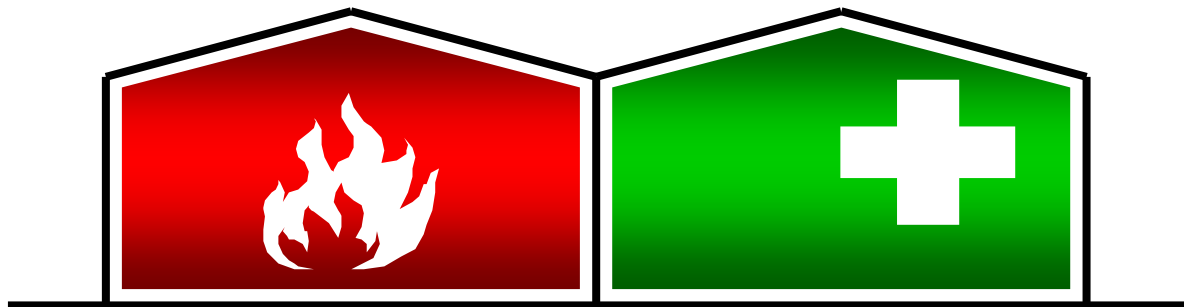


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RFS2-CR-2007-00032



Fire Safety of Industrial Hall

# Background Document





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## 1. Introduction

Since years, the fire resistance is one of the main hindrances to the development of the steel construction in multi-storey buildings. The new fire engineering methods issued from various recent research projects have shown that it is possible to obtain fire safe steel structure without passive fire protection.

Between 1983 and 1990, many research works have been dedicated to optimise the behaviour of steel or composite structure subjected to thermal loads similar to the ones of tests in laboratories, i.e. the standard fire curve also called ISO curve. Owing to these research works, the steel structural elements can be assessed with a full range of tools from tabulated data up to sophisticated tools based on Finite Element Method while the fire itself was defined by only one curve as function of time.

More recently, the research works are focused mainly on the study of steel structural behaviour under natural fire development since in this case, the temperature field is not homogenous inside the compartment and highly depends on different parameters such as fire loads, compartment boundaries and its ventilation condition. Moreover, the structural analysis is more and more considered in the scope of global behaviour rather than single member performance. With this type of approach, the analysis permits a much better understanding of what will really occur during a fire as far as steel structures of buildings are concerned because it provides the fire behaviour much closer to reality. In consequence, the outcome of all above works have brought the fire safety engineering of steel structures to a new era during which different advanced calculation tools are combined together to predict the real behaviour of steel structures in fire. The application of these advanced tools becomes also more and more common and leads already to some significant evolution of fire regulation toward much more consideration on real risks that the occupants and fire brigade may encounter during a fire.

On the basis of all above technical advancement, has been carried out with RFCS funds a specific research project [1] on the industrial halls. This project has deeply investigated the hidden resistance of steel structures provided by their 3D behaviour and the possible consequence of some local failure in fire situation. In the scope of this project, it is also clearly demonstrated with the help of advanced calculations using validated numerical models that steel structure, if designed appropriately, fulfils the safety requirements in case of fire which will be given in terms of "non-progressive collapse" and "non dangerous failure type". On the basis of a series of parametric studies, several simple design rules as well as some key construction details are proposed (see [2]) in order to help all engineers to design safe steel structures for single storey industrial buildings.

Considering the important progress obtained in above project, a new RFCS project is initiated with the objective of

⇒ summarizing all obtained simple design rules and construction details for single storey industrial buildings in a design guide



- ⇒ developing user-friendly software for more efficient application of simple design rules given in design guide
- ⇒ communicating through technical seminars all above design tools to engineers of several European countries for their fire design of single storey industrial buildings

However, the application of these design rules often needs the approval of corresponding authorities who in turn would like to understand the scientific basis of proposed design methods in order to get full confidence of them. In addition, a lot of experts and engineers are interested in knowing the background of these design methods for extended application of them. Therefore, this document is with the purpose of

- ⇒ giving a survey of real fire cases
- ⇒ providing a summary of several European national requirements in fire regulation
- ⇒ explaining in detail the mechanical basis of simple design rules
- ⇒ showing the validity of simple design rules with respect to advanced calculations

## **2. Survey of Real Fires in Industrial Halls**

### **2.1. Charleroi (Belgium)**

This building was a 6000 m<sup>2</sup> storage hall settled in Charleroi (Belgium). One part of this hall was composed of a prestressed concrete structure and another part was composed of steel structure.

The fire load in this industrial hall was big (it was a factory of clothes recycling). A big part of this hall was devoted to the storage of Textile bundles.



*Figure 2-1 Prestressed concrete structure fallen OUTSIDE (above) and steel structure fallen INSIDE (right)*

The particularity of this structure is the different materials used to compose it (Prestressed concrete and steel) and the difference of comportment of those parts of structure during the fire.

As you will see in the following figures, the structure in prestressed concrete falls OUTSIDE the compartment in fire while the steel structure falls INSIDE the compartment in fire.

## **2.2. Industrial hall (Spain)**

This industrial hall was used for the storage of Lucerne. This warehouse has not reached the total collapse.



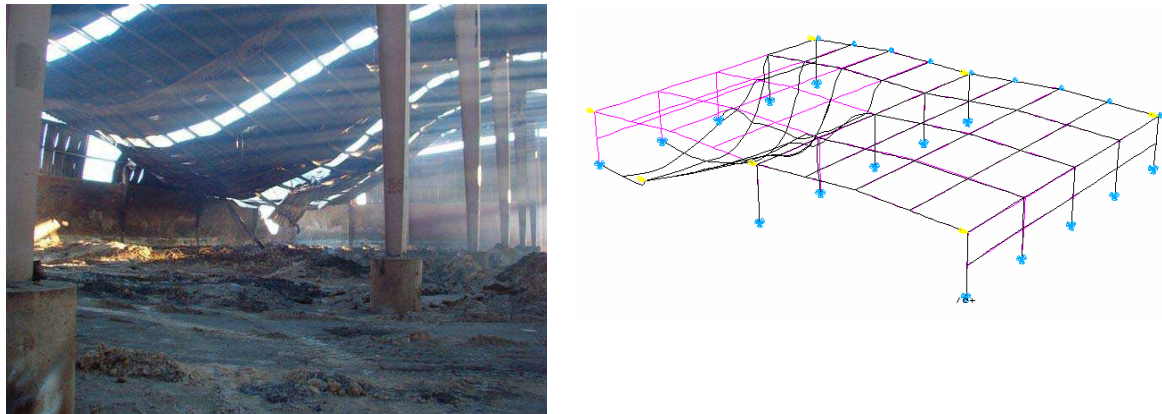
*Figure 2-2 After fire, partial collapse*





Partial collapse shown in the Figure 2-3 has been simulated numerically. The results are presented below. A similar behaviour of the roof and lateral structure is observed in both images, which indicates correct application of the software for prediction of this kind of behaviour.

It must be highlighted that lateral collapse has been produced inwards not affecting outside.



*Figure 2-3 Partial collapse and simulation*

### **2.3. Logs Santos Warehouse (Spain)**

This fire took place on 18th May 2001 in a warehouse of the firm FAGOR that belongs to MCC, located in Vitoria in the northern part of Spain.

The warehouse had two storage zones, one office zone, a dressing room, one custom and the room for the switchboards of alarms. In the east facade it had four exits and in the west facade had one exit and three doors for loading and unloading the lorries.

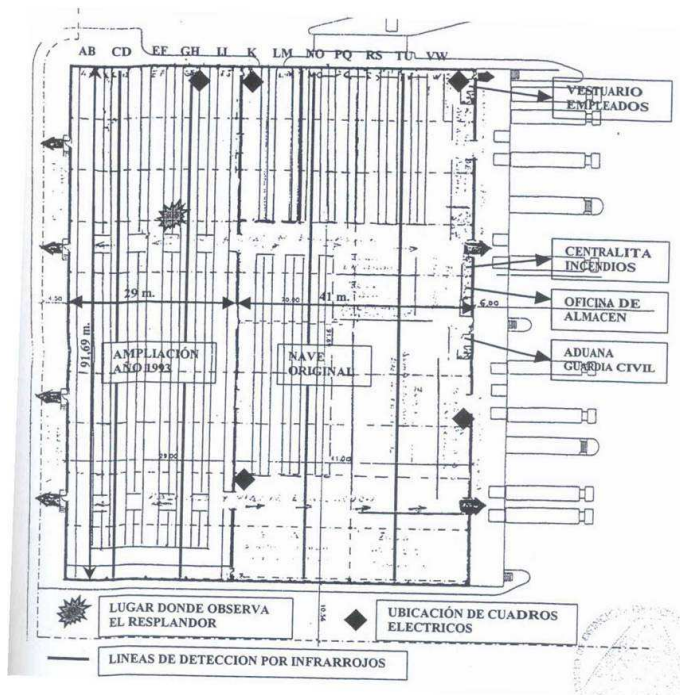


Figure 2-4 Fire scheme made by the Fire Brigade

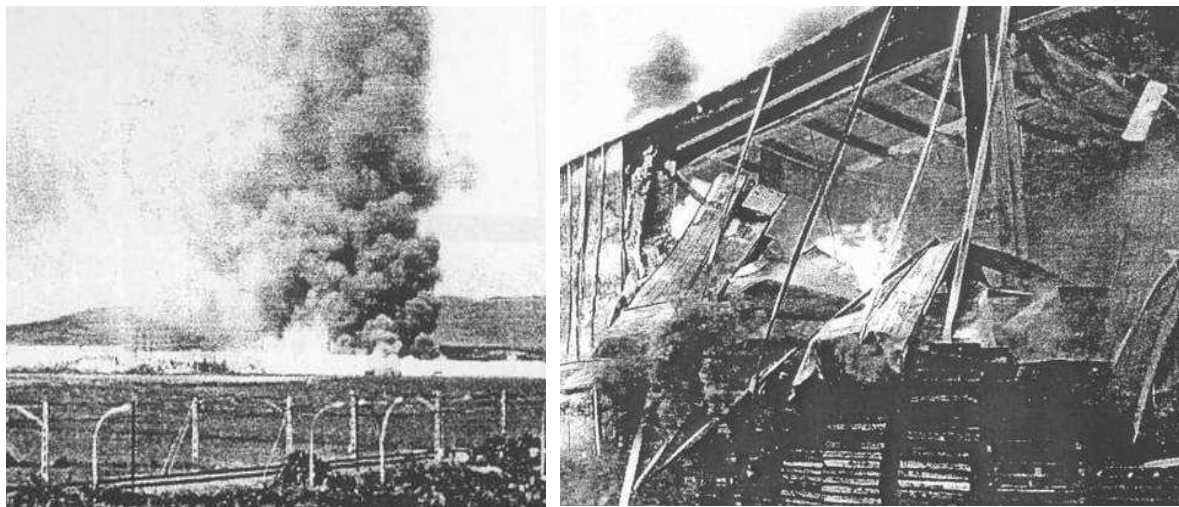


Figure 2-5 Photography of the fire

In terms of damages the A pavilion collapsed completely while the beams of pavilion B did not reach the collapse stage. All the installations were completely destroyed in both pavilions and stored products were destroyed.

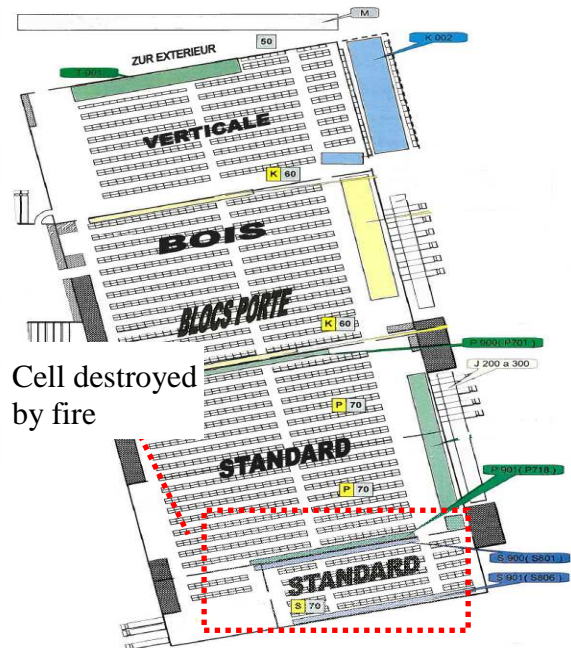
## 2.4. Industrial building (France 2007)





The storage building consists of several cells for various storage activities. The cell destroyed by fire is a steel framed structure and in flammable liquid storage activities. It is separated from the surrounding cells by firewalls equipped with sliding fire door. The cause of fire was most likely of electrical origin. As it can be seen in the following photos, the steel structure has fallen inside the cell during the fire and did not cause any damage to the juxtaposed structure. Except little and non structural damage, the fire wall was intact and there wasn't any significant heat transfer to neighbour cells.

Figure 2-6 Layout of the building



a) Damaged building after the fire



b) Collapse of the structure inside the cell



c) Firewall not damaged by the fire



d) Structure components after the fire

Figure 2-7 The storage hall damaged in fire

## 2.5. Steel industrial building in France

The storage building is composed of four parts as shown in Figure 2-8. The building consists of steel frameworks with unprotected steel columns and lattice beams. Façade elements are panels with double steel cladding containing fire insulating material. Partition walls between the two storage cells as well as the delivery cells are made with masonry blocs. The steel structure close to partition walls is embedded in walls and openings are not closed with doors. Separation between the small storage cell and the office building is ensured by a partition wall in masonry blocks with a door without any fire resistance.

Only 10 minutes after the fire was discovered the fire brigade arrived. They observed large quantity of smoke, which quickly filled in the whole building as the storage products were primarily paperboards and paper with 99% and plastics with 1%.

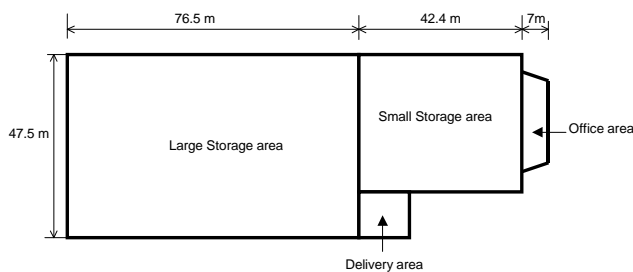


Figure 2-8: Layout of the storage building and development of fire (right)

The firemen observed important chimney effects and confronted to a violent flashover of smokes. Although the building was equipped with automatic extinguishing system, sprinklers didn't function or badly functioned and in consequence are not capable of stopping the fire at the beginning preventing therefore the generalized flashover.

After the fire (Figure 2-9), the large storage cell collapses entirely and the small storage cell doesn't reach the collapse. Only the external facings of the smallest cell remain stable. This is primarily due to the efforts of the firemen to protect the administrative building which was not touched by the fire. All storage products were destroyed in both cells, by fire or water.



Collapse of the large storage cell towards the inside of the building



Collapse of the lattice beams of the large storage cell



Figure 2-9: Collapse of the storage building

### 3. Fire Safety Regulations for Industrial Halls

#### 3.1. Belgium

Summary of the Belgian regulations for industrial buildings

The aim of the regulations is to prevent the beginning, the development and the propagation of a fire, ensure the safety of the users and facilitate the intervention of firemen.

The industrial buildings (IB) are sorted in three classes according to the characteristic fire load density (Class A  $\leq 350$  MJ/m<sup>2</sup>, Class B, and Class C  $> 900$  MJ/m<sup>2</sup>).

The general stability of the hall and the influence and interaction between the elements have to be considered taking into account the elongations and deformations produced by the increase of temperature (second order effects).

A distinction is made between two types of elements:

**type 1** : Element which, in case of collapse will lead to a progressive collapse that is not limited to the compartment where this element is located or to damages on the walls of this compartment.

**type 2** : Element which, in case of collapse lead to a progressive collapse that is limited to the compartment.

The requirement for type 1 elements is R60 for class A and R120 for classes B and C.

The requirement for type 2 elements is based on the equivalent time as defined in EN 1991-1-2.

The requirement for separating walls is EI 60 for Class A and EI 120 for Class B. Doors must be EI60 and be equipped with an automatic closing system.

Recommendations are given for connections between the compartment walls and the roof and between the compartment walls and the facades. The outside walls and the compartment walls must be designed in such a way that **the risk of collapse toward the outside is limited**.

The surface of the compartment  $A_{fi}$  cannot lead to a total design fire load higher than 5700 GJ without sprinklers and 34200 GJ with sprinklers. A one storey IB is deemed to satisfy the requirements if  $A_{fi}$  is lower than the values presented in the following table.

	Fire resistance of structural elements	
	Without sprinklers	With sprinklers



Class. of the hall	no determined R	R 30 or more	no determined R	R 30 or more
A	25 000	25 000	150 000	150 000
B	5 000 (*)	10 000	40 000	60 000
C	2 000(*)	5 000	7 000(*)	30 000
Storage class C	5 000(*)	5 000(*)	12 500(*)	30 000(*)

(\*) The surface of a one storey IB compartment can be increased by 60% if this hall has an improved accessibility.

The fire radiation to the neighbouring buildings cannot be higher than 15 kW/m<sup>2</sup>. Deemed to satisfy distance are given in the following table

Fire resistance of the façade	% of openings	Distance [m]
EI (i↔o) 60	0%	0
	0% ≤ % openings < 10%	4
	10% ≤ % openings < 15%	8
	15% ≤ % openings < 20%	12
	≥ 20% % openings	16
No determined resistance		16

("i" means inside and "o" means outside)

Other rules take into account that both buildings are on the same piece of land or not, the height of the highest façade, the eventual presence of sprinkler installations.

The IB must be equipped with an automatic fire detection installation (manual alarm is sufficient for Class A buildings with A<sub>fl</sub> not higher than 2000 m<sup>2</sup>)

Smoke and heat extraction is required except in the following cases:

- Class A with A<sub>fl</sub> ≤ 10 000 m<sup>2</sup> or Class B with A<sub>fl</sub> ≤ 500 m<sup>2</sup>.
- Compartments equipped with an automatic suppression installation (Sprinklers).

Every fire start has to be signalled to the firemen service.

The control functioning and the command of the active installation must be executed in a central control room (EI 60 wall).

A primary water supply has to exist near the building for the firemen.

### 3.2. France



### 3.2.1. Covered warehouses (storage of materials, products or combustible substances in quantities exceeding 500 tons)

*Classification:* If  $V$  is warehouse's volume then:

$V < 5\,000\text{ m}^3$	$5\,000\text{ m}^3 \leq V < 50\,000\text{ m}^3$	$V \geq 50\,000\text{ m}^3$
not classified	declaration	Authorization

*Requirement:*

The boundary walls of the warehouse or structural elements in case of an open warehouse must be located at a minimum distance of 20 m from the perimeter of the establishment.

Fire-fighters must have access to all exits of the warehouse by a path of 1.40 m wide at least.

The automatic fire detection in cells with storage transmission of the alarm to the operator is required.

With respect to structural fire resistance requirement of these storage buildings, it is summarized in following tables.

Height	$S < 3000\text{ m}^2$	$3000\text{ m}^2 < S < 6000\text{ m}^2$	$S > 6000\text{ m}^2$
$H < 12.5\text{ m}$	R0	R0 + sprinklers	R0 + Sprinklers + FSE
$H > 12.5\text{ m}$	R60 or Sprinklers + FSE	R60 + Sprinklers or Sprinklers + FSE	R60+Sprinklers+FSE or Sprinklers + FSE

Separating walls
<ul style="list-style-type: none"> <li>-REI 120 minimum</li> <li>-All elements ensure a equivalent REI level</li> <li>-The door between cells must be REI 120 with automatic shut-off.</li> <li>-Separating walls must be at least 1 m from roof.</li> <li>-If the exterior walls do not have a degree REI 60, the walls separating these cells are extended sideways to the exterior walls over a width of 1 m or 0.50 m protruding from the front in the continuity of the wall.</li> </ul>

The Fire Safety Engineering Study (FSE) must be carried out to demonstrate that the collapse of one cell does not create the chain collapse of the whole building and when building collapses in fire, it shall not collapse toward outside. Moreover this study must show that all the staff has enough time to evacuate from the building before the collapse occurs.





### 3.2.2. Storage of polymers, pneumatic and products of which at least 50% of the total mass unit is composed of polymers [plastics, rubber, synthetic resins and adhesives]

*Classification:*

If  $V$  is storage's volume then:

$V < 100 \text{ m}^3$	$100 \text{ m}^3 \leq V < 1000 \text{ m}^3$	$V \geq 1000 \text{ m}^3$
not classified	declaration	Authorization

*Requirement:*

The boundary walls of the structural elements must be located at a minimum distance of 15 m from the perimeter of the establishment or 10 m if the cell is equipped with a sprinkler system or the external wall is REI 120 exceeding at least 1 m of roof and 0.5 m laterally and of which doors have a fire rating REI of 60 minutes, equipped with a closed-door.

Regarding other elements, the requirement is:

Floor	Separating walls	External walls
up to REI 60	REI 120, door REI 60	R 30

### 3.3. Luxembourg

The safety regulation in Luxembourg is called Commodo/Incommodo described in a prescriptive law of 10 June 1999. It replaces the previous law of 1979 and was introduced for adapting reasons. It is enforced by the Ministry of Employment [13].

No fire resistance requirement is defined for industrial buildings.

### 3.4. Spain

Due to the law 2267/2004 of 3 December 2004, in case of industrial buildings (industries in general and industrial storages) and any type of storage building with a fire load bigger than 3.000.000 MJ, the regulation having jurisdiction is the "Fire Safety Regulation for Industrial buildings" called RSIEI.

This regulation can be accomplished in two different ways:

- ⇒ Fulfilling the prescriptive requirements of the RSIEI code.
- ⇒ With equivalent safety techniques, based on well known rules and regulations, properly described by the designers and approved by the authority having jurisdiction.

Buildings are classified according to:

- ⇒ Fire risk depending on the industrial activity carried out:
- ⇒ Low risk buildings: fire load  $< 850 \text{ MJ/m}^2$
- ⇒ Medium risk activities: fire load  $< 3400 \text{ MJ/m}^2$



⇒ High risk activities: fire load bigger than 3400 MJ/m<sup>2</sup>

Building typology: proximity of other occupancies within the same building or in neighbouring buildings:

⇒ Type A: industrial occupancy in a building shared with other industrial occupancies or even not industrial ones

⇒ Type B: industrial occupancies taking up a whole building detached less than 3 metres from any other one

⇒ Type C: industrial hall occupied completely by one occupancy and detached more than 3 metres from other buildings

⇒ Types D and E: occupancies covered by open structures without walls.

In function of this classification, the prescriptive requirements are established in terms of structural stability, compartment size and fire walls, distances for the evacuation of people...

Fire risk	Type A		Type B		Type C	
	Basement	Storey	Basement	Storey	Basement	Storey
Low	R120	R90 R60**	R90	R60 R15* R0**	R60	R30 R0* R0**
Medium	Not allowed	R120	R120	R90 R30* R15**	R90	R60 R15* R0**
High	Not allowed	Not allowed	R180	R120 R60* R30**	R120	R90 R30* R15** R0***

\* If the roof is light (<100kg/m<sup>2</sup>) and the collapse of the structure does not endanger other buildings or damage the compartmentation (smoke control system is necessary if the fire risk is medium or high)

\*\* Single storey buildings fitted with sprinklers and smoke control system

\*\*\* Single storey C buildings detached at least 10 meters from other buildings

*Table 3.1 : Structural fire resistance requirement for single storey building in Spain*



For general buildings, the requirements given in Table 3.1 are demanded for structural fire resistance. Some reductions are allowed in case of light roofs (up to 100 kg/m<sup>2</sup>) for buildings B and C for structural stability of the supporting structures of the roof. Also reductions are allowed for sprinkled halls. And finally, all single storey C buildings detached at least 10 meters from other buildings, no stability requirement is demanded.

## 4. Numerical Simulations

### 4.1. Software verification

Software applied to simulate structural behaviour of the building in fire has to cover the 3D structural behaviour including membrane and restrained effects as well as the failure mode so that post-local failure stage can be analysed. Such calculation models (ANSYS [9], ABAQUS and SAFIR [14]) have been compared true a benchmark. In this benchmark, two different users used ABAQUS.

#### 4.1.1. Benchmark definition

This benchmark is based on the following structure:

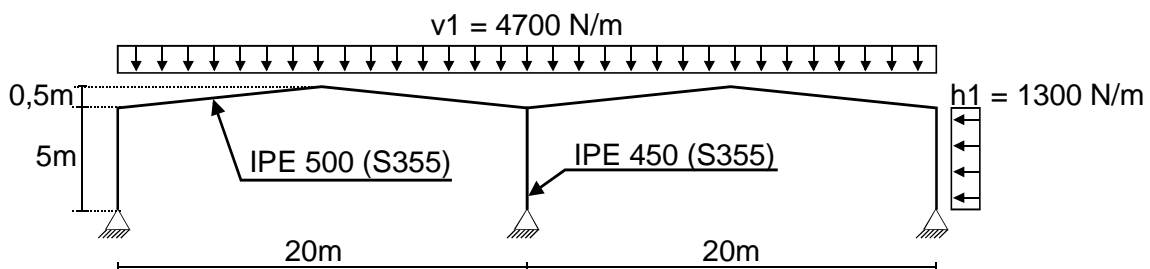


Figure 4-1 Benchmark 2D portal frame

- the material laws for thermal and mechanical properties come from the EC3 Fire parts [18];
- for the mechanical properties, the strain hardening is not considered;
- all the profiles will be assumed class 1 section during the fire;
- for the calculation of the temperature in the steel, an ISO fire curve is considered [19];
- for the thermal transfer, convection and radiation have been considered true the following parameters:

$$\alpha = 25 \text{ W/m}^2 \text{ K}$$

$$\varepsilon = 0.5$$

Equation 4-1

- no shadow effect has been taken into account.

The simple calculation method of EC3 [6] is used to evaluate the temperature curves of steel members (IPE 450, IPE 500). This lead to a uniform distributed temperature in the cross sections.

The study is composed of 4 parts as presented in Figure 4-2:

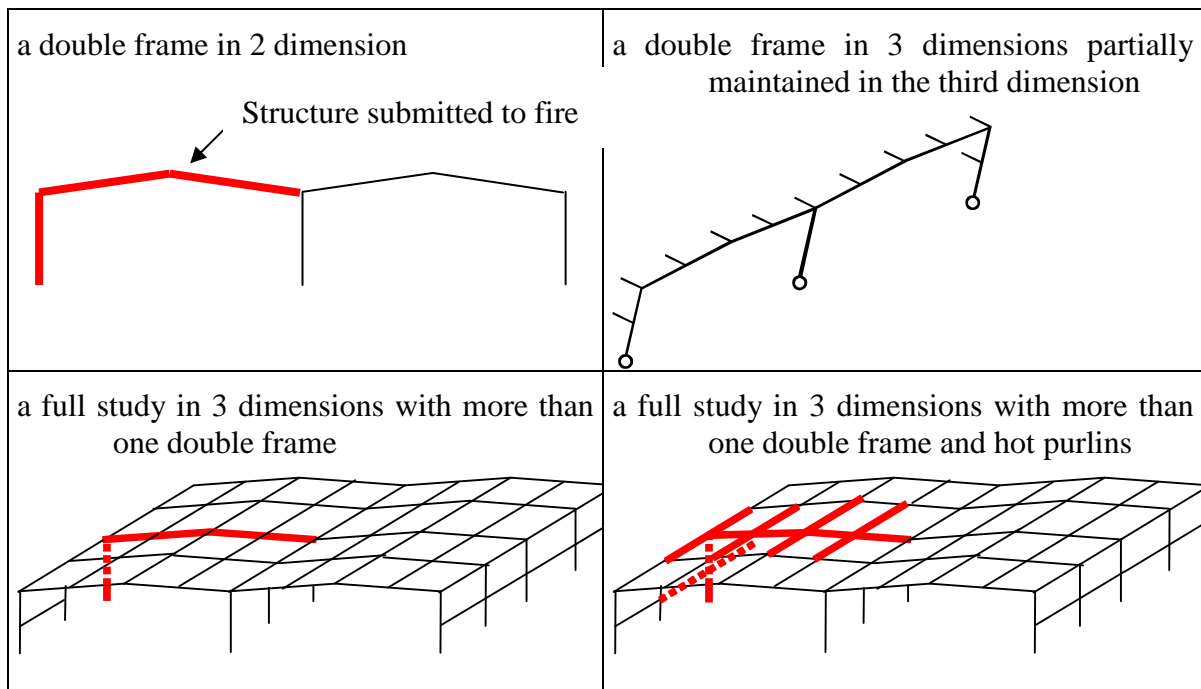


Figure 4-2 Illustration of the analysed models.

Unfortunately, the statistical finite element calculation stops before the real failure of the structure even for 2D analysis of single frame.

In order to avoid this numerical interruption, the possibility to perform a dynamic analysis of the structure has been studied with the different software [10]. Dynamic approach has been applied to the full 3D calculation.

#### 4.1.2. Results in 3 dimensions for one frame

The same frame has been analysed in 2D and in 3D by allowing the out-of-plane displacements. The frame is hinged frame with additional fixations added in the third dimension. In reality the restrains are provided by purlins (the 11 fixations in the third direction are shown in Figure 4-3).

The only initial deformation is in the frame plane XY according to the Y axis as shown in Figure 4-3. The maximum value is  $L/1000 = 0.01$  m. There is no initial deformation for the columns.

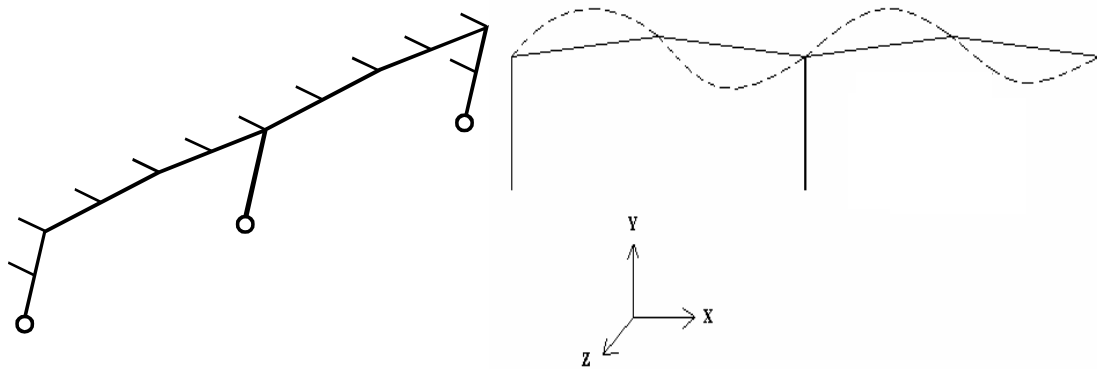
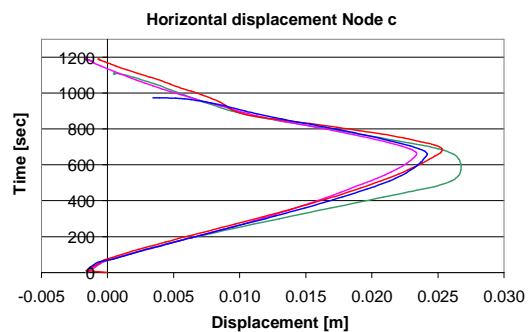
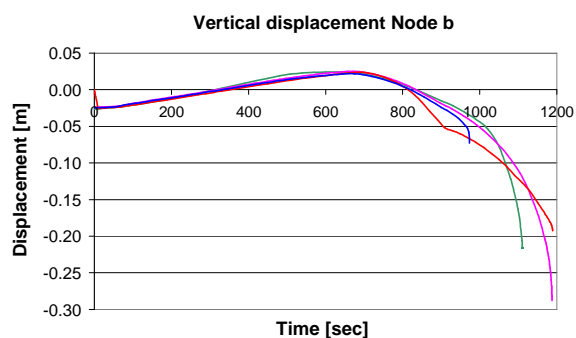
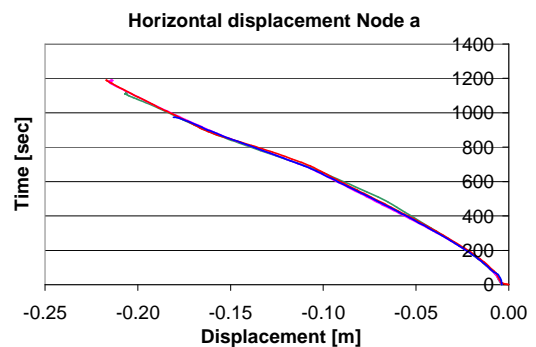
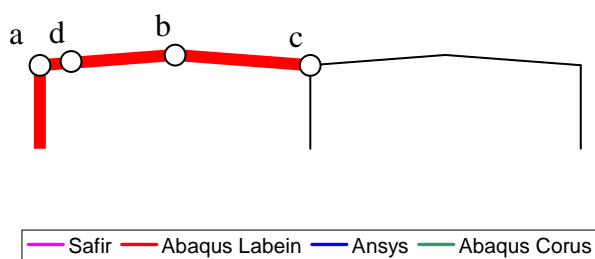


Figure 4-3 Illustration of the fixed points in the third dimension and a scheme of the initial deformations

### ➤ Evolution of the horizontal and the vertical displacements:

Evolution of the displacements in respect to time calculated using different software is presented in Figure 4-4. The displacements are measured in nodes “a” to “d”. As it is marked on the image below, the node “d” is located at 1/4 of the length of the first beam, which is heated (marked red):





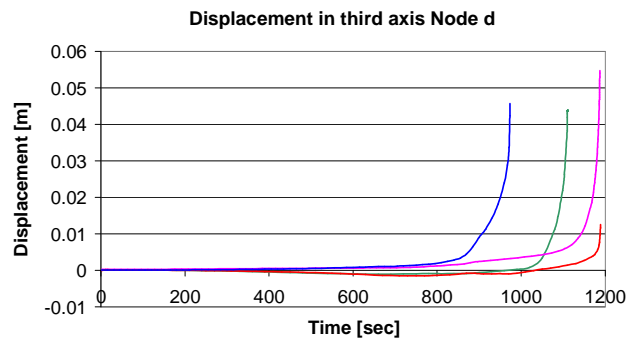


Figure 4-4 Displacement analysis of the 2D portal frame in space

The collapse of the structure occurs some minutes before the 2D analysis due to the lateral buckling of the beam under fire.

➤ **Evolution of the normal force with respect to the time:**

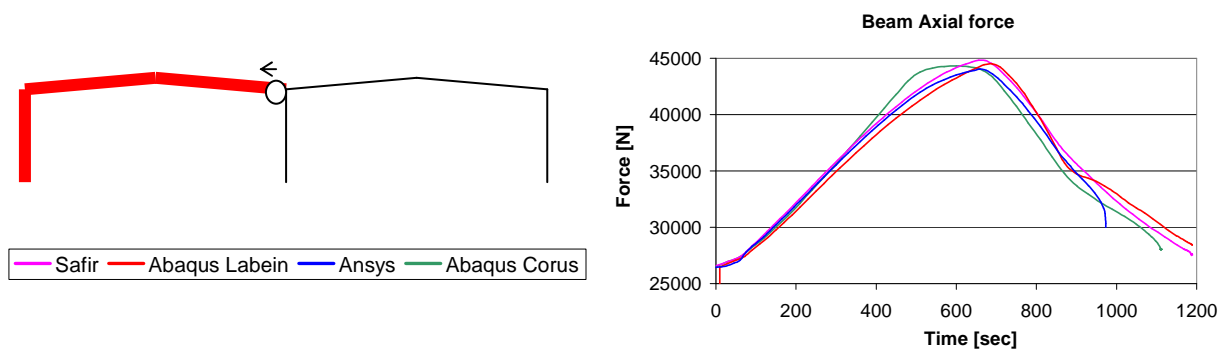


Figure 4-5 Axial force evolution

As marked in the Figure 4-5 the axial force is measured at the connection between the central column and the beam under fire and the connection of the central column and the “cold” beam.

The axial forces applied on the cold part of the structure have the same order of magnitude as the 2D analysis.

➤ **Deformation of the structure:**

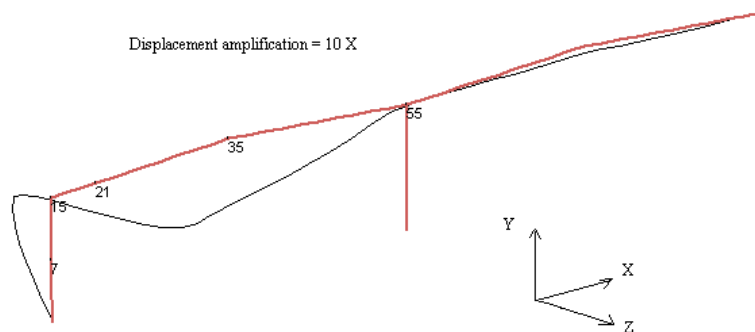


Figure 4-6 Spatial deformation of the frame



The deformation of the structure illustrated in Figure 4-6 is amplified 10 times.

#### 4.1.3. Results of the full 3 dimensional analysis - for more than one frame

The frame analysed independently in the previous sections is now included in a full 3D structure with other parallel frames connected to the first one by purlins. As in the precedent cases, the only central left frame is heated - marked red in Figure 4-7.

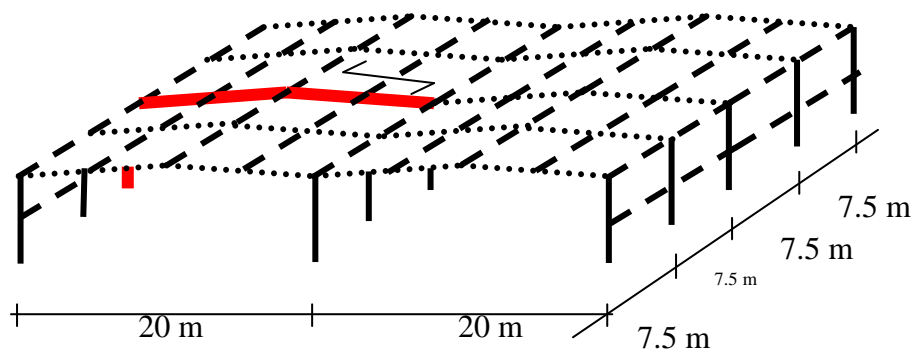


Figure 4-7 3D structure with multiple frames with marked single heated frame

The initial deformations applied to the central double frame only and they are the same as for the single bay analysed earlier.

The 3 displacements are the same for the purlins and for the beam in the connecting nodes.

For the rotation, the rotation around the Z axis ( Z axis is directed along the purlins) is the same for the beam and for the purlins because the purlins are fixed by 2 bolts on the beam. But the rotation around the X axis, Y axis and the warping are free between the purlins and the beam.

The structure is maintained in several points to simulate the presence of wind bracing and a load is applied to each purlin simulating a real load on the structure.

## 4.2. Numerical investigation of simple and multi-bay portal and lattice frame structures

The mechanical behaviour of simple and multi-bay framed structure under standard fire exposure has been investigated with a parametric study in which different main parameters affecting the performance of this type of steel structures were taken into account, such as **span of frames, height of columns, number of spans, fire location, position of fire walls**, etc.

### 4.2.1. Characteristic of the structures

All the analysed systems were built from the same type of hot-rolled profiles with the same type of connections as follows:

- **steel grade S235 was used for the frame systems;**



- **steel columns are hinged or semi rigid at bottoms;**
- **connections between beams and columns are rigid;**
- **columns are I or H hot rolled steel sections.**

Additionally for the lattice frame structures the following feature are considered:

- **lattice beams (top and bottom chord member and diagonals) are built from two equal leg angles back to back or crossed;**
- **equal leg angles are ranging from 50x50x5mm to 120x120x12mm according to beam span and column height. The depth of lattice beams is 2m;**
- **connections between lattice members (chords and diagonals) and connections between lattice beams and columns are rigid.**

#### **4.2.2. Assumptions of numerical modelling for analysis of the portal frames**

The simulations of the mechanical behaviour of structural steel frames exposed to fire with the computer code SAFIR and ABAQUS have been conducted using the following rules and assumptions:

- **2D numerical model was studied in a three dimensional space;**
- **dynamic simulations have been performed;**
- **steel columns and beams are modelled using beam finite element;**
- **loads applied on the building roof and on the columns are uniformly distributed, Figure 4-8;**

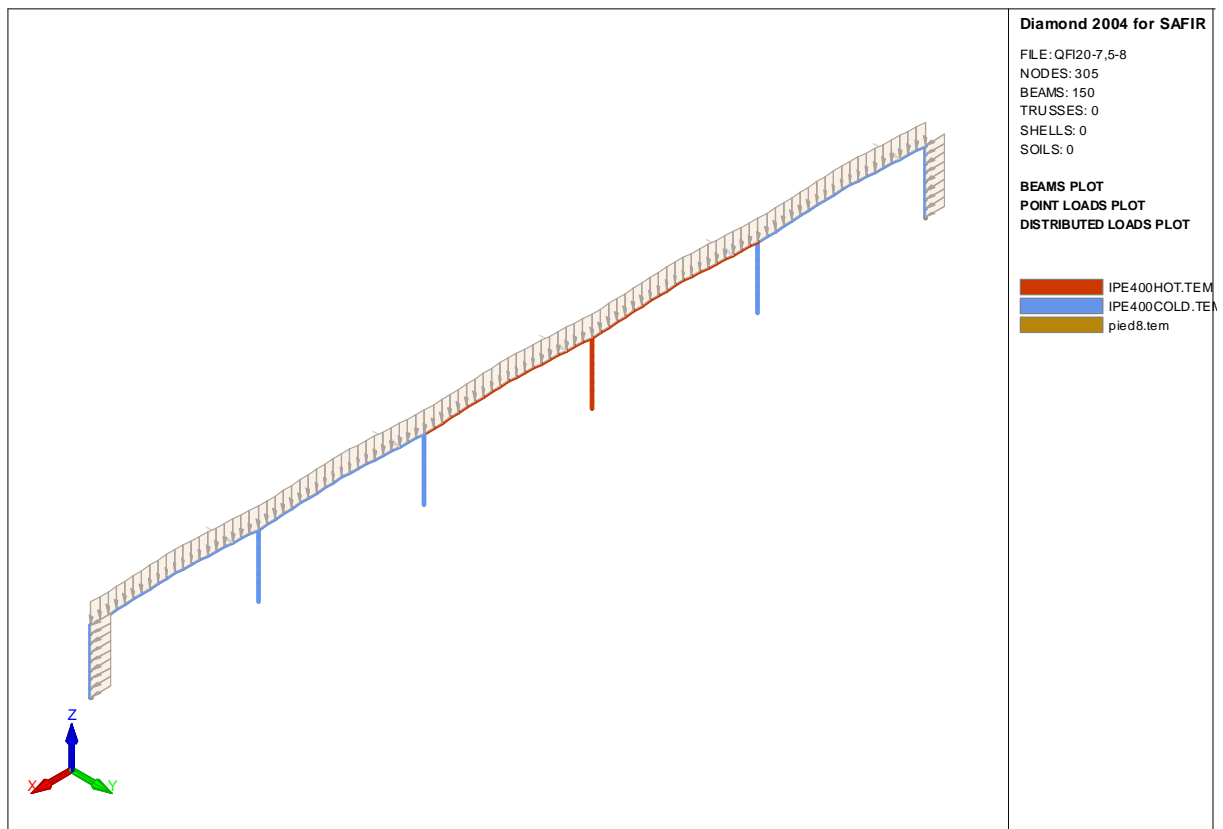


Figure 4-8 : Loading conditions of steel frames

- global out of plane imperfection was applied to the model (see Figure 4-9);
- no residual stresses taken into account;

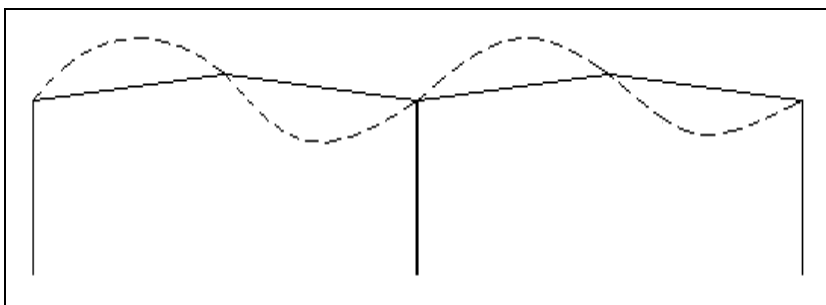


Figure 4-9 : Out of plane imperfection

- the mechanical materials properties according to EC3 Part 1.2;
- restrained lateral displacement of several points at position of purlins (see Figure 4-10).

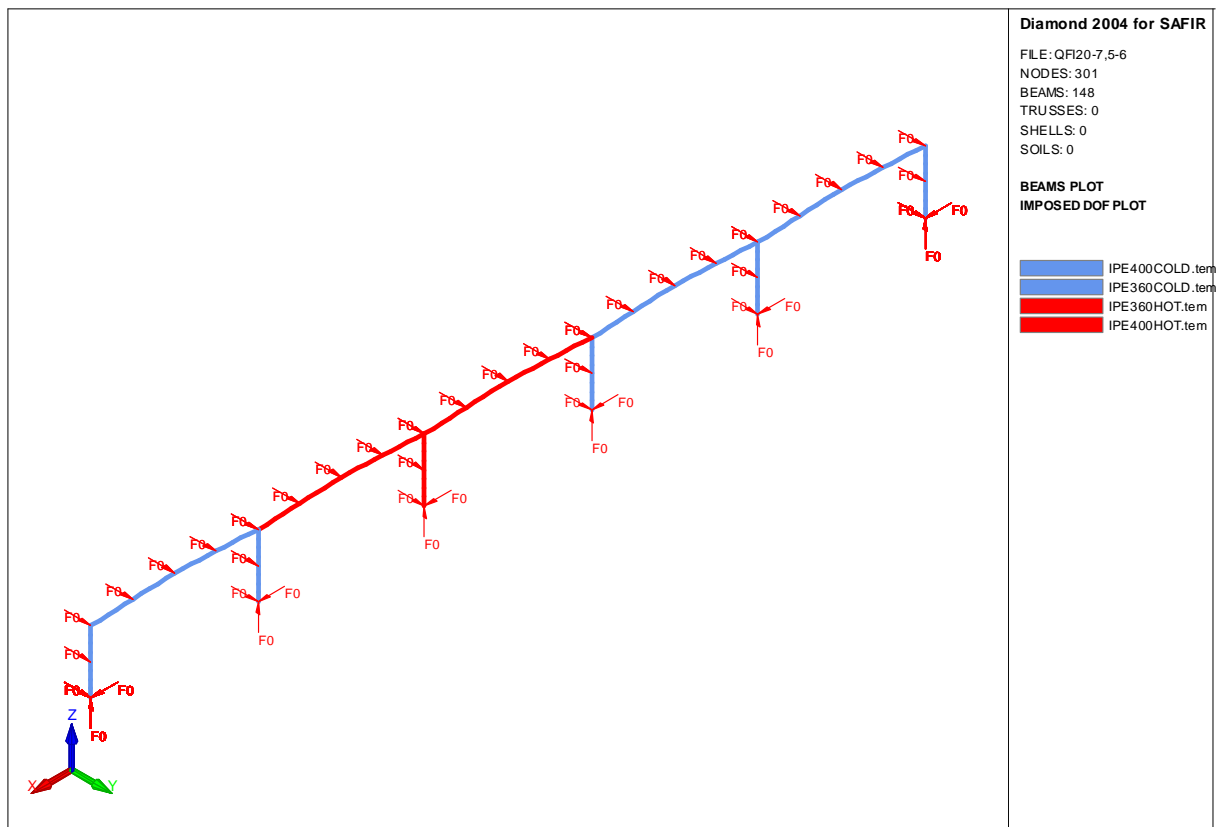


Figure 4-10 : Boundary conditions of steel frames

#### 4.2.3. Assumptions of numerical modelling for the lattice frames

The simulations of the mechanical behaviour of structural steel frames exposed to fire with the computer code ANSYS [9] have been conducted using the following assumptions:

- simulations have been performed under static and dynamic procedure;
- steel columns and lattice beams are modelled with finite element beam as shown in Figure 4-11;

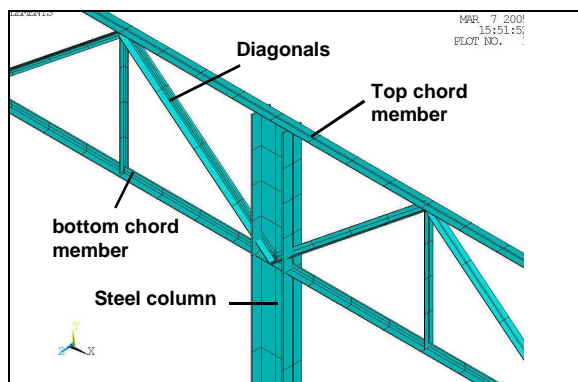




Figure 4-11 : Modelling of steel frames with beam elements

- loads applied on the building roof are taken into account as concentrated loads applied at nodes of top chords (Figure 4-12).
- the loads applied on the columns are uniformly distributed along the element;

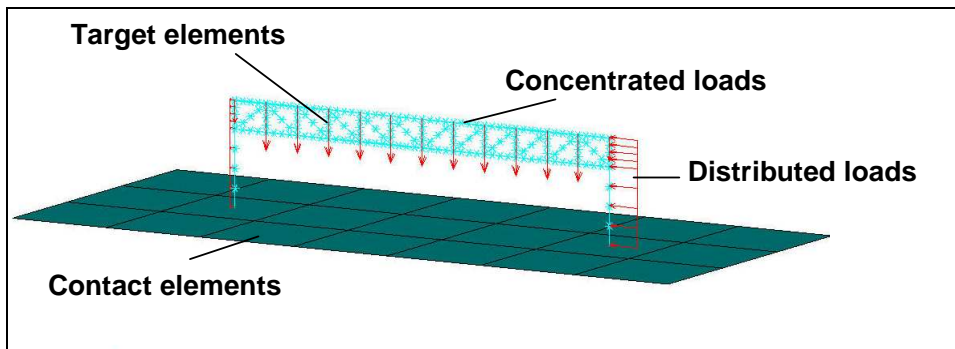


Figure 4-12 : Loading conditions of steel frames as well as modelling of ground level

- there is no sway or member imperfection in the model and the residual stresses are not taken into account;
- the mechanical materials properties are those given by EC3 Part 1.2;
- restrained lateral displacement of several points at position of purlins (see Figure 4-13).

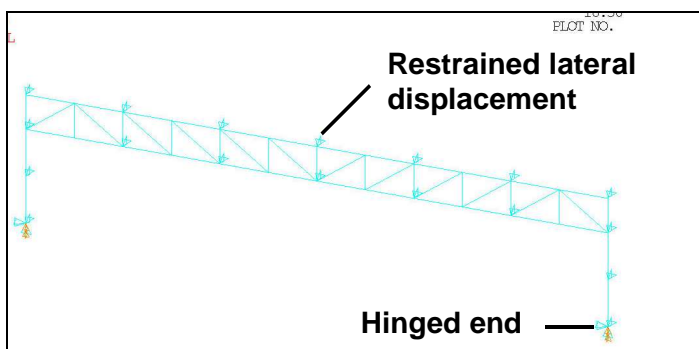


Figure 4-13 : Boundary conditions of steel frames

#### 4.2.4. Loading conditions

Steel frames have been dimensioned at room temperature on the basis of Part 1.1 of Eurocode 3 [21].

The various load values (**selfweight, effect of the wind and snow**) as well as their combinations under fire situation are described hereafter:

- **self weight ‘G’:**



- ⇒ Weight of roof is taken as  $250 \text{ N/m}^2$  ;
- ⇒ Weight of wall cladding is taken as  $150 \text{ N/m}^2$  ;
- **the snow load 'S' is taken as  $550 \text{ N/m}^2$ ;**
  - ⇒ This load corresponds to a building with a roof having a slope above 5%, located in zone 2a at altitude less than 200 m.
- **the wind load 'W' is taken as  $555 \text{ N/m}^2$ ;**
  - ⇒ This load will be reduced using appropriate pressure coefficients ( $C_{pe}$  and  $C_{pi}$ ) as shown in Figure 4-14 and Figure 4-15 respectively for portal and lattice frame. Numerical analyses have been carried out with only one the most unfavourable configuration of wind for fire condition has been considered;
- **no imposed loads have been considered.**

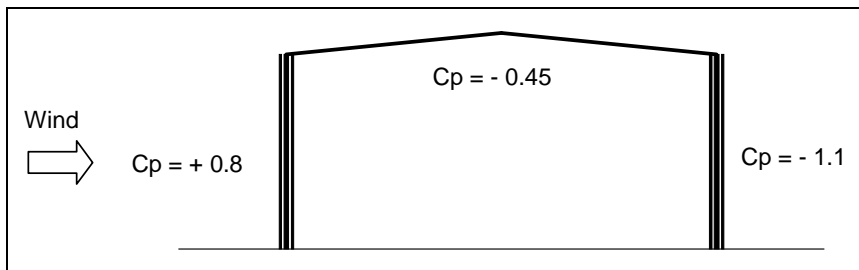


Figure 4-14 : Pressure coefficient  $C_p=(c_{pe}-c_{pi})$  in fire situation

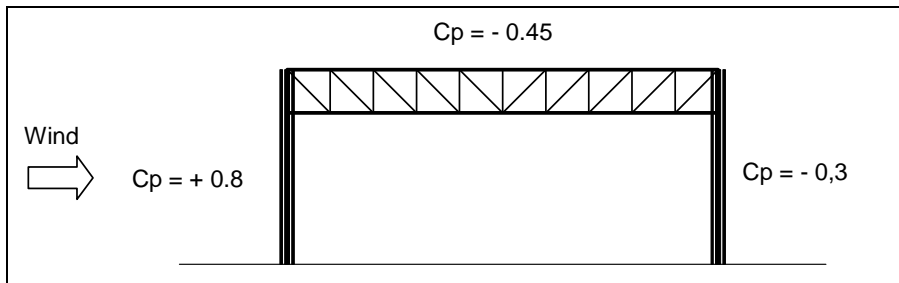


Figure 4-15 : Pressure coefficient  $C_p=(c_{pe}-c_{pi})$  in fire situation

From above loads, the load combinations taken into account in the numerical analyses are

- ⇒  $1.0 \times G + 0.2 \times W$  and
- ⇒  $1.0 \times G + 0.2 \times S$ .

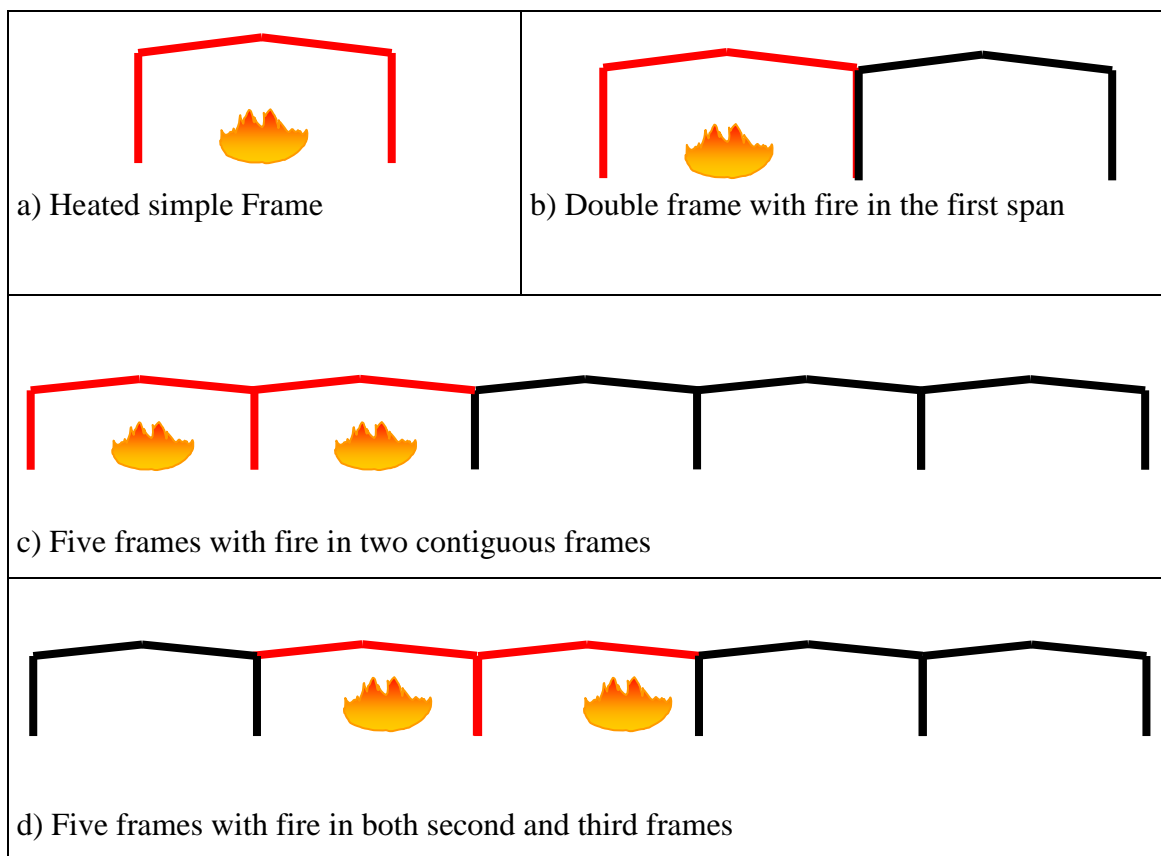
#### 4.2.5. Heating conditions:

- **steel frames are submitted to the standard time-temperature curve according to ISO 834;**
- **the material laws for thermal properties are those of EC3 Part 1.2;**
- **steel elements are assumed to be unprotected and heated from four faces;**

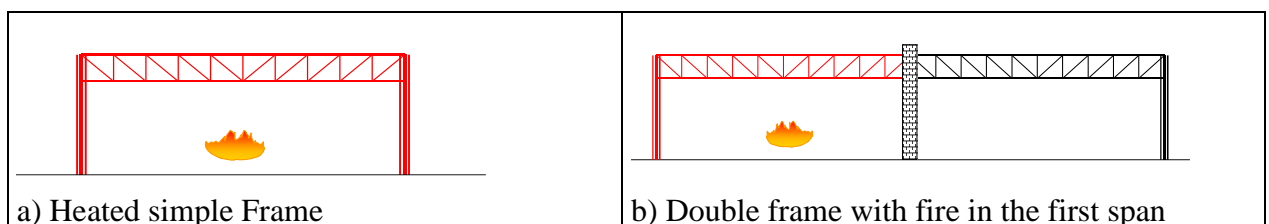


- **internal columns at the position of fire walls remain at room temperature;**
- **uniform temperatures on the cross-section as well as over the length of heated steel elements;**
- **heating rate of steel members exposed to fire has been determined using the section factor of the element according to EC3 Part1.2;**
- **all profiles have been assumed class 1 sections during the fire.**

For framed structures and lattice structures, different configurations have been investigated according to the frame number, the position of the fire walls and the fire location in the disaster cell (see Figure 4-16 and Figure 4-17 ).



*Figure 4-16 : Fire scenarios in portal frame structure*



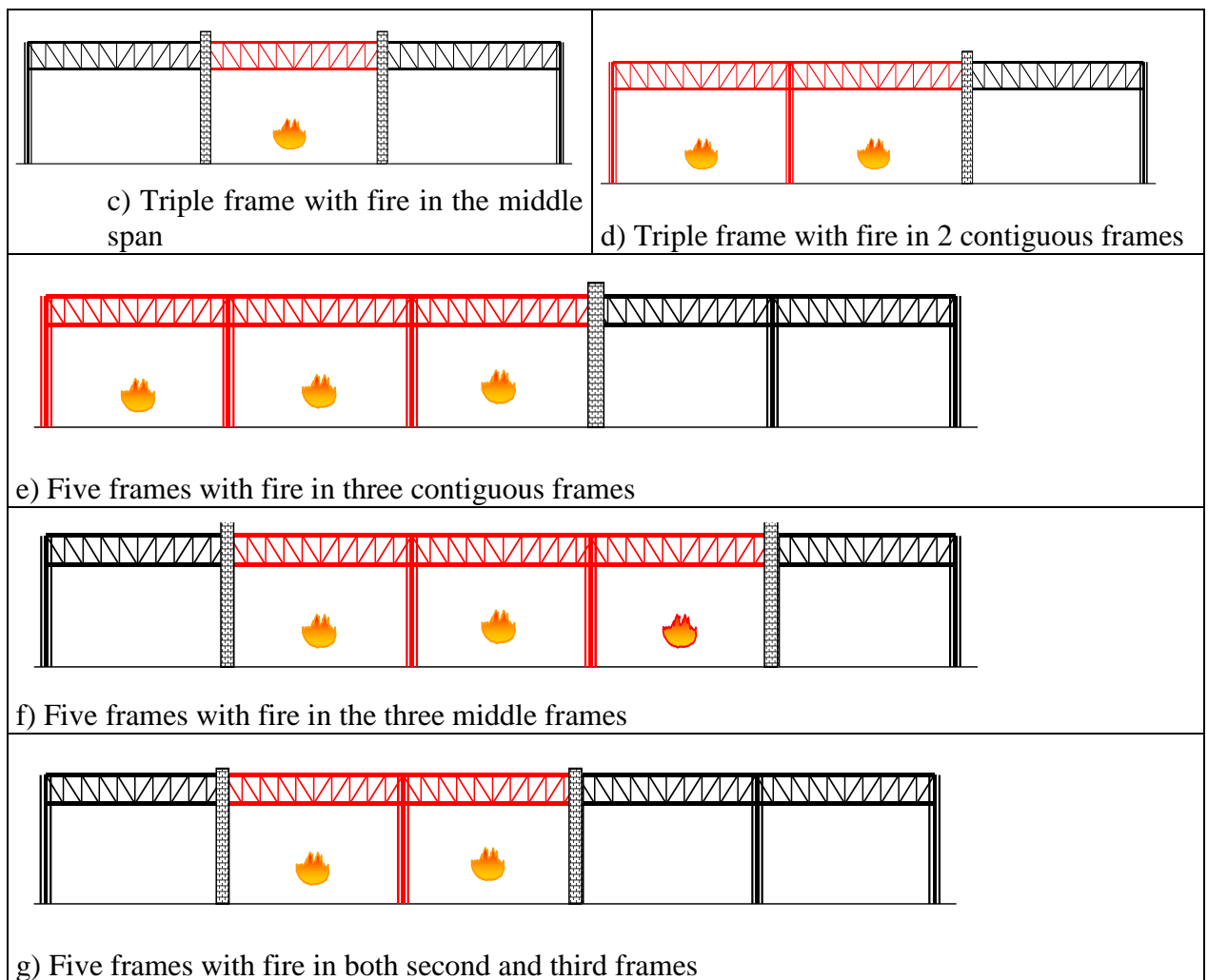


Figure 4-17 : Fire scenarios in multi-bay frames

For the calculation of temperatures, the following parameters have been considered:

- **coefficient of heat transfer by convection:  $h = 25 \text{ W/m}^2\text{K}$ ;**
- **emissivity:  $\epsilon = 0.5$ ;**
- **no shadow effect.**

#### 4.2.6. Range of parametrical tests

Below are listed parameters and their range used in the study of the behaviour of the portal frames in fire conditions:

- **frame systems: single, double and five frames;**
- **frame spans: 20m, 30m and 40 m;**
- **the column length is ranging from 7.5m to 20m;**
- **the frame spacing is taken as 6m, 8m and 10 m;**



- **two pitches: 1.5 ° and 10 °.**

And parameters used for analysis of the lattice frames:

- **frame systems: single, double, triple and five frames;**
- **frame spans: 20m and 30m;**
- **the column length is ranging from 7.5m to 17.5m;**
- **the frame spacing is taken as 15m and purlin spacing is taken as 4m or 5m according to beam span;**
- **equal leg angles are ranging from 50x50x5mm to 120x120x12mm according to beam span and column height.**

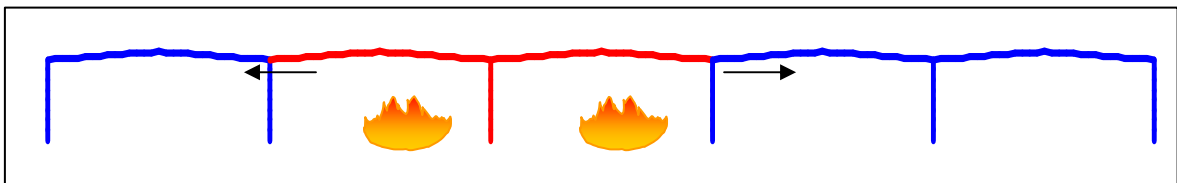
### 4.3. Results of parametric studies

#### 4.3.1. Fire behaviour of portal and lattice frame structure

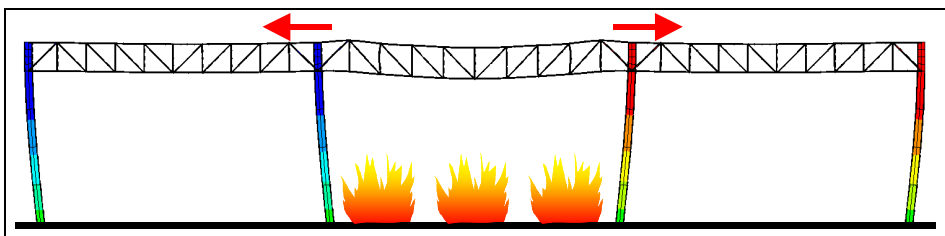
The analyses of numerical results show that the behaviour of multi portal framed structure can be divided in two successive phases leading to different structural behaviours.

One phase corresponds to thermal expansion of heated members (expansion phase). During this phase, the following observations have been made:

- **a progressive increase of lateral displacements at the top of columns towards the outside of the fire compartment (Figure 4-18, Figure 4-19);**



*Figure 4-18 : Lateral displacements at the top of columns*



*Figure 4-19 : Lateral displacements at the top of columns*

- **a progressive increase of internal forces (additional compressive force) in the heated beams. These compressive forces are due to the axial**



restraint against thermal elongation induced by the cold parts of the structure;

- in the case of the lattice beams the end of this phase occurs when the heated lattice beams fail mainly under compressive force. Stability depends on the fire resistance of steel members constituting the beam (Figure 4-20).

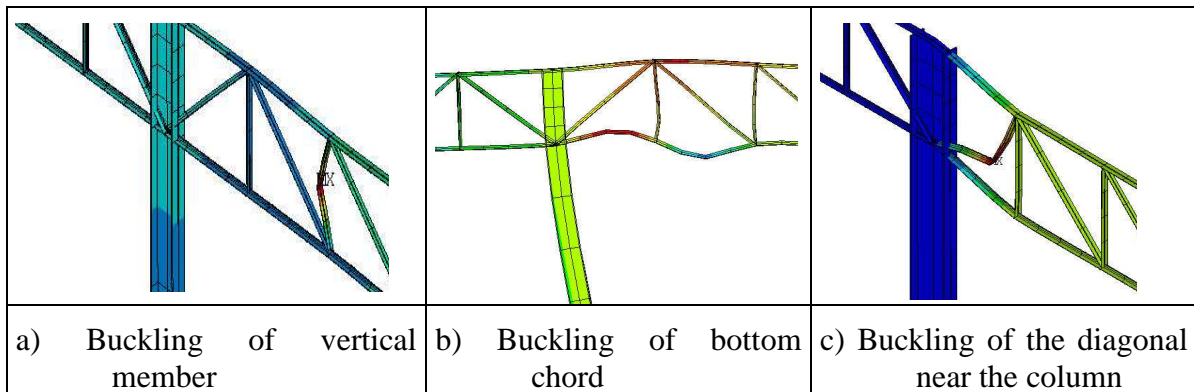


Figure 4-20 : Origin of the failure mode of heated lattice beam

A second phase refers to the collapse of the heated beam. During this phase the following events take place:

- beam changes progressively from combined compression and bending state to simple tensile state;
- from the beginning of this phase, displacement increments at the compartment ends change its direction: the top of columns go back to initial state and finally moves towards the fire compartment (see Figure 4-21, Figure 4-22);

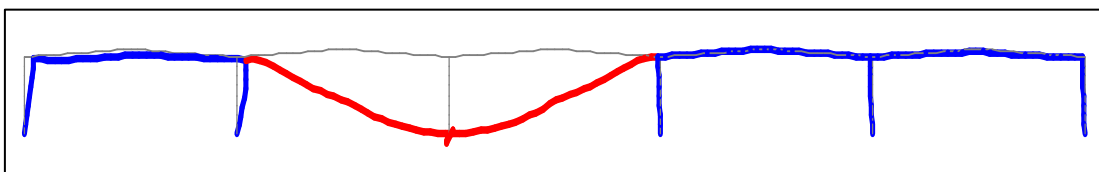


Figure 4-21 : Lateral displacements at the top of columns

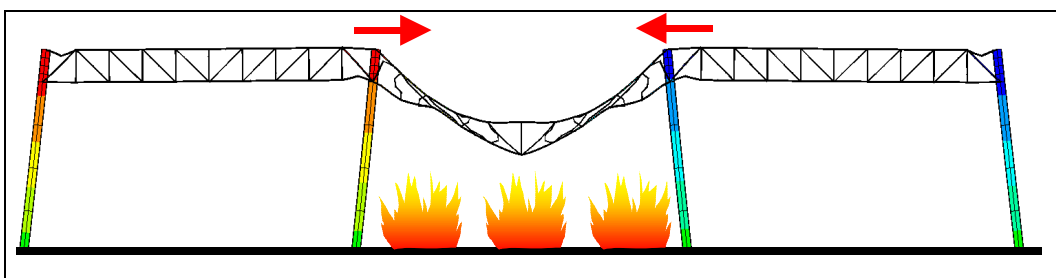


Figure 4-22 : Lateral displacements at the top of columns

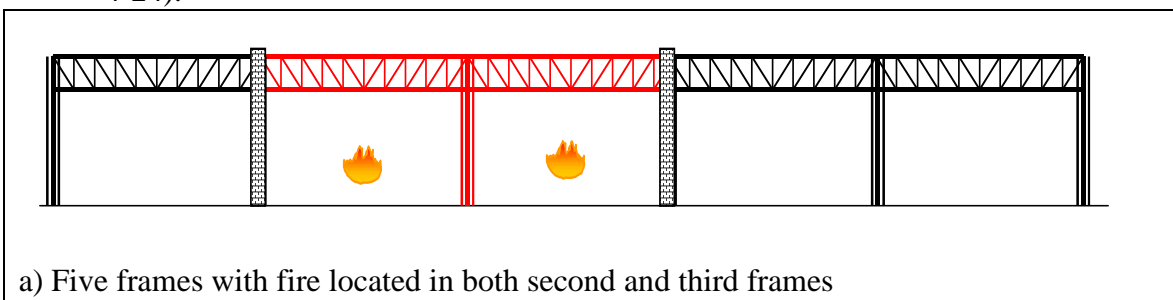


- **under important tensile force, heated beam behave as a chain;**
- **the lateral displacement at the top of compartment edge columns and the tensile force reach maximum points due to collapse of the beam and decrease then slightly;**
- **if the rigidity of the cold parts is not strong enough in the final phase, the structure collapses inside the fire compartment.**

#### 4.3.2. Parametric study observations

The structural behaviour of multi-bay frames under standard fire exposure have been investigated with a parametric study by varying the main parameters expected to influence the performance of this type of steel structure such as span, height of columns, number of bays, fire location and position of fire walls. Studied steel frames have been designed for room temperature on the basis of part 1.1 of Eurocode 3.

The analysis of obtained results shows clearly that the collapse of the multi-bay lattice frames is always caused by the failure of the heated beam as a result of important additional internal forces due to the axial restraint against thermal expansion induced by the cold parts of the structure (see Figure 4-23). In fact, under fully developed fire all the structural elements (beams and columns) of the same compartment are exposed to fire. In the fire conditions the beams fail always before columns as they tend to be made from smaller profile (especially the lattice beams). Additionally the temperature rise is much lower in the columns and the failure occurs later. Therefore, when beams fail before the collapse of columns, the chain effect will occur over one span alone (see force values at about 500 seconds of fire in Figure 4-24). It can be observed that the maximum horizontal tensile force created by chain effect is reached just after the failure of the beams. Afterwards, this force decreases progressively because the failing beams are continuously heated up and the plastic tensile resistance could be reached quite early leading to a significant increase of their elongation (in given example illustrated in Figure 4-24, this phenomenon occurs at about 900 seconds of fire). When steel columns collapse, this elongation is so important that even the chain effect with two spans will lead to smaller horizontal tensile forces for cold parts of the frame (see Figure 4-24).



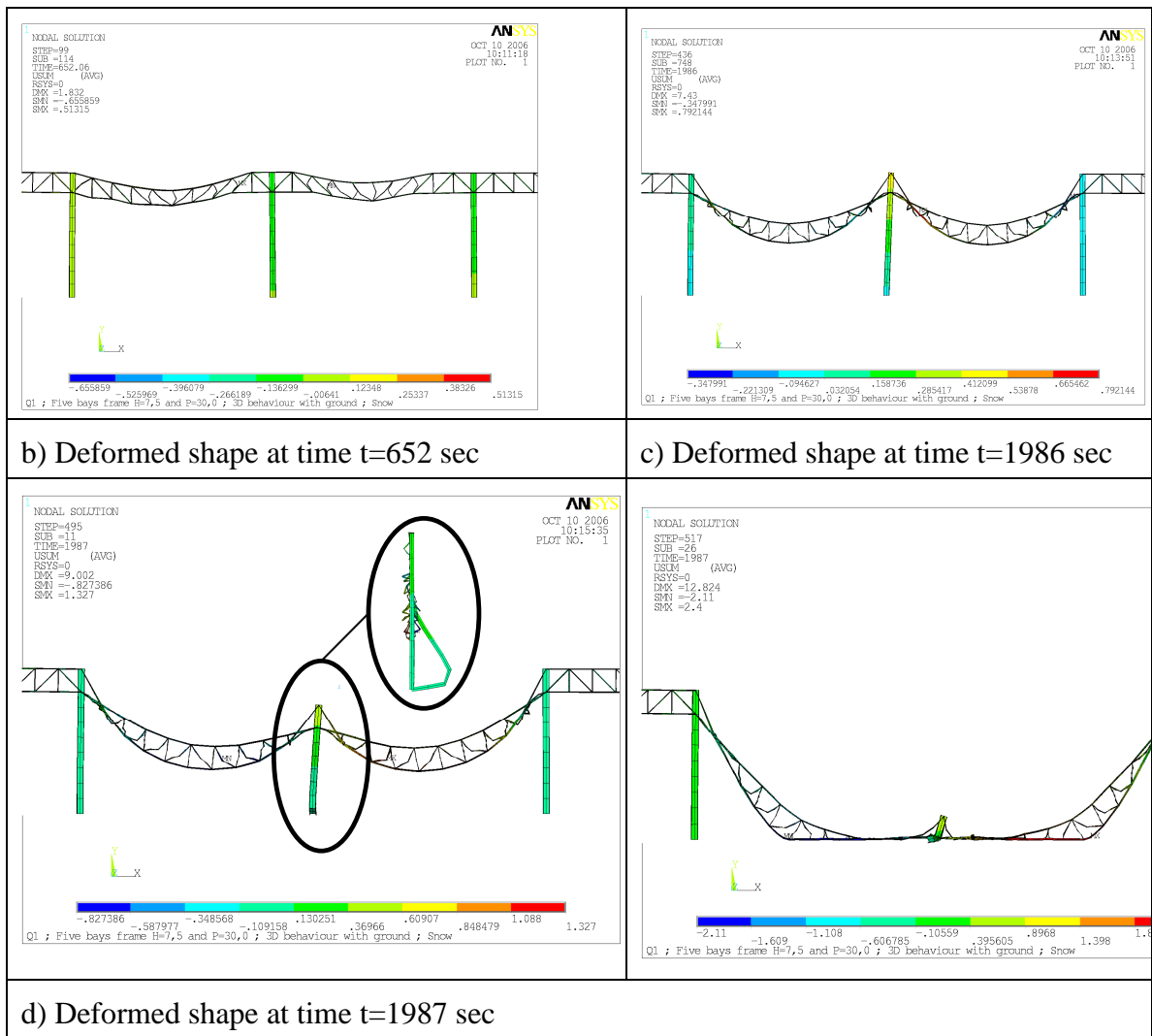


Figure 4-23 : Example of failure mode of five steel frames

In addition, the maximum tensile force in case of lattice beams has to be limited by the plastic tensile resistance of both top and bottom chords which are much less resistant than steel beams in case of portal frames. From this point of view, regarding the example given in Figure 4-24, if the heated column failed at about 18 minutes of fire, even the elongation of above two members is supposed to lead to the maximum chain effect at this moment, the horizontal force predicted by the simple calculation method using single span chain would not be exceeded. However, the failure of the column at this stage of fire is quite early.

As a consequence of above investigation, for lattice frames, the tensile force induced by the failure of the heated parts of the structure to check the performance of the lattice framework with respect to the progressive collapse of the storage building can be calculated by considering that each heated lattice beam behaves as a single span chain between their support columns.



- In the calculation method, heated columns are assumed to be sufficiently fire resistant to be considered as rigid support. So, the number of spans to take into account in the design method should not exceed 1, even if the number of spans of the fire compartment is more than 1.
- In real fire situation, the use of single span chain effect can be considered also as a realistic assumption because under general fire spread, roof beams will be much more heated than steel columns due to the hot gas layer formed in the upper part of the building at the early stage of fire.

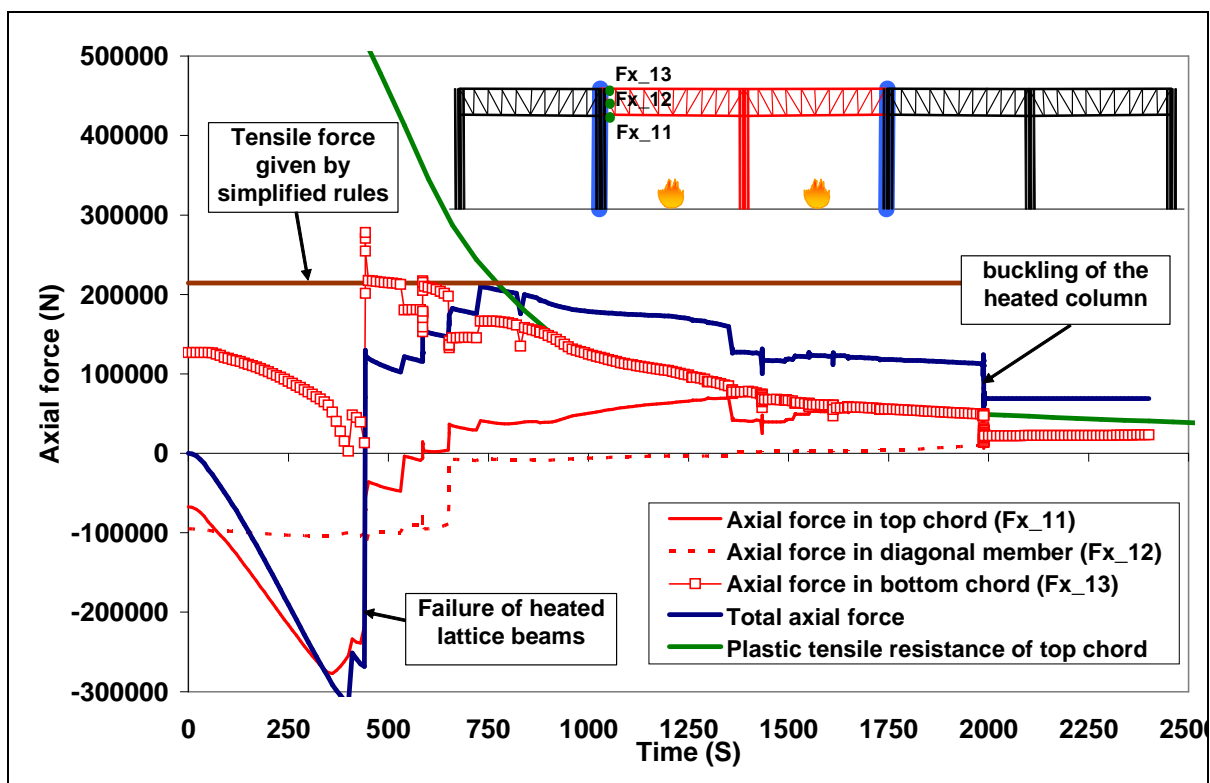


Figure 4-24 : Axial forces induced in heated lattice beams

## 5. Standardised Solution for Industrial Halls

### 5.1. Simplified rules for expansion and collapsing phase

The failure mode of steel framework of storage buildings depends on the resistance of the cold part of the structure, the resistance of the part of structure submitted to fire and on displacements generated at the compartment ends. These displacements may become the main criteria to evaluate the fire behaviour of partition walls and façade elements.



So design methods developed for industrial building with steel structure must allow:

- **On the one hand, to check the stability of the cold parts of the structure under the effect of the collapse of the heated part, and**
- **On the other hand to provide displacements induced at the fire compartment ends during both expansion phase and collapsing phase.**

As these calculations are performed on cold structures, so they can be assessed using room temperature design tools for structure analysis, provided that the forces induced by the behaviour of the heated structure can be evaluated.

Simple methods allowing a safe evaluation of these forces are given hereafter. Two types of steel structures are covered by these methods, namely:

- **Portal steel frames with cross section in standard H or I hot rolled profiles**
- **Steel frames making up lattice beams with columns in standard H or I hot rolled profiles**

#### 5.1.1. Catenary method and tension force

- **The numerical modelling and real fire observations show that steel frame behave as a chain under fire situation if columns are stable. For this reason, the evaluation of tensile force can be estimated in such a way to be as accurate as possible with the catenary theory.**

The following figure shows a general case of chain modelling, for which the two points of support are not at the same height.

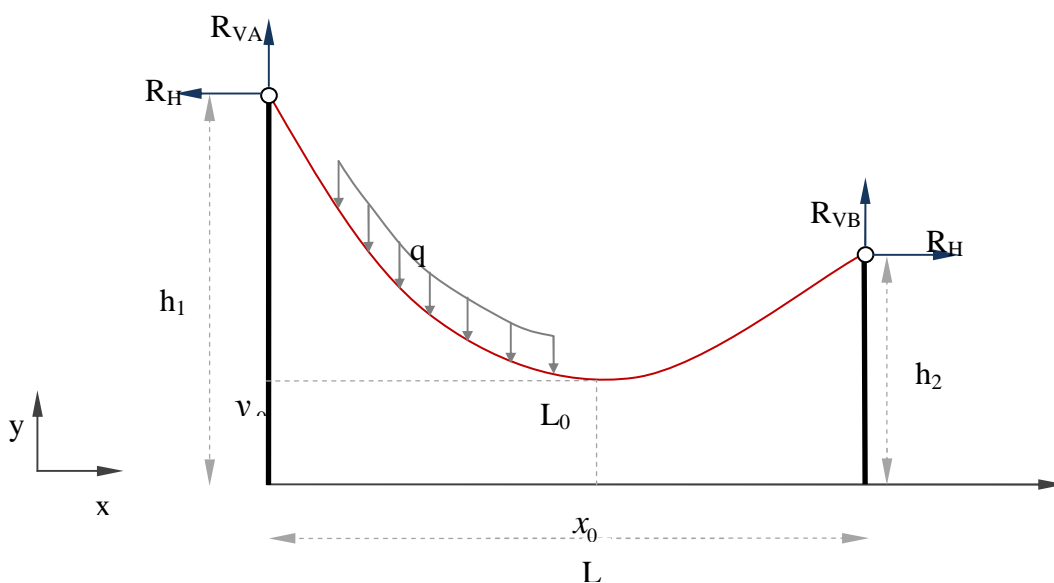


Figure 5-1: Parameters of the catenary.



According to catenary theory the horizontal tension  $R_H$  at the top of the frame is derived from the expression:

$$R_H = q \cdot a \quad \text{Equation 5-1}$$

Under constraints:

$$\begin{cases} x_0 < L, \text{ with } x_0 \text{ is such that } h_2 - h_1 = a \left( \cosh \left( \frac{x_0}{a} \right) - \cosh \left( \frac{L - x_0}{a} \right) \right), \\ y_0 > 0, \text{ with } z_0 = h_1 + a \left[ 1 - \cosh \left( \frac{x_0}{a} \right) \right], \\ h_2 > h_{2,min} \text{ with } h_{2,min} = h_1 - \sqrt{L_0^2 - L^2}. \end{cases}$$

Equation 5-2

In Equation 5-1,  $q$  is the linear load and  $a = \frac{L}{2X}$ , is a parameter function of  $X$  which can be estimated by,

$$\sinh(X) = \kappa \cdot X, \text{ where } \kappa^2 = \frac{L_0^2 - (h_1 - h_2)^2}{L^2} \quad \text{Equation 5-3}$$

Catenary parameters are as follows:

$h_1, h_2$  - heights of support columns

$L$  - distance between columns

$x_0, y_0$  - coordinates of the lowest point of the chain

$R_H, R_V$  - horizontal and vertical reactions (see Figure 5-1)

$L_0$  - length of the chain, given by the implicit equation,

$$L_0 = \frac{2R_H}{q} \sinh \left( \frac{qL}{2R_H} \right), \quad \text{Equation 5-4}$$

During fire, different situations can be met. Indeed, columns are considered as fixed at support and, under fire conditions, the unprotected intermediate column in the same cell determines the parameters of the catenary and then the generated forces in the top of columns. The following figures illustrate this connection in case of frames where two spans are heated.



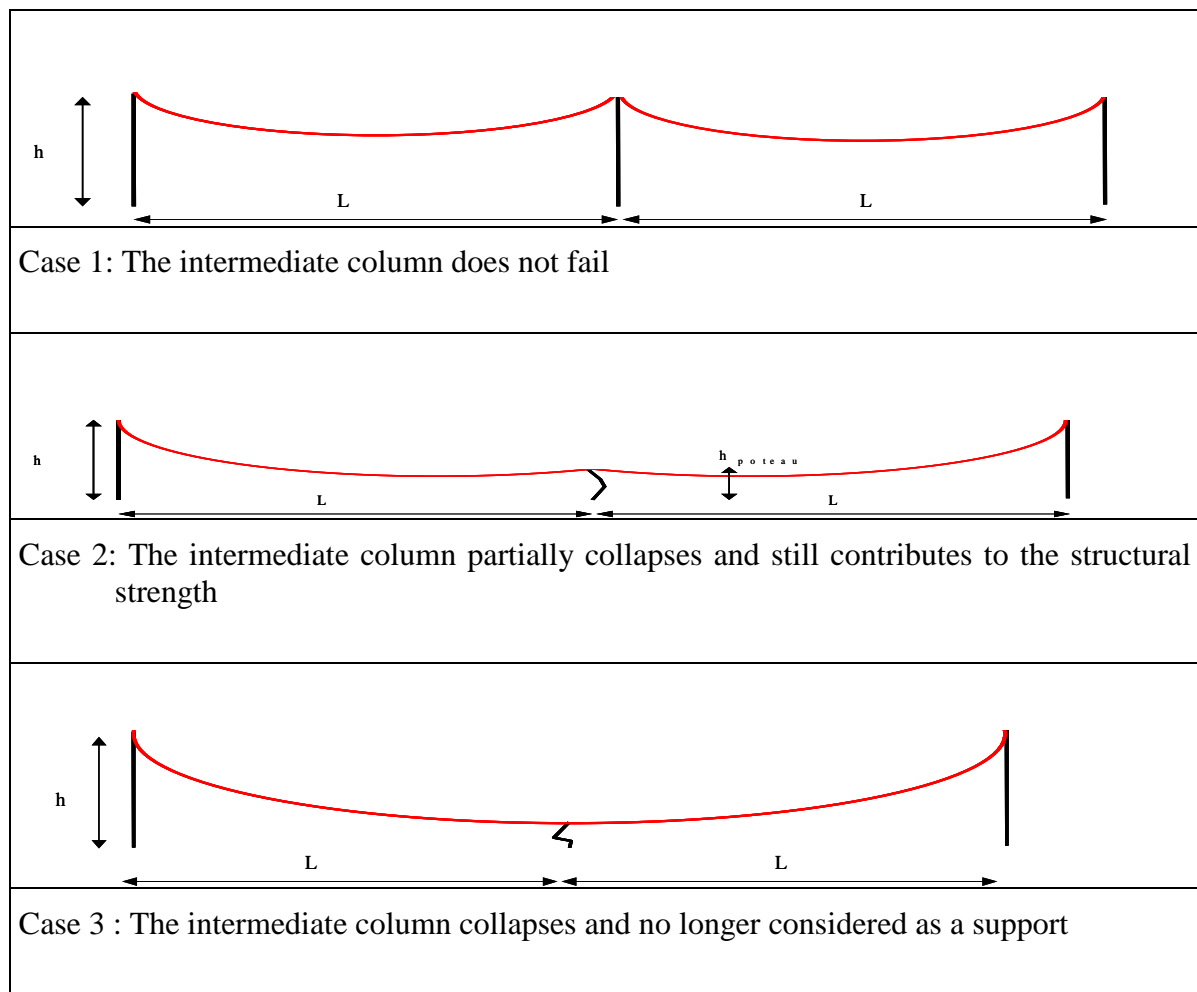


Figure 5-2: Different cases to be considered in the top load estimation.

The effective computational procedure consists of performing an iterative calculation of the tensile horizontal force according to the implicit Equations 5-1, 5-3 and 5-4 under constraints defined by Equation 5-2. For the above different situations (Figure 5-2) and for several constructive configurations, calculations have been performed in order to evaluate the horizontal tensile forces at compartment ends. It is obvious that the third case is the most unfavourable one and corresponding results has served as reference for the proposed simple method (Section 5.1.2 for portal steel frames and Section 5.1.3 for lattice frames). The catenary results for the case 3 are resumed in the table below;

	Load (KN/m)	Height (m)	Horizontal tensile force from catenary calculation (KN) Case 3
Span 20 m	2,16	7,5	102.79
		12,5	102.79
	2,88	7,5	138,8
		12,5	138,8



	3,6	7,5	173,49
		12,5	173,49
Span 30 m	2,16	7,5	156,14
		12,5	156,14
	2,88	7,5	208,19
		12,5	208,19
	3,6	7,5	260,24
		12,5	260,24
Span 40 m	2,16	7,5	208,19
		12,5	208,19
		20	208,19
	2,88	7,5	277,59
		12,5	277,59
		20	278,26
	3,6	7,5	342,86
		12,5	342,86
		20	342,86

Table 5.1: Horizontal tensile forces according to catenary theory.

### 5.1.2. Portal steel frames with cross section in standard H or I hot rolled profiles

Explanations given in this section deal with Section 4 of the design guide [1] and concern only the configurations when fire walls are perpendicular to portal frames of the storage building. When fire walls are parallel to portal frames, the risk of collapse towards the outside and progressive collapse (between different fire compartments) can be simply avoided with regard to several recommendations suggested in [2]. As well for expansion as for the collapse phase two fire configurations have been considered namely,

- ⇒ Fire compartment in the middle of the storage building (see Figure 5-3);
- ⇒ Fire compartment at the end of the storage building (see Figure 5-4);

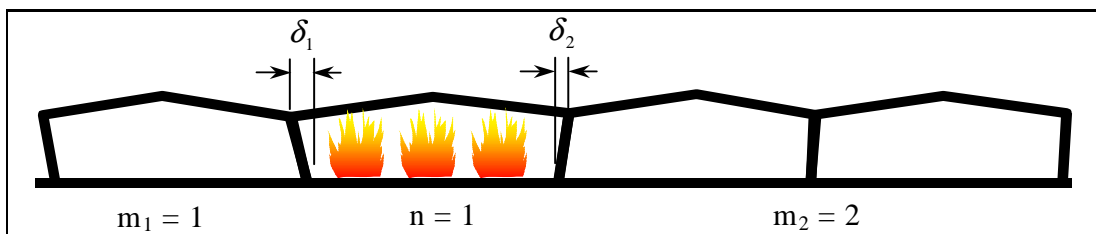


Figure 5-3 : Fire located in a cell at the middle of the building

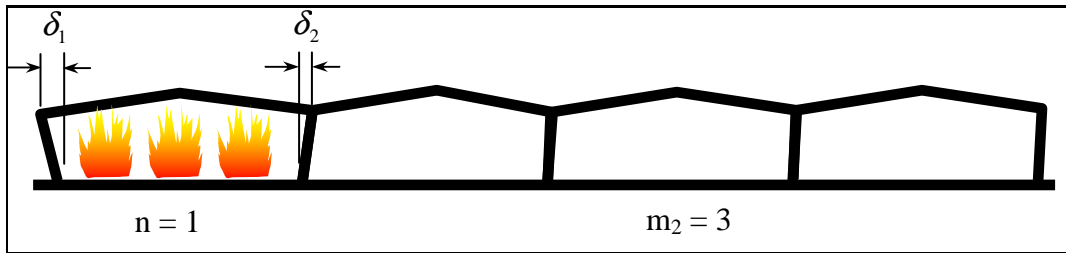


Figure 5-4 : Fire in a compartment at the end of the storage building

#### 5.1.2.1. Collapsing phase: horizontal tensile force and displacement induced

The design guide gives, in Eq. (4-8) (cf. [1] Section 4), the horizontal tensile force to be used in order to evaluate the stability of the cold parts in case of fire situation. One recalls here for convenience this tensile force,

$$F_t = c_p n_{eff} q \ell, \quad \text{Equation 5-5}$$

where

$q$  is the vertical applied load given by Equation 4-7,

$\ell$  is the span of one heated bay,

$n_{eff}$  is a coefficient given by Equation 4-5 as a function of the number of heated bays and the two studied fire configurations (fire in the middle or in the end of compartment),

$c_p$  is a coefficient given according to Equation 4-3 for different slope values.

It is to note that for intermediate slope values linear interpolation may be performed. The coefficient  $c_p$  is adjusted so that the horizontal tension force given by the simple method (Equation 5-5) is well correlated with catenary results (Table 5.1). Figure 5-5 gives the correlation between loads calculated using the catenary theory (cf. Table 5.1) and loads calculated using the simple method.

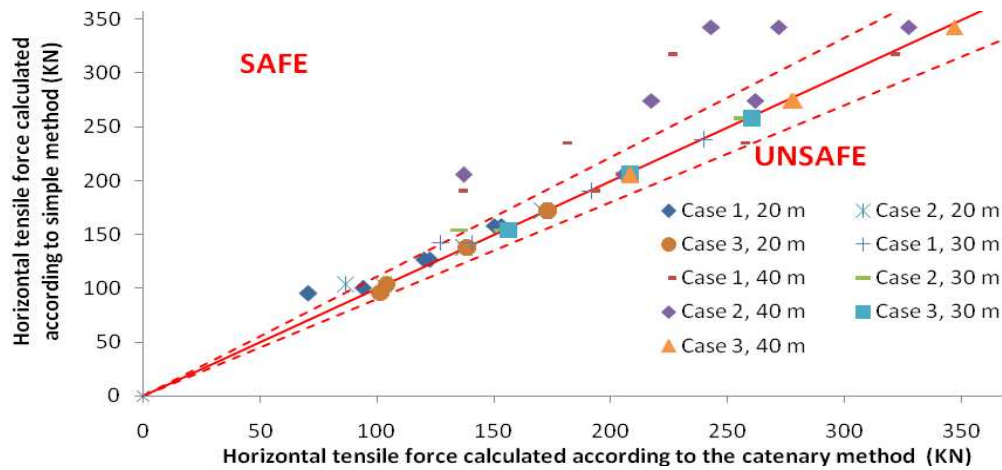


Figure 5-5 : Correlation between horizontal tensile forces calculated using the catenary method (Equations 5-1, 5-2 and 5-3) and those calculated using the proposed simple method.

The Figure 5-6 gives the correlation between loads determined by numerical simulations (where no failure of the cold parts of the structure occurred) and loads calculated according to the simple method.

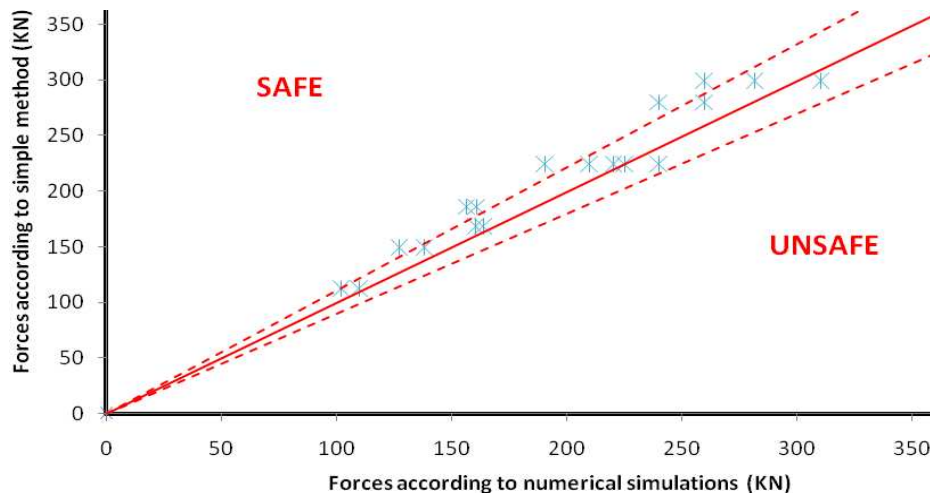


Figure 5-6: Correlation between tensile forces calculated using numerical simulations and those calculated using the proposed simple method.

#### 5.1.2.2. Expansion phase: force induced by thermal expansion

For the expansion phase, the only performance criteria to be checked concerns displacements induced at the ends of fire compartment and then forces generated by the thermal expansion of the beam.

When fire occurs in a compartment in the middle of the building, generated force can be given as a function of the slope of the roof according to,

$$F_p = c_p n q \ell$$

Equation 5-6

where



$n$  is the span number of the compartment submitted to fire. The number of spans “ $n$ ” to take into account in design is limited to 2, even if the total number of spans in the fire compartment is higher than 2;

$m_i$  is the span number of the neighbouring cold compartments;

$q=G+0,2S_n$  - is the linear load on roof [N/m] (equal to the load density multiplied by the spacing between frames) applied on the beam and calculated in fire situation (where  $G$  is the permanent load including self-weight of the steel frame and the equipment overloads and  $S_n$  is the snow load);

$\ell$  - is the span length [m];

$c_p$  - is an empirical coefficient (function of the slope of the roof) according to Table 5.2 (for intermediate values of slope, linear interpolation may be used),

Slope of the roof	$c_p$
0%	1,19
5%	1,16
10%	1,10

Table 5.2 : Slope values  $C_p$

- **For simplification reasons, the coefficient  $c_p$  in Equation 5-1 is taken the same as for Equation 5-5, which corresponds to the evaluation of the horizontal tensile force induced by the span deflection under fire.**

When fire occurs in a compartment at the end of the storage building (see Figure 5-4), pushing force induced at the compartment ends can be obtained in the following way:

$$F_p = K_t n c_{th} \ell, \quad \text{Equation 5-7}$$

where:

$$K_t = \frac{K_1 K_2}{K_1 + K_2}, \text{ with } K_1 \text{ and } K_2 \text{ equivalent stiffness for lateral displacements}$$

$\delta_1$  and  $\delta_2$  of steel frame;

$n$  is the span number of the cell submitted to fire;



$c_{th}$  is the reduction factor which corresponds to a thermal expansion at a temperature of 740°C. Values of  $c_{th}$  as a function of the slope are given in Table 5.3. For intermediate values of slope, linear interpolation may be used;

$\ell$  is the span length [m];

Slope of the roof	$c_{th}$
0%	0,01
5%	0,011
10%	0,015

Table 5.3 : Slope values  $C_{th}$

The value of  $K_1$  is defined as the lateral stiffness of the steel frame of the fire compartment which can be evaluated as follows:

$$K_1 = \begin{cases} 0.065 k & \text{for } n = 1 \\ 0.13 k & \text{for } n = 2 \\ 0.13 c k & \text{for } n > 2 \end{cases} \quad \text{Equation 5-8}$$

When the span number of the heated cell  $n$  is higher than 2,  $K_1 = 0,13ck$ , with  $k$  as defined in Equation 5-10 and  $c$  determined according to Equation 5-11 with  $m = n - 1$ .

The value of  $K_2$  is defined as the lateral stiffness of the steel frame of cold parts of the structure.  $K_2$  can be calculated using standard structural analysis software or, as for  $K_1$  formulas explained of the paragraph below.

#### Frame lateral stiffness evaluation

In practice, especially for the unequal steel frames, displacements will be calculated directly using standard software for structural analysis. For usual steel frames (constant range, even standard steel profiles from one span to another), equivalent lateral stiffness  $K_i$  can be calculated in an approximate way using the Daussy's relations [3]:

For  $m_i = 1$  :

$$K_i = k \quad \text{Equation 5-9}$$

with:

$$k = \frac{\alpha}{1 + 2\alpha} \cdot \frac{12EI_c}{(h + f)^3} [\text{N/m}], \quad \text{and} \quad \alpha = \frac{I_b}{I_c} \cdot \frac{h + f}{\ell} \quad \text{Equation 5-10}$$



where (see Figure 5-7):

$h$  - is the height of the portal frame [m];

$\ell$  - is the span length [m];

$I_b$  - is the second moment of area of the beam [ $\text{m}^4$ ];

$I_c$  - is the second moment of area of columns [ $\text{m}^4$ ];

$E$  - is the modulus of elasticity of steel for normal temperature [ $\text{N/m}^2$ ];

For  $m_i \geq 2$  :

$$K_i = ck \quad \text{with} \quad c = 1 + \sum_{i=2}^m \frac{i}{2} \frac{\alpha}{1 + 2i\alpha} \quad \text{Equation 5-11}$$

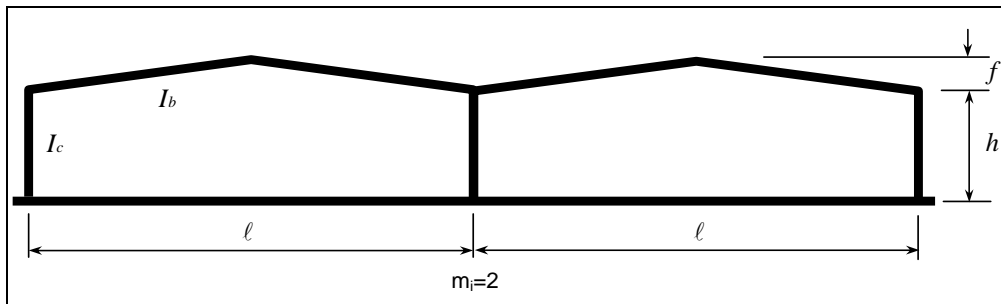


Figure 5-7 : Definition of parameters of cold parts

The Figure 5-8 shows the correlation between lateral displacement (and then lateral stiffness) calculated with structure software and that calculated using Eq. 5-3. Results show that the used formula gives, except for some cases, safe design values.



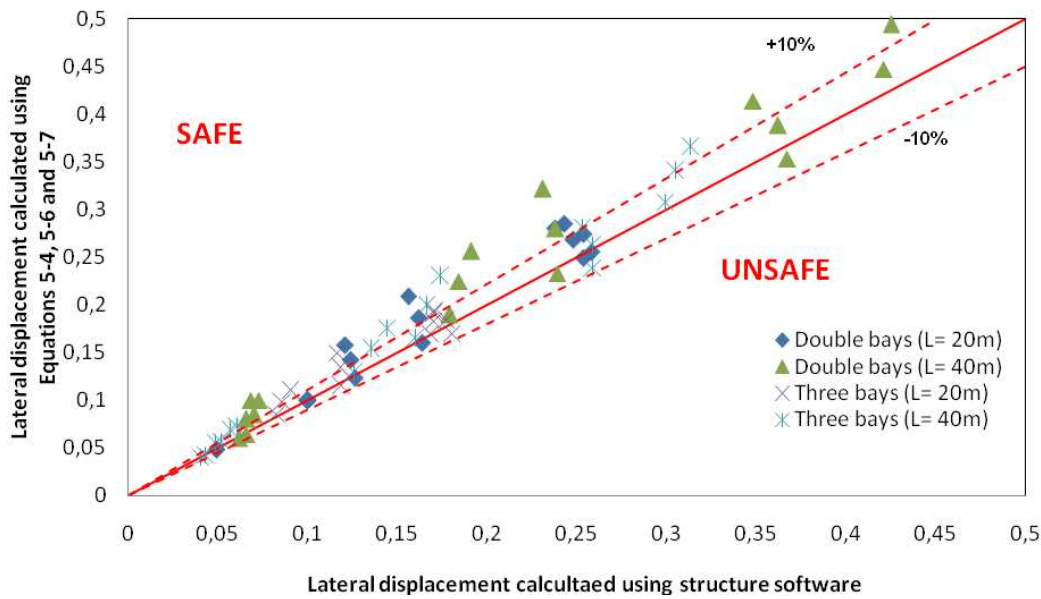


Figure 5-8: Correlation between lateral displacements calculated using structure software and those calculated using the simple method (Eqs. 5-9, 5-10 and 5-11)

This correlation can be improved (see Figure 5-9) by modifying the parameter  $\alpha$  such that Eq.(5-10) is replaced by

$$k = \frac{\alpha}{1+2\alpha} \cdot \frac{12EI_c}{(h)^3} \text{ [N/m]} \quad \text{and} \quad \alpha = \frac{I_b}{I_c} \cdot \frac{h+f}{\ell} \left( 1 - \frac{f}{0.6h} \right) \quad \text{Equation 5-12}$$

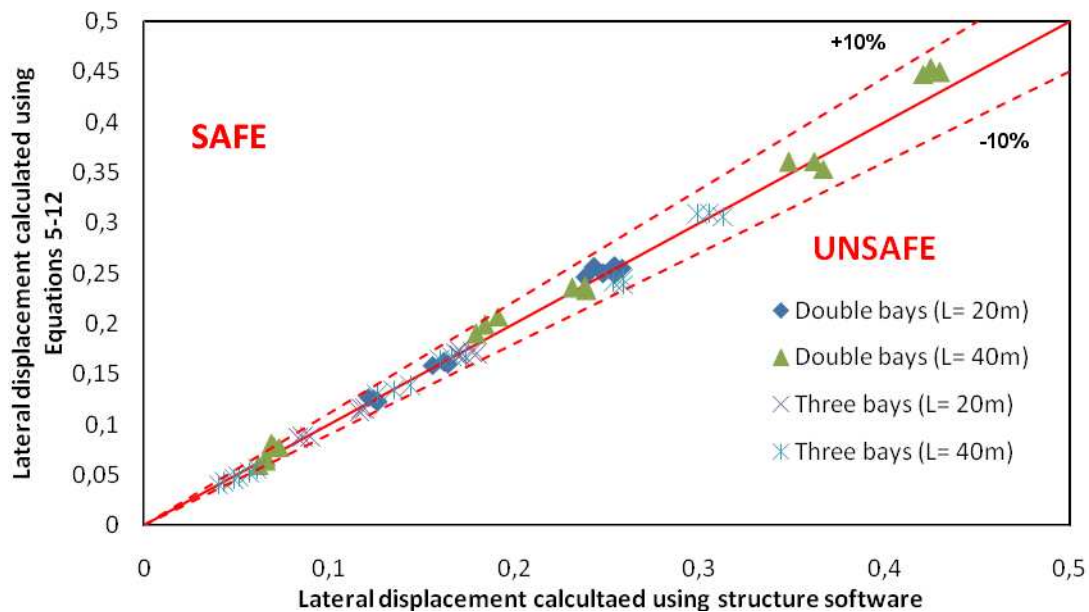


Figure 5-9: Correlation between lateral displacements calculated using structure software and those calculated using the simple method (Eq. 5-12)



### 5.1.2.3. Displacement at fire compartment ends

When the fire occurs in a compartment of the building, displacements  $\delta_i$  [m] induced at the compartment ends (see Figure 5-4) can be obtained according to

$$\delta_i = \begin{cases} \frac{\max\{F_p, F_t\}}{K_i}, & \text{at the neighbouring cold part,} \\ \frac{F_p}{K_i}, & \text{at the end of the frame.} \end{cases} \quad \text{Equation 5-13}$$

where

$F_p$  and  $F_t$  are forces induced by thermal expansion and tensile force given according to Equations 5-5 and 5-7 respectively.

$K_i$  is the equivalent lateral stiffness of the steel frames of cold compartments [N/m].

Displacements obtained allow checking that both facade and partition elements are compatible with the displacements developed at the ends of the fire compartment in order to avoid the collapse towards outside and the progressive collapse between different fire compartments.

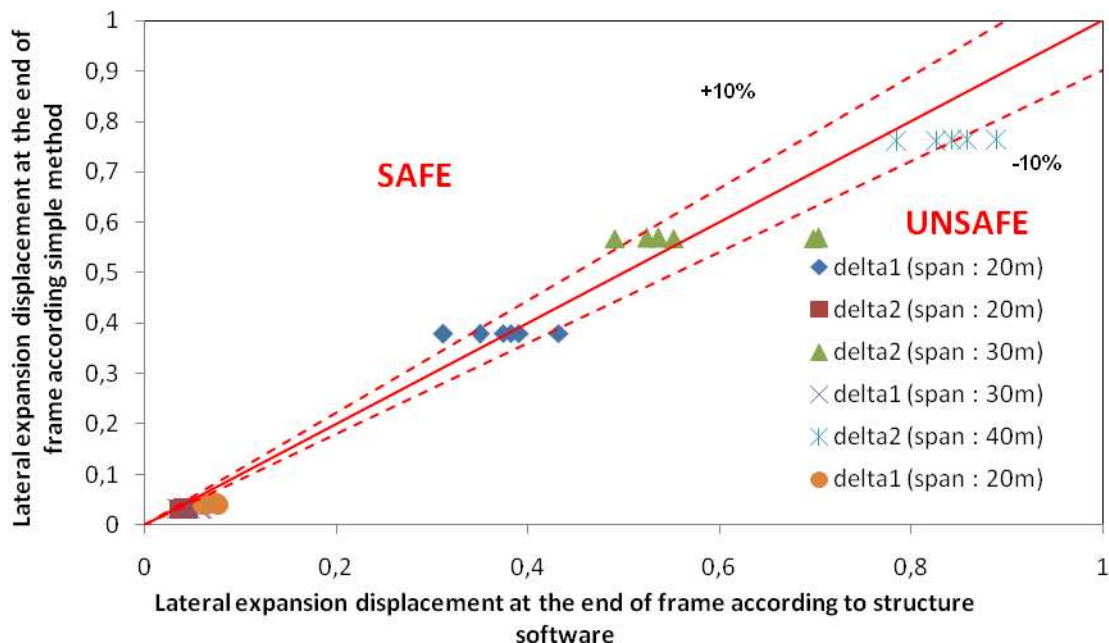




Figure 5-10: Correlation between lateral displacements calculated using structure software and those calculated using the simple method (Equation 5-13)

### 5.1.3. Steel frames with lattice beams and columns in standard H or I hot rolled profiles

#### 5.1.3.1. Expansion phase: Displacement at the fire compartment ends

For the expansion phase, the checking of the fire behaviour of lattice structures with respect to the fixed objectives only requires to evaluate maximum displacements at the ends of the fire compartment.

Lateral displacements  $\delta_i$  induced at the top of columns located at the compartment ends can be obtained using the following expression:

$$\delta_i = 0,009 \cdot \frac{K_t}{K_i} \cdot \sum_{i=1}^n \ell_i . \quad \text{Equation 5-14}$$

where:

$\ell_i$  is the length of the heated span  $i$  [m];

$n$  is the span number of the fire compartment;

$K_t = \frac{K_1 K_2}{K_1 + K_2}$  [N/m], where  $K_1$  and  $K_2$  are the equivalent stiffness of steel structures for the lateral displacements  $\delta_1$  and  $\delta_2$  (see Figure 5-11).

The partial coefficient (0.009) in Equation 5-14 corresponds to a thermal expansion at a temperature of 650°C. This coefficient is determined performing thermo-mechanical simulations which show that the collapse of lattice beams occurs at a maximum temperature of 650°C.

It should be noted that equivalent stiffness of the steel frameworks of the cold parts of the structure must be evaluated using standard software for structural analysis.

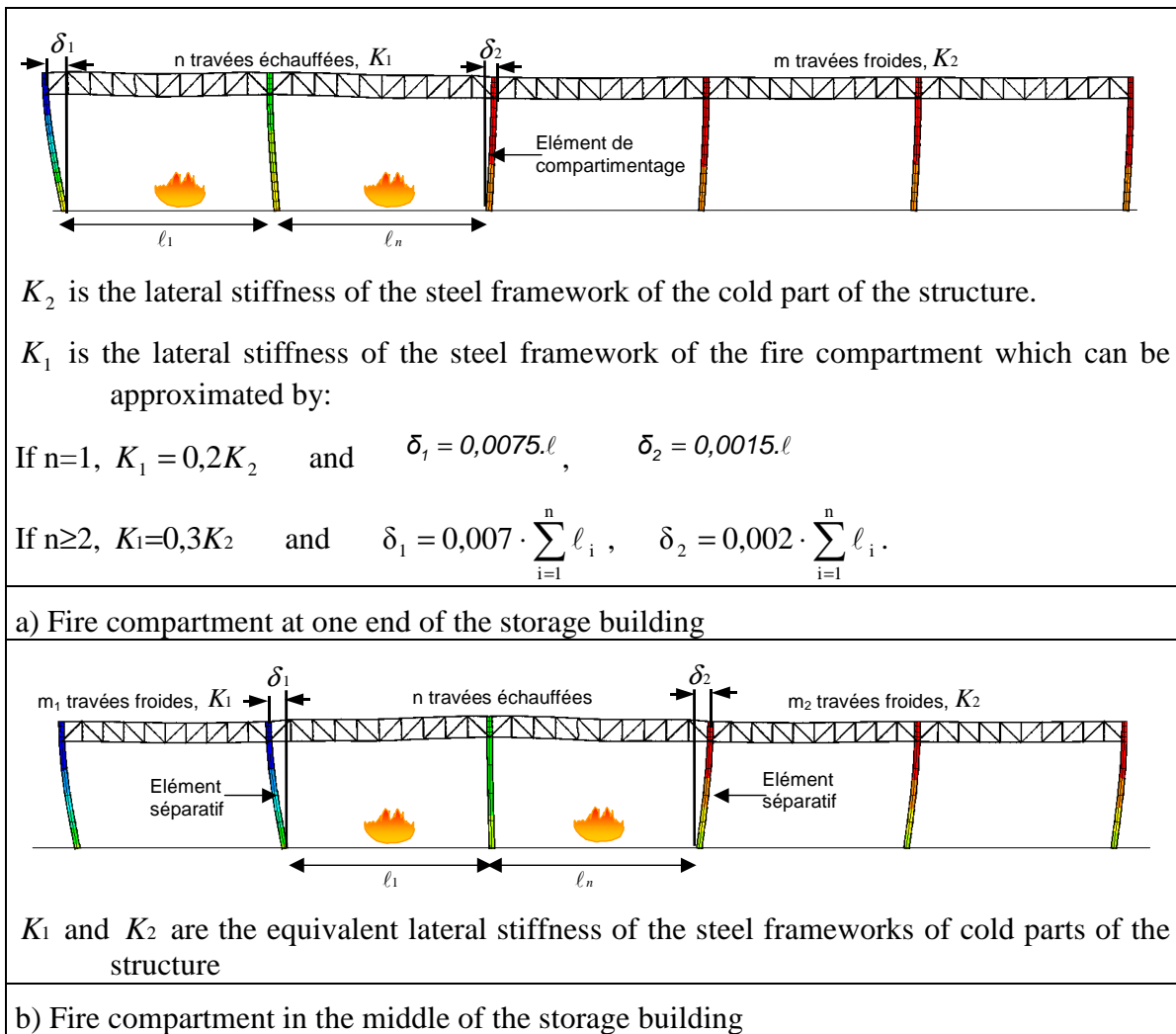


Figure 5-11 : Definition of lateral stiffness  $K_1$  and  $K_2$

Partial coefficients in previous expressions have been determined such that one obtains a good correlation between results of numerical simulations and those of the simple method. Figure 5-12 and Figure 5-13 show the correlation between the expansion displacements (for different structural configurations) at the top of the column calculated using numerical modeling and those calculated according to the simple method for the case a and b respectively (see Figure 5-11 for the two cases).

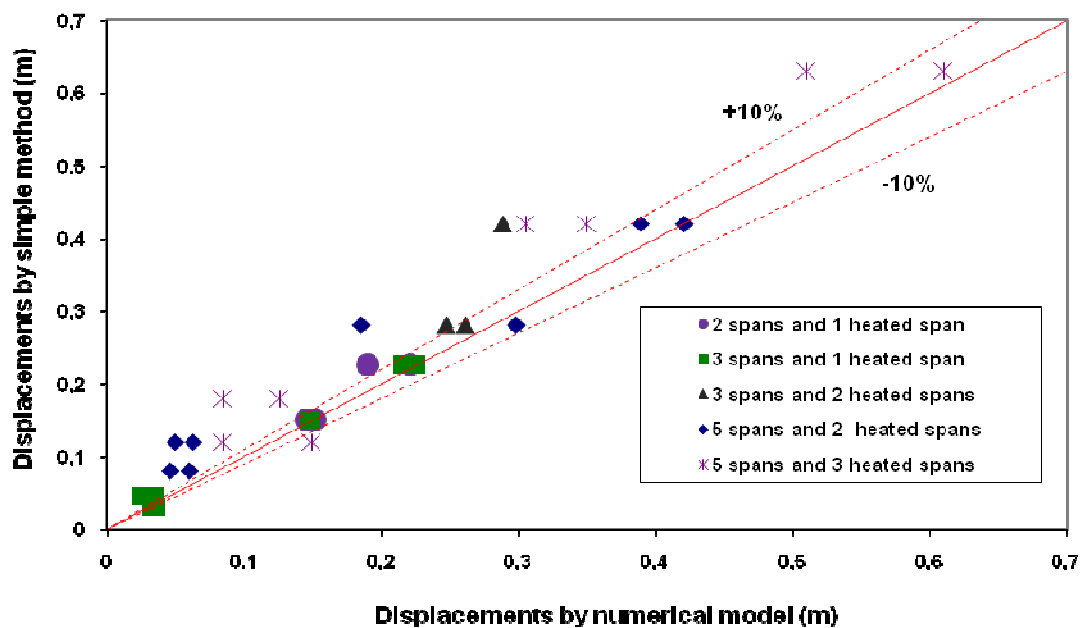


Figure 5-12 : Correlation between expansion displacements calculated using numerical modeling and those calculated using the simple method (Equation 5-14 and case a of Figure 5-11)

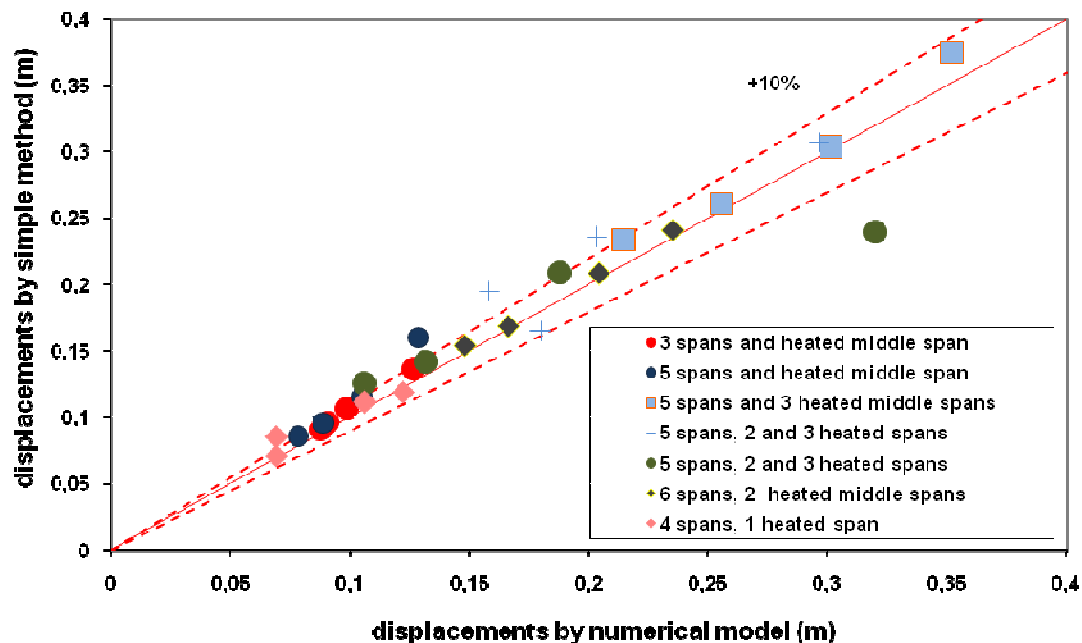


Figure 5-13 : Correlation between expansion displacements calculated using numerical modeling and those calculated using the simple method (Equation 5-14 and case b of Figure 5-11)

### 5.1.3.2. Collapsing phase: Stability of cold parts of the structure and displacement at the fire compartment ends



During the collapsing phase, chord members of heated lattice beams pass from a compression state to a simple tensile state. Then beams behave as chain subjected to uniform loads.

In the case of a simple heated span located at the middle of the building (see Figure 5-14), the horizontal tensile force applied at the ends of the fire compartment can be obtained from:

$$F = c_p \cdot q \cdot \ell \quad \text{Equation 5-15}$$

where:

$q=G+0,2S_n$  is the linear load on roof [N/m] (equal to the load density multiplied by the spacing between frames) applied on the beam and calculated in fire situation (where  $G$  is the permanent load including self-weight of the steel frame and the equipment overloads and  $S_n$  is the snow load);

$\ell$  is the length of the span [m];

$c_p$  is a coefficient taken as 1.45.

- **It is to note that the value of the coefficient  $c_p$  is calculated so that one obtains a good correlation between results of numerical modeling and those calculated using the simple method (see Figure 5-15 and Figure 5-17).**

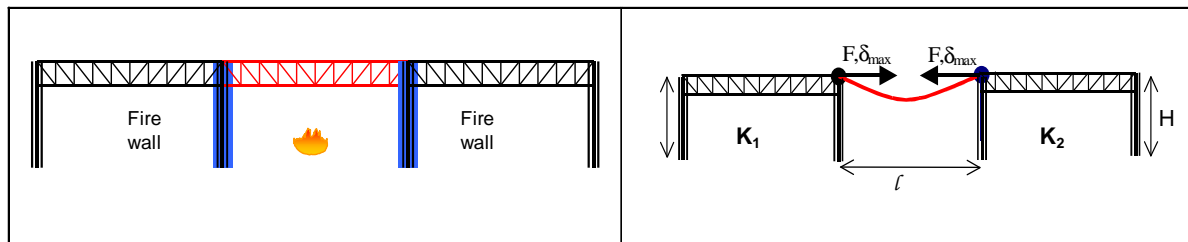


Figure 5-14 : Fire compartment in the middle of the storage building

The Figure 5-15 gives the correlation between tensile forces at the top of columns calculated according numerical simulations and those calculated using simple method according to Equation 5-14.

From previous maximum force  $F$ , displacements  $\delta_{max,i}$  at the top of the columns supports of the partition elements can be calculated in the usual way:

$$\delta_{max,i} = F / K_i \quad \text{Equation 5-16}$$

where



$K_i$  is the lateral stiffness of the examined cold part of the structure.

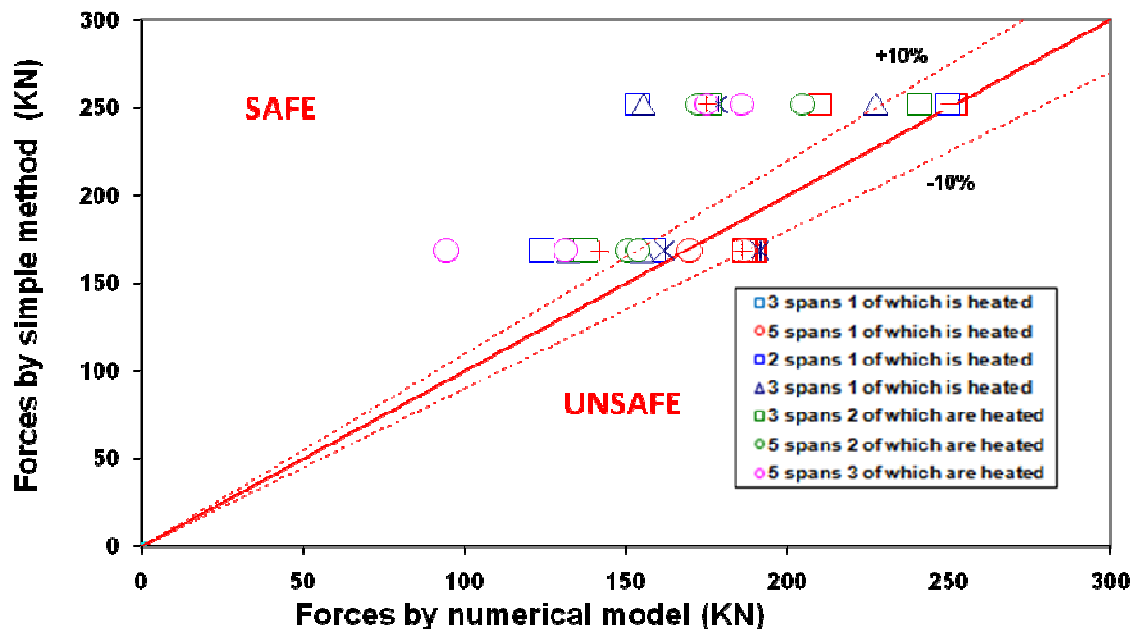
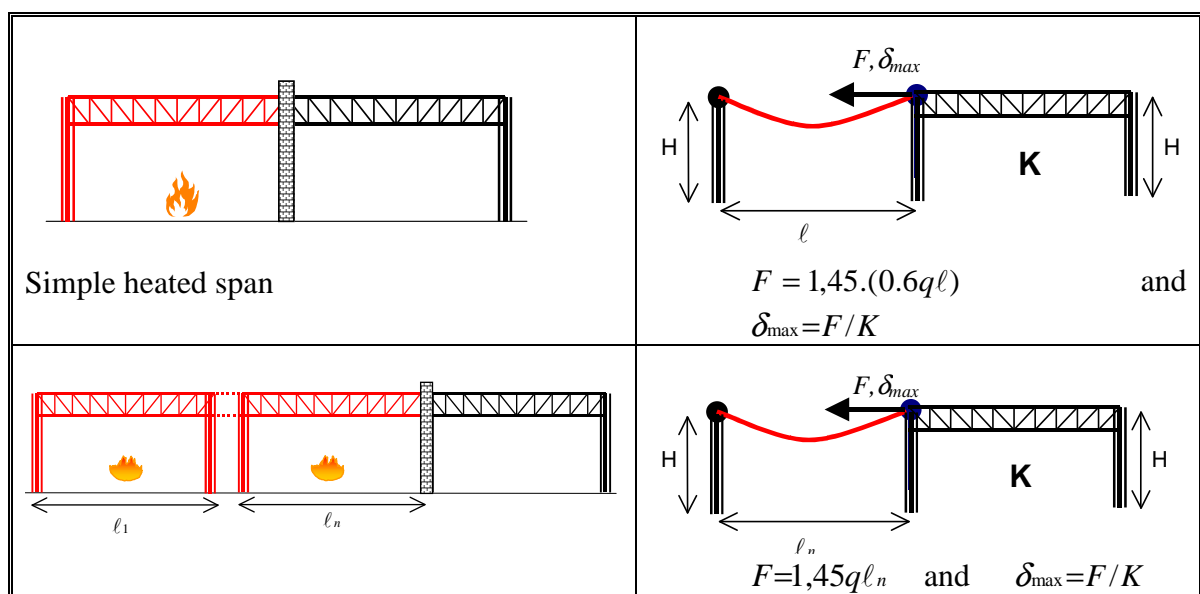


Figure 5-15 : Correlation between forces calculated by numerical methods and those calculated by the simple method (Eq. 5-14)

In the case of different partitioning (several heated spans; edge span heated) displacements at the top of columns supports of the façade or partition elements and forces transmitted to the cold parts of the structure can be calculated by applying the previous relations to the heated span(s) of the fire compartment close to cells not submitted to fire as indicated in Figure 5-16.







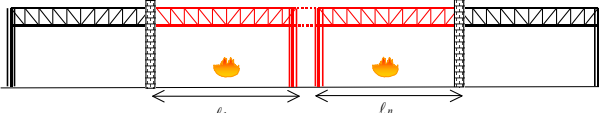
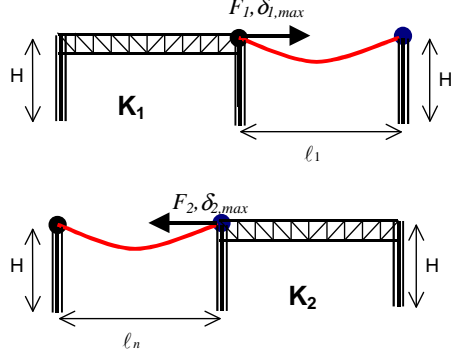
Case of n heated spans	
a) Fire compartment at the end of the storage building	
	 <p> <math>F_1 = 1,45q\ell_1</math> and <math>\delta_{1,max} = F_1 / K_1</math>  <math>F_2 = 1,45q\ell_n</math> and <math>\delta_{2,max} = F_2 / K_2</math> </p>
b) Fire compartment at the middle of the storage building;	

Figure 5-16 : Displacements and forces transmitted to cold parts of the structure

The Figure 5-17 gives the correlation between displacement calculated using numerical simulations and those calculated using the simple method (Equation 5-16).

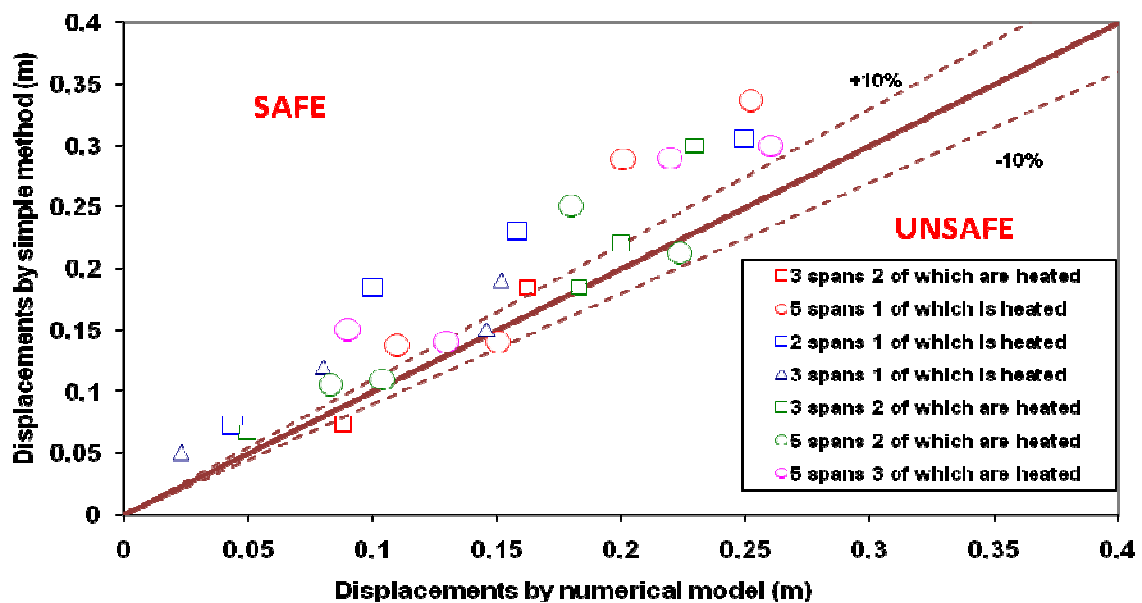


Figure 5-17 : Correlation between displacements calculated using numerical modelling and those calculated using the simple method (Equation 5-16)

## 5.2. Simple model for expansion phase



More accurate design method, therefore less simple of use, is presented hereafter for expansion phase. This method allows calculating maximum horizontal displacements at fire compartment ends.

### 5.2.1. Lattice structures with columns in standard H or I hot rolled profiles.

The method given hereafter aims at evaluating by an incremental calculation maximum displacements induced at the ends of a fire compartment during the expansion phase, taking into account the evolution and the distribution of temperatures as function of time, as well as their effects on the thermal properties (thermal expansion) and mechanical properties (reduction factors for yield strength and Young's modulus) of steel.

It should be noted that maximum displacements to use in the design of steel frameworks are those obtained when heated lattice beam fails, i.e. when the buckling resistance of one of the elements making up the beam is reached in fire situation.

The following procedure can be followed for the determination of maximum displacements:

- **Step 1: Choice of fire scenarios: i.e. choice of steel members (lattice beams) which will be heated. These scenarios are defined in accordance with the arrangement of the storage building (structure and partitioning) as illustrated in the Figure 5-18;**

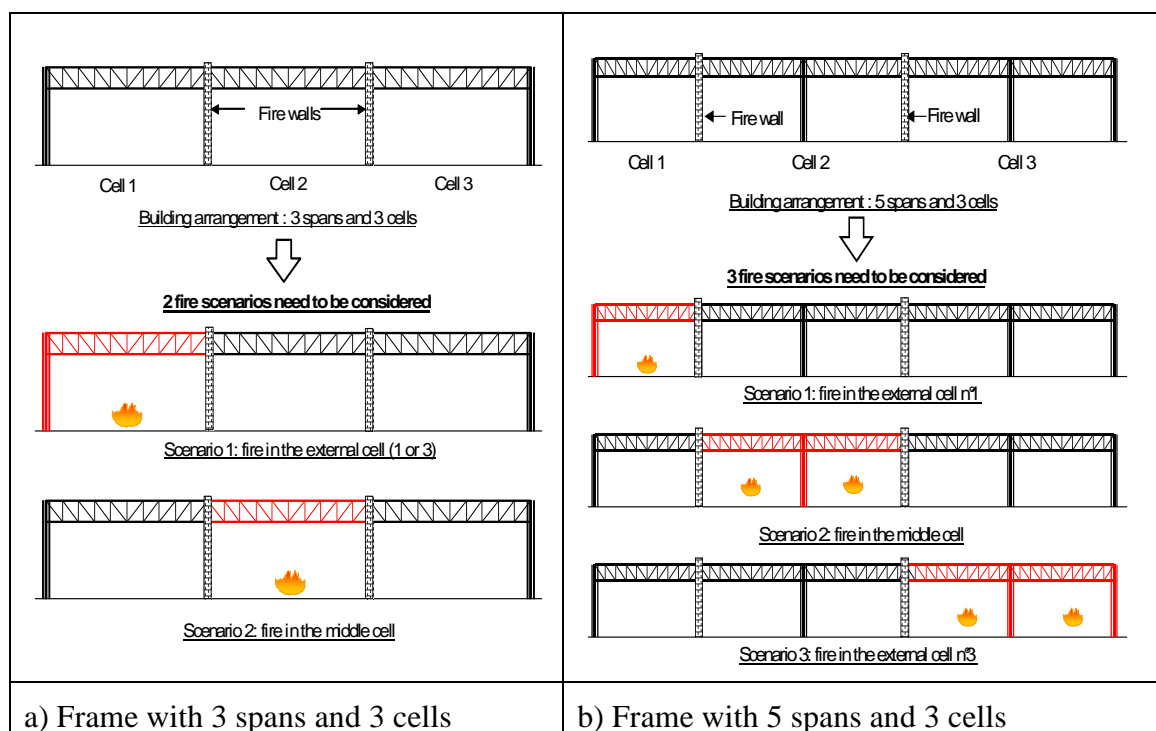


Figure 5-18 : Fire scenarios according to the arrangement of the storage building

- **Step 2: Calculation of temperatures in steel members making up the lattice beams in the fire compartment. Temperature distribution is**



**assumed to be uniform along the length and within the cross-section of steel profiles. So, no thermal gradients across section or along element length are considered.**

- **Step 3: Checking of fire resistance of heated lattice beams. From the temperature fields previously established, failure time of heated lattice beams which lead to the end of expansion phase should be predicted. More precisely, for each temperature level, the stability of various steel profiles making up the lattice beams (horizontal chords, vertical elements and compression diagonals) should be checked calculating:**

- ⇒ On the one hand, the design buckling resistance of these elements in fire situation (according to part 1-2 of Eurocode 3 [4]);
- ⇒ On the other hand, internal forces introduced in these elements in case of fire.

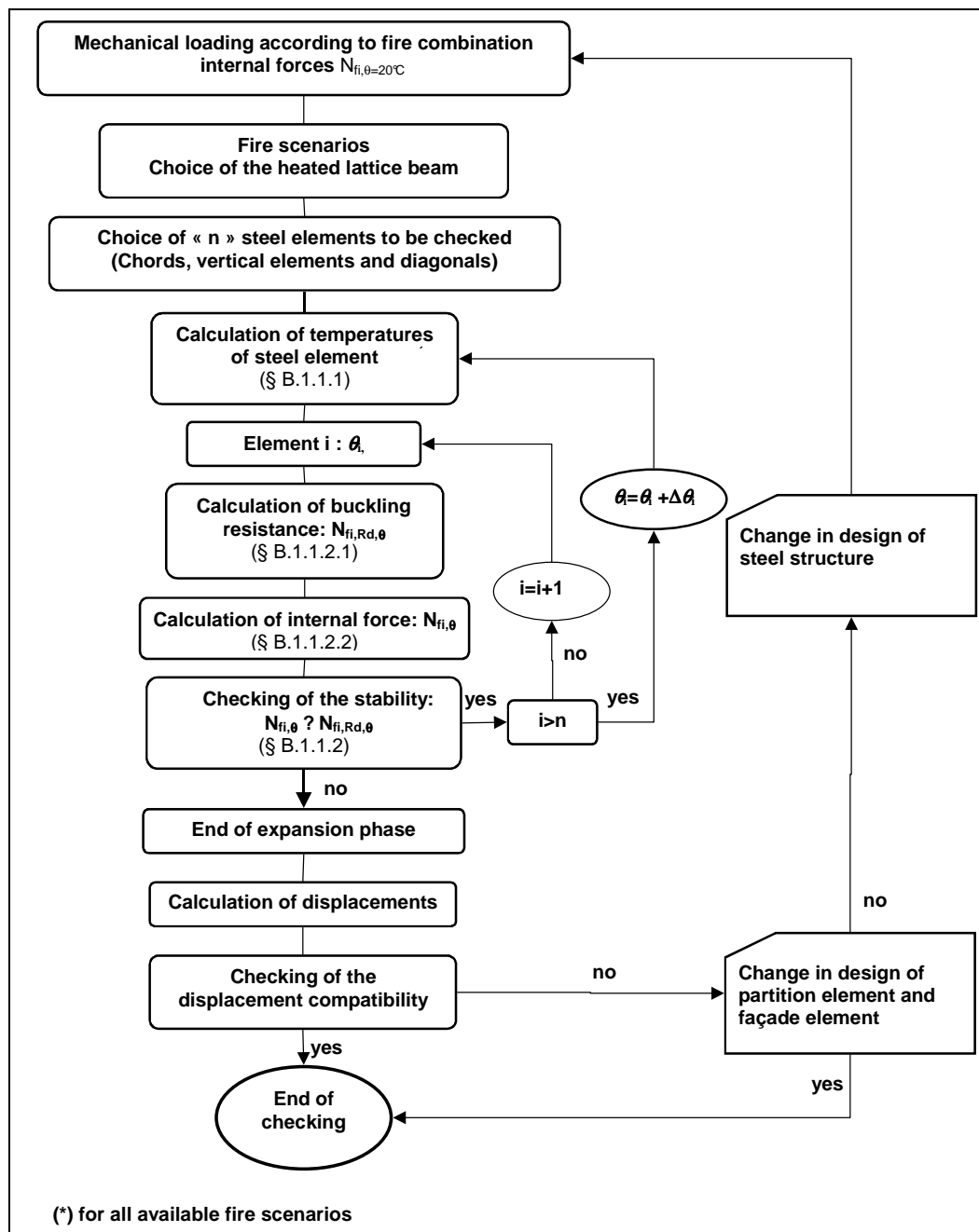


Figure 5-19 : Application flowchart of simple model for expansion phase

- **Step 4: Calculation of the maximum displacements at the top of columns supports of both partition and facade elements. Once these displacements obtained, it's possible to check the design for displacement compatibility between steel frame and partition walls.**

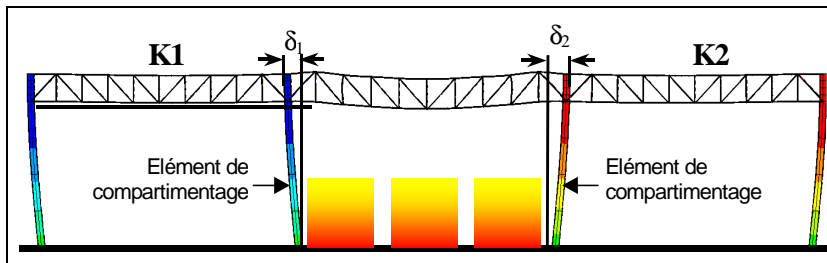
Application flowchart of the simple model is summarized in the Figure 5-19.

Two situations need to be considered, namely:



- **Fire compartment in the middle of the storage building;**
- **Fire compartment at the end of the storage building.**

#### 5.2.1.1. Fire compartment in the middle of the storage building: simple heated span



*Figure 5-20 : Fire compartment in a middle cell*

### Determination of temperatures in steel profiles:

Due to the difference between the section factor  $A_m/V$  of the several steel profiles making up lattice beams, the temperature level reached in each type of these elements must be calculated.

Temperatures in steel elements should be calculated according to the simplified method given in Part 1-2 of Eurocode 3 as function of time and section factor [4].

The calculation procedure summarised on Figure 5-19 is then performed taking into account successively the temperatures previously calculated.

The simple model is applied step by step until the failure of the heated lattice beam using the following temperatures.

Step	Chords	Diagonals	Vertical elements
1	201	265	359
2	258	335	435
3	314	399	496
...	...	...	...
10	604	661	693
...	...	...	...
n	...	...	...

*Table 5.4 : Step by step procedure*

### Checking of the fire resistance of heated lattice beams: End of the expansion phase

End of expansion phase occurs when one of the steel profiles making up the heated lattice beam (horizontal chord members, vertical members or diagonals) fails as a result of the progressive increase of internal forces due to the axial restraint against thermal expansion induced by the cold parts of the structure.



Also, to evaluate the maximum displacements to be used in the design method, it is necessary to estimate the temperature reached by the horizontal chord members at the failure time of heated lattice beam. This temperature is evaluated step by step by checking for each steel member the condition where the internal force applied to the element reaches its design buckling resistance in compression, i.e.:

$$N_{fi,\theta} = N_{fi,Rd,\theta} \quad \text{Equation 5-17}$$

where:

$N_{fi,Rd,\theta}$  is the design buckling resistance of the steel member in fire situation, for the temperature  $\theta$ ;

$N_{fi,\theta}$  is the internal force in fire situation for the temperature  $\theta$ , which is defined as:

$$N_{fi,\theta} = N_{fi,\theta=20^{\circ}\text{C}} + \Delta N_{fi,\theta} \quad \text{Equation 5-18}$$

where:

$N_{fi,\theta=20^{\circ}\text{C}}$  is the internal force in steel members obtained at room temperature with the load combination in fire situation. This force should be calculated using standard computer code for structure analysis;

$\Delta N_{fi,\theta}$  is the additional compressive force, for the temperature  $\theta$ , due to the partial restraint to the free elongation of the beam.

The checking of the resistance in the case of lattice beam can be limited to the following steel members:

- **Elements of the bottom chords close to the ends of fire compartment (i.e. close to the columns supports of the fire walls);**
- **For each type of steel profile used for vertical members, the element which is the more loaded at normal temperature (with load combination in fire situation);**
- **Diagonals loaded in compression.**

Calculation of the buckling resistance of steel profiles

The design buckling resistance at temperature  $\theta$ ,  $N_{b,fi,Rd}$ , of a steel member subjected to an axial compression should be obtained from:

$$N_{b,fi,Rd} = \chi_{fi} A k_{y,\theta} f_y / \gamma_{M,fi} \quad \text{Equation 5-19}$$

where:



$\chi_{fi}$  is the reduction factor for flexural buckling in fire situation which depends on the non-dimensional slenderness ratio ;

$k_{y,\theta}$  is the reduction factor for the yield strength of steel at the temperature  $\theta$ .

For a practical use, the buckling coefficient  $\chi_{fi}$  can be evaluated from values given in the following table, according to the steel grade and the non-dimensional ratio at room temperature  $\bar{\lambda}$ .

$\bar{\lambda}$	Steel grade			$\bar{\lambda}$	Steel grade		
	S235	S275	S355		S235	S275	S355
0.2	0.8480	0.8577	0.8725	1.7	0.1520	0.1549	0.1594
0.3	0.7767	0.7897	0.8096	1.8	0.1381	0.1406	0.1445
0.4	0.7054	0.7204	0.7439	1.9	0.1260	0.1282	0.1315
0.5	0.6341	0.6500	0.6752	2	0.1153	0.1172	0.1202
0.6	0.5643	0.5800	0.6050	2.1	0.1060	0.1076	0.1102
0.7	0.4983	0.5127	0.5361	2.2	0.0977	0.0991	0.1014
0.8	0.4378	0.4506	0.4713	2.3	0.0903	0.0916	0.0936
0.9	0.3841	0.3951	0.4128	2.4	0.0837	0.0849	0.0866
1	0.3373	0.3466	0.3614	2.5	0.0778	0.0788	0.0804
1.1	0.2970	0.3048	0.3172	2.6	0.0725	0.0734	0.0749
1.2	0.2626	0.2691	0.2794	2.7	0.0677	0.0686	0.0699
1.3	0.2332	0.2387	0.2473	2.8	0.0634	0.0642	0.0653
1.4	0.2081	0.2127	0.2200	2.9	0.0595	0.0602	0.0612
1.5	0.1865	0.1905	0.1966	3	0.0559	0.0565	0.0575
1.6	0.1680	0.1714	0.1766				

Table 5.5 : Reduction factor  $\chi_{fi}$  as a function of relative slenderness  $\bar{\lambda}$  and steel grade

The non dimensional slenderness at room temperature  $\bar{\lambda}$  is given by:

$$\bar{\lambda} = (\lambda / \lambda_d) (\beta_A)^{0.5} \quad \text{Equation 5-20}$$

where:

$$\lambda_d = 93.9 (235 / f_y)^{0.5} ;$$

$\lambda = \ell_{fi} / i$  is the slenderness of the element for the buckling about the weak axis;

$\ell_{fi}$  is the buckling length for the fire design situation about the weak axis ;

$i$  is the radius of gyration of the cross-section about the weak axis ;

$\beta_A = 1$  for class 1, 2 and cross-section ;

For steel members making up the lattice beams, the buckling length in the fire design situation may be taken as:





⇒ for horizontal chords:  $\ell_{fi} = 0,7\ell$  ;

⇒ for diagonals:  $\ell_{fi} = 0,65\ell$  ;

⇒ for vertical members:  $\ell_{fi} = 0,5\ell$  ;

where  $\ell$  is the member length. For horizontal chords, it's advisable to take the distance separating two successive vertical members.

### Calculation of the internal forces in the heated lattice beams

During the expansion phase, the temperature rise leads to a longitudinal elongation of the heated lattice beam which results in an increase of internal forces (additional compressive forces) due to axial restraint against thermal expansion induced by the cold parts of structure.

Two situations need to be considered, namely:

- **Additional compressive force in horizontal chords;**
- **Additional compressive force in vertical members and diagonals;**

#### a) Calculation of the additional internal forces in the horizontal chords

In order to check the stability of the heated lattice beam, and then to calculate the horizontal displacements at the ends of fire compartment, it is necessary to determine the additional compressive forces introduced in the bottom chord as well as in the top chord.

Assumptions:

- **The compressive force is assumed to be uniform along horizontal chords ;**
- **Horizontal chords of the lattice beam are modelled as simply supported isostatic beams (Figure 5-21) combined with horizontal spring taking into account the cold parts of the structure located beyond the partition elements. This spring acts in the horizontal direction and its stiffness  $K_{eq}$  is equivalent to the horizontal stiffness of the cold parts of the structure. Since the studied phase is the expansion phase, this springs are one-directional and provide a response to thermal expansion;**

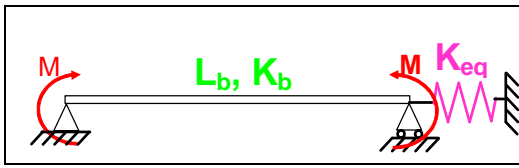


Figure 5-21 : Isostatic beam

- **Stress-strain relationships for steel are bilinear and derived of mechanical properties given in part 1-2 of Eurocode 3 (Figure 5-22).**

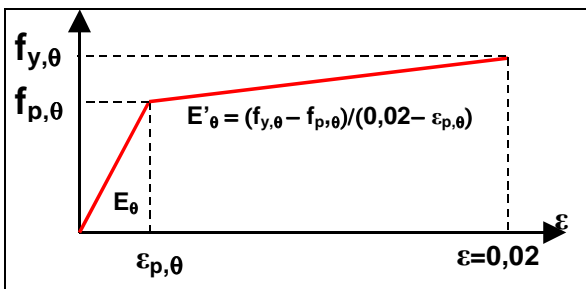


Figure 5-22 : Stress-strain relationship for steel

Restraint to free elongation of the beam developed by the cold parts of the structure introduces an additional compressive force in the bottom chord which can be calculated by the following formula:

$$\Delta N_{mi,\theta} = \frac{\alpha \cdot L_b \cdot (\theta - 20)}{1/K_{eq} + 1/K_{bi}} - \frac{1/K_b - 1/K_{bi}}{1/K_{eq} + 1/K_{bi}} N_{el,\theta} \quad \text{Equation 5-21}$$

where:

$\theta$  is the steel temperature;

$\alpha$  is the coefficient of linear thermal expansion (taken as  $14 \cdot 10^{-6}$ ) ;

$L_b$  is the span submitted to fire;

$N_{el,\theta}$  is the design resistance of the chord for the temperature  $\theta$  :  $N_{el,\theta} = A \cdot f_{y,\theta}$  ;

$K_{eq}$  is the equivalent lateral stiffness of the cold parts of the structure:  
 $1/K_{eq} = \sum 1/K_i$  where  $K_i$  is the lateral stiffness of the considered steel framework.

$K_b$  et  $K_{bi}$  are the axial stiffness (linear and non-linear elastic) of the chord for the temperature  $\theta$ .

The axial stiffness  $K_b$  et  $K_{bi}$  are defined for the temperature  $\theta$  by:



$$\Rightarrow K_b = A \cdot E_\theta / L_b$$

$$\Rightarrow K_{bi} = K_b \quad \text{if} \quad N_{mi, \theta=20^\circ\text{C}} + \Delta N_{mi, \theta} \leq N_{el, \theta} ;$$

$$\Rightarrow K_{bi} = A \cdot E'_\theta / L_b \quad \text{if} \quad N_{mi, \theta=20^\circ\text{C}} + \Delta N_{mi, \theta} > N_{el, \theta} ;$$

with:

$E_\theta$  et  $E'_\theta$  are the slope of the elastic linear range and the non-linear elastic range for steel at the temperature  $\theta$  (see Figure 5-22) and  $A$  is the cross-section area of the chord.

The additional compressive force developed in the top chord of the heated lattice beam can be calculated from:

$$\Delta N_{ms, \theta} = \frac{\alpha L_b (\theta - 20) - \delta_0}{1/K_{eq} + 1/K_{bi}} - \frac{1/K_b - 1/K_{bi}}{1/K_{eq} + 1/K_{bi}} N_{el, \theta} \quad \text{Equation 5-22}$$

where

$\delta_0 (= \Delta N_{mi, \theta} / K_{eq})$  is the displacement at temperature  $\theta$  due to the above additional compressive force in the bottom chord.

The axial stiffness  $K_b$  et  $K_{bi}$  are defined for the temperature  $\theta$  by:

$$\Rightarrow K_b = A \cdot E_\theta / L_b$$

$$\Rightarrow K_{bi} = K_b \quad \text{if} \quad N_{ms, \theta=20^\circ\text{C}} + \Delta N_{ms, \theta} \leq N_{el, \theta} ;$$

$$\Rightarrow K_{bi} = A \cdot E'_\theta / L_b \quad \text{if} \quad N_{ms, \theta=20^\circ\text{C}} + \Delta N_{ms, \theta} > N_{el, \theta} ;$$

**b) Calculation of the additional internal forces in the compression diagonals and vertical members:**

Studies performed on the basis of advanced calculations show that internal forces in the diagonals under compression of lattice beam remain approximately constant despite the temperature rise.

With regard to vertical members, the temperature rise as well as the axial restraint to free expansion induced by the horizontal chords initiate low additional compressive force in this type of element. However, numerical results show that instability of vertical members, when it takes place, always occurs for values of compressive force close to those obtained at normal temperature (with the load combination for the fire situation).

From the above comments, values of internal forces calculated at normal temperature with the load combination for the fire situation can be used to check the stability of diagonals under compression and vertical members.

For these elements, compressive forces are given by:



$$N_{fi,t} = N_{fi,\theta=20^{\circ}\text{C}} \quad \text{and} \quad \Delta N_{fi,\theta=0} = 0 \quad \text{Equation 5-23}$$

Calculation of maximum displacements at the ends of fire compartments

Displacements at the top of the columns supports of the partition elements can be calculated from:

$$\delta_{\max,i} = (\Delta N_{mi,\theta_C} + \Delta N_{ms,\theta_C}) / K_i \quad \text{Equation 5-24}$$

where:

$K_i$  is the lateral stiffness of the designed cold part of structure;

$\Delta N_{mi,\theta_C}$  is the additional compressive force in the bottom chord obtained for the temperature  $\theta_C$  (cf. Equation 5-20) ;

$\Delta N_{ms,\theta_C}$  is the additional compressive force in the top chord obtained for the temperature  $\theta_C$  (cf. Equation 5-21).

$\theta_C$  is the temperature reached in horizontal chords members at the end of expansion phase.

#### 5.2.1.2. Fire compartment in the middle of the storage building: Case of several heated spans

With regard to fire compartment with several spans, displacements at the top of columns supports of partition wall can be derived by the superposition and the combination of the basic case presented in Figure 5-20 with appropriate values of  $K_1$  and  $K_2$ .

For example, the displacement at the top of columns of the fire partition wall will be equal to the sum of the lateral displacement of each heated span, which can be obtained by applying the method of § 5.2.1.1 with suitable stiffness  $K_1$  and  $K_2$  as shown in Figure 5-23.

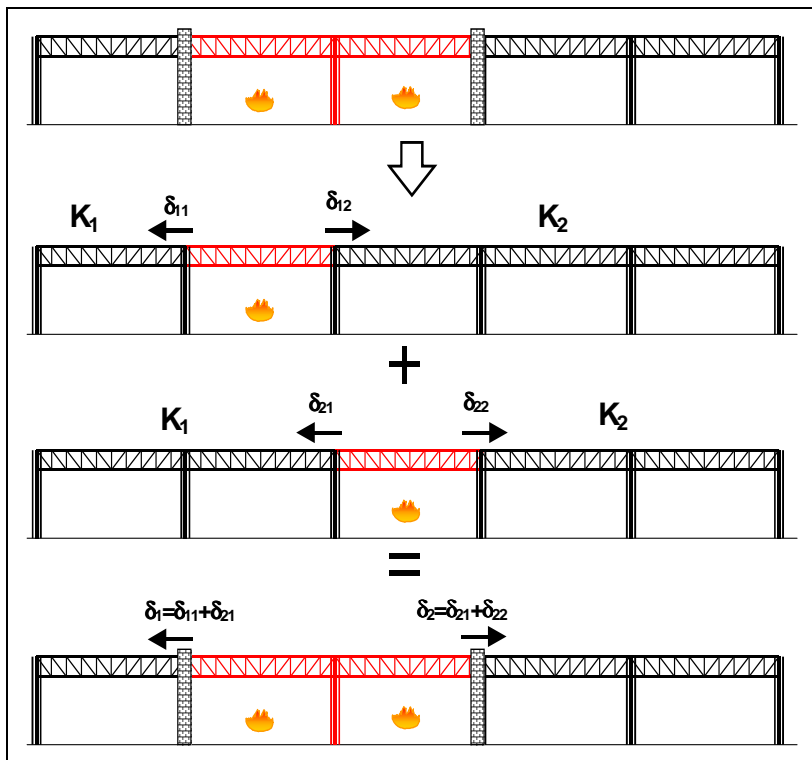


Figure 5-23 : Principle of displacement superposition

For a practical use, in alternative to the superposition method, displacements at the ends of fire compartments can be obtained by applying the basic case (see § 5.2.1.1) with a cell made up of only one equivalent heated span (of total length  $L$  equal to the sum of all heated spans) and with appropriate values of lateral stiffness  $K_1$  and  $K_2$  (see Figure 5-24).

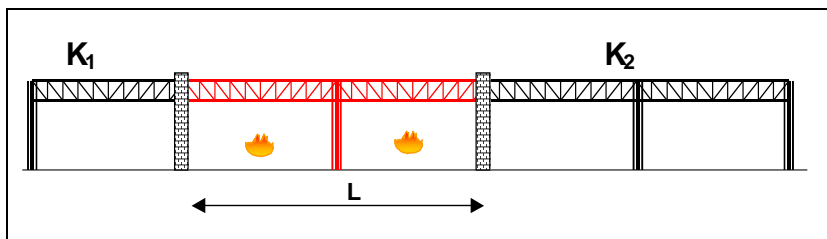


Figure 5-24 : Equivalent heated span

### 5.2.1.3. Fire compartment at the end of storage building

In the case of a fire compartment located at the end of the storage building, displacements at the top of columns supports of partition elements and facade elements can be calculated using the following rules:

- **Displacements at partition elements can be obtained by applying the simple model presented in paragraph 5.2.1.1 to the span of the fire compartment contiguous to the fire wall and considering**



appropriate values of lateral stiffness  $K_1$  and  $K_2$  (see Figure 5-25). In the case of only one heated span, the value of  $K_1$  should be taken as  $K_1 = 0.2 \times K$  (where  $K$  is the lateral stiffness of the span at normal temperature)

- Displacements at the end of the storage building can be calculated from the following formula:

$$\delta_1 = \alpha \cdot \sum_{i=1}^m \ell_i \cdot (\theta_i - 20) - \delta_2$$

Equation 5-25

where:

$\ell_i$  is the length of the heated span  $i$ ;

$n_1$  is the span number in the fire compartment ;

$\theta_i$  is the temperature reached in horizontal chords of lattice beam at the end of expansion phase;

$\alpha$  is the coefficient of linear thermal expansion (taken as  $14 \cdot 10^{-6}$ ).

### 5.3. Recommendation for bracing

Additional design recommendations must be put into practice to allow the collapse of the steel structure under fire condition on either side of the fire wall without causing any damage to the fire wall.

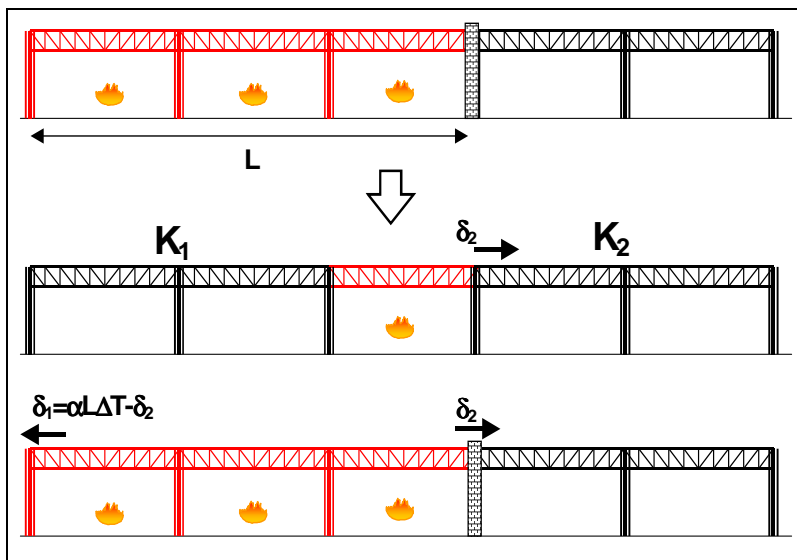


Figure 5-25 : Displacements in the case of a fire compartment at the end of building

### 5.3.1. Fire walls perpendicular to steel frames

Requirement of no collapse towards outside along the longitudinal direction (perpendicular to steel frames) can be satisfied using appropriate bracing systems. Specifically, each compartment must have its own bracing system (see Figure 5-26). So, the following solutions should be adopted:

- to use of additional vertical bracing system on each side of the fire wall. This bracing system should be designed to support a lateral load taken as 20% of the normal wind load (according to the load combination for the fire situation) calculated for a gable area “S” limited to the width of only one span ( $S=h \times l$ );
- to double the bracing on both sides of fire walls or to protect against fire the preceding bracing systems.

Nevertheless, these bracing systems shall be compatible with ambient temperature design; in a way that they will not cause problems e.g. to expansion of joint.

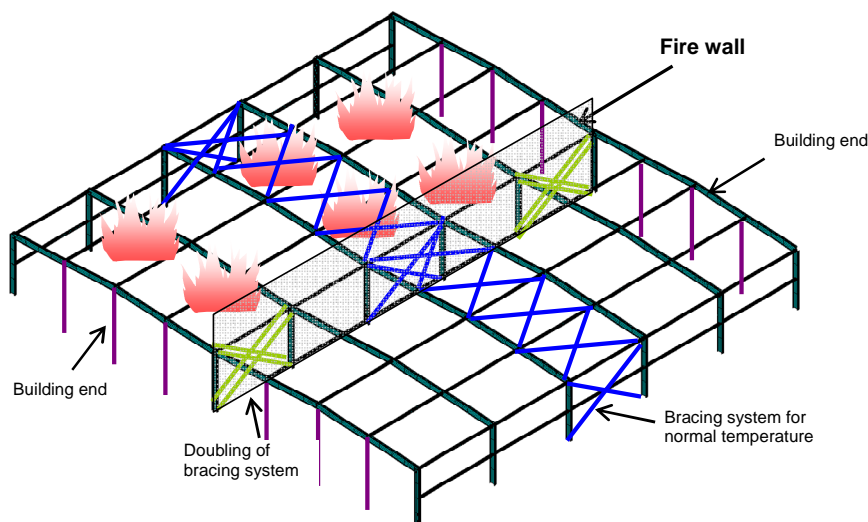


Figure 5-26 Bracing systems at the longitudinal end of the storage building

### 5.3.2. Fire wall parallel to steel frame

The bracing systems (vertical between columns or horizontal on the roof) are generally located inside the same compartment. When fire walls are parallel to steel frames, it is necessary to install an additional bracing system (vertical and horizontal on the roof) at each compartment, so that the collapse of the steel structure of the heated cell does not lead to the instability of the whole building (Figure 5-27). Each bracing system must be designed to support a horizontal uniform load taken as:

$$F = 1.19 q$$

Equation 5-26

where

$$q = G + 0.2 S$$

When the fire wall is mixed with the steel frame, elements of bracing systems must be fixed to rigid steel elements implemented to support the purlins on each side of the wall.

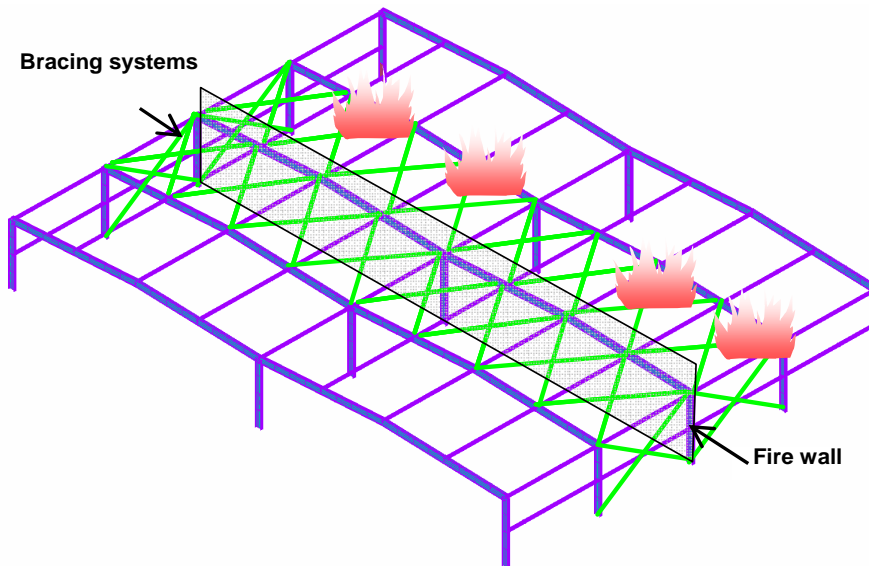


Figure 5-27 Bracing systems of storage buildings

## 5.4. Case study for lattice structures

As application example, design methods described previously in § 5.1.3 are used hereafter to evaluate maximum displacements and forces induced at the fire compartments ends of a building with lattice steel framework during both expansion phase and collapsing phase.

### 5.4.1. Description of chosen steel framework

The lattice steel structure characteristics and boundary conditions are summarized in Table 5.6 and Figure 5-28.

Span number	Span (m)	Column height (m)	Steel members				Cell number
			Column	Horizontal Chords	Vertical	Diagonal	
3	30	7.5	HEA 450	L100×100×10	L70×70×7 L50×50×5	L100×100×10 L80×80×8 L70×70×7	3





						L50×50×5	
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Table 5.6 : Main characteristics of steel framework

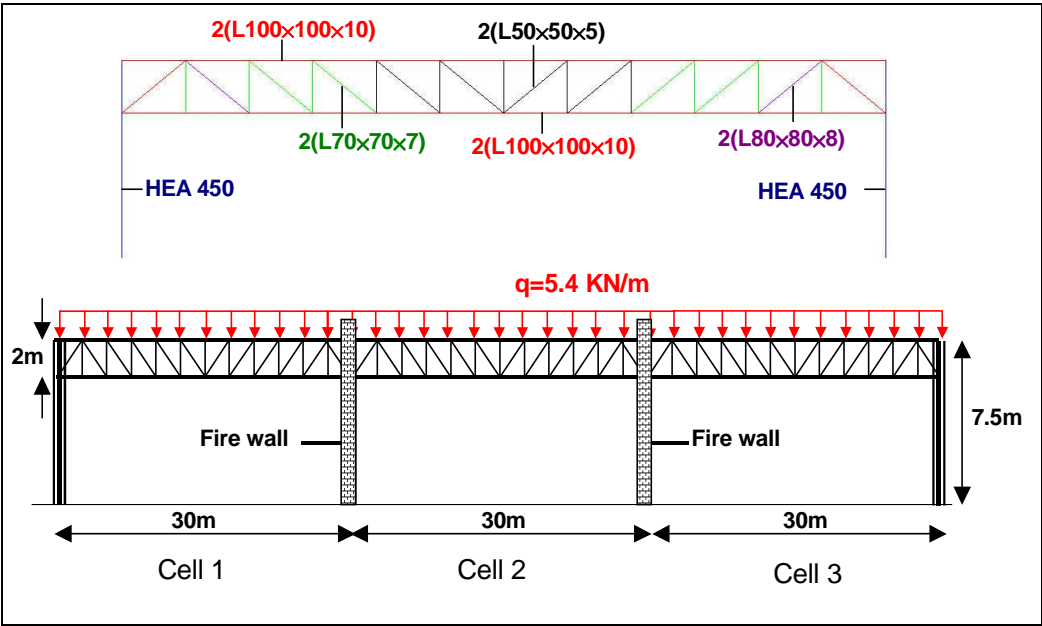


Figure 5-28 : Arrangement of steel framework

5.4.2. Choice of fire scenarios

In this study, building is divided into 3 cells separated by fire walls. Then, symmetry leads to consider only two fire scenario (see Figure 5-29):

- **Scenario 1: fire in the external cell (cell 1 or 3);**
- **Scenario 2: fire in the middle cell (cell 2);**

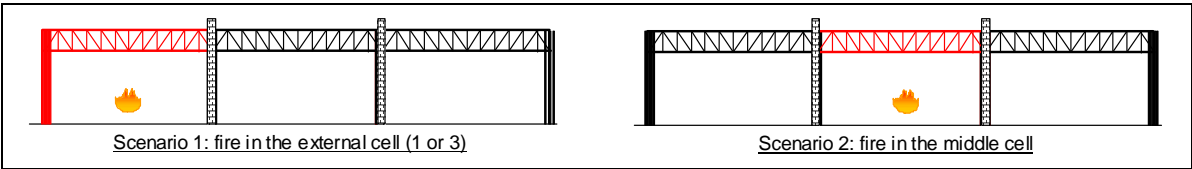


Figure 5-29 : Fire scenarios for studied steel framework

Lateral stiffness calculated using structure analysis software is given in Table 5.7.



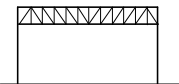
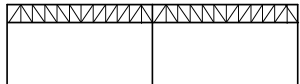
Span number		
Stiffness (N/m)	3538.57	4933.40

Table 5.7 : Lateral stiffness of steel frames

### 5.4.3. Summary of results

For each fire scenario displacement (for the expansion phase) and the forces (for the collapsing phase) are determined using the simple rules, simplified method and numerical simulations (ANSYS). Main results (displacements and forces) are reported in Table 5.8 and Table 5.9 respectively.

The results of simple design methods are compared to those obtained with numerical analysis (ANSYS). There is a good agreement between numerical model and simplified method.

Method	Fire scenario 1		Fire scenario 2	
	Left end	Right end	Left end	Right end
Simple rules	0.225	0.045	0.135	0.135
Simplified methods	0.188	0.031	0.105	0.105
Numerical results	0.17	0.026	0.10	0.10

Table 5.8 : Displacements for expansion phase

Method	Fire scenario 1		Fire scenario 2	
	Tensile Force (kN)	Displacement (m)	Tensile Force (kN)	Displacement (m)
Simple rules	171.0	0.035	285.0	0.081
Numerical results	141.0	0.03	270.0	0.08

Table 5.9 : Displacements and forces for collapsing phase

## 6. Façade Elements, Partitions and Fire Resistance Walls

To minimize the risk for people and to prevent any risk of fire spread between buildings or compartments separated from one to another by partition elements, safety regulation requires, in addition to the fire resistance rating usually needed for compartment elements (which depends on the use and height of the building), that



the localized failure of the first cell under fire condition doesn't lead to the progressive collapse of the load-bearing structure of the building and doesn't imply the collapse of the structure towards the outside. These requirements assumes that the movement of the load-bearing structure of the building don't lead to the prematurely collapse of the facades and partition walls. To reach this objective, adequate design recommendations should be put in practice.

After a short description of some systems currently used for industrial & storage buildings, recommendations for façades and partition walls as well as steel structure are suggested. These recommendations aim at preventing prematurely the failure of elements and so to avoid the risks of progressive collapse and collapse towards the outside.

## 6.1. Description of selected façades and wall systems

A short description of some type of façade and fire wall systems currently used for industrial & storage buildings is given hereafter:

- ⇒ Isocomposite panels
- ⇒ Fireproof panels
- ⇒ Frame walls with cold formed sections
- ⇒ Fire walls with hot rolled profiles and light weight concrete

### 6.1.1. Isocomposite panels

#### 6.1.1.1. Product description

Manufacture of sandwich panels of big length until 12m and width 1198 mm. Insulators are extruded polystyrene, expanded polystyrene, and Rockwool.

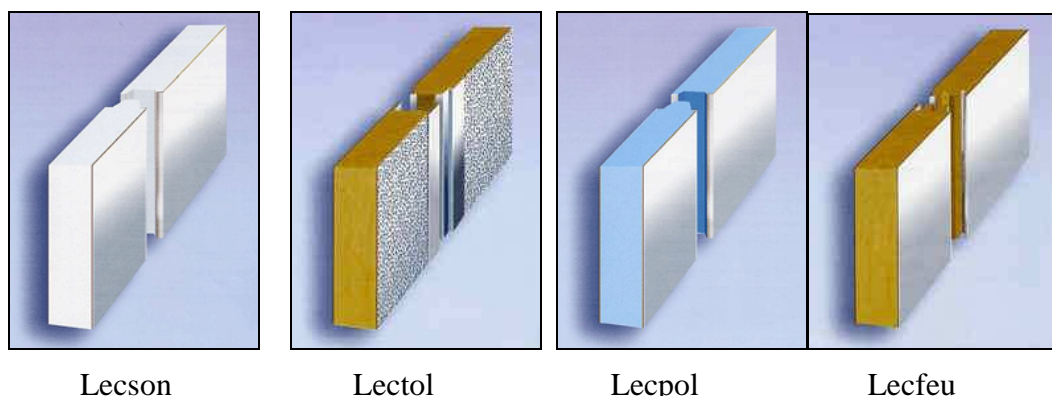


Figure 6-1 : Isocomposite systems

Sandwich panels for façades (Lecson, Lectol, Lectpol).



Sandwich panels for fire partition walls (Lecfeu).

	Products			
	Lecson	Lecfeu	Lectol	Lecpol
Width	1180 mm	1180 mm	1180 mm	1180 mm
Insulator thickness	60, 80, 100, 120 mm	60, 80, 100, 120 mm	60, 80, 100, 120 mm	60, 80, 100, 120 mm
Length	up to 6 m; Phonic isolation especially	up to 6 m; Fire resistance, especially	up to 12 m	up to 12 m
Loading resistance	thickness: 2 to 2.5 m	thickness: 2.5 to 4 m	thickness: 2 to 4 m	thickness: 2 to 4 m
Fire resistance	M0 (rock wool)	M0 (rock wool)	M1 (EPS)	M1 (PS)
Thermal performance	up to K = 0.34	up to K = 0.27	up to K = 0.26	up to K = 0.26
Durability (corrosion)	galvanized steel and painted;			

Table 6.1 Properties of the Isocomposite panels

## 6.1.2. Fireproof panels

### 6.1.2.1. Product description

The alternative to concrete in EI30, EI60, EI90 and **EI180**. The insulation core of the panel is non combustible acc. fire resistance class A1. An element thickness of 70mm responds to EI30 & W60, a thickness of 100mm responds to EI60 & W90. An element thickness of 120mm achieves EI90. The panels are available with different profile designs and thickness.

### 6.1.2.2.

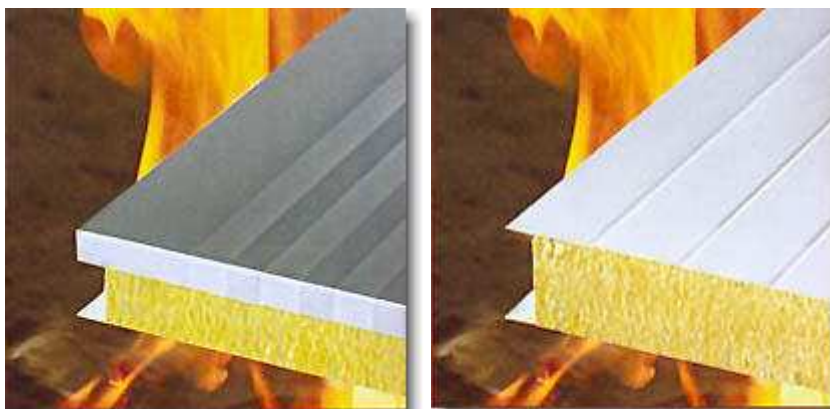


Figure 6-2 : Fireproof Panel

#### 6.1.2.3. Application field

The outstanding characteristics enable a great range of application. It ranges from external and internal wall construction to the construction of ventilation plants, office containers, roofs, ceilings enamelling chambers and drying installations

#### 6.1.2.4. Technical comments

##### Dimensions

Width: 915 mm or 1100 mm. Special construction widths: between 500 mm and 1200 mm.  
length: standard length at most 10 m. Thickness: 35, 40, 50, 60, 70, 80, 100, 120, 140, 160, 180, 200 mm. At panels of series L and V, 2 mm must be added to the standard thickness because of the profile design.

##### Durability (corrosion)

Resistance class K III / DIN 55 928 part 8. Outer and inner plate: 0,55 mm - 0,75 mm sendzimir galvanised steel acc. DIN 17162, polyester or PVDF coated. Insulation: Mineral fibre plates in a web form, non combustible acc. fire resistance class A1. Sealant: For tongue- and groove joint acc. demands of the fire resistance class. Higher steel thickness upon request. Polyester coating 25 µm, PVDF 80/20, Kynar 500 appr. 25 µm. Colours in stock: RAL 9002, RAL 9006, RAL 9010. Further colours acc. Pflaum-Colour-Spektrum (available upon request). Surface protection: The panels are supplied with a protection foil.

##### Fire Resistance

##### Behaviour in fire:

(thickness/class) 70mm / EI30; 80mm / EI60; 120mm / EI90; 100mm / EI180

##### Thermal performance

##### K-values:

⇒ for 35 mm  $K = 1.19 \text{ W/m}^2\text{K}$ ;



⇒ for 200 mm K = 0.24 W/m<sup>2</sup>K;

Acoustic performance

Sound insulation:

⇒ 35 to 60 mm: 34dB;

⇒ 70 to 200mm: 35dB;

### 6.1.3. Frame walls with cold formed sections

#### 6.1.3.1. Product description

Cold formed sections are introduced between two plaster boards. The thickness, the size and the shape of the cold formed section can be variable.

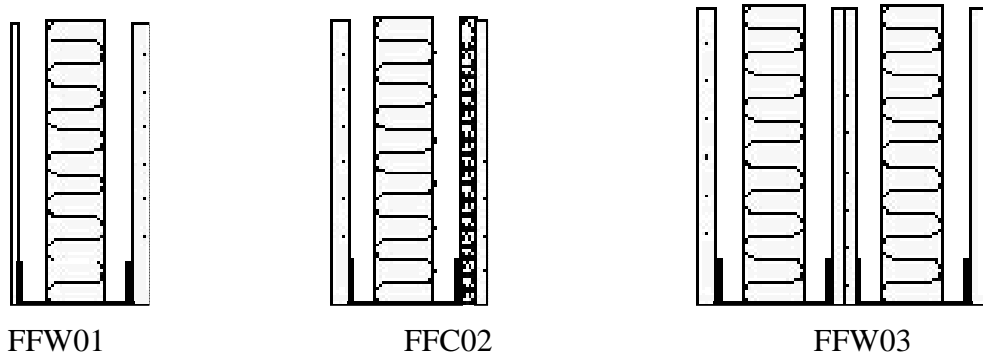


Figure 6-3 : Cold formed sections

#### 6.1.3.2. Application field

Partitions walls and fire resistant walls.

#### 6.1.3.3. Technical comments

Dimensions

FFW01

93mm steel channel 1.2mm gauge (CH9312). Internal lining: One layer 15mm Lafarge Megadeco plasterboard. Insulation: 50mm mineral wool density 33kg/m<sup>3</sup>. Weight 26 Kg/m<sup>2</sup>,

FFC02

93mm steel channel 1.2mm gauge (CH9312). Internal lining: One layer 15mm Lafarge Vapourcheck Megadeco plasterboard. External Lining: One layer 22mm Thermal Minerit. Insulation: 50mm mineral wool density 33kg/m<sup>3</sup>. Weight 27 Kg/m<sup>2</sup>,

FFW03

Internal Lining: One layer 15mm Lafarge Megadeco plasterboard. Sandwich lining:  
Two layers 9mm Minerit. Weight 52 Kg/m<sup>2</sup>,

	Products		
	FFW01	FFC02	FFW02
Fire resistance	EI30	EI60	EI120
Thermal performance	n.a	K= 0.35	n.a.
Acoustics performance	Sound insulation 45- n.a dB	Sound insulation 48- n.a dB	Sound insulation 55- n.a dB

Table 6.2 Properties of walls with cold form sections

#### 6.1.4. Fire walls with hot rolled profiles and light weight concrete

##### 6.1.4.1. Product description

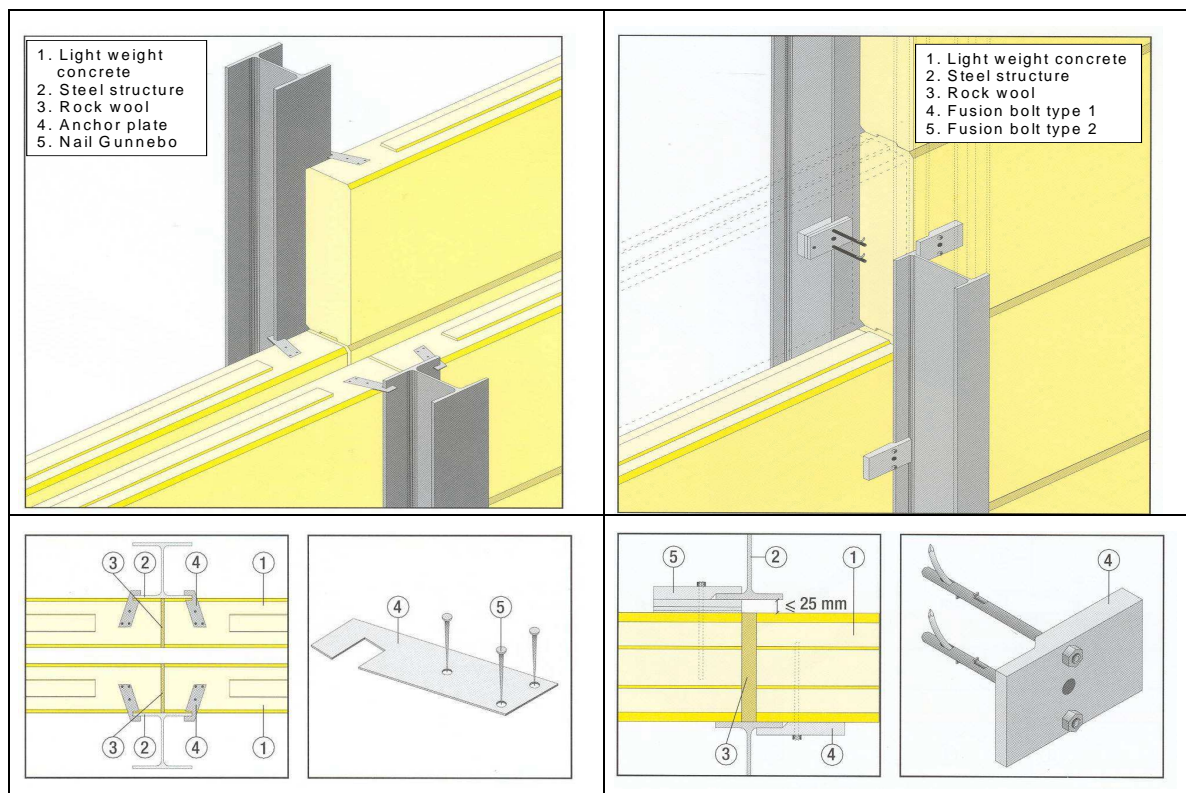


Figure 6-4 : Doubled wall (left side) and simple wall with fusion bolts (right side)

The wall is composed of hot rolled profile section and light weight concrete panels. The walls can be completely doubled or the steel structure can be doubled and connection on each side of the wall with fusion connections.





#### 6.1.4.2. Application field

Partitions walls and fire resistant walls.

#### 6.1.4.3. Technical comments

##### Dimensions

- ⇒ Width: 600 mm;
- ⇒ Length: 6000 mm maximum (Between two profiles);

##### Loading Resistance

Steel meshing in the light weight concrete is calculated for a wind pulling of  $800 \text{ N/m}^2$ .  
this quantity of steel meshing can be upgraded if necessary;

##### Fire Resistance

The concrete part of the wall for 150 mm resistance can reach a fire resistance of 6 hours.  
But the global fire resistance depends on the system itself;

##### Thermal performance

The thermal conductivity  $\lambda$  is  $0.15 \text{ W/mK}$ ;

### 6.2. Displacement of façades and fire walls

Studies performed on the basis of advanced calculations have shown that horizontal displacements of the load-bearing structure of industrial building under fire condition can be important.

The horizontal displacement can go up to several tens of centimeters and therefore could lead to the failure of facade or the partition element if it is not sufficiently ductile or not accurately fixed. It is thus important to ensure that displacements of the load-bearing structure can be absorbed by a partition wall (or a facade) in contact with it so that the integrity condition of partition element can be conserved. As a consequence, corresponding design methods easy to use and allowing to evaluate these displacements are given later.

### 6.3. Design recommendation

Recommendations proposed hereafter can be applied to any type of fire wall, such as in lightweight concrete, reinforced concrete, hollow block, steel sheeting with insulator, plasterboard, bricks, or built with any other material. However, fire wall must be fixed in a suitable way to remain compatible with the lateral displacements of the steel framework under fire condition.





Use of facade elements is not limited for storage buildings. However, whatever the type of facade is, its structural adequacy, its integrity and its compatibility with respect to the movement of the steel framework must be ensured in order that the collapse of these elements, if it takes place, occurs toward inside of the building. The self-stable facades must be proscribed as far as their movement occurs always towards outside as a consequence of thermal bowing effect. They will be used only if their behaviour is evaluated by advanced calculation model taking into account second order effects, or if their load-bearing structure is located outside, and thus sufficiently protected against heating to remain stable.

### 6.3.1. Attachment of façade and partition elements to steel structure

In order to prevent any failure of partition elements (fire walls) and facade elements due to significant lateral displacements of the steel structure, it is necessary to ensure that these elements remain solidly attached to the structure.

So, to avoid any risk of collapse of the facade elements towards the outside or collapse of partition elements, a solution consists in fixing these elements with the columns of the load-bearing structure, by means of suitable attachment systems. For example, horizontal steel plates or purlins uniformly distributed along the building height, arranged on columns and separated with a specific maximum depth. This maximum value will be fixed by the manufacturer of the walls, and it is recommended a maximum value of 3 m for made on side walls (concrete blocks, bricks...) (see Figure 6-5).

In addition, screws used to connect fire walls and facade elements on the columns must be designed to resist to the forces due to wind and self-weight of partition elements under the effect of the lateral displacement induced by the steel framework of the storage building.

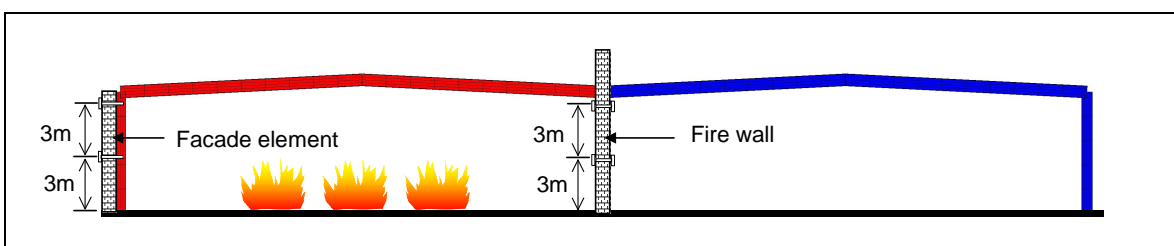


Figure 6-5 : Design detail for separation elements

### 6.3.2. Design recommendations for steel structures near to separation elements

Additional design recommendations must be put into practice to allow the collapse of the steel structure under fire condition on either side of the fire wall without causing any damage to the wall

The elements that could damage the walls (being near or crossing the walls) will remain stable with a fire resistant rate at least equal than the walls, to shift away the plastic hinges from the walls.

Thickness of fire protection applied to columns and beams can be simply calculated assuming a steel section exposed on four faces, for a standard fire exposure of one hour and a heating limited to 500°C.

Thickness of fire protection applied to lattice beams can be calculated assuming: a steel section exposed on four faces for bottom chords, vertical members and diagonals and three faces for top chords, for a standard fire exposure of one hour and a heating limited to 500°C.

### 6.3.3. Design recommendations for roof systems above the separation elements

In order to prevent that the collapse of the roofing structure close to the fire wall leads to the damage of the wall during the fire, some design details must be applied.

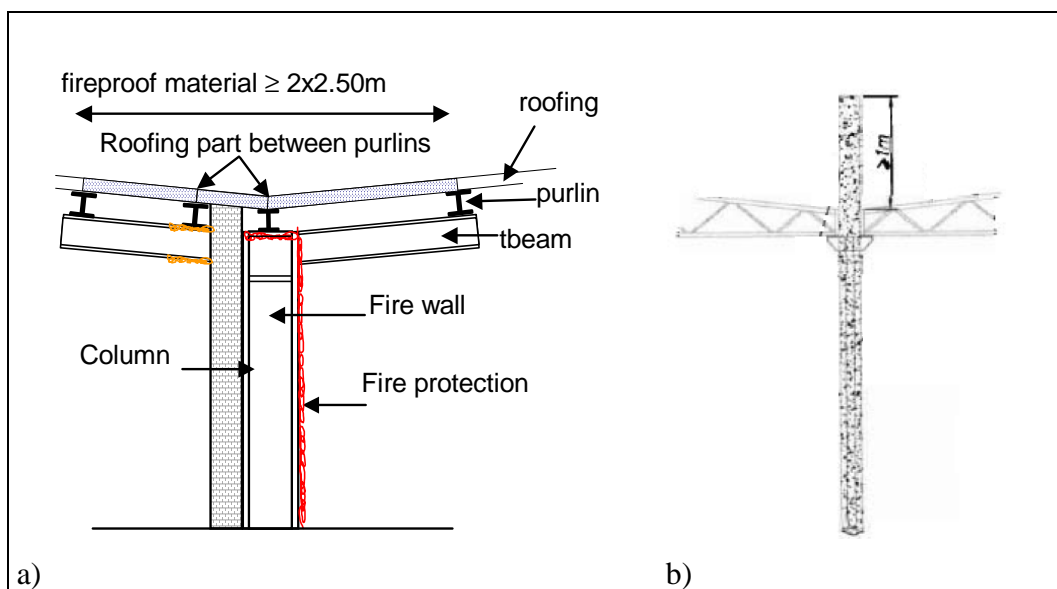


Figure 6-6 : Design details near to the roof

A solution consists (see Figure 6-6 a)):

- **In using purlins on both sides of the fire wall;**
- **In stopping the roof on both sides of the fire wall. Roof close to the fire wall (part located between the previous two purlins) must be designed so as to be supported by the wall. Then, the roof will be independent from one compartment to the others.**
- **And in using roof with fireproof material, over a width of 2.50 m on each side of the wall;**



Other possibility is to allow the wall exceed the roof up to a specific distance (Figure 6-6 b)).

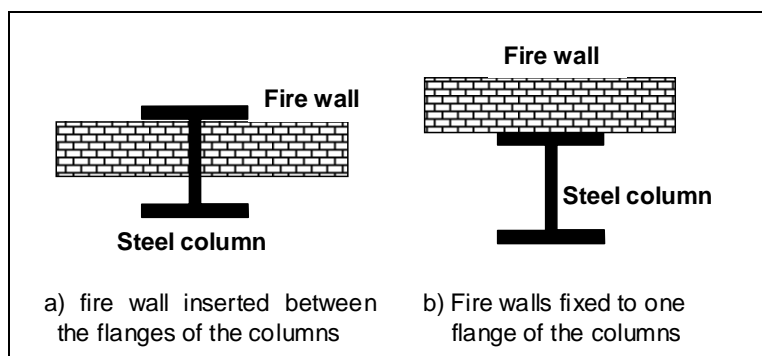
#### 6.3.4. Design recommendations for fire walls perpendicular to steel frames

In case of fire walls perpendicular to steel frames these design recommendations should be applied:

- **COLUMNS** that are into or near a wall must be always fire protected.
- **BEAMS** that cross walls must be protected over a specific distance from the wall. In case of portal frames this minimal length should be 200mm, and for lattice structures a minimal length equal to the distance separating the wall with the first vertical member.
- **PURLINS** never cross the walls so it is not necessary to be fire protected.

In storage buildings with steel frames, several solutions for partition elements need to be considered, namely (see Figure 6-7):

- **Fire wall inserted between the flanges of columns;**
- **Fire walls fixed to one flange of columns;**



*Figure 6-7 : Arrangement of separation elements*

In common cases, the fire requirements lead to apply a fire protection on columns of steel frames (see Figure 6-8 and Figure 6-9).

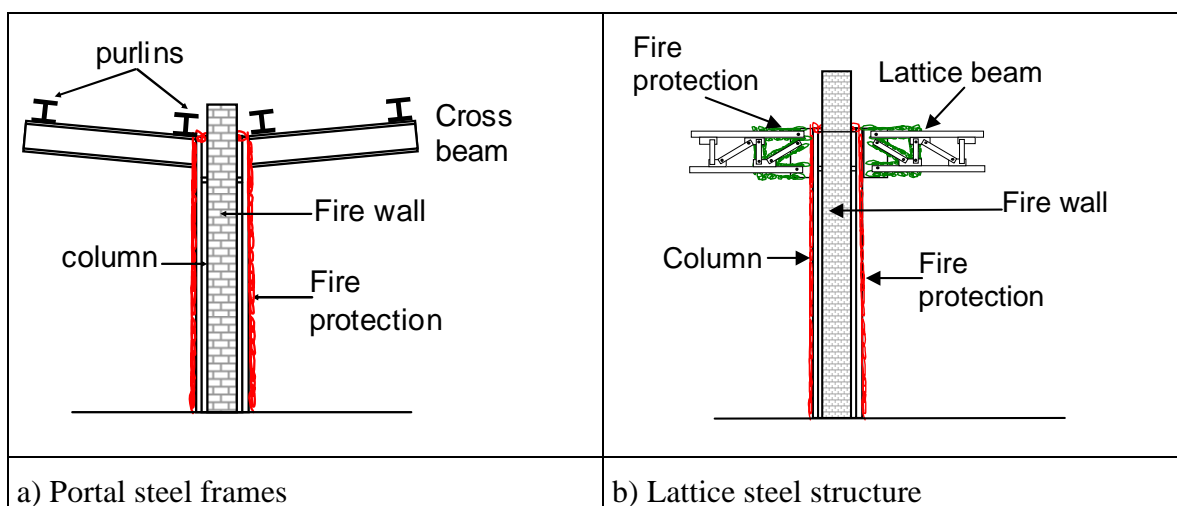
In addition, when the fire wall is inserted between the flanges of columns, no additional fire protection is needed for beams of portal steel frames (see Figure 6-8). On the contrary, lattice steel structures near fire wall must be protected to avoid the



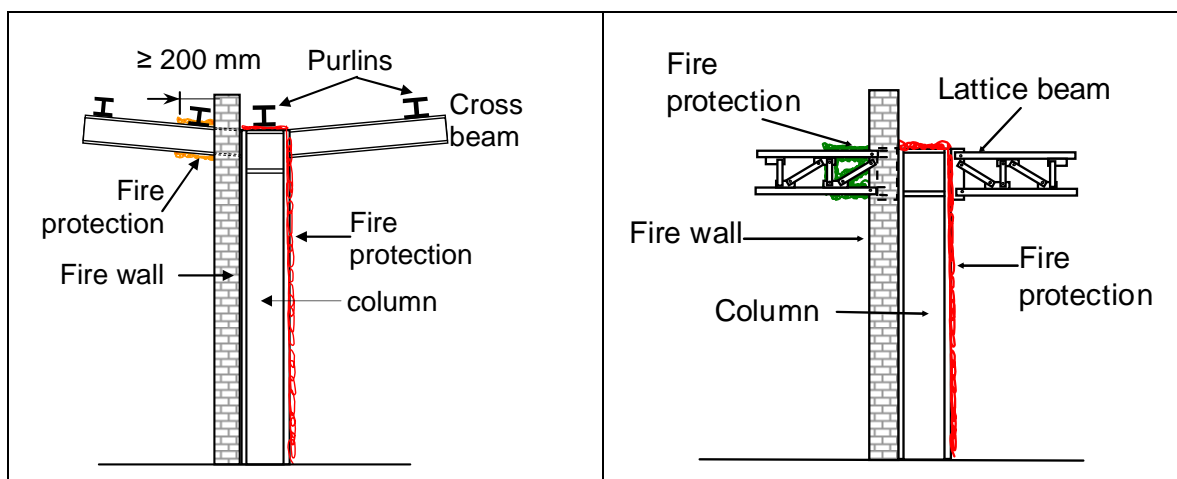
possible disorder induced by the failure of the lattice beam near to the fire wall. Consequently, a fire protection on both side of the wall must be applied to lattice beams (on horizontal chords, vertical members and diagonals) over a minimal length equal to the distance separating the wall with the first vertical member (see Figure 6-8 b).

In the similar way, when the fire wall is built beside one flange of columns, to prevent wall damage with the collapse of the beam near the fire wall, a fire protection must be used on beam (on the side of wall):

- **Over a minimal length of 200 mm beyond the wall limit, for portal steel frame (see Figure 6-9 a).**
- **Over a minimal length equal to the distance separating the wall with the first vertical member, for lattice structure (see Figure 6-9 b).**



*Figure 6-8 : Fire Protection when the fire wall is inserted between the flanges columns*





a) Portal steel frames	b) Lattice steel structure
------------------------	----------------------------

*Figure 6-9 : Fire protection when the fire wall is beside one flange of the columns*

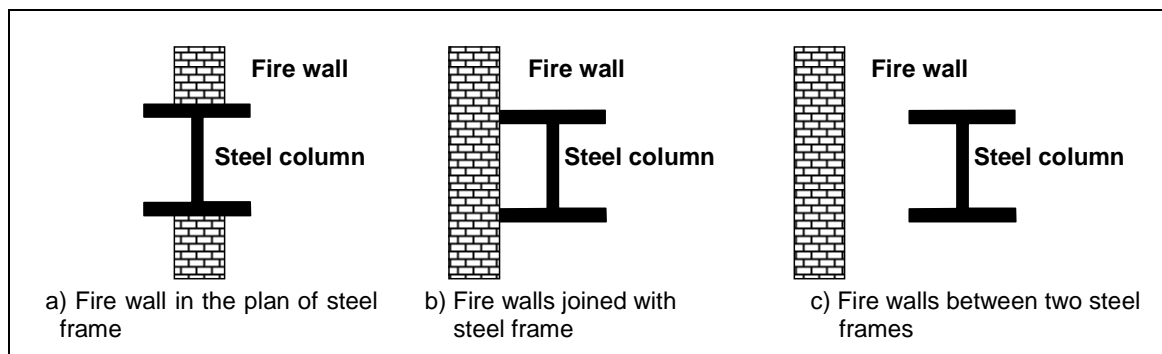
### 6.3.5. Design recommendations for fire walls parallel to steel frames

In case of fire walls parallel to steel frames these design recommendations should be applied:

- **COLUMNS** that are into or near a wall must be always fire protected.
- **BEAMS** that are into or near a wall must be always fire protected.
- **PURLINS** are going to cross the walls so it is necessary to fire protect the continuous purlins (over a distance of 200mm from the wall) or design a not continuous purlins system (see Figure 6-11).

Several solutions for partition elements can be considered (see Figure 6-10), namely:

- ⇒ Fire wall inserted in steel frame;
- ⇒ Fire walls beside and in contact with the steel frame;
- ⇒ Fire walls between two steel frames;



*Figure 6-10 : Arrangement of partition elements*

Requirements of no fire propagation and no progressive collapse between different compartments (stability of the cold parts of the structures) lead to apply a fire protection on steel frames (beams and columns) near fire walls (see Figure 6-11 and Figure 6-12).

When the roofing structure is made of lattice beams, lattice beams cannot allow inserting a continuous wall up to the roof. A solution consists in subdividing industrial building in two independent structures and inserts the fire wall between them. In

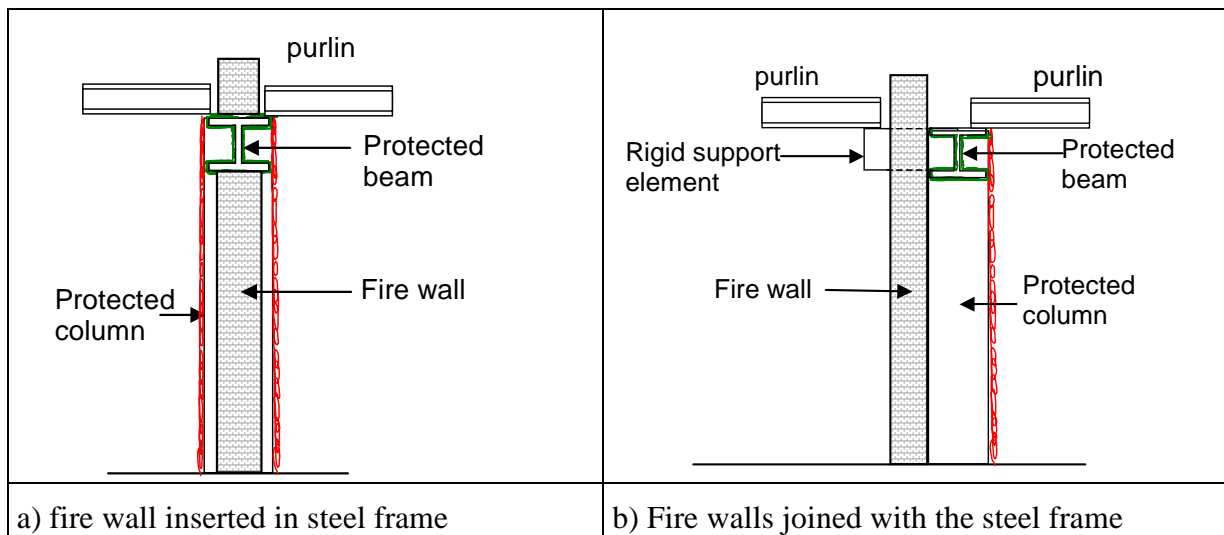


this case, no fire protection is required for the structure close to partition elements (see Figure 6-12b).

Steel elements going across a fire wall should not affect the fire performance of the wall (stability, thermal insulation qualities...). It is thus necessary to consider design solutions so that the collapse of the roofing structure nearest the fire wall doesn't involve the failure of the wall.

As example, a solution consists for portal steel frames:

- **When the fire wall is inserted in the steel framework, in putting through the wall rigid steel elements fixed on the beams to support the purlins (see Figure 6-11 b);**
- **In the case of continuous purlins, in putting on both sides of the wall a fire protection on purlins, over a minimal length of 200 mm beyond the wall. Thickness of fire protection can be calculated assuming steel section exposed on four faces, for a standard fire exposure of one hour and a heating limited to 500°C. In fact, the aim of this fire protection is to move away from the wall the plastic hinge which will be formed at elevated temperature.**



*Figure 6-11 : Design details of portal steel frame near to fire wall*

For lattice steel structure with a fire wall beside the steel frame, a solution consists:

- **When the roof structure is made of purlins, in protecting purlins and counters near the wall over a minimal length corresponding to the**



distance from the wall to the junction counter/ purlin (see Figure 6-12 a).

- When the roof structure is made of lattice beams, a fire protection must be applied to beams, located on the wall side, over a minimal length corresponding to the distance from the wall to the first vertical members of the beam.

Thickness of fire protection applied to lattice beam can be calculated assuming a steel section exposed on four faces for bottom chords, vertical members and diagonals and three faces for top chords, for an standard fire exposure of one hour and a heating limited to 500°C.

All the load bearing members on both sides of the wall must be capable of expanding and moving away from their supports without leading to the damage of the wall. If fire wall is not capable of bearing alone forces induced by thermal elongation of these members, design solutions must be taken so that these members come in contact with the wall creating an appropriate support for the fire wall.

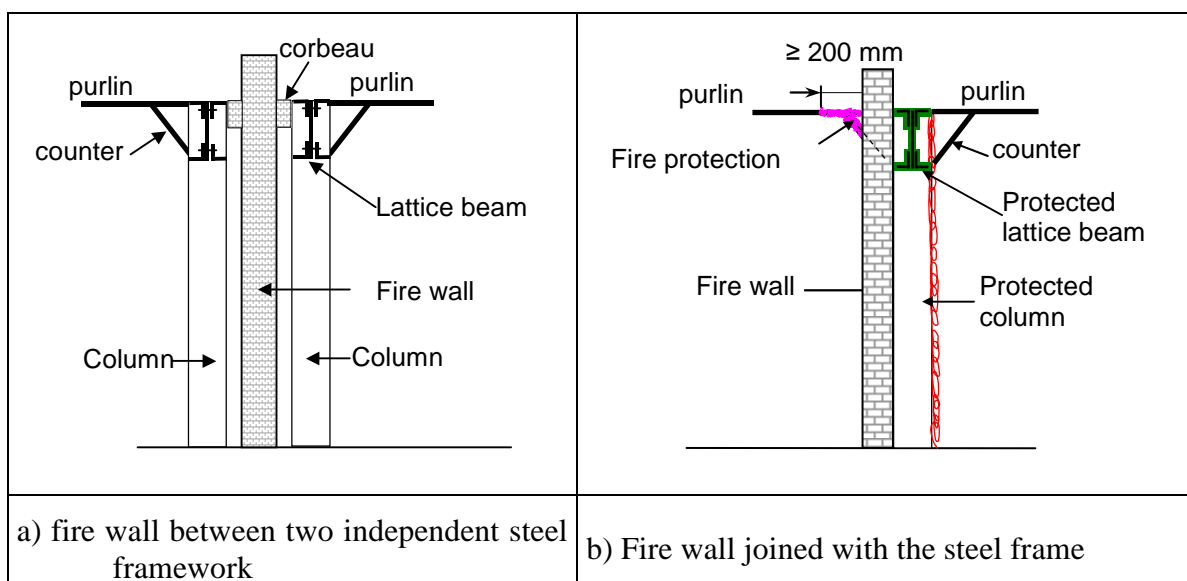


Figure 6-12 : Design details of lattice steel structure near to fire wall

When the fire wall is located between two steel frames, this wall is only loaded in normal situation by pressures or depressions due to the wind. However, in fire situation the deflection of the steel structure on a side or other of the wall will generate vertical loads on this wall. As a consequence this wall must be designed for the fire situation taking account of such additional loads.



## 7. Conclusions

In previous RFCS research project, with the help of advanced numerical models, parametric studies have been carried out to evaluate the structural behaviour (failure mode, displacement...) of single storey buildings with steel structure under fire condition. In these studies, the main parameters susceptible to affect the fire performance of two types of steel structures have been taken into account, such as span of frames, height of columns, number of spans, fire location, position of fire walls, etc.

On the basis of corresponding numerical results, simplified calculation methods have been proposed:

- ⇒ On the one hand, to check the stability of the cold parts of the structure under the effect of the collapse of the heated part of the structure, and
- ⇒ On the other hand, to evaluate maximum displacements developed at the fire compartment ends.

Two types of steel structures are covered by these methods, namely:

- ⇒ Portal steel frames with cross section in standard H or I hot rolled profiles;
- ⇒ Steel frames making up lattice beams with columns in standard H or I hot rolled profiles.

It have been shown through the comparison with numerical results that the proposed calculation methods allow, with a good precision, a safe evaluation of forces induced by the behaviour of the heated parts of the structure and displacements at the fire compartment ends.

The actual document has explained in detail the basis of developed simple design rules, their validity compared to advanced calculations as well as the fundamental principles of proposed construction details not only for main steel frames of single storage buildings but also for partition walls and facade elements. Finally, a brief description of the user-friendly design software is provided in order to facilitate its application by engineers in their fire design of single storage buildings in steel structure.

## 8. Working examples

### 8.1. Example 1

Figure 8-1 illustrates multi-bay portal frame with three fire compartments. Each of the bays is 30m wide and 10m high with 5% slope of the roof. Only columns near the fire walls are fire protected. The columns are made from IPE400 and beams from IPE360. Distance between the frames is 7m.





In this scenario fire occurs in the middle fire compartment.

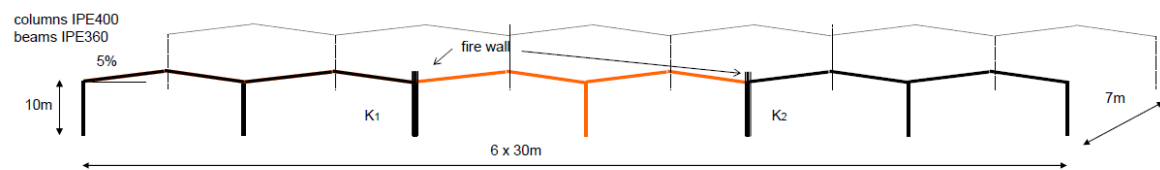


Figure 8-1 Multi-bay portal frame with 3 fire compartments

Following the methodology presented in the “FS+ Design Guide” the tensile forces and displacement that occur during fire in the middle frame will be presented hereafter.

### Tensile force

Step 1 Coefficient related to the slope of the roof

from Equation 4-3

$$c_p = 1.16 \quad \text{for portal frame with roof slope of 5\%}$$

Step 2 Coefficient related to the number of heated bays in the fire compartment

from Equation 4-5

$$n_{\text{eff}} = 2.00 \quad \text{for fire in the middle compartment and } n=2 \text{ bays in fire}$$

Step 3 Vertical load

weight of the roof	0.25 kN/m <sup>2</sup>
weight of the top frame	0.6573 kN/m
distance between frames	7 m
span of on heated bay connected to the column	30 m
snow load in fire condition in Belgium regul.	0 kN/m <sup>2</sup>

from Equation 4-7

$$q = 0.25 \text{ kN/m}^2 \cdot 7 \text{ m} + 0.6573 \text{ kN/m} = 2.4073 \text{ kN/m}$$

Step 4 Tensile force

from Equation 4-8

$$F = 1.16 \cdot 2.00 \cdot 2.4073 \text{ kN/m} \cdot 30 \text{ m} = 167.5504 \text{ kN}$$

### Lateral displacement

Step 1 Reduction factor related to the slope of the roof

from Equation 4-9

$$c_{\text{th}} = 0.011 \quad \text{for portal frame when roof slope equals 5\%}$$



## Step 2 Equivalent lateral stiffness of the cold part of the steel frame

from Equation 4-12

$K_1 = K_2 = c \cdot k$  for  $m = 2$  bays in “cold compartment” near the fire compartment

$I_b = 1.63E-04 \text{ m}^4$  second moment of area for beams for IPE 360  
 $f = 0.75 \text{ m}$  ridding  
 $h = 10 \text{ m}$  height of the column  
 $l = 30 \text{ m}$  span of one bay  
 $I_c = 2.31E-04 \text{ m}^4$  second moment of area for column for IPE 400  
 $E = 2.10E+08 \text{ kN/m}^2$  Young's modulus for steel for normal temperature

from Equation 4-13

$\alpha = 0.220550061$   
 $k = 71.8065082 \text{ kN/m}$   
 $c = 1.765646549$

$K_1 = K_2 = 126.7849134 \text{ kN/m}$

## Step 3 Lateral displacements in the expansion phase

from Equation 4-15

$\delta_1 = 1.321532629 \text{ m} = 132.15 \text{ cm}$   
 $\delta_2 = 1.321532629 \text{ m} = 132.15 \text{ cm}$

## Step 4 Maximum displacement induced by the tensile force

from Equation 4-16

$\delta_{\max 1} = 1.321532629 \text{ m} = 132.15 \text{ cm}$   
 $\delta_{\max 2} = 1.321532629 \text{ m} = 132.15 \text{ cm}$

## 8.2. Example 2

Figure 8-2 illustrates multi-bay portal frame with two fire compartments. Each of the bays is 24m wide and 7m high with 10% slope of the roof. Only columns near the fire walls are fire protected. The columns are made from IPE360 and beams from IPE330. Distance between the frames is 12m.

In this scenario fire occurs at the end of the portal frame with 2 bays.

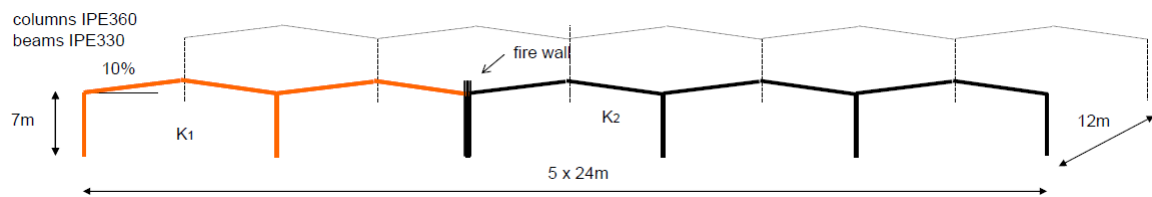


Figure 8-2 Multi-bay portal frame with 2 fire compartments

Following the methodology presented in the “FS+ Design Guide” the tensile forces and displacement that occur during fire at the end of the frame will be presented hereafter.

### Tensile force

Step 1 Coefficient related to the slope of the roof

from Equation 4-3

$$c_p = 1.10 \quad \text{for portal frame with roof slope of 10\%}$$

Step 2 Coefficient related to the number of heated bays in the fire compartment

from Equation 4-5

$$n_{\text{eff}} = 1.00 \quad \text{for fire at the end of the compartment and } n=2 \text{ bays in fire}$$

Step 3 Vertical load

weight of the roof	0.25 kN/m <sup>2</sup>
weight of the top frame	0.5721 kN/m
distance between frames	12 m
span of on heated bay connected to the column	24 m
snow load in fire condition in Belgium regul.	0 kN/m <sup>2</sup>

from Equation 4-7

$$q = 0.25 \text{ kN/m}^2 \cdot 12 \text{ m} + 0.5721 \text{ kN/m} = 3.57208 \text{ kN/m}$$

Step 4 Tensile force

from Equation 4-8

$$F = 1.10 \cdot 1.00 \cdot 3.57208 \text{ kN/m} \cdot 24 \text{ m} = 94.303 \text{ kN}$$

### Lateral displacement

Step 1 Reduction factor related to the slope of the roof

from Equation 4-9

$$c_{th} = 0.015 \quad \text{for portal frame when roof slope equals 10\%}$$



## Step 2 Equivalent lateral stiffness of the cold part of the steel frame

from Equation 4-14

$$K_1 = 0.13 \text{ k} \quad \text{for } n = 2 \text{ bays in the fire compartment at the end of frame}$$

from Equation 4-14

$$K_1 = c \cdot k \quad \text{for } m \geq 2 \text{ bays in "cold compartment" near the fire}$$

$$I_b = 1.18 \text{E-}04 \text{ m}^4 \quad \text{second moment of area for beams for IPE 330}$$

$$f = 1.2 \text{ m} \quad \text{ridging}$$

$$h = 7 \text{ m} \quad \text{height of the column}$$

$$l = 24 \text{ m} \quad \text{span of one bay}$$

$$I_c = 1.63 \text{E-}04 \text{ m}^4 \quad \text{second moment of area for column for IPE 360}$$

$$E = 2.10 \text{E+}08 \text{ kN/m}^2 \quad \text{Young's modulus for steel for normal temperature}$$

from Equation 4-13

$$\alpha = 0.17655$$

$$k = 97.0245 \text{ kN/m}$$

$$c = 2.77865$$

$$K_1 = 0.13 \cdot 97.0245 \text{ kN/m} = 12.6132 \text{ kN/m}$$

$$K_2 = 2.77865 \cdot 97.0245 \text{ kN/m} = 269.597 \text{ kN/m}$$

## Step 3 Lateral displacements in the expansion phase

from Equation 4-15

$$\delta_1 = 0.68782 \text{ m} = 68 \text{ cm}$$

$$\delta_2 = 0.03218 \text{ m} = 3.2 \text{ cm}$$

## Step 4 Maximum displacement induced by the tensile force

from Equation 4-16

$$\delta_{\max 1} = 7.47654 \text{ m} = 747.654 \text{ cm}$$

$$\delta_{\max 2} = 0.34979 \text{ m} = 34.979 \text{ cm}$$



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