The Institution of Structural Engineers

SECED

London 01.06.2022

Welcome Address and Introduction

Ahmed Elghazouli

Imperial College London

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

Imperial College London

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FAILNOMORE Project

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

- **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 and
  - available literature

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FAILNOMORE Project

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

- **Partners**
  - University of Liège (ULg) – Belgium
  - University of Coimbra – Portugal
  - Imperial College London - UK
  - University of Stuttgart – Germany
  - University of Trento – Italy
  - University of 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 & Dufferdange S.A.- Luxembourg

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FAILNOMORE Project

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

- **The proposed design guidelines are reported in a Design Manual (including design examples)**
- **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)**

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Workshop Programme

13:00-13:30	Status of Structural Eurocodes and Background to Codified Robustness Design (David Nethercot, Imperial College London)
13:30-14:30	Mitigating the Risk of Progressive Collapse under Exceptional Events –Failnomore Project (Jean-François Demonceau, University of Liège)
14:30-15:00	Coffee Break (and discussions)

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Workshop Programme

15:00-16:00	Overview of Design Procedures and Recommendations (Ahmed Elghazouli, Imperial College London) (Zeyad Khalil, Imperial College London)
16:00-17:00	Design Examples for Steel and Composite Buildings (Florea Dinu, University of Timisoara)
17:00-17:30	Workshop Closure (and discussions)

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Status of Structural Eurocodes and Background to Codified Robustness Design

David Nethercot

Imperial College London

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Outline

- **This presentation is organised as follows:**

1. Introduction
2. Normative context
3. FAILNOMORE project

2

Ronan Point (1968)

3

Alfred P. Murrah Building (1995)

4

Champlain Towers (2021)

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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

- **Progressive vs. Disproportionate collapse**

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1. INTRODUCTION

- **Extreme loading conditions**
  - Explosion, blast, impact, fire, snow overloading, tsunami, tornado, terrorist attacks .....
  - Identified vs. Unidentified threats
  - Large deformations experienced by structural components, including connections
  - Inadequacy of strength-based design methods alone
  - Need for ductility-based design approaches
  - Concrete vs. Steel/Composite structures

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2. NORMATIVE CONTEXT

- **Progressive collapse mitigation is covered in several European and International codes and guidelines**
- **Give general recommendations to achieve robustness requirements**
- **Mostly rely on prescriptive methods**

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2. NORMATIVE CONTEXT

CURRENT EUROCODES

- **EN 1990**
  - **EN 1990, 2.1 (4)P** sets out the basic principle related to structural robustness, where it is explicitly stated that:

    A structure shall be designed and executed in such a way that it will not be damaged by events such as: explosion, impact, and the consequences of human errors, to an extent disproportionate to the original cause

  - **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; (v) tying members together*

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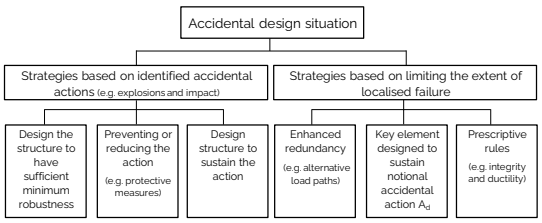
2. NORMATIVE CONTEXT

CURRENT EUROCODES

- **EN 1991-1-7 – Annex A**



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2. NORMATIVE CONTEXT

FUTURE EUROCODES

- **Current revision of Eurocodes**
- **Working Group CEN/TC 250/WG 6 "Robustness"**
- **EN 1990 - Section 4.4 and Informative Annex E covering strategies based on limiting the extent of damage**
- **The explicit design of structures for identified accidental action is covered within the scope of EN 1991**
- **Introduction of "Segmentation"**
- **Developments within EN 1993 and EN 1998 which may be of direct and indirect relevance to achieving robustness**

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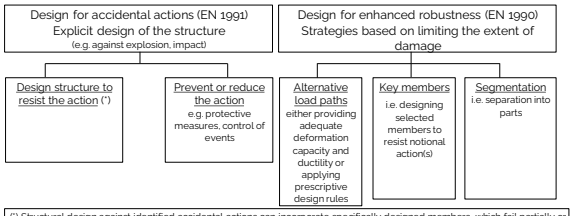
2. NORMATIVE CONTEXT

FUTURE EUROCODES

- **EN 1990 – Annex E**



(*) 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

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3. ASSESSMENT OF ROBUSTNESS REQUIREMENTS IN THE EUROCODES

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

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3. ASSESSMENT OF ROBUSTNESS REQUIREMENTS IN THE EUROCODES

- **The main objective of the FAILNOMORE project to address such shortcomings**
- **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 is proposed**
- **The proposed design guidelines are based on:**
  - recent research projects and
  - available literature
- **The proposed guidelines are demonstrated using a set of worked examples**

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3. ASSESSMENT OF ROBUSTNESS REQUIREMENTS IN THE EUROCODES

- **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)**

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London **01.06.2022**

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

steelconstruct.com/eu-projects/failnomore

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London 01-06-2022

Design for robustness

Jean-François DEMONCEAU[1]

[1] University of Liège, Belgium

FAILNOMORE

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1. INTRODUCTION

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

- **This presentation is organised as follows:**
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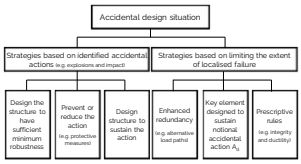
1. Introduction

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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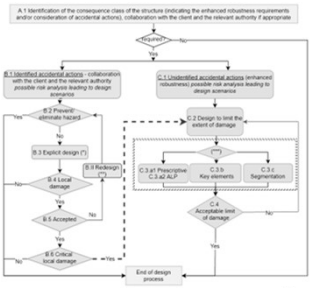
2. General design philosophy

- **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**



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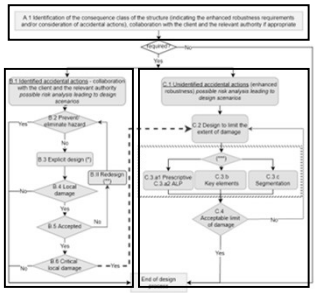
2. General design philosophy

- **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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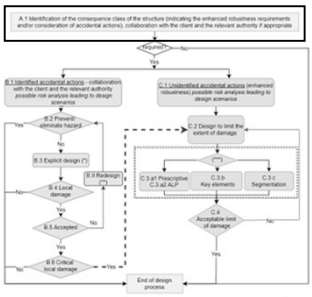

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3. Consequence classes

- **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, ...**

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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

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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**

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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

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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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FAIL NO MORE

1. **Introduction**
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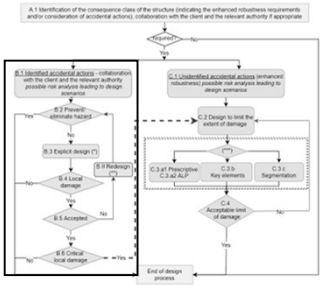

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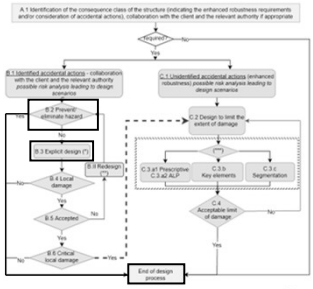
4. Design for identified acc. actions

- **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



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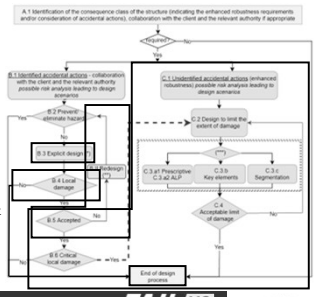
4. Design for identified acc. actions

- **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



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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 CC_2, the use of prescriptive methods or of simplified analysis considering static equivalent action models is possible
  - For CC_3, the use of refined methods (dynamic analysis, non-linear models..) may be required

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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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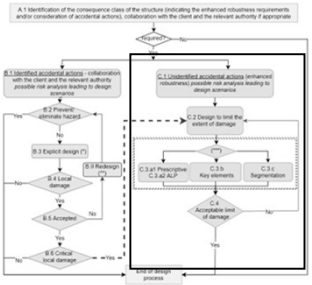

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5. Design for unidentified acc. actions

- **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**

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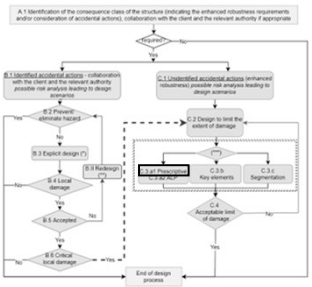
5. Design for unidentified acc. actions

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



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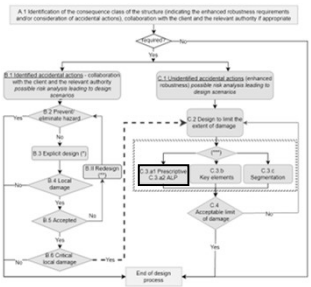
5. Design for unidentified acc. actions

- **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>



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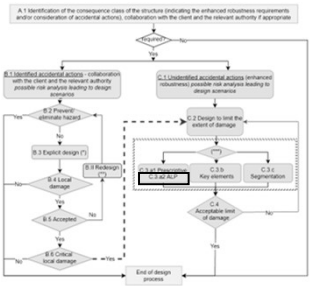
5. Design for unidentified acc. actions

- **For CC3, the use of the <u>ALP method</u> through refined approaches such as dynamic analyses is recommended**



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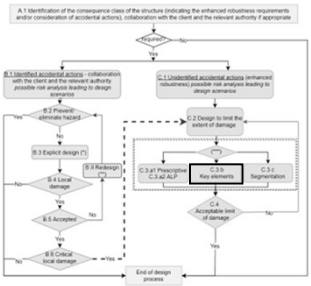
5. Design for unidentified acc. actions

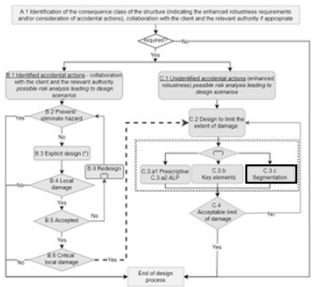
- **Where the loss of a supporting member generates a disproportionate collapse, the removed element should be labelled as a "<u>key element</u>"**
- **The design should turn towards methods of <u>local enhancement of resistance capacity</u> of the key element considering a notional accidental action**



30

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**



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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**

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CONTENT LIST

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

- **This presentation is organised as follows:**
  1. Introduction
  2. General design philosophy for robustness
  3. Definition of consequence classes
  4. Design for identified accidental actions
  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
  7. Conclusions

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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

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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

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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!

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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

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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

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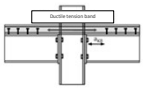
6.1 Minimum ductility requirements for joints

- **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



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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

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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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FAIL NO MORE

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7. Conclusions

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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

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Design for Robustness

London 01-06-2022

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

DEMONCEAU Jean-François
jfdemonceau@uliege.be

steelconstruct.com/eu-projects/failnomore

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London 01.06.2022

Overview of Design Procedures and Recommendations

Ahmed Elghazouli

Imperial College London

FAILNOMORE

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

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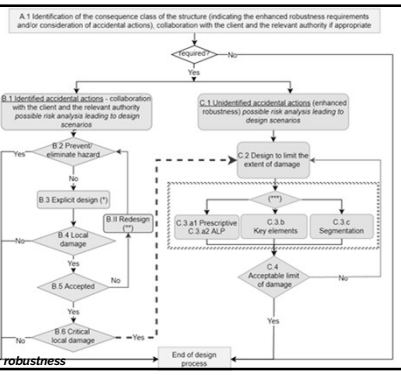
1

General design philosophy



2 Design for robustness

2

General design philosophy

Consequence Class 1	no specific consideration
Consequence Class 2	simplified analysis and/or prescriptive design/detailing rules
Consequence Class 3	Risk assessment / dynamic and/or non-linear analysis as appropriate

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3

IDENTIFIED THREATS

1. Introduction
2. Impact
3. Explosions
4. Fire as exceptional event
5. Earthquake as exceptional event
6. Conclusions

1. Introduction
2. Impact
   - 2.1 Equivalent static approach
   - 2.2 Simplified dynamic approach
   - 2.3 Full dynamic approach
3. Explosions
   - 3.1 External explosions
   - 3.2 Internal gas explosions
4. Fire as exceptional event
5. Earthquake as exceptional event
6. Conclusions

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2. IMPACT

- **Accidental action**
- **Low probability of occurrence but high consequence**
- **For buildings, vehicle impact is more common:**
  - Buildings near roads
  - Car parks or buildings with nearby parking
  - Buildings where cars and trucks are allowed

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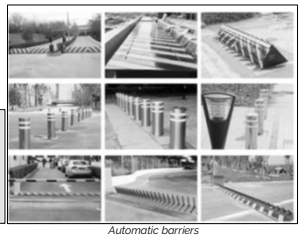
2. IMPACT

REDUCE/PREVENT THE ACTION

- **Reducing 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
  - Use of barriers (permanent or automatic)

Permanent barriers



Automatic barriers

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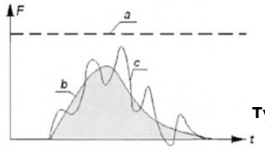
6

2. IMPACT

DESIGN STRATEGIES



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 all 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) (CC1/CC2)
- Simplified dynamic approach (Annex C EN1997-1-7) (Hard impact, up to CC2b) (Soft impact, up to CC3)
- Full dynamic approach (Hard or soft impact)

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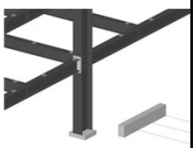
2. IMPACT (FULL DYNAMIC APPROACH)

- **Two approaches can be employed**
  - **The impact load explicitly modelled:**
    - Most realistic approach
    - Modelling of impact body (mass, stiffness), impact loading (velocity, direction, duration) structural response (stresses, strains, deflections), and structure - object interactions
    - Effect of strain rates 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 (analysis done using ELS)

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3. EXPLOSIONS

- **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 expand rapidly and generate shock waves - 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)

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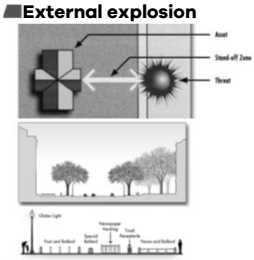
3. EXPLOSIONS

PREVENT/ELIMINATE HAZARD

- **External explosion**



- Maximizing stand-off distance significantly decreases 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 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

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3. EXPLOSIONS

PREVENT/ELIMINATE HAZARD

- **Internal gas explosion**
  - **Consider 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 venting**
  - **Vent areas:**
    - size and location → when sufficient venting is close to ignition point, flame speed will be low, and turbulence behind the obstacles will be limited
    - generally, gas explosion venting should be directed into open areas with minimum of obstructions
    - partial obstruction of a vent opening results in notable pressure increases

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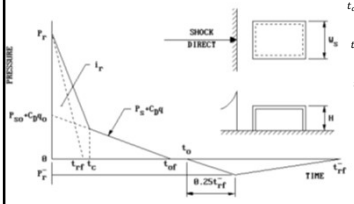
3.1 External explosion

EXPLICIT DESIGN

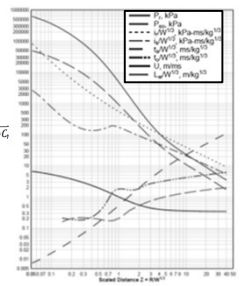
- **Scenario definition**
  - W – explosive charge weight
  - R –distance to the building



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

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

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



Pressure-impulse chart

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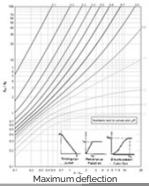
3.1 External explosion

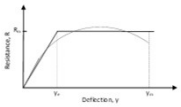
EQUIVALENT SDOF APPROACH

- **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 r_m

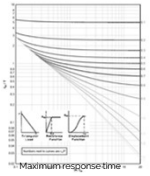


Maximum deflection

- **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 response time

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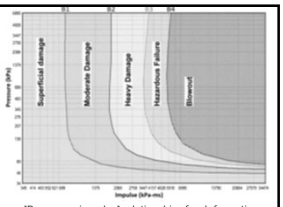
3.1 External explosion

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
- 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 (class of consequences) 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_{lim}	m_{max}	q_{lim}	m_{max}	q_{lim}	m_{max}	q_{lim}
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°	1	3°	1	3°
	Beam-column with limited ductility cross-section	0.7	-	0.85	3°	0.8	3°	0.85	3°
	Column (axial failure)	0.9	-	1.3	-	2	-	3	-

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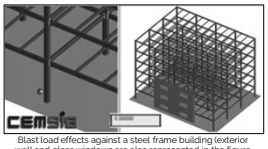
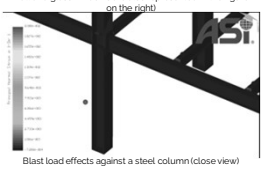
3.1 External explosion

FULL DYNAMIC APPROACH

- **Blast load**:
  - free and surface-field models of blast waves
  - function of explosive weight, distance to the explosive, and time
  - Alternatively, user-defined blast pressure. CFD may be used if necessary.
- **Material Models:**
  - Material models can be Linear, Bilinear, Multi-Linear, or User-Defined models
  - Steel, concrete, and composite models may be already integrated in program library
- **Failure criteria**
  - Elastic materials behave linearly without any plastic deformations.
  - Different failure criteria may be employed: (strain, stress, failure envelope, others)



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



Blast load effects against a steel column (close view)

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3.2 Internal gas explosions

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:

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

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*

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4. FIRE AS EXCEPTIONAL EVENT

- **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**
- **An example is a localised fire around a column which, in normal fire situation, should not take place**

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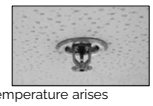
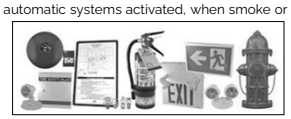
4. FIRE AS EXCEPTIONAL EVENT

REDUCE/PREVENT THE ACTION

- **Measures to prevent and/or reduce fire action and/or avoid spread**
- **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 fire spread:**
  - Fire extinguishers – activated manually, when fire appears
  - Sprinklers – automatic systems activated, when smoke or high temperature arises





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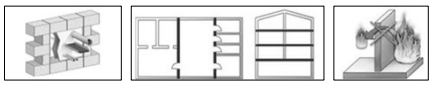
4. FIRE AS EXCEPTIONAL EVENT

REDUCE/PREVENT THE ACTION (continued)

- **Systems preventing fire spread :**
  - Fire walls – vertical isolation preventing fire spread
  - Vent insulators – insulation of any openings between compartments
  - Compartmentation – separation into quarters, between which fire cannot spread



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

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4. FIRE AS EXCEPTIONAL EVENT

- **Localised fire analysis can be undertaken using 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**

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

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_0 virtual origin of the fire

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

- **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)

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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 CFAST from NIST or OZONE (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 FDS from NIST

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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

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5. EARTHQUAKE AS EXCEPTIONAL EVENT

- **Earthquakes can be considered as exceptional events when:**
  - Structure is not designed for seismic action at all, or is designed for lower seismic demands hence the hazard is therefore exceptional
  - Structure is seismically vulnerable (pre-existing damage, not designed following modern code design requirements)

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5. EARTHQUAKE AS EXCEPTIONAL EVENT

PRESCRIPTIVE APPROACH

- **Particularly beneficial for non-seismic or low seismicity areas**
- **Such as Regularity Considerations in plan and in elevation**

(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, etc.)

VIBRATION CONTROL

- **Active, semi-active, passive control systems**
- **Base isolation and other common approaches**

DESIGN PROCEDURES

- **Simplified Code approaches (force reduction and capacity design)**
- **Performance based (detailed assessment of loading and response)**

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5. EARTHQUAKE AS EXCEPTIONAL EVENT

ROBUSTNESS ASSESSMENT

- **In the aftermath of an earthquake, the primary concern is the structural condition and whether it is safe from collapse under gravity loads, aftershocks, and other hazards (FEMA P-2090, 2021)**
  - If the structure lacks robustness, there is a risk of further damage 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.
- **Pushover approaches or full dynamic analysis procedures, depending on structural system and definition of seismic loading**

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6. CONCLUSIONS (Identified Threats)

- **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 are considered in manual.**
- **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 can be used**
- **The level of sophistication of the methods is strongly linked to the consequences class of the structure under consideration**

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FAIL NO MORE

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

UNIDENTIFIED THREATS

1. Introduction
2. Selection of the design strategy
3. Identification of local damage
4. Alternative load path methods (ALPM)
   - 4.1 ALPM-General
   - 4.2 ALPM- Prescriptive methods
   - 4.3 ALPM-Analytical methods
   - 4.4 ALPM-Simplified numerical approaches
   - 4.5 ALPM-Full numerical approach
   - 4.6 ALPM-Dynamic response from static response
5. Key element method
6. Segmentation method
   - 5.1 Weak segment borders
   - 5.2 Strong segment borders
7. Conclusions

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2. DESIGN STRATEGIES

- **Unidentified threats refer to accidental actions not specifically "identified" by the client or other stakeholders**
- **By definition, unidentified threats cannot be characterised and are unspecifiable**
- **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 structure belongs to**

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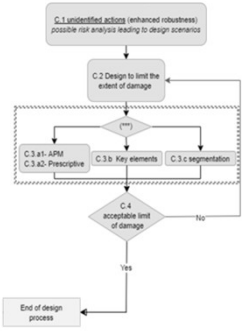
2. DESIGN STRATEGIES

- **For CC1 structures:**
  - No specific requirement
- **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



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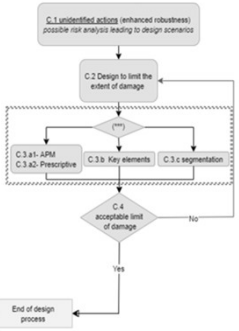
2. DESIGN STRATEGIES

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



30

3. LOCAL DAMAGE

- **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!**
- **Can reduce number of column loss scenarios to be considered, particularly for regular buildings**

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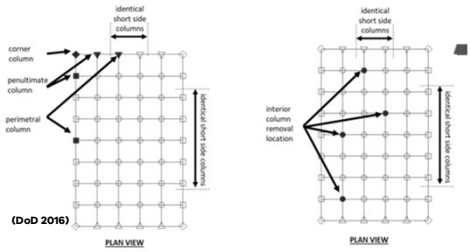
3. IDENTIFICATION OF LOCAL DAMAGE



(DoD 2016)

- **Storey Locations:**
  - 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

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3. IDENTIFICATION OF LOCAL DAMAGE

- **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**

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FAIL NO MORE

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

UNIDENTIFIED THREATS

1. Introduction
2. Selection of the design strategy
3. Identification of local damages
4. Alternative load path methods (ALPM)
   - 4.1 ALPM-General
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7. Conclusions

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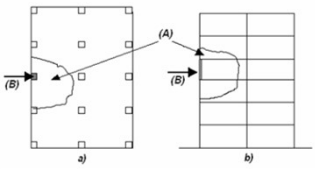
4.1 ALPM-GENERAL

- **When applying the alternative load path method, according to EN1991-1-7, 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 …
  - 100 m^2

  whichever is smaller, in each of the two adjacent storeys



(A) Local damage

(B) Notional column removed

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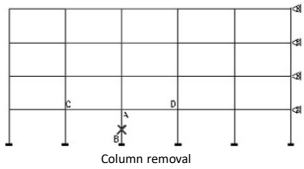
4.1 ALPM-GENERAL

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



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4.1 ALPM-GENERAL

- **Different design methods, with 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 approaches
  - *(4.6 ALPM-Prediction of dynamic response from statics analysis)*
  - 4.5 ALPM-Full numerical approach

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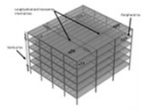
4.2 ALPM-PRESCRIPTIVE METHODS

- **The goal of such methods is to provide of a minimum level of robustness and resistance to progressive collapse for the structure**
- **They are indirect design approaches**
- **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;
  - selects the tie elements on the basis of level of risk and of the consequence classes;
  - is suggested for low/medium risk structures.
- **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.



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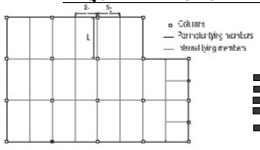
4.2 ALPM-PRESCRIPTIVE METHODS

- **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 RC slabs, profiled steel sheeting in composite floors can be used
- **Members & connections designed to resist a minimum level of tying forces**

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



Internal ties ⇨ $T_i = 0.8(g_k + \psi q_k)(s_r)(l_r) \geq 75\ kN$

Perimeter ties ⇨ $T_p = 0.4(g_k + \psi q_k)(s_r)(l_r) \geq 75\ kN$

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

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4.2 ALPM-PRESCRIPTIVE METHODS

- **To allow for the possible activation of tying members, a minimum level of ductility is required.**
- **In EN1991-1-7, 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 previous presentation.**

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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

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4.2 ALPM-PRESCRIPTIVE METHODS

- **A more refined method has been recently developed (CEN TC250 WG6 2020) which allows a better prediction of tensile loads**
- **It can be adapted to any structural systems through an appropriate calibration of several relevant coefficients**

$$T \geq \eta \rho \frac{i_r}{\bar{\alpha}} P$$

- 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_r 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

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4.2 ALPM-PRESCRIPTIVE METHODS

- **Composite floors offer an efficient structural solution to activate alternative load paths in case of a column loss scenario**
- **Allow activation of membrane forces within connected slab while requiring much less deformation capacity at beam extremities**
- **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 mobilisation of the slab, it is proposed to follow the recommendations from EN1992-1-1 (Section 9.10.2), where minimum requirements are given to provide the floor with a tying system**
- **Specific construction details for slabs made of precast concrete elements are also provided in the Design Manual**

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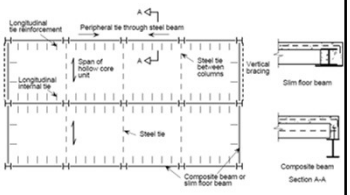
4.2 ALPM-PRESCRIPTIVE METHODS

Eurocode 2 requirements for slabs

- **Suitable tying system**
  - Peripheral ties
  - Internal ties
  - Horizontal column or wall ties
  - Vertical ties, if required (panel buildings of 5 or more storeys)
- **Criteria to arrange tie elements**
- **Design forces for ties**
- **Continuity and anchorage required for ties**

Example of tying system in precast slabs



Ref.: CEN/TC250/SC4 2020
"N 2040 Other Flooring Types Using Precast Concrete Elements"

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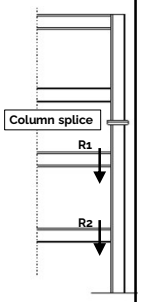
4.2 ALPM-PRESCRIPTIVE METHODS

- **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**
- **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**



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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 methods are covered in the next presentation**
- **The advanced method is detailed in the Design Manual (Annex A.8)**
- **All relevant application rules are provided in the Design Manual**

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4.4 ALPM- SIMPLIFIED NUMERICAL APPROACH

- **Robustness limit state**
  - Sudden column loss scenario
  - Prevent the collapse of the above floors
  - Large deformations allowed
  - Ductility limit
- **Multi-level ductility-centred assessment framework**
  - Nonlinear static response
  - Simplified dynamic assessment
  - Ductility assessment
- **Accounts for ductility, redundancy, energy absorption and dynamic effects**
- **Practice-oriented approach**
- **No need for detailed nonlinear dynamic analysis**

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4.5 ALPM-FULL NUMERICAL APPROACH

- **Design solutions based on advanced numerical programs (FEM, AEM, DEM) to simulate the building response under accidental loading**
- **Incorporate dynamics or equivalent dynamics, linear/non-linear analysis, large displacements, energy dissipation (plastic hinges and yield lines) and failure criteria**
- **Complexity of FE models mainly depends on the 'dimension' of the problem investigated and level of approximation and refinement adopted**
- **Different level of complexity for materials (nonlinearities, damage, contact, temperature, rate sensitivity, etc.), element types (line, surface, volume, mass, spring, etc.), joints (continuum, beams, constraints, springs, component models, etc.)**

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5. KEY ELEMENT METHOD

- This design strategy is 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 entails further damage that violates the performance objectives
- 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
- No capacity redistribution is allowed
- This approach is the only rational one in retrofitting of existing buildings

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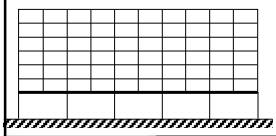
5. KEY ELEMENT METHOD

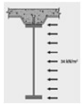
DESIGN STEPS

- **Identification of the key elements**
- **Design key elements to resist 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 key elements and any attached component**

EXAMPLE

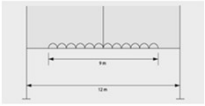
Key element ➡ Transfer girder





Horizontal direction:
- Side on the transfer girder

Codified accidental load (34 kN/m²)



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

50

6. SEGMENTATION METHOD

- **Spreading of failure is prevented/limited by isolating the failing part of a structure by segment/compartment borders**
- **Ensure that each part, (compartment or segment) can collapse independently without affecting the safety of the other parts**
- **Segmentation strategies can be based on either weak segment borders (fuse-hl strs) or strong segment (ALPM-vl strs) borders**
- **This method is desirable when the initial damage size is assumed to be of a large value**
- **Segmentation can be also combined with ALP methods, where ALP methods can be provided within the individual segments**

(Starossek, 2006)

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6. SEGMENTATION METHOD

- **WEAK SEGMENT BORDERS**
  - Allows the failure of a specific segment to take place without progression of failure to adjacent segments
  - Can be achieved by eliminating continuity between adjacent segments or reducing the stiffness to accommodate large deformations and displacements at segment borders, limiting the amount of force transmitted to the surrounding structure
  - Where alternative load paths are impractical or too expensive, segmentation by selectively eliminating continuity would be advantageous
- **STRONG SEGMENT BORDERS**
  - Designed to prevent an incipient collapse providing high local resistance that is able to accommodate relatively large forces
  - 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
  - Can be considered for vertical structures, such as multi-storey buildings with outrigger or belt trusses which can act along with vertical tying to allow redistribution of loads following local damage arresting falling debris and adding stability to surrounding structure

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7. CONCLUSIONS (Unidentified Threats)

- **Accidental actions not specifically considered by standards or indicated by the client or other stakeholders, or to any other actions resulting from unspecifiable causes**
- **Due to uncertainties in nature, magnitude and application point, the required structural performance cannot be determined**
- **Currently design strategies deemed to achieve an adequate level of robustness mainly seek to limit the extent of a localised damage, irrespective of the initiating cause**
- **Next presentation provides further details of practical analytical and simplified numerical methods**

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London **01.06.2022**

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

steelconstruct.com/eu-projects/failnomore

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London | 01.06.2022

Alternative Load Path Methods

Simplified Analytical and Numerical Approaches

Zeyad Khalil

Imperial College London

FAILNOMORE

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

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1

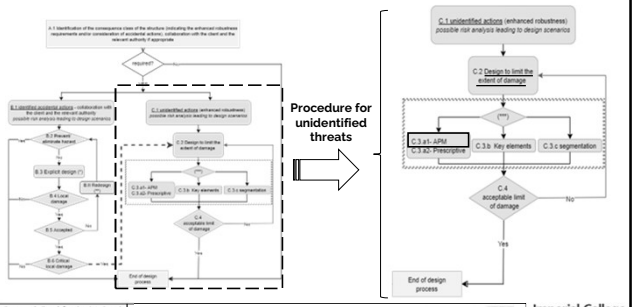
Outline

- **This presentation is organised as follows:**

1. General
2. Simplified analytical approaches
3. Simplified numerical approaches

2

1. GENERAL



3

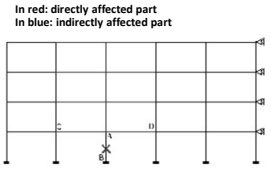
1. GENERAL

- Alternative load path (ALP) methods are performance-based design approaches which ensures that the structure allow the redistribution of loads in the case of the local failure of a supporting member
- The ALP design strategy is based on the limitation of the extent of localised failure by enhancing redundancy and considering the notional removal of columns for various local damage scenarios
- Such methods aim at demonstrating that the local damage is not spreading to an extent which is disproportionate to its original cause
- This goal can be achieved by providing the structure with adequate resources of ductility or deformation capacity and redundancy
- ALP methods do not require explicit modelling of specific actions associated to the damages; therefore, it is an event independent assumption, simplifying the analysis.

4

1. 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
  - The **indirectly affected part (IAP)**, which includes the rest of the structure



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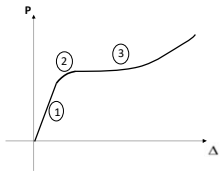
1. GENERAL

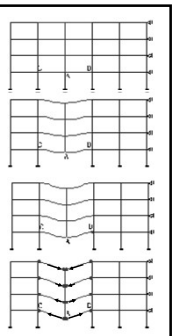
- **Visualisation of the global structural response**



Phase 1:
➡ Elastic response is observed

Phase 2:
➡ First plastic hinge forms
➡ Then, a complete plastic mechanism forms in the DAP

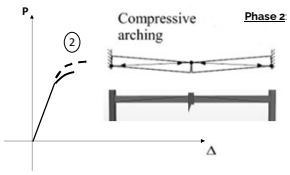
Phase 3:
➡ Significant displacements appear
➡ Development of 2nd order effects
➡ Tensile catenary



6

1. GENERAL

- **Visualisation of the global structural response**



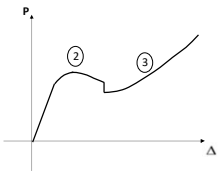
Phase 2 ⟹
- Plastic plate mechanisms in the slabs
- Plastic beam mechanisms in the DAP beams (steel or composite if the slabs are connected to the steel beams)

In addition, **arching effects** may have a considerable contribution to the overall resistance in the DAP beams further to the development of plastic mechanisms

7

1. GENERAL

- **Visualisation of the global structural response**



Phase 2 ⟹ 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

⬇

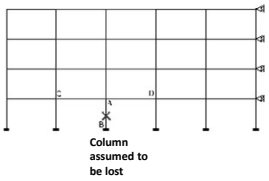
Catenary actions develop in the steel or composite DAP beams

8

1. GENERAL

- **Objectives are:**
  - Ensure that the **resistance** of the directly affected part (DAP) and of its components (beams and joints) is adequate;
  - Ensure that the different structural elements have a **sufficient ductility** and/or **rotation capacity** to reach the vertical displacement due to the column loss

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



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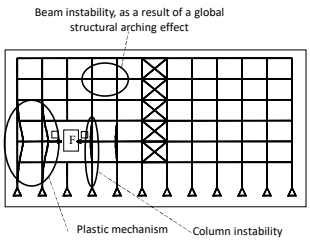
1. GENERAL



In addition, other possible relevant failure modes need to be checked:
- Buckling of the DAP columns adjacent to the lost column
- Global plastic mechanism in the IAP under the action of membrane forces transferred by the DAP to the IAP of the structure
- Buckling in compression of the upper beams of the DAP as a consequence of the development of arch effects

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1. 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 Prescriptive methods
  - 4.3 Analytical methods
  - 4.4 Simplified numerical approach
  - 4.5 Full numerical approach
  - 4.6 Prediction of the dynamic response from the static one

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1. 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 Prescriptive methods
  - **4.3 Analytical methods**
  - **4.4 Simplified numerical approach**
  - 4.5 Full numerical approach
  - **4.6 Prediction of the dynamic response from the static one**

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2. ANALYTICAL APPROACHES

- **The analytical 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**

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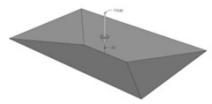
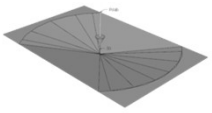
2. SIMPLIFIED ANALYTICAL APPROACHES

Contribution from the Slab

- The slab contribution to the overall resistance should be highlighted
- The plastic load resistance of the slab can be obtained by applying the classical first-order yield line theory
- Other methods are mentioned in the design manual and can account for the membrane forces that develop under large deflections for more accuracy
- Numerical models can also be used for this purpose





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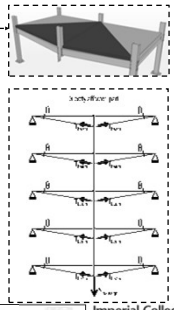
2. SIMPLIFIED ANALYTICAL APPROACHES

SIMPLIFIED METHOD FOR STRUCTURES WITH SIMPLE JOINTS

- No "plastic mechanism" robustness contribution may be expected
- Either the robustness is ensured by the slabs (yield mechanisms + membrane actions in the slab)
- Or the robustness is ensured by the DAP (catenary actions in the beams)



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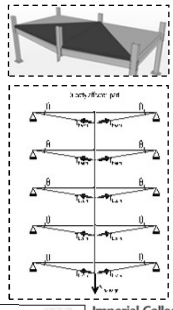
2. SIMPLIFIED ANALYTICAL APPROACHES

SIMPLIFIED METHOD FOR STRUCTURES WITH SIMPLE JOINTS

- The development of the membrane/catenary forces strongly depends on the stiffness of the indirectly affected part
- Slabs considered at each floor working as diaphragms
- Indirectly affected part may be assumed as infinitely stiff in the horizontal direction



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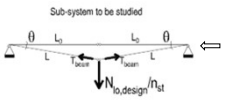
2. SIMPLIFIED ANALYTICAL APPROACHES

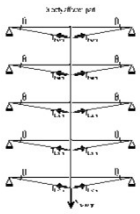
SIMPLIFIED METHOD FOR STRUCTURES WITH SIMPLE JOINTS

- **For a 2D sub-model:**

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

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





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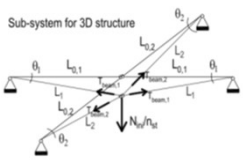
2. SIMPLIFIED ANALYTICAL APPROACHES

SIMPLIFIED METHOD FOR STRUCTURES WITH SIMPLE JOINTS

- **For a 3D sub-model:**

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



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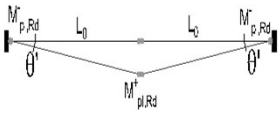
2. SIMPLIFIED ANALYTICAL APPROACHES

SIMPLIFIED METHOD FOR STRUCTURES WITH PARTIAL-STRENGTH JOINTS

- Development of a plastic mechanism in the directly affected part with plastic hinges forming at the level of the partial-strength joints
- Slab contribution can be added

$$N_{pl,i} = \frac{2.M^{-}_{Rd,i} + 2.M^{+}_{Rd,i}}{L} \qquad N_{pl} = \sum_i N_{pl,i}$$



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2. SIMPLIFIED ANALYTICAL APPROACHES

SIMPLIFIED METHOD FOR STRUCTURES WITH ARCHING EFFECT

- Arching effect can be mobilised in the beams of the directly affected part if the failure mode is associated to components in tension, in bending or in shear
- This means that the components in compression (column web in compression or beam flange and web in compression) have not reached their plastic resistance yet
- The design manual gives more details on how to estimate the additional resistance due to arching and will be demonstrated later in the worked examples

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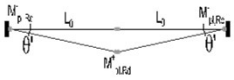
2. SIMPLIFIED ANALYTICAL APPROACHES

SIMPLIFIED METHOD FOR STRUCTURES WITH OVER-STRENGTH JOINTS

- Development of a plastic mechanism in the directly affected part with plastic hinges forming in the beams and not in the joints

$$N_{pl,i} = \frac{2.M^{-}_{pl,Rd,i} + 2.M^{+}_{pl,Rd,i}}{L} \qquad N_{pl} = \sum_i N_{pl,i}$$



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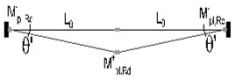
2. SIMPLIFIED ANALYTICAL APPROACHES

SIMPLIFIED METHOD FOR STRUCTURES WITH OVER-STRENGTH JOINTS

- "Beam arching effect" contribution cannot be activated here
- As the yielding zones are developing within the beam cross-sections, both parts of the cross-sections in the plastic hinges, respectively in tension and in compression, are yielded and so the resistance associated to the arching effect is equal to zero

$$N_{pl,i} = \frac{2.M^{-}_{pl,Rd,i} + 2.M^{+}_{pl,Rd,i}}{L} \qquad N_{pl} = \sum_i N_{pl,i}$$



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3. SIMPLIFIED NUMERICAL APPROACH

- **Robustness limit state**
  - **Sudden** column loss scenario
  - Prevent the collapse of the above floors
  - Maximum **dynamic** deformation exceeding the **Ductility limit**
- **Multi-level ductility-centred assessment framework**
  1. **Nonlinear static response**
  2. **Simplified dynamic assessment**
  3. **Ductility assessment**
- **Accounts for ductility, redundancy, energy absorption and dynamic effects**
- **No need for detailed nonlinear dynamic analysis**

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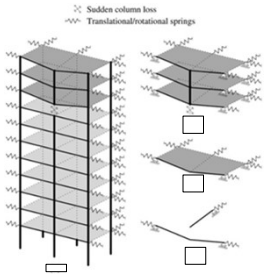
3. SIMPLIFIED NUMERICAL APPROACH

- **Structural idealisation**
  - Affected bay
  - Floor(s) above lost column
  - Single floor above lost column
  - Individual steel/composite beam above lost column
- **Response at higher levels can be assembled using low-level models**
- **Grillage approximation of floor can be assembled using individual beam models**
- **SDOF response of multiple floors can be assembled using floor model, when rigid columns are assumed**



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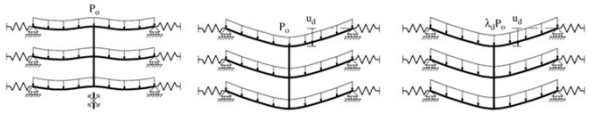
3. SIMPLIFIED NUMERICAL APPROACH

- **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 nonlinear static response of the structure is obtained with the exclusion of the damaged column such that the gravity loading is varied using a scale factor (λ) with $P = \lambda P_o$ and plotted against the static vertical displacement (u_s) at the location of the damage column.



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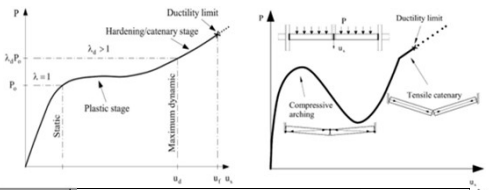
3. SIMPLIFIED NUMERICAL APPROACH

- **Nonlinear static response**
  - Takes into account: hardening, tensile catenary and compressive arching actions
  - Detailed and simplified models can be used at the desired level of structural idealisation as detailed in the design manual



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3. SIMPLIFIED NUMERICAL APPROACH

- **Simplified dynamic assessment**
  - The maximun 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.
    - After column loss, the structure accelerates from the rest where the gravity load exceeds the static structural resistance and where the difference between the work done by the load and the strain energy stored is transformed into kinetic energy
    - As the deformations increase, the static resistance exceeds the applied loading and the strain energy stored becomes more than the work done by the gravity load, which consequently leads to a continuous reduction in kinetic energy until the structure is brought back to rest at a maximum dynamic displacement
    - The maximum dynamic response is reached when the kinetic energy is reduced back to zero, i.e. when the work done by the gravity loads becomes identical to the energy absorbed by the structure
    - This gives rise to the concept of a "pseudo-static" response

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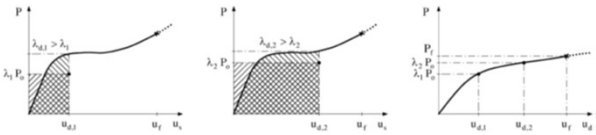
3. SIMPLIFIED NUMERICAL APPROACH

- **The maximun dynamic displacement associated with the sudden application of a gravity load λP_0 can be determined from the from 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 equating the hatched areas of the figure.**

$$\lambda_n P_o u_{d,n} = \int_0^{u_{d,n}} P du_s$$



(a) Dynamic response ($P = \lambda_1 P_o$) (b) Dynamic response ($P = \lambda_2 P_o$) (c) Pseudo-static response

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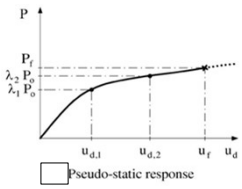
3. SIMPLIFIED NUMERICAL APPROACH

- **Ductility assessment**
  - The maximum dynamic displacement (u_d) obtained from the pseudo-static response at ($P=P_o$) is compared with the ductility limit (u_f)
  - The ductility limit (u_f) is determined as the minimum value of (u_d) such that the deformation demand exceeds the ductility supply in any of the joints
  - The deformations experienced by the joints corresponding to u_d can be determined
  - The ductility demands in the different components of the joint can then be obtained from the total joint deformations and compared to the ductility supply of the different components



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SUMMING UP

- **This presentation focused on:**
  1. Alternative load path methods to achieve robustness
  2. Simplified analytical approaches
     - Simplified method for structures with simple joints
     - Simplified method for structures with partial-strength joints
     - Simplified methods for structures with over-strength joints
  3. Simplified numerical approaches
     - Nonlinear static response
     - Simplified dynamic assessment
     - Ductility assessment

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London 01.06.2022

WORKED EXAMPLES

Florea Dinu [1]

[1] 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

1

1. Introduction
2. Steel structure in non-seismic area
3. Composite structure in non-seismic area
4. Steel structure in seismic area
5. Composite structure in seismic area

1. INTRODUCTION

- **Scope**: to demonstrate the applicability of the proposed guidelines for the design/evaluation for robustness of steel and composite steel-concrete building frames
- **The structures included in these worked examples are 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 design for robustness is done using two main approaches:**
  - **Identified accidental actions**
  - **Unidentified accidental actions**

2

TYPE 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 made in two configurations – one with steel columns and one with composite columns. In both cases, the beams and slabs are designed as composite

- **The design for robustness requires first the classification of the structure in terms of consequence classes for accidental actions**
- **All the structures included in these worked examples are included in Consequence Class 2b (Upper Risk Group)**

3

IDENTIFIED ACTIONS

- **Types of approaches for identified 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				X				X		
SS/S						X	X			
CS/S										

Recommended strategies for Consequence Class 2b (minimum requirements)

Additional strategies

4

UNIDENTIFIED ACTIONS

- **Types of approaches for unidentified 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	X				X	
SS/S	X		X			
CS/S	X					

Recommended strategies for Consequence Class 2b (minimum requirements)

Additional strategies

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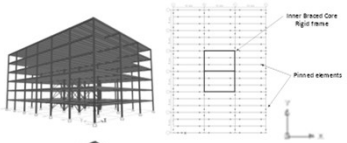
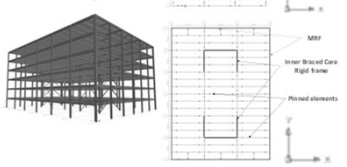
GEOMETRY AND STRUCTURAL SYSTEMS

- **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



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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_b = 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

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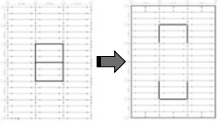
ACTIONS CONSIDERED IN THE DESIGN

- **SEISMIC DESIGN SITUATION - FOR SS/S and CS/S STRUCTURES (EUROCODE 8)**
  - Elastic response spectrum: Type 1
  - Ground type: B
  - Design ground acceleration, a_g =0.25 g
  - Behaviour factor q = 4.8 (dual frame CBF+MRF)
- **ADDITIONAL MEASURES WERE TAKEN FOR SEISMIC RESISTANT STRUCTURES:**
  - The braced spans were moved to the exterior (due to torsional effects)
  - MRFs were added on the perimeter on all sides (otherwise no differences compared to NS structures)
  - A dual steel frame seismic resistant system requires a minimum of 25% contribution from the MRFs to the total capacity. This condition led to the following:
    - The cross-section of beams and columns in the MRFs were increased to fulfil the conditions for dual frames
    - Intermediate columns were introduced on the short sides (X) of the perimeter. The spans remained unchanged at the interior.



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1. Introduction
2. Steel structure in non-seismic area
3. Composite structure in non-seismic area
4. Steel structure in seismic area
5. Composite structure in seismic area

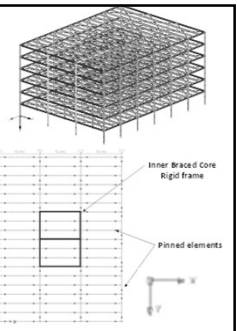
2. STEEL STRUCTURE IN NON-SEISMIC AREA

- **Identified actions**
  - **Seismic**
    - Prescriptive method
- **Unidentified actions**
  - **Prescriptive approach (Tying method)**
  - **Alternate load path method (ALPM)**
    - Analytical approach
    - Full numerical approach
  - **Segmentation**

9

ELEMENT SECTIONS

Element	Section	Steel grade	ID	ULS utilization factor	SLS deflection (rare combination)
Columns Y-facades	HEB 340	S355	1	0.95	-
Columns X-facades	HEB 360	S355	2	0.98	-
Inner columns	HEM 300	S355	3	0.95	-
Beams X-facades	IPE500	S355	A	0.52	43.8 mm
Beams Y-facades	IPE500	S355	A	0.77	29.8 mm
Inner X-beams	IPE550	S355	B	0.61	45.9 mm
Inner Y-beams	IPE600	S355	C	0.89	29.1 mm
Inner core beams	HEA300	S355	D	0.90	6.5 mm
Inner core braces	CHS 219.1x6.3	S355	-	0.90	-



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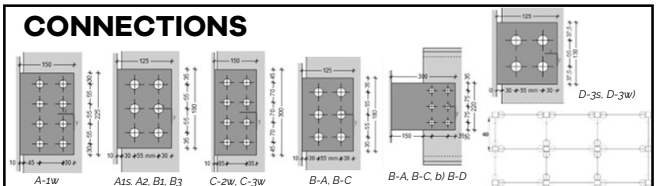
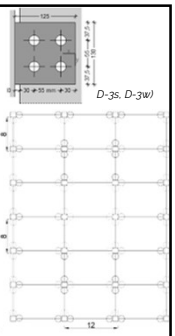
CONNECTIONS



Position s - strong axis w - weak axis	ULS load (kN)	Resistance (kN)	Failure mode	UF
A1s / A2	130	196	Fin plate in bearing	0.66
A1w	240	255	Fin plate in bearing	0.94
B1 / B3	180	196	Fin plate in bearing	0.92
C2w / C3w	430	443	Fin plate in bearing	0.97
D3s	60	102	Beam web in bearing	0.59
D3w	90	102	Beam web in bearing	0.88
BA / BC	180	196	Fin plate in bearing	0.92
BD	180	185	Fin plate in bearing	0.97



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IDENTIFIED ACTIONS

Seismic

Prescriptive method

- The structure considered in this example has been designed for ULS/SLS conditions only (persistent design situation). No calculations have been conducted with respect to any accidental 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
    - ✓ **Uniformity in plan and in elevation**
    - ✓ **Torsional resistance**
    - ✓ **Redundant systems**
    - ✓ **Direct load paths** (e.g., column supported on beam not recommended)
    - ✓ **Floor diaphragm**
  - Vibration control (base isolators, dampers – passive, active, semi-active)
  - Strength and stiffness
  - Ductility
    - ✓ Cross-section class (e.g., HEA300 class 3 sections for the beams may be replaced with class 1 HEB / IPE)
    - ✓ Joint typology (pinned joints replaced by ductile semi-rigid joints, allowing the formation of plastic hinges in the joints and dissipating part of the seismic induced energy)

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UNIDENTIFIED ACTIONS

Alternate load path method (ALPM)

Prescriptive method (Tying method)

This example shows the application of the tying method for beams and their connections (horizontal tying).

- **ACTIONS CONSIDERED FOR THE ACCIDENTAL DESIGN SITUATION**
  - Permanent loads DL
  - Live loads LL
  - No specific accidental action is taken into account

Tying forces are determined 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_p	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
$g_{k,HEB360}$	1,22 kN/m	$g_{k,HEM300}$	1,22 kN/m
$q_{k,HEB360}$	0,907 kN/m	$q_{k,HEM300}$	1,06 kN/m
$g_{k,HEB360}$	1,06 kN/m	$g_{k,HEM300}$	2,38 kN/m
$q_{k,HEB360}$	1,42 kN/m	h	4 m
h	4 m	n IPE550	4
n IPE550	1,5		
g_k facade	4 kN/m		
T_p	400,5 kN	T_i	694,2 kN

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

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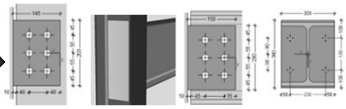
JOINTS VERIFICATIONS FOR TYING FORCES

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

REDESIGNED CONNECTIONS



B1/B3 *C2* *D3s/D3w* *3-3*

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

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UNIDENTIFIED ACTIONS

Alternate load path method (ALPM)

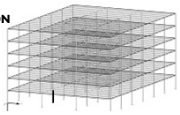
Analytical approach

This example gives information about 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
- **ELEMENTS UNDER INVESTIGATION**
  - Beam B1/B3 – IPE550
  - Beam C2w/C3w – IPE600
- **ASSUMPTIONS FOR JOINTS**
  - Solution 1: simple joints
  - **Solution 2: partial-strength joints**



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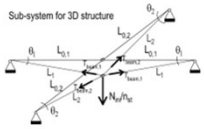
Solution 1 - Tying forces for simple joints

The procedure consists in solving the system of 4 equations



3D Structures with simple joints	
Eq. 1	$\frac{N_{inj}}{n_{st}} = 2.T_{beam,1}.\sin\theta_1 + 2.T_{beam,2}.\sin\theta_2$
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_{inj} is taken from the structural analysis by considering the accidental load case combination.

N_{inj}	n_{st}	E	A_1	$L_{0,1}$	A_2	$L_{0,2}$
4078.51 kN	6	210000 MPa	134 cm²	12 m	156 cm²	8 m

n_{st} – number of stories from DAP (activated)

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

Solution

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

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Solution 1 - Tying forces for simple joints

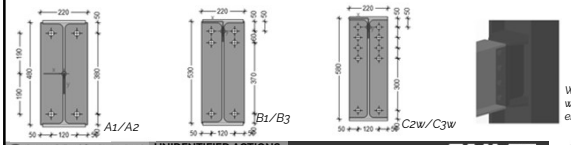
REMARKS
- The results indicate that a redesign of the structure for robustness is needed as the joints are not able to sustain such significant loads (see Tying Method).

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

T_p	268,8 kN	T_i	499,2 kN

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 will be replaced by flush end-plate joints.



A1/A2 *B1/B3* *C2w/C3w*

Welded part for weak axis flush end-plate joints

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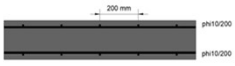
Solution 2 - Partial-strength joints, considering the following contributions:

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

If the sum of the above contributions is not sufficient, larger deformations develop and membrane effects in the beams are activated similarly as in the simple joint example. As this requires greater rotational capacity in the joints, the robustness design will be here performed alternatively by optimizing the three contributions mentioned above, so that no membrane effects occur.

CONTRIBUTION FROM THE SLAB

Class	t	c	Steel	A_{sx} (top and bottom)	A_{sy} (top and bottom)	M_{Rd} (sagging/ hogging)	Failure mode
C30/37	20 cm	20 mm	B500S	3.93 cm²/m	3.93 cm²/m	26.9 kNm	Yielding of reinforcement



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

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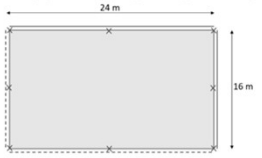
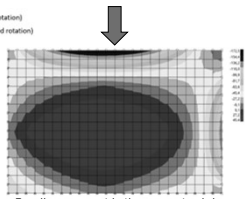
CONTRIBUTION FROM THE SLAB



Statical system of the concrete slab after column loss

The concrete slab contribution is expressed through the vertical point force P_{slab} (where the column is lost) needed for a plastic mechanism to develop.

Applying the accidental loading case (1 x G + 0.5 x Q) of 6.5 kN/m²



Bending moment in the concrete slab after column loss (M_{Ed} = -172.5 kNm)

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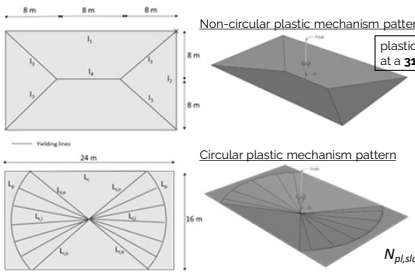
CONTRIBUTION FROM THE SLAB (2)



Non-circular plastic mechanism pattern

Circular plastic mechanism pattern

virtual works principle

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

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

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

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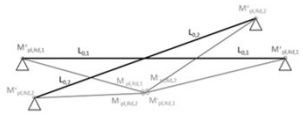
CONTRIBUTION FROM THE STEEL BEAM MECHANISM

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

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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 (i.e., a joint or a beam) in compression. In such conditions, an arch effect can be mobilised within the beams of the directly affected part, as soon as the plastic mechanism is formed.

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***
- Contribution from 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 THEN:

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

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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 of the directly affected part will develop 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 they disappear once large deformations are reached.

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

REDESIGN OF THE STRUCTURE WITH PARTIAL-STRENGTH JOINTS

In this worked example, the steel structure has been kept as it is (designed with internal forces with the simple joint modelling). Modelling semi-rigid joints as hinges is still a valid and safe assumption if these joints have enough ductility and rotation capacity.

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

- Modify the slab design to increase its contribution (slab mechanism ➤);
- Strengthen the joints in one or both directions to increase the contribution (beam mechanism ➤);
- Reinforce compression components to activate the arch effect (**activate arch effect**).

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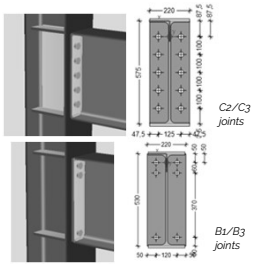
REDESIGN OF THE STRUCTURE WITH PARTIAL-STRENGTH JOINTS (2)

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 moment resistance of the joint and thus the beam mechanism.



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

CONTRIBUTION FROM THE SLAB

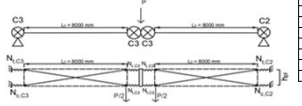
- As no changes have been made to the slab, the contribution of this component remains unchanged (N_{slab} = 313.6 kN)

CONTRIBUTION OF THE BEAMS MECHANISM

- N_{pl} is now equal to 334.7 kN

CONTRIBUTION FROM THE ARCHING EFFECTS

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



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

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_{tot}	1369.4 kN
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_u	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)	$N_{u,Rd}$	231.7 kN

$$N_{arch} = 51.0\ kN$$

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

$$N = N_{slab} + N_{pl} + N_{arch} = 313.6 + 334.7 + 51.0 = 699.3\ kN > 694.2\ \text{kN} \quad ✓$$

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.

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UNIDENTIFIED ACTIONS

Alternate Load Path Method

Full numerical approach

This example gives information about the design against unidentified threats using the ALPM through a full numerical approach.

ACTIONS FOR THE ACCIDENTAL DESIGN SITUATION

- Permanent loads DL
- Live loads LL

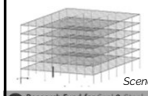
COMBINATION OF ACTIONS FOR ACCIDENTAL DESIGN SITUATION

$$DL + 0.5 \times LL$$

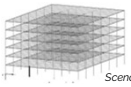
SCENARIOS CONSIDERED

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

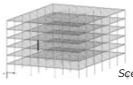
This example illustrates 3 column loss scenarios. However, in practical application, other column loss scenarios might be required). Therefore, it is up to the engineer to define which scenarios might be possible and which of them are the most relevant for the robustness design of the structure.



Scenario 1



Scenario 2



Scenario 3

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STRUCTURAL ANALYSIS

- The full numerical approach will be addressed using the finite element model developed for the ULS/SLS design of the structure. The aim is to remove a column and let membrane effects develop in the ties in the first step and then verify if the ties (members and joints) can withstand these tensile forces.
- Methodology and assumptions:
  - 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 without lost columns 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 force replaces the column;
  - 3rd step - A force of same magnitude in opposite direction is gradually applied at the same node. Load steps of 0.025 are used to ensure convergence. At the end of the analysis, the statical system corresponds to a complete column loss.

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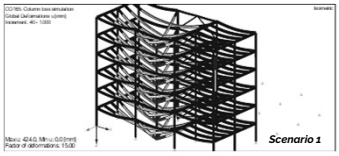
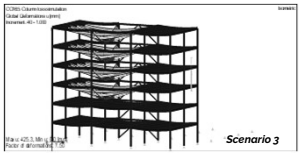
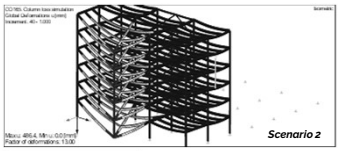
RESULTS



Scenario 1



Scenario 3



Scenario 2

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

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SCENARIO 1: VERIFICATIONS

- When a column is lost, compression force in neighbouring columns increases. However, these forces stay lower than the design compression forces from ULS ➔ no columns redesign required.
- The IPE550 members were designed to fulfil the SLS requirements (limitation of the deflection). In this case, the resistance of these members is still sufficient in case of a column loss.
- The IPE600 are not sufficient for the high tensile forces (15% of exceedance).

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 in the same order of magnitude that in Scenario 1 ➔ Scenario 3 is not further investigated in the following.

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REDESIGN OF STRUCTURE

SCENARIO 1 VERIFICATIONS

- Due to the section change of the **IPE600** to **IPE750x137**, the internal force distribution are modified

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

The utilization factor of the IPE750x137 is exceeded by 3%. <u>This exceedance can be considered as acceptable.</u>

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

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	

For joints C2w and C3w, no reasonable redesign could be found
➔ pinned joints replaced with semi-rigid joints (partial-strength)

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UNIDENTIFIED ACTIONS

Segmentation

Weak segment borders / Strong segment borders

REMARKS

- This method can be used either alone or in combination with other measures (e.g., local strengthening) or methods (e.g., ALPM). When the outputs of the ALPM indicate the need for redesign, the segmentation method may be used as an alternative solution to limit the extension of damage.
- In the case of the current low-rise building, a weak segmentation border strategy could be chosen. As it has been highlighted in results of both analytical and numerical approaches, the pinned fin plate joints designed for ULS are not able to withstand large tensile forces from membrane effects when considering a column loss scenario.
- Practically, these joints act as "fuses" in case of a column loss and the collapse will be limited to the area directly affected by the column loss (horizontal limitation of damage). If the joints response is ductile, they will develop large deformations before collapse, so preventing from a sudden brittle failure.

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1. Introduction
2. Steel structure in non-seismic area
3. Composite structure in non-seismic area
4. Steel structure in seismic area
5. Composite structure in seismic area

3. COMPOSITE STRUCTURE IN NON-SEISMIC AREA

Identified actions

- **Impact**
  - Equivalent static approach
- **External explosion**
  - Equivalent SDOF approach
- **Localised fire**
  - Localised fire models

Unidentified actions

- **Alternate load path method (ALPM)**
  - Prescriptive approach (Tying method)
  - Full numerical approach
- **Key element**

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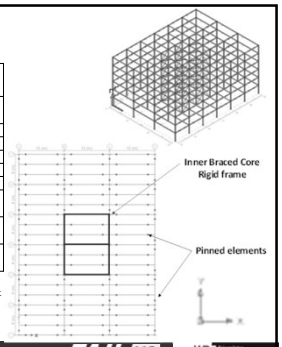
ELEMENT SECTIONS

Element	Type	Section	Utility		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 (see next slide).



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CONNECTIONS

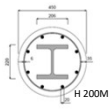
Position	Connection type	Shear resistance (kN)	Moment resistance (kNm)	Failure mode	UF
Perimeter	Header plate	289.38	-	Shear resistance of bolt group	0.73
	Fin plate	297.96	-	Shear resistance of bolt Group	0.71
Internal	Header plate	289.38	-	Shear resistance of bolt group	0.64
	Fin plate	265.89	-	Bolt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

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

COMPOSITE COLUMNS WITH EQUIVALENT PROPERTIES

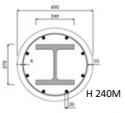
Perimeter columns

HD 360x162 <=>



Interior columns

HD 400x216 <=>



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IDENTIFIED ACTIONS

IMPACT

Equivalent static approach

This example gives information about the design against impact due to accidental collision of a vehicle, using the equivalent static approach.

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.

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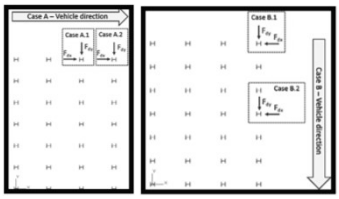
THE IMPACT LOADS

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

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** is made on the full 3D model using the software SCIA®. The section of the elements are those resulted from the initial design.
- The design of the composite columns was made using the software A3C®.

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RESULTS

Results of linear static analysis for impact on standard steel columns

Case	Section	Loading F_{dx} (kN)	Loading F_{dy} (kN)	Bottom support	UF (-) S355	UF (-) 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	1.46	1.10
				Hinged	1.72	1.30

Results of linear static analysis for impact on composite columns

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

- The results for S355 columns show that the yield strength is exceeded for both pinned and fixed conditions, with UFs up to 1.72.
- Using S460 steel grade, a considerable improvement is observed in terms of utilization factors.

COMPOSITE STEEL-CONCRETE COLUMNS

- UF are substantially higher ← pre-design of the sections and supporting conditions. Using a static approach for the impact analysis, the steel profile takes 65% to 70% → higher UFs.

- To mitigate the impact consequences:
  - Higher steel grade or larger cross section;
  - Column oriented to obtain maximum impact resistance / design bottom connections as fixed;
  - More advanced approaches to assess more accurately the capacity.

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UNIDENTIFIED ACTIONS

Alternate Load Path Method

Full numerical approach

This example gives information about the design against unidentified threats using the ALPM through a full numerical approach.

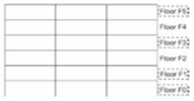
COMBINATION OF ACTIONS FOR THE ACCIDENTAL DESIGN SITUATION

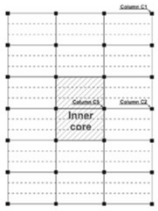
- Permanent loads DL
- Live loads LL

→ $DL + 0.5 \times LL$

SCENARIO CONSIDERED:

- Corner column (C1) at stories 0, 1, 3 and 5;
- Façade column (C2) at stories 0, 1, 3 and 5;
- Braced core columns (C5) at stories 0, 1, 3 and 5.





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STRUCTURAL ANALYSIS

- The objective of this analysis is to evaluate the behaviour of the building in case of accidental situation (column removal). The calculations are made using the software SAFIR®.
- A total of 20 simulations are made and divided into 2 different groups according to the assumed beam-to-column joint configuration:
  - 12 simulations with all pinned beam-to-column joints.
  - 8 simulations with rigid beam-to-column joints.
- In the cases where the column C1 is removed, two different assumptions are defined:
  - All beam-to-column joints are pinned (C1 "All pinned joints");
  - Rigid beam-to-column joints at the corner where the column is removed (C1 "Rigid joints").
- In the cases where the column C2 is removed, two different assumptions are defined:
  - All beam-to-column joints are pinned (C2 "All pinned joints");
  - Rigid beam-to-column joints where the column is removed (C2 "Rigid joints").

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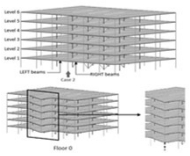
RESULTS

Maximum vertical displacements

Max. vertical displacement (m)	Floor	All pinned joints	Rigid joints
C1 Corner column	F0	1.340	0.081
	F1	1.340	0.083
	F3	1.320	0.088
	F5	1.380	0.720
C2 Façade column	F0	0.670	0.610
	F1	0.670	0.600
	F3	0.670	0.550
	F5	0.670	0.250
C5 Center core column	F0	0.016	-
	F1	0.017	
	F3	0.018	
	F5	0.018	

Detailed results for case C2

Maximum beam axial forces (Case C2)	Pinned joints: Left beam (kN)	Pinned joints: Right beam (kN)
Level 1	1381.6	1381.2
Level 2	1327.6	1326.8
Level 3	1340.4	1339.5
Level 4	1338.2	1337.4
Level 5	1337.6	1336.7
Level 6	1332.5	1331.7



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CONCLUSIONS

- Loss of column C1:
  - The structure shows very high vertical displacement as the only contribution in resisting the gravity loads is provided by the cantilevered concrete slab (beams have pinned ends);
  - Robustness behaviour can be improved by:
    - Reinforcing the beam-column joints along the vertical alignment of the columns (pinned -> semi-rigid - > rigid). The use of semi-rigid/rigid joints provides additional flexural capacity;
    - Improving the cantilever capacity of the slab (additional reinforcement at the corners of the building).
- Loss of columns C2 and C5:
  - The displacements are much smaller than for the corner column loss and the load is distributed through the floors;
  - Lateral displacements in columns adjacent to the lost column are small indicating the loads are relatively uniformly redistributed on all floors above the missing column;
  - Connections need to be redesigned;
  - These column loss scenarios do not lead to progressive collapse of the structure, but only to localised damage.

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FAIL NO MORE

1. Introduction
2. Steel structure in non-seismic area
3. Composite structure in non-seismic area
4. Steel structure in seismic area
5. Composite structure in seismic area

4. STEEL STRUCTURE IN SEISMIC AREA

- **Design approaches**
  - **Identified actions**
    - **External explosion:**
      - Equivalent SDOF approach
      - Full dynamic approach
    - **Internal explosion:**
      - Equivalent static approach
      - Dynamic approach (TNT equiv. method)
    - **Seismic:**
      - Advanced numerical analysis (multi-hazard)
  - **Unidentified actions**
    - **Alternate load path method (ALPM):**
      - Prescriptive approach (Tying method)
      - Simplified prediction of dynamic response
      - Full numerical approach

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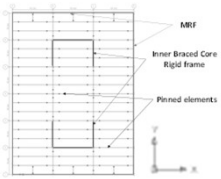
CROSS SECTIONS –UTILIZATION RATIOS

Columns	Section	Steel grade	UF
Corner columns	HE550B	S355	0.49
Perimeter columns	HE500B	S355	0.71
Inner Core columns	HD400X463	S355	0.95

Braces	Storey	Section	Steel grade	UF
Y Direction	1-3	HEA320	S355	0.41
	4	HEA260	S355	0.43
	5	HEA220	S355	0.46
	6	HEA200	S355	0.39
X Direction	1-3	HEB340	S355	0.41
	4-5	HEA320	S355	0.27
	6	HEA260	S355	0.26

Beams	Direction[1]	Storey	Section	Steel grade	UF	
					Strength	Deflection[2]
Perimeter beams	X	1-6	IPE550	S355	0.278	0.023
	Y	1-6	IPE600	S355	0.302	0.153
Interior beams	X	1-6	IPE550	S355	0.546	0.85
	Y	1-6	IPE550	S355	0.909	0.928
[4]Inner core beams	X	1-3	[3]H800	S460	0.936	-
		4-5	HEM800	S460	0.953	-
		6	HEM700	S460	0.789	-
	Y	1-3	HEM500	S460	0.859	-
		4-6	HEB500	S460	0.878	-

[1]See Figure for the orientation of the axes
[2]Deflection verification criteria: L/250 for secondary beams, L/350 for main beams
[3]H800 is a built-up section, h=814 mm, b=380 mm, t_f=50 mm, and t_w=30 mm.



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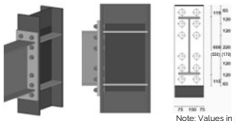
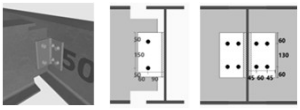
CONNECTIONS



Note: Values in parenthesis are for IPE550 beams



Moment resisting connections

Position Beam / column	Connection type	Moment resistance $M_{j,Rd}$(kNm)	Shear resistance (kN)	Failure mode in flexure	UF*	$\frac{M_{j,Rd}}{M_{pl,b}}$
A/1, A/7 IPE600-HEB550	Extended end plate	1173	1516	End plate in bending	0.29	0.94
A/1, A/7, A/2-6 IPE600-HEB500	Extended end plate	1169	1387	End plate in bending	0.26	0.94
1/A - 1/D IPE550-HEB500	Extended end plate	957	1409	End plate in bending	0.15	0.97

Note:
* Utilisation factor is defined for ULS, persistent design situation
** $M_{pl,b}$ is the plastic resistance of the beam

Pinned connections

Position	Story	Connection type	Shear resistance (kN)	Failure mode	UF*
A/1-7, D/1-7 IPE550-IPE600	1-6	Bolted angle cleat	196	Sec. beam bolts in shear	0.72
B/1-7, C/1-7 IPE550-IPE550	1-6	Bolted angle cleat	196	Sec. beam bolts in shear	0.72
B/2, B/5, C/2, C/5 IPE550-HEM500	1-3	Bolted angle cleat	196	Sec. beam at notch	0.67
B/2, B/5, C/2, C/5 IPE550-HEB500	4-6	Bolted angle cleat	196	Sec. beam bolts in shear	0.65

Note:
* Utilisation factor is defined for ULS, persistent design situation

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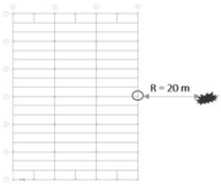
IDENTIFIED ACTIONS

External explosion

Equivalent single-degree-of-freedom approach

- **ACTION FOR THE ACCIDENTAL DESIGN SITUATION**
  Blast action A_{Ed}
- **DEFINITION OF BLAST SCENARIO**
  Column considered is perimeter column located in the middle of the long façade of the building
  - Standoff distance **20 m**
  - Explosive charge **100 kg of TNT**
- **STRUCTURAL ANALYSIS**
  A linear elastic analysis is performed using the simplified dynamic approach.



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- **BLAST LOADING PARAMETERS**
  - 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 → $\alpha_i = tan^{-1}\left(\frac{H_c}{W^{\frac{1}{3}}}\right) = tan^{-1}\left(\frac{1}{100^{\frac{1}{3}}}\right) = 12.158^{\circ}$

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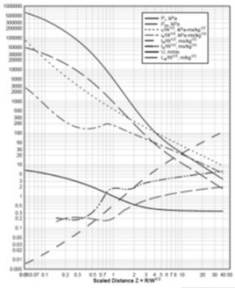
- **BLAST LOADING PARAMETERS (2)**
  - Next, using the chart for free-air bursts, the blast loading parameters are determined:

- 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)

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External explosions (high energy explosives)
- Standoff distance
- Direct blast effects vs. notional column removal
- (Secondary) Fragmentation

Codec Project (2018)

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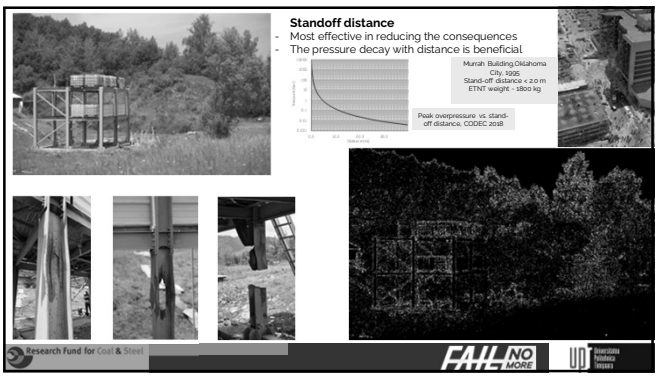

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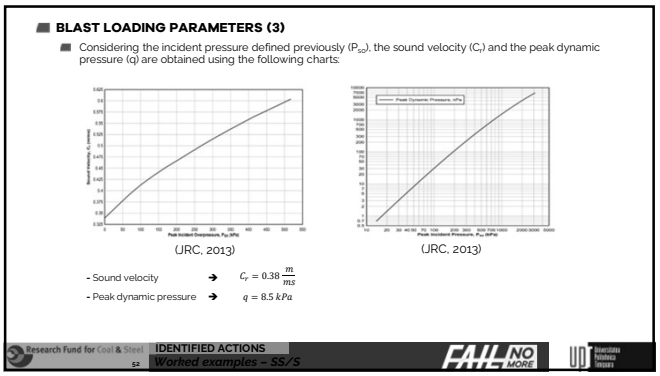

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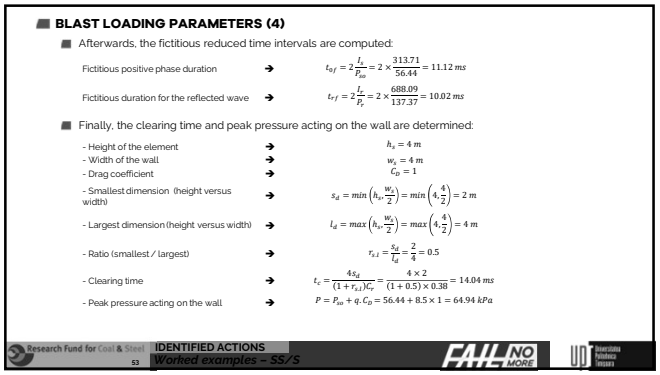

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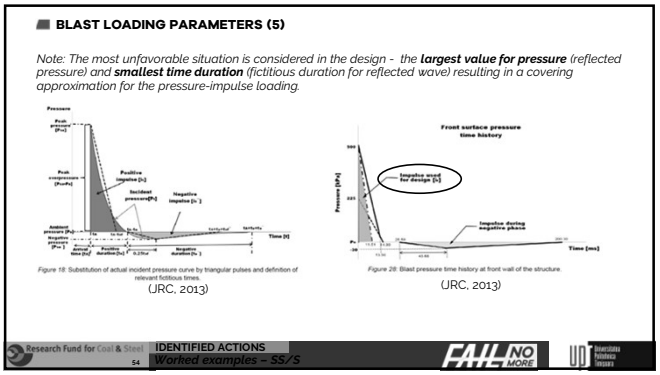

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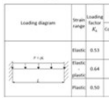
SINGLE DEGREE OF FREEDOM (SDOF)

- In this phase, the column is transformed in an equivalent SDOF system.
- The first step consists of calculating the load caused by the reflected pressure on the column. For the computation, a tributary width of 5m was assumed for the panel in front of the column.
- Since the analysis is static, DLF factor is used to consider the dynamic effects. The first iteration is performed using a **DLF of 1.4** to amplify the loading. Additionally, **DIF factor of 1.2** may be applied to the yield strength owing the strain rate effect.
- Further on, 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.

Distributed load from the blast on the column → $F_d = P_r w_p = 137.37 \times 5 = 686.85\ \frac{kN}{m}$

Point load from the blast on the column → $F_p = F_d h_c = 686.85 \times 3.5 = 2404\ kN$

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 Concentrated mass*	Mass factor K_M Uniform mass	Load-mass factor K_{LM} Concentrated mass*	Load-mass factor K_{LM} Uniform mass	Maximum resistance R_m	Spring constant k	Effective spring constant k_E	Dynamic reaction V
	Elastic	0.53	--	0.81	--	0.77	$\frac{12M_p}{L}$	$\frac{384EI}{L^3}$	--	$0.36R+0.14V$
	Elastic-Plastic	0.64	--	0.58	--	0.78	$\frac{16M_p}{L}$ (M_{pm})	$\frac{384EI}{L^3}$	$\frac{307EI}{L^3}$	$0.39R+0.11V$
	Plastic	0.50	--	0.33	--	0.66	$\frac{8M_p}{L}$ (M_{pm})	0	--	$0.38R_m+0.11V$

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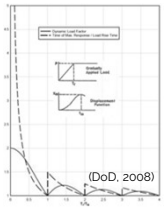
SINGLE DEGREE OF FREEDOM (SDOF)

- Thus, 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)

- Effective mass → $M_e = \frac{G_r \cdot h_c \cdot K_M}{g} = \frac{1.834 \times 3.5 \times 0.50}{9.81} = 327.3\ kg$
- Effective stiffness → $K_e = K_r K_L = 47471.8 \times 0.64 = 30382\ \frac{kN}{m}$
- Natural period of vibration → $T_r = 2\pi\sqrt{\frac{M_e}{K_e}} = 2 \times \pi \sqrt{\frac{327.3}{30382}} = 0.0206$
- Ratio between the fictitious duration of the reflected wave and the natural period → $\frac{t_{rf}}{T_r} = 0.49$
- Second iteration → $DLF = 1.6$
- Maximum applied moment → $M_{max} = \frac{F_p \cdot h_c}{8} DLF = \frac{2747.4 \times 3.5}{8} \times 1.6 = 1683\ kNm$
- Resistance force → $R_m = \frac{8(2M_{Ed})}{h_c} = \frac{8 \times 2 \times 550.4}{3.5} = 2516\ kN$

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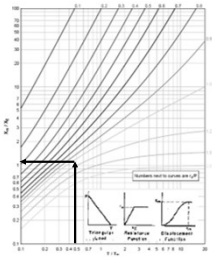
RESULTS

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

Ratio $\frac{R_m}{F_p} = 1.05 \quad \frac{t_{rf}}{T_c} = 0.49$

A ductility demand μ_t=1.05 (χ_M/χ_E) was obtained. Consequently, after the elastic displacement is determined, a maximum displacement of 87 mm was obtained.

- The process is performed similarly for the maximum response duration.



(DoD, 2008)

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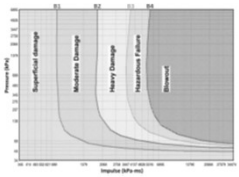
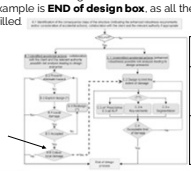
RESULTS (continued)

- The ductility demand will be compared with a capacity to assess if the damage is acceptable
- Using the pressure impulse relationships and response limits, one may establish a design objective.
- Hence, for this example, B1 objective (superficial damage) was chosen, and ratio $\frac{R_m}{R_{MAX}} = 1.05$ was obtained.
- Consequently, the damage may be considered acceptable (5% exceedance only).
- According to the chart for the design for robustness, from box B.6, the outcome for this example is **END of design box**, as all the requirements are considered fulfilled.



(CSA S850-12)



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	-
	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	-

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IDENTIFIED ACTIONS

External explosion

Full dynamic approach

ACCIDENTAL ACTION

Blast action A_{Ed}

DEFINITION OF BLAST SCENARIO

- For a relevant comparison, the blast scenario is the same as for the equivalent SDOF approach
- Loading parameters:
  - standoff distance R = 20 m;
  - explosive charge W = 100 kg of TNT;
  - tributary width of the column = 5 m (2.5 m on each side);
  - the blast pressure is considered to act on the 1st and 2nd stories columns.

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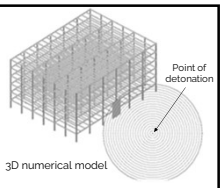
STRUCTURAL ANALYSIS

- A nonlinear dynamic analysis was performed using applied element method **(AEM)** on a full 3D model in Extreme Loading for Structure software **(ELS)**.



3D numerical model

COMPUTATION

- **The analysis is performed in two steps:**
  - 1st step: the permanent and live loads are applied on the structure in a nonlinear static analysis.
  - 2nd step: the charge is detonated, and the blast load is applied in a nonlinear dynamic analysis:
    - The time step for the analysis is $1E^{-6}$ s
    - Only the positive phase of the blast is considered
    - No reflection from the ground is considered

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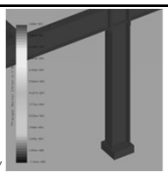
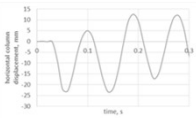
RESULTS

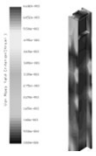
- The maximum horizontal displacement at the mid-height of the column is 24 mm.
- The maximum plastic strain amounts 1%.



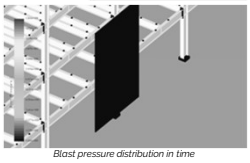
Strain history



Horizontal displacement at the mid-height of the column



Maximum plastic strain



Blast pressure distribution in time

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CONCLUSIONS

Full dynamic approach vs. equivalent SDOF approach

- The displacement in the full nonlinear dynamic analysis is less than the value obtained using tabular method (**24 mm** vs. **87 mm**) – **Box B.6** from robustness design flowchart;
- Nonlinear analysis can account for distribution of plasticity in the element;
- Full 3D model can account for real boundary conditions and interactions between elements;
- Full dynamic approach and 3D modelling can account for sequential application of blast pressure on the surface (different arrival times along the column length).

- Note that, in case of near field blasts, the effects can be amplified by the uplift pressure against the adjoining floors, which can result in higher dynamic effects and even risk of progressive collapse (Dinu et al. 2018).

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IDENTIFIED ACTIONS

Seismic

Advanced numerical analysis (multi-hazard)

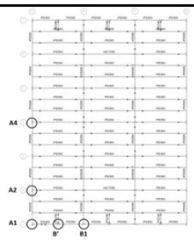
- **ACTIONS FOR THE ACCIDENTAL DESIGN SITUATION**
  - Permanent loads DL
  - Live loads LL
  - Seismic action A_{Ed} corresponding to ULS
- **COMBINATION OF ACTIONS FOR THE ACCIDENTAL DESIGN SITUATION**

  $DL + 0.3 \times LL + A_{Ed}$
- **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.



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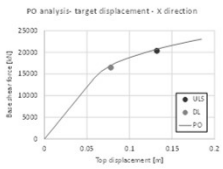
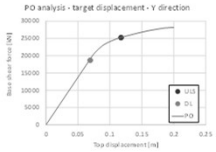
STRUCTURAL ANALYSIS

- 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.





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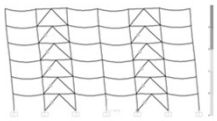
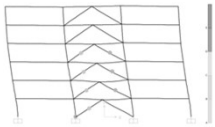
SEISMIC DEMANDS

- 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

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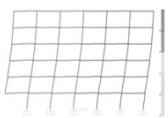
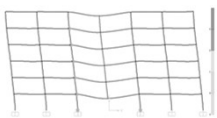
COLUMN REMOVAL AFTER THE EARTHQUAKE

- 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).

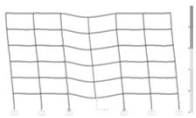
PLASTIC MECHANISM AFTER COLUMN REMOVAL FOR THE CONSIDERED SCENARIOS



Case A1



Case A2



Case A4

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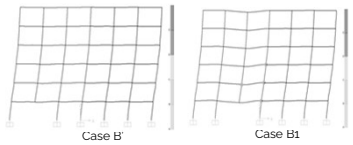
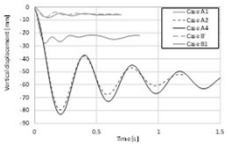
RESULTS

PLASTIC MECHANISM AFTER COLUMN REMOVAL FOR THE CONSIDERED SCENARIOS



TIME HISTORY RESPONSE FOR COLUMN REMOVAL SCENARIOS



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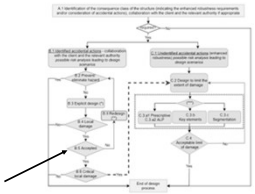
CONCLUSIONS

- 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 – Box B.5 ➔ End of design – robustness design flowchart.
- The level of damage in the elements (given by the level of plastic deformation in the plastic hinges) is small.
- Other performance objective (e.g., collapse prevention) may be employed to assess the structural behaviour.



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UNIDENTIFIED ACTIONS

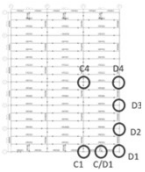
Alternate Load Path Method ALPM

Full numerical approach

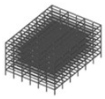
This example gives information about the design against unidentified threats using the ALPM and nonlinear dynamic analysis.

- **ACTIONS FOR THE ACCIDENTAL DESIGN SITUATION**
  - Permanent loads DL
  - Live loads LL
  - No specific accidental action is considered
- **COMBINATION OF ACTIONS FOR THE ACCIDENTAL DESIGN SITUATION**
  $DL + 0.5 \times LL$
- **COLUMN LOSS SCENARIOS**
  - 7 column removal locations are considered, see figure
  - All locations are at the ground floor
- **STRUCTURAL ANALYSIS**
  - 3D model
  - Extreme loading for structure ELS software

Column loss scenarios



View of the 3D model



The dynamic effects caused by the column loss are considered implicitly by means of the removal duration parameter.

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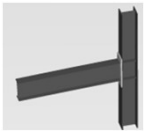
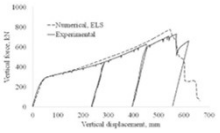
CALIBRATION OF THE NUMERICAL MODEL

- **Two experimental tests were used to calibrate the numerical model:**
  - **CODEC project:**
    - Experimental testing of a 3D steel frame system under column loss
    - Relevant for the local response of connections and global behavior (transition from flexural to catenary response, ultimate capacity)
  - **Equaljoints project:**
    - Monotonic and cyclic testing on seismic beam-to-column joints
    - Relevant for response of large size beam-to-column joints of similar arrangements
    - Note that the joints were tested under flexural conditions only

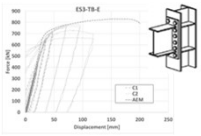


Joint configuration tested in Equaljoints

3D model tested in CODEC (stress map colours)



Force-displacement, experimental vs. numerical (CODEC)



Force – displacement curves, experimental vs. numerical (Equal-joints project)

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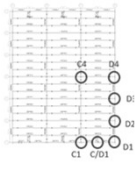
ANALYSIS

- **1st step:** All gravity loads assigned to the floors using a static analysis
- **2nd step:** Duration of column removal is 0.001 seconds

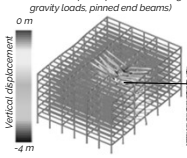
For cases C/D1, D1, D2, D3, D4, the structure has the capacity to resist the progressive collapse (The deformed shape is shown for scenario D2)

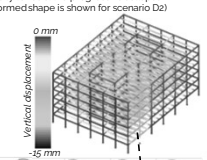
RESULTS

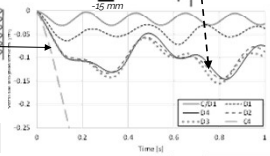


C4 column removal

- ➔ *Progressive collapse of the DAP*
- ➔ *DAP - not part of the SLRS (designed for gravity loads, pinned end beams)*







The results presented here were obtained using the design level of gravity loads: $DL + 0.5 \times LL$ (λ = 1).

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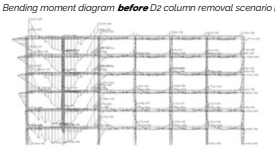
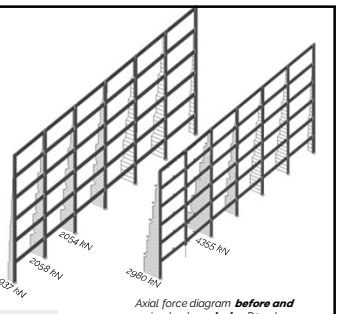
*Bending moment diagram **before** D2 column removal scenario [tf, m]*



*Maximal bending moment diagram **during** D2 column removal scenario [tf, m]*



*Axial force diagram **before and** maximal values **during** D2 column removal scenario [tf]*

The results presented here were obtained using the design level of gravity loads: $DL + 0.5 \times LL$ (λ = 1).

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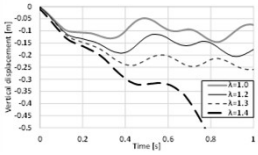
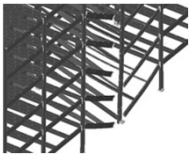
- The results presented before were obtained using the design level of gravity loads:

$$DL + 0.5 \times LL \qquad \text{(gravity load multiplier } \lambda = 1)$$

- To evaluate the strength reserve against progressive collapse, the gravity loads were increased proportionally by means of the factor λ until structure failure
- The dynamic analyses were repeated for each factor λ
- Results are presented for scenario D4



For D4 scenario, the progressive collapse is initiated for λ = 1.4 due to the failure of beam-to-column joints of IPE600 beams



Failure of beam-to-column joint triggers the progressive collapse
(scenario D4, λ = 1.4)

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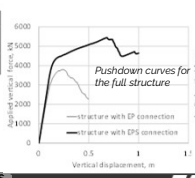
- **REMARKS**
  - In the case of C4 column removal, where all adjacent beams are pinned, the structure is not able to transfer the loads, thus undergoing progressive collapse ➔ **The structure needs to be redesigned**.
  - All other scenarios result in **safe response of the structure** (with plastic deformations, but without progressive collapse);
  - If higher gravity loads are present on the structure, progressive collapse may initiate – see case D4, λ = 1.4.

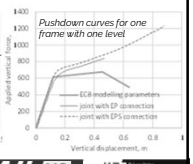
THE REDESIGN can be done using different strategies. Most efficient strategy is based on the activation of the catenary effects.

As the weak point is beam-to-column connection capacity ➔ strengthening strategy: connection reinforcement

end-plate rib stiffeners at both top and bottom sides of beam ends.

To compare the efficiency of the stiffening technique, a push-down analysis is performed on the structure with EP connections and structure with stiffened connections (EPS).





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- **CONCLUSIONS**
  - Lost column is part of a seismic resistant frame:
    - When the column loss affects a seismic resistant frame (i.e., perimeter frame), the damage is limited to the directly affected area.
    - The structure has the capacity to bridge over the missing column and redistribute the loads (alternate load paths available).
  - Lost column is not part of a seismic resistant frame (e.g. the internal structure with pinned beam ends, B4 and C4):
    - When the local damage affects the internal structure with pinned beam ends (B4 and C4), the damage propagates, and the progressive collapse develops on the entire affected area.
    - The pinned connections cannot resist the large axial force demands resulted from the column loss.
    - To limit the damage and prevent the progressive collapse, the alternatives to the strengthening of the pinned connections (which may be difficult to attain) are:
      - Use of moment resisting connections instead of pinned connections (redesign);
      - Use of composite action of the beam with the concrete slab;
      - Design the columns as key elements (to prevent the loss);
      - Reduction or prevention of hazards leading to column loss

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1. Introduction
2. Steel structure in non-seismic area
3. Composite structure in non-seismic area
4. Steel structure in seismic area
5. Composite structure in seismic area

5. COMPOSITE STRUCTURE IN SEISMIC AREA

- **Design approaches:**
  - **Identified actions**
    - **Impact**
      - Equivalent static approach
      - Simplified dynamic approach
      - Full dynamic approach
  - **Unidentified actions**
    - **Alternate load path method (ALPM)**
      - Prescriptive approach (Tying method)
      - Full numerical approach

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CROSS SECTIONS –UTILIZATION RATIOS

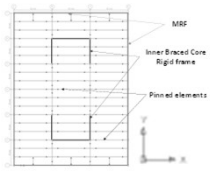
Columns	Section	Steel grade	UF
Corner columns	HE550B	S355	0.48
Perimeter columns	HE500B	S355	0.71
Inner Core columns	HD400X463	S355	0.95

Steel beams fully connected to 12cm solid slab **Nelson studs** Ø=19mm, h=100 mm /a 160 mm

Beams	Direction[1]	Storey	Section	Steel grade	UF Strength	UF Deflection[2]
Perimeter beams	X	1-6	IPE550	S355	0.278	0.178
	Y	1-6	IPE600	S355	0.302	0.157
Interior beams	X	1-6	IPE550	S355	0.627	0.971
	Y	1-6	IPE550	S355	0.874	0.94
Inner core beams	X	1-3	'H800'	S420	0.936	-
		4-5	HEM800	S420	0.953	-
		6	HEM700	S420	0.789	-
	Y	1-3	HEM500	S420	0.859	-
		4-6	HEB500	S420	0.878	-

[1]See Figure for the orientation of the axes
[2]Deflection verification criteria: L/250 for secondary beams, L/350 for main beams
[3]H800' is a built-up section, h=814 mm, b=380 mm, t_f=50 mm, and t_w=30 mm.

Braces	Storey	Section	Steel grade	UF
Y Direction	1-3	HEA320	S355	0.41
	4	HEA260	S355	0.43
	5	HEA220	S355	0.46
	6	HEA200	S355	0.40
X Direction	1-3	HEB340	S355	0.41
	4-5	HEA320	S355	0.39
	6	HEA260	S355	0.26



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IDENTIFIED ACTIONS

IMPACT

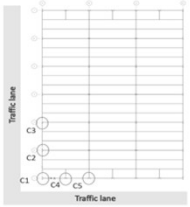
Equivalent static approach

This example gives information about the design against impact due to accidental collision of a vehicle, using the equivalent static approach

- **ACTIONS CONSIDERED FOR THE ACCIDENTAL DESIGN SITUATION**
  - Permanent loads DL
  - Live loads LL
  - Action due to impact A_{Ed}
- **COMBINATION OF ACTIONS FOR 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.



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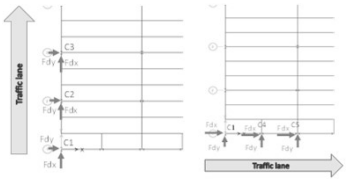
- **THE IMPACT LOADS**
  - Are calculated using data from Table 4.1 of (EN 1991-1-7 2006), considering the case: *Motorways and country national main roads.*

- **IMPACT ASSUMPTIONS**
  - Exposed columns: first floor (C1-C5)
  - Impact point height: 1.5m
  - Impact forces (see following Table)

Case	F_{dx} (kN)	F_{dy} (kN)
C1	1000	500
	500	100
C2	1000	500
C3	1000	500
C4	1000	500
C5	1000	500



- **STRUCTURAL ANALYSIS**
  - A **linear elastic analysis** is made on the full 3D model using SAP2000 software. The section of the elements are those resulted from the initial design (persistent and seismic design situations).

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- **RESULTS**

Case	Section	Impact force [kN]	Axis	Bottom support	N [kNm]	M [kNm]	U.F. [-]	Critical impact force ⁺⁺[kN]
C1	HEB550	1000	Major	Fixed	1048	670	0.478	2700
		500	Minor	Fixed	1053	230	0.656	800
		500	Major	Fixed				
		1000	Minor	Fixed	1074	625	1.313	-
C2	HEB500	1000	Major	Fixed	2218	677	0.899	1250
		500	Minor	Fixed	2216	342	1.044	-
C3	HEB500	1000	Major	Fixed	2229	681	0.9	1250
		500	Minor	Fixed	2238	342	1.048	-
C4	HEB500	1000	Major	Fixed	591	755	0.63	1300
		500	Minor	Fixed	647	339	0.74	700
C5	HEB500	1000	Major	Fixed	1687	787	0.864	1800
		500	Minor	Fixed	1696	340	0.954	550

⁺ The scenario is less demanding as the column was already verified for the same impact load applied against the weak axis of the section
⁺⁺ Impact force that causes the failure of the column (UF=1).

- Six out of nine impact scenarios satisfy the UF criterion, resulting in a safe design.
- Three out of nine impact scenarios result in capacity exceedance. However, instead of strengthening the columns (increasing the cross-section), the following measures may be taken:
  - To mitigate the impact, the hazard may be prevented or eliminated
  - Higher steel grade for columns
  - Column oriented to obtain maximum impact resistance.
  - The use of a more advanced approach

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IDENTIFIED ACTIONS

IMPACT

Simplified dynamic approach

This example gives information about the design against impact due to accidental collision of a vehicle using simplified dynamic approach

- **ACTIONS CONSIDERED FOR THE ACCIDENTAL DESIGN SITUATION**
  - Permanent loads DL
  - Live loads LL
  - Action due to impact A_{Ed}
- **COMBINATION OF ACTIONS FOR ACCIDENTAL DESIGN SITUATION**

  $DL + 0.5 \times LL + A_{Ed}$
- **IMPACT SCENARIOS**

  A single scenario is detailed, i.e., column C1 (UF = 1.313), minor axis impact, which has the highest U.F. according to equivalent static approach design.
- **IMPACT ASSUMPTIONS**
  - Impact direction: along the weak axis: **m=3.5 tons** (vehicle mass); **v_r= 90 km/h** (vehicle speed)
  - The column is made from HEB500, S355 steel, and is 4.0 m high with the following boundary conditions:
    - the column base is fixed
    - top of the column has all degrees of freedom fixed, except for the vertical displacement unrestrained.

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- **STRUCTURAL ANALYSIS**
  - A nonlinear dynamic analysis is made on a single column (isolated from the structure) using SAP2000 software.
  - The analysis is performed in two steps:
    - 1st step: vertical nodal load corresponding to the top of the column obtained from the static analysis in the accidental combination (DL + 0.5×LL) is applied as an axial compressive force using a static analysis.
    - 2nd step: the impact force is applied transversally on the weak axis direction, using a dynamic nonlinear analysis and hard impact approach:

$$F = v_r\sqrt{k \cdot m} \qquad \Delta t = \sqrt{m/k}$$

- v_r - impact velocity
- m - impact mass
- k - stiffness of the impact object

- v_r= 90 km/h = 25 [m/s]
- m = 3500 kg
- K= 300 [kN/m] = 300000 [N/m]

➔ $F = v_r\sqrt{k \cdot m} = 25\sqrt{300000 \cdot 3500} = 810\ kN$

  - In the dynamic analysis, the force is applied using a ramp function with instant rise and a duration of Δt = 0.07 s
  - The total duration of the dynamic analysis is one second (larger than the ramp function duration Δt), to verify if the column remains stable after the ramp function ends.
  - The nonlinear behaviour is modelled using plastic hinges at each column end and at the point of impact, using P-M2-M3 interaction. The plastic hinges are modelled using fibres.

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- The effect of the fast impact loading is considered using a DIF (strain rate effect) applied to the material resistance.
- The DIF formulation for hot-rolled steel with yield strength up to 420 N/mm2 can be expressed according to (CEB 1988) method.

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

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

- The strain rate ($\dot{\varepsilon}$) is obtained in an iterative procedure.
  - 1st iteration, the ratio between the specific deformation and the time up to the point of yielding is computed based on the analysis results.
  - Afterwards, the analysis is performed again with the modified material properties by using a DIF, followed by DIF recalculation.
  - If new DIF = previous step DIF ➔ no further iterations are needed.

$$\mathrm{DIF}\ (\text{for } f_y) = 1.118$$

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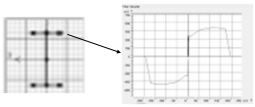
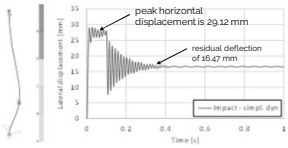
- **RESULTS**
  - The column can sustain the impact force, but with plastic deformations:
    - at the point of impact = 0.054% normal strain
    - at the bottom end = 0.073% normal strain
    - and 0.036% at the top end of the column.



Fiber definition (left) and maximum development of strain on the cross section (right)



Deformed shape of the column (left) and the lateral displacement vs. time (right)

- **CONCLUSIONS**
  - According to the chart for the design for robustness, from box B.6, the outcome for this example is END of design box, as all the requirements are considered fulfilled (i.e., the damage is not critical).

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IDENTIFIED ACTIONS

IMPACT

Full dynamic approach

This example gives information about the design against impact due to accidental collision of a vehicle using full dynamic approach

- **ACTIONS CONSIDERED FOR THE ACCIDENTAL DESIGN SITUATION**
  - Permanent loads DL
  - Live loads LL
  - Action due to impact A_{Ed}
- **COMBINATION OF ACTIONS FOR ACCIDENTAL DESIGN SITUATION**
  $DL + 0.5 \times LL + A_{Ed}$
- **IMPACT SCENARIOS**
  A single scenario is detailed, i.e., column C1 (UF = 1.313), minor axis impact, which has the highest U.F. according to equivalent static approach design.
- **IMPACT ASSUMPTIONS**
  - Impact direction: along the weak axis; **m=3.5 tons** (vehicle mass); **v_x= 90 km/h** (vehicle speed)
- **STRUCTURAL ANALYSIS**
  To analyse a complex structural behaviour (object collision followed by separation of elements and possible collapse) the impact with a vehicle was explicitly modelled. A nonlinear dynamic analysis was conducted on a full 3D model using the **ELS software**.

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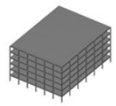
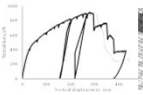
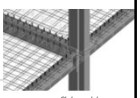
- **STRUCTURAL MODELLING AND ANALYSIS**
  ELS uses a nonlinear solver based on AEM (Applied Element Method) and allows the automatic detection and computation of yielding, hardening, failure of materials, separation of elements, contact at impact, buckling/post-buckling, crack propagation, membrane action, and *P*-Δ effect.
  For inertial effects, dead and live loads were assigned using **lumped masses**


General view of CS/S structural model


Connection detail

The analysis is performed in two steps.
- **1st step:** the permanent and live loads are applied on the structure in a static nonlinear analysis
- **2nd step**: the impact body is colliding with the C2 column in a dynamic nonlinear analysis.


Calibration curves


Slab model

- **MODEL ASSUMPTIONS FOR IMPACT**
  The impacting body (i.e., the vehicle) slides on the horizontal plane only, at a height of 1.5 m, and has 3.5 tonnes mass. The initial velocity of the object is 25 m/s.


Calibrated model


Experimental specimen before and after r.c. slab pouring

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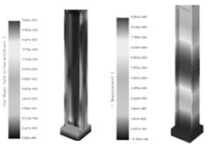
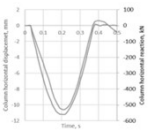
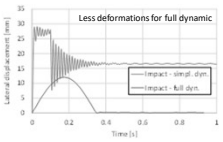
- **MODEL ASSUMPTIONS FOR IMPACT**
  The impacting body (car) is composed of a contact plate, a plate with assigned mass, and axial springs between them. The height of the contact zone between the car and the column is considered 0.6 m. The stiffness of the impacting body (car) is 300 kN/m and is modelled through elastic springs

- **RESULTS**
  Limited plastic deformations in the impacted column, with a maximum lateral deflection of 10.6 mm, was shown


Strains and deformation


Horizontal base reaction force (orange) and horizontal displacement at impact point (blue)



Comparison of results from dynamic approaches

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UNIDENTIFIED ACTIONS

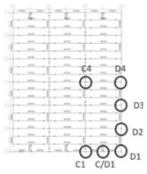
Alternate Load Path Method ALPM

Full numerical approach

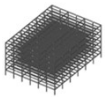
This example gives information about the design against unidentified threats using the ALPM and nonlinear dynamic analysis.

- **ACTIONS FOR THE ACCIDENTAL DESIGN SITUATION**
  - Permanent loads DL
  - Live loads LL
  - No specific accidental action is considered
- **COMBINATION OF ACTIONS FOR THE ACCIDENTAL DESIGN SITUATION**
  $DL + 0.5 \times LL$
- **COLUMN LOSS SCENARIOS**
  - 7 column removal locations are considered, see figure
  - All locations are at the ground floor
- **STRUCTURAL ANALYSIS**
  - 3D model
  - Extreme loading for structure ELS software

Column loss scenarios



View of the 3D model



The dynamic effects caused by the column loss are considered implicitly by means of the removal duration parameter.

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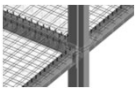
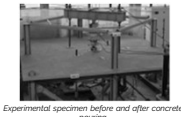
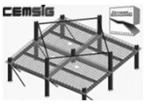
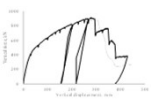
- **CALIBRATION OF THE NUMERICAL MODEL**
  - The model was calibrated using experimental results obtained in CODEC project
  - Experimental testing of a 3D system with steel columns and composite steel-concrete beams under column loss
  - The test is relevant for the local response of connections and global behavior (transition from flexural to catenary response, ultimate capacity) after the column is removed


Slab model


Connection detail


Experimental specimen before and after concrete pouring


3D model tested in CODEC – failure mode


Force-displacement curves, numerical vs experimental

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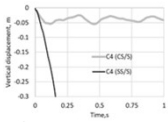
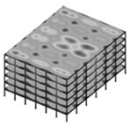
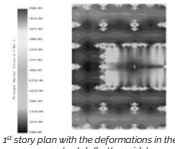
- **ANALYSIS**
  - **1st step:** All gravity loads assigned to the floors using a static analysis
  - **2nd step:** Duration of column removal is 0.001 seconds – nonlinear dynamic analysis
- **RESULTS**
  - The results are compared with those obtained for SS/S structure
  - The results of the nonlinear dynamic analysis show that the CS/S structure has the capacity to resist progressive collapse for all removal scenarios, **including scenario C4** (which is critical for structure SS/S)



Vertical force vs vertical displacement, CS/S vs SS/S


Deformed shape of the structure


1st story plan with the deformations in the concrete slab (bottom side)

- **CONCLUSIONS**
  - Steel-concrete interaction provides additional capacity to resist the column loss
  - The interaction is beneficial especially for frames with pinned beam ends

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