16TH EUROPEAN CONFERENCE ON
EARTHQUAKE ENGINEERING
THESSALONIKI 18 - 21 JUNE 2018

SEISMIC DESIGN OF STEEL STRUCTURES:

NEW TRENDS OF RESEARCH AND UPDATES OF EUROCODE 8

UNINA
UNIversità di NApoli "Federico II"
Prof. Raffaele Landolfo

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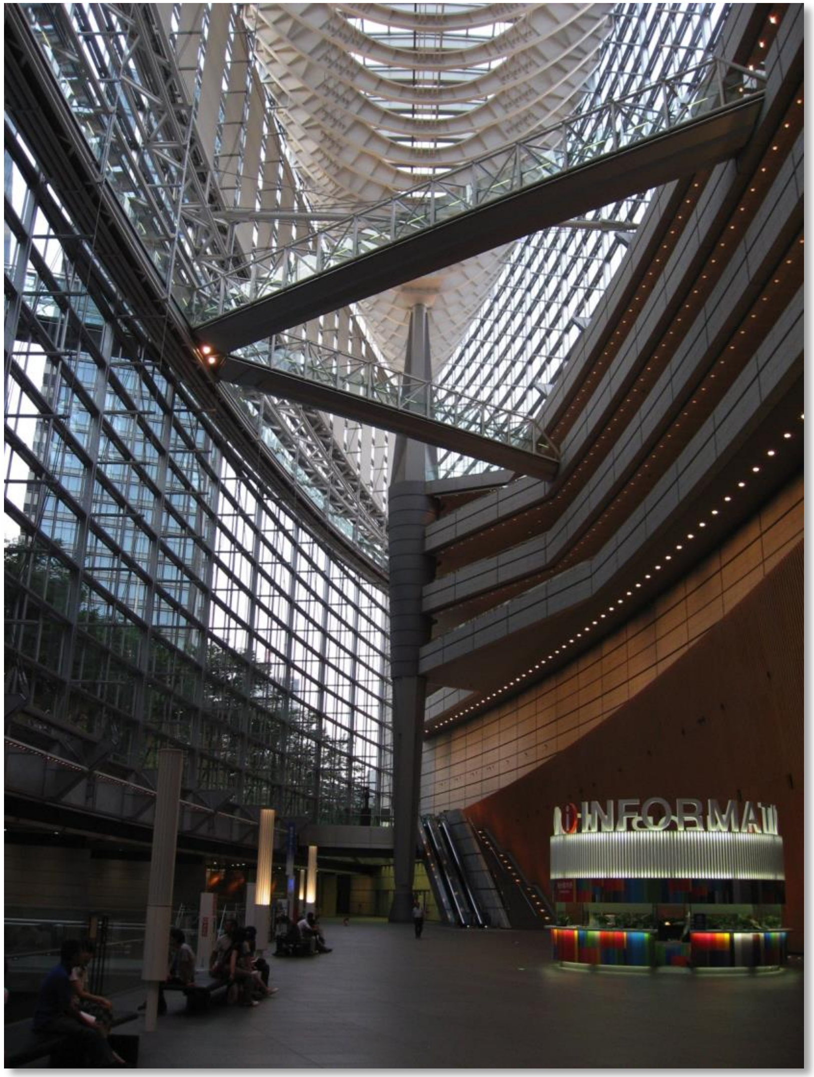
Contents



"Buildings of structural steel have performed excellently and better than any other type of substantial construction in protecting life safety, limiting economic loss, and minimizing business interruption due to earthquake-induced damage"

Yanev, P.I., Gillengerten, J.D., and Hamburger, R.O. (1991). *The Performance of Steel Buildings in Past Earthquakes.* The American Iron and Steel Institute

Terremoti recenti in Italia

City: Amatrice

Locality: -

Structural material: STEEL

Damage: no

Nonstructural material (partition walls): Masonery

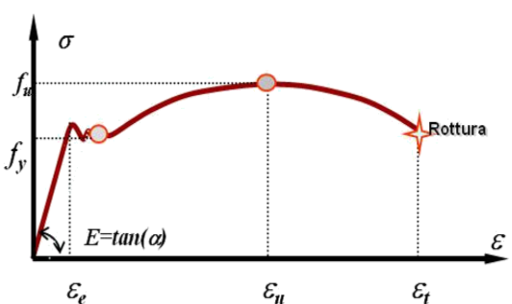
Damage: si (significativi)

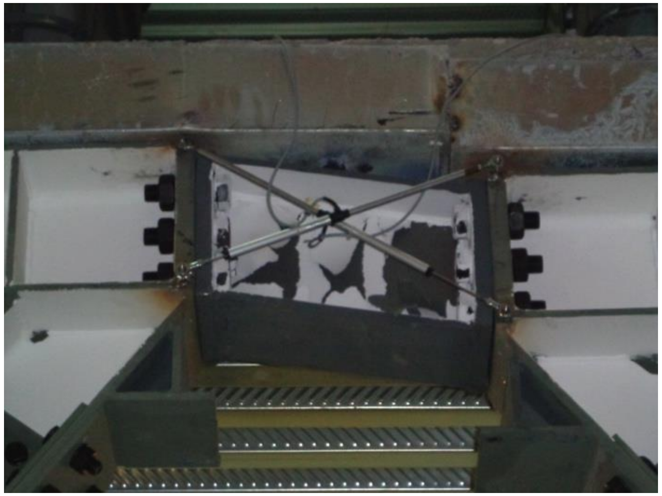
Fonte:
Rapporto fotografico relativo ai danni subiti da alcuni edifici a seguito del sisma del centro Italia del 2016
C. Menna*, R. Frascadore*, C. Moroni, G.P. Lignola*, G. De Martino*, A. Salzano*, M. Di Ludovico*, A. Prota*, G. Manfredi*, E. Cosenza*
*Dipartimento di Strutture per l'Ingegneria e l'Architettura, Università di Napoli Federico II
Disponibile al seguente link http://www.reluis.it.

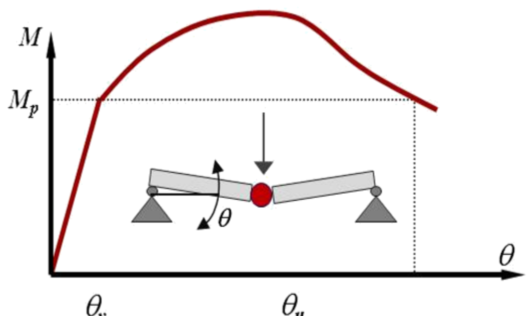
The ductility as design strategy

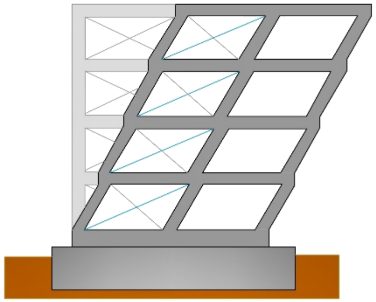
Ductility

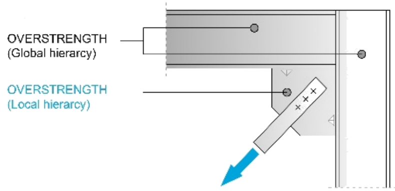
Ductility of material	Local ductility	System ductility

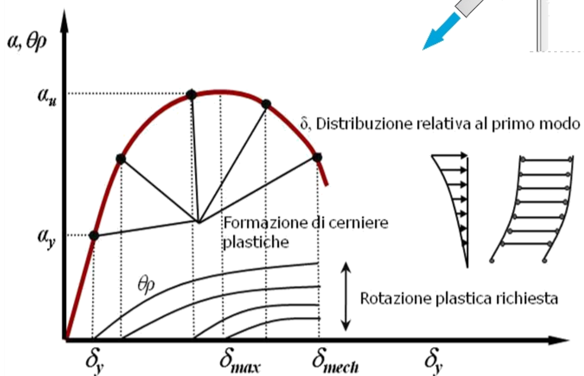






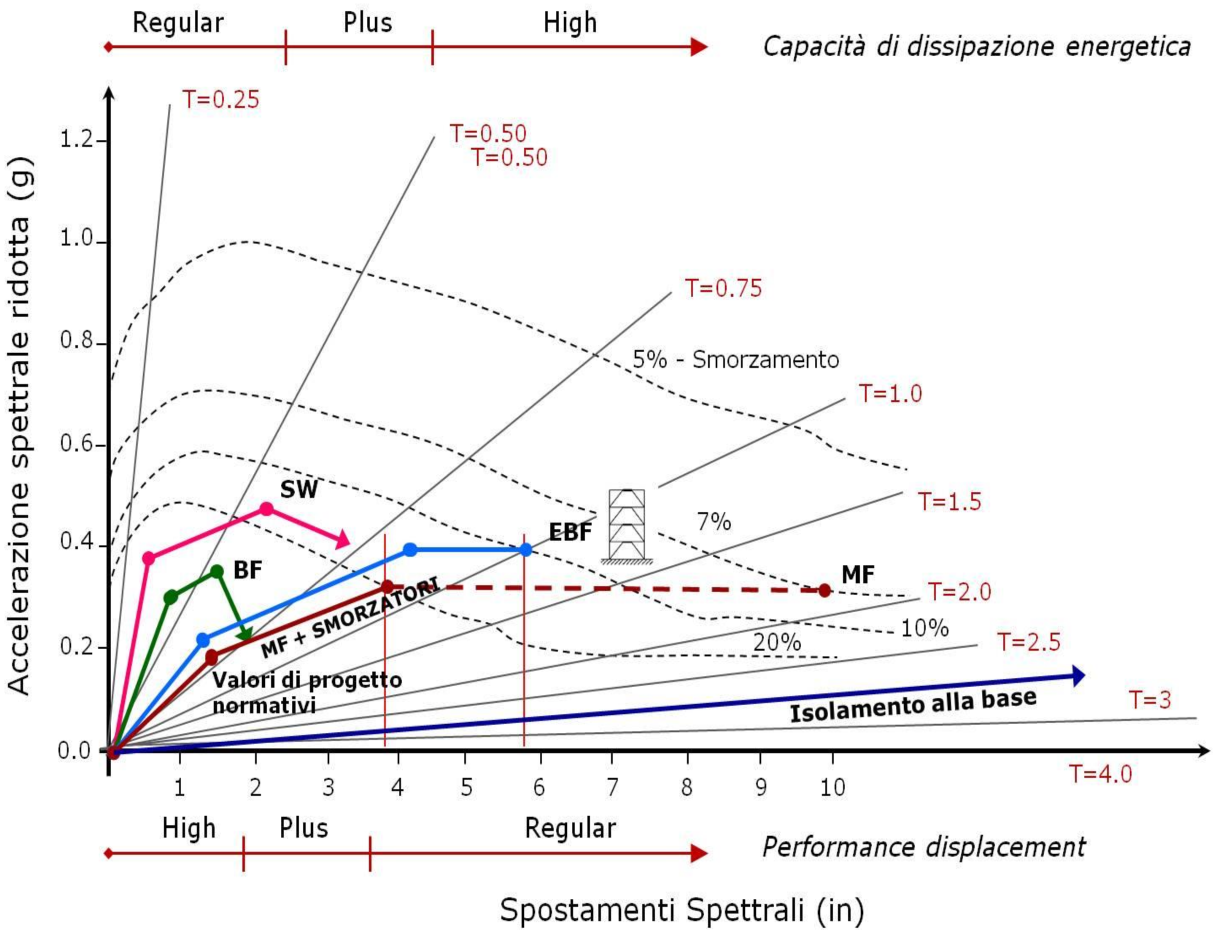






The ductility as design strategy

Ductility



Seismic design of steel structures in seismic areas

Damaged steel buildings in previous earthquake

Need of effective codified seismic design rules

Source:
G Charles Clifton, Gregory A MacRae, Lessons from the Field; Steel Structure Performance in Earthquakes in New Zealand from 2010 to 2016. *Key Engineering Materials 763:61-71*

EUROPEAN CONVENTION FOR CONSTRUCTIONAL STEELWORK

ECCS
CECM
EKS

- The aim of **ECCS is to promote the use of steelwork in the construction sector** by the development of standards and promotional information.
- ECCS is the only European organization which **brings together the Steel Industry, the Fabrication and Contracting specialists, and the Academic world** through an international network of construction representatives, steel producers, and technical centres.

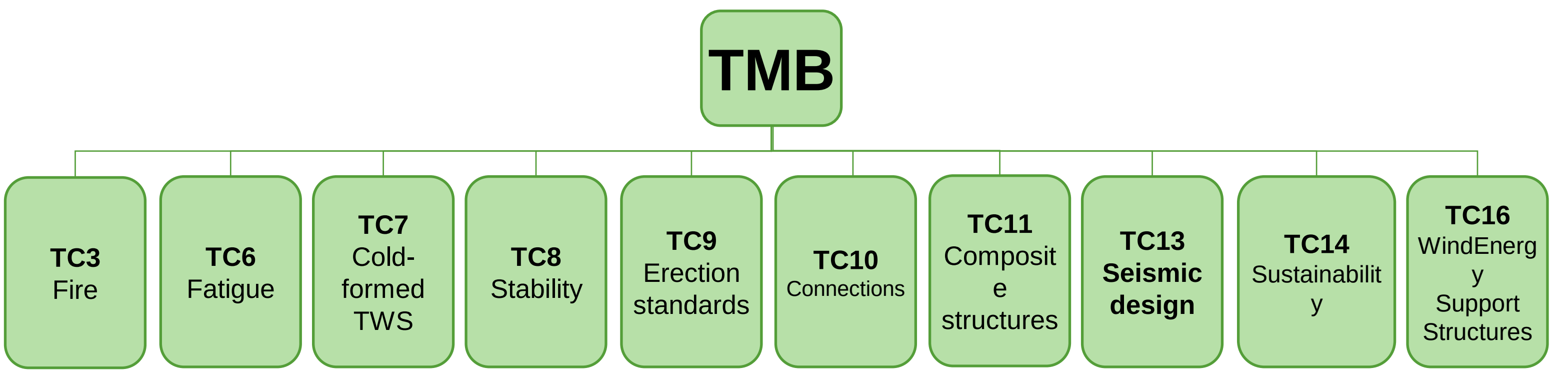
TECHNICAL COMMITTES

- The expert forum that establishes consensus on European practice and provides the undisputed background for normalization;

- The expert forum that identifies ongoing developments in specific fields;

ECCS
CECM
E K S

ECCS Technical Committees comprise over 200 experts

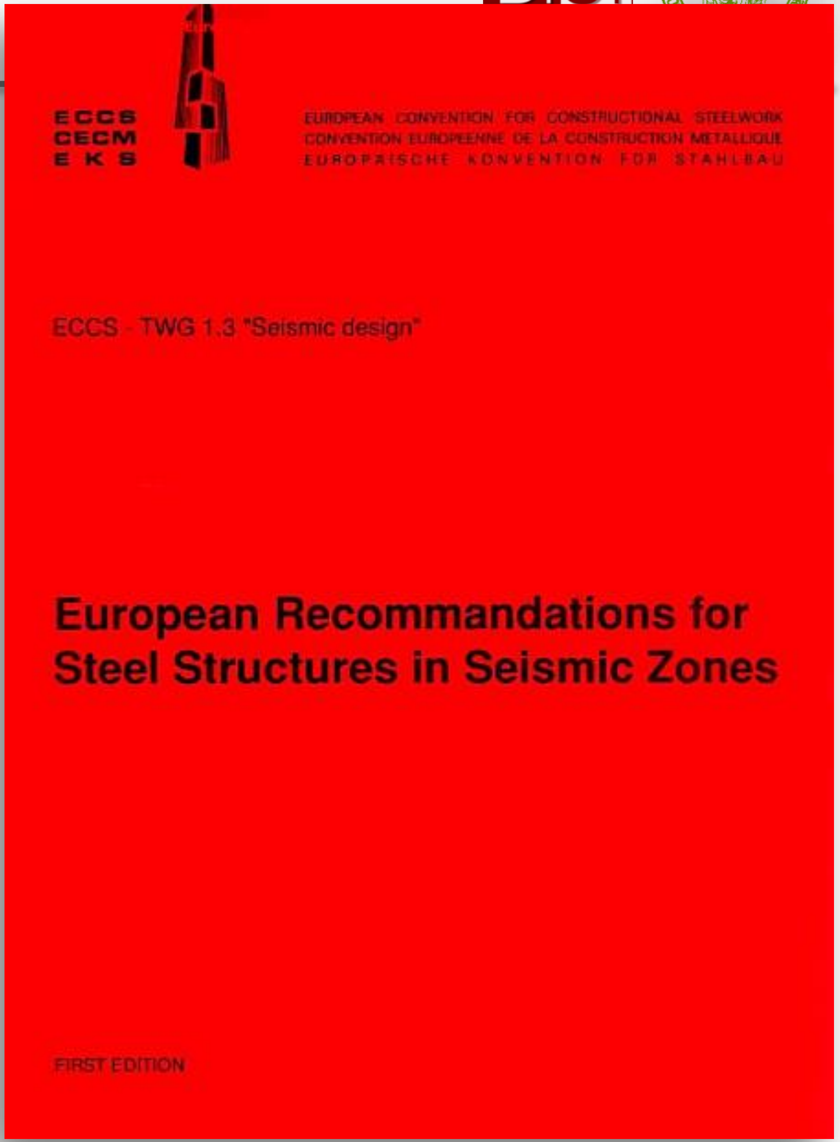


TC13 MISSION

TC13 is devoted to seismic design with the mission to promote the use of steel in seismic region

- This activity started in 1980's
- First EU seismic code:

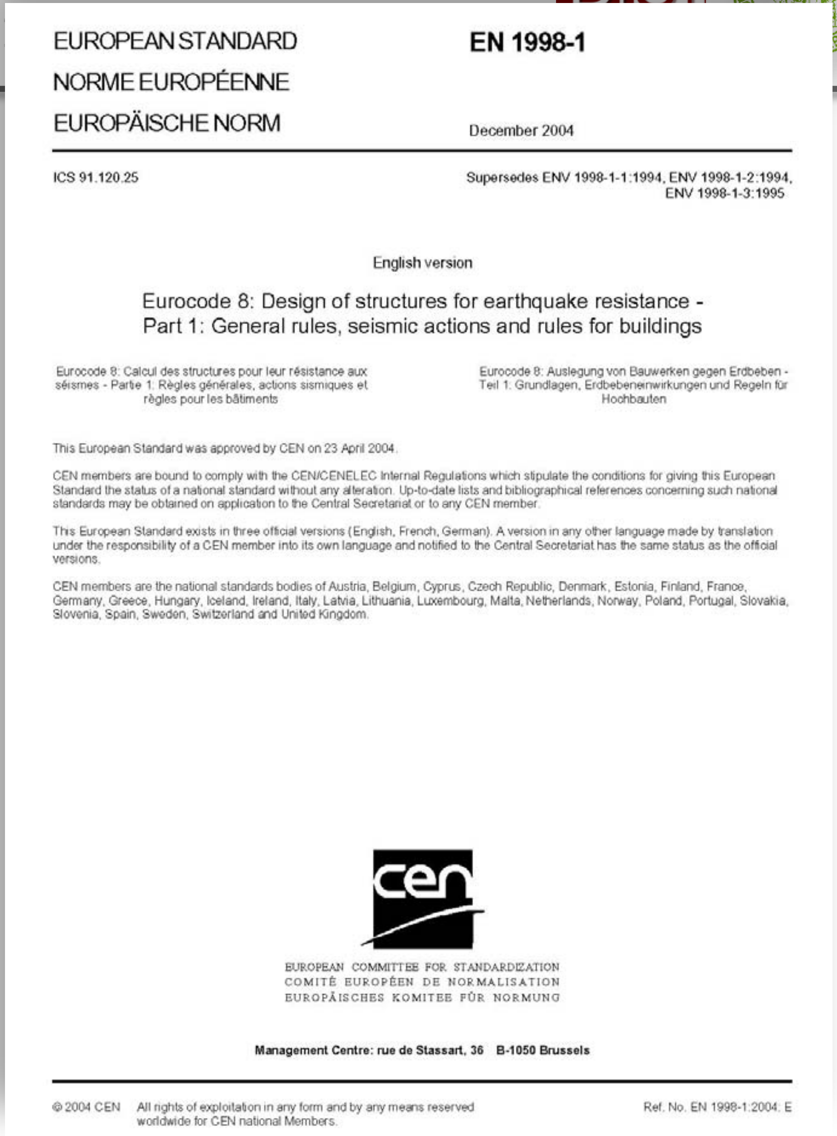
  ECCS code 1991
  European for Recommendations
  for Steel Structures in Seismic Zones



EUROCODES
BUILDING THE FUTURE

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

EN 1998-1 Section 6: Specific rules for steel building

CONTENTS

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

Ongoing revision process of European Standard

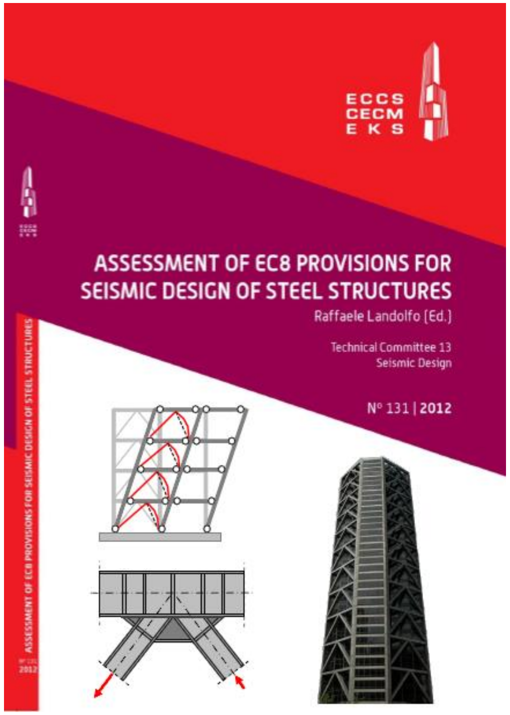
The efforts of TC13

Since **2007** TC13 worked to **improve the rules on seismic design of steel structures.**

In 2013 **"Assessment of EC8 Provisions for Seismic Design of Steel Structures " was published,** containing a critical and systematic review of current EC8 and identifying main criticisms and issues needing revisions and/or upgrading.

CONTENTS:

- *Material overstrength*
- *Selection of steel of toughness*
- *Local ductility*
- *Design rules for connections in dissipative zones*
- *New links in eccentrically braced frames*
- *Behaviour factors*
- *Capacity-design rules*
- *Design of concentrically braced frames*
- *Dual structures*
- *Drift limitations and second-order effects*
- *New structural types*
- *Low-dissipative structures*



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DIST

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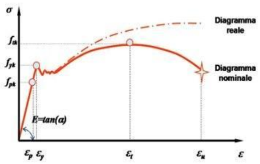
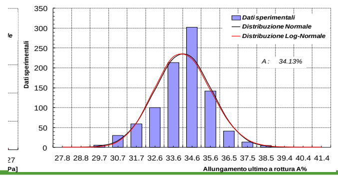
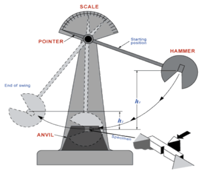
Design concept
Ductility class

Concept a) Low dissipative structural behaviour	DCL (Low)
Concept b) Dissipative structural behaviour	DCM (Medium)
	DCH (High)

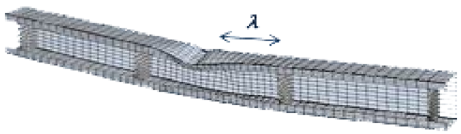
MATERIAL
Material overstrength
Toughness
Safety factors

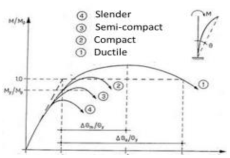




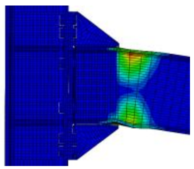


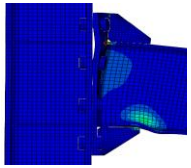
MEMBERS
Local ductility



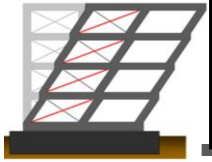


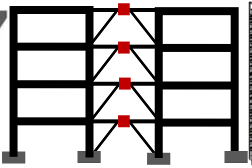
CONNECTIONS
NON-Dissipative B-To-C connections
Dissipative B-To-C connections

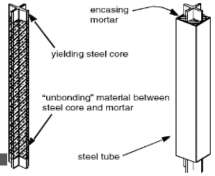




STRUCTURAL SYSTEMS
Traditional systems
Innovative systems and solutions







Under review

GENERAL
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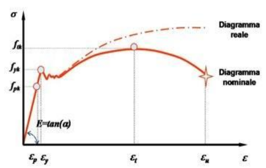
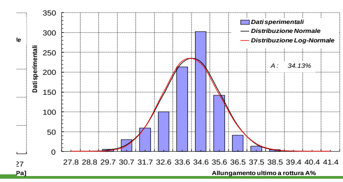
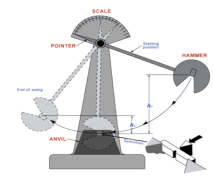
Material overstrength
Toughness
Safety factors







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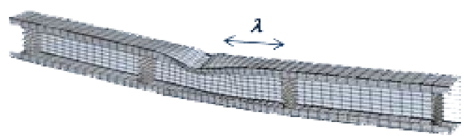

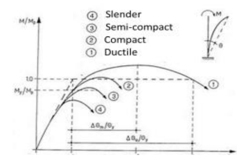

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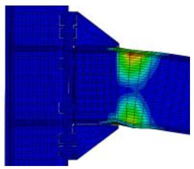

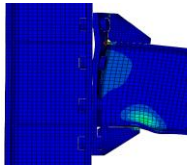

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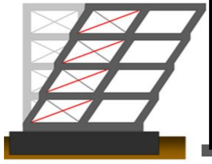
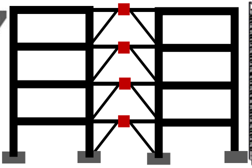
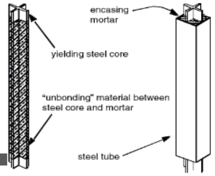




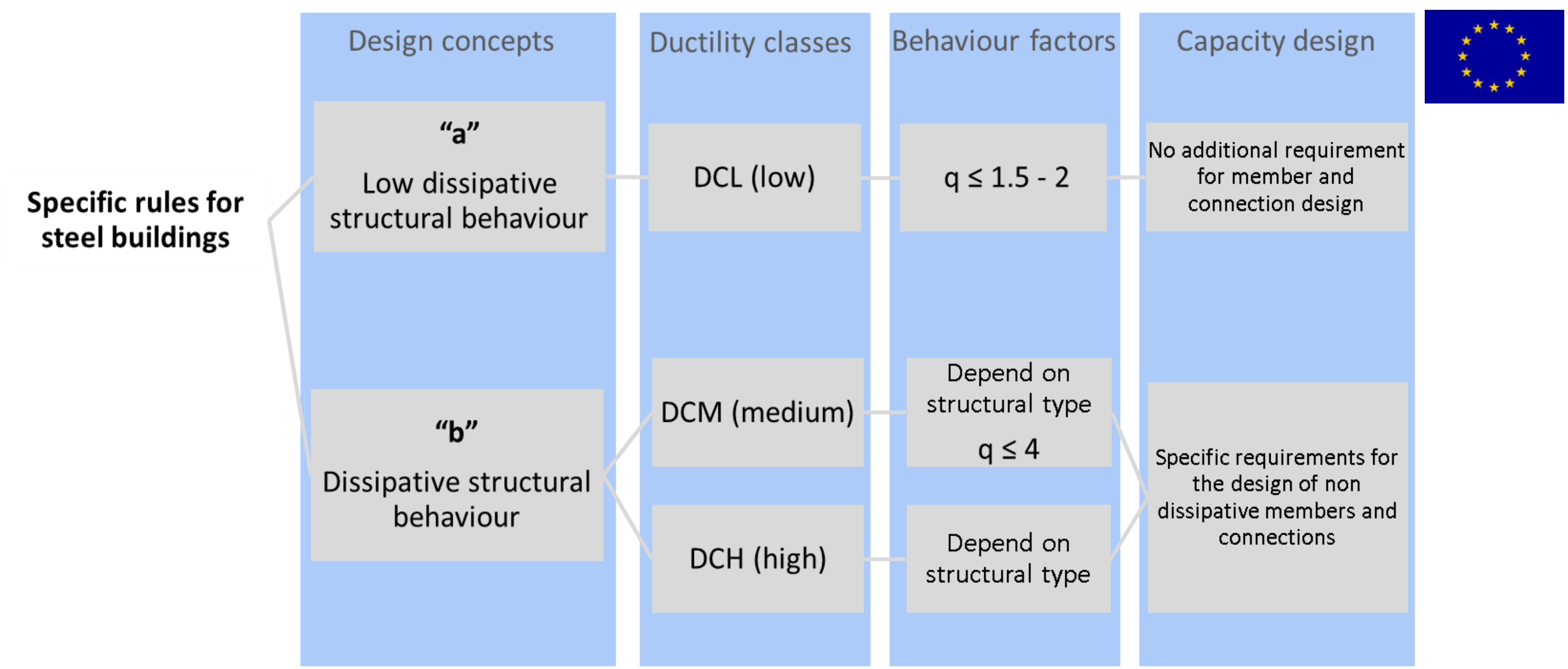


Under review

General: ductility class

CURRENT STAGE in EN 1998-1

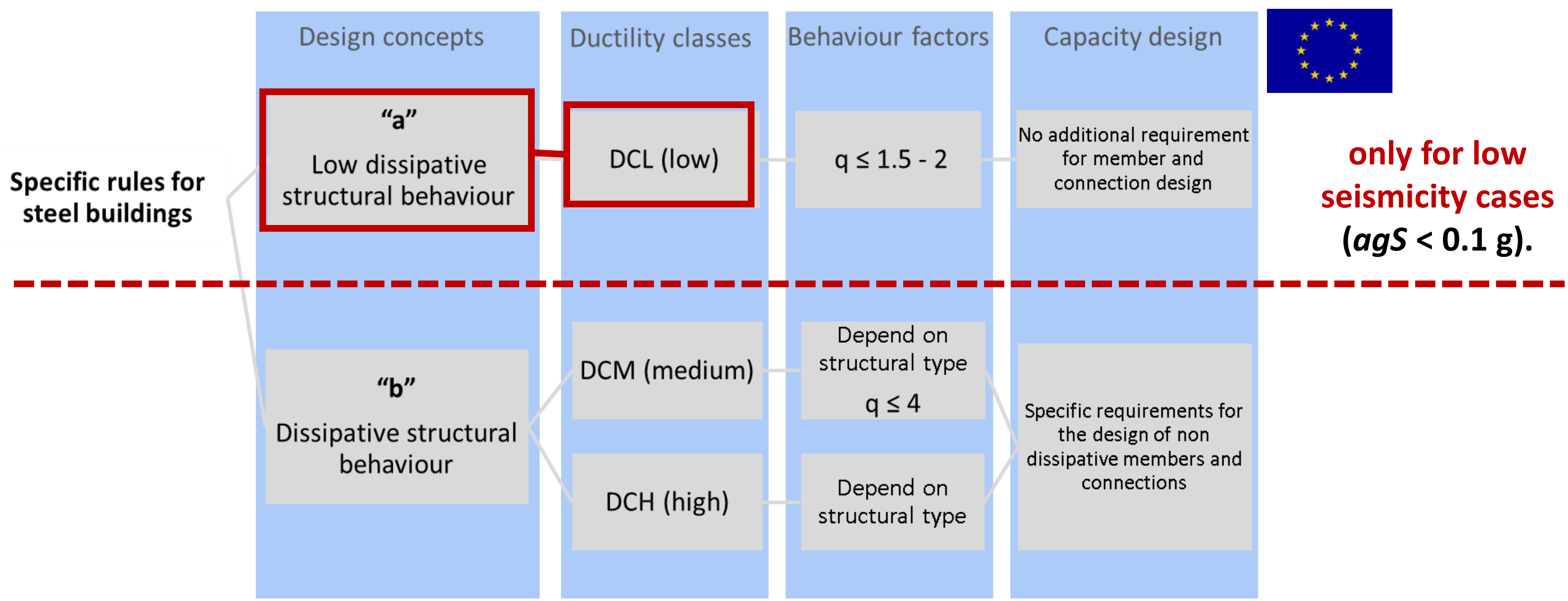
- Earthquake resistant steel buildings shall be designed in accordance with one of the following concepts (see Table 6.1) (CEN, 2005):
  - Concept **a)** Low-dissipative structural behaviour;
  - Concept **b)** Dissipative structural behaviour."



General: ductility class

CURRENT STAGE

- In concept **a)**, the resistance of the members and of the connections should be evaluated in accordance with **EN 1993 without any additional requirements**
- **For** buildings which are not seismically isolated design in accordance with **concept a) is currently recommended only for low seismicity cases (*agS* < 0.1 g).**



General: ductility class

CRITICISMS

- It is not clear from Eurocode 8 itself why DCL design could not be applied in moderate and even in high seismicity regions.

- **Allowing DCL or modified-DCL** detailing in EC8 **for moderate seismicity**, with an appropriate reserve capacity, may be desirable particularly for special or complex structures.

- Although suggesting the use of DCM or DCH for moderate and high seismicity often offers an efficient approach to providing ductility reserve against uncertainties in seismic action, **achieving a similar level of reliability could be envisaged through the provision of appropriate levels of over-strength, coupled with simple inherent ductility provisions where necessary.**

- **Need to develop new rules.**

General: ductility class
NEXT EN 1998:1

		DUCTILITY CLASS	BEHAVIOUR FACTOR	DESIGN RULES	
Current EN-1998	DCL	*Ductility Class Low*	*q ≤ 1.5-2*	***Elastic design*** *applies*	Same rules
	DCM	*Ductility Class Medium*	*q ≤ 4*	***Capacity design*** *applies with* ***identical*** *rules for both DCM and DCH*	
	DCH	*Ductility Class High*	*q > 4*		
Next EN-1998	DC1	*Negligible plastic engagement*	$q \leq 1.5$	***Elastic design*** *applies*	Different rules
	DC2	*Low dissipative engagement*	$q \leq 2.5$	***Elastic design*** *applies with* ***local ductility requirements***	
	DC3	*Medium-Large plastic engagement*	*q>2.5*	***Capacity design*** *applies*	

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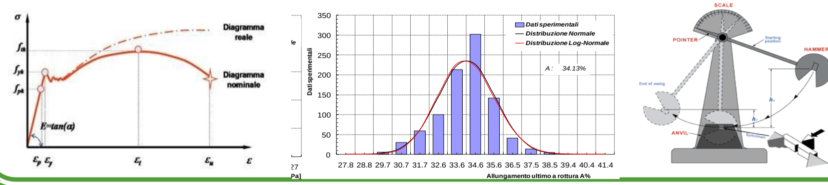
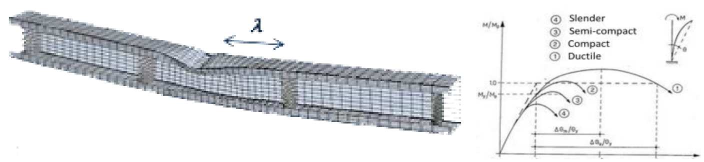
Material overstrength
Toughness
Safety factors



MEMBERS
Local ductility



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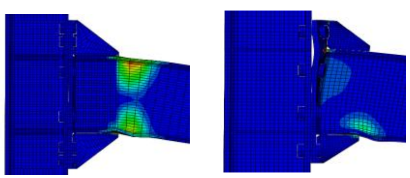

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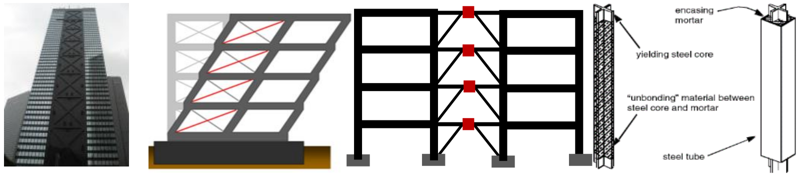
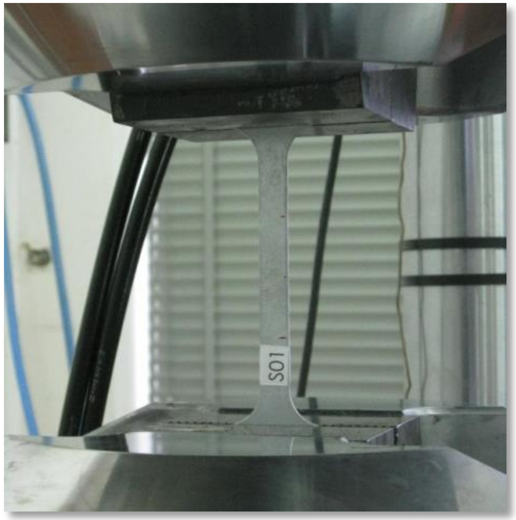


Under review

Material properties: overstrength

CURRENT STAGE



EN 1998-1 (CEN, 2005) states that "the distribution of material properties, such as yield strength and toughness, in the structure shall be such that dissipative zones form where they are intended to in the design".

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

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

f_y : Nominal yield strength specified for the steel grade

γ_{ov} : Overstrenght factor

$$\gamma_{ov} = \frac{f_{y,m}}{f_{y,k}}$$

$f_{y,m}$ → AVERAGE YIELD STRESS

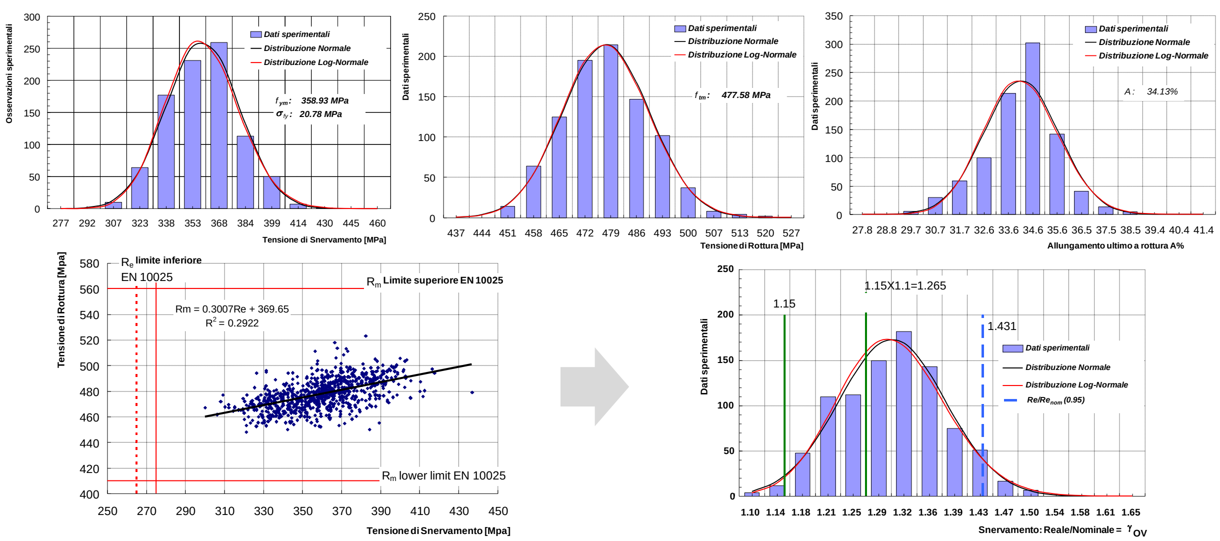
$f_{y,k}$ → CHARACTERISTIC YIELD STRESS

→ **AT THE CURRENT STAGE EC8 SUGGEST A CONSTANT VALUE γ_{ov} = 1.25**

Material properties: overstrength

CRITICISMS

- Experimental data suggest the actual steel strength depends on the steel grade and the type of steel product
- Also in AISC (2005), the overstrength factors are obtained in function of both steel grade and the type of steel product



(1st year annual report RFCS OPUS project-RFSR-CT-2007-00039)

Material properties: overstrength

NEXT EN 1998-1

Current EN-1998

γ_{ov} is the material overstrength factor used in design

NPD-Recommended Value 1,25

Next EN-1998

γ_{ov} is the ratio between the expected (i.e. average) yield strength $f_{y,average}$ and the relevant f_y.
This ratio is the material overstrength factor used in design, which depends on the steel grade

Steel grade	overstrength factor γ_{ov}
S235	1.45
S275	1.35
S355	1.25
S420	1.2
S460	1.1

These values are obtained by cross checking the findings obtained in OPUS and SAFEBRICTLE researches

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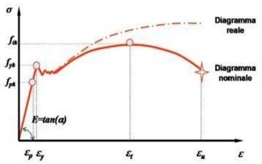
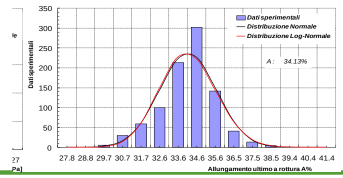
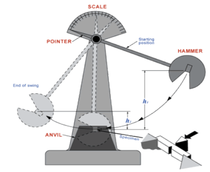
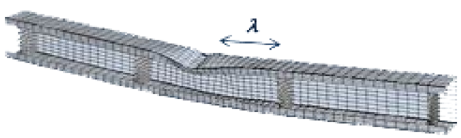
Toughness
Safety factors

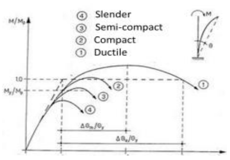






MEMBERS
Local ductility





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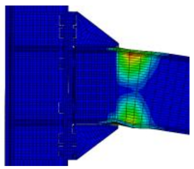
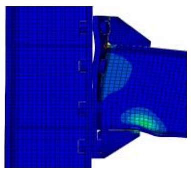
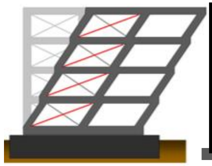
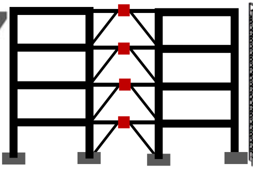
Dissipative B-To-C connections

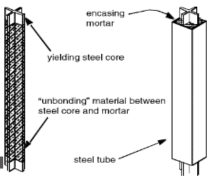




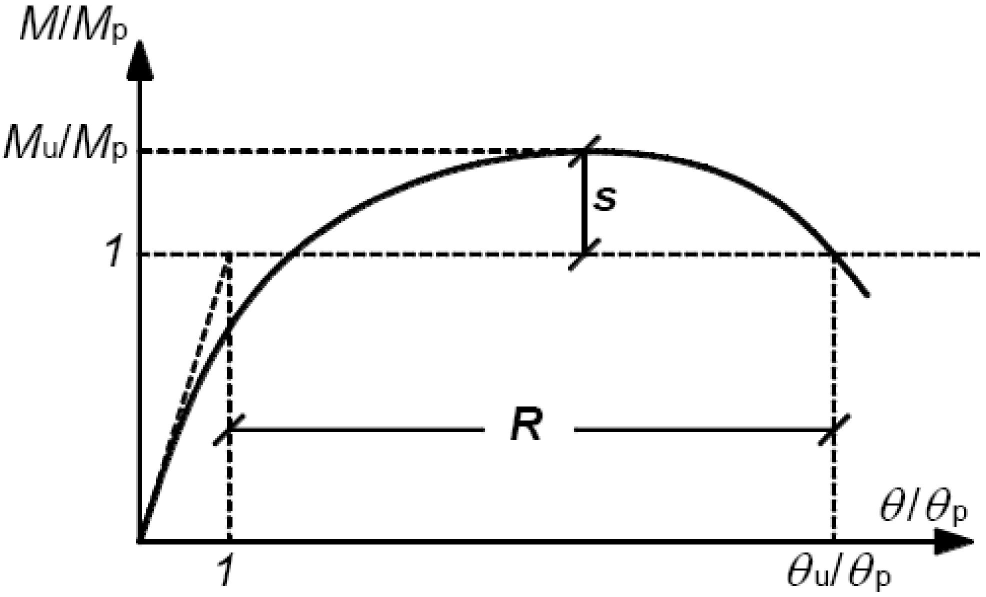
STRUCTURAL SYSTEMS
Traditional systems
Innovative systems and solutions







Under review


M/Mp
Mu/Mp
1
s
R
θ/θp
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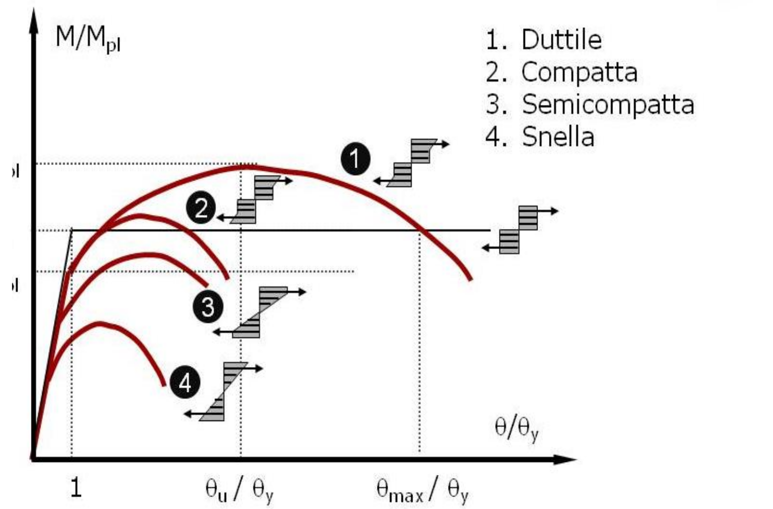
θu/θp

LOCAL DUCTILITY
CURRENT RULES



LOCAL DUCTILITY REQUIREMENTS → All intended plastic zones must fully develop

Local ductility of steel members in compression and bending is ensured in EN 1998-1 (2004) by restricting the width-thickness ratio b/t according to the cross-sectional classes specified in EN 1993-1-1 (2004).

Ductility class	Reference value of behaviour factor (q)	Required cross-sectional class
DCM	1,5 < q ≤ 2	class 1, 2 or 3
	2 < q ≤ 4	class 1 or 2
DCH	q > 4	class 1

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

LOCAL DUCTILITY

CRITICISMS

Rotation Capacity

- Ductile or plastic sections: $M_{max} > M_{pl}$
- Compact: $M_{max} > M_{pl}$

$R = ?$

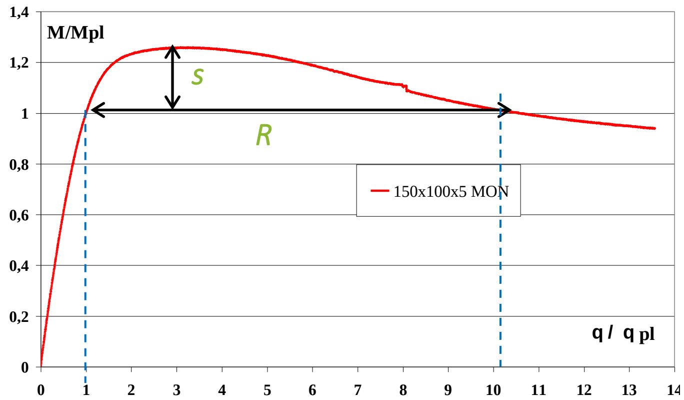
The minimum values are not clearly defined

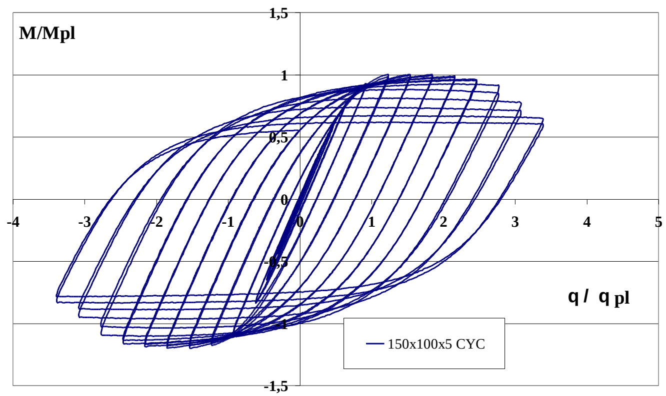
- Semi- compact: $M_y < M_{max} < M_{pl}$
- Slender: $M_{max} < M_y$

Flexural Overstrength:

S = 1.1 → **The overstrength factor is assumed constant ! ? !**

The relationship between R and s is not provided





Profilo 150x100x5

LOCAL DUCTILITY

POTENTIAL UPGRADE

In order to overcome the EC3 limits, a recent studies proposed novel empirical expressions for R and s under monotonic loading.

These expressions are defined on the bases of multiple regression of the experimental results obtained by monotonic tests

New empirical formulation for s

Expression of S is defined on the bases of multiple regression of the experimental results obtained by monotonic tests

$$\frac{1}{s} = C_1 + C_2\lambda_f^2 + C_3\lambda_w^2 + C_4\frac{b_f}{L} + C_5\frac{E}{E_h} + C_6\frac{\varepsilon_h}{\varepsilon_y}$$

	C_1	C_2	C_3	C_4	C_5	C_6
IPE - HE	1.709	0.167	0.005	-0.134	-0.007	-0.052
RHS - SHS	0.963	0.598	0.023	-1.112	0.012	-0.067

- λ_f the flange slenderness
- λ_w the web slenderness
- b_f the flange width, h the beam depth
- L the shear length factors

Steel	E/E_h	$\varepsilon_h/\varepsilon_y$
S 235	37.5	12.3
S 275	42.8	11.0
S 355	48.2	9.8

D'Aniello M., Landolfo R., Piluso V., Rizzano G. (2012). Ultimate Behaviour of Steel Beams under Non-Uniform Bending. *Journal of Constructional Steel Research* 78 (2012) 144–158.

LOCAL DUCTILITY

POTENTIAL UPGRADE

New empirical formulation for R

Expression of R is defined on the bases of multiple regression of the experimental results obtained by monotonic tests

$$R = C_1 + C_2 \frac{1}{\lambda_f^2} + C_3 \frac{1}{\lambda_w^2} + C_4 \frac{b_f h}{L^2} + C_5 \frac{b_f t_f}{hL} + C_6 \frac{A_f}{A_{TOT}} + C_7 \frac{L_m}{L} + C_8 s$$

	C_1	C_2	C_3	C_4	C_5	C_6	C_7	C_8
IPE - HE	-57.471	0.152	3.156	-73.360	-1454.240	40.210	328.986	-13.070
RHS - SHS	-18.425	-0.172	1.357	-120.527	1618.593	25.842	10.571	-31.800

- λ_f the flange slenderness
- λ_w the web slenderness
- b_f the flange width
- h the beam depth
- t_f the flange thickness
- L the shear length factors
- A_f area of the flange
- A_{TOT} the total cross section area
- L_m the length of the plastic hinge

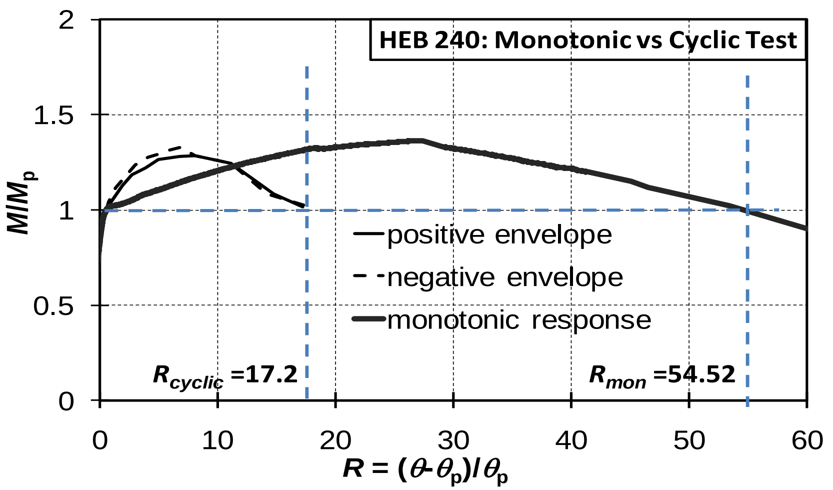
$$L_m = b_f \cdot 0.6 \cdot \left(\frac{t_f}{t_w}\right)^{\frac{3}{4}} \cdot \left(\frac{d}{0.5 \cdot b_f}\right)^{\frac{1}{4}}$$

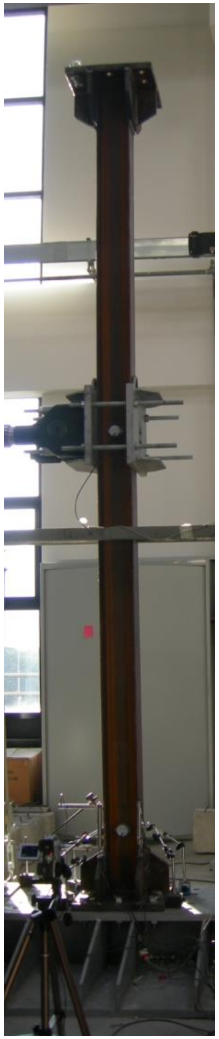
LOCAL DUCTILITY

CRITICISMS

- The EC8 adopts the classification criterion proposed **by the EC3 (NON SEISMIC CODE)** that is based on ductility criterion on the basis of the available rotation capacity (R): This classification is based on **monotonic response** of steel beams
- Seismic design require members rotation capacity (R) and overstrength (s) to be **properly evaluated, accounting for the cyclic behaviour induced by earthquake action**

Motononic vs. Cyclic behaviour: Rotation capacity (*R*)

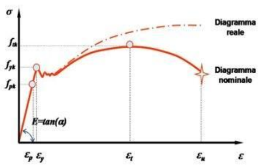


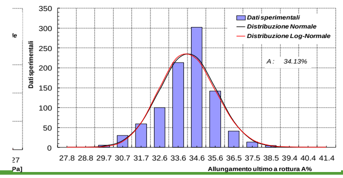


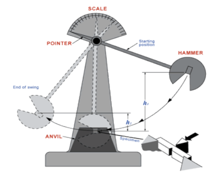
GENERAL
Design concept
Ductility class

Concept a) Low dissipative structural behaviour	DCL (Low)
Concept b) Dissipative structural behaviour	DCM (Medium)
	DCH (High)

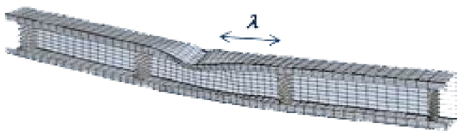
MATERIAL
Material overstrength
Toughness
Safety factors

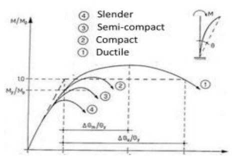




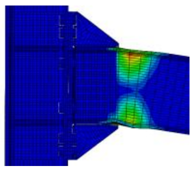


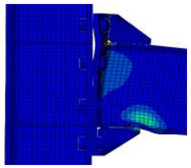
MEMBERS
Local ductility



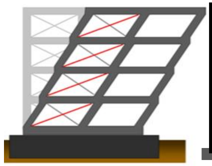


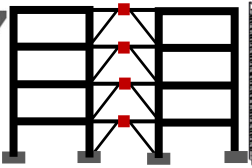
CONNECTIONS
NON-Dissipative B-To-C connections
Dissipative B-To-C connections

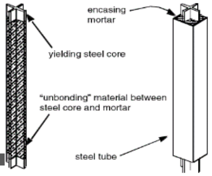




STRUCTURAL SYSTEMS
Traditional systems
Innovative systems and solutions



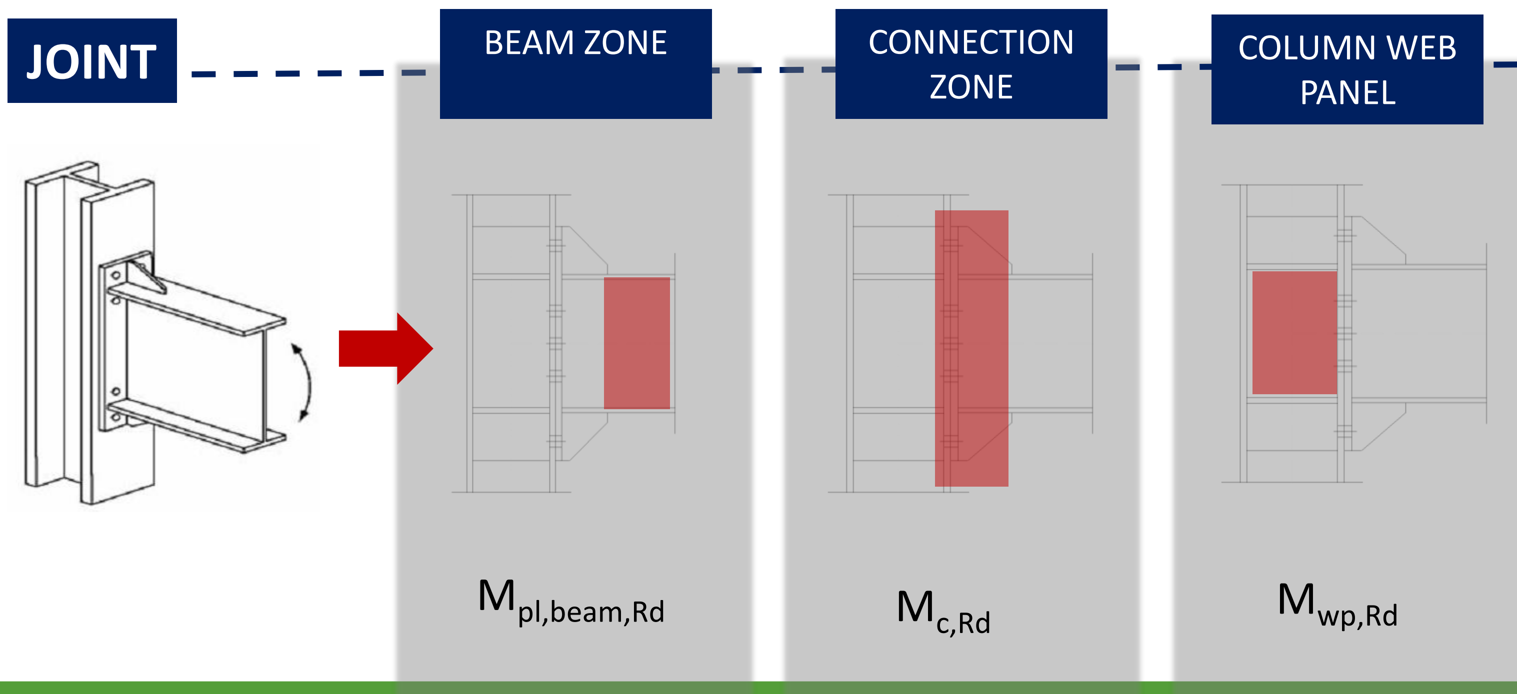




Under review

- The joint is conceived as made by **three macro-components**

- Each macro-component is individually designed according to specific assumptions and then **macro-component** are applied, in order to obtain three different **design objectives**

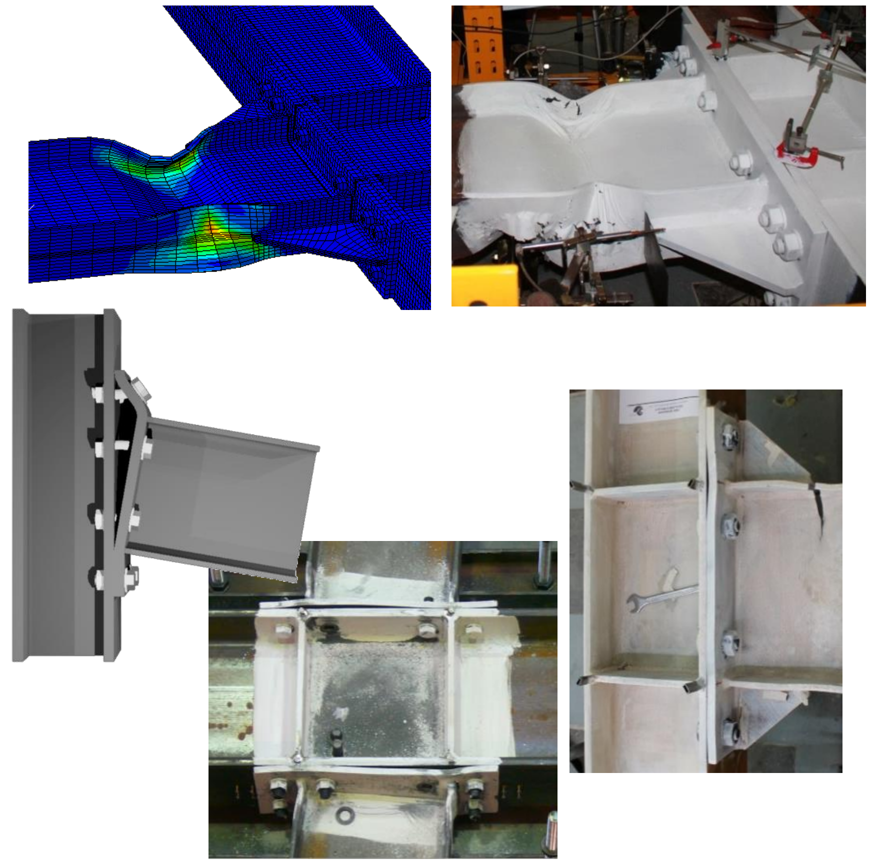


Design of steel beam-to-column connections

Design of beam-to-column connections accordin to EN 1998-1

SEISMIC DESIGN OF CONNECTIONS

- NON-DISSIPATIVE CONNECTION
- DISSIPATIVE CONNECTION



CONNECTIONS

NON-DISSIPATIVE JOINTS: current rules

Overstrength criterion for non dissipative connection of dissipative members made by means of full penetration butt welds

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

- R_d is the resistance of connection according to EN-1993
- R_{fy} is the plastic resistance of the connected dissipavie member based on the yield stress of the material as defined in EN-1993
- γ_{ov} is the material overstrength factor

Courtesy of Piluso

CONNECTIONS

NON-DISSIPATIVE JOINTS: CRITICISMS

Overstrength criterion for non dissipative connection of dissipative members made by means of full penetration butt welds

The hardening factor is assumed constant

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

It does not consider the posistion of plastic hinge (which can be far from the connection)


Courtesy of Piluso

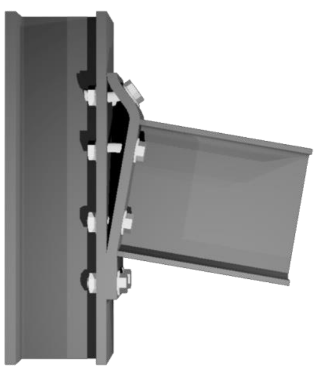
- R_d is the resistance of connection according to EN-1993
- R_{fy} is the plastic resistance of the connected dissipavie member based on the yield stress of the material as defined in EN-1993
- γ_{ov} is the material overstrength factor

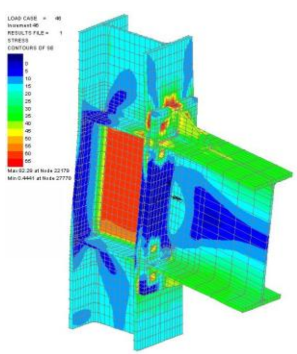
CONNECTIONS

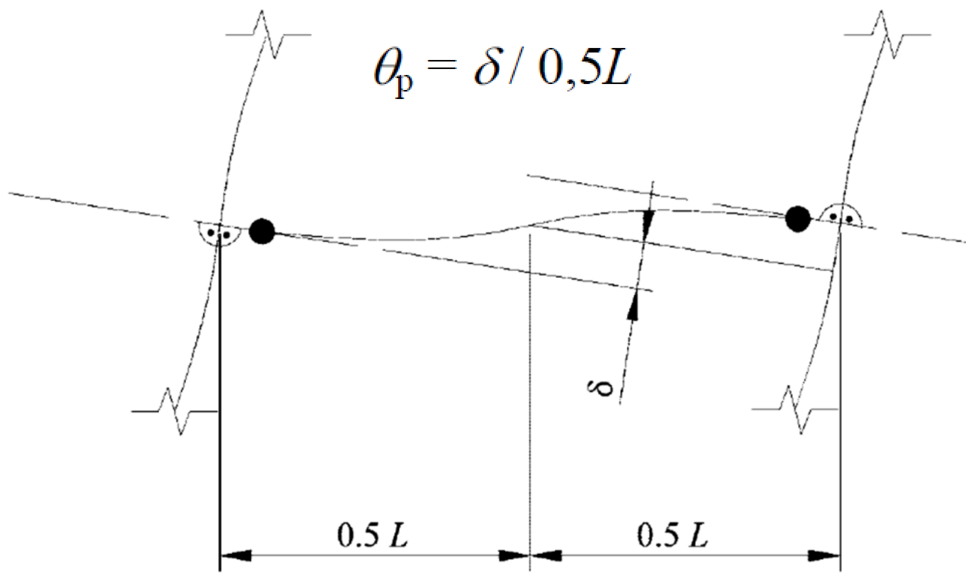
DISSIPATIVE JOINTS: current rules

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 with q >2
- Design is supported by specific experimental testing, resulting in impractical solutions within the typical time and budget constraints of r

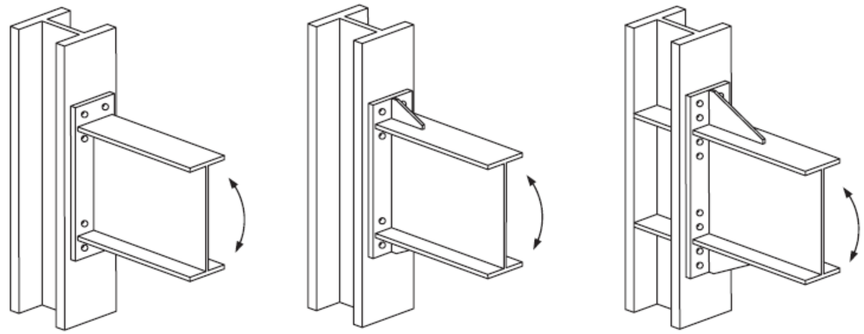






Prequalification of beam-to-column connections: *potential upgrade*

- In US and Japan this issue has been solved adopting **pre-qualified standard joints**.
- Within the FEMA/SAC program (1995), devoted to develop and evaluate guidelines for the inspection, evaluation, repair, rehabilitation, and construction of steel moment frame resisting structures.
- The US research effort was directed to feed into a specific standard (ANSI/AISC 358-05, 2005) **containing design, detailing, fabrication and quality criteria for a set of selected types of connections** including the most common used in US practice, that are prequalified for use in Special and Intermediate MRFs



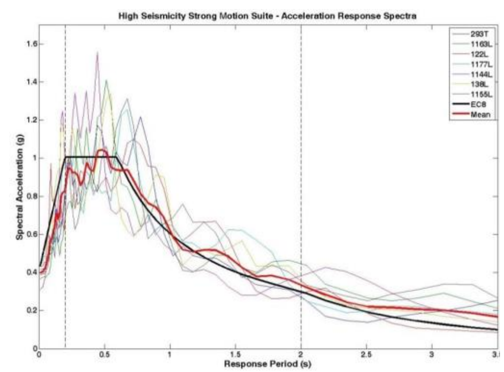
(a) (b) (c)

Fig. 6.1. Extended end-plate configurations: (a) four-bolt unstiffened, 4E; (b) four-bolt stiffened, 4ES; (c) eight-bolt stiffened, 8ES.

Limit of application of US prequalification procedure for EU practice

The type of seismic input, which affect the ductility demand on joints and connected members, differs between the different counties.

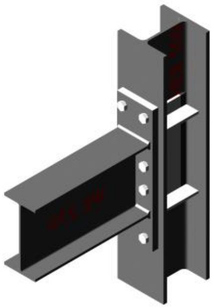
Seismic input



What about Europe?

In order to fill these gaps, the recently finished European research project "Equaljoints" was aimed at providing prequalification criteria of steel joints for the next version of EN 1998-1.

Research Fund for Coal & Steel



CONNECTIONS

DISSIPATIVE JOINTS: Potential upgrade

Research Fund for Coal & Steel

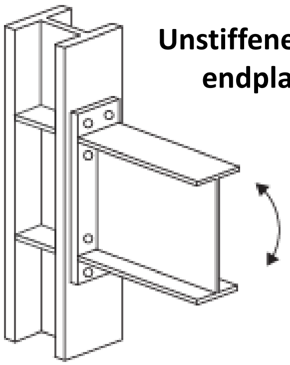
EQUALJOINTS PROJECT

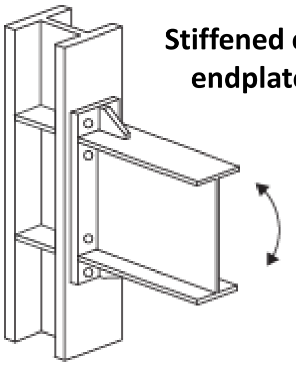
PARTNERS

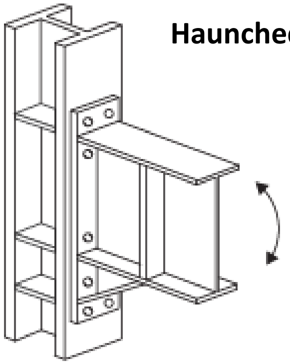
- Leader: **University of Naples Federico II** - CO1 -Italy
- Arcelormittal Belval & Differdange SA- BEN2 - Luxembourg
- Universite de Liege- BEN3 – Belgium
- Universitatea Politehnica din Timisoara BEN4 – Romania
- Imperial College of Science, Technology and Medicine- BEN5 - UK
- Universidade de Coimbra- BEN6 - Portugal
- European Convention for Constructional Steelwork Vereniging-BEN7 - Belgium
- CORDIOLI & C. S.P.A. – BEN8 – Italy

Prequalified joint typologies

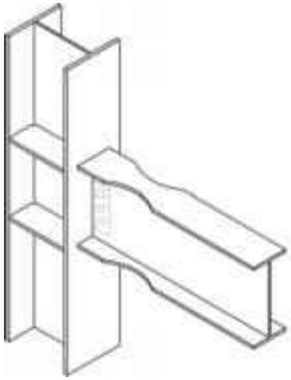
- Three bolted joint types:

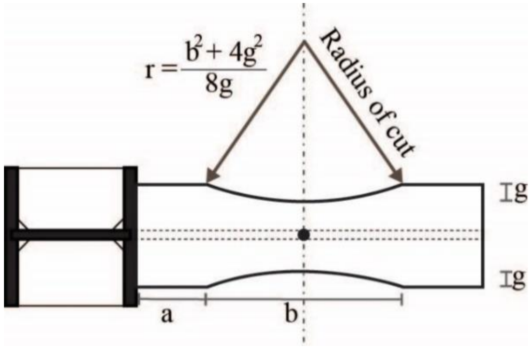






- And welded **dog-bone** joints:

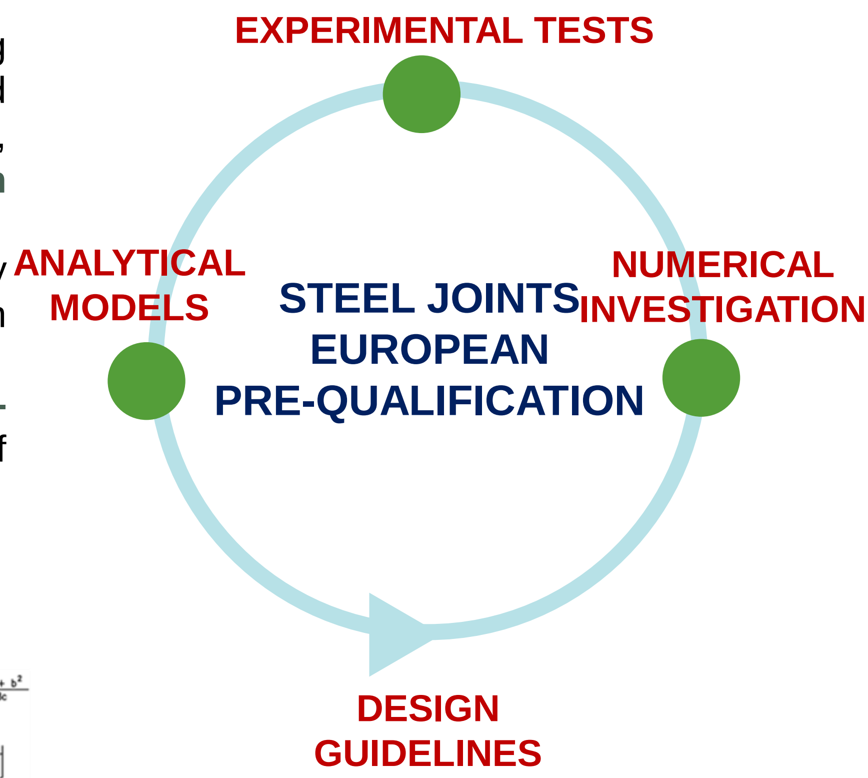




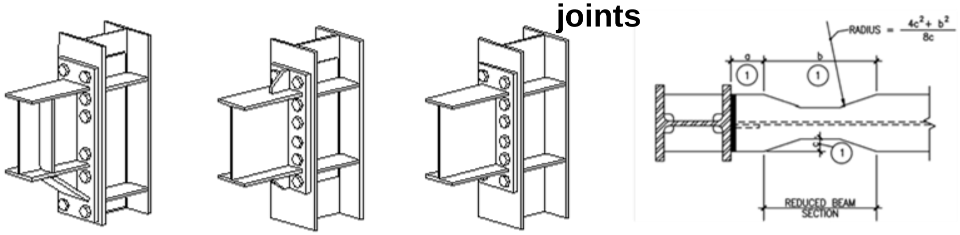
CONNECTIONS

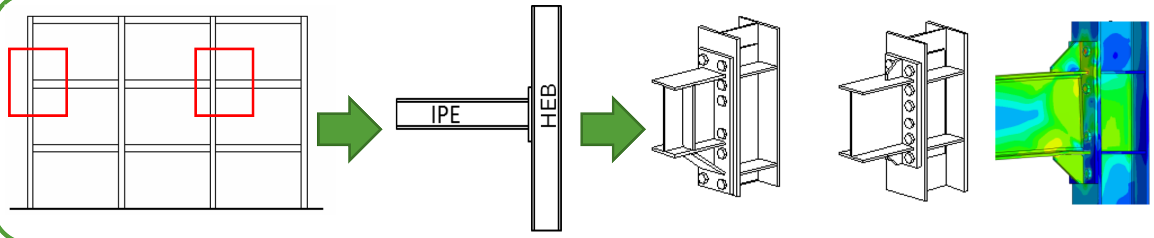
DISSIPATIVE JOINTS: Potential upgrade

- The **EQUALJOINTS** research project aims at providing **pre-qualification procedure** for a set of selected seismic resistant steel beam-to-column joints, introducing a **codified practice currently missing in Europe.**
- A large **experimental** programme supported by **theoretical** and **numerical** analyses has been proposed.
- The prequalification criteria will refer to both **full-strength** and **partial-strength** joints for three types of bolted configurations and one welded dog-bone joint.

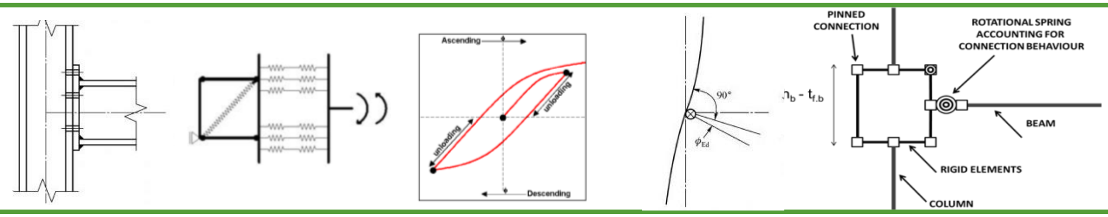


Haunched joints | **Extended Stiffened Endplate joints** | **Extended Unstiffened Endplate joints** | **Dogbone (welded)**

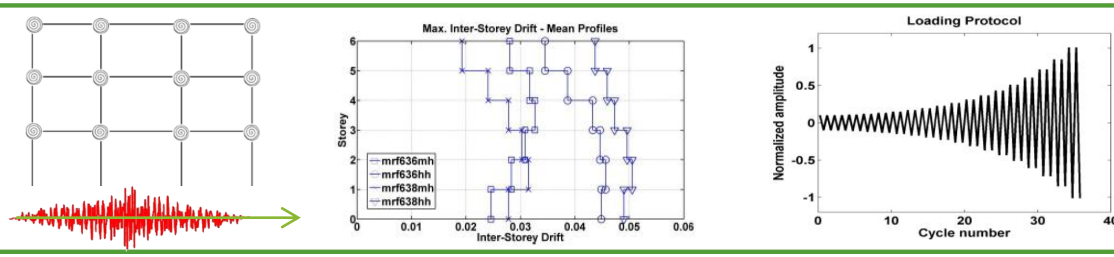




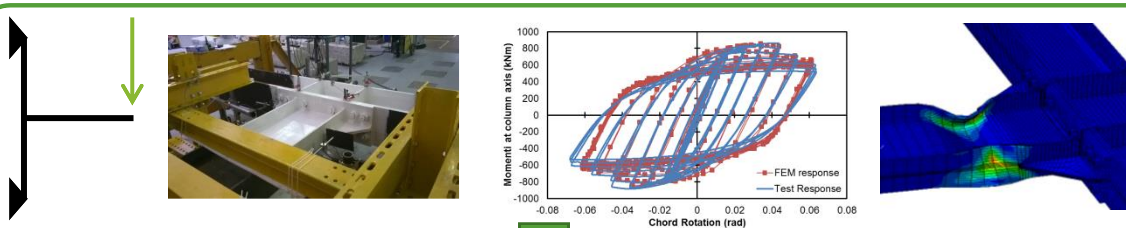
DESIGN OF REFERENCE STRUCTURES
SELECTION OF BEAM-TO-COLUMN ASSEMBLIES
DESIGN OF JOINTS



ANALYTICAL MODEL
DEFINITION OF HYSTERETIC JOINT BEHAVIOUR
CHARACTERIZATION OF ROTATIONAL SPRINGS



EVALUATION OF SEISMIC DEMAND ON JOINTS
DEFINITION OF NEW EU LOADING PROTOCOL

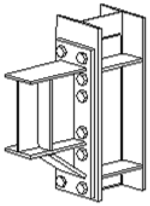


EXPERIMENTAL PROGRAMME
CALIBRATION OF FEM MODELS
FEM PARAMETRIC ANALYSES

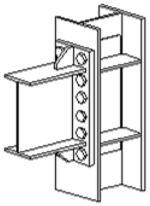
DESIGN GUIDELINES AND PREQUALIFICATION CHARTS

CONNECTIONS
Equaljoints experimental program

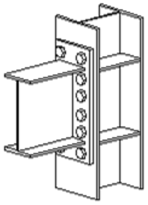
- **TESTS ON BASE MATERIAL**
- **CHARACTERIZATION OF BOLTS**
- **CYCLIC CHARACTERIZATION OF MILD CARBON STEEL**
- **76 JOINT SPECIMENS**



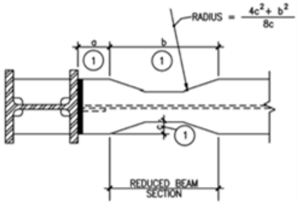
24 Tests



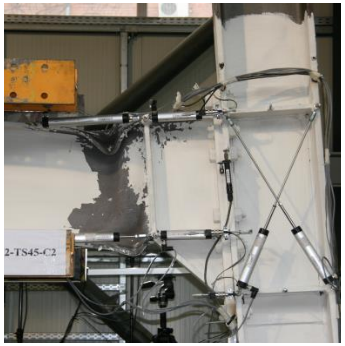
25 Tests

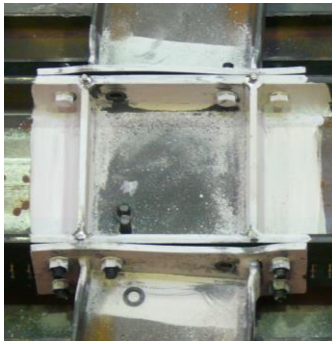


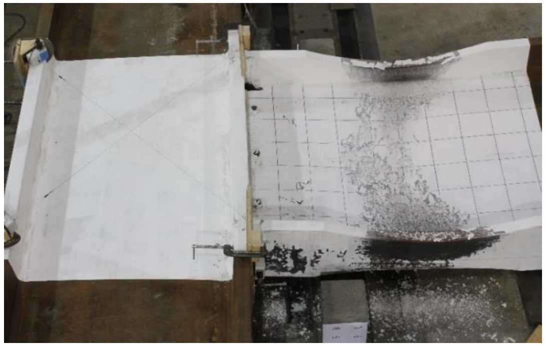
24 Tests



2 Tests







<u>Acceptance Criteria</u>:

EN1998-1:

Plastic rotation $\theta_p \geq 35$ mrad for DCH structures

Plastic rotation $\theta_p \geq 25$ mrad for DCM structures designed with q > 2

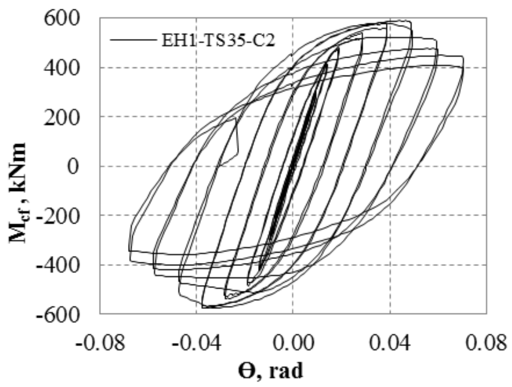
AISC 341-10:

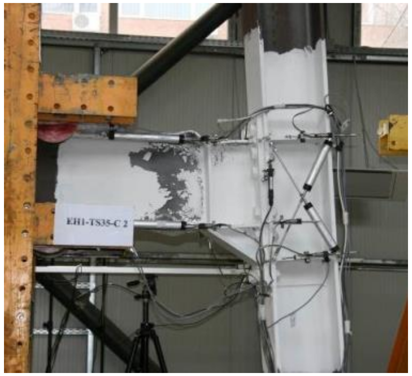
Residual bending strength larger or equal to 80% the plastic resistance of the beam M_p for chord rotation equal to 0.04 rad.

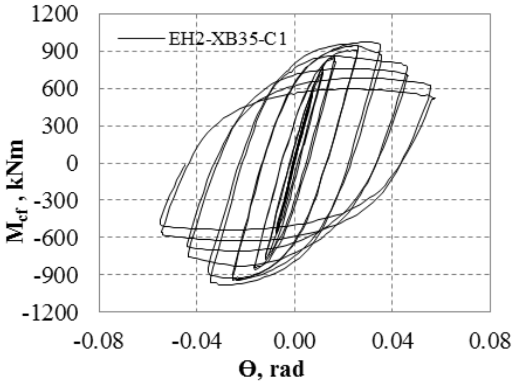
CONNECTIONS

Equaljoints experimental program

HAUNCHED JOINTS – 24 tests carried out Universitatea Politehnica di Timisoara







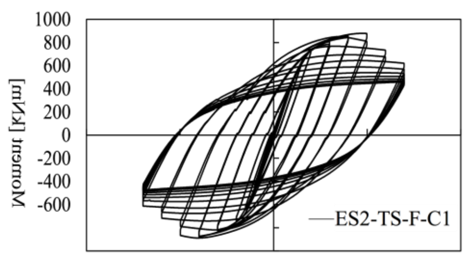
- 2 nominally identical tests per joint were carried out.
- The performance of the joints complies with both EC8 and AISC341

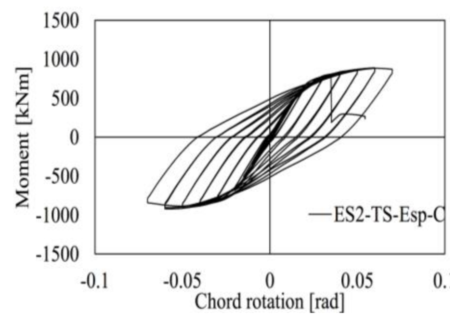
CONNECTIONS

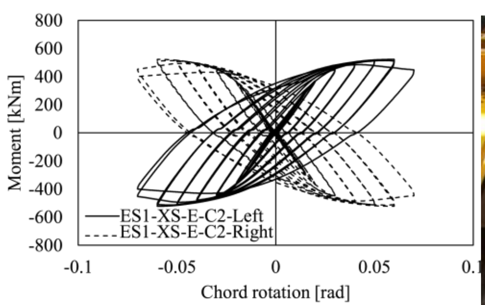
Equaljoints experimental program

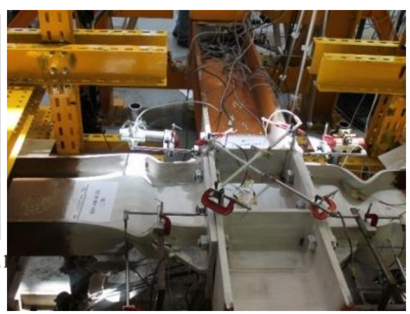
EXTENDED STIFFENED JOINTS – 17 tests carried out at Univ. of Napoli Federico II + 9 tests carried out at Univ. of Liege

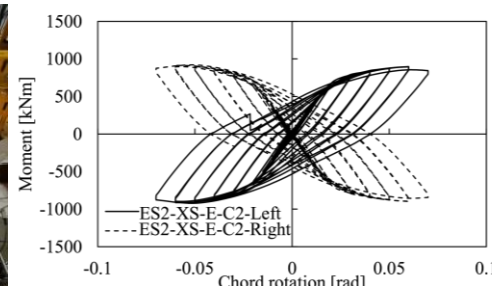
2 nominally identical tests per joint were carried out.











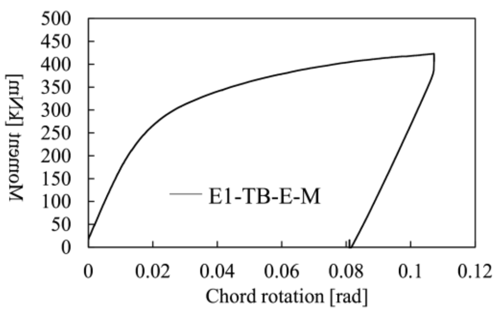
<u>The performance of the joints complies with both EC8 and AISC341</u>

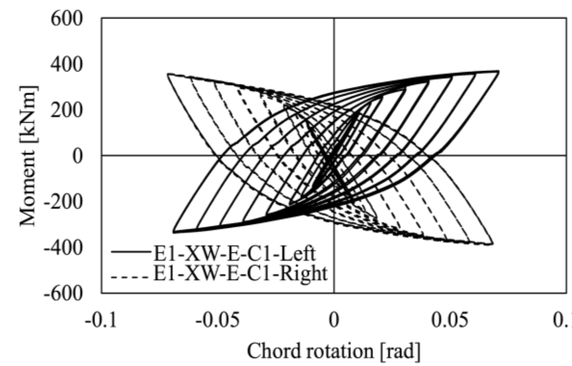
CONNECTIONS

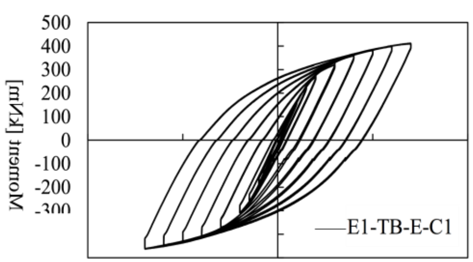
Equaljoints experimental program

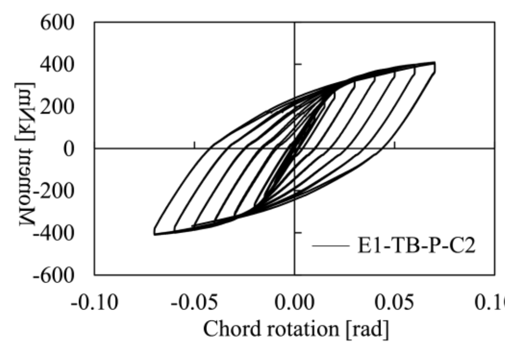
EXTENDED UNSTIFFENED JOINTS – 9 tests carried out at Univ. of Napoli Federico II + 15 tests carried out at University of Liege

2 nominally identical tests per joint were carried out.





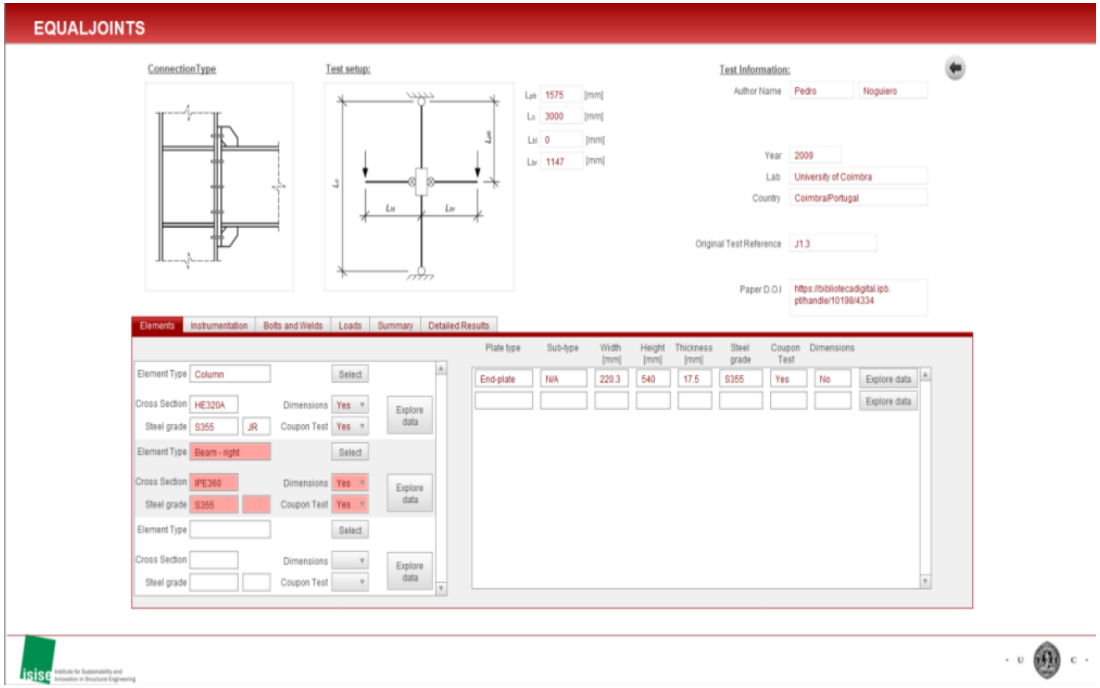




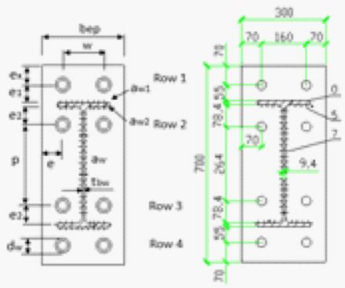
<u>The performance of the joints complies with EC8 requirements for DCM structures</u>

Experimental database

Experimental and numerical data are collected for producing prequalified charts. In this database all the available recorded data including the organization and source of the data, geometric properties of each element, material properties of each element, geometrical imperfection if available, loading protocols, hysteretic behavior of joint, failure mode and etc. are collected.



Prequalification charts and design guidelines

Component	Detail calculations	References
	Geometries a_{w1}=0 mm a_{w2}=5 mm a_w=7 mm b_{ep}=300 mm d_w=56 mm e=70 mm e_1=55 mm e_2=78.4 mm e_x=70 mm t_{wb}=9.4 mm w=160 mm  *"T-Stub" parameters* e_w=0.25*56=14 mm m, n and α parameters for the different bolt rows: *Row 1* $m = e_1 - 0.8a_{w1}\sqrt{2}$ =55.0-0.0=55.0 mm $n = \min[e_x, 1.25m]$ =min(70,1.25*55)=68.75 mm *Rows 2 and 3* $m = 0.5(b_p - 2e - t_{wb} - 1.6a_w\sqrt{2})$ =0.5*(300-2*70-9.4-1.6*7*1.414)=67.38 mm $n = \min[e, 1.25m]$ =min(70,1.25*67.38)=70 mm $m_2 = e_2 - 0.8a_{w2}\sqrt{2}$ = 78.4 -0.8*5*1.414 =72.74 mm $\lambda_1 = \frac{m}{m+e}$ =0.49 $\lambda_2 = \frac{m_2}{m+e}$ =0.53 →α =5.70 *Effective lengths* Bolt row 1 (individual) $l_{eff,1} = \min\begin{cases} 2\pi m,\ \pi m + w,\ \pi m + 2e \\ 4m + 1.25e_x,\ e + 2m + 0.625e_x,\ 0.5b_p,\ 0.5w + 2m + 0.625e_x \end{cases}$ $=\min\begin{cases} 2*3.14*55,\ 3.14*55+160,\ 3.14*55+2*70 \\ 4*55+1.25*70,\ 70+2*55+0.625*70,\ 0.5*300,\ 0.5*160+2*55+0.625*70 \end{cases}$ = 150 mm $l_{eff,2} = \min[4m + 1.25e_x,\ e + 2m + 0.625e_x,\ 0.5b_p,\ 0.5w + 2m + 0.625e_x]$ =min(4*55+1.25*70, 70+2*55+0.625*70, 0.5*300, 0.5*160+2*55+0.625*70] = 150 mm	EC3-1-8 6.2.6.5

Component	Detail calculations	References
Beam flanges and web in compression	$F_{fb,Rd} = M_{c,Rd}/(h - t_{fb})$ =604210000/(450-14.5)*10⁻³=1387.4 kN	EC3-1-8
Column web and continuity plates in compression	$b_{eff,c,wc} = t_{fb} + \sqrt{2}(a_{w1} + a_{w2}) + 5(t_{fc} + r_c) + 2t_p$ = 14.6+1.414*(0+5)+5*(21.5+27)+2*18=300.17 mm A_{cp}=18*(300-12)=5184 mm² $\omega = \frac{1}{\sqrt{1+1.3(b_{eff,c,wc}t_{wc}/A_{vc})^2}}$ =1/((1+1.3*(300.17*12/5609)²)^0.5)=0.807 k_{wc}=1.0 (supposing $\sigma_{com,Ed} < f_{y,wc}$) $F_{c,wc,Rd} = \frac{\omega k_{wc} b_{eff,c,wc} t_{wc} f_{y,wc}}{\gamma_{M0}} + \frac{A_{cp} f_{y,cp}}{\gamma_{M0}}$ =(0.807*300.17*12*355/1+5184*355/1)*10⁻³=2872.3 kN	
Beam web in tension	Bolt row 2(individual): $F_{t,wb,Rd,r2} = b_{eff,t,wb}\ t_{wb}\ f_{y,wb}\ /\ \gamma_{M0}$ =384.07*9.4*355*10⁻³/1=1281.64 kN Bolt row 3 (individual): $F_{t,wb,Rd,r3} = F_{t,wb,Rd,r2}$ = 1281.64 kN Group 1 (rows 2 +3): $F_{t,wb,Rd,g1} = b_{eff,t,wb}\ t_{wb}\ f_{y,wb}\ /\ \gamma_{M0}$ =(324.03+324.03)*9.4*355*10⁻³/1=2162.58 kN	EC3-1-8 6.2.6.8

CONNECTIONS

DISSIPATIVE JOINTS: Equaljoints

- **Equaljoints project was succesfully closed at the end of June 2016**
- **The final report has been accomplished.**


European Commission
Research Programme of the Research Fund for Co
TG S8

European pre-QUALified steel
EQUALJOINTS

Raffaele Landolfo, Mario D'Aniello, Silvia Costanzo,
Aurel Stratan, Dan Dubina, Cristian Vulcu, Cosmin M
Luis Da Silva, Carlos Rebelo, Hugo Augusto, Ashkan
Gentili, Jean-Pierre Jaspart, Jean-François Demoncea
Ahmed Elghazouli, Antonis Tsitos, Olivier Vassart, Edu
Veronique Dehan, Cecile Hamreza

Grant Agreement number RFSR-CT-2013-0
Reporting period: 1st July 2013 – 30th June 2


FINAL WORKSHOP
EQUALJOINTS PROJECT
European pre-QUALified steel JOINTS RFSR-CT-2013 – 00021
Napoli, June 21th 2016
Federico II Conference Centre, Via Partenope 36 (Seafront)

Final Workshop of EQUALJOINTS project
Program

9:00	Opening and welcome	
9:15	The Equaljoints Project	R. Landolfo
	Key note lecture 1: Steel Connection Failures from the 1994 to Today: The AISC 358 Effort to Prequalify Connections	R.T. Leon
10:15	Coffee Break	
10:45	Session 1: Design criteria and experimental performance	
	Haunched joints	D. Dubina
	Extended stiffened end-plate joints	M. D'Aniello
	Extended end-plate joints	J-F.Demonceau
	Dog-bone joints	O. Vassart
	Key note lecture 2: How to make EN 1993-1-8 easier to use?	M. Veljković
13:00	Lunch Break	
14:00	Key note lecture 3: Seismic design of bolted beam-to-column joints	V. Piluso
	Session 2: The influence of joints behaviour on global behaviour	
	Moment resisting frames	A.Elghazouli
	Dual-Concentrically braced frames	F. Gentili
	Dual-Eccentrically braced frames	A. Stratan
15:45	Coffee Break	
16:15	Session 3: Outcomes and dissemination	
	Analytical models	L. Da Silva
	Loading protocol for EU seismic pre-qualification of joints	A. Tsitos
	Design guidelines of pre-qualified seismic steel joints	J-P.Jaspart
	ECCS mission	V. Dehan
17:40	Workshop Summary	F.M.Mazzolani
	Discussion	
18:30	Closure	R. Landolfo

Imperial College London
ArcelorMittal
ECCS CECM EKS

Equaljoints PLUS Consortium

Coordinator		Università degli Studi di Napoli Federico II (UNINA)
Beneficiaries	Former EJ project	Arcelormittal Belval & Differdange SA (AM)
	Former EJ project	Universite de Liege (Ulg)
	Former EJ project	Universitatea Politehnica Timisoara (UPT)
	Former EJ project	Universidade de Coimbra (UC)
	Former EJ project	Convention Europeenne de la Construction Metallique (ECCS)
	Former EJ project	Universita degli Studi di Salerno (UNISA)
	Former EJ project	Imperial College of Science Technology and Medicine (IC)
		Centre Technique Industriel de la Construction Metallique (CTICM)
		National Technical University of Athens (NTUA)
		Ceske Vysoke Uceni Technicke V Praze (CVUT)
		Technische Universiteit Delft (TUD)
		Univerza V Ljubljani (UL)
		Universitet Po Architektura Stroitelstvo I Geodezija (UASG)
		Universitat Politecnica de Catalunya (UPC)
		Rheinisch-Westfaelische Technische Hochschule Aachen (RWTHA)

CONNECTIONS

DISSIPATIVE JOINTS: Equaljoints PLUS

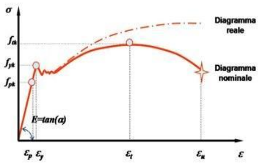
- **Equaljoint-PLUS** is a **24 month** RFCS project devoted to disseminate the knowledge achieved within the previous RCFS 36 months-project EQUALJOINTS

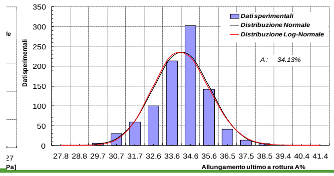
- **Equaljoint-PLUS** aims at the **valorisation**, the **dissemination** and the **extension** of the developed prequalification criteria **for practical applications to a wide audience** (i.e. academic institutions, Engineers and architects, construction companies, steel producers, etc.)

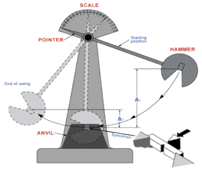
GENERAL
Design concept
Ductility class

Concept a) Low dissipative structural behaviour	DCL (Low)
Concept b) Dissipative structural behaviour	DCM (Medium)
	DCH (High)

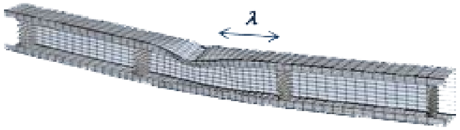
MATERIAL
Material overstrength
Toughness
Safety factors

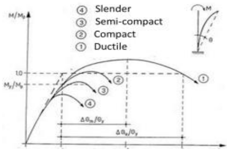




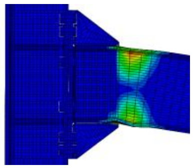


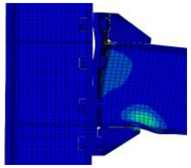
MEMBERS
Local ductility



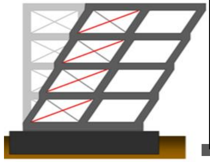


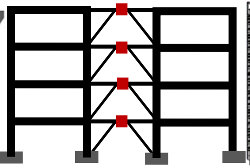
CONNECTIONS
NON-Dissipative B-To-C connections
Dissipative B-To-C connections

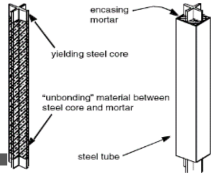




STRUCTURAL SYSTEMS
Traditional systems
Innovative systems and solutions







Under review

Structural systems

Traditional systems: current capacity design rules

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

where:

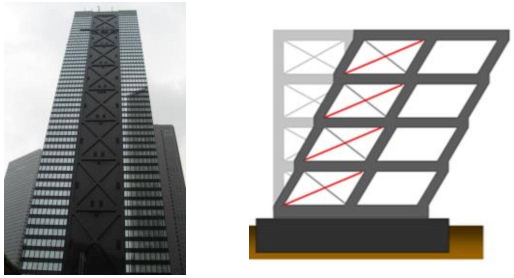
Ω is the minimum value of $\Omega_i = (R_{pl,Rd,i}/R_{Ed,i})$

being

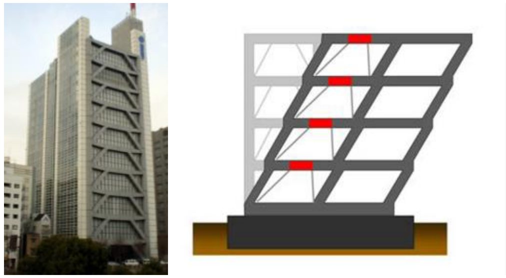
$R_{Ed,i}$ is the design value of the calculated elastic force in the dissipative part,i in the seismic design situation

$R_{pl,Rd,i}$ is the corresponding plastic strength

- **Moment resisting frames**



- **Concentrically braced frames**

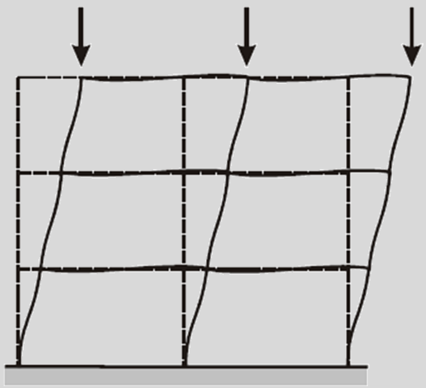


- **Eccentrically braced frames**

Moment resisting frames

Current rules and criticisms

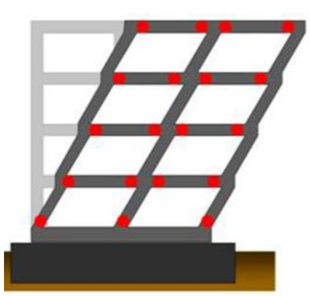
Frame stability

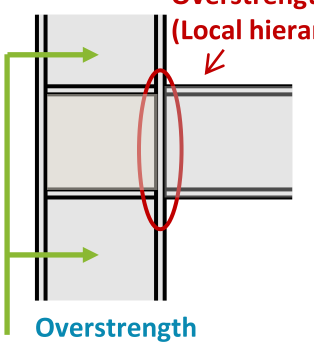


- *Frame instability is assumed for θ ≥ 0,3.*
- *If θ ≤ 0,1, second-order effects could be neglected*
- *for 0,1 < θ ≤ 0,2, P-Δ effects may be approximately taken into account in seismic action effects through the following multiplier:*

$$\theta = \frac{P_{tot} \cdot d_r}{V_{tot} \cdot h}$$

Capacity design requirements





Criticism !!!

- The need to satisfy stability requirements lead to oversize the beams of the frame.
- This leads to oversize the non-dissipative parts

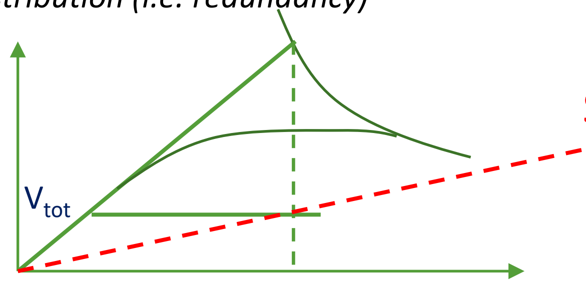
Moment resisting frames

Second order effects

Current EN-1998

Stability coefficient based on the secant stiffness of the idealized elastic-plastic response curve, which disregards the design overstrength and the plastic distribution (i.e. redundancy)

$$\theta = \frac{P_{tot} \cdot d_r}{V_{tot} \cdot h}$$

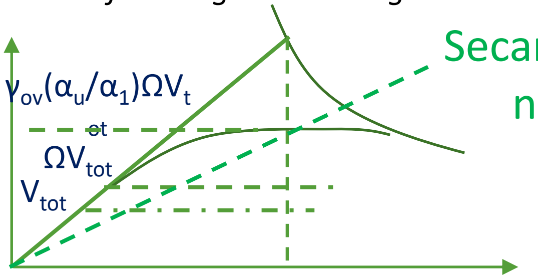


Secant Stiffness
current EC8

Next EN-1998

Modified stability coefficient based, which account for design overstrength and the plastic distribution

$$\theta = \frac{P_{tot} \cdot d_r}{V_{tot} \cdot h \cdot \gamma_{ov} \cdot \left(\alpha_u / \alpha_1\right) \cdot \Omega}$$



Secant Stiffness
next EC8

Moment resisting frames

Proposal for next EN 1998

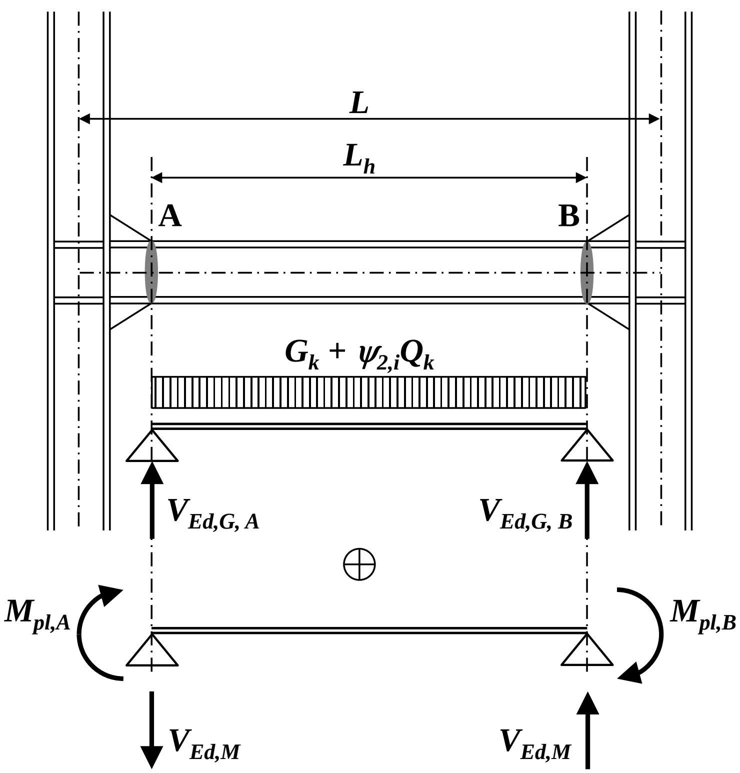
Design of beams

Using L is not conservative!

Current EN-1998

$$0.5V_{pl,i} \geq V_{Ed,i} = V_{Ed,G,i} + V_{Ed,M,i}$$

$$V_{Ed,M,i} = 2M_{pl,i}/L$$



Proposal for next EN-1998

$$V_{Ed,M,i} = 2M_{pl,i}/L_h$$

$$L_h = \left(L - 0.5d_c^{left} - 0.5d_c^{right} - s_h^{left} - s_h^{right}\right) = L$$

Moment resisting frames
Proposal for next EN 1998

Beam-to-column connections

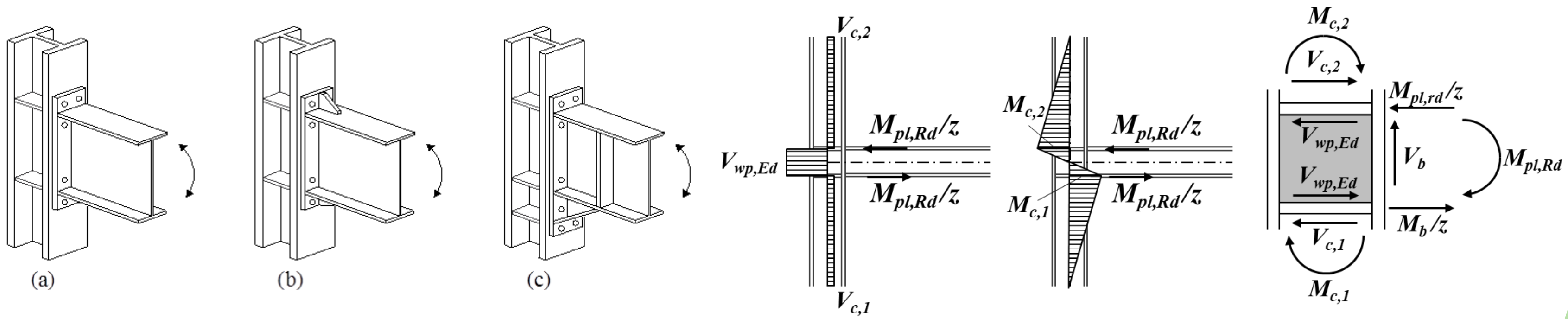
Current EN-1998

Consistently to the current codified provisions, the connections should satisfy :

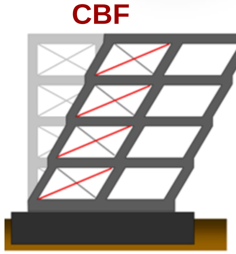
$$R_d \leq 1.1 \cdot \gamma_{ov} \cdot R_{fy}$$

Proposal for next EN-1998

Annex with design rules and pre-qualification of joints

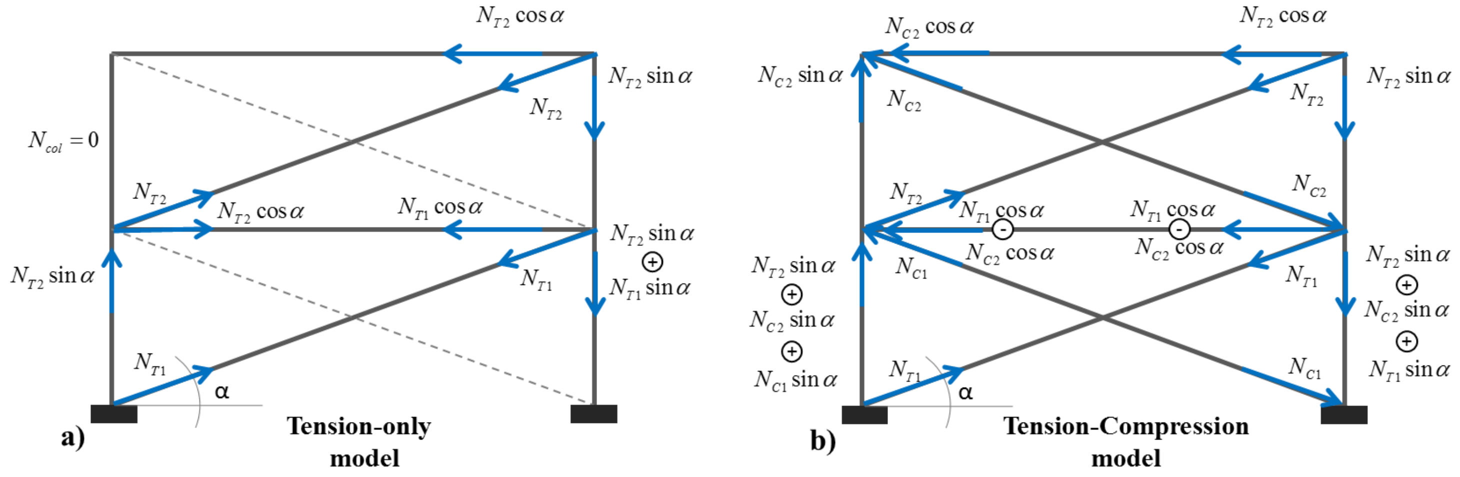


Concentrically braced frames
Current rules and criticisms



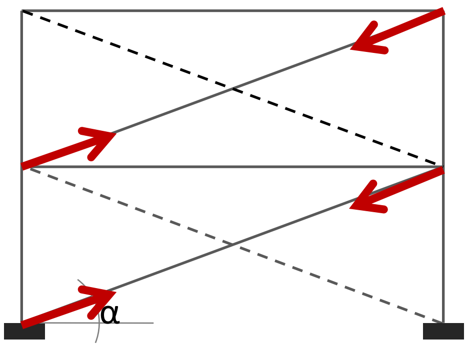
X-CBFs

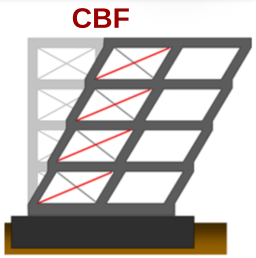
- In X-braced CBFs it is not allowed to consider the contribution of the compression diagonals, although it limits the slenderness of all diagonals. It should be clarified if compression diagonals should not be considered in analysis/design of CBFs or only in cases altogether or only in cases where there are no limits to the brace slenderness.
- **Potential upgrade:** Consider two cases in X-braced CBFs: a) where the contribution of the compression diagonal is taken into account in analysis/design and b) where it is not taken into account as in the existing clause.



X-CBF Proposal for next EN 1998

Current rules and criticisms





Slenderness limitation

$$1.3 \leq \bar{\lambda} \leq 2$$

Overstrength factor variation:

Ω should vary in a range (Ω, 1.25 Ω)

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

- **The requirements on brace slenderness and overstrength variation are pratically impossible to be simoultaneously fulfilled for the most common configurations.**
- **Lower slenderness limit for x-braced frames**: in addition to the upper limit for all frames, a lower slenderness limit of 1.3 imposed in EC8 for X-bracing, in order to limit the compression force in the brace. Satisfying this limit can result in **significant difficulties in practical design.**
- **Overstrength in diagonal members**: in order to mitigate the vulnerability of braced frames to the concentration of inelastic demand within critical storeys, EC8 introduces a 25% limit on the maximum difference in brace over-strength within the frame. Detailed studies show that this may not eliminate the problem and can impose additional design effort and difficulties in practical design.

X-CBF Proposal for next EN 1998

Design of bracings

Current EN-1998

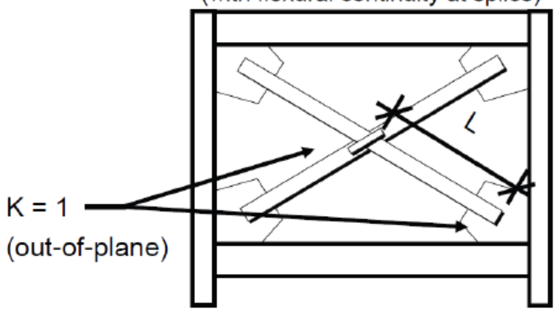
Brace slenderness

- At the current stage, EN 1998 solely mandates the slenderness ratio limitations, **but no specific guidance is provided for on how designers have to compute the buckling length** according to the stiffness of complementary brace and of the brace-to-brace connection
- In absence of specific instructions, the use of **T-O** (tension only) model can likely induce the designer to assume the **buckling length** considering the **whole length of the brace, and it would lead to inaccurate estimation of diagonal slenderness ratio.**

Proposal for next EN-1998

Brace slenderness

Cross Bracing
(with flexural continuity at splice)

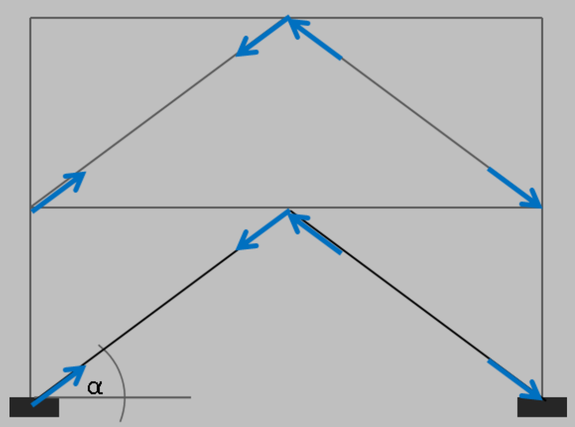


V-CBF Proposal for next EN 1998

Structural model and global analysis

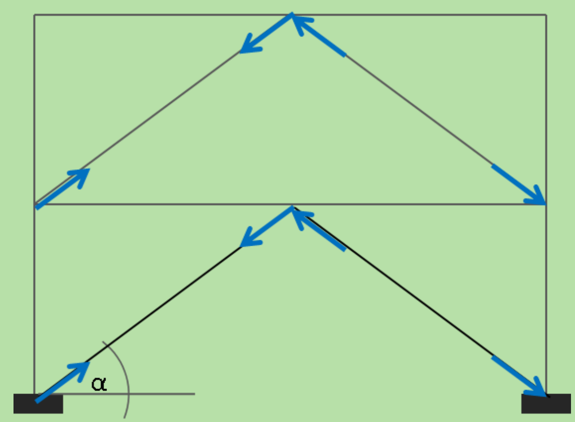
Current EN-1998

- Global elastic analysis is performed by assuming T-C model in which both tension and compression diagonals are specifically accounted for.



Proposal for next EN-1998

- Global elastic analysis is performed by assuming T-C model in which both tension and compression diagonals are specifically accounted for.



V-CBF Proposal for next EN 1998

Design of non-dissipative members: beams

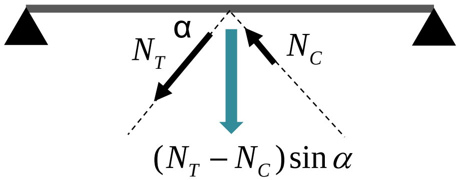
Current EN-1998

REQUIRED STRENGTH

The **required strength** is determined considering:

1) Non-seismic loadings without accounting for the intermediate support given by diagonals

2) **Seismic induced effects:** free-body distribution of plastic forces is considered as follows:



$$N_T = N_{pl,br,i} \qquad N_C = 0.3 N_{pl,br,i}$$

Proposal for next EN-1998

REQUIRED STRENGTH

The **required strength** is determined considering:

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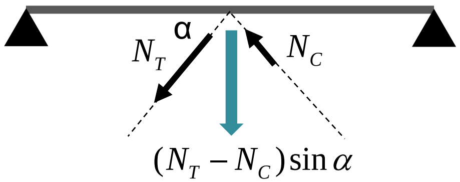


$$N_T = 1.1 \cdot \gamma_{ov} \cdot N_{pl,br,i} \qquad N_C = 0.3 \cdot \gamma_{ov} \cdot \chi \cdot N_{pl,br,i}$$

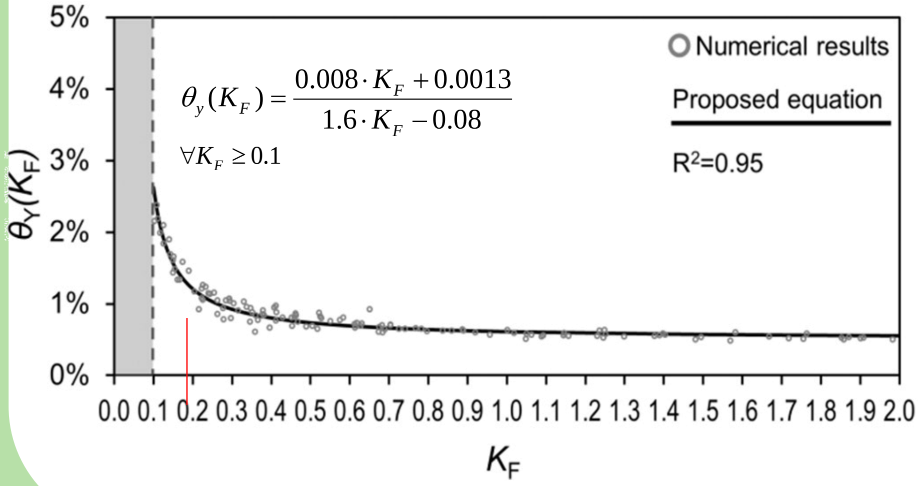
larger unbalanced force

V-CBF Proposal for next EN 1998

Design of non-dissipative members: beams

Proposal for next EN-1998

ADDITIONAL REQUIREMENT DEVOTED TO CONTROL THE BEAM FLEXURAL STIFFNESS



$$K_F \geq 0.2$$

$$K_F = \frac{k_b}{k_{br}} = \frac{\text{BEAM FLEXURAL STIFFNESS}}{\text{BRACES VERTICAL RIGIDITY}}$$

BEAM FLEXURAL STIFFNESS $k_b = 48\beta \frac{EI_b}{L_b^3}$

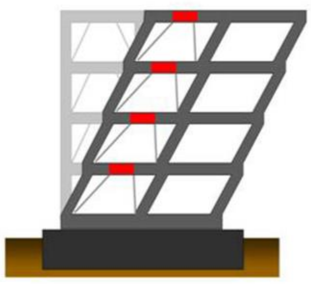
BRACES VERTICAL RIGIDITY

$$k_{br} = 2\frac{A_{br}E}{L_{br}} sen^2\alpha$$

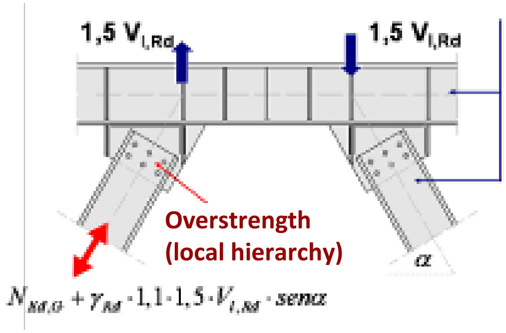
More detail in: D'Aniello M., Costanzo S., Landolfo R. (2015) The influence of beam stiffness on seismic response of chevron concentric bracings. Journal of Constructional Steel Research 112(112C):305-324.

Eccentrically braced frames
Current rules and criticisms

Global hierarchy



Local hierarchy



Required strength of braces:

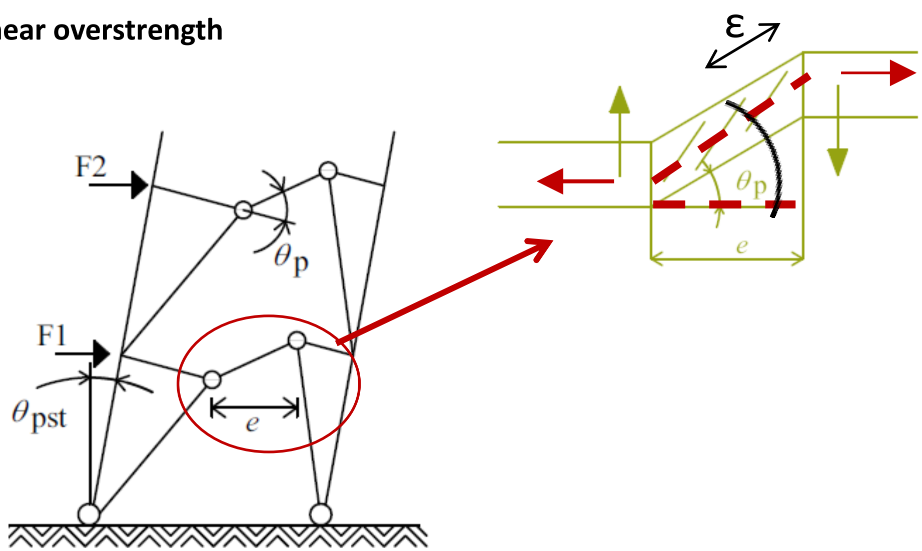
$$N_{Ed} = N_{Ed,G} + 1{,}1\gamma_{ov}\,\Omega N_{Ed,E}$$

Ω is the minimum value of
$\Omega_i = 1.5V_{pl,Rd,i}/V_{Ed,i}$ for short links
$\Omega_i = 1.5M_{pl,Rd,i}/M_{Ed,i}$ for long links

Criticism !!!

Braces tends to buckle under design forces

Shear overstrength



Criticism !!!

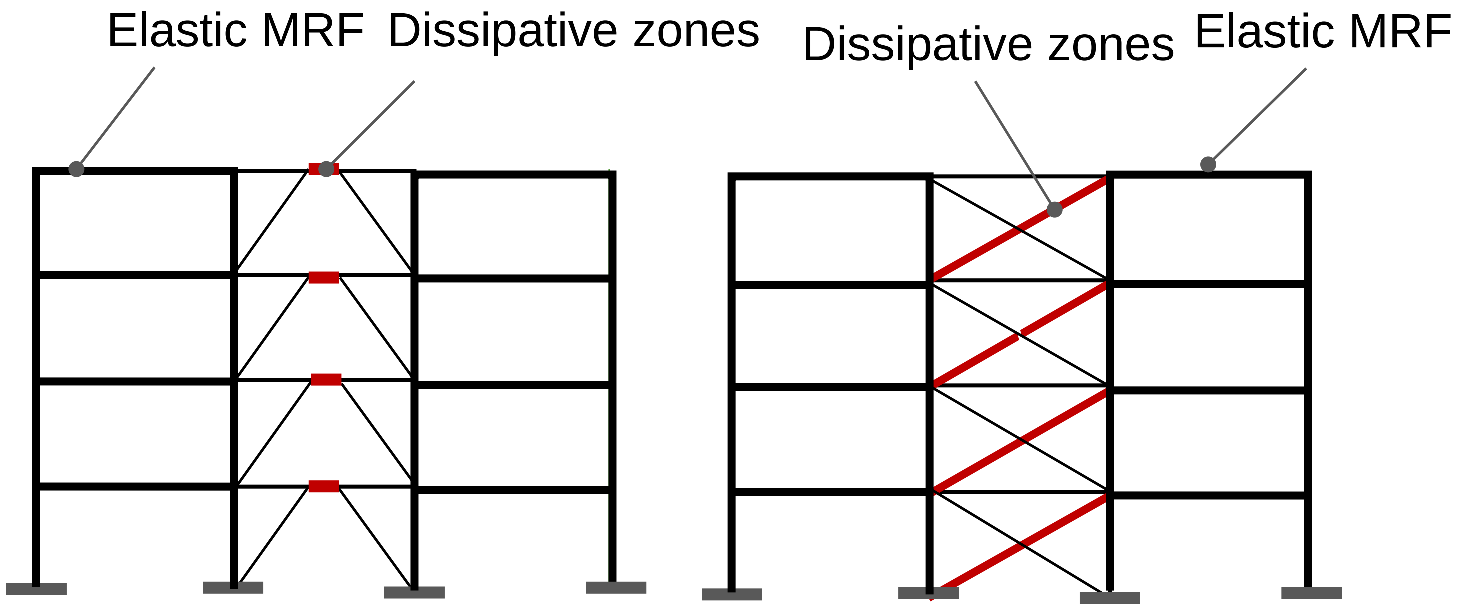
Under large deformations catenary effects occurr in the links; thereby, the EC8 disregards 2 effects:
- axial force in the link
- increasing link shear ovestrength (larger than 1.5 V_{pl})

SEISMIC DESIGN OF STEEL STRUCTURES:
NEW TRENDS OF RESEARCH AND UPDATES OF EUROCODE 8

Contents

Innovative solutions: reparability and re-centering dual-systems

- Recent studies were devoted to investigate **the re-centering capability of dual structures** with **removable dissipative members**.
- plastic deformations constrained to replaceable dissipative members
- re-centring capability of the structure provided by the elastic substructure



Innovative solutions: re-centering dual-systems

SEVENTH FRAMEWORK PROGRAMME

DUAREM PROJECT was devoted tu Validate the re-centering capability of dual-EBF structures with removable dissipative links

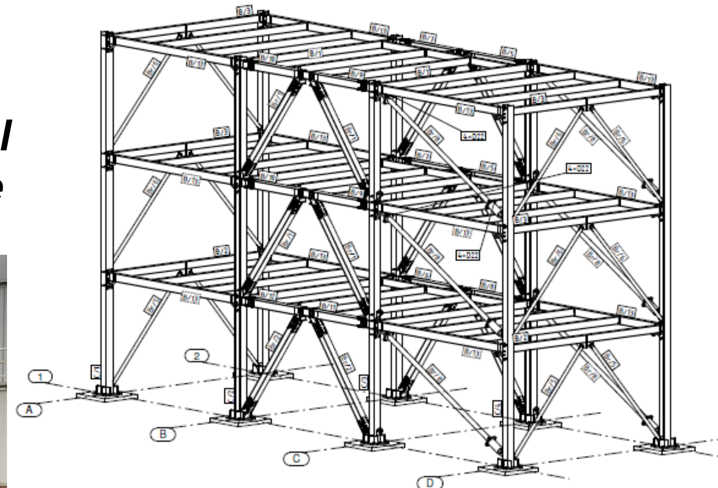
Acknowledgements: European Community's **Seventh Framework Programme** [FP7/2007-2013] for access to the European Laboratory for Structural Assessment of the European Commission – **Joint Research Centre** under grant agreement n° 227887”.

PARTNERS	• Leader: **Politechnica University of Timisoara** • University of Naples "Federico II" • University of Liege • University of Ljubljana • University of Coimbra

European Commission

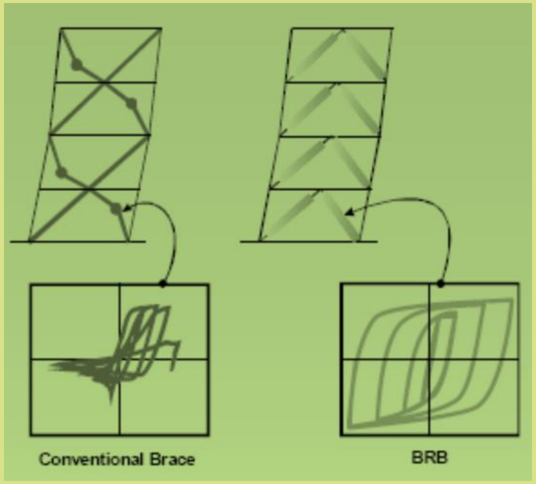
Innovative solutions: re-centering dual-systems

Duarem
"Full-scale experimental validation of dual eccentrically braced frame with removable links"

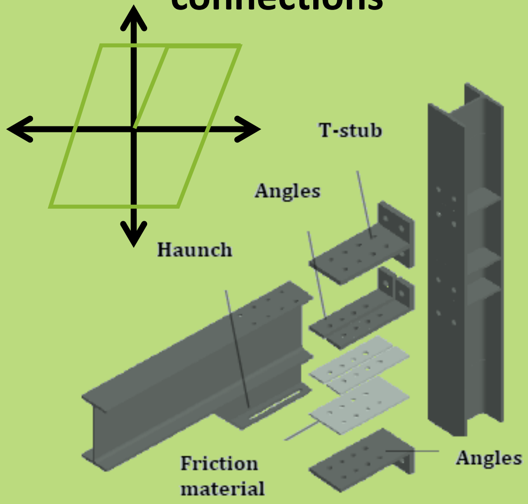


Low-damage systems

Buckling restrained braces (BRBs)

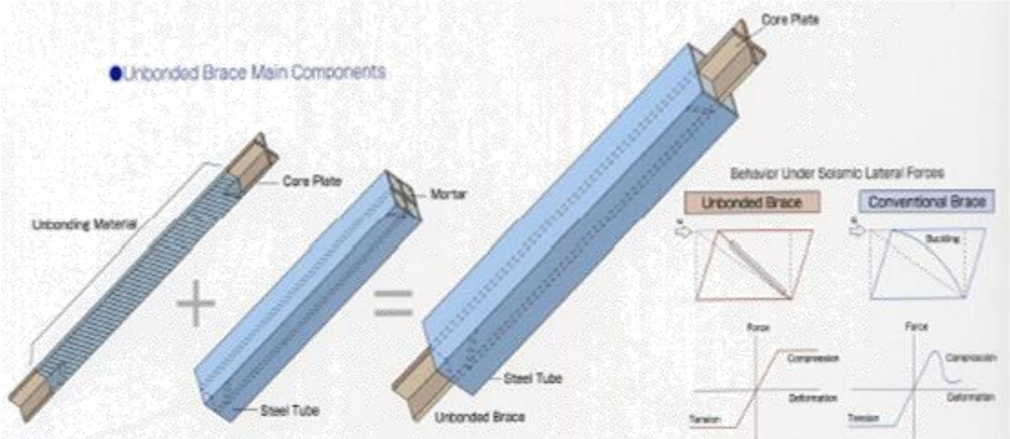


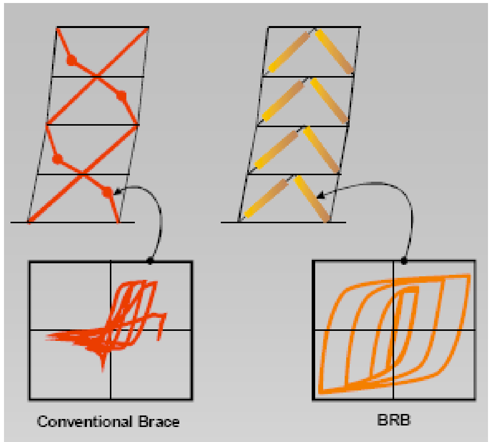
Free from damage connections



Lightweight steel-framed systems

Buckling restrained braces





Buckling restrained braces

Missing provisions

Structures with BRBs are not currently codified in Europe. Next version of EN-1998 needs to cover the following aspects:

- *q* factors
- Detailing rules for dissipative BRBs
- Qualification criteria for BRBs
- Detailing rules for non-dissipative members

BRBs Proposal for NEXT EN 1998-1

Design of bracings

Current EN-1998	Proposal for next EN-1998
X	

Proposal for next EN-1998

REQUIRED STRENGTH

i-th storey: $N_{pl,br,i} \geq N_{Ed,br,\mathrm{E},i} + N_{Ed,br,\mathrm{G},i}$

COMPRESSION STRENGTH ADJUSTMENT FACTOR

$$\beta = \frac{N_{C,Rd}}{N_{T,Rd}} \leq 1.30$$

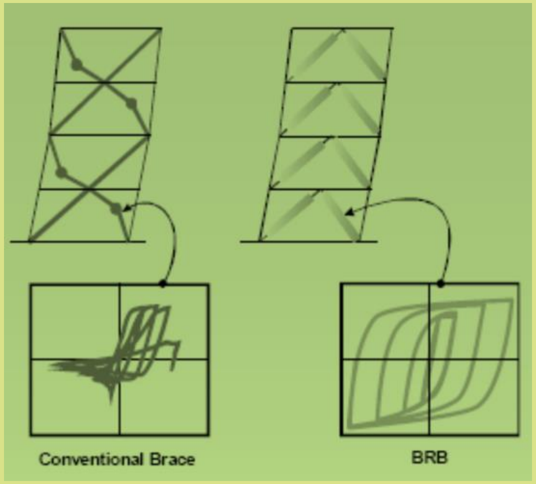
OVESTRENGTH HOMOGENEITY CONDITION

$$\left[\left(\Omega_i - \Omega\right)/\Omega\right] \leq 0.25$$

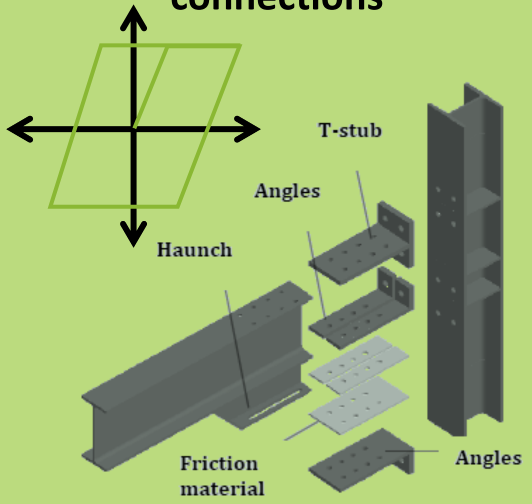
$$\Omega = \min\left(\Omega_i\right) = \min\left(\frac{N_{pl,br,Rd,i}}{N_{Ed,br,i}}\right) \quad i \in [1, n]$$

Low-damage systems

Buckling restrained braces (BRBs)

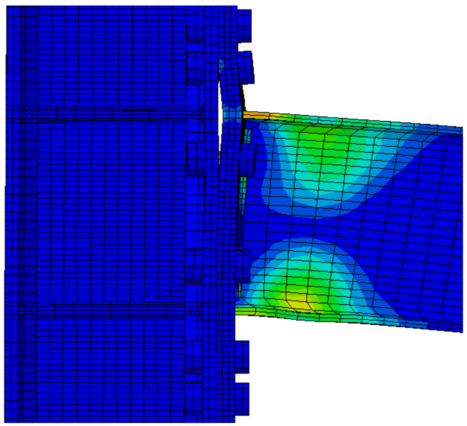


Free from damage connections

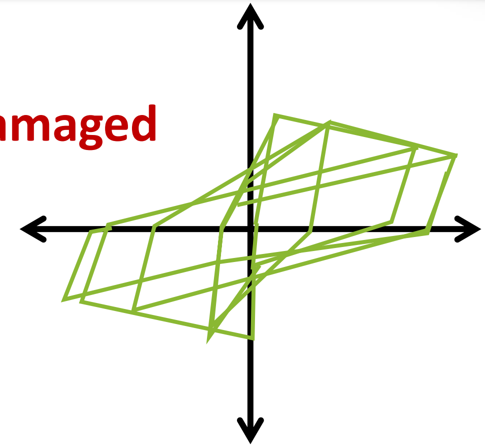


Lightweight steel-framed systems

FREE DAMAGE CONNECTIONS

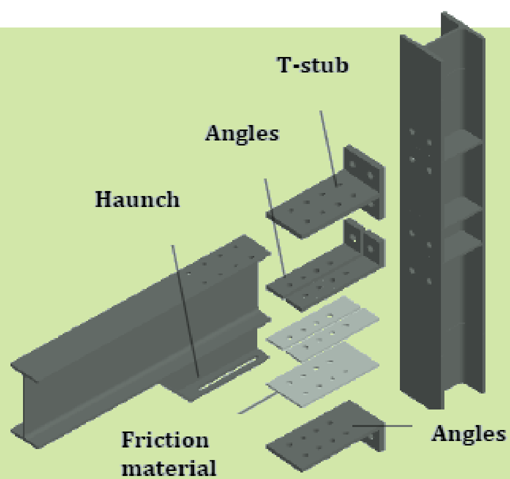


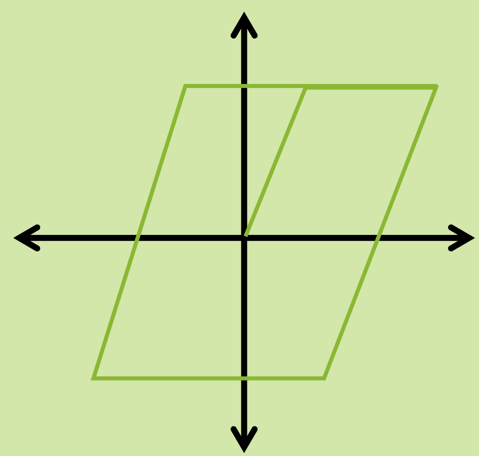
Dissipative but damaged



What's next?

Free damage connections





FREE DAMAGE CONNECTIONS

THE FREEDAM PROJECT

Research Fund for Coal & Steel

FREEDAM PROJECT

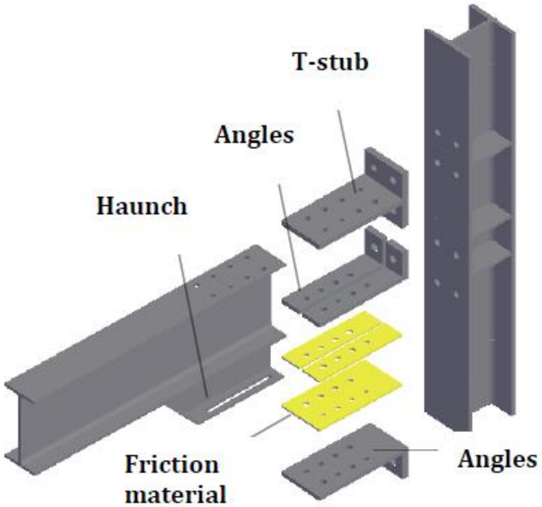
PARTNERS

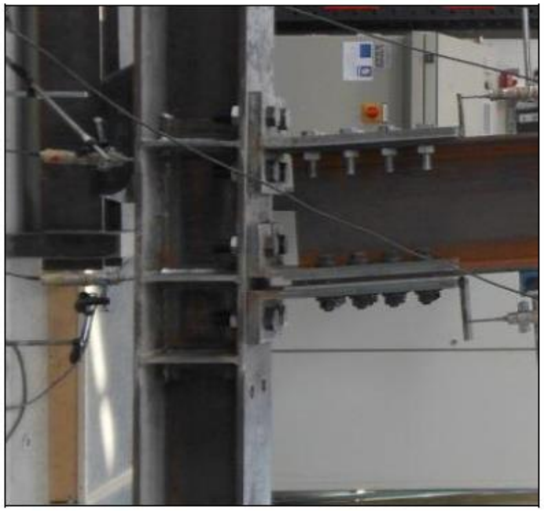
- Leader: **University of Salerno**
- University of Naples "Federico II"
- University of Liege
- University of Coimbra
- FIP
- O FELIZ

FREE DAMAGE CONNECTIONS

THE FREEDAM PROJECT

- The research is aimed at the development of innovative beam-to-column connections equipped with friction dampers which are located at the bottom flange level of the connected beam to dissipate the earthquake input energy.
- The friction resistance is calibrated by acting on the number and diameter of bolts and their tightening torque governing the preloading. The flexural resistance results from the product between the damper friction resistance and the lever arm. Such connections exhibit wide and stable hysteresis loops without any damage to the connection steel plate elements

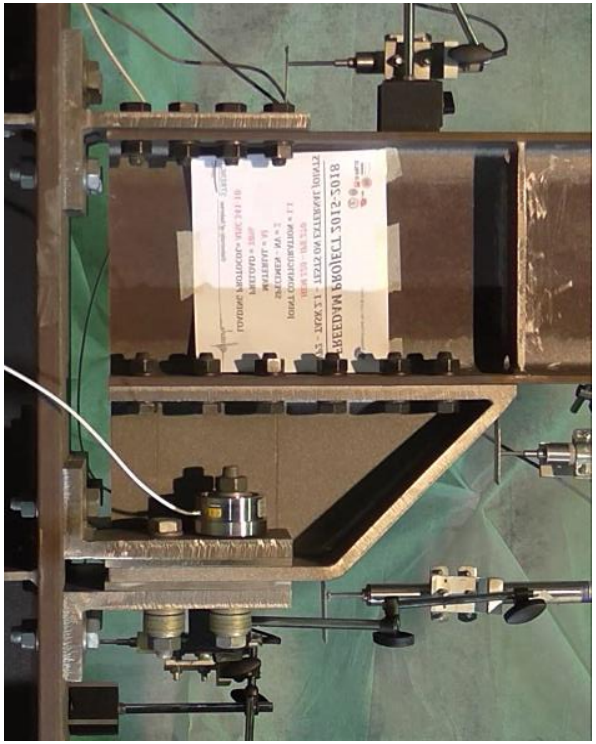


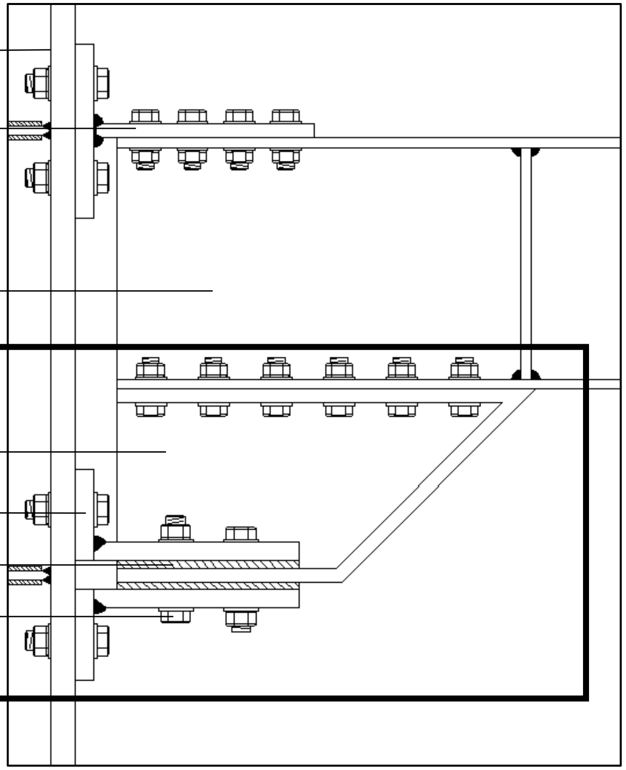


FREE DAMAGE CONNECTIONS

THE FREEDAM PROJECT

Tested Joints

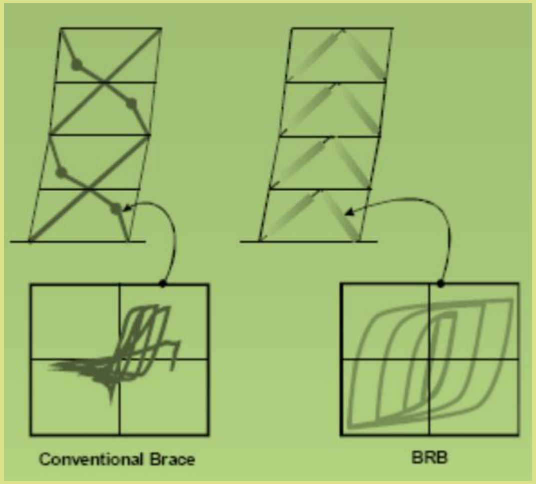




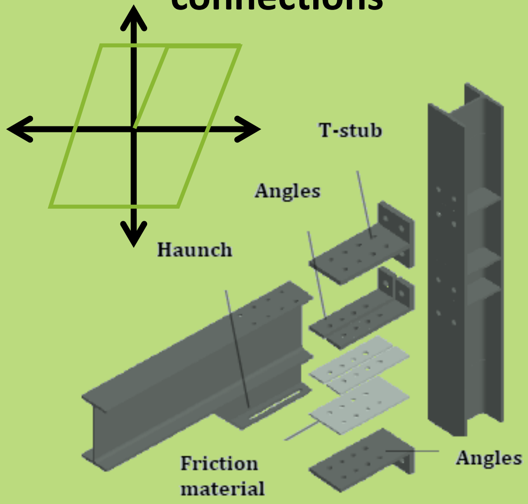
Latour M., D'Aniello M., Zimbru M., Rizzano G., Piluso V., Landolfo R. (2018) Removable friction dampers for low-damage steel beam-to-column joints. Soil Dynamics and Earthquake Engineering (accepted).

Low-damage systems

Buckling restrained braces (BRBs)



Free from damage connections



Lightweight steel-framed systems

Innovative systems: *Lightweight steel constructions*

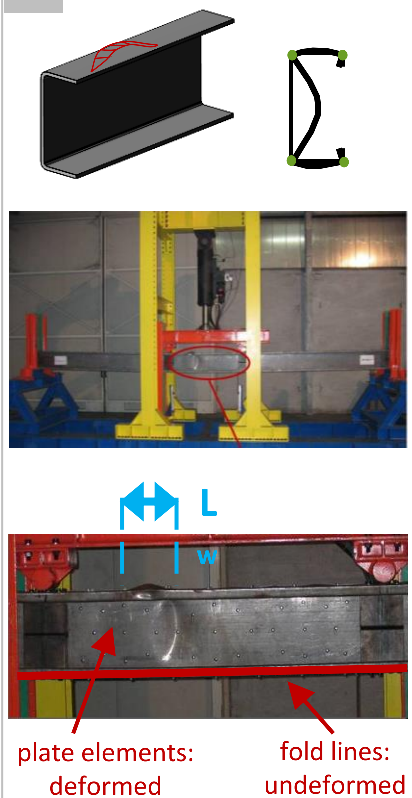
Lightweight Steel-Framed Construction using Cold-Formed Steel (CFS) profiles are even more light

Innovative systems: *Lightweight steel constructions*

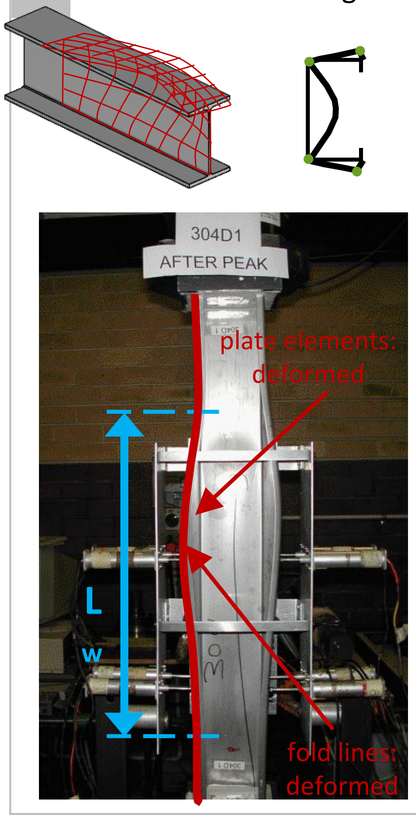
Cold-Formed Steel profiles

Effects of the high lightweight of CFS profiles: instability phenomena

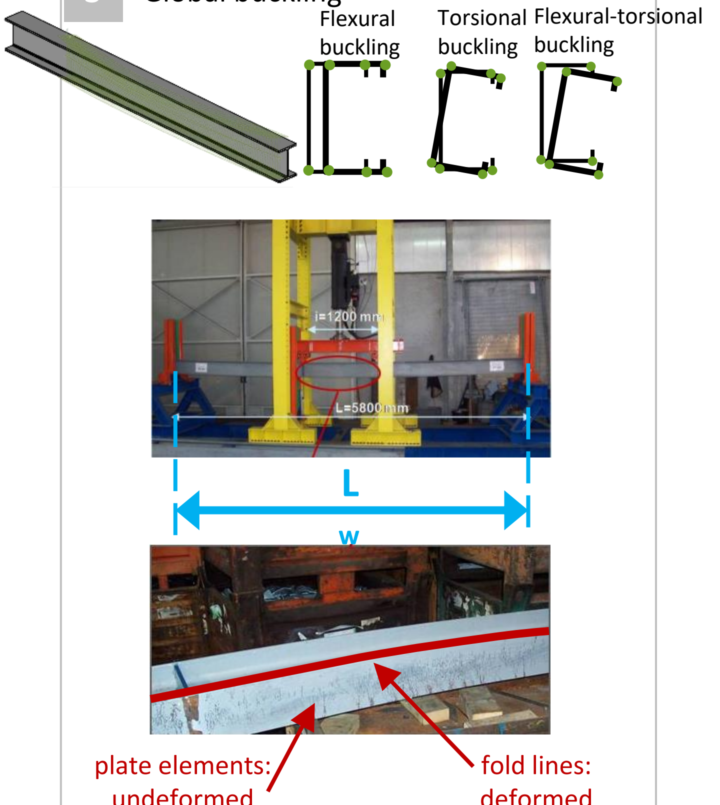
1 Local buckling



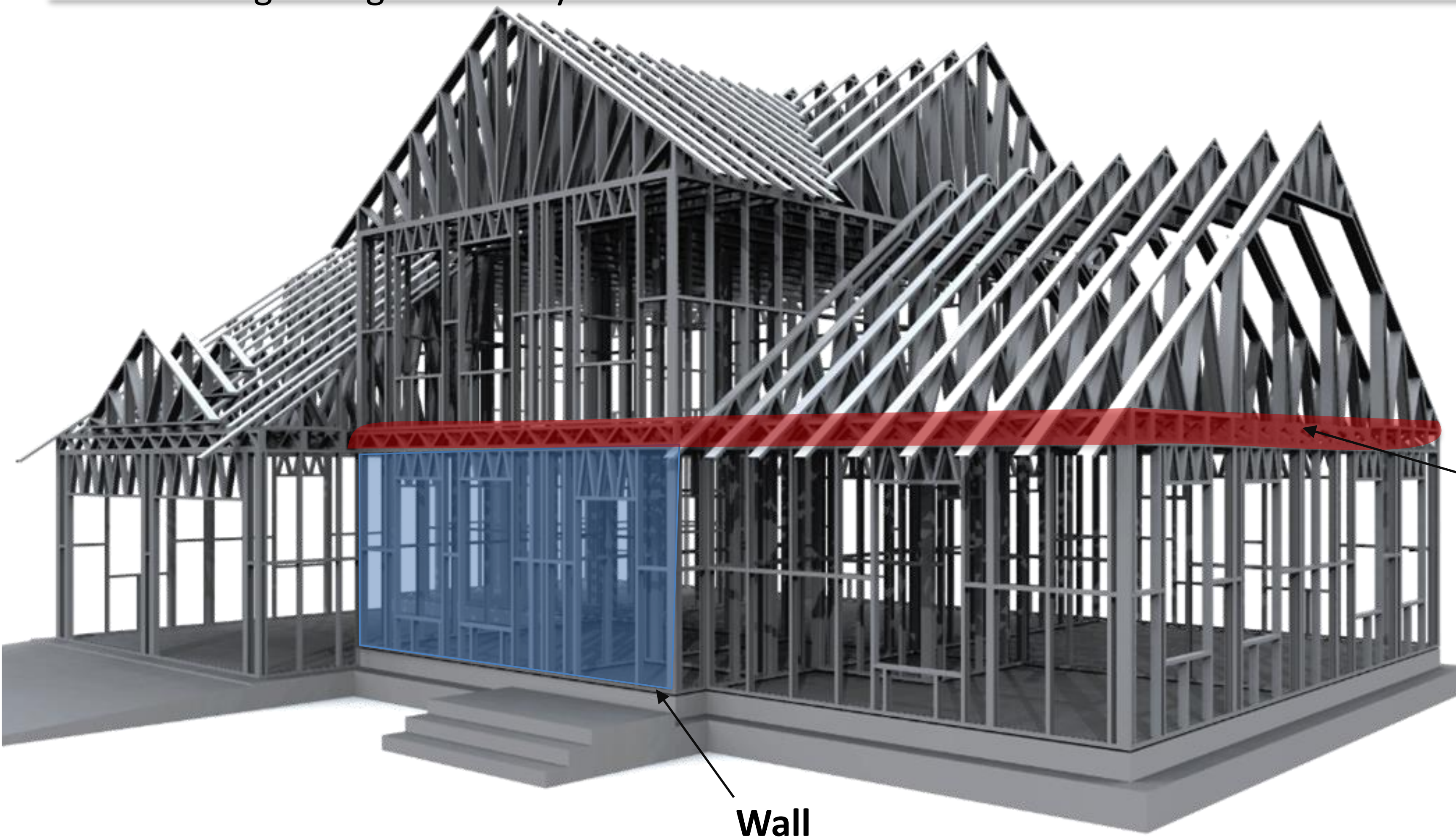
2 Distortional buckling



3 Global buckling

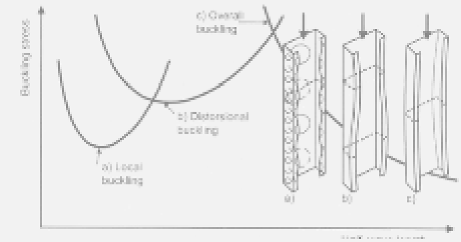


Ultimate Limit States: Structural applications of Lightweight steel drywall constructions -> **STRUCTURAL COMPONENTS**



Main **STRUCTURAL COMPONENTS**

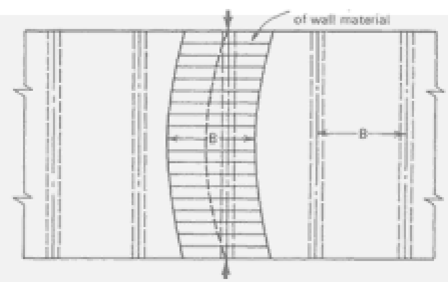
All-steel (strap-braced) design



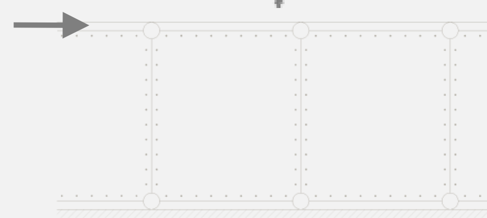
VERTICAL LOADS

HORIZONTAL LOADS

Sheathing-braced design



VERTICAL LOADS



HORIZONTAL LOADS

Innovative systems: *Lightweight steel constructions*

Open issues on seismic design

Seismic design criteria according to Eurocode 8

Eurocode 8 does not provide any specific prescription for the design of lightweight steel constructions in seismic area.

All-steel structure

All-Steel Structures could be designed according a **DCL (low dissipative approach for low seismicity zones)** by assuming the behaviour factor equal to **1.5** without capacity design rules.

However, this approach may be restrictive, since the lightness of these systems makes them a good solution also for high seismicity zones

Sheathing- braced structure

For Lightweight Steel-Framed Constructions designed according to **Sheathing-braced approach, EC8 in not applicable**

There is a gap between the European code specifications and the application of these systems in seismic areas

Innovative systems: *Lightweight steel constructions*

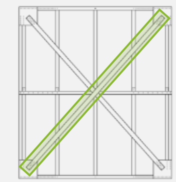
Open issues on seismic design

Seismic design criteria according to North American codes

North American Codes (**AISI S400** for USA, Mexico and Canada; **ASCE 7** for USA and Mexico; **NBCC** for Canada) allow the **dissipative design approaches according to the Capacity design.**

All-steel structure

Strap-braces act as the energy-dissipating elements



	ASCE 7	NBCC
Behaviour factor	4.0 (bearing wall systems)	2.47
Overstrength factor	the non-dissipative elements designed by considering the forces corresponding to the expected yield strength of diagonal	

Sheathing- braced structure

Sheathing connections act as the energy-dissipating elements

	ASCE 7		NBCC	
Behaviour factor	6.5 (bearing wall systems)	7.0 (building frame systems)	4.25 (shear walls with wood-based structural panel sheathing	
Overstrength factor	2.5 (building frame systems)	3.0 (bearing wall systems)	1.33 (DFP and OSB panels)	1.45 (CSP wood panels)

Innovative systems: *Lightweight steel constructions*

Ongoing researches

Structural all-steel systems

- Italian national research project
  ReLUIS-DPC, Line 1, years 2010-2013
- National research project
  Lamieredil-UNINA Project, years 2014-2017

Structural sheathing-braced systems

- National research project
  Prin, years 2001 - 2005
- European research project
  ELISSA Project, years 2013-2016

Drywall non-structural building components

- European research project
  Knauf-UNINA Project, years 2012 - 2016
- National research project
  Guerrasio-UNINA Project, years 2016 - 2017

Innovative systems: *Lightweight steel constructions*

Ongoing researches: Lamieredil-UNINA Project

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Innovative systems: *Lightweight steel constructions*

Ongoing researches: Lamieredil-UNINA Project

General experimental program

Test type		no. tests
Material, component and connections tests	Steel material	12
	Self-drilling screws	3
	Joints between gussets plate and strap-brace	6
	Hold-down device	4
Wall tests	In-plane monotonic tests	2
	In-plane quasi-static reversed cyclic tests	4
Shake table of 3D prototypes	Dynamic identification and earthquake tests	16 + 14 on 2 prototypes
	Total no. of tests	***61***

1:3 Reduced scale specimens

Innovative systems: *Lightweight steel constructions*

Ongoing researches: Lamieredil-UNINA Project

Tests on materials, components and connections

Tests on steel materials

Total tests: 12

Tests on Self-drilling screws

Total tests: 3

Tests on joints between gussets plate and strap-brace

Total tests: 6

Tests on hold-down devices

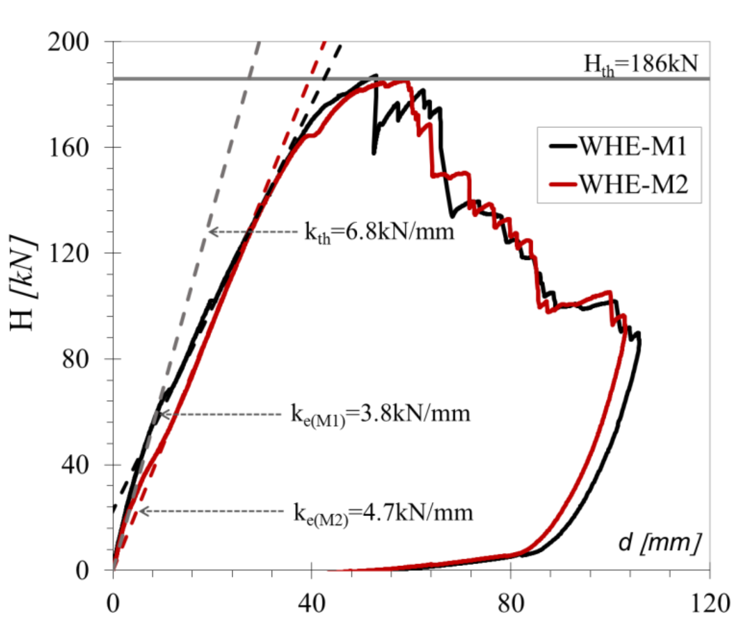
Total tests: 4

Innovative systems: *Lightweight steel constructions*

Ongoing researches: Lamieredil-UNINA Project

In-plane monotonic tests on wall specimens

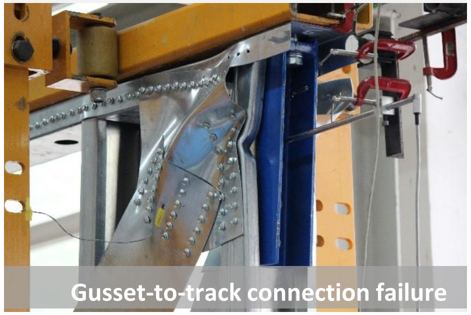
Test results



2 monotonic tests

Failure modes

W1-M1


Gusset-to-track connection failure

W1-M2


Gusset-to-track connection failure

Type	H_y (kN)	H_p (kN)	d_y (mm)	d_p (mm)	k_e (kN/mm)	FM
WHE-M1	160.2	187.1	36.6	52.9	3.8	GT
WHE-M2	164.1	185.7	38.9	58.7	4.7	GT

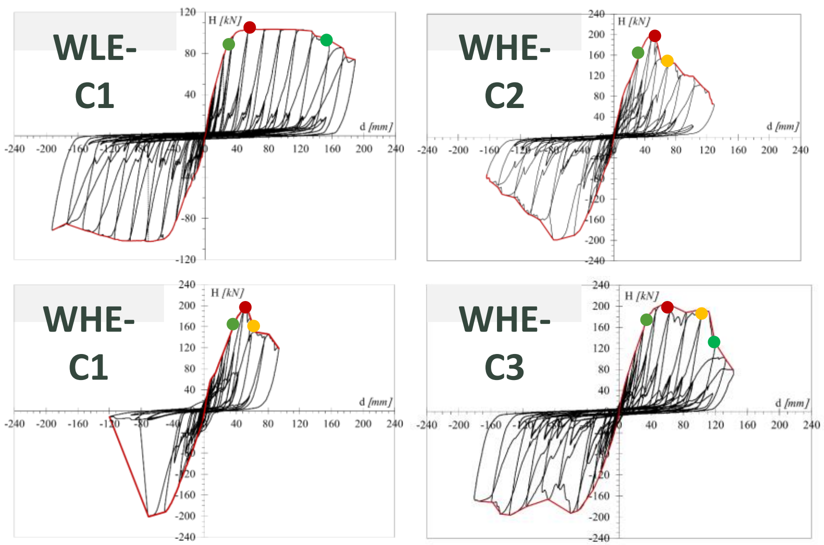
FM: Failure mode; GT: failure of gusset-to-track connection

Innovative systems: *Lightweight steel constructions*

Ongoing researches: Lamieredil-UNINA Project

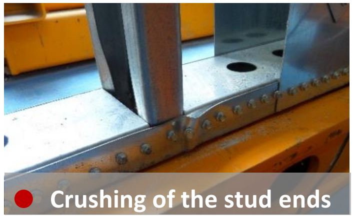
In-plane quasi-static reversed cyclic tests on wall specimens

Test results	Failure modes



4 cyclic tests

Local buckling of the tracks



Crushing of the stud ends

Gusset-to-track connection failure

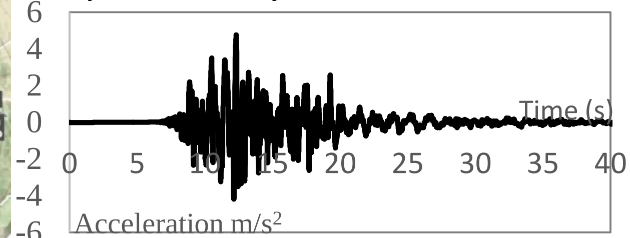
Plastic hinges in chord studs

Innovative systems: *Lightweight steel constructions*

Ongoing researches: Lamieredil-UNINA Project

Dynamic earthquake tests - Input: 2016 Norcia Earthquake

Input time history NRC-EW

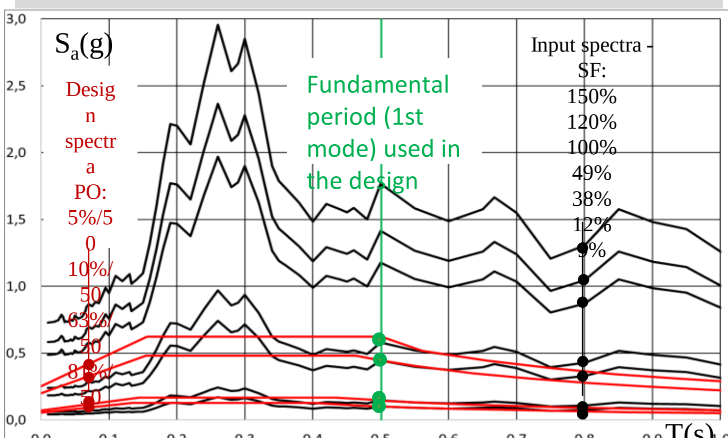


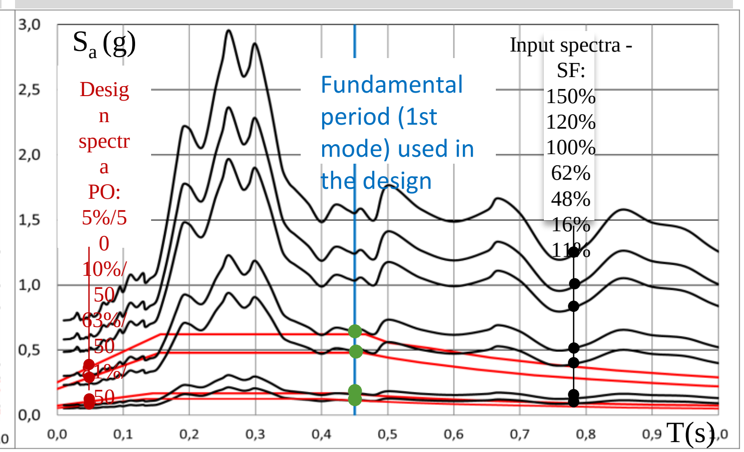
SELECTED GROUND MOTION

Event: *Norcia – October 30th, 2016 6:40 a.m.*
Magnitude: Mw= 6.5
Station: *Norcia*
Station code: *NRC*
PGA: *4.76 m/s²* (0,49 g)

Input spectrum vs. design spectrum (S_a-T format)

Concrete solution	OSB solution





SF: Scaling Factor; PO: Probability of occurrence

Innovative systems: *Lightweight steel constructions*

Ongoing researches: Lamieredil-UNINA Project

Earthquake test on shake table of the 1:3 reduced scale CFS three-storeys strap-braced stud structure (concrete solution)

Videos recorded during the Earthquake test with scaling factor of ***150%***

Innovative systems: *Lightweight steel constructions*

Ongoing researches: ELISSA Research Project

Structural all-steel systems

- Italian national research project
  ReLUIS-DPC, Line 1, years 2010-2013
- National research project
  Lamieredil-UNINA Project, years 2014-2017

Structural sheathing-braced systems

- National research project
  Prin, years 2001 - 2005
- European research project
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Drywall non-structural building components

- European research project
  Knauf-UNINA Project, years 2012 - 2016
- National research project
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Innovative systems: *Lightweight steel constructions*

Ongoing researches: ELISSA Research Project

Research funded by European Commission within the Project named "Energy Efficient LIghtweight-Sustainable-SAfe-Steel Construction" (Project acronym: **ELISSA**).

E2B Energy Efficient Buildings · SEVENTH FRAMEWORK PROGRAMME

ELISSA

ELISSA Research Project

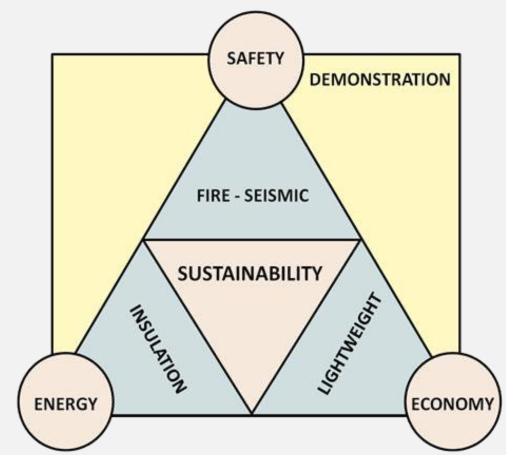
Energy Efficient LIghtweight – Sustainable – SAfe – Steel Construction

PARTNERS

- Takes the weight off your project.
- KNAUF Gips
- farbe Colori di ogni colore
- va-Q-tec Creating energy efficiency
- ZAE BAYERN
- stress
- University of ULSTER FireSERT
- Wölfel Beratende Ingenieure

Project objective

The ELISSA project was devoted to the development and demonstration of **nano-enhanced prefabricated lightweight Cold-Formed Steel (CFS) skeleton/dry wall constructions** with improved of energy efficiency, fire and seismic safety and sustainability.



Innovative systems: *Lightweight steel constructions*

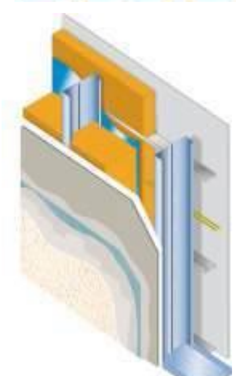
Ongoing researches: ELISSA Research Project

The reference structural system: The COCOON “Transformer”

The system already obtained the European Technical Approval for static loads and the upgrading to withstand also **seismic loads** is one of the main objective of the ELISSA project.

Research goal for DIST

Evaluation of the seismic response of sheathed CFS buildings by means experimental tests on connections, walls and 3D mock-up.



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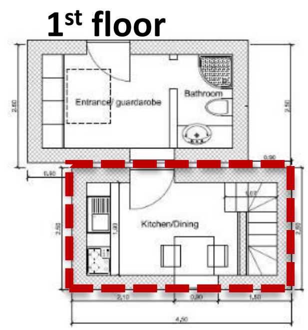
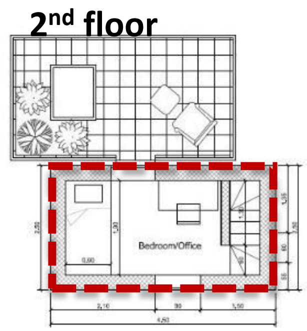
The case study: The "ELISSA house"

The case study consists of a three-rooms two-storeys dwelling named **"ELISSA house"**.

The load-bearing structure of ELISSA house is based on CFS frames (walls and floors) produced by COCOON sheathed with gypsum-based board panels produced by KNAUF (Diamant boards for walls and GIFAfloor boards for floors).





"ELISSA HOUSE" data

- **3 rectangular modules** of plan dimensions **2.5 x 4.5 m**, horizontally and vertically jointed
- **Two storeys building**
- **Total gross area:** 34 m^2 + terrace
- **Total height:** 5.4 m

The Elissa Mock-up

ELISSA MOCK-UP data

2 rectangular modules of plan dimensions 2.5 x 4.5 m, vertically jointed

- Two storeys building
- Total gross area: 22.5 m^2
- Total height: 5.4 m
- Weight of the complete building (w/ finishing) : 102 kN (**4.53 kN/m^2**)
- Weight of the structural part (w/o finishing): 46 kN (**2.04 kN/m^2**)

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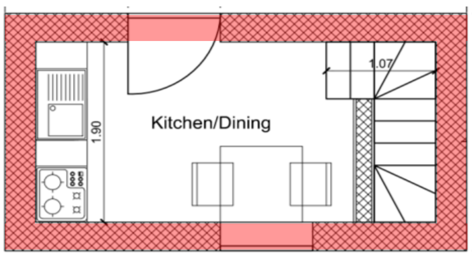
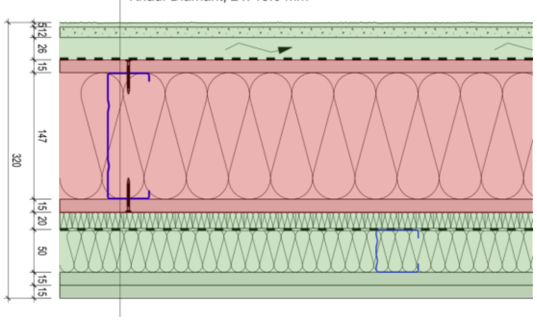
Structural and non-structural building components

Walls



Aquapanel Outdoor plasterboard with render system, 12.5 mm
Knauf slotted hat profile FLV 25/100 with air cavity, 25 mm
Knauf Insulation LDS 0.04 membrane
Knauf Diamant, 1 x 15.0 mm
Structure Cocoon C147/50/1.5 mm, centered at 625 mm
Knauf Insulation mineral wool, FCB 035, 147 mm
Knauf Diamant, 1 x 15.0 mm
Vacuum Insulation Panels, 20 mm
Knauf profile CW50/0.6 mm, centered at 625 mm
Knauf Insulation mineral wool, 50 mm
Knauf Diamant, 2 x 15.0 mm

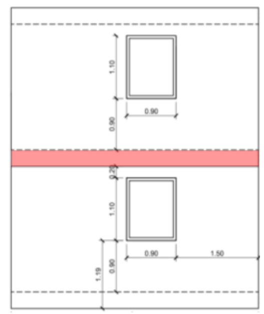


Floor/roof

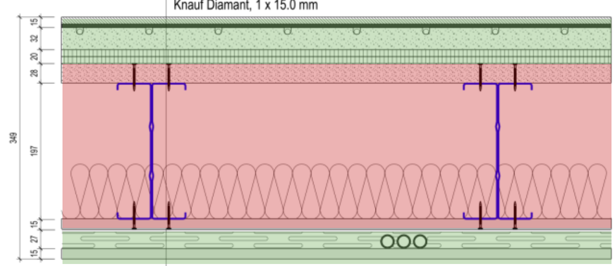
Structural elements

Non-structural elements

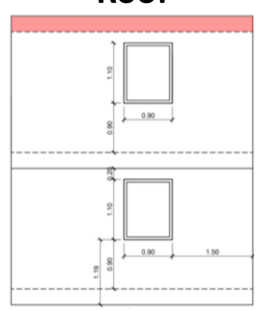
Floor



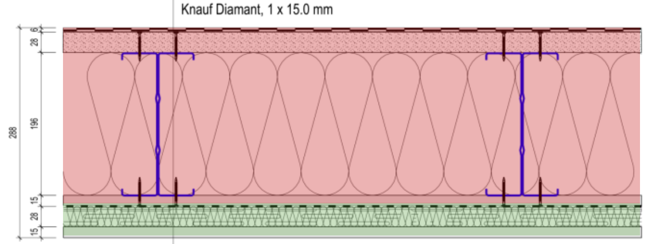
Floor covering
Floor heating / cooling system Knauf GIFAfloor Klima, 32 mm
Impact sound insulation Knauf WF, 2 x 10 mm
Load panels Knauf GIFAfloor, 28 mm
Structure Cocoon DT 2xC197/50/2.0 mm, centered at 500 mm
Knauf Insulation mineral wool, max. 180 mm
Knauf Diamant, 1 x 15.0 mm
Knauf resilient channel 60/27/0.6 mm, 27mm, centered at 500 mm
Knauf Diamant, 1 x 15.0 mm



Roof



Roof sealing film
Knauf GIFAfloor, 28 mm (load panel)
Structure Cocoon DT 2xC197/50/2.0 mm, centered at 500 mm
Knauf Insulation mineral wool, FCB 035, 200 mm
Knauf Insulation vapor barrier LDS 10 Silk
Knauf Diamant, 1 x 15.0 mm
Knauf resilient channel 60/27/0.6 mm, 27mm, centered at 500 mm
Aerogel high performance insulation, 30 mm
Knauf Diamant, 1 x 15.0 mm

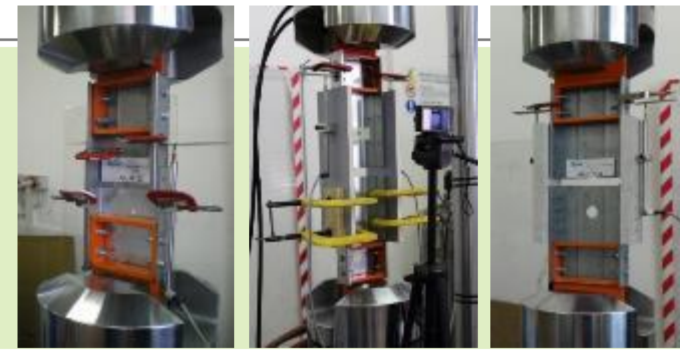


Innovative systems: *Lightweight steel constructions*

Ongoing researches: ELISSA Research Project

General experimental program

Test type		no. tests
MICRO-SCALE **Component (connections) tests**	Panel-to-steel connections for walls	11
	Panel-to-steel connections for floors	7
	Steel-to-steel connections	15
MESO-SCALE **Sub-structure (wall) tests**	In-plane monotonic tests	1
	In-plane quasi-static reversed cyclic tests	3
MACRO-SCALE **Shake table tests on the ELISSA mock-up**	Dynamic identification and earthquake tests	16 + 28 on 1 prototype (w/ and w/o finishing)
	Total no. of tests	***81***



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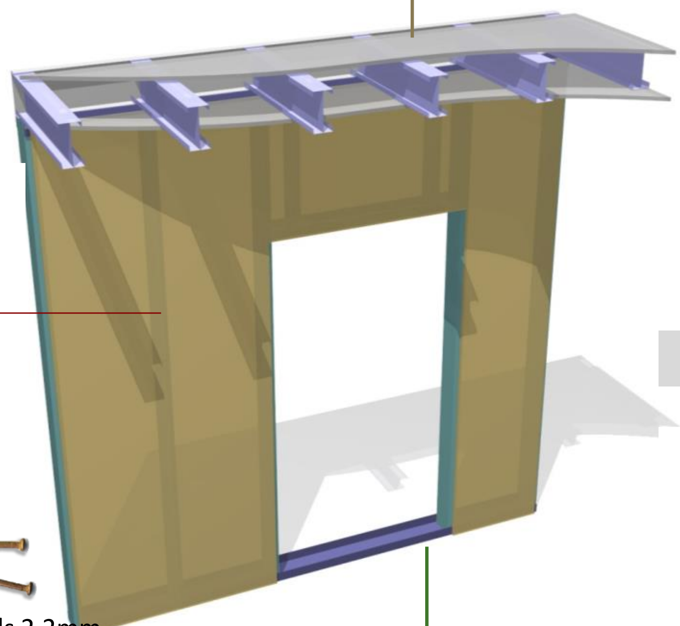
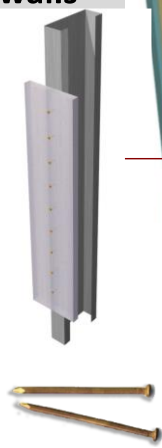
Ongoing researches: ELISSA Research Project

Micro-Scale tests: shear tests on connections



Panel-to-steel connections for walls

Total tests: 11



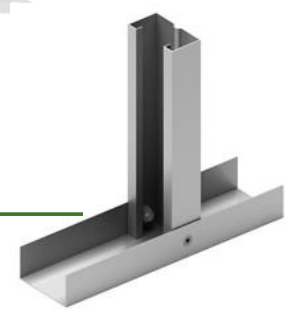
Balistic nails 2,2mm

Panel-to-steel connections for floors

Balistic nails 3,4mm

Total tests: 7

Steel-to-steel connections



Clinching 8mm

Total tests: 15

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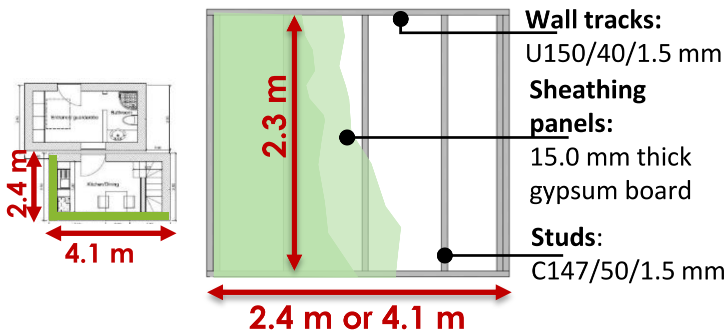
Meso-Scale tests: in-plane monotonic and cyclic tests on sub-structures

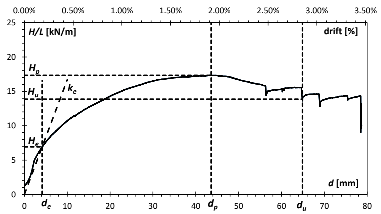
Specimen typologies and test program

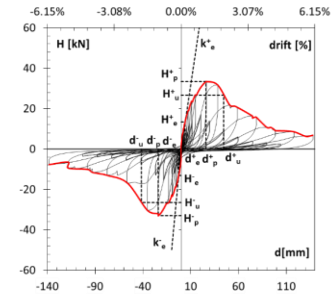


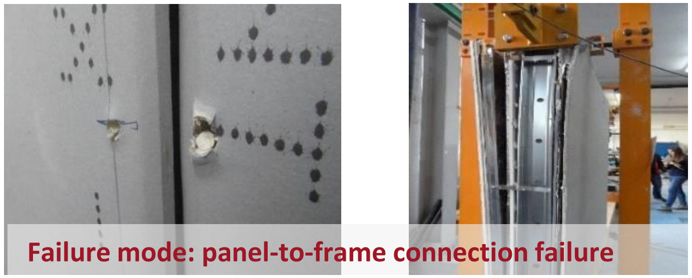
Label	Geometry	Finishing	Load type	No. tests
WS_2400_M	2.4 m x 2.3 m [A]	NO	Monotonic	1
WS_2400_C	2.4 m x 2.3 m [A]	NO	Cyclic	1
WS_4100_C	4.1 m x 2.3 m [B]	NO	Cyclic	1
WF_2400_C	2.4 m x 2.3 m [A]	YES	Cyclic	1

1 monotonic test and 3 cyclic tests

Experimental results







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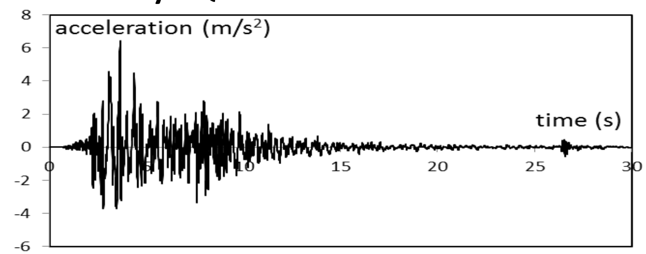
Dynamic earthquake tests - Input: 2009 L'Aquila Earthquake

Mercalli Intensity (effects): 8-9
Richter magnitude (energy): 5.8

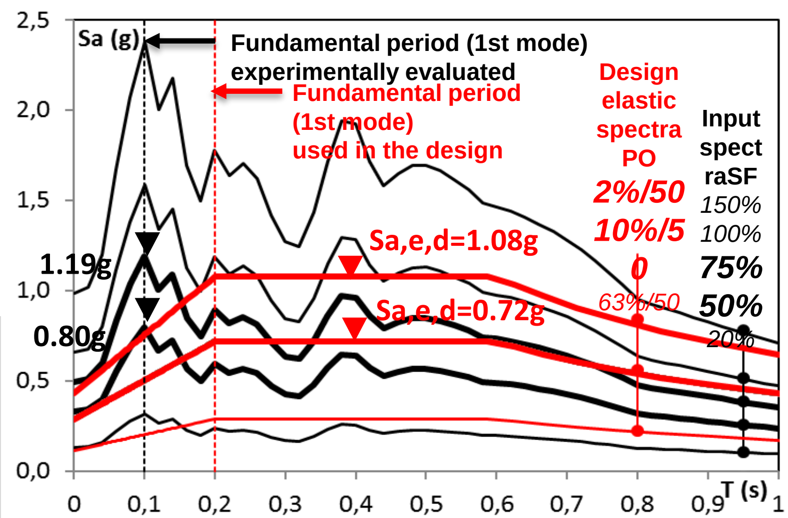
SELECTED GROUND MOTION

Event: *L'Aquila - April 6th, 2009 3:33 a.m.*
Magnitude: Mw= 6.2
Station: *L'Aquila - Valle Aterno - Centro Valle*
Station code: *AQV*
PGA: *6.44 m/s²* (0,66 g)

Input time history AQV-EW



Input spectrum vs. design spectrum (S_a-T format)



SF: Scaling Factor; PO: Probability of occurrence

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Earthquake test on shake table of the ELISSA mock-up

External view	Internal view (2nd floor)
	

Videos recorded during the Earthquake test with scaling factor of ***150%***

SEISMIC DESIGN OF STEEL STRUCTURES:
NEW TRENDS OF RESEARCH AND UPDATES OF EUROCODE 8

Contents

Conclusive remarks

- Nothwithstading some damages occurred in recent earthquakes, **steel structures demonstrated to behave very well under severe earthquakes**. However, **further improvements in codification are necessary** to avoid undesired brittle failures as well as to improve the repairability after moderate and severe sesmic events.
- Eurocode 8 is currently under maintenance and the main aspects concerning their revision process have been shown. **The main criticisms of the seismic design rules for steel structures have been also higlighted**.
- The ongoing activity within **TC250/ SC8 with the support of the ECCS Technical Committee 13 (TC13)** of the revision process of the rules for seismic design of steel structures has been shown.
- Such review represent an **extremely important and delicate phase**, because it will lead to the definition of the future structure of Eurocode 8. However, the work of the PT is still far from being completed.
- The discussion and **the proposals for the next EC8 have been supported on the basis of the outcomes obtained within several EU research projects** as well as among the consolidated and widely acknowledged findings matured in the field.

Thank You For Your Kind Attention

Prof. Raffaele Landolfo

UNINA