Research Programme of the Research Fund for Coal and Steel

FIre and Seismic performances of Hybrid fire WALLs in case of single-storey industrial and commercial steel buildings (FISHWALL)

Current practice for single storey steel framed buildings and review of fire and seismic regulations

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WP 1: Analysis of regulation requirements, fire and seismic actions applied to partitions walls and design of tests

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ABSTRACT

It is well known that the intrinsic fire resistance of single-storey unprotected steel-framed buildings is largely sufficient to guarantee the evacuation of occupants in the event of fire. In consequence, for this type of building, the main concern of national fire regulations in Europe is how to prevent the spread of fire to the whole building. To achieve this objective, two performances shall be usually satisfied, namely, the appropriateness of constructive systems to ensure that there is no progressive collapse between fire compartments, and the efficiency of fire walls to stop the fire inside the initial compartment regardless of the state of structures exposed to fire. In practice, many constructional solutions can be implemented in order to preserve the integrity of the fire walls, while accepting that the fire exposed part of the structure may collapse. One of the most common solutions is to place a non-load bearing wall between two independent steel structures and to connect it to them by means of "fusible" links. In fire situation, these fusible links have to allow the wall to be disconnected from the structure affected by fire without endangering the separating function of the wall, which shall remain fixed to the steel structure on the other side of the wall and therefore not exposed to fire. However, due to the lack of corresponding scientific evidence, questions are being very often raised about the real efficiency of such systems in fire situation, which, in certain cases, have also to provide an adequate seismic resistance, if they are used in seismic areas.

Today, concrete or masonry wall solutions are frequently used for the compartmentation of buildings, predominately for low-rise commercial and industrial steel buildings. However, as an alternative, lightweight sandwich panels (comprising two thin flat metal faces and an insulated core) could become an appropriate steel fire wall solution, offering numerous benefits in comparison to other solutions, including fire resistance, durability, flexibility, easy dismounting and fast construction times. But, there is an evident lack of technical information about the adequate fire performance of such type of wall solutions when they are implemented in single-storey buildings with unprotected steel structure, which constitutes a major obstacle for their large use.

In this context, the overall goal of the FISWHALL project is to develop a design guidance and recommendations for an innovative hybrid fire wall solution based on lightweight steel-faced sandwich panels associated with unprotected steel structure under both fire and seismic actions, but considered individually. This will be achieved through the following specific tasks: i) Establishing of a full range of experimental evidence about the fire and seismic behaviour of the investigated hybrid fire wall solution by carrying out a number of tests; ii) Investigating intensively the fire and seismic performances of the above hybrid fire wall solution in combination with unprotected single-storey steel structures through a variety of parametric numerical studies by means of validated FE numerical models; iii) Developing both cost-effective and innovative "fusible" connection systems for fire walls to be used in combination with unprotected steel structures of single-storey buildings; and iv) Developing a design guidance and practical recommendations for the studied hybrid fire wall and fusible links solutions, on the basis of above studies, from which engineers can carry out very efficient design.

This deliverable summarises the main characteristics of single-storey steel buildings, with industrial or commercial use. Typical steel framed structures, connections, and bracing systems are detailed, based on current practice. Investigation focuses mainly on the most common structures, which are portal frames with plain rolled or welded sections. From collected information, four reference steel framed structures are defined, for single medium or large sized compartments, with single or multi bay frames. In addition, an overview of the main requirements prescribed in both national fire and seismic regulations in relation with industrial and commercial buildings, including warehouses is also presented. The following European countries were included in the survey: Czech Republic, Belgium, Finland, France, Germany, Hungary, Italy, Luxembourg, Netherlands, Poland, Portugal, Spain, Switzerland and UK (England + Wales).

1 INTRODUCTION

In the scope of the project FISHWALL, it is intended to perform numerical studies to investigate the global structural behaviour of typical single-storey steel-framed buildings in the event of real fires or earthquake, when these events are considered individually. For this purpose, a brief review of typical single-storey industrial and commercial steel buildings was first undertaken over the European market. Investigation focused mainly on portal frames with plain rolled or welded sections, which are the construction systems identified as the most widely used in Europe. Thus, chapter 2 summarises shortly the main characteristics of single-storey steel buildings, with industrial or commercial use. Typical steel framed structures, connections, and bracing systems are also detailed, based on current practice. From collected information, four reference steel framed structures intended to be used in the various numerical studies planned throughout the project were defined. The selected structures are presented in chapter 3. They correspond to four existing buildings in France, corresponding to buildings enclosing medium to large-sized fire compartments, with single or multi-bay steel frames. In addition, an overview of both fire and seismic regulations of many European countries applying to industrial and commercial buildings of all sizes, including warehouses was conducted. The following European countries (equitably distributed among the partners) were included in the survey: Czech Republic, Belgium, Finland, France, Germany, Hungary, Italy, Luxembourg, Netherlands, Poland, Portugal, Spain, Switzerland and UK (England + Wales). Regarding the fire regulations, due consideration was given to requirements dealing with the structural fire resistance, the overall structural behaviour, the compartmentation and the smoke extraction asked for such type of building, which are summarised in chapter 4. The main purpose was to determine the most commonly fire resistance rating asked for partition fire walls, the permitted maximum sizes of fire compartments according to the building occupancy and the associated smoke and heat exhaust measures. Chapter 5 reports on provisions given in the reviewed European countries for the earthquake resistant design of single-storey buildings. The review mainly focused on the methods applicable in determining the design seismic actions, with specific reference to the provisions of EN 1998-1 [47]. In case of fire, the amount of releasable energy depends on the mass of combustible materials. The fire duration depends on this fire load and on the heat release rate. This is related to the composition of the fire load and its exposed surface, and to the fresh air supply. Fire load density, which is directly linked to fire load, is a dimensioning parameter and many numerical models use it. It was thus necessary to have reliable statistical data. Therefore, a short literature review of existing fire load density surveys in target buildings was conducted. The survey results are summarised in chapter 6.

The data thus collected will allow selecting the main key input parameters to take into account in preliminary numerical analyses (real fire development, seismic analyses) planned in WP1 on the selected reference buildings.

2 TYPICAL STEEL STRUCTURE FOR SINGLE STOREY BUILDINGS

2.1 Overview

Single storey steel buildings are generally built with an external cladding envelope, supported on relatively short span secondary steel members, which are in turn supported on the primary steel structure [1]. The most common primary structures used for industrial or commercial buildings are portal frames. They are very efficient for enclosing large volumes, which requires spaces that have internal columns reduced to a minimum. As shown in Figure 1, a portal frame building comprises a series of transverse frames braced longitudinally. The primary steelwork consists of columns and rafters, which form portal frames, and bracing. The end frame (gable frame) can be either a portal frame or a braced arrangement of columns and rafters. The secondary steelwork consists of side rails for walls and purlins for the roof. This secondary structure supports the building envelope, but also plays an important role in restraining the primary structure. The roof and wall cladding separate the enclosed space from the external environment as well as providing thermal and acoustic insulation. The structural role of the cladding is to transfer loads to the secondary structure and also to restrain the flange of the purlin or rail to which it is attached.

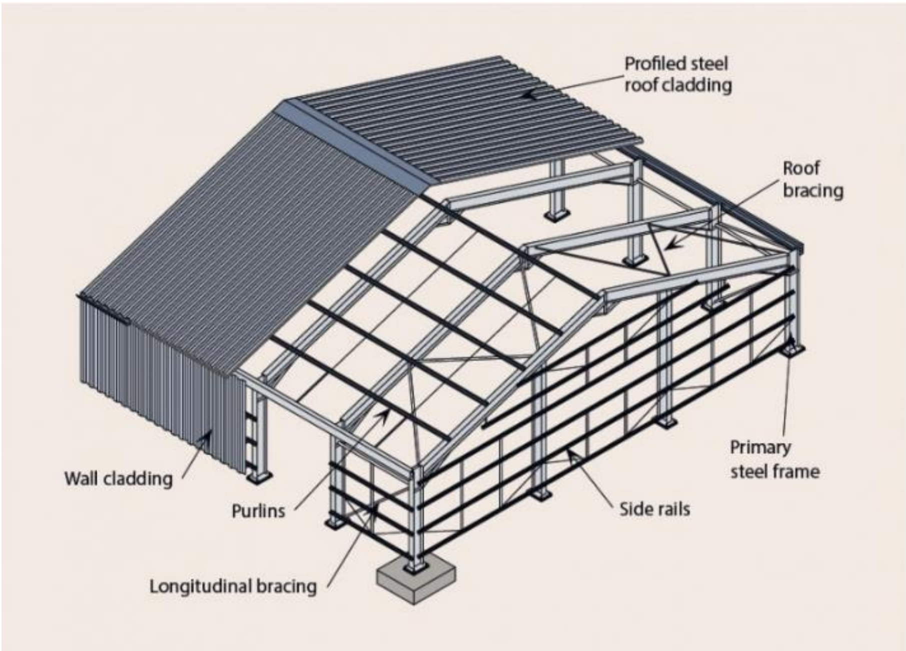


Figure 1: Principal components of a steel structure for single-storey buildings [3]

2.2 Primary steel structure

Typical primary steel structures for single-storey buildings are either:

- Rigid framed structures, or
- Pinned frame beam-and-column structures.

Rigid frames are achieved by providing a rigid (moment resisting) connection between the ends of the roof beams and the columns. The stiff frame that is created is much more efficient in carrying the imposed loads on the roof than a simply supported roof member (with pinned connections at its ends) and the frame also provides resistance against wind forces on the sides of the building. Because the frames are self-supporting in the plane of the frame, the bracing in the roof can be reduced, compared to a structure with simply supported roof beams [1].

Rigid framed structures fall into two categories, portal framed structures and truss framed structures.

2.2.1 Portal frames

Portal frames come in a variety of different shapes and sizes, with flat and pitched roofs. A portal frame may be single bay or multi bay, and has generally straight rafters. The members are either hot rolled sections or welded sections (built up sections consisting of two flange plates, welded to a web plate to form an I-section).

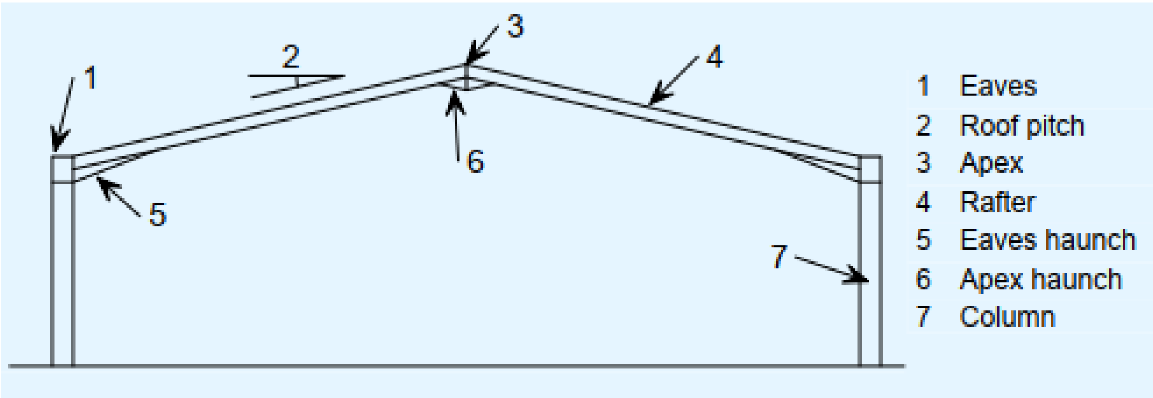


Figure 2: Single-span symmetric portal frame [1]

Figure 3: Portal frames with hot rolled sections

Welded sections are used where standard rolled sections are inadequate in terms of load carrying capacity or stiffness. Typical uses include long-span floors in buildings and crane girders in industrial buildings.

Two types of welded sections may be found:

- Members with a constant depth, but with a varying flange thickness if necessary;
- Tapered sections, generally with constant flange dimensions and a linearly varying web depth along the length of the member.

Welded sections are designed to resist the applied actions using proportions that ensure low self-weight and high load resistance. For efficient design it is common to use a relatively deep girder to minimise flange area for a given applied moment. A deep girder also provides a deep web whose area may be minimised by reducing its thickness to the minimum required to carry the applied shear. Such a deep web may be quite slender (a high web depth to thickness ratio) and may be susceptible to shear and local buckling. It is therefore relatively common to provide transverse or longitudinal stiffeners.

Figure 4: Portal frames with welded sections

In-plane stability of portal frames is provided by frame continuity and the rigid connection between rafters and columns. Connections are usually bolted, with the resistance of the rafter enhanced locally with a haunch (for hot rolled sections) or by increasing the section dimensions at the joint (for welded sections).

Stability in the longitudinal direction is provided by a combination of bracing in the roof, across one or both end bays, and vertical bracing in the elevations.

Column bases are either fixed or pinned. In many cases, the frame will have pinned bases in order to minimize foundations and base connections. In practice, fixed columns are mainly used under high horizontal forces, as they will deflect less.

The frames located at the end of buildings are generally called gable frames. Gable frames may be:

- simple frame structures with column and simply supported beams, together with vertical bracing or rigid panel for the lateral stability;
- or may be identical to the internal frames, even though they experience lighter loads. If future extension to the building is envisaged, portal frames are commonly used as the gable frames, to reduce the impact of the structural works.

A typical gable frame is shown in Figure 5.

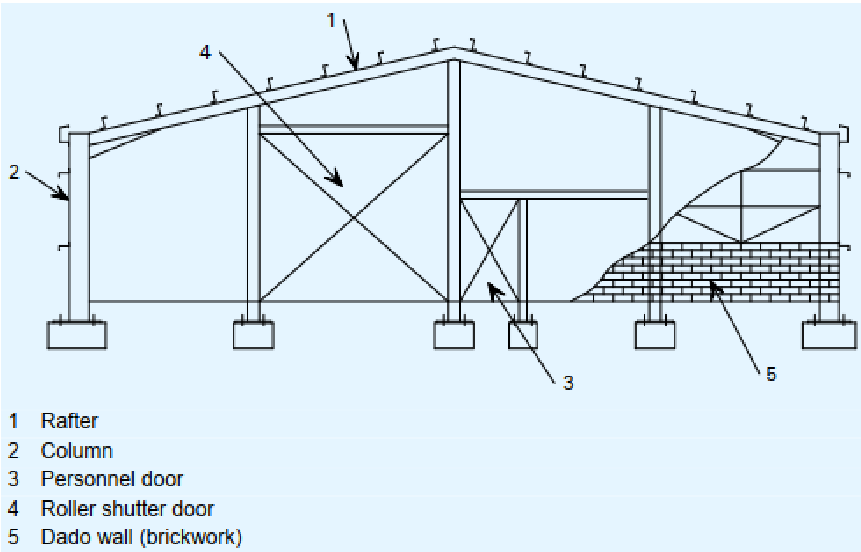


Figure 5: Typical details of an end gable of a portal frame building [1]

Some industrial buildings require overhead cranes. Most overhead cranes use single or twin beams spanning across the building and with a hoist mounted on the beams. The crane beams are supported on runway beams that run the length of the building.

Incorporating an overhead crane in a building always influences the design of the building structure, even when the hoisting capacity is very modest. Crane use also results in horizontal forces from movement of the loads, so additional bracing is usually provided.

A crane with a lifting capacity up to a safe working load of about 20 tons can usually be carried on runway beams that are supported off the columns that support the roof. For larger cranes, it is more economical to use separate columns (or vertical trusses) to support the runway beams and avoid excessive loads on the building structure.

Figure 6: Overhead crane

2.2.1.1 Portal frame with hot rolled sections

A single-span symmetrical portal frame is typically of the following proportions [1]:

- A span up to 40 m (25 m to 35 m is the most efficient).
- An eaves height (base to rafter centreline) of between 5 and 10 m. The eaves height is determined by the specified clear height between the top of the floor and the underside of the haunch.
- A roof pitch between 3° and 10°.
- A frame spacing between 5 m and 10 m.
- Depth of rafters ranges from 240 mm to 600 mm; for the columns, the depth is generally between 300 and 600 mm, although values between 800 and 900 mm may be used.
- Haunches are provided in the rafters at the eaves to enhance the bending resistance of the rafter and to facilitate a bolted connection to the column. The eaves haunch is typically cut from the same size rolled section as the rafter, or one slightly larger, and is welded to the underside of the rafter. The length of the eaves haunch is generally 10% of the span.
- Small haunches are provided at the apex, to facilitate the bolted connection.

2.2.1.2 Portal frames with welded sections

A single-span symmetrical portal frame is typically of the following proportions:

- A span L up to 60 m.
- An eaves height H (base to rafter centreline) of between 5 and 10 m.
- A roof pitch between 3° and 10°.
- A frame spacing between 5 m and 10 m.
- General rules of thumb for the dimensions are:
    - Depth of the rafters: L/30
    - Depth of the columns: H/10
    - Width of the flanges: between h/5 and h/2 (h is the depth of the member)
    - Thickness of the web: between h/160 and h/100.

2.2.2 Lattice trusses

For large spans, roof trusses provide an effective and economic alternative. Typical truss shapes are shown in Figure 8, and a framed truss is illustrated in Figure 7.

A lattice truss is usually composed of a roof truss and steel columns with hot rolled sections or welded sections. The truss is composed of a top and bottom chords, with a triangular system of internal members, alternatively in tension or compression.

Figure 7: Frame truss

Trusses are used in a broad range of buildings, mainly where there is a requirement for very long spans. Many configurations of lattice trusses exist, depending on the magnitude of the internal forces, ease of connections between members, aesthetics, etc…:

- N-truss or W-truss are the most common form (Figure 8);
- With straight or either a single or a double slope for the upper chord;
- For smaller spans, trusses are composed of small internal members:
    - Open sections, mainly angles, U, I and tee sections;
    - Built-up sections, such as double angles;
    - Hollow sections.
- The inclination of the diagonal members in relation to the chords are between 35° and 55°;
- The internal members may be bolted or welded;
- The connections between members of the truss and the column may be fixed or pinned.

A single-span truss frame is typically of the following proportions:

- A span L up to 100 m.
- A ratio of span to truss depth in the range 10 to 15.
- An eaves height inferior to L/12 for flat trusses, and in the range L/5 to L/6 for pitched trusses.
- A roof pitch between 3% and 10%.
- A frame spacing between 6 m and 10 m.

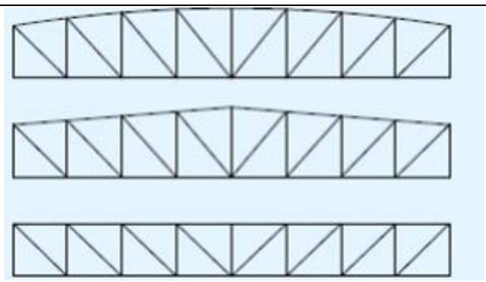
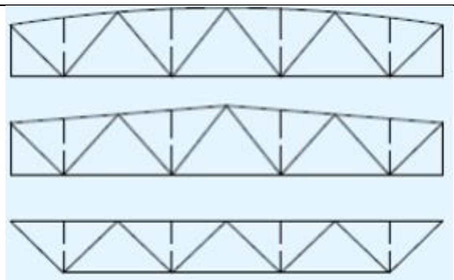
	
(a) N-truss	(b) W-truss

Figure 8: Typical truss shapes for single-storey buildings

It should be mentioned that for the same steel weight, it is possible to get better performance in terms of resistance and stiffness, with a truss than with an I beam. This difference is greater for long spans and/or heavy loads.

Truss buildings generally have roof bracing and vertical bracing in each elevation to provide stability in both orthogonal directions. For single-storey buildings, there are two types of general arrangement of the structure [2], shown in Figure 9:

- In the first case (a), the lateral stability of the structure is provided by a series of portal trusses; the connections between the truss and the columns provide resistance to a global bending moment. Loads are applied to the portal structure by purlins and side rails.
- In the second case (b), each truss and the two columns between which it spans, constitute a simple structure; the connection between the truss and a column does not resist the global bending moment, and the two column bases are pinned. Bracing in both directions is necessary at the top level of the simple structure; it is achieved by means of a longitudinal wind girder which carries the transverse forces due to wind on the side walls to the vertical bracing in the gable walls. Longitudinal stability is also provided by a wind girder in the roof and vertical bracing in the elevations.

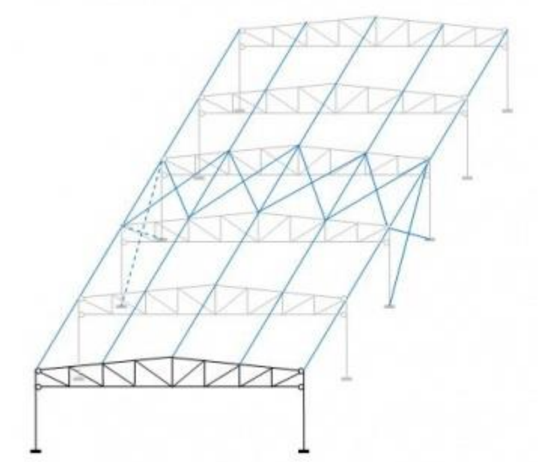
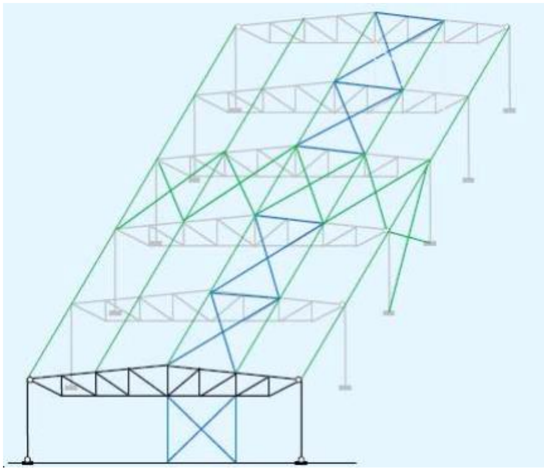
	
(a) Lateral stability provided by portal trusses	(b) Building braced in both directions

Figure 9: Typical truss building arrangements [2]

2.2.3 Bracing

Bracing is required to resist longitudinal actions due to wind and cranes, and to provide restraint to members.

For single-storey buildings, bracing is generally located in the plane of the roof (“plan bracing”) and vertically in the plane of the side walls (“vertical bracing”).

The primary functions of the plan bracing are:

- To transmit wind forces to the vertical bracing in the walls.
- To provide a stiff anchorage for the purlins which are used to restrain the rafters.

The diagonal steel members are double angles or circular hollow sections fixed to the portal frames.

The vertical bracing is often provided at both ends of the building, or in one bay only. The primary functions of the vertical bracing are:

- To transmit the horizontal loads to the ground.
- To provide a rigid framework to which side rails and cladding may be attached so that the rails can in turn provide stability to the columns.

Vertical bracing can be found in the following forms:

- Truss systems with diagonal steel members: cross bracing is the most common form, but K-bracing and V-bracing can be found.
- Rigid frame: when it is difficult or impossible to brace the frame vertically by conventional bracing, it is necessary to introduce moment-resisting frames in the elevations in one or more bays.
- Rigid walls.

(a) Plan bracing

(b) Rigid frame

(c) Cross bracing

Figure 10: Bracing systems for single-storey buildings

2.2.4 Connections

There are two types of connection:

- Welded connections, which are mainly used for link elements (gussets, plates) or welded sections;
- Bolted connections, which offer more simplicity for on-site assembly, and are less expensive to manufacture. Two types of bolt are commonly used:
    - Ordinary bolts: these bolts are used for connections where no preloading is to be applied (mainly secondary elements), and rely on the shear resistance of the bolt and the bearing resistance of the connected plates to establish the load path.
    - HR bolts: this type of bolt is tightened to a defined pre-load to clamp the connecting plates together to ensure that the load transfer occurs through the development of friction between contact surfaces. These bolts are mainly used to connect elements subjected to bending moments and shear forces.

2.2.4.1 Connections for portal frames

In a portal frame structure, the connections between beams and columns transfer bending moments, as well as shear and axial forces, and they must be designed as rigid connections [1].

A rigid connection typically has a full depth end plate. The roof beam is often haunched locally and the column web is stiffened in order to resist the local forces from the end of the roof beam.

The three major connections in a single bay portal frame are those at the eaves, the apex and the column base.

A typical eaves connection is shown in Figure 11(a). In almost all cases a compression stiffener in the column (as shown, at the bottom of the haunch) will be required. Other stiffeners may be required to increase the bending resistance of the column flange, adjacent to the tension bolts, and to increase the shear resistance of the column web panel. The haunch is generally fabricated from a similar size beam to the rafter (or larger), or fabricated from equivalent plate.

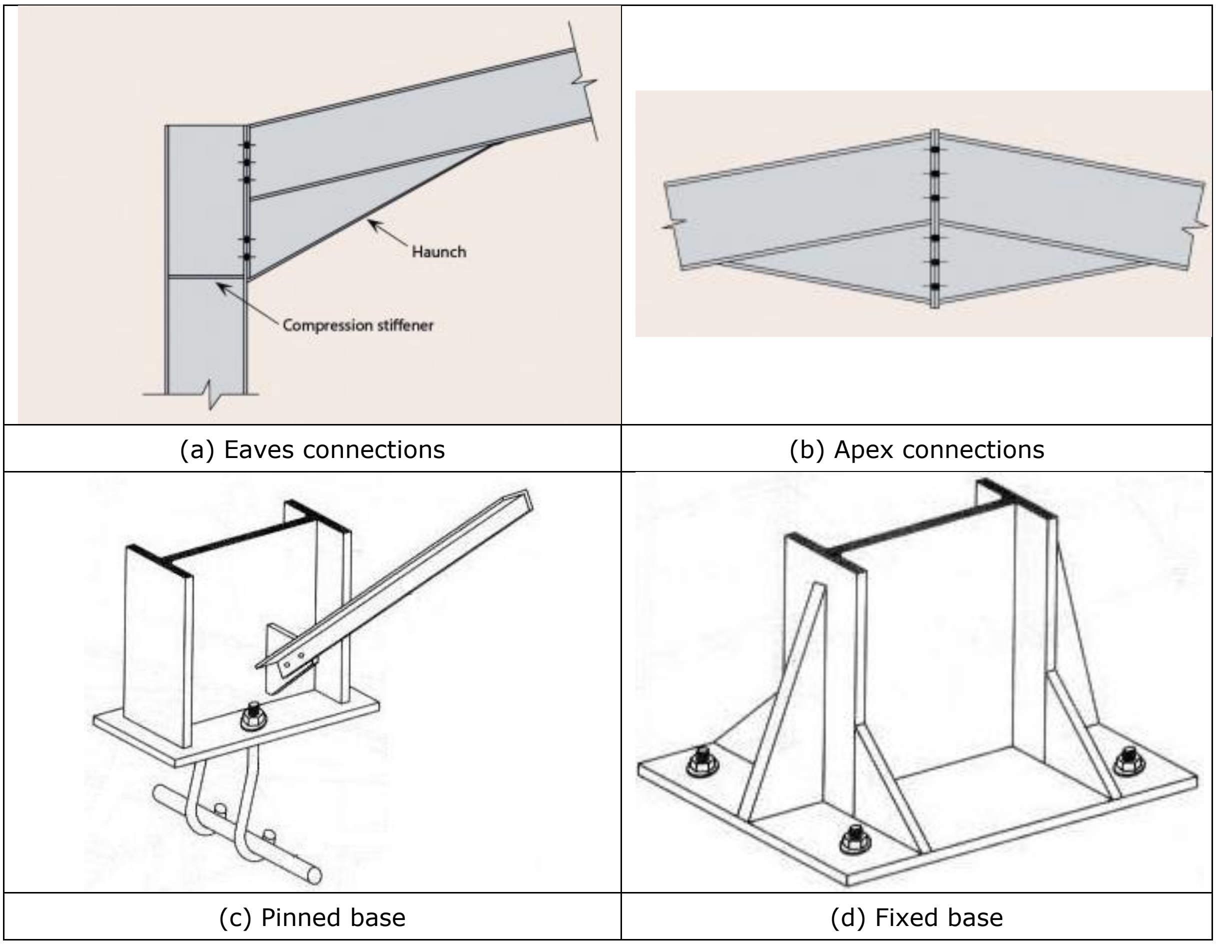
A typical apex connection is shown in Figure 11(b). The apex connection is primarily used to increase the depth of the member at the point of maximum bending moment, in order to increase the bending resistance. The apex haunch is usually fabricated from the same member as the rafter, or from equivalent plate.

The base of the column is often kept simple with larger tolerances in order to facilitate the interface between the concrete and steel-work. Pinned connections are often preferred in order to minimize foundation sizes, even if in some cases it will require much more steel than fixed base connections. When high horizontal forces are applied to the column, fixed based connections may be required.

A typical pinned base is shown in Figure 11(c). A plate is welded to the end of the column and attached to the foundation with holding down bolts. The plate is generally at least as thick as the flange of the column. Most authorities accept that even with four holding down bolts, the base is still pinned. Alternatively, the base may have only two holding down bolts, on the axis of the column, but this may make the erection of the steelwork more difficult.

For a typical fixed base connection, it will be necessary to use a thicker base plate, or additional stiffener plates as shown in Figure 11(d).

Holding down bolts are generally embedded in the foundation, with some freedom of lateral movement (tubes or cones) so that the steelwork can be aligned precisely. The holes in the base plate are usually 6 mm larger than the bolt diameter, to facilitate some lateral alignment [1].



(a) Eaves connections

(b) Apex connections

(c) Pinned base

(d) Fixed base

Figure 11: Typical connections in a portal frame [2]

(a) Eaves connection

(b) Apex connection

(c) Pinned base

(d) Fixed base

Figure 12: Typical connections in a portal frame (photos)

2.2.4.2 Connections for lattice trusses

Truss connections are either bolted or welded to the chord members, either directly to the chord, or via gusset plates.

Many configurations can be found, depending on the type of section to be assembled:

- Double angle members as chords, and single (or double) angles as internal members. The connections are generally made with a gusset plate welded or bolted between the angles forming the chords.
- Rolled sections as chords, with the web in the plane of the truss. The internal members are usually angle members, connected via a gusset plate welded to the flange of the chord.
- Rolled sections as chords, with the web perpendicular to the plane of the truss, and single (or double) angles as internal members. The connections to the chord members are usually via gusset plates welded to the web.
- Rolled sections for both chords and internal members, with the web perpendicular to the plane of the truss. A simple, effective design is to choose chords and internal members that have the same overall depth, and connect them with a gusset plate, welded or bolted to the outside of both flanges.

<table>
<tr><td></td><td></td></tr>
<tr><td>(a) Bolted connection to the gusset</td><td>(b) Welded connection to the chord</td></tr>
</table>

Figure 13: Typical connections for lattice trusses

2.2.4.3 Bracing connections

A triangulated bracing (truss system) is the most economical and efficient way to provide stability to a steel structure. Besides, being installed at the beginning of the structure erection, it offers the advantage of not requiring temporary bracing. Typically, bracing systems span between columns or rafters, creating lattice girders.

Either single diagonals are provided, in which case they must be designed for either tension or compression (and eventually subjected to buckling), or crossed diagonals are provided, in which case slender bracing members carrying only tension may be provided. When crossed diagonals are used, it is assumed that only the tensile diagonals provide resistance.

In most cases, the bracing member is attached by bolting to a gusset plate (except for tubular elements, which are generally welded), which is itself welded to the beam, to the column, or more commonly welded to the beam and its end connection.

The table below summarizes the different types of connections that could be found with triangulated bracing systems.

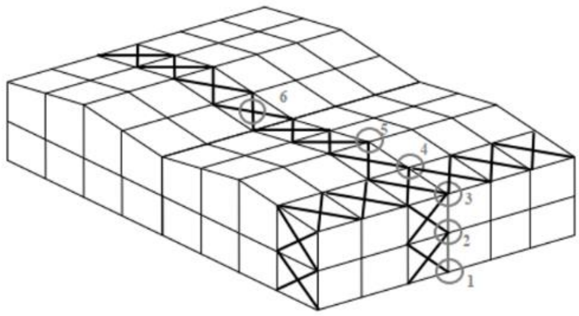
<table>
<tr><td>Node 1</td><td rowspan="3">Vertical bracing</td><td>Base column</td><td rowspan="6"></td></tr>
<tr><td>Node 2</td><td>Column + Beam</td></tr>
<tr><td>Node 3</td><td>Column + Rafter + Eave purlin</td></tr>
<tr><td>Node 4</td><td rowspan="3">Plan bracing</td><td>Rafter + Purlin</td></tr>
<tr><td>Node 5</td><td>Apex</td></tr>
<tr><td>Node 6</td><td>Cross diagonal</td></tr>
</table>

Table 1: Location of connection nodes

Figure 14 shows a bolted connection to the base of a column (node 1 in Table 2.1). A common practice is to minimize the eccentricity between the bracing member and the column axis. The gusset plate is welded to the column web and to the base plate.

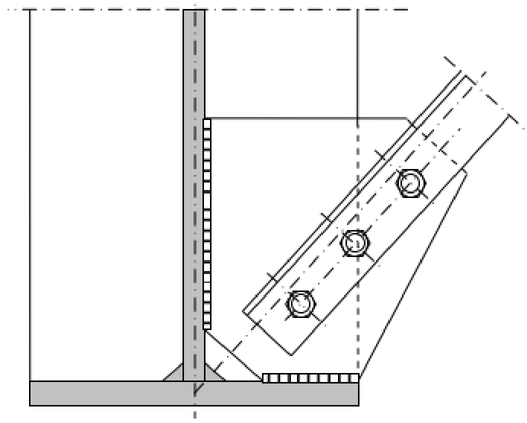
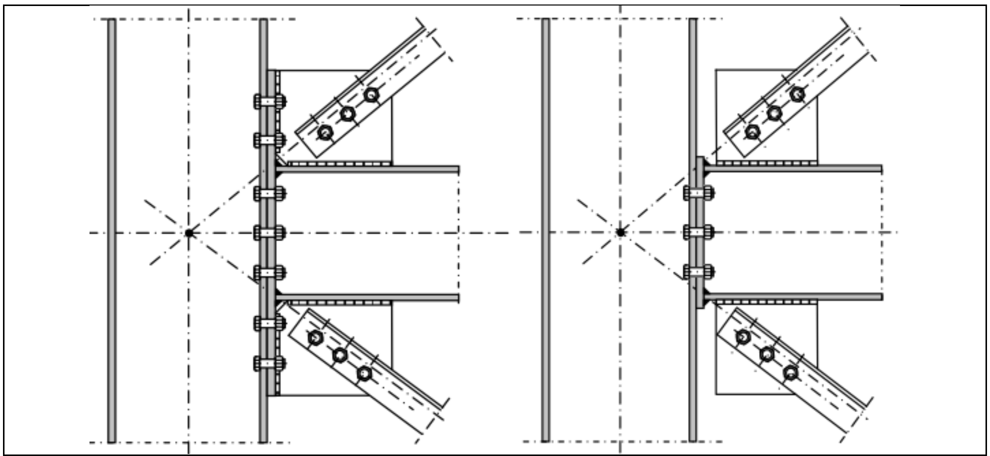


Figure 14: Bolted connection to the base

Figure 15 shows typical vertical bracing connections (node 2 in Table 2.1):



(a) Rigid connection	(b) Pinned connection

Figure 15: Vertical bracing connection

Figure 16 illustrates a typical plan bracing connection (node 4 in Table 2.1). The gusset is bolted to the flange of the rafter, preferably under the roof purlin.

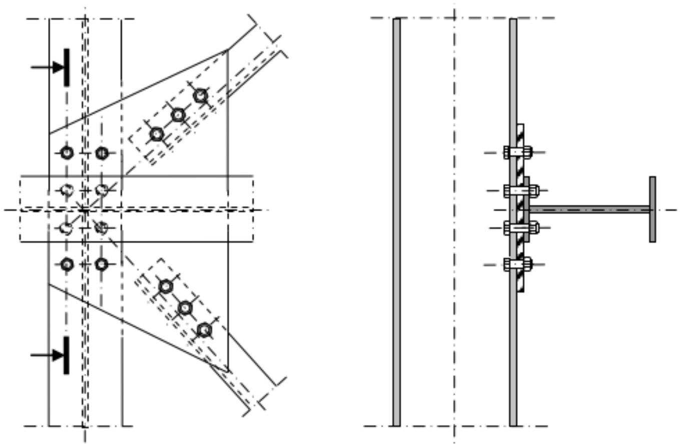


Figure 16: Plan bracing connection

Figure 17 shows a typical cross bracing connection (node 6 in Table 2.1), with only one continuous diagonal.

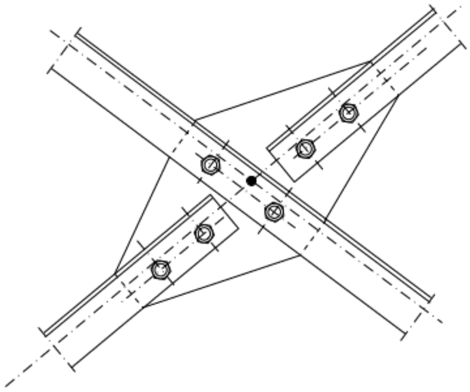


Figure 17: Cross bracing connection

2.3 Secondary steel structure

The secondary steelwork of a single-storey building consists of side rails for walls and purlins for the roof, spanning between the columns and rafters respectively. The secondary steelwork supports the building envelope, but also plays an important role in restraining the primary steelwork.

(a) Purlins	(b) Side rails

Figure 18: Secondary steel structure

2.3.1 Purlins

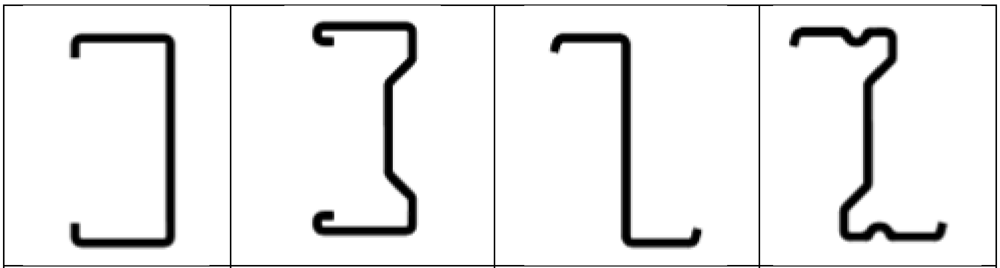
The main functions of the purlins are to:

- support the roof cladding;
- transfer the forces from the roof to the primary structural elements, i.e. rafters or lattice trusses;
- provide limited restraint against lateral torsional buckling of the rafters.
- in some cases, act as compression members as part of the bracing system.

For these secondary members, there is a choice between thin cold-formed sections and hot-rolled sections (usually I-section). The profiles of typical thin cold-formed sections are shown in Figure 19.

Purlins are typically of the following proportions:

- I-sections range from IPE 80 to IPE 240;
- For cold-formed sections, the depth of the section typically lies between 120 mm and 350 mm, with the profile thickness varying between 1,5 mm and 4 mm.
- They are used at relatively low spacing, typically between 1,5 m and 2,5 m.
- Span of hot rolled sections ranges from 5 and 7,5 m, whereas cold-formed sections can reach a span of 12 to 15 (thus resulting in fewer portal frames).



(a) C profile	(b) Sigma profile	(d) Z profile	(d) Zeta profile

Figure 19: Typical cross sections of thin cold-formed beams

The installation of the purlins starts at the eaves and progress up the slope to the apex.

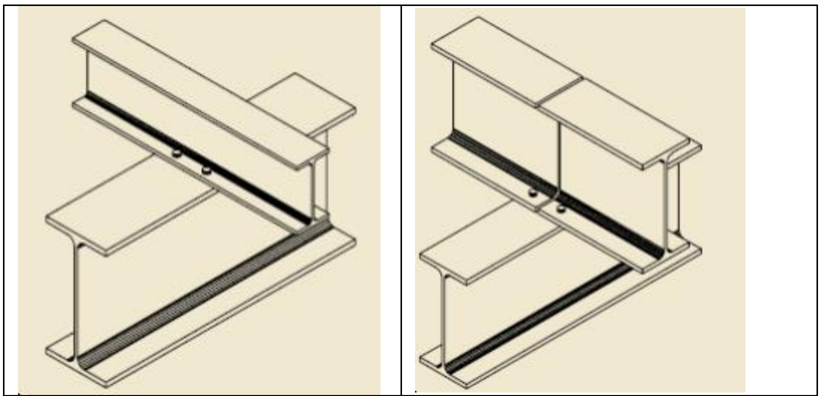
Connections between the purlins and the primary structure can be:

- A bolted connection between the upper flange of the rafter and the lower flange of the purlin (typically used for hot rolled purlins, Figure 20);
- Using a cleat, that is bolted to the purlins and either bolted or welded to the rafters (typically used for cold-formed purlins, Figure 21).

Besides, sleeves can be used to achieve a continuity of purlins, as shown in Figure 21(b). These sleeves are sheet thicker than the purlins because the moment acting on support is then maximum.

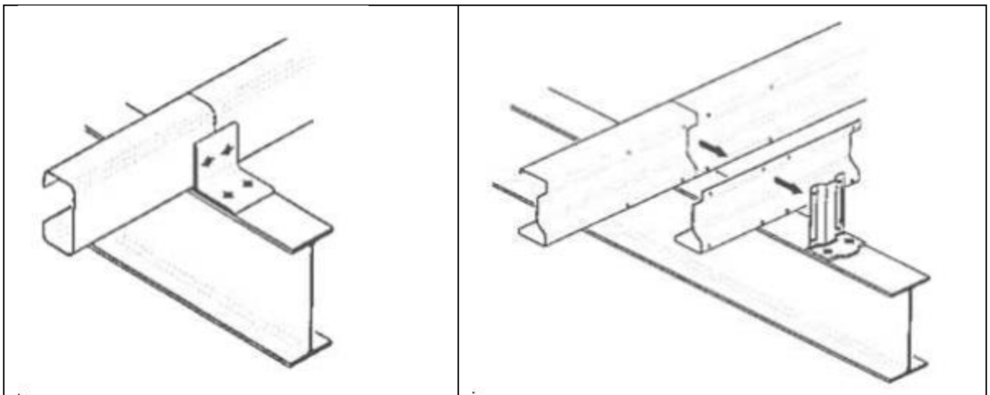
For Z profiles, the purlins can also be overlapped at the supports to have continuity.

Finally, eave purlins are directly connected to the columns. They are bolted to a plate and are located either between columns, or above columns. In the latter case, eave purlins are usually made continuous with a sleeve.



(a) Continuous purlin	(b) Single span purlin

Figure 20: Connections for hot rolled purlins [4]



(a) Single span purlin: bolted connection with a cleat	(b) Continuous purlin: bolted connection with a cleat and sleeve

Figure 21: Connections for thin cold-formed purlins

Figure 22: Typical connections for cold-form sections

2.3.2 Side rails

The main functions of the side rails are to:

- support the wall cladding;
- transfer the forces from the wall to the primary structural elements;
- provide limited restraint against lateral torsional buckling of the columns;
- in some cases, act as compression members as part of the bracing system.

In short, the side rails are at the walls what the purlins are at the roof of a building.

As for purlins, side rails can be thin cold-formed sections or hot-rolled sections. I-sections typically range from IPE 100 to IPE 180, and thin cold-formed sections are about the same depth.

The spacing between side rails depends on the frame spacing, wind load, and type of wall cladding. Generally, the spacing is about 2 m.

2.3.3 Restraint elements

Restraint elements are necessary to provide stability to the rafters and columns. The purlins and side rails are considered adequate to restrain the flange that they are attached to, but unless special measures are taken, the purlins and side rails do not restrain the inside flange. Restraint to the inside flange is commonly provided by bracing from the purlins and side rails, as shown in Figure 23. The bracing is usually formed of thin metal straps, designed to act in tension, or from angles designed in compression if bracing is only possible from one side [2].

	
(a) Stabilizing the inside flange of a column	(b) Stabilizing the bottom flange of a roof beam [1]

Figure 23: Restraint elements

In a same way, anti-sag bars can be used to provide lateral restraint to the purlins, especially for cold-formed sections (Figure 24). They are generally spaced every 3 m.

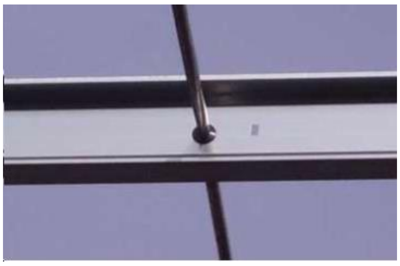


Figure 24: Anti-sag bars for the purlins

2.4 Building envelope

The roof and wall cladding are generally referred to as the building envelope.

There are two types of envelope: non structural envelope (CE marking 14782) for class III of the EN 1993-1-3 and structural envelope (CE marking EN 1090-1) for classes I and II of the EN 1993-1-3.

It has some or all of the following functions:

- Separating the enclosed space from the external environment;
- Transferring load to the secondary steelwork (class III);
- Restraining the secondary steelwork (class II);
- Providing thermal insulation;
- Providing acoustic insulation;
- Providing an airtight envelope;
- Providing ventilation.

The choice of wall and roof cladding can have a significant impact on the appearance of a building. The profile shape can have a significant impact on the appearance of a building due to its effect on the perceived colour and texture of the cladding (caused by the reflection of light). The orientation of the cladding (ribs horizontal or ribs vertical) will also influence the appearance of the building, due to the effects of shadow and reflection [3].

The principal options for cladding systems are:

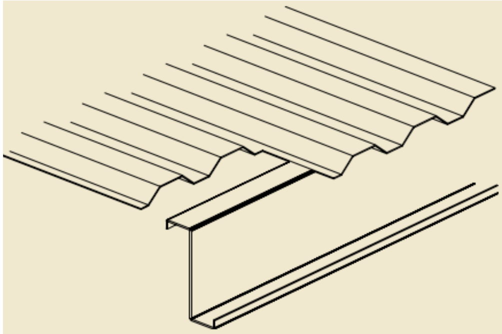
- Profiled steel sheeting:
    - Single-skin
    - Double-skin, built up on site from a liner panel, insulation and an outer sheet
    - Composite panels (also known as sandwich panels), pre-fabricated off site from an inner sheet, and outer sheet and insulation.
- Steel sheeting with insulation, covered by a waterproof membrane – commonly used on flat roofs.
- Wooden panels/decking
- Precast concrete slabs
- Blockwork

The steel sheet is generally coated with a zinc or zinc-aluminium alloy in a hot-dip process. The top coating is an organic coating to provide an attractive finish. Generally steel sheeting is made of galvanised steel grades S 280 GD, S 320 GD, S 350 GD or S 275 GD.

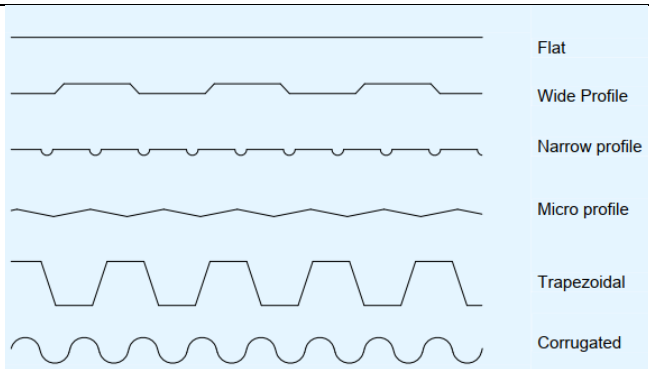
These different types of cladding system are presented in the following paragraphs.

2.4.1 Single-skin sheeting

Single-skin sheeting is widely used in industrial structures where no insulation is required, being the most economical solution. The sheeting is fixed directly to the purlins and side rails as shown in Figure 25(a), fastened with screws or bolts. The steel sheets are usually between 0,63 and 1,5 mm thick (including galvanisation), with a depth varying from 32 mm to 35 mm. Various profiles exist, as shown in Figure 25(b). The current span is 2 to 3m.



(a) Single-skin sheeting [4]



(b) Typical cladding profiles [1]

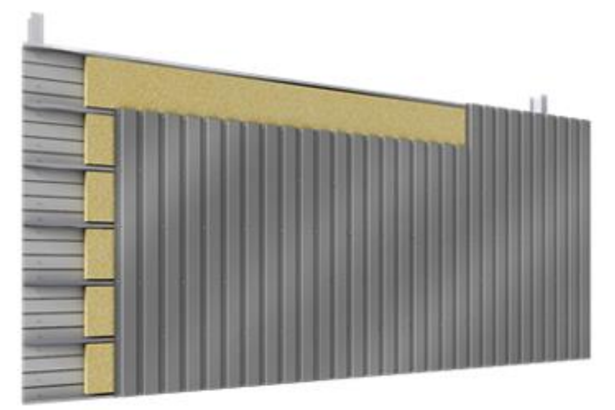
Figure 25: Single-skin cladding systems

2.4.2 Double-skin sheeting

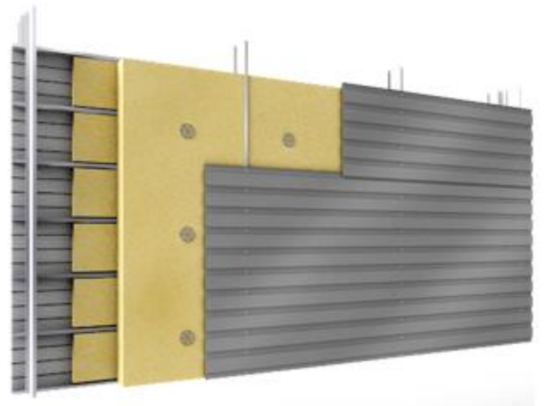
This common type of cladding consists of a metal liner tray, a layer of insulation material, a spacer system and an outer metal sheet, as illustrated in Figure 26.

The most common form of insulation in built-up cladding systems is mineral wool, which is favoured due to its light weight, low thermal conductivity, ease of handling and relatively low cost. The thickness ranges from 100 to 300 mm.

The span of such systems is limited by the spanning capability of the cladding sheets, which is typically in the order for the liner tray of 6 m to 8,5 m depending on the applied loading and for cladding sheet of 1 to 3 m. Built-up cladding systems must, therefore, be supported by secondary steelwork (purlins or side rails). As the name suggests, these systems are built up from their constituent parts on site.



(a) Vertical layout



(b) Horizontal layout

Figure 26: Double-skin cladding systems

2.4.3 Sandwich panels

Insulated panels consist of a rigid layer of insulation sandwiched between two metal skins, as shown in Figure 27(a). The result is a strong, stiff, lightweight panel with good spanning capabilities due to composite action in bending. These panels are commonly used on industrial buildings and retail 'sheds' in place of the built-up cladding described previously. In this case, the panels span between cold formed purlins or side rails. However, for commercial buildings, where the secondary steelwork is not needed for restraint purposes, it is quite common for composite wall cladding panels to span directly between the columns.

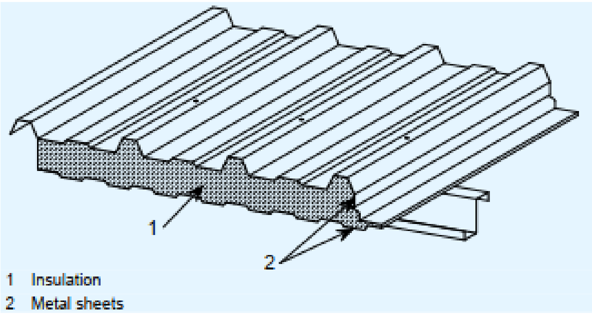
Unlike built-up systems, there is no need for a spacer system, as the rigid insulation is strong and stiff enough to maintain the correct spacing of the sheets.

Sandwich elements for roofs generally have a current width of 1000 mm with thicknesses between 40 and 150 mm 300 mm if cold store application according to DTU 45 in France), depending on the required insulation level and structural demands.

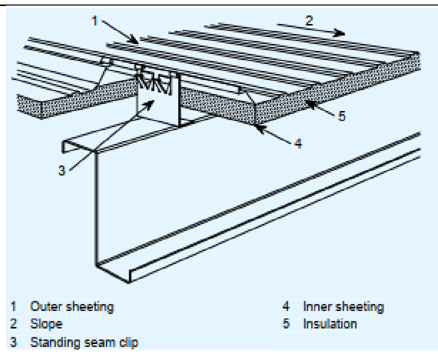
2.4.4 Standing seam systems

Standing seam systems use a specially designed profile for the weather sheet, which incorporates a clipped joint between adjacent sheets. This eliminates the need for exposed fasteners and improves the weather tightness of the cladding system. Insulated panel systems are also available with a standing seam joint in the weather sheet. A typical standing seam system is shown in Figure 27(b).

The disadvantage of this system is that significantly less restraint is provided to the purlins than with a conventionally fixed system. Nevertheless, a correctly fixed liner will provide adequate restraint.



(a) Sandwich panels



(b) Composite panels

Figure 27: Roof systems [3]

2.4.5 Fastening elements

Fastening techniques include the connections of the sheets to the supporting structure (primary fixing) and the connections between sheets (secondary fixing).

Fasteners are either stainless or protected with an anti-corrosion coating.

Exposed fasteners are supplied with a sealing element or washer. Sealing washers have generally a minimum diameter of 13 mm for steel sheets and 16 mm for aluminium sheets.

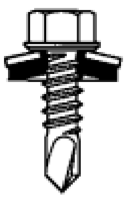
Most fasteners used for metal cladding applications are either self-tapping or self-drilling.

Self-drilling fasteners require no pre-drill operation and are therefore often preferred. The maximum drilling capacity of self-drilling fasteners is between 1,5 mm and 15 mm, and the minimum thread diameter is typically 5,5 mm (4 mm for secondary fixing).

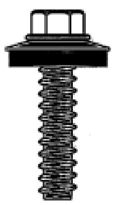
Self-tapping fasteners have no drill point and therefore a pre-drill operation is necessary. They typically have a thread diameter of 6,3 mm (4 mm for secondary fixing). They are mainly used when the substrate thickness is below 1,5 mm or above 15 mm (this is more specifically the case with roof claddings).

Many other types of fastener are available for specific primary and secondary applications within the metal cladding market. These include:

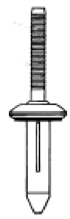
- Rivet type fasteners: these are most widely used for secondary fixing, typically for connection to thin materials such as side laps on profiled sheeting. As with self-tapping threaded fasteners, rivets need to be applied through correctly sized predrilled pilot-holes.
- Grommet type fasteners: when connecting profiled sheet side laps to a PVC roof material, a two part grommet type secondary fastener is usually recommended. A central threaded set pin is assembled into an elastomeric sleeve which has a nut encapsulated at its lower end.



(a) Self-drilling fastener



(b) Self-tapping fastener



(c) Rivet type fastener

(d) Grommet type fastener

Figure 28: Fastening elements for the building envelope

3 REFERENCE STEEL FRAMED STRUCTURES

3.1 Primary structure with hot rolled sections

3.1.1 Single bay portal frame (reference case n°1)

The first selected building has a floor area of about 1 400 m² (30 m x 46,3 m) as illustrated in Figure 29.

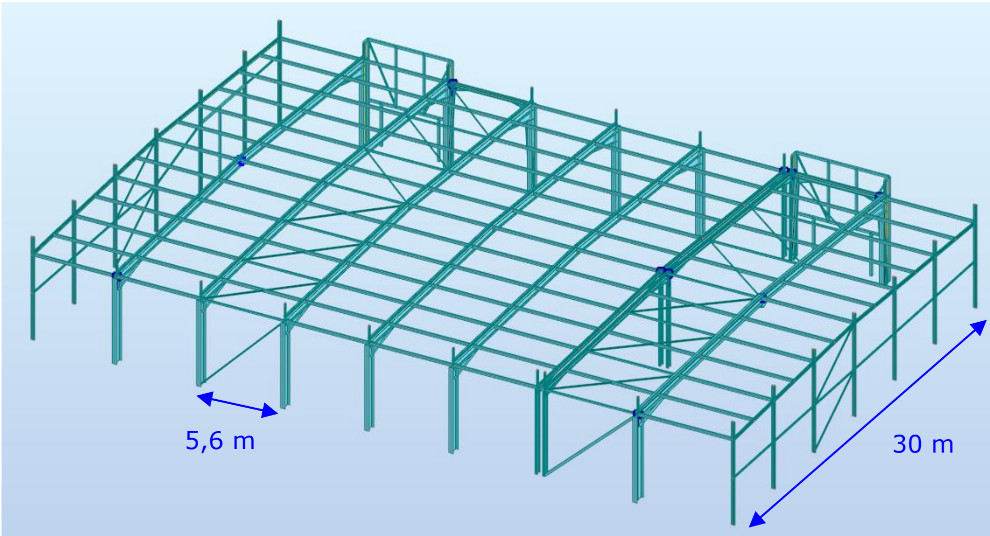


Figure 29: Perspective view of the reference case n°1 with floor area of 1 400 m²

The primary structure has the following characteristics:

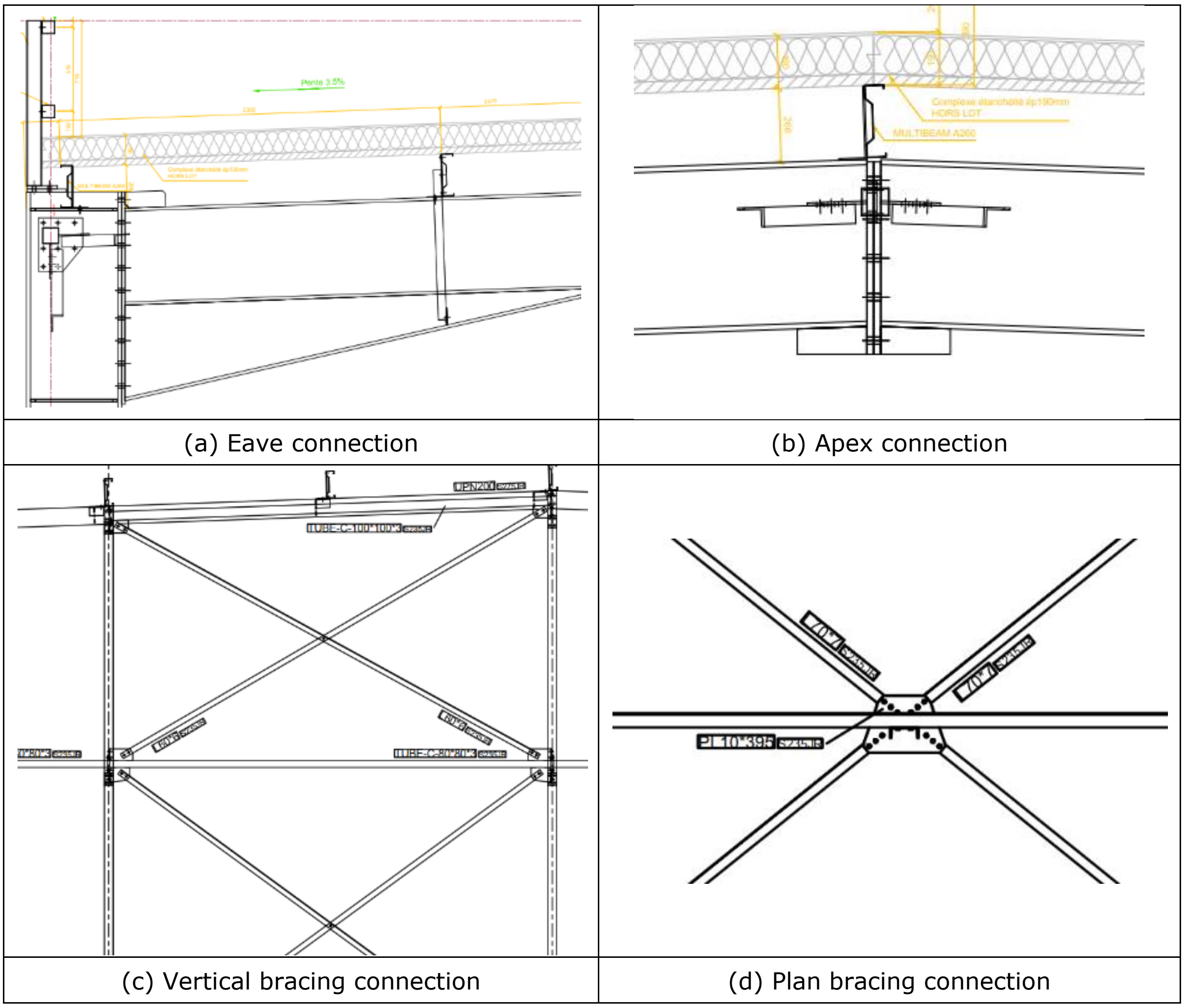
- Single bay portal frame
- Portal frame span: 30 m
- Frame spacing: 5,6 m
- Eave height: 5,7 m
- Roof pitch: 3,5 %
- Pinned base column
- Portal frames are composed of hot rolled sections:
  - HEA600 section for the columns and IPE600 section for the rafters (left part of the building)
  - HEA800 section for the columns and HEA550 section for the rafters (right part of the building, subjected to higher loads)
- The gable frame is composed of IPE200 sections for the gable posts, and cold-formed channel sections for the rafters
- Steel grade is S275.

Purlins are thin cold-formed sections (C profile), spaced every 2,5 m approximately.

Stability of the building is provided by cross bracing in the roof (L70x7 angles), and in the elevations by:

- vertical cross bracing (L70x7 angles) in one bay,
- diagonal bracing (square hollow section 140x4) in one bay,
- and rigid frames in two bays

Typical connections are detailed in the following figures:



(a) Eave connection

(b) Apex connection

(c) Vertical bracing connection

(d) Plan bracing connection

Figure 30: Typical connections of the reference structure n°1

The structure is designed with Eurocode rules, with the following loads (according to French National Annexes):

- Permanent loads:
  - Self-weight of structural elements
  - Roof panels: 35 kg/m²
  - Photovoltaic panels: 25 kg/m²
  - Other accessories: 25 kg/m² (left part of the building) and 35 kg/m² (right part)
- Snow load:
  - Zone D, altitude < 200 m
  - S = 72 kg/m²
- Wind load:
  - Zone 2, terrain category IIIb
  - basic wind velocity: 24 m/s
- Seismic load:
  - Seismic zone: 1 (weak)
  - Building importance class: II
  - Ground type: E

3.1.2 Two bay portal frame (reference case n°2)

The second building has a floor area of about 3 100 m² (61 m x 51 m) as illustrated in Figure 31.

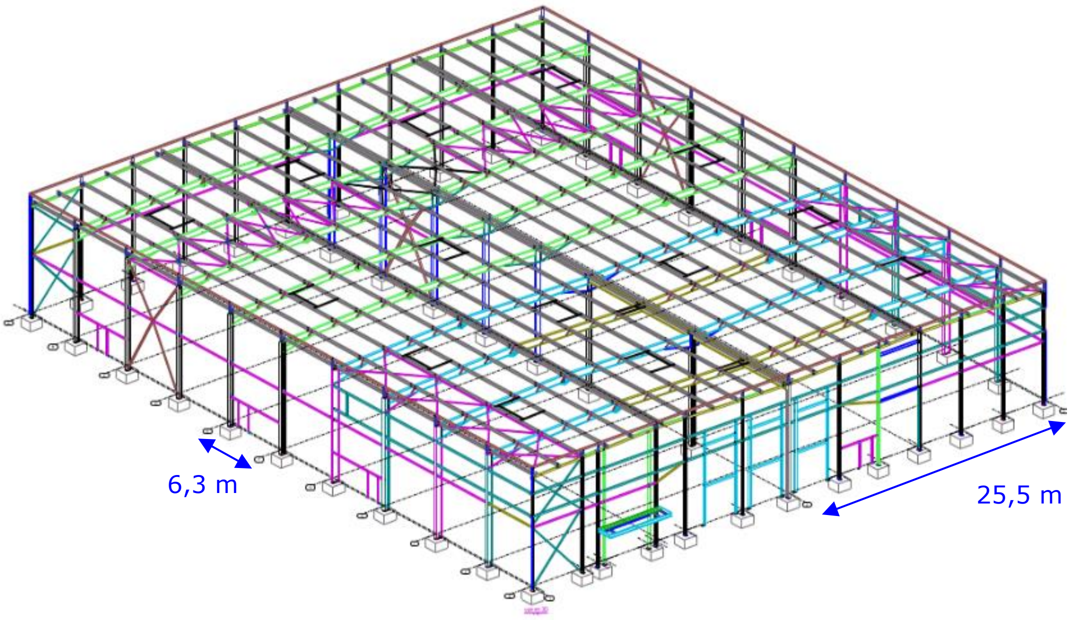


Figure 31: Perspective view of the reference case n°2 with floor area of 3 100 m²

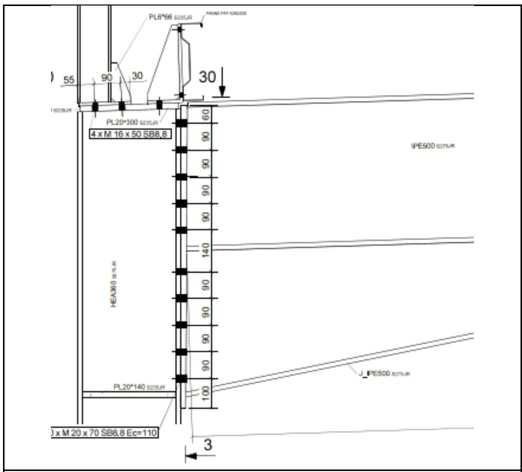
The primary structure has the following characteristics:

- 2 bay portal frame
- Portal frame span: 25,5 m
- Frame spacing: 6,3 m
- Eave height: 9,3 m
- Roof pitch: 3,1 %
- Pinned base column
- Portal frames are composed of hot rolled sections:
    - HEA360, HEA450 or IPE600 sections for the columns
    - IPE500 or IPE600 sections for the rafters
- The gable frame is composed of IPE240 sections for the gable posts, and cold-formed channel sections for the rafters
- Steel grade is S275.

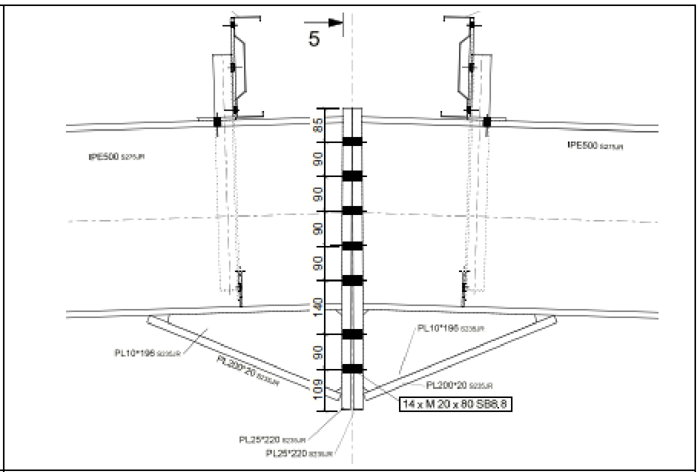
Purlins are thin cold-formed sections (sigma profile), spaced every 2,4 or 2,5 m.

Stability of the building is provided by a combination of cross bracing in the roof (L60x6 or L70x7 angles), and vertical cross bracing in the elevations (L70x7, L100x10 or L150x15 angles).

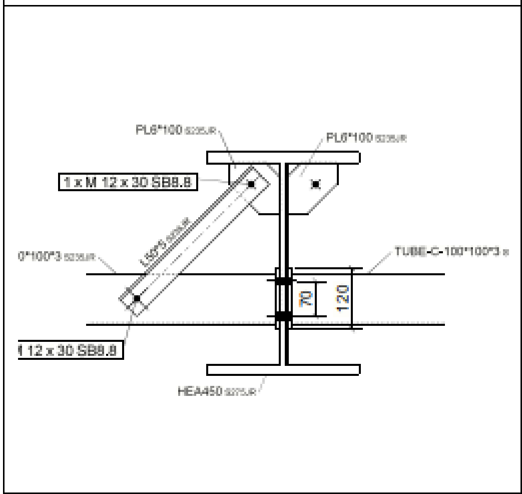
Typical connections are detailed in the following figures:



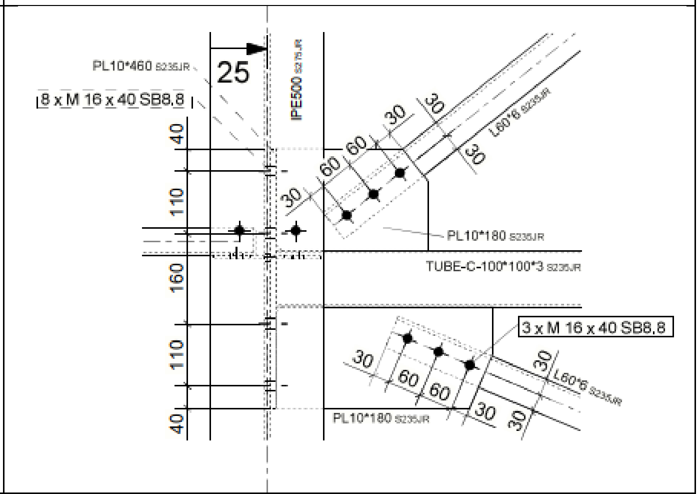
(a) Eave connection



(b) Apex connection



(c) Connection between side rail and column



(d) Bracing connection

Figure 32: Typical connections of the reference structure n°2

The structure is designed with Eurocode rules, with the following loads (according to French National Annexes):

- Permanent loads:
  - Self-weight of structural elements
  - Roof panels: 30 kg/m²
  - Photovoltaic panels: 25 kg/m²
  - Other accessories: 5 kg/m²
- Snow load:
  - Zone A1, altitude < 200 m
  - S = 36 kg/m²
- Wind load:
  - Zone 2, terrain category IIIb
  - Basic wind velocity: 24 m/s
- Seismic load:
  - Seismic zone: 3 (moderate)
  - Building importance class: II
  - Ground type: A

3.2 Primary structure with welded sections

3.2.1 Two bay portal frame (reference case n°3)

The third selected building has a floor area of 6 000 m² (100 m x 60 m) as illustrated in Figure 33.

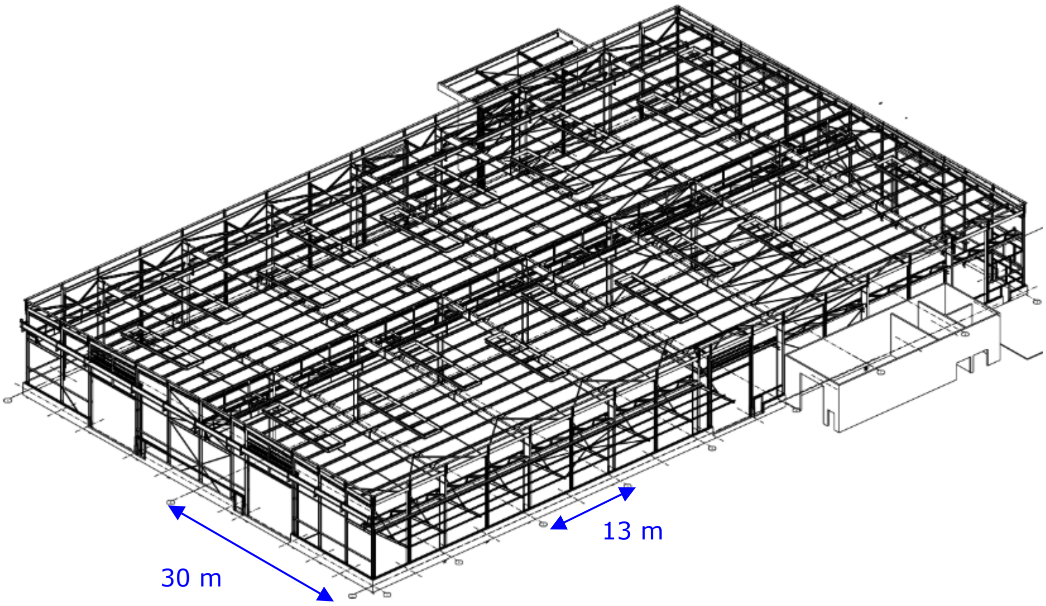


Figure 33: Perspective view of the reference case n°3 with floor area of 6 000 m²

The primary structure has the following characteristics:

- 2 bay portal frame
- Portal frame span: 30 m
- Frame spacing: 13 m
- Eave height: 10,3 m
- Roof pitch: 3,1 %
- Pinned base column
- Portal frames are composed of welded sections, as detailed in the figure below
- The gable frame is composed of IPE300 or IPE330 sections for the gable posts, and cold-formed channel sections for the rafters

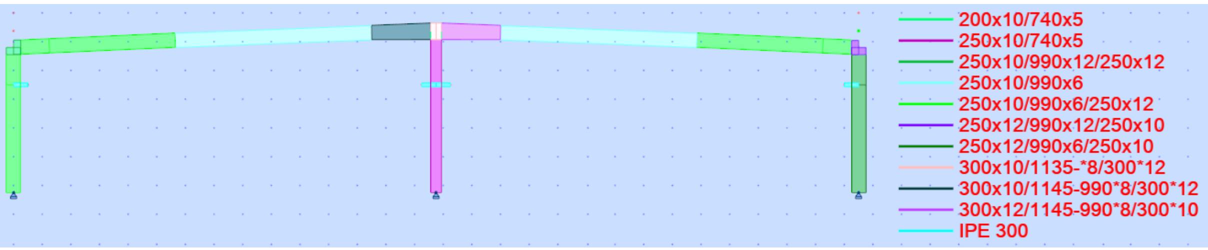


Figure 34: Main characteristics of portal frames of the reference structure n°3

The secondary structure is composed of:

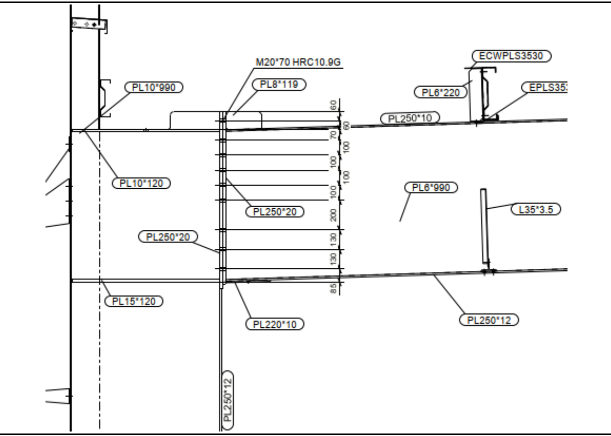
- Purlins in thin cold-formed sections (sigma profile), spaced every 2,4 m approximately
- Secondary columns made of IPE270 section

Steel grade is S355 for welded sections and S275 for hot rolled sections or cold-formed sections.

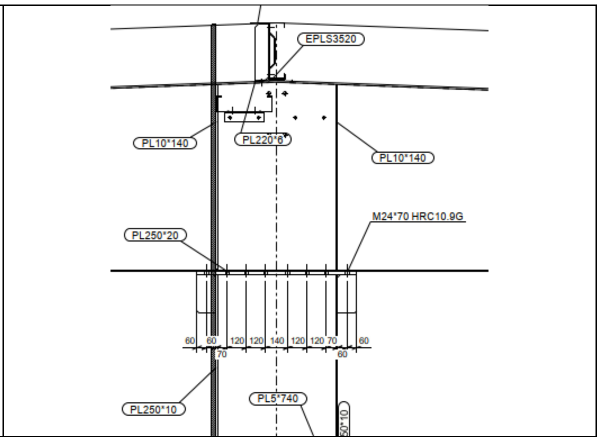
Stability of the building is provided by a combination of cross bracing in the roof, and vertical cross bracing in the elevations (L60x6 angles).

Two overhead cranes run the full length of the building, with a capacity of 6,3 tons. The crane beams are supported by beams made of IPE300 section.

Typical connections are detailed in the following figures:



(a) Eave connection



(b) Apex connection

Figure 35: Typical connections of the reference structure n°3

The structure is designed with Eurocode rules, with the following loads (according to French national annexes):

- Permanent loads:
  - Self-weight of structural elements
  - Roof panels: 24 kg/m²
- Imposed loads:
  - Other accessories: 5 kg/m²
- Snow load:
  - Zone A2, altitude < 200 m
  - S = 36 kg/m²
- Wind load:
  - Zone 1, terrain category IIIb
  - Basic wind velocity: 22 m/s
- Seismic load:
  - Seismic zone: 1 (weak)
  - Building importance class: II

3.2.2 Four bay portal frame (reference case n°4)

The last reference case is a building with a floor area of 12 000 m² (134 m x 89 m) as illustrated in Figure 36.

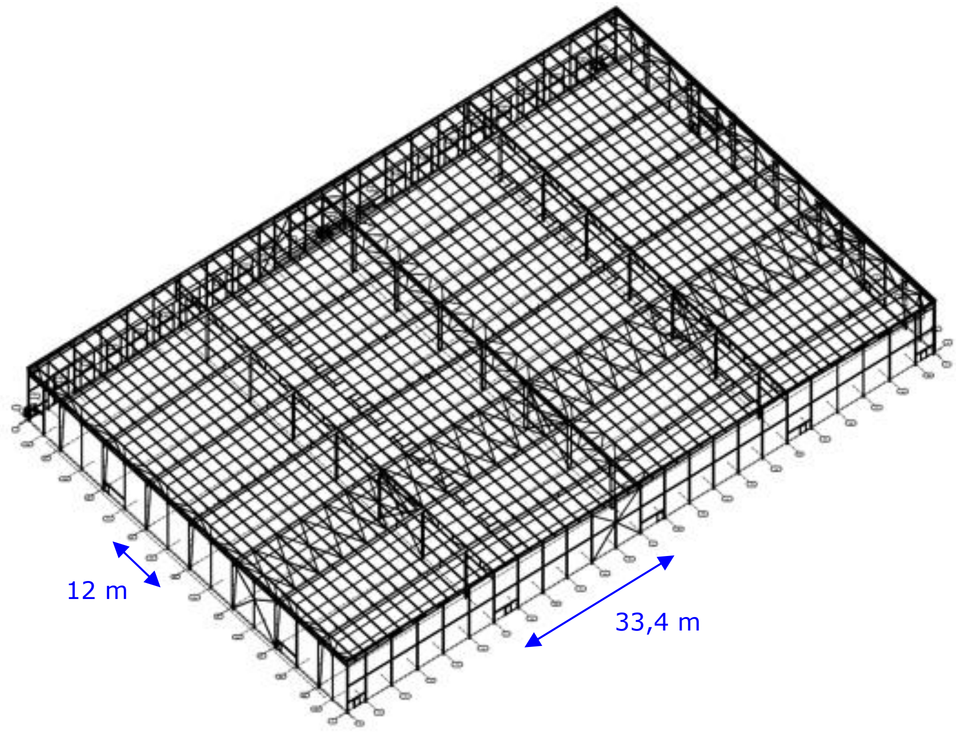


Figure 36: Perspective view of the reference case n°4 with floor area of 6 000 m²

The primary structure has the following characteristics:

- 4 bay portal frame
- Portal frame span: 33,4 m
- Frame spacing: 12 m
- Eave height: 12 m
- Roof pitch: 3,1 %
- Pinned base column
- Portal frames are composed of welded sections (except for intermediary columns which are HEA340 sections), as detailed in the figure below
- The gable frame is composed of IPE300 sections for the gable posts, and cold-formed channel sections for the rafters

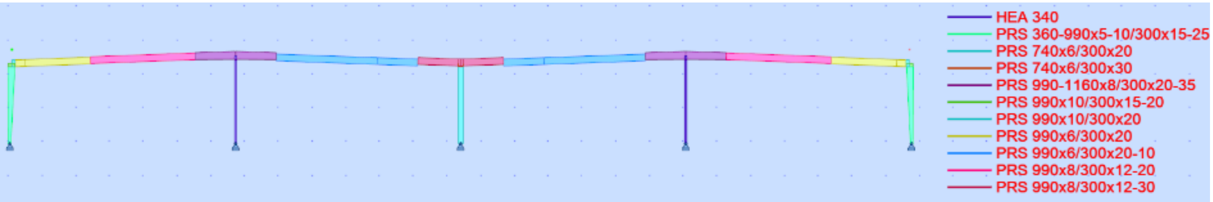


Figure 37: Main characteristics of portal frames of the reference structure n°4

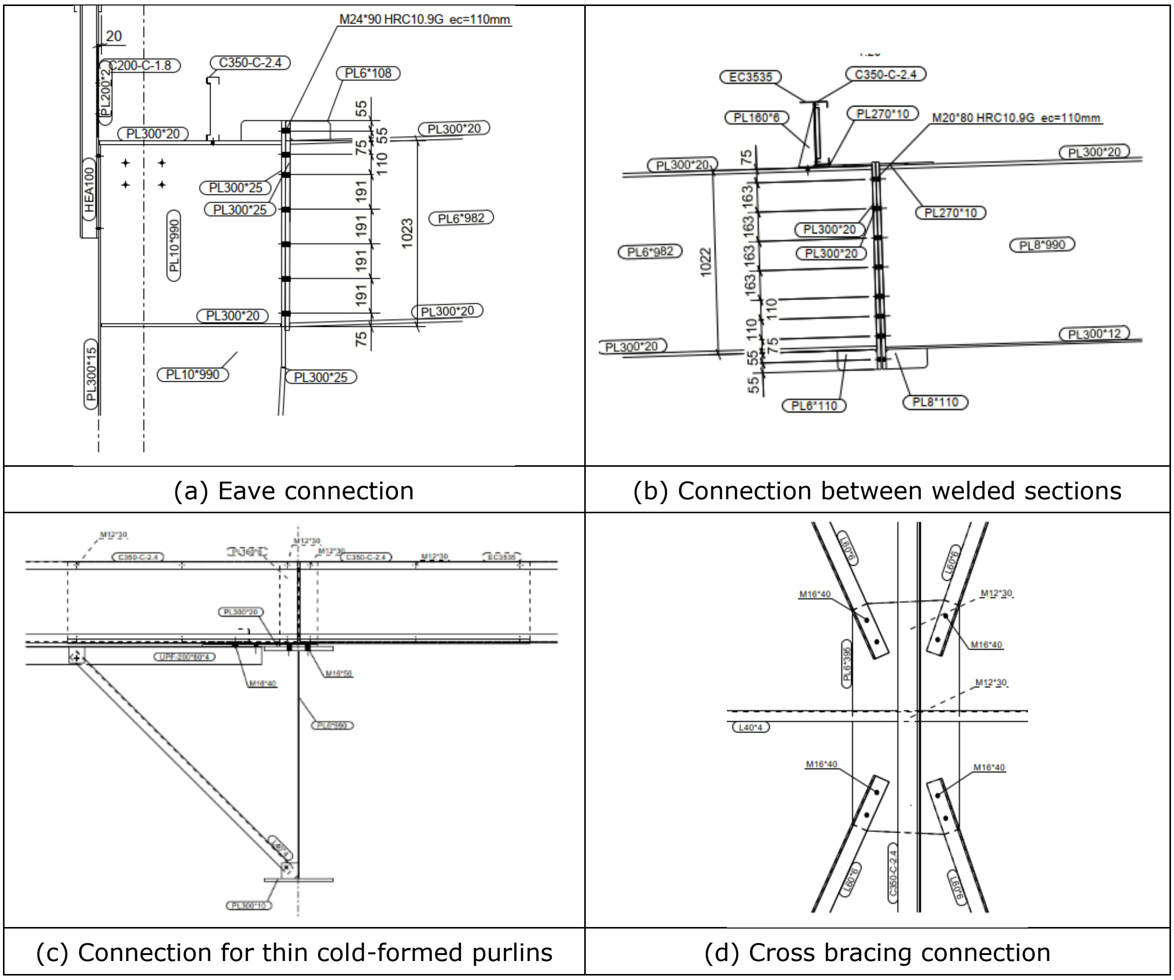
The secondary structure is composed of:

- Purlins in thin cold-formed sections (sigma profile), spaced every 2,7 or 2,8 m approximately
- Secondary columns made of IPE270 section

Steel grade is S355 for welded sections and S275 for hot rolled sections or cold-formed sections.

Stability of the building is provided by a combination of cross bracing in the roof (L60x6 angles), and vertical cross bracing in the elevations (L40x4 or L70x7 angles).

Typical connections are detailed in the following figures:



(a) Eave connection

(b) Connection between welded sections

(c) Connection for thin cold-formed purlins

(d) Cross bracing connection

Figure 38: Typical connections of the reference structure n°4

The structure is designed with Eurocode rules, with the following loads (according to French National Annexes):

- Permanent loads:
    - Self-weight of structural elements
    - Roof panels: 30,6 kg/m²
    - Sprinkler system: 5 kg/m² (purlins) or 10 kg/m² (portal frame)
    - Other accessories: 5 kg/m²
- Snow load:
    - Zone A1, altitude < 200 m
    - S = 36 kg/m²
- Wind load:
    - Zone 2, terrain category IIIa
    - Basic wind velocity: 24 m/s
- Seismic load:
    - Seismic zone: 3 (moderate)
    - Building importance class: II
    - Ground type: A

4 FIRE REGULATIONS

Usually, single-storey buildings have to meet various fire safety requirements, some of which are related to structural performances. Main requirements for fire resistance, compartmentation and smoke control imposed to commercial and industrial single-storey buildings in national fire regulations of reviewed European countries are summarized hereafter. Detailed information of the survey are provided in Annex A.

4.1 Structural fire resistance

Structural fire resistance is an important component of building's fire safety strategy. In particular, the structural fire resistance may contribute to maintaining the integrity of fire walls typically used for the sub-division of buildings into smaller areas to prevent the spread of flames, smoke and hot gases to the whole buildings. The fire resistance must also allow ensuring the stability of structures for a reasonable period of time in the event of fire to guarantee the safety of the building occupants during their evacuation and that of the firefighters during their intervention.

Structural fire resistance requirements vary significantly throughout Europe. The requirements for fire resistance of single-storey buildings are generally specified with regard to the building occupancy, the building height and the building fire risk level (fire loads), as shown in Table 2 and Table 4. Typically, the fire resistance requirements for single-storey buildings range from no fire resistance (R0) to 120 minutes (R120), but some national fire regulations may require up to 3 hours fire resistance for industrial buildings. As an illustration of the differences in fire requirements, the French regulation states that commercial buildings in France do not require any fire resistance (R0), while a minimum fire resistance R60 may be asked in Portugal. Similarly, no fire resistance may be required in Spain for type C industrial buildings detached at least 10 metres from any other building. According to the UK regulation, for industrial buildings built close to the site boundary, the external walls have to be designed to withstand a collapse of the roof, or the roof structure is required to have the same fire resistance as the walls. Fitting buildings with sprinklers often allow for reduction in the fire resistance requirements for many countries.

In addition to fire resistance, specific requirements in terms of overall structural behaviour may also be prescribed for single-storey buildings. A local failure of the heated parts of the structure may be accepted, as long as it does not affect the overall stability of the buildings. Structures must therefore be designed in such a way that no progressive collapse can occur in compartmented buildings, in order to avoid that the local collapse of the structure, due to a fire, endangers people (occupants and rescue services) or even property, located in a building area still protected from hot gases and smoke. Moreover, the structure of single-storey buildings (including façade elements) must collapse towards the inside of the building to provide a safe situation to firefighters located around the building. Such requirements are prescribed in many countries, like France, Belgium, Luxembourg, Spain or Italy.

At last, it is worth noting that most of the national fire regulations permit a performance-based approach to assess the structural fire behaviour, as an alternative to prescriptive rules.

4.2 Compartmentation

Buildings must be designed and constructed in such a way that, in the event of a fire, the fire is contained and its effects are limited within a specific section of buildings. For this purpose, buildings are usually subdivided into compartments of limited size, separated by means of partition fire walls with appropriate fire performance. The effects provided by compartmentation on property loss is that direct damage is confined to the content of the compartment in which the fire starts, reducing the chances of the fire growing large. As regards life safety, people in other parts of the building can use escape routes to get out safely without being exposed to the smoke or gases from the fire.

Each country surveyed, apart from UK, has requirements for the compartmentation of industrial and commercial single-storey buildings. Requirements vary significantly from one country to another, as shown in Table 3 and Table 4. Most of the fire regulations impose restrictions on the maximum permissible compartment size according to the building occupancy and the fire risk level. It is worth noting that fitting sprinklers allow increasing the maximum compartment size, but limits are still imposed in all countries but the UK.

In several regulations for industrial buildings, the fire resistant level of partition fire walls is different from the structural fire resistance. In Spain, the same fire resistance rating is required for fire walls and the structure. For the other countries, the fire resistance rating of partition fire walls varies from 60 minutes to 120 minutes, when the structural fire resistance varies usually from 0 to 60 minutes.

With regard to commercial buildings, the fire resistance required for the partition fire walls varies also widely. Some countries require as little as 30 minutes fire resistance, while others require 120 minutes. For most part of the countries, except Finland, Poland, Portugal and Switzerland, the REI requirement of partition fire walls is independent on fire risk.

In addition to fire resistance requirement, specific provisions may be asked for partition fire walls. For example, in France, partition fire walls in industrial buildings must extend at least 1 m beyond the top surface of the roof. In addition, if the external facades do not have a 1 hour fire resistance rating (EI 60 or REI 60), the partition fire walls must be laterally extended over a width of 1 m, or 0.5 m projecting from the façade in the line of the wall. Similar requirements exist in Belgium, Luxembourg, Spain and Portugal.

It could be noted that sandwich panels are suitable for internal and external fire wall applications. Usually, the fire performance of sandwich panel wall solutions is evaluated from standard fire tests conducted in accordance with the general requirements of EN 1363-1 [6], with alternative or additional procedures provided in the complementary standard EN 1363-2 [7], with the EN 1364-1 specific requirements [8] and EN 14509 specifications [9]. Based on fire test results, a classification report is drawn up following EN 13501-2[10], which should then be used to prove that the considered wall solutions have the required fire-resistance rating.

4.3 Fire suppression

Sprinklers may be required by national fire regulations. As already mentioned, in addition to their obvious effect in the reduction of the fire growth, their use usually leads to a reduction of the fire resistance rating required for the structure. In most countries, they also allow larger fire compartment sizes.

4.4 Smoke control systems

National fire regulations may require that smoke control systems are implemented in public buildings, storage buildings and industrial buildings in order to facilitate escape, by minimising risks of smoke inhalation and injury and to some extent, to enable fire-fighters to better see the fire and therefore, to extinguish it more speedily and effectively. Smoke control systems help in removing smoke from the fire area, and in limiting the spread of hot gas beneath the roof, which increases the time for the compartment to become smoke-logged, giving people more time to escape safely from the building. Traditionally, this is achieved by a combination of smoke exhaust systems (mechanical or natural) and smoke curtains (to prevent the lateral spread of smoke).

In the countries surveyed, the fire compartments should be usually subdivided into so called "smoke compartments" or "smoke zones" with maximum allowable area, bordered by smoke curtains. Typically, the maximum permissible size of « smoke zones » range more or less from 1600 and 2600m², depending if the building is naturally or mechanically ventilated. So, a smaller building may simply be split into two "smoke zones", while a very large warehouse may have multiple "smoke zones". In addition, national regulations often require that buildings are equipped with smoke and heat exhaust systems. In most countries, smoke extraction does not affect the size of fire compartments, although they are sometimes required as an additional system to support the operation of firefighters, e.g. after exceeding a specific maximum area. To facilitate design, in some cases, regulation prescribes a minimum lump sum value for the needed aerodynamic surface (effective area) of exhausts, usually 1 or 2% of the compartment area. For example, the French regulations state that fire compartments in industrial buildings must be divided into "smoke zones" bordered by fixed smoke curtains, with a maximum area of 1 650m² and a maximum length of 60m. Each smoke zone is equipped with a smoke exhaust system, with an effective area not less than 2% of the smoke zone area. Similar requirements may exist in Belgium, Spain and Portugal.

4.5 Fire detection and fire alarms

Adequate measures are traditionally necessary for detecting any outbreak of fire and for alerting the building occupants and the fire department of the occurrence of fire. In small single-storey buildings where all exits are visible, it is likely that any fire will be quickly detected by the occupants and a shout of 'Fire!' may be sufficient. In larger single-storey buildings, a simple sounder such as a battery powered alarm or rotary bell may be adequate. In an industrial building, the ambient noise has to be considered, to ensure that the alarm will be heard by the occupants.

In some countries, fire alarm may affect the maximum permissible size of fire compartments. For example, the fire compartment size in Finland may be increased by 50% with the installation of fire alarm system in buildings.

4.6 Egress facilities

For safe evacuation, appropriate means of escape are needed, such as a proper number and width of emergency exits and proper length, width and height of passages and evacuation accesses. Escape routes in small single-storey buildings generally lead directly to a safe location outside the building; they do not normally require any special treatment. In larger buildings, where travel distances are greater and where the fire is likely to make a single escape route unusable, an alternative mean of escape may be necessary. Fire regulations also take into consideration disabled people.

Traditionally, a maximum distance for evacuation up to an exit is given in each regulation. The prescribed distance ranges usually between 25 and 60 m according to the number of exits in the building.

Table 2: Structural fire resistance requirements for industrial & storage single-storey buildings

Countries	Fire risk			Comments
	Low	Medium	High	
France (***) V >5 000°m^3	R15 – H ≤ 13,70m R60 - H>13,70m	R15 – H ≤ 13,70m R60 - H>13,70m	R15 – H ≤ 13,70m R60 - H>13,70m	A performance-based design is permitted to assess the structural fire behaviour
Spain Type A	R90 / (R60)	R120 / (R90)	Not allowed	
Spain Type B	R60 / R15* / (0)	R90 / R30* / (R15)	R120 / R60* / (R30)	
Spain Type C	R30 / R0* / (R0)	R60 / R15* / (R0)	R90 / R30* / (R15)	If building detached at least 10m from anyother building R0
UK	R60 / (R30)	R60 / (R30)	R60 / (R30)	If building detached from any other building R0
Belgium	R60	R120	R120	Requirement for type I structural members. R0 respecting some maximum compartment size restrictions A performance based design is permitted to assess the structural fire behaviour
Luxembourg	R0	R30 R0****	R60 R30****	A performance-based design is permitted to assess the structural fire behaviour
Germany	R0	R30 / (R0)	PBD / (R0)	
Italy	Depends on the performance level assigned according to the risk profile******			A performance-based design is permitted to assess the structural fire behaviour
Switzerland	R0	R0	R0	
Portugal	R0	R60	R60	
Finland	R0	R 30 (R15) R15**	R 60 (R30) (R15**)	A performance-based design is permitted to assess the structural fire behaviour
Poland	R0	R60 R15*****	R240 R30*****	R0 for compartment with an area higher than 1000m² and equipped with a smoke exhaust system
Netherlands	R0	R0	R0	
Hungary	R30	R30	R60	

The values in brackets are for buildings fitted with sprinklers
PBD: Performance based design
(*): Reduction of required fire stability is admitted if height is lower than 15m, the roof is light (<100kg/m²) and the collapse of the structure does not endanger other buildings
(**): Reduction of required fire stability is admitted if flammability class 1 is non-combustible
(***): Regulation for buildings with more than 500 tonnes of combustibles
(****) Fire requirement for buildings with a ground surface area lower than 2 500m²
(*****) Fire requirement for the roof supporting structure
(******) Often, a performance level II is assigned, which entails a structural fire resistance of R30 or less based on the design fire load

Table 3: Maximum compartment size and fire wall requirements for industrial & storage single-storey buildings

Countries	Fire risk			Comments
	Low	Medium	High	
France (****) V >5 000°m^3	S≤3 000 m^2 (3 000<S≤6 000m^2) 6000 m^2<S* REI120	S≤3 000 m^2 (3 000<S≤6 000 m^2) 6000 m^2<S* REI120	S≤3 000 m^2 (3 000<S≤6 000 m^2) 6 000 m^2<S* REI120	The limit of 6 000m^2 can be increased to 12000 m^2 if the height of the compartment does not exceed 23m
Spain class A	S≤1000 m^2 S**≤1250 m^2 (S≤2 000 m^2) REI=R structure	300<S<500 m^2 375<S**≤625 m^2 (600<S≤1 000 m^2) REI=R structure	No permitted	
Spain ClassB	S≤4 000 m^2 S**≤5 000 m^2 (S≤8 000 m^2) REI=R structure	2 500<S≤3 500m^2 3 125<S**≤4 375m^2 (5 000 m^2<S≤7 000m^2) REI=R structure	1 500<S≤2 000m^2 1 875<S**≤2 500m^2 (3 000<S≤4 000m^2) REI=R structure	
Spain ClassC	S≤6 000m^2 S**≤7 500m^2 (S≤12 000m^2) REI=R structure	3 500<S≤5 000 m^2 4 375<S**≤6250 m^2 (6 000<S≤10 000 m^2) REI=R structure	2 000<S≤3 000 m^2 2 500<S**≤3 750 m^2 (4 000<S≤6 000 m^2) REI=R structure	No limit if the building is detached more than 10m from any other building and building sprinklered
UK	No limit	No limit	No limit	
Belgium	S≤25 000 m^2 (S≤150 000m^2) REI60	S≤5 000m^2 (S≤40 000m^2) REI120	S≤2 000m^2 (S≤7 000m^2) REI120	
Luxembourg	S ≤1 600 (S ≤3 200) REI 90	S ≤3 200 (S ≤6 400) REI 90	S ≤5 000 (S ≤10 000) REI 90	
Italy	depends on the life risk profile	depends on the life risk profile	depends on the life risk profile	The maximum compartment size for buildings ranges usually from 4 000 m^2 to 32 000 m^2.
Switzerland	S≤2 400m^2 / EI 30 (S≤4 800m^2) S>4 800m^2*** /EI 30	S≤2 400m^2 / EI 30 (S≤4 800m^2) S>4 800m^2*** /EI 30	S≤2 400m^2 / EI 30 (S≤4 800m^2) S>4 800m^2*** /EI 30	
Portugal Case I Case II Case III	 1 600 m^2 / REI 60 (3 200) m^2 6 400 m^2 / REI 60 (12 800 m^2) 12 800 m^2 / REI 60 (25 600 m^2)	 800 m^2 / REI 90 (1 600 m^2) 2 400 m^2 / REI 90 (4 800 m^2) 4 800 m^2 / REI 90 (9 600 m^2)	 400 m^2 / REI 120 (800 m^2) 800 m^2 / REI 120 (1 600 m^2) 2 400 m^2 / REI 120 (4 800 m^2)	No limit if the building is detached more than a relevant distance.
Finland fire hazard class 1 fire hazard class 2	 6 000[1)] (60 0000) EI-M 90 (EI-M 60) 2 000[1)] (12 0000) EI-M 120 (EI-M 60)	 4 000[1)] (36 0000) EI-M 90 (EI-M 60) 1 000[1)] (6 0000) EI-M 120 (EI-M 60)	 2 000 (12 000) EI-M 90 (EI-M 60) (2 000) (EI-M 60)	[1)] The surface area may be increased by 50% with the installation of fire alarm system
Poland	20 000[1)] m^2 (40 000 m^2) REI 0	8 000[1)] m^2 (16 000 m^2) REI 60	2 400[1)] m^2 4 800 m^2 REI 120	[1)] The surface area may be increased by 50% with the installation of smoke exhaust system
Netherlands	2 500 m^2 REI 60	2 500 m^2 REI 60	2 500 m^2 REI 60	
Hungary	10 000 (15 000) 30 000***** REI 180	5 000 (10 000) 20 000***** REI 180	1 000 (4 000) 8 000***** REI 180	

The values in brackets are for buildings fitted with sprinklers
(*): With safety engineering and agreement of the authority (CSIS)
(**): 50% of the perimeter accessible for fire brigade
(***): Technical measures are required (smoke exhaust system, sprinklers, ect.)
(****) Regulation for buildings with more than 500 tonnes of combustibles
(*****) Building fitted with an automatic fire-extinguishing system and equipped with a fire alarm system

Table 4: Structural fire resistance, maximum compartment size and fire wall resistance requirements for commercial (large space) single-storey buildings

Countries	Fire resistance of structures	Maximum compartment size S and fire wall resistance	Comments
France	R30 or R0*	No limit, except for storage area (V<5 000m^3) REI 30 to REI 120	A performance-based design is permitted to assess the structural fire behaviour
Spain	R90 or R30**	S ≤2 500m^2 (S ≤10 000m^2) REI 90	
Belgium	R30	S ≤2 500 (S ≤3 500) REI 30	The length of compartments shall not exceed 90 m. The compartment size can exceed the given limits with the agreement of authorities. A performance-based design is permitted to assess the structural fire behaviour
UK	R60 (R30)	S ≤ 2 000 m^2 (no limit) REI 60	No structural fire resistance if building detached from any other building
Portugal	R0 or R60 according to the risk category	S ≤1 600 m^2 (S ≤3 200m^2) REI 60	In case of single compartment, the maximum size may be up to 8 000 m^2 or 16°000 m^2 according to the circulation features of the building
Luxembourg	R30 (R0)	No limit REI 60 (REI 0)	A performance-based design is permitted to assess the structural fire behaviour.
Italy	Depends on the performance level assigned according to the risk profile	No limit	A performance-based design is permitted to assess the structural fire behaviour.
Finland	R60	S ≤1 600 m^2 EI 30	A performance-based design is permitted to assess the structural fire behaviour
Switzerland	R0	S ≤600 m^2 EI 30	The maximum size may be higher if the building is fitted with sprinklers and equipped with an smoke exhaust system
Poland	R30 (R0)	S ≤10 000 m^2 REI 60	The maximum compartment size may be increased by 100% if the building is fitted with sprinklers or equipped with a smoke exhaust system, or by 200% with both measures
Netherlands	R0	1 000 m^2 REI 60	
Hungary	Depends on the fire risk classification	Depends on the fire risk classification	

The values in brackets are for buildings fitted with sprinklers

(*): No stability required is admitted for single storey buildings with special conditions:

- the main structural members are made out of fireproof materials or materials specified in regulation
- the roof structure is visible from the floor, or supervised by an automatic fire detection system, or
- fire protected by an automatic fire sprinkler system, or isolated by a protective screen allowing to meet a fire resistance rating R30

(**): Reduction of required fire stability admitted if height is lower than 28m, the roof is light (<100kg/m^2) and the collapse of the structure does not endanger other buildings

5 SEISMIC REGULATIONS

As for fire, the main objective of seismic regulations is to protect people during earthquakes. Consequently, buildings must be designed so that no collapse occurs and structural damage remains limited in the event of an earthquake. Single-storey buildings may therefore be subject to seismic design requirements, with levels depending mainly on activities of the building, its importance and the seismic zone in which it is located.

This section provides an overview of the main regulation provisions given in the reviewed European countries for the earthquake resistant design of single-storey buildings, according to EN 1998-1 [31]. In particular, all the Nationally Determined Parameters (NDPs) needed for the representation of seismic actions (including the seismic zone maps, the corresponding reference ground accelerations, the importance factor assigned to buildings, the soil factor assigned to each ground type and the different parameters describing the response spectrums that can be used for design) are summarised hereafter. Detailed information can be found in Annex B. It is worth remembering that the NDPs are those parameters that were left open in EN 1998-1 for national choice, to be used for the design of buildings and civil engineering works, in order to take into account country specific geographical, geological or seismic conditions, as well as specific levels of protection.

It should be noted that there are no restrictions to the use of the EN 1998-1 (voluntary or mandatory use) for the design of new construction works in all reviewed countries, apart from Italy and Switzerland, where national seismic code are mandatory. The regulation provisions coming from the Italian code NTC-2018 [60] and the Swiss code SIA 261 [65] are summarised in Annex B.

5.1 Performance requirements and compliance criteria

Current seismic codes, like EN 1998-1, generally ask for a two level performance based design of structures, in satisfying the following two performance requirements:

- No-collapse requirement: The structure should withstand the seismic events without local or global collapse.
- Damage limitation requirement: The structure should withstand the seismic events without the occurrence of excessive damage and associated limitations of use.

The two previous requirements are to be checked against two different levels of seismic action. In case of ordinary structures (of ordinary importance), EN 1998-1 recommends the two following seismic actions:

- A seismic action with an exceedance probability of 10% within 50 years, which corresponds to a return period of 475 years, for the "no-collapse requirement".
- A seismic action with an exceedance probability of 10% within 10 years, which corresponds to a return period of 95 years, for the "damage limitation requirement".

5.2 Seismic zoning

In Europe, the seismic hazard is traditionally presented in the form of maps subdividing national territories into several seismic zones as a function of the local hazard. The seismic hazard maps are currently defined by National Authorities and can be found in each country's National Annex of EN 1998-1.

According to EN 1998-1, the seismic hazard is described from a single parameter, i.e. the value of the reference Peak Ground Acceleration on rock soil or other rock-like geological formations. In most of European countries, the peak ground acceleration specified by National Authorities for each seismic zone, corresponds to a reference return period of the seismic action equal to 475 years, or equivalently to a reference probability of exceedance of 10% in 50 years, i.e. the recommended values in EN 1998-1.

A typical European Seismic Hazard Map, in terms of peak ground acceleration with a 10% chance of being exceeded in 50 years, is given in Figure 39 for information purpose. National seismic hazard maps of all European countries surveyed are given in Annex B. In most of these countries, it is worth noting that the hazard within each seismic zone is assumed to be constant, apart for Spain and Italy where seismic hazard is defined according to geodetic coordinates.

Depending upon the value of the design ground acceleration and the ground type, it should be also mentioned that cases of low or very low seismicity, where reduced/simplified or no seismic design procedures for certain types or categories of structures may be followed, are usually specified in National Annexes.

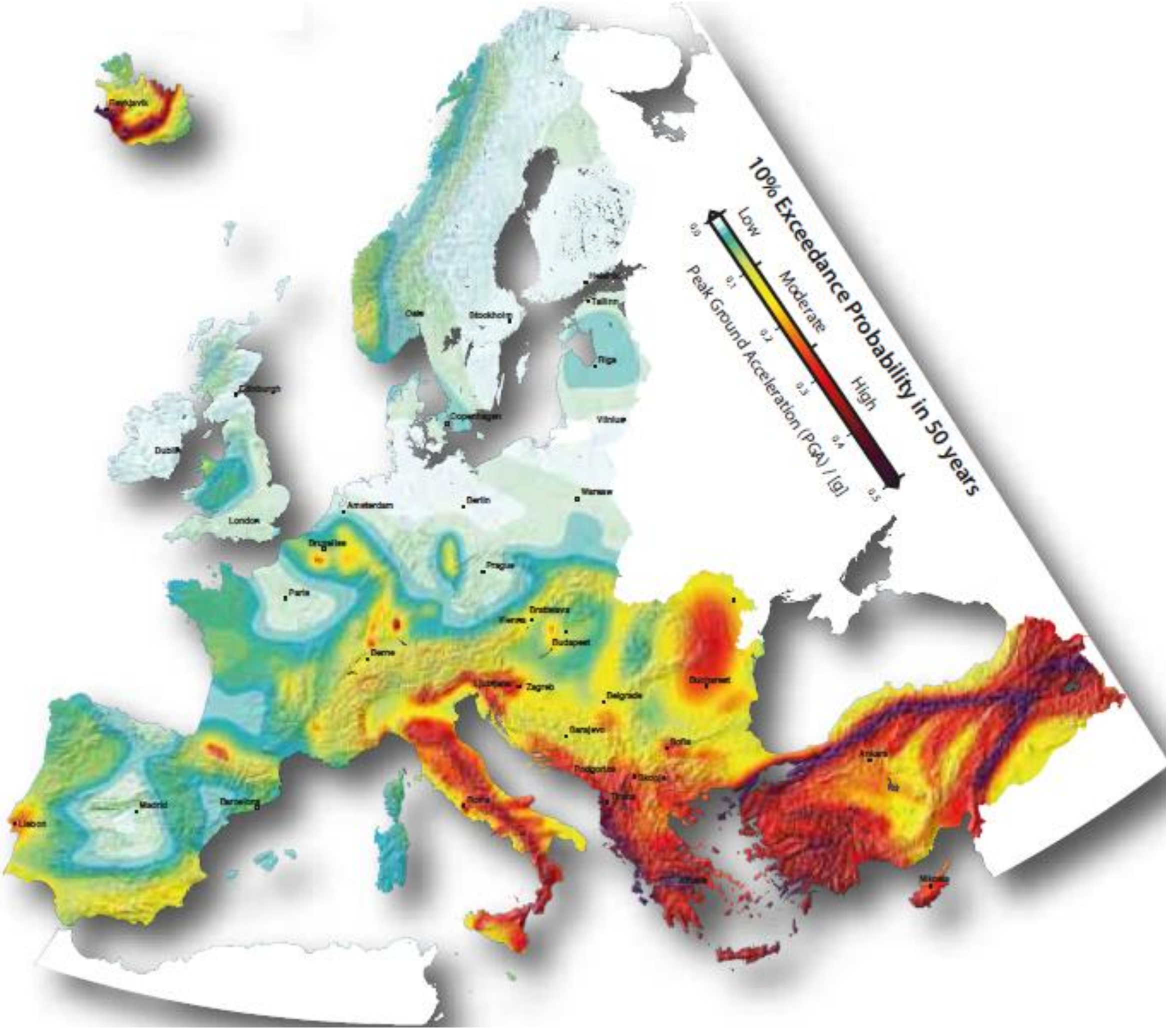


Figure 39: Euro-Mediterranean Seismic Hazard map [33]

5.3 Importance classes and importance factors

EN 1998-1 currently classifies buildings into four importance classes of increasing importance, from category I to category IV, depending on the consequences of collapse for human life, on their importance for public safety and civil protection in the immediate post-earthquake period, and on the social and economic consequences of collapse. According to this standard, an importance factor, γ_I, is assigned to each importance class, leading to a value modification of the reference seismic action to determine the design earthquake.

The building classification and corresponding importance factors, as defined in EN 1998-1, are reported in Figure 5. Values given in this table are the recommended ones. It is worth noting that importance factor values are NDP's, which may be changed by the National Authorities according to characteristics of the countries seismic hazard. Most of reviewed European countries have opted for the recommended values, except Portugal and UK. In addition, it should be mentioned that the French regulation distinguishes two types of buildings:

- The so-called "normal risk" buildings, which are categorised according to the four importance classes defined in EN 1998-1, and
- The "special risk" buildings, which are buildings, facilities and equipment for which the effects on people, property and the environment, even minor damage resulting from an earthquake, may not be limited to the immediate vicinity of these buildings. It corresponds to nuclear type installations, dams, bridges, facilities classified for environmental protection, which are

subject to special seismic regulations. An importance factor equal to 2.2 is assigned to such type of buildings by the French national authorities.

In practice, single-storey buildings for industrial, commercial and artisanal uses are most commonly classified in importance classes II or III.

Table 5: Definition of importance classes of buildings in EN 1998-1

Importance classes	Building	Importance Factor γ_I
I	Buildings of minor importance for public safety (e.g., agricultural buildings, etc.)	0,8
II	Ordinary buildings not belonging to the other categories (smaller residential and office buildings, workshops, etc.)	1,0
III	Buildings of which the seismic resistance is important in view of the consequences associated with a collapse (huge residential buildings, schools, assembly halls, malls, etc.)	1,2
IV	Buildings of which the integrity during earthquakes is of vital importance for civil protection (hospitals, important civil protection facilities, fire department, security staff, etc.)	1,4

It is worth mentioning that in a high-seismic prone country like Italy, the coefficient of use (CU) is assigned to each class. It has a different meaning than the importance class stated in EN 1998-1 and then they are not directly comparable. The Italian coefficient of use modifies the return period and consequently the seismic action, whereas the importance class directly changes the seismic action by taking into account a different return period.

5.4 Influence of the ground type

EN 1998 classifies the ground type in five main categories, denoted Ground types A to E, described by the stratigraphic profiles and parameters given in Table 6 in view of taking into account the ground influence on the seismic response of structures (i.e. allowing the selection of the relevant spectral shape, among various different possibilities, as shall be presented below).

Ground types A to D range from rock or other rock-like formations to loose cohesionless soils or soft cohesive soils. Ground Type E is essentially characterised by a sharp stiffness contrast between a (soft or loose) surface layer (thickness varying between 5 and 20 m) and the underlying much stiffer formation. Two additional soil profiles (S1 and S2) are also included in Table 6. For sites with ground conditions matching one of these ground types, special studies for the definition of the seismic action are required.

The ground classification of EN 1998-1 (ground type A to E) is adopted in the major part of the European countries surveyed. However, a particular ground classification is specified in the German and Spanish National Annexes.

Table 6: Ground types as defined in EN1998-1

Ground type	Description of stratigraphic profile	Parameters		
		$v_{s,30}$ (m/s)	N_{SPT} (blows/30cm)	c_u (kPa)
A	Rock or other rock-like geological formation, including at most 5 m of weaker material at the surface.	> 800	_	_
B	Deposits of very dense sand, gravel, or very stiff clay, at least several tens of metres in thickness, characterised by a gradual increase of mechanical properties with depth.	360 – 800	> 50	> 250
C	Deep deposits of dense or medium-dense sand, gravel or stiff clay with thickness from several tens to many hundreds of metres.	180 – 360	15 - 50	70 - 250
D	Deposits of loose-to-medium cohesionless soil (with or without some soft cohesive layers), or of predominantly soft-to-firm cohesive soil.	< 180	< 15	< 70
E	A soil profile consisting of a surface alluvium layer with v_s values of type C or D and thickness varying between about 5 m and 20 m, underlain by stiffer material with v_s > 800 m/s.			
S_1	Deposits consisting, or containing a layer at least 10 m thick, of soft clays/silts with a high plasticity index (PI > 40) and high water content	< 100 (indicative)	_	10 - 20
S_2	Deposits of liquefiable soils, of sensitive clays, or any other soil profile not included in types A – E or S_1			

5.5 Representation of the seismic action

A seismic design approach most commonly adopted nowadays is based on the use of response spectra. The EN 1998-1 specifies response spectra that may be used in seismic design while allowing that any country may use its own shape of response spectrum, by specifying values of parameters defining it in its National Annex.

According to EN 1998-1, response spectra are constructed from a basic shape, assumed to be independent of the level of seismic intensity, normalised by the peak horizontal ground acceleration (a_gS) applicable at the construction site.

5.5.1 Elastic response spectrum

Within the scope of EN 1998-1, the shape of the elastic response spectrum is taken as being the same for the two levels of seismic action for the no-collapse requirement and for the damage limitation requirement.

For the horizontal components of the seismic action, the elastic response spectrum $S_e(T)$ consists of four successive parts depending on periods TB, TC and TD. The first part starts from the design ground acceleration value, a_g, and increases linearly up to a value of $2{,}5 \times S \times \eta \times a_g$, where S is the soil factor whose values (along with the values of TB, TC and TD) depend on the ground type, and η is a structural damping correction factor with reference value η=1 for 5% viscous damping. The second part retains a constant value up to period TC. The third part decreases inversely linearly to the period up to a value of TD. Finally, the last part continues decreasing inversely to the square of the period up to a period equal to 4sec.

Specifically, the elastic response spectrum is defined by the following expressions:

- If $0 \leq T \leq T_B$ $\quad S_e(T) = a_g\, S\left[1 + \frac{T}{T_B}(2.5\,\eta - 1)\right]$
- If $T_B \leq T \leq T_c$ $\quad S_e(T) = a_g\, S\, 2.5\,\eta$
- If $T_c \leq T \leq T_d$ $\quad S_e(T) = a_g\, S\, 2.5\,\eta\left[\frac{T_C}{T}\right]$
- If $T_D \leq T \leq 4\text{s}$ $\quad S_e(T) = a_g\, S\, 2.5\,\eta\left[\frac{T_C\,T_D}{T}\right]$

where:

- T is the vibration period of a linear single-degree-of-freedom system;
- a_g is the design ground acceleration on type A ground, which could be obtained from the reference ground acceleration a_{gr} multiplied by the importance factor γ_I of the construction as follows: $a_g = \gamma_I\, a_{gr}$
- T_B, T_C are the limits of the constant spectral acceleration branch;
- T_D is the value defining the beginning of the constant displacement response range of the spectrum;
- S is the soil factor, allowing to take account for different ground types;
- η is the damping correction factor.

It should be noted that although 5% damping is taken as reference value, EN 1998-1 allows the use of other damping ratios corresponding to different materials, or for systems with artificial (active) damping devices.

EN 1998-1 defines two types of response spectra (type 1 and type 2) for varying seismicity conditions. The use of type 1 response spectrum is recommended for seismic areas where the surface-wave magnitude of such earthquakes exceeds 5,5 (typically very active regions of southern Europe, such as Greece or Italy) and type 2 otherwise (typically for stable areas of central and northern Europe, such as Germany). EN 1998-1 recommends two series of coefficients, C and S for taking into account soil conditions.

The values of parameters S, TB, TC and TD are, as already stated, Nationally Determined Parameters (NDPs) which may be specified in the National Annex to take into account the specific regional features of the seismic hazards. For information, the recommended values of the parameters S, T_B, T_C and T_D, for type 1 and type 2 response spectra are given in Table 7 and Table 8 according to the ground type. It can be noted that the recommended value for the soil factor is S = 1 for Ground Type A (Rock) and ranges from S = 1,2 to 1,4 for the other ground types in case of type 1 response spectra or from S = 1,35 to 1,8 in case of type 2 response spectra.

For the vertical component, the shape of the response spectrum and the values of both design ground acceleration and periods TB, TC and TD are different. It should be noted that the vertical component is usually small and may be, like vertical wind loading, disregarded. Furthermore, contrary to what is indicated for the horizontal components, it is considered that the vertical ground motion is not very much affected by the underlying ground conditions and so no use of the soil factor S is made.

Table 7: Values of the parameters describing the recommended type 1 elastic response spectra

Ground type	S	T_B (s)	T_C (s)	T_D (s)
A	1,0	0,15	0,4	2,0
B	1,2	0,15	0,5	2,0
C	1,15	0,2	0,6	2,0
D	1,35	0,2	0,8	2,0
E	1,4	0,15	0, 5	2,0

Table 8: Values of the parameters describing the recommended type 2 elastic response spectra

Ground type	S	T_B(s)	T_C (s)	T_D (s)
A	1,0	0,05	0,25	1,2
B	1,35	0,05	0,25	1,2
C	1,5	0,1	0,25	1,2
D	1,6	0,1	0,3	1,2
E	1,8	0,05	0,25	1,2

Table 9: recommended Values of parameters for the vertical elastic response spectrum

Spectrum	A_{vg}/a_g	T_B (s)	T_C (s)	T_D (s)
Type 1	0,90	0,05	0,15	1
Type 2	0,45	0,05	0,15	1

After reviewing all the National Annexes of the countries surveyed, it should be noted that:

- In France, the use of the type 2 elastic response spectra is specified for "normal risk" buildings in seismic zones 1 to 4 and for "special risk" buildings in seismic zones 1 to 3, with characteristic periods different from those recommended in EN 1998-1. The use of the recommended type 1 elastic response spectra is specified for "normal risk" buildings in seismic zone 5 and for "special risk" buildings in seismic zones 4 and 5, with the same soil factors and characteristic periods as with those recommended in EN 1998-1.
- In Belgium, Luxembourg and UK, the use of the recommended type 1 response spectra is specified in National Annexes, because the type 2 spectra is not typical for the local conditions in these countries In Germany, only type 1 response spectra are also applicable, but with amplification factors and cut-off periods different from recommended values.
- Contrary to general practice, the Portuguese National Annex of EN 1998-1 considers two seismic zonation maps corresponding to intraplate earthquakes of high-magnitude and continental earthquakes of moderate magnitude, respectively. The corresponding response spectra are built from type 1 response spectra in the first case and Type 2 in the second one, but with different soil factor and cut-off period values. In general, the plateau is wider in the Portuguese National Annex than in EN 1998-1. It is worth mentioning that in the Portuguese National Annex, non constant values of the soil factor S have been adopted. In fact, the S factor decreases as the ground acceleration increases in the different seismic zones. It ranges from 1,35 to 2,0 and is independent of the response spectra type.
- For Czech Republic, the use of either type 1 or type 2 response spectra is also specified according to the locality considered.
- For Spain, the shape of the response spectra is obtained from the formula given in EN 1998-1. However, the cut-off periods and the soil factor are different from recommended values, depending on two coefficients: C, defining the soil class, and K, which takes into account the influence of the long-distance earthquakes occurred in the Azores Gibraltar zone. The soil factor S depends on the soil coefficient C and the design ground acceleration a_g through a non-linear function, taking values in the range 0,8-2,0.
- For Italy, the two types of spectrum stated in the EN 1998-1 are not used because the national mandatory standard defines the elastic response spectrum for each limit state as a function of the class of use and the location of the building.

5.5.2 Design spectrum for elastic analysis

The capacity of structural systems to resist seismic actions in the non-linear range generally permits their design for forces smaller than those corresponding to a linear elastic response. According to EN 1998, in order to avoid explicit inelastic structural analysis in design, the capacity of the structure to dissipate energy, through mainly ductile behaviour of its elements and/or other mechanisms, is taken into account by performing an elastic analysis based on a design response spectrum, reduced with respect to the elastic one. This reduction is accomplished by introducing a behaviour factor q.

For the horizontal components of the seismic action, the design spectrum, $S_d(T)$, is defined by the following expressions:

- If $0 \le T \le T_B$ $\quad S_d(T) = a_g\, S\, \left[\frac{2}{3} + \frac{T}{T_B}\left(\frac{2.5}{q} - \frac{2}{3}\right)\right]$
- If $T_B \le T \le T_c$ $\quad S_d(T) = a_g\, S\, \frac{2.5}{q}$
- If $T_c \le T \le T_d$ $\quad S_d(T) = \begin{cases} = a_g\, S\, \frac{2.5}{q}\left[\frac{T_C}{T}\right] \\ \ge \beta a_g \end{cases}$
- If $T_D \le T$ $\quad S_d(T) = \begin{cases} = a_g\, S\, \frac{2.5}{q}\left[\frac{T_C\, T_D}{T}\right] \\ \ge \beta a_g \end{cases}$

where:

- a_g, S, T_b, T_C and T_D are as defined for the elastic spectrum;
- q is the behaviour factor;
- β is the lower bound factor. The β value for use in a country may be found in its National Annex. In all reviewed European countries, the β value is fixed to the recommended one, i.e. β = 0,2.

For the vertical component of the seismic action, the corresponding design response spectrum is given by expressions similar to those for the horizontal components, with the design ground acceleration in the vertical direction, a_{vg} replacing a_g and S taken as being 1.

The reference behaviour factors q assigned to steel-framed structures in EN 1998-1 are summarised in Table 10. These are upper values of q allowed for each system, provided that regularity criteria and capacity design requirements are met. For each system, the dissipative zones are specified in the standard (e.g. beam ends, diagonals, link zones in moment, concentrically braced and eccentrically braced frames, respectively). The α_u/α_1 ratio is related to the redundancy of structure, where α_1 is the first yielding strength of a structural member and α_u is the ultimate strength of the whole structure. It can be obtained from push-over analysis. Alternatively, default values can be used to determine q (as given in parenthesis in Table 10.

The application of q-factor greater than 2 must be combined with sufficient local ductility within dissipative zones. For DCM and q-factor equal to 4, class 1 or 2 cross-sections should be used, whereas for DCH and q-factor greater than 4, only class 1 cross-sections should be employed in dissipative zones. According to the French National Annex to EN 1998, it should be mentioned that the conditions for applying in France the ductility class DCL are fixed in a specific guidance document [28].

The upper limit of q for low dissipative behaviour is a Nationally Determined Parameters, which may be specified in the country National Annex. In all reviewed European countries, this upper limit is fixed to the recommended value, i.e. 1,5.

Table 10: Design concepts, structural ductility classes and upper limit reference values of the behaviour factors for systems regular in elevation

Design concept	Structural ductility class	Behaviour factor q
Non-dissipative	DCL	1,5-2
Moment resisting frames	DCM	4,0
	DCH	5 α_u/α_1 (5,5–6,5)
Concentric braced	DCM	4,0
	DCH	4,0
V-braced	DCM	2,0
	DCH	2,5
Eccentrically braced	DCM	4,0
	DCH	5 α_u/α_1 (6,0)
Dual moment-concentric braced	DCM	4,0
	DCH	4 α_u/α_1 (4,8)

Two aspects are highlighted hereinafter:

- In the Italian standard the medium ductility class is stated as DCB;
- The Swiss standard does not define the ductility class and the type of structure, but it assigns the behaviour factor taking into to account the dissipative character regardless of the type of structure.

5.6 Damage limitation

In addition to general deformation limits defined in EN 1998-1 in terms of a set limiting interstorey drifts, a reduction factor, ν, is also introduced in order to account for the lower return period of the seismic action to consider for the damage limitation requirement.

In Spain, Luxembourg, Belgium, Czech Republic and UK, values of the reduction factor ν to be used to verify the damage limitation conditions are the recommended values of EN 1998-1, i.e. 0,4 for important classes III and IV and 0,5 otherwise.

In France, a reduction factor equal to 0,4 is specified whatever the importance class of buildings.

For Germany, value of the reduction factor ν to be used for buildings of importance class IV is fixed to 0,4. For buildings of importance classes I to III, no values of ν are defined in National Annex since the verification of damage limitation in such cases is not needed in Germany.

In Portugal, the values of the reduction factor ν are 0,4 and 0,55 for the Type 1 and Type 2 seismic actions respectively, whatever the importance classes of buildings.

In Italy, a specific response spectrum is defined for the serviceability limit state. In Switzerland, the serviceability limit state is required only for the structures of class III. No requirements are stated for the other classes.

5.7 Design of non-structural elements

Installations and fittings that do not form part of the structure of buildings are described as non-structural elements. Non-structural elements are for instance building claddings, facades, partition walls or suspended ceilings.

Partition walls in buildings, as well as the building envelope, may be concerned by the obligations of seismic design, insofar as the fall of these elements is likely to cause injuries or to obstruct the exits during the building evacuation. In such cases, they are normally designed using the so-called equivalent static force method, by calculating an equivalent static force (seismic force) F_a acting on the element's centre of gravity.

According to EN 1998-1, the horizontal seismic force (equivalent static force) F_a acting on a non-structural element, may be calculated as follows:

$$F_a = \left[S_a W_a \gamma_a \right] / q_a$$

where:

- W_a weight of the non-structural element [kN].
- S_a seismic coefficient of the non-structural element [-].
- γ_a importance factor of the non-structural element [-].
- q_a behaviour factor of the non-structural element.

For non-structural elements, the importance factor γ_a is generally irrelevant (γ_a = 1.0). Additional safety, i.e. an importance factor $\gamma_a > 1.0$ has to be used only if the element is important for the function of vital systems (lifelines) or if the element may pose major risks in the case of earthquake damage.

According to EN 1998-1, the maximum value for behaviour factor q_a to be used for non-structural elements is equal to either 1.0 or 2.0, depending on their behaviour during earthquake shaking. For example, behaviour factor for cantilever parapets or ornamentation, signs and billboards, chimneys, and tanks are assigned as 1.0, while that for exterior and interior walls, partitions and facades, anchorage elements for false ceilings and light fixtures is assigned as 2.0.

6 TYPICAL FIRE LOADS

In case of fire, the amount of releasable energy is related to the mass of combustible materials. The fire duration depends on this fire load and on the heat release rate. This is related to the composition of the fire load and its exposed surface, the fresh air supply. Fire load density, which is directly linked to fire load, is a dimensioning parameter and many numerical models use it. It is thus necessary to have reliable statistical data. Such data can be found in the large amount of survey performed during the last decades [73], [74], [75], [76], [77] and [78].

In Switzerland, a risk analysis method exists for fire: the SIA 81 method [79]. It uses fire load densities measured in the sixties. Within the framework of an improvement of this method, a new series of measurement was performed in 2005 [80]. 95 industrial and commercial buildings located in 16 Swiss cantons and in Lichtenstein were investigated by ETH Zürich and VKF. An agreement between VKF/ETH and CTICM has been concluded to perform a statistical analysis of these data.

French data were collected within the framework of National Project for Fire Safety Engineering. This project is dedicated to improving the introduction of a performance-based approach in French regulations. It is based on the utilization of design fire scenarios in order to evaluate trial designs, as far as the fulfillment of fire safety objectives is concerned. As the fire load density is an important parameter for fire design, an investigation was conducted and led to the collection of 70 measurements [81]. It concerns public buildings (such as shopping centers, hotels and hospitals) and offices. They were performed by the Cergy-Pontoise University and Nancy Mines High-School in 2006-2007 [82], [83], [84] and [85].

6.1 Measurement method

The methods used for the two datasets are very similar even if they were conducted independently. Four sorts of information were recorded for every room:

- General information (date, company name, room ID…)
- Geometrical parameters:
    - Width, length, height
    - Opening size
    - Wall material
- Fire load data including mass or volume, material, heat of combustion… A distinction is made between mobile and immobile (fixed) fire load. In the French survey process a further distinction is made between objects with a simple geometrical shape (desk, cupboard…) and those where characteristic dimensions are hard to estimate (chair…). For the first category, weight or volume is measured (the easiest is chosen) and for the latter, the fire load is estimated by reference to a list of selected items representative of common products (wood chair, plastic chair…).

There are some differences between ETH/VKF and French protocols. The ETH/VKF protocol is described in detail in [80]. In the ETH/VKF survey prior to the visit, a first contact is established with the company to visit in order to identify what are the major fire loads and a literature search is carried out before the visit.

6.2 Fire load in warehouses

6.2.1 Global analysis

Here, the distinction between storage and production areas is made. Figure 40 shows the Cumulative Distribution Function (CDF) for the 131 production areas. 95% of fire load densities are lower than 2500 $MJ.m^{-2}$. The mean value is 1080 $MJ.m^{-2}$ and the standard deviation 1920 $MJ.m^{-2}$ which leads to a coefficient of variation c_v of 1.78 (ratio standard deviation / mean).

The lognormal law (cf. Table 11) is a common law in many fields where asymmetric distributions are expected. Nevertheless, in structural engineering, the Gumbel distribution (cf. Table 11) is frequently adopted. In Eurocode 1 part 1-2 [86], fire load densities are also supposed to follow the Gumbel law. So, the agreement between these two laws and that dataset is tested via the least squares method. A more complete description of statistical tools used here is available in chapters on probability and statistics in the SFPE Handbook [87]. The best values for the parameters are the couples (m = 6.33;

σ = 1.13) and (μ = 440; β = 530). As shown on Figure 40, the lognormal law gives better agreement (the curves are nearly merged).

Table 11: Lognormal and Gumbel laws.

Probability law	Lognormal	Gumbel (type I)
Probability density function	$f(x)=0 \quad \forall x \le 0$ $f(x)=\frac{1}{\sigma\sqrt{2\pi}}\frac{1}{x}\exp\left(-\frac{(\ln(x)-m)^2}{2\sigma^2}\right) \forall x>0$	$f(x)=\frac{1}{\beta}\exp\left(-\frac{x-\mu}{\beta}\right)\exp\left(-\exp\left(-\frac{x-\mu}{\beta}\right)\right)$ $\forall x \in R$
Cumulative distribution function	$F(x)=\frac{1}{2}+\frac{1}{2}erf\left(\frac{\ln(x)-m}{\sigma\sqrt{2}}\right) \quad \forall x>0$	$F(x)=\exp\left(-\exp\left(-\frac{x-\mu}{\beta}\right)\right) \quad \forall x>0$

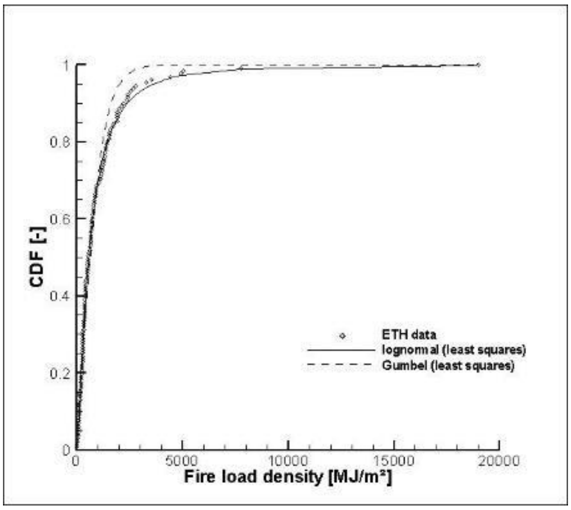


Figure 40: Comparison experimental data – theoretical laws for production rooms

For storage areas, fire load densities are mainly in the range [0 – 35000] MJ.m^{-2} (95% of values) with a mean value of 11874 MJ.m^{-2} and a standard deviation of 32774 MJ.m^{-2} (c_V: 2.76). Fire load densities are greater in storage areas than in production areas (by a factor 10).

The coefficient of variation is really high and is accentuated by the maximum measured value (433710 MJ.m^{-2}). This value was measured in a silo. The geometry of this kind of building is very different from other industrial and commercial buildings. Moreover, the construction provisions are specific regarding fire safety. If this value is omitted, the average would decrease to 9806 MJ.m-2 (17 %) and the standard deviation to 14055 MJ.m^{-2} (twice smaller than before).

6.2.2 Subset analysis

It is possible to divide the dataset into several subsets depending on the company activity. The following subsets are used:

- Chemical industrial,
- Diverse goods,
- Wood processing,
- Cardboard / Paper processing,
- Plastics processing,
- Food processing,
- Metal processing,
- Textiles,
- Offices.

In order to keep at least 15 values in every subset, no distinction is made between storage and production area. Major characteristics for each subset are gathered in Table 12. The analysis shows that the ratio mean value – standard deviation is always high and so, that a lognormal is probably more adequate than a Gumbel law. Means values are distributed on a wide range (factor 10) but extreme values are related to subset with a limited number of samples. Parameters given by the least square method are reported in Table 12 for every subset and the two probability laws.

Table 12: Subsets statistics, Lognormal and Gumbel law parameters.

Sector	Number of samples	Mean value [MJ.m-²]	Standard deviation [MJ.m-²]	c_v	Lognormal law		Gumbel law	
					m	σ	μ	β
Chemical industrial	21	13865	21860	1.58	8.11	1.97	2636	5919
Diverse goods	63	4712	7338	1.56	7.32	1.89	1301	2660
Wood processing	31	6235	8805	1.41	7.80	1.35	1811	2655
Paper processing	56	9880	16430	1.66	8.14	1.77	3064	6265
Plastics processing	41	5051	7865	1.56	7.60	1.27	1530	2040
Food processing	58	7945	12593	1.59	8.00	1.60	2641	4145
Metal processing	37	2461	3983	1.62	6.62	1.68	590	1175
Textiles	16	2609	4221	1.62	6.85	1.30	735	1069
Offices	12	1409	1854	1.32	6.33	1.14	429	546

6.3 Fire loads in commercial centers

Twenty six stores were surveyed, 90% of fire load densities are in the range [0 – 910] MJ.m^{-2}. The mean value and the standard deviation are respectively 571 MJ.m^{-2} and 372 MJ.m^{-2} leading to a coefficient c_v of 0.65. The least squares method gives the couples (m = 6.12; σ= 0.78) and (μ = 370; β = 306) as best parameters respectively for the lognormal and Gumbel law. As shown on Figure 41, the agreement is slightly better with Gumbel law but the lognormal law cannot be rejected with a chi-square test. This result is due to the fact that the coefficient of variation is low. The comparison with available data in the Fire Engineering Guidelines (F.E.G) [87] shows a good agreement with our sample, whereas parameters chosen in the Eurocode for the Gumbel law lead to lower values for the 90% and 95% fractiles (cf. Figure 41).

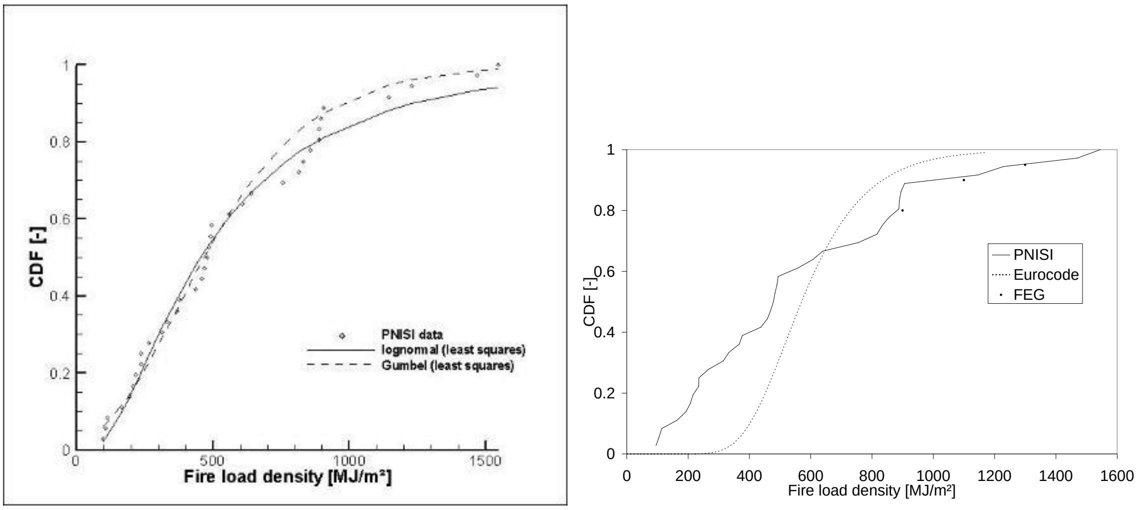


Figure 41: Shopping centers: comparison with theoretical laws (left), comparison with others surveys (right)

The combustible materials were classified in one of the following categories: wood, textiles, paper, plastics and miscellaneous for other combustible objects. However, fire load is mainly composed of wood (44%) and textiles (34%) (Figure 13). It reflects the fact that wood shelves are often used as a way to display goods whatever the type of store. Moreover, many stores are clothing stores, so textiles represent an important category. As for the survey conducted by Carleton University and the National Research Council of Canada [77], wood/paper and textiles are the main combustible materials in clothing stores with respectively 44.5% and 48.1%.

Table 13: Fire load densities extracted from [87]

Occupancy	Average [MJ/m²]	Fractile [MJ/m²]		
		80%	90%	95%
Hospital	230	350	440	520
Hotel bedroom	310	400	460	510
Offices	420	570	670	760
Shops	600	900	1100	1300

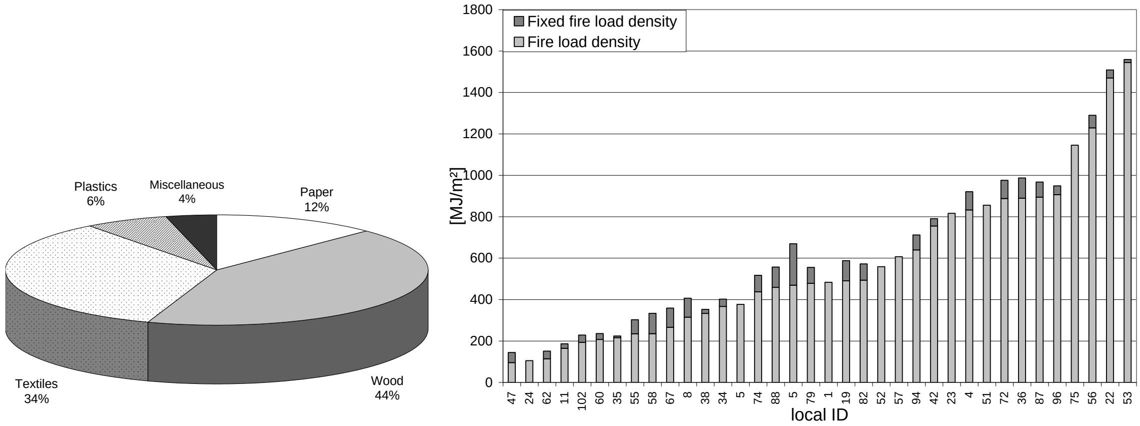


Figure 42: Mean composition (%) for shopping centers (left), measured fire load densities (right)

For shopping centers, the fire load induced by wall linings, floor, doors was measured, this is the fixed fire load density. Taking into account this part increases a little the mean value (622 MJ.m^{-2}) but it does not modify the standard deviation. This fire load density is called as the *total* fire load density.

7 CONCLUSIONS

This report aimed at summing up the main characteristics of single-storey steel buildings, with industrial or commercial use. From collected information, four reference steel-framed structures to be investigated in the scope of the project have been defined, for single medium or large sized compartments, with single or multi-bay frames. In addition, a summary of provisions and main requirements prescribed in both national fire and seismic regulations in relation with industrial and commercial buildings, including warehouses, has been presented. Collected data from regulation reviews will allow selecting the main key input parameters to be considered in the project, especially for the preliminary numerical studies planned in tasks T1.3 and T1.4 of the project, i.e.:

- Main parameters affecting the fire development in target buildings;
- Main parameters for the representation of seismic actions to be considered.

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ANNEX A. FIRE REGULATIONS

A.1. Luxembourg

In Luxembourg, industrial buildings are classified into three categories of increasing fire risk [30]:

- 1st category: establishments inducing low risks:

  Establishments that do not require the use and storage of hazardous materials.

- 2nd category: medium risk establishments:

  Establishments requiring the use of dangerous products and deposits, with high calorific value, in small quantities and not requiring the storage of dangerous products in large quantities.

- 3rd category: establishments involving significant risks:

  Establishments requiring the use of either dangerous products in large quantities, or deposits with a high calorific power in large quantities or allowing a rapid propagation of a possible fire and requiring the storage of dangerous materials.

It should be noted that the ground area of industrial buildings shall not exceed 10 000 m².

Commercial buildings are also classified into three categories [31], as follows:

- 1st category: establishments with a total area higher than 3 000m²;
- 2nd category: establishments with a total area ranging between 2 000 and 3 000m²;
- 3rd category: establishments with a total area lower than 2 000m².

A.1.1. Structural fire requirements

The fire resistance required in single-storey buildings in Luxembourg is indicated in Table 14. Typically, the fire resistance requirements range from 0 minutes (R0) to 60 minutes (R60), depending on the building type, the provision of sprinklers and the compartment size. In addition to fire resistance, specific requirements in terms of overall structural behaviour (no progressive collapse and collapse towards the building inside) are also prescribed for industrial buildings.

It is worth noting that a performance-based approach is permitted to assess the structural fire behaviour, as an alternative to prescriptive rules [32].

Table 14: Structural fire resistance requirements for single-storey industrial and commercial buildings in Luxembourg

Building type	Fire resistance requirement
Industrial building – 1th category	R0
Industrial building 2nd category	R0 for buildings with a ground surface area lower than 2 500m², R30 otherwise
Industrial building 3rd category	R30 for buildings with a ground surface area lower than 2 500m², R60 otherwise
Commercial building	R30 (R0)
( ) if the buildings is fitted with sprinklers	

A.1.2. Compartmentation

The maximum permissible compartment sizes for single-storey buildings are given in following table, according to the building type and the provision of an automatic sprinkler system.

For industrial buildings, it should be noted that the fire resistance rating required for partition fire walls is REI 90. In addition, partition fire walls must extend at least 0.5 m beyond the top surface of the roof. Doors in fire walls must have a one and half hour fire resistance rating (EI 90) and be equipped with self-closing mechanisms.

For commercial buildings, Luxembourgish fire regulations do not impose restrictions on the compartment size. However, shops installed in malls or shopping arcades must be separated by fire walls having a one hour fire resistance rating (REI 60), except if the building is sprinklered. Moreover, fire walls must be fitted with fire doors having one hour fire resistance rating (EI 60).

Table 15: Maximum permissible compartment sizes (m²) and fire wall resistance requirements for single-storey buildings in Luxembourg

Building category	Maximum compartment size (m²)	Fire resistance requirement for fire walls
Industrial - 1th category	1 600 (3 200)	REI 90
Industrial 2nd category	3 200 (6 400)	REI 90
Industrial - 3rd category	5 000 (10 000)	REI 90
commercial	No limit	REI 60 (REI 0)
( ) if the buildings is fitted with sprinklers		

A.1.3. Smoke control

Usually, following buildings must be fitted with a smoke and heat extraction system designed according to the technical guidelines ITM-SST 1552.2 [29]:

- The Industrial buildings of 1th category with a ground area higher than 600 m² and which do not meet a fire resistance R30.
- The Industrial buildings of 2nd category with a ground area higher than 300 m².
- All the industrial buildings of 3rd category.
- The commercial buildings with a ground area higher than 600 m² (or 300 m² for establishments with particular fire risks) as well as all storage rooms with a ground area higher than 600 m² (or 300 m² for the storage rooms with particular fire risks).

Moreover, fire compartments must be divided into “smoke zones” bordered by fixed smoke curtains, with a maximum area of 2 000 m² and a maximum length of 60m. Each smoke curtain must have a minimum height of 0.5 m. For sale areas, the maximum length should not exceed 40m.

A.2. Belgium

In Belgium [33], industrial buildings are classified according to their characteristic fire load, as follows:

- Class A: fire load $\leq$ 350 MJ/m²;
- Class B: 350 MJ/m² < fire load $\leq$ 900 MJ/m²;
- Class C: fire load $\geq$ 900 MJ/m².

A.2.1. Structural fire requirements

Two types of load-bearing structural members are distinguished in the Belgian fire regulation:

- Type I: Structural members, which in the event of collapse, give rise to a progressive collapse beyond the limits of to the fire affected compartment or cause damage to the walls of this compartment.
- Type II: Structural members, which in the event of collapse, give rise to a progressive collapse limited to the fire affected compartment.

The minimum fire resistance required for single-storey buildings is given in Table 16, according to the building occupancy and the type of structural members. According to the Belgian regulation, there may be a reduction of fire resistance requirements limiting the maximum dimensions of fire compartments (see Table 17). Thus, there is no fire resistance requirement for all class A buildings with a single compartment not exceeding 25 000m². For higher calorific loads (class B or C), the maximum permissible compartment size decreases to 5 000m², except for halls with high calorific load (>900 MJ/m²) where it decreases to 2 000 m². These limits can be increase by 60% according to the accessibility features of the building.

It is worth noting that a performance-based approach is permitted to assess the structural fire behaviour, as an alternative to prescriptive rules.

Table 16: Structural fire resistance requirements for single-storey industrial and commercial buildings in Belgium

Building type	Structural fire resistance
Industrial Class A	R60*
Industrial Class B and C	R120*
Commercial	R30**
* requirement for type I structural members ** requirement for all type structural member	

A.2.2. Compartmentation

In Belgium, the area of an industrial building or compartment is limited so that the total fire load of the compartment is less than or equal to 5700 GJ or, if the compartment is fitted with sprinklers to 34,200 GJ.

The maximum permissible compartment sizes in industrial buildings are given in following table, according to the building class, the structural fire resistance and the presence of a sprinkler system.

For building class A, the fire resistance rating required for partition fire walls is EI 60 (or REI 60), while for building class B and C, the fire resistance rating is EI 120 (or REI 120). Moreover, the partition fire wall must extend at least 1 m beyond the top surface of the roof. As an alternative solution, the partition fire wall should be mounted up to the underside of a roof having a fire resistance rating E 60 or R 120 over a 2m length from the wall, on either side of the wall. If the external facades do not have a 1 hour fire resistance rating (E 60 or E 120), the partition fire wall must be laterally extended over a width of 1 m, or 0.5 m projecting from the façade in the line of the wall.

Doors in fire wall must have a one hour fire resistance rating (EI 120) and be equipped with self-closing mechanisms.

Table 17: Maximum permissible compartment sizes (m²) for single-storey industrial and storage buildings in Belgium

Building class	Building without sprinklers		Building fitted with sprinklers	
	Structural fire resistance		Structural fire resistance	
	R0	R30 ou +	R0	R30 ou +
Class A	25 000	25 000	150 000	150 000
Class B	5 000(*)	10 000	40 000	60 000
Class C	2 000(*)	5 000	7 000(*)	30 000
Class C warehouse	5 000(*)	5 000(*)	12 500(*)	30 000
* the maximum permissible compartment size can be increase by 60% according to the accessibility features of the building				

Commercial buildings should be divided into compartments of 2 500 m² maximum size. In case of single compartment, the maximum size may be up to 3 500 m². However, the length of this compartment shall not exceed 90 m. Moreover, it can be noted that the compartment size of buildings can exceed the previous values with the agreement of Belgian authorities. Fire walls separating two compartments must have a half of one hour fire resistance rating (EI 30 or REI 30). Usually, the fire performance of doors in partition fire walls must be equivalent to the fire resistance rating needed for the wall. Doors should be equipped with self-closing mechanisms

A.2.3. Smoke control

In order to limit the development and spread of fire and smoke to the fire-affected compartment, industrial buildings should be equipped with a smoke and heat exhaust system (SHE system) complying with NBN S 21-208-1 [34]. This provision does not apply to:

- industrial buildings or fire compartments of Class A with a total ground area of 10 000 m² or less;
- industrial buildings or fire compartments of class B with a total ground area of 5 00 m² or less

- Fire compartments equipped with a sprinkler system.

In industrial buildings, for fire compartments with a total ground area lower or equal to 2 000 m², the effective area of smoke exhaust system (or smoke vent systems) must be not less than 2% of the roof area (with a minimum area of 1m² per smoke outlet), provided that the height of the stored goods is at most 70% of the height of the smoke exhaust system. More generally, fire compartments must be divided into "smoke zones" bordered by fixed smoke curtains, with a maximum area of 2 000 m² and a maximum length of 60m.

A.3. Spain

In Spain ([35], [36]), industrial and commercial buildings are classified according to:

- The fire risk depending on the industrial activity carried out:
    - <u>Low risk buildings</u>: fire load < 850 MJ/ m^2
    - <u>Medium risk activities</u>: fire load < 3400 MJ/m^2
    - <u>High risk activities</u>: fire load bigger than 3400 MJ/m^2
- The building typology:
    - <u>Type A</u>: industrial occupancy in a building shared with other industrial occupancies or even not industrial ones
    - <u>Type B:</u> industrial occupancies taking up a whole building detached less than 3 metres from any other one
    - <u>Type C:</u> industrial hall occupied completely by one occupancy and detached more than 3 metres from other buildings
    - <u>Types D and E</u>: occupancies covered by open structures without walls.

A.3.1. Structural fire requirements

For general buildings, the structural fire resistances reported in Table 18 are traditionally required. Some reductions are allowed for structural members supporting roofs, in case of lightweight roofs (up to 100 kg/m^2) for type B and type C buildings. Reductions are allowed for sprinklered buildings also. Moreover, no fire resistance is required for single-storey type C buildings detached at least 10 meters from other buildings.

Table 18: Structural fire resistance requirement for single-storey building in Spain

<table>
<tr><th rowspan="2">Building type</th><th colspan="3">Fire risk</th><th rowspan="2">comments</th></tr>
<tr><th>Low</th><th>Medium</th><th>High</th></tr>
<tr><td>Type A</td><td>R90
R60**</td><td>R120
R90**</td><td>Not allowed</td><td></td></tr>
<tr><td>Type B</td><td>R60
R15*
R0**</td><td>R90
R30*
R15**</td><td>R120
R60*
R30**</td><td></td></tr>
<tr><td>Type C</td><td>R30
R0*
R0**</td><td>R60
R15*
R0**</td><td>R90
R30*
R15**</td><td>If building Detached at least 10m from any other building R0</td></tr>
<tr><td colspan="5">* the structure supporting a lightweight roof (less than 100 kg/ m²), the building height is less than 15m and there is no risk of progressive collapse
** if the buildings is fitted with sprinklers</td></tr>
</table>

It can be noted that for industrial and storage buildings, the fire resistance requirement is only needed for the main supporting members (frames), but not for the roof structure.

For commercial buildings, a fire resistance R90 is usually required for the main structure. However, it may be reduced to R30, if the structure is supporting a lightweight roof (up to 100 kg/ m²), the building height is less than 28m and specific requirements in terms of overall structural behaviour (no progressive collapse) are fulfilled.

A.3.2. Compartmentation

The maximum permissible compartment sizes as well as the fire resistance rating required for partition fire walls in industrial and commercial single-storey buildings are given in Table 19.

Table 19: Maximum permissible compartment sizes (m²) and fire wall resistance requirements for single-storey industrial and storage buildings in Spain

Building category	Fire risk			Comments
	Low	Medium	High	
Type A	S < 1000 S*** < 1250 (S < 2000) REI=R structure	300 < S < 500 375 < S*** < 625 (600< S< 1 000) REI=R structure	No permitted	
Type B	S < 4 000 S*** < 5 000 (S < 8 000) REI=R structure	2 500 < S < 3 500 3 125 < S*** < 4 375 (5000 < S< 7 000) REI=R structure	1 500 < S < 2 000 1 875 < S*** < 2500 (3 000 < S < 4 000) REI=R structure	
Type C	S < 6 000 S*** < 7500 (S < 12 000) REI=R structure	3 500 < S < 5 000 4 375 < S*** <6 250 (6 000 < S < 10 000) REI=R structure	2 000 < S < 3 000 2 500 <S*** < 3 750 4 000 < S < 6 000) REI=R structure	No limit if the building is detached more than 10m from any other building and a fire suppression system is installed
( ) buildings fitted with sprinklers *** 50% of the perimeter accessible for fire brigade				

For industrial buildings, the same fire resistance rating is required for partition fire walls and the building structure. When a wall separating two fire compartments reaches the roof, it must be extended upwards at least 1 m, or the roof must have half the fire resistance rating required for the wall in 1 meter width. Moreover, when a wall separating different compartments meets a façade, the fire resistance of the façade shall be at least equal to half of the fire resistance rating required for the fire wall over a width of 1 m. Doors in a fire wall must have half of the fire resistance required for the wall.

For commercial buildings, the maximum permissible compartment size should not exceed 2 500 m², or 10 000 m² if the building is sprinklered. The partition fire wall must have at least one and a half hours fire resistance rating (EI 90 or REI 90). The same fire resistance is required for doors in a fire walls.

A.3.3. Smoke control

Regarding industrial buildings, a smoke and Heat Control System designed according to UNE 23585 [37] must be installed in following cases:

a) Compartments with production activities:

1. With average fire risk and ground area higher than 2 000 m²
2. With high fire risk and ground area higher than 1 000 m²

b) Compartments with storage activities:

1. With average fire risk and ground area higher than 1 000 m²
2. High fire risk and ground area higher than 800 m²

For smaller surface buildings with medium and high fire risks, a smoke and heat control system must be installed respecting the following minimum values for the aerodynamic surface (effective area):

a) Compartment with <u>production</u> activities for medium or high risk buildings:

– 0.5 m² for every 200 m² or fraction;

b) Compartment with <u>Storage</u> activities for medium or high risk buildings:

– 0.5 m² for every 150 m² or fraction.

A.4. France

A.4.1. Structural fire requirements

The minimum fire resistance required for commercial and industrial single-storey buildings in France [14] is given in Table 20. When fire stability requirements are prescribed, they usually range from 15 minutes (R15) to 60 minutes (R60), depending on the building occupancy, the provision of sprinklers, the building height and the compartment size. Stability requirement is only asked for the main structural framework, but not for the secondary structure. Indeed, no structural fire resistance level is generally required for secondary elements, as purlins, if they are not necessary for the stability of the structure (example for lateral buckling of beams).

In addition to fire resistance, structures of single-storey buildings must be designed in such a way that no progressive collapse can occur in compartmented buildings, in order to avoid that the local collapse of the structure, due to a fire, endangers people (occupants and rescue services) or even property, located in a building area still protected from hot gases and smoke. Moreover, the structure of industrial single-storey buildings (including façade elements) must collapse towards the inside of the building to provide a safe situation to firefighters located around the building.

Traditionally [12], commercial single storey buildings do not require structural fire resistance when simultaneously:

- The main structural members are made out of fireproof materials or materials specified in regulation.
- The roof structure is visible from the floor, or supervised by an automatic fire detection system, or
- Fire protected by an automatic fire sprinkler systems, or isolated by a protective screen allowing to meet a fire resistance rating R30.

None of these conditions is required if each room does not receive more than fifty people and has a direct exit on outside.

It should be noted that a performance-based approach is permitted to assess the structural fire behaviour as an alternative to prescriptive rules.

Table 20: Structural fire resistance requirements according to the building type in France

	Structural fire resistance requirements			
	R0	R15	R30	R60
Single-storey buildings	Industrial building [(b),(c)] Commercial building [(a)] (cat. 5)	Industrial building [(b)]	Commercial building[(a)] (cat. 1, 2, 3 and 4) Industrial building [(b)]	Industrial building [(b)]

(a) R0 according to clause C014 of ministerial order of 25 june 1980
(b) According the building occupancy, the quantity of combustible materials, the provision of sprinklers, the building height and the compartment size.
(c) Buildings regulated by the French Labor Code

A.4.2. Compartmentation

According to French fire regulations, fire resistance requirements prescribed for partition fire walls implemented in single-storey buildings are generally in the range of REI 60 to REI 120 (but sometimes up to REI 180). The maximum permissible size of fire compartments depends on the type of building considered.

<u>For industrial/storage buildings:</u>

- The maximum compartment size is equal to 3 000 m² if the building is not sprinklered, or 6 000 m² (and in some cases 12 000 m² where the height of the compartment does not exceed 23m) if the building is sprinklered. The compartment size can be increased more than 12 000m² with the agreement of authorities.

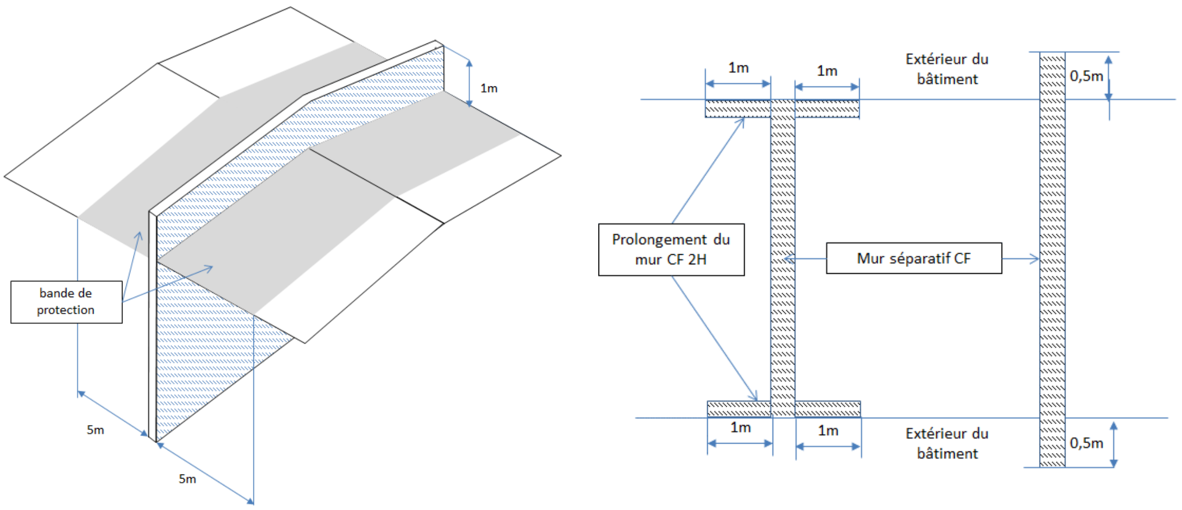
- The partition fire wall must have at least a two hours fire resistance rating (EI 120 or REI120);
- The partition fire wall must extend at least 1 m beyond the top surface of the roof and the roof must be covered, over a minimum width of 5 m on either side of the partition walls, with a fireproof material to prevent the spread of a fire from one compartment to another (Figure 43);
- As an alternative solution, a partition fire wall mounted up to the underside of the roof and a roof portion having an appropriate fire performance can be implemented provided that it leads to the same level of fire separation as the previous solution;
- If the external facades do not have a 1 hour fire resistance rating (EI 60 or REI60), the partition fire wall must be laterally extended over a width of 1 m, or 0.5 m projecting from the façade in the line of the wall (see Figure 29);
- Openings in fire wall must be properly sealed to ensure effective fire resistance ;
- Doors in fire wall must have a 2-hour fire resistance rating (EI 120) and be equipped with self-closing mechanisms.

Table 21: Maximum compartment size for single-storey industrial buildings

	Maximum compartment size (m²)
Not sprinklered building	3 000
sprinklered building	6 000 (12 000*)
*if the compartment height does not exceed 23m. The compartment size can be increased more than 12 000 m² with the agreement of authorities	

For commercial buildings:

- There is no restriction on the compartment size, except for storage areas (with a volume limited to 5 000 m^3);
- The partition fire wall must have at least half an hour fire resistance rating (R)EI 30;
- Usually, the fire performance of doors in partition fire walls must have at least one half hour fire resistance (EI 30l;
- The separation between a public building and third parties must be constituted by a partition fire wall with two hours fire resistance rating (EI120 or REI120). The fire resistance may be increased to three hours if one of the buildings contains an exploitation with particular fire hazards. In addition:
  - When the roofs of buildings are at the same level, one of the following constructional measures must be implemented:
    - The partition fire wall between buildings is extended at least 1m beyond the top surface of the roof. The wall extension must have at least one-hour fire resistance rating (E60 or RE60).
    - One of the roofs is made with building elements having a half-hour fire resistance rating (E30 or RE30) over a 4m length from the wall.
  - When the facade of one of the buildings dominates the roof of the other, one of the following constructional measures is to be realized:
    - The facade is (R)EI 120 over a height of 8 m from the lowest roof. Any openings are closed with building elements having two hours fire resistance rating (EI 120).
    - The lowest roof is made with building elements having half-hour fire resistance rating RE 30 (or RE 60 if one of the building can be considered as a “particular fire hazards building”. In general commercial building without sprinkler is considered like that), over a 4m length from the façade. The fire resistance is increased to one hour, over a length of 8 meters, if one of the buildings present particular fire hazards.



a) Extension of the wall from the roof b) Extension of the wall from the façade

Figure 43: Detailing applicable to fire walls in industrial single-storey building

A.4.3. Smoke control

According to French fire regulations, fire compartments in industrial buildings must be divided into "smoke zones" bordered by fixed smoke curtains, with a maximum area of 1 650m² and a maximum length of 60m. Each smoke curtain must have at least a quarter of an hour fire resistance rating and a minimum height of 1 m. Each smoke zone is equipped with a smoke exhaust system, with an effective area not less than 2% of the smoke zone area.

Regarding commercial buildings, for compartments under 1 000m², the effective area of smoke exhaust system (or smoke vent systems) must be not less than 2% of the ground floor area. For compartments over 1000m², the effective area of smoke exhaust systems must be calculated according to the French Technical Instruction 246 [27]. Compartments over 2 000 m² must be divided into sectors of not over 1 600m², with maximum length of 60m.

A.5. United Kingdom

A.5.1. Structural fire requirements

According to the building regulations in UK [38], single storey buildings do not normally require structural fire resistance. Exceptions may occur where the building structure provides support or stability to elements such as:

- A separating wall;
- A compartment wall or the enclosing structure of a protected zone;
- An external wall, which can needed fire resistance due to the proximity of another building.

In such cases, this fire resistance requirement does not apply to the structure unless its collapse may affect the stability of the wall.

Table 22: Structural fire resistance requirements according to the building use in UK

Building use	Minimum fire resistance	Comments
Industrial (assumed risk profile A2)	R60 (R30)	If building detached from any other building R0
Storage (assumed risk profile A3)	R60 (R30)	If building detached from any other building R0
Commercial (assumed risk profile B3)	R60 (R30)	If building detached from any other building R0
- The values in parentheses are for buildings fitted with sprinklers - The fire resistance requirements only apply when external walls or internal compartment walls requiring fire resistance		

A.5.2. Compartmentation

UK regulations only impose restrictions on the compartment size in single-storey building of B3 risk profile which includes retail purposes (see Table 23). Following the guidance contained in Section 6.5 of BS 9999 [40], the provision of an automatic sprinkler system would allow the design to consider a lower fire growth rate. This will decrease the risk profile of the building, which may lift the limit on compartment size within the building (e.g.: risk profile decreased from B3 to B2 for single storey buildings). Fire walls separating two compartments from one to another must have the fire resistance relating to the highest of the two separated parts, but never more than 60 minutes in the considered case. Elements of structure located within a compartment wall should be able to withstand fire for the same duration than that of the compartment wall. Moreover, the structural members supporting a fire wall must have the same fire resistance as the wall. Fire walls used to form a separated part of a building should run the full height of the building in a continuous vertical plane.

Note that requirements from the insurance UK industry restrict the maximum compartment size in industrial and storage buildings to 7 000m² in the absence of an approved automatic sprinkler system. Where such buildings exceed 7 000m² then either a sprinkler or a smoke control system is required and where such buildings exceed 14 000 m², then both a sprinkler and a smoke control system is required.

Table 23: Maximum permissible compartment sizes according to the building use in UK for single storey building

Building use	Maximum compartment size
Industrial (assumed risk profile A2)	No limit
Storage (assumed risk profile A3)	No limit
Commercial (assumed risk profile B3)	2 000 m² (No limit)
The values in parentheses are for buildings fitted with sprinklers	

Some constructional measures must be implemented at the wall/roof interface to limit the risk of fire spreading to the compartment walls. Especially, if a fire penetrates a roof near a compartment wall, there is a risk that it could spread over the roof to the adjoining compartment. To reduce this risk, a zone of roof 1.5 m wide on either side of the wall should have a covering of designation AA, AB or AC (national class) or BROOF(t4) (European class) on a substrate or deck of a material of limited combustibility, as set out in Figure below.

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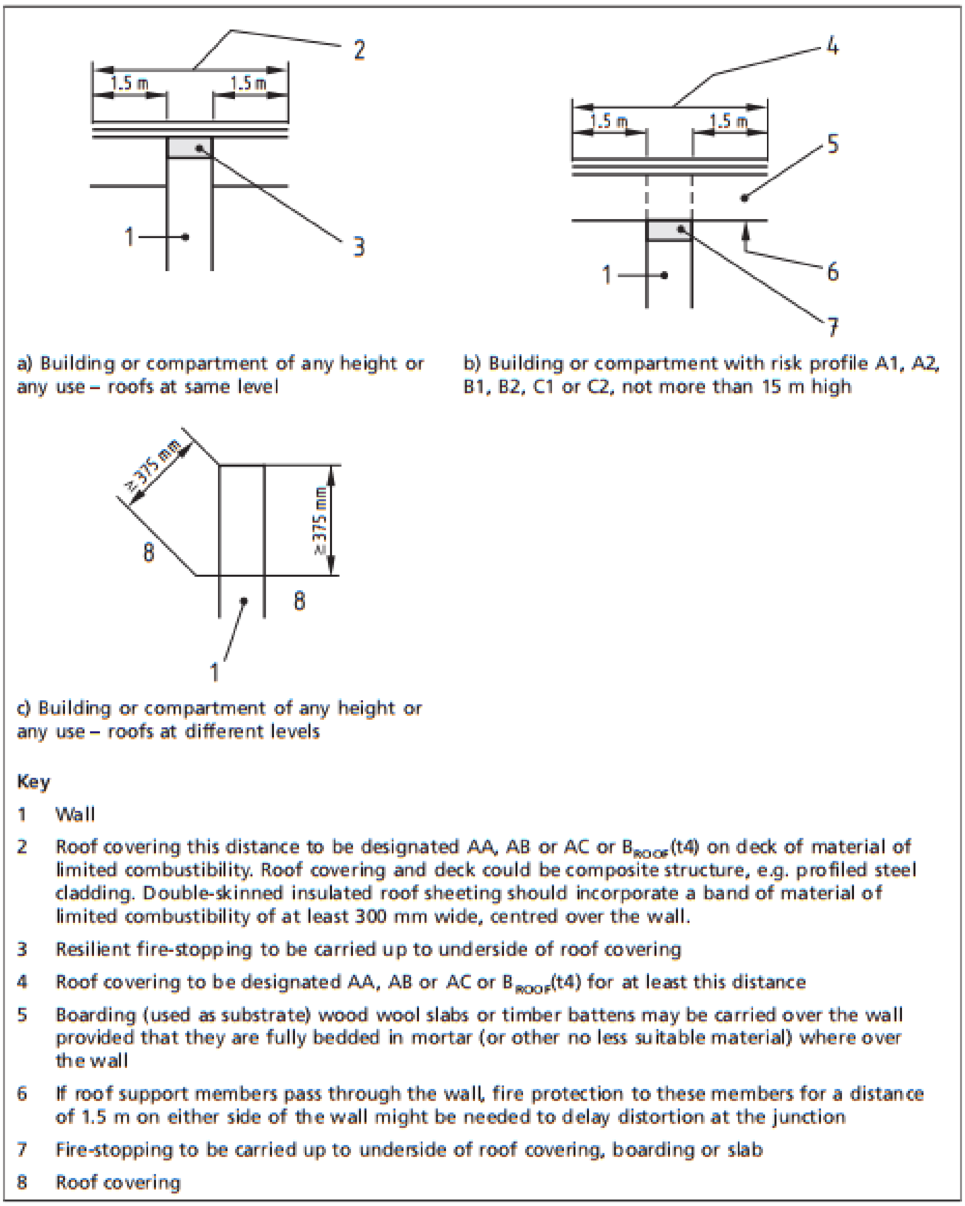

Figure 44: Junction of a compartment wall with a roof

The fire resistance of fire doors in a compartment wall separating buildings has to be as for the wall in which the door is fitted. In other cases, it should be in general half of the period of the fire resistance of the wall in which it is fitted, but not less than 30 minutes.

A.5.3. Smoke control

Smoke and Heat Control Systems may be designed according to different part of EN 12101 or BRE's report [39], if relevant.

Usually, "Smoke zones" are arranged in buildings, with a maximum area of 2 000/2 600m² and a maximum length of 60m in any direction. In shopping centres, the reservoir is assumed to be 50% in the shop and 50% in the mail.

A.6. Portugal

In Portugal [41], buildings are classified in twelve types, according to their functions and uses:

- Type I «Dwelling»
- Type II «Car parks»

- Type III «Administrative»
- Type IV «Schools»
- Type V «Hospitals and nursing homes»
- Type VI «Theatres/cinemas and public meetings»
- Type VII «Hotels and restaurants»
- Type VIII «Shopping and train stations»
- Type IX «Sports and leisure»
- Type X «Museums and art galleries»
- Type XI «Libraries and archives»
- Type XII «Industrial, workshops and storage»

If a building cannot be classified in one of these 12 Utilization-Types, it can be classified as "atypical danger" and Fire Safety Engineering can be used as well as Performance Based Design.

Each type of building is classified into four risk categories depending on their construction characteristics, height, gross area, number of floors below the reference floor, expected number of occupants, etc.

According to the previous list, commercial, Industrial, factory or storage buildings fall in the utilization type VIII and XII. The risk categories associated with such types of building are reported in Table 24 and Table 25 respectively.

Table 24: Risk categories for utilization-type XII "Industrial, workshops and storage buildings"

Category	Criteria referred to the utilization type XII	
	Inside building	Open air
	Modified fire load for UT XII	Modified fire load for UT XII
1st 2nd 3rd 4th	(*)≤500 MJ/m² (*)≤5000 MJ/m² (*)≤15000 MJ/m² (*)>15000 MJ/m²	(*)≤500 MJ/m² (*)≤5000 MJ/m² (*)≤15000 MJ/m² (*)>15000 MJ/m²
(*) For UT XII, intended for storage buildings only the limits for the modified fire load on this table should be 10 times higher than those indicated.		

Table 25: Risk categories for utilization-type VIII – Commercial buildings and transport stations

Category	Criteria referred to the utilization type VIII	
	Height	Effective (Number of occupants)
1st	≤9m	<100
2nd	≤28m	≤1000
3rd	≤28m	≤5000
4th	>28m	>5000

A.6.1. Structural fire requirements

According to the Portuguese fire regulations, single-storey buildings of 1st risk category, do not normally require structural fire resistance. On the other hand, a structural fire resistance R60 is required for other risk categories (2nd 3rd or 4th risk), as indicated in Table 26.

Table 26: Structural fire resistance requirements for commercial and industrial single-storey buildings according to risk category in Portugal

Utilization type	Risk category	
	1st	2nd to 4th
Commercial buildings	R0	R 60
Industrial, workshops and storage buildings	R0	R 60

A.6.2. Compartmentation

The maximum permissible compartment sizes as well as the fire resistance rating required for partition fire walls in industrial buildings are given in Table 27. In this table, different cases are distinguished:

- Case I corresponding to a building of utilization type XII that coexists with buildings from other utilizations types;
- Case II corresponding to a building exclusively for utilization type XII that shares a common façade with other buildings reserved for housing or that are open for public;
- Case III corresponding to a building exclusive of utilization type XII which may share a common façade with buildings of utilization type XII or that ensures an adequate spacing to buildings reserved for housing or that are open for public;
- Case IV corresponding to a detached building exclusively for utilization type XII without floors below the floor plan, ensuring an adequate spacing.

It should be noted that there is no more restrictions on the compartment size if the building is detached more than an appropriate distance, that depends on the risk category, from any other building. In addition, the maximum permissible areas indicated in Table 27.

Table 27: Maximum compartment sizes for utilization type XII for Industrial buildings in Portugal

Case	Risk category of utilization type XII			
	1st	2nd	3rd	4th
I	1 600 m^2 REI 60	800 m^2 REI 60	400 m^2 REI 60	400 m^2 REI 60
II	6 400 m^2 REI 60	2 400 m^2 REI 60	800 m^2 REI 60	400 m^2 REI 60
III	12 800 m^2 REI 60	4 800 m^2 REI 60	2 400 m^2 REI 60	1 200 m^2 REI 60
IV	No limit			

Regarding commercial buildings, the maximum compartment size is limited to 1 600 m² if the building is not sprinklered, or 3 200 m² if the building is fitted with sprinklers. In case of single compartment, the maximum size may be up to 8 000 m² or 16°000 m² according to the circulation features of the building. The partition fire wall must have at least a one hour fire resistance rating (R)EI 60.

Usually, the fire performance of doors in partition fire walls must be equivalent to the fire resistance rating needed for the wall.

The external walls of buildings of utilization type XII must guarantee at least the standard fire resistance class EI 60 or REI 60, and the spans must be equipped with fixed elements of E 30 or self-closing elements E 30 C when they face other buildings at a distance inferior to that indicated in following table:

Table 28: Minimum distances between buildings in Portugal

Risk category for utilization type XII	Greatest of heights of buildings "H"	Distance "L"
1st	$H \leq 9m$ $H > 9m$	L=4m L=8m
2nd	$H \leq 9m$ $H > 9m$	L=8m L=12m
3rd or 4th	Any	L=16m

Whenever the distances for the 2nd, 3rd and 4th risk categories are less than half of those referred to in Table 28, the standard fire resistance values of the external walls should be changed to EI 90 or REI 90 and the openings in them should be protected by E 45 or self-closing E 45 C elements.

The existence of openings in external walls above the roofs allocated to the utilization type XII of other buildings, or of other parts of the same building, is only permitted if the covering materials used guarantee fire class A1 in a band with a width of 8m measured from the wall.

If there are glazed elements on the roof referred to in the previous number, located within the 8m band mentioned, they must be fixed, have a standard fire resistance class E 60 or higher, and be 4m away from the façade above.

A.6.3. Smoke control

The spaces of Utilization Type XII of the 2nd risk category or higher, allocated to storage with an area greater than 400 m2, regardless of their location in the building, they must possess smoke control system.

Fire compartments must be divided into "smoke zones" bordered by fixed smoke curtains, with a maximum area of 1 600m² and a maximum length of 60m.

In Portugal, Smoke and Heat Control Systems may be designed according to different part of EN 12101.

A.7. Netherlands

A.7.1. Structural fire requirements

The Buildings Decree 2012 (Bouwbesluit 2012) [42] prescribes the minimum construction *requirements* that all structures in the Netherlands must meet. The fire resistance of building is depending on the type of building and the maximum floor of area of use. It can be different for new constructions, renovation or existing buildings. The main objective is to ensure safe egress for occupants in the event of a fire. The structure shall be capable, in case of fire, of being left and searched for a reasonable amount of time without danger of collapse. Therefore, collapse or failure of structural element should not happen before people are evacuated.

The Table 29 gives information about fire resistance required for buildings ranging in height from – 8 m to + 70 m. As can be seen, single storey buildings do not have specific stability requirements when exposed to fire.

Table 29: Structural fire resistance requirements according to the building type

Building type	Industrial function	Office function	Shop function
New construction / Renovation	○ Floor of area of use > 5 m above measurement level (mostly ground level) or < 5 m below measurement level → R90 ○ If according to NEN 6090 determined permanent fire load in compartment ≤ 500 MJ/m² → R60 (30 minutes less) ○ Floor/staircase/ramp over or under which an escape route leads → R30 when fire in sub-compartment in which escape route is not located	○ Floor of area of use > 5 m above measurement level or < 5 m below measurement level → R90 ○ according to NEN 6090 determined permanent fire load in compartment ≤ 500 MJ/m² → R60 (30 minutes less) ○ Floor/staircase/ramp over or under which an escape route leads → R30 when fire in sub-compartment in which escape route is not located	○ Floor of area of use > 5 m above measurement level or < 5 m below measurement level → R90 ○ according to NEN 6090 determined permanent fire load in compartment ≤ 500 MJ/m² → R60 (30 minutes less) ○ Floor/staircase/ramp over or under which an escape route leads → R30 when fire in sub-compartment in which escape route is not located
Existing building	○ Floor of area of use > 5 m above measurement level → R30 ○ Floor/staircase/ramp over or under which an escape route leads → R20 when fire in sub-compartment in which escape route is not located		

A.7.2. Compartmentation

The Table 30 gives information about the maximum surface for fire compartmentation according to the article 2.83, 2.85 and 2.89 of national building code – section 2.10.

Table 30: Maximum compartment size in Netherlands

Max surface / Fire compartmentation			
Building type	Industrial function	Office function	Shop function
New construction / Renovation	Max usable area : **2.500 m²** ○ A fire compartment does not extend over more than one plot. ○ A services space with a usable area exceeding 50 m2, or a services space where one or more heating appliances with a total nominal power exceeding 130 kW are installed, shall constitute a separate fire compartment.	Max usable area : **1.000 m²** ○ A fire compartment does not extend over more than one plot. ○ A services space with a usable area exceeding 50 m2, or a services space where one or more heating appliances with a total nominal power exceeding 130 kW are installed, shall constitute a separate fire compartment ○ In the case of a fire compartment of an industrial function with an area of use > 1000 m2 , the mentioned maximum area does not apply to one or more ancillary functions with a total area of use of ≤ 100 m2 located in that fire compartment.	
Existing building	Max usable area : **3.000 m²** - A fire compartment does not extend over more than one plot. - A technical room with an area of use > 100 m^2 or a technical room in which one or more combustion appliances with a total nominal load > 160 kW are installed, is a separate fire compartment.	Max usable area : **2.000 m²** - A fire compartment does not extend over more than one plot. - A technical room with an area of use > 100 m^2 or a technical room in which one or more combustion appliances with a total nominal load > 160 kW are installed, is a separate fire compartment. - In the case of a fire compartment of an industrial function with an area of use > 2000 m^2 , the mentioned maximum surface does not apply to one or more ancillary located in that fire compartment.	

The fire resistance of a fire compartment, with respect to spread of the fire to another fire compartment, to an enclosed space through with an additionally protected escape route passes, to a non-enclosed escape route, or to the lift shaft of a fire-fighting lift, can be find on the table below.

Table 31: Fire resistance of fire compartment in Netherlands

<table>
<tr><th colspan="4">Fire resistance of a fire compartment</th></tr>
<tr><th>Building type</th><th>Industrial function</th><th>Office function</th><th>Shop function</th></tr>
<tr><td rowspan="2">New construction</td><td colspan="3">o 60 minutes determined according to NEN 6068(1)

o 30 minutes if:
- The enclosed spaces as are located on the same plot, and
- No floor of area of use > 5 m above measurement level</td></tr>
<tr><td>30 min requirement is not valid when:
o compartment with area of use > 1.000 m2
o technical room (Only for light industry function for commercial animal keeping)
o separating of space through which a safety escape route passes</td><td>30 min requirement is not valid for:
o separating of space through which a safety escape route passes</td><td>30 min requirement is not valid for:
o separating of space through which a safety escape route passes</td></tr>
<tr><td>Renovation</td><td colspan="3">According to new construction rules, but level → legally acquired level and a minimal resistance to fire movement between spaces of 30 minutes.</td></tr>
<tr><td>Existing building</td><td>20 minutes determined according to NEN 6068</td><td>20 minutes determined according to NEN 6068</td><td>20 minutes determined according to NEN 6068</td></tr>
</table>

A fire compartment shall be divided into one or more fire subcompartments or traffic spaces through which a protected escape route passes. A protected escape route shall not be located in a fire subcompartment. The fire resistance of a fire subcompartment, with respect to another space in the fire compartment, as determined in accordance with NEN 6068, shall be at least 20 minutes, where the calculation of the fire resistance in relation to the dividing function of a partition shall be based exclusively on the assessment criterion of flame retardancy with respect to the sealing. It is worth noting that there is no need to divided industrial and shopping buildings into one or more smaller subcompartments.

If compartments need to be bigger than allowed according to the requirements in the Building Decree, than an equivalent level of safety should be achieved based on the equivalence principle.

A.7.3. Smoke control

Smoke control systems (as well as sprinklers) are not prescribed in the Dutch building code, but they can play a role (like any other measure) in applying equivalence as referred to in Article 1.3 of the Dutch building code:

"Any of the provisions of Chapters 2 to 7 will not need to be complied with if the structure or its use, other than by application of the relevant provision, provides at least the same level of safety, health protection, usefulness, energy efficiency, and environmental protection as envisaged with the provisions of those chapters.

An equivalent solution as referred to in the first paragraph shall be maintained during the use of the structure.

A.8. Switzerland

According to the Swiss national fire regulation [44], buildings are classified according to their functions and uses and their overall height, distinguishing:

- Low-rise buildings: up to 11 m in total height
- Medium height buildings: up to 30 m in total height

- High buildings: over 30 m in total height

A.8.1. Structural fire requirements

In Switzerland, single-storey buildings do not require normally any fire resistance. However, this requirement may changes if the building structure supports a separating firewall and the collapse of this structure may affect the stability of the wall.

A.8.2. Compartmentation

The maximum permissible compartment sizes as well as the fire resistance rating required for partition fire walls in industrial and commercial buildings are given in the following table. It is worth noting that the size limits indicated in this table may be widely increased if the building is equipped with a smoke exhaust system.

The fire resistance rating required for partition fire walls is EI 30 for both commercial and industrial single-storey buildings. The fire resistance may be increased to REI 90 if the building have high fire load. Doors in fire wall must have a one hour fire resistance rating (EI 30).

It should be noted that some design detailing regarding the connection of fire walls to the building envelope are given in a explanatory note published by the AEAI.

Table 32: Maximum permissible compartment sizes (m²) and fire wall resistance requirements for single-storey industrial and storage buildings in Switzerland

Building type	Maximum compartment size (m²)	Fire resistance requirement for fire walls
Industrial building	2400 4800*	EI 30
Commercial building	600 600*	EI 30
* Building fitted with an automatic fire-extinguishing system		

A.8.3. Smoke control

According to the national Swiss fire codes, in order to enhance the efficiency of firefighting and rescue operations, Smoke extraction system should be installed in buildings, in particular in large buildings. Smoke and Heat Control Systems may be designed according to different part of EN 12101 or any other relevant codes.

It should be noted that the Swiss fire protection regulations made smoke ventilation simulations mandatory for large public and private buildings.

A.9. Poland

In Poland [45], buildings are divided into 3 categories based on the usage type, with buildings intended for:

- people (ZL);
- industrial and storage (PM);
- and livestock buildings (IN).

A.9.1. Structural fire requirements

The fire resistance required in Poland for industrial single-storey buildings is indicated in Table 33. Typically, the fire resistance requirements for the main structure range from 0 minutes (R0) to 240 minutes (R240), according to the fire load density. It can be noted that the fire resistance required for the roof supporting structure is lower, ranging from 0 minutes to 30 minutes (R30) only.

For commercial buildings, a fire resistance R30 is usually required for the main structure, but not for the roof supporting structure. No fire resistance is required if the building is sprinklered.

Table 33: Structural fire resistance requirements for industrial single-storey buildings in Poland

Maximum fire load density in fire compartment Q [MJ/m²]	Main structure	Roof supporting structure
$Q \leq 500$	R0	R0
$500 < Q \leq 1000$	R30*	R0
$500 < Q \leq 1000$	R60*	R15*
$2000 < Q \leq 4000$	R120*	R30*
$Q > 4000$	R240*	R30*
* R0 for compartment with an area higher than 1000m² and equipped with a smoke exhaust system		

A.9.2. Compartmentation

The maximum permissible compartment size in industrial single-storey buildings is given in Table 34, according to the maximum fire load density present in the fire compartment. It is worth noting that the maximum permissible areas indicated here may be increased by 100% if the building if fitted with sprinklers or by 50% is the building is equipped with a smoke exhaust system. Moreover, there is no more restriction by equipping the building with a sprinkler system and an automatic smoke exhaust system.

For commercial building, the compartment size is limited to 10 000 m². This limit may be increased by 100% if the building is fitted with sprinklers or equipped with a smoke exhaust system, or by 200% with both measures. The partition fire walls must have at least one hour fire resistance rating (REI 60). Doors in a fire wall must have half of the fire resistance required for the wall.

The fire partition walls must be laterally extended over a width of at least 0.3 m beyond the building facade or a vertical band with a fireproof material of a width of at least 2 m and having a fire resistance EI60 must be installed over the entire height facade.

In buildings with a fire-spreading roof, the partition fire walls must extend at least 0.3 m beyond the top surface of the roof or a horizontal band with a fireproof material of a width of at least 1 m and having a fire resistance EI60 must be installed under the roof.

Table 34: Maximum permissible compartment sizes (m²) and fire wall resistance requirements for industrial single-storey buildings in Poland

Maximum fire load density in fire compartment Q [MJ/m²]	Maximum compartment size (m²)		Fire resistance requirement for fire walls
	non-explosive atmospheres	explosive atmospheres	
$Q \leq 500$	20 000	8 000	REI 0
$500 < Q \leq 1000$	15 000	6 000	REI 30
$500 < Q \leq 1000$	8 000	4 000	REI 60
$2000 < Q \leq 4000$	4 000	2 000	REI 60
$Q > 4000$	2 000	1 000	REI 120

A.9.3. Smoke control

In Poland, where required, smoke and Heat Control Systems may be designed according to different part of EN 12101 or any other relevant codes.

A.10.Italy

In Italy the juridical framework is quite articulated depending on the type of activity under consideration, as illustrated in Figure 45. Indeed, the code differentiate the type of activity into:

- Activities that are regulated by specific technical rules, such as the commercial buildings.
- Activities that are not regulated by specific technical rules but that can be subjected to the control of the fire brigades, such as the industrial buildings.

The main difference between activities that are regulated by specific technical rules is the possibility to apply the Natural Fire Safety Concept (NFSC) with or without derogation. A derogation procedure must be employed when specific ministerial decrees are used (for instance, for commercial buildings DM 27/7/2010)[55], whereas it is possible to avoid it when specific technical rules (RTV 23/11/2018) [61] that belong to the fire prevention code (DM 3/8/2015) [55] are followed. It is clear that a derogation procedure entails a longer administrative path and inevitably does not encourage the use of the natural fire safety concept.

For activities that are not regulated by specific technical rules but that can be subjected to the control of the fire brigades, there are two ways to evaluate the fire resistance: i) through the fire prevention code (DM 3/8/2015) [55]; ii) through ministerial decrees (DM 9/3/2007[58] and DM 9/5/2007[57]). In this document the rules in the framework of the fire prevention code (DM 3/8/2015) [55], which is the most recent regulation with its latest update dated back to 18 October 2019 (DM 18/10/2019)[56], will be presented.

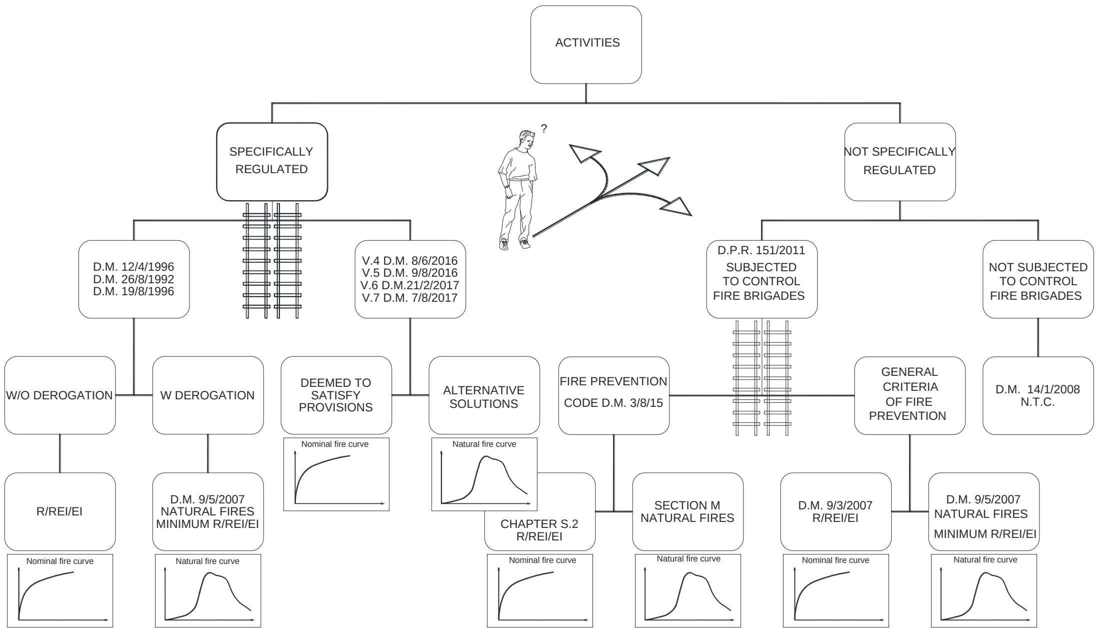

Figure 45: Italian juridical framework relative to fire prevention (Adapted from [60]).

A.10.1. Structural fire requirements

By following the fire prevention code (DM 3/8/2015) [55], the first step is to assign to the building a performance level, as shown in Figure 35. The attribution criteria are reported in Table 36, for which the relevant risk profiles for life and goods are illustrated in Table 37 and Table 39, respectively. In Table 38, examples of activities and the associated life risk profile are given.

Table 35: Performance levels in terms of fire resistance

Performance level	Description
I	No consequences due to structural collapse
II	Fire resistance requirements for a sufficient period of time that allows the safe evacuation from the building
III	Fire resistance requirements for a period of time that is adequate with respect to the fire duration
IV	Fire resistance requirements that, after the fire, guarantee a limited damage of the building
V	Fire resistance requirements that, after the fire, guarantee the complete functionality of the building

Table 36: Attribution criteria of the performance levels in terms of fire resistance

Performance level	Description
I	Buildings for which all the following conditions are satisfied: • Compartmented with respect to the adjacent structures and separated from the other buildings. A structural collapse does not endanger the other buildings. • There is only one responsible for the activities within the building with a risk profile for goods $R_{goods} = 1$. • There are no stable occupants, but only occasionally.
II	Buildings for which all the following conditions are satisfied: • Compartmented with respect to the adjacent structures. • Structurally separated from the other buildings or if not, the collapse of one portion does not affect the other parts and the adjacent area, i.e. collapse towards inside the perimeter. • There is only one responsible for the activities within the building with a risk profile for goods $R_{goods} = 1$ and a risk profile for life R_{life} comprised within A1, A2, A3, A4. • Occupant density ≤ 0.2 people/m^2. • Limited people with disabilities. • Floors located between -5 m and 12 m
III	Buildings not included in the previous two performance levels.
IV, V	Based on the stakeholder’s requirements for buildings with high strategic importance.

Table 37: Determination of R_{life}

Characteristics of the occupants	Fire growth rate			
	Slow	Medium	Fast	Ultra-fast
Occupants that are awake and are familiar with the building	A1	A2	A3	A4
Occupants that are awake and are not familiar with the building	B1	B2	B3	Not allowed
Occupants can be asleep	C1	C2	C3	Not allowed
• Individual and long-term activities	Ci1	Ci2	Ci3	Not allowed
• Managed and long-term activities	Cii1	Cii2	Cii3	Not allowed
• Managed and short-term activities	Ciii1	Ciii2	Ciii3	Not allowed
The occupants receive medical treatment	D1	D2	Not allowed	Not allowed
The occupants are in transit	E1	E2	E3	Not allowed

Table 38: Examples of activities and relative life risk profile.

Type of activity	R_{life}
Gymnastic hall	A1
Private car park	A2
Office not open to the public, archive	A2-A3
Commercial activity not open to the public	A2-A4
School laboratory, server room	A3
Productive activity, research laboratory, warehouse	A1-A4
Warehouse of dangerous substances	A4
Restaurant, waiting room	B1-B2
Public car park	B2
Office open to the public, conference hall, disco club, theatre, cinema, museum	B2-B3
Commercial activity open to the public	B2-B4
Residential building	Ci2-Ci3
Residence hall, dormitory	Cii2-Cii3
Hotel room	Ciii2-Ciii3
Hospital	D2
Railway station, airport, underground station	E2

Table 39: Determination of R_{goods}

		Activities with cultural heritage	
		No	Yes
Activities with strategic relevance	No	R_{goods} = 1	R_{goods} = 2
	Yes	R_{goods} = 3	R_{goods} = 4

For a deemed-to-satisfy design approach that employs nominal fire curves the structural fire requirements are classified according to the performance level. In particular:

- **Performance level I:**
  - The structure must be separated from other buildings with a distance that, at least, is its height.
  - The fire propagation should be limited to the other structures based on compartmentation measures described in Paragraph A.10.2.
  - No structural fire resistance is required.
- **Performance level II:**
  - The structure must be separated from other buildings with a distance that, at least, is its height.
  - The minimum fire resistance class is 30 min or less based on the design fire load reported in Table 40.
- **Performance level III:**
  - The minimum fire resistance class is based on the design fire load reported in Table 40.

Table 40: Minimum fire resistance class

Design fire load	Minimum fire resistance class
$q_{f.d} \leq 200$ MJ/m^2	No requirement
$q_{f.d} \leq 300$ MJ/m^2	15 min
$q_{f.d} \leq 450$ MJ/m^2	30 min
$q_{f.d} \leq 600$ MJ/m^2	45 min
$q_{f.d} \leq 900$ MJ/m^2	60 min
$q_{f.d} \leq 1200$ MJ/m^2	90 min
$q_{f.d} \leq 1800$ MJ/m^2	120 min
$q_{f.d} \leq 2400$ MJ/m^2	180 min
$q_{f.d} > 2400$ MJ/m^2	240 min

- **Performance level IV:**
  - The fire resistance requirements should be taken as the ones defined for the Performance level III.
  - In order to limit the damage, the following deformability limits should be satisfied:
    - $\delta_{v.max}/L$ = 1/100 the ratio between the maximum vertical deformability and the span of beams and mono-directional slabs.
    - $\delta_{v.max}/L$ = 1/100 the ratio between the maximum vertical deformability and the minimum span of bi-directional slabs.
    - $\delta_{h.max}/h$ = 1/100 the ratio between the maximum interstorey drift and the storey height.
  - The joints between compartmentation elements should be designed in order to allow the displacements in fire conditions. In particular, it is possible to employ joints that are tested in accordance with EN 1366-4.
  - In terms of compartmentation capability, the resistance to smoke should be at least EI S_{100} and the mechanical fire resistance to impact M of the walls should be equal to class determined for Performance level III.

- **Performance level V:**
  - The fire resistance requirements and the deformability limits should be taken as the ones defined for Performance level IV.
  - In order to limit the damage, the deformability limits for each structural element should be satisfied in accordance with the Italian Building Code (NTC, 2018)[59].

For industrial buildings, the performance level that is typically determined is Performance level II. Thus, a minimum fire resistance class of 30 min or less is assigned.

For commercial buildings, there are specific technical rules (RTV 23/11/2018)[61] that belong to the fire prevention code (DM 3/8/2015) [55].

The classification is conducted according to the building net carpet area and the storey height as shown Table 41 and Table 42.

Table 41: Classification according to the net carpet area

AA	$A \le 1500\ m^2$
AB	$1500\ m^2 < A \le 3000\ m^2$
AC	$3000\ m^2 < A \le 5000\ m^2$
AD	$5000\ m^2 < A \le 10000\ m^2$
AE	$A > 10000\ m^2$

Table 42: Classification according to the storey height.

HA	$-1\ m < h \le 6\ m$
HB	$-5\ m < h \le 12\ m$
HC	$-10\ m < h \le 24\ m$
HD	If not in other classes

Based on the aforementioned classification, the minimum fire resistance requirements are shown in Table 43.

Table 43: Fire resistance requirements for commercial buildings

Compartments	Activity classification			
	HA	HB	HC	HD
Overground	30 min(*)	60 min		90 min
Underground	-	90 min		

(*) For AA and AB activities that occupy a single-storey building that starts between -1 m and 1 m and that is compartmented from other buildings it is possible to attribute a class of 15 min.

A.10.2. Compartmentation

According to the fire prevention code, the compartmentation measures are selected according to the performance levels shown in Table 44 that are assigned based on the criteria highlighted in Table 45.

Table 44: Performance levels in terms of compartmentation

Performance level	Description
I	No requirement
II	The fire spread is prevented: • towards other activities • within the same activity
III	The fire spread is prevented towards other activities. The fire and cold smoke spread are prevented within the same activity.

Table 45: Attribution criteria of performance levels in terms of compartmentation

Performance level	Description
I	Not allowed for activities subjected to the control of the fire brigades.
II	Activities that are not comprised in the other criteria.
III	Based on the risk profile of the activity and of the nearby activities. It can be applied to compartments used for occupants that sleep or that receive medical treatment.

The maximum area of the compartment depends on the life risk profile, shown in Table 37, and the values of the maximum compartment gross area are reported in Table 46 based on the location of the compartment in the building. The fire resistance shall satisfy the requirements given in Paragraph A.10.1.

Table 46: Maximum compartment gross area in m^2.

R_{life}	Location of the compartment								
	<-15m	<-10m	<-5m	<-1m	≤12 m	≤24 m	≤32 m	≤54 m	>54 m
A1	2000	4000	8000	16000	No limitation	32000	16000	8000	4000
A2	1000	2000	4000	8000	64000	16000	8000	4000	2000
A3	Not allowed	1000	2000	4000	32000	4000	2000	1000	Not allowed
A4	Not allowed	Not allowed	Not allowed	Not allowed	16000	Not allowed	Not allowed	Not allowed	Not allowed
B1	Not allowed	2000	8000	16000	64000	16000	8000	4000	2000
B2	Not allowed	1000	4000	8000	32000	8000	4000	2000	1000
B3	Not allowed	Not allowed	1000	2000	16000	4000	2000	1000	Not allowed
Cii1 Ciii1	Not allowed	Not allowed		2000	16000	8000	8000	8000	4000
Cii2 Ciii2	Not allowed	Not allowed	Not allowed	1000	8000	4000	4000	2000	2000
Cii3 Ciii3	Not allowed	Not allowed	Not allowed	Not allowed	4000	2000	2000	1000	1000
D1	Not allowed	Not allowed	Not allowed	1000	2000	2000	1000	1000	1000
D2	Not allowed	Not allowed	Not allowed	1000	2000	1000	1000	1000	Not allowed
E1	2000	4000	8000	16000	No limitation	32000	16000	8000	4000
E2	1000	2000	4000	8000	No limitation	16000	8000	4000	2000
E3	Not allowed	Not allowed	2000	4000	16000	4000	2000	Not allowed	Not allowed

(*) For significant environmental profile risk R_{env} the maximum compartment gross area is reduced to 50%

Depending on the life risk profile, typically A1-A4, the maximum compartment gross area of industrial single-storey buildings with height less than 24 m ranges from 4000 m^2 to 32000 m^2.

For commercial buildings, there are specific technical rules (RTV 23/11/2018)[61] within the fire prevention code (DM 3/8/2015). For commercial areas that are open to the public (TA) the additional limitations and fire prevention measures are provided based on the location of the compartment.

Table 47: Additional compartmentation measures for commercial buildings open to the public

Compartment location	Limitations	Additional fire prevention measures
-1 m ≤ h ≤ 12 m	None	None
h > 12 m	None	• Detection and alarm of Performance level IV • Protected evacuation paths
-5 m ≤ h ≤ -1 m	AA with qf ≤ 600 MJ/m^2	None
h > 12 m	None	• Fire control of Performance level IV • Detection and alarm of Performance level IV • Smoke and heat control of Performance level III

A.10.3. Smoke control

According to the fire prevention code, the smoke and heat control measures are selected according to the performance levels shown in
Table 48 that are assigned based on the criteria highlighted in Table 49.

Table 48: Performance levels in terms of smoke and heat control.

Performance level	Description
I	No requirement
II	Smoke and heat shall be exhausted from the compartments in order to facilitate the operations of the fire brigades
III	The fire spread is prevented towards other activities The fire and cold smoke spread are prevented within the same activity

Table 49: Attribution criteria of performance levels in terms of smoke and heat control

Performance level	Description
I	Compartments that satisfy all following conditions: • Activities without occupants or occasional presence • Fire load density qf ≤ 600 MJ/m^2 • qf > 200 MJ/m^2; gross area ≤ 25 m^2 • qf ≤ 200 MJ/m^2; gross area ≤ 100 m^2 • no presence or little presence of dangerous substances • no dangerous operations
II	Compartments that are not comprised in the other criteria
III	Based on the risk evaluation of the activity and the nearby activities

For deemed-to-satisfy solutions, the following prescriptions shall be satisfy based on the performance level:

- **Performance level II**: for each compartment smoke and heat exhaust, that work emergency openings, must be in place. Based on the risk evaluation it is possible to install systems with capabilities of forced horizontal ventilation.
- **Performance level III**: A forced or natural system able to evacuate smoke and heat must be installed according to UN 9494-1 (natural) and UNI 9414-2 (forced).

Based on the evaluation risk a part of the openings used to evacuate smoke may have the properties listed in Table 50.

Table 50: Type of opening for smoke exhaust

Type	Description
SEa	Permanently open.
SEb	Automatic opening system.
SEc	Commanded opening from protected position.
SEd	Closing elements that can be opened from a non-protected position.
SEe	Permanent closing elements for which the opening is possible in the actual fire conditions.

The minimum total surface (SE) of the exhaust openings is computed as a function of the fire load density (MJ/m^2) and the gross area of the compartment (m^2), as shown in Table 51.

Table 51: Minimum total surface of the exhaust openings

Design	Fire load density	SE	Additional requirements
SE1	$qf \leq 600$ MJ/m^2	A/40	-
SE2	$600 < qf \leq 1200$ MJ/m^2	A qf/40000 + A/100	-
SE3	$qf > 1200$ MJ/m^2	A/25	10% of opening of type SEa or SEb or SEc

The rules for industrial buildings are described above for which the performance level is selected according to Table 49.
Additional provisions for commercial buildings that are open to the public are given according to the specific technical rules (RTV, 23/11/2018) [61] and they are shown in Table 52.

Table 52: Performance levels in terms of smoke and heat control for commercial buildings that are open to the public

Activity classification	Conditions	Performance level
AA	None	II
AB, AC	$qf \leq 600$ MJ/m^2 and fast fire growth rate	II
AB, AC, AD, AE	None	III

A.11. Finland

In Finland [43], buildings are categorized in three building fire classes (P1, P2 and P3). The fire classes are defined by building height, gross floor area, floor count and building occupancy. Within fire classes P1 and P2 there is further differentiation for the load bearing and fire separation requirements based on the expected fire load. The fire load categories are as follows:

1) less than 600 MJ/m^2;
2) at least 600 MJ/m2, but not more than 1,200 MJ/m^2;
3) over 1,200 MJ/m^2.

A.11.1. Structural fire requirements

The fire resistance required for industrial single-storey buildings in Finland is indicated in Table 53. Typically, the fire resistance requirements for the main structure range from 0 minutes (R0) to 60 minutes (R60), according to the fire class of building and the provision of sprinklers.

For commercial buildings, a fire resistance R60 is usually required for the structure.

Table 53: Structural fire resistance requirements for industrial single-storey buildings in Finland

Fire class of the building	Fire resistance
P1	R 60 (R30 *) (R15, A2 *)
P2	R 30 (R15 *) (R15, A2)
P3	R0
* Building fitted with an automatic fire-extinguishing system A2: Load-bearing structures must be at least A2-s1, d0-class.	

It is worth noting that a performance-based design is permitted to assess the structural fire behaviour. When the design of load-bearing structures is based on a design fire scenario, a building is considered sufficiently fire-safe with respect to load-bearing structures if the building does not collapse during the period of time required for securing evacuation, rescue operations and controlling the fire.

A.11.2. Compartmentation

The maximum permissible compartment size in industrial single-storey buildings is given in Table 54, according to the fire class of the building. It is worth noting that the maximum permissible areas may be increased by 50% if the compartment is equipped with a fire alarm system. The partition fire wall must have at least a one half hour fire resistance rating EI 30. For industrial buildings, the fire resistance rating required for partition fire walls range from EI 60 to EI 90.

For commercial building, the compartment size is limited to 1 600 m². The partition fire wall must have at least a one half hour fire resistance rating EI 30.

Usually, the fire resistance of doors in partition fire walls must be at least half of the fire resistance required for the walls.

Table 54: Maximum permissible compartment sizes (m²) and fire wall resistance requirements for industrial single-storey buildings in Finland

Fire class of the building	fire hazard class	Maximum compartment size (m²)	Fire resistance requirement for fire walls
P1	class 1	6 000[1)] (60 0000*)	EI-M 90, A1 (EI-M 60, A1*)
	class 2	2 000[1)] (12 0000*)	EI-M 120, A1 (EI-M 60, A1*)
P2	class 1	4 000[1)] (36 0000*)	EI-M 90, A1 (EI-M 60, A1*)
	class 2	1 000[1)] (6 0000*)	EI-M 120, A1 (EI-M 60, A1*)
P3	class 1	2 000 (12 000*)	EI-M 90, A1 (EI-M 60, A1*)
	class 2	2 000*	EI-M 60, A1 *
* The building is provided with an automatic fire-extinguishing system that is suitable for this purpose. [1)] The surface area of a fire compartment may be increased by a maximum of 50% if the area is provided with a fire alarm system that is linked to the emergency centre and effective extinguishing work can be commenced at a sufficiently early stage.			

A.11.3. Smoke control

Usually, in order to enhance the efficiency of firefighting and rescue operations, means of smoke extraction are installed in the buildings. Smoke extraction is designed on the basis of a risk analysis and, in most cases, the smoke extraction plan must be submitted to the rescue authorities for approval. Specifications for the design, scaling and installation of smoke exhaust ventilators are defined in the European standard SFS-EN 12101-2.

A.12. Hungary

In Hungary, fire safety requirements are regulated by legislative provision called National Fire Safety Code issued by Ministerial Decree 54/2014 (XII. 05.) BM as amended by Ministerial Decree 30/2019 (VII 26) BM [67]. This is much shorter than its predecessors, consisting only the basic fire safety design principles, the required safety level and the detailed fire safety requirements.

Besides the legislative provision, there are altogether 14 Fire Protection Technical Guidelines including Fire Protection Properties for Building Constructions. Accepted technical solutions, Best Practice examples and standard applications are included in the Fire Safety Guidelines. There are no extracts or parts from EU related and valid standards, but practical solutions how to fulfil the required safety level using the standards. For instance, the required fire safety per-romances are included in the NFSC, calculation methods are included in the Eurocode standards and the accepted fire safety design processes of loadbearing structures are shown in the Fire Protection Technical Guideline: Fire Protection Properties for Building Constructions.

Advantage of the guidelines are the followings: easy to use, free download, regular overwide is simpler than at the standards or at the legislative provisions. Fire Protection Technical Guidelines are completed by committees of professionals but issued by the Headquarter of National Dis-aster Recovery.

A.12.1. Structural fire requirements

According to the NFSC, Article 6, when designing building products and building structures, they shall be selected in a manner to ensure that

- supporting structures retain their load-bearing capacity and dividing structures retain their integrity and thermal insulation capacity for a period of time specified in this Decree considering the expected impact of fire,
- building structures and building products designed to achieve a fire protection related objective fulfil their role and remain functional for a period of time specified in this Decree and respond to the presence of fire effectively,
- they block, render more difficult or direct the propagation of fire and its concomitants in line with their function, and
- the volume of heat, smoke and combustion products they give off is kept to a bare minimum.

According to the NFSC, Article 7, initial parameters of fire protection design:

- the fire protection solutions of a building shall be designed and dimensioned taking into account the harmful effect of a single fire starting at any location inside the building at any time,
- the building is used according to its designated purpose when the fire starts,
- the number of people exposed to threat and their capacity to escape match the designated purpose,
- the fire covers a single fire compartment including the location where it started, and
- no simultaneous event occurs at the time of the fire, such that would pose a threat or risk or would render fire safety solutions inoperable.

Fire protection requirements shall be established on the basis of the fire hazard category of substances, the risk category of units of hazard and the standard risk category of independent building sections and special structures. To determine the risk that influences fire protection requirements, the following shall be specified:

- the units of hazard in a building and in an independent building section, the related risk categories and in turn the standard risk category of the building and the independent building section, and
- the risk category of special structures.

The unit of hazard may be:

- a unit with independent designated purpose,

- a group of adjacent units with independent designated purpose as defines in Article 11,
- a special structure or
- a part of the building, the independent building section or special structure identified by the person responsible for preparing fire safety documentation by taking the provisions of paragraph (3) into account.

Risk classifications of the different risk units and the design risk classification can be determined according to the followings:

- Based on the risk unit's highest floor level (+7,0 m below: VLR, between +7,01-+14,0 m: LR, between +14,01-+30,0 m MR, above 30 m: HR)
- Based on the risk unit's lowest floor level (between ±0,00 - -4,00: VLR, between- -4,01 - -7,00 LR, between - 7,01-+14,0 m: MR, below -14 m: HR).
- Based on the risk unit's room of maximum capacity (under 50 persons: VLR, between 51-300 persons: LR, above 300 persons: MR – there is no HR classification based on the room's capacity).
- Based on the escaping abilities of the users (escaping on their own: VLR,)
- Based on the stored objects, goods or materials (non-combustible materials: VLR, combustible materials included maximum 100 kg or 100 l explosive materials: LR, combustible materials included maximum 300 kg or 300 l explosive materials: MR, explosive materials: HR).
- Based on the industrial technology (according to list of examples from the Fire Protection Technical Guideline Risk Classification)

Table 55: Reaction to fire and fire resistance requirements for building's structure and fire walls

<table>
<tr><td>1</td><td>Design risk classification</td><td></td><td>VLR</td><td>VLR</td><td>VLR</td><td>LR</td><td>LR</td><td>LR</td><td>MR</td><td>MR</td><td>MR</td><td>HR</td><td>HR</td><td>HR</td></tr>
<tr><td rowspan="2">2</td><td rowspan="2">Number of levels of the building or building part</td><td rowspan="2"></td><td>1-2
Industrial, agricultural, storage functions</td><td>3
Industrial, agricultural, storage functions</td><td rowspan="2">4</td><td rowspan="2">1-2</td><td rowspan="2">3-4</td><td rowspan="2">5-6</td><td rowspan="2">1-2</td><td rowspan="2">3-6</td><td rowspan="2">7-15</td><td rowspan="2">1-2</td><td rowspan="2">3-15</td><td rowspan="2">>15</td></tr>
<tr><td>1-3
residential function</td><td>1-3
Public functions</td></tr>
<tr><td>3</td><td>Building structures</td><td>R</td><td>15
D</td><td>30
D</td><td>60
D</td><td>30
D</td><td>30
C</td><td>60
A2</td><td>30
A2</td><td>60
A2</td><td>90
A2</td><td>60
A2</td><td>90
A2</td><td>120
A2</td></tr>
<tr><td>10</td><td>Fire wall</td><td>REI</td><td colspan="6">120
A1</td><td colspan="3">180
A1</td><td colspan="3">180
A1</td></tr>
<tr><td>11</td><td>Fire compartment border wall and floor slab
EW criteria can be applied instead of EI at fire compartment border walls at least with B reaction-to-fire classification, over 2,10 m measured from the floor level of adjoining circulation or escape route
EW criteria can be applied instead of EI at external fire rated bordering wall, if not increase the fire spread hazard</td><td>EI
(EW)</td><td colspan="2">30
A2</td><td>60
A2</td><td>30
A2</td><td>30
A2</td><td>60
A2</td><td>30
A2</td><td>60
A2</td><td>90
A2</td><td>60
A2</td><td>90
A2</td><td>120
A2</td></tr>
<tr><td>12</td><td>Barrier against fire propagation</td><td></td><td colspan="12">the requirement is at least the same as the adjoining floor slab or wall, but maximum A2, R 90 A2</td></tr>
<tr><td>13</td><td>Fire rated partition wall
EW criteria can be applied instead of EI over 2,10 m measured from the floor level of adjoining circulation or escape route</td><td>EI
(EW)</td><td colspan="5">15</td><td colspan="7">30</td></tr>
</table>

Classified by the degree of risk, buildings, independent building parts and units of hazard may belong to the design risk category of

- Very Low Risk or VLR,
- Low Risk or LR,
- Medium Risk or MR,
- High Risk or HR.

The fire resistance required for buildings is indicated in Table 55. Typically, the fire resistance requirements for the main structure of single-storey buildings range from 15 minutes (R15) to 60 minutes (R60), according to the risk category. It can be noted that there are no fire requirement requirements in the following cases:

- Storage buildings containing non-fire-hazardous materials only;
- Industrial or storage buildings with a floor area of up to 1 000 m², classified as VLR or LR, if the number of persons occupying the building at any time does not exceed 10;

provided that protection against the spread of fire is ensured between the structure and adjacent buildings and outdoor storage areas.

A.12.2. Compartmentation

Notwithstanding the provisions, the standard risk category of buildings, independent building sections and special structures shall be identical to the most severe risk category assigned to their units of hazard. The supporting structure elements ensuring compartmentation must comply – depending the standard or design risk classification - with the requirements set forth in the following table.

The maximum permissible compartment size in industrial and commercial single-storey buildings is given in Table 56, according to the building risk category and the building occupation.

Table 56: Maximum permissible compartment sizes (m²) for single-storey buildings in Hungary

<table>
<tr><th rowspan="3">Building occupation</th><th colspan="5">Design risk classification</th></tr>
<tr><th colspan="2">VLR</th><th rowspan="2">LR</th><th rowspan="2">MR</th><th rowspan="2">HR</th></tr>
<tr><th>if the building has a moderate risk of VLR</th><th>if the building has a standard risk LR, MR or HR</th></tr>
<tr><td>Commercial</td><td>1 000
2 000*</td><td>4 000
8 000*</td><td>8 000
16 000*</td><td>7 000
14 000*</td><td>3 000
6 000*</td></tr>
<tr><td>warehouse</td><td colspan="2">10 000
20 000*</td><td>12000
24000*</td><td>7 000
14 000*</td><td>4 000
8 000*</td></tr>
<tr><td>Industrial</td><td colspan="2">8 000
12 000*
24 000**</td><td>10 000
15 000*
30 000**</td><td>5 000
10 000*
20 000**</td><td>1 000
4 000*
8 000**</td></tr>
<tr><td colspan="6">* Building fitted with an automatic fire-extinguishing system
** Building fitted with an automatic fire-extinguishing system and equipped with a fire alarm system</td></tr>
</table>

A.12.3. Smoke control

The buildings must be fitted with a smoke and heat extraction system designed according to the European guidelines EN 12101-1/2 and 2:2018 and EN 12101-2:2020 according to National Fire Safety Code 54/2014 (XII. 05.). This provision does not apply to compartments with a floor area lower than 1200 m².

Moreover, fire compartments must be divided into “smoke zones” bordered by fixed smoke curtains, with a maximum area of 1 600 m² and a maximum length of 60m.

A.13. Czech Republic

Technical standards are generally not obligatory in the Czech Republic (they are recommended), but they can be obligatory if required by a decree (especially in the case of protection of the public interest, such as thermal protection, acoustics or fire safety). Fire design standards (series of standards CSN 73 08xx [70], [71], [72]) and some other associated standards are binding because they are ordered by Decree 23/2008 Coll [68]. The calculation methods and requirements are found exclusively in the technical standards of the ČSN 73 08xx series (fire safety of buildings), in part specific requirements are given in Decree 23/2008 Coll. However, with regard to the age of the Decree, the requirements are translated into the current versions of the standards (Decree No. 23/2008).

The fire safety design system is prescriptive. A different procedure (performance-based design) is allowed by law (133/1985 Coll., § 99) and the framework methodology is then very briefly defined by the fundamental standards CSN 73 0802:2020, resp. CSN 73 0804:2020 (non-production or production objects) in the informative appendix. Acceptance criteria are not de-fined in national regulations.

The basic categorization in the field of fire safety designing is created by 2 fundamental stand-ards for non-production buildings (CSN 73 0802, residential and civil buildings) and production buildings (CSN 73 0804). The general content of the master standards is given in the previous section. This categorization is complementary to other project standards (CSN 73 08xx series) for specific constructions and operations (assembly areas, housing, accommodation, alterations to buildings, medical buildings, social care, postal operations, telecommunications links, agricultural buildings, warehouses).

A.13.1. Structural fire requirements

Fire resistance requirements are based on a fire resistance grade, which is determined from a combination of the following:

- construction system type based on the combustibility and combination of load-bearing and fire-separating construction elements, see Figure 46;
- fire risk – expressed by calculated fire load or equivalent fire duration (calculated fire load is in principle the same value as equivalent fire duration – standard furnace exposure);
- building height – from the floor level of the ground level storey to the floor level of the uppermost storey.

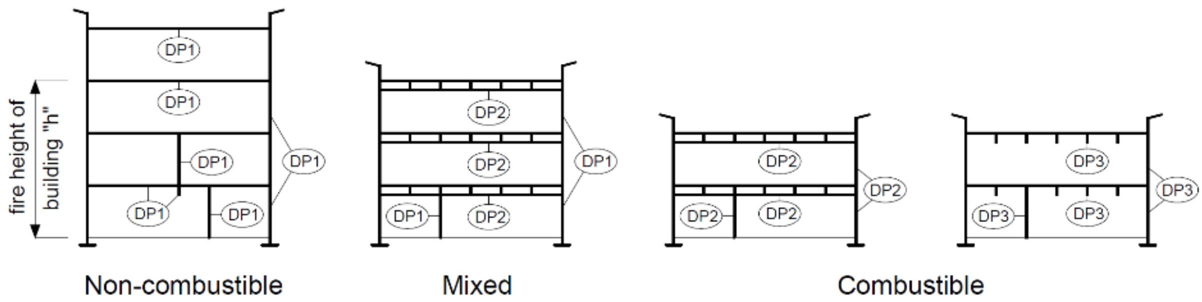

Figure 46: Classification of construction system types

Classification of construction element types is based upon the reaction to fire class of their individual components as shown in Figure 47.

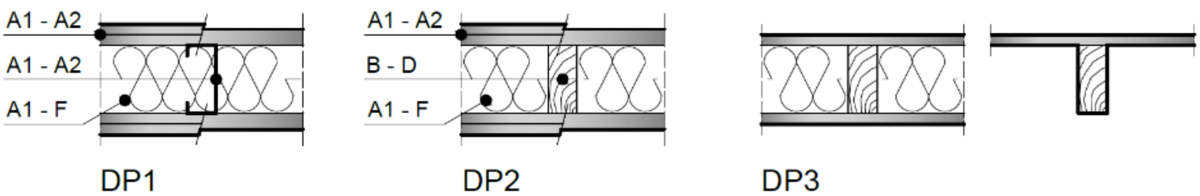

Figure 47: Classification of construction element types

The fire risk value is calculated for each compartment and counts in variable and fixed fire load, fuel type (fire growth rate) and ventilation conditions. There are a series of calculations, which are different for non-industrial and industrial buildings.

Once the three parameters listed above are calculated, then the fire resistance grade is established from a series of tabulated values. An example for non-industrial building is reproduced in Table 57.

Table 57: Determination of fire resistance grade

Construction system (CS) of building	The highest factored fire load (p_v [kg/m^2]) in fire compartment (FC)	The lowest fire resistance grade (FRG) in fire compartment						
		I.	II.	III.	IV.	V.	VI.	VII.
		Fire height of building h [m]						
Non-combustible	15	12	30	60	without limit			
	30	O	12	30	without limit			
	45	O	6	22,5	45	without limit		
	60	O	6	12	30	45	without limit	
	90	O_a	O	6	12	30	45	
	120	N_1	O_a	O	6	12	30	45
	above 120[1]	N_1	N_1	O_a	O	6	12	30
Mixed	10	6	12	12	18	22,5	N_2	N_2
	25	O	6	12	18	22,5	N_2	N_2
	35	O	6	12	18	22,5	N_2	N_2
	50	O_a	O	6	18	22,5	N_2	N_2
	75	N_1	O	6	12	22,5	N_2	N_2
	100	N_1	O	6	9	15	N_2	N_2
	above 100[1]	N_1	N_1	O	6	12	N_2	N_2
Combustible	10	4	9	12	12	12	N_2	N_2
	20	O	4	9	12	12	N_2	N_2
	30	O	4	9	12	12	N_2	N_2
	40	O_a	O	4	9	12	N_2	N_2
	60	N_1	O	4	4	9	N_2	N_2
	80	N_1	O_a	O	4	9	N_2	N_2
	above 80[1]	N_1	N_1	O_a	O	4	N_2	N_2

Example:
- block of flats
- h = 21 m
- non-combustible CS
- FC = flat
- p_v = 45 kg/m^2

▶ **III. FRG**

Table legend:
- N = FRG must not be applied
- O = FC in one-storey buildings only

Subsequently for each relevant type of construction element the duration of fire resistance rating is established from a series of tabulated values; an example for non-industrial building is reproduced in Table 58. The fire resistance criteria, e.g. integrity, insulation, etc. is defined by the ČSN 73 0810:2016 standard and depend on the function of the particular construction element.

Table 58: Determination of fire resistance grade

Ref.	Construction	Fire Resistance Grade (FRG) of Fire Compartment (FC)						
		I.	II.	III.	IV.	V.	VI.	VII.
		Fire Resistance of construction and required type						
1	Fire walls and ceilings:							
	a) in basement	30 DP1	45 DP1	60 DP1	90 DP1	120 DP1	180 DP1	180 DP1
	b) in upper floors	15	30	45	60	90	120 DP1	180 DP1
	c) in upmost floor	15	15	30	30	45	60 DP1	90 DP1
	d) between buildings	30 DP1	45 DP1	60 DP1	90 DP1	120 DP1	180 DP1	180 DP1
2	Fire openings:							
	a) in basement	15 DP1	30 DP1	30 DP1	45 DP1	60 DP1	90 DP1	90 DP1
	b) in upper floors	15	15	30	30	45 DP2	60 DP1	90 DP1
	c) in upmost floor	15	15	15	30	30	45 DP2	60 DP1
3a	External walls load-bearing:							
	a) in basement	30 DP1	45 DP1	60 DP1	90 DP1	120 DP1	180 DP1	180 DP1
	b) in upper floors	15	30	45	60	90	120 DP1	180 DP1
	c) in upmost floor	15	15	30	30	45	60 DP1	90 DP1
3b	External walls non-load-bearing:	15	15	30	30	45	60 DP1	90 DP1
4	Load-bearing elements of roof	15	15	30	30	45	60 DP1	90 DP1
5	Load-bearing elements inside Fire Compartment							
	a) in basement	30 DP1	45 DP1	60 DP1	90 DP1	120 DP1	180 DP1	180 DP1
	b) in upper floors	15	30	45	60	90	120 DP1	180 DP1
	c) in upmost floor	15	15	30	30	45	60 DP1	90 DP1
6	Load-bearing elements outside building	15	15	15	30	30 DP1	45 DP1	60 DP1
7	Load-bearing elements of independent building part inside Fire Compartment	15	15	30	30	45	45 DP1	60 DP1
8	Non-loadbearing elements inside Fire Compartment	-	-	-	DP3	DP3	DP2	DP1
9	Stairways, excl. PEW	-	15 DP3	15 DP3	15 DP1	30 DP1	45 DP1	45 DP1
10a	Evacuation and fire elevator shafts and shafts in buildings higher h > 45 m:							
	1. fire separation elements	According to line 1						
	2. fire openings	According to line 2						
10b	Other shafts:							
	1. fire separation elements	30 DP2	30 DP2	30 DP1	30 DP1	45 DP1	60 DP1	90 DP1
	2. fire openings	15 DP2	15 DP2	15 DP1	15 DP1	30 DP1	30 DP1	45 DP1
11	Roof facing	-	-	15	15	30	30 DP1	45 DP1
12	One-storey building:							
	a) fire walls	30 DP1	45 DP1	60 DP1	90 DP1	-	-	-
	b) fire openings	15 DP1	30 DP1	30 DP1	45 DP1	-	-	-
	c) vertical fire barriers in external walls between buildings	15 DP1	30 DP1	30 DP1	45 DP1	-	-	-

A.12.2. Compartmentation

The Tables Table 59 to Table 61 are a simplified representation of a complex system for establishing fire resistance requirements in the Czech Republic (CSN 73 0802:2020). The full system application has a finer resolution of fire resistance values, accounts for ventilation conditions and other factors. Requirements for fire walls are (R)EI xx where xx is the time value from the tables below. Fire load values are based on the occupancy type for individual rooms (spaces) and then averaged for the entire fire compartment.

Table 59: Non-combustible construction (concrete, gypsum covered steel, etc.)

	≤12 m	≤22,5 m	≤30 m	>30 m
Low (≤30 $kg.m^{-2}$)	30	45	60	60
Medium (≤60 $kg.m^{-2}$)	45	60	60	90
High (>60 $kg.m^{-2}$)	60	90	120	180

Table 60: Encapsulated combustible construction – horizontal
(e.g. gypsum covered timber frame)

	≤6 m	≤12 m	≤18 m	≤22,5 m
Low (≤30 $kg.m^{-2}$)	30	45	60	90
Medium (≤60 $kg.m^{-2}$)	45	60	60	90
High (>60 $kg.m^{-2}$)	60	90	not allowed	not allowed

Table 61: Encapsulated combustible construction – vertical
(e.g. gypsum covered timber frame) and exposed combustible construction (e.g. CLT panels)

	≤4 m	≤9 m	≤12 m
Low (≤30 $kg.m^{-2}$)	30	45	60
Medium (≤60 $kg.m^{-2}$)	45	90	not allowed
High (>60 $kg.m^{-2}$)	90	not allowed	not allowed

A.13.2. Smoke control

The buildings must be fitted with a smoke and heat extraction system designed according to the European guidelines ČSN EN 12101-1/2 and 2:2018 and ČSN EN 12101-2:2020 according to CSN 73 0804:2020.

ANNEX B. SEISMIC REGULATIONS

B.1. France

Since 2011, the French national legislative provisions refer to EN 1998-1, thus making compulsory its use for the seismic design of buildings. According to EN 1998-1, structures in seismic regions are recommended to be designed and constructed to withstand a design seismic action associated with a reference probability of exceedance of 10% in 50 years or a reference return period of 475 years. The hazard is described in terms of the value of the reference PGA, which may be derived from zonation maps found in National Annex.

B.1.1. Seismic zoning

The French territory is divided into five seismic zones of increasing seismic activity based on communal divisions [26], ranging from the very low seismicity zone "1" to the high seismicity zone "5" (see Figure 48). This hazard map corresponds to a mean return period of 475 years (10% probability of exceedance in 50 years), i.e. the recommended reference period in EN 1998-1.

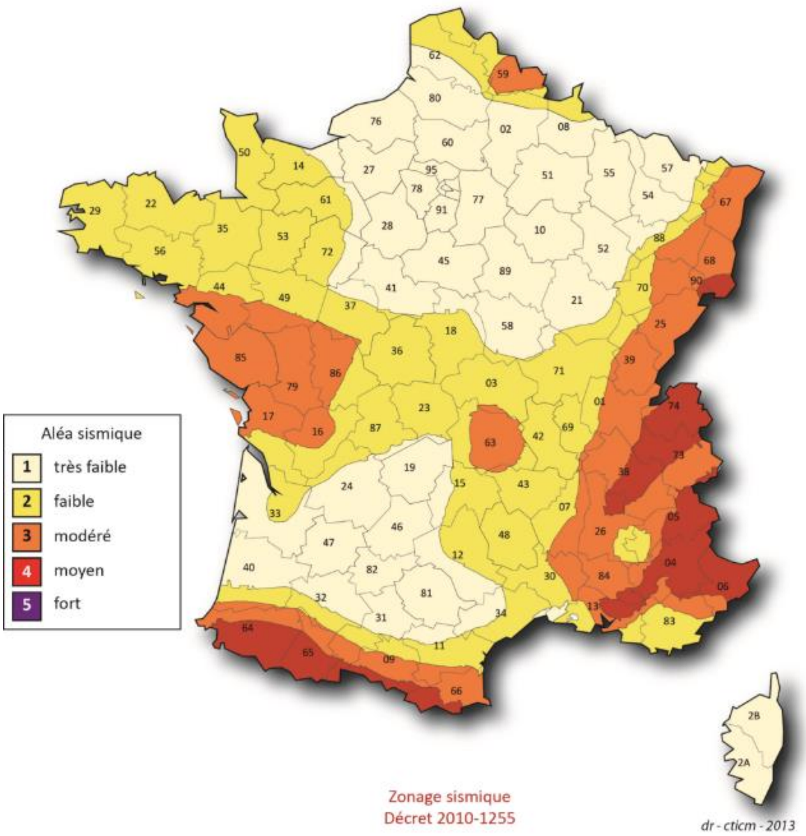

Figure 48: French seismic zoning (corresponding to a reference return period of 475 years with 10% probability of exceedance in 50 years)

B.1.2. Building classification and importance factors

The French regulation distinguish two types of buildings:

- The so-called "normal risk" buildings, which are buildings, installations and equipment for which the consequences of an earthquake are limited to their occupants and their immediate vicinity, and
- The "special risk" buildings, which are buildings, facilities and equipment for which the effects on people, property and the environment, even minor damage resulting from an earthquake, may not be limited to the immediate vicinity of these buildings. It corresponds to nuclear type installations, dams, bridges, facilities classified for environmental protection (ICPE), which are subject to special seismic regulations.

The "Normal risk" buildings are categorised into four classes of increasing importance [17], from category I to category IV, depending mainly on the activity taking place in the building and the number of people it can accommodate. The types of buildings associated with each importance class are reported in Table 62. It should be noted that the most restrictive building class is used in cases where the building is made up of parts categorised under different importance classes. Moreover, regarding the application of the regulation to existing buildings, the importance class to be used is that resulting from the classification after works have been carried out or the purpose of the building has changed.

The importance factor γ_I assigned to each importance class are reported in Table 62 also. They correspond to the recommended values given in EN 1998-1.

Table 62: Definition of importance classes for “normal risk” buildings

Importance classes	Importance factor γ_I	Buildings
I	0,8	No human activity requiring significant time spent in the building
II	1	Individual dwellings Class 4 and 5 buildings open to the public Communal dwellings less than or equal to 28 m high Offices or commercial premises not open to the public, less than or equal to 28 m high, accommodating a maximum of 300 people Industrial buildings accommodating a maximum of 300 people Car parks open to the public
III	1,2	Class 1, 2 and 3 buildings open to the public Communal dwellings and offices more than 28 m high Buildings accommodating more than 300 people Healthcare and social facilities Communal energy-generating facilities Educational establishments
IV	1,4	Buildings vital for civil protection, national defence and maintenance of public order Buildings essential for maintaining communications, production and storage of drinking water and public distribution of power Buildings essential for controlling air safety Medical establishments (intensive care) Weather centres

It could be noted that for "normal risk" buildings, the obligation to apply seismic design rules concerns only some new constructions (see Table 63), including:

- Buildings of importance classes III and IV, located in seismic zone 2;
- Buildings of importance classes II, III and IV, located in seismic zones 3, 4 and 5;

In practice, the single-storey buildings for industrial, commercial and artisanal uses are commonly classified in importance classes II or III, depending on various parameters like the number of people present in the building, the building height or the building occupation. For buildings of importance II located in seismic zone 2, it can be noted that none seismic design is needed. On the other hand, other buildings have to be dimensioned with regard to the seismic risk. In general, higher the seismicity or greater the stake importance is, higher is the level of seismic action that the building must withstand.

Table 63: New 'normal risk' buildings subject to earthquake requirements

Seismic zones	Importance classes I	II	III	IV
1	No requirement	No requirement	No requirement	No requirement
2	No requirement	No requirement	Mandatory seismic design	Mandatory seismic design
3	No requirement	Mandatory seismic design	Mandatory seismic design	Mandatory seismic design
4	No requirement	Mandatory seismic design	Mandatory seismic design	Mandatory seismic design
5	No requirement	Mandatory seismic design	Mandatory seismic design	Mandatory seismic design

B.1.3. Reference ground acceleration

Table 64 gives the values of the reference peak ground acceleration a_{gr} on ground type A for the defined seismic zones. It should be noted that these values are applicable to "normal risk" buildings only. With regard to "special risk" buildings, the French regulatory gives the design ground accelerations a_g to take into account in seismic design for each seismic zone.

Table 64: Reference peak ground acceleration a_{gr} (m/s²) for "normal risk" buildings

Seismic zones	1	2	3	4	5
a_{gr} (m/s²)	0,4	0,7	1,1	1,6	3

As can be noticed in Table 64, the reference peak ground acceleration ranges from 0,4 to 3 m/s².

B.1.4. Design ground acceleration

In France, it can be noted that the design ground acceleration is different for new and existing buildings. For new "normal risk" buildings, the values of the design ground acceleration (corresponding to $a_{gr} \times \gamma_I$) are given in Table 65, depending on the importance class of the building and the seismic zone considered. For existing buildings, the design ground acceleration can be assumed to be 60% of that required for a new buildings. Moreover, where it has to be considered, the design vertical ground acceleration a_{vg} is usually lower and taken as 90% of the value of the horizontal acceleration for seismic zones 1 to 4, and 80% for seismic zone 5.

Table 65: Design ground acceleration a_g (m/s²) on ground type A for "normal risk" buildings

Seismic zones	Importance classes			
	I	II	III	IV
1	0,32	0,40	0,48	0,56
2	0,56	0,70	0,84	0,98
3	0,88	1,10	1,32	1,54
4	1,28	1,60	1,92	2,24
5	2,40	3,00	3,60	4,20
		Application of EN 1998-1 is mandatory		

For "special risk" buildings, the design ground acceleration values are given in Table 66, depending on the seismic zone [19].

Table 66: Design ground accelerations a_g et a_{vg} en m/s² for « special risk » buildings

Seismic zones	Design horizontal ground acceleration a_g (m/s²)		Design vertical ground acceleration a_{vg} (m/s²)	
	New buildings	Existing buildings	New buildings	Existing buildings
1	0,88	0,74	0,79	0,67
2	1,54	1,3	1,39	1,17
3	2,42	2,04	2,18	1,84
4	3,52	2,96	2,82	2,37
5	6,60	5,55	5,28	4,44

B.1.5. Influence of the soil

Five soil classes are defined in France, with an associated value of the soil factor reported in following tables, distinguishing between the seismic zones 1 to 5 and the building type.

Table 67: Soil factor S according to the ground type for “normal risk” buildings

Ground type	Soil factor S	
	zones 1 to 4	Zone 5
A	1	1
B	1,35	1,2
C	1,5	1,15
D	1,6	1,35
E	1,8	1,4

Table 68: Soil factor S according to the ground type for « special risk » buildings

Ground type	Soil factor S	
	zones 1 to 3	Zones 4 to 5
A	1	1
B	1,35	1,2
C	1,5	1,15
D	1,6	1,35
E	1,8	1,4

B.1.6. Elastic response spectrum

The seismic action can be represented by an elastic ground response acceleration spectrum (also called elastic response spectrum), the form of which is defined in EN 1998-1 and the values of the various parameters are set by the ministerial decrees establishing the seismic design rules applicable to "normal risk" or "special risk" buildings. The response spectrum is thus established from standard formulas, which depend on the maximum design ground acceleration, the soil factor S and the characteristic periods noted TB, TC and TD.

EN 1998-1 distinguishes two types of elastic response spectra (type 1 and type 2) for varying seismicity conditions. The use of the so-called type 2 spectrum (Ms<5.5 that is low seismicity regions) is specified for “normal risk” buildings in seismic zones 1 to 4 and for “special risk‟ buildings in seismic zones 1 to 3, with characteristic periods different from those recommended in EN 1998-1. The use of the so-called type I spectrum (Ms>5.5 that is high and moderate seismicity regions) is specified for “normal risk” buildings in seismic zone 5 and for “special risk‟ buildings in seismic zones 4 and 5, with the same soil factors and characteristic periods as with those recommended in EN 1998-1. The values of characteristic periods specified in the French regulation are given in following tables for “normal risk” buildings and for « special risk » buildings.

Table 69: Periods TB, TC and TD for the horizontal elastic response spectrum for « normal risk » buildings

Ground type	Seismic zones 1 to 4			Seismic zone 5		
	T_B	T_C	T_D	T_B	T_C	T_D
A	0,03	0,2	2,50	0,15	0,40	2,00
B	0,05	0,25	2,50	0,15	0,50	2,00
C	0,06	0,40	2,00	0,20	0,60	2,00
D	0,10	0,60	1,50	0,20	0,80	2,00
E	0,08	0,45	1,25	0,15	0,50	2,00

Table 70: Periods TB, TC and TD for the vertical elastic response spectrum for « normal risk » buildings

Seismic zones	T_B	T_C	T_D
1 to 4	0,03	0,20	2,50
5	0,15	0,40	2,00

Table 71: Periods TB, TC and TD for the horizontal elastic response spectrum for « special risk » buildings

Ground type	Seismic zones 1 to 3			Seismic zones 4 and 5		
	T_B	T_C	T_D	T_B	T_C	T_D
A	0,03	0,2	2,50	0,15	0,40	2,00
B	0,05	0,25	2,50	0,15	0,50	2,00
C	0,06	0,40	2,00	0,20	0,60	2,00
D	0,10	0,60	1,50	0,20	0,80	2,00
E	0,08	0,45	1,25	0,15	0,50	2,00

Table 72: Periods TB, TC and TD for the vertical elastic response spectrum for « special risk » buildings

Seismic zones	T_B	T_C.	T_D.
1 to 3	0,03	0,20	2,50
4 and 5	0,15	0,40	2,00

Taking into account the rules defined by Eurocode 8 (in particular the threshold of 0.25g below which it is normally not necessary to take into account the vertical direction of the earthquake) and the values defined by the French regulations. It is interesting to note that the acceleration a_{vg} is everywhere lower than 0.25g in metropolitan France (seismic zones 1 to 4 only), whatever the category of importance of the building. Therefore, the vertical direction is normally never to be taken into account in metropolitan France for normal risk buildings.

B.1.7. Design response spectrum

According to EN 1998-1, in order to avoid explicit inelastic structural analysis in design, the capacity of the structure to dissipate energy, through mainly ductile behaviour of its elements and/or other mechanisms, is taken into account by performing an elastic analysis based on a design response spectrum, reduced with respect to the elastic one by introducing a behaviour factor q.

The behaviour factor is generally calculated by the ratio α_u/α_1, where α_u is the coefficient by which the seismic action has to be multiplied to reach the collapse of the structure, and α_1 to reach first yield in any of its components (equivalent to the formation of the first plastic hinge). Reference values and upper limits for behaviour factors q to be used in France for design of steel structure according to the wanted structural behaviour are given in the following table.

Moreover, in France the lower-bound coefficient β is fixed to the recommended value, i.e., 0,2.

Table 73: Values of the behaviour factor q according the structural behaviour design

Design concept	Structural ductility class	behaviour factor q
Low dissipative structural behaviour	DCL (low ductility)	$q \leq 1{,}5$
Dissipative structural behaviour	DCM (medium ductility)	$1{,}5 < q \leq 4$
	DCH (high ductility)	$q > 4$

It could be noted that according to the French National Annex to EN 1998-1, the conditions for applying the ductility class DCL are fixed in recommendations published by CNC2M [28].

In France, the use of the ductility class L is reserved only for cases where the ground acceleration $a_g = a_{gR}\ \gamma_I$ is between 0.78 m/s^2 and 1.1 m/s2 (NF EN 1998-1/NA). For ag < 0.78 m/s^2 or agS < 1 m/s^2, the seismic activity of the construction site is too low; the EN 1998-1 is not to be used, and the structure is to be designed by applying only the rules of EN 1993-1-1.

B.1.8. Damage limitation

In addition to general deformation limits defined in EN 1998-1 in terms of a set limiting interstorey drifts, a reduction factor, v, is also introduced in order to account for the lower return period of the seismic action to consider for the damage limitation requirement. In France, the reduction factor v to use to verify the damage limitation conditions is equal to 0.4, whatever the importance class of buildings (which is different to the recommended values in EN 1998-1, i.e. 0.4 for important classes I and II or 0.5 for important classes III and IV).

B.1.9. Design of non-structural elements

In France, the non-structural elements are covered by the EN 1998-1 and a national guidance [21] (part of the French law) that help to applied the standard.

B.2. Spain

B.2.1. Seismic zoning

The Spanish Annex National of EN 1998 [50] do not provide any seismic map. For information purpose, the seismic hazard map given in Figure 49 comes from the Spanish building design code NCS-02. This map provides values of basic seismic acceleration, a_b, for hard soil type, and values of a contribution factor K, which takes into account the influence of the different types of expected earthquakes on the seismic hazard. The Spanish territory is then divided into five zones of decreasing seismicity. This hazard map corresponds to a mean return period of 475 years (10% probability of exceedance in 50 years), i.e. the recommended reference period in EN 1998-1.

Table 74: reference peak ground acceleration a_b (m/s²) for soil type I

Seismic zones	1	2	3	4	5
a_b (m/s²)	$0.16g \leq a_b$	$0.12g \leq a_b < 0.16g$	$0.08g \leq a_b < 0.12g$	$0.04g \leq a_b < 0.08g$	$a_b < 0.04g$

Figure 49: Spanish seismic zoning (specified in building design code NCS-02)

According to the Spanish National Annex to EN 1998-1, a low seismicity case is defined as a region where the product a_g.S is not greater than 0,1g and a very low seismicity case a region having a reference peak ground acceleration a_{gr} not greater than 0.04g. It could be mentioned that in cases of very low seismicity, the provisions of EN 1998-1 need not to be observed.

B.2.2. Building classification and importance factor

Buildings in Spain are classified in three importance classes, from moderate to special, according to their intended use and independently of their type as indicated in following table. They are similar to those recommended in EN 1998-1.

Table 75: Definition of importance classes of buildings and corresponding importance factor

Importance classes		Importance factor γ_I	Building use
Moderate	Buildings with a negligible probability that their destruction may cause victims, result in interruption of primary services, or produce significant economic damages to third parties.	0,8	Warehouses (without hazardous substances) and archives
normal	Buildings for which destruction could cause victims, result in interruption of a non-essential service to community, or produce significant economic losses but without catastrophic effects.	1	Residential (dwellings and hotels) and Public (offices, shops) buildings
special	Buildings for which destruction may interrupt an essential service or result in catastrophic effects.	1.4	Hospitals, rescue services, basic communication facilities, gas and fuel tanks, water treatment facilities, pumping stations, power Plants, distribution networks, nuclear or thermal power stations, dams, transportation, industrial buildings involving dangerous substances., historical monuments, public buildings (like large commercial areas) where a very large number of people is foreseen.

B.2.3. Reference ground acceleration

Tabulated values of the reference peak ground accelerations a_{gr} according to geodetic coordinates are specified in the Spanish National Annex of EN 1998.

B.2.4. Influence of the soil

Four classes of soil are considered in Spain, ranging from class A to class D, as defined in EN 1998-1. The soil factor associated to each class are given in Table 76.

B.2.5. Elastic response spectrum

According to the Spanish National Annex of EN 1998-1, values of the main parameters defining the shape of the elastic response spectrum are reported in Table 76. It should be noted that the cut-off periods depend on two coefficients: C, defining the soil class, and K, which takes into account the influence of the long-distance earthquakes occurred in the Azores Gibraltar zone. The soil factor S depends on the soil coefficient C and the design ground acceleration a_g through a non-linear function, taking values in the range (0,8-2.0). Values of the coefficient K are specified in the Spanish National Annex of EN 1998 in the form of tabulated data depending on geodetic coordinates.

Table 76: Values of parameters describing the horizontal elastic response spectrum

Ground type	Soil factor S	T_B	T_C.	T_D.
A	1	$\frac{T_c}{5}$	$\frac{K}{4}$	2
B C	If $\rho a_g \leq 0{,}1g$ $S = C$ If $0{,}1g < \rho a_g \leq 0{,}4g$ $S = C + 3{,}33\left(\frac{a_g}{g} - 0{,}1\right)(1 - C)$ If $0{,}4g \leq \rho a_g$ $S = 1$	$\frac{T_c}{5}$	$\frac{KC}{4}$	2
D	If $\rho a_g \leq 0{,}1g$ $S = 2$ If$0{,}1g < \rho a_g \leq 0{,}4g$ $S = 2.33 + 3{,}33\frac{a_g}{g}$ If $0{,}4g \leq \rho a_g$ $S = 1$	$\frac{T_c}{5}$	$\frac{K}{2}$	2

Where $C = \left({800}/{v_{s,30}}\right)^{0.465}$ and K is the contribution factor

Table 77: Values of parameters for the vertical elastic response spectrum

A_{vg}/a_g	T_{vB}/T_B	T_{vc} /T_C.	T_{vD} /T_D.
0,7	1	0,75	1

B.2.6. Design response spectrum

According to the Spanish National Annex to EN 1998-1, reference values and upper limits for the behaviour factors q to be used in Spain for design of steel structure according to the wanted structural behaviour are given in the following table. Moreover, the lower-bound coefficient β is fixed to the recommended value, i.e., 0,2.

Table 78: Values of the behaviour factor q according the structural behaviour design

Design concept	Structural ductility class	behaviour factor q
Low dissipative structural behaviour	DCL (low ductility)	q ≤1,5
Dissipative structural behaviour	DCM (medium ductility)	1,5 < q ≤ 4
	DCH (high ductility)	q > 4

B.2.7. Damage limitation

In Spain, values of the reduction factor v to be used to verify the damage limitation conditions are the recommended values of EN 1998-1, i.e. 0,5 for important classes I and II or 0,4 for important classes III and IV.

B.3. Germany

B.3.1. Seismic zoning

According to the German National Annex to EN 1998-1 [52], the seismic hazard in Germany is defined by means of the seismic hazard map given Figure 50, which indicate the values of the spectral accelerations $Sa_{P,R}$.to use in seismic design. This hazard map corresponds to a mean return period of 475 years (10% probability of exceedance in 50 years), i.e. the recommended reference period in EN 1998-1. As can be noticed in Figure 50, the spectral accelerations $Sa_{P,R}$ ranges from 0,1 to 4,0 m/s².

According to the German National Annex to EN 1998-1, it could be noted that:

- Very low seismicity cases are those where the product $a_g.S$ is not greater than 0.5m/s² (or 0.05g).
- For accelerations $S_{aP,R}$<0.6m/s² , the condition for very low seismicity is always fulfilled for common elevated structures of all importance building classes whatever the subsoil classes.
- For accelerations $S_{aP,R}$<0.84m/s² , the condition for very low seismicity is always fulfilled for usual high-rise structures of importance class II whatever the subsoil classes.

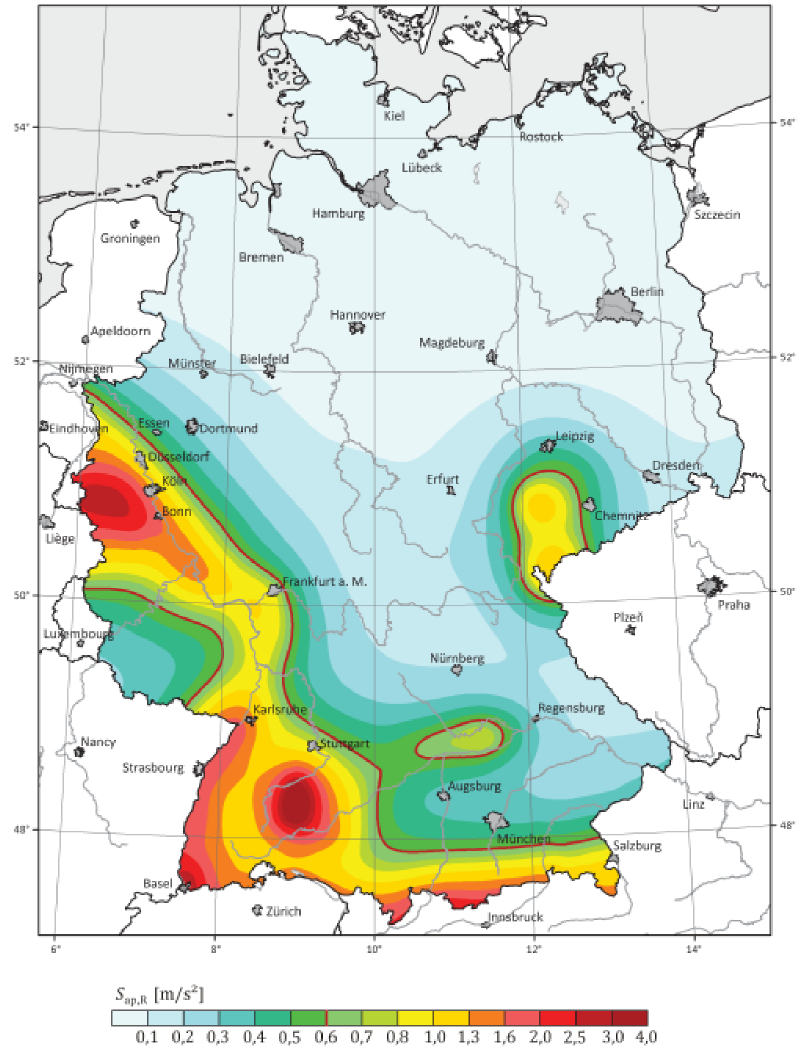

Figure 50: German seismic zoning

The importance factor to be associated with the four importance building classes defined in EN 1998-1 are those indicated in the following table. They correspond to the recommended values given in EN 1998-1.

Table 79: Definition of importance classes of buildings and associated importance factors

Importance classes	building	Importance Factor γ_I
I	Buildings of minor importance for public safety (e.g., agricultural buildings, etc.)	0,8
II	Ordinary buildings not belonging to the other categories (smaller residential and office buildings, workshops, etc.)	1,0
III	Buildings of which the seismic resistance is important in view of the consequences associated with a collapse (huge residential buildings, schools, assembly halls, malls, etc.)	1,2
IV	Buildings of which the integrity during earthquakes is of vital importance for civil protection (hospitals, important civil protection facilities, fire department, security staff, etc.)	1,4

B.3.2. Reference ground acceleration

According to the German National Annex to EN 1998-1, the reference peak ground acceleration on type A ground, a_{gR} may be calculated from the spectral acceleration $S_{aP,R}$ given in seismic hazard map as follows:

$$a_{gR} = S_{aP,R} / 2{,}5$$

B.3.3. Influence of the soil

In Germany, the soil types are differentiated according to the following subsoil classes:

- Ground Type A
  - Non-weathered (fresh) solid rocks with high strength
  - Dominant shear wave velocities are higher than about 800 m/s
- Ground Type B
  - Moderately weathered solid rocks or solid rocks with low strength
  - Coarse-grained (incohesive) or mixed-grain unconsolidated soil with high friction properties in a dense or solid consistency (for example, glacial loose rock)
  - Dominant shear wave velocities range between about 350 m/s and 800 m/s
- Ground Type C
  - Heavily or completely weathered solid rocks
  - Coarse-grained (incohesive) or mixed-grain unconsolidated soil in medium-dense soil or in at least a stiff consistency
  - Fine-grained (cohesive) soil in at least a stiff consistency
  - Dominant surface wave measurements range between about 150 m/s and 350 m/s

The changing subsoil between rock and sediment is divided into subsoil classes R, T, and S:

- Subsoil class R
  - Areas predominantly characterized by rocks
- Subsoil class T
  - Transition zones between subsoil class R and subsoil class S, as well as areas of relatively shallow sedimentary basins
- Subsoil class S
  - Areas with deep basin structures with thick sedimentary fill

The soil factor S associated to each soil class are indicated in the following table as a function of the spectral acceleration $S_{aP,R}$.

Table 80: Soil factor S according to the ground type

spectral acceleration $S_{aP,R}$ m/s^2	Ground type					
	A-R	B-R	C-R	B-T	C-T	C-S
$S_{aP,R}$. ≤ 1,0	1,00	1,25	1,50	1,05	1,45	1,30
1,0 < $S_{aP,R}$. ≤ 2,0	1,00	1,20	1,30	1,00	1,25	1,15
$S_{aP,R}$. > 2,0	1,00	1,20	1,15	1,00	1,10	0,95
For the subsoil B-S, the soil parameter S may be assumed as for C-S.						

B.3.4. Elastic response spectrum

According to the German National Annex of EN 1998-1, values of the main parameters defining the shape of the both horizontal and vertical elastic response spectra are reported in Table 81 and Table 82.

Table 81: Values of periods describing the horizontal elastic response spectrum

Ground type	T_B (s)	T_C. (s)	T_D. (s)
A-R	0,10	0,20	2,00
B-R	0,10	0,25	2,00
C-R	0,10	0,30	2,00
B-T	0,10	0,25	2,00
C-T	0,10	0,40	2,00
B-S	0,10	0,40	2,00
C-S	0,10	0,50	2,00

Table 82: Values of parameters for the vertical elastic response spectrum

A_{vg}/a_g	T_B (s)	T_C (s)	T_D (s)
0.7	0.05	0,2	1,20

B.3.5. Design response spectrum

Reference values and upper limits for behaviour factors q to be used in Germany for design of steel structure according to the wanted structural behaviour are given in the following table. Moreover, the lower-bound coefficient β is fixed to 0.

Table 83: Values of the behaviour factor q according the structural behaviour design

Design concept	Structural ductility class	behaviour factor q
Low dissipative structural behaviour	DCL (low ductility)	$q \leq 1{,}5$
Dissipative structural behaviour	DCM (medium ductility)	$1{,}5 < q \leq 4$
	DCH (high ductility)	$q > 4$

B.3.6. Damage limitation

In Germany, value of the reduction factor v to be used to verify the damage limitation conditions for buildings of importance class IV is fixed to 0,4. For buildings of importance classes I to III, no values of v are defined since the verification of damage limitation in such cases is not needed in Germany.

B.4. Portugal

B.4.1. Seismic zoning

According to the Portuguese National Annex of EN 1998-1 [51], five seismic zones are defined. Moreover, two types of seismic action must be considered: Type 1 seismic action and Type 2 seismic action corresponding to scenarios of Interplate earthquakes (Azores-Gibraltar) and Intraplate (continental) shocks, respectively. Each of these two types of seismic action has its own seismic zoning. This hazard map corresponds to a mean return period of 475 years (10% probability of exceedance in 50 years), i.e. the recommended reference period in EN 1998-1.

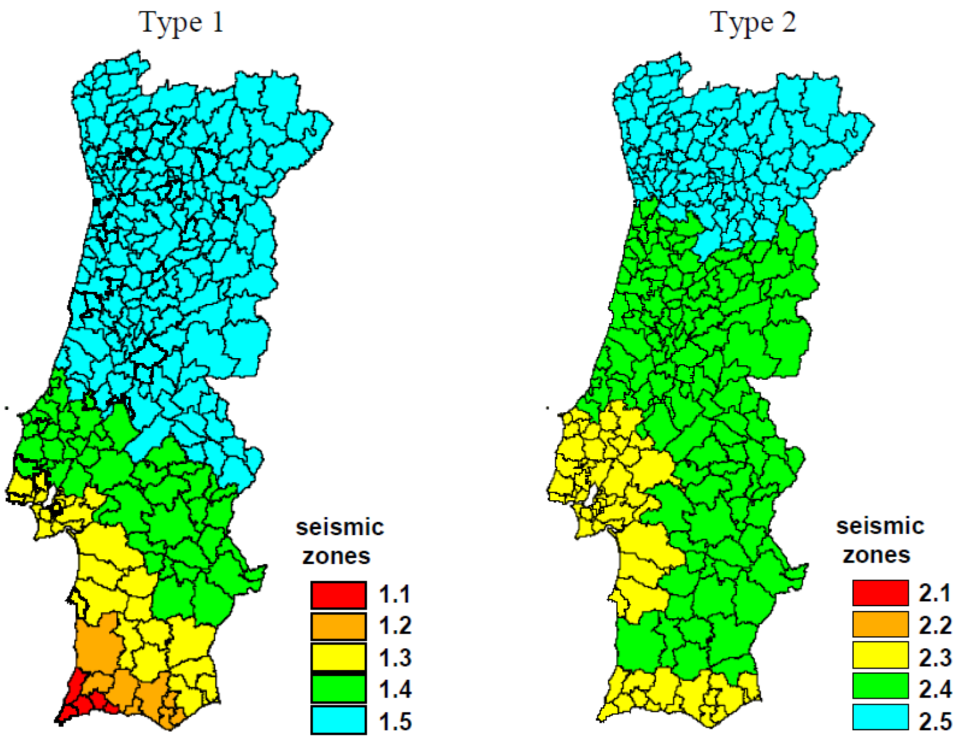

Figure 51: Portuguese seismic zoning

According to the Portuguese National Annex to EN 1998-1, a low seismicity case is defined as a region where the product a_g.S is not greater than 0,1g. Very low seismicity cases are regions having a reference peak ground acceleration a_{gr} not greater than 0,04g or where the product ag.S is not greater than 0.5m/s² (or 0.05g).

B.4.2. Building classification

In Portugal, the importance factor to be associated with the four importance building classes defined in EN 1998-1 are those indicated in the following table. They are quite different to the recommended values given in EN 1998-1.

Table 84: Values of importance factor associated with each importance building class

Importance classes	Importance Factor γ_I	
	Type 1 seismic action	Type 2 seismic action
I	0,65	0,75
II	1	1,0
III	1,45	1,25
IV	1,95	1,50

B.4.3. Reference ground acceleration

Table 85 presents the reference peak ground acceleration, a_{gR} , for the considered seismic zones and for the two seismic scenarios. As can be noticed, the reference peak ground acceleration in Portugal ranges from 0,5 to 2,5 m/s².

Table 85: Reference peak ground acceleration a_{gr} (m/s²) for soil class A

Type 1 seismic action		Type 2 seismic action	
Seismic zone	a_{gr} (m/s²)	Seismic zone	a_{gr} (m/s²)
1.1	2,5	2,1	2,5
1.2	2	2,2	2,0
1.3	1,5	2,3	1,7
1.4	1	2,4	1,1
1.5	0,5	2,5	0,8

B.4.4. Influence of the soil

In Portugal, the soil factor S should be taken equal as:

- If $a_g \leq 1m/s^2$ $\quad S = S_{max}$
- If $1m/s^2 \leq a_g \leq 4m/s^2$ $\quad S = S_{max} - \frac{(S_{max}-1)}{3}(a_g - 1)$
- If $a_g \geq 4m/s^2$ $\quad S = 1$

Where:

- a_g is the design ground acceleration on type A ground
- S_{max} is a parameter depending on the ground type as indicated in Table 86.

B.4.5. Elastic response spectrum

According to the Portuguese National Annex of EN1998, values of the main parameters defining the shape of the elastic response spectra are reported in following tables.

Table 86: Values of parameters describing the horizontal elastic response spectrum

Ground type	type 1 seismic action				type 2 seismic action			
	S_{max}	T_B (s)	T_C s)	T_D (s)	S_{max}	T_B (s)	T_C (s)	T_D (s)
A	1,0	0,1	0,6	2,0	1,0	0,1	0,25	2,0
B	1,35	0,1	0,6	2,0	1,35	0,1	0,25	2,0
C	1,6	0,1	0,6	2,0	1,6	0,1	0,25	2,0
D	2,0	0,1	0,8	2,0	2,0	0,1	0,25	2,0
E	1,8	0,1	0,6	2,0	1,8	0,1	0,25	2,0

Table 87: Values of parameters for the vertical elastic response spectrum

seismic action	A_{vg}/a_g	T_B (s)	T_C (s)	T_D (s)
Type 1	0,75	0,05	0,25	1
Type 2	0,95	0,05	0,15	1

B.4.6. Design response spectrum

According to the Portuguese National Annex to EN 1998-1, reference values and upper limits for behaviour factors q to be used in Portugal for design of steel structure according to the wanted structural behaviour are given in the following table. Moreover, the lower-bound coefficient β is fixed to the recommended value, i.e., 0,2.

It could be noted that according to the Portuguese National Annex to EN 1998-1, the ductility class DCL is not limited to low seismicity cases only and can be used for regular buildings which are not seismically isolated, having an importance class I or II.

Table 88: Values of the behaviour factor q according the structural behaviour design

Design concept	Structural ductility class	behaviour factor q
Low dissipative structural behaviour	DCL (low ductility)	$q \leq 1,5$
Dissipative structural behaviour	DCM (medium ductility)	$2,0 < q \leq 4$
	DCH (high ductility)	$q > 4$

B.4.7. Damage limitation

In Portugal, values of the reduction factor v to be used to verify the damage limitation conditions are fixed to 0,4 and 0,55 for the Type 1 and Type 2 seismic actions respectively, whatever the importance classe of buildings.

B.5. Luxembourg

B.5.1. Seismic zoning

According to the Luxembourgish National Annex of EN 1998-1, only one seismic zone is defined corresponding to the whole country. A low seismicity case is defined as a region where the product a_g.S is not greater than 0,1g or the design ground acceleration is lower than0,08g and a very low seismicity case a region having a reference peak ground acceleration a_{gr} not greater than 0,06g. It could be mentioned that in cases of very low seismicity, the provisions of EN 1998 need not to be observed.

B.5.2. Building classification

In Luxembourg, the importance factor to be associated with the four importance building classes defined in EN 1998-1 are those indicated in the following table. They correspond to the recommended values given in EN 1998-1.

Table 89: Definition of importance classes of buildings in Luxembourg

Importance classes	building	Importance Factor γ_I
I	Buildings of minor importance for public safety (e.g., agricultural buildings, etc.)	0,8
II	Ordinary buildings not belonging to the other categories (smaller residential and office buildings, workshops, etc.)	1,0
III	Buildings of which the seismic resistance is important in view of the consequences associated with a collapse (huge residential buildings, schools, assembly halls, malls, etc.)	1,2
IV	Buildings of which the integrity during earthquakes is of vital importance for civil protection (hospitals, important civil protection facilities, fire department, security staff, etc.)	1,4

B.5.3. Reference ground acceleration

The reference peak ground acceleration a_{gr} on ground type A in Luxembourg is a_{gR} = 0.39 m/s² or 0.04g.

B.5.4. Influence of the soil

In Luxembourg, the soil factor S associated to each soil class are those indicated in EN 1998-1.

B.5.5. Elastic response spectrum

According to the Luxembourgish National Annex of EN 1998-1, values of the main parameters defining the shape of the elastic response spectra are reported in the following tables. They correspond to the so-called type 2 spectrum (Ms<5,5 that is low seismicity regions) defined in EN 1998-1.

Table 90: Values of parameters describing the horizontal elastic response spectrum

Ground type	Type 2 seismic action			
	S_{max}	T_B (s)	T_C (s)	T_D (s)
A	1,0	0,05	0,25	1,2
B	1,35	0,05	0,25	1,2
C	1,5	0,1	0,25	1,2
D	1,6	0,1	0,25	1,2
E	1,8	0,05	0,25	1,2

Table 91: Values of parameters for the vertical elastic response spectrum

A_{vg}/a_g	T_B (s)	T_C (s)	T_D (s)
0.45	0,05	0,15	1

B.5.6. Design response spectrum

Reference values and upper limits for behaviour factors q to be used in Luxembourg for design of steel structure according to the wanted structural behaviour are given in the following table. Moreover, the lower-bound coefficient β is fixed to the recommended value, i.e., 0,2.

Neither the use of the design concept nor the use of the ductility classes are subject to any limitations. In particular, there are no geographical limitations on the use of ductility classes M and H.

Table 92: Values of the behaviour factor q according the structural behaviour design

Design concept	Structural ductility class	behaviour factor q
Low dissipative structural behaviour	DCL (low ductility)	$q \leq 1,5$
Dissipative structural behaviour	DCM (medium ductility)	$1,5 < q \leq 4$
	DCH (high ductility)	$q > 4$

B.5.7. Damage limitation

In Luxembourg, values of the reduction factor v to be used to verify the damage limitation conditions are fixed to 0,4 for importance classes III and IV and 0,5 otherwise.

B.6. Belgium

B.6.1. Seismic zoning

According to the Belgian National Annex of EN 1998-1, the seismic hazard in Belgium is defined by means of the seismic hazard map given in Figure 52, which defines four seismic zones of increasing seismic activity. A low seismicity case is defined as a region where the product a_g.S is not greater than 0,1g or the design ground acceleration is lower than0,08g and a very low seismicity case a region having a reference peak ground acceleration a_{gr} not greater than 0.06g. It could be mentioned that in cases of very low seismicity, the provisions of EN 1998 need not to be observed.

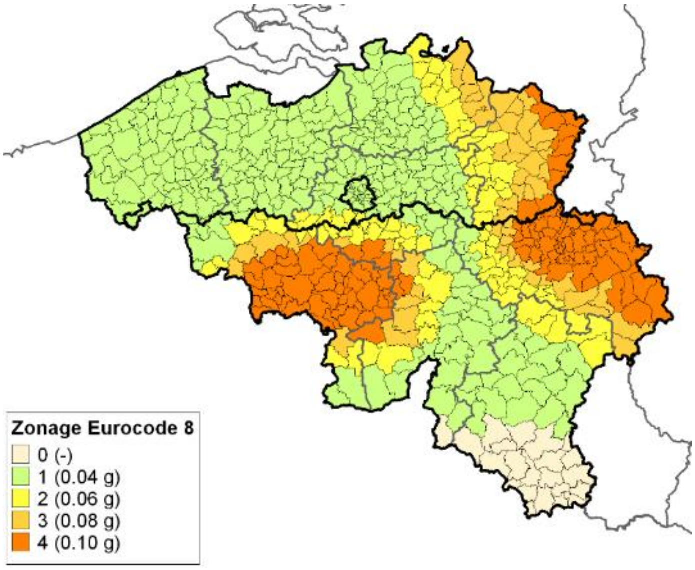

Figure 52: Belgian seismic zoning

B.6.2. Building classification

In Belgium, the importance factor to be associated with the four importance building classes defined in EN 1998-1 are those indicated in the following table. They correspond to the recommended values given in the standard.

Table 93: Definition of importance classes of buildings in Belgium

Importance classes	building	Importance Factor γ_I
I	Buildings of minor importance for public safety (e.g., agricultural buildings, etc.)	0,8
II	Ordinary buildings not belonging to the other categories (smaller residential and office buildings, workshops, etc.)	1,0
III	Buildings of which the seismic resistance is important in view of the consequences associated with a collapse (huge residential buildings, schools, assembly halls, malls, etc.)	1,2
IV	Buildings of which the integrity during earthquakes is of vital importance for civil protection (hospitals, important civil protection facilities, fire department, security staff, etc.)	1,4

B.6.3. Reference ground acceleration

Table 94 gives the values of the reference peak ground acceleration a_{gr} on ground type A for the defined seismic zones in Belgium.

Table 94: Reference peak ground acceleration a_{gr} (m/s²) in Belgium

Seismic zones	1	2	3	4
a_{gr} (m/s²)	0,39	0,59	0,78	0,98

As can be noticed in Table 94, the reference peak ground acceleration ranges from 0,39 to 0,98 m/s².

B.6.4. Influence of the soil

In Belgium, the soil factor S associated to each soil class are those indicated in EN 1998-1.

B.6.5. Elastic response spectrum

According to the Belgian National Annex of EN 1998-1, values of the main parameters defining the shape of the elastic response spectra are reported in following table. They correspond to the so-called type 2 spectrum (Ms<5,5 that is low seismicity regions) defined in EN 1998-1.

Table 95: Values of parameters describing the horizontal elastic response spectrum

Ground type	type 2 seismic action			
	S_{max}	T_B.(s)	T_C. .(s)	T_D. .(s)
A	1,0	0,05	0,25	1,2
B	1,35	0,05	0,25	1,2
C	1,5	0,1	0,25	1,2
D	1,6	0,1	0,3	1,2
E	1.8	0.05	0.25	1.2

Table 96: Values of parameters for the vertical elastic response spectrum

A_{vg}/a_g	T_B (s)	T_C(s)	T_D(s)
0.45	0.05	0.15	1

B.6.6. Design response spectrum

According to the Belgian National Annex to EN 1998-1, reference values and upper limits for behaviour factors q to be used in belgium for design of steel structure according to the wanted structural behaviour are given in the following table. Moreover, the lower-bound coefficient β is fixed to the recommended value, i.e., 0,2.

Table 97: Values of the behaviour factor q according the structural behaviour design

Design concept	Structural ductility class	behaviour factor q
Low dissipative structural behaviour	DCL (low ductility)	$q \leq 1,5$
Dissipative structural behaviour	DCM (medium ductility)	$1,5 < q \leq 4$
	DCH (high ductility)	$q > 4$

B.6.7. Damage limitation

In Belgium, values of the reduction factor v to be used to verify the damage limitation conditions are fixed to 0,4 for importance classes III and IV and 0,5 otherwise.

B.7. Czech Republic

B.7.1. Seismic zoning

According to the Czech Republic National Annex of EN 1998-1 [54], the seismic hazard in Czech Republic is defined by means of the seismic hazard map given in Figure 53, which defines five seismic zones of increasing seismic activity. A low seismicity case is defined as a region where the product a_g.S is not greater than 0,1g and a very low seismicity case a region having a reference peak ground acceleration a_{gr} not greater than 0.05g. It could be mentioned that in cases of very low seismicity, the provisions of EN 1998 need not to be observed.

Figure 53: Czech Republic seismic zoning

B.7.2. Building classification

In Czech Republic, the importance factor to be associated with the four importance building classes defined in EN 1998-1 are those indicated in the following table. They correspond to the recommended values given in EN 1998-1.

Table 98: Definition of importance classes of buildings in Czech Republic

Importance classes	building	Importance Factor γ_I
I	Buildings of minor importance for public safety (e.g., agricultural buildings, etc.)	0,8
II	Ordinary buildings not belonging to the other categories (smaller residential and office buildings, workshops, etc.)	1,0
III	Buildings of which the seismic resistance is important in view of the consequences associated with a collapse (huge residential buildings, schools, assembly halls, malls, etc.)	1,2
IV	Buildings of which the integrity during earthquakes is of vital importance for civil protection (hospitals, important civil protection facilities, fire department, security staff, etc.)	1,4

B.7.3. Reference ground acceleration

Table 99 gives the values of the reference peak ground acceleration a_{gr} on ground type A for the defined seismic zones.

Table 99: Reference peak ground acceleration a_{gr} (m/s²) in Czech Republic

Seismic zones	1	2	3	4	5
a_{gr} (m/s²)	0,03g	0,04g	0,05g	0,06g	0,07g

As can be noticed in Table 99, the reference peak ground acceleration ranges from 0,29 to 0,69m/s² approximately.

B.7.4. Influence of the soil

In Czech Republic, the soil factor S associated to each soil class are those indicated in EN 1998-1.

B.7.5. Elastic response spectrum

According to the Czech Republic National Annex of EN 1998-1, for construction works in the districts Břeclav, Frýdek-Místek, Hodonín, Karviná, Kroměříž, Nový Jičín, Opava, Ostrava-Město, Přerov, Vsetín, Uherské Hradiště and Zlín,, the horizontal seismic action according to the Type 1 elastic response spectrum where the values of the parameters are given in Table 100, shall be used.

For construction works in the remaining districts of the Czech Republic, the horizontal seismic action according to the Type 2 elastic response spectrum where the values of the parameters are given in Table 101 shall be used.

Table 100: Values of parameters describing the horizontal type 1 elastic response spectrum

Ground type	type 2 seismic action			
	S_{max}	T_B (s)	T_C (s)	T_D (s)
A	1,0	0,15	0,4	2,0
B	1,25	0,15	0,5	2,0
C	1,4	0,20	0,6	2,0
D	1,55	0,20	0,8	2,0
E	1,5	0,15	0,5	2,0

Table 101: Values of parameters describing the horizontal type 2 elastic response spectrum

Ground type	type 2 seismic action			
	S_{max}	T_B (s)	T_C (s)	T_D (s)
A	1,0	0,05	0,25	1,2
B	1,20	0,05	0,25	1,2
C	1,45	0,1	0,25	1,2
D	1,6	0,1	0,30	1,2
E	1,5	0,05	0,25	1,2

Table 102: Values of parameters for the vertical elastic response spectrum

Spectrum	A_{vg}/a_g	T_B (s)	T_C (s)	T_D (s)
Type 1	0,90	0,15	0,4	2,0
Type 2	0,45	0,05	0,15	1,0

B.7.6. Design response spectrum

According to the Czech Republic National Annex to EN 1998-1, reference values and upper limits for behaviour factors q to be used in Czech Republic for design of steel structure according to the wanted structural behaviour are given in the following table. The use of structures of ductility classes M and H is not limited provided that a qualified design, execution and quality control are guaranteed. Moreover, the lower-bound coefficient β is fixed to the recommended value, i.e., 0,2.

Table 103: Values of the behaviour factor q according the structural behaviour design

Design concept	Structural ductility class	behaviour factor q
Low dissipative structural behaviour	DCL (low ductility)	q ≤1,5
Dissipative structural behaviour	DCM (medium ductility)	1,5 < q ≤ 4
	DCH (high ductility)	q > 4

B.7.7. Damage limitation

In Czech Republic, values of the reduction factor v to be used to verify the damage limitation conditions are fixed to 0,4 for importance classes III and IV and 0,5 otherwise, as recommended in EN 1998-1.

B.8. United Kingdom

B.8.1. Seismic zoning

According to the UK National Annex of EN 1998-1 [53], the seismic hazard in UK is defined by means of the seismic hazard map given in Figure 54, which defines five seismic zones of increasing seismic activity. This hazard map corresponds to a mean return period of 2500 years A low seismicity case is defined as a region where the design ground acceleration is lower than 2m/s2 (for TNCR =2 500 years) and a very low seismicity case a region having a design ground acceleration ag not greater than 1.8m/s2 (for TNCR =2 500 years). It could be mentioned that in cases of very low seismicity, the provisions of EN 1998 need not to be observed.

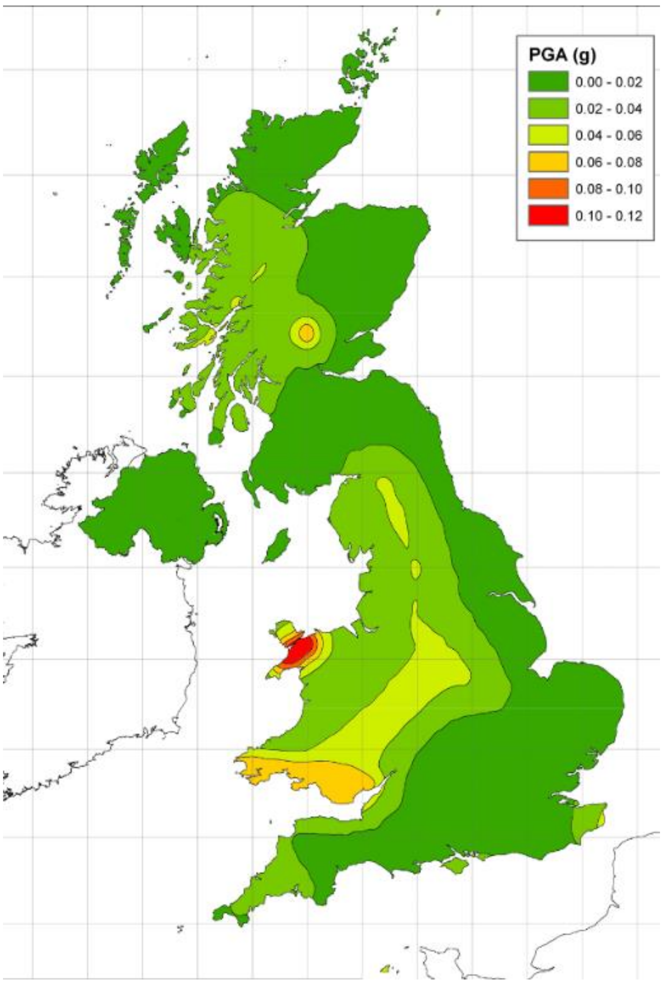

Figure 54: United Kingdom seismic zoning

B.8.2. Building classification

In UK, buildings are classified according to EN 1990, Table B1. In the absence of statutory requirements or contractual specifications, Seismic design is not required for buildings classified as being in consequence class CC1 or CC2, provided they are adequately designed for non seismic design conditions. For structures in consequence class CC3, the need for an seismic design should be considered taking into account an importance factor γI equal to 1.0.

B.8.3. Reference ground acceleration

Table 104 gives the values of the reference peak ground acceleration a_{gr} on ground type A for the defined seismic zones.

Table 104: Reference peak ground acceleration a_{gr} (m/s²) in UK

Seismic zones	1	2	3	4	5
a_{gr} (m/s²)	0,20	0,39	0,59	0,79	0,98

B.8.4. Influence of the soil

In UK, the soil factor S associated to each soil class are those indicated in EN 1998-1.

B.8.5. Elastic response spectrum

According to the UK National Annex of EN 1998-1, values of the main parameters defining the shape of the elastic response spectra are reported in the following table. They correspond to the so-called type 2 spectrum (Ms<5.5 that is low seismicity regions) defined in EN 1998-1.

Table 105: Values of parameters describing the horizontal elastic response spectrum

Ground type	Type 2 seismic action			
	S_{max}	T_B (s)	T_C (s)	T_D (s)
A	1,0	0,05	0,25	1,2
B	1,35	0,05	0,25	1,2
C	1,5	0,1	0,25	1,2
D	1,6	0,1	0,3	1,2
E	1,8	0,05	0,25	1,2

Table 106: Values of parameters for the vertical elastic response spectrum

A_{vg}/a_g	T_B (s)	T_C (s)	T_D (s)
0,45	0,05	0,15	1

B.8.6. Design response spectrum

Reference values and upper limits for behaviour factors q to be used in UK for design of steel structure according to the wanted structural behaviour are given in the following table. Moreover, the lower-bound coefficient β is fixed to the recommended value, i.e., 0,2.

Neither the use of the design concept nor the use of the ductility classes are subject to any limitations. In particular, there are no geographical limitations on the use of ductility classes M and H.

Table 107: Values of the behaviour factor q according the structural behaviour design

Design concept	Structural ductility class	behaviour factor q
Low dissipative structural behaviour	DCL (low ductility)	q ≤2
Dissipative structural behaviour	DCM (medium ductility)	1,5 < q ≤ 4
	DCH (high ductility)	q > 4

B.8.7. Damage limitation

In UK, values of the reduction factor v to be used to verify the damage limitation conditions are the recommended values of EN 1998-1, i.e. 0.4 for important classes I and II or 0.5 for important classes III and IV.

B.9. Netherlands

The Dutch building code [42] states the following about earthquakes in section 2.1 (overall strength of the building structure):

- In addition to the provisions of Articles 2.2 to 2.5a inclusive (general strength of load-bearing structure), further regulations may be given by ministerial regulation with regard to the loads on structures caused by earthquakes as a result of gas extraction in the province of Groningen.

At the time of writing, no ministerial regulation is given yet. For the time being, the competent authority is not allowed to access a calculation of earthquake load, because the Dutch Building Code 2012 does not impose any (public law) requirement for this.

The national Annex of the Eurocode 8 is in the Netherlands still in development. In the run-up to the National Annex, the NEN issued the NPR 9998:2018. The scope of the NPR is limited to the Northern Netherlands, where induced earthquakes as a result of gas extraction in the Groningen gas field (can) occur.

The NPR 9998:2020 "Assessment of the structural safety of buildings in case of erection, reconstruction and disapproval – induced earthquakes – Basis of design, actions and resistances" provides the guidelines for assessing whether:

a) New buildings to be built are sufficiently earthquake resistant,

b) Existing buildings are sufficiently earthquake resistant,

c) existing buildings after reinforcement are sufficiently earthquake resistant.

B.10. Italy

Seismic design in Italy is regulated by mandatory national standard, the NTC 2018 "Norme Tecniche per le Costruzioni" [59] that define both the main principles and rules to design the structures. The standard is supported by the explanatory document, "Circolare Esplicativa" NTC 2018 "Istruzioni per l'applicazione delle Nuove norme tecniche per le costruzioni" [63] that is not mandatory, and its focus is to be a guideline for the national standard.

B.10.1. Seismic zoning

According to the NTC 2018 Italian Standard, the seismic hazard is defined by means of the seismic hazard classification considering the local hazard. The Italian seismic zonation is based on the Probabilistic Seismic Hazard Assessment (PSHA) by means the MPS04-S1 model, see Figure 55, as of the European seismic hazard model SHARE.

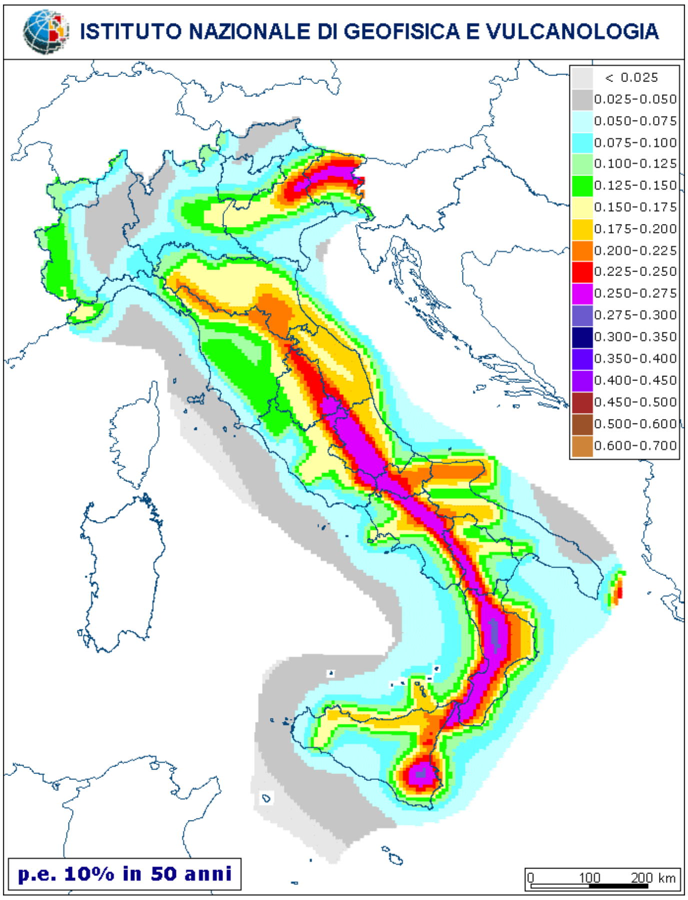

Figure 55: Italian seismic zonation by MPS04-S1 model [62].

The result of the MPS04-S1 classification is the definition of seismic parameters for 10751 points. In this way the national territory was subdivided into a grid whose nodes are no more than 10 km away, as shown in Figure 56. The local seismic parameters of the design location are obtained by interpolation of the node's parameters throughout the following design parameters:

- the maximum ground acceleration a_g;
- the maximum value of the amplification factor of the acceleration spectrum F_0;
- the reference value to define the upper limit of the period of the constant spectral acceleration branch T_C^*.

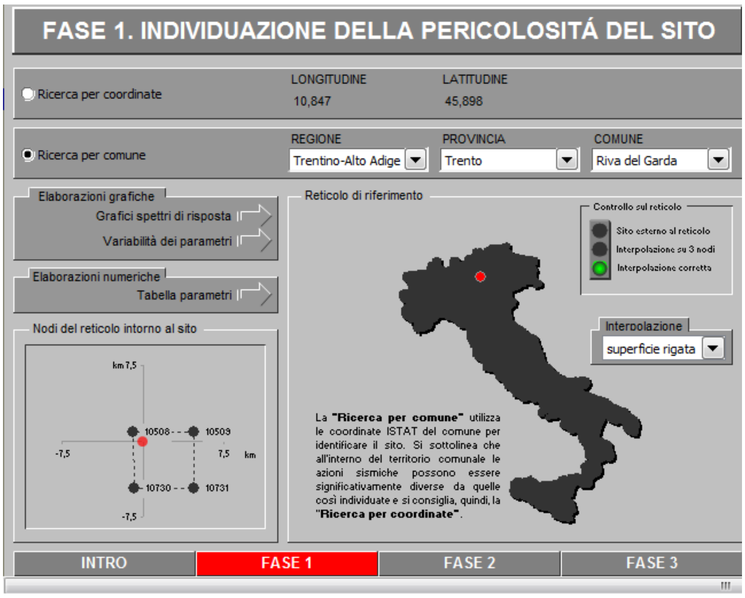

Figure 56: Italian seismic zonation to define design seismic parameters.

The seismic parameters are defined as a limit state function taking into account different probability of exceedance for the reference period, as summarized in Table 108.

Table 108: Limit state and probability of exceedance in reference period

Limit state	Probability of exceedance in the reference period	Return period associated to the reference period of 50 year
Operational - SLO	81%	30 years
Damage - SLD	63%	50 years
Life safety - SLV	10%	475 years
Collapse prevention - SLC	5%	975 years

The hazard seismic zonation exhibits a mean return period of 475 years for SLV (10% probability of exceedance in 50 years), i.e. the recommended reference period in EN 1998-1.

B.10.2. Building classification and class of use

The Italian regulation distinguish three types of buildings with an associated nominal period, as shown in Table 109, finalized to evaluate the reference period considering the coefficient of use, correlated to the use of structure and the importance, by means of following formulation:

$$V_R = V_N \cdot C_U$$

Table 109: Type of building with associated minimum reference period - V_N

Type of building		Minimum reference period
1	Temporary and provisional	10 years
2	With ordinary performance levels	50 years
3	With high performance levels	100 years

For building of type 1 the seismic design is not required.

The type of buildings is categorised into four classes depending *on* their usage with increasing importance, from class I to class IV, mainly depending on the activity taking place in the building, the number of people it can accommodate and the level of possible consequences.

The coefficient of use C_U assigned to each class is reported in Table 110. The meaning of this coefficient is different from the importance class stated in EN 1998-1, and then they are not directly comparable. The coefficient of use modifies the return period and consequently the seismic action, whereas the importance class directly changes the seismic action taking into account the different return period.

Table 110: Definition of classes of use of buildings

Class of use	Coefficient of use C_U	Buildings
I	0,7	Buildings with only occasional presence of people, e.g. agricultural buildings
II	1,0	Buildings whose use involves normal crowding, without dangerous content to the environment and without critical public and social functions. Bridges, infrastructural works, road networks that do not fall under Class of Use III or Class of Use IV, railway networks whose interruption does not cause emergency situations. Dams whose collapse does not cause significant consequences.
III	1,5	Buildings whose use involves significant crowding. Industries with dangerous activities for the environment. Extra-urban road networks that do not fall under Class of Use IV. Bridges and railway networks whose interruption leads to emergency situations. Dams with significant consequences related to their collapse.
IV	2,0	Buildings with important public or strategic role, also with reference to the management of civil protection in the event of a disaster. Industries with activities that are particularly dangerous for the environment. Road networks of type A or B, as per Ministerial Decree 5/11/2001, n. 6792, "Functional and geometric standards for road construction", and type C when belonging to routes connecting between provincial capitals not also served by type A or B roads. Bridges and railway networks of critical importance for the maintenance of communication routes, particularly after a seismic event. Dams connected to the operation of aqueducts and to electricity production plants.

In Italy the seismic design is mandatory for each type of structure and class of use. Simplified design is permitted only in the case of structures located in zone with low seismicity characterized by maximum horizontal ground acceleration amplified by soil factor lower than 0.075g.

Different checks are required as a function of the role played under seismic action from the elements, primary or secondary elements, and of systems such as the plumbing system, etc.

In the Italian standard the industrial buildings usually fall into the class of use II, III or IV as a function of the consequences of their damage due to the seismic event. The checks for different elements and systems as a function of the limit state are summarized in Table 111.

Table 111: Required checks as a function of class of use, limit state and type of element/system

LIMIT STATE		CU I	CU II			CU III and IV		
		ST	ST	NS	IM	ST	NS	IM
SLE	SLO					RIG		FUN
	SLD	RIG	RIG			RES		
SLU	SLV	RES	RES	STA	STA	RES	STA	STA
	SLC		DUT			DUT		

Legend:

- *CU:* *class of use;*
- *SLE:* *serviceability limit state;*
- *SLU:* *ultimate limit state;*
- *SLO:* *operational limit state;*
- *SLD:* *damage limit state;*
- *SLV:* *life safety limit state;*
- *SLC:* *collapse prevention limit state;*
- *ST:* *primary structural elements;*
- *NS* *non-structural elements;*
- *IM:* *system;*
- *RES:* *strength verification;*
- *RIG:* *stiffness verification;*
- *DUT:* *ductility verification;*
- *STA:* *stability verification;*
- *FUN:* *functional verification.*

B.10.3. Design ground acceleration

Due to the Italian seismic zoning through the definition of the main parameters for 10751 points there is not a value of ground acceleration for a defined zone like other standard. The value of maximum ground acceleration a_g together F_0 and T_C^* are given on ground type A for the location of the building.

In Italy, it can be noted that the design ground acceleration is the same for new and existing buildings. Moreover, where it has to be considered, the design vertical ground acceleration is defined on the same criteria of the horizontal acceleration adopting different coefficients.

B.10.4. Influence of the soil

Five soil classes are defined in Italy, see Table 112, with associated values of the stratigraphic amplification coefficient S_S and a coefficient correlated to the ground type C_C, as gathered in Table 113.

Table 112: Ground type in Italy

Ground type	Description of stratigraphic profile	v_s (m/s)
A	Rock masses outcropping or very rigid soils characterized by shear wave speed values exceeding 800 m/s, possibly including soil with characteristics on the surface of poorer mechanics with a maximum thickness of 3 m.	> 800
B	Soft rocks and deposits of very thickened coarse-grained soils or very consistent fine-grained soils, characterized by an improvement in mechanical properties with depth and by equivalent velocity values between 360 m/s and 800 m/s.	360÷800
C	Deposits of medium-thickened coarse-grained soils or medium-consistent fine-grained soils with substrate depths greater than 30 m, characterized by an improvement in mechanical properties with depth and equivalent velocity values between 180 m/s and 360 m/s.	180÷360
D	Deposits of poorly densified coarse-grained soils or poorly consistent ine-grained soils, with substrate depths greater than 30 m, characterized by improved mechanical properties with depth and equivalent velocity values between 100 m/s and 180 m/s.	100÷180
E	Soils with characteristics and equivalent velocity values attributable to those defined for categories C or D, with substrate depth not exceeding 30 m.	

Table 113: Stratigraphic amplification coefficient S_S and ground type coefficient C_C

Ground type	Stratigraphic amplification coefficient S_S	Ground type coefficient C_C
A	$1{,}00$	$1{,}00$
B	$1{,}00 \leq 1{,}40 - 0{,}40 F_0 \cdot \frac{a_g}{g} \leq 1{,}20$	$1{,}10 \cdot (T_C^*)^{-0{,}20}$
C	$1{,}00 \leq 1{,}70 - 0{,}60 F_0 \cdot \frac{a_g}{g} \leq 1{,}50$	$1{,}05 \cdot (T_C^*)^{-0{,}33}$
D	$0{,}90 \leq 2{,}40 - 1{,}50 F_0 \cdot \frac{a_g}{g} \leq 1{,}80$	$1{,}25 \cdot (T_C^*)^{-0{,}50}$
E	$1{,}00 \leq 2{,}00 - 1{,}10 F_0 \cdot \frac{a_g}{g} \leq 1{,}60$	$1{,}15 \cdot (T_C^*)^{-0{,}40}$

B.10.5. Elastic response spectrum

The horizontal component of the seismic action can be represented by an elastic ground response acceleration spectrum (also called elastic response spectrum), whose form is defined by the NTC 2018. The shape of response spectrum is correlate to the ground type by means of stratigraphic amplification coefficient S_S, the ground type coefficient C_C and topographic amplification coefficient S_T. The topographic amplification coefficient is correlated to the characteristics of topographic surface, Table 114, and the location of the building as stated in Table 115.

Table 114: Definition of the topographic categories

Topographic category	Characteristics of the topographic surface
T1	Flat surface, slopes and isolated elevations with an average inclination i ≤ 15 °
T2	Slopes with an average inclination i> 15 °
T3	Elevations with a much smaller width at the crest than at the base and an average inclination of 15 ° ≤ i ≤ 30 °
T4	Elevations with a much smaller width at the crest than at the base and with an average inclination of> 30 °

Table 115: Topographic amplification coefficient S_T

Topographic category	Building location	Topographic amplification coefficient S_T
T1	-	1,0
T2	At the top of the slope	1,2
T3	At the crest of elevations with average inclination less than or equal to 30 °	1,2
T4	At the crest of elevations with average inclination higher than 30 °	1,4

The NTC 2018 provisions define the elastic response spectrum for each limit state and as a function of the class of use and the location of the building contrary to EN 1998-1 that defines two types of elastic response spectra (type 1 and type 2) for varying seismicity conditions.

The shape of elastic response spectrum is defined starting from the location of the building and the correlated main parameters, maximum ground acceleration a_g, F_0 and T_C^*, whereas the other points of spectrum are defined taking into account the type of ground and the topographic aspect by the following equations.

- $T_C = C_C \cdot T_C^*$
- $T_B = T_C/3$
- $S = S_S \cdot S_T$
- $S_e(T) = a_g \cdot S \cdot \eta \cdot F_0 \cdot \left[\frac{T}{T_B} + \frac{1}{\eta \cdot F_0}\left(1 - \frac{T}{T_B}\right)\right]$ $\quad 0 \le T < T_B$
- $S_e(T) = a_g \cdot S \cdot \eta \cdot F_0$ $\quad T_B \le T < T_C$
- $S_e(T) = a_g \cdot S \cdot \eta \cdot F_0 \cdot \left(\frac{T_C}{T}\right)$ $\quad T_C \le T < T_D$
- $S_e(T) = a_g \cdot S \cdot \eta \cdot F_0 \cdot \left(\frac{T_C \cdot T_D}{T^2}\right)$ $\quad T_D \le T$

In some cases, the vertical component should be also considered. For instance, when the beam span exceeds 20 m, which is a common case for single-storey buildings.

B.10.6. Design response spectrum

According to NTC 2018, in order to avoid explicit inelastic structural analysis in design, the capacity of the structure to dissipate energy, through mainly ductile behaviour of its elements and/or other mechanisms, is taken into account by performing an elastic analysis based on a design response spectrum reduced with respect to the elastic one by introducing a behaviour factor q.

From a practical point of view the design response spectrum is obtained by replacing in the previous equations the η parameter with 1/q.

The behaviour factor is correlated with:
- ductility class;
- the type of structure;
- the value α_u/α_1.

Differently from EN 1998-1, the Italian standard, NTC 2018, defines only two ductility class, the medium ductility class CD "B" and the high ductility class CD "A" defining the maximum behaviour factor, q_0, for the different type of structure, as summarized in Table 116.

Table 116: Upper limit of reference values of behaviour factors for SLV

Structural type	q_0	
	CD "A"	CD "B"
Moment resisting frames Frame with eccentric bracings	$5{,}0\,\alpha_U/\alpha_1$	$5{,}0\,\alpha_U/\alpha_1$
Frame with concentric diagonal bracings	4,0	4,0
Frame with concentric V-bracings	2,0	2,0
Inverted pendulum	$2{,}0\,\alpha_U/\alpha_1$	2,0
Moment resisting frame with concentric bracing	$4{,}0\,\alpha_U/\alpha_1$	4,0
Moment resisting frames with masonry infills	2,0	2,0

In agreement with Table 116, the behaviour factor exhibits either a constant value or a variable value depending on the ratio α_u/α_1, where α_u is the coefficient by which the seismic action has to be multiplied to reach the collapse of the structure, and α_1 to reach first yield in any of its components (equivalent to the formation of the first plastic hinge). The α_u/α_1 values are summarized in Table 117.

Table 117: Upper limit values of α_u/α_1 factors for structures regular in plan

Characteristic of the structure	α_U/α_1
Single-story buildings	1,1
Multi-storey frame buildings, with a single span	1,2
Multi-storey and multi-span frame buildings	1,3
Multi-storey frame buildings with eccentric bracings	1,2
Inverted pendulum	1,0

The value of behaviour factor is correlated with cross-sectional classes of the steel elements that dissipate energy to allow a suitable ductile behaviour, as indicated in Table 118.

Table 118: Requirements on cross-sectional class of dissipative elements depending on Ductility Class and reference behaviour factor SLV

Ductility class	Reference value of behaviour factor q_0	Required cross-sectional class
CD "B"	$2{,}0 < q_0 \leq 4{,}0$	Class 1 or 2
CD "A"	$4{,}0 < q_0$	Class 1

B.10.7. Damage limitation

The NTC 2018 defines the elastic response spectrum for the serviceability limit state in similar way to the horizontal response spectrum. Analogously to EN 1998-1, the verification is based on deformability checks in terms of limiting the interstorey drifts correlated both to the typology of the infill elements and to class of use, as summarized in Table 119.

Table 119: Limitation of understorey drift

Characteristic of elements	CU I & CU II	CU III & CU IV
Brittle non-structural elements	$q \cdot d_r \leq 0{,}005h$	$q \cdot d_r \leq \frac{2}{3} 0{,}005h$
Ductile non-structural elements	$q \cdot d_r \leq 0{,}0075h$	$q \cdot d_r \leq \frac{2}{3} 0{,}0075h$
Elements that can be damaged due to their ductile behaviour	$q \cdot d_r \leq 0{,}010h$	$q \cdot d_r \leq \frac{2}{3} 0{,}010h$

B.11. Switzerland

Seismic design in Switzerland is regulated by national standard SIA 261 "Actions sur les structures porteuses" [65] that defines both the main principles and rules. The standard is supported by the explanatory document, SIA 261/1 "Actions sur les structures porteuses – Spécifications complémentaires" and its focus is to be a guideline for the national standard.

B.11.1. Seismic zoning

The Swiss territory is divided into four seismic zones of increasing seismic activity. For each zone the seismic hazard is assumed constant. The map of seismic zonation is given in the Annex F of the standard SIA 261 (see Figure 57). This hazard map corresponds to a mean return period of 475 years (10% probability of exceedance in 50 years), i.e. the recommended reference period in EN 1998-1.

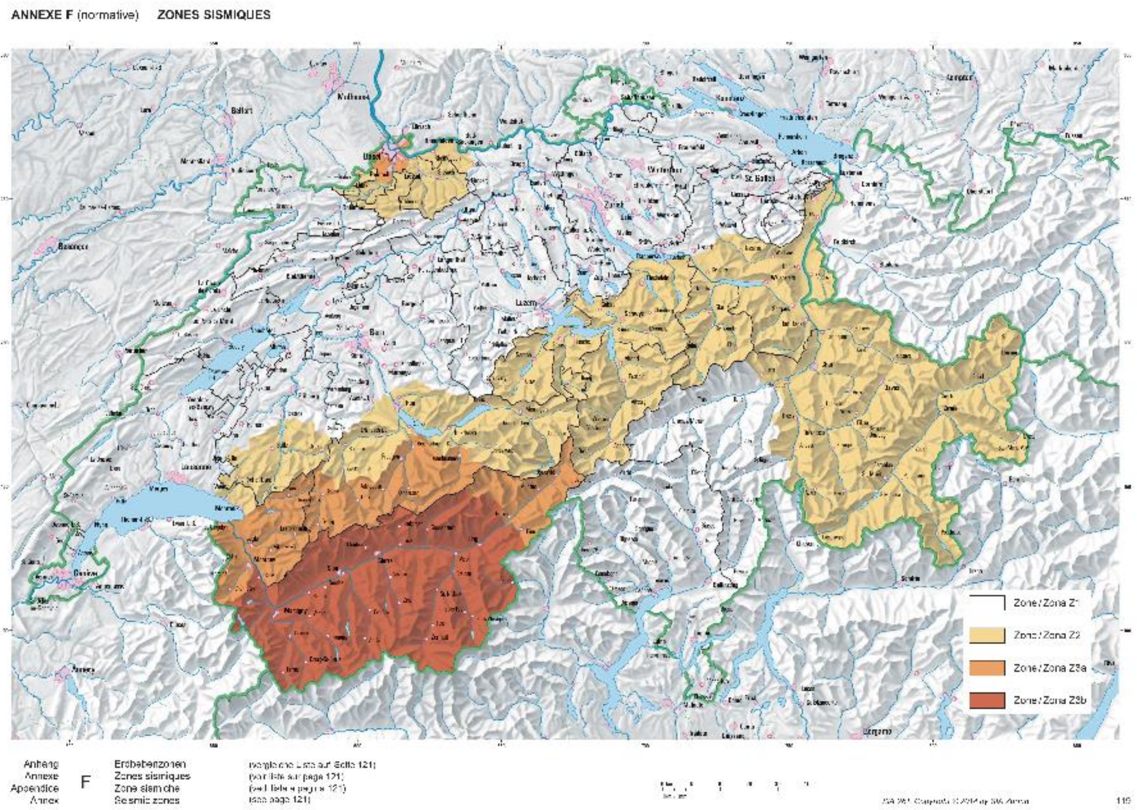

Figure 57: Swiss seismic zonation (corresponding to a reference return period of 475 years with 10% probability of exceedance in 50 years)

B.11.2. Building classification and importance factors

The Swiss standard categorises the buildings/structures into three classes of increasing importance, from category I to category III, mainly depending on the activity taking place in the building and the number of people it can accommodate. The types of buildings associated with each importance class as well as the importance factor γ_I assigned to each importance class are reported in Table 120. They correspond to the recommended values given in EN 1998-1.

Table 120: Definition of importance classes and importance factor

Classes	Factor γ_I	Characteristics	Examples
I	1,0	- Occupancy PB ≤ 50 people - No large gatherings of people - No goods or facilities of particular value - Harm to the population or the environment excluded	- Residential buildings, administrative and craft buildings - Industrial buildings and warehouses - Parkings - Bridges whose importance after an earthquake is low (e.g. pedestrian bridges and bridges for agricultural or forestry, if they do not cross important communication routes)
II	1,2	- Occupation PB> 50 people - Attendance possible by many people - Goods or facilities of special value - Infrastructure with an important function	- Hospitals with their equipment and installations, if they do not belong to class III - Shopping centres, stadiums, cinemas, theatres, schools and churches - Public administration buildings - Bridges of great importance after an earthquake or bridges crossing important communication routes after an earthquake - Retaining walls and embankments bordering important communication routes after an earthquake - Works, equipment and installations intended for supply, disposal and telecommunications, if not in class III - High chimneys
III	1,4	- Infrastructure with a vital function	- Emergency hospitals with their equipment and facilities - Structures, equipment and installations used for disaster protection (e.g. fire department buildings or ambulance garages) - Bridges of great importance for serving a region after an earthquake - Retaining walls and embankments bordering essential communication routes for serving certain structures or a region after an earthquake - Structures of vital importance for supply, disposal and telecommunications

Comparing the Swiss classification with EN 1998-1, it is important to highlight the absence of the lower class “Buildings of minor importance for public safety, e.g. agricultural buildings, etc.”

In practice, the single-storey buildings for industrial, commercial and artisanal uses are commonly classified as of importance classes I or II, depending on the number of people present in the building.

In Switzerland the seismic design, in term of strength, is required for each class of importance, whereas the serviceability limit state is required only for structures of class III.

Seismic design of structures is little different from the conception stated in EN 1998-1. The Swiss standard defines both the design and the structural requirements correlated to the seismic zone, as reported in Table 121, whereas the behaviour factor is linked to the cross-sectional class and the type of structure. Moreover, the structural behaviour can be dissipative or non-dissipative, but in the case of dissipative behaviour there are no different classes of ductility.

Table 121: Design and structural requirements

Requirement	* suggested ** exceptions to be justified *** mandatory	Z1/CO I Z1/CO II Z2/CO I	Z1/CO III Z2/CO II Z3/CO I	Z2/CO III Z3/CO II Z3/CO III
Plan view, construction provisions				
- The elements able to withstand the horizontal forces (frames, load-bearing walls with core, triangulated bracing, etc.) should be distributed in the most symmetrical way and shall be characterised by similar deformation capacities. Ensure a homogeneous behaviour of the structure by an appropriate arrangement of floors, bracing, etc.		*	**	**
- For the elements having to withstand horizontal forces, variations (in the vertical direction) of stiffness and resistance to bending, shearing force and torsion (exception: transition to basements) should be avoided.		*	**	**
- A rigid box at basement level should be created.		*	**	**
Construction				
- The prefabricated elements to the other elements of construction shall be joined.		***	***	***
- For prefabricated elements with movable supports, a support length equal to 1/70 of the span, but at least 150 mm, should be provided.		***	***	***
- Non-structural elements should be checked		***	***	***
Foundation				
- It is not recommended to build a load-bearing structure on a very variable ground.		*	**	***
- To ensure uniform movement, isolated foundations located in soft ground or connected by sleepers, etc. should be avoided.		*	**	***

B.11.3. Reference peak ground acceleration

Table 122 gives the values of the reference peak ground acceleration a_{gd} on ground type A for the each seismic zone.

Table 122: Reference peak ground acceleration a_{gd} (m/s²)

Seismic zones	Z1	Z2	Z3a	Z3b
a_{gd} (m/s²)	0,6	1,0	1,3	1,6

As can be noticed in Table 122, the reference peak ground acceleration ranges from 0,6 to 1,6 m/s².

B.11.4. Influence of the soil

Six soil classes are defined with an associated value of the soil factor reported in Table 123.

Table 123: Ground type classification and soil factor

Ground type	Description of stratigraphic profile	v_s (m/s)	Soil factor S
A	Rock or other rock-like geological formation, including at most 5 m of weaker material at the surface.	> 800	1,00
B	Deposits of very dense sand, gravel, or very stiff clay, at least several tens of metres in thickness, characterised by a gradual increase in mechanical properties with depth.	500÷800	1,20
C	Deep deposits of dense or medium dense sand, gravel or stiff clay with thickness from several tens to many hundreds of metres.	300÷500	1,15
D	Deposits of loose-to-medium cohesionless soil (with or without some soft cohesive layers), or of predominantly soft-to-firm cohesive soil.	< 300	1,35
E	A soil profile consisting of a surface alluvium layer with v_s values of type C or D and thickness varying between about 5 m and 20 m, underlain by stiffer material with v_s > 800 m/s.	-	1,40
F	Structurally sensitive, organic or very soft deposits (e.g. peat, lake chalk, soft silt) with a thickness greater than 10 m	-	-

B.11.5. Elastic response spectrum

The seismic action can be represented by an elastic ground response acceleration spectrum (also called elastic response spectrum), whose form and the values of the various parameters are defined by the national standard SIA 261. The response spectrum is thus established from standard formulas, which depend on the maximum design peak ground acceleration, the soil factor S and the characteristic periods noted TB, TC and TD.

Contrary to EN 1998-1, the national standard defines the parameters for only one elastic response spectrum correlated to the type of ground, but independent from the seismic zone. The main parameters are summarized in Table 124.

Table 124: Periods TB, TC and TD for the horizontal elastic response spectrum

Ground type	T_B (s)	T_C (s)	T_D (s)
A	0,15	0,4	2,0
B	0,15	0,5	2,0
C	0,20	0,6	2,0
D	0,20	0,8	2,0
E	0,15	0,5	2,0
F	-	-	-

The shape of elastic response spectrum is defined taking into account the type of ground and the soil factor by the following equations.

- $S_e = a_{gd} \cdot S \left[1 + \frac{(2{,}5 \cdot \eta - 1)T}{T_B}\right]$ $\quad 0 \leq T < T_B$
- $S_e(T) = 2{,}5 \cdot a_{gd} \cdot S \cdot \eta$ $\quad T_B \leq T < T_C$
- $S_e(T) = 2{,}5 \cdot a_{gd} \cdot S \cdot \eta \cdot \frac{T_C}{T}$ $\quad T_C \leq T < T_D$
- $S_e(T) = 2{,}5 \cdot a_{gd} \cdot S \cdot \eta \cdot \frac{T_C \cdot T_D}{T^2}$ 0 $\quad T_D \leq T$

In the case where vertical seismic actions are to be considered, the design vertical ground acceleration is obtained multiplying the horizontal spectrum by 0,7.

B.11.6. Design response spectrum

According to national standard SIA 261, in order to avoid explicit inelastic structural analysis in design, the capacity of the structure to dissipate energy, through mainly ductile behaviour of its elements and/or other mechanisms, is taken into account by performing an elastic analysis based on a design response spectrum, reduced with respect to the elastic one by introducing a behaviour factor q.

The behaviour factor is linked to the cross-sectional class and the type of structure in agreement to the SIA 263 standard [66], and contrary to EN 1998-1 it is independent of the ratio α_u/α_1, as reported in Table 125.

A noteworthy observation is that in agreement with the national standard SIA 263 the behaviour of the structures can be dissipative or non-dissipative, but in the case of dissipative behaviour there are no different classes of ductility.

Table 125: Maximum values of the behaviour factor q according to cross-section class and structural behaviour design

Behaviour of structure	Type of structure	Cross-section class			
		Class 1	Class 2	Class 3	Class 4
Non dissipative behaviour	-	q=2			q= 1,5
Dissipative behaviour	Moment resisting frames	q=5	q=4	q=2	No
	Frames with concentric active tension diagonal bracings	q=4	q=4	q=2	No
	Frames with concentric V bracings	q=2,5	q > 4	q=2	No

B.11.7. Damage limitation

The serviceability limit state is required only for the structures of class III. No requirement is indicated for the other classes. Verification is required for non-structural elements with potentially dangerous consequences.