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Robustness of car parks against localised fire

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Deliverable I: Definition of the problem and selection of the appropriate investigation ways

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I Introduction

The work package 1 objectives were fully contemplated during the first period of the project (i.e. from 01/07/2008 to 31/12/2008). These objectives were indentified within the project description as follows:

- Definition of the car park structures (constitutive elements, connection types, loading, bracing systems …), of the specific design requirements and of the risks to be possibly encountered in terms of localised fire (destruction of one column or more than one column according to the position of the column in the structure, intensity and duration of the fire …).
- Identification of the distribution of temperatures within the affected part of the structure all along the event on the basis of previous research works performed, in particular within past RFCS projects.
- Selection of the philosophy to be followed so as to derive robustness requirements and related design recommendations (indirect methods, direct methods – alternate load path method or specific load resistance methods - …).
- Identification of the appropriate scenario(s) to be considered later on in the studies and of the related situations.

Each contractor contributed more to some items according to their expertise:

- Fire aspects: FCTUCOIMBRA (3), ULGG (1), ARCELORPROFIL (4), CTICM (7)
- Robustness aspects: ICST (2), ULGG (1)
- Design aspects: GREISCH (6)
- Fabrication and erection aspects: ARCELORPROFIL (4)
- Design requirements: CSTB (5), CTICM (7)

The present deliverable summarises the main information which were collected and the investigations ways which have been agreed.

II Fire aspects

The University of Coimbra (FCTUCOIMBRA) has realised, as part of WP1, three documents reporting state of the art (reflecting also activities of some partners of the present project) on the behaviour of:

- Open car parks subjected to a local fire (Simoës da Silva et al., 2009(a) – see Annex A);
- Beam-to-column joints in fire (Simoës da Silva et al., 2009(b) – see Annex B);
- Steel columns in fire (Simoës da Silva et al., 2009(c) – see Annex C).

These topics are developed in the following paragraphs.

II.1 Open car park structures subjected to a local fire (Simoës da Silva et al., 2009(a))

The first document is a review of available knowledge on open car parks subjected to a local fire. The following main subjects have been investigated:

- Available publications on
    - o car fires;
    - o localised fire in car parks;
    - o experimental research works in entire buildings;
    - o experimental research works on sub-structures.
- Verification of a steel open car park under a localised fire:
    - o Fire scenarios;
    - o Rate of heat release (RHR) of cars;
    - o Structural behaviour.
- Design requirements for open car parks as far as fire is concerned.

Statistics of accidental fires in open car parks in France and in New Zealand completed the state-of-the-art. It was shown that car fires have never been dangerous neither for the stability of the structure nor for the people. The fire stays local, the maximum number of cars involved in a fire being three cars. It results that most unprotected steel in open sided steel-framed car parks has sufficient inherent resistance to withstand the effects of any fires that are likely to occur.

II.2 Beam-to-column joints in fire (Simoës da Silva et al., 2009(b))

Beam-to-column connections in a fire are exposed to a combination of forces and moments, significantly different to the single bending moment and shear force assumed in ambient design. Two different loading situations happen on the connection during the heating and the cooling phases of the fire:

- During the heating phase, members in frames that are restrained axially by other cooler members experience significant temperature-induced compression forces that are coupled with bending actions, which produce local buckling of the steel section. At high temperature, flange local buckling and yielding then reduce the stiffness at a localised region (typically near a connection), so that the member rapidly develops tensile actions that can result in catenary actions (Heidarpour, 2007).
- During the cooling phase, the beam recovers its strength and stiffness, the thermal expansion reduces, and the maximum tension in the beam increases. The reversal of bending moment leads to large sagging moments, since the Young modulus and resistance recover their values and the temperature decreases faster on the bottom flange than on the top (Santiago, 2008). Connections and bolts are particularly vulnerable, since tensile failures could happen.

The second document reports a state of the art on the behaviour of beam-to-column joints in fire and considers experimental, numerical and analytical research works. The state of the art is based on two review papers of Luís Simões da Silva (Simões da Silva et al., 2005) and Khalifa Al-Jabri (Al-Jabri et al., 2008). Recent developments do not appear in these two review papers, and the more relevant recent works are described in the document:

- The Ph. D. Thesis of Florian Block (Block, 2006),
- The Ph. D. Thesis of Ramli Sulong (Sulong, 2007),
- The Ph. D. Thesis of Amin Heidarpour (Heidarpour, 2007),

- The Ph. D. Thesis of Aldina Santiago (Santiago, 2008),
- The research project “Robustness of steel connections in fire” conducted by the Sheffield and Manchester Universities (Hu et al., 2008a-b; Yu et al, 2008a-b-c),
- The COSSFIRE European project (Zhao et al., 2007).

A complete list of references on the behaviour of steel and composite joints in fire has also been written, divided into 3 subsections referring to experimental, numerical and analytical research works.

II.3 Steel columns in fire (Simoës da Silva et al., 2009(c))

The behaviour of steel columns subjected to fire conditions is vastly different from that under normal ambient temperature. Elevated temperatures will notably induce additional compressive stress due to thermal axial restraint from adjoining unheated structure which leads to a lower failure temperature of the column.

The third document reports a state of the art on the behaviour of steel columns in fire and considers experimental, numerical and analytical research works:

- The review of experimental research works has been based on the review paper of João Paulo Rodrigues (Rodrigues et al., 2002), and on the Coimbra internal state-of-the-art report written by António Correia (Correia, 2008).
- The review of theoretical research works has been based on the two Ph.D. Theses of Bjorn Aasen (Aasen, 1985) and João Paulo Rodrigues (Rodrigues, 2000), on the Coimbra internal state-of-the-art report (Correia, 2008) and on recent publications.

A complete list of references on the behaviour of steel columns in fire has also been written, divided into 3 subsections referring to experimental, numerical and analytical research works.

III Robustness aspects

In the past, the University of Liège was involved within a RFCS project titled “Robustness – Robust structures by joint ductility“ (Kuhlmann et al, 2009). As a contribution to this project, the University of Liège investigated the exceptional event “Loss of a column in a steel or composite building frame”. The performed researches resulted in two PhD theses presented in 2008 ((Demonceau, 2008) and (Luu, 2008)), in which a state-of-the-art on the robustness aspects is available.

Within these two complementary theses, a general concept to predict the global response of a frame further to a column loss was developed. This concept has been validated to investigate the behaviour of 2D frame, where the column loss does not induce dynamic effects within the structure.

The developed concept could be adapted to the exceptional event considered within the present project, i.e. a localised fire in a car park, when the localised fire is close to a column and leads to the loss of the latter. To adapt this general concept, it is intended to take into account within the latter of the 3D behaviour of the slab, behaviour which has a major influence on the global response of a car park subjected to a localised fire, and the temperature distribution within the structural elements which can affect the properties of the latter (and in particular, in the beam-to-column joints at the top of the column where the fire is localised).

At Imperial College, literature, various references closely related to the project ((Izzuddin et al, 2007, 2008; Vlassis et al., 2007), and in particular WP3, were reviewed. In particular, the mechanisms of progressive collapse were reviewed and the various design standards aiming at enhancing the robustness of structures were fully studied. Furthermore, case studies of structures subject to fire were considered, especially relating to WTC failure and Cardington fire tests.

IV Design aspects, fabrication and erection aspects and design requirements

Within these different items, it was intended to collect information about the current practice for car parks in Europe.

Background information on UK design practice for car parks, involving the structural layout, connection types and load factors, was collected. Two popular types of car park structures are observed in the UK, namely, car parks with and without internal columns. It is found that the ideal layout for car parks in the UK utilizes clear spans without internal columns; therefore, if steel is chosen as the framing material, a clear span solution can be used for the majority of car parks.

A large number of car parks around the UK were surveyed to establish the common types of joints. Generally speaking, the connections adopted in UK car park structures are similar to the normal multi-storey steel or composite structures; however, some special aspects need to be carefully considered in car park connection design such as dynamic effects and fire resistance. For this project, it is commonly agreed to cover only braced structures. Accordingly, the steel connections to be investigated could be “simple” connections. For the major axis connections between beams and columns, three types are commonly used in the UK: angles, end plates and fin plates. With regard to minor axis beam-to-column connections, both end plates and fin plates are generally adopted in the UK. For the connection between the primary beams and secondary beams, angles and fin plates are preferred by engineers around the UK.

With regard to the load factors for car park design, British Standards and European codes were considered. According to BS 5950, the load factors for car parks are similar to those of other buildings. The dead load for car parks is determined by the material used which is similar to normal multi-storey buildings. The imposed loading applicable is given in standard codes e.g. BS 6399: Part 1 in the UK. For a normal mix of vehicles, subject to the maximum weight of any vehicle being 2500kg, the imposed uniformly distributed load given in BS 6399: Part 1 is 2.5kN/m^2. Unfortunately, no load factors are defined for removal of columns according to either BS5950 or Building Regulations.

In France, open car parks benefit from a particular regulation which accept the fire engineering as a method in order to verify fire safety criteria (Art. PS7). In order to help in the definition of relevant scenario to be taken into account, a guide, approved by the ministry in charge of this type of structures, has been published. It details 3 scenarios to be examined for usual car parks. On the other end, a guide dealing with fire safety design of steel car parks has been published. It contains case studies that often serve has reference in pre-design of steel car parks. That is why steel open car parks types are mainly limited to configurations fitting the scope of the guide.

By the way, the usual features of steel car parks in France are mainly:

- Columns: H profile partially encased or circular composite profile
- Beams: I or H profiles, span from 7.5 to 16 m (secondary beams) and from 5 to 10 m (primary beams)
- Decking: precast concrete slab or composite floor, span from 2.5 to 5 m, thickness 12 to 16 cm.
- Height: 2.5 to 4.5 m
- Material: steel S355 grade for beams and columns, S3500 grade for composite floor

For any new car parks, if design is done on the basis of fire engineering, the project has to be presented to a local safety commission, in charge of approving fire scenarios and safety criteria. Results of the engineering study have to get a positive expert advice from one of the approved technical centres.

In Portugal, the new national law concerning the buildings fire security has just been published (Portugal, 2008a). Concerning car parks, defined as a structure utilisation of 'type II', a distinction is done between open and closed car parks (see Table 1).

Table 1. Risks categories when the structure utilisation is of 'type II' (Car parks)

Category	Criteria's relative to an utilisation of 'type II', when integrate to the building structure			Open structure
	Height	Gross section	Number of storeys occupied by the type II utilisation below the reference level	
1st – Reduced risk	/			Yes
	≤ 9m	≤ 3 200m²	≤ 1	No
2nd – Moderate risk	≤ 28m	≤ 9 600m²	≤ 3	No
3rd – High risk	≤ 28m	≤ 32 000m²	≤ 5	No
4th – Very high risk	> 28m	> 32 000m²	> 5	No

The minimum fire resistance prescribed by the Portuguese code for structural elements of an open parking is R60 (Portugal, 2008b).

A two-storey composite open car park structure designed in Portugal is shown in Figure 1, and illustrates the typical Portuguese design of this type of structure. Dimensions and sections of elements are presented in Table 2. The car park is made of composite beams, steel columns, composite slabs and bolted connections.

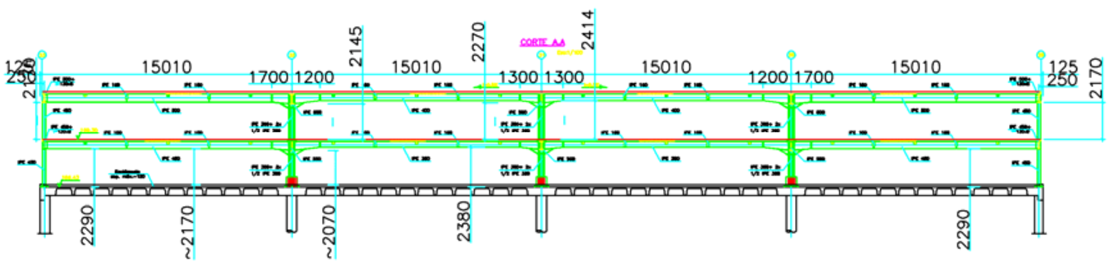

Figure 1. Elevation sight of the car park structure

Table 2. Typical dimensions and sections of a Portuguese open car park structure

Element	Steel or composite	Dimension	Section	Illustrations
Primary beam	Composite	Length = 7.2 m or 7.6 m	IPE 550 or IPE 600	Figure 2
Secondary beam	Composite	Length = 15.01 m (spaced of 2.4 m or 2.5 m)	IPE 450 or IPE 500	
External column	Steel	Free height ≈ 2.3 m	IPE 450	Figure 1, Figure 3
Internal column	Steel		IPE 360 + 2x ½ IPE 360	
Slab	Composite	Span = 2.4 m or 2.5 m	Thickness = 0.13 m	Figure 4

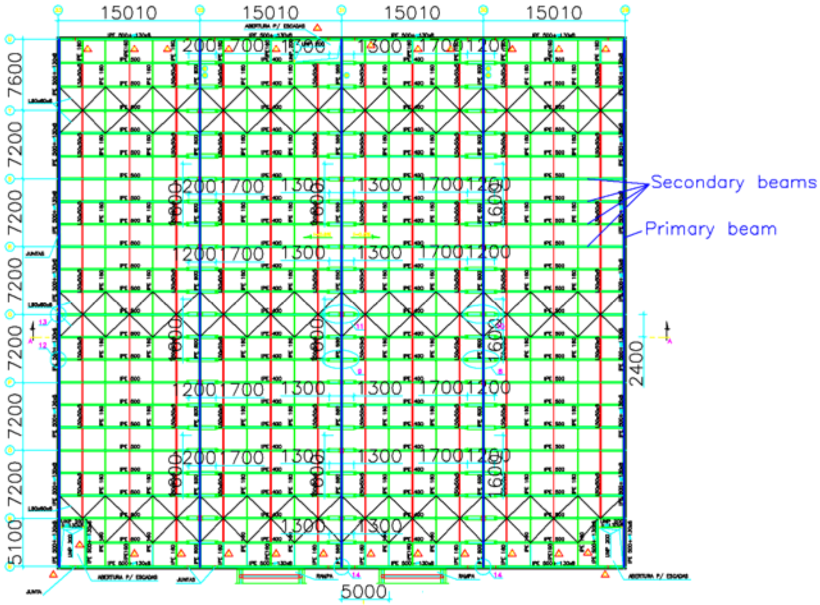

Figure 2. Plan of the car park structure – 2^{nd} floor

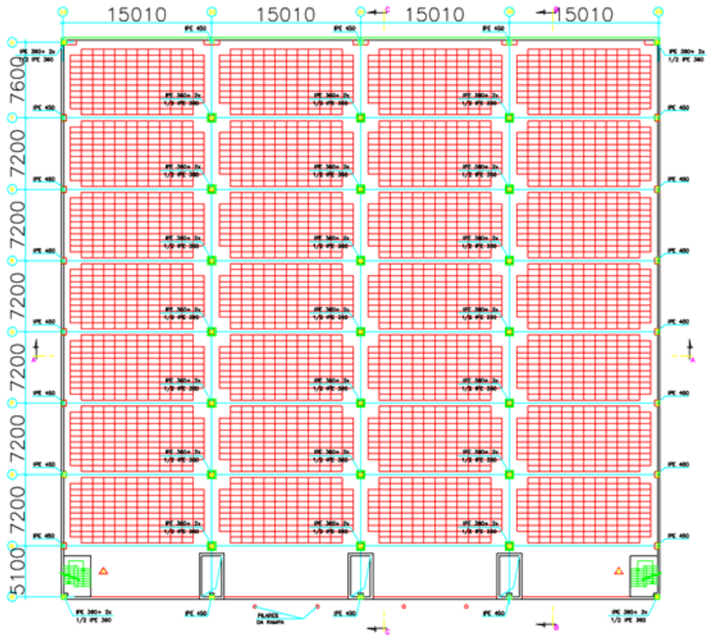

Figure 3. Plan of the car park structure – Position of the columns

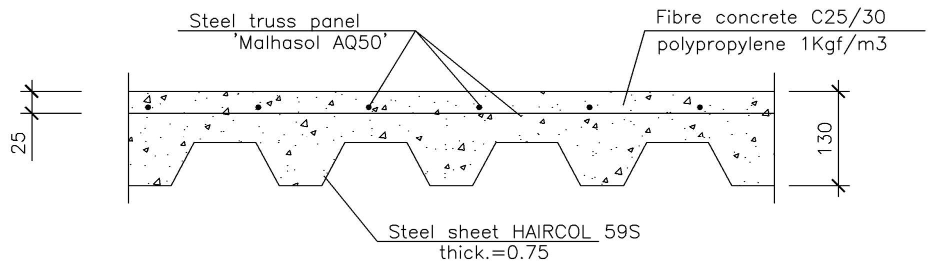

Figure 4. Composite slab section

The dimensions of the parking bay used in Portugal are 5.0 m long and 2.5 m wide as shown in Figure 5.

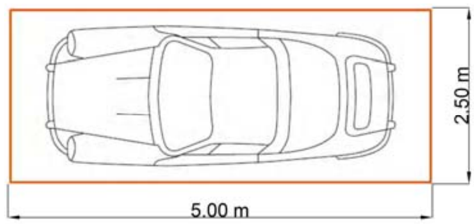

Figure 5. Parking bay dimensions in Portugal

V Conclusions

Founded on the information collected within the previous sections and, in particular, on the usual practices in Europe for car park structures, a typology of structure to be investigated within the project has been identified:

- Only open car park will be considered within the project;
- Two types of slabs will be contemplated: composite slab (steel sheet + concrete) and prefabricated slab elements;
- Simple steel connections will be considered for the beam-to-column joints although a semi-rigid behaviour of the latter should be observed as a result of the composite action;
- I profiles will be used for the beams (the cellular beams will not be considered as their behaviour when subjected to fire still be under investigation in other research projects);
- Steel H profiles for the columns will be used (composite columns will not be considered as this configuration of columns is less sensitive to a localised fire);
- Only braced buildings will be studied, what reflects the most usual configuration.

Concerning the scenario associated to the exceptional event to be considered, it has been decided to study localised fires leading to the progressive loss of column resistance; however, no particular scenario on how fire develops within the structure will be contemplated. It will be also assumed that the beam-to-column joints at the top of the loss column are subjected to fire action too, what will affect their mechanical properties.

According to these main decisions, the following philosophy to be followed within the project has been adopted:

- First, a reference building will be designed (based on the actual knowledge);

- Then, from this reference building, structural elements will be extracted in order to investigate the response of these individual elements numerically, analytically and experimentally (main objective of WP2);
- Founded on the knowledge gained from WP2, investigations on the global structural response will be conducted numerically and analytically (main objective of WP3);
- From the investigations made within WP2 and WP3, it will be then possible to derive design recommendations, what is the objective of WP4;
- Finally, the so-obtained design recommendations will be applied to an “actual” study case” in WP5. It is proposed, for the latter, to take, as “study case”, the reference building designed at the beginning of the project (design according to the actual knowledge) and to improve its design by applying the design recommendations developed within the present project.

For the design of the reference building, the following decisions have been taken (founded on the knowledge gained from the present work package):

- General layout and arrangement of beams and columns:
    - o It is decided to investigate structures with internal columns and what will be investigated, in a first step, is the loss of an internal column.
    - o It is proposed to place steel columns each 10m. The proposed layout for the reference frame is given here below. The primary composite beams are represented in green. The secondary composite beams are represented in blue. The steel columns are represented in red. The slab can be composite or made of prefabricated elements. For the first slab solution, more secondary beams are requested because the maximum span for a composite slab is 3,33m. However, it is possible to go up to 5m span with special deck systems (with a thicker slab); this solution will not be considered at a first step. Within the project, for the definition of the tested specimens, we will more focus on the composite solution while the prefabricated solution will be theoretically investigated.

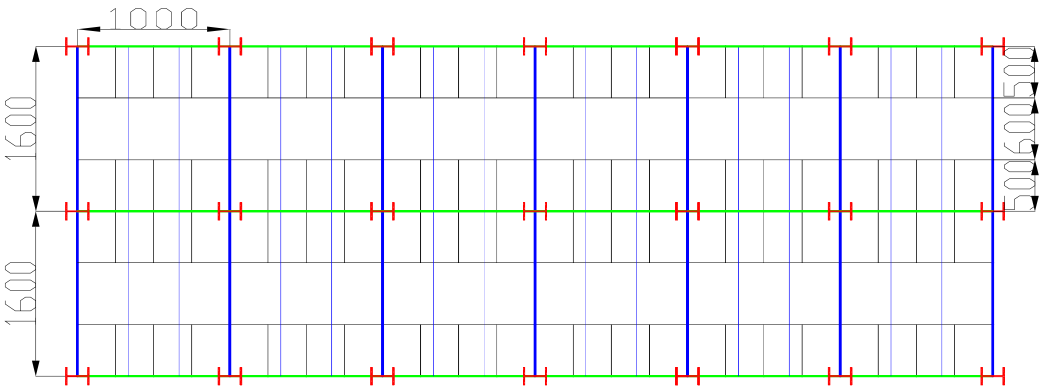

Figure 6. Chosen layout (dimensions in cm)

    - o The height of a storey is taken as equal to 3,0 m. It will be assumed that the building has 8 floors.
    - o It will be assumed that all the columns are the same within the building (i.e. one profile for the columns). Also, one profile for the secondary beams and one profile for the primary beams will be designed.
    - o For the bracing system, it is assumed that concrete ramps are each side of the building and are connected to the small façades. Accordingly, the horizontal displacements of the columns in the small façades can be assumed as blocked.
- Configuration of the joints

    - For the beam-to-beam joints (between the primary and secondary beams) and the minor axis beam-to-column joints, double web cleats will be chosen.
    - For the major axis beam-to-column joints, flush end plate solutions will be chosen.
    - For the column bases, hinges will be assumed.
- Slabs
    - As mentioned previously, two solutions for the slab will be investigated: composite slabs and slabs with precast elements.
    - The composite solutions with a span of 3,33m will be used to define the specimens to be tested in laboratory.
    - The beam-to-slab connection will be designed as a full strength one.
- Materials
    - S355 steel grade for beams
    - S355 and S460 steel grades for columns will be contemplated. Two solutions will be proposed and the solution for the tested specimens will be chosen according to the availability of the proposed profiles.
- Load cases and combinations (SLS and ULS)
    - The distributed load to be considered for the vehicles weight is 2,5 kN/m².
    - An estimated weight of safety barrier will also be put around the building.
    - For the combination of loads, the recommendations of the Eurocodes will be followed.
- Thermal action further to the event
    - This action will be defined more precisey later on within the project in the next WPs.

The design of this reference building will start during the next period of the project.

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ANNEX A – Open car parks subjected to a local fire

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ROBUSTFIRE Project

Document 1 – Car Parks – v1(11)

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EXECUTIVE SUMMARY

The increase of the market shares for steel and composite car parks in Europe is somewhat limited by the lack of information on how these structures behave under exceptional localised fire. In the present project, a general philosophy for the design of robust structures against exceptional events will be developed and practical design guidelines for its application to car parks under localised fire will be derived. In order to begin the project, a state-of-the-art relative to open steel car park structures and their resistance to a local fire is written in this report.

Previous or ongoing relevant projects are presented, relative to car fire tests and car park fire tests, but also to various individual aspects, like temperature distribution, joint and structural elements behaviour under natural fires, requirements for robustness, ductile joints, structural safety, … These previous or ongoing research works are completed by experimental fires in buildings or on sub-frames and by statistics of accidental fires in car parks in France and in New Zealand. Statistics about real fires in open car parks show that car fires have never been dangerous neither for the stability of the structure nor for the people. The fire stays local, the maximum number of cars involved in a fire being three cars. It results that most unprotected steel in open sided steel-framed car parks has sufficient inherent resistance to withstand the effects of any fires that are likely to occur.

The development of a localised car fire in a car park can be studied according to fire scenarios. In France, three basic scenarios were defined by the INERIS (Zhao et al., 2004) to design car park structures submitted to fire, including up to seven cars in a fire. The rate of heat release (RHR) curves were obtained from previous research works for different vehicle types. References RHR curves obtained by the CTICM tests in 1995-1996 (Schleich et al., 1999a) for a single class 3-car fire and three class 3-cars with a fire propagation time of 12 minutes are specially detailed in this report. The structural behaviour of the building can be studied by specific combinations for mechanical and thermal loadings in car park structures, defined in the EN 1991-1-1:2002 and the EN 1991-1-2:2002. The EN1993-1-2:2005 defines the rules to calculate the fire resistance of any structural element in car parks. The net heat flux to structural elements from each car fire is a function of the position, the height, the diameter of the fire source, the RHR and the distance from the element to the fire.

Only a few requirements are imposed in law concerning open car parks, as an unprotected steel structure becomes a generality in most countries, with eventually the condition of at least a partial connection between the steel beam and the concrete slab. Dimensions of the parking bay slightly vary from a country to another and according to the parking bays arrangement, but generally, for parking bays at a 90° angle against the traffic lane, dimensions are 2.5m wide and 5m long. As this type of structure is regular, typical dimensions of open car parks can easily be done. Some of them are presented, for some countries, at the end of this report.

TABLE OF CONTENTS

I Previous Research

I.1 Car fire tests

Car fires have been experimentally studied in several countries in the world. The Ph. D. Thesis of Li (2004) describes all these experiments. There are summarized hereafter, and RHR (Rate of Heat Release) curves resulted from some tests are showed in §III.2, Figure 35.

- ***Mangs and Keski-Rahkonen (1994a)***: See §I.4.1, *Fire safety in open car parks (ECCS, 1993)*.
- ***Shipp and Spearpoint (1995)***: Two full-scale calorimetric fire experiments on passenger cars carried out to obtain information on the consequences of a car fire in a shuttle train in the Channel Tunnel between England and France were realised in United Kingdom.
- ***Schleich et al. (1999a), Kruppa et al. (1998)*** for the CTICM tests: See §I.4.2, *Design rules for closed car park steel structures (1993 - 1996)*.
- ***Steinert (2000)***: Ten full scale car fire tests were realised at MFPA in Germany. Cars ranging from one to three in each test were put in a closed compartment measuring. The measurements for each test included temperatures, gas concentrations, heat fluxes, mass loss rate and RHR (Li, 2004).
- ***Stroup et al. (2001)***: Two fire tests were carried out in a 1995 passenger minivan with some exterior damage. The experiment was conducted under a calorimeter hood at NIST (National Institute of Standards and Technology) in the US (Li, 2004).
- ***General Motors***: i) Three fire tests were performed on crash tested vehicles at Factory Mutual test centre to obtain the behaviour of fire spread from the engine compartment into the passenger compartment. ii) Four fire tests were done on crash tested vehicles to study the spread of an underbody fuel pool fire into the passenger compartment. iii) Two full scale fire tests were carried out on two crash tested 1999 Chevrolet Camaros cars to investigate the effects of substituting plastic resins containing flame retardant chemicals in the HVAC (heating, ventilation, and air conditioning) module.

  The measurements taken for each test included air temperatures, heat fluxes and combustion gas concentrations all in the passenger compartment, as well as the RHR (Li, 2004).
- ***Small scale tests***: A series of tests were realised at NIST in the US in order to assess the fire hazard after a motor vehicle accident.

  Then 18 exterior automotive parts from a passenger van and a sport coupe were tested in a cone calorimeter by Janssens et al. (2004). Based on these test data, a model was shown to predict fire growth, for certain materials in post crash vehicle fires originating in the engine compartment (Li, 2004).
- ***Ingason (2001) and Shipp (2002)***: Fire tests of all types of vehicles were realised by Ingason (2001) and Shipp (2002) in order to measure the RHR of each vehicle (small passenger car, van, truck, bus, train,…). The RHR curves

from Ingason (2001) were for the condition of road tunnel ventilation (Li, 2004).

I.2 Open car park fire tests

I.2.1 Butcher et al. (1968) – UK

Three car fire tests were realised in a specially built steel scaffolding structure with an insulated ceiling approximately 2.1m above the floor (ECCS, 1993; Li, 2004). During the two firsts tests, the two ends of the structure were left opened. Nine cars in a three by three array were arranged with parallel spacing's ranging from 0.75 m to 1.2 m (Figure 1).

Figure 1. Fire test in the scaffolding structure – Butcher et al. (1968)

The central car was ignited, but the fire did not spread to any of the adjacent cars.

Maximal measured temperature:

- 840ºC in the air,
- 360ºC in the steel column,
- 275ºC in the steel beam.

Conclusions:

- A fire single parked vehicle is unlikely to cause uncontrollable fire spread within a car park;
- The damage to the car park building is not critical.

The wood equivalent fire load density for a car park was found to be 17 kg/m^2.

I.2.2 Nippon Steel (1970) – Japan

Five fire tests were realised in a steel framed parking building (ECCS, 1993).

Maximal temperature measured in the unprotected steel members:

- 245ºC in beam,
- 242ºC in column.

I.2.3 Gewain (1973) – USA, Pennsylvania

A full scale car fire test was realised in the multi-storey open car park showed in Figure 2, with unprotected steel frames and concrete decks (ECCS, 1993; Li, 2004).

Three cars were arranged, and the central one was ignited.

Figure 2. Open car park in Pennsylvania – Gewain (1973)

Thermal and structural results:

- Max. temperature of the air = 432ºC (above windscreen, after 11 minutes)
- Air temperatures for most parts in the building: < 204°C
- Max. temperature of the steel = 226ºC
- Deflection and elongation of elements = 0 after cooling.

Conclusions:

- The fire did not spread to any of the adjacent cars during the 50 minutes of test;
- Low fire hazard in an open air parking structure;
- Steel provide adequate safety against the structural collapse under a car fire;
- Confirm results of Butcher et al. (1968), § I.2.1.

The wood equivalent fire load density for a car park was found to be 9.8 kg/m^2.

I.2.4 Bennetts et al. (1985) – Australia

Two fire tests were realised in the two-level open-deck car park showed in Figure 3, with unprotected steel and concentred loads on the first floor. Five cars were arranged (ECCS, 1993; Li, 2004).

- First test: the fire did not spread to any of the adjacent vehicles. Maximal temperature in the steel: 285ºC.
- Second test: 3 cars involved, the fire spread from the car first ignited to two neighbouring cars at 14 and 35 minutes. Maximal temperatures: 340ºC in a beam and 320ºC in a column.

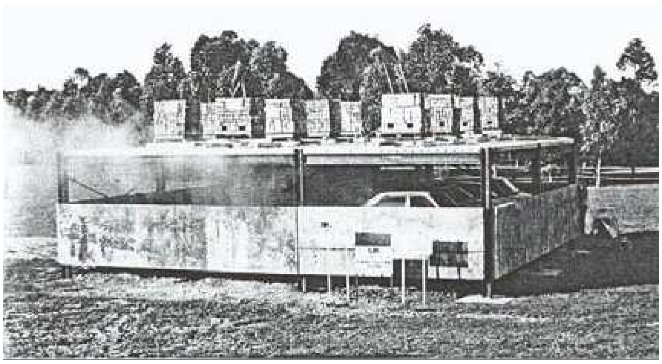


Figure 3. Open-deck car park – Bennetts et al. (1985)

Conclusions:

- The probability to involve more than 2 cars is very small (fire brigades arrive before);
- Security assured with unprotected steel.

I.2.5 CTICM (2000) – France, Vernon

Three car fire tests were realised in a two-level braced car park by the CTICM, with unprotected steel and concrete slabs (Anon, 2000; CTICM, 2000; Joyeux and Kruppa, 2000a, 2000b; Joyeux et al., 2002; Li, 2004; Zhao and Kruppa, 2002, 2004):

- Preliminary test (Test 1 - Figure 4): 3 cars involved, the central one is initially ignited;
- Demonstration test (Test 2 - Figure 4): 3 cars involved, the central one is initially ignited. More wind action than during the first test (2m/s);
- Fire propagation test (Test 3 - Figure 5): 2 cars parked on parking bays located in front.

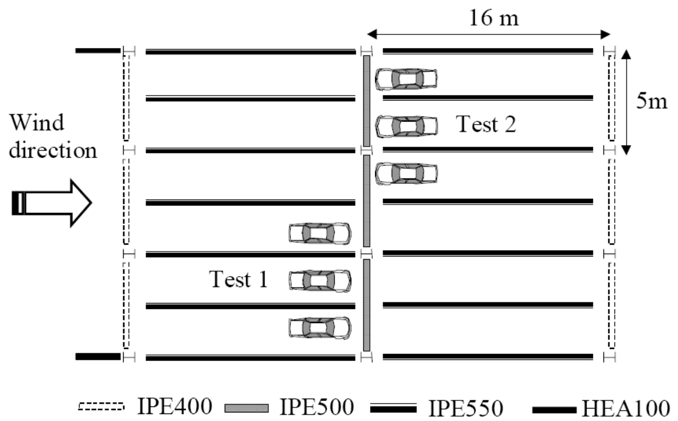


Figure 4. Fire scenarios for the tests 1 and 2 – Zhao and Kuppa (2002)

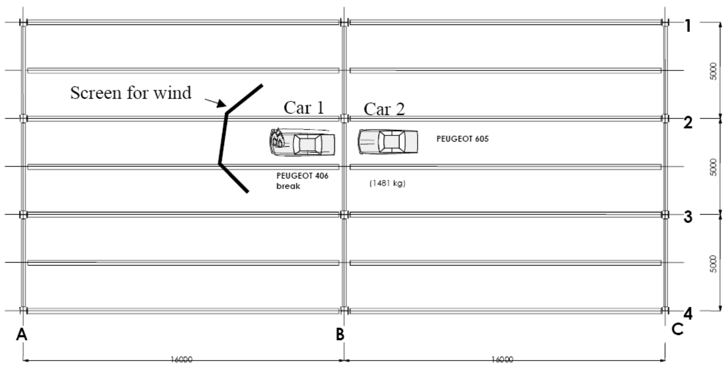


Figure 5. Fire scenario for the test 3: the fire propagation test – Joyeux et al. (2002)

I.2.5.1 Structure dimensions and connection types

The two-level braced steel car park structure is designed according to the worst fire scenario and has the following dimensions (Figure 6 and Figure 7):

- 15m width x 32m long x 3m height;
- Central columns: HEB 200 (line B);
- External columns: HEA 180 (lines A and C);
- Beams lines A and C: IPE 400 with 20 stud shear connectors of φ 19x100;
- Beams line B: IPE 500 with 20 stud shear connectors of φ 19x100;
- Beams lines 2 and 3 (secondary beams): IPE 550 with 52 stud shear connectors of φ 19x100;
- Beams lines 1 and 4 (secondary beams): IPE 550 with 34 stud shear connectors of φ 19x100;
- Composite slab, with a steel sheet of COFRASTRA 40 and a concrete slab of total height 12cm (Figure 8).

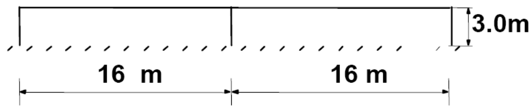


Figure 6. Frame dimensions– Joyeux et al (2002)

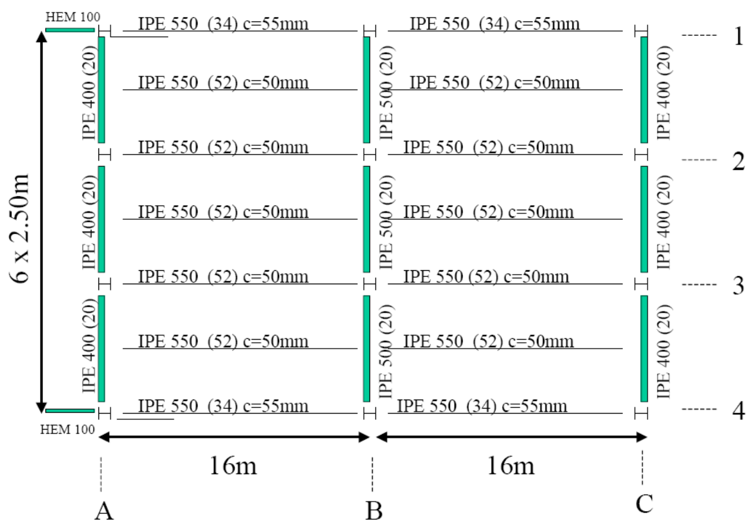


Figure 7. Unprotected steel car park dimensions – Joyeux et al (2002)

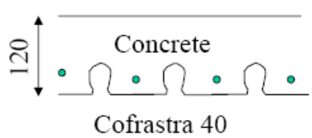


Figure 8. Concrete slab – joyeux et al (2002)

Connection types are the following:

- The columns are hinged at the bottom;
- For the beams, three types of solution have been analysed: rigid connections between columns and beams (connections with steel end plate), hinged connections (connections by angles) and semi-rigid connections (connections by angles, horizontal stiffeners and additional rebars in the slab) between secondary beams and main beams.

I.2.5.2 Preliminary test (Test 1)

Thermal results:

- Maximal air temperature under the ceiling: 1000ºC (between 45 and 60 min), with an average temperature on 15 min equal to 500ºC.
- The gradient temperature in the beam is upper than 200ºC, with a max. temperature in the lower flange of 700ºC.
- Max. temperatures in columns: 640ºC for the exposed flange and 500ºC for the unexposed flange.
- Max. temperatures in slab: 600ºC for the steel, 360ºC for steel in the rib, 260ºC at 25mm in the concrete, 120ºC at 50mm in the concrete, 100ºC at 70mm in the concrete.

Structural results:

- Cambering of the two exposed beams (frame beam and secondary beam): 2.5 cm after the cooling phase, and a maximal deflection of 0.4% of the length (span of 16m).
- Local instabilities in these two beams: lateral buckling and flange instabilities.
- Rigid connection damages: break of 6 bolts (out of 8) at the level of endplate of the main beam (but no risks of car park instability).

I.2.5.3 Demonstration test (Test 2)

Thermal results:

- Maximal air temperature under the ceiling: 1000ºC, with an average temperature on 10 min equal to 600ºC.
- The gradient temperature in the beam is upper than 270ºC, with a max. temperature in the lower flange of 640ºC.
- Max. temperatures in columns: 300ºC for the exposed flange, and 209ºC for the unexposed flange.
- Max. temperatures in slab: 191ºC for the steel, 177ºC for steel in the rib, 163ºC at 25mm in the concrete, 112ºC at 50mm in the concrete, 100ºC at 70mm in the concrete.

Structural results:

- Cambering of the two exposed beams (frame beam and secondary beam): 4 cm after the cooling phase, and a maximal deflection of 1% of the length (span of 16m).
- Local instabilities in these two beams: lateral buckling and flange instabilities.
- Rigid connection damages: break of 3 bolts (out of 8) at the level of endplate of the main beam (but no risks of car park instability).

Numerical simulations of the demonstration test (Joyeux et al., 2002):

- Results of a comparison of the accuracy of a 3D simulation (ANSYS) with a 2D simulation (SISMEF): the 3D modelling is more appropriate for predicting the structural fire behaviour of composite floor system than 2D simulations.

- Results of a parametrical study (additional reinforcement for secondary beams; joint condition between secondary beam and columns): the contribution of additional reinforcing steels for secondary beams is quite small under this fire scenario since the displacement difference is only about 7 %, and the difference is very small between rigidly and simply jointed structure.

I.2.5.4 Fire propagation test (Test 3)

The ignition of the second car was only after 56 minutes, due to the 37 minutes needed for the propagation in the cabin of the first car. The maximal temperatures in elements were lower than 200ºC, leading to low measured vertical and horizontal displacements of the steel structure.

I.2.5.5 Conclusions

The fire engineering methodology is sufficiently safe. In the second test, weather conditions (strong winds) clearly accelerated the fire development in comparison with the first one, but the effect was compensated by a simultaneous reduction of the thermal actions to the loadbearing structure. Then the fire spread from one car to another and the measured maximum vertical displacement were quite different for the two first tests, but deformations were far from leading to the collapse of the structure.

The Fire propagation test (Test 3) didn't lead to interesting conclusions.

I.2.6 Conclusions from the open car parks fire tests

Variation of temperatures in steel elements according to experiment results:

Table 1. Fire tests in various countries

Full scale fire tests	**Maximum measured steel temperature**	
	Beam	**Column**
UK, 1968 (Butcher et al., 1968)	275 ºC	360ºC
Japan, 1970 (ECCS, 1993)	245ºC	242ºC
USA, 1973 (Gewain, 1973)	226ºC	-
Australia, 1985 (Bennetts et al., 1985)	340ºC	320ºC
France, 2000 (Joyeux et al., 2002)	700ºC	640ºC

By comparing the test results and the limiting temperatures at which failure is expected to occur, which are 620℃ for beams carryi ng floor slabs and 550℃ for columns according to Corus (2004), the four first tests indicate a large security, but the French test obtained temperature values upper than the limiting ones. However, no risks for the stability of the structure appeared even with some bolt failures.

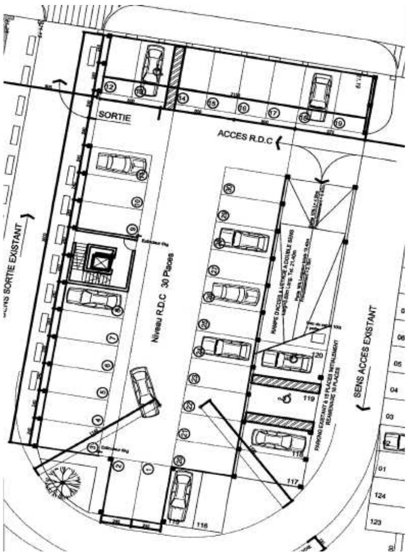
In conclusion, most unprotected steel in open sided steel-framed car parks has sufficient inherent resistance to withstand the effects of any fires that are likely to occur.

I.3 Numerical and analytical studies in car parks

I.3.1 Joyeux et al (2001) – France, Lille

A numerical study of the car park fire behaviour in Lille was realised in 2001. Dimensions, and steel columns and composite beams sections of the car park are given in Figure 9 and detailed hereafter:

- 16m wide x 37m long;
- 3-storey frames each 2m40 with steel columns and composite beams;
- S355 steel;
- IPE 400 for beams;
- HEB 240 for columns;
- C45/50 for concrete slabs, with a thickness of 100mm.



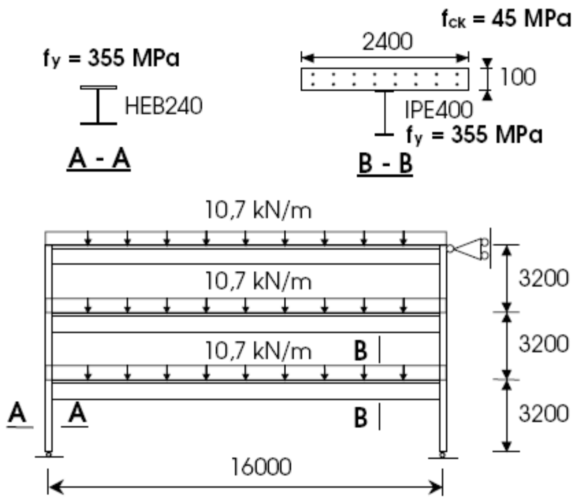
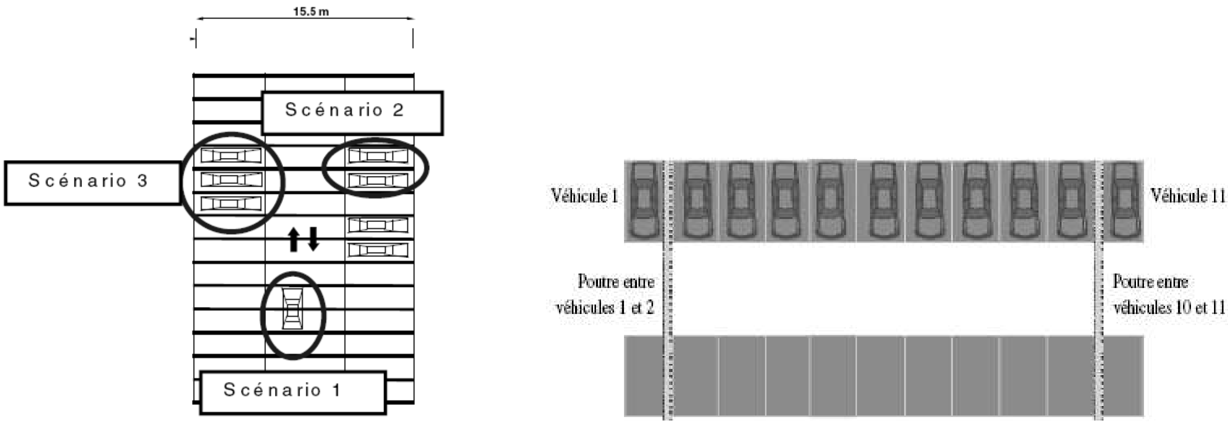


Figure 9. Composite steel-concrete car park in Lille, France – Joyeux et al. (2001)

Four scenarios using class 3-cars are simulated with the finite element program SISMEF and the software TASEF (Figure 10):

- Fire scenario 1: One car of class 3 situated under a steel beam;
- Fire scenario 2: Fire of 2 cars of class 3;
- Fire scenario 3: Fire of 3 cars of class 3;
- Fire scenario 4: Fire of 11 cars of class 3, more than 2 hours of fire.

Table 2 indicates the classification of cars.



(a) (b)

Figure 10. The 3 first scenarios of car fire (a) and the fire scenario 4 (b) – Joyeux et al. (2001)

Table 2. Classification of cars – Joyeux et al. (2002)

Type	Class 1	Class 2	Class 3	Class 4	Class 5
Peugeot	106	306	406	605	806
Renault	Twingo-Clio	Mégane	Laguna	Safrane	Espace
Citroën	Saxo	ZX	Xantia	XM	Evasion
Ford	Fiesta	Escort	Mondeo	Scorpio	Galaxy
Opel	Corsa	Astra	Vectra	Omega	Frontera
Fiat	Punto	Bravo	Tempra	Croma	Ulysse
Wolkswagen	Polo	Golf	Passat	//	Sharan

Structural conclusions: No risks for the stability of this car park under these four scenarios.

Thermal conclusions: a comparison is done between the numerical results and the Hasemi's computing method, and shows that computed temperatures are always upper than measured temperatures.

I.3.2 ArcelorMittal (2007) and CTICM (Zhao et al., 2004) – France

ArcelorMittal studied numerically the behaviour of seven car park structures and presented pre-studied results on ordinary conceptions of aerial car parks. Analyses were done at ambient temperature and at elevated temperature.

The dimensions of the seven structures studied are summarized in the Table 3, with the following characteristics:

- Secondary beams: length from 7.5m up to 16m, spaced from 2.5m to 5m, steel profile types IPE, HEA or HEAA, S355 steel, connected to the slab, hinged at the extremities;
- Primary beams: length from 5m to 10m, steel profile types IPE, HEA or HEAA, S355 steel, connected to the slab, hinged at the extremities;
- Steel columns HEB or HEM S355 or S460, partially-encased section (the concrete does not take part in the mechanical stability of the column, it serves only as a fire protection of the web);

- Concrete or composite steel-concrete slabs, with at least a general reinforcement grid (diameter 7/150/150);
- Full connection between the slab and beams;
- 5 common levels more one roof level.

The first structure is showed at Figure 11.

Table 3. Summary of the seven cases studied – ArcelorMittal (2007)

Case	Clearance under beam	SB* length	Nº of cars between SB*	SB* spacing	Columns spacing	PB** length	Thick. of the slab
1	2.1m	16m	1	2.5m	10m	10m	12cm
2	2.1m	16m	2	5m	10m	10m	16cm (6+10)
3	2.6m	15m	1	3.33m	10m	10m	12cm
4	2.1m	16m	1	3.33m	10m	10m	12cm
5	2.1m	16m	2	5m	5m	-	16cm (6+10)
6	2.1m	7.5m	1	2.5m	7.5m	7.5m	12cm
7	2.1m	16m	2	5m	5m	-	20cm (6+14)

* SB: Secondary beam, ** PB: Primary beam

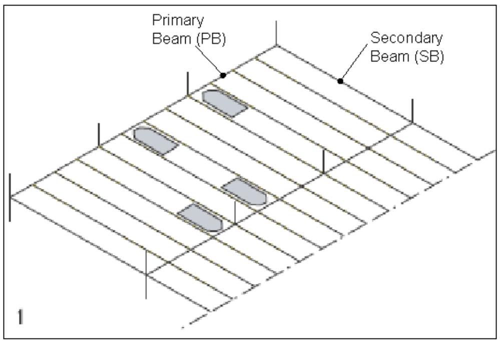
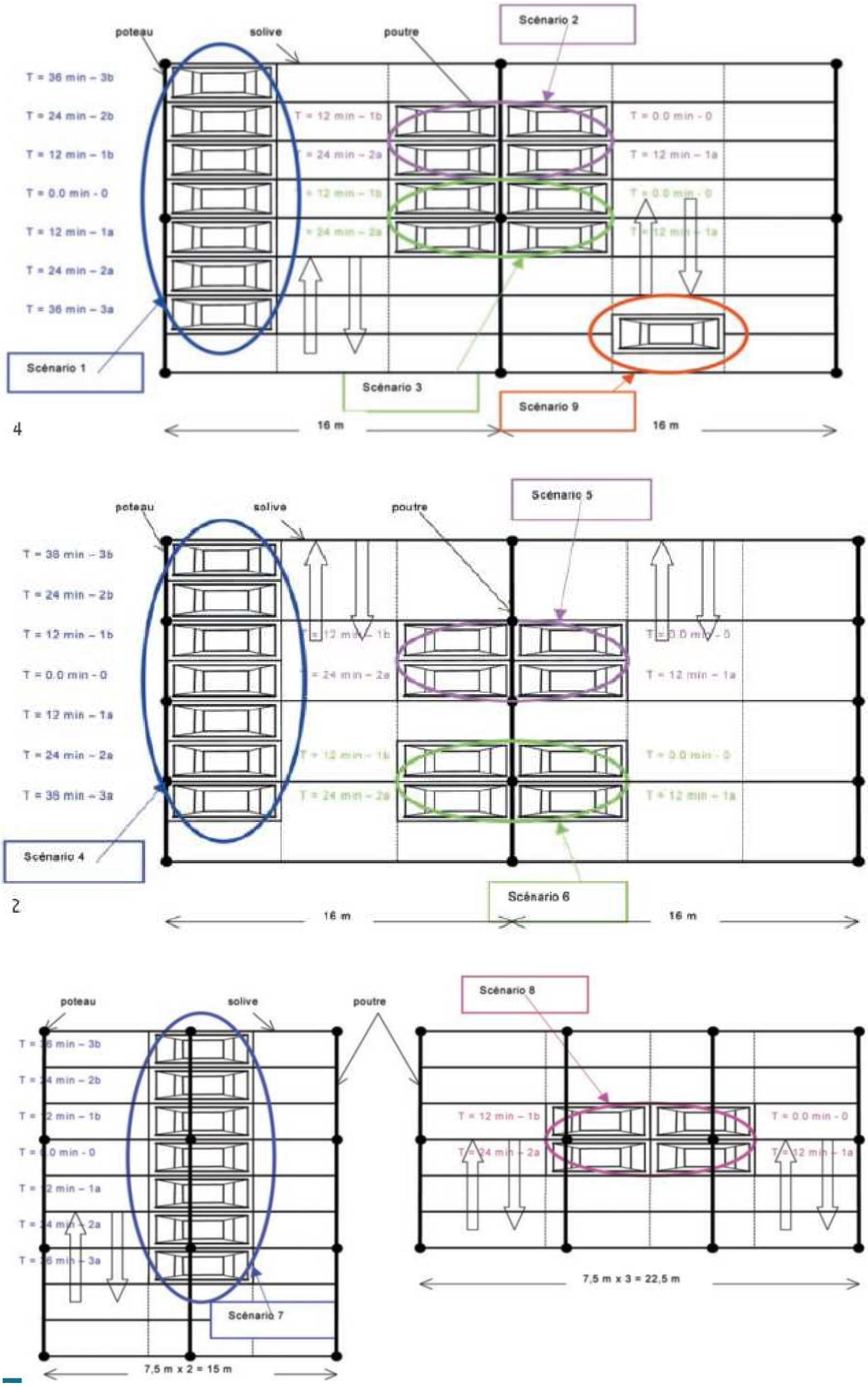


Figure 11. Schema of the structure number 1

Concerning the analysis at elevated temperatures, the INERIS propose 3 basic fire scenarios (Cwiklinski, 2001) that were used during this study. These 3 scenarios, using eventually a utilitarian vehicle, which is a small van full of inflammable products (19.5GJ, 18MW), were applied to the seven structures and declined into the 9 scenarios showed in Figure 12.

The synthesis of the analysed cases and the results are presented in Table 4. Steel column sections are given with the assumption of a partially-encased section. The structure stays stable and deflections disappear after the cooling phase. The maximal deformation of tensile steel rebars is of 4.45% and is lower than the maximal limit value of 5%. The maximal displacement of the floor is equal to 680 mm for the structure dimensions of 16.0 x 10.0 m and 15.0 x 10.0 m. For all beams and slabs, the deflection is lower than 1/20 of the length.



Scenario 1: fire propagation to 7 vehicles of class 3 to analyse primary and secondary beams,
Scenario 2: fire propagation to 4 vehicles of class 3 around a principle beam,
Scenario 3: fire propagation to 4 vehicles of class 3 around a column,
Scenarios 4, 5, 6 are equivalent to the scenarios 1, 2 and 3 but for other dimensions,
Scenario 7: fire propagation to 7 vehicles of class 3 to analyse primary beams and columns,
Scenario 8: fire propagation to 4 vehicles of class 3 around two columns,
Scenario 9: One utilitarian vehicle burning at mid-span under the beam.

Figure 12. ArcelorMittal Scenarios (2007)

Table 4. Synthesis of analysed cases – Zhao et al. (2004)

Data of the structure and the scenarios									Results			
Case	Structure dimensions; type and thick. of slab (m)	SB spacing (m)	Storey	Scenarios		Steel profiles	Concrete	Steel rebars R500: upper/lower rebar layer	Time (s)	Displ. (m)	Deformations of steel rebars	
											SB direction (%)	PB direction (%)
1	16.0 x 10.0 x 2.10 Cofraplus 60, 12cm	2.5	Common	S1	6C* + 1U**	SB***: IPE 450 (S355)	C30/37	HA7 150 x 150 (mesh of diameter 7, 150 x 150)	1920	440	0.85	1.55
				S2	3C + 1U	PB****: HEA 500 (S355)			1920	525	2	0.85
				S3	3C + 1U	Column: HEB 240 (S355)			1920	610	2.45	2.8
			Roof	S1	6C + 1U	SB: IPE 500 (S355)			Covered by the case 4			
				S2	3C + 1U	PB: HEA 550 (S355)						
				S3	3C + 1U	Column: HEB 240 (S355)						
2	16.0 x 10.0 x 2.10 Concrete planks, 16cm (6+10)	5.0	Common	S1	6C + 1U	SB: IPEA 600 (S355)	C30/37	HA7 150 x 150 / HA7, spaced of 100mm	1920	605	2.1	3.4
				S2	3C + 1U	PB: HEAA 650 (S355)			1920	610	2.5	1.9
				S3	3C + 1U	Column: HEM 300 (S355)			1860	575	2.85	3.7
			Roof	S1	6C + 1U	SB: HEAA 650 (S355)			1980	650	2.9	3.1
				S2	3C + 1U	PB: HEAA 700 (S355)			1920	670	2.1	1.9
				S3	3C + 1U	Column: HEB 240 (S355)			1920	**680**	3.4	**4.45**
3	15.0 x 10.0 x 2.60 Cofraplus 60, 12cm	3.33	Common	S1	6C + 1U	SB: IPE 500 (S355)	C30/37	HA7 150 x 150	2400	315	1.3	1.4
				S2	3C + 1U	PB: HEA 500 (S355)			1980	460	1	1.5
				S3	3C + 1U	Column: HEB 240 (S355)			1920	340	1.4	1.85
				S9	1U				900	220	0.5	1.1
			Roof	S1	6C + 1U	SB: IPE 500 (S355)			Covered by the case 4			
				S2	3C + 1U	PB: HEA 550 (S355)						
				S3	3C + 1U	Column: HEB 240 (S355)						
4	16,0 x 10,0 x 2,10 Cofraplus 60, 12cm	3.33	Common	S1	6C + 1U	SB: IPE 500 (S355)	C30/37	HA7 150 x 150	1920	395	0.9	1.5
				S2	3C + 1U	PB: HEA 500 (S355)			1920	450	1.45	1.2
				S3	3C + 1U	Column: HEM 300 (S460)			1980	510	1.85	2.4
			Roof	S1	6C + 1U	SB: IPE 550 (S355)			1920	410	1	1.6
				S2	3C + 1U	PB: HEA 550 (S355)			1980	515	1.7	1.25
				S3	3C + 1U	Column: HEB 240 (S355)			1920	620	2.5	3
5	16.0 x 5.0 x 2.10 Concrete planks, 16cm (6+10)	5.0	Common	S4	6C + 1U	SB: IPEA 600 (S355)	C30/37	HA7 150 x 150 / HA7, spaced of 100mm	1920	**480**	1.5	2.9
				S5	3C + 1U	PB: IPE 400 (S355)			1920	385	1.6	1.65
				S6	3C + 1U	Column: HEB 240 (S460)			1920	355	1.75	1.45
			Roof	S1	6C + 1U	SB: HEAA 650 (S355)			Covered by the case 2			
				S2	3C + 1U	PB: IPE 450 (S355)						
				S3	3C + 1U	Column: HEB 240 (S355)						
6	7.5 x 7.5 x 2.10 Cofraplus 60, 12cm	2.5	Common	S7	6C + 1U	SB: IPE 240 (S355)	C30/37	HA7 150 x 150	1860	425	2.9	3.3
				S8	3C + 1U	PB: IPE 400 (S355) Column: HEB 340 (S355)			2880	325	1.55	1.1
			Roof	S7	6C + 1U	SB: IPE 270 (S355)			1800	270	1.2	1.05
				S8	3C + 1U	PB: IPE 450 (S355) Column: HEB 160 (S355)			2940	**430**	3	2.95
7	16.0 x 5.0 x 2.10 Cofradalle 200, 20cm (6+14)	5.0	Common	S4	6C + 1U	SB: HEAA 650 (S355)	C30/37	SB: 4HA25	No failure			
				S5	3C + 1U	PB: IPE 400 (S355)		PB: 4HA12	Covered by the case 5			
				S6	3C + 1U	Column: HEB 400 (S460)		HA6 200x200	Covered by the case 2			
			Roof	S1	6C + 1U	SB: HEAA 700 (S355)		Cofradal 200: HA12	No failure			
				S2	3C + 1U	PB: IPE 450 (S355)		SB: 6HA25	Covered by the case 5			
				S3	3C + 1U	Column: HEB 240 (S355)		PB: 4HA12	Covered by the case 2			

* C: Car of class 3, ** U: Utilitarian vehicle, *** SB: Secondary beam, **** PB: Primary beam

Types of usual beam-to-column and beam-to-beam connections are showed in Figure 13.

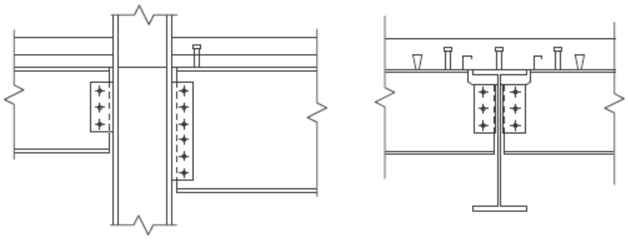


Figure 13. Two types of usual connections: beam-to-column and secondary beam to primary beam connections – ArcelorMittal (2007)

I.4 EC, ECSC and RFCS relevant projects

I.4.1 Fire safety in open car parks (ECCS, 1993)

The production of this document was to clear up differences between fire safety requirements of open car parks for all European countries, according to information and test results available throughout the world. Additional tests were performed at the VTT Fire Technology Laboratory of the Technical Research Centre of Finland (Mangs and Keski-Rahkonen, 1994a). Three full-scale fire tests on passenger cars from the 1970’s were realised in a simulated open car park. RHR curves, rate of mass change, gas temperature above the car and temperatures inside the car… were determined as a function of time.

→ RHR of a burning car;

→ Deduction of a “car fire model”. This model, combined to the air temperature calculation method, allowed obtaining the air temperature field around the structure;

→ Analyse of the steel temperature and the structural behaviour during the localised fire by numerical simulation using CEFICOSS and TASEF. It was resulted that no failure of the unprotected steel temperature will occur, provided that the sections of the beams have a composite behaviour.

I.4.2 Design rules for closed car park steel structures (1993 - 1996)

The purpose of this project was to develop a design guide for closed car park structures submited to localised natural fires, and therefore to change the regulation and to etablish more realistic standards in Europe (Schleich et al., 1999a):

→ Definition of the fire: one burning car or a wave of burning cars for which a RHR curve can be defined. Ten full-scale calorimetric fire experiments on old and recent European cars were realised by the CTICM in 1995 and 1996. A part of a closed car park has been simulated, and can accommodate two cars (Li, 2004);

→ Design of the ventilation to evacuate the CO produced by the running cars and the fumes in case of fire;

→ Air temperature calculation methods that provide the thermal action on the structure for different fire scenarios;

→ Thermo-mechanical calculation of the structure;

→ Design rules enabling steel structures to survive the fire;

→ Numerical thermo-mechanical simulations of a parking structure.

I.4.3 Design rules for large compartments steel structures (1993 - 1996)

The aim of this report is to provide new requirements which correspond in a better way to a real fire effect for large compartments (Schleich et al., 1999b):

→ Proposition that ISO requirement F30, F60, F90, F120 be replaced by the requirement “no failure at all”, which demonstrates an increase of safety;

→ Prediction that a structure can survive the required fire defined by its size and its rate of heat release (RHR);

→ Procedure to check whether the fire remains localized and to calculate the temperature field in a steel structure in that case;

→ Mechanical behaviour analyse using the Fire part of Eurocode 3;

→ New formula for a column situated in a 2 Zone environment;

→ New tool called TEFINAF providing the steel temperature field of floor beams for any type of localized fire.

I.4.4 NFSC 1 (1994 – 1998) and NFCS 2 (1999 - 2000)

The main aim of this project is to establish a more realistic and more credible approach for analysis of structural safety in case of fire that takes account of active fire-fighting measures and real fire characteristics (Schleich et al., 2001, 2002a, 2002b). This natural fire safety concept leads to financial benefits and better safety guidance.

I.4.5 NFSC 3 (1999 - 2002)

The objective of this project (Natural fire safety concept - the development and validation of a CFD-based engineering) is the development of an engineering methodology, exploiting the advanced capabilities of computational fluid dynamics (CFD), for determining the thermal behaviour of structural elements in steel/composite-framed buildings subjected to natural fires (Kumar et al, 2005).

→ An embedded mesh solver was developed and implemented into the SOFIE CFD code to accurately determine heat conduction within composite solid components;

→ A systematic and progressive validation of the CFD-based methodology was undertaken to ensure confidence in, and the robustness of, the procedure;

→ The heat transfer parameters pertaining to the simple models defined in the structural Eurocodes (EC1, EC3) were reviewed;

→ Recommendations were presented on equivalent parameters derived from the CFD simulations. In some situations, the theoretical ambiguities of the basic governing heat transfer equation may lead to significant errors in the calculated heat flux.

I.4.6 Real fire tests in car parks and high buildings (1998-2001)

The objective of this project was to confirm that open and closed car parks, and high buildings don’t need a large duration of fire resistance requirements, and it is generally not necessary to insulate the steel section (beam and/or columns) to obtain a satisfactory safety level. Demonstrated experimental fire tests and numerical simulations were performed in three different configurations (Joyeux et al., 2002):

- A large (high) compartment with a local fire (2 Fire tests realised in 1998);
- An open car park built in France with a fire scenario involving several cars (3 fire tests realised in 2000, see §I.2.5);
- A closed car park in The Netherlands with a fire scenario involving several cars (2 fire tests realised in 2001).

Statistics on the real fires in open and closed car parks mainly in Paris, but also in others big cities of Europe, and additional car fire tests under calorimetric hood, lead to improve and confirm the open car park design fire corresponding of class-3 car fires.

I.4.7 PRECIOUS (2002 - 2007)

Precious considers the situation of an earthquake immediately followed by significant conflagrations (Bursi et al., 2008). The purposes of the project are to develop fundamental data, design guidelines and prequalification of two typologies of ductile and fire-resistant composite beam-to-column joints with columns composed of partially reinforced-concrete-encased I-section steel profile, or of concrete filled steel tube.

The joints will be the result of a multi-objective advanced design able to guarantee structural, seismic and fire safety and will be conceived as prefabricated components to be cost-effective both from a designer and from an industrial viewpoint.

Finally, to optimise prefabrication processes, concrete slabs, reinforced by high ductile bars and electrowelded meshes, will be alternatively cast with profiled steel sheeting or electrowelded lattice girders.

→ Definition and design of configurations of joints for seismic and fire loadings;

→ Analysis of the mechanical and thermal behaviour of the chosen typologies and extension of the numerical investigation to other types of precast structural assemblies;

→ Derivation of design procedures and promotion of the invesigated joint solutions;

→ Conception of the joints as prefabricated components to be cost-effective, using slabs with high ductile rebars, electrowelded meshes, and electrowelded lattice girders or profiles steel sheetings;

→ Enhancement of seismic design procedures relevant to partially restrained and fully restrained composite joints and moment-resisting frames included in EC 3 (Eurocode 3), EC 4 and EC 8;

→ Moment-rotation characteristics at elevated temperatures, temperature distributions and the relevant mode of failures for the connection types studied;

I.4.8 ROBUSTNESS – Robust structures by joint ductility (2004 - 2007)

The objective of this project is to encourage and to promote the wider use of composite and steel frames by increasing the robustness of structures (Kuhlmann et al., 2008). Progressive failure of the whole structure caused by local damage (e. g. failure of a column caused by a vehicle impact, explosion, fire, earthquake …) can be prevented by robust design. Profiting from the inherent ductile behaviour of steel, this project analyses the requirements for robustness and develops new ductile joint solutions to allow for force redistribution within the structure so that a global collapse of the building is inhibited and structural safety is ensured. A particular attention is paid to the exceptional situation “loss of a column further to an impact”; criteria for robust structures, especially concerning steel and composite joints are elaborated. In particular, the behaviour of joints under combined bending moment and tension load is investigated in details. Within this project, analytical, numerical and experimental approaches have been used to achieve the objective of the project.

→ Defining requirements for ductile steel and composite joints and sections,

→ Developing new ductile joint solutions,

→ Experimental investigations: full scale test of a substructure for the event “loss of a column”, joint tests and component tests,

→ Numerical simulations realised on the behaviour of the joints under large deformations and combined loading of bending and tension.

I.4.9 COSSFIRE (2006 - 2009)

The main aim of this project is to enhance the scientific findings and to develop efficient, practical and economic design rules on steel and steel and concrete connections when exposed to real fire conditions (Zhao et al., 2007). The work programme of this project includes four principle tasks:

→ A detailed bibliography analysis of both test results and calculation models on connections under standard fire conditions. In addition to the collection of existing data, a review of commonly used types of steel joints over both UK and French markets is conducted.

→ Some tests under natural fire heating conditions on connection components, on full structural steel joints and on connections between concrete slabs and border steel members in case of composite steel members:

- An experimental investigation of the behaviour of different connection components under natural fire conditions has been done. 40 fire tests of bolts have been conducted up to now. The fire tests of welds will start very soon.
- 8 fire tests of connections will be done: 4 on steel connections and 4 on steel-concrete composite connections. The first two fire tests on steel connections have already been done. The experimental results of the Coimbra tests (Santiago, 2008) have been taken into account to design the connection tests predicted and a comparison of the tests results has been done.
- The fire behaviour of connections between concrete slab and steel members at the border of composite floor will be studied by a global floor test, including four different edge connection configurations, realistic restrained effects applied to connections with used global floor and a uniformly distributed load.

→ Numerical simulations of existing available experimental data on steel connection as well as the fire tests to be performed within the scope of both this project and the Precious project (§ I.4.7);

→ Development of simple design rules as well as practical design guidance on various commonly used types of connections for fire situations.

This project is still ongoing and should be finished on the 31 December 2009. The further tasks will be to end all connection component, steel and composite connections, and the global steel and concrete composite floor tests at elevated temperatures; to develop the relevant material model of joint components for real fire conditions; to make parametric numerical studies of steel joints and the composite floor under real fire conditions; to improve and develop a simple calculation method for predicting the restrained effects of steel members induced by the heating condition of real fires, and a global design guide.

I.5 Real fires in buildings

- Basingstoke fire accident.
- Churchill Plaza fire accident in the UK.
- Broadgate fire accident, SCIF (1991):

  On the 23rd June 1990 a major fire accident occured in a 14-storey building under construction in Broadgate, London. The structure of the building was a steel frame with composite steel deck concrete floors and was only partially protected at this stage of construction. The fire lasted 4.5 hours including 2 hours where the fire exceeded 1000℃. During and af ter the fire, despite large deflections in the elements exposed to fire, the structure behaved well and there was no collapse of any of the columns, beams or floors. The Broadgate phase 8 fire was the first opportunity to examine the influence of fire on the structural behaviour of a modern fast track steel framed building with composite construction.

- Chiado fire accident in Lisbon, Neves et al. (1995):

  The Chiado fire occurred in the historical centre of Lisbon the 25 August 1988, and destroyed 18 buildings built between the XIII and XX centuries, representing a significant loss for the city patrimony.

- Fire event at the World Trade Center in NY, 11 Sept. 2001:

  The fall of the 2 World Trade towers was due to the collisions by aircraft and its fuel. Two Boeing 767 jets were flown into the complex, one into each tower, in a coordinated suicide attack. After burning for 59 minutes, the South Tower collapsed, followed a half-hour later by the North Tower.

  Quintiere et al. (2002), Usmani et al. (2003), The Royal Society of Edinburgh (2004), Baum and Rehm (2005), Kuligowski and Mileti (2008)

I.6 Fire tests realised in real buildings or on sub-structures

I.6.1 Fire tests in real buildings

- ***Petersson et al. (1976)***: Gas time-temperature in different fire compartments were measured, and data in a form readily accessible to the practising engineer were written.

- ***Genes (1982)***: Large-scale fire test was done on a compartment designed to simulate 2 floors of a 20-storey building.
- ***Latham et al. (1985)***: Experimental tests realised in a fire compartment, with a floor area typical of an office in a multi-storey building or a six-bed hospital ward, varying the fire load, the ventilation conditions and the thermal properties of the enclosing surfaces. A total of 21 fire tests were carried out. Steel time-temperature curves for unprotected structural steelwork for several sections exposed in a large fire compartment were drawn.
- ***Anon (1986)***: Fire tests were realised in Germany.
- ***Thomas et al. (1992)***: William street fire tests by BHP, Australia.
- ***Proe and Bennetts (1994)***: Collin street fire test by BHP, Australia.
- ***Wald et al. (2008)***: Compartment fire test in a 3-storey steel frame building, 2006 (Figure 14).

  Composite slabs, steel beams, header plate connections, 3.8m x 5.95m, height of 2.78m, fire load of 58.5 kg/m².

Figure 14. The building of the Ammoniac Separator II before demolition – Wald et al. (2008)

2 fire tests:

- Local fire test: to evaluate the analytical model of the heat transfer into the ceiling and to evaluate analytical prediction models of the column temperature exposed to the natural fire;

- Compartment fire test: to learn more about the connection temperatures and the internal forces in a structure.

Maximum temperature in connections: 691ºC (lower bolt);

Maximum temperature in the lower flange of beam: 1050ºC.

Wald et al. (2007, 2008); Chlouba et al. (2008a-b); Sokol et al. (2008)

- ***Cardington steel framed building, UK***

Table 5. The seven fire tests realised in the Cardington building

Nº	*Name, description*	*Dimensions*	*Thermal action*	*Mech. load*	*Max. temperatures*	*Max. vertical displacement*	*Notes*
1	1D – One restrained beam with composite section (Corus)	8m x 3m 24m²	Gas fired furnace	30%	Gas: 913ºC Steel: 870ºC	230mm or span/35	No fire protection.
2	2D – Plane frame test (Corus)	21m x 2.5m 53m²	Gas fired furnace	30%	Beam lower flange: 820ºC Unprotected parts of columns: 750ºC	Beam deflection: span/34	Reproduction of the type of deformation observed in the Broadgate fire (protected columns, unprotected beam and connections)
3	3D – BRE Corner compartment test (BRE)	9m x 6m 54m²	Natural fire (40kg/m²)	30%	Beam bottom flange: 903ºC	Floor deflection: 265mm	Typical office fire
4	3D – BS Corner compartment test (Corus)	10m x 7.5m 70m²	Timber cribs (45kg/m²)	30%	Unprotected beams: between 900ºC and 1020ºC	- Beam deflection: 420mm over a 9m span - Floor deflection: 365mm or span/25	Main objective: determining the membrane action.
5	3D – Large compartment test (BRE)	21m x 18m 342m²	Natural fire (40kg/m²)	30%	Steel: 691ºC	557mm	
6	3D – Office-Demonstration test (Corus)	18m x 9m 136m²	Natural fire (45kg/m²)	30%	Max. steel temperature: in excess of 1000ºC	Floor deflection: 600mm or span/15	Sever fire scenario with real furniture
7	Structural Integrity test	11m x 7m 77m²	Natural fire (40kg/m²)	56%	- Beam bottom flange: 1087.5ºC - Connection beam-to-beam: 980ºC - Connection beam-to-column: 900ºC	1200mm	Aim: studying the behaviour of steel-concrete composite frame building subjected to a natural fire. Unprotected beams and connections.

References

Cardington tests in general: Armer et al. (1994); Martin and Moore (1997); Moore (1995); Newman et al. (2000); Wang (1998b, 2002).

Two BRE tests: Bailey et al. (1999a, 1999b); Bailey (2000); Cooke (1996); Lennon (1996); Moore (1996, 1997); Wald et al. (2006b); Wang (2000); Wang and Kodur (2000).

Four Corus tests: British Steel (1998); Franssen et al (1995); Kirby (1996, 1997); O'Connor et al. (1998).

Structural integrity test: Foster et al., (2007); Lennon and Moore (2004); Santiago (2008); Sokol and Wald (2005); Wald et al. (2004a, 2004b, 2004c, 2005, 2006a, 2006b).

I.6.2 Fire tests on sub-structures

I.6.2.1 Witteveen et al. (1977)

The first reported test to assess structural behaviour under fire conditions. The stability of braced and unbraced frame at elevated temperature was studied.

I.6.2.2 Koike et al. (1982), Ooyanagi (1983), Hirota (1984)

Model steel frame tests realised in Japan.

(1) 2D steel frame, low temperature test (Koike et al. 1982):

Bending moments in column increase when beam is heated, because of compressive forces in the restrained beam.

(2) 3D frames / only the beam heated at high temperature (Ooyanagi et al. 1983):

Buckling of the beams;

With bracing: lower temperature of buckling;

Buckling of some heated columns due to restraint by the adjacent structure.

(3) 3D model steel frames (Hirota et al. 1984):

Buckling of some heated columns due to restraint by the adjacent structure.

Electrical furnace, strain gauges and displacements transducers, but lack of information to assess the post-buckling behaviour of the columns.

I.6.2.3 Rubert and Schaumann (1986)

Series of quarter-to-half scale fire tests on steel sub-assemblies under fire in order to obtain the failure temperature of the heated steel members, Germany, 1986.

3 frames tested at elevated temperature (electrical heating): 1 braced and 2 unbraced.

LTB prevented by using stiffeners.

No information for strains and forces attained in the test frames.

Good tests for validation of numerical models.

I.6.2.4 Cooke and Latham (1987)

Test of Fire Research Station and Corus on a rugby post frame, UK. First test on a full-size loaded steel frame subjected to a natural fire using wooden cribs.

1 steel beam + 2columns + concrete slabs to give realistic heating conditions to the beam (no composite action); bracing; natural fire exposure (timber cribs); measures of gas temperature, steel temperature and deflections.

Observations:

- Initially deflection due to thermal bowing, then due to increased mechanical deflections (end: runaway);

- Pin joints: Hogging moment transferred – Plastic hinges near connections;

- Equivalent standard fire test time: 32min > simply supported beam (20min).

Conclusion:

The performance of the frame was better than that of the individual elements as a result of the connection continuity that may be exploited to provide increased fire resistance of the beam.

I.6.2.5 Kimura et al. (1990)

7 tests on beam-column assemblies in fire, Japan.

Column: concrete filled square steel tubes.

Conclusion: The influence of the initial bending moment in the column may be ignored.

I.6.2.6 Li et al. (1999)

3 tests on small scale ¼ rigid steel frames (2 bay – 1 storey) at Tongji University, China.

Span: 1620mm; Height: 1400mm; Welded beam to column connections; Gas fire furnace; Loads kept constant during heating; Columns: square steel tubes.

Aim: Prediction to nonlinear behaviour of steel frames subjected to fire.

Result: Procedure to calculate the non linear response and ultimate load of steel frames exposed to fire.

Li and Jiang (1999); Li et al. (1999).

I.6.2.7 Liu et al. (1999 – 2002)

Parametrical fire testing of full scale structural assemblies, University of Manchester, UK.

Subject: the effect of restraint on protected or partially protected beams in the event of fire, and effect of connection types on the failure temperature.

25 fire tests – 2D studies complemented with Cardington tests.

- 2 different types of beam-to-column connection, double web cleats or flush end-plates, while the whole sub-frame assembly is subjected to various levels of loading (3 levels loading);

- Connections and columns protected;

- Top flanges of the beams are wrapped with 5mm thick ceramic fibre blanket.

Numerical model of the tests with FEAST and VULCAN, and numerical study of the catenary action into beams with FEAST.

Results:

- Flush end-plates: high hogging bending moment transferred to columns and the failure temperature is 70ºC higher than the failure temperature of a simply supported beam;

- Web cleat connection: very little influence on the behaviour of the beam until beam in contact with column;

- No beam collapse when in catenary action because difficulty of large deflections in the furnace;

- Connections can enhance the fire resistance of a beam;

- Catenary action obvious only at large deflections (only in tests with higher axial restraint and lower load levels);

- Small variation in temperature in the web and bottom flange.

Fahad et al. (2000), Liu et al. (2001a, 2001b, 2002), Liu and Davies (2001).

I.6.2.8 Dong and Prasad (2009)

Thermal and structural response of a two-storey, two bay composite steel frame under fire loading (Figure 15).

3 furnace tests (conducted with different heating conditions); columns and connections protected.

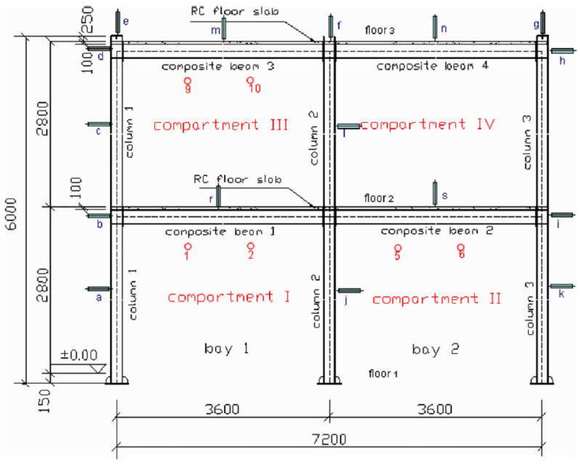


Figure 15. Two-storey, two-bay sway portal frame with fixed bases – Dong and Prasad (2009)

Results:

- Max. average gas temperature at the end of the test: 899ºC;

- Significant temperature gradient through the composite floor beam: 800ºC between the bottom flange of the steel beam and top of the concrete slab, and 550ºC between the top flange of the steel beam and top of the concrete slab;

- The top flange temperature was approximately 200ºC lower than that of the bottom flange;

- Structural failure observed in connections, composite beam and concrete slab.

Dong and Prasad (2008, 2009); Dong et al (2009)

II Real vehicle fires in car parks

II.1 Fire in Schiphol airport

Figure 16 shows a fire (10-2002) in a car park near Schiphol airport (Noordijk and Lemaire, 2005). Around 30 cars were on fire at the same time. Also the fire spread was much faster than currently assumed. However, the fire occurred in a car park of a car rental company, which led to some specific circumstances that might have caused the more rapid fire spread than normally expected:

- All cars were parked on a small distance of each other, which can enhance fire spread from car to car;
- All cars were new and new cars contain more plastic parts than older cars. Plastics can be ignited more easily and produce more heat;
- All fuel tanks of the cars were completely filled, leading to a high fire load;
- The fuel tanks were made of plastic and started leaking fuel, creating pool fires which can also cause spreading of fire, by draining away under other cars.

Figure 16. Fire in a car park of the Schiphol airport – Noordijk and Lemaire (2005)

II.2 Statistics on real fires in car parks (1997) – France, Paris

Hereafter is presented a summary of statistics on real fire in car parks in Paris realised in 1997 (Joyeux et al., 2001; Joyeux et al., 2002; Joyeux, 1999)).

II.2.1 Data from firemen

- 327 intervention reports of fires in underground car parks in 1997,
- 78 intervention reports concerning fires in open car parks during three years: 1995, 1996 and 1997 (34 reports).

II.2.2 Number of cars involved in open car park fires

Figure 17 shows the number of car involved in a fire, considering all fires or only car fires. The maximum number of cars involved in fires was 3, which corresponds to only 10 % of the fires. 30 % of fires are not due to cars.

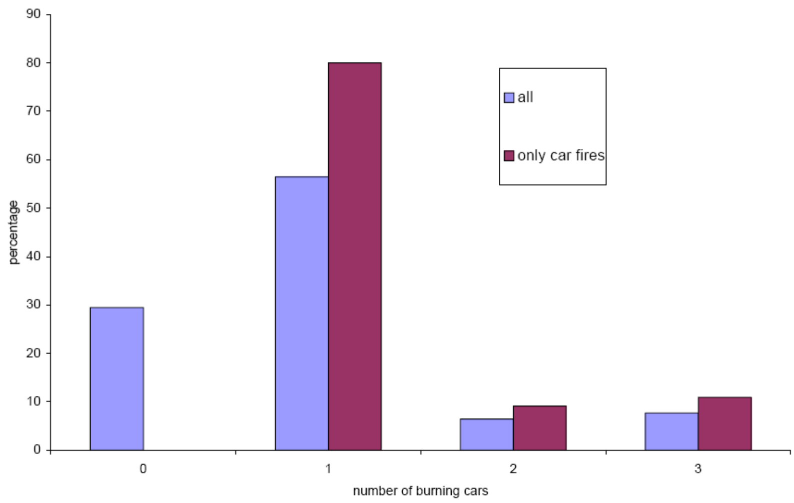


Figure 17. Open car park data: number of cars involved in a fire – Joyeux et al. (2002)

II.2.3 Classification of open car park burning cars

Figure 18 shows the classification of burning cars. The categories 4 and 5 represent 10% of cars. The classification of cars is in function of the calorific potential and is given in the Table 2, §I.3.1.

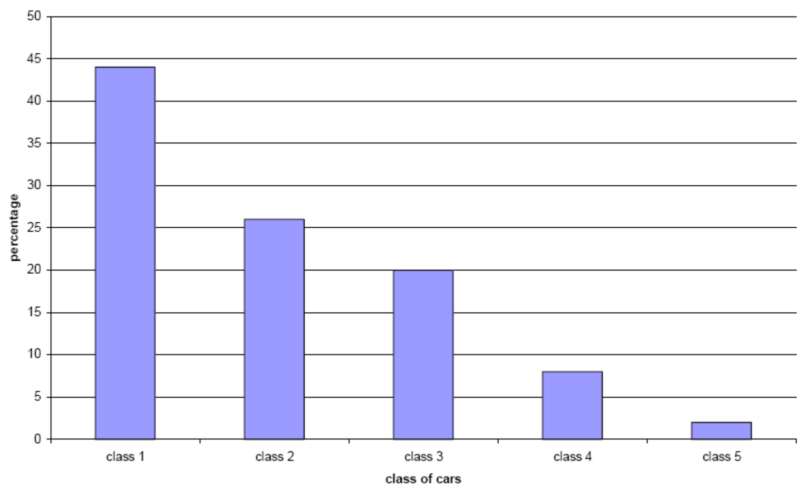


Figure 18. Open car park data: Classification of burning cars – Joyeux et al. (2002)

II.2.4 Time to extinction by firemen before their arrival in open car parks

Figure 19 gives the distribution of fire according to the time to extinction by firemen, or before their arrival.

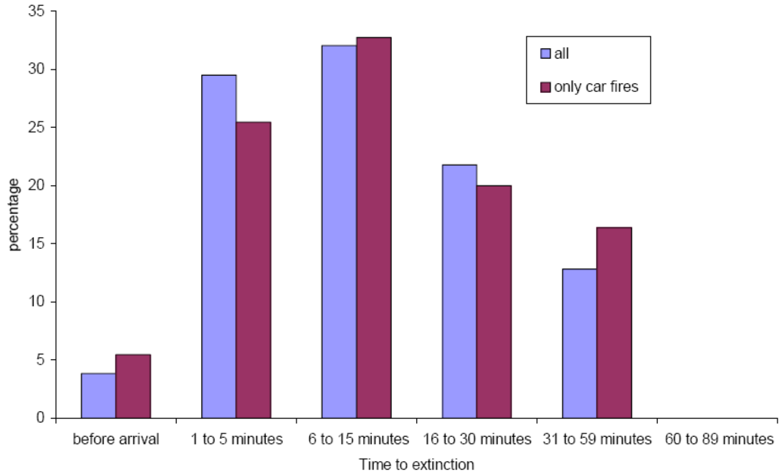


Figure 19. Open car park data: Time to extinction – Joyeux et al. (2002)

Additional comments:

- 5.5% of car fires were extinguished before the arrival of firemen;
- All fires were stopped in 1 hour;
- 16% of car fires required between 30 minutes and one hour to be extinguished;
- Only two people were injured, the owners of cars;
- No fire was recorded with more than one car of category 3, 4 or 5.

Figure 20 shows the percentage of car fires in function of the duration of the fire and the number of cars involved in an open car park:

- Fire duration's between 30 and 60 minutes are generally necessary for the extinction of fires involving several cars (50 % of cases) or of fires in boxes;
- 70 % of fires involving one car were stopped before 15 minutes;
- 80 % of fires involving two cars were stopped between 6 and 30 minutes;
- 84 % of fires involving 3 cars were extinguished by the firemen between 16 and 60 minutes after their arrival.

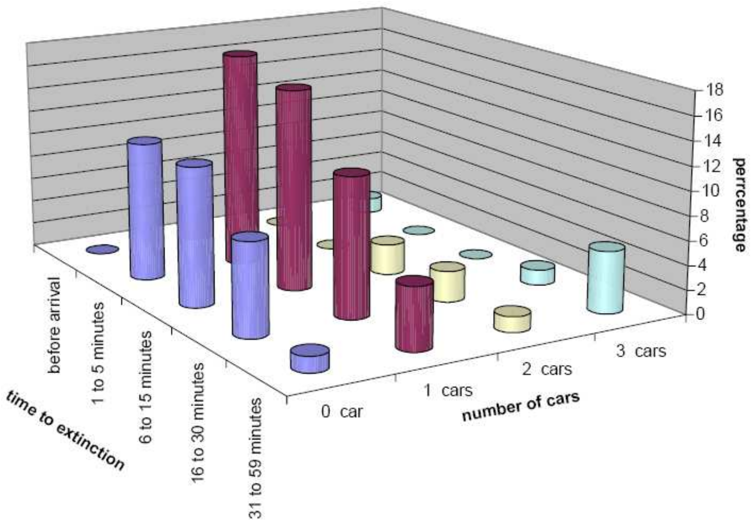


Figure 20. Open car park data – Joyeux et al. (2002)

II.3 Statistics on real fires in car parks (2004) – New Zealand

Statistics in New Zealand parking buildings were established during the eight-year period from 1995 to 2003 (Li and Spearpoint, 2007):

- No fire fatality was recorded in New Zealand car parks (neither in US or Canada (Denda, 1993; Harris, 1972), or in France (Joyeux et al., 2002));
- 96 fire incidents involving vehicles or on average 12 vehicle fire incidents each year;
- Eight vehicles involved in three multiple vehicle fire incidents, which equate to approximately 3% of total vehicle fire incidents.

II.3.1 Parking building and vehicle types

The breakdown of all vehicle fires in New Zealand parking buildings from 1995 to 2003, according to the FIRS coding definitions, is shown in Figure 21.

Figure 22 illustrates the breakdown of type of vehicles involved in fire incidents.

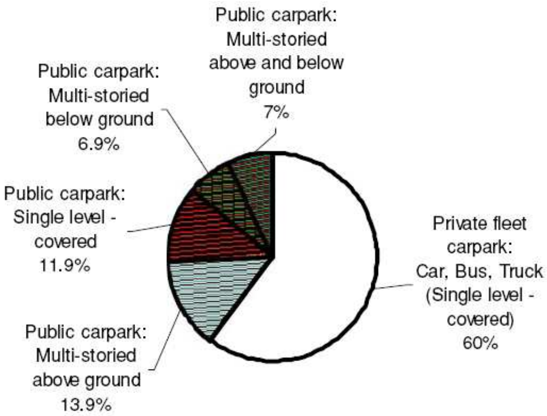


Figure 21. Specific type of parking buildings where vehicles were involved in fires, 1995-2003 – Li and Spearpoint (2007)

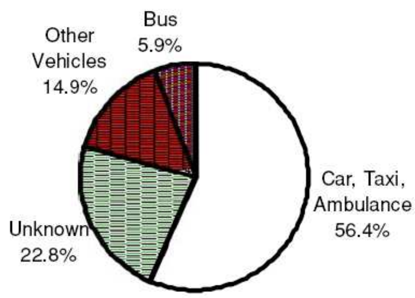


Figure 22. Type of vehicles involved in fires in parking buildings, 1995-2003 – Li and Spearpoint (2007)

II.3.2 Fire spread between vehicles

Figure 23 illustrates the vehicle fires in New Zealand parking buildings by number of vehicles involved due to a single ignition.

- Two incidents where two vehicles were involved in the parking building fire simultaneously (1995/1996 and 2002/2003);

- One incident involved four vehicles in the year of 1999/2000 and this fire started from a vehicle with recorded type as light truck (under one tonne) then spread to three buses.

→ 3% of the vehicle fire incidents in New Zealand parking buildings involved multiple vehicles which is lower than values published elsewhere (7% in US car parks (Denda, 1993), and 15% in French underground car parks (Joyeux et al., 2002)).

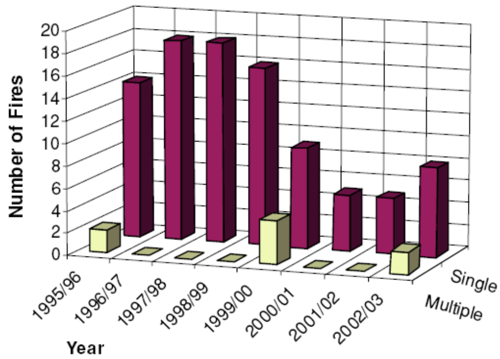


Figure 23. Vehicles fires in parking buildings by number of vehicles involved, 1995-2003 – Li and Spearpoint (2007)

II.3.3 Age of vehicles involved in parking building fires

The probability of a vehicle involved in parking building fire rises with the increase of the vehicle age is shown in Figure 24.

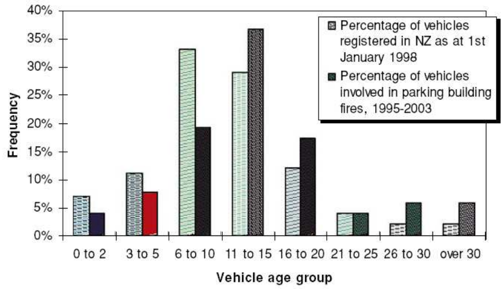


Figure 24. Distribution comparison between age of vehicles involved in parking building fires and age of all registered vehicles – Li and Spearpoint (2007)

III Verification of a steel open car park under a localised fire

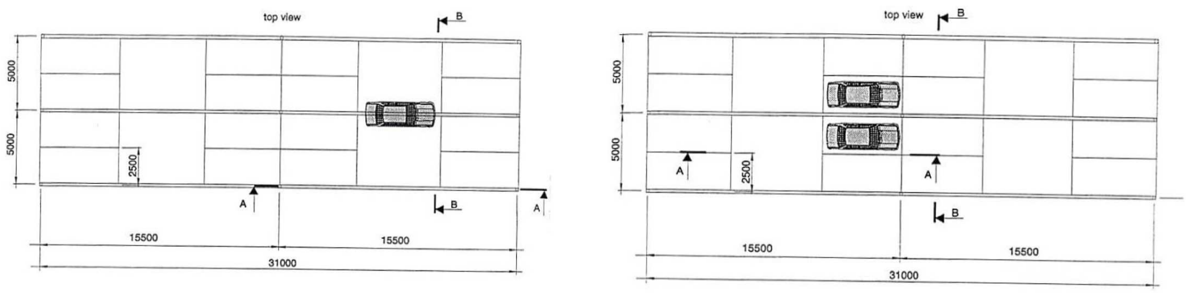
A fire engineering methodology for structural fire behaviour of open car parks is determined according to the following steps (Joyeux et al., 2002):

- Definition of the fire scenario,
- Determination of the rate of heat release of cars,
- Determination of the heat flux to structural elements from each car fire,
- Determination of the total heat flux to structural elements,
- Determination of the temperature of structural elements,
- Determination of fire behaviour of parts of the structure.

III.1 Fire scenarios

Figure 25 shows the two most critical scenarios in an open car park, according to the ECCS report (1993):

- Scenario 1: implies only one car burning at mid-span under the beam. It corresponds to the maximum bending moment position and so the most critical situation for the beams. This scenario is also defined by INERIS as a basic scenario (Zhao et al., 2004).
- Scenario 2: Two burning cars are considered, parked at each side of a beam.



Scenario 1 Scenario 2

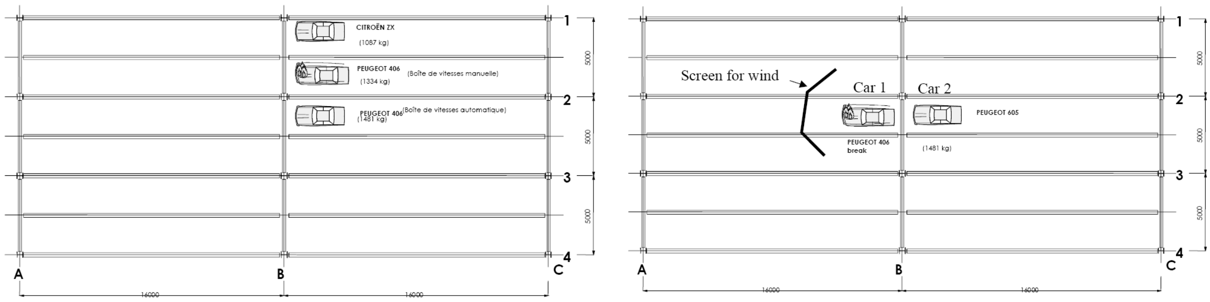
Figure 25. ECCS Scenarios (1993)

The car park constructed for the French experimental fire test realised by the CTICM (see §I.2.5 – Joyeux et al., 2002) was designed according to the two previous scenarios described by the ECCS, except that for the scenario 2, three cars were implied. It represents the critical case: when the car 1 is on one side of a beam, the fire propagates to the car 2 on the other side of the beam and then propagates to the car 3 close to the car 2.

Two other scenarios were tested during the French experimental fire tests (Figure 26):

- Scenarios 3: Three cars of class 3 parked on consecutive bays. The car in the middle is ignited. A scenario of 3 class-3 cars involved in a fire is an envelope scenario of about 98.7% of all possible scenarios from the statistics in open car parks (§II.2).

- Scenario 4: Two cars parked on parking bays located in front of, to assess the propagation of the fire from one car to another car in this configuration.



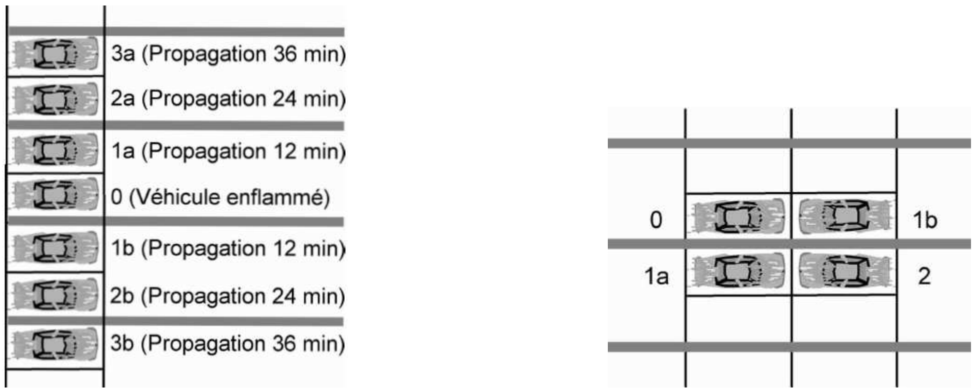
Scenario 3 Scenario 4

Figure 26. CTICM scenarios – Joyeux et al. (2002)

The INERIS define three basic scenarios (Zhao et al., 2004): the scenario 1 with a utilitarian vehicle under a beam, which is a small van full of inflammable products (19.5GJ, 18MW), and the two following ones:

- Scenario 5: Propagation of the fire to 7 vehicles of class 3 (see Table 2, §I.3.1 for the cars classification), in normal parking bay, with eventually a utilitarian vehicle at places 0 or 1b. Dimensions of a normal parking bay will be defined in the §IV.3.
- Scenario 6: Propagation of the fire to 4 vehicles of class 3 parked on parking bays located in front of, with eventually a utilitarian vehicle at places 0, 1a, 1b or 2.

Note that these last scenarios 5 and 6 have never been met in real fires, and for the scenarios with normal cars, the columns have to resist to the fire while scenarios with the utilitarian vehicle just have to be used for the stability check of the entire structure.



Scenario 5 Scenario 6

Figure 27. Two additional scenarios defined by INERIS, France – Cajot et al. (2003)

For all scenarios, the propagation time for the fire propagation from a car to another car is 12 minutes (Zhao et al., 2004).

III.2 RHR curves

III.2.1 CTICM car fire tests results (1995-1996)

Rate of heat release curves of cars in fire obtained by the tests of the CTICM (1995-1996, §I.4.2) were used to design the car park structure for the last experimental work realised in a real open car park in France (Joyeux et al., 2002), see §I.2.5. The six first tests were realised with vehicles from the 70' and 80'. Vehicles more recent (1995) were used for the four last tests. See Table 6. The sum of the rates of heat release included in the two fire plumes is equal to the total heat release of the car, which was measured during the tests.

Table 6. Summary of the CTICM tests (1995-1996)

Test	Number of vehicles involved	Vehicle 1	Vehicle 2	Height under ceiling (m)	Configuration of the test volume	Energy (E) and RHR (Q)
1	2	Class 2	Class 3	2.30	Angle (2 sides opened)	-
2	1	Class 3		2.30	Angle	-
3	1	Class 1		2.60	Angle	E = 2.1GJ Q = 3.5MW
4	1	Class 3		2.60	Angle	E = 3.08GJ Q = 2.1MW
5	2	Class 4	Class 1	2.60	Angle	-
6	2	Class 3	Class 2	2.60	Angle	-
7	1	Class 3		2.60	3 sides opened	E = 6.7GJ Q = 8.5MW
8	1	Class 1		2.60	3 sides opened	E = 4.1GJ Q = 4.1MW
9	2	Class 1	Class 3	2.60	3 sides opened	-
10	2	Class 3	Class 1	2.60	3 sides opened	-

From the results, reference curves were defined. Figure 28 shows the reference curve of a single class 3-car fire and Figure 29 gives reference curves of three burning class 3-cars with time propagation from one burning car to an other equal to 12 minutes. Values are detailed in the Table 7.

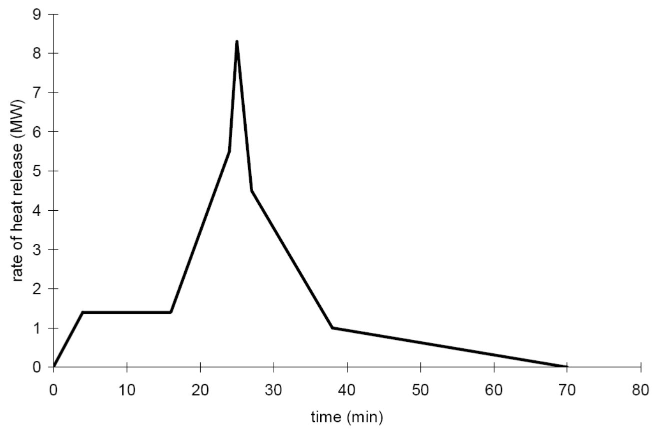


Figure 28. Reference curve RHR (MW) vs time (min) of a single class3-car fire – Joyeux et al. (2002)

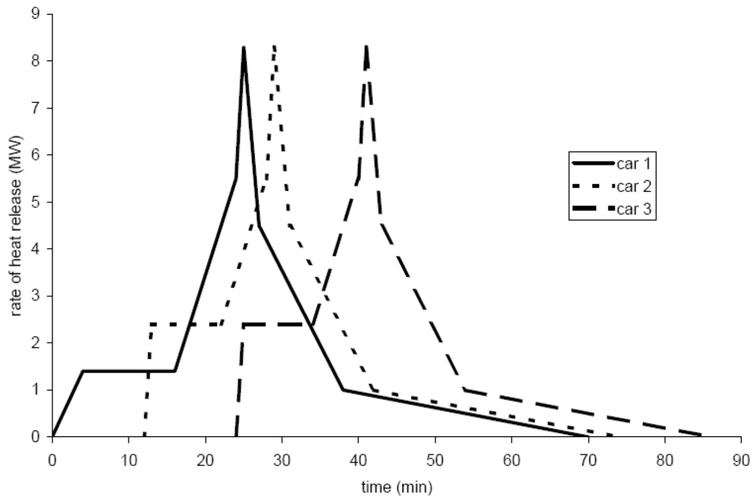


Figure 29. Reference curves RHR (MW) vs time (min) of the three burning class 3-cars – Joyeux et al. (2002)

Table 7. RHR for 3 burning cars – Joyeux et al. (2002)

Time (min)	Car 1	Time (min)	Car 2	Time (min)	Car 3
0	0	12	0	24	0
4	1.4	13	2.4	25	2.4
16	1.4	22	2.4	34	2.4
24	5.5	28	5.5	40	5.5
25	8.3	29	8.3	41	8.3
27	4.5	31	4.5	43	4.5
38	1	42	1	54	1
70	0	74	0	86	0

The RHR curve of a utilitarian vehicle is given in Figure 30 in pink, with two more curves of a burning class 3-car at the beginning (in red) and a burning class 3-car after propagation (in blue). The maximal RHR value of the utilitarian vehicle (18MW) is not obtained by experimental work but corresponds to a safe value (Zhao et al., 2004).

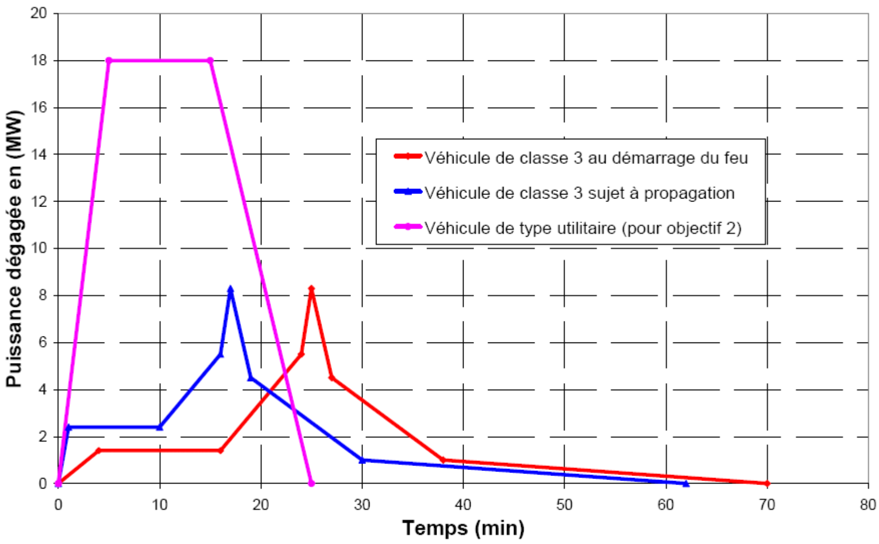


Figure 30. Reference curves RHR (MW) vs time (min) of a burning utilitarian vehicle and burnings of two class 3-cars – Zhao et al (2004)

Figure 31 shows RHR curves obtained for the fire scenarios number 5, including 7 vehicles and represented in blue in the figure, and number 6, including 4 vehicles and represented in pink in the figure, with a utilitarian vehicle in the second position of burning. The fire scenario number 1, with only a utilitarian vehicle burning under a beam, is represented in red. These scenarios are showed in Figure 25 and Figure 27.

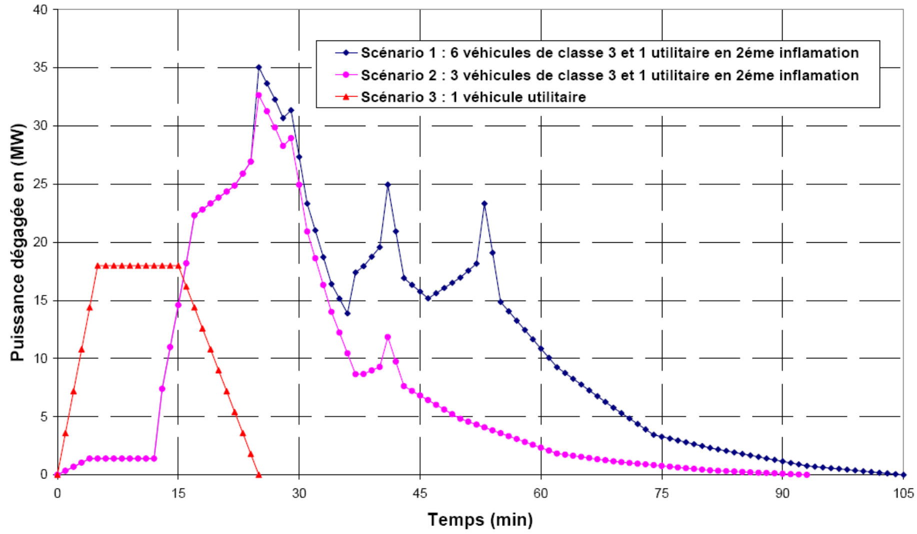


Figure 31. Reference curves RHR (MW) vs time (min) according to the scenario – Zhao et al (2004)

III.2.2 Two additionally car fire tests realised by the CTICM (2000)

Before the realisation of the French experimental test in a real car park in 2000, two other car fire tests were performed under calorimeter hood at the CTICM laboratory on a Peugeot 406 break and a Peugeot 406 family, cars that were used for the car park fire test. The evolution with time of the mass loss rate, the rate of heat release and the energy are showed respectively in Figure 32, Figure 33 and Figure 34. A comparison with a previous test on a Laguna is done (Joyeux et al., 2002).

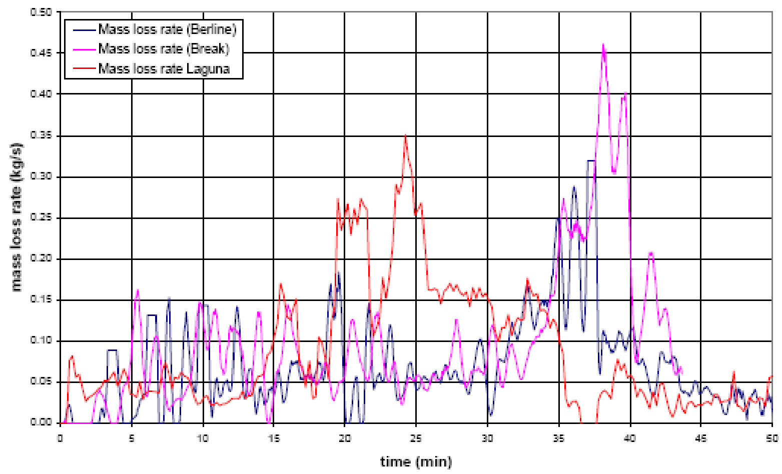


Figure 32. Mass loss rate (kg/s) vs time (min) for three cars: Berline, Break and Laguna – Joyeux et al. (2002)

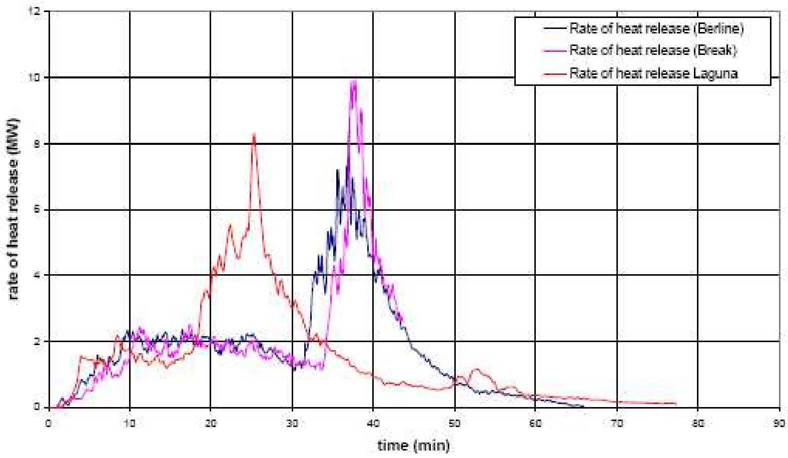


Figure 33. RHR (MW) vs time (min) for three cars: Berline, Break and Laguna – Joyeux et al. (2002)

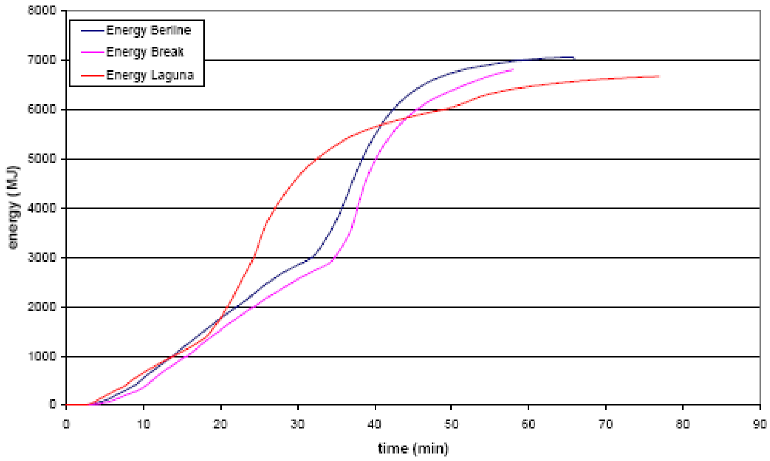


Figure 34. Energy (MJ) vs time (min) for three cars: Berline, Break and Laguna – Joyeux et al. (2002)

III.2.3 Comparison of passenger vehicles RHR curves

Figure 35 shows RHR curves from various car fire experiments (Li and Spearpoint, 2007):

- The three tests reported by Mangs and Keski-Rahkonen (1994a, 1994b) gave similar RHR results and only Test 2 is shown (§I.4.1);
- The study described by Schleich et al. (1999a) is represented by their proposed reference RHR curve;
- Single vehicle fire tests by Steinert (2000) yielded relatively low RHR, hence only Test 3 was shown to represent this work;
- Almost same peak RHR value (about 8 MW) for Maestro car by Shipp and Spearpoint (1995) and the reference RHR curve by Schleich et al. (1999a);
- More modern cars yield a higher maximum RHR than older generation cars when involved in fire. Ingason (2001) observed a tendency of maximum RHR increasing linearly with the total energy to be released from passenger cars, and further proposed that an average increase of 0.7 MW in maximum RHR can be expected from one GJ of energy. This value is within the range of RHR versus energy (0.55 to 0.85 MW/GJ) shown by Steinert (2000).

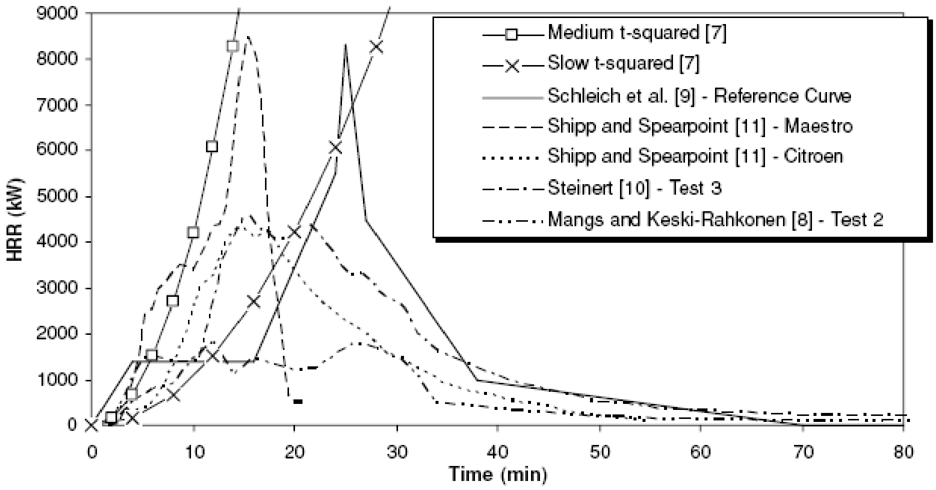


Figure 35. RHR (MW) vs time (min) curves from various car fire experiments – Li and Spearpoint (2007)

III.3 Structural Behaviour

III.3.1 Mechanical loading

The combination of mechanical loads for car parks is defined by the Eurocode 0 (En 1990:2002). According to this Eurocode, the mechanical action in case of fire is the following:

$$1.0\ g_k + 0.7\ q_k$$

where g_k is the permanent loads and q_k the live loads.

The load model which should be used, given by the Eurocode 1 part 1.1 (EN 1991-1-1:2002), is a single axle with a load Q_k with dimensions according to Figure 36 and a uniformly distributed load q_k.

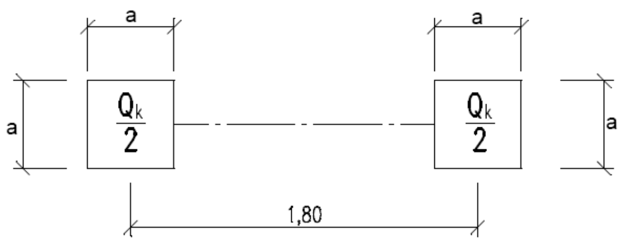


Figure 36. Dimensions of axle load – EN 1991-1-1:2002

Concerning characteristic values of the live loads, the Eurocode 1 part 1.1 (EN 1991-1-1:2002) recommends that q_k be selected within the range 1.5 to <u>2.5</u> kN/m^2 and Q_k be selected within the range 10 to <u>20</u> kN; the recommended values are underlined, but they may be set by the National annex. The axle load should be applied on two square surfaces with a 100 mm side in the possible positions which will produce the most adverse effects of the action.

Table 8. Values of the live loads for Belgium and Portugal according to National annexes

	q_k [kN/m^2]	Q_k [kN]	References
Belgium	2.5	20	NBN EN 1991-1-1 - ANB:2005
Portugal	2.5	15	NP EN 1991-1-1:2005

III.3.2 Thermal loading

III.3.2.1 Car fire model

The ECCS report (1993) describes a car fire model. According car fire tests, it was clearly observed that flames extended out of the car mainly through the windscreen and the rear window. The hot gases in the flames and above them move upwards due to the buoyancy. This buoyancy flow is referred to as a fire plume. The burning car is divided into two plumes which are called as the front fire plume and the rear fire plume. The axis of the fire plumes is assumed to be 2m apart according to the dimensions of ordinary passenger cars.

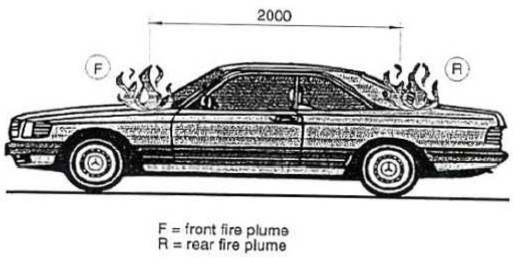


Figure 37. Front fire plume and rear fire plume, ECCS (1993)

III.3.2.2 Calculation of the thermal loading

In order to calculate the steel temperature inside a car park it should be considered two situations:

A) The temperature of the structural elements far away the fire,

B) The temperature of the structural elements near the fire.

In both situations two ways could be used to calculate the temperature in the structural elements:

i) a simplified method assuming an homogeneous temperature in the steel section,

ii) a finite element model calculating the temperature field in the steel section.

To calculate the temperature of the structural elements far away the fire, the EN 1991-1-2:2002 suggests that the homogeneous temperature in the steel section should be calculated based on the net heat flux h_{net} received by the fire exposed unit surface area:

$$\dot{h}_{net} = \alpha_c\left(\theta_g - \theta_m\right) + \Phi\varepsilon_m\varepsilon_f\sigma\left[\left(\theta_r + 273\right)^4 - \left(\theta_m + 273\right)^4\right] \quad (1)$$

Where α_c is the coefficient of heat transfer by convection [W/m^{2}K], θ_g is the gas temperature in the vicinity of the fire exposed member [℃], θ_m is the surface temperature of the member [℃], θ_r is the effective radiation temperature of the fire environment [℃], Φ is the configuration factor, ε_m is the surface emissivity of the member, ε_f is the emissivity of the fire and σ is the Stephan Boltzmann constant (= 5.67x 10^{-8} W/m^{2}K^4). The temperatures θ_g may be adopted as nominal temperature-time curves, or adopted according to the fire models (see EN 1991-1-2:2002).

However if it is intended to obtain a more sophisticated simulation, it should be used a **two-zone model** (EN 1991-1-2:2002, Annex D). A two-zone model is based on the assumption of accumulation of combustion products in a layer beneath the ceiling, with a horizontal interface. Different zones are defined: the upper layer, the lower layer, the fire and its plume, the external gas and walls. In the upper layer, uniform characteristics of the gas may be assumed. The exchanges of mass, energy and chemical substance may be calculated between these different zones. A computational fluid dynamic model may be used to solve numerically the partial differential equations giving, in all points of the compartment, the thermo-dynamic and aero-dynamic variables

To calculate the temperature of the structural elements near the fire, the EN 1991-1-2:2002 suggests a simplified method: the **localized fire model** (EN 1991-1-2:2002, Annex C). The net heat flux h_{net} received by the fire exposed unit surface area at the level of the ceiling, is given by:

$$\dot{h}_{net} = \dot{h} - \alpha_c(\theta_m - 20) - \Phi\varepsilon_m\varepsilon_f\sigma\left[(\theta_m + 273)^4 - (20 + 273)^4\right] \quad (2)$$

Where $\dot{h}$ is the heat flux received by the fire exposed unit surface area at the level of the ceiling when the flame is impacting the ceiling and the others coefficients are defined in expression (1).

The net heat flux ($\dot{h}_{net,car}$) to structural elements from each car fire is a function of the following parameters (Joyeux et al., 2002 - Figure 38):

Q (W) Rate of heat release of cars, given in the §III.2;

H_a (m) Vertical distance from the beam lower flange to the floor (in case of a heat flux to a beam);

H_s (m) Vertical distance from the fire source to the floor; for a car in fire, 0.3 m is used by ECCS (1993) and Joyeux et al. (2002), and 0.6m is used by Schleich et al. (1999a) for closed car parks;

D (m) Diameter of the fire or characteristic length of the fire source; for a car in fire, 3.9 m is used, with a surface equal to 12m^2 (Schleich, 1999a);

r (m) Horizontal distance between the fire axis and the point where the net heat flux is computed.

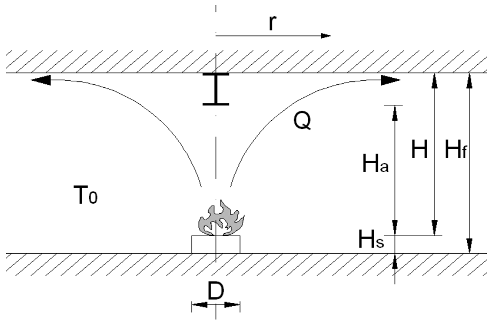


Figure 38. Parameters to model a car fire – ECCS (1993)

This net heat flux, $\dot{h}_{net,car}$, is computed according to the following expression (Zhao et al., 2004):

$$\dot{h}_{net,car} = 100 \qquad \text{if } y \leq 0.30$$

$$\dot{h}_{net,car} = 136.30 - 121.00y \qquad \text{if } 0.30 < y \leq 1.00 \tag{3}$$

$$\dot{h}_{net,car} = 15y^{-3.7} \qquad \text{if } 1.00 < y$$

Where y is computed by:

$$y = \frac{r + H + z'}{L_H + H + z'} \tag{4}$$

With L_H specified according to the following expression:

$$\frac{L_H + H}{H} = 2.90\, {Q_H^*}^{0.33} \tag{5}$$

$$H = H_a - H_S \tag{6}$$

$$Q_H^* = \frac{Q}{1.11 \times 10^6\, H^{2.5}} \tag{7}$$

$$z' = 2.4D\left(Q^{*2/5} - Q^{*2/3}\right) \qquad \text{when } Q^* < 1.00 \tag{8}$$

$$z' = 2.4D\left(1.0 - Q^{*2/5}\right) \qquad \text{when } Q^* \geq 1.00$$

$$Q^* = \frac{Q}{1.11 \times 10^6\, D^{2.5}} \tag{9}$$

A reduction factor equal to 0.85 is applied when some faces of the element are submitted to a lower heat flux because of their position more or less hidden.

The heat flux from several localised fires is given by the sum of heat flux obtained for each localised fire, with a maximum value equal to 100 kW/m^2, corresponding to the value being reached by an open fire unconfined (Zhao et al., 2004).

According to a report submitted at CECMI by Joyeux (Joyeux, INC-00/62-DJ/IM), the computing of the heat flux to columns varies with the column position in relation to the localised fire: either the column is between two vehicles, either it is at the extremity of the parking bay, or it is at the extremity of the parking bay in front of a wall (Zhao et al., 2004).

Generally, the free height for cars and vans is at least equal to 2.1m (ArcelorMittal, 2007). However, storey heights can vary from 2.1m to 5.0m, which gives a height under beam of about 4.5m. A higher storey height gives smaller thermal loading thanks to the higher spacing. This phenomenon is shown in Figure 39 (Zhao et al., 2004). In order to take into account this positive parameter, Zhao et al. (2004) defined a design extrapolation rule of car parks primary and secondary beams with a minimal height under beam of 3.0 m (Table 9).

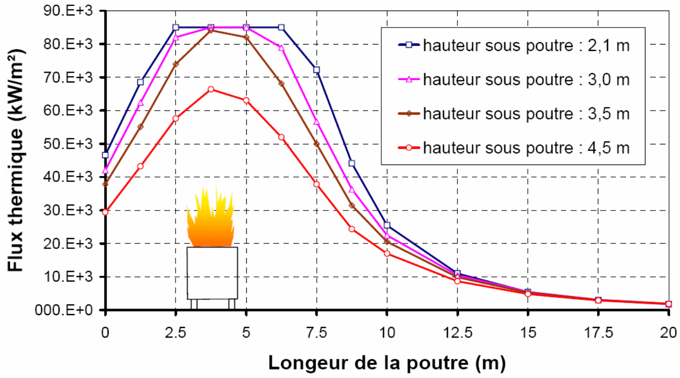


Figure 39. Heat flux (kW/m^2) received by the beam vs beam length (m) in function of the free height under beam for the scenario 6 (see Figure 27) – Zhao et al. (2004)

Table 9. Design extrapolation rule for beams of car park with a minimal height under beam equal to 3.0 m – Zhao et al. (2004)

Minimal section area for steel beams in car parks with a minimal height under beam of 3.0m				
$\frac{A_n \times f_{yn}}{A_{ref} \times f_y} \geq \beta$	A_n and f_{yn} are respectively the area and the steel grade of the new beam sections. A_{ref} and f_y are respectively the area and the steel grade of the beam after a fire design.	Height under beam		
		3.0 m	3.5 m	4.5 m
Values of β for secondary beams:		1.0	0.9	0.77
Values of β for primary beams:		1.0	0.85	0.77
Application conditions: Min. sections height: $H \geq 0.85\ H_{ref}$ Max. section factor: $A/V \leq 1.10\ (A/V)_{ref}$ H is the total height of the beam section after reduction H_{ref} is the total height of the beam section after a fire design A/V is the beam section factor considered on three faces				

III.3.3 Structural fire response

In the actual fire safety engineering, there exist two major assessment approaches to evaluate the mechanical response of structures exposed to fire:

- Fire tests (very expensive and take a long time),
- Design rules.

Depending of the desired sophistication, simplified calculation models or advanced calculation models could be employed. Simple calculation models are simplified design methods for individual members, which are based on conservative assumptions. Advanced calculation models are design methods in which engineering principles are applied in a realistic manner to specific applications (EN1993-1-2:2005).

As far as advanced calculation models are concerned, in principle, they can be applied for any type of structural member analysis in fire design. However, following features have to be considered:

- Advanced calculation methods for mechanical response should be based on the acknowledged principles and assumptions of the theory of structural mechanics, taking into account the changes of mechanical properties with temperature (see EN 1993-1-2:2005);

- Any potential failure modes not covered by the advanced calculation method (including local buckling and failure in shear) should be eliminated by appropriate means. For example, in case of numerical analysis using beam elements;
- Advanced calculation methods may be used in association with any heating curve, provided that the material properties are known for the relevant temperature range;
- The effects of thermally induced strains and stresses both due to temperature rise and due to temperature differentials, should be considered;
- The model for mechanical response should also take account of:
    - the combined effects of mechanical actions, geometrical imperfections and thermal actions,
    - the temperature dependent mechanical properties of the material (see EN 1993-1-2:2005),
    - geometrical non-linear effects,
    - the effects of non-linear material properties, including the unfavourable effects of loading and unloading on the structural stiffness.

IV Design requirements

IV.1 General requirements

According to ECCS (1993), steel structures do not need any fire protection in open car parks and therefore have additional economical advantages. The security of car parks based on unprotected steel structures is guaranteed under fire conditions, provided that the rules presented hereafter will be considered.

Construction rules (ECCS, 1993):

- Static calculation: "Cold" design according to the valid standards following the state of the art is the basic condition for the stability of the structure in the fire situation. No special precautions for fire neither a special "hot" design are required.
- Beams: Cross beams and support beams shall be built as composite beams. Rolled sections or welded I-sections may be used as floor beams. For economical reasons it is recommended to use lightweight sections (IPE, HEAA and UB).
- Columns: Columns shall consist of wide flange profiles (HEA, HEB, UC).
- Horizontal stability: Frames or bracings have to be provided in order to support the horizontal loads. These bracings have to be protected against the heating. It may be interesting to foresee staircases in concrete, which may supply the horizontal stability and which additionally may serve as fire escape routes.
- Steel grade: Material of steel construction may correspond to EN 10025 (Fe360, Fe430 and Fe510 – March 1990) or EN 10113 (S275, S355, S420 or S460 – March 1993).
- Shear connectors: The steel beams shall be connected by shear connectors to the concrete slab. Sufficient connection has to be provided and should be calculated according to the guidelines for either elastic or plastic design.
- Concrete floor: The concrete floor may be built with either in-situ concrete (e.g. profile sheets as permanent shuttering) or precast concrete (e.g. hollow-core units, etc…). The essential point is the statically and structural integration of the floor in the whole load bearing system.
- Additional reinforcement: Both hinged and rigid connections of cross and support beams with the columns are admissible. In case of a hinged connection the negative bending moment near the column, as it occurs during fire, should be taken up by an additional reinforcement of the concrete floor.

IV.2 Fire requirements country by country

IV.2.1 Worldwide fire resistance requirements for car parks

A car park may be considered as "open" if for every parking level, the ventilation areas in the walls are situated in at least two opposite facades, equal at least 1/3 of the total surface area of all the walls and correspond to at least 5% of the floor area of one parking level (ECCS, 1993).

Table 10. Worldwide fire resistance requirements for car parks (Y stands for YES, N for NO) – ECCS (1993), updated for UK, Portugal, Luxembourg, France and Belgium.

92-11-03	Car Parks			OPEN CAR PARKS									Bibliography
CCP = Closed Car Park		Distinction between		Limitations						General ISO fire resistance requirements	Conditional alternatives		
OCP = Open Car Park UGCP = Underground Car Park AGCP = Aboveground Car Park	Special Requirements	CCP/OCP	AGCP/UGCP	Minimum percentage of openings (%)		MAXIMUM					Unprotected steel	Natural fire (*5)	
						Distance between opposite facades (m)	Number of storeys	Height of Buildings (m)	Floor area per storey (m^2)				
COUNTRY				(*1)	(*2)								
Australia	Y	Y	N	-	50	-	-	-	-	Up to R60	Y	-	Australia
Austria	Y	Y	Y	33	-	70	-	22	-	Up to R90	Y	Y	Austria
Belgium	Y	Y	N	17 (1/6)	-	60	-	-	-	NONE (*6)	/	Y	Belg. 2007, Infosteel
Canada	Y	Y	N	25	-	120	-	22	10000	Up to R60	Y	-	Canada
Denmark	Y	Y	Y	5% of the floor area		24	-	-	-	Up to R60	Y	Y	Denmark
Finland	Y	Y	N	30	-	-	8	-	9000	R60	N (*7)	Y	Finland
France	Y	Y	N	50 5% of the floor area		75	-	-	-	Up to R60	N (*7)	Y	ArcelorM. 2007
Germany	Y	Y	-	33	-	70	-	22	-	NONE (*6)	/	/	Germany
Hungary	N	-	-	-	-	-	-	-	-	R30 to R90 (*4)	N	N	Hungary
Italy	Y	Y	Y	-	60	-	-	-	-	Up to R90	-	-	Italy
Luxembourg	Y	-	-	-	-	-	-	-	-	NONE (*6)	/	/	Lux., ArcelorM. 1996
Netherlands	Y	Y	Y	33	-	54	-	20	-	NONE (*6)	/	/	Neth.
New Zealand	N	-	-	-	-	-	-	-	-	(*4)	Y	-	New Zealand
Norway	Y	Y	N	33	-	-	6	-	5400	R10 to R60	Y	-	Norway
Poland	Y	N	Y	-	-	-	-	25	4000	R60	N	-	Poland
Portugal	Y	Y	N	-	-	-	-	-	-	R60	-	-	Portugal 2008a-b
Spain	N	-	-	-	-	-	-	-	-	R60 to R120 (*4)	-	-	Spain
Sweden	N	-	-	-	-	-	-	-	-	Up to R90 (*4)	Y	Y	Sweden
Switzerland	Y	Y	Y	25	-	70	-	-	-	NONE (*6)	/	/	Switz.
United Kingdom	Y	Y	Y	5% of the floor area		-	-	-	-	R15 to R60	Y	Y	Corus 2004
United States	Y	Y	-	-	50	120	9	-	4700	NONE (*6)	/	/	US

(*1): Total area of openings / total area of walls and facades surrounding one parking level.

(*2): Opening area of one side / wall area of this side.

(*3): In France, use of unprotected steel is allowed for AUTOMATIC open car parks.

(*4): General requirements of National Building Code.

(*5): Use of Natural Fire as an alternative to ISO-Fire to prove the fire resistance.

(*6): None means that there is no fire requirement and that unprotected structure is thus allowed.

(*7): Bare steel is allowed if this can be proved by tests or scientific studies.

IV.2.2 Car park structure definition in Portugal

In Portugal, the new national law concerning the buildings fire security has just been published (Portugal, 2008a). Concerning car parks, defined as a structure utilisation of 'type II', a distinction is done between open and closed car parks (Table 11). The minimum fire resistance prescribe by the Portuguese code for structural elements of an open parking is R60 (Portugal, 2008b).

Table 11. Risks categories when the structure utilisation is of 'type II' (Car parks) – Portugal (2008a)

Category	Criteria's relative to an utilisation of 'type II', when integrate to the building structure			Open structure
	Height	Gross section	Number of storeys occupied by the type II utilisation below the reference level	
1st – Reduced risk	/			Yes
	≤ 9m	≤ 3 200m^2	≤ 1	No
2nd – Moderate risk	≤ 28m	≤ 9 600m^2	≤ 3	No
3rd – High risk	≤ 28m	≤ 32 000m^2	≤ 5	No
4th – Very high risk	> 28m	> 32 000m^2	> 5	No

IV.2.3 Prescriptions from ArcelorMittal (1996) in Luxembourg

Prescriptions given by ArcelorMittal (1996):

- Definition of an open car park: each storey has openings equal to 1/3 of the total surface area of the outer walls of this level and effective and permanent ventilation is guaranteed (Figure 40);
- No fire resistance requirements for steel structures of open multi-storey car parks. For the countries where there are fire resistance requirements: fire engineering is considered as an alternative to the conventional ISO fire approach which requires passive protection of the steel elements.

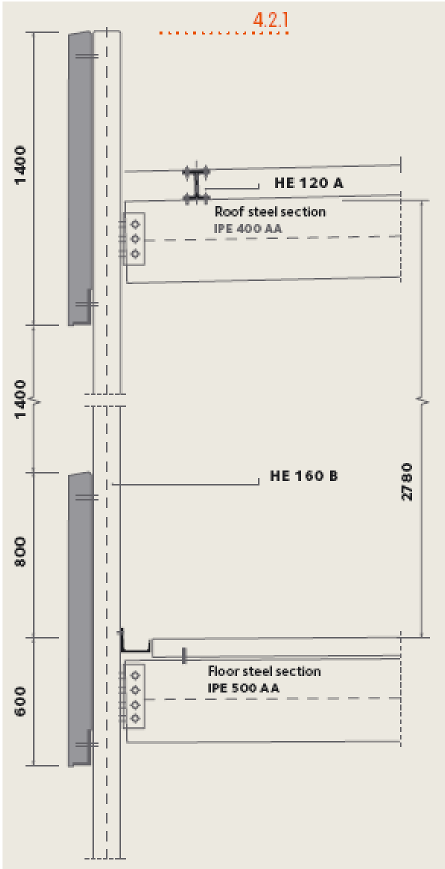


Figure 40. Detail of an open multi-storey car park – ArcelorMittal (1996)

IV.2.4 Prescriptions from ArcelorMittal (2007) in France

According to ArcelorMittal (2007):

- Distinctions are done between open car park, or largely ventilated car park, and closed car park, and between underground car park and aboveground car park;
- Aerial car parks are considered largely ventilated when, for every parking level, the ventilation areas in the walls are situated in at least two opposite facades, equal at least 50% of the total surface area of these facades and correspond to at least 5% of the floor area of one parking level, with a maximal distance of 75m between these facades (Figure 41);
- No fire resistance requirements for steel structures of open multi-storey car parks;
- Localised fire: Scenarios of car fire are proposed;
- Composite beams are required.

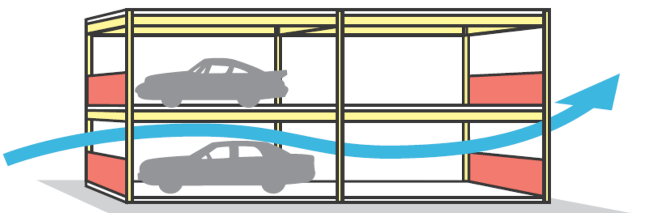


Figure 41. Car park largely ventilated

IV.2.5 Open car park definition in Belgium

A car park is considered opened when the two opposite facades respect the following conditions (Belgium, 2007):

- Distance maximal between these two facades equal to 60m,
- Openings are situated in these two opposite facades, and are equal to at least 1/6 of the total surface area of vertical walls of one parking level,
- Openings are uniformly distributed on the length of each of the two facades,
- Obstacles are admitted between these two facades under conditions (see Belgium, 2007),
- Horizontal distance between these facades and external obstacles has to be at least 5m.

IV.2.6 Prescriptions from Corus (2004) for England, Scotland and Ireland

According to Corus (2004):

- Definition of an Open car park: open ventilation of at least 5% of the floor area at each level, at least half of which should be in opposing walls;
- Dominant period: 15 minutes. Most universal beams and columns will achieve 15 minutes fire resistance without added protection although a small number of sections at the lower end of the range will do so only when less than fully loaded. (In general, where a section does not have 15 minutes inherent fire resistance, it is usually more efficient to increase the section size than to fire protect.)

Table 12. Open sided car parks – Corus (2004)

Requirements from regulations	**Height of top floor**	
	Up to 30m	**Over 30m**
England and Wales	15 minutes *+◊	60 minutes
Scotland	30 minutes #	30 minutes
Northem Ireland	15 minutes *+	Not permitted
Republic of Ireland	15 minutes *	Not permitted

* Increased to 60 minutes for compartment walls separating buildings

\+ Increased to 30 minutes for elements protecting a means of escape

◊ The European Supplement to Approved Document B outlines details of beams and columns, in terms of section factors, which are deemed to satisfy the requirement to achieve 15 minutes fire resistance without protection.

I. Beams supporting concrete floors, maximum section factor = $230m^{-1}$ or open section beams with a minimum lower flange thickness ≥ 9mm.

II. Free standing columns, maximum section factor = 180m-1 or open section columns with a minimum lower flange thickness ≥ 12.5mm.

III. Wind bracing and struts, maximum section factor = 210m-1 or flats, angles and channels with a thickness ≥ 10mm or hollow sections with a wall thickness ≥ 6mm.

A note similar to that in the European Supplement to Approved Document B exists in Technical Standards Part D, page 33D. This says that, where the topmost storey of the building is at a height not more than 18m above ground, the requirement for the structural frame will be met by:

I. Beams supporting concrete floors, each beam having a maximum section factor = 230m-1.

II. Free standing columns, each having a maximum section factor = 180m-1

III. Wind bracing and struts, each having a maximum section factor = 210m-1

IV.2.7 Open-deck car park definition in Australia

According to OneSteel (2004):

- Open-deck and sprinklered car parks can be constructed from bare steel construction provided that the columns and beams achieve certain requirements with respect to their surface-area-to-mass ratio;
- The BCA (Building Code of Australia) defines an open-deck car park as a car park which is cross ventilated using two approximately opposite sides. The sides that provide ventilation must be at least 1/3 of the area of any other side and the opening must be at least ½ of the wall area.

IV.3 Dimensions of the parking bay and the car

Dimensions slightly vary from a country to another. Some typical dimensions are presented hereafter, from the ECCS (ECCS, 1993), ArcelorMittal (ArcelorMittal, 1996, 2007), a German paper of Kurz (Kurz, 2007), Corus (Corus, 2004), and OneSteel in Australia (OneSteel, 2004). Table 13 shows the different standard parking dimensions defined for parking bay set at 90º against the traffic lanes.

Table 13. Bay dimensions for car park with spaces set at 90º

	Bay width (m)	Bay length (m)	Lane wide (m)	Building width (m)
ECCS (1993)	2.5	5.0	-	-
Luxembourg, France – ArcelorMittal (1996, 2007)	2.5	5.0	5.50	15.50
	2.3	5.0	6.50	16.50
Germany – Kurz (2007)	2.4	5.0	6.00	16.00
Britain – Corus (2004)	All	4.8	6.00 (1way aisle)	15.60
			6.95 (2way aisle)	16.55
Australia – OneSteel (2004) *Standard bay dimensions*	2.6	5.4	5.80	16.60

According to Zhao et al. (2004), the bay dimension in real fire scenarios will always be 2.5 x 5.0 m.

IV.3.1 ECCS (1993)

The dimensions of the standard parking bay defined by the ECCS report (1993) are 5.0m long and 2.5m wide when the spaces are set at right angles to the lanes (Figure 42). Car dimensions are 4.8m long and 1.8m wide.

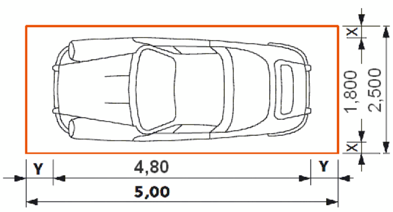


Figure 42. Parking bay and car dimensions – ECCS (1993)

IV.3.2 ArcelorMittal (1996, 2007)

Dimensions of the car slightly vary from the ECCS values: 4.7m long x 1.75m wide (Figure 43).

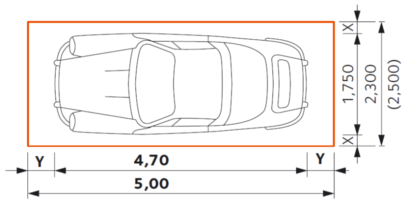
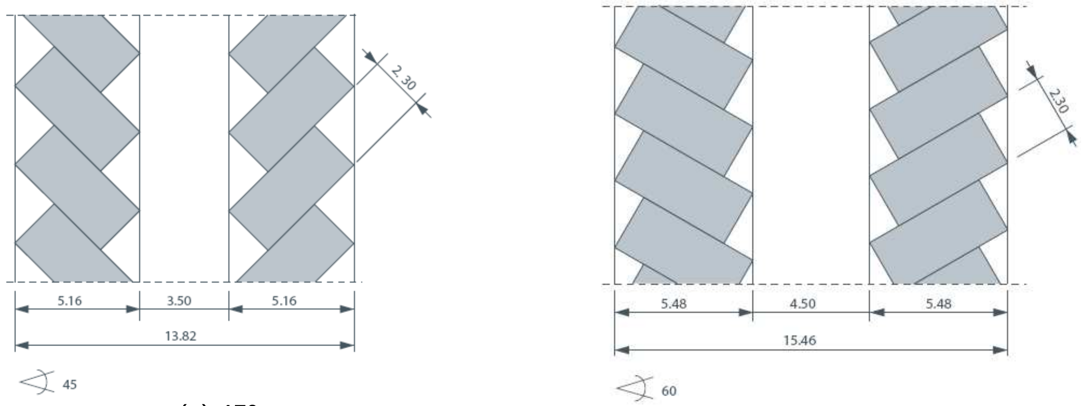


Figure 43. Parking bay and car dimensions – ArcelorMittal (1996)

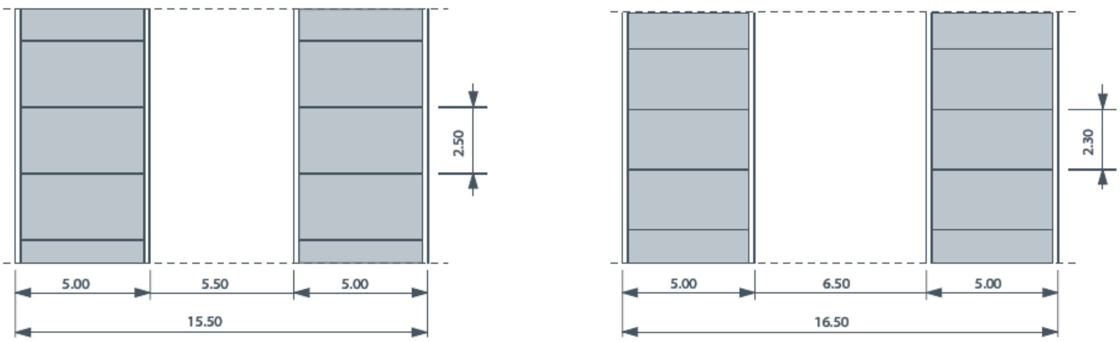
Figure 44 and Figure 45 show the typical dimensions of parking bays at three different angles against the traffic lanes (45º, 60º and 90º) proposed by ArcelorMittal. For spaces set at 90º, the bay wide depends of the lane wide, as shown in Figure 45.



(a) 45º

(b) 60º

Figure 44. Required car park widths for parking spaces set at various angles against the traffic lanes – ArcelorMittal (1996, 2007)



(a)

(b)

Figure 45. Car park widths in function of the lane width (90º) – ArcelorMittal (1996, 2007)

IV.3.3 Kurz (2007) – Germany

Figure 46 shows dimensions given for German car parks in the paper of Kurz (2007) that slightly vary from ArcelorMittal values.

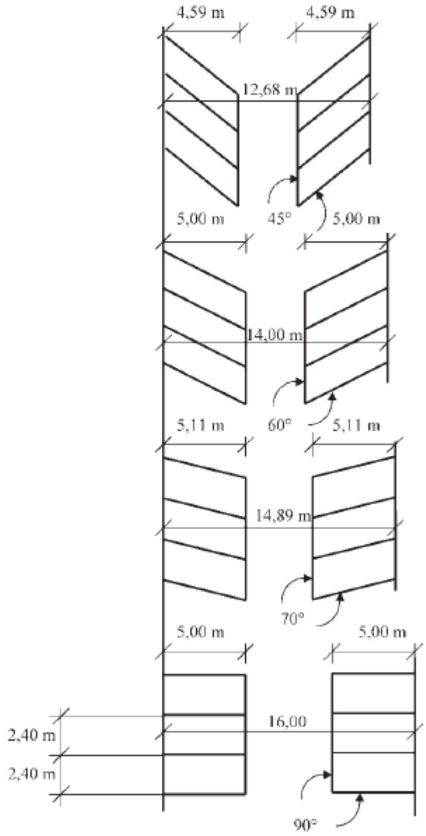


Figure 46. Car park widths – Kurz (2007)

IV.3.4 Corus (2004) – Britain

Typical dimensions for Britain car parks are given in the Table 14. Corus specifies dimensions for a one way aisle or a two way aisle in case of a 90º parking (Figure 47).

Table 14. Effect of varying parking angle on parking bin requirements – Corus (2004)

Parking angle	Stall width (m)	Stall width (m) parallel to aisle	Aisle width (m)	Bin width (m) (stall length 4.80m)
45º	2.3 2.4 2.5	3.25 3.39 3.54	3.60	13.65 13.80 13.95
60º	2.3 2.4 2.5	2.66 2.77 2.89	4.20	14.85 14.95 15.05
75º	2.3 2.4 2.5	2.38 2.49 2.59	4.98	15.45 15.50 15.55
90º	All	All	6.00 - 1 way 6.95 - 2 ways	15.60 16.55

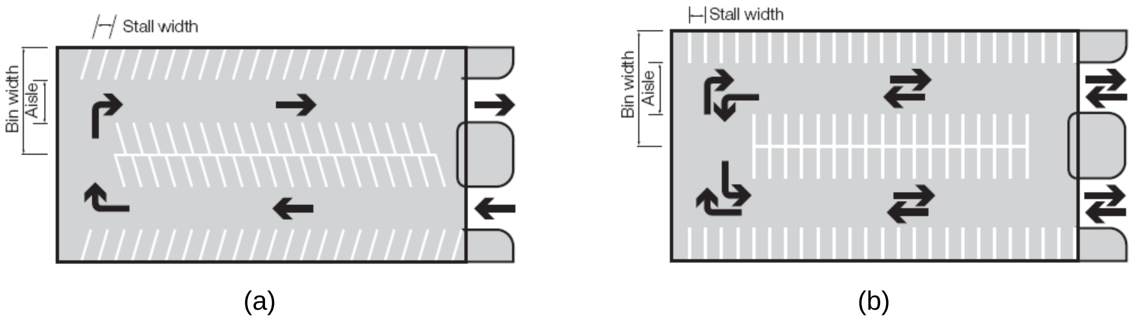


Figure 47. a) One way flow, angled parking; b) two way flow, 90º parking – Corus (2004)

IV.3.5 OneSteel (2004) – Australia

OneSteel defines dimensions according to the car park classification, as shown in Table 15.

Table 15. Car park classifications and dimensions – OneSteel (2004)

User Class	Examples of Uses	Space Width (m)	Aisle Width (m) (Parking at 90º)
1	Generally all day parking e.g. tenantm employee and commuter parking, universities	2.4	6.2
1A	Residential, domestic and employee parking – 3 point turn entry & exit	2.4	5.8
2	Generally medium term parking e.g. long term city and town parkingm sports facilities, entertainment centres, hotels, motels, airport visitors	2.5	5.8
3	Generally short term city and town centre parking, shopping centres, hospitals and medical centres	2.6	5.8
3A	Short term, high turnover parking generally at shopping centres	2.6 2.7	6.6 6.2
4	Parking for people with disabilities	2.4 (+2.4 Shared area)	5.8

IV.4 Suitable car park steel structures

Conception of an open car park slightly varies from a country to another. Some typical dimensions, for barking bay set at 90º against the traffic lane, are presented hereafter, from ArcelorMittal (ArcelorMittal, 1996, 2007), Corus (Corus, 2004), and OneSteel in Australia (OneSteel, 2004). A comparative table of the different steel sections of beams and columns is written in the §IV.4.5.

CTICM redacted the design guidelines in France for this type of structure ("Guide Parkings ouverts au feu CTICM-ARCELOR", Zhao et al. (2004)), which covers more than 85% of the market in Europe with the size and the shape covered within these design guidelines.

IV.4.1 ArcelorMittal (1996) – Luxembourg

IV.4.1.1 Spacing of columns

- Outer columns spacing's: corresponds to the width of one or more parking spaces (units of 2.30 m to 2.50 m);

- Ideally: the distance between columns should correspond to the one of the main girders (avoids secondary beams and optimises the weight of the structure);
- Optimal delimitation of the parking spaces when the distance between columns coincides with the border width of each parking space;
- Secondary beams between columns if the spacing between two columns exceeds 5m.

IV.4.1.2 Sections and steel grade of columns

- Steel grade value recommended: S355. In large-scale structures: high-strength steel S460 featuring a 30% higher yield strength than grade S355.
- Continuous steel columns, with partially encased H or I sections, or with overdesigned steel sections, or with circular hollow steel section full of concrete.

IV.4.1.3 Floor beams

a. Cast-in-place concrete slabs

- Can be made using temporary formwork or permanent formwork made of prefabricated concrete planks or metal deck;
- Temporary formwork: the spacing between the steel beams may be freely selected according to the thickness of the concrete slab. However, for economic reasons, this spacing should not exceed 5 m and, in any case, it is a good idea to take advantage of the effect of connecting the rolled steel sections with the reinforced concrete floor.

b. Composite concrete slabs

- Composite action between the concrete slab and the steel beam via metal studs welded on the upper flange of the H-section;
- Either savings of about 20% in the steel consumption can be achieved (Table 16) or the construction height can be reduced;
- Self-supporting metal decks from ArcelorMittal Construction in combination with cast-in place concrete: shortening construction time, act solely as formwork or in composite action, used for spans up to 3.33 m (special metal decks allowing spans up to 5 m without support during the construction phase).

Table 16. Design example of a composite floor beam with a construction height of 60 cm – ArcelorMittal (1996), updated with the EN1993-1-1:2005 for the partial factor

	Rolled steel sections in S355 steel with non composite, *prefabricated slabs*	Rolled steel sections in S355 steel with *composite slab*, poured on site (grade C25/30)
Span	l = 16.00 m	
Steel section spacing	***b = 2.50 m***	
Excess load	Q = 2.5 kN/m^2	
Slab thickness	100 mm	140 mm
	G = 7.00 kN/m Q = 6.25 kN/m	G = 9.25 kN/m Q = 6.25 kN/m
	Ed = 1.35*7.00 + 1.5*6.25 = 18.825 kN/m M = Ed*16^2/8 = 602.4 kNm	Ed = 1.35*9.25 + 1.5*6.25 = 21.86 kN/m M = Ed*16^2/8 = 700 kNm
Section	***IPE 500***	***IPE 400***
	$M_{ply.Rd}$ = 2194*355/(1.0*1000) = 779 kNm > 602 kNm	The neutral axis lies within the slab: $z_c = (A_a f_y/\gamma_a)/(b_{eff}\ 0.85\ f_{ck}/\gamma_c) = 77$ mm < 140 mm $M_{ply.Rd} = F_a*(h_a/2+h_p+h_c-z_c/2) = 822$ kNm > 700 kNm
Gauge	100 mm + 500 mm = 600 mm	140 mm + 400 mm = 540 mm
	To limit the final deformation, the floor steel sections are given a camber corresponding to a load of G + max 1.3Q	

c. Prefabricated concrete slabs

- *Prefabricated concrete planks*: thicknesses between 5 to 8 cm, can incorporate the lower reinforcement layer of the concrete deck, cast-in-place concrete and the upper layer of reinforcement added on top, unsupported spans up to 5 m (in case of longer spans, temporary supports during construction);
- *Pre-fabricated concrete slabs*: reduced construction time, made in the factory to very tight tolerances and assembled on site, frictional linking of the prefabricated slabs by fixing with prestressed, high-strength bolts to the upper flange of the steel beams, requires very precise construction work, Figure 48 shows cope cuts to create a composite action between the prefabricated concrete slabs and the steel beam via metal studs.

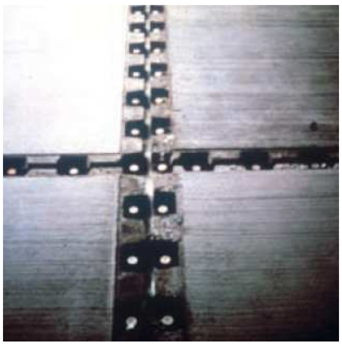


Figure 48. Composite action by sealing the joints with special mortar – ArcelorMittal (1996)

d. Composite beams

- Beams in S355 steel;
- Beams in S460 steel can also be used in composite floors, to save material and costs. Table 17 shows the influence of steel grades and composite or non

composite design for a span of 16 m on the construction height and the weight;

- Steel beams *precambered*: to compensate for the deformation resulting from the application of the dead load (weight of the concrete slab and the steel beam) and part of the life load (usually <30%) (Figure 49).

Table 17. Comparison between different steel grades for a non composite and a composite floor system – ArcelorMittal (1996)

	Non composite floor system			*Composite floor system*		
Span	16.00 m					
Steel section spacing	***5.00 m***					
Prefabricated slab thickness	120 mm			140 mm		
Life load	2.50 kN/m^2			2.50 kN/m^2		
Grade of steel	S235	S355	S460	S235	S355	S460
Section	IPE 750x196	IPE 750x147	IPE 600	IPE 600	***IPE 550***	IPE 500
Height of section (mm)	770	753	600	600	550	500
Height ratio	1.02	1.00	0.8	1.09	1.00	0.91
Linear weight of section (kg/m)	196	147	122	122	106	91
Linear weight ratio	1.33	1.00	0.83	1.15	1.00	0.86

Figure 49. Composite steel sections with camber before placement of the metal decks – ArcelorMittal (1996)

IV.4.1.4 Connection

Usually, beams and columns are connected by bolted angles, Figure 50.

Figure 50. Bolted beam-to-column connection – ArcelorMittal (1996)

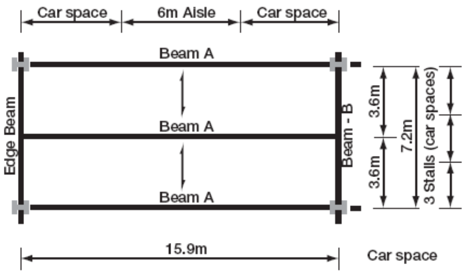
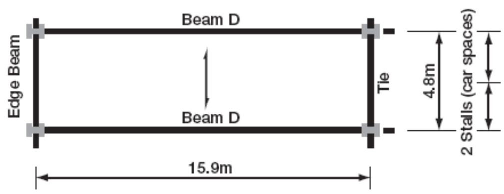
IV.4.2 Corus (2004) – England

Sizing data shits are given by Corus, based upon the following assumptions:

- Design is given for internal and edge beams where appropriate for one bay of a car park;
- Imposed loading is taken as 2.5kN/m^2;
- Grade S355 is used for all main and secondary beam steel;
- Approximate weights of steel given are based on a car park 72m long x 32m wide with car parking spaces at 2.4m wide;
- All in-situ concrete is normal weight, grade 50;
- Imposed load deflection limit: Span/360
- Total load deflection limit: Span/150
- Pre-camber approximately equivalent to deflection due to dead load + 1/3 deflection due to imposed load

Only two of the seven layouts written by Corus (2004) are presented in Table 18.

Table 18. Structural dimensions of a 90º car park with 3 spaces per bay (Layout 1) or 2 spaces per bay (Layout 2), and composite slabs – Corus (2004)

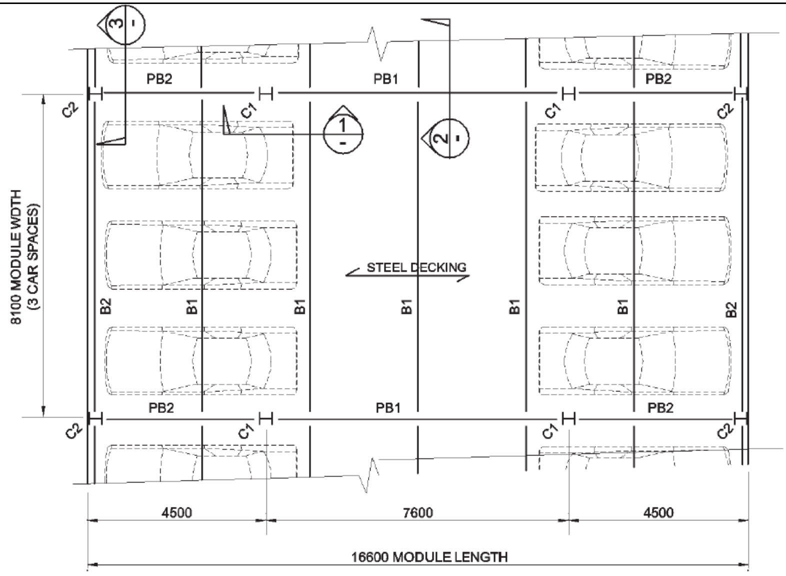
 Layout 1	 Layout 2
Floor construction: 75mm deep precast planks, Composite slab depth – 140mm overall; *Beam 'A':* 610 x 229 x 101 UB, grade S355, Composite with 104 – 19mm diameter shear studs x 95mm long, Camber approximately 80mm; *Beam 'B':* 610 x 229 x 101 UB, grade S355, Composite with 30 – 19mm diameter shear studs x 95mm long; *Columns:* 305 x 305 x 97 UC, grade S355; *Approximate weight of steel:* 45.2 kg/ m^2 (0.86 tonne/parking space); *Overall depth of construction:* 743mm.	*Floor construction:* 100mm deep precast planks, Composite slab depth – 150mm overall; *Beam 'D':* 610 x 229 x 125 UB, grade S355, Composite with 104 – 19mm diameter shear studs x 120mm long, Camber approximately 85mm; *Tie:* 152 x 152 x 23 UC, grade S275; *Columns:* 254 x 254 x 73 UC, grade S355; *Approximate weight of steel:* 36.2 kg/m^2 (0.69 tonne/parking space); *Overall depth of construction:* 762mm.

IV.4.3 OneSteel (2004) – Australia

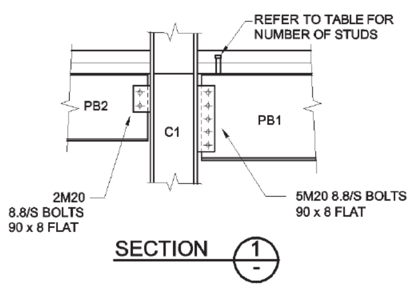
Typical dimensions of steel open car parks are also given by OneSteel, according to the Australian code. Some design criteria are reminded hereafter:

- Design loads: superimposed dead loads equal to 0.1 kPa and live load equal to 2.5 kPa;
- Spacing of secondary beams: about 2.8m centres;
- Partial connection for slabs;
- Columns: floor-to-floor height of 3m;
- 8-level car park.

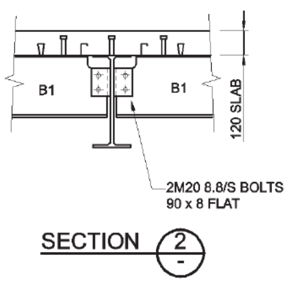
OneSteel presents 11 examples of designed structures as open car parks. Three of these examples are presented in Figure 51, Figure 52 and Figure 53. Details of columns connections are given from Figure 54 up to Figure 57.



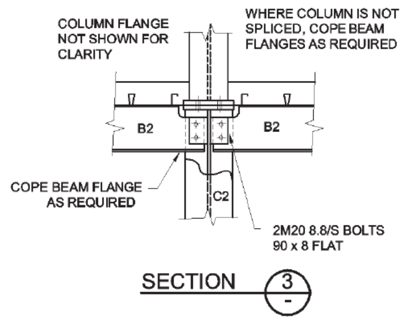
Framing plan – Typical module



Primary beam-to-column connection



Secondary beam to PB connection



External beam-to-column connection

BEAM SCHEDULE

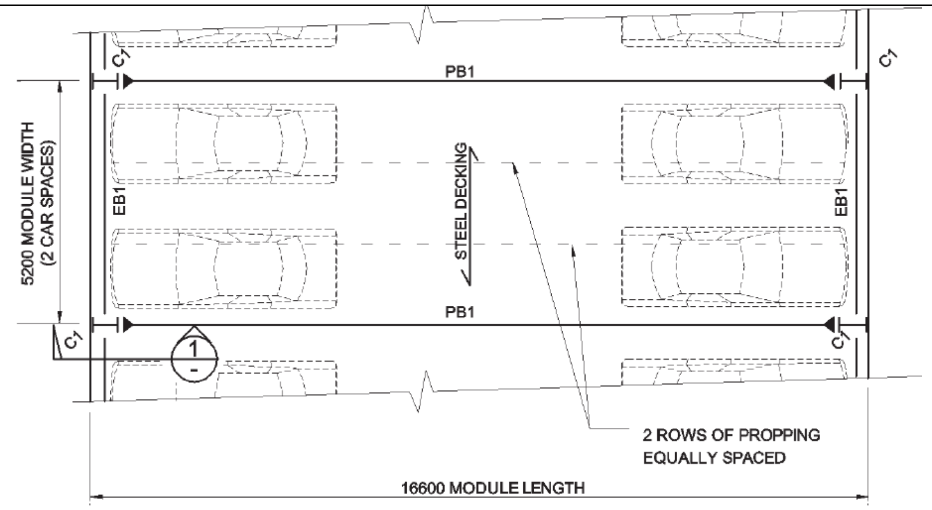
Beam Mark	Beam Size	No. Studs 19mm dia x 95mm high: 200 wide pans	300 wide pans	Camber (mm)
B1	310UB40.4	21	27	25
B2	250UB37.3	0	0	25
PB1	460UB67.1	30	30	20
PB2	360UB44.7	0	0	ncu

ncu - natural camber up

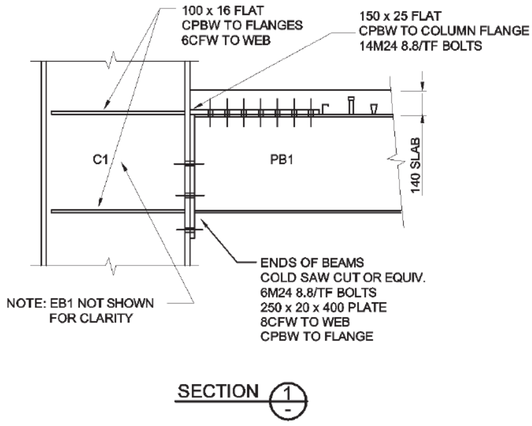
COLUMN SCHEDULE

Level	Column C1	Column C2
8 7	200UC46.2	150UC37.2
6 5	250UC72.9	200UC46.2
4 3	250UC89.5	250UC72.9
2 1	310UC118	250UC89.5

Figure 51. Structural dimensions of a 90º parking with 3 spaces per bay, primary beams in the perpendicular direction to the lane – CASE 1 – OneSteel (2004)



Framing plan – Typical module



BEAM SCHEDULE

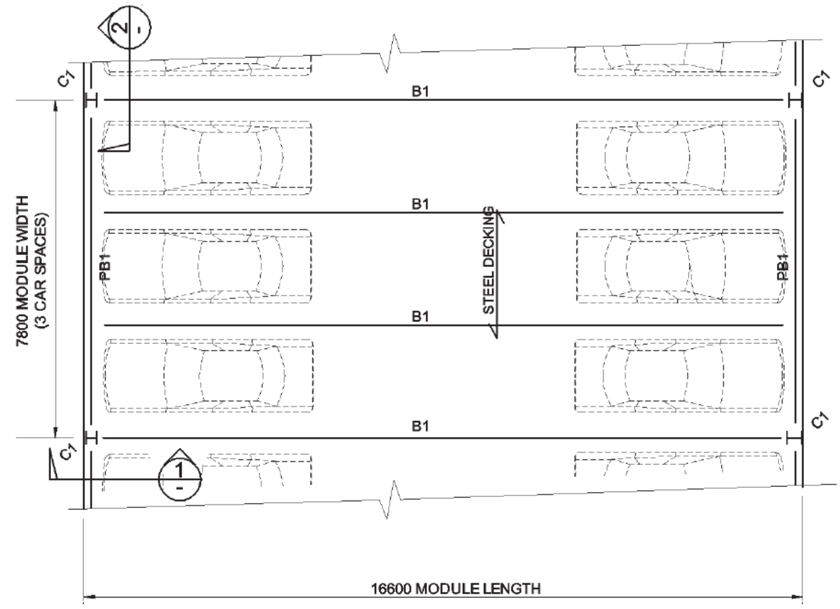
Beam Mark	Beam Size	No. Studs 19mm dia x 95mm high	Camber (mm)
PB1	530 UB 82.0	76	20
EB1	200 UB 25.4	13	ncu

ncu - natural camber up

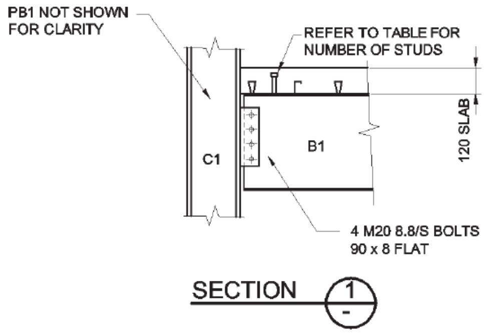
COLUMN SCHEDULE

Level	Column C1
8 7	800WB122
6 5	800WB122
4 3	800WB168
2 1	800WB192

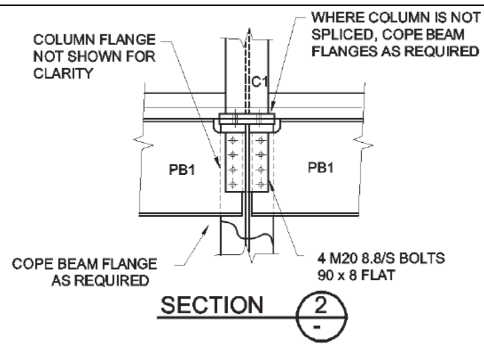
Figure 52. Structural dimensions of a 90º parking with 2 spaces per bay, primary beams in the perpendicular direction to the lane and a maximum span of 16.6m – CASE 2 – OneSteel (2004)



Framing plan – Typical module



Secondary beam-to-column connection



External beam-to-column connection

BEAM SCHEDULE

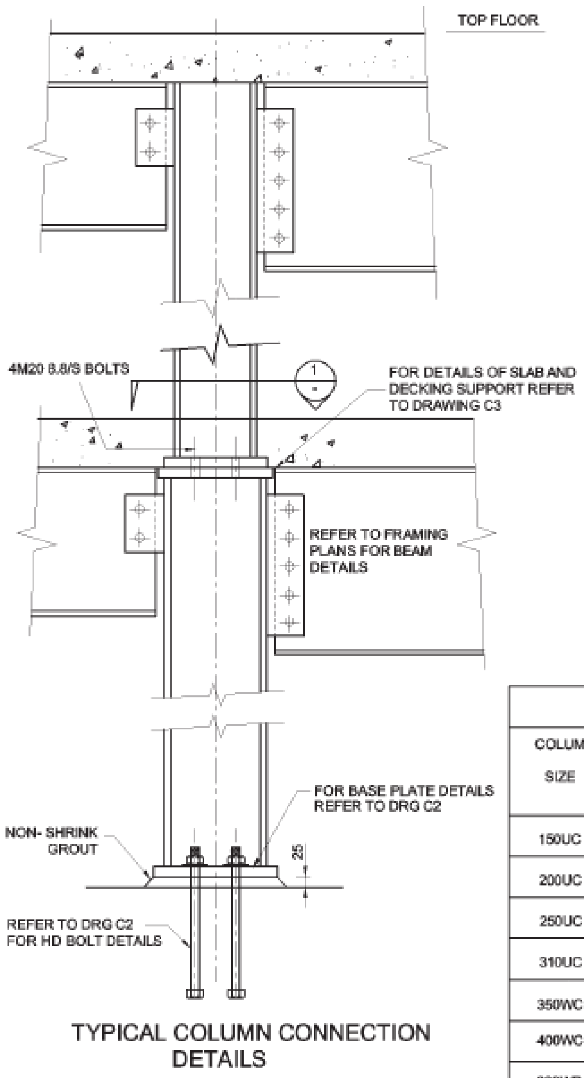
Beam Mark	Beam Size	No. Studs 19mm dia x 95mm high: 200 wide pans	300 wide pans	Camber (mm)
B1	610UB113	46	55	50
PB1	610UB101	0	0	ncu

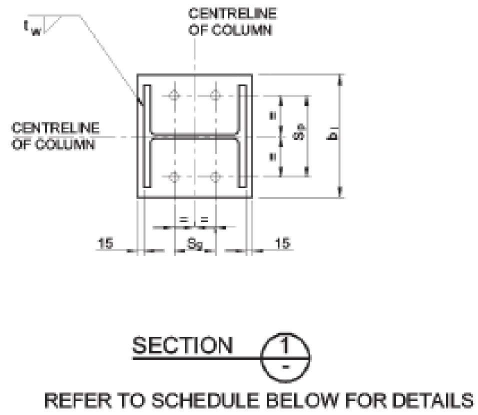
ncu - natural camber up

COLUMN SCHEDULE

Level	Column C1
8 7	200 UC 60
6 5	310 UC 97
4 3	310 UC 137
2 1	310 UC 158

Figure 53. Structural dimensions of a 90º parking with 3 spaces per bay, primary beams in the same direction than the lane and a maximum span of 16.6m – CASE 3 – OneSteel (2004)





COLUMN SPLICE SCHEDULE					
COLUMN SIZE	b_i mm	CAP PLATE THICKNESS mm	S_g mm	S_p mm	FILLET WELD t_w mm
150UC	300	20	100	200	6
200UC	300	20	100	200	6
250UC	300	20	100	200	6
310UC	350	20	150 (100) *	250 (200) *	6
350WC	400	20	150	250	8
400WC	450	25	150	250	8
800WB	350	25	600	200	8

* USE DIMENSION IN BRACKETS WHERE THERE IS A SMALLER COLUMN OVER

Figure 54. Columns connections details – OneSteel (2004)

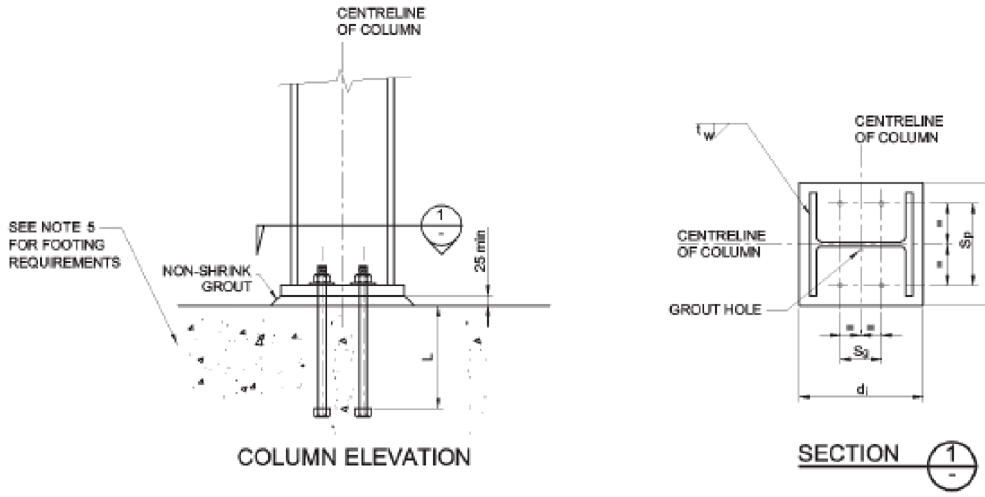


Figure 55. Typical base plate details – OneSteel (2004)

Table 19. Details of dimensions of typical base plate details – OneSteel (2004)

BASE PLATE SCHEDULE								
COLUMN SIZE	d_i mm	b_i mm	BASE PLATE THICKNESS mm	BOLT SIZE GRADE 4.6/S (GALVANISED)	EMBEDMENT LENGTH, L mm	S_g mm	S_p mm	FILLET WELD t_w mm
150UC	230	300	32	4M20	250	100	200	6
200UC	250	300	32	4M20	250	100	200	6
250UC	300	300	32	4M20	250	100	200	6
310UC	400	400	40	4M24	300	150	250	6
350WC	510	510	60	4M24	300	150	250	8
400WC	540	540	60	4M24	300	150	250	8
800WB	850	350	60	4M24	300	600	200	8

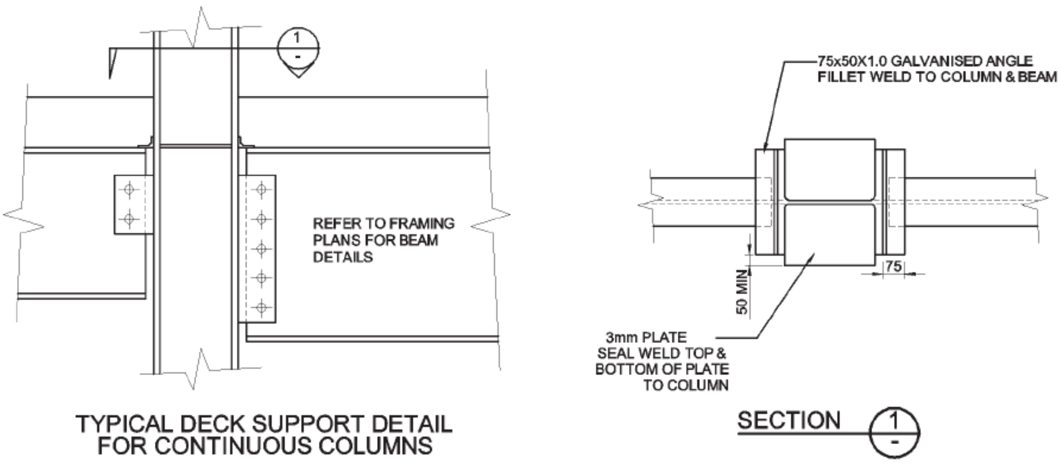


Figure 56. Typical deck support detail for continuous columns – OneSteel (2004)

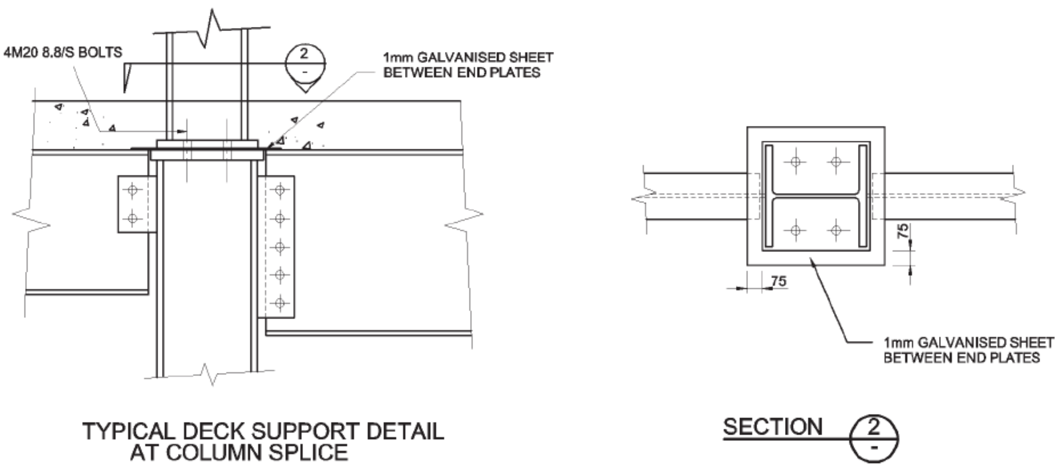


Figure 57. Typical deck support detail at column splice – OneSteel (2004)

IV.4.4 Kurz (2004, 2007) – Germany

The two papers written by the German Kurz about open steel car parks notably give examples of connection types:

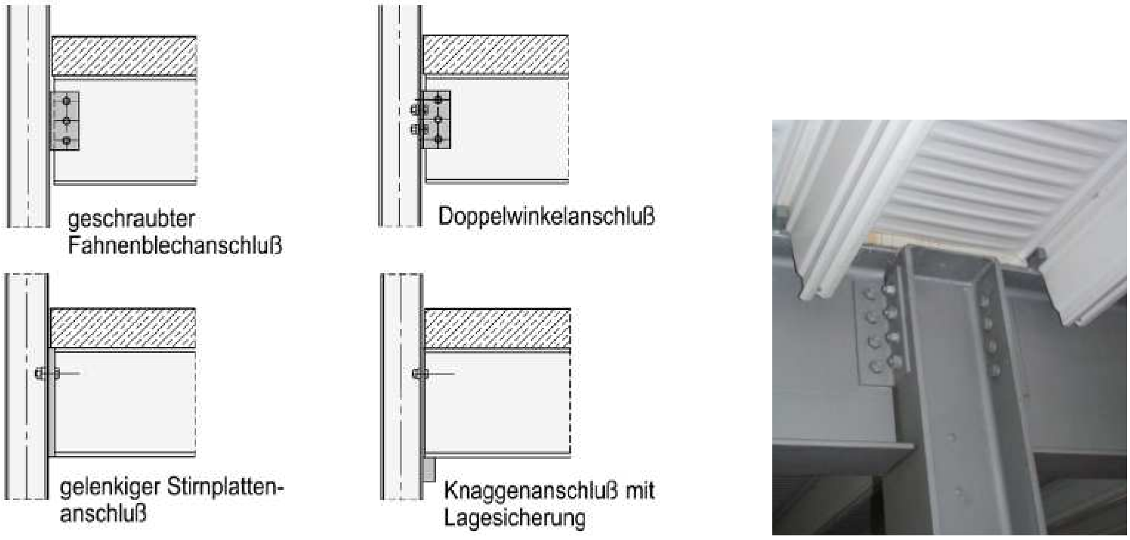


Figure 58. Connections between steel girder and column – Kurz (2004, 2007)

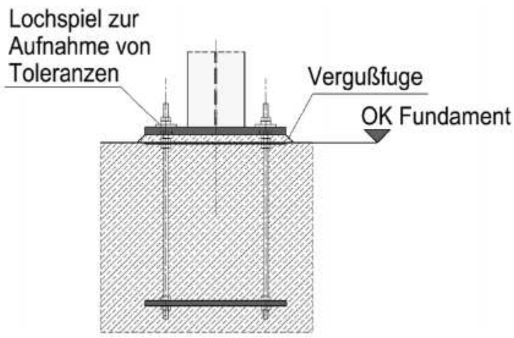


Bild 8. Stützenverankerung
Fig. 8. Fixing of column root

Figure 59. Fixing of column root – Kurz (2004)

IV.4.5 Summary table of typical steel car park structures dimensions

Table 20 gives a synthesis of typical car park structures characteristics from France, Luxembourg, England and Australia presented previously.

Table 20. Synthesis of typical structure dimensions country by country (Parking bay set at 90º against the traffic lane, Steel S355, Common storey)

<table>
<tr><th>Reference of the car park structure example</th><th>Steel column</th><th>Primary beam (Section/span)</th><th>Secondary beam (Section/ span/spacing)</th><th>Slab (Type/ thickness)</th><th>Connection type</th><th>Link in the report</th></tr>
<tr><td rowspan="2">ArcelorMittal (1996), Luxembourg</td><td rowspan="2">-</td><td>- / -</td><td>IPE 400/ 16m/2.5m</td><td>Composite slab poured on site / 140mm</td><td rowspan="2">Bolted beam-to-column connections (Bolted angles)</td><td>Table 16</td></tr>
<tr><td>- / -</td><td>IPE 550/16m/5m</td><td>Prefabricated slab / 140mm</td><td>Table 17</td></tr>
<tr><td rowspan="5">ArcelorMittal (2007), France, Table 4, §I.3.2</td><td>HEB 240*</td><td>HEA 500/ 10m</td><td>IPE 450/ 16m/2.5m</td><td rowspan="3">Composite slab / 120mm</td><td rowspan="5">Bolted connections</td><td>Case 1</td></tr>
<tr><td>HEB 240*</td><td>HEA500/ 10m</td><td>IPE 500/ 16m/3.33m</td><td>Case 3</td></tr>
<tr><td>HEB 340*</td><td>IPE 400/ 7.5m</td><td>IPE 240/ 7.5m/2.5m</td><td>Case 6</td></tr>
<tr><td>HEM 300*</td><td>HEAA 650/ 10m</td><td>IPEA 600/ 16m/5m</td><td rowspan="2">Concrete slab / 160mm</td><td>Case 2</td></tr>
<tr><td>HEB 400* (S460)</td><td>IPE 400/ 5m</td><td>IPEA 600/ 16m/5m</td><td>Case 5</td></tr>
<tr><td>Joyeux et al (2002), France</td><td>HEB 200 (centre) HEA 180 (extremities) - 2 levels</td><td>IPE 500 / 5m (centre) IPE 400 / 5m (extremities)</td><td>IPE 550/ 16m/2.5m</td><td>Composite slab / 120mm</td><td>End-plate connections, Angles connections, Columns hinged at the bottom</td><td>Figure 7</td></tr>
<tr><td>Joyeux et al (2001), France</td><td>HEB 240 - 4 levels</td><td>- / -</td><td>IPE 400/ 16m/2.4m</td><td>Concrete slab / 100mm (C45/50)</td><td>-</td><td>Figure 9</td></tr>
<tr><td rowspan="2">Corus (2004), England</td><td>305 x 305 x 97 UC</td><td>610x229x101 UB/ 7.2m</td><td>610x229x101 UB/ 15.9m/3.6m</td><td>Composite slab / 140mm</td><td rowspan="2">-</td><td rowspan="2">Table 18</td></tr>
<tr><td>254 x 254 x 73 UC</td><td>610x229x125 UB/ 15.9m/4.8m</td><td>152x152x23 UC/ 4.8m</td><td>Composite slab / 150mm</td></tr>
<tr><td rowspan="3">OneSteel (2004), Australia</td><td>250UC89.5 and 310UC118 (level 1)</td><td>460UB67.1/ 7.6m 360UB44.7/ 4.5m</td><td>310UB40.4 (interior) 250UB37.3 (external) / 7.5m/ 2.8m</td><td rowspan="3">Composite slab / 120 or 140mm</td><td rowspan="3">Bolted connections</td><td>Figure 51</td></tr>
<tr><td>800WB192 (level 1)</td><td>530UB82/ 16.6m</td><td>200UB82/ 5m/ 16.6m</td><td>Figure 52</td></tr>
<tr><td>310UC158 (level 1)</td><td>610UB101/ 7.5m</td><td>610UB113/ 16.6m/ 2.5m</td><td>Figure 53</td></tr>
</table>

*: Columns designed to the fire being partially-encased with concrete

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ANNEX B – Beam-to-column joints in fire

Departamento de Engenharia Civil
Faculdade de Ciências e Tecnologia da Universidade de Coimbra

ROBUSTFIRE Project

Document 2 – Joints – v1(9)

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UC/DEC/ISISE/2009-002 RFS-PR-07039-Joints-v1(9)

EXECUTIVE SUMMARY

Traditionally, beam-to-column joints are assumed to have sufficient fire resistance due to cooler temperatures and slower rate of heating, caused by the concentration of mass on the joint area. However, real fires and experimental observations show that on several occasions steel joints also fails because of high strains induced by the distortional deformation of the connected members (Santiago, 2008). Beam-to-column connections in a fire are exposed to a combination of forces and moments, significantly different to the single bending moment and shear force assumed in ambient design. The additional moments and axial forces in the beam originate from restraint thermal expansion, large vertical deflections and rotations and the effects of cooling on a plastically deformed structure (Block, 2006a).

During the heating phase, local buckling of the beam bottom flange and shear buckling at the web in the vicinity of the joint are the main failure mechanisms. These local buckling’s occur due to the restraint to thermal elongation provided by the adjacent cooler structure. Local buckling of the beam flange has been found to be highly significant in accelerating the development of catenary action in fire, since this action is reliant on hinges forming which may result from local buckling. Local buckling of the beam web, which experiences a non-uniform temperature variation, is also important because the mechanical properties of the steel are degraded non-uniformly from their ambient values (Heidarpour, 2007).

During the cooling phase, the failure of joints from their tensile components, such as bolts or end-plates, is an unwanted brittle failure. This results from reversal of bending moment, that leads to large sagging moments, since the Young modulus and resistance recover their values and the temperature decreases faster on the bottom flange than on the top (Santiago, 2008).

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I State of the art

I.1 Introduction

This document reports a state of the art on the behaviour of beam-to-column joints in fire and considers experimental, numerical and analytical research works. The state of the art is based on two review papers of Luís Simões da Silva (Simões da Silva et al., 2005) and Khalifa Al-Jabri (Al-Jabri et al., 2008) that are including in Annexes A and B. As the more recent review paper (Al-Jabri et al., 2008) has only been written in 2005, recent developments do not appear in it. The Ph. D. Theses of Florian Block (Block, 2006), Ramli Sulong (Sulong, 2007), Amin Heidarpour (Heidarpour, 2007) and Aldina Santiago (Santiago, 2008) have been presented since this review paper of Al-Jabri (Al-Jabri et al., 2008). The research project "*Robustness of steel connections in fire*" has also been conducted by the Sheffield and Manchester Universities (Hu et al., 2008a-b; Yu et al, 2008a-b-c), and the COSSFIRE European project (Zhao et al., 2007) is still ongoing. The two review papers, the Ph. D. Theses and the two research projects are presented in the following sections.

I.2 Simões da Silva et al., 2005

This review paper is reproduced in full in annex A.

Simões da Silva L., Santiago A., Vila Real P., Moore D. "Behaviour of steel joints under fire loading". International Journal of Steel and Composite Structures 5(6), 485-513, 2005.

I.3 Al-Jabri et al., 2008

This review paper is reproduced in full in annex B.

Al-Jabri K.S., Davison J.B., Burgess I.W. "Performance of beam-to-column joints in fire - A review". Fire Safety Journal 43, 50-62, 2008.

I.4 Recent developments

I.4.1 Ph. D. Thesis of Florian Mauricius Block (2006)

A component based connection element was developed which is able to predict the behaviour of bolted flush and extended end-plate beam-to-column connections. The new element represents the behaviour of such connections under the influence of bending moment, normal force in the beam and column, and increasing and decreasing temperatures. It was implemented into the specialised finite element software *Vulcan* which was developed to predict the behaviour of steel, composite and concrete structures in fire.

The research work presented in the thesis was inspired by the observation that beam-to-column connections in a fire are exposed to a combination of forces and moments, significantly different to the shear loading and bending moment of the connection assumed in ambient design. The additional moments and axial forces in the beam originate from restraint thermal expansion, large vertical deflections and rotations and the effects of cooling on a plastically deformed structure. As connections are normally not designed to withstand those additional forces they could fail to transfer the beam forces to the column which could lead to a progressive collapse of the frame structure.

Due to the combination of axial load and moment, together with the large variety of possible connections, the Component Method was used as a basis of the new element. This method separates a joint into its zones of fundamental behaviour i.e. tension, compression and shear which are then represented by force-displacement curves calculated from simplified mathematical models. Finally, the joint is reassembled as a rigid bar-spring model in which each zone is represented as a spring.

A large part of this thesis concentrated on the development of a simplified component model for the compression zone in the column web including the effects of elevated temperatures and axial load in the column. Experimental, numerical, statistical and mechanical studies were used to develop and validate this model.

The new connection element was compared with a number of ambient and elevated temperature experiments on connections with good success.

Finally, a plane frame connected with different end-plate connections was analysed during the heating and cooling phase of a fire, showing the vulnerability of connections and bolts in particular during the cooling phase of a fire.

I.4.2 Ph. D. Thesis of Nor Hafizah Ramli Sulong (2007)

A new analytical model for steel connections was presented and discussed in this thesis. The model is based on the 'component-based approach' and is capable of representing the behaviour of typical connection configurations in both steel and composite framed structures. Both monotonic and cyclic loading conditions can be represented at ambient as well as elevated temperatures. The implementation was undertaken within the advanced finite element program ADAPTIC, which accounts for material and geometric nonlinearities.

Validation of the proposed connection model was carried out by comparison against available experimental results for several joint types and loading conditions. The findings indicated the reliability and efficiency of the suggested model. A series of parametric studies for different structural configurations were presented in order to demonstrate the scope of application of the proposed model.

Finally, the implications of the studies carried out on performance-based structural fire design were assessed. Based on findings from the analytical examinations, more simplified approaches, suitable for implementation within design procedures, were proposed.

Some observations were underlined:

- For the endplate connection under combined bending and axial loading, the plastic mechanism and mode of failure changes from Mode 1 to Mode 2 as temperature increases, with the degradation of the material properties of the component with temperature;

- By incorporating the actual stiffness and strength characteristics of the connection, the fire resistance of a beam could be enhanced compared to the case of an idealized pin-representation;
- The modelling of a composite floor showed that the overall structural response was not significantly influenced by the behaviour of the connections. But by examining the connection axial and bending response, it was shown that the beam response can be limited by the capacity of the connection in both, compression and tension conditions. Then it is important to have a realistic representation of the connections in order to assess the local load and deformation demands imposed on the joints.

I.4.3 Ph. D. Thesis of Amin Heidarpour (2007)

This research work investigated the behaviour of steel and composite beams as well as beam-to-column connections at elevated temperatures. Current formulations for beam web buckling at ambient temperature needed substantial revision at elevated temperatures. Very large compressive forces developed within the beams initially was not considered and these induce large stresses in the column web. In this thesis were conducted:

- The development of a formulation representing the mechanics of the potential failure zone in the column web in the compression region of the connection;
- The prediction of the critical temperature in a connection that causes failure of the bolts, end-plate and column flange in the tension zone of the connection;
- The development of an elastic analysis of a panel zone in a rigid or semi-rigid joint in a steel frame at elevated temperatures, which is based on simple equilibrium considerations that takes into account the shear and flexural deformations of the panel zone;
- The presentation of a geometrical nonlinear formulation in order to model the structural response of a composite beam restrained. This formulation incorporates partial interaction between the concrete slab and steel component, as well as the degradation of the stiffness's of the components of the composite beam prior to yield at elevated temperature.

I.4.4 Robustness of steel connections in fire (2005-2008)

The Universities of Sheffield and Manchester conducted a joint research programme with the aim of investigating the capacity and ductility of steel connections at elevated temperatures. In the test programme the connections were subjected to combinations of shear force and tying force (Figure 1), and loaded to large deformation and fracture. Fin plate, flush end-plate, web cleat and flexible end-plate connections were studied.

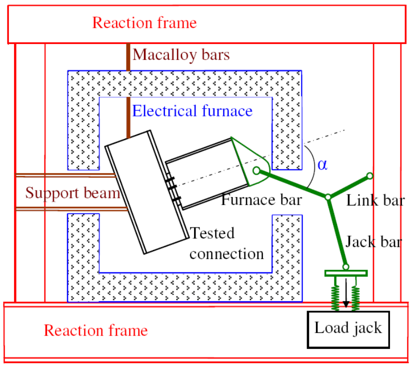

Figure 1. Test setup – Yu et al. (2008a)

In the case of fin plate connections, the test results indicated that bolts are vulnerable to shear fracture and that failure is usually controlled by bolt shear rather than by plate bearing. Fin plate connection resistance reduces rapidly with increase of temperature (Yu et al., 2008c).

For flush end-plate connections, observed failure types in the tests tended to change the failing component from the end-plate to the bolts with increase of temperature, as the bolt strength reduces faster than that of steel in fire. At elevated temperatures, the use of thick end-plates can enhance the peak resistance, but reduces the rotational capacity of the connection (Yu et al., 2008a).

Web cleat connections tests results showed that the tying capacity decreases rapidly with increase of temperature, and that the connection has little residual resistance at 650℃. Web cleat connections are capable of failing in a number of ways. The failure mode appears not to be sensitive to the load combinations, but is dependent on the temperature. Fracture of the web cleat close to its heel, and double shear of the bolts through the beam web are the two critical failure modes at elevated temperatures. In general, web cleat connections have extremely high rotational capacity compared with alternative types (Yu et al., 2008b).

The review of experimental results of flexible end-plate connections indicated that the minimum tying resistance of 75 kN might not be assured for flexible end-plate connections in a fire situation. As the rotation capacity of end-plate connections is reduced as increasing temperatures, the connections may not possess the extensive ductility required for catenary action and may rupture before catenary action is fully developed. In consequence, specifying partial depth end-plate connections seems inadvisable for robustness of steel structures where large connection rotations are anticipated (Hu et al., 2008b).

In order to investigate its resistance and ductility at ambient and elevated temperatures, a three dimensional numerical model has been created for a flexible end-plate connection, using the ABAQUS finite element code. In comparison with experimental test data, a good correlation with the finite element analysis is achieved and the method is suitable to study the tying resistance and ductility for simple steel connections with various dimensions at different temperatures (Hu et al., 2008a).

I.4.5 Ph. D. Thesis of Aldina Santiago (2008)

The research work reported in this thesis dealt with the following subjects: i) the characterisation of the behaviour of beam-to-column joints in fire, and ii) their influence on the overall behaviour of the structure subjected to a natural fire. Special emphasis was directed to the cooling phase, because it induces an unwanted failure mode: brittle failure of the tensile components. The experimental results from a full-scale building fire test and six beam-to-column sub-frames fire tests under a natural fire were presented and discussed based on their temperature development, structural deformability and failure modes. The results were used to support and validate numerical models.

The analytical approach of this research involved the proposal and validation of a design methodology for steel joints under fire loading. The model is based on the Eurocode 3 approach and includes the representation of all components to allow the application of any combination of bending moment and axial forces that characterise a fire situation.

The main conclusions of this thesis were:

- One of the main problems during the cooling of the fire is the tensile failure of the bolts;
- During the cooling phase, the lower bolt row showed the highest principal strains. This result from the reversal of bending moment, that leads to large sagging moments, since the Young modulus and resistance recover their values and the temperature decreases faster on the bottom flange than on the top;
- The application of a thin end-plate is in agreement with the ductility criteria expressed in EN 1993-1-8-2005, and it demonstrated to be a good option to reduce the large bolt deformations and consequently the bolt failure. However, even if no bolt failure was observed, the equivalent strains on the end-plate increased to values higher than those assumed to the end-plate cracking. This was observed not only near the lower bolt row but also in the upper one;
- Additional bolt rows in the lower zone of the connections should be considered in order to increase the joint resistance during the cooling phase;
- A header plate demonstrated to be the worst option. It reduced bending resistance and was not enough to resist to the increase of the hogging moments developed during the heating phase.

I.4.6 COSSFIRE Project, ongoing (2006-2009)

The main aim of this project is to enhance the scientific findings and to develop efficient, practical and economic design rules on steel and steel and concrete connections when exposed to real fire conditions (Zhao et al., 2007). The work programme of this project includes four principle tasks:

1. A detailed bibliography analysis of both test results and calculation models on connections under standard fire conditions. In addition to the collection of existing data, a review of commonly used types of steel joints over both UK

and French markets was conducted, and it concluded that the following types of bolted steel joints are very commonly used for both steel and composite frames: end-plates, double angle web cleats and fin plates.

2. Some tests under natural fire heating conditions on connection components, on full structural steel joints and on connections between concrete slabs and border steel members in case of composite steel members:

   - An experimental investigation of the behaviour of different connection components under natural fire conditions has been done. 40 fire tests of bolts have been conducted up to now. The fire tests of welds will start very soon.

     The first analysis of obtained test results showed clearly that the bolts start to loss their strength once heated beyond certain temperature level. During the cooling phase, the bolts recover their initial strength if the maximum heating does not exceed 400℃. However, wit h more important heating levels, the initial strength of the bolts can no longer fully recover. It can be found that the higher the maximum heating level is, the lower the recovered strength of the bolts will be. These test results are very useful because they enlighten in detail the performance of one of most important joint components, that is bolt in case of real fire condition. Apparently, the risk of bolt broken during cooling phase of real fire is really possible if two conditions are met, that is the high heating level and important restrained effect in tension.

   - 8 fire tests of connections will be done: 4 on steel connections and 4 on steel-concrete composite connections. The first two fire tests on steel connections have already been done. The experimental results of the Coimbra tests (Santiago, 2008) have been taken into account to design the connection tests predicted and a comparison of the tests results has been done.

     From the first two fire tests some major conclusions are: i) failure of connections can be caused by tension in beams, especially during the cooling down phase; ii) tension in the beam is likely to occur if the maximum deflection during heating is high; iii) tension in the beam will probably be much higher when the level of longitudinal restraint is high; iv) maximum tension in a beam will increase during cooling down, and at the same time strength of the bolts will (partly) recover. Whether or not failure of the bolts occurs depends on the developing tension force on one hand and the strength recovery on the other hand. Failure may occur during a late stage of cooling down.

   - The fire behaviour of connections between concrete slab and steel members at the border of composite floor will be studied by a global floor test, including four different edge connection configurations, realistic restrained effects applied to connections with used global floor and a uniformly distributed load.

3. Some numerical simulations of existing available experimental data on steel connection as well as the fire tests to be performed within the scope of both this project and the Precious project (Bursi et al., 2008);

4. A development of simple design rules as well as practical design guidance on various commonly used types of connections for fire situations.

This project is ongoing and should be finished on the 31 December 2009. The further tasks will be to end all connection component, steel and composite connections, and the global steel and concrete composite floor tests at elevated temperatures; to develop the relevant material model of joint components for real fire conditions; to make parametric numerical studies of steel joints and the composite floor under real fire conditions; to improve and develop a simple calculation method for predicting the restrained effects of steel members induced by the heating condition of real fires, and a global design guide.

I.4.7 Conclusions about the recent developments performed for steel and composite joints in fire

All recent developments underline that beam-to-column connections in a fire are exposed to a combination of forces and moments, significantly different to the single bending moment and shear force assumed in ambient design. Most of the previous research works studied the connection behaviour during the heating phase of the fire, while Santiago (Santiago, 2008) and COSSFIRE Project (Zhao et al., 2007) put emphasis also on the behaviour of connections during the cooling phase.

I.4.7.1 The heating phase

During the heating phase, members in frames that are restrained axially by other cooler members experience significant temperature-induced compression forces that are coupled with bending actions, which produce local buckling of the steel section. The combined compression, bending, geometric nonlinearity and thermally degraded material properties may lead to inelastic local buckling or to the development of plastic hinges as the temperature increases. At high temperature, flange local buckling and yielding then reduce the stiffness at a localised region (typically near a connection), so that the member rapidly develops tensile actions that can result in catenary actions (Heidarpour, 2007).

The experimental research work led by the Universities of Manchester and Sheffield studied failure modes of four types of connections under elevated temperatures (Hu et al., 2008b; Yu et al., 2007, 2008a-b-c)

Block (Block, 2006), Sulong (Sulong, 2007) and Santiago (Santiago, 2008) developed new models based on the component-based approach, to study numerically connections under elevated temperatures submitted to the combination of forces and moments. The analysis of connections behaviour were performed using Block and Sulong models implemented into, respectively, the finite element software Vulcan and the finite element program Adaptic. They notably concluded that, by incorporating the actual stiffness and strength characteristics of the connection, the fire resistance of a beam could be enhanced compared to the case of an idealized pin-representation, and that the beam response can be limited by the capacity of the connection in both, compression and tension conditions.

I.4.7.2 The cooling phase

During the cooling phase, the beam recovers its strength and stiffness, the thermal expansion reduces, and the maximum tension in the beam increases. The beam contraction depends on the temperature development and mainly on the tensile restraint. Several recent developments underline that during the cooling phase, connections and bolts are particularly vulnerable. One of the main problem is the tensile failure of the bolts. The lower bolt row shows the highest principal strains, which results from the reversal of bending moment, that leads to large sagging moments, since the Young modulus and resistance recover their values and the temperature decreases faster on the bottom flange than on the top (Santiago, 2008). According to the first results of the COSSFIRE project (Zhao et al., 2007), the risk of bolt broken during cooling phase of real fire is really possible if two conditions are met, that is the high heating level and important restrained effect in tension.

II Complete list of references on the behaviour of steel and composite joints in fire

II.1 Introduction

The present chapter proposes a complete list of references on the behaviour of steel and composite joints in fire. It is divided into 3 subsections referring to experimental, numerical and analytical research works. In each subsection, papers are classified by chronological order. The main author's name, the subject and the research date are first detailed; then relevant references to this research work are written.

II.2 Experimental Research

1) ***Kruppa, CTICM***, Tests of flexible to rigid connections (6 joint types) to establish the performance of high strength bolts, 1976.

   → First measurements of the temperature distribution around the joint area

   Kruppa J. "Resistance au feu des assemblages par boulons haute résistance". St. Rémy-les-Chevreuse, Centre Technique Industriel de la Construction Métallique, France, 1976.

   → Same type of tests realised by ***British Steel***, 1982.

   British Steel. "The performance of beam/column/beam connections in the BS476: Art 8 fire test". Reports T/RS/1380/33/82D and T/RS/1380/34/82D.

2) ***Lawson***, 8 bolted beam-to-column connection tests (cruciform arrangement) to quantify the benefits of structural continuity under fire conditions and to develop a design approach for steel beams taking the rotational restraint provided by the joints into consideration, 1990.

   → First tests to measure the structural continuity afforded by beam-to-column connection at elevated temperature.

   Lawson R.M. "Enhancement of fire resistance of beams by beam-to-column connections". Technical Report 086, The Steel Construction Institute, 1990a.

   Lawson R.M. "Behaviour of steel beam-to-column connections in fire". The Structural Engineer 68(14), 263-271, 1990b.

3) ***Leston-Jones***, The influence of semi-rigid connections on the performance of steel framed structures in fire, UK, 1997.

   Leston-Jones L.C. "The influence of semi-rigid connections on the performance of steel framed structures in fire". Ph.D. thesis, University of Sheffield, 1997.

   → 11 tests on small-scale specimens (bare steel and composite connections) to study their behaviour in fire; 2 tests at ambient temperature with cruciform connections and flush end-plates.

→ Modelling the response of bar steel and composite flush end-plates joints by a component-based model to study the elevated temperature joint's behaviour.

→ Analytical relationship developed by ***El-Rimawi et al.*** (Ramberg-Osgood equation) adapted to represent graphically the moment-rotation behaviour of semi-rigid connections in fire based on data from experimental tests.

Leston-Jones L.C., Burgess I.W., Lennon T., Plank R.J. "Elevated temperature moment-rotation tests on steelwork connections". Proceedings of the Institution of Civil Engineers, Structures & Buildings 122, 410-419, 1997.

Al-Jabri et al. extended the Leston-Jones study to include the following parameters: member size, connection type, different failure mechanisms, 1999.

Al-Jabri K.S. "The behaviour of steel and composite beam-to-column connections in fire". Ph.D. thesis, University of Sheffield, 1999.

→ 5 series of tests at elevated temperatures at the Building Research Establishment, including flush and flexible bare-steel connections, and flexible end-plate composite connections; Results were derived into moment-rotation-temperature curves for different connection type,

Al-Jabri K.S., Lennon T., Burgess I.W. and Plank R.J. "Behaviour of steel and composite beam-column connections in fire". Journal of Constructional Steel Research 46(1-3), 308-309, 1998.

→ Parametric study of a typical sub-frame, to study the effect of connection type, end-plate thickness, concrete strength, load ratio and connection temperature, using a finite element program developed at Sheffield University,

→ Comparisons with the large-scale fire tests on the composite building at BRE's Cardington laboratory,

→ Development of a simplified component-based model to predict the behaviour of steel and composite flexible end-plate joints at high temperatures; Good agreement when comparing to the experimental tests.

Al-Jabri K.S. "Component-based model of the behaviour of flexible end-plate connections at elevated temperature". Composite Structures, Vol. 66, 215-221, 2004.

Al-Jabri K.S., Burgess I.W., Plank R.J. "Spring-stiffness model for flexible end-plate bare-steel joints in fire". Journal of Constructional Steel Research, 61, 1672-91, 2005b.

→ Simple procedure to enhance the Ramberg-Osgood equation to generate the moment-rotation curves of steel and composite end-plate joints at high temperatures; Modelling of the behaviour of bare steel and composite flexible end-plate joints, 2003, 2005.

Al-Jabri K.S. "Numerical modelling of the behaviour of bare-steel and composite flexible end-plate connections at elevated temperatures". Proc. Conference Advances in Structures, 635-641, Australia, 2003.

Al-Jabri K.S., Burgess I.W., Lennon T., Plank R.J. "Moment-rotation-temperature curves for semi-rigid joints". Journal of Constructional Steel Research, 61, 281-303, 2005a.

→ Utilisation of the ABAQUS code to model the behaviour of flush end-plate steel joints: good comparison of the moment-rotation curves and the failure mode with experimental results, 2006.

Al-Jabri K.S., Seibi A., Karrech A. "Modelling of unstiffened flush end-plate bolted connections in fire". Journal of Constructional Steel Research, 62(1-2), 151-159, 2006.

4) ***Spyrou et al.***, Performance of the tension and compression zones of steel joints at elevated temperatures, 2002, 2004.

Spyrou S. "Development of a component-based-model of steel beam-to-column joints at elevated temperature". Ph.D. Thesis, University of Sheffield, UK, 2002.

→ 45 tests with tensile loads and T-stub assemblies at elevated temperature to investigate the 3 failure modes of the T-stub

+ Compression zone tests: 29 column web transverse compression tests (heated up to the required Tº before applying compression forces).

→ Development of components models at ambient and elevated temperature for the tension and compression zones, then combination of these models into a simple component-based model. Good correlation with previous elevated temperature moment-rotation tests (Leston-Jones, 1997; Al-Jabri, 1999).

Spyrou S., Davison J.B., Burgess I.W., Plank R.J. "Experimental and analytical investigation of the 'compression zone' component within a steel joint at elevated temperatures". Journal of Constructional Steel Research 60, 841-865, 2004a.

Spyrou S., Davison J.B., Burgess I.W., Plank R.J. "Experimental and analytical investigation of the 'tension zone' components within a steel joint at elevated temperatures ". Journal of Constructional Steel Research 60, 867-896, 2004b.

Spyrou S., Davison J.B., Burgess I.W., Plank R.J. "Experimental and analytical studies of steel joint components at elevated temperatures". Fire Mater. 28, 83-94, 2004c.

→ ANSYS used to model T-stubs specimens at elevated temperatures. Good comparisons with experimental results.

Spyrou S., Davison J.B., Burgess I.W., Plank R.J. "Component based-studies on the behaviour of steel joints at elevated temperatures". In: Lamas A., Simões da Silva L., editors. Proceedings of the third European conference on steel structures-Eurosteel, Coimbra, Portugal, cmm press, p.1469-78, 2002.

→ Development of a new empirical expression to predict the ultimate compressive force of the column web, using the Drdacky's formula (Drdacky and Novotny, 1977).

5) ***Block et al.***, Development of a component-based finite element for steel beam-to-column connections at elevated temperature, 2006.

Block F. "Development of a component-based finite element for steel beam-to-column connections at elevated temperature". Ph.D. Thesis, Department of Civil and Structural Engineering, University of Sheffield, UK, 2006.

→ Tests on small British column sections loaded both axially and transversally to study the compression zone in the column web, 2004.

→ Numerical and analytical studies to understand the effect of superstructure loading on the component column web in compression; Development of high-temperature models for the behaviour of the main components of steel end-plate beam-to-column connections in fire, using ANSYS.

→ Development of a simplified analytical approach to predict the force-displacement response of the compression zone.

Block F.M., Burgess I.W., Davison J.B., Plank R.J. "A component approach to modelling steelwork connections in fire: Behaviour of column webs in compression". Proceedings of ASCE Structures Congress, Nashville, Tennessee, pp. 1163-1170, 2004a.

Block F.M., Burgess I.W., Davison J.B. "Numerical and analytical studies of joint component behaviour in fire". Proceedings of Third International Workshop "Structures in Fire", Ottawa, 2004b.

Block F.M., Burgess I.W., Davison J.B., Plank R.J. "High-Temperature experiments on joint component behaviour". ICASS '05, Shanghai, China, 2005.

→ Development of the component model started by Spyrou for end-plate connections. This spring model was condensed into a new finite element and incorporated in the program Vulcan. The model is validated by experimental results [Leston-Jones, 1997].

Block F., Burgess I., Davison B., Plank R.J. "The development of a component-based connection element for end-plate connections in fire". Proceedings of the fourth International Workshop "Structures in Fire", SiF'06, Aveiro, Portugal, 345-56, 2006a.

Block F.M., Burgess I.W., Davison J.B., Plank R.J. "A Component-Based Approach for Modelling the Effect of Fire on Steel Joints". 19th Australasian Conference on the Mechanics of Structures and Materials, Canterbury, New Zealand, 2006b.

Block F., Burgess I., Davison B., Plank R.J. "The development of a component-based connection element for end-plate connections in fire". Fire Safety Journal, 42(6-7), 498-506, 2007.

6) ***Lou and Li***, 2 cruciform tests on 16mm thick extended end-plates with M20 bolts to evaluate temperature distributions and structural response of extended end-plate joints, 2006.

→ Utilisation of the ANSYS code to model the behaviour of the tests: the results compared well with the experimental results.

Lou G-B and Li G-Q. "Non-linear finite element modelling of behaviour of extended end-plate bolted moment connections in fire". Proceedings of the fourth International Workshop "Structures in Fire", SiF'06, Aveiro, Portugal, 327–43, 2006.

7) ***Wang***, Experimental study and spring-component modelling of extended end-plate joints in fire, 2007.

→ 4 full-scale extended end-plate joints tested under fire (simple column of 3m + cantilever beam of 1.35m protected to the fire)

→ Spring-component modelling of extended end-plate joints in fire

Wang W.Y., Li G.Q., Dong Y.L. "Experimental study and spring-component modelling of extended end-plate joints in fire". Journal of Constructional Steel Research, 63, 1127-1137, 2007.

8) ***Qian et al.***, Behaviour of steel beam-to-column joints at elevated temperatures, 2007.

→ 6 tests of typical steel extended end-plate beam-to-column joints at elevated temperatures with different axial compression forces applied to the beams to simulate restraint effects, results as moment-rotation-temperature curves; Showed the importance of the consideration of the beam web in shear on the joint behaviour.

→ Finite element analyses using the commercial finite element analysis software package MSC. Marc Mentat (2001).

→ Analytical component-based method extended to include the shear component in the end panel of the beam web.

Qian Z.H., Tan K.H., Burgess I.W. "Behaviour and mechanical model of steel beam-to-column joints at elevated temperatures". Proceedings of Fifth International Conference on Advances in Steel Structures, Singapore, 783-790, 2007.

Qian Z.H., Tan K.H., Burgess I.W. "Behavior of Steel Beam-to-Column Joints at Elevated Temperature: Experimental Investigation". ASCE, Journal of Structural Engineering, Vol. 134, No. 5, May 1, 2008.

Tan K.H., Qian Z.H., Burgess I.W. "Experimental investigation of steel beam-to-column joints at elevated temperatures". Proceedings of the 3rd International Conference of Steel and Composite Structures (Eds.: Wang Y.W., Choi C.K.), 565-570, Manchester, U.K., 2007.

9) ***Hu and Yu et al.,*** Robustness of steel connections in fire, 2005-2008.

→ Experimental investigation of the behaviour of fin plate, flush end-plate, web cleat and flexible end-plate connections in fire when the beams are in catenary action:

University of Sheffield: 14 fire resistance tests with fin plate connections and 17 tests with flush end-plate connections (single joints);

University of Manchester: fire resistance tests on structural sub-frames.

Hu Y., Davison B., Burgess I., Plank R. "Experimental study on flexible end-plate connections in fire". Eurosteel 2008, 5th European Conference on Steel and Composite Structures, Graz, Austria, 3-5 September, 2008b.

Yu H., Burgess I., Davison B. "Experimental Investigation of the Robustness of Fin Plate Connections in Fire". 5th International Conference on Advances in Steel Structures, Singapore, ICASS, Volume III, p.722-727, 2007.

Yu H., Burgess I.W., Davison J.B., Plank R.J. "Experimental Investigation of the Behaviour of Flush End-plate Connections in Fire". SIF 2008, Singapore, 28-30 May 2008a.

Yu H., Burgess I.W., Davison J.B., Plank R.J. "Experimental investigation of the tying capacity of web cleat connections in fire". Eurosteel 2008, 5th European Conference on Steel and Composite Structures, Graz, Austria, 3-5 September, 2008b.

Yu H., Burgess I.W., Davison J.B., Plank R.J. "Experimental investigation of the behaviour of fin plate connections in fire". Proceedings of the 3rd International Conference of Steel and Composite Structures (Eds.: Wang Y.W., Choi C.K.), 541-547, Manchester, U.K., 2007. Journal of Constructional Steel Research, Article in press, 2008c.

→ Analysis of temperatures in the steel members and joint components using the FE package ABAQUS and the calculation method in EN 1993-1-2: 2005.

Dai X.H., Wang Y.C., Bailey C.G. "Temperature distributions in unprotected steel connections in fire". Proceedings of the 3rd International Conference of Steel and Composite Structures (Eds.: Wang Y.W., Choi C.K.), 535-540, Manchester, U.K., 2007.

→ A three dimensional numerical model was created for a flexible end-plate connection, using the ABAQUS finite element code, in order to investigate its resistance and ductility at ambient and elevated temperatures; Comparisons with experimental tests.

Hu Y., Burgess I., Davison B., Plank R. "Modelling of flexible end-plate connections in fire using cohesive elements". Fifth International Workshop "Structures in Fire", Singapore, May of 2008a.

Yu H., Burgess I.W., Davison J.B., Plank R.J. "Numerical simulation of bolted steel connections in fire using explicit dynamic analysis". Journal of Constructional Steel Research, Article in press, 2008d.

10) ***Santiago et al.***, Global methodology for the application of the component method to characterize the behaviour of steel joints under a natural fire, 2008.

Santiago A. "Behaviour of beam-to-column steel joints under natural fire". Ph.D. Thesis, Department of Civil Engineering, University of Coimbra, October, 2008.

→ Large-scale natural fire test in a real building (7th Cardington fire test).

Wald F., Chaldná M., Moore D., Santiago A., Lennon T. "Temperature distribution in a full-scale steel framed building subject to a natural fire". International Journal of Steel and Composite Structures, 6(2), 159-182, 2006a.

Wald F., Simões da Silva L., Moore D., Lennon T., Chaldná M., Santiago A., Beneš M., Borges L. "Experimental behaviour of steel structure under natural fire". Fire Safety Journal, 41(7), 509-22, 2006b.

→ 6 experimental tests on beam-to-column sub-frames submitted to a natural fire: Results used to validate 3D FE analysis models.

Santiago A., Simões da Silva L., Vila Real P. "Experimental investigation of the behaviour of a steel sub-frame under a natural Fire". Proceedings of the 3rd International Conference of Steel and Composite Structures (Eds.: Wang Y.W., Choi C.K.), 681-686, Manchester, U.K., 2007.

Santiago A., Simões da Silva L., Vaz G., Vila Real P., Gameiro Lopes A. "Experimental Investigation of the Behaviour of a Steel Sub-frame Under a Natural Fire". Steel and Composite Structures, 8(3), 243-264, 2008b.

→ Numerical parametric study, using the finite element software's SAFIR and LUSAS, to carry out systematic parametric studies to complement the experimental work and to provide a better understanding of the main influences on the overwall behaviour of a steel beam under fire.

Santiago A., Simões da Silva L., Vila Real P., Veljkovic M. "Numerical study of a steel sub-frame in fire". Computers and Structures, 86, 1619-1632, 2008a.

Santiago A., Simões da Silva L., Vila Real P. "Effect of joint typologies on the 3D behaviour of a steel frame under a natural fire". Proceedings of the Fifth International Conference on Structures in Fire (SiF'08), 205-216, 2008c.

Santiago A., Simões da Silva L., Vila Real P., Gameiro Lopes A., Vaz G. "Experimental evaluation of the influence of connection typology on the behaviour of steel structures under fire". Engineering journal, 2008e, Accepted for publication.

→ Development and validation of a design methodology for steel joints under fire.

Santiago A. "Caracterização de ligações metálicas sob a acção do fogo". Master Thesis, Department of Civil Engineering, University of Coimbra, 2000 (in Portuguese).

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II.3 Numerical Simulations

1) ***Liu***, First 3D finite element code (FEAST) to simulate the influence of connections on the response of steel structures in the event of a fire, FIRES-T3 performs the thermal analysis (Iding et al., 1977), 1996, 1998, 1999.

→ Comparisons with experimental tests results (Lawson, 1990; Leston-Jones et al., 1997; Al-Jabri et al., 1998).

Iding R., Bresler B. and Nizamuddin D. "FIRES-T3: A computer program for the fire response of structures – thermal". Fire Research Group Report No UCB FRG 77-15, University of California, Berkeley, 1977.

Liu T.C.H. "Finite element modelling of behaviour of steel beams and connections in fire". Journal of Constructional Steel Research, 36(2), 181-118, 1996.

Liu T.C.H. "Effect of connection flexibility on fire resistance of steel beams". Journal of Constructional Steel Research, 45(1), 99-118, 1998a.

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Liu T.C.H. "Fire resistance of unprotected steel beams with moment connections". Journal of Constructional Steel Research, 51, 61-77, 1999a.

Liu T.C.H. "Moment-rotation-temperature characteristics of steel/composite joints". Journal of Structural Engineering, 125(10), 1188-1197, 1999b.

2) ***El-Houssieny et al.***, Development of a 3D FE model to simulate extended end-plate joints at room and elevated temperatures, 1998.

→ Parametric study to investigate the influence of connection typology on the behaviour of sub-frames at high temperatures,

→ Validation against experimental results,

→ Simple equations for the moment-rotation stiffness, the bolt forces and stresses.

El-Houssieny O.M., Abdel Salam S., Attia G.A.M. and Saad A.M. "Behavior of extended end-plate connections at high temperature". Journal of Constructional Steel Research, 46(1-3), pp.299, 1998.

3) ***Rahman et al.***, Study of the response of fin plate joints in fire using ANSYS, 2004.

   Rahman R., Hawileh R., Mahamid M. "The effect of fire loading on a steel frame and connection". High-performance structures and materials II, WIT press, p. 307-16, 2004.

4) ***Sarraj et al.***, Development of a complete 3D model of fin plate connection using ABAQUS and including the rotational behaviour of the connection and the influence of the heated beam at high temperature, 2006, 2007.

   → Validation against lap joints tests at room temperature (Richard et al., 1980) and beam tests at high temperatures (Wald et al., 2006).

   Sarraj M., Burgess I., Davison B., Plank R.J. "Finite element modelling on fin plate steel connections in fire". Proceedings of Fourth International Workshop "Structures in Fire", SiF'06, Aveiro, Portugal, 315-26, 2006.

   Sarraj M., Burgess I.W., Davison J.B., Plank R.J. "Finite element modelling of steel fin plate connections in fire". Fire Safety Journal, 42, p.408-415, 2007b.

   → Development of a representation of fin plate connection via a simplified component model, enabling prediction of the connection response at both ambient and elevated temperatures.

   Sarraj M., Burgess I.W., Davison J.B., Plank R.J. "Development of a component model approach to fin plate connections in fire". Proceedings of the 3rd International Conference of Steel and Composite Structures (Eds.: Wang Y.W., Choi C.K.), 549-555, Manchester, U.K., 2007a.

5) ***Ramli Sulong et al.***, Development of a component model to simulate the joint behaviour under fire, implementation in the finite element program Adaptic, 2007.

   → Validation against experimental: Rotational behaviour of the joint validated at elevated temperatures (Al-Jabri, 1999; Spyrou, 2002); Combined bending and axial effect only validated at room temperature (Simões da Silva et al., 2004; Lima et al., 2004).

   Ramli Sulong N.H., Elghazouli A.Y., Issuddin B.A. "Analytical modelling of steel connections at elevated temperatures". Proceedings of Fourth European Conference on Steel Structures "Eurosteel", Maastricht, Netherlands, p.5.1.25-32, 2005.

   Ramli Sulong N.H., Elghazouli A.Y., Issuddin B.A. "Behaviour and design of beam-to-column connections under fire conditions". Proceedings of Fourth International Workshop "Structures in Fire", SiF'06, Aveiro, Portugal, 379-90, 2006.

   Ramli Sulong N.H. "Structural fire performance of steel connections". Ph.D. Thesis, Department of Civil and Environmental Engineering, Imperial College London, South Kensington Campus, London, UK, 2007.

   Ramli Sulong N.H., Elghazouli A.Y., Issuddin B.A. "Behaviour and design of beam-to-column connections under fire conditions". Fire Safety Journal, 42(6-7), p.437-451, 2007.

Ramli Sulong N.H., Elghazouli A.Y., Issuddin B.A. Ajit N. "Deformation capacity and ductility demand of steel connections at ambient and elevated temperature". Eurosteel 2008, 5th European Conference on Steel and Composite Structures, 627, Graz, Austria, 3-5 September, 2008.

6) ***Anderson and Gillie***, Heat transfer analyses of typical connections using the commercial software ABAQUS to obtain temperature profiles; Creation of 3D composite models with linear, 8 noded brick, heat transfer elements; 2008.

Anderson K., Gillie M. "Heat transfer modelling of connections". Proceedings of the Fifth International Conference on Structures in Fire (SiF'08), 229-237, 2008.

7) ***Selamet and Garlock***, Development and validation of a finite element model of a single plate shear connection capable of predicting the limit states of bolt bearing, bolt shearing, flange local buckling, and the peak loads, 2008.

→ Validation performed against simple plate connection tests as well as a full-scale building test done at Cardington.

Selamet S., Garlock M.E.M. "Behavior of steel plate connections subjected to various fire scenarios". Proceedings of the Fifth International Conference on Structures in Fire (SiF'08), 139-149, 2008.

II.4 Analytical Simulations

1) ***El-Rimawi et al.***, Characterization of the joint behaviour at high temperatures and development of a modified form of the Ramberg-Osgood equation to describe the moment-rotation characteristics, 1989.

→ Calibration of parameters using data from the tests performed by Lawson (Lawson, 1990).

El-Rimawi J.A. "The behaviour of the flexural members under fire conditions". Ph.D. Thesis, University of Sheffield, UK, 1989.

El-Rimawi J.A. "NARR: A program for the structural analysis of 2-D frames at elevated temperatures". Research Report, University of Sheffield, January, 1993.

El-Rimawi J.A., Burgess I.W., Plank R.J. "The influence of connections stiffness on the behaviour of steel beams in fire". Journal of Constructional Steel Research, 43(1-3), 1-15, 1997.

El-Rimawi J.A., Burgess I.W., Plank R.J. "Studies of the behaviour of steel sub frames with semi-rigid connections in fire". Journal of Constructional Steel Research, 49, 83-98, 1999.

2) ***Heidarpour et al***, Analytical studies of the behaviour of steel and steel-concrete composite beams and beam-to-column connections at elevated temperatures, 2007, 2008.

Heidarpour A. "Behaviour of steel and steel-concrete composite beams and beam-to-column connections at elevated temperatures". Ph.D. Thesis, School of Civil and

Environmental Engineering, the University of New South Wales, Sydney, Australia, November 2007.

→ Behaviour of compression region of beam-to-column connections:

Local buckling of flange outstands in thin-walled steel beams; Elastic local buckling of the web of an I-section beam; Analytical model for determining the ultimate strength at elevated temperatures of stocky hot-rolled and plate girder webs subjected to patch loading induced by concentrated loads.

Heidarpour A. and Bradford M.A. "Local buckling and slenderness limits for flange outstands at elevated temperatures". Journal of Constructional Steel Research, 63, 591-598, 2007a.

Heidarpour A. and Bradford M.A. "Local buckling and slenderness limits for steel webs under combined bending, compression and shear at elevated temperatures". UNICIV Report, School of Civil and Environmental Engineering, The University of New South Wales, Sydney, Australia, 2007; Thin-Walled Structures, 46, 128-146, 2008a.

Heidarpour A. and Bradford M.A. "Failure mechanisms for stocky webs under patch loading at elevated temperatures". Journal of Constructional Steel Research, 64, 1103–1111, 2008b.

Heidarpour A., Bradford M.A. "Behaviour of a T-stub assembly in steel beam-to-column connections at elevated temperatures". Engineering Structures, 30, 2893-2899, 2008c.

Heidarpour A. and Bradford M.A. "In-plane non-Linear elastic spline finite strip analysis of thin plates at elevated temperatures". Proceedings of the 5th European Conference on Steel and Composite Structures, EuroSteel 2008, Graz, Austria, 1049-1054, 2008d.

→ Behaviour of the tension zone and panel zone of a semi-rigid joint:

Theoretical investigation of the mechanical properties of a T-stub assembly at elevated temperature and behaviour of the panel zone in a steel frame at elevated temperatures that develop during a compartment fire, and for which significant axial compressive forces are generated, analytical structural model and parametric study.

Heidarpour A. and Bradford M.A. "Modelling of bolted T-stub assemblies at elevated temperatures". Proceeding of the Third International Conference on Steel and Composite Structures, Manchester, UK, 581-586, 2007b.

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ANNEX A – Simões da Silva et al., 2005

Behaviour of steel joints under fire loading

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Abstract. This paper presents a state-of-the-art on the behaviour of steel joints under fire loading and some recent developments in this field, currently being carried out by the authors. Firstly, a review of the experimental research work on steel joints is presented, subdivided into isolated member tests, sub-structure tests and tests on complete building structures. Special emphasis is placed on the seventh Cardington test, carried out by the authors within a collaborative research project led by the Czech Technical University in Prague. Secondly, a brief review of various temperature distributions within a joint is presented, followed by a discussion of the behaviour of isolated joints at elevated temperature, focussing on failure modes and analytical procedures for predicting the moment-rotation behaviour of joints at elevated temperature. Finally, a description of the coupled behaviour of joints as part of complete structures is presented, describing previous work and investigations on real fire (including heating and cooling phases) currently being carried out by the authors.

Key words: axial restraint; component method; composite joints; experimental tests; fire engineering; steel joints; temperature.

1. Introduction

The behaviour of steel and composite joints under fire loading is a subject that has only recently received special attention by the research community. In fact, as recently as 1995, the European prestandard on the fire response of steel structures (EN1993-1-2: 1995) deemed it unnecessary to assess the behaviour of steel joints under fire conditions. This approach was supported by the argument of the increased massivity of the joint area. However, observations from real fires show that, on several

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occasions, steel joints fail, particularly their tensile components (such as bolts or end-plates) because of the high cooling strains induced by the distortional deformation of the connected members (Buchanan 2002, Bailey *et al.* 1999, Wald *et al.* 2005).

To characterize the behaviour of steel joints under fire loading, several aspects must be considered:

a) The time-temperature distribution in and around the joint;
b) The effect of high temperatures on the structural response of the joints;
c) The redistribution of internal forces with time acting on the joint from the global behaviour of the structure.

A consistent approach for the assessment of the fire response of steel joints requires a broader view than merely restricting the attention to the joint area. Two directly related aspects must be considered, namely the fire development and the structural fire resistance strategy. Although essential, these aspects will not be dealt with in detail in this paper, generic references are available (Drysdale 1999, Vila Real 2003).

It is the purpose of this paper to present a state-of-the-art of the behaviour of steel joints under fire loading and to describe some recent developments currently being carried out by the authors.

2. Experimental research

Experimental research has been used to establish the behaviour of steel structures in fire and, in particular, steel joints. The experimental results on the response of steel connections under fire conditions are relatively recent and are limited, partly because of the high cost of conducting fire tests and partly due to the limitations on the size of furnace used. Only a few connection tests have been performed and these have concentrated on obtaining the moment-rotation relationships of isolated connections. In the following, a summary of available test results is presented.

2.1. Tests on isolated joints

The first reported tests on steel beam-to-column connections in fire were performed at CTICM (Kruppa 1976) and by the British Steel (British Steel, 1982). Both experimental projects tested a range of “flexible” to “rigid” connections under the ISO 834 fire curve (ISO 834, 1975) and linear increases in temperature. The main aim of these tests was to establish the performance of high strength bolts. The results suggested that the bolts and their connected elements could undergo considerable deformations in fire.

However, it was in the last fifteen years that research into the behaviour of joints under fire loading experienced significant developments. Lawson (1990) was the first to quantify the benefits of structural continuity under fire conditions on the basis of the results from a series of fire tests on typical beam-to-column connections. The tests were set up to fit within the test furnace at the Warrington Fire Research Centre. The cruciform test arrangement was used, comprising beams connected to a column, as shown in Fig. 1.

Eight tests were carried out under the standard temperature-time fire curve ISO 834. These tests included four flush end-plate joints (two using steel beams, one composite beams and one shelf-angle beams), two extended end-plate joints (both using steel beams) and two web-cleat joints (one using steel beams and one composite beams). All the beams supported concrete or a concrete slab. However, they were each restricted to a single load level (two load levels in three cases), providing insufficient

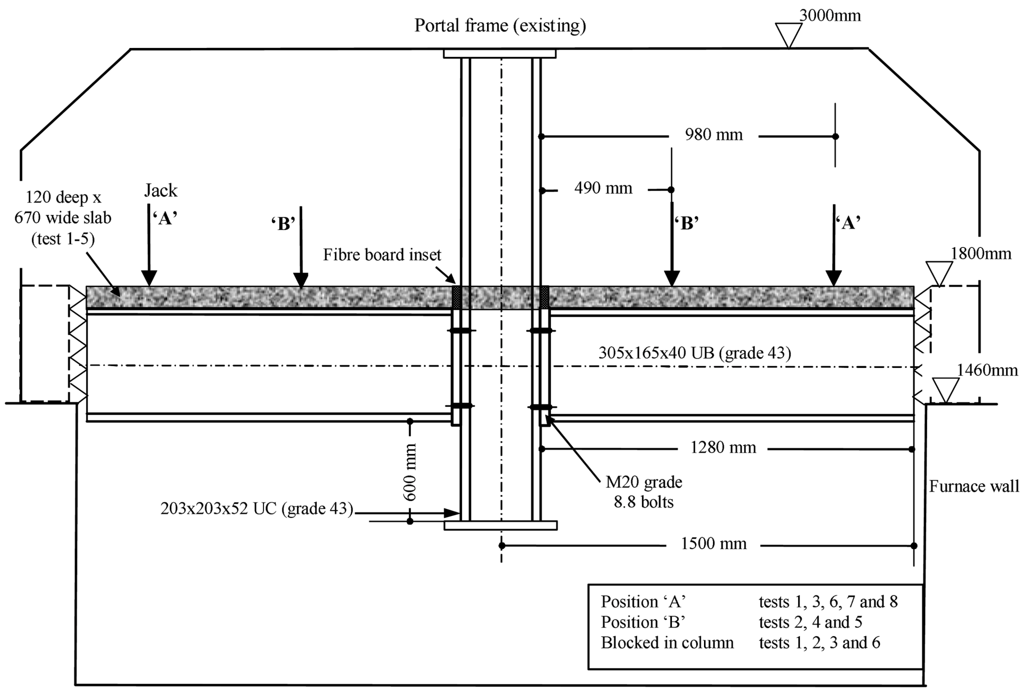


Fig. 1 Arrangement of the fire tests on beam-to-column connections (Lawson 1990)

data for the construction of the moment-rotation curves. This single load level varied between 0.10 M_p and 0.40 M_p, M_p - being the plastic moment capacity of the steel beam. From the experimental observations, it was found that the temperatures in the joint were much lower than that of the lower flange of the steel beam, which is usually the element that defines the limiting temperature of the beam. The maximum temperature on the lower beam flange was about 650 to 750°C and the maximum temperature in the upper exposed bolts was 150 to 200°C lower, and those inside the concrete slab were about 200 to 350°C lower than the lower beam flange. Failure was due to large deformation of the end-plate. In addition, it was suggested that composite action at elevated temperature contributed to the enhanced moment capacity of the connections. This was estimated by adding together the moment capacities of the bare-steel connection and the reinforced concrete slab.

The first characterization at high temperatures of the moment-rotation behaviour of commonly used joints was performed within a collaborative research programme involving the Building Research Establishment, the University of Sheffield and the Steel Construction Institute in the UK. This programme was divided into two phases; firstly, the behaviour of specimens with only one joint configuration was investigated (Leston-Jones *et al.* 1997); and secondly, the scope of the experimental programme was extended to include parameters such as member size, connection type and different failure mechanisms (Al-Jabri *et al.* 1998, Al-Jabri 1999).

In the first phase, a total of eleven tests were conducted, including two tests at room temperature, for both bare-steel and composite joints. The scope of the programme was restricted to cruciform connections with flush endplates (the most common connection type used in non-composite building frame). The high temperature tests were performed on a furnace that consisted of four 'barrel' modules suitable for testing beams or columns, and a junction furnace designed for testing connections within a

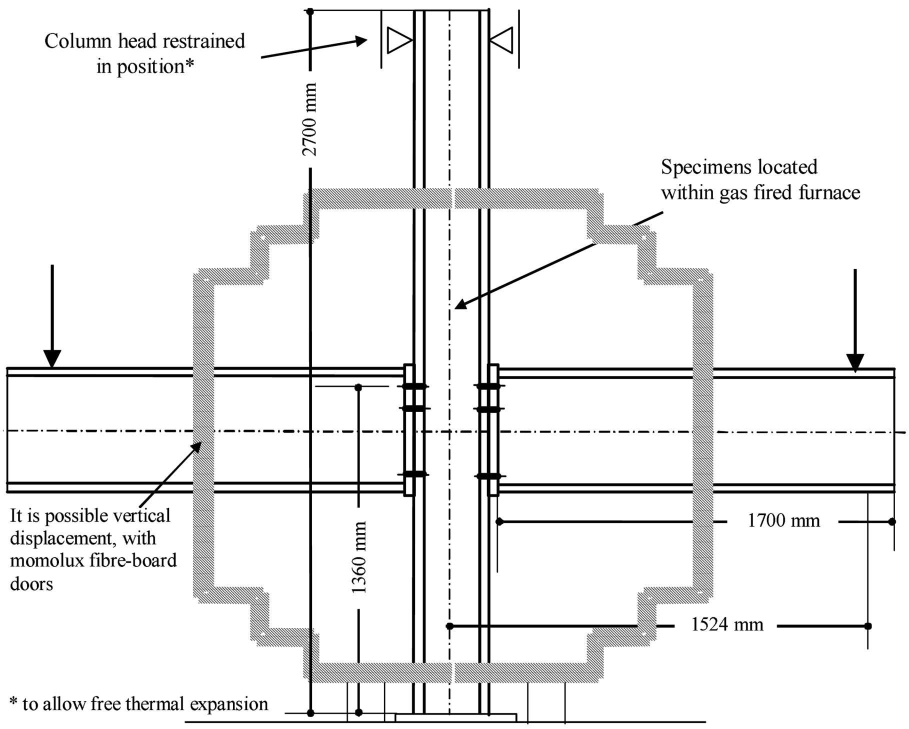


Fig. 2 High temperature test: schematic arrangement (Leston-Jones *et al.* 1997)

two- or three-dimensional framework. The high temperature test arrangement is illustrated in Fig. 2. In order to create a temperature distribution across the connection representative of real conditions, the concrete slab was simulated by a 50 mm thick ceramic fibre blanket wrapped around the top flange of the beam. All tests were performed by applying a fixed bending moment with a subsequent increase of the furnace temperature at a rate of about 10ºC/min while maintaining the applied load.

The tests results at room temperature have shown significant deformation of the column web in the compression zone and of the column flange in the tension zone. Very little damage to the beams, plates or bolts was observed. The joint was capable of resisting moments in excess of 30 kNm, with a proportional limit of about 15 kNm. Observation from the high-temperature tests showed an almost linear temperature profile through the depth of the connection. The uniform heating combined with the relatively slow rate of heating provided a somewhat unrepresentative fire condition when compared to a typical building fire. The failure modes observed at high temperatures were similar to those at room temperature. It was also observed that both the moment capacity and stiffness of the connection degraded with temperature. The critical temperature range of the connection, at which there is a market downturn in capacity, was between 500-600ºC.

In the second phase (Al-Jabri *et al.* 1998, Al-Jabri 1999), the general arrangement for the high temperature tests was the same as for the Leston-Jones tests (Fig. 2). In all cases, the test specimens consisted of a symmetric cruciform arrangement of a single column 2.7 m high with two cantilever beams 1.9 m long, connected on either side of the column flange. The programme of work is summarized in Table 1. The small member sizes associated with Group 1 tests were chosen for comparison with the Leston-Jones tests, the only difference being the adoption of a more realistically sized end-plate thickness

Table 1 High temperature connections tests.

Group	Beam	Column	End-plate	Material	Load Level
1	254×102×22UB	152×152×23UC	flush end-plate, $t_P = 8$ mm	bare steel	4 tests: 0.2 M_{c1}; 0.4 M_{c1}; 0.6 M_{c1}; 0.8 M_{c1}.
2	356×171×51UB	254×254×89UC	flush end-plate, $t_P = 10$ mm	bare steel	4 tests: 0.2 M_{c2}; 0.4 M_{c2}; 0.6 M_{c2}; 0.8 M_{c2}.
3	356×171×51UB	254×254×89UC	header-plate, $t_P = 8$ mm	bare steel	3 tests: 0.2 M_{c3}; 0.4 M_{c3}; 0.6 M_{c3}.
4	356×171×51UB	254×254×89UC	header-plate, $t_P = 8$ mm	composite	5 tests: 0.2 M_{c4}; 0.4 M_{c4}; 0.6 M_{c4}; 0.6 M_{c4}; full.
5	356×171×51UB	254×254×89UC	header-plate, $t_P = 10$ mm	composite	4 tests: 0.2 M_{c5}; 0.4 M_{c5}; 0.6 M_{c5}; 0.8 M_{c5}.

$M_{c1} = 20$ kNm; $M_{c2} = 140$ kNm; $M_{c3} = 60$ kNm; $M_{c4} = 105$ kNm; $M_{c5} = 227$ kNm

of 8 mm. The larger section sizes and variation in connection types associated with the bare steel tests in Groups 2 and 3 reflect the adoption of connections used in the eight storey building at Cardington. Groups 4 and 5 extend this philosophy to include the behaviour of the composite slab.

Observations showed little variation in temperature across the bare steel connections. However, in the composite connections, the concrete slab acted as a radiation shield and a heat sink, keeping the upper flange cooler and thus enhancing the failure temperature of the lower beam flange. The observed failure modes depended on the connection type: in group 1, localized deformation at the top of the end-plate was observed, particularly around the top bolt, accompanied by deformation of the column flange and buckling of the column web. In group 2 localized deformations occurred at the top of the end-plate, slipping of the top bolts in the tension zone (maybe due softening of the bolts at elevated temperatures); at high moment levels, cracking of the end-plate along the welds was observed in both the beam web and flange. In group 3 significant end-plate deformation was developed. In groups 4 and 5 failure in the concrete slab (due to separation of the shear stubs from the concrete slab) was observed. After failure of the concrete slab, the applied load was transferred to the connection and end-plate failure was observed. In general, all high temperature tests produced failure modes similar to those of the same connections at room temperature.

Recently, in order to assess the individual behaviour of each component of an end-plate connections at high temperatures, Spyrou *et al*. (2004a, 2004b) carried out an experimental programme on T-Stub and column web components using an electric furnace, where the temperature ranged from 20 to 800ºC. Twenty five specimens were tested to investigate the three failure modes of the T-Stub and twenty nine tests were conducted on the column web in compression. The tests at high temperature highlighted the bolt flexibility as a key parameter in the behaviour of the T-Stub. On the compression zone tests, some discrepancies were highlighted between the capacities calculated using the current design standards and the experimental results at room temperature.

2.2. Tests on sub-structures

As stated before, isolated member tests do not truly reflect the behaviour of a member under either normal or fire conditions. Many aspects only occur when steel members are connected together. Global and local failure and the force redistribution capability of highly redundant structural systems are some of the features occurring when the members interact with each other which cannot be represented by isolated element testing (Armer *et al.* 1994). In addition, other effects can be observed, as the behaviour associated with restraint to thermal expansion caused by the adjacent cooler structure, increases the axial force in the heated members. This can cause column instability (Bailey *et al.* 1996) and local buckling in the heated beams, neither of which would occur in isolated members.

In Germany, Rubert and Schaumann (1986) used electrical heating to test a series of three different arrangements of quarter-to-half scale sub-assemblies rigidly connected. No information was provided on the forces in the test frames to enable quantification of the effect between different frame members. Nevertheless, these high temperature tests have been used by various researchers to validate numerical models.

At the same time, the Fire Research Station, UK, carried out perhaps the first fire test on a full-scale structural assembly subjected to a natural fire using wooden cribs (Cooke and Latham 1987). The test structure was a goal post assembly consisting of a steel beam (406×178×54 UB) and two steel columns (203×203×52 UC). The column bases were pinned and the beam was connected to the columns using flush end-plate connections. A concrete slab was placed on top of the steel beam to give realistic heating conditions. Bracing was provided to the test frame near the beam-to-column connections to prevent sway and out-of-plane deflections. This test showed that the performance of the frame was better than that of the individual elements as a result of the continuity between the beam and columns.

At the end of 1990's, the University of Manchester developed an experimental project in order to investigate the effects of restraint to thermal expansion of unprotected beams, from protected columns and adjacent cooler beams (Liu *et al.* 2002). These tests focused on the effect that different connection types had on the failure temperature of the connected members at different load levels. The experimental work was based on two-dimensional studies and was complemented with the results from the Cardington full-scale frame fire tests. In total, 25 fire tests on 2D steel frames were conducted. Two types of connections were considered: flush end-plate and web cleat connections, coupled with three levels of loading (20%, 50% and 70% of the moment capacity of the beam) and three degrees of horizontal restraint (8 kN/m; 35 kN/m and 62 kN/m). The beams had varying amounts of insulation, including the case in which the lower flange was fully exposed and the upper flange was encased in a concrete slab. The columns were generally fire-protected and a section of the beam and column in the region of the connection was exposed to fire. The top flange of the beam was protected by a non-composite concrete slab. The major axis connection configuration consisted of a 178×102×19UB beam connected to a 152x152x30UC column, both in grade 43 (S 275) steel.

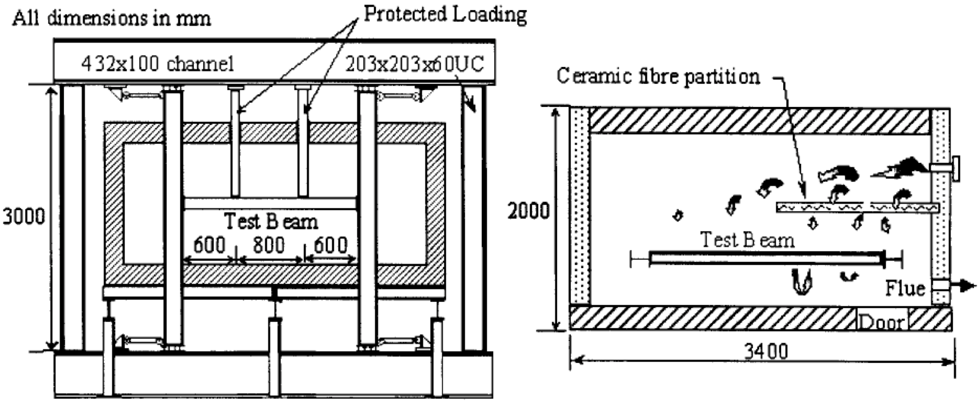


Fig. 3 (a) High temperature test, (b) Section through furnace (Liu *et al.* 2002)

The basic layout of the furnace and the specimens are illustrated in Fig. 3(a). The furnace box was constructed of light rectangular hollow steel sections supporting thin steel plates with ceramic fibre lining (with rapid heating times and light-weight construction). To expose the steel frame to uniform heating but to prevent direct exposure to the flame itself, the furnace box was partially divided by a ceramic fibre partition, as shown in Fig. 3(b). The burning system was programmed to follow the ISO 834 fire curve.

Observations showed a small variation in temperature in the web and bottom flange. Due to the fire protection around the top flange, its temperature initially rose at a slower rate, the temperature difference between the top flange and the web being about 300°C after 10 min. This difference was subsequently reduced to 100°C when the web temperatures reached 800°C. It was recognized that web-cleat connections had very little influence on the behaviour of the beam until the beam came into contact with the column. Flush end-plate connections were able to transfer a much higher bending moment to the column than the web-cleat connections. In some tests (those with higher axial restraint and lower load levels), catenary action at large deflections was clearly visible (Liu *et al.* 2001a).

2.3. Tests on complete building structures and observation of real fire events

Fire tests on sub-structures are necessary to understand the interaction between different structural members and to appreciate the difference between the behaviour of isolated members and members within a structure. However, in addition to the structural members a complete structure also includes floor slabs, walls and others non-structural members. A proper understanding of the behaviour of a complete building in fire can only be obtained if all such components are included. Although extremely expensive, fire tests on complete buildings are essential, complemented with the investigation of accidental fire events in buildings. Crucial to the understanding of the structural behaviour of a complete building in a real fire are the Broadgate fire accident (SCI, 1991) and the Cardington full-scale fire tests (Armer *et al.* 1994, Bailey *et al.* 1999, Wang 2002). Others tests and real fires included the William street fire tests by BHP (Thomas *et al.* 1992) and the Collin street fire test by BHP (Proe and Bennetts 1994) in Australia, the Basingstoke fire accident and the Churchill Plaza fire accident in the United Kingdom, the fire tests performed in Germany (Anon 1986) and the fire event at the World Trade Center, NY (Usmani *et al.* 2003, Quintiere *et al.* 2002). In the following paragraph, the Cardington full-scale fire tests will be described in more detail.

The Cardington Laboratory is a unique worldwide facility for the advancement of understanding of whole-building performance. This facility is located at Cardington, Bedfordshire, UK and consists of a former airship hangar with dimensions 48 m×65 m×250 m. It is used by industrial organizations, universities and research institutes, government departments and their agencies. The BRE's Cardington Laboratory comprises three experimental buildings: a six storey timber structure, a seven storey concrete structure and an eight storey steel structure.

The eight storey steel structure was built in 1993. It is a steel framed construction using composite concrete slabs supported by steel decking in composite action with the steel beams (Fig. 4). It has eight storeys (33 m) and is five bays wide (5×9 m = 45 m) by three bays deep (6 + 9 + 6 = 21 m) in plan, see Fig. 5. The structure was built as a no-sway frame with a central lift shaft and two end staircases providing the necessary resistance against lateral wind loads. The main steel frame was designed for gravity loads, the joints consisting of flexible end plates for beam-to-column connections and fin plates for beam-to-beam connections, designed to transmit vertical shear loads. The building simulates a real

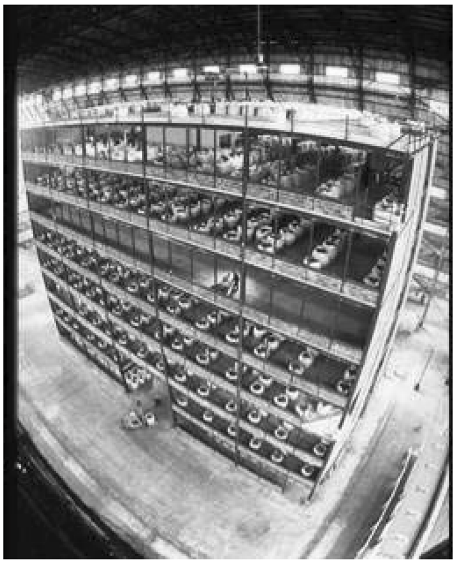


Fig. 4 The eight storey steel structure in Cardington

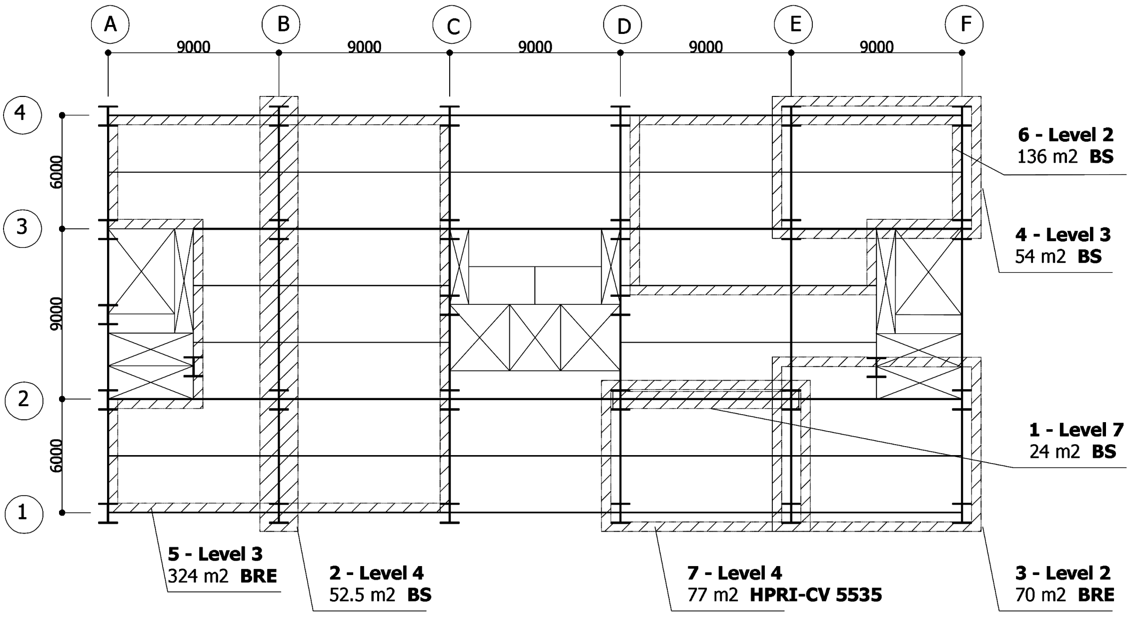


Fig 5. The Cardington fire tests on steel structure

commercial office in the Bedford area and all the elements were designed according to British Standards and checked for compliance with the provisions of the Structural Eurocodes.

The building was designed for a dead load of 3.65 kN/m^2 and an imposed load of 3.5 kN/m^2 (Bravery 1993). The floor construction consisted of a steel deck and a light-weight in-situ concrete composite floor, incorporating an A142 (142 mm^2/m) anti-crack mesh in both directions. The floor slab had an overall depth of 130 mm and the steel decking had a trough depth of 60 mm.

Table 2 Fire tests on the steel structure in the Cardington laboratory (Wald *et al.* 2005).

No.	Test	Fire compartment		Load	
		size, m	area, m^2	Fire	Mechanical
1	One beam heated by gas	8×3	24	Gas	30%
2	One frame heated by gas	21×2.5	53	Gas	30%
3	Corner compartment	9×6	54	40 kg/m^2	30%
4	Corner compartment	10×7	70	45 kg/m^2	30%
5	Large compartment	21×18	342	40 kg/m^2	30%
6	Office – Demonstration	18×9	136	45 kg/m^2	30%
7	Structural integrity	11×7	77	40 kg/m^2	56%

Seven large-scale structural fire tests at various positions within the experimental building were conducted; see Fig. 5 and Table 2 (Moore 1995, O'Conner and Martin 1998, Bailey *et al.* 1999, Wald *et al.* 2005). The main objective of the compartment fire tests was to assess the behaviour of structural elements with real restraint under a natural fire.

The first test performed at Cardington was a restrained beam test involving a single 305x165xUB40 composite beam section supporting part of the seventh floor of the building (Lennon 1997). A gas-fired furnace was used to heat the beam to approximately 900°C. The second test, a plane frame test, involved heating a series of beams and columns across the full width of the building. Again, a gas-fired furnace was used to heat the steelwork to approximately 800°C. The BRE corner compartment test was the first natural fire carried out at the Cardington Laboratory, representing a typical office fire (timber cribs were used to provide a fire load of 40 kg/m^2). The internal compartment walls were constructed using fire resistant board, one external wall was solid brick and the other external was formed by double glazing windows. All columns were protected up to and including the connections. It was observed that the fire development was largely influenced by the lack of oxygen in the compartment (Moore and Lennon 1997). The fourth test, the BS corner compartment test also used timber cribs to provide a fire load of 45 kg/m^2. In this test, both the perimeter beams and the columns were fire protected with the internal beam unprotected. Load bearing concrete blocks were used for the compartment walls. The fifth test was the largest compartment fire test in the world. The compartment was designed to represent a modern open-plan office (18 m × 21 m). The compartment was bounded by fire resistant walls. The main aim of this test was to investigate the ability of a large area of composite slab to support the applied load once the main beams had failed. Consequently, none of the beams had any fire protection and all columns were fire protected. Again, the ventilation conditions governed the fire severity. In the demonstration test (sixth test), unlike in the previous tests, real furniture (desks, chairs, filling cabinets, computer terminals, etc.) was used for the fire load. The ventilation was provided by windows and blank openings. The beams were unprotected while the columns were protected. This test was characterized by a rapid rise in temperature representing a severe fire scenario.

Finally, the seventh test, the structural integrity fire test, had the highest amount of mechanical loading and was carried out by the authors. It was developed in order to investigate the global structural behaviour of steel-concrete composite frame building subject to a natural fire test, focusing on the examination of the temperature development within the various structural elements, the corresponding (dynamic) distribution of internal forces and the behaviour of the composite slab, beams, columns and connections. The fire test was performed on the 4th floor, enclosing a plan area of 11 m by 7 m and it was bounded with three layers of plasterboard and an opening 1.27 m × 9 m long simulating an open

Fig. 6 7th Cardington fire test - compartment after fire, residual deformation 925 mm, no local collapse of structure

window. The columns, external joints and edge beam were fire protected to prevent global structural instability. The mechanical load was simulated using sandbags, each weighing 1100 kg and the fire load was provided by 40 kg/m^2 of wooden cribs. During the test, the predicted local collapse of the structure was not reached (Fig. 6). The maximum deflection was about 1 200 mm and the residual deflection was 925 mm. Fig. 7 compares the temperatures recorded in the compartment with the parametric curve calculated according to prEN1991-1-2 (2002). The maximum recorded compartment temperature near the wall (2 250 mm from D2) was 1107.8°C after 54 minutes, while the predicted temperature was 1078 °C after 53 min. The main damage mechanisms observed near the connections can be summarised as follows: local buckling of the beam lower flange, shear buckling of the beam web, buckling of the column flange in compression, fracture of the end-plate along the welds, elongation of the holes in the beam web, fracture in the concrete slab and slippage of the mesh. A detailed description of this fire test can be found in Wald *et al.* (2005).

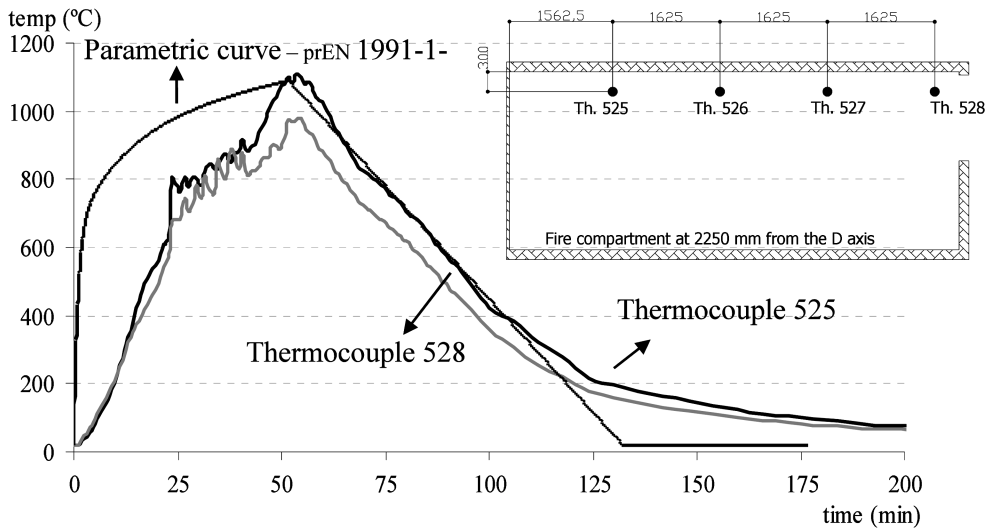


Fig. 7 7th Cardington fire test - compartment temperature

Table 3 Summary of the results from the major full-scale fire tests in the Cardington laboratory

No.	Org.	Level	Time, min to max. temp.	Maximum temperature (°C)		Deformations (mm)	
				gas	steel	maximal	residual
1	*BS	7	170	913	875	232	113
2	BS	4	125	820	800	445	265
3	**BRE	3	114	1000	903	269	160
4	BS	2	75	1020	950	325	425
5	BRE	3	70	-	691	557	481
6	BS	2	40	1150	1060	610	-
7	***HPRI-CV 5535	4	55	1108	1088	> 1000	925

*BS – British Steel (now Corus); **BRE – Building Research Establishment; ***HPRI-CV 5535 – Collaborative European project.

The main results from the Cardington tests are summarized in Table 3.

3. Experimental behaviour of joints at high temperature

The experimental results presented above provided information on the temperature distribution within and around the joint area as well as the characterization of the moment-rotation relationship at high temperatures, including some insight into the various failure modes. In the following sections, both aspects are summarized.

3.1. Temperature distribution within the joint

The thermal conductivity of steel is high. However, because of the mass concentration within the joint area, a differential temperature distribution should be considered within the joint. Various temperature distributions have been proposed or used in experimental tests by several authors. According to EN 1993-1-2: 2005, the temperature of a joint may be assessed using the local section factor (*A/V*) of the joint components or calculated using the maximum value of the ratios *A/V* of the adjacent steel members. For beam-to-column and beam-to-beam joints, where the beams support any type of concrete floor, the temperature may be calculated based on the temperature of the bottom flange at mid span. However, some recent studies do not support the EN 1993-1-2 recommendations. For example, Franssen (2002) has shown that the temperature in the components is higher than the local massivity would have indicated, probably because the dimensions of the components are of an order of magnitude smaller than the dimensions of the connected members and the influence of these members is felt on the components.

Thermocouples were used in some of the previous tests in order to quantify the temperature distribution within a joint. Table 4 summarizes the results, a detailed description being found in the literature. It shows that for deeper beams a web temperature similar to bottom flange temperature is observed while for small beams a smaller web temperature is observed. Additionally, the presence of the concrete slab above a joint causes a reduction in the temperature of the beam top flange.

Table 4 Temperature distribution with time within joint

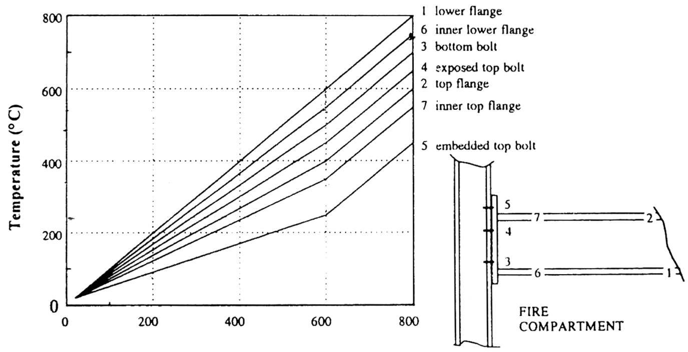
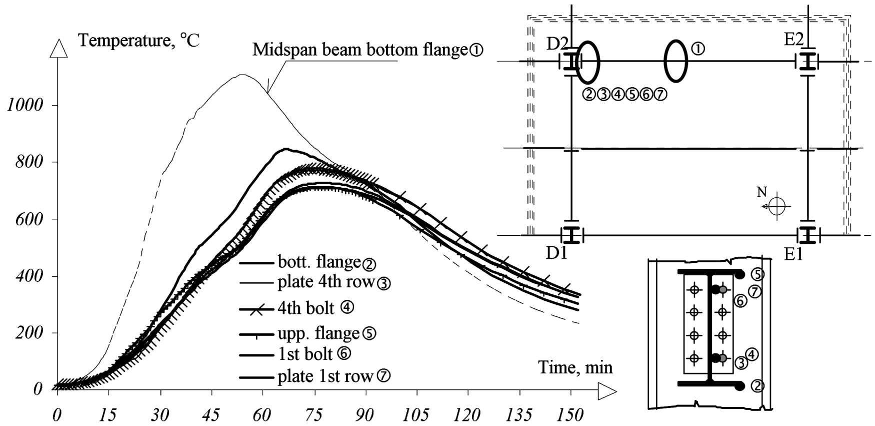
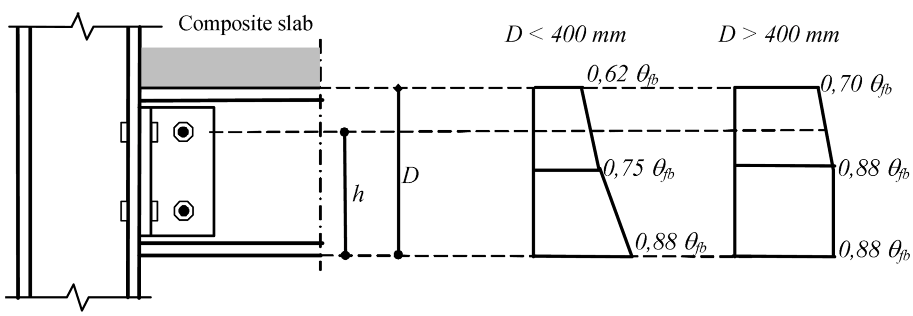
Authors	Temperature distribution	Main contribution
Kruppa (1976)	<u>Half of IPE500 on the top flange</u>: $\theta_{\text{joint (web)}} \approx 150$ - 200 ºC lower than $\theta_{\text{(web beam)}}$ $\theta_{\text{joint (flanges)}} \approx$ 150 - 400 ºC lower than $\theta_{\text{(beam flanges)}}$ <u>Fin-plate and flush end-plate joints</u>: $\theta_{\text{joint (web)}} \approx \theta_{\text{(web beam)}}$ $\theta_{\text{joint (flanges)}} \approx$ 50 ºC lower than $\theta_{\text{(beam flanges)}}$ <u>Extended end-plate joints</u>: $\theta_{\text{joint (flanges)}} \approx$ 50 ºC lower than $\theta_{\text{(beam flanges)}}$ <u>Web-cleats on the flanges and web</u>: $\theta_{\text{joint (flanges)}} \approx$ 250 ºC lower than $\theta_{\text{(beam flanges)}}$ <u>Web-cleats on the flanges and web (column - HEB340)</u>: $\theta_{\text{joint (web)}} \approx$ 100 ºC lower than $\theta_{\text{(web beam)}}$ $\theta_{\text{joint (flanges)}} \approx$ 150 ºC to 200ºC lower than $\theta_{\text{(beam flanges)}}$	First measurements of the temperature distribution around the joint area. *Scope of calibration*: Isolated fire test - steel beam-to-column connections (beam: IPE360; column: HEB340).
Lawson (1990)	θ_{fb} = 650 - 750 ºC (failure temperature) $\theta_{\text{exposed upper bolts}}$ = 150 - 200 ºC lower than θ_{fb} $\theta_{\text{bolts inside the concrete slab}}$ = 200 - 350 ºC lower than θ_{fb} $\theta_{\text{lower bolts}}$ = 100 - 150 ºC higher than $\theta_{\text{upper bolts}}$	Measurements of the temperature distribution in the joint components. Influence of the concrete slab on the temperature distribution. *Scope of calibration*: Isolated fire test - steel and composite beam-to-column connections.
SCI recom. (1990)	Joint with extended end-plate, considering embedded top bolts:  	
Leston-Jones *et al.* (1997)	$\theta_{\text{upper beam flange}}$ 0.677 × θ_{fb}; $\theta_{\text{beam centre web}}$ 0.985 × θ_{fb}; $\theta_{\text{top bolt}}$ 0.928 × θ_{fb}; $\theta_{\text{middle bolt}}$ 0.987 × θ_{fb}; $\theta_{\text{bottom bolt}}$ 0.966 × θ_{fb}; $\theta_{\text{column flange}}$ 1.036 × θ_{fb}; $\theta_{\text{end plate}}$ 0.982 × θ_{fb}	Description of the temperature distribution in each joint component. *Scope of calibration*: Isolated fire test - steel double-sided joint with flush end-plate.
Al-Jabri *et al.* (1998)	Hottest connection elements: column web. $\theta_{\text{column web}}$ (1.08 to 1.26) × θ_{fb}; $\theta_{\text{beam top flange – compos joint}}$(0.7-0.8) × $\theta_{\textit{beam top flange-steel joint}}$	Influence of the concrete slab on the joint temperature distribution. *Scope of calibration*: Isolated fire test - steel and composite beam-to-column connections.

Table 4 Continued

Authors	Temperature distribution	Main contribution
Liu *et al.* (2002)	$\theta_{\text{connection}}$ $0.8 \times \theta_{\text{beam mid-span}}$	*Scope of calibration*: Fire tests in restrained steel beams
Wald *et al.* (2004)	 Heating phase, the joint temperature is significantly lower than the remote bottom flange; in contrast, the cooling down in the joints was slower. For the max. ambient temperature: $\theta_{\text{joint}} \approx 200$°C lower than θ_{fb} The first bolt row from the top was significantly cooler than the lower bolts. The end-plate was hotter than the bolts.	Description of the temperature distribution in each joint component during a natural fire. *Scope of calibration*: Full-scale fire test – header plate beam-to-column and fin plate beam-to-beam
EN 1993-1-2 (2005)	The temperature of a joint may be assessed using: · the local massivity value (*A/V*) of the joint components, or · the maximum value of the ratios *A/V* of the adjacent steel members. · For beam-to-column and beam-to-beam joints, where the beams are supporting any type of con crete floor: 	

θ_{fb} – Lower beam flange temperature

3.2. Failure modes

In general, the high temperature tests on isolated steel joints produced failure modes which are similar to those observed for the same joint at room temperature. However, during a natural fire on a

Fig. 8 (a) Beam web in shear and local buckling of beam lower flange; (b) End-plate fracture

framed building, the joint behaviour is influenced by the restrained forces due to the interaction between the fire-affected members and the adjacent structure. In the following section the main failure modes observed during the experiments above are presented:

Steel joint tests:

(i) Header end-plate joint: 1 - Significant end-plate deformation (Al-Jabri *et al.* 1998).

(ii) Flush end-plate joint: 1- Localized deformation at the top of the end-plate, accompanied by deformation of the column flange and buckling of the column web, in cases of small cross-sections and, 2- Slipping of the top bolts in the tension zone. 3- At high temperatures and high load levels, fracture of the end-plate along the welds (Leston-Jones *et al.* 1997, Al-Jabri *et al.* 1998, Lawson 1990).

Steel joints supporting a concrete slab:

(iii) Flexible end-plate joint: 1- Local buckling in the lower beam flange and web adjacent to the joint, the concrete slab restraining the upper flange. This local buckling occurs during the heating phase, due to the restraint to thermal elongation provided by the adjacent cooler structure and the continuity of the structure. As the temperature and the associated deformations increase, the shear resistance of the beam's web is reached (Fig. 8a). This failure mode is only observed in global tests, where the thermal expansion is restrained by the adjacent cooler members. 2- Buckling of the column flange in compression in major axis joints. 3- Fracture of the end-plate along the welds caused by the large rotations

Fig. 9 Elongation of holes in the beam web in fin plate connection

observed during the heating phase or due to the horizontal tensile forces developed during cooling in restrained beams (Fig. 8b). Fracture occurred along one side of the connection only, while the other side remained intact. After one side has fractured, the increased flexibility allowed larger deformations without further fracture. 4- Large crack propagating from the face of the column flange parallel to the beam. After this, the joint stiffness decreased and secondary cracks occurred perpendicular to, and continuous across, the connections on both sides of the slab (Al-Jabri *et al.* 1998, Wald *et al.* 2005).

(iv) Fin plate beam-to-beam joint: 1- Elongation of the holes in the thinest component (beam web or fin plate) due to the associated large rotations (Fig. 9). The elongation of the holes leads to increased joint flexibility, allowing larger deformations (Wald *et al.* 2005).

4. Theoretical studies and design methodologies

4.1. Behaviour of isolated joints

From the experimental results it was possible to develop analytical or numerical procedures for the derivation of the moment–rotation relationship of isolated joints at high temperatures using joint properties at ambient temperature. These procedures are based on either (i) fitting of the Ramberg-Osgood type expressions to the global moment-rotation response of the joint, or (ii) application of the philosophy of the component method to deal with high temperatures. Both approaches are described in the following section.

4.1.1. Curve fitting methods

El-Rimawi was the first researcher who tried to characterize the joint behaviour at high temperatures. He developed a modified form of the Ramberg-Osgood equation (Ramberg and Osgood 1943) to describe the moment-rotation characteristics (El-Rimawi 1989). This relationship is defined by a single non-linear equation that always yields a positive slope, corresponding to the rotational stiffness of the joint, Eq. (1):

$$\phi = \frac{M}{A} + 0.01\left(\frac{M}{B}\right)^n \tag{1}$$

ϕ is the joint rotation at any given temperature, M is the applied moment to the joint and A, B and n are temperature dependent factors. Parameters A and B are directly related to the joint stiffness and capacity, respectively, while n defines the non-linear shape of the curve that is linked to the connection. El-Rimawi calibrated these parameters using data from the tests performed by Lawson (1990) (El-Rimawi *et al.* 1997, 1999). This analytical procedure is relatively easy to apply and gives a good representation of the observed behaviour. However it is limited to the tested connection and before extrapolating to others connections types it is necessary to perform a wide range of tests involving different failure mechanisms. To apply this equation to joints with different section sizes, a factor λ was introduced assuming that the moment capacity of a joint is proportional to the distance between the internal tensile and compressive forces (D):

$$\lambda = \frac{D-50}{303.8-50} \tag{2}$$

and the Ramberg-Osgood equation takes a more general form

$$\phi = \frac{M}{\lambda^2 A} + 0.01\left(\frac{M}{\lambda \cdot B}\right)^n \tag{3}$$

Recently, Al-Jabri developed a simple procedure to enhance the Ramberg-Osgood equation to generate the moment-rotation curves of steel and composite end-plate joints at high temperatures (Al-Jabri *et al.* 2004). First, the Ramberg-Osgood expression was fitted to the room temperature connection characteristics, then the temperature parameters A and B were reduced with the increasing temperatures based on the degradation rate of the strength of structural steel (EN1993-1-2:2005) and the levels of strain fitted with previous experimental tests (Al-Jabri 1999). This procedure predicts with reasonable accuracy the moment-rotation curves at high temperatures for the calibrated joints.

4.1.2. Component method

At room temperature, the component method (Weynand *et al.* 1995, Simões da Silva *et al.* 2002), which consists of modeling a joint as an assembly of extensional springs and rigid links, where the springs (components) represent specific parts of a joint that, dependent on the type of loading, make an identified contribution to one or more of its structural properties, has now become the *de facto* standard for the analysis of steel and composite joints at room temperature (EN1993-1-8, 2004). However, high temperature component models are rare, due to the lack of experimental data that describes the connection behaviour. The first author to use a component model at high temperatures was Leston-Jones (1997), to predict the response of steel and composite flush end-plate joints. Comparison with experimental results (Leston-Jones 1997, Leston-Jones *et al.* 1997) showed that there was good agreement for steel flush end-plate connections. However, the composite model showed a significant difference in the rate of degradation compared with experimental results.

The authors proposed a global methodology for the application of the component method to analyze the behaviour of steel joints at high temperatures (Simões da Silva *et al.* 2001). The component force-displacement response at room temperature is calculated according to part 1.8 of EC3 and the effect of high temperature is incorporated by the continuous change of yield stress and Young's modulus on each component. It is possible to calculate the joint moment-rotation response at high temperatures (Fig. 10), as well as the non-linear joint behaviour as the temperature increases. Eqs. (4) to (6) illustrate the change in the force-deformation response of component i with increasing temperature, for a given temperature variation θ:

$$F^y_{i,\theta} = k_{y,\theta} \times F^y_{i,20°C} \tag{4}$$

$$K^e_{i,\theta} = k_{E,\theta} \times K^e_{i,20°C} \tag{5}$$

$$K^{pl}_{i,\theta} = k_{E,\theta} \times K^{pl}_{i,20°C} \tag{6}$$

$k_{y,\theta}$ and $k_{E,\theta}$ are the reduction factors for effective yield stress and Young's modulus at temperature θ; $F^y_{i,20°C}$ is the limit load of component i at room temperature and $K^e_{i,20°C}$ is the elastic and post-limit stiffness of component i at room temperature. The joint moment-rotation response at temperature θ, when the component i yields, is expressed by:

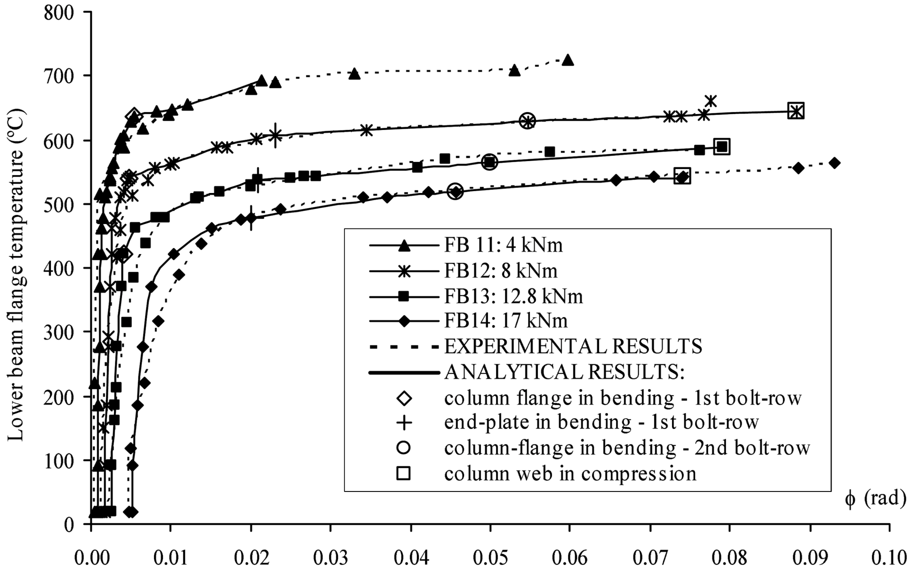


Fig. 10 Anisothermal temperature–rotation response (Simões da Silva *et al.* 2001)

$$M^y_{i;\theta} = k_{y;\theta} \times M^y_{i,20°C} \tag{7}$$

$$\phi^y_{i;\theta} = \frac{M^y_{i;\theta}}{S^y_{i;\theta}} = \frac{k_{y,\theta} \times M^y_{i,20°C}}{k_{E,\theta} \times S_{i,20°C}} = \frac{k_{y;\theta}}{k_{E;\theta}} \times \phi^y_{i,20°C} \tag{8}$$

where $M^y_{i,20°C}$ denotes the moment corresponding to the yield of component i at room temperature, $\phi^y_{i,20°C}$ is the joint rotation corresponding to the yield of component i at room temperature and $S_{i,20°C}$ is the rotational stiffness at room temperature. Fig. 10 compares the analytical procedure with the experimental results obtained by Al-Jabri *et al.* (1998) for a cruciform bolted end-plate beam-to-column steel joint under uniform temperature distribution. Excellent agreement is observed between the analytical procedure and the test results (Simões da Silva *et al.* 2001).

Recently, Al Jabri developed another simplified component-based model to predict the behaviour of steel and composite flexible header plate joints at high temperatures. In this model the elements of the connection are treated as springs with known stiffness and the overall connection response is obtained by assembling the stiffnesses of individual elements in the tension and compression zone. The high temperature effect is incorporated in each element at a given bolt-row (Fig. 11). This method is valid up to the point at which the bottom flange of the beam comes into contact with the column (Al-Jabri 1999, 2003, 2004). Therefore, the rotation at any given moment is expressed by

$$\phi = \frac{M}{S_t} \tag{9}$$

Where S_t is the rotational stiffness of the joint for a given temperature. This can be calculated using the methods given in EN1993-1-2 for steel joints. Anderson and Najafi (1994) have developed a model for composite connections. The methods for steel and composite joints are given below:

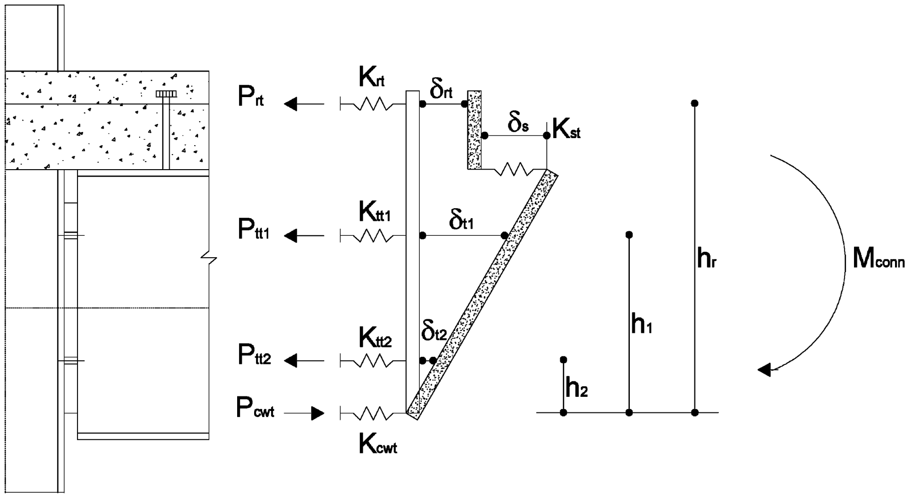


Fig. 11 Composite joint component model (Al-Jabri 2004)

$$
\begin{cases}
K_t^{-1} = S_{jt}^{-1} = (K_{eqt}z^2)^{-1} + (K_{cwt}z^2)^{-1} & \text{steel joint} \\
K_t^{-1} = S_{cct}^{-1} = \left[(K_{eqt} + K_{cwt})z^2 + \dfrac{K_{rt}K_{st}h_r h}{K_{rt} + K_{st}}\right]^{-1} & \text{composite joints}
\end{cases}
\tag{10}
$$

where K_{eqt} and K_{cwt}, are the stiffness of the tension zone and compression zone for a given temperature; K_{rt} and K_{st} are the stiffnesses of the reinforcement and the shear studs respectively for a given temperature; z is the level arm to the centerline of the equivalent spring in the tension zone; h is the distance from the beam-slab interface to the centre of rotation and h_r is the distance from the reinforcement to the centre of rotation.

Comparison of the results from the model with existing test data (Al-Jabri *et al.* 1998, Al-Jabri 1999) shows good agreement, especially in the elastic region.

4.1.3. Component characterization

The application of the component method requires the characterisation of the individual components. To this purpose, Spyrou *et al.* (2004a, 2004b) have developed analytical expressions for the force-displacement relationships for the column web in compression and the T-stub in tension at high temperatures.

Based on Drdacky's formula, a new empirical expression has been developed to predict the behaviour of the column web in compression (Spyrou *et al.* 2004a):

$$
P_u = t_{wc}^2 \sqrt{E_{wc}\sigma_{wc}} \sqrt{\frac{t_{fb}}{t_{wc}}} \left[0.65 + \frac{1.6c}{d_{wc}} \frac{2\beta}{2\beta + c}\right]
\tag{11}
$$

where t_{wc} and t_{fb} are the web and column flange thicknesses; d_{wc} is the total depth of the column web between fillets; E_{wc} is the Young's modulus of the column web; σ_{wc} is the column web yield stress; c is the width of the applied load and β is the development width of the bearing zone. The web column capacity at high temperature is obtained from Eq. (11) applying the reduction factors for yield stress and stiffness from EN1993-1-2.

On the tension zone and based on classical beam theory, a mathematical model was developed to assess the deformation modes for a T-stub assembly under various bending moments. This procedure is divided into two steps: first, the minimum tension force required to form the first plastic hinge or yielding of the bolts is calculated along with the corresponding deformation; secondly, the force and corresponding deformation to form the second plastic hinge in failure modes I and II is calculated. The temperature effect is defined by the reduction factors for yield stress and stiffness from EN1993-1-2 and by the bolt reduction factors obtained by Kirby (1995).

At the same time, Block (Block *et al.* 2004a, 2004b) carried out a numerical and analytical study to understand the behaviour of the column web compression zone in fire, including the effect of superstructure loading. The main aim was to develop a simplified analytical approach to predict the force-displacement response of the compression zone. The resistance of the compression zone at high temperatures was based on the room temperature approach applying the strength reduction factors in EN1993-1-2. The room temperature approach used in this study was chosen by statistical comparison of the design approaches (Block *et al.* 2004a):

$$F_{R,\theta} = k_{y,\theta} f_{yw} L_{eff} t_w k_{N,w} \tag{12}$$

$$k_{N,w} = \sqrt{1 - \left[\frac{\sigma_{N,w}}{k_{y,\theta} f_{yw}}\right]^2} \tag{13}$$

L_{eff} is the effective length and is related to the critical load, $\sigma_{N,w}$ is the axial stress in the column web, f_{yw} is the yield stress of the column web and t_w is the column web thickness. The factor $k_{N,w}$ was derived from the transverse load capacity due to bending stresses in plate girders (Djubek and Skaloud 1976).

Additionally, Block *et al.* (2004b) proposed a reduction factor for the displacement at ultimate load, which describes the full force-displacement response of the column web in compression:

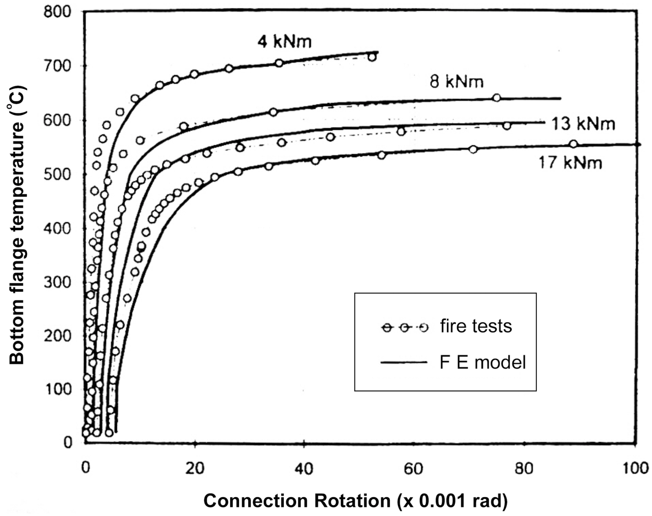


Fig. 12 Temperature-rotation curves at various moment levels for Leston-Jones flush end-plate connections (Liu 1999b)

$$\begin{cases} k_{N,\,\delta,\,20°\mathrm{C}} = -0.55\dfrac{\sigma_N}{f_y} + 1 \\ k_{N,\,\delta,\,\geq 400°\mathrm{C}} = -0.7\dfrac{\sigma_N}{f_y} + 1 \end{cases} \tag{14}$$

These studies recognized the interest in using the component method for the analysis of steel joints at high temperature. However, it must be stressed that much work still remains ahead to adequately characterize the behaviour of the various components at elevated temperatures.

4.1.4. Finite element models
A three-dimensional finite element code (FEAST) was developed at the University of Manchester to simulate the structural response of steel and composite joints at elevated temperatures (Liu 1999b). The program uses isoparametric shell finite elements to model the steel and concrete slab and a sophisticated beam-spring element to model the bolts. It is linked to FIRES-T3 (Iding *et al.* 1977), which performs the thermal analysis. Comparisons against available tests results (Lawson 1990, Leston-Jones *et al.* 1997, Al-Jabri *et al.* 1998), as shown in Fig. 12, yield good agreement.

4.2. Influence of axial restraint

4.2.1. Introduction
As stated in the introduction, the behaviour of a steel joint depends on the redistribution of internal forces with time acting on the joint as a result of the global behaviour of the structure. Under real fire conditions, the actual behaviour clearly deviates from the results of isolated joint tests (Wald *et al.* 2005), the joint being subjected to a full 3D stress state (N, M_y, M_z, M_t, V_y and V_z) resulting from local, distortional or global instability of the connected members. The effect of axial restraint and the associated thermal expansion of the structural members play an important role in inducing this 3D state of stress and is reviewed in the following paragraph. Several researchers have addressed this aspect, preliminary attempts being credited to Burgess *et al.* (1990), and Saab and Nethercot (1991). Table 5 summarizes the work on this topic.

In addition to the studies described in Table 5, and in order to characterize the effect of axial and rotational restraint under real fire conditions, the authors developed numerical models of structural sub-assemblies with different end restrictions under two alternative thermal loadings. There are a) the standard temperature-time exposure, ISO 834 curve; and b) a heating and cooling curve, both of which are illustrated in Fig. 13. (Santiago *et al.* 2003, 2004). The program SAFIR (Franssen 2003) was chosen to carry out the numerical simulations, which is a specialized finite element program which includes geometrical and material non-linear analysis for studying structures subjected to fire.

This numerical work was carried out in the following two stages:

1st stage – Numerical simulations using 2D beam elements.

2nd stage – Numerical simulations using shell elements.

4.2.2. Behaviour of a 2D steel beam under fire loading including the end joint response
A 2D model was developed consisting of a beam with different axial and rotational end restraints: one end of the beam (end B) was assumed to be built-in in all simulations, while several possibilities were tested for the opposite end (end A) (Fig. 14). The applied thermal load is illustrated in Fig. 12 and the

Table 5 Influence of end-restraints on structural response under fire loading

Authors		Scope	Main results
El-Rimawi *et al.* (1997, 1999)	ISO 834	Influence of connection stiffness on the behaviour of steel beams - Analytical investigation based on the secant-stiffness approach. Connection characteristics expressed by Ramberg-Osgood curves.	The connection details and its temperature are not critical parameters in determining failure. The connection restraint depends on the ability of the supporting structure to sustain the induced bending effects.
Bailey (1998)	Temps. obtained from previous analysis	Development of a numerical model to simulate the structural response of steel framed building in fire - INSTAF. Semi-rigid connections were simulated by a 3D spring using Ramberg-Osgood curves.	The failure temperatures increase as the connection rotational restraint becomes stiffer. This software INSTAF was re-named VULCAN and is under continuous development.
Liu (1996, 1998)	Temps. obtained from previous tests	Numerical investigation carried out with a purposely developed program (FEAST). Includes the influence of bolted end-plate connections and axial restraint on the behaviour of a steel beam	The connection details do not substantially affect the overall performance of the beam in the event of a fire.
Allam *et al.* (1999)	ISO 834	Experimental and numerical study (VULCAN and FEAST) to investigate the influence of axial restraint on the behaviour of a steel beam.	The joint rotational stiffness reduces beam deflection and the restraint to axial movement prevents the run-way behaviour at high deflections.
Liu *et al.* (2001a,b) Liu and Davies (2001)	ISO 834	Development of a three-dimensional mathematical model to simulate the response of steel structures in the event of fire and comparison with experimental results. Includes the influence of axial restraint on the behaviour of steel beam and catenary action	Includes a description and an explanation of whole beam behaviour during a fire. Evidence of catenary action in cases with lower load levels and high axial restraints. Proposes a simplified analytical approach to evaluate the axial forces developed during whole fire.
Huang *et al.* (2002)	ISO 834	Numerical investigation – FEMFAN. Includes the influence of axial restraint on the behaviour of steel beam and catenary action	The large deflections developed at high temperatures are not controlled by the steel degradation, but by the axial restraints. At high temperatures, the plastic and creep strain together with post-buckling large displacement retards the restrained beam failure.
Yin and Wang (2004, 2005a,b)	Steel temp. from Liu tests	Numerical investigation of the beam behaviour during whole fire, under the effect of axial and rotational end-restraints – ABAQUS. Development of an analytical tool. Comparison with experimental results.	Development of a simplified calculation method to analyze the beam behaviour during a fire. This method is applied to steel beams with non-linear axial and end-restraints and non-uniform temperature distribution over the cross section.

mechanical loading, was assumed to represent a typical serviceability condition for an office building and was taken as a constant uniformly distributed load of 14 kN/m. The beam and its joints were assumed unprotected and were exposed to the fire.

From the numerical analysis, it is found that compared to the usual assumptions of a "standard" fire (ISO 834 fire curve and simply-supported boundary conditions) the results are distinctly different. Firstly, even for low axial restraint (beam end A), an initial increase of internal forces is noted as the

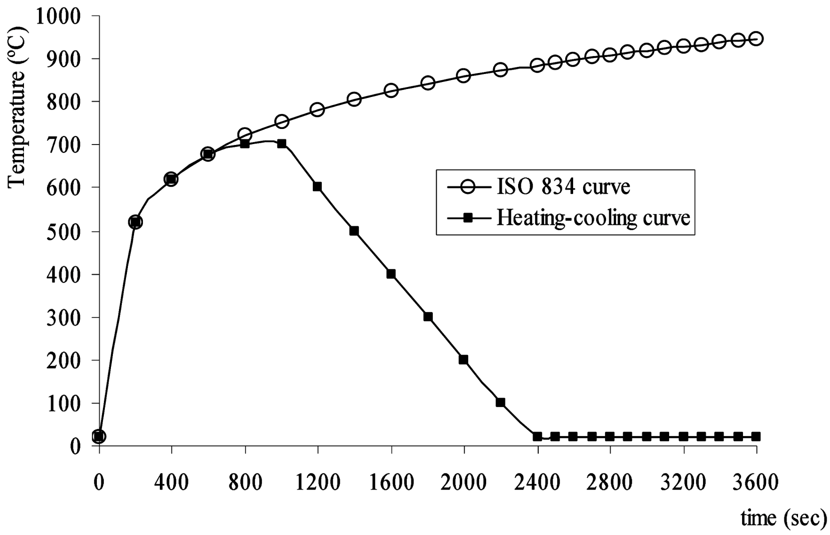


Fig. 13 Temperature-time curves (Santiago *et al.* 2003, 2004)

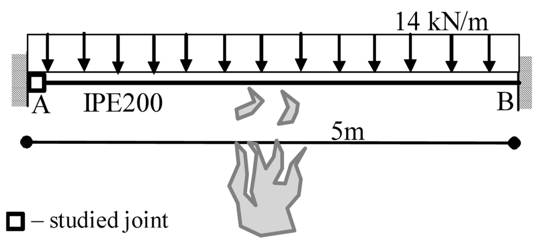


Fig. 14 Beam model

temperature increases, leading to a higher overall stress level in the beam without significant bending deformations. Then a reduction of internal forces took place, probably resulting from the bending deformation of the beam induced by the axial restraint and the initial deformation. This is followed by the development of catenary action for the case of high axial restraint, as clearly observed in Fig. 15(b). In the case of an ISO834 fire (continuous temperature increase) this eventually leads to numerical instability reflecting the reduction of the yield stress of steel below values that ensure equilibrium (strength based failure criteria). In contrast, for the real fire curve, complete reversal of the bending moment distribution is observed as the temperature decreases, thus allowing an increase of the axial force as the steel properties recover. Repeating the same analysis with a parametric variation of the rotation restraint at the beam end A reveals the same qualitative behaviour, the failure temperature slowly increasing with increased joint rotational restraint (Santiago *et al.* 2003).

4.2.3. Three-dimensional modeling of a beam-column subassembly subject to fire

The next step was the development of a 3D numerical model comprising two columns and a beam supporting a concrete slab (Fig. 16). Given that the behaviour is quite distinct from the results obtained using the ISO 834 fire curve, as shown by the 2D beam model, thermal loading consisted of a heating-cooling curve only, as illustrated in Fig. 13. The parametric study considered a corner column under two thermal boundary conditions: (i) column faces thermally protected and unexposed to the fire load;

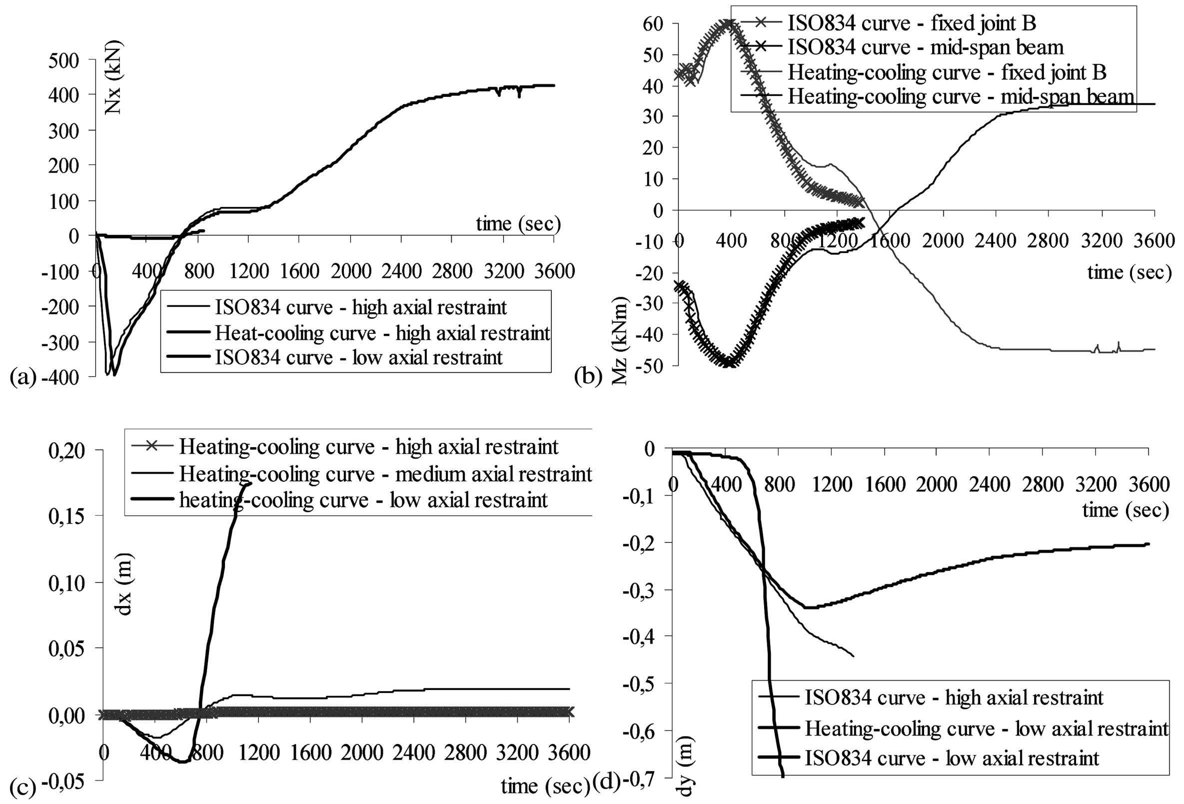


Fig. 15 Development of internal stresses and displacements: (a) Axial force; (b) Bending moment of beam high axial restrained; (c) Axial displacement; (d) Mid-span beam deflection

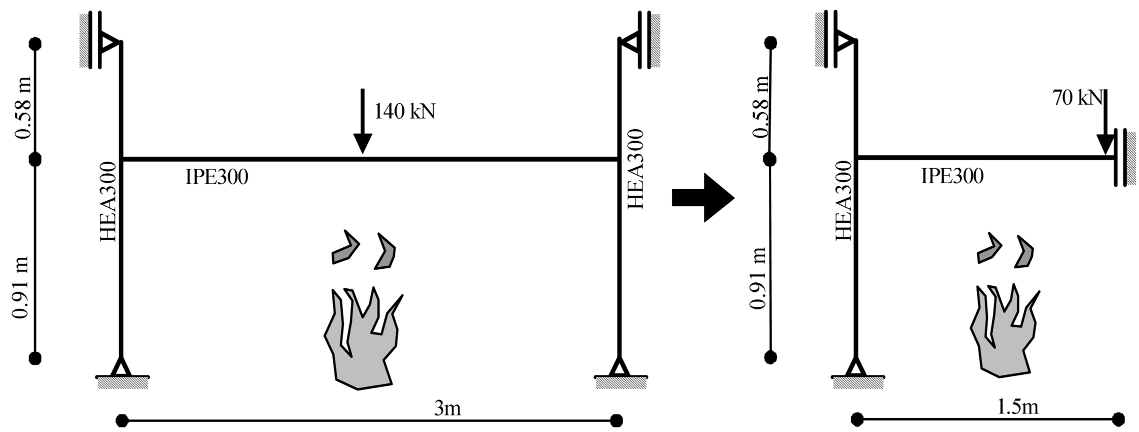


Fig. 16 Structural column - beam subassembly model

(ii) column faces unprotected and the external column flange protected from the fire load. These two cases correspond to two alternative design concepts, concerning the use of fire protection: (i) unprotected beams and protected columns, which have been shown to give good fire performance

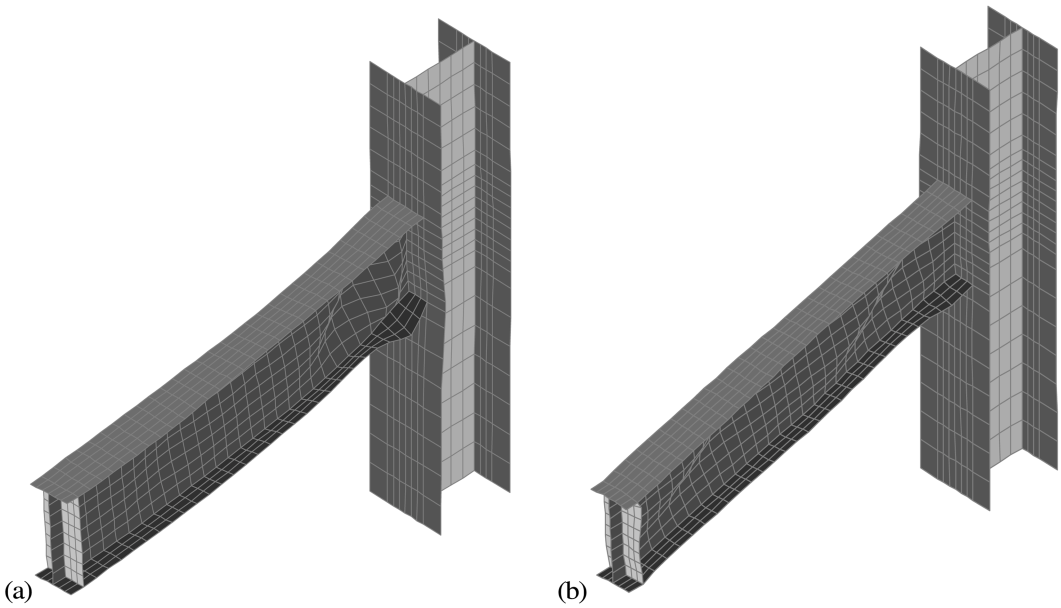


Fig. 17 Deformed structure: (a) at 900 sec; (b) at 3600 sec (end of the analysis)

(Wald *et al.* 2005), and (ii) unprotected beams and partially exposed columns. It should be mentioned that in case i) the axial and rotational stiffness as well as the strength of the columns remain constant during the whole fire, while in case ii) they were all reduced as the temperature increased. Finally, it was considered that the beam was continuously restrained against lateral-torsional buckling due to the supported concrete slab.

From the 3D numerical simulations it was possible to identify the evolution of local deformations. Case i) shows that the lower beam flange and the beam web adjacent to the connection started to buckle locally at around 610^{o}C due to the restraint to thermal elongation provided by the adjacent cooler column. The heated lower flange of the beam was unable to transmit the high normal forces generated in the lower flange of the beam where it meets the column. As the temperature increased, shear failure of the beam web was observed (Fig. 17).

Comparing the behaviour of the beam in both cases, it is noted that in case i) the beam starts to deflect faster than in case ii) due to the high column restraint that reduces the thermal expansion and increases the vertical deflection. However, at high temperatures, case i) does not deflect at an accelerating rate as observed in case ii), where the structural response is largely influenced by the column restraint behaviour: the column flexibility induces unrestrained beam axial displacement, see Fig. 18 (Santiago *et al.* 2004).

Fig. 18 compares the results obtained using shell and beam finite elements. Close examination shows that the resulting curves are qualitatively similar but the maximum values are quite different. These differences result from the fact that shell elements allow local buckling to develop, in contrast to the beam elements which constrain the distortion of the cross section.

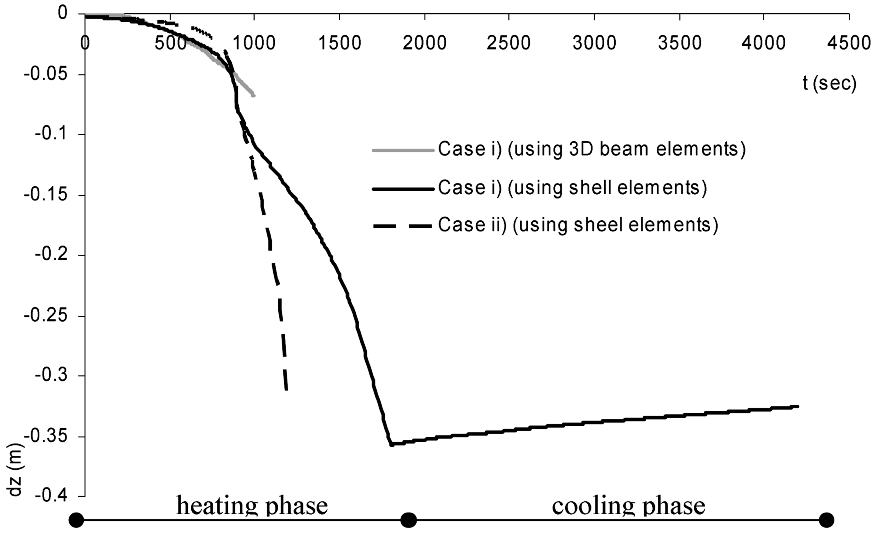


Fig. 18 Vertical displacement in the mid-span beam

5. Conclusions

The behaviour of steel joints under fire loading was reviewed in this paper. Historically, it was only in the last fifteen years that this subject started to receive greater attention from the scientific community. Despite the extensive work carried out in this period, the main conclusion is that the behaviour of steel joints under fire loading is still an open question.

The experimental evidence obtained from tests on isolated joints, tests on sub-structures and tests on complete structures (notably the Cardington fire tests) highlighted the following conclusions: i) during a fire event, the distribution of internal forces in the joint varies dramatically with time; ii) the redistribution of forces within the structure during the fire event strongly affects the joint behaviour; iii)

Fig. 19 Experimental layout

the joint response exhibits a true 3-D behaviour even when in cold-design the joint is only loaded in major axis bending; iv) expansion and subsequent contraction of the deformed steel members during fire induce high levels of axial force in the joint, probably leading to fracture of less ductile components.

These conclusions result in the need to develop more general design provisions that are currently available for the cold design of steel joints, the component method being an adequate compromise between accuracy and simplicity to address this issue. Several developments in this direction were also reviewed in this paper.

Finally, as an example of ongoing research, and to try to reach these goals, the authors are currently attempting to complement the results of the Cardington fire test (Wald *et al.* 2005) by carrying out seven substructure fire tests at the University of Coimbra. These tests use the observed and measured boundary conditions from the seventh Cardington test and include a thorough numerical simulation in line with the numerical results presented above (Santiago *et al.* 2003, 2004). Fig. 19 illustrates the experimental layout of a beam-column sub-frame with flush end-plate connections to be tested under a natural fire curve reproduced by a series of gas burners along each side of the beam.

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Performance of beam-to-column joints in fire—A review

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Abstract

This paper presents a state-of-the-art review on the behaviour of beam-to-column joints in fire and considers experimental and analytical research work on isolated joint configurations conducted with the prime objective of developing moment–rotation–temperature behaviour of joints. In addition, investigations on the effect of axial thrust on the behaviour of joints is presented because fire tests on a full-scale building, and observations from accidental fires, have demonstrated differences between the behaviour of joints when tested in isolation and considered as part of a complete building. Furthermore, joints that are routinely assumed as pinned at ambient temperature can provide considerable levels of both strength and stiffness at elevated temperature, albeit at large deformations, and this has been found to have a beneficial effect on the survival time of steel-framed buildings. It is noted that while FE analysis is capable of predicting accurately the performance of steel structures in fire, realistic models of joint performance are required. For this purpose, the use of a component approach for the prediction of joint performance is explained and appears to be a viable alternative to extensive joint testing or detailed FE analysis of joint details.

Keywords: Fire; Elevated temperature; Moment–rotation; Connections; Joints; Component models; Axial restraint; Cardington frame

1. Introduction

The properties of steel structures such as load-carrying capacity degrades rapidly in fire due to the reduction in both stiffness and strength of the material. Steel-framed buildings may be subjected to a fire and therefore must be constructed to ensure life safety and property protection. Using applied fire protection remains the most usual way of satisfying the fire resistance requirement for a steel-framed structure. This can be achieved by covering the exposed steel with a prescribed thickness of an insulating material. However, it is more rational to consider fire as an additional load case, like wind or earthquake [1], and design the structure to withstand fire without protection rather than designing the structure at ambient temperature and then applying fire protection. Therefore, in the past 20 years, there has been much interest in understanding the behaviour of different structural elements in fire and experiments have been conducted on steel members and joints either in isolation or as part of a sub-frame assembly or even a full-scale structure.

All structural members exposed to fire heat up, but the rate of temperature rise in each member is different. Joints in a steel-framed building tend to heat up slower than the material within the span of the beam because of the presence of additional materials (bolts, plates, angles, etc.) and due to their shielded location (i.e. usually beneath a composite floor). EN 1993-1-2:2005 [2] suggests temperatures at joints of between 62% and 88% of that in the beam lower flange temperature at mid-span. Alternatively, the temperature distribution may be calculated based on the exposed area: volume ratio of the component (plate, angle, bolt, etc.) but Franssen [3] reported that this method may be unconservative.

Traditionally, the design of steel-framed structures assumes that the actual behaviour between the beam and the column is either rigid (implying complete rotational continuity) or pinned. However, actual connection behaviour exhibits characteristics over a wide spectrum between

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these two limits; connections regarded as pinned generally possess some rotational stiffness while rigid connections display some flexibility [4]. Joints that were assumed pinned at ambient temperature could provide considerable levels of both strength and stiffness at elevated temperature. Therefore, joints can have significant influence on enhancing the survival time of structural members (such as beams) through redistribution of forces induced in the beam to the adjacent cold members [5,6]. Wang [7] sounds a word of caution; if the benefits of semi-continuous action are utilised in ambient temperature design, and this joint capacity is included when calculating the beam's load ratio, it is possible that the fire resistance may be lower than if the beam were designed as simply supported.

Experimental investigations conducted on the performance of steel joints at elevated temperature are relatively recent and rather limited in number, partly because of the high cost of the fire tests and the limitations on the size of furnace used. The joint tests conducted are primarily focused on establishing the moment–rotation relationships of isolated joints and comprehensive understanding of joint behaviour cannot be achieved based on the outcomes from experimental fire tests alone. Results from fire tests conducted on isolated joints provide very important fundamental data on the behaviour of joints but they do not truly reflect the actual behaviour of joints in the event of fire due to the absence of structural continuity. Thus, in recent years, extensive research has been carried out based on numerical simulation of joints and structural frames subjected to fire conditions [8]. This paper brings together research work that has been conducted to investigate the behaviour of beam-to-column joints at elevated temperatures in order to provide the reader with a broad understanding of the extent of research in this field. The review considers experimental work on isolated joints, various forms of analytical methods that have been developed to predict the behaviour of both bare steel and composite joints and the effect of structural continuity on the performance of joints in fire.

2. Experimental tests on isolated joints

The first experimental fire tests on joints were conducted by CTICM [9] in 1976 with six joint types ranging from "flexible" to "rigid". The primary purpose of these tests was to investigate the performance of high-strength bolts at elevated temperatures, and no indication of the performance of the joints was reported. Results suggested that the gross deformation of the other elements preceded failure of the bolts. Two tests were carried out by British Steel [10] in 1982 on a "rigid" moment-resisting joint. Despite the limited results obtained, it was concluded that joint elements could suffer significant deformation in fire. The first tests to measure the structural continuity afforded by beam-to-column joints at elevated temperatures were carried out by Lawson [11]. The aim of the programme was to develop a design approach for steel beams taking the rotational restraint provided by the joints into consideration. Five of these tests were on non-composite beams, two on composite beams and one on a shelf angle floor beam. Three types of typical joints were studied; an extended end plate, a flush end plate and a double-sided web cleat. The tests demonstrated the strength of these joints in fire and showed that significant moments (up to two-thirds of their ambient temperature design moment capacity) could be sustained in fire conditions. Lawson also noted that the bolts were not susceptible to premature failure, and that the rotation of the joints in all tests exceeded 6°. In addition, he suggested that composite action in fire contributed to an enhanced moment capacity of joints, which could be estimated by adding the moment capacities of the bare steel joint and the reinforced concrete slab. Lawson proposed simple rules for designing simply supported beams in fire taking into account the moment transferred via joints in fire conditions. Although the tests results provided insufficient data to describe the moment–rotation characteristics of the joints, they did provide essential information for the earliest attempts at joint modelling.

Experimental tests were conducted by Leston-Jones et al. [12] to develop moment–rotation relationships for flush end plate joints at elevated temperatures. Eleven tests were carried out on small-scale specimens, including two tests at ambient temperature, for both bare steel and composite joints. Results demonstrated that both the stiffness and moment capacity of the joint decreased with increasing temperature with a significant reduction in capacity for temperatures in the range of 500–600 °C. These tests provided useful data for connection modelling, but for a limited range of details, using relatively small section sizes for comparison with earlier ambient temperature joint testing work by Davison et al. [13]. A series of elevated-temperature joint tests was conducted by Al-Jabri et al. [14] to study the influence of parameters such as member size, end plate type and thickness and composite slab characteristics, on the joint response in fire. The joint types included two flush end plate and one flexible end plate bare steel joints and two flexible end plate composite joints. For each joint, a series of tests were conducted, each at a constant load level but with increasing furnace temperature. A family of moment–rotation–temperature curves were established for each joint [15,16]. Table 1 summarises some of the experimental fire tests conducted on isolated joints.

In 2004, Spyrou et al. [17,18] reported the results of an experimental investigation of the performance of the tension and compression zones of steel joints at elevated temperatures. A total of 45 T-stubs were tested at elevated temperatures and 29 column web transverse compression tests. Simplified analytical models of both the tension and compression zone were developed and compared with the experimental results. The analytical model for the tension T-stubs proved capable of predicting with reasonable accuracy the failure in any one of the three modes; formation of plastic hinges in the flange near the web

Table 1
Details of some experimental fire tests conducted on joints

References	Joint type	Beam size	Column size	Bolt size & number	End-plate thickness (mm)	Orientation & arrangement
Lawson [11]	(2) extended bare	305 × 165 × 40 UB (S275)	203 × 203 × 52 UC (S275)	6 M20	12	Major Beam-column
	(2) flush bare	305 × 165 × 40 UB (S275)	203 × 203 × 52 UC (S275)	6 M20	12	Major Beam-column
	(1) web cleat bare	305 × 165 × 40 UB (S275)	203 × 203 × 52 UC (S275)	6 M20	12	Major Beam-column
	(1) flush bare	305 × 165 × 40 UB (S275)	203 × 203 × 52 UC (S275)	6 M20	12	Minor Beam-column
	(1) flush bare	305 × 165 × 40 UB (S275)	203 × 203 × 52 UC (S275)	6 M20	12	Minor Beam-beam
	(1) web cleat bare	305 × 165 × 40 UB (S275)	203 × 203 × 52 UC (S275)	6 M20	12	Minor Beam-beam
	(1) flush composite	305 × 165 × 40 UB (S275)	203 × 203 × 52 UC (S275)	6 M20	12	Major Beam-column
	(1) web cleat composite	305 × 165 × 40 UB (S275)	203 × 203 × 52 UC (S275)	6 M20	12	Major Beam-column
	(1) flush composite	305 × 165 × 40 UB (S275)	203 × 203 × 52 UC (S275)	6 M20	12	Minor Beam-beam
Leston-Jones [12]	(4) flush bare	254 × 102 × 22 UB (S275)	152 × 152 × 23 UC (S275)	6 M16	12	Major Beam-column
	(5) flush composite	254 × 102 × 22 UB (S275)	152 × 152 × 23 UC (S275)	6 M16	12	Major Beam-column
Al-Jabri et al. [14,15]	(4) flush bare	254 × 102 × 22 UB (S275)	152 × 152 × 23 UC (S275)	6 M16	8	Major Beam-column
	(4) flush bare	356 × 171 × 51 UB (S355)	254 × 254 × 89 UC (S355)	8 M20	10	Major Beam-column
	(3) flexible bare	356 × 171 × 51 UB (S355)	254 × 254 × 89 UC (S355)	8 M20	8	Major Beam-column
	(4) flexible composite	356 × 171 × 51 UB (S355)	254 × 254 × 89 UC (S355)	8 M20	8	Major Beam-column
	(3) flexible composite	610 × 229 × 101 UB (S275)	305 × 305 × 137 UC (S275)	14 M20	10	Major Beam-column
Lou and Li [21]	(2) extended endplate	H300 × 160	H240 × 240	8M20	16	Major Beam-column

(*) Number of tests conducted.

followed by bolt yield and fracture, formation of plastic hinges in the flanges near the web and the bolt lines followed by bolt yield and fracture, or bolt fracture with the flanges remaining elastic (Fig. 1). Block et al. [19,20] further developed the work on the compression zone, conducting over tests at elevated temperatures and refining the analytical model for this zone. Lou and Li [21] recently reported the results of two cruciform tests conducted on 16 mm thick extended end plates with M20 bolts under ISO834 fire curve conditions. The well-instrumented tests recorded temperature distributions and structural response and were used to validate non-linear FE modelling. Burgess et al. [22] recently commenced a collaborative project with the University of Manchester to investigate the robustness of steel joints. This will involve testing a large number of joints under tensile forces at high temperatures to simulate catenary conditions.

3. Analytical simulation of the behaviour of joints

Although a large number of analytical investigations have been carried out to simulate the behaviour of joints at ambient temperature, very little research work has been conducted in fire conditions. This is mainly due to the large number of parameters that need to be taken into account when modelling the joint's response in fire. This is including modelling of material behaviour at elevated temperatures and complexities associated with the modelling process (i.e. modelling elements and areas of contact between elements) and the limited number of fire tests with which the models may be compared. Different types of models have been suggested ranging from simplified mathematical representation of the joint's response (curve-fitting equations) to simplified analytical methods such as component models and sophisticated finite element models.

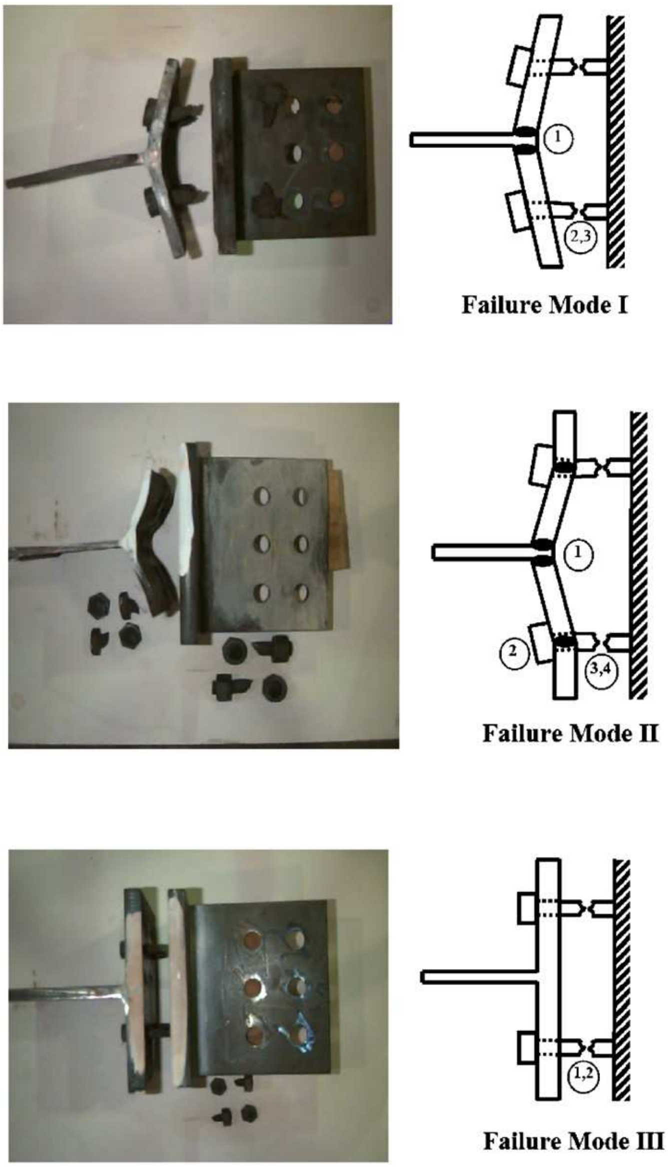

Fig. 1. Failure modes of T-stub arrangements in fire.

3.1. Mathematical representation of joint's characteristics

Mathematical representation of the joint characteristics (in the form of curve-fitting equations) have been proposed to represent the moment–rotation data of the joint generated from experimental tests in a numerical form suitable to be incorporated in the analytical models. In order to achieve acceptable results, these expressions should have the capability of representing the entire response of the joint. Due to the number of parts in a connection (plates, bolts, welds, sections), the response of joints to loading is usually complicated and moment–rotation relationships are generally non-linear. Such behaviour can be represented in a number of different ways with various levels of complexity. In simplified analytical models, simple forms of mathematical representation such as bi-linear, tri-linear and multi-linear forms of curve fit are adequate to describe the behaviour of the joint. More advanced mathematical expressions are usually adopted to represent the joint's characteristics in sophisticated analysis such as finite element models. Fig. 2 shows different forms of curve-fitting representations of joint's behaviour.

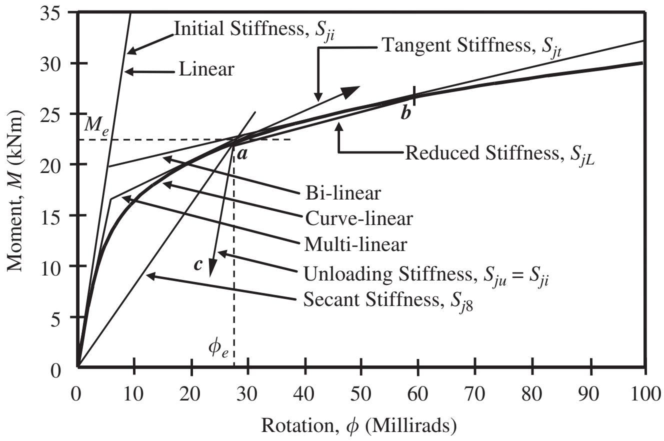

Fig. 2. Different forms of curve-fitting representation of joint's characteristics.

Initial attempts to model the elevated temperature moment–rotation characteristics by mathematical expression were made by El-Rimawi et al. [23]. They proposed a simple equation with three parameters, to represent the moment–rotation–temperature data obtained from experimental fire tests, using the modified Ramberg–Osgood expression [24] of the following form:

$$\phi = \frac{M}{A} + 0.01\left(\frac{M}{B}\right)^{n}, \tag{1}$$

where ϕ and M are the joint rotation and the corresponding level of moment, respectively; A, B and n are the temperature-dependent parameters.

The parameters A and B represent the joint's stiffness and strength, respectively, whereas n defines the curve sharpness. This relationship was then adapted by Leston-Jones et al. [12] and Al-Jabri et al. [15] to represent graphically the moment–rotation behaviour of semi-rigid connections in fire based on data from experimental tests.

Each test at a constant moment provides a set of temperature–rotation data. The complete data set therefore relates moment, temperature and rotation, and this can be used to fit a series of Ramberg–Osgood equations, each representing a moment–rotation curve for a particular temperature. The above equation is only valid for joints with single moment–rotation curves. However, for joints such as flexible end plates possessing two stages of moment–rotation behaviour (before and after contact of the beam flange with column flange), two separate moment–rotation expressions are necessary in order to represent the response of the joint accurately. The first moment–rotation curve can be represented by Eq. (1). A second Ramberg–Osgood curve can be created at an artificially defined origin (ϕ_1, M_1) coinciding with the location at which the beam bears against the column. The moment–rotation expression for the second stage may be defined as

$$\phi = \phi_1 + \frac{(M - M_1)}{A_1} + 0.01\left[\frac{(M - M_1)}{B_1}\right]^{n_1}, \tag{2}$$

where ϕ_1 is the rotation at which the beam flange comes into contact with the column; M_1 is the moment corresponding with ϕ_1; A_1, B_1 and n_1 are the temperature-dependent constants for stage two of response.

As an example of this technique, the response of a flexible end plate composite joint with two stages of moment–rotation behaviour is shown in Fig. 3 [15].

A simple procedure was proposed by Al-Jabri et al. [25] to predict the moment–rotation temperature response of different joints based on available ambient temperature–moment–rotation test data. The stress–strain characteristics of steel at elevated temperature are represented in EC3: part 1.2 [2] using the concept of a 'strength reduction factor' (which is really a strength-retention factor) to determine the residual strength of the steel at a particular temperature relative to its yield strength at room temperature. This reduction factor approach is applied to ambient-temperature joint rotational stiffness and strength based on moment rotation data. The proposed method can be summarized as follows:

1. Moment–rotation data of the joint at ambient temperature is represented mathematically using the modified Ramberg–Osgood equations as given by Eqs. (1) and (2).
2. The parameters representing the joint's stiffness (A and A_1) and strength (B and B_1) are reduced with increasing temperatures in accordance with the strength-reduction factors of structural steel presented in the design codes [2,26].
3. A family of moment–rotation curves for the joint with increasing temperatures is generated.

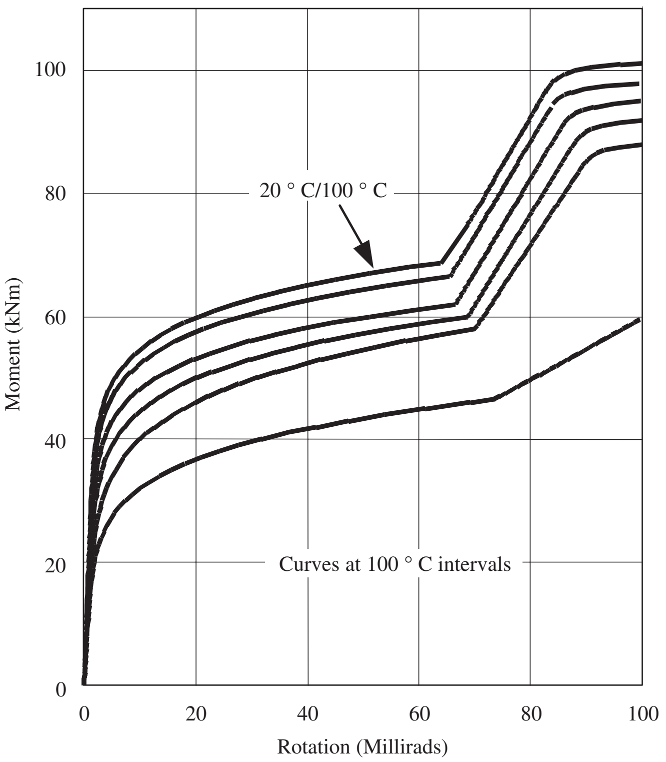

Fig. 3. Moment–rotation–temperature curves for a typical flexible end plate joint.

Comparison of the experimental results with the results obtained using the proposed method showed close agreement for both bare steel and composite joints as clearly shown in Fig. 4. The method can be used to predict, to an acceptable degree of accuracy, the elevated-temperature moment–rotation characteristics for both bare steel and composite joints provided that ambient-temperature data are available. It should be noted that the method was developed based on isolated joint tests where the joints were subjected to moments only. Therefore, it can be applied in analytical and numerical modelling of the steel-framed structures only in cases where the effect of restraint to thermal expansion of members is not likely to be serious since restraint to thermal expansion will produce axial forces in addition to moments.

3.2. Finite element models

The finite element method (FEM) provides an attractive means to investigate the beam-to-column joints in more detail than experimental tests would usually allow. A number of 3-D FE models has been developed by various authors taking into account geometrical and material non-linearities. Liu [27–31] was the first to attempt to use FEM in modelling connection behaviour at elevated temperature. He developed a finite element model; FEAST, to predict the behaviour of different types of joint at elevated temperature. The beam, column, end plate and stiffeners were modelled using eight-noded shell elements and considered the non-linear behaviour of the material with non-uniform thermal expansion across a section as well as large deformations in fire. The stress–strain–temperature characteristics were adopted based on recommended values from experimental tests. A close agreement was observed with experimental data for different types of joints [11,15]. A 3-D finite element model was developed by El-Houssieny et al. [32] to simulate the response of extended end plates at both ambient and elevated temperatures. Results from the developed model compared well with the experimental results and subsequent parametric studies were carried out to investigate the influence of connections on the behaviour of subframe elements in fire. ANSYS was used by Spyrou et al. [33] to model T-stub specimens at elevated temperatures. Good comparison was found between the experimental results and 3-D analyses. Rahman et al. [34] studied the response of fin plate joints in fire using ANSYS. Two types of 3-D solid elements, pre-tensioning

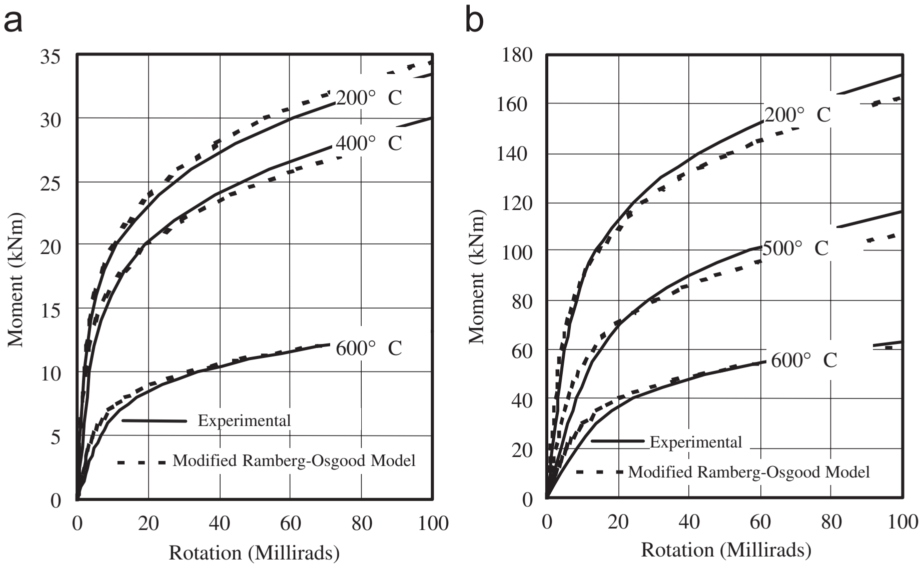

Fig. 4. Ramberg–Osgood model of the behaviour of bare steel and composite semi-rigid joints in fire.

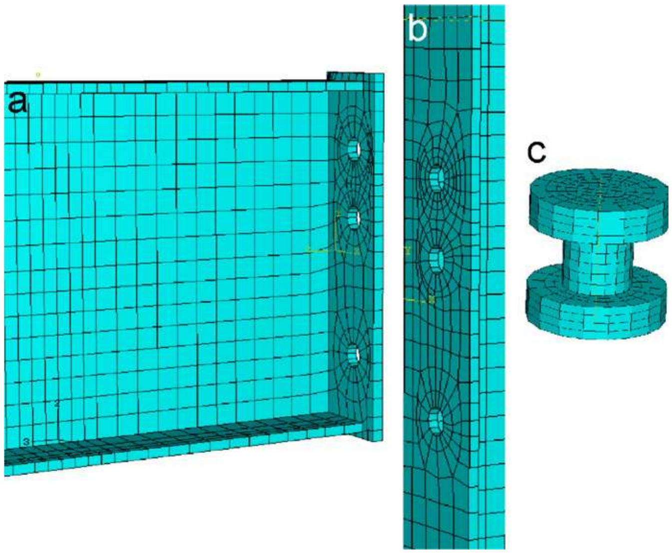

Fig. 5. 3-D finite element idealisation of the joint components.

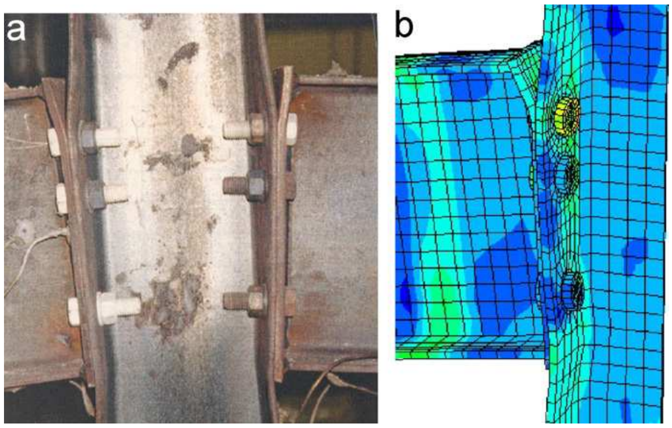

Fig. 6. Actual and predicted failure mode of flush end plate joint using FE analysis.

elements and contact elements were utilized in the modelling of beams, columns, fin plate and bolts, respectively. A transient time–temperature fire loading was applied. Despite realistic results predicted by the model, no experimental data were used to investigate its accuracy. Sarraj et al. [35] also developed 3-D ABAQUS models of fin plate connections, which include the important contact interaction between the bolts and the fin plate and beam web. The models were validated against lap joint data at ambient temperature [36] and a 2005 fire test conducted by Wald et al. [37] at the Czech Technical University. Sarraj has used the FE modelling to develop a component spring model assembly.

Recently, a finite element model was developed by Al-Jabri et al. [38] to study the behaviour of flush end plate bare steel joints at elevated temperatures using a general purpose finite element software ABAQUS. The finite element model was used to establish the moment–rotation characteristics of the flush end plate bare steel joints with a concentrated force and elevated temperatures. The joint components (Fig. 5) were modelled using 3-D brick elements, while contact between the various components was modelled using Coulomb friction. Material non-linearity was considered to model steel members and joint components. Degradation of steel properties with increasing temperatures was taken in accordance with the design code recommendations [2]. The obtained FE-simulated failure modes and moment–rotation–temperature characteristics of the joints compared well with the experimental data in both the elastic and plastic regions as shown in Figs. 6 and 7, respectively. Lou and Li [21] used ANSYS to model the behaviour of cruciform tests with extended end plates in fire. A sequential analysis was used; a transient thermal analysis was conducted first followed by a static structural analysis. Non-uniform thermal expansion, geometric non-linearity, temperature-dependent non-linear material behaviour, bolt pretension and surface-to-surface contact were all included in the analyses. Excellent correlation between the analyses and experimental results from two fire tests was achieved.

From the presented overview it is clear that FE methods provide a reliable technique, which can efficiently be used in predicting the elevated-temperature behaviour of joints to an acceptable degree of accuracy and enable a wider range of parameters to be considered than would be the case with a laboratory-based investigation.

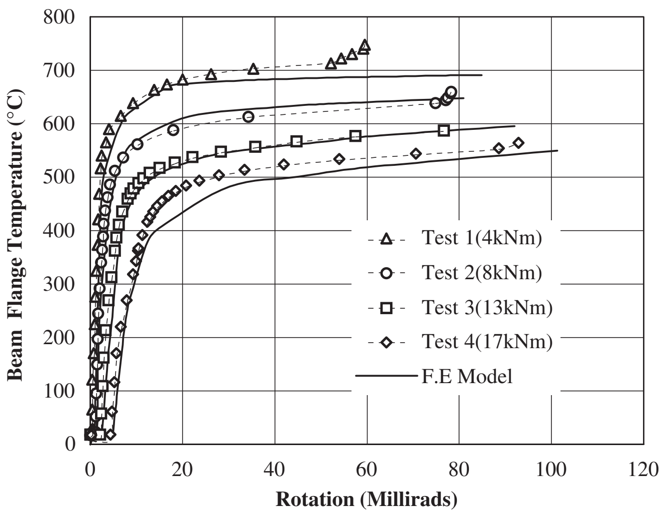

Fig. 7. Comparison of FE and experimental results of flush end plate bare steel joint for four different fire tests.

3.3. Prediction of the joint's characteristics using component-based models

Component-based (also known as spring-stiffness) models have recently become popular in predicting the joint's response at both ambient and elevated temperatures. They are based on dividing the joint into its basic components of known mechanical properties. Each joint component such as end plate, column flange, bolts, etc., is idealised as a spring of known stiffness as shown in Fig. 8. The behaviour of the joint may be determined by assembling the stiffnesses of individual components to obtain the global stiffness of the joint. The elevated temperature response of the joint may be predicted by allocating an individual temperature-stiffness profile to each element at a given bolt row, allowing the modelling of any form of temperature distribution based on test data. The joint's characteristics (i.e. the stiffness and strength) are degraded with increasing temperatures. Extensive research work [39] has been conducted in modelling the behaviour of isolated joints (without considering the effect of axial forces) at ambient temperature using the component method. Outcomes of the research work are reflected in the development of EC3: part 1.8 [40], which concerns the design of joints at ambient temperature using the component method.

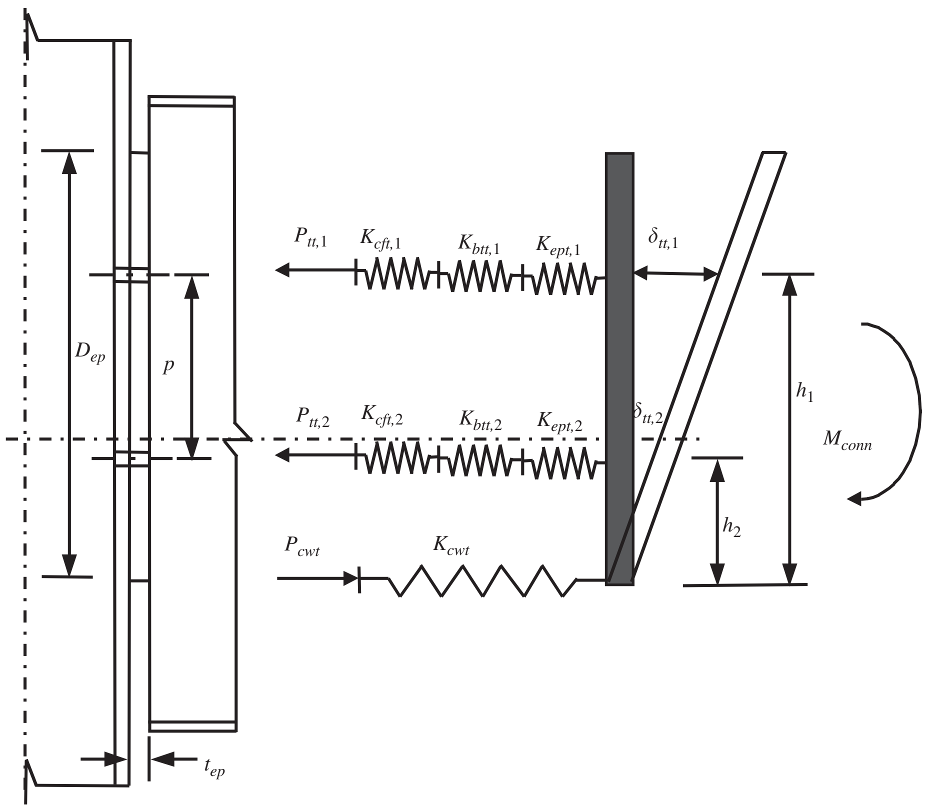

Fig. 8. Component model for flexible end plate joints.

Elevated-temperature component-based models are rare and the initial work to study the elevated temperature joint's behaviour was conducted by Leston-Jones [41] who modelled the response of bare steel and composite flush end plates joints. Comparison with experimental data showed that there was a close agreement between the results for bare steel flush end plate joints. However, the composite model showed a significant discrepancy in the rate of degradation compared with experimental results for elevated temperatures. Spyrou [18] followed up this work and successfully developed component models for the tension and compression zones [17]. He combined these spring models into a simple component-based model [42], shown in Fig. 9(a), which idealises the beam-to-column joint as two rigid bars, connected by two non-linear springs simulating the tension and the compression zones; the load–deformation characteristics of the two springs are shown schematically in Fig. 9(b) and a comparison of the resulting model predictions with Leston–Jones tests data is shown in Fig. 10.

A similar component model was proposed by da Silva et al. [43] for the behaviour of steel joints at elevated temperatures. The proposed model was restricted to bare steel flush end plate joints. Comparison with experimental results available in the literature has shown good agreement with the proposed method.

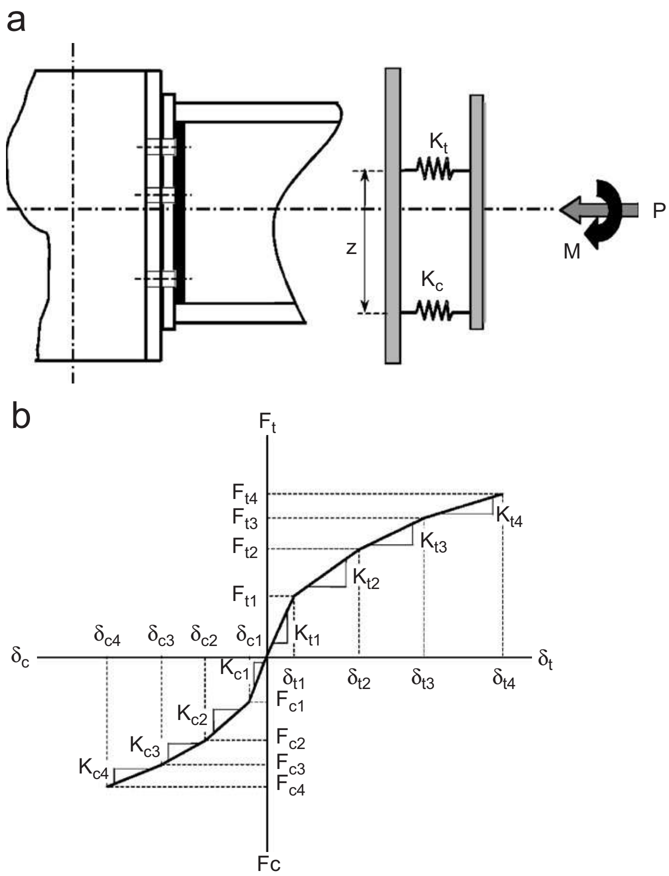

Fig. 9. Behaviour of joint subjected to axial thrust.

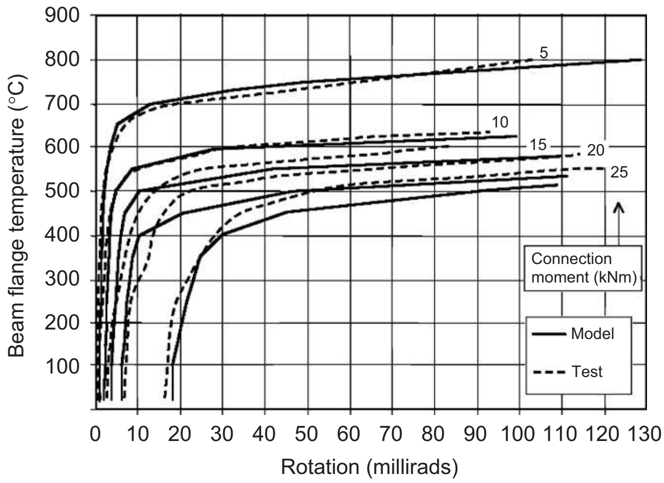

Fig. 10. Comparison between experimental and component model results.

Al-Jabri [44,45] adopted a similar approach to model the behaviour of bare steel and composite flexible end plate joints. Comparison of the bare steel component models with existing test data generated good results especially in the elastic zone. Also, the predicted rate of degradation of the joint stiffness and capacity compared well with the experimental results. For composite joints, the predicted and the measured responses agreed well for ambient temperature. Results were also encouraging for elevated temperatures, but in some respects more experimental data are required to draw positive conclusions.

Building on earlier work by Spyrou, Block et al. [46] further developed a component model for end plate connections, which includes the end plate in bending, the column flange in bending, bolts in tension and the column web in compression (see Fig. 11). The first three components form the tension zone of the connection and are combined as two T-stubs in series. A shear spring is included to transfer the vertical load from one node to another but it is assumed to be rigid at present, although the formulation of the element allows the implementation of slip and shear failure of the bolts. The shear zone in the column web is not yet included, which limits the use of the element to internal joints with equal moments in which the column web does not experience shear deformations. The model was validated against the test data of Leston–Jones [41] with good correlation, as seen in Fig. 12.

4. Effect of structural continuity on the performance of joints

The majority of research work that has been carried out to study the behaviour of beam-column joints was focused on testing and modelling isolated joint configurations, where the effect of structural continuity and the presence of axial forces (tension and/or compression) are usually ignored. Therefore, joints were predominantly assumed to act in bending and the major research efforts were directed at establishing moment–rotation responses in the absence

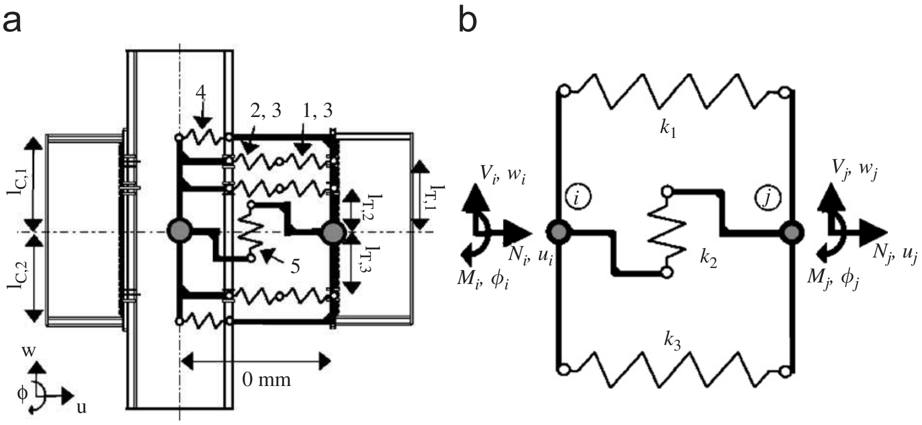

Fig. 11. Component model developed by Block for joint with shear deformation.

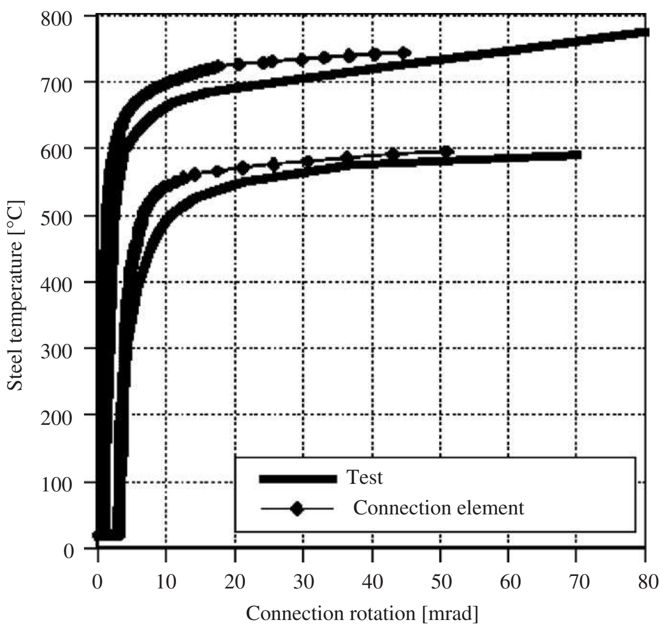

Fig. 12. Comparison between results from experimental tests and the component model with shear spring developed by Block.

of axial force in the beam. In some steel structures, such as pitched-roof portal frames, sway frames or frames with partially constructed floors, the magnitude of the generated axial forces in the beams is significant [47,48] which affects the ambient-temperature performance of joints. The effect of axial forces is insufficiently addressed in part 1.8 of EC3 [40], it only suggests a single empirical limitation on the allowable axial force of 10% of the beam axial plastic resistance, below which the axial force could be ignored [48].

Several research attempts have recently been made to study the effect of axial force using the component method [49–52] in order to establish bending moment–axial force (M–N) interaction curves and initial stiffness for different joint configurations. Unfortunately, no experimental data describing the behaviour of joints under combined axial force and moment were available to validate these models. However, Wald and Ŝvarc [53] and Lima et al. [47,48] studied experimentally the behaviour of beam-to-column joints in the presence of axial force and its influence on the overall behaviour of joints. In the former study, two tests were performed on beam-to-beam and beam-to-column joints while the latter examined 15 joint configurations, i.e. eight flush end plate and seven extended end plate joints. These studies confirmed that the presence of axial force significantly affects the joint's structural behaviour. From the above discussion, it is clear that the effect of the axial force on the behaviour of joints at ambient temperature is not fully understood and, therefore, further research work is needed in order to have better understanding to what extent axial restraint influences the joint's response.

The effect of axial force on the behaviour of the joints is much more critical when steel structures are subjected to fire. In fire, beams expand due to high temperatures, whereas they contract when the structure cools down causing development of thermal stresses in the beam. These thermal stresses induce very high axial forces in the beam, which considerably affects the behaviour of the connecting joints. Therefore, the joints should have the capability to redistribute the forces to the adjacent cold structures. Recent experimental and analytical studies [54–56], which were performed on a sub-frame assembly, have concentrated on investigating the effect of axial restraint on the behaviour of steel beams in fire without giving considerable attention to the behaviour of joints under bending and axial restraint. However, Liu et al. [54] concluded that joints could enhance the fire resistance of a beam by reducing some of the mid-span moment during most of the time when temperature is rising, despite the possibility of local buckling in the beam. Simões da Silva et al. [57] provided a useful summary table of the work conducted on the influence of end restraint on structural response under fire loading in their recently published review paper.

4.1. Lessons learned from the cardington full-scale frame tests

Following accidental fires, a complete structure behaves better than its individual members in isolation due to the interactions of structural members causing an increase in the inherent fire resistance of steel-framed buildings [1,58]. Thus, in order to understand the behaviour of a real structure under natural fire conditions and to collect data that would allow verification of the analytical models, which are capable of analysing structures in fire, the Building Research Establishment (UK) conducted a series of fire tests on a full-scale building structure at Cardington, UK. The structure was an eight-storey office building design, which was considered as a typical example of the type of braced structure and load levels that are commonly found in the UK. These tests were commonly known as Cardington tests. In total six fire tests were conducted on the composite frame during 1995–1996 which included a restrained beam test, a plane frame test, the first corner test, the second corner test, the large compartment test and a demonstration test. A full description of the Cardington full-scale frame and its associated tests is presented in detail elsewhere [59–62].

Two types of joints were used in the Cardington frame: flexible (partial depth) end plates and fin plates. Flexible end plates were used for beam-to-column joints and fin plates for beam-to-beam joints. These joints are usually considered as pinned joints that are assumed to transfer shear forces and have sufficient flexibility to allow rotation. Results from Cardington fire tests have pointed out the following critical structural aspects that need addressing with respect to the behaviour of joints in fire [61–64]:

- Despite the fracture of the end plates along one side, the connected beams showed no sign of collapse under very high deflections (Fig. 13).
- The temperature of the beam bottom flange was considerably higher than the temperature of the joint. At the maximum temperature, the joint temperature was almost 200 °C lower than the temperature of the beam. The temperature of bottom bolts was higher than the temperature of the top bolt due to the shielding by the composite slab, which acted as a heat sink to the top part of the joint. The end plate was hotter than the bolts at the same level because of the ratio of the bolt diameter to the end plate thickness.
- The joints showed signs of being subjected to high tensile forces. In the flexible end plate joints, the plates had fractured down one side while the other side remained intact as shown in Fig. 14 and in the fin plates (beam-to-beam joints) the bolts had sheared (Fig. 14(b)). This fracture occurred as a result of the high tensile forces developed during the cooling of the connected beam under large rotations associated with pinned joints. During cooling steel contracts, causing a reduction in the vertical deflection of the beams while a significant tensile force is applied to the weld connecting the partial-depth end plate to the beam flange. This tensile strain on cooling was relieved by the plate fracture or bolt shear of the joint as shown in Fig. 15. Also, the shear fracture of the bolts in fin plate joints suggesting that flexible end plate joints act more reliably in fire than fin plate joints. However, such behaviour was not observed in isolated member fire tests [65]. The behaviour of joints during cooling of the structure needs further investigation. An initial solution, to overcome

Fig. 13. Overall frame deformation after fire.

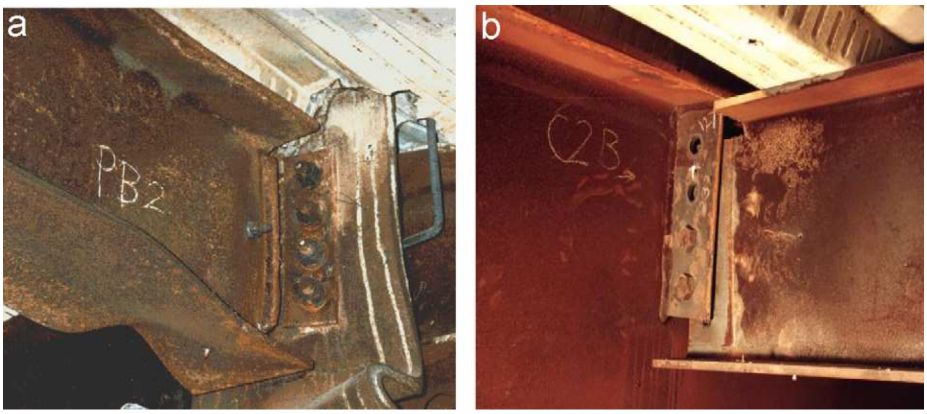

Fig. 14. Failure mode of flexible end plate and fin plate joints in fire.

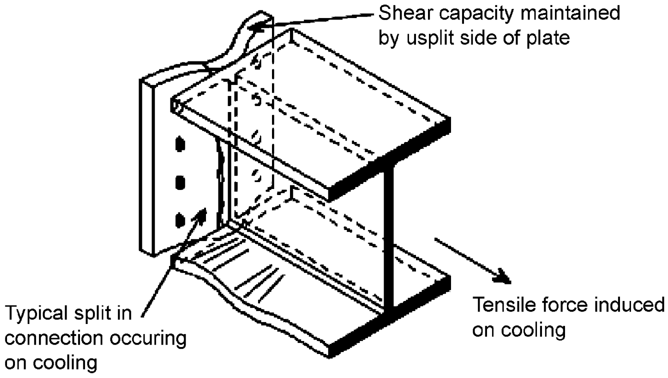

Fig. 15. Fracture of end plate during the cooling phase of the fire.

Fig. 16. Local buckling of the beam in the vicinity of the joint.

this situation, suggests designing the joint to have large ductile behaviour to ensure that shear capacity is maintained when subjected to high tensile forces in fire.

- Most internal beams showed signs of local buckling during the heating phase in the lower flange, and part of the web, in the vicinity of the joints (Fig. 16). This behaviour is caused by the restraint to thermal expansion and negative moment caused by the rotational restraint from the joint. The restraint to thermal expansion was provided by the surrounding cooler structure and the structural continuity of the test frame. This has taken place in the beam due to the inability of the lower flange of the beam to transfer the high axial forces induced in the beam to the adjacent beams–columns after closure of the gap in the lower part of the joints [64]. Therefore, conservatively, the joints should be assumed as 'pinned' and the connected beams as simply supported allowing larger mid-span deflections to develop. Results from a recent analytical study conducted on the first Cardington test [66] confirmed that the response of the structure was mainly dominated by the effects of thermal expansion and that material degradation and gravity loading were of secondary importance. Local buckling was found not to be a major concern in isolated member fire tests [63].

4.2. Towards a rational design method of joints in fire

During the last two decades, a number of numerical models have been introduced that have the capability of predicting accurately the performance of isolated joints in fire. Experimental studies on the behaviour of full-scale steel-framed structures will undoubtedly provide a more broad understanding of their real behaviour in fire. The fact that the frame survived the severe fire conditions experienced in the Cardington project raises a number of fundamental issues whether the current design methods are reflecting the true behaviour of structures in fire or not. However, it seems likely, given the huge expense of conducting full-scale fire tests that the fire engineering design of joints is going to be based on numerical modelling rather than experimental testing.

The mass of numerical data which has been generated and observations from the Cardington tests have already suggested a number of research routes which will lead to appropriate developments in standard numerical modelling techniques. Some of these are already in progress [67–71], and others will emerge as research progress. The use of modelling will remain crucial to the development of simplified design strategies and rules taking into account the interaction of structural members in fire. The improvements that have taken place within the past decade have been very significant in representing the reality of structural behaviour in fires. The ability to model such behaviour accurately will undoubtedly open new rational design approaches as confidence in its capabilities grows which reflect the real behaviour of joints as part of steel-framed structures in fire.

5. Conclusions

An overview of research conducted in the field of beam-to-column joints in fire has been presented. Compared with the volume of experimental work published on the behaviour of joints at ambient temperatures, relatively few test programmes have been conducted on the behaviour of joints in fire. Fire tests are expensive and complicated to conduct, and this combined with the large number of variables (joint dimensions, load levels, temperatures), makes joint modelling particularly attractive. FE modelling of joints can produce excellent predictions of test data and is a valuable tool for conducting parametric studies. A number of researchers have used FE modelling augmented by test data to develop simplified mechanical models of joint components with the aim of combining these into a sufficiently accurate model for use in global analysis. These component-based models are currently capable of accounting for joint dimensions and geometry, materials, temperature distributions and combinations of axial load and moment.

Also, the effect of structural continuity on the behaviour of joints at elevated temperature has been reviewed from which it has been concluded that the joints performed extremely well during fire despite being subjected high tensile forces during the cooling phase. Therefore, the effects of cooling on joint performance remain to be investigated.

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ANNEX C – Steel columns in fire

Departamento de Engenharia Civil
Faculdade de Ciências e Tecnologia da Universidade de Coimbra

ROBUSTFIRE Project

Document 3 – Steel Columns – v2(4)

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March 2009 IR_2009-003

UC/DEC/ISISE/2009-003 RFS-PR-07039-Steel_Columns-v2(4)

EXECUTIVE SUMMARY

The behaviour of steel columns subjected to fire conditions is vastly different from room temperature. At room temperature, local buckling and global buckling are the two failure modes of steel columns due to instability, governing by two factors: the width-to-thickness ratio of steel plates and the slenderness ratio of the columns respectively (Yang et al, 2006a). At high temperature, the stability of columns depends mainly on the column initial load, the column slenderness, and the restraint stiffness from adjoining unheated structure (Wang, 2004). This last factor, the thermal restraint, plays a key role in the stability of these columns.

A review of recent literature shows that the effects of axial restraint have been investigated numerically by Neves (Neves, 1995), Wang (Wang, 1997a) and Shepherd et al. (Shepherd et al., 1997), and experimentally by Simms et al. (Simms et al., 1997), Ali et al. (Ali et al., 1998), Rodrigues et al. (Rodrigues et al., 2000) and Tan et al. (Tan et al., 2007). Besides, the effects of rotational restraint have also been studied numerically by Wang (Wang, 1997a-b) and Valente and Neves (Valente and Neves, 1999), and experimentally by Ali and O'Connor (Ali and O'Connor, 2001), and Wang and Davies (Wang and Davies, 2003a-b). It was found that the critical temperatures of columns are reduced by axial restraint but increased by rotational restraint. Valente and Neves (Valente and Neves, 1999) found that even very weak rotational restraints, maintained at room temperature, can substantially increase the column critical temperature.

Slenderness and initial load were studied by a great number of authors, leading to the conclusion that the higher the column slenderness or the initial load, the lower the column failure temperature.

The instability of a column submitted to fire does not lead directly to the failure because of a postbuckling behaviour. The fire resistance of a steel column corresponds to the instant when the column compressive load in the postbuckling stage returns to its initial load. This definition was introduced by Neves (Neves, 1995) and confirmed later by Franssen (Franssen, 2000) and Wang (Wang, 2004).

Many analytical studies have been performed, leading to formulas able to predict the fire resistance of axially restrained steel columns (Huang and Tan, 2003a-b), or able to ascertain the column stability under a non-uniform temperature distribution without recourse to finite element modelling (Tan and Yuan 2008, 2009).

TABLE OF CONTENTS

I State of the art

I.1 Introduction

The present report introduces a state of the art on the behaviour of steel columns in fire and considers experimental, numerical and analytical research works. The review of experimental research works on steel columns in fire is described in the following paragraph (§I.2). This review is based on the review paper of João Paulo Rodrigues (Rodrigues et al., 2002), including in Annex, and on the Coimbra internal state-of-the-art report written by António Correia (Correia, 2008) for the recent developments. The review of theoretical research works is described in §I.3, based on the two PhD Theses of Bjorn Aasen (Aasen, 1985) and João Paulo Rodrigues (Rodrigues, 2000), on the Coimbra internal state-of-the-art report (Correia, 2008) and on recent publications.

I.2 Experimental research works

I.2.1 Rodrigues et al., 2002

This review paper about fire resistance tests on steel columns is reproduced in full in annex.

Rodrigues J.P.C., Gomes F.T., Neves I.C. "Fire Resistance Tests on Steel Columns – A Summary Review". XXX IAHS World Congress on Housing, Housing Construction – An Interdisciplinary Task, Coimbra, Portugal, pp. 1639-1651, September 2002.

I.2.2 Recent developments

I.2.2.1 Wang and Davies, 2003

In 2003, ***Wang and Davies*** published results of fire resistance tests performed by the Structures and Fire Research Group at the University of Manchester in England (Wang and Davies, 2003a-b). A non-sway loaded steel columns rotationally restrained by two loaded steel beams was exposed to fire. The objectives of the tests were to study:

- Changes in column bending moments due to variable column bending stiffness relative to the adjacent structure;
- How these changes could affect the columns critical temperature.

The column was in a horizontal position to suit the furnace, with the restraining beams connected to the web of the column (Figure 1), so that primary bending moments would act about the minor axis of the column and the column would fail by buckling in the minor axis direction. This ensures that the level of bending moments transferred by the beams could be sensitively reflected in the column behaviour. Two series of tests were carried out, one using fin plate beam-to-column connections and the other using extended end plate connections. Each assembly was loaded at the top of the column and on the two beams. In each series, 9 tests were performed, varying the level of total loads and beam loads. The applied loads were kept constant during tests in order to simulate a column with free thermal expansion.

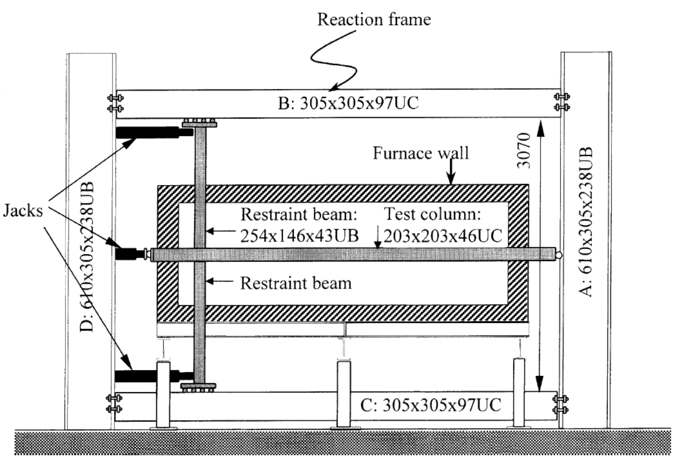

Figure 1. Test set up – Wang and Davies, 2003

After analysing the results, Wang and Davies concluded that:

- Bending moments in a steel frame column undergo complex changes under fire conditions. In all cases, the column bending moments reduced initially. With a high column axial load relative to the initial primary bending moment, the column bending moment reversed in direction at an early stage of fire exposure. With a low column axial load relative to the initial primary bending moment, the column bending moment also reversed direction, but this occurred at a very late stage of fire exposure just before column failure;
- Column failure temperatures were mainly dependent on the total applied load in the column and was not affected neither by the level of initially applied bending moments, nor by the types of beam-to-column connection;
- The final column effective length near failure was not sensitive to the types of beam-to-column connection;
- When using BS 5950 Part 8 or Eurocode 3 Part 1.2, the column failure temperatures and failure loads were most accurately calculated by ignoring the column bending moments and assuming an effective length ratio of 0.7.

I.2.2.2 Yang et al., 2005, 2006

In 2005, ***Yang et al.*** (Yang et al., 2005) performed, in Taiwan, experimental fire resistance tests on stub columns in fire-resistant steel. A new type of high-performance steel with an improved fire resistance characteristic of steel itself has been developed to enhance the fire resistant properties of steel structures. Using this type of steel could lead to a lower structural cost, avoiding, or at least reducing, the fire protection. However, the design criteria for the application of fire-resisting steel in steel columns are still limited. This research work examined the structural behaviour of this type of steel columns under fire load. The objectives were:

- To evaluate the variations of the ultimate strength of steel columns due to different width-to-thickness ratios under specific elevated temperatures;
- To investigate the reduction effects on column strength resulting in the increasing temperature;
- To establish the design guidelines of steel columns using fire-resisting steel.

A series of 24 specimens with tubular, quadratic and H sections were tested to the limit states under specified temperature level. They notably showed that:

- The ultimate loads and the ductility of the stub stiffened columns decrease while the temperature or the width-to-thickness ratio increase;
- The effect of the width-to-thickness ratio on the ultimate strength of box column at elevated temperature is more significant than that of H column.

Yang et al. concluded that temperature is the dominant factor affecting both the ultimate strength and the ductility of the specimen under fire. Design guidelines for the requirement of the width-to-thickness ratios of both box column and H column made of fire-resisting steel were proposed.

In 2006, ***Yang et al.*** (Yang et al., 2006a) presented results of fire resistance tests on steel H-columns (Figure 2). The width-to-thickness ratio of steel plates and the slenderness ratio of steel columns are two dominating factors linked to, respectively, local buckling and global buckling of columns. To evaluate the influence of these two factors on the structural behaviour of steel columns in fire, two series of H steel columns were loaded to their limit states at specified temperature levels (steady state tests). A total of 20 steel columns reached their limit states due to axial load under fire. The first series of tests on stub columns were performed in order to investigate the structural behaviour of steel columns under fire load, and, as previously, to examine the effects of the temperature and the width-to-thickness ratios on the ultimate strength of steel columns at elevated temperature. The objective of the second series of tests was to study the effect of the slenderness ratio on the fire resistance of long steel columns in fire.

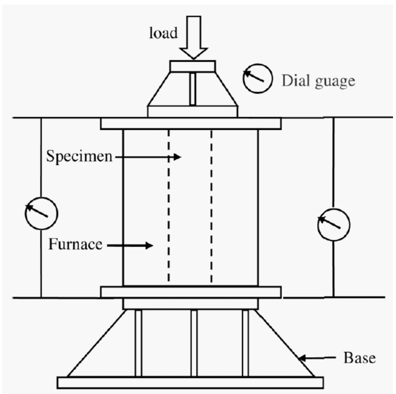

Figure 2. Test set-up – Yang et al., 2006a

The first series of tests showed that the column is able to retain 70% of the strength at room temperature, for temperatures lower than 500℃. The second series of tests was then performed at this critical temperature of 500℃. They showed that the strength drops seriously and the failure modes change from local to global buckling for a slenderness ratio greater than 50. So Yang et al. recommended designing the steel column with a slenderness ratio lower than 50, and with a critical temperature of 500℃ to ensure the columns retain more than 60% of their room temperature strength. The brittle failure by global buckling of the steel column should then be avoided.

Generally, the ultimate strength obtained by a steady-state test is larger than the ones obtained by a transient-state test. However, the difference is smaller when the strain is larger than 1%. In these tests, the differences were ignored due to strains corresponding to the ultimate strength of steel columns larger than 5%.

Also in 2006, ***Yang et al.*** (Yang et al., 2006b) presented results of fire resistance tests on fire-resistant steel H columns. The tests were similar to the previous ones performed on steel H columns. The first series studied the effects of the width-to-thickness ratios and the thermal effect on the residual stress of stub columns. The second series studied the factors affecting the global behaviour of long fire-resistant steel columns, the slenderness ratios and residual stress.

It was concluded from the tests that column strength is sensitive to slenderness ratio at elevated temperature. The strength of a slender column decreased sharply especially for temperatures above 600ºC. At 600ºC, fire-resistant steel columns retain more than 60% of the nominal yield strength. Due to the release of residual stress in fire, the failure mode changed from inelastic global buckling at room temperature to local buckling at elevated temperature. They concluded from this study that the compressive strength can be greatly improved by using this type of steel. Guidelines to design fire-resistant steel columns in fire conditions were proposed.

I.2.2.3 Tan et al., 2007 – Huang and Tan, 2007

In 2007, ***Tan et al.*** (Tan et al., 2007) performed experimental fire resistance tests, at Nanyang Technological University, on axially restrained columns in order to determine the failure time. Various axial restraint ratios were studied, simulating the adjacent cooler structure. Axial restraint was provided by a simply supported transverse steel beam placed at the column end, connected with an actuator (Figure 3). Rotational restraint was generated from unavoidable friction due to firm contact between knuckles.

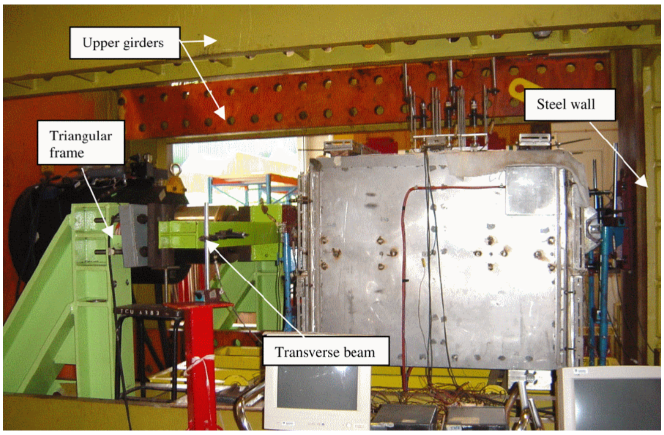

Figure 3. Experimental set-up – Tan et al., 2007

Test programme had three main objectives:

- Investigating the effect of column initial imperfections on failure times. For each test, the column initial out-of-straightness and load eccentricity were measured, while flexural springs were proposed for simulating the bearing friction in the FE models;
- Examining the axial restraint effect on the column failure time or temperature;
- Proposing a simple and direct approach for calculating the column failure time at elevated temperature.

Four series of bare steel columns were tested to failure, at realistic mean heating rate of 8ºC/min, varying the slenderness ratios. Each series was subjected to the following tests:

- One load capacity test at ambient temperature;
- One fire resistant test without axial restraint;
- Three fire resistant tests with various axial restraint ratios.

The working load N was taken as 50% of the associated experimental axial load capacity at room temperature.

The experimental results showed that:

- Column initial out-of-straightness and load eccentricity markedly reduce column failure time;
- Axial restraints significantly reduce the column failure time, due to the increase of the column internal axial force;
- Bearing friction significantly retards the column failure time and affects the structural behaviour during heating. Generally, friction effects are more noticeable in stocky columns than slender ones.

A simple Rankine approach (Huang and Tan, 2003a - §I.3.2.4) was used to compute the column failure times, which gave good predictions in comparison with the test results. So it can be used in the computation of steel column failure times/temperatures.

Experimental tests were also modelled using the finite element program FEMFAN2D (Huang and Tan, 2007), considering both material and geometrical nonlinearities. The objective was to propose a numerical approach to take into account the secondary effects:

- Proposition of a method to model the axial restraint and unavoidable boundary friction effects;
- Validation against experimental tests;
- Examination of the effects of load eccentricity, boundary friction and different steel material models at elevated temperatures.

Axial and rotational restraints were modelled by, respectively, an axial spring and a rotational spring. Analytical models were proposed.

From the evolution of the internal axial force P in function of the time, it was showed an increase of P at the beginning of heating, due to the thermal expansion, following by a reduction of P, due to the mechanical contraction. In some case, a short plateau was showed before a rapid decrease of P. This was due to the restoring effect of friction at the column ends that retarded or delayed column buckling.

Good agreement was shown between the test results and the FE predictions. The effects of load eccentricity, boundary friction and steel material models on structural responses were shown to substantially affect the column behaviour at elevated temperature.

I.2.2.4 Correia et al., 2007-2009 – Silva et al., 2008

Recently, a large program of fire resistant tests was carried out at the Laboratory of Testing Materials and Structures of the Faculty of Sciences and Technology of University of Coimbra (FCTUC), Portugal. ***Correia et al.*** (Correia et al., 2007, 2009 – Silva et al., 2008) performed a series of tests on steel columns embedded on brick walls. The walls have a favourable and an unfavourable effect on the fire resistance of the steel column:

- Thanks to the walls, a large part of the steel column is protected from heating;
- Because of the walls, the cross section is submitted to differential heating.

This differential heating will lead to the development of differential stresses fields in the cross-section that will have influence on the fire resistance of the element. The Eurocode 3 part 1.2 does not take into account this fact and the fire resistance is determined as if the heating was uniform.

The objectives of the tests were to study:

- The evolution of temperatures on elements embedded on walls;
- The influence of several parameters on the fire resistance of steel, as the stiffness of the surrounding structure to the column in fire and the slenderness of the element.

The experimental set-up is composed by a three-dimensional restraining frame where the column in test is inserted (Figure 4). This restraining frame has the objective to simulate different stiffnesses of the surrounding structure of the building to the column in fire.

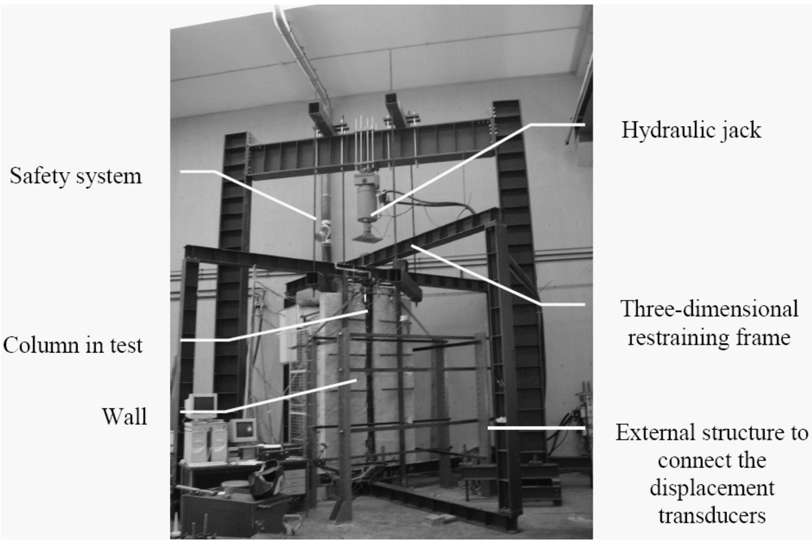

Figure 4. Test set up – Correia et al., 2007

The thermal action was only applied on one side of the element, to permit the analysis of the thermal gradient produced through the wall and across the cross section of the column. The column was subjected to the application of a constant vertical load. This load intended to simulate the serviceability load of a column when it is a part of a building structure.

Around 40 tests were planned for this study. Some specimens were tested with the strong axis parallel to the wall surface and other with that axis perpendicular to the wall surface. Two different column cross-sections and two thicknesses of the building walls were also tested.

The evolution of temperatures in different points of the cross-section of the elements obtained in experimental tests were compared with the ones obtained from calculations performed with the finite element program SUPER-TEMPCALC (STC), and with the simplified calculation method presented in EC3 part 1.2 for the determination of the temperature evolution in steel elements.

From the five first tests performed, it was found that the method of Eurocode 3 Part 1.2 and the calculations results from STC overestimate the temperatures in the cross section of steel columns embedded on walls in comparison with the experimental results, so they underestimate the real fire resistance of the element. Certain differences between results from the finite-element program STC and from experimental tests could be explained by the heat losses throughout the walls and the steel surfaces in contact with the air outside the furnace, which are not taken into account by the program STC.

The authors concluded that:

- The section factor defined in EC3 part 1.2 for the case of steel sections embedded on walls is not appropriate;

- For the cases with web parallel to the wall surface, the thicker wall plays a very important role in reducing the temperatures on the unexposed half of the flange and also in the web;
- For the cases with the web perpendicular to the wall surface:
  - The temperatures of the unexposed face of the flange were slight higher for the thicker wall,
  - The temperatures of the exposed face of the flange were much higher for the thinner wall.

I.2.2.5 Korzen et al., 2009 – Carvalho et al., 2009

Fire resistance tests on steel and composite steel and concrete columns subjected to axial restraints were also carried out at the Laboratory of Testing Materials and Structures of the Faculty of Sciences and Technology of University of Coimbra (FCTUC), Portugal (Carvalho et al., 2009; Korzen et al., 2009). These tests were performed in order to study the effects of the stiffness of the surrounding structure to the column in fire and the slenderness of the element.

The system of restraints of the FCTUC was described in the previous paragraph (§I.2.2.4 - Figure 4). A constant compressive load of 70% of the design value of buckling resistance of the column at room temperature was applied. Four stiffness ratios were studied (3, 13, 39 and 68 kN/mm).

The authors concluded from the tests that:

- The higher stiffness of the surrounding structure, the higher the restraining forces, the lesser the fire resistance;
- The lesser the slenderness, the higher the fire resistance.

The fire resistance tests realised on steel columns were simulated using the finite element program SAFIR (Carvalho et al., 2009). The temperatures considered in the numerical simulations were registered in the steel of the columns during the experimental tests. A uniform distribution of temperatures in the cross-sections was considered. The comparison of the tests and simulations results showed that the experimental tests are quite well described by the program SAFIR. Only some differences were observed in the restraining forces and in the axial displacements.

Some fire resistance tests on single steel columns with restrained thermal elongation were also performed at the Federal Institute for Materials Research and Testing (BAM), in Berlin, Germany, in order to compare with the ones carried out in Coimbra on single steel columns. The BAM system simulates the remaining building environment by a model on a computer (Figure 5). The forces and moments at the boundaries of the column specimens are measured and utilized for the computation of the corresponding displacements and angles, which are sent to the specimen (see *Rodrigues et al. 2002* in Annex).

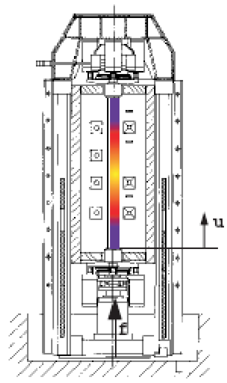

Figure 5. BAM system – Korzen et al., 2009

A comparison of results from the two laboratories, FCTUC and BAM, was presented by Korzen et al. (Korzen et al., 2009) and showed that the axial displacements experienced by the BAM specimens were in general higher than the ones of FCTUC specimens.

I.3 Theoretical research works

I.3.1 Review from the PhD Theses of Aasen (1985) and Rodrigues (2000) – Summary

Bauschinger, in 1887, recommended the Rankine-Gordon formula as basis for the design of fire exposed columns made of wrought iron and cast-iron. This formula has been improved later by Toh et al. (Toh et al., 2000) and then by Huang and Tan (Huang and Tan, 2003a, b).

Some numerical simulations to study the influence of the boundary conditions on the behaviour of heated steel columns were performed in 1972 and 1973 by ***Culver*** (Culver, 1972), ***Culver et al.*** (Culver et al., 1973) and ***Ossenbruggen et al.*** (Ossenbruggen et al., 1973). Various buckling curves were established depending on the following parameters: steel sections, different types of end conditions and thermal gradients either along the column or across the cross sections.

In 1981/82, ***Janss and Minne*** (Janss and Minne, 1981/82) described a simple design method for steel columns under concentric and eccentric loading in fire. The method adopted the European buckling curves for the design of steel elements at room temperature as the basic design curves for steel column in fire, modified to take into account the temperature effects on the steel properties.

Then, in 1982, the *Document Technique Unifié* (DTU) was published by the ***CTICM*** (Centre Technique Industriel de la Construction Métallique), presenting a calculation method for the prediction of the fire behaviour of steel structures (DTU, 1982). A new calculation process for the critical temperature of steel elements with restrained thermal elongation was described for the first time.

In 1992, ***Burgess et al.*** (Burgess et al., 1992) presented a complete numerical study on the influence of several parameters on the failure of steel columns in fire: the influence of slenderness, the effects of stress-strain relationships, the effect of residual stress levels, the influence of local buckling and the behaviour of blocked-in-web columns. Simulations were performed for pin-ended columns. Slenderness was identified as the parameter having the most influence on the critical temperature of steel columns, and columns with high and small slenderness behave better in fire situations than those with intermediate slenderness values.

In the same year, ***Jeyarupalingan and Virdi*** (Jeyarupalingan and Virdi, 1992) presented a new method for the analysis of steel beams and columns subjected to high temperatures. This method was implemented into the software SOSMEF. The resulting program allowed: the cross-section to vary along the length of the member, non-linear variations of temperature in the three dimensions, non-linear stress-strain temperature relationships and the use of more than one material in composite elements.

In 1995, ***Pow and Bennetts*** (Pow and Bennetts, 1995a-b) described a general numerical method to calculate the non-linear behaviour of load-bearing members under elevated temperature conditions. The method took into account, among other things, the combined actions of axial and biaxial bending, external restraints and temperature variation over the cross-section and along the member.

Also in 1995, ***Cabrita Neves*** (Neves, 1995) performed a theoretical study using the computer programme ZWAN to analyse the behaviour of steel columns with restrained thermal elongation. Neves introduced the idea that the fire resistance should correspond to the instant when the restraining forces reach the initial applied load again. He concluded that for columns with a centred load, the critical temperature, and consequently the fire resistance, decreases as the stiffness of the structure increases; fire resistance is highest in cases of null stiffness, no axial restraint. For columns with an eccentric load, the critical temperature is independent of the stiffness of the structure.

Then, in 1999, ***Valente and Neves*** (Valente and Neves, 1999) published a new work on this subject studying the influence of various parameters on the fire resistance of steel columns with axial and rotational restraints, using the finite element program FINEFIRE. When rotational restraint was considered, the restraining forces and the fire resistance were greater for higher grades of rotational restraint.

In 1997, ***Wang*** (Wang, 1997a-b) studied different aspects of the structural continuity, notably improved column rotational restraint and increased compressive load in column due to axial restraint to column thermal expansion. A parametric analytical study was performed using three different methods to investigate the effects of structural continuity on column critical temperature and fire protection thickness.

I.3.2 Recent developments

I.3.2.1 Shepherd et al., 1997 – Shepherd, 1999

Shepherd et al. studied, in 1997 (Shepherd et al., 1997; Shepherd, 1999), the effects of axial restraint on steel columns submitted to fire, using the non-linear finite element analysis program Vulcan developed at Sheffield University. A numerical analysis of the experimental fire resistance tests realised by Ali et al. at the University of Ulster (Ali et al., 1998) was performed, then extended to a parametric study. The series of tests included columns that were free to expand, as well as some which were subjected to various levels of axial restraint (see *Rodrigues et al., 2000* in annex).

The tested columns were only axially restrained during the expansion phase, then when a column began to be shortening than its original length, the test became totally unrestrained and the column's displacement went into run-away rather than allowing the restraint to stabilise the behaviour. However, in the numerical model, the restraint axial stiffness was modelled by a linear spring that continued to support the column when they become shorter than their original length. The rate at which the force is transferred from the column to the spring was determined by the stiffness of the spring.

Vulcan model showed good agreement with the test data before the columns have become shorter than their original length.

Then Shepherd (Shepherd, 1999) notably studied the effects of i) axial and rotational restraint stiffness, ii) the temperature distribution, and iii) the eccentricity of the axial load using the computer program Vulcan. He performed the following conclusions:

- If no axial restraints, failure occurs at a higher temperature since the axial force does not increase with temperature and failure is due to the reduction in material properties, specifically the tangent modulus.
- Compared to a pin-ended column, a rotational restrained column will have a less sudden and more progressively buckling failure due to the reduction of rotation of the ends of the column.
- The rotational restraints will be more beneficial for high slender columns failing by flexural buckling at low temperatures, as they are stiffer and moments can be transferred into the bearings if sufficient restraint is provided. Low stiffness columns will not be re-stabilised by rotational restraints.
- The level of rotational restraint does not affect the column behaviour during the initial heating phase. During this stage, the column does not undergo a great deal of buckling deflection and therefore the ends of the column will not attempt to rotate significantly.
- The behaviour of columns in fire is significantly affected by any thermal gradients introduced by the heating regime, be the lateral or longitudinal.
- The load eccentricity was simulated in Vulcan by dummy struts introduced on either end of the column. The effect of increasing load eccentricity is to reduce the maximum axial force supported by the column, and decrease the failure temperature.
- The lower the level of initial eccentricity, the more lateral deflection is seen after failure.

Shepherd (Shepherd, 1999) also developed a mathematical method of assessing the level of axial restraint. It was showed that the axial stiffness of columns above the fire compartment have an effect on the restraint stiffness to the heated columns. The presence of concrete floor-slabs has the effect of increasing axial restraint to a heated column, and the stiffness of beam-to-column connections dominates the level of imposed axial restraint. This method compared well with Vulcan analysis.

I.3.2.2 Toh et al., 2000, 2001, 2003

In 2000, ***Toh et al.*** (Toh et al., 2000, 2001) presented an analytical formula based on the Rankine principle to obtain a realistic estimate of the column fire resistance. The Rankine principle allows determining separately the strength and the stability of a steel column at a particular temperature (see equation (1)). The Rankine load $P_c(T)$ is given by the interaction between the strength and the stability of the columns:

$$\frac{1}{P_c(T)} = \frac{1}{P_p(T)} + \frac{1}{P_e(T)} \tag{1}$$

Where P_p is the rigid-plastic collapse load and P_e is the elastic buckling load.

The following assumptions were made:

- 3 different support conditions (pinned-pinned, pinned-fixed and fixed-fixed conditions);
- Temperature distribution uniform;
- No considerations of the influences of thermal gradients, initial out-of-straightness, residual stresses;
- Flexural buckling about the major or minor principal axis determined separately;
- Local and torsional buckling not considered;
- Finite deformations have no effects on the equilibrium state;
- No thermal restraints.

The new formula was found as being:

$$\frac{1}{\mu}\frac{P_c(T)}{P_y(T)} + \frac{P_c(T)}{P_e(T)} = 1 \tag{2}$$

Where P_y is the yield load in the absence of M(T) which is the maximum bending moment along the column length, P_e is the elastic buckling load at temperature T and μ is the reduction factor of the yield load at temperature T.

A good agreement was showed with both test results and other analytical formulas. It was also suggested that the influence of the initial out-of-straightness and residual stresses is not so significant and may be ignored under fire conditions.

In 2003, ***Toh et al.*** (Toh et al., 2003) completed the research work by two verifications, one using a finite element program, the other comparing with experimental fire resistance tests results from the literature. Good agreement showed that the proposed formula can be used as a quick tool to determine the fire resistance of steel columns.

I.3.2.3 Wang, 2004

Wang (Wang, 2004), in 2004, developed a simplified analytical method to consider the complex post-buckling behaviour of a column axially loaded with thermal expansion restrained. The effects of various factors on the column failure temperature were investigated, as the restraint stiffness during the column loading (expansion) and unloading (contraction) phases, column slenderness, and the initially applied column load ratio.

The complete behaviour of an axially restrained and axially loaded column is divided into three stages, the pre-buckling, the buckling and the post-buckling phase (Figure 6):

- Pre-buckling: the column is stable and its axial force is increasing due to the restrained thermal elongation;
- Buckling: when the column compressive load reached its buckling resistance, buckling occurs. The column undergoes a large lateral deflection, accompanied by contraction and a sharp reduction in its compressive load;
- Post-buckling: at increasing temperature, the column undergoes further lateral movement to accommodate the thermal expansion until an equilibrium position is found. Since the column bending moment caused by the lateral deflection is increasing, the column axial load has to reduce in order to maintain stability.

The simplified method described by Wang allows obtaining this entire load-temperature relationship. A uniform temperature is assumed in the column.

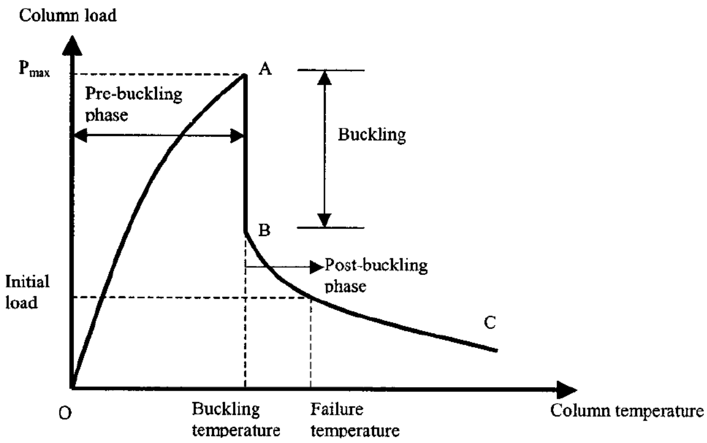

Figure 6. Complete load-temperature relationship of an axially restrained steel column – Wang, 2004

The simplified method was validated against the test results of Rodrigues (Rodrigues, 2000) and against the theoretical results of Franssen (Franssen, 2000). Good correlations were showed.

Wang then found that:

- The higher the restraint stiffness the lower the buckling temperature;
- The compressive resistance of a column after buckling may be higher than the initially applied column load;

- Generally, when the restraint stiffness is low and the applied column load is high, the effect of column postbuckling behaviour is small;
- With a combination of low initial load, high column slenderness and high restraint stiffness, the column failure temperature can be much higher than its buckling temperature.

He confirmed the idea about the column failure definition introduced by Neves (1995) and pointed out by Franssen (Franssen, 2000) as being the temperature at which the column compressive load in the postbuckling stage returns to its initial load. He showed that the column failure temperature can be much higher than its buckling temperature, which depends mainly on the column initial load, the column slenderness, and the restraint stiffness.

I.3.2.4 Huang, Tan et al., 2002, 2003, 2004, 2006

In 2002, ***Tan et al.*** (Tan et al., 2002) extended the FEM program FEMFAN (version 2), originally developed by Toh (Toh, 2000), in order to include elastoplastic behaviour, which considers unloading or cooling of structure, and creep effect. The analysis of steel plane frames showed excellent agreement at either room or elevated temperature, comparing to experimental and analytical benchmark tests. Factors affecting the structural behaviour at elevated temperatures were studied, such as the temperature-dependent material nonlinearity, the geometric nonlinearity, the thermal gradient, and the creep effect. It was found that:

- Creep effect generally starts to profoundly affect the structural response of heated columns beyond 400ºC. Thus, transient analysis is indispensable for steel structures at elevated temperature beyond 400℃;
- Under uniform heating, it was observed that for columns of intermediate slenderness ratio, the critical temperature is affected by the creep effect, and that creep has greater effect on stocky columns than on slender columns;
- Under non-uniform heating across the section, thermal bowing affects the critical temperature which is lower than for uniform heating case. Then, the creep has less effect on the critical temperature.
- The current simplified method for calculating the critical temperature in Eurocode 3 part 1.2 was shown to be unsafe for columns with intermediate slenderness ratio under both uniform heating and non-uniform heating.

In 2003, ***Huang and Tan*** (Huang and Tan, 2003a-b) published an extension of the Rankine equation in order to predict the fire resistance of an axially restrained steel column, the boundary restraints and the creep strain being incorporated. The column was assumed to be a pin-roller non-sway steel column with a linear spring attached at its top end to simulate the axial restraint, and a rotational spring to simulate the rotational restraints (Figure 7).

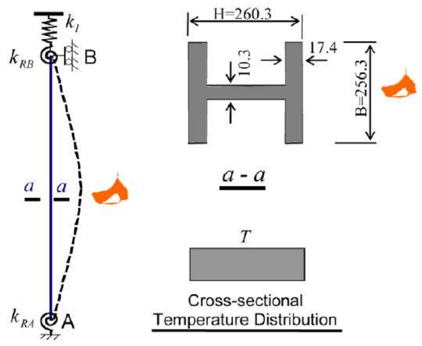

Figure 7. Axially and rotationally restrained column in fire – Huang and Tan, 2003b

It was assumed that a column attains the critical temperature when either one of the following two conditions are satisfied (Figure 8):

- For columns following curve a, as the Rankine formula only considers limited buckling conditions, the temperature associated with P^u is defined as the critical temperature. The term P^u denotes the maximum internal axial force developed during heating;
- Columns following curve c are considered to have buckled either when FEA fails to achieve a stable solution, or the decreasing rate of column internal axial force P exceeds $0.03P^{20}$/ºC (P^{20} is the initial value of internal axial force P).

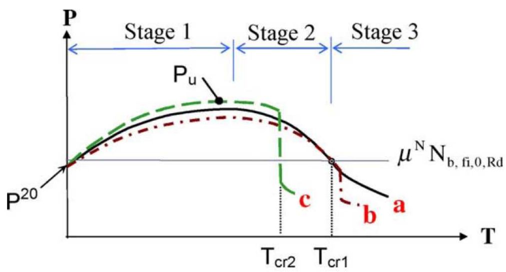

Figure 8. Development of column internal axial force – Huang and Tan, 2004

The proposed Rankine approach was verified experimentally and numerically, and results showed good predictions. The proposed Rankine formula, which served as a quick tool to ascertain the fire resistance of a column made of ASTM A36 steel, can be easily extended to other steel grades if corresponding creep models are available.

In 2004 and 2006, ***Huang et al.*** (Huang and Tan, 2004; Huang et al., 2006) wanted to shed light i) on the fundamental mechanism of a heated axially and rotationally restrained steel column subjected to both external axial load and bending moments, ii) on the interactions between the heated column and its surrounding cool structures, and iii) on the effect of different heating schemes on column critical temperature. They applied the FEM program FEMFAN2D for the analysis of the structural behaviour of a heated column in a compartment fire of a non-sway steel frame (Figure 9), in order to study:

- The axial restraint effect;
- The rotational restraint effect;

- The external axial load effect;
- The effect of two moments acting on opposite ends;
- The developments of column internal forces, cross-sectional stresses and strains;
- The effect of both rapid and slow heating schemes on column critical temperature;
- The column slenderness effect;
- The creep effect.

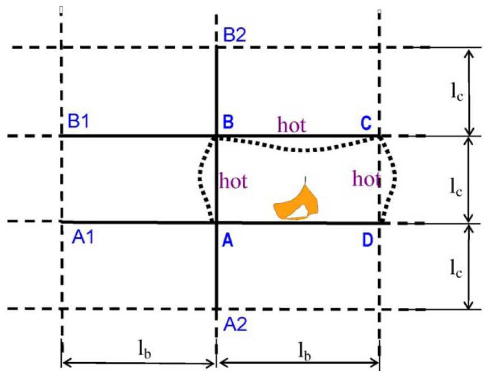

Figure 9. Compartment fire – Huang and Tan, 2004

Only bending behaviour about the strong axis and the cross-sectional uniform heating were investigated. The temperature along the column was assumed uniformly distributed.

It was showed that under the combined action of axial loads and bending moments, pin-roller column bends in a single curvature throughout the heating duration. However, rotationally restrained column bends in a single curvature at the beginning of heating, while at the later heating stage it bends in triple curvature, which retards the development of its lateral deformation. This is due to the restoring effect of friction, which retards lateral deformations. Then, external moments have limited effect on the critical temperature of a flexurally restrained column and can be neglected.

Figure 8 shows the development of the internal axial force. Curve ‘a’ shows a typical development of the axial force P. During the first stage of heating, P grows with temperature due to an increase in thermal restraint force. The growth rate will slow down towards the end of the first stage and subsequently P starts to decrease, due to the degradation of steel elastic modulus and the onset of column yielding.

Huang et al. concluded that:

- External moments influence the column response at elevated temperature in affecting the development of internal loads and in reducing the critical temperature for pin-roller columns;
- For rotationally restrained columns, external moments only affect the developments of internal forces at elevated temperatures and can be excluded from the calculation due to the restoring effect of rotational restraint;

- Even weak rotational restraints will substantially increase column critical temperature due to the restoring effect;
- The shorter the heating time the smaller is the creep strain;
- Creep generally starts to influence the structural behaviour of steel columns beyond 400ºC-450ºC, and should be included into numerical simulations;
- Axial restraint reduces the critical temperature;
- The critical temperature is defined as being either the buckling temperature (curve c, Figure 8), or the temperature at which the column compressive load in the postbuckling stage returns to its initial load (curve a, Figure 8). Provided that connections to adjoining structure remain intact, heated columns remain stable even if the internal axial force drops below the service load at ambient temperature;
- Slender columns with weak axial restraints exhibit snap-through failure;
- Effective lengths of rotationally restrained columns reduce at high temperatures. 0.55 can be taken for the structural fire design.

I.3.2.5 Tan and Yuan, 2007-2008-2009

Recently, ***Tan and Yuan*** (Yuan and Tan, 2007; Tan and Yuan, 2008, 2009) studied analytically the stability of a pin-ended steel column under non-uniform temperature distribution in the longitudinal direction. First, they studied the elastic buckling of restrained steel columns (Tan and Yuan, 2008), then they included bilinear material models in order to take account the effect of plasticity (Yuan and Tan, 2007; Tan and Yuan, 2009).

Temperature of columns in the longitudinal direction is not uniform but varies between top and bottom. This difference of temperature between extremities can be quite significant, particularly before flashover conditions, due to the upper hottest layer of air rising to the top, with a relatively cooler layer at the bottom. In absence of finite element analysis, the assumption of a uniform temperature in the longitudinal direction is usually used by engineers, which ascribe the hottest temperature at the column top to be the same throughout the column height. But the assumption about uniform temperature may cause inaccurate predictions. The objective of this work is to obtain closed-form solutions to enable engineers to quickly ascertain the column stability under preflashover fire conditions without recourse to time-consuming finite element modelling.

Elevated temperatures change the material properties of steel and induce additional compressive stress due to thermal restraints from the adjoining unheated structure. To simulate these restraints, two linear springs were attached to the column ends, then replaced by one equivalent spring (Figure 10). The temperature across a section was assumed to be uniform.

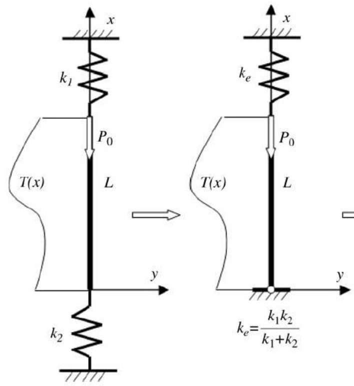

Figure 10. Column member under compressive load – Tan and Yuan, 2008

Tan and Yuan analysed two different idealizations of temperature distributions that represent more realistically the thermal response of a column: the linear and the step distributions. Analytical predictions showed good agreements with existing numerical results. Comparing these predictions with uniform temperature distribution based on the top column temperature showed that the buckling load or failure time is seriously underestimated.

The plasticity of the steel depends on the stress level instead of the temperature. Comparing the results based on elastic and inelastic assumptions, the effect caused by plasticity on the critical load is considerable, especially at high temperatures.

Because of a transient analysis (in order to consider the creep effect), results are not valid if the temperature rises very slowly.

I.4 Parameters influencing the behaviour of steel columns under fire conditions

This section tries to review the main parameters that affect the behaviour of steel columns at high temperature: i) the initial applied load, ii) the adjoining unheated structure that creates axial and rotational restraints, iii) the column slenderness, iv) the secondary effects and v) the creep effects. But first, a definition of the critical temperature of a steel column subjected to fire is given:

- The critical temperature is defined as being either the buckling temperature or the temperature at which the column compressive load in the postbuckling stage returns to its initial load (Huang and Tan, 2004).
- 95% of the columns tested by Knublauch in 1974 had a critical temperature above or equal to 500ºC (Knublauch et al., 1974).
- The experimental tests of Aribert and Randriantsara (1980, 1983 and 1984) showed that, at 600ºC, columns strength was significantly reduced.
- Recently, Yang et al. (Yang et al, 2006a) proposed to adopt a critical temperature of steel columns of 500ºC in order to ensure that the column strength keeps higher than 2/3 of the ambient temperature yield strength.

Axial load:

- The higher the initial axial load the lower the column failure temperature.
- The axial internal load is influenced by some of the following parameters.

Axial restraints:

- They are due to the connection of upper storey beams to the column itself.
- They prevent thermal expansion which generates additional axial force and reduces the failure temperature.
- Low slenderness steel columns axially restrained fail due to yielding of the cross-section and their failure temperature is governed by the material yield strength of the steel, while slender columns fail by flexural buckling and their failure temperatures are sensitive to initial geometrical imperfections (Ali et al., 1998).
- Initial axial load level affects the failure temperatures of columns of all slenderness (Shepherd, 1999).

Rotational restraints:

- They are provided by the beams framing into a column, as well as the column continuity to upper and lower storeys through splicing.
- They increase the critical temperatures under the same load.
- Aasen (Aasen, 1985) observed that the beam-to-column connections change the column behaviour, with a reduction of lateral deflections and a gradual type of buckling failure.
- Wang and Davies (Wang and Davies, 2003) found that i) the bending moment at heads of column decrease initially, then change direction, and ii) the final column effective length is not sensitive to the types of beam-to-column connection.
- More recently, Huang and Tan (Huang and Tan, 2007) described the restoring effect of friction, which retards the development of lateral deformation of a column rotationally restrained. Then, even weak rotational restraints will increase column critical temperature due to the restoring effect.

Slenderness of the steel column:

- An increase in slenderness causes a decrease in failure temperature (Shepherd et al., 1997).
- According to Yang et al. (Yang et al, 2006a), the slenderness ratio should be kept under 50 to prevent brittle failure under fire.

Secondary effects and creep effects:

- The initial out-of-straightness and accidental eccentricities of the columns diminish the fire resistance (Aasen, 1985 - Tan et al., 2007).
- The shorter the heating time the smaller is the creep strain.
- The creep generally starts to govern steel column behaviour beyond 400-450ºC (Huang and Tan, 2004).

II Complete list of references on the behaviour of steel columns in fire

II.1 Introduction

This chapter presents a complete list of references on the behaviour of steel columns in fire. It is divided into 2 subsections referring to experimental and theoretical research works. In each subsection, papers are classified by chronological order. The main author's name, the subject and the research date are first detailed; then relevant references to this research work are written.

II.2 Experimental Research

1) ***Bauschinger***, Fire resistance tests on metallic columns (cast iron, wrought iron and mild steel columns), 1885 and 1887.

2) ***Knublauch et al.***, 23 tests on steel columns protected by box insulation to determine the critical temperature, Germany, 1974.

   Knublauch E., Rudolphi R., Stanke J. "Theoretische Ermittlung der Feuerwiderstandsdauer von Stahlstützen und Vergleich mit Versuchen. 1.Teil: Berechnung der Stahltemperatur von Stahlstützen. 2.Teil: Bestimmung der Kristischen Stahltemperatur". Der Stahlau, H.6 und H.8, 1974.

3) ***Stanke***, 14 tests on steel columns with differing axial restraint grades, Germany, 1977.

   Stanke J. "Feuerwiderstandsdauer von Stahlstützen mit behinderter Längsdehnung". Bundesanstalt für Materialprüfung, (BAM), Berlin, 1977.

4) ***Minne, Vandamme and Janss***, 29 fire resistance tests on steel columns with different isolation types, Belgium, 1979, 1981, 1982.

   Janss J. and Minne R. "Buckling of Steel Columns in Fire Conditions". Fire Safety Journal, vol.4, England, 227-235, 1981/82.

   Minne R. and Vandamme M. "De Weerstand tegen Brand van Stalen Kolommen". Poefverslagen 3648-3659, Laboratorium voor Anwending der Brandstoffen en Warmte-Overdracht, R.U.G., Gent, 1979 (in Flemish).

   Vandamme M. and Janss J. "Buckling of Axially loaded Steel Columns in Fire Conditions". IABSE Proceedings, 43-81, 1981.

5) ***Olesen***, 24 fire resistance tests on steel columns without axial restraints, Denmark, 1980.

   Olesen F.B. "Brandforsog med centralt belastede stalsojler". Rapport nr 8003, Instituttet for Bygningsteknik, AUC, Aalborg, 1980 (in Danish).

6) ***Aribert and Randriantsara***, 33 fire resistance tests on steel columns to study the creep effect at high temperature, France, 1980, 1983, 1984.

Aribert J.M. and Randriantsara C. "Etude du flambement à des températures d'incendie - Action du fluage". Construction Métallique, No. 4, 1980.

Aribert J.M. and Randriantsara C. "Sur une modélisation du flambement des poteaux métalliques dans des conditions d'incendie". Third International Colloquium on Stability of Metal Structures, Preliminary Report, CTICM, Paris, 1983.

Randriantsara C. "Etude critique du flambement des poteaux métalliques dans les conditions d'incendie". Ph.D. Thesis, INSA, Rennes, 1984.

7) ***Hoffend***, Complete test program on steel columns subjected to high temperatures for a parametric study (slenderness, load level, buckling axis, load eccentricity, hinged and built-in column ends, thermal gradients along the column, steel cross-sections, rate of heating, axial restraint), Germany, 1977, 1980, 1983.

Hoffend F. "Brandverhalten von Stahlstützen bei ausmittiger Lasteinleitung, Dehnbehinderung oder teilweiser Bekleidung". Sonderforschungsbereich 148 - Brandverhalten von Bauteilen, Arbeitsbericht 1978-1980, Teil I, Technical University of Braunscheweig, Braunschweig, Germany, 29-59, 1980 (in German).

Hoffend F. "Zum Brandverhalten van Stahlbauteilen versuchsergebnisse und rechenerische Untersuchungen". Sonderforschungsbereich 148 - Brandverhalten von Bauteilen, Arbeitsbericht 1981-1983, Teil I, Technical University of Braunscheweig, Braunschweig, Germany, 57-80, 1983 (in German).

8) ***Cabrita Neves***, 2 fire resistance tests on steel columns with restrained thermal elongation, Germany, 1981.

Klingsch W. and Neves I.C. "Restraint Effects in Steel Frames by Local Fire". International Seminar on Steel and Composite Columns under Fire Action, Delft, The Netherlands, 1980.

Neves I.C. "The Column – Structure Interaction on the Behaviour of Steel Columns Under Local Fire, Ph.D. Thesis, Instituto Superior Técnico, Technical University of Lisbon, Lisbon, Portugal, 1982.

9) ***Aasen***, 18 fire resistance tests on steel columns with different load levels, column lengths and restraints, Norway, 1985.

Aasen B. "An Experimental Study on Steel Columns Behaviour at Elevated Temperatures". Ph. D. Thesis, Division of Steel Structures, Norwegian Institute of Technology, University of Trondheim, Trondheim, Norway, November 1985.

Poh K.W. and Bennetts I.D. "Comparison of Test and Calculated Behaviour of Steel Columns at Elevated Temperatures". Rep. BHPR/PPA/R/93/024/SG011A, BHP Research and New Technology, Melbourne Research Laboratories, Melbourne, Australia, November 1993.

10) ***FIRTO*** (Fire Insurers Research and Testing Organisation), Several fire resistance tests on steel columns (single columns, columns with blocks on the web and columns inserted in walls), England, 1980's.

Wainman D.E. and Kirby B.R. "Compendium of UK Standard Fire Test Data – Unprotected Structural Steel – 1" Ref. No. RS/RSC/S10328/1/87/B, British Steel Corporation, Swinden Laboratories, Rotherdam, UK 1988.

Wainman D.E. and Kirby B.R. "Compendium of UK Standard Fire Test Data – Unprotected Structural Steel – 2" Ref. No. RS/R/S1199/8/88/B, British Steel Corporation, Swinden Laboratories, Rotherdam, UK 1989.

11)***Franssen et al.***, Study of the instability of steel columns submitted to fire (pin-ended, unrestrained, symmetrically heated single steel column) with the objective to end up with analytical formulas for the instability of steel members centrally or eccentrically loaded, 1995 to 1998.

→ Experimental part: Two series of full-scale tests on steel columns at elevated temperatures under small or large eccentricities (21 fire resistance tests in Spain, at the LABEIN laboratory and 8 others in France, at the CTICM Fire Station).

Franssen J.M., Talamona D., Kruppa J., Cajot L.C. "Stability of Steel Columns in case of Fire: Experimental Evaluation". Journal of Structural Engineering, Vol. 124, No. 2, 1998.

→ Numerical part, using the finite element programs SAFIR and LENAS: Steel columns submitted to axial compressive forces, centrally or eccentrically loaded,

→ Analytical formulas for the buckling coefficient.

Franssen J.M., Schleich J.B., Cajot L.G. "A simple Model for the Fire Resistance of Axially-loaded Members – Members According to Eurocode 3". Journal of Constructional Steel Research, Vol. 35, 49-69, 1995.

Franssen J.M., Schleich J.B., Cajot L.G., Azpiazu W. "A simple Model for the Fire Resistance of Axially-loaded Members – Comparison with Experimental Results". Journal of Constructional Steel Research, Vol. 37, No. 3, 175-204, 1996.

Talamona D., Franssen J.M., Schleich J.B., Kruppa J. "Stability of Steel Columns in case of Fire: Numerical Modelling". Journal of Structural Engineering, Vol. 123, No. 6, 1997.

12)***Simms at al.***, Experimental study about changes in steel column axial loads due to axial restraints to the columns' thermal expansion, Ulster University, England, 1996.

→ 18 fire resistance tests on columns with one slenderness ratio, to investigate the effects of load level and axial restraint.

→ Simple analytical model of the restrained column condition.

Simms WI, O'Connor DJ, Ali F, Randall M. "An experimental investigation on the structural performance of steel columns subjected to elevated temperatures". Journal of Applied Fire Science 5(4), 269-84, 1995-96.

Simms W.I. "An experimental investigation of axially restrained steel columns in fire", Ph.D. Thesis, University of Ulster, 1997.

13)***Ali et al.***, The performance of axially and rotationally restrained steel columns during fire, Joint research between the University of Ulster and the University of Sheffield, England, 1998, 2001.

→ Ulster: 37 fire resistance tests on pin-ended steel columns that investigate three parameters: slenderness ratio, degree of axial restraint and loading ratio.

Ali F.A., Shepherd A., Randall M., Simms I.W., O'Connor D.J., Burgess I. "The Effect of Axial Restraint on the Fire Resistance of Steel Columns". Second World Conference on Constructional Steel Design, San Sebastian, Spain, Journal of Constructional Steel Research, Vol. 46, Nos. 1-3, paper no 177, 1998.

Randall M.J. "The effects of Axial Restraint on the Behaviour of Steel Columns in Fire". Ph. D. Thesis, University of Ulster, 1998.

→ Ulster: Parametric experimental investigation of 10 half-scale steel columns restrained axially and rotationally (2 parameters: degree of rotational restraint and the loading level imposed), and Method of estimating the effective length of fixed end columns (partial fixity).

Ali F. and O'Connor D. "Structural performance of rotationally restrained steel columns in fire". Fire Safety journal 36, 679-691, 2001.

Ali F., Nadjai A., Talamona D. "Effect of rotational restraint on performance of steel columns in fire". Journal of Applied Fire Science, vol. 13, Issue 1, 21-34, 2004.

→ Sheffield associated computational study: Numerical analysis of the tests using a non-linear finite element computer program named VULCAN that provided satisfactory accurate computer simulations of fire tests. The computer modelling was extended to parametric studies to include the effects of many more levels of axial restrain.

→ ***Shepherd et al.***, Parametrical numerical study of the effects of axial restraint on steel columns submitted to fire with the Vulcan program (load, geometrical imperfection, material properties, temperature profile, axial and rotational restraints), 1997.

Applicability of different method solution procedures to the analysis of columns (Arc length method introduced in Vulcan).

Shepherd P., Burgess I.W., Plank R.J., O'Connor D.J. "The performance in fire of restrained steel columns in multi-storey constructions". Fourth Kerensky International Conference, 333-42, 1997.

Shepherd P.G. "The Performance in Fire of Restrained Columns in Steel-Framed Construction", Ph.D. Thesis, University of Sheffield, 1999.

New tests realised by ***Tan et al.*** on the axial restraint effect on the fire resistance of steel columns, Singapore, 2004, 2007:

→ 19 I-section hot-rolled steel columns fire tested to determine the failure temperature time of unprotected columns with various axial restraint ratios, Focus on initial imperfections effects, 2007.

Tan K.H., Toh W.S., Phng G.H.E. "Fire Resistance of Steel Columns Subjected To Different Restraint Ratios". The Second International Conference on Steel & Composite Structures, Seoul, Korea, 625-635, September 2-4, 2004.

Tan K.H., Toh W.S., Huang Z.F., Phng G.H. "Structural responses of restrained steel columns at elevated temperatures. Part 1: Experiments". Engineering Structures 29, 1641-1652, 2007.

→ Numerical simulations of the tests using the finite element program FEMFAN, focus on accurate simulation of secondary effects arising from the experiments, namely, initial out-of-straightness and boundary friction

→ In addition, examination of the effects of load eccentricity, boundary friction and steel material models on column failure time

Huang Z.F., Tan K.H. "Structural response of restrained steel columns at elevated temperatures. Part 2: FE simulation with focus on experimental secondary effects". Engineering Structures 29, 2036-2047, 2007.

14) ***Rodrigues et al.***, Fire resistance of steel columns with restrained thermal elongation, IST, Portugal, 2000.

→ 190 tests on small pin-ended steel bars and 18 tests on small built-in steel bars with rectangular cross sections to study the critical temperature of compressed steel elements with restrained thermal elongation (4 slenderness, 2 eccentricities, 6 levels of restraint),

→ Simulations with the finite element programs ZWAN and FINEFIRE.

Rodrigues J.P.C. "Fire Resistance of Steel Columns with Restrained Thermal Elongation". Ph.D. Thesis, Instituto Superior Técnico, Technical University of Lisbon, Portugal, December 2000.

Rodrigues J.P.C., Neves I.C., Valente J.C. "Experimental research on the critical temperature of compressed steel elements with restrained thermal elongation". Fire Safety Journal 35, 77-98, 2000.

→ Model for predicting the critical temperature of restrained columns, and the reasons why the critical temperature can sometimes be lower than for columns free to elongate, 2002.

Neves I.C., Valente J.C., Rodrigues J.P.C. "Thermal restraint and fire resistance of columns". Fire Safety Journal 37, 753-771, 2002.

→ Comparison of the previous tests realised in 2000 on small pin-ended steel bars with real scale tests on axially compressed steel columns performed in the BAM column furnace, in Germany, where the remaining structure is numerically simulated by a computer model, 2006.

→ Numerical simulation of the BAM tests with the finite element program FineFire.

Korzen M., Neves I.C., Rodrigues J.P.C., Valente J.C. "Behaviour in fire of thermally restrained compressed steel members experimental and numerical approach". Proceedings of the fourth International Workshop "Structures in Fire", SiF'06, Aveiro, Portugal, 273-286, 2006.

15) ***Wang and Davies***, Fire resistance tests realised on non-sway loaded steel columns rotationally restrained to study i) changes in column bending moments due to variable column bending stiffness relative to the adjacent structure, and ii) how these changes could affect the columns critical temperature, UK, 2003.

Wang Y.C. and Davies J.M. "An experimental study of non-sway loaded and rotationally restrained steel column assemblies under fire conditions: analysis of test results and design calculations". Journal of Constructional Steel Research 59, 291-313, 2003a.

Wang Y.C. and Davies J.M. "Fire tests of non-sway loaded and rotationally restrained steel column assemblies". Journal of Constructional Steel Research 59, 359-383, 2003b.

→ Similar fire resistance tests on restrained concrete-filled columns realised by Kimura in 1990 (Kimura, 1990).

16) ***Kamikawa et al.***, Experimental tests to study the evolution of temperatures in steel columns (H sections and quadratic sections), 2002, 2006.

Kamikawa D., Hasemi Y., Wakamaty T., Kagiya K. "Experimental Flame Heat Transfer and Surface Temperature Correlations for a Steel Column Adjacent to and Surrouned by a pool fire". Proceedings of IFASS, 2002.

Kamikawa D., Hasemi Y., Wakamaty T., Kagiya K. "Prediction of the Thermal response of a metal column exposed to a localised fire". Proceedings of SiF'04, Third International Workshop Structures in Fire, 2004.

Kamikawa D. "Mechanical responses of a steel column exposed to a localised fire". SIF'06, Fourth International Workshop "Structures in Fire", Aveiro, Portugal, 244-253, 2006.

17) ***Yang et al.***, Study of the effects of width-thickness ratios, slenderness ratios, and residual stress on the behaviour of columns using steel and fire-resistant steel, 2005, 2006.

→ 24 Experimental tests on stub columns under uniform fire load using fire-resistant steel, 2005.

Yang K.C., Chen S.J., Lin C.C., Lee H.H. "Experimental study on local buckling of fire-resisting steel columns under fire load". Journal of Constructional Steel Research, vol. 61, 553-565, 2005.

→ Experimental fire resistance tests on steel and fire-resistant steel H columns, Steady-state method, 2006.

→ Analytical model able to predict the behaviour of fire-resistant steel H-columns under elevated temperatures and design guidelines for the design of fire-resistant steel columns in fire conditions.

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Yang K. C., Lee H. H., Chan O. "Experimental study of fire-resistant steel H columns at elevated temperature". Journal of Constructional Steel Research, vol. 62, 544-553, 2006b.

18) ***Correia et al.***, Study of the behaviour of steel columns embedded on walls, steel and composite steel and concrete columns, subjected to fire, with restrained thermal elongation, 2007-2009.

→ Fire resistance tests accomplished in steel columns embedded on masonry walls (FCTUC, Coimbra, Portugal)

→ Numerical analysis of the tests in steel columns embedded in walls using the finite element program SUPERTEMPCALC

Correia A.M., Rodrigues J.P.C., Silva V.P. "Studies on the fire behaviour of steel columns embedded on walls". 11th International Conference on Fire Science and Engineering – Interflam, London, 2007.

Correia A.M., Rodrigues J.P.C., Silva V.P. "Experimental research on the fire behaviour of steel columns embedded on walls". Application of Structural Fire Engineering, Prague, Czech Republic, 2009.

Silva V.P., Correia A.M., Rodrigues J.P. "Simulation on fire behavior of steel columns embedded on walls". XXXIII Jornadas Sudamericanas de Ingeniera Estructural, Santiago, Chile, 2008 (In Portuguese).

→ Fire resistance tests of steel and composite steel and concrete columns axially restrained at the Laboratory of Testing Materials and Structures of the Faculty of Sciences and Technology of University of Coimbra (FCTUC), Portugal, and other ones at the Federal Institute for Materials Research and Testing (BAM), in Berlin, Germany

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→ Numerical analysis of the tests of steel columns using the finite element program SAFIR

Carvalho L.T., Rodrigues J.P.C., Gonçalves M.C., Correia A.M. "Numerical simulation of steel columns in fire". Application of Structural Fire Engineering, Prague, Czech Republic, 2009.

II.3 Theoretical Research

1) ***Baushinger***, Use of the Rankine-Gordan formula as the basis for the design of wrought and cast iron columns exposed to fire, 1900.

Tang et al., Analytical formula to obtain a realistic estimate of column fire resistance under uniform temperature distribution, based on the Rankine principle, 2000, 2001.

Toh W.S. "Stability and strength of steel structures under thermal effects". Ph.D. Thesis, CSE, Nanyang Technological University, Singapore, 2000.

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Tang C.Y., Tan K.H., Ting S.K. "Basis and Application of a Simple Interaction Formula for Steel Columns under Fire Conditions". ASCE Journal of Structural Engineering, Vol. 127, No. 10, 1206-1213, October, 2001.

→ Verification study of the Rankine formula against a numerical model (the computer program FEMFAN) and a set of full-scale column test results (parameters: slenderness ratio, load eccentricity, steel grade, initial imperfections), 2003.

Toh W.S., Tan K.H., Fung T.C. "Rankine approach for steel columns in fire: numerical studies". Journal of Constructional Steel Research 59, 315-334, 2003.

→ Rankine Formula adapted for steel frames under fire.

Tang C.Y., Tan K.H., Ting S.K. "Basis and application of Simple Interaction Formula for Steel Frames under Fire Conditions". Journal of Structural Engineering, ASCE, Vol. 127, No. 10, October, 2001.

Toh W.S., Tan K.H., Fung T.C. "Strength and Stability of Steel Frames in Fire: Rankine Approach". Journal of Structural Engineering, Vol. 127, No. 4, April, 2001.

Huang and Tan, Extension of the Rankine equation to predict the fire resistance of an axially restrained steel column, boundary restraints and creep strain being incorporated, 2003.

Huang Z.F., Tan K.H. "Analytical Fire Resistance of Axially Restrained Steel Columns". ASCE Journal of Structural Engineering, Vol. 129, No. 11, 1531-1537, 2003a.

Huang Z.F., Tan K.H. "Rankine approach for fire resistance of axially-and-flexurally restrained steel columns". Journal of Constructional Steel Research 59, 1553-1571, 2003b.

2) ***Culver***, Numerical study of the influence of the boundary conditions on the behaviour of heated steel columns, 1972.

Culver C.G. "Steel Column Buckling Under Thermal Gradients". Journal of the Structural Division, American Society of Civil Engineers (ASCE), ST8, vol. 98, USA, 1853-1865, August 1972.

Culver, Ossenbruggen and Aggarwal, Two more works on the Culver topic: study of the buckling behaviour of pin-ended steel columns and of the behaviour of pin-ended steel columns with non-uniform temperature distribution,1973.

→ Analytical method used to calculate the ultimate load of steel columns.

Culver C.G., Aggarwal V. and Ossenbruggen P. "Buckling of Steel Columns at Elevated Temperatures". Journal of the Structural Division, American Society of Civil Engineers (ASCE), ST4, vol.99, USA, 715-726, April 1973.

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3) ***Janss and Minne***, Simple design method for steel columns under concentric and eccentric loading in fire conditions, 1981.

Janss J. and Minne R. "Buckling of Steel Columns in Fire Conditions". Fire Safety Journal, vol.4, England, 227-235, 1981/82.

4) ***CTICM***, Centre Technique Industriel de la Construction Métallique, Calculation method for the prediction of the fire behaviour of steel structures, 1976, 1982, 1983.

Document Technique Unifié (DTU). "Méthode de prévision par le calcul du comportement au feu des structures en acier". Construction Métallique No.3, France, 39-79, 1982 (in French).

5) ***Burgess et al.***, Complete numerical study on the influence of several parameters on the failure of steel columns in case of fire (slenderness,

stress-strain relationships, residual stress levels, local buckling, blocked-in-web columns), 1992.

Burgess I.W., Olawale A.O., Plank R.J. "Failure of Steel Columns in Fire". Fire Safety Journal, vol. 18, England, 183-201, 1992.

Olawale A.O. and Plank R.J. "The Collapse Analysis of Steel Columns in Fire Using a Finite Strip Method". International Journal for Numerical Methods in Engineering, vol. 26, England, 2755-2764, 1988.

→ Tests on this type of column realised by British Steel (Wainman and Kirby, 1988, 1989)

6) ***Jeyarupalingan and Virdi***, Numerical method for the analysis of steel beams and columns subjected to high temperatures, The finite element program SOSMEF is developed using this new method, 1992.

Jeyarupalingan N. and Virdi K.S. "Steel Beams and Columns Exposed to Fire Hazard". Conference Structural Design for Hazardous Loads, London, 429-438, 1992.

7) ***Pow and Bennetts***, Numerical simulations of the Aasen tests, General numerical method to calculate the non-linear behaviour of load-bearing members under elevated temperature conditions, 1995.

Poh K.W. and Bennetts I.D. "Analysis of Structural Members Under Elevated Temperature Conditions". Journal of Structural Engineering, American Society of Civil Engineers (ASCE), No. 4, vol. 121, USA, 664-675, April 1995a.

Poh K.W. and Bennetts I.D. "Behaviour of Steel Columns at Elevated Temperatures". Journal of Structural Engineering, American Society of Civil Engineers (ASCE), No. 4, vol. 121, USA, pp. 676-684, April 1995b.

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Neves I.C. "The Critical Temperature of Steel Columns with Restrained Thermal Elongation". Fire Safety Journal, vol. 24, England, 211-227, 1995.

Valente and Neves, Numerical study of the influence of various parameters on the fire resistance of steel columns with axial and rotational restraints (eccentricity of the load, slenderness of the column, stiffness of the surrounding structure and the grade of rotational restraint on the nodes of the column), FINEFIRE, 1999.

Valente J.C. and Neves I.C. "Fire Resistance of Steel Columns with Elastically Restrained Axial Elongation and Bending". Journal of Constructional steel Research, vol. 52, England, 319-331, 1999.

9) ***Wang et al.***, The effects of frame continuity on the behaviour of steel columns under fire conditions, 1994-2004.

→ Numerical parametric studies to investigate various aspects of steel frame behaviour and to quantify the effects of frame continuity on the behaviour of steel columns, 1994, 1995, 1997.

Wang Y.C. and Moore D.B. "The effect of frame continuity on the critical temperature of steel columns". Proc., 3rd Kenrensky Conf. on Global Trends in Structural Engineering, Singapore, 681-686, 1994a.

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Wang Y.C., Lennon T., Moore D.B. "The Behaviour of Steel Frames Subject to Fire". Journal of Constructional Steel Research, vol. 35, pp. 291-322, 1995.

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Comparisons with experimental results of João Paulo Rodrigues (Rodrigues, 2000) and numerical results of Franssen (Franssen, 2000).

Wang Y.C. "Postbuckling Behavior of Axially Restrained and Axially Loaded Steel Columns under Fire Conditions". Journal of Structural Engineering, ASCE, March 2004.

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FIRE RESISTANCE TESTS ON STEEL COLUMNS – A SUMMARY REVIEW

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Abstract
The traditional way of measuring the fire resistance of a structural member is to subject an identical sample to a standard fire resistance test. The resultant fire resistance is defined as the time in minutes that the member (or assembly) is able to withstand the standard fire before a specific condition of failure is reached.

A lot of tests on structural members have been performed worldwide. Columns with and without loading and axial restraining have been tested. In this paper, a general summary of the main fire resistance tests carried out on steel columns, in the last 30 years, is presented.

1. Fire Resistance Tests on Steel Columns

The fire resistance of steel columns has been largely studied by means of experimental tests. Tests with and without loading and axial restraining have been performed worldwide.

Most of the fire resistance tests conducted on steel columns have been carried out in the last thirty years. The major part was carried out in Europe, Table 1. Those tests used large furnaces where the columns were heated by means of gas or naphtha combustion, or electricity (fig. 1). To provide a means of comparison between the different tests, a standard heating process was established, the ISO 834 fire curve [1].

Knublauch et al. [2], in 1974, reported a series of 23 tests on steel columns protected by box insulation made of vermiculite plates. The tests were conducted at BAM - Bundesanstalt für Materialforschung und -prüfung, Berlin, Germany. The columns were heated with a large furnace. Part of the length of the columns was inside the furnace but another part was outside. The applied load was constant for all the tests and was calculated according to DIN 4114. Thermal elongation was not prevented. Columns with different types of cross-section and the same length were tested. The main conclusion of the tests was that 95% of the columns had a critical temperature above or equal to 500 ºC.

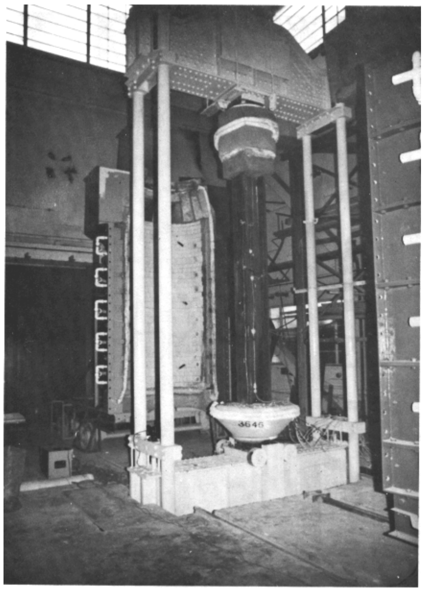

Fig. 1. Column furnace fire test facility (FIRTO, Borehamwood, England) [26]

In 1977, **Stanke** [2] described 14 tests carried out at the same Laboratory on steel columns with differing axial restraint grades. In these tests, only measured the loads and temperatures were measured. The main findings from the tests were that, during the initial heating, the load increased rapidly due to the axial restraint to thermal elongation. After reaching the maximum restraint load, the columns "collapsed" and the load started to diminish gradually. This behaviour is typical of columns with restrained thermal elongation. For the first time, it was observed that steel columns could withstand the initial loading for longer than initially predicted.

Minne, Vandamme and Janss [2, 3], in 1979, 1981 and 1982 reported a series of 29 fire resistance tests on steel columns. The tests were conducted at University of Ghent in Belgium. A total of 27 tests with different types of insulation and two tests without insulation were realised. These tests were an improvement on the earlier ones in that the whole length of the column was inside the furnace. The columns had slenderness ratios between 25 and 102. The loading was kept constant during the tests and no axial restraint was imposed. The columns were clamped in special end fixtures, with the aim of providing a perfect rotational restraint at both ends.

Olesen [2], in 1980, reported the results of a series of 24 fire resistance tests on steel columns carried out at the University of Aalborg in Denmark. Hinged columns with different lengths and without axial restraint were tested. The columns were placed in the horizontal position with the weak axis vertical, relative to the buckling occurring in the horizontal plane. In 18 tests, the columns were heated up to certain temperature levels and then subjected to a constant loading rate until buckling. In the remaining 6 tests, the columns were tested with constant loading during the heating process, until buckling occurred. Outside the furnace, the columns were connected to a frame.

Aribert and Randriantsara [2], in 1980, 1983 and 1984, reported a series of fire resistance tests on steel columns carried out at the University of Rennes in France. The tests were designed to study the creep effect at high temperatures on steel columns. Thirty-three tests were conducted on non-insulated pin-ended columns with the same length and cross-section, placed in the vertical position. The columns were heated throughout their length by means of an electric furnace. Some of the columns, with and without end moment at the top, were heated up to certain temperature levels and were then loaded until buckling. Other columns, with and without axial restraint, were first loaded and then heated until buckling. The tests showed that creep started to influence the strength of columns at 450 ºC. At 600 ºC columns strength was significantly reduced.

Hoffend [4, 5] reported a complete test program on steel columns subjected to high temperatures, in 1977, 1980 and 1983. The tests were carried out at iBMB - Institut für Baustoffe Massivbau und Brandschutz, Technical University of Braunschweig in Germany. A lot of tests on steel columns subjected to high temperatures were performed. The parameters studied were: the slenderness of the column, load level, buckling axis, load eccentricity, hinged and built-in column ends, thermal gradients along the column, various steel cross-sections, rate of heating and the degree of axial restraint.

Some conclusions drawn from the tests were:

- The critical temperature was slightly higher for slender than for stocky columns.
- The load level was more important for the less slender columns owing to the tendency to inelastic buckling.
- The critical temperature for weak axis buckling was lower than that for strong axis buckling.
- The load eccentricity had a greater effect on diminishing the critical temperature of slender columns than it did for stocky columns. This was more pronounced for buckling around the weak axis.
- The thermal gradients in height had a minor effect on the strength of hinged columns.

Table 1. European fire resistance tests on steel columns

Laboratory	Researchers/dates of publication of results	Tests
BAM – Bundesanstalt für Materialforschung und -prüfung, Berlin, Germany	Knublauch et al (1974) Stanke (1977)	23 14
iBMB – Institut für Baustoffe Massivbau und Brandschutz, Technical University of Braunschweig, Germany	Hoffend (1977) (1980) (1983)	8 32 35
University of Ghent, Belgium	Minne and Vandamme (1979) Vandamme and Janss (1981) Janss and Minne (1981/82)	29
Instituttet for Bygningsteknik, University of Aalborg, Denmark	Olesen (1980)	24
iBMB – Institut für Baustoffe Massivbau und Brandschutz, Technical University of Braunschweig, Germany	Cabrita Neves (1981)	2
INSA – Instituts Nationaux des Sciences Appliquées University of Rennes, France	Aribert and Randriantsara (1980) (1983) Randriantsara (1984)	33
Norwegian Institute of Technology, University of Trondheim, Norway	Aasen (1984)	20
FIRTO - Fire Insurers Research and Testing Organisation Borehamwood, England	(1980/85)	11
University of Ulster- England	Ali (1998) (2001)	37

In 1981, at the same Laboratory, ***Cabrita Neves*** [6, 7] performed two fire resistance tests on steel columns with restrained thermal elongation. A frame located outside the furnace created the thermal restraint. Two values of axial restraint were studied; 33 and 44.5 kN/mm. Axial displacements, lateral deflections and restraining forces were measured in the columns tested. The evolution of the restraining forces showed the typical behaviour of increasing up to a maximum, then decreasing to values under the initial load. The same behaviour was observed for the axial displacements. With regard to the lateral deflections, these started to increase slowly followed by a sudden drop after column failure.

Aasen [2], in 1985, reported the results of a series of 18 fire resistance tests on steel columns carried out at the Norwegian Institute of Technology. Twelve tests were conducted on pin-ended columns with free thermal elongation, four tests on columns with end moment restraint and free thermal elongation, and, finally, two tests on pin-ended columns with axial restraint. This series was complemented by two compression tests on columns at room temperature, one with rotational restraint.

The columns were made of rolled I-section IPE 160. Four column lengths were tested, 3.1, 2.21, 1.75 and 1.7 m, which corresponded to slenderness ratios about the weak axis of 169, 120, 95 and 92. The load was applied centrally except in two tests on pin-ended columns with free thermal elongation where it had an eccentricity of 14 and 20 mm. Different load levels were used.

The rotational restraint in the tests, where envisaged, was achieved by connecting the columns ends to restraining beams through web-cleats or end-plates. The axial restraint in other tests was effected by means of a restraining frame.

For the fire resistance tests with free elongation and rotational restraint beyond the temperatures and the fire resistance times, the values of the axial and lateral deformations at mid-height of the column were measured. In general, axial deformation increased with temperature, followed by a decrease after column buckling. Lateral deformation increased very slowly with temperature until column buckling occurred, after which it increased rapidly.

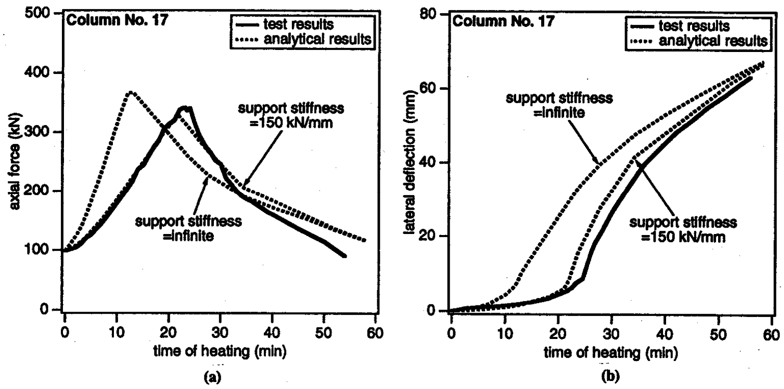

Fig. 2. Comparison of calculated and measured behaviour for Aasen's test column 17:
(a) axial force; (b) lateral deflection [8]

The restraining forces were also measured in the tests with restrained thermal elongation, beyond the parameters measured on previous tests. The typical behaviour for this kind of test was again observed. The restraining forces increased up to a maximum and then decreased to values under the initial load. The axial deformations showed the same behaviour. Lateral deflection increased slowly until the maximum restraining forces were reached and then increased very rapidly with the decrease of restraining forces. This behaviour is the same as that observed in the earlier tests carried out by Cabrita

Neves. These tests were later simulated numerically by Pow and Bennetts and the results published in 1995 (fig. 2.) [8].

The numerical simulations showed some deviation from the test results in the case of infinite stiffness of the restraining frame. It was subsequently concluded that the restraining frame had lower stiffness. This stiffness was not measured in the tests. A value of 150 kN/mm for the stiffness of the restraining frame showed a better agreement between the test results and the numerical simulations.

The main conclusions of these tests were outlined by Aasen in 1985 in a report from the Norwegian Institute of Technology, University of Trondheim, Norway [2].

For unrestrained columns it was concluded that:

- High load levels diminished the fire resistance of the columns.
- The initial out-of-straightness and accidental eccentricities of the columns diminished the fire resistance.
- The slenderness of the columns and the heating rate affected slightly the strength of the columns.

For rotationally restrained columns it was concluded that:

- The beam-to-column connections change the column behaviour, with a reduction of lateral deflections and a gradual type of buckling failure. The end-plated connections performed better than the web-cleat connections.
- The columns with intermediate slenderness ratios showed a flexural - torsional buckling mode of failure.

For axially restrained columns it was concluded that:

- For the higher applied load level tested, the maximum restraining forces were reached earlier and the fire resistance was lower.
- The initial geometrical imperfections of the columns changed the shape of the restraining forces curves and the lateral deflections curves as a function of the temperature. Columns with imperfections showed rounder curves with a smooth change between the loaded and the unloaded phases.

In Borehamwood (England), at ***FIRTO - Fire Insurers Research and Testing Organisation*** [9, 10], several fire resistance tests were performed on steel columns in the 1980s. Single columns without protection, columns with blocks on the web and columns inserted in walls were tested. The columns were tested with constant loading that was a percentage of the design value, calculated according to BS (British Standard) 449:Part 2: 1969. Single columns with and without blocks on the web were exposed on all four sides, while columns inserted in walls were exposed only on one side. Some results of the tests are reported, such as the temperatures attained in the steel profiles and the displacements as a function of time.

In 1998, ***Ali*** et al. [11] published a study on the effect of axial restraint on the fire resistance of steel columns, carried out at the University of Ulster in England. This study incorporated 37 fire resistance tests on pin-ended steel columns. A new testing system was developed to perform the tests. Axial restraint and loads could be applied to the test columns separately or at the same time.

The tests were conducted on pin-ended 1.8 m long columns. The tested sections were UK 152 x 152 x 23 UC, 178 x 102 x 19 UB and 127 x 76 x 13 UB steel profiles, with slenderness ratios λ of 49, 75 and 98. Four load levels (α_L = 0, 0.2, 0.4 and 0.6) and three axial restraint levels (α_K= 0, 0.1 and 0.2) were tested. The load level α_L was defined as a percentage of the BS 5950: 1989 ultimate load of the tested column. The axial restraint level α_K was defined by a relation between the structure stiffness, K_s and the column axial stiffness K_c.

$$\alpha_K = \frac{K_s}{K_c} \qquad K_c = \frac{AE_s}{L} \tag{1}$$

where:

A- column cross-section area
E_s- Young's modulus at room temperature
L- length of the column

The results of the tests showed the typical behaviour of steel columns when axial restraint is imposed. As an example, figure 3 shows the results of the restraining forces, axial displacements and lateral displacements measured in a 127 x 76 x 13 UB column. The restraining forces rise relatively linearly up to a maximum, followed by a sudden decrease until a certain value. This sudden drop in the restraining forces was associated with an abrupt instability of the column. The equilibrium was restored before the initial applied load was reached. The same behaviour was observed for axial displacements. The lateral displacements showed a large increase after the instability of the column.

The tests were also analysed using a non-linear finite element computer programme named VULCAN. This programme is able to analyse composite steel and concrete frames with semi-rigid connections at elevated temperatures. The results of the numerical simulations for the 127 x 76 x 13 UB column are also presented in figure 3. A lack of fit, due to the bedding-in of column bearings, was observed in the test. The restraining forces did not immediately increase with temperature from the start of the test. This behaviour was also modelled with the computer programme (fig. 3).

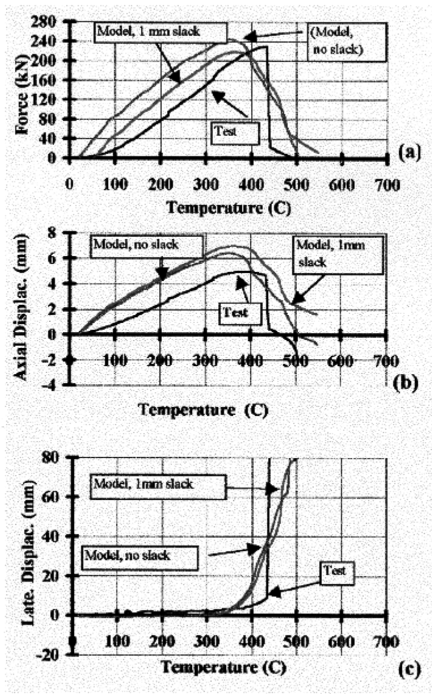

Fig. 3. Test results, UK 127 x 76 x 13 UB column, λ= 98, α_K= 0.2, α_L= 0.2 [11]

It was concluded in this work that:

- The failure temperature decreased and the restraining forces increased when the degree of axial restraint was increased.
- Increasing the load level reduced the maximum restraining force.

- Higher values for the restraining forces were measured for the more stocky columns. The failure of these columns was more gradual than it was for the slender columns. Some of the slender columns exhibited sudden instability.

In 2001, ***Ali*** et al. [12] reported a study of 10 tests on the effect of rotational restraint on the fire resistance of steel columns, carried out at the University of Ulster in England. The previous testing system, used to test pin-ended columns, was modified for these tests in order to impose rotational restraint in conjunction with axial restraint.

The tests were conducted on rotational end-restrained 1.8 m long columns. The section tested was the UK 127 x 76 x 13 UB steel profile. Five load levels (α_L = 0, 0.2, 0.4, 0.6 and 0.8) and one axial restraint level (α_K= 0.29) were tested. As in the previous tests, the load level α_L was defined as a percentage of the BS 5950: part 1 - 1989 ultimate load of the tested column. The axial restraint level α_K was defined by a relation between the structure stiffness, K_s and the column axial stiffness K_c. The degree of rotational restraint ρ imposed on a column was defined as the ratio of the rotational stiffness of the column ρ_c and the structure ρ_s

$$\rho = \frac{\rho_s}{\rho_s + \rho_c} \qquad (2)$$

When $\rho_s >> \rho_c$, then $\rho \approx 1 \rightarrow$ the column end is fully fixed
When $\rho_s \approx 0$, then $\rho \approx 0 \rightarrow$ the column is pin-ended.

Two values of rotational restraint were considered in the experimental tests, a low value of ρ= 0.186 and a high value of ρ= 0.936.

The main conclusions drawn in this work were:

- The rotational restraint had a minor effect on the generated restraint forces but critical temperatures were greatly increased under the same load.
- The rotationally restrained columns usually do not show a sudden drop in the generated restraint force as in the axially restrained columns.
- Increasing the load level caused a significant drop in the failure temperature.

In 2000, ***João Paulo Rodrigues*** [13, 14] presented a systematic study of the influence of certain parameters on the fire resistance of compressed steel elements with restrained thermal elongation. A large number of fire resistance tests on restrained small steel elements was carried out at the Instituto Superior Técnico (IST) in Lisbon (fig. 4). The parameters studied were: the slenderness of the element, the stiffness of the surrounding structure and the eccentricity of the loading. Nearly 190 tests were performed on small pin-ended steel bars and 18 tests on small built-in steel bars. For the pin-ended bars, four different slenderness values (319, 199, 133 and 80), two eccentricities of the compression load (1mm and another equal to

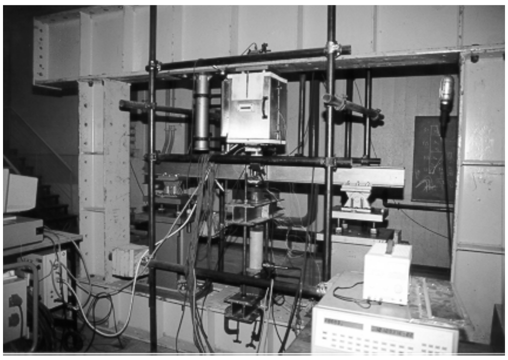

Fig. 4 Experimental model – IST tests [13, 14]

the thickness of the bar) and six values of restraint to axial elongation (0, 1, 10, 24, 42 and 98 kN/mm) were tested. For the built-in bars, two slenderness values (80 and 32) and three values of restraint to axial elongation (1, 10 and 98 kN/mm) were tested.

Figures 5 and 6 show two exemples of the evolution of the measured restraining forces as a function of the maximum temperature in the bars. The evolution of the restraining forces for the bar slenderness values of 80 with centred and eccentric loading is given. In each graph, one curve for each value of stiffness of the surrounding structure, ranging from zero to 98 kN/mm, is presented. The restraining forces P are related to the initial value P_0. The initial load P_0 was 70% of the design value of the buckling load at room temperature, calculated according to Eurocode 3 – Part 1 [15].

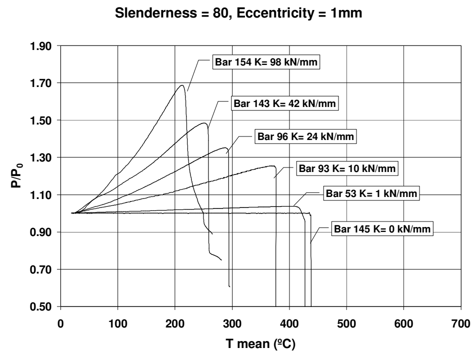

Fig. 5 – Evolution of the restraining force - centred load

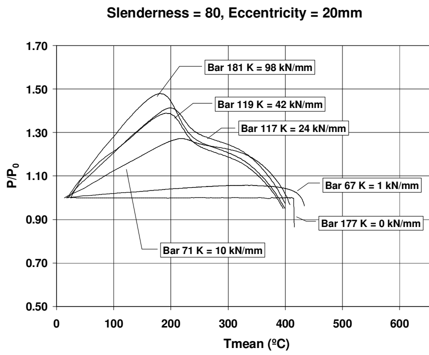

Fig. 6 - Evolution of the restraining forces - eccentric load

In Figure 5, for the bars with slenderness 80 and centred loading, it can be observed that when the stiffness of the stiffness-beam increases, the curves intersect the line P/P_0=1 for successively lower values of the maximum steel temperature. The restraining forces also increase with increasing stiffness. For a stiffness of 98 and 42 kN/mm, a tendency to intersect the line P/P_0=1 at the same temperature could be observed.

Figure 5 also shows that the restraining forces increase as a function of the temperature up to a maximum, followed by a sharp decay. This sharp decay was related to a sudden instability of the bars.

In Figure 6 a slight tendency for the critical temperature to be lower when the stiffness increases is also observed, although this is not so clear as in the case of centred loading. In spite of that, it was concluded, in general, the critical temperature of the bar is not influenced by the stiffness.

The eccentric loading is characterised by gentle increasing and decreasing of the restraining forces, as seen in the curves presented.

2. Recent developments

In real situations, steel columns are incorporated into building structures. This fact has inspired several researchers to develop new testing techniques where the restraint imposed by the structure surrounding the steel columns is taken into account. In the early 1990s, at ***iBMB in Braunschweig and BAM in***

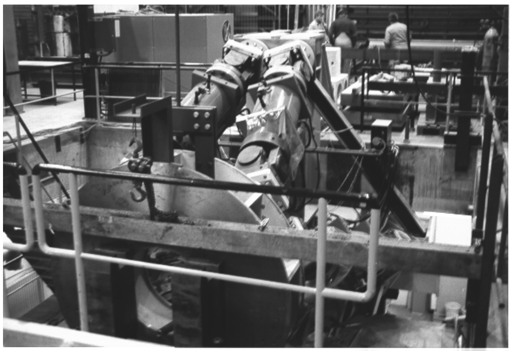

Fig. 7. SALM - Several Axis Loading Machine (iBMB, Braunschweig, Germany)

Berlin, both in Germany, some researchers started to develop new testing techniques to study the problem (figs.7 and 8) [16, 17].

The systems developed in Braunschweig and Berlin are identical, with a difference on the position of the element in testing. In the system of Braunschweig the element is placed in the horizontal position while in the system of Berlin the element is placed in the vertical position. The technique used on these systems, makes it possible to have only one element of the building structure inside the furnace, while computers simulate the behaviour of the remaining structural system. An interface node interconnects the element inside the furnace with the simulated system. The movements of the interface node are realised by several hydraulic jacks, while the computers are solving, in an iterative process, the equations of the equilibrium of forces and displacements between the element in the furnace and the simulated system.

The system has been developed for future use with standard and natural fire tests, and structural systems composed of all relevant construction materials. It may be used to study the behaviour of steel columns in a fire situation as a part of a building structure. Thermal restraint and subsequent restraining forces can be studied with this technique.

Some problems have been found with respect to the capacity of the computers to perform the calculations, accompanying the development of the test inside the furnace. This technique is still under development at iBMB.

In 1993, a real 8-storey steel-framed building was built in the Large Building Test Facility at ***BRE - Building Research Establishment***, in Cardington, England, for testing in different conditions (fig. 9). This building is part of a major EC (European Community) research project that involves two more test buildings: a 6-storey timber-framed building, and a 7-storey in situ concrete-framed building.

The steel framed building, representing a multi-occupancy office block, is 45 m long, 21 m wide and 33.5 m high. This building provided the opportunity to investigate the behaviour of Europe’s main construction materials and improve our understanding of how real buildings behave under different environmental and accidental loads. The results are intended to contribute to the validation of theoretical models and to calibrate the recommendations established in the European Standards.

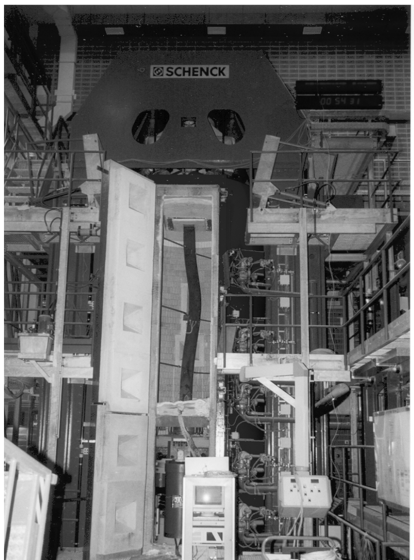

Fig. 8. “Substructuring in Fire Engineering”, BAM, Berlin, Germany

Fig. 9 Eight-storey steel–framed building (BRE, Cardington, England)

A lot of tests have been performed on this building, including seismic, explosion and fire tests. The fire tests were conducted on steel beams, columns and frames. Fire tests were carried out in different compartments with different fire loads. The steel columns were tested as though in real fires, because the axial and rotational restraint could be considered. Sand bags were used to simulate the service load of the building.

In 2001, **João Paulo Rodrigues** carried out two tests on two different columns of the ground floor of this building. One of the tested columns was in the middle and the other was on the border of the building structure. In the first test the loading was centred and in the second one was eccentric. The results of these tests will be compared with the results of tests carried out by the author at IST in Lisbon on restrained small steel elements and with others carried out at BAM in Berlin and will be published as soon as possible.

The columns were heated by enclosing them in a portable gas-fire furnace, while the remaining building structure was kept cold. Sand bags placed on the different floors simulated the serviceability load of the building. In the tests, the temperatures on the steel columns and in other points of the surrounding structure, as well as the restraining forces and displacements, were measured.

3. Conclusions

Traditionally, the fire resistance of load-bearing members has been assessed by means of standard fire tests with constant load. These tests do not describe particularly well the behaviour of load-bearing members in building structures subjected to real fires. There are differences arising from the applied loads, restraint conditions and fire exposure characteristics. Analytical methods are nowadays more used for determining the behaviour of structures in a fire [18, 19]. The high cost of carrying out real fire tests on full-scale structures justifies the use of computational or analytical methods. The results of the world wide experimental work performed in the last decades represent a valuable database for the calibration of the computer tools that have been developed more recently and that will certainly constitute in the future the main option for assessing the fire resistance of steel columns.

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