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INNOSEIS

Valorization of innovative anti-seismic devices

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CONTENTS

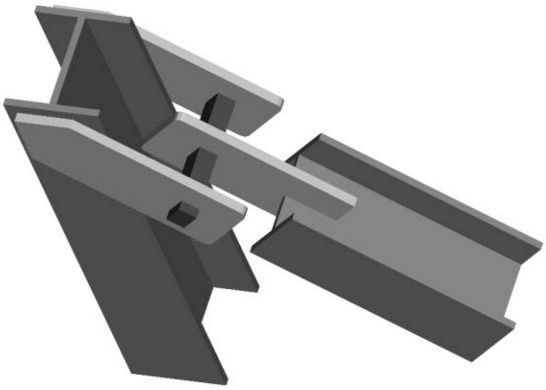
1 INTRODUCTION

This Volume presents relevant design guidelines for 9 innovative anti-seicmic devices which include supplementary clauses of EN 1998-1 in its current version, May 2004. Reference is made to the clauses of this version. Figures, tables and equation numbering is indicative. The systems under discussion are dissipative connections, dissipative links, dissipative beam splices, replaceable shear links and shear panels and modified braces.

2 INERD PIN CONNECTIONS

2.1 ADDITIONS TO PAR. 6.3.1 STRUCTURAL TYPES

(1) Frames with concentric braces and dissipative connections are those in which the connections of braces to adjacent members are dissipative and partial strength compared to the brace so that energy can be dissipated in the connections, while the braces and other parts are protected from buckling and yielding. The connection medium is a pin that runs through two external plates connected to the frame columns/beams, and one or two internal plates connected to the brace (Fig. 6.1). Pin connections may be placed at one or both ends of the diagonals.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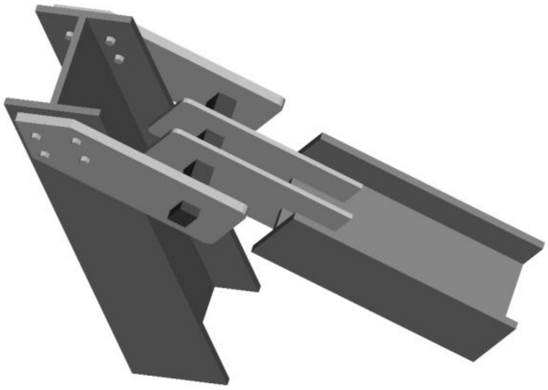
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Fig. 6.1: Possible configurations of the dissipative pin connection

2.2 ADDITIONS TO PAR. 6.3.2, TABLE 6.2 BEHAVIOR FACTORS

Table 6.2: Upper limit of reference values of behavior factors for systems regular in elevation

STRUCTURAL TYPE	Ductility class	
	DCM	DCH
INERD pin connections		
At both diagonal’s ends	3.0	4.0
At one diagonal’s end	2.0	3.0

2.3 ADDITIONS TO 6.5.3, DESIGN RULES FOR DISSIPATIVE ELEMENTS IN COMPRESSION OR BENDING

(3) In order to ensure that the dissipative pins will be loaded primarily in bending, their length shall be such that

$$a \geq h \qquad \text{Eq. (6.1)}$$

where h the height of the pin
a the clear distance between the internal and external plates

2.4 ADDITIONS TO 6.5.5, DESIGN RULES FOR DISSIPATIVE ELEMENTS IN COMPRESSION OR BENDING

(8) The resistance R_d of welds or bolts of the dissipative pin connection must satisfy the criterion:

$$R_d \geq 1.1 \cdot \gamma_{ov} \cdot P_{u,Rd} \qquad \text{Eq. (6.2)}$$

where $P_{u,Rd}$ is the ultimate resistance of the pin connection under consideration
γ_{ov} = 1.25 is the recommended overstrength factor

For bolted connections, High Strength Friction Bolts should be used (Categories B, C or E according to EN1993-1-8).

2.5 ADDITIONS TO PAR. 6.7 DESIGN AND DETAILING RULES FOR FRAMES WITH CONCENTRIC BRACINGS

6.7.1 Design criteria

(4)P Concentric braced frames with dissipative pin connections shall be designed in such a way that yielding of the pins in bending will take place before buckling of the braces or yielding of the adjacent members and parts.

6.7.2 Analysis

(2)P - in frames with dissipative pin connections both the tension and compression diagonals shall be taken into account. The pin connection may be modelled as an axial spring with spring constant:

- For one internal plate:

$$K_{pin} = \frac{32 \cdot EI}{\ell^3} \qquad \text{Eq. (6.3)}$$

- For two internal plates:

$$K_{pin} = \frac{8 \cdot EI}{a \cdot \ell^2 \cdot \alpha \cdot (3 - 4 \cdot \alpha)} \qquad \text{Eq. (6.4)}$$

where EI the bending stiffness of the pin
ℓ the axial distance of the external plates
α = a / ℓ

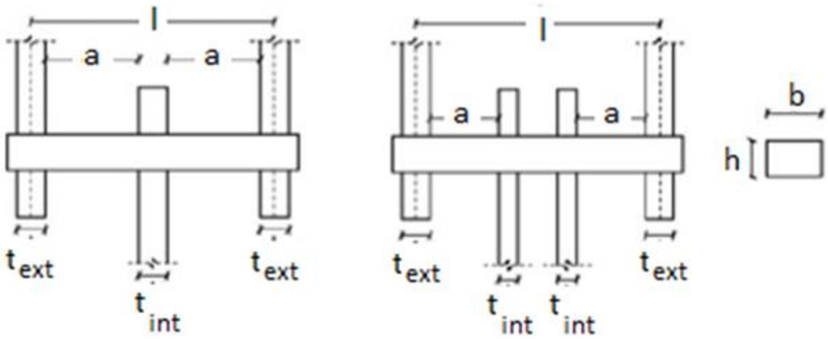


Fig. 6.2: Geometric properties of dissipative pin connections

6.7.3 Diagonal members

(10) Dissipative pins are designed for the highest brace forces in the seismic design situations according to:

$$P_{Ed} \leq P_{u,Rd} \qquad \text{Eq. (6.5)}$$

where P_{Ed} the design axial force of the brace and the connection
$P_{u,Rd}$ the ultimate resistance of the connection

The resistance of the connection due to bending and shear of the pin are defined in eq. (6.6a) and (6.6b) respectively. The factor β_{III} defines the percentage of the pin that has undergone significant plastic deformation on each side, with $0 \leq \beta_{III} \leq 0.5$. The ultimate resistance of the connection is found through an iterative process by changing factor β_{III}, so that the two values of equations (6.6a) and (6.6b) become equal.

$$P_{u,M,Rd} = k_{pin} \cdot \frac{4 \cdot M_u}{a_{red,III} \cdot \gamma_{pu}} \qquad \text{Eq. (6.6a)}$$

$$P_{u,V,Rd} = k_{pin} \cdot \frac{2 \cdot b \cdot \left(1 - 2 \cdot \beta_{III}\right) \cdot h \cdot f_y}{\sqrt{3} \cdot \gamma_{pu}} \qquad \text{Eq. (6.6b)}$$

where $\mathrm{M_u} = \mathrm{W_{u,pl}} \cdot \mathrm{f_{mid}}$ the ultimate plastic resistance of the pin

$f_{mid} = f_y + \left(f_u - f_y\right) \cdot \lambda_f / 2$ the maximum normal stress of the pin

$\lambda_f = \left(\frac{a-h}{2 \cdot h}\right)^2$ a factor for the influence of shear with $0 \le \lambda_f \le 1$

$W_{u,pl} = b \cdot h^2 \cdot \left[\beta_{III} - \beta_{III}^2 + \chi \cdot \left(0.5 - \beta_{III}\right)^2\right]$ the plastic modulus of the pin, taking into account the reduction due to the shear stresses.

$\chi = \sqrt{1 - \left(f_y / f_{mid}\right)^2}$

(11) Overstrength of a pin *i* is defined by the expression:

$$\Omega_i = \frac{P_{u,Rd,i}}{P_{Ed,i}} \qquad \text{Eq. (6.7)}$$

The selection of pin's dimensions shall be such that the value of Ω_i is close to 1.
In order to achieve a homogeneous global dissipative behaviour of the structure, it should be checked that the maximum overstrength ratio Ω_{max} over the entire structure does not differ from the minimum value Ω_{min} by more than 25%:

$$\frac{\Omega_{max}}{\Omega_{min}} \le 1.25 \qquad \text{Eq. (6.8)}$$

(12) Diagonal members shall be verified to yielding and buckling assuming the exhaustion of the capacity of the pins at their ends:

$$N_{Ed} = \Omega_{\max} \cdot P_{u,Rd} \qquad \text{Eq. (6.9)}$$

where Ω_{max} is the maximum value of all the pinned connections of the diagonals

6.7.4 Beams and columns

Beams and columns connected to braces with flexible INERD connections should meet the following minimum resistance requirement:

$$N_{pl,Rd}\left(M_{Ed}\right) \ge N_{Ed,G} + 1.1 \cdot \gamma_{ov} \cdot \Omega \cdot N_{Ed,E} \qquad \text{Eq. (6.10)}$$

where $N_{pl,Rd}\,(M_{Ed})$ is the axial design resistance of the frame member according to EN1993, taking into account the interaction with the bending moment M_{Ed}
$N_{Ed,G}$ is the axial force of the frame member due to non-seismic actions of the seismic combinations

$N_{Ed,E}$ is the axial force of the frame member due to the seismic action of the seismic combinations
Ω_{min} is the minimum value of all the pinned connections of the diagonals

This requirement can be expressed in the following clause:

(4) Beams and columns connected to braces with dissipative pin connections can be verified according to 6.7.4 (1), where Ω is the the minimum value of all the pinned connections of the diagonals.

The total magnification factor ($1.1 \cdot \gamma_{ov} \cdot \Omega$) cannot exceed the value of the behaviour factor *q* used in analysis.
The actual maximum yield strength stress of the pin steel should be as close as possible to its nominal value in order to achieve an economic design. This can be achieved by if the steel of the pin complies with 6.2 (3)a or 6.2 (3)c.

6.7.5 Pin modelling for nonlinear static (pushover) analyses

The dissipative pin connection may be represented by a nonlinear axial spring at the diagonal's end with properties illustrated in Figure 3. Characteristic points that define the axial spring properties are given in Table 1.

Point	P	δ_{pl}
A	0	0
B	P_{yd}	0
C	P_{ud}	$0.5 \cdot h$
D	P_{ud}	a
E	$0.5 \cdot P_{ud}$	a
F	$0.5 \cdot P_{ud}$	$1.5 \cdot a$
Acceptance criteria (δ_{pl})		
IO	$0.25 \cdot h$	
LS	$0.6 \cdot h$	
CP	$0.8 \cdot a$	

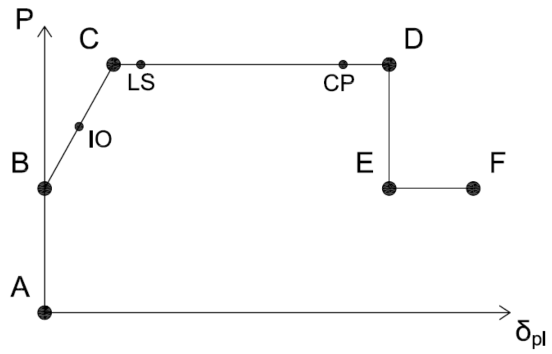


Fig. 6.3: Nonlinear properties of the dissipative pin connection spring and performance levels

6.7.6 Pin modelling for nonlinear dynamic analyses

(1) The static nonlinear law of dissipative pins described in 6.7.5 can be expanded in order to exhibit an adequate hysteretic behaviour. A typical hysteretic law is shown in Fig. 6.4 where it can be seen that special attention is needed to model the pinching that is observed during cyclic loading.

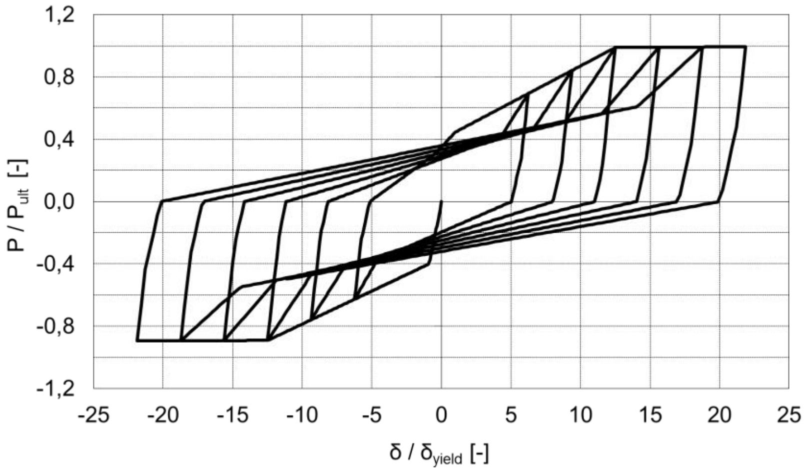


Fig. 6.4: Hysteretic behaviour of the dissipative pin connection nonlinear spring

(2) When performing nonlinear dynamic analysis, pin damage due to low-cycle fatigue must be examined. The following damage curve should be considered for dissipative pins:

$$\log N = 6 - 3 \cdot \log S \qquad \text{Eq. (6.11)}$$

The damage index can be determined from the stress history of the pin connection according to Annex A of EN1993-1-9.

3 INERD U CONNECTIONS

3.1 ADDITIONS TO PAR. 6.3.1 STRUCTURAL TYPES

(2) The U-Connection is a suitable solution for concentric braced frames (Fig. 3.1). The U-connection consists of one or two bent, U-shaped thick plates (Fig. 3.2) that connects the brace to the adjacent member. The connection of the bracing with the U-Device can be parallel or perpendicular (Fig. 3.3).

(3) The U-Connection is designed as dissipative connection.

(4) The U-Connection is suitable for structures not too sensitive to large displacements. In the case multi-storey buildings, maximum of 6 storeys.

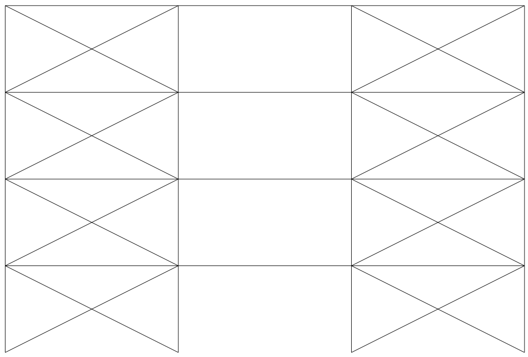


Fig. 3.1: Type of frame for implementation U-connection: concentric braced frames

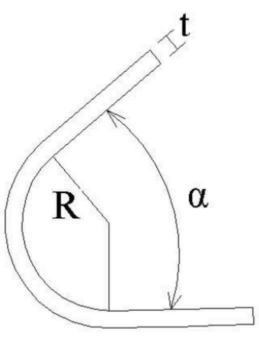
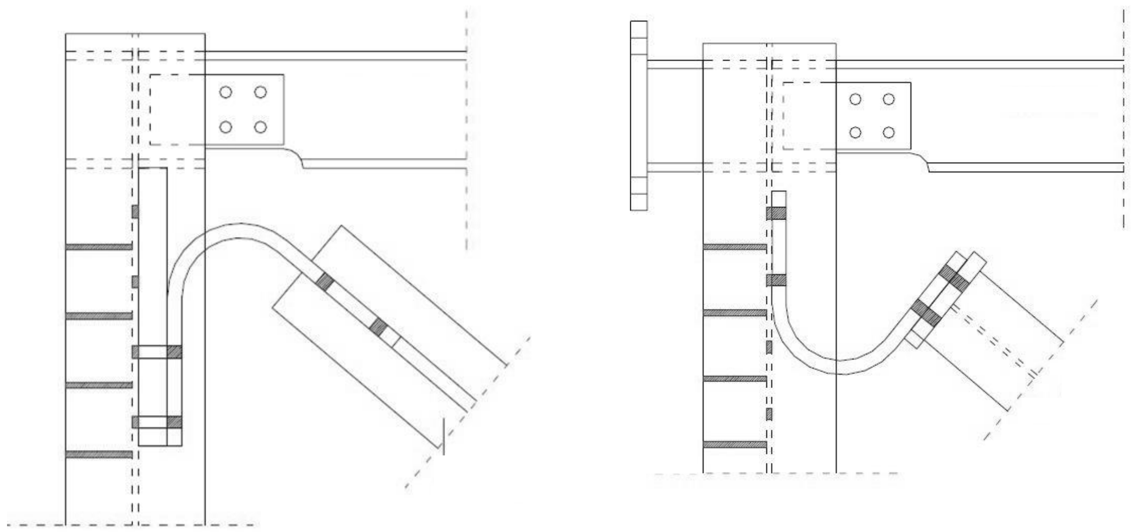


Fig. 3.2: U-Device



a) Load parallel to U-Connection b) Load perpendicular to U-Connection

Fig. 3.3: Bracing to U-Device connection typology

3.2 ADDITIONS TO PAR. 6.3.2, TABLE 6.2 BEHAVIOR FACTORS

Table 6.2: Upper limit of reference values of behavior factors for systems regular in elevation

U-Connection	3.0

3.3 ADDITIONS TO PAR. 6.12 (NEW) DESIGN AND DETAILING RULES FOR FRAMES WITH U-CONNECTIONS

6.12.1 Analysis

The U-Connections may be simulated as follows:

- By means of beam elements. The number of elements has to be sufficient to reproduce the curvature of the device. The connection between the U-Connection elements and the structure members (columns and bracings) is rigid (continuous).
- By means of equivalent spring. In the structural model the connection between the members (columns and bracings) is performed using a spring element. The spring element behaviour approximates the behaviour of the U-Connection.

Beam-to-column and column bases are modelled as pinned.

6.12.2 U-Connections

U-Connections shall be verified for the design axial load on the bracings:

$$\frac{N_{Ed}}{N_{U,Rd}} \leq \qquad \text{Eq. (3.1)}$$

where:
N_{Ed} is the design axial force on the bracing
$N_{U,Rd}$ is the design resistance of the U-Connection.

Overstrength of a U-Connection is defined by the expression:

$$\Omega = \frac{N_{pl,U,Rd}}{N_{Ed}} \qquad \text{Eq. (3.2)}$$

The selection of U-Connection dimensions shall be such that the value of Ω is close to 1.
To achieve a global dissipative behavior of the frame, it should be checked that the maximum ratios Ω over the entire structure do not differ from the minimum value Ω by more than 25%.

$$\frac{max\Omega}{min\Omega} \leq 1.25 \qquad \text{Eq. (3.3)}$$

6.12.3 Columns and bracings in braced frames using U-Connections

The columns and bracings connected through the U-Connection device shall be verified to resist the capacity design action effects as following:

System’s columns connected to pin links, and receptacle beams shall be verified to resist the capacity design action effects as following:

$$N_{Col,Ed} = N_{Ed,G} + 1{,}1 \cdot \gamma_{Ov} \cdot \Omega \cdot N_{Ed,E} \qquad \text{Eq. (3.4)}$$

$$N_{Brac,Ed} = 1{,}1 \cdot \gamma_{Ov} \cdot \Omega \cdot N_{Ed,E} \qquad \text{Eq. (3.5)}$$

where:
$N_{Ed,G}$ are the axial forces due to the non-seismic actions included in the combination of actions for the seismic design situation,
$N_{Ed,E}$ are the axial forces due to the design seismic action,
$\Omega = min\Omega_i = min\left\{\frac{M_{pl,pin,Rd,i}}{M_{Ed,i}}\right\}$ $\Omega = min\Omega_i = min\left\{\frac{N_{U,Rd,i}}{N_{Ed,i}}\right\}$ is the minimum overstrength factor for all U-Connections in the building, see Eq. (1.2), and γ_{ov} =1.25 is the material overstrength factor.
The total magnification factor of seismic forces cannot exceed the value of the behavior factor q used in analysis. The real yield stress of the steel should be as close as possible to its nominal value in order to achieve an economic design.

6.12.4 U-Connection modelling for non-linear static (pushover) analysis

The structural model used for a non-linear static pushover analysis shall include the response of the structural elements and connections beyond the elastic state. Depending on the model used for the U-Connection, the following should be used:

- Beam element: material model should consist in a elasto-perfectly-plastic or elasto-plastic with strain hardening constitutive law;
- Spring element: the spring element should be non-linear and the behavior should reproduce the post-elastic behavior of the device. An approximation of the real behavior can be done through a multi-linear law.

6.12.5 U-Connection modelling for non - linear dynamic analysis

The structural model used for a non-linear dynamic analysis shall include the response of the structural elements and connections beyond the elastic state and under cyclic loading. Depending on the model used for the U-Connection, the following should be used:

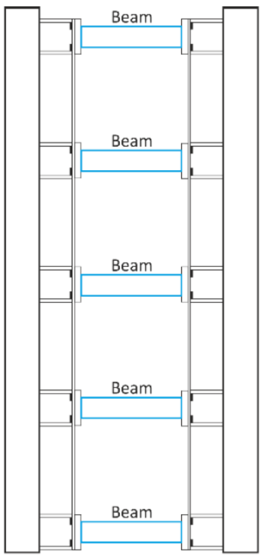
- Beam element: material model should consist in a cyclic law (with kinematic hardening);
- Spring element: the spring element should be non-linear and the behavior should reproduce the hysteric behavior of the device.

4 FUSEIS BEAM LINKS

4.1 ADDITIONS TO PAR. 6.3.1 STRUCTURAL TYPES

h) FUSEIS beam link systems are composed of two closely spaced strong columns rigidly interconnected by multiple beams. The beams run from column to column and can be of different cross section types, as for example RHS, SHS, CHS or I-shaped sections. The FUSEIS beam link system resists lateral loads as a vertical Vierendeel beam and acts as the seismic load resisting system in a frame (Fig. 1.1)

(6) The horizontal beams in the FUSEIS beam link system are the primary dissipative zones where the energy dissipation capability is mainly located. Reduced beam sections (RBS) are recommended to clearly define the dissipative zones (Fig. 1.2). Reduced beam sections (RBS) are to be designed according to EN 1998-3. Joints between floor beams and columns may be pinned or semi-rigid. Semi-rigid joints are preferred to obtain an almost self-centring system with less residual displacements. Columns may be fixed or pinned.



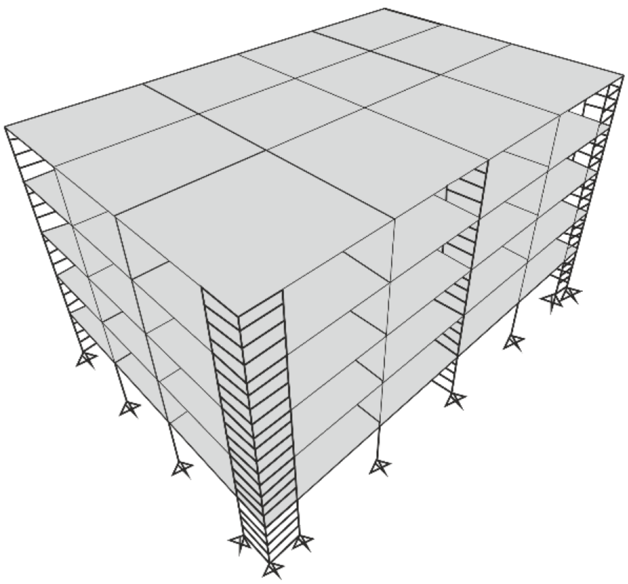


Fig. 4.1: FUSEIS beam link system (left) and several systems located in a building (right)

Fig. 4.2: Dissipative zones in FUSEIS beam links by using reduced beam sections (RBS)

4.2 ADDITIONS TO PAR. 6.3.2, TABLE 6.2 BEHAVIOR FACTORS

Table 6.2: Upper limit of reference values of behavior factors for systems regular in elevation

STRUCTURAL TYPE	Ductility class	
	DCM	DCH
h) FUSEIS beam links	3	5

4.3 ADDITIONS TO 6.5.3, DESIGN RULES FOR DISSIPATIVE ELEMENTS IN COMPRESSION OR BENDING

(3) In the case of FUSEIS beam link systems, in order to avoid significant interaction between shear and moment action, the following Equation should be fulfilled:

$$l_{RBS} > \frac{2 \cdot M_{pl,RBS,Rd}}{V_{b,pl,Rd}} = \frac{4 \cdot W_{pl,RBS}}{A_v/\sqrt{3}} \qquad \text{Eq. (6.1)}$$

Where:

l_{RBS} = axial distance between reduced beam sections (RBS)

$M_{pl,RBS,Rd} = W_{pl,RBS} \cdot f_y$ is the design moment resistance of the reduced beam section (RBS), whereby $W_{pl,RBS}$ is the corresponding plastic section modulus and f_y is the yield strength

$V_{b,pl,Rd}$ = design shear resistance of beam section

A_v = shear area of beam section

4.4 ADDITIONS TO PAR. 6.12 (NEW) DESIGN AND DETAILING RULES FOR FRAMES WITH FUSEIS BEAM LINKS

6.12.1 Analysis

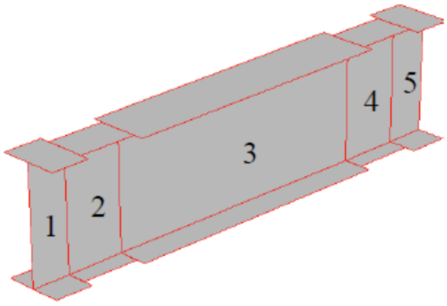


Fig. 4.3: Numerical modelling of FUSEIS beam link

FUSEIS beam link systems shall be represented by appropriate beam-column FE-elements. The net beam length shall be subdivided into 5 zones as shown in Fig. 1.3. These zones shall represent the full sections and the reduced beam sections (RBS). Beam-to-column joints as well as column bases shall be represented as rigid, semi-rigid or hinged in accordance to the connection detailing. Rigid zones shall be provided from column centers to column faces to exclude non-existent beam flexibilities.

6.12.2 Dissipative element verification

The dissipative elements of the system, i.e. beam links, shall be verified to resist the internal forces and moments as determined from structural analysis. Beam links shall be verified assuming a formation of a plastic hinge at the reduced beam section (RBS).

(1) The moment capacity at the reduced beam section (RBS) shall be verified as following:

$$\frac{M_{Ed}}{M_{pl,RBS,Rd}} \leq 1.0 \qquad \text{Eq. (6.2)}$$

where:

M_{Ed} = design bending moment

$M_{pl,RBS,Rd}$ = plastic, resistance design moment of reduced beam section (RBS)

(2) The shear resistance shall be verified in accordance to:

$$\frac{V_{CD,Ed}}{V_{b,pl,Rd}} \leq 1.0 \qquad \text{Eq. (6.3)}$$

Where:

$$V_{CD,Ed} = \frac{2 \cdot M_{pl,RBS,Rd}}{l_{RBS}} \qquad \text{Eq. (6.4)}$$

$V_{CD,Ed}$ = capacity design shear force

$V_{b,pl,Rd}$ = design shear resistance of beam section

If 6.5.3 (3) is fulfilled, Eq 6.3 is automatically guaranteed.

(3) The beam end moment resistance shall be verified in accordance with:

$$\frac{M_{CD,Ed}}{M_{b,pl,Rd}} \leq 1.0 \qquad \text{Eq. (6.5)}$$

Where:

$M_{CD,Ed} = \frac{l_b}{l_{RBS}} \cdot M_{pl,RBS,Rd}$ = capacity design bending moment, whereby:

l_b = net beam length

l_{RBS} = axial distance of reduced beam sections (RBS)

$M_{b,pl,Rd}$ = design bending moment of reduced beam section (RBS)

(4) Lateral torsional buckling verifications for the FUSEIS beam links are generally not necessary due to their small length.

6.12.3 FUSEIS beam link system strong columns verification

(1) The FUSEIS columns shall be verified to resist the capacity design action effects as following:

$$N_{CD,ED} = N_{Ed,G} + 1.1 \cdot \gamma_{ov} \cdot \Omega \cdot N_{Ed,E} \qquad \text{Eq. (6.6)}$$

$$M_{CD,ED} = M_{Ed,G} + 1.1 \cdot \gamma_{ov} \cdot \Omega \cdot M_{Ed,E} \qquad \text{Eq. (6.7)}$$

$$V_{CD,ED} = V_{Ed,G} + 1.1 \cdot \gamma_{ov} \cdot \Omega \cdot V_{Ed,E} \qquad \text{Eq. (6.8)}$$

Where:

$N_{Ed,G}, V_{Ed,G}, M_{Ed,G}$ = axial forces, shear forces and bending moments respectively in columns due to the non-seismic actions included in the combination of actions for the seismic design situation

$N_{Ed,E}, V_{Ed,E}, M_{Ed,E}$ = axial forces, shear forces and bending moments in columns due to the design seismic action

$\Omega = \min \Omega_i = min\{M_{pl,RBS,Rd,i}/M_{Ed,i}\}$ = minimum value of the relevant ratios for all FUSEIS beam links in one building direction

6.12.4 Connection verifications

Connections between FUSEIS beam links and columns shall be verified with the following capacity design actions:

(1) If reduced beam sections are used the capacity bending moment shall be derived as following:

$$M_{CD,con,Ed} = max\{M_1, M_2\} \quad \text{Eq. (6.9)}$$

Where

$$M_1 = 1.1 \cdot \gamma_{ov} \cdot \frac{l_b}{l_{RBS}} \cdot M_{pl,RBS,Rd} \quad \text{Eq. (6.10)}$$

$$M_2 = 1.1 \cdot \gamma_{ov} \cdot M_{u,b} \quad \text{Eq. (6.11)}$$

Where

$$M_{u,b} = W_{pl,b} \cdot f_u \quad \text{Eq. (6.12)}$$

$\gamma_{ov} = f_{y,act}/f_y$ if the actual yield strength of the beam is known or if not $\gamma_{ov} = 1.25$

l_b = net beam length

l_{RBS} = axial distance of reduced beam sections (RBS)

$f_{y,act}$ = actual yield strength of the beam

f_u = ultimate strength of the beam

$W_{pl,b}$ = plastic moment of the beam section at beam end

The design shear of the connection may be calculated from:

$$\cdot \gamma_{ov} \cdot \frac{2 \cdot M_{pl,RBS,Rd}}{l_{RBS}} \quad \text{Eq. (6.13)}$$

(2) If reduced beam sections (RBS) are not used and alternatively the connection region is strengthened by means of additional plates (Fig. 1.4), the strengthened area and the connection shall have a capacity design moment equal to:

$$M_{CD,con,Ed} = \frac{l_b}{l_{net}} \cdot M_{u,b} \quad \text{Eq. (6.14)}$$

Where

l_b = net beam length

l_{net} = net un-strengthened beam length

$M_{u,b} = W_{pl,b} \cdot f_u$

The design shear of the connection may be calculated from:

$$V_{con,CD} = \frac{2 \cdot M_{CD,con,Ed}}{l_b} \quad \text{Eq. (6.15)}$$

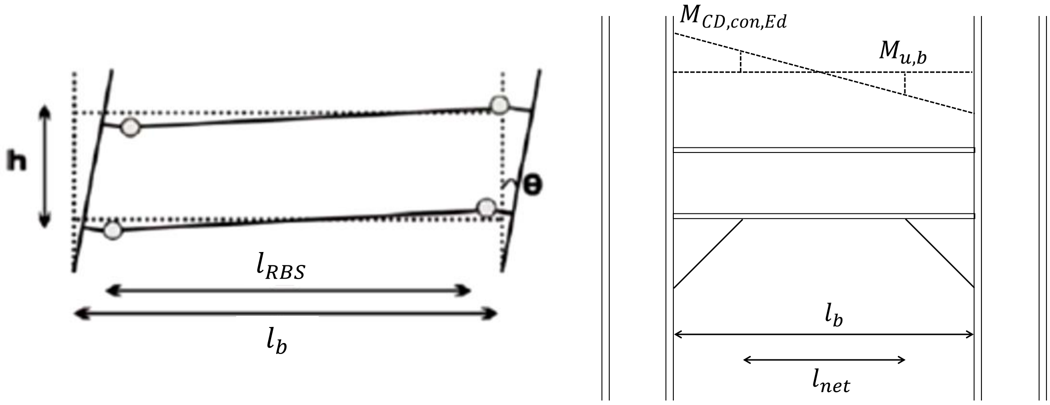


Fig. 4.4: Plastic hinges with reduced beam sections (RBS) and end strengthening of the beam

6.12.5 Plastic hinge modelling for non-linear static (pushover) analyses

For the dissipative elements, which are the reduced beam sections (RBS) of the FUSEIS beam link system, the nonlinear hinge properties of Fig. 1.5 according to a multi-linear plastic kinematic model may be used.

HINGE PROPERTIES (α_{pl}=shape factor)						
	IPE		SHS		CHS	
Point	M/M_{pl}	Φ/Φ_{pl}	M/M_{pl}	Φ/Φ_{pl}	M/M_{pl}	Φ/Φ_{pl}
A	0	0	0	0	0	0
±B	±1	0	±0.6	0	±1	0
±C	$\pm\alpha_{pl}$	±40	$\pm\alpha_{pl}$	±25	$\pm\alpha_{pl}$	±25
±D	±0.6	±40	±0.4	±25	±0.2	±25
±E	±0.6	±45	±0.4	±30	±0.2	±30

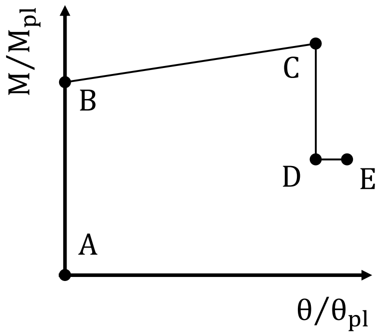


Fig 1.5: Non-linear hinge parameters for IPE, SHS and CHS sections useable for multilinear model.

During nonlinear simulations performance might be assessed by checking the acceptance criteria shown in Fig. 1.6. Three different performance levels are classified: Damage Limitation (DL), Significant Damage (SD) and Near Collapse state (NC). Performance levels are defined by rotation ratios for IPE, SHS and CHS sections.

ACCEPTANCE CRITERIA (Φ/Φ_{pl})			
	IPE	**SHS**	**CHS**
DL	15	5	6
SD	25	12	10
NC	35	18	16

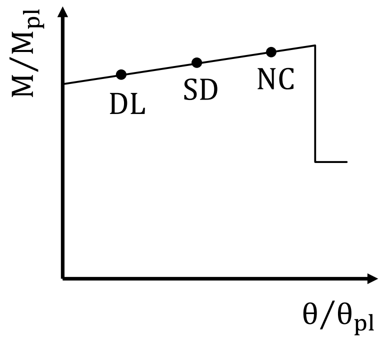


Fig 1.6: Definition of limit states for FUSEIS beam link plastic hinges

5 FUSEIS PIN LINKS

5.1 ADDITIONS TO PAR. 6.3.1 STRUCTURAL TYPES

(5) Frames with FUSEIS pin links are those in which the horizontal forces are mainly resisted by a number of pin links rigidly connected to strong columns (Fig. 3.1). Each pin link is composed of two receptacle beams connected through a short steel pin (Figure 2a). Alternatively, receptacles are omitted and pins are provided with threads in different directions (one left one right) at their ends and directly bolted to end-plates that are connected to the column flanges Figure 2b). Joints between floor beams and columns may be pinned or semi-rigid.

(6) In frames with FUSEIS pin links the dissipative zones are located in the middle part of the pins where the pin section is reduced, so that energy is dissipated by means of cyclic bending of the pins. The design criteria listed in par. 6.5.2 for dissipative zones apply for the pins.

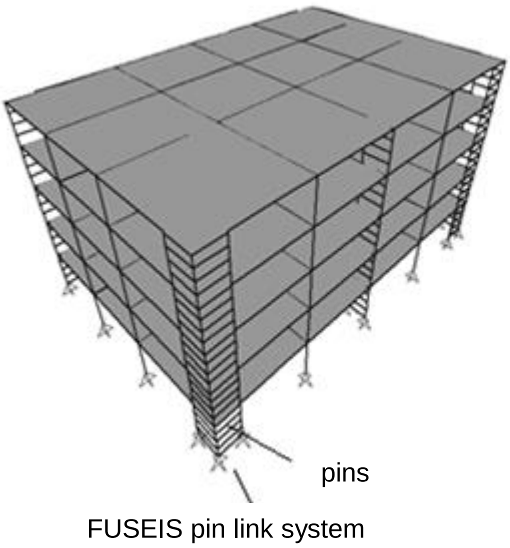


Fig. 5.1: FUSEIS pin link system in a building

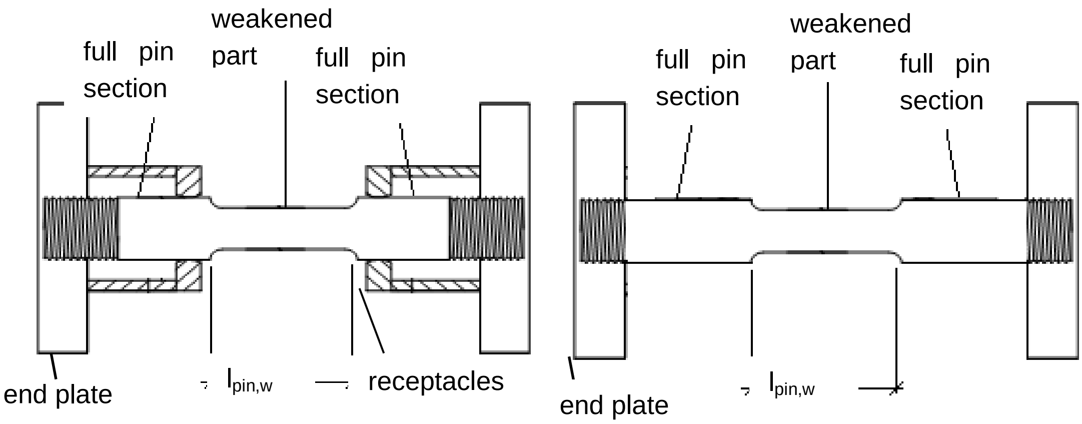


Fig. 5.2: FUSEIS pin link a) with receptacles, b) without receptacles

5.2 ADDITIONS TO PAR. 6.3.2, TABLE 6.2 BEHAVIOR FACTORS

Table 6.2: Upper limit of reference values of behavior factors for systems regular in elevation

STRUCTURAL TYPE	Ductility class	
	DCM	DCH
FUSEIS pin links	2.5	3.0
Condition	$l_{pin,w} < 6 \cdot M_{pl,pin} / V_{pl,pin}$	$l_{pin,w} \geq 6 \cdot M_{pl,pin} / V_{pl,pin}$

where:

$l_{pin,w}$ is the length of the weakened part of the pin

$M_{pl,pin} = W_{pl,pin} \cdot f_y$

$M_{pl,pin}$ is the plastic moment resistance of the weakened pin section

$W_{pl,pin}$ is the plastic section modulus of the weakened part of the pin

f_y is the yield stress of the pin

$V_{pl,pin} = \frac{A_v \cdot f_y}{\sqrt{3}}$ and A_v is the area of the weakened part of the pin

$V_{pl,pin}$ is the plastic shear resistance of the weakened section of the pin

A_v is the shear area of the weakened part of the pin

5.3 ADDITIONS TO 6.5.3, DESIGN RULES FOR DISSIPATIVE ELEMENTS IN COMPRESSION OR BENDING

(3) The length of the weakened part of the pins shall be such that

$$l_{pin,w} \geq 4 \cdot M_{pl,pin} / V_{pl,pin} \quad \text{Eq. (5.1)}$$

in order to ensure the development of a bending mechanism for the pin.

5.4 ADDITIONS TO PAR. 6.12 (NEW) DESIGN AND DETAILING RULES FOR FRAMES WITH FUSEIS PIN LINKS

6.12.1 Analysis

Pin links may be simulated by beam elements that are divided in three parts with different cross sections as following.

- Links with receptacles

The receptacle beam sections at the two ends and the weakened pin in the middle.

- Links without receptacles
  The full pin section at the two ends and the weakened pin in the middle.

The joints between the receptacle beams and the system columns are simulated as rigid. Rigid zones shall be provided from column centers to column faces to consider their clear length in the analysis and thus exclude non-existent beam flexibilities.

Connections between floor beams and system columns are formed as simple. However, for composite buildings some degree of semi-rigidity develops due to the presence of the slab reinforcement. Column bases may be either pinned or fixed.

6.12.2 Pin links

Pin links shall be verified assuming a formation of a plastic hinge at the ends of its weakened cross section. The most stressed end in the seismic design situation should be verified as following:

$$\frac{M_{Ed}}{M_{pl,pin,Rd}} \leq 1 \qquad \text{Eq. (5.2)}$$

$$\frac{N_{Ed}}{N_{pl,pin,Rd}} \leq 1 \qquad \text{Eq. (5.3)}$$

where:

M_{Ed} is the design bending moment

N_{Ed} is the design axial force

$M_{pl,pin,Rd}$ is the design plastic moment resistance of the weakened pin section.

$N_{pl,pin,Rd}$ is the design axial force resistance of the weakened pin section.

Pin chord rotations within the weakened length shall be limited according to the following condition:

$$\theta_{pin} \leq \theta_{pin,lim} = 0.14 \text{ radians} \quad \text{Eq. (5.4)}$$

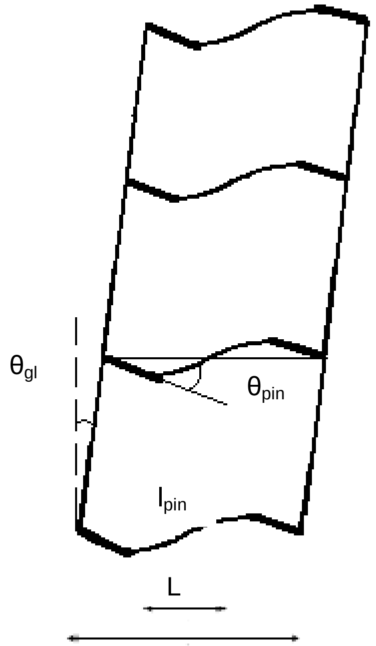


Fig. 5.3: Pin chord rotations

Overstrength of a pin link is defined by the expression:

$$\Omega = \frac{M_{pl,pin,Rd}}{M_{Ed}} \quad \text{Eq. (5.5)}$$

The selection of pin's dimensions shall be such that the value of Ω is close to 1.
To achieve a global dissipative behavior of the frame, it should be checked that the maximum ratios Ω over the entire structure do not differ from the minimum value Ω by more than 25%.

$$\frac{max\Omega}{min\Omega} \leq 1.25 \quad \text{Eq. (5.6)}$$

6.12.3 Columns connected to pin links, receptacle beams and connections to columns

System's columns connected to pin links, and receptacle beams shall be verified to resist the capacity design action effects as following:

$$N_{CD,Ed} = N_{Ed,G} + 1.1 \cdot \alpha \cdot \gamma_{ov} \cdot \Omega \cdot N_{Ed,E}$$ Eq. (5.7)

$$M_{CD,Ed} = M_{Ed,G} + 1.1 \cdot \alpha \cdot \gamma_{ov} \cdot \Omega \cdot M_{Ed,E}$$ Eq. (5.8)

$$V_{CD,Ed} = V_{Ed,G} + 1.1 \cdot \alpha \cdot \gamma_{ov} \cdot \Omega \cdot V_{Ed,E}$$ Eq. (5.9)

where:
$N_{Ed,G}$ ($V_{Ed,G}$, $M_{Ed,G}$) are the axial forces (shear forces and bending moments accordingly) due to the non-seismic actions included in the combination of actions for the seismic design situation,
$N_{Ed,E}$ ($V_{Ed,E}$, $M_{Ed,E}$) are the axial forces (shear forces and bending moments accordingly) due to the design seismic action,
$\Omega = min\Omega_i = min\left\{\frac{M_{pl,pin,Rd,i}}{M_{Ed,i}}\right\}$ is the minimum overstrength factor for all pins in the building, see (5),
γ_{ov} =1.25 is the material overstrength factor and
α=1.5 is an additional overstrength factor of the system.
The total magnification factor of seismic forces or moments cannot exceed the value of the behavior factor q used in analysis. The real yield stress of the steel should be as close as possible to its nominal value in order to achieve an economic design.

6.12.4 Full sections of pin links

The moment resistance of the full section of pin links shall be verified at its contact area with the face plate of the receptacles, in accordance with:

$$\frac{M_{CD,Ed}}{M_{pl,Rd}} \leq 1$$ Eq. (5.10)

where:

$$M_{Cd,Ed} = \frac{l_{pin}}{l_{pin,w}} \cdot M_{pl,pin,Rd}$$ Eq. (5.11)

l_{pin} is the length between the face plates of the receptacles or the end plates of the pin
$l_{pin,w}$ is the length of the weakened part of the pins and
$M_{pl,Rd}$ is the design plastic bending moment resistance of full pin section.

6.12.5 Connections of pin links

Categories B and C of bolted joints with high strength bolts of category 8.8 or 10.9 should be employed between system columns and end plates. The connections must have sufficient overstrength to ensure that they do not fail when plastic hinges develop in the pins. They shall be designed for the capacity design bending moment and shear force, determined from (12) and (13):

$$M_{Cd,con,Ed} = 1.1 \cdot \gamma_{ov} \cdot \frac{l_{pin}}{l_{pin,w}} \cdot M_{pl,pin,Rd} \qquad \text{Eq. (5.12)}$$

$$V_{Cd,con,Ed} = 1.1 \cdot \gamma_{ov} \cdot \frac{2 \cdot M_{pl,pin,Rd}}{l_{pin,w}} \qquad \text{Eq. (5.13)}$$

6.12.6 Pin modelling for non-linear static (pushover) analyses

The structural model used for elastic analysis shall be extended to include the response of structural elements beyond the elastic state and estimate expected plastic mechanisms and the distribution of damage. Plastic hinge properties of pin links are placed at the ends of their weakened sections and are illustrated in Figure 3, while values of the parameters are given in Table 1 where M represent moments, θ chord rotations.

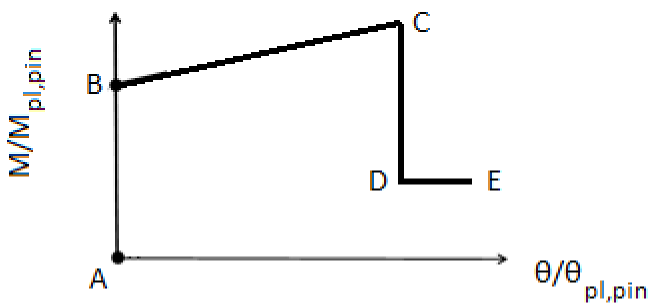


Fig. 5.4: Non - linear plastic hinge properties of pin links

Table 5.1: Values at characteristic points of pin links

Point	$M/M_{pl,pin}$	$\theta/\theta_{pl,pin}$
A	0	0
B	1	0
C	2	100
D	0,5	100

E	0,5	150

Table 5.2 provides plastic rotation capacities of pin links at the three considered limit states that are marked in Figure 4.

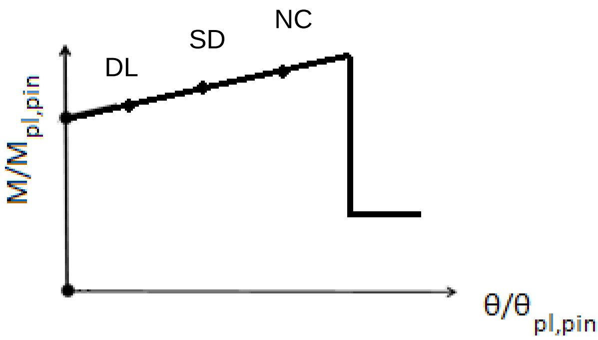


Fig. 5.5: Limit states for pin links

Table 5.2: Plastic rotation capacities of pin links

Limit state	DL(damage limitation)	SD(significant damage)	NC (near collapse)
$\theta/\theta_{pl,pin}$	30	45	60

6.12.7 Pin modelling for non - linear dynamic analysis

- Modelling

In non-linear dynamic analysis, pin links are represented by multi-linear plastic link elements positioned at the ends of the weakened part of the pin. The behavior of the non-linear link is defined only for the rotational degree of freedom with respect to the major axis of inertia while the remaining degrees of freedom are modelled as linear. The nonlinear properties applied include a moment rotation input with positive and negative moment capacities equal to the plastic moment capacity and initial stiffness of the pin under positive and negative moments (Table 3). The hysteresis type should be the one provided by the Multi-linear plastic kinematic model (Figure 5).

Table 5.3: Multi-linear force – deformation definition

Point	Moment	Rotation
1	-2 $M_{pl,pin}$	-100 $\theta_{pl,pin}$
2	-1 $M_{pl,pin}$	-20 $\theta_{pl,pin}$
3	0	0
4	1 $M_{pl,pin}$	20 $\theta_{pl,pin}$
5	2 $M_{pl,pin}$	100 $\theta_{pl,pin}$

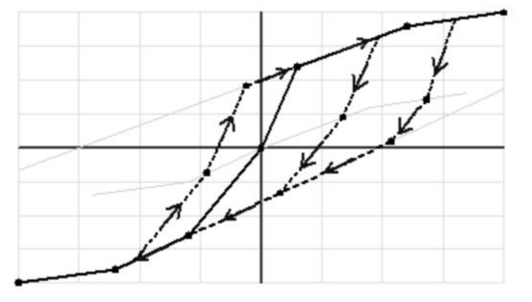


Fig. 5.6: Limit states for pin links Multi-linear plastic kinematic model

The remaining part is modelled as following:

- Links with receptacles
  The middle part of the pin and the receptacle beams are introduced as beam elements with the corresponding cross sections (Figure 6a).
- Links without receptacles
  The middle part of the pin is represented by a beam element with cross section the weakened pin section, while the end parts of the pin by beam elements with the full pin cross section (Figure 6b).

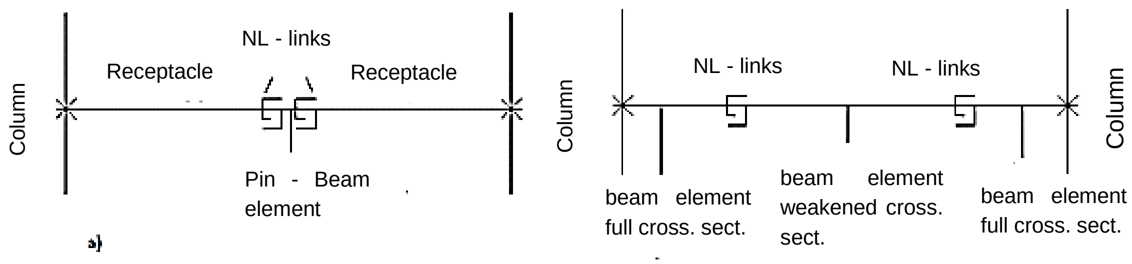


Fig. 5.7: Representation of pin links a) with b) without receptacles

- Low cycle fatigue verifications

When performing non-linear cyclic analysis, pin damage due to low cycle fatigue shall be examined. Following damage curve for pins applies:

$$logN = -0.90 - 3 \cdot log\Delta\theta \qquad \text{Eq. (5.14)}$$

where:

$\Delta\theta$ is the pin chord rotation range and

N is the corresponding number of cycles to failure

The damage index D may be determined by the Palmgren – Miner law of damage accumulation as follows:

$$D = \frac{n_1}{N_1} + \frac{n_2}{N_2} + ... + \frac{n_i}{N_i} \leq 1$$ Eq. (5.15)

where:
n_i is the number of cycles carried out at the same stress range S_i,
N_{fi} is the number of cycles at which failure occurs in case of constant amplitude and
i is the total number of constant amplitude cycles.
The histogram of deformation ranges may be determined by application of the reservoir method.

6 FUSEIS BOLTED BEAM SPLICES

6.1 DESIGN GUIDELINES TO BE INCLUDED IN CHAPTER 7 OF EN1998-1-1

<u>7.1 Generalities</u>

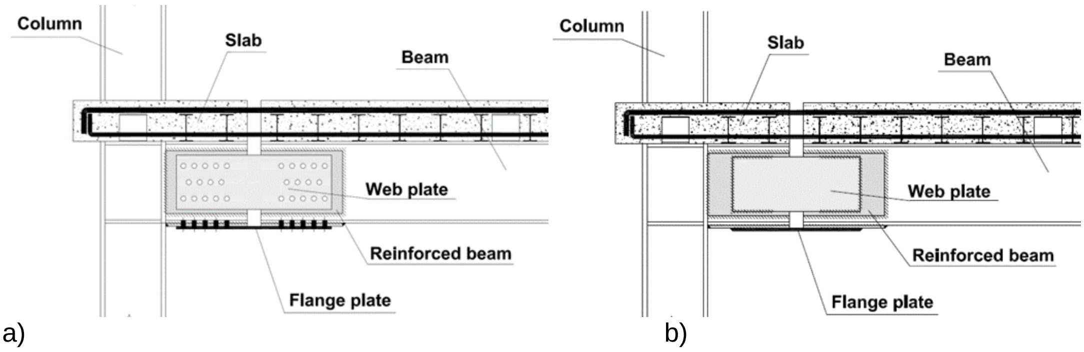
7.1.2 Design principles

(5)P at the end of the sentence, add the following: “For composite moment resisting frames with dissipative beam splices see 7.8.5”.

<u>7.3 Types of structure and behaviour factor</u>

7.3.1 Types of structure

g) Composite moment resisting frames with dissipative beam splices: structures with the same definition and limitations as indicated in 7.3.1(1)a, but with beam splices as the dissipative connections. In the dissipative beam splice, the interrupted composite steel-concrete beams are restored by steel cover plates that connect the web and the lower flange of the beams. The steel cover plates may be bolted or welded to the beam. The part of the beam near the interruption is reinforced with additional steel plates welded to both its web and flange, as well as the column is strengthened in correspondence of the beam to column joint. The gap in the concrete slab just over the fuse is intended to avoid major damage to the concrete, by allowing the fuse to develop larger rotations, avoiding both crushing of the concrete as well as damage to the floor finishes. The configuration of the device on a typical beam-to-column connection is shown below.



a) b)

Fig. 7.1: FUSEIS beam splices with a) bolted cover plates, b) welded cover plates

7.3.2 Behaviour factor

Table 7.2: Upper limit of reference values of behavior factors for systems regular in elevation

STRUCTURAL TYPE	Ductility class

	DCM	DCH
g) Composite moment resisting frames with dissipative beam splices	3.0	4.0

7.8 Design and detailing rules for moment resisting frames with dissipative beam splices

7.8.1 Specific criteria

(1)P 6.6.1(1)P is applied but with plastic hinges formed at the beam splices. The concentration of inelastic behavior on dissipative beam splices shall prevent the spreading of damage into the beams and the columns. In order to assure that irreplaceable parts remain undamaged, these have to be designed such that they remain in the elastic regime when the beam splice achieves its resistant capacity.

(2)P 7.7.1(2)P is applied.

(3) Regarding the location of dissipative zones, 7.5.2(5)P is applied.

(4) The required configuration to form plastic hinges should be obtained following the rules presented in 4.4.2.3, 7.8.3, 7.8.4, 7.8.5.

7.8.2 Analysis

(1)P 7.7.2(1)P is applied.

(2) 7.7.2(2) is applied.

(3) 7.7.2(4) is applied.

7.8.3 Rules for beams, columns, beam splices and reinforcing steel

(1) 7.7.3(2)P is applied.

(2) 6.6.2(2) is applied with $M_{pl,Rd}$, $N_{pl,Rd}$ and $V_{pl,Rd}$ substituted with $M_{FUSE,pl,Rd}$, $N_{FUSE,pl,Rd}$ and $V_{FUSE,pl,Rd}$ which are the plastic moment, axial and shear resistance of the beam splice, respectively.

(3) 6.6.3(1)P is applied for the columns but with overstrength factor being the minimum value of $\Omega = M_{FUSE,pl,Rd,i}/M_{Ed,i}$; $M_{FUSE,pl,Rd,i}$ is the beam splice plastic moment at beam i.

(4) Reinforced beam cross-sections and their necessary length are designed such that the current composite beam zone, immediately after the interruption of the reinforcement, and the beam-column joint section remain elastic. In this respect,

6.6.3(1)P is applied for the quantification of the acting forces. Regarding the safety verification of the referred cross-sections, elastic resistance should be considered.

(5) In order to avoid brittle failures of the welds or bolts that connect the fuse plates to the beam, these should be designed to guarantee that the maximum stresses developed by the fuse can be transmitted safely to the beam.

(6) 7.7.3(6) to (9) are applied.

(7) To ensure a global dissipative behaviour of the structure, it should be checked that the maximum ratios Ω over the entire structure do not differ from the minimum value Ω by more than 25%.

$$\frac{max\Omega}{min\Omega} \leq 1.25 \qquad \text{Eq. (7.1)}$$

7.8.4 Beam-to-column joint

(1) 6.6.4 is applied with beam splices considered as the joint.

7.8.5 Condition to disregard the composite nature of the beams with the slab

(1) Since the dissipative beam splices are composed solely with steel dissipative elements (interruption of the concrete slab and continuous rebar designed as non-dissipative), principle c) may be considered for 7.5.2(2)P.

(2) In this sense, to validate the assumption made in 7.8.5(1), rebar should be designed such that they remain in the elastic regime.

7.8.6 Beam splice modelling for non-linear static and dynamic analyses

(1) The multi-linear plastic link model can be used as reference model for the elastic-plastic behaviour of the dissipative connections. The behaviour of the non-linear link is defined only for the rotational degree of freedom with respect to the major axis of inertia while the remaining degrees of freedom are modelled as linear. The hysteresis type should be the one provided by the Multi-linear plastic pivot model (Dowell, Seible and Wilson, 1998).

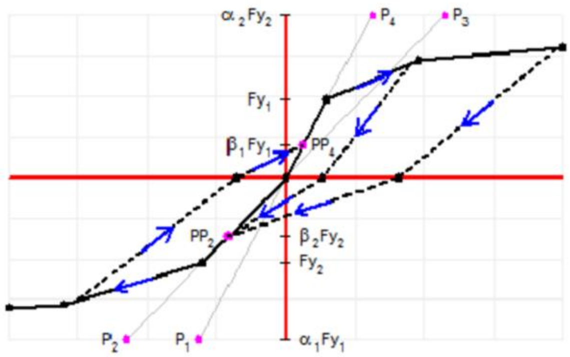


Fig. 7.2: Multilinear plastic pivot model

(2) The moment-rotation relationship of the beam splice adopted in the nonlinear analyses shall take into account appropriately the asymmetry of the behaviour under hogging and sagging moment, as well as the onset of the buckling failure mechanisms. A schematic behaviour of the dissipative connection is represented in the following figure.

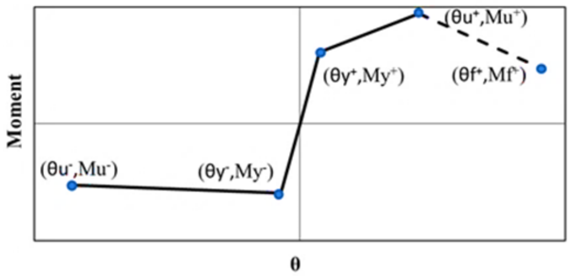


Fig. 7.3: Schematic multilinear modelling

<u>Note: The sections 7.8; 7.9; 7.10; 7.11 and 7.12</u> of EN 1998-1:2004 will be considered as sections 7.9; 7.10; 7.11; 7.12 and 7.13, respectively.

6.2 PRINCIPLES: DESIGN PROCEDURE TO SUPPORT THE GUIDELINES TO BE INCORPORATED IN EN1998-1-1

1) Aiming to avoid excessive overstrength, the steel material of the dissipative fuses shall have controlled properties. In accordance with *EN1998-1-1*, their yield strength must have a maximum value of:

$$f_{y,\max} \leq 1.1 \cdot \gamma_{ov} \cdot f_y \qquad \text{Eq. (2.1)}$$

where $\gamma_{ov} = 1.25$ is the overstrength factor and f_y is the nominal value of the yield strength.

2) The gap in the slab just over the fuse is intended to avoid major damage to the concrete, by allowing the fuse to develop larger rotations, without concrete-to-concrete contact. The gap width in the reinforced concrete part of the fuse can 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, 10% of the height of the slab and 10% of the total height of the composite cross-section.

3) Design of the beam splice and rebar are such that the rebar remain in the elastic regime. It is recommended that the upper layer rebar area is twice the area of the flange plate fuse.

4) The resistance of the reinforcement plates at the beam splice zone as well as its minimum span from the beam-column joint should be such that the beam-column joint section and the current composite beam section remain elastic (Fig. 2.1).

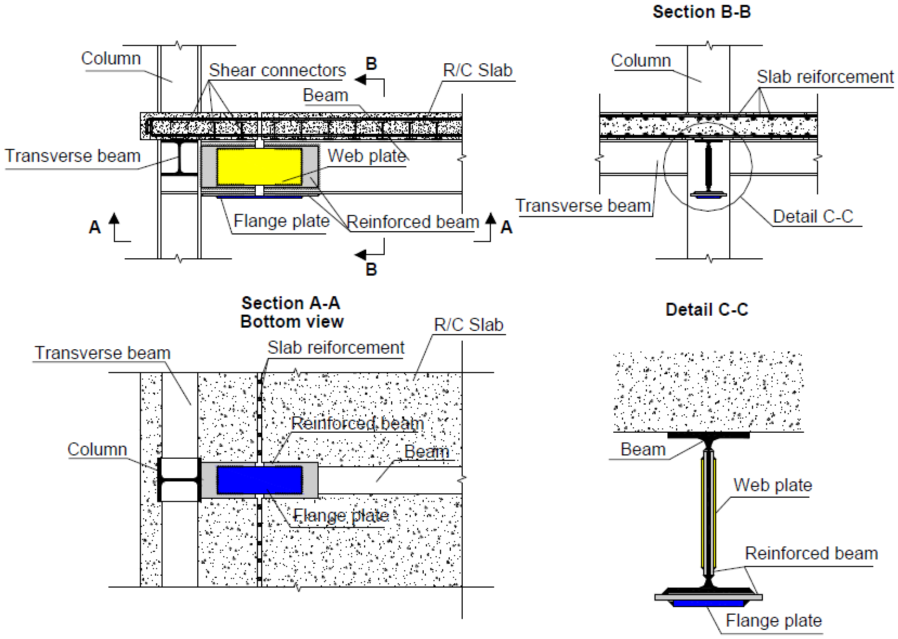


Fig. 2.1: Schematic representation of the welded FUSEIS beam splices

5) The FUSEIS welded or bolted beam splices should verify the following resistance checks:
Firstly, it should be verified that the full plastic moment of resistance and shear forces are not decreased by compression forces.

$$\frac{N_{Ed}}{N_{pl,fuse,Rd}} \leq 0.15 \qquad \text{Eq. (2.2)}$$

The shear resistance shall be verified with capacity design criteria, considering that plastic hinges are developed at both ends of MRFs' beams simultaneously. The shear resistance of the beam splice is assumed to be solely conferred by the web plates.

$$\frac{V_{CD,Ed}}{V_{pl,fuse,Rd}} \leq 1.0 \qquad \text{Eq. (2.3)}$$

where $V_{CD,Ed} = 2M_{max,fuse}/L_{fuses,ij}$ is the capacity design shear force, $M_{max,fuse}$ is the maximum developed by the fuses, $L_{fuses,ij}$ is the distance between the fuses of the same beam and $V_{pl,fuse,Rd}$ is the resistance conferred by the web plates.

$$\frac{M_{Ed}}{M_{\max,fuse}} \leq \frac{1}{\Omega} \leq 1.0 \qquad \text{Eq. (2.4)}$$

where M_{Ed} is the design moment, $M_{max,fuse}$ is the maximum moment of the fuse and Ω is the overstrength factor.
6) To achieve a global dissipative behaviour of the structure, it should be checked that the maximum ratios Ω over the entire structure do not differ from the minimum value Ω by more than 25%.

$$\frac{\max\Omega}{\min\Omega} \leq 1.25 \qquad \text{Eq. (2.5)}$$

7) The non-dissipative elements (columns, composite beams) shall be capacity designed for increased values of internal forces compared to the ones derived from the analyses with the most unfavourable seismic combination, to ensure that the failure of the fuses occurs first. All the elements shall consider the following capacity design actions:

$$N_{CD,Ed} = N_{Ed,G} + 1.1 \cdot \gamma_{ov} \cdot \Omega \cdot N_{Ed,E} \qquad \text{Eq. (2.6)}$$

$$M_{CD,Ed} = M_{Ed,G} + 1.1 \cdot \gamma_{ov} \cdot \Omega \cdot M_{Ed,E} \qquad \text{Eq. (2.7)}$$

$$V_{CD,Ed} = V_{Ed,G} + 1.1 \cdot \gamma_{ov} \cdot \Omega \cdot V_{Ed,E} \qquad \text{Eq. (2.8)}$$

where $N_{Ed,G}$, $M_{Ed,G}$ and $V_{Ed,G}$ are respectively the axial forces, shear forces and bending moments due to the non-seismic actions included in the combination of actions for the seismic design situation. $N_{Ed,E}$, $M_{Ed,E}$ and $V_{Ed,E}$ are respectively the axial forces, shear forces and bending moments due to the design seismic action. $\Omega = \min \Omega_i = \min\{M_{max,fuse,i}/M_{Ed,i}\}$ is the minimum overstrength factor for all dissipative connections in the building, see Eq. (2.4). $\gamma_{ov} = 1.25$ is the material overstrength factor, see Eq. (2.1).

7 FUSEIS WELDED BEAM SPLICES

7.1 DESIGN GUIDELINES TO BE INCLUDED IN CHAPTER 7 OF EN1998-1-1

7.1 Generalities

7.1.2 Design principles

(5)P at the end of the sentence add the following: “for composite moment resisting frames with dissipative beam splices see 7.8.5.

7.2 no changes

7.3 Types of structure and behaviour factor

7.3.1 Types of structure

g) Composite moment resisting frames with dissipative beam splices: structures with the same definition and limitations as indicated in 7.3.1(1)a, but with beam splices as the dissipative element.

7.3.2 Behaviour factor

Maximum behaviour factor for g): 3 for DCM and 4 for DCH.

7.4 no changes

7.5 no changes

7.6 no changes

7.7 no changes

7.8 Design and detailing rules for moment resisting frames with dissipative beam splices

7.8.1 Specific criteria

(1)P 6.6.1(1)P is applied but with plastic hinges formed at the beam splices.

(2)P 7.7.1(2)P is applied.

(3) Regarding the location of dissipative zones, 7.5.2(4) or 7.5.2(5) is applied.

(4) The required configuration to form plastic hinges should be obtained following the rules presented in 4.4.2.3, 7.8.3, 7.8.4, 7.8.5.

7.8.2 Analysis

(1)P 7.7.2(1)P is applied.

(2) 7.7.2(2) is applied.

(3) 7.7.2(4) is applied.

7.8.3 Rules for beams, columns, beam splices and reinforcing steel

(1) 7.7.3(2)P is applied.

(2) 6.6.2(2) is applied with $M_{pl,Rd}$, $N_{pl,Rd}$ and $V_{pl,Rd}$ substituted with $M_{FUSE,pl,Rd}$, $N_{FUSE,pl,Rd}$ and $V_{FUSE,pl,Rd}$ which are the plastic moment, axial and shear resistance of the beam splice, respectively.

(3) 6.6.3(1)P is applied for the columns but with overstrength factor being the minimum value of $\Omega = M_{FUSE,pl,Rd,i}/M_{Ed,i}$; $M_{FUSE,pl,Rd,i}$ is the beam splice plastic moment at beam i.

(4) Reinforced beam cross-sections and their necessary length are designed such that the current composite beam zone, immediately after the interruption of the reinforcement, and the beam-column joint section remain elastic. In this respect, 6.6.3(1)P is applied for the quantification of the acting forces and yielding resistance should be considered for the safety verification of the referred cross-sections.

(5) 7.7.3(6) to (9) are applied.

(6) To ensure a global dissipative behaviour of the structure, it should be checked that the maximum ratios Ω over the entire structure do not differ from the minimum value Ω by more than 25%.

$$\frac{\max\Omega}{\min\Omega} \leq 1.25 \qquad \text{Eq. (1.1)}$$

7.8.4 Beam-column joint

(1) 6.6.4 is applied with beam splices considered as the joint.

7.8.5 Condition to disregard the composite nature of the beams with the slab

(1) Since the dissipative beam splices are composed solely with steel dissipative elements (interruption of the concrete slab and continuous rebar designed as non-dissipative), principle c) may be considered for 7.5.2(2)P.

(2) In this sense, to validate the assumption made in 7.8.5(1), rebar should be designed such that they remain in the elastic regime.

The old sections 7.8; 7.9; 7.10; 7.11 and 7.12 will be considered as sections 7.9; 7.10; 7.11; 7.12 and 7.13, respectively.

7.2 PRINCIPLES: DESIGN PROCEDURE TO SUPPORT THE GUIDELINES TO BE INCORPORATED IN EN1998-1-1

1) Aiming to avoid excessive overstrength, the steel material of the dissipative fuses shall have controlled properties. In accordance with *EN1998-1-1*, their yield strength must have a maximum value of:

$$f_{y,max} \leq 1.1 \cdot \gamma_{ov} \cdot f_y \qquad \text{Eq. (2.1)}$$

where $\gamma_{ov} = 1.25$ is the overstrength factor and f_y is the nominal value of the yield strength.

2) The nominal yield strength of the flange fuses shall be low and preferably not exceed 235 MPa.

3) The gap in the slab just over the fuse is intended to avoid major damage to the concrete, by allowing the fuse to develop larger rotations, without concrete-to-concrete contact. The gap width in the reinforced concrete part of the fuse can 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, 10% of the height of the slab and 10% of the total height of the composite cross-section.

4) Design of the beam splice and rebar are such that the rebar remain in the elastic regime. It is recommended that the upper layer rebar area is twice the area of the flange plate fuse.

5) The resistance of the reinforcement plates at the beam splice zone as well as its minimum span from the beam-column joint should be such that the beam-column joint section and the current composite beam section remains elastic (Fig. 2.1).

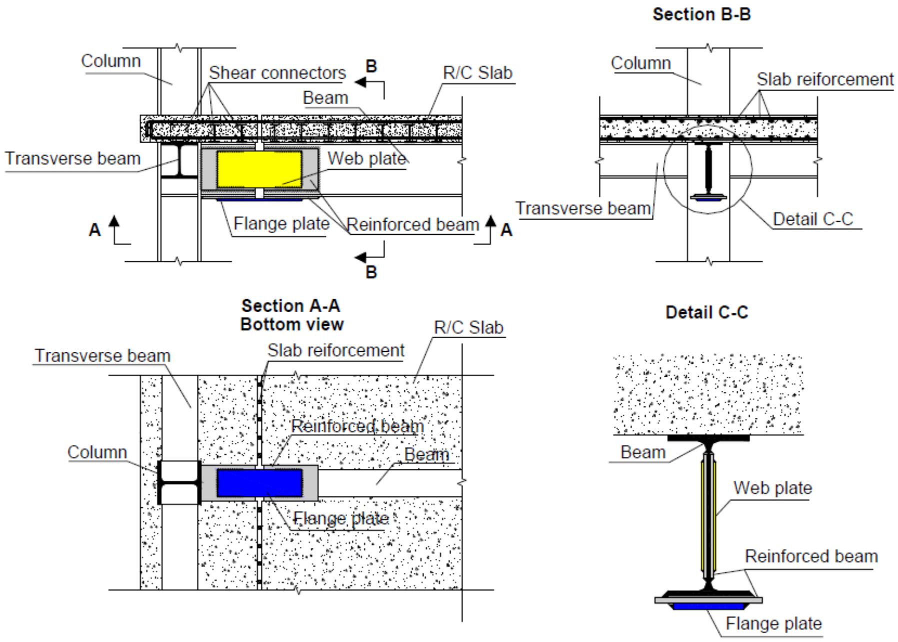


Fig. 2.1: Schematic representation of the welded FUSEIS beam splices

6) The welded FUSEIS beam splices should verify the following resistance checks: Firstly, it should be verified that the full plastic moment of resistance and shear forces are not decreased by compression forces.

$$\frac{N_{Ed}}{N_{pl,fuse,Rd}} \leq 0.15 \qquad \text{Eq. (2.2)}$$

The shear resistance shall be verified with capacity design criteria, considering that plastic hinges are developed at both ends of MRFs' beams simultaneously. The shear resistance of the welded FUSEIS is assumed to be solely conferred by the web plates.

$$\frac{V_{CD,Ed}}{V_{pl,fuse,Rd}} \leq 1.0 \qquad \text{Eq. (2.3)}$$

where $V_{CD,Ed} = 2M_{max,fuse}/L_{fuses,ij}$ is the capacity design shear force, $M_{max,fuse}$ is the maximum developed by the fuses, $L_{fuses,ij}$ is the distance between the fuses of the same beam and $V_{pl,fuse,Rd}$ is the resistance conferred by the web plates.

$$\frac{M_{Ed}}{M_{max,fuse}} \leq \frac{1}{\Omega} \leq 1.0 \qquad \text{Eq. (2.4)}$$

where M_{Ed} is the design moment, $M_{max,fuse}$ is the maximum moment of the fuse and Ω is the overstrength factor.

7) To achieve a global dissipative behaviour of the structure, it should be checked that the maximum ratios Ω over the entire structure do not differ from the minimum value Ω by more than 25%.

$$\frac{\max\Omega}{\min\Omega} \leq 1.25 \qquad \text{Eq. (2.5)}$$

8) Fuse rotations

9) The non-dissipative elements (columns, current and reinforced composite beams) shall be capacity designed for increased values of internal forces compared to the ones derived from the analyses with the most unfavourable seismic combination, to ensure that the failure of the welded FUSEIS occurs first. All the elements shall consider the following capacity design actions:

$$N_{CD,Ed} = N_{Ed,G} + 1.1 \cdot \gamma_{ov} \cdot \Omega \cdot N_{Ed,E} \qquad \text{Eq. (2.6)}$$

$$M_{CD,Ed} = M_{Ed,G} + 1.1 \cdot \gamma_{ov} \cdot \Omega \cdot M_{Ed,E} \qquad \text{Eq. (2.7)}$$

$$V_{CD,Ed} = V_{Ed,G} + 1.1 \cdot \gamma_{ov} \cdot \Omega \cdot V_{Ed,E} \qquad \text{Eq. (2.8)}$$

where $N_{Ed,G}$, $M_{Ed,G}$ and $V_{Ed,G}$ are respectively the axial forces, shear forces and bending moments due to the non-seismic actions included in the combination of actions for the seismic design situation. $N_{Ed,E}$, $M_{Ed,E}$ and $V_{Ed,E}$ are respectively the axial forces, shear forces and bending moments due to the design seismic action. $\Omega = \min \Omega_i = \min\{M_{max,fuse,i}/M_{Ed,i}\}$ is the minimum overstrength factor for all welded FUSEIS in the building, see Eq. (2.4). $\gamma_{ov} = 1.25$ is the material overstrength factor, see Eq. (2.1).

8 REPLACEABLE BOLTED LINK

8.1 ADDITIONS TO 6.3.1 STRUCTURAL TYPES

(6) h) Replaceable bolted links systems are dual frames obtained by combining moment resisting frames with frames with eccentric bracings and replaceable links in which the links are bolted with the intention of providing energy dissipation capacity, by means of cyclic shear, and being replaceable, while the more flexible moment resisting frames are kept elastic in order to provide the restoring force necessary to re-centre the structure upon removal of damaged links.

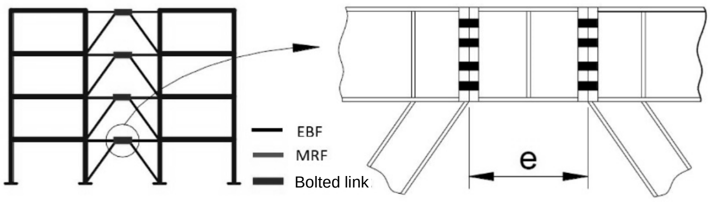


Fig. 6.10: Possible configuration of replaceable bolted links systems

8.2 ADDITIONS TO 6.3.2 BEHAVIOR FACTOR

Table 6.2: Upper limit of reference values of behavior factors for systems regular in elevation

STRUCTURAL TYPE	Ductility class	
	DCM	DCH
h) Replaceable bolted links systems	2.5	4

8.3 ADDITIONS TO 6.8.1 DESIGN CRITERIA

(4)P Frames with eccentric bracings and replaceable links shall be designed so that specific elements or parts of elements called seismic links are removable (bolted) and able to dissipate energy by the formation of plastic shear mechanisms (short links).

8.4 ADDITIONS TO 6.8.4 CONNECTIONS OF THE SEISMIC LINKS

(4) If seismic links are designed to be removable and replaceable, they should be bolted. Contact surfaces should be class B (blasted with shot or grit with zinc paint), providing a coefficient of friction of at least 0.4 and bolts should be preloaded.

(5) Flush end-plate link-beam connection may be used and it should be kept elastic. Therefore, the connection should have a design shear force $V_{j,Ed}$ and bending moment $M_{j,Ed}$ corresponding to a fully yielded and strain hardened link:

$$V_{j,Ed} = \gamma_{sh} \cdot \gamma_{ov} \cdot V_{p,link} \tag{Eq. (6.32)}$$

$$M_{j,Ed} = \frac{V_{j,Ed} e}{2} \tag{Eq. (6.33)}$$

where

γ_{sh} is the strain hardening factor.

NOTE 1 The recommended value is γ_{sh} = 1.8 for DCH and γ_{sh} = 1.5 for DCM.

(5) To achieve the connection over-strength, very short dissipative members may need to be adopted (with length e as small as $0.8M_{p,link}/V_{p,link}$).

(6) The flexibility of the link's bolted connection should be considered in the global analysis.

NOTE 1 If flush end-plate connection with preloaded bolts is used, it should be considered infinitely rigid.

(7) Estimation of seismic performance of link-beam bolted connections under cyclic loading should be supported by experimental evidence.

(8) Experimental evidence may be based on existing data. Otherwise, tests should be performed.

8.5 ADDITIONS TO 6.10.2 MOMENT RESISTING FRAMES COMBINED WITH CONCENTRIC BRACINGS

(7) In dual frames obtained by combining moment resisting frames with braced frames, the weaker, more flexible, subsystem (moment resisting frames) should

provide a minimum strength of the structure. Therefore, the duality of the structure should be checked by verifying that the moment resisting frames should be able to resist at least 25% of the total seismic force:

$$F_y^{MRF} \geq 0.25 \cdot (F_y^{MRF} + F_y^{BF}) \qquad \text{Eq. (6.34)}$$

$$F_y^{MRF} = \frac{4M_{pl,b}}{H} \qquad \text{Eq. (6.35)}$$

where

F_y^{MRF} is the yield strength of moment resisting frames;

F_y^{BF} is the yield strength of braced frames;

L is the frame span (see Fig. 6.15);

H is the frame story height (see Fig. 6.15);

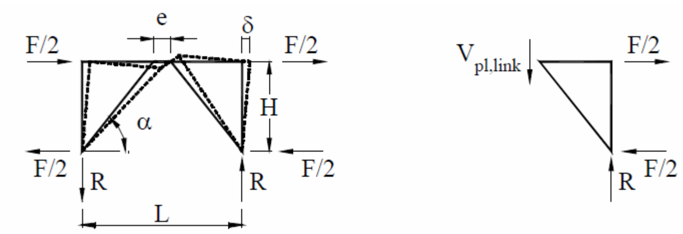
$M_{pl,b}$ is the design value of plastic moment resistance of the end of a beam from moment resisting frames, in accordance with EN 1993.

(8) The yield strength of frames with eccentric bracings should be computed as follows:

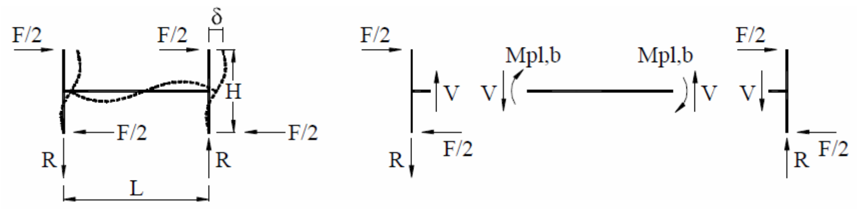
$$F_y^{EBF} = \frac{L}{H} \cdot V_{p,link} \qquad \text{Eq. (6.36)}$$

where

$V_{p,link}$ is the design value of shear resistance of the seismic link (see 6.8.2).



a)



b)

Fig. 6.15: Basic one-storey frame with eccentric bracings and moment resisting frame.

8.6 ADDITIONS TO CHAPTER 6 SPECIFIC RULES FOR STEEL BUILDINGS

6.12 Design and detailing rules for frames with steel shear panels - see ch. 8

6.13 Re-centering capacity of steel dual frames

(1) Re-centring capacity of dual configurations should be verified by preventing yielding in moment resisting frames up to the attainment of ultimate deformation capacity in dissipative frames. This may be achieved by keeping the ultimate displacement of dissipative frames (at ultimate limit state) smaller than the yield displacement of elastic frames (moment resisting frames):

$$\delta_u^{DIS} < \delta_y^{MRF} \qquad \text{Eq. (6.37)}$$

where

δ_u^{DIS} is the ultimate displacement of dissipative frames at ultimate limit state;

δ_y^{MRF} is the yield displacement of moment resisting frames;

6.13.1 Re-centring capacity of replaceable bolted links systems

6.13.1.1 Analytical check

(1) Dissipative frames in replaceable bolted links systems are the frames with eccentric bracings. Their ultimate displacement corresponds to achievement of plastic deformation capacity of the links and should be computed as follows:

$$\delta_u^{EBF} = \delta_y^{EBF} + \delta_{pl}^{EBF} = \frac{F_y^{EBF}}{K^{EBF}} + \frac{e}{L \cdot e} \; H \cdot \gamma_{pl,u} < \delta_y^{MRF} = \frac{F_y^{MRF}}{K^{MRF}} \qquad \text{Eq. (6.38)}$$

$$K^{EBF} = \frac{K_{link}^{EBF} \cdot K_{br}^{EBF}}{K_{link}^{EBF} + K_{br}^{EBF}} \qquad \text{Eq. (6.39)}$$

$$K_{link}^{EBF} = \frac{L}{H^2} \cdot (L \cdot e) \cdot \frac{G \cdot A_S}{e} \qquad \text{Eq. (6.40)}$$

$$K_{br}^{EBF} = 2 \cdot \frac{E \cdot A}{l_{br}} \cdot \cos^2 \alpha \qquad \text{Eq. (6.41)}$$

$$K^{MRF} = \frac{4}{H^2 \cdot \left(\frac{L}{6 \cdot E \cdot I_b} + \frac{H}{12 \cdot E \cdot I_c} \right)} \qquad \text{Eq. (6.42)}$$

where

δ_u^{EBF}	is the ultimate displacement of frames with eccentric bracings at ultimate limit state;
δ_y^{EBF}	is the yield displacement of frames with eccentric bracings;
δ_{pl}^{EBF}	is the plastic displacement of frames with eccentric bracings;
K^{EBF}	is the stiffness of frames with eccentric bracings;
e	is the link's length (see Fig. 6.15);
$\gamma_{pl,u}$	is the plastic deformation capacity of the link;
K^{MRF}	is the stiffness of moment resisting frames;
K_{link}^{EBF}	is the link's stiffness;
K_{br}^{EBF}	is the braces stiffness;
G	is the shear modulus;
A_s	is the link's shear area;
E	is the Young's modulus;
A	is the brace cross-section area;
l_{br}	is the brace length;
α	is the brace angle;
I_b	is the beam's inertia moment;

I_c is the column's inertia moment.

(2) The analytical procedure should be used as a pre-design of re-centring capacity.

NOTE 1 It may solely be used for checking re-centring capacity of low-rise structures, where lateral deformation of the structure is dominated by a shear-type response.

NOTE 2 For checking re-centring capacity of mid-rise and high-rise buildings (where a global bending behaviour may arise in elevation) is strongly recommended to use also nonlinear static and/or dynamic analyses.

(3) Because using formulas is an approximate and simplified approach, nonlinear static and/or dynamic analyses are recommended for all structures in order to check re-centring capability.

6.13.1.2 Link modelling for nonlinear static analysis

(1) Short bolted links nonlinear behaviour in shear should be described by the following backbone curve:

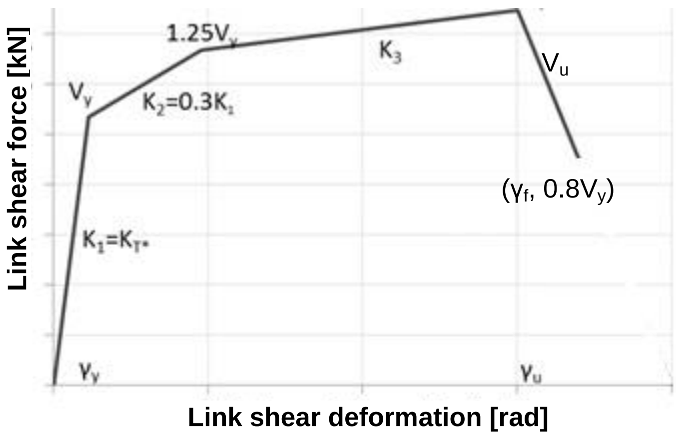


Fig. 6.16: Shear links nonlinear behavior.

where

K_1 is the initial stiffness of the link;

V_y is the shear resistance of the link ($V_{p,link}$);

V_u is the ultimate resistance of the link;

NOTE 1 It is recommended to use $1.8V_y$ in case of DCH and $1.5V_y$ in case of DCM.

γ_u is the ultimate shear rotation of the link;

NOTE 1 It is recommended to use 0.15 rad for DCH and 0.1 rad for DCH.

γ_f is the failure shear rotation of the link;

NOTE 1 It is recommended to use 0.17 rad for DCH and 0.11 rad for DCM.

6.13.1.3 Link modelling for nonlinear dynamic analysis

(1) Hysteretic behaviour of bolted shear links should be considered. The hysteresis loop should be formulated with rules for stiffness and strength degradation, and pinching. Parameters for hysteretic rules presented in Table 6.4 may be used.

Table 6.4: Parameters for hysteretic behaviour of shear bolted links

Specific parameter	Value
Stiffness degrading parameter	20
Ductility-based strength decay parameter	0.001
Hysteretic energy-based strength decay parameter	0.001
Smoothness parameter for elastic-yield transition	10
Parameter for shape of unloading	0.5
Slip length parameter	0
Slip sharpness parameter	100
Parameter for mean moment level of slip	0
Exponent of gap closing spring	10
Gap closing curvature parameter	1000
Gap closing stiffness coefficient	1

9 REPLACEABLE SHEAR PANEL

9.1 ADDITIONS TO PAR. 6.3.1 STRUCTURAL TYPES

(9) Frames with replaceable thin walled steel shear panels are those in which the horizontal forces are mainly resisted by members subjected to shear.

Moment resisting frames combined with replaceable thin walled steel shear panels.

(6) In frames with replaceable shear panels, the dissipative zones should be mainly located in the panels.

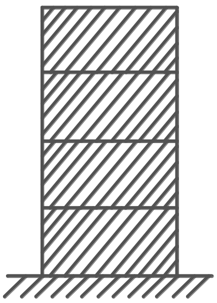


Fig. 6.10: Frames with replaceable thin walled steel shear panels (dissipative zones in replaceable shear panels only). Default values for α_u/α_1 (see 6.3.2(3) and Table 6.2).

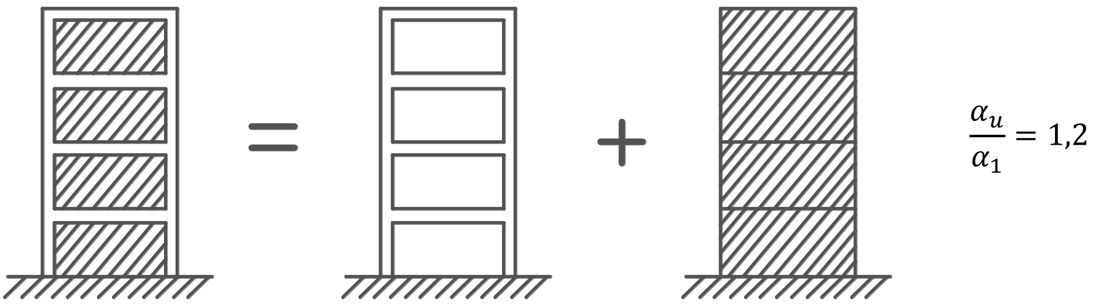


Fig. 6.11: Moment resisting frames combined with steel shear panels (dissipative zones in bending and shear panels). Default values for α_u/α_1 (see 6.3.2(3) and Table 6.2).

9.2 ADDITIONS TO PAR. 6.3.2, TABLE 6.2 BEHAVIOR FACTORS

Table 6.2: Upper limit of reference values of behavior factors for systems regular in elevation

STRUCTURAL TYPE	Ductility class	
	DCM	DCH
h) Frames with steel	4	5 α_u/α_1

shear panels		
Moment resisting frames with steel shear panels	4	5 α_u / α_1

9.3 ADDITIONS TO PAR. 6.10, DESIGN RULES FOR STEEL STRUCTURES WITH CONCRETE CORES OR CONCRETE WALLS AND FOR MOMENT RESISTING FRAMES COMBINED WITH CONCENTRIC BRACINGS OR INFILLS

6.10.4 Moment resisting frames combined with replaceable thin walled steel shear panels.

(1) Dual structures with both moment resisting frames and braced frames acting in the same direction should be designed using a single *q* factor. The horizontal forces should be distributed between the different frames according to their elastic stiffness.

(2) The moment resisting frames and the braced frames should be conform to 6.6 and 6.12.

(3)P The duality of the structure shall be checked by verifying that the moment resisting frames is be able to resist at least 25% of the total seismic force:

$$F_y^{MRF} \geq 0.25 \cdot (F_y^{MRF} + F_y^{SPSW}) \qquad \text{Eq. (6.34)}$$

where

F_y^{MRF} is the yield strength of moment resisting frames;

F_y^{SPSW} is the yield strength of frame with replaceable thin walled steel shear panels.

9.4 ADDITIONS TO CHAPTER 6 SPECIFIC RULES FOR STEEL BUILDINGS

6.12 Design and detailing rules for frames with replaceable shear panels

6.12.1 Design criteria

(1)P Frames with shear panels shall be designed so that yielding of the steel shear panels in shear will take place before failure of the connections and before yielding or buckling of the beams or columns.

(2)P The structural system shall be designed so that a homogeneous dissipative behaviour of the whole set of steel shear panels is realised.

(3) The application of the steel shear panels should be limited to panels having aspect ratio 0.8 < L/h < 2.5.

NOTE Other shear panel aspect ratio performance should be verified experimentally.

6.12.2 Analysis

(1)P Under gravity load conditions, only beams and columns shall be considered to resist such loads, without taking into account the steel shear panels.

(2) For preliminary design, the size of the steel shear panels and boundary elements (beams and columns) may be determined by approximation of the steel shear panels with tension only diagonals (Fig. 6.12).

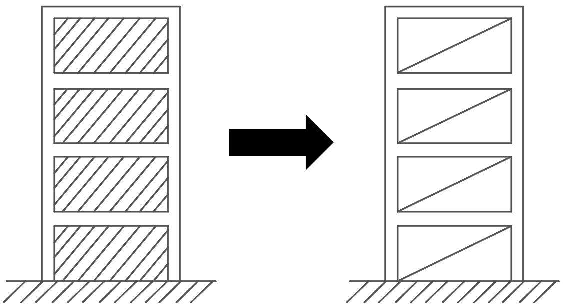


Figure 6.12: Approximation of the steel shear panels by tension only diagonals.

(3)P The frame with tension only diagonals shall be designed in accordance with criteria and rules given in 6.7 for frames with concentric bracings.

6.12.3 Horizontal and vertical boundary elements

(1)P The horizontal and vertical boundary element shall be designed to resist the maximum forces developed under the tension field action of the fully yielded panels.

(2)P The vertical boundary elements shall have moments of inertia about an axis taken perpendicular to the plane of the web, I_c, not less than:

$$I_c \geq \frac{0.00307 \cdot t_w \cdot h^4}{L} \qquad \text{Eq. (6.32)}$$

where
t_w is the thickness of the steel shear panel;
h is the height of steel shear panel, between horizontal boundary element centrelines;
L is the width of steel shear panel, between vertical boundary element centrelines;

NOTE If different sections are used for vertical boundary elements, then the average values of moment of inertia may be used in calculation.

(3)P The horizontal boundary elements shall have moments of inertia about an axis taken perpendicular to the plane of the web, I_b, not less than:

$$I_b \geq 0.0031 \cdot \frac{\Delta t_w \cdot L^4}{L} h \qquad \text{Eq. (6.33)}$$

where
Δt_w is the difference in steel shear panel thicknesses above and below horizontal boundary element;

6.12.4 Steel shear panels

(1) The steel shear panel thickness, may be calculated using the tension-only diagonal (see **6.12.2**) area with the following expression:

$$t_w = \frac{2 \cdot A_{brace} \cdot \Omega \cdot \sin\theta}{L \cdot \sin 2\alpha} \qquad \text{Eq.(6.34)}$$

where
A_{brace} is the area of the tension-only diagonal;
Ω is the overstrength factor, defined in **6.7.4 (1)**;
θ is the angle between the vertical and the longitudinal axis of the tension-only diagonal;
α is the angle of inclination of the tension field of the panel, measured from the vertical, may be taken as 40°, or may be calculated with expression (6.35):

$$\tan^4 \alpha = \frac{1 + \dfrac{t_w \cdot L}{2 \cdot A_c}}{1 + t_w \cdot h \cdot \left(\dfrac{1}{A_b} + \dfrac{h^3}{360 \cdot I_c \cdot L} \right)} \qquad \text{Eq. (6.35)}$$

where
I_c is the column moment of inertia;
t_w is the thickness of the steel shear panel;

A_c is the columns;
A_b is the areas of beams;
I_c is the moment of inertia of the vertical boundary element, may be taken as average between both vertical boundary elements.

(2) The plastic shear strength of a steel shear panels may be calculated with expression (6.34) based on the assumption that each panel may be modelled by a series of inclined pin-ended strips (see **6.12.6**):

$$V_n = 0.42 F_y t_w L_{cf} \sin 2\alpha \qquad \text{Eq. (6.34)}$$

where
L_{cf} is the clear distance between vertical boundary element flanges;
F_y is the yielding strength of the steel shear panel;

6.12.5 Horizontal-to-vertical boundary elments connection

(1)P For frames with steel shear panels, the plastic resistance of the connected dissipative member, R_d, calculated according to **6.5.5**, and shall account for the shear force resulting from the yield strength, in tension of the diagonal panel yielding.

6.12.6 Shear panels to boundary elements connection

(1)P The required strength of steel shear panel connection to the surrounding boundary elements shall equal the expected yield strength, in tension, of the panel.

(2) Two typical details of connections of steel shear panel to boundary beams and columns may be used, see 6.13.

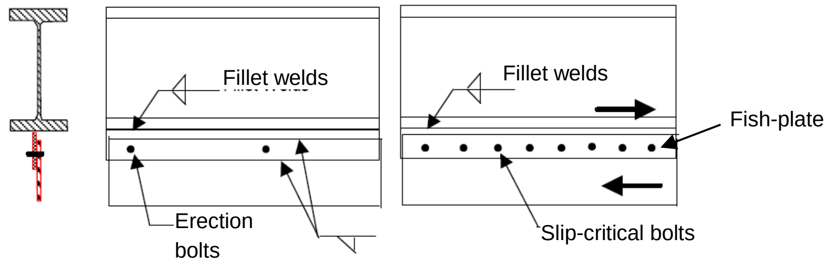


Figure 6.13: Shear panel to boundary elements connection

(3)P The welded connection shall be designed such that the fish plates and welds develop the shear strength of the panel.

(4) If re-centring capacity is of interest, bolted connections are recommended. The bolts should be slip-resistant and able to develop the shear strength of the panels.

(5) It is expected that during the cyclic loading of the steel shear panels, the bolts slip before the tension field yields. Therefore, the design shear and bearing resistance should be also verified, according to EN 1993-1-8.

(6) In case of very thin steel shear panels, welded strengthening plates may be used in order to increase the bearing resistance.

6.13 Re-centering capacity of steel dual frames

(1)P Re-centring capacity of dual configurations shall be verified by preventing yielding in moment resisting frames up to the attainment of ultimate deformation capacity in dissipative frames. This may be achieved by keeping the ultimate displacement of dissipative frames (at ultimate limit state) smaller than the yield displacement of elastic frames (moment resisting frames):

$$\delta^{DIS} < \delta^{MRF} \qquad \text{Eq. (6.35)}$$

where

δ_u^{DIS} is the ultimate displacement of dissipative frames at ultimate limit state;
δ_y^{MRF} is the yield displacement of moment resisting frames;

6.13.2 Re-centering capacity of steel dual frames with replaceable shear panels

(1) Nonlinear static and/or dynamic analyses are recommended for all structures in order to check re-centring capability.

6.13.1.1 Shear panel modelling for nonlinear static (pushover) analyses

(1) The shear panels may be represented by minimum 10 inclined tension only pin-ended strip members at angle α with respect to vertical, and oriented in the same direction as the principal tensile stresses in the panel (strip model), see figure 6.15. Characteristic points that define the strip properties are given in Table 6.4 and 6.5.

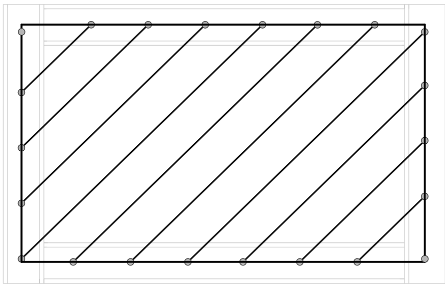


Fig. 6.15: Strip model for static nonlinear analysis

Table 6.4: Nonlinear properties of the tension only strips

Hinge	A	B	C	D	E
P/P_y	0	0.8	1.4	1.4	1.2
Δ/Δ_y	0	0	14	20	27

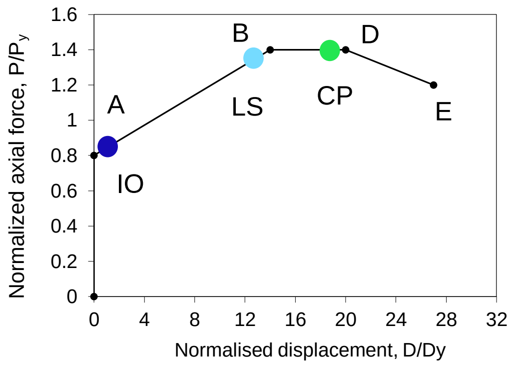


Fig. 6.16: Nonlinear properties of the tension only strips

Table 6.5: Acceptance criteria

Criteria	IO	LS	CP
Δ / Δ_y	0.5	13	19

(2) The area of the strips may be calculated using equation 6.32:

$$A_S = \left(L \cdot \sin\alpha + h \cdot \cos\alpha\right) / n \qquad \text{Eq. (6.32)}$$

where
n is the number of strips per panel;

6.13.1.1 Shear panel modelling for nonlinear dynamic analyses

(1) The panel may be replaced with minimum of 10 strips oriented in both directions (dual strip model), having the properties described in par. **6.13.1.1**, see

figure 6.17.

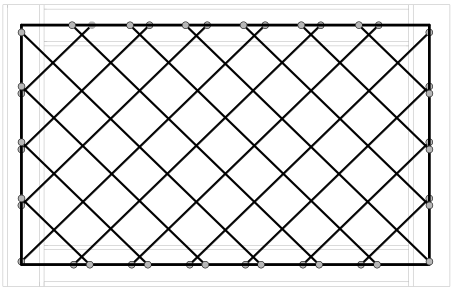


Fig. 6.17: Strip model for static nonlinear analysis

(2) The hysteretic law of the dissipative panels is shown in Fig. 6.18. Special attention is needed to model the pinching that occur during cyclic loading.

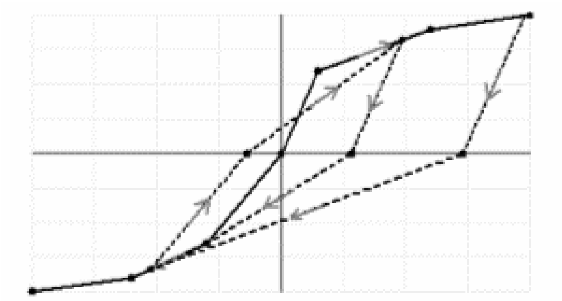


Fig. 6.18: Takeda type hysteretic law

10 CONCENTRICALLY BRACED FRAME WITH MODIFIED BRACES (CBF-MB)

10.1 ADDITIONS TO PAR. 6.3.1 STRUCTURAL TYPES

(10) Concentrically braced frames with modified braces (CBF-MB) are those meeting the following requirements:

- The diagonals of the bracing are intersected by a splitting beam (Fig. 1.1);
- Each diagonal contains a variable H-shaped built-up cross section (Fig. 1.2);
- The joints connecting the bracings to the column are pinned while the splitting beam – column joint is rigid;
- Joints between beam and columns may be pinned or semi-rigid.

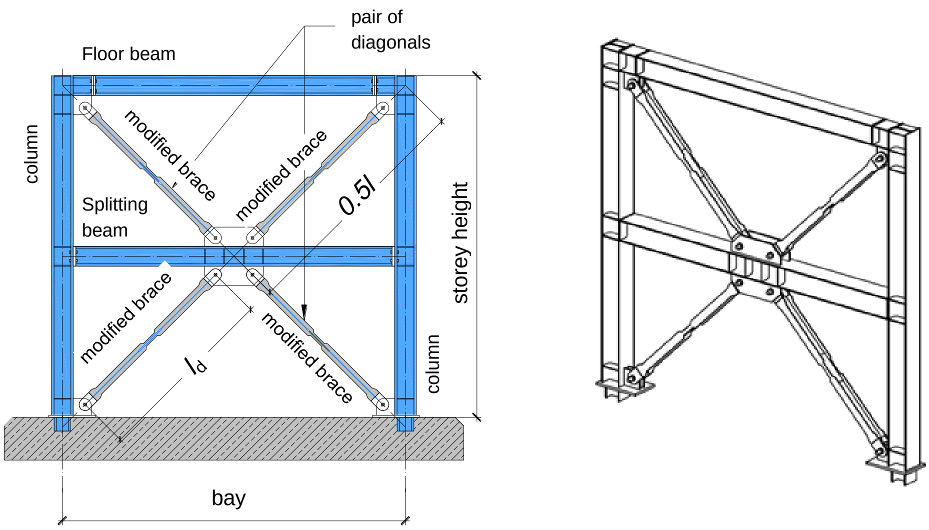
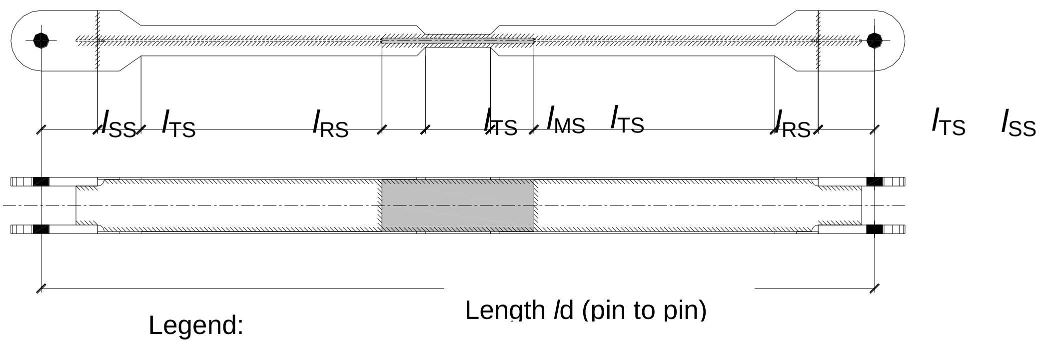


Fig. 10.1: CBF-MB system



Legend:

MS – modified section   SS – strong section
RS – reduced section   TS – transition section
l – length of section (e.g. l_{RS} – length of reduced section)

Fig. 10.2: Overview of modified brace member

(3) In CBF-MB, the dissipative zones should be mainly located in the diagonals. The CBF-MB belongs to the following categories:
− active tension diagonal bracings, in which the horizontal forces can be resisted by the tension diagonals only, neglecting the compression diagonals. The intersection point of these diagonals lies on a horizontal member (splitting beam) which shall be continuous.

(7) In CBF-MB the dissipative zones are located in the diagonals. They should be designed in a manner to separate the zones that yield in tension from those where compression post-buckling plastic strains occur. The design criteria listed in par. 6.5.2 for dissipative zones apply for the modified braces.

10.2 ADDITIONS TO PAR. 6.3.2, TABLE 6.2 BEHAVIOUR FACTORS

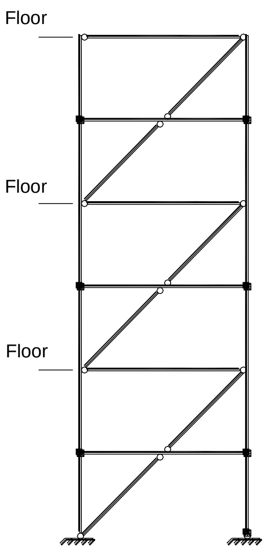
Table 6.2: Upper limit of reference values of behaviour factors for systems regular in elevation

STRUCTURAL TYPE	Ductility class	
	DCM	DCH
CBF-MB	4.0	5.0
Condition as per 6.12.4	ρ =1.00	ρ =1.15

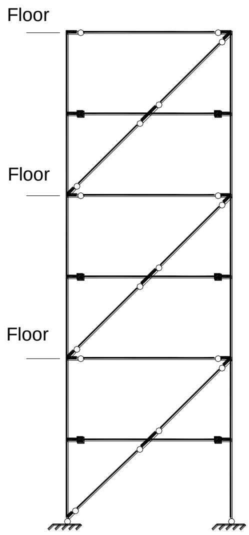
10.3 ADDITIONS TO PAR. 6.12 (NEW) DESIGN AND DETAILING RULES FOR CONCENTRICALLY BRACED FRAMES WITH MODIFIED BRACES (CBF-MB)

6.12.1 Analysis

Elastic multimodal analysis may be performed based on 6.7.2 (1) and (2) with the following specific requirements.
The modified braces should be defined by constant H-shape section with characteristics of RS and joined to the frame by simple pin connections. Columns should be continuous through all stories. Joints between columns and floor beam and column bases may be modelled as nominally pined or semi-rigid. General interpretation is shown on Figure (1.3).



a)



b)

Fig. 1.3: a) Centre Line -to- Centre Line model for elastic analysis; b) Joint offset model.

6.12.2 Design of modified braces

- Length of MS, RS and TS (Figure 1.2)

The length l_d of the modified brace should be (0,375-0,4)l where l is the system length of the diagonal.

The length of the modified section l_{MS} should be defined by Equation (1.1). The reduced section (RS) length l_{RS} should be designed as long as possible, allowing for the required length of smooth transition section (TS) from RS to the strong section (SS). Preliminary estimation may be made by Equation (1.2).

$$l_{MS} = (0.067 \div 0.085) \cdot l_d \qquad \text{Eq. (1.1)}$$

$$l_{RS} \approx (0.3) \cdot l_d \qquad \text{Eq. (1.2)}$$

- Area ratio

The area of RS should fulfil 6.7.3 (5). Additionally, the following condition shall be satisfied:

$$A_{MS} / A_{RS} \geq 1.4 \qquad \text{Eq. (1.3)}$$

Where:

A_{MS} is the modified section's area,

A_{RS} is the reduced section's area.

The strong section (SS) dimensions and area should be chosen to provide fully elastic response in the net section for pin connection and to fulfil the bearing checks of bolts.

- Section modulus ratio

To ensure that modified section has lower bending capacity than the reduced section including in stage of large plastic strains and strain hardening, the Condition (1.4) should be fulfilled:

$$W_{pl,RS} / W_{pl,MS} \geq 2.0 \qquad \text{Eq. (1.4)}$$

Where:

$W_{pl,RS}$ is the reduced section's plastic modulus,

$W_{pl,MS}$ is the modified section's plastic modulus.

- Buckling length of modified brace

Since there is modified section inserted in the mid-length, the real buckling length $l_{cr} = \mu . l_d$ will be longer than l_d. The effective length l_{cr} may be obtained by FE elastic buckling analysis or by Equation (1.5),

$$\mu = l_{cr} / l_d = 0.88 \cdot K_L^{(0.033)} \cdot K_I^{(0.1\ln(K_L)-0.36)} \qquad \text{Eq. (1.5)}$$

Where:

$K_L = l_{RS} / l_{MS}$ is the section length ratio,

l_{MS} is the length of the modified section,

l_{RS} is the length of the reduced section,

$K_I = I_{MS} / I_{RS}$ is the inertial moment ration,

I_{MS} is the moment of inertia of the modified section,

I_{RS} is the moment of inertia of the reduced section,

μ is the buckling length parameter.

- Limitation of slenderness

The non-dimensional slenderness of the modified brace should be calculated based on the buckling length and should be in accordance with 6.7.3 (1).

- Yield resistance of modified brace

The yield resistance $N_{pl,Rd}$ of the modified brace should follow 6.7.3 (5) and should be obtained by Equation (1.6).

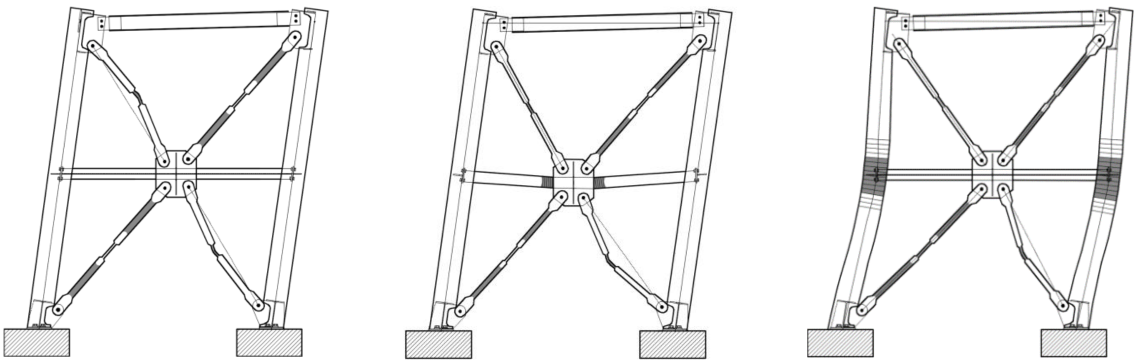
$$N_{pl,Rd} = A_{RS} \cdot f_y / \gamma_{M0} \quad \text{Eq. (1.6)}$$

- Modified brace connections

The connections of the modified braces to floor beams and splitting beams should satisfy the design rules of 6.5.5 (3).

6.12.3 Design of splitting beam

Formation of two types of storey plastic mechanisms is possible in CBF-MB named favourable and unfavourable (Figure 1.4). The favourable mechanism is when both compressed diagonals in a pair buckle and plastic elongations are within the pair of tensioned diagonals. The unfavourable one is when only one from the pair of compressed diagonals buckles and additional plastic hinges appear in the splitting beam or even in the columns (Figure 1.4 b), c)). Unfavourable mechanisms have to be avoided by a proper design of the splitting beam, assuring sufficient resistance and bending stiffness.



a) b) c)

Fig. 1.4: Plastic mechanisms: a) Favourable; b) Weak splitting beam; c) Weak columns

CBF-MB should be designed with splitting beam fixed to the columns thus forming an H-shaped frame. Splitting beam and columns are non-dissipative elements and should remain elastic until reaching ULS (significant damage).

- Transition stage

The stage when the H-shaped frame provides sufficient elastic stiffness and thus forces the unbuckled diagonal to buckle is illustrated in Figure 1.5 and is named transition stage ("just before buckling"). In that stage unbalanced horizontal and vertical forces appear. They may be determined through Equations (1.7) and (1.8), where $N_{b,Rd}$ (Equation (1.9)) is the buckling resistance of the brace according to EN 1993-1-1.

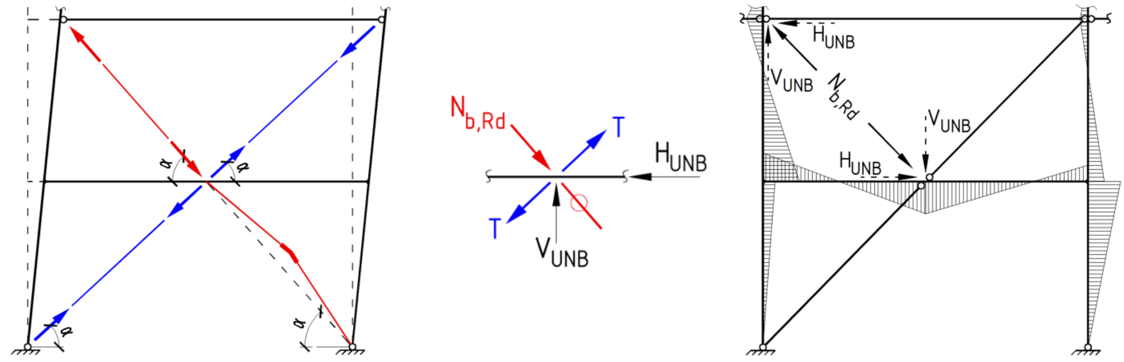


Fig. 1.5: a) Transition stage; b) Unbalanced forces; c) Internal moments (MUNB) resulting from the unbalanced forces (load case UNB)

$$V_{UNB} = N_{b,Rd} \cdot sin\,\alpha \qquad \text{Eq. (1.7)}$$

$$H_{UNB} = N_{b,Rd} \cdot cos\,\alpha \qquad \text{Eq. (1.8)}$$

$$N_{b,Rd} = \chi \cdot A_{RS} \cdot f_y / \gamma_{M1} \qquad \text{Eq. (1.9)}$$

The transition stage is characterized with additional bending moments and axial forces (load case UNB) that occur within the storey H-frame – Figure 1.5 c). That effect has to be accounted for into design. It may be simulated in the model for elastic analysis by introducing unbalanced forces separately for each storey as shown in Figure 1.5 c) or integrally in all stories simultaneously.

- Additional requirements to splitting beam

The splitting beam shall be designed to avoid lateral-torsional buckling by satisfying Equation (1.10).

$$\bar{\lambda}_{LT} \leq 0.40 \qquad \text{Eq. (1.10)}$$

The cross sections of splitting beam shall be chosen to satisfy Equation (1.11) in accordance with 4.4.2.3 (4).

$$2.M_{Rc} \geq 1.3 \cdot M_{Rb}$$ Eq. (1.11)

Where:
M_{Rc} is the relevant design bending resistance of the column ($M_{y,Rd}$ or $M_{z,Rd}$) jointed to the splitting beam,
M_{Rb} is the design bending resistance of the splitting beam.

6.12.4 Design of non-dissipative elements

The non-dissipative CBF-MB's elements are the columns, floor beams and the splitting beams.
They should be designed considering the internal forces from gravity loads in seismic design situation and internal forces including second order effects M_E, V_E and N_E from the seismic load case. The former should be obtained through elastic analysis by tension-only diagonal model and corrected with capacity multiplier $1{,}1.\gamma_{ov}.\Omega_{MIN}.\rho$.
Where:
γ_{ov} is the material overstrength factor according to 6.2 (3),
$\Omega_{MIN} = min\left\{\frac{N_{pl,Rd,i}}{N_{Ed,i}}\right\}$ is the minimum overstrength factor for the modified braces along the building height and
ρ is factor accounting for the available overstrength of the system and the possible higher actual buckling resistance of the brace. The value of ρ depends on the ductility class adopted (Table 6.2).
The design of non-dissipative elements should consider the additional internal forces M_{UNB}, V_{UNB} and N_{UNB} caused by the unbalanced forces, formed in the transition stage (6.12.3).

- Columns

Columns shall be verified to fulfil Equation (1.11) and to resist design forces obtained through Equations (1.12) to (1.14):

$$N_{col,Ed} = N_{Ed,G} + 1{,}1 \cdot \gamma_{OV} \cdot \Omega_{\min} \cdot \rho \cdot (N_E + N_{UNB})$$ Eq. (1.12)

$$M_{col,Ed} = M_{Ed,G} + 1{,}1 \cdot \gamma_{OV} \cdot \Omega_{\min} \cdot \rho \cdot (M_E + M_{UNB})$$ Eq. (1.13)

$$V_{col,Ed} = V_{Ed,G} + 1{,}1 \cdot \gamma_{OV} \cdot \Omega_{\min} \cdot \rho \cdot (V_E + V_{UNB})$$ Eq. (1.14)

- Splitting beams

Splitting beams shall be verified to fulfil Eq. (1.10) and Eq. (1.11) and to resist design forces obtained through Equations (1.15) – (1.17):

$$N_{sb,Ed} = N_{Ed,G} + 1{,}1 \cdot \gamma_{OV} \cdot \Omega_{\min} \cdot \rho \cdot (N_E + N_{UNB}) \qquad \text{Eq. (1.15)}$$

$$M_{sb,Ed} = M_{Ed,G} + 1{,}1 \cdot \gamma_{OV} \cdot \Omega_{\min} \cdot \rho \cdot (M_E + M_{UNB}) \qquad \text{Eq. (1.16)}$$

$$V_{sb,Ed} = V_{Ed,G} + 1{,}1 \cdot \gamma_{OV} \cdot \Omega_{\min} \cdot \rho \cdot \leq (V_E + V_{UNB}) \qquad \text{Eq. (1.17)}$$

- Floor beams

Floor beams shall be verified to resist design forces obtained through Equations (1.18) to (1.20):

$$N_{b,Ed} = N_{Ed,G} + 1{,}1 \cdot \gamma_{OV} \cdot \Omega_{\min} \cdot \rho \cdot (N_E + N_{UNB}) \qquad \text{Eq. (1.18)}$$

$$M_{b,Ed} = M_{Ed,G} + 1{,}1 \cdot \gamma_{OV} \cdot \Omega_{\min} \cdot \rho \cdot (M_E + M_{UNB}) \qquad \text{Eq. (1.19)}$$

$$V_{b,Ed} = V_{Ed,G} + 1{,}1 \cdot \gamma_{OV} \cdot \Omega_{\min} \cdot \rho \cdot (V_E + V_{UNB}) \qquad \text{Eq. (1.20)}$$

6.12.5 Modified braces modelling for non-linear static (pushover) analysis

The joint offset structural model according to Figure 1.3 should be used. The lateral force distribution should comply with 4.3.3.4.2.2. Plastic hinge properties of MB are placed in the middle of each modified brace and the backbone curves are illustrated in Figure 1.6, while values of the parameters are given in Table 1.1.

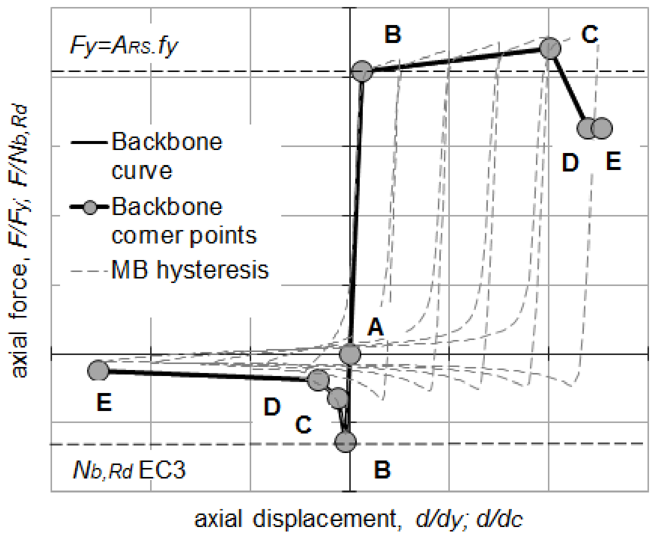


Fig. 1.6: MB backbone curve for static non-linear analysis

Table 1.1: Backbone curve characteristic points

Point	Tension	Point	Compression

	Force	Displacement		Force	Displacement
A	0	0	A	0	0
B	F_y=A_{RS}.f_y	δy	B	$N_{b,Rd}$	δc
C	F_{SH}	16.5δy	C	0,5$N_{b,Rd}$	3δc
D	0,8F_y	19δy	D	0,3$N_{b,Rd}$	8δc
E	0,8F_y	20δy	E	0,2$N_{b,Rd}$	20δy

The following equations (1.21) to (1.25) shall be used for definition of the characteristic points. $N_{b,Rd}$ is the buckling resistance of the brace and χ is the buckling reduction facto as per EN 1993-1-1.

$$\delta_y = f_y \cdot l / E \qquad \text{Eq. (1.21)}$$

$$F_y = A_{RS} \cdot f_y \qquad \text{Eq. (1.22)}$$

$$F_{SH} = F_y + \left(F_y / \delta_y \; 0.005\right) \cdot \left(16.5 \cdot \delta_y\right) \qquad \text{Eq. (1.23)}$$

$$N_{b,Rd} = \chi \cdot A_{RS} \cdot f_y \qquad \text{Eq. (1.24)}$$

$$\delta_C = N_{b,Rd} \cdot \delta_y / F_y \qquad \text{Eq. (1.25)}$$

Table 1.2 provides axial deformation capacities of modified braces in compression and tension at the 3 considered limit states that are marked in Figure 1.7.

Table 1.2: Axial deformation capacities of modified braces in compression and tension

EN 1998-1 limit states	SLS	ULS	
Limit state	DL (damage limitation)	SD (significant damage)	NC (near collapse)
δ / δ_y (Tension)	+2.5	+9.5	+16
δ / δ_c (Compression)	-2.5δy	-9.5δy	-16δy

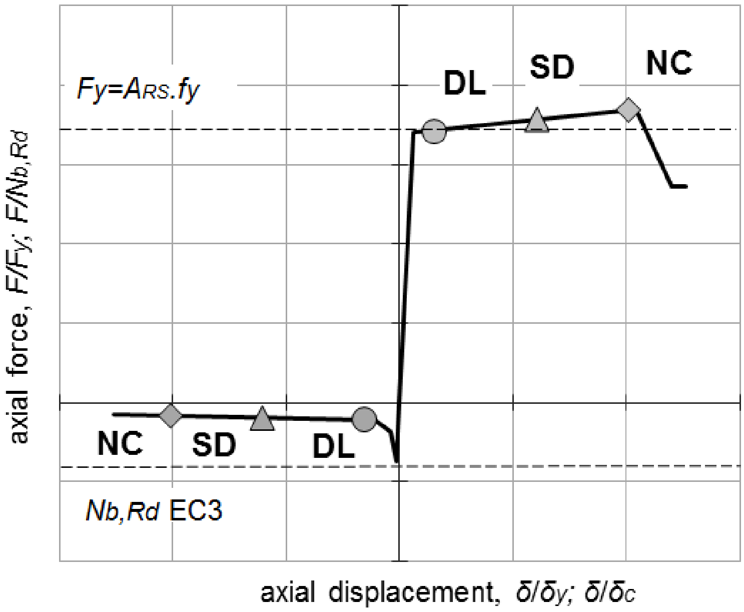


Fig. 10.7: Limit states for modified braces

6.12.6 Modified braces modelling for non--linear dynamic analysis

In non-linear dynamic analysis (NDA) MB should be simulated by multi-linear plastic link elements with pivot hysteresis type. Link element should be connected to diagonal joint offsets – Figure 1.8 a). The non-linear link should be constituted by the parameters α_1, α_2, β_1 and β_2, presented in Table 1.3 and Fig. 1.8. b). The α_1 value identifies the scaling pivot point for unloading to zero from positive force, α_2 locates the point for unloading to zero from a negative force, β_1 locates the pivot point for reverse loading from zero toward positive force and β_2 locates the pivot point for reverse loading from zero towards negative force. The behaviour of the non-linear link should be defined only for the single degree of freedom with respect to the axial elongation/shortening while the remaining degrees of freedom should be modelled as linear.

Table 1.3: Pivot points description

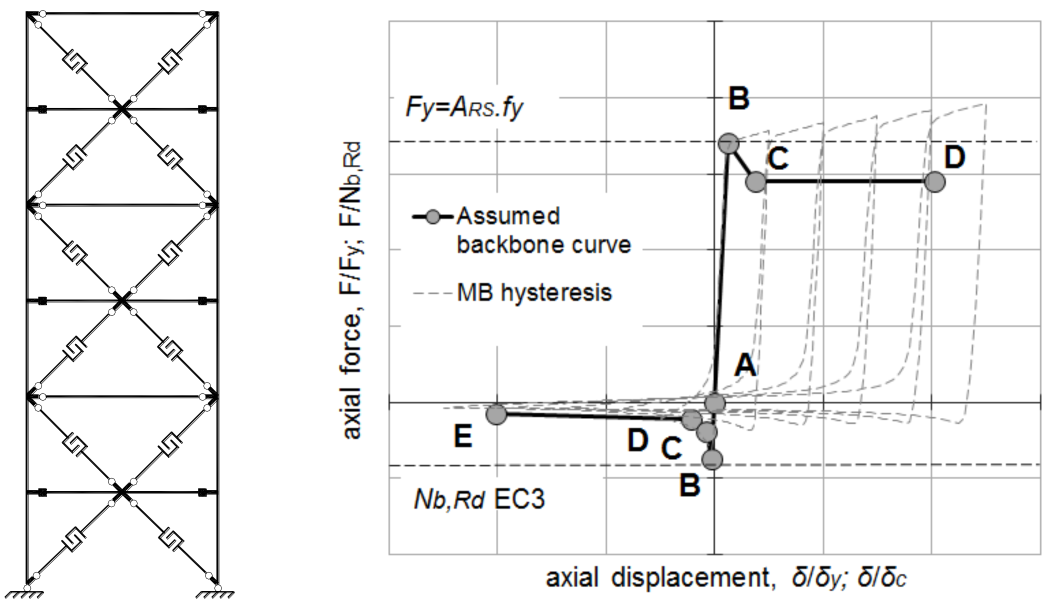
Pivot point parameter	α_1	α_2	β_1	β_2	η
Value	100	0.1	0.02	0.4	0.0

For proper definition of hysteresis behaviour, the multi-linear plastic link requires definition of backbone curve. Table 1.4 summarizes the backbone curve characteristic points. Fig. 1.8 b) represents the backbone curve where cyclic strength degradation in extend to 15% is adopted.

Table 1.4: Backbone curve characteristic points

Point	Tension		Point	Compression	
	Force	Displacement		Force	Displacement
A	0	0	A	0	0
B	F_y=$A_{RS}.f_y$	δy	B	$N_{b,Rd}$	δc
C	0,85F_y	3δy	C	0,5$N_{b,Rd}$	3δc
D	0,85F_y	16.5δy	D	0,3$N_{b,Rd}$	8δc
			E	0,2$N_{b,Rd}$	16.5δy

The representative axial forces and displacements are defined following equations (1.21) to (1.25). $N_{b,Rd}$ is the buckling resistance of the brace as per EN 1993-1-1.



a) b)
Fig. 1.8: CBF-MB model for NDA: a) Multi Linear Plastic Link; b) Backbone curve

6.12.7 Low cycle fatigue verifications

When performing non-linear cyclic analysis the accumulated damage in the modified braces due to low cycle fatigue shall be examined. The representative relation between the MB axial deformation amplitude, δ, corresponding to the number of cycles to failure, N is given by equation (1.26).

$$\delta\,(N) = 110 - 52 \cdot \log(N) \quad \text{Eq. (10.26)}$$

The damage index D may be determined by the Palmgren – Miner law of damage accumulation, Equation (1.27), as follows:

$$D = \frac{n_1}{N_1} + \frac{n_2}{N_2} + ... + \frac{n_i}{N_i} \leq 1 \qquad \text{Eq. (10.27)}$$

Where:
n_i is the number of cycles carried out at the same axial deformation amplitude δ_i,
N_i is the number of cycles at which failure occurs in case of constant axial deformation amplitude and
i is the total number of constant amplitude cycles.