Brussels 10-05-2022

Welcome!

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FAILNOMORE

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

Research Fund for Coal & Steel

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1. INTRODUCTION

Ronan Point (1968)

1. INTRODUCTION

World Trade Center (2001)

1. INTRODUCTION

- **STRUCTURAL ROBUSTNESS can be defined as:**

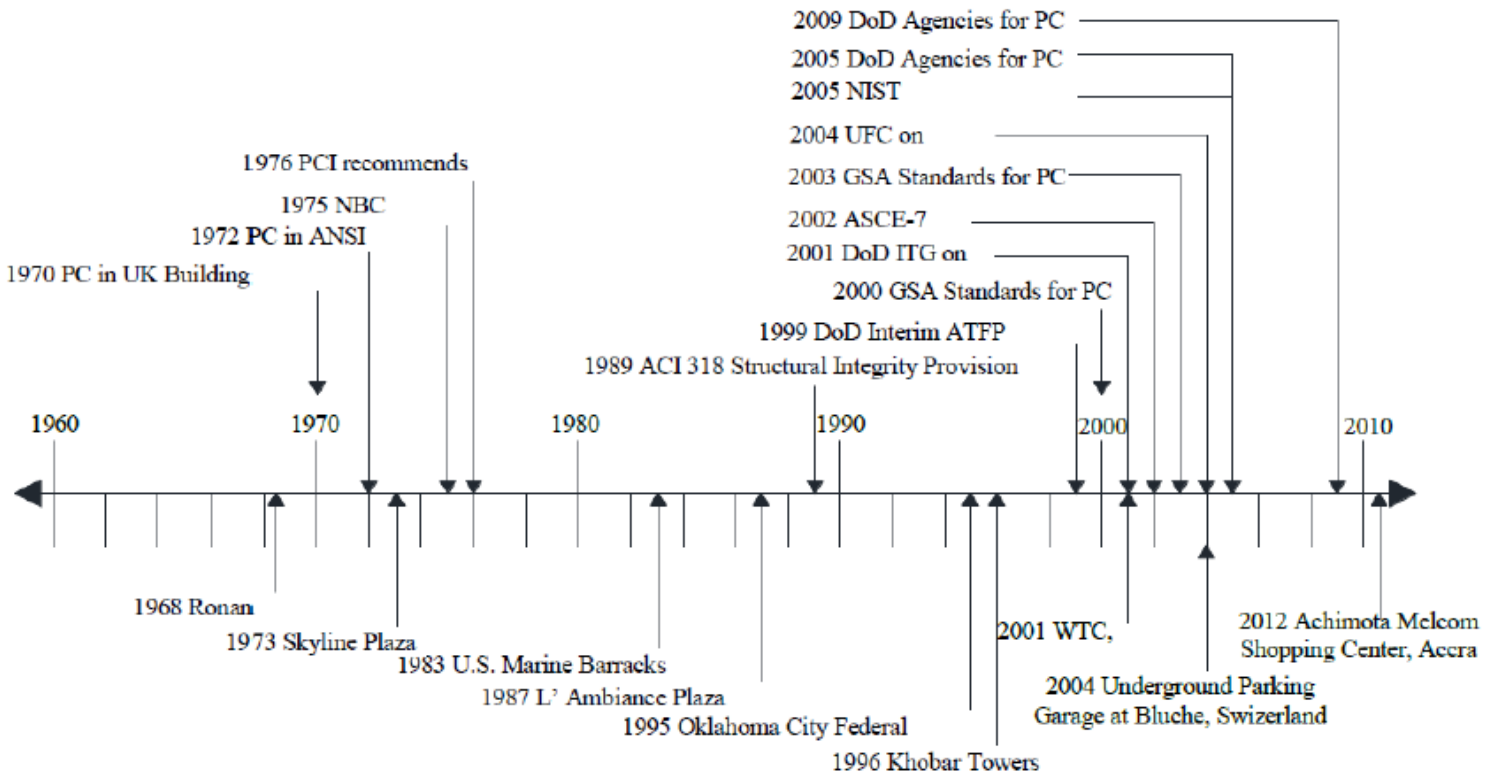
  Ability of a structure to withstand events like fire, explosions, impact or the consequences of human error, without being damaged to an extent ***disproportionate*** *to the original cause*

- **PROGRESSIVE/DISPROPORTIONATE COLLAPSE can be defined as:**

  Progressive/Disproportionate 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

2. NORMATIVE CONTEXT

- **Since the progressive collapse of the Ronan Point tower in 1968, some codes and standards have included some recommendations to limit the risk of occurrence of progressive collapse of buildings**



Qian et al., 2016

2. NORMATIVE CONTEXT

CURRENT EUROCODES

- **EN 1990**
  - **EN 1990, 2.1 (4)P** sets out the basic principle related to structural robustness:

    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

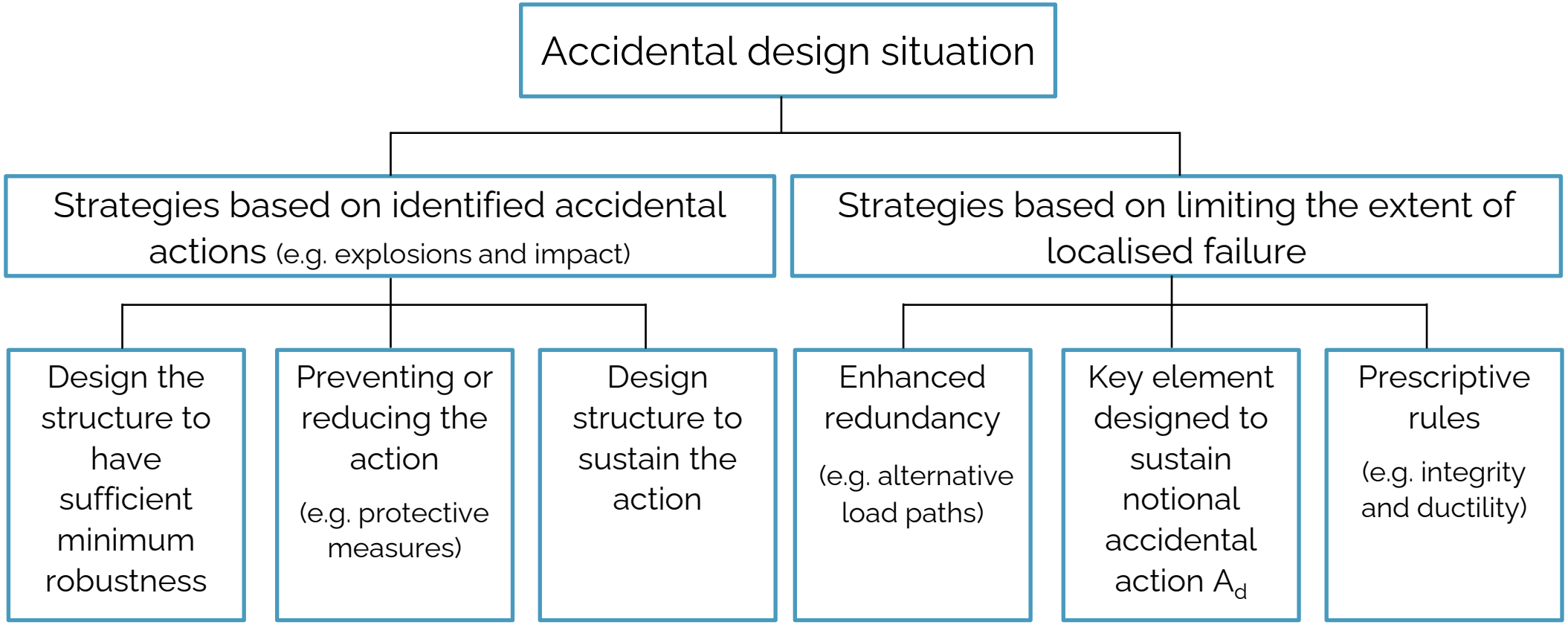
  - **EN 1990, 2.1 (5)P** states that *Potential structural damage should be avoided or limited by one or more of the following:*
    - *avoiding, eliminating or reducing hazards applied on the structure;*
    - *selecting a structural form with low sensitivity to the hazard;*
    - *selecting a form and design which can survive removal of individual or limited parts of the structure;*
    - *avoiding systems that collapse without warning;*
    - *tying members together*

    For more details, EN 1990 refers to **EN 1991-1-7**

2. NORMATIVE CONTEXT

CURRENT EUROCODES

EN 1991-1-7 – Annex A



2. NORMATIVE CONTEXT

FUTURE EUROCODES

EN 1990 – Annex E

Design for accidental actions (EN 1991)
Explicit design of the structure
(e.g. against explosion, impact)

- Design structure to resist the action (*)
- Prevent or reduce the action
  e.g. protective measures, control of events

Design for enhanced robustness (EN 1990)
Strategies based on limiting the extent of damage

- Alternative load paths
  either providing adequate deformation capacity and ductility or applying prescriptive design rules
- Key members
  i.e. designing selected members to resist notional action(s)
- Segmentation
  i.e. separation into parts

(*) 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.

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2. NORMATIVE CONTEXT

OTHER INTERNATIONAL CODES AND GUIDELINES

- **International codes and guidelines**
  - The Unified Facilities Criteria (UFC): Design of Buildings to Resist Progressive Collapse (UFC 4-023-03, developed by USA Department of Defense) - 2016
  - The USA General Services Administration: Alternate Path Analysis and design guidelines - 2016
  - ASCE 7-16 - 2017
  - The International Building Code (IBC) - 2018
  - UK Building Regulations 2010 Approved Document A - 2013
  - Chinese Code for Anti-Collapse Design of Building Structures (CECS 392) – 2014
  - Others

2. NORMATIVE CONTEXT

ASSESSMENT OF ROBUSTNESS REQUIREMENTS IN EUROCODE

- General/broad requirements that may be difficult to interpret
- No consistent set of rules available
- Lack of ductility requirements
- Need to incorporate latest research outcome to date
- Need for practical simplified methods and tools

The objective of the **FAILNOMORE project** was to address/solve these issues!

3. FAILNOMORE Project

- **Project funded by the Research Fund for Coal and Steel (RFCS) from the European Commission**

- **The main objective is to produce a set of practical and user-friendly design guidelines to mitigate the risk of progressive collapse of steel and composite structures subjected to exceptional events**

- **The proposed design guidelines are based on:**
  - recent research projects
  - available literature

3. FAILNOMORE Project

Partners:

- University of Liège – Belgium
- University of Coimbra – Portugal
- Imperial College London - UK
- University of Stuttgart – Germany
- University of Trento – Italy
- Politehnica University Timisoara – Romania
- Czech Technical University of Prague – Czech Republic
- Rzeszow University of Technology - Poland
- Technical University of Delft – The Netherlands
- Universitat Politècnica de Catalunya – Spain
- INSA de Rennes – France
- ECCS – Europe
- Feldmann + Weynand GmbH – Germany
- ArcelorMittal Belval & Differdange S.A.- Luxembourg

3. FAILNOMORE Project

Partners:

3. FAILNOMORE Project

- The proposed design guidelines are reported in a Design Manual
- The Design Manual is freely available (through the ECCS website - https://www.steelconstruct.com/eu-projects/failnomore/) in 10 different languages (English, Portuguese, German, Italian, Romanian, Czech, Polish, Dutch, Spanish and French)

3. FAILNOMORE Project

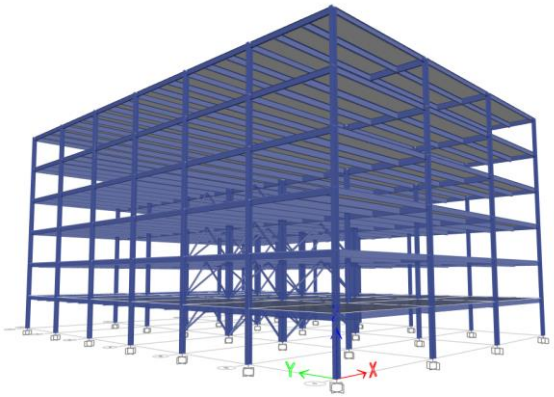
FAILNOMORE Design Manual – Table of contents

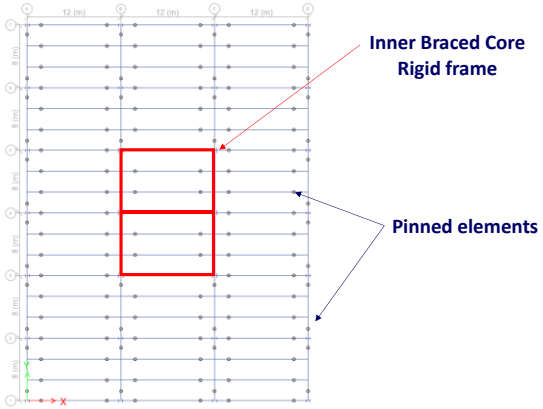
Part 1 – Design for robustness

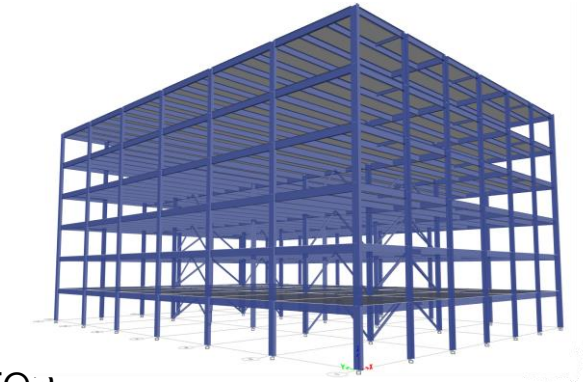
1. Normative context
2. Design for robustness
3. Consequence classes
4. Identified threats
5. Unidentified threats
6. Risk assessment
7. Conclusions

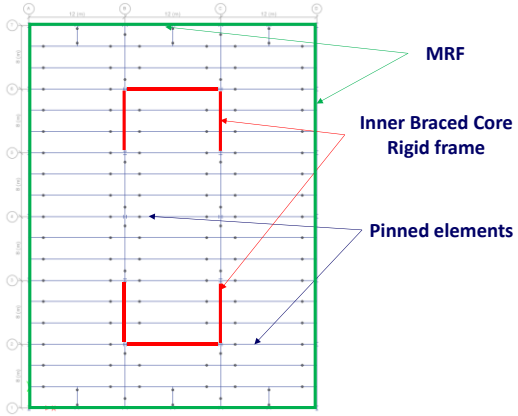
Part 2 – Worked examples

- A steel structure designed in non seismic area
- A steel structure designed in seismic area
- A composite structure designed in non seismic area
- A composite structure designed in seismic area









3. FAILNOMORE Project

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- A composite structure designed in seismic area

Part 3 – Annexes

Part 4 - References

3. FAILNOMORE Project

The presentations of the FAILNOMORE workshop will present the different design strategies and methods proposed in the Design Manual:

- **How to design for robustness under exceptional events**

  Jean-Pierre Jaspart, University of Liège

- **Design against identified threats**

  Florea Dinu, University of Timisoara

- **Design against unidentified threats**

  Jean-François Demonceau, University of Liège

- **Application of the practical design recommendations to worked examples**

  Tudor Golea, University of Liège

Design for Robustness

Brussels 10-05-2022

Děkuji! Dank je! Thank you! Merci! Dankeschön! Grazie! Dziękuję Ci! Obrigado! Mulțumesc! Gracias!

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steelconstruct.com/eu-projects/failnomore

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Design for robustness

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1. INTRODUCTION

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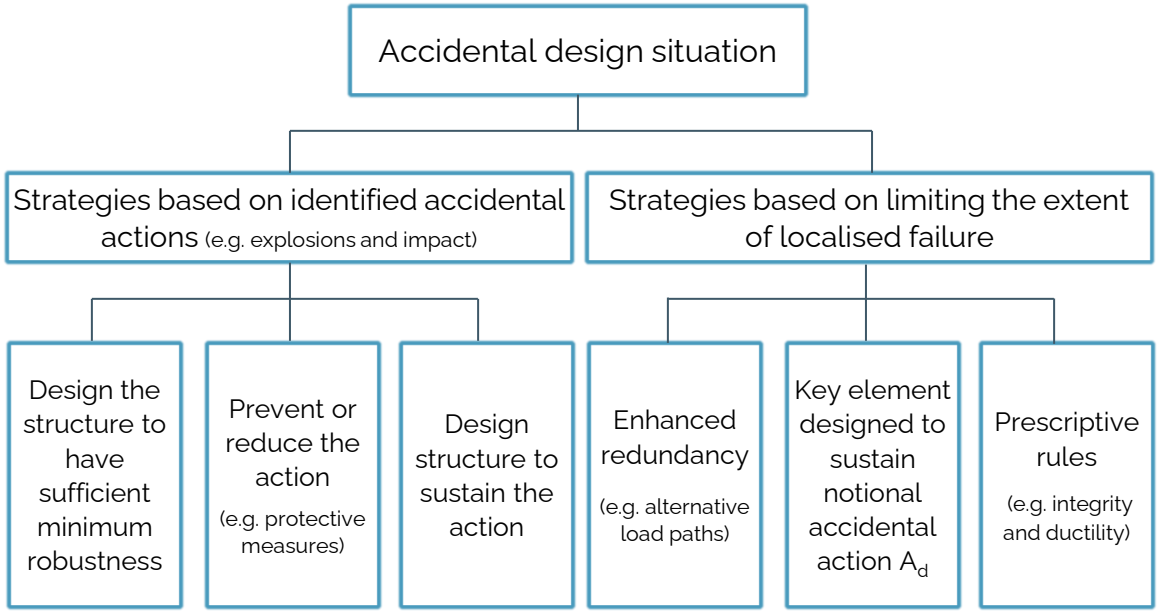
1. Introduction

Design strategies proposed in EN 1991-1-7



- **Identified weaknesses:**
  - No clear indications on how to select the design strategies to be applied
  - No consistent set of rules available

→ No clear guidance on how to <u>design for robustness</u>

→ The FAILNOMORE Design Manual aims to overcome these weaknesses by proposing, in its Chapter 2, a <u>general design philosophy</u> commonly agreed at European level

→ This general design philosophy is presented hereafter

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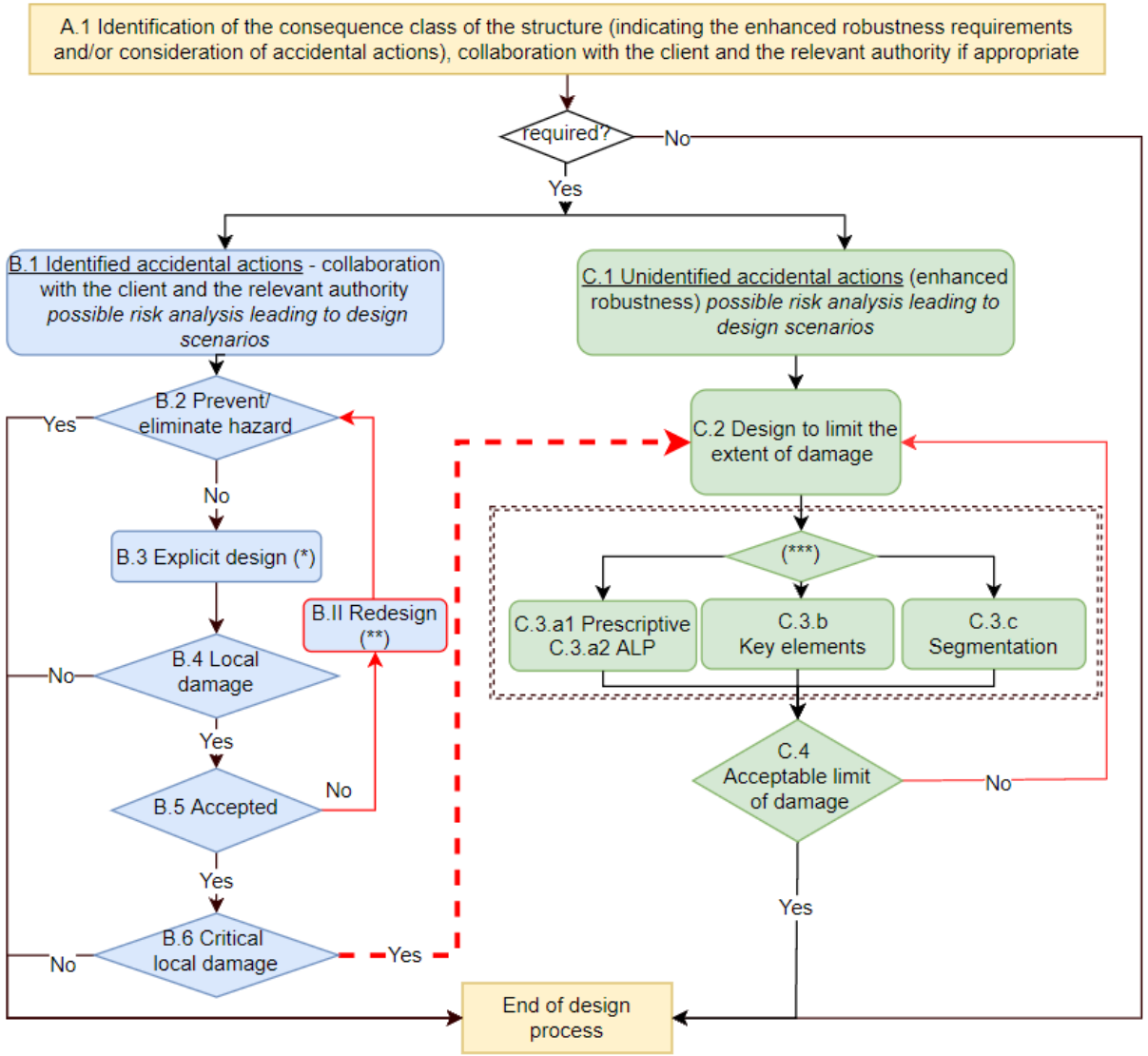
2. General design philosophy

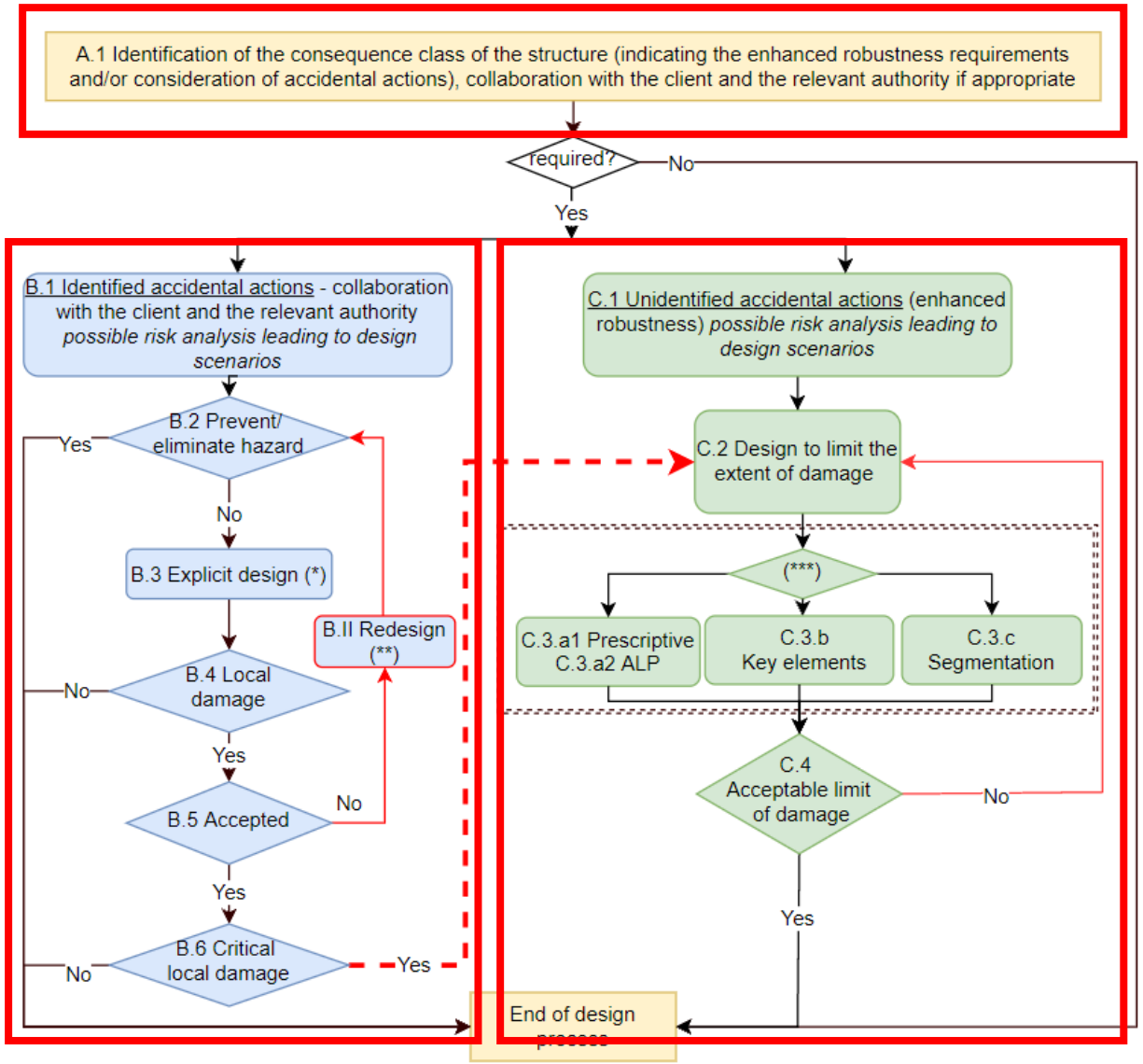
- The design for structural robustness is proposed as a step-by-step procedure presented in a general flowchart

- This flowchart is the backbone of the FAILNOMORE Design Manual



2. General design philosophy

- **This flowchart can be subdivided in three main parts:**
  A. Definition of the consequence class of the studied structure
  B. Design strategies for identified accidental actions
  C. Design strategies for unidentified accidental actions



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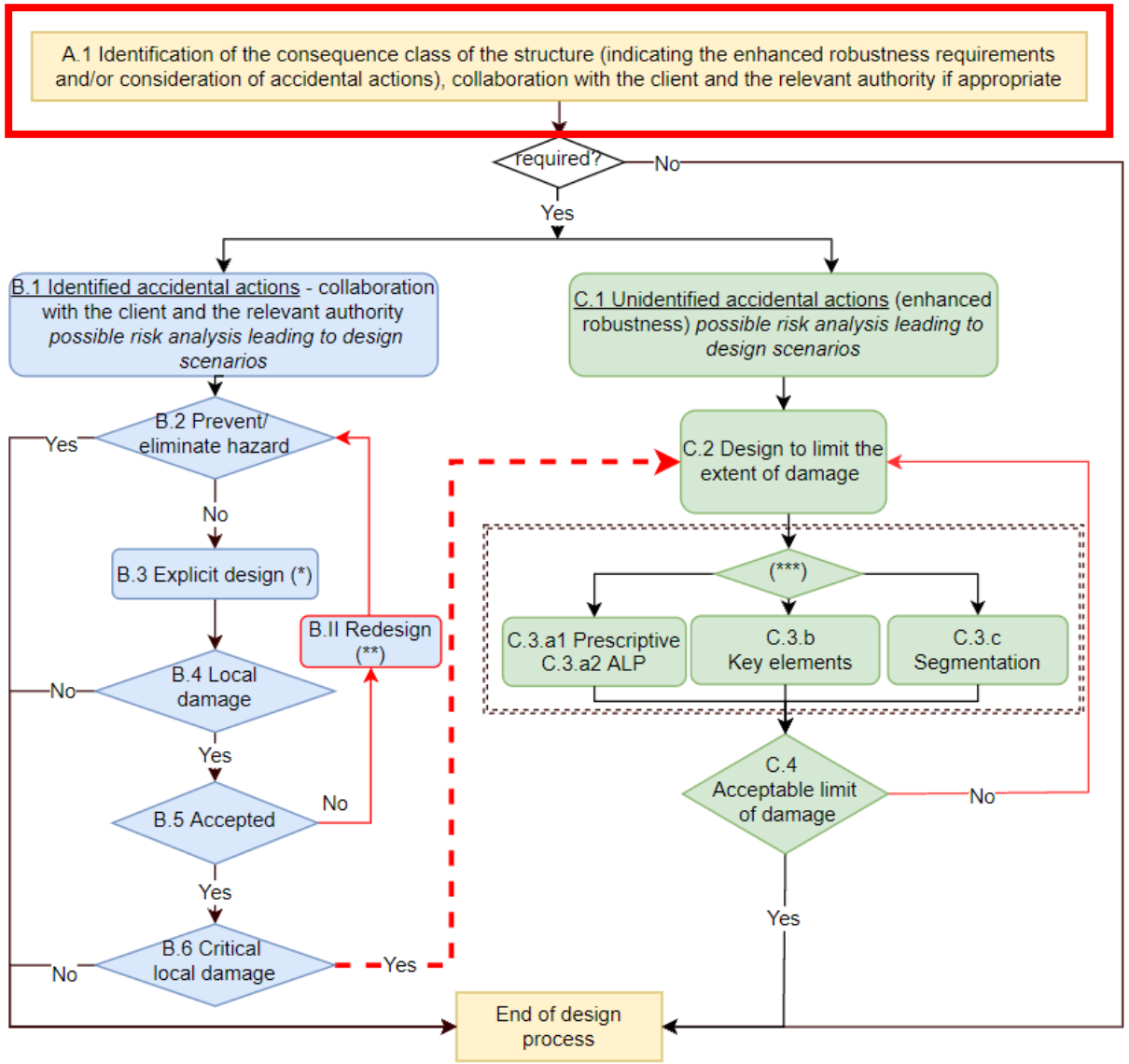

3. Consequence classes

- **Building structures are classified into "consequence classes" reflecting the consequences of structural failure in terms of:**
  - Loss of human live
  - Personal injury
  - Economic loss
  - Social loss
  - Environmental loss

- **This classification is a simplification of a complex risk-based system related to building type, height, occupancy, nature of materials, ...**

3. Consequence classes

- **In Annex A of EN 1991-1-7 and in EN 1990, three consequence classes are identified:**

Consequence class (CC)	Categorization of building type and occupancy
1	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)	5 storey single occupancy houses - Hotels, residential, offices ≤ 4 storeys - Industrial ≤ 3 storeys - Retailing premises ≤ 3 storeys and < than 1000 m² floor area in each storey - Single storey educational buildings - Buildings ≤ 2 storeys admitting public with floor areas ≤ 2000 m² 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 m2 but ≤ 5000 m2 at each storey - Car parking ≤ 6 storeys
3	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

CC1: Low consequence for loss of human life, **and small or negligible one** for economic, social or environmental aspects

CC2: Medium consequence for loss of human life **and considerable one** for economic, social or environmental aspects

CC3: High consequence for loss of human life, **or significant one** for economic, social or environmental aspects

3. Consequence classes

- In Annex A of EN 1991-1-7 and EN 1990, three consequence classes are identified:

Consequence class (CC)	Categorization of building type and occupancy
1	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)	5 storey single occupancy houses - Hotels, residential, offices ≤ 4 storeys - Industrial ≤ 3 storeys - Retailing premises ≤ 3 storeys and < than 1000 m² floor area in each storey - Single storey educational buildings - Buildings ≤ 2 storeys admitting public with floor areas ≤ 2000 m² 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 m2 but ≤ 5000 m2 at each storey - Car parking ≤ 6 storeys
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3. Consequence classes

- In Annex A of EN 1991-1-7 and EN 1990, three consequence classes are identified:

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3. Consequence classes

- Building structures are not all precisely covered by this table

  → Sound engineering judgment required

- Some additional guidelines are provided in Chapter 3 of the FAILNOMORE Design Manual

3. Consequence classes

- **The consequence class of the building allows the practitioner to assess the design approach to be adopted in view of achieving an adequate level of robustness**

- **Consequence Class 1 (CC1):**

  The design for robustness doesn't imply any specific considerations as long as the design is carried out in full compliance with the rules given in the series of Eurocodes

- **Consequence Classes 2 and 3 (CC2a, CC2b 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

3. Consequence classes

- **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**

- **The identification of the threats and of the relevant scenarios will enable the designer to adopt either:**
  - an explicit design for a specific identifiable accidental action and/or
  - a design strategy that limits the extent of initial damage as a consequence of any unidentifiable accidental event

- **In addition, for CC3, a systematic risk assessment is generally required**

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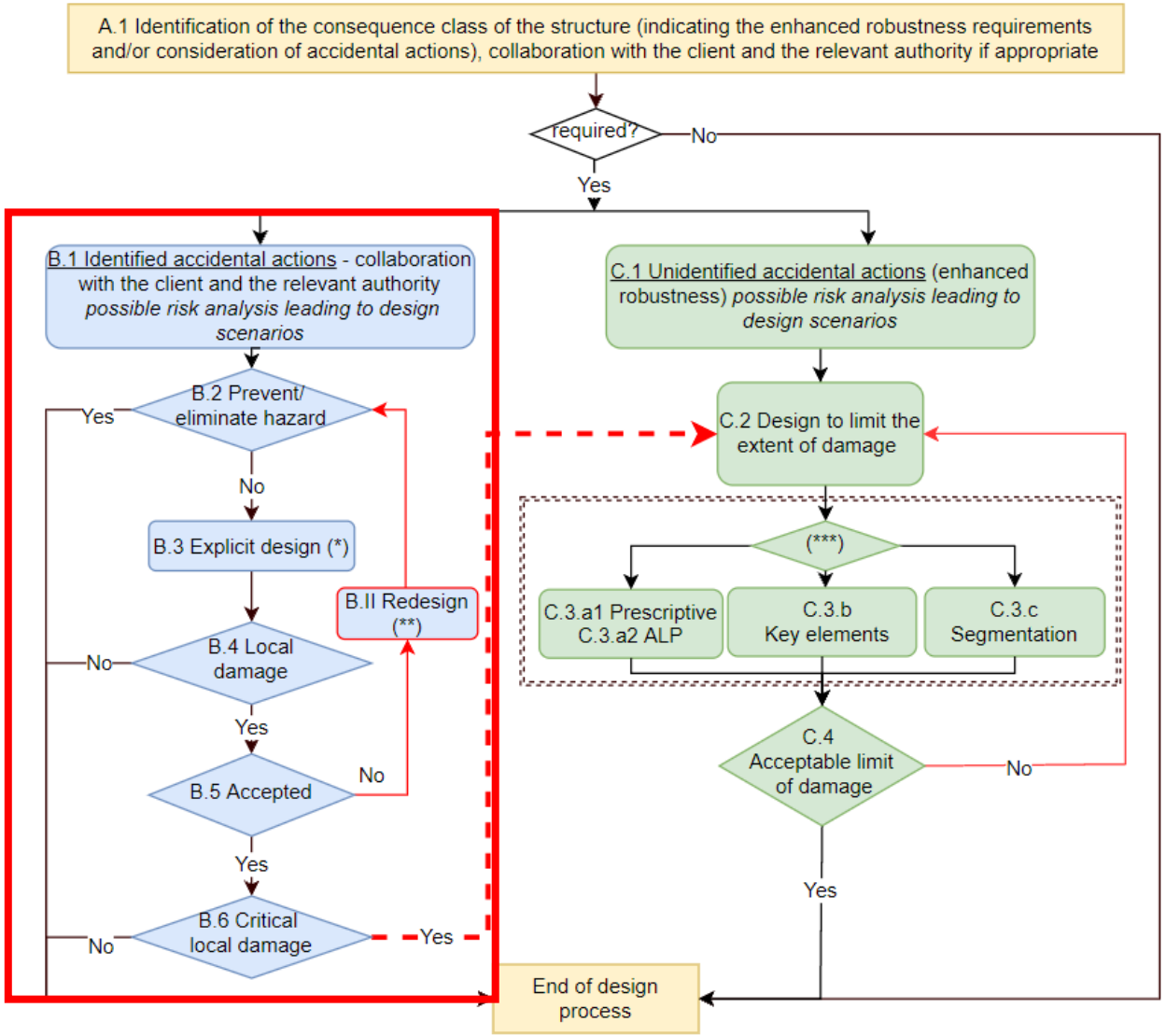
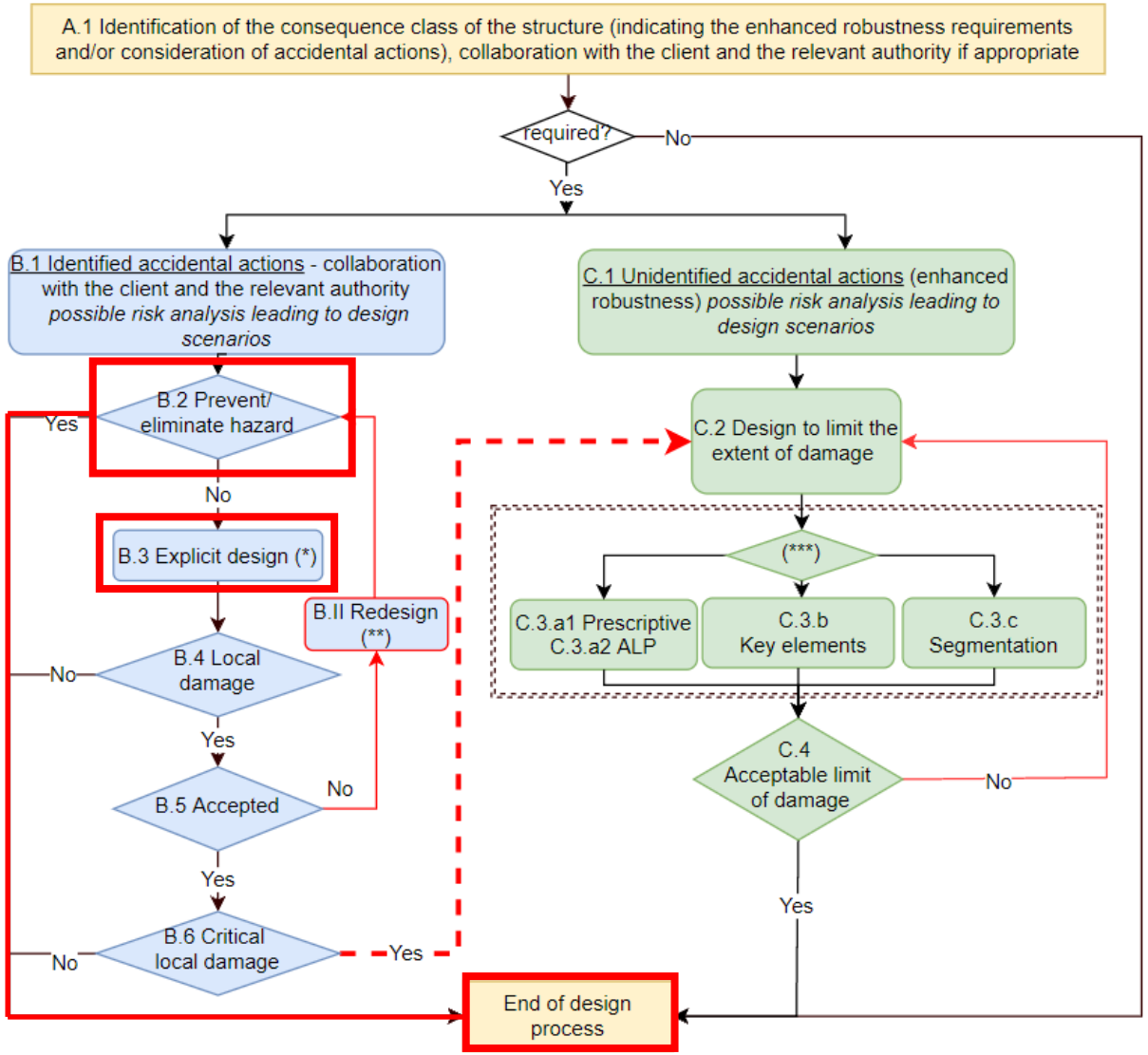


4. Design for identified acc. actions

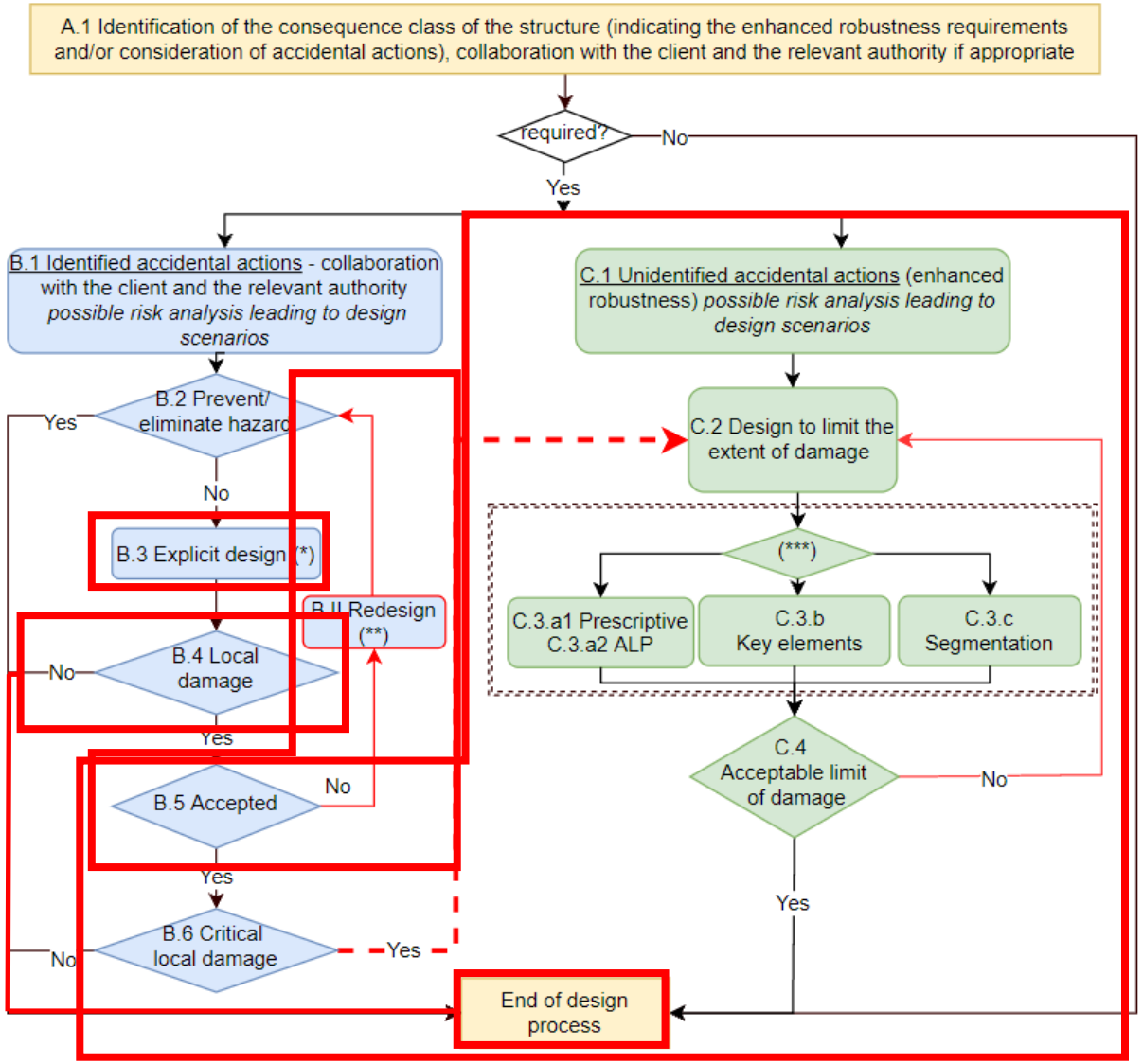
- **Check the possibility of using preventive and/or protective measures to reduce or mitigate the accidental actions**
  - If the action is fully prevented, it ends the design process for this specific threat
  - If the action is only reduced, an assessment of possible damages is required through an explicit design



4. Design for identified acc. actions

Explicit design for the considered accidental action

- If there is no damage, it ends the design process
- If there is a damage which is not acceptable ➔ a redesign of the structure is required
- If there is an acceptable damage, its extent should be prevented using appropriate design strategies as proposed for unidentifiable accidental actions



4. Design for identified acc. actions

- **The design for identified accidental actions may rely on analytical and/or numerical methods**

- **The level of sophistication of the methods is strongly linked to the consequence class:**
  - For CC2, the use of prescriptive methods or of simplified analysis considering static equivalent action models is possible
  - For CC3, the use of refined methods (dynamic analysis, non-linear models…) may be required

4. Design for identified acc. actions

- **Within Chapter 4 of the FAILNOMORE Design Manual, four accidental actions are considered:**
  - Impacts
  - Internal and external explosions
  - Fire as exceptional action
  - Earthquake as exceptional action

- **Different approaches with different levels of sophistication are proposed**

- **The latter will be presented in a specific presentation**

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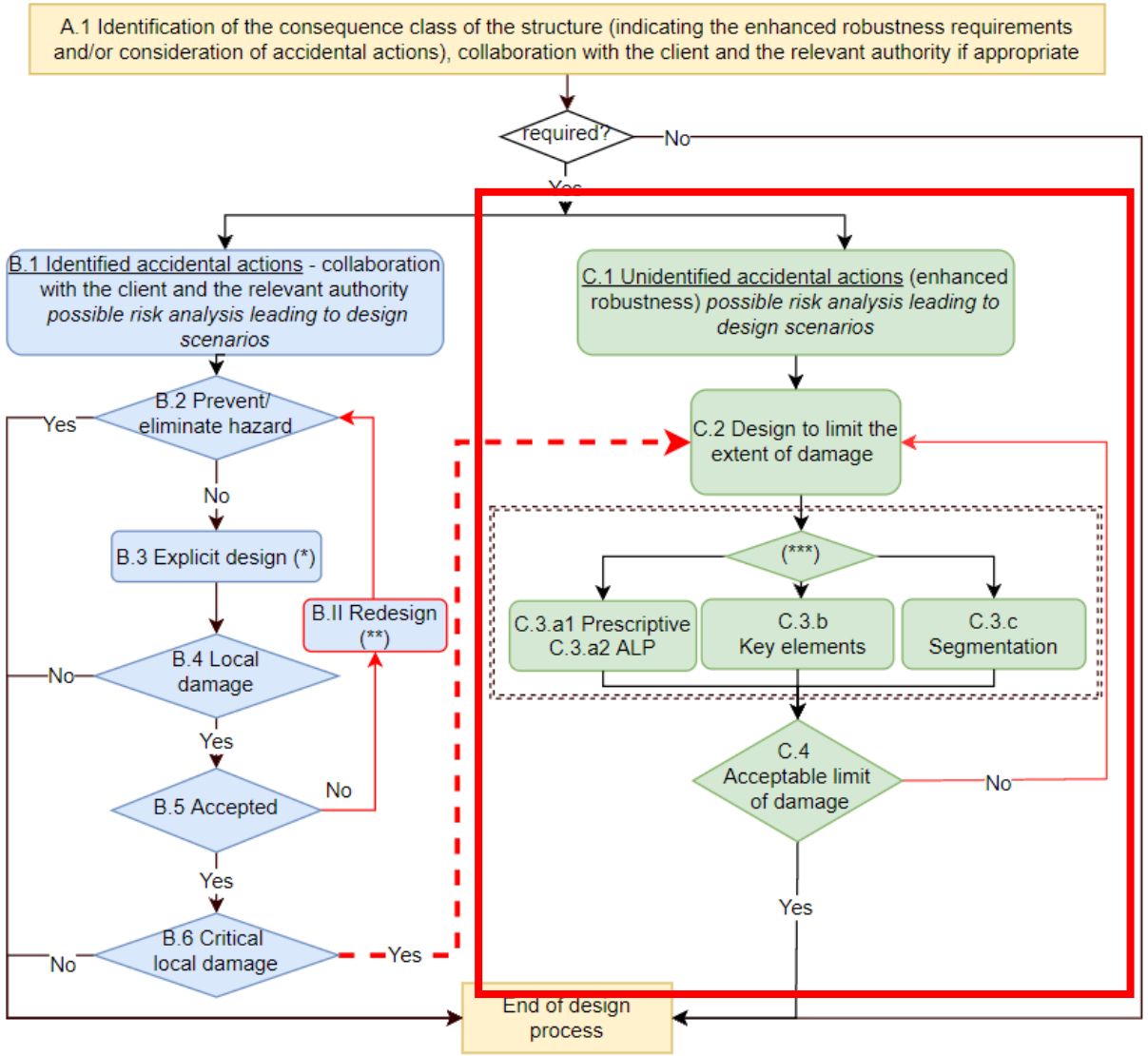


5. Design for unidentified acc. actions

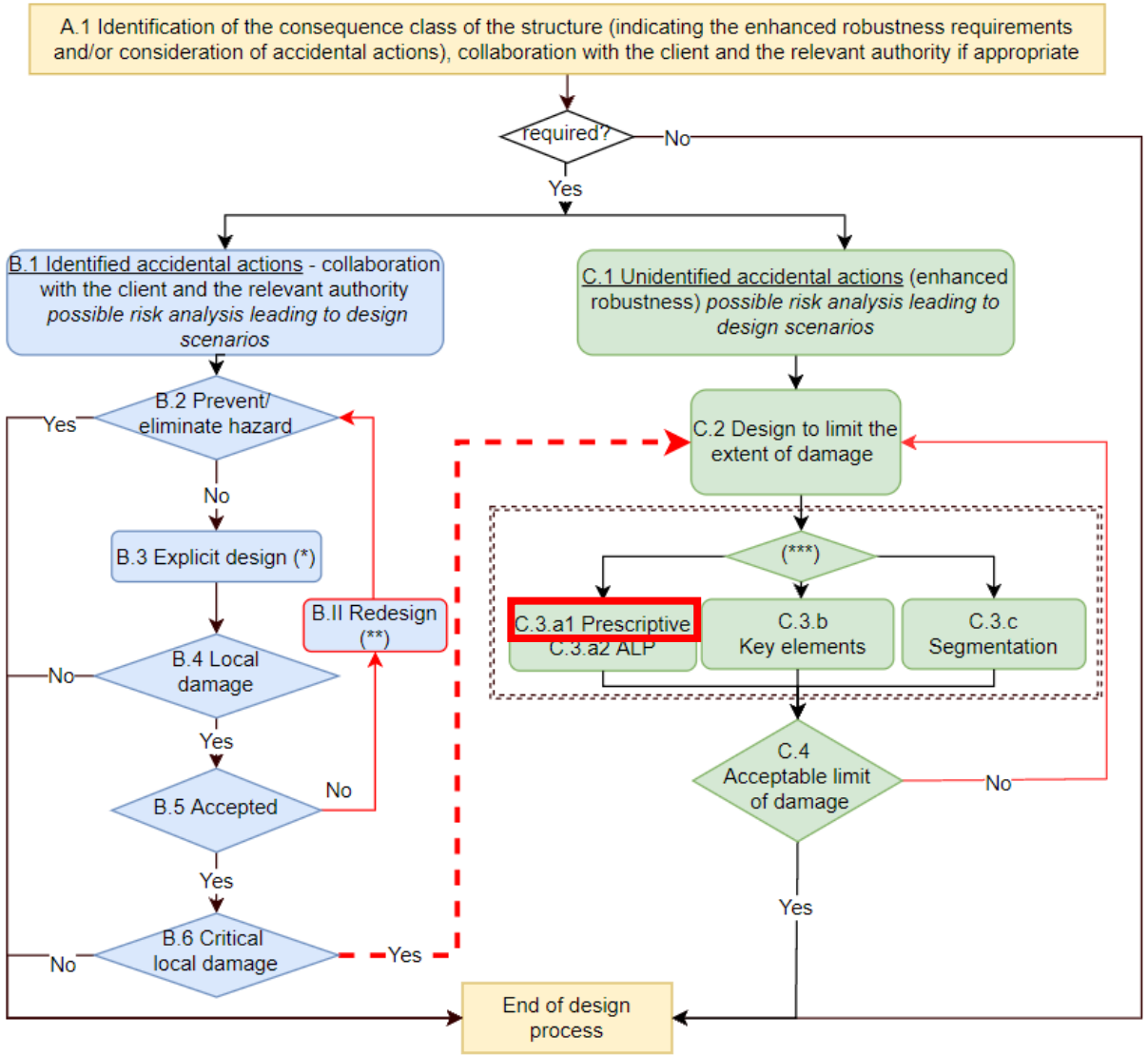
- **The design for unidentified accidental actions is founded on strategies aiming at limiting the extent of a localised damage, whatever is its initiating cause:**
  - The Alternative Load Path (ALP) method
  - The key element method
  - The segmentation method

- **The level of sophistication of the proposed methods will range from prescriptive methods to sophisticated full non-linear analyses**

- **The selection of the method to be applied is strongly linked to the consequences class**

5. Design for unidentified acc. actions

- For CC2a, EN 1991-1-7 suggests providing the structure with an efficient horizontal tying system using a prescriptive method

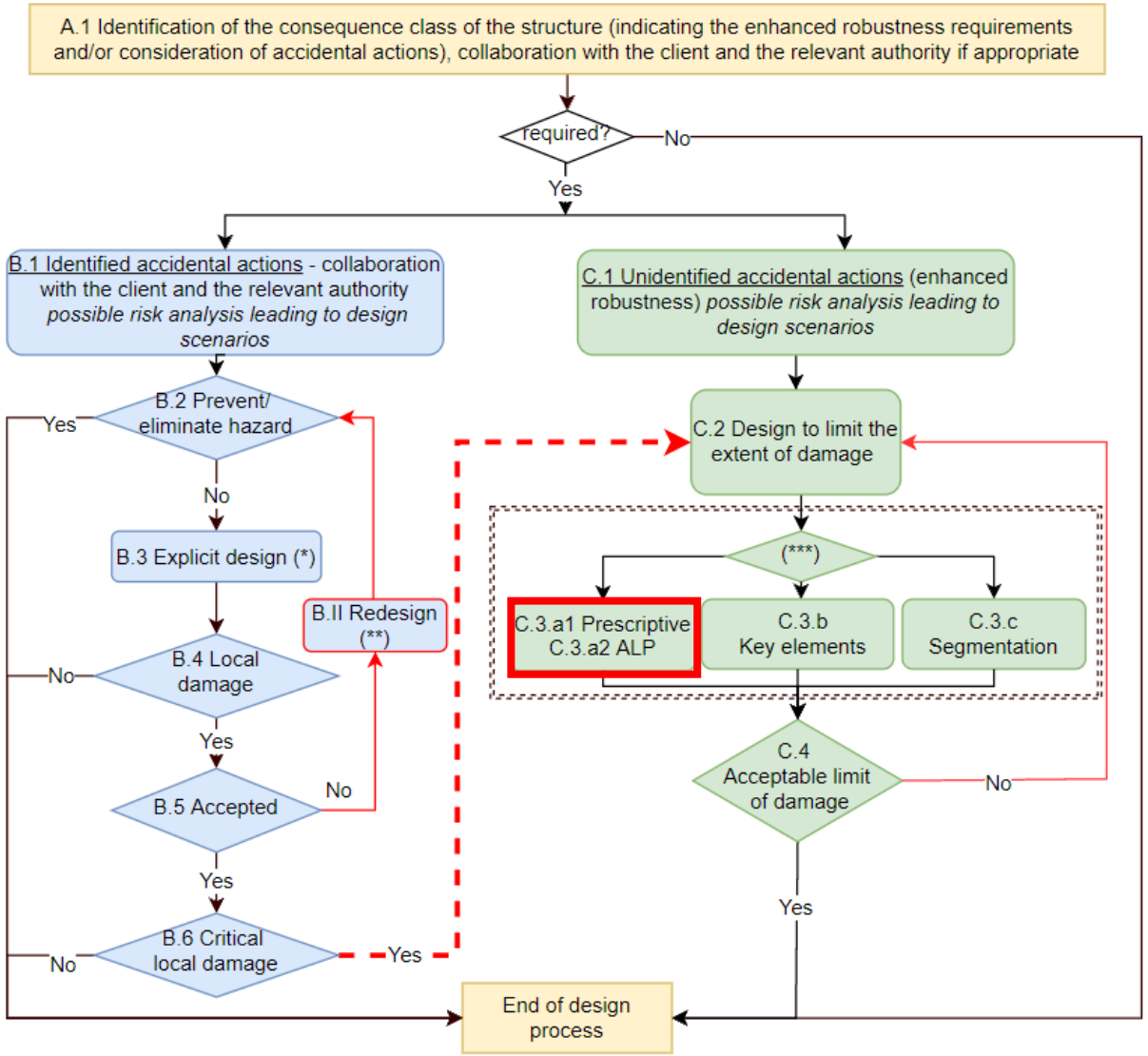


5. Design for unidentified acc. actions

- For CC2b, EN 1991-1-7 suggests providing the structure with an efficient <u>horizontal and vertical tying system</u> using a prescriptive method

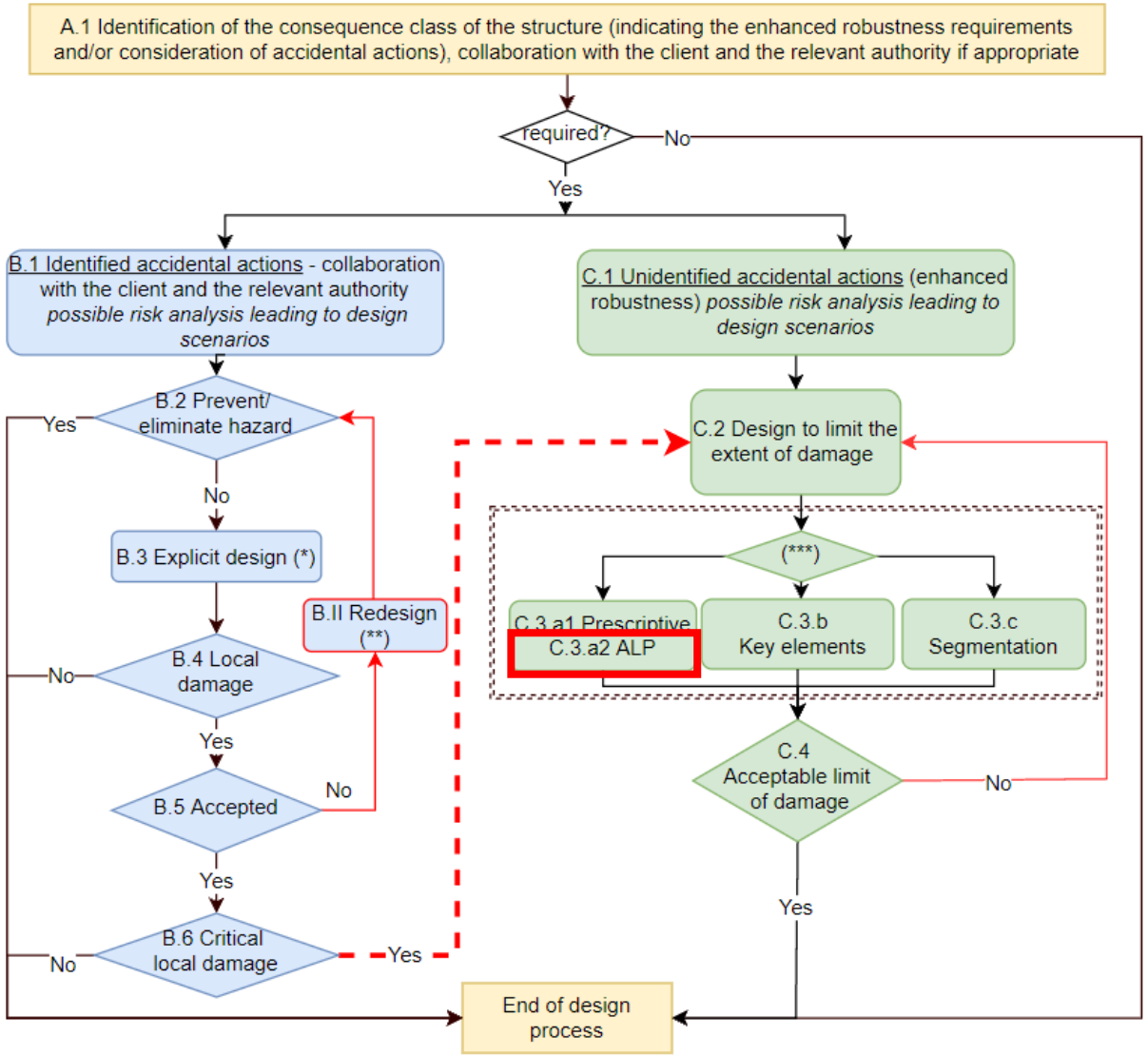
or

To consider the complete removal of supporting elements → <u>ALP method</u>



5. Design for unidentified acc. actions

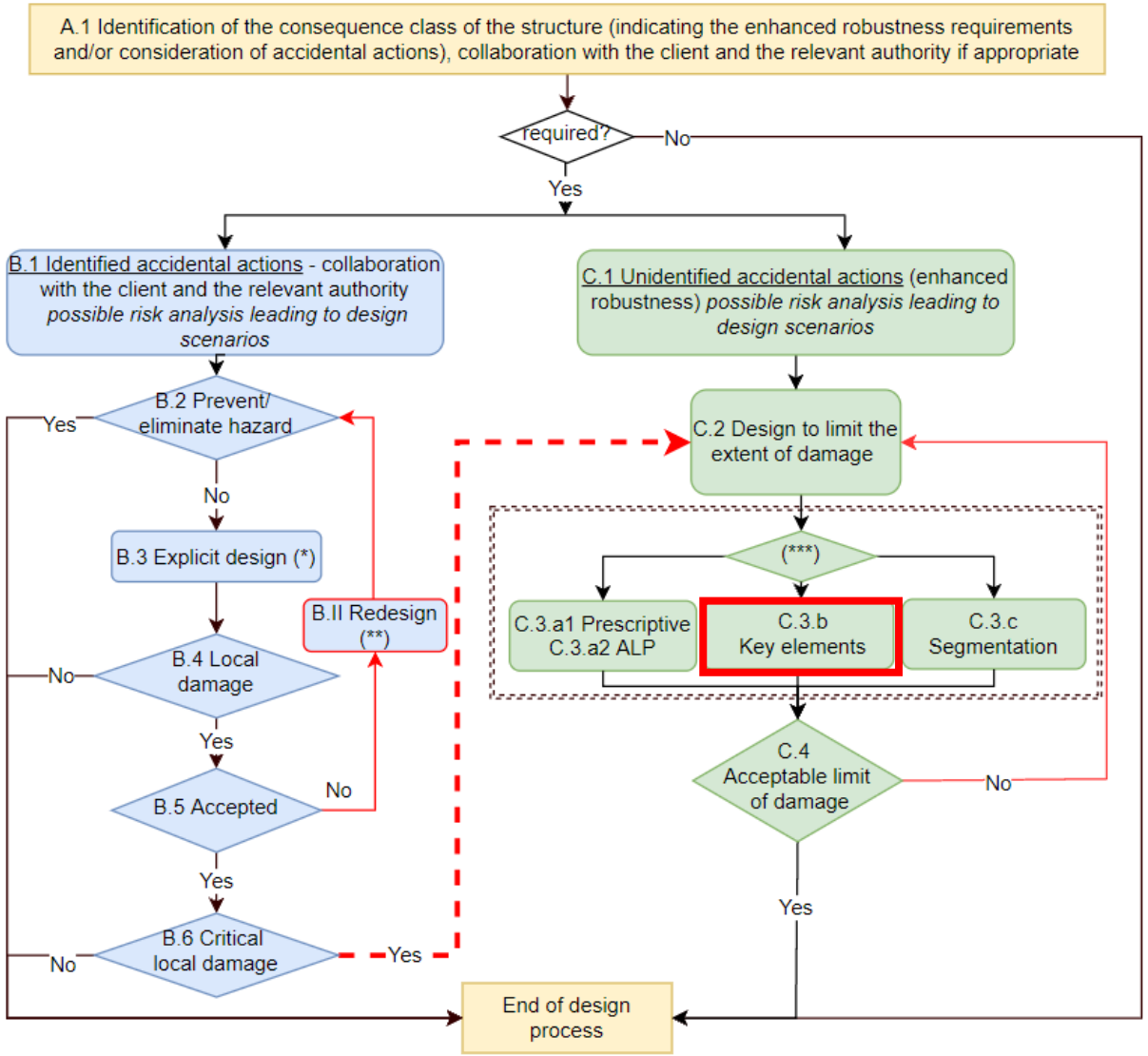
- For CC3, the use of the ALP method through refined approaches such as dynamic analyses is recommended



5. Design for unidentified acc. actions

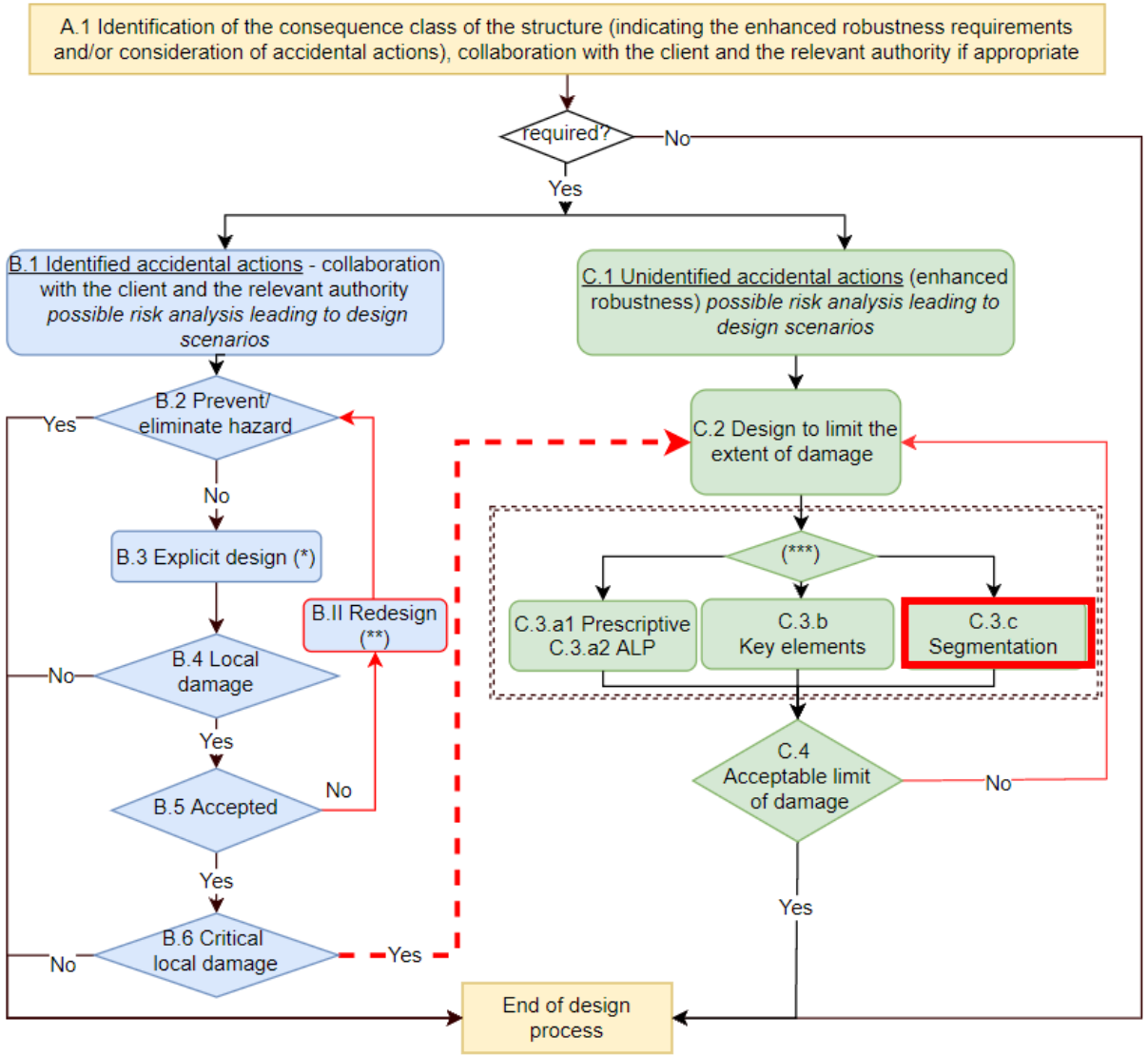
- Where the loss of a supporting member generates a disproportionate collapse, the removed element should be labelled as a "key element"

- The design should turn towards methods of local enhancement of resistance capacity of the key element considering a notional accidental action



5. Design for unidentified acc. actions

- An alternative to these methods is the use of <u>segmentation</u>
- Segmentation is a design strategy to prevent or limit an initial damage by <u>isolating the failing part</u> of a structure from the remaining structure
- Segmentation strategies can generally be based on either <u>weak segment</u> borders or <u>strong segment</u> borders



5. Design for unidentified acc. actions

- The different methods behind these design strategies are presented in Chapter 5 of the FAILNOMORE Design Manual

- They will be detailed in a specific presentation

1. Introduction
2. General design philosophy
3. Consequence classes
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5. Unidentified accidental actions
6. Structural joints
7. Conclusions

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This presentation is organised as follows:

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5. Design for unidentified accidental actions
6. Importance of structural joints:
   - 6.1 Minimum ductility requirements for joints
   - 6.2 Simplified method for endplate joints
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6. Importance of structural joints

- **Structural joints are important structural elements influencing the global response of a steel or composite building**

- **A joint can be classified in terms of stiffness, resistance and ductility:**

Stiffness	Resistance	Ductility
Nominally pinned	Nominally hinged	Brittle
Semi-rigid	Partial-strength	Ductile for plastic verification
Fully rigid	Full-strength	Ductile for plastic analysis

Covered in EN 1993-1-8

Not explicitly covered in EN 1993-1-8

6. Importance of structural joints

- **The component method is the analytical method recommended in EN 1993-1-8 and in EN 1994-1-1 for the characterisation of steel and composite joints in terms of stiffness, resistance and ductility**

- **The calculation of these joint properties is basically possible, whatever is the applied loading but:**
  - the codes are only providing precise application rules for joints subjected to bending moments while, under accidental/exceptional events, interactions between bending moments and axial forces may develop at the level of the joints
  - the joint loading sequence under accidental/exceptional events usually differ significantly from those considered at ULS

- **In the FAILNOMORE Design Manual, methods allowing for an accurate prediction of the joint properties when subjected to M-N interaction are provided in Annex A.1**
  These rules are not detailed in this presentation

6. Importance of structural joints

- **The use of rigid full-strength joints allows to neglect the effect of the joints on the internal forces distribution and on the design resistance of the system but does not usually represent the most economical option!**

- **However, if a plastic analysis is performed, 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 (if the joint ductility cannot be ensured)**
  → introduction of a new resistance classification: <u>OVER-STRENGTH joints</u>

Resistance
Nominaly hinged
Partial-strength
Full-strength
Over-strength

with $f_{ov} = 1{,}1 \times \gamma_{ov} \times f_y$

If a joint is not "over-strength", the joint behaviour will have to be accounted for in the design for robustness!

6. Importance of structural joints

- Under exceptional events, it is generally required to take advantage from the development of large deformations and from the ultimate resistance of the material with the objective of finding a new state of equilibrium in the deformed shape after an event

- So, ductility and large deformation capacity are seen as important properties to be provided to the structural joints

- Regardless the nature of the event or of the adopted design strategy, the preliminary design of all structural joints for ductility appears as a prerequisite (except for over-strength joints)

Ductility
Brittle
Ductile for plastic verification
Ductile for plastic analysis

To achieve it, **minimum ductility requirements** which should be all the time respected by the joints are provided in the FAILNOMORE Design Manual

6.1 Minimum ductility requirements for joints

- **The objective is to avoid the activation of brittle joint components at failure**

- **For pinned joints, requirements are expressed for the welds and for the bolt diameter**
  - For the welds, the use of full penetration or full-strength welds is recommended
    ➔ Design criteria to guarantee the full-strength character of welds are provided in the FAILNOMORE Design Manual
  - For bolts in tension, it is recommended to respect the design criterion provided in EN 1993-1-8 which links the diameter "d" of the bolt to the thickness "t" of the component in bending

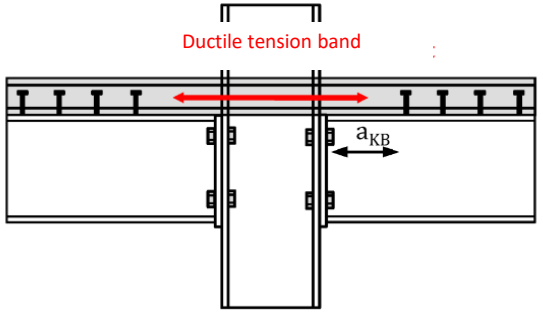
$$t \leq 0{,}36d\sqrt{f_{ub}/f_y}$$

This criterion guarantees the activation of a ductile failure mode at the level of the component in bending

  - Moreover, to allow a sufficient rotation, detailing requirements specific for pinned joints are also provided in the FAILNOMORE Design Manual

6.1 Minimum ductility requirements for joints

- **For partial-strength joints :**
  - the use of full-penetration or full-strength welds is also recommended
  - If a component in bending is activated at failure, it is also recommended to respect the design criterion provided in EN 1993-1-8 which links the diameter “d” of the bolt to the thickness “t” of the component in bending
  - Failure modes “column web in transverse compression” and “beam flange and web in compression” should be avoided as they involve local instability phenomena
- **Some specific recommendations are also provided for partial-strength composite joints regarding:**
  - The properties of the rebars to be used in the vicinity of the joints
  - The positioning of the first shear stud to be kept at a certain distance from the column



6.1 Minimum ductility requirements for joints

- **For full-strength joints :**
  - The same recommendations as for partial-strength joints apply as the connected members may exhibit an overstrength which could lead to the activation of the joints at failure

- **For over-strength joints :**
  - There is no specific requirement

- **As stated in the title, these propositions have to be seen as minimum ductility requirements**
  Addition requirements, more specific to the adopted design strategy, may also have to be respected

6.2 Simplified method for endplate joints

- **As alternative to the previously presented criteria, the simplified method proposed by Rölle can be used**

- **Respecting some constructive criteria, this method:**
  - Allows for an easy characterisation of joint
  - Guarantees the ductility with a total joint rotations > 80 mrad
  - Validated by experiments

- **Applicable for:**
  - Flush and extended endplate all-steel joints
  - Steel-concrete composite joints - 2 bolt rows

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7. Conclusions

- In this presentation, the general philosophy to design for robustness as proposed in the FAILNOMORE Design Manual has been presented

- A specific attention has been paid to the joints identified as crucial elements when designing for robustness
  In particular, minimum ductility requirements to be respected have been identified

- In the next presentations, the different design methods which can be applied in the framework of the adopted general design philosophy will be detailed and applied to worked examples

Design for Robustness

Brussels 10-05-2022

Děkuji! Dank je! Thank you! Merci! Dankeschön! Grazie! Dziękuję Ci! Obrigado! Mulțumesc! Gracias!

JASPART Jean-Pierre

Jean-pierre.jaspart@uliege.be

steelconstruct.com/eu-projects/failnomore

Research Fund for Coal & Steel

FAIL NO MORE

LIÈGE université

Bruxelles 10.05.2022

IDENTIFIED THREATS

Florea Dinu

Politehnica University Timisoara

FAILNOMORE

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

Research Fund for Coal & Steel

FAIL NO MORE

Universitatea Politehnica Timișoara

1. Introduction
2. Impact
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4. Fire as exceptional event
5. Earthquake as exceptional event
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1. INTRODUCTION

- **This section describes mitigation strategies and design approaches for five types of identified accidental actions**
- **The presentation is organized as follows:**
  1. Introduction
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     - 2.2 Simplified dynamic approach
     - 2.3 Full dynamic approach
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     - 3.1 External explosion
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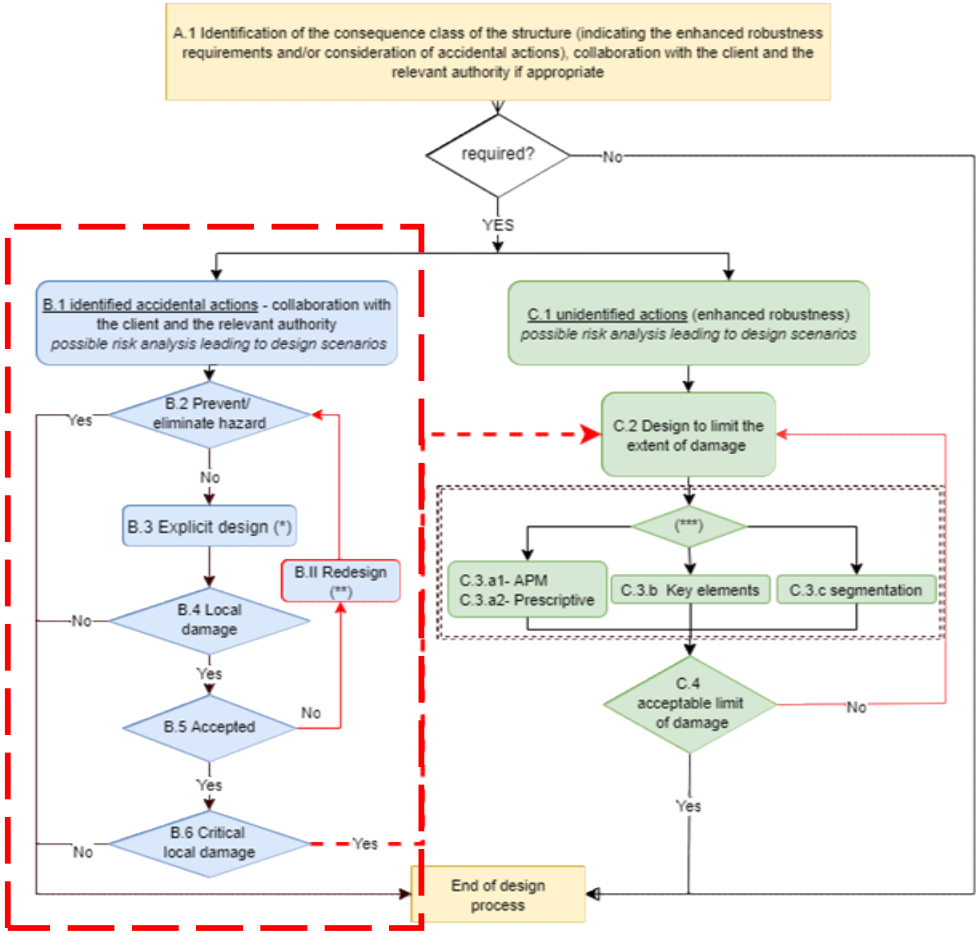
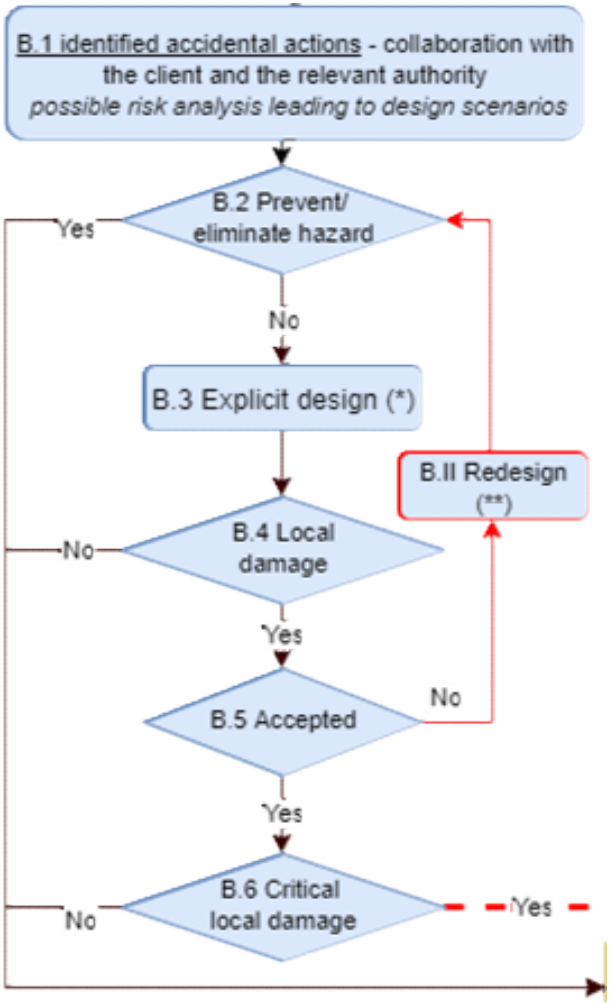
1. INTRODUCTION





1. INTRODUCTION

- The design for robustness can be done considering the direct effects of the extreme action **(identified actions)**
- When the result of the design is not acceptable, the structure needs to be redesigned/**strengthened** (hazard mitigation/more advanced methods if appropriate / unidentified actions, see Section C)
- The design should be done considering appropriate methods of analysis, which depends on the safety category, or consequence class (CC)

Consequences class 1	no specific consideration
Consequences class 2	simplified analysis and/or applying prescriptive design/detailing rules
Consequences class 3	dynamic analysis / non-linear analysis if appropriate

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2. IMPACT

- **Accidental action occurring in a very short time (usually fractions of a second)**
- **Action with low probability of occurrence but high consequences**
- **For buildings, car impact is more common:**
  - Buildings near the roadways
  - Car park buildings or buildings with nearby parking
  - Buildings where cars and trucks are allowed

2. IMPACT

Reduce/prevent the action

- **Prevention measures are focused on reducing the velocity of the impact object (e.g. vehicle) and/or reducing its accessibility to the building**
- **The most common measures are:**
  - Proper planning of the access roads
  - Acces control
  - Use of barriers (permanent or automatic)

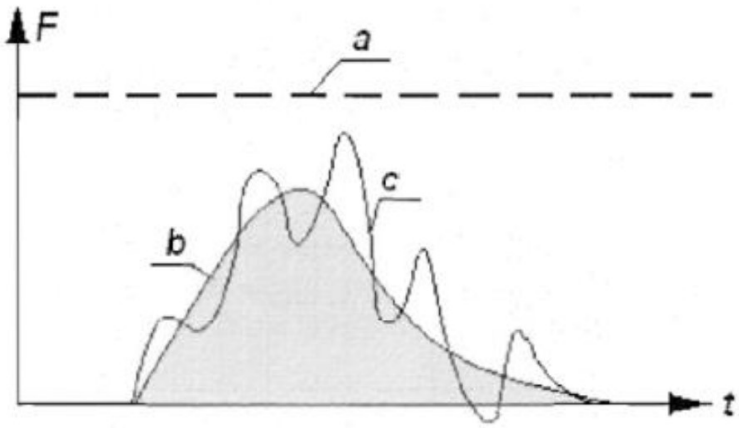
Permanent barriers

Mobile barriers

2. IMPACT

Design strategies

- ***"Impact is an interaction phenomenon between a moving object and a structure, in which the kinetic energy of the object is suddenly transformed into energy of deformation"*** **EN 1991-1-7**



Key :
a : equivalent static force
b : dynamic force
c : structural response

Impact force vs. time (EN 1991-1-7:2006)

- **Hard impact** (or rigid impact) – The structure is consider rigid, and the impact object dissipates the energy introduced by the collision **(conservative assumption)**
- **Soft impact** – The structure deforms to dissipate the energy introduced by the collision

Types of analysis

- Equivalent static approach (Hard impact)
- Simplified dynamic approach (Hard or soft impact)
- Full dynamic approach (Hard or soft impact)

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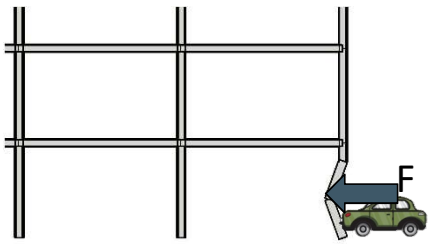
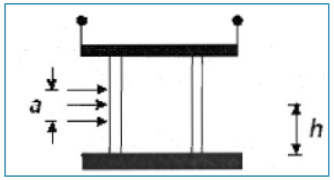
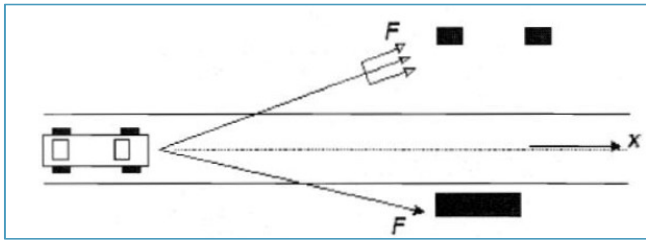
2.1 Equivalent static approach

- **In this procedure, the action is represented by an equivalent static force**
- **The approach can be applied to hard impact and structures belonging to CC1 and CC2 structures**
- **Allows verifying the static equilibrium, resistance, and deformations of the structure**







a *- the height of the recommended force application area - ranges from 0,25 m (cars) to 0,50 m (lor*
h *- the location of the resulting collision force F, i.e., the height above the level of the carriageway - ranges from 0,50 m (cars) to 1,50 m (lorries)*
x *- is the centre of the lane*

Category of traffic	Force F_{dx} [a] [kN]	Force F_{dy} [a] [kN]
Motorways and country national and main roads	1000	500
Country roads in rural area	750	375
Roads in urban area	500	250
Courtyards and parking garages with access to:		
- Cars	50	25
- Lorries [b]	150	75

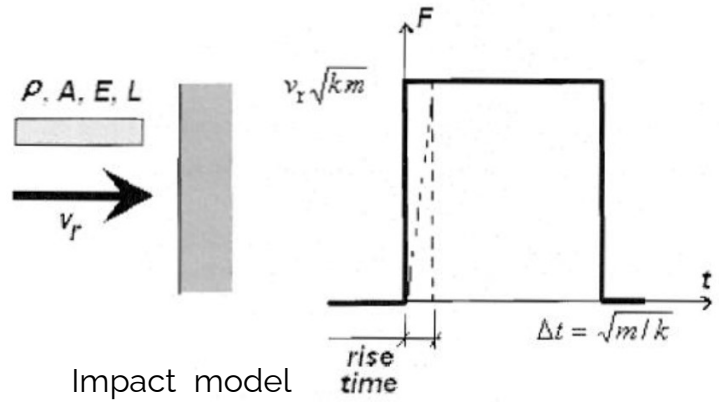
[a] x = direction of normal travel, y = perpendicular to the direction of normal travel.

[b] The term "lorry" refers to vehicles with maximum gross weight greater than 3,5 tonnes.

Collision force on supporting substructures near traffic lanes for bridges and supporting structures for buildings (EN 1991-1-7:2006)

2.2 Simplified dynamic approach

- **This procedure allows two different definitions of action (Annex C of EN 1991-1-7):**
  - Hard impact → for structures up to CC2b
  - Soft impact → for structures up to CC3



Impact model

Hard impact

Maximum force at the exterior face of the structure (rectangular impulse):

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

v_r – collision velocity

k, m – stiffness and mass of the impact object

- Dynamic load factor (DLF) can be used to amplify the force due to dynamic effects:
  - Step function (rise time = 0 sec): **DLF= 2.0**
  - Pulse load: **DLF** needs to be quantified (**DLF = 1.0** up to **1.8**)

Soft impact

A. Structure resists through elastic deformations

- See hard impact, but with k being the stiffness of the structure

B. Structure resists through plastic deformations

- The structure should be ductile enough to absorb the total kinetic energy of the impact object:

$$E = {}^{1}/_{2} \cdot m \cdot v_r^2$$

- If the response is plastic-rigid

$$^{1}/_{2} \cdot m \cdot v_r^2 \leq F_o y_0$$

F_o – Plastic resistance of the structure

y_o – Deformation capacity

2.3 Full dynamic approach

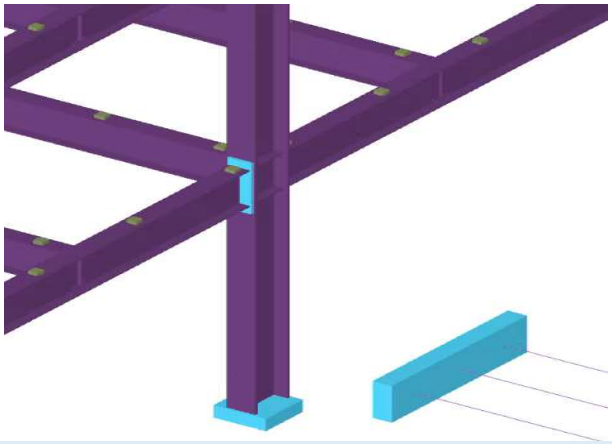
Two approaches can be employed

The impact load explicitly modelled:

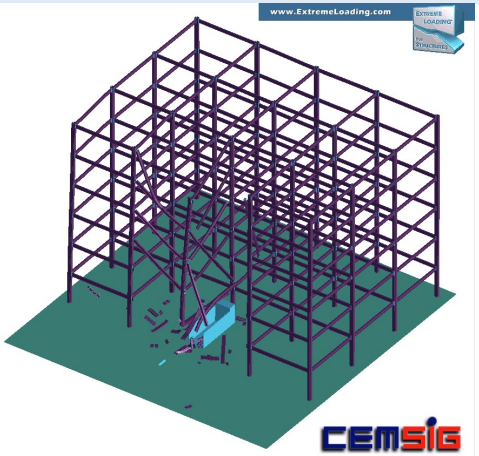
- Most realistic approach
- The numerical model includes modelling of the impact body (mass, stiffness), impact loading (velocity, direction, duration) structural response (stresses, strains, deflections), and structure - object interactions
- Effect of strain rate on material: through DIF factors

Alternate load path analysis

- Column is considered removed due to impact (see unidentified threat section)
- More practical than previous approach, still allows a good estimation of the structure robustness



The impact load explicitly modelled (analyses done using ELS)



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3. EXPLOSIONS

- **Explosion - an extremely rapid release of energy in the form of pressure wave, heat, sound, and light**
- **Explosive materials can be solids, gases, vapours, or dust**
- **Deflagration vs detonation:**
  - Depending on the nature of the explosive material and the local conditions, the explosion may develop as a deflagration or as a detonation
  - Deflagration - propagation of a combustion zone at a velocity that is less than the speed of sound in the unreacted medium (e.g., internal gas explosion in buildings)
  - Detonation - propagation of a combustion zone at a velocity that is greater than the speed of sound in the unreacted medium (e.g., industrial explosives)

1 – incident wave
2 – ground reflected wave
3 – structure reflected wave

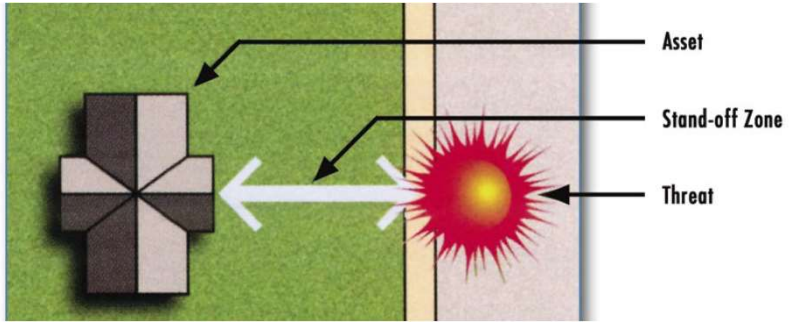
Detonation of a high explosive charge near a building (SAFE-WALL, 2021)

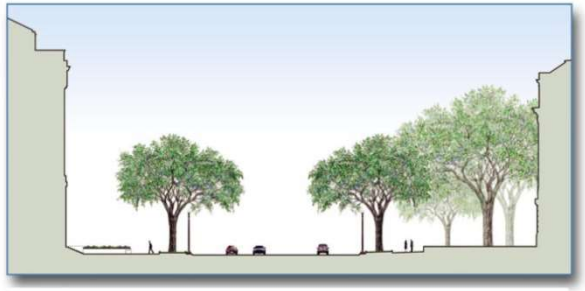
Detonation of a high explosive charge near a building (FRAMEBLAST, 2018)

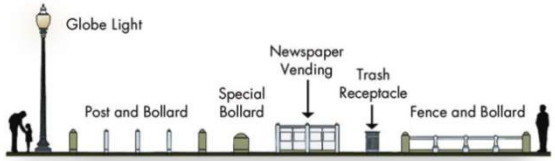
Detonation of a high explosive charge near a building (FRAMEBLAST, 2018)

3. EXPLOSIONS

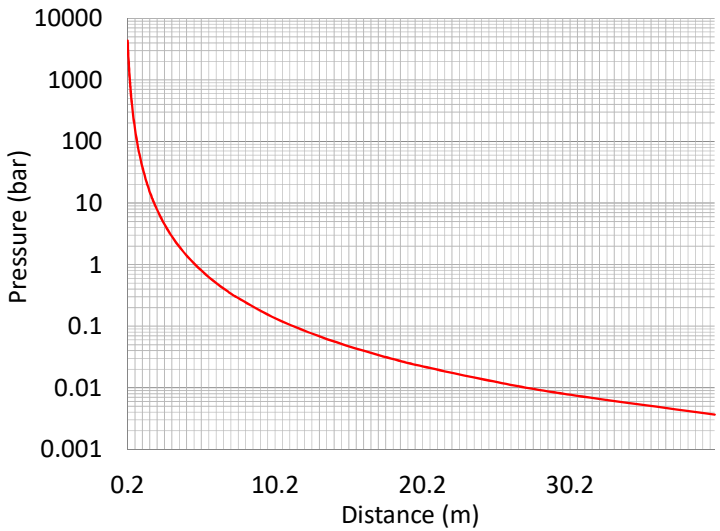
Prevent/eliminate hazard - External explosion







- The blast pressure reduces significantly with increase of the distance ➔ maximizing stand-off distance will decrease the effects of a blast
- Public spaces: bollards, trees, street furniture can be used as obstacles
- Higher risk area: a blast resistant wall (barricade that protects the structure - keeps the energy from reaching the structure)
- Avoidance of exterior attached non-structural elements → limits flying debris, emergency exits remain in operation
- Windows can cause severe injuries: appropriate type of glazing, reduced windows area
- Structural shapes and dimensions: **edge section** ➔ long rectangular shapes induces less peak reflected pressure than square shapes; **parabolic** or **cubic** shaped facades perform better than an upright faced facade



Peak pressure vs. stand-off distance, CODEC 2016

Stand-off distance < 2.0 m
Equivalent TNT weight ~ 1800 kg

Alfred P. Murrah Building,Oklahoma City, 1995

3. EXPLOSIONS

Prevent/eliminate hazard - Internal gas explosion

- **Take into consideration the gas explosion hazard from the beginning of the project:**
  - separation of areas
  - overall layout
- **Strong frame structures supporting roof and intermediate floors: if a solid wall is needed, use low weight wall panels to facilitate early explosion venting**
- **Vent areas:**
  - size and location ➔ when sufficient venting is close to the ignition point, the flame speed will be low, and the turbulence generated behind the obstacles will be limited
  - generally, the gas explosion venting should be directed into open areas with minimum of obstructions
  - partial obstruction of a vent opening can result in strong pressure increases

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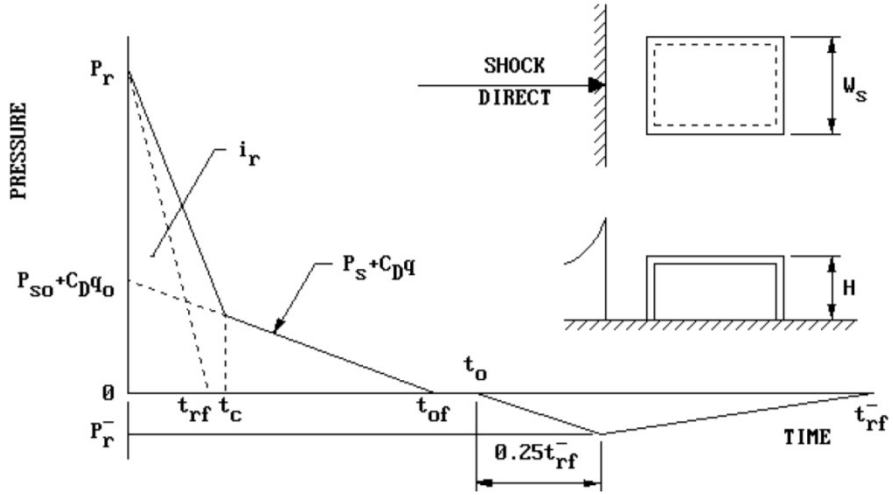
3.1 External explosion

EXPLICIT DESIGN

Scenario definition

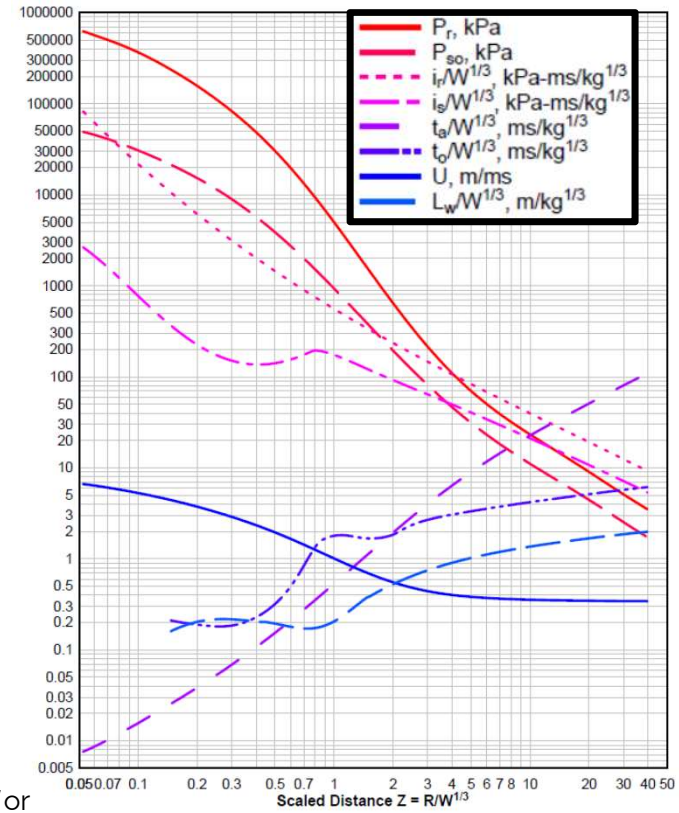
- W – explosive charge weight
- R –distance to the building



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

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

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



Pressure-impulse chart (free-air bursts)

- 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_o 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;
- i_r is the total reflected impulse; P_r is the peak reflected pressure.

3.1 External explosion

Equivalent single-degree-of-freedom approach

- The element loaded by blast is first transformed into an equivalent single degree of freedom SDOF
- The mass distribution, boundary conditions, resistance function, and load history are idealized:

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

$$K_L = \frac{F_E}{F(t)} \qquad K_m = \frac{m_E}{m}$$

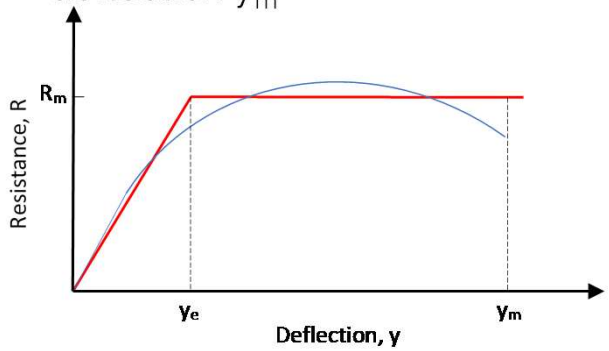
T_n	period of the SDOF system
m_E	mass of the equivalent system
k_E	equivalent spring constant
K_L	load factor
F_E	equivalent load
F	actual total load on the structure
K_M	mass factor
m	total mass of the actual element

Elastic SDOF systems

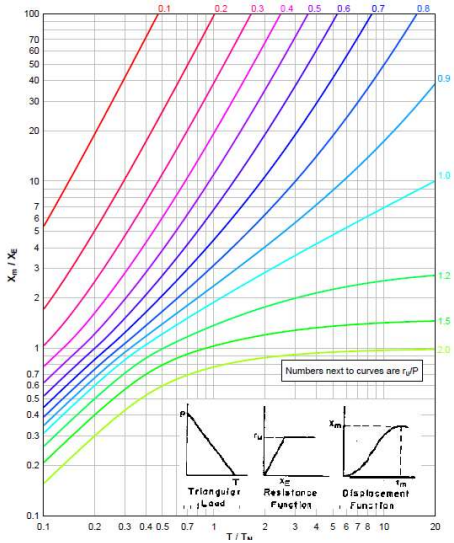
- The results can be obtained using diagrams
- y_{max} is the maximum dynamic deflection
- y_{st} is the deflection resulted from the static application of the peak load $_{Fm}$

Elasto-plastic SDOF systems

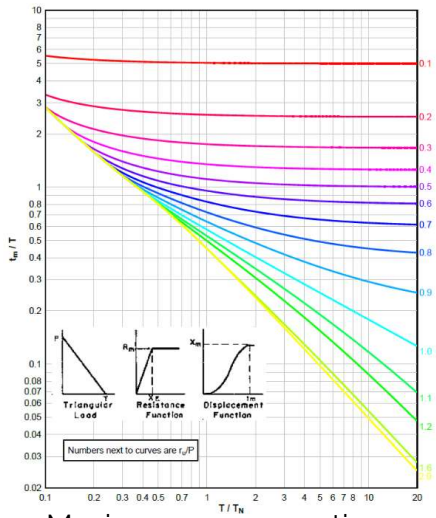
- The response of the SDOF system with elasto-plastic response is idealised in terms of its ultimate resistance R_m, and maximum deflection y_m



Resistance – deflection curve



Maximum deflection



Maximum response time

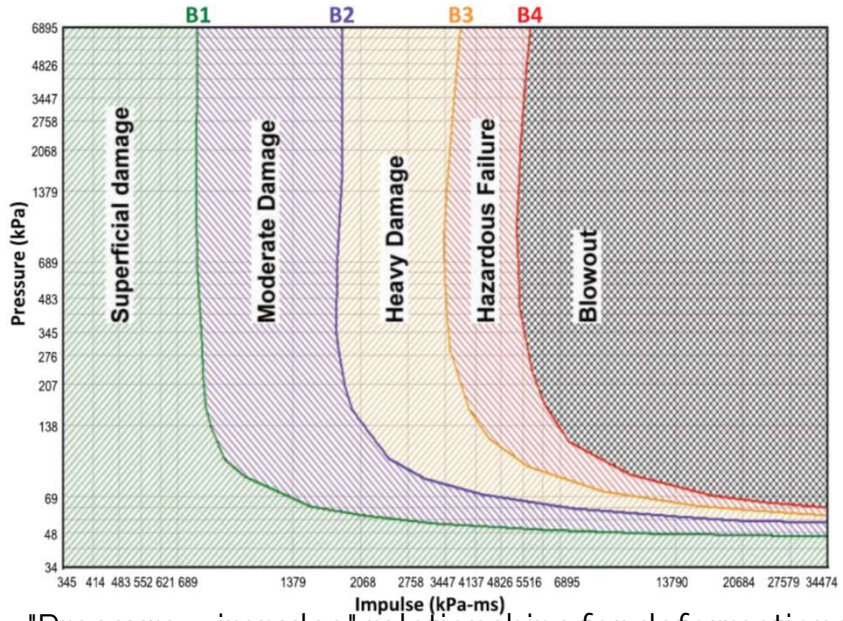
3.1 External explosion

Pressure-impulse diagrams method

- Step 1: the load shape is defined. This should be consistent with the explosion threat
- Step 2: SDOF analysis (or other approaches) are used to determine the response of the component in the form of end rotation, θ, and ductility factor, μ

  μ is 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

- Step 3: the response is compared with the system limits (available for entire buildings, individual structural members, windows)
- Step 4: based on the damage level determined in previous step, the level of protection (consequences class) is provided by comparing the results with the acceptable limits



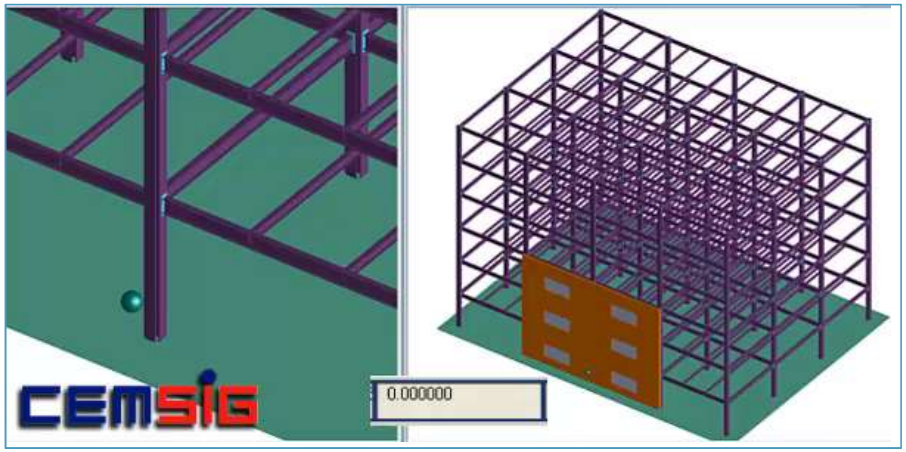
"Pressure – impulse" relationships for deformations corresponding to damage limits

Element type		B1		B2		B3		B4	
		m_{max}	q_{max}	m_{max}	q_{max}	m_{max}	q_{max}	m_{max}	q_{max}
Flexure	Beam with ductile cross-section	1	-	3	3°	12	10°	25	20°
	Beam with limited ductility cross-section	0.7	-	0.85	3°	1	-	1.2	-
	Plate bent about weak axis	4	1°	8	2°	20	6°	40	12°
Compr.	Beam-column with ductile cross-section	1	-	3	3°	3	3°	3	3°
	Beam-column with limited ductility cross-section	0.7	-	0.85	3°	0.85	3°	0.85	3°
	Column (axial failure)	0.9	-	1.3	-	2	-	3	-

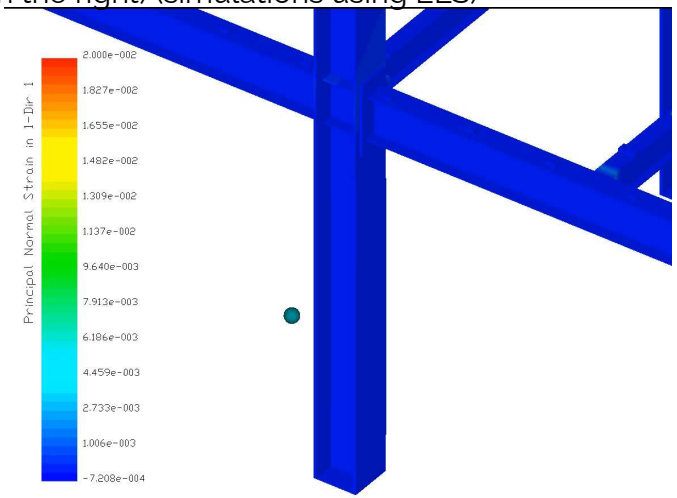
3.1 External explosion

FULL DYNAMIC APPROACH

- **Blast load** and structural behaviour (internal forces, stresses, strains, deflections) are function of time, thus requiring appropriate definition and modelling
- **Material Models:**
  - In steel and steel-concrete structures, the material models can be Linear, Bilinear, Multi-Linear, or User-Defined models
  - Steel, concrete, and composite models may be already integrated in the library of the program
- **Failure criteria**
  - Elastic materials behave linearly without any plastic deformations. A predefined failure point can also be set
  - Different failure criteria may be employed:
    - principal tensile strain
    - tensile strength (steel)
    - compressive strength;
    - shear strength
    - Mohr-Coulomb failure envelope (concrete)
    - other acceptance criteria based on test results, tabulated values, best design practice



Blast load effects against a steel frame building (exterior wall and glass windows are also represented in the figure on the right) (simulations using ELS)



Blast load effects against a steel column (close view) (simulations using ELS)

FULL DYNAMIC APPROACH (continued)

- **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
  - Time step too short ➔ very long analysis time // large time step ➔ less accurate analysis
  - If time step = ΔT, then the shortest period that can be considered in the analysis is 2ΔT (highest frequency $\pi/\Delta T$).
  - Blast analysis usually requires ΔT of 0.00001 sec.
- **Blast scenarios:**
  - Blast effects are modelled using free 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. CFD may be used if necessary.
- **Boundary conditions and initial state:** displacements or rotations at supports can be fixed or free to develop (partial fixity can be also employed). Initial conditions are required (velocity and acceleration values at the start of motion, t = 0.0)
- **Equilibrium equations**

$$[M]\{\ddot{x}\} + [C]\{\dot{x}\} + [K]\{x\} = \{F\}$$

M – mass (inertia) C – damping (energy dissipation) K – stiffness (restoring force)
F - applied force x – displacement $\dot{x}$ - velocity $\ddot{x}$ - acceleration

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CONTENT LIST

- **This section describes mitigation strategies and design approaches for five types of identified accidental actions**
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3.2 Internal gas explosion

EXPLICIT DESIGN

Consequences class 1	No specific consideration
Consequences class 2	Key elements of the structure can be designed using equivalent static load model
Consequences class 3	Dynamic analysis

- **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**

3.2 Internal gas explosion

Equivalent static pressure approach

$$p_d = 3 + p_{stat}$$

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

p_d	the nominal equivalent static pressure to design the structure in [kN/m²];
p_{stat}	the uniformly distributed static pressure at which venting components will fail in [kN/m²];
A_v	the area of venting components in [m²];
V	the volume of rectangular enclosure in [m³].

The ratio of the area of venting components and the volume should comply with the following formula:

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

3.2 Internal gas explosion

Dynamic approach (TNT equivalence method)

- **The mass of the gas (or vapour cloud) is converted into a TNT equivalent charge**
- **The equivalent TNT charge is estimated from the energy content in the exploding gas cloud**

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

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

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

V [m³]	smaller of *the total volume of the congested region* or *the volume of the gas cloud*

- **Limitations of TNT equivalence method**
  - For explosion pressures below 1 bar, the TNT equivalence 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
  - It 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).

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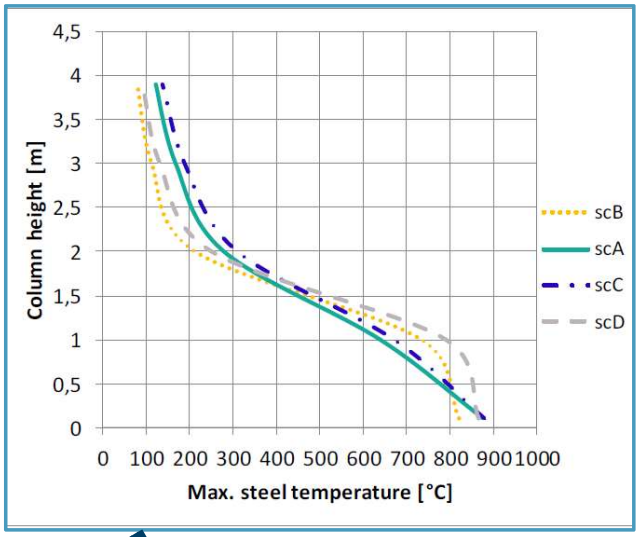
4. FIRE AS EXCEPTIONAL EVENT

- Structures should be designed to fulfil a *minimum required fire resistance*, which is usually based on the concept of *minimum required fire time*. This concept limits, to an acceptable level, the probability of human life losses and/or extensive damage of structures
- Part 1-2 of EC 3 and EC4 present the design fire rules for steel and composite structures
- An exceptional fire event is an event beyond the cases considered by the codes:
  - localised fire around a column when, which, in normal fire situation, should not take place
  - sequential exceptional events (e.g., fire after earthquake, fire after impact/explosion)

4. FIRE AS EXCEPTIONAL EVENT

- **Localized fire around a column when, which, in normal fire situation, should not take place**

Scenario	Diameter of the fire basis	Rate of heat release density	Fire load density	Fire growth rate
A	2 m	250 kW/m² (office building)	511 MJ/m² (office building)	300 sec (office building)
B	1 m	500 kW/m² (office building)	511 MJ/m² (office building)	300 sec (office building)
C	2 m	250 kW/m² (commercial area)	730 MJ/m² (commercial area)	150 sec (commercial area)
D	1 m	500 kW/m²	730 MJ/m² (commercial area)	150 sec (commercial area)



- When the structure is designed for fire following Eurocode rules, it is unlikely that the fire will lead to the collapse of the bearing elements and cause loss of stability

4. FIRE AS EXCEPTIONAL EVENT

- **Localised fire analysis can be taken into account using the Annex C of EN 1991-1-2**
  - Flashover unlikely to occur
  - Depending on the size of the compartment and of the fire, can or cannot impinge the ceilling of the compartment

Flame length

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

When $L_f \geq H$ → fire impinges the ceiling

H – compartment height

Q – rate of heat release

D – fire diameter

Temperature of the flame

(when $L_f < H$)

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

Q_c convective part of the rate of heat release (=0.8Q)

Z height of the flame along its axis;

Z_o virtual origin of the fire

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

Net heat flux at ceiling level

(when $L_f \geq H$)

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

$\dot{h}$ heat flux lux received by the fire exposed per unit of surface at the level of the ceiling

α_c heat transfer coefficient by convection

θ_m surface temperature

Φ configuration factor

ε_m surface emissivity of the member

ε_f emissivity of the fire

σ Stephan Boltzmann constant (5.67 x 10-8 W/m2K4)

4. FIRE AS EXCEPTIONAL EVENT

- **Alternatively, advanced fire models can be used:**
  - **Zone models -** Annex D of Eurocode 1 Part 1-2 provide basic equations of conservation of mass and energy. Examples of software that can be used if the Consolidated Fire and Smoke Transport **CFAST** from NIST or the **OZONE** developed in the University of Liege
  - **CFD model (Computational fluid dynamic model)** –Annex D of Eurocode 1 Part 1-2. An example of software that can be used for CFD analysis is the Fire Dynamics Simulator **FDS** from NIST

4. FIRE AS EXCEPTIONAL EVENT

Sequential exceptional events

- **Another situation where fire is considered as exceptional load is for cases where the fire appears after a first exceptional event, such as:**
  - Fire after earthquake
  - Fire after impact/explosion

Kobe, 1995 – Fire after earthquake

In these situations, the structure is already damaged by the first event, thus, the fire standard design is no longer valid

Twin Towers, 2001
Fire after impact/explosion

4. FIRE AS EXCEPTIONAL EVENT

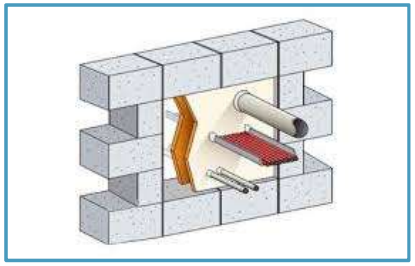
REDUCE/PREVENT THE ACTION

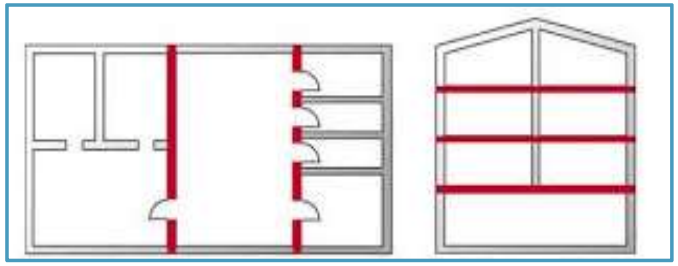
- **Measures to prevent and/or reduce the fire action and/or avoid the spread of the fire should be considered**
- **Aspects regulated by law:**
  - Storage near columns not allowed
  - Control of the materials for facades
  - Distance between buildings to reduce risk of spreading
- **Systems preventing the fire spread:**
  - Fire extinguishers – activated manually, when fire appears
  - Sprinklers – automatic systems activated, when smoke or high temperature arises

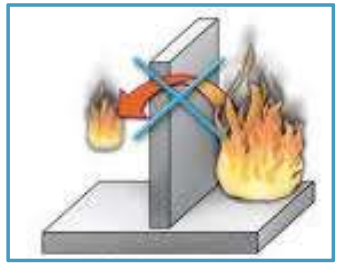
4. FIRE AS EXCEPTIONAL EVENT

REDUCE/PREVENT THE ACTION (continued)

- **Systems preventing the fire spread :**
  - 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







- **Systems for quick detection and early warning:**
  - Smoke detectors
  - Thermal detectors
  - Alarm systems
  - Exit road marking

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5. EARTHQUAKE AS EXCEPTIONAL EVENT

An **earthquake** is a sudden release of strain energy accumulated in the Earth's crust, caused mostly by the rupture of geologic faults. Due to its nature, it is virtually not possible to prevent or eliminate the seismic hazard

➔ the reduction and prevention of consequences are strictly associated with the building structure and the integrated systems, which help the building to adequately respond to the seismic action

Earthquake can be considered as exceptional event when:

- Structure is not designed for a seismic action at all, or it 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)

Christchurch, New Zealand (2010, 2011 events):

- 2010 earthquake: damages predominantly to pre-1970s buildings

- 2011 earthquake: many buildings that collapsed were already weakened by the 2010 event

5. EARTHQUAKE AS EXCEPTIONAL EVENT

PRESCRIPTIVE APPROACH

- **This approach is especially beneficial for non-seismic areas where seismic actions may occur but with a very low probability**

  Building configuration (building's size and shape, and structural and non-structural elements) determines the way seismic forces are distributed within the structure, their relative magnitude, and other design concerns.

- **Torsional effects 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 recommended**

5. EARTHQUAKE AS EXCEPTIONAL EVENT

PRESCRIPTIVE APPROACH (continued)

- **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

5. EARTHQUAKE AS EXCEPTIONAL EVENT

PRESCRIPTIVE APPROACH (continued)

- **Vibration control**: buildings are, in general, poor resonators to dynamic shocks and therefore dissipate vibration by absorbing it
- **To improve the response:**
  - **Base isolation** can be used to detach (isolate) the building from the ground greatly reducing seismic energy transferred to the superstructure. *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 (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

5. EARTHQUAKE AS EXCEPTIONAL EVENT

PRESCRIPTIVE APPROACH (continued)

- **To improve the building response to earthquake the practitioners can act on :**
  - strength and stiffness properties which should be selected considering the balance between deformability and force resistant capacity.
  - ductility allowing the dissipation of part of the energy by plastic deformations.
    - **Ductile elements** 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

5. EARTHQUAKE AS EXCEPTIONAL EVENT

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.
  - The residual capacity after an earthquake can be defined as:
    - lateral force-resisting system - the minimum spectral acceleration that corresponds to local or global collapse during an aftershock.
    - gravity load-carrying capacity - the minimum level of gravity loads that corresponds to local or global collapse after a damaging earthquake.

5. EARTHQUAKE AS EXCEPTIONAL EVENT

PROCEDURE FOR THE EVALUATION OF THE SEISMIC ROBUSTNESS

Step 1: Design/evaluation for persistent / seismic design situations

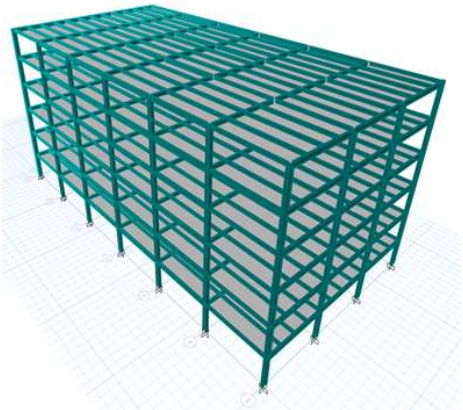
- The structure is first designed to meet the code-based requirements (for new structures, only). The seismic response can be calculated using a Nonlinear Static analysis (N2 method, EN 1998) following the recommendations from EN 1993-1-14 (2020) regarding the behaviour laws to be used for the materials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

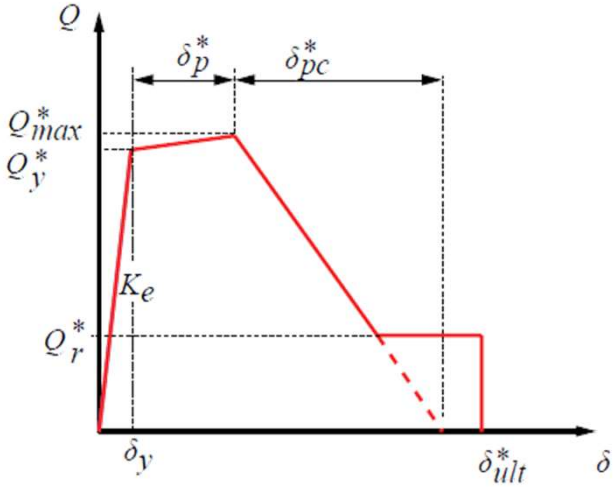
5. EARTHQUAKE AS EXCEPTIONAL EVENT

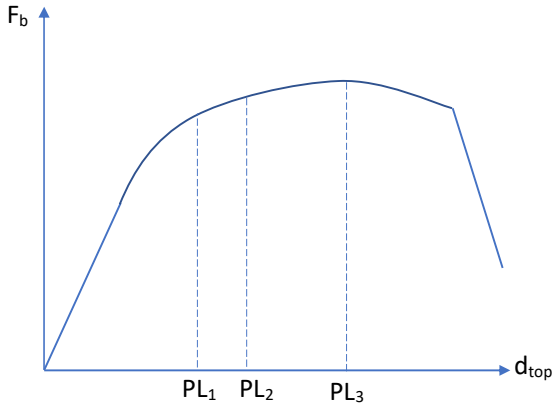
PROCEDURE FOR THE EVALUATION OF THE SEISMIC ROBUSTNESS

Step 1: Design/evaluation for persistent / seismic design situations

- The global seismic performance can be presented in the form of a base shear force - top displacement F_b - d_{top} curve.
- Performance levels (PL) are defined by the corresponding maximum top displacement, e.g., P_{L1} (limited damage), P_{L2} (moderate damage) and D_3 (large damage). Depending on the level of hazard, a certain damage level is expected





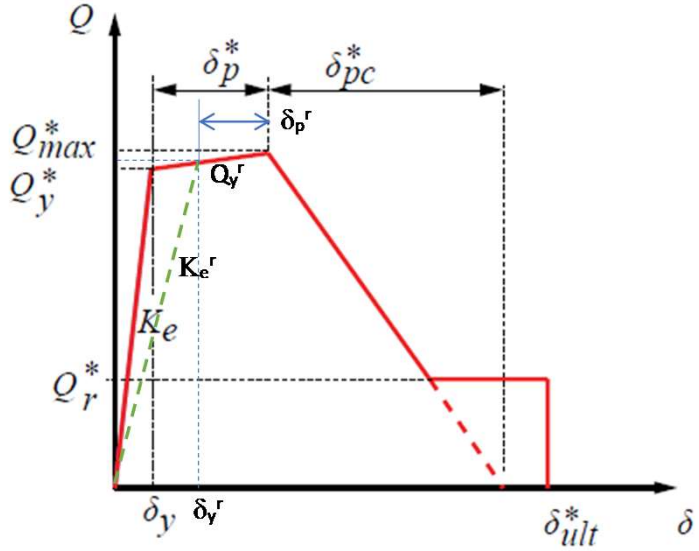


5. EARTHQUAKE AS EXCEPTIONAL EVENT

PROCEDURE FOR THE EVALUATION OF THE SEISMIC ROBUSTNESS

Step 2: Evaluation of the residual capacity after an earthquake

- After the local and global ductility demands are evaluated (Step 1), modifications of plastic hinges are introduced for the damaged elements (i.e., elements with plastic deformations), resulting in a modified nonlinear model. 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



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

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6. CONCLUSIONS

- In this presentation, design approaches to robustness in case of Identified threats have been presented
- Design approaches require the identification of the threats and the definition of the associated actions
- The cases of impact, explosion, fire as exceptional event and earthquake as exceptional events have been considered.
- For some actions the level of threat can be reduced or even eliminated with preventive or protective measures
- For the explicit design under identified accidental actions analytical and/or numerical methods are used
- The level of sophistication of the methods is strongly linked to consequences class of the structure under consideration

IDENTIFIED THREATS

Bruxelles 10.05.2022

Děkuji! Dank je! Thank you! Merci! Dankeschön! Grazie! Dziękuję Ci! Obrigado! Mulțumesc! Gracias!

Florea Dinu

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steelconstruct.com/eu-projects/failnomore

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Brussels 10/05/2022

UNIDENTIFIED THREATS

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FAILNOMORE

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

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1. Introduction
2. Selection of the design strategies
3. Identification of local damages
4. Alternative load path methods (ALPM)
5. Key element method
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1. INTRODUCTION

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   - 4.2 ALPM- Prescriptive methods
   - 4.3 ALPM-Analytical methods
   - 4.4 ALPM-Simplified numerical approach
   - 4.5 ALPM-Full numerical approach
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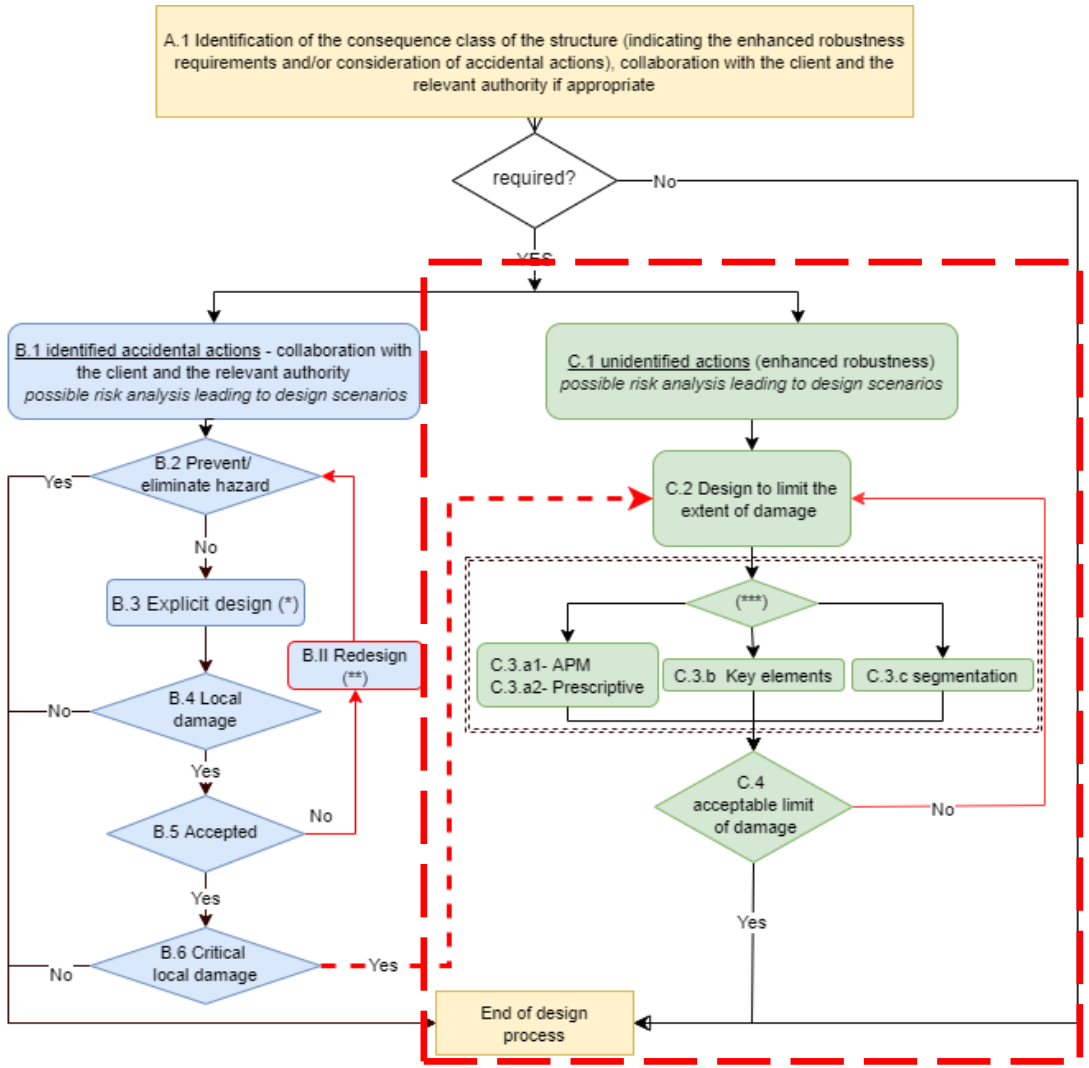
5. Key element method
6. Segmentation method
   - 5.1 Weak segment borders
   - 5.2 Strong segment borders
7. Conclusions

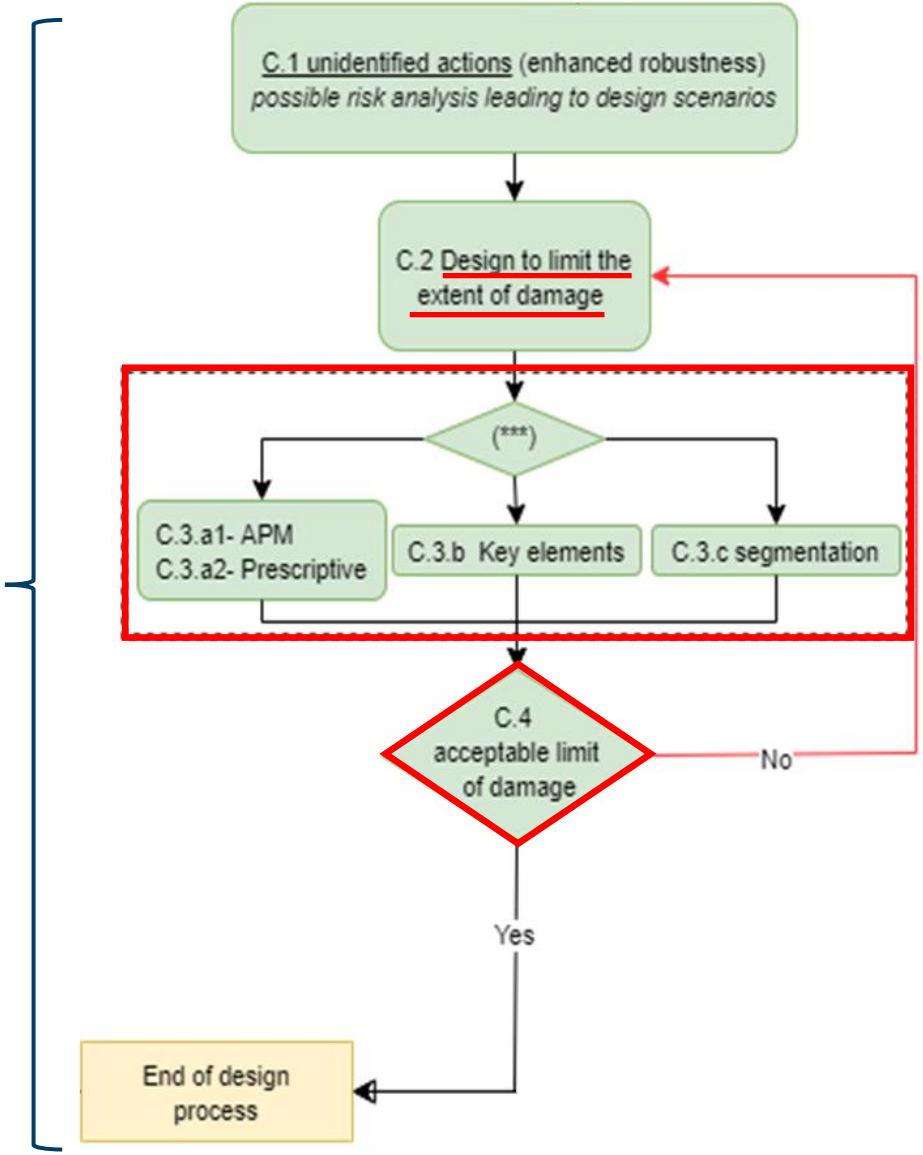
1. Introduction

- Unidentified threats refer to accidental actions not specifically "identified" by the client or other stakeholders
- By definition, unidentified threats cannot be characterised and are so unspecifiable
- Unidentified threats can also be associated to a combination of exceptional events, i.e. to unidentified scenario
- Accordingly, the adopted design strategies aim at limiting the extent of a localised damage, whatever is the initiating cause
- The selection of the design strategy to be adopted is dependent on the consequences class (CC) to which the considered structure belongs to

1. INTRODUCTION-OVERVIEW



Procedure for unidentified threats



1. Introduction
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5. Key element method
6. Segmentation method
7. Conclusions

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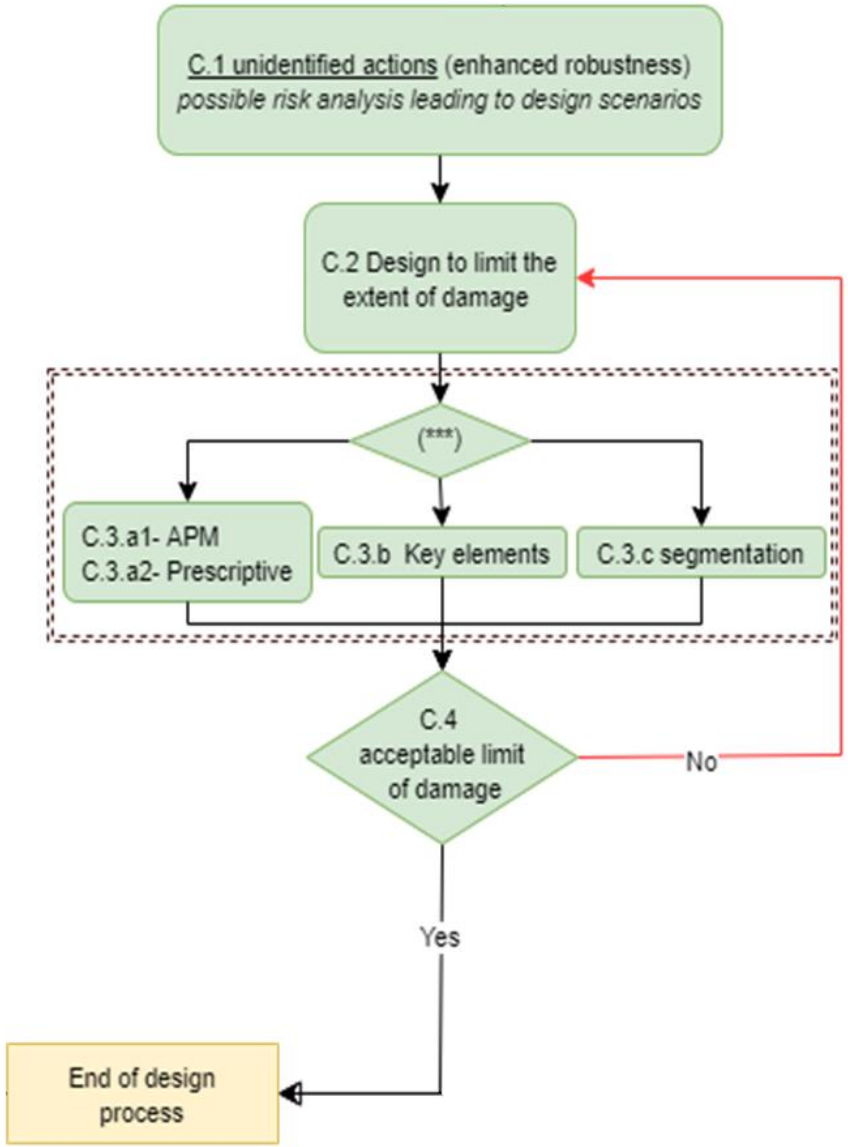
2. DESIGN STRATEGIES

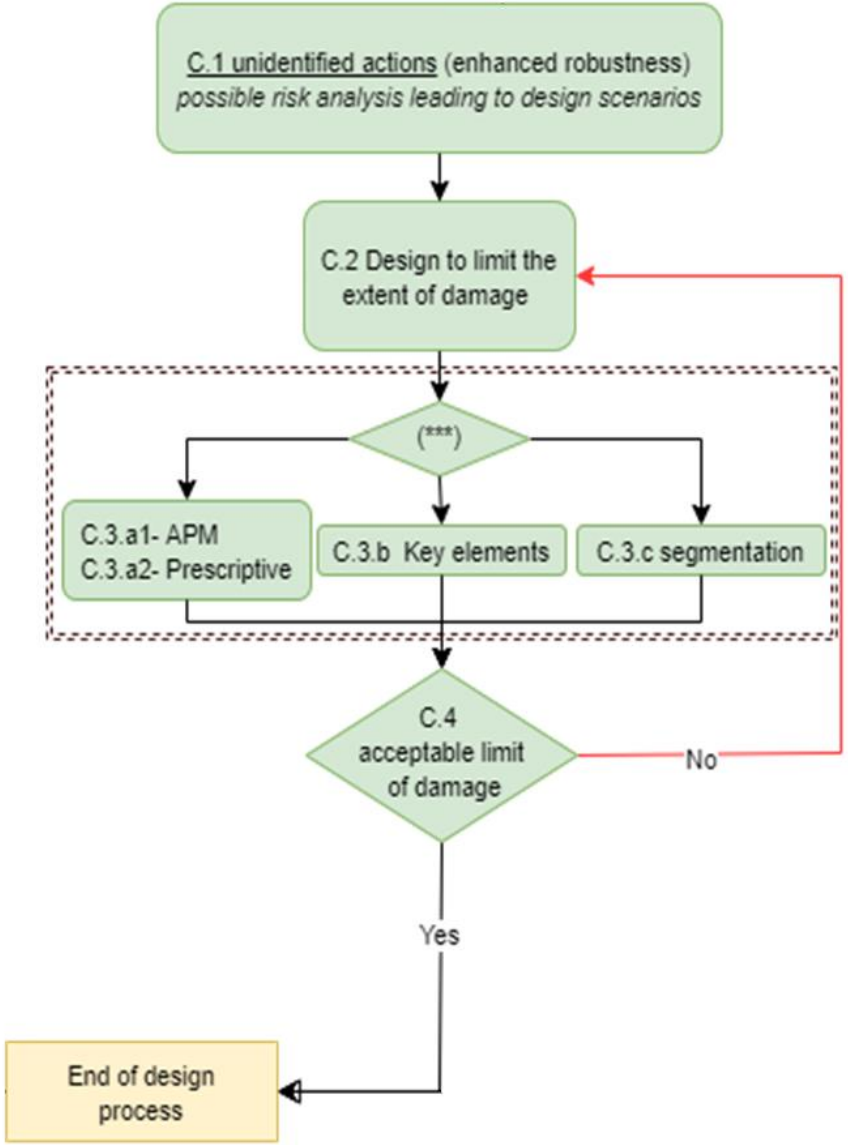
- **For CC1 structures:**
  - No specific request
- **For CC2 structures – lower risk group (CC2a):**
  - Prescriptive approach to secure effective horizontal ties
- **For CC2 structures – upper risk group (CC2b):**
  - Prescriptive approach to secure effective horizontal and vertical ties or …
  - Alternative load path method (ALPM) or …
  - Key element method or …
  - Segmentation



2. DESIGN STRATEGIES

- **For CC3 structures:**
  - The design approaches proposed for CC2, upper risk group (CC2b), remains valid **but**…
  - It may require a risk analysis and the use of refined methods, i.e. full dynamic analyses, non-linear models…

➔ The level of sophistication of the design approach to be applied is increasing with the consequences class



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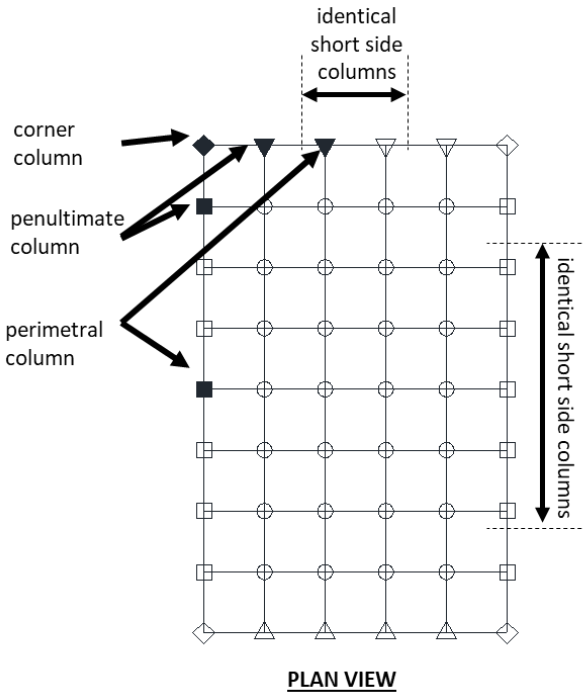
3. LOCAL DAMAGES

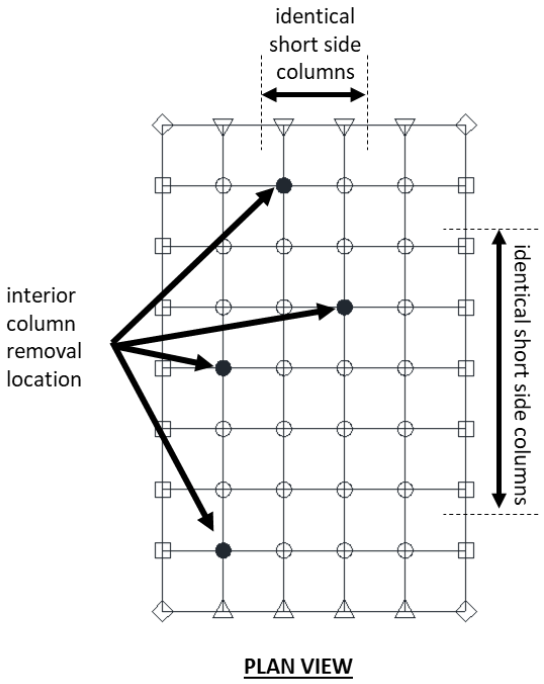
- According to EN1991-1-7, the local damage to be considered for building structures included in CC2b is the notional removal of each supporting column, or of each beam supporting a column

- This could represent a significant amount of work!

- Possibilities to reduce the number of column loss scenarios to be considered exist, in particular for regular buildings

3. IDENTIFICATION OF LOCAL DAMAGES





(DoD 2016)

- **For locations in terms of storey:**
  - First storey above ground;
  - Storey directly below roof;
  - Storey at mid-height;
  - Storey above the location of a column splice or a change in column size

3. IDENTIFICATION OF LOCAL DAMAGES

- **In EN1991-1-7, it is not stated if the column removal has to be assumed as instantaneous or as « quasi-static »**
- **The consideration of a quasi-static column loss allows:**
  - The use of simple analysis tools as no dynamic effects need to be accounted for
  - To have a good indication on the ability of a structure to activate alternative load paths under dynamic effects
- **The consideration of an instantaneous column loss offers an upper bound on the response of building structures**
- **In this presentation, methods to predict the response of frame assuming a quasi-static column loss will be first presented. Then, methods to account for the dynamic response of frames will be addressed.**

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4.1 ALPM-GENERAL

- Design approaches aim at providing the structure with alternative load paths that allow the redistribution of loads in the case of the local failure of a supporting member

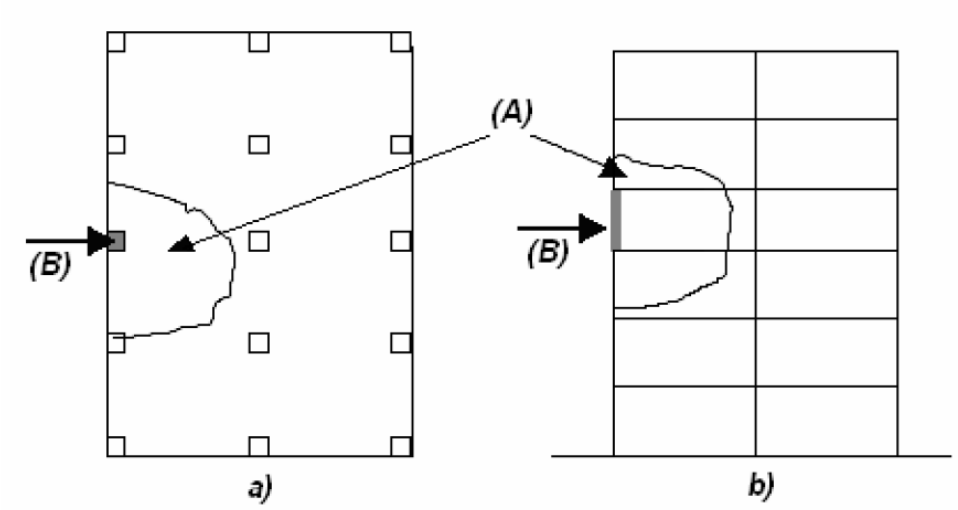
- Objective: to find a new state of equilibrium in the deformed shape

- This goal can be achieved by providing the structure with adequate resources of ductility, deformation capacity and redundancy, and/or applying prescriptive design rules, such as tying

4.1 ALPM-GENERAL

- **When applying the alternative load path method, according to EN 1991-1-7, it should be demonstrated that the local damage is not spreading to an extent which is disproportionate**
- **Limit of admissible local damage:**
  - 15% of the floor or …
  - 200 m^2

  whichever is smaller, in each two adjacent storeys



(A) Local damage

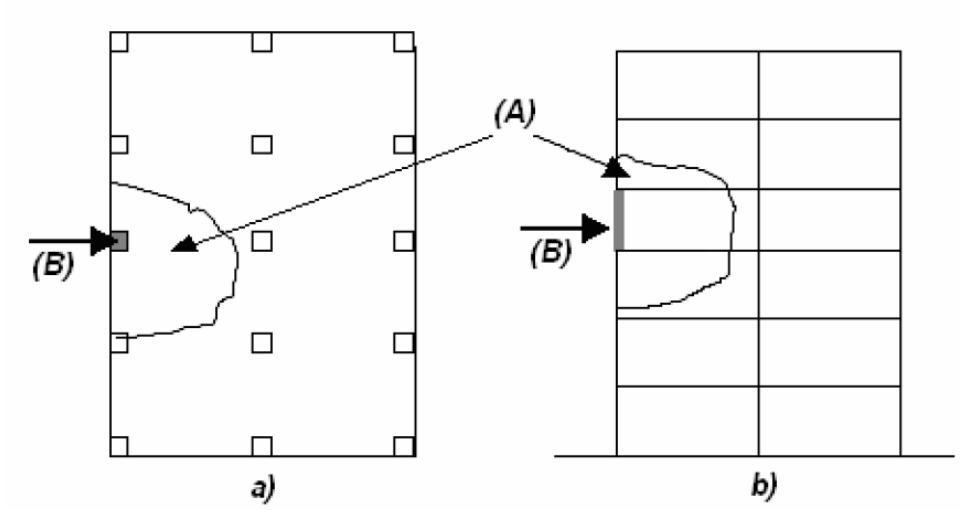
(B) Notional column removed

4.1 ALPM-GENERAL

- **When applying the alternative load path method, according to EN 1991-1-7, it should be demonstrated that the local damage is not spreading to an extent which is disproportionate**
- **Limit of admissible local damage:**
  - 15% of the floor or …
  - 200 m^2

  whichever is smaller, in each two adjacent storeys

What does it mean?



(A) Local damage

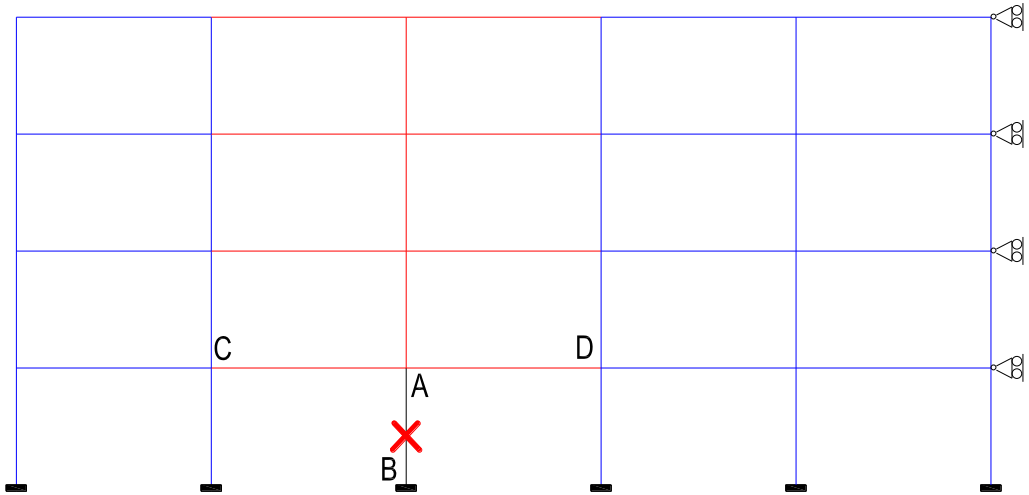
(B) Notional column removed

4.1 ALPM-GENERAL

- **Assumed scenario: loss of a column**

- **A building structure losing a column can be divided in two main parts:**
  - 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 and …
  - the indirectly affected part (IAP), which includes the rest of the structure

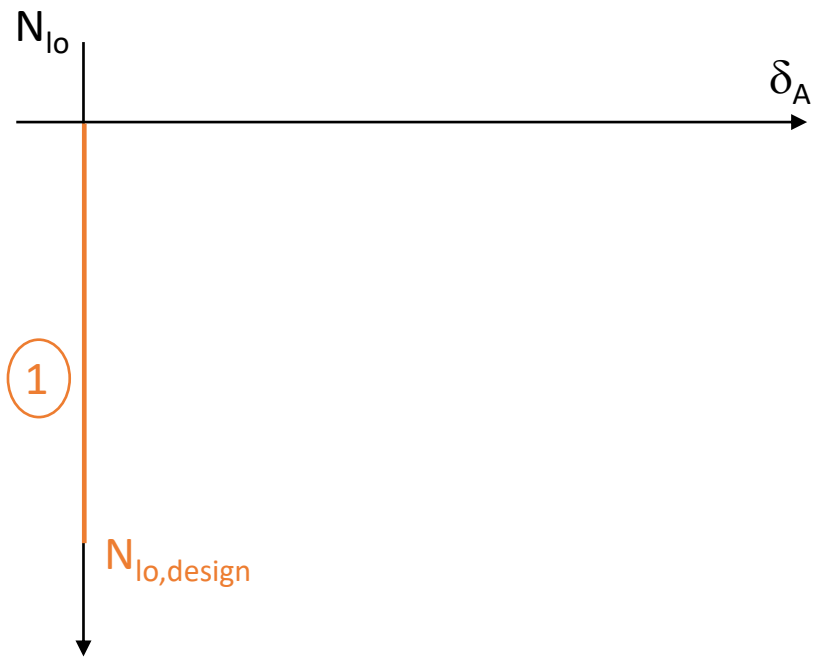
In red: directly affected part
In blue: indirectly affected part



Column assumed to be lost

4.1 ALPM-GENERAL

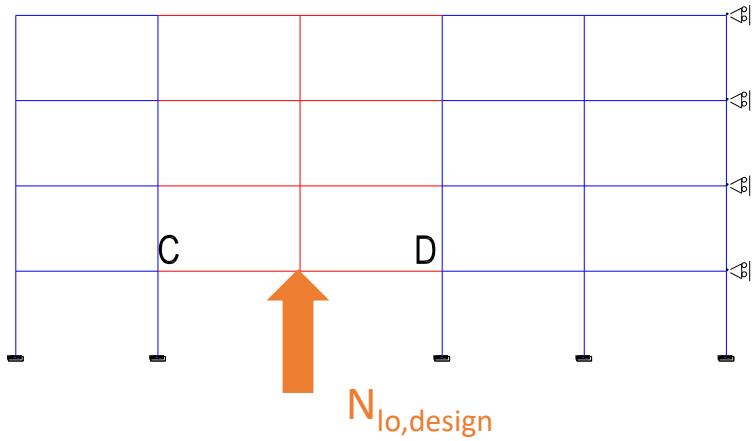
- **The response of a frame further to a column loss can be subdivided in different successive phases, which develop or not according to the cases**



Phase 1 ⇨ Building under accidental load combination

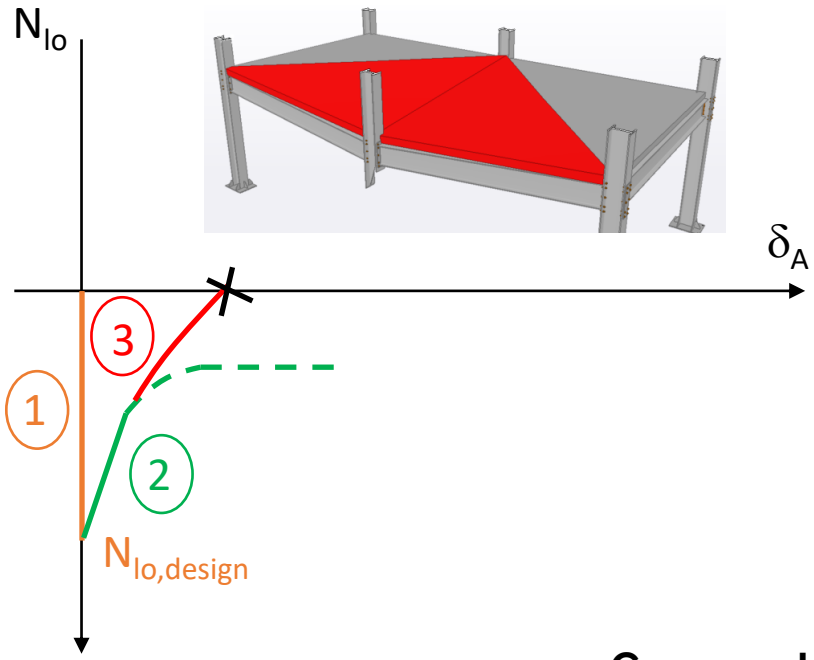
⇩

Column to be lost subjected to an axial load $N_{lo,design}$



4.1 ALPM-GENERAL

Robustness may possibly be ensured by the slabs alone, if present (slabs connected or not to the steel beams)



Phase 2 ⇒ Beginning of the column loss

⇓

First, an elastic response is observed
Then, **plastic** plate **mechanisms** form in the slabs

Phase 3 ⇒ Significant displacements appear

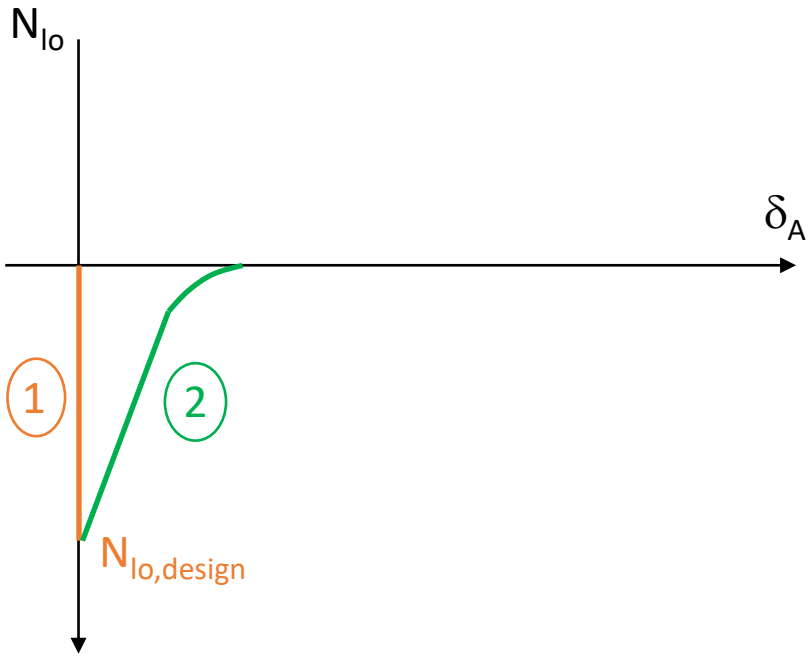
⇓

Catenary actions within the slabs develop until brittle failure occurs due to the lack of ductility of the rebars (e.g.)

Comment: if robustness is not sufficient, then other structural contributions will have to be activated, requiring significant displacements that the slab could not afford. In this case, « Phase 3 » in the slab will have to be ignored

4.1 ALPM-GENERAL

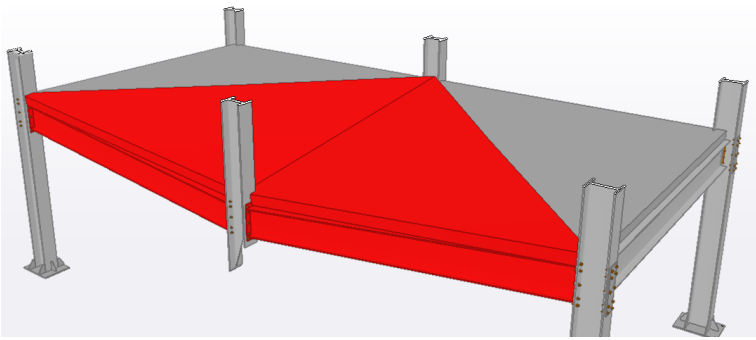
- **If robustness is not ensured by the slabs alone, yielding of the slabs and of the DAP beams may be activated**



Phase 2: ⇨ Beginning of the column loss

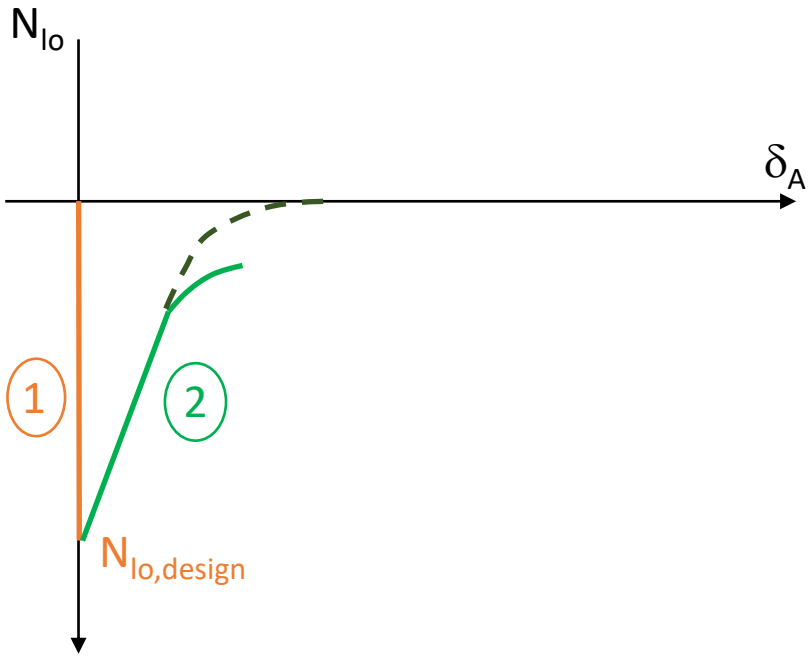
⬇

- Plastic plate mechanisms in the slabs
- Plastic beam mechanisms in the DAP beams (steel or composite if the slabs are connected to the steel beams)



4.1 ALPM-GENERAL

- **If robustness is not ensured by the slabs alone, yielding of the slabs and of the DAP beams may be activated + arching effects in beams**

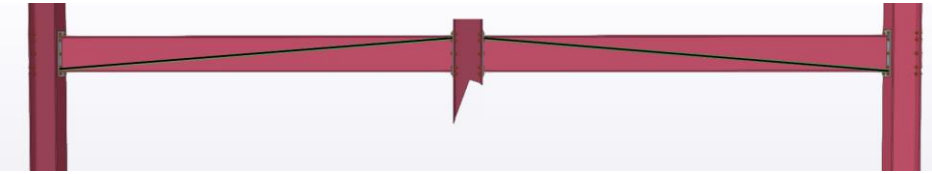


Phase 2: ⇨ Beginning of the column loss

⇩

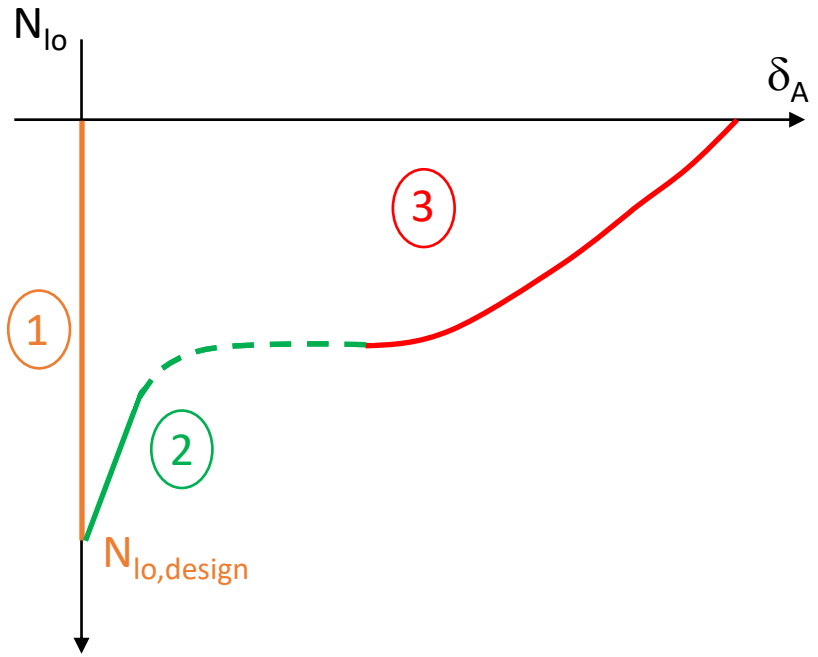
- Plastic plate mechanisms in the slabs
- Plastic beam mechanisms in the DAP beams (steel or composite if the slabs are connected to the steel beams)

In addition, in some specific cases, **arching effects** may develop in the DAP beams further to the development of plastic mechanisms



4.1 ALPM-GENERAL

If robustness is not yet sufficient, catenary actions in the beams may be activated



Phase 2: ⇒ Beginning of the column loss

⇓

- Plastic plate mechanisms in the slabs
- Plastic beam mechanisms in the DAP beams (steel or composite if the slabs are connected to the steel beams)
- Beam arching effect

A decrease of the resistance is contemplated as a result of a **"snap-through"** in the beams; it is followed by a drop due to the brittle failure of the rebars in the slabs, if present

Phase 3 ⇒ Significant displacements appear

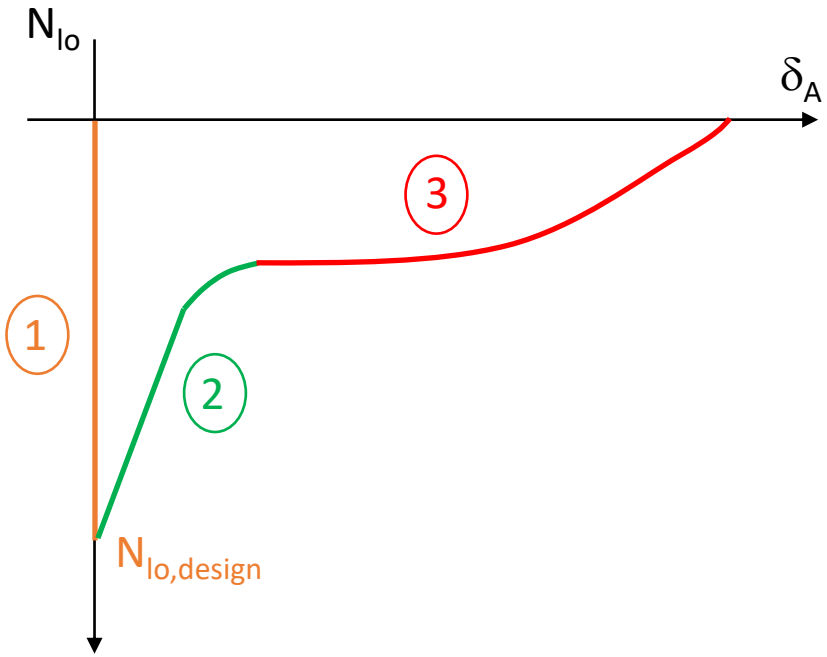
⇓

Catenary actions develop in the steel or composite DAP beams

Comment: Finally, two contributions to the robustness are to be considered : **beam mechanisms** and **catenary actions** in the DAP beams

4.1 ALPM-GENERAL

Visualisation of the global structural response



Phase 1: ➡ Building under accidental load combination

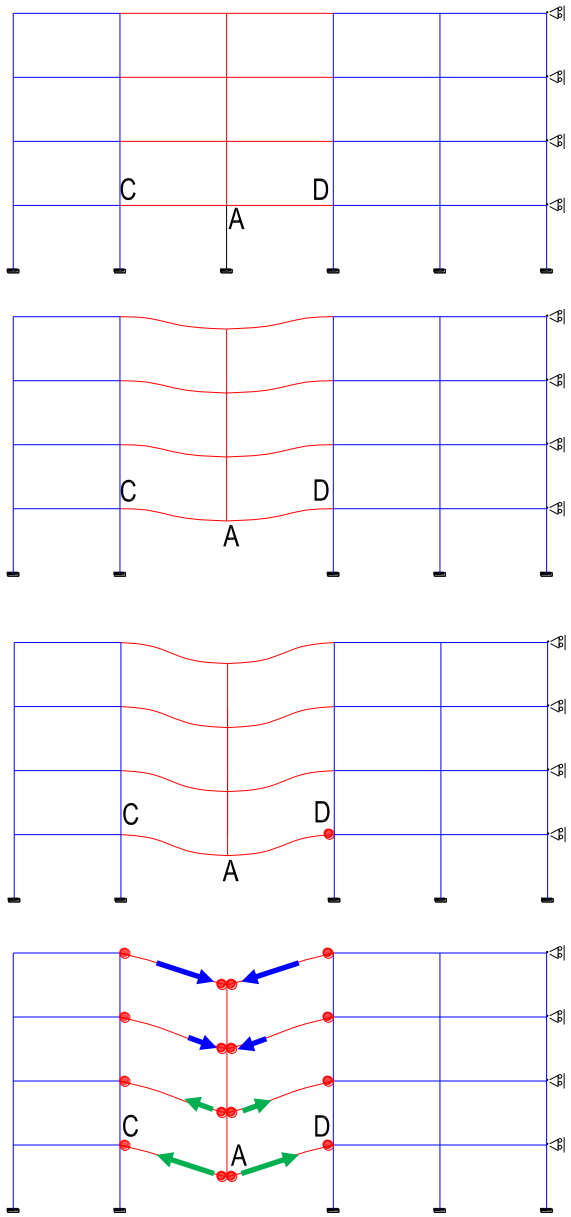
- Column to be lost subjected to an axial load $N_{lo,design}$

Phase 2: ➡ Beginning of the column loss

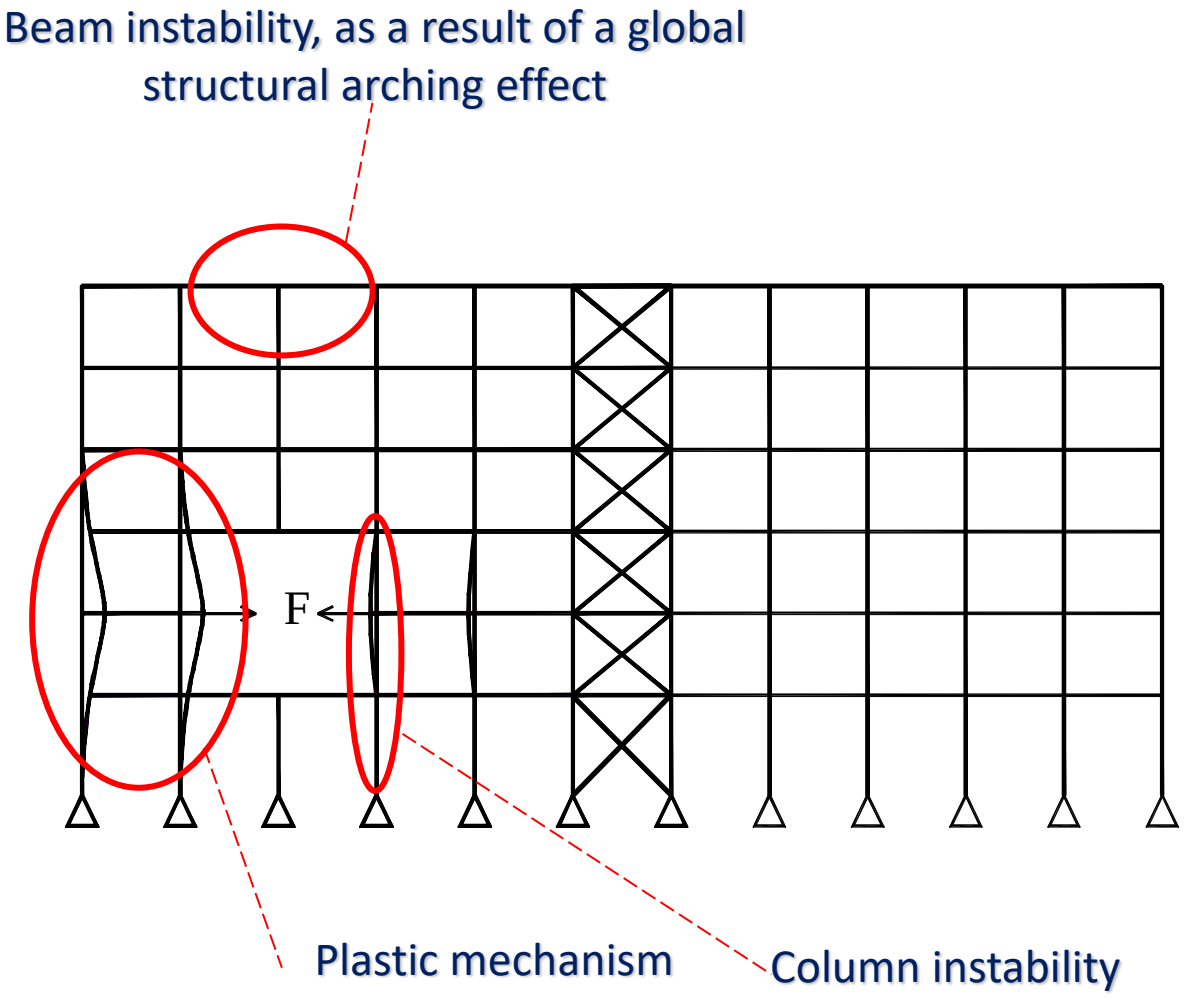
- First, an elastic response is observed
- Then, a first plastic hinge forms
- Finally, a complete plastic mechanism forms in the DAP

Phase 3: ➡ Significant displacements appear

- Development of 2nd order effects
- Axial forces in the beams appear (membrane/catenary forces)



4.1 ALPM-GENERAL



Other possible relevant failure modes need to be checked:

- **Buckling of the IAP columns adjacent to the lost column**
- **Global plastic mechanism in the IAP under the action of membrane forces transferred from the DAP to the IAP**
- **Buckling in compression of the upper beams of the DAP as a consequence of the development of arch effects**

4.1 ALPM-GENERAL

- **Different design methods, characterised by different complexity levels, can be adopted to implement the alternative load path approach**
- **The following are considered:**
  - 4.2 ALPM-Prescriptive methods
  - 4.3 ALPM-Analytical methods
  - 4.4 ALPM-Simplified numerical approach
  - 4.5 ALPM-Full numerical approach
  - 4.6 ALPM-Prediction of the dynamic response from the static one

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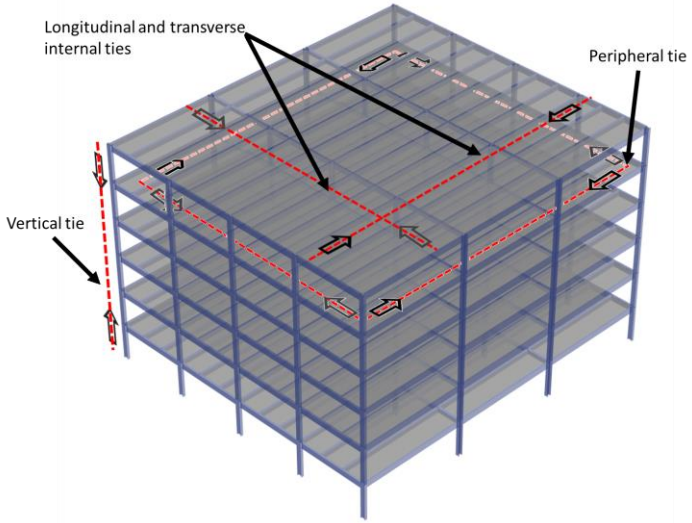
4.2 ALPM-PRESCRIPTIVE METHODS

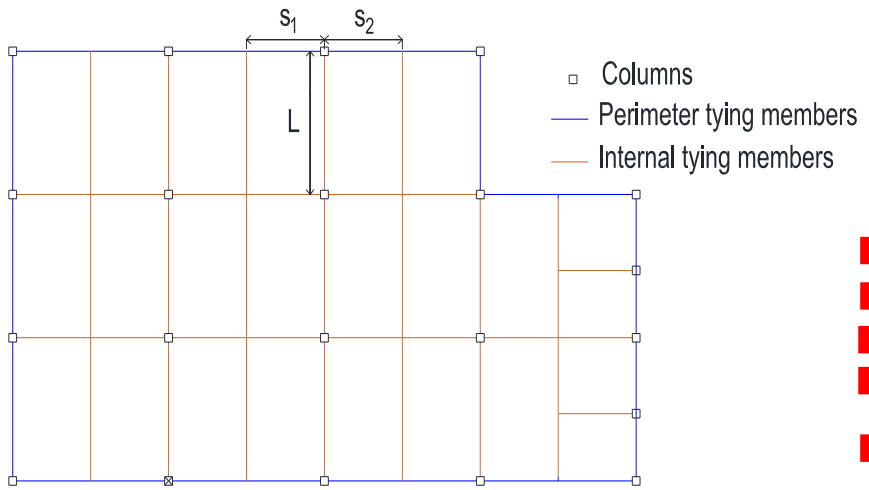
- **The goal of such methods is to provide a minimum level of robustness and resistance to progressive collapse for the structure**
- **The Tying force method is a prescriptive design method which:**
  - provides a minimum level of continuity and strength between structural members
  - uses horizontal and vertical tie elements
- **This approach is considered by the EN 1991-1-7 and prescribes:**
  - Horizontal tying to be used for CC2a buildings
  - Both horizontal and vertical tying to be used for CC2b buildings



4.2 ALPM-PRESCRIPTIVE METHODS

- **Horizontal ties should be located at each floor and roof level**
  - Around the plan perimeter and….
  - Internal ties at two right angle to tie the column and wall elements and…
  - Steel beams, reinforcing rebars and fabric reinforcement in r.c. slabs, profiled steel sheeting in composite floors can be used
- **Members and connections have to be designed to resist a minimum level of tying forces**

REQUIREMENTS FOR FRAMED STRUCTURES (EN 1991-1-7)



Internal ties ⇒ $T_i = 0.8(g_k + \psi q_k)sL \geq 75\ kN$

Perimeter ties ⇒ $T_p = 0.4(g_k + \psi q_k)sL \geq 75\ kN$

- g_k permanent surface action applied on the considered floor
- q_k variable surface action applied on the considered floor
- s average spacing of adjacent ties ($s = (s_1 + s_2)/2$)
- L span of the tie
- ψ relevant combination factor of action effects for accidental design situations (EN 1990)

4.2 ALPM-PRESCRIPTIVE METHODS

- **To allow for the possible activation of tying members, a minimum level of ductility is required**
- **In EN 1991-1-7, no clear indications are provided**
- **The FAILNOMORE design manual is filling this gap:**
  - If over-strength joints are used at the beam extremities, the use of Class 1 cross-section under hogging and sagging bending is recommended
  - If full-strength joints are used, ductility is required from the joint and the beam
  - If partial-strength or simple joints are used, ductility/deformation capacity is required at the level of the joints
- **When ductility/deformation capacity is required at the level of the joints, it is recommended to apply the criteria to ensure a minimum deformation capacity already discussed in the "Design for Robustness" presentation**

4.2 ALPM-PRESCRIPTIVE METHODS

- The minimum tensile design forces computed using the prescriptive method are defined to ensure a minimum level of continuity/redundancy in the floor
- These forces do not at all reflect the actual level of tensile forces, which could occur in the case of complete loss of column (as highlighted through the worked examples)
- Finally, a solid link between the tying capacity and the actual resistance to progressive collapse cannot be established

↓

Efficiency of this method remains questionable

4.2 ALPM-PRESCRIPTIVE METHODS

- A more refined method has been recently developed by B. Izzuddin from Imperial College London and proposed to normative committees
- This method allows for the prediction of tensile loads closer to the ones which would occur in the case of column loss scenario
- It can be adapted to any structural systems through an appropriate calibration of some coefficients

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

- T tensile load to be supported by the considered tying member
- η amplification coefficient to account for possible dynamic effects
- ρ reduction factor to account for different effects such as strain hardening or interaction between tensile load and bending
- i_f tying force intensity factor depending of the system under consideration

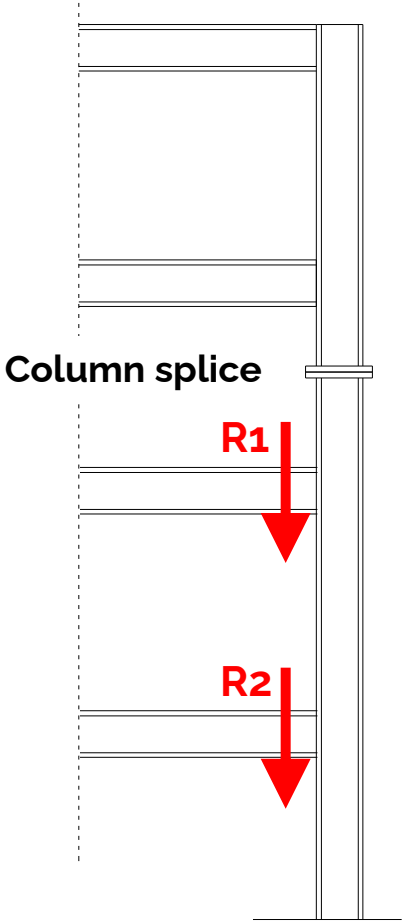
- $\bar{\alpha} = \frac{\alpha}{0.2}$ coefficient to account for the chord rotation capacity α (in rad) for different structural typologies
- P equivalent load to account for the loads applied to the considered floor

4.2 ALPM-PRESCRIPTIVE METHODS

- Composite floors are an efficient structural solution to activate alternative load paths in case of a column loss scenario
- These allow for the activation of membrane forces within the connected slab while requiring much less deformation capacity at the level of the beam extremities
- The use of steel beam grids with the upper flange of the beams connected to the slab in the two main directions is recommended
- For the slab, it is proposed to follow the recommendations from EN 1992-1-1, where minimum requirements are given to provide the slab with a tying system
- Specific construction details for slabs made of precast concrete elements are also provided in the Design Manual

4.2 ALPM-PRESCRIPTIVE METHODS

- Vertical tying should be provided for CC2b structures
- In framed buildings, the columns should be capable of resisting an accidental design tensile force equal to the largest design vertical permanent and variable load reaction applied to the column from any one storey (not applied with normal loading)
- In practice, the structural elements to be checked are the column splices
- Column splices should be designed to carry the largest total beam end reactions applied at a single floor
- End reactions should be calculated for the normal design case and not for the accidental load case
- Check of the column splices subjected to tensile load is not explicitely covered in the Eurocodes → information provided in the design manual



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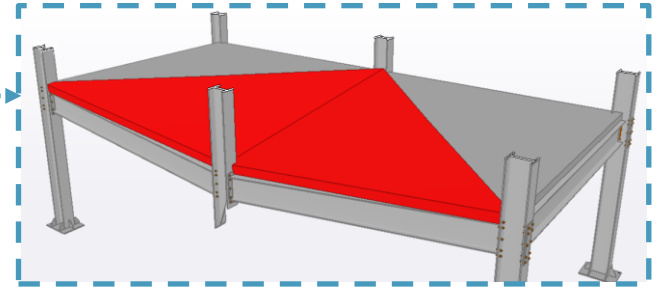
4.3 ALPM-ANALYTICAL METHODS

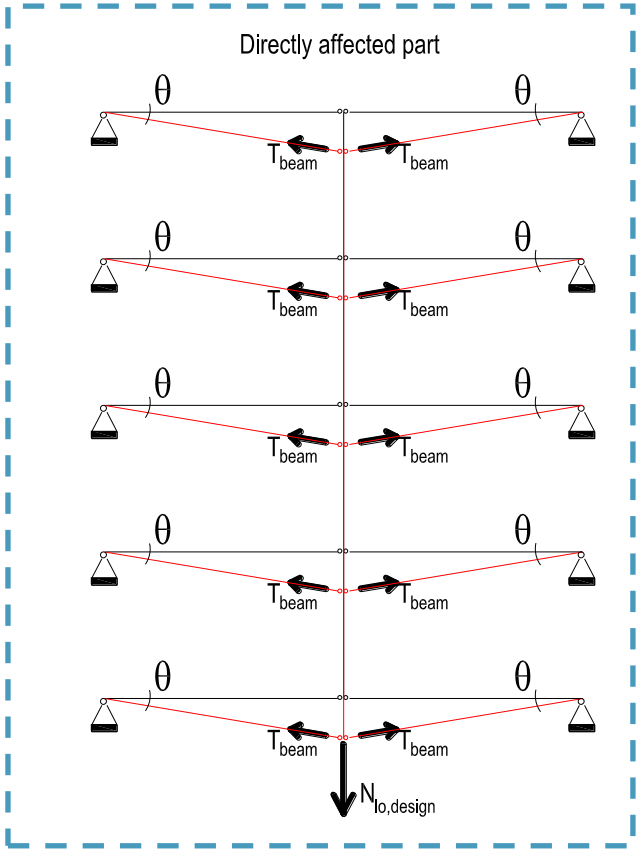
- **The robustness assessment methods proposed in the Design Manual are:**
  - Simplified method for structures with simple joints
  - Simplified method for structures with partial-strength joints
  - Simplified methods for structures with over-strength joints
  - Advanced method
- **The simplified ones are briefly described herein after**
- **The advanced one is detailed in the Design Manual (Annex A.8)**
- **All relevant application rules are provided in the Design Manual**

4.3 ALPM-ANALYTICAL METHODS

SIMPLIFIED METHOD FOR STRUCTURES WITH SIMPLE JOINTS

- Slabs considered at each floor (working as diaphragms)
- Two possibilities may be contemplated:
  - Either the robustness is ensured by the slabs (yield mechanisms + catenary actions in the slab)
  - Or the robustness is ensured by the DAP (catenary actions in the beams)
- IAP check: columns adjacent to the lost one





4.3 ALPM-ANALYTICAL METHODS

SIMPLIFIED METHOD FOR STRUCTURES WITH PARTIAL-STRENGTH JOINTS

- Slabs considered at each floor (working as diaphragms)
- Two possibilities may be contemplated:
  - Either the robustness is ensured by the slabs
    (yield mechanisms in the slab+catenary actions in the slab)
  - Or the robustness is ensured by the slab + DAP beams
    (yield mechanisms in the slab and in the beams + arch effects)
  - ~~Or the robustness is ensured by the DAP beams~~
    ~~(yield mechanisms in the beams + catenary actions in the beams)~~
- IAP check: columns adjacent to the lost one

Need for advanced models

4.3 ALPM-ANALYTICAL METHODS

SIMPLIFIED METHOD FOR STRUCTURES WITH OVER-STRENGTH JOINTS

- Slabs considered at each floor (working as diaphragms)
- Two possibilities may be contemplated:
  - Either the robustness is ensured by the slabs
    (yield mechanisms in the slab+catenary actions in the slab)
  - Or the robustness is ensured by the slab + DAP beams
    (yield mechanisms in the slab and in the beams)
  - ~~Or the robustness is ensured by the DAP beams~~
    ~~(yield mechanisms in the beams + catenary actions in the beams)~~

Need for advanced models

- IAP check: columns adjacent to the lost one

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4.4 ALPM- SIMPLIFIED NUMERICAL APPROACH

- **Numerical approach for a column loss scenario requires**
  - To account for material non-linearities
  - To account for geometrical non-linearities
  - To account for possible dynamic effects
  - …

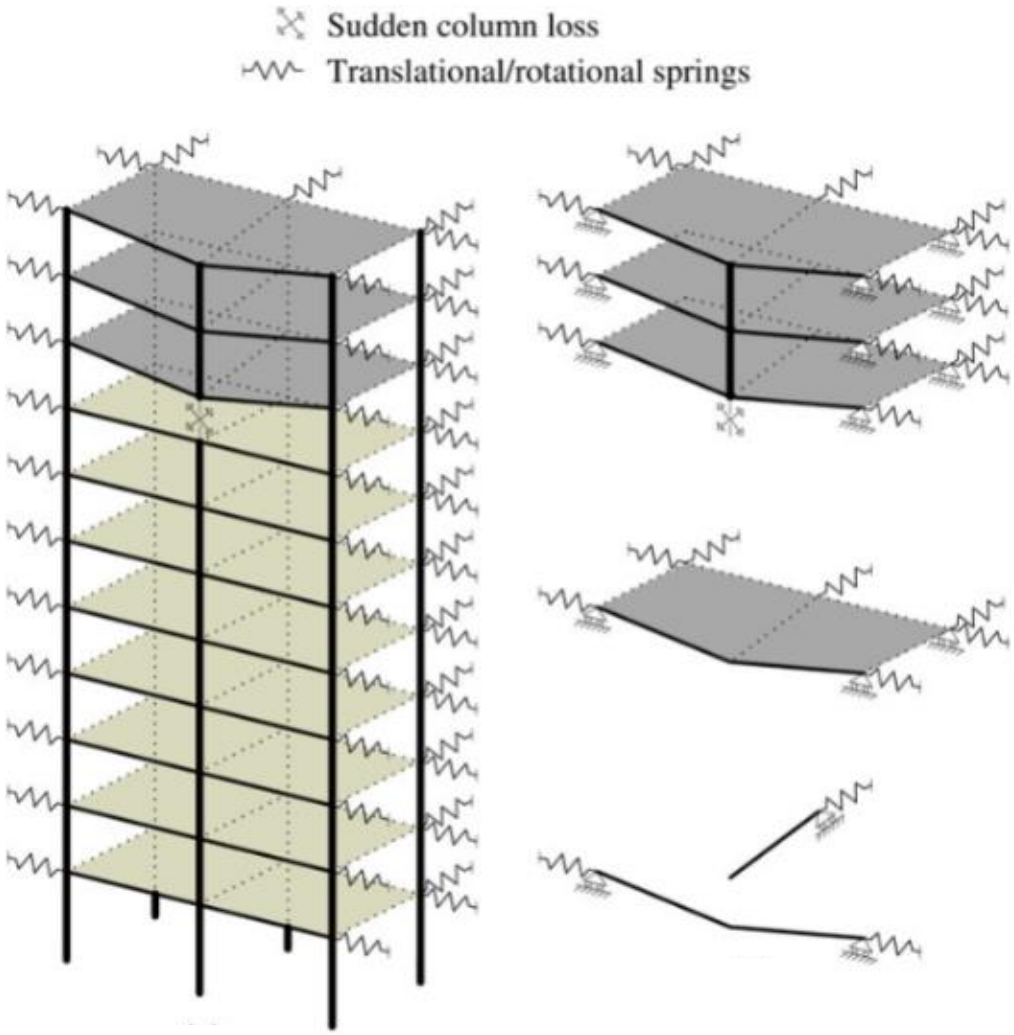
→ Requires the use of sophisticated tools and time-consuming analyses

- **Possibility to adopt simplified numerial approach addressed in the design manual**
  - Adopting simplified material behaviour laws
  - Defining substructure models
  - Realising simplified dynamic assessment → no need for detailed and sophisticated dynamic analysis

→ Practice-oriented approach

4.4 ALPM- SIMPLIFIED NUMERICAL APPROACH

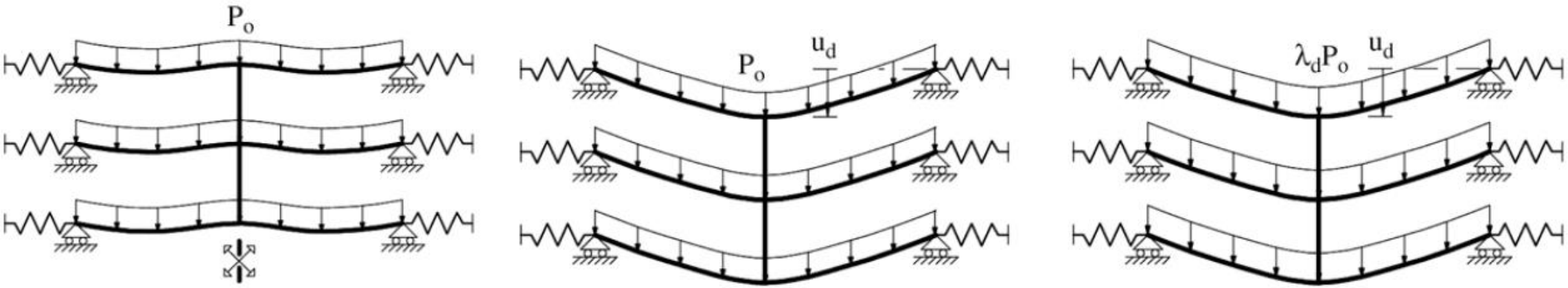
- **Possible substructure idealisations**
  - Modelling of the Directly Affected Bays
  - Floor(s) above lost column
  - Single floor above lost column
  - Individual steel/composite beam above lost column



4.4 ALPM- SIMPLIFIED NUMERICAL APPROACH

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
- The maximum dynamic response to be estimated accurately from the nonlinear static response under amplified gravity loading ($\lambda_d P_o$)



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4.5 ALPM-FULL NUMERICAL APPROACH

- Design solutions based on the use of advanced numerical programs (FEM, AEM, DEM) able to manage the building response in accidental loading conditions
- The effectiveness strongly depends on the ability of the designer to identify and manage the key factors affecting the structural response
- Attention has to be paid to phenomena associated with large displacements, energy dissipation (plastic hinges and yield lines) and failures associated with the constitutive relationship adopted for the materials
- Complexity of FE models mainly depends on the 'dimension' of the problem investigated and on the level of approximation and refinement adopted

4.5 ALPM-FULL NUMERICAL APPROACH

In the FE modelling phase, attention should be paid to:

- **Material models**
  - Non-linear material models are the most appropriate to investigate large displacements scenarios induced by progressive collapse
  - More complex materials' cumulative damage models would allow to catch local collapses as well as the potential detachment of the components
  - Depending on the investigated problem, the temperature dependence and the strain rate sensitivity of the material properties should be considered
- **Joints models**
  - connections 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, springs)
  - simplified models, such as the component method can be adopted with the requirement that stiffness, strength and deformation capacity are caught with accuracy

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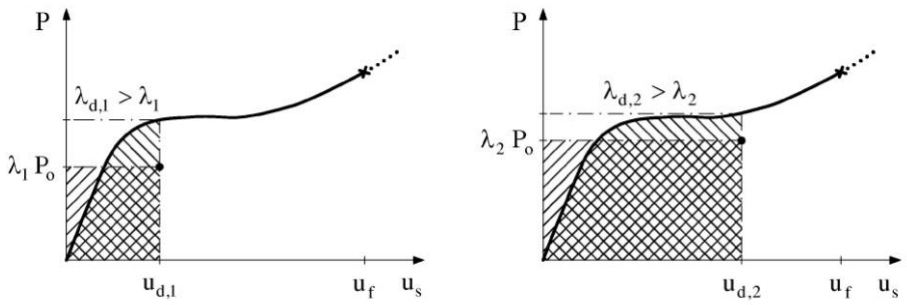
4.6 ALPM-PREDICTION OF THE DYNAMIC RESPONSE FROM THE STATIC ONE

- The maximum dynamic response can be determined from the non-linear response through a simplified approach
- Starting point: the sudden column loss resembles the sudden application of the gravity load on the DAP

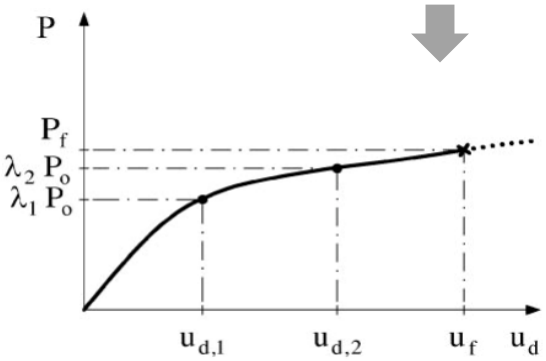
→ This gives rise to the concept of a "pseudo-static" response

4.6 ALPM-PREDICTION OF THE DYNAMIC RESPONSE FROM THE STATIC ONE



(a) Dynamic response ($P = \lambda_1 P_o$) (b) Dynamic response ($P = \lambda_2 P_o$)

- **The «pseudo-static» response can be obtained by plotting the applied gravity load (P_n) against the maximum dynamic displacement ($u_{d,n}$) for different levels of loading (λ_n).**



(c) Pseudo-static response

- **The maximun dynamic displacement associated with the sudden application of a gravity load λP_0 can be determined from the energy balance between the work done by the load and the internal energy stored.**
- **For a single degree of freedom mode the equivalence between the external work (W_n) and the internal energy (U_n) can be obtained by integrating the hatched areas of the figure.**

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

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

area under the nonlinear static (P, u_s) curve up to $u_{d,n}$

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5. KEY ELEMENT METHOD

- This design strategy is an alternative to ALP
- It is based on the design of structural Key Element, i.e. structural component (or part of a structure) the failure of which would induce a disproportionate collapse
- The design of key elements is carried out for a specific level of load
- Key elements, connections and attached components have to be designed to develop their full resistance without failure

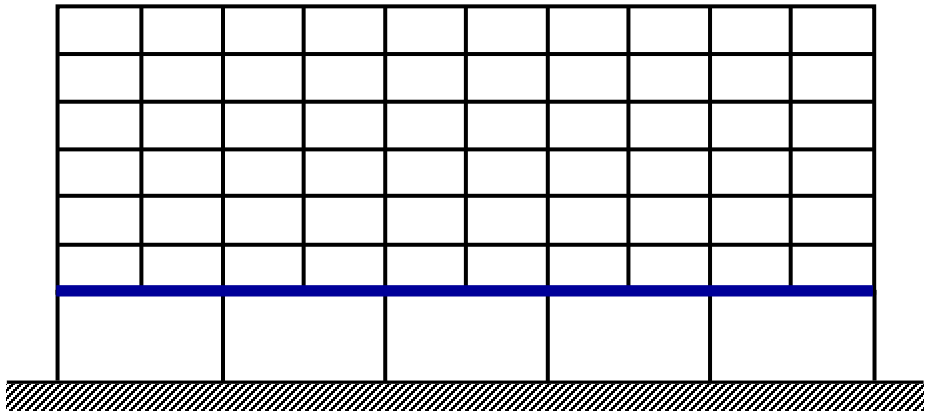
5. KEY ELEMENT METHOD

STEPS OF THE DESIGN

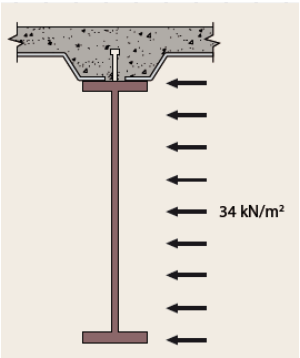
- **Identification of the key elements**
- **Design of the key elements to resist to a specificed accidental design action**
  - accidental load combination of EN 1990
  - EN1991-1-7 recommends 34 kN/m² applied in any direction
- **Accidental action applied to the key elements and to any attached component**

EXAMPLE

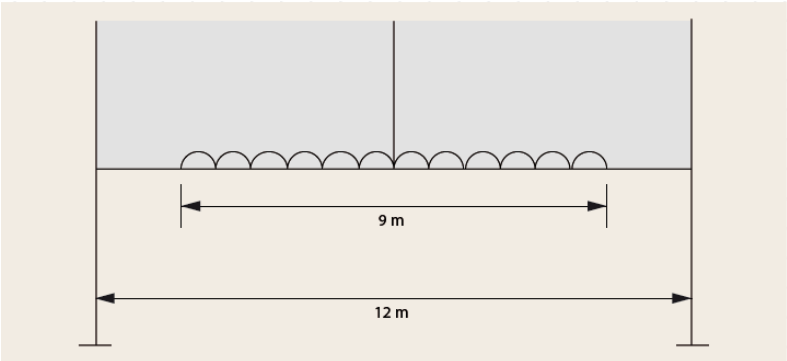
Key element ➡ Transfer girder



Codified accidental load (34 kN/m²)



Horizontal direction:
- Side on the transfer girder



Vertical direction
- Downwards on the top of the floor slab
- Upwards on the underside of the girder and the floor slab

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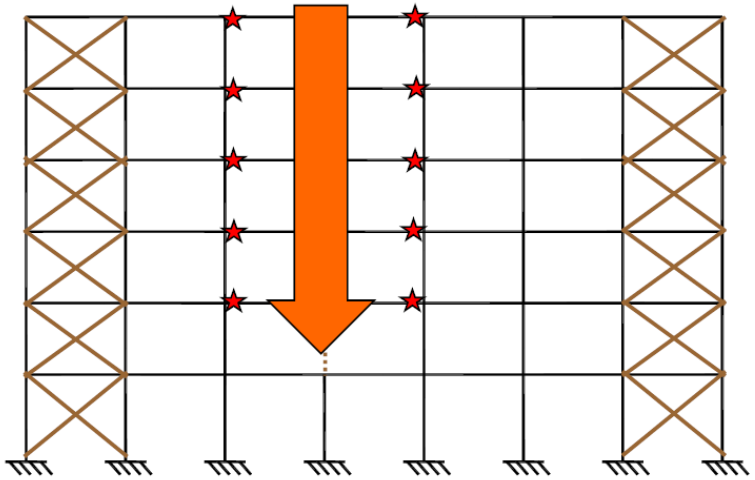
6. SEGMENTATION METHOD

- The spreading of failure is prevented/limited by isolating the failing part of a structure from the remaining structure by segment/compartment borders
- This approach would ensure that each part (compartment or segment) is able to collapse independently without affecting the safety of the other parts
- Segmentation strategies can be based on either **weak segment** borders or **strong segment** borders

6. SEGMENTATION METHOD

WEAK SEGMENT BORDERS

- This method would allow the failure of a specific segment to take place without the progression of failure to adjacent segments
- This method can be achieved by eliminating continuity between adjacent segments or reducing the stiffness to accommodate large deformations and displacements at the segment borders limiting the amount of force transmitted to the surrounding structure
- In the case where alternative load paths are impractical, or too expensive, segmentation by selectively eliminating continuity would be advantageous



6. SEGMENTATION METHOD

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

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7. CONCLUSIONS

- In this presentation, the philosophy of the design for robustness in case of **unidentified threats** has been presented
- Unidentified threats refer to accidental actions not specifically considered by standards or indicated by the client or other stakeholders or to any other actions/scenarios resulting from unspecifiable causes
- 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

UNIDENTIFIED THREATS

Brussels 10/05/2022

Děkuji! Dank je! Thank you! Merci! Dankeschön! Grazie! Dziękuję Ci! Obrigado! Mulțumesc! Gracias!

DEMONCEAU Jean-François

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steelconstruct.com/eu-projects/failnomore

Research Fund for Coal & Steel

FAIL NO MORE

LIÈGE université

Brussels 10-05-2022

WORKED EXAMPLES

Tudor GOLEA [1]

[1] University of Liège, Belgium

FAILNOMORE

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

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1. INTRODUCTION

1. Introduction
2. Identified accidental actions
3. Unidentified accidental actions

- **Scope**: to illustrate the applicability of the proposed guidelines for the robustness assessment of steel and steel-concrete composite framed buildings
- **The structures included in the worked examples were initially designed to fulfil the requirements for the ultimate limit state (ULS) and serviceability limit state (SLS) for:**
  - **persistent design situation - non-seismic resistant structures**
  - **persistent and seismic design situations - seismic resistant structures** (with additional requirements for damage limitation state DL)
- **The robustness assessment is conducted relying on two types of accidental actions:**
  - **Identified accidental actions**
  - **Unidentified accidental actions**

TYPES OF STRUCTURES

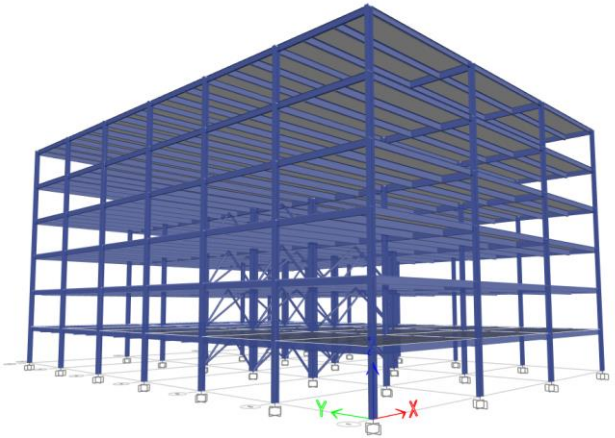
Reference name	Type of structure
SS/NS	**S**teel **S**tructure in **N**on-**S**eismic area
CS/NS*	**C**omposite **S**tructure in **N**on-**S**eismic area
SS/S	**S**teel **S**tructure in **S**eismic area
CS/S	**C**omposite **S**tructure in **S**eismic area

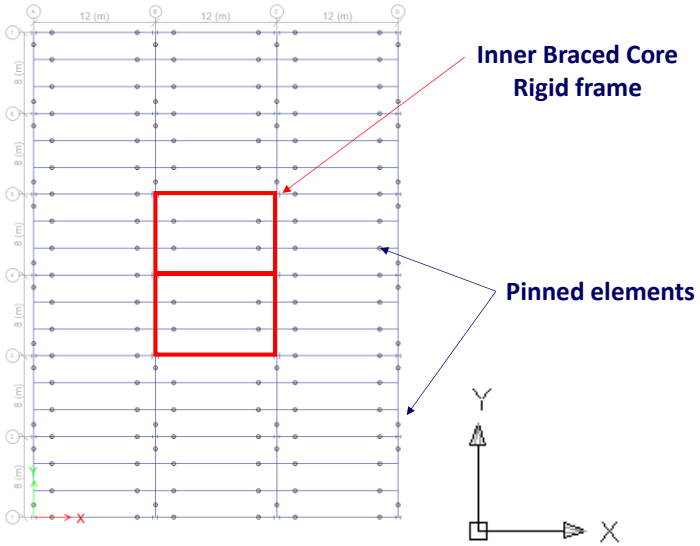
* The structure is designed in two configurations – one with steel columns and one with composite columns. In both cases, the beams and slabs are designed as composite

- **All the structures included in the worked examples fall into the Consequence Class 2b (Upper Risk Group)**

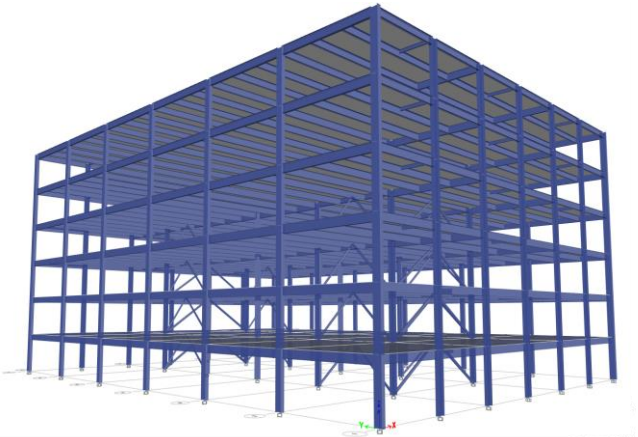
GEOMETRY AND STRUCTURAL SYSTEMS

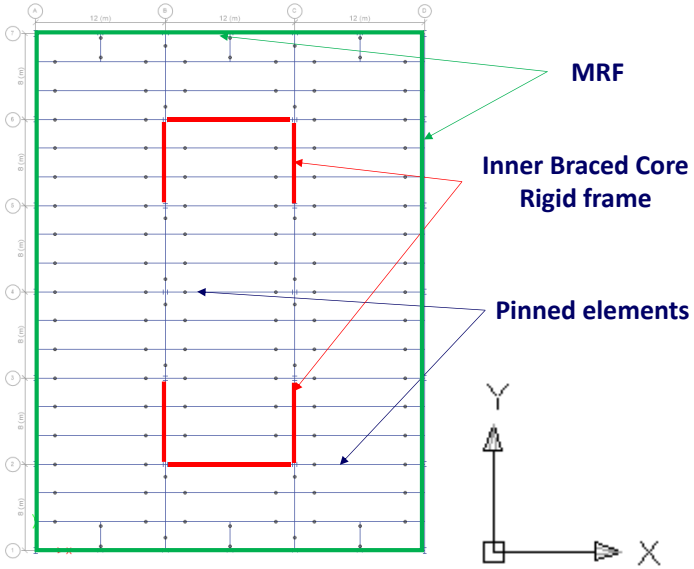
- **Non-seismic area:**
  - 6 storeys of 4.0 m height each
  - 6 bays of 8.0 m in the Y direction
  - 3 bays of 12.0 m in the X direction





- **Seismic area**
  - 6 storeys of 4.0 m height each;
  - 6 bays of 8.0 m in the Y direction
  - 3 bays of 12.0 m in the X direction – internal frames
  - 6 bays of 6.0 m in the X direction – perimeter frames





ACTIONS CONSIDERED IN THE DESIGN

Persistent design situation – for all structures

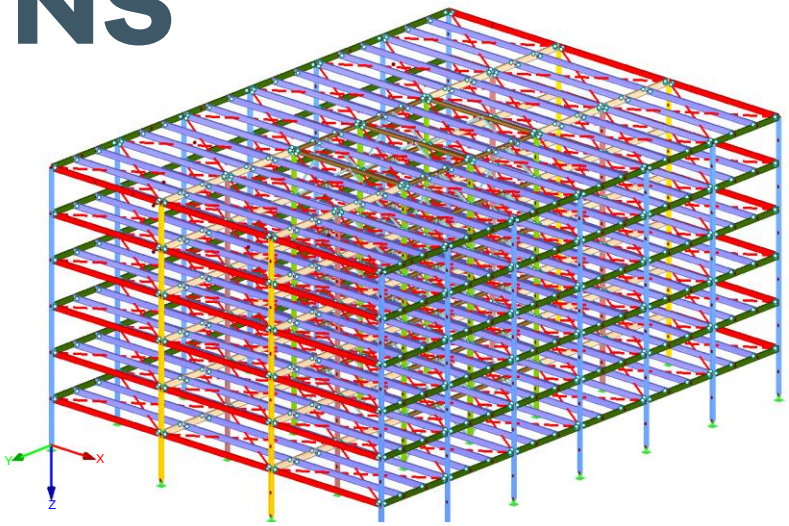
Loads	Type of Structures		
	SS/S & CS/S	**CS/NS**	**SS/NS**
	Location		
	Timis County, RO	Luxembourg	Aachen, DE
Dead load	- Floors: g_k = 5 kN/m² - Façade (supported by the perimeter beams): g_k= 4 kN/m		
Live load	- Live load for office buildings: q_k = 3 kN/m² - Construction load q_k = 1 kN/ m² (general floors and roof)		
WIND			
Wind speed	$v_{b,0}$ = 25 m/s	$v_{b,0}$ = 24 m/s	$v_{b,0}$ = 25 m/s
Equivalent wind pressure	q_b = 0.4 kN/m2	q_b = 0.36 kN/m2	q_p = 0.9 kN/m2*
Terrain category	III	III	"Binnenland"*
Snow load	s_k = 1.5 kN/m2	s_k = 0.5 kN/m2	s_k = 0.85 kN/m2**

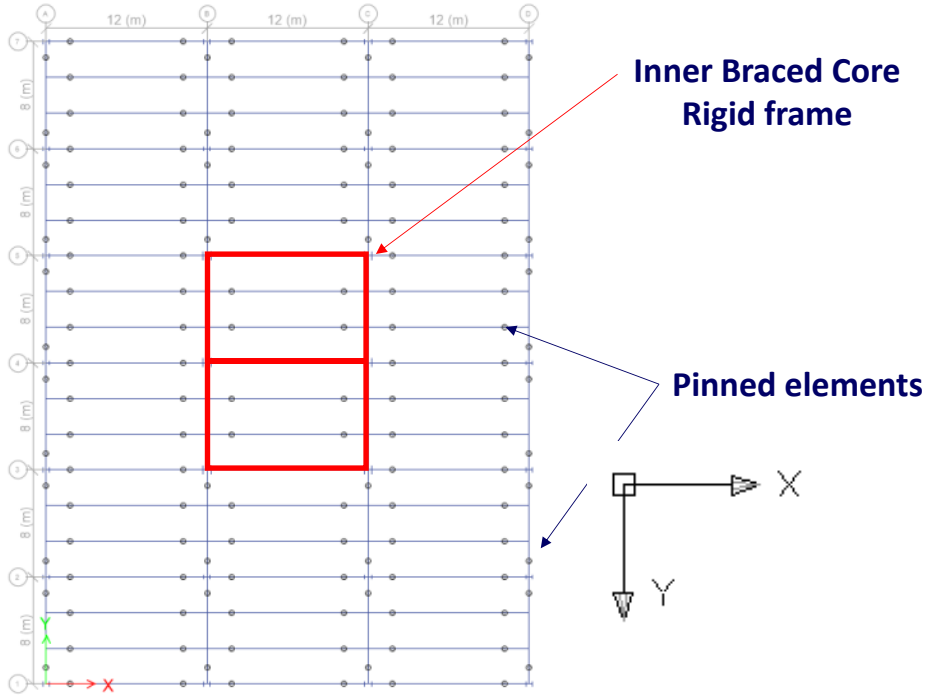
* Simplified wind pressure acc. to DIN EN 1991-1-4/NA Tab. NA.B.3 as commonly used in Germany. This replaces the concept of terrain category. "Binnenland" can be translated with "inland region" or "interior region" and is used to be distinguished from island and coastal regions.

** Snow zone 2 according to DIN EN 1991-1-3/NA

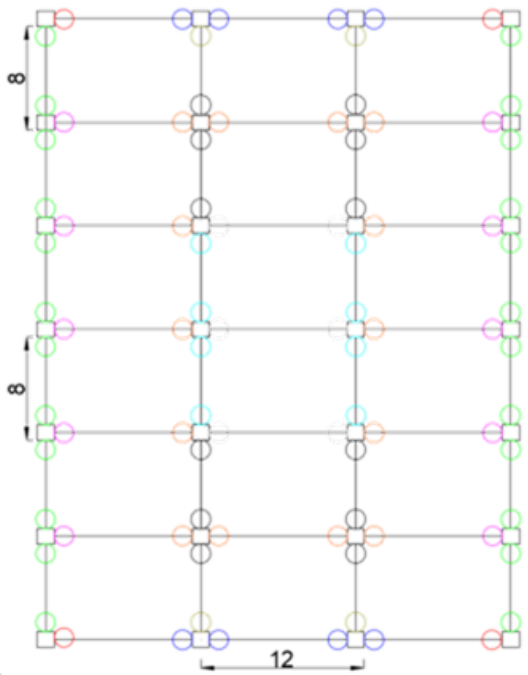
ELEMENT SECTIONS – SS/NS

Element	Section	Steel grade	ID	Utilization factor (UF)	
				Strength	Deflection
Columns Y-facades	HEB 340	S355	1	0.94	-
Columns X-facades	HEB 360	S355	2	0.97	-
Inner columns	HEM 300	S355	3	0.95	-
Beams X-facades	IPE500	S355	A	0.51	0.89
Beams Y-facades	IPE500	S355	A	0.75	0.83
Inner X-beams	IPE550	S355	B	0.62	0.93
Inner Y-beams	IPE600	S355	C	0.87	0.89
Inner core beams	HEA300	S355	D	0.90	0.19
Inner core braces	CHS 219.1x6.3	S355	-	0.90	-

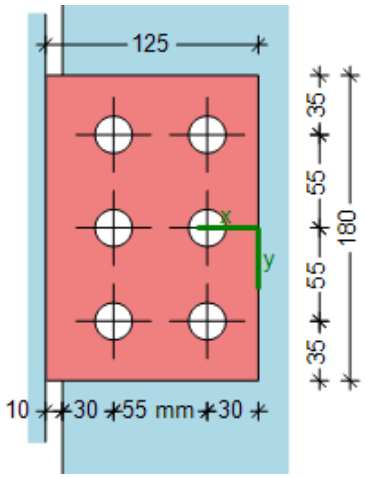




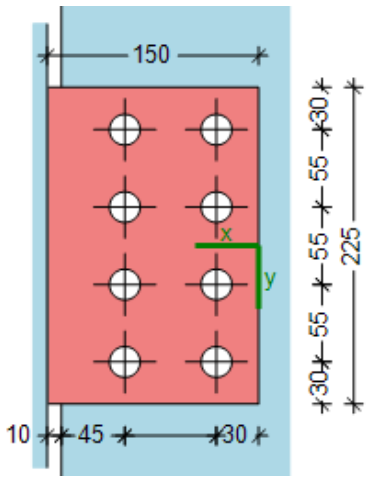
CONNECTIONS – SS/NS



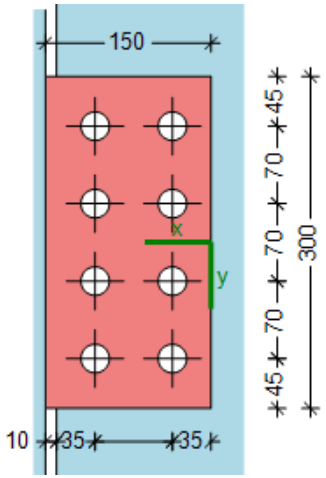
- A1w : IPE500-HEB340 weak axis
- A1s : IPE500-HEB340 strong axis
- A2 : IPE500-HEB360 strong axis
- B1 : IPE550-HEB340 strong axis
- B3 : IPE550-HEM300 strong axis
- C2 : IPE600-HEB360 weak axis
- C3 : IPE600-HEM300 weak axis
- D3w : HEA300-HEM300 weak axis
- D3s : HEA300-HEM300 strong axis



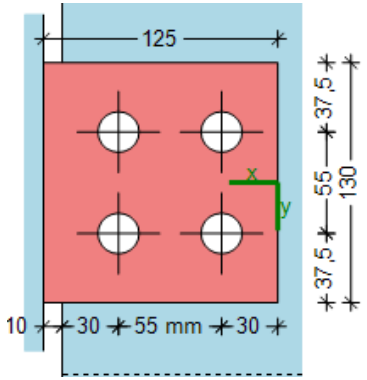
A1s, A2, B1, B3



A1w



C2w, C3w



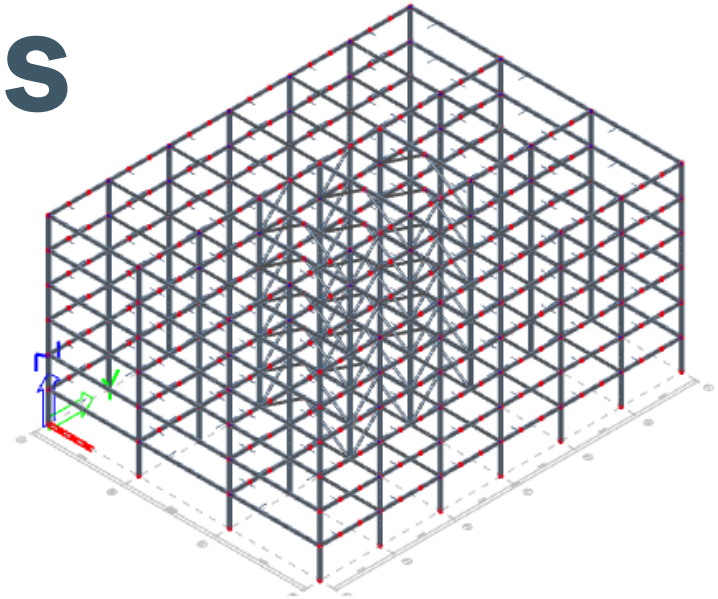
D3s, D3w

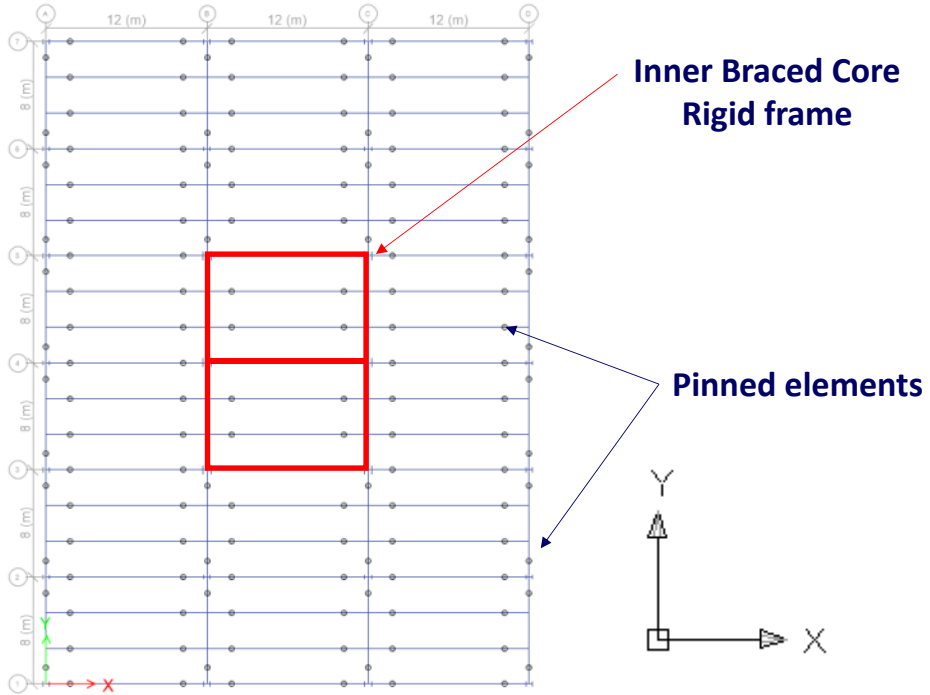
Position s = strong axis w = weak axis	Connection type	Shear resistance (kN)	Moment resistance (kNm)	Failure mode	UF
A1s / A2	Fin plate	196	-	Fin plate in bearing	0.66
A1w	Fin plate	255	-	Fin plate in bearing	0.94
B1 / B3	Fin plate	196	-	Fin plate in bearing	0.92
C2w / C3w	Fin plate	443	-	Fin plate in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

ELEMENT SECTIONS – CS/NS

Element	Type	Section	UF		Critical design (ULS / SLS)
			ULS	SLS	
Beams	Perimeter beams	IPE 450	0.93	0.80	- Final Stage - Crushing concrete flange
					- Final Stage - Deflection
	Interior beams	IPE 500	0.96	0.86	- Final Stage - Bending
					- Final Stage - Deflection
		IPE 360	0.95	0.98	- Final Stage - Bending
					- Final Stage - Deflection
Columns*	Perimeter columns	HD 360x162	0.61	-	- Final Stage - Bending and axial compression
	Interior columns	HD 400x216	0.78	-	- Final Stage - Bending and axial compression
Bracing system	Circular hollow sections	CHS219.1x5.0	0.71	-	- Final Stage - Bending and axial compression

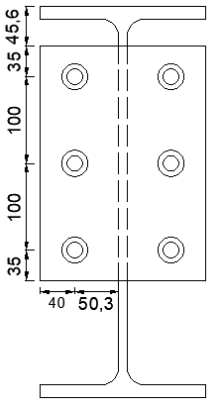
* Composite columns with equivalent properties were also considered



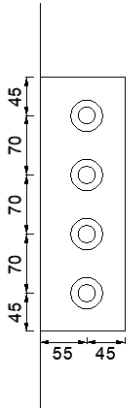


CONNECTIONS – CS/NS

Position	Connection type	Shear resistance (kN)	Moment resistance (kNm)	Failure mode	UF
Perimeter	Header plate	289.38	-	Bolts in shear	0.73
	Fin plate	297.96	-	Bolts in shear	0.71
Internal	Header plate	289.38	-	Bolts in shear	0.64
	Fin plate	265.89	-	Beam web in bearing	0.70



Header plate connection

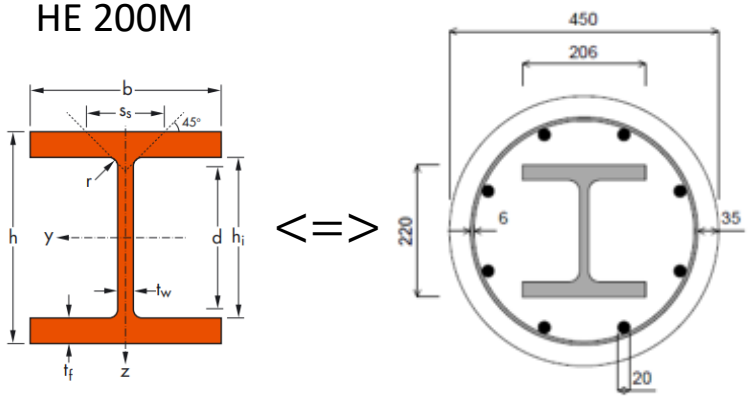


Fin plate connection

COMPOSITE COLUMNS WITH EQUIVALENT PROPERTIES

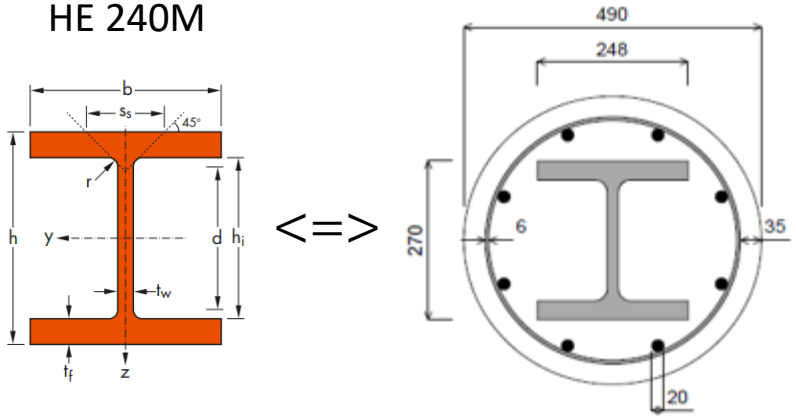
Perimeter columns

HE 200M



Interior columns

HE 240M



"Nelson studs d=19mm, h=100 mm" were used in a single row and a longitudinal and transversal reinforcement of ϕ12//100 provided in the slab.

IDENTIFIED ACCIDENTAL ACTIONS

Approaches for identified accidental actions and their application

	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/NS										
CS/NS										
SS/S										
CS/S										

Recommended strategies for Consequence Class 2b (minimum requirements)

Additional strategies

UNIDENTIFIED ACCIDENTAL ACTIONS

Approaches for unidentified accidental actions and their application

	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/NS						
CS/NS						
SS/S						
CS/S						

Recommended strategies for Consequence Class 2b (minimum requirements)

Additional strategies

2. IDENTIFIED ACCIDENTAL ACTIONS

1. Introduction
2. Identified accidental actions
3. Unidentified accidental actions

- **Impact**
  - Equivalent static approach – CS/NS
- **External explosion**
  - Equivalent SDOF approach – CS/NS
- **Seismic action**
  - Prescriptive approach – SS/NS
  - Advanced numerical analysis (multi-hazard) – SS/S

IDENTIFIED ACTIONS

IMPACT

Equivalent static approach – CS/NS

This example presents the design against impact due to accidental collision of a vehicle.

ACTIONS FOR THE ACCIDENTAL DESIGN SITUATION

- Permanent loads DL
- Live loads LL
- Action due to impact A_{Ed}

COMBINATION OF ACTIONS FOR THE ACCIDENTAL DESIGN SITUATION

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

IMPACT SCENARIOS

- Include perimeter columns along traffic lines;
- In the example, both long facade (along vertical traffic lane) and short façade (along horizontal traffic lane) are exposed.

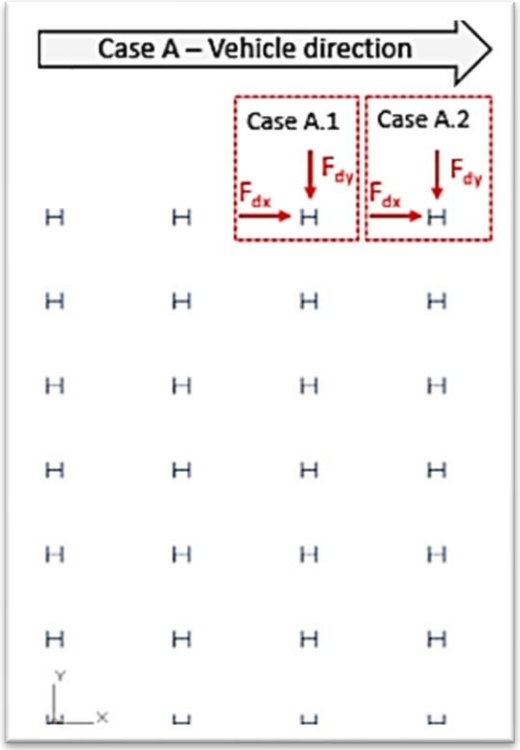
THE IMPACT LOADS

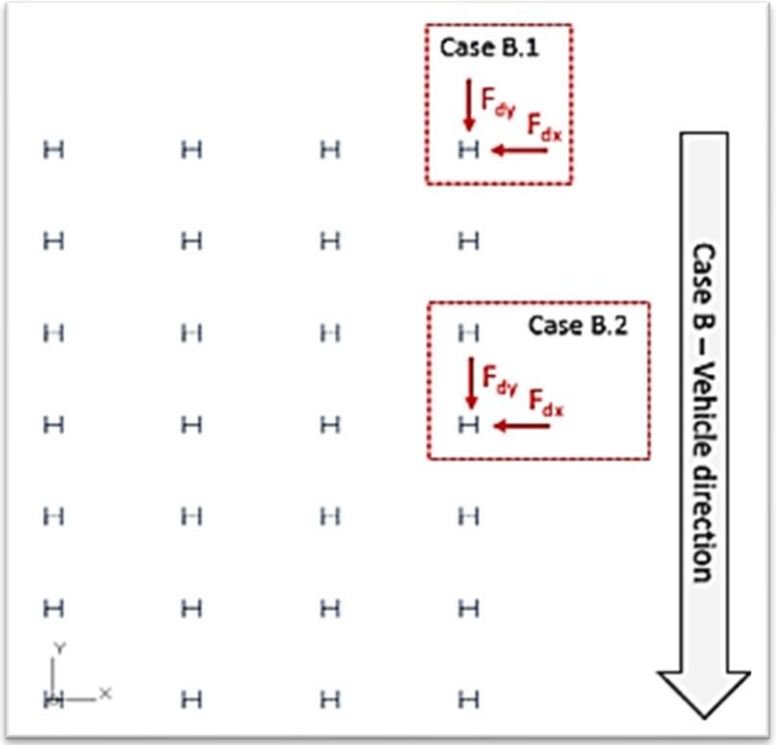
- Calculated using data from Table 4.1 of (EN 1991-1-7 2006), considering the case: *Motorways and country national main roads.*

IMPACT ASSUMPTIONS

- Exposed columns: ground floor (A.1 / A.2 / B.1 / B.2);
- Impact point height: 1.5m;
- Impact forces:

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

- A **linear elastic analysis** was performed on the full 3D structural model using the software SCIA®.
- The design of the composite columns was performed with the A3C® software.

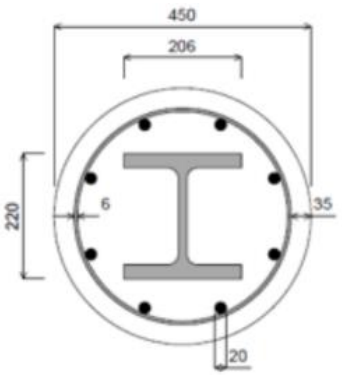
RESULTS

Results of linear static analysis for impact on standard 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

Results of linear static analysis for impact on composite columns

Case	Loading		Upper and bottom supports	UF (-)
	F_{dx} (kN)	F_{dy} (kN)		S355
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



STANDARD STEEL COLUMNS

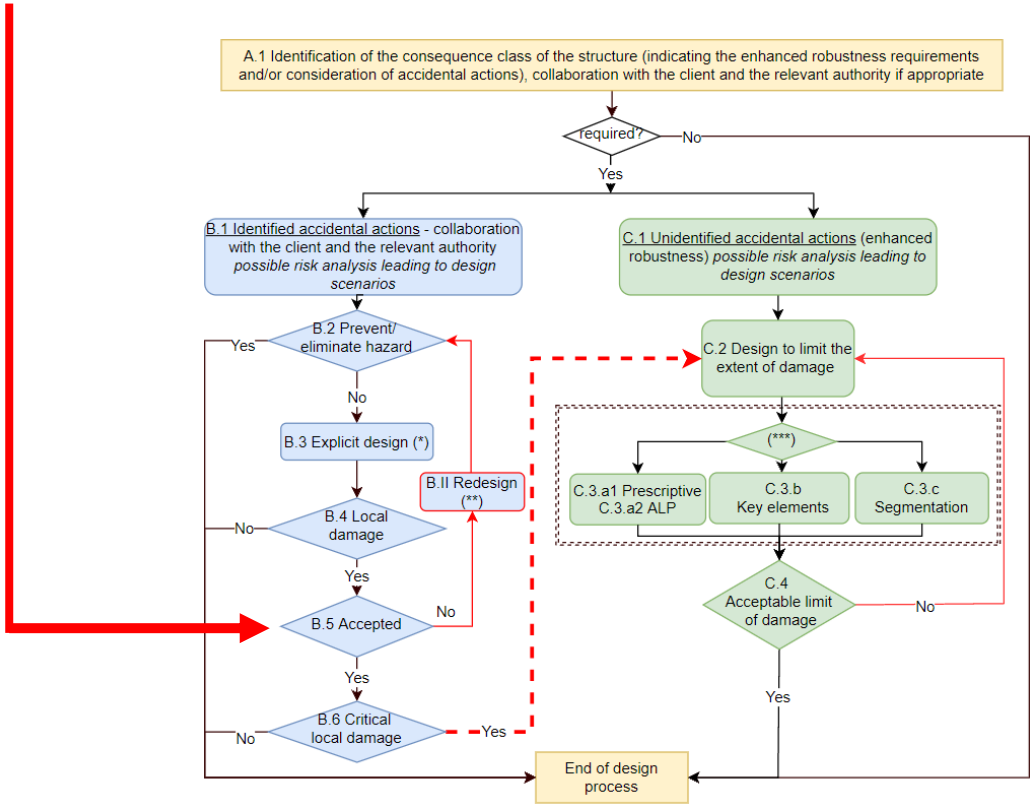
- For S355 columns, the yield strength is exceeded for both pinned and fixed conditions.
- Using S460 steel grade, a considerable improvement is observed.

COMPOSITE STEEL-CONCRETE COLUMNS

- UFs are substantially higher due to the pre-design of the sections and supporting conditions. Using a static approach for the impact analysis, the steel profile takes 65% to 70% of the bending resulted from the impact➔ higher UFs.

CONCLUSIONS

- To mitigate the consequences of impact:
  - The impact may be prevented or eliminated by using preventive measures;
  - Adopt higher steel grade for columns / increase section size;
  - Orient the columns to maximize the impact resistance / design bottom connections as fixed;
  - More advanced methods of analysis may be used to assess more accurately the capacity.



IDENTIFIED ACTIONS

External explosion

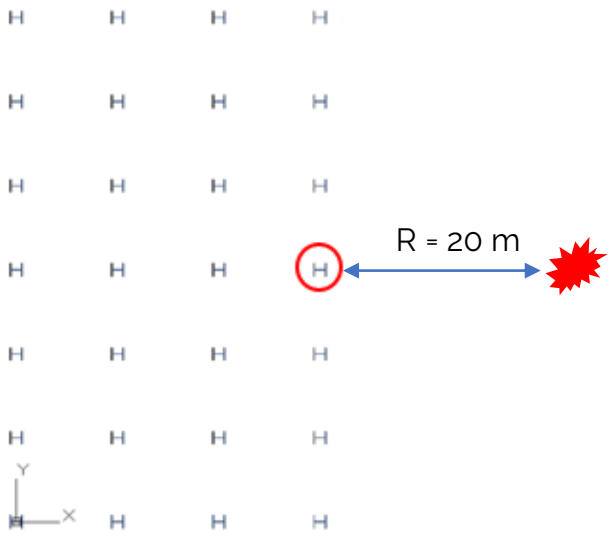
Equivalent single-degree-of-freedom approach – CS/NS

ACCIDENTAL ACTION

- Blast action A_{Ed}.

DEFINITION OF BLAST SCENARIO

- The blast acts on a perimeter column located at the ground floor at the middle of the long façade.
- Standoff distance R = **20 m**
- Explosive charge W = **100 kg of TNT**



STRUCTURAL ANALYSIS

- A linear elastic analysis is performed using the equivalent single-degree-of-freedom approach.

BLAST LOADING PARAMETERS (1)

- The scaled distance and angle of incidence are calculated based on:
  - Charge mass;
  - Standoff distance;
  - Point of detonation (from the ground surface).

- 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 → $Z = \frac{R}{W^{\frac{1}{3}}} = \frac{20}{100^{\frac{1}{3}}} = 4.309\ \frac{m}{kg^{\frac{1}{3}}}$

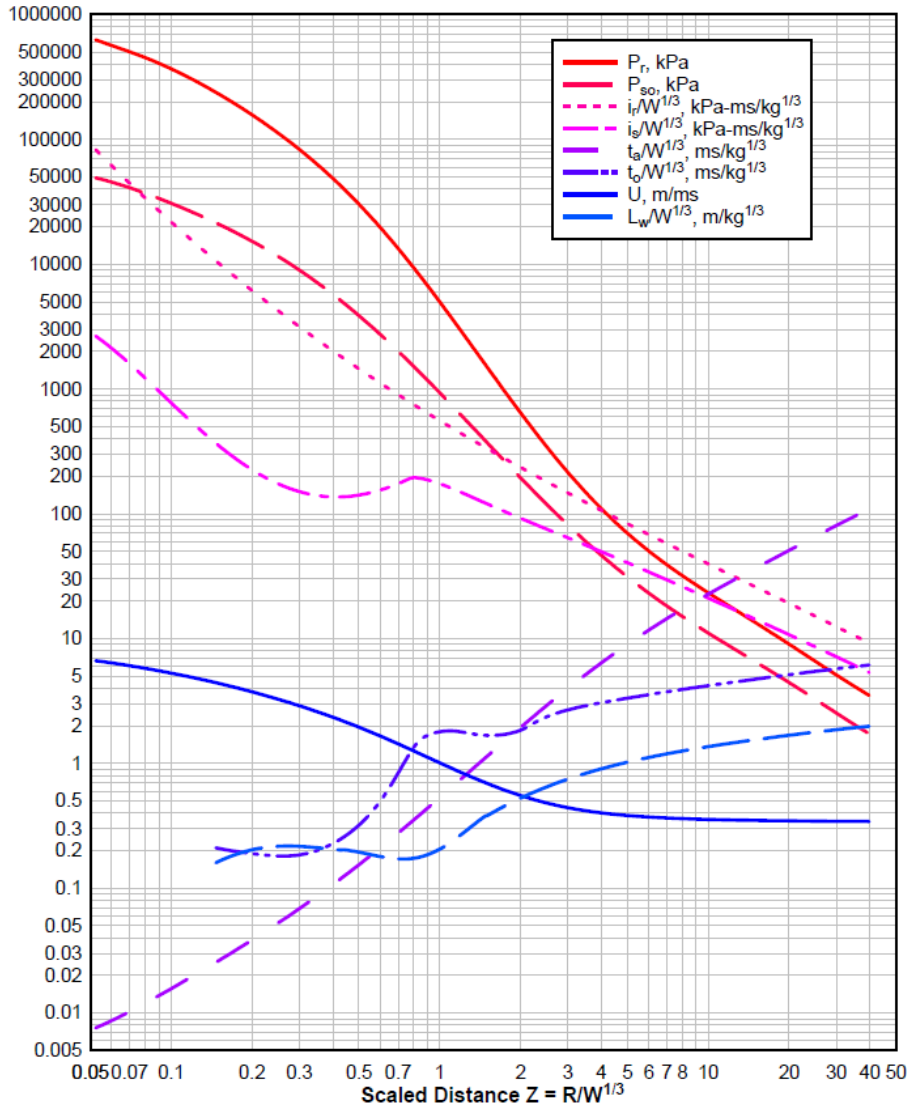
- Distance from blast source → $R_h = \sqrt{R^2 + H_c^2} = \sqrt{20^2 + 1^2} = 20.025\ m$

- Angle of incidence → $\alpha_i = tan^{-1}\left(\frac{H_c}{W^{\frac{1}{3}}}\right) = tan^{-1}\left(\frac{1}{100^{\frac{1}{3}}}\right) = 12.158^o$

BLAST LOADING PARAMETERS (2)

- Next, using the chart for free-air bursts, the blast loading parameters are determined:

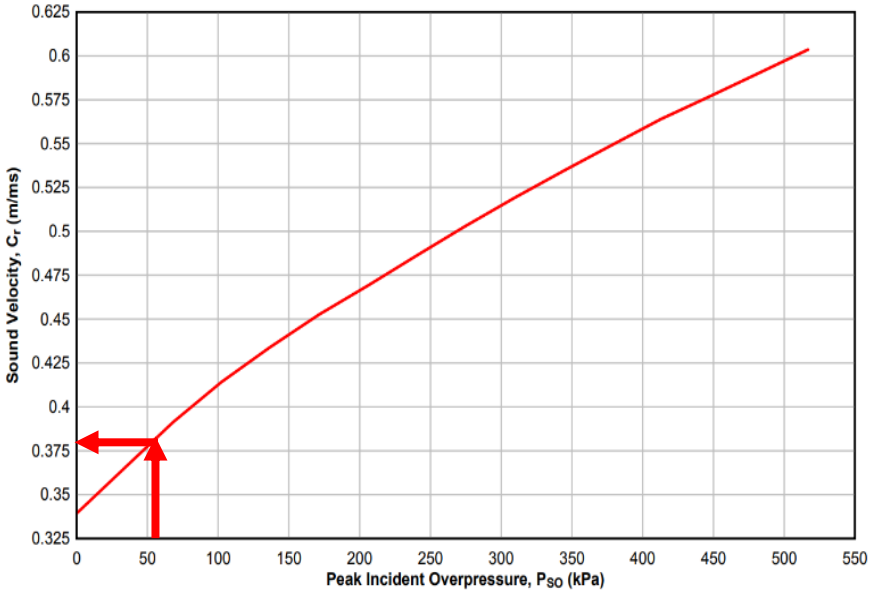
- Incident pressure → $P_{so} = 56.44\ kPa$
- Incident impulse → $I_s = 313.71\ kPa.ms$
- Reflected pressure → $P_r = 137.37\ kPa$
- Reflected impulse → $I_r = 688.09\ kPa.ms$
- Time of arrival → $t_a = 30.29\ ms.W^{\frac{1}{3}} = 140.59\ ms$
- Positive phase duration → $t_0 = 16.49\ ms$
- Blast wavelength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



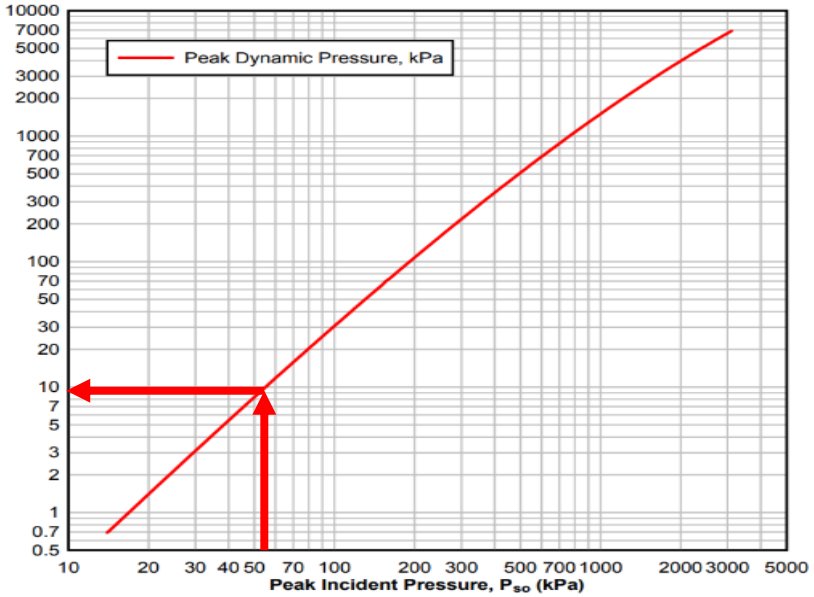
(JRC, 2013)

BLAST LOADING PARAMETERS (3)

- Considering the incident pressure defined previously (P_{so}), the sound velocity (C_r) and the peak dynamic pressure (q) are obtained using the following charts:



(JRC, 2013)



(JRC, 2013)

- Sound velocity → $C_r = 0.38\ \frac{m}{ms}$
- Peak dynamic pressure → $q = 8.5\ kPa$

BLAST LOADING PARAMETERS (4)

- Afterwards, the fictitious reduced time intervals are computed:

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 → $t_{rf} = 2\frac{I_r}{P_r} = 2 \times \frac{688.09}{137.37} = 10.02\ ms$

- Finally, the clearing time and peak pressure acting on the wall are determined:

- Height of the element → $h_s = 4\ m$
- Width of the wall → $w_s = 4\ m$
- 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 → $P = P_{so} + q.C_D = 56.44 + 8.5 \times 1 = 64.94\ kPa$

BLAST LOADING PARAMETERS (5)

*Note: The most unfavorable situation is considered in the design - the **largest value for pressure** (reflected pressure) and **smallest time duration** (fictitious duration for reflected wave) resulting in a covering approximation for the pressure-impulse loading.*

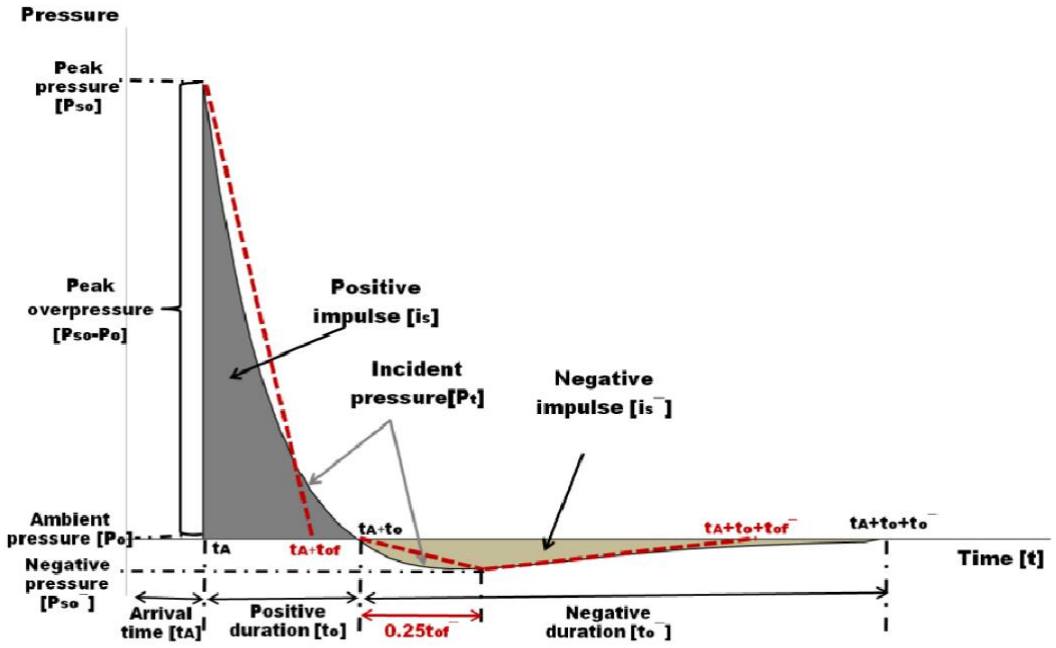


Figure 18: Substitution of actual incident pressure curve by triangular pulses and definition of relevant fictitious times.

(JRC, 2013)

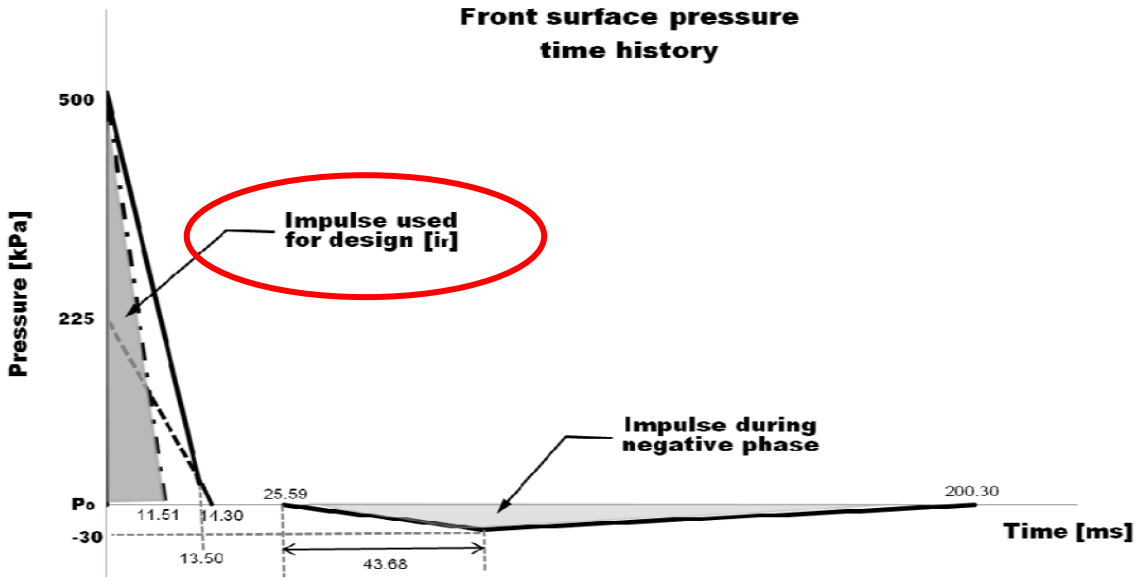


Figure 28: Blast pressure time history at front wall of the structure.

(JRC, 2013)

SINGLE DEGREE OF FREEDOM (SDOF) APPROACH (1)

- The column is transformed in an equivalent SDOF system. Two solutions were calculated: **steel columns** and **composite columns**.
- The first step consists of calculating the load caused by the reflected pressure on the column.
- A width of 5m was assumed for the panel attached to the column.

- 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 → $t_{rf} = 10.02\ ms$
- Selfweight of the column (Steel; Composite) → $G_c = (1.834\ ;\ 4.721)\ \frac{kN}{m}$

- Distributed load from the blast on the column → $F_d = P_r w_p = 137.37 \times 5 = 686.85\ \frac{kN}{m}$
- Point load from the blast on the column → $F_p = F_d h_c = 686.85 \times 4.0 = 2747.4\ kN$

- Since the analysis is static, a DLF is used to account for dynamic effects.
- The first iteration is performed using a **DLF** of **1.45** to amplify the loading. Additionally, to account for the strain rate effects, the yield strength of the material may be affected by a **DIF** of **1.2**.

SINGLE DEGREE OF FREEDOM (SDOF) APPROACH (2)

- To determine the response of the SDOF system in terms of ultimate resistance R_m, different transformation factors (loading K_L, mass K_M, etc.) for beams and one-way slabs are used.

- Column stiffness (Steel; Composite) ↓

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

- Maximum resistant moment (Steel; Composite) ↓

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

- Effective mass (steel; composite) ➔

$$M_e = \frac{G_c . h_c . K_M}{g} = \frac{(1.834\ ;\ 4.721) \times 4 \times 0.50}{9.81} = (374.03\ ;\ 962.82)\ kg$$

- Effective stiffness (steel; composite) ➔

$$K_e = K_c K_L = (130762.8\ ;\ 53221.04) \times 0.64 = (83688.19\ ;\ 34061.47)\ \frac{kN}{m}$$

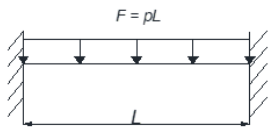
- Natural period of vibration (steel; composite) ➔

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

- Ratio between the fictitious duration of the reflected wave and the natural period (steel; composite) ➔

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

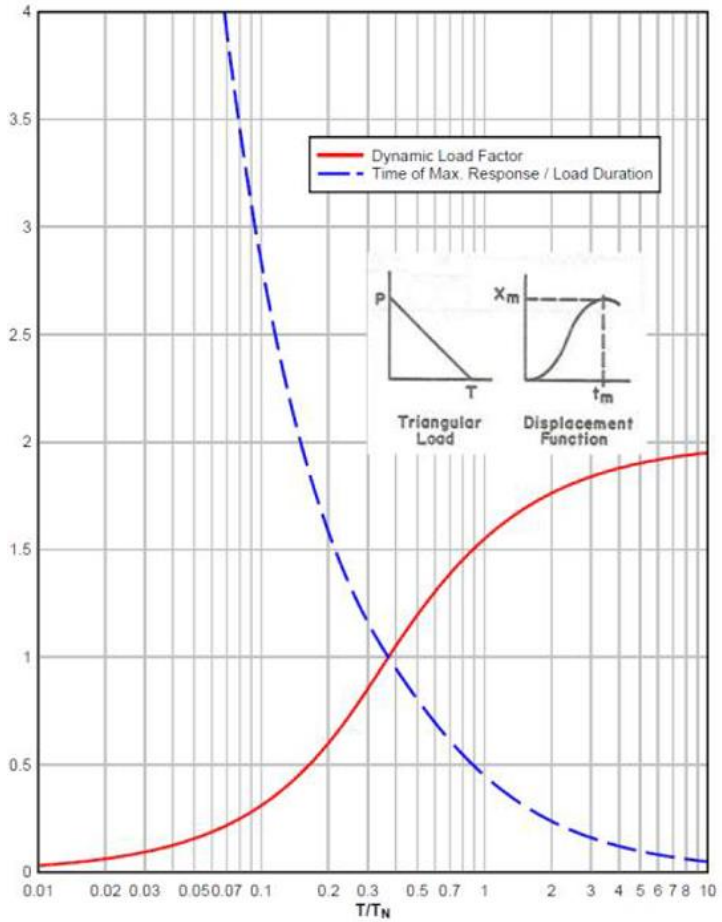
Table 67. Transformation Factors for Beams and One-way Slabs double fixed beam (Biggs and Biggs, 1964)

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	....	$0.38R_m+0.12F$

- Loading factor ➔ $K_L = 0.64$
- Mass factor ➔ $K_M = 0.50$

SINGLE DEGREE OF FREEDOM (SDOF) APPROACH (3)

- Based on the previous data, the natural period of vibration and the ratio between the reflected time duration and period of vibration are obtained. Function of this ratio, a second iteration for the DLF may be performed.



(DoD, 2008)

- Second iteration (steel; composite) ➔ $DLF = (1.30\ ;\ 1.80)$

- Maximum applied moment (steel; composite) ➔ $M_{max} = \frac{F_p . h_c}{8} DLF = \frac{2747.4 \times 4}{8} \times (1.30\ ;\ 1.80)$
$= (1785.81\ ;\ 2472.66)\ kNm$

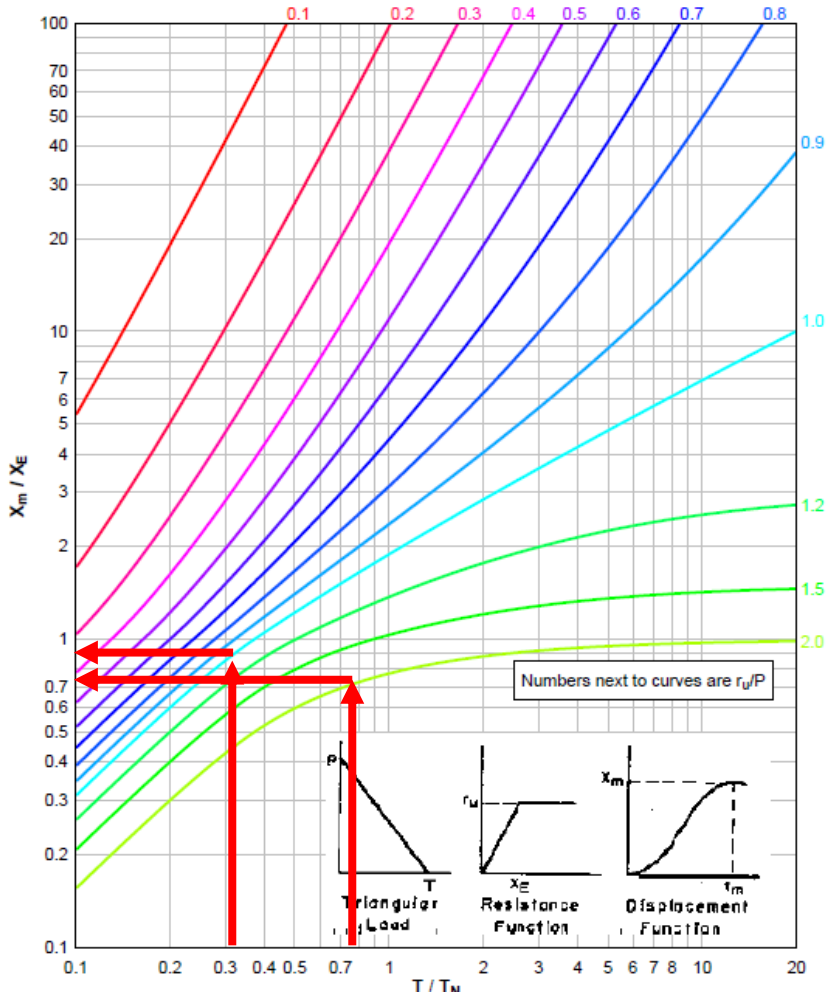
- Resistance force (steel; composite) ➔ $R_m = \frac{8(2M_{Rd})}{h_c} = \frac{8 \times 2 \times (1347.01\ ;\ 759.42)}{4}$
$= (5388.05\ ;\ 3037.7)\ kN$

RESULTS (1)

- The ratio, the maximum resistance and point load is used to determine the ductility demand μ using the following chart.

Ratio (Steel; Composite) $\frac{R_m}{F_P} = (1.96\,;1.11)$ $\frac{t_{rf}}{T_c} = (0.75\,;0.30)$

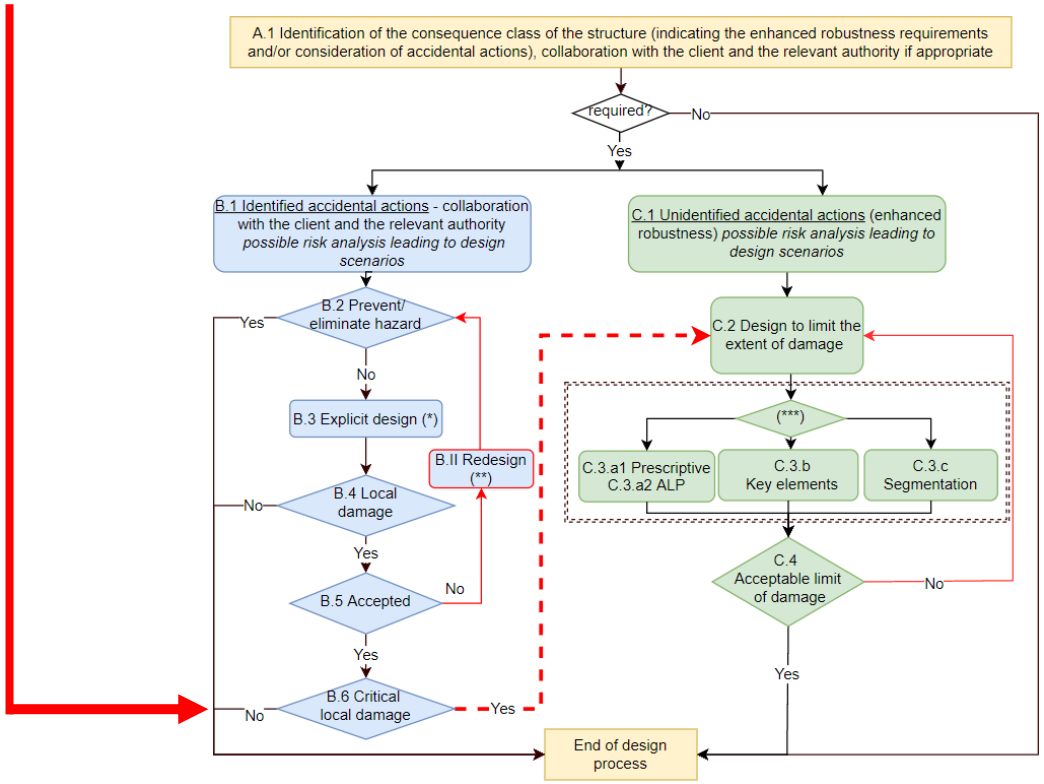
- A ductility demand (Steel ; Composite) μ_1=(0.80 ; 0.95) (χ_M/χ_E) was obtained.
- Consequently, after the elastic displacement is determined, maximum displacements of 51.51 mm (Steel) and 84.72 mm (Composite) mm were obtained.
- The process is performed similarly for the maximum response duration.

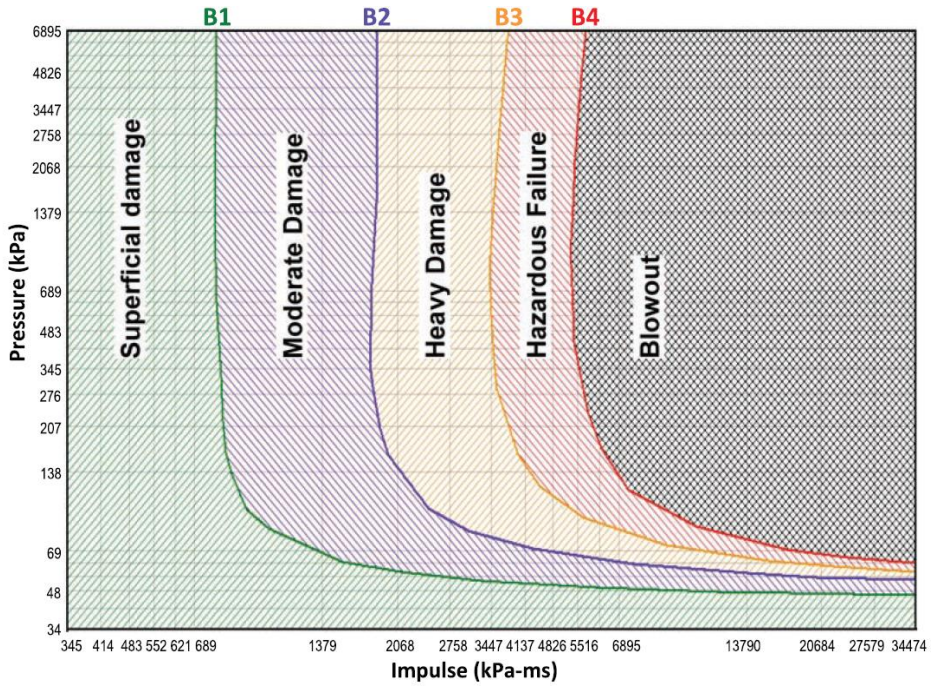


(DoD, 2008)

RESULTS (2)

- The ductility demand will be compared with a capacity to assess if damage is acceptable.
- Using the pressure impulse relationships and response limits, one may establish a design objective.
- B1 objective (superficial damage) was chosen, and a ratio $\frac{\mu_1}{\mu_{max}} = (0.80\,;0.95)$ was obtained.
- Consequently, the damage may be considered acceptable.





(CSA S850-12)

Table 5. Example of response limits for hot-rolled structural steel (CSA, 1991)*

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	-

IDENTIFIED ACTIONS

Seismic action

Prescriptive method – SS/NS

- The structure considered in this example has been designed for ULS/SLS conditions only (persistent design situation). No calculations were conducted with respect to any seismic action. Consequently, the seismic action is considered as exceptional.
- In practice, simple recommendations can be followed when the seismic action is less demanding than the wind-based design. This is mainly done for low-rise buildings to optimize engineering costs.

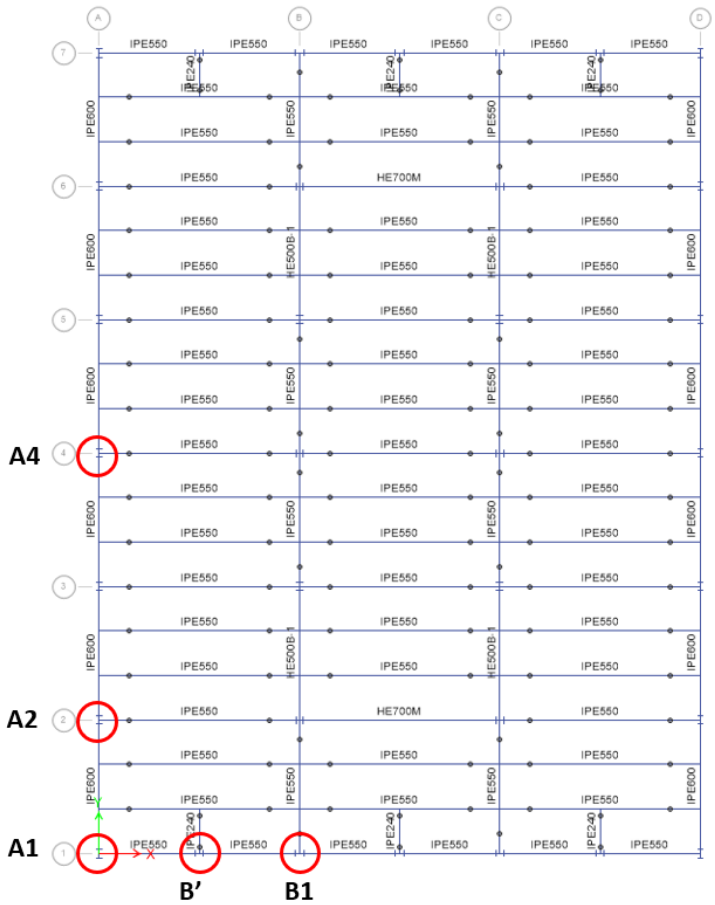
RECOMMENDATIONS

- Building configuration:
  - ✓ Low height to base ratio
  - ✓ 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 (symmetrical arrangements, regularity in plan and elevation)
- Vibration control (base isolators, dampers – passive, active or semi-active)
- Strength and stiffness
- Ductility
  - ✓ Cross-section class (e.g., HEA300 class 3 sections for the beams should be replaced with class 1 HEB300)
  - ✓ Joint typology (pinned joints could be replaced by ductile semi-rigid joints, allowing the formation of plastic hinges in the joints and dissipating part of the seismic induced energy)

IDENTIFIED ACTIONS

Seismic action

Advanced numerical analysis (multi-hazard) - SS/S



- **ACTIONS FOR THE ACCIDENTAL DESIGN SITUATION**
  - Permanent loads DL
  - Live loads LL
  - Seismic action A_{Ed} corresponding to ULS
- **COMBINATION OF ACTIONS FOR THE ACCIDENTAL DESIGN SITUATION**

$$DL + 0.3 \times LL + A_{Ed}$$

- **HAZARD SCENARIO**

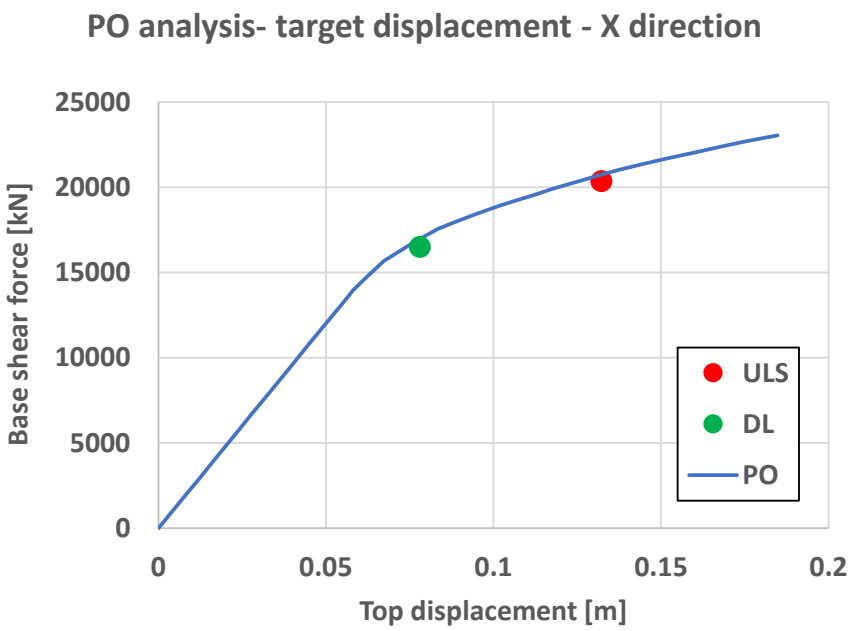
After the structure is subjected to an earthquake, a column can be lost, thus making the structure vulnerable to subsequent hazards. In the following, column loss approach is applied to verify the capacity of the structure to resist progressive collapse.

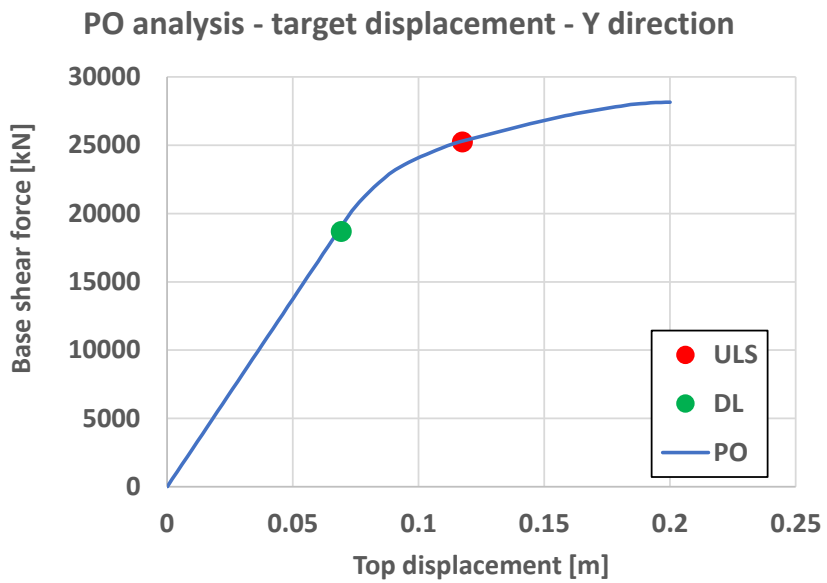
Step 1: Seismic analysis – The structure is subjected to a design level earthquake.

Step 2: Column loss scenarios: Lost columns are located at A1, A2, A4, B1, B' – they are assumed to be lost one at a time.

STRUCTURAL ANALYSIS

- The seismic analysis is performed using push-over analysis.
- The damage assessment is done using the N2 method (EN 1998).
- After the gravity loads are applied, the structure is subjected to monotonically increasing lateral forces.
- Minimum two patterns of lateral forces should be used:
  - Uniform
  - Variable
- The results are reported for the distribution with higher demands.

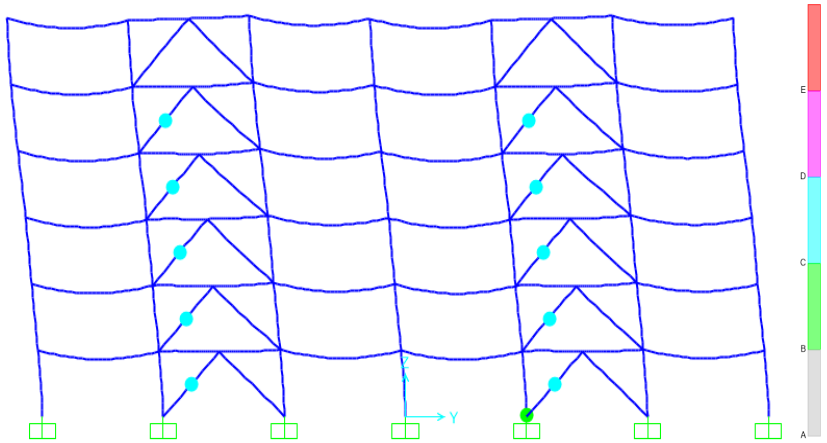




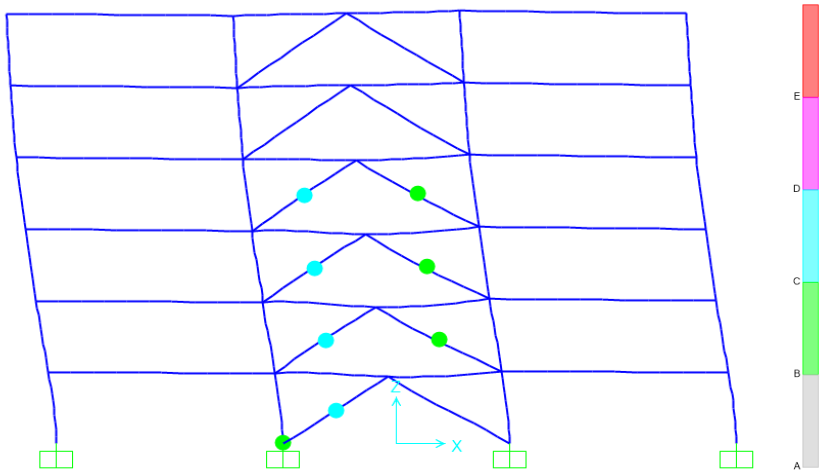
SEISMIC DEMANDS

- To evaluate the seismic demands for ULS, the structure is pushed to its target top displacement D_t.
- No plastic hinges develop in perimeter moment resisting frames in neither X nor Y direction at ULS, but only in the braced frames.

PLASTIC MECHANISM AT ULS TARGET DISPLACEMENT



Longitudinal braced frame



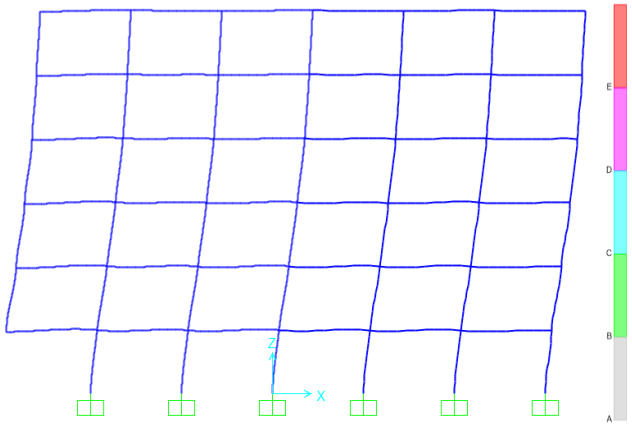
Transversal braced frame

COLUMN REMOVAL AFTER THE EARTHQUAKE

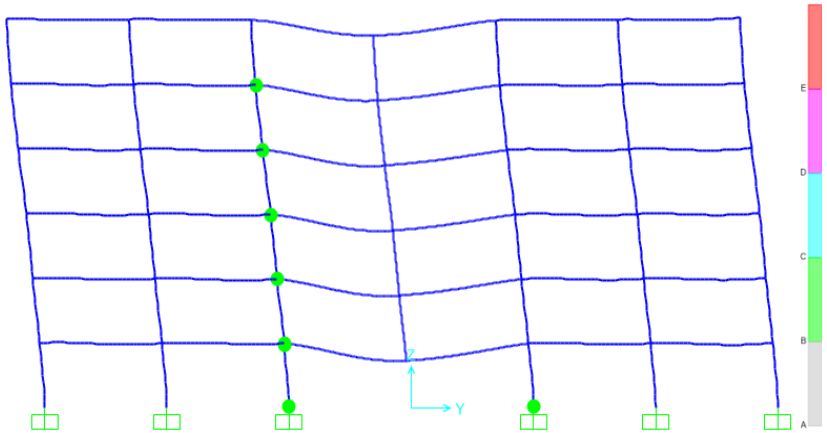
- The assessment of progressive collapse resistance is done using the alternative load path (**ALP**) method and nonlinear dynamic procedure (**NDP**), in accordance with the UFC 4-023-03 guidelines.
- The gravity loads were applied in first stage; then, in the second stage, the element is removed almost instantaneously (removal duration of 0.005 seconds).

RESULTS

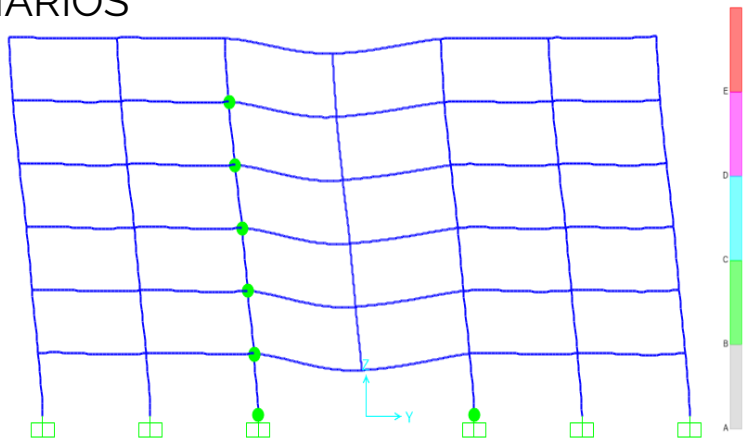
PLASTIC MECHANISM AFTER COLUMN REMOVAL FOR THE CONSIDERED SCENARIOS



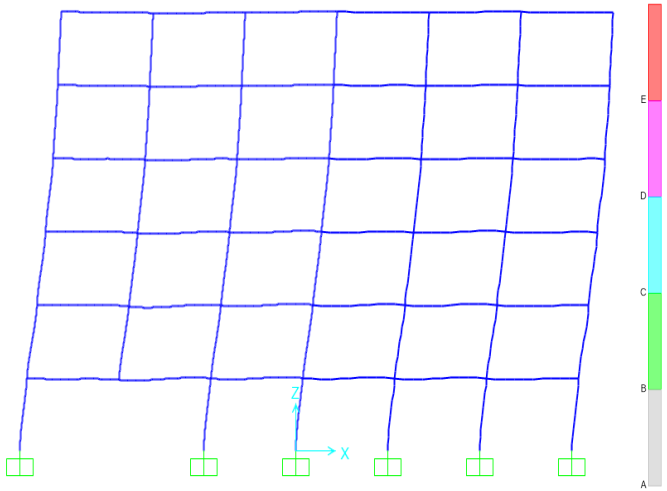
Case A1



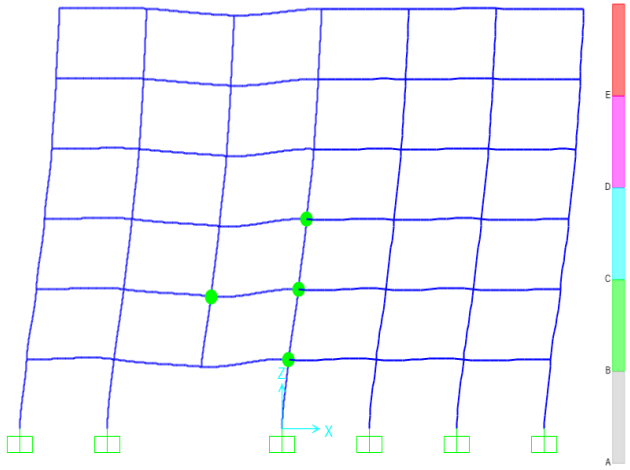
Case A2



Case A4

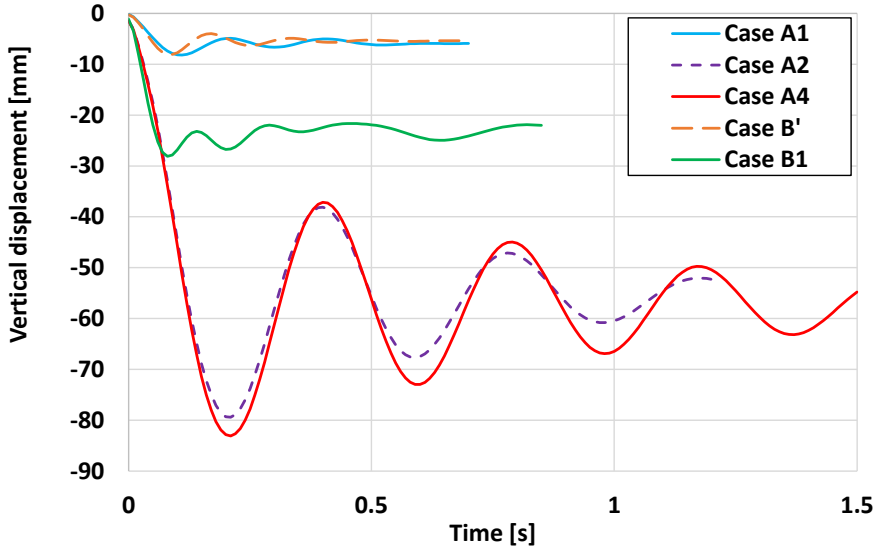


Case B'



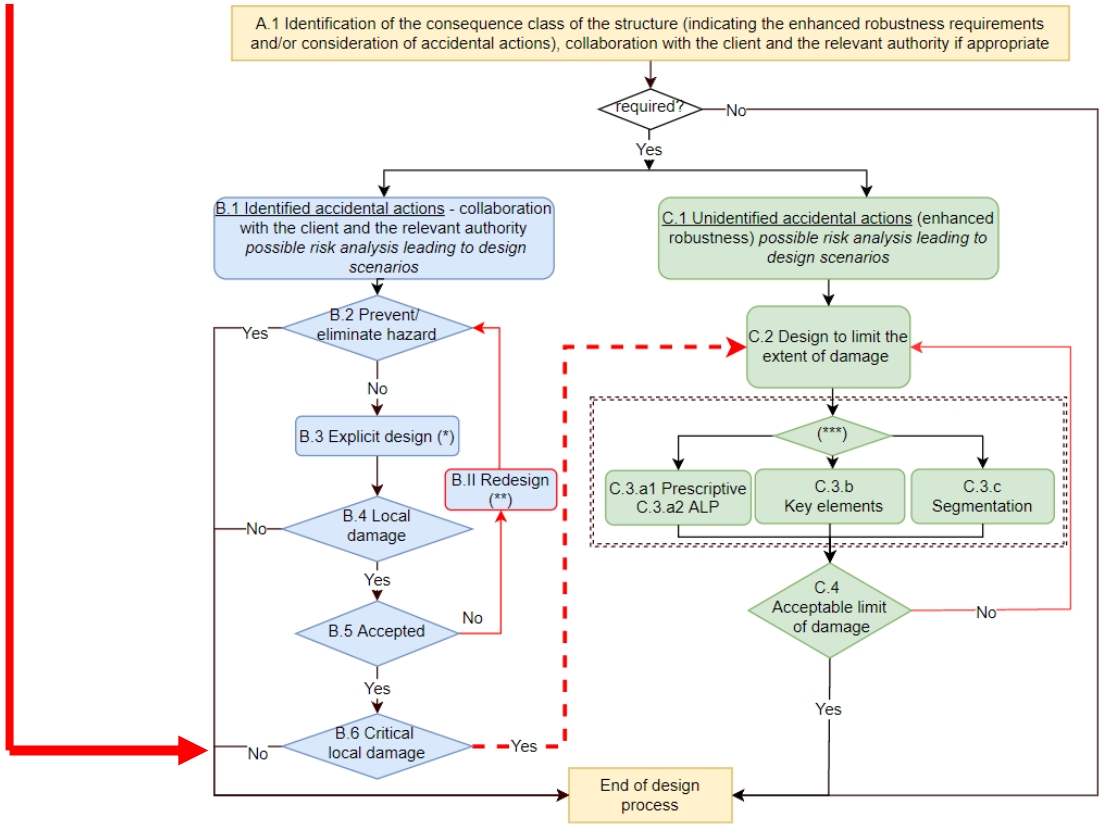
Case B1

TIME HISTORY RESPONSE FOR COLUMN LOSS SCENARIOS



CONCLUSIONS

- 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.



2. UNIDENTIFIED ACCIDENTAL ACTIONS

1. Introduction
2. Identified accidental actions
3. Unidentified accidental actions

- **Alternate load path method (ALPM)**
  - Prescriptive approach – SS/NS
  - Analytical approach – SS/NS
  - Full numerical approach – SS/NS

UNIDENTIFIED ACTIONS

Alternate load path method (ALPM)

Prescriptive method – SS/NS

ACTIONS CONSIDERED

- Permanent loads DL
- Live loads LL

Tying forces according to EN 1991-1-7

$T_i = 0.8(g_k + \psi q_k)sL$	*or*	*75 kN, whichever is greater*
$T_p = 0.4(g_k + \psi q_k)sL$	*or*	*75 kN, whichever is greater*

HORIZONTAL TYING FORCES

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**

VERTICAL TYING FORCES

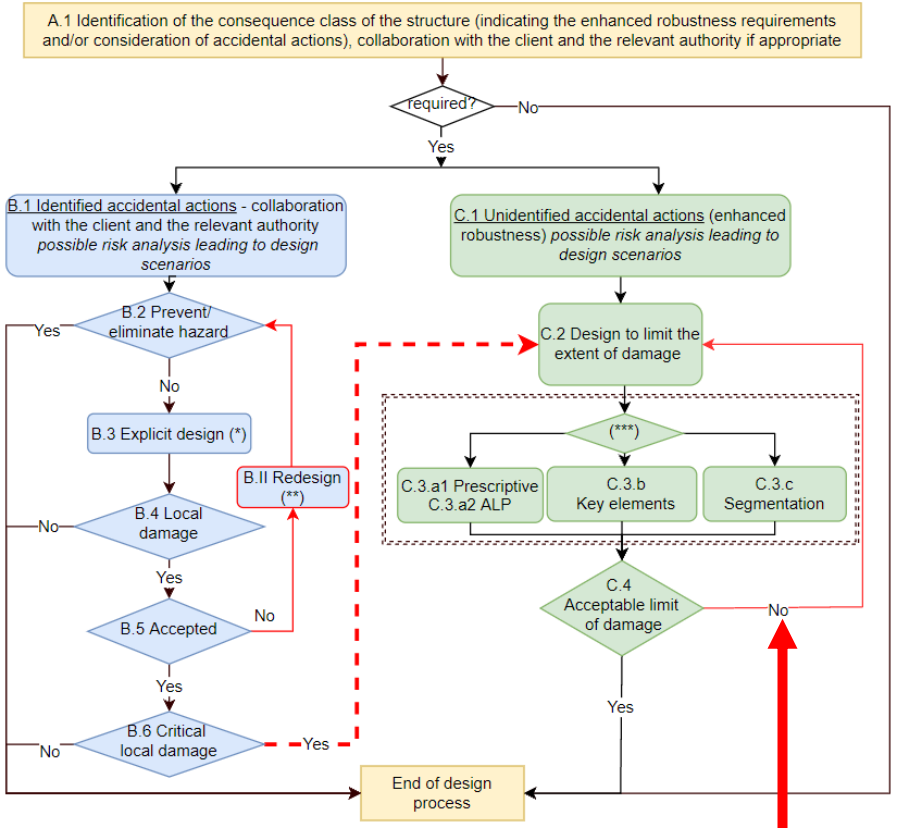
External tie (HEB360)		Internal tie (HEM300)	
s	8 m	s	8 m
L	12 m	L	12 m
ψ	0,5	ψ	0,5
g_k	5 kN/m2	g_k	5 kN/m2
q_k	3 kN/m2	q_k	3 kN/m2
g_{IPE600}	1,22 kN/m	g_{IPE600}	1,22 kN/m
g_{IPE500}	0,907 kN/m	g_{IPE550}	1,06 kN/m
g_{IPE550}	1,06 kN/m	g_{HEM300}	2,38 kN/m
g_{HEB360}	1,42 kN/m	h	4 m
h	4 m	n IPE550	4
n IPE550	1,5		
g_k facade	4 kN/m		
T_e	**400,5 kN**	**T_i**	**694,2 kN**

VERIFICATION FOR TYING FORCES

MEMBER VERIFICATIONS for horizontal tying forces according to EN 1993-1-1 ➔ all members check **OK**

JOINTS VERIFICATIONS

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 JOINTS

B1/B3

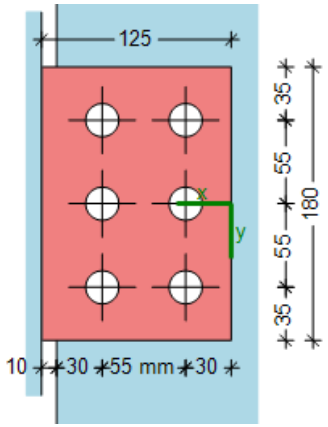
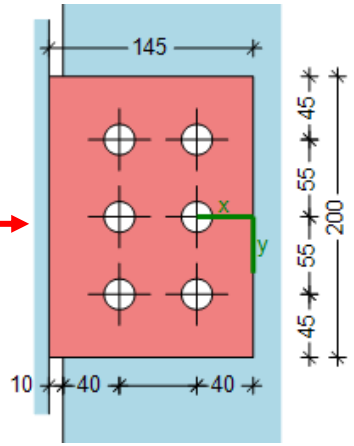
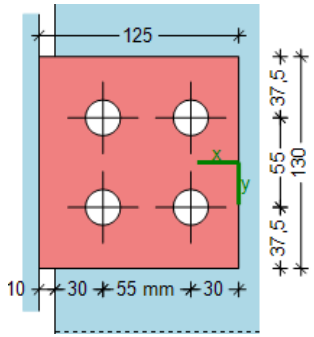


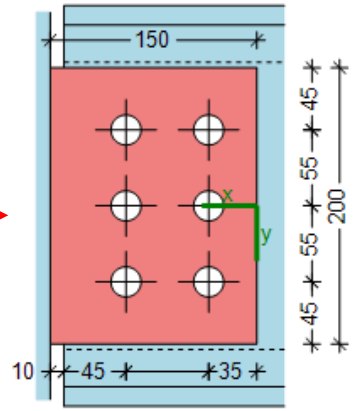
Plate geometry



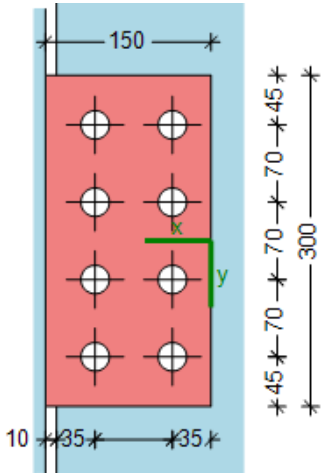
D3s/D3w



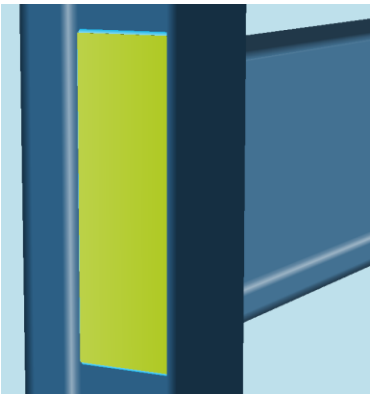
2 bolts added



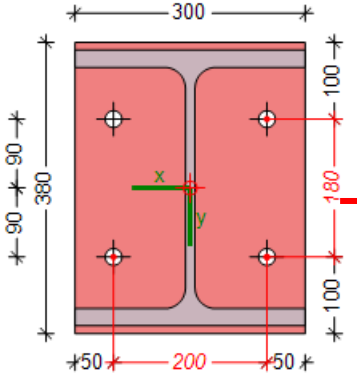
C2w



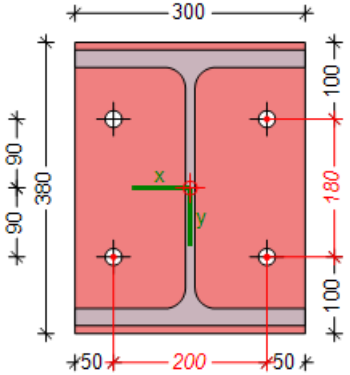
Additional column web plate



3-3



M20->M24
EP 15->20 mm



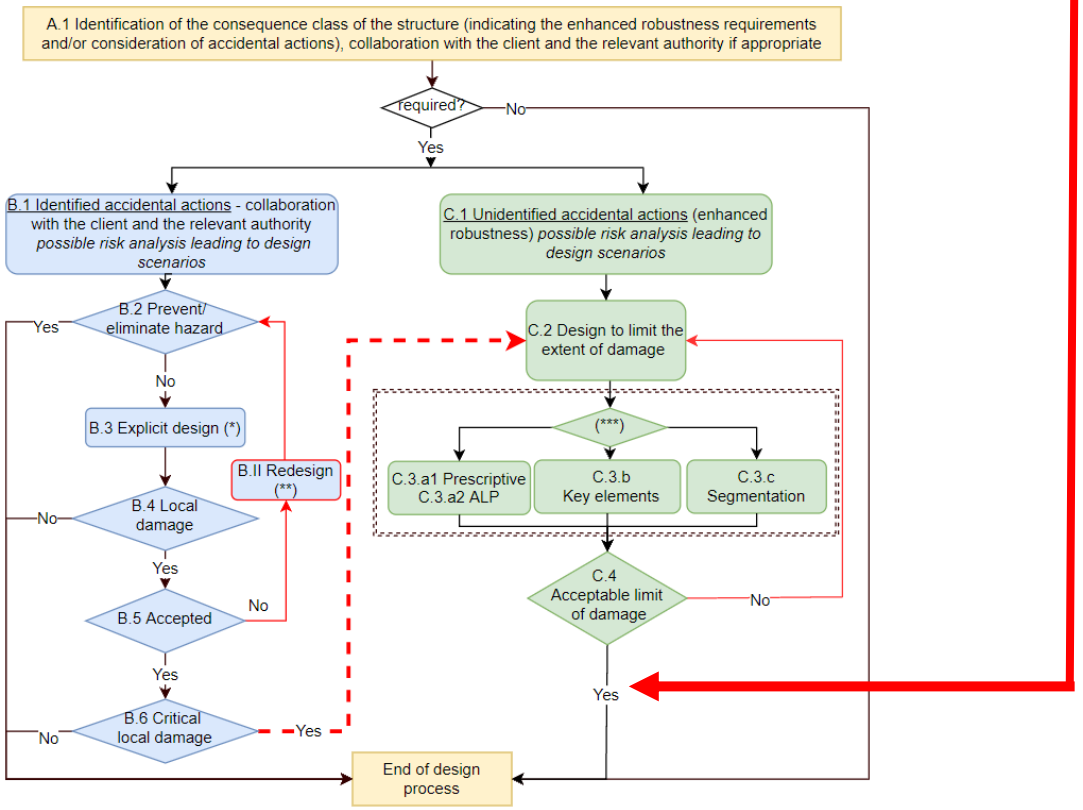
REDESIGNED JOINTS VERIFICATIONS FOR TYING FORCES

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

Check of the D3s/D3w joints is exceeded by 3%

↓

Small exceedance accepted



UNIDENTIFIED ACTIONS

Alternate load path method (ALPM)

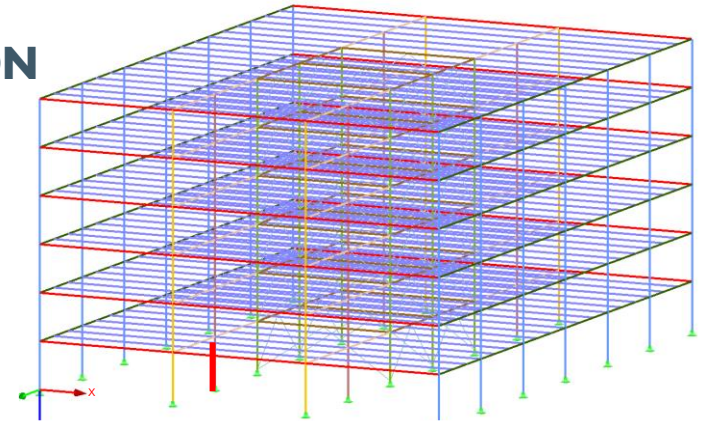
Analytical approach – SS/NS

This example presents the design against unidentified threats using the analytical approach from ALPM.

- **ACTIONS CONSIDERED FOR THE ACCIDENTAL DESIGN SITUATION**
  - Permanent loads DL
  - Live loads LL
- **COMBINATION OF ACTIONS FOR THE ACCIDENTAL DESIGN SITUATION**

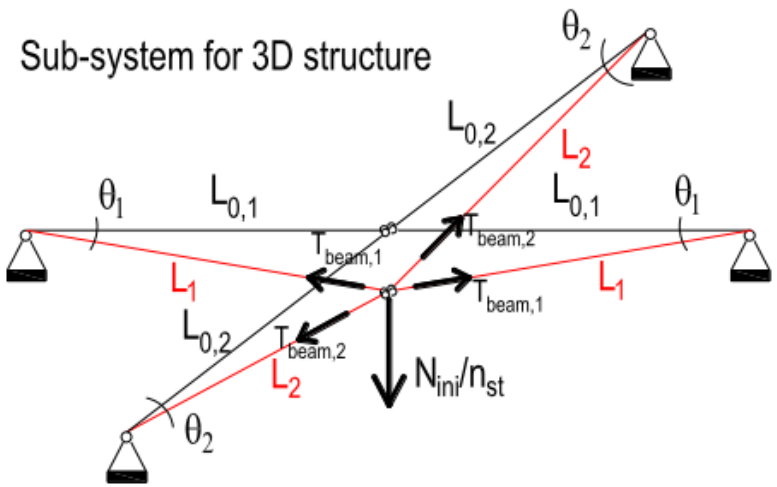
$$DL + 0.5 \times LL$$

- **SCENARIO CONSIDERED:** column removal at location B2, ground level
- **ASSUMPTIONS FOR JOINTS**
  - Solution 1: simple joints
  - Solution 2: partial-strength joints



Solution 1 - Tying forces for simple joints

The procedure consists in solving the system of 4 equations



	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	$L_{0,1}.\tan\theta_1 = L_{0,2}.\tan\theta_2$

beam,1 - IPE550

beam,2 - IPE600

N_{ini} is taken from the structural analysis in the accidental load combination.

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

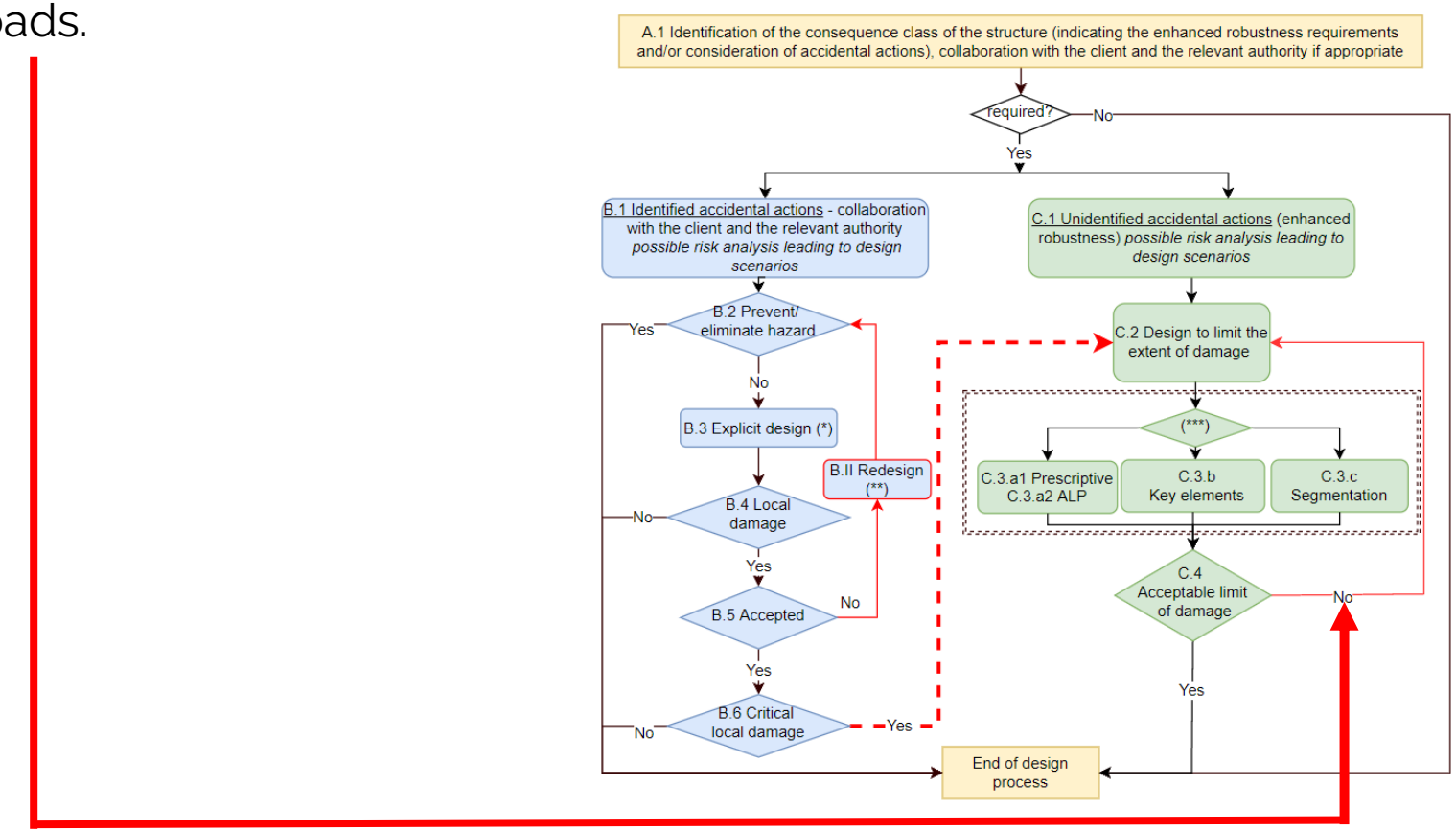
$$17\,866.67\tan(x)\left(1-\cos\left(\tan^{-1}(0.67\tan(x))\right)\right)+31\,200\tan(x)\,(1-\cos(x))-3.24=0$$

Solution

θ_1	θ_2	$T_{beam,1}$ - IPE550	$T_{beam,2}$ - IPE600
0.03659 rad	0.05485 rad	**1884 kN**	**4934 kN**

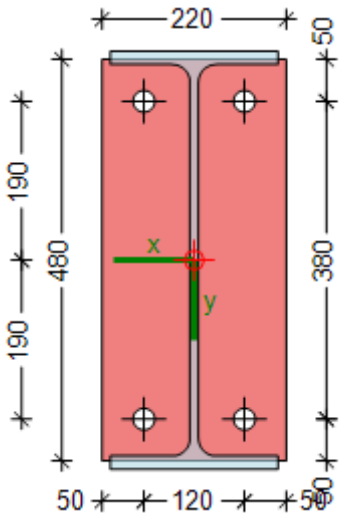
Solution 1 - Tying forces for simple joints

- When compared with the numerical approach (next example), the results obtained are approximately 8% higher (1741 kN for IPE550 and 4565 kN for IPE600). However, 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.

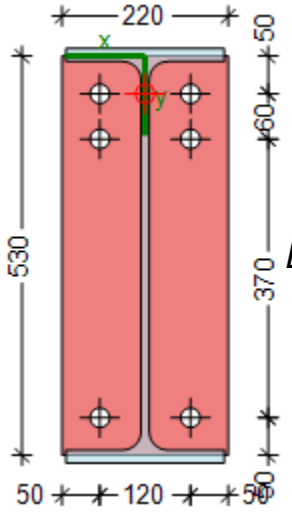


Solution 2 - Alternative approach with partial-strength joints

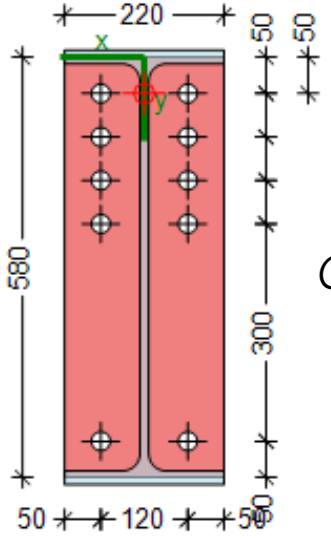
Partial-strength joints may be a good alternative to pinned joints to increase the robustness of the structure. To investigate this, main beam-to-column joints were replaced by flush end-plate joints.



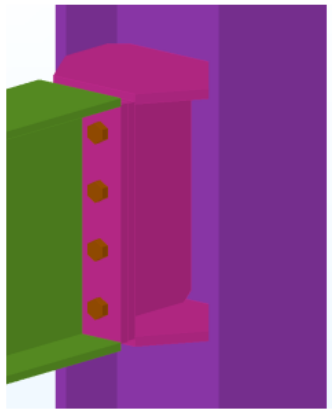
A1/A2



B1/B3



C2w/C3w



Welded part for weak axis flush end-plate joints

The simplified analytical method with partial-strength joints takes into account the following effects

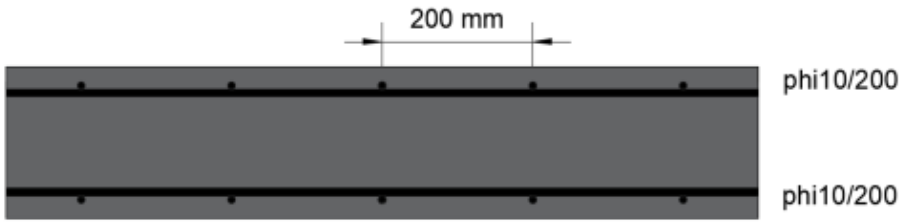
- Contribution from the plastic mechanism of beams
- Contribution from the slab
- Contribution from the arching effects

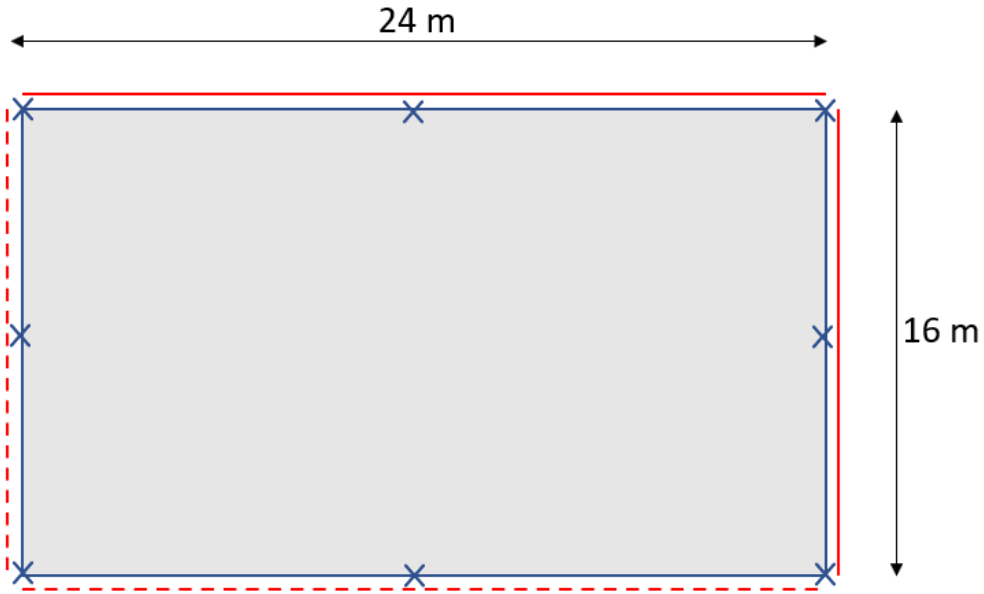
CONTRIBUTION FROM THE SLAB (1)

The RC slab contribution is expressed through the vertical point force P_{slab} (where the column is lost) leading to the development of a plastic mechanism.

The slab is designed to fulfil SLS/ULS requirements. The steel reinforcement is defined by the minimal constructive reinforcement according to EN 1992-1 Chap. 9.

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 cm²/m	3.93 cm²/m	26.9 kNm	Yielding of reinforcement





Static system of the concrete slab after column loss

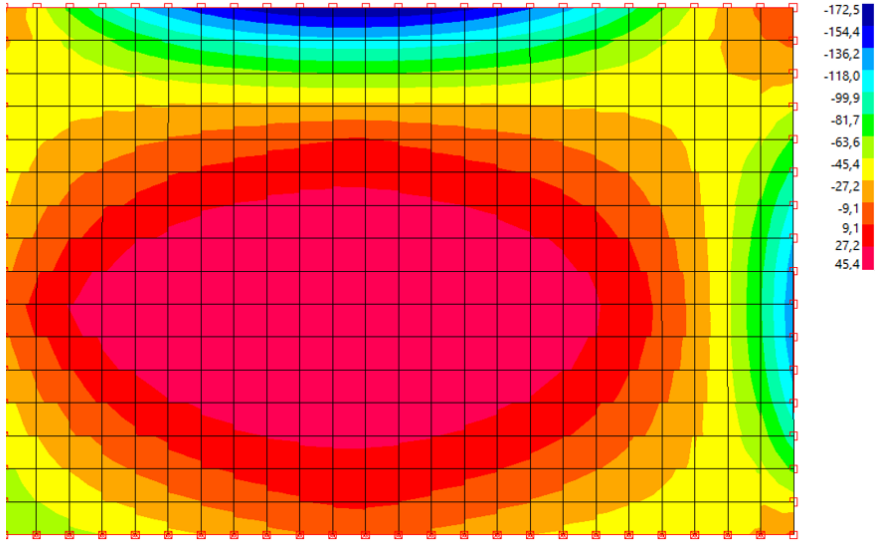
Peripheral beams (free rotation)

Slab continuity (restrained rotation)

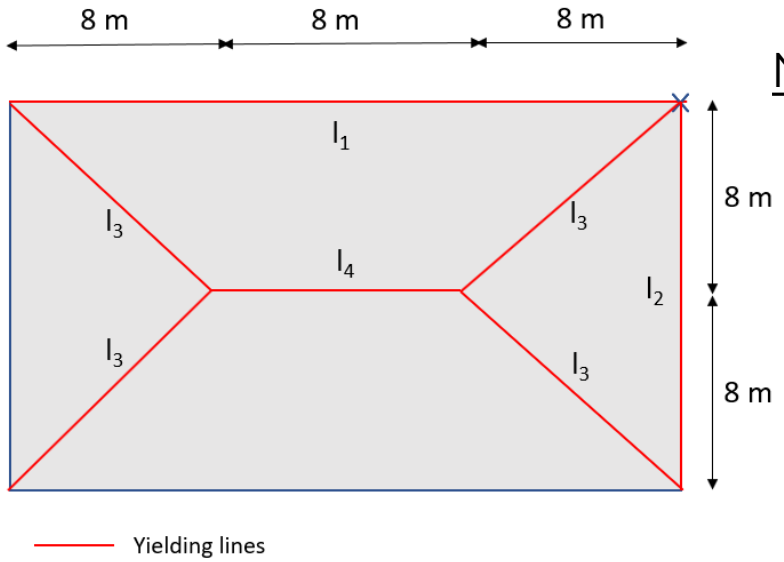
Columns

Accidental load combination (1 x G + 0.5 x Q) of 6.5 kN/m²

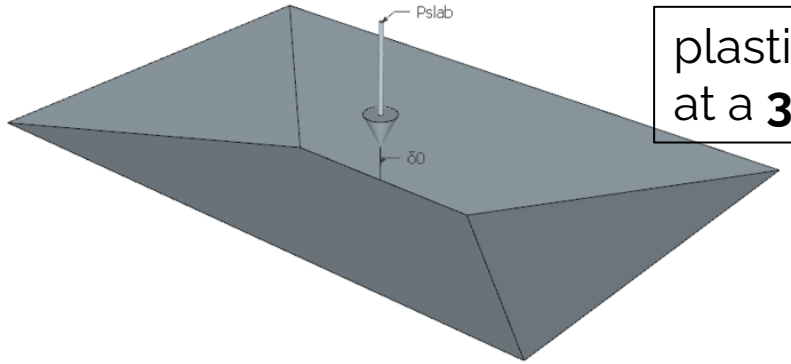
M_{Ed} = -172.5 kNm



CONTRIBUTION OF THE SLAB (2)

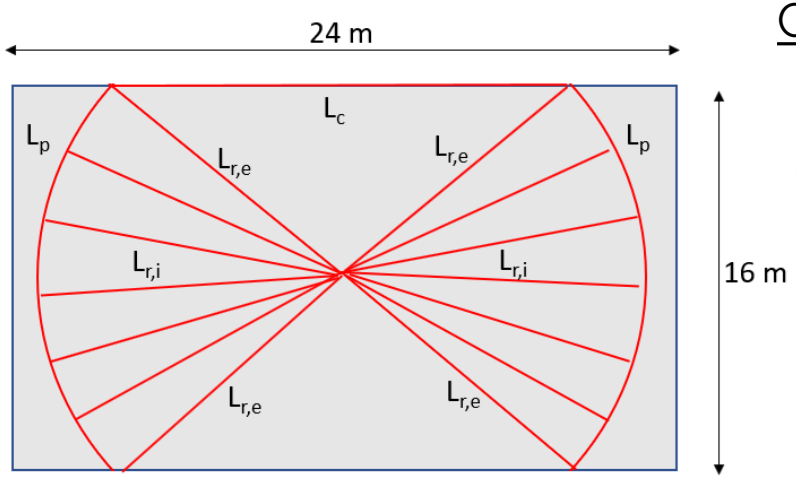


Non-circular plastic mechanism pattern

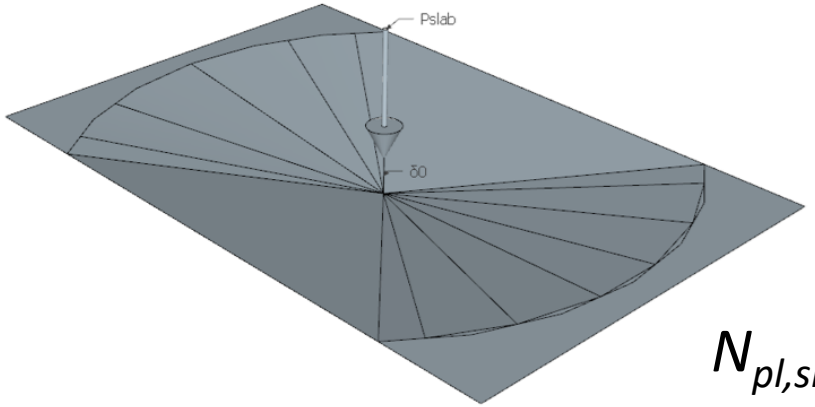


virtual works principle

plastic mechanism develops at a **313.6 kN** force



Circular plastic mechanism pattern



plastic mechanism develops at a **330.4 kN** force

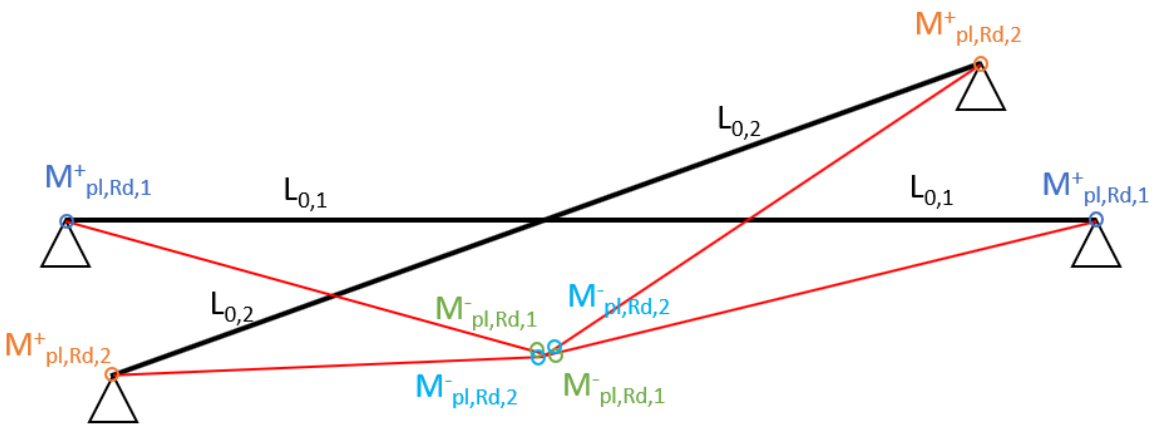
$$N_{pl,slab} = min(313.6\ kN; 330.4\ kN)$$

$$= \mathbf{313.6\ kN}$$

CONTRIBUTION OF THE STEEL BEAM MECHANISM

Partial-strength joints ➔ vertical force associated to the development of a plastic beam mechanism due to the formation of plastic hinges in the joints can be computed for both directions.

$$N_{pl} = \frac{2.M^{-}_{pl,Rd,1} + 2.M^{+}_{pl,Rd,1}}{L_{0,1}} + \frac{2.M^{-}_{pl,Rd,2} + 2.M^{+}_{pl,Rd,2}}{L_{0,2}}$$



Moment resistances of the 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

➡ $N_{pl} = 269$ kN

CONTRIBUTION OF THE ARCHING EFFECT

- N_{arch} is the vertical point force required to overcome the arching effect.
- The arching effect is activated if the failure mode of the system is not a component in compression. In such conditions, an arch effect can be mobilised within the beams of the directly affected part.

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***
- The contributions of the slab, the beam mechanism and the arch effect can be cumulated as their activation requires limited deformation capacities, as soon as the plastic mechanism has formed.

THE TOTAL RESISTANCE IS:

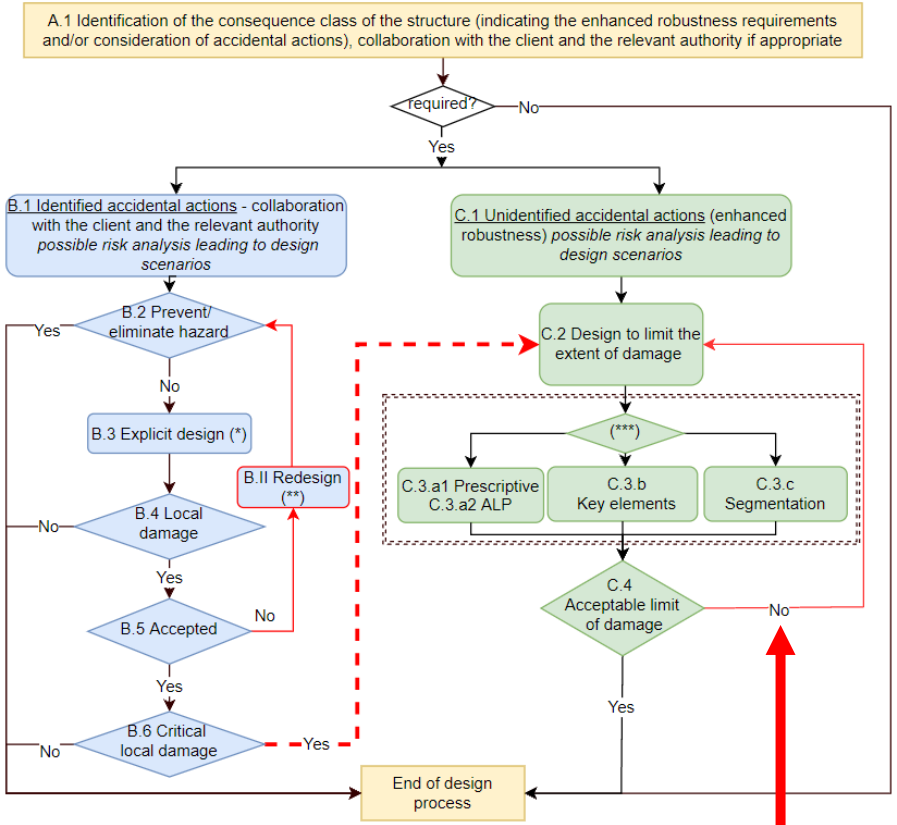
$$N = N_{slab} + N_{pl} + N_{arch} = 313.6 + 269.0 + 0.0 = 582.6\ kN$$

VERIFICATION OF THE STRUCTURE

$$N = N_{slab} + N_{pl} + N_{arch} = 313.6 + 269.0 + 0.0 = 582.6\ kN < 694.2\ kN\ (Force\ in\ the\ removed\ column)$$

➔ Significant vertical displacements will develop in the directly affected part with the apparition of membrane forces $\boldsymbol{N_{membrane}}$ in the beams. Such membrane forces cannot be cumulated with the contributions coming from the slab and from arching effects as the latter disappear once large deformations are reached.

- The contribution $\boldsymbol{N_{membrane}}$ requires significant deformation capacities at the level of the partial-strength joints. The failing component in the joints is here the column web in compression (not ductile) ➔ joints need to be redesigned.



REDESIGN OF THE STRUCTURE WITH PARTIAL-STRENGTH JOINTS

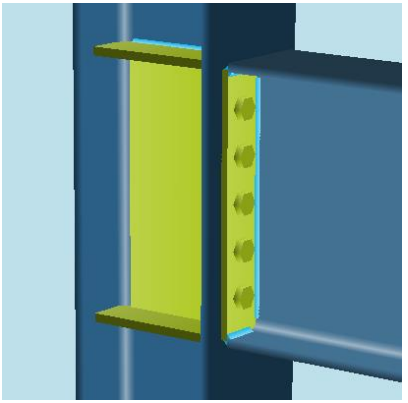
There are several ways of achieving the robustness requirements, such as:

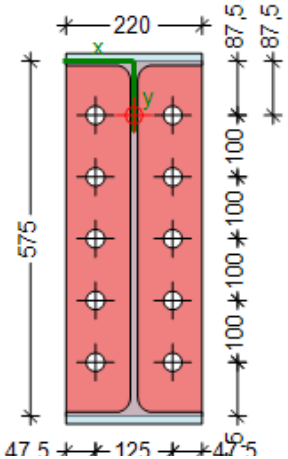
- Redesign the slab to increase the contribution from the slab mechanism;
- Strengthen the joints in one or both directions to increase the contribution of the beam mechanism;
- Reinforce compression components in the joints to activate the arch effect.

CHANGES FOR JOINTS C2/C3 AND B1/B3

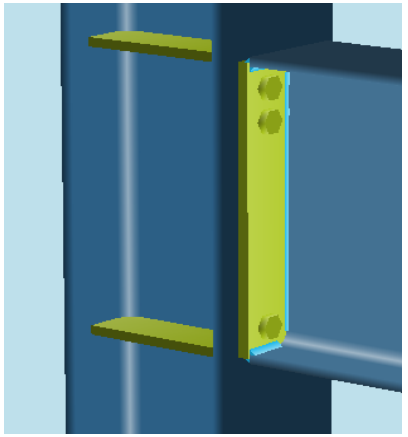
- Column stiffeners (same thickness as beam flanges);
- Web stiffener (just for C2/C3);
- Adapted bolt pattern;
- Flange welds changed from 6 to 7 mm;
- M24 bolts increased to M27 (just for C2/C3).

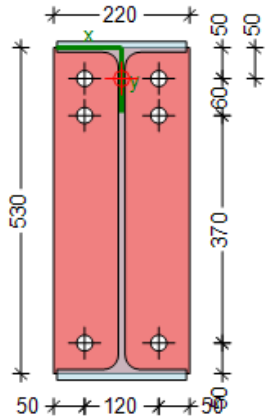
Changes in these joints allow to increase the bending resistance of the joints and thus increase the contribution of the beam mechanism.





C2/C3 joints





B1/B3 joints

REDESIGN OF THE STRUCTURE WITH PARTIAL-STRENGTH JOINTS (3)

CONTRIBUTION FROM THE SLAB

- As no changes have been made to the slab, the contribution of this component remains unchanged (N_{slab} = **313.6 kN**)

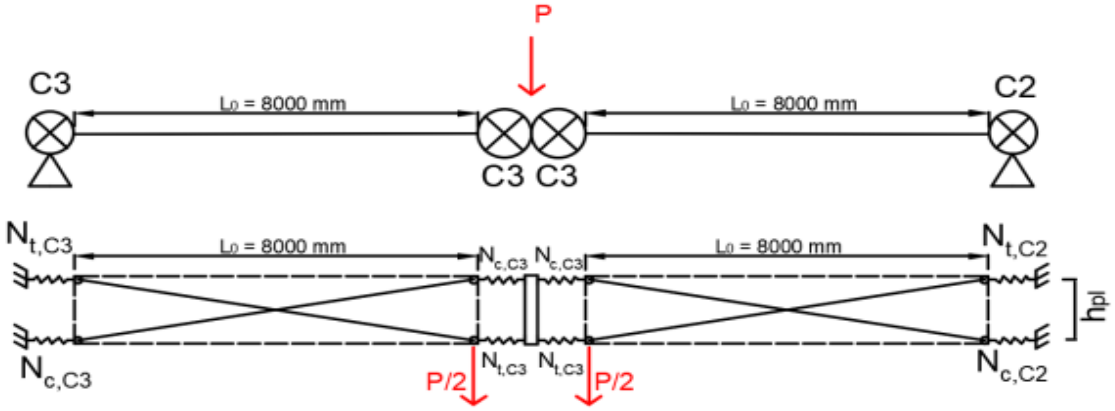
CONTRIBUTION OF THE BEAMS MECHANISM

- N_{pl} is now equal to **334.7 kN**

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

CONTRIBUTION FROM THE ARCHING EFFECTS

- Only the arching effect coming from the short frame (IPE600 with C2/C3 joints) is considered



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

N_{arch} = 51 kN

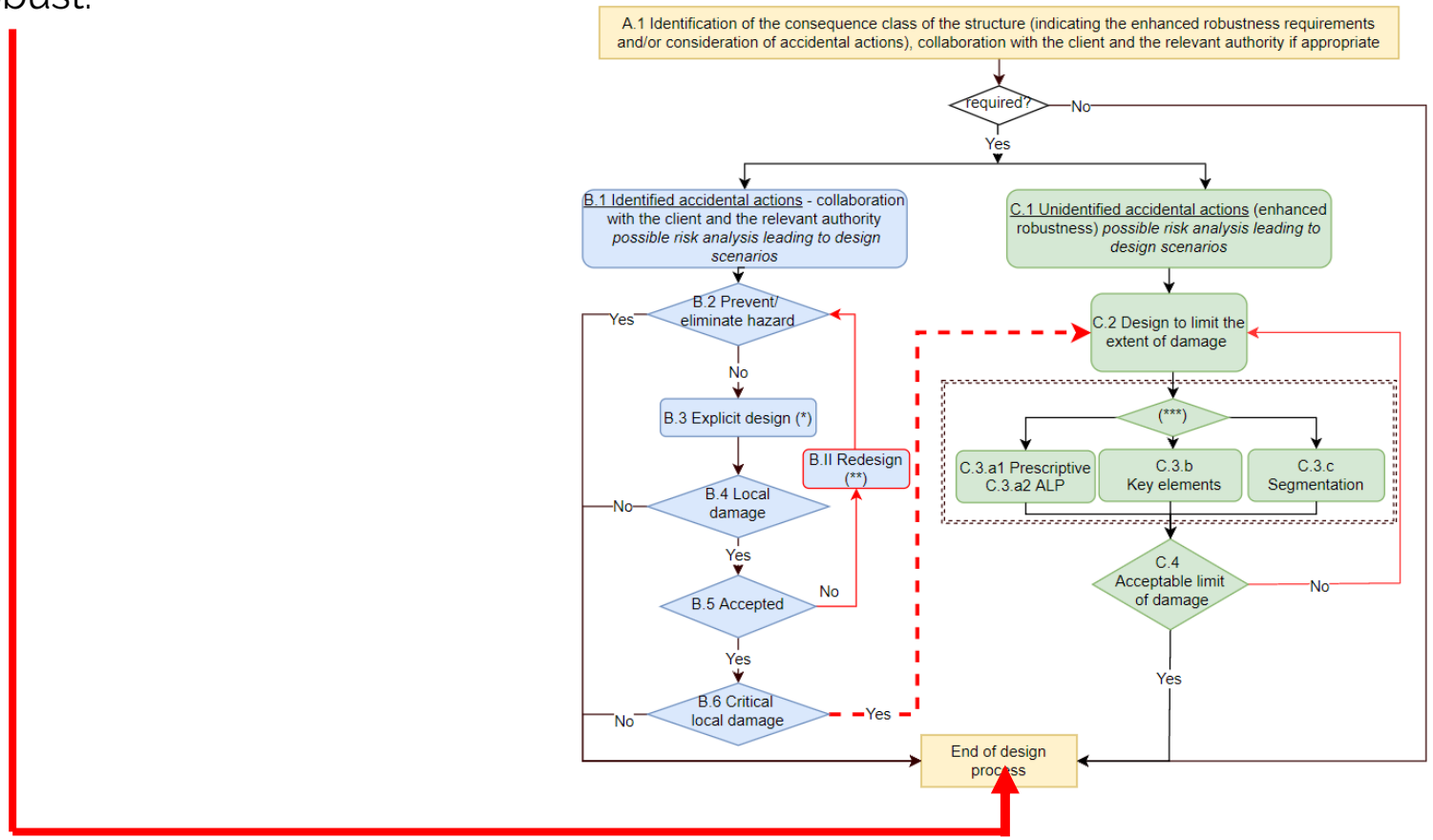
VERIFICATION OF THE STRUCTURE

By cumulating all of the three contributions, **the total resistance** is now:

$$N = N_{slab} + N_{pl} + N_{arch} = 313.6 + 334.7 + 51 = 699.3\ kN > 694.2\ kN$$

CONCLUSIONS

- 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.



UNIDENTIFIED ACTIONS

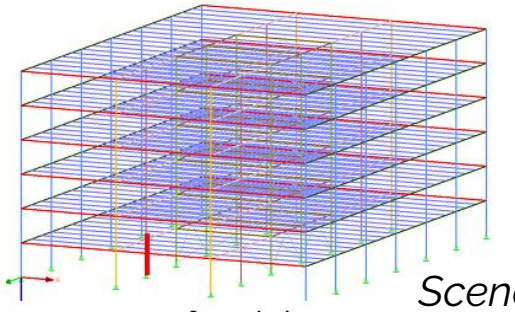
Alternate Load Path Method

Full numerical approach – SS/NS

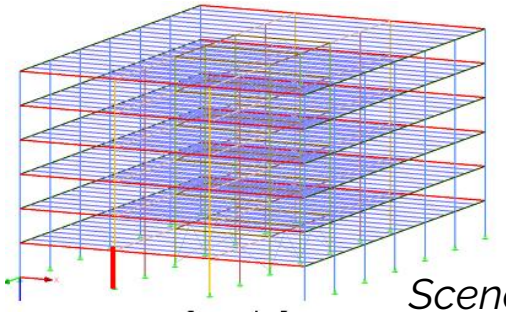
- **ACTIONS FOR THE ACCIDENTAL DESIGN SITUATION**
  - Permanent loads DL
  - Live loads LL
- **COMBINATION OF ACTIONS FOR ACCIDENTAL DESIGN SITUATION**

$$DL + 0.5 \times LL$$

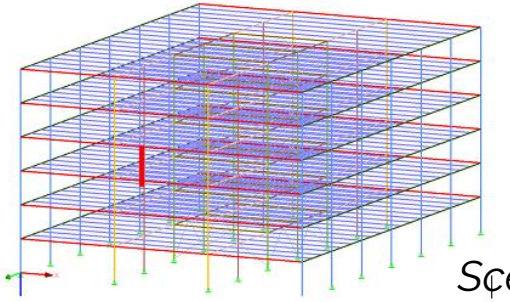
- **SCENARIOS CONSIDERED**
  - Scenario 1: Inner column loss at floor 0;
  - Scenario 2: Facade column loss at floor 0;
  - Scenario 3: Inner column loss above column splice



Scenario 1



Scenario 2

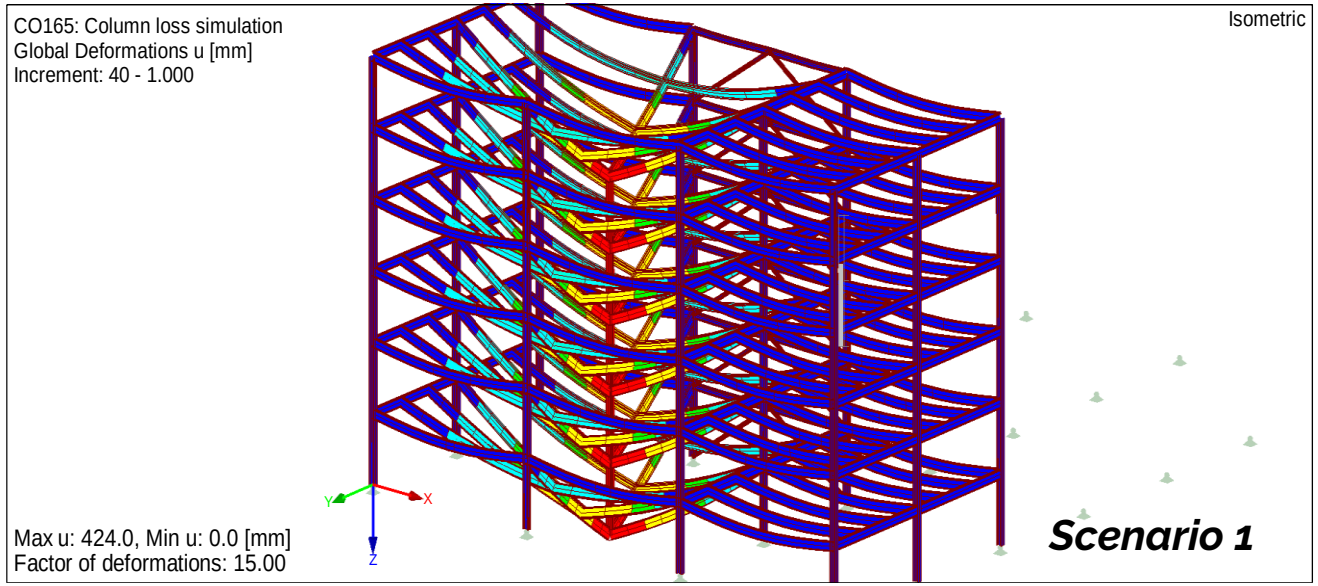


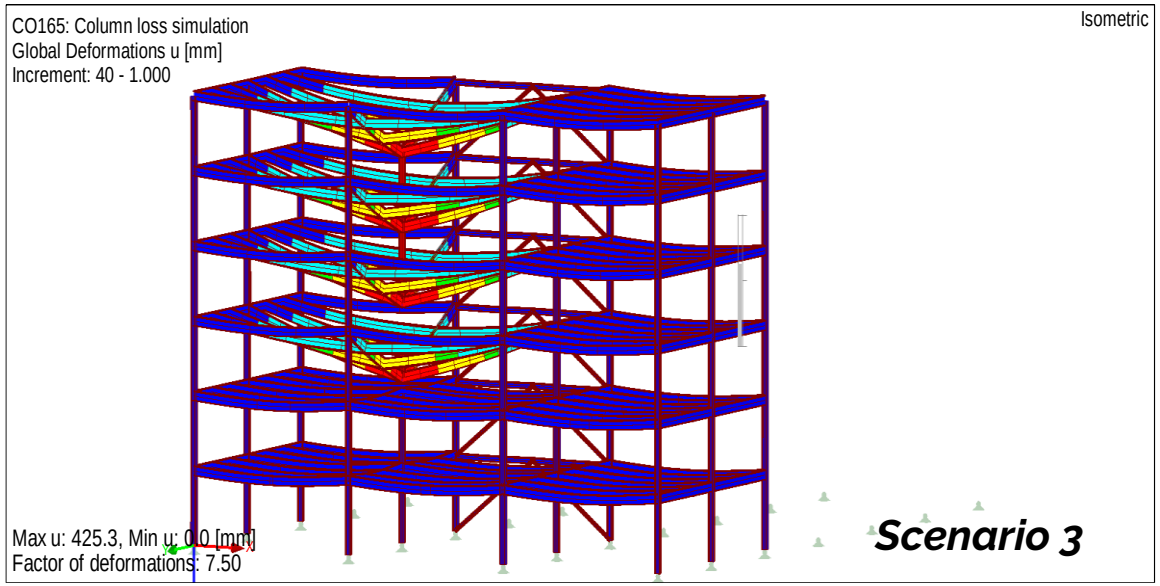
Scenario 3

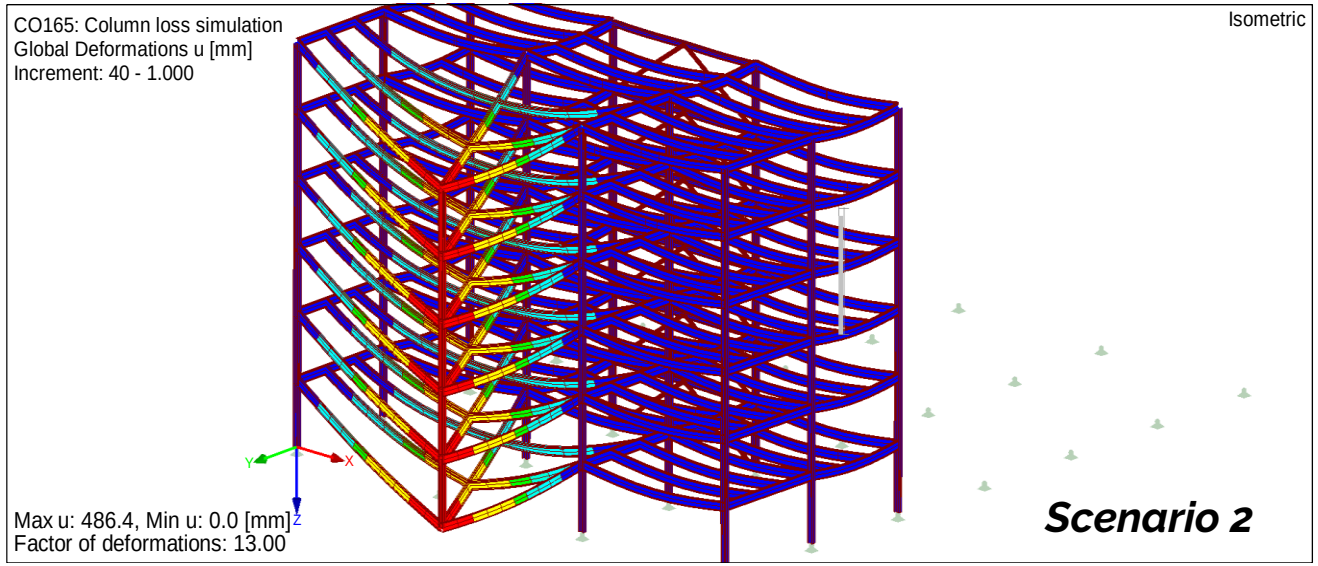
STRUCTURAL ANALYSIS

- The aim is to remove a column allowing for the development of membrane effects in the ties in the first step and then verify if the ties (members and joints) can withstand these tensile forces.
- Methodology and assumptions:
  - FE analysis is performed using a Newton-Raphson algorithm allowing the integration of large deformations;
  - Lateral-torsional buckling of the beams which in reality are restrained by the diaphragms, is prevented by fictitiously increasing the torsional inertia of the beam members.
- Modelling of column loss scenario:
  - 1st step –The structure is analysed with all the columns in place under the accidental load case combination ➔ the compression force in the column to be lost is known;
  - 2nd step - This force is applied at the upper node of the column to be lost and the column is removed, so that this reaction force replaces the column;
  - 3rd step - A force of same magnitude is gradually applied at the same node but in opposite direction. Load steps of 0.025 are used to ensure convergence.

RESULTS





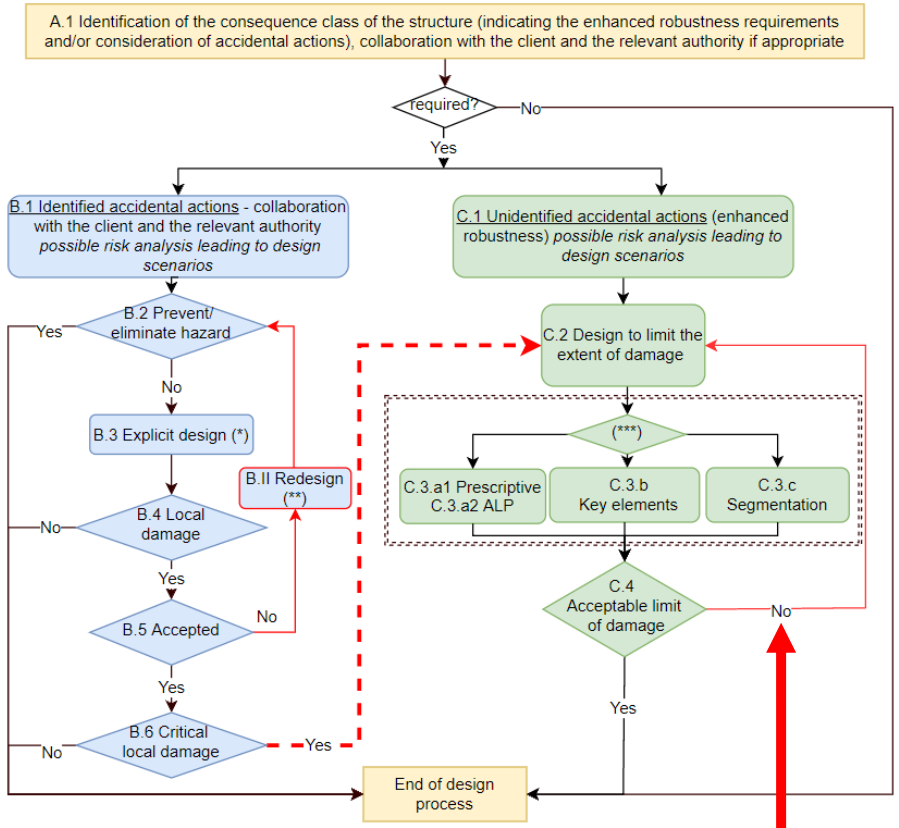


Scenario	Member	Joint	Tying force (kN)	Moment (kNm)
1	IPE550	B1/B3	1741	274
	IPE600	C2/C3	4565	536
2	IPE500	A1s/A2	1620	195
3	IPE550	B1/B3	1715	275
	IPE600	C2/C3	4493	537

SCENARIO 1: VERIFICATIONS

- In the absence of the removed column, compression forces in adjacent columns increased. However, these forces remain lower than the design compression forces at ULS ➔ no column redesign required.
- The IPE550 beams were designed to fulfil the SLS requirements (deflection), yet the resistance of these members is still sufficient to withstand a column loss.
- The IPE600 beams are not sufficiently resistant for the high tensile forces (15% of exceedance).

Member	Section	Tying / 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

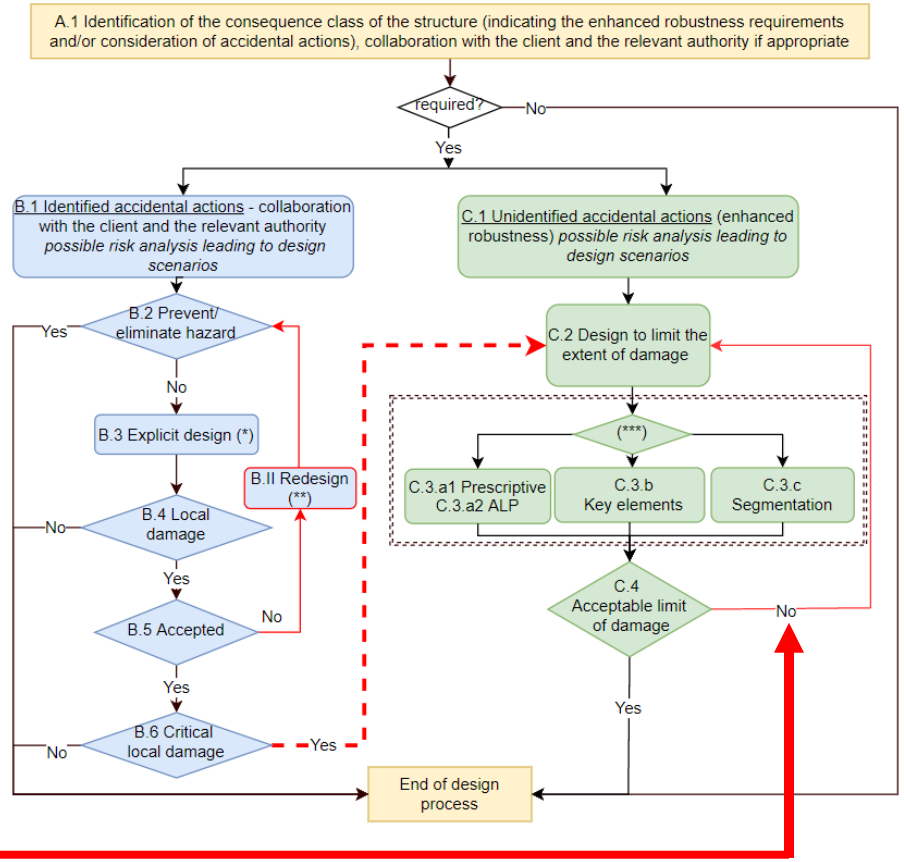


SCENARIO 2: VERIFICATIONS

- All members verify the requirement;
- Not fulfilled for joints A1s/A2s ➔ **redesign**

Member	Section	Tying / 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

Position s = strong axis w = weak axis	Tying force (kN)	Connection Failure mode	UF
A1s / A2s	1620	Fin plate in bearing	3.71



SCENARIO 3: VERIFICATIONS

- No tying forces in vertical ties, but tensile forces in horizontal ties. These tensile forces are similar to the ones reached in scenario 1.

REDESIGN OF STRUCTURE

SCENARIO 1 VERIFICATIONS

- Due to the section change (IPE600 to IPE750x137), the internal force distribution has changed:

Member	Section	Tying / 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

JOINT VERIFICATIONS

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:

- 2 bolts added
- M27 instead of M24
- additional welded web plate to the beam
- modified fin plate geometry and thickness (25 mm)
- thicker weld for ductility requirements (15 mm).

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	

REDESIGN OF STRUCTURE

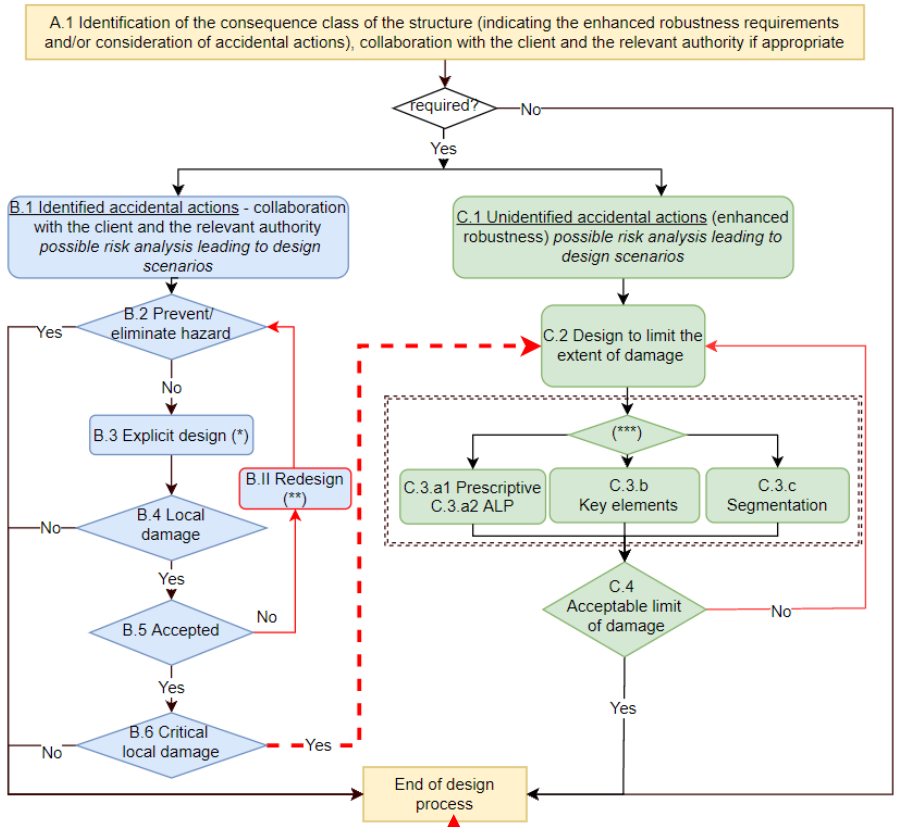
SCENARIO 2 VERIFICATIONS

Redesigned joint A1s/A2s:

- 4 bolts added;
- M24 instead of M20;
- additional welded web plate to the beam;
- modified fin plate geometry and thickness (20 mm);
- thicker weld for ductility requirements (12 mm).

JOINT VERIFICATIONS

Position s = strong axis w = weak axis	Tying force (kN)	Failure mode	UF
A1s / A2s	1620	Bolts in shear	1.01



WORKED EXAMPLES

Brussels 10-05-2022

Děkuji! Dank je! Thank you! Merci! Dankeschön! Grazie! Dziękuję Ci! Obrigado! Mulțumesc! Gracias!

Tudor GOLEA

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steelconstruct.com/eu-projects/failnomore

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