2C09
Design for seismic and climate change

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European Erasmus Mundus Master Course
Sustainable Constructions
under Natural Hazards and Catastrophic Events
520121-1-2011-1-CZ-ERA MUNDUS-EMMC

List of Lectures

1. Earthquake-Resistant Design of Structures I
2. Earthquake-Resistant Design of Structures II
3. Seismic Design of Steel Structures

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Seismic Design of Steel Structures

1. Benefits of steel structures
2. Design criteria for steel structures
3. Detailing rules for steel structures
4. Innovative solutions

Seismic Design of Steel Structures

1. Benefits of steel structures
2. Design criteria for steel structures
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Lesson learned from past earthquakes

L'AQUILA, APRIL 2009

Which are the reasons why steel buildings have such a good behavior?

TOKYO, MARCH 2011

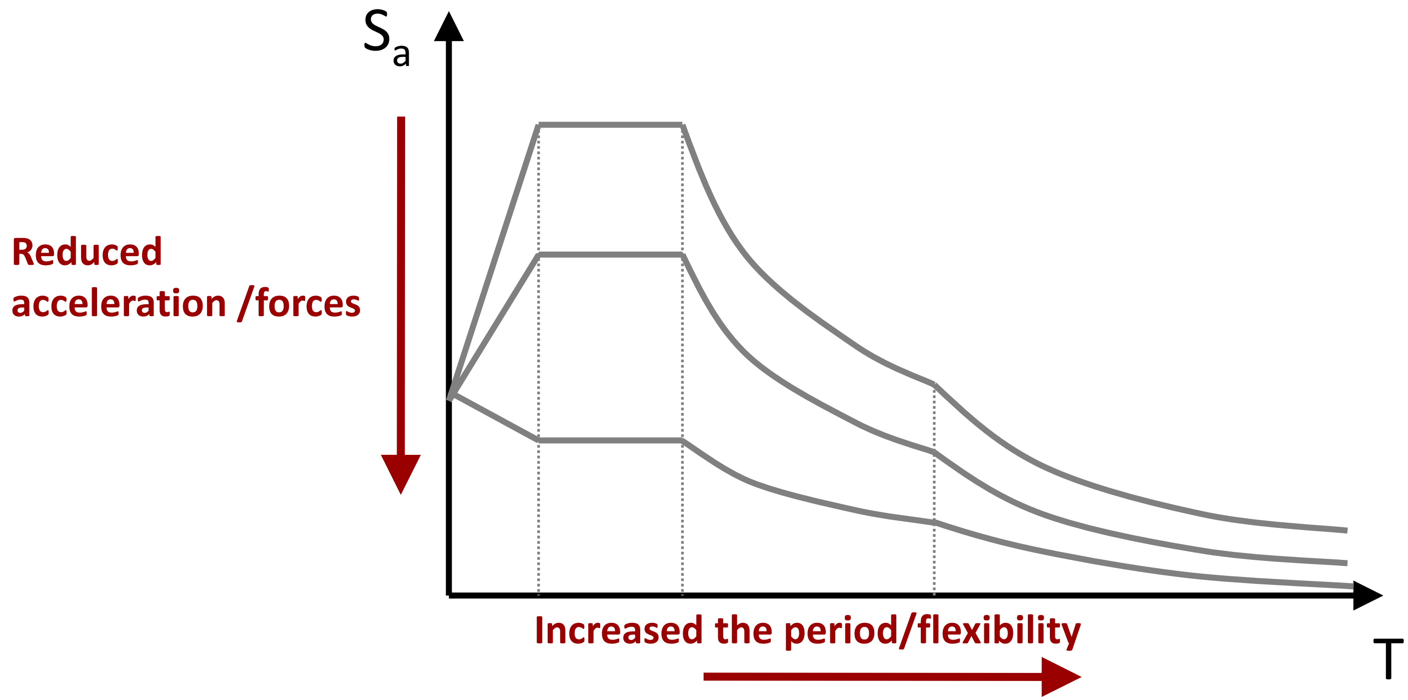
Benefits of Steel Structures

List of contents:

Flexibility & lower weight

Steel structures are generally more flexible than other types of structure and lower in weight if compared with RC and masonry buildings.

In addition, since earthquake forces are inertia force due to accelerating mass: the lower is the mass, the lower is the seismic design forces



Benefits of Steel Structures

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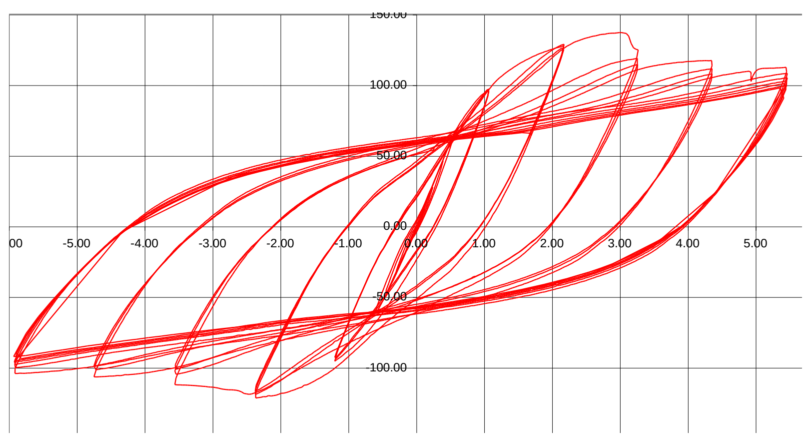
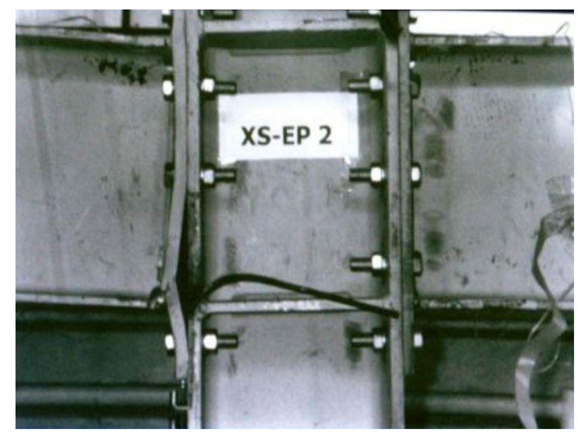
High ductility

The steel is characterized by the **ductility** that *is the capability to perform plastic deformations without failure*





Benefits of Steel Structures

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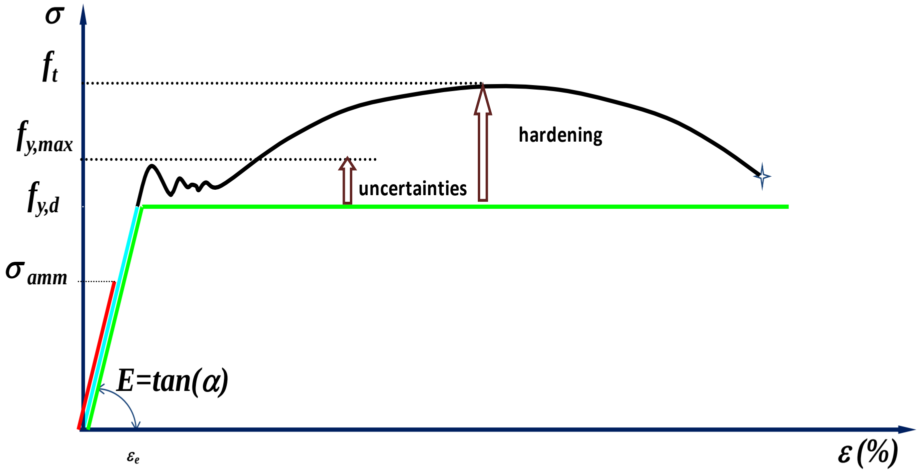
Ductility levels: **1. MATERIAL DUCTILITY**



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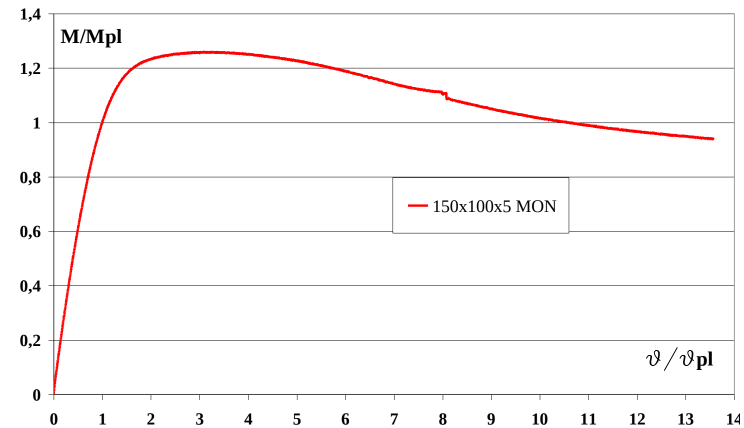
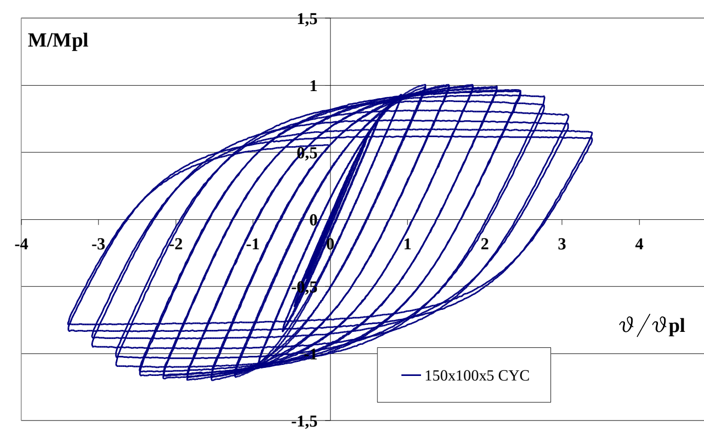
Ductility levels: **2. LOCAL DUCTILITY**

PROFILE 150x100x5





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Benefits of steel structures

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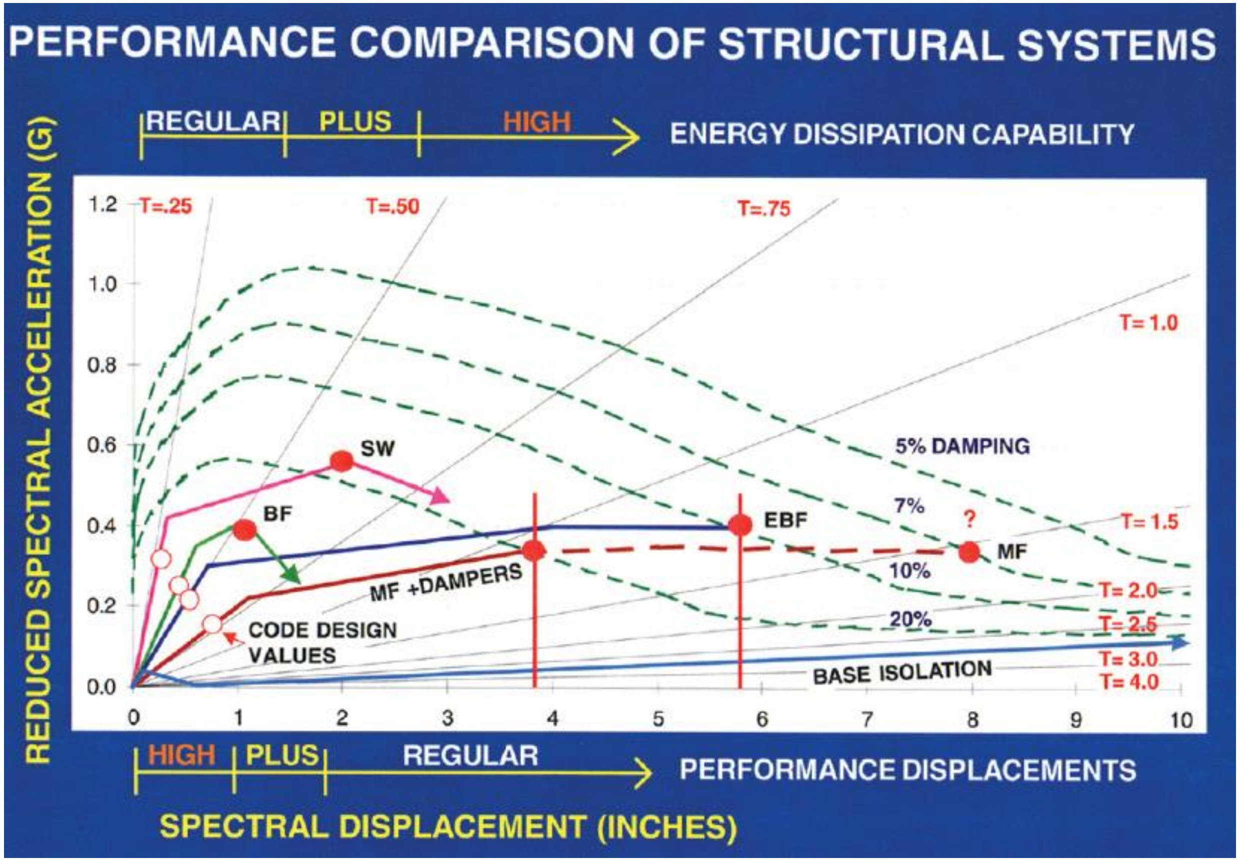
Innovative solutions

Ductility levels: **3. SYSTEM DUCTILITY**

The great variability of structural typologies allows designing to get different seismic performances



Source: FEMA 454 / *December 2006*

Seismic Design of Steel Structures

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4. Innovative solutions

Eurocode 8: Steel Buildings

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Eurocode 8: Steel Buildings

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Eurocode 8: Design Concept

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EN 1998-1-6	2 DESIGN CONCEPTS	3 DUCTILITY CLASSES	SEVERAL BEHAVIOUR FACTORS
Specific rules for steel buildings*	"a" Low dissipative structural behaviuor	DCL (low)	q ≤ 1.5 - 2
	"b" Dissipative structural behaviuor	DCM (medium)	q=f *(structural type)* and q ≤ 4
		DCH (high)	q=f *(structural type)*

* For the design of non dissipative steel structures see: EN1993

Eurocode 8: Rules for Dissipative Structures

Design criteria for dissipative structures

- Structures with dissipative zones shall be designed such that yielding or local buckling or other phenomena due to hysteretic behaviour do not affect the overall stability of the structure

- Dissipative zones shall have adequate ductility and resistance. The resistance shall be verified in accordance with EN 1993

- Dissipative zones may be located in the structural members or in the connections

- If dissipative zones are located in the structural members, the non-dissipative parts and the connections of the dissipative parts to the rest of the structure shall have sufficient overstrength to allow the development of cyclic yielding in the dissipative parts

- When dissipative zones are located in the connections, the connected members shall have sufficient overstrength to allow the development of cyclic yielding in the connections

Eurocode 8: Steel Buildings

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REQUIRED STEEL PROPERTIES

Eurocode 8: Rules for Dissipative Structures

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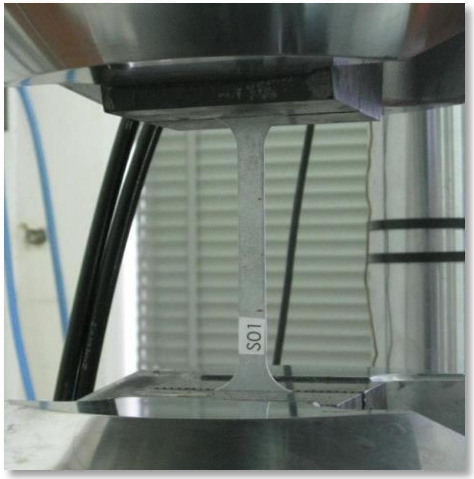
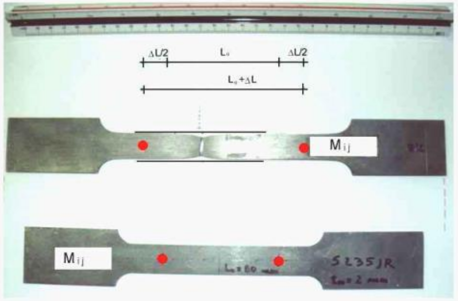
Material Properties

Estimate the actual yield strength of dissipative members/connections, which can be substantially larger than the nominal one.





$$f_{y,max} \leq 1{,}1\,\gamma_{ov}\,f_y$$

RECOMMENDED EC8 VALUE

γ_{ov} **= 1.25**

$f_{y,max}$: Actual maximum yield strength of the steel of dissipative zone

f_y : Nominal yield strength specified for the steel grade

γ_{ov} : Overstrenght factor

Eurocode 8: Rules for Dissipative Structures

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Material Toughness

The choice of material **to avoid brittle fracture** in view of toughness is another key issue in the seismic design of steel structures .

EC8 requires that the toughness of the steels should satisfy the requirements for the seismic action at the **quasi-permanent value of the service temperature** according to see EN 1993-1- 10.

Recent studies have shown that the limitation given in Eurocode 8 is **safe-sided** for European earthquakes.


JRC Scientific and Technical Reports

CHOICE OF STEEL MATERIAL FOR THE DESIGN OF SEISMIC RESISTANT STEEL STRUCTURES

M. Feldmann, B. Eichler, G. Sedlacek, X.XXX

Background documents in support to the implementation, harmonization and further development of the Eurocodes

Joint Report

Prepared under the JRC – ECCS cooperation agreement for the evolution of Eurocode 3 by representatives of CEN / TC 250

Editors: x. xxxxx, x. xxxxx and x. xxxxx

First Edition, xxx 2010
EUR xxxxx EN – 2010

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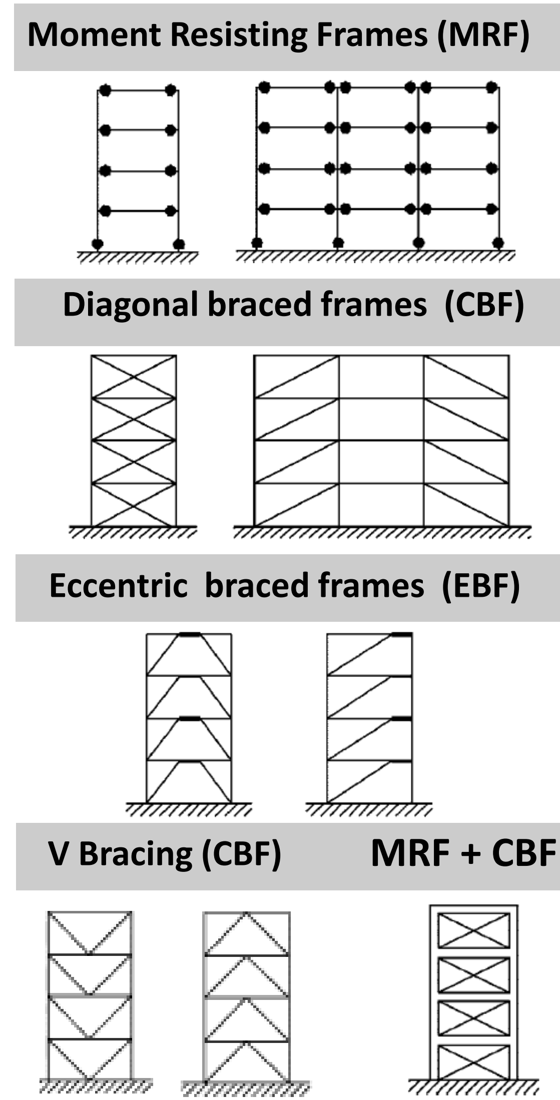
Structural typologies and behaviour factors

Code behaviour factors are mostly empirical, and are supposed to account for ductility, redundancy and overstrength of different structural typologies.

STRUCTURAL TYPE	Ductility Class DCM	Ductility Class DCH
a) MRF	4	$5\alpha_u/\alpha_1$
b) CBF Diagonal bracings V-bracings	 4 2	 4 2,5
c) EBF	4	$5\alpha_u/\alpha_1$
d) Inverted pendulum	2	$2\alpha_u/\alpha_1$
e) Concrete cores/walls	See section 5	
f) MRF + CBF	4	$4\alpha_u/\alpha_1$
g) MRF + infills Unconnected infills	2	2
Connected infills	See section 7	
Isolated infills	4	$5\alpha_u/\alpha_1$



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DUCTILITY REQUIREMENTS: RULES FOR DISSIPATIVE MEMBERS AND FOR CONNECTIONS

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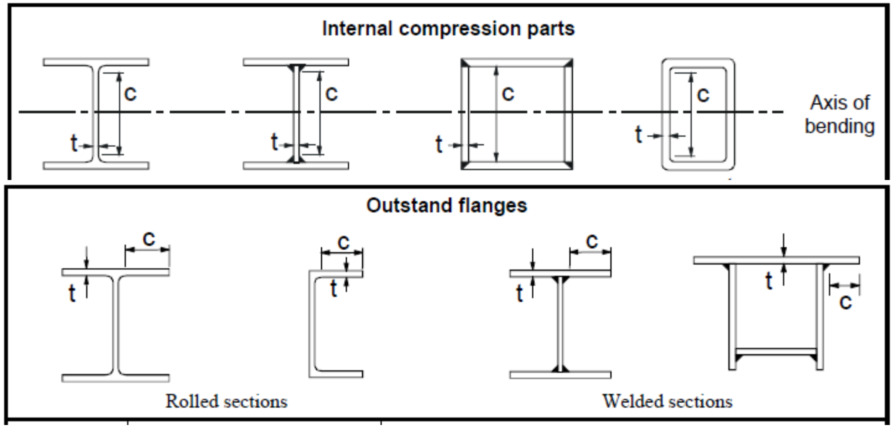
Element in compression of dissipative zones

Sufficient local ductility of members which dissipate energy in **compression** or **bending** shall be ensured by restricting the width-thickness ratio b/t according to the cross-sectional classes specified in EN 1993-1-1

Local slenderness b/t and local ductility



Requirements on cross-sectional class of dissipative elements depending on Ductility Class and reference behaviour factor

Ductility class	Reference value of behaviour factor q	Required cross-sectional class
DCM	$1{,}5 < q \leq 2$	class 1, 2 or 3
	$2 < q \leq 4$	class 1 or 2
DCH	$q > 4$	class 1

Eurocode 8: Rules for Dissipative Structures

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Element in tension of dissipative zones

For tension members or parts of members in tension, the ductility requirement of EN 1993-1-1 should be met.

Where capacity design is requested, the design plastic resistance $N_{pl,Rd}$ should be less than the design ultimate resistance of the net section at fasteners holes $N_{u,Rd}$ so the following expression should be satisfied:

$$\frac{A_{res}}{A} \geq 1{,}1 \cdot \frac{\gamma_{M2}}{\gamma_{M0}} \cdot \frac{f_{yk}}{f_{tk}}$$

A_{res} : net resistant area

A : gross area

γ_{M0} : safety factor for the resistance of the members without holes

γ_{M2} : safety factor for the resistance of the members with holes

Eurocode 8: Rules for Dissipative Structures

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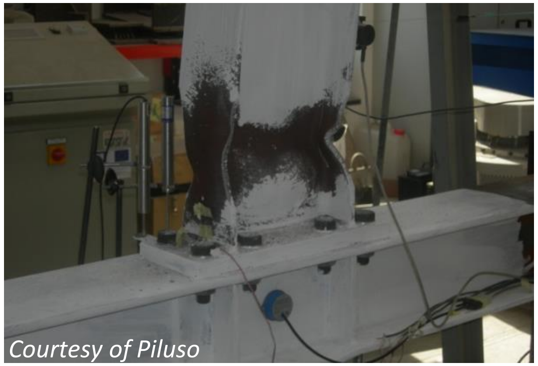
Connections

CAPACITY DESIGN PRINCIPLES

Dissipative zones may be located in the structural members or in the connections.

If dissipative zones are located in the structural members, the connections of the dissipative parts to the rest of the structure shall have sufficient overstrength to allow the development of cyclic yielding in the dissipative parts.



Courtesy of Piluso

When dissipative zones are located in the connections, the connected members shall have sufficient overstrength to allow the development of cyclic yielding in the connections

Courtesy of Piluso

Eurocode 8: Rules for Dissipative Structures

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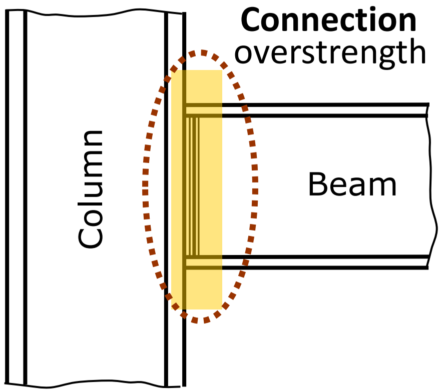
Non dissipative connections in dissipative zones

The design of connections shall be such as to satisfy the overstrength criterion.
For fillet weld or bolted non dissipative connections, the following expression should be satisfied:

The hardening factor is assumed constant

$$R_d \geq 1{,}1\,\gamma_{ov}\,R_{fy}$$

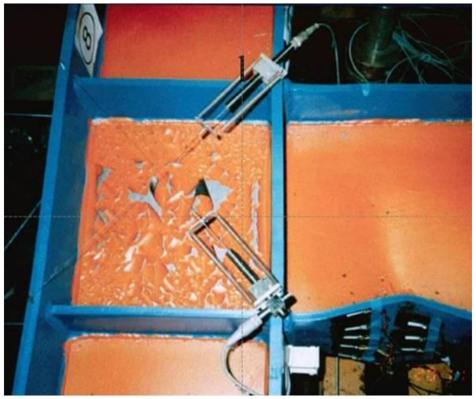


R_d : resistance of the connection in accordance with EN 1993

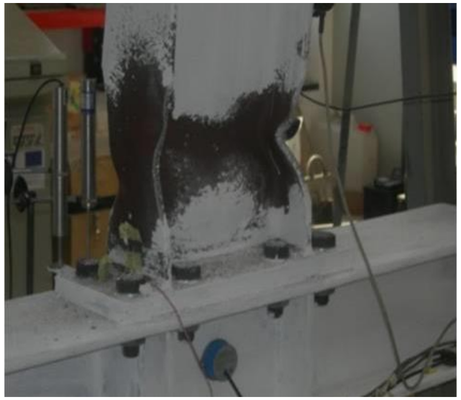
R_{fy} : plastic resistance of the connected dissipative member

γ_{ov} : overstrenght factor

The hardening factor should be related to cross section classification



WELDED CONNECTION



BOLTED CONNECTION

Eurocode 8: Rules for Dissipative Structures

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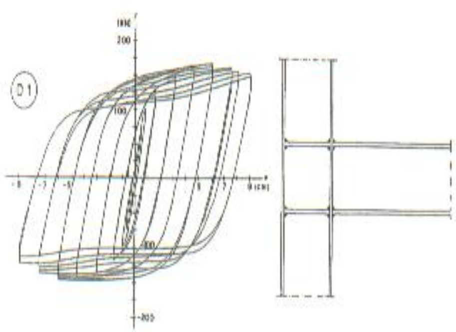
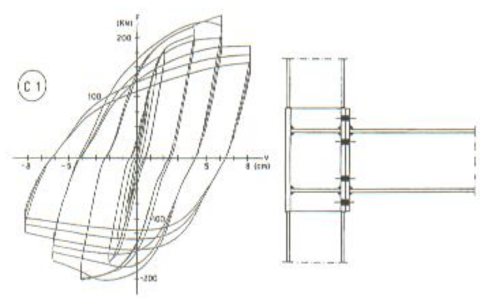
Provisions for dissipative connections

EN 1998 allows the formation of plastic hinges in the connections in case of partial-strength and/or semi-rigid joints, provided that :

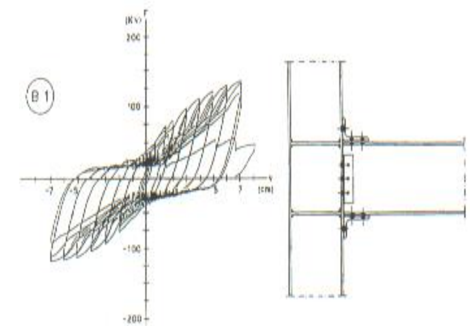
Joint cyclic rotation capacity should be at least **0.035 rad** in case of DCH or **0.025 rad** in case of DCM



Welded joint



End plate joint



Angle cleat joint

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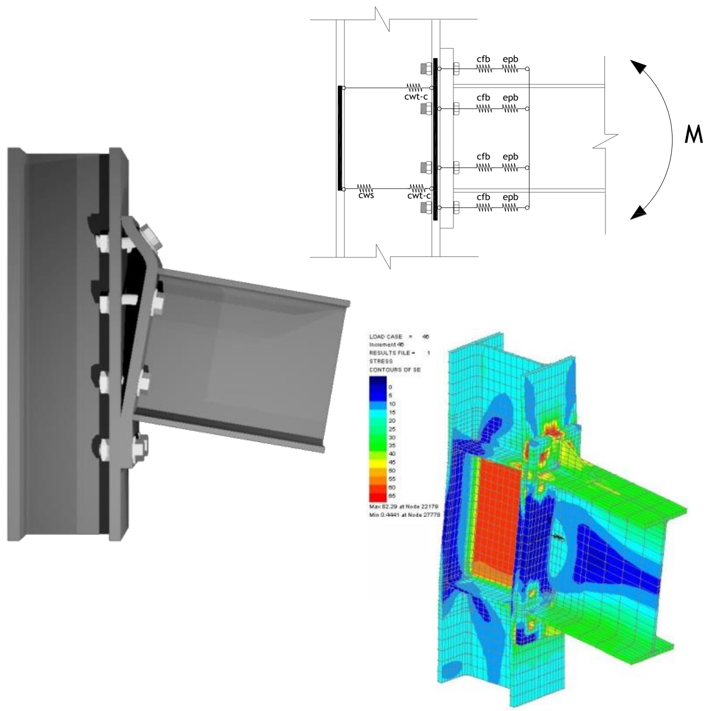
Dissipative connections

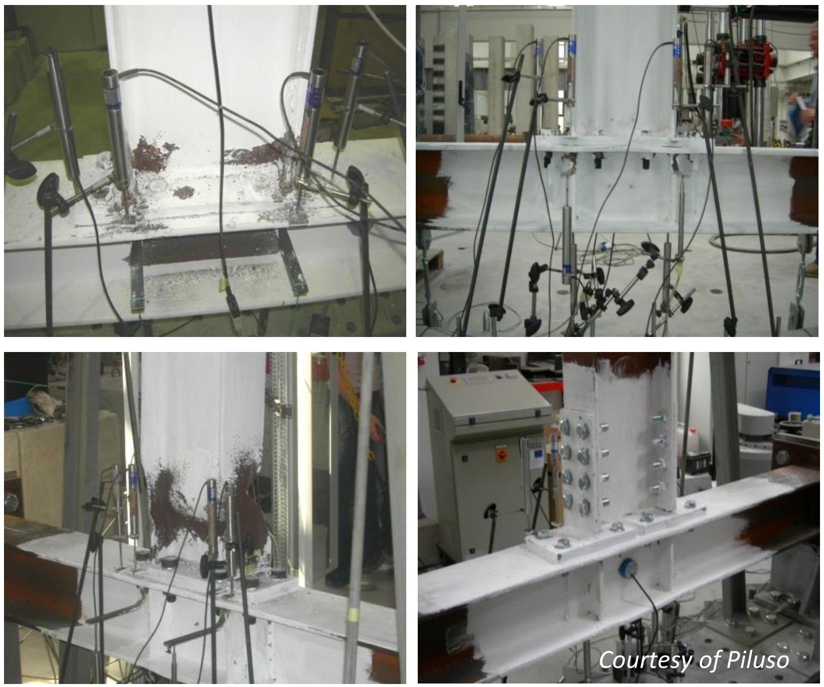
How computing Joint cyclic rotation capacity ?

EN 1998-1 (2004) requires design supported by specific experimental testing, resulting in impractical solutions within the typical time and budget constraints of real-life projects.

Joint modelling



Experimental tests



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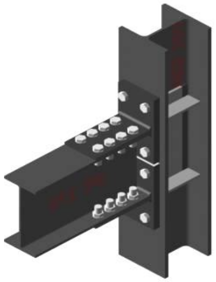
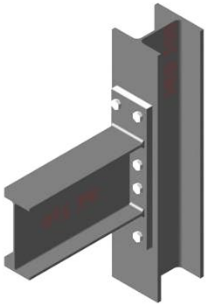
Dissipative connections

Potential upgrade

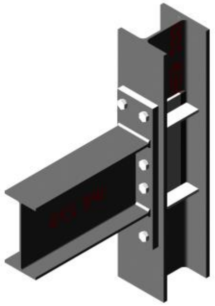


It is clear that this procedure is **unfeasible from the designer's** point of view



As an alternative to design supported by testing, the code prescribes to **find existing data on experimental test performed on similar connections in the scientific literature**, matching the typology and size of to the project.

In US and Japan this issue has been solved adopting **pre-qualified standard joints**.



Unfortunately, the standard joints adopted in the current US and Japan practice cannot be extended to Europe (different materials, section shapes and welding process)

New European research projects will be carried out to solve this issue

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Structural Typologies for Steel Buildings

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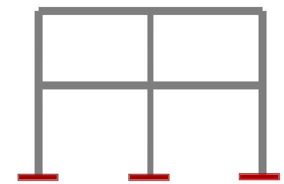
Steel buildings shall be assigned to one of the following **structural typologies** according to the behaviour of their primary resisting structure under seismic actions:

- **Moment Resisting Frames (MRF)**
- **Frames with Concentric Bracings (CBF)**
- **Frames with Eccentric Bracings (EBF)**
- **Inverted Pendulum structures**
- **Structures with concrete cores or concrete walls**
- **Moment Resisting Frames** combined with **concentric bracings**
- **Moment Resisting Frames** combined with **infills**

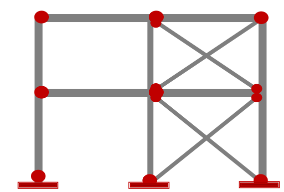
Structural Typologies for Steel Buildings

List of contents:

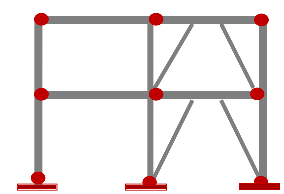
- **Moment Resisting Frames (MRF)**
horizontal forces are resisted by members acting in an essentially flexural manner



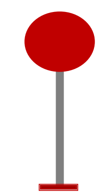
- **Frames with Concentric Bracings (CBF)**
horizontal forces are mainly resisted by members subjected to axial forces



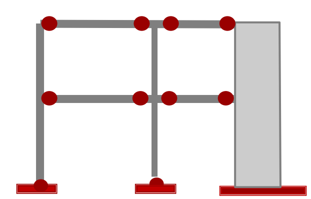
- **Frames with Eccentric Bracings (EBF)**
horizontal forces are mainly resisted by seismic links by cyclic bending or cyclic shear



- **Inverted Pendulum structures**
dissipative zones are located at the bases of columns



- **Structures with concrete cores or concrete walls**
are those in which horizontal forces are mainly resisted by these cores or walls



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Detailing Rules for MRF

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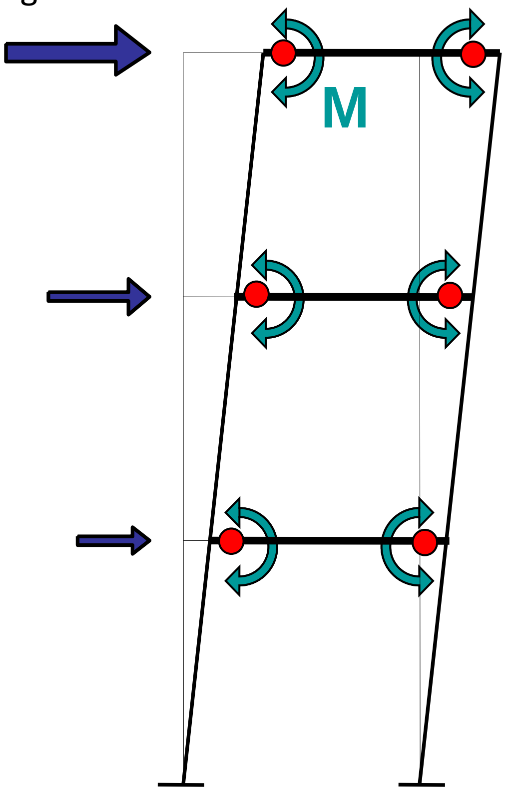
Moment Resisting Frames

The horizontal forces are mainly resisted by members acting in essentially flexural manner. Energy is thus dissipated by means of cyclic bending.



Detailing Rules for MRF

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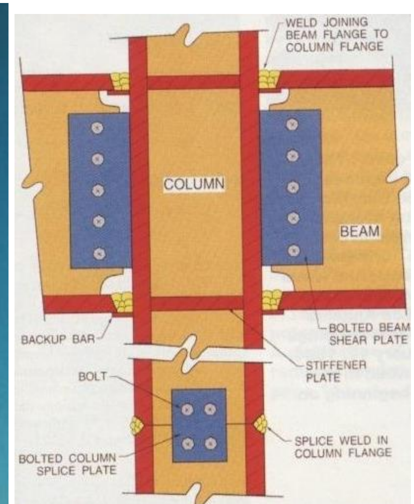
Moment Resisting Frames



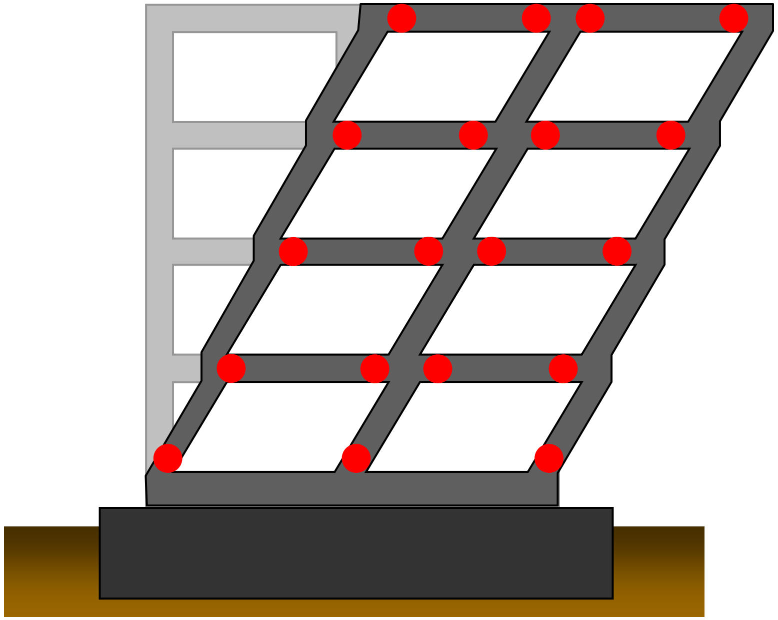
Detailing Rules for MRF

Design Concept

Global mechanism:
Plastic hinges in **beams** not in columns

The dissipative zones should be mainly located in plastic hinges in the beams or in the beams-to-columns joints

Dissipative zone in columns may be located:
- at the base of the frame
- at the top of the column in the upper story of multi storey building



Detailing Rules for MRF

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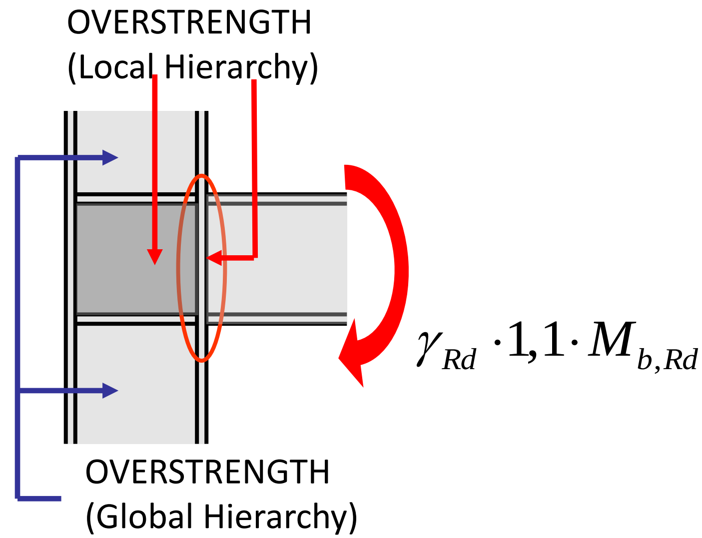
Basic Principles

Global capacity design:
Allows the formation of the global dissipative mechanisms

Local capacity design:
Allows the formation of local plastic mechanisms and ensures the transfer of full plastic forces
Concerns mainly connections



Detailing Rules for MRF

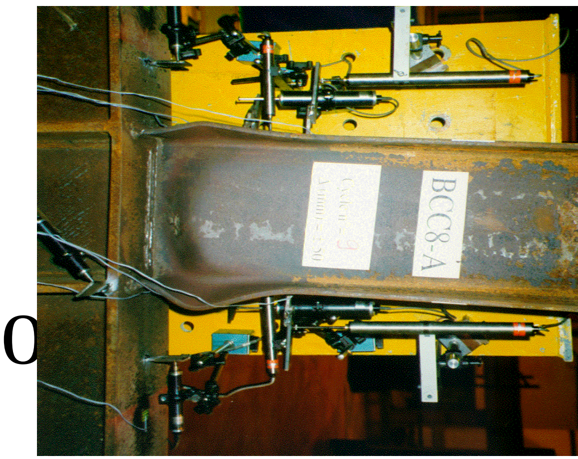
Beams

For plastic hinges in the beams it should be verified that the full plastic moment resistance and rotation capacity are not decreased by compression and shear force. At the location of the expected plastic hinge it should be verified:

$$M_{Ed} / M_{pl,Rd} \leq 1,0$$

$$N_{Ed} / N_{pl,Rd} \leq 0,15$$

$$\left(V_{Ed,G} + V_{Ed,M}\right) / V_{pl,Rd} \leq 0,50$$



where:

M_{Ed} , N_{Ed} , V_{Ed} design values of bending moment, axial force and shear force

$M_{pl,Rd}$, $N_{pl,Rd}$, $V_{pl,Rd}$ design plastic moment, axial forces, and shear resistance

$V_{Ed,G}$ design value of shear force due to non seismic actions

$V_{Ed,M}$ is the design value of the shear force due to two plastic moments $M_{pl,Rd}$ with the same sign at the location of plastic hinges

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Detailing Rules for MRF

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Columns

Columns shall be verified considering the most unfavourable combination of the axial force and the bending moment assuming the following design values:

$$N_{Ed} = N_{Ed,G} + 1{,}1 \cdot \gamma_{0V} \cdot \Omega \cdot N_{Ed,E}$$

$$M_{Ed} = M_{Ed,G} + 1{,}1 \cdot \gamma_{0V} \cdot \Omega \cdot M_{Ed,E}$$

$$V_{Ed} = V_{Ed,G} + 1{,}1 \cdot \gamma_{0V} \cdot \Omega \cdot V_{Ed,E}$$

The column shear force shall satisfy the relation:

$$V_{Ed} / V_{pl,Rd} \leq 0{,}50$$

where:

$M_{Ed,G}$, $N_{Ed,G}$, $V_{Ed,G}$ are the design values of the effect of the non seismic actions

$M_{Ed,E}$, $N_{Ed,E}$, $V_{Ed,E}$ are the design value of the effects of seismic actions

γ_{0V} is the overstrength factor

Ω is the minimum value of $\Omega_i = M_{pl,Rd,i} / M_{Ed,i}$ of all beams in which dissipative zones are located

Detailing Rules for MRF

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Capacity Design (Beam-Column)

In order to allow the development of the global collapse mechanism it has to be ensured the local capacity design.

In frame buildings the following condition should be satisfied at all beam to column joints:

$$\sum M_{Rc} \geq 1,3 \cdot \sum M_{Rb}$$

where:

ΣM_{Rc} is the sum of the design values of the moments of resistance of the columns framing the joint. The minimum value of column moments of resistance within the range of column axial forces produced by the seismic design situation should be used in the previous expression

$\Sigma M_{,Rb}$ is the sum of the design values of the moments of resistance of the beams framing the joint. When partial strength connections are used, the moments of resistance of these connections are taken into account in the calculation of $\Sigma M_{,Rb}$

Detailing Rules for MRF

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Beam-Column connections

If the structure is designed to dissipate energy in the beams, the beam to column connections of the whole frame must provide adequate overstrength to permit the formation of the plastic hinges at the ends of the beams.

So the following relationship must be achieved:

$$M_{j,Rd} \geq 1{,}1 \cdot \gamma_{0V} \cdot M_{b,pl,Rd}$$

where:

$M_{j,Rd}$ is the bending moment resistance of the connection

$M_{b,pl,Rd}$ is the bending moment resistance of the connected beam

γ_{0V} is the overstrength factor

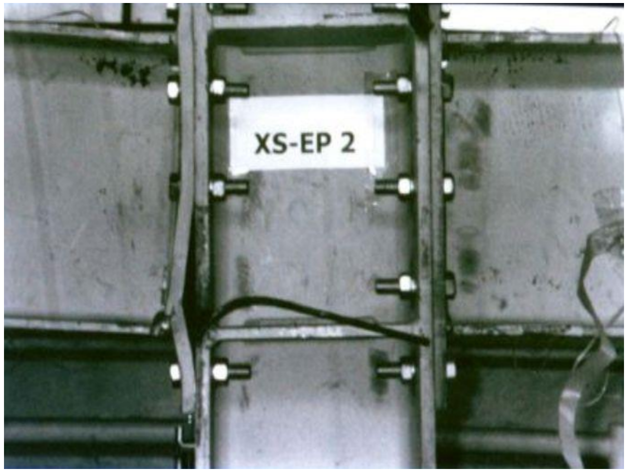
Detailing Rules for MRF

List of contents:

Nodal Web Panels

In beam to column connections the web panels of the columns must provide adequate overstrength to permit the development of the expected dissipative mechanism, avoiding their plasticization or shear buckling.

This requirement is satisfied if:



$$V_{vp,Ed} / \min\left(V_{vp,Rd} ; V_{vb,Rd}\right) < 1$$

where:

$V_{vp,Ed}$ is the design shear force in the web panel due to the action effects

$V_{vp,Rd}$ is the shear resistance of the web panel

$V_{vb,Rd}$ is the shear buckling resistance of the web panel

Detailing Rules for MRF

Column-Foundation connections

The beam to Foundation connection has to be designed in such a way to have adequate overstrength with respect to the column.

In particular, the bending moment resistance of the connection must achieve the following relationship:

$$M_{C,Rd} \geq 1{,}1 \cdot \gamma_{0V} \cdot M_{c,pl,Rd}\left(N_{Ed}\right)$$

where:

$M_{c,pl,Rd}$ is the design plastic bending moment of the column, taking into account the axial force N_{Ed} acting in the column, that give the worst condition for the base connection

γ_{0V} is the overstrength factor

Detailing Rules for MRF

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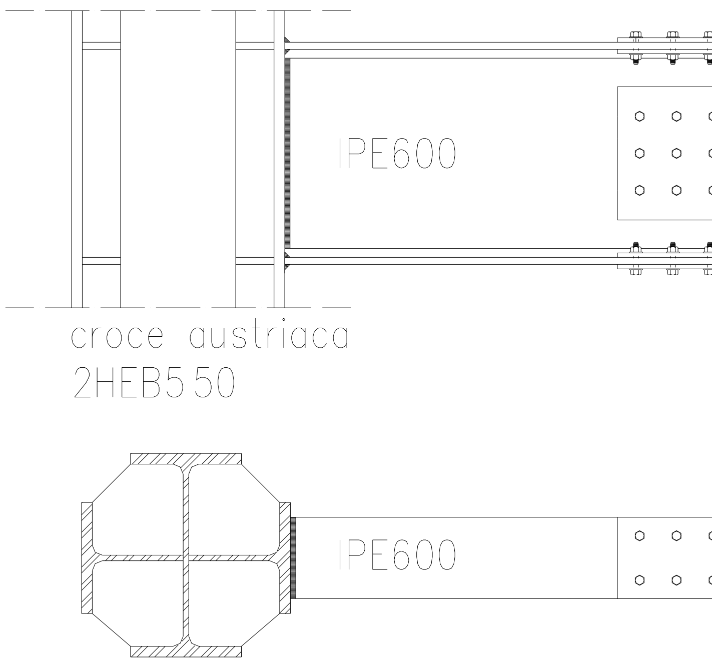
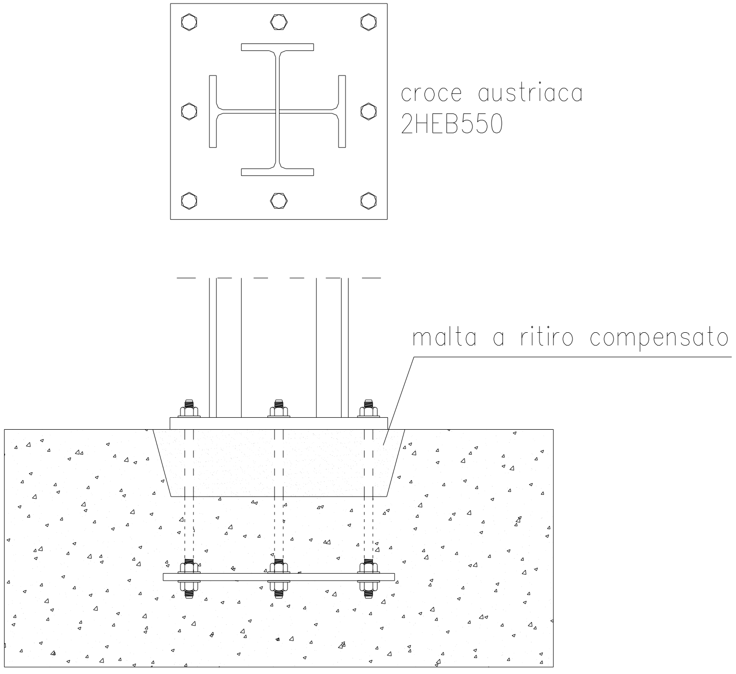
Connections: typical joints



Beam to Column



Column to Foundations

Innovative solutions for MRF

List of contents:

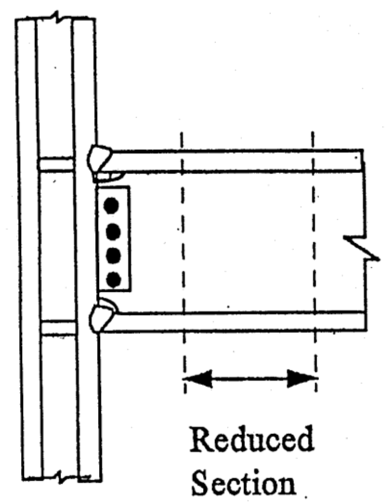
Problems with MRF		Possible innovative solutions
Excessive Overstrength of Columns	►	DOG BONE
Serviceability Limit State requirements	►	DISSIPATIVE PANELS

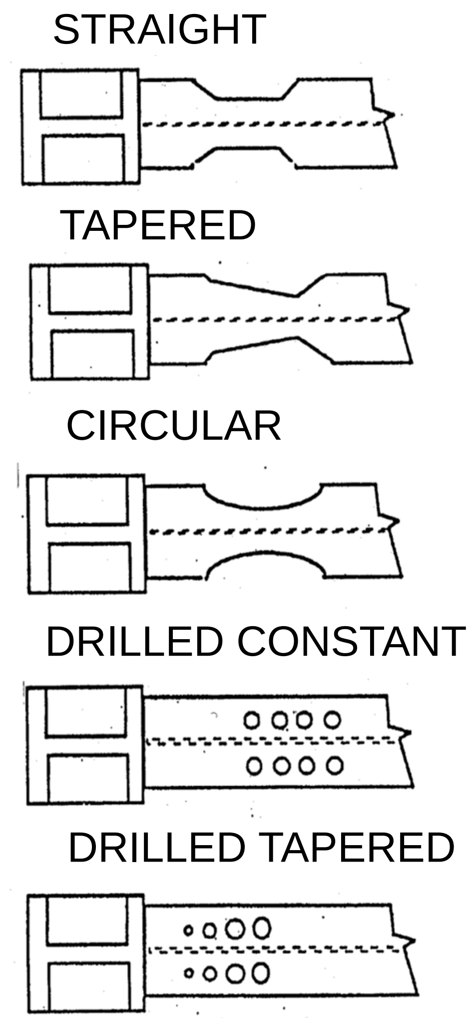
Innovative solutions for MRF

List of contents:

DOG BONE

The reduced section of the beam is intended to force the formation of the plastic hinge away from the face of the column, and it forces the large stresses and inelastic strains further into the beam.



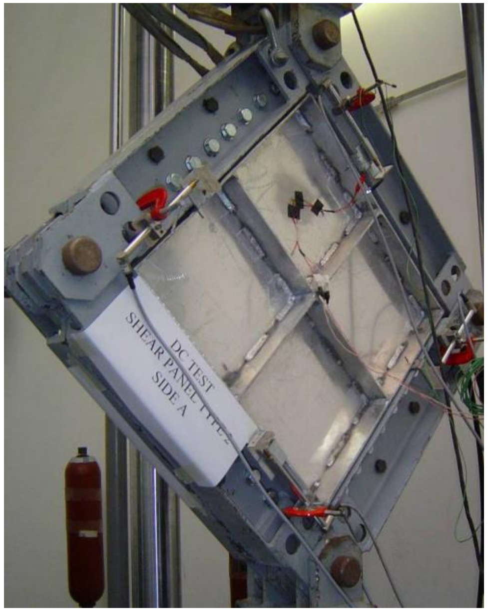


Innovative solutions for MRF

List of contents:

DISSIPATIVE PANELS

The insertion of metal shear panels in a beam and column frame, with or without stiffening, gives a good lateral resistance to the structure.
Metal shear panels are typical energy dissipation systems, based on the principle of the metal yielding, that are activated by the inter-storey drift, when the structure is subjected to horizontal forces.



Seismic Design of Steel Structures

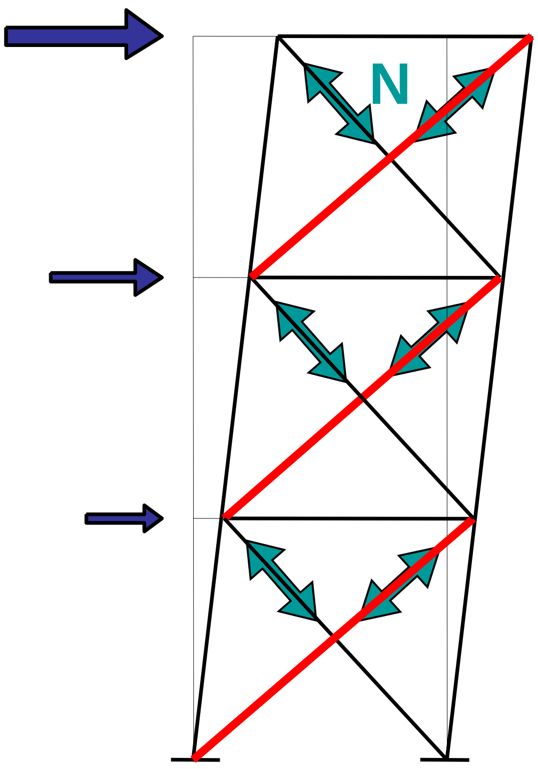
1. Benefits of steel structures
2. Design criteria for steel structures
3. Detailing rules for steel structures: CBF
4. Innovative solutions

Detailing Rules for CBF

List of contents:

Concentric Braced Frames

The horizontal forces are resisted by diagonal members acting in tension.



Detailing Rules for CBF

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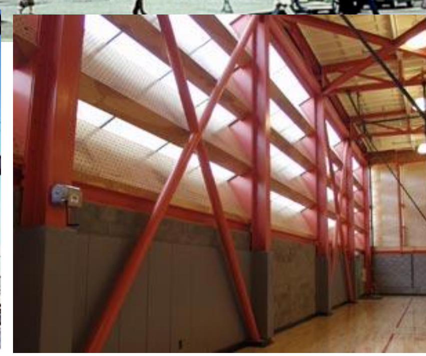
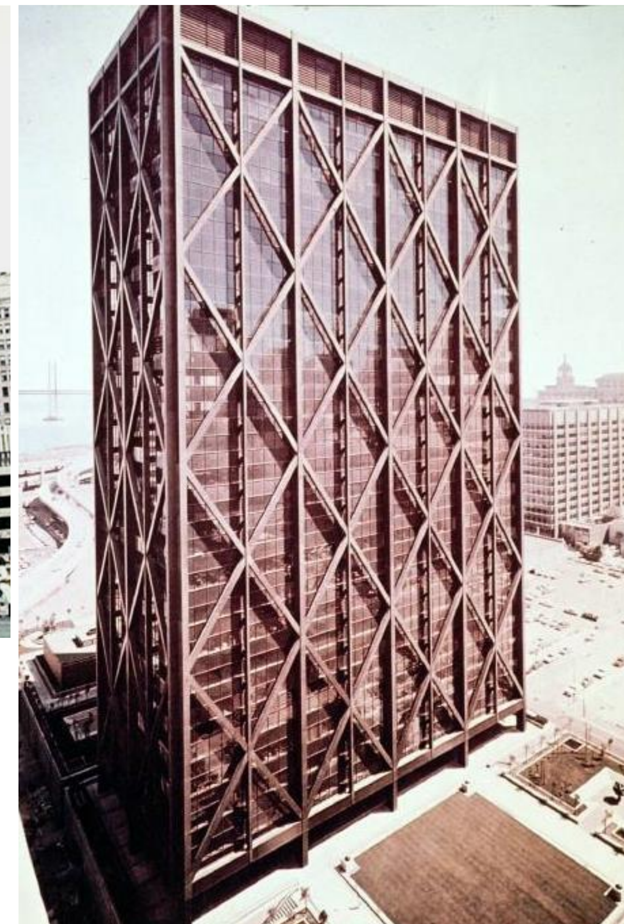
Concentric Braced Frames



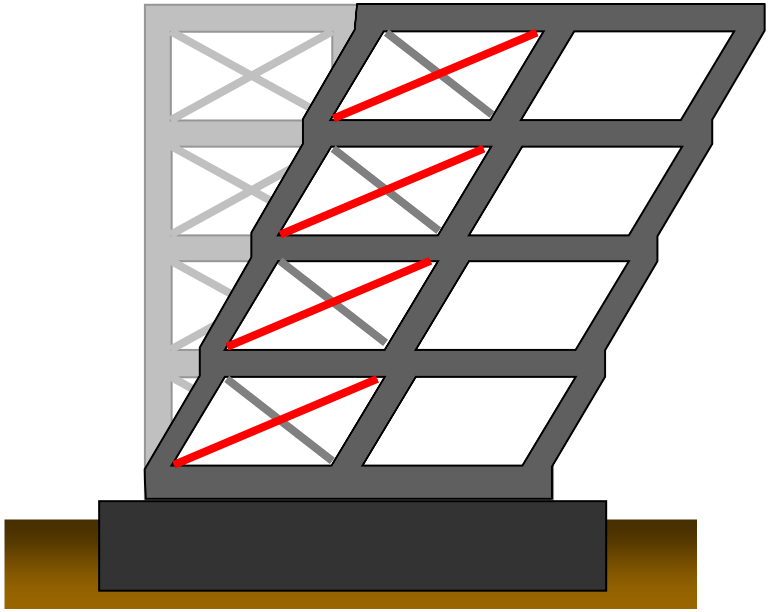


Detailing Rules for CBF

Design Concept

Global mechanism:
The dissipative elements are the bracings in tension.

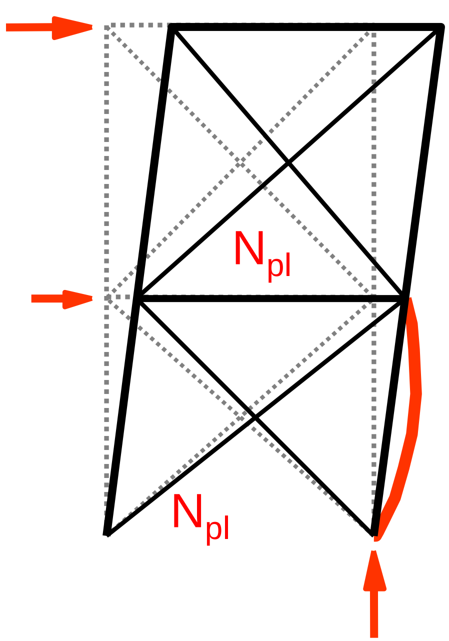
Concentric braced frames shall be designed so that yielding of the diagonals in tension will take place before failure of the connections and before yielding or buckling of the beams or columns.



Detailing Rules for CBF

Basic Principles

Global capacity design:
Allows the formation of the global dissipative mechanisms



Detailing Rules for CBF

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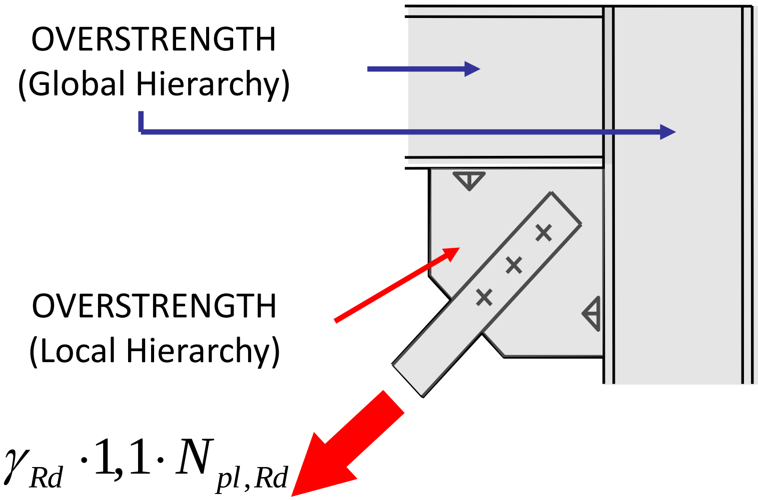
Basic Principles

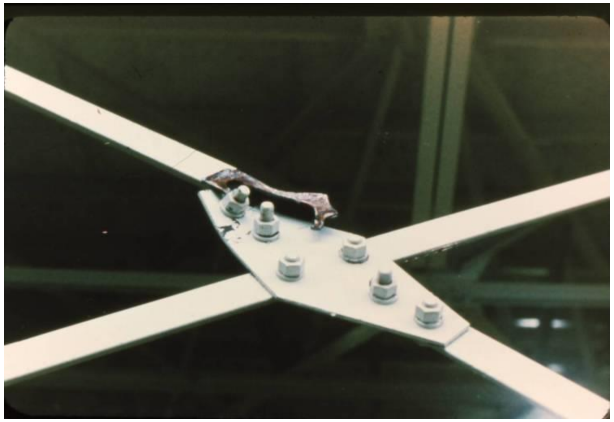
Local capacity design:

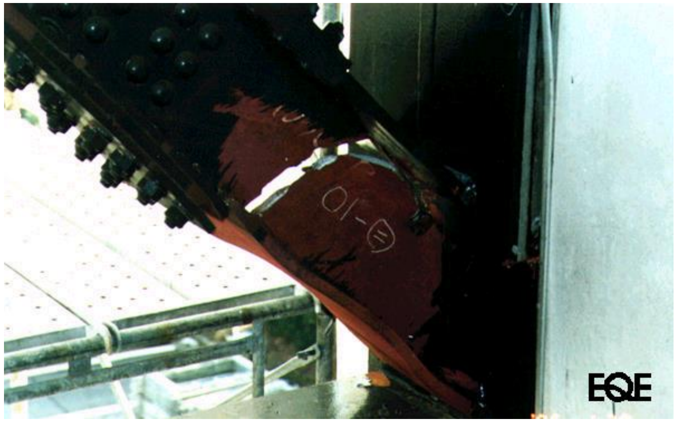
Allows the formation of local plastic mechanisms and ensures the transfer of full plastic forces

Concerns mainly connections



$$\gamma_{Rd} \cdot 1{,}1 \cdot N_{pl,Rd}$$





Detailing Rules for CBF

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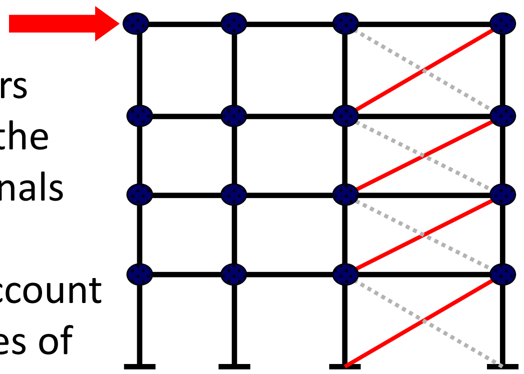
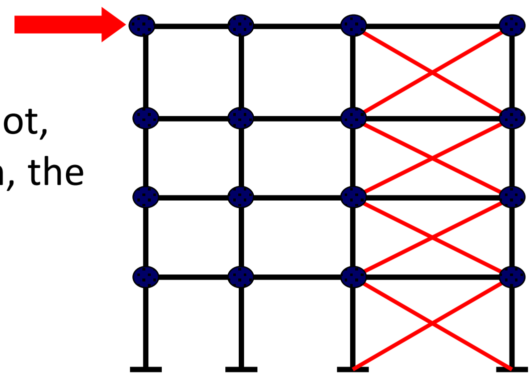
Diagonal Bracings

Modeling:
Since horizontal forces are resisted by diagonal members acting in tension, applying the capacity design criteria, the contribution of the resistance of the compressed diagonals has to be neglected.
Anyway, the compressed diagonals can be taken into account to calculate the natural frequencies, the vibration modes of the structure, and the design seismic forces, granted that the compressed diagonals stability is checked.



Resistance:
Whether the compressed diagonals are considered or not, when occur the buckling of the diagonal in compression, the diagonal members in tension have to be checked to be sufficient to resist the design seismic forces determined before.



Detailing Rules for CBF

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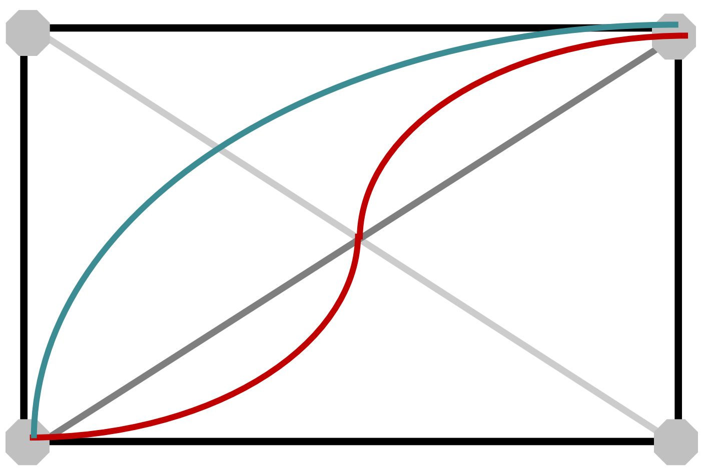
Diagonal Bracings

Slenderness :
The non-dimensional slenderness of diagonals is the ratio between the geometrical slenderness λ and the elastic critical slenderness λ_y



IN PLANE
$$\lambda_{ip} = \frac{0.5 \cdot L_d}{\rho_{ip}}$$

OUT OF PLANE
$$\lambda_{op} = \frac{L_d}{\rho_{op}}$$

NON-DIMENSIONAL SLENDERNESS $\bar{\lambda} = \frac{\lambda}{\lambda_y}$ with $\lambda_y = \pi\sqrt{\frac{E}{f_y}}$

Detailing Rules for CBF

Diagonal members

In structures of more than two storeys the non-dimensional slenderness of diagonal members should be:

$$1{,}3 \leq \bar{\lambda} \leq 2$$

in frames with X bracings.

The overstrength factor to apply the capacity design criteria is:

$$\Omega_i = \frac{N_{pl,Rd,i}}{N_{Ed,i}}$$

Calculated over all the diagonals of the braced system. In order to satisfy a homogeneous dissipative behaviour of the **diagonals**, it should be checked that the maximum value does not differ from the minimum value by more than **25%**.

Detailing Rules for CBF

List of contents:

Beams and Columns

Beams and columns with axial forces should meet the following minimum resistance requirement:

$$N_{Ed} / N_{pl,Rd}(M_{Ed}) \leq 1$$

where:

$$N_{Ed} = N_{Ed,G} + 1{,}1 \cdot \gamma_{0V} \cdot \Omega \cdot N_{Ed,E}$$

and $N_{pl,Rd}$ is the design buckling resistance of the beam or the column in accordance with EN 1993, taking into account the interaction of the buckling resistance with the bending moment defined as its design value in the seismic design situation:

$$M_{Ed} = M_{Ed,G} + 1{,}1 \cdot \gamma_{0V} \cdot \Omega \cdot M_{Ed,E}$$

Detailing Rules for CBF

Connections

The connections of diagonal members to the structure have to provide adequate overstrength to permit the development of the expected dissipative mechanism.

For fillet weld or bolted non dissipative connections, the following expression should be satisfied:

$$R_{j,d} \geq \gamma_{0V} \cdot 1{,}1 \cdot R_{pl,Rd} = R_{U,Rd}$$

where:

$R_{j,d}$ is the design resistance of the connection;

$R_{pl,Rd}$ is the plastic resistance of the connected dissipative member based on the design yield stress of the material

$R_{U,Rd}$ is the upper bound of the plastic resistance of the connected dissipative member;

γ_{0V} is the overstrength factor

Detailing Rules for CBF

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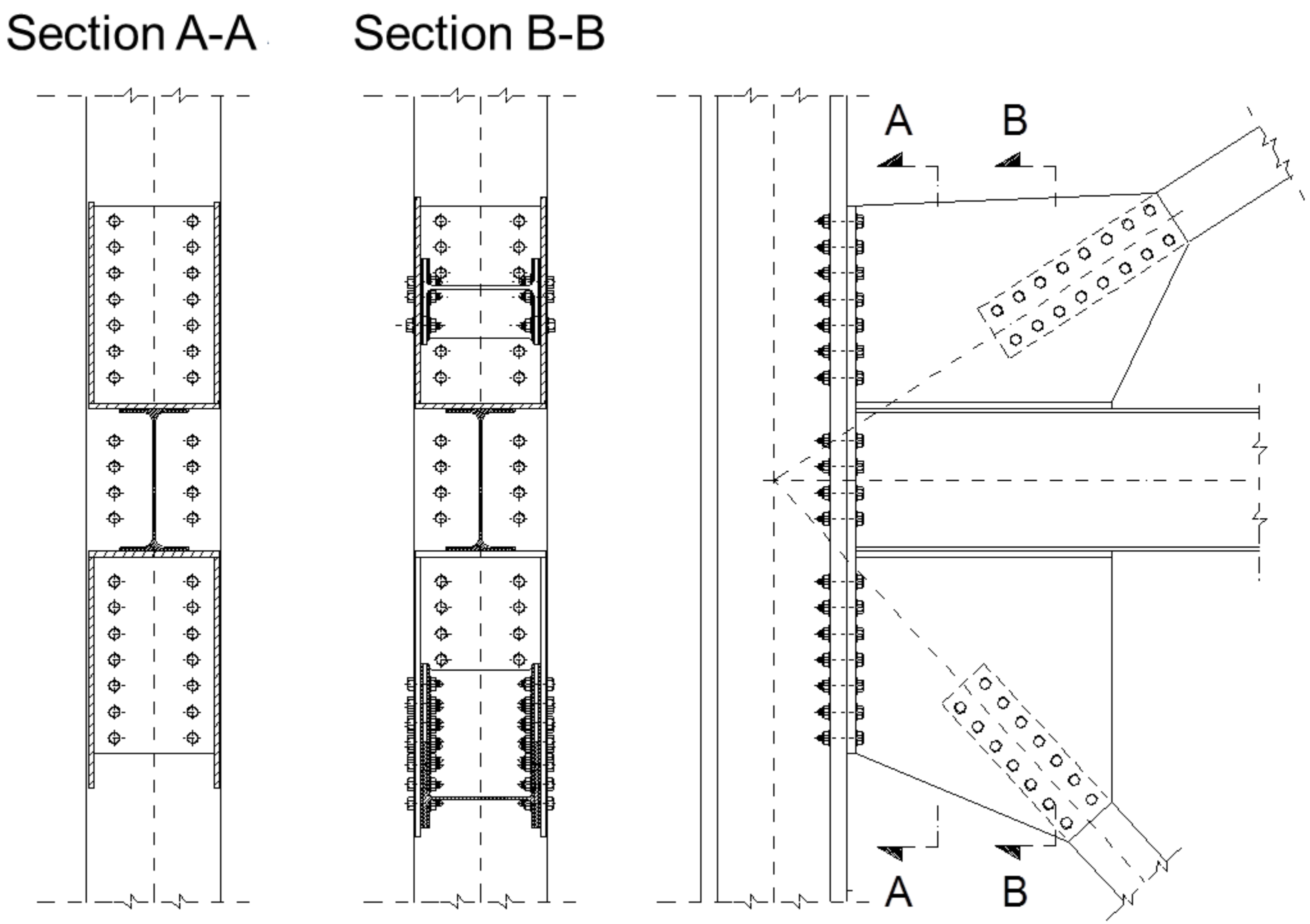
Connections: typical joint



Detailing Rules for CBF - V bracings

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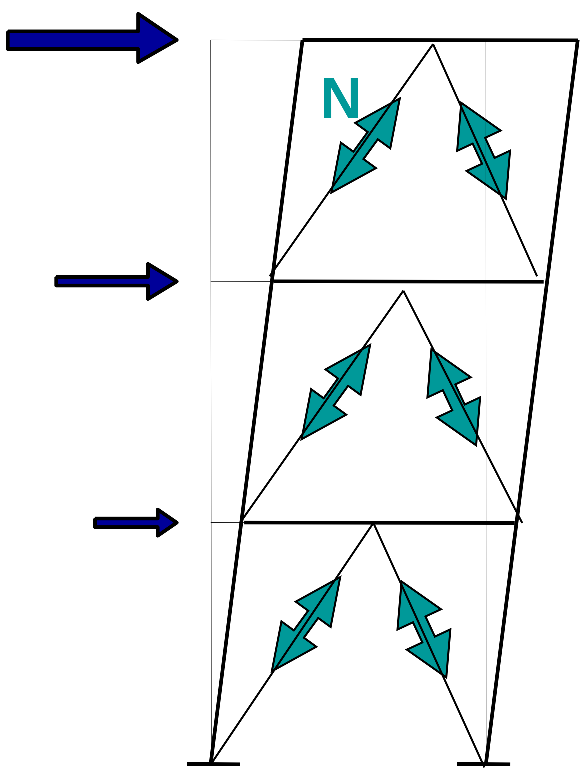
SUSCOS

Sustainable Constructions under Natural Hazards and Catastrophic Events

Concentric Braced Frames – V bracings

The horizontal forces are resisted by diagonal members acting in tension.



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Concentric Braced Frames – V bracings

Detailing Rules for CBF - V bracings

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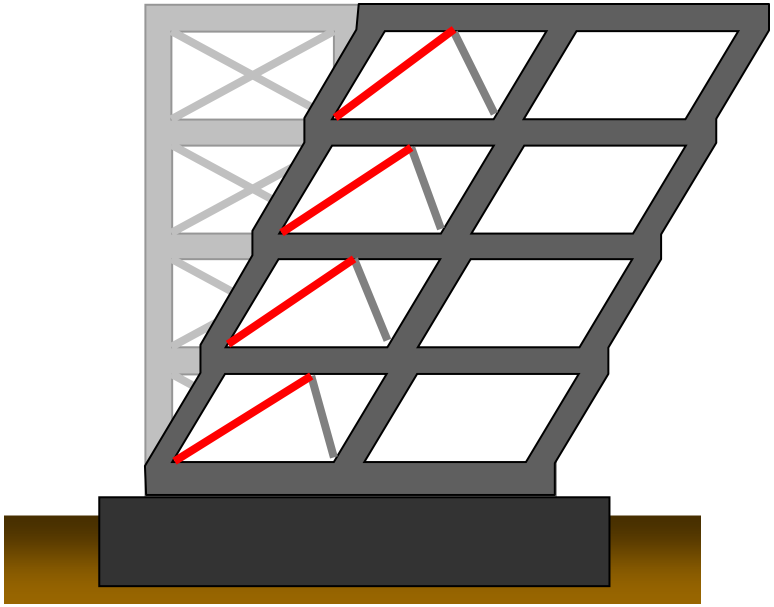
Design Concept

Global mechanism:
The dissipative elements are the bracings in tension.

Concentric braced frames shall be designed so that yielding of the diagonals in tension will take place before failure of the connections and before yielding or buckling of the beams or columns.

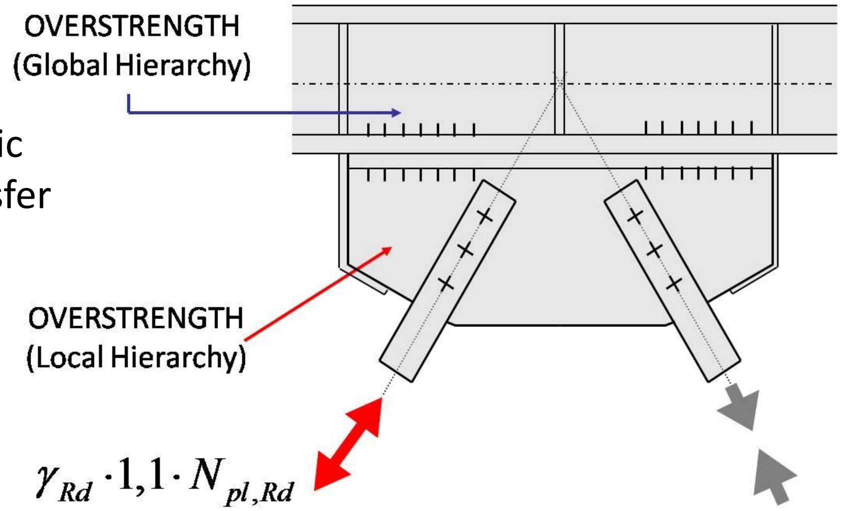


Detailing Rules for CBF - V bracings

Basic Principles

Global capacity design:
Allows the formation of the global dissipative mechanisms

Local capacity design:
Allows the formation of local plastic mechanisms and ensures the transfer of full plastic forces
Concerns mainly connections



Detailing Rules for CBF - V bracings

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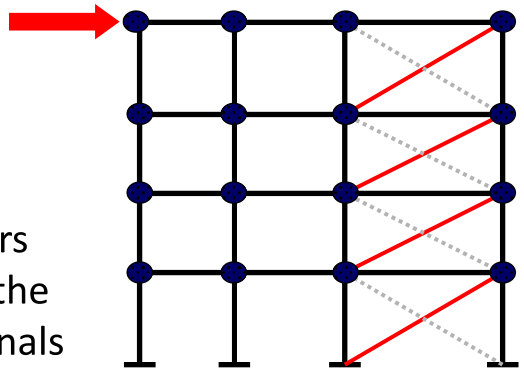
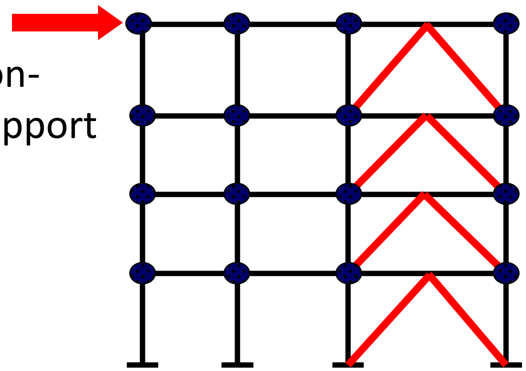
Diagonal Bracings



Modeling:

Since horizontal forces are resisted by diagonal members acting in tension, applying the capacity design criteria, the contribution of the resistance of the compressed diagonals has to be neglected.

In frames with **V bracings**, both the tension and compression diagonals shall be taken into account.
Moreover, the beams should be designed to resist all non-seismic actions without considering the intermediate support given by the diagonals.



Detailing Rules for CBF - V bracings

List of contents:

Diagonal Bracings

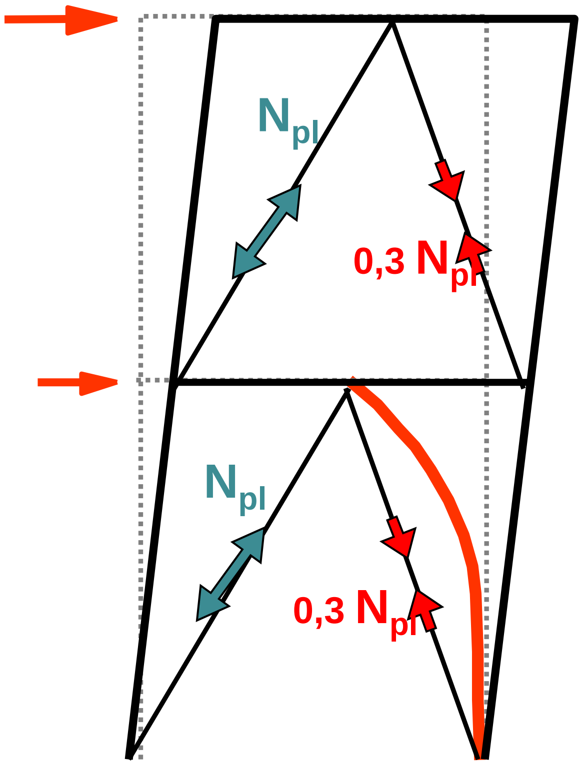
During the design it should be taken into account both the tension and compression diagonals.

During the safety checks, should be considered the buckling of the diagonal in compression.

The unbalanced vertical seismic action effect applied to the beam by the braces after the buckling of the compression diagonal is calculated considering:

$N_{pl,Rd}$ in tension diagonals

$\gamma_{pb}\, N_{pl,Rd}$ in compression diagonals



with γ_{pb} = 0,30 is the factor used for the estimation of the post buckling resistance of diagonals in compression.

Detailing Rules for CBF - V bracings

List of contents:

Diagonal members

In structures of more than two storeys the non-dimensional slenderness of diagonal members should be:

$$\bar{\lambda} \leq 2$$

in frames with V bracings

The overstrength factor to apply the capacity design criteria is:

$$\Omega_i = \frac{N_{pl,Rd,i}}{N_{Ed,i}}$$

Calculated over all the diagonals of the braced system. In order to satisfy a homogeneous dissipative behaviour of the **diagonals**, it should be checked that the maximum value does not differ from the minimum value by more than **25%**.

Detailing Rules for CBF - V bracings

Beams and Columns

Beams and columns with axial forces should meet the following minimum resistance requirement:

$$N_{Ed} / N_{pl,Rd}(M_{Ed}) \leq 1$$

where:

$$N_{Ed} = N_{Ed,G} + 1{,}1 \cdot \gamma_{0V} \cdot \Omega \cdot N_{Ed,E}$$

and $N_{pl,Rd}$ is the design buckling resistance of the beam or the column in accordance with EN 1993, taking into account the interaction of the buckling resistance with the bending moment defined as its design value in the seismic design situation:

$$M_{Ed} = M_{Ed,G} + 1{,}1 \cdot \gamma_{0V} \cdot \Omega \cdot M_{Ed,E}$$

Detailing Rules for CBF - V bracings

Connections

The connections of diagonal members to the structure have to provide adequate overstrength to permit the development of the expected dissipative mechanism.

For fillet weld or bolted non dissipative connections, the following expression should be satisfied:

$$R_{j,d} \geq \gamma_{0V} \cdot 1{,}1 \cdot R_{pl,Rd} = R_{U,Rd}$$

where:

$R_{j,d}$ is the design resistance of the connection;

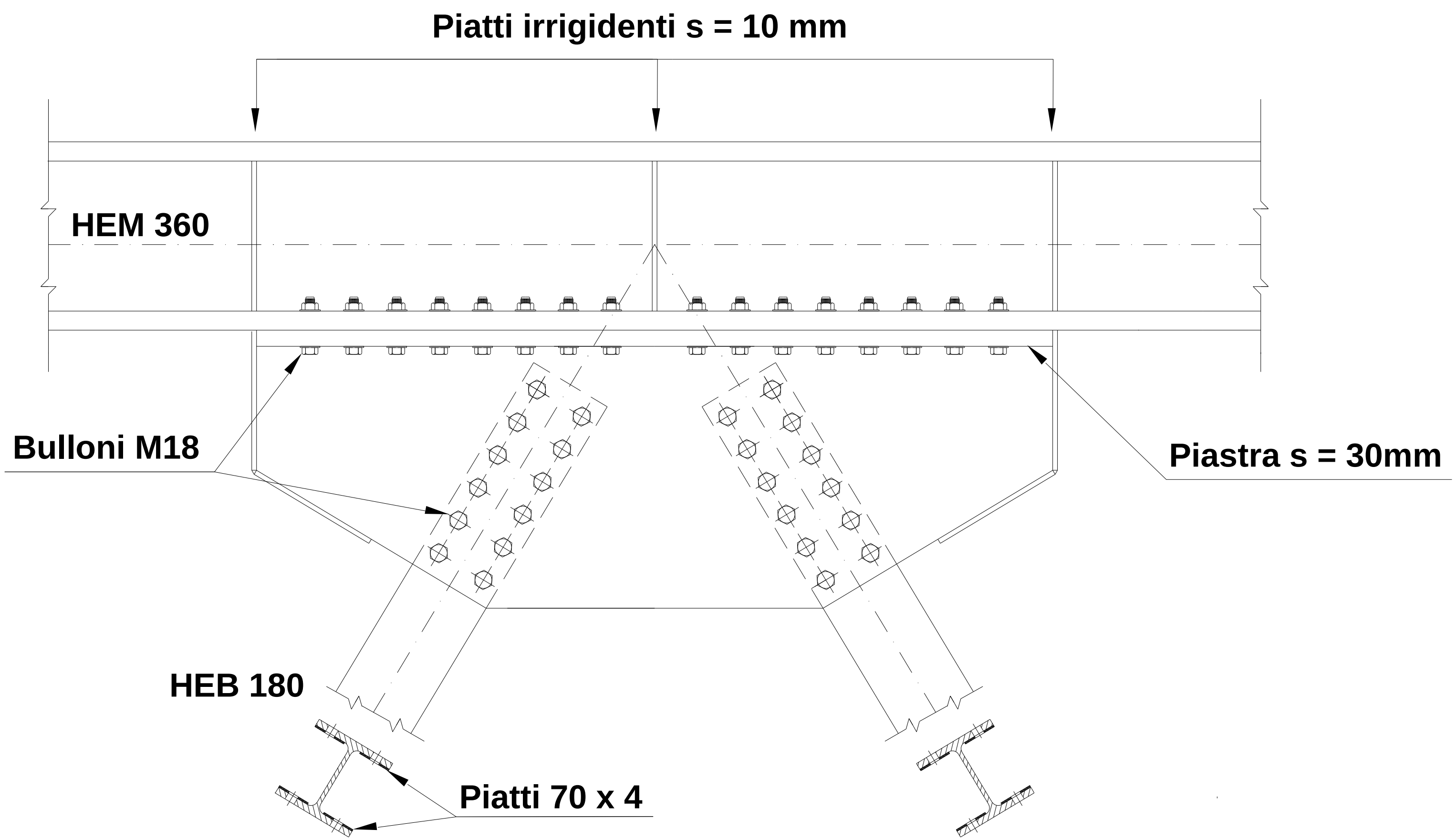
$R_{pl,Rd}$ is the plastic resistance of the connected dissipative member based on the design yield stress of the material

$R_{U,Rd}$ is the upper bound of the plastic resistance of the connected dissipative member;

γ_{0V} is the overstrength factor

Detailing Rules for CBF - V bracings

Connections: typical joint



Innovative solutions for CBF

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Sustainable Constructions under Natural Hazards and Catastrophic Events

Innovative solutions

• developed in the field of seismic design and retrofitting to improve the performances of traditional CBF

• based on the weakening of the end sections of some elements, to induce the plasticization in specific parts of the structure, or on the insertion of special devices

• advantage of simple substitution of damaged parts after the earthquake

- SEMI-RIGID JOINTS
- BUCKLING RESTRAINED BRACES (BRB)
- REDUCED SECTION SOLUTION (RSS)

Innovative solutions for CBF

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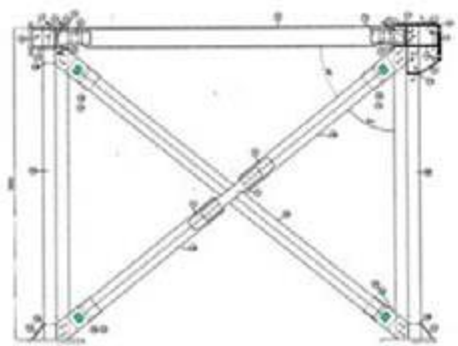
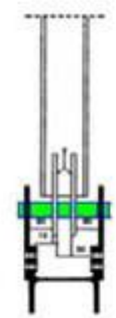
SEMI-RIGID JOINTS

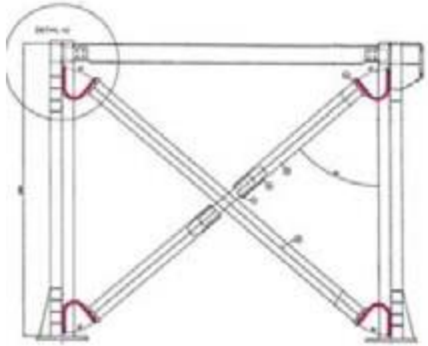
The solution consists in replacing traditional joints with special dissipative joints. They are semi-rigid joints (pin-connections and U-connections) designed with a lower resistance with respect to the one corresponding to the diagonal member instability, to avoid the brace yielding.

Pin-connection





U-connection



Innovative solutions for CBF

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BUCKLING RESTRAINED BRACES (BRB)

They are special devices able to decouple the resistance to the axial load from the buckling resistance. The cyclic response of the BRB is more stable than the response of a conventional concentric bracing.

Advantages: substitution after the damage is simpler, devices can be simply hided in the external walls, so it can be used successfully also for retrofitting of existing buildings.

The internal core

The external tube

Innovative solutions for CBF

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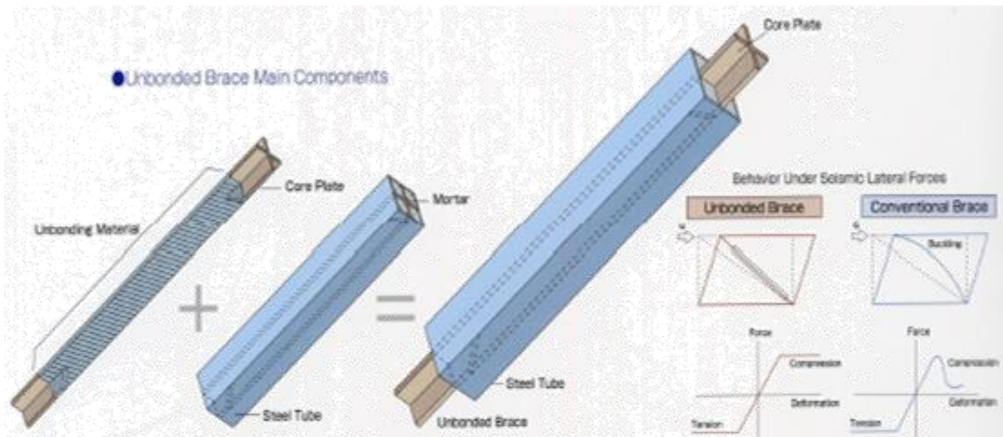
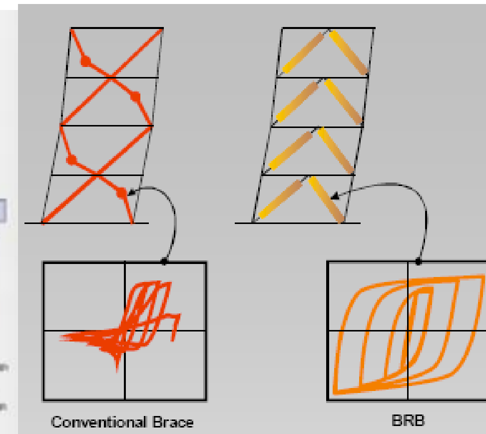
BUCKLING RESTRAINED BRACES (BRB)





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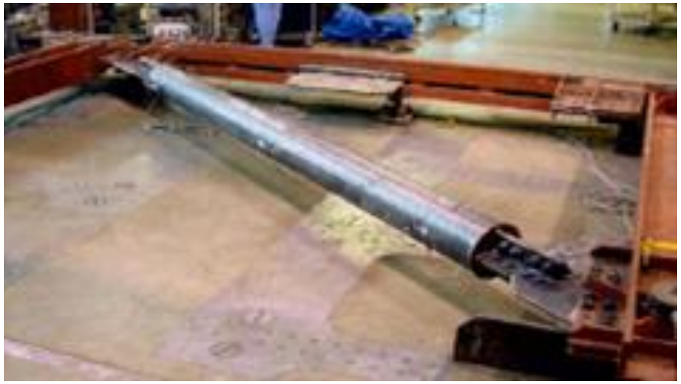
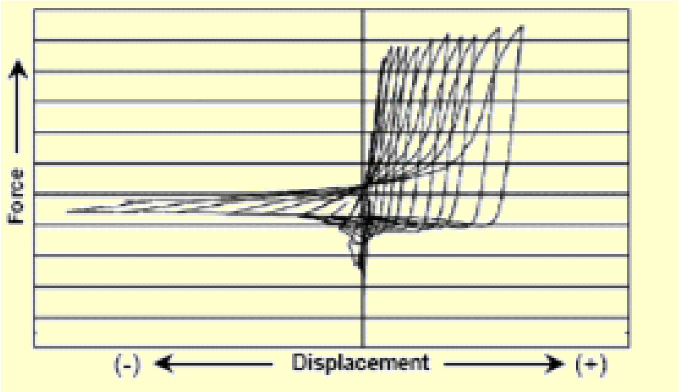
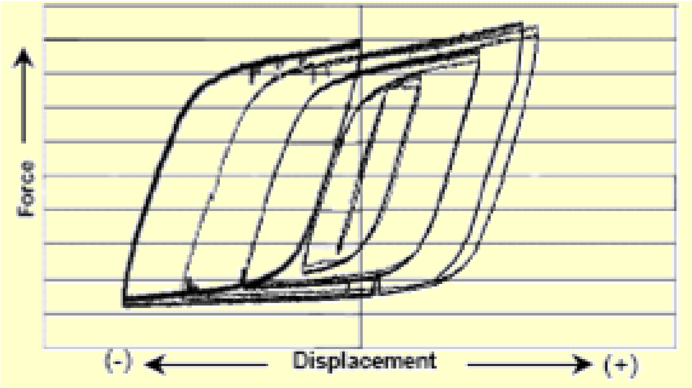
BUCKLING RESTRAINED BRACES (BRB)







Tremblay et al. (2006)

Innovative solutions for CBF

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BUCKLING RESTRAINED BRACES (BRB)

Innovative solutions for CBF

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BUCKLING RESTRAINED BRACES (BRB)

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BUCKLING RESTRAINED BRACES (BRB)

Innovative solutions for CBF

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REDUCTION SECTION SOLUTION (RSS)

The strategy used is the weakening of the end sections of diagonal members, obtained through holes (to induce plastic deformations of these zones).
This solution was created to solve a problem of CBF structures, due to seismic codes, that prescribe slenderness limits that cause over-dimensioning of diagonal members, resulting in non-global collapse mechanisms.
Advantage: the global ductility of the structure is increased.

The internal core

The external tube

Seismic Design of Steel Structures

1. Benefits of steel structures
2. Design criteria for steel structures
3. Detailing rules for steel structures: EBF
4. Innovative solutions

Detailing Rules for EBF

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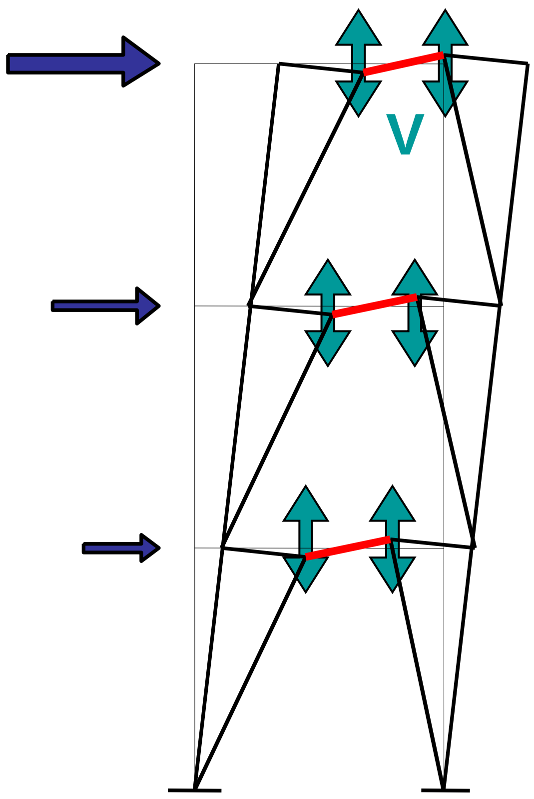
Sustainable Constructions under Natural Hazards and Catastrophic Events

Eccentric Braced Frames

The horizontal forces are resisted by specific elements called “**seismic links**” acting in bending and/or shear.



Detailing Rules for EBF

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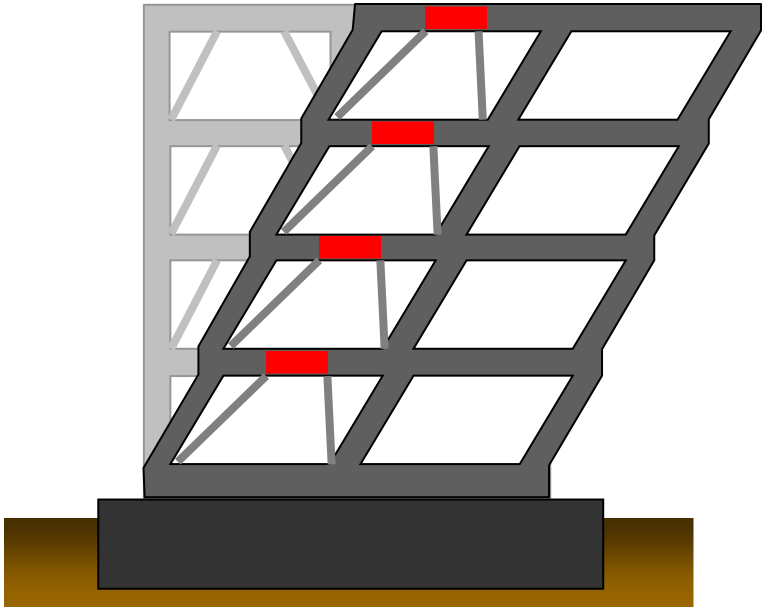
Eccentric Braced Frames

Detailing Rules for EBF

Design Concept

Global mechanism:
The dissipative elements are the seismic links.

Frames with eccentric bracings shall be designed so that specific elements or parts of elements called “seismic links” are able to dissipate energy by the formation of plastic bending and/or plastic shear mechanisms, before failure of the connections and before yielding or buckling of the beams, columns and diagonal members.

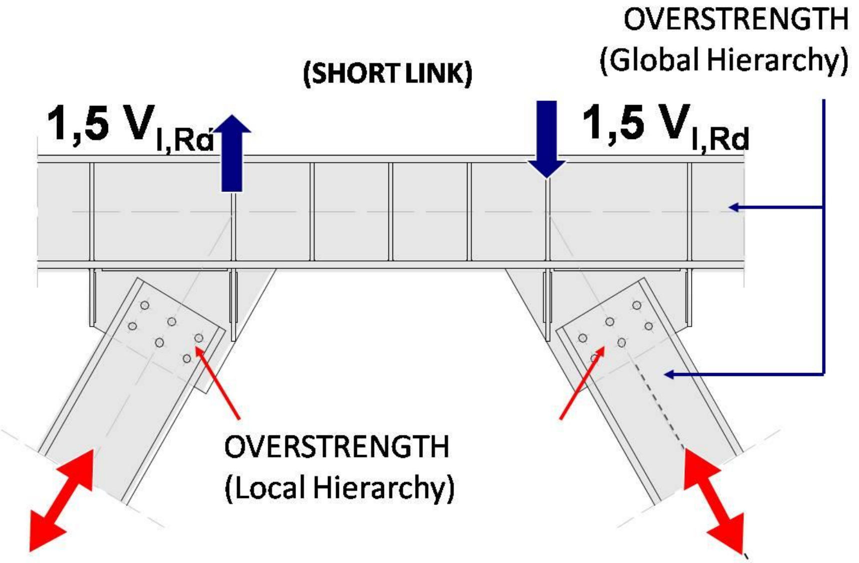


Detailing Rules for EBF

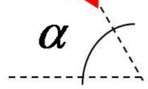
Basic Principles

Global capacity design:
Allows the formation of the global dissipative mechanisms

Local capacity design:
Non dissipative elements and connections are designed with adequate overstrength respect to dissipative zones (link)



$$N_{Ed,G} + \gamma_{Rd} \cdot 1{,}1 \cdot 1{,}5 \cdot V_{l,Rd} \cdot sen\alpha$$



Detailing Rules for EBF

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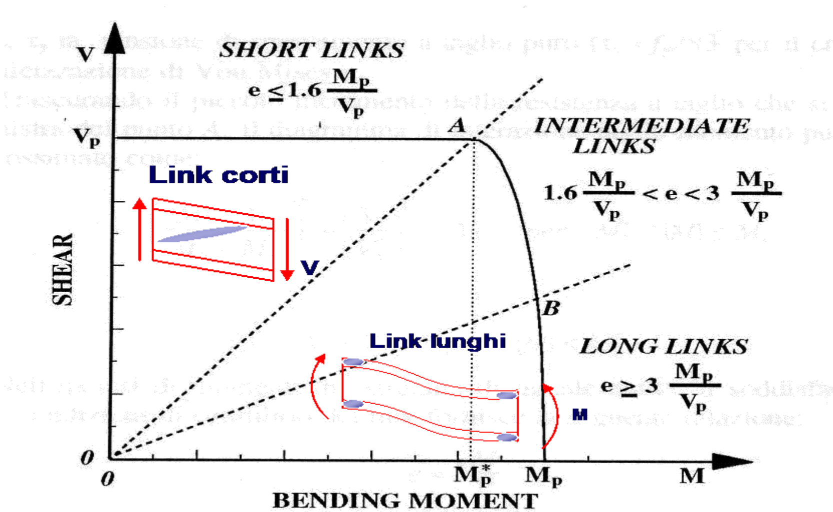
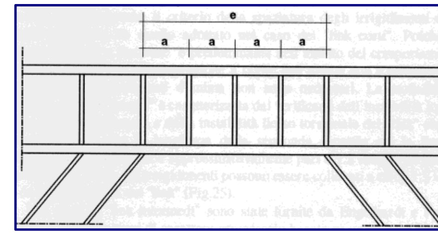
Classification of seismic links

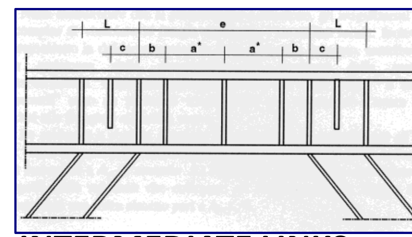
Seismic links are classified into 3 categories according to the type of plastic mechanism developed:



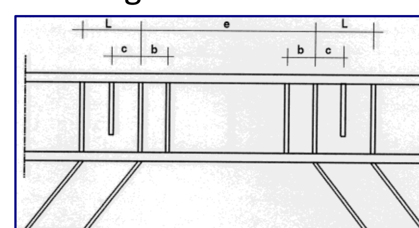
SHORT LINKS
dissipate energy by yielding essentially in shear



LONG LINKS
dissipate energy by yielding essentially in bending



INTERMEDIATE LINKS
the plastic mechanism involves bending and shear



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Sustainable Constructions under Natural Hazards and Catastrophic Events

Detailing Rules for EBF

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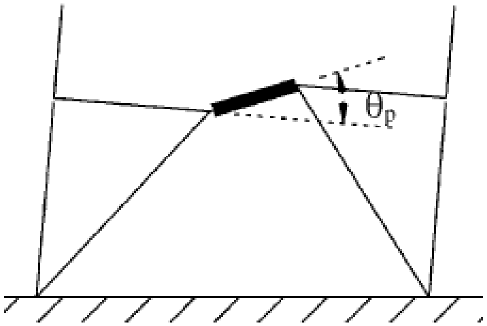
Sustainable Constructions under Natural Hazards and Catastrophic Events

Classification of seismic links

In designs where equal moments would form simultaneously at both ends of the link (see Figure), links may be classified according to the length e.

For I sections, the categories are:



SHORT LINKS:

$$e \leq 1,6\frac{M_{p,link}}{V_{p,link}}$$

LONG LINKS:

$$e \geq 3,0\frac{M_{p,link}}{V_{p,link}}$$

INTERMEDIATE LINKS:

$$1,6\frac{M_{p,link}}{V_{p,link}} < e < 3,0\frac{M_{p,link}}{V_{p,link}}$$

where $M_{p.link}$ and $V_{p,link}$ are the design bending moment and shear resistance of the links and they are calculated here for I sections :

$$M_{p,link} = f_y \cdot b \cdot t_f \cdot \left(d - t_f\right)$$

$$V_{p,link} = \frac{f_y}{\sqrt{3}} \cdot t_w \cdot \left(d - t_f\right)$$

Detailing Rules for EBF

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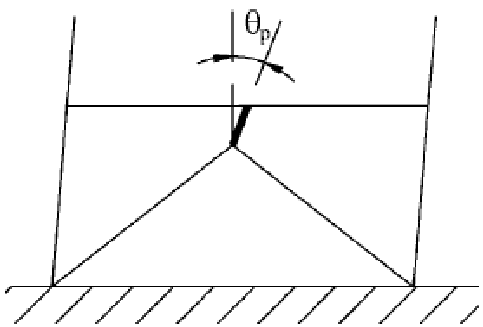
Classification of seismic links

In designs where only one plastic hinge would form at one end of the link (see Figure), links may be classified according to the length e.
For I sections, the categories are:



SHORT LINKS:

$$e \le 0,8(1+\alpha)\frac{M_{p,link}}{V_{p,link}}$$

LONG LINKS:

$$e \ge 1,5(1+\alpha)\frac{M_{p,link}}{V_{p,link}}$$

INTERMEDIATE LINKS:

$$0,8(1+\alpha)\frac{M_{p,link}}{V_{p,link}} < e < 1,5(1+\alpha)\frac{M_{p,link}}{V_{p,link}}$$

where

α is the ratio between the smaller and the greater bending moments at the ends of the link in the seismic design situation;

$M_{p.link}$ and $V_{p,link}$ are the design bending moment and shear resistance of the links and they are calculated here for I sections :

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Seismic Links

If $N_{Ed}/N_{pl,Rd} \leq 0{,}15$ the design resistance of the link should satisfy both of the following relationships at both ends of the link:

$$V_{Ed} \leq V_{p,link}$$

$$M_{Ed} \leq M_{p,link}$$

If $N_{Ed}/N_{pl,Rd} > 0{,}15$ the design resistance of the link should satisfy both of the previous relationships at both ends of the link with the reduced values $V_{p,link,r}$ and $M_{p,link,r}$

$$V_{p,link,r} = V_{p,link}\left[1-\left(N_{Ed}/N_{pl,Rd}\right)^2\right]^{0,5}$$

$$M_{p,link,r} = M_{p,link}\left[1-\left(N_{Ed}/N_{pl,Rd}\right)\right]$$

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Web Stiffeners

Ductility of seismic links is guaranteed by the disposal of web stiffeners.
For rotation angle should not exceed:

For Short Links

$$\Theta_p \leq 0{,}08 \ rad$$

For Long Links

$$\Theta_p \leq 0{,}02 \ rad$$

Links should be provided with intermediate web stiffeners as follows:

• intermediate web stiffeners spaced at intervals not exceeding ($30t_w$ – d/5) for a rotation angle Θ_p of 0,08 radians or ($52t_w$ – d/5) for rotation angles Θ_p of 0,02 radians;

• for Long Links one intermediate web stiffener placed at a distance of 1,5 times b from each end of the link where a plastic hinge would form;

• the intermediate web stiffeners should be full depth, on only one side of the link web for links that are less than 600 mm in depth, and on both sides of the web for links that are 600 mm in depth or greater.

Detailing Rules for EBF

Members not containing Seismic Links

Columns and diagonal members, if horizontal links in beams are used, and also the beam members, if vertical links are used, should be verified in compression considering the most unfavourable combination of the axial force and bending moments:

$$N_{Rd}\left(M_{Ed}, V_{Ed}\right) \geq N_{Ed,G} + 1{,}1 \cdot \gamma_{0V} \cdot \Omega \cdot N_{Ed,E}$$

The overstrength factors calculated for each member not containing seismic links are:

for Short Links the minimum value of $\Omega_i = 1{,}5 \cdot V_{p,link,i} / V_{Ed,i}$

for Long Links the minimum value of $\Omega_i = 1{,}5 \cdot M_{p,link,i} / M_{Ed,i}$

In order to achieve a global dissipative behaviour of the structure, it should be checked that the maximum value does not differ from the minimum value by more than 25%.

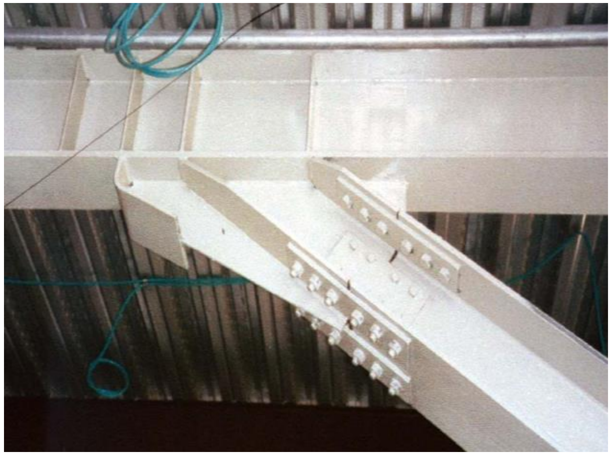
Detailing Rules for EBF

Connections

The connections of links to the other members have to provide adequate overstrength to permit the development of the expected dissipative mechanism, avoiding their plasticization or buckling.

Non dissipative connections of dissipative members made by means of full penetration butt welds may be deemed to satisfy the overstrength criterion.

$$R_{j,d} \geq \gamma_{0V} \cdot 1{,}1 \cdot R_{pl,Rd} = R_{U,Rd}$$



where:

$R_{j,d}$ is the design resistance of the connection;

$R_{pl,Rd}$ is the plastic resistance of the dissipative member;

$R_{U,Rd}$ is the upper bound of the plastic resistance of the dissipative member;

γ_{0V} is the overstrength factor.

Detailing Rules for EBF

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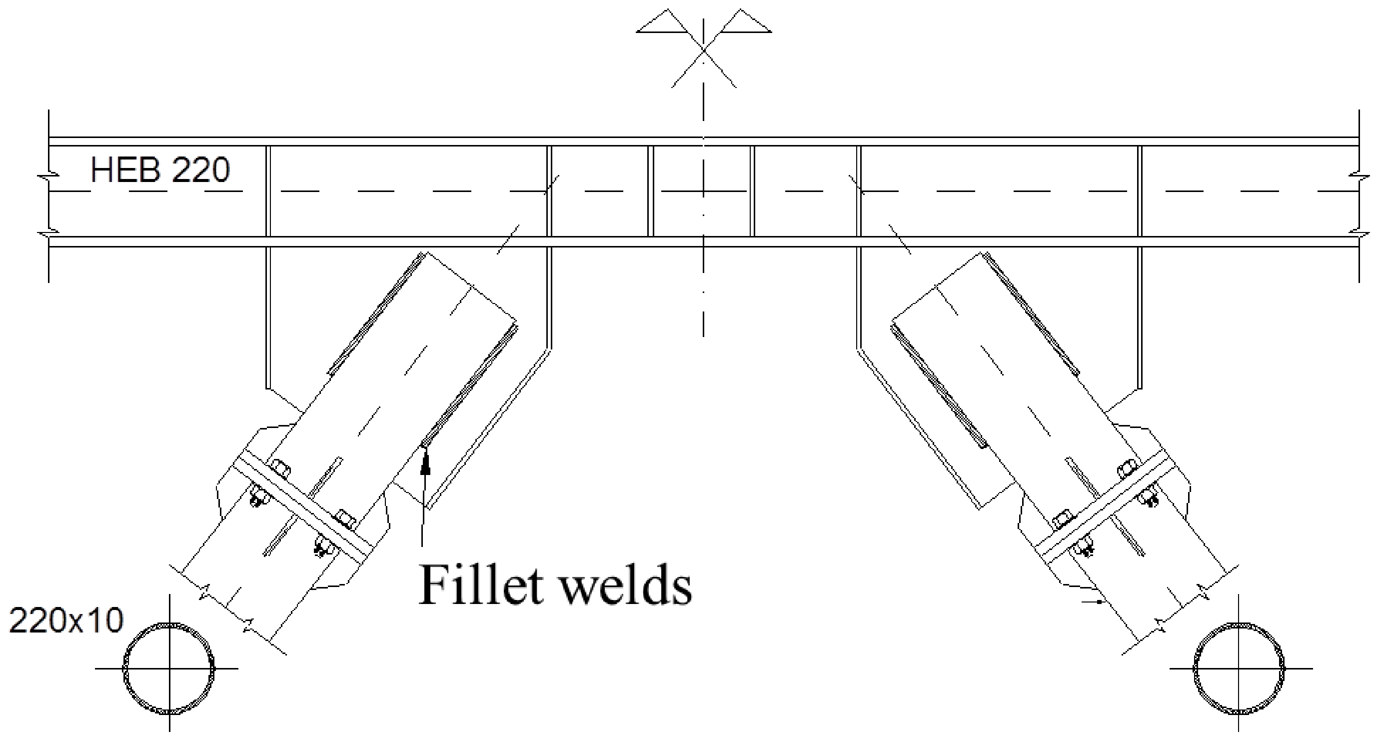
Connections: typical joint



Seismic Design of Steel Structures

1. Benefits of steel structures
2. Design criteria for steel structures
3. Detailing rules for steel structures
4. Innovative solutions

Innovative systems: Cold Formed Structures

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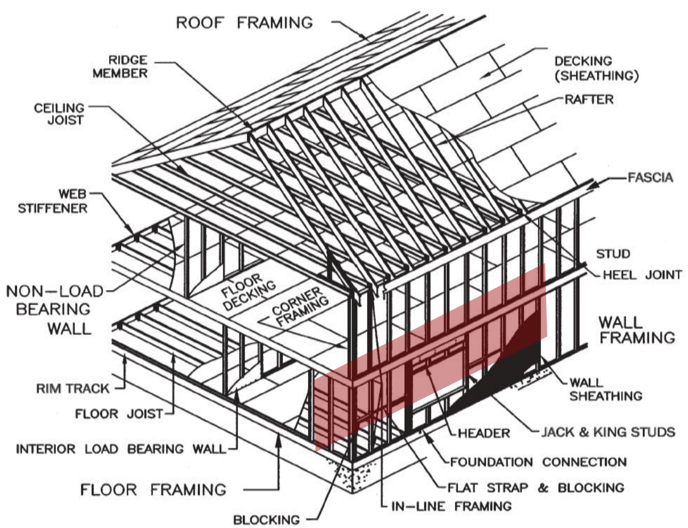
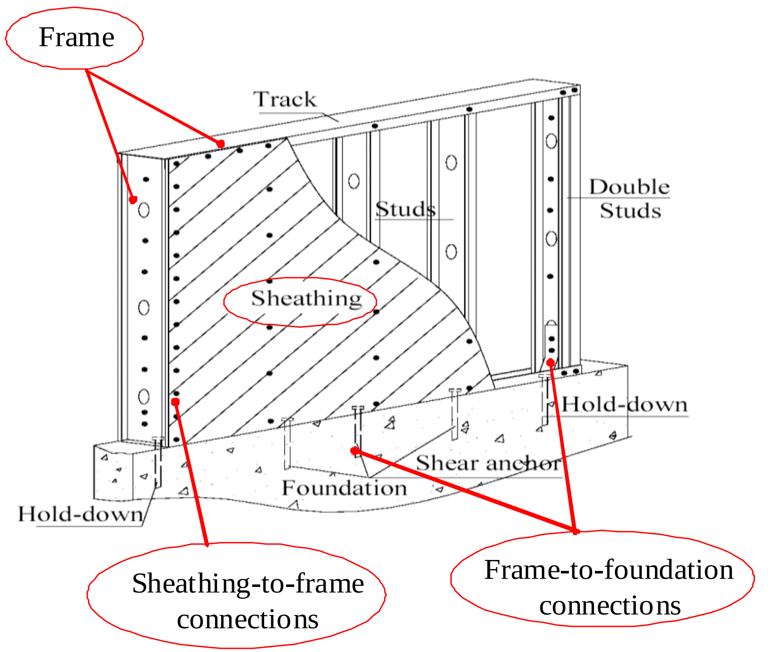
Cold Formed Structures (CFS) in seismic areas





- The seismic behaviour of cold-formed steel (CFS) structures sheathed with panels is influenced by the response of shear walls.
- Cold-formed steel shear walls are characterized by strong nonlinearity of the monotonic lateral-load response and pinching of hysteresis loops.
- Generally, CFS Shear walls are designed according to capacity design criteria, in such a way to promote the development of the most ductile failure mechanism

CFS: Cold Formed Structures

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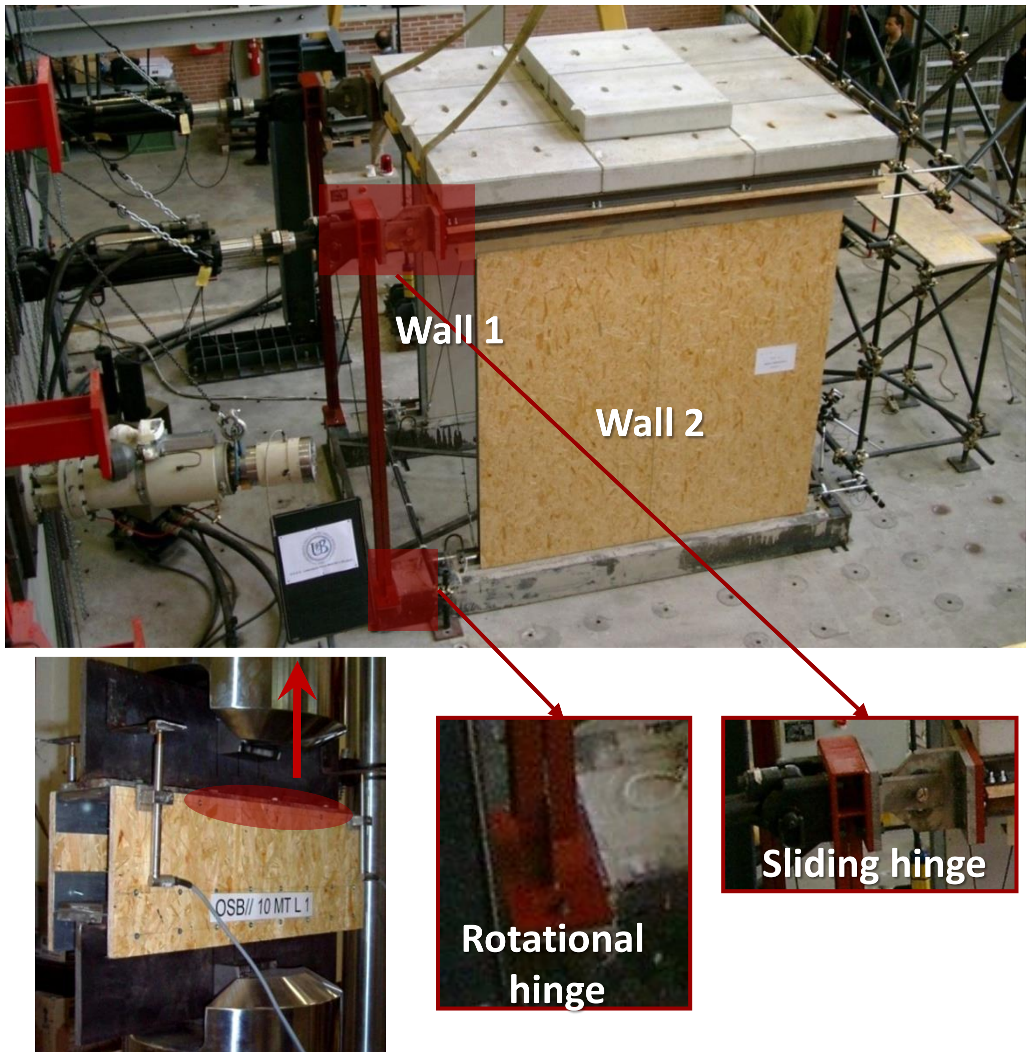
Cold Formed Structures (CFS) in seismic areas



At the University of Naples ''Federico II'' the structural behaviour of sheathed CFS frame structures has been one of the main research subjects developed in the last years.

These studies have included:

1. physical tests and numerical analysis for the seismic performance assessment of single-storey sheathed CFS houses;

2. experimental tests on typical panel-to-frame screw connections;

3. modeling of sheathed CFS shear walls based on screw connection tests;

4. architectural, technological and structural design of sustainable house prototypes for contemporary living needs.

CFS: Cold Formed Structures

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Benefits of steel structures

Design criteria for steel structures

Detailing rules for steel structures

MRF

Innovative solutions

APPLICATION*: Building construction of foundation and primary stage school - Bfs Naples - Lago Patria - Naples*

CFS: Cold Formed Structures

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CFS: Cold Formed Structures

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CFS: Cold Formed Structures

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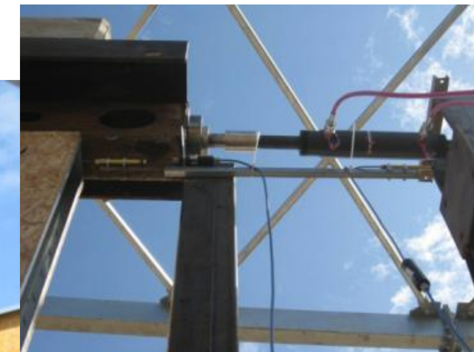
APPLICATION*: Building construction of foundation and primary stage school - Bfs Naples - Lago Patria - Naples*



Full Scale Wall On Site Tests

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Sustainable Constructions under Natural Hazards and Catastrophic Events

The reverse of the medal:
Bad design and inadequate execution

The importance of rational design

Teheran-Iran

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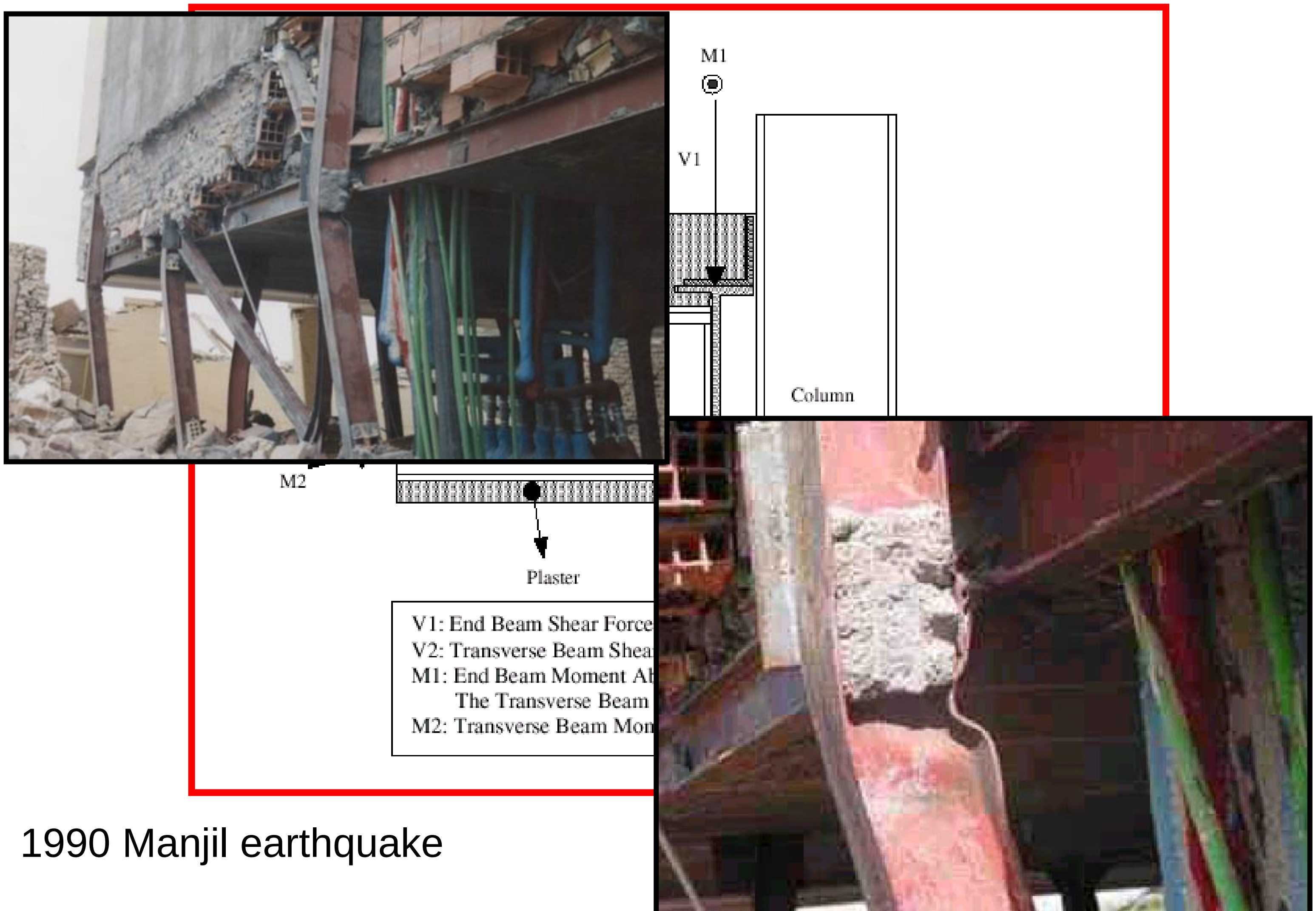
Sustainable Constructions under Natural Hazards and Catastrophic Events

The reverse of the medal:
Bad design and inadequate execution

The importance of rational design

Beam- to -column end-span “Khorjini” connection



1990 Manjil earthquake

Thank you
for your attention

http://steel.fsv.cvut.cz/suscos

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