New fire safety regulations for single-storey buildings have appeared in several countries (Belgium, Spain and France) that could significantly reduce the application of steel in this type of building. In order to provide strong technical arguments and solutions to avoid the introduction of excessive fire resistance requirements in the single-storey field throughout Europe, an ECSC research project 'Fire safety of industrial halls and low-rise buildings' has been carried out, with completion in 2007.

The project clearly demonstrated that a steel structure, if designed appropriately, fulfils the safety requirements in case of fire in terms of 'non-progressive collapse' and 'non-dangerous failure type'. On the basis of a series of parametric studies, several simple design rules and some key construction details have been proposed in order to help engineers to design safe steel structures for single-storey industrial buildings.

Dissemination of these results was an important aim of the project. Therefore the following actions have been taken.

— The simple design rules and construction details worked out for single-storey industrial buildings have all been summarised in a design guide.

— A background document has been created in order to give more detailed information from previous research, provide a summary of several European national requirements in fire regulation and include a survey of real fire cases.

— User-friendly 'LUCA' software has been developed for more efficient application of the design guide.

— Technical seminars have been organised in order to communicate all the abovementioned design tools to engineers in several European countries

Additionally, a simplified method to evaluate heat flux depending on the distance from the façade is reported on in this project. The method has been developed within a French national project, and includes large number of real scale fire tests in order to validate the methodology.

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Fire safety of industrial halls — A valorisation project

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Table of Content

1 Final Summary

WP1

Investigation of real fires in industrial halls presented in the scope of the previous research project has been updated with new cases. Review of Fire Safety Regulations in Luxembourg, France, Spain and Belgium has been updated. Results of the WP1 are reported in the Background Document.

WP2

Based on the results from the previous project a Design Guide has been edited covering design rules for industrial buildings in respect to structural behaviour as well as the behaviour of attached elements of steel industrial buildings.

The study related to calculation of heat flux emitted in case of fire in steel storage buildings has been developed mainly within a scope of French national project. Due to the delay in real scale fire tests performed in the French project the methodology has been developed too late to be included in the Design Guide of this project. However, it is presented in the following section of this report. The heat flux emission is an important aspect only for the French partner in this project and it has been presented during the seminar organised in France.

WP3

In order to be able to present the different documents in the mother tongue to all seminar participants in Spain, Luxembourg, Belgium and France, the documents realised in the scope of WP1 and 2 were translated into French and Spanish. Additionally, considering specific character of the Belgium market the Design Guide has been also translated in Dutch, which was outside of the project requirements.

WP4

The calculation method developed for industrial halls has been implemented in software called "LUCA". The software refers clearly to the equations presented in the Design Guide and produces report of the calculations performed also in pdf format.

WP5

In order to disseminate deliverables prepared in the previous project seminars have been organised in France, Spain and Belgium (in French and Dutch). Before the event, invitations have been prepared and distributed to the target audience; stakeholders involved in the construction market, architects, engineers, steel fabricator and general constructors, students, and professors as well as last but not least decision makers with authorities, insurance companies and firemen. During the workshops, printed documents as well as CD-Rom or USB memory sticks were distributed.

It is very important that alignment with the National Regulations was strongly highlighted during the seminars, when the regulations were also presented.

2 Objectives and Introduction

New Fire Safety Regulations for single storey buildings were introduced in different countries (France, Spain, and Belgium) that could result a significant loss of market share for steel. The steel industry must be ready to provide strong technical arguments and solutions to avoid the introduction of excessive fire resistance requirements in the single storey field throughout Europe. With this aim, the ECCS Project “Fire Safety Of Industrial Hall And Low Rise Building” (CEC Agreement 7210-PR-378) has been launched, focusing on the industrial halls where there is also hidden resistance provided by the 3D behaviour and where it is needed to analyse the structure after some local failure in order to demonstrate that the structure fulfils the safety requirements in case of fire which will be given in terms of "non-progressive collapse" and "failure type".

In the scope of this project research obtained in the RFCS Project “Fire Safety of Industrial Halls and Low Rise Building” has been disseminated. This project has proved that the fire safety of steel single storey buildings is sufficient, in the absence of passive fire protection, by means of risk assessment showing that the safety of people and firemen is ensured.

The work performed in this project has been organised in Work Packages, which clearly specify expected deliverables.

- To realise a design guide for Single Storey Industrial Building taking into account the structural behaviour in case of fire, the façade systems and the fire walls.
- To translate this design guide into French and Spanish,
- To implement Software to design (using the simplified methods) the Single storey structure in case of fire
- And the final objective is to organise various Workshop with the local authorities in order to aware them about this new concept of calculation.

3 Guidelines and Background Document

Within the project two documents have been created; Design Guide and Background Document. The English version of the documents is attached in the Annex hereafter. The Design Guide contains all the simple design rules and construction details for single storey industrial buildings, which have been developed in the previous research project.
The design rules apply to the specific range of sizes of frames used in the parametric. Design beyond the scope of analysis will not be recommended unless treated as preliminary design, which will be farther validated.

- Length of bays for simple bay: 15m , 20m and 30m
- Length of bays for multiple bay: 20m, 30m and 40m
- Height – simple bay: 5m, 7.5m and12.5m
- Height – multiple bay: 7.5m, 12.5m and 20m
- Slope: 5 degrees
- Number of bays: 1, 3 and 5
- Lattice beam: equal angles 50x50x5 till 120x120x12

The design rules have been further validated and modification of certain parameters has been introduced in order to improve the design. Detailed description of the study and alteration are described in the Background Document.
In particular the changes comprise the following equations:

1. Formula for calculation of tensile force for fastening has now form $F = W + 5 p \delta_i d / n$

   Initial coefficient 3 has been replaced with the coefficient 5. The explanation is following: from the static calculation the coefficient is equal to 0.5. Considering that in 720 degrees C the steel has 20% of the strength the coefficient is increased to 2.5. Furthermore, due to the simplified assumption (no second order effect) an uncertainty factor of 2 has been included which leads finally to the factor 5 in the formula.
2. CTICM made study to identify the best value of a multiplier within the coefficient k related to stiffness. The coefficient is described by the formula: $k = \frac{\alpha}{1+2\alpha} \frac{12EI_c}{(h+f)^3}$
3. Also the coefficient α will is changed as a result of study done by CTICM: $\alpha = \frac{I_b}{I_c} \frac{h+f}{l} (1 - \frac{f}{0.6h})$
   where
   h – is the height of the columns [m]
   f – is the ridging [m]
   l – is the length of the span [m]
   Ib – is the second moment of area of the beam [m^4]
   Ic – is the second moment of area of the column [m^4]
   E – is the modulus of elasticity of steel for normal temperature [N/m²]

The Background Document in comparison to the previous project has been updated with the new survey of real fires in industrial halls. The new cases include fire in Logs Santos Warehouse (Spain) and two industrial buildings in France.

Also the actualisation of the National Regulations has been made. Additionally, regulation for Belgium has been added as a partner from this country joined the project. It has been also agreed between the partners not to include information regarding situation in the United Kingdom, which has been present before, due to lack of the partner from this country.

In parallel to this project a French national project is carried on by CTICM, which concentrates on emission of heat flux through the facades of industrial building during fire. Number of real scale tests had been performed and the methodology to calculate the heat flux was developed and validated.

4 Design guide - Heat flux emitted in case of fire in steel storage buildings

4.1 Background

Nowadays, the radiation effect associated with the fires of warehouse is based on "simple" calculation tools. The core method is essentially derived from pool fire tests (fires with hydrocarbon liquids). In practice, experts and engineers have developed their own hypotheses to take into account the influence of various characteristics of warehouses. These assumptions can differ a lot from one expert to another and delay the acceptance of technical report by authorities.

Within the scope of a French national project involving four technical institutes (CNPP, CTICM, INERIS and IRSN), which have already made several fire engineering studies on this topic, a new method aimed at estimating heat fluxes and their consequences has been developed. This method takes into account main characteristics of warehouse which can evolve during the fire (e.g. influence of roof or wall collapsing).

The validity of above method is evaluated in comparison with experimental tests. Eight tests on a 100m² warehouse were conducted and one real scale test performed on 850 m² warehouse (first worldwide).

4.2 General Description Of The Method

4.2.1 Field of application

The method is related to ordinary warehouses. Its application complies with the risk analysis to be realized for the installations subjected to high damage consequences due to important quantity of combustible materials according to French regulation. For this simple calculation method, the following assumptions are used:

- Active protection measures did not succeed in preventing the extension of initial fire.
- Heat release rate varies over time.
- If several compartments are present and separated by fire resistant walls then an analysis is conducted in the compartment where fire has developed.

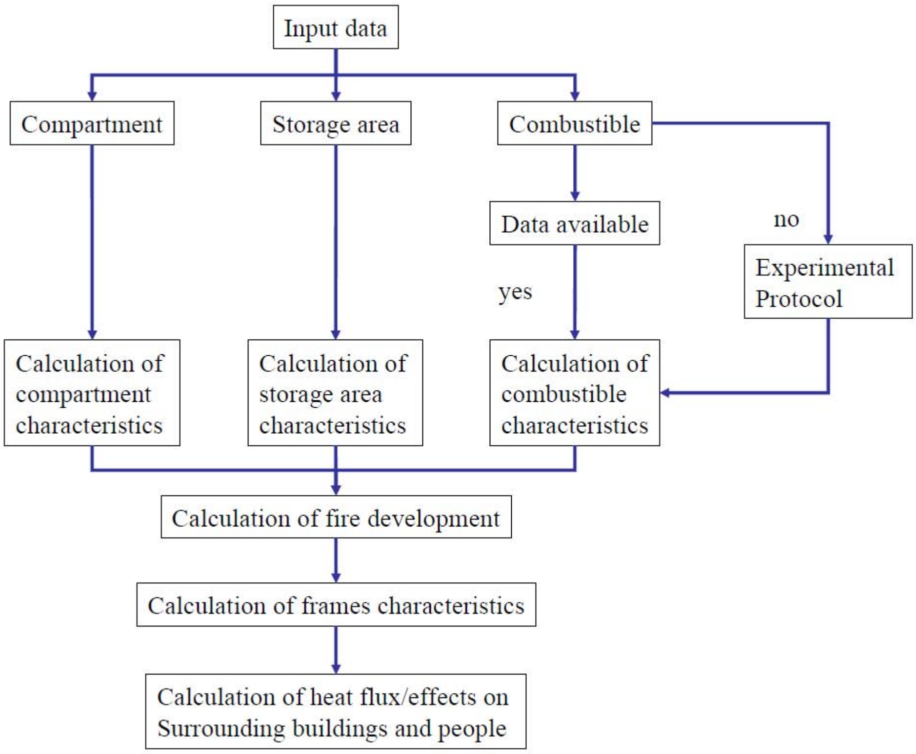
Passive protection measures (fire resistant elements) are efficient and prevent the fire propagation to adjacent compartments. Consequently, fire fighters could contain the fire from surrounding compartments. Nevertheless, the owner must demonstrate the efficiency of these protection measures.

In this case, consequences of fire include:

- radiation of flames that may induce a thermal heating on potential target around the building: people, other buildings...
- toxicity and smoke spreading.

However, the latter point is not yet included within the framework of French risk analysis. In fact, only the distances associated to heat flux of 3 and 5 $kW.m^{-2}$ are given in the case of a scenario where the fire is fully developed within the compartment. When spreading to other compartments cannot be prevented, a calculation is done for each compartment individually then effects are summed (a delay corresponding to the fire resistance capacity of wall is taken into account).

4.2.2 Main steps



The application of this simple calculation method needs to follow a specific flowchart given above.

All these application steps are detailed in the following sections.

4.3 Input data

This section explains the input parameters that are needed in order to apply the method. If any parameter is not known, default value is proposed. In that case, the value will be equivalent to the worst situation (e.g. highest heat of combustion).

Another option will be to estimate their value. In order to do this, a method of estimation is proposed specially for. This option is intended specifically to new products or if no consensual value can be found in existing technical references.

- Data relevant to compartment include length, width, height of structure, characteristics of wall and roof.
- Data relevant to storage area include size, number of levels and mode of storage (rack, loose).
- Data relevant to combustible material include size, composition of pallet.

In the method, characteristics of materials listed in Table 4.1 are included.

Combustible	Incombustible
Wood pallet	Steel
Wood	Water
PE	Glass
Cardboard	
PVC	
Polystyrene	

Cotton cloth	
Synthetic cloth	
PMMA	

Table 4.1 List of combustible and incombustible material

4.4 Calculation of combustible characteristics

The heat release rate depends mainly on the nature of pallets and on ventilation. The following parameters are directly related to the heat release rate:

- combustible material (wood, cardboard, PE,...)
- incombustible material (steel, water,...)
- compactness (state of division) and packaging

They are used to characterize:

- heat release rate of one pallet
- mass loss rate
- heat of combustion
- spreading of fire
- combustion delay of a pallet
- horizontal velocity
- vertical velocity

4.4.1 Rate of mass loss

A pallet is considered globally (in terms of composition) and the rate of mass loss is calculated by mass-weight averaging:

$$V_{comb\_palette} = \frac{\sum_{comb\_i} M_{comb\_i} V_{comb\_i}}{\sum_{comb\_i} M_{comb\_i}} \qquad \text{Equation 4-1}$$

In the section on fire spreading, a rate of mass loss per unit area is then deducted where the area considered is the external area of a pallet.

4.4.2 Heat of combustion

As for rate of mass loss, a mean heat of combustion is obtained by mass-weight averaging of each material into the pallet:

$$\Delta H_{comb\_palette} = \frac{\sum_{comb\_i} M_{comb\_i} \Delta H_{comb\_i}}{\sum_{comb\_i} M_{comb\_i}} \qquad \text{Equation 4-2}$$

4.4.3 Combustion delay for a pallet

It is used to estimate the total surface of rack in fire at a given time by eliminating pallets of which the combustible materials have been fully consumed. This is achieved by considering that surfaces which begin to burn at a time t will disappear at time $t + t_{comb\_palette}$ with:

$$t_{comb\_palette} = \frac{\sum_{comb\_i} M_{comb\_i}}{V_{comb\_palette}} \qquad \text{Equation 4-3}$$

4.4.4 Horizontal and vertical flame spreading velocity

Horizontal and vertical flame spreading velocities are estimated for several groups of material (mix wood, steel, water, PE). Those groups serve as the references and other materials are also classified into groups (some in reference groups, some on new groups). Experimental results of Cleary, Quintière and Ingason were used to estimate velocity ratio between each group. These results are detailed in Table 4.2.

Flame spreading velocities are assumed to be the same independently of mode of storage (rack or loose).

Group	Horizontal flame spreading velocity	Vertical flame spreading velocity
1	$V_{\_prop\_horiz\_pal}$ x 5	$V_{\_prop\_vert\_pal}$ x 5
2	$V_{\_prop\_horiz\_pal}$	$V_{\_prop\_vert\_pal}$
3	$V_{\_prop\_horiz\_pal}$ / 5	$V_{\_prop\_vert\_pal}$ / 5

Table 4.2 Flame spreading velocities

4.5 Fire spreading

Heat release rate $P(t)$ is calculated according to :

$$P(t) = S_{feu\_dev}(t) \times \tau_{occupation} \times V_{comb\_moyen} \times \Delta H_{comb\_moyen} \qquad \text{Equation 4-4}$$

Where:

$\tau_{occupation}$ is the occupancy ratio equal to storage capacity over maximal capacity. It is supplied by the owner and will not have to be exceeded afterward. Default value is 1.

$V_{comb\_palette}$ and $\Delta H_{comb\_palette}$ are detailed in paragraphs 4.4.1 and 4.4.2.

$S_{feu\_dev}(t)$ is the fire area at a given time *t*. The area considered is the external surface of rack as indicated on Figure 4-1.

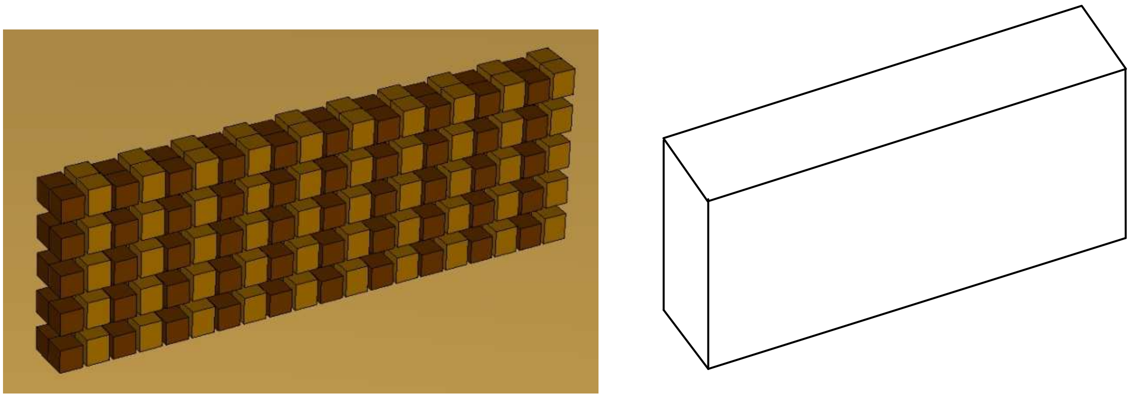


Figure 4-1 left: double rack, right: extern surface considered

4.5.1 Calculation of fire area

The general formula is:

$$S_{feu\_dev}(t) = \sum_i S_{pyro}(i,t) - S_{chute\_toit}(t) \qquad \text{Equation 4-5}$$

where:

$S_{pyro}(i,t)$, fire area for rack i at time t considering that for:

i=1: double rack where fire starts

i=2 et i=3: racks next to rack 1

i=4 et i=5: racks next to racks 2 and 3

i=6 et i=7: racks next to racks 4 and 5

…

It is assumed that the fire starts in the center of compartment.

$S_{chute\_toit}(t)$: area of rack which is covered by parts of roof after its collapsing. In reality, the elements of roof which collapse can reduce fire intensity by preventing air supply.

Calculation of fire area for each rack

Between t and $t+dt$, fire area evolves according to two phenomenon: firstly the increase of fire area induced by flame propagation (term $\Delta S_{propagation}(t)$) and secondly the decrease of fire area induced by extinction of totally burnt pallets (last term). This term is equal to the increase of fire area at time t - $t_{comb\_palette}$.

$$S_{feu\_dev}(t+dt) = S_{feu\_dev}(t) + \Delta S_{propagation}(t) - \Delta S_{propagation}(t - t_{comb\_palette}) \qquad \text{Equation 4-6}$$

Moreover, flame spreading is different between primary racks (racks 1, 2 and 3) and secondary racks (racks 4 and so on).

4.5.2 Flame spreading between adjacent racks

The flame spreading is considered in following way:

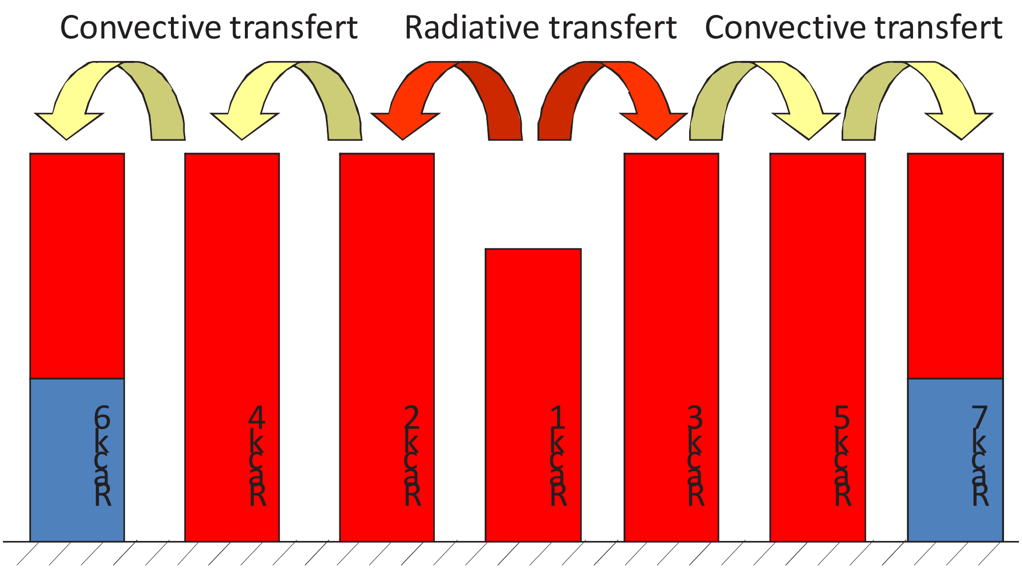


Figure 4-2 flame spreading between racks

From rack 1 to racks 2 and 3, the radiative transfer -involving lateral faces- dominates the convective transfer (the hot layer is not enough wide). Then propagation is mainly governed by convective transfer as the hot layer develops.

Increase of fire area for first racks involved in fire

Flame spreading is divided into two phases as described in Figure 4-3. The fire starts on middle of rack and spreads toward the edge parts of racks. During this phase the shape of fire area seems to be a "V" on lateral side (and rectangular on upper side). Then, flames spread gradually downward to the floor.

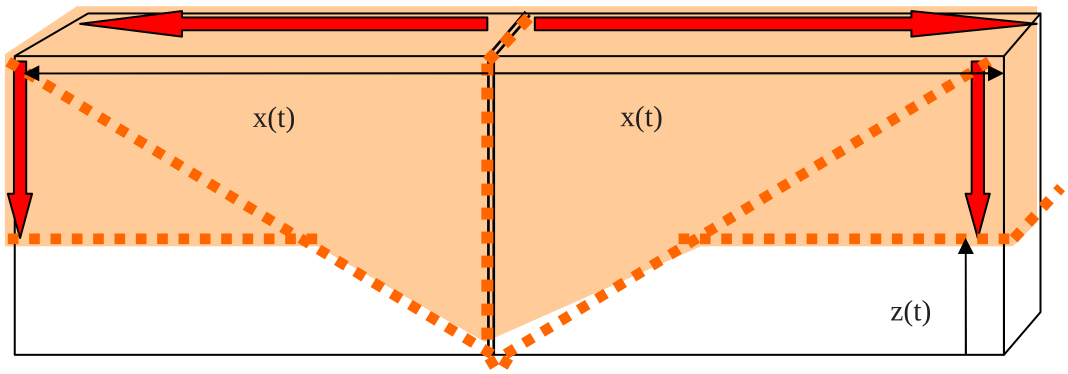


Figure 4-3 flame spreading for primary racks

Input data:

x(t): abscise of flame front on upper side at time t

z(t): height of flame front at time t

vit_prop_hor: horizontal spreading velocity

vit_prop_ver : vertical spreading velocity

t : time

Δt : time step

Output data:

x(t+dt): absciss of flame front on upper side at time t+dt

z(t+dt): height of flame front at time t+dt

ΔS(t+dt): fire area increase at time t+dt

$$x(t+dt) = \min(x(t) + vit\_prop\_hor \times dt; \frac{Longueur\_stock}{2}) \quad \text{Equation 4-7}$$

$$x\_virt1 = vit\_prop\_hor \times t$$

$$x\_virt2 = vit\_prop\_hor \times (t + dt)$$

If $x(t+dt) < \dfrac{Longueur\_stock}{2}$ then

$$z(t+dt) = hauteur\_stockage$$

$$\Delta S(t+dt) = 4^a \left(or\, 2^b\right) \times (x(t+dt) - x(t)) \times (largeur\_stock + hauteur\_stock)$$

([a]: in case of a simple rack

[b]: in case of a double rack)

Else

$$z(t+dt) = \max(0; z(t) - vit\_prop\_ver \times dt) \quad \text{Equation 4-8}$$

$$\Delta S(t+dt) = (x(t+dt) - x(t)) \times (2l\,arg\,eur\_stock)$$
$$+4^{a}(or\,2^{b}) \times \left(Longueur\_stock - \frac{z(t) \times x\_virt1}{hauteur\_stock} \right) \times z(t) \qquad \text{Equation 4-9}$$
$$-4^{a}(or\,2^{b}) \times \left(Longueur\_stock - \frac{z(t+dt) \times x\_virt2}{hauteur\_stock} \right) \times z(t+dt)$$

([a]: in case of a simple rack

[b]: in case of a double rack)

Increase of fire area for secondary racks

As flame spread to surrounding racks is done by contact with the hot layer of smoke, it is considered that the upper part burns instantaneously then flames spread toward the floor as shown on Figure 4-4.

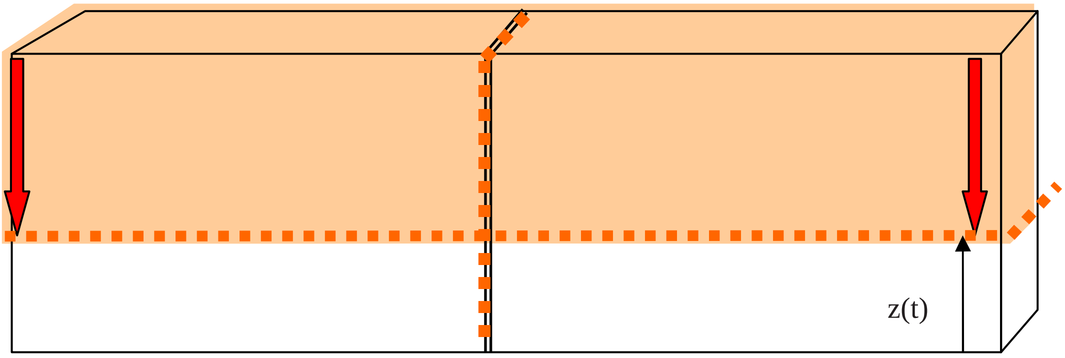


Figure 4-4 flame spreading for secondary racks

Input data:
z(t): height of flame front at time t
vit_prop_ver: vertical spreading velocity
Δt : time step

Output data:
z(t+dt): height of flame front at time t+dt
ΔS(t+dt): fire area increase at time t+dt

$$z(t+dt) = \max(0; z(t) - vit\_prop\_ver \times dt) \qquad \text{Equation 4-10}$$

$$\Delta S(t+dt) = (z(t+dt) - z(t)) \times (périmètre\_stockage) \qquad \text{Equation 4-11}$$

périmètre_stockage :
4Longueur_stock+2largeur_stock for double racks
2Longueur_stock+2largeur_stock for simple racks

4.5.3 Calculation of area hidden by collapsed elements of roof

The following formula is used:

$$S_{chute\_toit}(t) = S_{feu\_plafond}(t - t_{retard\_toit}) \times Coef\_toit \qquad \text{Equation 4-12}$$

$S_{chute\_toit}(t)$ is related to the flaming of the upper part of rack as described on Figure 4-5. It is assumed that the area of collapsed roof at time t equals to the ground area of flamed rack at time t-t_{retard_toit}. This area is extended to adjacent alley on half width leading to $S_{feu\_plafond}(t)$ then the coefficient *Coef_toit* is applied.

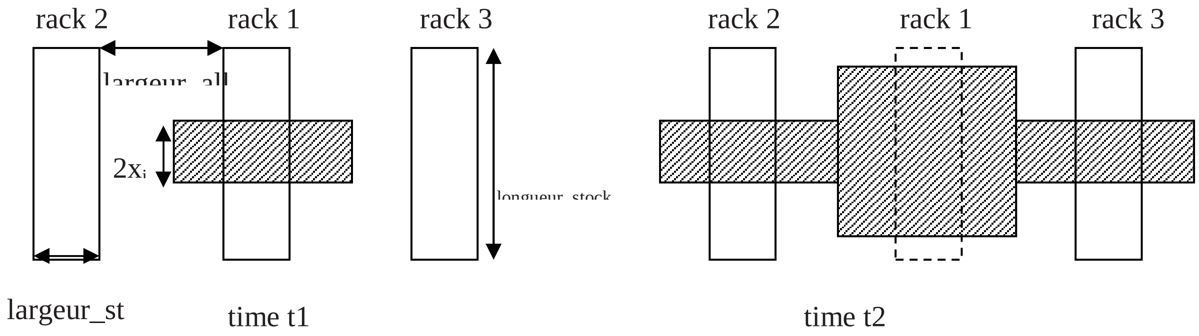


Figure 4-5 Evolution of roof surface which collapses

$$S_{feu\_plafond}(t) = \sum_i x(i,t) \times (largeur\_stock + largeur\_allée) \quad \text{Equation 4-13}$$

where *x(i,t)* =

for primary racks (i=1,2,3), abscise of flame front
for secondary rack (i>3), 0 if the rack do not burn else longueur_stock

t_{retard_toit} as well as *Coef_toit* depends on the roof structure and nature. A time limit is also introduced after which roof above storage area collapses totally.

4.6 Final Output

4.6.1 Fire characteristics: flame height and emissivity

Flame height H is estimated via a formula based on Zukoski correlation :

$$H = \min\left[\left(\frac{Ps'}{223}\right)^2 ; 0{,}026\left(P's.D\right)^{2/3}\right] \quad \text{Equation 4-14}$$

where :

Ps' : heat release rate per unit area [kW/m²]. It is the ratio $P(t)$ / $S_{chute\_toit}(t)$

D : hydraulic Diameter [m] based on 4 x $S_{chute\_toit}(t)$ / *Per*

Per : perimeter of collapsed roof surface

This formula is applicable only if $P's < 14130\ \sqrt{D}$

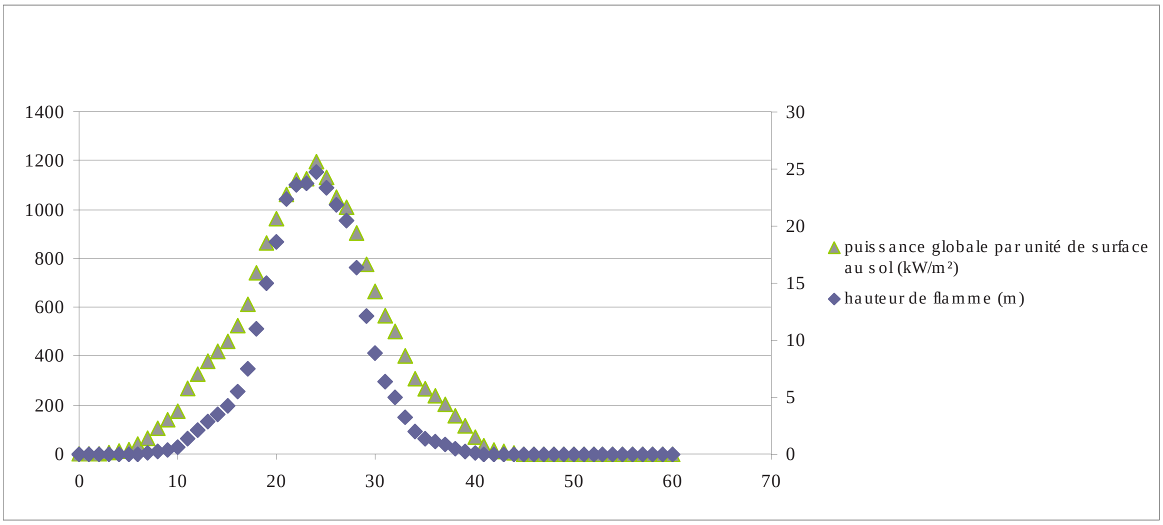


Figure 4-6 Example of flame height

Mudan's correlation is used for flame emissivity :

$$E_{moy} = E_{\max} . e^{-s.D} + E_S . \left(1 - e^{-s.D}\right) \quad \text{Equation 4-15}$$

E_{max} : equivalent blackbody emissive power, 140 kW/m² (1000 °C)

s : extinction coefficient, 0.12 m^{-1}

D : hydraulic diameter (m)

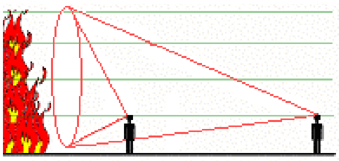
E_s : emissive power of smoke, 20 kW/m² (500 °C)

4.6.2 Flame shape, heat fluxes and associated effects

For each wall, the flame area is modeled as rectangular surface with height H, its width is limited to storage area and it is positioned at the top of collapsed roof.

Depending on their fire resistance, walls can hide radiation of lower part of the flame. Here, RE walls assumed to hide all radiation in the time limit of their fire resistance. Moreover, analysis of real fires shows that beyond this limit of stability, walls may continue to filter consequently the radiation.

Thus, wall surfaces capable of stopping radiation are calculated for the duration of fire and taken into account.



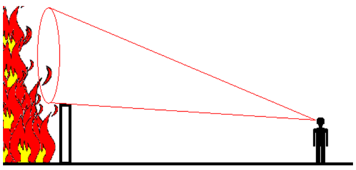


Figure 4-7 Left: without wall; right: with wall

Target position

Two types of target are considered: human people and surrounding buildings.

Human

Target's elevation is fixed to 1.8 m.

Human head is modeled forms a horizontal and vertical target. So, the total view factor is:

$$f = \sqrt{fh^2 + fv^2}$$

where f_h (respectively f_v) is the horizontal view factor (respectively vertical view factor)

Buildings

Two cases are analyzed; firstly a vertical target and secondly a horizontal target.

The vertical target corresponds to walls and target's height must be that where heat fluxes are maximum (in the limit of wall height). The horizontal target corresponds to the roof ant its height is the roof's height.

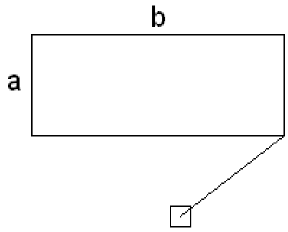
View factor

For a differential surface parallel to a finite rectangular surface, the expression for view factor is:

$$f_V = \frac{1}{2.\pi}\left(\frac{X}{\sqrt{1+X^2}}\tan^{-1}\left(\frac{Y}{\sqrt{1+X^2}}\right)+\frac{Y}{\sqrt{1+Y^2}}\tan^{-1}\left(\frac{X}{\sqrt{1+Y^2}}\right)\right)$$

$$X = \frac{a}{d}$$

$$Y = \frac{b}{d}$$

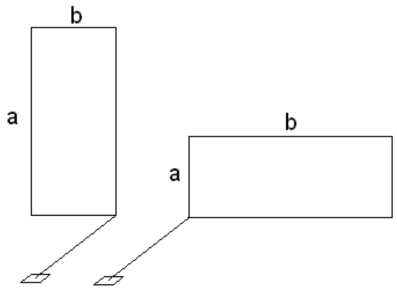


And for a differential surface perpendicular to a finite rectangular surface, the expression for view factor is:

$$f_h = \frac{1}{2.\pi}\left(\tan^{-1}\left(\frac{1}{Y}\right)-\frac{Y}{\sqrt{X^2+Y^2}}\tan^{-1}\left(\frac{1}{\sqrt{X^2+Y^2}}\right)\right)$$

$$X = \frac{a}{b}$$

$$Y = \frac{d}{b}$$



4.6.3 Atmospheric attenuation

Atmospheric attenuation is mainly due to water vapor present in the atmosphere. The Bagster correlation is used:

$\tau = 2{,}02 \,.\, (p_w \,.\, x)^{-0.09}$ Equation 4-16

Where:

x = distance between flame and target [m]

p_w = partial pressure of water in the air [Pa]

$p_w = RH.e^{\left(14{,}4114-\frac{5328}{Ta}\right)} \,.\, 1{,}013 \,.\, 10^5$

RH : relative humidity [%], default value : 70%

τ: ambient temperature [K], default value : 298 K

4.6.4 Heat fluxes

The heat flux F emitted by flame and received by the target is estimated according to:

$$F = \tau \, . \, f \, . \, E_{moy}$$ Equation 4-17

where

E_{moy} : emissivity (paragraph 4.6.1)

f : view factor (paragraph 4.6.2)

τ : atmospheric attenuation (paragraph 4.6.3)

Calculations are relatively simple because only analytical or empirical formulas are used but due to a great number of calculations (heat fluxes are calculated along all the walls and for the whole duration of fire). Therefore, it is only possible to apply this method with a software which under preparation.

In French fire regulations, several limit values of heat fluxes are of interest:

3 kW/m²

5 kW/m²

8 kW/m²

They constitute the main criterion adopted in the method to evaluate the acceptable distance.

5 Software

The objective of the software "LUCA" is to simplify work of the different engineering offices while applying calculation method presented in the Design Guide. With this simple tool, the integrity verification of the single storey building in case of fire is assured.

The Software is currently delivered in three languages (Spanish, French & English). But it gives the possibility to users to implement another language by translating a series of words and sentences in a file that is provided with the software.

The launch window contains information about the applicability conditions and the description of how to treat the results given by the software.

The user has to select between the different types of frames (simple frame, frames with cross section in H or I hot rolled profiles and frames with lattice beams and columns in H or I) he/she wishes to calculate. Further, detailed characteristic of the frame has to be specified. For example, the user has to give the type of profile of the beams and columns, the length and height of the frames, the span number in the fire compartment and in the cold part, the position of the fire compartment, the position of the fire wall (parallel or perpendicular to the frame), the total design value of the load in the roof (fire situation), etc.

All the calculation results (displacement in the expansion phase, displacements in the collapse phase, tensile forces on the top of the columns, etc.) obtained are illustrated with schematic pictures for an easier understanding and control of data.

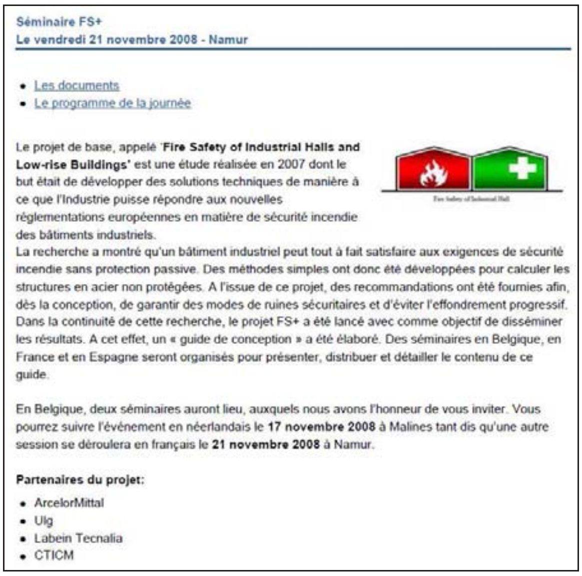
The software also gives the possibility to produce and print a report from the calculation in pdf format. This document will bring together the data that has been implemented, the intermediate results used for the final calculations, the final results and a summary of the equations used for the calculations.

6 Dissemination of Knowledge

Results of the project were presented during seminars organised in Belgium, France and Spain. Each seminar included part related to national fire regulations and alignment of the proposed calculation with the regulations.

6.1 Belgium

- There were two seminars organised in Belgium in order to respond to the bi-lingual market requirements.
- Seminars were organised by CIA (Belgium IPO) on the 17^{th} of November 2008 in Mechelen in Dutch and the 21^{st} of November 2008 in Namur in French
- The Dutch seminar was attended by 93 participants and the French seminar by 59 participants, which significantly exceeded our expectations
- The seminars had been advertised on the CIA website http://www.infosteel.be where at this moment all the documents and presentations are available for download
- All the documents and software were distributed during the seminars on USB keys.



Séminaire FS+
Le vendredi 21 novembre 2008 - Namur

- Les documents
- Le programme de la journée

Le projet de base, appelé '**Fire Safety of Industrial Halls and Low-rise Buildings'** est une étude réalisée en 2007 dont le but était de développer des solutions techniques de manière à ce que l'Industrie puisse répondre aux nouvelles réglementations européennes en matière de sécurité incendie des bâtiments industriels.
La recherche a montré qu'un bâtiment industriel peut tout à fait satisfaire aux exigences de sécurité incendie sans protection passive. Des méthodes simples ont donc été développées pour calculer les structures en acier non protégées. A l'issue de ce projet, des recommandations ont été fournies afin, dès la conception, de garantir des modes de ruines sécuritaires et d'éviter l'effondrement progressif. Dans la continuité de cette recherche, le projet FS+ a été lancé avec comme objectif de disséminer les résultats. A cet effet, un « guide de conception » a été élaboré. Des séminaires en Belgique, en France et en Espagne seront organisés pour présenter, distribuer et détailler le contenu de ce guide.

En Belgique, deux séminaires auront lieu, auxquels nous avons l'honneur de vous inviter. Vous pourrez suivre l'événement en néerlandais le **17 novembre 2008** à Malines tant dis qu'une autre session se déroulera en français le **21 novembre 2008** à Namur.

Partenaires du projet:

- ArcelorMittal
- Ulg
- Labein Tecnalia
- CTICM

'Studiedag FS+
17 november 2008 te Mechelen

- Documenten van die dag
- Het programma van die dag

Het basisproject, "Fire Safety of Industrial Halls and Low-rise Buildings" is een studie uitgevoerd in 2007. Het doel hiervan was technische oplossingen te ontwikkelen zodat de industrie zou kunnen beantwoorden aan de nieuwe Europese reglementeringen inzake brandveiligheid van industriële gebouwen.

Onderzoek heeft aangetoond dat een industrieel gebouw volledig kan voldoen aan de eisen van de brandveiligheid zonder daarom verdere passieve bescherming aan te brengen. Er werden dus eenvoudige methodes ontwikkeld om onbeschermde stalen structuren te berekenen. Na afloop van dit project werden aanbevelingen opgesteld om, vanaf het concept, een veilige bezwijkvorm te garanderen en om progressieve instorting te vermijden.

In navolging van dit basisproject en met als doel de resultaten ervan te verspreiden werd FS+ gelanceerd. Een ontwerpgids werd opgesteld. In België, Frankrijk en Spanje worden nu studiedagen georganiseerd om de inhoud van deze gids voor te stellen, te verspreiden en toe te lichten.

Partners van het project:

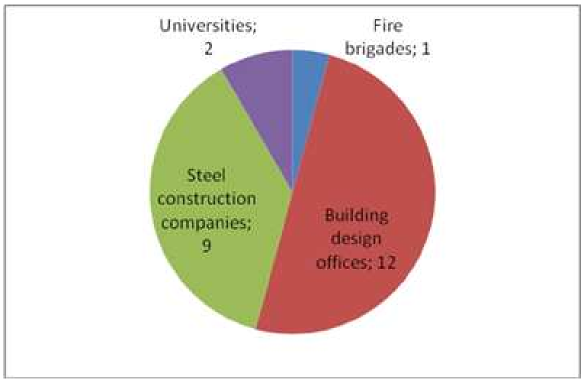
- ArcelorMittal
- Ulg
- Labein Tecnalia
- CTICM

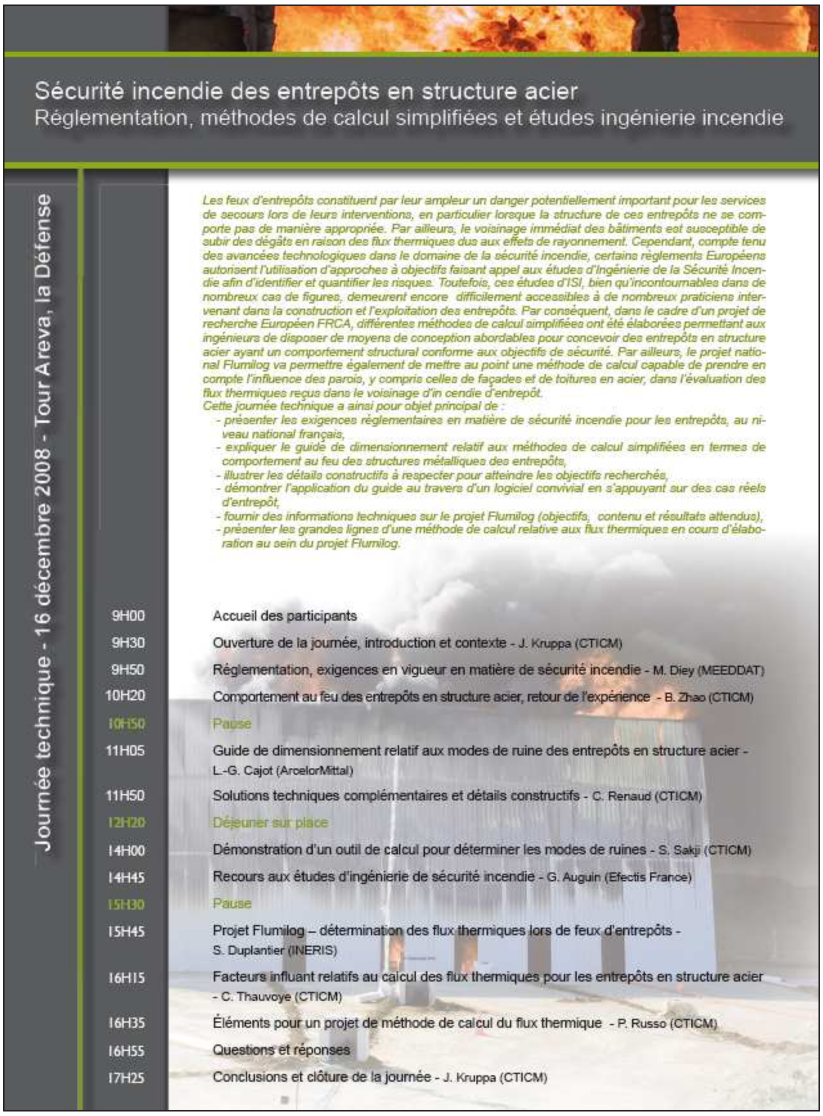
09h00	Accueil des invités
09h30	**Introduction et programme de la journée** *Jo Naessens – Centre Information Acier*
09h45	**Feux réels dans les halls industriels** *Audrey Debackere – Centre Information Acier*
10h00	**Réglementation nationale : Annexe 6** *Jean-Paul Renier – Brigade des pompiers de La Louvière*
10h45	**Champs d'application** *Jean-Marc Franssen – ULg*
11h00	Pause café
11h15	**Comportement en cas d'incendie des structures en acier et conséquences** *Jean-Marc Franssen – ULg*
11h40	**Guide de conception –** *Jean-Marc Franssen – ULg*
12h30	**Démonstration des softwares de calcul** *Jean-Baptiste Lansival – ULg*
12h50	**Questions et conclusions**
13h00	Lunch
14h00	Fin du séminaire

09u30	**Inleiding – Echte branden in industriële hallen** *Jo Naessens – Staalinfocentrum*
09u40	**Nationale reglementering : Bijlage 6 en schoolvoorbeelden uit België** *Guy Van De Gaer – Directeur van Brandweervereniging Vlaanderen*
10u25	**Toepassingsgebieden** *Luc Schueremans – Katholiek Universiteit van Leuven*
10u35	**Handelingen in geval van brand bij stalen structuren en gevolgen** *Lincy Pyl – Hogeschool Denayer*
11u00	Koffiepauze
11u15	**Ontwerpgids – Controlemiddelen en aanbevelingen ontwerp** *Luc Schueremans – Katholiek Universiteit van Leuven*
12u00	**Demonstratie van rekensoftware** *Lincy Pyl – Hogeschool Denayer*
12u25	**Toepassingsvoorbeelden** *Lincy Pyl – Hogeschool Denayer*
12u50	**Vragen en conclusies**
13h00	Lunch
14u00	Einde van de studiedag

6.2 France

- The FS+ French seminar was organised by the CTICM and took place the 16th of December in 'Tour Areva, La Defence' in Paris
- The number of participants was exactly 25 persons of different activities as explained in the next diagram





Journée technique - 16 décembre 2008 - Tour Areva, la Défense

Sécurité incendie des entrepôts en structure acier

Réglementation, méthodes de calcul simplifiées et études ingénierie incendie

Les feux d'entrepôts constituent par leur ampleur un danger potentiellement important pour les services de secours lors de leurs interventions, en particulier lorsque la structure de ces entrepôts ne se comporte pas de manière appropriée. Par ailleurs, le voisinage immédiat des bâtiments est susceptible de subir des dégâts en raison des flux thermiques dus aux effets de rayonnement. Cependant, compte tenu des avancées technologiques dans le domaine de la sécurité incendie, certains règlements Européens autorisent l'utilisation d'approches à objectifs faisant appel aux études d'Ingénierie de la Sécurité Incendie afin d'identifier et quantifier les risques. Toutefois, ces études d'ISI, bien qu'incontournables dans de nombreux cas de figures, demeurent encore difficilement accessibles à de nombreux praticiens intervenant dans la construction et l'exploitation des entrepôts. Par conséquent, dans le cadre d'un projet de recherche Européen FRCA, différentes méthodes de calcul simplifiées ont été élaborées permettant aux ingénieurs de disposer de moyens de conception abordables pour concevoir des entrepôts en structure acier ayant un comportement structural conforme aux objectifs de sécurité. Par ailleurs, le projet national Flumilog va permettre également de mettre au point une méthode de calcul capable de prendre en compte l'influence des parois, y compris celles de façades et de toitures en acier, dans l'évaluation des flux thermiques reçus dans le voisinage d'in cendie d'entrepôt.

Cette journée technique a ainsi pour objet principal de :

- *présenter les exigences réglementaires en matière de sécurité incendie pour les entrepôts, au niveau national français,*
- *expliquer le guide de dimensionnement relatif aux méthodes de calcul simplifiées en termes de comportement au feu des structures métalliques des entrepôts,*
- *illustrer les détails constructifs à respecter pour atteindre les objectifs recherchés,*
- *démontrer l'application du guide au travers d'un logiciel convivial en s'appuyant sur des cas réels d'entrepôt,*
- *fournir des informations techniques sur le projet Flumilog (objectifs, contenu et résultats attendus),*
- *présenter les grandes lignes d'une méthode de calcul relative aux flux thermiques en cours d'élaboration au sein du projet Flumilog.*

9H00	Accueil des participants
9H30	Ouverture de la journée, introduction et contexte - J. Kruppa (CTICM)
9H50	Réglementation, exigences en vigueur en matière de sécurité incendie - M. Diey (MEEDDAT)
10H20	Comportement au feu des entrepôts en structure acier, retour de l'expérience - B. Zhao (CTICM)
10H50	Pause
11H05	Guide de dimensionnement relatif aux modes de ruine des entrepôts en structure acier - L.-G. Cajot (ArcelorMittal)
11H50	Solutions techniques complémentaires et détails constructifs - C. Renaud (CTICM)
12H20	Déjeuner sur place
14H00	Démonstration d'un outil de calcul pour déterminer les modes de ruines - S. Sakji (CTICM)
14H45	Recours aux études d'ingénierie de sécurité incendie - G. Auguin (Efectis France)
15H30	Pause
15H45	Projet Flumilog – détermination des flux thermiques lors de feux d'entrepôts - S. Duplantier (INERIS)
16H15	Facteurs influant relatifs au calcul des flux thermiques pour les entrepôts en structure acier - C. Thauvoye (CTICM)
16H35	Éléments pour un projet de méthode de calcul du flux thermique - P. Russo (CTICM)
16H55	Questions et réponses
17H25	Conclusions et clôture de la journée - J. Kruppa (CTICM)

- Mr Kruppa opened the presentations speaking about the new approaches applied in fire safety and the French regulations concerning fire safety of warehouses (Mr Diey who was supposed to give this presentation was unable to attend the seminar). The following presentations were held according the program given in the flyer.

- An USB key including all technical documents and presentations of the FS+ project as well as the software was distributed to the participants. Participants received also a folder with printed PowerPoint presentations.

6.3 Spain

- Spanish seminar has been organised in Madrid on the12th of November 2008

- Considering that the target, objectives and subjects were similar for this project as well as for DIFISEK+ (RFS-CT-2007-00030), it was decided to develop an unique seminar with a wider scope, to reach a bigger impact in the Spanish construction agents (constructors, engineering offices, authorities…)

- 143 participants attended the seminar
- Invitation and the agenda of the meeting is presented on the pictures below
- During the seminar participants received all the presentations and documentations on the CD

7 Summary and Conclusion

The project has been very important for the construction especially steel construction market. It has to be remembered that the industrial halls created considerable market for steel.
Thanks to the research the engineers and steel fabricators obtained a tool enabling them to argue safety of their designs for industrial halls. Also, involvement of firemen and authorities in the seminars had a significant impact on their understanding of the material.
Number of participants of the seminars indicates clearly an importance of the subject for the construction. Fire was always considered an issue for steel structures. And this project clearly showed that this issue that can be easily overcome and the industrial halls made of steel comply with the fire safety regulations. Good knowledge and understanding of the structural behaviour developed in the previous research project had been successfully communicated to the designers, fabricators, authorities and fireman.

List of figures

Main document

Design Guide

Background Document

List of tables

Main Document

Background Document

8 References

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Appendices

Project carried out with a financial grant of the
Research Programme of the Research Fund for Coal and Steel
RFS2-CR-2007-00032

Fire Safety of Industrial Hall

Design Guide

1 Introduction

This design guide is a response of the steel industry to the new fire safety regulations introduced recently in many European countries.

As a result of extensive research works, financed by RFCS project [6], the methodology and recommendations for design of single storey industrial halls were developed. The results are derived from numerical and parametric study performed for portal and lattice frames on number of various height and spans of the structures. The ISO fire curve has been used for the simulations.

Design beyond the scope of analysis will not be recommended unless treated as preliminary design, which will be farther validated.

The sizes considered in the project are typical for industrial halls as listed:

- ⇒ Length of a span for simple bay: 15m, 20m and 30m
- ⇒ Length of a span for multiple bay: 20m, 30m and 40m
- ⇒ Height – simple bay: 5m, 7.5m and 12.5m
- ⇒ Height – multiple bay: 7.5m, 12.5m and 20m
- ⇒ Slope: 0° to 10°
- ⇒ Number of bays: 1, 3 and 5
- ⇒ Lattice beam: equal angles 50x50x5 till 120x120x12

The primary aim of the research work was to prove that in absence of passive fire protection the fire safety of steel single storey industrial buildings is sufficient. By means of risk assessment and structural simulation it has been shown that the safety of occupants and firemen is guaranteed by the following criteria:

- **criteria of “no collapse towards the outside”. In case of fire occurring in one of the building compartments, the structure does not collapse towards the outside of the building.**
- **criteria of “no progressive collapse”. In case of fire occurring in one of the building compartments, the localized failure of the compartment does not lead to the collapse of the adjacent compartments.**

The objective of this design guide is to provide engineering offices with simplified design rules and calculation methods ensuring that the structural behaviour (load-bearing structure, façade elements, roofing and fire walls) of the industrial building follows the above criteria satisfying the safety objectives for peoples (occupants and firemen) in terms of structural behaviour.

2 Behaviour of structures in fire

The behaviour of multiple bay portal frame structures in fire conditions can be divided in two successive phases leading to different structural behaviours. [1]

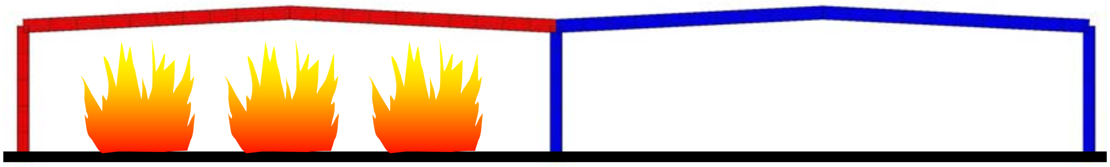


Figure 2.1 Heating condition

First phase corresponds to thermal expansion of heated members. During this phase, the following events are observed:

- **a progressive increase of lateral displacements towards the outside of the fire compartment at the top of the columns supporting the roof structures;**
- **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 part of the structure;**

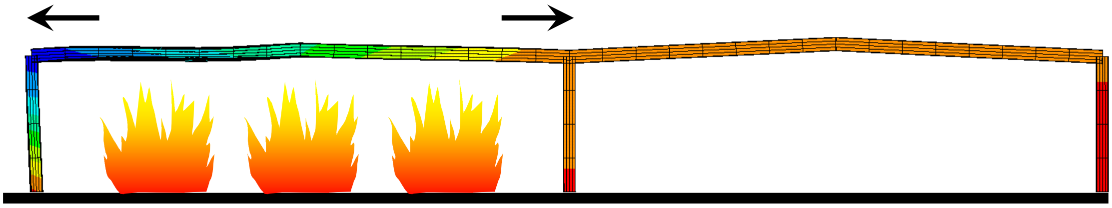
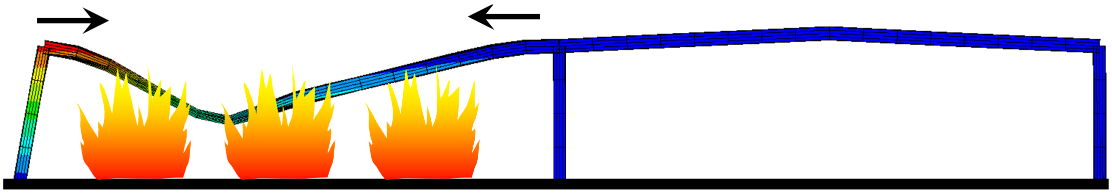


Figure 2.2 Deformed shape during the expansion phase

The second phase corresponds to the collapse of the heated part of the structure. During this phase the following observations are made:

- **beam changes progressively from combined compression and bending state to simple tensile state;**
- **from the beginning of this phase, displacements at the compartment ends change in direction: the top of external columns go back to initial state and finally move towards the fire compartment;**



[1] Note: A very important assumption in the behaviour analyses presented hereafter is that the internal columns at the position of fire walls remain at room temperature.

Figure 2.3 Deformed shape during the collapsing phase

- **the heated beam behaves as a chain under significant tensile force;**
- **the lateral displacement at the top of unheated compartment edge columns and the tensile force reach the maximum point and then decrease slightly due to the collapse of the heated beam;**
- **if the stiffness of the cold part is strong enough, in final phase, the heated structure collapses inside the fire compartment. If the strength of the cold part is strong enough, the cold part remains standing, without further collapse.**

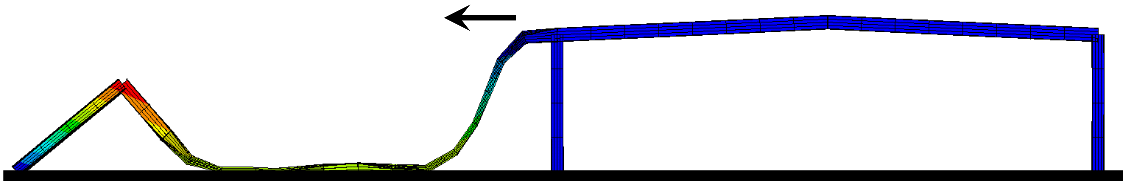


Figure 2.4 Deformed shape at he end of the collapsing phase

3 Field of application

3.1 What the guide does not do

This design guide does not:

- **explain how to calculate fire resistance of structures;**
- **define fire resistance requested by regulations;**
- **explain how to calculate stability of cold structure;**
- **show how to design facades or fire walls.**

3.2 What the guide does do

This design guide does illustrate possible failure modes of industrial halls that have to be avoided and proposes some methods to avoid these failure modes. The failures discussed are as follow:

- **collapse of a structure towards the outside;**
- **collapse of facades and fire walls towards the outside;**

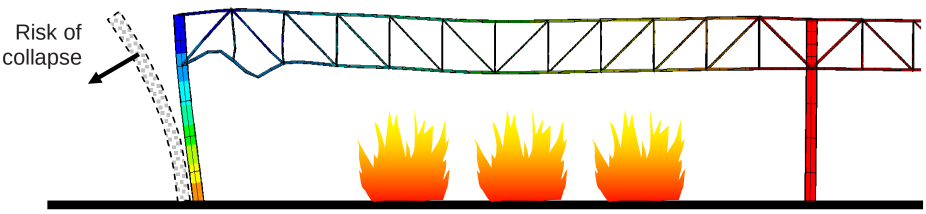


Figure 3-1 Risk of collapse of facade elements towards the outside during the expansion phase

- **collapse of adjacent cold structures – progressive collapse.**

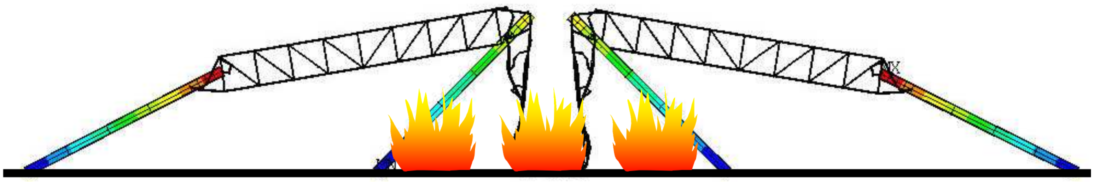


Figure 3-2 Risk of progressive collapse of the load bearing structure

3.3 Structure and compartmentalisation of storage buildings

The present document applies to storage buildings satisfying the following conditions:

- **storage buildings with steel structure; either in steel portal frames with cross section in standard H or I hot rolled profiles or equivalent welded plate girders, or steel frames based on lattice beams with columns in standard H or I hot rolled profiles or equivalent welded plate girders;**
- **storage buildings divided in one or several cells separated one from each other by fire walls. These walls can be either perpendicular to the steel portal frames or parallel to the steel portal frames.**

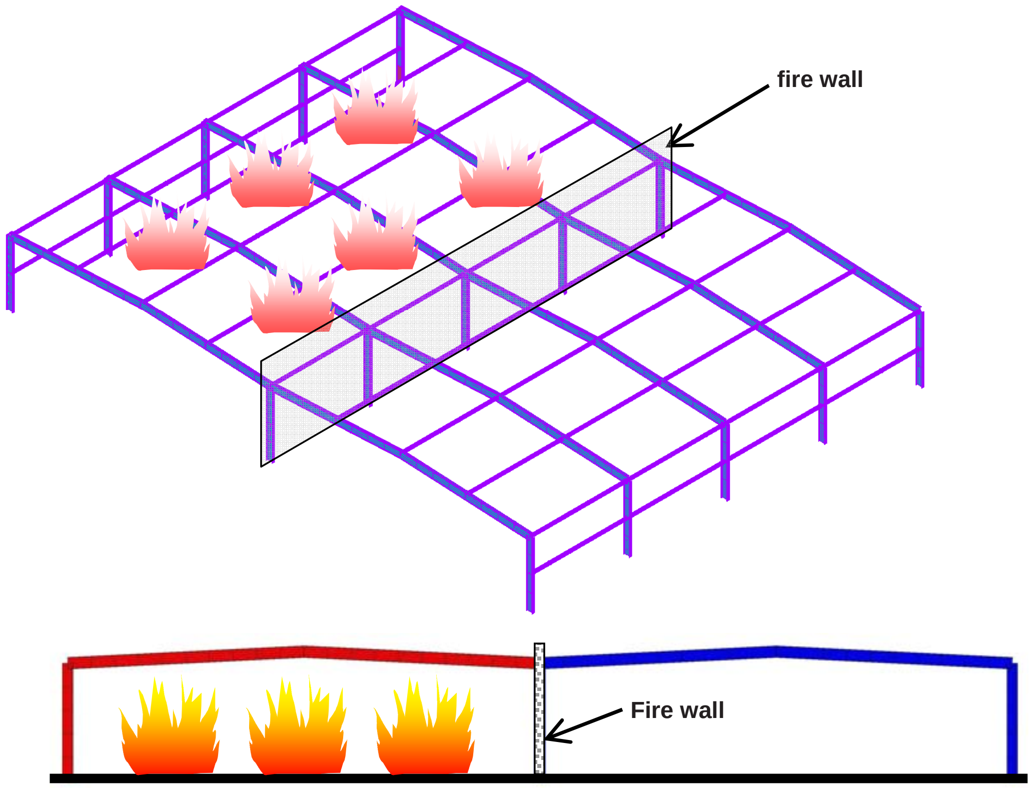


Figure 3-3 Fire wall perpendicular to portal frame

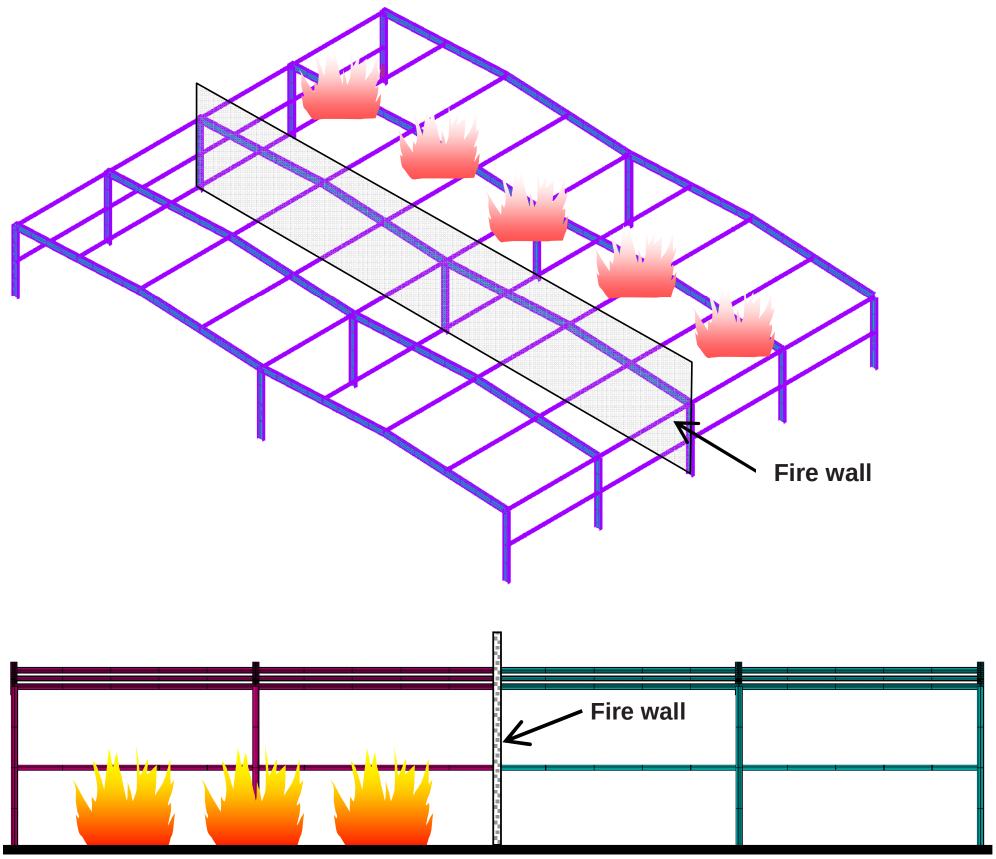


Figure 3-4 Fire wall parallel to portal frame

3.4 Fire walls and façade elements

Recommendations proposed in Section 5 of the present document can be applied to any type of fire wall, such as walls made of lightweight concrete, reinforced concrete, hollow block, steel sheeting with insulator, plasterboard, bricks, or built with any other material.

However, the fire walls must be sufficiently flexible or fixed in a suitable way to remain compatible with the lateral displacements of the steel framework under fire condition.

Use of façade elements is not limited for storage buildings. Nevertheless, whatever the type of façade, its structural adequacy, its integrity and its compatibility with respect to the movement of the steel framework must be ensured. In this way, the elements will fail with the framework towards the inside of the building in case of collapse.

The utilisation of self-stable façades is not recommended because, as a consequence of thermal bowing effects, they always move towards the outside. These façades 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.

In addition, during the expansion phase the structure moves towards the outside although it may not collapse at that stage. Consequently, façade elements must be capable of absorbing this movement. Afterwards, the structure moves in opposite direction and falls down towards the inside (see Section 2). The façade elements must be attached to the steel structure in a way that they fall down together towards the inside of the building.

4 Design method

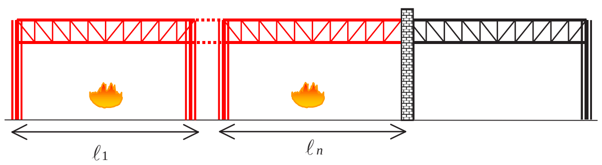
4.1 Means of checking

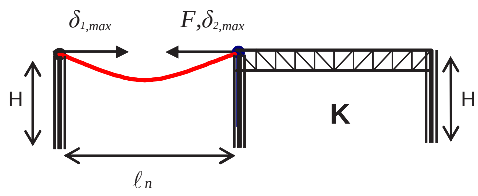
➢ **Collapse toward the outside:**

Assessment of possible collapse of the structure towards the outside of the fire compartment.

➢ **Tensile force:**

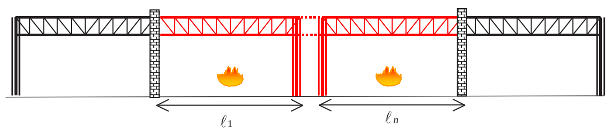
Calculation of tensile forces that appear at the top of cold part of the portal frame as a result of fire in the adjacent compartment. The forces enable stability check of the remaining cold structure.



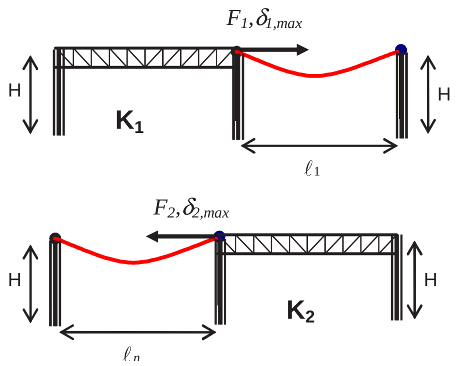


Case of n heated spans

a) Fire compartment at the end of the storage building



Case of n heated spans

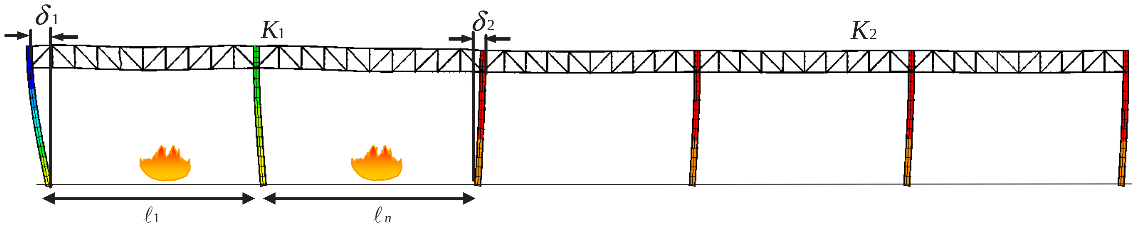


b) Fire compartment at the middle of the storage building

Figure 4-1 Maximum displacements and forces transmitted to cold parts of the structure

➢ **Lateral displacements:**

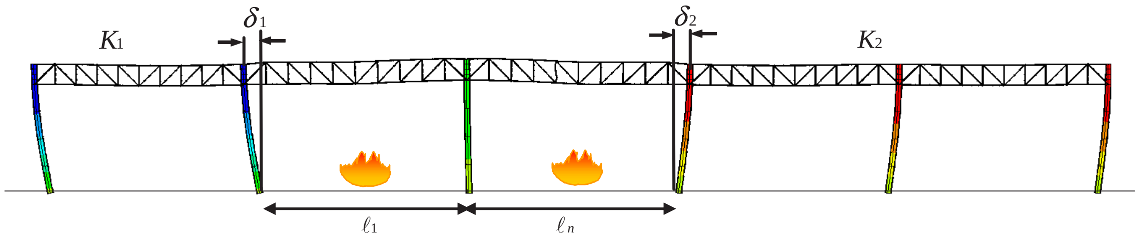
Calculation of maximum lateral displacements that appear at the top of the heated part of the frame as a result of the thermal expansion of the beams in the fire compartment. The maximum lateral displacement is used to assess the stability of the fire walls and facades.



K_2 is the lateral stiffness of the steel framework of the cold part of the structure.

K_1 is the lateral stiffness of the steel framework of the fire compartment compartment

a) Fire compartment at one end of the storage building



K_1 and K_2 are the equivalent lateral stiffness of the steel frameworks of cold parts of the structure

b) Fire compartment in the middle of the storage building

Figure 4-2 Lateral displacements of the structure during expansion phase

4.2 Cases

4.2.1 Single bay

- **Collapse towards the outside:**

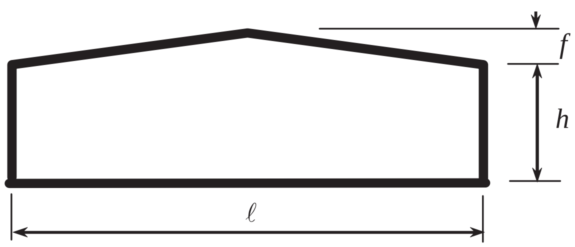
The collapse towards the outside of the compartment is avoided if Equation 4-1 is fulfilled:

$$h/l < 0.4 \qquad \text{Equation 4-1}$$

where

$h-$is the height of the columns

$l-$is the span of the beam



- **Tensile force:**

Not applicable

- **Lateral displacement:**

$$\delta_i = 0.5\% \qquad \text{Equation 4-2}$$

4.2.2 Multiple bays – fire wall perpendicular to the main frames – portal and lattice frames

- **Collapse towards the outside:**

Never occurs [6] for buildings up to 20m of height

- **Tensile force:**

Step 1 – Coefficient related to the slope of the roof c_p

⇒ Portal frame

$$c_p = \begin{cases} 1.19 & for \quad 0\%\ slope \\ 1.16 & for \quad 5\%\ slope \\ 1.10 & for \quad 10\%\ slope \end{cases}$$ Equation 4-3

⇒ Lattice frame

$$c_p = 1.45$$ Equation 4-4

Step 2 – Coefficient related to the number of heated bays n in the fire compartment n_{eff}

⇒ Portal frame

$$n_{eff} = \begin{cases} \left.\begin{matrix} 0.5 & at\ the\ end\ of\ the\ frame \\ 1.0 & in\ the\ middle\ of\ the\ frame \end{matrix}\right\} & for\ \ n = 1\ (one\ bay\ under\ fire) \\ \left.\begin{matrix} 1.0 & at\ the\ end\ of\ the\ frame \\ 2.0 & in\ the\ middle\ of\ the\ frame \end{matrix}\right\} & for\ \ n \geq 2\ (more\ than\ one\ bay\ under\ fire) \end{cases}$$

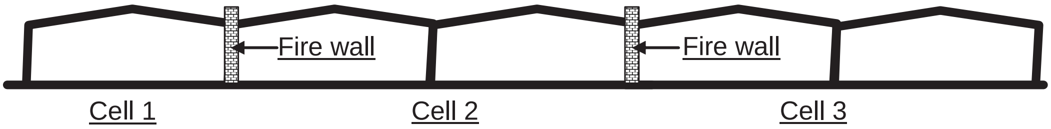
Equation 4-5

⇒ Lattice frame

$$n_{eff} = \begin{cases} \left.\begin{matrix} 0.6 & at\ the\ end\ of\ the\ frame \\ 1.0 & in\ the\ middle\ of\ the\ frame \end{matrix}\right\} & for\ \ n = 1\ (one\ bay\ under\ fire) \\ \left.\begin{matrix} 1.0 & at\ the\ end\ of\ the\ frame \\ 1.0 & in\ the\ middle\ of\ the\ frame \end{matrix}\right\} & for\ \ n \geq 2\ (more\ than\ one\ bay\ under\ fire) \end{cases}$$

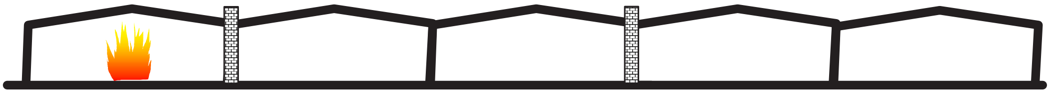
Equation 4-6

Example

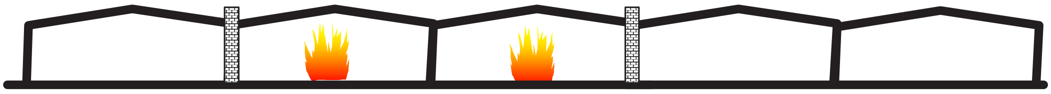


Configuration of a storage building (portal frame): 5 spans and 3 compartments

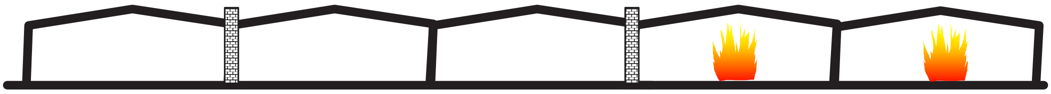
3 fire scenarios need to be considered



Scenario 1: fire in cell 1, end of the frame, one bay under fire n=1, n_{eff}=0.5



Scenario 2: fire in cell 2, middle of the frame, two bays under fire n=2, n_{eff}=2.0



Scenario 3: fire in cell 3, end of the frame, two bays under fire n=2, n_{eff} =1.0

Figure 4-3 Possible fire scenarios in a storage building with 3 compartments

Step 3 – Vertical load[2] q [N/m]

$$q = G + 0.2S \qquad \text{Equation 4-7}$$

where

G – is the dead load

S – is the characteristic snow load in fire conditions

Step 4 – Tensile force F [N] on top of the columns (Figure 4-1)

$$F = c_p \, n_{eff} \; q \, l \qquad \text{Equation 4-8}$$

where

l – is the span of on heated bay connected to the column

[2] Note: The design value of the applied load in the fire situation “q” shall be calculated, if necessary, according to load combination coefficients defined in corresponding national annexes instead of using Equation 4-7.

- **Lateral displacement:**

Step1 – Reduction factor related to the slope of the roof c_{th}

⇒ Portal frame

$$c_{th} = \begin{cases} 0.01 & for\ 0\%\ slope \\ 0.011 & for\ 5\%\ slope \\ 0.015 & for\ 10\%\ slope \end{cases}$$ Equation 4-9

⇒ Lattice frame

$$c_{th} = 0.009$$ Equation 4-10

Step 2 – Equivalent lateral stiffness K_i [N/m] of the cold part of the steel frame

⇒ If fire compartment is in the middle of the frame as illustrated in Figure 4-8

K_1 and K_2 should be calculated by one of the classical elastic methods.

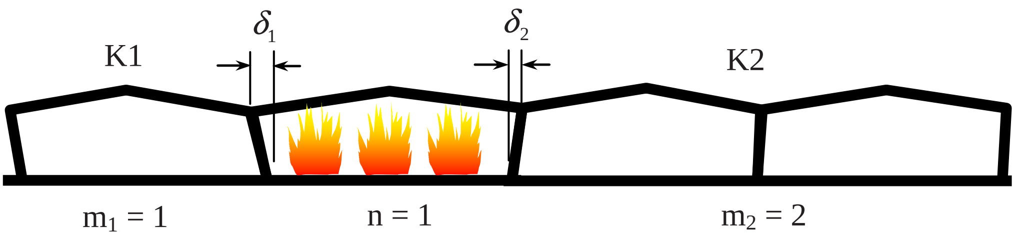


Figure 4-4 Fire located in a cell at the middle of the storage hall

Notice:

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 according to the cold span number mi using the following relations:

For $m_i = 1$:

$$K_i = k$$ Equation 4-11

For $m_i \geq 2$:

$$K_i = c\,k$$ Equation 4-12

$$k = \frac{\alpha}{1+2\alpha}\frac{12EI_c}{(h+f)^3}$$

$$c = 1 + \sum_{i=2}^{m} \frac{i}{2}\frac{2\alpha+1}{1+2i\alpha} \qquad \text{Equation 4-13}$$

$$\alpha = \frac{I_b}{I_c}\frac{h+f}{l}(1-\frac{f}{0.6h})$$

where (as indicated in Figure 4-4):

h – is the height of the columns [m]

f – is the ridging [m]

l – is the length of the span [m]

I_b – is the second moment of area of the beam [m4]

I_c – is the second moment of area of the column [m4]

E – is the modulus of elasticity of steel for normal temperature [N/m²]

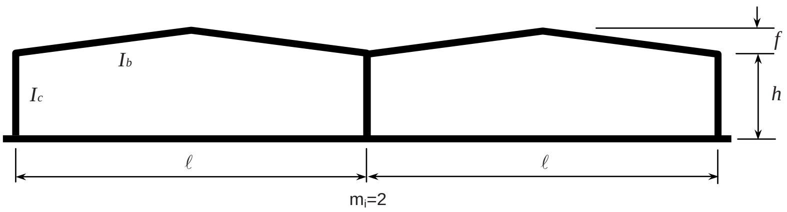


Figure 4-5 Definition of parameters of cold parts

⇒ If fire compartment is at the end of the frame

K_2 should be calculated as for fire in the middle compartment

K_1, which is defined as the lateral stiffness of the steel frame of the heated fire compartment, should be calculated as follows:

$$K_1 = \begin{cases} \left.\begin{array}{ll} 0.065\,k & for\ n=1 \\ 0.013\,k & for\ n=2 \\ 0.013\,ck & for\ n>2 \end{array}\right\} & for\ \ portal\ \ frame \\ \left.\begin{array}{ll} 0.2\,K_2 & for\ n=1 \\ 0.3\,K_2 & for\ n\geq 2 \end{array}\right\} & for\ \ lattece\ \ frame \end{cases} \qquad \text{Equation 4-14}$$

where k and c calculated from Equation 4-13 with $m = n - 1$, hence n is the number of heated bays as shown in Figure 4-6.

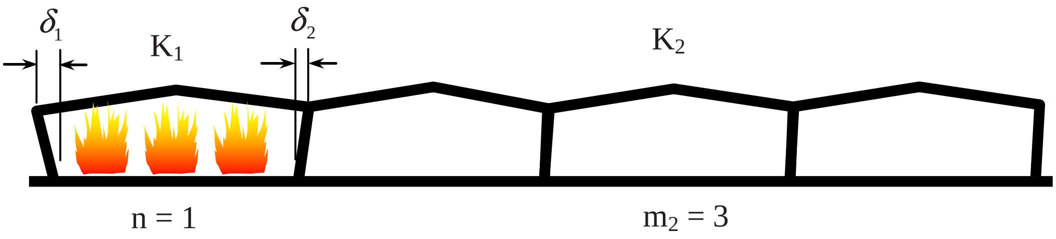


Figure 4-6 Fire in a compartment at the end of the storage building

Step 3 – Lateral displacements δ_i in the expansion phase (Figure 4-2, 4-6)

$$\delta_i = \begin{cases} \dfrac{K_t}{K_i} c_{th} \sum_{i=0}^{n} l_i & \text{for the lattice frame} \\ \dfrac{K_t}{K_i} c_{th}\ n\, l & \text{at the end of portal frame} \\ \dfrac{c_p\, q}{K_i} n_{eff}\ l & \text{for the middle of portal frame} \end{cases} \qquad \text{Equation 4-15}$$

where

n - is a number of heated spans

$K_t = \dfrac{K_1\, K_2}{K_1 + K_2}$, with K_1, K_2 equivalent stiffness for the lateral displacements of steel frame (Figure 4-6)

Step 4 – Maximum displacement $\delta_{max,i}$ induced by tensile force at the top of columns (Figure 4-1)

$$\delta_{max,i} = {F}/{K_i} \qquad \text{Equation 4-16}$$

where

F - is the tensile force calculated in $F = c_p\, n_{eff}\ q\ l$ Equation 4-8

4.2.3 Multiple bays – fire wall parallel to the main frames – portal and lattice frames

Risk of collapse towards the outside and progressive collapse (between different fire compartments) can be avoided simply just complying with some recommendations given in section 6.2.

4.3 How to use the values

The tensile force F calculated at the top of the cold frame (Equation 4-8) should be used as additional horizontal load for stability check of the frame remaining after the fire.

The stability check should be done with steel considered at ambient temperature but in the fire situation according to national annex for Eurocode (adequate load combination and coefficients).

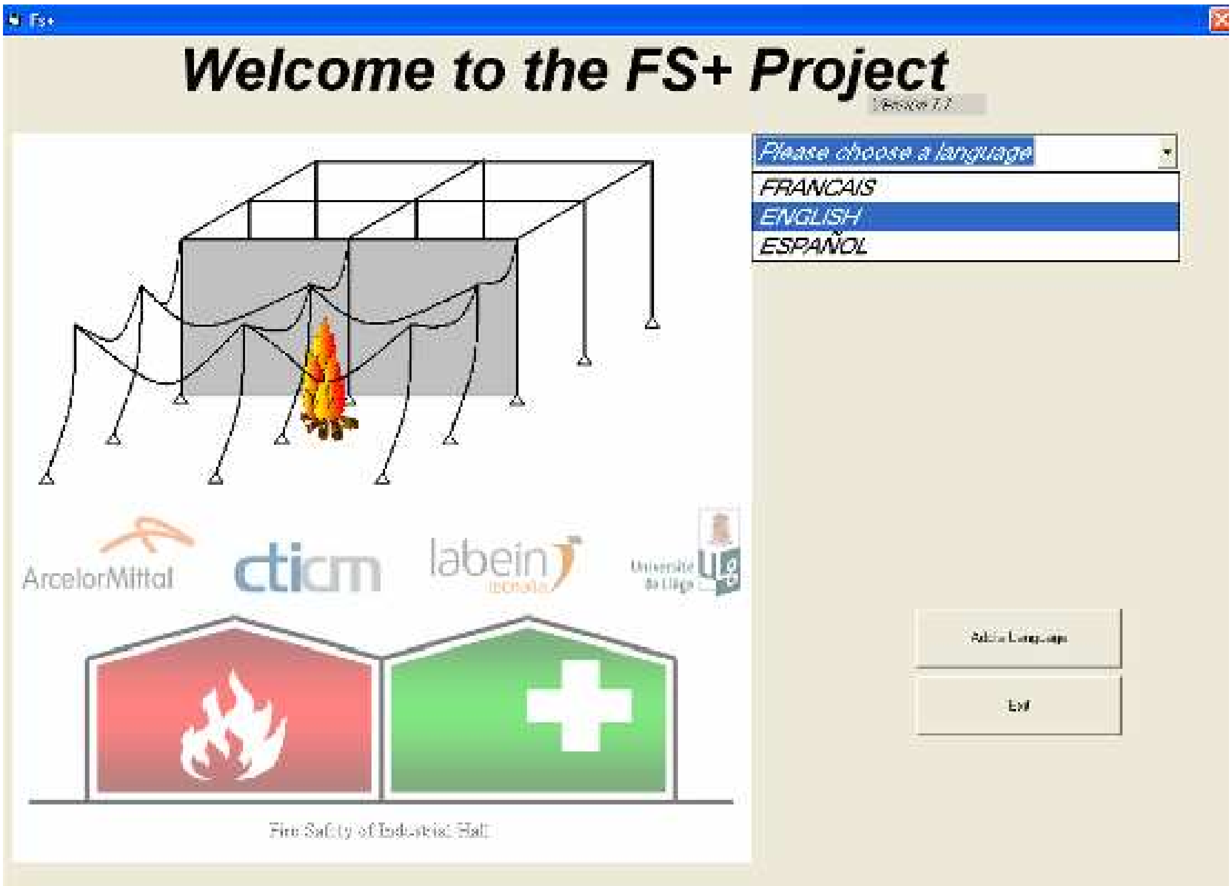
The maximum lateral displacement calculated at the top of the remaining cold frame should be used to check stability of the fire wall and façade elements. Method for this verification depends on the type of the wall, connections to the frame etc. and therefore it is not included in this design guide.

5 Software “LUCA”

5.1 Introduction

The objective of the software LUCA is to simplify the works of the different engineering offices while applying calculation method presented in this Design Guide.

With this simple tool, the integrity verification of the single storey building in case of fire is simplified.



5.2 Description, Input & Output

The Software is delivered in three languages (Spanish, French & English). But the whole Program FS+ has been implemented to give the capability to users to translate it easily into another language. The user, who wants to work with the program written in another language than the one previously given, will just have to translate a series of words and sentences in a file that will be given with the software.

The launch window is configured to propose the choice of languages (English, Spanish and French). Once the language is selected from the drop down menu all the following comments are in this language and the second window appears.

This window contains the applicability conditions and the description of how to treat the results given by the software.

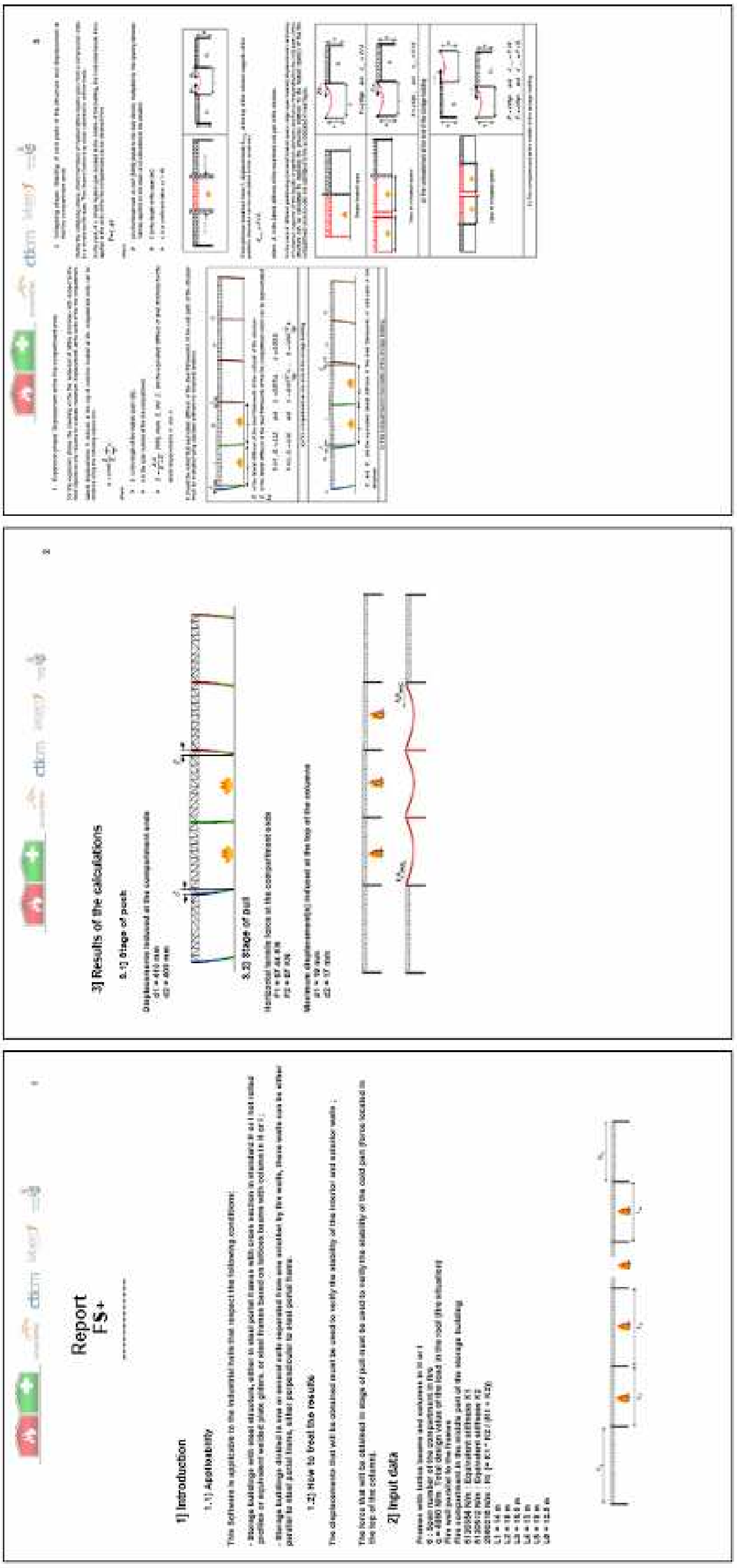
On the third window the user must select between the different types of frames (simple frame, frames with cross section in H or I hot rolled profiles and frames with lattice beams and columns in H or I).

Once this choice made, another window appears listing data that is necessary for the calculations and has to be specified by the user. For example, the user has to give the type of profile of the beams and columns, the length and height of the frames, the span number in the fire compartment and in the cold part, the position of the fire compartment, the position of the fire wall (parallel or perpendicular to the frame), the total design value of the load in the roof (fire situation), etc.

Once this information is provided, a button called ‘Next’ appears at the bottom of the page. If this button is ‘clicked’, another page appears with all the calculation results (displacement in the expansion phase, displacements in the collapse phase, tensile forces on the top of the columns, etc.).

All these results are illustrated with schematic pictures for an easier understanding and control of data.

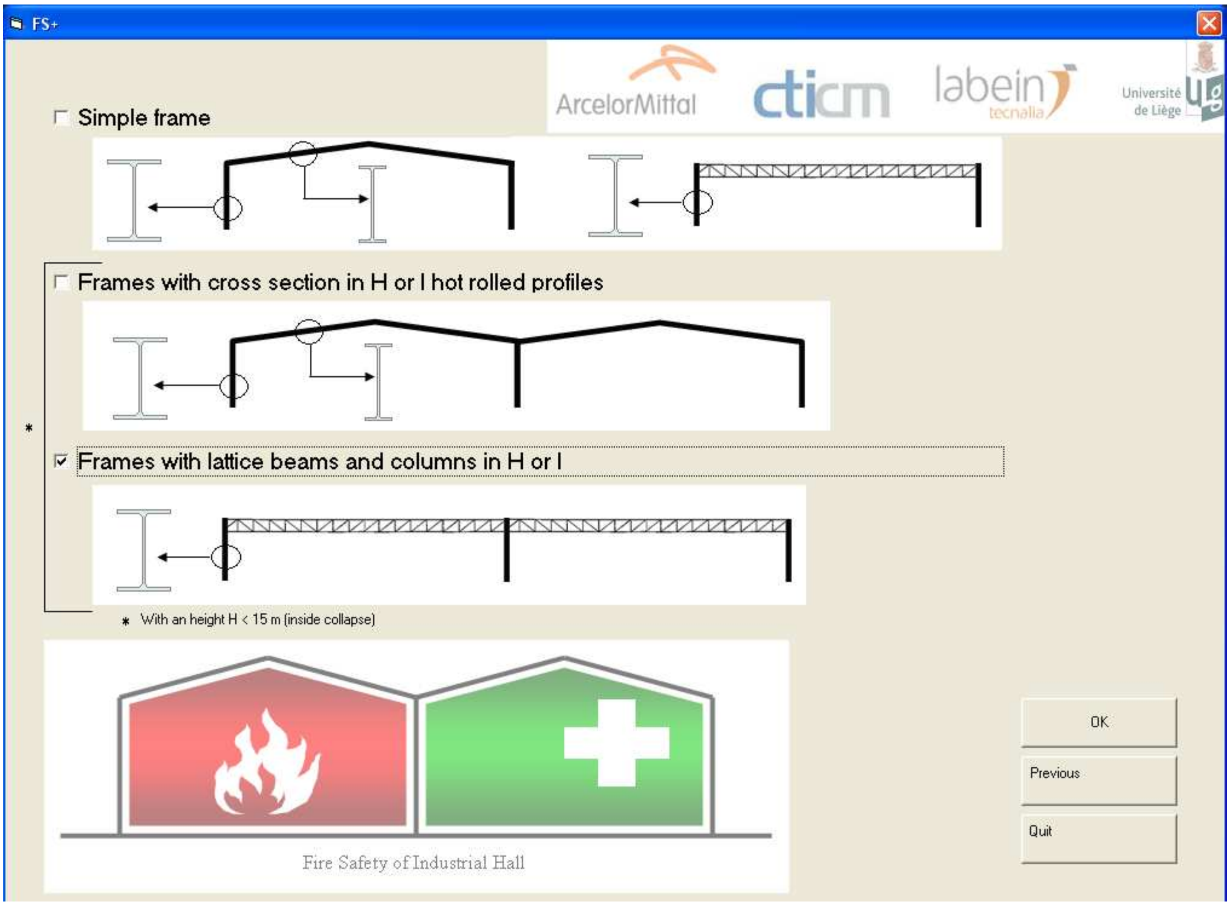
5.3 Reports

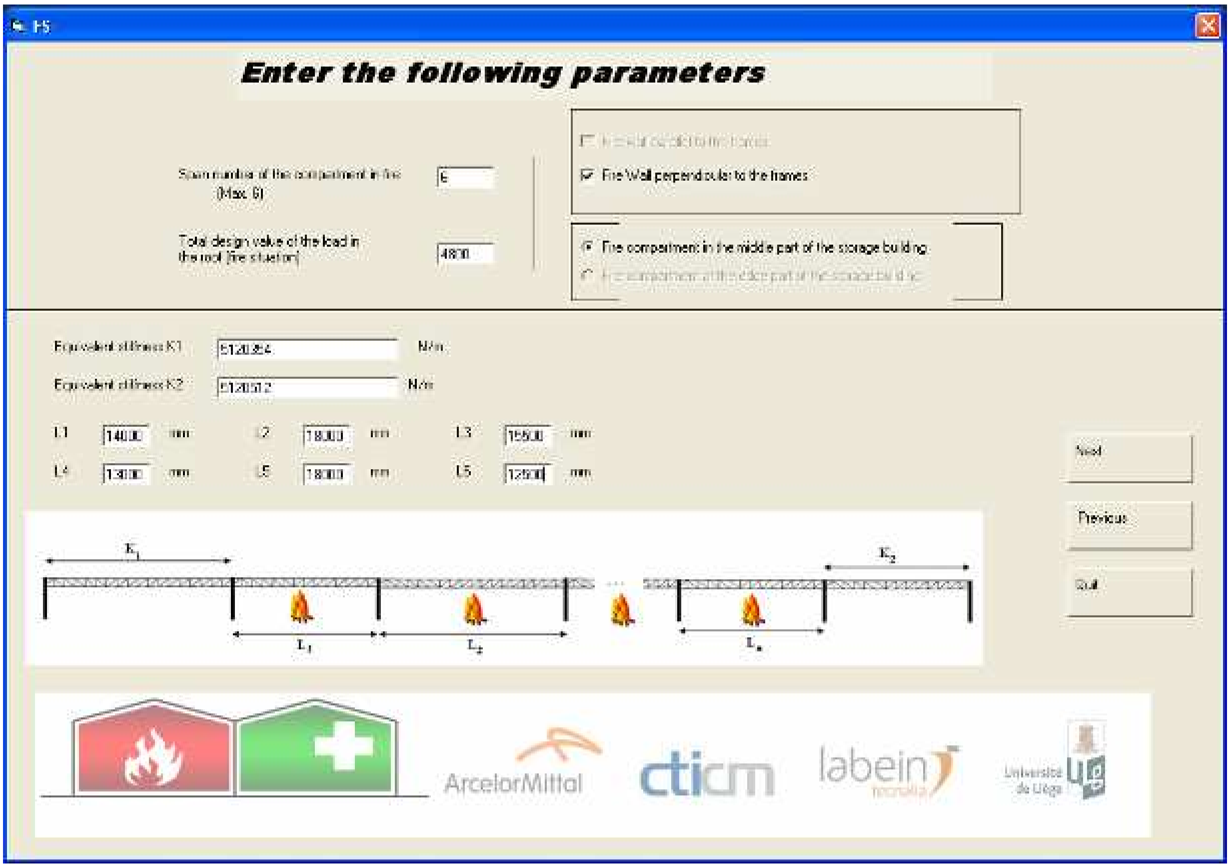


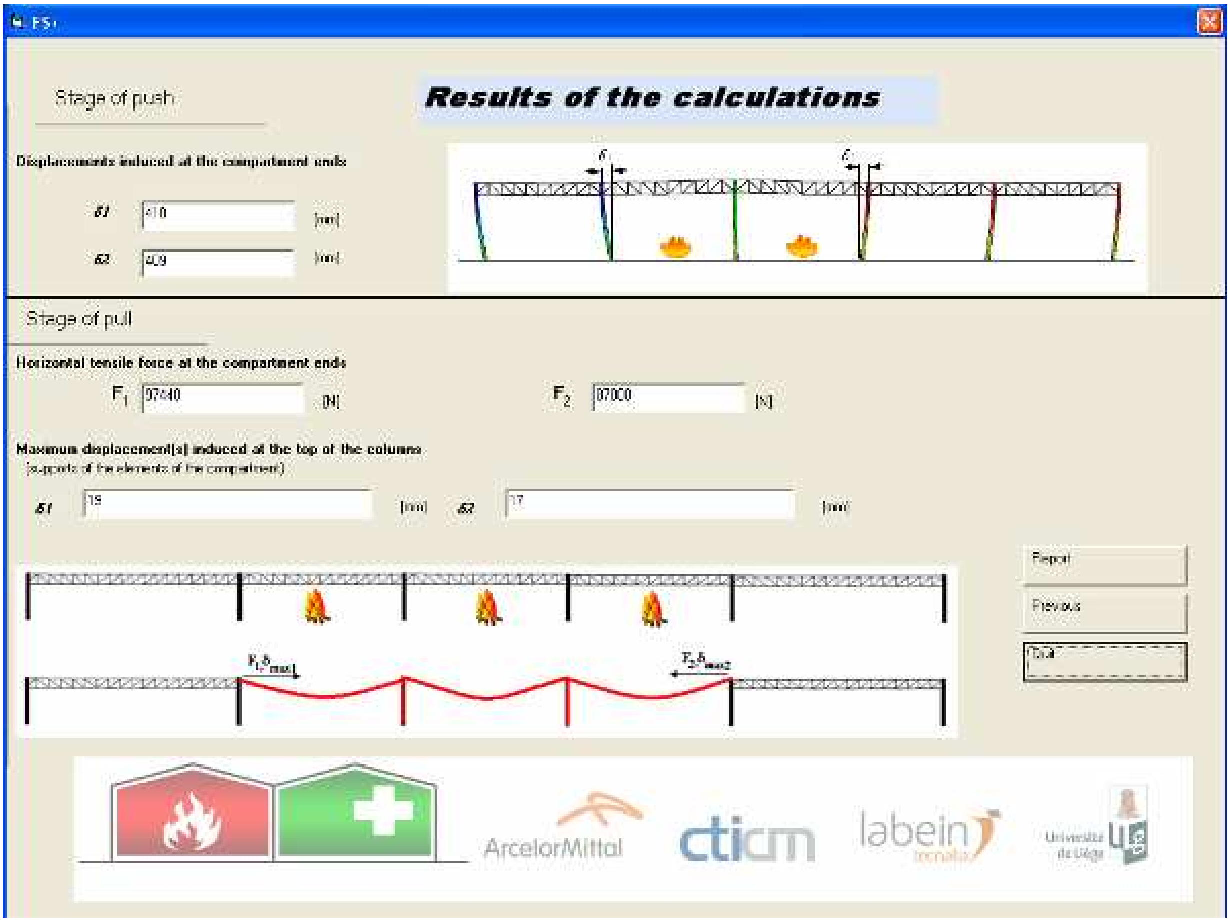
By clicking on a button ‘Print’ the software will produce document in pdf format. The document will contain a report of the calculation performed. The software will identify a ‘pdfwritter’ to produce the report in electronic form or if the user’s computer does not have the ‘pdfwritter’ it will directly print the report on the default printer. This document will bring together the data that has been implemented, the intermediate results used for the final calculations, the final results and a summary of the equations used for the calculations.

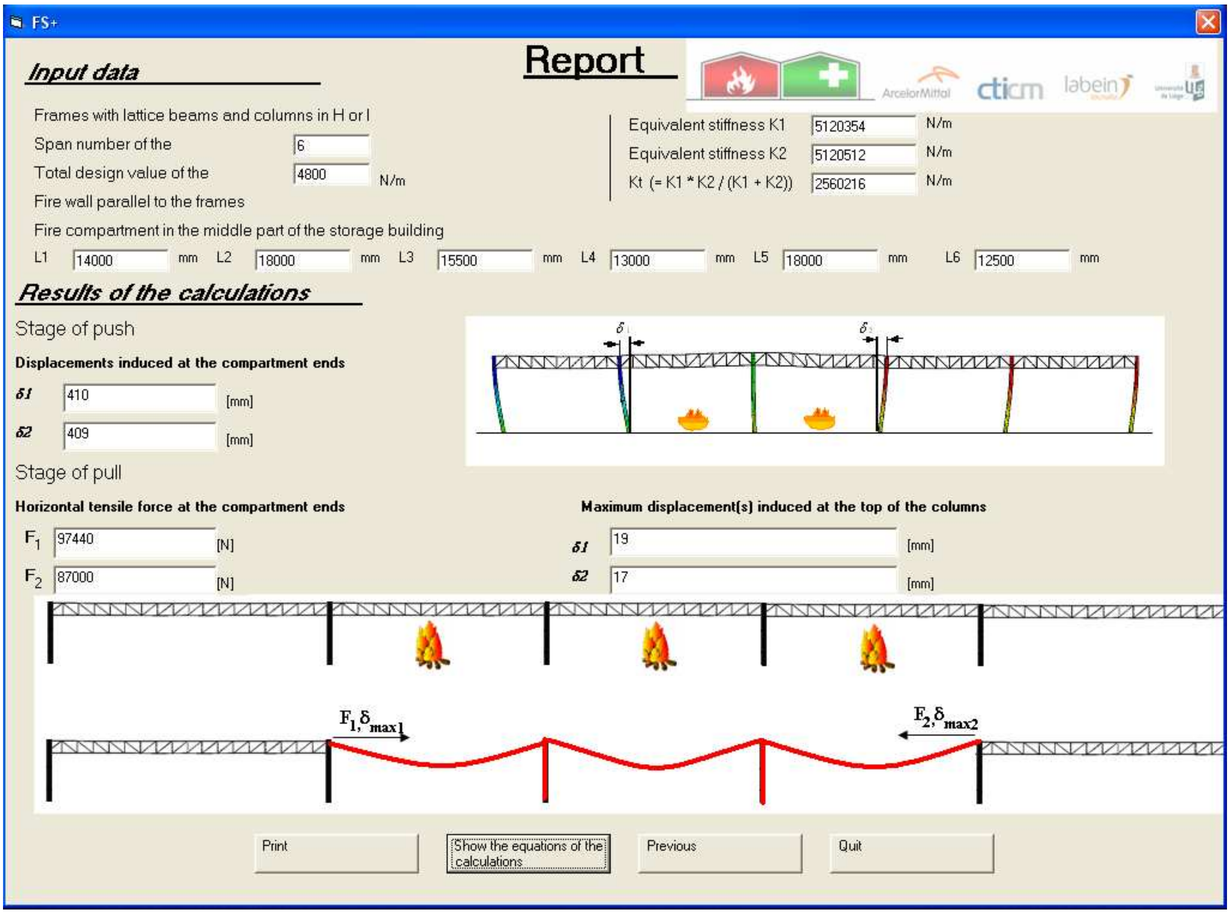
The summary of this Design Guide can also be open directly by a ‘click’ on a button called ‘see the equations of the calculations’.

5.4 Screen shots from the software




Enter the following parameters
Fire Wall perpendicular to the frames
Span number of the compartment in fire (Max 6)
6
Total design value of the load in the roof (fire situation)
4800
Fire compartment in the middle part of the storage building
Equivalent stiffness K1
5120264
N/m
Equivalent stiffness K2
5120512
N/m
L1 14000 mm
L2 18000 mm
L3 15500 mm
L4 13000 mm
L5 18000 mm
L6 12500 mm
Next
Previous
Quit
ArcelorMittal
cticm
labein tecnalia
Université de Liège


Stage of push
Results of the calculations
Displacements induced at the compartment ends
δ1 410 [mm]
δ2 409 [mm]
Stage of pull
Horizontal tensile force at the compartment ends
F1 97440 [N]
F2 97000 [N]
Maximum displacement(s) induced at the top of the columns
(supports of the elements of the compartment)
δ1 19 [mm]
δ2 17 [mm]
Report
Previous
Quit
ArcelorMittal
cticm
labein tecnalia
Université de Liège


FS+
Report
Input data
Frames with lattice beams and columns in H or I
Span number of the
6
Total design value of the
4800
N/m
Fire wall parallel to the frames
Fire compartment in the middle part of the storage building
Equivalent stiffness K1
5120354
N/m
Equivalent stiffness K2
5120512
N/m
Kt (= K1 * K2 / (K1 + K2))
2560216
N/m
L1
14000
mm
L2
18000
mm
L3
15500
mm
L4
13000
mm
L5
18000
mm
L6
12500
mm
Results of the calculations
Stage of push
Displacements induced at the compartment ends
δ1
410
[mm]
δ2
409
[mm]
Stage of pull
Horizontal tensile force at the compartment ends
F1
97440
[N]
F2
87000
[N]
Maximum displacement(s) induced at the top of the columns
δ1
19
[mm]
δ2
17
[mm]
F1, δmax1
F2, δmax2
Print
Show the equations of the calculations
Previous
Quit

6 Design recommendation

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.

6.1 Fire walls

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.

6.1.1 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.

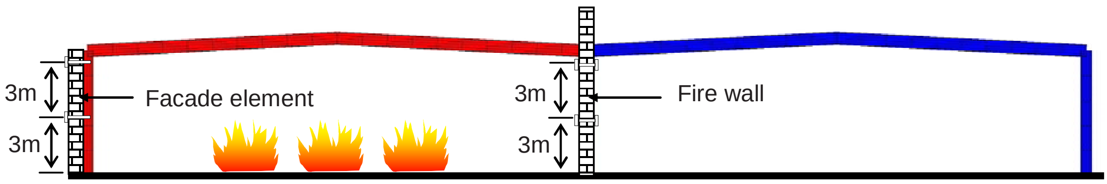


Figure 6-1 Design detail for separation elements

A solution consists in fixing these elements with the columns of the load-bearing structure, by means of suitable attachment systems 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-site walls (concrete, masonry…)

In addition, fastenings used to connect fire walls and façade elements on the columns must be designed to resist the forces produced due to wind and self-weight of partition elements under the effect of the lateral displacement induced by the steel frame of the storage building. If these fastenings are in steel and unprotected against fire, each of them must be designed at ambient temperature to support the following extracting force:

$$F = W + 5\, p\, \delta_i\, d / n \qquad \text{Equation 6-1}$$

where

W – is the characteristic wind load used for the design at ambient temperature and applied to each fastening

p – is the self-weight of the wall

d – is the spacing between frame

n – is the total number of fastening (uniformly distributed along the height)

δ_i – is the lateral displacement by the steel structure

6.1.2 Steel structures near separation elements

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. Therefore fire protection has to be applied to some part of the beam and columns:

- **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.**

6.1.3 Roof system above the separation elements

The roof shall be independent from one compartment to the others.

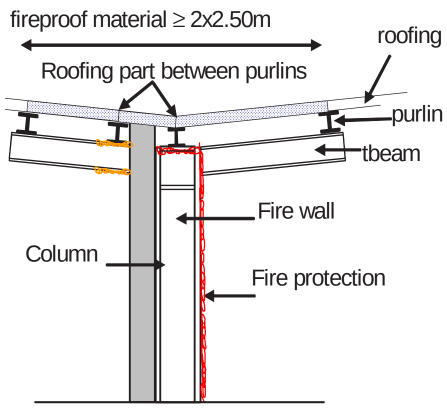


Figure 6-2 Protection of the roof

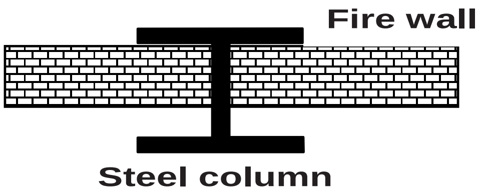
- **Purlins on both sides of the fire wall;**
- **Stop the roof on both sides of the fire wall.**
- **Roof with fireproof material, over a width of 2.50 m on each side of the wall;**
- **Other possibility is to allow the wall exceed the roof up to a specific distance**

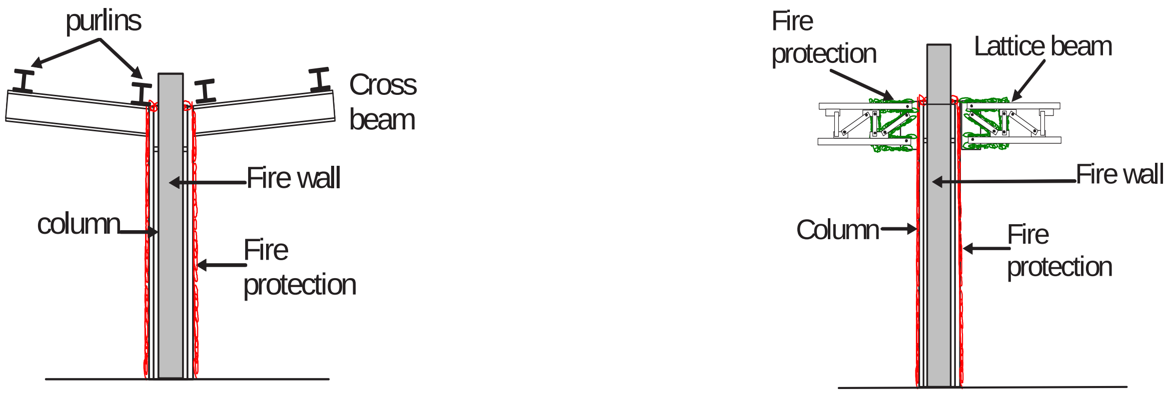
6.2 Fire walls perpendicular to steel frames

General recommendation regarding fire protection of columns, beams and purlins:

- **COLUMS 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.**

6.2.1 Fire wall inserted between the flanges of the column



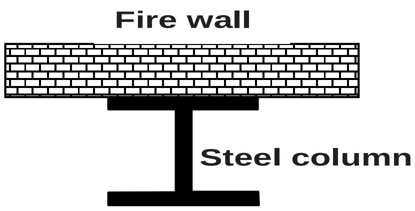


a) b)

Figure 6-3Fire protection required for near the fire wall inserted between the flanges of the column a) portal frame b) lattice frame

Apart from the column the lattice steel structures near fire wall must be protected over a minimal length equal to the distance separating the wall with the first vertical member for lattice frame to avoid the possible disorder induced by the failure of the lattice beam near to the fire wall

6.2.2 Fire wall fixed to one flange of the column



If the fire wall is built beside one flange of columns, to prevent wall damage caused by the collapse of the beam adjacent to the fire wall, a fire protection must be applied to the beam:

- **over a minimal length of 200 mm beyond the wall edge for portal steel frame**
- **over a minimal length equal to the distance separating the wall with the first vertical member for lattice frame**

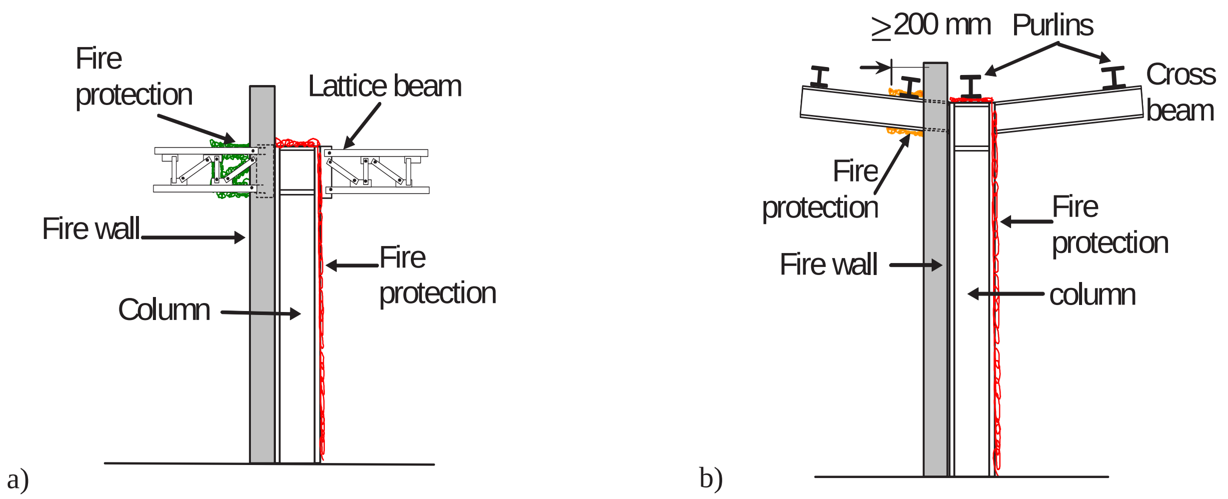
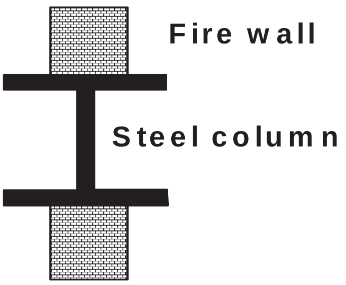


Figure 6-4 Fire protection required for beams and columns near fire wall fixed to one flange of the column a) lattice frame b) portal frame

6.3 Fire Walls Parallel To Steel Frames

- **COLUMS that are into or near a wall must be always fire**
- **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.**

6.3.1 Fire wall in the plan of steel frame



In this situation the beam and the column has to be fire protected.

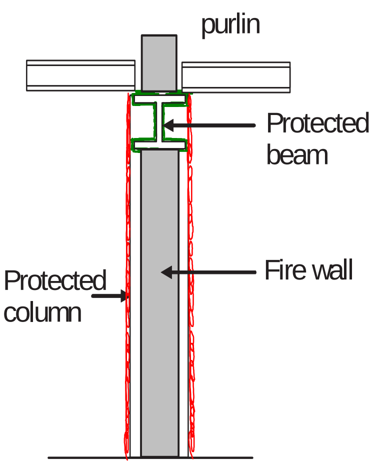
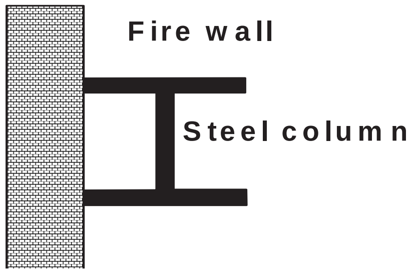


Figure 6-5 Fire protection for the column when fire wall is in the plane of the frame

6.3.2 Fire wall attached to the steel frame



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 closest to the fire wall doesn't initiate the failure of the wall.

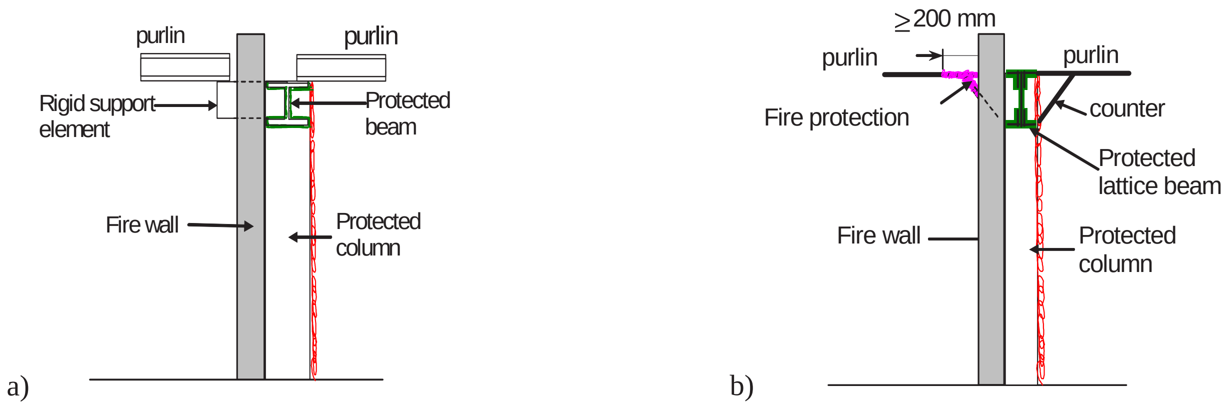


Figure 6-6 Design details for elements near fire wall

In case of the portal frame the following recommendations are suggested:

- **when the fire wall is inserted in the steel framework, rigid steel elements fixed on the beams should be inserted through the wall to support the purlins;**
- **in case of continuous purlins, a fire protection should be applied to purlins on both sides of the wall, over a minimal length of 200 mm beyond the wall.**

In case of lattice frame the recommendations are following:

- **protection of purlins and counters near the wall over a minimal length corresponding to the distance from the wall to the junction counter/ purlin when the roof structure is made of purlins;**
- **application of fire protection to beams, located at the wall side, over a minimal length corresponding to the distance from the wall to the first vertical members of the beam when the roof structure is made of lattice beams.**

6.3.3 Fire wall between two steel frames

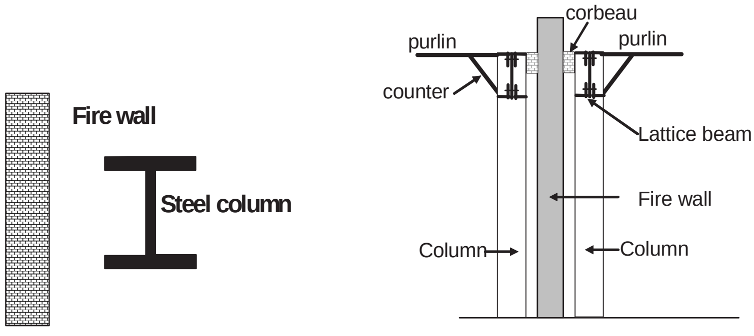


Figure 6-7 Fire wall between two portal frames

Lattice beams cannot allow inserting a continuous wall up to the roof, so a solution consists in subdividing industrial building in two independent structures and inserts the fire wall between them.

6.4 Recommendations for bracing system

6.4.1 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. So, the following solutions should be adopted:

- **use 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×l);**
- **to double the bracing on both sides of fire walls or to protect against fire the preceding bracing systems.**

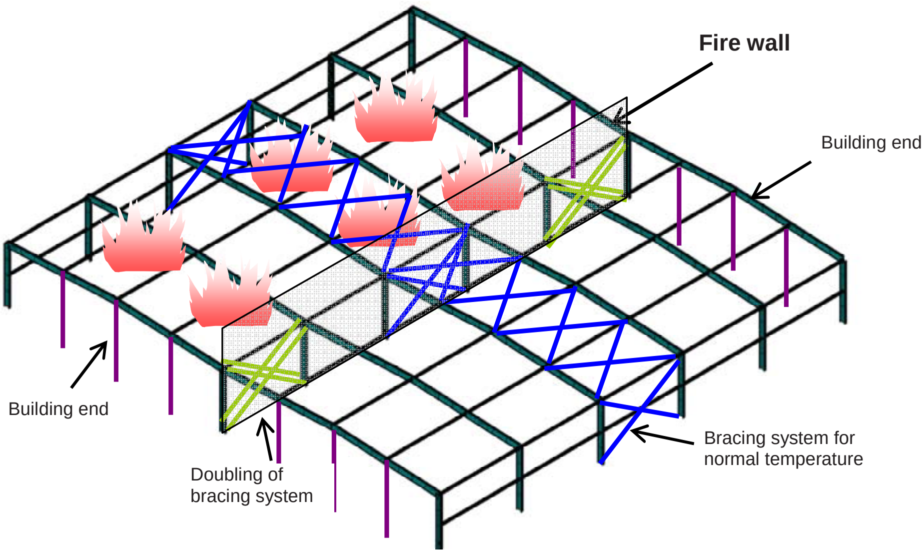


Figure 6-8 Bracing systems at the longitudinal end of the storage building

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.

6.4.2 Fire walls parallel to the steel frames

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. Each bracing system must be designed to support a horizontal uniform load taken as:

$$F = 1.19\, q \qquad \text{Equation 6-2}$$

where

$$q = G + 0.2\, S \quad \Rightarrow \quad Equation\ 4\text{–}7$$

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.

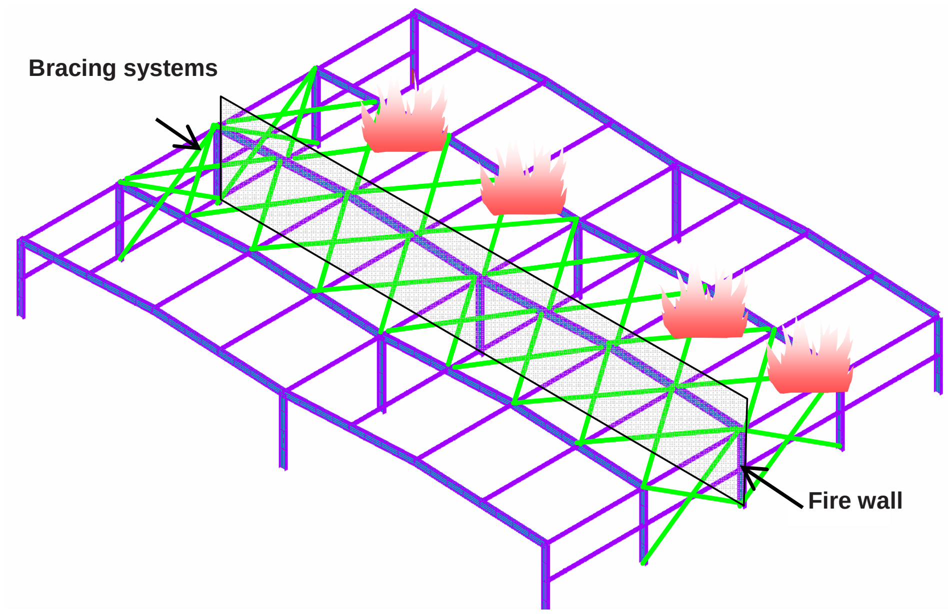


Figure 6-9 Bracing systems of storage buildings

6.5 Additional design recommendation for simple portal steel frames

Parametric studies [6, 11, 12] performed with the advanced numerical model SAFIR [5, 10] showed that the collapse could occur towards the outside in the case of storage buildings with simple portal steel frame in some conditions .

In such cases, the failure mode towards the outside can be avoided by providing to the connections between columns and foundation, as well as to the resistance capacity of the foundation, an ultimate resistance at ambient temperature. The resistance should be such that the vertical loads corresponding to the fire situation can be carried with an additional bending moment equal to 20 % of the ultimate plastic moment of the column at ambient temperature.

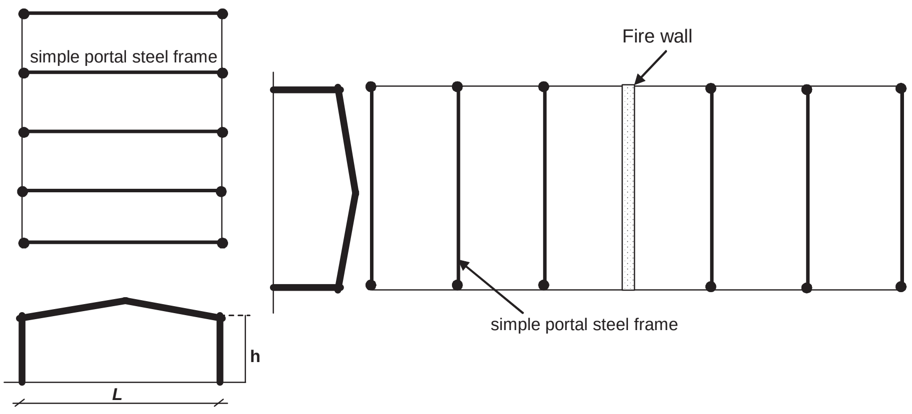


Figure 6-10 Industrial buildings with simple portal steel frame

Project carried out with a financial grant of the
Research Programme of the Research Fund for Coal and Steel
RFS2-CR-2007-00032

Fire Safety of Industrial Hall

Background Document

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 m2 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.

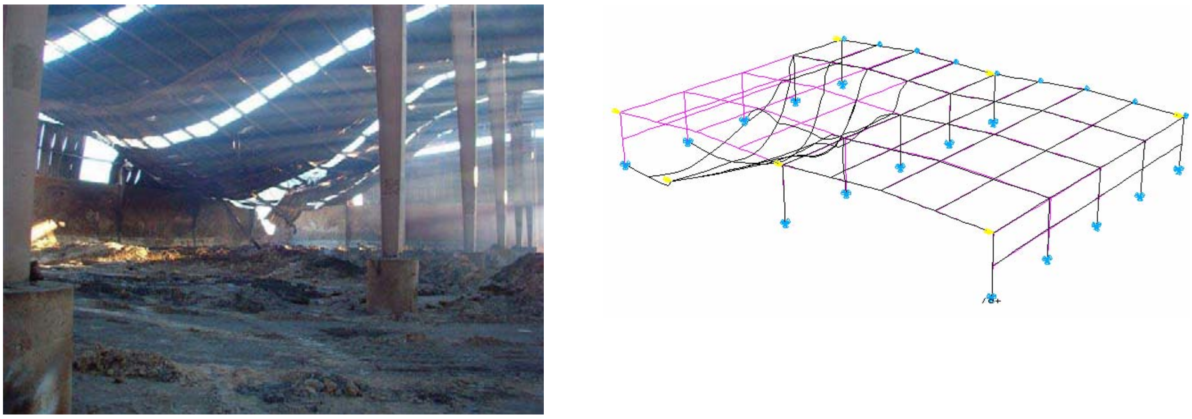


Figure 2-3 Partial collapse and simulation

2.3 Logs Santos Warehouse (Spain)

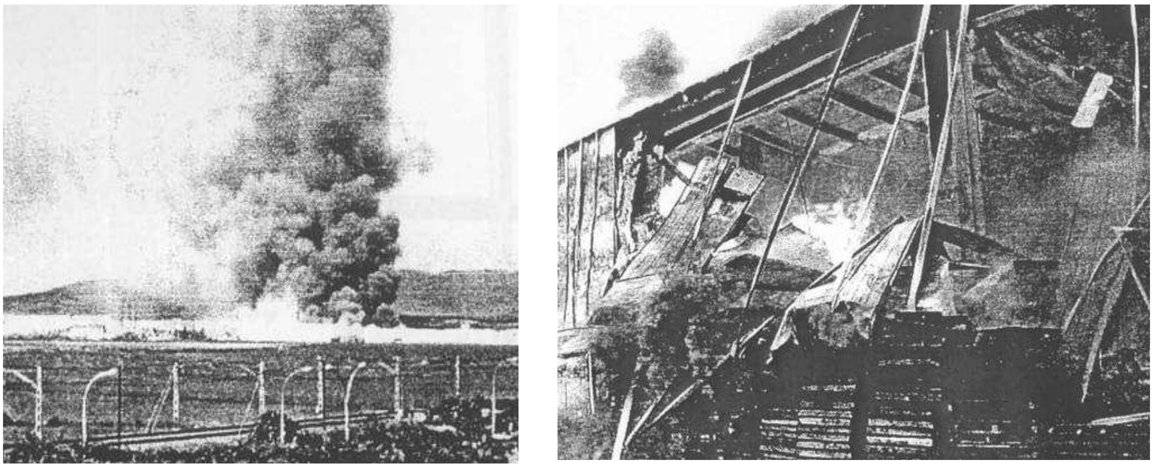


Figure 2-4 Photography of the fire

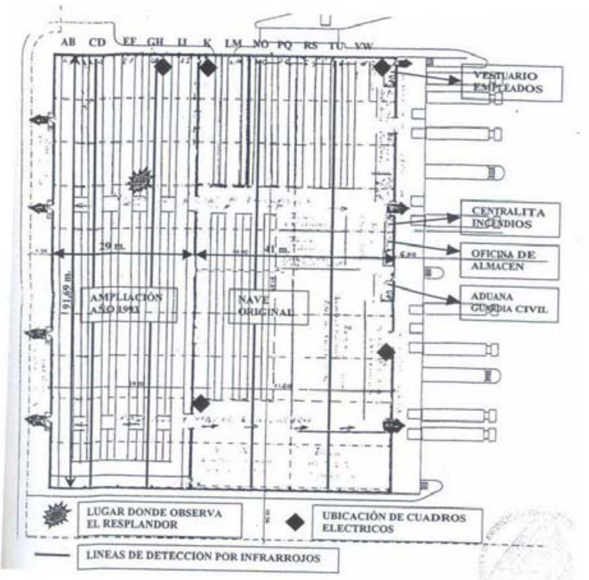


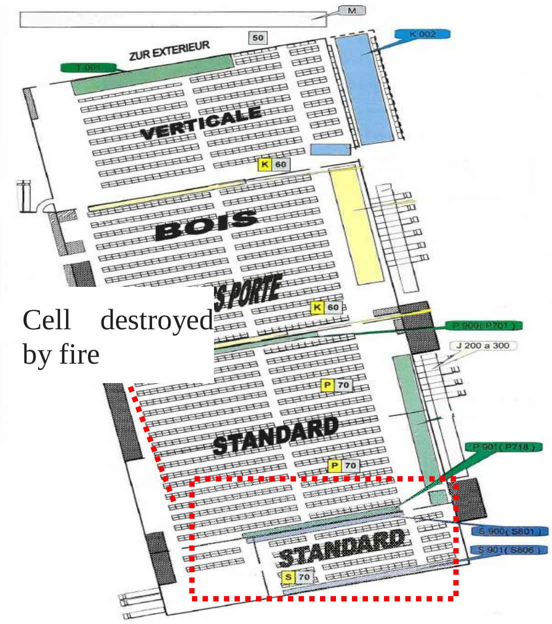
Figure 2-5 Fire scheme made by the Fire Brigade

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.

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.

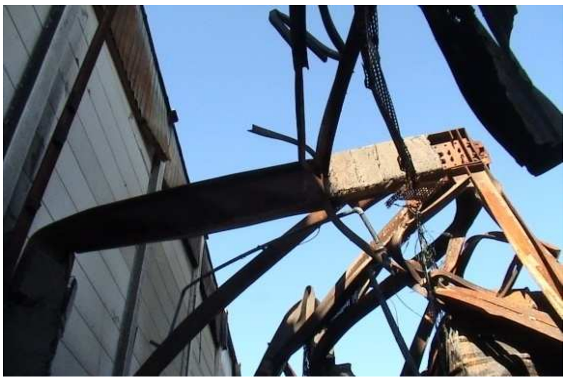
In addition, all façades of the cell in fire has collapsed together with steel frame toward inside of buildings constituting a safe failure mode for fire brigade fighting against the fire.

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%.

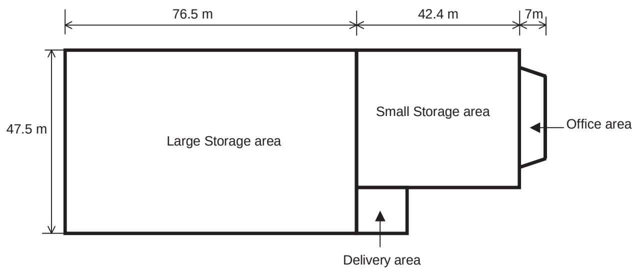


Figure 2-8 Layout of the storage building and development of fire (right)

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 ≤ 350 MJ/m², Class B, and Class C > 900 MJ/m².

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_{fl} 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_{fl} 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/m². Deemed to satisfy distance are given in the following table

Fire resistance of the façade	% of openings	Distance [m]
EI (i↔0) 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_{fl} not higher than 2000 m²)

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

- Class A with $A_{fl} \leq 10\,000$ m² or Class B with $A_{fl} \leq 500$ m².

- 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 m³	5 000 m³ ≤ V < 50 000 m³	V ≥ 50 000 m³
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 m²	3000 m² < S < 6000 m²	S > 6000 m²
H < 12.5 m	R0	R0 + sprinklers	R0 + Sprinklers + FSE
H > 12.5 m	R60 or Sprinklers + FSE	R60 + Sprinklers or Sprinklers + FSE	R60+Sprinklers+FSE or Sprinklers + FSE

Separating walls
-REI 120 minimum -All elements ensure a equivalent REI level -The door between cells must be REI 120 with automatic shut-off. -Separating walls must be at least 1 m from roof. -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.

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:

$V < 100\ m^3$	$100\ m^3 \leq V < 1000\ m^3$	$V \geq 1000\ m^3$
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 MJ/ m^2
- Medium risk activities: fire load < 3400 MJ/m^2
- High risk activities: fire load bigger than 3400 MJ/m^2

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/m²) 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^2) 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:

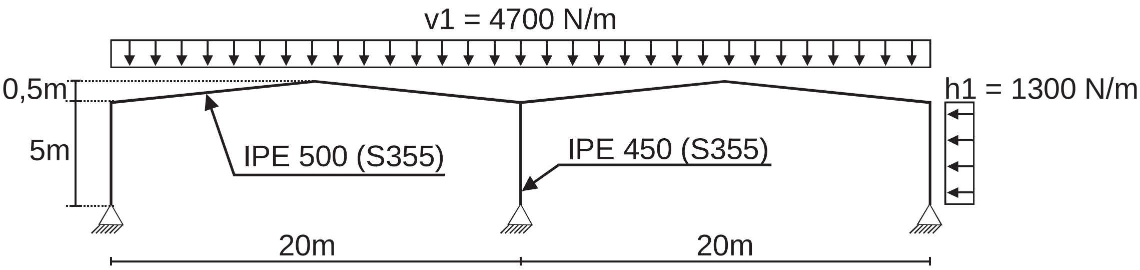


Figure 4-1 Benchmark 2D portal frame

- **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:**

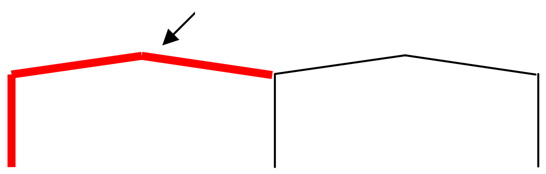
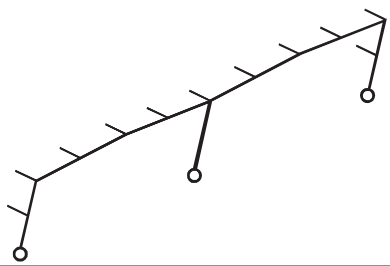
$$\alpha = 25 \quad W/m^2 K$$
$$\varepsilon = 0.5$$

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:

a double frame in 2 dimension	a double frame in 3 dimensions partially maintained in the third dimension
	

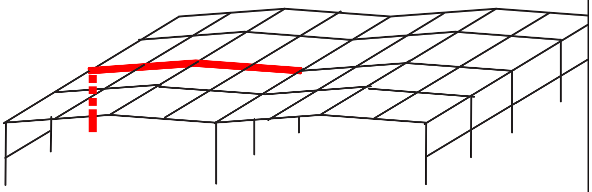
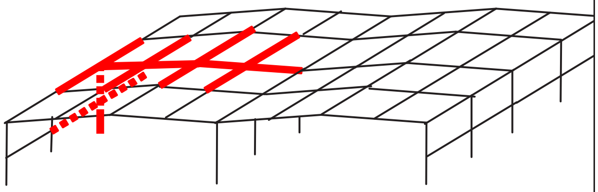
a full study in 3 dimensions with more than one double frame	a full study in 3 dimensions with more than one double frame and hot purlins
	

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.

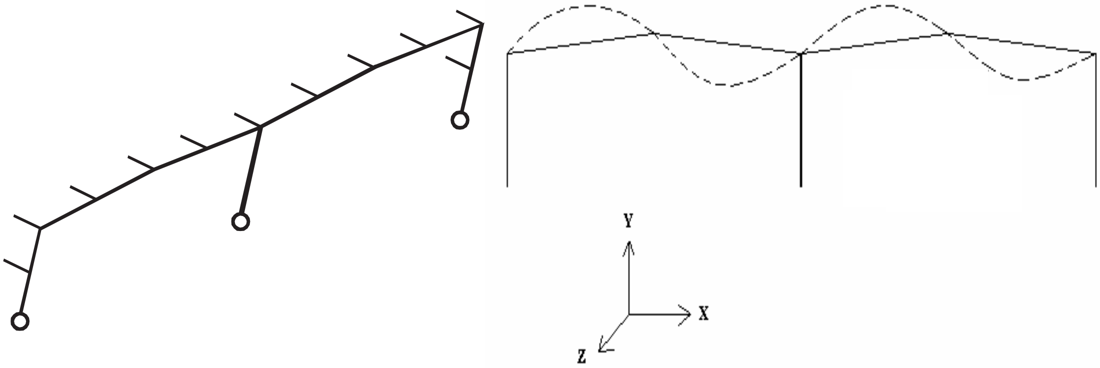
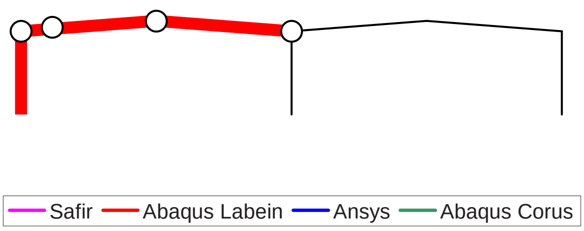
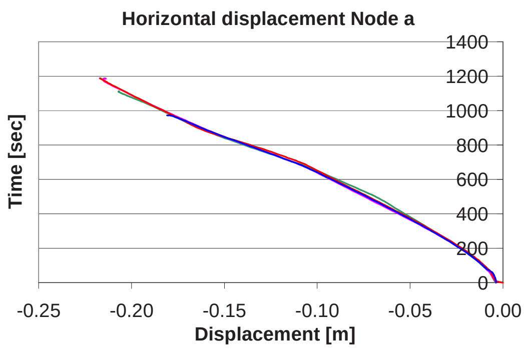


Figure 4-3 Illustration of the fixed points in the third dimension and a scheme of the initial deformations

➢ **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):





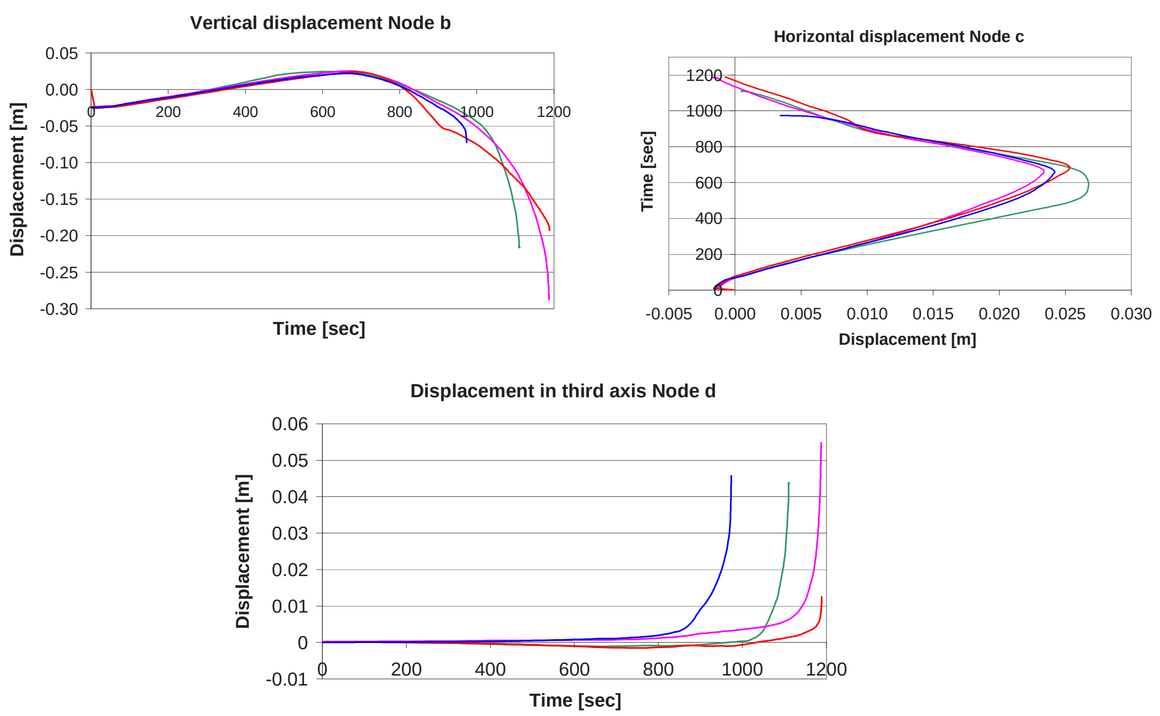


Figure 4-4 Displacement analysis of the 2D portal frame in space

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:**

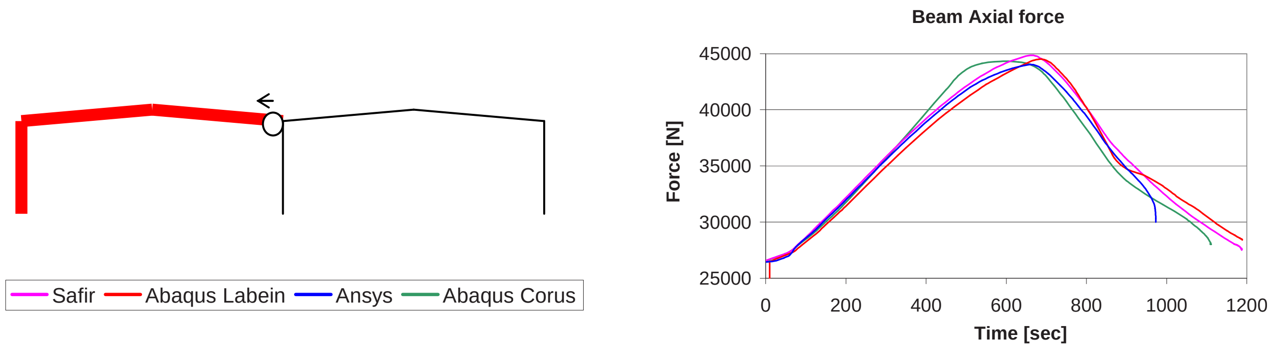


Figure 4-5 Axial force evolution

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:**

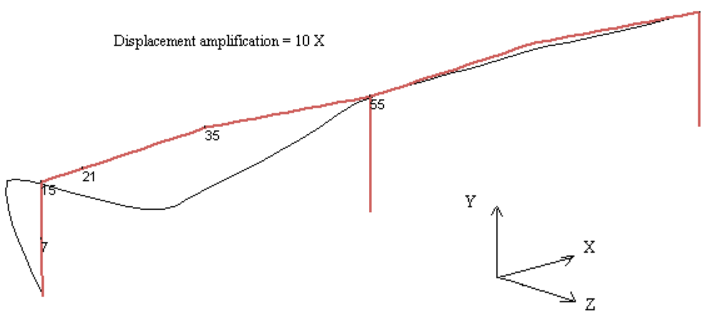


Figure 4-6 Spatial deformation of the frame

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.

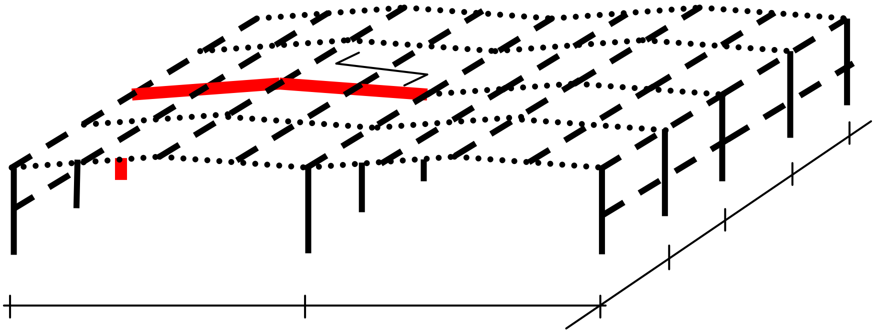


Figure 4-7 3D structure with multiple frames with marked single heated frame

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;**

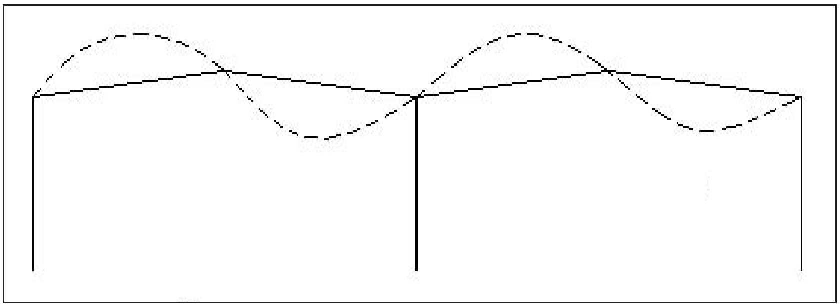


Figure 4-8 Out of plane imperfection

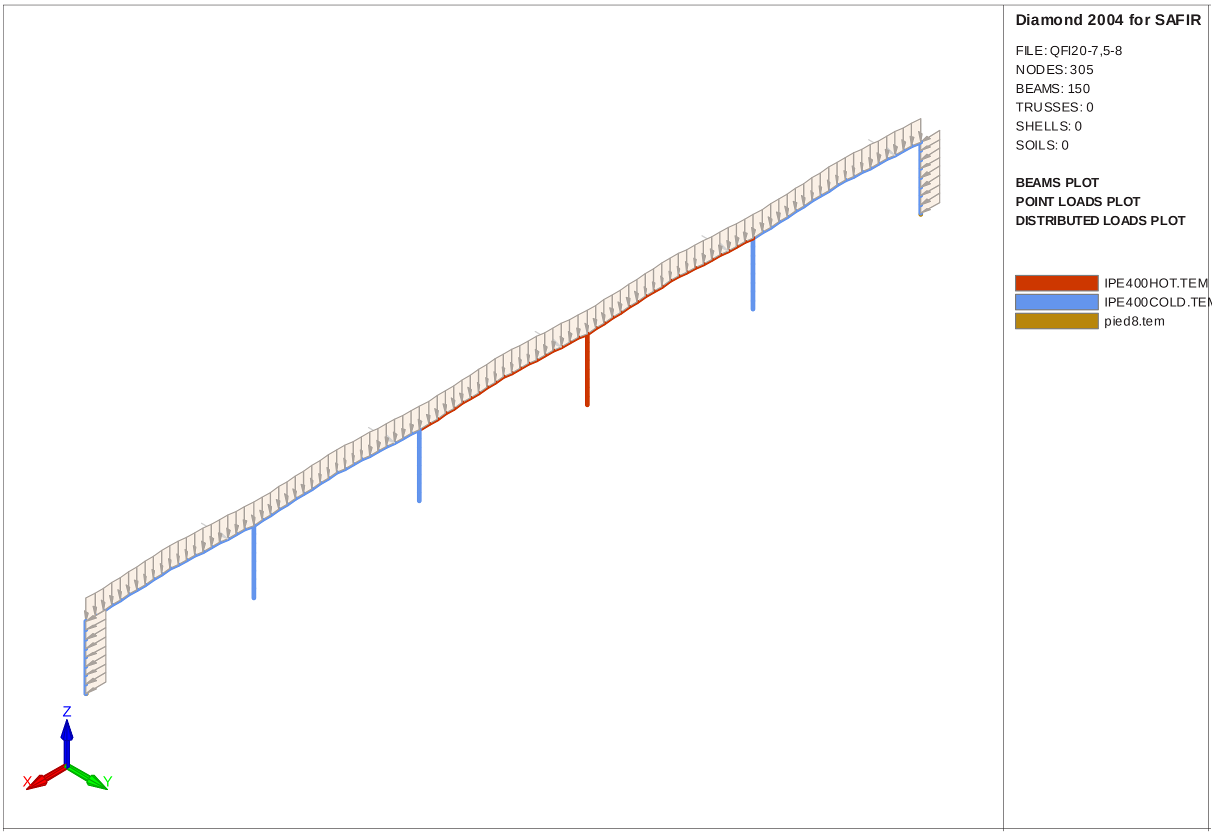


Figure 4-9 Loading conditions of steel frames

- ➢ **global out of plane imperfection was applied to the model (see Figure 4-8);**
- ➢ **no residual stresses taken into account;**
- ➢ **the mechanical materials properties according to EC3 Part 1.2;**

➢ **restrained lateral displacement of several points at position of purlins (see Figure 4-10).**

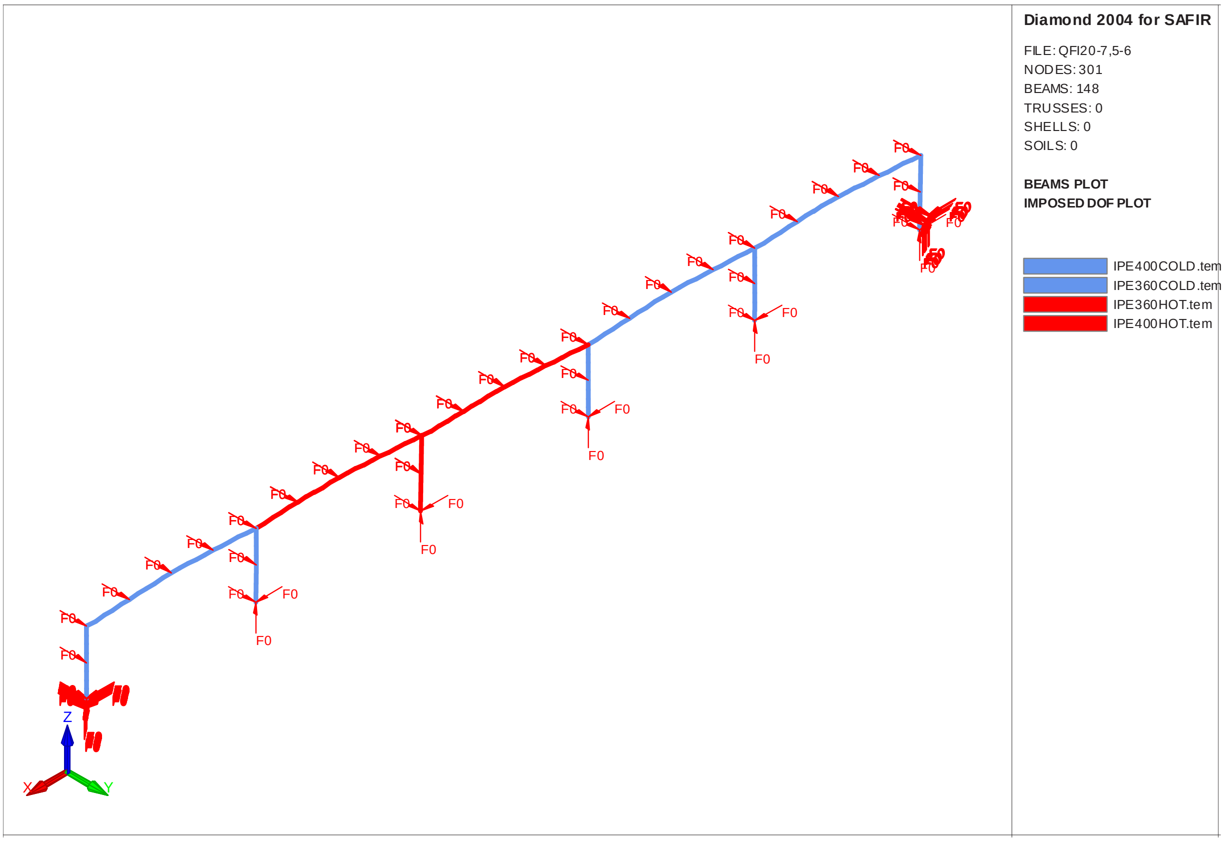


Figure 4-10 Boundary conditions of steel frames

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;**

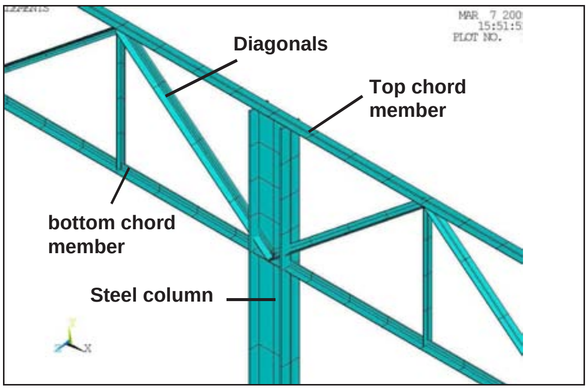


Figure 4-11 Modelling of steel frames with beam elements

➢ **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;**

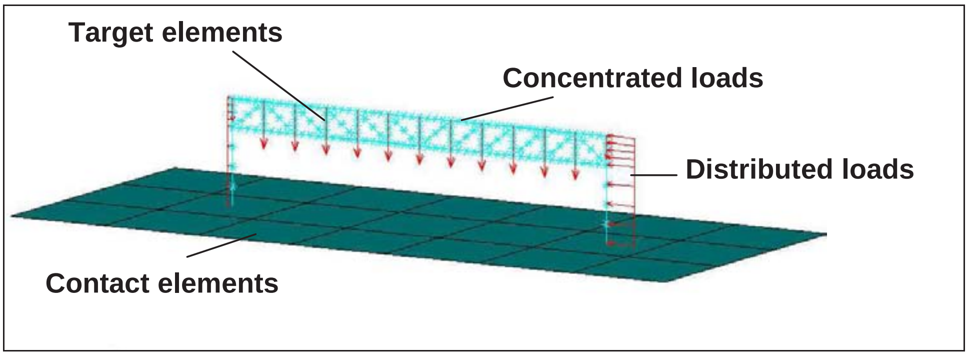


Figure 4-12 Loading conditions of steel frames as well as modelling of ground level

- **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).**

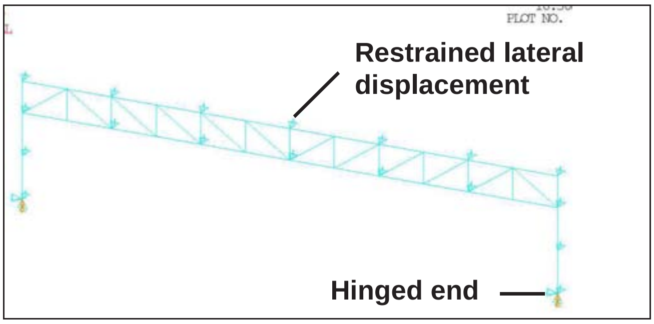


Figure 4-13 Boundary conditions of steel frames

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 N/m² ;
  - ⇒ Weight of wall cladding is taken as 150 N/m² ;
- **the snow load 'S' is taken as 550 N/m²;**
  - ⇒ 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 N/m²;**
  - ⇒ 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.**

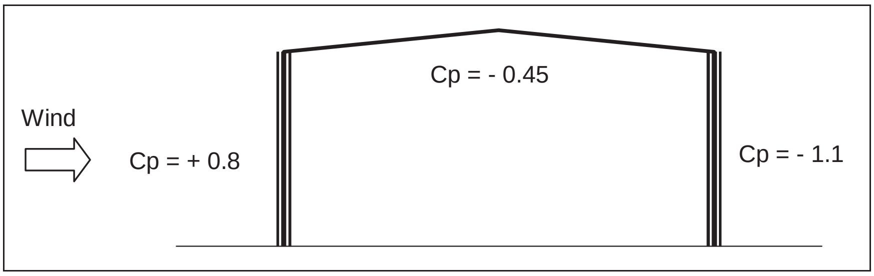


Figure 4-14 Pressure coefficient $C_p=(c_{pe}-c_{pi})$ in fire situation

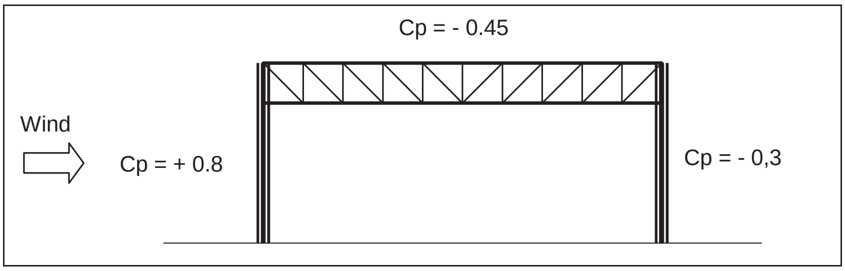


Figure 4-15 Pressure coefficient $C_p=(c_{pe}-c_{pi})$ in fire situation

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

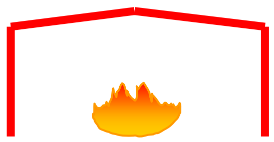
⇒ $1.0\times G + 0.2\times W$ and

⇒ $1.0\times G + 0.2\times S$.

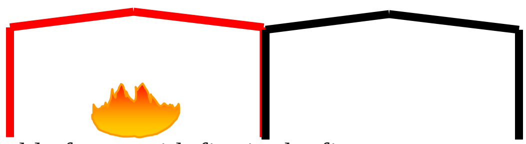
4.2.5 Heating conditions:

- **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.**

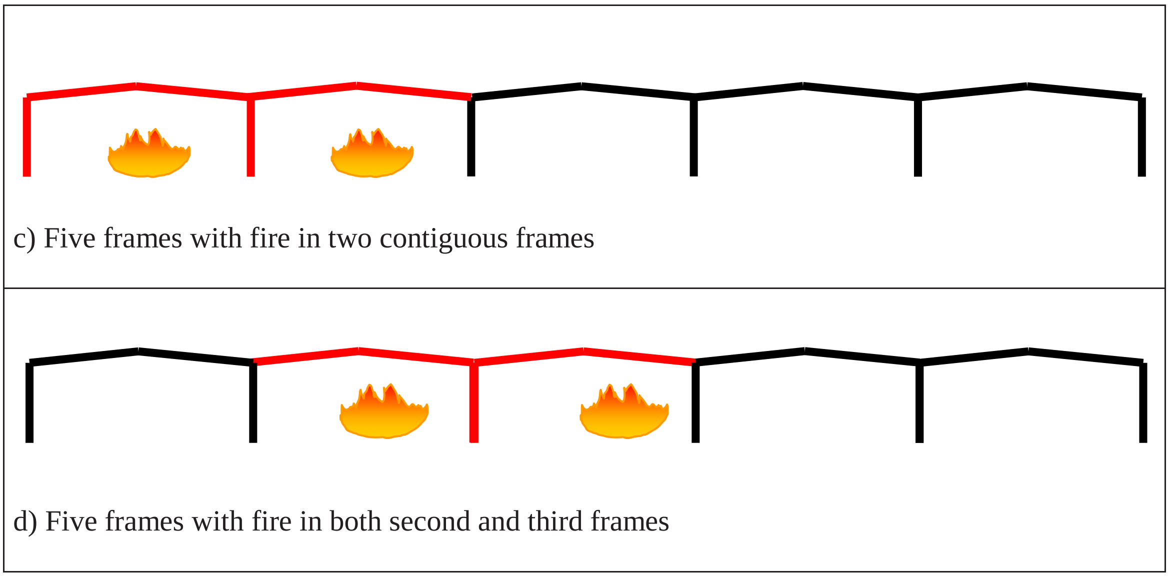
For framed structures and lattice structures, different configurations have been investigated according to the frame number, the position of the fire walls and the fire location in the disaster cell (see Figure 4-16 and Figure 4-17).



a) Heated simple Frame



b) Double frame with fire in the first span



c) Five frames with fire in two contiguous frames

d) Five frames with fire in both second and third frames

Figure 4-16 Fire scenarios in portal frame structure

a) Heated simple Frame

b) Double frame with fire in the first span

c) Triple frame with fire in the middle span

d) Triple frame with fire in 2 contiguous frames

e) Five frames with fire in three contiguous frames

f) Five frames with fire in the three middle frames

g) Five frames with fire in both second and third frames

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 W/m^{2}K;**
- **emissivity: ε = 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);**

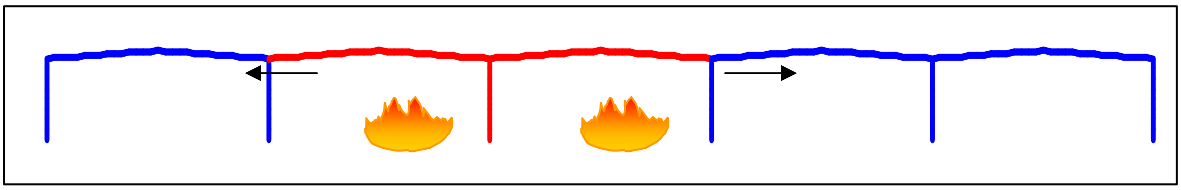


Figure 4-18 Lateral displacements at the top of columns

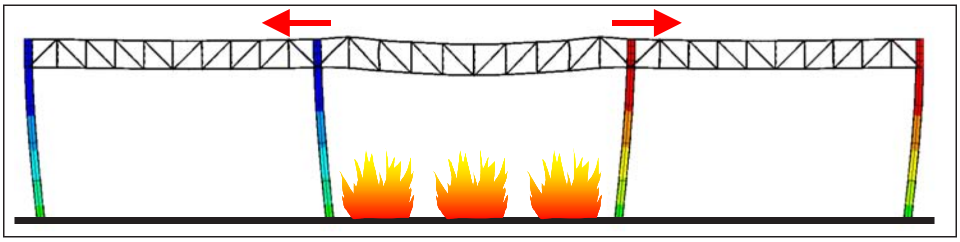
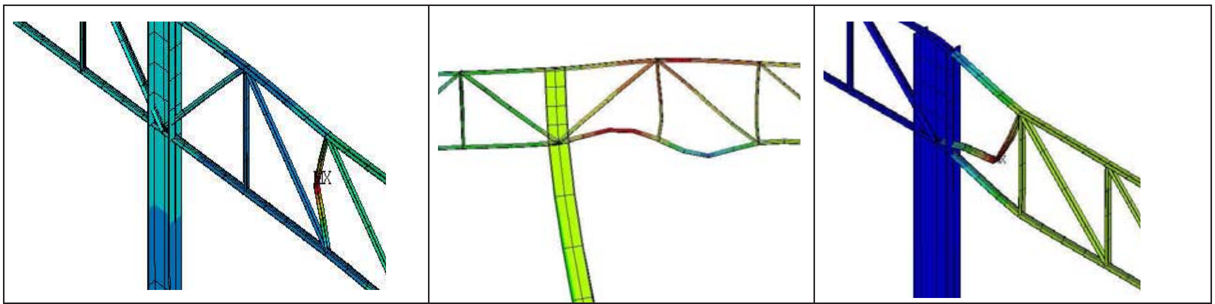


Figure 4-19 Lateral displacements at the top of columns

- **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).**



a) Buckling of vertical member	b) Buckling of bottom chord	c) Buckling of the diagonal near the column

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);**

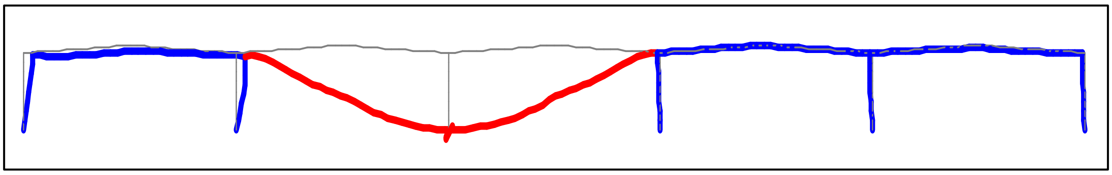


Figure 4-21 Lateral displacements at the top of columns

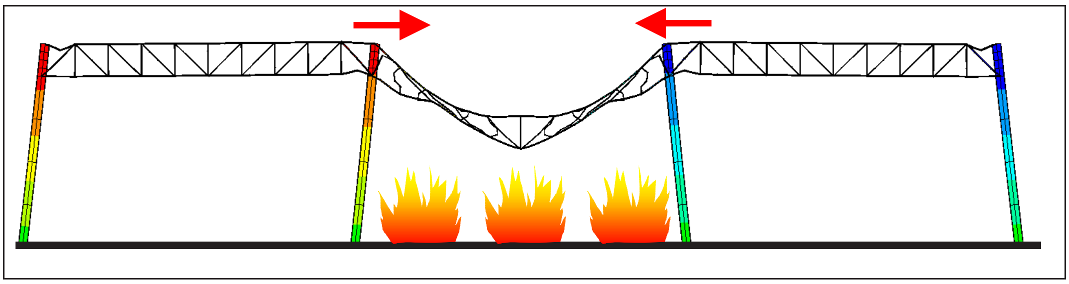


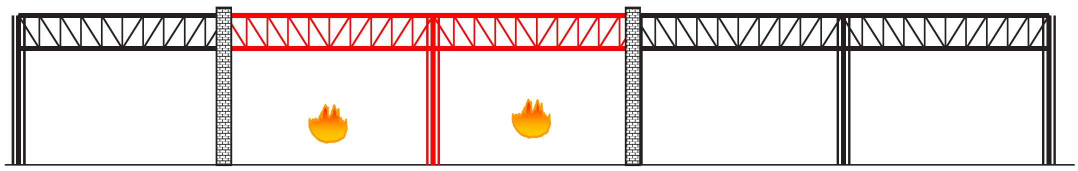
Figure 4-22 Lateral displacements at the top of columns

- **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.**

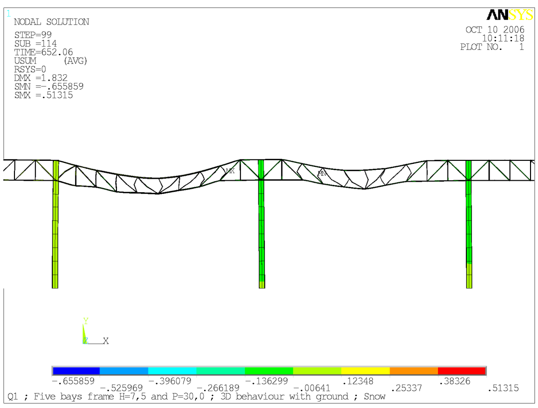
4.3.2 Parametric study observations

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.

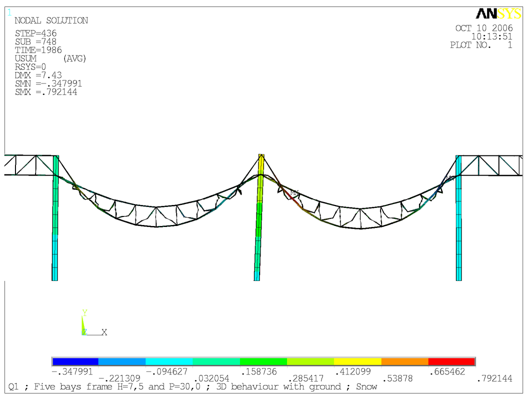
The analysis of obtained results shows clearly that the collapse of the multi-bay lattice frames is always caused by the failure of the heated beam as a result of important additional internal forces due to the axial restraint against thermal expansion induced by the cold parts of the structure (see 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).



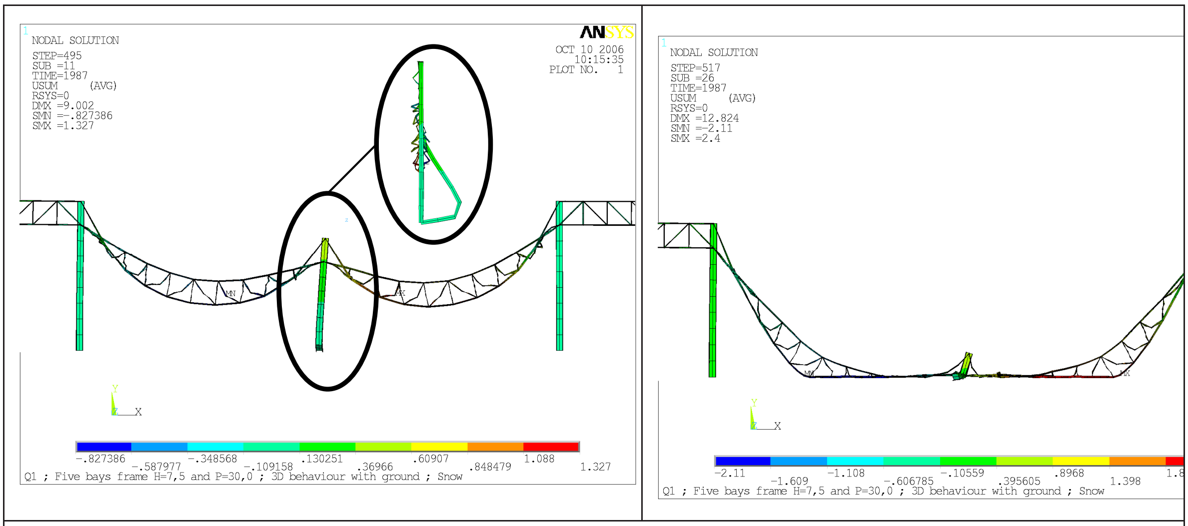
a) Five frames with fire located in both second and third frames



b) Deformed shape at time t=652 sec



c) Deformed shape at time t=1986 sec



d) Deformed shape at time t=1987 sec

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.

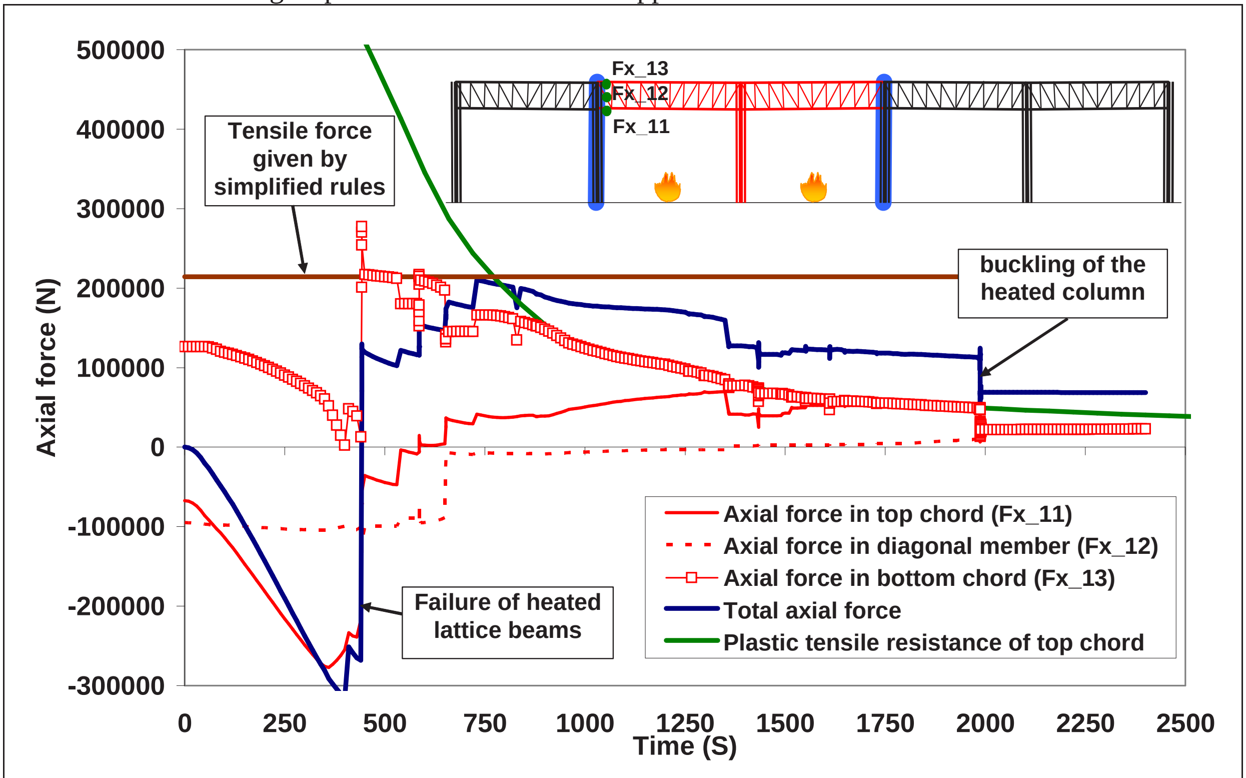


Figure 4-24 Axial forces induced in heated lattice beams

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.

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.

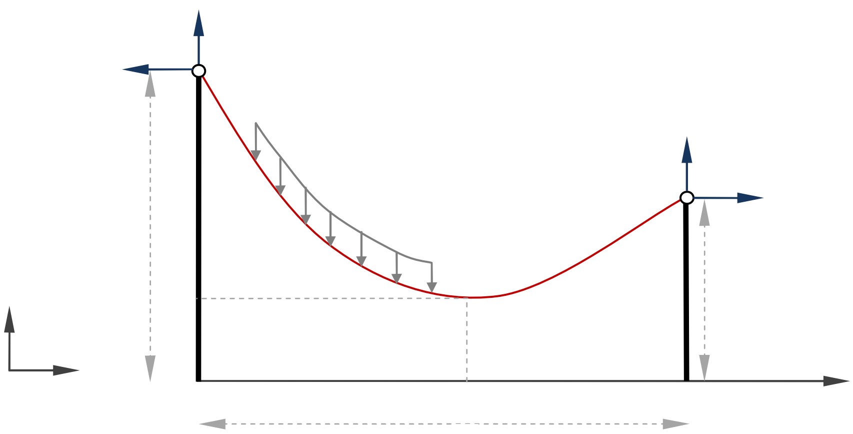


Figure 5-1 Parameters of the catenary.

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

$$R_H = q.a \qquad \text{Equation 5-1}$$

Under constraints:

$$\begin{cases} x_0 < L,\ with\, x_0\ is\ such\ that\ \ h_2 - h_1 = a\left(cosh\left(\dfrac{x_0}{a}\right) - cosh\left(\dfrac{L - x_0}{a}\right)\right), \\ y_0 > 0, with\ \ z_0 = h_1 + a\left[1 - cosh\left(\dfrac{x_0}{a}\right)\right], \\ h_2 > h_{2,min}\ with\ h_{2,min} = h_1 - \sqrt{L_0^2 - L^2}. \end{cases}$$ Equation 5-2

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

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

Catenary parameters are as follows:

h_1, h_2 - heights of support columns

L - distance between columns

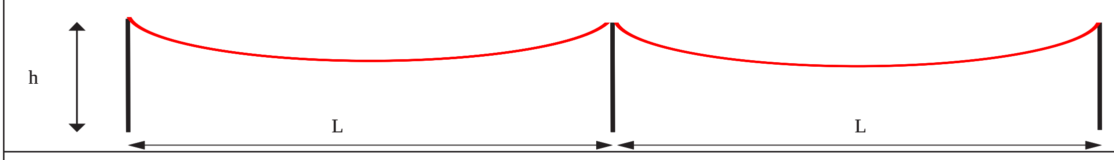
x_0, y_0 - coordinates of the lowest point of the chain

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

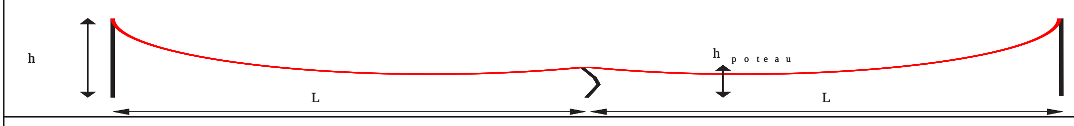
L_0 - length of the chain, given by the implicit equation,

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

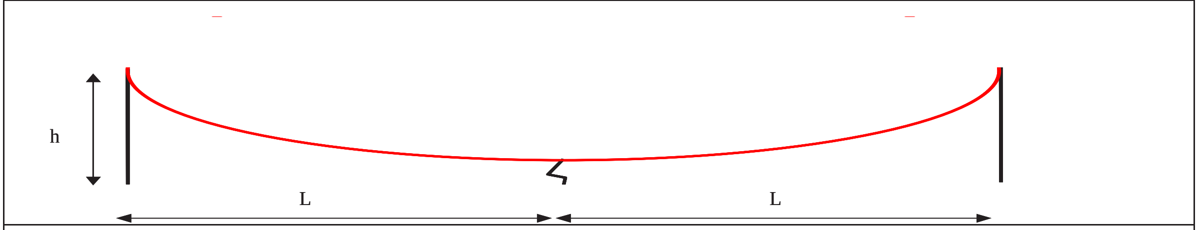
During fire, different situations can be met. Indeed, columns are considered as fixed at support and, under fire conditions, the unprotected intermediate column in the same cell determines the parameters of the catenary and then the generated forces in the top of columns. The following figures illustrate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



Case 1: The intermediate column does not fail



Case 2: The intermediate column partially collapses and still contributes to the structural strength



Case 3 : The intermediate column collapses and no longer considered as a support

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 5.1.4 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);

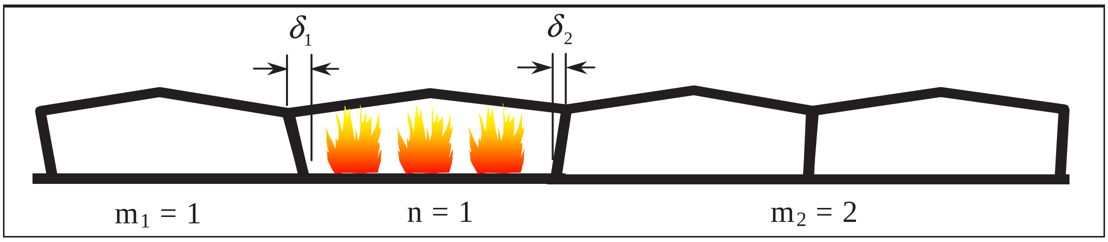


Figure 5-3 Fire located in a cell at the middle of the building

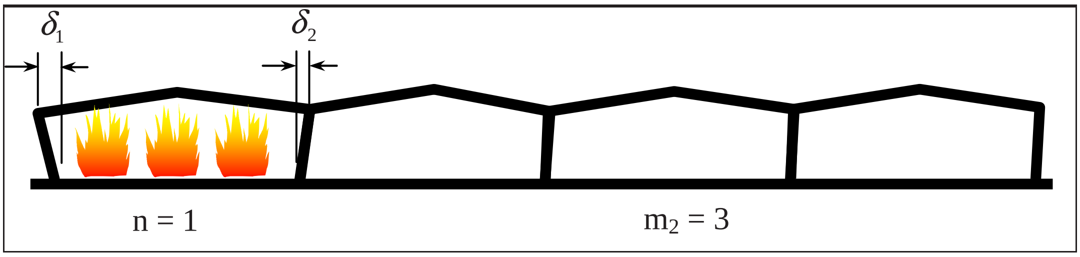


Figure 5-4 Fire in a compartment at the end of the storage building

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,

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

where

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

ℓ 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.

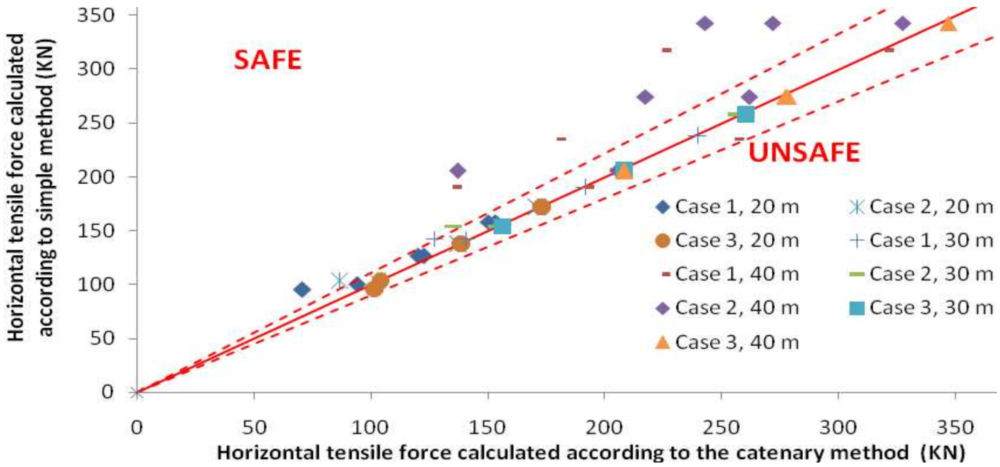


Figure 5-5 Correlation between horizontal tensile forces calculated using the catenary method (Equations 5-1, 5-2 and 5-3) and those calculated using the proposed simple method.

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.

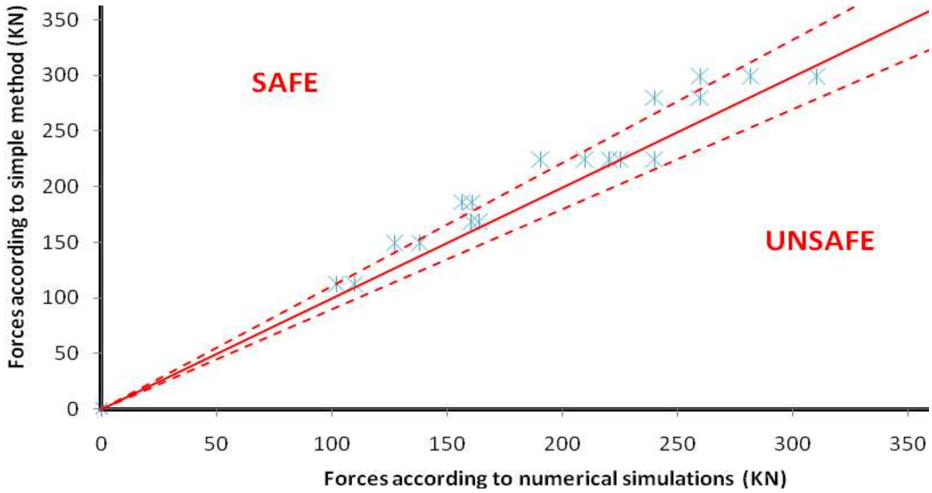


Figure 5-6 Correlation between tensile forces calculated using numerical simulations and those calculated using the proposed simple method.

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,

$$F_p = c_p n q \ell \qquad \text{Equation 5-6}$$

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);

ℓ - 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:

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

where:

$K_t = \dfrac{K_1 K_2}{K_1 + K_2}$, with K_1 and K_2 equivalent stiffness for lateral displacements δ_1 and δ_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;

ℓ 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:

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

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 $m_i = 1$:

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

with:

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

where (see Figure 5-7):

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

ℓ - is the span length [m];

I_b - is the second moment of area of the beam [m^4];

I_c - is the second moment of area of columns [m^4];

E - is the modulus of elasticity of steel for normal temperature [N/m²];

For $m_i \geq 2$:

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

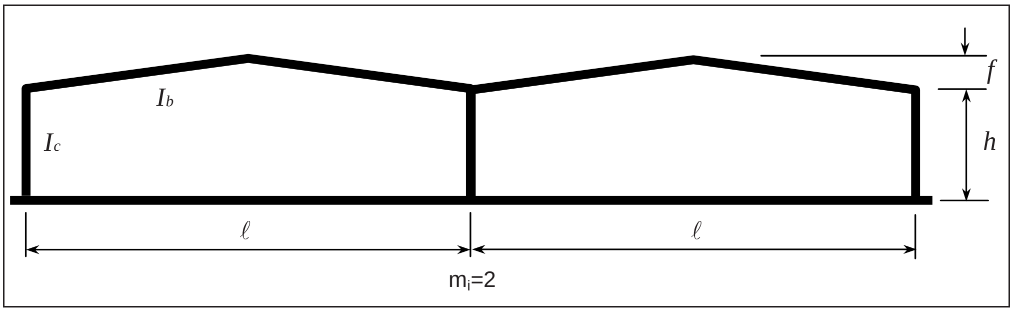


Figure 5-7 Definition of parameters of cold parts

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.

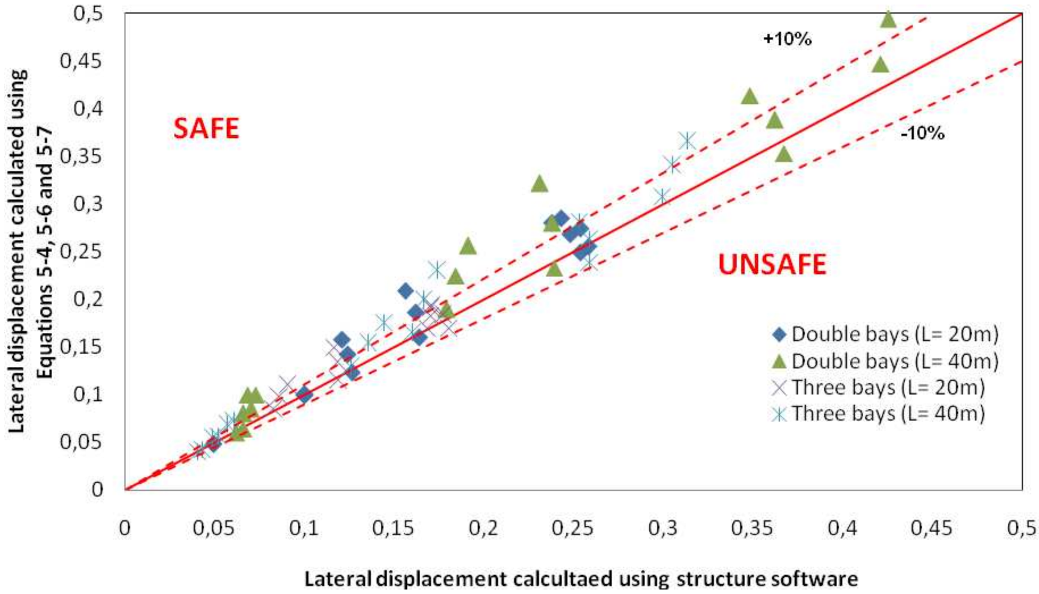


Figure 5-8 Correlation between lateral displacements calculated using structure software and those calculated using the simple method (Eqs. 5-9, 5-10 and 5-11)

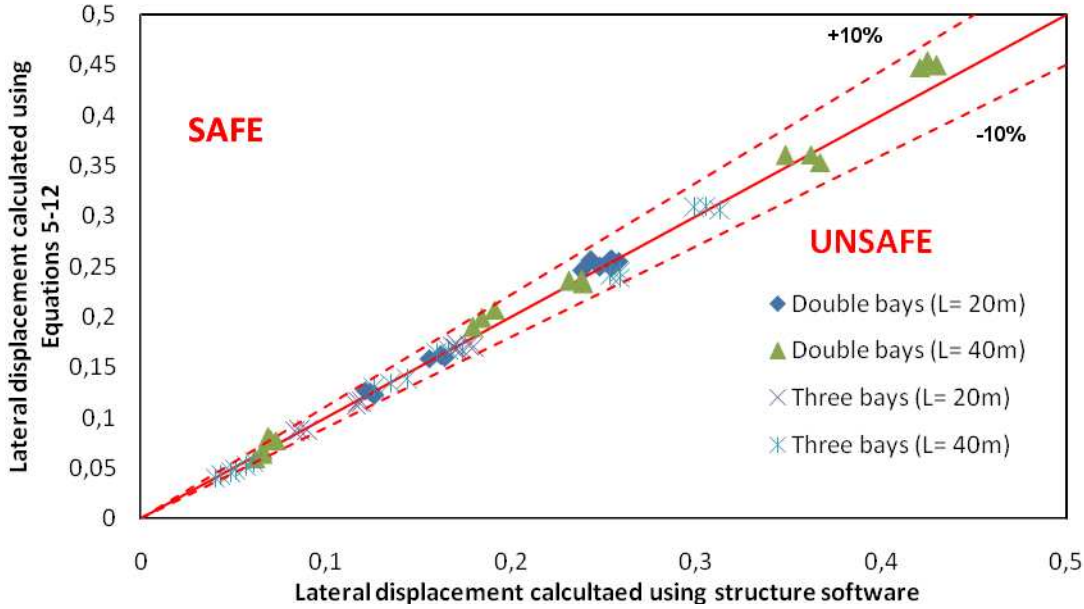


Figure 5-9 Correlation between lateral displacements calculated using structure software and those calculated using the simple method (Eq. 5-12)

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

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

5.1.3 Displacement at fire compartment ends

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

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

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.

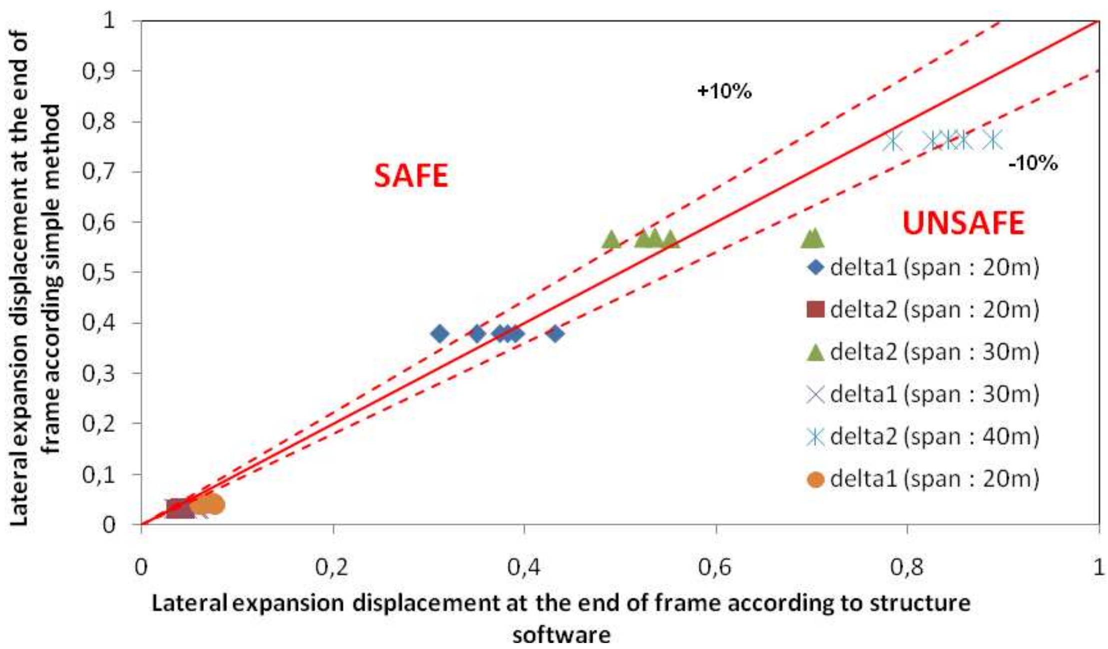


Figure 5-10 Correlation between lateral displacements calculated using structure software and those calculated using the simple method (Equation 5-13)

5.1.4 Steel frames with lattice beams and columns in standard H or I hot rolled profiles

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 δ_i induced at the top of columns located at the compartment ends can be obtained using the following expression:

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

where:

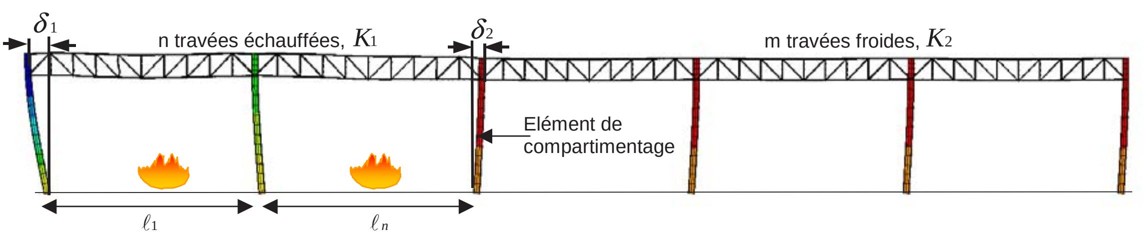
ℓ_i is the length of the heated span i [m];

n is the span number of the fire compartment;

$K_t = \dfrac{K_1K_2}{K_1+K_2}$ [N/m], where K_1 and K_2 are the equivalent stiffness of steel structures for the lateral displacements δ_1 and δ_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.



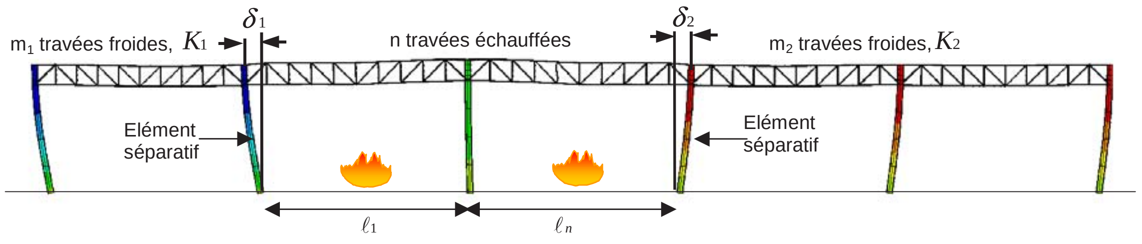
K_2 is the lateral stiffness of the steel framework of the cold part of the structure.

K_1 is the lateral stiffness of the steel framework of the fire compartment which can be approximated by:

If $n=1$, $K_1 = 0{,}2K_2$ and $\delta_1 = 0{,}0075.\ell$, $\delta_2 = 0{,}0015.\ell$

If $n \geq 2$, $K_1 = 0{,}3K_2$ and $\delta_1 = 0{,}007 \cdot \sum_{i=1}^{n}\ell_i$, $\delta_2 = 0{,}002 \cdot \sum_{i=1}^{n}\ell_i$.

a) Fire compartment at one end of the storage building



K_1 and K_2 are the equivalent lateral stiffness of the steel frameworks of cold parts of the structure

b) Fire compartment in the middle of the storage building

Figure 5-11 Definition of lateral stiffness K_1 and K_2

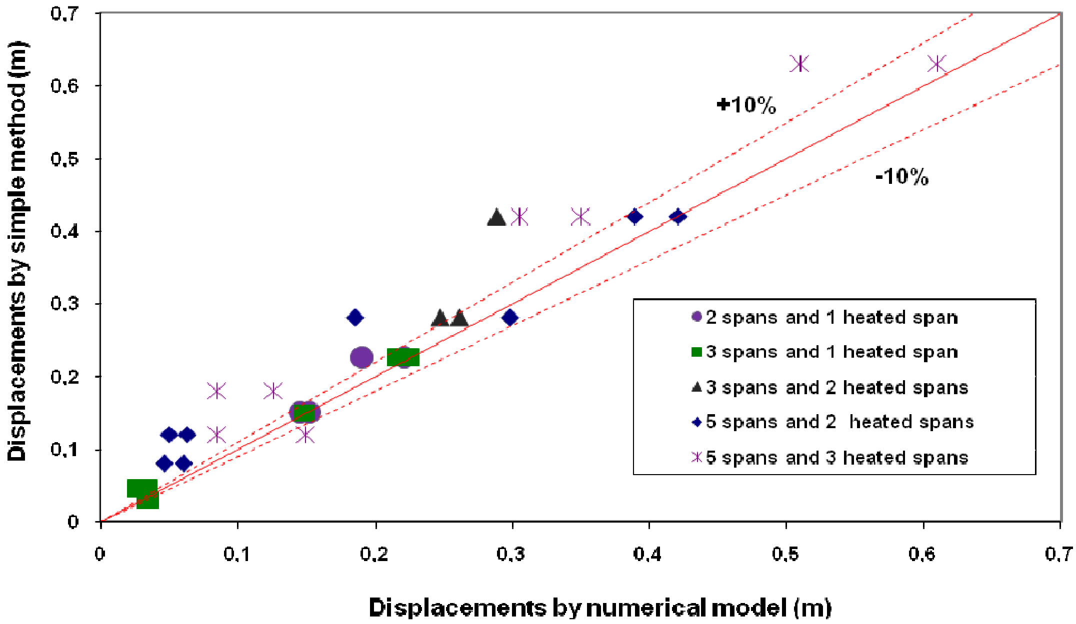


Figure 5-12 Correlation between expansion displacements calculated using numerical modeling and those calculated using the simple method (Equation 5-14 and case a of Figure 5-11)

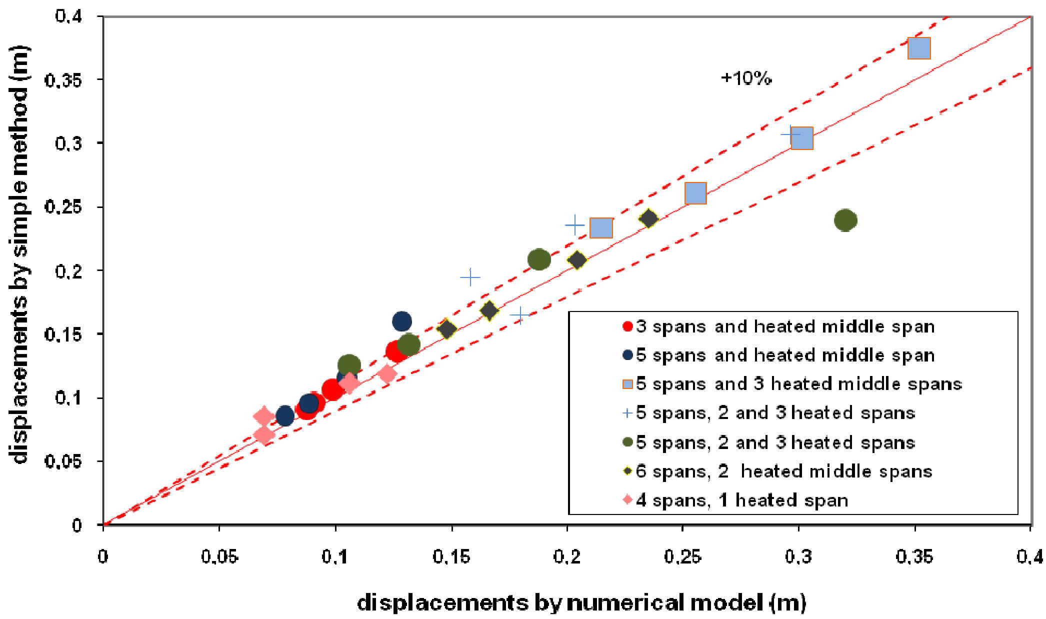


Figure 5-13 Correlation between expansion displacements calculated using numerical modeling and those calculated using the simple method (Equation 5-14 and case b of Figure 5-11)

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).

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, the horizontal tensile force applied at the ends of the fire compartment can be obtained from:

$$F = c_p.q.\ell \qquad \text{Equation 5-15}$$

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);

ℓ 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 cp 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).**

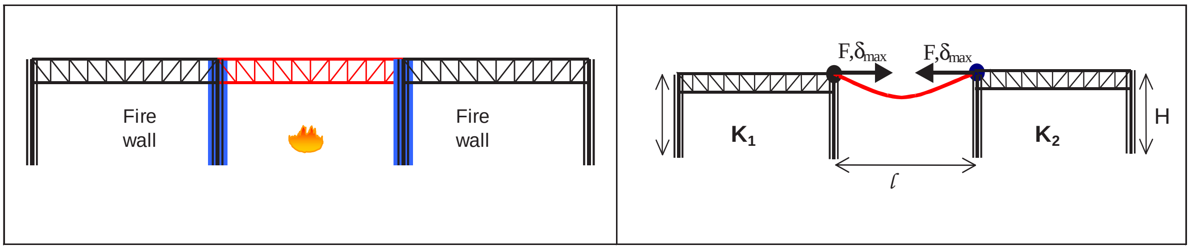


Figure 5-14 Fire compartment in the middle of the storage building

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:

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

where

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

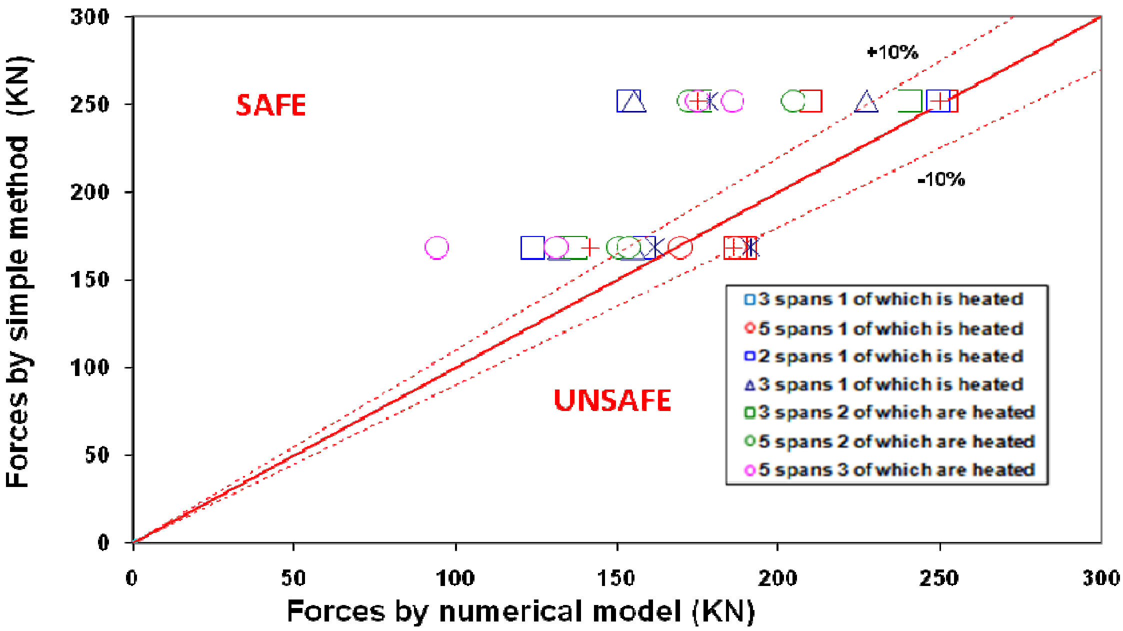
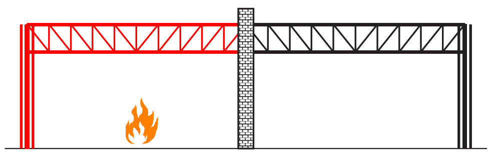
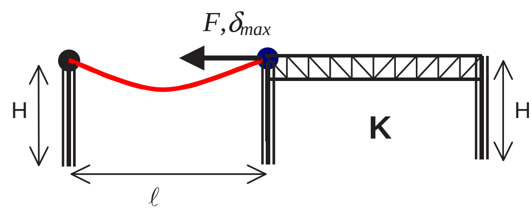
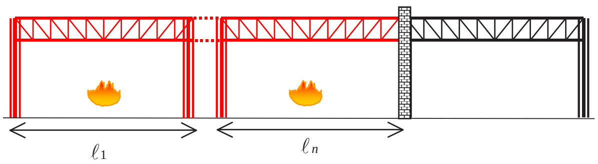
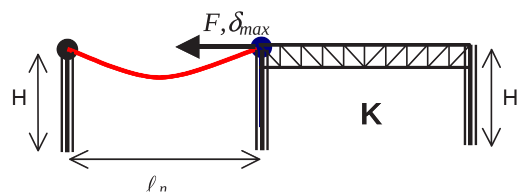
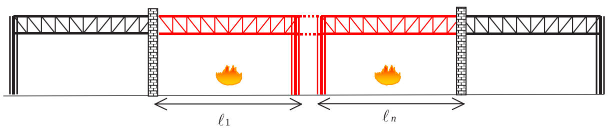


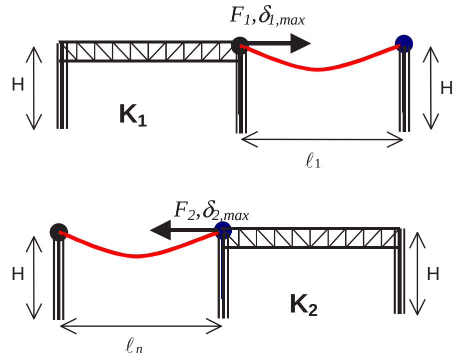
Figure 5-15 Correlation between forces calculated by numerical methods and those calculated by the simple method (Eq. 5-14)

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.

 Simple heated span	 $F = 1{,}45.(0.6q\ell)$ and $\delta_{max} = F / K$
 Case of n heated spans	 $F = 1{,}45q\ell_n$ and $\delta_{max} = F / K$
a) Fire compartment at the end of the storage building	



Case of n heated spans



$F_1 = 1{,}45q\ell_1$ and $\delta_{1,max} = F_1 / K_1$

$F_2 = 1{,}45q\ell_n$ and $\delta_{2,max} = F_2 / K_2$

b) Fire compartment at the middle of the storage building;

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

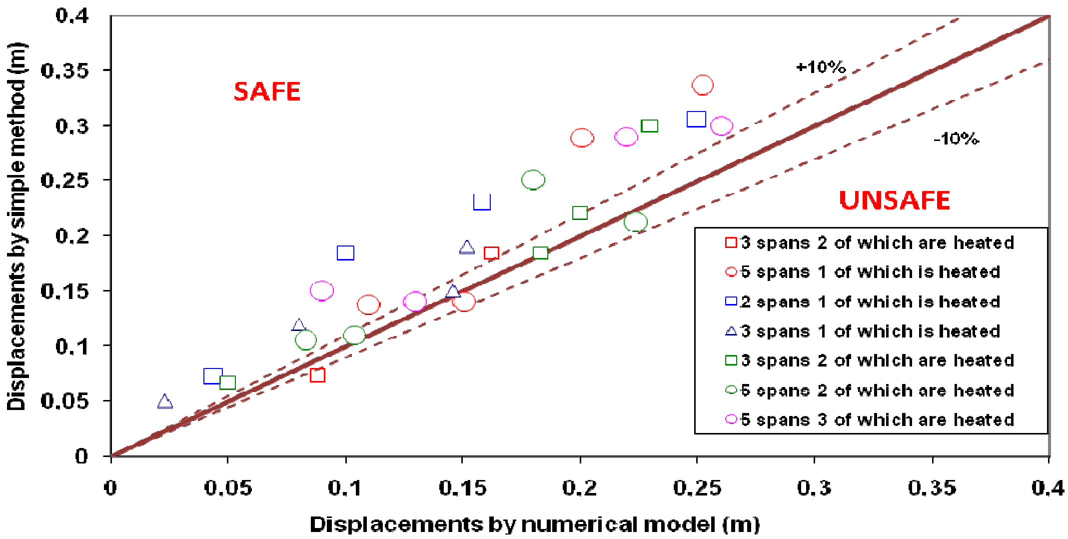


Figure 5-17 : Correlation between displacements calculated using numerical modelling and those calculated using the simple method (Equation 5-16)

The Figure 5-17 gives the correlation between displacement calculated using numerical simulations 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.

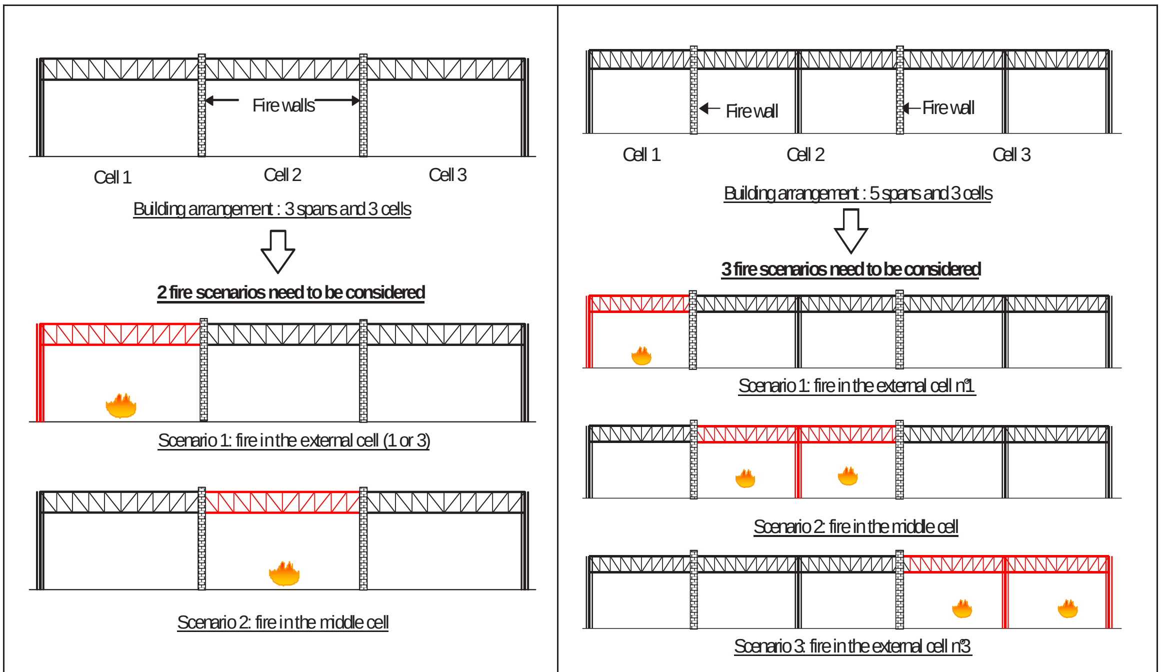
5.2.1 Lattice structures with columns in standard H or I hot rolled profiles.

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;**



a) Frame with 3 spans and 3 cells | b) Frame with 5 spans and 3 cells

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:**
  - ⇒ the design buckling resistance of these elements in fire situation (according to part 1-2 of Eurocode 3 [4]);
  - ⇒ internal forces introduced in these elements due to fire.

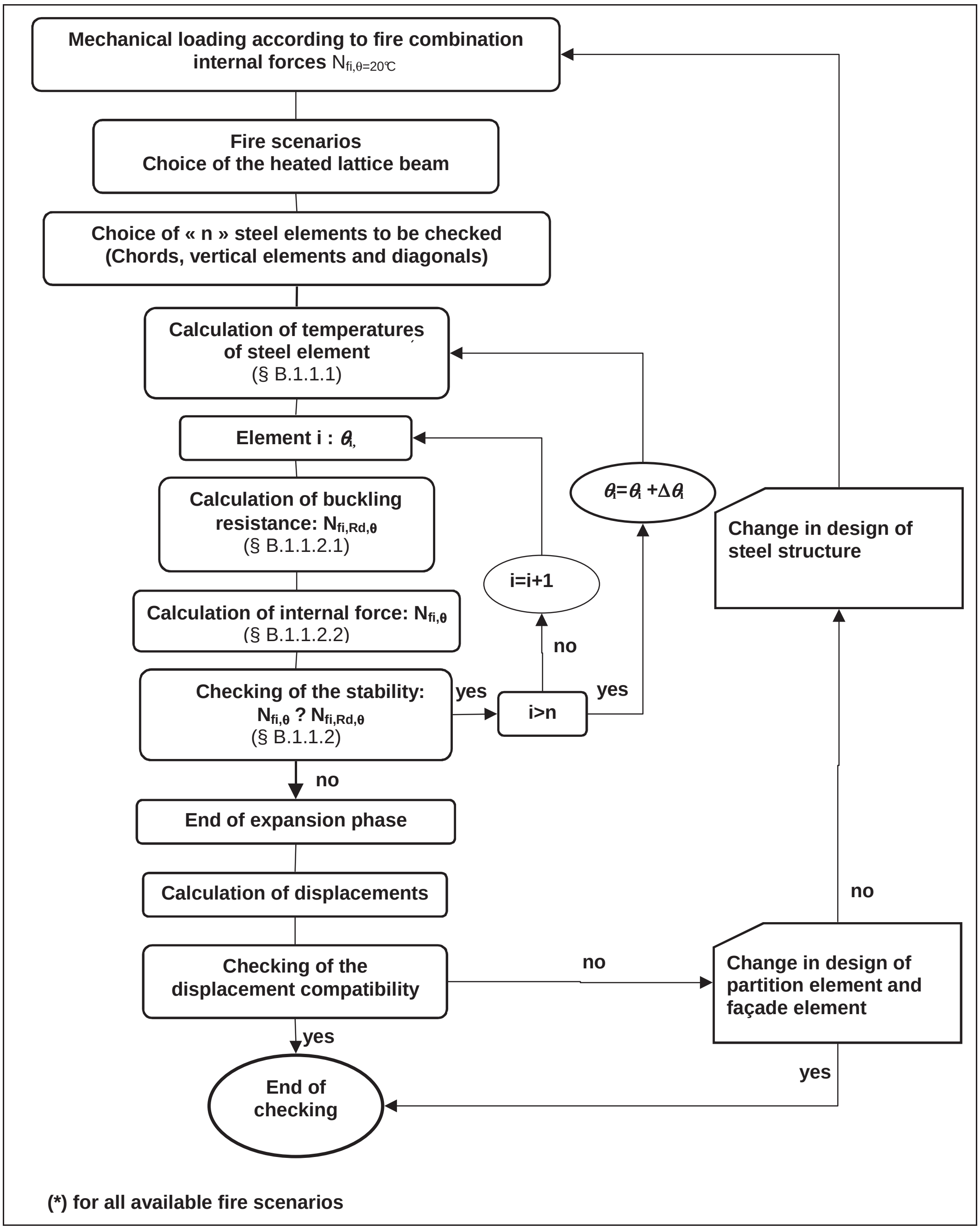


Figure 5-19 Application flowchart of simple model for expansion phase

- **Step 4: Calculation of the maximum displacements at the top of columns supports of both partition and facade elements. Once theses 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.**

5.2.2 Fire compartment in the middle of the storage building: simple heated span

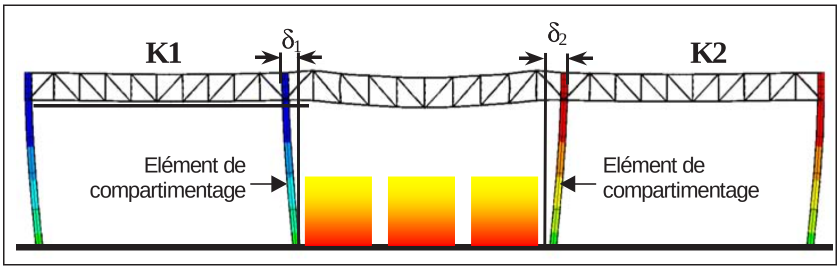


Figure 5-20 Fire compartment in a middle cell

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.:

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

where:

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

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

$$N_{fi,\theta} = N_{fi,\theta=20°C} + \Delta N_{fi,\theta} \qquad \text{Equation 5-18}$$

where:

$N_{fi,\theta=20°C}$ 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 θ, 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 θ, $N_{b,fi,Rd}$, of a steel member subjected to an axial compression should be obtained from:

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

where:

χ_{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 θ.

For a practical use, the buckling coefficient χ_{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 χ_{fi} as a function of relative slenderness $\bar{\lambda}$ and steel grade

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

$$\bar{\lambda} = (\lambda / \lambda_1).(\beta_A)^{0.5}$$ Equation 5-20

where:

$\lambda_1 = 93.9.(235/f_y)^{0.5}$;

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

ℓ_{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 ;

β_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 ℓ 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;

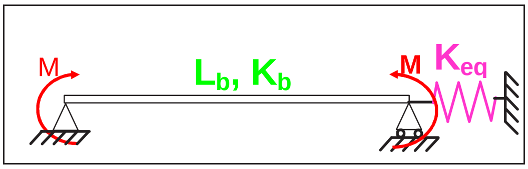


Figure 5-21 Isostatic beam

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

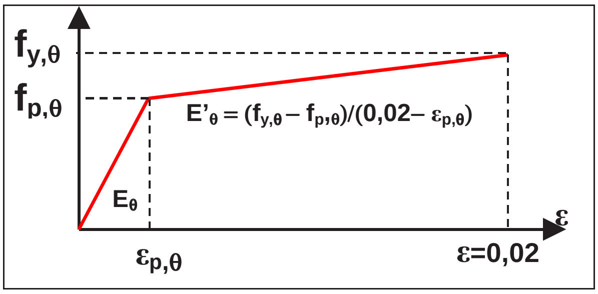


Figure 5-22 Stress-strain relationship for steel

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:

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

where:

θ is the steel temperature;

α is the coefficient of linear thermal expansion (taken as 14.10^{-6}) ;

L_b is the span submitted to fire;

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

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, K_{bi} are the axial stiffness (linear and non-linear elastic) of the chord for the temperature θ.

The axial stiffness K_b, K_{bi} are defined for the temperature θ by:

⇒ $K_b=A.E_\theta/L_b$

⇒ $K_{bi}=K_b$ if $N_{mi,\theta=20°C}+\Delta N_{mi,\theta} \leq N_{el,\theta}$;

⇒ $K_{bi}= A.E'_\theta /L_b$ if $N_{mi,\theta=20°C}+\Delta N_{mi,\theta} > N_{el,\theta}$;

with:

E_θ and E'_θ are the slope of the elastic linear range and the non-linear elastic range for steel at the temperature θ (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:

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

where

δ_0 $(= \Delta N_{mi,\theta} / K_{eq})$ is the displacement at temperature θ due to the above additional compressive force in the bottom chord.

The axial stiffness K_b and K_{bi} are defined for the temperature θ by:

- ⇒ $K_b = A.E_\theta / L_b$
- ⇒ $K_{bi} = K_b$ if $N_{ms,\theta=20°C} + \Delta N_{ms,\theta} \leq N_{el,\theta}$;
- ⇒ $K_{bi} = A.E'_\theta / L_b$ if $N_{ms,\theta=20°C} + \Delta N_{ms,\theta} > N_{el,\theta}$;

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:

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

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:

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

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 θ_c (cf. Equation 5-20) ;

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

θ_c is the temperature reached in horizontal chords members at the end of expansion phase.

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 § 5.2.2 with suitable stiffness K_1 and K_2 as shown in Figure 5-23.

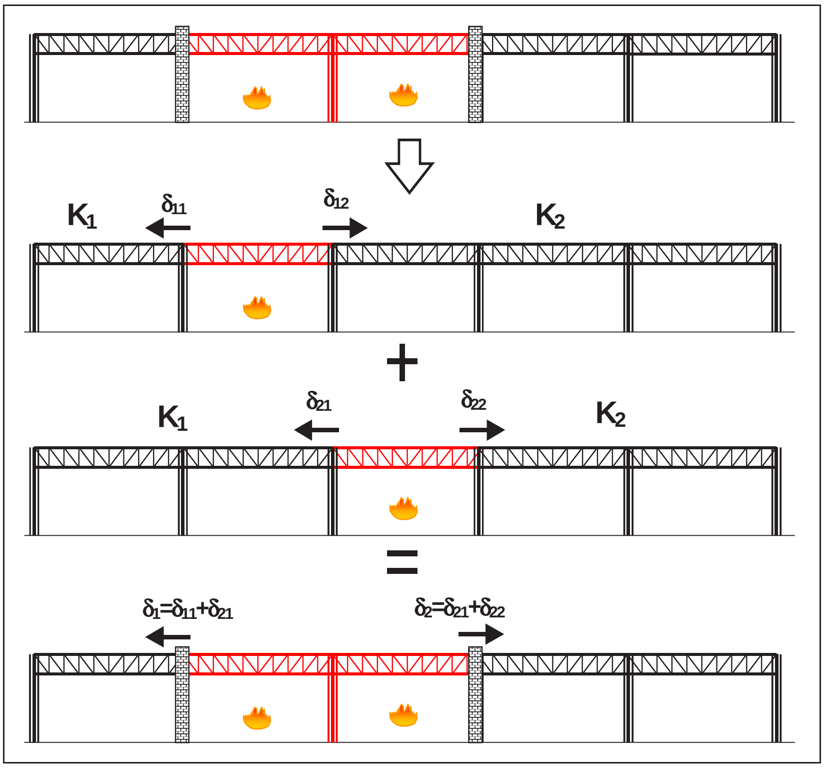


Figure 5-23 Principle of displacement superposition

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 § 5.2.2) 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).

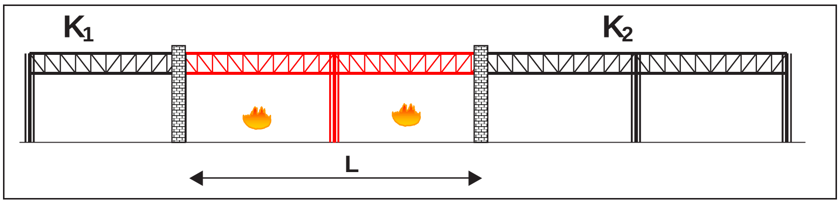


Figure 5-24 Equivalent heated span

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 5.2.2 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:**

$$\delta_1 = \alpha . \sum_{i=1}^{n_1} l_i \, . \, (\theta_c - 20) - \delta_2 \qquad \text{Equation 5-25}$$

where:

ℓ_i is the length of the heated span i;

n_1 is the span number in the fire compartment ;

θ_c is the temperature reached in horizontal chords of lattice beam at the end of expansion phase;

α is the coefficient of linear thermal expansion (taken as 14.10^{-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.

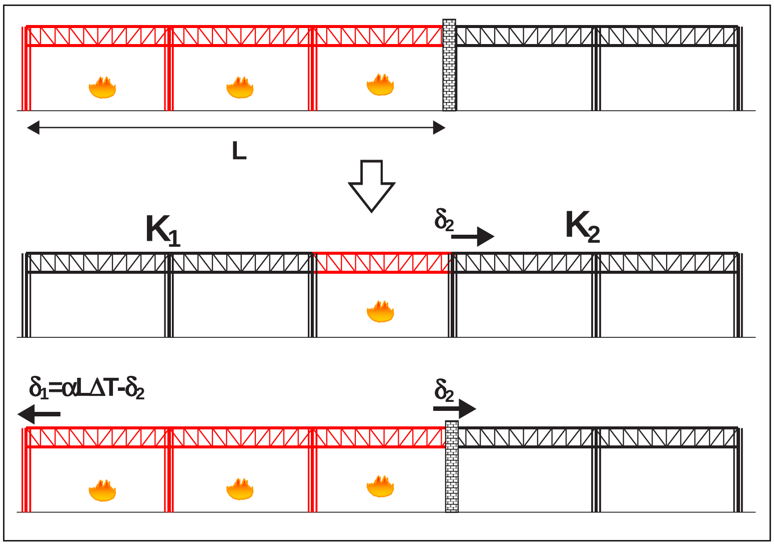


Figure 5-25 Displacements in the case of a fire compartment at the end of building

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. 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*×*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.

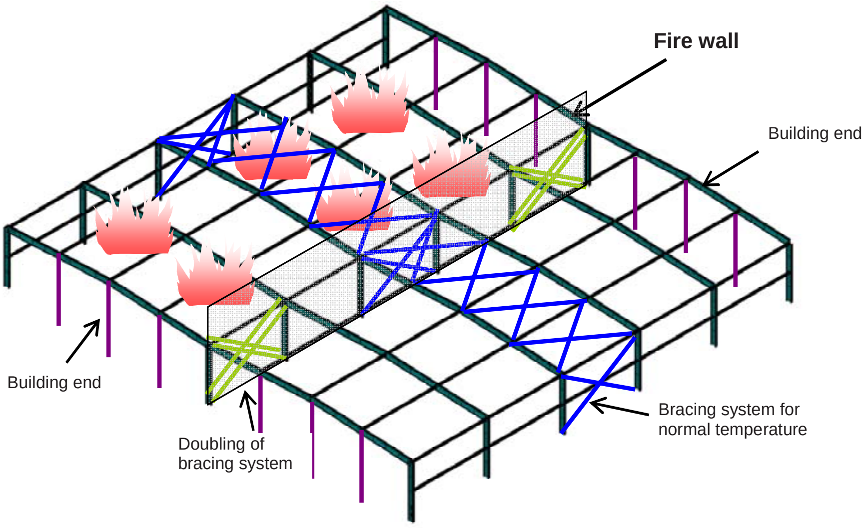


Figure 5-26 Bracing systems at the longitudinal end of the storage building

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.

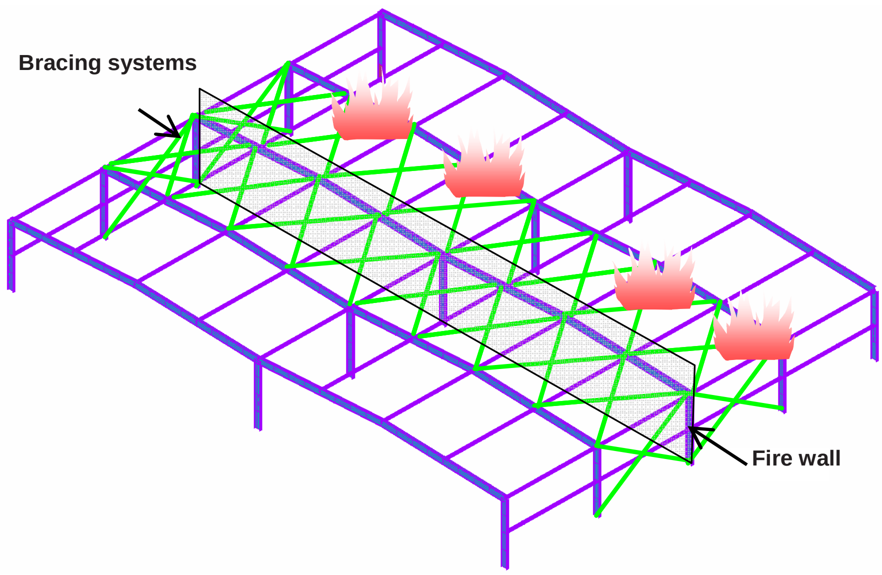


Figure 5-27 Bracing systems of storage buildings

Each bracing system must be designed to support a horizontal uniform load taken as:

$$F = 1.19\, q \qquad \text{Equation 5-26}$$

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.

5.4 Case study for lattice structures

As application example, design methods described previously in § 5.1.4 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 L50×50×5	3

Table 5.6 Main characteristics of steel framework

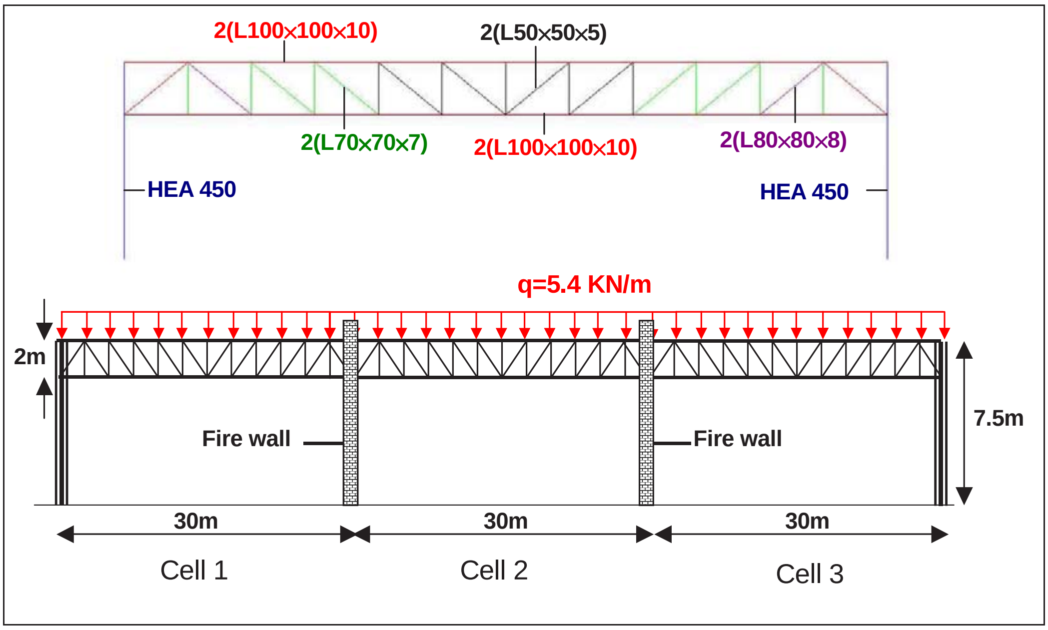


Figure 5-28 Arrangement of steel framework

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:

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

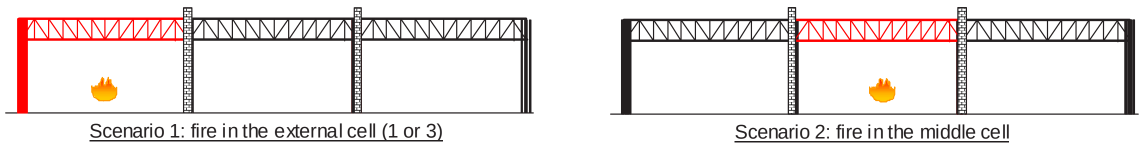


Figure 5-29 Fire scenarios for studied steel framework

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

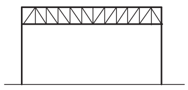
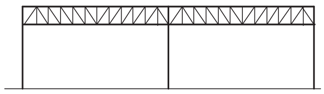
Span number		
Stiffness (N/m)	3538.57	4933.40

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

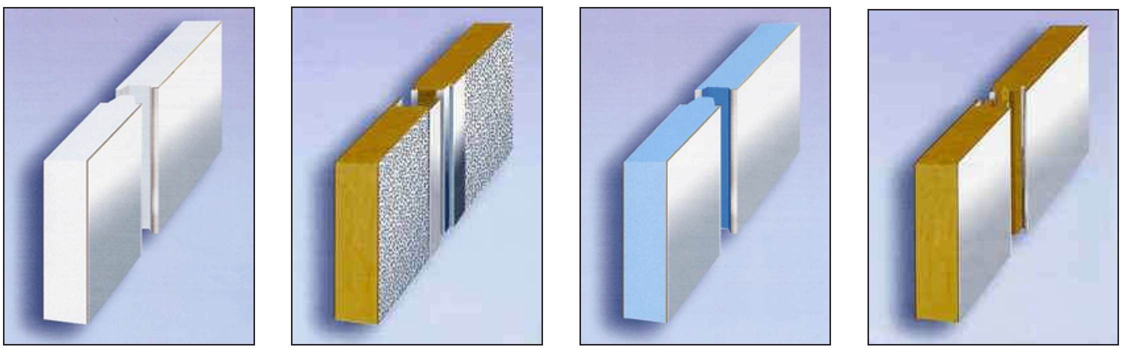
⇒ Frame walls with cold formed sections

⇒ Fire walls with hot rolled profiles and light weight concrete

6.1.1 Isocomposite panels

Product description

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



Lecson Lectol Lecpol Lecfeu

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

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.

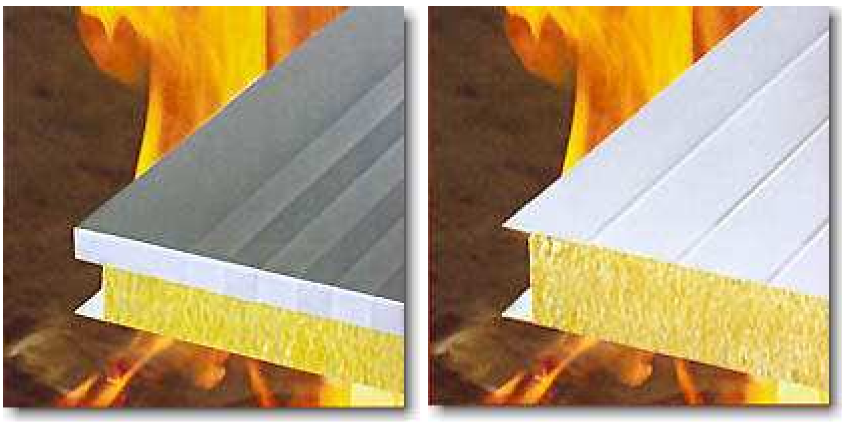


Figure 6-2 Fireproof Panel

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

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 W/m²K;

⇒ for 200 mm K = 0.24 W/m²K;

Acoustic performance

Sound insulation:

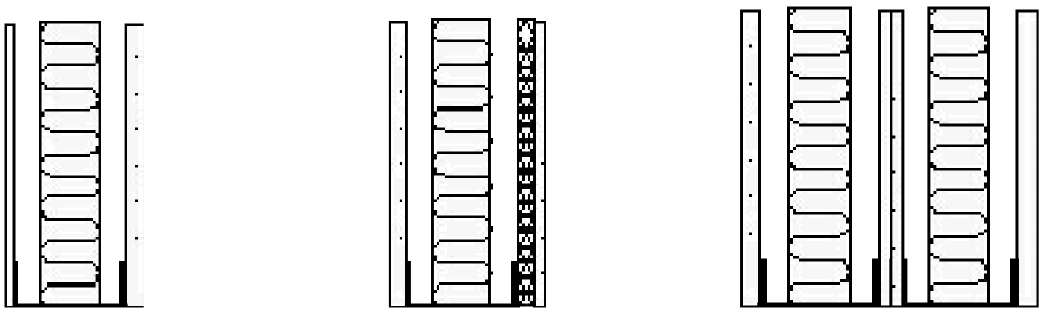
⇒ 35 to 60 mm: 34dB;

⇒ 70 to 200mm: 35dB;

6.1.3 Frame walls with cold formed sections

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.



FFW01 FFC02 FFW03

Figure 6-3 Cold formed sections

Application field

Partitions walls and fire resistant walls.

Technical comments

Dimensions

FFW01

93mm steel channel 1.2mm gauge (CH9312). Internal lining: One layer 15mm Lafarge Megadeco plasterboard. Insulation: 50mm mineral wool density 33kgm³.Weight 26 Kg/m², 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/m³.Weight 27 Kg/m²,

FFW03

Internal Lining: One layer 15mm Lafarge Megadeco plasterboard. Sandwich lining: Two layers 9mm Minerit. Weight 52 Kg/m²,

	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

Product description

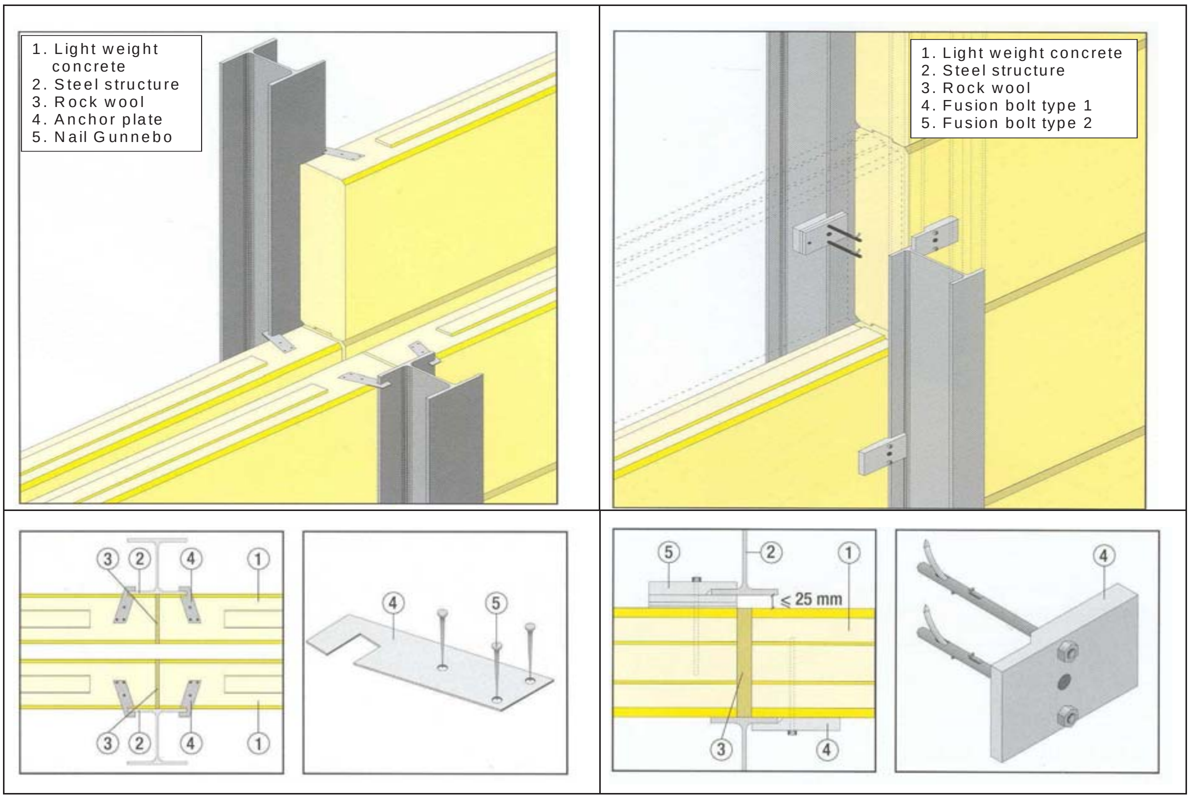


Figure 6-4 Doubled wall (left side) and simple wall with fusion bolts (right side)

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.

Application field

Partitions walls and fire resistant walls.

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 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 Lamba is 0.15 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.

6.2.1 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.

6.2.2 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.

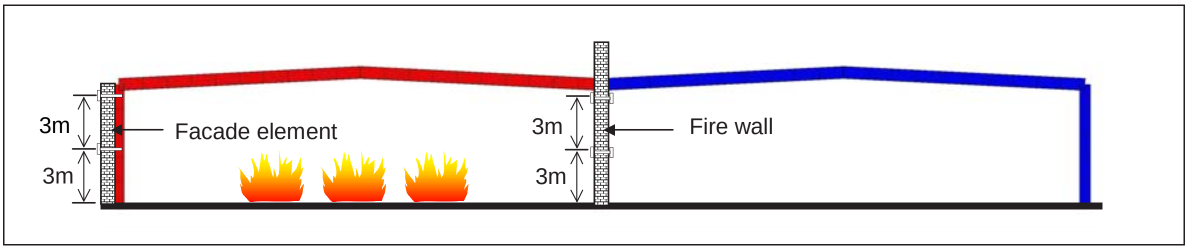


Figure 6-5 Design detail for separation elements

6.2.3 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.

6.2.4 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.

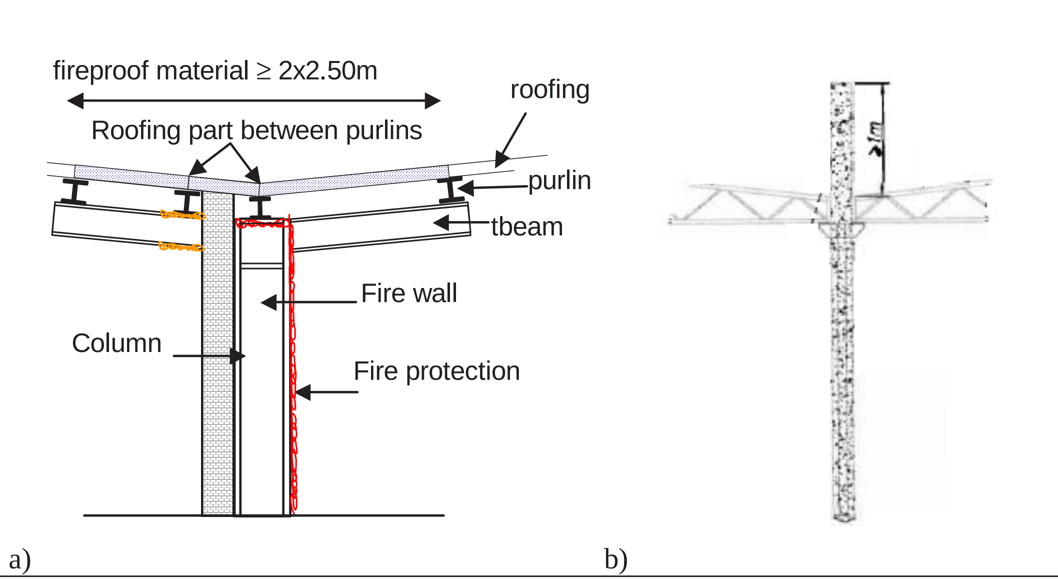


Figure 6-6 Design details near to the roof

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)).**

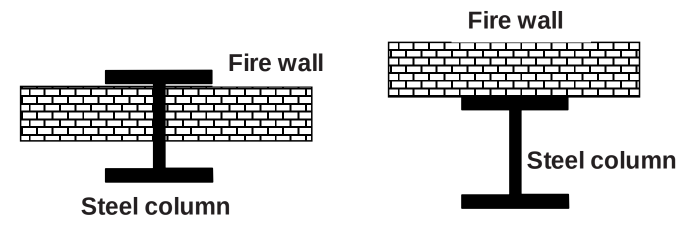
6.2.5 Design recommendations for fire walls perpendicular to steel frames

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

- **COLUMS 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:

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



a) fire wall inserted between the flanges of the columns

b) Fire walls fixed to one flange of the columns

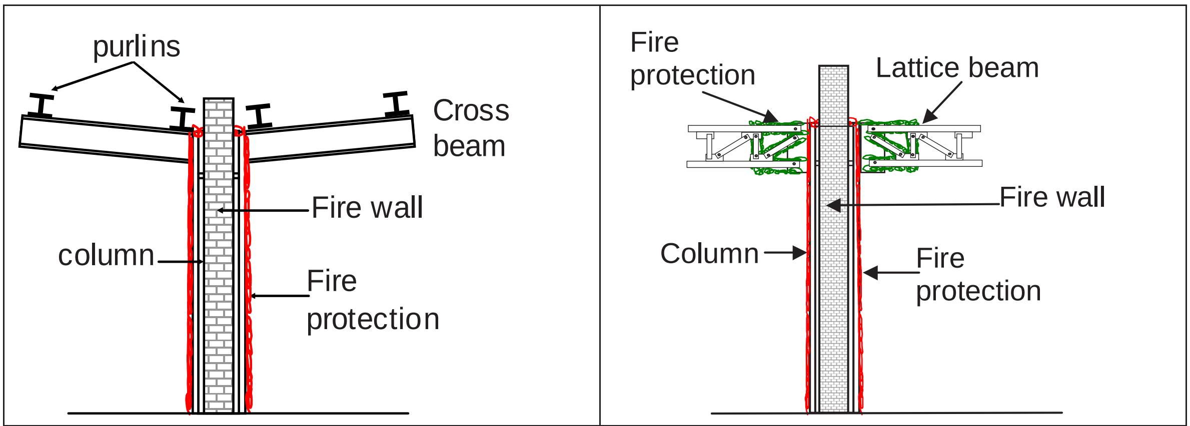
Figure 6-7 Arrangement of separation elements

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).

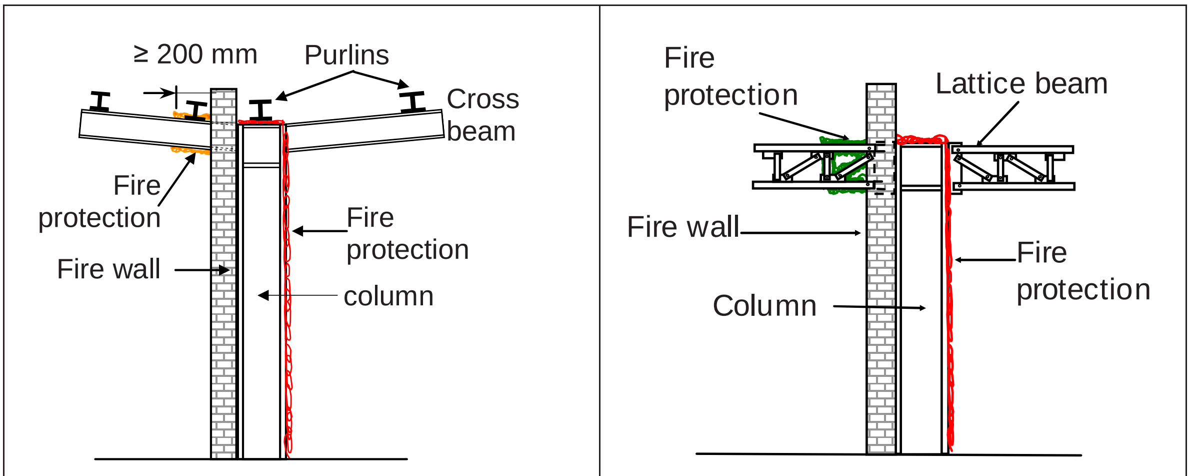
In the similar way, when the fire wall is built beside one flange of columns, to prevent wall damage with the collapse of the beam near the fire wall, a fire protection must be used on beam (on the side of wall):

- **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).**



a) Portal steel frames | b) Lattice steel structure

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

6.2.6 Design recommendations for fire walls parallel to steel frames

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

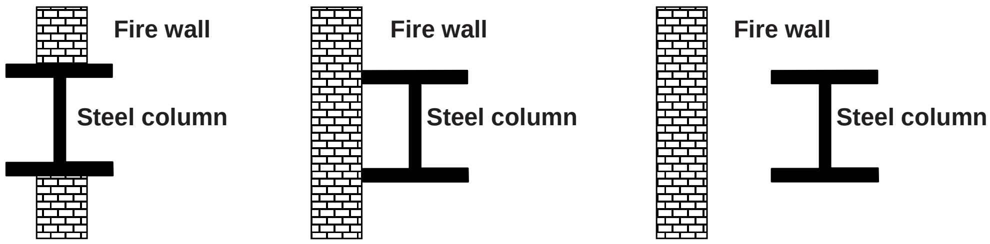
- **COLUMS that are into or near a wall must be always fire protected.**
- **BEAMS that are into or near a wall must be always fire protected.**
- **PURLINS are going to cross the walls so it is necessary to fire protect the continuous purlins (over a distance of 200mm from the wall) or design a not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



a) Fire wall in the plan of steel frame

b) Fire walls joined with steel frame

c) Fire walls between two steel frames

Figure 6-10 Arrangement of partition elements

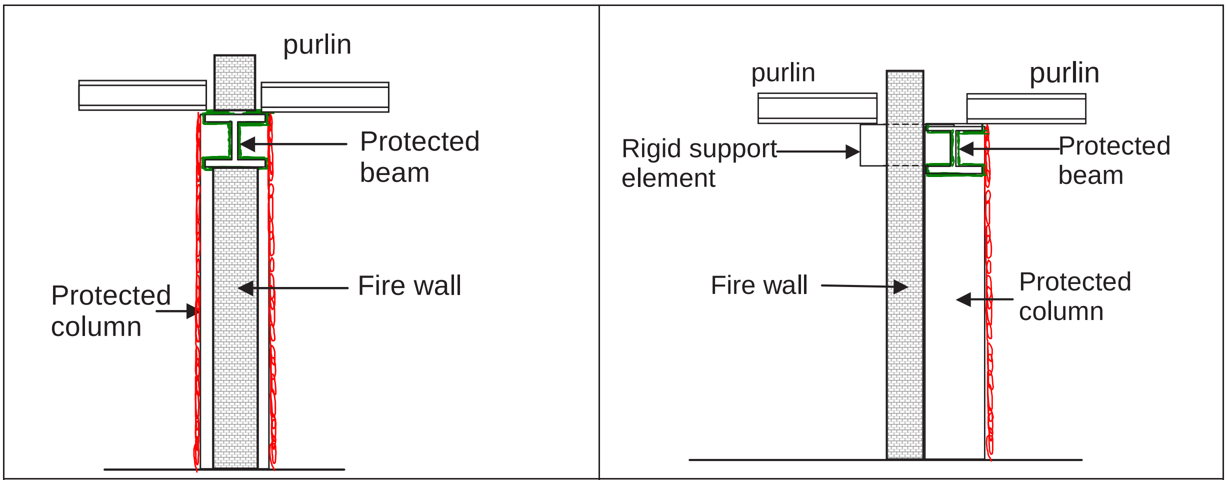
Requirements of no fire propagation and no progressive collapse between different compartments (stability of the cold parts of the structures) lead to apply a fire protection on steel frames (beams and columns) near fire walls (see Figure 6-11 and Figure 6-12).

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.**



a) fire wall inserted in steel frame | b) Fire walls joined with the steel frame

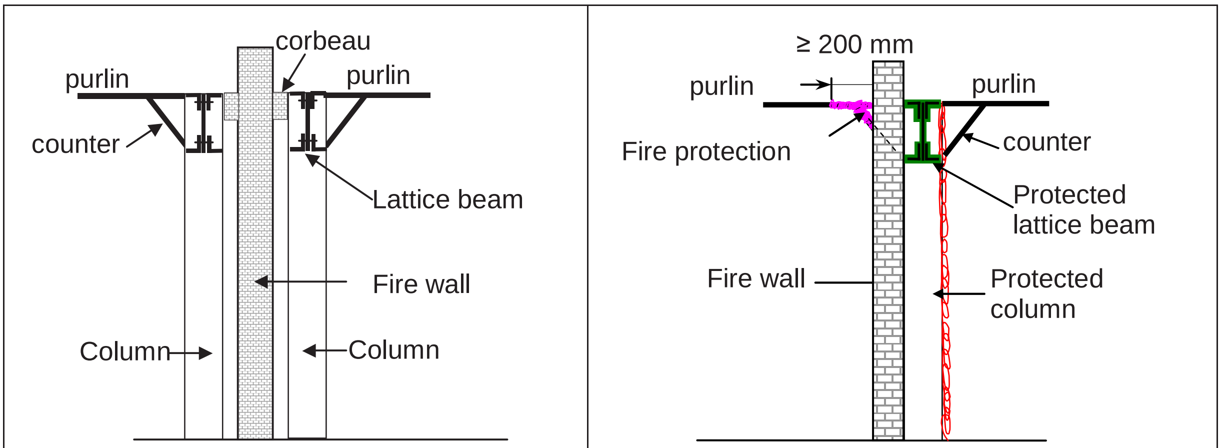
Figure 6-11 Design details of portal steel frame near to fire wall

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.



a) fire wall between two independent steel framework | b) Fire wall joined with the steel frame

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.

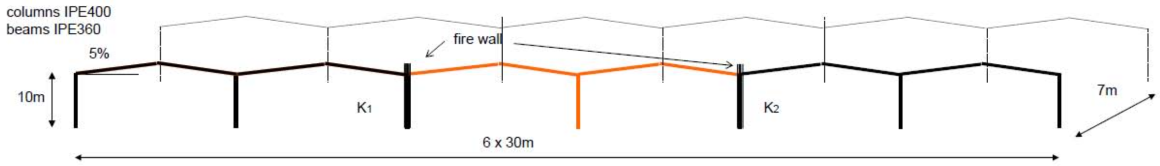


Figure 8-1 Multi-bay portal frame with 3 fire compartments

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

$c_p = 1.16$ 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

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

Step 3 Vertical load

weight of the roof	0.25 kN/m^2
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^2

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

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

Lateral displacement

Step 1 Reduction factor related to the slope of the roof

from Equation 4-9

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

Step 2 Equivalent lateral stiffness of the cold part ot 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 m^4 second moment of area for beams for IPE 360

f = 0.75 m ridging

h = 10 m height of the column

l = 30 m span of one bay

$I_c =$ 2.31E-04 m^4 second moment of area for column for IPE 400

E = 2.10E+08 kN/m^2 Young's modulus for steel for normal temperature

from Equation 4-13

$\alpha =$ 0.220550061

k = 71.8065082 kN/m

c = 1.765646549

$K_1 = K_2 =$ 126.7849134 kN/m

Step 3 Lateral displacements in the expansion phase

from Equation 4-15

$\delta_1 =$ 1.321532629 m = 132.15 cm

$\delta_2 =$ 1.321532629 m = 132.15 cm

Step 4 Maximum displacement induced by the tensile force

from Equation 4-16

$\delta_{max1} =$ 1.321532629 m = 132.15 cm

$\delta_{max2} =$ 1.321532629 m = 132.15 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.

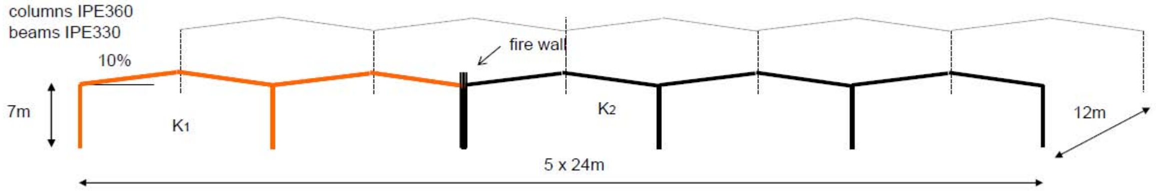


Figure 8-2 Multi-bay portal frame with 2 fire compartments

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

$c_p = 1.10$ 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

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

Step 3 Vertical load

weight of the roof 0.25 kN/m^2

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^2

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 for portal frame when roof slope equals 10%

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

from Equation 4-14

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

from Equation 4-14

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

$I_b =$ 1.18E-04 m^4 second moment of area for beams for IPE 330

f = 1.2 m ridging

h = 7 m height of the column

l = 24 m span of one bay

$I_c =$ 1.63E-04 m^4 second moment of area for column for IPE 360

E = 2.10E+08 kN/m^2 Young's modulus for steel for normal temperature

from Equation 4-13

$\alpha =$ 0.17655

k = 97.0245 kN/m

c = 2.77865

$K_1 = 0.13 \cdot 97.0245$ kN/m = 12.6132 kN/m

$K_2 =$ 2.77865· 97.0245 kN/m = 269.597 kN/m

Step 3 Lateral displacements in the expansion phase

from Equation 4-15

$\delta_1 =$ 0.68782 m = 68 cm

$\delta_2 =$ 0.03218 m = 3.2 cm

Step 4 Maximum displacement induced by the tensile force

from Equation 4-16

$\delta_{max1} =$ 7.47654 m = 747.654 cm

$\delta_{max2} =$ 0.34979 m = 34.979 cm

European Commission

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