Multi-storey steel framed buildings under natural fire conditions

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In this article, a methodology is presented by which the structural behaviour of composite steel framed building under natural fire conditions can be analysed. As a first step, a fire model is employed in order to predict the temperature development in the fire compartment. Secondly, a thermal response model is used to calculate the temperature distribution and development in the various structural elements. Finally, by means of a mechanical response model and given the thermal response, the structural performance of the full structure is predicted.
This methodology is demonstrated by means of two case studies on two existing high-rise buildings in The Netherlands.

Key words: Fire resistance, natural fire conditions, composite steel frame structures

1 Introduction

Traditionally, the fire resistance of load-bearing structures is assessed by considering the behaviour of single structural components, rather than the composite behaviour of a complete structure. In the traditional approach, the elements which are considered to be critical are isolated from the whole structure. The fire resistance is assessed on the basis of the behaviour under fire of this isolated element. This assessment could either be based on an existing design rule. Or, when such design rule is not available, the isolated element is taken to a furnace for fire testing. In order to perform the fire test, the single element is incorporated in a test rig and thus subjected to a certain set of boundary conditions. Also, a certain schematised loading is applied. By this approach, the composite behaviour of a whole construction is not taken into account properly. In a complete structure, the boundary conditions of a single element is influenced by the behaviour of the surrounding structure, also the loading on the single element can be variable depending on the deformation of the structure.

Also in the single structural component approach, strongly schematised, standard fire conditions are taken into account. Whereas in reality, fire conditions can be quite different from case to case. Depending on various parameters, such as the amount of flammable material and the ventilation in the fire compartment etc., the fire development in a compartment can differ significantly, both in the time and the temperature regime. This is not taken into account in the standard approach for fire resistance testing.
The Cardington Demonstration project has shown that, under well monitored, fully developed fire conditions, the floors and steel beams of composite steel framed buildings may remain unprotected, without failure of the building structure [1]. In order to use these results for practical design purposes and the conditions under which such conclusions hold were specified more precisely in a subsequent research project. This research project is referred to as the "Cardington (2) project". As a results of this project, operational design guidance with regard to the structural behaviour of multi-storey composite steel framed buildings under natural fire conditions were obtained. The approach developed in the Cardington (2) project is presented in this article.
For a complete analysis of the structural behaviour of composite steel framed building under natural fire conditions, the following models are needed:

- a fire model, by which the temperature development in the fire compartment is predicted as function of the various parameters involved (dimension & lay out of the fire compartment, fire load density, ventilation conditions etc.);
- a thermal response model, by which the temperature distribution and development in the various structural elements (beams, columns, slabs) is predicted, given the thermal loading;
- a mechanical response model, by which the structural performance (deflections, deformations, moment distribution etc.) is predicted, given the thermal response.

These models are further explained in Chapter 3. However, in order to complete the history of this research program, a brief summary of the Cardington Demonstration project is presented first, see Chapter 2. Finally, in Chapter 4, the presented methodology is illustrated with two case studies, which are taken from two existing high-rise building located in The Netherlands.

2 Full-scale fire tests

In the past various full-scale fire tests have been performed. The fire tests which were used in order to calibrate the present modelling are referred to as the Cardington Demonstration project [1].
Within this Demonstration project, a total of 6 full-scale fire tests were carried out within the eight-storey steel framed structure located within the BRE Large Building Test Facility at Cardington, Bedfordshire, United Kingdom. See Figure 1 for a picture of this structure.
The fire tests, which were performed can be summarised as follows:

• Test 1 – restrained beam	unprotected beam, heated over 8 meter of its 9 meter length at a controlled speed of approx. 3 – 10 °C per minute.
• Test 2 – plane frame	unprotected beams and protected columns over the entire width of the building, heating was controlled by a furnace.
• Test 3 – 1st corner	area of the fire compartment was 76 m^2, fire load 45 kg of wood per square meter.
• Test 4 – 2nd corner	area of the fire compartment was 54 m^2, fire load 40 kg of wood per square meter.
• Test 5 – large compartment	area of the fire compartment was 340 m^2, fire load 40 kg of wood per square meter. This compartment was located between rows A – C and rows 1 – 4, see Figure 1.
• Test 6 – simulated office	area of the fire compartment was 135 m^2, containing "normal" office materials, fire load equivalent of 46 kg of wood per square meter. This compartment was located between rows D – F and rows 3 – 4, see Figure 1.

In Figure 1 a typical floor plan of the building is shown in which the locations of the various fire tests are noted.

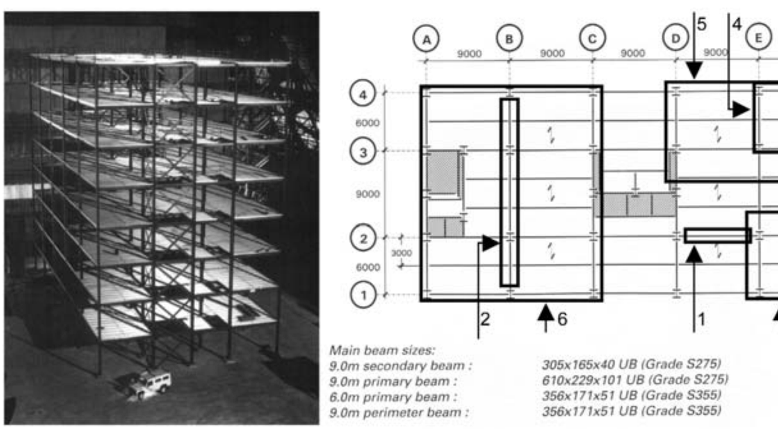

Figure 1. Photo of the steel framed structure at Cardington (left) and floor plan with locations of various fire tests (right)

The most important finding of the demonstration project was that the eight-storey steel framed structure in Cardington, in which the fire tests were performed, possessed a very significant degree of inherent fire resistance even although the steel floor beams remained entirely unprotected against fire attack.

3 Modelling

3.1 ***Fire model***

The fire model utilised for the analyses presented in chapter 4 is composed of a one-zone and a two-zone model as well as a model to switch from the two-zone to the one-zone model [2]. Within the main model, various sub-models are adopted which enable the evaluation of:

- The heat and mass transfer between the inside of the compartment and the ambient external environment through vertical and horizontal openings and boundaries and forced vents (vent model).
- The heat and mass produced by the fire (combustion model).
- The mass transfer from the lower to the upper layer by the fire plume (air entrainment model).

Figure 2 shows a schematic view of the two-zone model and its sub-models for heat and mass transfer. In the two-zone model, the compartment is divided in an upper and a lower layer. In each layers the gas properties (temperature, density, etc.) are assumed to be uniform. The pressure is assumed to be constant throughout the whole compartment volume, except when it is evaluated that mass exchanges through vents.

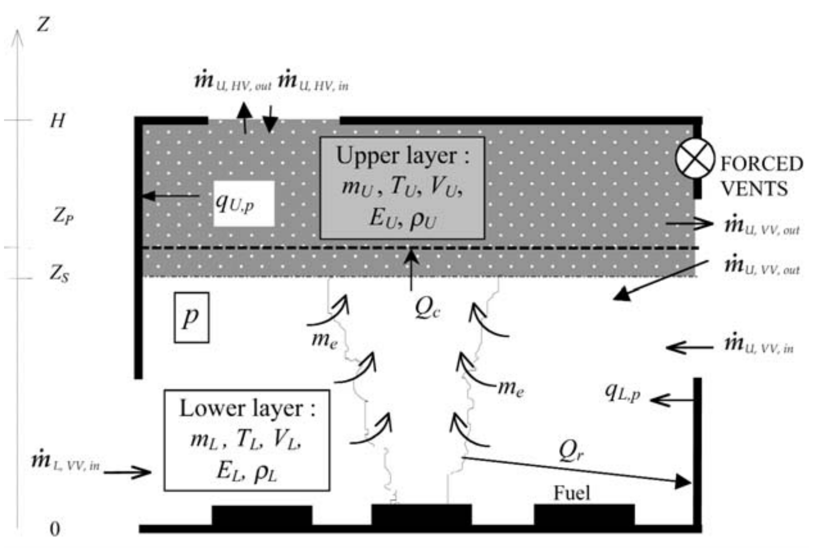

Figure 2. Schematic view of two-zone model and associated sub-models

Some switch criteria are defined so that they represent a limit beyond which one-zone model assumptions becomes closer to the physics of the fire situation than the two-zone model. The switch is made so that the total energy and mass present in the two-zone model system at time of switch are fully conserved in the one-zone model system.

Figure 3 shows a schematic view of the one-zone model and its sub models for heat and mass transfer. In the one-zone model, a single zone represents the compartment. In this zone the temperature and density are assumed to be uniform. The pressure is assumed to be constant on the whole compartment volume (except while evaluating mass exchange through vents). The zone is supposed to be opaque. Radiative and convective heat transfers connect partitions to it.

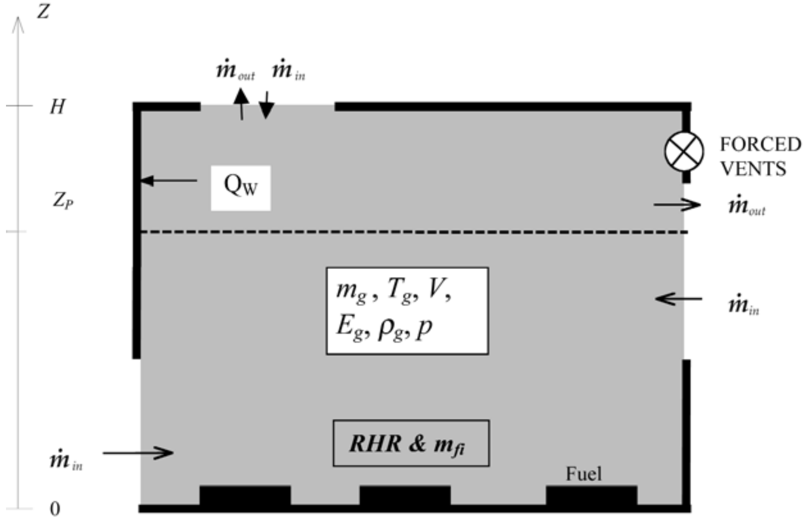

Figure 3. Schematic view of one-zone model and associated sub-models

3.2 ***Thermal response***

From the fire model, based on various input parameters, the temperatures in the fire compartment are predicted. From these predicted air temperatures, the structural response will be assessed. This is carried out on the following steps. Firstly, the temperature distribution in the structural members are calculated (thermal response). Secondly, based on the calculated temperatures of the structural members, and given the mechanical loading, the mechanical behaviour of the structure is calculated (mechanical response).

The thermal analysis is performed while the structure is exposed to fire. For a complex structure, the sub-structuring technique is used, where the total structure is divided into several sub-structures and a temperature calculation is performed successively for each of the substructures. The thermal analysis is made using 2-D solid elements, to be used later on cross sections of the finite elements which will be used in the analysis of the mechanical response. For the calculation of temperatures in beams, the temperature is non-uniform in the sections of the beam, but there is no heat transfer along the axis of the beams. For shells, the temperature is non-uniform through the thickness of the shell, but there is no heat transfer in the plane of the shell.

3.3 *Mechanical response*

For the analyses presented in Chapter 4, the finite element model DIANA is employed [3]. DIANA is a general-purpose three-dimensional finite element programme, suitable for the simulation of the thermal and structural response of structures including physical and geometrical non linear behaviour, dynamic effects and time and temperature dependent problems.

The physical non-linear behaviour of steel has been modelled with a Von Mises yield contour including hardening according to Eurocode 4 [4]. Concrete stress-strain behaviour has been modelled with a Drucker-Prager yield contour for compression including hardening.

After evaluation of the effect of the inclusion of cracking, it appeared that the effect on the response was minimal while the effect on the numerical stability was detrimental. Therefore, in the analyses, no additional cracking criterion was applied. Later on, in the newest version of DIANA (8.1), the numerical instability was overcome and cracking could be applied, see the case study of the *Delftse Poort* in Chapter 4.

In the modelling as presented, steel members such as beams and columns and ribs of steel-concrete composite slabs are being modelled with numerically integrated curved beam elements. These beam elements can be subdivided into zones to describe the actual cross sectional shape. Over each zone a time dependent temperature and temperature gradient are prescribed. The slabs have been modelled with numerically integrated curved shell elements, also provided with temperatures and temperature gradients. In both beam and shell elements, embedded reinforcement was placed where appropriate.

If two structural elements are connected to the same node, all degrees of freedom, i.e. the displacements and rotations, are compatible. If appropriate, the joints between structural elements have been modelled as hinges or with different nodes that were tied only for the required degrees of freedom. Figure 4 shows the adopted finite element mesh, which is used to model a typical section of the composite steel/concrete floor. The distribution of the various elements to model this section of the composite floor is primarily chosen with the aim to obtain a proper temperature distribution of the modelled structure.

The simulations have been carried out in an incremental-iterative way. First the mechanical load has been applied. Hereafter, time has been increased incrementally. In each time step, the temperatures increase according to the results of the thermal response analyses. The temperature increase results in thermal expansion and degradation of the mechanical properties. Within each time or load step, the equilibrium has been searched in an iterative way using a secant stiffness approach. Within each iteration, the strain decomposition in each element is also carried out in an iterative way.

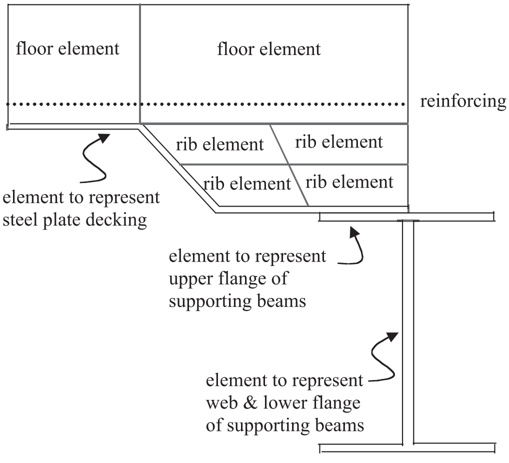

Figure. 4 Schematic view of finite element mesh of a typical section of the composite steel-concrete floor

4 Examples

4.1 ***Case: Rembrandt Tower Amsterdam***

4.1.1 Introduction

As a first case, the Rembrandt Tower in Amsterdam, see Figure 5, was analysed using the tools described in the previous paragraphs [5] and [6]. This high-rise office building has a height of 135 m. It consists of a steel frame structure with steel columns in the faced braced by a square concrete core in which the vertical transport systems are incorporated. The floors are made of composite decks using steel sheets supported on steel beams. A typical storey is 3.4 m high and each storey consists of one fire compartment.

Dutch regulations require an equivalent fire safety level in for building beyond 70 m as for buildings lower than 70 m. However, no method is prescribed to assess the safety level. Therefore, one has to use general fire safety engineering principles and tools to meet the requirements. From fire safety engineering it is known that several aspects need special consideration in high rise buildings:

- The smoke movement is affected by the chimney effect, i.e. a natural draught exists in the building which can hamper the smoke spread control.
- Fire spread to other compartments can not be stopped by the fire brigade from the outside of the building. The fire brigade can only attack the fire through the building.
- High hydrostatic pressure differences exist in the suppression systems
- A complete evacuation of the building is takes more time than for a normal building if it is possible at all.

The consequences of an eventual collapse with respect to the urban environment are far greater than for a low rise building.

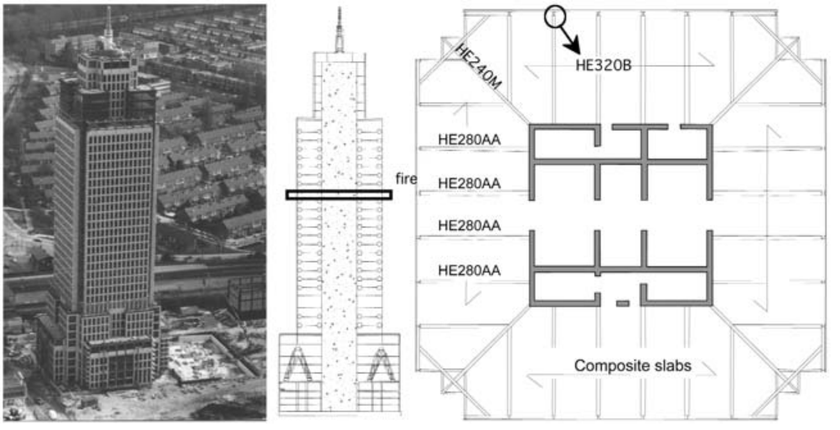

Figure 5. Photo of the Rembrandt tower office building in Amsterdam (left), cross section of the structural system with the fire exposed storey (middle) and floor plan of the structural system (right)

4.1.2 Approach

A finite element model of the structural system of the tower was developed with the computer code DIANA, in which a fully developed fire was assumed in one fire compartment. In order to achieve a consistency of crudeness between the model for the fire development and the structural model, the standard fire was replaced by a simulation of the fire development of a typical fire compartment in the building with the computer programme Ozone [3], i.e. the model described in chapter 3.

The size of the braced steel columns reduces towards the top because of the lower loads. A simple fire analyses of the columns at each storey, based on the standard fire exposure, showed that the columns at the 21st storey were most critical, see Figure 5. Therefore, this storey was modelled. In order to optimise the design, the FE model was used for three design scenario's:

1. the insulation of each member according to a standard design analyses of the single element with the initial applied load and the natural fire exposure.
2. no insulation on the beams and 20 mm of Promatect-H on the columns.
3. without any insulation on both the steel columns and the steel beams but partition walls dividing one storey into four fire compartments.

4.1.3 Fire development

The fire development was modelled with Ozone. Since, most office spaces in the tower are furnished without partition walls, one big compartment was modelled of 32.4 x 32.4 m excluding the central core of 14.4 x 14.4 m. Ozone assumes one-dimensional heat conduction through the boundaries. The actual thermal properties of the concrete core, the composite floors and the sandwich construction of the façade (steel sheet – mineral wool – granite) were modelled with nominal values for concrete, steel and mineral wool as given by the programme. The characteristic value with 80 % reliability of the fire load density in standard offices was used of 593 MJ/m².

A big uncertainty is the ventilation resulting from the breaking of the windows. A small parameter study showed that the effect of the assumptions for the breaking of the windows on the temperature of the steel members in the compartment is relatively small. The results based on the assumption that all windows break directly at the start of the fire were finally used as input for the finite element model.

4.1.4 Thermal response models

Separate finite element models were made for the determination of the time dependent and non-uniform temperature distribution of cross sections of the steel concrete composite slab, the regular HE280AA beams and the heavy corner beams HE240M. The temperature of the columns was obtained by Ozone, as a uniform temperature distribution could be assumed for these columns which were exposed from all four sides.

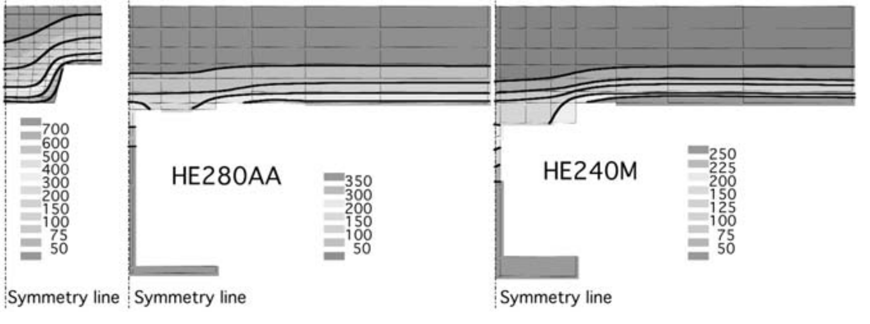

Figure 6. Cross sectional models for the determination of the thermal response of the steel concrete composite deck after 75 min. of fire exposure (left), the bare regular beams after 50 min. (middle) and the bare corner beams also after 50 min. (right)

4.1.5 Structural response model

The entire floor of the 21st level was modelled including the columns. For the scenario in which the fire compartment was reduced to ¼ of the floor area of that storey, one additional floor was modelled on top of the fire exposed construction representing the rest of the building in order to simulate the capability of the higher floors to redistribute the vertical loads to the unexposed columns.

At the bottom side and top side, the columns were modelled with clamped supports, the top side allowing for vertical displacements. At the top side of the columns, the momentary part of the vertical loads of the rest of the building were applied, considering partial safety factors equal to unity, which is in accordance with the recommended values in national and international codes for the fire situation.

The beams, columns and reinforced ribs of the composite slabs were modelled with numerically integrated beam elements based on the Mindlin-Reissner theory. The reinforced concrete deck was modelled curved shell elements. The steel sheet was modelled as reinforcement, considering a separate temperature development for the lower flange, the web and the upper flange. The non- linear temperature distribution in the ribs and the deck obtained with the thermal response models were simplified to linear temperature distributions over the beam and shell elements. For that purpose the average temperature was taken equal to the average temperature over the symmetry line of the thermal response models and the thermal gradient was derived such that the temperature of the reinforcement in the structural model equalled the temperature of the node in the thermal response model at the location of rebar.
From the structural models it was learnt that the deflections during fire were primarily driven by the thermal deformations rather than the applied loads. In design scenario 1 and 2, no failure occurred during the entire fire duration. In the cooling phase, the deflections reduced. In design scenario 3, the columns collapsed after 38 minutes. It appeared that the cold construction above the fire compartment was not capable to redistribute the loads sufficiently.

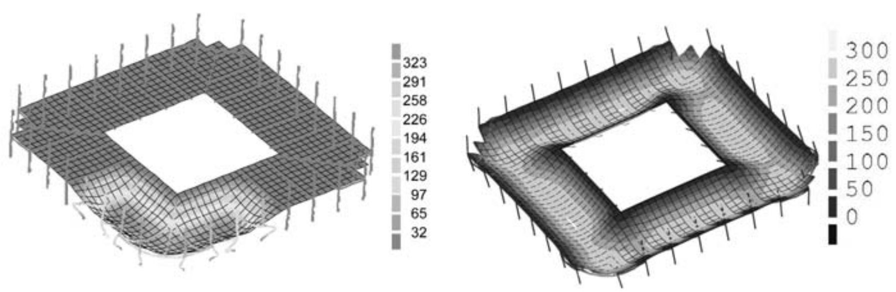

Figure 7. Collapse of the columns after 38 minutes in the scenario with partition walls dividing the floor plan into 4 fire compartments without any insulation on the steel columns and the steel beams (left) and displacements after 95 minutes without for the scenario with no insulation of the beams but 20 mm Promatect-H on the columns (right)

4.2 ***Case study Delftse Poort***

The building named *Delftse Poort* (1991) is currently still the highest building (150 m.) of The Netherlands and located close to Rotterdam Central Station. The building comprises four parts constructed of pre-cast concrete. In this case, the fire resistance with regard to collapse in case of a fire at the 25th storey of building part 1, see Figure 8, was studied. The fire compartment that is modelled is in use as an office area. Although the building is equipped with a sprinkler system, in this case study it is assumed that the sprinkler system malfunctions [7].

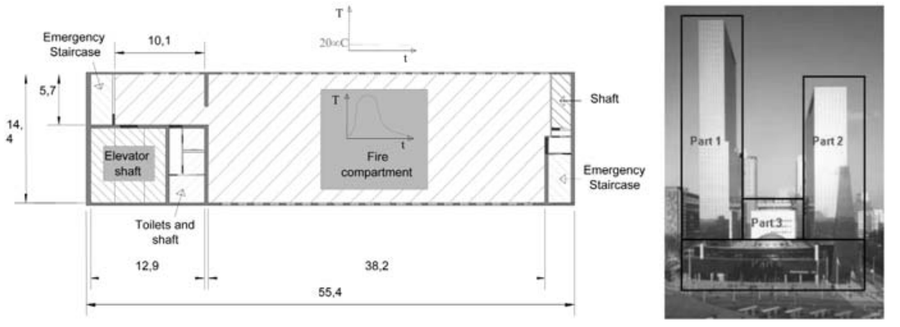

Figure 8. Floor plan of 25_{th} storey. Left: Structural core with e.g. the elevator shaft. Middle: Office area constructed with prefabricated concrete elements. Right: Shaft and emergency staircase

4.3 ***Results***

4.3.1 Natural Fire Model

Once again, the breaking of the window has an important effect on the development of the fire. However, opposite to a steel framed structure, it was not possible to predict in advance would be most severe. Therefore, three scenario's were evaluated, i.e.

1. Fast fuel controlled scenario
2. Slow oxygen controlled scenario
3. Intermediate scenario

The first two scenarios describe the outer borders of the spectrum of possible time-temperature curves. The third scenario is a best guess based on engineering judgement.

4.4 ***Thermal response model***

The fire compartment is not fully taken into account in the system approach. Only that part of the fire compartment that is build up of prefabricated concrete elements is taken into account in the thermal response and mechanical FEM models. The uniform time temperature curves calculated by use of Ozone are applied as a boundary condition to three different thermal response models representing respectively the wall of the 25th storey except for the column, the column of the 25th storey and the floor elements of the 26th storey (see Figure 9). The floor of the 25th storey is considered "cold".

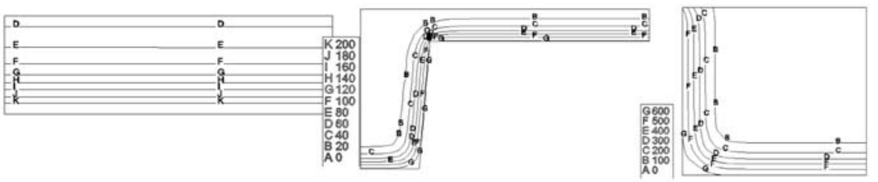

Figure 9. From left to right: wall model – slow scenario, floor model – intermediate scenario and column model – fast scenario, all taken at times at which the maximum air temperature is reached

4.5 ***Mechanical FEM model***

In figure 10 the finite element mesh is presented together with the boundary conditions in a cross-section. The curved shell elements at the bottom and the top of the model represent together with the constraints at the bottom and the top of the model, the structure below and above the model. At the left side of the model, where it meets the structural core of the building, the translation in Z-direction is fixed.

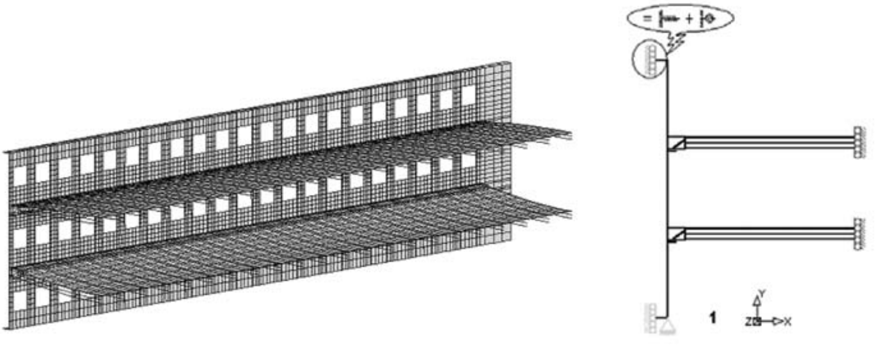

Figure 10. Final mechanical FEM model (left) and boundary condition (right)

When the structure is heated up by the fire the floor expands in both directions. In global X-direction this expansion is more restrained compared to the global Z-direction. This restraint leads to a distributed load on the wall in global X-direction of approximately 350 kN per repetitive structural element of 1.8m’.

Furthermore the upper part of the wall, the column and the lower part of the wall of the 25th storey are heated up by both an average temperature and a temperature gradient. This average temperature has little influence on the mechanical behaviour because the structure just moves upwards. The gradient however leads curvature that is partially restrained by its surrounding structural parts. Consequently a change in the bending moments results.

It is found, that the top part of the column of the 25th storey is a weak part of the structure in case of fire due to a relatively small moment of inertia. Furthermore the distributed load on the wall caused by the restrained expansion of the floor leads to an extra “negative” bending moment at the top of the column. The relatively cold rear side of the column cracks horizontally and the concrete at the hot front side of the column crushes.

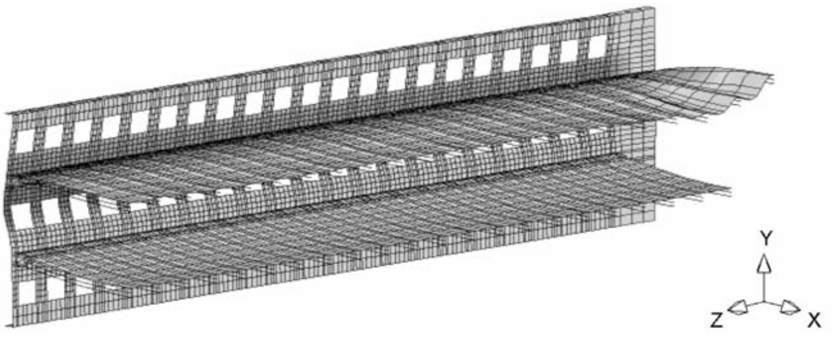

Figure 11. Thirty times enlarged deformation in case of slow –scenario with soft interface configuration

5 Conclusions

A methodology has been developed by which the structural behaviour of composite steel framed building under natural fire conditions can be analysed. As the first step in this methodology, a fire model has been employed in order to predict the temperature development in the fire compartment. Secondly, a thermal response model was used to calculate the temperature distribution and development in the various structural elements. Finally, by means of a mechanical response model and given the thermal response, the structural performance of the full structure can be predicted. This methodology was successfully validated against full scale natural fire tests in the Cardington building in the UK. The practical relevance was demonstrated by means of two case studies on existing high-rise buildings in The Netherlands. The first case study focussed on the building named Rembrandt Tower in Amsterdam (135 m height). The second case study dealt with the building named Delftse Poort (1991, 150 m. height) located close to Rotterdam Central Station.

These two case studies provide a crucial understanding of the failure modes, which could develop in case of fire in those buildings. Using the methodology leads to an improved structural design. In this way the methodology can contribute to the safety level of vulnerable high rise buildings.

Acknowledgement
The authors wish to thank ECSC for the financial support for the research projects into the Natural Fire Safety Concept as well as both Cardington projects.

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