Fire Safety of Industrial Hall

Design Guide

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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:

$\Rightarrow$ Length of a span for simple bay: 15m, 20m and 30m

$\Rightarrow$ Length of a span for multiple bay: 20m, 30m and 40m

$\Rightarrow$ Height – simple bay: 5m, 7.5m and 12.5m

$\Rightarrow$ Height – multiple bay: 7.5m, 12.5m and 20m

$\Rightarrow$ Slope: 0° to 10°

$\Rightarrow$ Number of bays: 1, 3 and 5

$\Rightarrow$ 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]
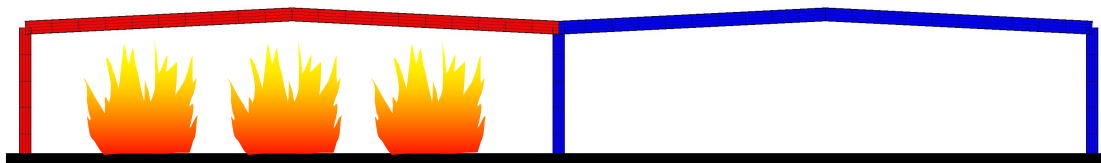
Figure 2.1 Heating condition

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;**
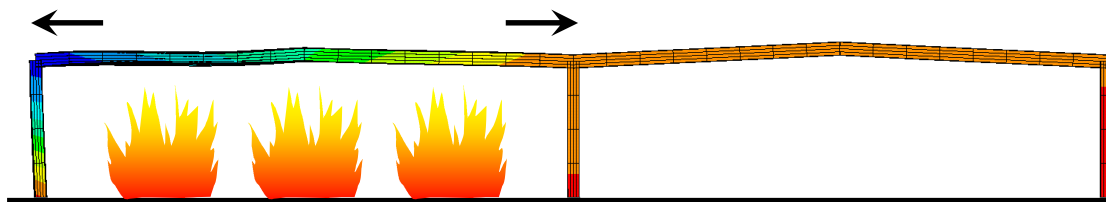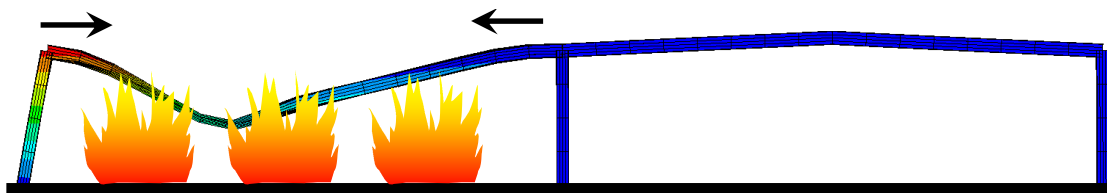

Figure 2.2 Deformed shape during the expansion phase

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[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.

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;**

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.**
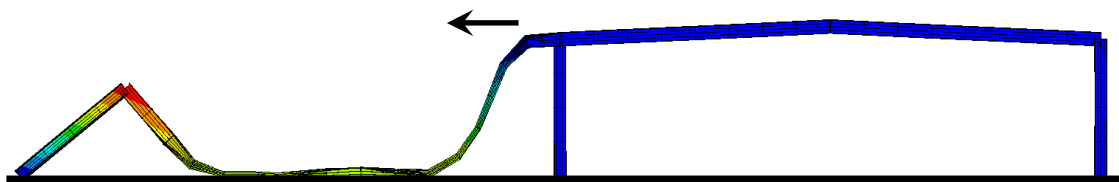
Figure 2.4 Deformed shape at he end of the collapsing phase

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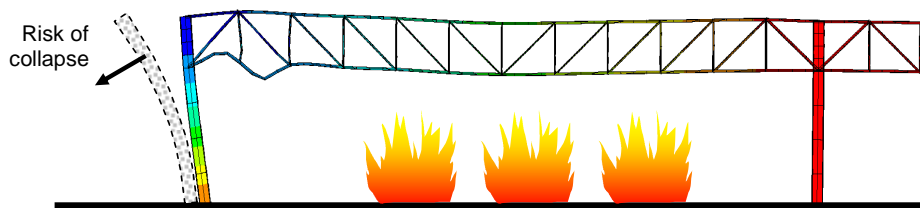
Figure 3-1 Risk of collapse of facade elements towards the outside during the expansion phase

➢ **collapse of adjacent cold structures – progressive collapse.**
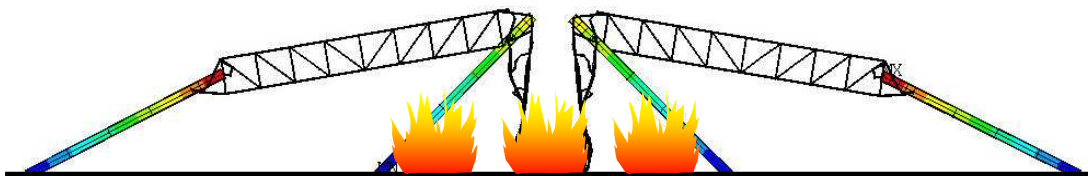
*Figure 3-2 Risk of progressive collapse of the load bearing structure*

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 and Figure 3-4 show examples of steel frames as well as possible locations of the fire walls in the building.

Fire is assumed to be developed in the whole fire compartment.
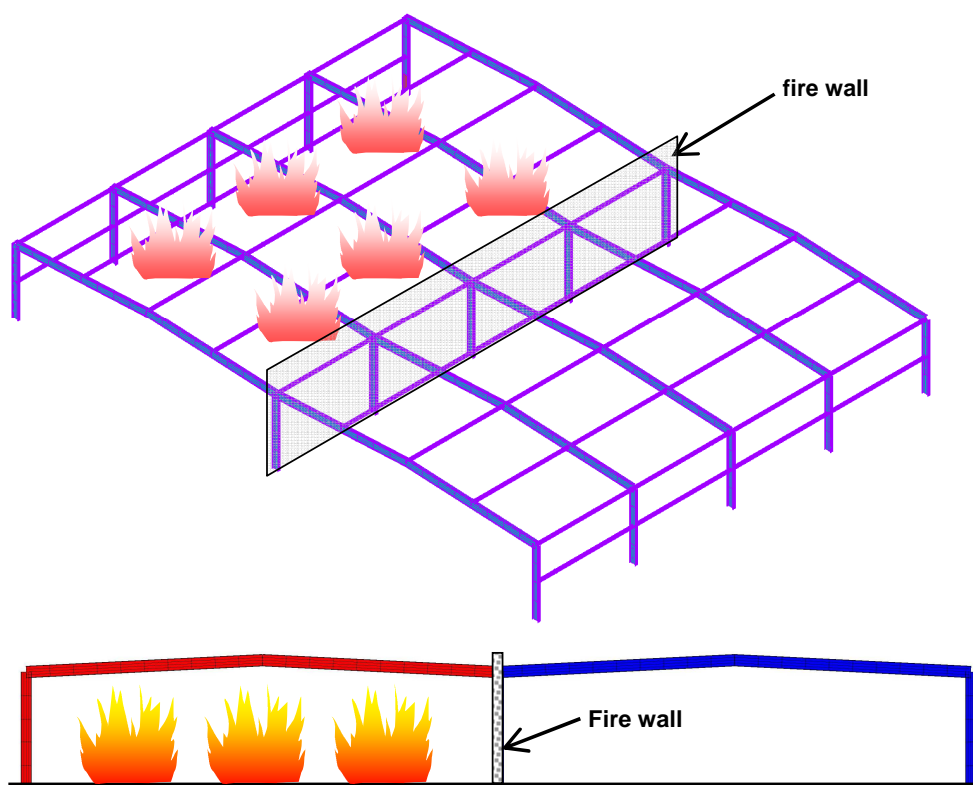
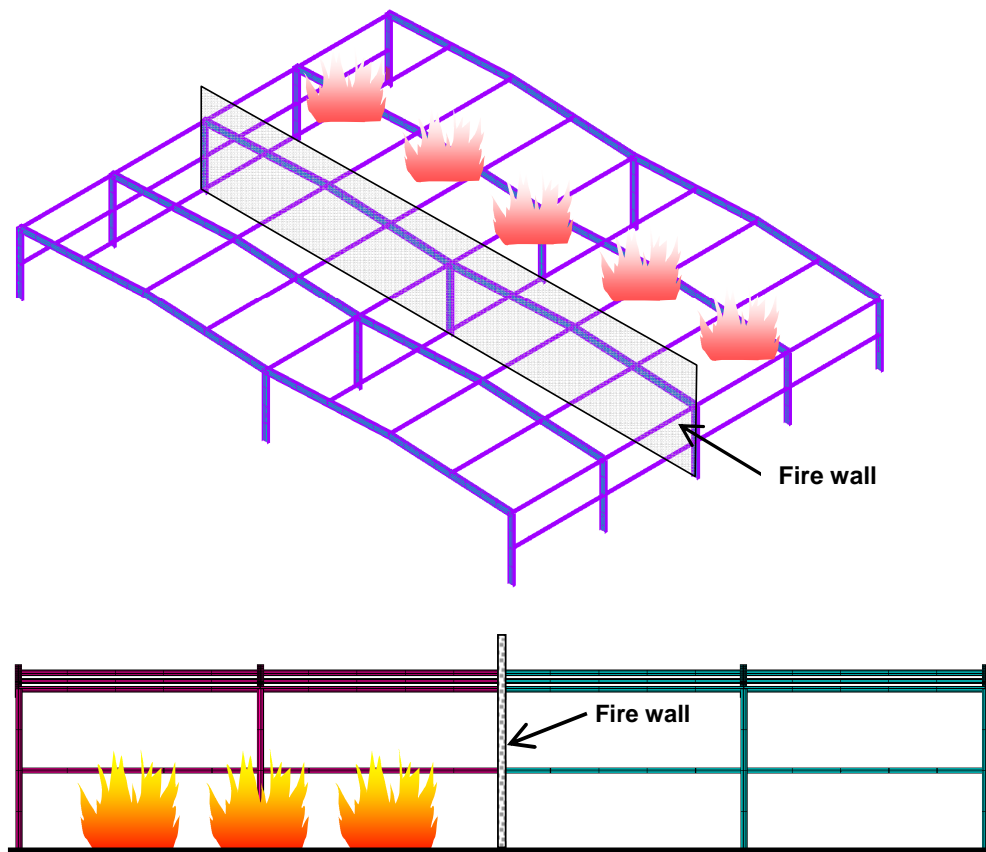*Figure 3-3 Fire wall perpendicular to portal frame*

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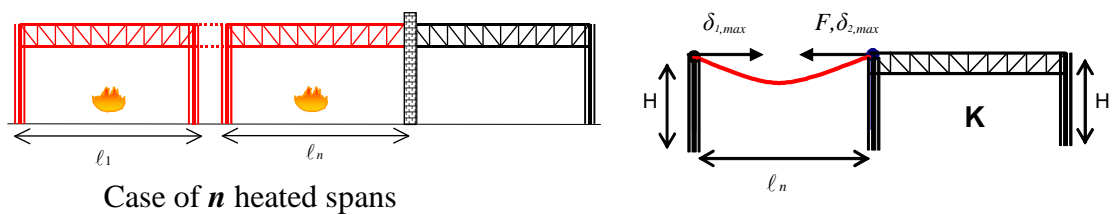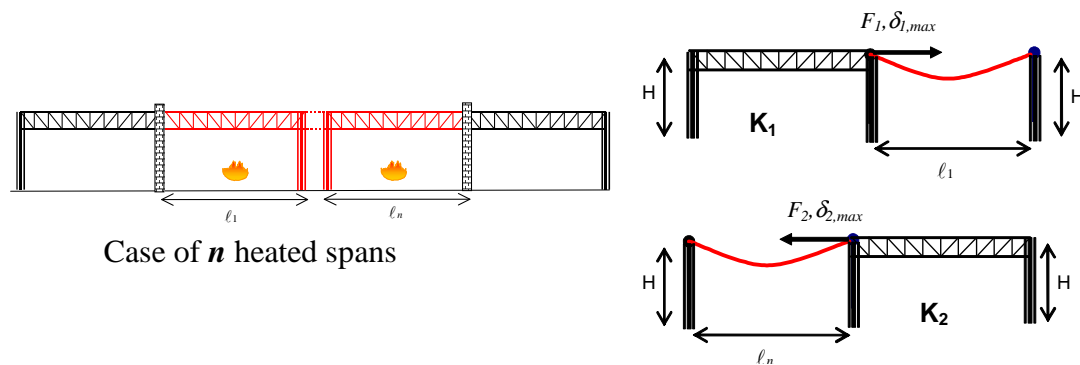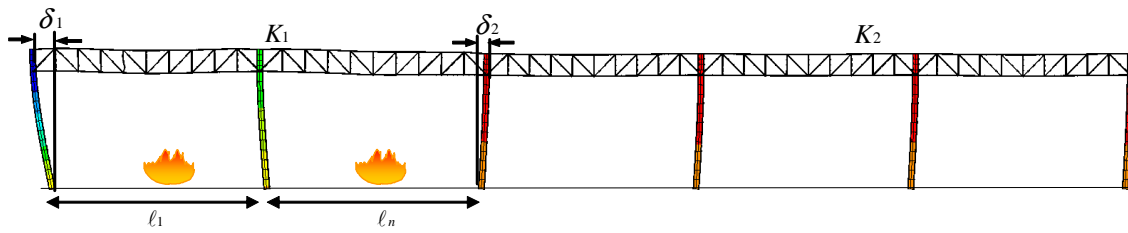
b) Fire compartment at the middle of the storage building

Figure 4-1 Maximum displacements and forces transmitted to cold parts of the structure
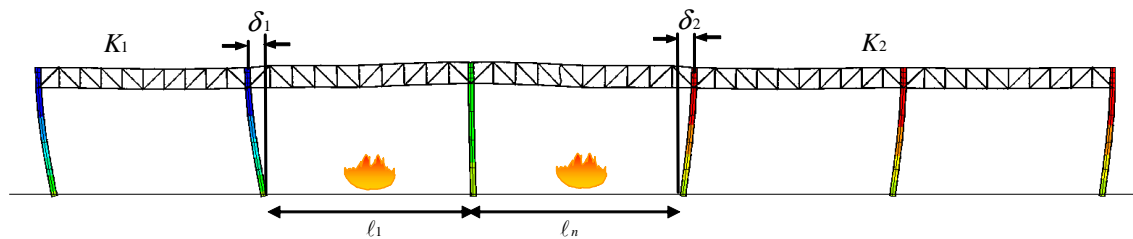
➢ **Lateral displacements:**

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

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:

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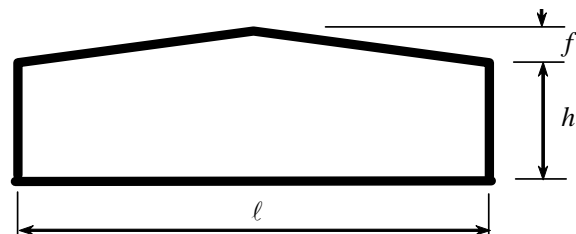
where

h − is the height of the columns

l − is the span of the beam

➢ **Tensile force:**

Not applicable

➢ **Lateral displacement:**

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

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 – <u>Coefficient related to the slope of the roof c_p</u>

⇒ Portal frame

$$c_p = \begin{cases} 1.19 & \text{for} \quad 0\% \ \text{slope} \\ 1.16 & \text{for} \quad 5\% \ \text{slope} \\ 1.10 & \text{for} \ 10\% \ \text{slope} \end{cases}$$
<div align="right">Equation 4-3</div>

⇒ Lattice frame

$$c_p = 1.45$$
<div align="right">Equation 4-4</div>

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

⇒ Portal frame

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

Equation 4-5

⇒ Lattice frame

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

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]

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

where

G – is the dead load

S – is the characteristic snow load in fire conditions

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^{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.

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

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

where

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

➢ **Lateral displacement:**

Step1 – <u>Reduction factor related to the slope of the roof c_{th}</u>

⇒ Portal frame

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

⇒ Lattice frame

$$c_{th} = 0.009 \qquad\qquad \text{Equation 4-10}$$

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

⇒ If fire compartment is in the middle of the frame as illustrated in Figure 4-3

K_1 and K_2 should be calculated by one of the classical elastic methods.
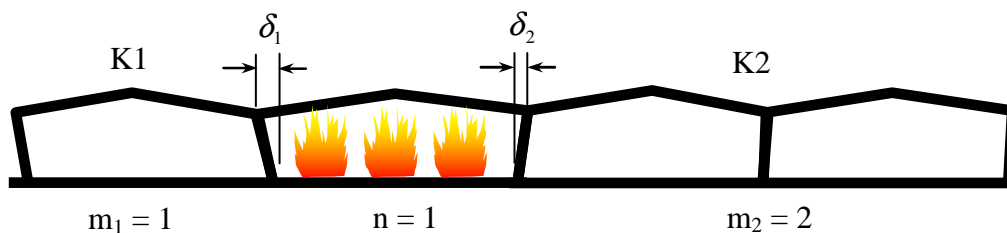
Figure 4-4 Fire located in a cell at the middle of the storage hall

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 m_i 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

with

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

$$c = 1 + \sum_{i=2}^{m} \frac{i}{2}\ \frac{2\alpha+1}{1+2i\alpha}$$

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

Equation 4-13

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 [m^4]

I_c – is the second moment of area of the column [m^4]

E – is the modulus of elasticity of steel for normal temperature [N/m²]
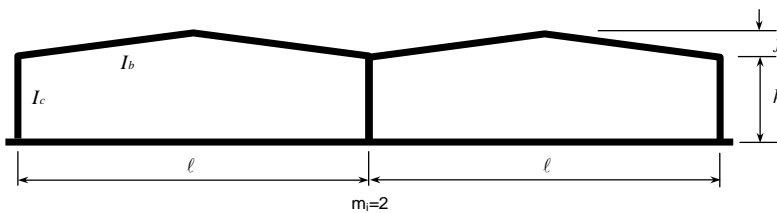
Figure 4-5 Definition of parameters of cold parts

$\Rightarrow$ 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:

$$K_1 = \begin{cases} \begin{cases} 0.065\,k & \text{for } n = 1 \\ 0.13\,k & \text{for } n = 2 \\ 0.13\,c\,k & \text{for } n > 2 \end{cases} & \text{for portal frame} \\ \\ \begin{cases} 0.2\,K_2 & \text{for } n = 1 \\ 0.3\,K_2 & \text{for } n \geq 2 \end{cases} & \text{for lattice frame} \end{cases} \qquad \text{Equation 4-14}$$

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 4-5.
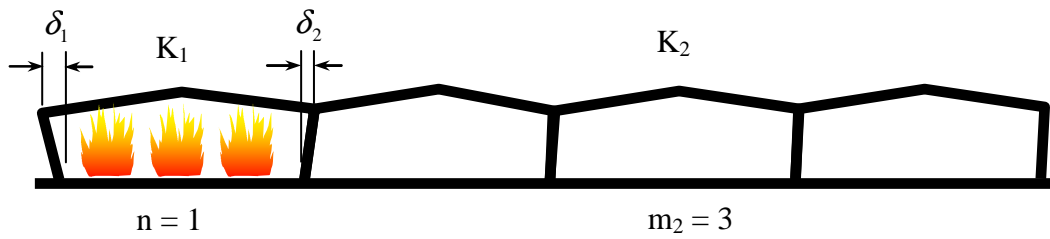
Figure 4-6 Fire in a compartment at the end of the storage building

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

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

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

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

where

F - is the tensile force calculated in Equation 4-17

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

Report FS+

1] Introduction

1.1] Applicability

This Software is applicable to the industrial halls that respect 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 H or I;

- Storage buildings divided in one or several cells separated from one another by fire walls, these walls can be either parallel to steel portal frames, either perpendicular to steel portal frames.

1.2] How to treat the results

The displacements that will be obtained in stage of pull must be used to verify the stability of the interior and exterior walls ;

The force that will be obtained in stage of pull must be used to verify the stability of the cold part (force located in the top of the column).

2] Input data

3] Results of the calculations

3.1] Stage of push

Displacements induced at the compartment ends
d1 = 410 mm
d2 = 405 mm

3.2] Stage of pull

Horizontal tensile forces induced at the top of the columns

Maximum displacement(s) induced at the top of the columns
d1 = 19 mm
d2 = 17 mm

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.
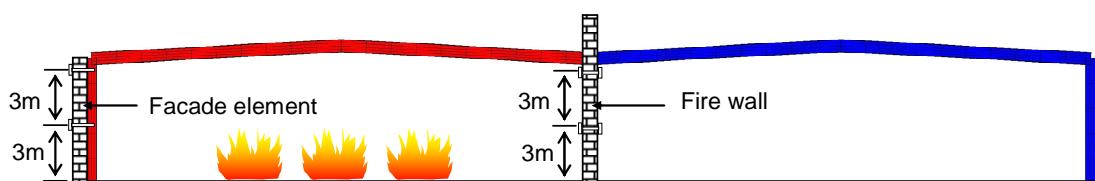
Figure 6-1 Design detail for separation elements

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:

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

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 <u>columns and beams</u> 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 <u>lattice beams</u> 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.
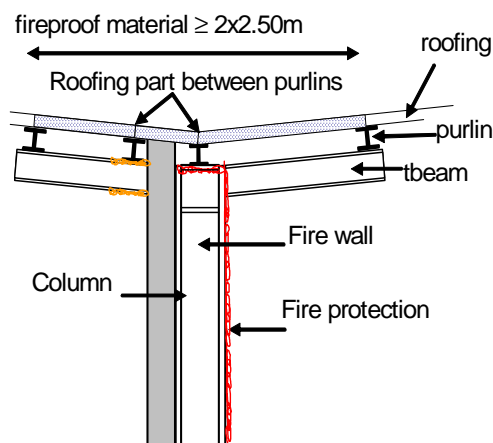
Figure 6-2 Protection of the roof

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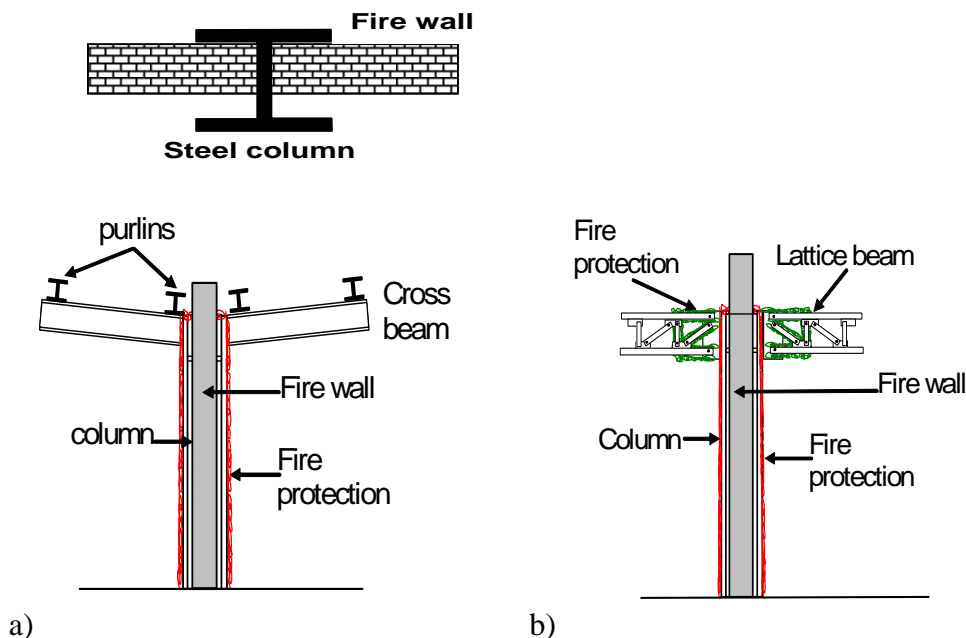
Figure 6-3Fire protection required for near the fire wall inserted between the flanges of the column a) portal frame b) lattice frame

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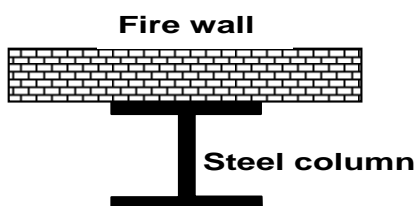
If the fire wall is built beside one flange of columns, to prevent wall damage caused by the collapse of the beam adjacent to the fire wall, a fire protection must be applied to the beam:

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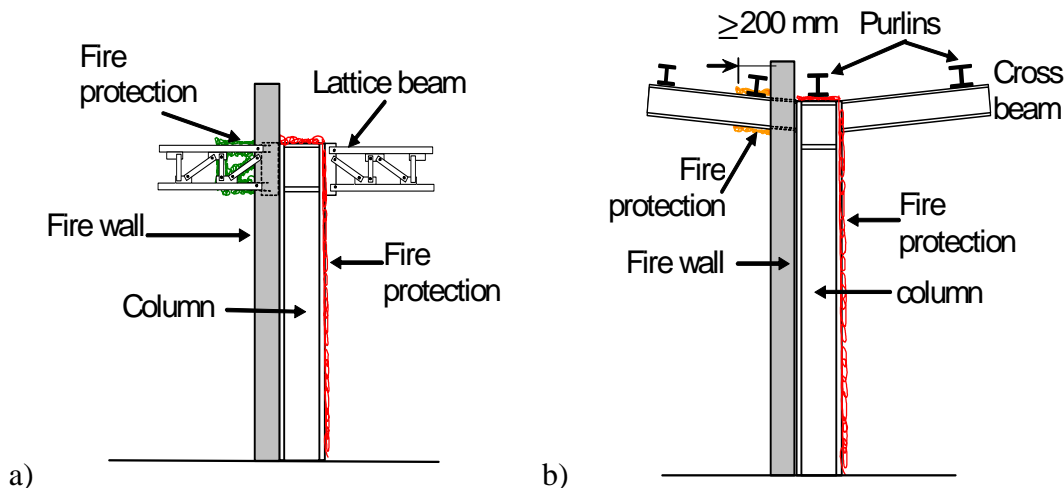
Figure 6-4 Fire protection required for beams and columns near fire wall fixed to one flange of the column a) lattice frame b) portal frame

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
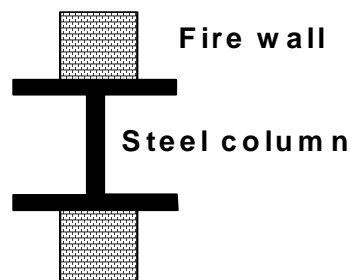
In this situation the beam and the column has to be fire protected.
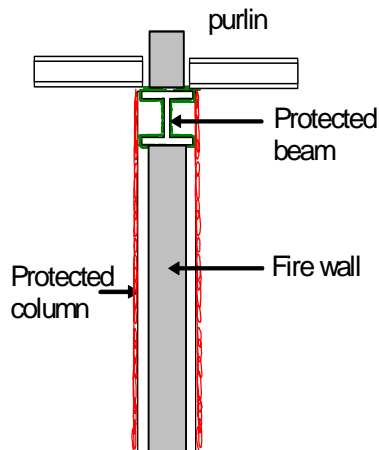
Figure 6-5 Fire protection for the column when fire wall is in the plane of the frame

6.3.2. Fire wall attached to the steel frame
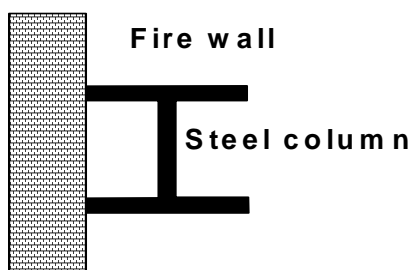
Steel elements going across a fire wall should not affect the fire performance of the wall (stability, thermal insulation qualities…). It is thus necessary to consider design solutions so that the collapse of the roofing structure closest to the fire wall doesn't initiate the failure of the wall.
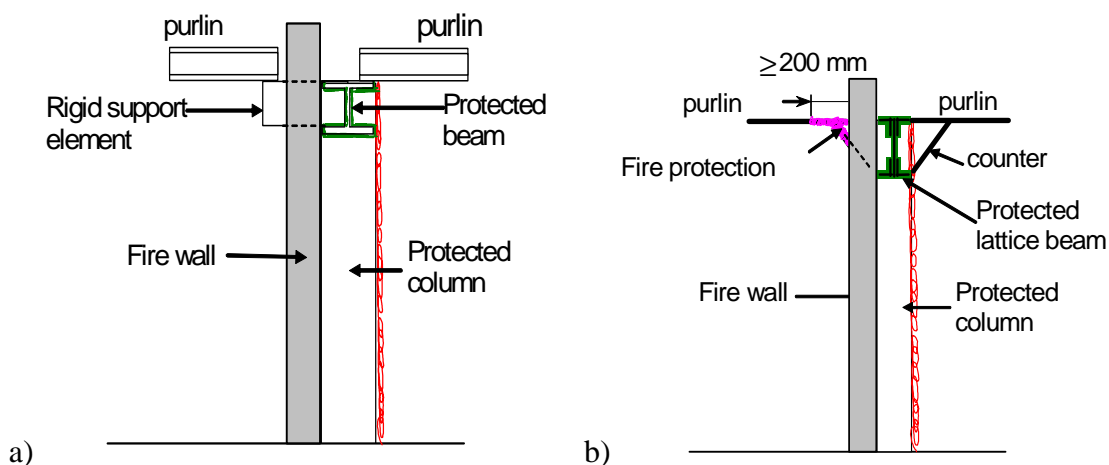
Figure 6-6 Design details for elements near fire wall

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
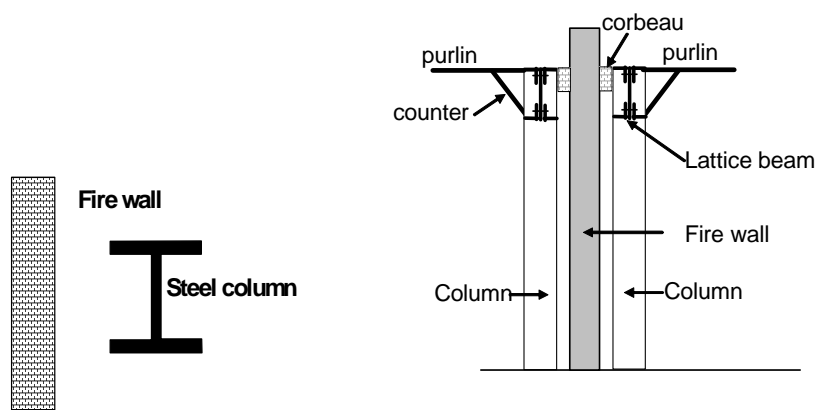
Figure 6-7 Fire wall between two portal frames

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 (see Figure 6-8). 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.**

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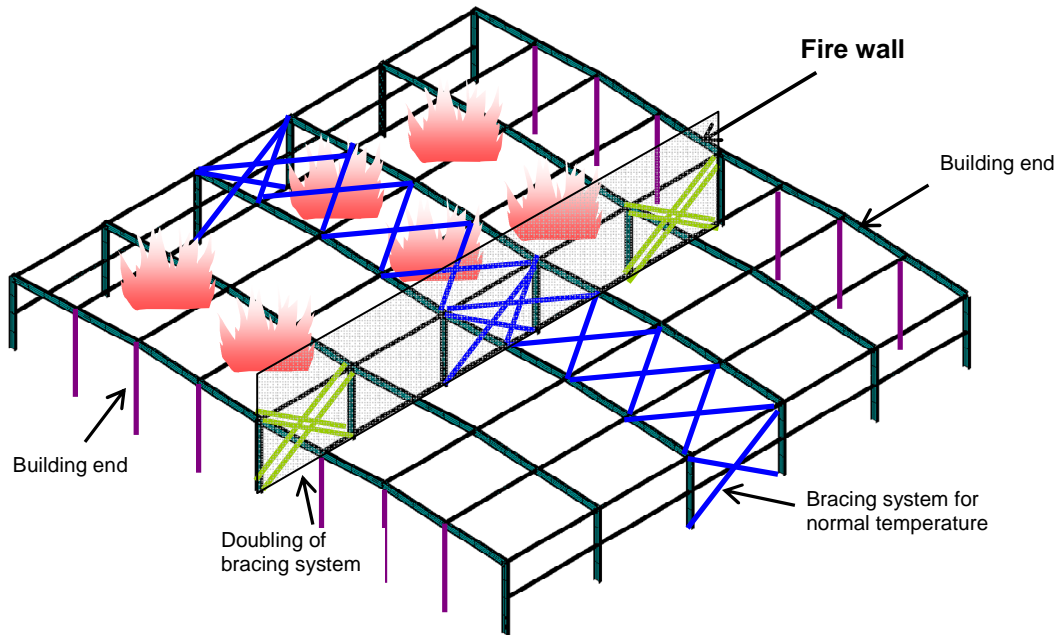
Figure 6-8 Bracing systems at the longitudinal end of the storage building

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 (Figure 6-9). Each bracing system must be designed to support a horizontal uniform load taken as:

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

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.

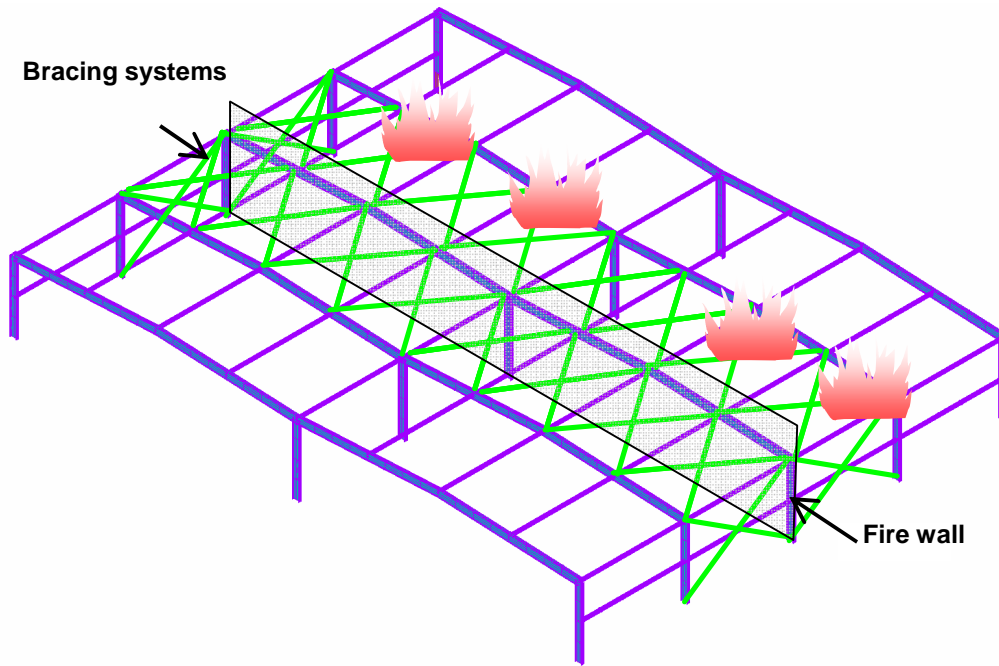*Figure 6-9 Bracing systems of storage buildings*

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 (Figure 6-10).

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.
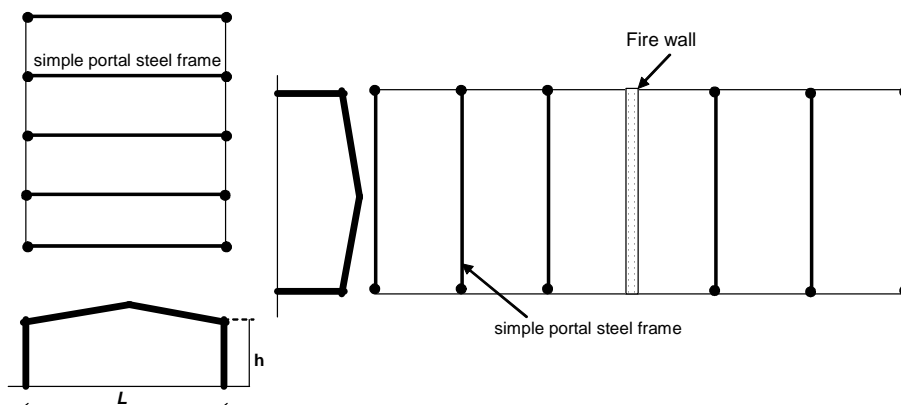

Figure 6-10 Industrial buildings with simple portal steel frame

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