European Commission

Eurocodes
Background and Applications

Design of **Steel Buildings**
with worked examples

16-17 October 2014
Brussels, Belgium

Organised and supported by

European Commission
DG Enterprise and Industry
Joint Research Centre
European Convention for Constructional Steelwork
European Committee for Standardization
CEN/TC250/SC3

Seismic design of steel structures

Raffaele Landolfo

Department of Structures for Engineering and Architecture
University of Naples Federico II

Joint Research Centre

Contents:

PART 1

GENERAL PRINCIPLES

PART 2

CHAPTER 6:
SEISMIC DESIGN OF STEEL STRUCTURES

PART 1

GENERAL PRINCIPLES

Eurocode 8

General

Performance requirements and compliance criteria

Ground conditions and seismic action

Design of buildings

Low seismicity zones

GENERAL PRINCIPLES

EUROCODES
BUILDING THE FUTURE

Eurocode 8

General

Performance requirements and compliance criteria

Ground conditions and seismic action

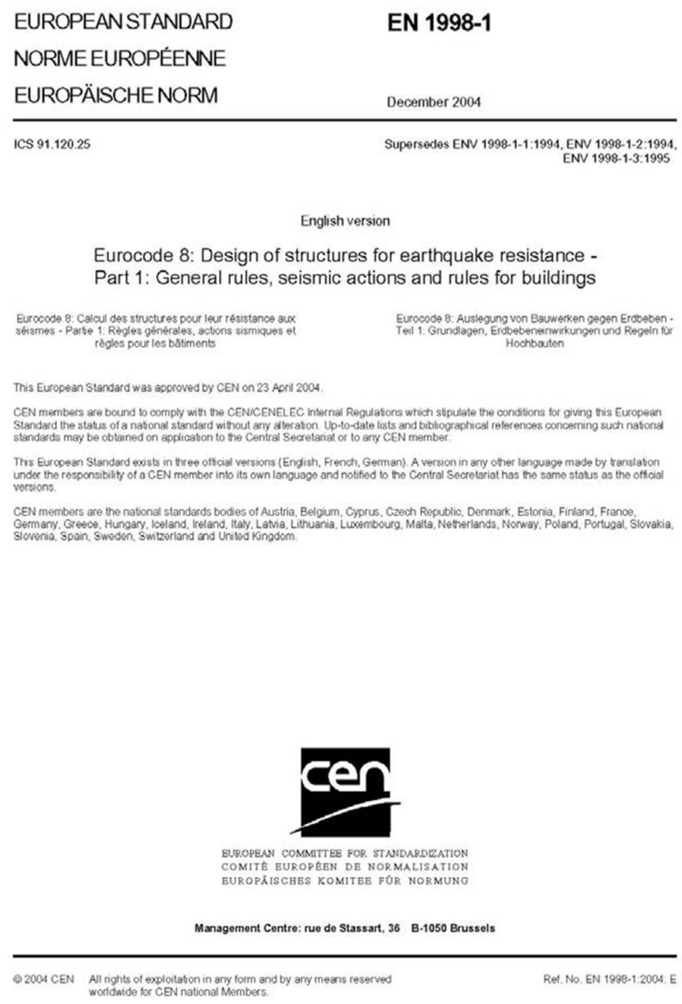
Design of buildings

Low seismicity zones

Eurocode 8

Design of structures for earthquake resistance.

Part 1. General rules, seismic actions and rules for buildings


EUROPEAN STANDARD
NORME EUROPÉENNE
EUROPÄISCHE NORM

EN 1998-1

December 2004

ICS 91.120.25

Supersedes ENV 1998-1-1:1994, ENV 1998-1-2:1994, ENV 1998-1-3:1995

English version

Eurocode 8: Design of structures for earthquake resistance - Part 1: General rules, seismic actions and rules for buildings

Eurocode 8: Calcul des structures pour leur résistance aux séismes - Partie 1: Règles générales, actions sismiques et règles pour les bâtiments

Eurocode 8: Auslegung von Bauwerken gegen Erdbeben - Teil 1: Grundlagen, Erdbebeneinwirkungen und Regeln für Hochbauten

This European Standard was approved by CEN on 23 April 2004.

CEN members are bound to comply with the CEN/CENELEC Internal Regulations which stipulate the conditions for giving this European Standard the status of a national standard without any alteration. Up-to-date lists and bibliographical references concerning such national standards may be obtained on application to the Central Secretariat or to any CEN member.

This European Standard exists in three official versions (English, French, German). A version in any other language made by translation under the responsibility of a CEN member into its own language and notified to the Central Secretariat has the same status as the official versions.

CEN members are the national standards bodies of Austria, Belgium, Cyprus, Czech Republic, Denmark, Estonia, Finland, France, Germany, Greece, Hungary, Iceland, Ireland, Italy, Latvia, Lithuania, Luxembourg, Malta, Netherlands, Norway, Poland, Portugal, Slovakia, Slovenia, Spain, Sweden, Switzerland and United Kingdom.

EUROPEAN COMMITTEE FOR STANDARDIZATION
COMITÉ EUROPÉEN DE NORMALISATION
EUROPÄISCHES KOMITEE FÜR NORMUNG

Management Centre: rue de Stassart, 36 B-1050 Brussels

Ref. No. EN 1998-1:2004: E

CONTENTS

EN1998-1 to be applied in combination with other Eurocodes

GENERAL PRINCIPLES

Eurocode 8

General

Performance requirements and compliance criteria

Ground conditions and seismic action

Design of buildings

Low seismicity zones

1. GENERAL
1.1 SCOPE

THE **main goals** in the **DESIGN OF SEISMIC RESISTANT STRUCTURES**

In the event of earthquakes:

- Human lives are protected
- Damage is limited
- Structures important for civil protection remain operational



CONTENTS

EN1998-1 to be applied in combination with other Eurocodes

GENERAL PRINCIPLES

Eurocode 8

General

Performance requirements and compliance criteria

Ground conditions and seismic action

Design of buildings

Low seismicity zones

2. PERFORMANCE REQUIREMENTS AND COMPLIANCE CRITERIA

2.1 FUNDAMENTAL REQUIREMENTS

Two different limit states are defined for the achievement of fundamental requirements

Damage Limitation Requirement	No Collapse Requirement
Damage Limitation State (DLS)	**Ultimate Limit State (ULS)**
• For ordinary structures this requirement should be met for a seismic action with 10 % probability of exceedance in 10 years • Return Period= 95 years • Performance level required: • Withstand the design seismic action without damage • Avoid limitations of use and high repair costs	• For ordinary structures this requirement should be met for a reference seismic action with 10 % probability of exceedance in 50 years Return Period= 475 years • Performance level required: • Withstand the design seismic action without local or global collapse • Retain structural integrity and residual load bearing capacity after the event

GENERAL PRINCIPLES

Eurocode 8

General

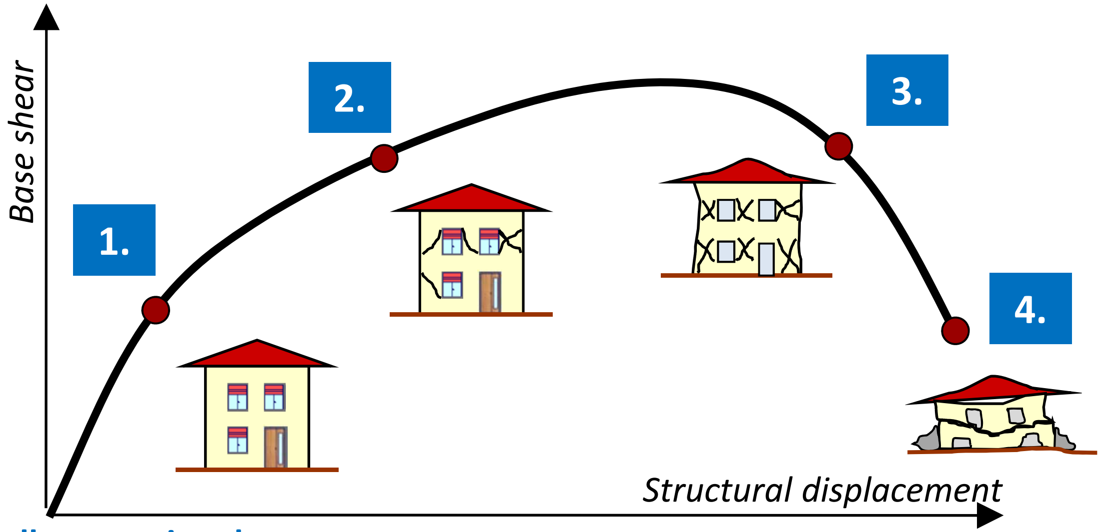
Performance requirements and compliance criteria

Ground conditions and seismic action

Design of buildings

Low seismicity zones

Performance Based Design



1. Fully operational:
Continuous service. Negligible structural and nonstructural damage.

2. Operational:
Most operations and functions can resume immediately. Structure safe for occupancy. Essential operations protected, non-essential operations disrupted. Repair required to restore some non-essential services. Damage is light.

3. Life Safety:
Damage is moderate, but structure remains stable. Selected building systems, features, or contents may be protected from damage. Life safety is generally protected. Building may be evacuated following earthquake. Repair possible, but may be economically impractical.

4. Near Collapse:
Damage severe, but structural collapse prevented. Nonstructural elements may fall. Repair generally not possible

Performance Based Design

GENERAL PRINCIPLES

Eurocode 8

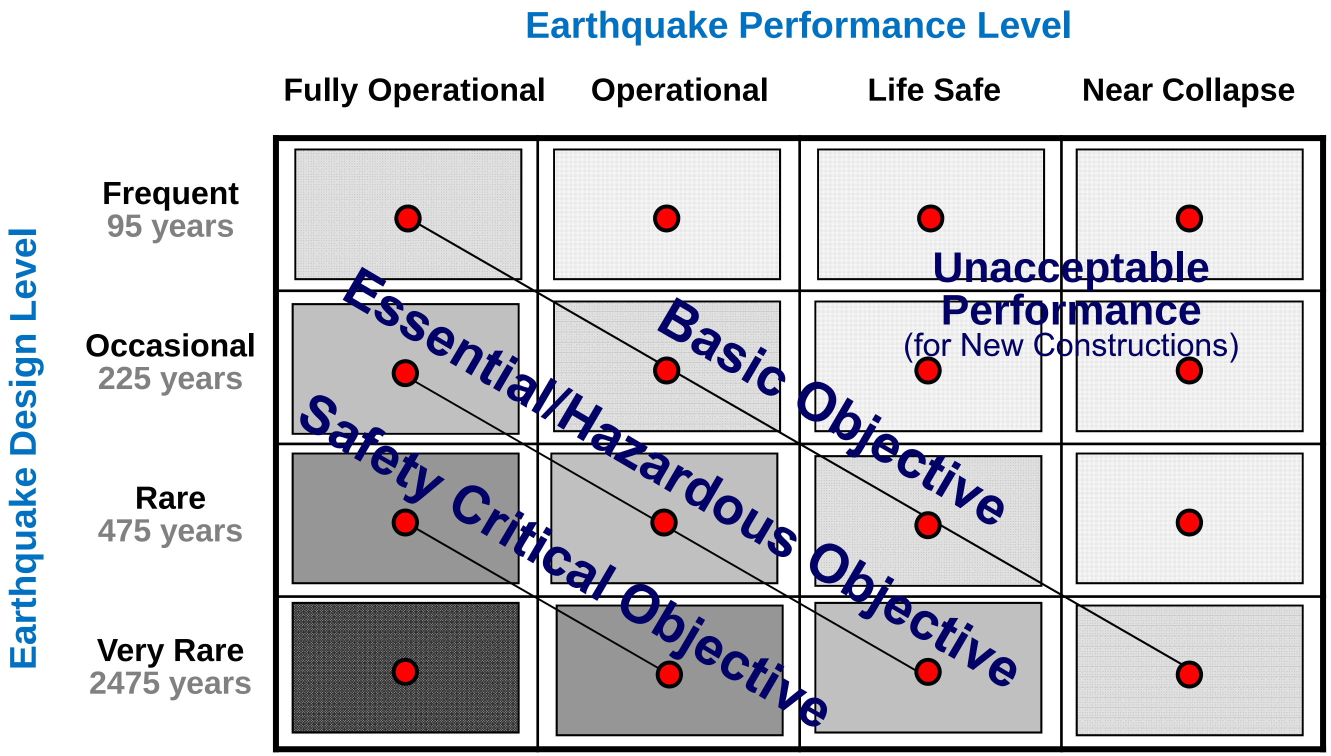
General

Performance requirements and compliance criteria

Ground conditions and seismic action

Design of buildings

Low seismicity zones



CONTENTS

DEFINITION OF SEISMIC ACTION

EN1998-1 to be applied in combination with other Eurocodes

3. GROUND CONDITIONS AND SEISMIC ACTION

3.1 GROUND CONDITIONS

Identification of ground types

Ground type	Description of stratigraphic profile
A	Rock or other rock-like geological formation, including at most 5 m of weaker material at the surface.
B	Deposits of very dense sand, gravel, or very stiff clay, at least several tens of metres in thickness, characterised by a gradual increase of mechanical properties with depth.
C	Deep deposits of dense or medium-dense sand, gravel or stiff clay with thickness from several tens to many hundreds of metres.
D	Deposits of loose-to-medium cohesionless soil (with or without some soft cohesive layers), or of predominantly soft-to-firm cohesive soil.
E	A soil profile consisting of a surface alluvium layer with v_s values of type C or D and thickness varying between about 5 m and 20 m, underlain by stiffer material with v_s > 800 m/s.

GENERAL PRINCIPLES

Eurocode 8

General

Performance requirements and compliance criteria

Ground conditions and seismic action

Design of buildings

Low seismicity zones

3. GROUND CONDITIONS AND SEISMIC ACTION

3.2 SEISMIC ACTION

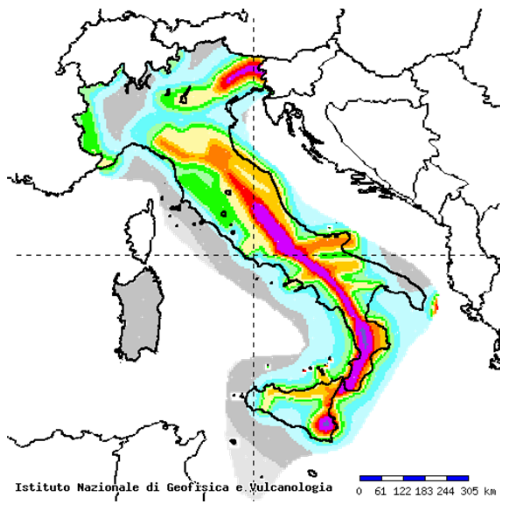
Seismic zones

National territories shall be subdivided by the National Authorities into **seismic zones**, depending on the local hazard.

• The hazard is described by **a_{gR}**, the value of the reference peak ground acceleration on type A, derived from zonation maps in National Annex;

• The parameter **a_{gR}** corresponds to the reference return period **T_{NCR}** of the seismic action for the no-collapse requirement;

• The parameter **a_{gR}** is modified by the **Importance Factor γ_I** to become the design ground acceleration (on type A ground) $a_g = a_{gR} \cdot \gamma_I$



the Italian case

GENERAL PRINCIPLES

Eurocode 8

General

Performance requirements and compliance criteria

Ground conditions and seismic action

Design of buildings

Low seismicity zones

3. GROUND CONDITIONS AND SEISMIC ACTION

3.2 SEISMIC ACTION

Importance factors

Target **reliability of requirement** depending on consequences of failure:

- Classify the structures into importance classes
- Assign a higher or lower return periodto the design seismic action

In operational terms multiply the reference seismic action by the importance factor γ_I

Importance class	Buildings	γ_I
I	Buildings of minor importance for public safety, e.g. agricultural buildings, etc.	0,8
II	Ordinary buildings, not belonging in the other categories.	1,0
III	Buildings whose seismic resistance is of importance in view of the consequences associated with a collapse, e.g. schools, assembly halls, cultural institutions etc.	1,2
IV	Buildings whose integrity during earthquakes is of vital importance for civil protection, e.g. hospitals, fire stations, power plants, etc.	1,4

GENERAL PRINCIPLES

Eurocode 8

General

Performance requirements and compliance criteria

Ground conditions and seismic action

Design of buildings

Low seismicity zones

3. GROUND CONDITIONS AND SEISMIC ACTION

3.2 SEISMIC ACTION

Basic representation of the seismic action

The earthquake motion at a given point on the surface is represented by an **elastic ground acceleration response spectrum**, called "ELASTIC RESPONSE SPECTRUM".

- Common shape for the ULS and DLS verifications
- Two orthogonal independent horizontal components
- Vertical spectrum shape different from the horizontal spectrum (common for all ground types)
- Possible use of more than one spectral shape (to model different seismo-genetic mechanisms)

GENERAL PRINCIPLES

Eurocode 8

General

Performance requirements and compliance criteria

Ground conditions and seismic action

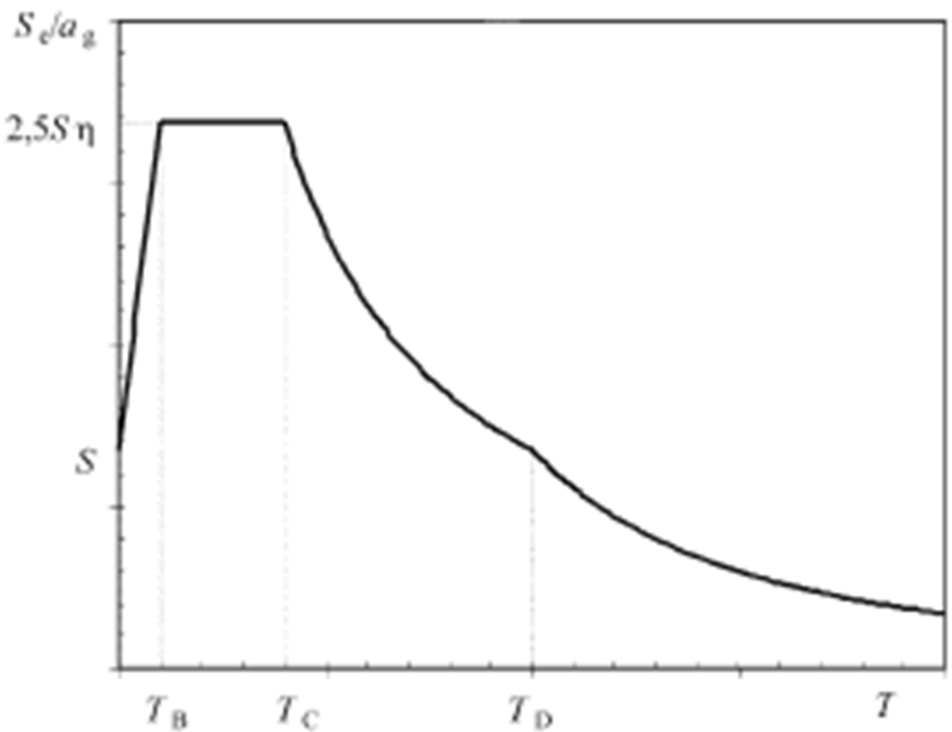
Design of buildings

Low seismicity zones

3. GROUND CONDITIONS AND SEISMIC ACTION

3.2 SEISMIC ACTION

ELASTIC RESPONSE SPECTRUM



GENERAL PRINCIPLES

Eurocode 8

General

Performance requirements and compliance criteria

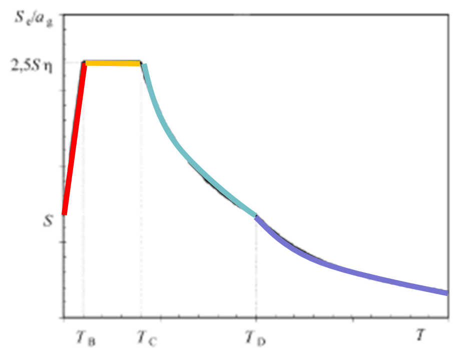
Ground conditions and seismic action

Design of buildings

Low seismicity zones

3. GROUND CONDITIONS AND SEISMIC ACTION

3.2 SEISMIC ACTION



The **ELASTIC RESPONSE SPECTRUM** is divided in 4 different branches defined by the following expressions:

- $0 \leq T \leq T_B$ $\quad S_e(T) = a_g \,.\, S \,.\, (1+T/T_B \,.\, (\eta \,.\, 2{,}5 - 1))$
- $T_B \leq T \leq T_C$ $\quad S_e(T) = a_g \,.\, S \,.\, \eta \,.\, 2{,}5$
- $T_C \leq T \leq T_D$ $\quad S_e(T) = a_g \,.\, S \,.\, \eta \,.\, 2{,}5\,(T_C / T)$
- $T_D \leq T \leq 4$ s $\quad S_e(T) = a_g \,.\, S \,.\, \eta \,.\, 2{,}5\,(T_C \,.\, T_D / T^2)$

$S_e(T)$	**elastic** response spectrum
a_g	**design ground acceleration** on type A ground
T_B T_C T_D	**corner periods** in the spectrum (NDPs)
S	**soil** factor (NDP)
η	**damping** correction factor (η = 1 for 5% damping)

GENERAL PRINCIPLES

Eurocode 8

General

Performance requirements and compliance criteria

Ground conditions and seismic action

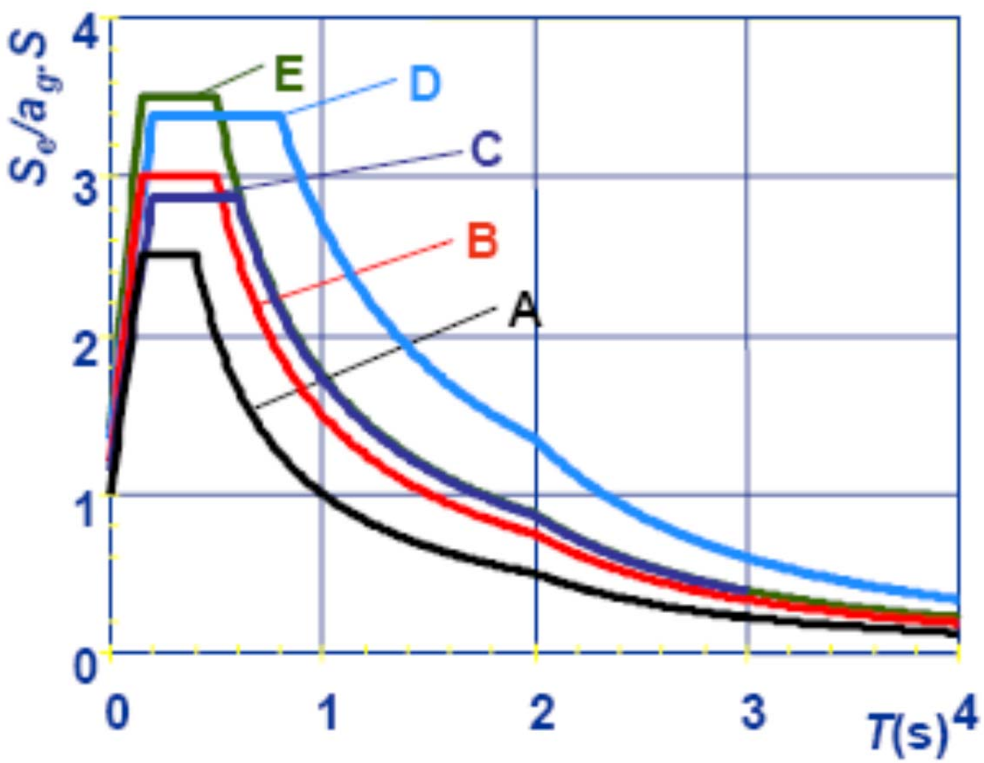
Design of buildings

Low seismicity zones

3. GROUND CONDITIONS AND SEISMIC ACTION

3.2 SEISMIC ACTION

The **ELASTIC RESPONSE SPECTRUM** is different for the different grounds types:



3. GROUND CONDITIONS AND SEISMIC ACTION
3.2 SEISMIC ACTION

VERTICAL **ELASTIC RESPONSE SPECTRUM**

The vertical component of the seismic action shall be represented by an elastic response spectrum, $S_{ve}(T)$, derived using expressions :

$$0 \le T \le T_B : S_{ve}(T) = a_{vg} \cdot \left[1 + \frac{T}{T_B} \cdot (\eta \cdot 3{,}0 - 1)\right]$$

$$T_B \le T \le T_C : S_{ve}(T) = a_{vg} \cdot \eta \cdot 3{,}0$$

$$T_C \le T \le T_D : S_{ve}(T) = a_{vg} \cdot \eta \cdot 3{,}0 \left[\frac{T_C}{T}\right]$$

$$T_D \le T \le 4\mathrm{s} : S_{ve}(T) = a_{vg} \cdot \eta \cdot 3{,}0 \left[\frac{T_C . T_D}{T^2}\right]$$

3. GROUND CONDITIONS AND SEISMIC ACTION

3.2 SEISMIC ACTION

DESIGN RESPONSE SPECTRUM

The capacity of structural systems to resist seismic actions in the non-linear range generally permits their design for resistance to seismic forces smaller than those corresponding to a linear elastic response.

The capacity of the structure to dissipate energy is taken into account by performing an elastic analysis based on a reduced response spectrum, called ''**DESIGN SPECTRUM**''

This reduction is accomplished by introducing the **behaviour factor “q”.**

3. GROUND CONDITIONS AND SEISMIC ACTION

3.2 SEISMIC ACTION

BEHAVIOUR FACTOR “q”

The **behaviour factor q** is an approximation of the ratio of the seismic forces that the structure would experience if its response was completely elastic with 5% viscous damping, to the seismic forces that may be used in the design, with a conventional elastic analysis model, still ensuring a satisfactory response of the structure.

The **values** of the behaviour factor q, which also account for the influence of the viscous damping being different from 5%, are given for various **materials** and **structural systems** according to the relevant **ductility classes**.

3. GROUND CONDITIONS AND SEISMIC ACTION

3.2 SEISMIC ACTION

GENERAL PRINCIPLES

Eurocode 8

General

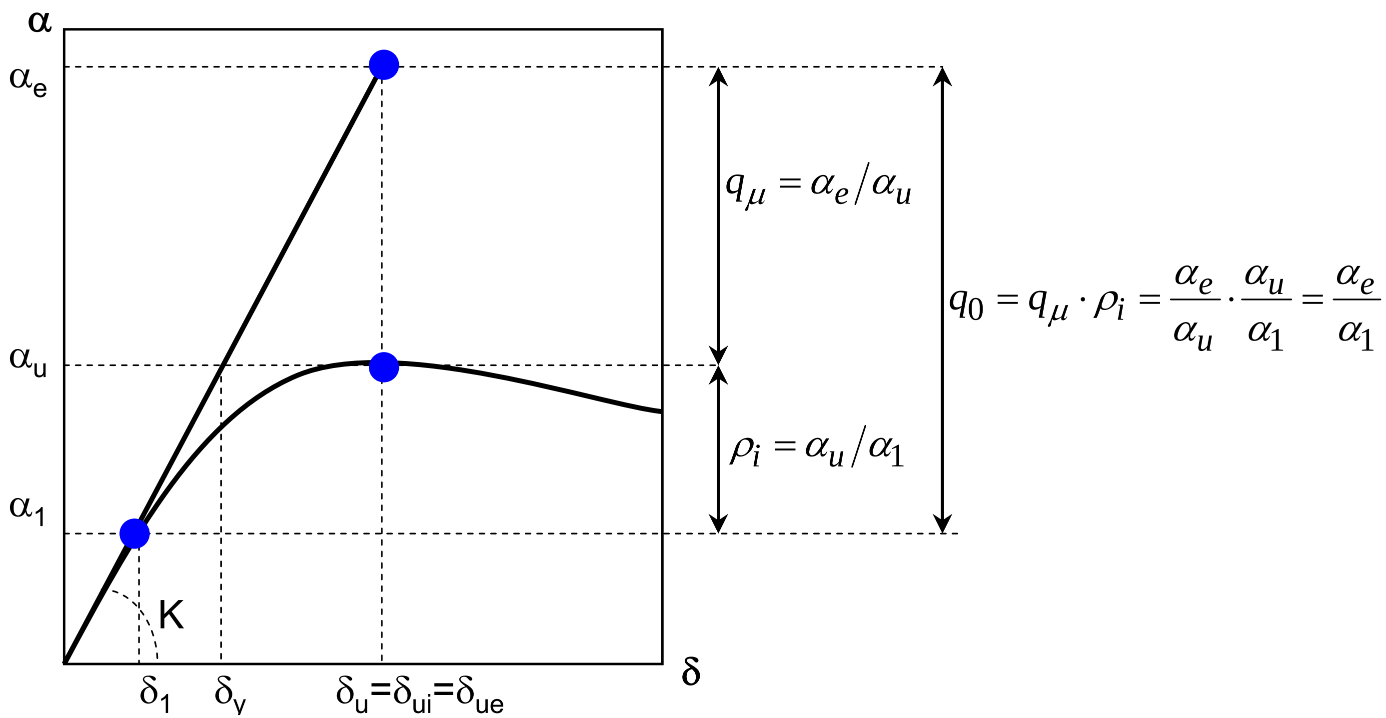
Performance requirements and compliance criteria

Ground conditions and seismic action

Design of buildings

Low seismicity zones

BEHAVIOUR FACTOR "q"



GENERAL PRINCIPLES

Eurocode 8

General

Performance requirements and compliance criteria

Ground conditions and seismic action

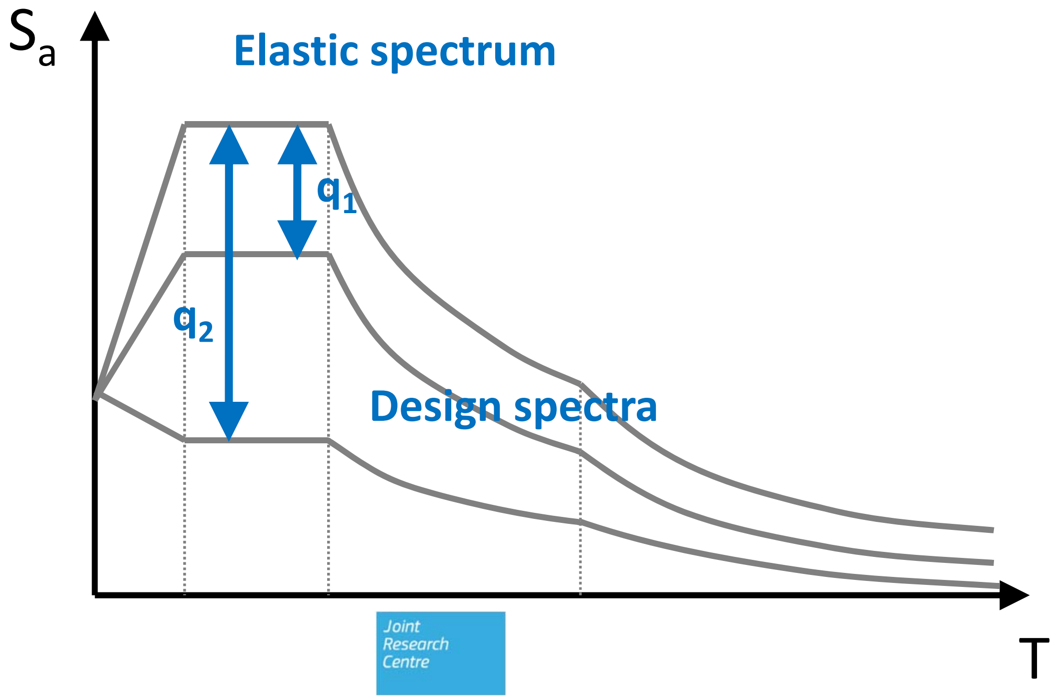
Design of buildings

Low seismicity zones

3. GROUND CONDITIONS AND SEISMIC ACTION
3.2 SEISMIC ACTION

BEHAVIOUR FACTOR "q"

The behaviour factor takes into account the dissipative capacity of the structural systems. Hence, it varies with the structural typology.



GENERAL PRINCIPLES

Eurocode 8

General

Performance requirements and compliance criteria

Ground conditions and seismic action

Design of buildings

Low seismicity zones

3. GROUND CONDITIONS AND SEISMIC ACTION
3.2 SEISMIC ACTION

COMBINATIONS OF SEISMIC ACTIONS WITH OTHER ACTIONS

The design value *Ed* of the effects of actions in the seismic design situation shall be determined in accordance with EN 1990:2002

$$E_d = E\{G_{k,j} \,;\, P \,;\, A_{Ed} \,;\, \psi_{2,i} Q_{k,i}\} \quad j \geq 1 \,;\, i \geq 1$$

The inertial effects of the design seismic action shall be evaluated by taking into account the presence of the masses associated with all gravity loads appearing in the following combination of actions:

$$\Sigma G_{k,j} \text{ "+" } \Sigma \psi_{E,i} \cdot Q_{k,i}$$

$\Psi_{E,i}$ is the combination coefficient for variable action *i*, it takes into account the likelihood of the loads $Q_{k,i}$ not being present over the entire structure during the earthquake. These coefficients may also account for a reduced participation of masses in the motion of the structure due to the non-rigid connection between them.

GENERAL PRINCIPLES
Eurocode 8
General
Performance requirements and compliance criteria
Ground conditions and seismic action
Design of buildings
Low seismicity zones

3. GROUND CONDITIONS AND SEISMIC ACTION
3.2 SEISMIC ACTION

COMBINATIONS OF SEISMIC ACTIONS WITH OTHER ACTIONS

The combination coefficients Ψ_{Ei} for the calculation of the effects of the seismic actions shall be computed from the following expression:

$$\psi_{Ei} = \varphi \cdot \psi_{2i}$$

Table 4.2: Values of φ for calculating ψ_{Ei}

Type of variable action	Storey	φ
Categories A-C*	Roof Storeys with correlated occupancies Independently occupied storeys	1,0 0,8 0,5
Categories D-F* and Archives		1,0

* Categories as defined in EN 1991-1-1:2002.

CONTENTS

EN1998-1 to be applied in combination with other Eurocodes

GENERAL PRINCIPLES
Eurocode 8
General
Performance requirements and compliance criteria
Ground conditions and seismic action
Design of buildings
Low seismicity zones

4. DESIGN OF BUILDINGS
4.2 CHARACTERISTICS OF EARTHQUAKE RESISTANT BUILDINGS

BASIC PRINCIPLES OF CONCEPTUAL DESIGN

In seismic regions the aspect of seismic hazard shall be taken into account in the early stages of the **conceptual design** of the building.
The guiding principles governing this conceptual design are:

- structural simplicity;
- uniformity, symmetry and redundancy;
- bi-directional resistance and stiffness;
- torsional resistance and stiffness;
- diaphragmatic behaviour at storey level;
- adequate foundation.

4. DESIGN OF BUILDINGS
4.3 STRUCTURAL ANALYSIS

Modeling and methods of analysis

• The model of the building shall adequately represent the distribution of stiffness and mass in it so that all significant deformation shapes and inertia forces are properly accounted for under the seismic action considered. In the case of non-linear analysis, the model shall also adequately represent the distribution of strength.

•Accidental torsional effects

•Methods of analysis

GENERAL PRINCIPLES

Eurocode 8

General

Performance requirements and compliance criteria

Ground conditions and seismic action

Design of buildings

Low seismicity zones

4. DESIGN OF BUILDINGS

4.4 SAFETY VERIFICATIONS

For the safety verifications the ultimate limit state (ULS) and the damage limitation state (DLS) shall be considered

ULTIMATE LIMIT STATE (4.4.2)

The no-collapse requirement (ultimate limit state) under the seismic design situation is considered to have been met if the following conditions regarding resistance, ductility, equilibrium, foundation stability and seismic joints are met.

- Resistance condition (including P-Δ effects)
- Global and local ductility condition (including capacity criteria for MRF)
- Equilibrium condition
- Resistance of horizontal diaphragms
- Resistance of foundations
- Seismic joint condition

GENERAL PRINCIPLES

Eurocode 8

General

Performance requirements and compliance criteria

Ground conditions and seismic action

Design of buildings

Low seismicity zones

4. DESIGN OF BUILDINGS
4.4 SAFETY VERIFICATIONS

DAMAGE LIMITATION STATE (4.4.3)

Limitation of interstorey drift

The following limits shall be observed:

a) for buildings having non-structural elements of brittle materials attached to the structure:

$$d_r \nu \leq 0{,}005\, h\,;$$

b) for buildings having ductile non-structural elements:

$$d_r \nu \leq 0{,}0075\, h\,;$$

c) for buildings having non-structural elements fixed in a way so as not to interfere with structural deformations, or without non-structural elements:

$$d_r \nu \leq 0{,}010\, h$$

Buildings in low seismicity zones

DEFINITION OF LOW SEIMICITY

It is recommended to consider as **low seismicity zones** either those in which the design ground acceleration on type A ground**, a_g, is not greater than 0,08 g** (0,78 m/s2), or those where the product **a_gS is not greater than 0,1 g** (0,98 m/s2).

The selection of whether the value of a_g, or that of the product a_gS will be used in a country to define the threshold for low seismicity cases, may be found in the **National Annex**."

DESIGN OF BUILDINGS

In cases of low seismicity, **reduced or simplified seismic design procedures** for **certain types or categories of structures** may be used.

The selection of the categories of structures for which the provisions of low seismicity apply may be found in the **National Annex**

Buildings in very low seismicity zones

DEFINITION OF VERY LOW SEIMICITY

It is recommended to consider as **very low seismicity zones** either those in which the design ground acceleration on type A ground**, a_g, is not greater than 0,04 g** (0,39 m/s2), or those where the product **$a_g S$ is not greater than 0,05 g** (0,49 m/s2).

The selection of whether the value of a_g, or that of the product $a_g S$ will be used in a country to define the threshold for low seismicity cases, may be found in its National Annex."

DESIGN OF BUILDINGS

For building in very low seismicity zones, the provisions of EN 1998 need not be observed. Basically we can just forget about seismic design (**EXCLUSION CRITERIA**)

The selection of the categories of structures for which EN 1998 provisions need not be observed may be found in the **National Annex**

PART 2

CHAPTER 6: SEISMIC DESIGN OF STEEL STRUCTURES

Design criteria for steel structures

Detailing rules for steel structures

MRF, CBF, EBF

Innovative solutions

Eurocode 8: Steel Buildings

Genesis of EU seismic code for steel buildings

The development of seismic design provisions for steel structures is ongoing for over thirty years in the framework of ECCS.

- First activities started in 1980's

- First EU seismic code:
  ECCS code 1991
  European for Recommendations
  for Steel Structures in Seismic Zones.

Eurocode 8: Steel Buildings

CONTENTS

CONTENTS

CHAPTER 6

Design criteria for steel structures

Detailing rules for steel structures

CHAPTER 6

Design criteria for steel structures

Detailing rules for steel structures

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)	$q \leq 1.5 - 2$
	"b" Dissipative structural behaviuor	DCM (medium)	q=f *(structural type)* and $q \leq 4$
		DCH (high)	q=f *(structural type)*

Rules for Dissipative Structures

Design criteria for low dissipative structures

• Structures with **low dissipative behaviour** con be designed according to **Low Ductility Class** concept (DCL)

• The DCL concept is a design approach where the strength assigned to the structure is "sufficiently" large to make the plastic deformation demand from the design earthquake "sufficiently" small. This implies that **some detailing rules** for ductility can be waived.

• There is always **large uncertainty about the seismic actions**. There are chances that the intensity of a real earthquake occurring at the building site is exceeding the design value. If the real earthquake intensity is exceeding the design value, the demand is being larger than assumed for the design.

•Because of the previous argument regarding the uncertainties, the DCL concept should be **used with caution** (EC8 suggests **only in case of low seismicity zone**)

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

CONTENTS

Rules for Dissipative Structures

Material Properties

CHAPTER 6

Design criteria for steel structures

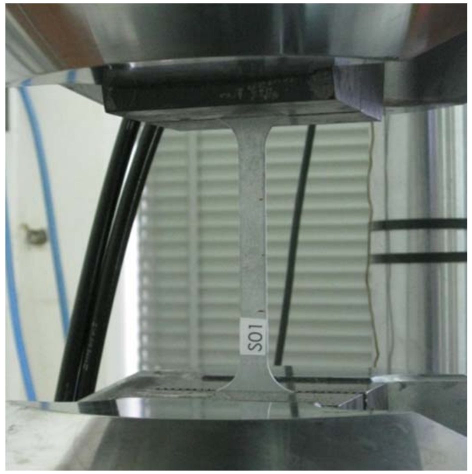
Detailing rules for steel structures

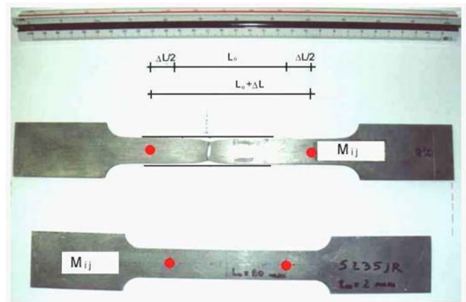
Estimate the actual yield strength of dissipative members/connections, which can be substantially larger than the nominal one.

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

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

CHAPTER 6

Design criteria for steel structures

Detailing rules for steel structures

Rules for Dissipative Structures

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

JRC EUROPEAN COMMISSION

ECCS CECM EKS

CONTENTS

CHAPTER 6

Design criteria for steel structures

Detailing rules for steel structures

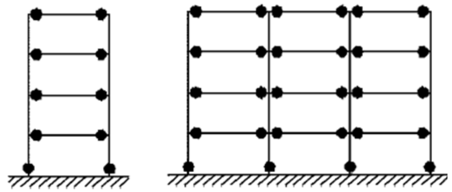
Rules for Dissipative Structures

Structural typologies and behaviour factors

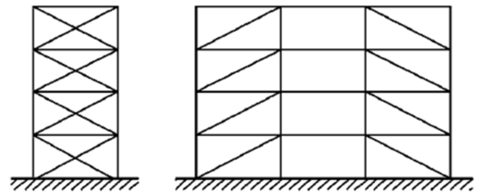
Code behaviour factors are mostly empirical, and are supposed to account for ductility, redundancy and overstrength of different structural typologies.

STRUCTURAL TYPE	Ductility Class	
	DCM	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	$5\alpha_u/\alpha_1$

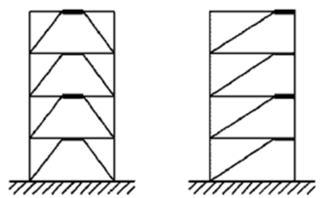
Moment Resisting Frames (MRF)



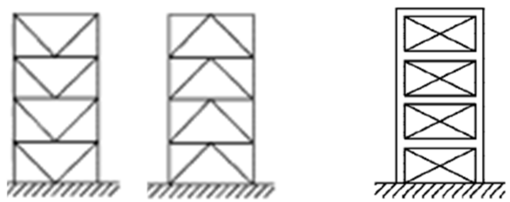
Diagonal braced frames (CBF)



Eccentric braced frames (EBF)



V Bracing (CBF) **MRF + CBF**



CONTENTS

DUCTILITY REQUIREMENTS: RULES FOR DISSIPATIVE MEMBERS AND FOR CONNECTIONS

CHAPTER 6

Design criteria for steel structures

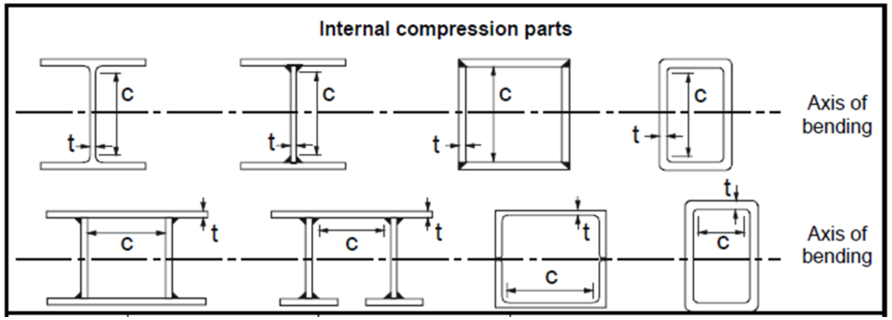
Detailing rules for steel structures

Rules for Dissipative Structures

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



Rules for Dissipative Structures

Element in tension of dissipative zones

CHAPTER 6

Design criteria for steel structures

Detailing rules for steel structures

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

Rules for Dissipative Structures

Connections

CHAPTER 6

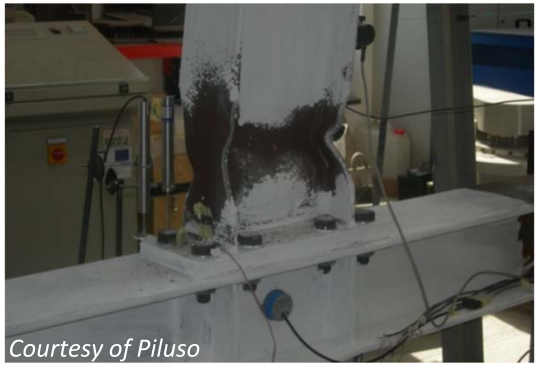
Design criteria for steel structures

Detailing rules for steel structures

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

Rules for Dissipative Structures

CHAPTER 6

Design criteria for steel structures

Detailing rules for steel structures

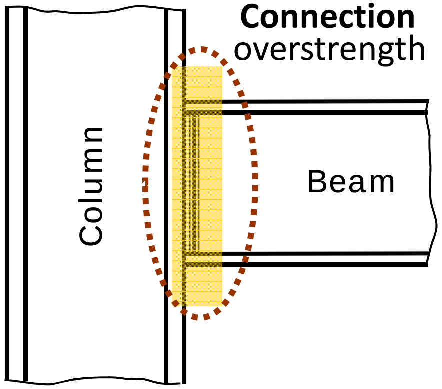
Non dissipative connections in dissipative zones

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

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

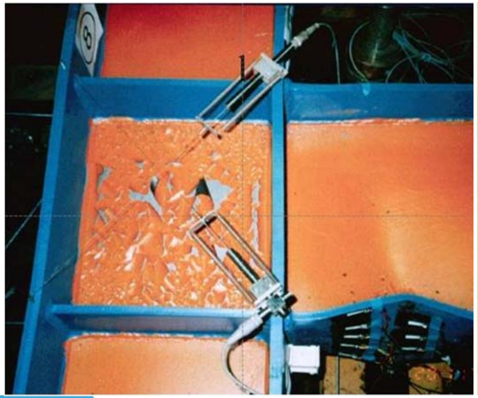


R_d : resistance of the connection in accordance with EN 1993

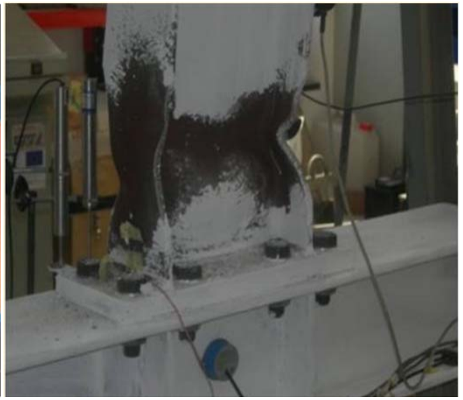
R_{fy} : plastic resistance of the connected dissipative member

γ_{ov} : overstrenght factor

The hardening factor should be related to cross section classification



WELDED CONNECTION



BOLTED CONNECTION

Rules for Dissipative Structures

Provisions for dissipative connections

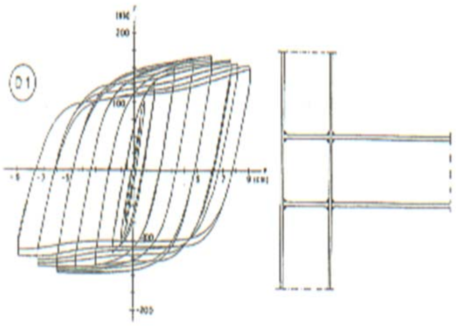
CHAPTER 6

Design criteria for steel structures

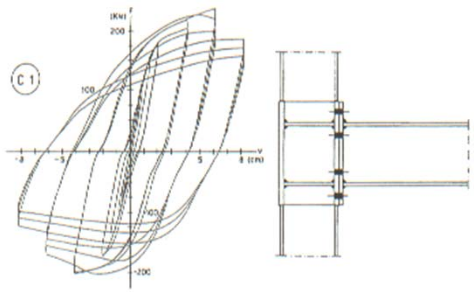
Detailing rules for steel structures

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

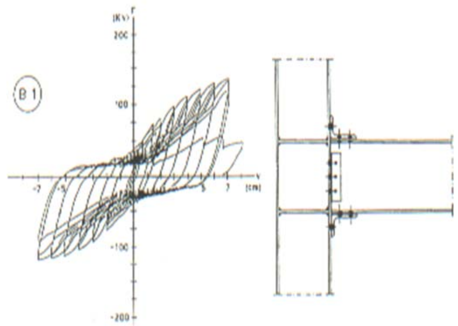
Joint cyclic rotation capacity should be at least **0.035 rad** in case of DCH or **0.025 rad** in case of DCM



Welded joint



End plate joint



Angle cleat joint

CHAPTER 6

Design criteria for steel structures

Detailing rules for steel structures

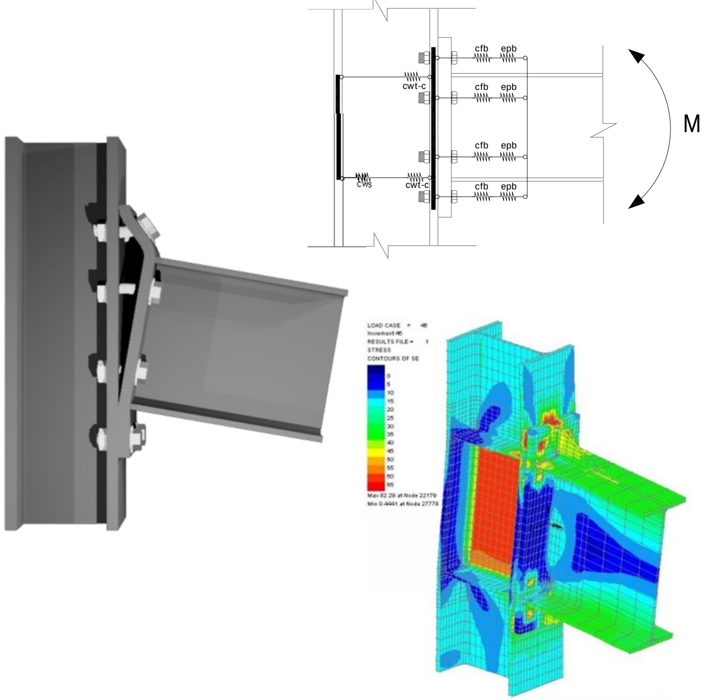
Rules for Dissipative Structures

Dissipative connections

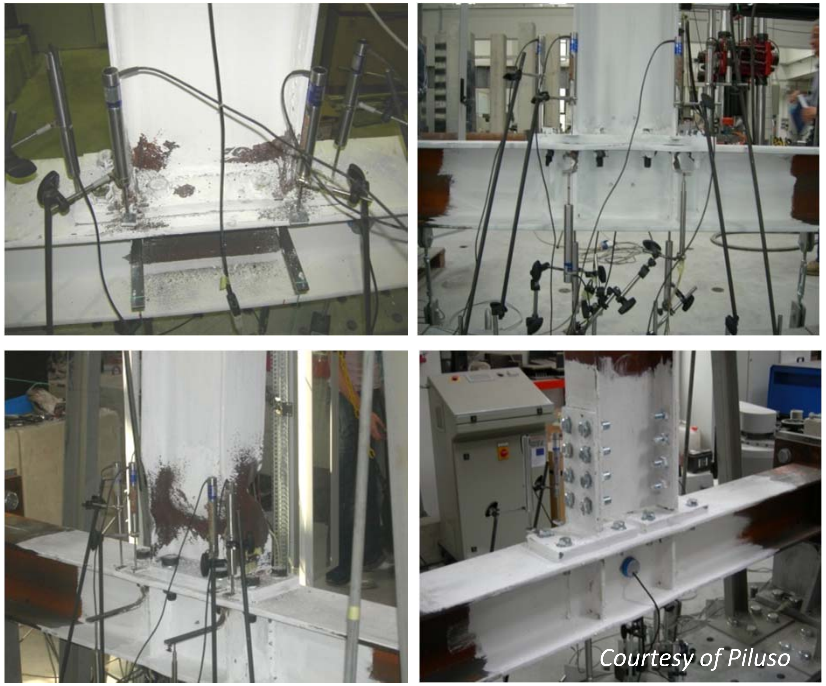
How computing Joint cyclic rotation capacity ?

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



Courtesy of Piluso

CHAPTER 6

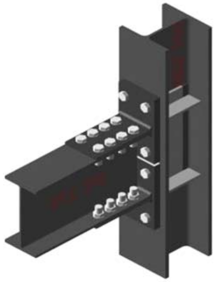
Design criteria for steel structures

Detailing rules for steel structures

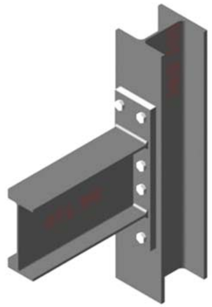
Rules for Dissipative Structures

Dissipative connections

Potential upgrade

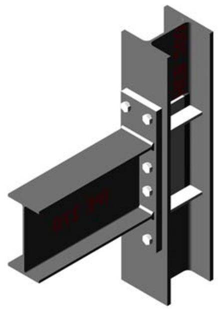


It is clear that this procedure is **unfeasible from the designer's** point of view



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)

European pre-qualified steel joints (EQUALJOINTS) project is currently ongoing to solve this issue

CONTENTS

CHAPTER 6

Design criteria for steel structures

Detailing rules for steel structures

RULES FOR GLOBAL HIERARCHY AND LOCAL CAPACITY DESIGN RULES FOR THE SPECIFIED DISSIPATIVE STRUCTURAL TYPES

Structural Typologies for Steel Buildings

CHAPTER 6

Design criteria for steel structures

Detailing rules for steel structures

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**

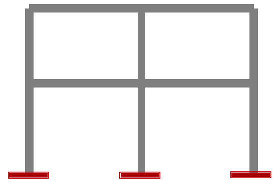
CHAPTER 6

Design criteria for steel structures

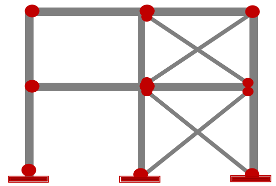
Detailing rules for steel structures

Structural Typologies for Steel Buildings

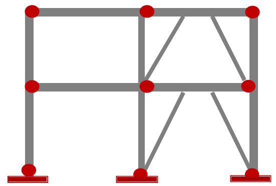
- **Moment Resisting Frames (MRF)**
horizontal forces are resisted by members acting in an essentially flexural manner



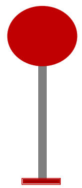
- **Frames with Concentric Bracings (CBF)**
horizontal forces are mainly resisted by members subjected to axial forces



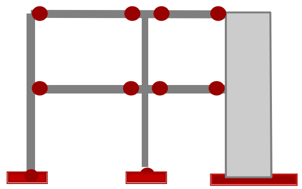
- **Frames with Eccentric Bracings (EBF)**
horizontal forces are mainly resisted by seismic links by cyclic bending or cyclic shear



- **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



Detailing rules for MRF

CHAPTER 6

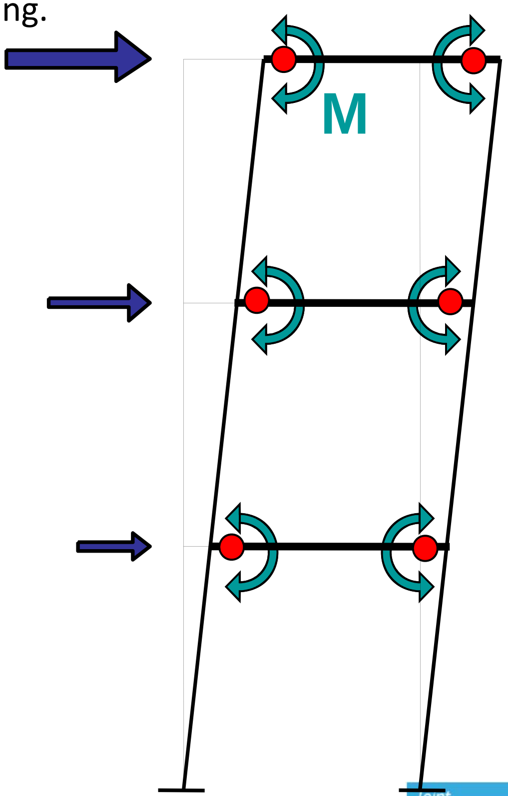
Design criteria for steel structures

Detailing rules for steel structures

MRF

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.



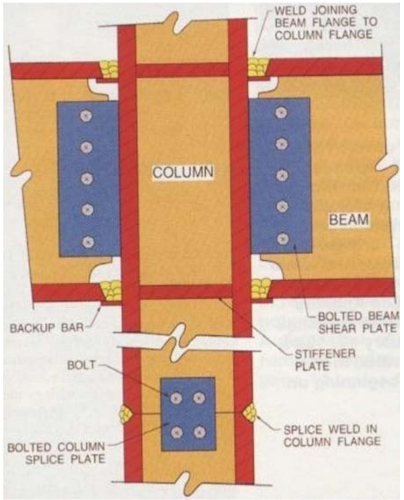
Detailing rules for MRF

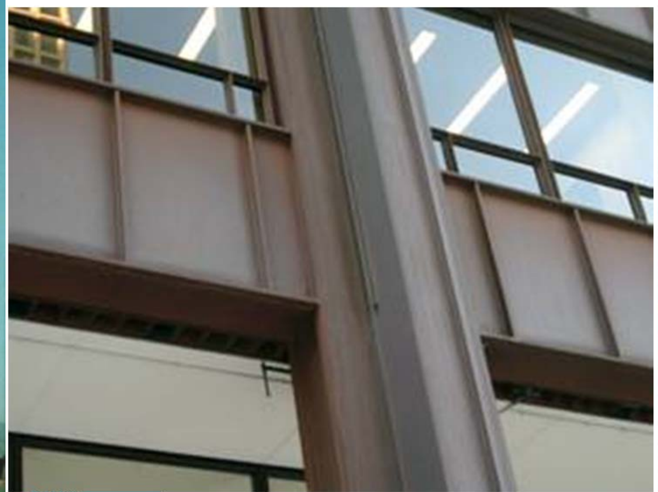
CHAPTER 6

Design criteria for steel structures

Detailing rules for steel structures

MRF





Detailing rules for MRF

Design Concept

CHAPTER 6

Design criteria for steel structures

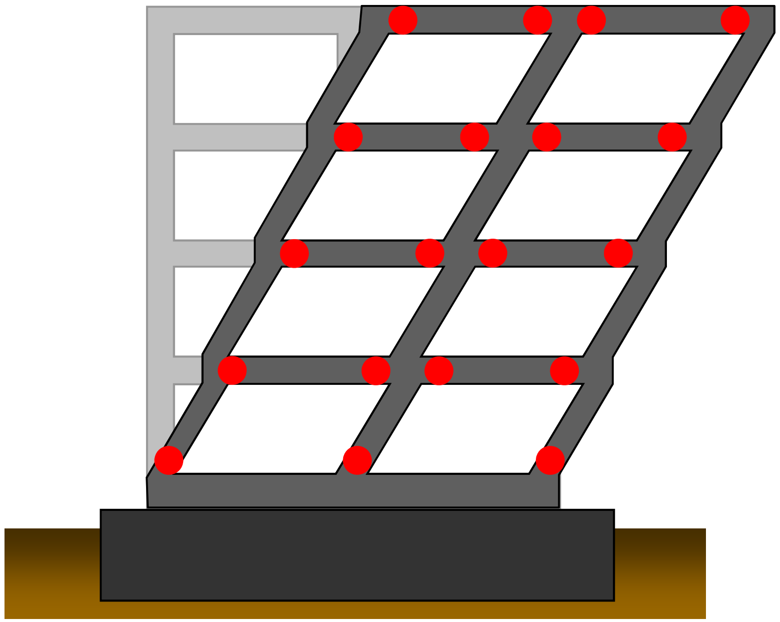
Detailing rules for steel structures

MRF

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

Basic Principles

CHAPTER 6

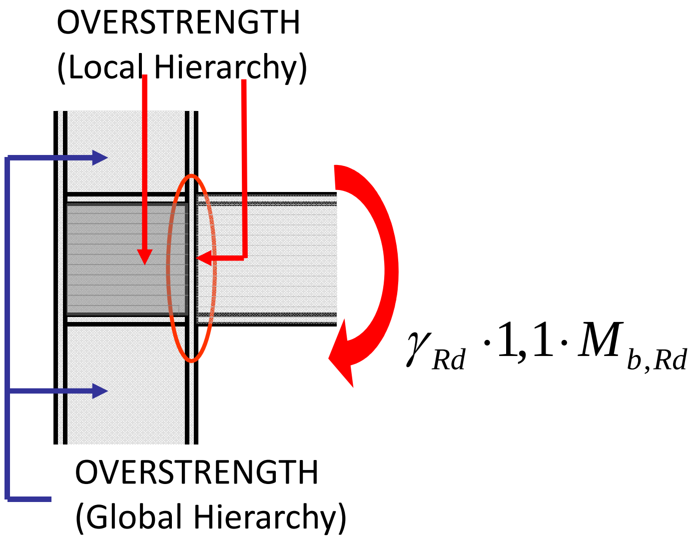
Design criteria for steel structures

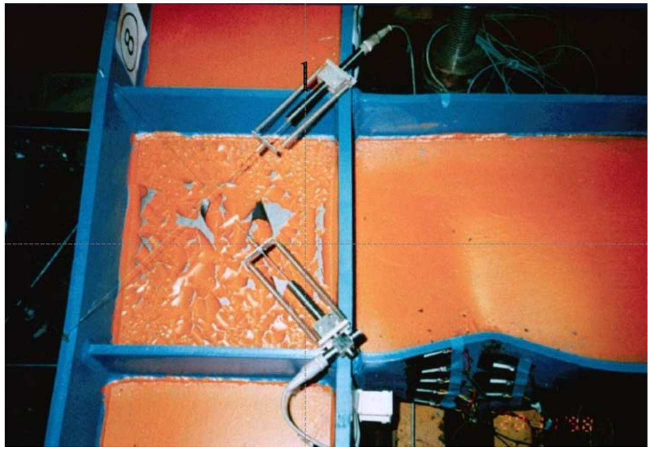
Detailing rules for steel structures

MRF

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





CHAPTER 6

Design criteria for steel structures

Detailing rules for steel structures

MRF

Detailing rules for MRF

Beams

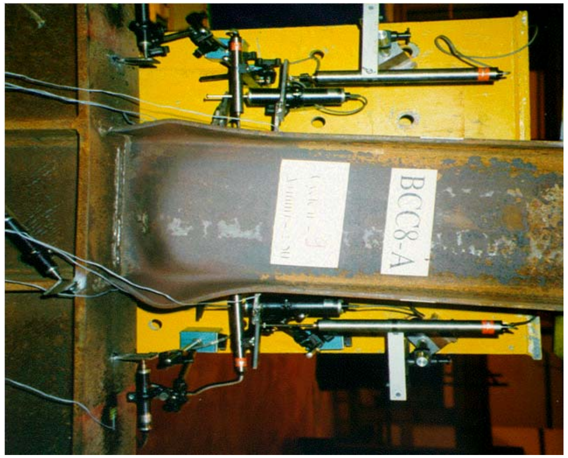
For plastic hinges in the beams it should be verified that the full plastic moment resistance and rotation capacity are not decreased by compression and shear force.

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

CHAPTER 6

Design criteria for steel structures

Detailing rules for steel structures

MRF

Detailing rules for MRF

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

CHAPTER 6

Design criteria for steel structures

Detailing rules for steel structures

MRF

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

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:

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

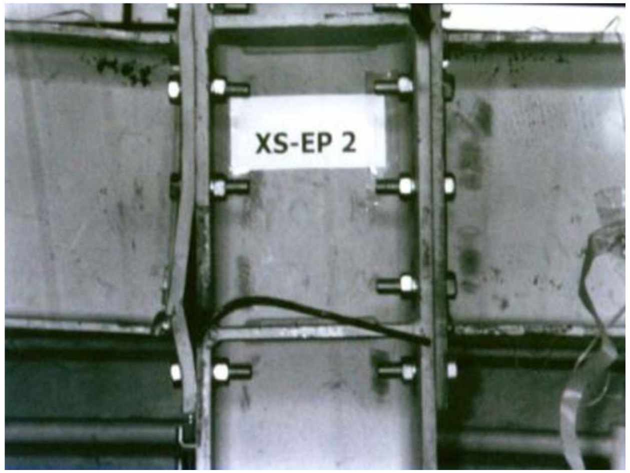
where:

$M_{j,Rd}$ 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



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:

CHAPTER 6

Design criteria for steel structures

Detailing rules for steel structures

MRF

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

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

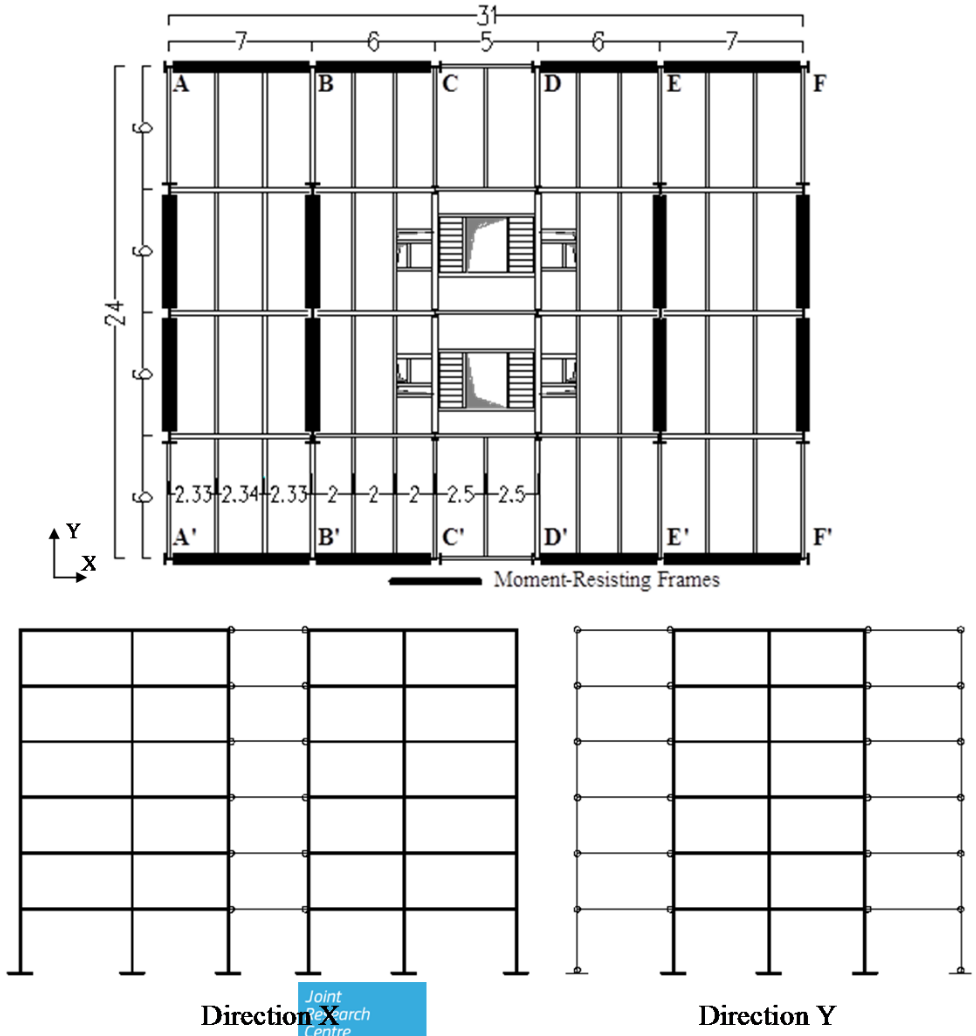
Calculation example

CHAPTER 6

Design criteria for steel structures

Detailing rules for steel structures

MRF



Calculation example

Characteristic values of vertical persistent and transient actions

	G_k (kN/m^2)	Q_k (kN/m^2)
Storey slab	4.20	2.00
Roof slab	3.60	0.50 1.00 (Snow)
Stairs	1.68	4.00
Claddings	2.00	

Calculation example

Seismic action:

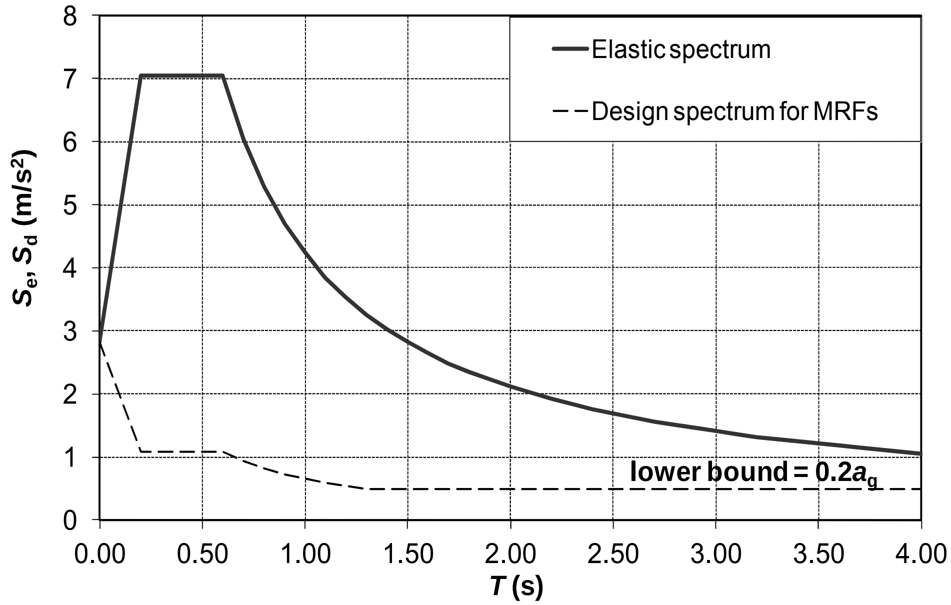
Elastic and design response spectra

CHAPTER 6

Design criteria for steel structures

Detailing rules for steel structures

MRF



behaviour factor q was assigned according to EN 1998-1 (6.3.2) as follows:

$$q = \frac{\alpha_u}{\alpha_1} \cdot q_o = 1.3 \cdot 5 = 6.5$$

CHAPTER 6

Design criteria for steel structures

Detailing rules for steel structures

MRF

Calculation example

Combination of actions

In case of buildings the seismic action should be combined with permanent and variable loads as follows:

$$\sum G_{k,i} "+" \sum \psi_{2,i} \cdot Q_{k,i} "+" A_{Ed}$$

Type of variable actions	ψ_{2i}
Category A – Domestic, residential areas	0.30
Roof	0.30
Snow loads on buildings	0.20
Stairs	0.80

CHAPTER 6

Design criteria for steel structures

Detailing rules for steel structures

MRF

Calculation example

Masses

In accordance with EN 1998-1 3.2.4 (2)P, the inertial effects in the seismic design situation have to be evaluated by taking into account the presence of the masses corresponding to the following combination of permanent and variable gravity loads:

$$\sum G_{k,i} \text{"+"} \sum \psi_{E,i} \cdot Q_{k,i}$$

Type of variable actions	ψ_{2i}	φ	ψ_{Ei}
Category A – Domestic, residential areas	0.30	0.50	0.15
Roof	0.30	1.00	0.30
Snow loads on buildings	0.20	1.00	0.20
Stairs	0.80	0.50	0.40

Calculation example

Seismic weights and masses in the worked example

CHAPTER 6

Design criteria for steel structures

Detailing rules for steel structures

MRF

Storey	G_k (kN)	Q_k (kN)	Seismic Weight (kN)	Seismic Weight (kN/m^2)	Seismic Mass (kN s^2/m)
VI	3256.27	1326.00	3579.67	4.81	364.90
V	3992.08	1608.00	4233.28	5.69	431.53
IV	3994.08	1608.00	4235.28	5.69	431.73
III	4020.54	1608.00	4261.74	5.73	434.43
II	4034.87	1608.00	4276.07	5.75	435.89
I	4092.99	1608.00	4334.19	5.83	441.81

Calculation example

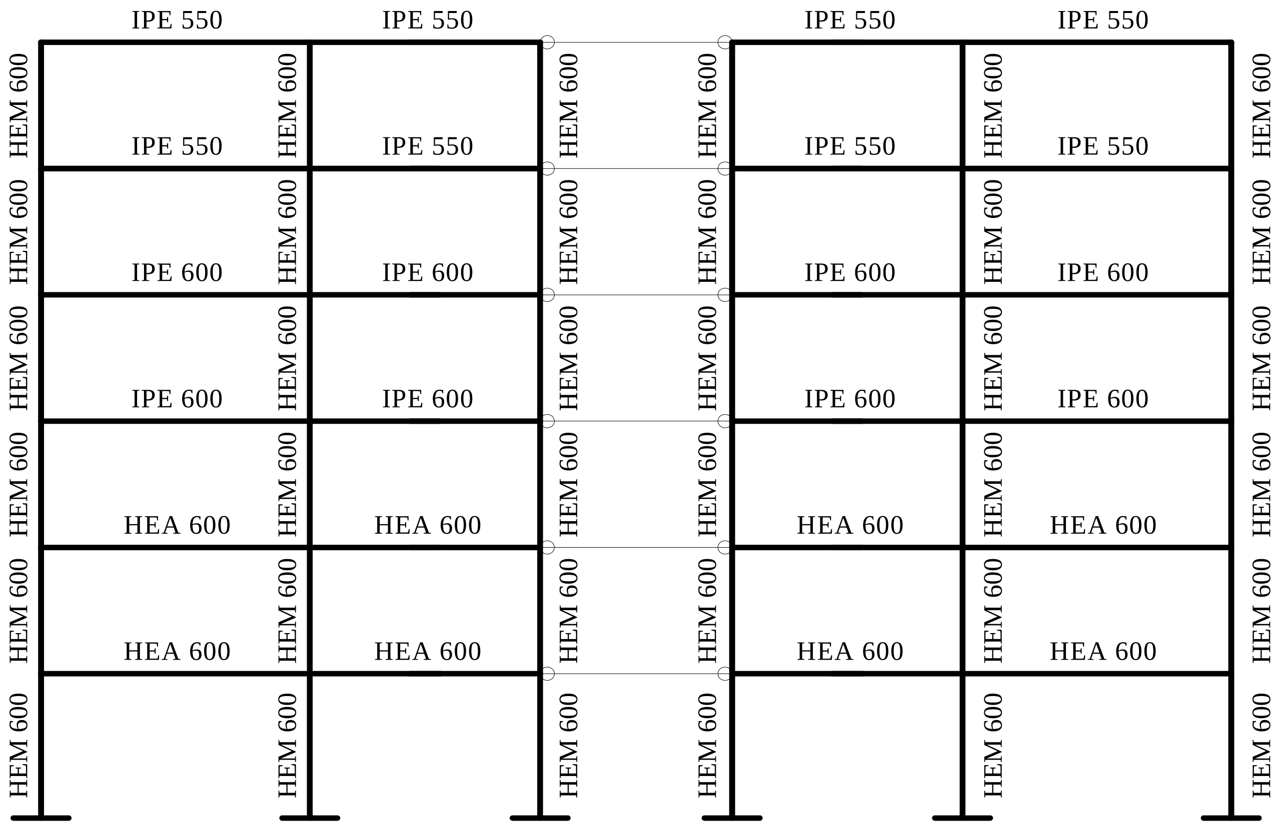
Verifications

CHAPTER 6

Design criteria for steel structures

Detailing rules for steel structures

MRF



Calculation example

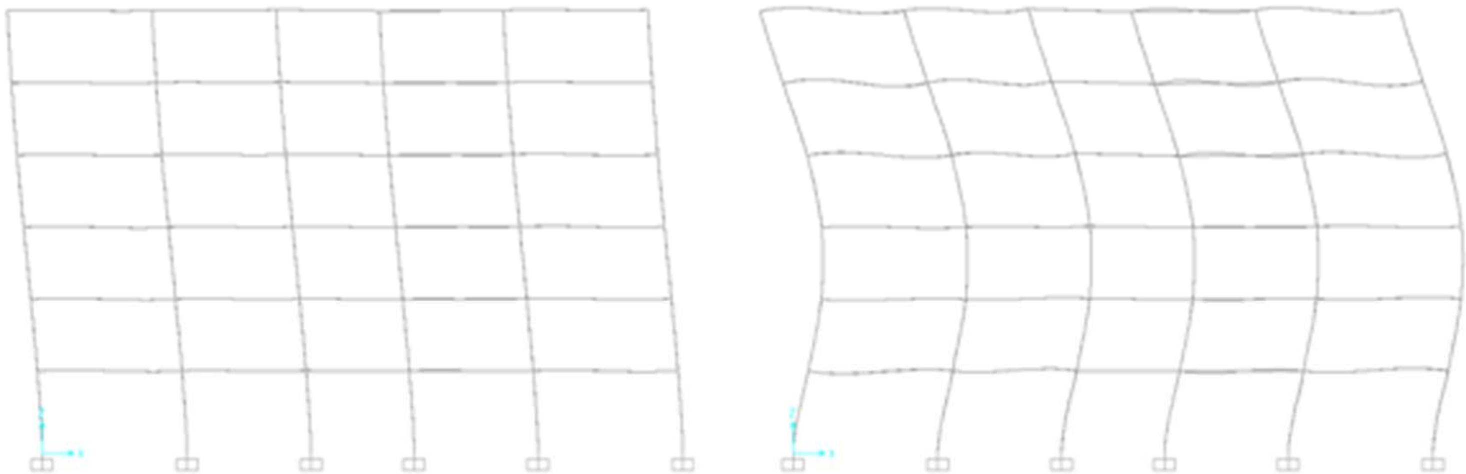
Verifications

CHAPTER 6

Design criteria for steel structures

Detailing rules for steel structures

MRF



$T_1 = 1.089\text{s}; M_1 = 0.793$ $T_2 = 0.356\text{s}; M_2 = 0.131$

Dynamic properties in X direction

CHAPTER 6

Design criteria for steel structures

Detailing rules for steel structures

MRF

Calculation example

the coefficient ϑ ranges within 0.1÷0.2 at the lower storeys (namely those indicated in bold, which are from storey 1 to 4). Hence, to take into account second order effects the seismic effects were magnified through the relevant multiplier α , which is calculated at each storey having $\vartheta > 0.1$

Stability coefficients calculated in X direction

Storey	P_{tot}	V_{tot}	h	$d_r = d_e \times q$	$\theta = \frac{P_{tot} \cdot d_r}{V_{tot} \cdot h}$	$\alpha = \frac{1}{(1-\theta)}$
	(kN)	(kN)	(mm)	(mm)	(-)	(-)
VI	1841.93	313.29	3500	253	0.05	1.05
V	4004.27	539.46	3500	223	0.09	1.09
IV	6176.76	707.88	3500	183	0.12	**1.14**
III	8351.27	842.52	3500	135	0.14	**1.17**
II	10563.00	953.75	3500	84	0.15	**1.18**
I	12807.69	1025.18	4000	36	0.11	**1.13**

Centre

CHAPTER 6

Design criteria for steel structures

Detailing rules for steel structures

MRF

Calculation example

flexural checks for beams belonging to MRF in X direction:

Span	Storey	Left end $M_{Ed,G}$ (kNm)	Left end $M_{Ed,E}$ (kNm)	Left end M_{Ed} (kNm)	Left end Ω_i	Right end $M_{Ed,G}$ (kNm)	Right end $M_{Ed,E}$ (kNm)	Right end M_{Ed} (kNm)	Right end Ω_i	Ω_{min}
A-B	VI	59.39	110.45	169.84	4.51	55.86	102.81	158.67	4.83	
	V	82.89	156.86	239.75	3.20	74.49	152.34	226.84	3.38	
	IV	82.60	252.84	369.95	2.61	74.68	240.38	347.87	2.78	
	III	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	
	II	81.62	381.72	531.76	2.77	77.92	354.88	496.41	2.96	
	I	76.72	339.87	$\boxed{460.10}$	3.20	81.50	312.77	434.31	3.39	**2.32**
Span B-C	VI	46.19	106.74	152.93	5.01	41.96	115.27	157.23	4.87	
	V	57.96	163.18	221.14	3.47	58.59	168.04	226.63	3.38	
	IV	60.56	258.75	354.62	2.72	59.52	272.17	368.82	2.62	
	III	59.83	294.20	403.36	2.39	57.45	307.83	416.89	**2.32**	
	II	62.30	383.11	514.09	2.86	56.59	411.86	542.27	2.71	
	I	65.58	340.87	450.09	3.27	52.92	369.45	469.66	3.13	

$$\boxed{M_{Ed} = M_{Ed,G} + \alpha \cdot M_{Ed,E} = 76.72kNm + 1.13 \cdot 339.87kNm = 460.10kNm}$$

CHAPTER 6

Design criteria for steel structures

Detailing rules for steel structures

MRF

Calculation example

flexural checks for columns belonging to MRF in X direction:

$$M_{\text{Ed}} = M_{\text{Ed,G}} + \alpha \cdot 1.1 \cdot \gamma_{\text{ov}} \cdot \Omega \cdot M_{\text{Ed,E}} =$$
$$= 12.06kNm + (1.13 \cdot 1.1 \cdot 1.25 \cdot 2.32) \cdot 509.64kNm = 1843.28kNm$$

$$N_{\text{Ed}} = N_{\text{Ed,G}} + \alpha \cdot 1.1 \cdot \gamma_{\text{ov}} \cdot \Omega \cdot N_{\text{Ed,E}} =$$
$$= 590.11kN + (1.13 \cdot 1.1 \cdot 1.25 \cdot 2.32) \cdot 410.72kN = 2065.89kN$$

	V	38.24	164.55	562.40	167.59	74.53	405.00	3114.06	5.54
	I	31.91	115.02	445.20	578.91	410.72	2054.69	3114.06	6.99
bottom end	VI	44.65	45.37	189.17	75.37	30.47	172.41	3114.06	16.46
	V	40.81	97.71	352.04	177.40	74.53	414.80	3114.06	8.85
	IV	41.13	142.34	556.43	279.95	144.11	801.63	3114.06	5.60
	III	40.08	213.96	835.91	382.24	221.10	1204.62	3114.06	3.73
	II	44.81	240.19	947.06	486.08	322.09	1695.95	3114.06	3.29
	I	12.06	509.64	1843.28	590.11	410.72	2065.89	3114.06	1.69

Calculation example

Local hierarchy criterion for external and inner columns in X

CHAPTER 6

Design criteria for steel structures

Detailing rules for steel structures

MRF

$$\sum M_{\text{Rc}} = 2 \cdot M_{\text{Rc}} = 2 \cdot 3114.06kNm = 6228.12kNm$$

$$\sum M_{\text{Rb}} = 2 \cdot M_{\text{Rb}} = 2 \cdot 1471.25kNm = 2942.5kNm$$

$$\frac{\sum M_{\text{Rc}}}{\sum M_{\text{Rb}}} = 2.12 > 1.3$$

						ertical B)	
						b, right side (kNm)	$\frac{\sum M_{\text{Rc}}}{\sum M_{\text{Rb}}}$
VI	3114.06	766.43	4.06	3114.06	766.43	766.43	2.03
V		766.43	8.13		965.80	965.80	4.06
IV		965.80	6.45		1471.25	1471.25	3.22
III		965.80	6.45		766.43	766.43	3.22
II		1471.25	4.23		965.80	965.80	2.12
I		1471.25	4.23		1471.25	1471.25	$\boxed{2.12}$

Detailing rules for CBF

CHAPTER 6

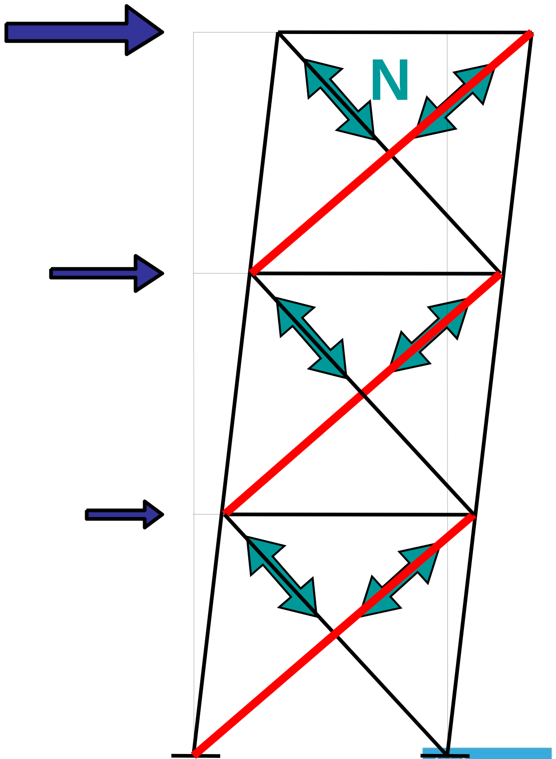
Design criteria for steel structures

Detailing rules for steel structures

CBF

Concentric Braced Frames

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



Detailing rules for CBF

CHAPTER 6

Design criteria for steel structures

Detailing rules for steel structures

CBF

CHAPTER 6

Design criteria for steel structures

Detailing rules for steel structures

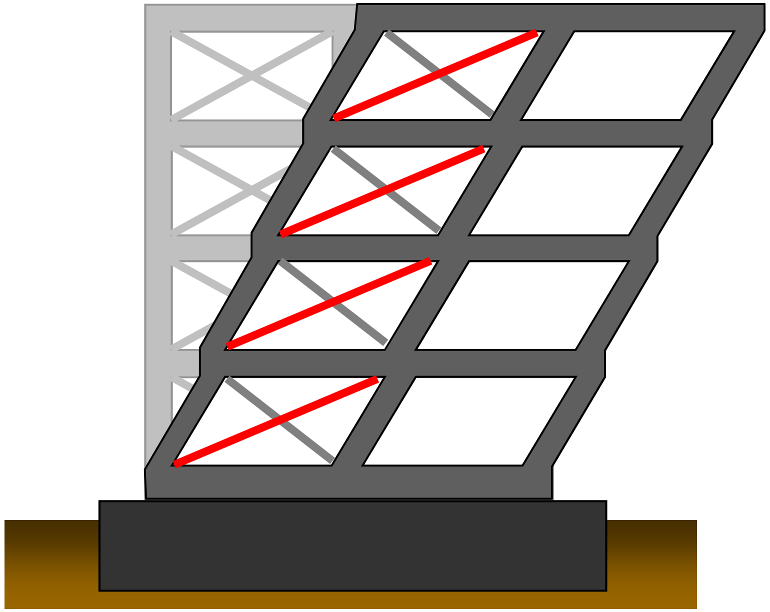
CBF

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

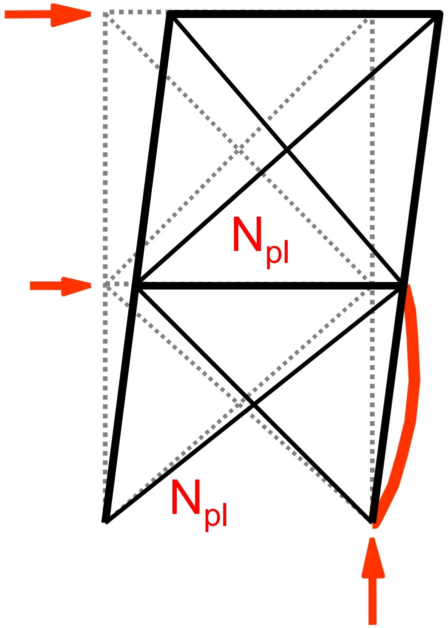
CHAPTER 6

Design criteria for steel structures

Detailing rules for steel structures

CBF

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



CHAPTER 6

Design criteria for steel structures

Detailing rules for steel structures

CBF

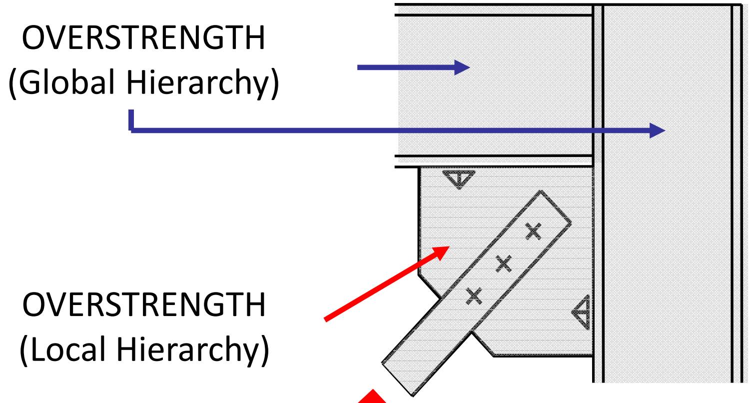
Detailing rules for CBF

Basic Principles

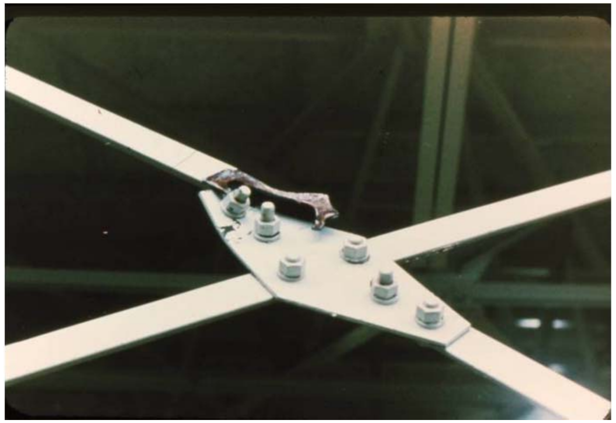
Local capacity design:

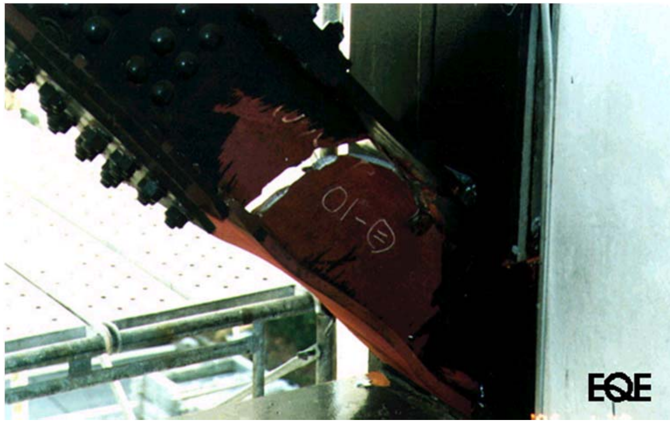
Allows the formation of local plastic mechanisms and ensures the transfer of full plastic forces

Concerns mainly connections



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





Detailing rules for CBF

CHAPTER 6

Design criteria for steel structures

Detailing rules for steel structures

CBF

Diagonal members

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

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

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%**.

CHAPTER 6

Design criteria for steel structures

Detailing rules for steel structures

CBF

Detailing rules for CBF

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:

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

CHAPTER 6

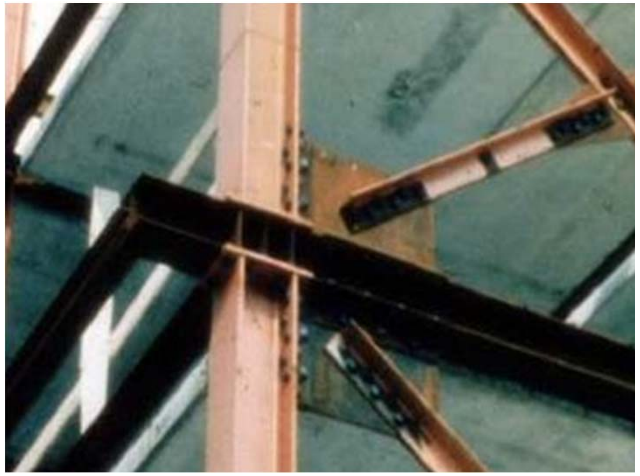
Design criteria for steel structures

Detailing rules for steel structures

CBF

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 -V bracing

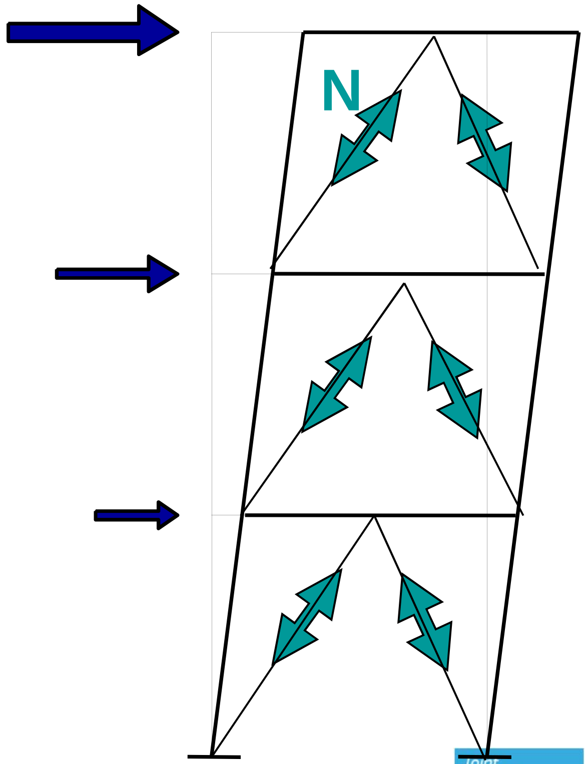
CHAPTER 6

Design criteria for steel structures

Detailing rules for steel structures

CBF

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



Detailing rules for CBF -V bracing

CHAPTER 6

Design criteria for steel structures

Detailing rules for steel structures

CBF

Detailing rules for CBF -V bracing

CHAPTER 6

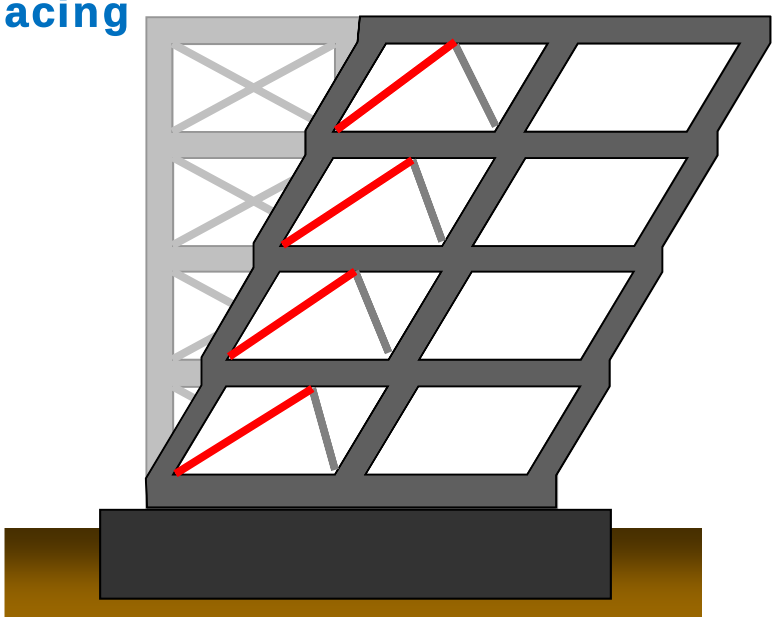
Design criteria for steel structures

Detailing rules for steel structures

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 -V bracing

CHAPTER 6

Design criteria for steel structures

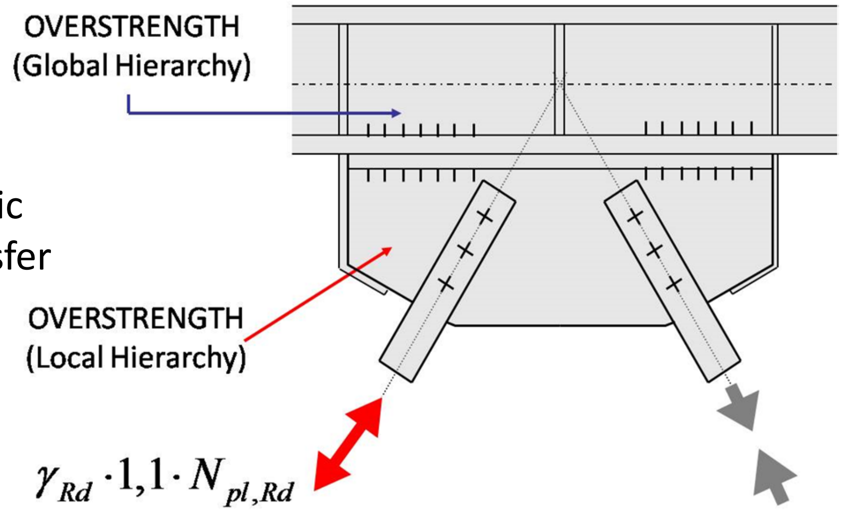
Detailing rules for steel structures

CBF

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



CHAPTER 6

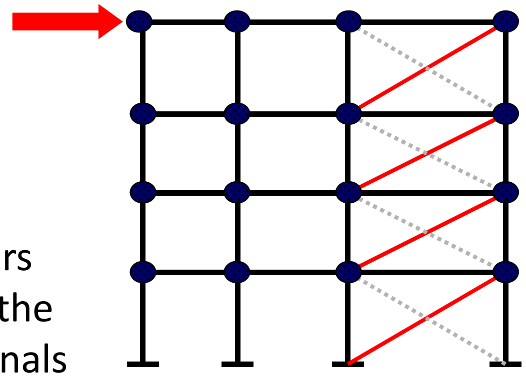
Design criteria for steel structures

Detailing rules for steel structures

CBF

Detailing rules for CBF -V bracing

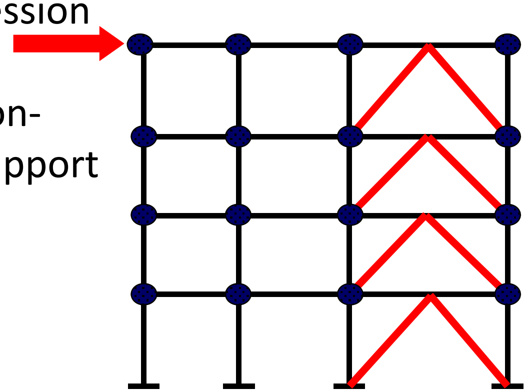
Diagonal Bracings



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.



CHAPTER 6

Design criteria for steel structures

Detailing rules for steel structures

CBF

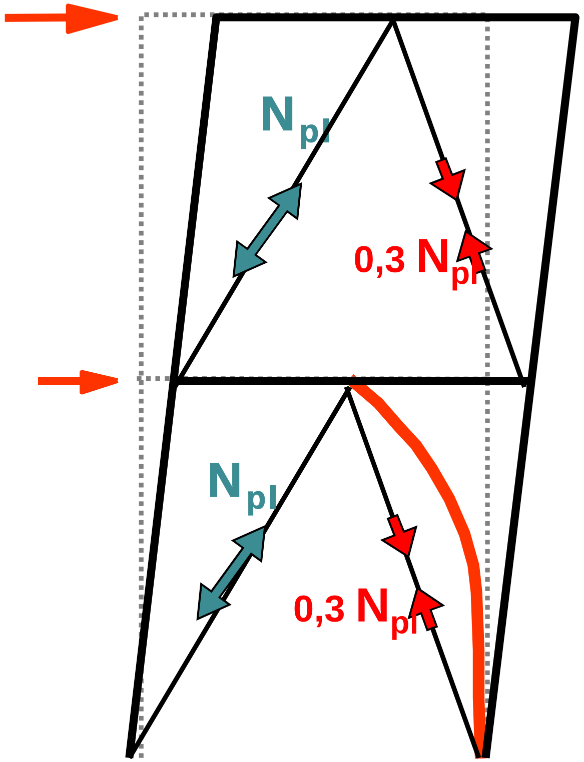
Detailing rules for CBF -V bracing

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

γ_{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 bracing

CHAPTER 6

Design criteria for steel structures

Detailing rules for steel structures

CBF

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 bracing

CHAPTER 6

Design criteria for steel structures

Detailing rules for steel structures

CBF

Beams and Columns

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

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

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 bracing

Connections

CHAPTER 6

Design criteria for steel structures

Detailing rules for steel structures

CBF

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

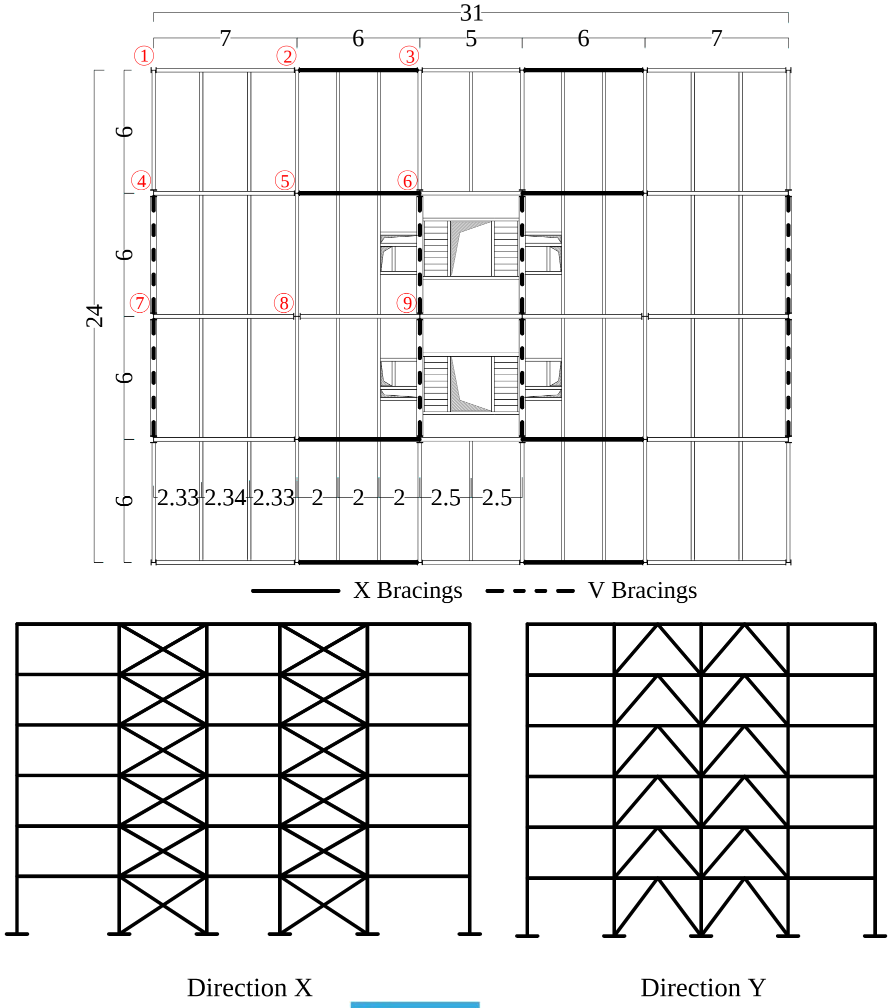
Calculation example

CHAPTER 6

Design criteria for steel structures

Detailing rules for steel structures

CBF



CHAPTER 6

Design criteria for steel structures

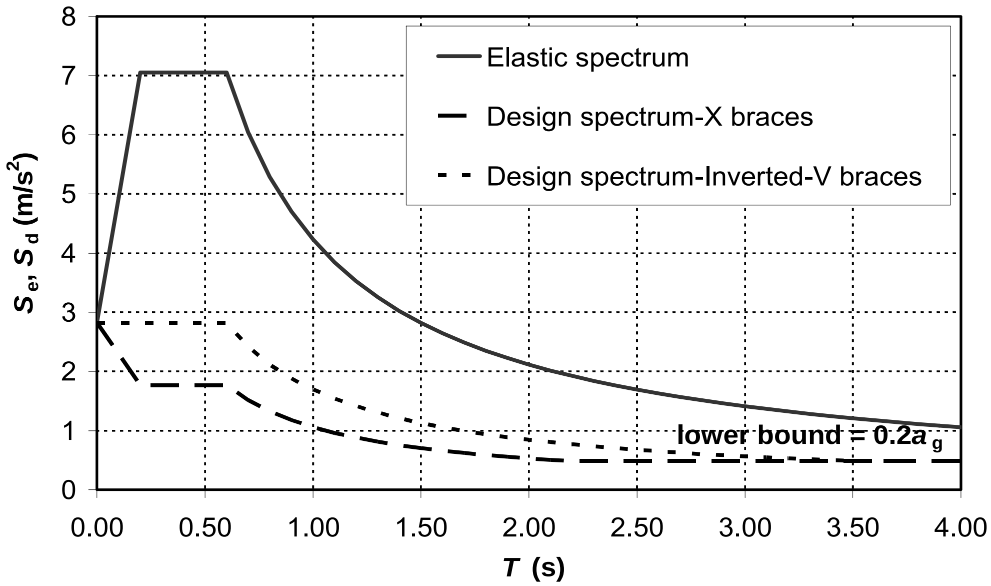
Detailing rules for steel structures

CBF

Calculation example

Seismic action

Elastic and design response spectra



behaviour factor q was assigned according to EC8 (DCH concept) as follows:

$$q = 4 \quad \text{for X-CBFs}$$

$$q = 2.5 \quad \text{for inverted V-CBFs}$$

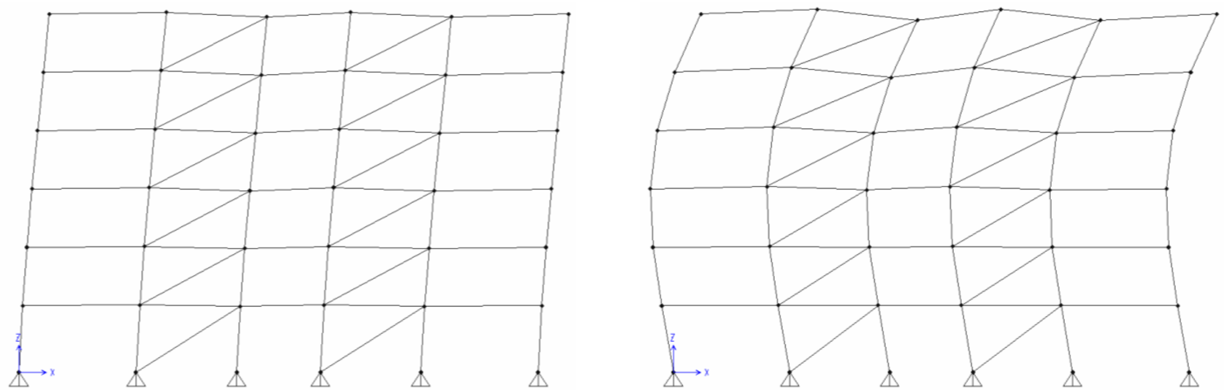
Calculation example

CHAPTER 6

Design criteria for steel structures

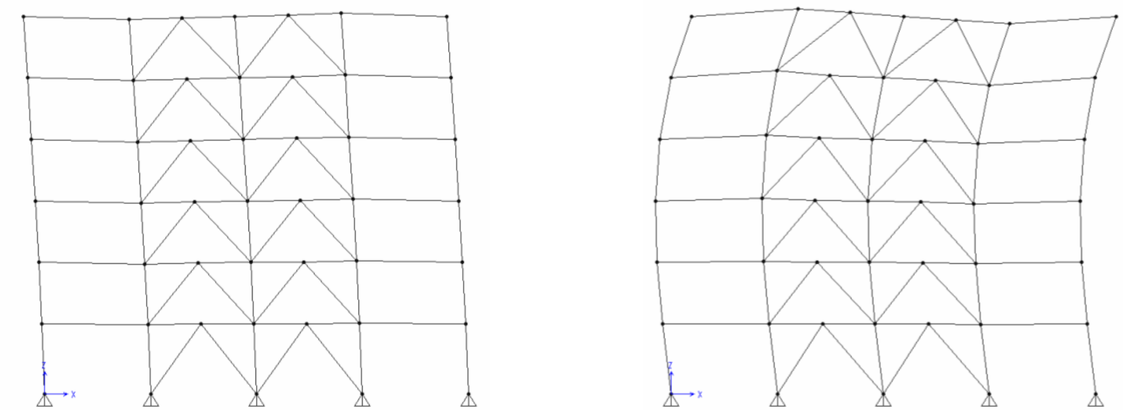
Detailing rules for steel structures

CBF



$T_1 = 0.874\text{s}; M_1 = 0.759$ $T_2 = 0.316\text{s}; M_2 = 0.161$

Dynamic properties in X direction



$T_1 = 0.455\text{s}; M_1 = 0.765$ $T_2 = 0.176\text{s}; M_2 = 0.156$

Dynamic properties in Y direction

CHAPTER 6

Design criteria for steel structures

Detailing rules for steel structures

CBF

Calculation example

Verification of Braces: X-CBF

The circular hollow sections are suitable to satisfy both the slenderness limits ($1.3 < \bar{\lambda} \leq 2.0$) and the requirement of minimizing the variation among the diagonals of the overstrength ratio Ω_i, whose maximum value (Ω_{max}) must not differ from the minimum one (Ω_{min}) by more than 25%. .

Storey	Brace cross section (d x t) (mm x mm)	λ	$\bar{\lambda}$	$N_{pl,Rd}$ (kN)	N_{Ed} (kN)	$\Omega_i = \frac{N_{pl,Rd}}{N_{Ed}}$	$\frac{\Omega_i - \Omega_{min}}{\Omega_{min}}$ (x 100)
VI	114.3x4	178.10	1.90	326.65	180.65	1.81	16.70
V	121x6.3	171.08	1.82	533.45	325.70	1.64	5.71
IV	121x8	173.22	1.85	667.40	430.74	**1.55**	0.00
III	121x10	176.29	1.88	820.15	517.46	1.58	2.29
II	133x10	159.31	1.70	907.10	576.19	1.57	1.61
I	159x10	136.57	1.45	1099.80	650.07	1.69	9.19

Calculation example

CHAPTER 6

Design criteria for steel structures

Detailing rules for steel structures

CBF

Verification of beams: X-CBF

Storey	Section	N_{Rd} (kN)	$N_{Ed,G}$ (kN)	$N_{Ed,E}$ (kN)	$N_{Ed}=N_{Ed,G}+1.1\gamma_{ov}\Omega N_{Ed,E}$ (kN)	$\frac{N_{Rd}}{N_{Ed}}$
VI	IPE 360			156.05	265.96	9.70
V	IPE 360			281.34	479.51	5.38
IV	IPE 360	2580.85	0.00	372.07	634.15	4.07
III	IPE 360			446.98	761.82	3.39
II	IPE 360			497.72	848.29	3.04
I	IPE 360			540.90	921.90	2.80

Storey	$N_{Ed,G}$ (kN)	$N_{Ed,E}$ (kN)	$N_{Ed}=N_{Ed,G}+1.1\gamma_{ov}\Omega N_{Ed,E}$ (kN)	$M_{Ed,G}$ (kNm)	$M_{Ed,E}$ (kNm)	$M_{Ed}=M_{Ed,G}+1.1\gamma_{ov}\Omega M_{Ed,E}$ (kNm)	$M_{N,Rd}$ (kNm)	$\frac{M_{Rd}}{M_{Ed}}$
VI		78.02	132.98	64.28		64.28	361.75	5.63
V		218.70	372.74	86.27		86.27	361.75	4.19
IV	0.00	326.71	556.83	86.27	0.00	86.27	355.97	4.13
III		409.53	697.99	86.27		86.27	331.14	3.84
II		472.35	805.06	86.27		86.27	312.31	3.62
I		510.16	869.51	86.27		86.27	300.98	3.49

CHAPTER 6

Design criteria for steel structures

Detailing rules for steel structures

CBF

Calculation example

Verification of columns: X-CBF

Storey	Section	A (mm^2)	χ	$N_{pl,Rd}$ (kN)	$N_{Ed,G}$ (kN)	$N_{Ed,E}$ (kN)	$N_{Ed}= N_{Ed,G}+1.1\gamma_{ov}\Omega N_{Ed,E}$ (kN)	$\frac{\chi N_{pl,Rd}}{N_{Ed}}$
column type "a"								
VI	HE180A	4530	0.59	1608.15	103.77	0.00	103.77	9.12
V	HE180A	4530	0.59	1608.15	237.62	91.03	392.76	2.41
IV	HE240B	10600	0.75	3763.00	372.52	253.90	805.26	3.52
III	HE240B	10600	0.75	3763.00	507.15	465.92	1301.24	2.18
II	HE240M	19960	0.77	7085.80	646.06	716.86	1867.85	2.94
I	HE240M	19960	0.71	7085.80	786.00	994.39	2480.80	2.03
column type "b"								
VI	HE180A	4530	0.59	1608.15	92.33	91.03	247.47	3.82
V	HE180A	4530	0.59	1608.15	214.20	253.90	646.94	1.46
IV	HE240B	10600	0.75	3763.00	338.31	465.92	1132.41	2.50
III	HE240B	10600	0.75	3763.00	461.08	716.86	1682.87	1.68
II	HE240M	19960	0.77	7085.80	586.39	994.39	2281.19	2.40
I	HE240M	19960	0.71	7085.80	710.44	1341.94	2997.59	1.68

CHAPTER 6

Design criteria for steel structures

Detailing rules for steel structures

CBF

Calculation example

Verification of Braces: inverted V-CBF

design checks in tension

Storey	Brace cross section (d x t) (mm x mm)	$N_{pl,Rd}$ (kN)	$N_{Ed,\ D1}$ (kN)	$\Omega_i = \dfrac{N_{pl,Rd}}{N_{Ed\ d,D1}}$	$\dfrac{\Omega_i - \Omega}{\Omega}$(x 100)
VI	127x6.3	561.65	245.60	2.29	2.04
V	193.7x8	1097.45	461.96	2.38	6.00
IV	244.5x8	1395.90	622.87	**2.24**	0.00
III	244.5x10	1722.55	756.68	2.28	1.58
II	273x10	1941.10	843.92	2.30	2.63
I	323.9x10	2317.10	986.84	2.35	4.77

CHAPTER 6

Design criteria for steel structures

Detailing rules for steel structures

CBF

Calculation example

Verification of Braces: inverted V-CBF

design checks in compression

Storey	Brace cross section (d x t) (mm x mm)	λ	$\overline{\lambda}$	χ	$N_{b,Rd}$ (kN)	$N_{Ed,\ D1}$ (kN)	$\frac{N_{b,Rd}}{N_{Ed,D1}}$
VI	127x6.3	107.94	1.15	0.56	315.86	245.60	1.29
V	193.7x8	70.15	0.75	0.82	904.70	461.96	1.96
IV	244.5x8	55.07	0.59	0.89	1249.31	622.87	2.01
III	244.5x10	55.53	0.59	0.89	1538.50	756.68	2.03
II	273x10	49.51	0.53	0.92	1777.16	843.92	2.11
I	323.9x10	45.05	0.48	0.93	2155.83	986.84	2.18

Calculation example

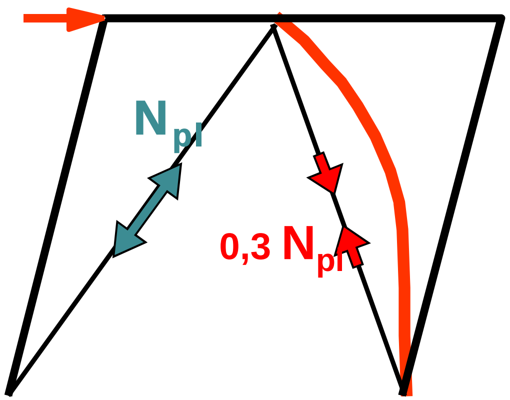
CHAPTER 6

Verification of beams: inverted V-CBF

Design criteria for steel structures

Detailing rules for steel structures

CBF



Storey	Section	N_{Ed} (kN)	$M_{Ed,G}$ (kNm)	$M_{Ed,E}$ (kNm)	M_{Ed} (kNm)	M_{Rd} (kNm)	$\frac{M_{Rd}}{M_{Ed}}$
VI	HE320 B	475.25	41.90	447.83	489.73	762.90	1.56
V	HE320 M	928.63	58.13	875.05	933.19	1574.43	1.69
IV	HE360 M	1181.17	58.35	1113.02	1171.38	1771.10	1.51
III	HE450 M	1457.57	58.62	1373.48	1432.10	2247.51	1.57
II	HE500 M	1642.50	59.24	1547.74	1606.98	2518.37	1.57
I	HE550 M	1807.34	61.28	1946.36	2007.64	2816.22	1.40

Calculation example

CHAPTER 6

Verification of columns: inverted V-CBF

Design criteria for steel structures

Detailing rules for steel structures

CBF

Storey	Section	A (mm^2)	χ	$N_{pl,Rd}$ (kN)	$N_{Ed,G}$ (kN)	$N_{Ed,E}$ (kN)	$N_{Ed}= N_{Ed,G}+1.1\gamma_{ov}\Omega N_{Ed,E}$ (kN)	$\frac{\chi N_{pl,Rd}}{N_{Ed}}$
VI	HE180A	4530	0.59	1608.15	94.72	0.00	94.72	9.99
V	HE180A	4530	0.59	1608.15	225.44	182.06	674.27	1.40
IV	HE240M	19960	0.77	7085.80	384.77	527.24	1684.50	3.26
III	HE240M	19960	0.77	7085.80	534.95	984.00	2960.71	1.85
II	HE320M	31200	0.85	11076.00	694.41	1535.70	4480.22	2.10
I	HE320M	31200	0.81	11076.00	847.88	2139.46	6122.07	1.46

Detailing rules for EBF

Eccentric Braced Frames

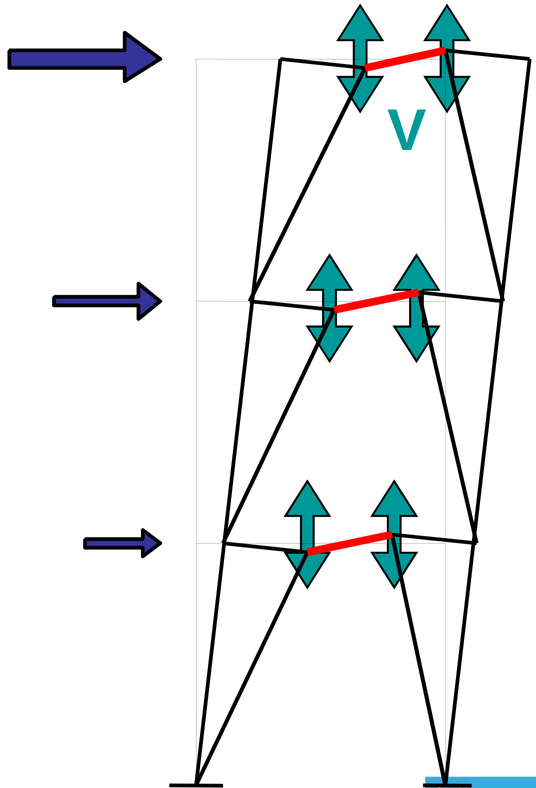
CHAPTER 6

Design criteria for steel structures

Detailing rules for steel structures

EBF

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



Detailing rules for EBF

CHAPTER 6

Design criteria for steel structures

Detailing rules for steel structures

EBF

Detailing rules for EBF

Design Concept

CHAPTER 6

Design criteria for steel structures

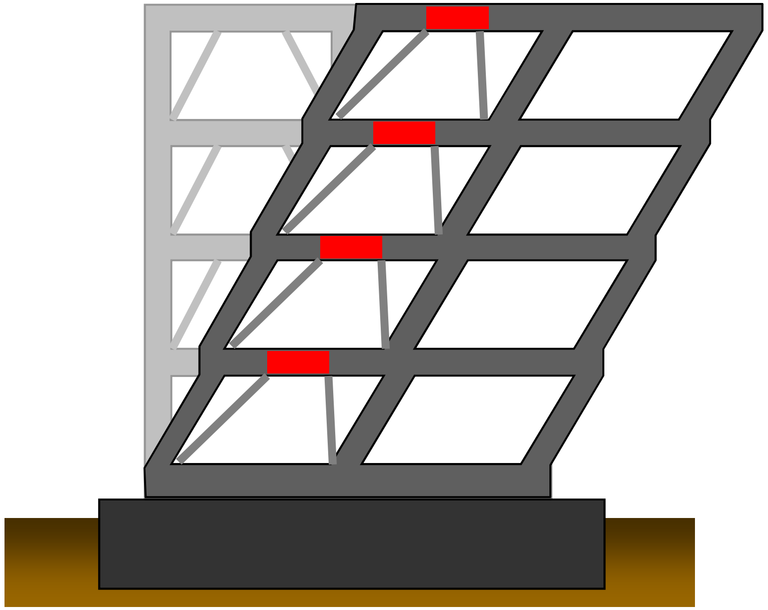
Detailing rules for steel structures

EBF

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.

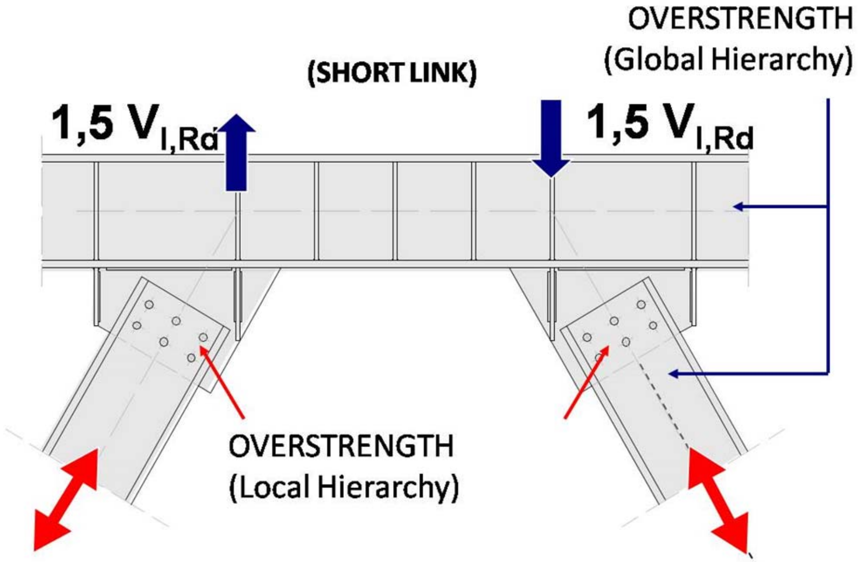


Detailing rules for EBF

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

Classification of seismic links

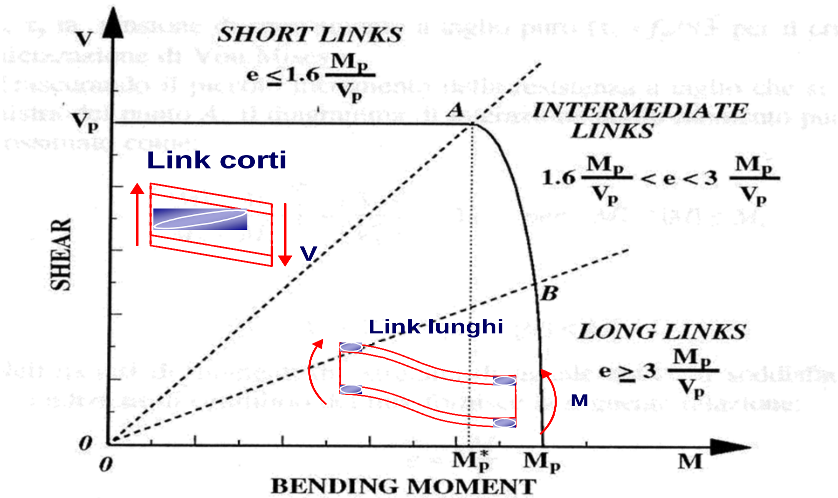
CHAPTER 6

Design criteria for steel structures

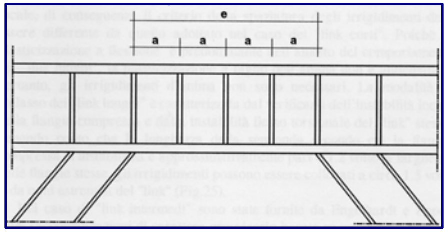
Detailing rules for steel structures

EBF

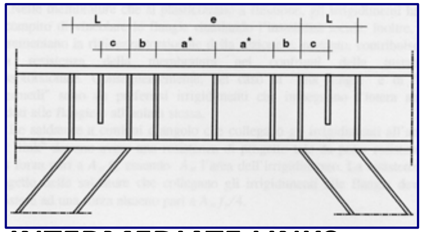
Seismic links are classified into 3 categories according to the type of plastic mechanism developed:



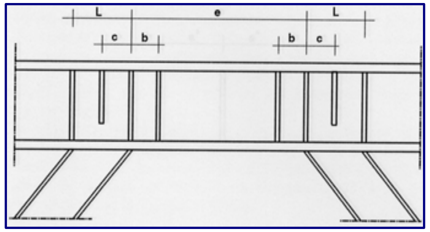
SHORT LINKS
dissipate energy by yielding essentially in shear



LONG LINKS
dissipate energy by yielding essentially in bending



INTERMEDIATE LINKS
the plastic mechanism involves bending and shear



Detailing rules for EBF

CHAPTER 6

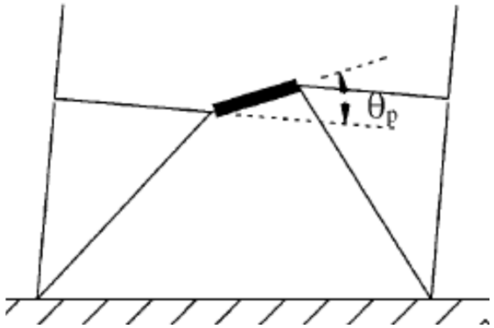
Design criteria for steel structures

Detailing rules for steel structures

EBF

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 :

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

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

CHAPTER 6

Design criteria for steel structures

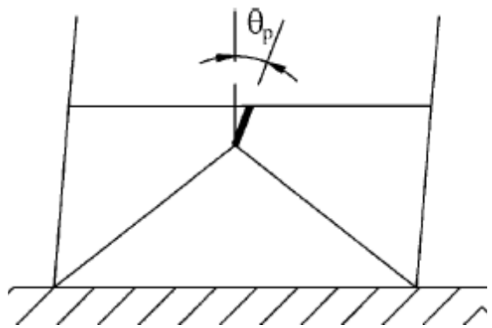
Detailing rules for steel structures

EBF

Detailing rules for EBF

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:

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

LONG LINKS:

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

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 :

Detailing rules for EBF

CHAPTER 6

Design criteria for steel structures

Detailing rules for steel structures

EBF

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 $M_{p,link,r}$

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

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

CHAPTER 6

Design criteria for steel structures

Detailing rules for steel structures

EBF

Detailing rules for EBF

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

CHAPTER 6

Design criteria for steel structures

Detailing rules for steel structures

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

CHAPTER 6

Design criteria for steel structures

Detailing rules for steel structures

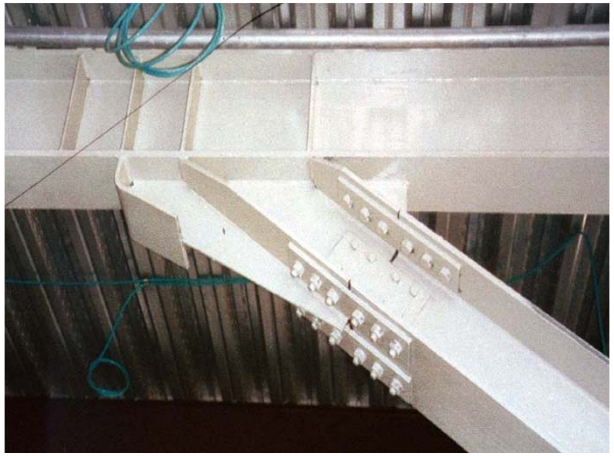
EBF

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.

Future of EU seismic codes

ECCS CECM EKS

European Convention for Constructional Steelwork

Technical Committee n.13 “Seismic Design” - TC13

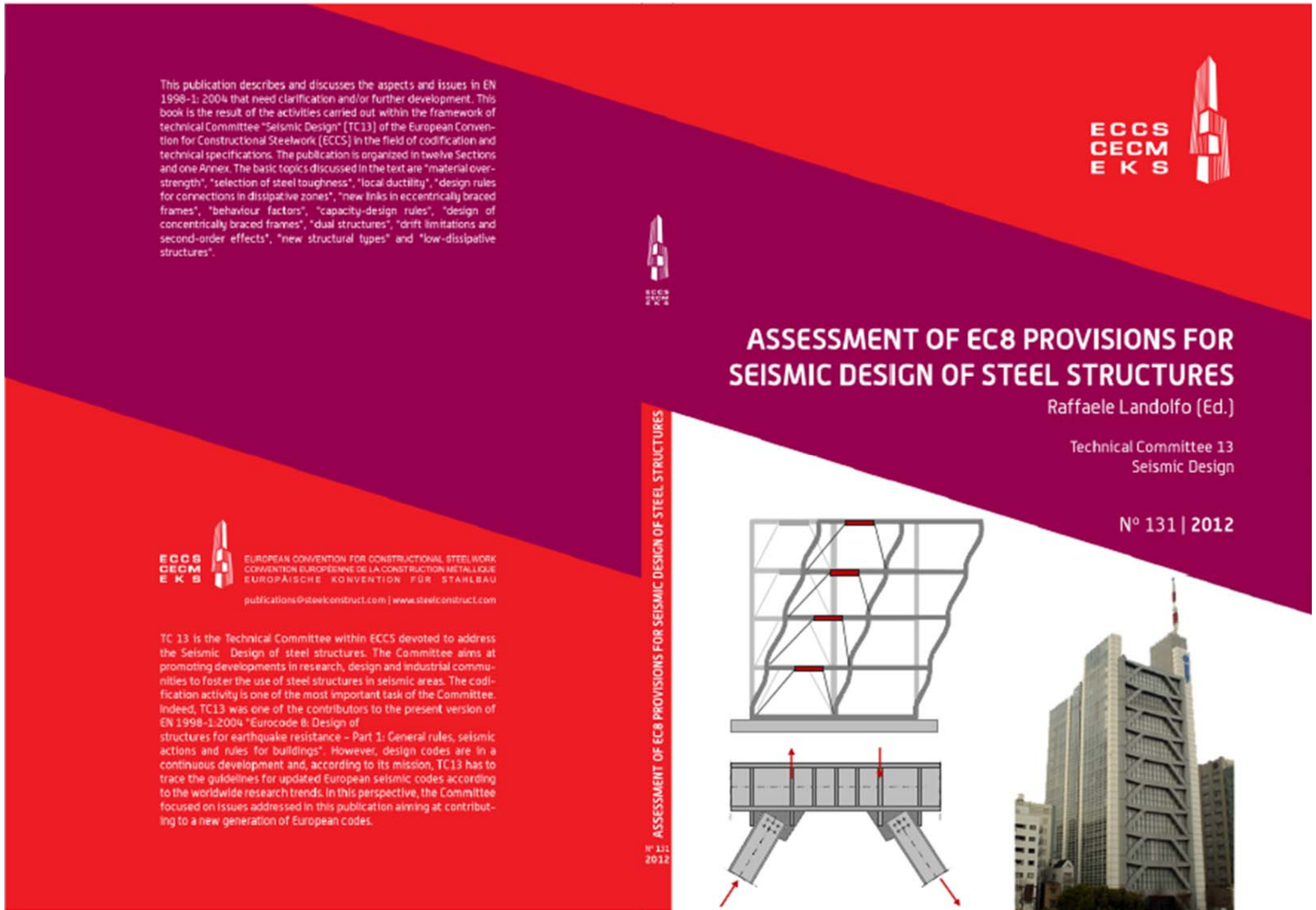
TC13 MISSION

TC13 is devoted to the topic of seismic design with the mission to promote the use of steel in seismic regions.

TC13 CHAIRMAN

Raffaele Landolfo

TC13 PUBLICATIONS



References

EN 1998-1-1. Eurocode 8: Design of structures for earthquake resistance—Part 1:General rules, seismic actions and rules for buildings. CEN; 2004.

EN 1998-3. Eurocode 8: Design of structures for earthquake resistance—Part 3: Assessment and retrofitting of buildings. CEN; 2005.

EN 1993-1. Eurocode 3: Design of steel structures—Part 1-1: General rules and rules for buildings. CEN; 2005.

Landolfo Raffaele, editor. Assessment of EC8 provisions for seismic design of steel structures. Technical Committee 13—Seismic Design, No 131/2013. ECCS—European Convention for Constructional Steelwork; 2013.

R. Landolfo, F. M. Mazzolani, D. Dubina, L. da Silva, M. D'Aniello. DESIGN OF STEEL STRUCTURES FOR BUILDINGS IN SEISMIC AREAS. ECCS—European Convention for Constructional Steelwork; (in preparation)

Thanks for your kind attention

landolfo@unina.it