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DESIGN RECOMMENDATIONS AGAINST PROGRESSIVE COLLAPSE IN STEEL AND STEEL-CONCRETE BUILDINGS

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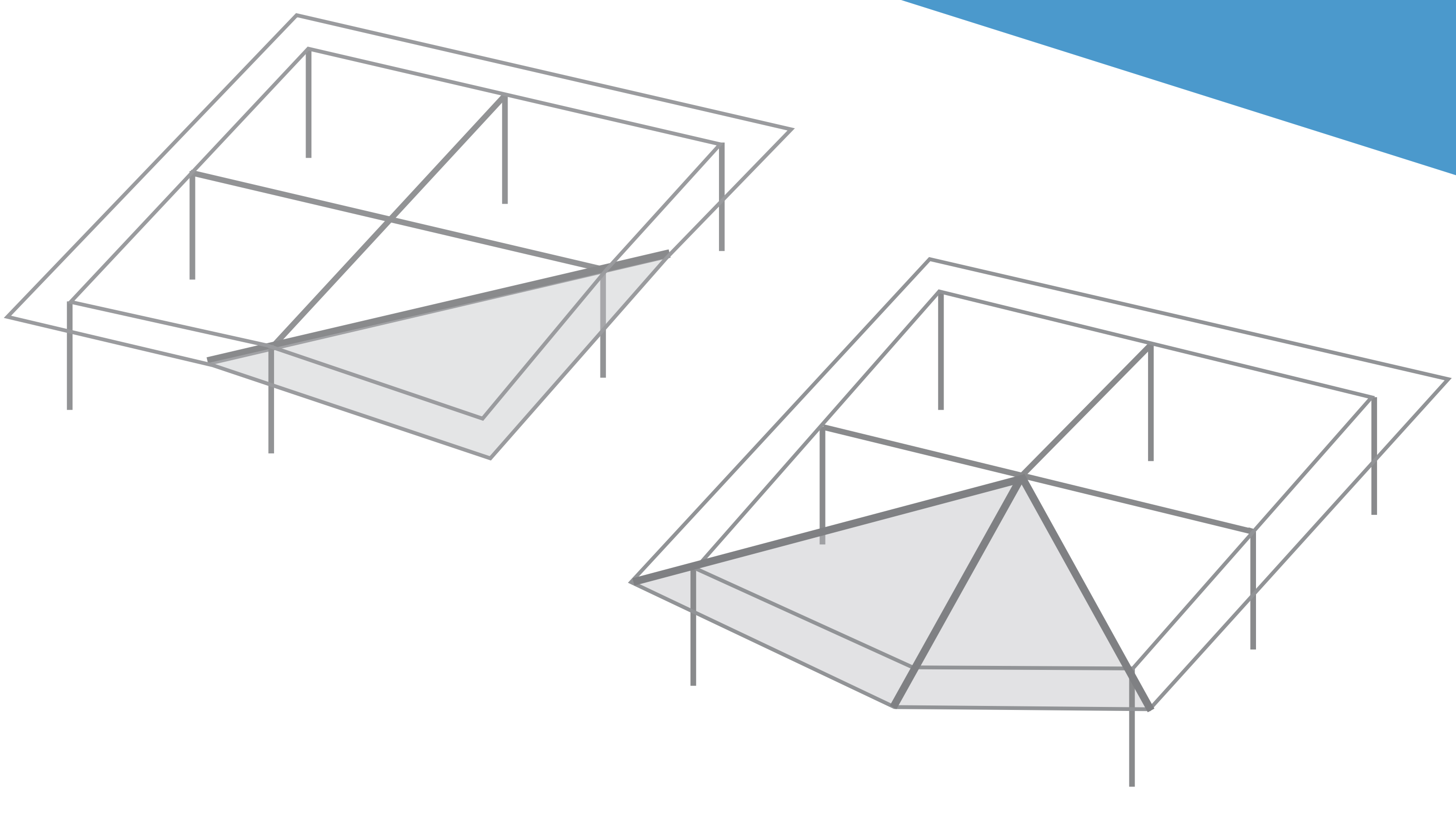

Mitigation of the risk of progressive collapse
in steel and composite building frames
under exceptional events

FAILNOMORE

D3-1

D3-1: Design recommendations against progressive collapse in steel and steel-concrete buildings

December 2021

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Definitions

In the present section, definitions of important terms relevant to the topic and proposed in the literature are listed.

Accidental actions / events

(EN 1990, 2002) – Action, usually of short duration but of significant magnitude, that is unlikely to occur on a given structure during the design working life.

Consequence

(EN 1991-1-7, 2006) – A possible result of an event. Consequences may be expressed verbally or numerically in terms of loss of life, injury, economic loss, environmental damage, disruption to users and the public, etc. Both immediate consequences and those that arise after a certain time has elapsed are to be included.

Deflagration

(EN 1991-1-7, 2006) - Propagation of a combustion zone at a velocity that is less than the speed of sound in the unreacted medium.

Detonation

(EN 1991-1-7, 2006) - Propagation of a combustion zone at a velocity that is greater than the speed of sound in the unreacted medium.

Dynamic force

(EN 1991-1-7, 2006) – Force that varies in time and which may cause significant dynamic effects on the structure; in the case of impact, the dynamic force represents the force with an associated contact area at the point of impact.

Dynamic increase factor (DIF - dynamic properties of the materials)

Multiplication factor for the mechanical properties under static loading to account for the effects of strain rates.

Dynamic load factor (DLF - dynamic amplification of the load)

Multiplication factor for the static load to account for the effects of the kinetic energy.

Equivalent static force

(EN 1991-1-7, 2006) - Alternative representation for a dynamic force including the dynamic response of the structure.

Hazard

(EN 1990, 2002) – An unusual and severe event, e.g., an abnormal action or environmental influence, insufficient strength or resistance, or excessive deviation from intended dimensions.

Hazard Scenario

(EN 1991-1-7, 2006) – A critical situation at a particular time consisting of a leading hazard together with one or more accompanying conditions which leads to an unwanted event (e.g., complete collapse of the structure).

Key element

(EN 1991-1-7, 2006) - A structural member upon which the stability of the remainder of the structure depends.

Localised failure

(EN 1991-1-7, 2006) – Part of a structure that is assumed to have collapsed, or been severely disabled, by an accidental event.

Progressive collapse (disproportionate collapse)

(JRC, 2012) – Progressive collapse of a building can be regarded as the situation where local failure of a primary structural component leads to the collapse of adjoining members and to an overall damage which is disproportionate to the initial cause.

Resilience

(Adam et al., 2018) – The resilience of a building includes not only the structural property of robustness, which contributes to the capacity of absorbing an extreme event, but also a recovery capacity that allows the pre-event performance level to be quickly restored or even improved.

Risk

(EN 1991-1-7, 2006) – A measure of the combination (usually the product) of the probability or frequency of occurrence of a defined hazard and the magnitude of the consequences of the occurrence.

Risk analysis

(EN 1991-1-7, 2006) – A systematic approach for describing and/or calculating risk. Risk analysis involves the identification of undesired events, and the causes and consequences of these events.

Risk assessment

(CSA, 1991) – A process of risk analysis and risk evaluation (with risk evaluation containing risk acceptance and option analysis).

Robustness

(EN 1991-1-7, 2006) - Ability of a structure to withstand events like fire, explosions, impact or the consequences of human error, without being damaged to an extent disproportionate to the original cause.

Structural integrity

(ASCE 7-05, 2006) – Property of being able “to sustain local damage with the structural system as a whole remaining stable and not being damaged to an extent disproportionate to the original local damage”.

Vulnerability

(Starossek and Haberland, 2010) – Susceptibility of a structure to suffer initial damage when affected by abnormal events. A structure is vulnerable if abnormal events easily lead to initial damage.

Introduction

Structural robustness and mitigation of progressive collapse is a specific safety consideration which is now addressed in modern codes and standards, including the Eurocodes, and which requires particular care from all professionals involved in the construction industry, including architects, designers, constructors, control officers, and insurance managers. The importance of the robustness design has been recognised by world shaking disasters such as the 9/11 collapse of Twin Towers in New York City and the need for practical guidelines has been triggered. Indeed, the availability of such guidelines for practical application addressed to various construction professionals considering different use and risk of buildings helps to ensure confidence in safety of steel and composite construction.

During the past decade, a significant number of research projects related to the structural response of steel and composite buildings under various exceptional loading situations (impact, fire, earthquake,…) have been carried out, especially in Europe and in the USA. As an outcome of these recent scientific actions, different possible practical methods have been proposed to achieve the mitigation of progressive collapse through effective designs and accounting for the full potential of material characteristics in steel and composite structures.

Purpose of the project "Mitigation of the risk of progressive collapse in steel and composite building frames"- FAILNOMORE, is to consolidate the knowledge developed in the aforementioned research and transform it into practical recommendations and guidelines. The set of practical and user-friendly design guidelines for mitigating the risk of progressive collapse is focused on steel and composite structures subjected to exceptional events such as impact, explosions, fire, seismic, referring also to available normative documents, in order to propose a commonly agreed European design methodology. The project was funded for 24 months (starting from July 2020) by the Research Fund for Coal and Steel (RFCS) under grant agreement No 899371.

The FAILNOMORE project partners are:

- University of Liège (ULG) – Belgium
- University of Coimbra (UC) – Portugal
- Imperial College London (IC) – UK
- University of Stuttgart (USTUTT) – Germany
- University of Trento (UNITN) – Italy
- Politehnica University Timisoara (UPT) – Romania
- Czech Technical University of Prague (CVUT) – Czech Republic
- Rzeszow University of Technology (PRZ) – Poland
- Technical University of Delft (TUD) – The Netherlands
- Universitat Politècnicade Catalunya (UPC) – Spain
- INSA de Rennes (INSAR) – France
- European Convention for Constructional Steelwork (ECCS) – Europe
- Feldmann+ Weynand GmbH (F+W) – Germany
- ArcelorMittal Belval & Differdange S.A. (AM) – Luxembourg

The present design manual is a part of dissemination material and reflects the main outcomes of the FAILNOMORE project. The present document is divided into three parts:

- Part 1 entitled "Design for robustness" which reflects the design strategies and the design approaches to be adopted. In particular, the normative context is first presented in Section 1. Then, in Section 2, the design methodology to be followed and the design strategies which can be adopted are briefly introduced. Finally, detailed and practice-oriented design guidelines on how to apply the different proposed design approaches are provided from Section 3 to 6 with general conclusions drawn in Section 7.

- Part 2 entitled “Worked examples” which illustrates how the different design approaches can be applied on four actual design examples: a steel and a composite structure designed in a non-seismic area and a steel and a composite structure designed in a seismic area.
- Part 3 entitled “Annexes” which provides the reader with additional detailed information regarding some of the proposed design concepts.
- Part 4 entitled “References” which collects the references mentioned in the present document.

Part 1 – Design for Robustness

1 Normative context

1.1 Overview

This chapter provides a brief overview of current robustness-related procedures in existing codes and guidelines, with particular emphasis on the requirements available in EN 1990 and EN 1991-1-7. Relevant design provisions in other international codes, as well as in other Eurocodes such as EN 1993, EN 1994 and EN 1998 are referred to where necessary in other parts of this document, and are discussed in detail in the background document (Demonceau et al., 2021) and the design guide (Elghazouli et al., 2021). Selected robustness-related developments which are currently under consideration for possible inclusion within the process of revision and evolution of the second generation of the Eurocodes are also outlined herein.

1.2 Robustness requirements in Eurocodes

1.2.1 Basic principles

EN 1990, 2.1 (4)P (EN 1990, 2002) sets out the basic principle related to structural robustness, where it is explicitly stated that: "A structure shall be designed and executed in such a way that it will not be damaged by events such as: explosion, impact, and the consequences of human errors, to an extent disproportionate to the original cause". In addition, to deal with hazard mitigation related to structural collapse, EN 1990, 2.1 (5)P states that: "Potential structural damage should be avoided or limited by one or more of the following: (i) avoiding, eliminating or reducing hazards applied on the structure; (ii) selecting a structural form with low sensitivity to the hazard; (iii) selecting a form and design which can survive removal of individual or limited parts of the structure; (iv) avoiding systems that collapse without warning; (v) tying members together".

1.2.2 Design situations

According to EN 1990, 3.2 (2)P, the design situation of relevance to structural robustness is the *Accidental Design Situation* which refers to exceptional conditions applicable to the structure or to its exposure to: e.g., fire, explosion, impact or the consequences of localised failure. EN 1990 also separates accidental design situations (e.g., fire, impact, blast, localised failure) from seismic ones (Elghazouli, 2013). Besides, EN 1990, 3.2 (3)P states that the selected design situations shall be sufficiently severe and varied so as to encompass all conditions that can reasonably be foreseen to occur during the execution and use of the structure. On the other hand, "Robustness" is defined in EN 1991-1-7 (EN 1991-1-7, 2006) as "the ability of a structure to withstand events like fire, explosion, impact or the consequences of human error, without being damaged to an extent disproportionate to the original cause". This definition, therefore, links robustness to accidental design situations, where the combination of actions for such situations is given in EN 1990, 6.4.3.3, Eq. (6.11b). It is also worth noting that deliberate malicious/terrorist actions are not strictly within the definition of accidental scenarios in Eurocodes; however, it is the responsibility of the engineer to consider the robustness of structures under all these extreme loads. A similar situation is also related to the stability and residual capacity of buildings following seismic or fire actions, which is not directly covered by the robustness requirements in EN 1991-1-7. However, EN 1998-1 (EN 1998-1, 2004) gives general rules on seismic actions and rules for buildings, while EN 1993-1-2 (EN 1993-1-2, 2005) and EN 1994-1-2 (EN 1994-1-2, 2005) give general rules for structural fire design of steel and composite buildings respectively.

1.2.3 Consequence classes

The design strategies for accidental design situations to meet the robustness requirements are based on the consequence class of the structure. The proposed classification in Annex A of EN 1991-1-7 categorises buildings into four consequence classes (CC) considering the building type, occupancy, and size. In EN 1990 and EN 1991-1-7, Cl 3.4, only three consequence classes are identified. However, in Annex A of EN 1991-1-7, Table A.1, Consequence Class 2 is subdivided into CC2a (medium consequences-lower risk group) and CC2b (medium consequences-upper risk group), with the other classes being CC1 (low consequences of failure) and CC3 (high consequences). More details regarding the consequence class of buildings as adopted herein can be found in Section 3.

1.3 Robustness strategies

1.3.1 General

As stipulated in EN 1991-1-7, the strategy adopted for hazard mitigation and the design of structures for accidental actions would depend on whether the accidental actions are identified or unidentified as summarised in Figure 1.

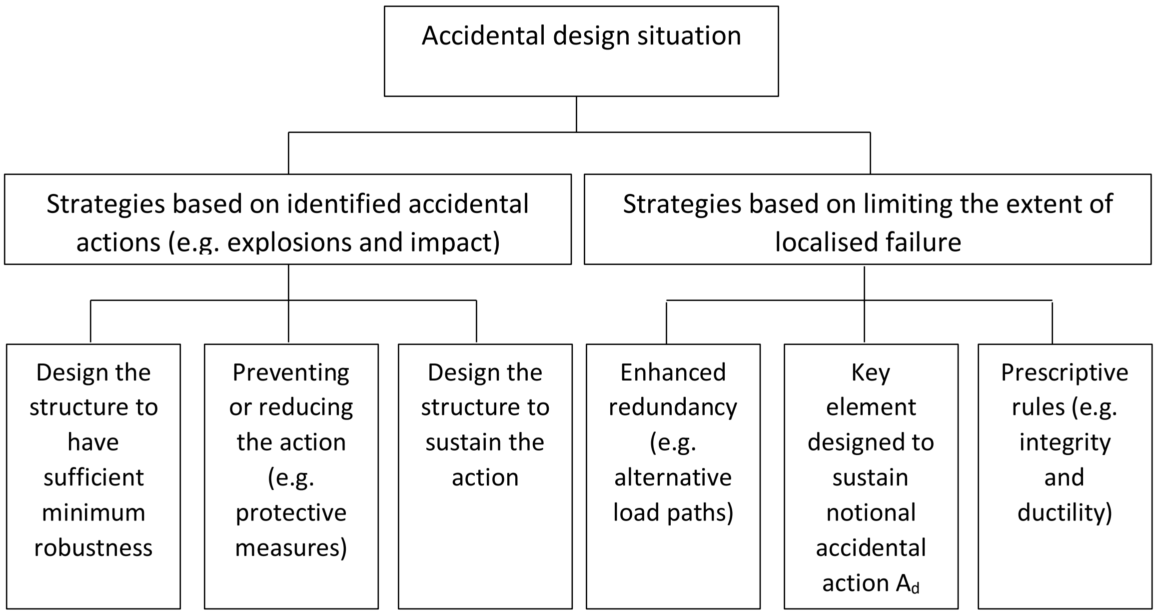


Figure 1. Robustness strategies for accidental design situations in (EN 1991-1-7, 2006)

1.3.2 Strategies based on identified accidental actions

EN 1991-1-7, 3.2 states that when accidental actions are identified and taken into account, the following factors should be also considered: (i) the measures taken for preventing or reducing the severity of an accidental action; (ii) the probability of occurrence of the identified accidental action; (iii) the consequences of failure due to the identified accidental action; (iv) public perception; (v) the level of acceptable risk. It also states that under such actions, localised failure may be acceptable provided it will not endanger the stability of the whole structure, and that the overall load-bearing capacity of the structure is maintained and allows necessary emergency measures to be taken.

Additionally, it emphasises that measures should be taken to mitigate the risk of accidental actions and these measures should include, as appropriate, one or more of the following strategies: (i) preventing the action from occurring or reducing the probability and/or magnitude of the action to an acceptable level through the structural design process; (ii) protecting the structure against the effects of an accidental action by reducing the effects of the action on the structure; (iii) ensuring that the structure has sufficient robustness by adopting one or more of the following approaches: a) designing certain components of the structure, upon which stability depends, as key elements to increase the likelihood of the structure survival following an accidental event; b) designing structural members, and selecting materials, to have sufficient ductility capable of absorbing significant strain energy without rupture; c) incorporating sufficient redundancy in the structure to facilitate the transfer of actions to alternative load paths following an accidental event.

The notional values for identified accidental actions (e.g., in the case of impact or internal explosion) are proposed in EN 1991-1-7. These values may be changed in the National Annex for individual countries or for a specific project and agreed in the design by the relevant authority and the client.

1.3.3 Strategies based on limiting the extent of localised failure

Strategies based on limiting the extent of localised failure cover a wide range of possible events and are mostly related to unidentified accidental actions. The adoption of strategies for limiting the extent of localised failure may provide adequate robustness against other accidental actions apart from those covered by EN 1991-1-7 (e.g., external explosions and terrorist attacks) or any other actions resulting from unspecified causes. For most building structures, potential accidental actions are mostly unidentified, hence designing structures for such situations would involve robustness strategies largely based on limiting the extent of failure using one of the following approaches, as stated in EN 1991-1-7, Cl 3.3: (i) designing key elements, on which the stability of the structure depends, to sustain the effects of a representation of accidental action; (ii) in the event of localised failure, such as failure of a single primary member, the stability of the structure or a significant part of it is not endangered; (iii) applying prescriptive design and detailing rules that provide acceptable robustness for the structure. Such strategies include prescriptive tying force methods, alternative load paths methods and key element design methods. They aim to provide an acceptable level of robustness to sustain localised failure without a disproportionate level of collapse.

Annex A of EN 1991-1-7 further details the application of such strategies to the different building categories. More stringent requirements are recommended going from CC1 to CC3, reflecting the increased level of risk due to structural collapse.

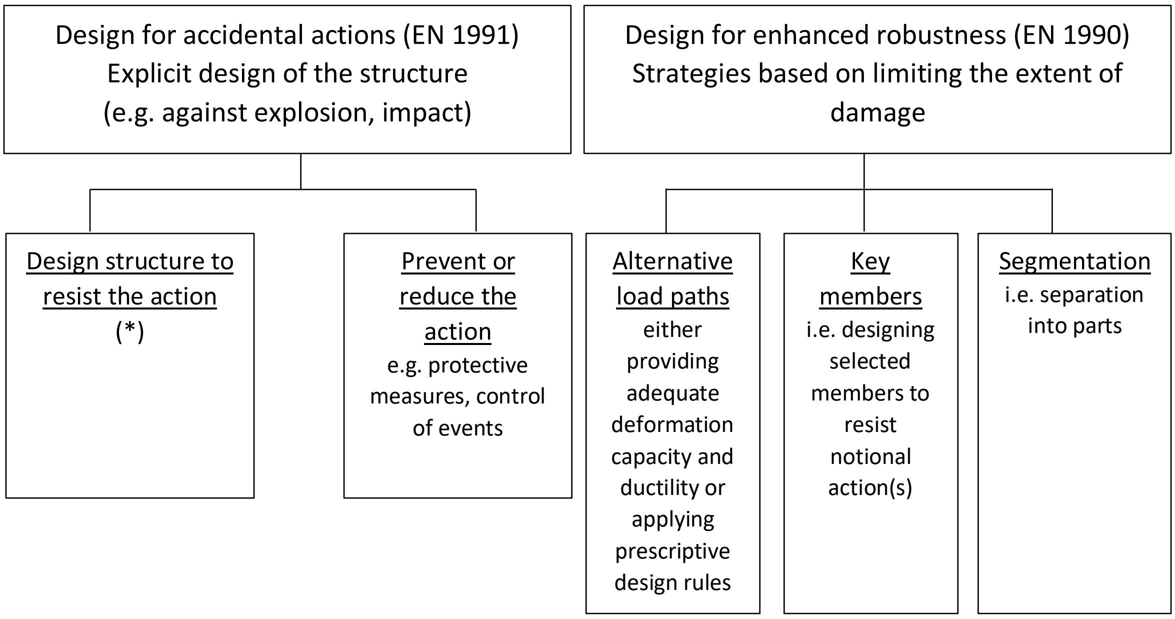
Both EN 1993 and EN 1994 provide recommendations which may be either directly or indirectly relevant to the design and detailing for robustness, including information related to the ductility and rotation capacity of beams and partial-strength joints, amongst others.

Various robustness requirements also exist in other international guidelines. These include, but are not limited to: the Unified Facilities Criteria (UFC): Design of Buildings to Resist Progressive Collapse (UFC 4-023-03, developed by USA Department of Defense (DoD, 2016), the USA General Services Administration (GSA, 2016) Alternate Path Analysis and design guidelines, recommendations included within ASCE 7-16 (ASCE, 2017b) and the International Building Code (IBC) (ICC, 2018), in addition to the stipulations in the UK Building Regulations 2010 Approved Document A (ODPM, 2013) as well as the Chinese Code for Anti-Collapse Design of Building Structures (CECS 392) (CECS, 2014). As mentioned before, these provisions are referred to where necessary in other parts of this document, and are described in more detail in the background document (Demonceau et al., 2021).

1.4 Current normative developments

The current draft revision of EN 1990 (prEN 1990, 2019) for the forthcoming second generation of Eurocodes introduces Section 4.4 and Informative Annex E, which are exclusively dedicated to structural robustness. Section 4.4 states that: “A structure should be designed to have an adequate level of robustness so that, during its design service life it will not be damaged by adverse and unforeseen events, such as the failure or collapse of a structural member or part of a structure, to an extent disproportionate to the original cause”. It also notes that for most structures, design in accordance with the Eurocodes provides an adequate level of robustness without the need for any additional design measures to enhance structural robustness; if such measures are needed, it should be specified by the relevant authority or agreed for a specific project by the relevant parties. On the other hand, Annex E of the draft EN 1990 gives informative guidance for enhancing the robustness of buildings and bridges. It provides strategies based on limiting the extent of damage, while the explicit design of structures for identified accidental action is covered within the scope of EN 1991.

The proposed robustness strategies follow the typical methods discussed above, with the addition of a “Segmentation Strategy”. To this end, Table E.1 in Annex E gives recommendations for indicative design methods for enhancing robustness for consequence classes CC1, CC2 and CC3. It is also worth noting that the new EN 1990 adds two more consequence classes, CC0 and CC4. CC4 is considered to have extreme risk of loss of human life or personal injury and a considerable economic, social or environmental risk. The provisions in the Eurocodes do not entirely cover design rules needed for structures classified as CC4. For these structures, additional provisions to those given in the Eurocodes may be needed. On the other side, CC0 has the lowest risk, where either the Eurocodes or alternative provisions may be used and where elements other than structural may be classified as CC0. Therefore, the provisions in Eurocodes mainly cover design rules for structures classified as CC1 to CC3.



(*) Structural design against identified accidental actions can incorporate specifically designed members, which fail partially or fully, provided their failure does not lead to further structural collapse as agreed with authorities.

Figure 2. Design strategies for identified accidental actions and for general enhanced robustness according to (prEN 1990, 2019)

In addition to the proposed revisions in EN 1990 (prEN 1990, 2019), there are developments within EN 1993 and EN 1998 which may be of direct and indirect relevance to the satisfaction of the robustness requirements. These include guidance on the rotation capacity assessments in beams and joints in EN 1993, as well as the provision of load-deformation relationships for steel and composite steel-concrete components for nonlinear static (pushover) analysis in EN 1998. These provisions are referred to where necessary in other parts of this document, and are described and critically assessed in more detail in the background document (Demonceau et al., 2021).

1.5 Concluding remarks

The present chapter has highlighted the requirements and available strategies for robustness design as currently stipulated and proposed in the Eurocodes. Although design for robustness is normatively approached through the general principles available in EN 1990 and EN 1991-1-7, no consistent set of rules is available. Key parameters for the performance of design for robustness, such as those required from the system and available from local ductility, require further treatments, guidance, and clarifications in normative design.

More generally, despite the presence of a substantial body of research dealing with robustness in various structural forms, at both the overall and local level, there is a need to transfer this knowledge into simplified methods and tools to the engineering practice. This document therefore aims at distilling information available from recent research studies on steel and composite framed structures in the form of design provisions ranging from detailed to simplified, for the benefit of different levels of practical design, which are supported and illustrated by a number of realistic design case studies.

2 Design for robustness

2.1 Design strategies

2.1.1 Introduction

In order to comply with the requirements set by the current design standards (Section 1), the design for structural robustness is proposed herein as a step-by-step procedure relying on the consequences class of the building, the nature of accidental action to be considered and the structural layout of the building. This procedure is readily organised in a general flowchart that reflects the design process to be followed as shown in Figure 3.

This flowchart can be seen as the backbone of the present design manual and will be comprehensively presented in this section. More insightful details on the approaches and procedures to be applied throughout the design process will be then addressed in the following chapters.

2.1.2 General design philosophies

EN 1991-1-7 (2006) prescribes the avoidance or limitation of potential damages in accidental scenarios by preventing or reducing the accidental action, by protecting the structure against the effects of the accidental action (through adequate protective systems), or by designing the structure to withstand the accidental action or its effects. These measures lead consequently either to a low probability of hazard occurrence or to a robust structure that resists the accidental action by limiting the propagation of the initial damage.

Following closely the guidelines of EN 1991-1-7, the naturally suggested starting point in the design for robustness is the identification of consequences class of the building under consideration (Box A.1 in Figure 3). The consequences class of the building allows the practitioner to assess the design approach to be adopted in view of achieving an adequate level of robustness. For instance, the design for robustness of a low consequences of failure class (CC1) doesn't imply any specific considerations as long as the design is carried out in full compliance with the rules given in the suite of Eurocodes (EN 1990 to EN 1999). On the other hand, for buildings with higher consequences of failure, such as those identified as CC2 and CC3, the design for robustness implies specific approaches which could range from simple prescriptive rules to advanced risk analyses and complex analytical or numerical methods. More details about the definition of the consequences classes are provided in Section 3.

Once the consequences class is established, the potential threats and the relevant accidental loading scenarios shall be identified by the designer in close collaboration with the client and the relevant authorities. Consequently, the identification of potential threats enables the practitioner to plead either for an explicit design for a specific identifiable accidental action (Boxes B in Figure 3) and/or for a design strategy that limits the extent of initial damage arose as a consequence of any unidentifiable accidental event (Boxes C in Figure 3). For buildings with high consequences of failure (CC3), a systematic risk assessment is generally required to identify the accidental scenarios that are most likely to occur during the life span of the structure (see Chapter 6).

2.1.3 Design for well-identified accidental actions

Generally, the design for a well-identified accidental event implies the use of preventive and protective measures that would mitigate the risk of hazard occurrence or would reduce the latter's destructive effects (Box B.2 in Figure 3). Such measures can range from conceptual solutions (selecting structural forms with low hazard sensitivity) to measures for the reduction of the effects of an accidental action (e.g., safety barriers or protective bollards).

Where the measures taken to prevent the exceptional events lead to a complete avoidance of the full range of possible threats, it is reasonable to consider that the design complies fully with the robustness requirements. Conversely, as long as these protective measures only reduce the magnitude (or probability of occurrence) of the accidental action, or simply cannot be implemented, local damages are imminent and an assessment of possible local damages through an explicit design is required (Box B.3 to Box B.6 in Figure 3). If the predicted local damages are unacceptable and could trigger a disproportionate collapse of the structure, a redesign of the structure has to be carried out so that local damages are counteracted (Box B.2 in Figure 3). Where such damages are acceptable, their extent should be prevented through appropriate design strategies as proposed for unidentifiable accidental actions (see Section 2.1.4).

Generally, for the explicit design under identified accidental actions, specific design strategies relying on analytical and/or numerical methods are used. The level of sophistication of the methods is strongly linked to the consequences class of the structure under consideration. The strategies and the methods currently available are presented in detail in Chapter 4. Within this chapter, four specific accidental actions will be contemplated: impact (Section 4.2), explosion (Section 4.3), fire as exceptional event (Section 4.4) and earthquake as exceptional event (Section 4.5).

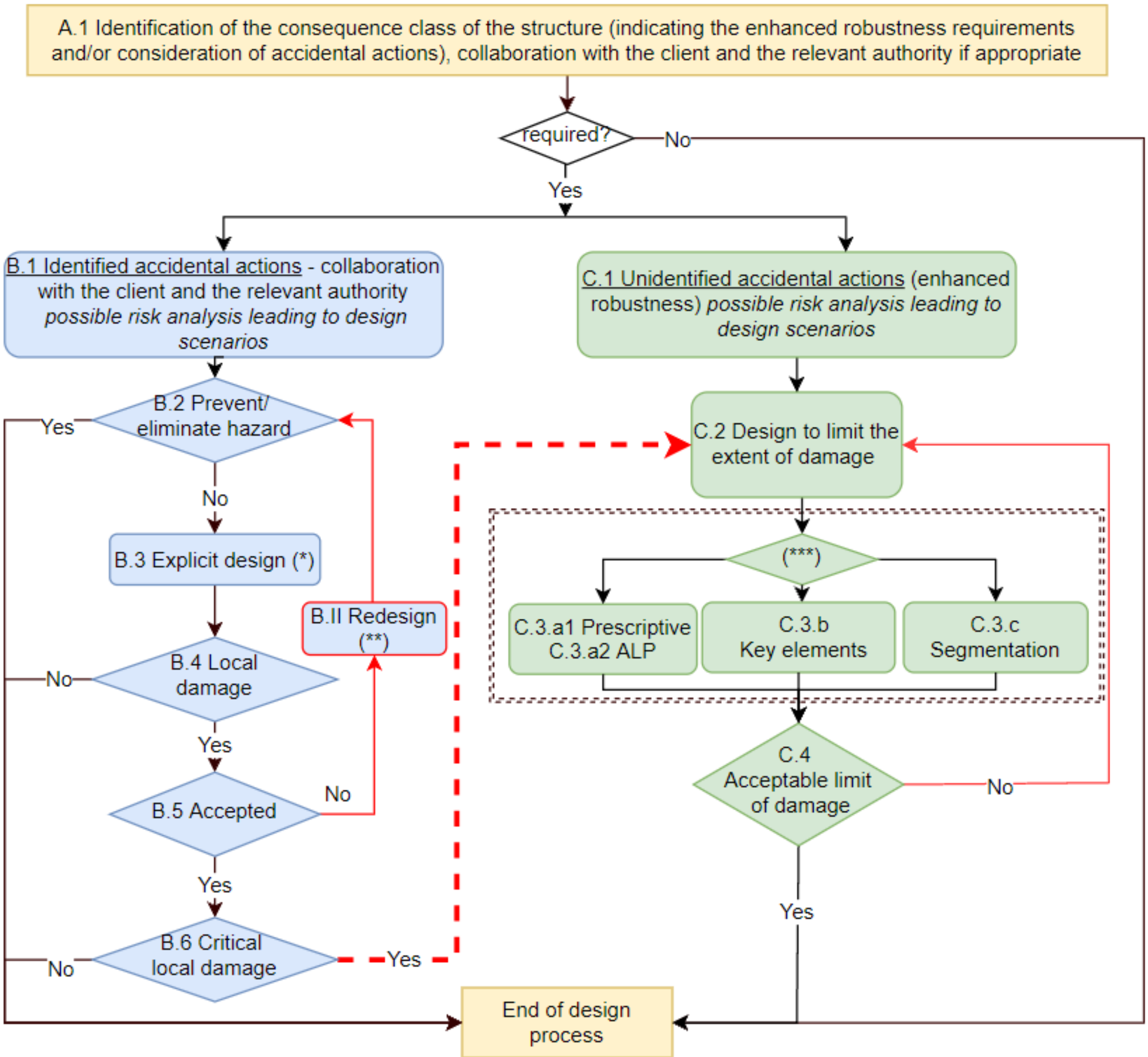
2.1.4 Design for unidentifiable accidental actions

Unidentified threats refer to accidental actions not specifically considered by standards or indicated by the client or other stakeholders or to any other actions resulting from unspecifiable causes. Due to uncertainties regarding the nature, the magnitude, and the application point (region) of an unidentifiable accidental action, the required structural performance is usually impossible to be estimated. In this case, the design for robustness requires pragmatic solutions covering a wide range of potential accidental scenarios. Currently, the design strategies deemed to achieve an adequate level of structural robustness mainly seek to limit the extent of a localised damage (Box C.2 in Figure 3), whatever is the initiating cause. These design strategies are addressed in Chapter 5.

For buildings in the lower consequences classes (CC2a – see Chapter 3), EN 1991-1-7 suggests providing the structure with an efficient horizontal tying system using a prescriptive method named the "tying method" (Box C.3.a2 1n Figure 3). This method allows ensuring a minimum level of continuity between the different structural members by means of horizontal ties and thus the development of catenary actions in the damaged structure in view of activating alternative load paths. Nevertheless, due to the impossibility to estimate the level of robustness achieved through the tying method, the efficiency of the latter remains questionable, and it is seen rather as a necessary but not sufficient measure. Also, the development of catenary actions requires a sufficient ductility in key structural locations, but this point is not specifically addressed in the code which confirms the previous statement. Within Section 5.3.1, proposals will be made to overcome these identified weaknesses.

For buildings in the upper consequences classes (CC2b – see Chapter 3), different alternatives are proposed. The first one is the use of the tying method as proposed for CC2a but adding the request for an efficient vertical tying system (see Section 5.3.1).

The second one is the consideration of the complete removal of supporting elements (Box C.3.a2 in Figure 3). This situation simulates the case where a supporting element is lost completely further to an accidental event and allows to assess whether the structure is able to activate an alternative load path to survive to the loss of the supporting element. The current normative context defines this approach as the "notional removal of supporting elements" and, as EN 1991-1-7 prescribes, it should be applied for all supporting elements (columns, beams supporting columns, or any nominal section of load-bearing walls) considered to be removed one at a time in each storey of the building. Even though such a method could prove to be tedious and time-consuming as it requires the use of advanced analysis tools, it provides the possibility to verify whether the building remains stable and whether the observed damages remain acceptable. From Section 5.3.2 to Section 5.3.4, analytical and numerical tools presenting different levels of sophistications will be proposed to apply this approach.



* Appropriate design approaches for higher and lower consequences classes can be required

**When redesign/retrofit, more advanced methods may be used where appropriate

***Strategies for designing for robustness are not mutually exclusive and may be used singly or in combination

Figure 3. Flowchart reflecting the design for robustness process

Where the loss of a supporting member generates a disproportionate collapse or the extent of the local damage exceeds a specific agreed or prescribed limit, the removed element should be labelled as a "key member" and the design should turn towards methods of local enhancement of resistance capacity of the element defined as the key element method (Box C.3.b in Figure 3). Moreover, the key member should be designed so that it resists a notional accidental action, and its failure should be prevented by any means. This method is detailed in Section 5.4.

An alternative to these methods is the use of segmentation (Box C.3.c in Figure 3). Segmentation is a design strategy that can offer a possibility to prevent or limit an initial damage by isolating the failing part of a structure from the remaining structure by what can be referred to as segment borders. Segmentation strategies can generally be based on either weak segment borders or strong segment borders. More details are provided in Section 5.5.

For buildings in CC3, the design approaches to be adopted are similar to the ones proposed for CC2b but could require the use of refined methods such as dynamic analyses (Section 5.3.5) and should be accompanied by a risk analysis (Chapter 6) as previously mentioned.

2.2 Importance of structural joints in the design for robustness

2.2.1 Classical design at SLS and ULS

Structural joints are key elements which highly influence the global response of a steel building. As stated in EN 1993-1-8, joints may be classified in terms of rotational stiffness, resistance, and ductility.

Three levels of rotational stiffness are considered: nominally pinned, semi-rigid and fully rigid. Stiffness classification boundaries are provided in EN 1993-1-8 but for their application to pinned joints, reference is made to (Jaspart et al., 2009). In reality, deformations occur also under axial or shear forces, but these ones remain quite limited, and they are usually assumed not to significantly influence the response of the structure.

In terms of bending resistance, EN 1993-1-8 and (Jaspart et al., 2009) refer to three classes, namely nominally hinged, partial-strength, and full-strength joints, for which classification criteria are also presented. The concept of partial/full-strength joints may be easily extended to any other loading situation (axial force, combination of moment and axial forces…).

As far as ductility is concerned, three categories exist also, but they are unfortunately not explicitly identified in EN 1993-1-8: brittle joints, ductile joints for plastic verification and ductile joints for plastic analysis. Similar to member cross-sections, one could speak about classes of joints.

The use of rigid full-strength joints does not usually represent the most economical option, because of their high fabrication costs, but this allows to neglect the effect of the joints on the distribution of internal forces and on the design resistance of the system, yielding being only likely to develop in the member cross-section, at least if an elastic analysis is carried out together with an elastic or plastic verification of the cross-section resistance. As soon as a plastic structural analysis is carried out, thus requiring plastic rotation capacity for the development of the plastic mechanism, the risk of developing a plastic hinge in the joint adjacent to the cross-section, due to material overstrength in the member should be avoided, especially if the ductile response of the full-strength joint has not been checked. In EN 1993-1-8, the consideration of an initial “over-resistance” of the joints is then required, as compared to the nominal resistance of the cross-section. Here one could speak about “over-strength joints”.

The component model available in (EN 1993-1-8, 2005) constitutes the main analytical method for the calculation of the mechanical properties, (i) stiffness, (ii) resistance and (iii) rotation capacity of the joints. It finds application both for the elastic and plastic design of any steel or steel-concrete composite (EN 1994-1-1, 2004) joint configuration. Details for the implementation procedure and supplementary information to (EN 1993-1-8, 2005) and (EN 1994-1-1, 2004) are available in (Jaspart and Weynand, 2016a) and (Demonceau et al., 2021). In order to extend its application field, (Demonceau, 2008) characterised a not yet available component for steel-concrete composite joints, the “composite slab in compression”, and made proposals for the effective slab area and the component’s contribution to steel-concrete composite joints under sagging moments, see Chapter VIII.4.2 of (Demonceau et al., 2021). Another interesting reference is the following one in which a review of the design rules for components available in the codes and in the technical literature is presented (Jaspart et al., 2005). Finally, for components met in tubular joints, reference is made to (Weynand et al., 2015).

The calculation of the stiffness and resistance design properties of joints is basically possible, whatever is the loading (moment M only, axial force N only and combination of moment M and axial force N, in addition to shear forces), through the use of the component method approach.

In the above-mentioned normative documents, however, precise application rules are not provided for joints subjected to bending moments and axial forces, except for column bases. When a joint is also subjected to axial force N_{Ed}, a rough approach is just proposed in which, first, the influence of the M-N interaction is disregarded as long as N_{Ed} is smaller than 5% of the axial design plastic resistance of the connected beam cross-section ($N_{pl,Rd}$).

In (Demonceau et al., 2019), it has been shown that the Eurocode M-N interaction predicts sometimes rather precisely, but often very safely the joint resistance, while the 5% rule leads generally to a significant overestimation of the joint resistance. Besides that, Eurocode 3 Part 1-8 is not defining the way on how to evaluate the axial joint resistance N_{Rd}. In the same publication, an improved design analytical assembly procedure is also presented for steel and steel-concrete composite joints. It has been validated through comparisons to results obtained from experimental tests performed on composite beam-to-column joints in various loading situations, including fire and progressive collapse. This advanced procedure, fully compatible with the design principles followed by the Eurocodes, is described in Annex A.1.

2.2.2 Design of joints under exceptional events

Under exceptional events, classical SLS/ULS design criteria in terms of material yielding and deformation may widely be exceeded. As the final objective is to limit the local damages of the structure or the extension of these local damages to the rest of the structure, advantages from these large deformations and from the ultimate resistance of the material in the robustness assessment can be envisaged. In other words, the aim is to demonstrate that the structure can pass from an initial stable undamaged configuration, before the event, to another stable damaged configuration possibly at the cost of extremely large deformations and use of ultimate material resistance. For joints, very large extensional or rotational deformations may be involved, with a level of loading nearly equal to the joint ultimate resistance. For joints not able to exhibit such large deformations, brittle failure may prematurely occur, which adversely affects the possibility to mitigate the risk of progressive collapse. As a conclusion, ductility and large deformation capacity are seen as important properties to be provided to the structural joints.

Moreover, exceptional events often induce internal forces in the joints which significantly differ from those considered as SLS/ULS. These forces vary according to the nature of the event. In addition, the possible loss of an element further to the event may drastically modify the distribution of internal forces in the undamaged part of the structure. As a conclusion, ideally, brittle failure modes should be avoided all along the complex and unforeseen loading sequence of the joint during the event.

Regardless the nature of the event or of the adopted design strategy, the preliminary design of all structural joints for ductility in ULS conditions appears as a prerequisite, even if this is not strictly requested. It simply starts from the principle that a joint, which is even not ductile under ULS will probably not “suddenly” exhibit tremendous deformation or rotation capacities under exceptional events.

The characterisation of the joint properties in such extreme situations is not covered by EN 1993-1-8 but it has been shown in various research projects (Kuhlmann et al., 2008; Demonceau et al., 2013; Ulrike Kuhlmann et al., 2017) that the use of the component approach may be extended to address extreme loading situations in joints, and it will therefore be again used as a reference for joint calculations in the present design manual.

In conclusion, in the present design manual, it is recommended:

- First, to design systematically ductile joints under SLS/ULS conditions. To achieve it, general guidelines are provided further in this section. The conditions are expressed in terms of minimum ductility requirements which should be all the time respected by the joints independently from the global structural analysis and design process implemented to check ULS.
- To respect specific complementary design criteria under exceptional conditions. These ones differ according to the design strategies presented in Section 2.1. They will be detailed accordingly in Sections 4 and 5. In some cases, ductility requirements will have to be fulfilled by some joints for loading situations, which differ from those met under ULS conditions (for instance, in the case of a loss of a structural element further to the exceptional event).

2.2.3 Minimum ductility requirements for structural joints

2.2.3.1 General recommendations for all steel and composite joints at ULS

Under ULS conditions, various levels of ductility are required according to the specific situation encountered in design. In each of the above-listed cases, the minimum ductility requirements at ULS are specified.

<u>Use of pinned joints</u>

Ductility requirements for such joints are provided in (Jaspart et al., 2009), as well as procedures for the evaluation of the design shear resistance (in the form of design sheets allowing an easy application in practice). In this publication, requirements are expressed in terms of full-strength welds and minimum values for the ratio *d/t* between the diameter of the bolts and the thickness of the connected plates (header plate, for instance).

Concerning the welds, the use of full penetration welds or full-strength welds is recommended. While the use of full penetration welds may induce extra costs, full-strength welds can be reasonably achieved respecting the design criteria provided in Table 1.

Table 1. Recommended weld thicknesses “a” to obtain full-strength double fillet welds for plate thickness “t” smaller than 40mm (Jaspart et al., 2009)

Steel grade	**S235**	**S275**	**S355**	**S420M**	**S420N**	**S460M**	**S460N**
f_y (N/mm²)	235	275	355	420	420	460	460
f_t (N/mm²)	360	430	510	520	550	550	580
β_w	0.80	0.85	0.90	1.00	1.00	1.00	1.00
$f_{w,u,end}$ (N/mm²)	255	286	321	294	311	311	328
Full strength double fillet welds	a ≥ 0.46t	a ≥ 0.48t	a ≥ 0.55t	a ≥ 0.71t	a ≥ 0.68t	a ≥ 0.74t	a ≥ 0.70t

To avoid the brittle failure of the bolts and to guarantee a sufficient ductility to the bolted joint, EN 1993 1-8 provides a criterion in Section 6.4.2. This criterion links the thickness "*t*" of the component in bending (thickness of the column flange, of the endplate or of the flange of the cleat) to the diameter of the bolt "*d*":

$$t \leq 0{,}36d\sqrt{f_{ub}/f_y} \tag{1}$$

where f_{ub} is the ultimate strength of the bolt material and f_y the elastic strength of the material of the component in bending. This condition should at least by satisfied by one of the two connected plates.

Moreover, to allow sufficient rotation capacity without any significant development of bending moment in the connection, detailing requirements are also required. Some examples are provided in Annex A.2.

<u>Use of partial-strength joints</u>

If an elastic structural analysis is carried out at ULS and is associated to an elastic verification of the joint resistance, no ductility is to be ensured as no yielding is assumed to develop in the joints.

If an elastic structural analysis is carried out at ULS and is followed by a plastic verification of the joint resistance, minimum requirements must be checked, as for Class 2 member cross-sections, so as to allow a full plastic redistribution of internal forces in the joints. EN 1993-1-8, in its Section 6.2.7.2(9), specifies the rules to be respected to avoid the premature failure of the bolts in tension.

Finally, if a plastic structural analysis involving plastic hinges in the joints is performed, plastic hinges are assumed to form in the joints and rotate. Therefore, some failure modes, such as bolt and weld fracture, have to be avoided. Provisions expressed in Table 1 and Equation (2) must be followed to prevent their appearance. These provisions have not to be applied if a ductile failure mode in another weaker component of failure prevails and limits the plastic resistance of the joints. The yielding of the "column web in transverse tension" is one of these ductile components, as well as the development of bearing deformation prior to bolt fracture in bolted sub-assemblies subjected to shear (for instance, in joints with bolted flange cover plates).

Other failure modes to be avoided are the "column web in transverse compression" and the "beam flange and web in compression" which may involve local instability phenomena.

According to the global design approach followed at ULS (elastic/plastic analysis; elastic/plastic verification), the requirements in terms of ductility may vary, while under exceptional events it has been stated in Section 2.2.2 that "the preliminary design of all structural joints for ductility in ULS conditions appears as a prerequisite". To achieve it, it is recommended, for all the structural joints, to adopt the ductility requirements associated to a "plastic structural analysis involving plastic hinges", i.e., a level of ductility which allows a significant plastic rotation capacity. This requirement often allows to avoid later reinforcement of the joints when checking the robustness of the structures.

The same approach applies to steel-concrete composite joints. For the latter it is recommended, in addition, to satisfy the ductility requirements in the steel part of the connection (i.e., without the slab) so as to ensure a proper "residual" response of the joints after the fracture of the rebars, at high rotation level.

The ductility of the slab in tension depends on the diameter of the rebars, the reinforcement ratio and the ductility class of the rebars (min class B to be used).

In this regard, general requirements given in EN 1994-1-1 (e.g., the minimum reinforcement rate) should be followed.

In (Duarte da Costa, 2018), the ductility of composite joints subjected to hogging moments is investigated. In particular, two minimum ductility conditions that guarantee a sufficient ductility to perform a plastic analysis are provided:

- Effective reinforcement ratio (i.e., ratio between the area of the reinforcement and the area of the concrete $A_{c,eff}$ as defined in Section 7.3.2(3) of (EN 1992-1-1, 2005)): 2,0% ≤ ρ_{eff} ≤ 3,5%;
- Diameter of longitudinal reinforcement ϕ ≥ 12mm.

Moreover, in (Schäfer, 2005), the placement of the first shear stud at a certain distance a_{KB} from the column is recommended to allow for the formation of a tension band in the concrete slab, see Figure 4, and to improve the ductility of the joint.

Under sagging moment, the slab is in compression; it can be assumed that its ductility is sufficient to form a ductile hinge in the composite joint.

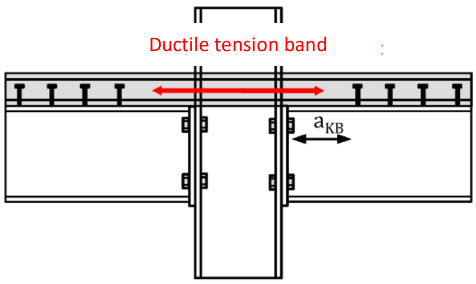


Figure 4. Ductile tension band in the concrete slab

<u>Use of full-strength joints</u>

In the case of an elastic structural analysis at ULS with an elastic or plastic verification of the member cross-sections, no ductility requirement is to be ensured as yielding develops in the connected members.

In the case of plastic structural analysis involving plastic hinges in the members, no ductility or rotational capacity is normally expected at ULS from the joints as they should not undergo any significant yielding, being stronger than the members. But due to overstrength effects, the actual member plastic resistance could increase in such a way that the plastic hinges develop in the joints. Consequently, an unexpected premature brittle failure of the joint could then occur.

The overstrength of the material can be estimated using the recommendation from Eurocode 8 (EN 1998-1, 2004), Section 6.5.5: $f_{ov} = 1{,}1 \times \gamma_{ov} \times f_y$ where f_{ov} is the material strength accounting for overstrength, 1,1 is a coefficient to account for strain hardening, γ_{ov} is the overstrength factor (recommended value = 1,25, but can be based more accurately on the values given in Table 2 proposed in the prEN 1998 forthcoming new version of Eurocode 8 (prEN 1998-1-2, 2019)) and f_y is the nominal yield strength of the material. This means that the resistance of the “over-strength” joint (see Section 2.2.1) should be at least 1,38 times higher than the resistance of the weakest connected members to account for the overstrength effects. Otherwise, the joints might become the weakest structural elements and should be able to exhibit a minimum level of deformation capacity as requested for partial-strength joints.

Table 2. Randomness material factor γ_{ov} (prEN 1998-1-2, 2019)

Steel grade	γ_{ov}
S235	1,45
S275	1,35
S355	1,25
S460	1,2

Preliminary request for ductility

As a conclusion, as a "good measure" to help the structure to accommodate exceptional events, it is recommended to always design all structural joints at ULS in such a way that plastic hinges may form and rotate, i.e., as it would be in the case of a structural plastic analysis performed at ULS.

The only exception to this principle relates to "over-strength joints". It has to be however highlighted that the over-strength character of the joints should be ensured not only under bending moments, but for all the loading situations encountered in the joints during the event, for instance involving tying forces.

2.2.3.2 Specific recommendations for partial-strength steel and composite bolted joints with endplates at ULS

A user-friendly alternative to the explicit use of the component model of EN 1993-1-8, when it comes to the evaluation of the bending resistance of moment resisting bolted endplate joints, see Figure 5, is provided in (Rölle, 2013). The method assumes the product of bolts' tensile strength and relevant lever arm to be the factor that predominantly defines the joint moment capacity, while other parameters with an influence on the moment capacity are considered indirectly through the application of a correction factor, see Annex A.3.1. With the help of certain constructive criteria, the design procedure aims to reach an optimal Mode 2 failure of the joint combining sufficient ductility and a satisfying not too low resistance, as this would be the case for Mode 1 failure. The validity of the latter approach has been demonstrated through experimental tests performed on specimens made of steel grades up to S355.

Additionally to the general recommendations for ductility addressed for all joints in Section 2.2.3.1, specific recommendations have been expressed, and are given in Table 3, for highly ductile partial-strength joints, in which plastic hinges should form and rotate, see also (Vogel et al., 2014). In particular, a certain distance of the bolt from the profile (see m and m_2 or m_x in Figure 5) has to be ensured as this has been proved to have a significant influence on the ductility (Rölle, 2013). If all six criteria of Table 3 are fulfilled, total joint rotations of 80 to 180 mrad may be achieved by mainly activating the "end-plate in bending" component at failure.

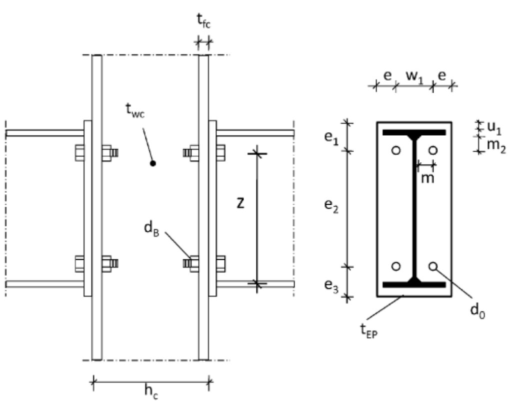


Figure 5. Relevant geometric parameters of an end-plate joint

Table 3. Constructive criteria for the design of highly ductile end-plate beam-to-column steel joints

Name of parameter	Parameter symbol	Criterion
Ratio of end-plate thickness to bolt diameter	t_{EP}/d_B	$< 0,65$
Steel grade of construction steel	f_y	$\leq S355$
Steel grade of bolt	f_{uB}	≥ 8.8
Horizontal distance of bolt (mm)	m	$\geq 3.0d_B$
Vertical distance of bolt (mm)	$m_2(m_x)$	$\geq 2.5d_B$
Beam height (mm)	h_b	≤ 500
In this particular table, the criteria limits are linked to the specified range of steel grades for which tests were available, but it should not be concluded that the higher steel grades do not allow for high ductility.		

2.2.3.3 *Specific recommendations for partial-strength and full-strength steel joints with endplates in structures subjected to seismic actions at ULS*

In steel moment resisting frames, the beam extremities are generally used as dissipative zones, and the beam-to-column joints are designed to resist to the internal forces corresponding to the development of plastic hinges at the beam extremities. However, possible overstrength and strain hardening effects occurring in the dissipative zones have to be taken into account when designing the non-dissipative zones.

The use of partial-strength joints as dissipative zones is allowed but, in this case, the ductility and the dissipation capacity of the joints should be demonstrated by means of experimental tests according to the current design standard.

For this reason, a European RFCS project named EQUALJOINTS involving academic and industrial partners has been launched with the objective of studying and prequalifying three types of bolted joints that are commonly used in European practice.

As a result, a design guide, a software, and an app for mobile devices were developed and translated in 12 European languages bringing directly the outcomes of the EQUALJOINTS research project to the engineering practice. These design tools are made available for free on the web site of the European Convention of Constructional Steelwork (ECCS - https://www.steelconstruct.com/eu-projects/equaljoints/). These recommendations will be implemented in the new forthcoming version of Eurocode 8.

2.2.3.4 *Evaluation of the plastic rotation capacity of joints at ULS*

General and specific approaches for an explicit determination of the plastic rotation capacity of steel and composite joints are presented in Annex A.4.

2.2.3.5 *Summary of the conclusions in view of the design of joint under exceptional loadings*

According to the structure and to the design procedure followed to mitigate the risk of progressive collapse, the requests in terms of resistance and ductility to be exhibited by the joints may significantly differ. These requests will be specified for each design strategy in Sections 4 and 5.

It is required to design all structural joints at ULS so that hinges may form (this is implicit for pinned joints) and rotate.

Requirements to achieve this goal are provided in this section for pinned, partial-strength and full-strength joints. They are expressed:

- in general terms for all steel and composite joints (Section 2.2.3.1);
- under the form of simplified design approaches for partial-strength joints with endplate connections (Section 2.2.3.2);
- under the form of a prequalification procedure for partial-strength and full-strength steel joints with endplate connections in a structure subjected to earthquakes (Section 2.2.3.3).

Reference to evaluation procedure for the available plastic rotational capacity of steel and composite joints is finally made in Section 2.2.3.4

3 Consequence classes

Building structures are classified into consequences classes based on the consequences of structural failure in terms of loss of human lives or personal injury and of economic, social, or environmental losses. Such classification is considered to be a simplification of a complex risk-based system related to building type, height, occupancy, societal perception, type of load, structural type, nature of materials, amongst others. In EN 1990 and EN 1991-1-7, Cl 3.4, three consequence classes are identified. However in Annex A of EN 1991-1-7, Table A.1, Consequence Class 2 is subdivided into CC2a (medium consequences-lower risk group) and CC2b (medium consequences-upper risk group), with the other classes being CC1 (low consequences of failure) and CC3 (high consequences), as summarised in Table 4. It is worth mentioning that Annex A is considered to be informative rather than normative, where the guidance provided does not need to be followed. However, it is the decision of individual countries whether to recommend the application of Annex A or not. More practical guidance related to building classification for robustness can also be found elsewhere (Way, 2011). On the other hand, the current draft revision of EN 1990 (prEN 1990, 2019) adds two more consequence classes, CC0 and CC4. CC4 is considered to have extreme risk of loss of human life or personal injury and a considerable economic, social or environmental risk. The provisions in the Eurocodes do not entirely cover design rules needed for structures classified as CC4. For these structures, additional provisions to those given in the Eurocodes may be needed. On the other side, CC0 has the lowest risk, where either the Eurocodes or alternative provisions may be used and where elements other than structural may be classified as CC0. Therefore, the provisions in Eurocodes mainly cover design rules for structures classified as CC1 to CC3. Additionally, the draft revision of EN 1990 allows consequence classes CC1 to CC3 to be divided into upper and lower sub-classes in other Eurocodes.

There are some cases when practicing engineers may encounter difficulties if building structures may not directly follow the descriptions provided in Table 4. In such cases, engineering judgement is required, and it is the responsibility of the engineer to ensure that the safety of the structure is not compromised. Some of the common cases are listed in the following (see (Way, 2011) for more details):

- Including mezzanine floors in counting the number of storeys for building classification will depend on the size and use of such floor. For an approximate guide, SCI P391 (Way, 2011) recommends the mezzanine floor to be counted if it is greater than 20% of the building footprint, which can be increased if the floor is not accessed on a daily basis.
- Habitable areas of roof floors should be counted in the number of storeys regardless of the roof’s slope.
- Buildings with different numbers of storeys that fall into different consequence classes should be classified relating to the most onerous class.
- Mixed use buildings that fall into different consequence classes should be classified depending on the most onerous class.
- Basement storeys are defined such that the external ground level should be at least 1.2 m above the top surface of the basement floor for a minimum of 50% of the building’s plan. In determining the number of storeys, basement storeys may be excluded, provided such basement storeys fulfil the requirements of "Consequence Class 2b Upper Risk Group". In case of Consequence Class 3, basement floors shall follow the requirements of such class.
- The ground floor storey can be excluded from the total number of storeys for building classification if all of its structural elements including the connections are designed as key elements. In the case of using the ground storey as parking, it can be excluded from the storey count if all of the following apply:

    - Parking is exclusively for users of the building.
    - The ground floor storey must not be accessible to or contain a right of way for the general public.
    - All the structural elements of the ground floor storey, and their connections, are designed as key elements.
- For buildings that undergo conversions, alterations or extensions resulting in a change of consequence class, the building should then be classified to the most onerous class.

Table 4. Categorisation of Consequence Classes in current EN 1990 and EN 1991-1-7 - Annex A

Consequence Class (CC)	**Description**	**Type and Occupancy Examples**
1	Low consequence for loss of human life, and economic, social or environmental consequences small or negligible	Single occupancy houses ≤ 4 storeys Agricultural buildings where people do not normally enter (e.g., storage buildings), greenhouses Buildings into which people rarely go, provided at distance 1.5 times height away from others
2a (Lower Risk Group)	Medium consequence for loss of human life, economic, social or environmental consequences considerable	5 storey single occupancy houses Hotels, residential, offices ≤ 4 storeys Industrial ≤ 3 storeys Retailing premises ≤ 3 storeys and < than 1000 m^2 floor area in each storey Single storey educational buildings Buildings ≤ 2 storeys admitting public with floor areas ≤ 2000 m^2 at each storey
2b (Upper Risk Group)		Hotels, residential, offices > 4 storeys but ≤ 15 storeys Educational buildings > single storey but ≤ 15 storeys Retailing premises > 3 storeys but ≤ 15 storeys Hospitals ≤3 storeys Offices greater than 4 storeys but not exceeding 15 storeys. Buildings admitting public with floor areas > 2000 m^2 but ≤ 5000 m^2 at each storey Car parking ≤ 6 storeys
3	High consequence for loss of human life, or economic, social or environmental consequences very great	Buildings defined above as Class 2a and 2b that exceed limits on area and storeys Buildings to which members of the public are admitted in significant numbers (e.g., concert halls, grandstands, …etc.) Stadia accommodating more than 5000 spectators Buildings with hazardous substances/processes
Note: Table is not exhaustive and can be adjusted.		

4 Identified threats

4.1 Introduction

The design for robustness of building structures can be done considering either the direct effects of an extreme action, or a specific extent of damage from an unknown / unforeseen event. Obviously, the methods from the first category require the identification of the threat and the definition of the action. Typical examples are fire, explosion, blast, or impact. For some actions, the level of threat can be reduced or even eliminated with non-structural or other measures, e.g., active fire protection - sprinklers, vent openings for gas explosions, protecting the structure against the impact using traffic bollards, or increased stand-off for blast. Also, in some cases, localised damage may be allowed to develop, but not to an extent disproportionate to the original cause. Some accidental actions are treated in detail in the Eurocodes:

- Earthquake: the design of structures subjected to earthquake is covered by a specific Eurocode, EN 1998;
- Fire: the design of structures subjected to fire is covered in Part 1-2 of the different "material related" Eurocodes.

However, under some circumstances, the actions can exceed the conditions considered in the codes, for example under cascading loading scenarios, e.g., earthquake after earthquake, fire after earthquake, or fire after blast.

The effects of the identified actions on a structure should be done considering appropriate methods of analysis, which depends on the safety category, or consequences class (EN 1991-1-7):

- Consequences class 1: no specific consideration of accidental actions;
- Consequences class 2: depending on the specific circumstances of the structure in question: a simplified analysis by static equivalent load models for identified accidental loads and/or by applying prescriptive design/detailing rules;
- Consequences class 3: extensive study of accident scenarios using dynamic analyses and non-linear analyses if appropriate.

The next section covers the design for the following identified accidental actions:

- Impact loads due to road traffic (Section 4.2);
- External explosion (Section 4.3.2);
- Internal explosion due to natural gas (Section 4.3.3);
- Fire (Section 4.4);
- Earthquake (Section 4.5).

4.2 Impact

4.2.1 Prevent/eliminate hazard

The hazard coming from impact is typically associated with an incident involving vehicles. The consequences of the vehicle impact depend strongly on the weight, and speed, and direction (in relation with the building) of the vehicle. The preventive measures are part of the building safety focused on slowing the vehicle down and reducing access to the building. This can be achieved by appropriate design of access roads, which do not allow for large cars to directly approach the building, and which limit the vehicle speed. There are also various equipment starting from simple speed bumpers on the road, through automatic blockers and security barriers as illustrated in Figure 6 to Figure 9.

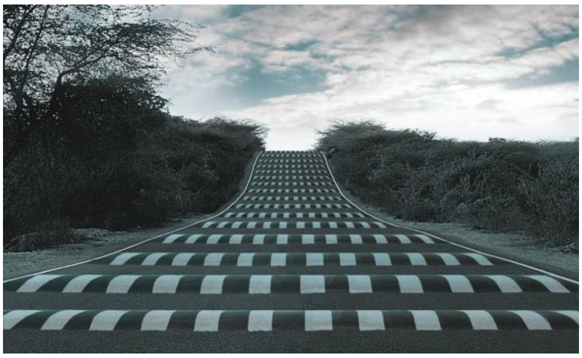


Figure 6. Speed bumpers on the hill road

Figure 7. Speed bumpers in the car park

Figure 8. Automatic road blocker and barrier

Figure 9. Hydraulic road security blockers

4.2.2 Explicit design

Impact actions are covered in Chapter 4 of EN 1991-1-7 (EN 1991-1-7, 2006). This code covers several different situations in which an impact action may occur. Independently of the situation, the impact always involves an interaction between a colliding object (source of impact) and the impacted object (e.g., a column of a building).

Depending on the consequences class of the structure, the following simplifications are allowed (see Figure 10):

- For structures in low and/or medium consequences class (up to CC2 - see Section 3), a static analysis such as the equivalent static approach of EN 1991-1-7 is sufficient as described in Section 4.2.2.1.
- For structures in high consequence class (CC3 - see Section 3), a dynamic analysis is required. This analysis can be a simplified (EN 1991-1-7) or full dynamic analysis – see Sections 4.2.2.2 and 4.2.2.3 respectively.

When using the equivalent static approach of EN 1991-1-7, the impacted object is always considered rigid, i.e., the colliding object absorbs all the impact energy (*hard impact*), what is conservative. However, when dynamic analyses are used, *hard impact* or *soft impact* are both allowed. In the *soft impact*, the capacity of the impact object in dissipating the impact load is taken into account.

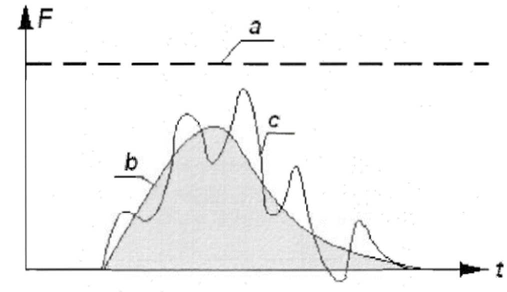


Figure 10. Representation of an impact action (EN 1991-1-7, 2006)

4.2.2.1 Equivalent static approach

In this approach, the impact load is replaced by an equivalent static force *F* accounting for the effects of the load on the structure. For all the types of impact dealt with in Section 4 of EN 1991-1-7, values of static equivalent forces for different types of vehicles (cars, lorries, trains, ships…) are reported with explanations on how to apply them to the structures.

The most common situation in buildings is the impact of a vehicle with one of the supporting columns. The application of this approach for this case is shown in Figure 11, the position (height *h* and area *a*) of the force in the column depends on the type of vehicle (car or lorry), while the magnitude of the force *F* is dependent of the type of road where the vehicle is travelling (i.e., the maximum velocity that it can achieve).

The impacted member (and the surrounding structure) should be checked when subjected to the equivalent static force F and to the other permanent and variable loads, considering an accidental load combination. For this member, the ULS should be checked, without any limitations in terms of deformation.

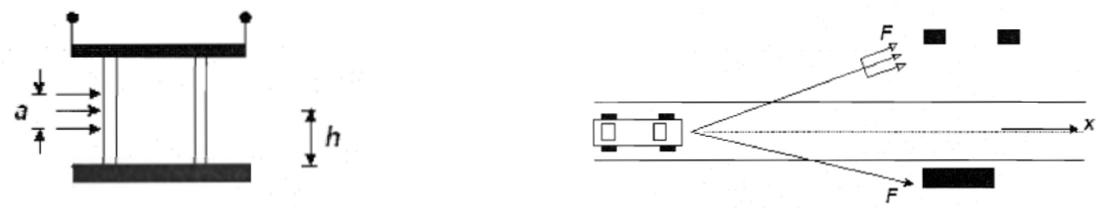


Figure 11. Collision force on supporting substructures near traffic lanes for bridges and supporting structures for buildings (Eurocode 1 2006)

4.2.2.2 Simplified dynamic approach

This approach can be found in Annex C of EN1991-1-7 and it can be generally described by the model provided in Figure 12. The assessment of the impact force *F* depends on the type of impact (soft or hard impact):

- For *hard* or *soft* impact in which the colliding object or impacted object, respectively, deform linearly, Eq. (2) can be used, where *k* is the stiffness of the colliding object (hard impact) or the impacted object (soft impact); v_r is the impact velocity and *m* is the mass of the colliding object.

$$F = v_r \sqrt{k \cdot m} \tag{2}$$

- For *soft impact*, when the impact energy is absorbed through plastic deformations, it is required that the structure ductility is enough to absorb the total kinetic energy ½mv_r^2 of the colliding object. Assuming a rigid plastic response of the structure, this requirement is satisfied if the condition given by Eq. (3) is respected, where F_0 is the plastic strength of the structure and y_0 its deformation capacity.

$$\tfrac{1}{2} \cdot m \cdot v_r^2 \leq F_0 y_0 \tag{3}$$

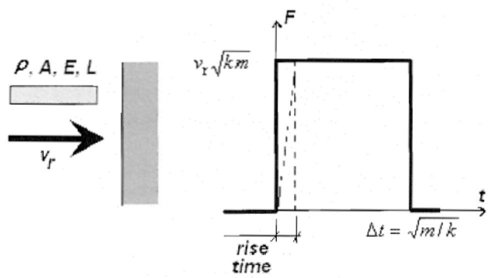


Figure 12. Impact model (EN 1991-1-7, 2006)

For the specific case of impact of a vehicle with a supporting member of a structure, EN 1991-1-7 suggests some values for the parameters that influence the impact force such as the mass, velocity, plastic strength F_0, deceleration of the vehicle, etc., depending on the type of vehicle and the type of road. In the informative annex of EN 1991-1-7, this particular case is explained in more detail.

4.2.2.3 Full dynamic approach

In a full dynamic analysis, the designer can decide between an analysis where the impact is explicitly modelled or an alternative load paths analysis (or column loss analysis) where the action is not explicitly modelled but its consequence, i.e., a column loss, is simulated. In practical terms, this second approach is more appealing because it can provide a good estimation of the structure robustness, without the complexity required to model impact actions.

Several options are considered within the alternate load path analysis, differing from their complexity (linear/ non-linear, static/dynamic, etc…); the design guidelines to apply them are described in Section 5 of this manual, thus, these rules are not described here. However, some parameters are here highlighted as important for a good estimation of the robustness capacity of a structure subjected to an impact action through an alternative load path analysis, such as:

- *Dynamic effects* can be taken into account by evaluating the time removal of the column or bearing element loss. For example, the GSA suggestions (GSA, 2003) can be followed.
- *Effect of the strain rates* imposed by the action on constitutive laws of materials composing the structure can be easily assessed through *DIF* parameter. For impact loads inducing strain rate usually between 10^{-1} to 10, the DIF coefficient to be applied on the elastic strength of the steel material varies from 1,1 to 1,3. For the bolt mechanical properties, a DIF coefficient of 1,1 can be reasonable assumed. There are also many models available in the literature to account more accurately for this, such as the Johnson-Cook model (Johnson and Cook, 1983).

4.3 Explosion

An explosion is an extremely rapid release of energy in the form of pressure wave, heat, sound, and light (Hall, 2017). The output of an explosion may also include the impact with primary fragments and/or secondary fragments. Even though all these can affect buildings and occupants in different ways, this chapter is mainly limited to the response of structures to the pressure load. No guidance is given to account for thermal or flying debris impact, even if the effects can be significant in some cases.

Explosive materials can be solids, gases, vapours, or dust. Depending on the nature of the explosive material and the local conditions, the explosion may develop as a deflagration or expand rapidly and generate shock waves (detonation). More information can be found in (Demonceau et al., 2021).

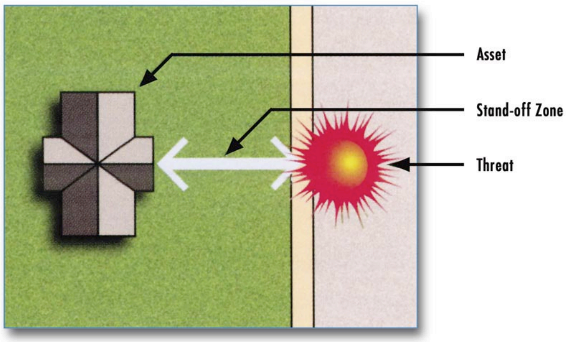
Generally, buildings are not designed for loading conditions generated by explosions, excepting the facilities designed to resists such actions (e.g., blast resistant buildings) or buildings where gas is burned or regulated. Thus, when buildings are subjected to such extreme loads, they may sustain extensive damages (Ellingwood et al., 2007; Somes, 1973; Burnett, 1975a; EN 1991-1-7, 2006). The possibility that primary structural components may fail shall be recognized and measures shall be taken to mitigate this risk, for example by preventing the progressive collapse after a column loss (CSA, 2012). In the following, details about the main characteristics of explosions and possible design approaches are given. Also, measures to reduce or prevent the explosion threat are given.

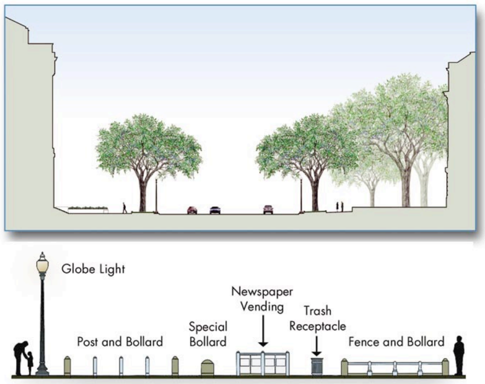
4.3.1 Prevent/eliminate hazard

4.3.1.1 External explosion

There are several available methods to reduce or eliminate the external explosion threats without any intervention to the structural systems. The blast pressure reduces significantly with increase of the distance, therefore maximizing stand-off distance will decrease the effects of a blast (Figure 13a). In case of public spaces, where it is not possible to create / control a certain stand-off distance, bollards, trees, street furniture can be used as obstacles, as illustrated in Figure 13b. For higher risk area, a blast resistant wall can be built, which is a barricade that protects the structure from an explosion. The main aim of the wall is to keep the energy imparted by the explosion from reaching the structure, which is now protected from permanent damage and can continue the operation after the blast.

The selection of the building shape and materials can also mitigate the effect of an explosion. Non-structural elements attached to the building exterior have to be avoided to limit flying debris and improve emergency egress by ensuring that exits remain passable. If used, they should be designed using lightweight materials with connections designed to resist the capacity of the element. Windows are the most vulnerable part of building causing severe injuries. Depending on the risk level, appropriate type of glazing should be used as well as reduced area of windows on the exposed facades. It has been identified that structural shapes and dimensions have considerable influence on the design blast load. A square edge section results in higher peak reflected over pressure when compared with long rectangular edge section subjected to blast loads. In case of a circular shaped structure, the highest peak reflected over pressure is observed at a point on the edge, which is nearest to the explosion. This pressure diminishes in magnitude towards both the sides of the center. Further, in modern buildings, it is observed that a parabolic or cubic shaped facade performs better than an upright faced facade. Thus, by analysing the shape of the building, the design can be adjusted to use the shape that results in minimum design blast load and simultaneously provides usable area.





a) *b)*

Figure 13. Mitigation of explosion effects: a) concept of stand-off distance; b) streetscape security elements (FEMA 426)

4.3.1.2 Internal gas explosion

Lessons learned from previous accidents show that prevention of gas explosions by reducing the probability of the accidental releases and ignition only, is not sufficient. If effectively implemented, a good engineering practice may help reducing the consequences (Bjerketvedt et al., 1997a):

- Take into consideration the gas explosion hazard from the beginning of the project. It is in the early phase of the development project that major decisions such as location of different areas, separation of areas and overall layout (that will influence the vent arrangement and the process itself) are made.
- Buildings subjected to possible internal explosions should have a strong frame structure supporting roof and intermediate floors. The "walls" should be open, if possible. If a solid wall is needed, use low weight wall panels to facilitate early explosion venting.
- Vent areas are important not only in terms of the size, but also in terms of location. Thus, when there is sufficient venting close to the ignition point (see also *a)* for the importance of the conceptual design), the flame speed will be low, and the turbulence generated behind the obstacles will be limited.
- As a general principle, the gas explosion venting should be directed into open areas with a minimum of obstructions.
- Partial obstruction of a vent opening can result in strong pressure increases.

4.3.2 External explosion – Explicit design

4.3.2.1 Definition of the structural blast loads

An explosion scenario is defined first, including the expected charge weight *W*, type of explosion, and distance to the building *R*.

The evolution of the pressure vs time associated to a front wave can be idealised through the curve presented in Figure 14. Unless better information is available, the blast load parameters can be determined using the diagrams presented in Figure 15, which involves the computation of the scaled distance, *Z*, which depends on the explosive mass W (in kg of TNT), and the actual distance from the centre of the spherical explosion R (in m). Except for the pressures and velocity, all the other values in Figure 15 are scaled by a factor $W^{1/3}$ so as to take into account the actual size of the charge.

The idealised pressure-time diagram for the front wall can be constructed using the following relationships:

$$t_c = \frac{4S}{(1+R)C_r} \tag{4}$$

$$t_{of} = \frac{2i_s}{P_{s0}} \tag{5}$$

$$t_{rf} = \frac{2i_r}{P_r} \tag{6}$$

where:

- t_c is the clearing time;
- S is the smallest of the surface's height H or half width $W/2$;
- C_r is the sound velocity in the reflected medium;
- R is the ratio S/G, where G is the largest of the surface's height H or the half width $W/2$;
- t_{of} is the fictitious time ($t_{of} < t_o$, where t_0 is the actual duration of the positive phase) of the incident wave;
- i_s is the impulse value of the positive phase of the blast wave;
- P_{so} is the peak incident pressure;
- t_{rf} is the fictitious duration of the reflected wave;
- i_r is the total reflected impulse;
- P_r is the peak reflected pressure.

The peak dynamic pressure q_o is calculated from Figure 16. This parameter is required to compute the value of $P_{so} + C_D q_o$ (see Figure 14) which is determined by using C_D=1 for the drag coefficient for the front face of the structure.

Note: For the positive phase of the reflected pressure, two curves P_r-t are constructed and compared: one corresponding to infinite surface conditions, and another derived using the assumption that the finite surface geometry influences the value of the reflected pressure. The curve to be used for loading the structure is the one that produces the smallest impulse value (JRC).

The loads computed for front face of the structure are applied in the structural design of the building using the load combination rules given in EN 1990 for the accidental design situations. Depending on the complexity of the building and class of consequences, different types of analysis may be required (e.g., equivalent SDOF, dynamic non-linear analysis) as reported in the next sections.

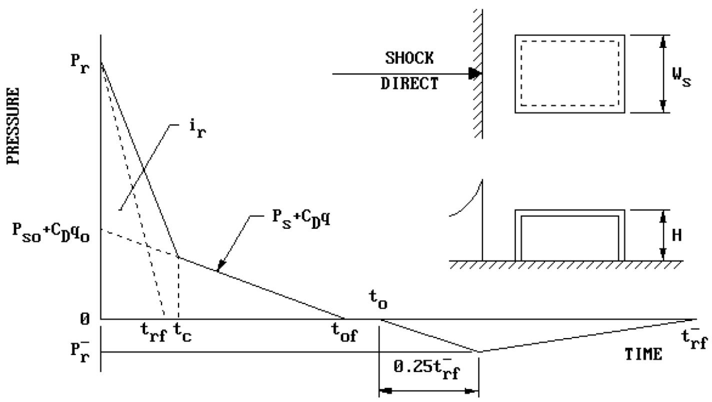


Figure 14. Front wall pressure

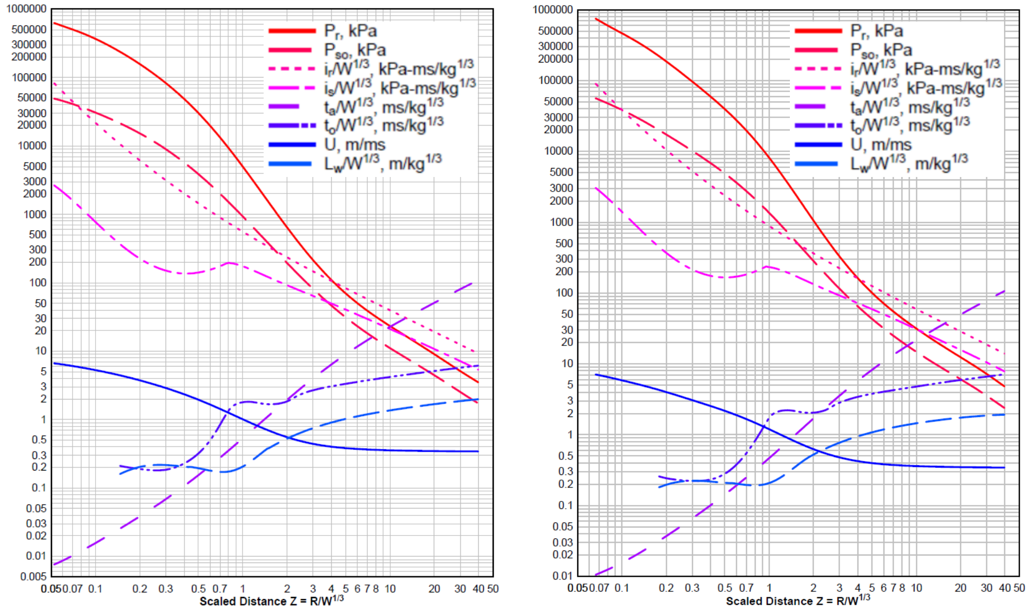


Figure 15. Parameters of positive phase of shock spherical wave of TNT charges from free-air bursts (left) and surface bursts (right) (modified from (DoD, 2008))

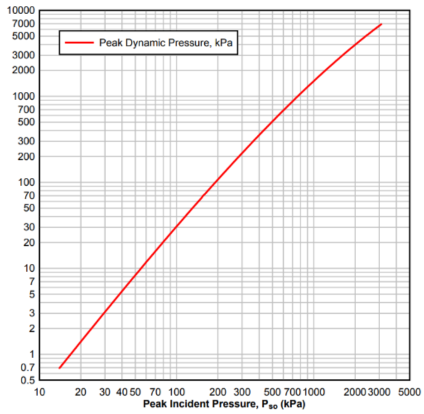


Figure 16. Variation of peak dynamic pressure q_o versus peak incident pressure (modified from (DoD, 2008))

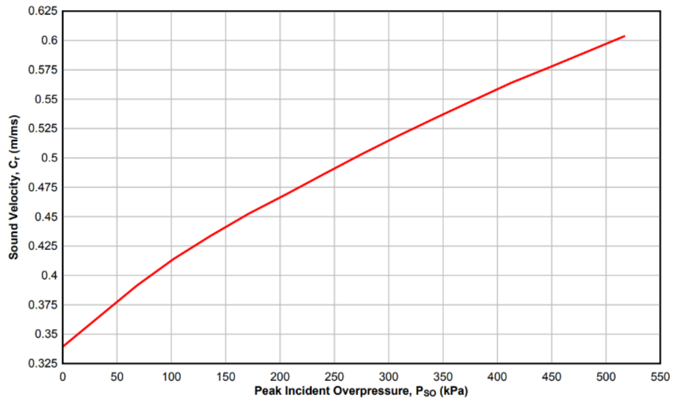


Figure 17. Sound velocity in reflected overpressure region (modified from (DoD, 2008))

4.3.2.2 Equivalent single-degree-of-freedom analysis

External explosions are dynamic events by definition. For simple structures, a rigorous dynamic analysis can be performed to evaluate the response. For practical design purposes however, approximations need to be made to allow the design with reasonable accuracy. This can be done by transforming the structure into an equivalent single degree of freedom SDOF system, where the mass distribution, boundary conditions, resistance function, and load history are idealized.

To define the equivalent SDOF system, it is necessary to evaluate the parameters of that system; namely, the equivalent mass m_E, the equivalent spring constant k_E and the equivalent load F_E. Additionally, the load-time function *F(t)* must be established. Most accidental loads, including explosions, can be defined by one of the following four types of load-time functions with limited duration t_d: suddenly applied constant load, triangular load, symmetrical triangular load, and constant force with finite rise time. Depending on the ratio between positive duration, t_o, and the natural period, T_n, the blast load can be modelled using a triangular load-time function (small t_o, large T_n) or suddenly applied constant load (large t_o, small T_n). But in general, the blast load can be modelled using a triangular load-time function.

The period of the SDOF system can be calculated using the relationship:

$$T_n = 2\pi\sqrt{m_E/k_E} \tag{7}$$

Also, the equivalent characteristics of mass and load can be defined and obtained by means of the transformation factors using the following equations:

$$K_L = \frac{F_E}{F(t)} \tag{8}$$

$$K_m = \frac{m_E}{m} \tag{9}$$

where:

- K_L is the load factor, F_E is the equivalent load, and F is the actual total load on the structure;
- K_M is the mass factor, m_E is the mass of the equivalent system, and m is the total mass of the actual element.

In practice, tabulated values for different structural systems are provided in literature. Examples of such values are provided in Annex A.6. Then, on the basis of these values, it is possible to predict the response of the SDOF system in terms of maximum displacement and so, in terms of ductility demands, using simple approaches or abacus according to the assumed behaviour, i.e., elastic or elasto-plastic (see here below).

Elastic SDOF systems

The maximum response of the SDOF systems with elastic response is defined by the dynamic load factor, DLF, and maximum response time, t_m, where DLF, defined as in Eq. (10), can be determined using Figure 152 from Annex A.6.2.

$$DLF = \frac{y_{max}}{y_{st}} \tag{10}$$

where:

- y_{max} is the maximum dynamic deflection;
- y_{st} is the deflection resulted from the static application of the peak load F_m.

Elasto-plastic SDOF systems

The response of the SDOF system with elasto-plastic response is defined in terms of its ultimate resistance R_m, and maximum deflection y_m. The resistance functions R - y are idealized as bilinear functions characterised by the following parameters: elastic stiffness (k), elastic deflection (y_e), maximum deflection (y_m), and ultimate resistance R_m (see Figure 18.

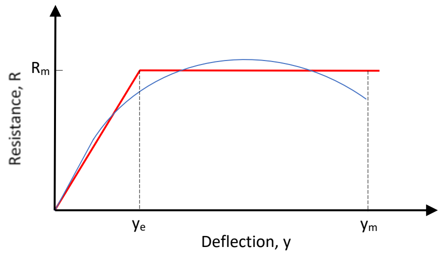


Figure 18. Resistance function vs. deflection for an elasto-plastic SDOF system

The result for the maximum displacement is presented in chart form, as a family of curves R_m/F_m, and gives the required ductility μ, given by the ratio y_m/y_e, as a function of t_d/T_n, according to tabular data given in Annex A.6.

4.3.2.3 Pressure-impulse diagrams

The design approach presented in the previous section (4.3.2.2) considers the effects of the overpressure to describe the element response. However, for very short positive phase durations, t_o (relative to the natural period, T_n), the structural response is sensitive to the associated impulse and not to the maximum pressure. Thus, the response of an element can be fully represented by an iso-response curve called pressure-impulse (P-I) diagram. P-I diagrams may be used to evaluate the performance of a structural system or component, provided that the parameters used in the generation of the selected P-I diagram represent the loading (from explosion), failure mode, and dynamic characteristics of the system under consideration. P-I diagrams may be generated using SDOF or numerical analysis (FEM, AEM), or may be fitted to appropriate experimental data. The evaluation of the performance follows the next steps:

- First the load shape is defined. This should be consistent with the explosion threat.
- SDOF analysis (or other approaches) is used to determine the response of the component in the form of end rotation, θ, and ductility factor, μ, defined as the ratio between the maximum displacement under the considered blast load and the elastic displacement, i.e., the displacement when a plastic hinge forms in the considered system.
- The response computed above is compared with the system limits. Such limits are available for entire buildings, individual structural members, or windows, see Table 5 as an example.
- Based on the damage level determined in previous step, the level of protection (class of consequences) is provided by comparing the results with the information in Figure 19.

Table 5. Example of response limits for hot-rolled structural steel (CSA, 2012)*

Element type		B1		B2		B3		B4	
		μ_{max}	θ_{max}	μ_{max}	θ_{max}	μ_{max}	θ_{max}	μ_{max}	θ_{max}
Flexure	Beam with compact section†	1	-	3	3°	12	10°	25	20°
	Beam with noncompact section†,‡	0.7	-	0.85	3°	1	-	1.2	-
	Plate bent about weak axis	4	1°	8	2°	20	6°	40	12°
Compression	Beam-column with compact section†,§	1	-	3	3°	3	3°	3	3°
	Beam-column with noncompact section†,§	0.7	-	0.85	3°	0.85	3°	0.85	3°
	Column (axial failure)**	0.9	-	1.3	-	2	-	3	-

** Where a dash (-) is shown, the corresponding parameter is not applicable as a response limit.*

† *Limiting width-to-thickness ratios for compact and noncompact sections are defined in CSA 2012.*

‡ *These response limits are applicable for flexural evaluation of existing elements that satisfy the design requirements of Clauses 6 through 8 but do not satisfy the detailing requirements in Clause 9, and shall not be used for design of new elements.*

§ *If a shear plane through the anchor bolts connecting the column base plate to the foundation exists, the response limit for superficial damage shall apply, using the shear capacity of this connection, rather than the element flexural capacity, as the ultimate resistance for analysis.*

** *Ductility ratio is based on axial deformation, rather than flexural deformation.*

Note: *Adapted from PDC-TR 06-08*

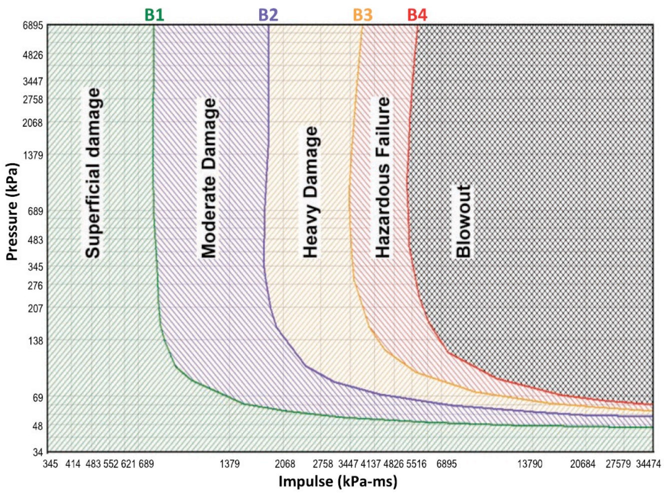


Figure 19. Pressure–Impulse relationships for deformations corresponding to damage limits (B1 to B4) (CSA, 2012)

4.3.2.4 Full dynamic approach

The previous sections showed that the assessment of structural capacity under the effects of a blast entails the calculation of the strength and ductility. Due to the complexity of the problem, more sophisticated full dynamic numerical approaches, such as the Applied Element Analysis (AEM), can be used. However, the use of constitutive models has to be handled carefully and the user must be aware of advantages and limitations of the models (NISTIR). Guidelines for such analysis are provided here below:

i) Blast load
- An explosion is a release of energy in a very short time; therefore, the resulting load is dynamic. As the dynamic load varies with time, the structural behaviour, internal forces, and geometry are also function of time.
- If the element mass is set as zero, then the analysis is static as the inertia forces will be zero. So, an appropriate definition of the element masses is required.

ii) Material Models
- In steel and steel-concrete structures, the material models can be Linear, Bilinear, Multi-Linear, or User-Defined models;
- Steel and concrete models can already be integrated in the library of the program.

iii) Failure criteria
- Elastic materials behave linearly without any plastic deformations. A predefined failure point can also be set.
- Different failure criteria may be employed.

- Steel: the failure criterion is based on the principal tensile strain.
- Steel reinforcement: the failure criterion is reached when the resultant stress reaches the ultimate strength of the reinforcing bar.
- Concrete: Tensile strength; compressive strength; shear; If the material is subjected to combined normal compressive stresses and shear stresses, the failure of the material can follow the Mohr-Coulomb failure envelope (Applied Science International, 2021).
- Other acceptance criteria based on test results, tabulated values or best design practice can be adopted.

iv) Calculation time step

- Time effects are continuous during the entire analysis. However, the numerical solution assumes a small-time step that can follow the structural behaviour.
- A too short time step will result in very long analysis time, while using a large time step will result in less accurate analysis and the numerical solution may fail to converge.
- If the time step is ΔT, then the shortest period that can be considered in the analysis is $2\Delta T$ (highest frequency is $\pi/\Delta T$). All frequencies higher than this frequency will not affect the analysis.
- Blast analysis usually requires ΔT of 0.00001 sec.

v) Blast scenarios

- Blast effects are modelled using free-air and surface-field models of blast waves. The pressure resulting from the blast wave is a function of explosive weight, distance to the explosive and time. Alternatively, user-defined blast pressure may be adopted (see (Laszlo et al. 2020)). Advanced load prediction techniques including CFD may be used if necessary.

vi) Boundary conditions and initial state

- The boundary conditions can be either displacement restraints or rotational restraints. Also, supports can be rigid supports and/or deformable supports.
- To solve a time-dependent problem numerically, initial conditions are required (velocity and acceleration values at the start of motion, t = 0.0). By default, the body (structure) initial conditions are set equal to zero as long as the motion starts from rest.

vii) Equilibrium equations

- The overall equilibrium set of equations in the dynamic problem is as follows:

$$[M]\{\ddot{x}\} + [C]\{\dot{x}\} + [K]\{x\} = \{f\} \qquad (11)$$

where $[M]$ is the mass matrix, $[C]$ is the damping matrix, $[K]$ is the stiffness matrix, $\{f\}$ is the external load vector, and $[x]$ is the displacement vector.

The solution for dynamic problems adopts the step-by-step integration method. The solution of the equilibrium equations is solved using an exact solver (in ELS, Cholesky upper-lower decomposition).

4.3.3 Internal gas explosion - Explicit design

4.3.3.1 General (EN 1991-1-7 2006)

Internal explosions shall be considered in the design of all parts of the building where gas is burned or regulated, or where explosive material such as explosive gases, or liquids forming explosive vapor or gas is stored or transported (e.g., dwellings with gas installations). Structures shall be designed to resist progressive collapse resulting from an internal explosion. Design may permit failure of a limited part of the structure provided this does not include key elements upon which the stability of the whole structure depends (see Table 6). The explosive pressure should be assumed to act effectively simultaneously on all the bounding surfaces of the enclosure in which the explosion occurs. When calculating the structural response, a triangular load-time function with a duration of 0,2 s may be adopted. A sensitivity study on the load-time function should be performed to identify the peak load time within the 0,2 s duration.

Table 6. Design considerations for gas explosion as a function of consequence classes (EN 1991-1-7, 2006)

CC1	No specific consideration of the effects of an explosion should be necessary other than complying with the rules for connections and interaction between components provided in EN 1992 to EN 1999
CC2	Key elements of the structure can be designed to resist actions by using an equivalent static load model
CC3	A dynamic analysis should be used

4.3.3.2 Equivalent static pressure approach

According to EN 1991-1-7, the nominal equivalent static pressure associated to natural gas explosions can be computed using the following formulae:

$$p_d = 3 + p_{stat} \tag{12}$$

or

$$p_d = 3 + \frac{p_{stat}}{2} + \frac{0.04}{(A_v/V)^2} \tag{13}$$

whichever is the greater.

where:

- p_d is the nominal equivalent static pressure to design the structure in [kN/m^2];
- p_{stat} is the uniformly distributed static pressure at which venting components will fail in [kN/m^2];
- A_v is the area of venting components in [m^2];
- V is the volume of rectangular enclosure in [m^3].

Where building components with different p_{stat} values contribute to the venting area, the largest value of p_{stat} should be used. No value of p_d greater than 50 kN/m^2 should be taken into account. The ratio of the area of venting components and the volume should comply with the following formula:

$$0.05m^{-1} \leq A_v/V \leq 0.15m^{-1} \tag{14}$$

4.3.3.3 Dynamic approach (TNT equivalence method)

The principle of the TNT equivalent method is the conversion of the mass of the gas (or vapour cloud) into a TNT equivalent charge. The equivalent TNT charge is estimated from the energy content in the exploding gas cloud. Equivalent mass W_{TNT} can be calculated based on the following equation material:

$$W_{TNT} = \eta \frac{W_g \times E_c}{E_{TNT}} \tag{15}$$

where:

- η is the explosive yield (or efficiency) factor;
- W_g is the mass of vapour in cloud of gas (equal to the mass of the air and flammable gas mixture);
- E_C is the heat of the combustion of the flammable material;
- E_{TNT} is the detonation energy of TNT.

For typical hydrocarbons (e.g., methane, propane, butane), the energy equivalence ($\frac{E_C}{E_{TNT}}$) can be taken as 10. For a natural gas explosion, if the explosive yield (or efficiency) factor is considered as equal to 20 % (η=0.2), the equivalent mass of TNT can be estimated (assuming atmospheric pressure initially) with the following formula (Bjerketvedt et al., 1997b; Harris and Wickens, 1989):

$$W_{TNT} \cong 0.16V \ [kg] \tag{16}$$

where:

- V [m^3] is the smaller of either the total volume of the congested region or the volume of the gas cloud.

When the equivalent mass of TNT is known, then the wavefront parameters (pressure, impulse, duration) can be determined using the methods presented in Section 4.3.2.

The limitations of the TNT equivalent method are:

- This method can be applied with satisfactory results to strong gas cloud explosions. For explosion pressures below 1 bar, the TNT equivalent method will overestimate the pressure.
- The deviation is small for describing the far field effects, while it is large for describing the near field effects.
- The TNT equivalent method can be useful as a rough approximation if one uses a yield factor of 20% and appropriate value for *V* (or the corresponding mass of hydrocarbon).

4.4 Fire as exceptional event

Fire actions should always be considered when designing steel and composite structures, using the prescriptive approach or a performance-based approach prescribed by the Eurocode. These design approaches are presented in Parts 1-2 of EN 1993 and EN 1994 and it is detailed in the FAILNOMORE background document (Demonceau et al., 2021).

A fire action as exceptional event should be seen as fire events not directly covered by the building regulation, in terms of intensity or location, due to their low probability of occurrence, but which could be associated with significant consequences. This is the situation which is addressed in the present section.

4.4.1 Prevent/eliminate hazard

Fire in buildings can be a result of different events such as blast and earthquake or it can be ignited directly (careless use of matches, cigarettes and pipes, faulty wiring or electrical equipment, careless use of cooking equipment, etc). The first step to avoid the ignition of a fire is avoiding any self-igniting materials in the building (storage of chemicals, petrol).

Building regulations are specifying rules regarding the storage of such materials in buildings – often storage around columns is prohibited as a general rule. The other aspects regulated by law are materials used for facades and the distance between buildings to reduce the risk of fire spread between buildings and along the building. Among other systems preventing fire spread (passive and active) and mitigate the effect of hazard are:

- Fire extinguishers – activated manually, when fire appears;
- Sprinklers – automatic systems activated, when smoke or high temperature arises;
- Fire walls – vertical isolation preventing fire spread;
- Vent insulators – insolation of any openings between compartments;
- Compartmentation – separation of building into quarters, between which the fire cannot be spread.

Very important is a quick detection of fire and early warning and evacuation systems enabling fast evacuation of occupants and quick activation of fire man and systems to stop the fire. For this issue, the use of the following equipment can be contemplated:

- Smoke detectors;
- Thermal detectors;
- Alarm systems;
- Exit road marking.

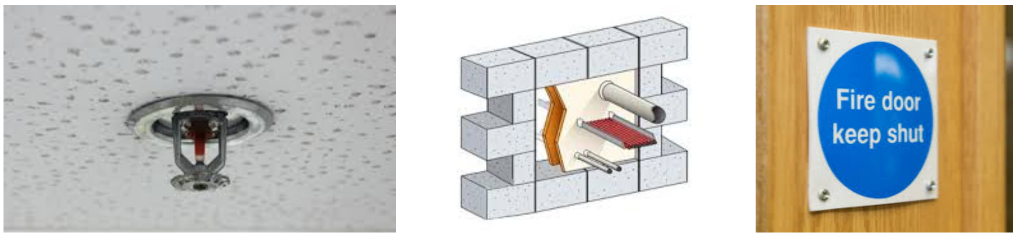


Figure 20. Sprinkler, vent insulators, fire door

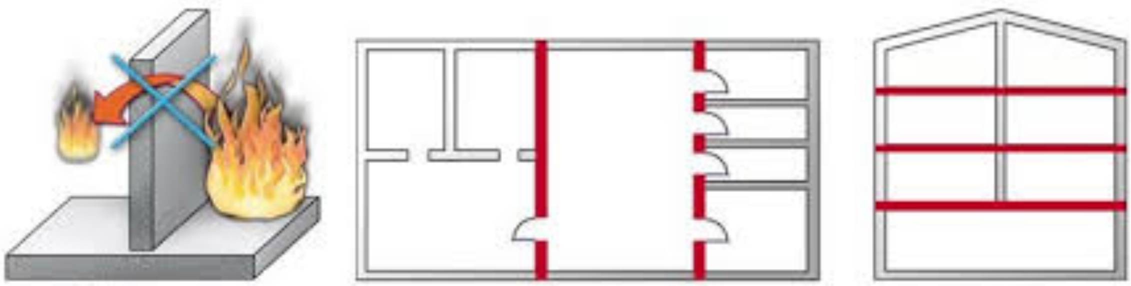


Figure 21. Fire wall and compartmentation

Figure 22. Fire extinguishers and early warning systems.

4.4.2 Design strategy

An example of fire as exceptional event is a fire localised around a column (when in normal situation fire load should not be located here) due to exceptional thermal loading. This action can be taken into account using a model defined in Annex C of EN 1991-1-2 (see next subsection) and/or by advanced fire models such as Zone models or CFD models. The model of Annex C is presented in Section 4.4.2.1 while recommendations for advanced fire modelling are reported in Section 4.4.2.2.

However, based on recent research results, it can be highlighted that the increase of temperature due to an unexpected localised fire situation is unlikely to lead to the collapse of some of the bearing elements and consequently the loss of stability of the structure, when the structure was designed for fire following the rules of the Eurocode and the National requirements.

Table 7 shows four fire scenarios and load that have been considered for localised fire next to a column. The resulted distribution of temperature along the columns is illustrated in Figure 23. As it can be seen, only at the bottom of the column (up to 1 m) significant steel temperatures is reached that could cause some local buckling or plastic failure.

Table 7. Different localised fire scenarios for office buildings and commercial areas (Demonceau et al., 2021)

Scenario	Diameter of the fire basis	Rate of heat release density	Fire load density	Fire growth rate
A	2 m	250 kW/m^2 (office building)	511 MJ/m^2 (office building)	300 sec (office building)
B	1 m	500 kW/m^2 (office building)	511 MJ/m^2 (office building)	300 sec (office building)
C	2 m	250 kW/m^2 (commercial area)	730 MJ/m^2 (commercial area)	150 sec (commercial area)
D	1 m	500 kW/m^2 (commercial area)	730 MJ/m^2 (commercial area)	150 sec (commercial area)

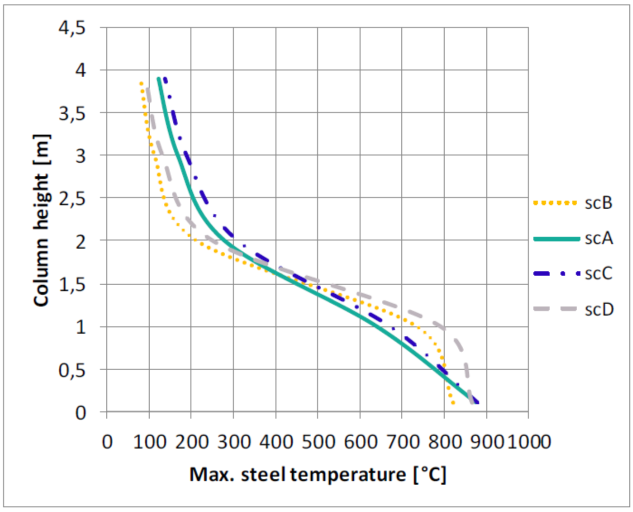


Figure 23. Increase of temperature along the height of the column for different localised fire scenarios (Demonceau et al., 2021)

A different and more severe situation in terms of robustness with fire defined as an exceptional load is when a sequence of exceptional event scenarios is considered, such as fire after an earthquake or after an impact or explosion. In those situations, the structure is already damaged after the first event, thus, the normal fire design is not valid anymore, since this design always considers that the structure is undamaged. Therefore, the fire event should be considered as an exceptional load. For these cases, column loss scenarios (see Section 5) could be considered for the design for robustness as a safe sided approach.

4.4.2.1 Localised fire models

In the model of Eurocode, a localised fire (or pre-flashover fire) is a fire where a flashover is unlikely to occur. Depending on the size of the fire and of the compartment, a localised fire can or not impinge on the ceiling of the compartment. In this model, the temperature in the flame and plume and the surrounding gases are not uniform. This model is described in Annex C of EN 1991-1-2 (EN 1991-1-2, 2002).

A localised fire impinges the ceiling of the compartment when the length of the flame (L_f), estimated through Eq. (17), is equal to or higher than the distance between the fire source and the ceiling (H).

$$L_f = 0.0148Q^{0.4} - 1.02D \tag{17}$$

with D, the diameter of the fire and Q, the rate of heat release (Annex E of EN 1991-1-2) .

The temperature of the flame along the symmetrical vertical flame axis when L_f < H can be obtained by Eq. (18)

$$\theta_{(z)} = 20 + 0.25Q_c^{2/3}(z - z_0)^{-5/3} \leq 900 \tag{18}$$

with Q_c, the convective part of the rate of heat release (=0.8Q); Z, the height of the flame along its axis; Z_0, the virtual origin of the fire (Eq.(19))

$$Z_0 = -1.02D + 0.00524Q^{2/5} \tag{19}$$

For cases when the flame impinges the ceiling, the net heat flux received by the fire exposed per unit of surface at the level of the ceiling is given by Eq. (20).

$$\dot{h}_{net} = \dot{h} - \alpha_c(\theta_m - 20) - \Phi\varepsilon_m\varepsilon_f\sigma[(\theta_m + 273)^4 - (20 + 273)^4] \tag{20}$$

with $\dot{h}$, the heat flux received by the fire exposed per unit of surface at the level of the ceiling; α_c, the heat transfer coefficient by convection; θ_m, the temperature at the surface of the element; Φ, the configuration factor; ε_m, the surface emissivity of the member (0.7 – carbon steel; 0.8 stainless steel); ε_f, the fire emissivity; σ, is the Stephan Boltzmann constant (5.67 x 10^{-8} W/m^{2}K^4).

4.4.2.2 Advanced fire models

To use advanced fire models, it is always required to use specific software.

- Zone models – see Annex D of EN 1991-1-2 for basic equations of conservation of mass and energy. Examples of software that can be used are the CFAST from NIST or the OZONE developed in the University of Liege

- CFD model (Computational fluid dynamic model) – see Annex D of EN 1991-1-2 for suggestions. An example of software that can be used for CFD analysis is the FDS from NIST

4.5 Earthquake as exceptional event

Seismic risk results from the interaction of seismic hazard and structural vulnerability. Therefore, an earthquake can be considered exceptional when:

- Structure is not designed for a seismic action at all, e.g., it is designed for gravity and wind loads only (e.g., when the building site is not considered as seismic at the time of construction) or is designed for lower seismic demands – the hazard is therefore exceptional.
- Structure is seismically vulnerable (pre-existing damages, system not designed following modern code design requirements).

4.5.1 Prevent/eliminate hazard

An earthquake is a sudden release of strain energy accumulated in the Earth's crust. Earthquakes are caused mostly by the rupture of geologic faults. Other causes include volcanic activity, landslides (all with natural causes), but also human activities (mine blasts, nuclear tests, oil/gas drilling). Due to its nature, it is not possible to prevent or eliminate the seismic hazard. Therefore, the reduction and prevention of consequences (e.g., structural / non-structural damages) are strictly associated with the building structure and the integrated systems, which help the building to adequately respond to the seismic action, see next sections.

4.5.2 Prescriptive approach

Even not directly evaluating the performance of the structure in case of a seismic event, prescriptive requirements can improve the seismic response with minimum engineering effort and structural interventions. This approach is especially beneficial for non-seismic areas where seismic actions may occur but with a very low probability of occurrence, at least sufficiently low to not consider it in the design process. Indeed, this approach favours systems, materials, and detailing with performances demonstrated in past seismic events. Selection of the structural configuration and knowledge about the building's period, torsion, damping, ductility, strength, stiffness, can help one determine the most appropriate design strategy to employ:

- Building configuration: this term defines a building's size and shape, and structural and non-structural elements. Building configuration determines the way seismic forces are distributed within the structure, their relative magnitude, and other design concerns. Regular configuration buildings generally have:
    - Low Height to Base Ratios
    - Equal Floor Heights
    - Symmetrical Plans
    - Uniform Sections and Elevations
    - Maximum Torsional Resistance
    - Short Spans and Redundancy
    - Direct Load Paths
    - Design of secondary/non-structural elements to avoid debris.
- Torsional effects: they develop due to the asymmetric distribution of inertial masses and/or rigidities. Symmetrical arrangements will result in balanced stiffness and reduced torsional effects. Regularity in plan and in elevation is also recommended.
- Vibration control: buildings in general are poor resonators to dynamic shocks and dissipate vibration by absorbing it. The following systems can be employed to improve the response:
    - Base isolation can be used to detach (isolate) the building from the ground in such a way that seismic energy that is transferred to the superstructure is greatly reduced. Most suitable candidates for base-isolation are low to medium-rise buildings constructed on stiff soils; high-rise buildings or buildings constructed on soft soils are not suitable for base isolation.
    - Passive damping systems. The most common application is a tuned mass damper (TMD) device, which consists of a mass, a spring, and a damper that is attached to a structure. The seismic energy is dissipated by the damper inertia force acting on the structure.
    - Active damping systems. Active tuned mass dampers cancel out speed-dependent vibrations by counteracting the excitation forces of a disrupted main system. Each TMD consists of an actuator, a control system, and a power electronic unit. All the

components of the TMD are mutually balanced so that the TMD force acts in precisely the opposite direction of the excitation force.
    - Semi-active control systems, which take advantage of the best features of both passive and active control systems. The term “semi-active” is used to indicate that the operation of these systems requires a very small amount of external power. The control forces are developed through appropriate adjustment of damping or stiffness characteristics.
- Strength and Stiffness: Strength is a property of a material to resist the applied forces within a safe limit. Stiffness of a material is a degree of resistance to deflection. Selection of strength and stiffness properties should be done considering the balance between deformability and force resistant capacity.
- Ductility: Ductility is the characteristic of a material (such as steel) or element to dissipate part of the energy by plastic deformations. Ductile elements typically fail only after the development of considerable plastic deformations. Non-ductile elements, such as poorly reinforced concrete members, fail by brittle fracture, with no plastic deformations. The ductility demands can refer both to elements and to their joints.
    - For elements, the main requirements target the slenderness and the prevention of instability (e.g., lateral-torsional buckling of beams in flexure) before reaching their plastic strength. At the level of the section, ductile or semi-ductile cross-sections (class 1, class 2) are favoured.
    - For joints, symmetrical configurations are recommended, as they can provide a more stable hysteretic response throughout subsequent cycles. Also, components that fail in a brittle mode (e.g., welds, bolts) need to be provided with overstrength. To ensure a ductile behaviour of the joints, the recommendations from Section 2.2 can be followed.

4.5.3 Design strategies

In the aftermath of an earthquake, the primary concern is the structural condition and whether it is safe from collapse under gravity loads, earthquake aftershocks, and other hazards (FEMA P-2090, 2021). If the structure lacks the robustness, there is a risk of further damages or progressive collapse under an aftershock or other hazards, even the structure initially resists the ground motion. To avoid such a disastrous scenario, the building’s residual capacity needs to be assessed. The residual capacity after an earthquake can be defined as:

- load-carrying capacity of the lateral load-resisting system – the minimum spectral acceleration that corresponds to local or global collapse during an aftershock.
- load-carrying capacity of the gravity load-resisting system – the minimum level of gravity loads that corresponds to local or global collapse after a damaging earthquake.

In the following, a procedure for the evaluation of the seismic robustness is presented.

i) <u>Step 1: Design/evaluation for persistent / seismic design situations</u>

The structure is first designed to meet the code-based requirements (see Figure 24.a) (for new structures, only). The seismic response can be calculated using a Nonlinear Static analysis (N2 method, EN 1998) following the recommendations from EN 1993-1-14 (2020) regarding the behaviour laws to be used for the materials and the modelling of the structural elements.

The general load-deformation relation of a structural component can be characterized using prEN 1998-1-2:2019.3, Annex L (Figure 24.b). Thus, the component model shall be defined by:

- an effective elastic stiffness, K_e considering both flexural and shear deformations.

- the yield point, which is defined by the effective yield strength, Q_y*, and the corresponding yield deformation, δ_y*.
- the post-yield range, in which the structural component exhibits hardening prior to reaching its maximum strength, Q_{max}* (i.e., peak response).
- the pre-peak plastic deformation, δ_p* defines the plastic deformation up to the peak response of the structural component.
- the post-peak response is represented by the post-peak plastic deformation, δ_{pc}* of the component.

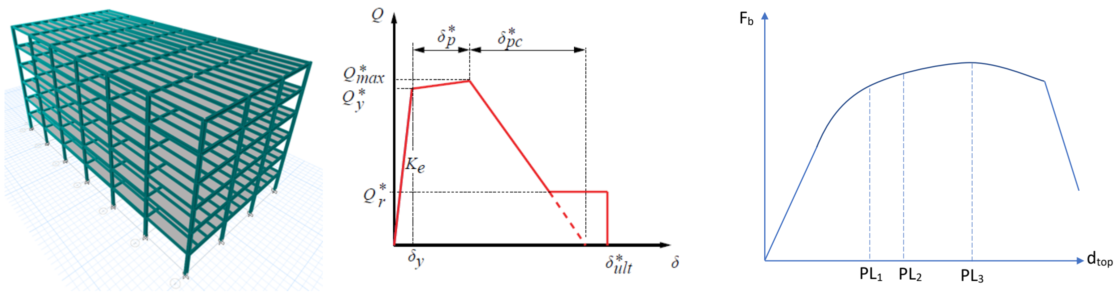
The global seismic performance can be presented in the form of a base shear force – top displacement $F_b - d_{top}$ curve, see Figure 24.c. Performance levels (*PL*) are defined by the corresponding maximum top displacement, e.g., PL_1 (limited damage), PL_2 (moderate damage) and PL_3 (large damage). Depending on the level of hazard, a certain damage level is expected.

ii) <u>Step 2: Evaluation of the residual capacity after an earthquake</u>

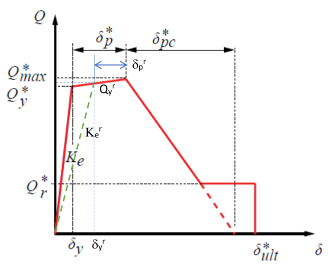
After the evaluation of the local and global ductility demands (Step 1), modifications of flexural hinges are introduced for the damaged elements (i.e., elements with plastic deformations), resulting in a modified nonlinear model (see Figure 24.d). The residual strength of a column shall be conservatively assumed as zero if the peak response is attained during the seismic motion. P-Δ effects must be accounted for (especially when residual lateral deformations after the earthquake are significant).

The resistance of the frame structure against a seismic aftershock can be evaluated using a nonlinear analysis (e.g., pushover analysis). The analysis is done on the damaged model.

The resistance of the frame structure against progressive collapse under gravity loads can be evaluated using a pushdown (vertical) static analysis using the methods proposed in Section 5.3.



a) b) c)



	Stiffness	Strength	Ductility
Initial (intact)	K_e	Q_y	δ_p
Damaged	K_e^r	Q_y^r	δ_p^r

d)

Figure 24. The steps of the seismic robustness assessment for framed structures (adapted from Polese et al., 2012): a) view with the building model; b) general definition of load-deformation relationship for steel and steel-composite structural components; c) seismic capacity curve obtained in the nonlinear static analysis for the undamaged (initial) structure; d) modelling parameters for the damaged plastic hinges

5 Unidentified threats

5.1 Selection of appropriate design strategies

Unidentified threats refer to accidental actions not specifically considered by standards or indicated by the client or other stakeholders or to any other actions resulting from unspecifiable causes. Due to uncertainties regarding the nature, the magnitude and the application point (region) of an unidentifiable accidental action, the required structural performance is usually impossible to estimate. Currently, the design strategies deemed to achieve an adequate level of structural robustness mainly seek to limit the extent of a localised damage, whatever is the initiating cause.

The identification of the localised damage to consider is addressed in Section 5.2 while the design strategies to check the adequate level of robustness are described in Section 5.3 (alternative load path methods), 5.4 (key element method) and 5.6 (segmentation method).

5.2 Identification of local damages

Generally, the main objective of robustness design is to ensure that any local damage resulting from unforeseen extreme events does not cause disproportionate collapse. In this regard, any local damage scenario has to be threat-independent. Accordingly, this requires the identification of local damages to be considered in the design process.

If reference is made to the present draft of (EN 1991-1-7, 2006), the local damage to be considered for building structures included in the upper group of Consequences Classes (CC 2b and CC3) is the notional removal of each supporting column, or each beam supporting a column, or of any section of load-bearing wall (one at a time in each storey of the building).

The concept of "*Notional column removal*" stated in (EN 1991-1-7, 2006) represents the removal of the entire column on the clear height between the connections at the level of the floors. Elements are removed without affecting end joints / connections. Notional removal of a column may not always be conservative, due to the infinite possibilities of loading scenarios and load-structure interaction, but for an accomplishable assessment of the structural system capacity to transfer loads through alternative paths, notional removal is seen as an efficient and practical analysis scenario.

In the Eurocodes, it is not stated if this notional column removal has to be assumed as instantaneous or as "quasi-static". The consideration of a "quasi-static" removal allows (i) the use of more simple tools as no dynamic effects need to be accounted for and (ii) to have a good indication on the ability of a structure to activate alternative load paths. However, the consideration of an instantaneous of local part of the structure maximises inertial effects; in particular, sudden column loss was shown to offer an upper bound on the subsequent response of building structures in comparison with column damage due to a blast event (Gudmundsson and Izzuddin, 2010). In addition, damaged elements may have residual capacity, which is not conservatively taken into consideration implicitly, except in case of applying residual strength method.

As stated previously, the removal of each supporting element, one at a time, should be contemplated according to (EN 1991-1-7, 2006), what could require a significant amount of design work. However, possibilities of reducing the number of local damage scenarios to be considered in the design process exist, in particular in regular building structures for which design scenarios can be identified considering possible structural symmetry, similarity of boundary conditions and other engineering reasoning principles. In UFC 04-023-03 (DoD, 2016), it is required to consider at least the following column loss for a storey plan as a minimum of scenarios (see Figure 25):

- external columns and internal columns near the middle of the short side and the middle of the long side;
- the corner of the building;
- columns at locations where the plan geometry of the structure changes significantly, such as abrupt decrease in bay size or re-entrant corners;
- columns with adjacent columns lightly loaded or adjacent bays with different tributary sizes;
- locations where members frame in at different orientations or elevations;
- locations where the structure has any vertical load discontinuity (i.e., transfer conditions) (GSA, 2016).

For locations in terms of the storey itself, the following should be considered:

- First storey above ground;
- Storey directly below the roof;
- Storey at mid-height;
- Storey above the location of a column splice or change in column size.

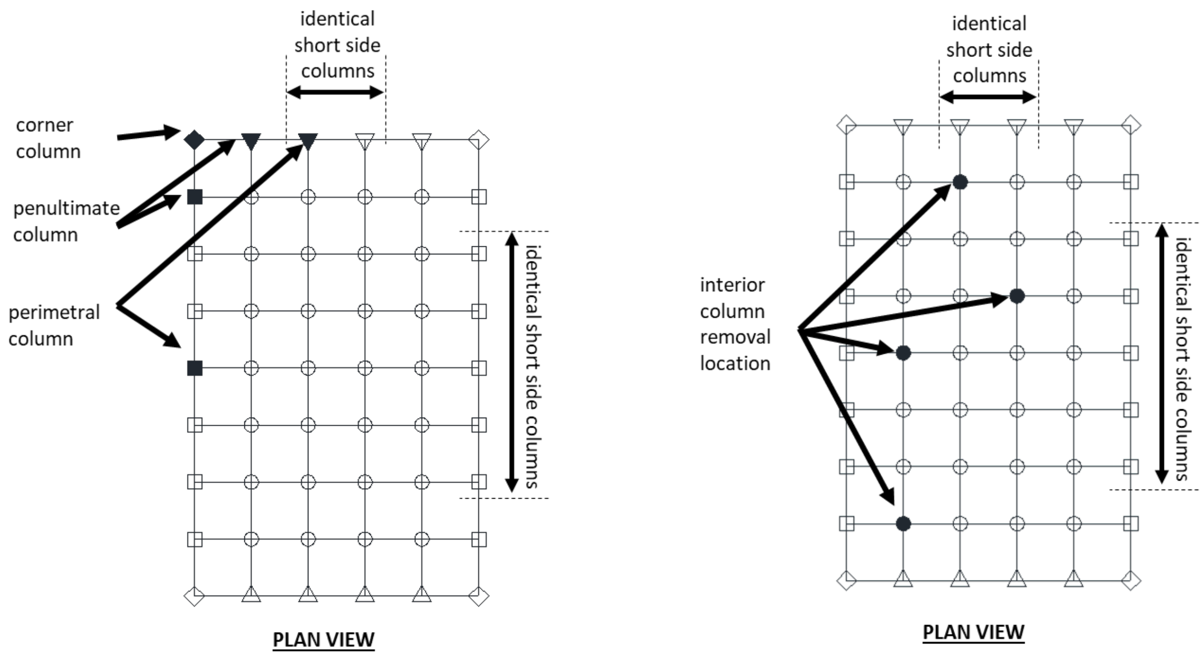


Figure 25. External and internal column removal scenarios (DoD, 2016)

For the considered local damage scenario, the extent of damage it would create should be limited. The Annex A of the current version of EN 1991-1-7 (EN 1991-1-7, 2006) and Annex E of the proposed draft of the imminent second generation EN 1990 (prEN 1990:2019, 2019) specifies this limit as 15 % of the floor area or 100 m^2, whichever is smaller in each of the two adjacent storeys to the one where the column was removed. However, in principle, the acceptable limit of damage can be defined by the client or relevant authorities based on performance objectives related to the importance of the structure and the consequences of such damage on life safety, protection of valuable contents or minimisation of operational downtime of the structure.

If for the respective scenario the damage limit cannot be respected, it means that this scenario (local damage) cannot be allowed to occur, and so the supporting element which was assumed to be lost has to be prevented from failure and designed as a key element.

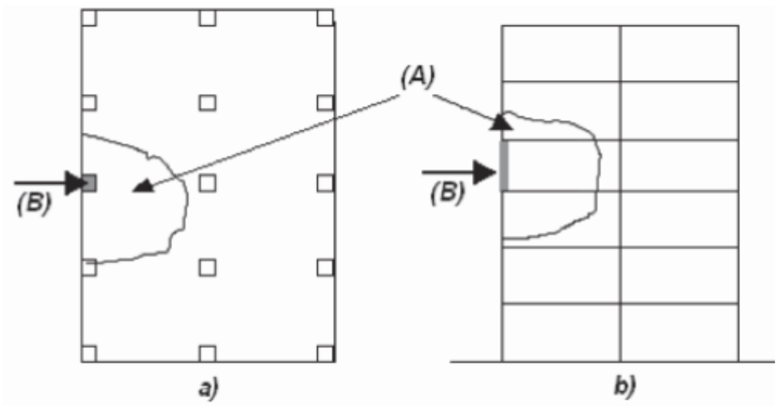


Figure 26. Acceptable limit of damage in case of removal of a column in a framed structure. The limit 'A' is 15 % of the floor area, or 100 m^2, whichever is smaller, in each of two adjacent storeys. 'B' is the column notionally removed. A) Plan and b) Section (EN 1991-1-7, 2006)

5.3 Alternative load path methods

A building structure losing a column can be divided in two main parts, as illustrated in Figure 27:

- the directly affected part (DAP) which represents the part of the building directly affected by the column loss, i.e., the beams, the columns, and the beam-to-column joints which are just above the failing column;
- the indirectly affected part (IAP) which includes the rest of the structure; this one is affected by the loads developing within the directly affected part; but obviously, these forces are themselves influenced by the own response of the indirectly affected part.

If a cut is realised in the structure at the top of the failing column (see Figure 27), different internal forces in the vertical direction are identified: (i) the shear forces V_1 and V_2 at the beam extremities close to the failing column, (ii) the axial force N_{up} in the column just above the failing column and (iii) the axial force N_{lo} in the failing column.

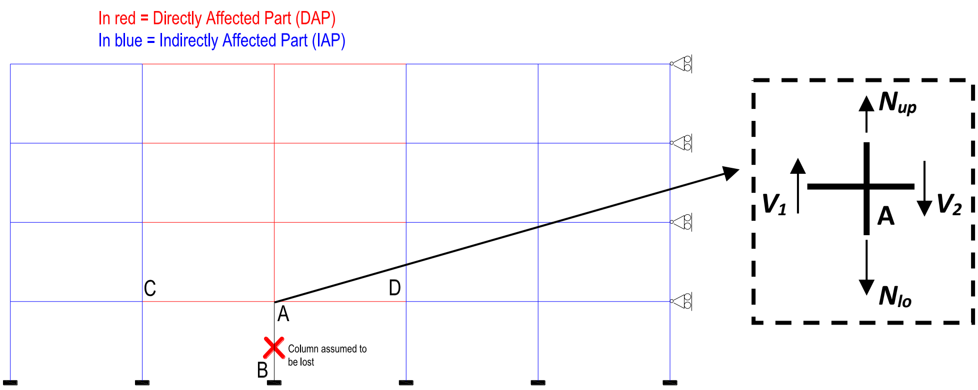


Figure 27. Schematic representation of a frame during a column loss

In Figure 28, a curve representing the evolution of the vertical displacement Δ_A according to the normal load N_{lo} in the failing column during the exceptional event (see Figure 27) is illustrated.

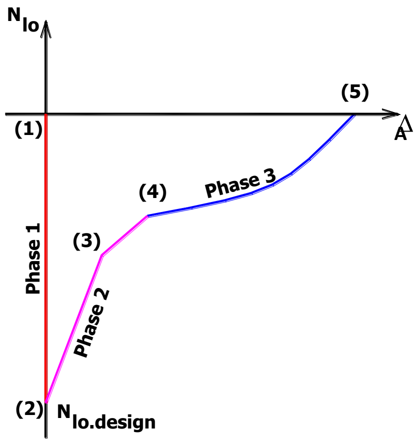


Figure 28. Evolution of N_{lo} according to the vertical displacement at the top of the loss column

- From point (1) to (2) (Phase 1), the axial force N_{lo} in the column AB increases until it reaches a value $N_{lo.design}$ associated to the accidental load combination assumed to act when the event occurs (N_{lo} is reported with a negative sign in compression in the figure) while Δ_A remains approximately equal to 0 during this phase.
- From point (2) to (5), the column is progressively removed as a consequence of the exceptional event. The compression force in column AB decreases until it reaches a value equal to 0 at point (5) where the column is considered as fully removed. In the same time, the value of Δ_A increases. Along this unloading path, different types of structural responses which could potentially develop may be identified:
  - From point (2) to (4) (Phase 2): during this phase, the directly affected part passes from a fully elastic behaviour (from point (2) to (3)) to a global plastic mechanism. At point (3), the first plastic hinges appear in the directly affected part while, at point (4), complete beam mechanisms have developed. Obviously, this contribution, which is illustrated in Figure 29, may only occur when partial-strength or full-strength joints connect the beam extremities to the columns; for partial-strength joints, the plastic hinges occur in the joints, while for full-strength or over-strength joints, the plastic hinges develop at beam ends (see Section 2.2.1).

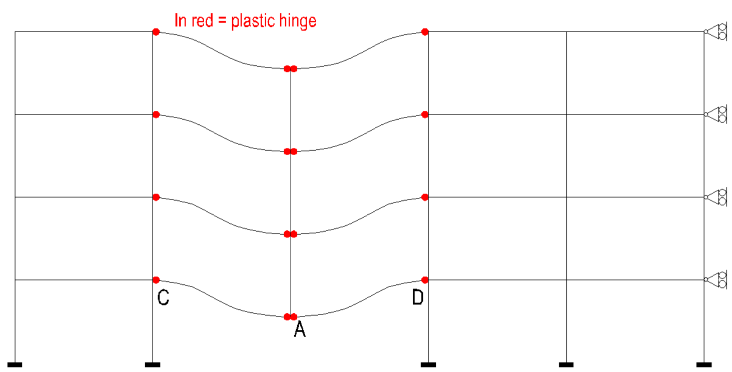


Figure 29. Development of beam plastic mechanisms in the DAP

In some specific situations that will be detailed in the following sections, this “plastic mechanism” contribution to the structural robustness is complemented by a so-called "beam arch" contribution. This "arching" effect is illustrated in Figure 30 for the lower DAP beams of the structure shown in Figure 29. In fact, because of the actual non-zero height of the beam and the cinematic conditions to be respected to form the mechanism, points C and D must first move away from each other before coming progressively closer when the vertical displacement becomes significant. This induces membrane compression forces in the beams and so an arching effect develops (which may be visualised in the form of an arch resulting from the inclination of the beam diagonals – dashed lines in Figure 30). The resistance of this arch is widely dependent on the capacity of points C and D to move relatively each to another. The longitudinal spring $K_{H,c}$ in Figure 30 represents this relative movement capacity. In the particular structure shown in Figure 30, the horizontal displacement of point D is prevented by the presence of an efficient bracing system at the right side of the structure (this one is materialised by lateral supports at each storey level in Figure 29). This results in an infinite value of $K_{H,c,right}$. For point C, on the other hand, the capacity to move laterally is linked to the stiffness $K_{H,c,left}$ of the left side of the IAP under the action of the DAP compression force generated by the arch. If a second bracing system is installed on the left side of the structure, then the spring stiffness of $K_{H,c,left}$ would be also almost infinite and the resulting arching effect would be quite significant. If, on the contrary, $K_{H,c,left}$ is rather low, the arching effect will be quite negligeable. For the sake of simplicity, both values of $K_{H,c,right}$ and $K_{H,c,left}$ are merged as indicated in Figure 30 in an equivalent stiffness coefficient $K_{H,c}$. The geometry and the properties of the beam-to-column joints may also influence the arching effect; this will be discussed further in the relevant sections.

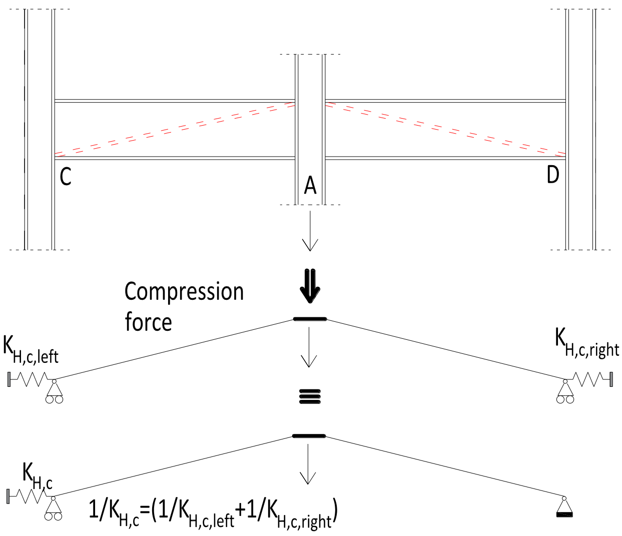


Figure 30. Development of arching effects in the DAP

- From point (4) to (5) (Phase 3): during this phase (Figure 31), high displacements are observed in the directly affected part and therefore second order effects play an important role. Significant catenary actions develop in the bottom beams of the directly affected part. As explained below, after the beam mechanisms are formed, both C and D points come closer together and, if this movement is somewhat prevented ($K_{H,c}$ is now substituted by $K_{H,t}$), axial tensile membrane forces appear in the beams and lead to a new significant contribution to the robustness of the structure.

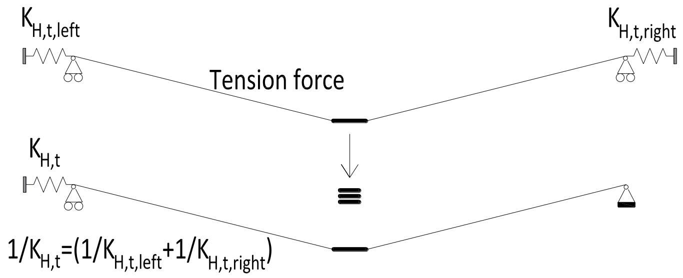


Figure 31. Development of catenary effects in the DAP

According to the type of joints and of the structural system, these three contributions to the robustness may occur or not. This point will be addressed when required in the following sections.

But it must be noted that the following conditions have imperatively to be respected to benefit from the various above-listed robustness contributions:

- the resistance of the directly affected part and of its components (beams and joints) is adequate;
- the different structural elements have a sufficient ductility and/or rotation capacity to reach the vertical displacement Δ corresponding to point (5); close to full-strength joints, ductility is required from the joint and from the connected beam while, in the case of over-strength joints, ductility is only required in the beam section.

Moreover, the loads which are transferred from the directly affected part to the indirectly affected part should not induce the premature failure of elements in the latter. From that point of view, three failure modes may be identified (Figure 32):

- the buckling of IAP columns adjacent to the lost column, which will be subjected to additional compression forces;
- the development of a global plastic mechanism in the indirectly affected part under the action of the membrane forces transferred by the DAP to the IAP of the structure;
- the buckling in compression of the upper beams of the IAP as a result of a possible progressive development, in the whole structure, of an arching effect induced, in the specific case of Figure 32 by the sway deformability of the left part of the IAP of the structure.

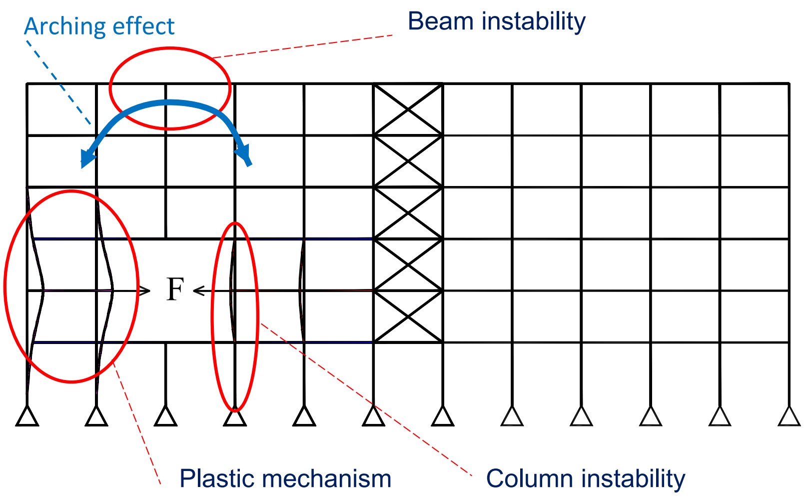


Figure 32. Possible failure modes in the IAP

All these conditions, for both DAP and IAP parts, will always have to be considered and duly checked to guarantee the requested level of structural robustness to be associated to the three above-defined contributions (plastic mechanism, beam arching effects and catenary effects).

Finally, the significant contribution of the concrete floor slabs to the structural robustness needs to be highlighted. When these slabs are adequately connected to the steel beams, so developing a composite action, their positive benefit on the DAP (beam mechanism / arching effect / catenary effect) may be directly covered through the definition of composite steel-to-concrete beams. When no composite action is contemplated between the concrete slabs and the supporting beams, the own level of resistance of the slab may also be considered in the robustness evaluation, but in a specific way. In addition, whatever the case (composite action or not), the slabs have a positive influence on the values of the restraining K_c and K_t stiffness coefficients.

These different aspects will be raised in the next sections successively addressing the four possible design methods to implement the alternative load-path approach: the prescriptive method (Section 5.3.1), the analytical method (Section 5.3.2), the simplified numerical method (Section 5.3.3), the full numerical method (Section 5.3.4).

Amongst these methods, three of them aim at quantifying the three above-listed structural contributions to the robustness: the analytical method, the simplified numerical method, and the full numerical method. On the contrary, the prescriptive method proposes a set of verifications which are not directly linked to the actual response of the structure.

When dynamic aspects are to be considered which would not have been covered by the application of the advanced analytical method, the simplified numerical method, or the full numerical method, a possibility exists to derive the dynamic response from the static one. This procedure is presented in Section 5.3.5.

5.3.1 Prescriptive methods

The tying force method is a prescriptive indirect design method that is assumed to provide a minimum level of structural robustness and resistance to progressive/disproportionate collapse. In particular, the method ensures that a minimum level of continuity and strength is achieved between the different structural members by means of horizontal and vertical ties as illustrated in Figure 33, resulting in an enhanced overall structural integrity. This approach is adopted by most of the design codes and recommended by different design guidelines to increase the resistance to progressive and/or disproportionate collapse of low and medium risk structures, e.g. Eurocode EN 1991-1-7 (2006), UFC 4-023-03 (DoD, 2016), ASCE/SEI 7-16 (ASCE, 2017a) and IBC 2009 (ICC, 2018).

Tying requirements are typically specified in either horizontal members/components only or both horizontal and vertical members/components depending on the level of risk associated with the structure and the consequences of its collapse. In particular, in EN 1991-1-7, horizontal tying is required for the lower group of Class of Consequences 2 (CC2a – see Chapter 3) while both horizontal and vertical tying is required for the upper group of Class of Consequences 2 (CC2b – see Chapter 3).

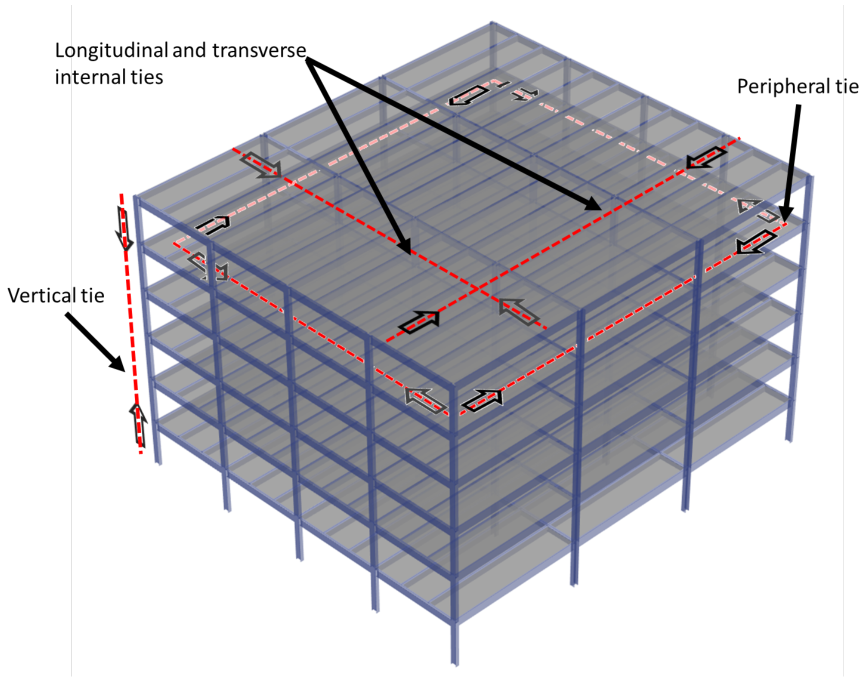


Figure 33. Typical tying for framed structures (Stylianidis, 2011)

5.3.1.1 Horizontal tying

5.3.1.1.1 Method proposed in EN 1991-1-7

According to EN 1991-1-7, horizontal ties should be provided around the perimeter of each floor and roof level and internally in two orthogonal directions to tie the column and wall elements securely to the structure of the building (see Figure 34). Generally, horizontal tying can be provided by steel beams (and their end connections), steel bar reinforcement in concrete slab or fabric mesh reinforcement and profiled steel sheeting in composite floors. To rely on the steel sheeting, the tie should act in the same direction of the profiled sheeting and the sheeting should be directly fixed to the steel beam with shear connectors (shear connectors welded to the beam flange through the sheeting).

In the current codes and regulations, the horizontal ties, including both members and connections, have to be designed to be capable of resisting minimum levels of tying forces. In particular, minimum tensile design forces are proposed in EN 1991-1-7. For framed structures, they can be estimated using the following formula:

$$T_i = 0.8(g_k + \psi q_k)sL \qquad or \qquad 75\ kN, \text{whichever is greater} \tag{21}$$

$$T_p = 0.4(g_k + \psi q_k)sL \qquad or \qquad 75\ kN, \text{whichever is greater} \tag{22}$$

where:

T_i is the design tensile load for internal ties;
T_p is the design tensile load for perimeter ties;
g_k is the permanent surface action applied on the considered floor;
q_k is the variable surface action applied on the considered floor;
s is the average spacing of adjacent ties (s = (s_1 + s_2)/2 – see Figure 34);
L is the span of the tie (Figure 34);
ψ is the relevant combination factor of action effects for accidental design situations as defined in EN 1990 (CEN 2005).

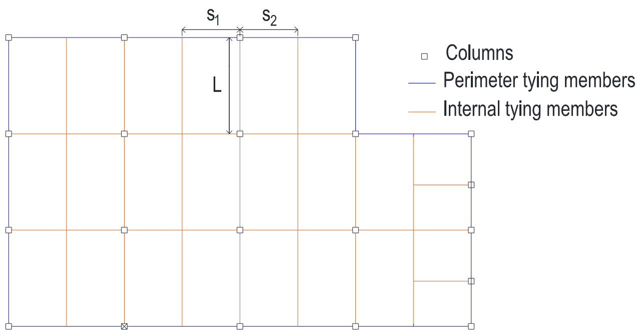


Figure 34. Horizontal tying in a building floor

In addition, EN 1991-1-7 specifies some other provisions to be applied. It states that the horizontal ties should be provided:

- in continuous lines;
- in the case of the perimeter ties, arranged as close as possible to the floor edges;
- in the case of ties intended to be provided on the column lines, arranged as close as possible to the lines of columns;
- such that 30% or more of the ties are located at close vicinity of the column grid lines.

As stated before, the members and the joints at their extremities have to be able to support the tying forces assumed to be applied alone, even if these members are also used to support gravity loads (for instance floor beams used as tying members). The members subjected to tensile loads can be easily checked. However, the characterisation of joints subjected to tensile loads and, in particular, the prediction of their plastic and ultimate tensile resistance is not explicitly covered in the present draft of the Eurocodes. Rules are proposed in Annex A.1 allowing for the characterisation of structural joints under axial loads. In the framework of the present approach, the tensile loads can be compared to the ultimate resistance of the structural elements.

In addition, to ensure the efficiency of the prescriptive method and so the possible activation of the tying members, it is also required to guarantee a minimum level of ductility, in particular at the extremities of the beams to allow for a minimum deformation capacity of the structural system. However, even if this need for minimum level ductility is clearly stated in EN 1991-1-7, no specific recommendations are provided on what is meant by “minimum level” of ductility and on how to guarantee it.

If over-strength joints are used at the extremities of the beams, this ductility will be required at the level of the beam itself. In such situation, it is recommended to use Class 1 cross-sections under bending moment (sagging and hogging). In the case of full-strength joints, ductility is required from the joint and the beam while, if partial-strength or simple joints are used, this ductility/deformation capacity will be required at the level of the joints. Reference can be made to Section 2.2 where criteria to ensure a minimum deformation capacity to structural joints are provided.

It has to be highlighted that the minimum tensile design forces computed using the above-mentioned procedure are defined in order to ensure a minimum level of continuity/redundancy in the floor and does not at all reflect the level of tensile forces which could occur in case of a complete loss of column, which are generally much higher. Also, a solid link between the tying capacity and the actual resistance to progressive collapse cannot be established (Nethercot et al., 2010; Vlassis et al., 2008) and so the efficiency of this method remains questionable.

5.3.1.1.2 Specificities of composite floors

In case of composite floors, i.e., floors made of steel profiles with the upper flange connected to the slab, the method presented here above can be safely used neglecting the composite character of the floor. However, in the case of a column loss scenario, such a structural solution allows the development of membrane action in the composite beams and in the connected slab, so leading to the activation of an alternative load path. The efficiency of this solution has been demonstrated through experimental tests performed in Europe (Kuhlmann et al., 2017; Zandonini et al., 2014).

To ensure the efficiency of the composite solution, the use of steel beam grids with the upper flange of the beams in the two main directions connected to the slab is recommended to guarantee a good collaboration between the steel members and the slab in both directions but also to allow for a proper anchorage of the slab on the lateral beams when membrane forces develop.

Through recent studies (Demonceau et al., 2013; Kuhlmann et al., 2017), it has been demonstrated that (i) membrane forces mainly developed in the slab of composite floors while limited tensile forces develop in the composite beams and (ii) the activation of these membrane forces requires much less deformation capacity at the level of the structural beams. Accordingly, the activation of alternative load path in composite floors will require (i) composite beams with a minimum level of ductility at their extremities to allow for the development of a plastic mechanism and (ii) a collaborative slab with appropriate constructive details, in particular in terms of reinforcement.

As previously mentioned, the composite beams will be mainly subjected to bending moments, the tensile forces developing in the beams being limited. Accordingly, the ductility at the level of the composite beam extremities is required under bending moments only. Four situations can be met in practice according to the nature of the joints at the extremities of the composite beams:

- *Over-strength joints are used and so the ductility is required at the level of the composite beams*. As the objective is to develop a plastic mechanism with a minimum level of deformation capacity, the use of Class 1 cross-sections under sagging and hogging moments is recommended.
- *Partial-strength joints are used and so the ductility is required at the level of the joints*. In such situation, reference is made to Section 2.2 where design recommendations are provided to ensure a minimum level of ductility to partial-strength joints.
- *Simple joints are used and so a minimum level of rotation capacity is required at the level of the simple joints*. In such a situation reference is again made to Section 2.2 where design recommendations are provided to ensure a minimum level of rotation capacity to simple joints.
- *Full-strength joints are used and so ductility is required at the level of the joints and of the beams.*

Regarding the collaborative slab, different solutions can be contemplated: reinforced concrete slab fully cast on site, reinforced concrete slab using precast concrete elements or composite slabs. No specific design recommendations or constructive details are provided in the present draft Eurocode 4 (EN 1994-1-2, 2005) to guarantee the possibility of activating membrane forces within the slab while minimum requirements are given in Eurocode 2 (EN 1992-1-1, 2005), more precisely in Section 9.10.2, to provide a floor with a tying system. So, for composite floor, it is suggested here to follow the minimum requirements of Eurocode 2. The application of this recommendation can be seen as the application of a prescriptive tying method specific to composite floor.

For collaborative reinforced concrete slab cast on site, the above-mentioned requirements from Eurocode 2 can be directly applied. For collaborative slab using precast concrete elements, specific rules, coming in addition to the above-mentioned requirements of Eurocode 2, are proposed in (CEN/TC250/SC4, 2020) to ensure a proper anchoring of the slab to their supports (see Figure 35).

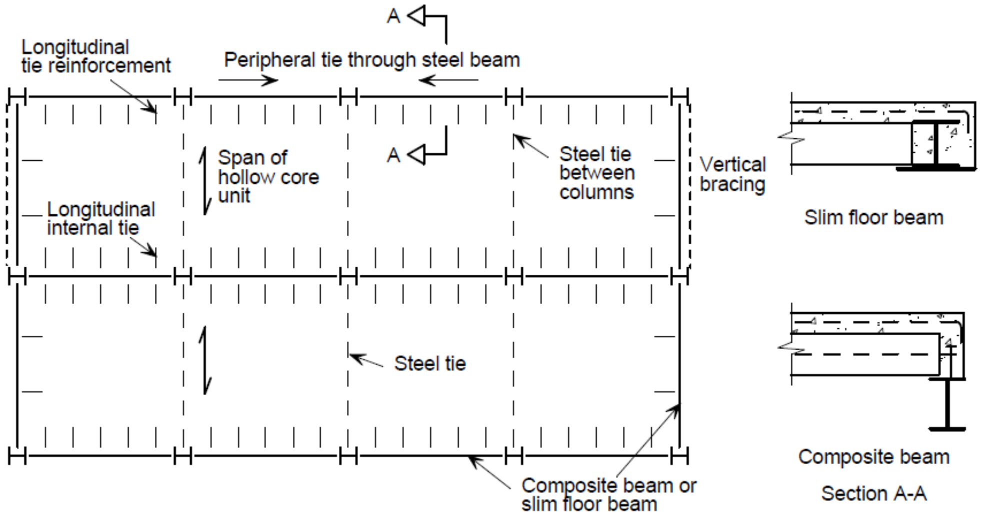


Figure 35. Tying action in the floor plate using precast slabs (after N 2040, 2020)

For collaborative composite slabs, no specific recommendations are available yet. Through tests recently performed at the Politehnica University Timisoara (Dinu et al., 2015), it has been demonstrated that a debonding between the composite slabs and the steel deck may occur when significant deformations develop which could limit the possibility of developing significant membrane forces in the composite slab. This requires further investigations to propose appropriate constructive details to avoid this debonding and so to effectively activate the slab in case of column loss scenario.

5.3.1.1.3 New method proposed by PT 2 of CEN TC250 Working Group 6

In (CEN/TC250/WG6, 2020), a new method developed by B. Izzuddin and presented as an alternative to the prescriptive tying method presently recommended in EN 1991-1-7 is proposed. This method allows for a better prediction of the tensile loads to be supported by the tying members in case of column loss scenario, accounting for variable levels of ductility, the floor typology and possible dynamic effects. The general formulation for the computation of the minimum tensile force to be supported is as follows:

$$T \geq \eta . \rho . \left(\frac{i_f}{\bar{\alpha}}\right) . P \tag{23}$$

where:

- T is the tensile load to be supported by the considered tying member;
- η is an amplification coefficient to account for possible dynamic effects;
- ρ is a reduction factor to account for different effects such as strain hardening of interaction between tensile load and bending;
- i_f is a tying force intensity factor depending of the system under consideration;
- $\bar{\alpha} = \frac{\alpha}{0.2}$ is a coefficient to account for the chord rotation capacity α (in rad) for different structural typologies;
- P is an equivalent load to account for the loads applied to the considered floor.

This formulation is presented as a "universal" one which can be used whatever the used materials and structural typology are. It requires an appropriate characterisation of the constitutive coefficients. In (CEN/TC250/WG6, 2020), different values of i_f and P are proposed for double-span beams, two-way floor tying and one-way floor tying subjected to different loading conditions. For the computation of α, it is clearly stated that the rotation capacity to be computed is not the one corresponding to the rotation ductility, i.e., the rotation during which the structural member is able to sustain its plastic resistance, but the one corresponding to the failure of the structural member. However, up to now, no easy-to-apply methods are available to predict such rotation capacity, in particular for steel and composite structures. Nevertheless, in case of partial-strength joints, the values of the rotation ductility predicted using the recommendation of Section 2.2.3 can be safely used for α. It is also stated that the proposed formulation is valid if a minimum level of rotation capacity α_{min} is available. Criteria for the definition of α_{min} values are proposed in (CEN/TC250/WG6, 2020).

For the dynamic amplification η, it is mentioned that, in absence of information, the most realistic value is to consider $\eta = 2$. This can be seen as a safe estimation of this coefficient. Formulations to compute more refined values for this coefficient are also proposed in (CEN/TC250/WG6, 2020).

In addition, this method is founded on assumptions regarding the behaviour of the surrounding structure and it is recommended to check the latter under tying forces and to check if it exhibits a sufficient stiffness under the action of the tying loads as the proposed formulation is based on the assumption that the horizontal stiffness at the extremities of the tying members is high. In (CEN/TC250/WG6, 2020), criteria to check if this stiffness is sufficient are provided. For structures in which it is possible to activate diaphragm effects, it can be assumed that these criteria are satisfied.

As can be observed through this brief description, this method requires the characterisation of different parameters and, in particular, the analysis of the surrounding structure. It is the reason why this method can be seen as a hybrid method combining prescriptive criteria and analytical approaches.

5.3.1.2 Vertical tying

Vertical tying can allow the redistribution of loads through the development of alternative load paths as illustrated in Figure 36. Vertical tying is mainly governed by the tensile capacity of the column splices. Therefore, the splices must be able to resist the tensile forces that can arise due to the loss of column support, in order to hang the above floors and redistribute the load to the rest of the undamaged structure.

In Eurocode 1 Part 1-7 (EN 1991-1-7, 2006), requirements for vertical ties are given:

- all the columns in the structure should be tied continuously from the foundation to the roof;
- the tie should be able to resist a tensile force corresponding to the largest design vertical permanent and variable reaction applied in normal design conditions to the column from any one storey.

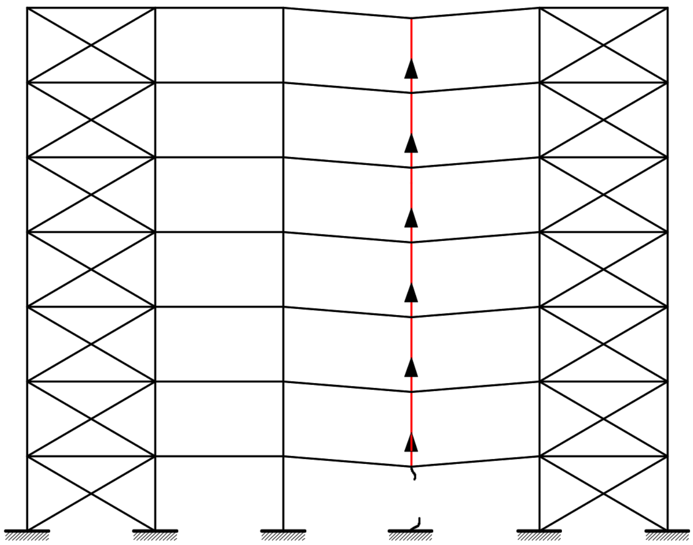


Figure 36. Alternative load paths developed through vertical tying

The check of column splices subjected to tensile loads is not explicitly covered in the Eurocodes. Rules are proposed in Annex A.1 for the characterisation of joints under tensile loads.

5.3.2 Analytical methods

In the present section, different analytical approaches will be proposed with different level of sophistication, from the simplest ones to the most advanced ones. The simplest approaches are founded on assumptions which allows a safe estimation of the structural response exposed to a column loss scenario in comparison to the most advanced ones, which allows for a more accurate prediction.

This section will first focus on the possible contribution from the slab. Then, simplified analytical methods will be proposed for different structural typologies. Finally, a more advanced analytical method will be briefly addressed.

But before tackling these subjects, an important preliminary remark must be made. In Sections 5.3.2.2 (simplified analytical methods for structures with pinned joints), 5.3.2.3 (simplified analytical methods for structures with partial-strength joints) and 5.3.2.4 (simplified analytical methods for structures with over-strength joints), the presence of concrete slabs acting as efficient diaphragms is assumed at each storey level. As a result, the indirectly affected part may be assumed as infinitely stiff in the horizontal direction, what induces an equal distribution of membrane forces in the storeys located above the lost column. In structures where this condition is not satisfied, the membrane forces will distribute amongst these storeys according to their relative lateral stiffnesses.

In such a situation, more advanced models are required and reference can be made to Section 5.3.2.5 (advanced analytical approach) or to Sections 5.3.3 and 5.3.4 (numerical approaches). Specific failure modes related to this variation of stiffness in the height of the indirectly affected part may then occur, which will have to be checked; they have been illustrated in Figure 32:

- the development of a global plastic mechanism in the indirectly affected part under the action of the membrane forces transferred by the DAP to the IAP of the structure;

- the buckling in compression of the upper beams of the DAP as a result of a possible progressive development, in the whole structure, of an arching effect induced by the sway deformability of the left part of the IAP of the structure.

Finally, it must be noted that the verification of the buckling resistance of the IAP columns adjacent to the lost column (see Figure 32) will have to be achieved in all cases, whatever the stiffness of the IAP. This check will be performed by assuming an individual overloading of these columns, further to the event, equal to one-half of $N_{lo,design}$ in 2D structures, and one fourth of $N_{lo,design}$ in 3D structures.

5.3.2.1 Contribution from the slab

As mentioned in Section 5.1, the slabs can play a key role in the way on how the structure will behave further to the loss of a column.

This event leads, for the slabs located above the lost column, to the loss of one of their vertical supports and therefore to a significant increase of their free span and to the development of large deflections.

The behaviour of reinforced concrete slabs undergoing large deflections is investigated since many years and models with different degrees of complexity have been proposed in the literature. Most of these ones are based on the preliminary application of the well-known first-order yield line theory proposed by Johansen (Hognestad, 1953). This theory requires first to select a failure plastic mechanism in the slab, and then, by applying the principle of virtual works, to compute the plastic load resistance as an upper bound solution. When the plastic load is reached, it is assumed that the cracks and the curvature of the slab are concentrated along yield lines (see examples in Figure 37). The blocks surrounded by these yield lines are assumed to remain elastic and planar, and rotate rigidly around the yield lines. The yield line pattern is affected by several parameters, such as the plastic moment capacities of the slab cross-sections, the support conditions, and the geometry of the slab. The models initially proposed for reinforced concrete slabs can be easily and safely extended to composite slabs by only considering the contribution of the reinforced concrete slabs located above the ribs, i.e., by neglecting the contributions from the steel sheet and the concrete inside the ribs. This procedure applies when the slabs are connected or not to the steel beams, but the presence of a connected slab may influence the yield-line pattern. In Figure 37, the left pattern may apply to both situations while the right one represents a possible yield line pattern only in the absence of connected beams.

By applying this theory to the slabs located above the lost column, a plastic resistant surface load may be derived which has to be compared to the applied surface load (for an accidental combination). If this plastic load is bigger than the applied one at each storey above the lost column, the slabs are able themselves to sustain the applied accidental loads and, therefore, the structure can be assumed as robust. If it is not the case, plastic mechanisms will form in the slabs and other structural contributions will have to be activated to survive the column loss scenario.

In this case, one first possibility is to activate membrane effects. If an internal column is lost, the method developed by Bailey (Bailey, 2001) can be used. In his work, Bailey investigated the load-bearing capacity of orthotropic laterally unrestrained slabs with only one layer of reinforcement, by referring to an equilibrium method and accounting for the membrane forces. By applying this method, the load carrying capacity of the slab can be evaluated. In the framework of the RobustImpact RFCS project (Kuhlmann et al., 2017), the effectiveness of the combined Johansen/Bailey method has been tested on different column loss scenarios. The results were compared with the outcomes of FE numerical model with a good agreement. As an alternative, numerical tools can also be used to predict the response of the concrete slabs, but it falls out of the scope of the present section dedicated to analytical approach. Reference can be made to Section 5.3.4 for this alternative.

When an external column is lost, the contribution coming from the development of plastic mechanism in a slab can be accounted for by considering the yielding lines illustrated in Figure 38. However, the possibility to activate membrane forces is very limited and can therefore be neglected.

Possibilities to activate other structural "robustness" contributions than the "yield mechanism" and "membrane effects" ones in the slab strongly depends on the configuration of the floor and more globally of the structure. These possibilities will be addressed in the following subsections for different structural typologies.

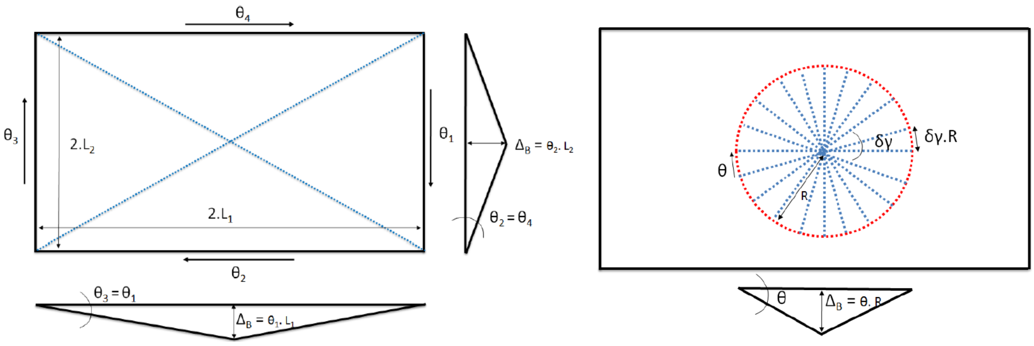


Figure 37. Examples of failure mechanisms for an internal column loss (Lemaire 2010)

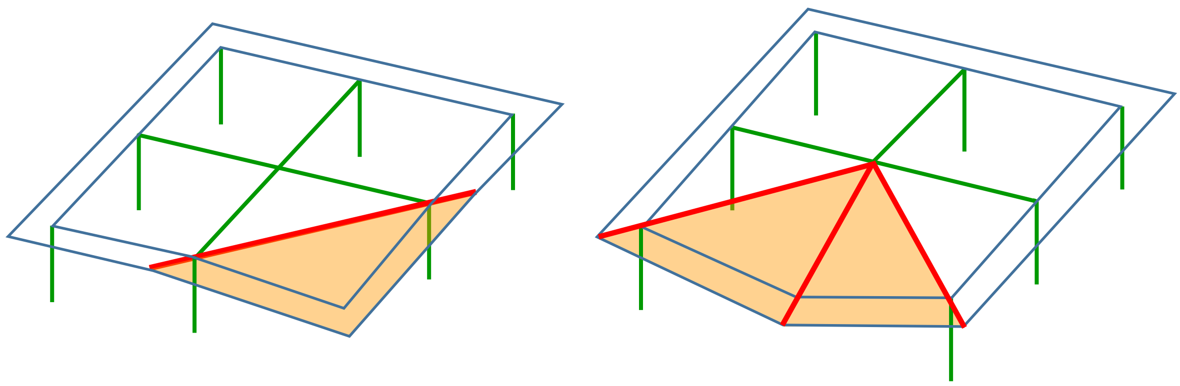


Figure 38. Examples of failure mechanisms for an external column loss

5.3.2.2 Simplified analytical methods for structures with simple joints

If the slabs are not able to sustain the loads associated to the column loss scenario (see Section 5.3.2.1), it remains to check the possible contribution from the supporting steel structure (see Section 5.3). As simple joints act at the ends of the beams, no "plastic mechanism" robustness contribution may be expected, and the possibility to develop beam arching effects is also quite questionable. But on the other hand, large displacements may occur in the structural system leading to potential high membrane forces

However, the contribution resulting from the activation of these membrane forces cannot be cumulated with the contribution from the slab. Indeed, as above-mentioned, the activation of the membrane forces in the beams is only possible for large displacements which are not compatible with the deformation capacity of the slab. Accordingly, the final objective here is so to see if an equilibrium between the so-activated membrane forces in the beams only and the load associated to the column loss can be found as explained here after.

The development of the membrane forces strongly depends on the stiffness $K_{H,t}$ of the indirectly affected part (see Section 5.3). If this stiffness is very small, negligible membrane forces will develop in the directly affected part and so the structure will be considered as "not robust". On the other hand, if this stiffness is significant, large membrane forces will develop and a new state of equilibrium may be found in the deformed shape.

If the slab has been initially designed to work as a diaphragm, it may be assumed to be rigid in its plane. As a consequence, the value of the $K_{H,t}$ stiffness of the indirectly affected part introduced in Section 5.3 may be taken as infinite, the extremities of the directly affected part being totally fixed in the horizontal direction. Indeed, when these beam extremities intent to move horizontally, the structure comes into direct contact with the slabs at the different storeys; these contacts prevent these extremities from moving by activating the slab in compression in its plan. Based on this assumption, the response of the structure further to a column loss may be easily predicted using the static and kinematic theorems, i.e., using the equations of equilibrium and expressing the compatibility of displacement.

An example is given for the 2D frames with simple joints illustrated in Figure 39 in which the concrete slabs acting as diaphragms are placed at each floor level. For this frame, the membrane forces T_{beam} developing in the beams of the directly affected part may be predicted referring to the sub-system illustrated in Figure 39. Because of the presence of the slabs at each storey (infinite value of $K_{H,t}$), the same tension force develops in all the beams (assumed to be the same at each floor level) of the directly affected part. Accordingly, each double-beam will resist in the same way to the force $N_{lo,design}$, the axial load which is initially present in the column before the event and which can be evaluated under the accidental load combination (EN 1990, 2002). Consequently, the behaviour of the frame can finally be studied using the sub-system of Figure 39 submitted to a force $N_{lo,design}/n_{st}$, n_{st} being the number of storeys activated in the directly affected part.

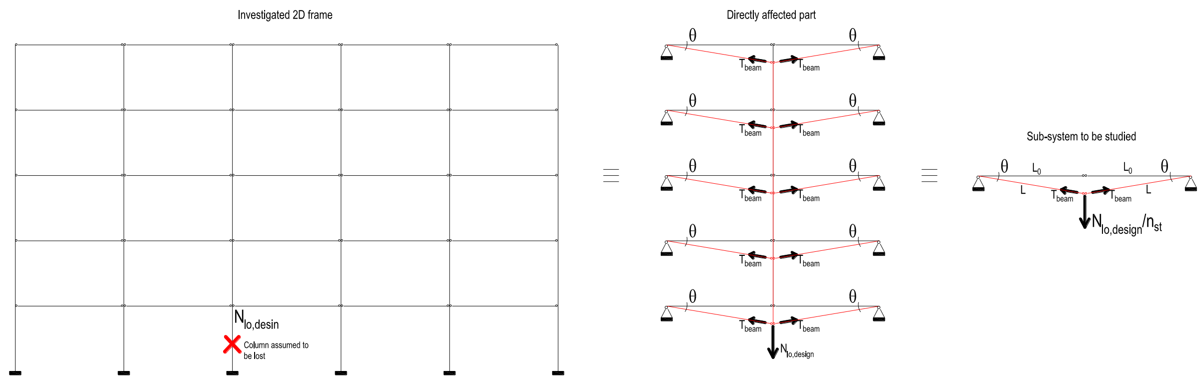


Figure 39. Simplified analytical approach – from a 2D frame to a sub-system model

For the so-defined sub-system, the following equations can be written based on equilibrium and geometrical considerations:

$$\frac{N_{lo,design}}{n_{st}} = 2.T_{beam}.\sin\theta \qquad (24)$$

$$L = L_0/\cos\theta \qquad (25)$$

where L is the length of the individual beams in the deformed system and L_0 is their initial length. In the elastic range, the elongation of the beams is related to the tension force they sustain:

$$\Delta L = L - L_0 = T_{beam} \cdot \frac{L_0}{E.A} \tag{26}$$

where E is the Young modulus of the beam material and A is the beam cross-section area.

Replacing L by $L_0/\cos\theta$ in this equation, a system of two equations with two unknowns, T_{beam} and θ, is obtained:

$$\frac{N_{lo,design}}{n_{st}} = 2.T_{beam}.\sin\theta \tag{27}$$

$$T_{beam} = \frac{1-\cos\theta}{\cos\theta}.E.A \tag{28}$$

By solving this system of equation, it is possible to predict (i) the tensile load T_{beam} to be supported by the beams and the joints at the extremities, and (ii) the rotation capacity demand θ for the simple joints.

As previously mentioned, it is assumed that the beams of the directly affected part are remaining in the elastic range; accordingly, it is required to check if these beams subjected to bending moments (coming from the gravity load) and to the tensile load T_{beam} remains elastic. For the check of the simple joints both in terms of resistance (under T_{beam}) and rotation capacity (θ), reference can be made to Sections A.5.1 and A.2 respectively.

This model can be easily extended to 3D structures as reflected in Section A.7.

5.3.2.3 Simplified analytical methods for structures with partial-strength joints

If partial-strength joints are used at the extremities of the beams, the column loss scenario will result first to the development of a plastic mechanism in the directly affected part (see Section 5.1) with plastic hinges forming at the level of the partial-strength joints. The plastic load associated to the formation of a plastic mechanism in a beam with partial-strength joint (Figure 40) is obtained through the following equation (assuming the joints at the beam extremities are the same):

$$N_{pl,i} = \frac{2.M_{Rd,i}^{-} + 2.M_{Rd,i}^{+}}{L} \tag{29}$$

where $M_{Rd,i}^{-}$ is the design plastic resistance of the partial-strength joint at the extremities of beam i under hogging moment while $M_{Rd,i}^{+}$ is the one under sagging moment.

This formula can be used for the beams of each storey above the lost column and the sum of the so-obtained $N_{pl,i}$ values corresponds to the plastic load N_{pl} required to form a plastic mechanism in the directly affected part:

$$N_{pl} = \sum_i N_{pl,i} \tag{30}$$

If the so-obtained value of N_{pl} is greater than $N_{lo,design}$ (see Section 5.3.2.2), then the beams of the directly affected part can sustain the column loss and the structure can be assumed as robust.

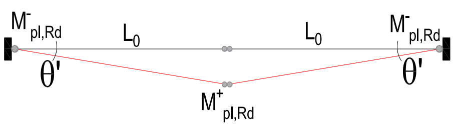


Figure 40. Beam plastic mechanism developing in a beam with partial-strength joints

If not, the contribution from the beams (Equation (30)) can be cumulated with the contribution resulting from the development of a yield plastic mechanism in the slabs (see Section 5.3.2.1). This one has here to be evaluated by applying a transverse concentrated load on the slab at the level of the lost column. For this specific loading condition, a concentrated load $N_{pl,slab,i}$ associated to the formation of a plastic mechanism in the slab at each storey i may be computed using the Johansen theory (see Section 5.3.2.1). Finally, the plastic load $N_{pl,slab}$ corresponding to the formation of a plastic mechanism in all the slabs of the directly affected part writes:

$$N_{pl,slab} = \sum_{i} N_{pl,slab,i} \tag{31}$$

If $N_{pl} + N_{pl,slab}$ is greater than $N_{lo,design}$ (see Section 5.3.2.2), then the beams and the slabs of the directly affected part can sustain the column loss and the structure can be assumed as robust.

If not, it is required to look for other possible contributions. The activation of the latter strongly depends on the nature of the failure mode at the level of the partial-strength joints as explained here below.

If the failure mode is associated to components in tension, in bending or in shear, this means that the components in compression (column web in compression or beam flange and web in compression) have not reached their plastic resistance yet. In such conditions, an arching effect can be mobilised in the beams of the directly affected part, as schematically illustrated in Figure 41, as soon as the plastic mechanism is formed. This arching effect (i) prevents the apparition of significant vertical displacements within the directly affected part and (ii) allows for the mobilisation of extra resisting forces in the system. This arching effect vanishes when the resistance of the row in compression at one of the extremities of the rod representing the arch members ($F_{rd,c}$ – see Figure 41 in which it is assumed that the joints at the extremities of the beams are the same) is reached.

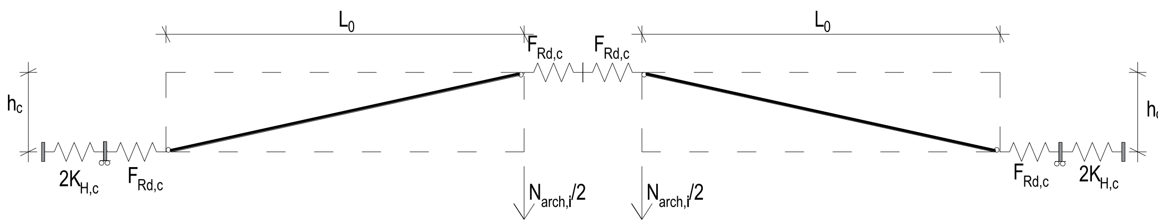


Figure 41. Schematic view of the arching effect within a beam of the directly affected part

To predict the extra forces which can be mobilised through this arc effect, the following procedure can be applied in which it is assumed that the stiffness of the indirectly affected part $K_{H,c}$ is infinite (see Figure 41). The proposed procedure can be adapted to other situations but will require more refined analytical models described in Section 5.3.2.5.

The first step consists in evaluating the vertical displacement $\Delta_{pl,i}$ of the beams at each storey level i, when the beam mechanism has formed. The corresponding value, obtained by means of a second order analysis, is equal to:

$$\Delta_{pl,i} = \frac{N_{pl,i}.(2.L_0)^3}{192.E.I_{y,i}} + L_0.Tan\left(\frac{M_{Rd,i}}{\frac{S_{j,ini,i}}{\eta}}\right) \tag{32}$$

where E is the young modulus of the steel, $I_{y,i}$ is the moment of inertia of the beams, $M_{Rd,I}$ is the bending resistance of the joint at the beam extremities, $S_{j,ini,i}$ is the initial stiffness of the joint, all three values at level i. η is the stiffness modification coefficient as defined in Table 5.2 of (EN 1993-1-8 2005).

This equation is valid for steel beams with joints exhibiting the same stiffness and resistance under sagging and hogging moment at each extremity, but it can be adapted to other configurations.

When the plastic mechanism forms in the beams at level i, the horizontal springs from Figure 41, representing the components in compression, are already subjected to a force F_t (corresponding to the sum of the tensile loads in the rows in tension, for sake of horizontal equilibrium within the joints). Accordingly, these springs exhibit a shortening equal to:

$$\delta_{c,el} = \frac{F_t}{k_{eff,c}.E} \tag{33}$$

where $k_{eff,c}$ is the effective stiffness coefficient of the row in compression computed according to Section 6.3.3.1 of (EN 1993-1-8 2005).

The position of the arch rod when the plastic mechanism forms in the beam is illustrated in orange in Figure 42. The length of the arch rod L_D at that moment is equal to:

$$L_D = \sqrt{\left(L_0 + 2\delta_{c,el}\right)^2 + \left(h_c - \Delta_{pl,i}\right)^2} \tag{34}$$

The resistance of the arching effect is reached when the resistance of the joint row in compression $F_{Rd,c}$ is reached, which corresponds to a deformation at the level of the joint row in compression equal to:

$$\delta_{c,pl} = \frac{F_{Rd,c}}{k_{eff,c}.E} \tag{35}$$

and to an inclination of the arch rod θ (see Figure 42) equal to:

$$\theta_r = Acos\left(\frac{L_0 + 2.\delta_{c,pl} + \delta_K}{L_D}\right) = Acos\left(\frac{L_0 + 2.\delta_{c,pl}}{L_D}\right) \tag{36}$$

where $\delta_K = \frac{F_{Rd,c} - F_t}{K_{H,c}}$ is the horizontal displacement of the indirectly affected part; it is here equal to 0 as $K_{H,c}$ is assumed to be infinite. In this equation, it is reasonably assumed that the length of the arch rod L_D remains constant. It has to be highlighted that the horizontal spring reflecting the behaviour of the indirectly affected part is only activated when the plastic mechanism is formed, i.e., when the arching effect develops. Indeed, prior to the development of the plastic mechanism, no horizontal forces are reported to this part as the beams are working in bending only.

Knowing this value of θ_r, the contribution coming from the arching effect $N_{Arch,i}$ is finally obtained by expressing the horizontal equilibrium of the system:

$$N_{Arch,i} = 2.Tan(\theta_r).\left(F_{Rd,c} - F_t\right) \tag{37}$$

Obviously, if the resistance of the joint at extremities of the beams is associated to a component in compression, F_t is equal to $F_{Rd,c}$ (for sake of equilibrium) and so, no arching effect can be mobilised at level i ($N_{arch,i}$ = 0).

This contribution can only be accounted for if the arch rod subjected to a compression equal to $N_{Arch,i}/cos(\theta_r)$ is able to sustain this force. The resistance of the arch rod can be reasonably assumed as equal to the resistance of the beam in compression $N_{b,Rd}$. If the resistance of the rod is reached, $N_{Arch,i}$ can so be safely assumed as equal to $N_{b,Rd}$. $cos(\theta_r)$ The same applies for the indirectly affected part which has to be able to sustain an applied horizontal load equal to $\left(F_{Rd,c} - F_t\right)$.

Finally, the contribution of this arching effect N_{Arch} for the directly affected part is equal to:

$$N_{Arch} = \sum_i N_{Arch,i} \tag{38}$$

This contribution can be cumulated to the ones resulting from the beam and slab plastic mechanisms as the activation of this arching effect required limited deformation capacities.

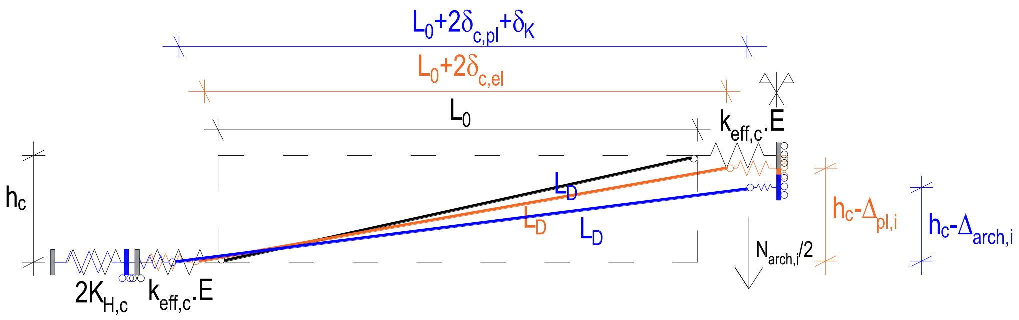


Figure 42. Positions of the arch rod during the column loss

Accordingly, if N_{pl} + $N_{pl,slab}$ + N_{Arch} is greater than $N_{lo,design}$ (see Section 5.3.2.2), the structure can be assumed as robust.

If it is not the case, significant vertical displacements of the directly affected part will develop as soon as this arching effect will be overcome. With the apparition of these vertical displacement, the development of membrane forces within the directly affected part can be foreseen but this new contribution $N_{membrane}$ cannot be cumulated with both the contributions from the arching effect (which vanishes after the mechanism has formed) and from the slab plastic mechanism (which vanishes when significant deformations are reached, because of its reduced deformation capacity). Accordingly, this contribution is of interest if:

$$N_{pl} + N_{m,embrane} > N_{lo,design} > N_{pl} + N_{pl,slab} + N_{Arch} \tag{39}$$

The prediction of the contribution $N_{membrane}$ requires to perform second order analysis. In addition, one has to account for the M-N interaction (see Section 2.2.1) in the partial-strength joints (see A.1), what requires the adoption of advanced design methods. Reference can be made to Sections 5.3.2.5, 5.3.3 or 5.3.4 where analytical and numerical advanced design methods are also proposed. However, it has to be reminded that the development of large displacements in the system requires significant deformation capacities at the level of the partial-strength joints (subjected to M-N interaction). Unfortunately, in such a situation, a sufficient deformation capacity cannot be exhibited by most of the classical joints.

5.3.2.4 Simplified methods for structures with over-strength joints

In such structures, when beam plastic mechanism forms, the hinges develop in the beams and not in the joints. Accordingly, the formulas to account for this specific robustness contribution (addressed in the previous section) becomes:

$$N_{pl,i} = \frac{2.M_{pl,Rd,i}^{-} + 2.M_{pl,Rd,i}^{+}}{L} \text{ and } N_{pl} = \sum_i N_{pl,i} \tag{40}$$

where $M_{pl,Rd,i}^{-}$ is the design plastic resistance of the beam sections at level *i* under hogging moment and $M_{pl,Rd,i}^{+}$ - the one under sagging moment.

If the so-obtained value of N_{pl} is greater than $N_{lo,design}$ (see Section 5.3.2.2), then the beams of the directly affected part can sustain the column loss and the structure can be assumed as robust.

If it is not the case, the possible "plastic mechanism" contribution from the slab can be accounted for as described in Section 5.3.2.3. If N_{pl} + $N_{pl,slab}$ is greater than $N_{lo,design}$ (see Section 5.3.2.2), then the beams and the slabs of the directly affected part can sustain the column loss and the structure can be assumed as robust.

If it is not sufficient, the "beam arching effect" contribution, described in the previous section, cannot be activated here. Indeed, as the yielding zones are developing within the beam cross-sections, both parts of the cross-sections in the plastic hinges, respectively in tension and in compression, are yielded and so the resistance associated to the arching effect is equal to zero.

Accordingly, the only additional contribution which can be accounted for is the one associated to the development of membrane effects in the beams belonging to the directly affected part but, as reported in the previous section, this additional contribution cannot be cumulated with the contribution coming from the slab plastic mechanism as the request in terms of deformation capacity at the level of the yielding lines would be too high. As for structures with partial-strength joints, the prediction of the contribution $N_{membrane}$ requires the adoption of advanced design methods. Reference can be made to Sections 5.3.2.5, 5.3.3 or 5.3.4.

5.3.2.5 Advanced analytical approach

A more general and detailed analytical approach has been developed by (Huvelle et al., 2015). This model allows the prediction of the structural response of a 2D frame with simple, partial-strength or over-strength joints when membrane forces developed in the directly affected part further to the formation of beam plastic mechanisms in case of partial-strength or over-strength joints acting at the extremities of the beams.

The model is founded on the definition of a substructure and on its characterisation, under the following assumptions:

- a progressive (static) column loss is assumed;
- the plastic hinges can develop in the beam cross-sections or in the beam-to-column joints;
- all columns are made of a unique cross-section type, and it is the same for the beams;
- only the loss of internal columns (i.e., columns which are not at the corners) is considered;
- no yielding develops in the rest of the structure, called the indirectly affected part (i.e., its behaviour is assumed to be infinitely elastic).

Through the proposed analytical approach, a set of N equations with N unknows is obtained and can be solved using mathematical solvers. The main result of this approach is the prediction of the evolution of the vertical displacement of the directly affected part vs. the load in the failing column (identified as $N_{membrane}$ in the previous sections).

Details about this advanced analytical approach are provided in Annex A.8. In this annex, it is also explained how this model can be extended to predict the response of 3D framed structures.

5.3.3 Simplified numerical approaches

5.3.3.1 Introduction

In this section, a simplified approach is presented for the assessment of the resistance of multi-storey steel framed building structures to progressive/disproportionate collapse using sudden column loss as a design scenario. The method offers a quantitative framework that takes into account ductility, redundancy, energy absorption and dynamic effects. The simplicity of the framework allows it to be directly applied in design practice. Additionally, it enables the quantification of structural robustness for sudden column loss scenarios, unlike the prescriptive methods discussed in Section 5.3.1.

Generally, the objective of robustness design is making sure that any local damage resulting from unforeseen extreme events does not cause disproportionate collapse. Sudden column loss, as illustrated in Figure 43 (Izzuddin et al., 2008), represents an appropriate design scenario, which includes the dynamic effects that can be associated with the failure of vertical members under extreme events, such as blast and impact; however, it is event-independent. This design scenario is not necessarily identical in dynamic effect to column damage resulting from blast or impact. However, it can provide an upper bound on the structural deformation demands which is approached in the limit as the level of blast loading on the affected column becomes very large (Gudmundsson and Izzuddin, 2010; Izzuddin, 2010). In addition, it can capture the influence of column failure occurring over a relatively short duration to the response time of the structure. Therefore, it can be considered as a standard dynamic test of structural robustness, and may be applied to various other extreme dynamic events via calibrated design factors.

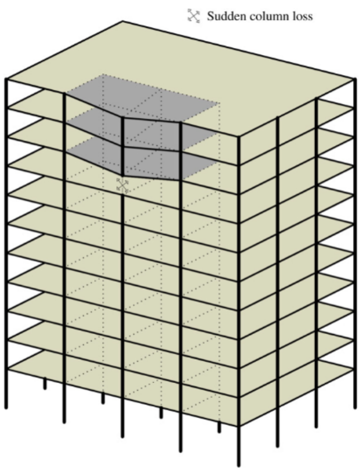


Figure 43. Multi-storey framed structure under sudden column loss scenario (Izzuddin et al., 2008)

5.3.3.2 Robustness limit state

For a sudden column loss scenario, a definition of a robustness limit state is needed, beyond which local damage progresses to disproportionate collapse at the global structural level. The robustness limit state shall be based on preventing the collapse of the above floors in the event of sudden column loss and ensuring that the surrounding columns can resist the redistributed loads. The collapse of only one floor can lead to onerous demands on the lower impacted floors that also have to sustain the debris loading which will in turn increase the vulnerability of the structure to progressive/disproportionate collapse. Additionally, as the robustness limit state deals with rare and extreme accidental events, it is acceptable/reasonable to allow relatively large deformations in the upper floor. This would allow mobilisation of the beneficial effects of compressive arching and tensile catenary/membrane actions that are not normally utilised under the typical loading conditions as to attain better design economy. Otherwise, designing the floors to resist gravity loading in the absence of the column support using conventional strength-based methods would lead to structures that are excessively over-designed for normal loading conditions.

In the current approach, the robustness limit state for sudden column loss is defined in terms of the maximum dynamic deformation in the upper floors exceeding the ductility limit. For steel-framed structures having simple or partial-strength joints, sudden column loss can lead to a notable concentration of deformations in the joints within the above floors. Such limit is in turn based on first component failure, such that the ductility demand at the maximum dynamic response is equal to the ductility supply in one of the joints. This can also be generalised to account for the successive failures of more than one component. Consequently, the most general case to a robustness limit state can be defined in terms of the level of gravity loading that exceeds the maximum pseudo-static resistance of the floor system prior to complete collapse. This forms the underlying principle of the proposed ductility-centred approach, as discussed in the following sections.

5.3.3.3 Multi-level ductility-centred assessment framework

The response of a multi-storey structure subjected to a sudden column loss is dynamic and highly nonlinear involving considerable material and geometric nonlinearities. The limit state discussed in 5.3.3.2 is evaluated by determining the maximum dynamic response of the structure under gravity loading after sudden column removal then evaluating if the ductility supply provided by the joint is sufficient to accommodate the resulting demands. Nonlinear dynamic finite element analysis is considered to be the most accurate method to determine the maximum demands imposed on the joints. However, it is relatively complex and requires special expertise that may not be readily available for typical design practices. Alternatively, a more practical approach is presented that requires nonlinear static rather than dynamic analysis with the dynamic effects incorporated in a simplified and accurate way.

The proposed framework is composed of three main stages:

1. Nonlinear static response of the damaged structure under gravity loading.
2. Simplified dynamic assessment to determine the maximum dynamic response in the event of a sudden column loss.
3. Ductility assessment of the connections.

This proposed framework is based on the limit state discussed where the main design objective is to prevent the collapse of the upper floors in the event of a sudden column loss. The proposed framework also offers an important feature which can be applied at various levels of structural idealisation depending on the regularity of the structure and the applied loading as discussed in the following section.

5.3.3.4 Structural idealisation

The proposed framework can be applied on the overall global structural level, as shown in Figure 43, and at different sub-structural levels as well, as shown in Figure 44 (Izzuddin et al., 2008). The level of structural idealisation is determined according to the required modelling detail and whether the structural model reduction is feasible or not, which is largely dependent on the regularity of the building with regard to the structural and loading arrangements. The first level of structural idealisation/reduction consists of the affected bay only in a multi-storey building as shown in Figure 44a. In that level, appropriate boundary conditions must be assigned to represent the interaction of the bay under consideration with the surrounding structure. If the surrounding columns are assured to be able to withstand the redistributed load, only the floors above the lost column can be considered where the deformation is concentrated resulting in further model reduction as shown in Figure 44b (zone called "DAP – directly affected part" in the previous sections). If the affected floors are similar in structure, loading and IAP restraints (see Section 5.3), a single floor system can be considered as shown in Figure 44c, where the axial force in the columns directly above the lost column can be assumed to be negligible. Lastly, individual steel/composite beams can be considered, as shown in Figure 44d, subjected to appropriate proportions of the gravity loading while ignoring the planar effects within the floor slab (i.e., by disregarding the possible own resistance of the slabs, in contrast to what is done, when justified, in Section 5.3.2).

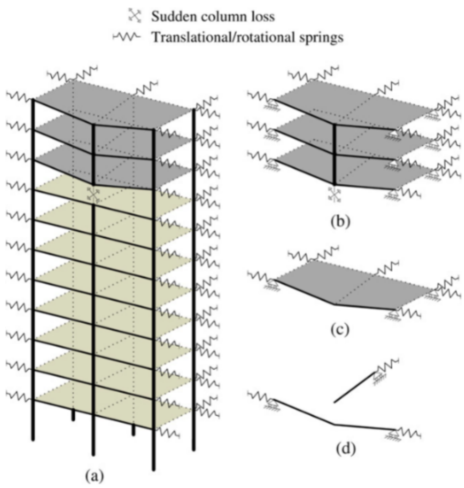


Figure 44. Structural idealisation levels for progressive/disproportionate collapse assessment. (a) Affected bay, (b) Floor(s) above lost column, (c) Single floor above lost column and (d) Individual steel/composite beam above lost column (Izzuddin et al,. 2008)

5.3.3.5 Nonlinear static response

The effect of the sudden removal of a column can be regarded similar to the sudden application of the gravity load (P_o) for a given structure, as shown in Figure 45 (Izzuddin et al., 2008), especially when the amount of deformations sustained by the structure is significant. The sudden application of gravity loading is associated with dynamic effects, where all the ductility demands for all the deformation states leading to the maximum dynamic response should be met with a sufficient ductility supply to prevent failure.

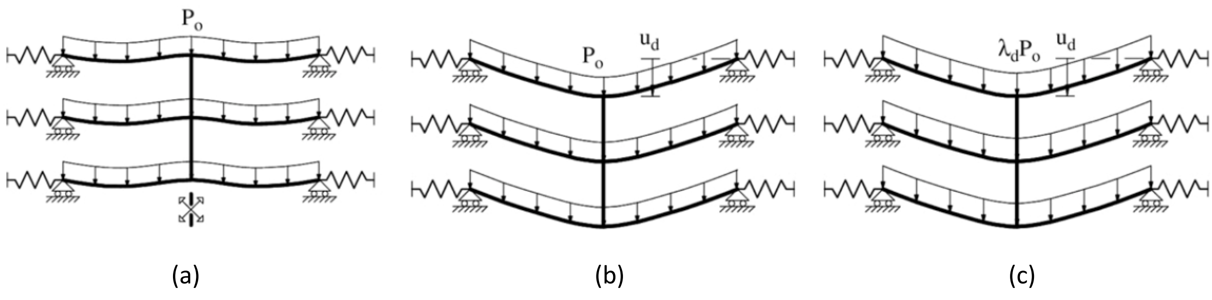
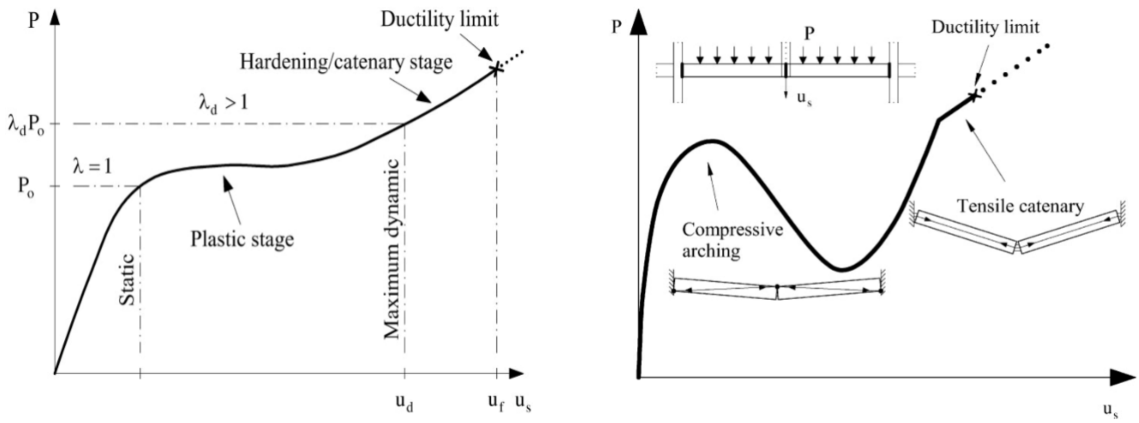


Figure 45. Modelling of sudden column removal. (a) Sudden column loss, (b) Maximum dynamic response and (c) Amplified static load (Izzuddin et al., 2008)

As illustrated in Figure 45c, the proposed framework enables the maximum dynamic response to be estimated accurately from the nonlinear static response under amplified gravity loading ($\lambda_d P_o$) without the need of performing any complex dynamic nonlinear analysis. In this respect, the nonlinear static response of the structure is obtained with the exclusion of the damaged column such that the gravity loading is varied using a scale factor (λ) with $P = \lambda P_o$ and plotted against the static vertical displacement (u_s) at the location of the damage column. A typical nonlinear static response can be shown in Figure 46 where such response is the basis for the determination of the maximum dynamic response (u_d) as will be illustrated in the following section. As illustrated in Figure 46 (Izzuddin et al., 2008), for realistically designed structures, the plastic bending resistance is not sufficient to sustain the amplified static loading ($\lambda_d > 1$) in the event of sudden column loss and further reliance on hardening and/or catenary action is needed. It is also clear that the maximum dynamic displacement (u_d) should be below a certain limit referred to here as the "ductility limit" at which first failure in one of the joints occurs. In addition, some systems may undergo a softening static response as a result of compressive arching action as (Izzuddin, 2010).



(a) (b)

Figure 46. Nonlinear static response under proportional gravity loading (P = λPo) (Izzuddin, 2010)

5.3.3.5.1 Nonlinear static response of individual beams

5.3.3.5.1.1 Detailed modelling

Detailed finite element modelling can be used to determine the nonlinear static response at the different levels of structural idealisation. On the beam level, elasto-plastic beam-column elements can be employed accounting for material and geometric nonlinearity. In addition, the composite action between the concrete slab and the steel beam can be modelled taking into account the effect of partial/full shear connectors. The nonlinear joint behaviour can be considered using a component-based mechanical model based on the principles proposed in EN 1993-1-8 (2005) and EN 1994-1-1 (2004). Developments on component-based modelling of steel and composite bolted joints have been recently proposed which take into account the combined effect of bending moments and axial forces (M-N interaction) and consider the so-called "group effects" between successive bolt-rows (Demonceau et al., 2019; Alhasawi et al., 2017) as illustrated in other parts of this document. More guidance can be found in EN 1993-1-14 (2020) which gives some rules for the use of numerical methods in the design of steel structures.

5.3.3.5.1.2 Simplified modelling

The nonlinear static response for individual steel or composite beams can be obtained using analytical expressions that account for the explicit modelling of the joint behaviour and employ traditional structural analysis principles without the need for the detailed and complex finite element modelling as illustrated in Section 5.3.2.5. Such approach would be more practical in common design practices than nonlinear finite element models allowing the easy shift from the prescriptive rules and the associated limitations to a more accurate approach in assessing structural robustness. This approach has been evoked in Section 5.3.2.5 and details are provided in Annex A.8.

5.3.3.5.2 Simplified assembly of the nonlinear static response for a single floor

Simplified modelling can be utilised to determine the nonlinear static response of a single floor system though the assembly of the responses of individual beams in a grillage approximation ignoring the membrane effects of the slab. As shown in Figure 47, for a dominant deformation mode, the overall system response of the single floor system (P, u_s) can be assembled from that of the individual beams (P_i, $u_{s,i}$) as:

$$P = \frac{1}{\alpha}\sum_{i} \alpha_i \beta_i P_i \tag{41}$$

where β_i is a compatibility factor relating the individual beam displacement to the floor reference displacement ($u_{s,i} = \beta_i u_s$), as illustrated in Figure 47, α_i is a non-dimensional work-related factor that depends on the assumed load distribution on the beam and may depend on the incremental deformation mode at the current level of loading (i.e., 0.5 for uniformly distributed load and 1 for point load), and α is also work-related factor that depends on the gravity load distribution on the beam (i.e. 0.25 for uniformly distributed load).

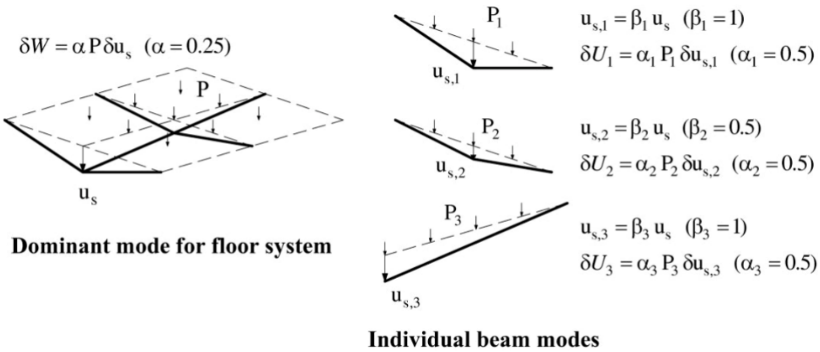


Figure 47. Grillage approximation of a single floor system

5.3.3.5.3 Simplified assembly of the nonlinear static response for multiple floors

Similarly, simplified modelling can be utilised to determine the nonlinear static response of multiple floors system above the damaged column though the assembly of the responses of individual floors. Assuming an SDOF deformation mode as shown in Figure 48 (Izzuddin, 2010) in which the floor displacement ($u_{s,j}$), measured along the failed column line, is constant for all floors, the overall response from individual floors can be expressed as:

$$P = \frac{1}{\alpha}\sum_{j} \alpha_j P_j \tag{42}$$

where α_j is the work-related factor for floor (*j*) (i.e., 0.25 for uniformly distributed load). While α is the overall work-related factor for the whole system (i.e., 0.25 for uniformly distributed load).

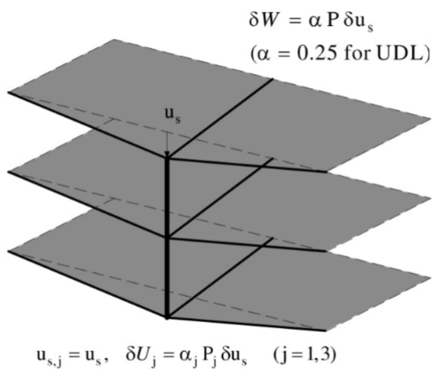


Figure 48. Simplified model for multiple floor system (Izzuddin, 2010)

Lastly, it is worth mentioning that detailed modelling can be applied at bay, multiple floors and single floor levels, where shell elements can be utilised accounting for material and geometric nonlinearity coupled to the beam elements and can capture the 2D membrane effects within the floor slab. This would be more accurate than the case of simplified modelling of single floors using a grillage approximation which inherently cannot account for the membrane floor action. Additionally, detailed models can be used in combination with simplified models where detailed modelling can be applied on the beam level and the nonlinear static response at the higher levels of structural idealisation can then be assembled using simplified modelling.

5.3.3.6 Simplified dynamic assessment

In the event of a sudden column loss, the typical response of a building structure is highly nonlinear and dynamic, therefore, the maximum dynamic response of the structure should be considered when assessing the resulting ductility demands. In this framework, the maximum dynamic response is determined through a simplified approach, as illustrated in Section 5.3.5, without the need of any complex nonlinear dynamic analysis which is not practical for typical/common design situations. The proposed approach is more accurate than the traditional dynamic amplification factor approach with the amplification factor depending on both the level of gravity loading and the nature of the nonlinear response thus lacking generality for common forms of nonlinear static response (Izzuddin, 2010).

5.3.3.7 Ductility assessment

The last stage of the proposed assessment framework is to compare the maximum dynamic displacement (u_d) obtained from the pseudo-static response at ($P = P_o$) with the ductility limit (u_f) to evaluate the limit state as shown in Figure 49. The ductility limit (u_f) is determined as the minimum value of (u_d) such that the deformation demand exceeds the ductility supply in any of the joints as discussed in other sections of this document. Alternatively, the limit state can be established by comparing P_o to the pseudo-static capacity (P_f), where P_f is defined as:

$$P_f = \max(P_n) \qquad \text{for} \qquad 0 \le u_{d,n} \le u_f \tag{43}$$

P_f would normally corresponds to u_f on the pseudo-static response curve, however, this wouldn't be the case if the response undergoes a softening behaviour due to compressive arching.

In the case of using simplified modelling at the system assessment level where the system response is obtained from simplified assembly of lower-level models, the displacements of the sub-systems can be determined from u_d using the appropriate compatibility conditions. The deformations experienced by the joints can then be determined for the displacements at the lowest level of the considered sub-system which would be either represented by detailed beam/floor models or by simplified beam models. Both the rotational and axial joint deformations must be considered, particularly when sufficient axial restraint is present that can lead to the development of catenary action. The ductility demands in the different components of the joint can then be obtained from the total joint deformations and compared to the ductility supply of the different components. It is important to note here that the system limit state is defined by the failure of a single joint such that the ductility demand exceeds the ductility supply in one or more of the joint components. However, if the failure of a single joint would not lead to a system failure in the presence of sufficient residual redundancy and ductility, this limit state can then be re-established for the system with the exclusion of the failed joint and any affected sub-systems beyond the associated ductility limit.

5.3.3.8 Assessment of floor systems subject to failed floor impact

The collapse of only one floor can lead to onerous demands on the lower impacted floors that also have to sustain the debris loading which will in turn increase the vulnerability of the structure to progressive/disproportionate collapse. However, under specific circumstances, it may be possible for the lower part of the structure to arrest impact and prevent progressive collapse. The factors that primarily influence such possibility include: (i) the number of failed floors above the level under consideration, (ii) the reduction in kinetic energy through energy absorption within the failed floors as well as energy loss upon impact, and (iii) the ability of the lower structural floor system to sustain the additional load from debris, accounting for the associated dynamic effects. Vlassis et al. (2007; 2009) proposed a design-oriented methodology for the assessment of progressive collapse resistance of floor systems in multi-storey buildings subject to impact from one upper failed floor. The proposed method can also be generalised to deal with the initial failure of more than one floor. The underlying basis of the proposed framework is that the ability of the lower floor to arrest the falling floor mainly depends on the amount of kinetic energy that is transmitted from the upper floor during impact. Similar to the simplified assessment procedure discussed above for multi-storey buildings under sudden column loss scenarios discussed above, the proposed approach uses the nonlinear static response of the impacted floor along with an energy balance approach to estimate the maximum dynamic deformation demands without the need for detailed nonlinear dynamic analysis. The study demonstrates the extremely onerous conditions imposed on the impacted floor that can result in an increased vulnerability to progressive collapse for structures of this type. Importantly, the likelihood of shear failure modes in addition to inadequate ductility supply under combined bending/axial actions is identified, thus establishing the need for further research work on the dynamic shear capacity of various joint types subject to extreme events.

5.3.4 Full numerical approach

In recent years, the increased computational capacities and the availability of advanced numerical programs (FEM, AEM, DEM) able to manage most of the phenomena characterising the building response in accidental loading conditions opened the way to design solutions based on a full numerical approach. The effectiveness of this approach, which is nowadays commonly used, strongly depends on the ability of the designer to identify and model the key factors affecting the structural response. In this framework, great attention should be paid to phenomena associated with energy dissipation due to the activation of local plasticity, such as plastic hinges and yield lines, and failures associated with the constitutive behavioural laws adopted for the materials.

Different degree of accuracy can be used when modelling the materials, ranging from the simplest ones, i.e., the linear elastic models, to the more complex non-linear ones, also incorporating strength and stiffness degradation. The linear elastic laws can be used in elastic models which can be adopted in the preliminary design phases to identify critical issues of the structural response to be investigated in further more accurate studies. However, collapse scenarios induce large displacements into the structure and activate the non-linear-response of materials, non-linear material models are the most appropriate. Materials yielding provides the main contribution to the energy dissipation capacity and to the redistribution of the internal forces. Steel yielding must be properly represented because it enables development of plastic hinges and activation of the catenary effect in beams. At this aim, models of different degree of accuracy and complexity can be used. A useful guide about the steel stress-strain relationships to be adopted can be found in EN 1993-1-14 (2020). For concrete also, the constitutive law should be properly modelled, incorporating its asymmetric response in tension and compression, in order to enable simulation of cracking, which is vital for catching the development of yielding lines in the concrete slab.

Besides, highly refined materials models can be adopted incorporating the unloading and reloading branches from an inelastic state. Moreover, depending on the level of complexity and accuracy of the analysis, considering the materials' cumulative damage would allow to catch local collapses as well as the potential detachment of the components.

When required by the specific design scenario, other material features should be properly modelled. As an example, when investigating fire scenarios, the temperature dependence of the mechanical properties of the materials should be accounted for. At this aim, a guide is provided by part 1-2 of the Eurocodes and in particular EN 1992-1-2 (2004) and the EN 1993-1-2 (2005), for concrete and steel, respectively. Similarly, when the scenarios involve dynamically applied actions (e.g., explosions or impact loading), the strain rate sensitivity of the material properties should be considered. The effects of strain rate on material strength are usually implemented into models considering a dynamic increase factor (DIF) (Johnson and Cook, 1983; Malvar and Crawford, 1998).

A second key element of the modelling phase is the choice of the finite element types (line, surface, volume or special elements such as mass, spring…). In detail, the order and type of the chosen finite elements are related to the structural behaviour (magnitude of deflections, strains, rotations, stresses), the chosen method of analysis (linear and non-linear) and the material representation (linear or non-linear).

In framed structures, beams and columns are usually modelled via beam elements with centroidal axis coincident with the centroid of the cross-sections. Nevertheless, when significant for the structural response, eccentricities have either to be accounted for explicitly or considered in the interpretation of the results of analysis. The selection of the beam element, in terms of DOFs depends on the investigated problem. Local behaviour such as web crippling or plate buckling are not covered by the beam modelling and they should be taken into account with more sophisticated models or additional calculations.

Shell or solid elements are usually used to account for the slab contribution in 3D models. The first approach, although characterised by high computational efficiency, makes difficult to catch in detail the behaviour of slabs in the various phases of the response from flexural to membrane-like. A full 3D solution with solid elements should hence be adopted to account for the various combined effects of the composite floor response, such as the yielding and fracture in the steel reinforcement mesh and deck, the crushing in the concrete slab, the nonlinear behaviour of the shear connectors. In order to reduce the computational demand a hybrid approach may offer a "mid-way" solution, which consists in the use of shell elements combined with a smeared steel layer for modelling the reinforcement in the two directions. In addition, the possible steel deck can be simulated with beam elements in the direction of the steel deck ribs and then connected to the slab through tie constraints. This approach allows considering the steel components needed to simulate the tensile resistance of the slab when the membrane effect develops. Eventually, shear connections between beam and slab can be simply modelled through links characterised by laws suitable to represent the shear connection.

As the joints (in particular the beam-to-column joints) are key structural components in preventing progressive collapse, an adequate modelling is required. In detail, depending on the level of the analysis, the joints may be modelled either in a "sophisticated" way (i.e., using solid or shell elements) or through a simplified approach, i.e., using beam elements, constraints or springs. To limit the complexity of the analysis, simplified models, such as the component method (EN 1993-1-8, 2005; EN 1994-1-1, 2004), are usually adopted with the requirement that the key parameters of stiffness, strength and deformation capacity of the steel joints are caught with adequate accuracy. More details about the characterisation of joints are provided in Section 2.2.

Another important issue is the definition of the boundary conditions: they should reflect in a realistic way the actual restraint conditions providing a kinematically stable static system, and should be consistent with the DOFs of the type of finite element used. Interaction between different parts or components of a model usually requires definition of contacts. The FE programs nowadays available allow the designer to select different types of contact models. Their calibration requires a set of parameters to be accurately identified. For this reason, incorporating contacts between the parts in the model enables a more realistic simulation of the structural response but at the cost of higher design and computational time.

Finally, the choice of the type of analysis: it depends on the problem to be investigated. Linear analysis is simpler to be developed and can be performed via commercial software. Nevertheless, linear analysis cannot activate the main source of non-linearities typical of progressive collapse scenarios which arise from: i) large displacements and large strains (geometric non-linearity); ii) non-linear stress-strain relationships (yield and material non-linearity); iii) change of contacts between elements (topological/contact, non-linearity). Therefore, non-linear analysis, which requires the use of advanced design tools, should be performed.

The numerical analyses should aim at providing the background necessary to evaluate the structural ability to activate alternative load paths. Based on the displacement field, it is possible to estimate the deformation capacities required in the plastic zones and evaluate the additional design forces in the structural elements; accordingly, it is possible to check if the structure is sufficiently robust to reach this new state of equilibrium (Demonceau et al., 2018). These additional forces may lead to different potential failure modes which needs to be considered:

- Joint failure: the beam-to-column joints which are initially designed for bending moment and shear forces have to support additional tension forces which arise from the presence of the catenary action. This may lead to failure of some joint components. Also, if partial-strength joints are used, the latter will yield and failure may occur due to excessive deformations, i.e., by lack of ductility.
- Beam failure: for structures with full-strength joints, the entire plastic zone may develop at the beam extremities. As plastic hinge develops due to bending moment, followed by significant deformations under M-N interaction, this yielded zone may fail by lack of deformation capacity. Also, the beams at the top of the frames may fail by instability under bending and compression, this compression being associated with the development of an arching effect in the structure.

- Column instability: extra compression forces are developing in the columns adjacent to the lost one which may lead to column buckling. In addition, columns on which catenary forces act may be more sensible to buckling as high forces can lead to significant out of plane displacements.

Detailed numerical simulations of explosive events may also be contemplated. Nevertheless, it is important to be aware that numerical models and analysis procedures still need experimental validation. One such tool is the Extreme Loading for Structures (ELS) software, which allows structural engineers to design and analyse a structure subjected to blast loads with full 3-D nonlinear dynamic analysis. The results allow users to visualize in 3D how the building or different structural components inside the building will behave under the prescribed conditions. Moreover, because ELS is based on the Applied Element Method (AEM), engineers can visualize the after-blast effect of the resultant debris and its effect on other structural components, creating a "true damage" picture of the occurrence. In this software, blast pressure loading curves can be created automatically using UFC 3-340-02 (Structures to resist the effects of accidental explosions) or by importing custom pressure time history loads.

5.3.5 Prediction of the dynamic response from the static one

The maximum dynamic response can be determined from the nonlinear static response through a simplified approach. The main concept behind this proposed simplified approach is that the sudden column loss resembles in effect the sudden application of the gravity load on the directly affected sub-structure, especially when large deformations are sustained. Immediately after column loss, the structure accelerates from rest where the gravity load exceeds the static structural resistance and where the difference between the work done by the load and the strain energy stored is transformed into kinetic energy. As the deformations increase, the static resistance exceeds the applied loading and the strain energy stored becomes more than the work done by the gravity load, which consequently leads to a continuous reduction in kinetic energy until the structure is brought back to rest at a maximum dynamic displacement. Considering the response being dominated by a single deformation mode, the maximum dynamic response is reached when the kinetic energy is reduced back to zero, or in other words, when the work done by the gravity loads becomes identical to the energy absorbed by the structure. This gives rise to the concept of a pseudo-static response.

Considering the nonlinear static load-deflection response for a given appropriate level of idealisation of the structural system at two levels of suddenly applied loading ($P = \lambda_1 P_o$) and ($P = \lambda_2 P_o$) as shown in Figure 49a and Figure 49b (Izzuddin et al., 2008), the maximum dynamic displacements ($u_{d,1}, u_{d,2}$) associated with the sudden application of gravity load (λP_o) can be determined from energy balance between the work done by the load and the internal energy stored.

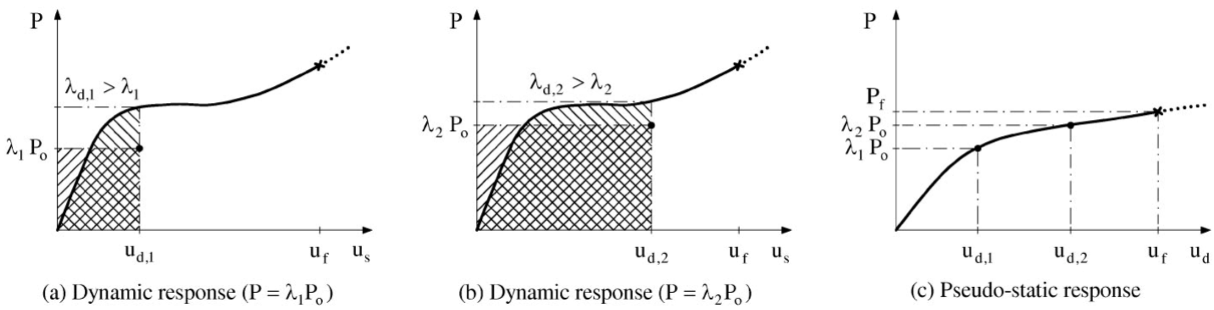


Figure 49. Simplified dynamic assessment and definition of pseudo-static response (Izzuddin et al., 2008)

With the assumption of a SDOF mode, the equivalence between external work (W_n) and internal energy (U_n) can be obtained such that the two depicted hatched areas become identical. Utilising the nonlinear static load-deflection response allows the level of the suddenly applied gravity loading ($P_n = \lambda_n P_o$) that result in a certain maximum dynamic displacement ($u_{d,n}$) to be obtained as follows:

$$W_n = \alpha\lambda_n P_o u_{d,n}; \qquad U_n = \int_0^{u_{d,n}} \alpha P \mathrm{d}u_s\,; \qquad W_n = U_n \tag{44}$$

$$P_n = \lambda_n P_o = \frac{1}{u_{d,n}} \int_0^{u_{d,n}} P \mathrm{d}u_s \tag{45}$$

such that the integral represents the area under the nonlinear static (P, u_s) curve up to $u_{d,n}$.

If the suddenly applied gravity loading (P_n) is plotted against the maximum dynamic displacement ($u_{d,n}$) for different levels of loading (λ_n), the "pseudo-static" response can then be obtained as shown in Figure 49c. For the actual gravity loading (P_o), the maximum dynamic displacement can be easily obtained from the pseudo-static response at ($P = P_o$). In addition, the complete pseudo-static response provides useful information about the impact of different levels of gravity loading in the event of sudden column loss and the sensitivity of the maximum dynamic displacement to the slight changes in the applied gravity load. Ultimately, this proposed simplified approach allows the pseudo-static response to be directly obtained from the nonlinear static response, unlike the use of detailed nonlinear dynamic analysis that would require a large number of simulations under different levels of gravity loading.

A simple and straightforward procedure for establishing the pseudo-static response curve and the maximum dynamic displacement is provided as follows (Izzuddin et al., 2008). Assuming a nonlinear static response defined as a (P, u_s) curve, whether from detailed finite element modelling or using simplified analytical expressions, the presented algorithm can be used to establish the pseudo-static response (P, u_d) curve and the dynamic displacement corresponding to the suddenly applied gravity loading ($P = P_o$). In the following proposed algorithm, $P_{m\backslash n}$ refers to the suddenly applied load ($\lambda_{m\backslash n} P_o$), while $P_{d,m\backslash n}$ refers to the amplified static load ($\lambda_{d,m\backslash n} P_o$), with *m* and *n* indicating the start and end of the current increment, respectively.

1. Initialise: $P_{d,m} = P_m = 0$, $u_{d,m} = 0$, $A_m = 0$; choose a small displacement increment Δu_d.
2. Set: $u_{d,n} = u_{d,m} + \Delta u_d$.
3. Determine $P_{d,n}$ corresponding to $u_{d,n}$from nonlinear static response (P, u_s) curve; obtain current area under the (P, u_s) curve: $A_n = A_m + (P_{d,m} + P_{d,n})\Delta u_d/2$.
4. Determine current pseudo-static load: $P_n = A_n/u_{d,n}$; establish new point (P_n, $u_{d,n}$) on pseudo-static response (P, u_d) curve.
5. If ($P_m < P_o \leq P_n$), obtain and output dynamic displacement corresponding to P_o: $u_d = u_{d,m} + (u_{d,n} - u_{d,m})(P_o - P_m)/(P_n - P_m)$.

If more points are required for pseudo-static response curve: update: $P_{d,m} = P_{d,n}$, $P_m = P_n$, $u_{d,m} = u_{d,n}$, $A_m = A_n$; repeat from step 2.

5.4 Key element method

According to the literature, a key element is a structural component or a part of the structure whose failure entails further damage that violates the performance objective. In order to avoid local damages exceeding an assumed limit value, such elements have to be properly identified and designed.

Following the "recent" codes (EN 1991-1-7, 2006), the strength of the key elements has to be enhanced to withstand a specified level of load.

This design strategy is frequently adopted for structures possessing a limited level of redundancy such as tensile structures, 2D and 3D trussed systems, cable stayed and suspension structures. Key elements can also be used in addition to other design features to improve the robustness of high-risk buildings (Arup, 2011). Furthermore, this design approach is often the only rational approach when retrofitting existing buildings. Depending on the context, examples of potential key elements could be columns, load-bearing walls of a building, piers of continuous bridges or cables in a cable-supported structure (Starossek and Haberland, 2012).

According to EN 1991-1-7 (2006), the accidental design action for checking key elements is of 34 kN/m^2 applied in any direction. This load, based on the Ronan Point collapse in London, 1968 (Way, 2011), is intended to represent a possible range of impact and blast events and is used as a tool for designing key elements to be more robust than what is required for normal design cases.

The key elements must be designed to develop their full resistance without failure of either the member itself or of its joints. Therefore, the design action should be applied to the key element and any components attached to it, unless the attached components or their joints cannot sustain the 34 kN/m^2. Hence, for the design of a key element, it is necessary to consider which components, or portion of components, will remain attached to the element in the event of an accident. This implies that various cases of loading would need to be considered for a wall or slab attached to the member, with due account of upper and lower limits of the attachment capacity. In this design approach no capacity for load redistribution needs to be provided.

Therefore, the key element approach includes the following steps:

- Identification of the key structural members.
- Design of the key elements to resist to an accidental design action A_d applied in the horizontal and vertical direction, one direction at a time. According to EN 1991-1-7, the recommended value for A_d is 34 kN/m^2. However, if appropriate, other accidental actions can be considered.
- The accidental design action has to be applied to the key element and to any attached component.

In the design process, the accidental load combination of Eurocode 0 (EN 1990, 2002) should be used in the design of key elements and their attached components.

5.5 Segmentation method

Segmentation/compartmentalisation is a design strategy that can offer a possibility to enhance the robustness of a structure. In such an approach, the spreading of failure following an initial damage can be prevented or limited by isolating the failing part of a structure from the remaining structure by what can be referred to as segment/compartment borders. Such approach would ensure that each part/compartment/segment is able to collapse independently without affecting the safety of the other parts. Segmentation strategies can generally be based on either weak segment borders or strong segment borders where the locations of the segment borders are selected by the design engineer within the scope of the design objectives and in accordance with the requirements of the client and the relevant authorities depending on the type and importance of the structure (Starossek, 2007; Starossek and Haberland, 2012; CEN/TC250/WG6, 2020).

In alternative load path methods, the extent of collapse increases, and the effectiveness of the method decreases with an increase in initial damage size. It is thus preferable in the case when the size of the initial damage is assumed to be small. On the other hand, when using the segmentation strategy, the extent of collapse and the effectiveness of the method are considered to be insensitive to the size of the initial damage, provided that the segments sizes are not too small. However, the fixed extent of collapse due to segmentation is relatively large corresponding to the failure of the whole segment. Consequently, such method is desirable when the initial damage size is assumed to be of a large value. Segmentation can also be combined with alternative load path methods, where alternative paths can be provided within the individual segments. In such case, the extent of failure spreading will not be significantly larger than the initial assumed damage for both small and large initial damage sizes (Starossek and Haberland, 2012).

5.5.1 Weak segment borders

Segmentation achieved using weak segment borders would allow the failure of a specific segment to take place without the progression of failure to other adjacent segments. In such mode, segmentation can act as a structural fuse, where failing parts can safely disconnect from the structure. This segmentation method can be achieved by eliminating continuity between adjacent segments/ compartments or reducing the stiffness to accommodate large deformations and displacements at the segment borders and thus limiting the amount of force transmitted to the surrounding structure (Starossek, 2007; Starossek and Haberland, 2012; CEN/TC250/WG6, 2020).

It should be also noted that providing continuity in general has a desirable effect on the overall performance of the structure in extreme events; however, continuity can be detrimental when the resulting alternate load paths are not provided with the required strength that is able to withstand the forces transmitted by continuity. Therefore, in the case where alternative load paths are impractical, or too expensive to be provided, segmentation by selectively eliminating continuity would be advantageous. This is also the case if alternate load paths (or collapse-isolating elements) are strong enough, but the corresponding verification proves difficult or unconvincing (Starossek, 2006).

For building structures, such a form of segmentation is commonly applied to horizontal low-rise buildings which have a relatively large footprint. In such low structures, it can be assumed that collapse would involve the full height of the building; however, it is limited in horizontal extent at locations where collapse forces cannot be transferred across the boundary to the surrounding structure. As stated, it is desirable that alternative paths are provided within the individual segments. It should mentioned that the surrounding structure should be checked under the highest possible level of tying forces such that failure in adjacent segments should be avoided.

5.5.2 Strong segment borders

Segmentation based on strong segment borders is designed to prevent an incipient collapse providing high local resistance that is able to accommodate relatively large forces. In this mode, segmentation can offer an alternate load path, such that resistance to local damage is achieved at relatively small deformations, or it can stop the collapse of part of the structure. This form of segmentation can be considered for vertical structures, such as the case of multi-storey buildings with outrigger or belt trusses at regular intervals, where such trusses can act along with vertical tying to allow for the redistribution of the loads following local damage arresting falling debris and adding stability to the surrounding structure (CEN/TC250/WG6, 2020; Starossek, 2007; Starossek and Haberland, 2012; Starossek, 2018; Ellingwood et al., 2007).

A third possibility of creating segment borders is to provide them with high ductility and large energy dissipation capacity (to accommodate large forces and large displacements at the same time) (Starossek, 2009).

6 Risk assessment

A risk analysis is based on the assessment and mitigation of the risk of structural damage and the consequences that could arise from the damage state, after an occurrence of low-probability, high-consequence accidental hazards, such as impact, fire, explosions, human errors, etc.

Within the Eurocode framework, a risk analysis is only required for buildings falling into the high consequences class CC3 according to EN 1991-1-7. Two types of risk analysis can be used, namely, i) *qualitative* and ii) *quantitative* analysis, being the main steps required for both analyses summarized in *Figure 50*.

In practice, a risk analysis based on a quantitative approach is quite complex to carry out since it requires the quantification, in terms of probabilities, of the likelihood of occurrence of each considered hazard as well as all of the possible consequences of its occurrence in the building, requiring the use of robust risk models and high amounts of data. For these reasons, the quantitative approach is rarely applicable by designers. However, if necessary, some guidelines on quantitative structural risk analysis are provided in EN 1991-1-7.

On the other hand, a risk analysis based on a qualitative assessment can be performed at any time or stage of a project even if it is strongly recommended to initiate it at an early stage of the design process. One of the crucial tasks to be achieved is the identification of hazards to be considered. In Annex B (informative) of EN 1991-1-7, conditions which could present hazards to a structure are identified (see Section 4 dedicated to identified threats); the identification of the hazards should be performed in close interaction with the future owner of the building and/or with the authorities. Then, for the so-identified hazards, it is asked to describe the possible consequences in case of occurrence of the latter and to define the required measures if these consequences are not acceptable. The qualitative assessment is easier to apply than the quantitative approach, therefore is most often the preferable approach even if it tends to be more subjective.

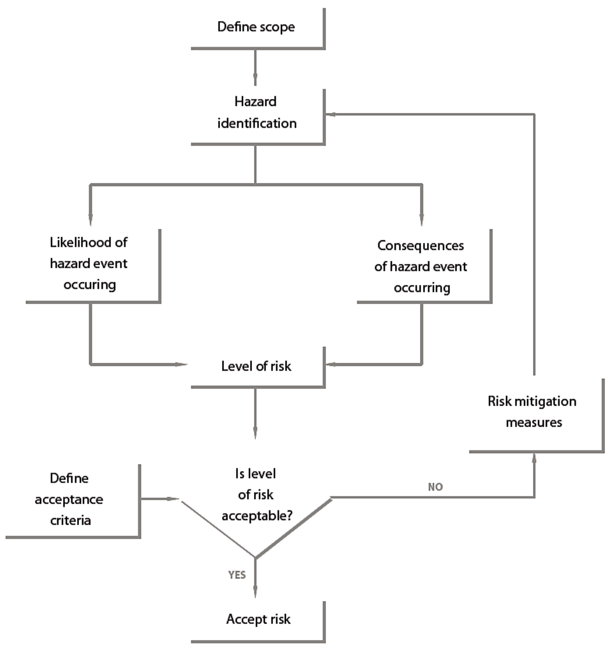


Figure 50. Risk assessment

7 Conclusions

Structural robustness and mitigation of progressive collapse is a specific safety consideration which is now addressed in modern codes and standards, including the Eurocodes, and which requires particular care from all professionals involved in the construction industry, including architects, designers, constructors, control officers, and insurance managers. However, looking more deeply to the Eurocode clauses related to this issue, only general design recommendations are provided, and the latter are sometimes unclear, uncomplete and too general in order to take into account of the specificities of the different structural typologies. This leads to difficulties for practitioners as clear design guidelines on how to meet this request for robustness are missing.

The FAILNOMORE project financed by the Research Fund for Coal and Steel of the European Commission aimed at filling this gap for steel and steel-concrete composite structures by collecting the most recent research outcomes in these field and transforming it into practical recommendations and guidelines. The present design manual constitutes the main result of this project reflecting the different practice-oriented and user-friendly design strategies for robustness which have been commonly agreed at European level as summarised here below.

To assist the practitioner, a general flowchart for the design for robustness is proposed in Section 2.1. This representative scheme guides the designer through the decision-making process and facilitates the judgment on the adoption of a suitable design strategy for any type of accidental scenario while being in full compliance with the requirements and recommendations of the Eurocodes.

The selection of the design strategies to be adopted is founded on the concept of consequences classes which is introduced in Chapter 3.

Then, a first set of design methods for structural robustness for identified accidental action including impact, explosion, fire, and earthquake as exceptional events are presented in Chapter 4. The proposed approaches range from protective measures aimed at preventing the occurrence of the accidental event to explicit design against a specific action. Various design methods with different levels of complexity and accuracy are presented in detail. The practitioner may opt for any of these approaches based on the complexity of the structural layout and the sought accuracy in the design.

Since a realistic examination of all accidental situations which could lead to collapse initiation is not feasible, a second set of threat-independent design methods are reported in Chapter 5. These methods aimed at enhancing the robustness of a structure to limit the propagation of a local damage within a structure. Valuable guidance is first given on the identification of the localised damage to be considered in the design. Then, approaches with different levels of complexity using simplified analytical to full numerical tools are presented with particular emphasis given to the ductility and rotation capacity required at the level of the structural elements and/or joints.

Finally, Chapter 6 provides a brief introduction to risk assessment and analysis which is sometimes requested for specific structures included in the most severe consequences class.

In conclusion, in the present design manual, the practitioners will find valuable information and methods, with different levels of sophistication, to meet the request for robustness for steel and composite structures in their daily practice using the full potential of their constitutive materials and elements. The applicability of the proposed design methods is illustrated in the second part of this design manual using relevant examples.

Part 2 – Worked examples

8 Introduction

8.1 General

This section illustrates and demonstrates the applicability of the proposed guidelines for the design/verification for robustness of steel and composite building frames. Four structural configurations are selected for the present study, see Table 8. Two structures are initially designed for the persistent design situation (non-seismic area) and two for persistent and seismic design situations (seismic area) according to the current version of the Eurocodes.

Table 8. Types of structures

Reference name	Type of structure
SS/NS	Steel Structure in Non-Seismic area
CS/NS*	Composite Structure in Non-Seismic area
SS/S	Steel Structure in Seismic area
CS/S	Composite Structure in Seismic area
* The structure has two variations – one with steel columns and one with composite columns. In both cases the floor beams and slabs are designed as composite.	

In the worked examples, the structures are initially designed for the ultimate limit state (ULS) and serviceability limit state (SLS) (with additional requirements for damage limitation state (DL) for seismic systems i.e., SS/S and CS/S), and the results are presented from Section 8.3 to Section 8.6.

Then, the design for robustness is considered in Section 8.7 (for identified accidental actions) and in Section 8.8 (for unidentified accidental actions), respectively. The examples illustrate the application of most of the methods presented in the previous sections.

The design for robustness requires first the classification of the structure in terms of consequences in case of accidental actions (see Section 3). All the studied structures are included in Consequences Class 2b (Upper Risk Group).

The design for identified actions (Section 8.7) will include verifications against five accidental actions, as reported in Table 9, using the recommendations of Section 4. As reflected in Table 9, all the studied structures will not be verified for all the considered accidental actions; the objective here is to apply the different methods presented in Section 4 on at least one worked example. According to the type of the accidental actions, the worked examples are labelled from I.1 to I.5 for identified accidental actions (impact, explosion, fire, and earthquake) and from II.1 to II.4 for unidentified actions (see Table 9 and Table 10).

For impact and gas explosion (see Section 4.2 and Section 4.3.1.1 respectively), methods with different levels of sophistication will be applied for the sake of completeness (Table 9), even though the recommendations for Class 2B structures (see EN 1991-1-7 and green boxes in Table 9) limit the analysis to simplified static equivalent action models. For external blast actions, which are not explicitly covered by the Eurocodes (see Section 4.3.1.1), the application of simplified rules and advanced dynamic analysis will be illustrated on the SS/S structure. The application of simplified rules and of thermal numerical analysis will be illustrated for the CS/NS structure under fire scenario (see Section 4.4). For the seismic actions (see Section 4.5), two design situations will be considered:

- Application of prescriptive recommendations on the SS/NS structure, which is assumed to be built in Aachen, i.e., in a region where the seismic action should not be explicitly accounted for in the design process but could however occur;
- Application of advanced numerical analysis considering multi-hazard scenarios on the SS/S structure.

For the design for unidentified accidental actions (Section 8.8), different alternatives are proposed for Class 2B structures (see Section 2.1.4).

The first one is the use of a prescriptive method to ensure the possibility of activating horizontal and vertical ties in case of accidental actions (see Section 5.3.1). This method is the easiest to be applied and its application is illustrated for all the worked examples as highlighted in Table 10 (green column).

The second one is the consideration of the notional removal of supporting elements considered to be removed one at a time in each storey of the building. The application of this second alternative can be contemplated using approaches with different levels of sophistication:

- Use of an analytical method (see Section 5.3.2) – the application of this method is exemplified for the SS/NS structure;
- Use of a simplified numerical approach (see Section 5.3.3) – this approach is applied to the SS/S structure;
- Use of an advanced numerical approach (see Section 5.3.4) – this approach is applied to all the worked examples.

A third alternative is the use of the key element method as proposed in EN 1991-1-7 (see Section 5.4). This method will be considered for the CS/NS structure.

Finally, the last alternative is the use of segmentation which will be briefly addressed for the SS/NS structure.

Table 9. Types of approaches for identified actions and their application

	Identified actions									
	Impact			**External explosion**		**Internal explosion**		**Localised fire**	**Seismic**	
Structure	Equivalent static approach	Simplified dynamic approach	Full dynamic approach	Equivalent SDOF approach	Full dynamic approach	Equivalent static approach	Dynamic approach (TNT equiv. method)	Localised fire models	Prescriptive method	Advanced numerical analysis (multi-hazard)
SS/S				I.2.2/ SS/S	I.2.3/ SS/S	I.3.1/ SS/S	I.3.2/ SS/S			I.5.2/ SS/S
CS/S	I.1.1/ CS/S	I.1.2/ CS/S	I.1.3/ CS/S							
SS/NS									I.5.1/ SS/NS	
CS/NS	I.1.4/ CS/NS			I.2.1/ CS/NS				I.4.1/ CS/NS		

Table 10. Types of approaches for unidentified actions and their application

	Unidentified actions					
	Alternate load path method (ALPM)				**Key element**	**Segmentation**
Structure	Prescriptive approach (Tying method)	Analytical approach	Simplified prediction of dynamic response	Full numerical approach	Normative approach	Weak segment borders / Strong segment borders
SS/S	II.1.1/ SS/S		II.4.2/ SS/S	II.4.3/ SS/S		
CS/S	II.1.2/ CS/S			II.4.4/ CS/S		
SS/NS	II.1.3/ SS/NS	II.4.1/ SS/NS		II.4.5/ SS/NS		II.3.1/ SS/NS
CS/NS	II.1.4/ CS/NS			II.4.6/ CS/NS	II.2.1/ CS/NS	

In the present design manual, only a summary on the conducted investigations on the worked examples is proposed. More details are available in Deliverable D2-2 of the FAILNOMORE project, which is freely available (in English) on the official website of the project (https://www.steelconstruct.com/eu-projects/failnomore/).

8.2 Geometry and structural systems proposed for investigation

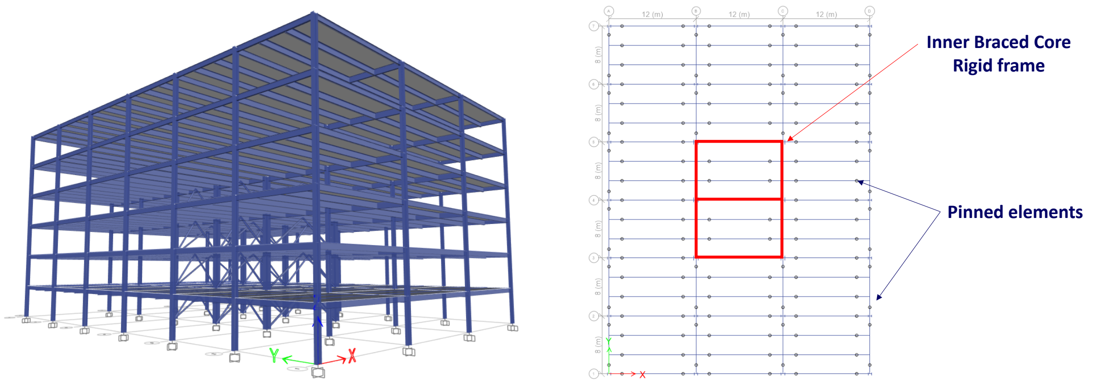
The geometry of the structures is shown in Figure 51, and consist of:

- Non-seismic area:
    1. 6 storeys of 4.0 m height each;
    2. 6 bays of 8.0 m in the Y direction;
    3. 3 bays of 12.0 m in the X direction.
- Seismic area:
    1. 6 storeys of 4.0 m height each;
    2. 6 bays of 8.0 m in the longitudinal direction;
    3. 3 bays of 12.0 m in the transversal direction – internal;
    4. 6 bays of 6.0 m in the transversal direction – perimeter.

The main structural system consists of:

- Non-seismic area (Figure 51a):
    1. An internal V-braced core to resist lateral loads from wind;
    2. A beam grid with main beams and secondary beams to resist gravity loads.
- Seismic area (Figure 51b):
    1. A dual system made of an internal V-braced core and perimeter moment resisting frames (MRFs) to resist lateral loads from wind and earthquakes;
    2. A beam grid with main beams and secondary beams to resist gravity loads.

The initial design used S355 steel and C30/37 concrete. Additionally, for the structures in seismic areas, S460 steel grade was used for the non-dissipative beams in the braced frames. H and circular hollow sections were used for steel elements. The joints were designed according to the EN 1993-1-8 provisions, with additional requirements for seismic resistant systems in terms of minimum capacity (see EN 1998-2). More details about the structural systems are given in the next sections.



a)

Figure 51. Presentation of the structural systems: a) non-seismic structures; b) seismic structures

b)

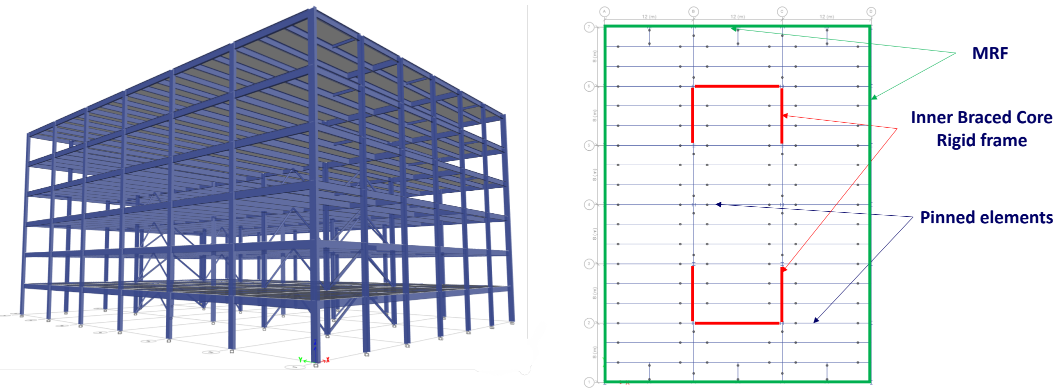


Figure 51. Presentation of the structural systems: a) non-seismic structures; b) seismic structures (cont.)

8.3 Actions, combination of actions

The actions that were used in the design of each structural typology are presented in Table 11. Combination of actions for ULS and SLS were considered in compliance with EN 1990. Additionally, according to EN 1998, damage limitation limit state (DL) was considered for SS/S and CS/S cases.

Table 11. Actions considered in the design process

Loads	Structures		
	SS/S & CS/S	**CS/NS**	**SS/NS**
	Location		
	Timișoara, RO	Luxembourg	Aachen, DE
Dead load	- Floors: g_k = 5 kN/m^2 - Façade (supported by the perimeter beams): g_k= 4 kN/m		
Live load	- Live load for office buildings: q_k = 3 kN/m^2 - construction load q_k = 1 kN/ m^2 (general floors and roof).		
WIND			
Wind speed	$v_{b,0}$ = 25 m/s	$v_{b,0}$ = 24 m/s	$v_{b,0}$ = 25 m/s
Equiv. wind pressure	q_b = 0.4 kN/m^2	q_b = 0.36 kN/m^2	q_p = 0.9 kN/m^2*
Terrain category	III	III	“Binnenland”*
Snow load	s_k = 1.5 kN/m^2	s_k = 0.5 kN/m^2	s_k = 0.85 kN/m^2**
Seismic load			
Elastic response spectrum	Type 1		
Ground type	B		
Design ground acceleration, a_g	0.25 g		
Behaviour factor, q	q = 4.8 (dual frame CBF+MRF)		

* Simplified wind pressure acc. to DIN EN 1991-1-4/NA Tab. NA.B.3 as commonly used in Germany. This replaces the concept of terrain category. “Binnenland” can be translated with “inland region” or “interior region” and is used to be distinguished from island and coastal regions.
** Snow zone 2 according to DIN EN 1991-1-3/NA

8.4 Design requirements and output

The structural analysis was carried out using 3D models and linear elastic analyses.

For SS/NS, calculations are performed using the following software:

- Dlubal RSTAB 8.22 for FE analyses including the STEEL EC3 module for cross-section member verifications;
- COP 2.1.3 Premium for the verification of connections.

For CS/NS, the design of the building was performed using the software SCIA (version 2019), while the connections were designed using calculation spreadsheets.

For the structures designed in seismic areas (SS/S and CS/S), Etabs v.19 and SAP2000 v23 software were used. The design of connections was performed using STeelCON software. For the seismic design, a modal response spectrum analysis was conducted. Also, the plastic mechanism and seismic response by means of non-linear static analysis procedure (push-over analysis) was evaluated using the N2 method.

The verifications performed for all structures included:

- ULS verifications for which the results are reported through utilization factors (UFs).
- SLS verifications, which were done using the following admissibility criteria:
    1. The maximum deflection of secondary beams limited at $L/250$;
    2. The maximum deflection of main beams limited at $L/350$;
    3. The maximum top displacement under wind action limited at $H/500$;

where L is the length of the beams and H is the height of the structure.

Additionally, for the seismic resistant structures, the following verifications were performed:

1. Interstorey drift limited at 0.75% H_{st} to comply with the damage limitation requirement (buildings with ductile non-structural elements);
2. Second order effects: $\theta \leq 0.2$;
3. Verification of dissipative members and connections in CBFs and MRFs;
4. Verification of non-dissipative members and connections in CBFs and MRFs;

where H_{st} is the storey height and θ is the interstorey drift sensitivity coefficient.

The output of the design is presented in Table 12 to Table 14. Table 12 shows the cross-sections for the different categories of beams and the utilization factors for strength (including buckling resistance where appropriate) and stiffness. Table 13 presents the cross-sections for the different categories of columns and the utilization factors for strength (including buckling resistance). For the structures designed in the seismic area, utilization factors for the columns of the Lateral Load Resisting System LLRS refer to maximum demand between combinations with wind or seismic action.

The SLS verification for all structures subjected to the wind action is presented in Table 16. All structures having the same height H (24 m), the ratio between the lateral top displacement and the acceptable limit has a maximum value of approximately 0.5.

Regarding the specific verifications for the structures in the seismic area, Table 17 presents the interstorey drift check at damage limitation state. As it may be observed, the structures successfully fulfil the limitation to 0.75%, having a maximum interstorey drift of 0.24%. The SS/S and CS/S have also been checked at ULS in terms of interstorey drift limitation using the following equation:

$$d_r^{ULS} = c \cdot q \cdot d_{re} \leq d_{r,a}^{ULS} \tag{46}$$

where c is the amplification factor (considered 1 since $T_1 \geq T_C$), q is the behaviour factor, and d_{re} is the relative displacement obtained from static calculation.

The acceptable limit for this verification is 2.5% H_{st}. As presented in Table 18, all values are below this limit, the largest being of 0.49%.

In addition, the results for the second-order verification are provided in Table 19. As it may be observed, the largest value for θ is 0.096. Consequently, as it is mentioned in Eurocode 8, the second order effects may be neglected, having a value for θ smaller than 0.1.

The seismic loading for the design of the non-dissipative elements takes into account the utilization factor (UF) of the braces. Consequently, having an UF of 0.46 for the most stressed brace, an overstrength factor of 1/0.46 = 2.16 was obtained. Considering also the strain hardening effect, the total overstrength factor considered for the design of the non-dissipative elements was Ω_T = 3.0.

Finally, the contribution of the perimeter MRFs was checked. In (RFCS, 2017), it is mentioned that the duality should be checked by verifying that the MRFs carry at least 25% of the seismic force. Considering the equilibrium of a simple frame and the development of plastic hinges at the ends of the beams, the capacity of a MRF is twice the plastic capacity of the beam divided by the storey height. The necessary flexural resistance of the beam may be determined using the following expression:

$$M_{pl,b} = \frac{F_y^{MRF}}{2} \cdot \frac{H_{st}}{n} \tag{47}$$

where F_y^{MRF} is the capacity of the frame, H_{st} is storey height, and n is the number of beams.

In the above formula, the capacity of the frame is taken as equal to 0.25 of the storey seismic force and n as equal to 12 since there are 6 beams per frame and 2 frames per direction resisting the seismic actions.

As presented in Table 20, the necessary flexural capacity is smaller than the efficient one in both directions; so, the duality condition is checked.

Table 12. Sections and utilization factors for beams

Case	Element	Direction[4]	Storey	Section	Utilization factor (UF)	
					Strength	Deflection[1]
SS/S	Perimeter beams	X	1-6	IPE550	0.278	0.023
		Y	1-6	IPE600	0.302	0.153
	Interior beams	X	1-6	IPE550	0.546	0.85
		Y	1-6	IPE550	0.909	0.928
	[5]Inner core beams	X	1-3	[6]H800*	0.936	-
			4-5	HEM800	0.953	-
			6	HEM700	0.789	-
		Y	1-3	HEM500	0.859	-
			4-6	HEB500	0.878	-

Table 12. Sections and utilization factors for beams (cont)

Case	Element	Direction[4]	Storey	Section	Utilization factor (UF)	
					Strength	Deflection[1]
CS/S	Perimeter beams[2]	X	1-6	IPE550	0.278	0.178
		Y	1-6	IPE600	0.302	0.157
	Interior beams[2]	X	1-6	IPE400	0.627	0.971
		Y	1-6	IPE450	0.874	0.94
	[5]Inner core beams	X	1-3	[6]H800*	0.936	-
			4-5	HEM800	0.953	-
			6	HEM700	0.789	-
		Y	1-3	HEM500	0.859	-
			4-5	HEB500	0.878	-
SS/NS	Perimeter beams	X	1-6	IPE500	0.51	0.89
		Y	1-6	IPE500	0.75	0.83
	Interior beams	X	1-6	IPE550	0.62	0.93
		Y	1-6	IPE600	0.87	0.89
	Inner core beams	X, Y	1-6	HEA300	0.9	0.19
CS/NS	Perimeter beams[3]	X, Y	1-6	IPE 450	0.93	0.8
	Interior beams[3]	X	1-6	IPE360	0.95	0.98
		Y	1-6	IPE500	0.96	0.86
	Inner core beams	X, Y	1-6	IPE500	0.45	-

[1]Deflection verification criterion: L/250 for secondary beams, L/350 for main beams
[2]Nelson studs d=19mm, h=100 mm / 160 mm – steel beams fully connected to a solid slab of 12cm
[3]Nelson studs d=19mm, h=100 mm / 160 mm – steel beams connected to a composite slab of 13 cm and with a Cofraplus 60 decking (0.88 mm)
[4]See Figure 51 for the orientation of the axes
[5]S460 steel grade used for the inner core beams.
[6]H800* is a built-up section, having the same height as regular HEM800, with b = 380mm, t_f = 50 mm, and t_w = 30 mm.

Table 13. Sections and utilization factors for columns

Case	Element		Section	UF
SS/S	Corner columns		HE550B	0.49
	Perimeter columns		HE500B	0.71
	Inner core columns		HD400X463	0.95
CS/S	Corner columns		HE550B	0.48
	Perimeter columns		HE500B	0.71
	Inner core columns		HD400X463	0.95
SS/NS	Perimeter columns	X	HEB 360	0.97
		Y	HEB 340	0.94
	Inner core columns		HEM300	0.95
CS/NS	Perimeter columns		HD360X162	0.61
	Inner columns		HD400X216	0.78

Table 14. Sections and utilization factors for braces

Case	Element	Direction	Storey	Section	UF
SS/S	Brace	Y	1-3	HEA320	0.41
			4	HEA260	0.43
			5	HEA220	0.46
			6	HEA200	0.39
		X	1-3	HEB340	0.41
			4-5	HEA320	0.27
			6	HEA260	0.26
CS/S		Y	1-3	HEA320	0.41
			4	HEA260	0.43
			5	HEA220	0.46
			6	HEA200	0.40
		X	1-3	HEB340	0.41
			4-5	HEA320	0.39
			6	HEA260	0.26
SS/NS		X, Y	1-6	CHS 219.1x6.3	0.90
CS/NS		X, Y	1-6	CHS 219.1x5	0.71

It may be observed that, for SS/S and CS/S structures, the condition for homogeneity (25% maximum difference between UF elements on elevation) was fulfilled for most elements. The difference between the most stressed and least stressed braces is 16% in case of Y direction. However, on X direction, the condition was not fulfilled on the last two stories due to the requirement of using Class 1 cross-sections for high ductility class.

Table 15 presents the slenderness verification for diagonal members according to the seismic design. It may be observed that all the braces fulfilled the condition, the maximum value of the non-dimensional slenderness $\bar{\lambda}$ being 0.76, which is lower than the admissible limit of 2.0.

Table 15. Slenderness check

Case	Direction	Storey	Section	A (mm^2)	f_y (MPa)	I (mm^4)	L_{cr} (mm)	N_{cr} (kN)	$\bar{\lambda}$ (-)
SS/S and CS/S	X	6	HEA260	8680	275	36680000	3605500	5848.1	0.639
		5-4	HEA320	12400	275	36950000	3605500	5891.2	0.761
		1-3	HEB340	17090	275	96900000	3605500	15449.4	0.552
	Y	6	HEA200	2570	275	13360000	2828500	3461.1	0.654
		5	HEA220	3030	275	19950000	2828500	5168.3	0.585
		4	HEA260	3310	275	36680000	2828500	9502.5	0.501
		1-3	HEA320	3710	275	69850000	2828500	18095.6	0.434

Table 16. SLS check for LLRS against wind action

Case	Direction	Top displacement (mm)	Maximum allowable displacement (mm)
SS/S	X	4.62	48
	Y	3.2	
CS/S	X	4.61	
	Y	3.16	
SS/NS	X	12.4	
	Y	7.3	
CS/NS	X	8.6	
	Y	5.6	

Table 17. Interstorey drifts for the structures in seismic zones – DL

Case	Storey	Direction	Drift (%)	Case	Storey	Direction	Drift (%)
SS/S	6	X	0.171	CS/S	6	X	0.172
	5		0.209		5		0.210
	4		0.244		4		0.243
	3		0.222		3		0.220
	2		0.224		2		0.222
	1		0.183		1		0.182
	6	Y	0.190		6	Y	0.190
	5		0.241		5		0.241
	4		0.238		4		0.238
	3		0.203		3		0.203
	2		0.193		2		0.192
	1		0.148		1		0.148

Table 18. Interstorey drifts for the structures in seismic zones - ULS

Case	Storey	Direction	Drift (%)	Case	Storey	Direction	Drift (%)
SS/S	6	X	0.343	CS/S	6	X	0.343
	5		0.419		5		0.419
	4		0.486		4		0.486
	3		0.440		3		0.440
	2		0.445		2		0.444
	1		0.364		1		0.364
	6	Y	0.380		6	Y	0.381
	5		0.482		5		0.482
	4		0.476		4		0.476
	3		0.406		3		0.406
	2		0.385		2		0.385
	1		0.297		1		0.296

Table 19. Second order effects for the structures in seismic zones

Case	Storey	h (mm)	P_x (kN)	V_x (kN)	d_x (mm)	θ_x (rad)	Case	Storey	h (mm)	P_y (kN)	V_y (kN)	d_y (mm)	θ_y (rad)
SS/S	6	4000	10867	1753	60.77	0.094	SS/S	6	4000	10867	1881	59.12	0.085
	5	4000	21734	2983	52.77	0.096		5	4000	21734	3176	50.10	0.086
	4	4000	32602	3912	42.80	0.089		4	4000	32602	4094	38.57	0.077
	3	4000	43469	4628	31.02	0.073		3	4000	43469	4810	27.01	0.061
	2	4000	54336	5193	20.18	0.053		2	4000	54336	5376	17.01	0.043
	1	4000	65203	5524	9.09	0.027		1	4000	65203	5707	7.42	0.021
CS/S	6	4000	10867	1753	60.73	0.094	CS/S	6	4000	10867	1883	59.11	0.085
	5	4000	21734	2985	52.73	0.096		5	4000	21734	3178	50.08	0.086
	4	4000	32602	3914	42.76	0.089		4	4000	32602	4097	38.54	0.077
	3	4000	43469	4630	30.99	0.073		3	4000	43469	4813	26.98	0.061
	2	4000	54336	5195	20.16	0.053		2	4000	54336	5379	16.98	0.043
	1	4000	65203	5526	9.10	0.027		1	4000	65203	5710	7.40	0.021

Table 20. Contribution of the MRF frames for the LLRS – SS/S and CS/S

Case	Storey label	Direction	V_i (kN)	$0.25V_i$ (kN)	n	$M_{Rd,nec}$ (kNm)	W_{nec} (mm^3)	Section	W_{eff} (mm^3)	$M_{RD,eff}$ (kNm)
SS/S	6	X	1752.5	438.1	12	73.0	205695.6	IPE550	2787000	989.4
	5		2983.3	745.8	12	124.3	350149.8	IPE550	2787000	989.4
	4		3911.9	978.0	12	163.0	459139.5	IPE550	2787000	989.4
	3		4628.3	1157.1	12	192.8	543229.7	IPE550	2787000	989.4
	2		5192.7	1298.2	12	216.4	609469.1	IPE550	2787000	989.4
	1		5523.6	1380.9	12	230.2	648313.6	IPE550	2787000	989.4
	6	x	1881.3	470.3	12	78.4	220813.2	IPE600	35112000	12464.8
	5		3176.0	794.0	12	132.3	372765.1	IPE600	35112000	12464.8
	4		4094.4	1023.6	12	170.6	480560.5	IPE600	35112000	12464.8
	3		4810.2	1202.5	12	200.4	564574.4	IPE600	35112000	12464.8
	2		5376.1	1344.0	12	224.0	630999.7	IPE600	35112000	12464.8
	1		5707.5	1426.9	12	237.8	669894.1	IPE600	35112000	12464.8
CS/S	6	X	1753.4	438.3	12	73.1	205796.4	IPE550	2787000	989.4
	5		2984.7	746.2	12	124.4	350314.6	IPE550	2787000	989.4
	4		3913.5	978.4	12	163.1	459332.4	IPE550	2787000	989.4
	3		4630.1	1157.5	12	192.9	543444.7	IPE550	2787000	989.4
	2		5194.7	1298.7	12	216.4	609711.1	IPE550	2787000	989.4
	1		5526.1	1381.5	12	230.3	648600.5	IPE550	2787000	989.4
	6	x	1882.8	470.7	12	78.4	220980.2	IPE600	35112000	12464.8
	5		3178.0	794.5	12	132.4	373009.1	IPE600	35112000	12464.8
	4		4096.9	1024.2	12	170.7	480855.4	IPE600	35112000	12464.8
	3		4813.0	1203.2	12	200.5	564905.2	IPE600	35112000	12464.8
	2		5378.9	1344.7	12	224.1	631327.2	IPE600	35112000	12464.8
	1		5710.0	1427.5	12	237.9	670185.7	IPE600	35112000	12464.8

8.5 Joints

8.5.1 SS/NS

Beam-to-beam as well as beam-to-column joints are pinned fin plate joints. Brace joints as well as column base joints are not detailed here. Column splices are moment resisting end-plate joints. The position of column splices is assumed approximately at mid-height of the building. The design of column splices is constructive (only compression forces and negligible bending moments).

The nomenclature of the joints throughout the worked examples is based on members IDs reported in Figure 52. Joint labels, ULS shear forces, and resistances are summarized in Table 21.

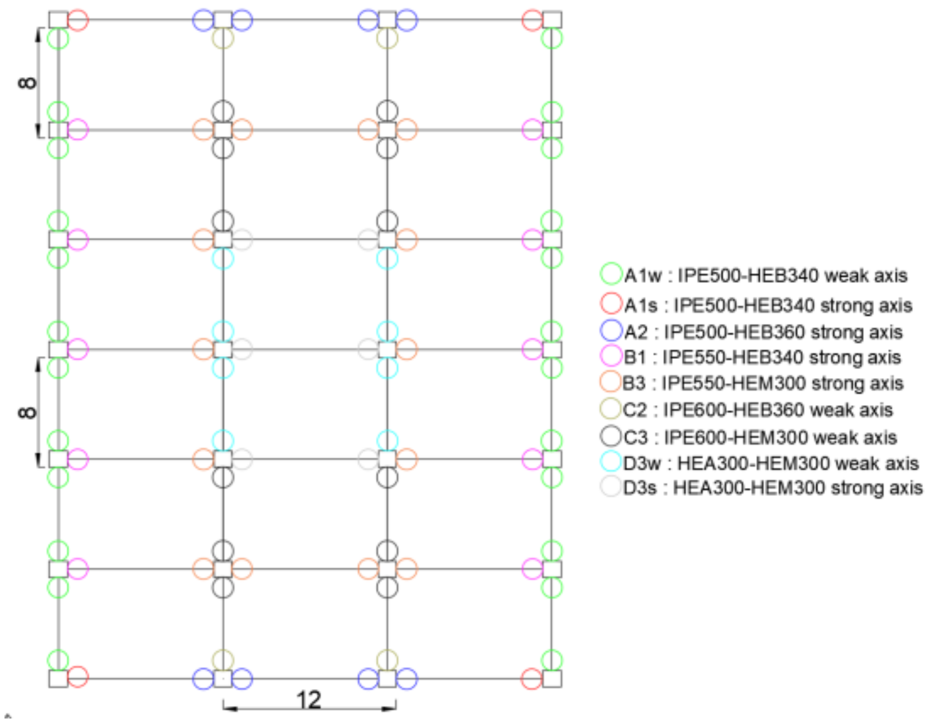


Figure 52. Joint positions

Table 21. Verifications of joints at ULS, SS/NS

Position s = strong axis w = weak axis	**Connection type**	**Shear resistance (kN)**	**Moment resistance (kNm)**	**Failure mode**	**UF**
A1s / A2	Fin plate	196	-	Fin plate in bearing	0.66
A1w	Fin plate	255	-	Fin plate in bearing	0.94
B1 / B3	Fin plate	196	-	Fin plate in bearing	0.92
C2w / C3w	Fin plate	443	-	Fin plate in bearing	0.97
D3s	Fin plate	102	-	Beam web in bearing	0.59
D3w	Fin plate	102	-	Beam web in bearing	0.88
BA / BC	Fin plate	196	-	Fin plate in bearing	0.92
BD	Fin plate	185	-	Fin plate in bearing	0.97

The design of joints has been performed using the aforementioned software COP. Notice that the design of such joints is not directly covered by the current version of the Eurocode; so the verification is based on the document from ECCS (ECCS, 2009). These verifications also contain ductility requirements for a proper pinned assumption of the joints. All failure modes are here ductile (fin plate or beam web in bearing).

8.5.2 CS/NS

Two different types of connections were calculated:

- Header plate;
- Fin plate.

A comparison between header plate and fin plate connections was performed for the joints of the perimeter beams (IPE450) and internal beams (IPE360) to the columns (HD360x162).

The summary of the results for the joints may be found in the Table 22.

Table 22. Verifications of joints at ULS, CS-NS

Position	Connection type	Shear resistance (kN)	Moment resistance (kNm)	Failure mode	UF
Perimeter	Header plate	289.38	-	Shear resistance of bolt group	0.73
	Fin plate	297.96	-	Shear resistance of bolt group	0.71
Internal	Header plate	289.38	-	Shear resistance of bolt group	0.64
	Fin plate	265.89	-	Plate bearing in supported beam web	0.70

8.5.3 SS/S and CS/S

Prequalified seismic moment resisting joints were adopted for the perimeter MRFs of both SS/S and CS/S. Extended end-plate joint configuration was preferred from the typologies available from the European RFCS EqualJoints project. Equal strength joints were chosen for MRFs and the joints adopted for the SS/S were also used for CS/S since no cross-sectional changes were made for the MRFs. Moreover, as the slab is considered totally disconnected from the steel frame in a circular zone around a column (see EN 1998-2), the composite character of beams with the slab was disregarded in the calculation of the joints.

For the other elements (beam-to-beam as well as beam-to-column, except the MRFs and the braced core) pinned joints were used. Angle cleats were used in both cases (SS/S and CS/S), with minor changers from one case to another.

The summary of the results for the moment resisting joints may be found in Table 23, while Table 24 provides the verification of pinned joints.

Table 23. Verifications of moment resisting joints at ULS, SS/S and CS/S

Position	Connection type	Moment resistance (kNm)	Shear resistance (kN)	Failure mode in bending	UF*	$\frac{M_{Rd}}{M_{pl,b}}$
A/1, A/7 IPE600-HEB550	Extended end-plate	1173	1516	End-plate in bending	0.29	0.94
A/1, A/7, A/2-6 IPE600-HEB500	Extended end-plate	1169	1387	End-plate in bending	0.26	0.94
1/A - 1/D IPE550-HEB500	Extended end-plate	957	1409	End-plate in bending	0.15	0.97
Note: * Utilization factor is defined for ULS, persistent design situation only						

Table 24. Verifications of pinned joints at ULS, SS/S and CS/S

Case	Position	Storey	Connection type	Shear resistance (kN)	Failure mode	UF*
SS/S	A/1-7, D/1-7 IPE550-IPE600	1-6	Cleat angle	196	Sec. beam bolts in shear	0.72
	B/1-7, C/1-7 IPE550-IPE550	1-6	Cleat angle	196	Sec. beam bolts in shear	0.72
	B/2, B/5, C/2, C/5 IPE550-HEM500	1-3	Cleat angle	196	Sec. beam at notch	0.67
	B/2, B/5, C/2, C/5 IPE550-HEB500	4-6	Cleat angle	196	Sec. beam bolts in shear	0.65
CS/S	A/1-7, D/1-7 IPE400-IPE600	1-6	Cleat angle	196	Sec. beam in bearing	0.90
	B/1-7, C/1-7 IPE400-IPE450	1-6	Cleat angle	196	Sec. beam in bearing	0.97
	B/2, B/5, C/2, C/5- IPE550-HEM500	1-3	Cleat angle	196	Sec. beam at notch	0.74
	B/2, B/5, C/2, C/5 IPE550-HEB500	4-6	Cleat angle	196	Sec. beam at notch	0.84
Note: * Utilization factor is defined for ULS, persistent design situation, only						

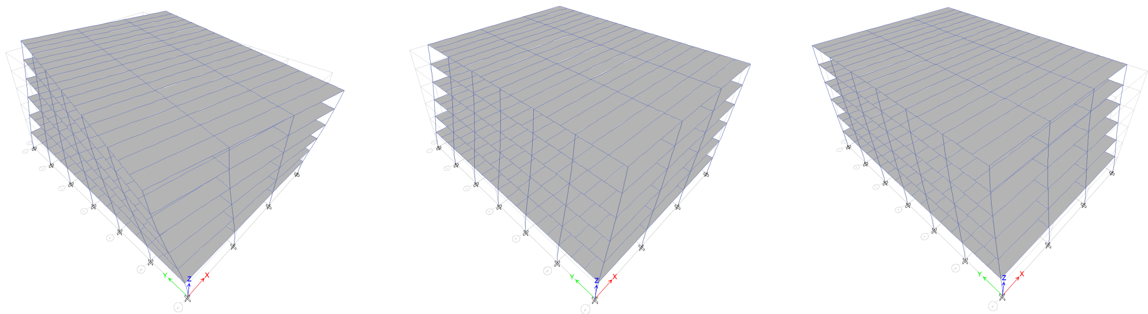
8.6 Comments on the final selection of the worked example structures

8.6.1 Seismic vs. non-seismic

The structural configurations were mainly designed to cover both seismic and non-seismic areas but keeping similar main structural schemes to allow for some direct comparisons in the design against accidental actions. Thus, same spans, bays, and storey heights were adopted. However, some adjustments were necessary for seismic resistant structures, i.e.:

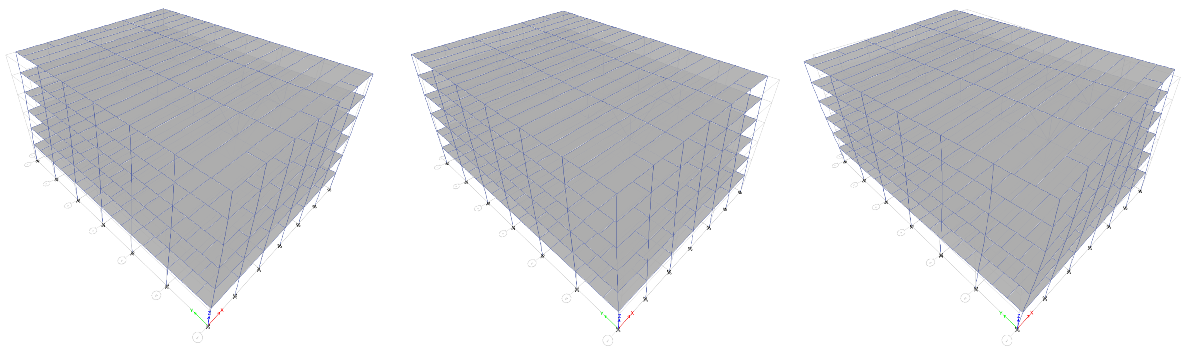
- The position of the braced spans close to the centre of rigidity (Figure 51a) makes the structure sensitive to torsional effects (Figure 53a). For seismic design, this is a feature to avoid, as it may cause collapse or heavy damages during earthquakes. As a result, the braced spans were moved to the exterior (Figure 51b) and additionally, MRFs were added on the perimeter on all sides. This resulted in a better response with first two translational modal shapes (Figure 53b).

- A dual steel frame seismic resistant system requires a minimum of 25% contribution from the MRFs to the total capacity (see EN 1998-2). To fulfil this requirement, the cross-sections of the beams and columns in the MRFs needed to be increased, and additionally, intermediate columns were introduced on the short sides (X) of the perimeter. The spans remained unchanged at the interior.



mode 1 mode 2 mode 3

a)



mode 1 mode 2 mode 3

b)

Figure 53. Modal shapes of the seismic resistant systems: a) initial, with a 1st torsional mode; b) after reconfiguration, with mode 1 and 2 translational

The condition that the effective modal mass should sum up to at least 90% of the total effective mass is fulfilled and the values are provided in Table 25 for the SS/S structure. The first mode is translation on X direction, the second is translation on Y direction, and in the third is torsion about Z axis, as presented in Figure 53b. The behaviour of CS/S structure (modal shapes) is very similar and the results are not presented.

Table 25. Modal parameters for SS/S structure

Case	Mode	Period [s]	SumUX	SumUY	SumRZ
Modal	1	0.769	0.7972	0	0
Modal	2	0.729	0.7972	0.7672	0
Modal	3	0.709	0.7972	0.7672	0.8153
Modal	4	0.271	0.9343	0.7672	0.8153
Modal	5	0.256	0.9343	0.9289	0.8153
Modal	6	0.25	0.9343	0.9289	0.9356
Modal	7	0.159	0.9692	0.9289	0.9356
Modal	8	0.147	0.9692	0.9289	0.9701
Modal	9	0.145	0.9692	0.9675	0.9701
Modal	10	0.113	0.9888	0.9675	0.9701
Modal	11	0.105	0.9888	0.9862	0.9701
Modal	12	0.105	0.9888	0.9862	0.9891

8.6.2 Steel vs. composite

The benefit of using composite beams yields in reduction of the cross-sections for the gravitational loading resisting system. For the LLRS of the structures in seismic zones, no changes in cross-sections were made. Thus, the same sections and UFs were obtained for the non-dissipative beams, columns, and perimeter beams for the MRF for the CS/S as in the case of SS/S structure. Moreover, as the loading remains the same (see Table 19), the second order effects for both structures are almost identical.

The composite systems were simply derived from the bare steel frames, by considering the composite action of the beams (for seismic resistant structures) and composite action of beams and floors (non-seismic structures). For the second category, a full composite structure was also designed by replacing the steel columns with equivalent composite steel-concrete columns. The interest for this structural choice was mainly for impact and accidental blast loading as it will be highlighted here after.

8.7 Identified exceptional events

8.7.1 Impact

8.7.1.1 Design for impact using equivalent static approach (CS/S)

FAIL NO MORE Worked example	Title	Design for impact using equivalent static approach			1 of 3 pages
	Structure	Composite structure in seismic zone	Made by	UPT	Date: 06/2021
	Document ref.	I.1.1 / CS/S			

Example: Design for impact of first storey perimeter columns in a composite structure in seismic zone using the equivalent static approach — Design manual § 4.2.2.1

This example gives information about the design against impact due to accidental collision of a vehicle.

Basic data of the structure — Design manual § 8.2.

- For geometry, sections, and materials, see Section 8.2.

Actions for Accidental Design Situation

The following actions are considered:

- Permanent loads DL (see Table 11);
- Live loads LL (see Table 11 for CS/S structure);
- Impact action A_{Ed} (see section below).

Combination of actions for Accidental Design Situation — EN 1990 §6.4.3.3, Eq 6.11b

The combination of actions is:

$$DL + 0.5 \times LL + A_{Ed}$$

Definition of impact scenarios

Impact scenarios include perimeter columns along traffic lanes. In this example, both long (along vertical traffic lane – see Figure 54) and short (along horizontal traffic lane – see Figure 54) facades are exposed.

The impact gives rise to a collision force that has components parallel and perpendicular to the direction of travel. In the design process, the two components can be considered as independent, i.e., the two components should not be applied at the same time.

Impact assumptions:

- Exposed columns: first floor (C1-C5 – see Figure 54 and Figure 55)
- Impact point height: 1.5m
- Impact forces (see Table 26)

The impact loads are calculated using data from Table 4.1 of (EN 1991-1-7 2006), considering the case: *Motorways and country national main roads.* — EN 1991-1-7 2006

Structural analysis

A **linear elastic analysis** is conducted on a full 3D model using SAP2000 software. The sections of the elements are those resulted from the initial design (persistent and seismic design situations). The acceptance criteria are given in terms of utilization factors (UFs) for accidental combinations only.

Worked example I.1.1 / CS/S	Design for impact using equivalent static approach – CS/S	2 of 3 pages

Table 26. Impact forces for linear static analysis – CS/S

Case	F_{dx} (kN)	F_{dy} (kN)
C1	1000	500
	500	1000
C2	1000	500
C3	1000	500
C4	1000	500

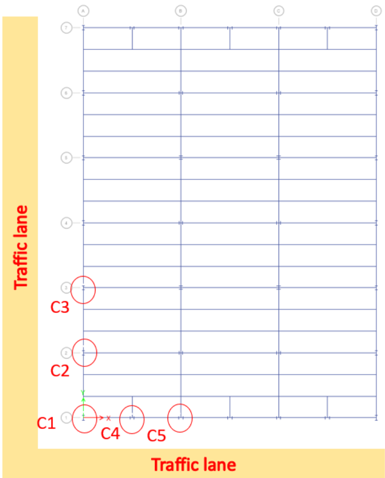


Figure 54. Road layout

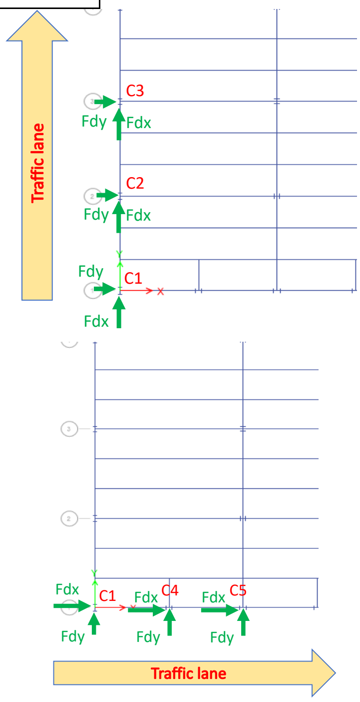


Figure 55. Plan views with direction of impact for each traffic lane

Results

Table 27. Results of linear static analysis

Case	Section	Impact force (kN)	Axis	Bottom support	*N* (kN)	*M* (kNm)	UF (-)	Critical impact force** (kN)
C1	HEB550	1000	Major	Fixed	1048	670	0.48	2700
		500	Minor	Fixed	1053	230	0.66	800
		500	Major	Fixed	*			
		1000	Minor	Fixed	1074	625	1.31	-
C2	HEB500	1000	Major	Fixed	2218	677	0.90	1250
		500	Minor	Fixed	2216	342	1.04	-
C3	HEB500	1000	Major	Fixed	2229	681	0.9	1250
		500	Minor	Fixed	2238	342	1.05	-
C4	HEB500	1000	Major	Fixed	591	755	0.63	1300
		500	Minor	Fixed	647	339	0.74	700
C5	HEB500	1000	Major	Fixed	1687	787	0.86	1800
		500	Minor	Fixed	1696	340	0.95	550

Worked example I.1.1 / CS/S	Design for impact using equivalent static approach – CS/S	3 of 3 pages

* The scenario is less demanding as the column was already verified for the same impact load applied according to the weak axis of the section. ** Impact force that causes the failure of the column (UF=1) *<u>Conclusions</u>* • Six out of nine impact scenarios satisfy the UF criterion, resulting in a proper design.	Flowchart Figure 3 – Box B.4 ➔ End of design

• Three out of nine impact scenarios result in capacity exceedance. However, the results may be conservative, as they are obtained using a simplified static analysis. Therefore, for the verifications that are not fulfilled using this approach, a capacity assessment with more sophisticated approaches may be used instead (see Worked Example (W.E.) I.1.2 / CS/S).
• To mitigate the impact, the hazard may be prevented or eliminated (see Section 4.2.1).
• In order to improve the design and response to impact load, other measures can be implemented:
  o Adopting higher steel grade for columns;
  o Orienting the columns (acording to their cross-sections's strog axis) to maximize the resistance to impact.
	Flowchart Figure 3 – Box B.5 ➔ Box B.II **OR** Box B.6

8.7.1.2 Design for impact using simplified dynamic approach (CS/S)

FAIL NO MORE Worked example	Title	Design for impact using simplified dynamic approach			1 of 3 pages
	Structure	Composite structure in seismic zone	Made by	UPT	Date: 06/2021
	Document ref.	I.1.2 / CS/S			

Example: Design for impact of first storey perimeter columns in a composite structure in seismic zone using the simplified dynamic analysis

Design manual § 4.2.2.2

This example gives information about the design against impact due to accidental collision of a vehicle.

Basic data of structure

Design manual § 8.2.

- For geometry, sections, materials, see Section 8.2.

Actions for Accidental Design Situation

The following actions are considered:

- Permanent loads DL (see Table 11);
- Live loads LL (see Table 11 for CS/S structure);
- Impact action A_{Ed} (see section below).

Combination of actions for Accidental Design Situation

EN 1990 §6.4.3.3, Eq 6.11b

The combination of actions is:

$$DL + 0.5 \times LL + A_{Ed}$$

Definition of impact scenarios

W.E. I.1.1 / CS/S

The impact scenarios include perimeter columns along traffic lanes, as previously defined in W.E. I.1.1 / CS/S. In this example, however, a single scenario is detailed, i.e., column C1 (UF = 1.31), minor axis impact, which has the highest UF according to the equivalent static approach design – see Table 26, W.E. I.1.1 / CS/S for the forces considered.

Structural analysis

A **nonlinear dynamic analysis** is conducted on a single column (isolated from the structure) using SAP2000 software.

The impact direction is along the weak axis, similar with the application of force F_{dx}, considering a vehicle speed and mass of v_r= 90 km/h and m=3.5 tons, respectively.

The column is made from HEB500, S355 steel, and is 4.0 m high. The column is studied in isolation and has the following boundary conditions:

- the column base is fixed;
- top of the column has all degrees of freedom fixed, except for the vertical displacement, which is unrestrained.

The analysis is performed in two steps:

1st step: vertical nodal load corresponding to the top of the column obtained from the static analysis in the accidental combination ($DL + 0.5 \times LL$) is applied statically as an axial compressive force.

Worked example I.1.2 / CS/S	Design for impact using simplified dynamic approach – CS/S	
2nd step: the impact force is applied transversally on the weak axis direction using a dynamic nonlinear analysis and hard impact approach as follows:		
<u>*Computation*</u> $\boldsymbol{F = v_r\sqrt{k \cdot m}}$		EN1991-1-7, formula C.1
where v_r is the impact velocity, m is the impact mass, and K – the stiffness of the impact object.		
The parameters are calculated considering the same type of road (*Motorways and country national main roads*):		
K= 300 kN/m = 300000 N/m		EN1991-1-7
v_r= 90 km/h = 25 m/s		Hypothesis
m = 3500 kg		Hypothesis
This results in: $F = v_r\sqrt{k \cdot m} = 25\sqrt{300000 \cdot 3500} = 810\ kN$		
***Note:** If the impact force is amplified by the DLF (recommended value of DLF = 1.4), for a vehicle velocity of 90 km/h (see Table C1 of EN1991-1-7) the equivalent dynamic impact force, F_{equiv}, is close to the one applied in the static analysis (see W.E I.1.1 / CS/S):* $F_{equiv} = 1.4 \cdot 25\sqrt{300000 \cdot 3500} = 1134.1\ kN$		Formula (4.1.5) from (Vrouwenvelder et al., 2005)
In the dynamic analysis, the force is applied using a ramp function with instant rise and a duration of: $\Delta t = \sqrt{k/m} = \sqrt{300000/3500} = 0.108\ \mathrm{s}$		EN1991-1-7
The total duration of the dynamic analysis is one second (larger than the ramp function duration Δt), to verify if the column remains stable after the ramp function ends.		SAP2000
The nonlinear behaviour is modelled using plastic hinges at each column end and at the point of impact using P-M2-M3 interaction. The plastic hinges are modelled using fibres.		
The effects of the fast impact loading (strain rate effects) are considered using a dynamic increase factor (DIF) applied to the strength of material.		
The DIF formulation for hot-rolled steel with yield strength up to 420 N/mm^2 can be expressed according to (CEB 1988) method.		
The strain rate ($\dot{\varepsilon}$) is obtained through an iterative procedure. In the first iteration, the ratio between the specific deformation and the time up to the point of yielding is computed based on the analysis results without applying a DIF. Afterwards, the analysis is performed again with the modified material properties by using a DIF, followed by DIF recalculation. If the new DIF values are comparable with the ones from the previous step (convergence), no further iterations are needed.		

Worked example I.1.2 / CS/S	Design for impact using simplified dynamic approach – CS/S	3 of 3 pages

$$\mathrm{DIF} = \frac{f_{dy}}{f_y} = 1 + \frac{6.0}{f_y} \ln \frac{\dot{\varepsilon}}{5 \times 10^{-5}}$$

$$\mathrm{DIF} = \frac{f_{du}}{f_u} = 1 + \frac{7.0}{f_u} ln \frac{\dot{\varepsilon}}{5 \times 10^{-5}}$$

At the end of the iterative process, one obtains DIF (f_y) = 1.118

Results

The column can sustain the impact force, but with incipient plastic deformations at the point of impact 0.054% normal strain, 0.073% at the bottom end and 0.036% at the top end of the column.

The figure below shows the lateral displacement time history of the column at the impact point. The peak horizontal displacement is 29.12 mm, with a residual deflection of 16.47 mm.

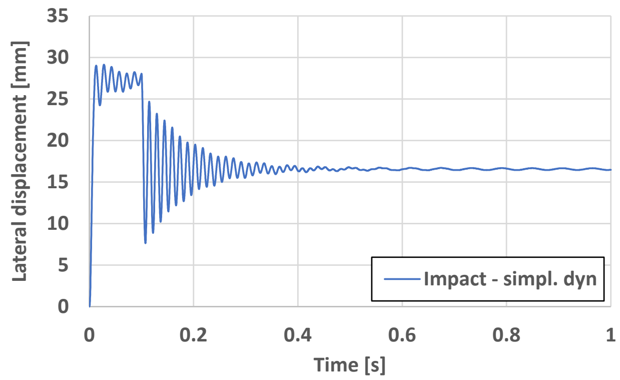


Figure 56. Lateral displacement time history at point of impact – CS S

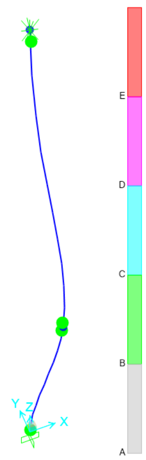


Figure 57. Plastic hinges – CS S

The application of equivalent static approach (W.E. I.1.1 / CS/S) indicated that the utilization factor exceeds unity ➔ redesign is needed. However, if plastic deformations are allowed to develop in the column, the design becomes **acceptable** by applying a *simplified dynamic approach* ➔ end of design.	Flowchart Figure 3 – Box B.6 ➔ End of design

8.7.1.3 Design for impact using full dynamic approach (CS/S)

FAIL NO MORE Worked example	Title	Design for impact using full dynamic approach			1 of 4 pages
	Structure	Composite structure in seismic zone	Made by	UPT	Date: 06/2021
	Document ref.	I.1.3 / CS/S			

Example: Design for impact of first storey perimeter columns in a composite structure in seismic zone using the full dynamic approach	Design manual § 4.2.2.3
This example gives information about the design against impact due to accidental collision of a vehicle.	
<u>*Basic data of structure*</u> • For geometry, sections, and materials, see Section 8.2.	Design manual § 8.2.
<u>*Actions for Accidental Design Situation*</u> The following actions are considered: • Permanent loads DL (see Table 11); • Live loads LL (see Table 11 for CS/S structure); • Impact action A_{Ed} (see section below).	
<u>*Combination of actions for Accidental Design Situation*</u> The combination of actions is:	
$DL + 0.5 \times LL + A_{Ed}$	EN 1990 §6.4.3.3, Eq 6.11b
<u>*Definition of impact scenarios*</u> • For definition of impact scenarios, see example W.E. I.1.1 / CS/S, with specific details reported in W.E. I.1.2 / CS/S. • The impact parameters are calculated considering the same type of road (*Motorways and country national main roads*):	W.E. I.1.1 / CS/S and W.E. I.1.2 / CS/S
K= 300 kN/m = 300000 N/m - stiffness of the impact object; v_r= 90 km/h = 25 m/s - impact velocity; m = 3500 kg - impacting mass.	EN 1991-1-7
<u>*Structural analysis*</u> To analyse a complex structural behaviour, such as an object collision followed by separation of elements and possible collapse, the impact with a vehicle was explicitly modelled. A nonlinear dynamic analysis was conducted on a full 3D model using the ELS software. ELS uses a nonlinear solver based on AEM (Tagel-Din and Meguro, 2000) and allows the automatic detection and computation of yielding, hardening, failure of materials, separation of elements, contact at impact, buckling/post-buckling, crack propagation, membrane action, and *P-Δ* effect. In the AEM modelling technique the structural elements are modelled as small solid elements (discretization is made both along the length of the member and of the cross-section) connected by normal and shear springs that follow the constitutive law of the corresponding material (including plastic	

Worked example I.1.3 / CS/S	Design for impact using full dynamic approach – CS/S	2 of 4 pages

behaviour, separation, contact). After reaching the separation strain, springs are removed. Then, if the separated elements come in contact, springs are generated at the surface of elements that are forced towards each other (Applied Science International, 2021).

Columns and beams were defined as solid objects with a constant I / H shape cross-section. The objects were discretized into small solid elements, generating 25 sets of springs at each surface. Link elements were used to model vertical braces and horizontal ties (anchored to perimeter columns). Beam-to-column joint properties were modelled with 8-node objects for end-plates and individual springs for each bolt. Pinned joints were defined by connecting the secondary beams with the main beams using just the springs representing the bolts. Column bases were considered as fixed. Reinforced concrete (RC) slabs were modelled as solid concrete elements with steel springs at the level of the reinforcement. Springs also simulated the connectors linking the beams to the RC slab.

To take into account the inertial effects, dead and live loads were assigned on the floors using lumped masses, which simulate better inertia effects in comparison to load assignments.

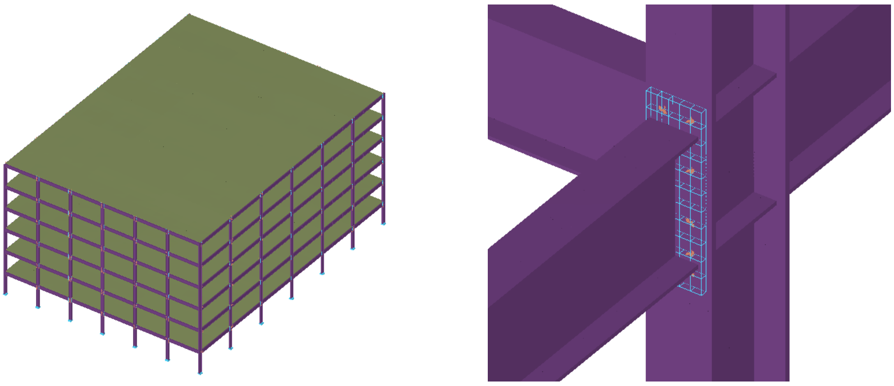
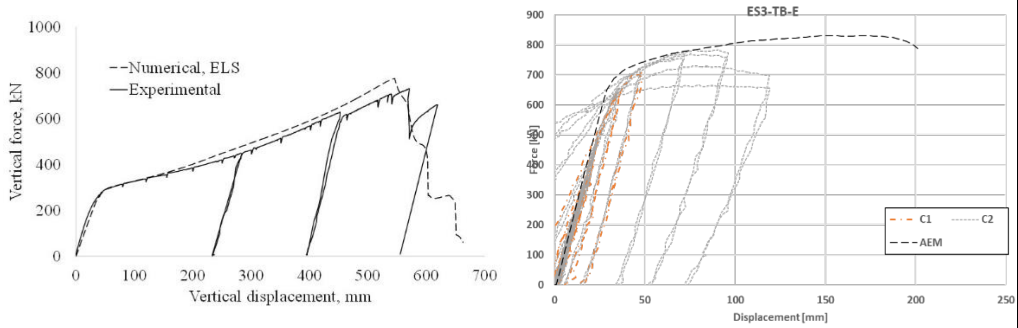


Figure 58. CS/S structure model (general view and connection detail)

To improve the accuracy of the AEM model, fine meshing was applied to the structural elements and joints which are contributing to the load redistribution capacity. The calibration was done against relevant experimental data from tests on subassemblies and joints (see Figure 59). Thus, Figure 59a shows the force-displacement curves in a column loss scenario from an experimental test and the corresponding numerical prediction in ELS, while Figure 59b shows the beam-to-column hysteretic and backbone curves from tests on joints. Based on these two comparisons, the accuracy of the numerical model in reproducing the structural response is considered adequate.

Worked example I.1.3 / CS/S	Design for impact using full dynamic approach – CS/S	3 of 4 pages



a) b)

Figure 59. System calibration on CODEC Tests and Connections calibration on Equaljoints Tests: a) force-displacement in a column loss scenario (Dinu et al., 2016); beam-to-column hysteretic and backbone curves (Landolfo et al., 2018)

The analysis is performed in two steps.

1st step: the permanent and live loads are applied on the structure in a static nonlinear analysis

2nd step: the impact body is colliding with the C2 column in a dynamic nonlinear analysis.

<u>Model assumptions for impact</u>

The impacting body (i.e., the vehicle) is allowed to slide on the horizontal plane only, at a height of 1.5 m, and has a mass of 3.5 tones. The initial velocity of the object is 25 m/s. The impacting body is composed of a contact plate, a plate with assigned mass, and axial springs between them. The height of the contact zone between the lorry and the column is considered as equal to 0.6 m. The stiffness of 300 kN/m is modelled through elastic springs.

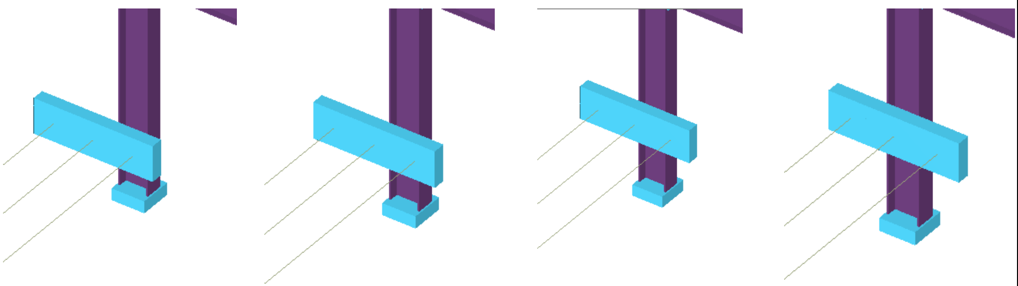


Figure 60. Collision object moving towards the column

Worked example I.1.3 / CS/S	Design for impact using full dynamic approach – CS/S	4 of 4 pages

Results

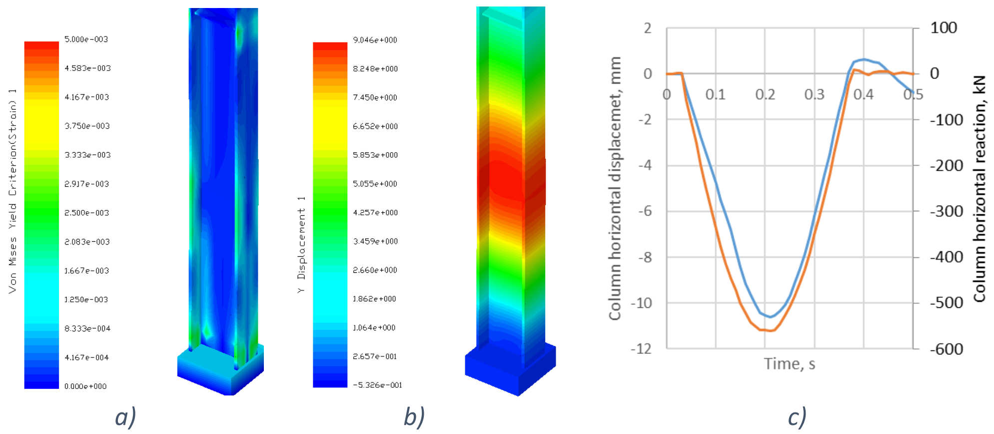


Figure 61. Results for impacted column: a) strains; b) deformations; c) horizontal base reaction force (orange) and horizontal displacement at impact point (blue)

The results show limited plastic deformations in the impacted column, with a maximum lateral deflection of 10.6 mm.

Conclusions

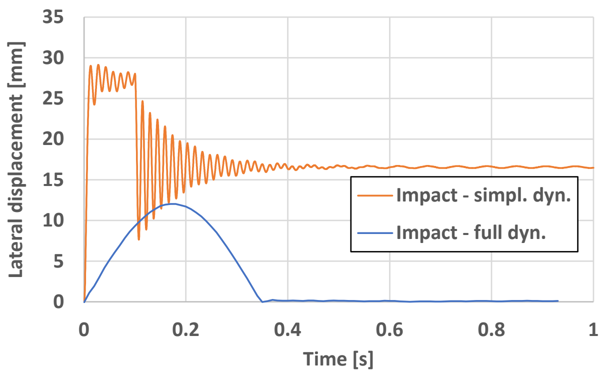


Figure 62. Lateral displacement in time – comparison of dynamic approaches

Compared with the W.E. I.1.2 / CS/S, full dynamic approach results in less deformation (as presented in Figure 62), as the restraining provided by the adjacent structure (especially the vertical restraining) is taken into account, and the “real” rise function of the impact force is less steep than the one applied for simplified dynamic approach.

Flowchart Figure 3 – Box B.6 ➔ End of design

Note that explicit consideration of *impact object-structure* interaction may result in much higher demands than typically considered in simplified dynamic analysis (Dubina et al., 2019).

8.7.1.4 Design for impact using equivalent static approach (CS/NS)

FAIL NO MORE Worked example	Title	Design for impact using equivalent static approach			1 of 3 pages
	Structure	Composite structure in non-seismic zone	Made by	AM	Date: 06/2021
	Document ref.	I.1.4 / CS/NS			

Example: Design for impact of first storey perimeter columns in a composite structure in non-seismic zone using the equivalent static approach

Design manual § 4.2.2.1

This example gives information about the design against impact due to accidental collision of a vehicle.

Basic data of structure

Design manual § 8.2.

- For geometry, sections, and materials, see Section 8.2.

Actions for Accidental Design Situation

The following actions are considered:

- Permanent loads DL (see Table 11);
- Live loads LL (see Table 11 for CS/NS structure);
- Impact action A_{Ed} (see section below).

Combination of actions for Accidental Design Situation

EN 1990 §6.4.3.3, Eq 6.11b

The combination of actions is:

$$DL + 0.5 \times LL + A_{Ed}$$

Definition of impact scenarios

Impact scenarios include perimeter columns along two traffic lanes (see Figure 63). In this example, both long façade and short façade are exposed.

The impact gives rise to a collision force that has components parallel and perpendicular to the direction of travel.

Each impact direction (short side – case A, long side – case B) results in two loading situations (according to the traffic flow) for the columns located at the ground floor of the building, i.e., one along the lane and one perpendicular to the lane. The location of the columns considered in the analysis is presented in Figure 63.

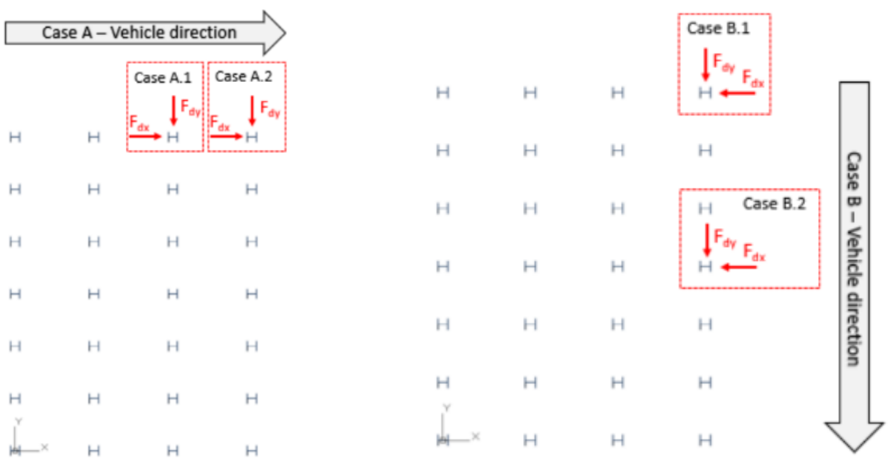


Figure 63. Plan view with columns layout, traffic lanes and position of columns under impact

Worked example I.1.4 / CS/NS	Design for impact using equivalent static approach – CS/NS	2 of 3 pages

The impact loads are calculated using table 4.1 of EN 1991-1-7, considering the case *Country roads in rural area.*

EN 1991-1-7, 2006

Table 28. Impact forces for linear static analysis CS/NS

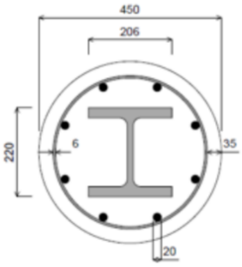
Case	F_{dx} (kN)	F_{dy} (kN)
A.1	750	375
A.2	750	375
B.1	375	750
B.2	375	750

Structural analysis

The **linear elastic analysis** is performed on a full 3D model using the software SCIA®. In a first step, the cross-sections of the members are those resulting from the initial design (persistent design situation). In a second step, the use of composite columns instead of steel ones is considered; the composite columns are designed using the software A3C®(see the selected cross-section here below). The acceptance criteria reported here are given in terms of utilization factors (UFs) for accidental combinations only.

Details of the composite columns:

- Steel section - HE200M
- Concrete class – C30/37
- Rebar (A500) – ϕ20 mm / ϕ6 mm



Results

Table 29. Results of linear static analysis for impact on steel columns

Case	Section	Loading		Bottom support	UF (-)	
		F_{dx} (kN)	F_{dy} (kN)		S355	S460
A.1	HD 360x162	750	375	Fixed	1.30	0.91
				Hinged	1.50	1.05
A.2	HD 360x162	750	375	Fixed	1.08	0.78
				Hinged	1.23	0.92
B.1	HD 360x162	375	750	Fixed	1.29	0.98
				Hinged	1.54	1.17
B.2	HD 360x162	375	750	Fixed	1.45	1.10
				Hinged	1.72	1.30

Table 30. Results of linear static analysis for impact on composite columns

Case	Loading		Upper and bottom supports	UF (-) S355
	F_{dx} (kN)	F_{dy} (kN)		
A.1	750	375	Hinged	2.63
A.2	750	375	Hinged	2.04
B.1	375	750	Hinged	2.25
B.2	375	750	Hinged	2.34

<table>
<tr><td>Worked example I.1.4 / CS/NS</td><td>Design for impact using equivalent static approach – CS/NS</td><td>3 of 3 pages</td></tr>
<tr><td colspan="2">
<u>Conclusions</u>
Standard steel columns
The results for the columns made of S355 steel grade show a surpass of the yield strength for both pinned and fixed conditions with UFs up to 1.72.
When using S460 steel grade, a considerable improvement is observed in terms of utilization factors.
Composite steel-concrete columns
Regarding the composite columns, the utilization factors are substantially higher. This is mainly related to the pre-design of the sections and supporting conditions. The columns were pre-designed considering the same capacity as the steel columns and pinned supports at both extremities (the steel cross-sections used for the composite elements are substantially smaller). When an impact load is applied (considering an equivalent static approach), the element will be subjected to bending which will be taken for the most part by the steel profile when it comes to the composite section (approximately 65% to 70%). Due to this, the composite columns show a higher utilization factor for impact analysis.
It is concluded that, for the non-composite steel columns, if the standard design is made considering around 60% to 65% utilitzation, the columns can still be able to sustain the impact load (static approach), assuming that the bottom supports remain fixed. For the sections that are failing using this approach, a capacity assessment with more sophisticated approaches should be made.
It is reminded that this example considered less demanding road conditions for impact with respect to W.E. I.1.1 / CS/S.
As shown previously, the main improvement that can be made is by increasing the steel grade to S460; by doing so, the columns have a better behaviour for the majority of cases. Other measures can also be implemented to improve the response to the impact load:

Orientate the columns (acording to their cross-sections's strong axis) to maximize the resistance to impact;
Increase the size of the sections;
Design the end-connections of the columns with higher stiffness and resistance (i.e., fixed (rigid) column bases);
Use of composite columns, to achieve an optimum solution in terms of size, used grade of steel, used concrete;

</td><td>
Flowchart Figure 3 – Box B.5 ➔ Box B.II OR Box B.6
Flowchart Figure 3 – Box B.5 ➔ Box B.II OR Box B.6
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</table>

8.7.2 Blast analysis

8.7.2.1 External blast

8.7.2.1.1 Design for external blast using equivalent single-degree-of-freedom approach (CS/NS)

FAIL NO MORE Worked example	Title	Design for external blast using equivalent SDOF approach			1 of 6 pages
	Structure	Composite structure in non-seismic zone	Made by	AM	Date: 06/2021
	Document ref.	I.2.1 / CS/NS			

Example: Design for external blast of perimeter columns of a composite structure in non-seismic zone using the equivalent SDOF approach

Design manual, § 4.3.2.2

This example gives information about the design against blast due to accidental external explosion.

Basic data of structure

Design manual, § 8.2.

- For geometry, sections, and materials, see Section 8.2 for the steel column solution and W.E. I.1.4 / CS/NS for the composite column.

Actions for Accidental Design Situation

The following action is considered:

- Blast action A_{Ed} (see section below).

Note: *No other loads are considered as acting on the column.*

Definition of blast scenario

The column considered in the analysis is a perimeter column located in the middle of the longest façade of the building – see Figure 64.

The blast scenario assumes that a car is placed at a standoff distance of 20 m from the column and carries an explosive charge equal to 100 kg of TNT (or equivalent).

The burst is defined as a free-air burst with a free height from the ground of 1 m.

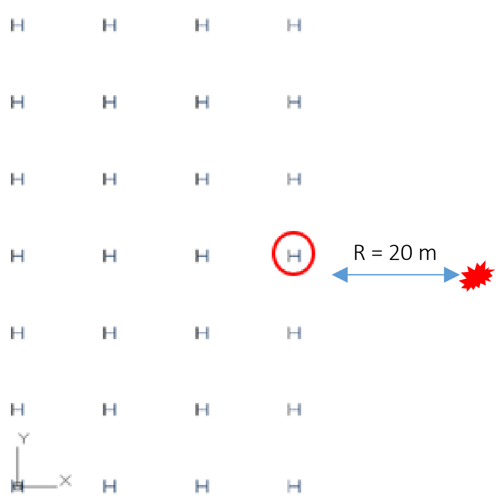


Figure 64. Plan view of the columns under blast load – CS/NS

Worked example I.2.1 / CS/NS	Design for external blast using equivalent SDOF approach – CS/NS	2 of 6 pages

Structural analysis

A **linear elastic analysis** is performed using the simplified dynamic approach as described in Section 4.3.2.2.

Computation

Structural blast loads

The first step consists in defining the peak dynamic pressure by calculating the scaled distance (*Z*), distance from blast source (R_h) and angle of incidence (α_i) according to the previously defined scenario.

TNT equivalent mass of the explosive charge	$W = 100\ kg$
Standoff distance	$R = 20\ m$
Height of the blast	$H_c = 1\ m$
Scaled distance	$Z = \frac{R}{W^{\frac{1}{3}}} = \frac{20}{100^{\frac{1}{3}}} = 4.309\ \frac{m}{kg^{\frac{1}{3}}}$
Distance from blast source	$R_h = \sqrt{R^2 + H_c^2} = \sqrt{20^2 + 1^2} = 20.025\ m$
Angle of incidence	$\alpha_i = tan^{-1}\left(\frac{H_c}{W^{\frac{1}{3}}}\right) = tan^{-1}\left(\frac{1}{100^{\frac{1}{3}}}\right) = 12.158^o$

By using the previous values, the data necessary to define the pressures and additional parameters are computed according to (Kingery and Bulmash 1984). Several other tools could be employed as well (i.e., (UN SaferGuard n.d.)) such as the graph provided in Figure 15 in Section 4.3.2.1.

Design manual, § 4.3.2.1, Figure 15

Incident pressure	$P_{so} = 56.44\ kPa$
Incident impulse	$I_s = 313.71\ kPa.ms$
Reflected pressure	$P_r = 137.37\ kPa$
Reflected impulse	$I_r = 688.09\ kPa.ms$
Time of arrival	$t_a = 30.29\ ms.W^{\frac{1}{3}} = 140.59\ ms$
Positive phase duration	$t_0 = 16.49\ ms$
Blast wavelength	$L_w = 0.4\ \frac{m}{kg^{\frac{1}{3}}}$
Shock front velocity	$U = 413.93\ \frac{m}{s}$

Worked example I.2.1 / CS/NS	Design for external blast using equivalent SDOF approach – CS/NS	3 of 6 pages

Note: *The difference between using the UN Saferguard website and Figure 15 from Section 4.3.2.1 is in the scaling of the parameters. Using the UN SaferGuard website, the values are already scaled ($W^{1/3}$). Only the wavelength was obtained from Figure 15 and it needed to be scaled. When Figure 15 is used, only the values for the time intervals, impulses, and wavelength need to be scaled (i.e., multiplied by $W^{1/3}$).*

Considering the incident pressure defined previously (P_{so}), the sound velocity (C_r) and the peak dynamic pressure (q) are obtained using the graphs: Figure 16 and Figure 17 from Section 4.3.2.1. — Design manual, § 4.3.2.1, Figure 16 and Figure 17

Sound velocity $C_r = 0.38\ \frac{m}{ms}$

Peak dynamic pressure $q = 8.5\ kPa$

Afterwards, the fictitious reduced time intervals need to be computed. This process is necessary since the blast wave and the formulation were initially defined for an infinite reflecting surface.

Fictitious positive phase duration $t_{0f} = 2\frac{I_s}{P_{so}} = 2 \times \frac{313.71}{56.44} = 11.12\ ms$

Fictitious duration for the reflected wave $t_{rf} = 2\frac{I_r}{P_r} = 2 \times \frac{688.09}{137.37} = 10.02\ ms$

Height of the element $h_s = 4\ m$ — Height of the column

Width of the wall $w_s = 4\ m$ — Assumption

Drag coefficient $C_D = 1$

Smallest dimension (height versus width) $s_d = \min\left(h_s, \frac{w_s}{2}\right) = \min\left(4, \frac{4}{2}\right) = 2\ m$

Largest dimension (height versus width) $l_d = \max\left(h_s, \frac{w_s}{2}\right) = \max\left(4, \frac{4}{2}\right) = 4\ m$

Ratio (smallest / largest) $r_{s.l} = \frac{s_d}{l_d} = \frac{2}{4} = 0.5$

Clearing time $t_c = \frac{4s_d}{(1 + r_{s.l})C_r} = \frac{4 \times 2}{(1 + 0.5) \times 0.38} = 14.04\ ms$

Peak pressure acting on the wall $P = P_{so} + q.C_D = 56.44 + 8.5 \times 1 = 64.94\ kPa$

Single degree of freedom approach (SDOF)

The first step in applying the SDOF method consists in calculating the uniformly distributed load (F_d) and point load (F_p) caused by the blast on the column.

Worked example I.2.1 / CS/NS	Design for external blast using equivalent SDOF approach – CS/NS	4 of 6 pages
Reflected pressure	$P_r = 137.37\ kPa$	
Height of the column	$h_c = 4\ m$	
Width of the panel in front of the column	$w_p = 5\ m$	
Fictitious duration of the reflected wave	$t_{rf} = 10.02\ ms$	
Self weight of the column (Steel; Composite)	$G_c = (1.834\ ;\ 4.721)\ \frac{kN}{m}$	
Distributed load from the blast on the column	$F_d = P_r w_p = 137.37 \times 5 = 686.85\ \frac{kN}{m}$	
Pont load from the blast on the column	$F_p = F_d h_c = 686.85 \times 4 = 2747.4\ kN$	
A first proposal of t_d/T = 2/3 (relation between reflected wave duration and period) is assumed such that a DLF may be considered using Figure 152 from Annex A.6.2.		Figure 152 §A.6.2
Dynamic load factor	$DLF = 1.45$	
The maximum moment corresponding to the load considering the DLF may be calculated, together with the different properties of the sections (see Table 67 from Annex A.6.1).		Table 67 §A.6.1
Loading factor	$K_L = 0.64$	
Mass factor	$K_M = 0.50$	
For the steel column:		
Plastic section modulus	$W_{pl.c} = 3162\ cm^3$	
Second moment of area	$I_c = 51890\ cm^4$	
For the composite column:		
Stiffness	$E.I_{eff} = 44350.87\ kNm^2$	
Maximum resistant moment	$M_{Rd.cp} = 632.85\ kNm$	
Dynamic increase factor	$DIF = 1.2$	
Steel yield strength	$f_y = 355\ MPa$	
Steel elastic modulus	$E = 210\ GPa$	
Column stiffness (Steel; Composite)		

Worked example I.2.1 / CS/NS	Design for external blast using equivalent SDOF approach – CS/NS	5 of 6 pages

$$K_c = \left(\frac{384E.I_c}{5h_c^3};\frac{384E.I_{eff}}{5h_c^3}\right) = \left(\frac{384\times210\times10^6\times51890\times10^{-8}}{5\times4^3};\frac{384\times44350.87}{5\times4^3}\right)$$

$$K_c = (130762.8\ ;\ 53221.04)\ \frac{kN}{m}$$

Maximum resistant moment (Steel; Composite)

$$M_{Rd} = \left(W_{pl.c}.f_y.DIF\ ;\ M_{Rd.cp}.DIF\right)$$

$$=(3162\times10^{-6}\times355\times10^3\times1.2\ ;\ \ 632.85\times1.2)$$

$$M_{Rd} = (1347.01\ ;\ 759.42)\ kNm$$

Maximum applied moment

$$M_{max} = \frac{F_p.h_c}{8}DLF = \frac{2747.4\times4}{8}\times1.45 = 1991.87\ kN.m$$

Effective mass (Steel; Composite)

$$M_e = \frac{G_c.h_c.K_M}{g} = \frac{(1.834\ ;\ 4.721)\times4\times0.50}{9.81} = (374.03\ ;\ 962.82)\ kg$$

Effective stiffness (Steel; Composite)

$$K_e = K_cK_L = (130762.8\ ;\ 53221.04)\times0.64 = (83688.19\ ;\ 34061.47)\ \frac{kN}{m}$$

Natural period of vibration (Steel; Composite)

$$T_c = 2\pi\sqrt{\frac{M_e}{K_e}} = 2\times\pi\sqrt{\frac{(374.03;962.82)}{(83688.19;34061.47)}} = (0.01;0.03)\ s$$

Ratio (Steel; Composite) $\frac{t_{rf}}{T_c} = \frac{10.02}{(13.28;33.41)} = (0.75\ ;\ 0.30)$

The new determined ratio allows for a second, more precise iteration.

Second interaction (Steel; Composite) $DLF = (1.30\ ;\ 1.80)$

Maximum applied moment (Steel; Composite)

$$M_{max} = \frac{F_p.h_c}{8}DLF = \frac{2747.4\times4}{8}\times(1.30\ ;\ 1.80) = (1785.81\ ;\ 2472.66)\ kNm$$

Resistance force (Steel; Composite)

$$R_m = \frac{8(2M_{Rd})}{h_c} = \frac{8\times2\times(1347.01\ ;\ 759.42)}{4} = (5388.05\ ;\ 3037.7)\ kN$$

Dynamic reaction (Steel; Composite)

$$V_m = 0.39R_m + 0.11F_p + G_c . h_c 0.5$$

$$V_m = 0.39 \times (5388.05\ ;\ 3037.7) + 0.11 \times 2747.4 + (1.834\ ;\ 4.721) \times 4 \times 0.5$$

$$V_m = (2407.22\ ;\ 1496.36)\ kN$$

Ratio (Steel; Composite) $\frac{R_m}{F_P} = \frac{(5388.05; 3037.68)}{2747.4} = (1.96\ ;\ 1.11)$

The ratio between the maximum resistance and the point load is used to determine the ductility demand from Figure 148 from Annex A.6.2. — Figure 148 from §A.6.2.

Results

Ratios (Steel; Composite) $\mu_1 = (0.80\ ;\ 0.95)$ — (χ_M/χ_E)

$\mu_2 = (0.55\ ;\ 1.2)$ — (t_m/T)

Yield displacement (Steel; Composite)

$$\chi_e = \frac{R_m}{K_e} = \frac{(5388.05\ ;\ 3037.7)}{(83688.19\ ;\ 34061.47)} = (64.38\ ;\ 89.18)\ mm$$

Maximum displacement (Steel; Composite)

$$\chi_M = \mu_1 \times \chi_e = (0.80\ ;\ 0.95) \times (64.38\ ;\ 89.18) = (51.51\ ;\ 84.72)\ mm$$

Maximum response time (Steel; Composite)

$$t_m = \mu_2 \times T_c = (0.55\ ;\ 1.2) \times (13.28\ ;\ 33.41) = (7.331\ ;\ 40.09)\ ms$$

The response limits in Table 5 Section 4.3.2.3 are used to evaluate the performance of a structural system / component. — Design manual, § 4.3.2.3, Table 5

$\mu_{max} = 1$ Flexure - > Beam - column with compact section -> B1 — Flowchart Figure 3 – Box B.5

Check (Steel; Composite) $\frac{\mu_1}{\mu_{max}} = (0.80\ ;\ 0.95)$ OK — Flowchart Figure 3 – Box B.6 ➔ End of design

Conclusions

According to the results, the steel and composite columns do not surpass the maximum response limits and both elements are able to withstand the blast load. The verification for superficial damage (class B1) was fulfilled.

8.7.2.1.2 Design for external blast using equivalent single-degree-of-freedom approach (SS/S)

FAIL NO MORE Worked example	Title	Design for external blast using equivalent SDOF approach			1 of 4 pages
	Structure	Steel structure in seismic zone	Made by	UPT	Date: 06/2021
	Document ref.	I.2.2 / SS/S			

Example: Design for external blast of a perimeter column of steel structure in seismic zone using the equivalent single-degree-of-freedom approach

Design manual § 4.3.2.2

This example gives information about the design against blast due to accidental external explosion.

Basic data of structure

Design manual § 8.2.

- For geometry, sections, and materials, see Section 8.2.

Actions for Accidental Design Situation

The following action is considered:

- Blast action A_{Ed} (see section below).

Note: No other loads are considered to act on the column.

Definition of blast scenario

The column considered in the analysis is a perimeter column located in the middle of the long façade of the building. The blast scenario assumes that a car is placed at a standoff distance of 20 m from the column and carries an explosive charge equal to 100 kg of TNT (or equivalent). The burst is defined as a free-air burst with a free height from the ground of 1m.

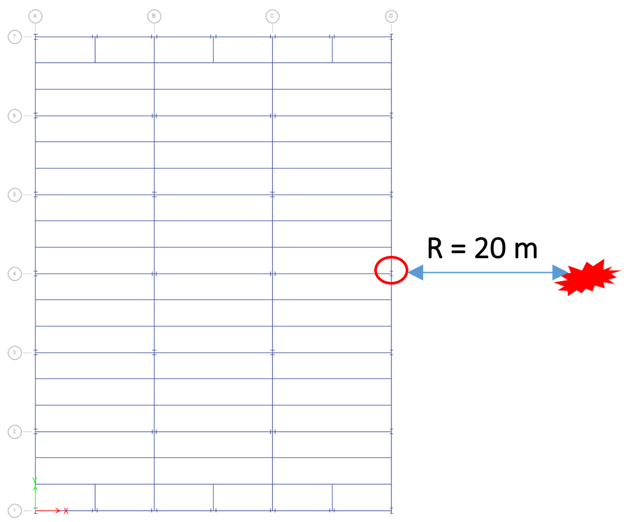


Figure 65. Plan view of the columns under blast load – SS S

Structural analysis

A **linear elastic analysis** is performed using the simplified dynamic approach following the procedure described below.

Worked example I.2.2 / SS/S	Design for external blast using equivalent SDOF approach – SS/S	2 of 4 pages

Computation

The blast loading parameters (incident pressure, incident impulse, reflected pressure, reflected impulse, time of arrival, positive phase duration, blast wavelength, shock front velocity) are identical to W.E. I.2.1/ CS/S**.** — W.E. I.2.1/ CS/S

Additionally, the parameters which follow (sound velocity, peak dynamic pressure, fictitious durations, clearing time) have the same values.

Single degree of freedom approach (SDOF)

The first step in applying the SDOF method consists in calculating the uniformly distributed load (F_d) and point load (F_p) caused by the blast on the column.

Reflected pressure	$P_r = 137.37\ kPa$	
Height of the column	$h_c = 3.5\ m$	Assumption

Note: If the rigid zone formed by the connection is accounted for, the height of the column can be assumed less than 4 m as effective length.

Width of the panel in front of the column	$w_p = 5m$
Self weight of the column	$G_c = 1.834\ \frac{kN}{m}$
Distributed load from the blast on the column	$F_d = P_r w_p = 137.37 \times 5 = 686.85\ \frac{kN}{m}$
Point load from the blast on the column	$F_p = F_d h_c = 686.85 \times 3.5 = 2404\ kN$
Dynamic load factor	$DLF = 1.4$
Loading factor	$K_L = 0.64$
Mass factor	$K_M = 0.50$
Plastic section modulus	$W_{pl.c} = 1292\ cm^3$
Second moment of area	$I_c = 12620\ cm^4$
Dynamic increase factor	$DIF = 1.2$

Yield strength affected by an amplification factor of 1.2 for the strain rate

Steel yield strength	$f_y = 355 \times 1.2 = 426\ MPa$
Steel elastic modulus	$E = 210\ GPa$

Worked example I.2.2 / SS/S	Design for external blast using equivalent SDOF approach – SS/S	3 of 4 pages

Column stiffness

$$K_c = \frac{384.I_c}{5h_c^3} = \frac{384 \times 210 \times 10^6 \times 12620 \times 10^{-8}}{5 \times 3.5^3} = 47472\ \frac{kN}{m}$$

Maximum resistant moment

$$M_{Rd} = W_{pl.c}.f_y.DIF = 1292 \times 10^{-6} \times 426 \times 10^3 = 550.4\ kNm$$

Maximum applied moment $\quad M_{max} = \frac{F_p.h_c}{8} DLF = \frac{2747.4 \times 3.5}{8} \times 1.4 = 1472\ kNm$

Effective mass $\quad M_e = \frac{G_c.h_c.K_M}{g} = \frac{1.834 \times 3.5 \times 0.50}{9.81} = 327.3\ kg$

Effective stiffness $\quad K_e = K_c K_L = 47471.8 \times 0.64 = 30382\ \frac{kN}{m}$

Natural period of vibration $\quad T_c = 2\pi \sqrt{\frac{M_e}{K_e}} = 2 \times \pi \sqrt{\frac{327.3}{30382}} = 0.0206$

Ratio between the fictious duration of the reflected wave and the natural period $\quad \frac{t_{rf}}{T_c} = 0.49$

The new determined ratio allows for a **second**, more precise **iteration**.

Second iteration $\quad DLF = 1.6$

Maximum applied moment $\quad M_{max} = \frac{F_p.h_c}{8} DLF = \frac{2747.4 \times 3.5}{8} \times 1.6 = 1683\ kNm$

Resistance force $\quad R_m = \frac{8(2M_{Rd})}{h_c} = \frac{8 \times 2 \times 550.4}{3.5} = 2516\ kN$

Dynamic reaction

$$V_m = 0.39R_m + 0.11F_p + G_c.h_c 0.5$$

$$V_m = 0.39 \times 2516 + 0.11 \times 2747.4 + 1.834 \times 3.5 \times 0.5 = 1248.92\ kN$$

Ratio $\quad \frac{R_m}{F_P} = 1.05$

The ratio between the maximum resistance and the point load is used to determine the ductility demand μ through Figure 148 from Annex A.6.2. — Figure 148 from §A.6.2.

Worked example I.2.2 / SS/S	Design for external blast using equivalent SDOF approach – SS/S	4 of 4 pages
Results		
Ratio	$\mu_1 = 1.05$ (χ_M/χ_E) $\mu_2 = 0.82$ (t_m/T)	
Yield displacement	$\chi_e = \frac{R_m}{K_e} = \frac{2516}{30382} = 82.82\,mm$	
Maximum displacement	$\chi_M = \mu_1 \times \chi_e = 1.05 \times 82.82 = 86.96\,mm$	
Maximum response time	$t_m = \mu_2 \times T_c = 0.82 \times 0.0206 = 16.90\,ms$	
Simplified dynamic approach (Pressure-impulse relationships)		Design manual, §4.3.2.3, Table 5
$\mu_{max} = 1$	Flexure - > Beam -column with compact section -> B1	Flowchart Figure 3 – Box B.5
Check	$\frac{\mu_1}{\mu_{max}} = 1.05$	Flowchart Figure 3 – Box B.6 ➔ End of design
Conclusions According to the results, the column can withstand the blast load (the value may be considered admissible), the requirement from class B1 (superficial damage) being fulfilled.		

8.7.2.1.3 Design for external blast using full dynamic approach (SS/S)

FAIL NO MORE Worked example	Title	Design for external blast using full dynamic approach			1 of 3 pages
	Structure	Steel structure in seismic zone	Made by	UPT	Date: 06/2021
	Document ref.	I.2.3 / SS/S			

Example: Design for external blast of perimeter columns of a steel structure in seismic zone using the full dynamic approach — Design manual § 4.3.2.4

This example gives information about the design against blast due to accidental external explosion.

Basic data of structure — Design manual § 8.2.

- For geometry, sections, and materials, see Section 8.2.

Actions for Accidental Design Situation

The following action is considered:

- Blast action A_{Ed} (see section below).

Definition of blast scenario — W.E. I.2.1/ CS/S

The column considered in the analysis is a perimeter column located in the middle of the long façade of the building.

Loading parameters:

- standoff distance = 20 m;
- explosive charge = 100 kg of TNT;
- tributary width of the column 5 m (2,5 m on each side);
- the blast pressure is considered to act on the 1st and 2nd storeys columns.

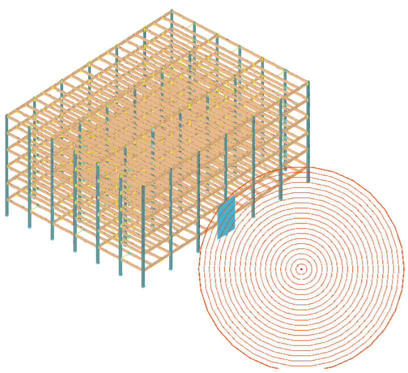


Figure 66. 3D model with the position of the charge

Note that, for a relevant comparison, the blast scenario considered in this example is the same with the one employed in W.E. I.2.1 / CS/S.

Structural analysis

The numerical analysis has been performed in ELS (Extreme Loading for Structure) software, using a full 3D model where the entire structure has been modelled.

Model assumptions in AEM – see W.E. I.1.3 / CS/S for details. — W.E. I.1.3 / CS/S

Worked example I.2.3 / SS/S	Design for external blast using full dynamic approach – SS/S	2 of 3 pages

To account for the tributary area loaded by the blast, rigid plates were modelled to transfer the pressure horizontally to the 1st and 2nd storey columns.

The blast loading parameters given below are computed automatically by the integrated blast pressure generator of ELS:

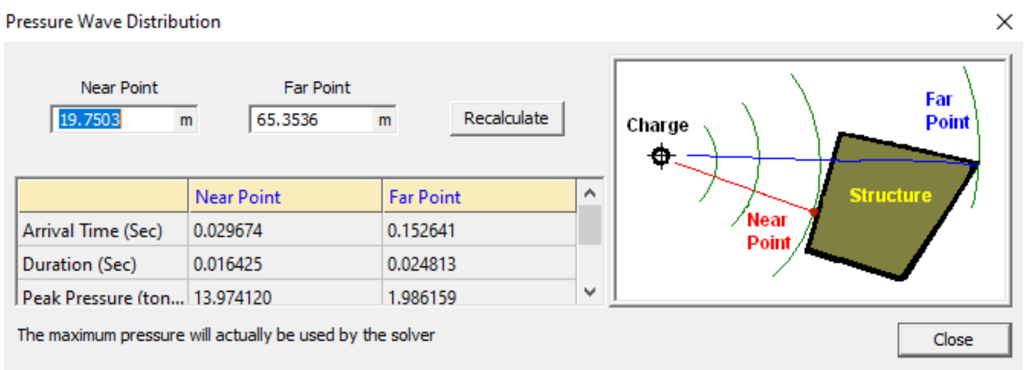


Figure 67. Blast parameters computed automatically in ELS software

The analysis is performed in two steps.

1st step: the permanent and live loads are applied on the structure in a nonlinear static analysis.

2nd step: the charge is detonated, and the blast load is applied in a nonlinear dynamic analysis. The time step for this analysis is $1E^{-6}$ s.

Only the positive phase of the blast is considered; no reflection from the ground is considered in the analysis.

__Results__

The maximum horizontal displacement at the mid-height of the column is 24 mm – see Figure 68 (left). The maximum reached plastic strain is 1%.

Flowchart Figure 3 – Box B.5

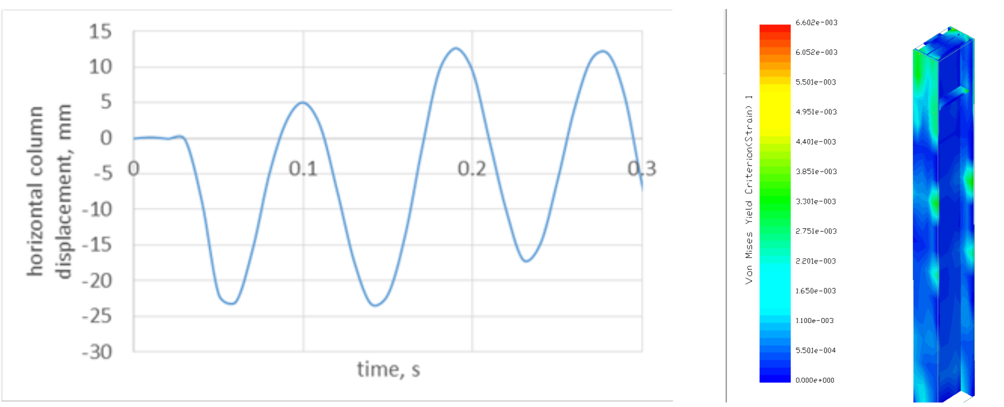


Figure 68. Horizontal deformation vs time at column mid-height (left) and Von Mises strains (right)

<table>
<tr><td>Worked example I.2.3 / SS/S</td><td>Design for external blast using full dynamic approach – SS/S</td><td>3 of 3 pages</td></tr>
<tr><td colspan="2"><u>Conclusions</u>

Full dynamic approach vs. equivalent SDOF approach:
The displacement in the full nonlinear dynamic analysis is less than the value obtained using tabular method (24 mm vs. 87 mm, see W.E. I.2.2 / SS/S);
Nonlinear analysis can account for distribution of plasticity in the element;
Full 3D model can account for real boundary conditions and interactions between elements;
Full dynamic approach and 3D modelling can account for sequential application of blast pressure on the surface (different arrival times along the column length).
Note that, in case of near field blasts, the effects can be amplified by the uplift pressure against the adjoining floors, which can result in higher dynamic effects and even risk of progressive collapse (Dinu et al. 2018).</td><td>Flowchart Figure 3 – Box B.6 ➔ End of design</td></tr>
</table>

8.7.2.2 Internal explosions

8.7.2.2.1 Design for internal explosions using equivalent static approach (SS/S)

FAIL NO MORE Worked example	Title	Design for internal explosions using equivalent static approach			1 of 3 pages
	Structure	Steel structure in seismic zone	Made by	UPT	Date: 06/2021
	Document ref.	I.3.1 / SS/S			

Example: Design for internal explosions on columns of a steel structure in seismic zone using equivalent static approach

Design manual § 4.3.3.2

This example gives information about the design against internal blast due to accidental internal gas explosion.

§ 5.4, EN1991-1-7

<u>Basic data of structure</u>

Design manual § 8.2.

- For geometry, sections, and materials, see section 8.2.

<u>Actions for Accidental Design Situation</u>

The following actions are considered:

- Permanent loads DL (see Table 11);
- Live loads LL (see Table 11 for SS/S structure);
- Gas pressure A_{Ed} (see section below).

<u>Combination of actions for Accidental Design Situation</u>

EN 1990 §6.4.3.3, Eq 6.11b

The combination of actions is:

$$DL + 0.5 \times LL + A_{Ed}$$

<u>Definition of gas explosion scenario</u>

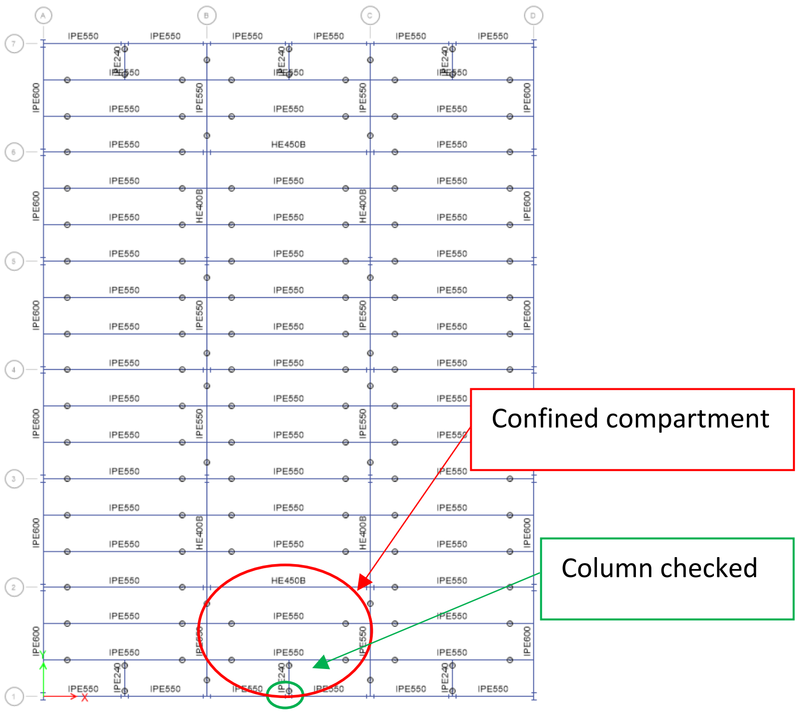


Figure 69. Position of the confined compartment and checked column – SS/S

Worked example I.3.1 / SS/S	Design for internal gas explosions using equivalent static approach – SS/S	2 of 3 pages

The compartment is located at the ground floor. The venting surface is considered on the external wall and is made of glass windows, while the other 3 internal walls are made of stronger materials. The column considered for the verification is circled in green in Figure 69.

Computation

Table 31. Geometry of the compartment

L	12	m	length
B	8	m	width
H	4	m	height
A_v	48	m^2	venting area
V	384	m^3	compartment volume

The venting area and volume of the enclosure were computed considering that the glass wall is placed on the enclosure of the building and on the entire storey height. After succesfully checking that the pressure model from EN 1991-1-7 can be applied for the current example (limits function of the venting area and volume of the enclosure), the following equivalent static pressure for the internal gas explosion was obtained:

$$p_d = 3 + p_{stat}$$

or

$$p_d = 3 + \frac{p_{stat}}{2} + \frac{0.04}{(A_v/V)^2}$$

whichever is the greater.

It was assumed that $p_{stat} = 3\ kN/m^2$, which represents the static uniformly distributed pressure at which venting components fail.

Consequently, the design pressure in case of accidental situation is:

$p_d = 7.06\ kN/m^2$

Hereinafter, the pressure was applied as a linear load acting on the height of the column considering a tributary width of 6 m.

Structural analysis

A **linear elastic analysis** is conducted on a full 3D model using SAP2000 software. The sections of the elements are those resulted from the initial design (persistent and seismic design situations). The acceptance criteria are given in terms of utilization factors (UFs) for accidental combinations only.

Results

The results of the linear static analysis of the column is presented in Table 32.

Worked example I.3.1 / SS/S	Design for internal gas explosions using equivalent static approach – SS/S	3 of 3 pages

Table 32. Results of linear static analysis

Section	Axis	Bottom support	*N* (kN)	*M* (kNm)	UF (-)	Lateral deflection (mm)
HEB500	Minor	Fixed	612	72	0.279	0.57

Conclusions

The column analysed with this approach does not exceed its capacity and does not require a redesign. However, since no local damage occurs, more sophisticated approaches may be used to quantify the damage that might appear.

Flowchart Figure 3 – Box B.4→ End of design

8.7.2.2.2 Design for internal explosions using dynamic approach – TNT equivalence method (SS/S)

Worked example	Title	Design for internal explosions using dynamic approach			1 of 6 pages
	Structure	Steel structure in seismic zone	Made by	UPT	Date: 06/2021
	Document ref.	I.3.2 / SS/S			

Example: Design for internal gas explosion of a steel structure in seismic zone using dynamic approach – TNT equivalence method

Design manual § 4.3.3.3

This example gives information about the design against internal gas explosions.

Under specific conditions, an internal gas explosion may be approximated with an equivalent TNT explosion (Bjerketvedt et al., 1997b).

(Bjerketvedt et al., 1997b), (DoD, 2008)

Note: *The procedure proposed to solve this case is a simplification of the actual procedure. The volume of the gas is replaced in the computations with an equivalent TNT charge. Onwards, the procedure applied in case of external blast, as described in W.E. I.2.1 / CS/NS and I.2.2 / SS/S is used for this example also. Thus, the effect of the frangibility of the walls, pressure leakage from the compartment etc. are neglected. However, a very complex procedure based on the recommendations from (DoD, 2008) is shown in the Deliverable D2-2 of the FAILNOMORE project available on the following website:* https://www.steelconstruct.com/eu-projects/failnomore/.

<u>*Basic data of structure*</u>

Design manual § 8.2.

- For geometry, sections, and materials, see Section 8.2.

<u>*Actions for Accidental Design Situation*</u>

The following action is considered:

- Gas action A_{Ed} (see section below).

Note: No other loads are considered to act on the column.

<u>*Definition of gas explosion scenario*</u>

For the internal explosion scenario, a 48 m^3 compartment and 6% methane concentration in the air were assumed.

<u>*Computation*</u>

Equivalent TNT mass

(DoD, 2008)

According to section 4.3.3.3, the following definition of the equivalent TNT charge may be used:

§4.3.3.3 Relation (15) and (16)

$$W_{TNT} = \eta \frac{W_g \times E_c}{E_{TNT}}$$

where:

η	0.2	[-]	energy release rate
E_c	55	MJ/kg	heat of methane
W_g	1.91	kg	total leakage of fuel
E_{TNT}	4.2	MJ/kg	detonation heat of TNT

The total leakage of fuel (W_g) *was computed using the following formula:*

$$W_g = V_{enclosure} \cdot \gamma_{methane} \cdot 6/100$$

To compute the mass of the gas, the volume of the enclosure was considered 48 m^3, as stated in the scenario, and the specific weight of the methane was 0.668 kg/m^3. Consequently, a mass of 1.91 kg was obtained for the methane, which was replaced by an equivalent mass $W_{TNT} = 5.0\ kg$ of TNT.

<u>*Structural analysis*</u>

A **linear elastic analysis** is performed using the simplified dynamic approach following the procedure described previously in W.E. I.1.1 / CS/NS.

The charge is assumed to be placed in the middle of the compartment allowing for a 4 m standoff distance on the transversal direction.

TNT equivalent mass of the explosive charge	$W = 5\ kg$
Standoff distance	$R = 4\ m$
Height of the blast	$H_c = 1\ m$
Scaled distance	$Z = \frac{R}{W^{\frac{1}{3}}} = \frac{4}{5^{\frac{1}{3}}} = 2.339\ \frac{m}{kg^{\frac{1}{3}}}$
Distance from blast source	$R_h = \sqrt{R^2 + H_c^2} = \sqrt{4^2 + 1^2} = 4.123\ m$
Angle of incidence	$\alpha_i = tan^{-1}\left(\frac{H_c}{W^{\frac{1}{3}}}\right) = tan^{-1}\left(\frac{1}{5^{\frac{1}{3}}}\right) = 30.32^o$
Blast parameters	
Incident pressure	$P_{so} = 198.87\ kPa$
Incident impulse	$I_s = 198.46\ kPa.ms$
Reflected pressure	$P_r = 663.44\ kPa$
Reflected impulse	$I_r = 514.65\ kPa.ms$

Worked example I.3.2 / SS/S	Design for internal gas explosions using dynamic approach – TNT equivalence method – SS/S	3 of 6 pages
Time of arrival	$t_a = 3.87ms.W^{\frac{1}{3}} = 6.62\ ms$	
Positive phase duration	$t_0 = 3.7ms.W^{\frac{1}{3}} = 6.33\ ms$	
Blast wavelength	$L_w = 0.85\ \dfrac{m}{kg^{\frac{1}{3}}}$	
Shock front velocity	$U = 557.06\ \dfrac{m}{s}$	
Sound velocity	$C_r = 0.47\ \dfrac{m}{ms}$	
Peak dynamic pressure	$q = 100\ kPa$	
Fictitious positive phase duration	$t_{0f} = 2\dfrac{I_s}{P_{so}} = \dfrac{396.96\ kPa.ms}{198.87\ kPa} = 1.996\ ms$	
Fictitious duration for the reflected wave	$t_{rf} = 2\dfrac{I_r}{P_r} = \dfrac{1029.3\ kPa.ms}{663.44\ 7kPa} = 1.551\ ms$	
Height of the element	$h_s = 4\ m$	Column in analysis
Width of the wall	$w_s = 4\ m$	Assumption
Drag coefficient (wall)	$C_D = 1$	
Smallest dimension (height versus width)	$s_d = \min\left(h_s, \dfrac{w_s}{2}\right) = \min\left(4, \dfrac{4}{2}\right) = 2\ m$	
Largest dimension (height versus width)	$l_d = \max\left(h_s, \dfrac{w_s}{2}\right) = \max\left(4, \dfrac{4}{2}\right) = 4\ m$	
Ratio (smallest / largest)	$r_{s.l} = \dfrac{s_d}{l_d} = \dfrac{2}{4} = 0.5$	
Clearing time	$t_c = \dfrac{4s_d}{(1 + r_{s.l})C_r} = \dfrac{4 \times 2}{(1 + 0.5)0.47} = 11.348\ ms$	
Peak pressure acting on the wall	$P = P_{so} + q.C_D = 198.87 + 100 \times 1 = 298.87\ kPa$	
Single degree of freedom approach (SDOF)		
Reflected pressure	$P_r = 663.44\ kPa$	
Fictitious duration of the reflected wave	$t_{rf} = 1.551\ ms$	

Worked example I.3.2 / SS/S	Design for internal gas explosions using dynamic approach – TNT equivalence method –SS/S	4 of 6 pages

Height of the column $h_c = 3.5\ m$

Width of the panel in front of the column $w_p = 4\ m$

Self-weight of the column $G_c = 1.834\ \frac{kN}{m}$

Distributed load from the blast on the column $F_d = P_r w_p = 663.44 \times 4 = 2653.76\ \frac{kN}{m}$

Pont load from the blast on the column $F_p = F_d h_c = 2653.76 \times 3.5 = 9288.2\ kN$

Dynamic load factor $DLF = 1.4$

Loading factor $K_L = 0.64$

Mass factor $K_M = 0.50$

Plastic modulus $W_{pl.c} = 1292\ cm^3$

Inertia $I_c = 12620\ cm^4$

Dynamic increase factor $DIF = 1.2$

Steel yield strength affected by an amplification factor of 1.2 for the strain rate

Steel yield strength $f_y = 355 \times 1.2 = 426\ MPa$

Steel elastic modulus $E = 210\ GPa$

Column stiffness

$$K_c = \frac{384E.I_c}{5h_c^3} = \frac{384 \times 210 \times 10^6 \times 12620 \times 10^{-8}}{5 \times 3.5^3} = 47472\ \frac{kN}{m}$$

Maximum resistant moment

$$M_{Rd} = W_{pl.c}.f_y.DIF = 1292 \times 10^{-6} \times 426 \times 10^3 = 550.4\ kNm$$

Max. applied moment $M_{max} = \frac{F_p.h_c}{8} DLF = \frac{9288.2 \times 3.5}{8} \times 1.4 = 5689\ kNm$

Effective mass $M_e = \frac{G_c.h_c.K_M}{g} = \frac{1.834 \times 3.5 \times 0.50}{9.81} = 327.3\ kg$

Effective stiffness $K_e = K_c K_L = 47471.8 \times 0.64 = 30382\ \frac{kN}{m}$

Natural period of vibration $T_c = 2\pi \sqrt{\frac{M_e}{K_e}} = 2 \times \pi \sqrt{\frac{327.3}{30382}} = 0.0206$

Worked example I.3.2 / SS/S	Design for internal gas explosions using dynamic approach – TNT equivalence method – SS/S	5 of 6 pages

Ratio $\dfrac{t_{rf}}{T_c} = 0.08$

Second interaction $DLF = 1.9$

Maximum applied moment $M_{max} = \dfrac{F_p.h_c}{8} DLF = \dfrac{9288.2 \times 3.5}{8} \times 1.9 = 7721\ kNm$

Resistance force $R_m = \dfrac{8(2M_{Rd})}{h_c} = \dfrac{8 \times 2 \times 550.4}{3.5} = 2516\ kN$

Dynamic reaction

$$V_m = 0.39R_m + 0.11F_p + G_c.h_c0.5$$

$$V_m = 0.39 \times 2516 + 0.11 \times 9288.2 + 1.834 \times 3.5 \times 0.5 = 2006.18\ kN$$

Ratio $\dfrac{R_m}{F_P} = 0.27$

__Results__

Ratio $\mu_1 = 0.9$ (χ_M/χ_E)

$\mu_2 = 3.6$ (t_m/T)

Yield displacement $\chi_e = \dfrac{R_m}{K_e} = \dfrac{2516}{30382} = 82.82\ mm$

Maximum displacement $\chi_M = \mu_1 \times \chi_e = 0.9 \times 82.82 = 74.35\ mm$

Maximum response time $t_m = \mu_2 \times T_c = 0.80 \times 0.0206 = 74.24\ ms$

Simplified dynamic approach (Pressure-impulse relationships) — Design manual, §4.3.2.3, Table 5

$\mu_{max} = 1$ Flexure - > Beam - column with compact section -> B1 — Flowchart Figure 3 – Box B.5

Check $\dfrac{\mu_1}{\mu_{max}} = 0.9$ OK — Flowchart Figure 3 – Box B.6 ➔ End of design

Worked example I.3.2 / SS/S	Design for internal gas explosions using dynamic approach – TNT equivalence method – SS/S	6 of 6 pages
Conclusions Using the TNT equivalent method, a more detailed analysis was performed. According to the equivalent static approach (W.E. I.3.1 / SS/S), the column remained with a UF of 0.28, meaning that there was no local damage.		Flowchart Figure 3 – Box B.4 ➔ End of design
However, using equivalent TNT method, local damage occurs, but it is not critical for the stability of the structure. To mitigate the effects of an accidental gas explosion, several measures may be implemented – see Section 4.3.1.		Flowchart Figure 3 – Box B.6 ➔ End of design

8.7.3 Localised fire analysis

8.7.3.1 Design for internal localised fire using localised fire models (CS/NS)

FAIL NO MORE Worked example	Title	Design for localised fire using localised fire models			1 of 2 pages
	Structure	Composite structure in non-seismic zone	Made by	AM	Date: 06/2021
	Document ref.	I.4.1 / CS/NS			

Example: Design for localised fire on columns of a composite structure in non-seismic zone using localised fire models

Design manual § 4.4.2.1

This example gives information about the design against fire in case of accidental situation.

Basic data of structure

Design manual § 8.2.

- For geometry, sections, and materials, see Section 8.2.

Actions for Accidental Design Situation

The following actions are considered:

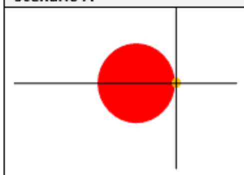
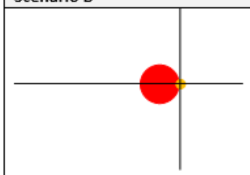
- Permanent loads DL (see Table 11);
- Live loads LL (see Table 11 for CS/NS structure);
- Fire A_{Ed} (see section below).

Definition of localised fire scenarios

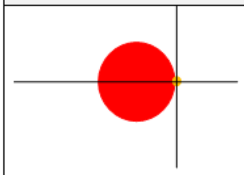
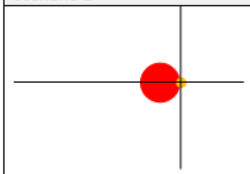
In this analysis, four scenarios are defined, starting from a baseline scenario considering standard values for an office building. The three other scenarios assume "*exaggerated values*": either for the rate of heat release (a double value of 500 kW/m^2) or for the fire load density and the fire growth rate (values for the "commercial area" occupancy, which are more severe than for office buildings).

(Brasseur et al. 2018), EN1991-1-2

Together with the previous assumptions, two realistic fire basis diameters are considered: 1 m and 2 m. For all scenarios, a safe-sided assumption is made, considering that the localised fire is placed just next to the column, i.e., there is a null distance between the exterior of the fire circular basis and the column.

Scenario A		
	Diameter of the fire basis	2 m
	Rate of Heat Release density	250 kW/m^2 (office building EN 1991-1-2)
	Fire load density	511 MJ/m^2 (office building EN 1991-1-2)
	Fire growth rate	300 sec (office building EN 1991-1-2)
Scenario B		
	Diameter of the fire basis	1 m
	Rate of Heat Release density	500 kW/m^2
	Fire load density	511 MJ/m^2 (office building EN 1991-1-2)
	Fire growth rate	300 sec (office building EN 1991-1-2)

Worked example I.4.1 / CS/NS	Design for localised fire using localised fire models – CS/NS	2 of 2 pages

Scenario C		
	Diameter of the fire basis	2 m
	Rate of Heat Release density	250 kW/m² (commercial area EN 1991-1-2)
	Fire load density	730 MJ/m² (commercial area EN 1991-1-2)
	Fire growth rate	150 sec (commercial area EN 1991-1-2)
Scenario D		
	Diameter of the fire basis	1 m
	Rate of Heat Release density	500 kW/m²
	Fire load density	730 MJ/m² (commercial area EN 1991-1-2)
	Fire growth rate	150 sec (commercial area EN 1991-1-2)

Structural analysis

For each scenario, the software OZone® (Cadorin, 2003) is used, applying the LOCAFI (Brasseur et al., 2018) model as well as the equations from EN 1991-1-2, to evaluate the steel temperatures of a bare steel column made of an unprotected hot rolled profile HEB340 (as an example).

Conclusions

The maximum steel temperatures along the height of the column for the 4 scenarios were calculated and compared (see Figure 70). This comparison highlights that, although different assumptions are made to characterize the localised fire, the same trend and order of magnitude are achieved. Significant temperatures develop at the bottom of the steel columns which can cause buckling or a local plastic failure.

Flowchart Figure 3 – Box B.5

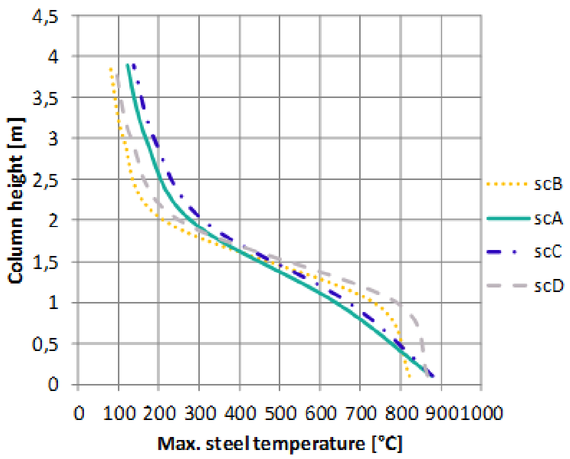


Figure 70. Steel temperature variation on column height

In another approach performing a full numerical analysis, a study was made where specific columns were removed and the building behaviour evaluated (Alternative load path method), see example II.4.6/ CS-NS.

In order to avoid fire damage, fire protection can be used instead of designing the structural elements for specific fire resistance or increase the size of the section.

8.7.4 Seismic analysis

8.7.4.1 Seismic design using prescriptive method (SS/NS)

FAIL NO MORE Worked example	Title	Seismic design using prescriptive method			1 of 1 pages
	Structure	Steel structure in non-seismic zone	Made by	F+W	Date: 06/2021
	Document ref.	I.5.1 / SS/NS			

Example: Design recommendations for steel structures in non-seismic zone (prescriptive method) This example gives recommendations about the application of prescriptive measures for improving the response of non-seismically designed steel structures in case of exceptional earthquakes.	Design manual § 4.5
Basic data of structure • For geometry, sections, and materials, see Section 8.2. The structure considered in this example has been designed for ULS/SLS conditions only (persistent design situation). No particular calculations have been conducted with respect to any accidental seismic action. So here, the seismic action is considered as exceptional. In practice, simple recommendations as proposed in Section 4.5.2 can be followed when the seismic action is less critical than the wind-based design. This is mainly done for low-rise buildings to optimize engineering costs. *Remarks* • Due to the symmetry in plan and regularity in elevation, the structure stiffness is well distributed, thus offering a favourable response to the seismic action. • Equal floor heights also contribute to the good behaviour of the structure in case of earthquakes. • Ductility requirements: 1. To increase the overall ductility of the structure, the HEA300 beams can be replaced with HEB300 ones, as HEA300 S355 profiles are class 3 and HEB300 S355 are class 1. All the other members are already class 1 profiles. 2. To optimize the structure response, the originally designed pinned joints could be replaced by ductile semi-rigid joints as described in Section 8.8.4.1 where the alternative load path method is applied (W.E. II.4.1 / SS/NS). This would allow the formation of plastic hinges in the joints and dissipate part of the seismic induced energy.	Design manual § 8.2.

8.7.4.2 Seismic design using advanced numerical analysis (multi-hazard) (SS/S)

FAIL NO MORE Worked example	Title	Seismic design using advanced numerical analysis (multi-hazard)			1 of 4 pages
	Structure	Steel structure in seismic zone	Made by	UPT	Date: 06/2021
	Document ref.	I.5.2 / SS/S			

Example: Design of a steel structure for multi-hazard scenarios using advanced numerical analysis

Design manual § 4.5

This example gives information about the design of a steel structure considering multi-hazard events, i.e., column failure after an earthquake.

Basic data of structure

Design manual § 8.2.

- For geometry, sections, and materials, see Section 8.2.

Actions for Seismic Design Situation

The following actions are considered:

- Permanent loads DL (see Table 11);
- Live loads LL (see Table 11 for SS/S structure);
- Seismic action A_{Ed} corresponding to ULS (see section below).

Combination of actions for Seismic Design Situation

The combination of actions is:

$$DL + 0.3 \times LL + A_{Ed}$$

Definition of hazard scenario

After the structure is subjected to an earthquake, a column can be lost, thus making the structure vulnerable to subsequent hazards. In the following, this procedure is applied to verify the capacity of the structure to resist progressive collapse using column loss approach.

Step 1: Seismic analysis – The structure is subjected to a design level earthquake

Step 2: Column loss scenarios: Lost columns are located at A1, A2, A4, B1, B' (Figure 71) – they are assumed to be lost one at a time.

Worked example I.5.2 / SS/S	Seismic design using advanced numerical analysis (multi-hazard) – SS/S	2 of 4 pages

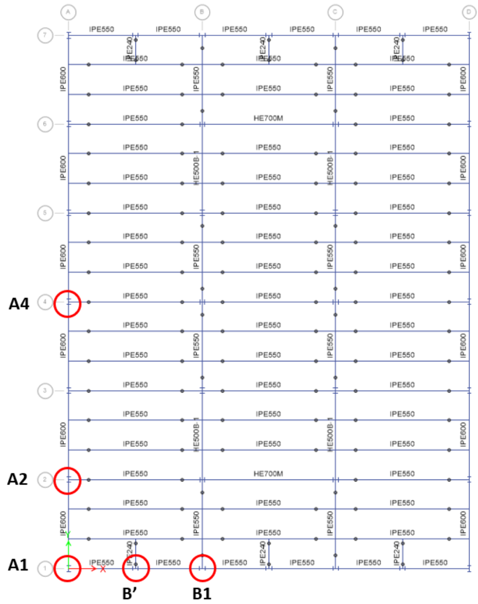
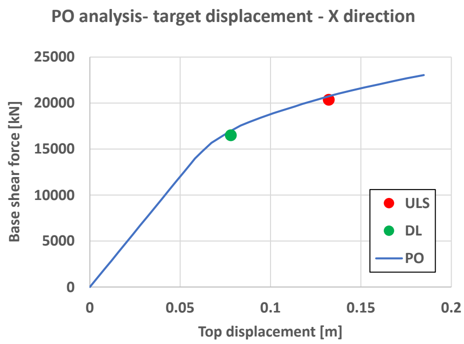


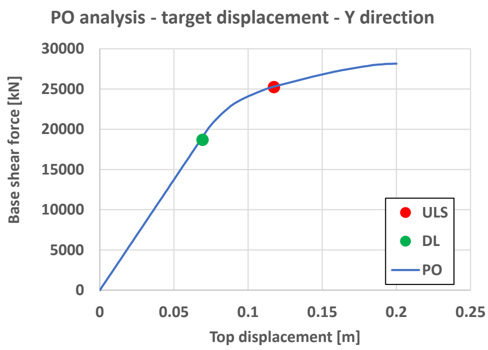
Figure 71. Position of the columns to be removed after eathquake

__*Structural analysis*__

The seismic analysis is performed using push-over analysis and the damage evaluation is done using the N2 method (EN 1998). After the gravity loads are applied, the structure is subjected to a monotonically increasing pattern of lateral forces, representing the inertial forces which would be experienced by the structure when subjected to ground shaking. Under incrementally increased loads, some structural elements may yield. Consequently, after each plastic hinge is formed, the structure experiences a loss in stiffness and load capacity. To evaluate the seismic demands for ULS, the structure is pushed to its target top displacement D_t. Figure 72 shows the capacity curves for transversal and longitudinal directions and the target points for ULS and DL. Figure 73 shows the plastic mechanisms at failure for transversal and longitudinal directions. No plastic hinges develop in perimeter moment resisting frames in neither X nor Y direction at ULS, but only in the braced frames.



a)



b)

Figure 72. Seismic analysis: a) push-over curve with the position of the target point – X direction; b) push-over curve with the position of the target point – Y direction

Worked example I.5.2 / SS/S	Seismic design using advanced numerical analysis (multi-hazard) – SS/S	3 of 4 pages

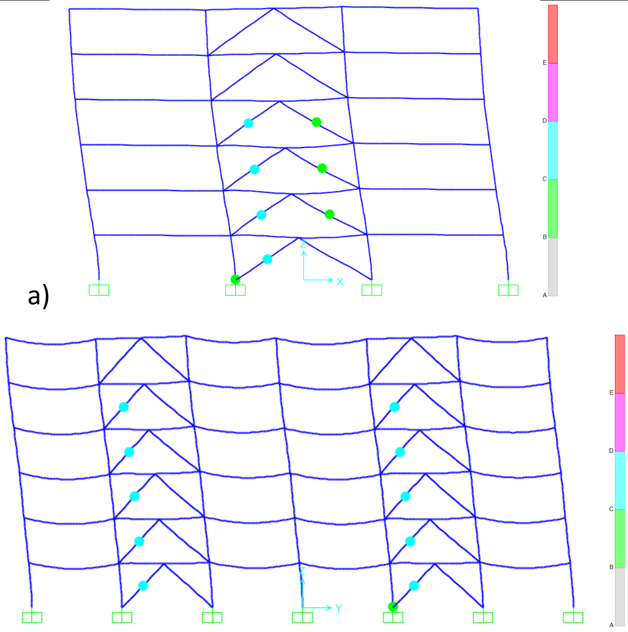


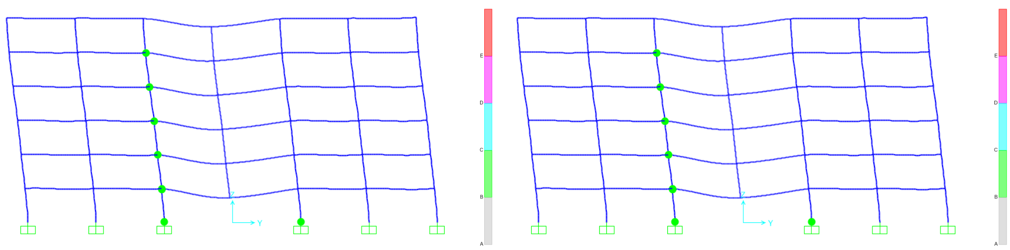
Figure 73. Seismic analysis: a) plastic mechanism at D_t ULS – current transversal frame; b) plastic mechanism at D_t ULS – current longitudinal frame

Column removal in the aftermath of the earthquake

Five removal scenarios are considered, i.e., perimeter, penultimate, and corner columns located at the ground floor. The scenarios involve columns on the short and long sides of the facade. The assessment of progressive collapse resistance is done using the alternative load path (ALP) method and nonlinear dynamic procedure (NDP), in accordance with the UFC 4-023-03 guidelines. The gravity loads are applied in first stage; then, in the second stage, the element is removed almost instantaneously (removal duration of 0.005 seconds).

Results

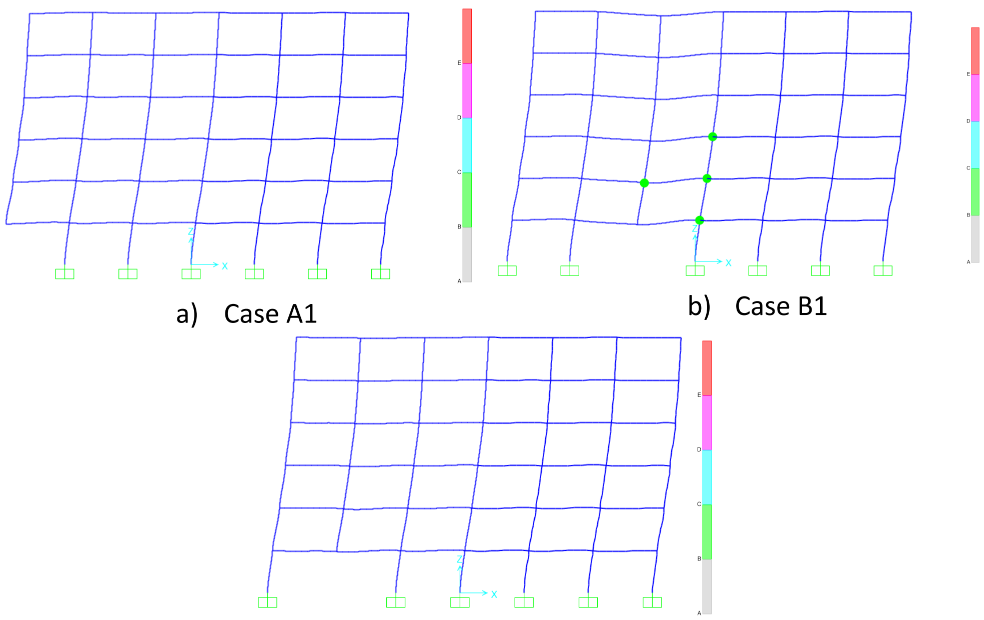
Below are presented the formation of the plastic mechanisms which occur in perimeter frames in the scenarios mentioned above. For each case, the plastic mechanisms (Figure 74a) to e)) and history of vertical displacement above the removed column (Figure 75) are presented.



a) Case A4

b) Case A2

Worked example I.5.2 / SS/S	Seismic design using advanced numerical analysis (multi-hazard) – SS/S	4 of 4 pages



a) Case A1 b) Case B1

c) Case B’

Figure 74. Plastic mechanism after column removal for the considered scenarios

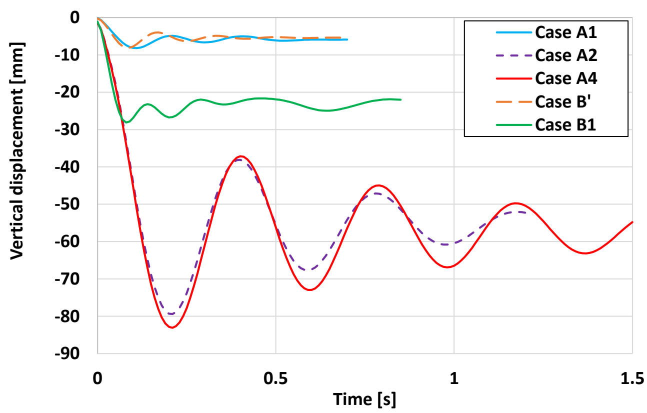


Figure 75. Time history response for column removal scenarios

<u>*Conclusions*</u>

- It may be concluded that the structure has the capacity to resist the progressive collapse even with the loss of a column after an earthquake.
- The level of damage in the elements (given by the level of plastic deformation in the plastic hinges) is small.
- Other performance objective (e.g., collapse prevention) may be employed to assess the structural behaviour.

Flowchart Figure 3 – Box B.5 ➔ End of design

8.8 Unidentified exceptional events

8.8.1 Prescriptive approach (tying method)

8.8.1.1 Design for unidentified threats using prescriptive approach - tying method (SS/S)

FAIL NO MORE Worked example	Title	Design for unidentified threats using prescriptive approach			1 of 3 pages
	Structure	Steel structure in seismic zone	Made by	UPT	Date: 06/2021
	Document ref.	II.1.1 / SS/S			

Example: Design for unidentified threats of a steel structure in seismic zone using prescriptive approach (tying method)

Design manual § 5.3.1

This example shows the application of the tying method for beams and their connections (horizontal tying).

The vertical tying should also have been checked but this check is not performed in the framework of this worked examples. The check of vertical ties is performed in W.E. II.4.1 / SS/NS.

Basic data of structure

Design manual § 8.2.

- For geometry, sections, and materials, see Section 8.2.

Actions for Accidental Design Situation

The following actions are considered:

- Permanent loads DL (see Table 11);
- Live loads LL (see Table 11 for SS/S structure);
- No specific accidental action is taken into account.

Definition of tensile loading

Eq. A1 and A2 from (EN 1991-1-7 2006)

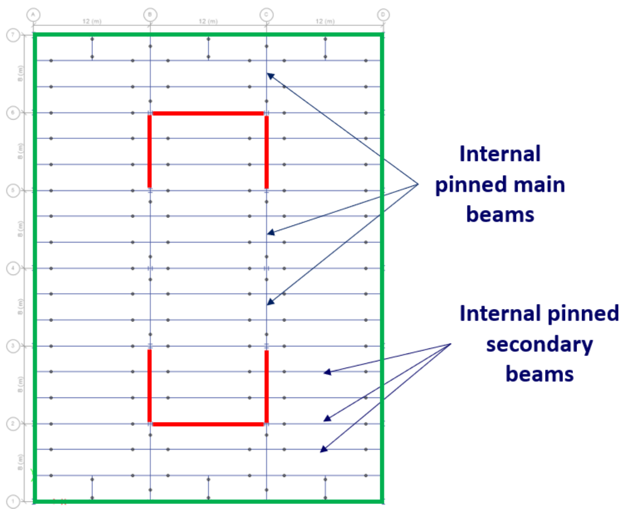


Figure 76. Horizontal ties considered for using prescriptive method – SS/S

Horizontal ties

- for internal ties: $T_i = 0.8 \cdot (g_k + \psi \cdot q_k)$ or 75 kN, whichever is greater
- for perimeter ties: $T_p = 0.4 \cdot (g_k + \psi \cdot q_k)$ or 75 kN, whichever is greater

Worked example II.1.1 / SS/S	Design for unidentified threats using prescriptive approach – tying method – SS/S	2 of 3 pages

Computation

- Internal pinned secondary beams (IPE550, all on short direction, see Figure 77 for joint configuration)

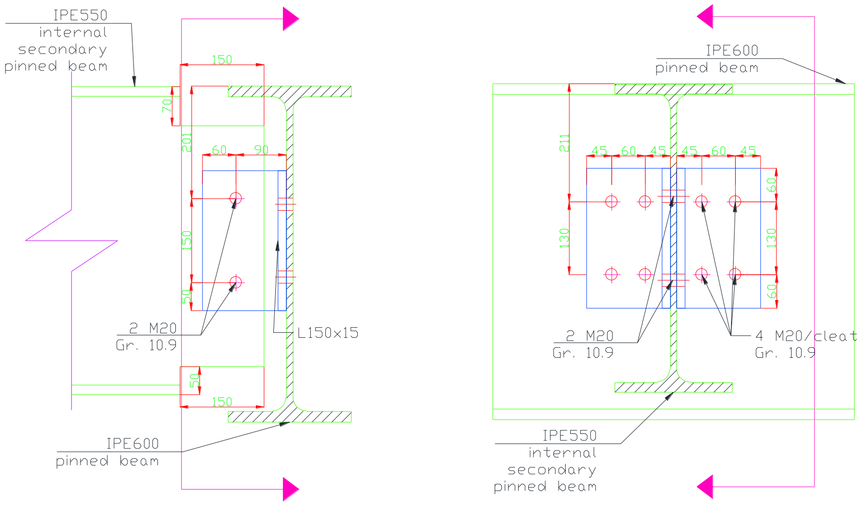


Figure 77. Joint configuration of pinned connection for a secondary beam

Spacing between ties (secondary beams) $s = 2.66\ m$

Span of the tie $L = 12\ m$

Design tensile load for internal ties

$T_i = max[0.8 \cdot (g_k + \Psi \cdot q_k)s.L;\ 75\ kN] = max[0.8 \times (5 + 0.5 \times 3) \times 2.66 \times 12;\ 75\ kN]\ = 166\ kN$

- Internal pinned main beams (IPE550, all on long direction, see Figure 78 for joint configuration)

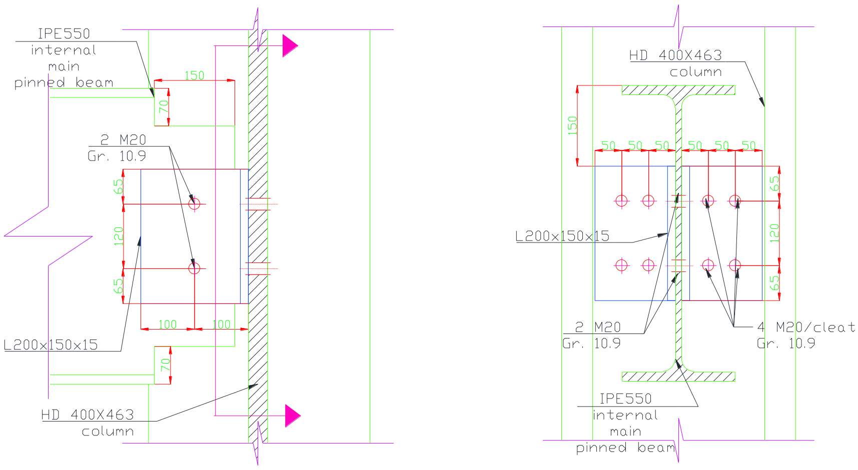


Figure 78. Joint configuration of pinned connection for a main beam

Spacing between ties (main beams) $s = 12m$

Span of the tie $L = 8\ m$

Worked example II.1.1 / SS/S	Design for unidentified threats using prescriptive approach – tying method – SS/S	3 of 3 pages

Design tensile load for internal ties

$$T_i = max[0.8 \cdot (g_k + \Psi \cdot q_k) \cdot s \cdot L;\ 75\ kN]$$
$$= max[0.8 \times (5 + 0.5 \times 3) \times 12 \times 8;\ 75\ kN] = 499.2\ kN$$

Results

The shear resistances and UFs for the connections of the internal ties considered for the verification are presented in Table 33.

Table 33 Connection check for tying forces according to the prescriptive method

Element	Tying force (kN)	Shear resistance (kN)	Failure mode	UF (-)
Internal pinned secondary beams	166	392	Sec. beam in bearing	0.42
Internal pinned main beams	499.2	392	Main beam bolts in shear	1.27

***Note**: The capacity of the connection in tension at the extremities of the secondary beams was verified without any verification to the main beam. Care is needed as the main beam web can become the critical component.*

Conclusions

For the connections of the internal pinned secondary beams, the UF of 0.42 results in an appropriate design.

Flowchart Figure 3 – Box C.4 ➔ End of design

For the connections of the pinned internal main beams, an UF of 1.27 requires a redesign of the joint.

Flowchart Figure 3–Box C.4➔C.2

Consequently, another bolt row was added (3 rows in total). It increased the shear capacity to **588 kN** which gives an UF of **0.85** for the connection – see Figure 79 for the redesigned configuration.

Flowchart Figure 3 – Box C.4 ➔ End of design

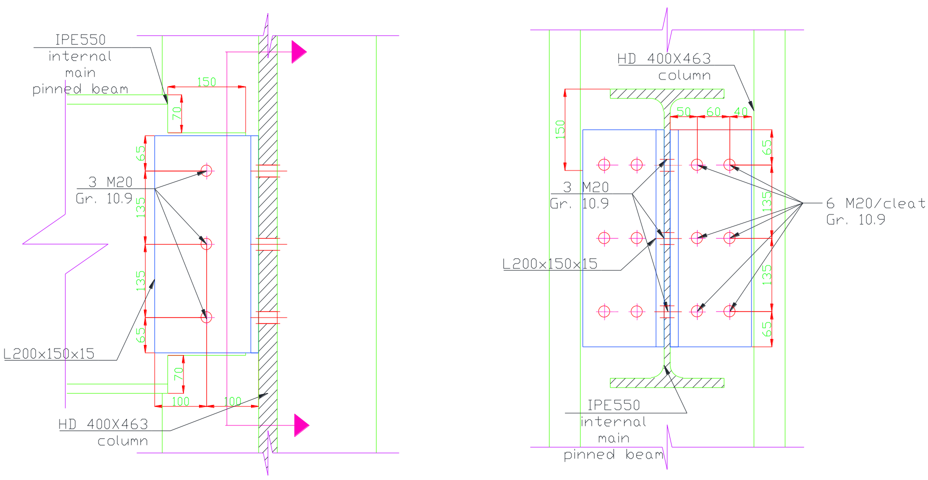


Figure 79. Redesign of the main beam joints

8.8.1.2 Design for unidentified threats using prescriptive approach – tying method (CS/S)

FAIL NO MORE Worked example	Title	Design for unidentified threats using prescriptive approach			1 of 2 pages
	Structure	Composite structure in seismic zone	Made by	UPT	Date: 06/2021
	Document ref.	II.1.2 / CS/S			

Example: Design for unidentified threats in a composite structure in seismic zone using prescriptive method (tying method)

Design manual § 5.3.1

This example shows the application of the tying method for beams and their connections (horizontal tying).

The vertical tying should also have been checked but this checked is not performed in the framework of this worked examples. The check of vertical ties is performed in W.E. II.4.1 / SS/NS.

Basic data of structure

Design manual § 8.2.

- For geometry, sections, and materials, see Section 8.2.

Actions for Accidental Design Situation

The following actions are considered:

- Permanent loads DL (see Table 11);
- Live loads LL (see Table 11 for CS/S structure);
- No specific accidental action is taken into account.

Definition of tensile loading

The verification is performed similarly to W.E. II.1.1 / SS/S for the main beams. Additionally, the longitudinal reinforcement in the effective length of the beam is taken into consideration.

Computation

- internal pinned main beams

Spacing between ties (main beams) $s = 12\,m$

Span of the tie $L = 8\,m$

Design tensile load for internal ties

$$T_i = max[0.8(g_k + \Psi . q_k)s.L;\ 75\,kN] = max[0.8(5 + 0.5 \times 3)12 \times 8;\ 75\,kN]$$

$$= 499.2\,kN$$

The axial force capacity of the main beam connection is the sum of the tension force transferred through the bolts and the tension force transferred through longitudinal reinforcement in the effective width of the reinforced concrete slab.

Worked example II.1.2 / CS/S	Design for unidentified threats using prescriptive approach – tying method – CS/S	2 of 2 pages
Results $N_u = 392\ kN + 73kN = 465 < T_i = 499.2\ kN$ ➔ connection redesign is required. Therefore, 3 bolts M20 10.9 were provided instead of 2, as presented in Figure 79 for the previous worked example. $N_u^* = 661\ kN > T_i = 499.2\ kN$, UF = 0.76 *Conclusions* All internal main pinned beams and their connections fulfil the verification for required tying forces with limited changes required in the design. As previously stated in W.E. II.1.1 / SS/S, it may be concluded that the design for gravity loads may be insufficient for tying force requirements in case of large tributary areas.	Flowchart Figure 3 – Box C.5 ➔ Redesign Flowchart Figure 3 – Box C.4 ➔ End of design	

8.8.1.3 Design for unidentified threats using prescriptive approach - tying method (SS/NS)

FAIL NO MORE Worked example	Title	Design for unidentified threats using prescriptive approach			1 of 4 pages
	Structure	Steel structure in non-seismic zone	Made by	F+W	Date: 06/2021
	Document ref.	II.1.3 / SS/NS			

Example: Design for unidentified threats in a steel structure in non-seismic zone using prescriptive method (tying method)

Design manual § 5.3.1

This example shows the application of the tying method for beams and their connections (horizontal and vertical tying).

Basic data of structure

Design manual § 8.2.

- For geometry, sections, and materials, see section 8.2.

Actions for Accidental Design Situation

The following actions are considered:

- Permanent loads DL (see Table 11);
- Live loads LL (see Table 11 for SS/NS structure);
- No specific accidental action is taken into account.

Definition of tensile loading

In this approach, only surface loads are taken into account. The line loads (facade loads) are considered by converting them into surface loads for external ties.

Computation

Horizontal and vertical tying forces are detailed in the tables below. Note that only members along frames are defined as ties here, so that beam-to-beam joints are not subjected to tying forces.

Table 34. Horizontal tying forces according to the prescriptive approach – SS/NS

External tie		**Internal tie**	
s	8 m	s	8 m
L	12 m	L	12 m
ψ	0,5	ψ	0,5
g_k	5 kN/m2	g_k	5 kN/m2
q_k	3 kN/m2	q_k	3 kN/m2
g_k facade	4 kN/m		
g_k facade equ.	0,5 kN/m2		
T_e	**268,8 kN**	**T_i**	**499,2 kN**

Worked example II.1.3 / SS/NS	Design for unidentified threats using prescriptive approach – tying method – SS/NS	2 of 4 pages

Table 35. Vertical tying forces according to the prescriptive approach – SS/NS

External tie (HEB360)			Internal tie (HEM300)		
s	8	m	s	8	m
L	12	m	L	12	m
ψ	0,5		ψ	0,5	
g_k	5	kN/m2	g_k	5	kN/m2
q_k	3	kN/m2	q_k	3	kN/m2
g_{IPE600}	1,22	kN/m	g_{IPE600}	1,22	kN/m
g_{IPE500}	0,907	kN/m	g_{IPE550}	1,06	kN/m
g_{IPE550}	1,06	kN/m	g_{HEM300}	2,38	kN/m
g_{HEB360}	1,42	kN/m	h	4	m
h	4	m	n IPE550	4	
n IPE550	1,5				
g_k facade	4	kN/m			
T_e	**400,5 kN**		**T_i**	**694,2 kN**	

<u>*Verification of the structure*</u>

- Member verification

The tying members have to be checked when subjected to the tying forces assumed to be applied alone. Accordingly, they are easily checked comparing their plastic axial resistance $N_{pl,Rd}$ to the tying forces T_e or T_i. All the tying members have their plastic axial resistance higher than the applied tying forces; the details of the computation are not reported here.

- Joints verification

The position of the joints in the structure is provided in Figure 52. The column splices (Figure 80) are characterised using the component method. Fin plate joint verifications (Figure 81) are carried out according to (ECCS, 2009). Results are given in Table 36.

As it can be observed in Table 36, joints B1, B3, C2w, D3s, D3w and 3-3 don't have a sufficient resistance to withstand tying forces according to the prescriptive approach.

Notice that, for double sided weak axis beam-to-column configurations, the component "column web in bending" is considered as not activated while this component is taken into account when characterising the single sided weak axis beam-to-column configurations.

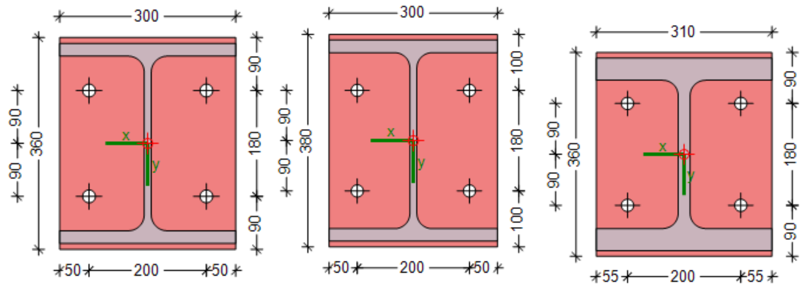
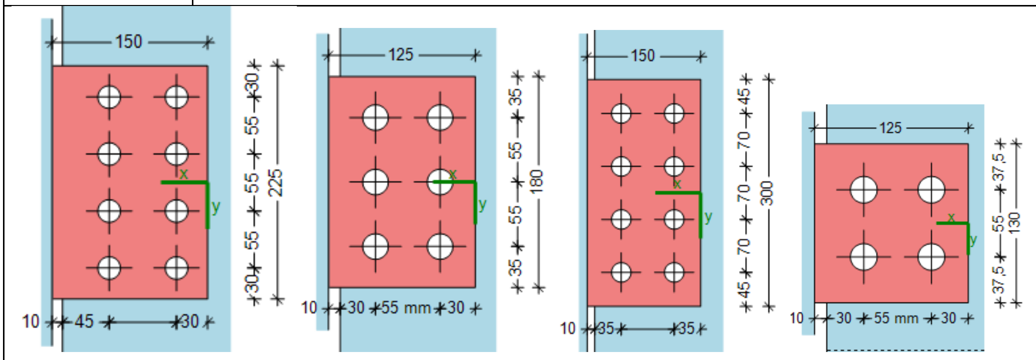


Figure 80. Column splices with 4xM20 (left: 1-1, center: 2-2, right: 3-3) – 15 mm thick S355 end-plate – 10.9 bolts – 5 mm flange welds and 4 mm web welds

Worked example II.1.3 / SS/NS	Design for unidentified threats using prescriptive approach – tying method – SS/NS	3 of 4 pages



a) 8xM20 b) 6M20 c) 8M24 d) 4M20

Figure 81. Fin plate beam-to-column joints (a): A-1w, b) A1s, A2, B1, B3, c) C-2w, C-3w, d) D-3s, D-3w) – 10 mmS355 thick fin plate – 10.9 bolts – 6 mm welds for the fin plates

Table 36. Joints verifications for tying forces according to the prescriptive approach

Position s = strong axis w = weak axis	**Tying force (kN)**	**Failure mode**	**UF**
A1s / A2	268.8	Fin plate in bearing	0.63
A1w	268.8	Column web in bending	0.73
B1 / B3	499.2	Fin plate in bearing	1.16
C2w	499.2	Column web in bending	1.15
C3w	499.2	Fin plate in bearing	0.67
D3s/D3w	499.2	Beam web in bearing	2.02
D3w	90	Beam web in bearing	0.88
1-1 / 2-2	400.5	End-plate in bending	0.88
3-3	694.2	End-plate in bending	1.31

Redesign of the structure

The redesign of joints B1, B3, C2w, D3s, D3w, and 3-3 consists of:

Flowchart Figure 3 – Box C.5 ➔ Box C.2

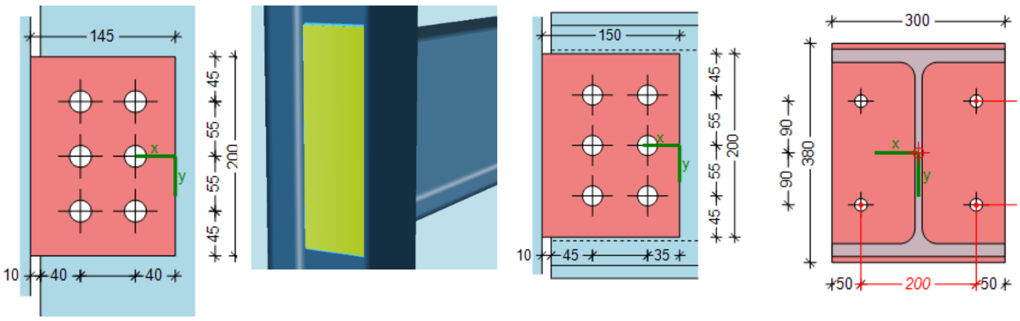
B1/B3 : slight modification of fin plate geometry;

C2w : welded column web plate added ;

D3s/D3w : 2 bolts added and modification of the fin plate geometry;

3-3 : M24 bolts (instead of M20) and 20 mm end-plate instead of 15 mm.

Worked example II.1.3 / SS/NS	Design for unidentified threats using prescriptive approach – tying method – SS/NS	4 of 4 pages



B1/B3 C2 D3s/D3w 3-3

Figure 82. Redesigned joints to fulfil tying forces verifications according to the prescriptive approach

Updated utilization factors for these joints are summarized in Table 37.

Table 37. Redesigned joints verifications for tying forces according to the prescriptive approach

Position s = strong axis w = weak axis	**Tying force (kN)**	**Failure mode**	**UF**
B1 / B3	499.2	Fin plate in tension (net)	0.93
C2w	499.2	Column web in bending	0.88
D3s/D3w	499.2	Beam web in tension (net)	1.03
3-3	694.2	End-plate in bending	0.83

The check of the D3s/D3w joints is exceeded by 3%. This small exceedance was accepted herein as it is usually done in practice. A solution to fulfil this check could be to replace the HEA300 with HEB300 beams. This might be also an enhancement for the postcritical behaviour in case of a column loss as HEB300 S355 are class 1 profiles while HEA300 are class 3.

Flowchart Figure 3 – Box C.4 ➔ End of design

8.8.1.4 Design for unidentified threats using prescriptive approach - tying method (CS/NS)

FAIL NO MORE Worked example	Title	Design for unidentified threats using prescriptive approach			1 of 4 pages
	Structure	Composite structure in non-seismic zone	Made by	AM	Date: 06/2021
	Document ref.	II.1.4 / CS/NS			

Example: Design for unidentified threats in a composite structure in non-seismic zone using prescriptive approach (tying method)

Design manual § 5.3.1

This example shows the application of the tying method for beams and their connections (horizontal tying).

The vertical tying should also have been checked but this evaluation is not performed in the framework of this worked examples. The check of vertical ties is performed in W.E. II.4.1 / SS/NS.

Basic data of structure

Design manual § 8.2.

- For geometry, sections, and materials, see section 8.2.

Actions for Accidental Design Situation

The following actions are considered:

- Permanent loads DL (see Table 11);
- Live loads LL (see Table 11 for CS/NS structure);
- No specific accidental action is taken into account.

Definition of tensile loading

In this example the calculations are made for the beams connected to the column located at the middle of the longest façade of the building – see Figure 83.

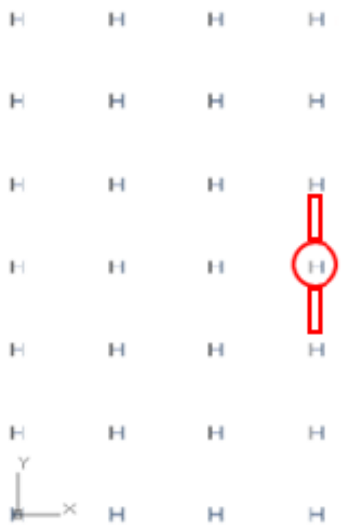


Figure 83. Beams considered for prescriptive method – CS/NS

Computation

For framed structures, the minimum tensile forces to be resisted by an effective horizontal tying can be estimated using the following calculations:

EN 1991-1-7 2006

Worked example II.1.4 / CS/NS	Design for unidentified threats using prescriptive approach – tying method – CS/NS	2 of 4 pages

- Horizontal tying

Permanent action	$g_k = 5 \dfrac{kN}{m^2}$
Variable action	$q_k = 3 \dfrac{kN}{m^2}$
Office floor loading factor	$\Psi = 0.5$
Spacing between ties (primary beams)	$s = 12\,m$
Span of the tie	$L = 8\,m$

Design tensile load for internal ties

$$T_i = max[0.8 \cdot (g_k + \Psi \cdot q_k) \cdot s \cdot L\,, 75\,kN]$$
$$= max[0.8 \times (5 + 0.5 \times 3) \times 12 \times 8\,, 75\,kN] = 499.2\,kN$$

Design tensile load for perimeter ties

$$T_p = max[0.4 \cdot (g_k + \Psi \cdot q_k) \cdot s \cdot L\,, 75\,kN]$$
$$= max[0.4 \times (5 + 0.5 \times 3) \times 12 \times 8\,, 75\,kN] = 249.6\,kN$$

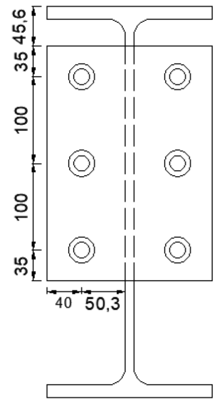
Cross-sectional area: internal beams (IP360)	$A_{s.i} = 7270\,mm^2$
Cross-sectional area: perimeter beams (IP450)	$A_{s.p} = 9880\,mm^2$
Plastic resistance of the internal beams	$N_{pl.i} = A_{s.i}.f_y = 7270 \times 10^{-6} \times 355 \times 10^3 = 2581\,kN$
Plastic resistance of the perimeter beams	$N_{pl.p} = A_{s.p}.f_y = 9880 \times 10^{-6} \times 355 \times 10^3 = 3507.4\,kN$
Utility check – Internal beams	$U_i = \dfrac{T_i}{N_{pl.i}} = \dfrac{499.2}{2581} = 0.19$
Utility check – Perimeter beams	$U_p = \dfrac{T_p}{N_{pl.p}} = \dfrac{249.6}{3507.4} = 0.07$

The calculations show that the beams are able to sustain the tensile loads defined in the standards.

The joints at the extremities of the beams should also be able to resist to the tying forces calculated previously. Two different joints solutions have been considered for the beam-to-column connections: header plate and fin plate joints.

Worked example II.1.4 / CS/NS	Design for unidentified threats using prescriptive approach – tying method – CS/NS	3 of 4 pages

Header plate



Bolt Type:

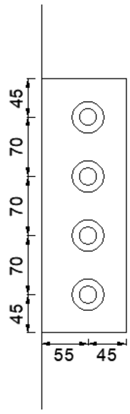
M16 Gr.8.8 (6 bolts)

Plate:

Thickness $t_p = 10mm$
Height $h_p = 270mm$
Width $b_p = 190mm$

Weld $a_w = 2X6mm$

Fin plate



Bolt Type:

M20 Gr.8.8 (4 bolts)

Plate:

Thickness $t_p = 10mm$
Height $h_p = 300mm$
Width $b_p = 100mm$

Weld $a_w = 2X6mm$

The verification of the connections mentioned above were made according to Annex A.5.

Bolts in tension	$N_{u1} = 602.88\ kN$	Bolts in shear	$N_{u1} = 376.32\ kN$
Header Plate in bending	$N_{u2} = 271.17\ kN$	Fin plate in bearing	$N_{u2} = 512.73\ kN$
Supporting member in tension	$N_{u3} = 383.08\ kN$	Fin plate in tension: Gross	$N_{u2} = 1128.00\ kN$
Beam web in tensions	$N_{u4} = 954.29\ kN$	Fin plate in tension: Net	$N_{u4} = 717.41\ kN$
Tying resistance of the joint	$N_u = 271.17\ kN$	Beam web in bearing	$N_{u5} = 481.96\ kN$
		Beam web in tension: Gross	$N_{u6} = 1060.32\ kN$
		Beam web in tension: Net	$N_{u7} = 674.36\ N$
		Supporting member in bending	$N_{u8} = 350.99\ N$
		Tying resistance of the joint	$N_u = 350.99\ kN$

Results

Utility check $U = \frac{T_p}{N_u} = 0.92$

Utility check $U = \frac{T_p}{N_u} = 0.71$

Worked example II.1.4 / CS/NS	Design for unidentified threats using prescriptive approach – tying method – CS/NS	4 of 4 pages

Conclusions

The calculations show that the beams can sustain the tensile loads defined in the standards.

Flowchart Figure 3 – Box C.4 ➔ End of design

It can be observed that the joints were computed assuming pinned connections, i.e., neglecting the possible composite actions which could develop at the level of these joints. This is considered as a safe approach if ductility is guarantee which is the case here. In fact, the rebars at the level of the joints can act as tying elements if the rebar arrangement is continuous throughout the building floor and their contribution could be then simply added to the joint resistance.

Table 38 Joints checked according to tying method – CS/NS

Type	ULS UF	Tying UF	Remarks
Header Plate	0.73	0.92	Bolt Group / Header plate in bending
Fin Plate	0.71	0.71	Bolt group / Support member in bending

According to Table 38, it is possible to conclude that in this example, using a connection targeted for a basic design with 70% utility, is an adequate approach when performing the pre-design in compliance with the tying requirements.

Flowchart Figure 3 – Box C.4 ➔ End of design

8.8.2 Key element method

8.8.2.1 Design for unidentified threats using key element method – normative approach (CS/NS)

FAIL NO MORE Worked example	Title	Design for unidentified threats using key element method – normative approach			1 of 4 pages
	Structure	Composite structure in non-seismic zone	Made by	AM	Date: 06/2021
	Document ref.	II.2.1 / CS/NS			

Example: Design for unidentified threats in a composite structure in non-seismic zone using key element method – normative approach	Design manual § 5.4

This example gives information about the design against unidentified threats using the key element method.

<u>Basic data of the structure</u> (Design manual § 8.2.)

- For geometry, sections, and materials, see Section 8.2.

<u>Actions for Accidental Design Situation</u>

The following actions are considered:

- Permanent loads DL (see Table 11);
- Live loads LL (see Table 11 for CS/NS structure);
- Accidental loading A_d (see section below).

<u>Combination of actions for Accidental Design Situation</u> (EN 1990 §6.4.3.3, Eq 6.11b)

The combination of actions is:

$$DL + 0.5 \times LL + A_d$$

<u>Definition of key elements</u>

In this example, a set of columns (A, B and C) are identified as key elements. According to (EN 1991-1-7 2006), the magnitude of the accidental design action for checking key elements is 34 kN/m² applied in any direction (individually). Figure 84 presents the columns that are checked using the key element method. ((EN 1991-1-7 2006))

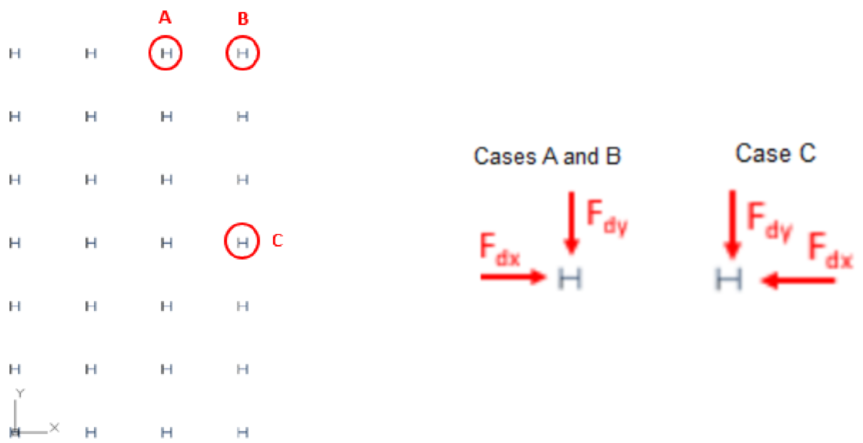


Figure 84 Columns considered to be checked using key element method – CS/NS

Worked example II.2.1 / CS/NS	Design for unidentified threats using key element method – normative approach – CS/NS	2 of 4 pages

Computation

Accidental load $A_d = 34\ \frac{kN}{m^2}$

Length of the column $l_c = 4\ m$

Height of the column section (Steel; Composite) $h_c = (364; 450)\ mm$

Width of the column section (Steel; Composite) $b_c = (371; 450)\ mm$

Width of the panel in front of the column $w_p = 5\ m$

Assumption as this value is not fixed in EN 1991-1-7

Point load (panel width) $F_p = A_d.w_p.l_c = 34 \times 5 \times 4 = 680\ kN$

Point load (section height) (Steel; Composite) $F_{s.h} = A_d.h_c.l_c = (34 \times 364 \times 10^{-3} \times 4\,; 34 \times 450 \times 10^{-3} \times 4 = (49.5\,; 61.2)\ kN$

Point load (section width) (Steel; Composite) $F_{s.w} = A_d.w_c.l_c = (34 \times 371 \times 10^{-3} \times 4\,;\ 34 \times 450 \times 10^{-3} \times 4 = (50.46\,; 61.2)\ kN$

Table 39. Accidental loads used for key elements – Steel columns – CS/NS

Case	F_{dx} (kN)	F_{dy} (kN)
A.1	50.46	0
B.1	50.46	0
C.1	680	0

Case	F_{dx} (kN)	F_{dy} (kN)
A.2	0	680
B.2	0	680
C.2	0	49.5

Table 40. Accidental loads used for key elements – Composite columns – CS/NS

Case	F_{dx} (kN)	F_{dy} (kN)
A.1	61.2	0
B.1	61.2	0
C.1	680	0

Case	F_{dx} (kN)	F_{dy} (kN)
A.2	0	680
B.2	0	680
C.2	0	61.2

Structural analysis

The point loads presented in Table 39 are applied directly as horizontal loads in the SCIA® model (steel columns model) at the mid-height of each key element column (A, B and C) along both axes individually, considering the accidental load combination. This constitutes a safe approach; an alternative could have been to better account for the actual redistribution of the horizontal load from the panel to the column when A_d is assumed to be applied to the panel or to consider A_d as a linear load applied along the height of the column in the other direction.

Worked example II.2.1 / CS/NS	Design for unidentified threats using key element method – normative approach – CS/NS	3 of 4 pages

Regarding the composite columns, the approach was similar as for impact analysis (W.E. I.1.4 / CS/NS, using the previous loads and the software A3C®.

Results

Table 41. UFs and deflection for steel columns – key elements – CS/NS

Case	Section	Loading		Bottom support	UF (-)		Lateral deflection* S355 (mm)
		F_{dx} (kN)	F_{dy} (kN)		S355	S460	
A.1	HD 360x162	50.46	0	Fixed	0.39	0.28	0.7
				Hinged	0.39	0.28	0.8
A.2	HD 360x162	0	680	Fixed	1.03	0.82	-
				Hinged	1.25	1.00	-
B.1	HD 360x162	50.46	0	Fixed	0.22	0.16	0.7
				Hinged	0.23	0.17	0.8
B.2	HD 360x162	0	680	Fixed	0.95	0.75	9.1
				Hinged	1.14	0.92	-
C.1	HD 360x162	680	0	Fixed	0.68	0.54	5.0
				Hinged	0.83	0.65	8.1
C.2	HD 360x162	0	49.5	Fixed	0.40	0.29	1.4
				Hinged	0.42	0.31	1.4

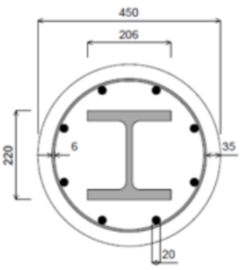
* The lateral deflection is reported here for sake of information as no check of deflection is required here.

Table 42. UFs for composite columns– key elements – CS/NS

Case	Loading		Upper & Bottom supports	UF (-) S355
	F_{dx} (kN)	F_{dy} (kN)		
A.1	61.2	0	Hinged	0.42
A.2	0	680	Hinged	2.29
B.1	61.2	0	Hinged	0.24
B.2	0	680	Hinged	1.84
C.1	680	0	Hinged	1.34
C.2	0	61.2	Hinged	0.40

Details of the composite columns:

- Steel section - HE200M
- Concrete class – C30/37
- Rebar (A500) – ϕ20 mm / ϕ6 mm



Worked example II.2.1 / CS/NS	Design for unidentified threats using key element method – normative approach – CS/NS	4 of 4 pages
Conclusions • **Standard steel columns** The results presented in Table 41 show that, for steel columns using fixed supports, the resistance does not exceed the yield strength (1.03 can be admissible).		Flowchart Figure 3 – Box C.4 ➔ End of design
However, with hinged supports for cases A.2 and B.2 this limit is exceeded.		Flowchart Figure 3 – Box C.2 ➔ Redesign
• **Composite steel-concrete columns** Regarding composite columns, as presented in Table 42, the utilization factors are considerably higher as explained previously for the impact analysis (W.E. I.1.4 / CS/NS). Overall it is concluded that, for non-composite steel columns, the standard design is able to sustain the developed loads, when the lower connection of the column is fixed. The composite columns however show worst results due to the fact that the main contribution for the resistance will be from the steel element which is substantially smaller than the one used for the standard steel design. As shown previously, the main improvement that can be made is increasing the steel grade to S460; by doing so, the columns utilization factors are all below or equal to 1.0 for standard steel sections. In order to improve the response of the key element under A_d, a set of other changes could be implemented: • Increase the size of the sections; • Design considering more advantageous boundary condition for the joints; • A combination of the previous solutions could be contemplated for the composite columns.		Flowchart Figure 3 – Box C.2 ➔ Redesign

8.8.3 Segmentation method

8.8.3.1 *Design for unidentified threats using segmentation method (SS/NS)*

FAIL NO MORE Worked example	Title	Design for unidentified exceptional loads using segmentation method			1 of 1 pages
	Structure	Steel structure in non-seismic zone	Made by	F+W	Date: 06/2021
	Document ref.	II.3.1 / SS/NS			

Example: Design for unidentified threats in a steel structure in non-seismic zone using segmentation	Design manual § 5.5
The example gives information about the design against unidentified threats using segmentation method.	
<u>*Basic data of structure*</u> • For geometry, sections, and materials, see Section 8.2.	Design manual § 8.2.
<u>*Remarks*</u>	
The segmentation method (weak segment border, strong segment border) can be used either alone or in combination with other measures (e.g., local strengthening) or methods (e.g., ALPM). When the outputs of the ALPM indicate the need for redesign, the segmentation method may be used as an alternative solution to limit the extension of damage. In the case of the current low-rise building, a weak segmentation border strategy could be chosen. As it will be highlighted from the results of both analytical and numerical approaches, the pinned fin plate joints designed for ULS are not able to withstand the large tensile forces from membrane effects when considering a column loss scenario. Practically, these joints act as “fuses” in case of a column loss, and the collapse will be limited to the area directly affected by the column loss (horizontal limitation of damage). If the joints response is ductile, they will develop large deformations before collapse, so preventing from a sudden brittle failure.	Flowchart Figure 3 – Box C.4

8.8.4 Alternate load path method

8.8.4.1 Design for unidentified threats using ALPM - analytical approach (SS/NS)

FAIL NO MORE Worked example	Title	Design for unidentified threats using ALPM – analytical approach			1 of 11 pages
	Structure	Steel structure in non-seismic zone	Made by	F+W	Date: 06/2021
	Document ref.	II.4.1 / SS/NS			

Example: Design for unidentified threats in a steel structure in non-seismic zone using alternate load path method - analytical approach

Design manual § 5.3.2

This example gives information about the design against unidentified threats using the analytical approach from ALPM.

Basic data of structure

Design manual § 8.2.

- For geometry, sections, and materials, see Section 8.2.

Actions for Accidental Design Situation

The following actions are considered:

EN 1990 §6.4.3.3, Eq 6.11b

- Permanent loads DL (see Table 11);
- Live loads LL (see Table 11 for SS/NS structure).

Combination of actions for Accidental Design Situation

The combination of actions is:

$$DL + 0.5 \times LL$$

Definition of column loss scenarios

- Scenario 1 – column removal at location B2

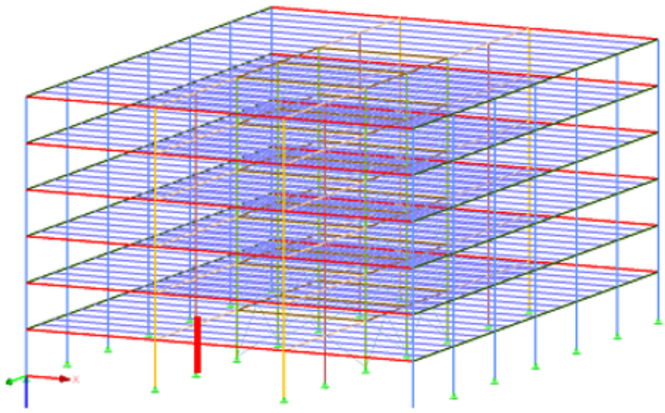


Figure 85. Assumed column loss (column B2)

Elements under investigation

- Beam B1/B3 – IPE550
- Beam C2w/C3w – IPE600

Assumptions for joints

- Solution 1: simple joints
- Solution 2: partial-strength joints

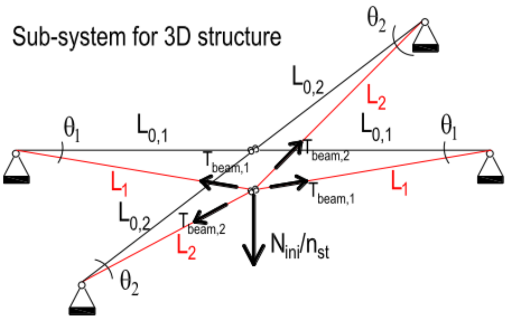
Worked example II.4.1 / SS/NS	Design for unidentified threats using ALPM – analytical approach – SS/NS	2 of 11 pages

<u>*Computation*</u>

The method applied is the simplified analytical approach for structures with horizontal diaphragms presented in Section 5.3.2.

- **Solution 1 - tying forces for simple joints (Section 5.3.2.2)**

The procedure consists in solving the system of 4 equations as shown in Figure 86.



	3D Structures with simple joints
Eq. 1	$\frac{N_{ini}}{n_{st}} = 2.T_{beam,1}.\sin\theta_1 + 2.T_{beam,2}.\sin\theta_2$
Eq. 2	$T_{beam,1} = \frac{1-\cos\theta_1}{\cos\theta_1}.E.A_1$
Eq. 3	$T_{beam,2} = \frac{1-\cos\theta_2}{\cos\theta_2}.E.A_2$
Eq. 4	$L_{0,1}.\tan\theta_1 = L_{0,2}.\tan\theta_2$

Figure 86. Equation system of the analytical approach for simple joints

Beam with index 1 is the IPE550, while beam with index 2 is the IPE600. The initial force in the column N_{ini} is taken from the numerical approach by considering the accidental load case combination.

Table 43. Input parameters for the analytical approach with simple joints – SS/NS

N_{ini}	n_{st}	E	A_1	$L_{0,1}$	A_2	$L_{0,2}$
4078.51 kN	6	210000 MPa	134 cm^2	12 m	156 cm^2	8 m

By reworking the equation system and embedding values from Table 43, the first equation can be written for x = θ_2 as follows:

$$17\,866.67\tan(x)\left(1-\cos\left(\tan^{-1}(0.67\tan(x))\right)\right) + 31\,200\tan(x)\,(1-\cos(x)) - 3.24 = 0$$

The solution of this equation is x = θ_2 = 0.05485 rad. The results for the four unknowns are summarized in Table 44.

Table 44. Solution of the equation system for the analytical approach in scenario 1 – SS/NS

θ_1	θ_2	$T_{beam,1}$ - IPE550	$T_{beam,2}$ - IPE600
0.03659 rad	0.05485 rad	1884 kN	4934 kN

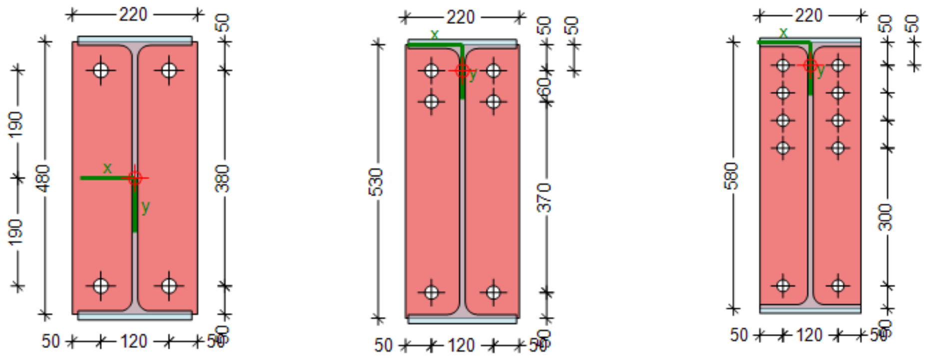
<u>*Remarks*</u>

- When compared with the numerical approach – W.E. II.4.5 SS/NS, the results obtained are approximately 8% higher (1741 kN for IPE550 and 4565 kN for IPE600). However, it is known that the analytical approach overestimates the tensile forces, so that the order of magnitude here is coherent and validates the tensile forces obtained with the numerical approach.
- The results indicate that a redesign of the structure for robustness is needed as the joints are not able to sustain such significant loads (see W.E. II.1.3 / SS/NS).

Worked example II.4.1 / SS/NS	Design for unidentified threats using ALPM – analytical approach – SS/NS	3 of 11 pages

- **Solution 2 – alternative approach with partial-strength joints**

As stated in W.E. II.4.5 / SS/NS, partial-strength joints may be a good alternative to pinned joints to increase the robustness of the structure. To investigate this, main beam-to-column joints will be replaced by flush end-plate joints. All these joints have M24 Gr. 10.9 bolts and 15 mm thick end-plates, as presented in Figure 87.



A1/A2 B1/B3 C2w/C3w

Figure 87. Alternative partial-strength flush end-plate joints for the analytical approach

These semi-rigid joints have been designed to withstand the ULS shear forces and possible N-V interaction in bolts. Note that for beam-to-column joints bolted on the weak axis of the column (through the column web), a welded part is needed to rebuild a “strong axis” type joint, as illustrated in Figure 88.

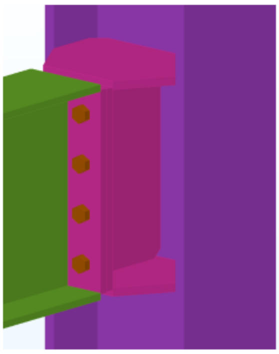


Figure 88. Welded part for weak axis flush end-plate joints (bolt pattern not representative)

The simplified analytical method with partial-strength joints takes into account the following effects (see Section 5.3.2.3):

- Contribution from the plastic mechanism of beams;
- Contribution from the slab;
- Contribution from the arching effects.

If the sum of the above contributions is not sufficient, larger deformations develop and membrane effects in the beams are activated similarly as in the simple joint example. As this requires greater rotational capacity in the joints, the robustness design will be here performed alternatively by optimizing the three above-mentioned contributions so that no membrane effects occur.

Worked example II.4.1 / SS/NS	Design for unidentified threats using ALPM – analytical approach – SS/NS	4 of 11 pages

- Contribution of the slab

The cross-section and the properties of the slab are summarized in Figure 89 and Table 45.

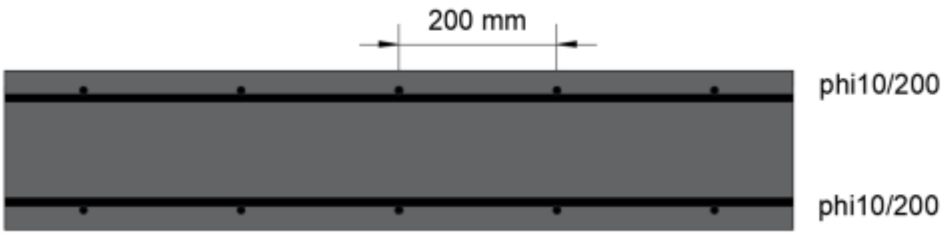


Figure 89. Cross-section of the concrete slab

Table 45. Properties of the concrete slab

Class	t	c	Steel	A_{sx} (top and bottom)	A_{sy} (top and bottom)	M_{Rd} (sagging/ hogging)	Failure mode
C30/37	20 cm	20 mm	B500S	3.93 cm^2/m	3.93 cm^2/m	26.9 kNm	Yielding of reinforcement

The slab is designed to fulfill SLS/ULS requirements. The steel reinforcement is defined by the minimal constructive reinforcement according to DIN EN 1992-1 Chap. 9.

For the considered column loss scenario, the static system of the concrete slab without accounting for any restraints coming from the inner beams is illustrated in Figure 90.

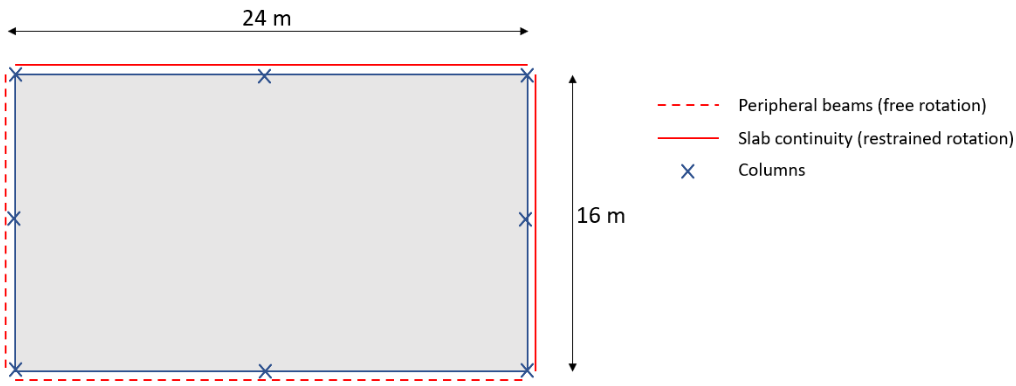


Figure 90. Statical system of the concrete slab after column loss

The accidental loading (1 x G + 0.5 x Q) of 6.5 kN/m² (by neglecting facade loads) leads to large bending moments for which the slab was not designed, see Figure 91.

Worked example II.4.1 / SS/NS	Design for unidentified threats using ALPM – analytical approach – SS/NS	5 of 11 pages

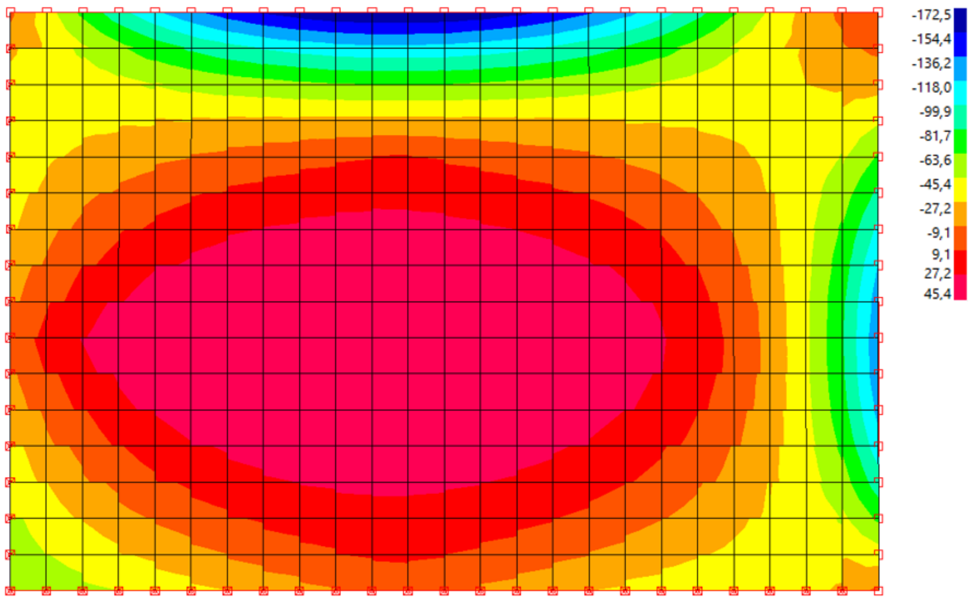


Figure 91. Accidental bending moment in the concrete slab after column loss (M_{Ed} = -172.5 kNm)

Consequently, the concrete slab would not be sufficient by itself to ensure the robustness of the structure. However, together with other effects as listed above, the slab can still contribute to ensure robustness. This contribution is expressed through the vertical point force P_{slab} (where the column is lost) needed for a plastic mechanism to develop. As the failure mode of the slab is ductile (yielding of the steel reinforcement), the slab will be able to maintain the plastic moment along yielding lines.

The plastic mechanism is obtained according to the Johansen method. Two failure patterns were investigated: a non-circular and a circular one. Both are illustrated in the following figures.

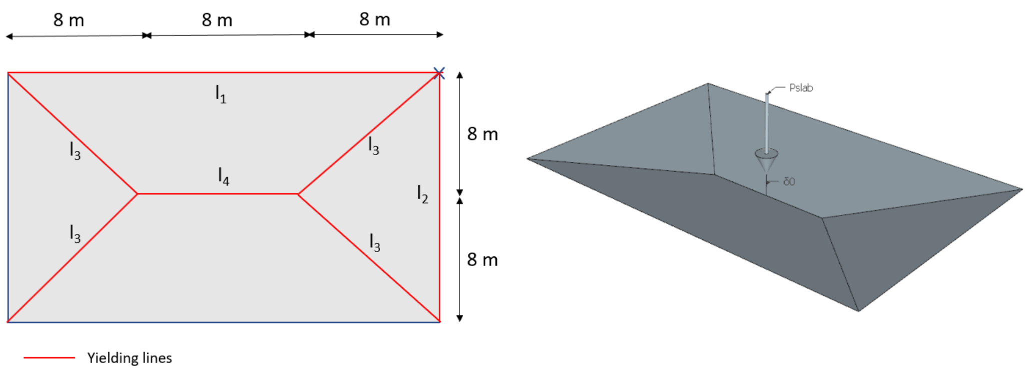


Figure 92. Non-circular plastic mechanism pattern

Worked example II.4.1 / SS/NS	Design for unidentified threats using ALPM – analytical approach – SS/NS	6 of 11 pages

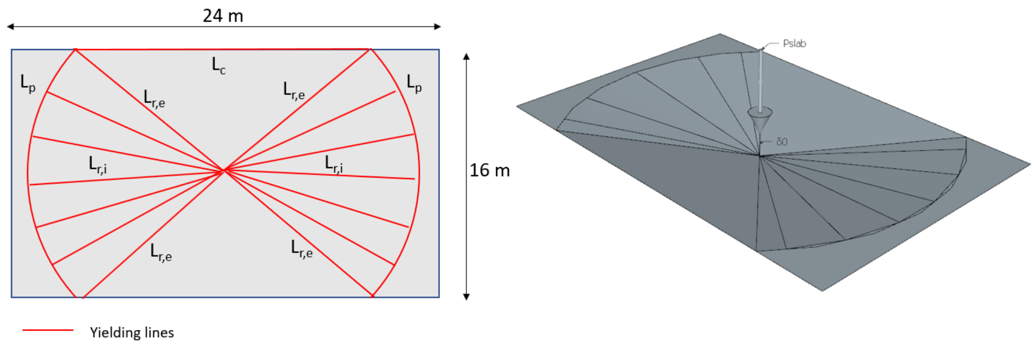


Figure 93. Circular plastic mechanism pattern

Using the virtual works principle, the following forces are obtained:

- Non-circular pattern: 313.6 kN
- Circular pattern: 330.4 kN

The value of $N_{pl,slab}$ is given by the minimum of both above values, i.e., 313.6 kN.

More details about the derivation of these values from the plastic mechanisms are found in the detailed calculation and in (Vermeylen, 2021).

- Contribution of the steel beam mechanism

Due to the use of partial-strength joints, the vertical force associated to the development of a plastic beam mechanism due to the formation of plastic hinges in the joints can be computed.

Since the joints on both directions are partial-strength joints, this force is given by the following equation (adapted from the 1D version), see Figure 94 for the illustrated mechanism.

$$N_{pl} = \frac{2.M^-_{pl,Rd,1} + 2.M^+_{pl,Rd,1}}{L_{0,1}} + \frac{2.M^-_{pl,Rd,2} + 2.M^+_{pl,Rd,2}}{L_{0,2}}$$

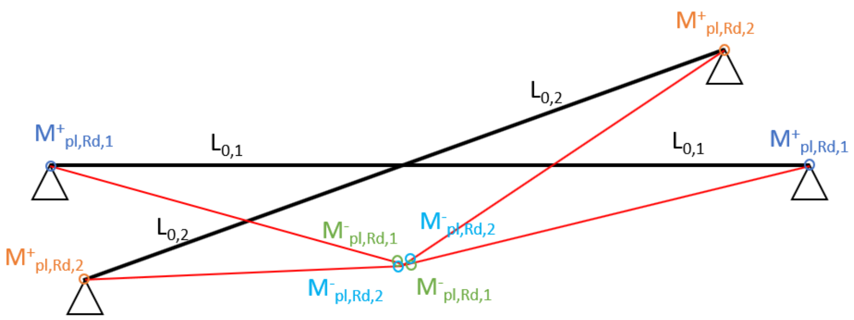


Figure 94. Plastic beam mechanism developing in the beams with partial-strength joints

Sagging and hogging moment resistances of the joints are given in Table 46.

Worked example II.4.1 / SS/NS	Design for unidentified threats using ALPM – analytical approach – SS/NS	7 of 11 pages

Table 46. Moment resistance of partial-strength joints

Joint B1/B3		Joint C2/C3	
$M_{pl,Rd,1}^{+}$ (hogging)	$M_{pl,Rd,1}^{-}$ (sagging)	$M_{pl,Rd,2}^{+}$ (hogging)	$M_{pl,Rd,2}^{-}$ (sagging)
306.1 kNm	224.7 kNm	416.6 kNm	305.6 kNm

Note: *The sign for the bending moment is here defined according to the German sign convention.*

Based on these values, a force N_{pl} equal to 269 kN is obtained.

- Contribution of the arching effects

In analogy to previous sections, the vertical point force N_{arch} needed to overcome the arching effect is computed.

The arching effects are activated if the compression resistance of any activated component of the system once the above mechanism has developed is not governing, in other words if the failure mode of the platform is not a component (i.e., a joint or a beam) in compression. In such conditions, an arch effect can be mobilised within the beams of the directly affected part as soon as the plastic mechanism has formed. The following table summarizes the failure modes of concerned joints.

Table 47. Failure modes of the partial-strength joints

Joint	Sagging / hogging	Failure mode
B1/B3	hogging (+)	Column web in compression
B1/B3	sagging (-)	Column web in compression
C2/C3	hogging (+)	Column web in compression
C2/C3	sagging (-)	Column web in compression

As all joints fail in compression, no arch effect can be activated, so that N_{arch} = 0 kN.

Verification of the structure with partial-strength joints

Contribution from the slab, the beam mechanism and the arch effect can be cumulated as their activation requires limited deformation capacities. The total resistance is then:

$$N = N_{slab} + N_{pl} + N_{arch} = 313.6 + 269.0 + 0.0 = 582.6\ kN$$

The vertical action applied when the column is lost equals the vertical axial force in internal columns and has been estimated as equal to 694.2 kN. As the sum of resistances of all the above contributions is lower than the vertical axial force, the structure cannot be assumed as robust.

This means that significant vertical displacements of the directly affected part will develop with the apparition of membrane forces $N_{membrane}$ in the beams. Such membrane forces cannot be cumulated with the contributions coming from the slab and from the arching effects as they disappear once large deformations are reached.

Worked example II.4.1 / SS/NS	Design for unidentified threats using ALPM – analytical approach – SS/NS	8 of 11 pages

The contribution $N_{membrane}$ requires the adoption of advanced design methods due to M-N interaction in the joints. This contribution would require significant deformation capacities at the level of the partial-strength joints. In such situation, the required levels of deformation capacities are not achievable in most of the cases, so that this contribution won't be assessed here.

As already stated above, ductile joints (ductile joint failure modes) are required for the assumption of plastic hinges forming at the level of the joints. The failure mode of joints is here column web in compression under bending moments. As this component is not considered as ductile, these joints need to be redesigned. This will be assessed with the robustness redesign in the next part.

- Redesign of the structure with partial-strength joints

Before performing the redesign, it has to be noted that, in a consistent way, the use of semi-rigid joints would modify the internal forces distribution in the structure. Smaller beam deflections at SLS could be expected, so that smaller beam cross-sections could be used. But bending moments in columns would also appear so that larger column cross-sections might be required. However, for usual buildings, the column cross-sections don't need to be upgraded due to the additional restraint coming from the beam-to-column joint stiffnesses. In the framework of this worked example, the steel structure has been kept as it is (designed with internal forces with the simple joint modelling). Modelling semi-rigid joints as hinges is still a valid and safe assumption if these joints have enough ductility and rotation capacity.

There are several ways of achieving the robustness requirements, such as:

- Modify the slab design to increase the contribution from the slab mechanism;
- Strengthen the joints in one or both directions to increase the contribution of the beam mechanism;
- Reinforce compression components to activate the arch effect.

In order to show the contribution of the arch effect in practice, we mainly chose here to modify the joints C2/C3 as shown in the following figure.

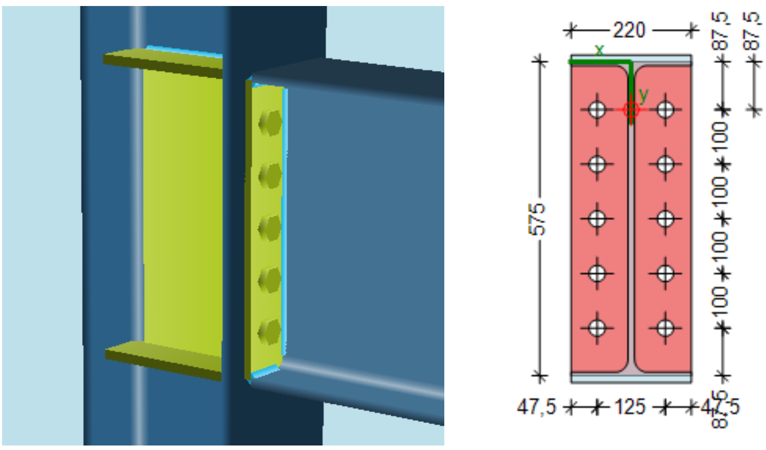


Figure 95. Redesign of joint C2/C3 to fulfill robustness requirements

Worked example II.4.1 / SS/NS	Design for unidentified threats using ALPM – analytical approach – SS/NS	9 of 11 pages

Changes are as follows:

- Column stiffeners (same thickness as beam flanges);
- Web stiffener;
- Adapted bolt pattern;
- Flange welds changed from 6 to 7 mm.

Column and web stiffeners are needed to activate the arch effect (see below). Note that specific rules from the EN 1993-1-8 have to be fulfilled in order to take web plates into account in the joint verification. As hogging and sagging bending moments play a role in the beam mechanism as well as in the arch effect, the unsymmetrical bolt pattern has been modified to a symmetrical one. M27 bolts (instead of M24) have been chosen to still fulfill the ULS shear force verification. Finally, the flange welds have been increased for ductility issues.

Modifications of the B1/B3 joint are needed to increase the contribution of the beam mechanism and reach the robustness requirements.

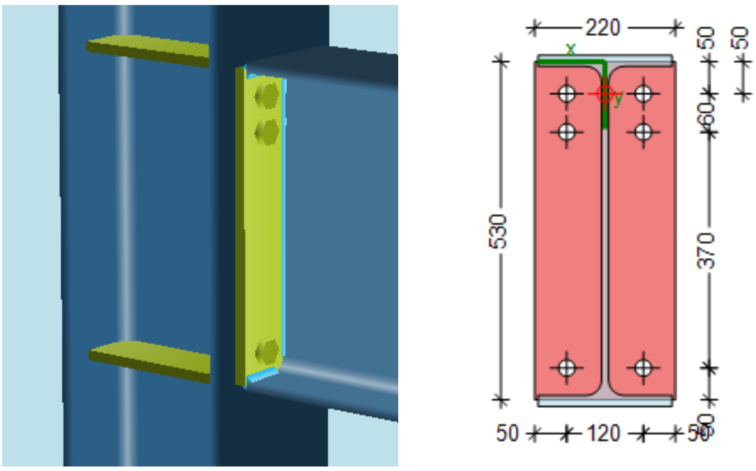


Figure 96. Redesign of joint B1/B3 to fulfill robustness requirements

Changes are as follows:

- Column stiffeners (same thickness as beam flanges);
- End-plate thickness changed from 15 to 20 mm;
- Flange welds changed from 6 to 7 mm.

Changes in this joint allow to increase the bending moment resistance of the joint and thus the beam mechanism. The bolt pattern remains unchanged.

a) Contribution of the slab

As no changes have been made to the slab, the contribution of this component remains unchanged (N_{slab} = 313.6 kN).

b) Contribution of the beam mechanism

The sagging and hogging bending moment resistances of the redesigned joints are given in the table below.

Table 48. Bending moment resistances of the redesigned joints B1/B3 and C2/C3

Joint B1/B3		Joint C2/C3	
$M_{pl,Rd,1}^{+}$ (hogging)	$M_{pl,Rd,1}^{-}$ (sagging)	$M_{pl,Rd,2}^{+}$ (hogging)	$M_{pl,Rd,2}^{-}$ (sagging)
368.9 kNm	285.4 kNm	451.3 kNm	451.3 kNm
CWS	CWS	EPB	EPB

From these values, N_{pl} is obtained as equal now to 334.7 kN

a) Contribution of the arching effect

In the framework of this example, only the arching effect coming from the short frame (IPE600 with C2/C3 joints) is accounted for, as illustrated in two dimensions in Figure 97.

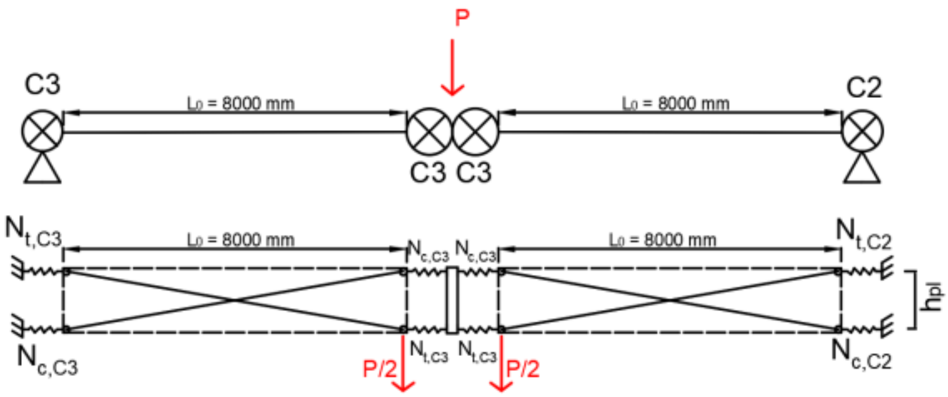


Figure 97. Model applied for the arching effect

Indeed, there will be no contribution coming from the other direction as the failure mode of the joints B1/B3 is column web in shear. This means that once the moment resistance of these joints is reached, there is no way to increase the tension forces in the rows to contribute to an extra arching effect.

For the redesigned joint C2/C3, the failure mode is end-plate in bending and the main joint properties are listed in the table below. As the joint is now symmetrical, values for hogging and sagging are identical.

Table 49. Properties of the C2/C3 joint

Moment resistance	M_{Rd}	451.3 kNm
Initial rotational stiffness	$S_{j,ini}$	114000 kNm/rad
Sum of tension forces in rows	F_t	1369.4 kN
Stiffness coefficient of BFC	k_7	$+\infty$
Stiffness coefficient of CWS	k_1	9.461 mm
Stiffness coefficient of BFC	k_2	$+\infty$
Compression resistance	F_c	1783 kN

Worked example II.4.1 / SS/NS	Design for unidentified threats using ALPM – analytical approach – SS/NS	11 of 11 pages

Note that the compression resistance of the joint is governed by the component column web in shear.

Intermediate results of the method are summarized in the table below. More details about the method can be obtained in the Deliverable D2.2 of the FAINOMORE project freely available on the project's website. A similar calculation can also be found in (Vermeylen, 2021) for other input parameters.

The following assumptions have been made:

- Since the IAP is made of diaphragms, its lateral displacement has been neglected;
- Since joints C2 and C3 are similar, they have been considered as identical in terms of stiffness and resistance.

Table 50. Parameters of the arching effect method

Vertical displacement of the beam	Δ_{beam}	36.9 mm
Vertical displacement due to joints rotation	Δ_{joints}	63.3 mm
Total vertical displacement due to the beam mechanism	Δ_{pl}	100.3 mm
Sum of tensile loads in the joint when mechanism forms	F_t	1369.4 kN
Effective compression stiffness of the joint	$k_{eff,c}$	9.461 mm
Elastic compression shortening of the joint	$\delta_{c,el}$	0.689 mm
Length of arch rod when plastic mechanism forms	L_D	8017.0 mm
Compression resistance of the joint	F_c	1783 kN
Plastic compression shortening of the joint at failure	$\delta_{c,pl}$	0.897 mm
Inclination of the arch rod at failure	ϑ	0.062 rad
Buckling resistance of the arch rod (safe approach)	$N_{b,Rd}$	231.7 kN

From these values, a force N_{arch} equal to 51.0 kN is obtained.

This contribution can be cumulated to the ones coming from the beam and slab plastic mechanisms as the activation of this arching effect required limited deformation capacities.

By cumulating all of the above contributions, the total resistance is now:

$$N = N_{slab} + N_{pl} + N_{arch} = 313.6 + 334.7 + 51.0 = 699.3\ kN$$

The resistance is now greater than the vertical axial force of 694.2 kN, so that the redesigned structure can now be assumed as robust.

As noted in W.E. II.4.5 SS/NS, further to the column loss, the axial forces in the columns adjacent to the DAP are increasing, yet they remain lower than the axial forces associated to the ULS. Therefore, since the same accidental load combination has been considered in the present worked example, it is reasonable to assume that the buckling of these columns will not occur, and the robustness of the structure will not be affected by this failure mode.

8.8.4.2 Design for unidentified threats using ALPM - simplified prediction of the dynamic response (SS/S)

FAIL NO MORE Worked example	Title	Design for unidentified threats using ALPM – simplified prediction of the dynamic response			1 of 2 pages
	Structure	Steel structure in seismic zone	Made by	UPT	Date: 06/2021
	Document ref.	II.4.2 / SS/S			

Example: Design for unidentified threats in a steel structure in seismic zone using alternate load path method - simplified prediction of the dynamic response

Design manual § 5.3.3

This example gives information about the design against unidentified threats using the simplified approach to predict the dynamic response further to a column loss scenario.

Basic data of structure

Design manual § 8.2.

- For geometry, sections, and materials, see Section 8.2.

Actions for Accidental Design Situation

The following actions are considered:

- Permanent loads DL (see Table 11);
- Live loads LL (see Table 11 for SS/S structure);
- No specific accidental action is taken into account.

Combination of actions for Accidental Design Situation

EN 1990 §6.4.3.3, Eq 6.11b

$$DL + 0.5 \times LL$$

Definition of computation hypothesis

The simplified numerical method adopted for the current example allows one to establish the maximum ductility demand and verifying the demand versus capacity ratio. However, to determine the response of the structure for a column removal scenario, a nonlinear static analysis was performed. Consequently, considering the energy balance between the work done by the loading and the internal energy stored, the pseudo-static response was determined.

Considered column removal scenario:

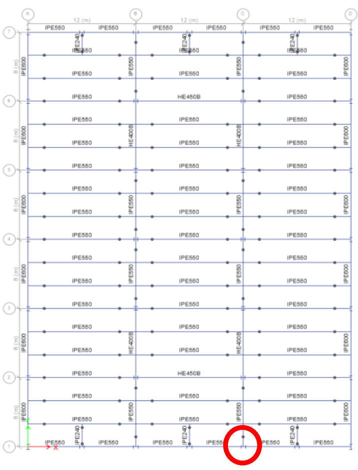


Figure 98. Column removal scenario – ALPM -simplified method – SS/S

Worked example II.4.2 / SS/S	Design for unidentified threats using ALPM – simplified prediction of the dynamic response – SS/S	2 of 2 pages

According to scenario presented in Figure 98, the column considered to be removed is at the ground floor.

Structural analysis

For this method, a 3D nonlinear static numerical analysis was performed on the model in SAP2000 software. The gravitational loading was assigned according to the previously mentioned combination. The loading was applied only on the zone connected with the column, i.e., two marginal frames on X direction and one frame in Y direction. Furthermore, the column displacement was imposed downwards up to the attainment of failure.

Geometrical and material nonlinearities (plastic hinges) were considered in the analysis.

The pushdown curve for scenario C1 is curve PD in Figure 99. On the vertical axis, the force has been normalized with gravity load multiplier λ (λ=1 for an applied load of 1.0 DL + 0.5 LL). The analysis was performed up to reaching failure.

After performing the energy balance (Izzuddin et al., 2008), the pseudo-static curve was determined and plotted comparatively with the pushdown curve – pseudo-static curve in Figure 99.

See Section §5.3.5

Results

The results show that, for λ=1, limited plastic deformations in the pseudo static curve are presented in Figure 99.

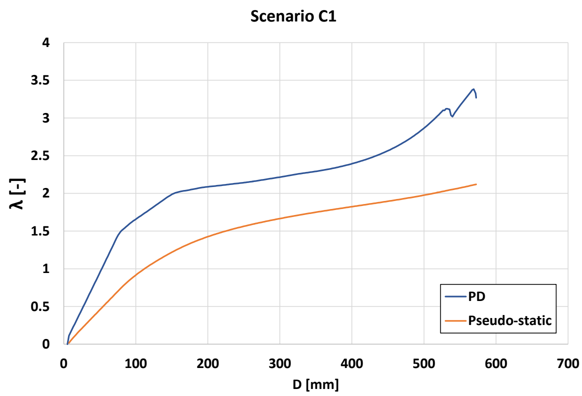


Figure 99 Normalized force multiplier vs. vertical displacement for push-down and pseudo-static curves – ALPM – simplified numerical approach – SS/S

Conclusions

- For the considered column removal scenario, the structure has resistance and ductility capacity to find alternate load paths and to avoid the progressive collapse.

Flowchart Figure 3 – Box C.4 → End of design

- The simplified numerical approach starting from a nonlinear static analysis provides a practical assessment of the ductility demand for design against progressive collapse. Compared with the full numerical analysis, the procedure is engineering oriented and may be performed faster. Even though the nonlinear dynamic analysis allows for more precise results, taking implicitly the dynamic amplification of the loading, the results provided using this method are comparable.

8.8.4.3 Design for unidentified threats using ALPM - full numerical approach (SS/S)

FAIL NO MORE Worked example	Title	Design for unidentified threats using ALPM - full numerical approach			1 of 7 pages
	Structure	Steel structure in seismic zone	Made by	UPT	Date: 06/2021
	Document ref.	II.4.3 / SS/S			

Example: Design for unidentified threats in a steel structure in seismic zone using alternate load path method - full numerical approach

Design manual §5.3.4

This example gives information about the design against unidentified threats using the ALPM and nonlinear dynamic analysis.

Basic data of structure

Design manual § 8.2.

- For geometry, sections, and materials, see section 8.2.

Actions for Accidental Design Situation

The following actions are considered:

- Permanent loads DL (see Table 11);
- Live loads LL (see Table 11 for SS/S structure);
- No specific accidental action is taken into account.

Combination of actions for Accidental Design Situation

EN 1990 §6.4.3.3, Eq 6.11b

$$DL + 0.5 \times LL$$

Note: *This combination is valid for dynamic analysis only, because the dynamic effects caused by the column loss are considered implicitly by means of the removal duration parameter.*

Definition of column removal scenarios

The scenarios taken into consideration are presented in Figure 100.

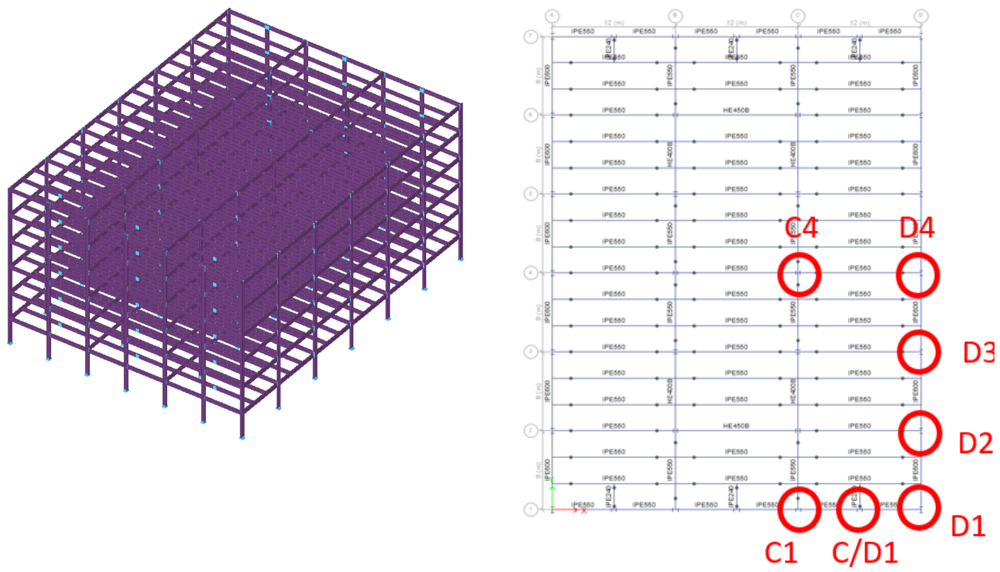


Figure 100. Isometric view of the structure (left) and location of columns to be removed for ALPM – full numerical approach – SS/S

Worked example II.4.3 / SS/S	Design for unidentified threats using ALPM – full numerical approach – SS/S	2 of 7 pages

Structural analysis

The objective of this analysis is to evaluate the behaviour of the building in case of accidental situation (column removal). The calculations are made using the ELS (Extreme Loading for Structures software) using the full 3D model of the structure.

Details about the numerical model are given in W.E. I.1.3 / CS/S. The model has been calibrated against relevant tests. The gravity loads were calculated using the combination of actions defined above and assigned to all floors.

Analysis:

- **1st step:** All gravity loads assigned to the floors using a static analysis
- **2nd step:** Duration of column removal is 0.001 seconds

Results

Figure 101 presents the time-history vertical displacement curves for each column removal scenario. As it can be seen, for case C4, the column removal causes progressive collapse on the entire affected area - see

Figure 102.

For cases C/D1, D1, D2, D3, D4 the structure has the capacity to resist the progressive collapse.

Figure 103 presents the deformed shape in case of D2 column removal scenario. The deformations are small and the resisting mechanism is based on flexural capacity (see Figure 104 and Figure 105), without the initiation of catenary action in beams (see Figure 106).

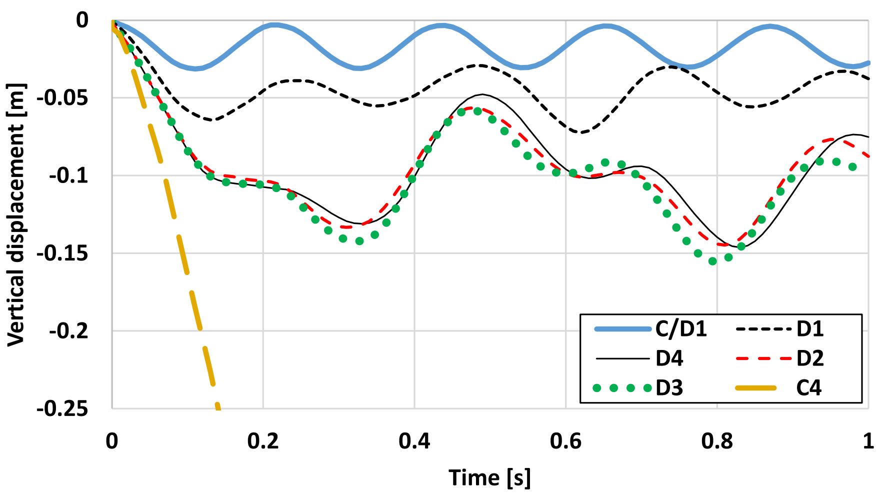


Figure 101. Time-history vertical displacement curves for removed columns

Worked example II.4.3 / SS/S	Design for unidentified threats using ALPM – full numerical approach – SS/S	3 of 7 pages

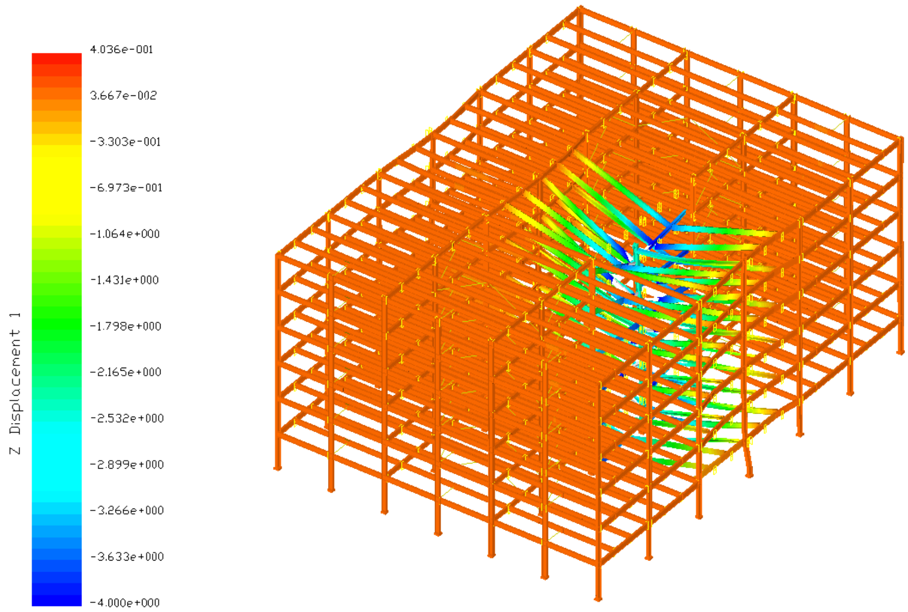


Figure 102. Failure mode after C4 after column removal

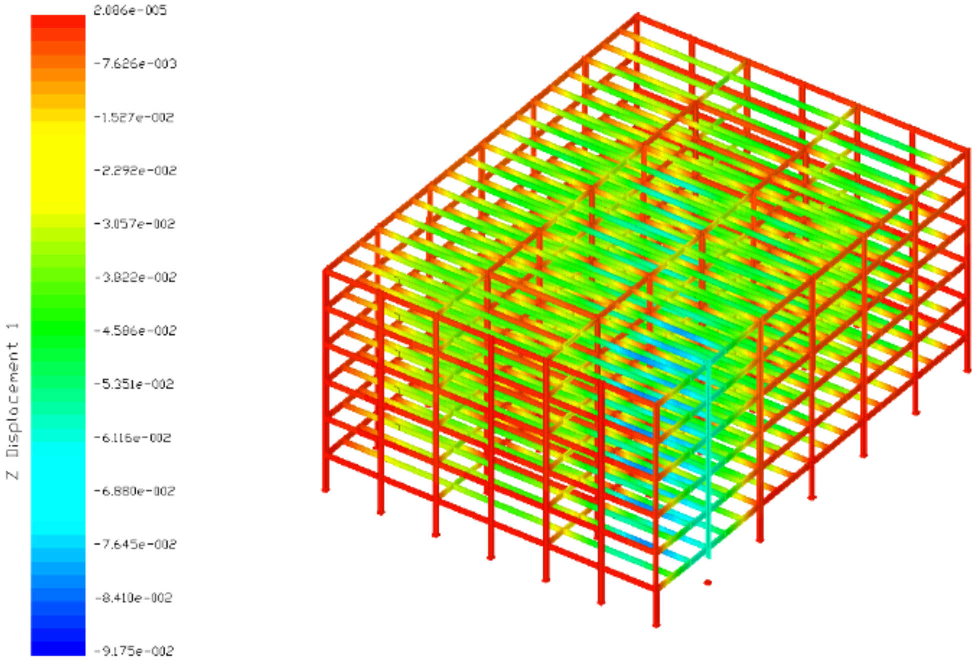


Figure 103. Vertical displacement of the structure in case of D2 column removal scenario [m]

Worked example II.4.3 / SS/S	Design for unidentified threats using ALPM – full numerical approach – SS/S	4 of 7 pages

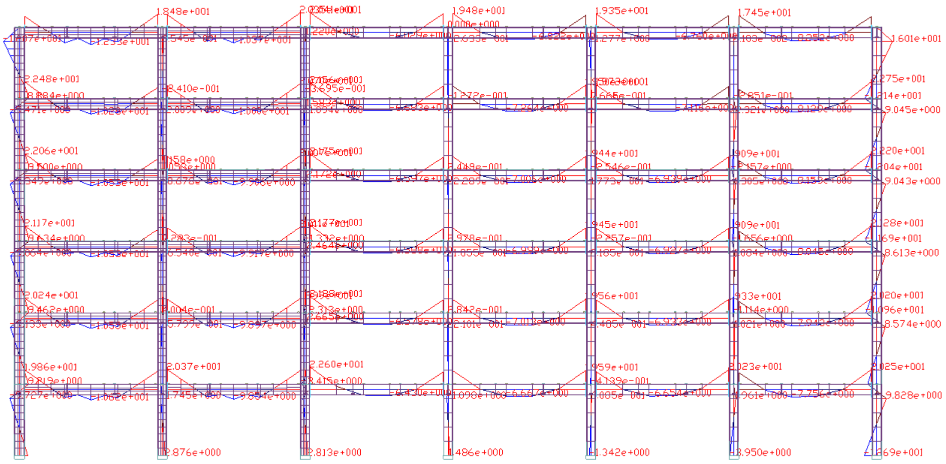


Figure 104. Bending moment diagram before D2 column removal scenario [tf m].

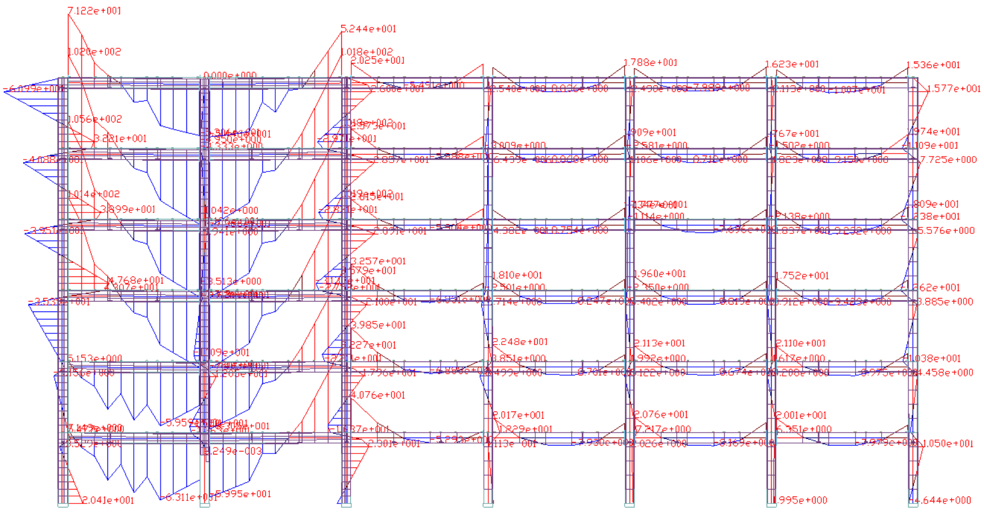


Figure 105. Bending moment diagram after D2 column removal scenario [tf m].

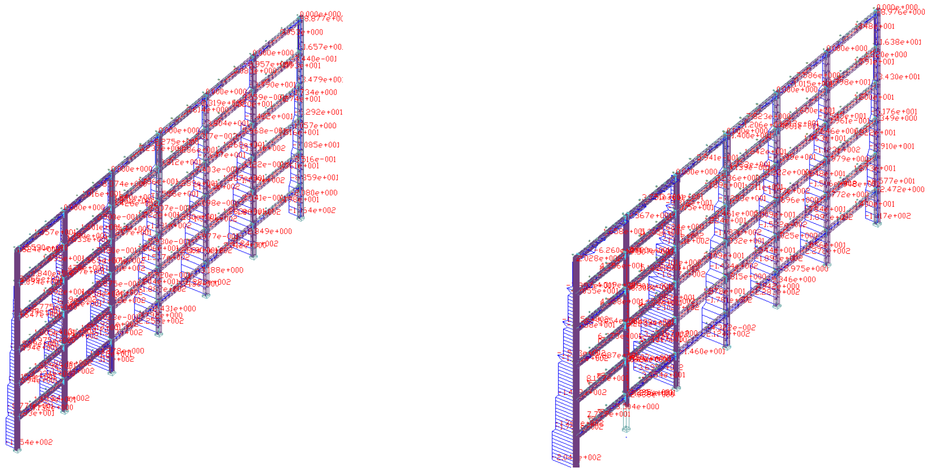


Figure 106. Axial force diagram before and after D2 column removal scenario[tf].

Worked example II.4.3 / SS/S	Design for unidentified threats using ALPM – full numerical approach – SS/S	5 of 7 pages

The results presented above were obtained using the design level of gravity loads: $DL + 0.5 \times LL$ (i.e., $\lambda = 1$). To evaluate the strength reserve against progressive collapse for cases C/D1, D1, D2, D3, D4, the gravity loads were increased by means of the gravity load multiplier λ. Then, the columns were removed using the same procedure as described above.

In the following, only the results for scenario D4 are discussed. As it can be seen from Figure 107, the progressive collapse is initiated for $\lambda = 1.4$ due to the failure of beam-to-column joints of IPE600 beams.

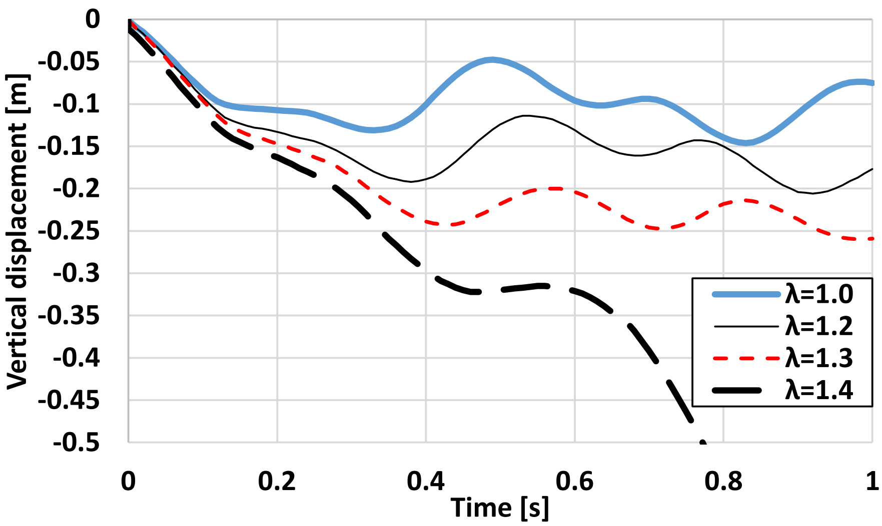


Figure 107. Time-history vertical displacement curves for scenario D4 at different gravity load multiplier λ

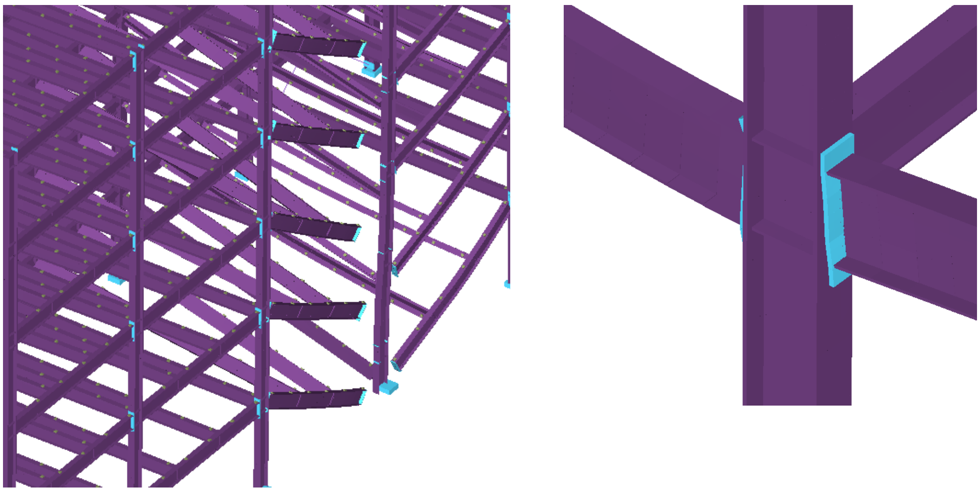


Figure 108. Failure of beam-to-column joint triggers the progressive collapse (scenario D4, λ = 1.4)

<u>*Remarks*</u>

- In the case of C4 column removal, where all adjacent beams are pinned, the structure is not able to transfer the loads, thus undergoing progressive collapse. The structure needs to be redesigned.

Worked example II.4.3 / SS/S	Design for unidentified threats using ALPM – full numerical approach – SS/S	6 of 7 pages

- All other scenarios result in safe response of the structure (plastic deformations develop but progressive collapse is prevented);
- If higher gravity loads are present on the structure, progressive collapse may also initiate – see case D4, λ = 1.4.

The redesign can be done using different strategies. The most efficient strategy is based on the activation of the catenary effects. Considering that the weak point is the capacity of beam-to-column connection, the strengthening strategy should involve reinforcement of the connection by means of end-plate rib stiffeners at both top and bottom sides of beam ends. The effects of this reinforcement are illustrated below for D4 column loss.

- Results of improving the connection typology

To compare the efficiency of the stiffening technique, a push-down analysis is performed on the structure with EP connections and on the structure with stiffened connections (EPS).

The loss scenarios analysis assumes the column D4 is removed, then the gravity load on the floors is incremented up to the attainment of failure, obtaining the so-called capacity curve. Figure 109 (left) presents comparatively the capacity curves before and after the strengthening of the connection (EP vs. EPS for scenario D4). As seen, the unstiffened end-plate connection has a limited deformation capacity and fails before the development of any catenary action in beams. The stiffened connections have a resistance higher than the beam. They are full-strength joints and the plastic deformation develop in the beam ends rather than in the connections (see Figure 110). This allows a significant increase in capacity, partly in flexural, but mostly in catenary.

In Figure 109 (right) the results obtained in the numerical analysis are compared with the EC8 load deformation relation given in prEN 1998-1-2:2019.3, Annex L. As it may be seen, the EPS connection has a large capacity reserve, therefore the use of seismic based acceptance criteria (EC8) can be quite conservative, column removal situations where adjacent main beams have continuous connections result in limited vertical deflections.

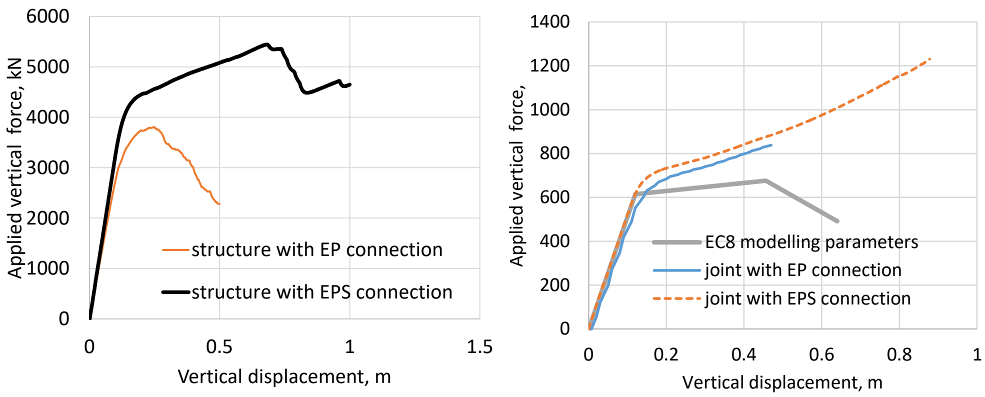


Figure 109. pushdown curves for structure (left) and for one frame with one level (right)

Worked example II.4.3 / SS/S	Design for unidentified threats using ALPM – full numerical approach – SS/S	7 of 7 pages

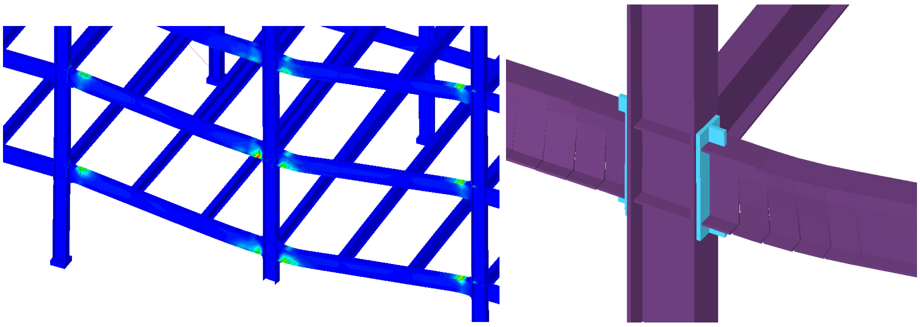


Figure 110. Structure with SEP: stain map on failure mode (left) and detail (right)

<u>*Conclusions*</u>

- The loss of perimeter column does not lead to propagation of damage and the structure has the capacity to resist the loss. The perimetral columns have no problems in finding alternate load pats to redistribute the load for a gravity load multiplier of λ=1, withstanding almost double the load.
- When the column loss affects a seismic resistant frame (i.e., perimeter frame), the damage is limited to the directly affected area, and the progressive collapse is prevented.
- When the local damage (i.e., column loss) affects the internal structure with pinned beam ends (B4 and C4), the damage propagates, and the progressive collapse develops on the entire affected area. The pinned connections cannot resist the large axial force demands resulted from the column loss. To limit the damage and prevent the progressive collapse, the alternatives to the pinned connection strengthening (which may be difficult to attain) are:

Flowchart Figure 3 – Box C.4 ➔ End of design

- use of moment resisting connections instead of pinned connections (redesign);
- use of composite action of the beam with the concrete slab (see II.4.4 / CS/S);
- the columns designed as key elements;
- the hazard leading to column loss reduced or eliminated.

Flowchart Figure 3 – Box C.4 ➔ C.2

8.8.4.4 Design for unidentified threats using ALPM - full numerical approach (CS/S)

FAIL NO MORE Worked example	Title	Design for unidentified threats using ALPM - full numerical approach			1 of 2 pages
	Structure	Composite structure in seismic zone	Made by	UPT	Date: 06/2021
	Document ref.	II.4.4 / CS/S			

Example: Design for unidentified threats in a composite structure in seismic zone using alternate load path method - full numerical approach

Design manual §5.3.4

This example gives information about the design against unidentified threats using the full numerical approach from ALPM.

Basic data of structure

Design manual § 8.2.

- For geometry, sections, and materials, see Section 8.2.

Actions for Accidental Design Situation

The following actions are considered:

- Permanent loads DL (see Table 11);
- Live loads LL (see Table 11 for CS/S structure);
- No specific accidental action is taken into account.

Combination of actions for Accidental Design Situation

EN 1990 §6.4.3.3, Eq 6.11b

$$DL + 0.5 \times LL$$

Definition of column removal scenarios

The same scenarios are used as in the case of W.E II.4.3 / SS/S, see Figure 111.

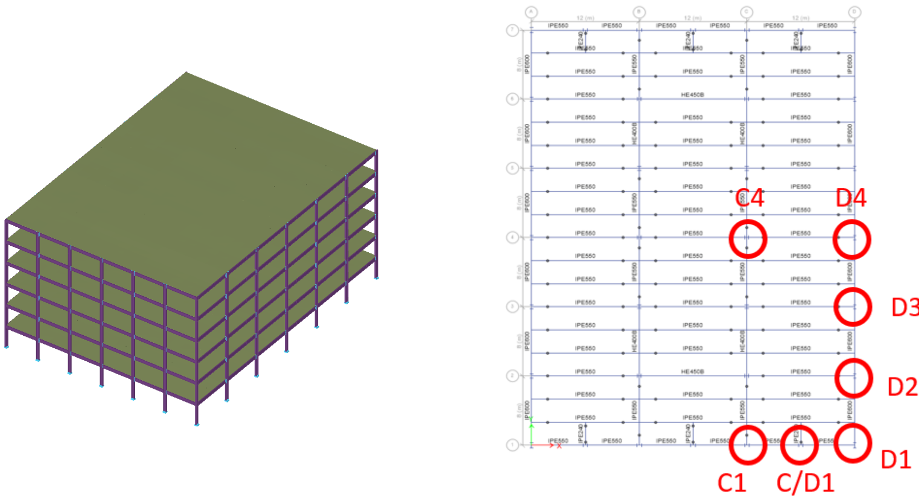


Figure 111. Isometric view of the structure (left) and location of columns to be removed for ALPM – full numerical approach (right)– CS/S

Structural analysis

Modelling assumptions and analysis procedure follow the same methods as presented in W.E. II.4.3 / SS/S. The only difference is the addition of the concrete slab (concrete and reinforcement) and the interaction with the steel structure (shear studs). Details are given in Table 12. Note that the steel structure (elements and connections) is the same as in case of the bare steel structure SS/S.

Worked example II.4.4 / CS/S	Design for unidentified threats using ALPM – full numerical approach – CS/S	2 of 2 pages

Results

The results of the NDP show that the CS/S structure has the capacity to resist progressive collapse for all removal scenarios, including scenario C4 which proved to be critical for structure SS/S. Figure 112a shows comparatively the force displacement curve CS/S and SS/S for scenario C4 and gravity load multiplier λ = 1. Figure 112b shows the deformed shape for CS/S. The structure exhibits limited plastic deformation in steel elements and concrete slab in the area affected by the column loss – see Figure 112c,d.

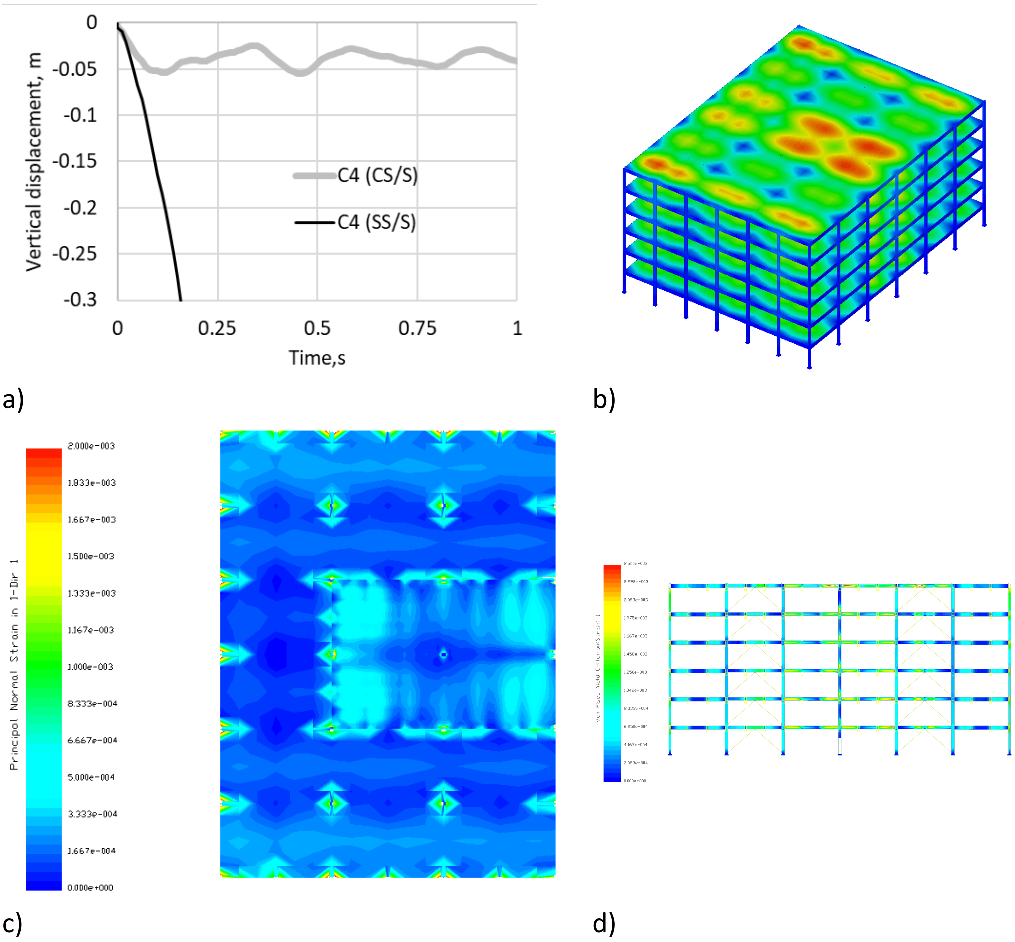


Figure 112. Results for CS/S and scenario C4: a) vertical force vs vertical displacement – CS/S vs SS/S, b) isometric view of the deformed structure, c) current plan view with the deformations in the concrete slab (bottom side), d) deformations in steel elements frame C/ 3-5

Conclusions

- The interaction between steel frame and concrete slab provides additional capacity to resist the column loss without the development of progressive collapse.
- The steel concrete interaction is beneficial especially for frames with pinned beam ends as the axial force requirement in beams to allow the development of catenary action can be excessive.

Flowchart Figure 3 – Box C.4 ➔ End of design

8.8.4.5 Design for unidentified threats using ALPM - full numerical approach (SS/NS)

FAIL NO MORE Worked example	Title	Design for unidentified threats using ALPM - full numerical approach			1 of 11 page
	Structure	Steel structure in non-seismic zone	Made by	F+W	Date: 06/2021
	Document ref.	II.4.5 / SS/NS			

Example: Design for unidentified threats in a steel structure in non-seismic zone using alternate load path method - full numerical approach	Design manual §5.3.4

This example gives information about the design against unidentified threats using the ALPM through a full numerical approach.

Basic data of structure

- For geometry, sections, and materials, see Section 8.2;
- For the initial joint properties, see W.E. II.4.1 / SS/NS.

Actions for Accidental Design Situation

The following actions are considered:

- Permanent loads DL (see Table 11);
- Live loads LL (see Table 11 for SS/NS structure);
- No specific accidental action is taken into account.

Combination of actions for Accidental Design Situation — EN 1990 §6.4.3.3, Eq 6.11b

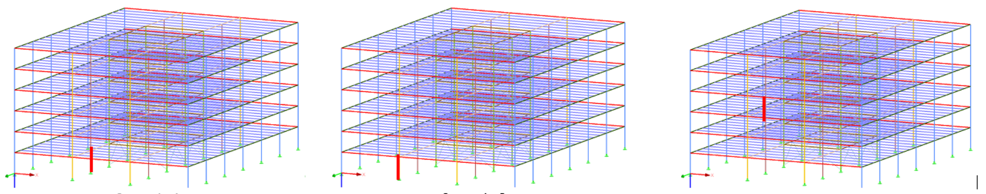
$$DL + 0.5 \times LL$$

Definition of column removal scenarios

Three possible different column loss scenarios are considered for this worked example:

- Scenario 1: Inner column loss at floor 0;
- Scenario 2: Facade column loss at floor 0;
- Scenario 3: Inner column loss above column splice.

Those 3 scenarios are illustrated in Figure 113 (lost column highlighted in red).



Scenario 1 Scenario 2 Scenario 3

Figure 113. Investigated column loss scenarios in the ALPM - full numerical approach – SS/NS

Remarks

This example illustrates 3 column loss scenarios. However, in practical application, other column loss scenarios might be required (see Section 5.2). Therefore, it is up to the engineer to define which scenarios might be possible and which of them are the most relevant for the robustness design of the structure. (DoD 2008)

Worked example II.4.5 / SS/NS	Design for unidentified threats using ALPM – full numerical approach – SS/NS	2 of 11 page

Structural analysis

The full numerical approach will be addressed using the finite element model developed for the ULS/SLS design of the structure. The aim is to remove a column and let membrane effects develop in the ties in the first step and then verify if the ties (members and joints) can withstand these tensile forces.

- Methodology and assumptions:

The FE analysis is performed using a Newton-Raphson algorithm allowing the integration of large deformations. As this can lead to lateral-torsional buckling of the beams for which no instability can occur in reality as they are maintained by the diaphragms, we prevent this instability to occur by fictitiously increasing the torsional inertia of the beam members.

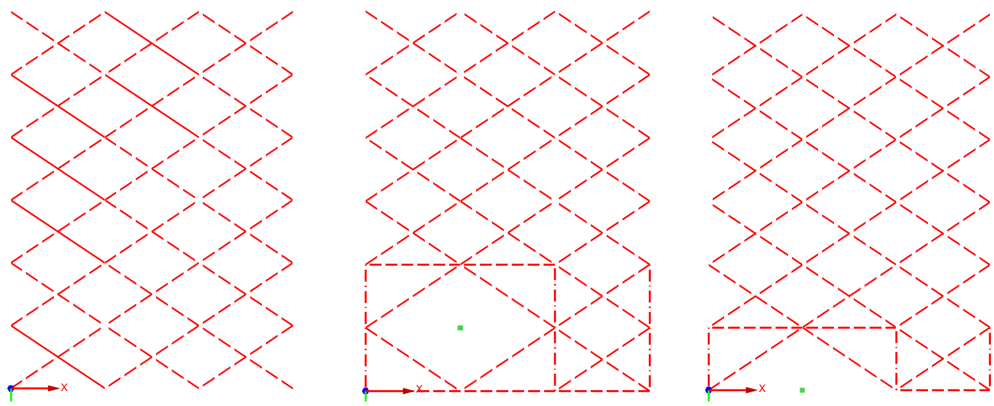
Remark

- Even if plastic deformations may develop following a column loss, material nonlinearities (plasticity) were not taken into account.

To ensure convergence of the algorithm, the column loss scenario is modelled as follows:

- First, the structure is analysed without any column loss under the accidental load case combination. From this, the actual compression force in column to be lost is known;
- Then at the upper node of the column to be lost, this force is applied and the column is removed, so that this force replaces the column;
- The last step simulates the column loss: A force of same magnitude in opposite direction is gradually applied at the same node. Load steps of 0.025 are used to ensure convergence. At the end of the analysis, the statical system corresponds to a complete column loss. Note that dynamic effects of the column loss are not considered in this worked example.

To avoid any composite action between diaphragms and the steel structure but still keep the diaphragm effect (infinitely rigid decks), diaphragm models have to be modelled and adapted for column loss scenarios, which are presented in Figure 114.



No column loss   Scenario 1 and 3   Scenario 2

Figure 114. Coupling elements pattern for diaphragm modelling in various column loss scenarios

Worked example II.4.5 / SS/NS	Design for unidentified threats using ALPM – full numerical approach – SS/NS	3 of 11 page

Tying forces and deformations:

Results of all considered column loss scenarios are illustrated in the following figures.

- Scenario 1: Inner column loss at floor 0 (Figure 115 to Figure 119)

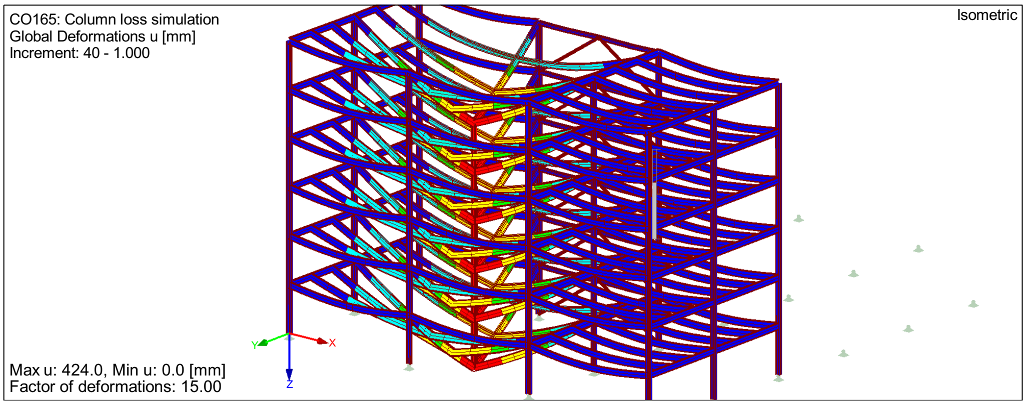


Figure 115. Deformed system (directly affected part) after column loss (scenario 1)

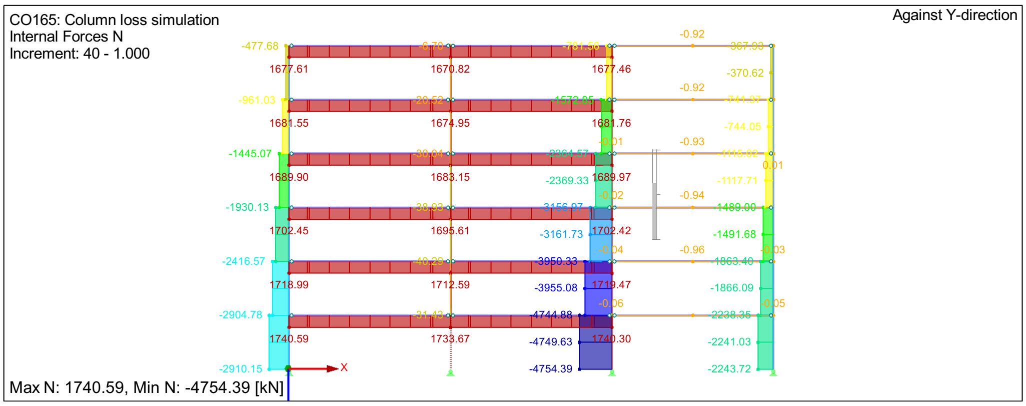


Figure 116. Normal internal forces in IPE550 frame after column loss (scenario 1)

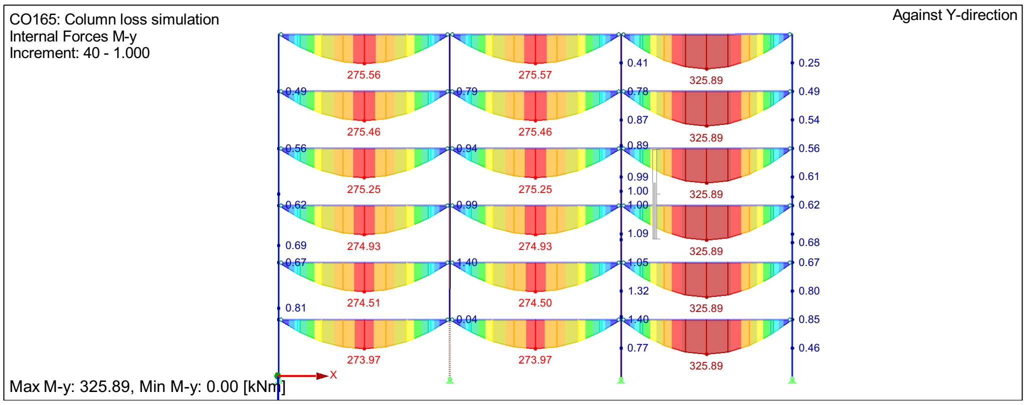


Figure 117. Bending moments in IPE550 frame after column loss (scenario 1)

Worked example II.4.5 / SS/NS	Design for unidentified threats using ALPM – full numerical approach – SS/NS	4 of 11 page

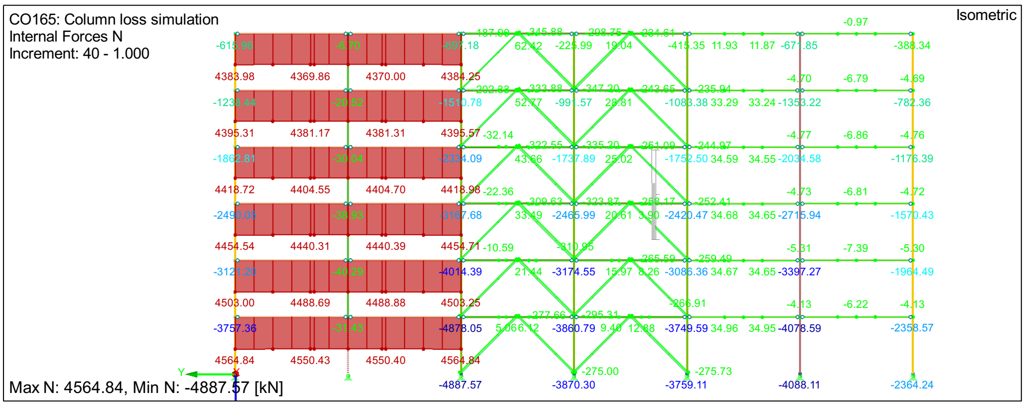


Figure 118. Normal internal forces in IPE600 frame after column loss (scenario 1)

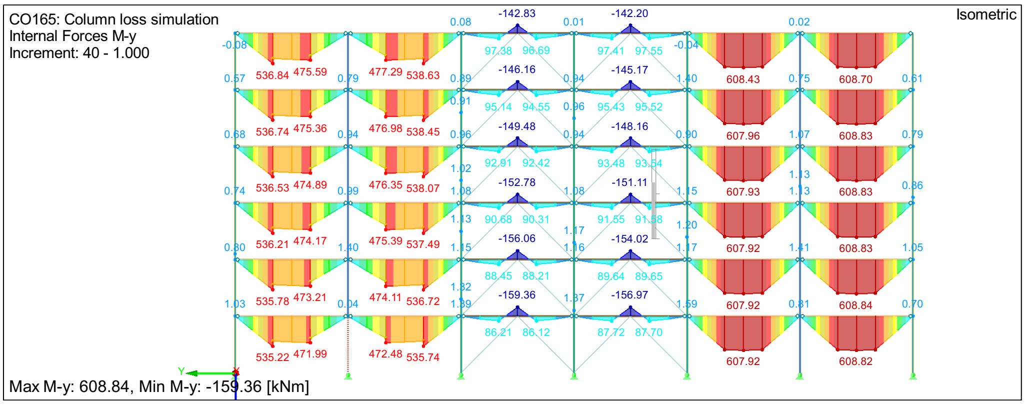


Figure 119. Bending moments in IPE600 frame after column loss (scenario 1)

- Scenario 2: Facade column loss at floor 0 (Figure 120 to Figure 122)

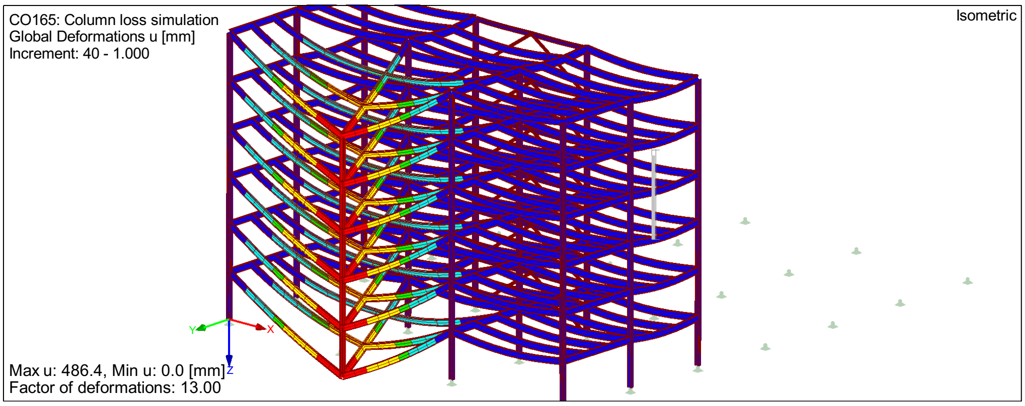


Figure 120. Deformed system (directly affected part) after column loss (scenario 2)

Worked example II.4.5 / SS/NS	Design for unidentified threats using ALPM – full numerical approach – SS/NS	5 of 11 page

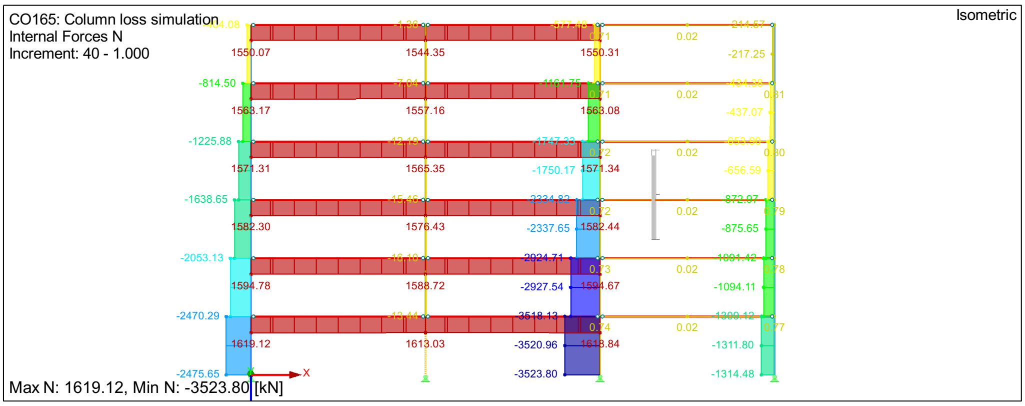


Figure 121. Normal internal forces in IPE500 frame after column loss (scenario 2)

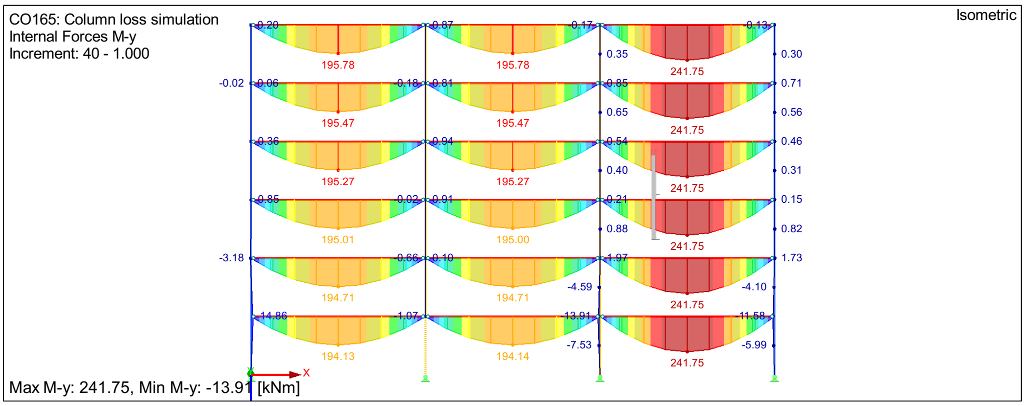


Figure 122. Bending moments in IPE500 frame after column loss (scenario 2)

- Scenario 3: Inner column loss above column splice (Figure 123 to Figure 127)

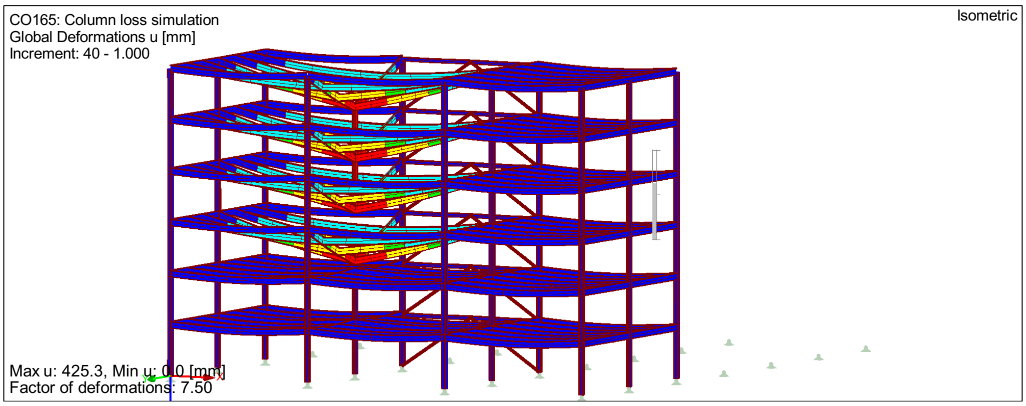


Figure 123. Deformed system (directly affected part) after column loss (scenario 3)

Worked example II.4.5 / SS/NS	Design for unidentified threats using ALPM – full numerical approach – SS/NS	6 of 11 page

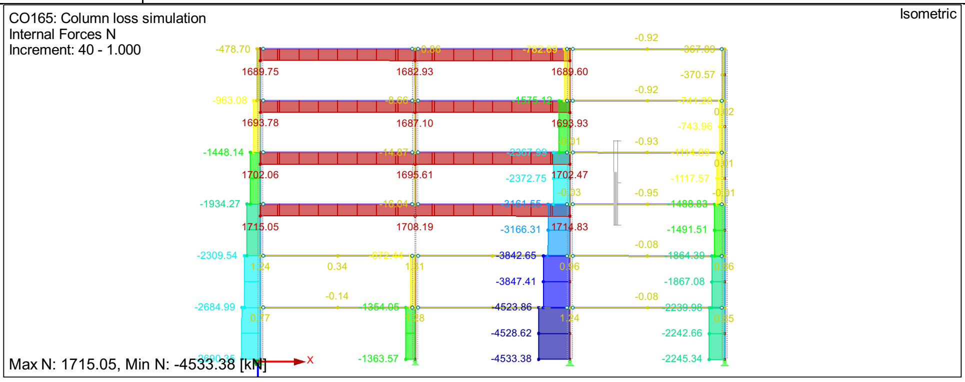


Figure 124 Normal internal forces in IPE550 frame after column loss (scenario 3)

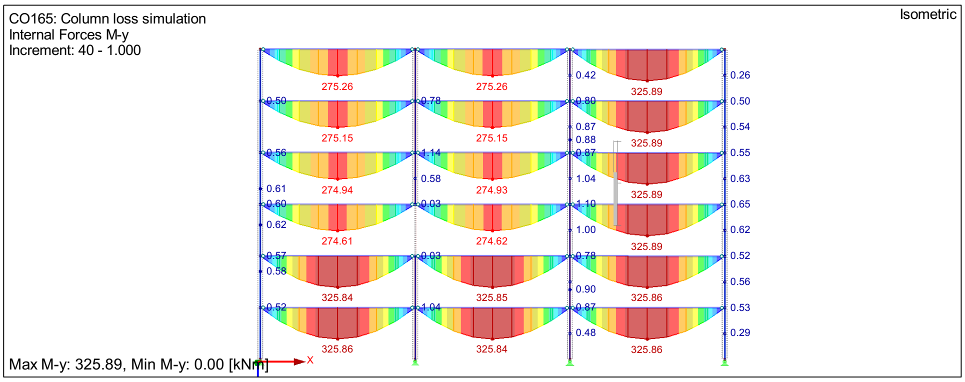


Figure 125. Bending moments in IPE550 frame after column loss (scenario 3)

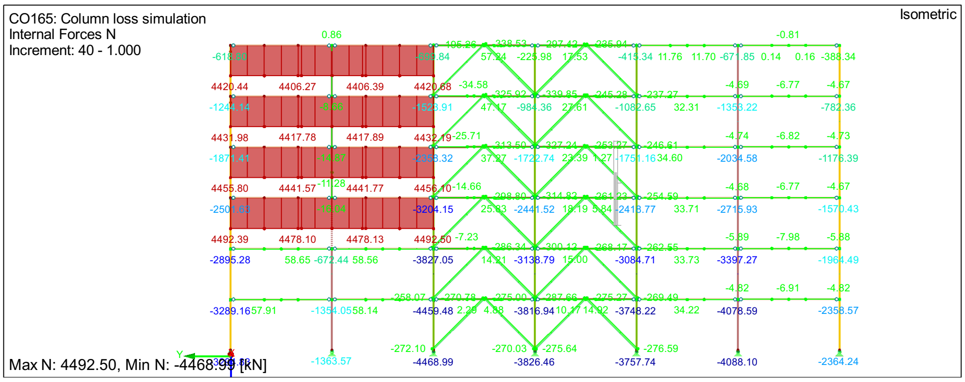


Figure 126. Normal internal forces in IPE600 frame after column loss (scenario 3)

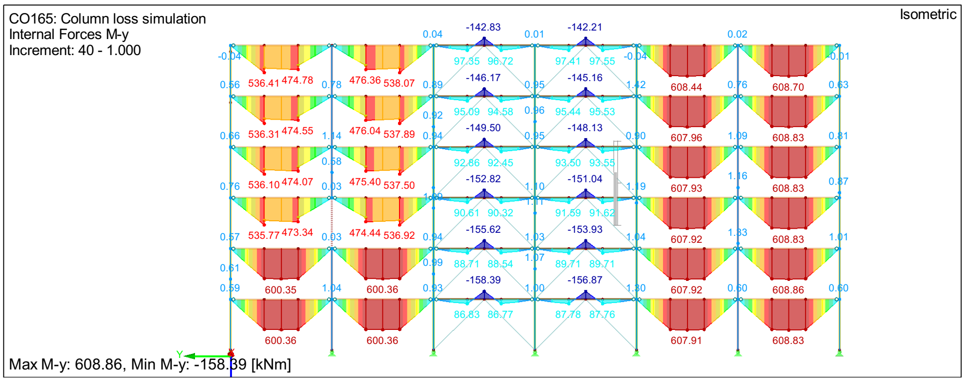


Figure 127. Bending moments in IPE600 frame after column loss (scenario 3)

Remarks

- 2D membrane effects develop for scenarios 1 and 3 (internal column loss) while only 1D membrane effects appears for scenario 2 (facade column loss);
- Note that a corner column loss could not work as no membrane forces (with simple joints at least) could develop. Corner columns should then be designed as key elements.

Results of the column loss scenarios in the directly affected part are summarized in Table 51.

Table 51. Internal forces in members/joints after column loss according to the numerical approach

Scenario	Member	Joint	Tying force (kN)	Moment (kNm)
1	IPE550	B1/B3	1741	274
	IPE600	C2/C3	4565	536
2	IPE500	A1s/A2	1620	195
3	IPE550	B1/B3	1715	275
	IPE600	C2/C3	4493	537

Verification of the structure

Scenario 1: Inner column loss at floor 0

The verification procedure is automatically performed within RSTAB using the STEEL EC3 module. Results from scenario 1 are summarized in Table 52.

Table 52. Member verifications for tying forces according to the numerical approach (scenario 1)

Member	Section	Tying force / compr. force (kN)	Moment (kNm)	UF
Columns Y-facades	HEB 340	-2910	0	0.66
Columns X-facades	HEB 360	-3763	0	0.72
Inner columns	HEM 300	-4887	0	0.60
Inner X-beams	IPE550	1736	274	0.58
Inner Y-beams	IPE600	4562	536	1.15

Remarks

- Due to the missing column, compression forces in neighbour columns are increased. However, in this worked example, these forces stay lower than the design compression forces from ULS, so that no redesign of columns is required.
- The IPE550 members were designed to fulfil the SLS requirements (limitation of the deflection). In this case, the resistance of these members is still sufficient in case of a column loss.
- The IPE600 are not sufficient for the high tensile forces (15% of exceedance). From an engineering point of view, it is expected that, due to the development of plastic hinges, the real tensile force in these profiles should be lower than the value obtained from the second order analysis, so that the IPE600 might be sufficient.

Worked example II.4.5 / SS/NS	Design for unidentified threats using ALPM – full numerical approach – SS/NS	8 of 11 page

On the contrary, the tensile force in the IPE550 would then be larger. In any case, the design was performed elastically and from this point of view, a cross-section change is required. This will lead to a modification of tensile forces in joints, so that joints verification will be performed after the redesign of the structural members. However, it can already be stated that the fin plate connections designed for ULS would not be resistant enough to withstand those high tensile forces.

Scenario 2: Facade column loss at floor 0

For scenario 2, member verifications are summarized in Table 53.

Table 53. Member verifications for tying forces according to the numerical approach (scenario 2)

Member	Section	Tying force / compr. force (kN)	Moment (kNm)	UF
Columns Y-facades	HEB 340	-2473	15	0.58
Columns X-facades	HEB 360	-3521	14	0.77
Inner columns	HEM 300	-5383	3	0.69
Beams X-facades	IPE500	1615	195	0.59

The joints verifications for the tying forces are summarized in the following table.

Table 54. Joints verifications for tying forces according to the numerical approach (scenario 2)

Position s = strong axis w = weak axis	Tying force (kN)	Failure mode	UF
A1s / A2s	1620	Fin plate in bearing	3.71

<u>*Remarks*</u>

- All members verify the requirement;
- Verification not fulfilled for joints A1s/A2s which need to be redesigned.

Scenario 3: Inner column loss above column splice

It appears that, for this structure, the loss of an internal column above a column splice doesn't lead to tying forces in vertical ties, but in tensile forces in horizontal ties. These tensile forces are in the same order of magnitude that in scenario 1 so that scenario 3 won't be investigated further in the following.

<u>**Redesign of the structure**</u>

Scenario 1: Inner column loss at floor 0

Due to the section change of the IPE600, the internal force distribution will be modified. In the following, the column loss scenario 1 was simulated again by replacing all IPE600 members with IPE750x137. This leads to the following modified tensile forces in horizontal ties and compression forces in columns as well as modified utilization factors:

Worked example II.4.5 / SS/NS	Design for unidentified threats using ALPM – full numerical approach – SS/NS	9 of 11 page

Table 55. Redesign members verifications for tying forces according to the numerical approach

Member	Section	Tying force / compr. Force (kN)	Moment (kNm)	UF
Columns Y-facades	HEB 340	-2862	0	0.66
Columns X-facades	HEB 360	-3827	0	0.82
Inner columns	HEM 300	-4941	0	0.61
Inner X-beams	IPE550	1658	276	0.56
Inner Y-beams	IPE750x137	4850	565	1.03

The utilization factor of the IPE750x137 is exceeded by 3%. This exceedance can be considered as acceptable.

Due to the cross-section change, inner Y-beams now have a larger axial stiffness, so that the tensile forces from membrane effects in those members are larger, too. In the same way, the tensile forces in the inner X-beams (IPE550) are now smaller. Alternatively, it has been tried to modify the IPE550 members for IPE600 members, in order to reduce the tensile force in the inner Y-beams. However, the positive effect for inner Y-beams was neglectable, so that changing to IPE750x137 for inner Y-beams with an elastic analysis is the only solution retained here.

Joint verifications with modified tying forces are summarized in Table 56.

Table 56. Joints verifications for tying forces according to the numerical approach (scenario 1)

Position s = strong axis w = weak axis	Tying force (kN)	Failure mode	UF
B1 / B3	1662	Fin plate in bearing	3.80
C2w	4852	Column web in bending	11.20
C3w	4852	Fin plate in tension (net)	6.17

Redesigned joint B1/B3 requires the following: 2 added bolts, M27 instead of M24, additional welded web plate to the beam, modified fin plate geometry and thickness (25 mm) as well as thicker weld for ductility requirements (15 mm).

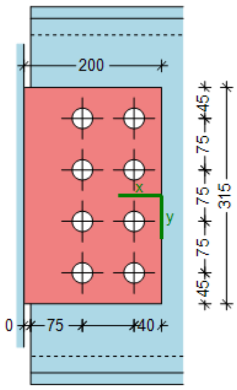


Figure 128. Redesigned joint B1/B3 to fulfil tying forces verifications according to the numerical approach

Worked example II.4.5 / SS/NS	Design for unidentified threats using ALPM – full numerical approach – SS/NS	10 of 11 page

Table 57. Redesigned joints verifications for tying forces according to the numerical approach (scenario 2)

Position s = strong axis w = weak axis	**Tying force (kN)**	**Failure mode**	**UF**
B1 / B3	1662	Bolts in shear	1.00
C2w / C3w	4852	Not feasible	

This leads to a utilization factor of 1.00 with bolts in shear as failure mode. Welded web plates to the beam are preferred to changing the beam cross-section in order to reduce the weight and thus the cost of the structure.

For joints C2w and C3w, no reasonable redesign could be found. For C2w, even a welded 40 mm column web plate would still not be sufficient to sufficiently reinforce the component column web in bending. And for both joints, 14 M36 10.9 bolts would be required to fulfil the verification of bolts in shear, however this would be not feasible geometrically speaking due to the limited beam height and required bolts and pitch distances, together with an impossible verification of the net section of the beam. Changing the beam cross-section would also lead to an unreasonable solution in terms of beam height and overall weight. Even by taking into account the plasticity in the numerical analysis, the tensile force would be of the same order of magnitude.

An alternative could be to use pinned header plate joints. This would solve the problem of lack of beam net section resistance as there won't be holes in the beam web anymore. However, the number of required bolts would still be unreasonable and column flanges should also be greatly reinforced to withstand high bending moments in column flanges.

It appears that pinned joints are not a reasonable choice to ensure sufficient robustness to this structure. Another suitable approach might be to replace pinned joints with semi-rigid joints (partial-strength). This alternative is discussed by applying the analytical method in W.E. II.4.1 / SS/NS.

Scenario 2: Facade column loss at floor 0

In this scenario, no member redesign is needed. However, IPE500 beam-to-column joints (A1s and A2s) have to be redesigned. Joints verifications for tying forces are illustrated in the following.

Worked example II.4.5 / SS/NS	Design for unidentified threats using ALPM – full numerical approach – SS/NS	11 of 11 page

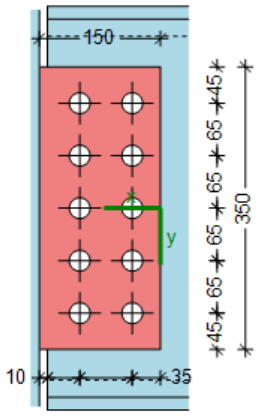


Figure 129. Redesigned joint A1s / A2s to fulfil tying forces verifications according to the numerical approach

Redesigned joint A1s / A2s requires the following: 4 added bolts, M24 instead of M20, additional welded web plate to the beam, modified fin plate geometry and thickness (20 mm) as well as thicker weld for ductility requirements (12 mm).

Table 58. Redesigned joints verifications for tying forces according to the numerical approach (scenario 2)

Position s = strong axis w = weak axis	**Tying force (kN)**	**Failure mode**	**UF**
A1s / A2s	1620	Bolts in shear	1.01

The utilization factor is exceeded by 1%. This exceedance can be considered as acceptable. The redesigned solution could still be considered as feasible.

8.8.4.6 Design for unidentified threats using ALPM - full numerical approach (CS/NS)

FAIL NO MORE Worked example	Title	Design for unidentified threats using ALPM - full numerical approach			1 of 3 pages
	Structure	Composite structure in non-seismic zone	Made by	AM	Date: 06/2021
	Document ref.	II.4.6 / CS/NS			

Example: Design for unidentified threats in a composite structure in non-seismic zone using alternate load path method - full numerical approach

Design manual §5.3.4

This example gives information about the design against unidentified threats using the full numerical approach from ALPM.

Basic data of structure

Design manual § 8.2.

- For geometry, sections, and materials, see Section 8.2.

Actions for Accidental Design Situation

The following actions are considered:

- Permanent loads DL (see Table 11);
- Live loads LL (see Table 11 for CS/NS structure);
- No specific accidental action is taken into account.

Combination of actions for Accidental Design Situation

EN 1990 §6.4.3.3, Eq 6.11b

$$DL + 0.5 \times LL$$

Definition of column removal scenarios

The behaviour of the building is studied for different accidental situations where certain column loss scenarios are considered as presented in Figure 130:

- Corner column (C1) at stories 0, 1, 3 and 5;
- Façade column (C2) at stories 0, 1, 3 and 5;

Braced core columns (C5) at stories 0, 1, 3 and 5.

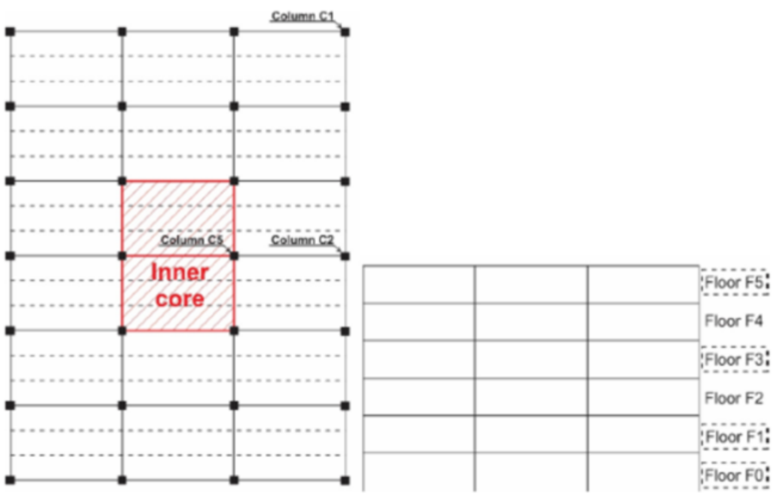


Figure 130. Structure plan and transversal frame view to identify column loss scenarios

Worked exampleII.4.6 / CS/NS	Design for unidentified threats using ALPM – full numerical approach – CS/NS	2 of 3 pages

Structural analysis

The objective of this analysis is to evaluate the behaviour of the building in case of accidental situation (column removal). The calculations are made using the software SAFIR®.

A total of 20 simulations are made and divided into 2 different groups according to the assumed beam-to-column joint configuration met at the extremities of the beams above the removed column:

- 12 simulations with all pinned beam-to-column joints;
- 8 simulations with rigid beam-to-column joints.

In the cases where the column C1 is removed, two different assumptions are defined:

- All beam-to-column joints are pinned (C1 "All pinned joints");
- Rigid beam-to-column joints at the corner where the column is removed (C1 "Rigid joints").

In the cases where the column C2 is removed, two different assumptions are defined:

- All beam-to-column joints are pinned (C2 "All pinned joints");
- Rigid beam-to-column joints where the column is removed (C2 "Rigid joints").

The numerical particularities of pinned and rigid joints are presented in Figure 131.

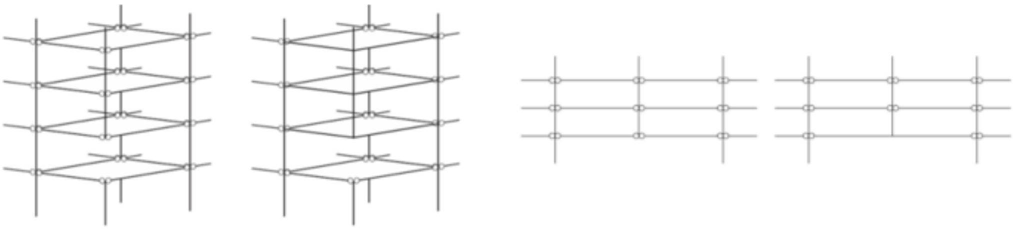


Figure 131. C1 "Pinned" versus "Rigid" joints and C2 "Pinned" versus "Rigid" joints – ALPM – full numerical approach – CS/NS

Results

The output of the SAFIR calculations is summarized in Table 59, according to the maximum vertical displacement at the location of the column loss.

Table 59. Maximum vertical displacement

Max. vertical displacement (m)	Floor	All pinned joints	Rigid joints
C1 Corner column	F0	1.340	0.081
	F1	1.340	0.083
	F3	1.320	0.088
	F5	1.380	0.720
C2 Façade column	F0	0.670	0.610
	F1	0.670	0.600
	F3	0.670	0.550
	F5	0.670	0.250
C5 Center core column	F0	0.016	-
	F1	0.017	
	F3	0.018	
	F5	0.018	

Worked example II.4.6 / CS/NS	Design for unidentified threats using ALPM – full numerical approach – CS/NS	3 of 3 pages

As an example, the obtained forces in the beams of the directly affected part considering the removal of column C2 at ground level F0 are provided in Figure 132.

Maximum beam axial forces (Case 2)		Pinned joints	
		Left beam (kN)	**Right beam (kN)**
Floor 0	Level 1	1381.6	1381.2
	Level 2	1327.6	1326.8
	Level 3	1340.4	1339.5
	Level 4	1338.2	1337.4
	Level 5	1337.6	1336.7
	Level 6	1332.5	1331.7

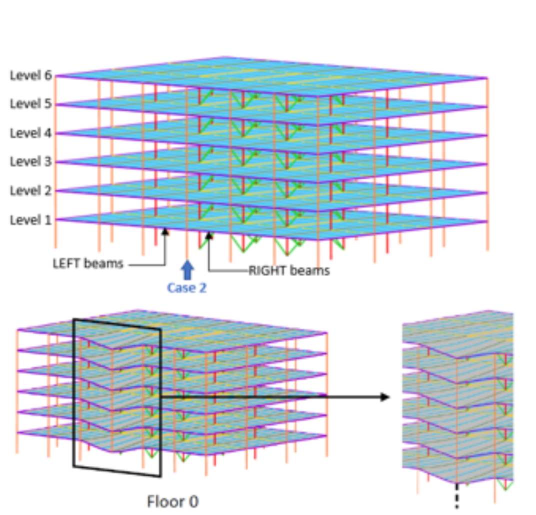


Figure 132. Forces and global displacements resulted from the removal of column C2 at F0.

It is interesting to note that the so-obtained forces are substantially higher than the forces calculated using the tying method (W.E. II.1.4 / CS/NS). The IPE 450 perimeter beams are still able to accommodate these axial loads, but the connections would need to be redesigned.

<u>*Conclusions*</u>

Loss of column C1:

Flowchart Figure 3 – Box C.4 ➔ C.2

- For the loss of the corner column C1, the structure shows very high vertical displacement (approximately 1.35m), as the only contribution in resisting the gravity loads is provided by the cantilevered concrete slab (beams have pinned ends);
- Robustness behaviour can be improved by:
  - Reinforcing the beam-column joints along the vertical alignment of the columns (pinned -> semi-rigid -> rigid). The use of semi-rigid/rigid joints provides additional flexural capacity;

Improving the cantilever capacity of the slab (additional reinforcement at the corners of the building).

Loss of columns C2 and C5:

Flowchart Figure 3 – Box C.4 ➔ End of design

- The displacements are much smaller than for the corner column loss and the load is distributed through the floors (see Figure 132);
- These column loss scenarios do not lead to progressive collapse of the structure, but only to localised damage;

Lateral displacements in columns adjacent to the lost column are small indicating the loads are relatively uniformly redistributed on all floors above the missing column.

8.9 Conclusions for worked examples

The design of steel and composite frame structures to resist the progressive collapse against identified and unidentified exceptional events can follow different approaches.

If the threat is identified, the design can be done using methods with different levels of sophistication; the required level is fixed according to the consequences class of the considered structure. The structures investigated in the worked examples presented herein are all included in the consequences class 2, upper risk group (CC2b), which allows the use of prescriptive methods or of simplified methods of analysis considering static equivalent actions. However, within the present design manual, all the methods, including the sophisticated ones requiring the use of dynamic analyses, have been applied to the worked examples for the sake of completeness.

In case of unidentified events or if the identified events induce too severe damages, the design for robustness involves strategies aiming at limiting the extent of a localized damage. Through the worked examples, the application of the different design approaches proposed within the present manual have been illustrated, going from prescriptive methods to more sophisticated ones employing dedicated finite element software.

For both families of strategies, i.e., design against identified or unidentified threats, the worked examples showed that the adoption of more advanced methods allow for a better and more accurate capture of the actual response of the structure and, in some cases, can limit or even avoid the need for strengthening measures.

The application of the different methods also highlighted that the choices made through the initial design, in particular in terms of constructional details such as the orientation of the columns in the facades (in case of impact or blast) or the selection of the joint configurations, have large impact on the robustness of the structure and, so, on its capacity to resist to progressive collapse. The activation of the composite action between steel beams and concrete slabs provides additional redistribution capacity and considerably reduces the local damages and the risk of progressive collapse.

In particular, for the seismically designed structures, it is observed that the seismic design principles leading to requirements in terms of regularity in plan and elevation, continuity at joints, lateral strength and stiffness, local and global ductility, but also in terms of failure hierarchy for members and joints, provide the steel and composite building structures with appropriate properties in terms of design for robustness. Stronger columns provide better protection against impact and explosion, while minimum flexural requirements and ductility at beam-to-column joints provide higher capacities in case of a column loss scenario.

For what concerns the joints, it has been demonstrated that their behaviour strongly influences the global response of the structure. Accordingly, it is crucial to respect the design recommendations provided in Section 2.2.3 which allow to guarantee a minimum level of ductility or of deformation capacity to the structural joints.

The results also indicated that some loading scenarios can still lead to significant damage and partial progressive collapse, for example in frames equipped with simple joints subjected to a column loss scenario. In such cases, the use of partial-strength beam-to-column joints is seen as a good alternative as it does not prevent the designer to still use simple methods of analysis considering the joints as pinned (if the ductility of the joints is guaranteed through the use of the recommendations of Section 2.2.3) while profiting from the extra resistance provided by the joints in case of exceptional events.

Considering the application of the alternative load path method, it has been clearly highlighted that the level of tensile loads obtained using the prescriptive method as recommended in EN 1991-1-7 are much smaller than the ones obtained through more sophisticated methods that imply explicit column loss simulations. This confirms that the prescriptive method is not aimed at predicting the loads associated to a column loss scenario but at ensuring a minimum level of continuity in the structure.

It also means that the use of the prescriptive method is not sufficient to guarantee that the structure will survive to a column loss scenario. To achieve this objective, the analytical or numerical methods proposed within the present design manual have to be employed in the design process.

For practitioners, the analytical approach is seen as a good alternative to the full numerical approach which requires the use of finite element models and a good knowledge on the use of such finite element tools.

Part 3 – Annexes

A.1 Design resistance of joints under combined bending moments and axial forces

Based on the static theorem, it is possible to predict the resistance of a connection at failure by expressing the equilibrium between the external applied forces and the internal forces. When a connection is subjected to *M* and *N*, the equations of equilibrium write:

$$M = \sum_{i=1}^{n} h_i . F_i$$
$$N = \sum_{i=1}^{n} F_i \qquad (48)$$

where F_i designates the force in row *i* and h_i the associated lever arm; this one is obtained computing the vertical distance between the considered row and the reference axis of the beam, i.e., the axis where *M* and *N* are considered to be applied (h_i is positive for the rows located above the reference axis).

The applied axial force and bending moment are linked through the concept of load eccentricity e as follows (*N* is positive when tension is applied and the positive value of *M* is defined in Figure 135):

$$M = e.N \qquad (49)$$

A.1.1 Resistance criteria with due account of group effects

The resistance of a row is taken as equal to the resistance of the weakest component active in the considered row. To respect the static theorem, this resistance should never be exceeded. This looks easy when looking to the individual resistance of rows but is more difficult when group effects develop in the connections (see Section 2).

In the model, any group of rows [*m,p*] for which group effects can develop is studied as an equivalent fictive row with an equivalent lever arm and a group resistance equal to that of the weakest component. Accordingly, the resistance criterion for each of the rows as part of the [*m,p*] group, for any component α, can be written as follows:

$$\sum_{i=m}^{p} F_i \le F_{mp}^{Rd\ \alpha} \qquad \text{m = 1,...n and for each value of m, p varies from m to n} \qquad (50)$$

where $F_{mp}^{Rd\ \alpha}$ is the resistance of the [*m,p*] group for component α computed according to Eurocode 3 Part 1-8. If *m* equals *p*, $F_{mp}^{Rd\ \alpha}$ becomes the individual resistance of the component α included in row *m*. Such a resistance criterion may be derived for each of the constitutive row components and the final resistance of the group of rows [*m,p*], called F_{mp}^{Rd}, may be defined as the smallest of the $F_{mp}^{Rd\alpha}$ values.

This criterion is illustrated in Figure 133 representing the application of this criterion for a connection with two bolt rows; the application of the criterion for a connection with three bolt rows, noted 1, 2 and 3, is presented in Figure 134. More globally, these figures cover cases met in any connections with n rows for which group effects can develop in two or three successive rows.

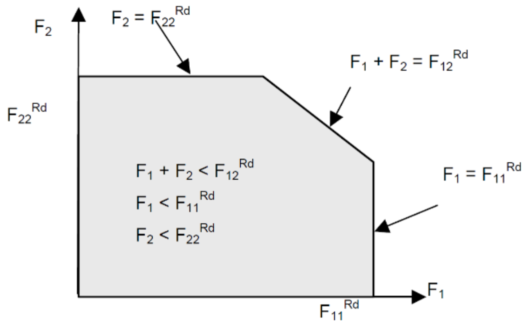


Figure 133. Interaction between two bolt rows

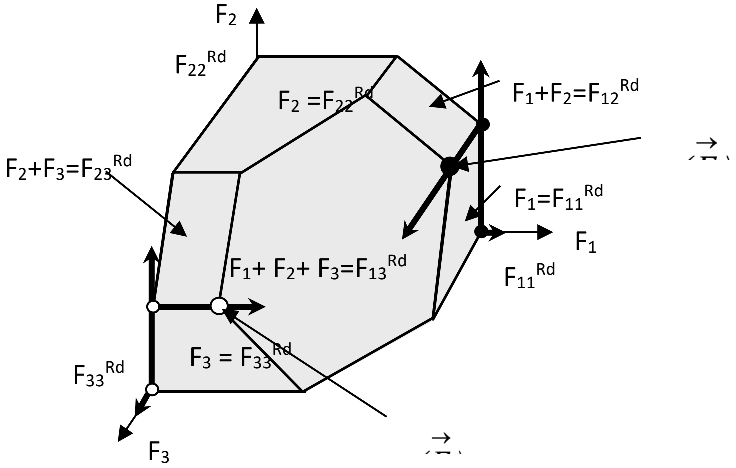


Figure 134. Interaction between three bolt rows and definition of F_{jRd} - Possible group effects between three bolt rows and successive steps for the evaluation of a connection resistance (black and white dots respectively).

A.1.2 Definition of the failure criterion for the whole connection

The resistance M-N interaction curve is obtained using the failure criterion provided by the following equation:

$$M = h_k.N + \sum_{i=1}^{n} (h_i - h_k).F_i^c \tag{51}$$

In this equation, the value of *k* varies from 1 to *n* where *n* represents the total number of rows. *k* designates the number of the particular row where the plastic neutral axis is assumed to be located for the computation of different points of the M-N resistant curve (between these points of the M-N resistant curve, the plastic neutral axis is passing from one row to the following one); accordingly, by varying the value of *k*, different distributions of resistant forces amongst the rows are obtained (respecting the static theorem) and so, different M-N couples are obtained. Knowing that for each position of plastic neutral axis, two distributions of resistant forces can be obtained (one assuming that the part above the plastic neutral axis is under compression and the part below is under tension and one assuming the reverse situation), "*2k*" M-N couples are obtained using Equation (51). This equation is defined so as to obtain the maximum resistance in bending by adopting an optimised distribution of the internal loads amongst the activated rows, taking into account the possible group effects as explained here below.

In this expression, two different resistances F_i^c can be attributed to row i (F_i^{Rd+} and F_i^{Rd-}) with the objective to maximize the absolute value of the bending resistance by maximising the loads in the rows which are the most distant from the row "*k*". This is illustrated for a connection with two bolts rows in

Figure 135. Let assume that, in this connection, the resistance of the two bolt rows in tension is governed by the component "End-plate in bending" and that the corresponding group resistance is equal to 100 kN and is smaller than the sum of the individual resistances of the two bolt rows (2 * 60 kN = 120 kN). In Figure 135, two situations are considered in which the number of the row *k* is respectively considered as equal to 1 and 4. The distribution of the tensile loads in the two bolt rows, for *k* = 1 and for *k* = 4, is illustrated in Figure 135. If k is equal to 4 and a positive moment is applied to the connection, it means that the resistance of the upper bolt row *F2Rd+* is equal to 60 kN and the one of the lower bolt row *F3Rd+* is equal to 40 kN (= 100 kN – 60 kN) while, if *k* = 1 and a negative moment is applied to the connection, the resistance of the upper bolt row *F2Rd-* is equal to 40 kN and the one of the lower bolt row is equal to 60 kN. Such a procedure is illustrated in Figure 134 for a joint in which three bolt rows are possibly concerned by group effects. The black dots show the successive steps to estimate F_i^{Rd+} respecting the group resistances while the white ones show the steps to estimate F_i^{Rd-}. Accordingly, F_i^{Rd+} and F_i^{Rd-}. can be defined as the maximum (or minimum in case of negative values) resistance of row *i* under positive and negative moments respectively considering the group effects and maximising the resistant moment.

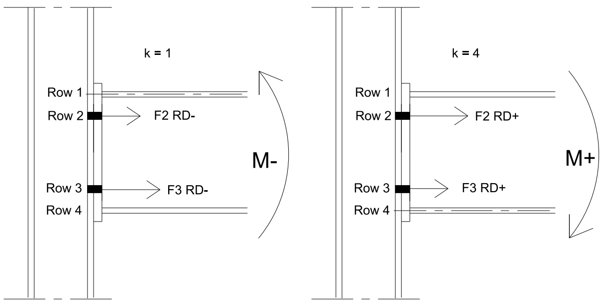


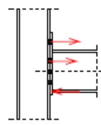
Figure 135. Examples of distribution of resistant forces amongst the rows in tension on a connection with two bolt rows

The interaction resistance criterion between the bending moment (*M*) and the axial force (*N*) at failure is provided by a set of 2 n parallel straight line segments; the slope of each of the 2 n parallel segments is equal to the value of the lever arm (h_k) and along these segments, the force (F_k) varies between 0 at one end and the maximum resistance row resistance at the other end.

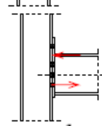
The application of Equation (51) may be written with more details as follows:

$$M = h_k . N + \sum_{i=1}^{n} (h_i - h_k) . F_i^c$$

$$\text{either} \quad \left. \begin{aligned} F_i^c &= \max(F_i^{Rd+}; 0) \text{ if } i < k \\ F_i^c &= \min(F_i^{Rd+}; 0) \text{ if } i > k \end{aligned} \right\} \text{ tension in the rows at the top } (\mathrm{M}^+)$$



$$\text{or} \quad \left. \begin{aligned} F_i^c &= \min(F_i^{Rd-}; 0) \text{ if } i < k \\ F_i^c &= \max(F_i^{Rd-}; 0) \text{ if } i > k \end{aligned} \right\} \text{ tension in the rows at the bottom } (\mathrm{M}^-)$$



$$\text{with} \quad F_i^{Rd+} = \min(F_{mi}^{Rd} - \sum_{\substack{j=m \\ i \neq up,lo}}^{i-1} F_j^{Rd+}, m = 1, \ldots, i) \text{ for } i < k \text{ and } F_i^{Rd+} = F_i^{Rd} \text{ for } i = up, lo > k$$

$$F_i^{Rd-} = \min(F_{im}^{Rd} - \sum_{\substack{j=i+1 \\ i \neq up,lo}}^{m} F_j^{Rd-}, m = i, \ldots, n) \text{ for } i > k \text{ and } F_i^{Rd-} = F_i^{Rd} \text{ for } i = up, lo < k$$

The non-ductile behaviour of some components may lead to a reduction of the connection resistance capacity. The way on how to adapt the M-N interaction curve accordingly is explained in (Demonceau et al., 2019), as well as the way to evaluate the elastic stiffness of the joint under M and N.

Finally, the response of a joint subjected to axial forces only appears as a specific loading case for which the here-above calculation procedure may be also applied.

A.2 Detailing requirements to allow sufficient rotation capacity of simple joints

A.2.1 Joints with a header plate

With the aim to enable a rotation without increasing too much the bending moment which develops into the joint, the contact between the lower beam flange and the supporting member has to be strictly avoided. So, it is imperative that the height h_p of the plate is lower than that of the supported beam web (Figure 136):

$$h_p \leq d_b \tag{52}$$

where d_b is the clear depth of the supported beam web.

If such a contact takes place, a compression force develops at the contact place; it is equilibrated by tension forces in the bolts and a significant bending moment develops (Figure 136).

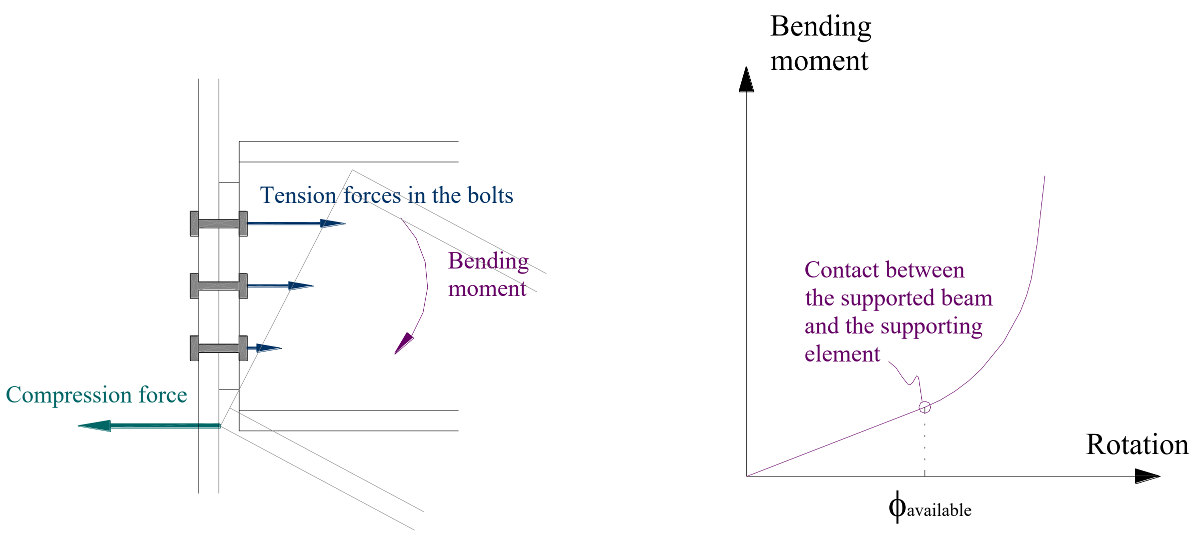


Figure 136. Contact and evolution of the bending moment

The level of rotation at which the contact occurs is obviously dependent on the geometrical characteristics of the beam and of the header plate, but also on the actual deformations of the joint components.

In order to derive a simple criterion that the user could apply, before any calculation, to check whether the risk of contact may be disregarded, the following rough assumptions are made (see Figure 137):

- the supporting element remains undeformed;
- the centre of rotation of the beam is located at the lower extremity of the header plate.

On the basis of such assumptions, a safe estimation (i.e., a lower bound) of the so-called "available rotation of the joint" $\phi_{available}$ may be easily derived:

$$\phi_{available} = \frac{t_p}{h_e} \tag{53}$$

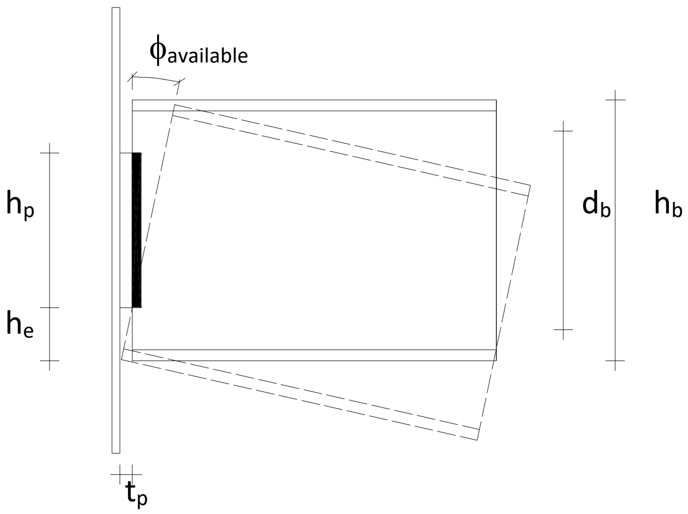


Figure 137. Geometrical characteristics of the joint and illustration of the contact between the beam and the supporting element

This available rotation has to be greater than the "required rotation capacity" which varies according to the structural system and loading. A simple criterion ensuring the sufficient joint rotation capacity may be written as:

$$\phi_{available} > \phi_{required} \tag{54}$$

For instance, the required rotation capacity, for a beam (length L and inertia I) simply supported at its extremities and subjected to a uniformly distributed load (factored load γp at ULS), writes:

$$\phi_{required} = \frac{\gamma\, p\, L^3}{24\ EI} \tag{55}$$

By expressing that $\phi_{available} > \phi_{required}$, a simple criterion ensuring a sufficient joint rotation capacity may be derived. It writes:

$$\frac{t}{h_e} > \frac{\gamma\, p\, L^3}{24\ EI} \tag{56}$$

Similar criteria may be derived for other load cases.

A.2.2 Joints with a fin plate

To permit a rotation without increasing too much the bending moment which develops into the joint, the contact between the lower beam flange and the supporting member has to be strictly avoided. To achieve it, the height h_p of the fin plate should be lower than that of the supported beam web (Figure 138):

$$h_p \leq d_b \tag{57}$$

where d_b is the clear depth of the supported beam web.

If such a contact takes place, a compression force develops at the contact place which is equilibrated by tension forces in the welds and in the plate, and additional shear forces in the bolts.

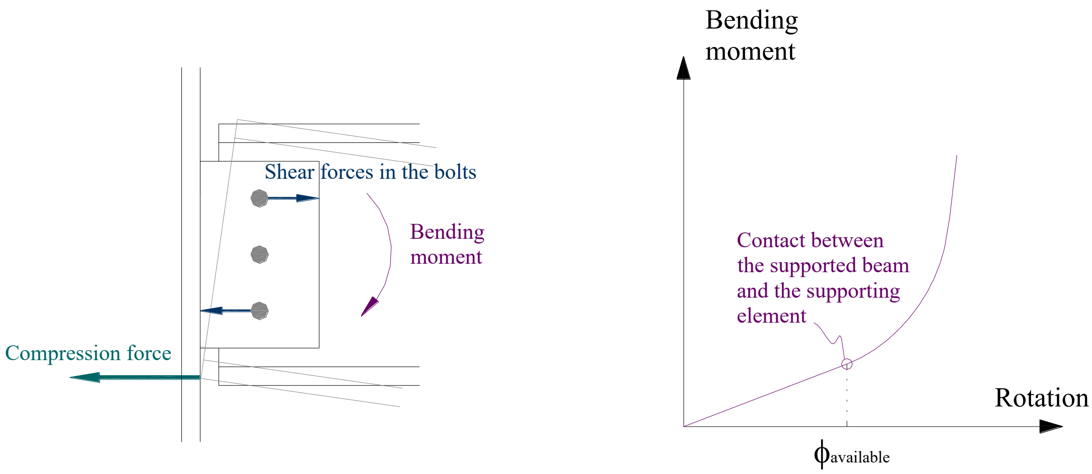


Figure 138. Contact and evolution of the bending moment

The level of rotation at which the contact occurs is obviously dependent on the geometrical characteristics of the beam and of the fin plate, but also on the actual deformations of the joint components.

In order to derive a simple criterion that the user could apply, before any calculation, to check whether the risk of contact may be disregarded, the following rough assumptions are made (see Figure 139):

- the supporting element and the fin plate remain undeformed;
- the centre of rotation of the beam is located at the gravity centre of the bolt group.

On the basis of such assumptions, a safe estimation (i.e., a lower bound) of the so-called "available rotation of the joint" $\phi_{available}$ may be easily derived:

if $\quad z > \sqrt{(z - g_h)^2 + \left(\frac{h_p}{2} + h_e\right)^2}$:

$$\phi_{available} = \text{"}\infty\text{"}$$

else:

$$\phi_{available} = \arcsin\left(\frac{z}{\sqrt{(z-g_h)^2 + \left(\frac{h_p}{2} + h_e\right)^2}}\right) - \text{arctg}\left(\frac{z - g_h}{\frac{h_p}{2} + h_e}\right)$$

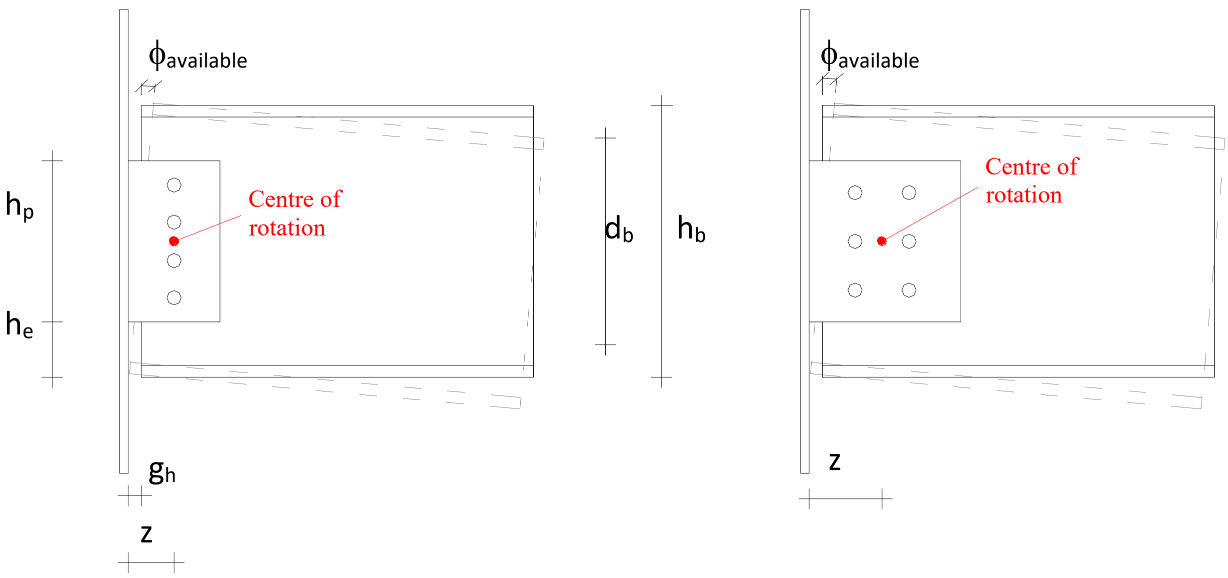


Figure 139. Geometrical characteristics of the joint and illustration of the contact between the beam and the supporting element

This available rotation has to be greater than the "required rotation capacity" which varies according to the structural system and loading. A simple criterion ensuring the sufficient joint rotation capacity may be written as:

$$\phi_{available} > \phi_{required} \tag{58}$$

A.3 Specific ductility recommendations for partial-strength steel and composite bolted joints with endplates

As introduced in Section 2.2.3.2, (Rölle, 2013) provides a simplified method for the derivation of the moment resistance of all-steel and composite end-plate joints as well as constructive criteria for the design of highly ductile joints, see also Table 3. Component failures other than tension failure are excluded by defined criteria, see Step 1 below in the section dedicated to the application of the method. Also, ductility criteria for the T-stub are given, see Step 2 below.

The method assumes the product of bolts' tensile strength and lever arm of force couple to be the factor that predominantly defines the moment capacity of the joints. Other parameters that have an influence on the joints' moment capacity are considered indirectly through the application of a correction factor. The formula for the calculation of the plastic moment capacity is given in Equation (59). For the lever arm, the assumption is that the sum of the tension axial forces acts in the centre of gravity of the bolts in tension and the sum of the compression forces acts in the centre of gravity of the beam´s flange in compression. The method has been developed using the experimental results of (Kuhlmann et al., 2008) and the numerical analyses performed in (Rölle, 2013). It has also been validated with the help of the analytical equations of the "real" component model and yields values on the safe side comparing to those of the component model.

The all-steel flush endplate joint is the standard joint configuration for which the method has been developed. It can additionally be applied for all-steel extended endplate and steel-concrete composite joints with 3 bolt-rows. For the all-steel extended endplate joints, the model considers exclusively the case that the two upper bolt rows are symmetrically placed around the beam flange. For the application of the model certain strength, geometrical, and configuration criteria should be satisfied and the method is valid under the following conditions:

- The bolt load bearing capacity should be the weakest component for the joint failure;
- $M_{j,pl,Rd} < 0{,}7\ M_{b,pl,Rd}$;
- Only one bolt-row per beam flange;
- For extended end-plates only one bolt-row above the beam flange;
- Only two bolts per bolt-row;
- The thickness of the endplate should not exceed 90% of column flange´s thickness: $t_{EP} \leq 0{,}9\ t_{fc}$.

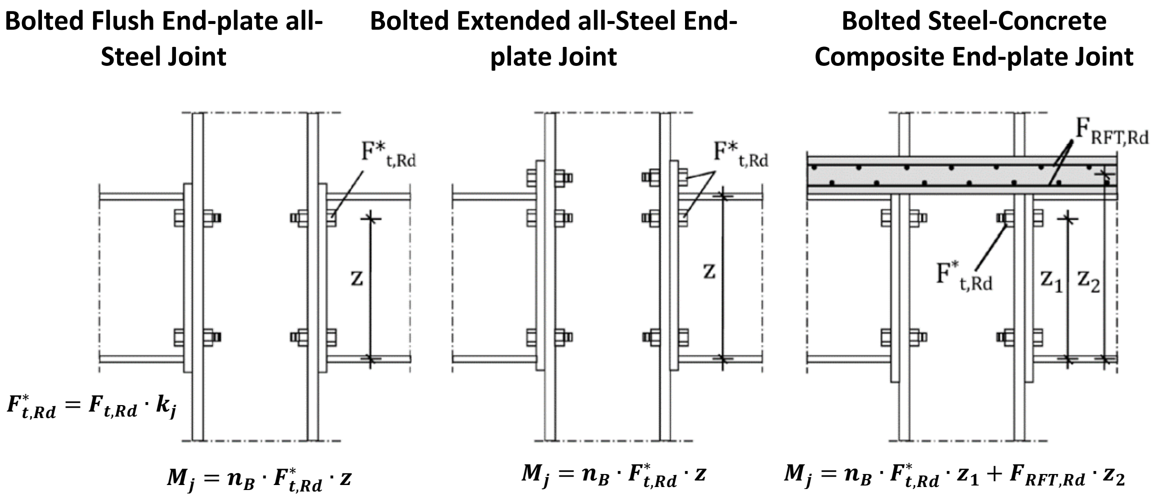


Figure 140. Typical joint configurations for the application of the simplified Rölle method (Rölle, 2013)

$$M_{j,pl,Rd} = n_B \cdot F_{t,Rd} \cdot k_j \cdot \alpha \cdot \mathrm{z} \tag{59}$$

- $F_{t,Rd}$→ *axial load capacity of bolts (design value)*
- k_j → *correction factor considering the influence of different parameters on the joint´s moment capacity*
- α→ *adjustment factor*
- z → *lever arm*
- n_B → *number of bolts in tension*

When referring to composite joints, the formula for the calculation of the joints' plastic moment capacity differentiates from Equation (59) through the addition of the term that considers the force of the steel reinforcement and is formed as follows:

$$M_{j,pl,Rd} = n_B \cdot F_{t,Rd} \cdot k_j \cdot \alpha \cdot \mathrm{z} + F_{T,RFT,Rd} \cdot z_2 \tag{60}$$

- $F_{T,RFT,Rd}$→ *tension strength of steel reinforcement (design value)*
- z_2 → *lever arm, see Figure 140*

This method ensures that the decisive component is the T-stub and that by the enhanced ductility criteria, a mode 2 failure occurs. Within the thesis (Rölle, 2013), it has been proven that assuming a braced frame structure, the rotation requirement $\phi_{available}$ / $\phi_{required}$ of at least 2 is fulfilled. This is also shown in (Keller et al., 2021).

A.3.1 Application of the simplified method (Rölle, 2013)

The step-by-step process for the application of the simplified Rölle method for the 3 joint configurations **(i)** all-steel flush end-plate joints, **(ii)** all-steel extended end-plate joints and **(iii)** steel-concrete composite joints can be described in 3 steps.

1. Check of the range validity for the column profile
2. Definition of the T-stub admissible thickness range – T-stub ductility criteria
3. Derivation of the moment resistance

These 3 steps are presented for each of configurations below:

i. Flush end-plate steel joints

Step 1

Table 60. Range of validity - column profile

Column Web in Compression	$\frac{\sqrt{h_c \cdot d_B}}{t_{wc}^2} \cdot \sqrt[3]{\frac{355}{f_{y,c}}} \cdot \sqrt{\frac{f_{uB}}{1.000}} < 7{,}0$
Column Web in Tension	$t_{wc} > 0{,}092 \cdot d_B \cdot \frac{f_{uB}}{f_{y,c}}$
Column Web in Shear	$t_{wc} > 1{,}12 \cdot \frac{d_B^2 \cdot f_{uB}}{h_c \cdot f_{y,c}}$

Step 2

Table 61. Ductility criteria for the T-stub

Lower limit (punching shear)	$t_{EP} \geq 0{,}186 \cdot d_B \cdot \frac{f_{uB}}{f_{u,EP}}$
Upper limit (ductility) - Stiffened T-stub (case of end-plate)	$t_{EP} \leq 0{,}33 \cdot d_B \cdot \sqrt{\frac{f_{uB}}{f_y}} \cdot \sqrt{\left(\frac{m}{2{,}5d_B}\right) \cdot \sqrt{\frac{m_2}{2{,}0d_B}}}$
For $0{,}9 \cdot t_{EP} \leq t_{fc} \leq t_{EP}$ – Unstiffened T-stub (case of column flange)	$t_{fc} \leq 0{,}4 \cdot d_B \cdot \sqrt{\frac{f_{uB} \cdot m}{f_y \cdot 2{,}5d_B}}$

Step 3

Table 62. Resistance model for the moment capacity of flush end-plate all-steel joints

Plastic moment capacity	$M_{j,pl,Rd} = 0{,}9 \cdot n_B \cdot F_{t,Rd} \cdot k_j \cdot z$
Joint´s correction factor	$k_{j(FEP)} = 1{,}95 \cdot \left(\frac{t_{EP} \cdot t_{cf} \cdot f_y}{m \cdot m_2 \cdot f_{uB}}\right)^{0{,}25} \leq 1{,}0$
Axial load capacity of the bolt (tension)	$F_{t,Rd} = \frac{0{,}9 \cdot f_{uB} \cdot A_s}{\gamma_{M2}}$

ii. Extended end-plate all-steel joints

For the case of extended end-plate steel joints, the definition of the end-plate´s thickness value range is performed as for the flush end-plate joints, while the check of the column´s profile value range and calculation of the joint´s plastic-moment capacity should be done as follows:

Step 1

Table 63. Range of validity for column profile	
Column Web in Compression	$\frac{\sqrt{h_c \cdot 2 \cdot d_B}}{t_{wc}^2} \cdot \sqrt[3]{\frac{355}{f_{y,c}}} \cdot \sqrt{\frac{f_{uB}}{1.000}} < 10{,}0$
Column Web in Tension	$t_{wc} > 0{,}092 \cdot d_B \cdot \frac{f_{uB}}{f_{y,c}}$
Column Web in Shear	$t_{wc} > 1{,}67 \cdot \frac{d_B^2 \cdot f_{uB}}{h_c \cdot f_{y,c}}$

Step 3

Table 64. Resistance model for the moment capacity of extended end-plate all-steel joints	
Plastic moment capacity	$M_{j,pl,Rd} = 0{,}9 \cdot n_B \cdot F_{t,Rd} \cdot k_j^* \cdot z$
Joint´s correction factor	$k_{j(EEP)}^* = 0{,}75 \cdot 1{,}95 \cdot \left(\frac{t_{EP} \cdot t_{cf} \cdot f_y}{m \cdot m_x \cdot f_{uB}}\right)^{0{,}25} \leq 1{,}0$
Axial load capacity of the bolt (tension)	$F_{t,Rd} = \frac{0{,}9 \cdot f_{uB} \cdot A_s}{\gamma_{M2}}$

iii. Steel-concrete composite joints

If the tension resistance of the reinforcement laying in the slab´s effective width is larger than the theoretical load carrying capacity of the upper bolt-row of a hypothetical extended part of the end-plate, then the component column web in compression should be checked separately.

The definition of the value range for the endplate´s thickness is performed according to the relative one for flush end-plate joints. In such a way, only the 3rd step for the application of the Rölle method differentiates as follows in Table 65 for composite joints compared to the other 2 type of joints:

Step 3

Table 65. Resistance model for the moment capacity of composite steel-concrete end-plate joints	
Plastic moment capacity	$M_{j,pl,Rd} = 0{,}9 \cdot n_B \cdot F_{t,Rd} \cdot k_j \cdot z_1 + F_{T,RFT,Rd} \cdot z_2$
Joint´s correction factor	$k_{j(EEP)}^* = 0{,}75 \cdot 1{,}95 \cdot \left(\frac{t_{EP} \cdot t_{cf} \cdot f_y}{m \cdot m_x \cdot f_{uB}}\right)^{0{,}25} \leq 1{,}0$
Axial load capacity of the bolt (tension)	$F_{t,Rd} = \frac{0{,}9 \cdot f_{uB} \cdot A_s}{\gamma_{M2}}$
Axial load capacity of steel reinforcement	$F_{T,RFT,Rd} = \frac{f_{sk} \cdot A_s}{\gamma_s}$

The formulae given above refer to the plastic bending moment. They may be transferred to the level of ultimate resistance as used under accidental load situation by transferring the bolt´s tension resistance from $F_{t,Rd}$ to $F_{t,u} = A_s f_{ub}$.

A.4 Evaluation of the plastic rotational capacity of joints at ULS

A.4.1 General principles and method

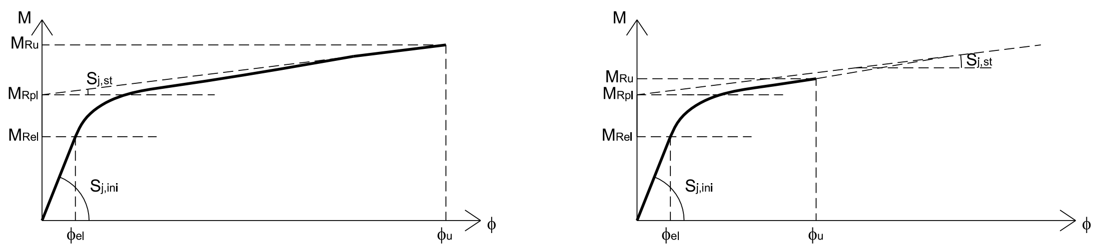
The rotational response of a joint is presented in the form of a *M-φ* moment-rotation curve where *M* and φ represent respectively the bending moment to which the joint is subjected and the resultant relative rotation between the connected members. This curve may be drawn as well for joints in bending and for joints subjected to more complex loading, including additional axial forces. But for sure, in the case of joints subjected to tension only, the rotational response is meaningless and an extensional behavioural curve *N-Δ* has to be considered.

In the robustness context, the evaluation of the maximum deformation capacity (extensional, rotational or both) is of key importance and a general procedure for its determination is here introduced. For the sake of clarity, it is presented in the case of a joint in bending, but it may be directly applied to any other joint loading situation.

For classical steel or composite joints made of welded and bolted connections, the shape of the *M-φ* curve is approximately bi-linear and may therefore be characterized by four key parameters:

- an initial stiffness $S_{j,ini}$;
- a plastic bending resistance M_{Rpl};
- a strain hardening (more generally post-plastic) stiffness $S_{j,st}$;
- an ultimate bending resistance M_{Ru}.

When no instability or early brittle failure occurs in the joint at ultimate state, M_{Ru} differs significantly from M_{Rpl}, and the bi-linear shape of the *M-φ* curve is well marked (Figure 141a); when instability or early brittle failure occurs - for instance in the column web in compression or in bolts in tension – M_{Ru} comes closer to M_{Rpl}, what tends to give a more or less round final shape to the *M-φ* curve (Figure 141b). Whatever the case, the ultimate rotation capacity ϕ_u may be derived at the intersection of the *M-φ* curve with the M_{Ru} horizontal line.



a – Well marked bi-linear response

b – Less marked bi-linear response

Figure 141. Main joint properties characterising actual M-φ curves

So, the ultimate plastic rotation capacity of the joints may be evaluated as equal to (Jaspart et al., 2019):

$$\varphi_u = (M_{Ru} - M_{Rpl})/S_{j,st} \tag{61}$$

The values of M_{Rpl} and $S_{j,ini}$ may be derived according to Eurocode 3 Part 1-8. The strain-hardening stiffness of the joint $S_{j,st}$ may therefore be evaluated as follows:

$$S_{j,st} = \frac{Ez^2}{\sum \frac{1}{k^*}} \tag{62}$$

where:

$$\sum \frac{1}{k^*} = \sum_m \left(\frac{1}{k_{i,m}}\right)_{M_{Rpl,comp,m} > M_{Rpl,limit}} + \sum_k \left(\frac{1}{k_{st,k}}\right)_{M_{Rpl,comp,k} > M_{Rpl,limit}} \tag{63}$$

k and *m* are component indices and $M_{Rpl,limit} = 1{,}65 M_{Rpl}$

A good estimation of the ultimate moment resistance M_{Ru} of the joint may simply be obtained by substituting:

- the yield stress of the steel material f_y by the ultimate stress f_u;
- the design resistance of the bolt in tension by the ultimate resistance of the bolt in tension (stress area times ultimate yield strength);

in the formulae proposed in Eurocode 3 for the evaluation of the joint design moment resistance M_{Rpl}. The risks of instability of the column web in compression and of the beam flange in compression have however not to be forgotten. As for M_{Rpl}, the ultimate moment resistance M_{Ru} is associated to the ultimate resistance of the weakest component.

A.4.2 Simplified method of Keller for the deformation capacity of composite joints

In (Keller, 2019) simplified equations for the prediction of the rotation capacity of composite joints are provided, both under positive and negative bending moment. These are based on the experiments conducted in (Kuhlmann et al., 2017), on the FE analyses conducted in (Rölle, 2013) and (Keller, 2019) and on the concept of the so-called component model. For the derivation of these equations, only the components which have a significant influence on the deformation capacity of composite joints have been considered.

Deformation capacity of joints under negative bending moment

For composite joints under negative bending, the proposal from (Keller, 2019) is reflected in Equation (64), whereas the considered components together with the relevant lever arm are presented in Figure 142. The deformation of the parts in tension and those in compression are given by Equations (65) and (66).

The given values correspond to the mean level of test results. Partial safety factors and correction factors for the design level are derived in (Keller, 2019).

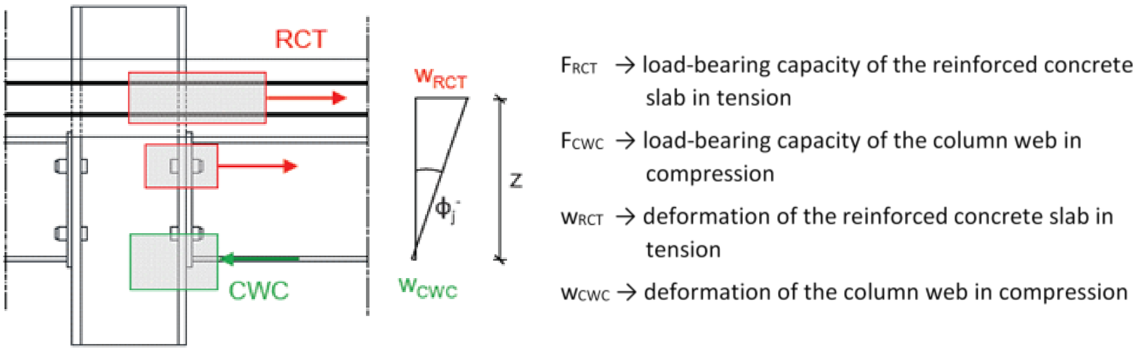


Figure 142. Total rotation φ_j^- of the composite joint under negative bending moment (Keller, 2019)

$$\varphi_{j,available}^{-} = \frac{w_{RCT} + w_{cwc}}{z} \tag{64}$$

where:

w_{RCT}	[mm]	Deformation of the reinforced concrete slab (see Equation (65))
w_{CWC}	[mm]	Deformation of the column web in compression (see Equation (66))
z	[mm]	Distance between the center of compression and the center of gravity of the reinforcement bars, see Figure 142

- $$w_{RCT} = \varepsilon_{su} \cdot \kappa \cdot l_z \cdot k_M \tag{65}$$

  where:

ε_{su}	[%]	Ultimate strain of the steel reinforcement bar
$\kappa = 0{,}49 \cdot \dfrac{\rho^{0{,}51}}{f_{ctm}{}^{0{,}34} \cdot d_r{}^{0{,}68}}$	[-]	Factor for the consideration of the different influencing factors
ρ	[%]	Reinforcement ratio
$l_z = 0{,}9 \cdot \left(\dfrac{h_{col}}{2} + \mathrm{a}\right)$	[mm]	Length of elongation of the steel reinforcement (on a single side of the joint)
h_{col}	[mm]	Height of the column profile
a	[mm]	Distance of the first shear stud to the column flange
$k_M = \begin{cases} 1{,}00 & \rightarrow for\ pure\ axial\ loading \\ 0{,}61 & \rightarrow for\ negative\ bending\ moment \end{cases}$	[-]	Factor considering the loading conditions

- $$w_{CWC} = \frac{M_{j,u}^{-}}{z \cdot k_{CWC} \cdot E} \cdot \delta_{smu} \tag{66}$$

  where:

$M_{j,u}^{-}$	[kNm]	Negative ultimate moment capacity of the joint
z	[mm]	Lever arm
$k_{CWC} = 0{,}7 \cdot b_{eff,c,wc} \cdot \dfrac{t_{wc}}{d_c}$	[mm]	Stiffness coefficient for CWC according to EN 1993-1-8
$\delta_{smu} = 1{,}12 \cdot 10^4 \cdot \varepsilon_{smu}{}^{2{,}35}$	[-]	Factor for the consideration of the strain of the reinforced concrete slab in tension
$\varepsilon_{smu} = \left(\dfrac{w_{RCT}}{l_Z}\right)$	[-]	Strain of the reinforced concrete slab in tension

Deformation capacity of joints under positive bending moment

The deformation capacity of joints under positive bending is given by Equation (67) and the considered components together with the relevant lever arm are illustrated in Figure 143.

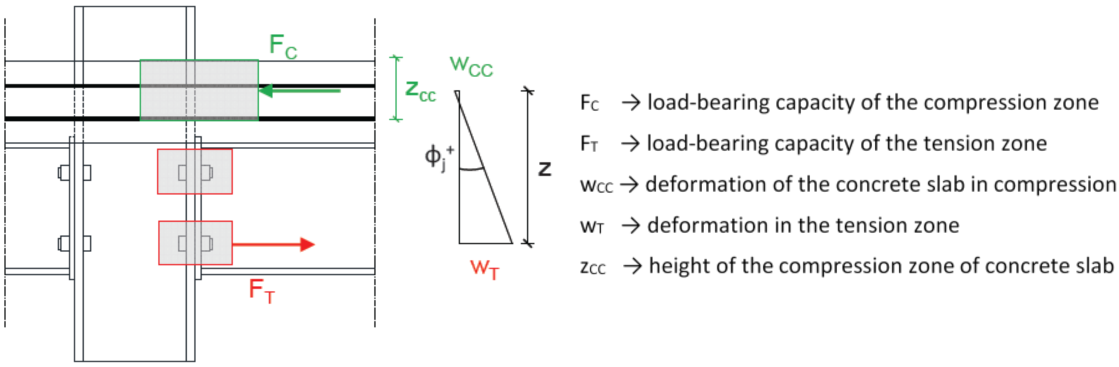


Figure 143. Total rotation φ_j^+ of the composite joint under positive bending moment (Keller, 2019)

$$\varphi^+_{j,avail.} = \frac{w_T}{z} \tag{67}$$

where:

$w_T = \frac{(t_{ep} + t_{cf}) \cdot f_y}{E} \cdot \delta_{pl,T}$	[mm]	Deformation of the joint´s steel part in tension
$\delta_{pl,T} = a \cdot 1{,}07 \cdot 10^{-4} \cdot h_b \cdot \sqrt{m_x} \cdot \left(\frac{d_b}{t_{ep}}\right)^3 \cdot \left(\frac{m}{t_{cf}}\right)^{1,8} \cdot \left(\frac{f_{ub}}{f_y}\right)^{2,8}$	[-]	Factor considering the different influencing parameters
$a = \begin{cases} 1{,}00 & for\ all\text{-}steel\ joints \\ 0{,}65 & for\ composite\ joints \end{cases}$	[-]	Factor for all-steel or composite joints
z	[mm]	Lever arm – distance between centre of compression and bolt row in tension
m_x	[mm]	Vertical distance between bolt and beam flange
m	[mm]	Horizontal distance between bolt and beam web

As an alternative, (Duarte da Costa, 2018) proposes an analytical procedure for the prediction of the ultimate rotation capacity of composite joints subjected to hogging moments with the activation of S500B rebars.

The first step consists in the determination of the ultimate ε_{smu} and yield strain limit ε_{smy} of the reinforced concrete component as the tension stiffening effect plays a non-negligible role in the elongation capacity of the component "slab rebars in tension". Hence, design charts to easily determine ε_{smu} and ε_{smy} are presented in Figure 144. These charts are given in function of concrete class and effective reinforcement ratio ρ_{eff}. In order to determine ε_{smu} and ε_{smy} with the help of these charts, the following steps must be followed:

1. Calculate the effective reinforcement ratio ρ_{eff} considering the effective area $A_{c,eff}$ of concrete around the longitudinal reinforcement according to (EN 1992-1-1, 2005), Figure 7.1 and derive the first crack stress σ_{sr1} with the upper chart in Figure 144.

2. Multiply the first crack stress σ_{sr1} obtained in the previous step by the factor k_b which is equal to k_c as defined in Section 7.4.2(1) of (EN 1994-1-1 2004). This factor considers the linear stress distribution

in the section prior to cracking. On this basis, determine the ultimate strain limit σ_{smu} of the reinforced concrete member with the inferior chart of Figure 144. On the same chart, determine the yield strain limit σ_{smy} of the reinforced concrete member.

In the second step, the effective joint length L_j is calculated: $L_j = \frac{h_c}{2} + n \cdot 2 \cdot \frac{\emptyset}{6.4 \cdot \rho_{eff}}$

with:

- h_c is the column depth;
- n = 1.5 for 1.0% ≤ ρ_{eff} ≤ 1.6%; 2.5 for 1.6% < ρ_{eff} ≤ 1.9%; 3.5 for 1.9% < ρ_{eff} ≤ 2.2%; 4.5 for 2.2% < ρ_{eff} ≤ 2.9%; 5.5 for 2.9% < ρ_{eff} ≤ 3.5%;
- $\emptyset$ is the diameter of the rebars

In the last step, the rotation capacity of the joint is calculated with the following equation by implementing the values obtained in the two previous steps:

$$\emptyset_u = \left[\varepsilon_{smu} \cdot \frac{h_c}{2} + \frac{\varepsilon_{smu} + \varepsilon_{smy}}{2} \cdot \left(L_j - \frac{h_c}{2}\right)\right] \cdot \frac{1}{h_r} \tag{68}$$

with h_r the internal lever arm between compression point and reinforcement layer.

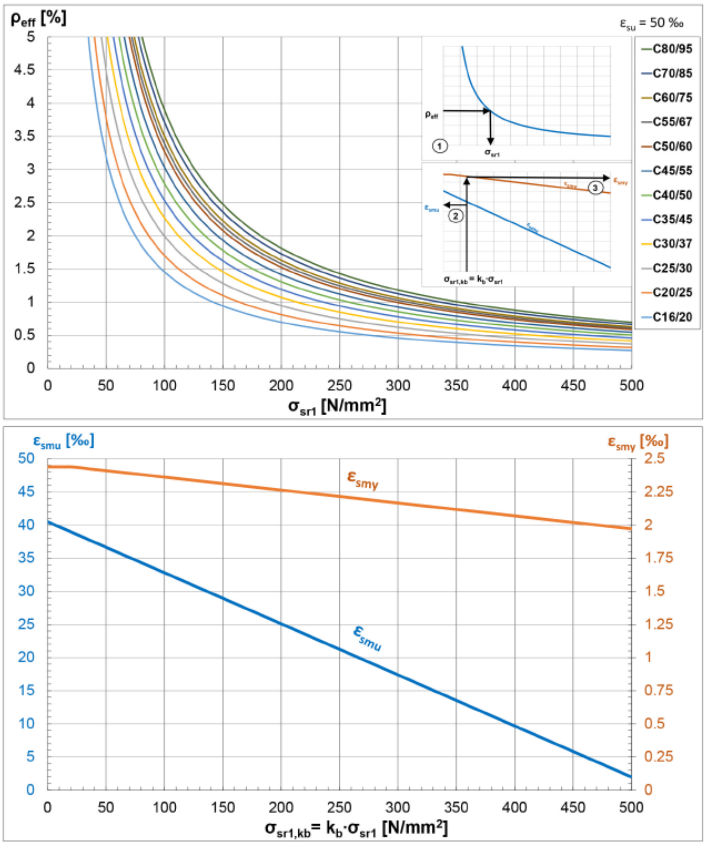


Figure 144. Design aid to determine the rotation capacity of composite joints according to (Duarte da Costa, 2018)

A.5 Resistance of joints under tension

Within the present annex, rules for the prediction of the axial resistance of simple joints, partial-strength joints, and column splices are provided.

A.5.1 Simple joints under tension

Specific design sheets for the derivation of the tying resistance of commonly used simple joint configurations have been prepared in (Jaspart et al., 2009) as part of European design recommendations; in particular, rules are provided for the prediction of the ultimate axial resistance of these joints. These rules are reported here below as they constitute easy-to-apply calculations procedures. It has however to be pointed out that their application is strictly limited to joints satisfying the minimum requirements in terms of ductility reported in Section 2.2.

A.5.1.1 General data for connections with header plate, fin plate or web cleats

- For the bolts:
  - n Total number of bolts
  - A Nominal area of a bolt
  - A_s Resistant area of a bolt
  - d Nominal diameter of a bolt shank
  - d_0 Diameter of a bolt hole
  - $f_{u,b}$ Ultimate strength of a bolt
  - $f_{y,b}$ Yield strength of a bolt

- For the welds:
  - a Throat thickness of the welds
  - β_w Correlation factor for the evaluation of the weld resistance

- For the supporting and supported elements:
  - t Thickness of the supporting plate (t_{cf} and t_{cw} for respectively a column flange and web, t_{bw} for a beam web)
  - t_w Thickness of the supported beam web
  - $A_{b,v}$ Gross shear area of the supported beam
  - $A_{b,v,net}$ Net shear area of the supported beam
  - f_u Ultimate strength of a steel element (index b_w for beam web, c_f and c_w for respectively column flange and web)
  - f_y Yield strength of a steel element (index b_w for beam web, c_f and c_w for respectively column flange and web)

- Safety coefficients:
  - γ_{M0} Partial safety factor for steel sections; it is equal to 1,0
  - γ_{M2} Partial safety factor for net section at bolt holes, bolts, welds, and plates in bearing; it is equal to 1,25

- Loading:
  - V_{Ed} Shear force applied to the joint

- Resistance:
  - V_{Rd} Shear resistance of the joint

$F_{v.Rd}$ Design resistance in shear

A.5.1.2 Particular notations for header plate connections

h_p Height of the header plate
t_p Thickness of the header plate
A_v Gross shear area of the header plate
A_{vnet} Net shear area of the header plate
f_{yp} Yield strength of the header plate
n_1 Number of horizontal rows
n_2 Number of vertical rows
e_1 Longitudinal end distance
e_2 Transverse end distance
p_1 Longitudinal bolt pitch
p_2 Transversal bolt pitch
m_p Distance between the bolt columns and the toe of the weld connecting the header plate to the beam web (definition according to EN 1993-1-8)

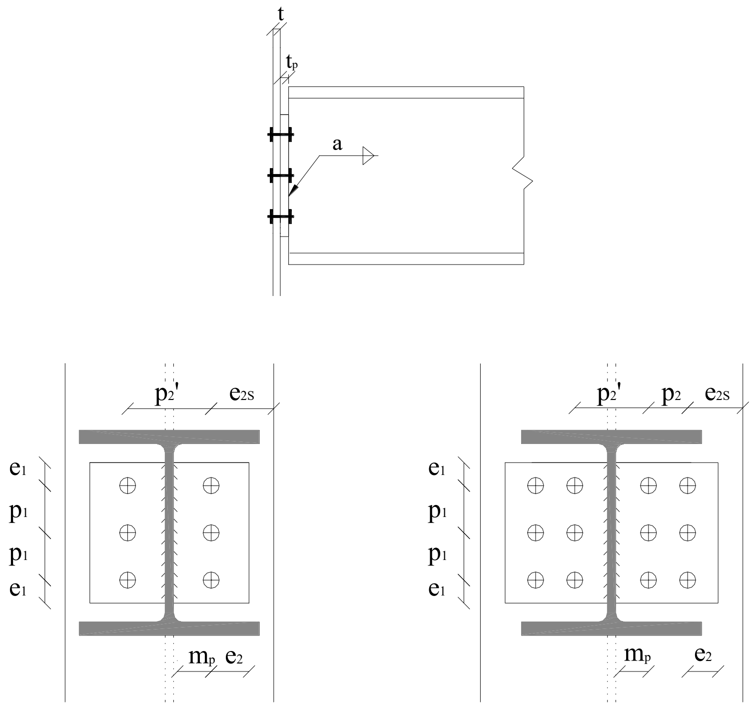


Figure 145. Header plate notations

A.5.1.3 Particular notations for fin plate connections

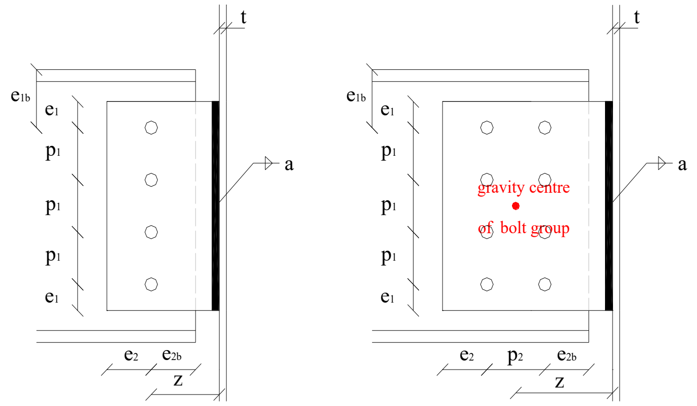
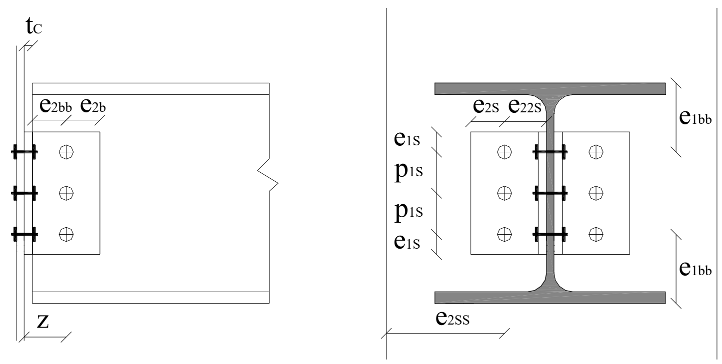


Figure 146. Fin plate notations

h_p Height of the fin plate
t_p Thickness of the fin plate
A_v Gross shear area of the fin plate
A_{vnet} Net shear area of the fin plate
f_{yp} Yield strength of the fin plate
n_1 Number of horizontal rows
n_2 Number of vertical rows
e_1 Longitudinal end distance (fin plate)
e_2 Transverse end distance (fin plate)
e_{1b} Longitudinal end distance (beam web)
e_{2b} Transverse end distance (beam web)
p_1 Longitudinal bolt pitch
p_2 Transverse bolt pitch

I Moment of inertia of the bolt group

A.5.1.4 Particular notations for cleat web connections



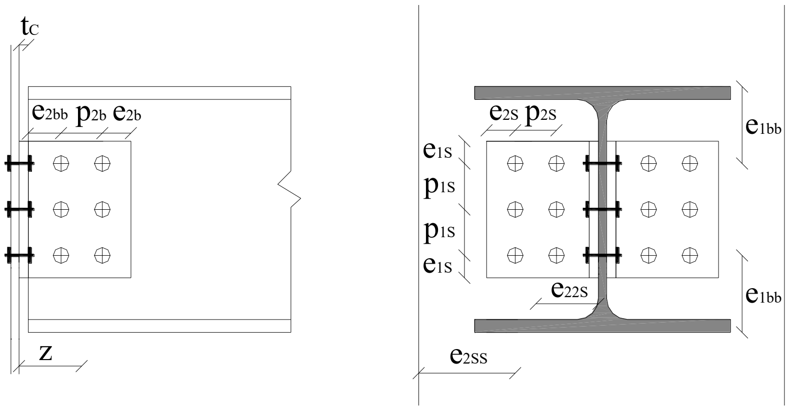


Figure 147. Web cleat notations

h_c Height of the cleat
t_c Thickness of the cleat
A_v Gross shear area of the cleat
A_{vnet} Net shear area of the cleat

Supported beam side:

d_{sb} Nominal diameter of a bolt shank
d_{0sb} Diameter of a bolt hole
n_b Total number of bolts
n_{1b} Number of horizontal rows
n_{2b} Number of vertical rows
e_{1b} Longitudinal end distance (cleat)
e_{2b} Transverse end distance (cleat)
p_{1b} Longitudinal bolt pitch
p_{2b} Transverse bolt pitch
e_{2bb} Transverse end distance (beam web)
e_{1bb} Longitudinal end distance (beam flange)
z Lever arm
I Moment of inertia of the bolt group

Supporting element side:

d_s Nominal diameter of a bolt shank
d_{0s} Diameter of a bolt hole
n_s Total number of bolts
n_{1s} Number of horizontal rows
n_{2s} Number of vertical rows
e_{1s} Longitudinal end distance (cleat)
e_{2s} Transverse end distance (cleat)
p_{1s} Longitudinal bolt pitch
p_{2s} Transverse bolt pitch
e_{2ss} Transverse end distance (supporting element)
e_{22s} Longitudinal distance between the inner bolt column and the beam web

A.5.1.5 Tying resistance of header plate connections

FAILURE MODE	VERIFICATION
Bolts in tension	$N_{u\,1} = n\, B_{t,u}$ with: $B_{t,u} = f_{ub}\, A_s/\gamma_{Mu}$
Header plate in bending	$N_{u\,2} = \min\,(F_{hp,u,1};\, F_{hp,u,2})$ $$F_{hp,u,1} = \frac{(8\,n_p - 2\,e_w)\,l_{eff.p.t,1}\;m_{u.p}}{2\,m_p\;n_p - e_w\;(m_p + n_p)}$$ $$F_{hp,u,2} = \frac{2\,l_{eff.p.t,2}\;m_{u.p} + n\,B_{t.u}\;n_p}{m_p + n_p}$$ where $n_p = \min\,(e_2;\, 1{,}25\, m_p)$ $$m_{u.p} = \frac{t_p^2\; f_{up}}{4\gamma_{Mu}}$$ $$l_{eff.p1} = l_{eff.p2} = h_p$$ (usually safe value; see EC3 – table with effective lengths for end-plates, case “Bolt-row outside tension flange of beam” – for more precise values; the effective lengths given in the table have however to be multiplied by a factor 2 before being introduced in the two above-written expressions)
Supporting member in bending	$N_{u\,3} =$ See EN 1993-1-8 for column flanges (with substitution of $B_{t.Rd}$ by $B_{t,u}$, f_y by f_u and γ_{M0} by γ_{Mu}).
Beam web in tension	$N_{u\,4} = t_w\, h_p\, f_{ubw}/\gamma_{Mu}$
Welds	The full-strength character of the welds is ensured through recommendations for weld design given in Section 2.2
Tying resistance of the joint	$$N_u = \min_{i=1}^{4} N_{u\,i}$$

A.5.1.6 Tying resistance of fin plate connections

FAILURE MODE	VERIFICATION
Bolts in shear	$N_{u\,1} = n\, F_{v,u}$ with: $F_{v,u} = \alpha_v \; f_{ub} \; A/\gamma_{Mu}$ • where the shear plane passes through the threaded portion of the bolt: A = A_s (tensile stress area of the bolt) • for 4.6, 5.6 and 8.8 bolt grades: α_v = 0,6 • for 4.8, 5.8, 6.8 and 10.9 bolt grades: α_v = 0,5 • where the shear plane passes through the unthreaded portion of the bolt: A (gross cross area of the bolt) and α_v = 0,6
Fin plate in bearing	$N_{u\,2} = n\, F_{b,u,\,hor}$ with: $F_{b,u,hor} = k_1 \; \alpha_b \; f_{up} \; d \; t_p/\gamma_{Mu}$ where: $\alpha_b = \min\left(\frac{e_2}{3d_0}\; ; \; \frac{p_2}{3d_0} - \frac{1}{4}\; ; \; \frac{f_{ub}}{f_{up}} \text{ ou } 1{,}0\right)$ $k_1 = \min\left(2{,}8\frac{e_1}{d_0} - 1{,}7\; ; 1{,}4\frac{p_1}{d_0} - 1{,}7\; ; 2{,}5\right)$
Fin plate in tension: Gross section	$N_{u\,3} = t_p\, h_p\, f_{up}/\gamma_{Mu}$
Fin plate in tension: Net section	$N_{u\,4} = 0{,}9\, A_{net,p}\, f_{up} / \gamma_{Mu}$ with: $A_{net,p} = t_p\, h_p - d_0\, n_1\, t_p$
Beam web in bearing	$N_{u\,5} = n\, F_{b,u,hor}$ with: $F_{b,u,hor} = k_1 \; \alpha_b \; f_{ubw} \; d \; t_{bw}/\gamma_{Mu}$ where: $\alpha_b = \min\left(\frac{e_{2b}}{3d_0}\; ; \; \frac{p_2}{3d_0} - \frac{1}{4}\; ; \; \frac{f_{ub}}{f_{ubw}} \text{ ou } 1{,}0\right)$ $k_1 = \min\left(1{,}4\frac{p_1}{d_0} - 1{,}7\; ; 2{,}5\right)$
Beam web in tension: Gross section	$N_{u\,6} = t_{bw}\, h_{bw}\, f_{ubw}/\gamma_{Mu}$
Beam web in tension: Net section	$N_{u\,7} = 0{,}9\, A_{net,bw}\, f_{ubw}/\gamma_{Mu}$ with: $A_{net,bw} = t_{bw}\, h_{bw} - d_0\, n_1\, t_{bw}$
Supporting member in bending	$N_{u\,8}$ = See EN 1993-1-8 for column flanges (with substitution of $B_{t.Rd}$ by $B_{t,u}$, f_y by f_u and γ_{M0} by γ_{Mu}).

FAILURE MODE	VERIFICATION
Welds	The full-strength character of the welds is ensured through recommendations for weld design given in Section 2.2
Tying resistance of the joint	$N_u = \min_{i=1}^{8} N_{u\,i}$

A.5.1.7 Tying resistance of connections with web cleats

The corresponding sheet could be added but, in fact, it combines the formulae presented here above as the two legs of the cleats may be easily assimilated respectively to a fin plate and to a header plate.

A.5.2 Partial-strength joints and column splices under tension

The component method can be easily adapted to allow the characterisation of joints under axial forces and in particular, under tensile loads which is the loading condition to be considered when applying, for instance, the tying method approach.

Indeed, the components activated under axial loads are similar to the ones activated under bending. Accordingly, applying the component method concepts, only the assembly procedure needs to be adapted in order to be able to predict the design axial resistance of joints:

$$N_{j,Rd} = \sum_i F_{Rd,i} \tag{69}$$

where $N_{j,Rd}$ is the axial design resistance of the considered joint and $F_{Rd,i}$ the design resistance of the component "i" activated under N (EN 1993-1-8, 2005).

Accordingly, the axial resistance of a joint can be simply predicted by summing the contribution of the different activated components. This formula is only valid if the ductility criteria reported in Section 2.2 are satisfied.

It is also possible to predict the ultimate axial resistance of joints replacing the design resistance of the components by their ultimate resistance:

$$N_{j,u} = \sum_i F_{u,i} \tag{70}$$

where $N_{j,u}$ is the axial ultimate resistance of the considered joint and $F_{u,i}$ the ultimate resistance of the component "i" activated under N obtained by substituting $B_{t.Rd}$ by $B_{t,u}$, f_y by f_u and γ_{M0} by γ_{Mu} in the rules provided in (EN 1993-1-8 2005).

A.5.3 Simplified method for the characterisation of steel or composite joints with bolted endplates under axial force

Following the concept presented in A.3.1 and using a reduction factor for the pure tension loading conditions, (Rölle, 2013) provided a formula (on the level of ultimate resistance) for the calculation of the joint's capacity under pure tension loading conditions:

$$N_{j,u} = k_j \cdot k_{j,T} \cdot F_{t,u} + F_{RFT,u} \tag{71}$$

where:

$$k_j = 1{,}95\left(\frac{t_{ep}\cdot t_{cf}\cdot f_y}{m\cdot m_x\cdot f_{ub}}\right)^{0{,}25}$$ Joint correction factor

$$k_{j,T} = \left(\left(\frac{m_x}{3{,}0\cdot d_b}\right)\cdot\left(1-\frac{m}{p}\right)\right)^{0{,}25}$$ Reduction factor for tension loading

$$F_{t,u} = A_s\cdot f_{ub}$$ Axial load carrying capacity of the bolts (failure)

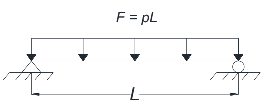
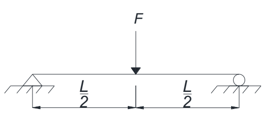
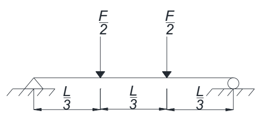
$$F_{RFT,u} = A_{s,RFT}\cdot f_{s,u}$$ Ultimate load carrying capacity of steel reinforcement

A.6 Tabular tools for response estimation of SDOF systems

A.6.1 Transformation factors for beams and one-way slabs

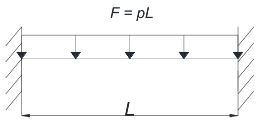
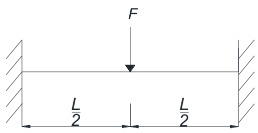
To determine the response of the SDOF systems with elasto-plastic behaviour, the ultimate resistance R_m, loading factors (K_L), mass factors (K_M), load mass factors (K_{LM}), spring constant (k) and dynamic reactions (V), can be determined for beams and one-way slabs from the following tables.

Table 66. Transformation Factors for Beams and One-way Slabs - simply supported beam (Biggs and Biggs, 1964)

Loading diagram	Strain range	Loading factor K_L	Mass factor K_M		Load-mass factor K_{LM}		Maximum resistance R_m	Spring constant k	Dynamic reaction V
			Concentrated mass*	Uniform mass	Concentrated mass*	Uniform mass			
	Elastic	0.64	...	0.50	...	0.78	$\frac{8M_P}{L}$	$\frac{384EI}{5L^3}$	0.39R+0.11F
	Plastic	0.50	...	0.33	...	0.66	$\frac{8M_P}{L}$	0	0.38R_m+0.12F
	Elastic	1.0	1.0	0.49	1.0	0.49	$\frac{4M_P}{L}$	$\frac{48EI}{L^3}$	0.78R-0.28F
	Plastic	1.0	1.0	0.33	1.0	0.33	$\frac{4M_P}{L}$	0	0.75R_m-0.25F
	Elastic	0.87	0.76	0.52	0.87	0.60	$\frac{6M_P}{L}$	$\frac{56.4EI}{L^3}$	0.525R-0.025F
	Plastic	1.0	1.0	0.56	1.0	0.56	$\frac{6M_P}{L}$	0	0.52R_m-0.02F

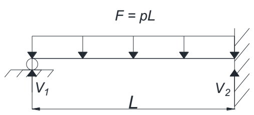
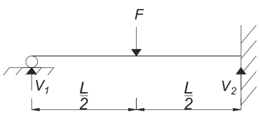
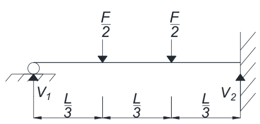
* Equal parts of the concentrated mass are lumped at each concentrated load.
Source: “Design of Structures to Resist the Effects of Atomic Weapons”, U.S Army Corps of Engineers Manual EM 1110-345-415, 1957.

Table 67. Transformation Factors for Beams and One-way Slabs double fixed beam (Biggs and Biggs, 1964)

Loading diagram	Strain range	Loading factor K_L	Mass factor K_M		Load-mass factor K_{LM}		Maximum resistance R_m	Spring constant k	Effective spring constant k_E	Dynamic reaction V
			Concentrated mass*	Uniform mass	Concentrated mass*	Uniform mass				
	Elastic	0.53	…	0.41	…	0.77	$\frac{12M_{Ps}}{L}$	$\frac{384EI}{L^3}$	….	0.36R+0.14F
	Elastic-plastic	0.64	…	0.50	…	0.78	$\frac{8}{L}(M_{Ps}+M_{Pm})$	$\frac{384EI}{5L^3}$	$\frac{307EI}{L^3}$	0.39R+0.11F
	Plastic	0.50	…	0.33	…	0.66	$\frac{8}{L}(M_{Ps}+M_{Pm})$	0	….	0.38R_m+0.12F
	Elastic	1.0	1.0	0.37	1.0	0.37	$\frac{4}{L}(M_{Ps}+M_{Pm})$	$\frac{192EI}{L^3}$	….	0.71R-0.21F
	Plastic	1.0	1.0	0.33	1.0	0.33	$\frac{4}{L}(M_{Ps}+M_{Pm})$	0	….	0.75R_m-0.25F

M_{Ps} – ultimate moment capacity at support
M_{Pm} – ultimate moment capacity at midspan
* Concentrated mass is lumped at the concentrated load.
Source: "Design of Structures to Resist the Effects of Atomic Weapons", U.S Army Corps of Engineers Manual EM 1110-345-415, 1957.

Table 68. Transformation Factors for Beams and One-way Slabs simply supported and fixed beam (Biggs and Biggs, 1964)

Loading diagram	Strain range	Load factor K_L	Mass factor K_M		Load-mass factor K_{LM}		Maximum resistance R_m	Spring constant k	Effective spring constant k_E	Dynamic reaction V
			Concentr. mass*	Uniform mass	Concentr. mass*	Uniform mass				
	Elastic	0.58	…	0.45	…	0.78	$\frac{8M_{Ps}}{L}$	$\frac{185EI}{L^3}$	$\frac{160EI}{L^3}$	V_1 = 0.26R+0.12F V_2 = 0.43R+0.19F
	Elastic-plastic	0.64	…	0.50	…	0.78	$\frac{4}{L}(M_{Ps}+2M_{Pm})$	$\frac{384EI}{5L^3}$		V = 0.39R+0.11F±M_{Ps}/L
	Plastic	0.50	…	0.33	…	0.66	$\frac{4}{L}(M_{Ps}+2M_{Pm})$	0		V = 0.38R_m+0.12F±M_{Ps}/L
	Elastic	1.0	1.0	0.43	1.0	0.43	$\frac{16M_{Ps}}{3L}$	$\frac{107EI}{L^3}$	$\frac{106EI}{L^3}$	V_1 = 0.25R+0.07F V_2 = 0.54R+0.14F
	Elastic-plastic	1.0	1.0	0.49	1.0	0.49	$\frac{2}{L}(M_{Ps}+2M_{Pm})$	$\frac{48EI}{L^3}$		V = 0.78R-0.28F±M_{Ps}/L
	Plastic	1.0	1.0	0.33	1.0	0.33	$\frac{2}{L}(M_{Ps}+2M_{Pm})$	0		V = 0.75R_m-0.25F±M_{Ps}/L
	Elastic	0.81	0.67	0.45	0.83	0.55	$\frac{6M_{Ps}}{3L}$	$\frac{132EI}{L^3}$	$\frac{122EI}{L^3}$	V_1 = 0.17R+0.17F V_2 = 0.33R+0.33F
	Elastic-plastic	0.87	0.76	0.52	0.87	0.60	$\frac{2}{L}(M_{Ps}+3M_{Pm})$	$\frac{56EI}{L^3}$		V = 0.525R-0.025F±M_{Ps}/L
	Plastic	1.0	1.0	0.56	1.0	0.56	$\frac{2}{L}(M_{Ps}+3M_{Pm})$	….		V = 0.52R_m-0.02F±M_{Ps}/L

M_{Ps} – ultimate bending capacity at support
M_{Pm} – ultimate bending capacity at midspan
* Equal parts of the concentrated mass are lumped at each concentrated load.
Source: "Design of Structures to Resist the Effects of Atomic Weapons", U.S Army Corps of Engineers Manual EM 1110-345-415, 1957.

A.6.2 Maximum deflection and maximum response time of elasto-plastic SDOF systems

To determine the response of the SDOF system with elasto-plastic behaviour, the required ductility μ, given by the ratio y_m/y_e, as a function of t_d/T_n is presented in chart form, as a family of curves R_m/F_m.

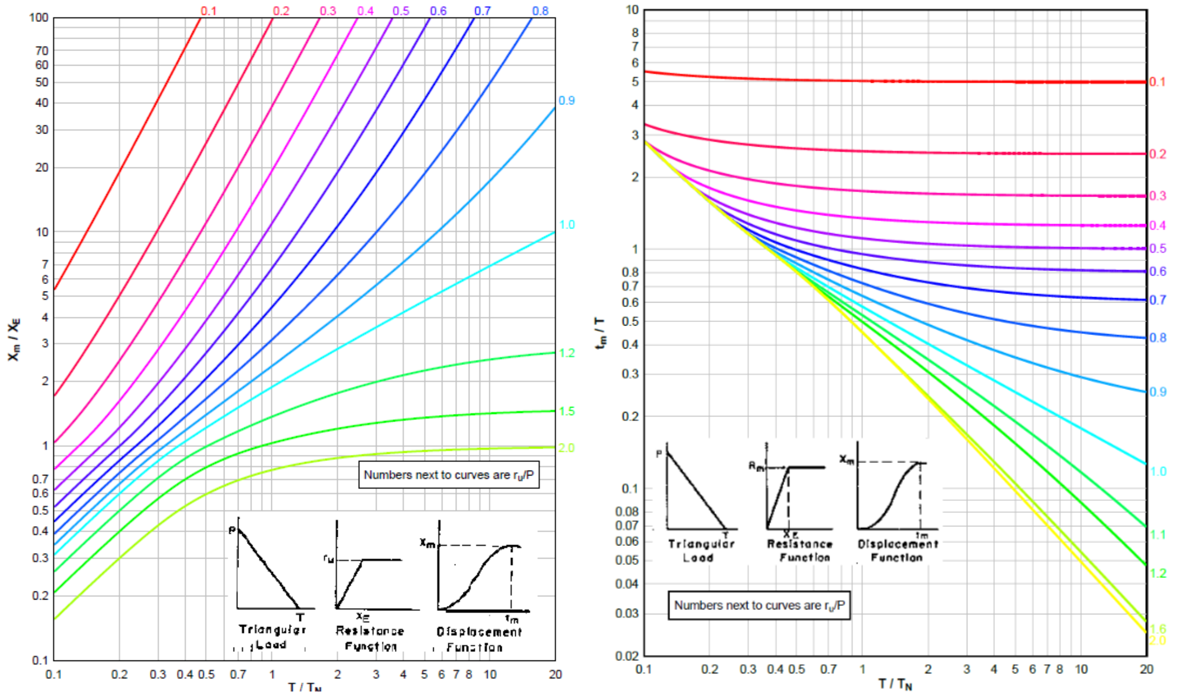


Figure 148. Maximum deflection (a) and maximum response time (b) of elasto-plastic SDOF system for triangular load (DoD, 2008)

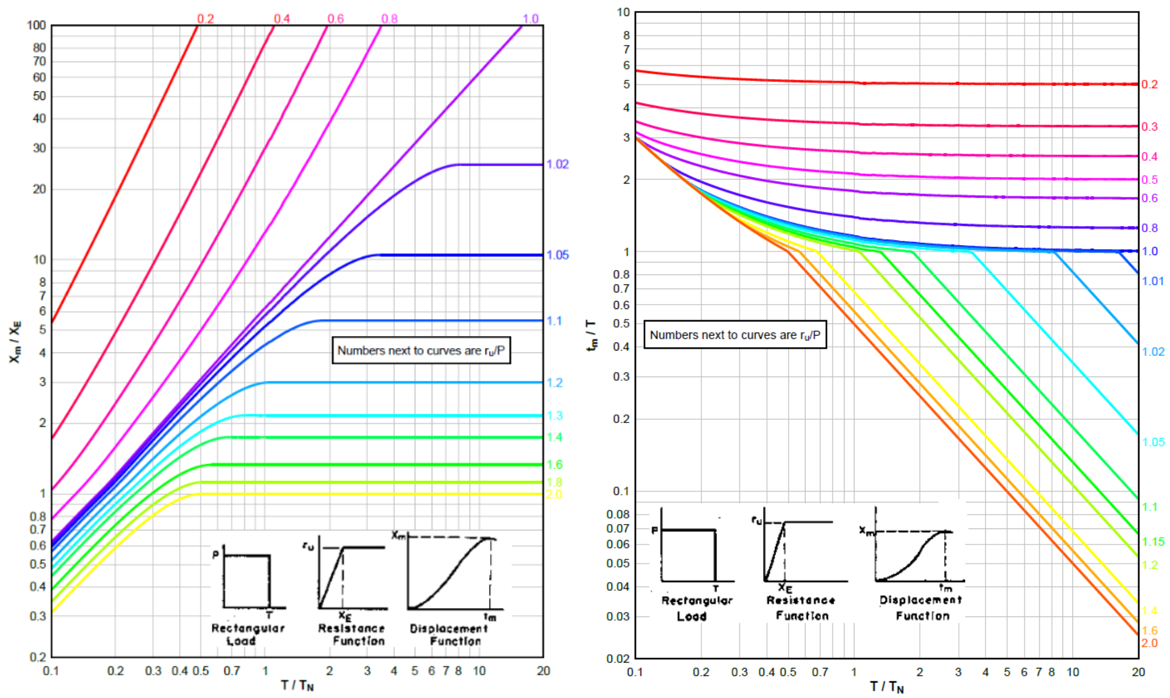


Figure 149. Maximum deflection (a) and maximum response time (b) of elasto-plastic SDOF system for rectangular load (DoD, 2008)

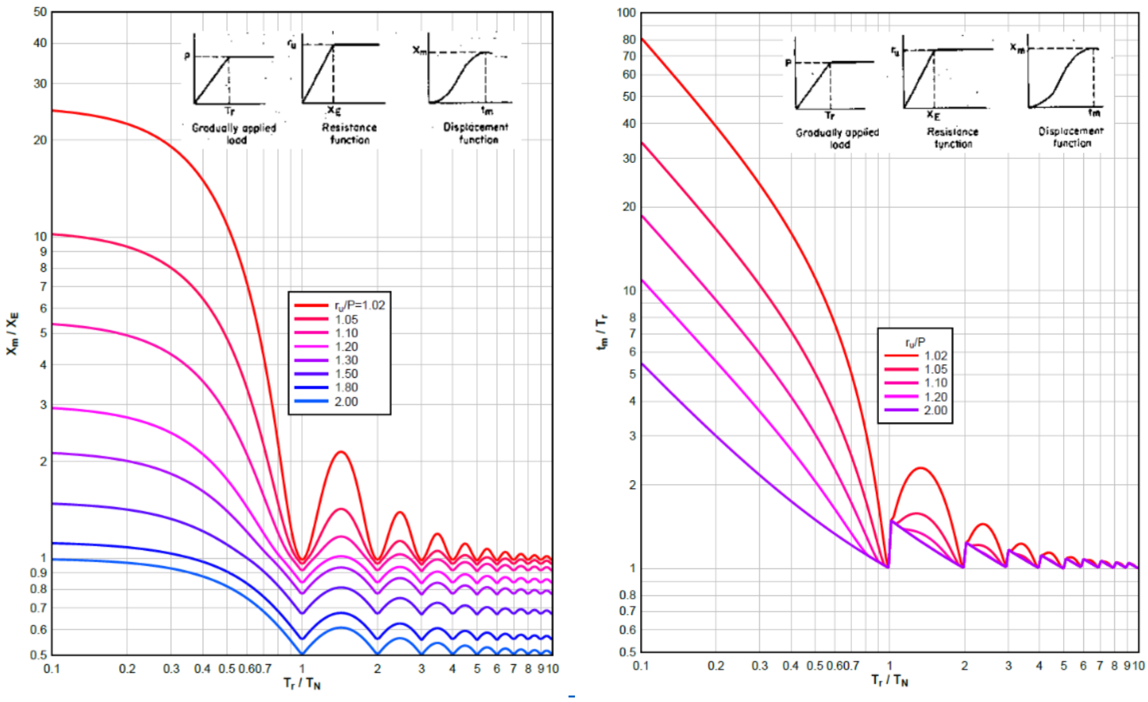


Figure 150. Maximum deflection (a) and maximum response time (b) of elasto-plastic SDOF system for gradually applied load

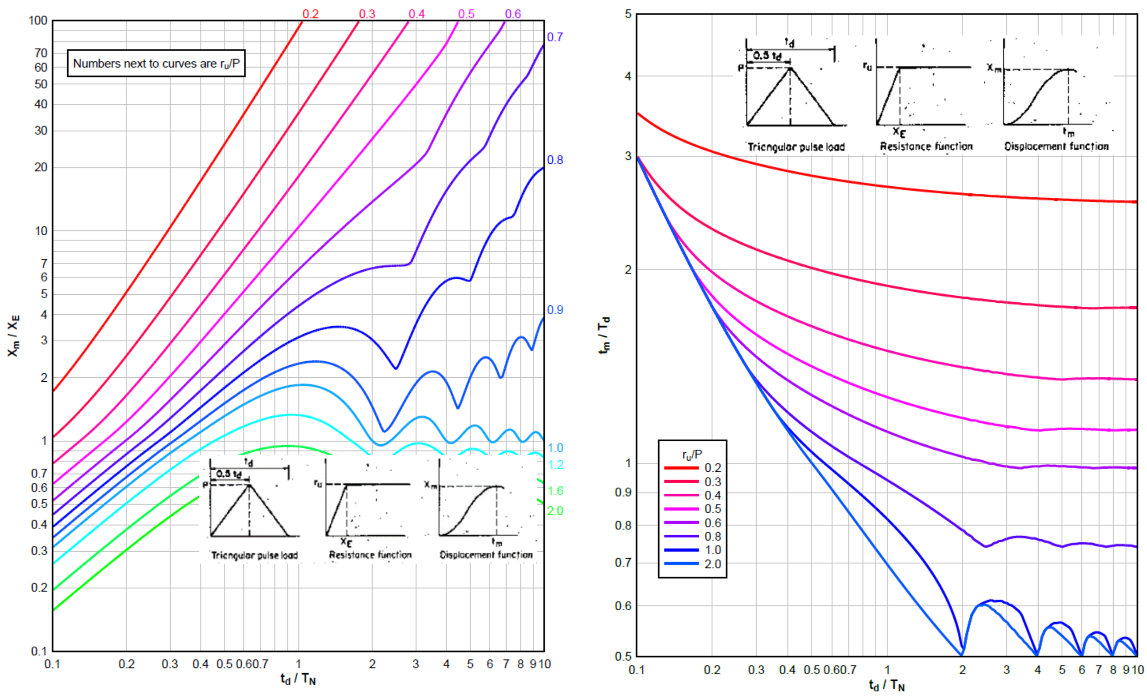


Figure 151. Maximum deflection (a) and maximum response time (b) of elasto-plastic SDOF system for triangular pulse load

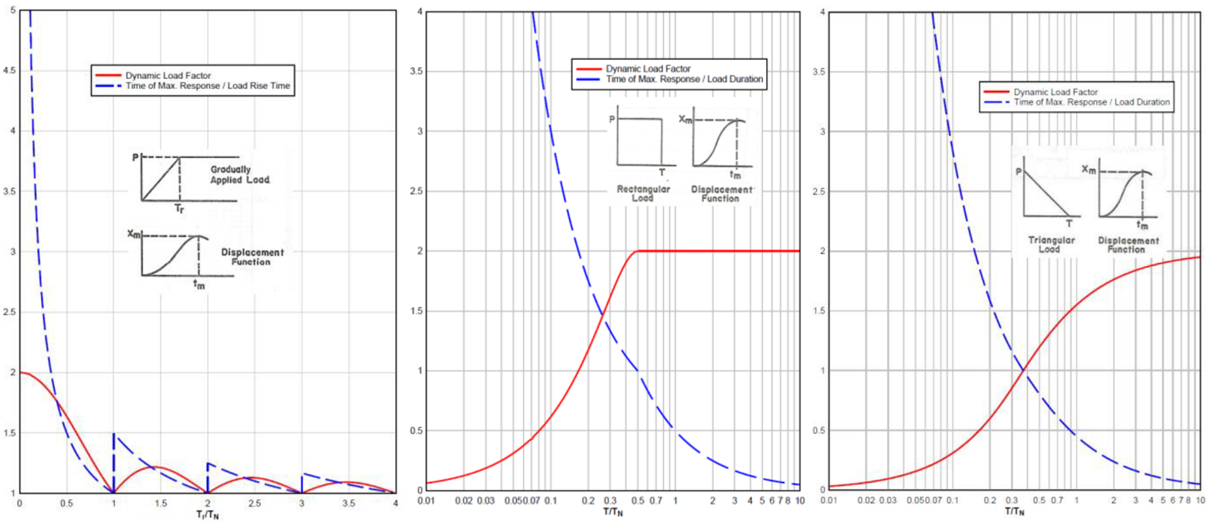


Figure 152. Maximum response of elastic, one-degree-of-freedom system for gradually applied load (a), for rectangular load (b) and for triangular load (c)

A.7 Simplified analytical method for 3D structures with simple joints

The formulas reported in Section 5.3.2.2 to predict the membrane forces and the required rotation at the level of the joints of 2D structures assuming an infinitely stiff diaphragm effect coming from the slab can be extended to 3D structures by means of small adaptations. For 3D structures, the sub-system to be considered becomes the one reported in Figure 153.

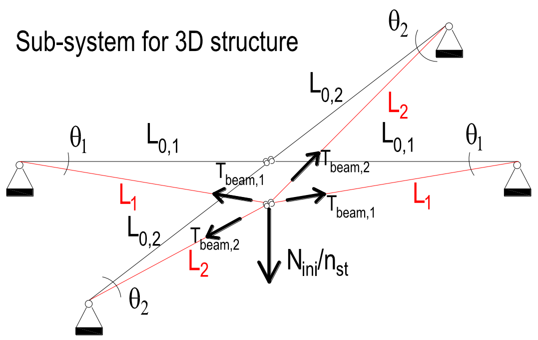


Figure 153. Sub-system for 3D structures

For this sub-system, it is again possible to predict its response using the equations of equilibrium and expressing the compatibility of displacement. In this system, four unknowns have to be determined: $T_{beam,1}$, $T_{beam,2}$, θ_1 and θ_2. The obtained systems of equations are provided in Table 69.

Table 69. System of equations for 3D structures with simple joints

	3D Structures with simple joints
Eq. 1	$\frac{N_{ini}}{n_{st}} = 2.T_{beam,1}.\sin\theta_1 + 2.T_{beam,2}.\sin\theta_2$
Eq. 2	$T_{beam,1} = \frac{1-\cos\theta_1}{\cos\theta_1}.E.A_1$
Eq. 3	$T_{beam,2} = \frac{1-\cos\theta_2}{\cos\theta_2}.E.A_2$
Eq. 4	$L_{0,1}.\tan\theta_1 = L_{0,2}.\tan\theta_2$

where the geometrical parameters are defined in Figure 153, $T_{beam,1}$ and $T_{beam,2}$ are the tensile loads in the beams in both directions, A_1 and A_2 are the cross-section of the beams in both directions.

A.8 Advanced analytical approach

A more general and detailed analytical approach has been developed and presented in (Huvelle et al., 2015). This model allows the prediction of the response of a 2D frame with simple, partial-strength or full-strength joints when membrane forces develop within the directly affected part during a column loss scenario.

The model is founded on the definition of a substructure and on its characterisation through analytical formula (see Figure 154) adopting the following assumptions:

- a progressive (static) column loss is assumed;
- the hinges can develop in the beam cross-sections or in the beam-to-column joints;
- all columns are made of a unique cross-section type, and it is the same for the beams;
- only the loss of internal columns (i.e., columns which are not at the corners) is considered;
- no yielding develops in the rest of the structure, called the indirectly affected part (i.e., its behaviour is assumed to be infinitely elastic).

The effect of the indirectly affected part on the response of the directly affected part is simulated through the definition of horizontal springs each side of each storey characterised by a stiffness K_H (as the behaviour of the indirectly affected part is assumed to be fully elastic, only a stiffness is required for its characterisation). Also, in the proposed model, one of the main parameters affecting the response of the substructure is the behaviour of the yielded zone which appears first under bending moment and then is submitted to bending moment and axial load while the catenary action is developing. These yielded zones are simulated by a multi-layer spring model as illustrated in Figure 154 with elastic-perfectly plastic behaviour laws assigned to each spring. The use of this multi-layer spring model allows considering situations for which the yielded zones are developing in beam cross-sections or at the level of beam-to-column joints using the component method principles.

The proposed analytical model consists in solving a system of N equations with N unknowns defined in Table 70. These equations have been derived using the static and the cinematic theorems, i.e. expressing the equilibrium of the system and the compatibilities of displacement. This system of equations is easily solvable through the use of a mathematic software. Through the model, the following results can be obtained:

- Vertical displacement – u, in particular:
    - the maximum displacement, and;
    - the remanent displacement
- Deformations at the level of the yielded zone;
- Horizontal deflections of the IAP;
- Internal forces in the system.

Table 70. System of equations and unknowns for the analytical model (Huvelle et al., 2015)

Unknowns	Number	Equations
u	1	u = input data
θ	n_{st}	$\sin(\theta) = u/(L_0 - 2L + \Delta_L)$
δ	n_{st}	$\cos(\theta) = (L_0 - 2L - \delta_H - 2\delta)/(L_0 - 2L + \Delta_L)$
$\delta_{H,l}$	n_{st}	$\delta_{H,l}(n_{st} \times 1) = S_l(n_{st}n_{st})F_H(n_{st})$
$\delta_{H,r}$	n_{st}	$\delta_{H,r}(n_{st}1) = S_r(n_{st}n_{st})F_H(n_{st})$
Δ_L	n_{st}	$\Delta_L = F_H(L_0 - 2L)/(EA)$
M	n_{st}	$M = \sum F_i h_i$
F_H	n_{st}	$F_H = \sum F_i$
F_i $(i = [1{:}6])$	$6 * n_{st}$	$F_i = f(\delta_i)$
δ_i $(i = [1{:}6])$	$6 * n_{st}$	$\delta_i = \delta + h_i\theta$
P	n_{st}	$-0.5P(L_0 - 0.5(\delta_{H,l} + \delta_{H,r})) + F_H u + 2M = 0$
P_{tot}	1	$P_{tot} = \sum P$

where:

- the geometrical parameters are defined in Figure 154;
- n_{st} is the number of storeys in the directly affected part;
- F_i is the axial force in each spring of the multi-layer spring models;
- δ_i is the elongation of each spring of the multi-layer spring models;
- F_H is the tensile force applied at the level of the yielded zones;
- M is the applied bending moment at the level of the yielded zones;
- $\delta_{H,l}$ and $\delta_{H,r}$ are respectively the elongation of the horizontal spring on the left and on the right of each storey;
- S_l and S_r are respectively the flexibility coefficients of the indirectly affected part on the left and on the right of each storey (these coefficients can be obtained through a linear elastic analysis performed on the indirectly affected part - see (Huvelle et al., 2015) for more details);
- P is the vertical load supported by each storey at the level of the lost column and;
- P_{tot} is the load associated to the column loss.

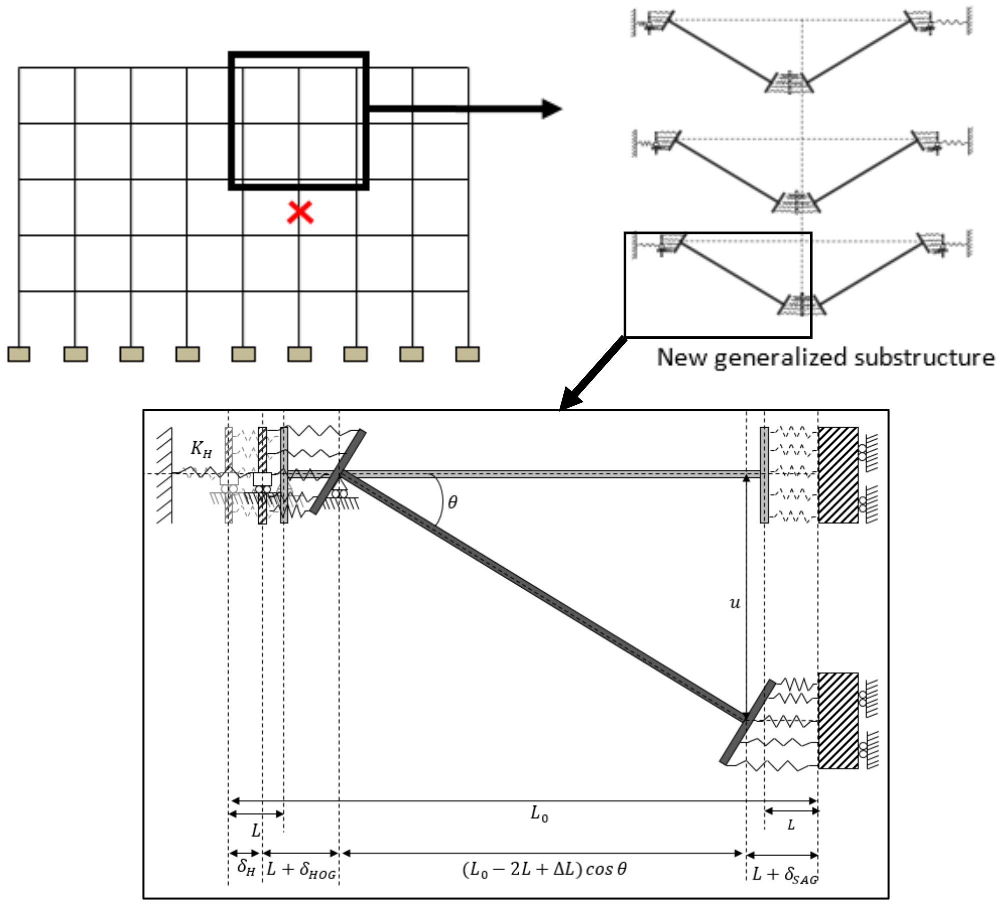


Figure 154. Definition of the substructure (Huvelle et al., 2015)

In (Kulik, 2014) and (Ghimire, 2016), it has been demonstrated how this model can be extended to 3D structures made of linear members. The extension of the analytical model consists in considering the response of a 3D structure as the sum of the response of two 2D frames intersecting at the level of the loss column as illustrated in Figure 155 and expressing the compatibility of displacement at the point of column loss.

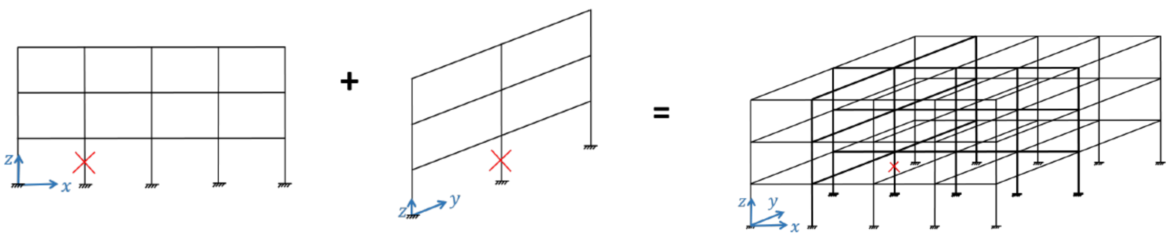


Figure 155. Superposition of the longitudinal and transversal frame response to obtain the 3D response (Jacques, 2019)

The directly affected part is checked for the state of stresses and deformations resulted from the analytical model for the maximum displacement. As mentioned in Section 5.1, compression forces may also develop in the top beams of the directly affected part in addition to the bending moments. So, in some cases, beams at the top levels have to be checked as beam-columns for stability.

Also, within the analytical model, no deformation limits are included. Accordingly, the deformation capacity of the different yielded zones of the DAP has to be checked for the maximum vertical deflection obtained through the analytical model when P_{tot} is equal to N_{ini} (see Section 5.3.2).

The indirectly affected part has also to be checked for the state of stresses and deformations associated to the maximum observed displacement, i.e., when the column is assumed to be lost is fully removed. Knowing the internal forces at the extremities of the substructure model, it is then possible to predict the internal forces in the IAP applying to the latter the loads at the extremities of each storey of the substructure. The check of the IAP is then performed according to Eurocode 3 and/or Eurocode 4. A specific attention has to be paid to the columns close to the lost column, which are supporting additional axial compression forces but also bending moments coming from the development of the membrane forces in the system. Also, the joints at the extremities of the beams of the IAP need to be checked as they are subjected to additional axial forces associated to the development of the membrane forces in the DAP (see Section 2.2.2).

Part 4 – References

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Structural robustness for the mitigation of progressive collapse is a specific safety consideration which is now addressed in modern codes and standards, including the Eurocodes, and which requires particular care from all professionals involved in the construction industry, including architects, designers, constructors, control officers- and insurance managers. The importance of the robustness design has been recognised by world shaking disasters such as the 9/11 collapse of Twin Towers in New York City and the need for practical guidelines has been triggered at this occasion. Indeed, the availability of practical guidelines addressed to the various construction professionals and covering specific use and risk situations for buildings helps to give confidence in the safety of steel and composite constructions.

During the past decade, a significant number of research projects related to the structural response of steel and composite buildings under various exceptional loading situations (impact, fire, earthquake…) have been carried out, especially in Europe and in the USA. As an outcome of these recent scientific actions, different design approaches have been proposed to mitigate progressive collapse accounting for the full potential of materials used in steel and composite structures.

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The purpose of the project entitled “Mitigation of the risk of progressive collapse in steel and composite building frames- FAILNOMORE” was to consolidate the knowledge developed in the aforementioned research and transform it into practical recommendations and guidelines. The set of practical and user-friendly design guidelines considered in the project focuses on steel and composite structures subjected to unidentified threats and identified threats such as impacts, explosions, fires and earthquakes; it refers also to the available normative documents so as to form in itself a commonly agreed European design methodology.

The project was funded for 24 months (starting from July 2020) by the Research Fund for Coal and Steel (RFCS) under grant agreement No 899371.

The so-developed design guidelines are promoted through the preparation of a design manual made available in English, Portuguese, German, Italian, Romanian, Czech, Polish, Dutch, Spanish and French which will be presented through national workshops organised in 11 European countries before the end of June 2022.