown in Figure 7.5 are classified as:

**a)** Category A non-preloaded shear resistant single bolt connection; the shear resistance of the bolt at elevated temperature ($\mathbf{F_{v,Rd}}$ ) has been measured during the test.

**b)** Category D non-preloaded tension resistant single bolt connection; the tensile resistance of the bolt ($\mathbf{F_{t,Rd}}$ ) has been measured during the test.

The material used for the grips was the heat resistant alloy NIMONIC 115. A heat resistant alloy was chosen to ensure that failure occurs in the bolted connection itself and not in the grips. Using a heat resistant alloy also meant that prying effects (arising from increased tensile stresses causing the bolt thread to extend) were avoided. The testing procedure defined on the basis of the experimental work by Kirby on carbon steel bolts at elevated temperature is shown in Figure 7.6 in terms of displacement and heating rates. The tests were carried out in displacement control, measuring machine crossbeam displacement. The temperature was feedback-controlled by a thermocouple applied inside the furnace; another thermocouple was applied on the bolt itself and enabled the temperature differences between the two zones to be monitored (see Figure 7.7).

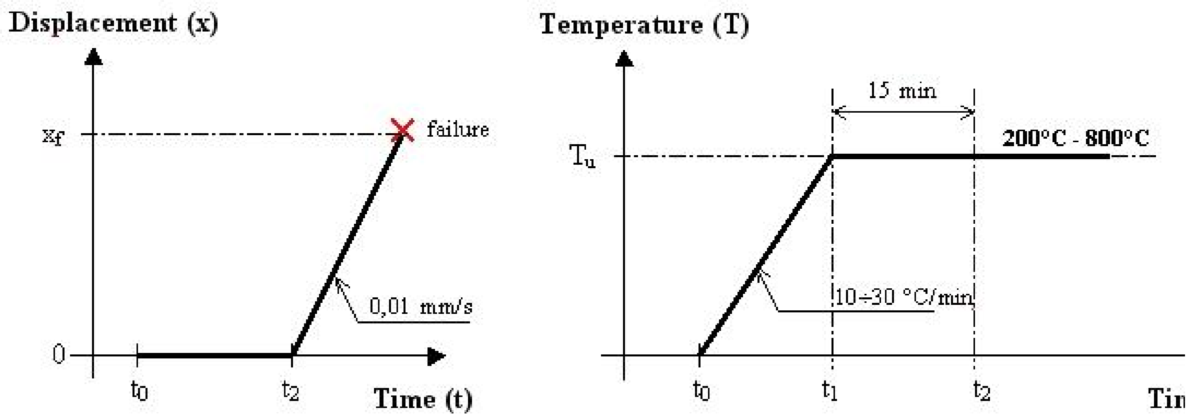



**Figure 7.6** *Testing procedures for bolted connections at elevated temperature: Left: displacement rate Right: heating rate*

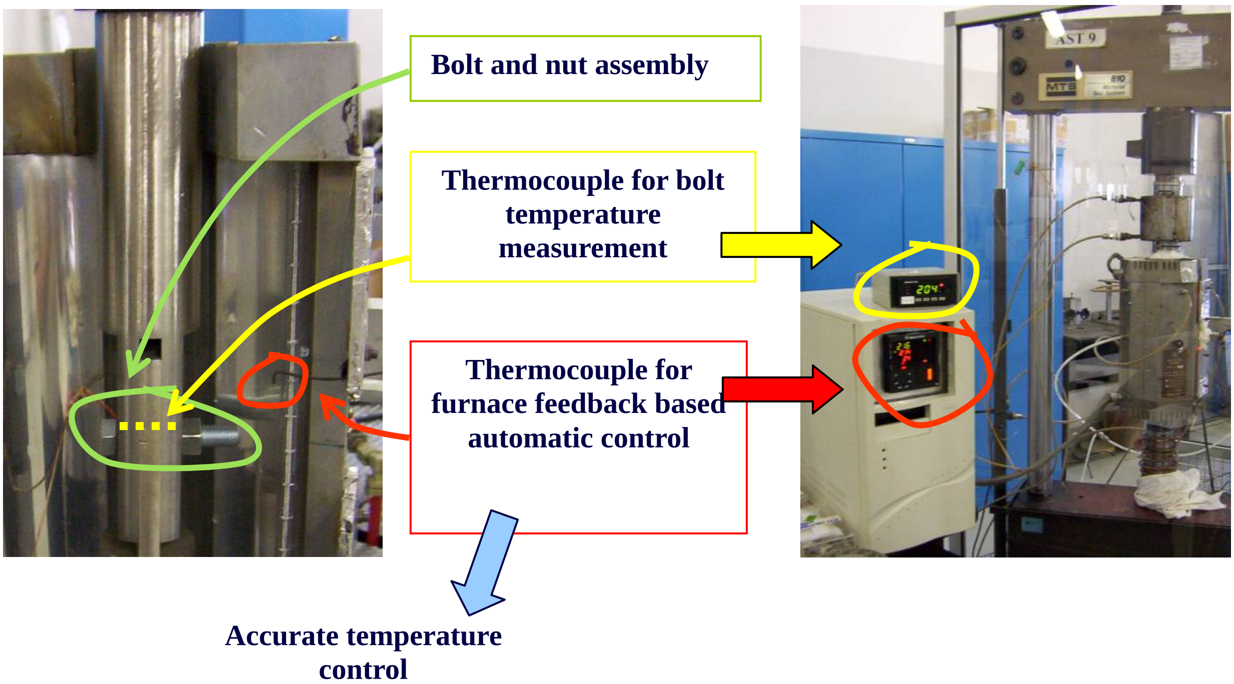



**Figure 7.7** *Bolt-nut assembly in the furnace (left) and high temperature testing appliance (right)*

Detailed test results at several temperatures are shown in Table 7.2 and Table 7.3, respectively for A2-70 and for A4-80 bolts. Tensile tests up to temperatures of 300 to 400°C highlighted ambiguous behaviour of the bolts: failure sometimes occurred in the shank rather than in the thread, even if the shank had a greater cross-section than the thread. Stainless steel is far more sensitive to cold working than carbon steel and this leads to the cold worked threads having increased mechanical resistance. The results can be understood by considering 300 to 400°C as a transition temperature range, above which cold working effects on stainless steel are overtaken by the effect of temperature related grain growth. Figure 7.8 shows A4 80 bolts after failure at 300°C, in the two cases of shank and thread failure.

**Table 7.2** *Detailed results of shear and tensile tests carried out on A2-70 bolt-nut assemblies*

| | Test temperature | Shear – $P_{max}$ [kN] | Tensile | |
|---|---|---|---|---|
| | | | $P_{max}$ [kN] | Failure mode |
| A2-70 | RT | 117.3 | 83.0 | thread |
| | 200 | 84.0 | 67.2 | shank |
| | 300 | 79.5 | 64.5 | shank |
| | | 80.0 | 63.3 | thread stripped |
| | 400 | 73.8 | 61.6 | shank |
| | 500 | 68.1 | 58.1 | thread |
| | 600 | 56.7 | 49.0 | thread |
| | | 57.2 | 47.0 | thread |
| | 800 | 23.6 | 17.5 | thread |
| | 900 | 12.9 | 5.6 | thread |

**Table 7.3** *Detailed results of shear and tensile tests carried out on A4-80 bolt-nut assemblies*

| | Test temperature | Shear – $P_{max}$ [kN] | Tensile | |
|---|---|---|---|---|
| | | | $P_{max}$ [kN] | Failure mode |
| A4-80 | RT | 105.9 | 73.6 | thread stripped |
| | | | 77.6 | thread |
| | 200 | 75.6 | 52.4 | thread stripped |
| | 300 | 75.2 | 60.0 | shank |
| | | 71.8 | 60.0 | thread |
| | 400 | 71.2 | 58.5 | thread |
| | 500 | 64.5 | 56.7 | thread |
| | 600 | 56.6 | 49.3 | thread |
| | | 57.6 | 49.5 | thread |
| | 800 | 24.4 | 13.3 | thread |
| | 900 | 13.8 | 6.3 | thread |

**Figure 7.8** *A4-80 bolts different failure modes at T = 300 °C*
*Left: Shank failure, Right: Thread failure*

# 7.3 Design guidance

## 7.3.1 Welded connections

EN 1993-1-4 refers to EN 1993-1-8[21] for the design of welds. The only specific information given is that $\beta_w$ for fillet welds should be taken as 1.0 unless a lower value is justified by tests. A consequence of the reference to EN 1993-1-8 is that the filler metal should have a strength at least equal to that of the base material.

For the fire design of stainless steel structures, EN 1993-1-4 refers to Annex C of EN 1993-1-2 for properties of the base material at elevated temperatures. No information is given about the filler metal. EN 1993-1-2 gives, however, some design rules for welds in carbon steel. For butt welds it is stated that the same strength as the base material may be assumed up to 700°C and for higher temperatures that the strength reduction factors for fillet welds given in Annex D of EN 1993-1-2 may be applied. These factors are lower than those for the base material. No guidance is given for stainless steel welds at elevated temperatures because of a lack of information. The recommendations given below are applicable to annealed grades; cold worked grades are not covered.

### *Butt welds*

Butt welds made with filler metal at least matching the base material are considered as full strength at room temperature, which means that they are at least as strong as the base material. This means that no design calculations for the weld strength are needed. The tests showed that this also is true at elevated temperatures up to 1000°C for two austenitic stainless steel grades, 1.4318 and 1.4571 combined with filler metal G 19 9 L, X2CrNiN18-7 and G19 12 3 Nb, X6CrNiMoTi17-12-2, respectively. This is a positive sign compared to the rules in Annex D of EN 1993-1-2 for carbon steel. Generalizing these results requires information that the high temperature properties of the filler metal are similar to those of the base material. However, this is not easy to do as different stainless alloys have different strength reductions at elevated temperatures.

### *Fillet welds*

Fillet weld and partial penetration welds are designed according to clause 4.5.3 of EN 1993-1-8 with $\beta_w$ =1.0 for stainless steel. This may be conservative in the case of a filler metal substantially stronger than the base material. For fire design the strength reduction factors in Annex D of EN 1993-1-2 for carbon steel may be used. This is likely to be very conservative but no better information is available.

A particular problem should be noted when fillet welds are designed for calculated forces that are smaller than the member resistance (partial strength connections). For restrained members, the dilatation caused by heating may cause big forces, which have to be considered for the design of connections. Alternatively, full strength connections should be used for such members.

### 7.3.2 Bolted connections

The high temperature tests highlight that stainless steel acts better than carbon steel at high temperatures beyond 400 to 450°C; this is really the more interesting range when studying fire effects. Omitting A4-80 grade thread stripping at 200°C, this grade performs slightly better than A2-70. It is observed that stainless steel bolts loaded in tension can fail either in the shank or in the threaded area at low temperatures (up to 300 to 400°C); this is probably due to the increased resistance of the threaded part after cold working.

Based on the test results, strength retention factors have been derived. The proposed values are minimum values, omitting the result for A4-80 tensile test at 200°C with thread stripping (see Table 7.4: bold fonts identify results chosen as suggested values). Figure 7.9 and Figure 7.10 show the strength factors together with the ones give in EN 1993-1-2 for carbon steel bolts, compared with the tensile and shear tests results, respectively. The slightly higher behaviour of grade A4-80 contributes to an extra safety margin.

**Table 7.4** *Suggested values for stainless steel strength reduction factors, related to experimental tests results*

| Test temperature °C | A2 70 | | A4 80 | | Suggested $K_{b,\theta}$ |
|---|---|---|---|---|---|
| | $K_{b,\theta\ shear}$ | $K_{b,\theta\ tensile}$ | $K_{b,\theta\ shear}$ | $K_{b,\theta\ tensile}$ | |
| RT | **1.00** | **1.00** | **1.00** | **1.00** | 1.00 |
| 200 | 0.72 | 0.81 | **0.71** | 0.69 | 0.71 |
| 300 | **0.68** | 0.77 | 0.69 | 0.79 | 0.68 |
| 400 | **0.63** | 0.74 | 0.67 | 0.77 | 0.63 |
| 500 | **0.58** | 0.70 | 0.61 | 0.75 | 0.58 |
| 600 | **0.49** | 0.58 | 0.54 | 0.65 | 0.49 |
| 800 | 0.20 | 0.21 | 0.23 | **0.18** | 0.18 |
| 900 | 0.11 | **0.07** | 0.13 | 0.08 | 0.07 |

## 7.4 Conclusions

Steady state (isothermal) tests were carried out on butt welded joints in grades 1.4318 and 1.4571 austenitic stainless steel. The strength retention factors for the butt welded joints for both the stainless steel grades were compared to factors for the base material given in the *Design Manual for Structural Stainless Steel*. It was concluded from the test results that the design strength of a full penetration butt weld, for temperatures up to 1000°C, could be taken as equal to the strength of the base material for grades 1.4318 and 1.4571 in the annealed condition. More tests need be carried out on other steel grades in order to verify the values of the mechanical properties of butt welded joints. Further studies are also necessary to verify the test results for cold-worked grade CP350 and C700 and higher strength levels.

Over forty isothermal tests from room temperature up to 900°C were performed on bolt assemblies in tension and shear; two grades of bolt were tested, A2-70 and A4-80. The tests showed that stainless steel bolts behave better than carbon steel at high temperatures beyond 400 to 450°C. Grade A4-80 bolts retain their strength slightly better than grade A2-70. Based on the test results, strength retention factors were derived.

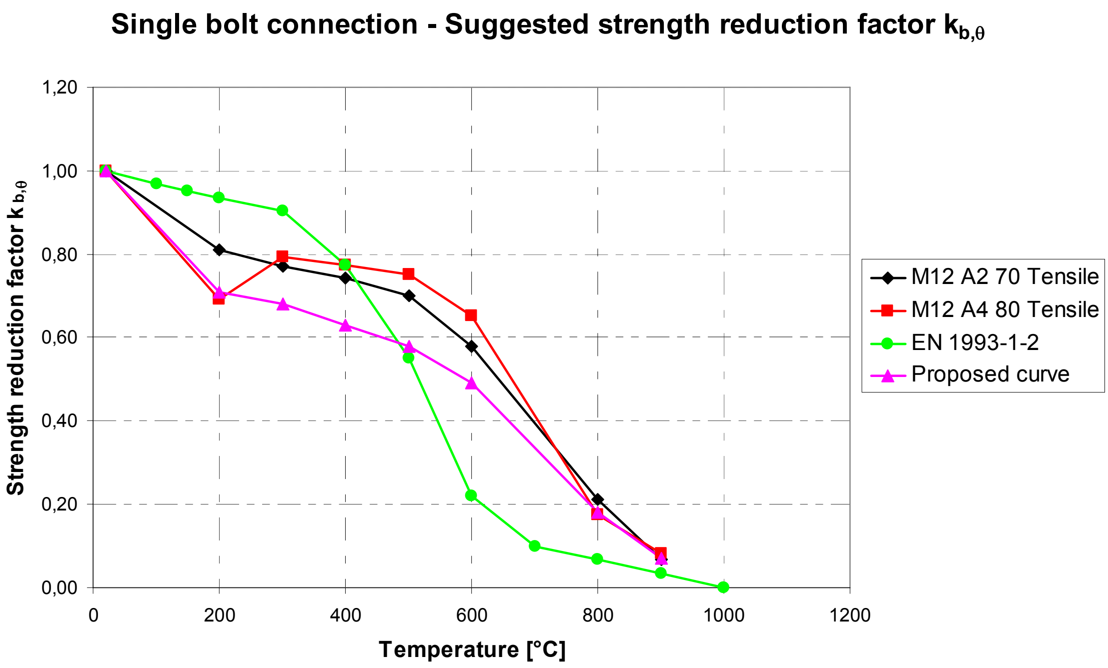



**Figure 7.9** *Single bolt connection proposed strength reduction factor $k_{b,\theta}$ and its comparison with EN 1993-1-2 standard and tensile experimental tests results.*

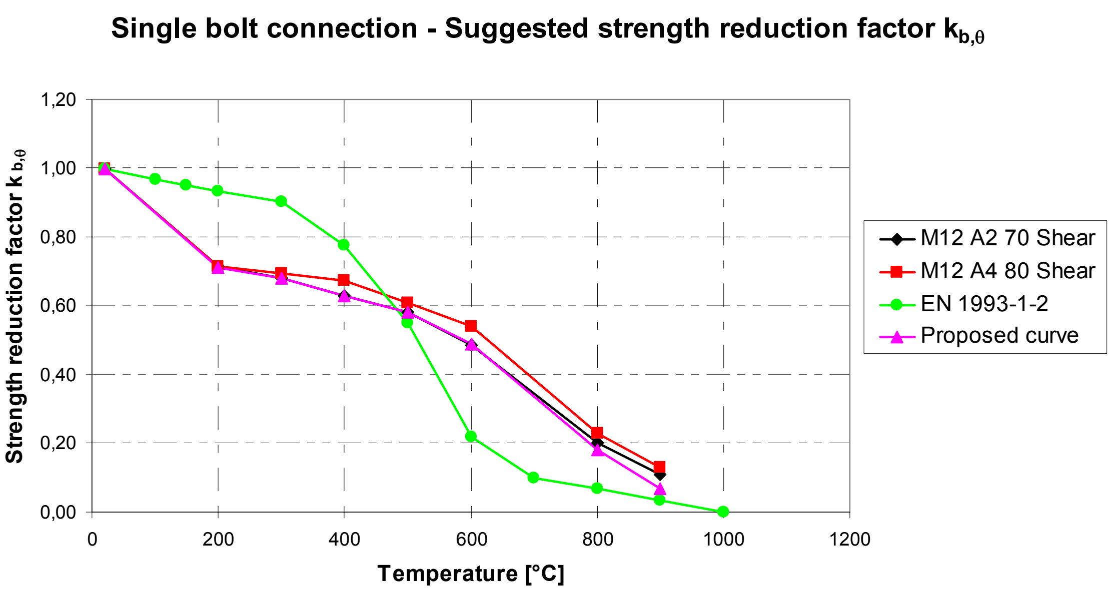



**Figure 7.10** *Single bolt connection proposed strength reduction factor $k_{b,\theta}$ and its comparison with EN 1993-1-2 standard and shear experimental tests results*

# 8 WP6: PARAMETRIC FIRE DESIGN

Detailed descriptions of the activities carried out under this work package are given in the relevant Final Work Package Report listed in Section 11.

## 8.1 Objectives

The purpose of this work package was to analyse the behaviour of stainless steel in fire in the following two applications:

- Exposed stainless steel columns located outside buildings
- Unprotected stainless steel columns in open car parks.

The aim is to show that unprotected stainless steel is a feasible alternative to protected carbon steel and to develop design guidance for the fire situation.

## 8.2 External structures

### 8.2.1 Numerical analysis

ANSYS was used to investigate the behaviour of external stainless steel columns in a fire. The building configuration analysed is presented in Figure 8.1.

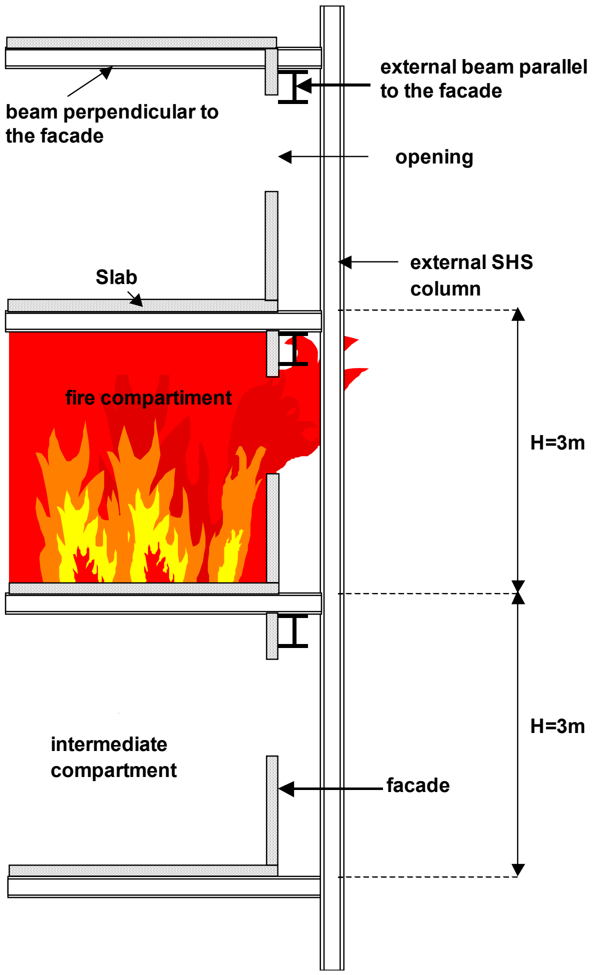



**Figure 8.1** *Details of external columns investigated*

The column length was taken as 9 m. Beams were not modelled but connections were represented by appropriate boundary conditions restraining lateral displacements and rotations at the positions of beams. Columns were pinned at the ends. A vertical load was applied at the top of the column and was kept constant during fire exposure. A geometrical imperfection of L/100 was considered.

Mechanical material properties of the column as a function of temperature were taken in accordance with EN 1993-1-2. At room temperature: $f_y = 240$ N/mm$^2$, $f_u = 530$ N/mm$^2$, $E = 200000$ N/mm$^2$, $\upsilon = 0.3$

A temperature gradient was introduced along the column length. The heating of external stainless steel members was calculated using a 2D simple calculation model developed in a previous ECSC project[22] which gives a good estimate of transient heat fluxes to carbon steel external members during a fire. The simple method was validated against results obtained from advanced numerical models. A validation case study was also carried out by comparison with results of a test performed by CTICM. Full details are given in the Final Work Package Report.

The model predicts thermal actions in three stages:

- Zone model for the compartment fire
- External flames model
- Thermal actions calculation

The performance of the external stainless steel columns was assessed by varying the main parameters that affect the fire severity:

- Compartment sizes
- Thermal properties of the wall
- Location of external member
- Total area of windows
- Fire load density
- External members
- Load level of column

The results of this study show that very strong thermal gradients occurred both along the member and across the cross-section. Across the section of the column, temperature differences can easily reach 500°C between the exposed and unexposed sides of the column. Along the length of the column, the thermal gradient can be about 500°C/m. Table 8.1 shows some of the more severe cross-sectional temperatures obtained at failure of the cases investigated. It can be seen that stainless steel columns have a mean failure temperature higher than 550°C and the column failure temperature increases as the load levels decrease. It is clear that unprotected external stainless steel columns perform adequately at load levels lower than 0.5. Table 8.2 shows equivalent results using S235 carbon steel column. It can be seen that some carbon steel columns fail before 30 minutes fire exposure, whereas the stainless steel columns remained stable during the whole fire duration.

**Table 8.1** *Temperature field at failure of external columns with stainless steel engulfed in fire (°C)*

| Cross-section | | Compartment Size: 3×4m (fire load density 1500MJ and 50% of windows opening) | | | Compartment Size: 9×8m (Fire load density 1500MJ and 50% of windows opening) | | |
|---|---|---|---|---|---|---|---|
| | | $\eta_{fi,t}$=0.3 | $\eta_{fi,t}$=0.5 | $\eta_{fi,t}$=0.7 | $\eta_{fi,t}$=0.3 | $\eta_{fi,t}$=0.5 | $\eta_{fi,t}$=0.7 |
| RHS 150x8 | $T_{max}$ (exposed side) | N.F* | 980 | 870 | N.F* | 921 | 813 |
| | $T_{int}$(intermediate side) | N.F* | 738 | 566 | N.F* | 783 | 690 |
| | $T_{min}$ (unexposed side) | N.F* | 534 | 234 | N.F* | 361 | 214 |
| | $T_{mean} = (T_{max}+2T_{int}+T_{min})/4$ | - | 748 | 559 | - | 712 | 602 |
| | Failure time (min) | - | **38.2** | **15.3** | - | **20.4** | **14.7** |
| RHS 300x8 | $T_{max}$ (exposed side) | N.F* | N.F* | 883 | N.F* | 945 | 833 |
| | $T_{int}$ (intermediate side) | N.F* | N.F* | 576 | N.F* | 802 | 696 |
| | $T_{min}$ (unexposed side) | N.F* | N.F* | 242 | N.F* | 392 | 208 |
| | $T_{mean} = (T_{max}+2T_{int}+T_{min})/4$ | - | - | 571 | - | 735 | 596 |
| | Failure time (min) | - | **-** | **15.8** | - | **22.4** | **15** |

* Column remains stable during all fire exposure
$\eta_{fi,t}$ is the load level under the fire situation

**Table 8.2** *Temperature field at failure of external columns with carbon steel engulfed in fire (°C)*

| Cross-section | | Compartment Size: 3×4m (fire load density 1500MJ and 50% of windows opening) | | | Compartment Size: 9×8m (Fire load density 1500MJ and 50% of windows opening) | | |
|---|---|---|---|---|---|---|---|
| | | $\eta_{fi,t}$=0.3 | $\eta_{fi,t}$=0.5 | $\eta_{fi,t}$=0.7 | $\eta_{fi,t}$=0.3 | $\eta_{fi,t}$=0.5 | $\eta_{fi,t}$=0.7 |
| RHS 150x8 | $T_{max}$ (exposed side) | 908 | 602 | 467 | 877 | 686 | 538 |
| | $T_{int}$ (intermediate side) | 768 | 601 | 514 | 709 | 586 | 503 |
| | $T_{min}$ (unexposed side) | 451 | 115 | 75 | 320 | 177 | 125 |
| | $T_{mean} =(T_{max}+2T_{int}+T_{min})/4$ | 699 | 480 | 392 | 654 | 509 | 417 |
| | Failure time (min) | 25.1 | 9.9 | 7.8 | **11.2** | **8.7** | **7.6** |
| RHS 300x8 | $T_{max}$ (exposed side) | 794 | 593 | 423 | 862 | 600 | 449 |
| | $T_{int}$ (intermediate side) | 750 | 583 | 481 | 700 | 539 | 448 |
| | $T_{min}$ (unexposed side) | 390 | 106 | 63 | 293 | 132 | 91 |
| | $T_{mean} = (T_{max}+2T_{int}+T_{min})/4$ | 670 | 471 | 362 | 623 | 434 | 339 |
| | Failure time (min) | 20.1 | 8.7 | 6.6 | **10.8** | **8.0** | **7.0** |

* Column remains stable during all fire exposure
$\eta_{fi,t}$ is the load level under the fire situation

### 8.2.2 Development of design guidance

The buckling resistance of external stainless steel columns under axial compression (Class of cross section ≤ 3) in fire can be obtained from:

$$N_{\mathrm{b,fi,t,Rd}} = \chi_{\mathrm{fi}}(\bar{\lambda}_{\theta}).\sum_{\mathrm{i=1}}^{4} A_{\mathrm{i}}.f_{2,\theta\mathrm{i}} / \gamma_{\mathrm{Mfi}} \tag{8.1}$$

where:

| | | |
|---|---|---|
| $A_{\mathrm{i}}$ | is | the area of plane element i defining the hollow cross-section (exposed side, lateral side or unexposed side) |
| $\theta_{\mathrm{i}}$ | is | the temperature of plane element i calculated from the simplified 2D heat transfer model developed for hollow steel section |
| $f_{2,\theta\mathrm{i}}$ | is | the 2% proof characteristic strength at temperature $\theta_{\mathrm{i}}$ |
| $\gamma_{\mathrm{M,fi}}$ | is | the partial factor for the fire situation |
| $\bar{\lambda}_{\theta}$ | is | the non dimensional slenderness at elevated temperature $\theta$ |
| $\chi_{\mathrm{fi}}$ | is | the reduction factor for flexural buckling in the fire design situation obtained from an appropriate buckling curve and depending on the non-dimensional slenderness |

The reduction factor $\chi_{\mathrm{fi}}$ for buckling resistance in the fire design situation is determined according to EN 1993-1-2, clause 4.2.3.2.

The non dimensional slenderness $\bar{\lambda}_{\theta}$ at temperature $\theta$ is given by: $\bar{\lambda}_{\theta} = \sqrt{\dfrac{\sum_{\mathrm{i}} A_{\mathrm{i}}\, f_{\mathrm{y},\theta\mathrm{i}}}{N_{\mathrm{fi,cr}}}}$ (8.2)

where:

$N_{\mathrm{fi,cr}}$ is the Euler elastic critical load obtained as follows: $N_{\mathrm{fi,cr}} = \dfrac{\pi^2 (EI)_{\mathrm{eff}}}{L_{\mathrm{e}}^{2}}$

$(EI)_{\mathrm{fi,eff}}$ is the effective stiffness: $(EI)_{\mathrm{fi,eff}} = \sum_{\mathrm{i}} E_{\mathrm{i},\theta} I_{\mathrm{i}}$

$E_{\mathrm{i},\theta}$ is the modulus of elasticity of plate element *i* at the appropriate elevated temperature $\theta_{\mathrm{i}}$

$I_{\mathrm{i}}$ is the second moment of area of plate element *i*

$L_{\theta}$ is an equivalent buckling length in fire situation taking into account effects of thermal gradient and column continuity on the fire resistance of the column:

Intermediate storeys $L_{\theta} = L - h_{\mathrm{s}}$

Top storeys $L_{\theta} = L$

In which $h_{\mathrm{s}}$ is the height of sill and $L$ is the system length in the relevant storey

The external column must satisfy the following condition:

$$N_{\mathrm{fi,Ed}} \leq N_{\mathrm{fi,Rd}} \tag{8.3}$$

where:

$N_{\mathrm{fi,Ed}}$ is the design value of the axial compression for the combination of actions considered in the fire situation according to EN 1991-1-2.

To show the accuracy of this simple calculation method, a comparison was carried out between the critical temperatures obtained with the proposed design method and those obtained with the numerical analyses (Figure 8.2). The difference between the values does not exceed 10%, although some points are on the unsafe side. It is concluded that the proposed simple calculation rules are suitable for predicting the fire resistance of external stainless steel members under axial compression to an acceptable degree of accuracy.

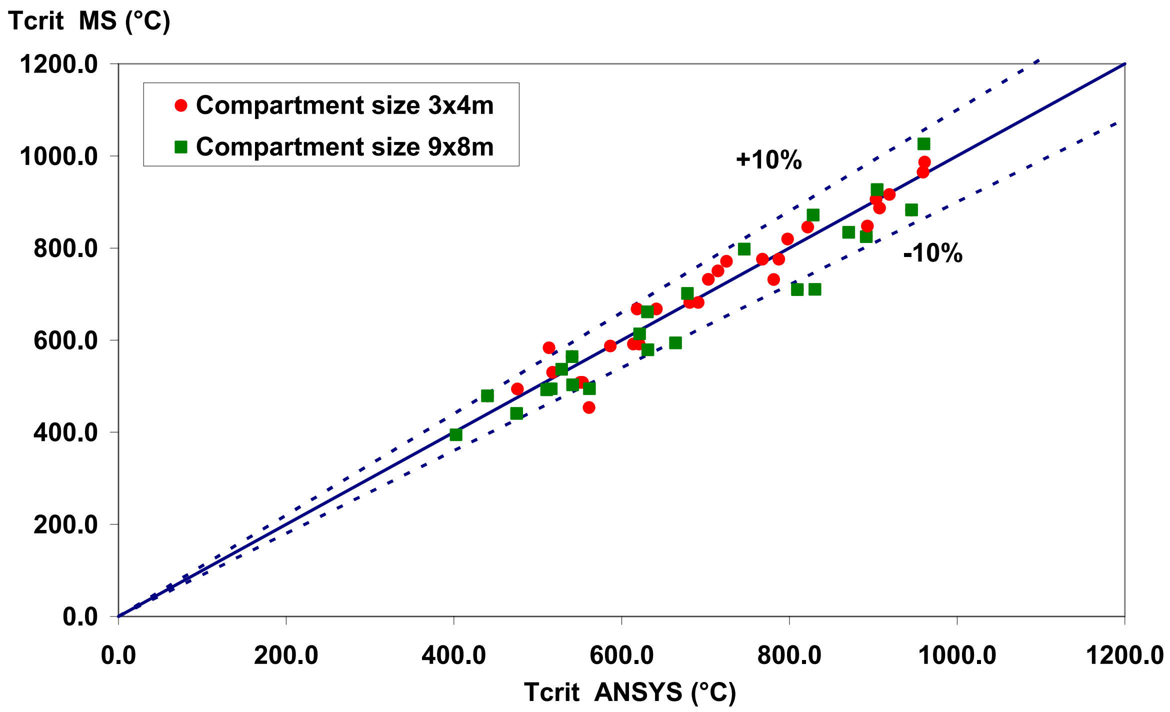



**Figure 8.2** *Comparison of critical temperatures calculated using simplified method ($T_{crit}$ MS) and numerical model ($T_{crit}$ ANSYS)*

# 8.3 Car park buildings

The behaviour of stainless steel columns in open car parks of steel and concrete composite construction was studied using a fire safety engineering procedure developed in France and validated against experimental results. Numerical investigations enabled the maximum load level for unprotected stainless steel hollow columns to be determined.

## 8.3.1 Numerical analysis

***Fire engineering procedure for open car parks***

The behaviour of open car parks in fire has been investigated by a fire engineering procedure based on the natural fire concept which involves the following stages:

- The most unfavourable fire scenario with respect to the fire stability of the structure and car park arrangement (number of cars involved in the fire and their positions) is determined. For the purpose of this project a single fire scenario was considered consisting of a column fully engulfed in a fire with four vehicles around. The fire starts in one of the four vehicles and spreads to the other three vehicles (scenario 2 in Figure 8.3).
- The thermal actions applied to different structural members are evaluated as a function of time according to the fire scenario based on the heat release rate of the vehicles involved as well as the propagation of fire between them. For open car parks, structural members near the fire are generally subjected to heat flux derived from fire flames; structural members far from the fire will only be heated by a layer of hot gas. Therefore, generally the thermal action for structural members will be a combination of both actions.

- The heating of the structural members is estimated considering a temperature gradient on the cross section and along the length of the member using an advanced calculation model.
- The mechanical behaviour of the car park structure is determined based on a 3D numerical model with the predicted structural heating characteristics. This global analysis takes account of lateral buckling of steel beams, membrane and diaphragm effects on the floor and load redistributions from the heated part of the structure to the cold parts. Material mechanical properties at elevated temperatures recommended in EN 1994-1-2 have been used for steel and concrete. In addition, the concrete is considered as a strength irreversible material. The possible rupture of reinforcing steel due to large elongations or movement of vehicles has also been taken into consideration by modifying the initial loading of the floor when the deflection of the floor becomes too high. As a consequence, two special criteria have been systematically checked:
  - maximum mechanical strain of reinforcing steel not exceeding 5%
  - maximum deflection of floor not higher than 1/20 of secondary beam span

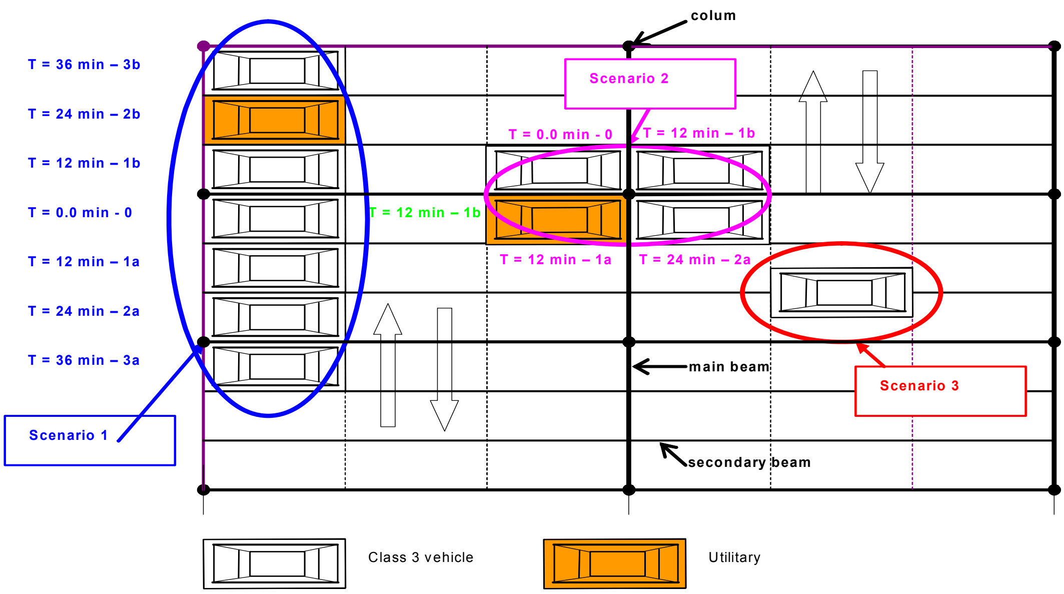



**Figure 8.3** *Basic fire scenario for open car parks*

### *Numerical analysis of open car parks*

Two different structural systems were investigated, whose characteristics are summarised in Table 8.3. The two systems consist of two storey structures, with steel beams, a composite floor system and steel columns protected by concrete. Before starting the analysis, preliminary calculations were carried out to find the smallest stainless steel CHS columns (leading to the maximum load ratio) to replace the hot rolled carbon steel columns given in Table 8.3. Firstly the stainless steel column with the same design buckling resistance at room temperature as the partially encased carbon steel section was determined. The load level of the carbon steel columns was 0.35. Then the temperature development in the stainless steel column (considering temperature gradient along the length of the member) was estimated and the buckling resistance of the column calculated as a function of time for the critical cross-sectional temperature previously obtained. These results give the maximum load level of the column (the ratio of the buckling resistance in fire over the ultimate load at room temperature design). The stainless steel column cross-section was designed so that the column load level becomes equal to the previously obtained value.

Stainless steel columns with the same buckling resistance at room temperature as carbon steel columns are given in Table 8.4. Buckling resistances of carbon steel columns at room temperature have been calculated according to EN 1993-1-1[23]. Buckling resistance of stainless steel columns have been calculated according to EN 1993-1-4 using the buckling parameters given for cold-formed sections (i.e. $\alpha = 0.49$ and $\overline{\lambda}_0 = 0.40$).

**Table 8.3** *Summary of systems studied*

| Framing system | | Case 1 | Case 2 |
|---|---|---|---|
| Level height (m) | First level | 4.10 | 3.10 |
| | Second level | 2.67 | 2.67 |
| Cross section of members (standard level) | Column | HEB 240 (S355) | HEA 340 (S355) |
| | Main beam | HEA 500 (S355) | IPE 400 (S355) |
| | Secondary beam | IPE 500 (S355) | IPE 240 (S355) |
| | Composite slab | Cofraplus60 | Cofraplus60 |
| Span of secondary beams (m) | | 15 | 7.5 |
| Spacing of secondary beams (m) | | 3.33 | 2.50 |
| Width of parking place (m) | | 2.5 | 2.5 |
| Number of vehicles between two successive secondary beams | | 1 | 1 |
| Spacing of columns (m) | | 10.0 | 7.50 |
| Span of main beams (m) | | 10.0 | 7.50 |
| Total depth of concrete slab (cm) | | 12 | 12 |

**Table 8.4** *Preliminary design of stainless steel column*

| Frame structure | Carbon steel column | | Stainless steel column | |
|---|---|---|---|---|
| | Cross section Length (m) | Buckling resistance at room temperature (kN) | Cross section | Buckling resistance at room temperature (kN) |
| Case 1 | HEB240 (S355) L = 4.1m | 2741 | CHS 323.9 x 12 mm | 2800 |
| Case 2 | HEB340 (S355) L = 2.67m | 5479 | CHS 610 x 12 mm | 5410 |

A comparison between the heating up characteristics of a stainless steel column filled with concrete and one not filled with concrete of identical geometry shows that after 30 minutes exposure to a natural fire, the protected column reaches 720°C and the unprotected column reaches 900°C.

The load level was defined as the ratio between the buckling resistance in fire (calculated with the following buckling parameters: $\alpha = 0.49$ and $\overline{\lambda}_0 = 0.40$) and the buckling resistance at room temperature. The resistances were calculated assuming no contribution of the concrete. For a fire duration of 30 minutes, the load ratio decreased during fire exposure, reaching minimum values of 0.45 and 0.2 for the concrete-filled and empty columns respectively. Stainless steel columns (filled with concrete) designed for a load level of 0.45 are reported in Table 8.5.

**Table 8.5** *Buckling resistance at room temperature*

| Framing system | Stainless steel column (protected with concrete) | |
|---|---|---|
| | Cross section | Buckling resistance at room temperature (kN) |
| Case 1 | CHS 273 × 12 mm | 2235 |
| Case 2 | CHS 407 × 14 mm | 4148 |

Numerical analyses were carried out for the fire scenario outlined in Cases 1 and 2. For Case 1 a grid of 3 x 3 was analysed and for Case 2 a grid of 4 x 4 (see Figure 8.4). The fire was considered to start on the lower floor of the car park and the upper level was represented in the model by the appropriate boundary conditions.

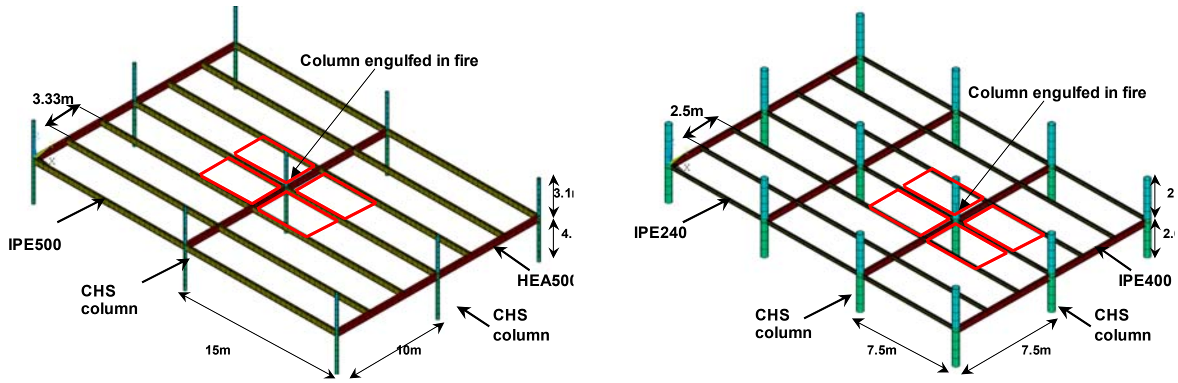



**Figure 8.4** *Structure of framing system, Left: Case 1, Right: Case 2*

The compressive resistance of concrete was taken as 30 MPa and the yield strength of the structural carbon steel grade S355 was 355 N/mm$^2$. The stainless steel was grade EN 1.4401/1.4404 with $f_y$ = 240 N/mm$^2$, $f_u$ = 530 N/mm$^2$, $E$ = 200,000 N/mm$^2$. The yield strength of the reinforcing steel was 500 MPa. The imposed load was 140 kg/m$^2$ plus the dead load. Full shear connection between steel beams and concrete slab was assumed. It was assumed that the concrete in the stainless steel columns did not contribute to the resistance of the column. The loads were uniformly distributed on the concrete slab and the resultants of the loads applied on the upper concrete slab were applied on the top of each column.

The following observations were noted with Case 1:

- The local temperature in the beams reaches over 900°C.
- The vertical deflection increases from 52 mm to 430 mm after 15 and 33 minutes of fire exposure. After 60 minutes the maximum heating phase of the fire had finished and the cooling phase had started. At this point the maximum deflection of the floor decreased to 395 mm.
- The area subject to deformation increased as the fire developed.
- The maximum deflection of the steel beams was approximately 412 mm for secondary beams and 318 mm for primary beams. This was below the failure criteria (span / 20).
- The maximum elongation of the reinforcing steel was 4.2%, which is below the limit of 5% established by EN 1992-1-2.

Frame system Case 1 with concrete filled stainless steel columns is a satisfactory structural solution for open car parks with regard to the fire scenario investigated.

The following observations were noted with Case 2:

- The maximum deflections of the steel beams were 127 mm for secondary beams and 44 mm for primary beams. These values were lower than the failure criteria (span/20).
- The maximum elongation of reinforcing steel did not exceed the failure criterion of 5%.

Frame system Case 2 with concrete-filled stainless steel columns is a suitable structural solution for open car parks with regard to the fire scenario investigated in this project.

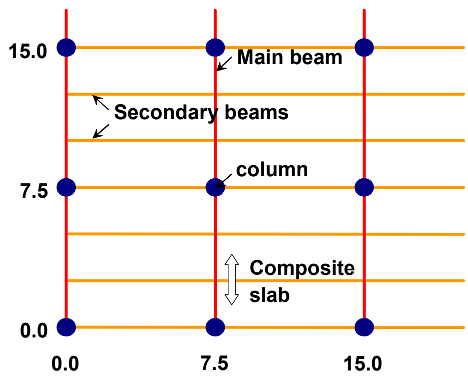
It can therefore be concluded that numerical analysis performed on specific framing systems for open car parks subject to natural fires has shown that a load ratio of 0.45 can be achieved with unprotected stainless steel columns in steel grade EN 1.4401/1.4404. This value is higher than the maximum load ratio achieved with partially encased carbon steel columns (0.35). The use of stainless steel enables reduced column cross-sections in comparison with a carbon steel column. However, stainless steel columns should be filled with concrete to limit their temperature rise and to ensure their stability during fire.

### 8.3.2 Development of design guidance

Design tables and construction details for using different carbon steel structural systems in open car parks are available in a practical design guide[24]. This study has enabled design tables for different structural systems for open car parks developed for carbon steel to be extended to cover stainless steel also. An example is shown in Figure 8.5.

## 8.4 Conclusions

Simple design guidance has been developed based on the results of numerical analyses and the fire design approach of EN 1993-1-2 for external stainless steel columns and stainless steel columns in open car parks. However no experimental investigation was carried out and it would be interesting to carry out some fire tests to produce evidence that the guidance developed is absolutely reliable.




Slab span: 2.5 m
Secondary beam span : 7.5 m
Main beam span: 7.5 m
Spacing of columns : 7.5 m

Applied loads (except self weight):
Standard level:
- Dead load: 0.20 kN/m²
- Imposed load: 2.50 kN/m²

Last level:
- Dead load: 1.45 kN/m²
- Imposed load: 2.50 kN/m²

Self-weight of facade: 7.5 kN/m²

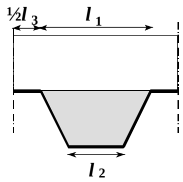
Orientation of parking place:
- Perpendicular to secondary beam

Net height beneath steel beam:2.1 m

| | | | |
|---|---|---|---|
| Minimum size of secondary beam cross section | Standard level | | IPE240 |
| | last level | | IPE270 |
| Minimum size of main beam cross | Standard level | | IPE400 |
| | last level | | IPE450 |
| Design of column cross-section | Carbon steel grade | Available of section type | HEA, HEB or HEM |
| | | Maximum load level (**) | 0.35 |
| | Stainless steel grade EN1.4404 | Available of section type | CHS or RHS |
| | | Maximum load level (**) | 0.45 |
| Requirement to be applied to concrete slab | Total depth of slab | | ≥ 120 mm and ≤ 140 mm |
| | Maximum height of steel deck | | 62 mm |
| | Minimum compactness of rib of steel deck (*) | | 0.393 |
| | Minimum thickness of steel sheet | | 0.75 mm |
| | Minimum mesh of reinforcing steel | | $\phi$7 150x150 mm |
| | Location of reinforcing steel mesh | | 30 mm from top of slab |

(*)compactness of rib of steel deck:

$$\frac{(\ell_1 + \ell_2)}{2(\ell_1 + \ell_3)}$$




(**) Load level: ratio of applied load under fire situation over ultimate load at room temperature design

**Figure 8.5** *Design guidance for open car parks with carbon steel and stainless steel columns.*

# 9 WP7: DESIGN AIDS AND SOFTWARE

## 9.1 Objectives

The objective of this work package is to prepare summary design recommendations which draw together the design guidance developed in Work Packages 1 to 6 and also to develop a web-based fire design software facility.

A summary of the design guidance recommendations is given at the end of the sections describing each Work Package. In the following sections, three activities are described which are independent of the preceding work packages.

## 9.2 Mechanical properties of stainless steel at elevated temperatures

Over the last 20 years, strength and stiffness retention factors have been derived from steady state (isothermal) and transient state (anisothermal) test data for a number of grades of stainless steel used in structural applications. It is generally accepted that the results of steady state tests are only accurate up to temperatures of about 400ºC; above this temperature they give unconservatively high results and data from transient state tests should be used which more closely replicate a real fire situation. Figure 9.1 shows the 0.2% proof strength retention curves for a number of austenitic grades, including two grades in the work hardened condition C850, one ferritic (1.4003) and one duplex (1.4462). Figure 9.2 shows the ultimate tensile strength retention curves for the same grades.

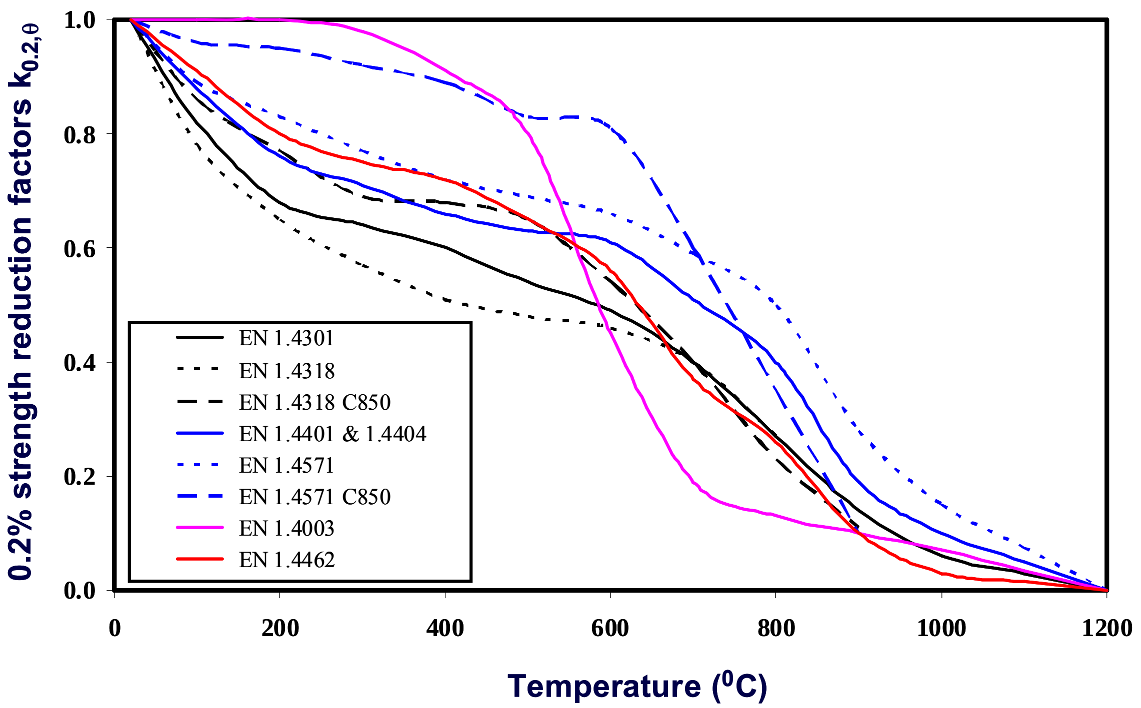



**Figure 9.1** *0.2% proof strength retention curves for stainless steels*

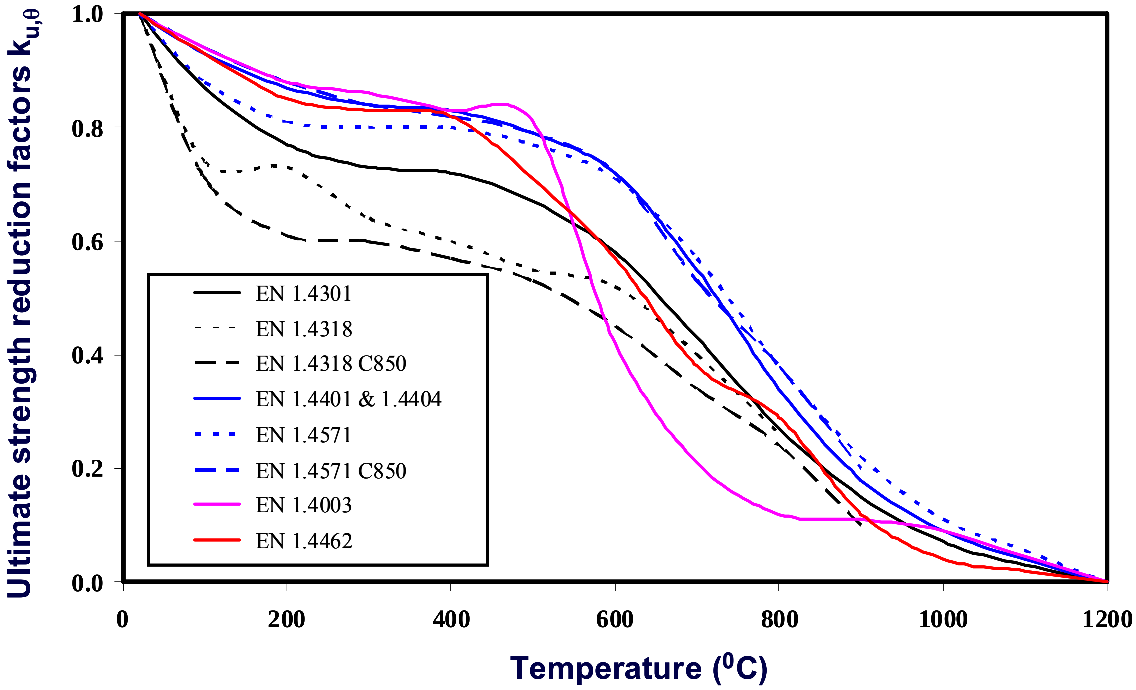



**Figure 9.2** *Ultimate tensile strength retention curves for stainless steels*

Having a unique set of retention factors for each different grade is awkward for designers and unjustified due to the high scatter in the test data for each grade. Bearing in mind that all structural carbon steels are currently described in EN 1993-1-2 by one set of strength retention curves, some preliminary work was carried out rationalising the stainless steel curves into a smaller number of generic curves. It was proposed that the following five generic curves should be developed:

- Chromium-nickel austenitic grades (e.g. 1.4301, 1.4318)
- Chromium-nickel-molybdenum austenitic grades (e.g. 1.4401, 1.4404)
- Stabilised austenitic grades (e.g. 1.4571, 1.4541)
- Ferritic grades (e.g. 1.4003)
- Duplex grades (e.g. 1.4462, 1.4362)

However, since this study was carried out, the test results from WP4 became available and indicated that grade 1.4541 exhibited much poorer performance than grade 1.4571. Outokumpu has commissioned its own test programme of transient state tests to investigate this further, outside this RFCS project. Outokumpu's study includes 1.4541 as well as a range of duplex grades. At the time of writing this Final Summary Report, the results of the tests had only just become available. New generic strength retention curves are under development.

## 9.3 Design of stainless steel beams and columns in fire

Early on in the Stainless Steel in Fire project, while the Third Edition of the *Design Manual for Structural Stainless Steel*[2] was being prepared, two new approaches to fire resistant design of stainless steel were compared to that in EN 1993-1-2 for carbon steel. The approaches are described below and Table 9.1 highlights the key differences between them.

### *Euro Inox Design Manual approach*

This approach is included in the Second Edition of the Euro Inox *Design Manual for Structural Stainless Steel* and is basically aligned with the approach in EN 1993-1-2 except it does not include the 0.85 factor into the expression for section classification. Compared to the room temperature design approach in EN 1993-1-4 and the Design Manual, for Class 1-3 cross-sections the Euro Inox method uses a lower buckling curve (the fire buckling curve derived from test data on carbon steel columns) and a higher (2%) material strength. For Class 4 cross-sections it uses the fire buckling curve with the 0.2% proof strength.

### *CTICM approach*

The CTICM approach, developed in an ECSC project studying cold worked austenitic stainless steel[5], uses the room temperature buckling curve and 0.2% proof strength for all cross-sections.

**Table 9.1** *Summary of differences between fire design approaches for stainless steel*

<table>
<tr><th>EN 1993-1-2</th><th>Euro Inox Design Manual (Second Ed)</th><th>CTICM method for cold worked stainless steel</th></tr>
<tr><td colspan="3">Section classification in fire – value of $\varepsilon$</td></tr>
<tr><td>$\varepsilon = 0.85\sqrt{\frac{235}{f_y}}$</td><td>$\varepsilon = \sqrt{\frac{235}{f_y}\frac{E}{210000}}$</td><td>$\varepsilon = \sqrt{\frac{235}{f_y}\frac{E}{210000}}$</td></tr>
<tr><td>Lower value of $\varepsilon$ leads to stricter classification limits, i.e. more sections become Class 4. Effective section properties also reduce as $\varepsilon$ is used in the calculation of effective widths.</td><td></td><td></td></tr>
<tr><td colspan="3">Buckling curve</td></tr>
<tr><td>'Fire' buckling curve<br>$\varphi_\theta = \frac{1}{2}\left(1+0.65\sqrt{\frac{235}{f_y}}\bar{\lambda}_\theta + \bar{\lambda}_\theta^2\right)$</td><td>'Fire' buckling curve<br>$\varphi_\theta = \frac{1}{2}\left(1+0.65\sqrt{\frac{235}{f_y}}\bar{\lambda}_\theta + \bar{\lambda}_\theta^2\right)$</td><td>'Room temperature' buckling curve<br>$\varphi_\theta = \frac{1}{2}\left(1+\alpha\left(\bar{\lambda}_\theta - \bar{\lambda}_0\right)+\bar{\lambda}_\theta^2\right)$<br>$\alpha$=0.49, $\bar{\lambda}_0$=0.4 for cold formed open sections & hollow sections<br>$\alpha$=0.76, $\bar{\lambda}_0$=0.2 for welded open sections (minor axis)</td></tr>
<tr><td colspan="2">'Fire' buckling curve lies below 'room temperature' buckling curve at all practical values of $\bar{\lambda}_\theta$ and $f_y$ for hollow sections.</td><td></td></tr>
<tr><td colspan="3">Material strength</td></tr>
<tr><td>Class 1-3 $f_{2,\theta}$<br>Class 4 $f_{0.2\%\mathrm{proof},\theta}$</td><td>Class 1-3 $f_{2,\theta}$<br>Class 4 $f_{0.2\%\mathrm{proof},\theta}$</td><td>Class 1-4 $f_{0.2\%\mathrm{proof},\theta}$</td></tr>
<tr><td colspan="2">$f_{2,\theta}$ is between 20 and 25% higher than $f_{0.2\%\mathrm{proof},\theta}$<br>The material strength is used in the calculation of $\bar{\lambda}_\theta$ and the buckling resistance at temperature $\theta$.</td><td></td></tr>
<tr><td colspan="3">SUMMARY</td></tr>
<tr><td>EN 1993-1-2 gives lowest critical temperatures/fire resistances</td><td></td><td>CTICM approach generally gives the highest critical temperatures/fire resistances</td></tr>
</table>

The factor 0.85 was introduced into the expression for ε used in section classification in EN 1993-1-2 because it is considered an average (for the relevant range of temperature) value of $\sqrt{\frac{k_E}{k_y}}$ for carbon steel (see Figure 9.3). Using an average value rather than calculating the stiffness to strength ratio at a given temperature simplified the calculations considerably as the Classes are not dependent on the temperature, which would make the calculations very difficult because a profile could be in Class 2 for some temperatures and in Class 3 for other ones. However, Figure 9.3 also shows the variation of $\sqrt{\frac{k_E}{k_y}}$ for stainless steel. Above temperatures of 200°C, this ratio rises from 1.0 to over 1.4. Applying a factor of 0.85 is therefore not appropriate for stainless steel.

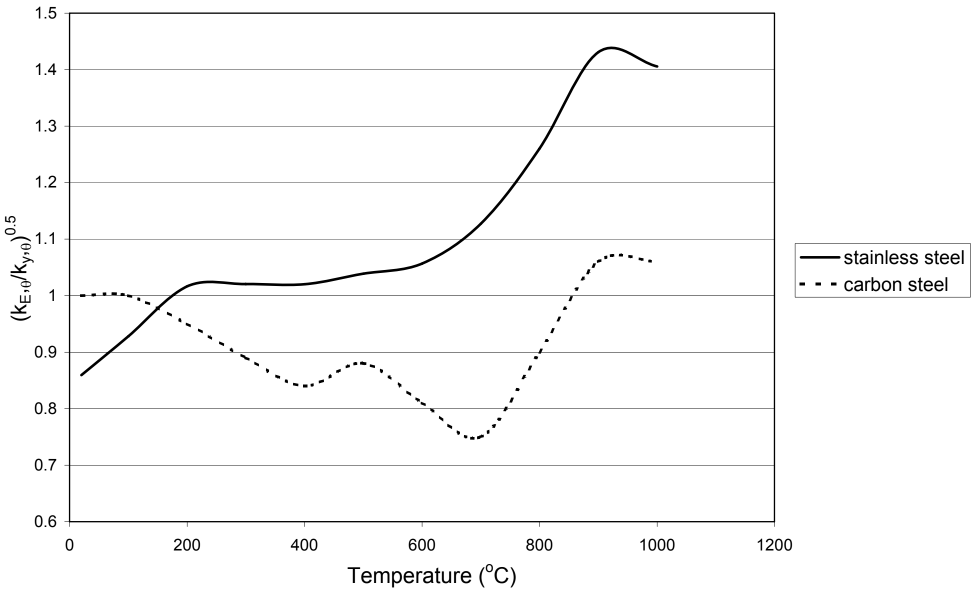



**Figure 9.3** *Variation of* $(k_E/k_y)^{0.5}$ *with temperature for carbon and stainless steel*

The fire resistances predicted by the two methods were compared against all available test data from stainless steel fire tests. (Appendix B gives a summary of column fire test data available at the time of carrying out this study in 2005.) Generally, the Euro Inox method gives slightly more conservative results than the CTICM method, although there is not a huge difference between the design curves (Figure 9.4). Figure 9.5 and Figure 9.6 show the relationship with temperature of the different material reduction factors and imperfection factors for grade EN 1.4301 adopted by the two methods. The figures show that these factors go some way to compensating for each other, thus explaining why there is no substantial difference between the two methods.

The Euro Inox method is a little more complicated because it involves a larger number of parameters: it needs the evaluation of the stress reduction factor at a total elongation (elastic and plastic) equal to 2% ($k_{2\%\theta}$) which implies the knowledge of the actual value of $f_u$, while the method proposed by CTICM does not and is independent of $f_u$.

Design curves in EN 1993-1-2 and EN 1994-1-2 were derived by the relevant Project Teams from a 'mean' assessment of the predictions against the test data points with no further reliability statistical analysis. Assuming a 'mean' assessment gives an acceptable level of safety, the CTICM approach gives an adequately safe prediction of the behaviour of stainless steel columns in fire. It was therefore decided to adopt this approach in the Third Edition of the *Design Manual for Structural Stainless Steel,* which was published in June 2006. The approach is summarised in Table 9.2. It represents advances in understanding of the behaviour of stainless steel members in fire and is less conservative than the approach in EN 1993-1-2.

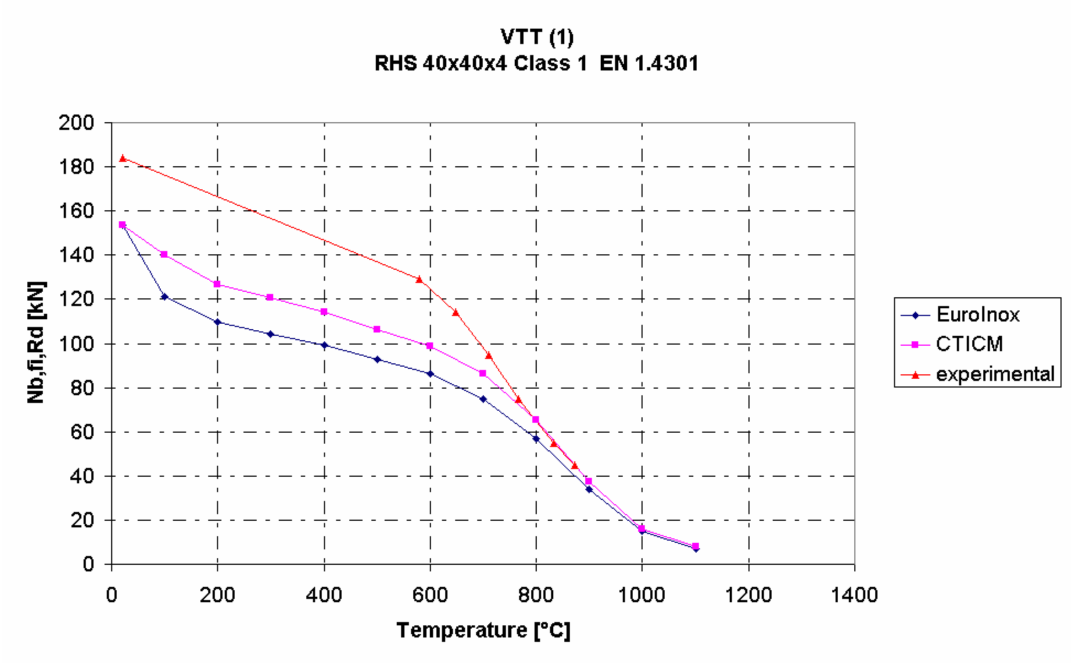



**Figure 9.4** *Column buckling tests at elevated temperature from VTT: comparison with design curves from Euro Inox and CTICM methods: grade 1.4301*

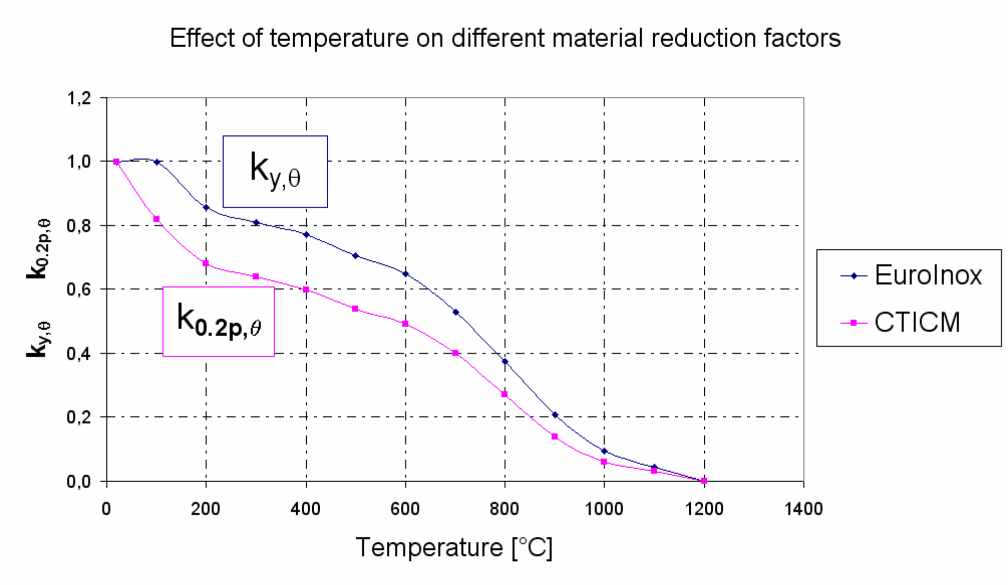



**Figure 9.5** *Grade EN 1.4301: comparison of material reduction factors for the two methods*

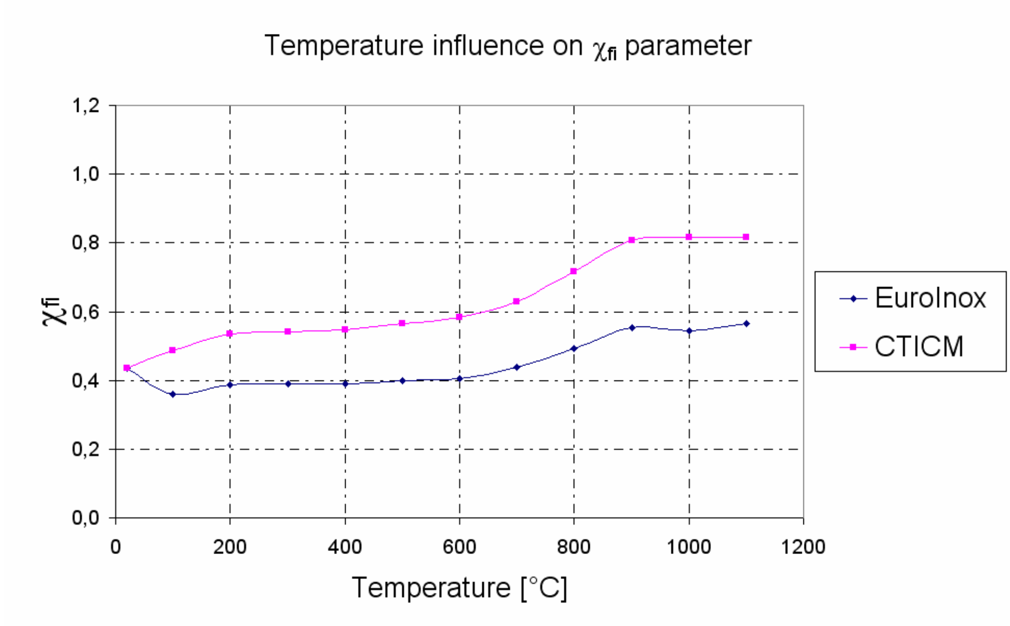



**Figure 9.6** *Grade EN 1.4301: comparison of* $\chi_{fi}$ *with temperature for the two methods*

**Table 9.2** *New approach for fire resistant design*

| Member | Strength and buckling curve for use in design |
|---|---|
| Columns | $f_{0,2proof,\theta}$ (all cross-section Classes) and the appropriate room temperature buckling curve |
| Restrained beams | $f_{2,\theta}$ (Class 1-3) and $f_{0,2proof,\theta}$ (Class 4) |
| Unrestrained beams | $f_{0,2proof,\theta}$ (all cross-section Classes) and the appropriate room temperature lateral torsional buckling curve |
| Tension members | $f_{2,\theta}$ (all cross-section Classes) |

## 9.4 Development of online software

Section 9.3 described the work carried out to develop a less conservative approach for determining the fire resistance of a stainless steel structural member which was subsequently published in the Third Edition of the *Design Manual for Structural Stainless Steel*[2].

Online software for designing stainless steel structural members at room temperature was developed during a previous Valorisation Project: Development of the Use of Stainless Steel in Construction (Contract No. 7215-PP-056), completed in 2003 and can be found at www.steel-stainless.org/software . The software was subsequently extended to cover cold worked stainless steel grades and fully aligned with EN 1993-1-4 in 2006. Under this project, the software was extended further to implement the fire design approach included in the Third Edition of the *Design Manual for Structural Stainless Steel*. The design software calculates the fire resistance of a structural member after certain time intervals, highlighting when the resistance of the member exceeds the presumed loading.

The main assumptions made by the software are as follows:

- There is no fire protection on the members
- There is a uniform temperature distribution across the cross-section and along the member (except for beams subject to bending with the upper flange of the beam protected)
- The temperature rise of the fire is defined by the standard time-temperature curve in EN 1363-1
- The heating-up rate is estimated from eq (7.36) in the Design Manual (eq (4.25) in EN 1993-1-2)
- There is no reduction of temperature due to the shadow effect
- The material strength and stiffness retention factors are taken from EN 1993-1-2 and the Design Manual

Firstly the user specifies the type of loading and the cross-section of the member being designed; from this information, the software calculates the section factor (the exposed surface divided by the cross-sectional area of the member). The temperature the section will reach after a series of time periods exposed to the standard fire curve is then calculated, conservatively assuming the resultant emissivity of stainless steel is 0.4. At each of these temperatures, the fire resistance is then calculated for the load type specified and compared to the assumed loading at the fire limit state. Figure 9.7 shows the web page where the user defines the type of loading and the web page where the results are given.

On the first input page, the user has the opportunity to specify the value of the reduction factor, $\eta_{fi}$, which is a factor used to calculate the loading at the fire limit state from the loading at the ultimate limit state. The software default value is 0.65, which is recommended in EN 1993-1-2 clause 2.4.2(3) except for imposed loads according to load category E (areas susceptible to accumulation of goods, including access areas).

It is assumed that loading at the fire limit state is equal to the normal temperature resistance multiplied by the utilization factor. Therefore, failure in the fire situation occurs when:

$$R_{fi,d,t} = \eta_{fi}\, E_d = \eta_{fi}\, \nu\, R_d \tag{9.1}$$

where:

| | |
|---|---|
| $R_d$ | is the design value of the member resistance at normal temperature, for a fundamental combination of actions from Equation 6.10 in EN 1990 |
| $\eta_{fi}$ | is the reduction factor for the design load level in the fire situation |
| $\nu$ | is the utilization factor |

The utilization factor is the ratio of the design effect at normal temperature to the corresponding design resistance. The software only uses it for the fire resistance calculations which assume that the loading at the fire limit state is equal to the resistance at normal temperature multiplied by the utilization factor. If the section is not fully utilized at room temperature (i.e. $\nu = E_d / R_d$ is less than 1.0), then the loading at the fire limit state can be reduced by entering the normal temperature utilization factor. This will result in a longer period of fire resistance (i.e. the member will be able to resist the load for a longer time). The default value for the utilization factor is 1.0.

For beams, the software offers a choice: if the user defines the upper flange of the beam as `protected`, then the section factor is calculated on the basis that only three sides of the member are exposed to the fire. In all other cases, the section factor is calculated on the basis that all four sides of the member are exposed to the fire. A lower section factor will lead to lower temperatures in the cross-section which leads to higher strength and longer period of fire resistance.

The fire resistances are expressed in terms of a ratio which shows the utilization of the section in fire after a given time. The ratio is the resistance of the member divided by the loading at the fire limit state. If the ratio is less then 1.0 then the member will fail and the ratio is displayed in orange.

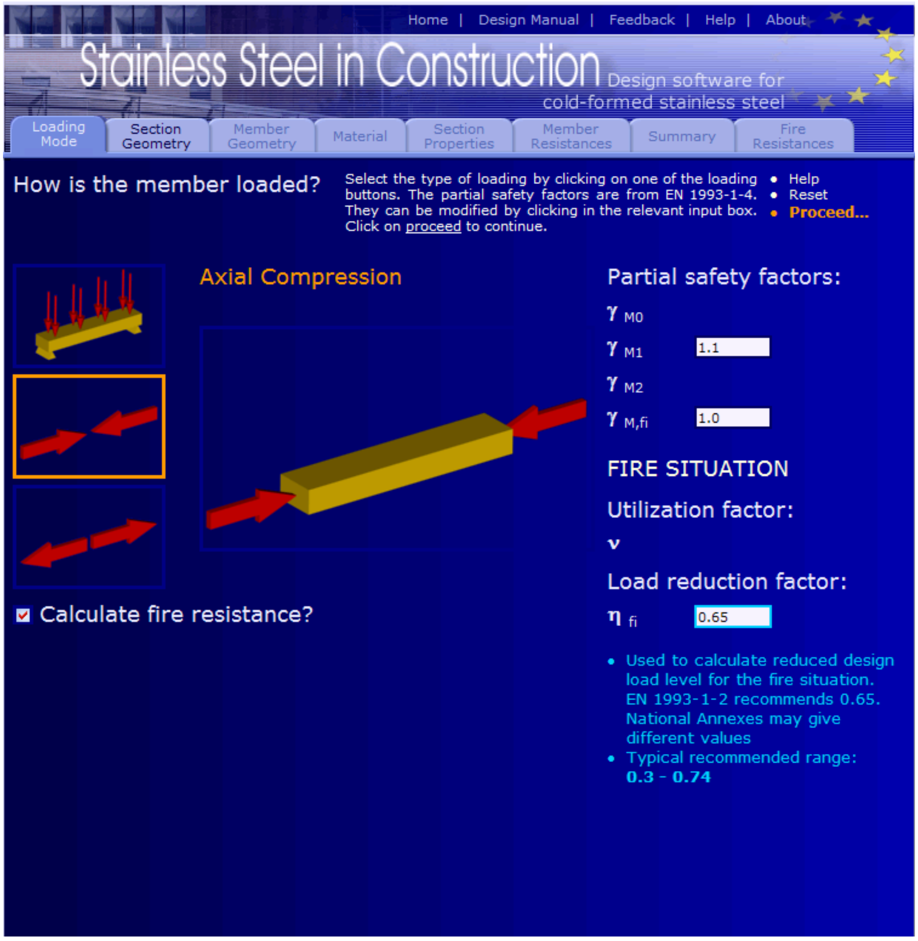
The software gives the following recommendations on how to improve the fire resistance of a section:

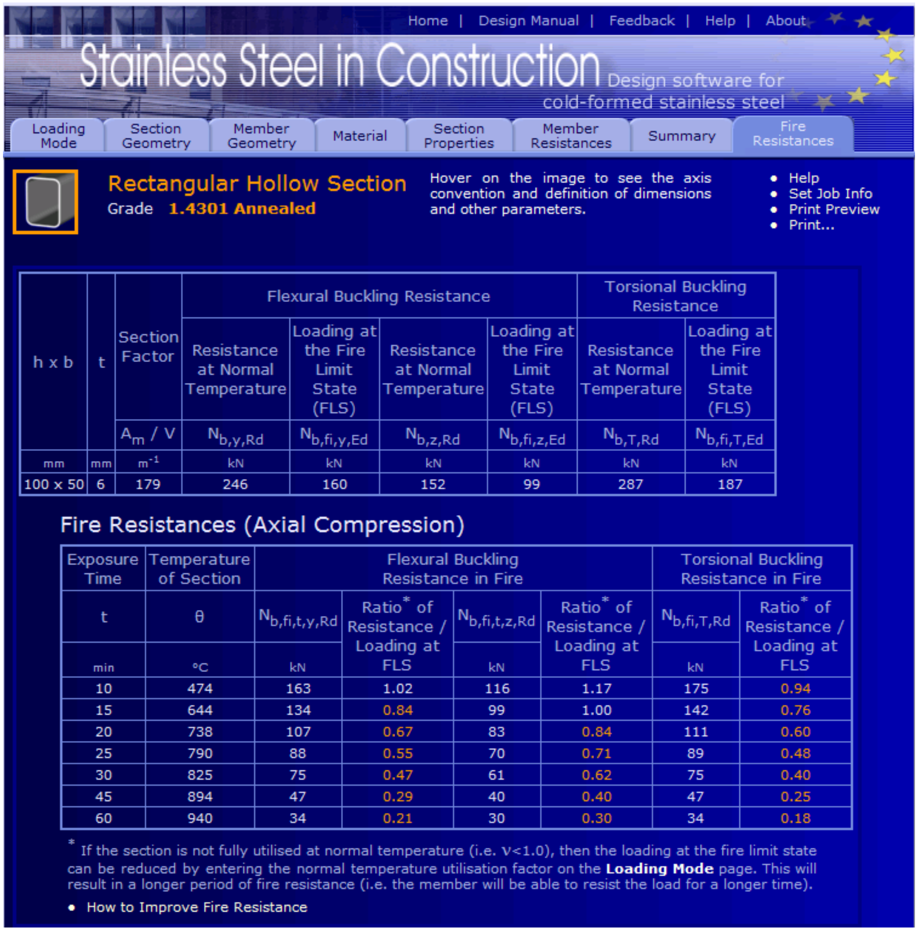
- increasing the thickness of the section
- increasing the size of the section
- using a section with a bigger section factor
- entering a utilisation factor (located on the loading mode tab).

Design software is very important for stainless steel sections because design tables are not readily available as there is no standard family of structural section sizes. A further advantages of this software is that it enables the effect of varying parameters such as the wall thickness, section size and utilisation factor to be quickly determined.

The software includes an online detailed contextual and conceptual help system.

The prototype fire package was demonstrated to partners at a project meeting and modifications were made in response to their comments. It was then subject to SCI Quality Assurance procedures for software prior to being moved into the public domain.




| h x b | t | Section Factor | Flexural Buckling Resistance | | | | Torsional Buckling Resistance | |
|---|---|---|---|---|---|---|---|---|
| | | | Resistance at Normal Temperature | Loading at the Fire Limit State (FLS) | Resistance at Normal Temperature | Loading at the Fire Limit State (FLS) | Resistance at Normal Temperature | Loading at the Fire Limit State (FLS) |
| | | $A_m / V$ | $N_{b,y,Rd}$ | $N_{b,fi,y,Ed}$ | $N_{b,z,Rd}$ | $N_{b,fi,z,Ed}$ | $N_{b,T,Rd}$ | $N_{b,fi,T,Ed}$ |
| mm | mm | $m^{-1}$ | kN | kN | kN | kN | kN | kN |
| 100 x 50 | 6 | 179 | 246 | 160 | 152 | 99 | 287 | 187 |

Fire Resistances (Axial Compression)

| Exposure Time | Temperature of Section | Flexural Buckling Resistance in Fire | | | | Torsional Buckling Resistance in Fire | |
|---|---|---|---|---|---|---|---|
| t | θ | $N_{b,fi,t,y,Rd}$ | Ratio* of Resistance / Loading at FLS | $N_{b,fi,t,z,Rd}$ | Ratio* of Resistance / Loading at FLS | $N_{b,fi,T,Rd}$ | Ratio* of Resistance / Loading at FLS |
| min | °C | kN | | kN | | kN | |
| 10 | 474 | 163 | 1.02 | 116 | 1.17 | 175 | 0.94 |
| 15 | 644 | 134 | 0.84 | 99 | 1.00 | 142 | 0.76 |
| 20 | 738 | 107 | 0.67 | 83 | 0.84 | 111 | 0.60 |
| 25 | 790 | 88 | 0.55 | 70 | 0.71 | 89 | 0.48 |
| 30 | 825 | 75 | 0.47 | 61 | 0.62 | 75 | 0.40 |
| 45 | 894 | 47 | 0.29 | 40 | 0.40 | 47 | 0.25 |
| 60 | 940 | 34 | 0.21 | 30 | 0.30 | 34 | 0.18 |

* If the section is not fully utilised at normal temperature (i.e. ν<1.0), then the loading at the fire limit state can be reduced by entering the normal temperature utilisation factor on the **Loading Mode** page. This will result in a longer period of fire resistance (i.e. the member will be able to resist the load for a longer time).

- How to Improve Fire Resistance

**Figure 9.7** *Fire design software: first input page and results page*

# 10 WP8: PROJECT CO-ORDINATION

Throughout the course of the project, six project meetings were held:

15 September 2004 in Stockholm

17 May 2005 in Dusseldorf

22 November 2005 in Ascot

16 May 2006 in Paris

28 November 2006 in Ascot

14 May 2007 in Brussels

At the meetings, partners discussed the work they had carried out and their plans for future activities. Solutions to problems were debated and suggestions made on how the research results could be converted into practical applications.

A password-protected project web site was set up where all project documents were posted including minutes of meetings, progress reports etc.

A number of the test programmes were completed later than originally scheduled due to difficulties encountered in procuring suitable test specimens. Only a few furnaces are available in Europe for testing structural members. These furnaces are very much in demand for testing commercial products, so further delays in the test programme occurred due to the furnaces being busy with other test work.

In general, the work was completed in accordance with the initial planned activities. A few changes to some of the test programmes were made when preliminary analysis work indicated that the original test programme could be improved upon.

A six month extension to the project was granted to enable activities in WP5 to be completed.

# 11 FINAL WORK PACKAGE REPORTS

All deliverables from this project can be downloaded from www.steel-stainless.org/fire. The Final Work Package Reports are listed below.

**WP1 Fire resistant structures and products**
Tests and analysis of fire resistant structures and products
Tiina Ala-Outinen,
VTT, 2007

**WP1 Fire resistant structures and products**
Numerical studies on fire resistant structures and products
Peter Schaumann, Oliver Bahr, Alexander Heise, Florian Kettner,
Leibniz University Hannover, 2007

**WP2 Composite members in fire**
Christophe Renaud
CTICM, 2007

**WP3 Members with Class 4 cross-sections in fire**
Fire tests on RHS cross-sections
Tiina Ala-Outinen,
VTT, 2007

**WP3 Members with Class 4 cross-sections in fire**
Analysis and design guidance on Class 4 members in fire
Björn Uppfeldt
SBI, 2007

**WP4 Properties at elevated temperatures**
Andrea Montanari and Giuliana Zilli
CSM, 2007

**WP5 Bolts and welds at elevated temperatures**
Bolts at elevated temperatures
Giuliana Zilli and Andrea Montanari
CSM, 2008

**WP5 Bolts and welds at elevated temperatures**
Isothermal tests on butt welded joints
Jukka Säynäjäkangas
Outokumpu, 2007

**WP5 Bolts and welds at elevated temperatures**
Numerical studies on welds at elevated temperatures
Bernt Johansson
SBI, 2007

**WP6 Natural fire design**
Christophe Renaud
CTICM, 2007

# 12 EXPLOITATION AND IMPACT OF RESEARCH RESULTS

## 12.1 Technical and economic potential

Stainless steel has unique properties which can be taken advantage of in a wide variety of applications in the construction industry. Stainless steel structural members are most likely to be used for structures in unusually corrosive environments or where maintenance is expensive or because a particular visual effect is required. Applications for structural members include canopies, entrances and atria, industrial structures for food, paper and pulp and chemical industries, swimming pool buildings and car park structures.

The results of the project generally highlight opportunities for stainless steel where 30 or 60 minutes fire resistance can be achieved with an unprotected stainless steel structural member, often where carbon steel would require protection to achieve similar periods of fire resistance. Additional cost and construction schedule savings arise from the absence of externally applied fire protection. Economic considerations mean it would be unlikely that stainless steel would be chosen solely because of its superior fire resistance. However, for specifiers considering stainless steel because of its aesthetic and durability properties, the additional benefit of providing fire resistance for a significant period whilst unprotected, might sway the balance in the favour of stainless steel. In applications where good corrosion resistance coupled with good fire resistance are required, stainless steel offers an excellent solution.

The hybrid stainless-carbon steel composite beam tested in WP2 is an interesting concept in which the lower exposed flange is stainless steel and the web and upper flange are carbon steel. The system has considerable architectural appeal due to the attractive exposed soffit of the stainless steel lower flange. Further product development activities should be carried out on this system. Figure 12.1 shows the very attractive stainless steel sinusoidal composite floor deck at the Luxembourg Chamber of Commerce.

**Figure 12.1** *Exposed stainless steel soffit at Luxembourg Chamber of Commerce*

## 12.2 Dissemination of project results

A web page has been developed at www.steel-stainless.org/fire for disseminating the project deliverables. From this page the Final Summary Report and each Final Work Package Report can been downloaded. A link to the design software is also given on this web page. A screen shot of this web page is shown in Figure 12.2.

Section 12.3 lists papers prepared describing the outcomes of the project which have been presented at conferences or included in journals.

The design guidance developed in this project now needs to be presented in a simple-to-use format and disseminated to practising engineers. Once feedback has been obtained from practitioners, the guidance should be prepared in a form suitable for submitting to the CEN Technical Committees responsible for preparing amendments and revisions to Eurocodes 3 and 4.


**Figure 12.2** *Stainless Steel in Fire web page at www.steel-stainless.org/fire*

## 12.3 Publications and conference presentations resulting from the project

| | | | |
|---|---|---|---|
| Light weight structures exposed to fire: a stainless steel sandwich panel | Tiina Ala-Outinen, Peter Schaumann, Olli Kaitila, Florian Kettner | Fourth International Workshop Structures in Fire, Aveiro, Portugal | May 2006 |
| Class 4 Stainless Steel Box Columns in Fire | Bjorn Uppfeldt and Milan Veljkovic | Cost Action 26, Prague Workshop | March 2007 |
| A design model for stainless steel box columns in fire | Björn Uppfeldt, Tiina ala Outinen, and Milan Veljkovic | Stainless Steel in Structures: Third International Experts Seminar, Ascot | 29-30 November 2007 |
| Fire behaviour of steel-concrete composite members with austenitic stainless steel | Christophe Renaud and Bin Zhao | Advanced Steel Construction journal | 2008 |
| A fire engineering approach to the design of stainless steel structural systems | Nancy Baddoo and Bassam Burgan | Sixth European Stainless Steel Conference Science and Market, Helsinki, Finland | June 2008 |

# 13 CONCLUSIONS

This report summarises the results of a 3½ year European research project studying the behaviour of a range of structural stainless steel systems subject to fire loading. As a result of the superior strength and stiffness retention, stainless steel columns and beams generally retain their load-bearing capacity for a longer time than equivalent carbon steel columns. A conservative approach to fire resistant design of stainless steel structures is covered in an informative annex to EN 1993-1-2, despite fire test data on stainless steel structural members being sparse. This project was carried out in an attempt to develop more comprehensive and economic design guidance. The project included tests on materials, members and connections, numerical analysis and development of design guidance aligned to the Eurocodes.

Stainless steel in buildings is almost always exposed, so this project aimed to identify structural solutions which give a specified period of fire resistance without any fire protection applied to the surface of the steel. Benefits of eliminating fire protection include lower construction costs, shorter construction time, more effective use of the internal floor area and more attractive appearance.

This project is significant because a number of the test programmes were highly innovative, being the first of their kind to be carried out on stainless steel. The topics studied and key outcomes of the project were:

- A range of concepts for load-bearing and separating systems designed to suppress temperature rise was developed and tested; 30 and 60 minutes fire resistance was achieved
- From a programme of tests and numerical studies, simplified design methods were developed for stainless steel concrete filled hollow sections and hybrid stainless-carbon steel composite floor beams in fire
- More economical fire design guidance for slender cross-sections was developed based on a test programme and numerical analysis.
- Strength retention curves for two grades of stainless not previously studied were derived through a programme of transient state tests.
- Tests on welded and bolted connections in fire enabled design guidance to be derived.
- The performance of external stainless steel columns and internal columns in open car parks when subjected to a realistic parametric fire was studied. Stainless steel exhibited superior performance to equivalent carbon steel columns in all cases.
- A set of preliminary generic strength retention curves for stainless steels were developed.
- A less conservative approach for determining the fire resistance of stainless steel structural members was developed and published in the Third Edition of the *Design Manual for Structural Stainless Steel*.
- Online software for fire resistant design of cold formed stainless steel structural members was developed.

The project has achieved its objective of developing more comprehensive guidance on the design of stainless steel structural systems in fire. The guidance developed now needs to be tested out in practice before it can be submitted to the CEN Technical Committees responsible for preparing amendments and revisions to Eurocodes 3 and 4. Despite significant progress in understanding the performance of stainless steel structural members in fire, a number of areas require further study. These include the development of simple design rules for stainless steel columns subject to non-uniform temperature distributions.

# LIST OF FIGURES

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# APPENDIX A COEFFICIENTS FOR DESIGN OF COMPOSITE COLUMNS

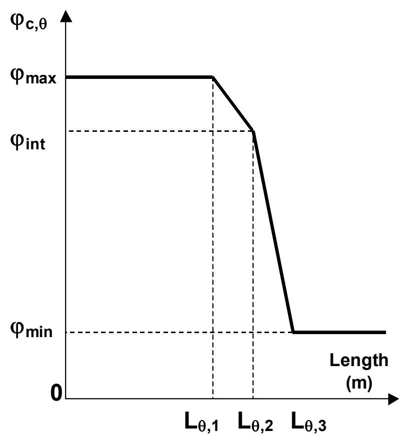
***Table A.1*** *Values of coefficient $\varphi_{a,\theta}$ for steel hollow sections*

| **Fire Rating** | **R30** | **R60** |
|---|---|---|
| $\varphi_{a,\theta}$ | 0.75 | 0.575 |

For concrete: $\varphi_{c,\theta}$ is defined by means of six parameters $L_{\theta,1}$, $L_{\theta,2}$, $L_{\theta,3}$, $\varphi_{max}$, $\varphi_{int}$ and $\varphi_{min}$ depending on of the cross-section size (external dimension ($b$) and thickness ($e$) of the hollow steel section), the column buckling length $L_\theta$, the ratio of reinforcement $A_s/(A_s+A_c)$ and the fire rating.

***Table A.2*** *Values of $\varphi_{c,\theta}$ for different column buckling lengths*

| **Column buckling length, $L_\theta$** | $\varphi_{c,\theta}$ |
|---|---|
| $L_\theta \leq L_{\theta,1}$ | $\varphi_{c,\theta} = \varphi_{max}$ |
| $L_{\theta,1} \leq L_\theta < L_{\theta,2}$ | $\varphi_{c,\theta} = \dfrac{\varphi_{max} - \varphi_{int}}{L_{\theta,1} - L_{\theta,2}} L_\theta + \dfrac{\varphi_{int} \times L_{\theta,1} - \varphi_{max} \times L_{\theta,2}}{L_{\theta,1} - L_{\theta,2}}$ |
| $L_{\theta,2} \leq L_\theta < L_{\theta,2}$ | $\varphi_{c,\theta} = \dfrac{\varphi_{int} - \varphi_{min}}{L_{\theta,2} - L_{\theta,3}} L_\theta + \dfrac{\varphi_{min} \times L_{\theta,2} - \varphi_{int} \times L_{\theta,3}}{L_{\theta,2} - L_{\theta,3}}$ |
| $L_{\theta,3} \leq L_\theta$ | $\varphi_{c,\theta} = \varphi_{min}$ |




Values of parameters $L_{\theta,1}$, $L_{\theta,2}$ and $L_{\theta,3}$ are given in the following table as a function of the cross-section size and the ratio of reinforcement $A_s/(A_s+A_c)$. For intermediate values of the external size and thickness of hollow steel section, linear interpolation may be used to calculate $L_{\theta,1}$, $L_{\theta,2}$ and $L_{\theta,3}$.

***Table A.3*** *Values of parameters $L_{\theta,1}$, $L_{\theta,2}$ and $L_{\theta,3}$ for fire ratings R30*

| **Ratio of reinforcement $A_s/(A_s+A_c)$.** | **Cross-section size** | | **Fire rating R30** | | |
|---|---|---|---|---|---|
| | **b (mm)** | **e (mm)** | **$L_{\theta,1}$ (m)** | **$L_{\theta,2}$ (m)** | **$L_{\theta,3}$ (m)** |
| 0 | 100 | 4 | 0.50 | 0.70 | 1.25 |
| | | 8 | 0.50 | 0.60 | 0.90 |
| | 250 | 4 | 1.90 | 2.90 | 4.00 |
| | | 8 | 1.50 | 2.25 | 3.25 |
| | 500 | 4 | 8.25 | 9.40 | 9.75 |
| | | 8 | 6.20 | 7.70 | 9.50 |
| 1 to 5 | 150 | 4 | 0.75 | 1.50 | 2.40 |
| | | 8 | 0.60 | 1.20 | 2.00 |
| | 500 | 4 | 5.50 | 9.50 | 15.00 |
| | | 8 | 5.00 | 8.00 | 12.00 |

**Table A.4** *Values of parameters $L_{\theta,1}$, $L_{\theta,2}$ and $L_{\theta,3}$ for fire ratings R60*

| Ratio of reinforcement $A_s/(A_s+A_c)$. | Cross-section size b (mm) | e (mm) | Fire rating R60 $L_{\theta,1}$ (m) | $L_{\theta,2}$ (m) | $L_{\theta,3}$ (m) |
|---|---|---|---|---|---|
| 0 | 150 | 4 | 0.50 | 0.90 | 1.80 |
| | | 8 | 0.50 | 0.80 | 1.30 |
| | 300 | 4 | 2.40 | 3.20 | 3.50 |
| | | 8 | 1.80 | 2.40 | 2.90 |
| | 400 | 4 | 4.80 | 5.50 | 6.00 |
| | | 8 | 3.50 | 3.90 | 4.30 |
| | 500 | 4 | 7.70 | 8.60 | 9.20 |
| | | 8 | 5.60 | 6.50 | 7.10 |
| 1 | 150 | 4 | 0.60 | 1.00 | 2.00 |
| | | 8 | 0.60 | 0.80 | 1.25 |
| | 500 | 4 | 5.00 | 7.00 | 10.00 |
| | | 8 | 3.50 | 5.50 | 9.00 |
| 2 to 5 | 150 | 4 | 0.70 | 1.12 | 2.45 |
| | | 8 | 0.70 | 0.90 | 1.80 |
| | 500 | 4 | 4.50 | 6.25 | 11.00 |
| | | 8 | 3.00 | 5.00 | 9.50 |

**Table A.5** *Values of parameters $\varphi_{max}$, $\varphi_{int}$ and $\varphi_{min}$ for fire ratings R30*

| Ratio of reinforcement $A_s/(A_s+A_c)$. | Cross-section size, b (mm) | Fire rating R30 $\varphi_{max}$ | $\varphi_{int}$ | $\varphi_{min}$ |
|---|---|---|---|---|
| 0 | 100 | 1 | 0.8 | 0 |
| | 500 | | | |
| 1 | 100 | 1 | 0.8 | 0.1 |
| | 500 | | | 0.15 |
| 2 | 100 | 1 | 0.8 | 0.12 |
| | 500 | | | 0.3 |
| 3 | 100 | 1 | 0.8 | 0.15 |
| | 500 | | | 0.45 |
| 5 | 100 | 1 | 0.8 | 0.2 |
| | 500 | | | 0.6 |

**Table A.6** *Values of parameters $\varphi_{max}$, $\varphi_{int}$ and $\varphi_{min}$ for fire ratings R60*

| Ratio of reinforcement $A_s/(A_s+A_c)$. | Cross-section size, b (mm) | Fire rating R60 $\varphi_{max}$ | $\varphi_{int}$ | $\varphi_{min}$ |
|---|---|---|---|---|
| 0 | 150 | 1 | 0.85 | 0 |
| | 500 | | | |
| 1 | 150 | 1 | 0.85 | 0.05 |
| | 500 | | | 0.05 |
| 2 | 150 | 1 | 0.85 | 0.08 |
| | 500 | | | 0.20 |
| 3 | 150 | 1 | 0.85 | 0.10 |
| | 500 | | | 0.35 |
| 5 | 150 | 1 | 0.85 | 0.20 |
| | 500 | | | 0.60 |

# APPENDIX B SUMMARY OF STAINLESS STEEL COLUMN TESTS IN FIRE

**Table B.1** *Experimental data on stainless steel column buckling behaviour at elevated temperature.*

| ID | Cross-section | Class | A ($A_{eff}$) [$mm^2$] | J ($J_{eff}$) [$mm^4$] | Grade | $f_y$ [MPa] | $f_u$ [MPa] | E [GPa] | $l_0$ [mm] | $\overline{\lambda}$ | $N_{b,Rd}$ [kN] | $F_{applied}$ [kN] | Critical temp. [°C] |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| **SCI (1)** | RHS 150 × 100 × 6 | 1 | 2852 | 4472392 | 1.4301 | 262 | 625 1) | 200 | 1700 | 0,49 | 705 | 268 | 801 |
| **SCI (2)** | RHS 150 × 75 × 6 | 1 | 2555 | 2299500 | 1.4301 | 262 | 625 1) | 200 | 1700 | 0,65 | 561 | 140 | 883 |
| **SCI (3)** | RHS 100 × 75 × 6 | 1 | 1973 | 1799455 | 1.4301 | 262 | 625 1) | 200 | 1700 | 0,65 | 435 | 156 | 806 |
| **SCI (4)** | Double C 200 × 150 × 6 | 4 | 3233 | 2) | 1.4301 | 262 | 625 1) | 200 | 1700 | 0,66 | 704 | 413 | 571 |
| **CTICM (1)** | RHS 100 × 100 × 4 | 2 | 1470 | 2260000 | 1.4301 | 298 | 625 1) | 200 | 3990 | 1,25 | 190 | 80 | 835 |
| **CTICM (2)** | RHS 200 × 200 × 4 | 4 | 2111 | 2) | 1.4301 | 298 | 625 1) | 200 | 3990 | 0,51 | 587 | 230 | 820 |
| **CTICM (3)** | RHS 100 × 100 × 3 | 4 | 979 | 1770000 | 1.4318 C700 | 360,5 | 750 1) | 200 | 3140 | 1,00 | 207 | 52 | 835 |
| **CTICM (4)** | RHS 100 × 100 × 3 | 4 | 813 | 1770000 | 1.4318 C800 | 629 | 850 1) | 200 | 3140 | 1,20 | 236 | 52 | 880 |
| **VTT (1)** | RHS 40 × 40 × 4 | 1 | 535 | 111000 | 1.4301 | 592 | 736 | 170 | 887 | 1,16 | 237 | 8 different points[18] | |
| **VTT (2)** | RHS 40 × 40 × 4 | 1 | 535 | 111000 | 1.4571 | 545 | 670 | 170 | 887 | 1,11 | 237 | 4 different points[18] | |
| **VTT (3)** | RHS 30 × 30 × 3 | 1 | 301 | 35000 | 1.4301 | 576 | 712 | 170 | 887 | 1,52 | 75 | 4 different points[18] | |

1) Experimental value not available, so value indicated by the Standard is used.

2) Value not available.

SCI and CTICM tests 1 and 2 are reported in Ref [3], CTICM tests 3 and 4 are reported in [5] and VTT tests are reported in Ref [18].

# APPENDIX C TECHNICAL ANNEX

EUROPEAN COMMISSION
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Directorate G – Industrial Technologies
**Research Fund for Coal and Steel**

**ANNEX IV**
**Form 1-1**

# TECHNICAL ANNEX

| | |
|---|---|
| **Project acronym:** | **SSIF** |
| **Proposal No:** | **RFS-PR-03143** |
| **Contract No:** | **RFS-CR-04048** |

TITLE: **STAINLESS STEEL IN FIRE**

## 1. OBJECTIVES

The objective of this project is to develop more comprehensive and economic guidance on the design of stainless steel structural members and connections when exposed to fire, including specific products meeting the requirements for 30 and 60 minutes fire resistance without fire protection.

The technical objectives are:

- To generate structural solutions where it is possible to use stainless steel structural members in buildings without fire protection, both considering the 'standard' fire and lower, more realistic fire loads.
- To generate test results on commonly used grades of stainless steel in structures; this will include tests on material, members and connections.
- To develop numerical models based on standardised methods and validated against the test results in order to generate additional data upon which a basis of design for a range of grades and types of members and connections can be established.

The commercial objectives are:

- To develop a methodology in the form of fire resistant design rules suitable for incorporation into standards that enable stainless steel members and connections to be designed cost effectively and safely in structures.
- To ensure that the deliverables of the project are in a format that is readily disseminated and used in the EU by incorporating them into European Standards. This will be achieved by the direct involvement of many of the key members of CEN committees in the project. This will maximise the likelihood of acceptance and incorporation of the rules in the standards within the necessary timescales.

| B.0 | WORK PACKAGE DESCRIPTION | WP No | 1 |
|---|---|---|---|

| **Workpackage Title** | Fire resistant structures and products | **No of man hours** |
|---|---|---|
| **WP Leader** | VTT (5) | 1180 |
| **Contractor (s)** | University of Hannover (6) | 924 |
| | Outokumpu Stainless (4) *(material supply)* | - |
| **Total** | | **2104** |

**1 -Objectives**

The main objective is to develop new stainless steel products without passive or active fire protection that can achieve 30 or 60 minutes fire resistance in a standard fire or in a natural fire. The new products will include load-bearing structures and fire separating members.

**2 - Work programme**

***(Participating contractors indicated by Contractor number after Task title)***

**Task 1.1 The applications of stainless steel in demanding constructions (5)**

Taking into account the demands of ease of maintenance, corrosion resistance and aesthetic appearance, potential applications will be chosen. Public buildings, where steel is used together with glass as well as paper, chemical and wood pulp industries are applications where good fire resistance properties are required along with the other special characteristics of stainless steel. When relevant applications are chosen, preliminary design calculations will be carried out.

**Task 1.2 Structural solutions for load-bearing structures meeting R30 or R60 requirements (5)**

The development of structural solutions for load–bearing structures will be based on task 1.1. Possible structural solutions include special cross-sections, tubes within each other and the use of stainless steel box shielding with beams and columns. Members can also be protected on one side by other materials. Verification of these structures is based on calculations;.

**Task 1.3 Structural solutions for fire separating structures meeting EI30 or EI60 requirements (5)**

The low emissivity of stainless steel is utilised in the development of separating structures. Problems due to heat expansion will be eliminated by structural solutions. Wall structures which meet load-bearing and separating requirements will also be considered. The structures can be sandwich panels, corrugated core sandwich panels, etc. Numerical analysis will be carried out to develop suitable structures. Simple preliminary tests will be carried out.

**Task 1.4 Experimental fire tests (5)**

***Load-bearing structures:***

Based on tasks 1.1 and 1.2, load-bearing structures will be chosen for the test programme. Fire resistance tests will be performed to develop and verify the calculation method for determining the strength of the structures exposed to fire. The temperature development will be measured as well the load-bearing capacity. A maximum of 6 different types of structures will be tested.

***Fire separating structures:***
Preliminary tests with small unloaded wall specimens will be performed before the final large-scale tests are carried out. A maximum of 5 different types of structures will be tested

**Task 1.5 Numerical studies (6)**
Numerical simulations of the heating up and load-bearing capacity of structures will be carried out. Parametric studies will extend the range of cross-sections and material types under investigation.

**3 - Interrelation with other workpackages: WP 2, 3, 4 and 7**

**4 - Deliverables and milestones**
Structural solutions for load-bearing members meeting the requirements of R30 and R60 and separating members meeting the requirements of EI30 and EI60.
*Completion: End of Year2*

| B.0 | WORK PACKAGE DESCRIPTION | WP No | 2 |
|---|---|---|---|

| **Workpackage Title** | Composite members in fire | **No of man hours** |
|---|---|---|
| **WP Leader** | CTICM (2) | 1777 |
| **Contractor (s)** | Outokumpu Stainless (4) *(material supply)* | - |
| **Total** | | **1777** |

**1 - Objectives**
To develop design guidance for composite members in fire by a programme of fire tests on concrete filled RHS and CHS columns and tests on floor beams with concrete fire protection.

**2 - Work programme**
***(Participating contractors indicated by Contractor number after Task title)***

**Task 2.1 Tests on concrete filled RHS and CHS columns in fire (2)**
It is necessary to carry out about 5 fire tests in order to generate adequate experimental evidence on the fire resistance of stainless steel RHS and CHS columns filled with unreinforced concrete. In these tests, two dimensions and two eccentricities will be used. These tests will seek to achieve a fire rating of both 30 minutes and 60 minutes.

**Task 2.2 Tests on floor beams with concrete fire protection (2)**
Two types of Slimflor beam will be tested (one fabricated entirely from stainless steel and one where the exposed flange is stainless steel and the web and upper flange is carbon steel). In these tests, some additional small specimens will also be included in order to study the heating behaviour of other types of beams.

**Task 2.3 Analysis of test results (2)**
The above fire tests will be systematically modelled with the help of advanced non-linear finite element analysis packages. Stainless steel material models from either earlier ECSC research projects or WP4 of the current research project will be adopted. In the analyses both geometrical and material non-linearity will be included. The temperature distributions measured in the tests will be taken into account. Parametric studies will look at a wider range of cross-section geometries, rates of heating and loading than were tested and will provide a good basis for the development of design rules.

**Task 2.4 Development of design guidance (2)**
From the results of the fire tests and numerical analyses, design rules for practical use will be developed for both concrete filled hollow section stainless columns meeting R30 and R60 fire ratings and partially protected floor beams.

**3 - Interrelation with other workpackages (please give WP No) :WP1, WP4 and WP7**

**4 - Deliverables and milestones**
Simple fire design rules for hollow section columns filled with unreinforced concrete for fire ratings of R30 and R60 and partially protected floor beams.
*Completion: Ist quarter, Year3*

| B.0 | WORK PACKAGE DESCRIPTION | WP No | 3 |
|---|---|---|---|

| Workpackage Title | Class 4 cross-sections in fire | No of man hours |
|---|---|---|
| **WP Leader** | SBI (7) | 925 |
| **Contractor (s)** | VTT (5) | 870 |
| **.** | Outokumpu Stainless (4) *(material supply)* | - |
| **Total** | | **1795** |

**1 – Objectives**
The aim of this work package is to provide design rules for structural members with Class 4 cross-sections in fire.

**2 - Work programme**
***(Participating contractors indicated by Contractor number after Task title)***

**Task 3.1 Fire tests on RHS sections (5)**
8-10 tests will be performed on rectangular hollow sections with a Class 4 cross-section. The tests will be designed such that failure is by local buckling.

**Task 3.2 Numerical analysis (7)**
The above fire tests will be systematically modelled with the help of advanced non-linear finite element analysis packages. Stainless steel material models from either earlier ECSC research projects or WP4 of the current research project will be adopted. In the analyses both geometrical and material non-linearity will be included. The temperature distributions measured in the tests will be taken into account. Parametric studies will look at a wider range of cross-section geometries, rates of heating and loading than were tested and will provide a good basis for the development of design rules.

**Task 3.3 Development of design guidance (7)**
From the results of the fire tests and numerical analyses, design rules for practical use will be developed for structural members with Class 4 cross-sections.

**3 - Interrelation with other workpackages:** WP 1, 6 and 7

**4 - Deliverables and milestones**
Simple fire design rules for practical use for structural members with Class 4 cross-sections.
*Completion: 2nd quarter, Year2*

| B.0 | WORK PACKAGE DESCRIPTION | WP No | 4 |
|---|---|---|---|

| Workpackage Title | Properties at elevated temperatures | No of man hours |
|---|---|---|
| **WP Leader** | CSM (3) | 2100 |
| **.** | Outokumpu Stainless (4) *(material supply)* | - |
| **Total** | | **2100** |

**1 – Objectives**
To obtain information on the mechanical properties at elevated temperatures for grades of stainless steel not tested before.

**2 - Work programme**
***(Participating contractors indicated by Contractor number after Task title)***

**Task 4.1 Transient state tests (3)**
Transient state tests will carried out on one grade of stainless steel applying a constant load to flat specimens and a heating rate of 10°C/min up to failure. Strain and temperature during the tests will be registered. Families of Strain vs Temperature curves will be obtained and parameterised at different load levels (from 0.1 to 0.9 of the yield stress, at a minimum of 10 steps). The post data processing will generate stress vs strain curves which are representative of the true behaviour of the steels in fire.

**Task 4.2 Material models for ENV 1993-1-2 (3)**
From the test results, strength and stiffness retention parameters for the grades tested will be developed for use with the material model for stainless steel in EN 1993-1-2.

**3 - Interrelation with other workpackages:** WP 2, 3, 6 and 7

**4 – Deliverables and milestones**
Report containing details of applied test methodologies, test results and material model.
*Completion: 2nd quarter, Year2*

| B.0 | WORK PACKAGE DESCRIPTION | WP No | 5 |
|---|---|---|---|

| **Workpackage Title** | Bolts and welds at elevated temperatures | **No of man hours** |
|---|---|---|
| **WP Leader** | U&A (8) | 1388 |
| **Contractor (s)** | Outokumpu Stainless Oy (4) *(material supply)* | 308 |
| | SBI (7) | 140 |
| | CSM (3) | 1400 |
| **Total** | | **3236** |

**1 – Objectives**
To obtain information on the mechanical properties of stainless steel bolts and welds at elevated temperatures, including:

- tension and shear tests of bolts
- tension tests on weld materials
- tension and shear tests of butt welded and fillet weld joints
- development of design guidance

**2 - Work programme**
***(Participating contractors indicated by Contractor number after Task title)***

**Task 5.1 Tension and shear tests of bolts (3)**
Tensile and shear tests at room temperature will be carried out according to current practice. In order to test bolts to failure at high temperatures, proper superalloy grips will be improved and manufactured. Tests at temperatures up to 1000°C will be carried out (a minimum of 4 different temperatures). M12 stainless steel bolts at two strength levels (according to EN ISO 3506) will be considered.

**Task 5.2 Tension tests on weld materials (8)**
Tensile characterizations at room and high temperatures will be carried out.

**Task 5.3 Tension and shear tests of butt weld and fillet weld joints (4) and (8)**
Butt plate joints welded with selected weld materials will be manufactured and tested in tension from room temperature to 1000°C. Shear tests on T-fillet joints will be performed at room temperature.

**Task 5.4 Development of design guidance (3) and (7)**
On the basis of the results obtained from the experimental activities, design guidance for stainless steel bolts and welds in fire structures in fire events will be prepared. The intention is to modify simple design rules from EN 1993-1-8 to make them suitable for inclusion in EN 1993-1-2.

**3 – Interrelation with other workpackages:** WP 1 to 5 and 7

**4 - Deliverables and milestones**
Development of design guidance for stainless steel bolts and welds
*Completion: 2nd quarter, Year3*

| B.0 | WORK PACKAGE DESCRIPTION | WP No | 6 |
|---|---|---|---|

| **Workpackage Title** | Parametric fire design | **No of man hours** |
|---|---|---|
| **WP Leader** | CTICM (2) | 461 |
| **Total** | | **461** |

**1 – Objectives**

- To analyse the behaviour of some typical examples of unprotected external structural stainless steel in fire and develop guidance
- To analyse the behaviour of some typical examples of unprotected structural stainless steel in natural fires and develop design guidance
- Applications will be investigated where it is possible to use stainless steel unprotected, whereas carbon steel in the same application would require protection.

**2 - Work programme**
***(Participating contractors indicated by Contractor number after Task title)***

**Task 6.1 External structures (balconies, stairs etc) (2)**
Numerical analysis will be carried out to investigate the possibility of using exposed stainless steel members located outside buildings. In this analysis, the heating-up characteristics of stainless steel members will be based mainly on the existing simple calculation methods in the Eurocodes, in combination with the results of the current ECSC funded project 'External structures in fire'. (This ongoing project is developing more accurate rules for calculating the heating of bare and protected external steel sections and balconies in fire.)

**Task 6.2 Large sheds (factory sheds, sports halls, etc) and car park buildings (2)**
Natural fires are different in nature and in effect to the standard fire conventionally adopted in fire resistant design calculations. A parametric fire is a mathematical idealisation of a natural fire in a compartment. A number of research projects have investigated the behaviour of carbon steel structure in natural fire conditions, sometimes called parametric fire design. The results of these projects (particularly the studies of fire development), will be extended to investigate the fire resistance of unprotected stainless steel members, such as columns of open car parks, etc. .

**Task 6.3 Development of design guidance (2)**
Simple design rules will be developed for external bare stainless steel members in fire and stainless steel members exposed to natural fires.

**3 - Interrelation with other workpackages:** WP 1 to 4 and 7

**4 - Deliverables and milestones**
Design guidance on external bare stainless steel members in fire and stainless steel members exposed to natural fires.
*Completion: 3rd quarter, Year3*

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| B.0 | WORK PACKAGE DESCRIPTION | WP No | 7 |
|---|---|---|---|

| Workpackage Title | Design aids and software | No of man hours |
|---|---|---|
| WP Leader | SCI (1) | 1252 |
| **Total** | | **1252** |

**1 – Objectives**
To produce design tools for practical design

- to prepare a design guide which draws together the design guidance developed in WP 1 to 6, including guidance on standard solutions for achieving 30 and 60 minutes fire resistance without passive or active fire protection
- development of fire design software
- to ensure a consistent format and methodology is adopted throughout the design guidance and to generate the necessary input for submission to the appropriate standards committees.

**2 - Work programme**

**Task 7.1 Develop design guide**
In order to streamline the process of design guidance development, this activity has been divided among the work packages 1 to 6, and in each case allocated to an appropriate partner closely associated with the generation of the basic data. This approach will increase the efficiency of the design development process by eliminating the communication delays and difficulties that are likely to result if the design development was centralised with a single partner.
However, the final deliverable from the project must be coherent, integrated and capable of being efficiently integrated into European standards. The principle activities within this work package is as follows:

- Unify the approach to the development of the recommendations to ensure uptake by European Standards
- Ensure the European guidance for the development of design rules (ENV 1993-1-1, Annex Z, to become Annex D of EN 1990) is consistently interpreted and applied
- Compile a final report comprising the design guidance generated by the project, in a form suitable for adoption by the appropriate CEN drafting committee.

**Task 7.2 Development of fire design software**
An internet-based software package will be developed. The package will calculate the fire resistance of a structural member after certain time intervals and advise on methods of fire protection (if necessary) or alternative ways of enhancing the fire resistance of the member. The package will be an extension of the software for designing stainless steel structural members at room temperature developed during the previous Valorisation Project: Development of the Use of Stainless Steel in Construction (Contract No. 7215-PP-056), completed in 2003 and available at http://www.steel-stainless.org/software.

**3 - Interrelation with other workpackages:** WP 1 to 6

**4 - Deliverables and milestones:**
A design guide and web-based software.
*Completion: 4th quarter, Year3*

| B.0 | WORK PACKAGE DESCRIPTION | WP No | 8 |
|---|---|---|---|

| Workpackage Title | Project co-ordination | No of man hours |
|---|---|---|
| **WP Leader** | SCI (1) | 790 |
| **Total** | | **790** |

**1 – Objectives**

- to manage and co-ordinate the project and maintain adequate lines of communication between all the partners and sub-contractors involved in the project in order to achieve the project objectives within the time and budget allocated.
- to prepare the output of the project including the final report and software.

**2 - Work programme**

- This work package covers all management and co-ordination activities required to keep the project on schedule and to cost. This includes:
- Planning of work packages and their inter-relationships;
- Liaison with partners and sub-contractors to monitor progress and identify difficulties;
- Adoption of measures to rectify any problems;
- Progress reporting;
- Organisation and running of progress meetings;
- Liaison with RFCS;
- Project administration

**3 - Interrelation with other workpackages: WP 1 to 7**

**4 – Deliverable and milestones**

Progress reports, progress meetings, final report, etc
*Completion: Continuous activity throughout the project duration*

**ANNEX IV**
**Form 1-3**

| Workpackages | | Deliverables | Hours on project/Contractor(s) | | | | | | | | 1st year | | | | 2nd year | | | | 3rd year | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | 1 | 2 | 3 | 4 | 5 | 6 | 7 | 8 | I | II | III | IV | I | II | III | IV | I | II | III | IV |
| WP 1 | Fire resistant structures and products | | | | | | | | | | | | | | | | | | | | | |
| Task 1.1 | The applications | Identification of specific applications | | | | | 100 | | | | | | | | | | | | | | | |
| Task 1.2 | Load-bearing structures | Structural solution | | | | | 100 | | | | | | | | | | | | | | | |
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| Task 1.4 | Fire tests | Report of tests | | | | | 780 | | | | | | | | | | | | | | | |
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| Workpackages | | Deliverables | Hours on project/Contractor(s) | | | | | | | | 1st year | | | | 2nd year | | | | 3rd year | | | |
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| | | | 1 | 2 | 3 | 4 | 5 | 6 | 7 | 8 | I | II | III | IV | I | II | III | IV | I | II | III | IV |
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| Total Hours on Project | | | 2042 | 2238 | 3500 | 308 | 2050 | 924 | 1065 | 1388 | | | | | | | | | | | | |