Seismic behaviour of composite steel-concrete frames with dissipative devices

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List of the content

List of the Figure .................................................................................................................. 6
List of the Table ...................................................................................................................... 10
Abstract ................................................................................................................................... 11
Sommario ................................................................................................................................. 12
1. Introduction .......................................................................................................................... 13
   1.1. Introduction ..................................................................................................................... 13
   1.2. Previous Research on Dissipative Devices ................................................................. 15
   1.3. Objective ....................................................................................................................... 25
2. Description of FUSEIS (Bolted Cover Plate Device System) ............................................ 29
   2.1. Introduction ..................................................................................................................... 29
   2.2. Experimental Investigations on FUSEIS2-1................................................................. 31
       2.2.1. Experimental Investigations on Individual Device .............................................. 31
       2.2.2. Experimental Set-Up .............................................................................................. 31
   2.3. Analysis of the Experimental Results .......................................................................... 33
       2.3.1. Cyclic behaviour ..................................................................................................... 33
       2.3.2. Monotonic and cyclic behaviour .............................................................................. 34
       2.3.3. Failure mode .......................................................................................................... 35
       2.3.4. Composite behaviour ............................................................................................. 36
   2.4. Experimental Investigations on Overall Frames ............................................................ 37
       2.4.1. Experimental Set-Up .............................................................................................. 37
       2.4.2. Test Results ........................................................................................................... 38
   2.5. Numerical Investigations ............................................................................................... 40
       2.5.1. Numerical Investigations on Individual Device .................................................... 40
       2.5.2. Numerical Investigations on Overall Frame ........................................................ 42
3. Design Rules for FUSEIS Systems ..................................................................................... 44
   3.1. Design Rules for Linear Dynamic Analysis .................................................................. 44
       3.1.1. Composite beam plastic resistance ....................................................................... 44
       3.1.2. Fuses resistance design .......................................................................................... 46
       3.1.3. Bending Resistance of the Fuse .............................................................................. 47
       3.1.4. Free buckling length ............................................................................................... 47
       3.1.5. Design of Flange plate ............................................................................................ 51
3.1.6. Longitudinal Reinforcement ................................................................. 52
3.2. Anchorage length according to EN 1992-1-1 8.4 .................................. 53
  3.2.1. Overlaps .......................................................................................... 55
3.3. Concrete confinement gap ..................................................................... 56
3.4. Design of the Fuse Devices for Shear .................................................... 56
3.5. Design of the Non-Dissipative Connecting Elements ............................. 57
3.6. Design of the Bolted Connection .......................................................... 58
  3.6.1. Influence of distance ........................................................................ 58
  3.6.2. Plate resistance according to EN 1993-1-8 ...................................... 59
  3.6.3. Bolts resistance under shear according to EN 1993-1-8 ................. 59
  3.6.4. Bearing resistance according to EN 1993-1-8 ................................. 60
3.7. Additional Detailing Remarks .................................................................. 60
4. Validation of non-linear plate behaviour ................................................ 62
  4.1. Fuses characterization .......................................................................... 62
  4.2. Validation of individual fuse non-linear plastic behaviour ................. 66
  4.3. Comparison of Numerical and Experimental – Individual frame ....... 68
  4.4. Comparison of Numerical and Experimental - Overall frame ............. 71
5. Case Studies .............................................................................................. 79
  5.1. Introduction .......................................................................................... 79
  5.2. Description of the case studies ............................................................. 79
  5.3. Loads .................................................................................................. 80
  5.4. Materials ............................................................................................. 81
  5.5. Analysis and Design of the Building without fuses .............................. 81
  5.6. Numerical Modelling of the case studies with FUSEIS devices ........ 83
    5.6.1. 2D Numerical model ...................................................................... 83
  5.7. Pushover ............................................................................................. 90
  5.8. 8 Story building .................................................................................. 91
  5.9. 4 Story building .................................................................................. 98
  5.10. 2 Story building ................................................................................ 105
6. Verification ................................................................................................ 109
  6.1. Classes of steel section ....................................................................... 109
  6.2. Integrity of the concrete slab ............................................................... 110
6.3. Effective column length

6.3.1. 8 Story Building - base internal columns effective length

6.3.2. 4 Story Building - base internal columns effective length

6.3.3. 2 Story Building - base internal columns effective length

6.4. Buckling at internal base columns

6.4.1. 8 Story Building buckling check

6.4.2. 4 Story Building buckling check

6.4.3. 2 Story Building buckling check

6.5. Modal analysis

6.5.1. 8 Story building – modal information

6.5.2. 4 Story building – modal information

6.5.3. 2 Story Building – modal information

6.6. Damage limitation – Verification from NTC 394 Circular C7341

6.6.1. 8 Story Building – Inter-story drift

6.6.2. 4 Story Building – Inter-storey drifts

6.6.3. 2 Story Building – inter-story drifts

6.7. II° effect analysis

6.7.1. 8 Story Building – drift sensitivity coefficient

6.7.2. 4 Story Building – drift sensitivity coefficient

6.7.3. 2 Story Building – drift sensitivity coefficient

6.8. Weak Beam-Strong Column check

6.8.1. 8 Story building - WBSC

6.8.2. 4 Story building – WBSC

6.8.3. 2 Story building - WBSC

7. Behaviour factor

7.1. Method 1

7.2. Method 2

7.3. Method 3

7.4. Method 4 and 5

7.5. Method 6 and 7

7.6. Method 8 and 9

7.7. Method 10
7.8. Method 11 ................................................................. 130
7.9. Method 12 ................................................................. 131
7.10. Method 13 and 14 ..................................................... 131
7.11. Method 15 and 16 ..................................................... 132
7.12. Method 17 and 18 ..................................................... 132
7.13. 8 Story behaviour factor value comparison factor ........... 133
8.  Conclusion .................................................................... 134
Annex A: Load combination ............................................. 136
Annex B: Behaviour factor of different building .................. 137
References ....................................................................... 141
List of the Figure

Figure 1.1: ADAS and TADAS innovative dissipative devices ........................................ 16
Figure 1.2: Buckling Restrained Braces Components ......................................................... 16
Figure 1.3: Details of Pi Damper as a beam-to-column connections .................................. 17
Figure 1.4: Slit damper ....................................................................................................... 18
Figure 1.5: Steel Self Centering Device ............................................................................. 18
Figure 1.6: Reduced Beam Section ..................................................................................... 19
Figure 1.7: Removable link device ..................................................................................... 20
Figure 1.8: FUSEIS 1: (a) FUSEIS1-1,(b) FUSEIS 1-2 and (c) ........................................ 21
Figure 1.9: FUSEIS 2: (a) FUSEIS2-1,(b) FUSEIS 2-2 .................................................. 21
Figure 1.10: INERD device example .................................................................................. 22
Figure 1.11: INERD™ Devices (a) Pin and (b) U-Shape .................................................... 23
Figure 1.12: Structure for test done in Milan .................................................................... 23
Figure 1.13: Example of U INERD device location ........................................................... 24
Figure 1.14: Graph of Inerd 14 - Global frame Response .................................................... 24
Figure 1.15: INERD 14 Loading History ......................................................................... 25
Figure 1.16: Dual system frame - concept ....................................................................... 26
Figure 1.17: Global mechanism and local story mechanism ............................................. 27
Figure 1.18: CBF typology ............................................................................................... 27
Figure 1.19: Dual frame system ......................................................................................... 28
Figure 2.1: FUSEISE Main Section Elements ................................................................. 29
Figure 2.2: Fuse Device in a Moment Resisting frame ..................................................... 30
Figure 2.3: Experimental Test Set-Up a) Experimental Test Overview b) Free Buckling Length c) Positioning of the Bolted Fuse Device ......................................................... 32
Figure 2.4: Moment -rotation (M - θ) Diagram of Fuse C ................................................... 33
Figure 2.5: Comparison in terms of Moment -rotation (M-θ) of Fuse C and Fuse D ....... 33
Figure 2.6: Comparison between Monotonic and Cyclic Tests Conducted on Fuse C-140 and B-140 in both, Sagging and Hoggibg respectively ............................................. 35
Figure 2.7: Failure Modes Example a)Gross-Mid and b) Net Section .................................. 36
Figure 2.8: Force-Displacement Diagram for Beam-Slab Connection ......................... 36
Figure 2.9 Progressive Slab Damage a) First, b) Second, c) Third and d) Forth Tests... 37
Figure 2.10: Schematic Representation of the Bolted Fuse Device ................................. 38
Figure 2.11: Frame Displacement under Loading in the a) –X and b) +X Direction ........ 39
Figure 2.12: An Example of Moment Rotation (M-θ) Diagram (Plate D) .................... 40
Figure 2.13: a) Sagging and b) Hoggibg (Approximate Device Rotation=40mrad) ....... 41
Figure 2.14: M-θ Comparison between Experimental and Numerical Results ............. 41
Figure 2.15: Comparison between Experimental and Numerical Results .................... 42
Figure 2.16: Von Misses Stress of the Deformed Shape .................................................... 43
Figure 2.17: An Example of Moment-Rotation Diagram (Plate D) ............................... 43
Figure 3.1: Fiber Layout .................................................................................................... 47
Figure 3.2: Buckling mechanism of the FUSEIS .................................................. 50
Figure 3.3: Overlaps position limits for the reinforced rebar’s .................................. 55
Figure 3.4: Yield and Ultimo strength of the bolts ..................................................... 58
Figure 3.5: Parameter individuation ........................................................................... 60
Figure 4.1: Material uniaxial stress-strain curve B450C ............................................. 62
Figure 4.2: Material uniaxial stress-strain curve B450C ............................................. 62
Figure 4.3: Characterization Plate D .......................................................................... 63
Figure 4.4: Characterization Plate A .......................................................................... 64
Figure 4.5: Characterization Plate B .......................................................................... 64
Figure 4.6: Characterization Plate D .......................................................................... 64
Figure 4.7: Hogging curve parameter ......................................................................... 65
Figure 4.8: Moment-rotation Diagram of the Plate D .................................................. 66
Figure 4.9: Moment-rotation Diagram of the Plate A .................................................. 66
Figure 4.10: Moment-rotation Diagram of the Plate B ............................................... 67
Figure 4.11: Moment-rotation Diagram of the Plate C ............................................... 67
Figure 4.12: Numerical Modelling of the Individual Fuse .......................................... 68
Figure 4.13: Material uniaxial .................................................................................... 68
Figure 4.14: Material uniaxial .................................................................................... 68
Figure 4.15: Moment-rotation Diagram of the Plate D ................................................ 69
Figure 4.16: Moment-rotation Diagram of the Plate A ............................................... 69
Figure 4.17: Moment-rotation Diagram of the Plate C ............................................... 70
Figure 4.18: Moment-rotation Diagram of the Plate B ............................................... 70
Figure 4.19: Numerical Modelling of the Overall Frame with Fuses ................................ 71
Figure 4.20: Moment-rotation Diagram of the Plate D – in bay plate ......................... 72
Figure 4.21: Moment-rotation Diagram of the Plate D – out of bay plate ................... 72
Figure 4.22: Force-displacement Diagram of the Plate D – Global frame response .... 73
Figure 4.23: Moment-rotation Diagram of the Plate A – in bay plate ......................... 73
Figure 4.24: Moment-rotation Diagram of the Plate A – out of bay plate ................... 74
Figure 4.25: Force-displacement Diagram of the Plate A – Global Frame ................. 74
Figure 4.26: Moment-rotation Diagram of the Plate B ............................................... 75
Figure 4.27: Moment-rotation Diagram of the Plate B ............................................... 75
Figure 4.28: Force-displacement Diagram of the Plate B – Global frame response .... 76
Figure 4.29: Moment-rotation Diagram of the Plate C – in bay plate ......................... 76
Figure 4.30: Moment-rotation Diagram of the Plate C – out of bay plate ................... 77
Figure 4.31: Force-displacement Diagram of the Plate C – Global frame response .... 77
Figure 5.1: Plan of the 2/4/8-Story Archetype Structures ........................................... 79
Figure 5.2: Elevation View of the 2/4/8-Story Archetype Structures ........................... 80
Figure 5.3 Composite Slab Section ........................................................................... 82
Figure 5.4: Total beam influence area ....................................................................... 83
Figure 5.5: Total columns influence area ................................................................. 84
Figure 5.6: Dead point load ....................................................................................... 85
Figure 5.7: Live point load ......................................................................................... 85
Figure 5.8: Superimposed point load ................................................................. 85
Figure 5.9: Frame hinge definer window .......................................................... 86
Figure 5.10: Effective slab width according to EC4 – 5.4.1.2. ............................. 87
Figure 5.11: Effective slab width according to EC8 – 7.6.3. .............................. 87
Figure 5.12: Frame hinge properties of the beams (moment in kNm) .................. 88
Figure 5.13: Disposition of different elements type ........................................... 88
Figure 5.14: Link moment-rotation curve definer windows ................................. 89
Figure 5.15: Summary of lumped plasticity modelling-approach ....................... 89
Figure 5.16: 8 Story Building Fuse Layout....................................................... 93
Figure 5.17: 8 Story - Constitutive laws of Fuse................................................ 94
Figure 5.18: 8 Story building ............................................................................ 95
Figure 5.19: 8 Story building ............................................................................ 95
Figure 5.20: 8 Story building without fuse (Uniform Acc)- δ=57cm ............... 96
Figure 5.21: 8 Story building without fuse (1st Mode Acc)- δ=57cm .............. 96
Figure 5.22: 8 Story building whit fuse - activation link step (Uniform Acc) ..... 96
Figure 5.23: 8 Story building whit fuse activation link step (1st Mode Acc) ...... 96
Figure 5.24: 8 Story building without fuse (Uniform Acc), step 34 ................. 97
Figure 5.25: 8 Story building (Uniform Acc), step 74 ..................................... 97
Figure 5.26: Base plate link moment-rotation evolution in hogging side .......... 97
Figure 5.27: Base plate link moment-rotation evolution in sagging side .......... 98
Figure 5.28: Top plate link moment-rotation evolution ..................................... 98
Figure 5.29: 4 Story Building Fuse Layout ....................................................... 101
Figure 5.30: 4 Story - Constitutive laws of Fuse............................................... 102
Figure 5.31: 4 Story building (Uniform Acc)- δ=1m ...................................... 103
Figure 5.32: 4 Story building (1st Mode Acc)- δ=1m ..................................... 103
Figure 5.33: 4 Story building whiteout fuse – (Uniform Acc)- δ=1m .......... 103
Figure 5.34: 4 Story building without fuse- (1st Mode Acc)- δ=0,3m .......... 103
Figure 5.35: 4 Story building whit fuse - activation fuse step (Uniform Acc) .... 104
Figure 5.36: 4 Story building whit fuse – activation fuse step (1st Mode Acc) ... 104
Figure 5.37: 4 Story building (Uniform Acc),δ=24cm .................................... 104
Figure 5.38: 4 Story building (Uniform Acc), δ=26cm .................................... 104
Figure 5.39: 4 Story building (1st Mode Acc), δ=20cm .................................... 104
Figure 5.40: 4 Story building (Uniform Acc), δ=22cm .................................... 104
Figure 5.41: 2 Story Building Fuse Layout ....................................................... 106
Figure 5.42: 2 Story - Constitutive laws of Fuse............................................. 106
Figure 5.43: 2 Story building – ................................................................. 107
Figure 5.44: 2 Story building – ................................................................. 107
Figure 5.45: 4 Story building without fuse – (Uniform Acc)- Step 13 .......... 107
Figure 5.46: 4 Story building without fuse– (1st Mode Acc)-Step 15 ............ 107
Figure 5.47: 2 Story building whit fuse - activation fuse step (Uniform Acc) .... 108
Figure 5.48: 2 Story building whit fuse – activation fuse step (1st Mode Acc) .... 108
Figure 5.49: 2 Story building – (Uniform Acc), step 12 ................................ 108
Figure 5.50: 2 Story building – (1st Mode Acc) step 15 ....................................................... 108
Figure 6.1: Alignment chart for uninhibited frame ............................................................. 110
Figure 6.2 Buckling curve .................................................................................................. 112
Figure 6.3: Design spectrum considered in the modal analysis ........................................... 113
Figure 6.4: 8 Story - Deformed shape in vibration mode 1 2 and 3 respectively .............. 114
Figure 6.5: 8 Story - Deformed shape in vibration mode 1 2 and 3 respectively .......... 115
Figure 6.6: 2 Story - Deformed shape in vibration mode 1 2 and 3 respectively ............ 116
Figure 7.1 Determination of the idealized elastic - perfectly plastic force – displacement relationship ............................................................................................................. 126
# List of the Table

Table 1: Conventional and Dissipative Structural System and Main Characteristics .... 15
Table 2: Dimensions of the Flange Plates of the Fuse Specimens (in mm) .......... 31
Table 3: Dimension and result of plate tested ........................................... 51
Table 4: Design friction tension for different concrete condition .......... 54
Table 5: L_(b,rqd) value for B450C rebar in function of diameter and concrete class .. 54
Table 6: Summary of the check that as to be provided .................................. 61
Table 7: Sensible plate parameter for each plate ......................................... 62
Table 8: Columns section for the 2 storey building ....................................... 82
Table 9: Columns section for the 4 storey building ....................................... 82
Table 10: Columns section for the 8 storey building ....................................... 82
Table 11: Effective slab width ........................................................................ 87
Table 12: Composite beam section property ............................................... 87
Table 13: 8 Story Building free buckling length ........................................... 93
Table 14: Dimension of fuse members .......................................................... 94
Table 15: 8 Story - property of the Fuses ..................................................... 94
Table 16: 4 Story Building Effective Length of Fuse ....................................... 101
Table 17: 4 Story - Dimension of the Fuse members ....................................... 101
Table 18: 4 Story - property of the Fuses ..................................................... 101
Table 19: 2 Story Building Effective Length of Fuse ....................................... 106
Table 20: 2 Story - Dimension of the Fuse members ....................................... 106
Table 21: 2 Story - property of the Fuses ..................................................... 106
Table 22: 8 story whit fuse - Fundamental vibration mode and associated mass ...... 114
Table 23: 4 story without fuse Fundamental vibration mode and associated mass ..... 114
Table 24: 4 story whit fuse - Fundamental vibration mode and associated mass ...... 115
Table 25: 4 story without fuse - Fundamental vibration mode and associated mass .... 115
Table 26: 2 story with fuse - Fundamental vibration mode and associated mass .... 116
Table 27: 2 story without fuse - Fundamental vibration mode and associated mass ... 116
Table 28: 2 story without fuse - Fundamental vibration mode and associated mass ... 116
Table 29: 8 Story building – Inter-storey drifts ............................................. 117
Table 30: 8 Story building – Inter-storey drifts ............................................. 118
Table 31: 8 Story building - drift sensitivity coefficient .................................... 119
Table 32: 4 Story building - drift sensitivity coefficient .................................... 120
Table 33: 2 Story building - drift sensitivity coefficient .................................... 120
Table 34: 8 Story building – WBSC check ................................................... 121
Table 35: 4 Story building – WBSC check ................................................... 123
Table 36: 2 Story building – WBSC check ................................................... 124

10
Abstract

A device devoted to improve the seismic resistance of steel and composite steel-concrete structures is analyzed. After a first part that focuses on experimental tests already carried out, the device is applied to different structures. The devices are properly design for each type of structure to obtain best performance in term of global collapse mechanism.

Three composite steel-concrete frames are considered: all are made of S355 structural steel; first (Frames 1) is eight story building, the second (Frames 2) is four story building and the third (Frame 3) two story building. Non-linear static analyses (Pushover) was performed for building design to MDC whit PGA of 0.2g. The software SAP2000 is used for all the analyses executed, both for the phase concerning the experimental tests and the one concerning the analyses performed on the frames considered.

The key concept of the device is the concentration of the damage inside it, forcing the plastic hinge to remain within the fuse and avoiding the spreading of inelasticity near the beam to column connection. Moreover the results show the prevention of the formation of the soft story plastic mechanism.

Global frame performance was investigate through behaviour factor assessment based on static pushover capacity curve. Most approaches for behaviour factor assessment are presented. Each proposal comes with its own definition of safety target and seismic performance assessment method. Comparison between different method result and supposed initial behaviour factor conclude this thesis work.

Key words: Dissipative devices, Steel-concrete composite moment resisting frame, Finite Element Method, Nonlinear static analysis, overstrenght factor, ductility factor, behaviour factor
Sommarrio

Viene analizzato un dispositivo sviluppato per migliorare la resistenza sismica di acciaio e strutture in acciaio-calcestruzzo. Dopo una prima parte che si concentra su prove sperimentali già effettuate, il dispositivo viene applicato a differenti strutture. I dispositivi sono adeguatamente progettati per ogni tipo di struttura per ottenere le migliori prestazioni in termini di meccanismo di collasso globale.

Tre telai in acciaio-calcestruzzo vengono presi in considerazione: tutti sono fatti di acciaio strutturale S355; il primo (Frame 1) è un edificio di otto piani, il secondo (Frame 2) è di quattro e il terzo (Telai 3) è a due piani. Analisi statiche non lineari (Pushover) sono state fatte per verificare la progettazione degli edifici di media duttilità progettati con accelerazione 0,2g. Il software utilizzato per tutte le analisi SAP2000 è, sia per la fase relativa alle prove sperimentali che per quella relativa alle analisi effettuate sui telai considerati.

Il concetto fondamentale è la concentrazione del danneggiamento dovuto al sisma, forzando la cerniera plastica a rimanere all'interno del fusibile ed evitando la diffusione del danno al vicino collegamento trave-colonna. Inoltre i risultati mostrano la prevenzione della formazione del meccanismo di soft-story.

Prestazioni globali del telaio sono state indagate attraverso valutazione dei fattori di comportamento sulla base di curva di capacità. La maggior parte degli approcci per la valutazione del fattore di comportamento sono presentati. Ogni metodo è dotato di una propria definizione del target di sicurezza e di valutazione delle prestazioni sismiche. Il confronto tra i diversi risultati e il presunto fattore di comportamento iniziale concludere questo lavoro di tesi.

Parole chiave: Dispositivi dissipativi, telaio momento resistente in acciaio-calcestruzzo, metodo degli elementi finiti, analisi statica non lineare, fattore di comportamento
1. Introduction

1.1. Introduction

"Seismic activity" is defined as the vibration of the ground due to the release of elastic energy from the breakage of rock within the earth or an explosion. These events are unavoidable and unpredictable, which causes economic as well as human losses. Although modern design approaches continue to propose innovative solutions to withstand strong earthquakes, the post-disaster restoration is still too expensive and impactful, from economic, social and environmental points of view. Today one of the international interests of companies, researchers as well as designers is looking forward to design innovative structures cheaper to be repaired after a strong earthquake. A key opportunity to reduce resource consumption and environmental loads lies in the post-disaster resiliency. Resilience can be defined as “the ability to prepare and plan for adsorb, recover from, and more successfully adapt to adverse events” [1]. This resilience can be defined with reference to four main properties as (1) robustness which refers to the strength of a system, or ability to withstand and absorb a given level of stress, (2) rapidity which measures the rate of a system recovery and capacity to restore to a given performance level, (3) redundancy captures the extent of components or systems that are capable of satisfying functional requirements in the event of hazard-induced disruption and damage and (4) resourcefulness refers to the capacity to mobilize resources to do so [2].

Civil structures subjected to strong earthquakes need to dissipate large amounts of energy. This energy dissipation is achieved through development of inelastic deformation in specific zones called “dissipative zones” of structural members. This means that conventional systems suffer significant inelastic deformations (costly damages) in main structural elements (steel beams, columns and concrete slabs) and residual interstory drifts after a strong seismic event. Repair work in these cases is most of the time not feasible, or too expensive, also because of the long interruption of the functionality of the building, leading to additional costs and discomforts for building owners and occupants. Reduction of damage to structural and non-structural elements after a disaster becomes a fundamental aspect for improving the long-term sustainability and resource conservation. The resources that are spent in the process of reconstruction after a disaster can be significantly reduced through innovative dissipative device to be introduced in new buildings and retrofit measures for existing buildings, lowering environmental and economic costs in a life cycle perspective.

The introduction of these devices aims to dissipate the seismic energy through their plastic deformation, leaving main structural elements undamaged. They must provide adequate characteristics of ductility and energy dissipation in order to absorb large strains due to concentration of plasticity. Although the “dissipative” character of the innovative devices has been studied intensively, reparation performance of the developed devices remains uncertain. Since it is crucial to restore the buildings and its
functions as quickly as possible after an earthquake, it is strongly advisable to develop structural systems that are simple to repair.

The most common seismic resistant systems in composite steel-concrete structures according to Eurocode 8 [3] are currently: moment resisting frames (MRF), concentric braced frames (CBF), eccentric braced frames (EBF). MRF systems are designed to dissipate energy through flexural yielding in the beams, near the beam-to-column connections, while the energy dissipation of CBF is achieved through yielding and buckling of the braces when under tension or compression. Similarly an EBF consists of a diagonal brace system, where the braces connect eccentrically to the frame beams through a link element which is dissipating energy due to shear and flexural yielding. These systems accept the development of damage on main structural elements under earthquake loads, thus resulting in significant economic losses due to reparability costs (when reparability is feasible), and interruption of services.

Accordingly, anti-seismic devices can be installed in buildings in order to protect the structural elements during the seismic events. They modify the seismic response of the structure by dissipating energy or creating restraints through a rigid connection. Among all innovative systems dissipative concentrating their plastic deformations into pre-define locations and leaving other structural elements in elastic deformation, two cases namely INERD and FUSEIS-2 will be deeply discus in this paper. The evaluation of the existing conventional structural systems as well as the two introduced new systems, in respect to stiffness and ductility by means of their advantages and disadvantages is shown in [4].
### Table 1: Conventional and Dissipative Structural System and Main Characteristics

<table>
<thead>
<tr>
<th>Structural Type</th>
<th>Stiffness</th>
<th>Ductility</th>
<th>Dissipative Zones</th>
</tr>
</thead>
<tbody>
<tr>
<td>Moment Resisting Frames (MRF)</td>
<td>0</td>
<td>++</td>
<td>Beam Ends</td>
</tr>
<tr>
<td>Concentric Braced Frames (CBF)</td>
<td>++</td>
<td>0</td>
<td>Tension Braces</td>
</tr>
<tr>
<td>Eccentric Braced Frames (EBF)</td>
<td>+</td>
<td>++</td>
<td>Beam Links</td>
</tr>
<tr>
<td>Steel Shear Walls</td>
<td>+</td>
<td>+</td>
<td>Steel Plate</td>
</tr>
<tr>
<td>Composite Shear Walls</td>
<td>++</td>
<td>0</td>
<td>Shear Wall</td>
</tr>
<tr>
<td>INERD Concentric Braced Frames</td>
<td>+</td>
<td>++</td>
<td>Pins *</td>
</tr>
<tr>
<td>Fuseis 2</td>
<td>0</td>
<td>++</td>
<td>Beam’s Fuses *</td>
</tr>
</tbody>
</table>

* Pins and fuses are exclusively made of steel that undergo inelastic deformations and possible damage when subjected to cyclic loading. These devices are designed to act as fuses that may be easily dismantled and replaced if damaged after a ground motion of intensity equal to or higher than the design earthquake [5], [6].

#### 1.2. Previous Research on Dissipative Devices

Researchers have focused on economical, feasibility, reparability and environmental performances of dissipative devices (seismic devices) after the Northridge (USA 1994) and Kobe (Japan 1995) earthquakes which led to the widespread failure of a large number of structural elements [7], but still the use of these such devices is questionable.

Dissipative devices can be characterized as (a) displacement-dependent for hysteretic yielding dampers, (b) velocity-dependent for viscous fluid dampers, (c) acceleration-dependent for mass dampers and (d) modified input devices for base isolation systems [8]. The following research will only focus on the steel-hysteretic yielding dampers, which combine some of the most effective and economical mechanisms for the dissipation of seismic energy input, obtained through the inelastic deformation of steel. Hysteretic steel yielding dampers dissipate seismic energy through plastic behaviour of their steel parts and they are introduced in steel frames in order to act as seismic fuses.
• **ADAS and TADAS**: The frame should be designed to desirably concentrate yielding in the pre-defined place, so that the remaining structural elements remain elastic and, therefore, undamaged. These devices are mostly applied in bracing systems making use of flexural deformation of metals including the patented ADAS and TADAS [9] and Cu-ADAS [10] and the Steel Slit Damper (SSD) [11]. The seismic performance of these devices has been proved experimentally [12], [13] and numerically[13]–[15], and their fabrication, implementation and replacement characteristics were proved to be easy [16].

![ADAS and TADAS innovative dissipative devices](image)

**Figure 1.1: ADAS and TADAS innovative dissipative devices**

**Problems Found:** Cost-benefit framework (economic and environmental balance in the life cycle) and associated models that can reasonably quantify the impact of sustainability aspects (LCA, LCC) are not addressed.

• **Buckling Restrained Braces (BRBs)** have been developed and tested for over 30 years worldwide and the most used type of brace appears to be that of a yielding steel core encased in a mortar filled hollow steel shape (see Figure 1.2). Kiggins and Uang [17] showed that although BRB frames can exhibit a favourable energy-dissipating mechanism, the low post-yield stiffness of the braces leave the system vulnerable to large permanent drifts. Residual storey drifts can be reduced, if used in dual systems only, which can be more costly than expected.

![Buckling Restrained Braces Components](image)

**Figure 1.2: Buckling Restrained Braces Components**
**Problems Found:** This system requires the post-disaster replacement of all damaged structural bracing elements. It may result in more significant costs (economic and environmental) and efforts compared to other dissipative system confined in small members.

- **Pi damper:** first developed by Koetaka et al. (2004) [18] they proposed alternative connections applicable to the column’s weak axis in both weld-free and MRF structures. In this system, a wide-flange beam is joined to a wide-flange column by bolting splices at the top flange and cast steel U-shape hysteretic dampers at the bottom flange (PI-dampers) (see Figure 1.3). This connection arrangement was designed so that the center of the beam rotation is at the end of the beam top flange. As a result, plastic deformations are concentrated on PI-dampers at the bottom of the beam without causing significant damage to the concrete floor slab under large storey drifts, ensuring lower repair costs. Seismic performance of this system was verified through quasi-static cyclic tests on full-scale beam-column subassemblies, capable of achieving stable hysteresis behavior in the large deformation range and yielding limited only in the dampers.

![Figure 1.3: Details of Pi Damper as a beam-to-column connections](image)

**Problems Found:** The global behaviour of the entire real steel-concrete construction subjected to realistic seismic actions is not adequately investigated. The repair process and feasibility of members’ reassembly are not adequately addressed. Direct economic savings from the installation of devices, as well as potential environmental benefits, are not quantified.

- **Slit damper:** Oh et al. (2009) [19] proposed a beam-to-column connection system for composite steel-concrete MRFs where the slit damper is connected to the bottom flange of the steel beam using high-strength bolts (see Figure 1.4). Cyclic tests of three beam-to-column connection were performed to verify the seismic performance of the proposed connection. Specimens exhibited stable hysteretic behaviour under large storey drift and plastic deformations were concentrated at the slit dampers while the beams and columns remained almost
elastic. Since the damper is located only on the lower beam flange, which is generally accessible for repair work after an earthquake, there is no need of removing the slab for replacement of the device. Experimental tests were combined with numerical analysis in order to obtain an effective theoretical model.

![Figure 1.4: Slit damper](image)

**Problems Found:** Although the fuses reached 40 mrad rotations under cyclic tests, for rotations higher than 20 mrad pinching occurred for some specimens due to slippage of the device, inducing an important loss of efficiency in terms of energy dissipation and making instable the hysteretic behaviour. The global behaviour of the entire steel-concrete construction subjected to realistic seismic actions is not investigated. The repair process and feasibility of members’ reassembly are not adequately addressed. Direct savings from the installation of devices are not quantified.

- **Steel Self Centering Device (SSCD)** was developed, designed and experimentally validated [20], [21]. The device is characterized by a hysteretic dissipative system and a steel pretension system for re-centering. This system is completely steel-based which can be easily assembled by steelworkers (see Figure 1.5). The dissipative system consists of steel fuses that are easily changeable after a seismic event. Experimental tests executed on the prototype of the SSCD confirmed the results obtained by the numerical modeling, highlighting the good dissipative and re-centering capacity of the SSCD [21].

![Figure 1.5: Steel Self Centering Device](image)
Problems Found: This system requires the post-disaster replacement of all damaged structural bracing elements, which may result in significant costs and efforts. Cost-benefit framework and associated models that can reasonably quantify the impact of sustainability efforts are not addressed.

- **Reduced Beam Section** (RBS) also called “dog-bone” which is a weakening approach based on an intentionally beam strength reduction in order to allow the development of plastic deformation in a particular beam section, this first introduced and tested by (Plumier, 1990) [22] and then recurred in several variants of flange shapes (see Figure 1.6). In this method, inelastic moment capacity will be achieved at that particular part of the section first. Positioning of the plastic hinge can be controlled by a simple design procedure [23].

![Figure 1.6: Reduced Beam Section](image)

Problems Found: this system requires to post-disaster replacement of the entire beam section after a severe earthquake which may result in significant costs and efforts. Cost-benefit framework and associated models that can reasonably quantify the impact of sustainability efforts are not addressed.

- **Removable Link Device**: DUAREM project (2013) [24] tested on the full-scale dual eccentrically braced structures with removable dissipative devices, at the European Laboratory for Structural Assessment of the Joint Research Centre in Ispra (Italy). The dissipative devices consisted in short links that are bolted to the floor beams in eccentric braced frames (see Figure 1.7). 2D Numerical analyses have been performed to evaluate their dynamic performance and to simulate the removal process of the links [24]. Numerical and experimental results showed that the structure is capable of withstanding the level of earthquake it was designed for, localizing all damage in the seismic links that were being easily replaceable.
Problems Found: Investigations were limited to a unique type of structural system, i.e. eccentrically braced dual steel frame system. Tests have been performed in one-direction only. Reparability and sustainability problems found were not adequately addressed. In case of composite steel-concrete beam the crushing of the slab can affect the serviceability and comfort of the users that means increase in the reparability cost.

- The FUSEIS research project introduced other two innovative dissipative devices for Moment Resisting Frame (Vayas et al., 2013): [5]
  
a) FUSEIS 1, shown in Fig1.2.9, consists of two closely spaced strong columns, rigidly connected to multiple beams that run from column to column placed between floor levels (FUSEIS 1-1) or alternatively interrupted and connected by short pins (FUSEIS 1-2); this system combines a shear resistant wall with the advantage of energy dissipation through plastic deformations of beams and is easy to be repaired or replace if required. These devices are not usually subjected to vertical loads, whereas the system acts as a vertical Vierendeel beam to overcome for the horizontal loads.

b) FUSEIS 2 that consists in a cross-sectional weakening located at the beam ends at a certain distance from the beam-to-column connections, obtained introducing a discontinuity on the composite beams and assembling the two parts through steel plates a) bolted or b) welded to the web and flange of the beam, as shown in Figure 1.9.

The behaviour of these fuse devices was studied numerically and experimentally during the research project, and the results have been published in many articles [25]–[29].
INERD pin devices were developed during the RFCS-supported INERD project (2004) [1]. The devices are composed of a steel pin connecting the ends of braces to the columns in concentric braced frames (Figure 2). INERDTM pins transfer brace axial forces through three-point bending. The devices act as semi rigid ductile dissipative brace connections. The devices are of partial strength in order to protect the braces from yielding ad buckling, so that energy dissipation occurs exclusively in them and not in the braces. They can be pinned shape or U shape. Provide fast and easily reparation procedure after strong seismic events.
INERD\textsuperscript{TM}, This type of dissipative devices is designed for concentrically braced steel frame with semi rigid ductile brace to column connections were first developed by Plumier et al. (2006) \cite{6} within the INERD research project: the pinned and the U-shape connections (see Figure 1.11) where introduced as the two types of innovative connections.

a) Pin device: The system consists of two external eye-bars welded or bolted to the column flanges, of another one or two internal eye-bars welded to the end of the diagonal member and a pin running through the eye-bars shown in Figure 1.11 (a).

b) U-shape device: this system consists of one or two bent, U-shaped thick plates that connect the brace to the adjacent member where energy dissipation takes place in the bent plate(s) shown in Figure 1.11 (b).

There after only Pin devi c e will be considered. The axial force of the brace is transferred through the internal plates to the pin which is thus subjected to three or four-point bending. The connections behave similarly whether the diagonal is in tension or in compression. In contrast with conventional frames, this type of innovative dissipative device allows for energy dissipation in the connections rather than in the structural members. The devices are of partial strength in order to protect the braces from yielding and buckling and therefore assure that all diagonals remain active regardless of being under tension or compression. The frames with INERD\textsuperscript{TM} connections besides their high ductility have the advantage that in case of damage after a strong seismic event, they can be easily repaired demanding significantly less material, time and equipment compared to conventional braced frames. Experimental as well as numerical results show that these devices concentrate damage, preserving the bracing system from buckling and assuring that all the steel members behave in the elastic range \cite{30}–\cite{32}.
Pin was round and rectangular; the round shape have the advantage to avoid torsion effect and concentration of the stress in the edge of the shape, but the disadvantage to be less chipper.
In terms of time history two different cases have been implemented in the test: a) using indication provided by ECCS and b) applying time history recorded during an earthquake in Greece in 1978. Dissipative connections have shown high capacity to absorb energy and withstand displacement among different stories, maintaining high resistance and rigidity, allowing to maintain an unreplaceable part in the elastic range.

*Figure 1.13: Example of UNERD device location*

*Figure 1.14: Graph of Inerd 14 - Global frame Response*
Problems Found: In terms of the dissipative behaviour, U-Devices loaded parallel to the connecting plate behave better than those loaded perpendicularly to connecting plate. A better dissipative behaviour of connections exists when the angle is different from some specific angle.

1.3. Objective

Steel elements are expected to be able to sustain large plastic deformation in the bending and shear thanks to dissipative features that reduce seismic energy through hysteresis cycles. After 1994 Northridge earthquake when more than 150 steel moment resisting frame structure suffered damage, primarily brittle fractures at the welded beam-to-columns connection. Codes were developed to implement the regulation and ensure seismic safety common rules. In according to the UNI EN 1998-1, 2005 the seismic-resistance steel structure can be classified on their behaviour against horizontal force and the dissipation energy mechanism. The seismic resistance behaviour can allow the designer to reduce the elastic design response spectrum in order to take into account benefit of nonlinear behaviour of the structure in the redistribution of the action acting. How much they can be reduced depend on the factor q- named as “behaviour factor” that intrinsically represent the behaviour of the structure to hinder the seismic action, higher behaviour factor means higher ductility capacity.

This work intends to promote and investigate coupling of two innovative seismic resistance system in the main frame direction that at the same time improve reparability after seismic event and reflecting the indication rules of hierarchy of the resistance.
In Moment Resisting Frame, the resistance to lateral forces is provided primarily by rigid frame action that is, by the development of bending moment and shear force in the frame members and joints. By virtue of the rigid beam–column connections, a moment frame cannot displace laterally without bending the beams or columns. The bending rigidity and strength of the frame members is therefore the primary source of lateral stiffness and strength for the entire frame. Flexural regime of beams, columns and connection where the seismic energy dissipation occurs through formation as high number of dissipative zone depend on the chosen design philosophy. Innovative MRF using FUSEIS system device reflect the seismic resistance concept to shift the plastic hinge region from beam-columns joint to beam in other hands associated large plastic deformation away from the potentially brittle failure connection areas. Furthermore this work is aimed to design the different FUSEIS for different storey’s promoting an almost simultaneously activation of the device during displacement in order to optimize global ductility and global energy dissipation capacity of the overall frame, avoiding the development of local collapse mechanisms which is not able to explore the full plastic reserves of the structure.

Global collapse mechanisms are preferred because lead to the development of the maximum number of plastic hinges before the collapse of the structure, achieve more safety and allow prediction of the collapse mechanism. As show in the figure below, a soft story collapse mechanism can compromise the stability of the whole structure considerably. This occurs especially in the story where the resistance of the beam exceeds, under the proportional load acting, the resistance of the columns providing a “strong beam-weak columns” mechanism.
Concentrically Braced Frame can be defined as an arrangement of elements in an orthogonal pattern in both elevation and on plan. The resistance to horizontal forces is provided by members subject only to compression and tension load. Regulation have traditionally assessed less seismic resistance to concentrically bracing frames compared to the MRF. In order to induce the plasticization of braces in tension, all the members, beams, columns, and beam-to-column connections, must be designed with an appropriate over strength so they behave in the elastic range. Innovative INERD device implies that the design axial resistance of beams and columns must be bigger than the correspondent plasticization value of the braces.

Moment resisting frames with concentric bracing is a coupling of those two single frame system defined as dual system in which the horizontal forces are absorbed by both moment frames and concentric acting in the same plane.

When earthquakes occur, a building undergoes dynamic motion. This is because the building is subjected to inertia forces that act in opposite direction to the acceleration of earthquake excitations. These inertia forces, called seismic loads, are usually dealt with by assuming forces external to the building. Since earthquake motions vary with time and inertia forces vary with time and direction, seismic loads are not constant in terms
of time and space. In designing buildings, the maximum story shear force considered to be the most influential, is the maximum acting in one of the two orthogonal direction of the building, in plane X,Y. According to clause 4.3.3.5.1.(3) of EC8 action effects due to the combination of the horizontal components of the seismic action may be computed using both of the two following combinations:

\[ E_{Ed,x} + 0.3E_{Ed,y} \]

\[ 0.3E_{Ed,x} + E_{Ed,y} \]

Where “+” indicate in combination whit, \( E_{Ed,x} \) represents the action effects due to the application of the seismic action along the chosen horizontal axis x of the structure and \( E_{Ed,y} \) represents the action effects due to the application of the same seismic action along the orthogonal horizontal axis y of the structure. The MRF are more flexible than CBFs – for this reason MRF whit Fuseis were placed in the direction where less mass is seismically active while CBFs whit INERD device were placed in the direction where more mass is seismically active. Response of the coupled system will be investigate in a 3D numerical model in order to assess performance in terms of local and global seismic resistance behaviour.

![Dual frame system](image)

*Figure 1.19: Dual frame system*
2. Description of FUSEIS (Bolted Cover Plate Device System)

2.1. Introduction

The FUSEIS bolted cover plate devices are a kind of seismic fuses for steel and composite steel-concrete moment resisting frames that provide good seismic performance and easiness of the repair work. They consist in a cross-sectional weakening located at the beam ends at a certain distance from the beam-to-column connections, avoiding in this way potential brittle failures at the welds. It acts as dissipative seismic fuses, force the plastic hinge to develop at the fuse device through concentration of inelastic behavior, preventing the spreading of damage into the beams and columns concentrating all the damage efficiently and are easily replaceable, so that repair work after an earthquake is limited to replacing the fuses by new ones, thus ensuring low-cost and very quick repair work. Basically, they have a simple detail and calculation procedure, making them easy to manufacture.

Figure 2.1: FUSEIS Main Section Elements
FUSEIS 2-1 achieves seismic resistance performance by introducing a discontinuity on the composite beams of a moment resisting frame and assembling the two parts of the beam through steel plates bolted to the web and flange of the beam. The connections between the steel plates and the beams are obtained by means of high strength friction grip (HSFG) bolts. These bolts are tightened according to the provisions given in EN 14399-2:2005 [36]. The part of the beam near the connection is reinforced with steel plates welded to the web and to the flanges. Also the part of the column near to the connection is reinforced in order to obtain an adequate over-strength and hence concentrates all the damage to the fuse device. The gap in the reinforced concrete slab also prevents damage due to concrete crushing. The longitudinal rebars are continuous over the gap, thus ensuring the transmission of stresses. The configuration of the device on a typical beam-to-column connection

In detail, the beam zone close to the fuse is reinforced with additional welded plates, both at the web and flanges, hence to avoid any sort of damage (e.g. spreading of plasticity) at the connections and in the adjacent irreplaceable steel parts of the beam. There are no strict design indications for these reinforcement plates, but the tested specimens were fitted with reinforcement plates with cross sectional areas roughly equivalent to those of the corresponding parts of the steel profile (web or flange). The duplication of the web and flange areas prevents the deformation that might otherwise develop at the holes, simplifying the repair procedures and limiting slippage at the corresponding bolts.

Figure 2.2: Fuse Device in a Moment Resisting frame

To avoid cracking of the concrete in the fuse section due to flexural deformation, a gap (see Figure 2.2) is left in the concrete slab in the section of the fuse, though the steel reinforcement is not interrupted in the gap section with the additional steel rebar’s in order to keep higher the level arm and consequently the rotation center during the rotation. In other hands the scope of this gap is to allow concentrated rotational deformation to occur in the gap section, avoiding both crushing of the concrete as well as damage to the floor finishes (such as tiles and so on).
2.2. Experimental Investigations on FUSEIS2-1

2.2.1. Experimental Investigations on Individual Device

Experimental tests were conducted on three different sub-assemblies of a beam-to-column of connection to the structural engineering laboratory of the Instituto Superior Tecnico of the University of Lisbon.

2.2.2. Experimental Set-Up

The basic test assembly consisted of a typical beam-to-column sub-assembly, comprising a composite beam with an IPE300 profile supporting a 150 mm thick and 1450 mm wide reinforced concrete slab, with a HEB240 profile column.

The gap width in the reinforced concrete part of the fuse could be different from that of the steel parts of the fuse. The recommended values for the gap width in the reinforced concrete (slab) and in the steel parts are, respectively, 20% of the height of the slab and 10% of the total height of the composite cross-section. Non-linear behaviour is expected to concentrate only in the fuse plates, which can be easily replaced by unbolting of the damaged plates and bolting the new ones.

The difference between the test specimens is the free buckling length \( L_0 \) measured between the innermost bolt rows of the fuses. Within this length both flange and web plates are unrestrained and therefore, are free to buckle. The following three different values of \( L_0 \) were chosen for each sub-assembly: 140, 170 and 200 mm. These fuses differed in terms of the geometric parameters of the flange plates, while the web plates designed to withstand shear forces should have the same dimensions in all tests. Therefore, the only cross-section dimensions that changed between tests, were the thickness \( t_f \) and width \( b_f \) of the flange plate. Each test was performed until complete failure of the fuse flange plate, after which the fuse plates were replaced by new ones and another test had been carried out. The testing order was as follows: first plates D, A, B, and C, followed by a repetition of this set of plates, performed in the same order. illustrates the dimensions of the flange plates of the fuse specimens. The monotonic tests were conducted after the conclusion of the cyclic tests.

Table 2: Dimensions of the Flange Plates of the Fuse Specimens (in mm)

<table>
<thead>
<tr>
<th>Flange Plate</th>
<th>A</th>
<th>B</th>
<th>C</th>
<th>D</th>
</tr>
</thead>
<tbody>
<tr>
<td>( t_f )</td>
<td>10</td>
<td>10</td>
<td>12</td>
<td>8</td>
</tr>
<tr>
<td>( b_f )</td>
<td>120</td>
<td>170</td>
<td>150</td>
<td>140</td>
</tr>
</tbody>
</table>
The dimensions presented provide the fuses with different values of capacity ratio $\alpha$ which can be defined by the following equation.

$$\alpha = \frac{M_{\text{Max, fuse}}}{M_{\text{pl, beam}}} \quad (1)$$

Where $M_{\text{max, fuse}}$ is the maximum moment developed by the fuse device which can be estimated analytically for each fuse type, taking the hardening and buckling phenomena into consideration with the help of a MatLab code developed by Espinha (2011) (this clearly introduced in design section), and $M_{\text{pl, beam}}$ is the plastic resistant moment of the non-reinforced area of the composite cross-section of the beam.

---

Figure 2.3: Experimental Test Set-Up a) Experimental Test Overview b) Free Buckling Length c) Positioning of the Bolted Fuse Device

The latter is a controlling design parameter that define rotation of the plate. Energy dissipation capacity is strictly correlated to ductility that plays one of the most important roles in describing the seismic performance, it results from a variety of non-linear phenomena, such as yielding and buckling of the plates, friction between the plates on the bolted connection interface and cracking of the concrete.
2.3. Analysis of the Experimental Results

2.3.1. Cyclic behaviour

The analysis of the results is based on Moment-Rotation diagrams of the fuse (M-θ). As shown in Errore. L'origine riferimento non è stata trovata. (a) the hysteretic behaviour of the fuses is stable, characterized by a marked pinching phenomenon, due to the slippage of the bolts and the buckling of the fuse plates.

![Diagram of Fuse C](image1)

*Figure 2.4: Moment -rotation (M - θ) Diagram of Fuse C*

![Comparison of Fuse C and Fuse D](image2)

*Figure 2.5: Comparison in terms of Moment -rotation (M-θ) of Fuse C and Fuse D*
Asymmetry of the diagram in terms of moments, is due to the strength loss caused by buckling of the fuse plates when subject to hogging bending moment. Nevertheless, deformation capacity is achieved because all specimens are able to perform ±41 mrad rotations, which is higher than the minimum value recommended by EC8 (35 mrad for DCH structures).

By comparing the moment-rotation diagrams between two tests of the same fuse specimen, becomes clear that there is a slight deterioration in terms of strength and energy dissipation. This is a consequence of the damage accumulation on the parts of the test assembly that are not replaced between tests, such as that due to cracking on the concrete slab.

Comparison between Fuses whit same buckling length but different area Fig 2.14b represent an higher moment capacity for the once whit an higher area, consequently widely spread hysteresis cycle for the Fuse C compared to Fuse D that have respectively an area 1800 mm² and 1120 mm².

2.3.2. Monotonic and cyclic behaviour

The monotonic behaviour can be compared with the cyclic behaviour in Figure 2.6. Both diagrams are very similar in terms of the initial stiffness and yield moments. The monotonic diagram seems to adjust well to the cyclic diagram, for the same rotation range, closely resembling the cyclic envelope curve. The differences in terms of the deformation capacity shown by the monotonic tests, can be attributed to damage accumulation due to low cycle fatigue effects. This aspect is more marked in the case of the hogging tests, because the hogging monotonic test was conducted after the sagging one, stiffness at the end of each cycle, which imposed high ductility demands to the specimen, resulting in increased deterioration of the concrete slab.
2.3.3. Failure mode

Two different failure modes were identified, consisting in the development of cracks at the gross mid-section or at the net section of the flange plate under tension, as shown in Figure 2.7. The observation of the tests indicates that the severity of the buckling and the low cycle fatigue effects precipitate the opening of cracks in the middle of the flange plate, where buckling deformation is also higher. On the other hand, the smaller the net area (when compared with the corresponding gross area of the cross-section of the flange plate) the higher the tendency towards spreading of cracks near the holes of the bolts, leading to the occurrence of the failure at the net cross-section. In order to evaluate both effects simultaneously, a dimensionless parameter was identified, aiming at understanding the causes of these different failure modes. The failure mode parameter $\zeta$ is given by:

$$\zeta = \frac{10}{\lambda} \frac{b_{f,beam} - 2\phi_{bh}}{b_f - 2\phi_{bh}} \left( \frac{t_f}{t_{f,beam}} \right)^3$$  \hspace{1cm} (2)

Where $\phi_{bh}$ is the diameter of the holes of the bolts, $b_{f,beam}$ and $t_{f,beam}$ are the width and thickness of the flanges of the steel beam, respectively. By observing the results, for values of $\zeta$ below 0.55, the failure takes place on the gross cross-section of the flange plate of the fuse. This type of failure mode is related to the large deformations on the flange plate caused by the buckling phenomenon, which causes the appearance of a stress concentration at the mid-length of the gross section of the plate, leading to the opening of cracks on the steel's surface. The stress fluctuations in the plastic range induced by the repetitive buckling along the cycles impose a gradual crack opening and propagation, due to low cycle fatigue [37]. These phenomena result in a progressive...
damage accumulation with the cycles and, consequently, the approach towards failure is also more progressive. On the other hand, for values of ζ above 0.85, fuse plates start to show failure on the net section of the flange plate. This failure mode is caused by the accumulation of tensile stresses near the bolt’s hole, leading to the sudden fracture of the flange plate and, therefore, to a failure mode which is less adequate. Measurements showed that both the column and the composite beam remained in the elastic range, moving similarly to rigid bodies with little elastic deformations. This aspect is very important, since it proves that the fuse is able to successfully protect the remaining irreplaceable parts.

![Figure 2.7: Failure Modes Example a)Gross-Mid and b) Net Section](image)

2.3.4. Composite behaviour

The specimens have shown a significant composite behaviour, where the slippage at the slab-beam interface proved to be relatively small, with values below 0.3 mm for all specimens, as may be observed in the typical slippage diagram shown in Figure 2.8, where F is the force measured by the load cell.

![Figure 2.8: Force-Displacement Diagram for Beam-Slab Connection](image)
Therefore, it is possible to conclude that the initial stiffness of each fuse is almost independent of its geometric characteristics, but rather dependent on the testing order. The deterioration of the irreplaceable parts of the sub-assembly resulting from the damage accumulation on previous tests has, therefore, a fundamental influence on the initial stiffness of the fuse. The deterioration of the irreplaceable parts comprises, among others, cracking of the concrete slab and smoothening of the connecting steel surfaces. Progression of cracking of the concrete along the tests may be observed in Figure 2.9. Among other consequences, this effect contributes to the increase of the free length of the longitudinal rebars, which therefore become more flexible. On the other hand, the smoothening of the steel surfaces induces a stiffness decrease due to the loss of friction between connecting plates. In this way, the connection presents more slippage of the bolts.

![Figure 2.9 Progressive Slab Damage](image)

**Figure 2.9** Progressive Slab Damage a) First, b) Second, c) Third and d) Forth Tests

### 2.4. Experimental Investigations on Overall Frames

In order to simulate a more realistic case, eight full scale tests are implemented and tested under seismic actions. Test results are analysed in terms of global force displacement behaviour, energy dissipation, drift capacity, frame stability as well as composite steel frame performance as were carried out on composite steel frame with fuse devices in the structural engineering laboratory of Politecnico di Milano (POLIMI).

#### 2.4.1. Experimental Set-Up

The frame specimens consist of four HEB240 steel columns, two IPE300 steel beams, and a 150-mm thick reinforced concrete slab. The slab is supported by IPE160 secondary-transverse beams placed every 1.4 m, in addition to a pair of transverse beams that are placed at each beam–column connection. Full shear connection is provided between the slab and the steel beam by means of IPE100 sections welded on top of the beam flange, acting as shear studs. The design of the composite slab is made according to Eurocode 4. High strength friction grip (HSFG) bolts are used in order to connect the steel plates to the beams in the fuse parts. The bolts are tightened according to the provisions given in BS EN 14399-2:2005. Longitudinal reinforcements (designed
according to the provision of EC 8, Annex J) consist of B450C Ø20/100 bars on the upper level, and Ø16/200 +Ø12/200 bars on the lower level. The transverse reinforcements consist of Ø12/72 bars near the fuse section and Ø10/72 bars in the rest of the slab.

It is evident that the seismic response of a frame with FUSEIS 2-1 depends mainly on the stiffness and strength of the flange plate. Therefore in order to achieve a controlled yielding of the plate and improve the behaviour of the frame under cyclic loading, design should try to archive a sequential yielding of the fuses. During the full-scale tests, the web plate and the free buckling length of the flange plate were left constant while changing only the thickness (t_f) and width (b_f) of the flange plate.

The frame, hence, is subjected to four cyclic (quasi static according to ECCS) loadings which are displacement-controlled (pushover tests) with velocity of 21 mm/min are implemented. The tests are considered satisfactory when a drift causing at least a 35 mrad rotation in the fuse devices will be obtained without significant inelastic deformation on the structural elements as well as on the reinforced concrete slab.

![Figure 2.10: Schematic Representation of the Bolted Fuse Device](image)

### 2.4.2. Test Results

Eight cyclic tests are implemented on the steel-composite frame with four different fuses. Each test is performed until complete failure of the fuse flange plate occurs, whichever fails first. The fuse elements had to be designed weaker than the adjacent members in order to force the position of the plastic hinge, to remain within the fuse and to avoid that damage spreads to the non-dissipative zones. In order to do so, the just previously described testing parameter α was introduced, which relates the resistance capacity of the fuse with the plastic resistance of the cross-section of the composite beam.
Measurements of relative rotations and displacements in the vicinity of beam-to-column connection showed that the columns and beams remained elastic with no evidence of plastic deformation or local buckling. The beam-to-column connections, which have larger moment capacity than the fuse parts, remained almost perfectly rigid. At the end of each test, the damage plates were dispatched instantly then a new plate was installed (the time required to replace one fuse device was approximately 30 min).

The deformations in the steel reinforcement did not go beyond the elastic range, as expected. Maximum relative displacement between the slab and the beam was 0.5 mm, which means the composite action between the reinforced concrete slab and the steel beam had satisfactorily achieved. Since the rotation center (plastic neutral axis) is quite above (between the two layers of the steel reinforcement of the slab), all the deformation is concentrated in the plates of the fuse devices. Both rotations and moments are computed at the mid-section of the fuse. The maximum rotation observed in the fuse devices is 40 mrad and after all the tests implemented, there was not any significant damage in the concrete slab (see Figure 2.11). Considering the fact that Eurocode 8 provisions require the connections to have a rotation capacity of the plastic hinge zone of at least 35 mrad (achieved with a strength degradation less than 20%) for the structures of high ductility class (DCH), and 25 mrad for the structures of medium ductility class (DCM), it can be concluded that the fuse devices performed well reaching plastic rotations larger than 35 mad without significant reduction in strength and stiffness. The displaced shape of the frame can be observed in Figure 2.11.

![Figure 2.11: Frame Displacement under Loading in the a) –X and b) +X Direction](image)

The overall behaviour of the fuse devices is summarized by means of moment rotation diagrams. The hysteretic behaviour of the fuses is stable and characterized by a pinching phenomenon, due to the slippage of the bolts and to the buckling of the fuse
plates when they are under hogging rotations. The fuse elements deform beyond their yield limit and contribute to the energy dissipation in the frame.

![Moment Rotation Diagram](image)

*Figure 2.12: An Example of Moment Rotation (M-θ) Diagram (Plate D).*

The loss of resistance in hogging bending that can be seen on the negative side of the moment rotation diagrams is caused by the buckling of the lower plate connected to the flange. The max. Plastic capacity achieved by the fuse elements was up to 335 kNm during sagging and up to 260 kNm during hogging of the element. The area under the hysteresis loops represents the energy dissipated in the fuse system during horizontal cyclic loading. Maximum displacement achieved without any significant damage to the structure and the composite slab was 55 mm at the top joint of the frame which means an inter-storey drift of 1.9%.

2.5. **Numerical Investigations**

2.5.1. **Numerical Investigations on Individual Device**

A qualitative comparison was performed, by comparing the experimental deformed shape with that of the numerical model for one single specimen, as displayed in 0 for both a) sagging and b) hogging. In order to do so, a series of numerical finite element analyses were developed by Abaqus. These models were initially calibrated based on the experimental results, assuming that both the beam and column had a rigid behaviour and that the composite beam presented full shear connection. Concrete was modelled as a linear elastic behaviour, considerably reducing the computational costs. The properties of the steel were modelled with linear hardening and the Von Mises yield criterion was used, bearing in mind the provisions of the EN 1993-1-1 and EN 1993-1-5. After achieving a good calibration with the solid FEM-models by Abaqus, further investigations were performed using beam elements in Perform 3D and SAP2000, using for the purpose non-linear spring models to model the constitutive behavior of the devices in a rather simple way.
As far as the stiffness is concerned, the finite element model is stiffer than the tested experimental models (see Figure 2.13). The difference could therefore be a consequence of the elastic stiffness loss shown by those specimens, due to the damage accumulation on the irreplaceable parts and also to cracking of the concrete and other low cycle fatigue effects.

Figure 2.13: a) Sagging and b) Hogging (Approximate Device Rotation=40mrad)

In order to simulate the correct behaviour of the fuse device in building design models, the parameters of the link must be calibrated according to the results of the tests implemented during the research project. The process starts with calibrating the
numerical analysis parameters of the component model. First, the fuse cross section behaviour is obtained from the analytical model based on the stress-strain relationship of the materials. Then this cross section behaviour is given as moment-rotation input to the Pivot Hysteresis model, and finally the parameters of the hysteresis model are calibrated comparing the experimental result with the analysis one (see Figure 2.15). The results showed a very good agreement between the experimental and numerical results, mainly, in terms of moment capacities, initial and nonlinear stiffness, buckling and pinching behaviour.

![Comparison between Experimental and Numerical Results](image)

**Figure 2.15: Comparison between Experimental and Numerical Results**

### 2.5.2. Numerical Investigations on Overall Frame

The fuse device behaviour is studied by means of two different numerical approaches. In order to have a better understanding of the connection response and to allow for the development of a simple engineering model, first a refined finite element modelling technique is used (by adopting the software package ABAQUS) in which the computational effort is very expensive, when it is necessary to demonstrate that the whole plasticization occurs in the fuses only (see Figure 2.16). Then, a simple engineering model is developed with the commercial software SAP2000. In order to do so, multilinear plastic pivot hysteresis has been defined to model the moment-rotation capacity of the fuse.
Also in this case, the fuse cross section behaviour obtained from the analytical model based on the stress-strain relationship of the materials are given as a moment rotation diagram input along with the pivot model parameters that are calibrated with the component tests. Then the results of the analyses are compared with the experimental results of the frame in terms of global force displacement behaviour. The model consists of a simple beam and link with the same geometry used in the experimental test set-up.

Figure 2.17: An Example of Moment-Rotation Diagram (Plate D)
3. Design Rules for FUSEIS Systems

Based on the experimental and analytical research carried out during the project Design Rules were developed giving all necessary information for conceptual design, analysis and design of building frames with FUSEIS systems, retaining the format of Eurocode 8. The design of a building with FUSEIS2-1 devices should be in compliance with the disposals of the relevant EN, in particular with EN1993-1-8.

Since the damage and energy dissipation may only occur due to inelastic behaviour of the replaceable parts i.e. FUSEIS device, irreplaceable parts i.e. beams and columns must be elastically designed to ensure that they remain undamaged when the device achieves its resistant capacity. On the other hand, beams must be locally reinforced at the “interface” with the fuse, to reduce any sort of damage which might develop at the holes (e.g. spreading of plasticity) at the connection and in the adjacent irreplaceable steel parts of the beam, simplifying the repair procedures and limiting slippage at the corresponding bolts. The local reinforcement of the beam may consist of an additional steel plate welded to both sides of the web and lower flange for the same length of the fuses cover plates. The bending resistance of the devices can be evaluated by defining the value of the capacity ratio \( \alpha \) of the fuse. In general, the results showed that fuses with higher values of capacity ratios (\( \alpha \)) provide higher performance levels in terms of stiffness, resistance, ductility and dissipated energy. Nevertheless, fuses with values of \( \alpha \) close to unity and, therefore, whose strength is like that of the composite beam, induce more damage in unreplaceable part and thus fail to concentrate plasticity within the fuse section.

3.1. Design Rules for Linear Dynamic Analysis

The fuse devices should be included at the all beam ends that belong the MRF. In the building design process, the cross-sections of the relevant structural elements should be first pre-designed for the same building but without fuses, considering the relevant limit states.

3.1.1. Composite beam plastic resistance

The maximum moment resistance of the composite beams \( M_{pl,Rd,beam} \) is the value of maximum bending in hogging and sagging which can be analytically calculated as follows in function of the position of neutral axes the depend from the resistance of the section parts.
Resistance of the elements

Resistance of the concrete

\[ R_c = h_c \times b_{eff} \times 0.85 \times f_{cd} \]  \hspace{1cm} (3)

Resistance of the steel section

\[ R_a = A \times f_{yd} \]  \hspace{1cm} (4)

Resistance of the upper beam flange

\[ R_f = b \times t_f \times f_{yd} \]  \hspace{1cm} (5)

Resistance of the webs

\[ R_w = b \times t_f \times f_{yd} \]  \hspace{1cm} (6)

Resistance of the longitudinal rebar’s

\[ R_s = A_s \times f_{sd} \]  \hspace{1cm} (7)

In function of the resistance is possible to know where the neutral axes are located and assess the resistance of the composite beam under positive bending

If \( R_c > R_a \) the neutral axis is located within the concrete slab:

\[ M_{pl,Rd}^+ = R_a \times \left( \frac{h}{2} + h_p + h_c - \frac{R_a}{R_c} \times \frac{h_c}{2} \right) \]  \hspace{1cm} (8)

If \( R_c = R_a \) the neutral axis is in the steel deck:

\[ M_{pl,Rd}^+ = R_a \times \left( \frac{h}{2} + h_p + \frac{h_c}{2} \right) \]  \hspace{1cm} (9)

If \( R_c < R_a \) and \( R_c > R_w \) the neutral axis is in the upper flange of the beam:

\[ M_{pl,Rd}^+ = R_a \left( \frac{h}{2} \right) + R_c \left( \frac{h_c}{2} + h_p \right) - 2 \left[ \frac{(R_a - R_c)^2}{4R_f} t_f \right] \]  \hspace{1cm} (10)

If \( R_c < R_a \) and \( R_c < R_w \) the neutral axis is in the web:

\[ M_{pl,Rd}^+ = R_c \left( \frac{h_c}{2} + h_p + \frac{h}{2} \right) + \left( \frac{1 - \frac{R_c}{R_a}}{1 - 0.5 \times \frac{b \times t_f}{A}} \right) \times W_{pl,Y} \times f_{yd} \]  \hspace{1cm} (11)
Where:
- $h_c$ is the concrete depth slab above the profiled decking
- $h$ is the high of the beam
- $h_s$ is the distance from the rebar’s and top of the beams

The same procedure can be performed for the negative bending moment to assess the resistance under hogging movement.

If $Rs < Ra$ and $Rs > Rw$ the neutral axes are in the upper flange of the beams:

$$M_{pl,Rd}^- = Ra * \left(\frac{h}{2}\right) + Rs * h_s - \frac{(Ra - Rs)^2}{4 * R_f} * t_f$$  \hspace{1cm} (12)

If $Rs < Ra$ and $Rs < Rw$ the neutral axis is in the upper flange of the beams:

$$M_{pl,Rd}^+ = Rs * \left(\frac{h}{2} + h_s\right) + \left(W_{pl,y} f_{yd}\right) \left(1 - \frac{1 - \left(\frac{Rs}{Rs}\right)}{0.5(\Delta - 2b_t)}\right)$$  \hspace{1cm} (13)

### 3.1.2. Fuses resistance design

With the design, resistant values of the pre-designed composite beams $M_{pl,Rd,beam}$, the resistant moment of the fuses $M_{Rd,fuse}$ may be computed through:

$$\alpha = \frac{M_{Rd,fuse}}{M_{pl,Rd,beam}}$$  \hspace{1cm} (14)

Note that the higher values of $\alpha$ provide higher resistant capacities, energy dissipation and stiffness. On the other hand, lower values of $\alpha$ are more efficient to prevent the spreading of damage to the irreplaceable parts of the structure.

A parametric study based on numerical analyses is underway to define the optimal values of this parameter. In any case, based on the results already achieved, it can be suggested that, to obtain the best performance of the fuse device in terms of capacity and energy dissipation, $\alpha$ values should be assumed in the range:

$$0,60 \leq \alpha^+ \leq 0,75$$
$$0,30 \leq \alpha^- \leq 0,50$$

The fuse devices should comply according to the classification of joints of EN1993-1-8 (5.2) for both strength and stiffness. In the relevant expressions, the variables which are defined for the joints should be replaced by the same variables defined for the fuses. Most of the fuses will be classified as partial strength, semi-rigid joints and be modelled...
according to the provisions of part 5.1 of the same code. To achieve a good behaviour, the structural steel should be very ductile, matching the ductility requirements of 3.2.2(1) of EN1993-1-1. The bending stiffness of the fuses may be obtained by dividing the yield moments $M_{\text{fuse}}$ with the yield rotations $\theta_p$ of the fuse for both sagging and hogging. The yield moments should be obtained by an elastic analysis considering that the flange plate is in yielding. The design yield stress $f_{yd}$ of the steel flange plate should be used for sagging and the design yield stress at buckling $f_{yd,b}$ for hogging:

$$f_{yd,b} = \min\{f_{yd}; \sigma_b(\varepsilon_y)\}$$  \hspace{1cm} (15)

The yield rotations $\theta_y$ are obtained by the equation $\chi_y = L_0 \theta_y$, if cross-sections remain plane and regarding the strain diagram on the fuse at yielding for both sagging and hogging.

### 3.1.3. Bending Resistance of the Fuse

Since the fuse plates may buckle at hogging rotations, the bending behaviour of the fuses is asymmetric in most of the cases. During earthquake both case can occur and the global behaviour is governed by the lower resisting once. Therefore, there is a need for computing both sagging and hogging resistant moments of the fuse, $M_{Rd,fuse}^+$ and $M_{Rd,fuse}^-$, respectively.

![Fiber Layout](image)

**Figure 3.1: Fiber Layout**

The buckling behaviour of the fuse plates may be controlled by the geometric slenderness that is function of the free buckling length $L_0$. Procedure aimed to obtain proper length in function of $t_f$ was developed to design fuses plate.

### 3.1.4. Free buckling length

Investigation show that fuses are stressed not only in compression but also in bending so that in the theoretical model the influence of the moment in the design of the free
buckling length needs to be accounted. To assess the reliability of the approach two theoretical models were developed and the results compared. First methods start from the equation for bending and compressed columns:

\[ \frac{P_{b,Ed}}{P_{cr}} + \frac{M}{M_u} = 1 \]  \hspace{1cm} (16)

Assumption that the moment acts only on one axes can be satisfied thanks to the restraint provided by the web plate that hinder distortions of the fuses out of plane. The equation can be rewritten in the form:

\[ \frac{P_{b,Ed}}{P_{cr}} = 1 - \frac{M}{M_u} = (1 - k) \]  \hspace{1cm} (17)

In terms of stress equation can be obtained dividing by the cross-section:

\[ \sigma_{b,Ed} = \sigma_{cr}(1 - k) \]  \hspace{1cm} (18)

Calibration on the test result shows that the moment acting on the fuse \( M \) during the formation of kinematic mechanism is 45% of the \( M_u \). The correction coefficient can be indicated as:

\[ \sigma_{b,Ed} = 0.55 \times \sigma_{cr} \]  \hspace{1cm} (19)

Slenderness of the plate can be described as the ration between the effective length and the inertial radius in the following:

\[ \lambda = \frac{L_0}{i} = \frac{\beta \times L}{i} \]  \hspace{1cm} (20)

The \( \beta \) coefficient consider the reduction of the length due to presence of \( M \). For a rectangular plate, if the width is assumed equal to width of the beam flange, the inertial radius can be written in equivalent form as follows:
\[ i = \frac{l}{\sqrt{A}} = \frac{t^2}{12} \]  

(21)

Slenderness can be assumed as the follows:

\[ \lambda = \frac{\beta * L * \sqrt{12}}{t} \]  

(22)

Substituting the \( \lambda \) in the equation of the Eulerian stress and impose a limit that keep critical stress below the yielding one is possible write the new equation impose the limit fixed.

\[ \frac{\pi^2 * E * t^2}{\beta^2 * L^2 * 12} < \frac{f_y}{\gamma_{m0}} \]  

(23)

Finally, is possible figure out the effective length for each plate in function only of the supposed thickness:

\[ L = \frac{\pi * t}{2 * \beta} \sqrt{\frac{E * \gamma_{m0}}{3 * f_y}} \]  

(23)

\( \beta \) coefficient allows to consider the Moment involved in the buckling of the plate during the deformation of the plate and is equal to 1.5; indeed, substituting this value is possible demonstrate the initial formula that consider \( M \) as 45% of the \( M_u \):

\[ \sigma_{b,E,d} = (1 - \frac{1}{(1,5)^2}) * \sigma_{cr} = (1 - \frac{1}{2,25}) * \sigma_{cr} = (1 - 0,45) * \sigma_{cr} = 0,55 * \sigma_{cr} \]

Second procedure was developed to obtain the free buckling length from the intersection of the stress-strain elastic limit curve and the buckling mechanism curve. The model is based on a previously developed model by Gomes (1992), initially applied to the buckling problem of longitudinal rebar’s in concrete columns, treated also in the Miguel work thesis [reference].
Figure 3.2: Buckling mechanism of the FUSEIS

\[ P = \frac{M_p}{w} \]  \hspace{1cm} (24)

Where \( M_p \) is plastic moment of the rectangular cross-section of the fuses plate,

\[ M_p = \frac{b_f \cdot t_f^2}{4} \cdot f_y \]  \hspace{1cm} (24)

By writing the transversal and vertical displacement, \( \omega \) and \( \delta \) respectively the coordinate \( \theta \).

\[ \omega = \frac{L_0}{2} \cdot \sin \theta \]  \hspace{1cm} (25)

\[ \delta = L_0 \cdot (1 - \cos \theta) \]  \hspace{1cm} (26)

Expanding in series these trigonometric expression, it is possible to obtain for small displacement:

\[ \omega = \sqrt{\frac{\delta \cdot L_0}{2}} \]  \hspace{1cm} (27)

By combining the result the equation can be expressed in terms of a stress-strain relationship through the respective definitions,
\[ P = \frac{M_p}{w} = \frac{2\sqrt{2}M_p}{\sqrt{\frac{L_0}{\sqrt{\delta}}}} \]  

(28)

Introduction elastic linear relationship in the buckling stress, imposing a value of \( \sigma \) equal to the yield strength, substituting the \( \sqrt{\varepsilon} = 2\% \) and ordinate the formula in function of \( L_0 \) can be obtained the following direct expression:

\[ L_0 = \frac{2\sqrt{2}M_p}{A_f y \sqrt{\varepsilon}} \]  

(29)

Comparison of output from both was done for all the plate with different thickness involved in the Lisbon test to assess the reliability of the formula. Table below summarize the result.

**Table 3: Dimension and result of plate tested**

<table>
<thead>
<tr>
<th>Plate</th>
<th>Plate dimension [mm]</th>
<th>Real Length [mm]</th>
<th>Lo, Method 1 [mm]</th>
<th>Lo, Method 2 [mm]</th>
<th>Err Meth 1 %</th>
<th>Err Meth 2 %</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>10x80</td>
<td>170</td>
<td>167</td>
<td>158</td>
<td>1%</td>
<td>7%</td>
</tr>
<tr>
<td>C</td>
<td>12x110</td>
<td>200</td>
<td>200</td>
<td>189</td>
<td>0%</td>
<td>5%</td>
</tr>
<tr>
<td>F</td>
<td>8x140</td>
<td>140</td>
<td>133</td>
<td>126</td>
<td>5%</td>
<td>10%</td>
</tr>
</tbody>
</table>

Both the expression is valid and provide average error lower than the 5%. By assuming a plastic distribution of forces for bending-shear interactions, the contribution of the web plates of the fuse to the bending resistance should be neglected but web plate has the same free length of the flange plate.

3.1.5. Design of Flange plate

The dimensions of the flange plate of the fuse device control the resistant bending moment of the cross-section of the fuse and is therefore dependent on the value of the capacity ratio of the device. If the plastic neutral axis is coincident with the centre of gravity of the longitudinal reinforcement the area of the flange plate may be estimated in pre-design by the expression:
\[ A_{f,\text{fuse}} = \frac{M_{\text{Rd,fuse}}^+}{f_{\text{yd}} z} \]  

Where \( M_{\text{Rd,fuse}}^+ \) is the sagging resistant moment of the fuse device is, \( f_{\text{yd}} \) is design yield strength of the structural steel according to EN1993-1-1 and \( z \) is the distance between the flange plate and the center of gravity of the rebar layers (see Fig 4.1). The hogging resistance of the fuse \( M_{\text{Rd,fuse}}^- \) should be obtained through an elastic-plastic analysis on the cross section with a modified constitutive relationship for the flange plate \( \sigma_{\text{mod,b}}(\varepsilon) \), given by:

\[ \sigma_{\text{mod,b}}(\varepsilon) = \min\{\sigma_t(\varepsilon); \sigma_b(\varepsilon)\} \]  

Where \( \sigma_t(\varepsilon) \) is the stress-strain relationship obtained through experimental tensile tests or according to the Annex C.6 of EN1993-1-5 and \( \sigma_b(\varepsilon) \) is the buckling stress-strain relationship given by

\[ \sigma_b(\varepsilon) = \frac{f_{\text{yd}}}{\lambda_f \sqrt{2\varepsilon}} \]  

Where \( \lambda_f \) is the geometric slenderness of the flange plate.

### 3.1.6. Longitudinal Reinforcement

The longitudinal reinforcement should be computed to remain elastic when the maximum resistant moment is developed by the fuse. In order to avoid yielding of the rebar, their area has to be computed so that the plastic neutral axis lies between the upper and lower rebar layers of the slab. It is recommended to provide the upper rebar layer with the double of the area of the lower layer. One should notice that only the rebar that are located within the effective width of the concrete flange of the composite beams at the sections adjacent to the fuse should be accounted for the bending resistance. The effective widths should be computed according to EN1993-1-8 (7.6.3) and EN1994-1-1 (5.4.1.2). The position of the plastic neutral axis should be obtained by an elastic-plastic analysis of the cross section with the material properties obtained experimentally or as defined in annex C.6 of EN1993-1-5. The non-yield condition should be verified by imposing the plastic curvature \( \chi_p \) to the cross-section of the device at sagging, assuming that the ultimate strain of the structural steel \( \varepsilon_u \) is developed at the flange plate. The plastic curvature is given by \( \theta_p = L_0 \chi_p \), where \( \theta_p \) is the plastic rotation as defined in 6.6.4 (3). The verification consists in performing an elastic-plastic analysis and checking that the strains on both rebar layers \( \varepsilon_s \) are lower than the yield strain of the material \( \varepsilon_{sy} \) according to EN1993-1-1.

The following total longitudinal rebar quantity \( A_{\text{sl,\text{total}}} \) is recommended to be used as a first iteration at a pre-design stage:
\[ A_{st,tot} = 7 A_{f,use} \frac{f_{yd}}{f_{sd}} \]  

(33)

Where \( A_{f,use} \) is the area of the cross-section of the flange plate of the fuse with \( f_{sd} \) the design yield strength of the reinforcement steel according to EN1992-1-1. The length of the gap on the concrete slab should be detailed to assure that the plastic rotation \( \theta_p \) is developed without contact of the concrete parts. After determining \( A_{st,tot} \) and \( A_{f,use} \) the sagging and hogging bending resistances may be computed and the new values of the capacity ratios \( \alpha^+ \) and \( \alpha^- \) should be determined. Finally, the total upper and lower area of the rebar can be determined from the following equation.

\[ A_{\text{upper rebar}} = 10 A_{\text{flange plate}} \frac{f_{yd}}{f_{sd}} \]  

(34)

\[ A_{\text{lower rebar}} = \frac{A_{\text{upper rebar}}}{2} \]  

(35)

### 3.2. Anchorage length according to EN 1992 1-1 8.4

The anchorage length \( L_a \) is defined according to EC2 in order to be effectively anchored, so that they can perform function of reinforcement of the concrete and be able to absorb the tensions expected in relation to materials characteristics. Under hypothesis that uniform tension is distributed along surface of the bar, bonding force can be expressed by relation:

\[ F_b = \pi \phi L_a f_b \]  

(36)

Where:

- \( f_b \) indicate the ultimate tensile resistance offered by the concrete
- \( L_a \) indicate the anchorage length
- \( \phi \) is diameter of the rebar’s

From equilibrium between the effort absorbed by the bars \( F_s = A_s \sigma_{sb} \) and the friction force in possible to obtain the anchorage length the formula in function of the anchorage length

\[ L_{b,\text{rqd}} = \frac{\phi}{4} \frac{\sigma_{sd}}{f_{bd}} \]  

(37)

Under hypothesis that acting tension in the rebar’s is equal to design yield tension finally the anchorage length is:
\[ L_{b,rqd} = \frac{\varnothing}{4} \times \frac{f_{yd}}{f_{bd}} \tag{38} \]

Where \( f_{bd} \) is the design bond tension that can be assumed in function of the concrete class and can be extracted by the following table in simplified approach.

**Table 4: Design friction tension for different concrete condition**

<table>
<thead>
<tr>
<th>Concrete class</th>
<th>( f_{bd} ) in condition of good adherence [Mpa]</th>
<th>( f_{bd} ) in condition of mediocre adherence [Mpa]</th>
</tr>
</thead>
<tbody>
<tr>
<td>C(20/25)</td>
<td>2.38</td>
<td>1.67</td>
</tr>
<tr>
<td>C(25/30)</td>
<td>2.69</td>
<td>1.88</td>
</tr>
<tr>
<td>C(28/35)</td>
<td>2.98</td>
<td>2.09</td>
</tr>
</tbody>
</table>

For and B450C steel rebar’s the design yielding strength \( f_{yd} = 391 \) Mpa is possible obtain the following value of anchorage length.

**Table 5: \( L_{(b,rqd)} \) value for B450C rebar in function of diameter and concrete class**

<table>
<thead>
<tr>
<th>Steel Class</th>
<th>Concrete Class</th>
<th>Good adherence condition</th>
<th>Mediocre adherence condition</th>
</tr>
</thead>
<tbody>
<tr>
<td>B450C</td>
<td>C(20/25)</td>
<td>41 ( \varnothing )</td>
<td>59 ( \varnothing )</td>
</tr>
<tr>
<td></td>
<td>C(25/30)</td>
<td>36 ( \varnothing )</td>
<td>52 ( \varnothing )</td>
</tr>
<tr>
<td></td>
<td>C(28/35)</td>
<td>33 ( \varnothing )</td>
<td>47 ( \varnothing )</td>
</tr>
</tbody>
</table>

The design anchorage length \( L_{bd} \) result be in according to 8.4.4 prEN 1992-1 and is the same of lap length.

\[ L_{bd} = \alpha_1 \alpha_2 \alpha_3 \alpha_4 \alpha_5 L_{b,rqd} \geq L_{b,min} \tag{39} \]

Where \( \alpha_1 \) consider effect of the form of the bars assuming adequate cover, \( \alpha_2 \) consider effect of concrete cover, \( \alpha_3 \) consider effect of confinement by transverse reinforcement,
α₄ consider influence of one or more welded transverse bars along the design anchorage length \( L_{bd} \) and α₅ the effect of the pressure transverse to the plane of splitting along the design anchorage length. Mandatory the product \( \alpha_2 \alpha_3 \alpha_5 \geq 0.7 \). Value of α parameter can be figure out by table 8.3 prEN 1992-1.

The anchorage of links and shear reinforcement should normally be effected by means of bends and hooks, or by welded transverse reinforcement. A bar should be provided inside a hook or bend.

### 3.2.1. Overlaps

The detailing of overlaps between bars shall be such that the transmission of the forces from one bar to the next is assured, spalling of the concrete in the neighbourhood of the joints does not occur and large cracks which affect the performance of the structure do not occur. The arrangement of the overlaps should comply part 8.7.2 prEN 1992-1.

![Overlaps position limits for the reinforced rebar's](image)

Provision are satisfied if the following conditions are fulfilled:

- The clear transverse distance between two lapped bars should not be greater than 4 \( \varnothing \) or 50 mm, otherwise the lap length should be increased by a length equal to the clear space exceeding 4 \( \varnothing \) or 50 mm;
- The longitudinal distance between two adjacent laps should not be less than 0.3 times the lap length, \( L_{bd} \);
- In case of adjacent laps, the clear distance between adjacent bars should not be less than 2 \( \varnothing \) or 20 mm.
3.3. **Concrete confinement gap**

Distance between the concrete parts has to be provided in order to ensure that damage don’t affect the slab during rotation of the fuse. Starting from the yield deformation of the steel rebar and elongation was been developed the following approach:

\[ \varepsilon_{ys} = 0.002 \]  \hspace{1cm} (40)

\[ \frac{\Delta L}{L} = \varepsilon \]  \hspace{1cm} (41)

Where:

- \( L \) = is the initial length
- \( \theta \) = is the design rotation angle equal to the 30 mrad

The elongation is assumed to be proportional to half of the slab width multiplied for plastic rotation of the fuses.

\[ \frac{\theta \cdot \frac{h}{2}}{\varepsilon} \leq L_{bd} + \frac{G}{2} \]  \hspace{1cm} (42)

Where:

- \( G \) = is defined as the Gap length between the concrete in order to allow movement;
- \( L_{bd} \) = is the total anchorage length as defined in prEN 1992-1;
- \( h_s \) = is the distance between the rebar’s layer

Reordering the formula in function of \( G \) is possible obtain the fundamental relationship that express the concrete gap:

\[ G = \frac{\theta \cdot \frac{h}{2}}{\varepsilon} - 2L_{bd} \]  \hspace{1cm} (43)

3.4. **Design of the Fuse Devices for Shear**

The web plates should be considered alone for the shear resistance of the fuse. The resistance of the web plates should be computed according to EN1993-1-1 (6.2.6), considering a shear area \( A_p \) equal to the area of the cross-section of the web plates. Special attention should be given to the verification of shear buckling, as specified in EN1993-1-5 (5). Shear deformability may be neglected for common spans in buildings. Hence, the minimum area of the web plate of the fuse can be determined by the following equation:
\[ A_w = \frac{V_{Ed}}{f_{yd}} \sqrt{3} \]  \hspace{1cm} (44)

Where \( V_{Ed} \) is the total shear force \( V_{Ed} = V_{Ed,M} + V_{Ed,G} \)

\[ V_{Ed} = V_{Ed,M} + V_{Ed,G} \]  \hspace{1cm} (45)

\( V_{Ed,M} \) is the shear force due to moment resistance of the fuse

\[ V_{Ed,M} = \frac{M_{fuse,Rd}^+ - M_{fuse,Rd}^-}{d} \]  \hspace{1cm} (46)

\( V_{Ed,G} \) is the shear force due to gravity loads, \( d \) is the distance between the fuses. Verification of shear buckling can be examined by the following equation:

\[ \frac{h_w}{t_w} < 72 \frac{\sqrt{235}}{\eta f_{yd}} \]  \hspace{1cm} (47)

Where \( \eta \) is a parameter that may be assumed to be equal to 1.2 steel grades up to and including S460. For higher steel grades \( \eta = 1.00 \) is recommended.

### 3.5. Design of the Non-Dissipative Connecting Elements

The connection between the fuse plates and the steel beam should be rigid and have full strength. Based on the disposals of EN1998-1 (6.5.5(3)), the following expression should be used for the design force that shall be considered on non-dissipative connections between each fuse plate and the corresponding connected part of the beam:

\[ F_{Ed} = 1.1 \gamma_{ov} A f_u \]  \hspace{1cm} (48)

Where \( A \) is the tensile resistant area of the fuse plate under consideration which should be taken as the net cross section for bolted fuses; \( \gamma_{ov} \) is the overstrength factor, according to of EN1998-1 (6.2 (3)a)). The recommended value is 1.25; \( f_u \) is the ultimate tensile strength of the steel of the fuse plate according to EN1993-1-1 (Table 3.1). All the disposals of EN1993-1-8 that are not mentioned here should also be verified.

The reinforcement of the composite beam with additional welded steel plates prevents the spreading of plasticity near the high concentration of stresses due to the transmission of forces near the fuse device. For beams with bolted devices, the thickness of the reinforcement plates should be sufficient to avoid the opalization of the holes on the beam with bolted devices, to avoid the enhancement of difficulties in the replacing operation of the fuse. The resistance of the reinforced cross-section of the composite beam should be checked for the relevant ULS at the critical sections.
The width of the flange reinforcement plate should be larger or equal than the width of the flange plate of the fuse. The reinforced beam should be provided with a length of at least the height of the steel profile in both directions of the fuse. As a recommendation, the area of the reinforcing flange plates and the area of both web plates should at least be equal to the area of the flange and web of the steel profile, respectively.

3.6. Design of the Bolted Connection

The bolts that connect the fuse plates to the beam should be designed to remain elastic when the fuse develops its maximum moment. Despite being replaceable parts, irrecoverable deformations on the bolts could compromise the unbolting process when trying to replace the fuse plate and so, these should remain elastic and be treated as non-dissipative elements. The following expression should be satisfied for non-dissipative bolted connections:

\[ F_{v,Rd} > \frac{F_{sd}}{n} \]  

(49)

Where \( F_{v,Rd} \) is the shear resistance per shear plane, according to EN1993-1-8 (Table 3.4) computed with the yield strength of the bolts \( f_{yd} \), \( F_{sd} \) is the design force of the non-dissipative connections, \( n \) is the number of bolts used to transmit the shear forces. The bolts should be pre-loaded and designed to behave as type B shear connections according to of EN1993-1-8 (3.4 and 3.9). In case of high-strength structural bolting for preloading the connection must satisfy rules included in EN 14399. Nominal value for bolts have to assessed in function of the class of the bolts in terms of yielding strength and ultimate tensile strength.

<table>
<thead>
<tr>
<th>Bolts class</th>
<th>4.6</th>
<th>4.8</th>
<th>5.6</th>
<th>5.8</th>
<th>6.8</th>
<th>8.8</th>
<th>10.9</th>
</tr>
</thead>
<tbody>
<tr>
<td>( f_{yb} ) [N/mm(^2)]</td>
<td>240</td>
<td>320</td>
<td>300</td>
<td>400</td>
<td>480</td>
<td>640</td>
<td>900</td>
</tr>
<tr>
<td>( f_{ub} ) [N/mm(^2)]</td>
<td>400</td>
<td>400</td>
<td>500</td>
<td>500</td>
<td>600</td>
<td>800</td>
<td>1000</td>
</tr>
</tbody>
</table>

*Figure 3.4: Yield and Ulitmo strength of the bolts*

3.6.1. Influence of distance

Nominal transversal distances must be check according to ENV 1993-3-1, in particular following value must be satisfied in order to permit to exploit the resistance of the plate in the zone adjacent to the hole:
3.6.2. Plate resistance according to EN 1993-1-8

The resistance of the beam web flange is:

$$F_{RP0} = \min \left\{ \frac{bt_0 f_{yk}}{\gamma_{M0}}, \frac{b_{net} t_0 f_{yk}}{\gamma_{M2}} \right\}$$  \hspace{1cm} (50)$$

Where \( b \) is the length dimension of the flange covered by the plate, \( t_0 \) is the thickness of the flange, \( b_{net} = b - nd_0 \) where \( d_0 \) is the is the bolt diameter \( d_b \) increased for the tolerance of the bolts (assumed 1 mm without specific information), \( f_{yk} \) is characteristic yielding strength of the plate material, \( \gamma_{M0} = 1.05 \) and \( \gamma_{M2} = 1.25 \).

The resistance of the cover joint is:

$$F_{RP1} = 2 \min \left\{ \frac{b t_1 f_{yk}}{\gamma_{M0}}, \frac{b_{net} t_1 f_{yk}}{\gamma_{M2}} \right\}$$  \hspace{1cm} (51)$$

Where \( t_1 \) is the thickness of the cover joint. In case of double cover joint \( t_1 = \sum t_i \). In case of the fuse is the thickness of the flange plate and the thickness of the reinforced plate welded.

Result are satisfied when:

$$\min \left\{ F_{RP0}; F_{RP1} \right\} > F_{Sd}$$  \hspace{1cm} (52)$$

3.6.3. Bolts resistance under shear according to EN 1993-1-8

The bolts have to be check under shear action according to EN1993-1-8 under hypothesis that all bolts share the load equally.

$$F_{v,Rd} = n \frac{k f_{tb} A_{res}}{\gamma_{M2}}$$  \hspace{1cm} (52)$$

Where \( n \) is the number of face connected and \( \gamma_{M2} = 1.25 \) and \( A_{res} \) is the gross area reduced for the presence of the hole in the section perpendicular to the acting force. \( k \) is 0.5 for for bolts class 4.8, 5.8, 6.8 and 10.9 and \( k \) is 0.6 for bolts class 4.6, 5.6, and 8.8

Check result satisfied when \( F_{v,Rd} \geq F_{v,Sd} \)
3.6.4. Bearing resistance according to EN 1993-1-8

The bolts must be check under bearing in order to limit the bearing deformation at the bolt hole and ensure reparability after earthquake without opalization of the hole. Bearing stress effects are independent of the bolt type because the bearing stress acts on the connected plate not the bolt. The stress will be highest at the radial contact point (A). However, the average stress can be calculated as the applied force divided by the projected area of contact.

\[
F_{b,Rd} = k \frac{f_u a_b d t}{\gamma_{M2}} < F_{sd}
\]

Where \(d\) is the bolts diameter, \(f_u\) is the ultima tensile resistance of the bolts, and \(t\) is the thick of the plate evaluated. In case of edge bolts: \(a_b = \min\left(\frac{e_2}{d_0}, \frac{f_{th}}{f_{tk}}, 1\right); k = \min(2.8 \frac{e_2}{d_0} - 1.7, 2.5)\) In case of internal bolts \(a_b = \min\left(\frac{p_1}{3d_0} - 0.25; \frac{f_{th}}{f_{tk}}\right); k = \min(1, 4 \frac{e_2}{d_0} - 1.7, 2.5)\)

Bearing resistance is the minimum between the internal and external case and it must be compared whit the acting force \(F_{sd}\).

3.7. Additional Detailing Remarks

The detailing rules which are not mentioned in this guide should be done considering the provisions of the relevant EN. In particular, special attention should be given to the provisions of EN1998-1 regarding the detailing of the concrete slab of the composite beam. The transverse reinforcement of the beam flange of the composite beam should be computed according to the provisions of EN1994-1-1 and EN1998-1. In particular, they should be designed taking into account the shear resistance of the shear connectors and the axial forces on the concrete flange and on the steel profile, according to the design procedures of EN1994-1-1 (6.6.6).
### Table 6: Summary of the check that as to be provided

<table>
<thead>
<tr>
<th>Moment resisting of the composite beam section</th>
<th>As described</th>
</tr>
</thead>
<tbody>
<tr>
<td>Free buckling length</td>
<td>( L_0 = \frac{2 \sqrt{2} M_p}{A f_y \sqrt{\varepsilon}} )</td>
</tr>
<tr>
<td>Anchorage length according to EC</td>
<td>( L_a = \frac{\phi}{4} \frac{f_{yd}}{f_{bd}} )</td>
</tr>
<tr>
<td>Concrete Gap</td>
<td>( G = \frac{\theta \cdot h}{\varepsilon} - L_a )</td>
</tr>
<tr>
<td>Area of flange plate</td>
<td>( A_{f,fuse} = \frac{M_{Rd,fuse}}{f_{yd} z} )</td>
</tr>
<tr>
<td>Longitudinal reinforcement</td>
<td>( A_{\text{upper rebar}} = 10 A_{\text{flange plate}} \frac{f_{yd}}{f_{sd}} )</td>
</tr>
<tr>
<td></td>
<td>( A_{\text{upper rebar}} = 2 A_{\text{lower rebar}} )</td>
</tr>
<tr>
<td>Area web plate</td>
<td>( A_w = \frac{V_{Ed} \sqrt{3}}{f_{yd}} )</td>
</tr>
<tr>
<td>Non dissipative connection elements</td>
<td>( F_{Ed} = 1,1 \gamma_{ov} A f_u &lt; F_{Rd} )</td>
</tr>
<tr>
<td>Plate resistance</td>
<td>( \min {F_{Rp0};F_{Rp1}} &gt; F_{Sd} )</td>
</tr>
<tr>
<td>Shear resistance of the bolts</td>
<td>( n \frac{k f_{th Ares}}{\gamma_{M2}} &gt; F_{Sd} )</td>
</tr>
<tr>
<td>Laps length - anchorage length</td>
<td>( L_{bd} = \alpha_1 \alpha_2 \alpha_3 \alpha_4 \alpha_5 L_{b,reqd} \geq L_{b,min} )</td>
</tr>
</tbody>
</table>
4. Validation of non-linear plate behaviour
In order to assess reliability of the numerical data, non-linear behaviour of the plate must be represented in terms of Moment rotation diagram. Thanks to the experimental data it is possible to calibrate the numerical model using the FEM software. The constitutive stress strain relationship of the material involved in the test are represented and are the same for all the tests.

![Figure 4.1: Material uniaxial stress-strain curve B450C](image1)

![Figure 4.2: Material uniaxial stress-strain curve B450C](image2)

Table 7: Sensible plate parameter for each plate

<table>
<thead>
<tr>
<th>Flange Plate</th>
<th>A</th>
<th>B</th>
<th>C</th>
<th>D</th>
</tr>
</thead>
<tbody>
<tr>
<td>Thickness [mm]</td>
<td>10</td>
<td>10</td>
<td>12</td>
<td>8</td>
</tr>
<tr>
<td>Width [mm]</td>
<td>120</td>
<td>170</td>
<td>150</td>
<td>140</td>
</tr>
<tr>
<td>I [mm$^4$]</td>
<td>10000</td>
<td>14167</td>
<td>21600</td>
<td>5973</td>
</tr>
<tr>
<td>Npl [kN]</td>
<td>330</td>
<td>476</td>
<td>423</td>
<td>263</td>
</tr>
<tr>
<td>Lo [mm]</td>
<td>170</td>
<td>170</td>
<td>170</td>
<td>170</td>
</tr>
<tr>
<td>Ncr [kN]</td>
<td>716</td>
<td>1015</td>
<td>1547</td>
<td>427</td>
</tr>
</tbody>
</table>

4.1. Fuses characterization
In order to figure out the Maximum bending moment at which the fuse is able to sustain before buckling is necessary figure out the acting axial force in the edge plate. Elastic critical axial force for flexural buckling is governed by Euler, plastic axial force and plastic moment are the governing parameters

$$N_{cr} = \pi^2 \frac{EI}{L_0^2}$$

(53)
\[ N_{pl} = f_y A = f_y bt \]  
\[ M_{pl} = f_y W_{pl} \]  

In order to take into account these governing parameter and figure out the axial force needed to buckle the plate formulation for beam-columns was adopted.

\[ \frac{N}{N_{pl}} + \frac{M}{M_{pl}} = 1 \]  

The moment acting in the middle of the plate \( M \), considered as the II ord moment, is function of the axial force that act and the deflection of the plate. Rearranging the formula in terms of \( N \) considering an initial imperfection \( v_o \) of the plate equal to 0.1 mm the buckling curve for each plate was been plotted in function of \( v_m \).

\[ N (v_m) = \frac{N_{pl} M_{pl}}{M_{pl} + v_m N_{pl}} \]  

These curve was been intersected by the \( \sigma - \varepsilon \) relationship of the specific plate, that tend to an horizontal asymptotic value equal to \( N_{pl} \) that is exactly the one needed to plasticize the section.

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![Figure 4.3: Characterization Plate D](image-url)

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63
Figure 4.4: Characterization Plate A

Figure 4.5: Characterization Plate B

Figure 4.6: Characterization Plate D
The maximum plastic bending moment for the flange plate is obtained multiplying the axial force identified by intersection point of two lines by the level arm. Under hypothesis that this value doesn’t vary and enforcing the center of rotation in the middle of the rebar’s layer in order to keep them in elastic range, for a composite beams cross-section made with IPE 300 and a 150 mm concrete, with two centimetres of concrete cover this value is 375mm.

<table>
<thead>
<tr>
<th>Plate</th>
<th>( M_{Rd,max.fuse} ) [kNm]</th>
<th>( \theta_{Rd,max.fuse} ) [mrad]</th>
</tr>
</thead>
<tbody>
<tr>
<td>D</td>
<td>114</td>
<td>3</td>
</tr>
<tr>
<td>A</td>
<td>91</td>
<td>3</td>
</tr>
<tr>
<td>B</td>
<td>178</td>
<td>6</td>
</tr>
<tr>
<td>C</td>
<td>184</td>
<td>6</td>
</tr>
</tbody>
</table>

*Figure 4.7: Hogging curve parameter*

From the rigidity of the elastic positive branch is possible to assess the rotation angle yield stress in the negative branch assuming for the elastic part the same rigidity of the positive one. The propagation of the plasticity zone during the post-buckling process is analytically characterized both for the hardening or the softening plasticity laws. The Link used to reproduce this Moment-Rotation behaviour of the Fuse in SAP200 are needed to consider the dissipation of energy required during the seismic action. The modes of insertion differ depending on the model (in our case, 2D).

The sudden loss of strength identified by the softening part is often unrealistic and difficult to analyse, reason why it is possible to consider the negative bending diagram from the yielding moment \( M_{Rd}^{\gamma} \) to the ultima moment \( M_{Rd}^{\delta} \) as a single segment, that ends with the failure.
4.2. Validation of individual fuse non-linear plastic behaviour

The comparison between the experimental curve of the fuses recorded during the test and analytical monotonic behaviour is show for all the plates. Validation of the after mentioned assumptions is needed in order to assess how the Fuses withstand seismic action.

Figure 4.8: Moment-Rotation Diagram of the Plate D

Figure 4.9: Moment-Rotation Diagram of the Plate A
The data show good agreement between the Fuse moment diagram. Reduction is observed in terms of initial stiffness and maximum moment rotation capacity but results still remain on the safe side. The order of the test was the same of the represented figure. It is possible to appreciate the reduction of the initial resistance due to damage of the irreplaceable parts in the experimental tests (concrete slab and reinforced rebar’s) when pass from the first test to the third once. This affect the initial stiffness of the cyclic curve. In order to reproduce this phenomena in the characterization of the Fuse slippage between the bolt was considered in the analytical calculation providing 2 mm for plate B and C. Ovalization of the hols were modelled numerically in order to build up a more representative curve of the second half plate series.
4.3. **Comparison of Numerical and Experimental – Individual frame**

Laboratory tests were carried out at the Department of Civil Engineering and Architecture of the Technical University of Lisbon, in order to characterize the behaviour of the bolted devices. In this case the experimental set up wasn’t a full scale frame, but a simpler system, basically made by a column, a composite beam and an actuator (namely a node). The steel of which these components were made was a S275 structural steel. The composite beam was made by an IPE300 and a 150 mm thick reinforced concrete slab; the longitudinal reinforcement made in B450C consist in a $\phi 20/100$ top layer, and a $\phi 16/100 + \phi 12/200$ bottom layer (dimensions in mm); the column was an HEB240. The fuse is installed near the joint between beam and column, as can be seen from the figure.

![Figure 4.12: Numerical Modelling of the Individual Fuse](image)

Four different fuses were tested numerically compared with the experimental model, keeping always constant the dimension of the web plate and changing only the flange. Material constitutive relationship are reported in table below.

![Figure 4.13: Material uniaxial stress-strain curve for S275](image)  
![Figure 4.14: Material uniaxial stress-strain curve for B450C](image)
The numerical simulation are carried out using SAP2000. The option Link/Support properties is adopted in order to model the Fuse, choosing a type of multilinear plastic. The property of the Link are not once experimental test but those validated in the comparison just described. More in detail: at the rotational degree of freedom of the link correspondent to the biggest bending moments, nonlinear properties are assigned, following the Pivot hysteresis model, that proved to be the best hysteresis model in our case, the only one capable to suit the data of the tests. The factor that vary are only the constitutive law of the moment rotation diagram, the other parameters were kept as default value.

**Figure 4.15: Moment-Rotation Diagram of the Plate D**

**Figure 4.16: Moment-Rotation Diagram of the Plate A**
The results show that the numerical model captures adequately the test specimen behaviour as concerns the initial stiffness and moment capacity under cyclic load, providing relatively accurate results, regarding the conducted simulations and its practical objectives. The most reliable model are the first and second one, in terms of cyclic and absence of major influence of locking phenomena, which were controlled successfully, leading to a good stiffness adjustment of the numerical model to the real behaviour of the fuses. The other test present deterioration in terms of initial stiffness and maximum resistance capacity due to damage accumulation in the irreplaceable part (concrete and rebar’s elements) showing difference between experimental and numerical once.
4.4. Comparison of Numerical and Experimental - Overall frame

In this case the tests were carried out at the Department of Structural Engineering of the Politecnico di Milano. A full-scale frame was built, made by beams, columns and an actuator. These elements were made by S275 structural steel. The composite beam was made by an IPE300 and a 150 mm thick reinforced concrete slab, the longitudinal reinforcement of the slab consist in a $\phi 20/100$ top layer, and a $\phi 16/100 + \phi 12/200$ bottom layer (dimensions in mm); the columns were HEB240.

![Figure 4.19: Numerical Modelling of the Overall Frame with Fuses](image)

Four fuses were tested; the web ones were kept constant instead of the flange plate vary according to the test performed in laboratory. The parameters regarding the Moment rotation of plastic hinges are obtained to assess the performance of the Fuses in the non-linear range. Two hinge constraints are applied at the base of the frame. All the columns are HE240B with S275 structural steel, while all the beams at the “first floor” are modelled as composite beam as used in the real test.

In the model we can assume that the only degree of freedom relevant for the plastic hinge formation in the beams is the moment M3, that is the moment around the z axis, so we consider only the effect of this action. The shear deformation is neglected because it leads to a brittle collapse of the elements, that isn’t the case analysed with the plastic hinge model, and the axial forces are negligible. The bending resistance of composite beam was assessed analytically and considered in the plastic hinge property in the model.

Three fuses are applied in the composite beam, 17 cm long and 36 cm spaced from the columns, modelled using the option multilinear plastic link, with the features already explained. Test frame has been restrained out of plane by means constrains. There is a rigid beam transfer the displacement between two top node.
Figure 4.20: Moment-Rotation Diagram of the Plate D – in bay plate

Figure 4.21: Moment-Rotation Diagram of the Plate D – out of bay plate
Figure 4.22: Force-Displacement Diagram of the Plate D – Global frame response

Figure 4.23: Moment-Rotation Diagram of the Plate A – in bay plate
Figure 4.24: Moment-Rotation Diagram of the Plate A – out of bay plate

Figure 4.25: Force-Displacement Diagram of the Plate A – Global Frame
Figure 4.26: Moment-Rotation Diagram of the Plate B

Figure 4.27: Moment-Rotation Diagram of the Plate B
Figure 4.28: Force-Displacement Diagram of the Plate B – Global frame response

Figure 4.29: Moment-Rotation Diagram of the Plate C – in bay plate
Figure 4.30: Moment-Rotation Diagram of the Plate C – out of bay plate

Figure 4.31: Force-Displacement Diagram of the Plate C – Global frame response
The numerical model represents correctly the experimental behaviour of the frame build experimentally in terms of moment rotation of the Fuses. As it can be seen from the graphs, the fuse elements deform beyond their yield limit and contribute to the energy dissipation in the frame in terms of global response Force-Displacement. The loss of resistance in the hogging bending that can be seen on the negative side of the moment rotation diagrams is caused by the buckling of the lower plate connected to the flange. Difference between numerical model and experimental are more evident in the last test done in terms of maximum resistance and initial rigidity. Initial stiffness of the device is almost independent of its geometric characteristics, but rather dependent on the testing order. The deterioration of the elements of the experimental set-up due to previous tests profoundly influences the initial stiffness of the fuse. Looking at our validations (and of course at the experimental curves) it is confirmed; indeed the testing order in all the three cases was: D, A, B and C, starting from the plate with the smallest area and ending with the plate with the largest; it emerges that the plate D has always a value of stiffness sensibly higher than the others and fit better the numerical test.
5. Case Studies

5.1. Introduction

In order to investigate the behaviour of composite steel-concrete frames using dissipative devices (FUSES) and examine the contribution of these devices to the energy dissipation, three buildings with different height will be examined in this session. All building have composite steel concrete slab and secondary beams which transfer the loads to the main frames, where the innovative device are employed.

5.2. Description of the case studies

Three archetype configurations which are vertically regular and square-plan, have been selected. They, considered as general office (class-B) buildings, are designed according to EN1993-1 [63] /EN1998-1 [3] and to the specific design guideline of the dissipative system. The case studies comprise three configurations as follows:

- 2 storeys, low-rise buildings
- 4 storeys, mid-rise buildings
- 8 storeys, high-rise buildings

For all buildings a common plan view has been selected. The number of bays in both direction is 3 with a span length of 8 m. The height of each story is 4m. They consist of a steel-concrete composite moment resisting frame in the Y direction and concentrically braced steel frame in the middle span of the X direction. Fuse devices are included in the structure at the end of the all beams in Y direction, (FUSES), while the INERDTM device are equipped at the end of all steel bracing elements in X direction. The concentric bracing system is located to accommodate the columns around their weak axis bending and the FUSEIS 2-1 are located in the direction along which the column are placed with strong axes bending.

![Figure 5.1: Plan of the 2/4/8-Story Archetype Structures](image-url)
5.3. **Loads**

In this framework dead loads, superimposed dead loads, and live loads have been considered. A summary of the applied loads is given in the following:

- **Dead Loads:**
  
  2.75 kN/m² composite slab + steel sheeting

- **Superimposed Loads:**
  
  Services, ceiling, raised floor: 0.70 kN/m² for intermediate floors

  1.00 kN/m² for top floor

  Perimeter walls 4.00 kN/m

- **Live Loads:**
  
  Offices (Class B): 3.00 kN/m²

  Movable partitions 0.80 kN/m²

  Total live load: 3.80 kN/m²

  Snow load to be ignored.
• Seismic Load:
  Importance factor: $\gamma_I = 1.0$
  Peak ground acceleration: $\alpha_{gR} = 0.20 \cdot g$
  Ground Type C – Type 1 spectrum:
  \[ S = 1.15 \quad TB = 0.20 \text{ sec} \quad TC = 0.60 \text{ sec} \quad TD = 2.00 \text{ sec} \]
  Lower bound factor: $\beta = 0.2$
  Vertical ground acceleration to be ignored.
  Behaviour factor $q = 4$

5.4. Materials

The materials used in the three buildings are given below:

- Structural steel: S355
- Concrete: C25/30
- Steel sheeting: Fe320
- Reinforcing steel: B500C

5.5. Analysis and Design of the Building without fuses

This work is part of European project INNOSEIS aimed to valorisation of innovative anti-seismic devices. Each studied device has a specific group of partner and a coordinator that work on it. Regarding INNOSEIS device the coordinator is National Technical University of Athens (NTUA) and Politecnico di Milano (POLIMI) is a beneficiaries. As indicated by European Commission, 3 archetype 3D frame will be investigate whit assigned dissipative device for each partners. This thesis work study a part of this 3D model assigned to POLIMI, the direction of moment resisting frame whit FUSEIS device.

The modelling, analysis and design of the buildings, was performed by means of the finite element program SAP2000. All beams and columns were simulated as beam elements, while no-section shell elements were used for the correct distribution of the load’s area. The composite slabs were designed by the program SymDeck Designer, which takes into account construction phases both for the ultimate and serviceability limit states. Columns are designed as steel members, with their section varying depending on the floor and the building. The assigned sections are given in detail in the following table. For all floors and buildings, IPE360 has been chosen for primary composite beams. Secondary beams are composite and simply supported with steel profile HEA200. Construction phases were critical for the design of these beams, so temporary supports need to be placed in order to reduce both bending deformation and
section size. Slabs are composite for all floors. They have been designed and checked according to the requirements of Eurocode 4 for all possible situations and no temporary supports are needed during construction phases. The thickness of the steel sheet is 0.80mm and the longitudinal reinforcement is Ø8/100. The steel beam is assumed to be connected to the concrete slab with the full shear transfer.

![Composite Slab Section](image)

**Figure 5.3 Composite Slab Section**

**Table 8: Columns section for the 2 storey building**

<table>
<thead>
<tr>
<th>Floor</th>
<th>Centre</th>
<th>Perimeter</th>
</tr>
</thead>
<tbody>
<tr>
<td>1-2</td>
<td>HEM320</td>
<td>HEB320</td>
</tr>
</tbody>
</table>

**Table 9: Columns section for the 4 storey building**

<table>
<thead>
<tr>
<th>Floor</th>
<th>Centre</th>
<th>Perimeter</th>
</tr>
</thead>
<tbody>
<tr>
<td>1-2</td>
<td>HEM340</td>
<td>HEB340</td>
</tr>
<tr>
<td>3-4</td>
<td>HEM300</td>
<td>HEB280</td>
</tr>
</tbody>
</table>

**Table 10: Columns section for the 8 storey building**

<table>
<thead>
<tr>
<th>Floor</th>
<th>Centre</th>
<th>Perimeter</th>
</tr>
</thead>
<tbody>
<tr>
<td>1-2</td>
<td>HEM550</td>
<td>HEB550</td>
</tr>
<tr>
<td>3-4</td>
<td>HEM500</td>
<td>HEB500</td>
</tr>
<tr>
<td>5-6</td>
<td>HEM450</td>
<td>HEB450</td>
</tr>
<tr>
<td>7-8</td>
<td>HEB360</td>
<td>HE360</td>
</tr>
</tbody>
</table>
5.6. Numerical Modelling of the case studies with FUSEIS devices

5.6.1. 2D Numerical model

The 2D model must capture accurately the behaviour of the 3D model in the direction considered. Subdivision of the load have to be performed for the influence area of the structural element considered. The following figure describe the influence area for the beam.

![Figure 5.4: Total beam influence area](image)

For each primary beam and influence width of 2.6 m. Each assigned load kN/m² will be applied as distributed load multiply it for the beam width, but the section of the beam will be modelled as the effective reacting width of the composite slab.

- Dead load: 2.75 kN/m² -> 7.32 kN/m
- Live load - top beam: 1.50 kN/m² -> 3.99kN/m
- Live load - intern. beam: 3.80 kN/m² -> 10.11 kN/m
- Superimposed load- top beam: 0.57 kN/m² -> 2.66 kN/m
- Superimposed load- intern beam: 0.70 kN/m² -> 1.86 kN/m

Each column has an area of influence higher than beams one because ultimately the columns would need to resist the imposed loads on the secondary beams which are the two internal frames and the two outer frames.
The difference between the influence area of the beam and influence area of the columns was evaluated and the correspond load applied at the columns as point load.

For end columns  
\[ A_{columns} - A_{beam} = 4(8 - 2,6) = 21,6 \, m^2 \]

For internal columns  
\[ A_{columns} - A_{beam} = 8(8 - 2,6) = 43,2 \, m^2 \]

The point load area the following:

- Dead load – end column:  
  \[ 2,75 \, kN/m^2 \rightarrow 58 \, kN \]
- Dead load – internal column:  
  \[ 2,75 \, kN/m^2 \rightarrow 117 \, kN \]
- Live load – end column - top:  
  \[ 1,50 \, kN/m^2 \rightarrow 31 \, kN \]
- Live load – internal column top:  
  \[ 1,50 \, kN/m^2 \rightarrow 63 \, kN \]
- Live load – end column - mid:  
  \[ 3,80 \, kN/m^2 \rightarrow 80 \, kN \]
- Live load – internal column - mid:  
  \[ 3,80 \, kN/m^2 \rightarrow 161 \, kN \]
- Superimposed load- end column - top:  
  \[ 0,57 \, kN/m^2 \rightarrow 21 \, kN \]
- Superimposed load- internal column top:  
  \[ 0,70 \, kN/m^2 \rightarrow 42 \, kN \]
- Superimposed load- end column – mid:  
  \[ 0,57 \, kN/m^2 \rightarrow 14 \, kN \]
- Superimposed load- internal column - mid:  
  \[ 0,70 \, kN/m^2 \rightarrow 29 \, kN \]
Nonlinear static analysis will be performed in order to evaluate the preliminary q-factor. The model incorporates all geometrical and material non-linearity that are fundamental parameters for a realistic modelling of a pushover analysis. The mass and stiffness of
secondary structural and non-structural elements also incorporated according to the state-of-practice approaches, e.g. via a leaning P-Δ column or a leaning frame.

A 2D nonlinear model of the structural system of each archetype building (previously pre-designed) was developed by using commercial finite element software SAP2000. A lumped plasticity modelling approach was employed for the non-linear models of the frames. Nonlinear material property was concentrated at the ends/mod-span of the frame elements by using particular features provided by the software for this scope (i.e. link and hinge property). Beam and column elements were modelled as frame elements and non-linearity was concentrated in the plastic hinges at their ends/mid-span. To characterize the non-linear behaviour of a plastic hinge, the generalized force–deformation properties suggested in FEMA 356 [15] were implemented. The plastic hinge property of the columns considers the interaction between axial force and bending moment.

![Frame hinge definer window](image)

**Figure 5.9: Frame hinge definer window**

Since the primary beams are composite steel-concrete sections, effective slab width need to be defined according to EC4 and EC8. The table report the value obtained for a 8m span beam near the support.
Table 11: Effective slab width

<table>
<thead>
<tr>
<th>Effective slab width</th>
<th>Sagging moments</th>
<th>Hogging moments</th>
</tr>
</thead>
<tbody>
<tr>
<td>EC4 Elastic</td>
<td>Not defined</td>
<td>1400mm</td>
</tr>
<tr>
<td>EC8 Elastic analysis</td>
<td>600 mm</td>
<td>800 mm</td>
</tr>
<tr>
<td>EC8 Plastic analysis</td>
<td>1200 mm</td>
<td>1600 mm</td>
</tr>
</tbody>
</table>

The beam resistance moment derived with effective width according to EC8 – plastic analysis hogging moment are reported in the following table for positive and negative bending.

Table 12: Composite beam section property

<table>
<thead>
<tr>
<th>M_{x,beam}^+ [kNm]</th>
<th>M_{pl,Rd}^+ [kNm]</th>
<th>M_{pl,Rd}^- [kNm]</th>
</tr>
</thead>
<tbody>
<tr>
<td>1011</td>
<td>1064</td>
<td>661</td>
</tr>
</tbody>
</table>

Therefore, after calculating the ultime bending resistance of the composite cross-section, they are implemented within the hinge property of the beams.
Beam-to-columns joint were considered as rigid in accordance with the connection detailing of the experimental tests. The length of the beams was subdivided into different elements in order to take into account both the presence of the fuses device and the part of the beam reinforced with additional welded plates. The part of the beam reinforced with additional welded plates, aimed at avoiding the spreading of plasticity to the connection, was reproduced in the numerical models by using different cross-section and plastic hinges properties around the device. The fuses were modelled as non-linear link elements inserted in the beams with a length equal to the free buckling length of the plates. The fuse devices were placed at a distance equal to the beam depth from the beam-to-column connections.

The multi-linear plastic pivot model was used as a hysteresis rule for the fuses. The values of the parameters used for the hysteretic model are defined after designing the fuse dimension and properties.
Figure 5.14: Link moment-rotation curve definer windows

The distribution of nonlinear material properties on the numerical model are summarized in the following figure.

Figure 5.15: Summary of lumped plasticity modelling-approach
5.7. **Pushover**

The pushover analysis is a static nonlinear analysis useful to investigate the behaviour of a structure under a particular load; it consists in “pushing” the structure until it collapses or a controlled deformation parameter reaches a predefined limit value. The pushing effect is obtained by applying a monotonically increasing load pattern (or displacement) and solving iteratively the static equilibrium equations. By plotting the total load applied versus the monitored point’s displacement is possible to obtain a characteristic force-displacement graph named “capacity curve”. This type of analysis is named static because the load increases in a long time span, so that no dynamic forces arise, and non-linear because it takes into account both geometrical non-linear properties (such as p-delta effects) and material nonlinear properties (i.e. plastic deformations of elements). According to clause 4.3.3.5.2. (5) and (1), the vertical component of the seismic action may be neglected because pushover analysis was performed and in the seismic design situation $a_{vg}$ was no greater than $0.25g$.

Applied vertical load are choose according to the following combination $G + 0.3Q$; then a horizontal static load was applied. Two different pushover mode was performed for each structure. At least two vertical distributions of the lateral loads should be applied:

- a “uniform” pattern, based on lateral forces that are proportional to mass regardless of elevation (uniform response acceleration);
- a “modal” pattern, proportional to lateral forces consistent with the lateral force distribution in the direction under consideration determined in elastic analysis.

Both the acceleration was performed in a control displacement push up to 4 meters. The sequence of hinge formation and the color-coded state of each hinge was viewed graphically, on a step-by-step basis, for each step of the pushover. If the plastic hinge formation occur at the columns means that the plastic resistance capacity of the beams is not so weak to avoid soft story mechanism. In other hands the story upper the column plastic hinge formation has fuse whit too strong bending resistance, that need to be replaced in an iterative process in order to slide up the formation of plastic hinge as much as possible. After different iteration was obtain the following archetype configuration in terms of fuse distribution.
5.8. 8 Story building

In order to provide a global collapse mechanism, different fuses were designed for different buildings. As a first iteration, web plates have been supposed with an equivalent area equal to the web of the steel beams, later according to capacity design concept they will be redesigned in order to reach final resisting cross-section. Different buckling length were modelled in function of the different designed fuses.

5.8.1. Fuse 1

As first iteration it was assumed $\alpha = 0.50$

$$M_{Rd, fuse} = M_{pl, Rd, beam, max} \times 0.50 = 1064 \text{kNm} \times 0.50 = 532 \text{kNm}$$

The level arm $z$ from the centre of rotation in the middle of the rebar’s and the flange plate is

$$h_a + h_p + \frac{h_c}{2} = 450mm + 73mm + \frac{77}{2}mm = 561mm$$

The bending resistance of the flange plate is consequently:

$$R_{flange plate} = \frac{M_{Rd, fuse}}{z} = \frac{532000 \text{kNm}}{561 mm} = 1128 \text{kN}$$

The area of the flange plate of the fuse may then be selected dividing the previous value with the yield stress of the material

$$A_{flange plate} = \frac{R_{flange plate}}{f_{yd}} = \frac{1128 \text{kN}}{0.323 \text{kN/mm}^2} = 3178 \text{mm}^2$$

Where $f_{yd} = 323 \text{N/mm}^2$ according to EC8 6.2. (2)P. It indicates that the distribution of material properties, such as yield strength and toughness, in the structure shall be such that dissipative zones form where they are intended to in the design. In order to satisfy this clause the yield strength of the steel of dissipative zones and the design of the structure actual maximum yield strength $f_{y, max}$ of the steel of dissipative zones should satisfy the following expression $f_{y, max} \leq 1.1\gamma_{ov} f_y$ where $\gamma_{ov}$ is the over strength factor used in design whit recommended value is $\gamma_{ov} = 1.25$. As indicate in the part (3)P this condition normally leads to the use of steels of grade S355 for non-dissipative members and non-dissipative connections (designed on the basis of the $f_y$ of S235 steels) and to the use of steels of grade S235 for dissipative members or connections where the upper yield strengths of steels of grade S235 is limited to $f_{y, max} = 355 \text{N/mm2}$. For steels of grade S235 and with $\gamma_{ov} = 1.25$ this method gives a maximum of $f_{y, max} = 323 \text{N/mm2}$ that is the value used in the calculation. Hereafter this value will be considered.

The following dimensions were adopted for the flange plate: 170x16mm
(\(A_{\text{flange plate}} = 3400 \, \text{mm}^2\)).

To compute the rebar quantity, one could start by computed the area of reinforcement according to the following criteria:

\[
A_{\text{upper rebar}} = 10 \, A_{\text{flange plate}} \frac{f_{yd}}{f_{sd}} = 10 \times 3400 \, \text{mm}^2 \times 0.396 = 13464 \, \text{mm}^2
\]

\[
A_{\text{lower rebar}} = \frac{A_{\text{upper rebar}}}{2} = 6732 \, \text{mm}^2
\]

Free buckling length was performed whit the formula (29) previous described:

\[
L_0 = 2 \sqrt{\frac{2}{3}} \frac{M_p}{f_{y}} \sqrt{\frac{1}{4} \cdot 203170} \frac{1}{4} \cdot 355 \sqrt{0.002} = 316 \approx 320 \, \text{mm}
\]

5.8.2. Fuse 2

As first iteration it was assumed \(\alpha = 0.45\)

\[
M_{Rd, fuse} = M_{pl,Rd,beam,max} \times 0.45 = 1064 \, \text{kNm} \times 0.45 = 478 \, \text{kNm}
\]

The level arm \(z\) from the centre of rotation in the middle of the rebar’s and the flange plate is

\[
h_a + h_p + \frac{h_c}{2} = 450 \, \text{mm} + 73 \, \text{mm} + \frac{77}{2} \, \text{mm} = 561 \, \text{mm}
\]

The bending resistance of the flange plate is consequently:

\[
R_{\text{flange plate}} = \frac{M_{Rd, fuse}}{z} = \frac{478800 \, \text{kNm}}{561 \, \text{mm}} = 1015 \, \text{kN}
\]

The area of the flange plate of the fuse may then be selected dividing the previous value with the yield stress of the material

\[
A_{\text{flange plate}} = \frac{R_{\text{flange plate}}}{f_{yd}} = \frac{1015 \, \text{kN}}{0.323 \, \text{kN/mm}^2} = 2860 \, \text{mm}^2
\]

The following dimensions were adopted for the flange plate: 170\times18\text{mm}

(\(A_{\text{flange plate}} = 3060 \, \text{mm}^2\)).

To compute the rebar quantity, one could start by computed the area of reinforcement according to the following criteria:
\[ A_{\text{upper rebar}} = 10 \frac{A_{\text{flange plate}} f_{yd}}{f_{sd}} = 10 \times 3060 mm^2 \times 0.396 = 12117 mm^2 \]

\[ A_{\text{lower rebar}} = \frac{A_{\text{upper rebar}}}{2} = 6058 mm^2 \]

Free buckling length was performed with the formula (29) previously described:

\[ L_0 = \frac{2 \sqrt{2} M_p}{A f_y \sqrt{\epsilon}} = \frac{2 \sqrt{2} \left( \frac{1}{4} \times 183170 \right)}{3060 \times 355 \sqrt{0.002}} = 284 \approx 280 mm \]

5.8.3. Summary of the model parameters

Detailed description of all key parameters involved in the model is presented below.

<table>
<thead>
<tr>
<th>Table 13: 8 Story Building free buckling length</th>
</tr>
</thead>
<tbody>
<tr>
<td>Free buckling length [mm]</td>
</tr>
<tr>
<td>Fuse 1</td>
</tr>
<tr>
<td>Fuse 2</td>
</tr>
</tbody>
</table>

Figure 5.16: 8 Story Building Fuse Layout
### Table 14: Dimension of fuse members

<table>
<thead>
<tr>
<th>Fuse</th>
<th>Web plate</th>
<th>Flange plate</th>
<th>Upper rebar</th>
<th>Lowe rebar</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Dimension</td>
<td>Dimension</td>
<td>Dimension</td>
<td>Area</td>
</tr>
<tr>
<td></td>
<td>[mm]</td>
<td>[mm]</td>
<td>[mm]</td>
<td>[mm²]</td>
</tr>
<tr>
<td></td>
<td>Area</td>
<td>Area</td>
<td>Area</td>
<td>Area</td>
</tr>
<tr>
<td>1</td>
<td>400x10</td>
<td>170*20</td>
<td>11Ø20</td>
<td>11Ø14</td>
</tr>
<tr>
<td></td>
<td>4000</td>
<td>13470</td>
<td>13823</td>
<td>6732</td>
</tr>
<tr>
<td>2</td>
<td>400x10</td>
<td>170*18</td>
<td>15Ø16</td>
<td>10Ø14</td>
</tr>
<tr>
<td></td>
<td>4000</td>
<td>12120</td>
<td>12063</td>
<td>6157</td>
</tr>
</tbody>
</table>

The upper and lower rebar’s was applied in the effective width cross-section, close to fuse device location for a length equal to beam depth. This procedure want enforce centre of rotation to lie in the middle of the rebar’s layer in order to don’t damage irreplaceable part.

### Table 15: 8 Story - property of the Fuses

<table>
<thead>
<tr>
<th>Fuse</th>
<th>M$_{pl}^+$ [kNm]</th>
<th>M$_{pl}^-$ [kNm]</th>
<th>M$_{rd,fus}^+$ [kNm]</th>
<th>M$_{rd,fus}^-$ [kNm]</th>
<th>$\alpha^+$</th>
<th>$\alpha^-$</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>1064</td>
<td>661</td>
<td>718</td>
<td>388</td>
<td>0,67</td>
<td>0,58</td>
</tr>
<tr>
<td>2</td>
<td>1064</td>
<td>661</td>
<td>679</td>
<td>349</td>
<td>0,63</td>
<td>0,52</td>
</tr>
</tbody>
</table>

![Fuse 1](image1.png) ![Fuse 2](image2.png)

**Figure 5.17: 8 Story - Constitutive laws of Fuse**
5.8.4. 8 Story building- Pushover curve

Nonlinear static analysis was performed and base shear-top displacement curve plotted. The following is comparison for same multi degree of freedom structure under different load pattern.

Frames under uniform acceleration was subjected to higher base shear force, in fact the curve reach higher value than model subject to acceleration pattern according to 1st mode. Initial rigidity seems to be similar until the model go out from elastic range. Important difference between bare frame and frame with fuse can be observed, in terms of ductility exploited by the structure: frame with fuse sustain less base shear and allow to achieve larger top displacement.

5.8.5. 8 Story building- Pushover result

Deformed shaper for building with and without fuse is plotted and showed.

Figure 5.18: 8 Story building (Uniform Acc) - δ=100cm

Figure 5.19: 8 Story building (1st Mode Acc) - δ=100cm
The result of pushover analysis show that at top joint displacement 100 cm, structure with fuse perform well against soft story mechanism providing global collapse without plasticization of the columns except at columns base. Model without fuse is not able to reach displacement 100 cm because collapse of the structure occurs early. By the comparison, it is clear the improvement guaranteed by the disposition of the fuse device.

5.8.6. 8 Story building- First plastic hinge activation

Activation of the first plastic hinge is investigated through each link of the most load columns in the model. The governing activation is lower step between the two side of the same joint. Each step corresponds to 1 cm top displacement.

The result of pushover analysis show that at top joint displacement 100 cm, structure with fuse perform well against soft story mechanism providing global collapse without plasticization of the columns except at columns base. Model without fuse is not able to reach displacement 100 cm because collapse of the structure occurs early. By the comparison, it is clear the improvement guaranteed by the disposition of the fuse device.

5.8.6. 8 Story building- First plastic hinge activation

Activation of the first plastic hinge is investigated through each link of the most load columns in the model. The governing activation is lower step between the two side of the same joint. Each step corresponds to 1 cm top displacement.
The plastic hinge activation occurs in beams instead of in the columns for building with innovative dissipative device. The governing activation steps is the lower side of same node. In both pushover case for building with fuse, activations are present in the range between 17 and 42 step in the most loaded columns (internal columns). This means an optimal behaviour because almost all the floor activates at same step, so hinge occurs simultaneously providing global collapse mechanism instead of local collapse mechanism exploiting better the plastic capacity of whole structure. Only last two floor present no activation of the fuse. Different iteration was performed but changing the last two floor fuse to a more thinner plate provide value not acceptable in terms of sensitivity drift ratio and inter-story drift.

Figure 5.26: Base plate link moment-rotation evolution in hogging side
Figure 5.27: Base plate link moment-rotation evolution in sagging side

Figure represent behaviour occur in the link element: the diagrams show that moment-rotation curve exceeds the elastic branch but it doesn’t reach the resistance capacity, developing few plastic resources. This procedure was performed for all the link aimed to discover the step activation of the link.

Figure 5.28: Top plate link moment-rotation evolution

The previous diagrams is related to the moment-rotation relation of the links placed at top level, at ultima step is possible see that they don’t reach the plastic field but remain undamaged in elastic field.

5.9. 4 Story building

Design of the fuses used in the four-story building is presented and summary of the data included in the model showed below.
5.9.1. Fuse 1

As first iteration, it was assumed \( \alpha = 0.45 \)

\[
M_{Rd,fuse} = M_{pl,Rd,beam,max} \times 0.45 = 1064 \text{ kNm} \times 0.45 = 478 \text{ kNm}
\]

The level arm \( z \) from the centre of rotation in the middle of the rebar’s and the flange plate is

\[
h_a + h_p + \frac{h_c}{2} = 450\text{mm} + 73\text{mm} + \frac{77}{2}\text{mm} = 561\text{mm}
\]

The bending resistance of the flange plate is consequently:

\[
R_{flange\ plate} = \frac{M_{Rd,fuse}}{z} = \frac{478800 \text{kNm}}{561 \text{mm}} = 1015 \text{kN}
\]

The area of the flange plate of the fuse may then be selected dividing the previous value with the yield stress of the material

\[
A_{flange\ plate} = \frac{R_{flange\ plate}}{f_{yd}} = \frac{1015 \text{kN}}{0.323 \text{kN/mm}^2} = 2860 \text{mm}^2
\]

The following dimensions were adopted for the flange plate: 170x18mm

\( (A_{flange\ plate} = 3060 \text{ mm}^2) \).

To compute the rebar quantity, one could start by computed the area of reinforcement according to the following criteria:

\[
A_{upper\ rebar} = 10 \times A_{flange\ plate} \times \frac{f_{yd}}{f_{sd}} = 10 \times 3060 \text{ mm}^2 \times 0.396 = 12117 \text{ mm}^2
\]

\[
A_{lower\ rebar} = \frac{A_{upper\ rebar}}{2} = 6058 \text{ mm}^2
\]

Free buckling length was performed with the formula (29) previous described:

\[
L_0 = \frac{2\sqrt{2} \times M_p}{A_{fy} \sqrt{\varepsilon}} = \frac{2\sqrt{2} \left(\frac{1}{4} \times 18^3 \times 170\right)}{3060 \times 355 \sqrt{0.002}} = 284 \approx 280 \text{ mm}
\]

5.9.2. Fuse 2

As first iteration, it was assumed \( \alpha = 0.42 \)
\[ M_{Rd, fuse} = M_{pl, Rd, beam, max} \times 0.42 = 1064 \text{ kNm} \times 0.42 = 446.8 \text{ kNm} \]

The level arm \( z \) from the centre of rotation in the middle of the rebar’s and the flange plate is

\[ h_a + h_p + \frac{h_c}{2} = 450 \text{ mm} + 73 \text{ mm} + \frac{77}{2} \text{ mm} = 561 \text{ mm} \]

The bending resistance of the flange plate is consequently:

\[ R_{\text{flange plate}} = \frac{M_{Rd, fuse}}{z} = \frac{466880 \text{ kNm}}{561 \text{ mm}} = 795.87 \text{ kN} \]

The area of the flange plate of the fuse may then be selected dividing the previous value with the yield stress of the material

\[ A_{\text{flange plate}} = \frac{R_{\text{flange plate}}}{f_{yd}} = \frac{795.87 \text{ kN}}{0.323 \text{ kN/mm}^2} = 2464 \text{ mm}^2 \]

The following dimensions were adopted for the flange plate: 170x16mm

\( (A_{\text{flange plate}} = 2720 \text{ mm}^2) \).

To compute the rebar quantity, one could start by computed the area of reinforcement according to the following criteria:

\[ A_{\text{upper rebar}} = 10 A_{\text{flange plate}} \frac{f_{yd}}{f_{sd}} = 10 \times 2720 \text{ mm}^2 \times 0.396 = 10771 \text{ mm}^2 \]

\[ A_{\text{lower rebar}} = \frac{A_{\text{upper rebar}}}{2} = 5385 \text{ mm}^2 \]

Free buckling length was performed whit the formula (29) previous described:

\[ L_0 = \frac{2 \sqrt{2} M_p}{A_f \sqrt{\varepsilon}} = \frac{2 \sqrt{2} \left( \frac{1}{4} \cdot 16^3 \cdot 170 \right)}{2720 \times 355 \sqrt{0.002}} = 252 \approx 250 \text{ mm} \]

5.9.3. Summary of the model parameters
Detailed description of all key parameters involved in the model is presented below referring to plate dimension, rebar number and diameter for composite reinforced beams part and different free buckling length of different fuse.

Table 16: 4 Story Building Effective Length of Fuse

<table>
<thead>
<tr>
<th>Free buckling length</th>
<th>[mm]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Fuse 1</td>
<td>280</td>
</tr>
<tr>
<td>Fuse 2</td>
<td>250</td>
</tr>
</tbody>
</table>

Figure 5.29: 4 Story Building Fuse Layout

Table 17: 4 Story - Dimension of the Fuse members

<table>
<thead>
<tr>
<th>Fuse</th>
<th>Web plate</th>
<th>Flange plate</th>
<th>Upper rebar</th>
<th>Lowe rebar</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Dimension</td>
<td>Area</td>
<td>Dimension</td>
<td>Area</td>
</tr>
<tr>
<td></td>
<td>[mm]</td>
<td>[mm²]</td>
<td>[mm]</td>
<td>[mm²]</td>
</tr>
<tr>
<td>Fuse 1</td>
<td>400*10</td>
<td>4000</td>
<td>170*18</td>
<td>15050</td>
</tr>
<tr>
<td>Fuse 2</td>
<td>400*10</td>
<td>4000</td>
<td>170*16</td>
<td>2820</td>
</tr>
</tbody>
</table>

Table 18: 4 Story - property of the Fuses

<table>
<thead>
<tr>
<th>Fuses</th>
<th>M_pl+</th>
<th>M_pl−</th>
<th>M_rd,fus+</th>
<th>M_rd,fus−</th>
<th>α+</th>
<th>α−</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>[kNm]</td>
<td>[kNm]</td>
<td>[kNm]</td>
<td>[kNm]</td>
<td></td>
<td></td>
</tr>
<tr>
<td>1</td>
<td>1064</td>
<td>661</td>
<td>679</td>
<td>349</td>
<td>0.63</td>
<td>0.52</td>
</tr>
<tr>
<td>2</td>
<td>1064</td>
<td>661</td>
<td>632</td>
<td>292</td>
<td>0.66</td>
<td>0.44</td>
</tr>
</tbody>
</table>
5.9.4. 4 Story building- Pushover curve

Nonlinear static analysis was performed for four story building and result plotted below in terms of base shear-top joint displacement.

Building without fuse show brittle failure after reach maximum base shear. As in the eight story building ductility for building with fuse reach satisfy vale, showed in the hardening part of the curve. Again, frame with uniform acceleration is subject to higher horizontal force than once loaded according to 1st mode patterns.
5.9.5. 4 Story building - Pushover result

Deformed shaper for building with and without fuse is plotted and showed

Deformed shape of building under pushover analysis show the different behaviour of the frame. Four story building with frame equipped by fuse provide optimal collapse mechanism: at 1 meter displacement of top joint no soft story in both analysis arise. Only one plastic column hinge occurs in acceleration mode at third story but, as indicate by blue colour it remains in Occupancy level, so safety side. The frame without fuse clear show soft story mechanism at base column level, that is the most collapse dangerous situation.
5.9.6. Story building- First plastic hinge activation

As for eight story activation of the first plastic hinge was investigated through each link of the most load columns in the model, than the model was compared for each case.

As before the plastic hinge activation occurs in beams instead of in the columns for building whit fuse. Generally, both pushover case present activations of fuse the range between 17 and 24 step instead of building bare frame present activation of soft-story mechanism at base and third story in only 2 cm range for both case.
5.10. 2 Story building

Design of the fuses used in the four-story building is presented and summary of the data included in the model showed below.

5.10.1. Fuse 1

As first iteration it was assumed $\alpha = 0.55$

$$M_{Rd,fuse} = M_{pl,rd,beam,max} \times 0.55 = 1064 \text{kNm} \times 0.55 = 585.2 \text{kNm}$$

The level arm $z$ from the centre of rotation in the middle of the rebar’s and the flange plate is

$$h_d + h_p + \frac{h_c}{2} = 450 \text{mm} + 73 \text{mm} + \frac{77}{2} \text{mm} = 561 \text{mm}$$

The bending resistance of the flange plate is consequently:

$$R_{flange plate} = \frac{M_{Rd,fuse}}{z} = \frac{585200 \text{kNm}}{561 \text{mm}} = 1042 \text{kN}$$

The area of the flange plate of the fuse may then be selected dividing the previous value with the yield stress of the material

$$A_{flange plate} = \frac{R_{flange plate}}{f_{yd}} = \frac{1042 \text{kN}}{0.323 \text{kN/mm}^2} = 2935 \text{mm}^2$$

The following dimensions were adopted for the flange plate: 170x18mm

$$(A_{flange plate} = 3060 \text{ mm}^2).$$

To compute the rebar quantity, one could start by computed the area of reinforcement according to the following criteria:

$$A_{upper rebar} = 10 \ A_{flange plate} \ \frac{f_{yd}}{f_{sd}} = 10 \times 3060 \text{ mm}^2 \times 0.396 = 12117 \text{ mm}^2$$

$$A_{lower rebar} = \frac{A_{upper rebar}}{2} = 6058 \text{ mm}^2$$

Free buckling length was performed with the formula (29) previous described:

$$L_0 = \frac{2 \sqrt{Z} M_p}{A_{fy} \sqrt{\varepsilon}} = \frac{2 \sqrt{2} \left(\frac{1}{4} 18^3 170\right)}{3060 \times 355 \times \sqrt{0.002}} = 284 \cong 280 \text{ mm}$$
5.10.2. Summary of modelled parameters

Detailed description of all key parameters involved in the model is presented below.

Table 19: 2 Story Building Effective Length of Fuse

<table>
<thead>
<tr>
<th>Free length [mm]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Fuse 1 280</td>
</tr>
</tbody>
</table>

Table 20: 2 Story - Dimension of the Fuse members

<table>
<thead>
<tr>
<th>Web plate</th>
<th>Flange plate</th>
<th>Upper rebar</th>
<th>Lowe rebar</th>
</tr>
</thead>
<tbody>
<tr>
<td>Dimension [mm]</td>
<td>Area [mm²]</td>
<td>Dimension [mm]</td>
<td>Area [mm²]</td>
</tr>
<tr>
<td>400*10</td>
<td>4000</td>
<td>170*18</td>
<td>3060</td>
</tr>
</tbody>
</table>

Table 21: 2 Story - property of the Fuses

<table>
<thead>
<tr>
<th>Fuse</th>
<th>M_pl+ [kNm]</th>
<th>M_pl− [kNm]</th>
<th>M_rd,fus_+ [kNm]</th>
<th>M_rd,fus_− [kNm]</th>
<th>α+</th>
<th>α−</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>1066</td>
<td>663</td>
<td>679</td>
<td>349</td>
<td>0,63</td>
<td>0,52</td>
</tr>
</tbody>
</table>

Figure 5.41: 2 Story Building Fuse Layout

Figure 5.42: 2 Story - Constitutive laws of Fuse
5.10.3 2 Story building- Pushover curve

Nonlinear static analysis was performed for two story building and result plotted below ini terms of base shear-top joint displacement.

![Base shear vs. displacement graph](image)

Same consideration done for the eight and four story building are valid for two story building. Considerable displacement at top joint is achieved and high ductility is exploited using fuse. This indicator reflects requirements of the European building code.

5.10.3 2 Story building- Pushover result

Deformed shaper for building with and without fuse is plotted and result reflect the trend observed in eight and four story building.

![Deformed shapes graphs](image)
5.10.4. 2 Story building- First plastic hinge activation

As for eight and fours story activation of the first plastic hinge was investigated through each link of the most load columns in the model, than the model was compared for each case.

Both model without fuse present soft story mechanism at base level whit plasticization of the column. At same top displacement building with fuse is remain undamaged and the column remain in elastic range. Once again is showed the benefit of fuse device to global collapse mechanism.
6. Verification

6.1. Classes of steel section

Eurocode EN 1998 (section 6.1.2 and 7.1.2 for steel and composite structures) requirements depend on the value of selected behaviour factor:

- Class 1 for $4.0 < q$. (For high dissipative structural behaviour)
- Class 2 for $2.0 < q \leq 4$. (For medium dissipative structural behaviour)
- Class 3 for $1.5 < q \leq 2$. (For low dissipative structural behaviour)

Class sections of structural elements is as follows. For composite beams, in according to clause 5.5.1(1) of EN 1994-1-1 and clause 5.6 and table 5.2 of EN 1993-1-1 we have flange and web subjected to compression.

Flange:

$$c \left( \frac{t_f}{t_f} \right) = \frac{0.5b - r - 0.5t_w}{t_f} = \frac{(0.5 \times 170 - 18 - 0.5 \times 8)}{12.7} = 4.96 <$$

$$4.96 < 9 \varepsilon = 9 \frac{\sqrt{335}}{355} = 9$$

Flanges are classified into class 1

Web:

$$c = h_a - 2t_f - 2r = 298.6 \quad \alpha = \frac{(Z_b - t_f - r)}{c} = 0.994$$

Since $\alpha > 0.5$

$$\frac{c}{t_w} = 37,325 > 396 \quad \frac{\varepsilon}{(13\alpha - 1)} = 33.21$$

Web are classified into class 2

Follow that composite beam of steel section is classified into class 2, can be employed for medium dissipative structural behaviour.
6.2. Integrity of the concrete slab

To maintain the integrity of the concrete slab during the seismic event, while yielding takes place in the bottom part of the steel section and/or in the rebar's of the slab, the limit values of \((x/d)\) ratio for ductility of composite beams with slab should satisfy the values given in the clause 6.2 (1.8) and table 7.4 of ENV 1998-1-1. For DCM with \(q=4\), \(f_y = 355 \text{ N/mm}^2\) the \(x/d\) limit is 0.27 where \(d\) is the section's height of composite beam and \((x)\) is the difference between the top of the slab and the position of neutral axis (in case of positive moment and seismic situation). In this case, all the buildings have the same composite beam, since \(R_c < R_a\) and \(R_w < R_c\) the neutral axis is in the upper flange of the beam and result \(x=159\) mm. That ratio satisfy that satisfy the requirement.

6.3. Effective column length

The effective column length (buckling length) is calculated as \(L_{cr} = KL\). Where the buckling coefficient \(K\) is the ratio of the effective column length to the unbraced length \(L\). Values of \(K\) depend on the support conditions of the column to be designed, and the design values of \(K\) for use with idealized conditions of rotation and translation at column supports must be done according to national annex. \(K\) factor was assessed using American method for sideways uninhibited frame.

![Alignment chart for uninhibited frame](image)

*Figure 6.1: Alignment chart for uninhibited frame*
6.3.1. 8 Story Building - base internal columns effective length

For a HEM550 the inertia ray around strong axes is $I_x = 198000$ cm$^3$

$$G_A = \frac{\sum L_c I_c}{\sum \alpha L_b I_b} = \frac{2 \times \frac{198000}{400}}{2 \times 0.625 \times \frac{48200}{400}} = 6.57$$

$G_B = 0$

Assuming $\alpha$ as the average between positive and negative of the fuse in the beams (0,625), $K = 1.6$ from graph for sideway uninhibited.

6.3.2. 4 Story Building - base internal columns effective length

For a HEM340 the inertia ray around strong axes is $I_x = 76370$ cm$^3$

$$G_A = \frac{\sum L_c I_c}{\sum \alpha L_b I_b} = \frac{2 \times \frac{76370}{400}}{2 \times 0.575 \times \frac{48200}{400}} = 2.75$$

$G_B = 0$

Assuming $\alpha$ as the average between positive and negative of the fuse in the beams (0,57) $K = 1.35$ from graph for sideway uninhibited.

6.3.3. 2 Story Building - base internal columns effective length

For a HEM320 the inertia ray around strong axes is $I_x = 68130$ cm$^3$

$$G_A = \frac{\sum L_c I_c}{\sum \alpha L_b I_b} = \frac{340}{138} = 2.45$$

$G_B = 0$

Assuming $\alpha$ as the average between positive and negative of the fuse in the beams (0,57) $K = 1.3$ from graph for sideway uninhibited.

6.4. Buckling at internal base columns

Internal columns should be checked under buckling action. For all the building since have the same plant, influence area of 64 m$^2$ was considered.

The combination 1.35G+1.5Q = 701 kN/storey.
6.4.1. 8 Story Building buckling check

Considering 8 story the compression at the base level is 5262 kN. For a story high 4000 mm the effective buckling length is L*K=6400mm. Considering HEM550 the inertia ray around the weaker axes is $i_y = 73,5$ mm $\tilde{\lambda} = 78$. $\lambda_e = 76,10$ so the relative slenderness is:

$$\frac{\lambda}{\lambda_e} = 1,03$$

Considering the ratio h/b=1,86>1,2 and tf=40mm<100mm the instability should be check around y-y using buckling curve a and around z-z using curve b. Using a correction coefficient $\chi=0,64$ the check is satisfied:

$$N_{b,rd} = 10681 \text{ kN} > N_{b,Ed} = 5262 \text{ kN}$$

6.4.2. 4 Story Building buckling check

Considering 4 story the compression at the base level is 1428 kN. For a story high 4000 mm the effective buckling length is L*K=5400Mm. Considering HEM340 the inertia ray around the weaker axes is 79,0 mm $\lambda = 67$. $\lambda_e = 76,10$ so the relative slenderness is:

$$\frac{\lambda}{\lambda_e} = 0,72$$
Considering the ratio $h/b=1.22>1.2$ and $f=40\text{mm}<100\text{mm}$ the instability should be check around y-y using buckling curve a and around z-z using curve b. Using a correction coefficient $\chi=0.83$ the check is satisfied:

$$N_{b,rd} = 1652 \text{ kN} > N_{b,Ed} = 1428 \text{ kN}$$

### 6.4.3. 2 Story Building buckling check

Considering 2 consecutive story the compression at the base level is 1128 kN. For a story high 4000 mm the effective buckling length is $L*K=5200\text{mm}$. Considering HEM320 the inertia ray around the weaker axes is $79.5 \text{ mm } \lambda_e = 76.10$ so the relative slenderness is:

$$\frac{\lambda}{\lambda_e} = 0.85$$

Considering the ratio $h/b=1.09<1.2$ and $f=40\text{mm}<100\text{mm}$ the instability should be check around y-y using buckling curve b and around z-z using curve c. using a correction coefficient $\chi=0.72$ the check is satisfied:

$$N_{b,rd} = 7974 \text{ kN} > N_{b,Ed} = 1128 \text{ kN}$$

### 6.5. Modal analysis

Response-modal analysis of one plane frame to evaluate the earthquake action effects was performed and the result are summarized in the table for each of the case study. The first second and third mode activate more than 90% of the mass. The table below provides the real design values as well as EN 1998 values of design spectrum and the corresponding period values using $q=4$.

![Figure 6.3: Design spectrum considered in the modal analysis](image)
Consideration on the modal information of all the structure allow to compare the first period. Structures with fuse have periods always higher than respective once without fuse due to the presence of weak device.

6.5.1. 8 Story building – modal information

The following tables summarize most important modal information for eight story building without fuse in terms of periods mass participating and cumulative mass

*Table 22: 8 story whit fuse - Fundamental vibration mode and associated mass*

<table>
<thead>
<tr>
<th>StepType</th>
<th>StepNum</th>
<th>Period Sec</th>
<th>Mass part UX %</th>
<th>Sum Mass part UX %</th>
</tr>
</thead>
<tbody>
<tr>
<td>Mode</td>
<td>1</td>
<td>2.06</td>
<td>80</td>
<td>80</td>
</tr>
<tr>
<td>Mode</td>
<td>2</td>
<td>0.65</td>
<td>8</td>
<td>88</td>
</tr>
<tr>
<td>Mode</td>
<td>3</td>
<td>0.36</td>
<td>5</td>
<td>93</td>
</tr>
</tbody>
</table>

*Figure 6.4: 8 Story - Deformed shape in vibration mode 1 2 and 3 respectively*

*Table 23: 8 story without fuse Fundamental vibration mode and associated mass*

<table>
<thead>
<tr>
<th>StepType</th>
<th>StepNum</th>
<th>Period Sec</th>
<th>Mass part UX %</th>
<th>Sum Mass part UX %</th>
</tr>
</thead>
<tbody>
<tr>
<td>Mode</td>
<td>1</td>
<td>1.47</td>
<td>79</td>
<td>79</td>
</tr>
<tr>
<td>Mode</td>
<td>2</td>
<td>0.49</td>
<td>10</td>
<td>89</td>
</tr>
<tr>
<td>Mode</td>
<td>3</td>
<td>0.28</td>
<td>4</td>
<td>94</td>
</tr>
</tbody>
</table>
6.5.2. 4 Story building – modal information

The following tables summarize most important modal information for four story building without fuse in terms of periods mass participating and cumulative mass.

*Table 24: 4 story whit fuse - Fundamental vibration mode and associated mass*

<table>
<thead>
<tr>
<th>StepType</th>
<th>StepNum</th>
<th>Period</th>
<th>Mass part UX</th>
<th>Sum Mass part UX</th>
</tr>
</thead>
<tbody>
<tr>
<td>Mode</td>
<td>1</td>
<td>0,96</td>
<td>0,81</td>
<td>81</td>
</tr>
<tr>
<td>Mode</td>
<td>2</td>
<td>0,33</td>
<td>0,13</td>
<td>94</td>
</tr>
<tr>
<td>Mode</td>
<td>3</td>
<td>0,18</td>
<td>0,05</td>
<td>99</td>
</tr>
</tbody>
</table>

*Figure 6.5: 8 Story - Deformed shape in vibration mode 1 2 and 3 respectively*

*Table 25: 4 story without fuse - Fundamental vibration mode and associated mass*

<table>
<thead>
<tr>
<th>StepType</th>
<th>StepNum</th>
<th>Period</th>
<th>Mass part UX</th>
<th>Sum Mass part UX</th>
</tr>
</thead>
<tbody>
<tr>
<td>Mode</td>
<td>1</td>
<td>0,89</td>
<td>84</td>
<td>84</td>
</tr>
<tr>
<td>Mode</td>
<td>2</td>
<td>0,30</td>
<td>11</td>
<td>95</td>
</tr>
<tr>
<td>Mode</td>
<td>3</td>
<td>0,18</td>
<td>3</td>
<td>98</td>
</tr>
</tbody>
</table>
6.5.3. 2 Story Building – modal information

The following tables summarize most important modal information for two story building without fuse in terms of periods mass participating and cumulative mass.

*Table 26: 2 story with fuse - Fundamental vibration mode and associated mass*

<table>
<thead>
<tr>
<th>StepType</th>
<th>StepNum</th>
<th>Period Sec</th>
<th>Mass part UX %</th>
<th>Sum Mass part UX %</th>
</tr>
</thead>
<tbody>
<tr>
<td>Mode</td>
<td>1</td>
<td>0.68</td>
<td>91</td>
<td>91</td>
</tr>
<tr>
<td>Mode</td>
<td>2</td>
<td>0.23</td>
<td>8</td>
<td>99</td>
</tr>
</tbody>
</table>

*Figure 6.6: 2 Story - Deformed shape in vibration mode 1 2 and 3 respectively*

*Table 27: 2 story without fuse - Fundamental vibration mode and associated mass*

<table>
<thead>
<tr>
<th>StepType</th>
<th>StepNum</th>
<th>Period Sec</th>
<th>Mass part UX %</th>
<th>Sum Mass part UX %</th>
</tr>
</thead>
<tbody>
<tr>
<td>Mode</td>
<td>1</td>
<td>0.46</td>
<td>91</td>
<td>91</td>
</tr>
<tr>
<td>Mode</td>
<td>2</td>
<td>0.15</td>
<td>9</td>
<td>100</td>
</tr>
</tbody>
</table>
6.6. Damage limitation – Verification from NTC 394 Circular C7341

We must verify at this stage whether the damage limitations of non-structural elements are satisfied according to clause 4.4.3.2.(c) of EN 1998-1:

\[ d_r \nu \leq 0.075 h \text{ where } d_r = qd_s \]

Where \( \nu \) is the reduction factor, considering the lower return period of the seismic action associated with the damage limitation requirement, \( \nu = 0.5 \) for a building of an importance class II. And the other parameters are defined previously. The values, shown in table below, show that the precedent inequality is well satisfied and the inter-storey drifts are limited.

6.6.1. 8 Story Building – Inter-story drift

Table 28: 8 Story building – Inter-storey drifts

<table>
<thead>
<tr>
<th>STORY</th>
<th>RS 1</th>
<th>EQ 1</th>
<th>SLS 1</th>
<th>SLS2</th>
<th>SLS3</th>
<th>ULS 1</th>
<th>ULS2</th>
<th>max</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.0050</td>
<td>0.0057</td>
<td>0.0002</td>
<td>0.0002</td>
<td>0.0002</td>
<td>0.0003</td>
<td>0.0003</td>
<td>0.0057</td>
</tr>
<tr>
<td>2</td>
<td>0.0093</td>
<td>0.0105</td>
<td>0.0005</td>
<td>0.0005</td>
<td>0.0004</td>
<td>0.0008</td>
<td>0.0006</td>
<td>0.0108</td>
</tr>
<tr>
<td>3</td>
<td>0.0105</td>
<td>0.0125</td>
<td>0.0006</td>
<td>0.0005</td>
<td>0.0005</td>
<td>0.0009</td>
<td>0.0007</td>
<td>0.0125</td>
</tr>
<tr>
<td>4</td>
<td>0.0101</td>
<td>0.0122</td>
<td>0.0007</td>
<td>0.0006</td>
<td>0.0005</td>
<td>0.0010</td>
<td>0.0008</td>
<td>0.0122</td>
</tr>
<tr>
<td>5</td>
<td>0.0093</td>
<td>0.0113</td>
<td>0.0007</td>
<td>0.0006</td>
<td>0.0005</td>
<td>0.0010</td>
<td>0.0008</td>
<td>0.0113</td>
</tr>
<tr>
<td>6</td>
<td>0.0074</td>
<td>0.0092</td>
<td>0.0007</td>
<td>0.0006</td>
<td>0.0005</td>
<td>0.0009</td>
<td>0.0008</td>
<td>0.0092</td>
</tr>
<tr>
<td>7</td>
<td>0.0061</td>
<td>0.0074</td>
<td>0.0010</td>
<td>0.0008</td>
<td>0.0007</td>
<td>0.0013</td>
<td>0.0011</td>
<td>0.0074</td>
</tr>
<tr>
<td>8</td>
<td>0.0041</td>
<td>0.0047</td>
<td>0.0011</td>
<td>0.0010</td>
<td>0.0009</td>
<td>0.0016</td>
<td>0.0014</td>
<td>0.0047</td>
</tr>
<tr>
<td>ds</td>
<td>0.0105</td>
<td>0.0125</td>
<td>0.0011</td>
<td>0.0010</td>
<td>0.0009</td>
<td>0.0016</td>
<td>0.0014</td>
<td>0.0105</td>
</tr>
<tr>
<td>( d_r = ds \times q )</td>
<td>0.0419</td>
<td>0.0498</td>
<td>0.0046</td>
<td>0.0040</td>
<td>0.0037</td>
<td>0.0063</td>
<td>0.0055</td>
<td></td>
</tr>
<tr>
<td>( d_r \times \nu \text{ [mm]} )</td>
<td>20,9740</td>
<td>24,9200</td>
<td>2,30</td>
<td>2,01</td>
<td>1,83</td>
<td>3,17</td>
<td>2,75</td>
<td></td>
</tr>
<tr>
<td>( 0.0075h\text{[mm]} )</td>
<td>30</td>
<td>30</td>
<td>30</td>
<td>30</td>
<td>30</td>
<td>30</td>
<td>30</td>
<td>30</td>
</tr>
</tbody>
</table>

This 8 story fuse configuration satisfy check 4.4.3.2. (c) of EN 1998-1 in all seismic design combination. The best fuse configuration is presented. Different Fuse was applied for different the story, satisfy global collapse mechanism and inter-story drift check.
6.6.2. 4 Story Building - Inter-storey drifts

Table 29: 4 Story building – Inter-storey drifts

<table>
<thead>
<tr>
<th>STORY</th>
<th>RS 1</th>
<th>EQ 1</th>
<th>SLS 1</th>
<th>SLS2</th>
<th>SLS3</th>
<th>ULS 1</th>
<th>ULS2</th>
<th>max</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.007</td>
<td>0.008</td>
<td>0.0003</td>
<td>0.0002</td>
<td>0.0004</td>
<td>0.0003</td>
<td></td>
<td>0.0078</td>
</tr>
<tr>
<td>2</td>
<td>0.009</td>
<td>0.011</td>
<td>0.0006</td>
<td>0.0005</td>
<td>0.0004</td>
<td>0.0008</td>
<td>0.0007</td>
<td>0.0113</td>
</tr>
<tr>
<td>3</td>
<td>0.0081</td>
<td>0.0099</td>
<td>0.0007</td>
<td>0.0006</td>
<td>0.0005</td>
<td>0.0009</td>
<td>0.0008</td>
<td>0.0099</td>
</tr>
<tr>
<td>4</td>
<td>0.0048</td>
<td>0.0060</td>
<td>0.0006</td>
<td>0.0005</td>
<td>0.0005</td>
<td>0.0008</td>
<td>0.0007</td>
<td>0.0060</td>
</tr>
<tr>
<td>ds</td>
<td>0.0081</td>
<td>0.0099</td>
<td>0.0007</td>
<td>0.0006</td>
<td>0.0005</td>
<td>0.0009</td>
<td>0.0008</td>
<td></td>
</tr>
<tr>
<td>dr=ds*q</td>
<td>0.0322</td>
<td>0.0396</td>
<td>0.0026</td>
<td>0.0022</td>
<td>0.0020</td>
<td>0.0037</td>
<td>0.0031</td>
<td></td>
</tr>
<tr>
<td>dr * v [mm]</td>
<td>16.12</td>
<td>19.80</td>
<td>1.31</td>
<td>1.11</td>
<td>0.98</td>
<td>1.83</td>
<td>1.54</td>
<td></td>
</tr>
<tr>
<td>0.0075h[mm]</td>
<td>30</td>
<td>30</td>
<td>30</td>
<td>30</td>
<td>30</td>
<td>30</td>
<td>30</td>
<td></td>
</tr>
</tbody>
</table>

6.6.3. 2 Story Building – inter-story drifts

Table 30: 2 Story building – Inter-storey drifts

<table>
<thead>
<tr>
<th>STORY</th>
<th>RS 1</th>
<th>EQ 1</th>
<th>SLS 1</th>
<th>SLS2</th>
<th>SLS3</th>
<th>ULS 1</th>
<th>ULS2</th>
<th>max</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>0.0064</td>
<td>0.0073</td>
<td>0.0004</td>
<td>0.0004</td>
<td>0.0003</td>
<td>0.0006</td>
<td>0.0005</td>
<td>0.0136</td>
</tr>
<tr>
<td>2</td>
<td>0.0072</td>
<td>0.0080</td>
<td>0.0008</td>
<td>0.0007</td>
<td>0.0007</td>
<td>0.0012</td>
<td>0.0010</td>
<td>0.0137</td>
</tr>
<tr>
<td>ds</td>
<td>0.0072</td>
<td>0.0080</td>
<td>0.0008</td>
<td>0.0007</td>
<td>0.0007</td>
<td>0.0012</td>
<td>0.0010</td>
<td></td>
</tr>
<tr>
<td>dr=ds*q</td>
<td>0.0286</td>
<td>0.0321</td>
<td>0.0033</td>
<td>0.0029</td>
<td>0.0027</td>
<td>0.0046</td>
<td>0.0040</td>
<td></td>
</tr>
<tr>
<td>dr * v [mm]</td>
<td>14.31</td>
<td>16.05</td>
<td>1.67</td>
<td>1.47</td>
<td>1.33</td>
<td>2.30</td>
<td>2.00</td>
<td></td>
</tr>
<tr>
<td>0.0075h[mm]</td>
<td>30</td>
<td>30</td>
<td>30</td>
<td>30</td>
<td>30</td>
<td>30</td>
<td>30</td>
<td></td>
</tr>
</tbody>
</table>

As for eight story, also four and two story satisfy criteria imposed by Eurocode. Configuration that provide higher inter story drift are the those maximize horizontal action. Result are presented in terms of millimetre.
6.7. II° effect analysis

Based on the Eurocode 8-1, the value of inter-storey drift sensitivity coefficient (\( \theta \)) is calculated according to the following expression according to clause 4.4.2.2(2)

\[
\theta = \frac{P_{tot} \cdot d_r}{V_{tot} \cdot h} \leq 0.1
\]

Where \( P_{tot} \) is the total gravity load at and above the storey considered in the seismic design situation, \( V_{tot} \) is the total seismic storey shear and \( h \) is the inter-storey height. The EN 1998-1 states that \( d_r \) is the real relative displacement, i.e. inelastic displacement, evaluated as the difference of average lateral displacements (\( d_s \)) at top and bottom of the storey under consideration and calculated by multiplying the elastic displacement (\( d_e \)), induced by a linear analysis based on design seismic action, by the displacement behaviour factor (\( q \)) (clause 4.3.4)

\[
d_r = d_s(i+1) - d_s(i)
\]

If (0.1 \( \leq \theta \leq 0.2 \)), the second-order effects may approximately be taken into account by multiplying the relevant seismic action effects by a factor equal to \( 1/(1 - \theta) \) according to clause 4.4.2.2(3) of EN 1998-1, and the structural design can be done by a linear elastic analysis.

If (0.2 \( \leq \theta \leq 0.3 \)), the structure is designed according to a plastic non-linear analysis (Pushover analysis). The value of the coefficient \( \theta \) shall not exceed 0.3 according to clause 4.4.2.2(4) of EN 1998-1.

6.7.1. 8 Story Building – drift sensitivity coefficient

<table>
<thead>
<tr>
<th>Floor</th>
<th>disp(Rspec) [m]</th>
<th>drift,real [m]</th>
<th>dr/h [-]</th>
<th>Vtot [kN]</th>
<th>Ptot [kN]</th>
<th>( \theta )</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0,0056</td>
<td>0,022</td>
<td>0,006</td>
<td>-321</td>
<td>7978</td>
<td>0,14</td>
</tr>
<tr>
<td>2</td>
<td>0,0108</td>
<td>0,043</td>
<td>0,011</td>
<td>-313</td>
<td>7068</td>
<td>0,24</td>
</tr>
<tr>
<td>3</td>
<td>0,0125</td>
<td>0,050</td>
<td>0,013</td>
<td>-297</td>
<td>6158</td>
<td>0,26</td>
</tr>
<tr>
<td>4</td>
<td>0,0123</td>
<td>0,049</td>
<td>0,012</td>
<td>-272</td>
<td>5250</td>
<td>0,24</td>
</tr>
<tr>
<td>5</td>
<td>0,0114</td>
<td>0,046</td>
<td>0,011</td>
<td>-240</td>
<td>4341</td>
<td>0,21</td>
</tr>
<tr>
<td>6</td>
<td>0,0092</td>
<td>0,037</td>
<td>0,009</td>
<td>-164</td>
<td>2650</td>
<td>0,15</td>
</tr>
<tr>
<td>7</td>
<td>0,0071</td>
<td>0,028</td>
<td>0,007</td>
<td>-115</td>
<td>1740</td>
<td>0,11</td>
</tr>
<tr>
<td>8</td>
<td>0,0043</td>
<td>0,017</td>
<td>0,004</td>
<td>-59</td>
<td>833</td>
<td>0,06</td>
</tr>
</tbody>
</table>
6.7.2. 4 Story Building – drift sensitivity coefficient

*Table 32: 4 Story building - drift sensitivity coefficient*

<table>
<thead>
<tr>
<th>Floor</th>
<th>disp(Rspec) [m]</th>
<th>drift,real [m]</th>
<th>dr/h [-]</th>
<th>Vtot [kN]</th>
<th>Ptot [kN]</th>
<th>θ</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.0078</td>
<td>0.031</td>
<td>0.008</td>
<td>-271.00</td>
<td>3003</td>
<td>0.09</td>
</tr>
<tr>
<td>2</td>
<td>0.0113</td>
<td>0.045</td>
<td>0.011</td>
<td>-235.00</td>
<td>2103</td>
<td>0.10</td>
</tr>
<tr>
<td>3</td>
<td>0.0099</td>
<td>0.040</td>
<td>0.010</td>
<td>-164.00</td>
<td>1202</td>
<td>0.07</td>
</tr>
<tr>
<td>4</td>
<td>0.0060</td>
<td>0.024</td>
<td>0.006</td>
<td>-89.00</td>
<td>577</td>
<td>0.04</td>
</tr>
</tbody>
</table>

6.7.3. 2 Story Building – drift sensitivity coefficient

*Table 33: 2 Story proposed building - drift sensitivity coefficient*

<table>
<thead>
<tr>
<th>Floor</th>
<th>disp(Rspec) [m]</th>
<th>drift,real [m]</th>
<th>dr/h [-]</th>
<th>Vtot [kN]</th>
<th>Ptot [kN]</th>
<th>θ</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.0073</td>
<td>0.029</td>
<td>0.007</td>
<td>-249.00</td>
<td>1737</td>
<td>0.05</td>
</tr>
<tr>
<td>2</td>
<td>0.0080</td>
<td>0.032</td>
<td>0.008</td>
<td>-160.00</td>
<td>832</td>
<td>0.04</td>
</tr>
</tbody>
</table>

6.8. Weak Beam-Strong Column check

The plastic resistance of columns subjected to combined bending and axial compression are known, and in accordance with the value of behaviour factor, it is important to ensure that the actual ruin of the structure will be based on the occurrence of a global plastic mechanism (and not on a local mechanism in one or two levels). This is clearly indicated, for steel and composite structures, by Eurocode 8. At each node of the structure, the strong-column, weak-beam condition shall be satisfied by applying the following inequality according to clause 4.4.2.3 of [8]. The Weak Beam-Strong Column check is:

\[ \sum M_{Rc} \geq 1.3 \sum M_{Rb} \]

At interior nodes, there are 2 beams and 2 columns intersecting, so the WBSC check becomes:

\[ \sum W_{pl,columns} \geq 1.3 \sum W_{pl,beams} \]

At exterior nodes, there is 1 beam and 2 columns intersecting so the WBSC check becomes:

\[ 2 \sum W_{pl,columns} \geq 1.3 \sum W_{pl,beams} \]
### 6.8.1. 8 Story building - WBSC

*Table 34: 8 Story building – WBSC check*

<table>
<thead>
<tr>
<th>Floor 1-2 - internal joint</th>
<th>HEM550</th>
<th>( M_{pt,rd , colun} )</th>
<th>2815 kNm</th>
<th>Verif.</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>( M_{pt,rd , fuse} )</td>
<td>718 kNm</td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Floor 1-2 - external joint</th>
<th>HEB550</th>
<th>( M_{pt,rd , colun} )</th>
<th>1984 kNm</th>
<th>Verif.</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>( M_{pt,rd , fuse} )</td>
<td>718 kNm</td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Floor 3-4 - internal joint</th>
<th>HEM500</th>
<th>( M_{pt,rd , colun} )</th>
<th>2518 kNm</th>
<th>Verif.</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>( M_{pt,rd , fuse} )</td>
<td>718 kNm</td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Floor 3-4 - external joint</th>
<th>HEB500</th>
<th>( M_{pt,rd , colun} )</th>
<th>1709 kNm</th>
<th>Verif.</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>( M_{pt,rd , fuse} )</td>
<td>718 kNm</td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Floor 5-6 - internal joint</th>
<th>HEM450</th>
<th>( M_{pt,rd , colun} )</th>
<th>2247 kNm</th>
<th>Verif.</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>( M_{pt,rd , fuse} )</td>
<td>679 kNm</td>
<td></td>
</tr>
<tr>
<td>Floor 5-6 - external joint</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>--------------------------</td>
<td>---</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>HEB450</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>$M_{pl,rd\ column}$</td>
<td>1413 kNm</td>
<td>Verif.</td>
<td></td>
<td></td>
</tr>
<tr>
<td>$M_{pl,rd\ fuse}$</td>
<td>679 kNm</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Floor 7-8 - internal joint</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>HEM300</td>
<td></td>
</tr>
<tr>
<td>$M_{pl,rd\ column}$</td>
<td>1447 kNm</td>
</tr>
<tr>
<td>$M_{pl,rd\ fuse}$</td>
<td>679 kNm</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Floor 7-8 - external joint</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>HEB300</td>
<td></td>
</tr>
<tr>
<td>$M_{pl,rd\ column}$</td>
<td>663 kNm</td>
</tr>
<tr>
<td>$M_{pl,rd\ fuse}$</td>
<td>679 kNm</td>
</tr>
</tbody>
</table>

The checks are satisfy for all the floor in 8 Story building configurations. The WBSC check is waived at the top level of multi-storey buildings in according to EC8 but result satisfy. Global and local ductility condition according to clause 4.4.2.3. of EC8 are verified to prevent formation of soft story plastic mechanism.
### 6.8.2. 4 Story building – WBSC

*Table 35: 4 Story building – WBSC check*

<table>
<thead>
<tr>
<th>Floor 1-2 - internal joint</th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>HEM340</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>$M_{pl,rd \text{ column}}$</td>
<td>1674 kNm</td>
<td>Verif.</td>
</tr>
<tr>
<td>$M_{pl,rd \text{ fuse}}$</td>
<td>718 kNm</td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Floor 1-2 - external joint</th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>HEB340</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>$M_{pl,rd \text{ column}}$</td>
<td>854 kNm</td>
<td>Verif.</td>
</tr>
<tr>
<td>$M_{pl,rd \text{ fuse}}$</td>
<td>718 kNm</td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Floor 3-4 - internal joint</th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>HEM300</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>$M_{pl,rd \text{ column}}$</td>
<td>1447 kNm</td>
<td>Verif.</td>
</tr>
<tr>
<td>$M_{pl,rd \text{ fuse}}$</td>
<td>663 kNm</td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Floor 3-4 - external joint</th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>HEB280</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>$M_{pl,rd \text{ column}}$</td>
<td>544 kNm</td>
<td>Verif.</td>
</tr>
<tr>
<td>$M_{pl,rd \text{ fuse}}$</td>
<td>633 kNm</td>
<td></td>
</tr>
</tbody>
</table>

Four story building satisfy condition of WBSC check ensuring that structural elements and the structure possess adequate ductility. No further iterations are necessary for this frame.
### 6.8.3. 2 Story building - WBSC

*Table 36: 2 Story building – WBSC check*

<table>
<thead>
<tr>
<th></th>
<th>Floor 1-2 - internal joint</th>
<th></th>
<th>Floor 1-2 - external joint</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>HEM320</td>
<td>$M_{pl,rd , colun}$</td>
<td>1574 kNm</td>
<td>$M_{pl,rd , fuse}$</td>
</tr>
<tr>
<td></td>
<td>$M_{pl,rd , fuse}$</td>
<td>679 kNm</td>
<td></td>
</tr>
<tr>
<td>HEB320</td>
<td>$M_{pl,rd , colun}$</td>
<td>762 kNm</td>
<td></td>
</tr>
<tr>
<td></td>
<td>$M_{pl,rd , fuse}$</td>
<td>679 kNm</td>
<td></td>
</tr>
</tbody>
</table>

Two story building satisfy condition of WBSC check ensuring that structural elements and the structure possess adequate ductility. No further iterations are necessary for this frame.
7. Behaviour factor

In the context of modern earthquake resistant design the behaviour factor also called response modification factor plays an important role. This factor modifies the design elastic response spectrum to the nonlinear elastic one. Behaviour factor generally compute by the product of ductility dependent factor and over-strength dependent factor as follows:

\[ q = \mu_T \cdot \Omega \]

Where according to 4.3.3.4.2.4 EC8:

- \( \mu_T = \frac{\alpha_e}{\alpha_u} \) is the period-based ductility dependent factor
- \( \Omega = \frac{\alpha_u}{\alpha_1} \) is the overstrength dependent factor
- \( \mu_T = \frac{d_m}{d_y} \) is the period-based ductility dependent factor
- \( \Omega = \frac{F_y}{F_1} \) is the overstrength dependent factor

According to 6.3.2.(3) of

- \( \alpha_u \) is the value by which the horizontal seismic design action is multiplied, in order to form plastic hinges in a number of sections sufficient for the development of overall structural instability, while all other design actions remain constant. The factor \( \alpha_u \) may be obtained from a nonlinear static (pushover) global analysis.
- \( \alpha_1 \) is the value by which the horizontal seismic design action is multiplied in order to first reach the plastic resistance in any member in the structure, while all other design actions remain constant.

There are different ways to calculate the ductility factor, some of them take into account the first global plastic mechanism while others may be calculated as the first plastic mechanism which may occur at any member of the structure. According to EuroCode 8, two load pattern cases may apply to the structure, the behaviour factor is then the minimum of all calculations. \( F_1 \) defines as the first global mechanism, \( F_y \) is the maximum base shear obtains by the pushover analysis and \( d_m \) is the corresponding displacement to the maximum base shear. \( dy \) is the bilinear idealized curve which can be calculates as follows:

\[ F_i = m_i \phi_i \]

Where \( F_i \) is the lateral forces, \( \phi_i \) is the normalized displacements and \( m_i \) is the mass in the i-th storey. Displacements are normalized in such a way that \( \Phi n = 1 \), where \( n \) is the control node (usually, \( n \) denotes the roof level), meaning, \( F_n = m_n \). The mass of an equivalent SDOF system \( m^* \) is determined as:
\[ m^* = \sum m_i \phi_i = \sum \bar{F}_i \]

and the transformation factor is given by:

\[ \Gamma = \frac{m^*}{\sum m_i \phi_i^2} \]

The force \( F^* \) and displacement \( d^* \) of the equivalent SDOF system are computed as:

\[ F^* = \frac{F_b}{\Gamma} \]

\[ d^* = \frac{d_n}{\Gamma} \]

Where \( F_b \) and \( d_n \) are, respectively, the base shear force and the control node displacement of the Multi Degree of Freedom (MDOF) system. The yield force \( F_y^* \), which represents also the ultimate strength of the idealized system, is equal to the base shear force at the formation of the plastic mechanism. The initial stiffness of the idealized system is determined in such a way that the areas under the actual and the idealized force – deformation curves are equal (see Figure 8.1). Based on this assumption, the yield displacement of the idealised SDOF system \( d_{y^*} \) is

\[ d_{y^*} = 2 \left( d_{m^*} - \frac{E_{m^*}^*}{F_{m^*}^*} \right) \]

where \( E_{m^*}^* \) is the actual deformation energy up to the formation of the plastic.

![Figure 7.1 Determination of the idealized elastic - perfectly plastic force – displacement relationship.](image)

126
7.1. Method 1

In this method $F_1$ defines as the first global plastic mechanism.

7.2. Method 2

In this method $F_1$ defines as the intersection between the first global plastic mechanism and bilinear curve.
7.3. Method 3

In this method, $F_1$ defines as the first local plastic mechanism such a way that, the first plastic mechanism at any member of the structure.

7.4. Method 4 and 5

In this method, $dm^*$, is defined as 95% of strength loss of the structure. $F_1$ defines as the first global plastic mechanism. The different between method 4 and 5 is:

Meth 4: $\Omega = \frac{f_{ymax}}{f_1}$

Meth 5: $\Omega = 0.95 \frac{f_{ymax}}{f_1}$
7.5. Method 6 and 7

In this method, dm*, is defined as 95% of strength loss of the structure. F_1 defines as the intersection between the first global plastic mechanism and bilinear curve. The different between method 6 and 7 is:

\[
\text{Meth 6: } \Omega = \frac{f_{\text{ymax}}}{f_1} \quad \text{Meth 7: } \Omega = 0.95 \times \frac{f_{\text{ymax}}}{f_1}
\]

7.6. Method 8 and 9

In this method, dm*, is defined as 95% of strength loss of the structure. F_1 defines as the first local plastic mechanism such a way that, the first plastic mechanism at any member of the structure. The different between method 8 and 9 is:

\[
\text{Meth 8: } \Omega = \frac{f_{\text{ymax}}}{f_1} \quad \text{Meth 9: } \Omega = 0.95 \times \frac{f_{\text{ymax}}}{f_1}
\]
7.7. Method 10

In this method $\alpha u^*$ is defined at the level of the maximum base shear, $ae^*$ is the elastic base shear corresponding to the maximum plastic displacement obtained at the maximum base shear and $\alpha_1^*$ is the base shear in which the global plastic mechanism takes place.

\[ \begin{align*}
\alpha e^* & \quad \alpha u^* \quad \alpha_1^* \\
\text{dy}^* & \quad \text{dm}^*
\end{align*} \]

7.8. Method 11

In this method $\alpha u^*$ is defined at the level of the maximum base shear, $ae^*$ is the elastic base shear corresponding to the maximum plastic displacement obtained at the maximum base shear and $\alpha_1^*$ is the base shear in which the local plastic mechanism at any member of the structure takes place.

\[ \begin{align*}
\alpha e^* & \quad \alpha u^* \quad \alpha_1^* \\
\text{dm}^*
\end{align*} \]
7.9. Method 12

Maximum displacement in FEMA is where 80\% loss of strength occur instead of the maximum one in other methods. FEMA uses design base shear which is calculated as following simplified formula rather than F1.

\[ \text{Vmax} \]

\[ \text{Ve} \]

\[ \text{Vd} \]

d.des dmax

7.10. Method 13 and 14

In Italian code dm* is defined where 85\% loss of strength achieved. F1* is the base shear corresponding to global plastic mechanism and Fy* is the maximum base shear. The different between method 13 and 14 is:

Meth 13: \( \Omega = \frac{f_{y_{\text{max}}}}{f_t} \)

Meth 14: \( \Omega = 0.95 \times \frac{f_{y_{\text{max}}}}{f_t} \)

\[ \text{0.85Fy*} \]

\[ \text{F1*} \]

\[ \text{d1*} \]

dy* dm*(0.85Fy*)
7.11. Method 15 and 16

dm* is defined where 85% loss of strength achieved. F1 defines as the intersection between the first global plastic mechanism and bilinear curve and Fy* is the maximum base shear. The different between method 16 and 17 is:

Meth 16: \( \Omega = \frac{f_{y}^{\max}}{f_1} \)  
Meth 17: \( \Omega = 0.95 \times \frac{f_{y}^{\max}}{f_1} \)

7.12. Method 17 and 18

dm* is defined where 85% loss of strength achieved. F1* is the base shear corresponding to local plastic mechanism and Fy* is the maximum base shear. The different between method 17 and 18 is:

Meth 17: \( \Omega = \frac{f_{y}^{\max}}{f_1} \)  
Meth 18: \( \Omega = 0.95 \times \frac{f_{y}^{\max}}{f_1} \)
8. Conclusion

Present thesis is part of INNOSEIS European project aimed to valorisation of innovative anti-seismic device. POLIMI is a beneficiary partner of this project, tasked to study whit IST number three different archetype. Those building will be equipped in the two orthogonal direction whit INERD and FUSEIS device. BRF and MRF characterize the frame in two direction studied. Design of those innovative seismic device was performed and dissipative behaviour of the system was investigated through a nonlinear static analysis. Series of mandatory check ensure serviceability and ultima safety in building assigned, according to European building construction rules.

First chapter summarize the data collected from different previous project research putting in result most of self-made anti-seismic device and problem found during specific device implementation. INERD was treated more in detail whit experimental description and result example of different test. Description of coupled moment resisting system conclude fist chapter.

Second part is dedicate exclusively on FUSEIS bolted cover plate device and data recorded on it. The introduction to specific studied device was done whit indication of governing parameters. Comparison between cyclic and monotonic behaviour was performed in order to explain and predict failure model. Fundamental argument from engineering point of view as energy dissipation and nonlinear behaviour were treated. Investigation performed by IST and POLIMI in laboratory ere included whit test specimen, loading sequence and cyclic loading. Follow numerical modelling of both experimental test to reproduce and compare numerical result obtained.

Design rules were implemented, in order to give at designer practice and direct tools making design of innovative ant seismic device more detailed and quickly. Initially the composite beam was treated in order to figure out resisting moment of composite section under hogging and sagging. Indication and suggestion for fuse design help reader in the first iteration. Design of free buckling length was described supported by theory and calibration on the result of test done. Anchorage length was implemented in design rule for design the concrete gap up to device location, in order to accommodate the minimum rotation required by the code whiteout crack of concrete. Conclude the chapter check for bolted plate and summary of check list.

In order to design ant seismic device for every building archetype whit size element different from once tested, validation of the fuse characterization was needed. Starting from theory of plasticization and bending-axial compressed columns procedure was described. Moment-rotation constitutive law of different plate tested was reproduced whit analytically procedure. Modelling of experimental frame in FEM was performed and each plate tested. Results of model were compared to experimental test record.
Consideration in terms of Moment-Rotation and global frame Force-Displacement conclude the chapter.

Modelling of new archetype of building, reporting size, material, load, and boundary condition used are included in chapter number five. Lumped plasticity modelling of fuse behaviour was treated with description of the plastic hinge in the model. Pushover analysis was performed in order to control soft-story mechanism formation and ensure a global collapse mechanism. Each archetype was described including design of each fuse implemented and relative calculation, fuse moment-rotation constitutive law and element size. Pushover result between building with fuse and whiteout fuse conclude each case study, including investigation of link activation step in the most loaded column.

Check in accordance to most relevant clause of Eurocode 8 and Eurocode 4 are included in the Chapter number 7. Classification of composite steel section characterize dissipative structural behaviour. Integrity of concrete slab was checked in order to ensure safety during moment rotation. Effective column length was assessed for each base internal column in according to American method and axial buckling effect checked. Inter-story drift, drift sensitivity coefficient and capacity design are the main part of the chapter according to of EN 1998-1. Pushover curve comparison between different acceleration for each structure conclude the chapter.

Numerical result carried out applying the fuses show that periods of the structures with fuses always rise due to of the presence of the weak devices, consequently top displacement increase allowing to exploit more ductility and avoiding soft story mechanism. The base shear is always lower in the structures with fuses and numerical result confirms the beneficial effect of the application of the devices. Limitation in fuses moment resistance has to be considered to satisfy Eurocode verification. Different iteration was performed in order to reach optimal configuration of columns size and fuse device.

Increasing strength of fuse increase too possibility of soft-story mechanism and formation of plastic hinge in columns but permit designer to reduce size of columns, during this operation the hierarchy of resistance and buckling effect in the base column are the most severe check. Decreasing resistance of the fuse increase ability to exploit structure more ductility but inter-story drift and sensitivity drift ratio are the most severe parameters to be accounted. Important consideration come the columns of the MRFs, applying the fuses to the frames only the plastic hinges at the ground are activated, and the soft storey mechanism is prevented. looking at the moment rotation diagrams of the fuses, it appears that the fuses are more stressed in the hogging zone because activation is always in the side of the column where go under negative moment. Behaviour factor for the structure with innovative device result be generally higher than the one indicate by the Eurocode this class of building, reflecting the high ductility showed in pushover curve.
Annex A : Load combination

Combination of action are set of design values used for the verification of the structural reliability for a limit state under the simultaneous influence of different actions. The load combination of gravity and seismic loads was considered according to EN 1990 6.4.3.4 (section 2.5.6). EN 1998-1 applies to the design of buildings and civil engineering works in seismic regions and is subdivided into 10 sections; section 3 gives the rules for the representation of seismic actions and for their combination with other actions. Seismic combination was considered according to section 3. For the selected design situations and the relevant limit states the individual actions for the critical load cases should be combined with following factor:

- $\psi_0$ = Factor for combination value of a variable action
- $\psi_1$ = Factor for frequent value of a variable action
- $\psi_2$ = Factor for quasi-permanent value of a variable action

Annex A1 of EN 1990:2002 gives rules and methods for establishing combinations of actions for buildings: for Office area the value in table A1.1 are the following

$\psi_0 = 0,7 ; \psi_1 = 0,5 ; \psi_2 = 0,3$

Check of structure was performed according to:

1. Characteristic combination used for irreversible limit states

$$\sum G_{K,i} + P + \psi_0 Q_{K,1} + \sum \psi_0 Q_{K,i}$$

2. Frequent combination used for reversible limit state

$$\sum G_{K,i} + P + \psi_1 Q_{K,1} + \sum \psi_2 Q_{K,i}$$

3. Quasi-permanent combination used for reversible limit state

$$\sum G_{K,i} + P + \sum \psi_2 Q_{K,i}$$

Combination nomenclature:

- ULS1 = fundamental (Live Dominant): $1,3G + 1,5L$
- ULS1 = fundamental (Superimposed dominant): $1,3G + 1,05L$
- SLS1 = characteristic (Superimposed dominant): $1G + 0,7L$
- SLS2 = frequent (Live dominant): $1G + 0,5L$
- SLS3 = frequent (Superimposed dominant): $1G + 0,3L$
- RS1 = Response spectrum $1G + 0,3L + 1RS$
- EQ1 = Static horizontal force $1G + 0,3L + 1Ex$
Annex B: Behaviour factor of different building

8 Story behaviour factor result – Uniform acceleration mode

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8 Story behaviour factor value – 1st Mode acceleration

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References


REHABILITATION OF BUILDINGS. Washington, D.C.: FEDERAL


of beam-to-column moment connection with hysteretic dampers for column weak


[20] P. H. A Braconi, A Osta, A Dall’Asta, G Leoni, S Möller, B Hoffmeister, S. A
Karamanos, G Varelis, E Alderighi, C Coscetti, W Salvatore, J Gracia, E Bayo, R
Mallardo, L Bianco, P Filipuzzi, D Vasilikis, P Tsintzos, S Estanislau, J Lobo, L
Fülöp, Prefabricated steel structures for low-rise buildings in seismic areas

Gündel, S. A Karmanos, G Varelis, Renata Obiala, P Tsintzos, D Vasilikis, J. B
Lobo, P Bartlam, S. C Estanislau, L Nardini, F Morelli, W Salvatore, D Dubina,
A Dogariu, S Bordea, G Bor, Steel solutions for seismic retrofit and upgrade of
existing constructions (Steelretro). European Commission, 2010.


[23] F. Mazzolani, Moment resistant connections of steel frames in seismic areas:
design and reliability. 2000.


response of dissipative devices for seismic resistant steel frames: Experimental
behaviour and numerical simulation,” Moment, 2011.

[28] I. Vayas et al, “Dissipative devices for seismic resistant steel frames,” in
Eurosteel, 2011.

(FUSEIS1) to seismic loading,” in Eurosteel, 2011.


C. A. C. and L. Calado, “Seismic Behaviour Of Steel Braced Frames With Dissipative Connections,” in the 4th International Conference on Advances in Steel Structures (ICASS’05), Shanghai.


E. Commission, BS En 15129, Anti-seismic devices. British Standards.


F. Mazzolani and V. Piluso, Theory and design of seismic resistant steel frames. 1996.


8, no. 31, 1993.


[59] and H. K. Nassar, Aladdin Aly, seismic demands for SDOF and MDOF systems. John A. Blume Earthquake Engineering Center, Department of Civil Engineering, Stanford University, USA, 1991.

