

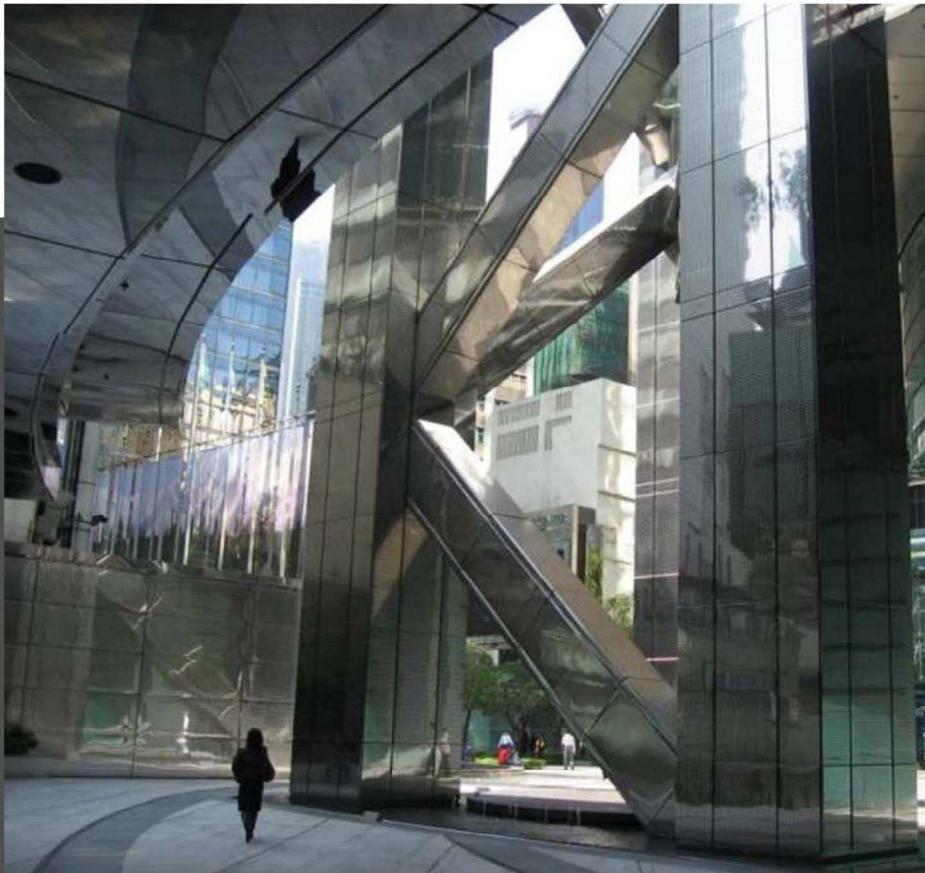
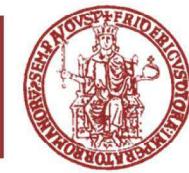


**Costruire oggi in acciaio per il domani**



**Seminario**

**UNINA**  
UNIversità di NApoli “Federico II”



# **COSTRUIRE IN ACCIAIO IN ZONA SISMICA**

LECCE

13 novembre 2017

Campus Ecotekne – Corpo Y, Aula Y-1

***Prof. Raffaele Landolfo***

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**1. Considerazioni introduttive**

**2. La normativa e la ricerca**

**3. What's next?**

**4. Conclusioni**



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**1. Considerazioni introduttive**

**2. La normativa e la ricerca**

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## 1. Considerazioni introduttive



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**EMILIA**, Maggio 2012

M = 5.8



**CENTRO ITALIA**, Agosto 2016 – Gennaio 2017

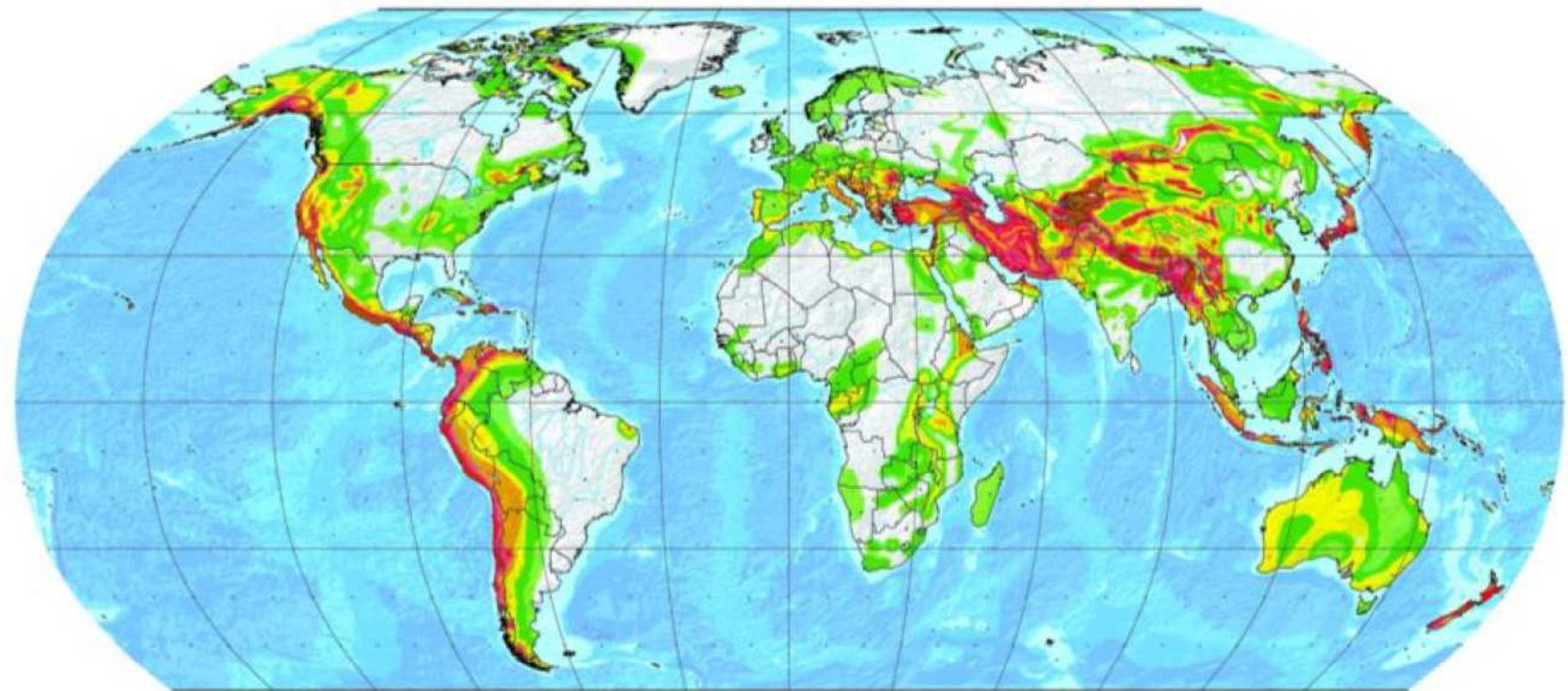
M = 4.5 ÷ 6.0



**L'AQUILA**, Aprile 2009

M = 6.2





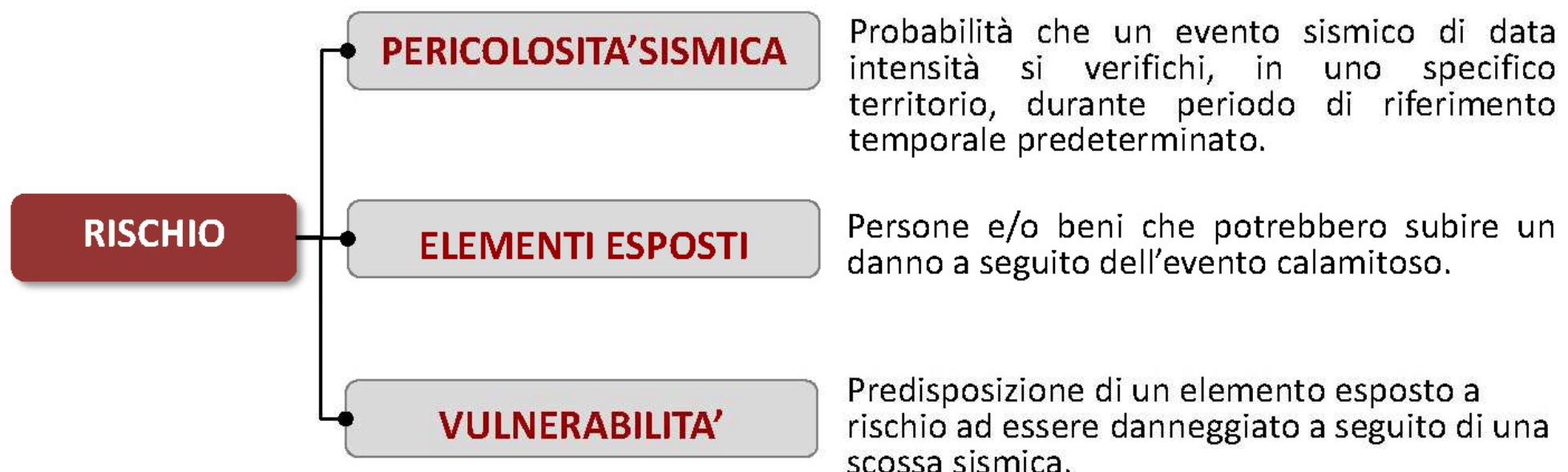
Situazione globale:

- 40% interessa le aree popolate
- Numerose perdite umane ed economiche ogni anno
- Impatto significativo sugli interessi globali

### Il rischio sismico

Il termine “**rischio**” si riferisce alle perdite attese di un dato elemento rispetto ad un possibile danno.

**Il rischio sismico** è la probabilità che un terremoto, in base alla complessità del danno sismico e della vulnerabilità degli elementi a rischio, provochi perdite di vite umane, perdita di edifici o delle loro funzioni.

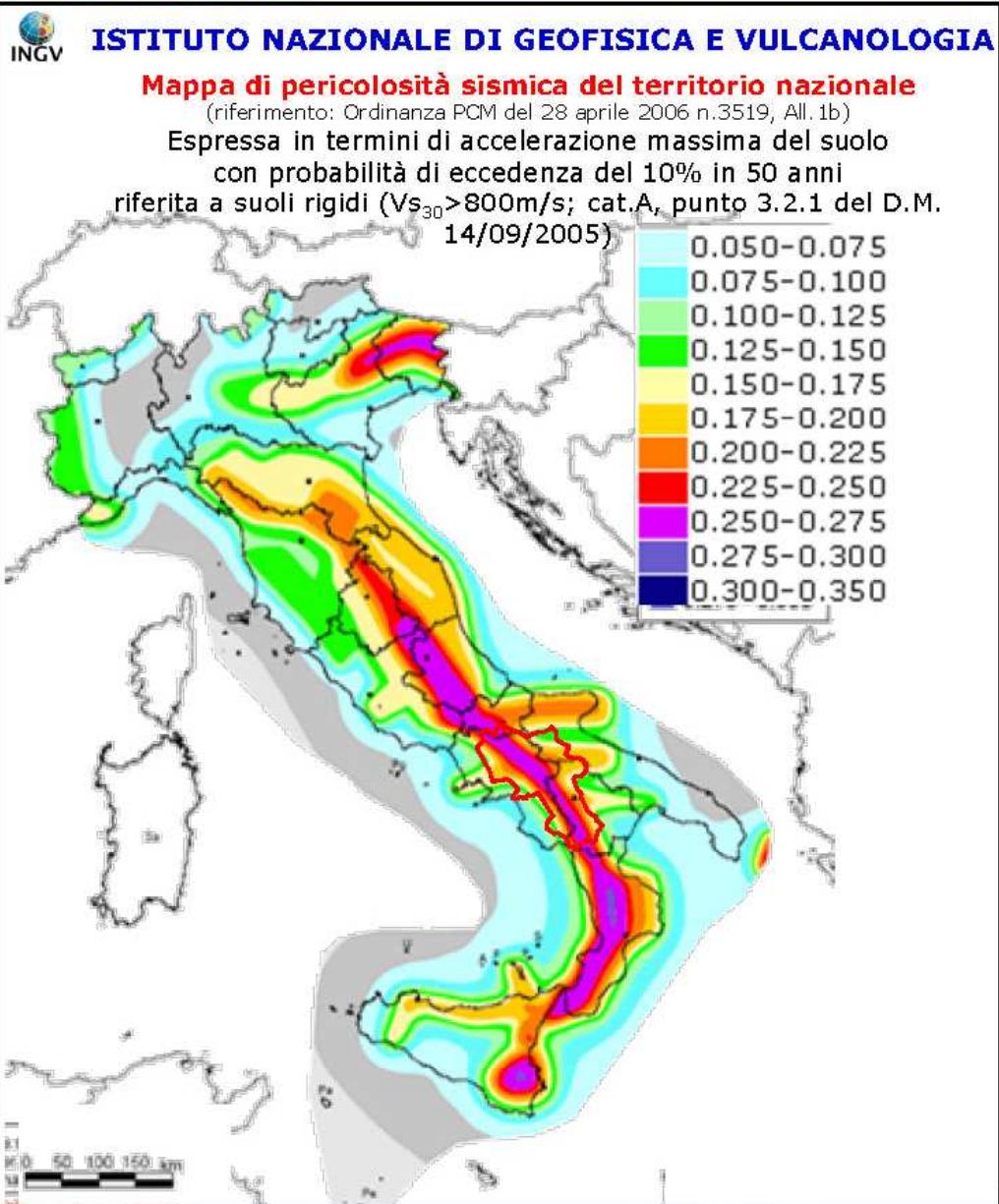


Source: Vulnerability and risk assessment United nation, Disaster management training programme

# Il rischio sismico

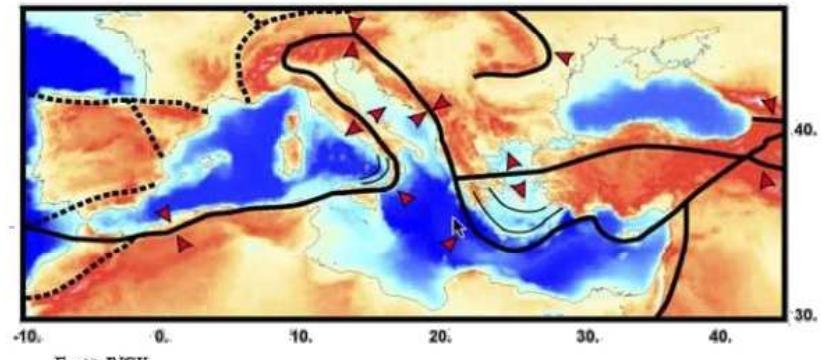


### Il rischio sismico



Costruire in acciaio in zona sismica

La sismicità della penisola italiana è legata alla sua particolare posizione geografica.



Fonte: INGV

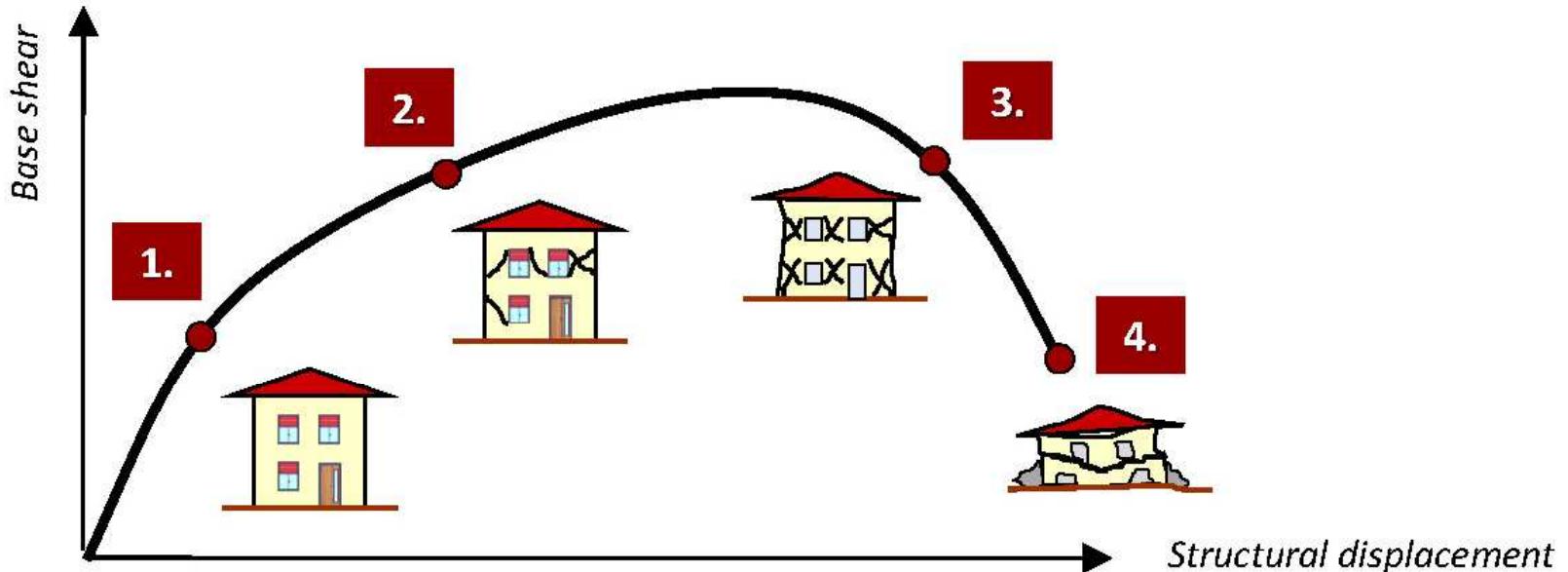
L'Italia è infatti situata nella zona di convergenza tra la **zolla africana** e quella **eurasiatica**.

Dall'andamento di tale frattura si evince perché in Italia, solo la **Sardegna** sia immune dai terremoti.

La sismicità è concentrata nella parte **centro-meridionale** della penisola ed in **alcune aree settentrionali**

Prof. R. Landolfo

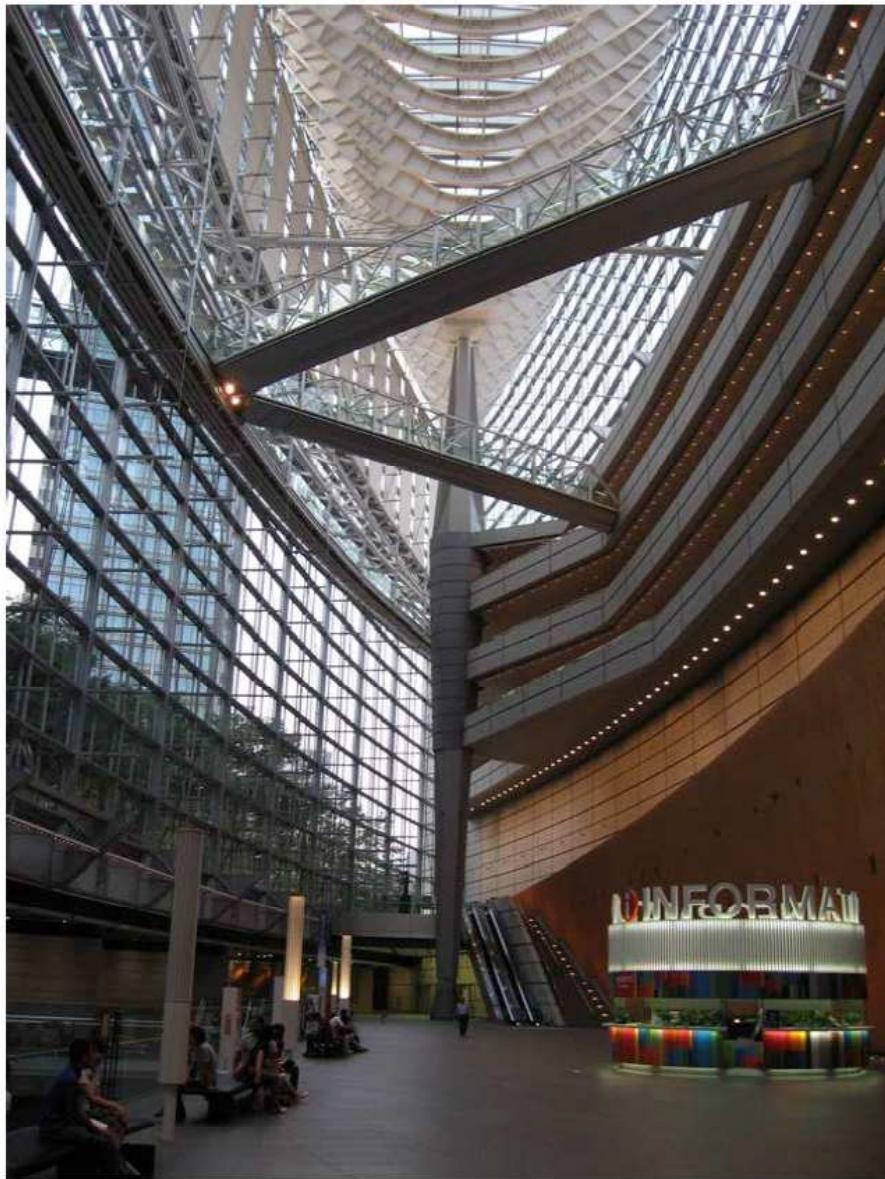
# Performance Based Seismic Design



1. SLO (Stato Limite di Operatività): la costruzione nel suo complesso, non deve subire danni ed interruzioni d'uso significativi
2. SLD (Stato Limite di Danno): la costruzione nel suo complesso, subisce danni tali da non mettere a rischio gli utenti e da non compromettere significativamente la capacità di resistenza e di rigidezza nei confronti delle azioni verticali ed orizzontali
3. SLV (Stato Limite di salvaguardia della Vita): a seguito del terremoto la costruzione subisce significativi danni dei componenti strutturali
4. SLC (Stato Limite di Collasco): a seguito del terremoto la costruzione subisce gravi rotture e crolli dei componenti non strutturali

SLE

SLU



*"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

L'Aquila, April 2009



$M = 6.2$



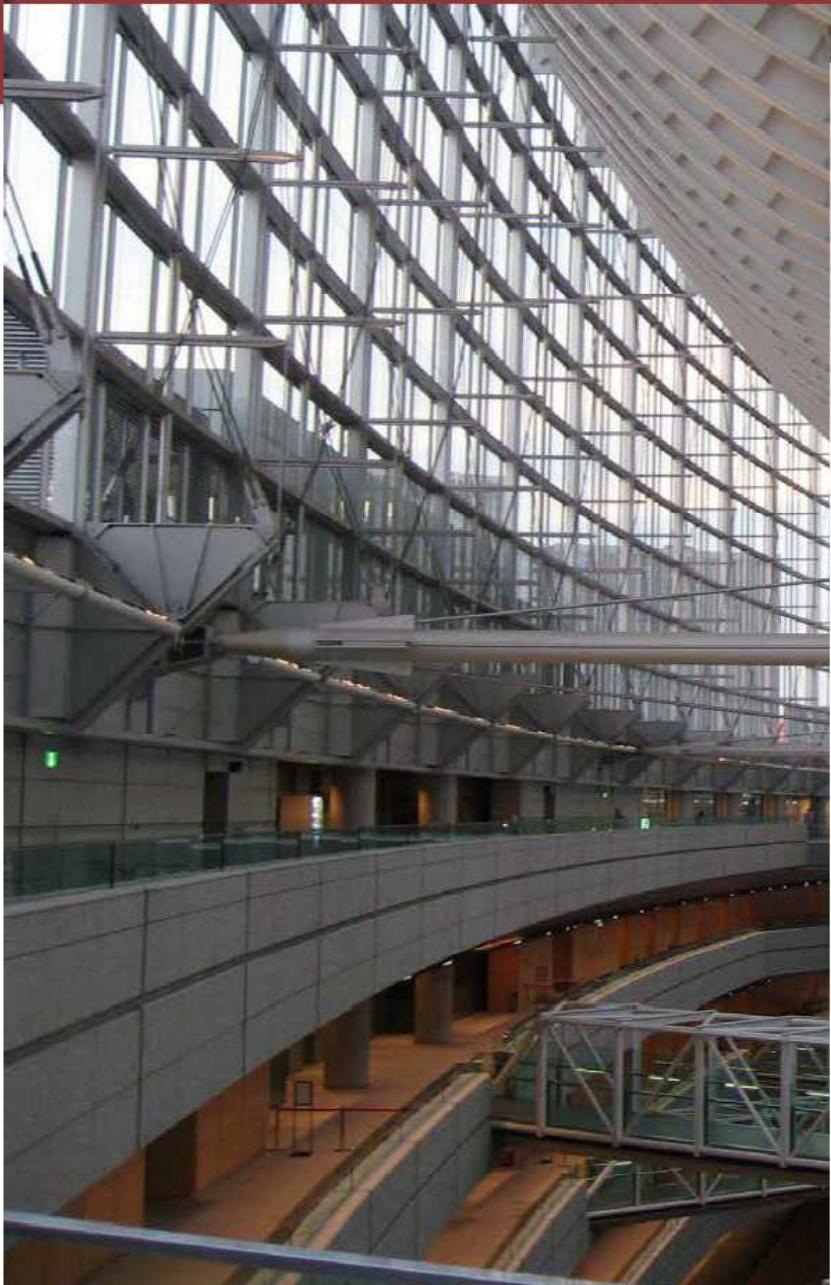
$M = 9.0$



Tokyo, March 2011

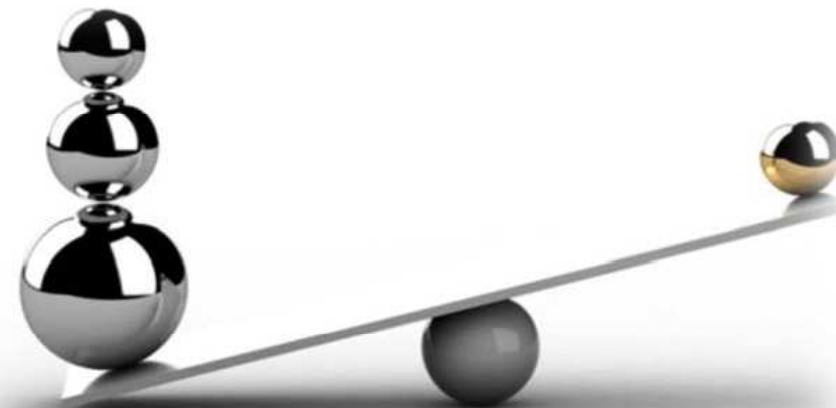
Perchè  
le costruzioni in acciaio  
resistono bene  
al sisma





## Peculiarità delle strutture in acciaio

**Resistenza  
Duttilità  
Leggerezza**



Strutture  
in acciaio

**VS**

Altre  
strutture

## La leggerezza

Chi vincerà ?



NTC



## Tecnologie costruttive

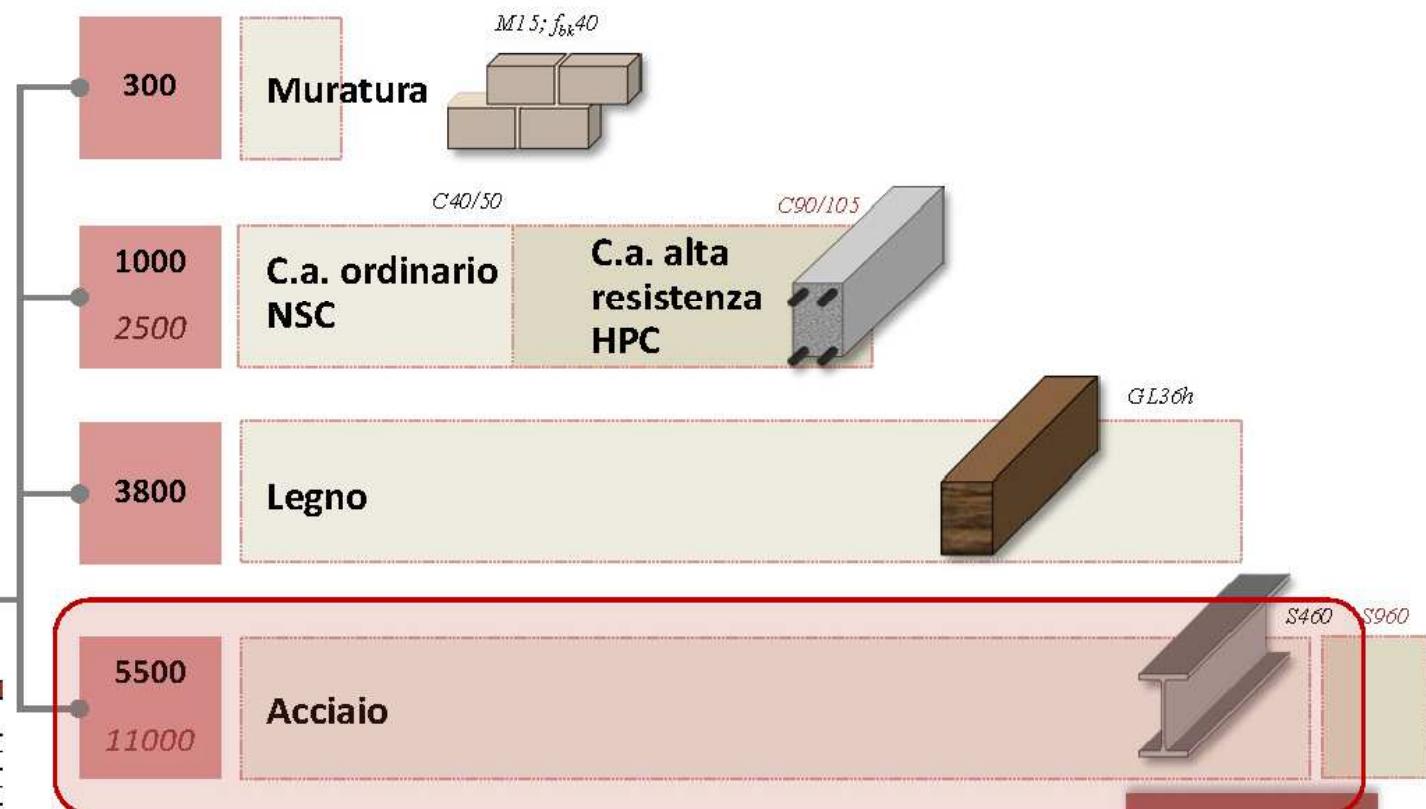
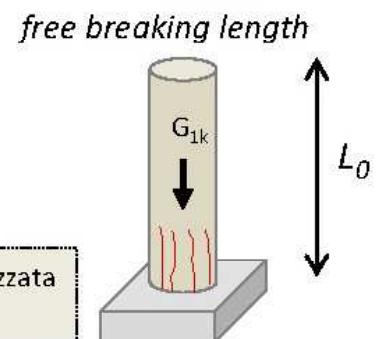
Proprietà meccaniche					
	$f_d$ [kg/m <sup>2</sup> ]	$\gamma_d$	$f_d$ [kg/m <sup>2</sup> ]	$\gamma'$ [kg/m <sup>3</sup> ]	$L_0$ [m]
Muratura M15/20/40	14	2,50	6	18,0	318
cbs NSC C40/50	40	1,50	27	24,0	1111
cbs HPC C80/105	90	1,50	60	24,0	2500
GL36h	31	1,81	17	4,5	3801
S460	460	1,05	438	78,5	5581
S960	960	1,05	914	78,5	11647

## Rendimento o grado di efficienza del materiale (m)

$$L_0 = \frac{f_d}{\gamma}$$

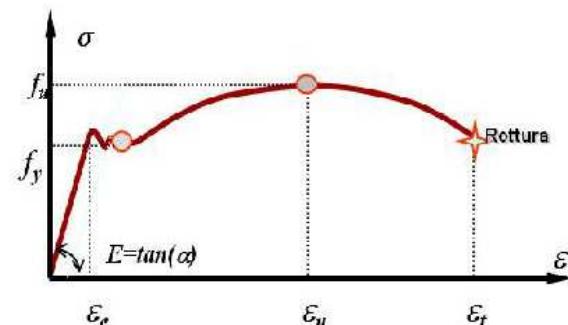
- $f_d$  resistenza di progetto [FL<sup>-2</sup>]
- $\gamma$  peso specifico [FL<sup>-3</sup>]

N.B.  $L_0$  rappresenta l'altezza teorica che può raggiungere una torre realizzata con un preassegnato materiale prima che il suo peso la porti a rottura

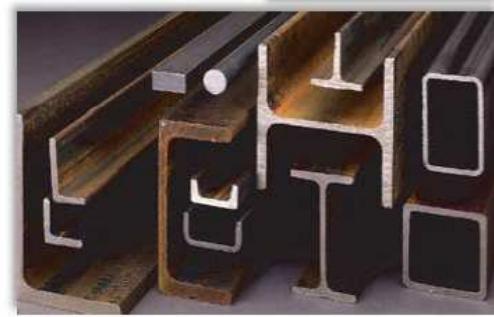
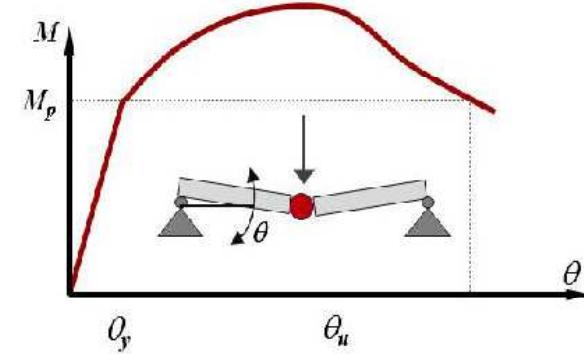


## La duttilità

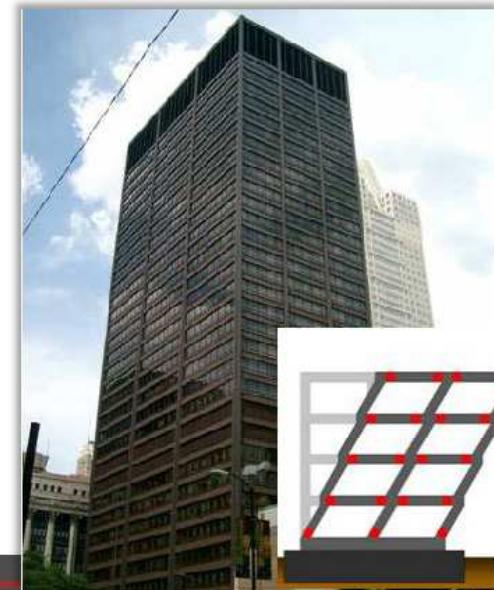
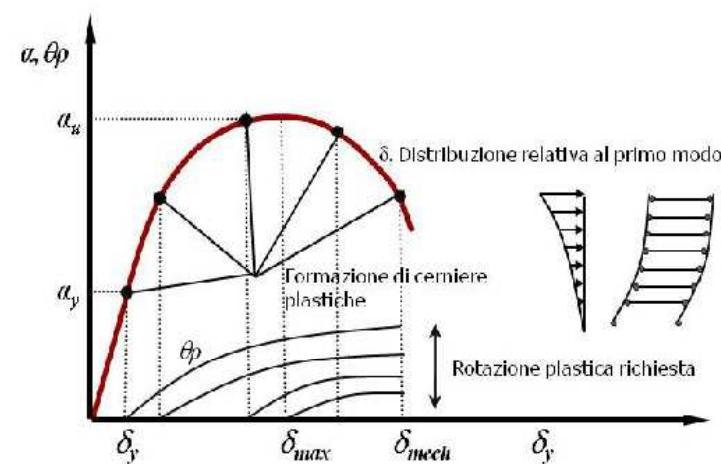
### 1. Duttilità di materiale



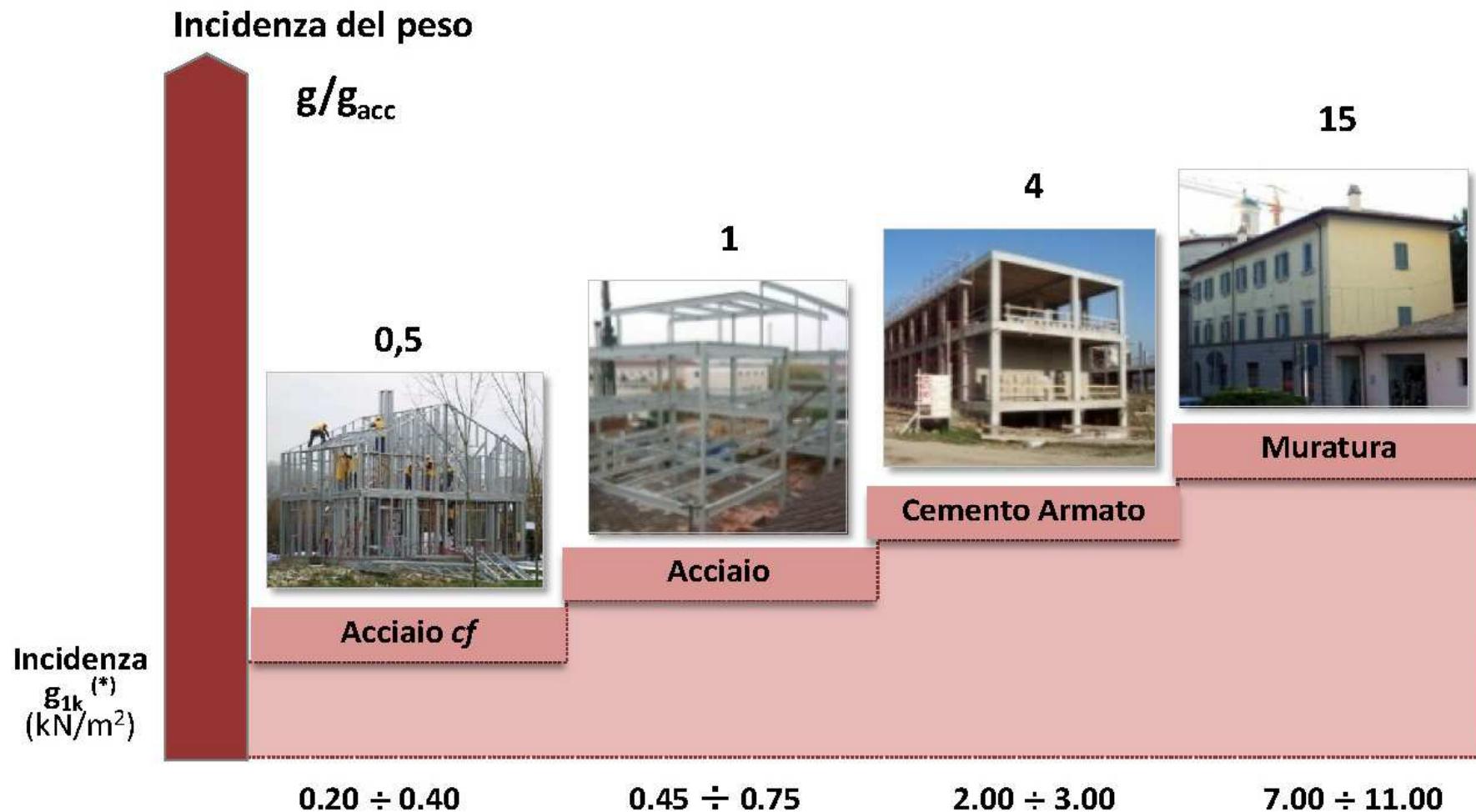
### 2. Duttilità di sezione



### 3. Duttilità di struttura

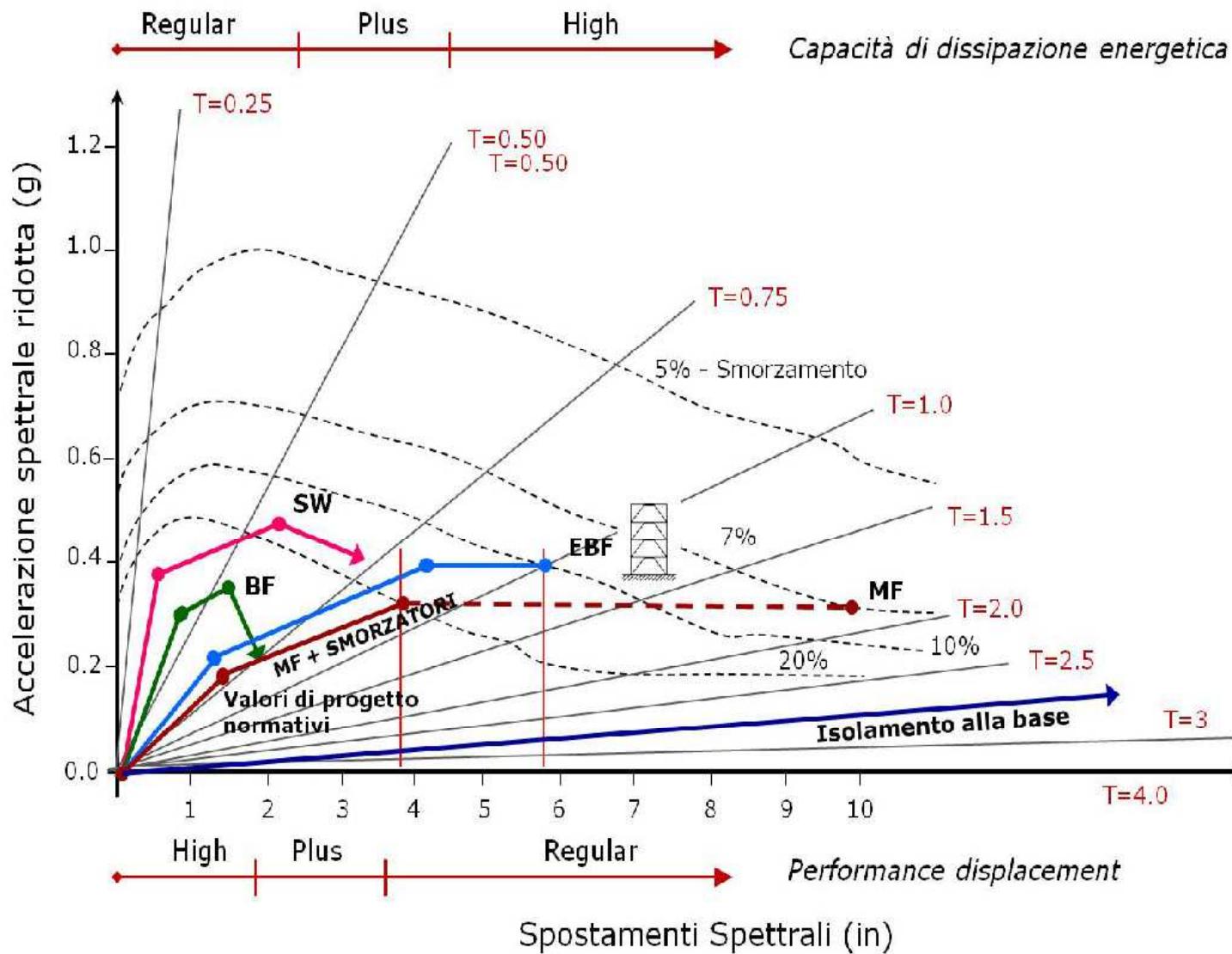


## La leggerezza



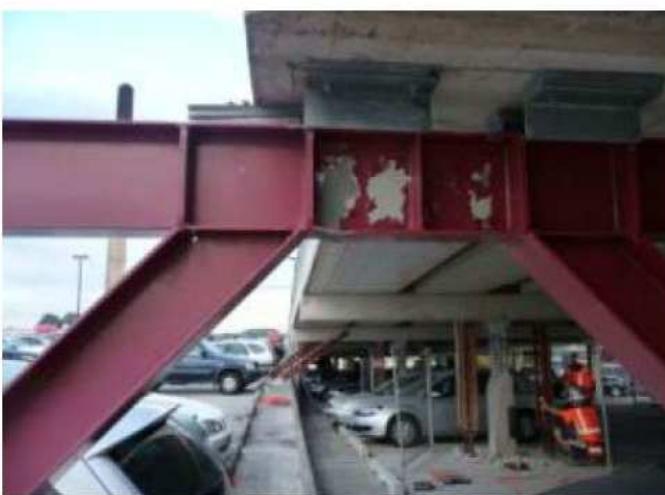
(\*) Confronto sulle incidenza per unità di superficie del **sistema strutturale** ad esclusione dei solai  
(ovvero del solo scheletro)

## La duttilità



### L'importanza di una corretta progettazione

Parking garage on St Asaph St and Antigua St, Christchurch



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**1. Considerazioni introduttive**

**2. La normativa e la ricerca**

**3. What's next?**

**4. Conclusioni**



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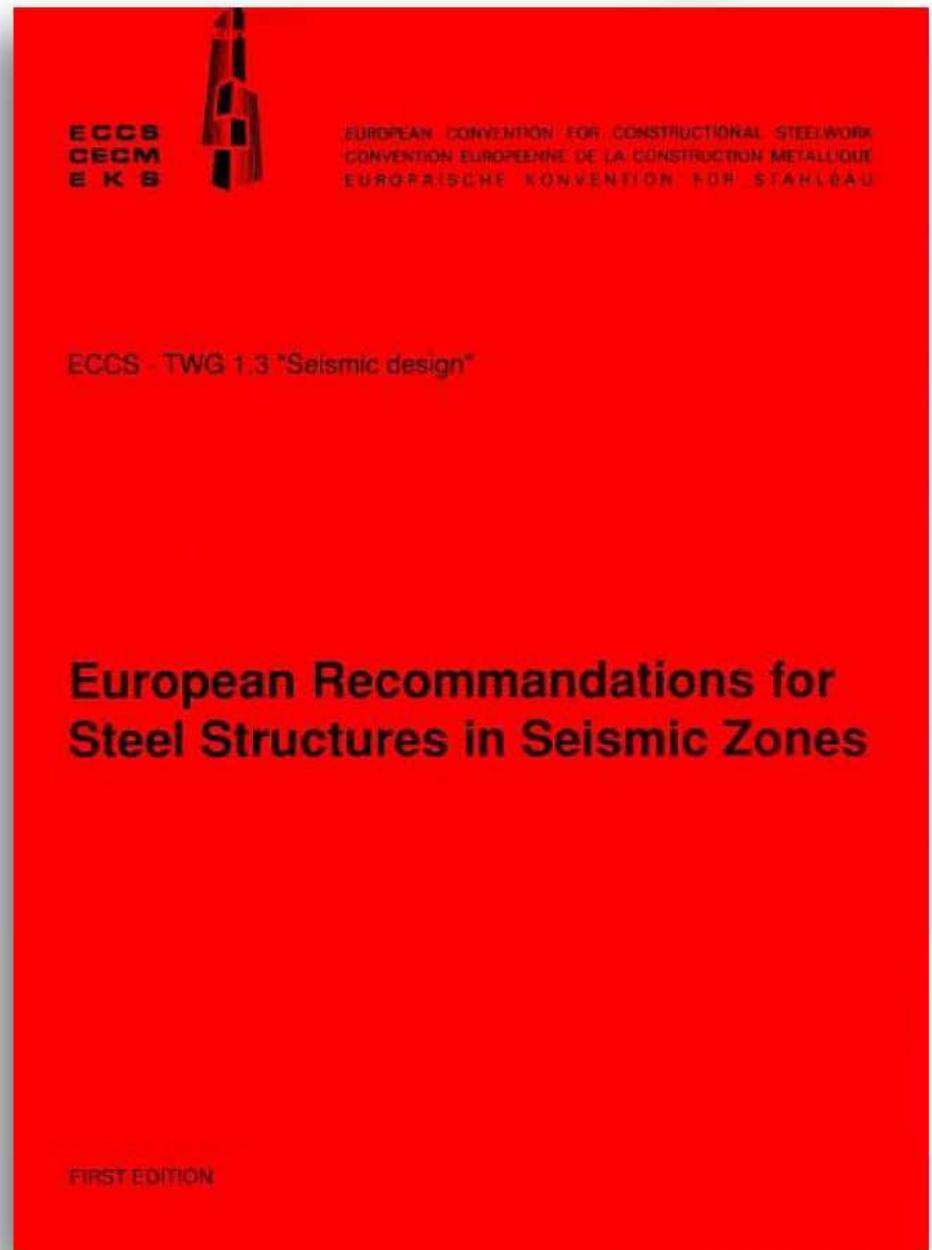
### ECCS code 1991



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

#### **ECCS code 1991**

*European Recommendations  
for Steel Structures in Seismic Zones*



# Eurocode 8 (EN1998-1)



## 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, Erdbebenwirkungen 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.

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

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

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

## D.M. 14-01-2008 'Nuove Norme Tecniche Costruzioni'



IL MINISTRO DELLE INFRASTRUTTURE  
di concerto con  
IL MINISTRO DELL'INTERNO  
e con  
IL CAPO DEL DIPARTIMENTO DELLA PROTEZIONE CIVILE

### 7. PROGETTAZIONE PER AZIONI SISMICHE

...

#### 7.5 COSTRUZIONI D'ACCIAIO

7.5.1 *Caratteristiche dei materiali*

7.5.2 *Tipologie strutturali e fattori di struttura*

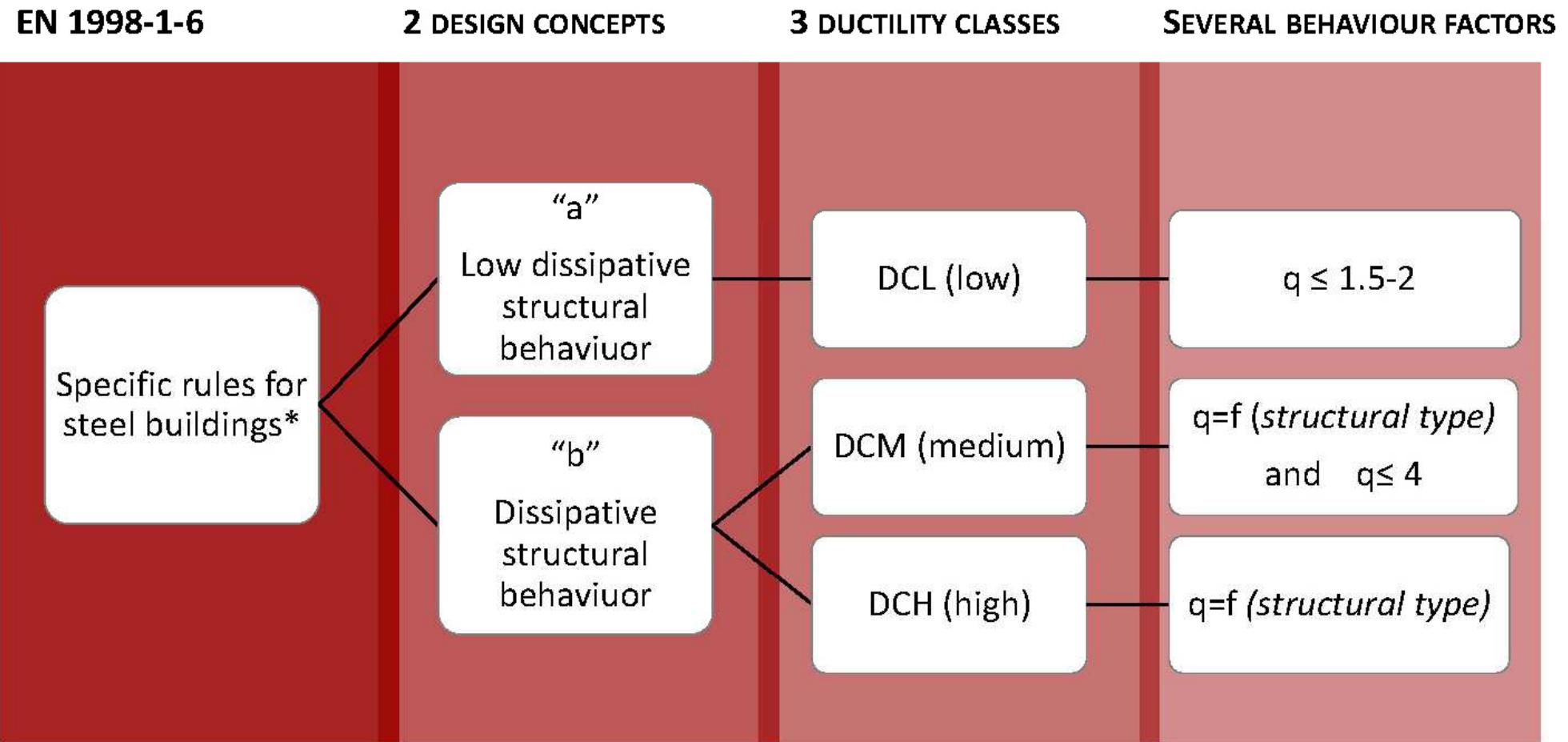
7.5.3 *Regole di progetto generali per elementi strutturali dissipativi*

7.5.4 *Regole di progetto specifiche per strutture intelaiate*

7.5.5 *Regole di progetto specifiche per strutture con controventi concentrici*

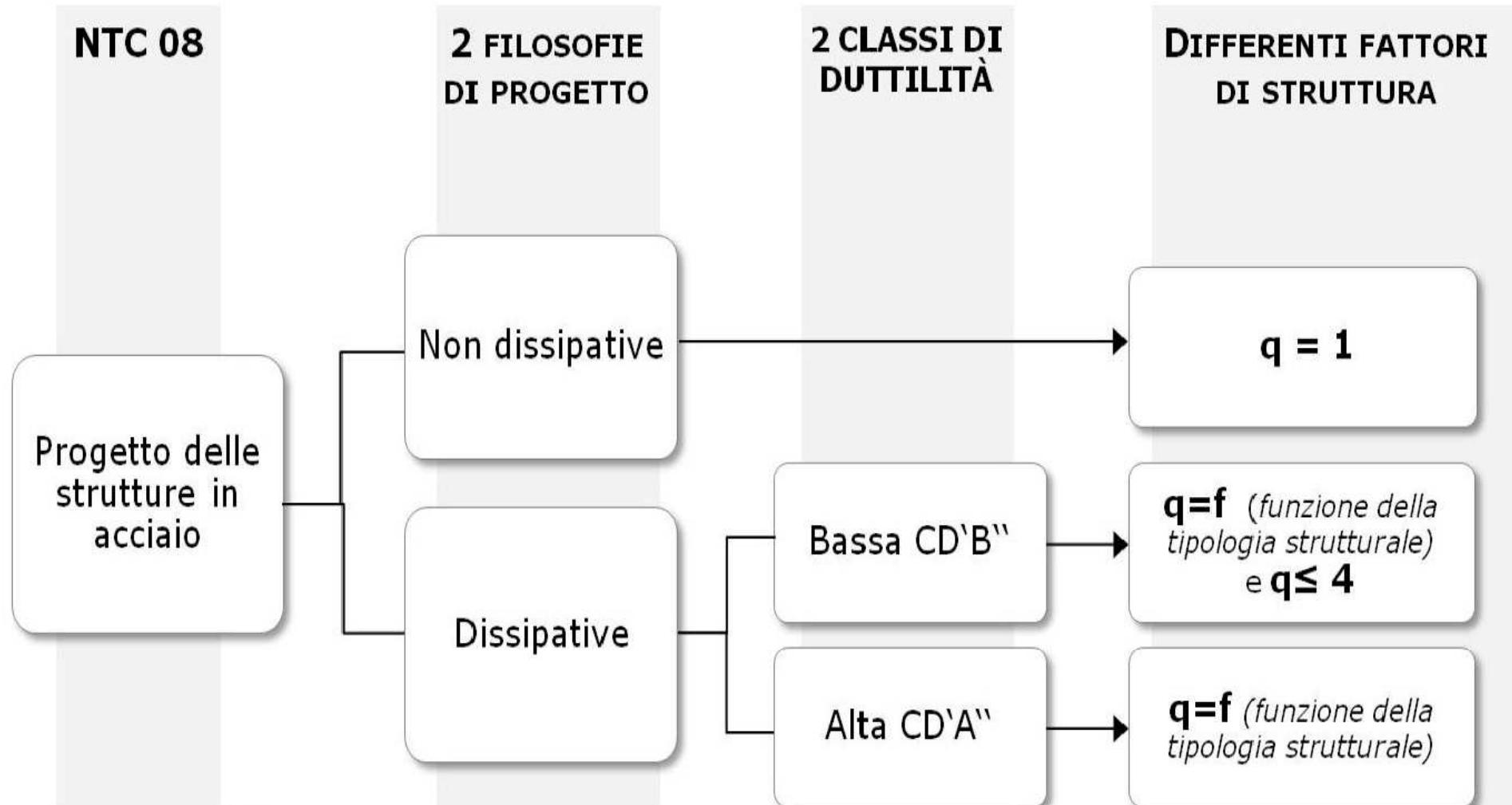
7.5.6 *Regole di progetto specifiche per strutture con controventi eccentrici*

## Design concept and safety verifications



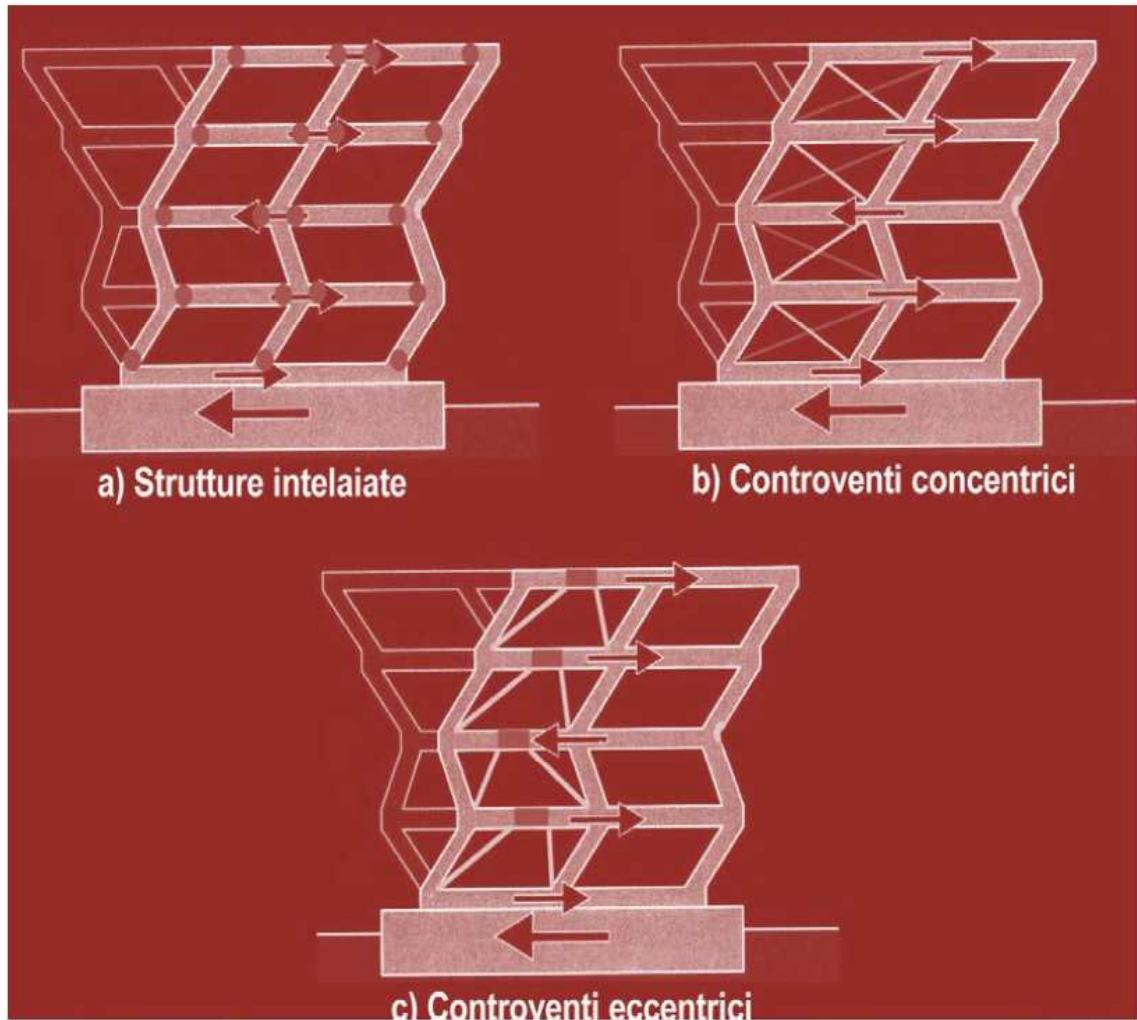
\* For the design of non dissipative steel structures see: EN1993

## Fattori di struttura

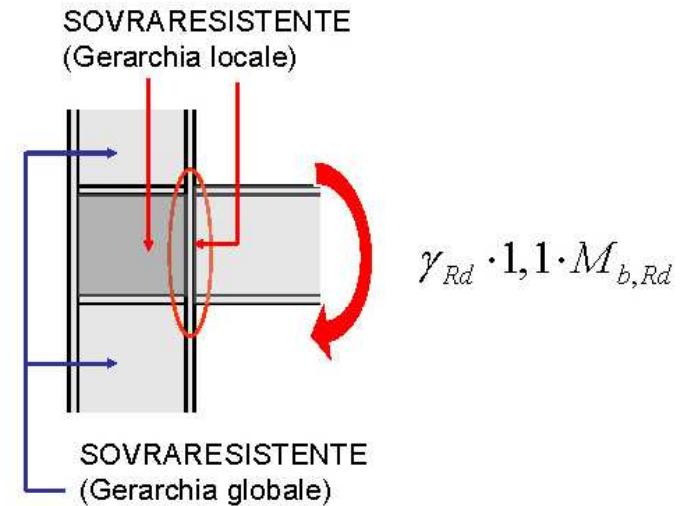
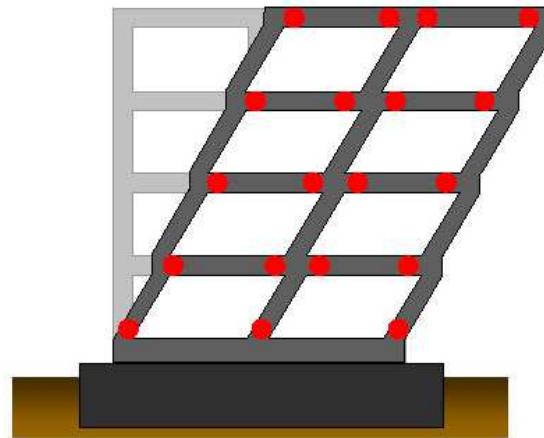
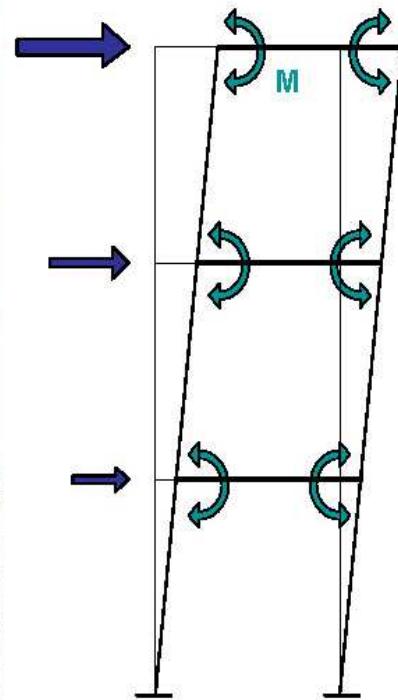


### Il Capacity Design

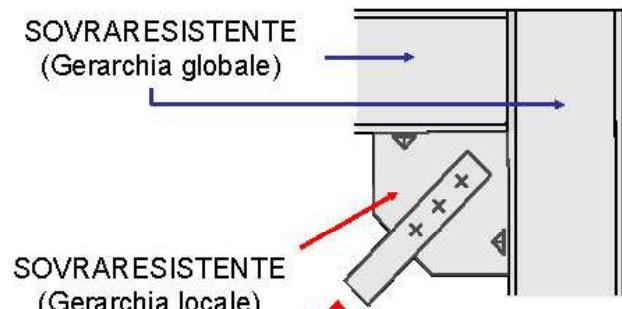
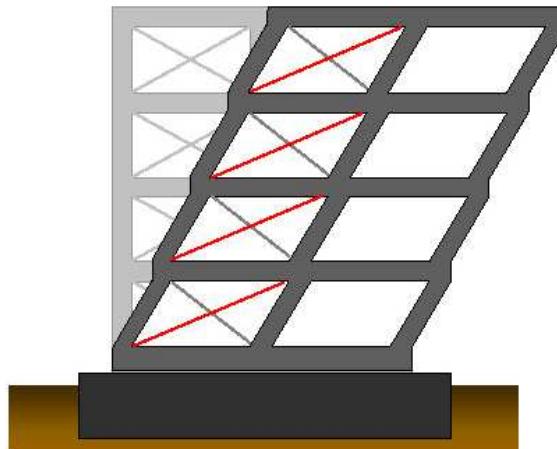
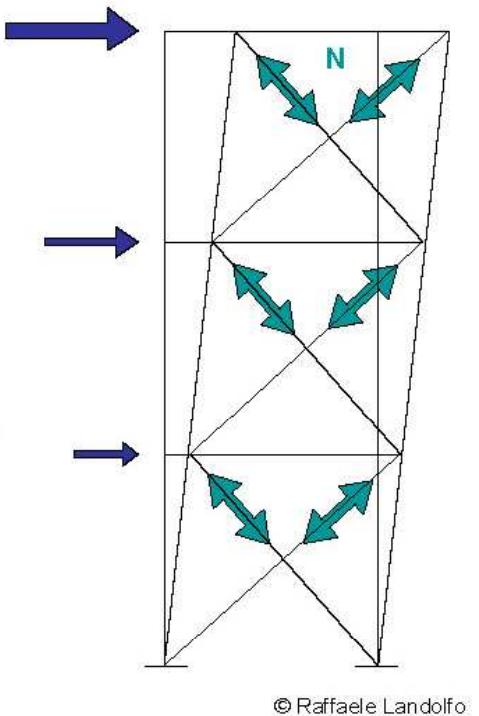
- In un approccio alle forze, si controlla il meccanismo di collasso distribuendo in modo opportuno le resistenze
- Le zone dissipative (zone critiche) fungono da fusibili favorendo la formazione di meccanismi di collasso globali
- Le parti non dissipative ed i collegamenti delle parti dissipative al resto della struttura devono possedere sufficienti sovraresistenze



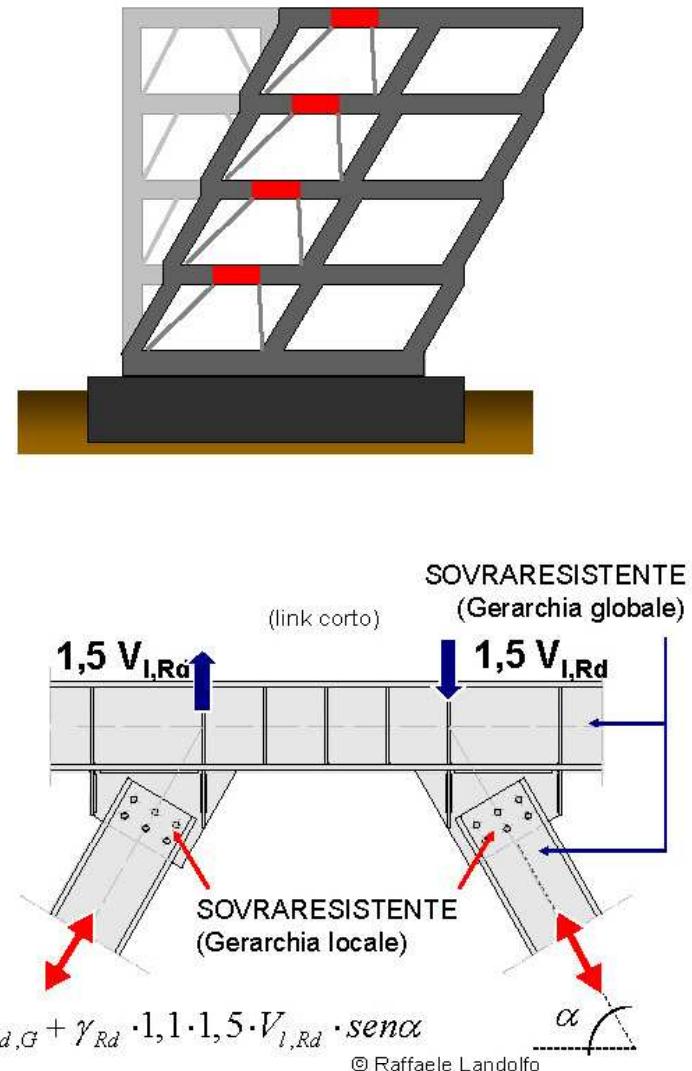
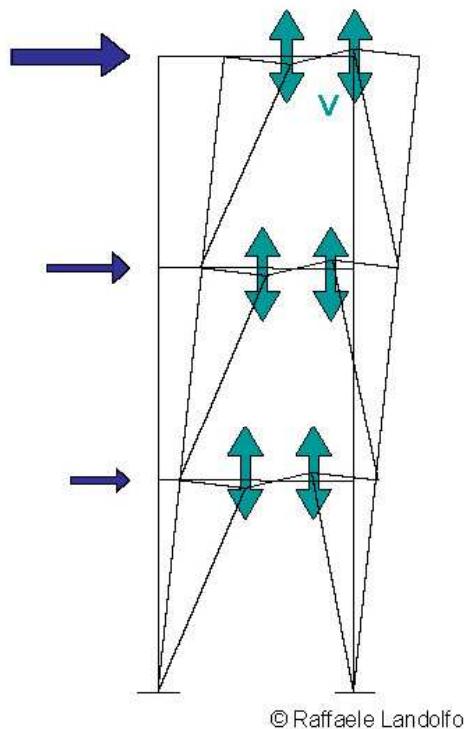
## Moment Resisting Frame (MRF)



## Concentric Braced Frames (CBF)

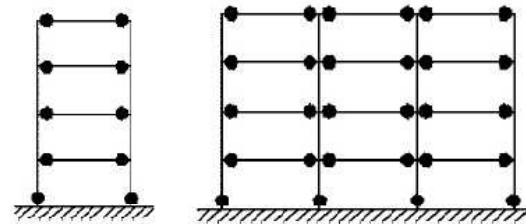


## Eccentric Braced Frames (EBF)



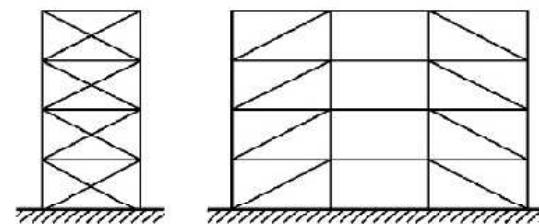
## Tipologie strutturali e fattori di struttura

Moment Resisting Frames (MRF)



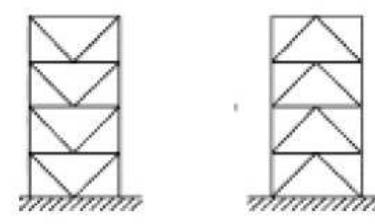
Even combined with isolated concrete infills

Diagonal Braced Frames (CBF)



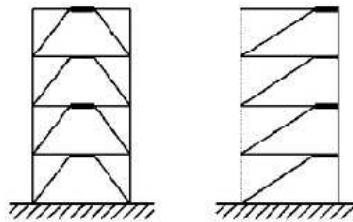
$DCM \rightarrow q = 4$ ;  $DCH \rightarrow q = 4$

V Bracing (CBF)



$DCM \rightarrow q = 2$ ;  $DCH \rightarrow q = 2.5$

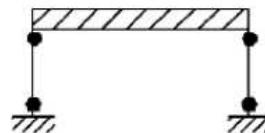
Eccentric Braced Frames (EBF)



$DCM \rightarrow q = 4$

$DCH \rightarrow q = 5 \cdot \alpha_u / \alpha_1$

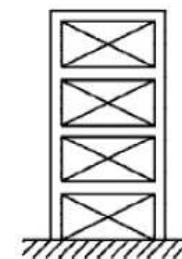
Inverted pendulum



$DCM \rightarrow q = 2$

$DCH \rightarrow q = 2 \cdot \alpha_u / \alpha_1$

MRF + CBF



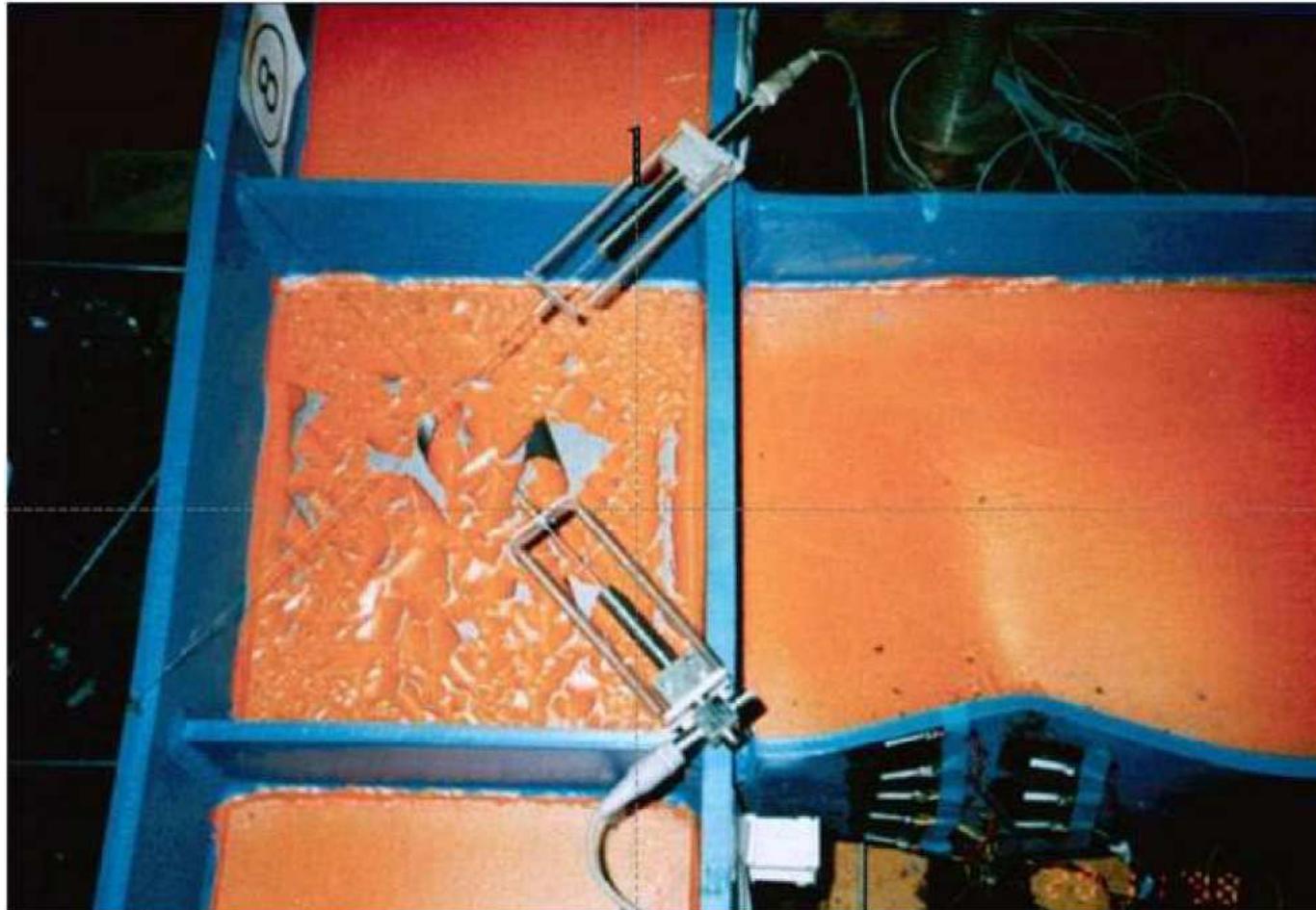
MRF+UNCONNECTED  
CONCRETE OR MASONRY  
INFILLS, IN  
CONTACT WITH THE FRAME

$DCM \rightarrow q = 4$

$DCH \rightarrow q = 4 \cdot \alpha_u / \alpha_1$

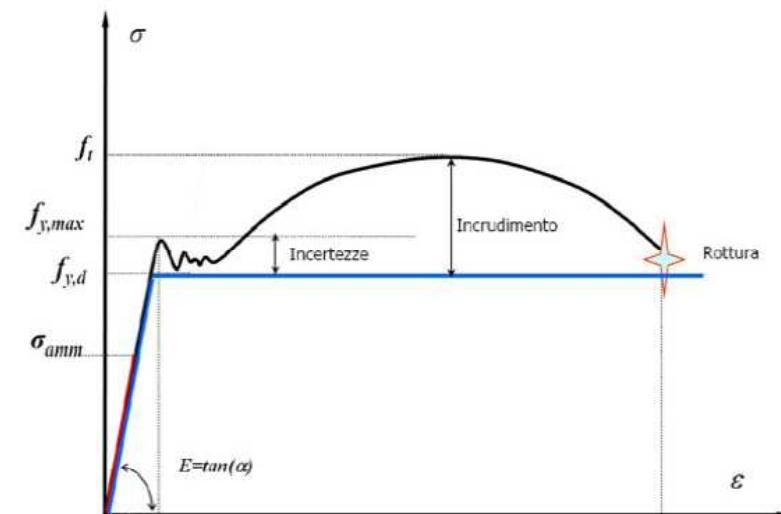
$DCM = DCH \rightarrow q = 2$

## La gerarchia delle resistenze

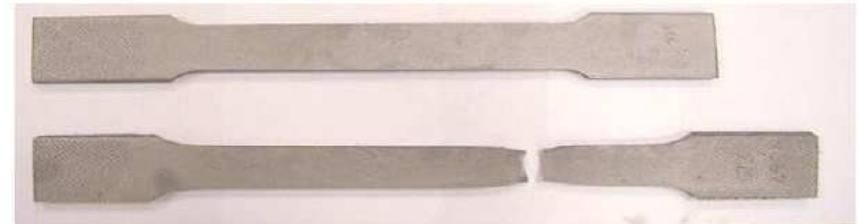


$$\gamma_{Rd} \cdot s \cdot M_{pl}$$


### Il materiale



Legame costitutivo e comportamento meccanico dell'acciaio  
Raffaele Lando



## Il materiale



### NTC 11.3.4.9

(Acciaio per zone dissipative)

$$\left\{ \begin{array}{l} f_{y,\max} \leq 1.2 \cdot f_{yk} \\ \frac{f_{tk}}{f_{yk}} \geq 1.20 \end{array} \right.$$

### EN 10025-2:2005

(Tensione di snervamento nominale)

alpha- numeric	numerical	≤16
S235JR	1.0038	235
S235J0	1.0114	235
S235J2	1.0117	235
S275JR	1.0044	275
S275J0	1.0143	275
S275J2	1.0145	275
S355JR	1.0045	355
S355L0	1.0553	355
S355J2	1.0577	355
S355K2	1.0596	355

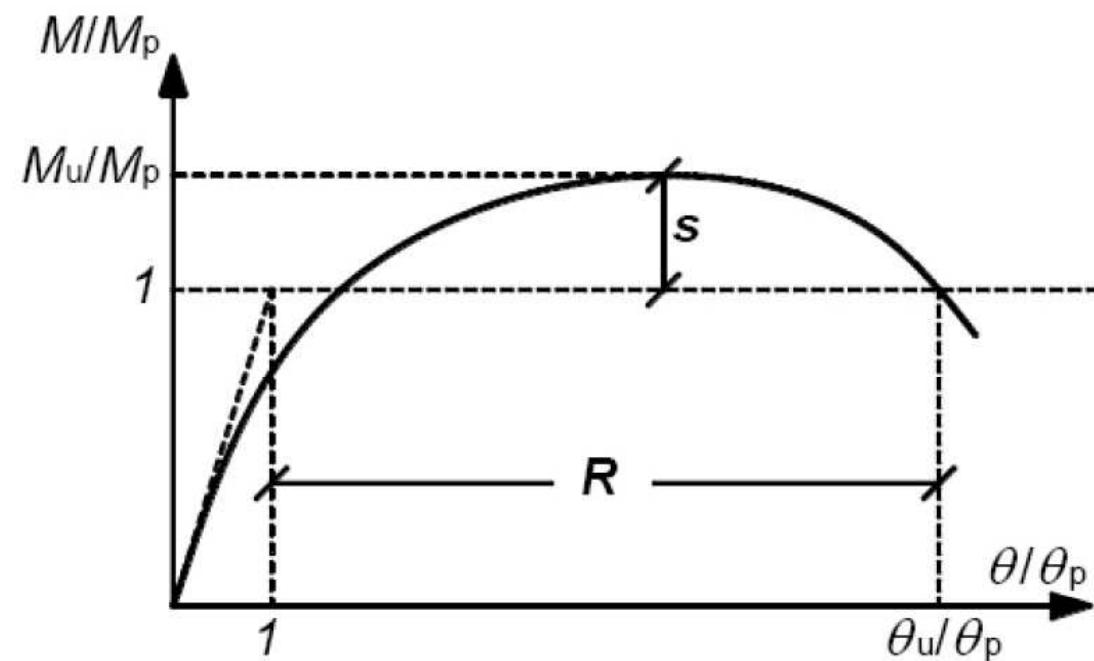
### Eurocode 8

$$\left\{ \begin{array}{l} f_{y,\max} \leq 1.1 \cdot \gamma_{Ov} \cdot f_y \\ \gamma_{Ov} = 1.25 \end{array} \right.$$

### NTC 7.5.1

Acciaio	$\gamma_{Rd} = \frac{f_{y,m}}{f_{yk}}$
S 235	1,20
S 275	1,15
S 355	1,10
S 420	1,10
S 460	1,10

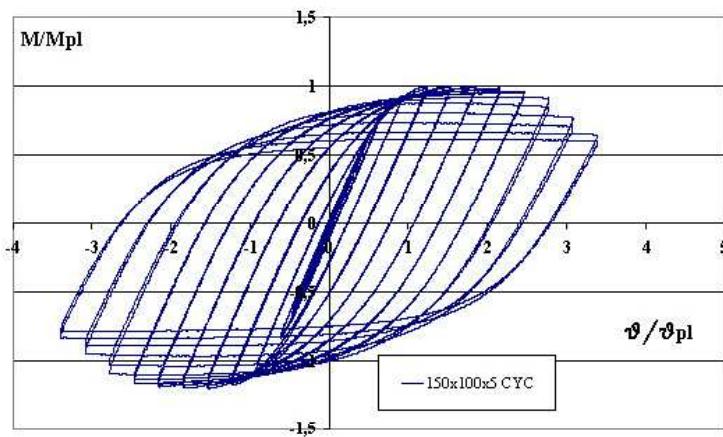
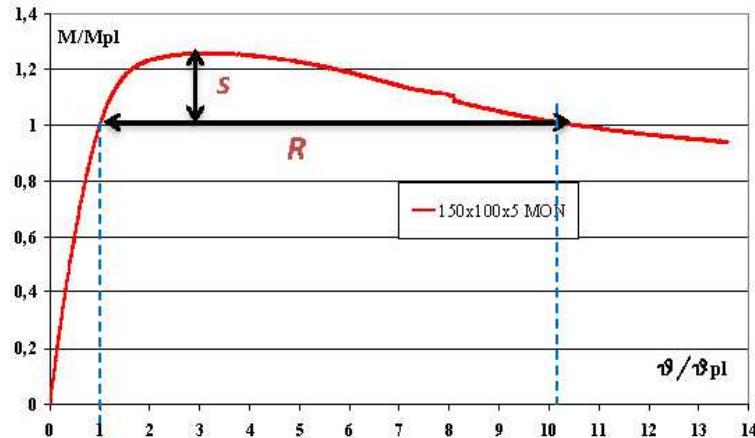
## Le membrature



## Le membrature

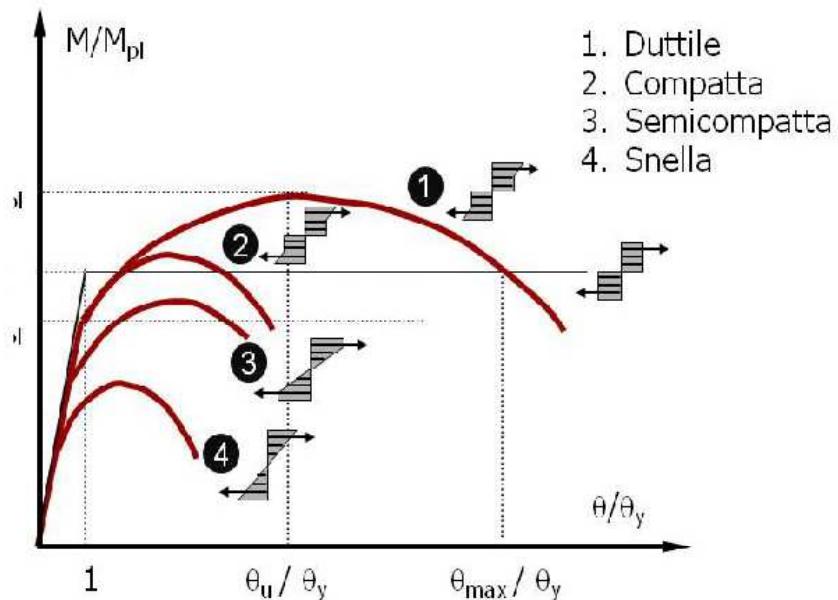
### NTC 11.3.4.9

(Acciaio per zone dissipative)



Profilo  $150 \times 100 \times 5$

## La classificazione delle sezioni



Classe di duttilità	Valore di riferimento del fattore di struttura $q_0$	Classe di sezione trasversale richiesta
CD"B"	$2 < q_0 \leq 4$	Classe 1 o 2
CD"A"	$q_0 > 4$	Classe 1

## Le membrature

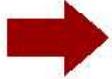


### EC3 cross section classification: limits of validity for seismic application

#### Rotation Capacity

- Ductile or plastic sections:  $M_{max} > M_{pl}$   $R = ?$   
**The minimum values are not clearly defined**
- Compact:  $M_{max} > M_{pl}$
- 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

## Le membrature



### New empirical formulation for S

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

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

- $\lambda_f$  the flange slenderness
- $\lambda_w$  the web slenderness
- $b_f$  the flange width,  $h$  the beam depth
- $L$  the shear length factors

	$C_1$	$C_2$	$C_3$	$C_4$	$C_5$	$C_6$
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

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

## Le membrature



### New empirical formulation for R

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

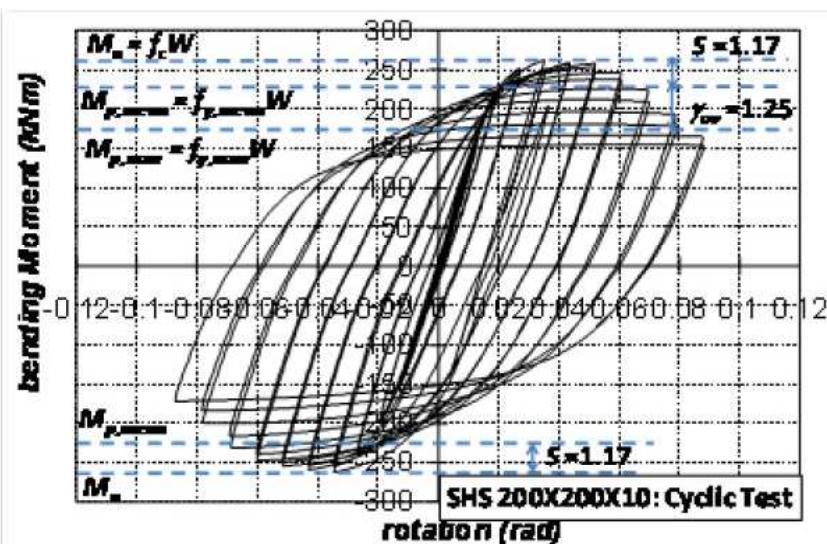
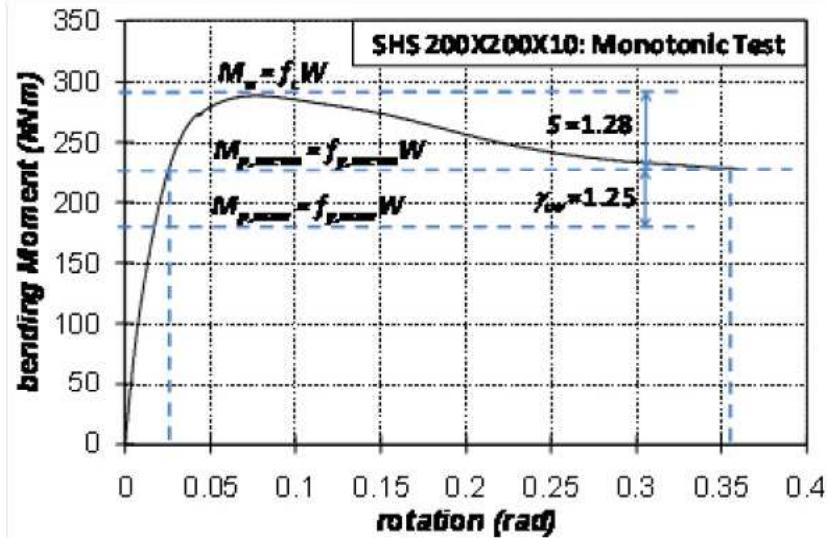
$$R = C_1 + C_2 \frac{1}{\lambda_f^2} + C_3 \frac{1}{\lambda_w^2} + C_4 \frac{b_f h}{L^2} + C_5 \frac{b_f t_f}{h L} + C_6 \frac{A_f}{A_{TOT}} + C_7 \frac{L_m}{L} + C_8 s$$

	$C_1$	$C_2$	$C_3$	$C_4$	$C_5$	$C_6$	$C_7$	$C_8$
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

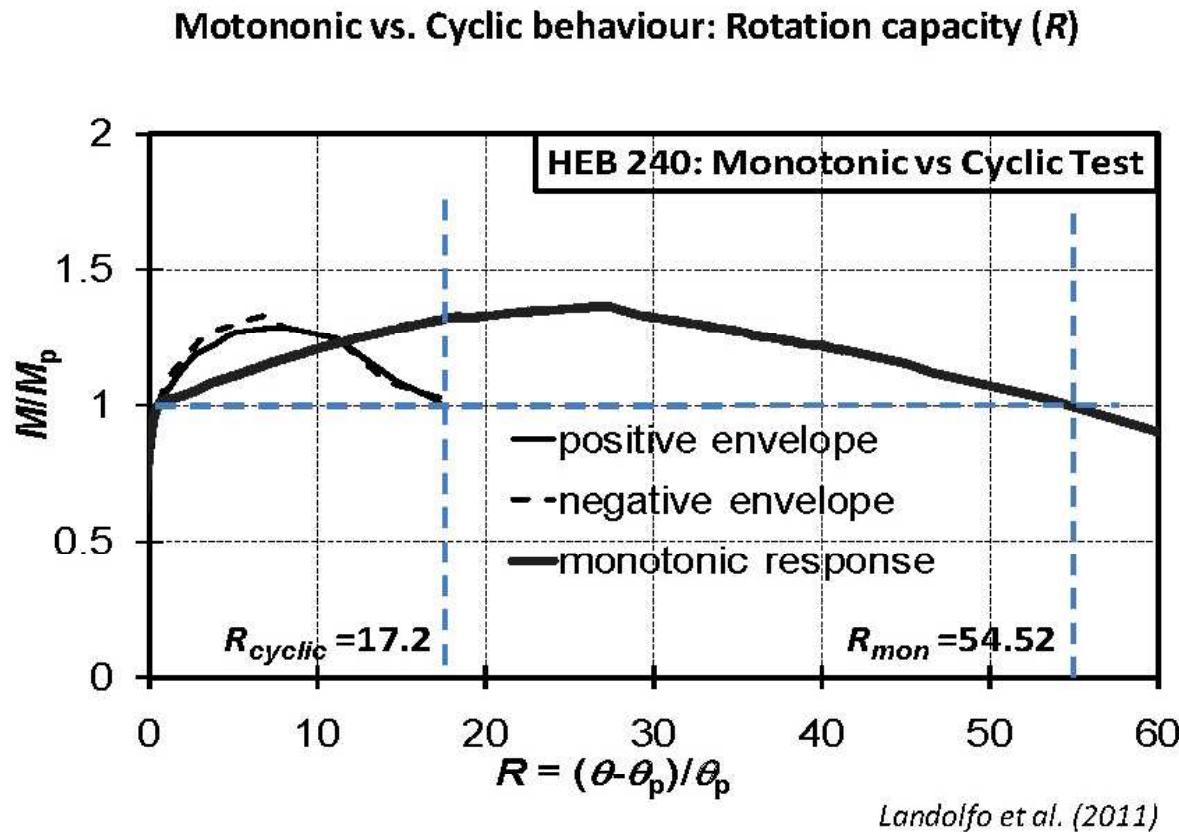
- $\lambda_f$  the flange slenderness
- $\lambda_w$  the web slenderness
- $b_f$  the flange width
- $h$  the beam depth
- $t_f$  the flange thickness
- $L$  the shear length factors
- $A_f$  area of the flange
- $A_{TOT}$  the total cross section area
- $L_m$  the length of the plastic hinge

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

## Le membrature



## Le membrature



Hence, in case of seismic application there is the need to classify steel beams under cyclic loading

## I collegamenti



### 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.



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

## Il collegamento “non dissipativo”



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

The hardening factor should be related to cross section classification!!

The hardening factor is assumed constant

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

$R_d$  is the resistance of the connection in accordance with EN 1993;

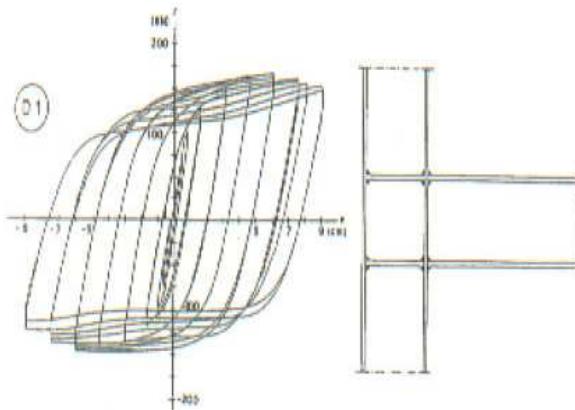
$R_{fy}$  is the plastic resistance of the connected dissipative member based on the design yield stress of the material as defined in EN 1993.

$\gamma_{ov}$  is the overstrength factor (see 6.1.3(2) and 6.2).

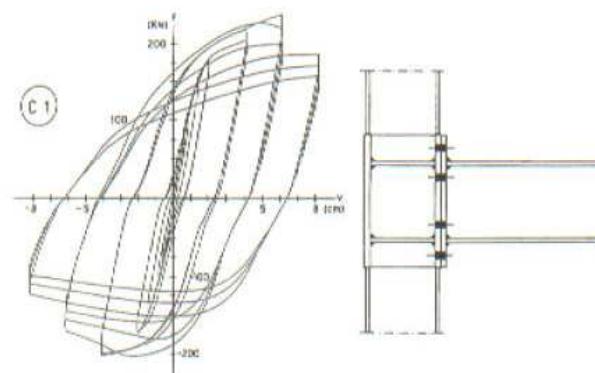
## I collegamenti “dissipativi”

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

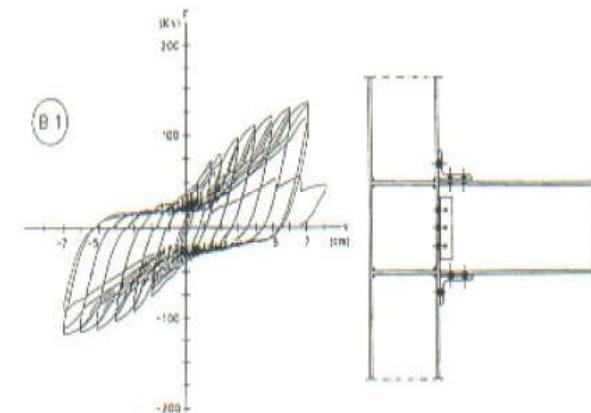
Joint cyclic rotation capacity should be at least **0.035 rad** in case of DCH or **0.025 rad** in case of DCM



*Welded joint*

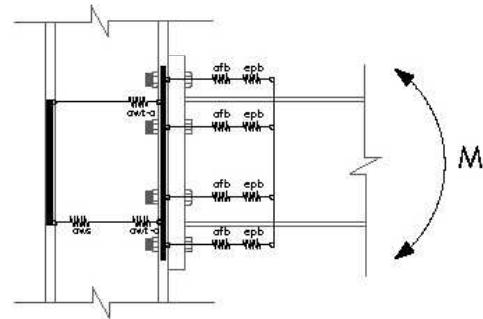
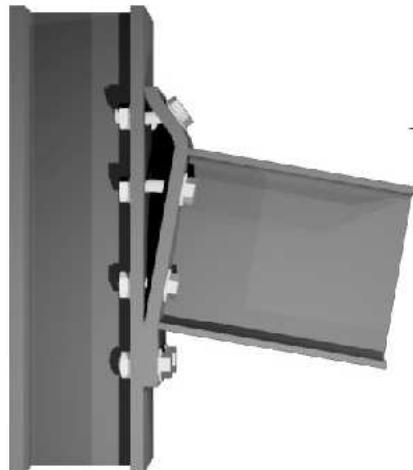


*End plate joint*

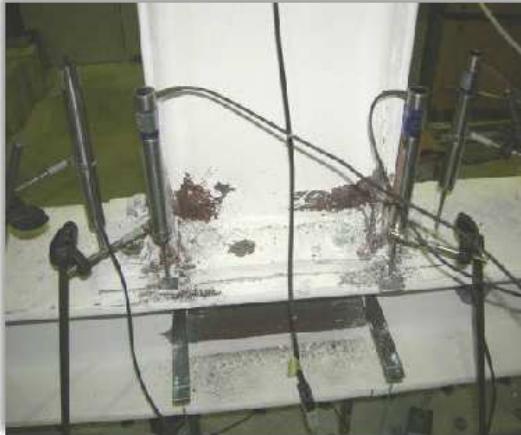
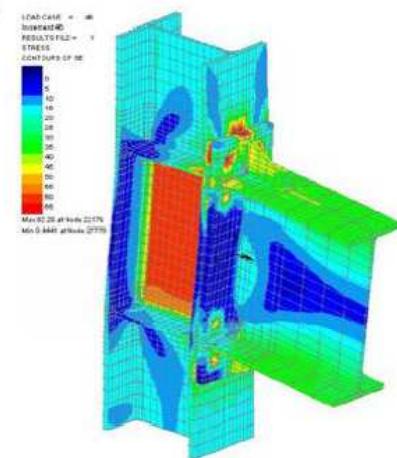


*Angle cleat joint*

### I collegamenti “dissipativi”



*Joint modelling*



*Experimental tests*



*Courtesy of Piluso*

## European pre-QUALified steel JOINTS

(RFSR-CT-2013-00021)



Obiettivo fondamentale del progetto:

Fornire procedure di pre-qualificazione per un set di connessioni trave-colonna soggette a carico ciclico.



## European pre-QUALified steel JOINTS

(RFSR-CT-2013-00021)



### Partners

1.  UNIVERSITY OF NAPLES FEDERICO II (Coordinator)
2.  ARCELORMITTAL BELVAL & DIFFERDANGE SA
3.  UNIVERSITE DE LIEGE
4.  UNIVERSITATEA POLITEHNICA DIN TIMISOARA
5.  IMPERIAL COLLEGE OF SCIENCE, TECHNOLOGY AND MEDICINE
6.  UNIVERSIDADE DE COIMBRA
7.  EUROPEAN CONVENTION FOR CONSTRUCTIONAL STEELWORK VERENIGING

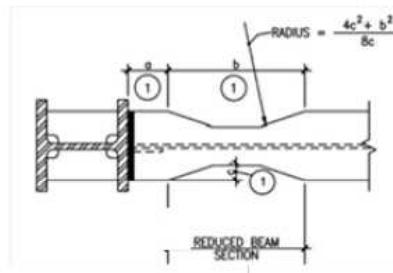
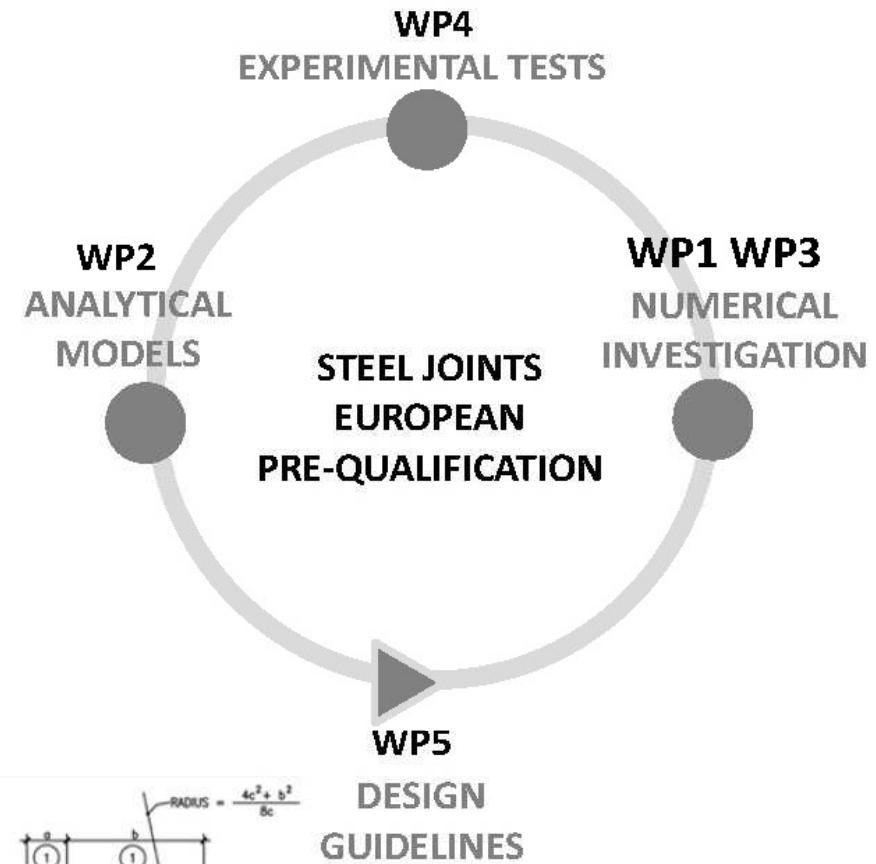
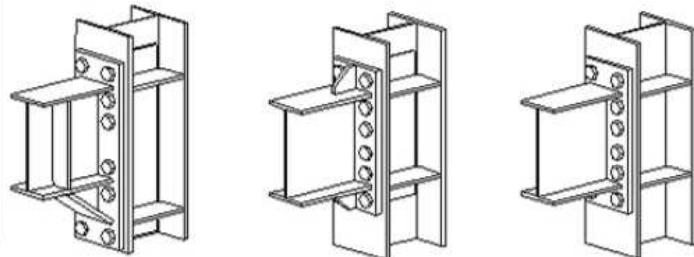
# European pre-QUALified steel JOINTS

(RFSR-CT-2013-00021)



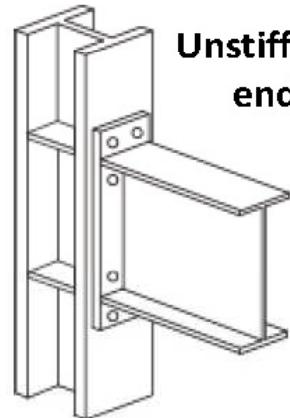
## Objectives

- 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.

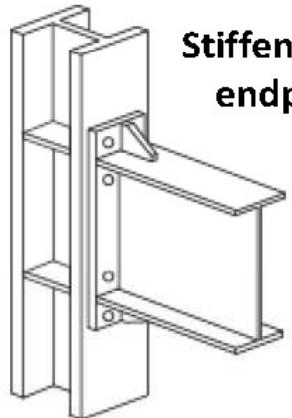


## Prequalified joint typologies

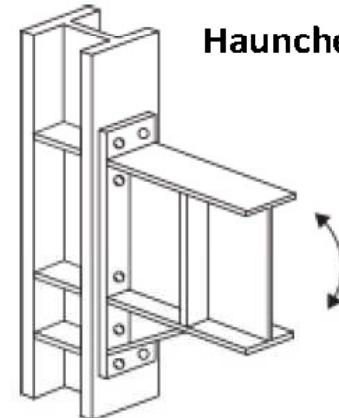
- Three bolted joint types:



Unstiffened extended endplate joints

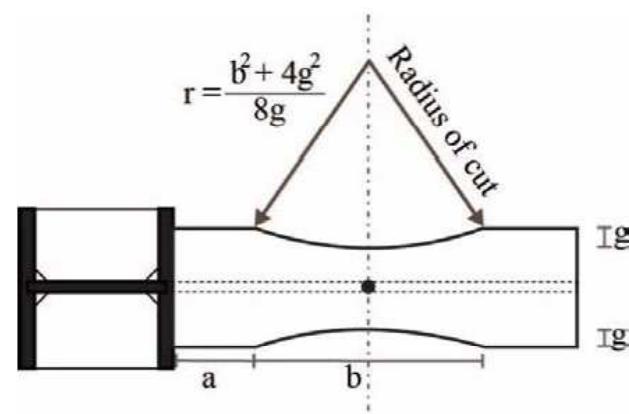
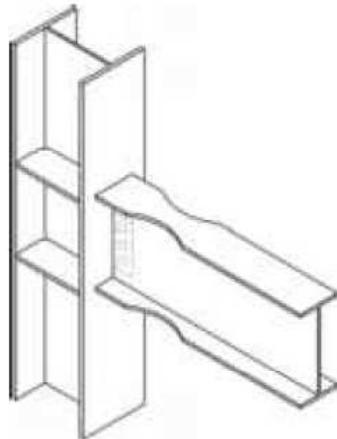


Stiffened extended endplate joints

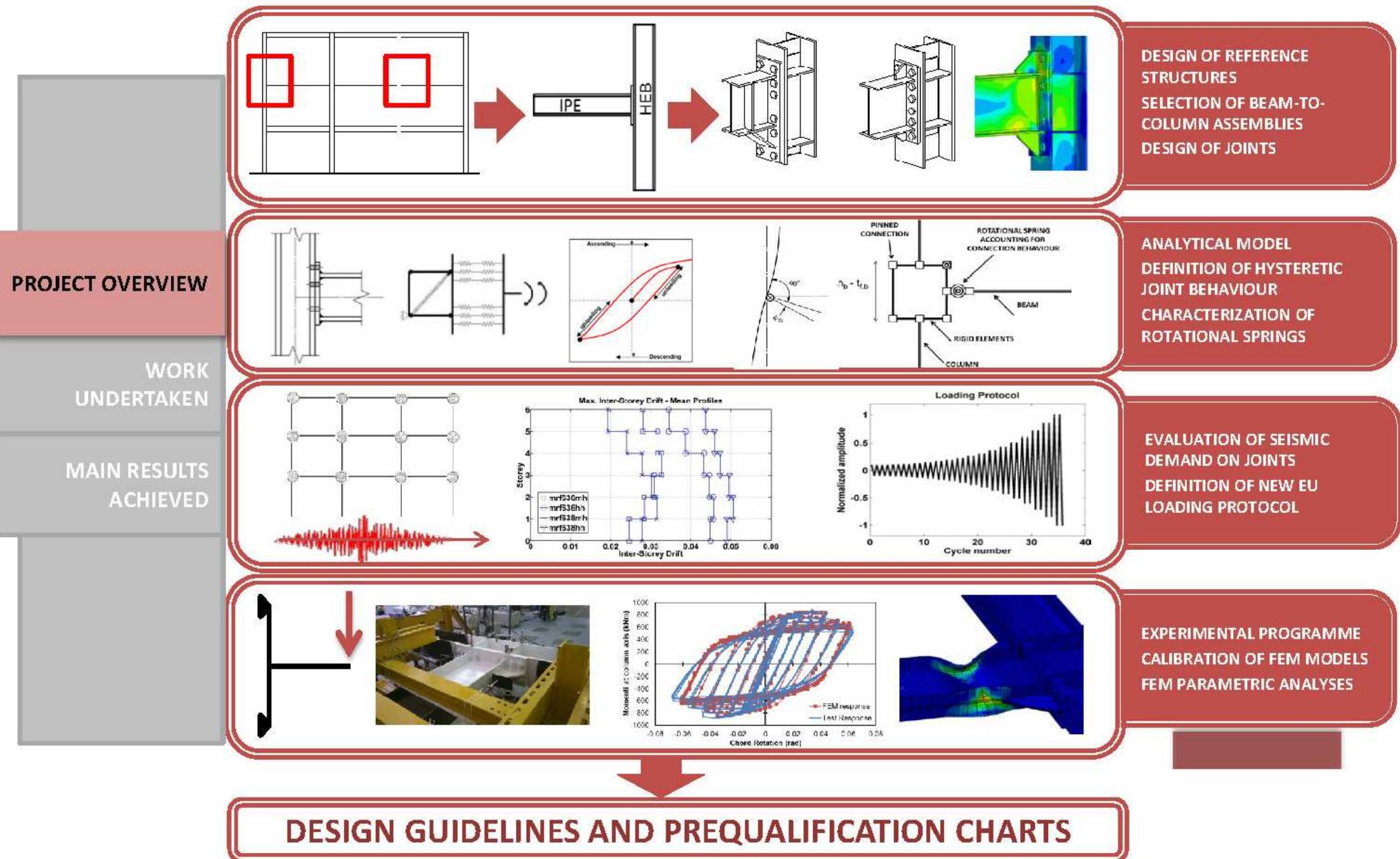


Haunched joints

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



## European pre-QUALified steel JOINTS - Project flow chart



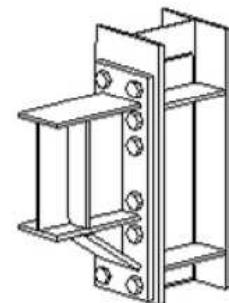
## Experimental investigation

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

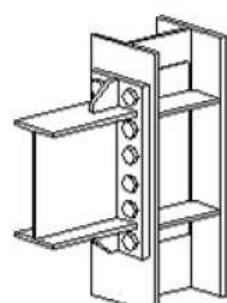
### PROJECT OVERVIEW

### WORK UNDERTAKEN

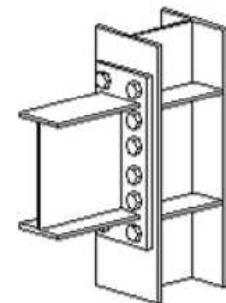
### MAIN RESULTS ACHIEVED



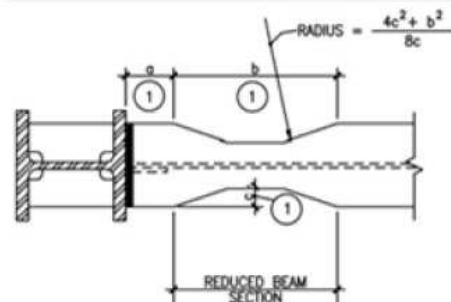
**25 Tests**



**24 Tests**



**24 Tests**



**2 Tests**

### Experimental programme – 76 joint specimens

#### beam to column assemblies

small beam (ipe 360) – medium beam (ipe450) – deep beam (ipe600)  
*\*dogbone connection designed for w-type us profiles.*

#### joint type

haunched – extended stiffened endplate – unstiffened endplate - dogbone

#### joint configuration

internal and external

#### performance objectives

full strength – equal strength – partial strength

#### loading protocol

monotonic – cyclic aisc – cyclic proposed

#### shot peening

yes/no

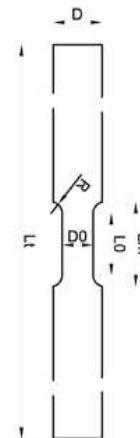
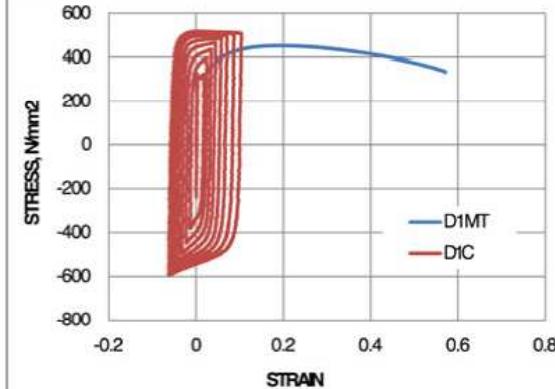
# Characterization of materials and bolts

- The tests on base material and bolts
- Cyclic characterization of HV and HR bolt assemblies
- Cyclic characterization of European mild carbon steel have been carried out.

## TESTS ON BASE MATERIAL



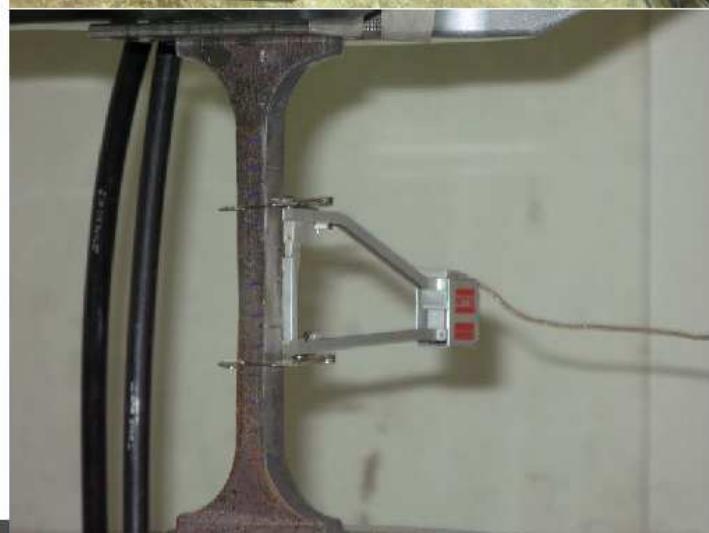
## CYCLIC CHARACTERIZATION OF EUROPEAN MILD CARBON STEEL



PROJECT OVERVIEW

WORK  
UNDERTAKEN

MAIN RESULTS  
ACHIEVED

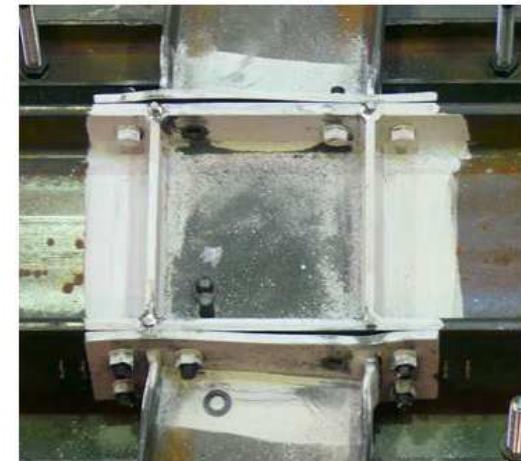


## TESTS ON BOLTS



### Tests on joints

- Tests on Haunched joints (UPT)
- Tests on Extended Stiffened joints (UNINA)
- Tests on Extended Unstiffened joints (Ulg)
- Tests on Dog-bone (AM)



#### Acceptance performance level:

EN1998-1. "The connection design should be such that the rotation capacity of the plastic hinge region  $\theta_p$  is not less than 35 mrad for structures of ductility class DCH and 25 mrad for structures of ductility class DCM with  $q > 2$ ".

AISC 341-10. "The degradation of flexural strength should not be less than 80% of the nominal flexural strength,  $M_p$ , at a drift angle of 0.04 rad".

## Calibration of FEMs and parametric analyses

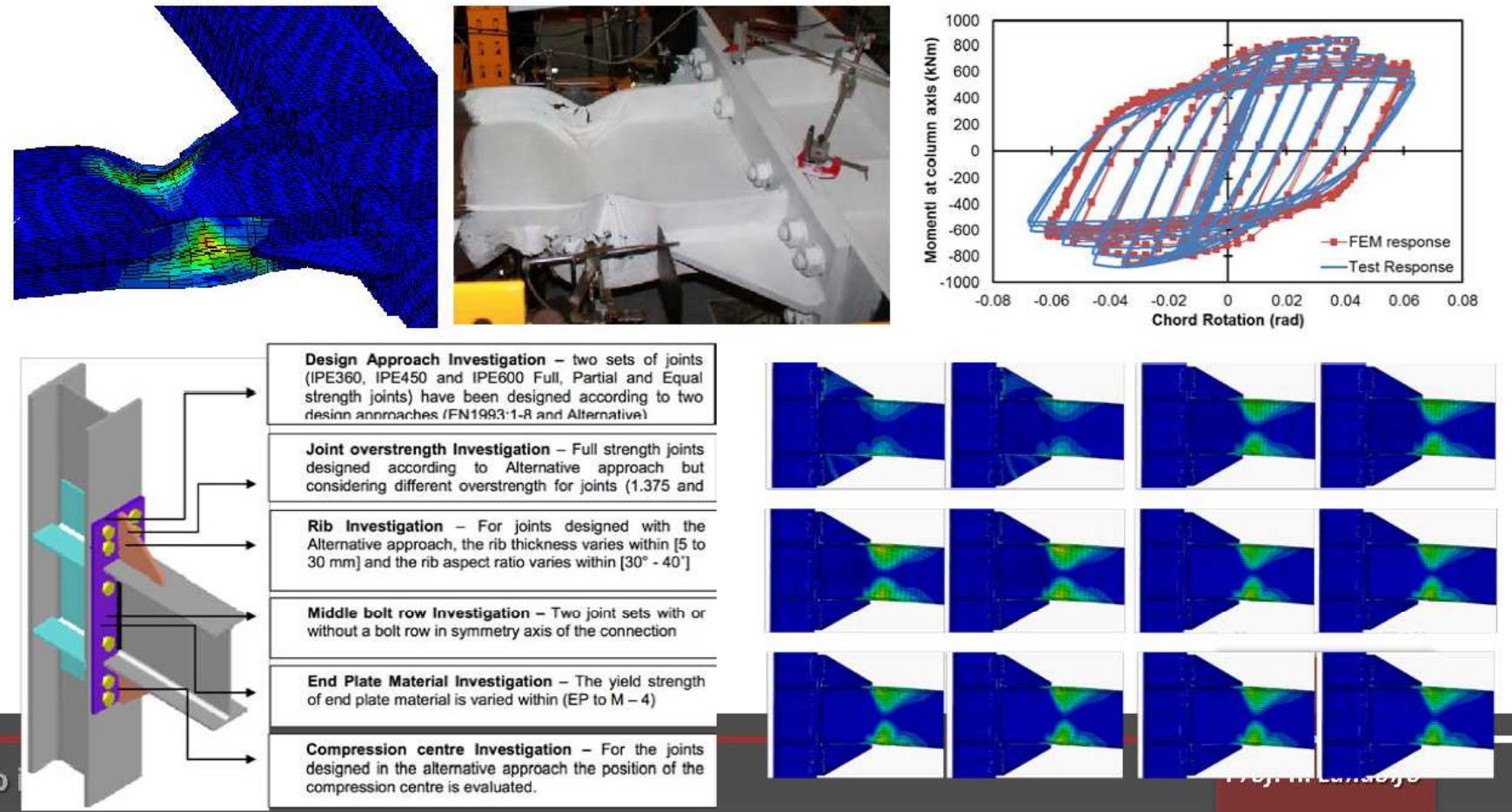
- The numerical finite elements models have been calibrated on the basis of experimental results found in literature and carried out within the project.
- The calibrated models have been used to perform a FEM parametric analyses performed in order to extend **test outcomes** and to deepen the knowledge of the behaviour of semi-rigid steel joints

### Extended stiffened end-plate joints

#### PROJECT OVERVIEW

#### WORK UNDERTAKEN

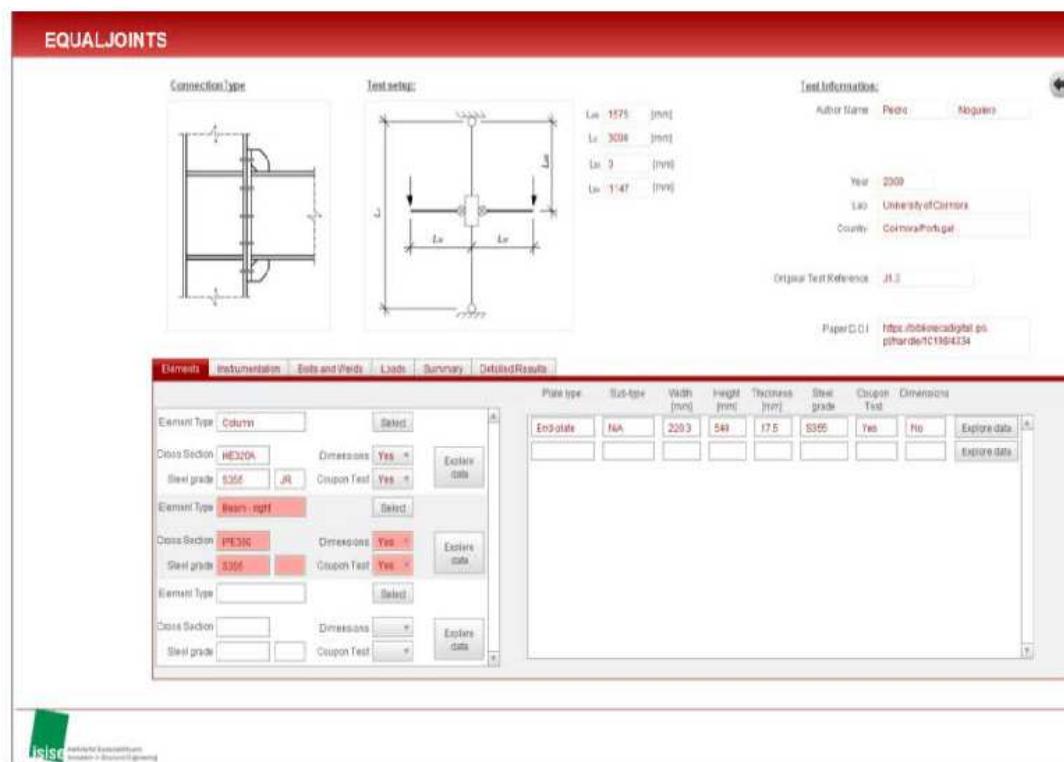
#### MAIN RESULTS ACHIEVED



## 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.

PROJECT OVERVIEW

WORK  
UNDERTAKENMAIN RESULTS  
ACHIEVED

## Prequalification charts and design guidelines



### PROJECT OVERVIEW

### WORK UNDERTAKEN

### MAIN RESULTS ACHIEVED

Component	Detail calculations	References
<b>Geometries</b>	<p><math>a_{w,1}=0 \text{ mm}</math>  <math>a_{w,2}=5 \text{ mm}</math>  <math>a_{w,3}=7 \text{ mm}</math>  <math>b_w=300 \text{ mm}</math>  <math>d_w=56 \text{ mm}</math>  <math>e=70 \text{ mm}</math>  <math>e_1=55 \text{ mm}</math>  <math>e_2=78.4 \text{ mm}</math>  <math>e_3=70 \text{ mm}</math>  <math>t_w=9.4 \text{ mm}</math>  <math>w=160 \text{ mm}</math></p>	EC3-1-G 6.2.6.5
<b>Column web and continuity plates in compression</b>	$F_{p,1,w} = M_{p,w} / (k - i_p)$ $= 604210000 / (450 - 14.5) * 10^3 = 1387.4 \text{ kN}$	EC3-1-G
<b>Column web and continuity plates in tension</b>	$\delta_{eff,e} = i_p + \sqrt{a_w(a_w + a_{w,1}) + 2(i_p + r_e) + 2e}$ $= 14.5 + 1.414 * (0+5) + 5 * (21.5 + 27) + 2 * 18 = 300.17 \text{ mm}$ $A_w = 18 * (300 - 12) = 5184 \text{ mm}^2$ $\sigma_c = 1$ $\sigma_t = \frac{1}{\sqrt{1 + 3.3 \cdot k_{eff,e} \cdot A_w / A_w^2}}$ $= 1 / (1 + 1.3 * (300.17 * 12 / 3609)^{1/2}) = 0.807$ $\lambda_{ct} = 1.0 \text{ (assuming } \sigma_{max,t} < \sigma_{p,1,w})$ $F_{w,1,t} = \frac{\delta_{eff,e} \cdot k_{eff,t} \cdot f_{y,w}}{A_w} + \frac{A_w f_{y,w}}{A_w}$ $= (0.807 * 300.17 * 12 * 355 / 1 + 5184 * 355 / 1) * 10^3 = 2872.3 \text{ kN}$	EC3-1-G 6.2.6.6
<b>Bolt rows 2 (individual)</b>	$F_{w,2,t,1} = b_{eff,e} \cdot t_{w,1} \cdot f_{y,w} / \gamma_{M0} = 554.07 * 9.4 * 355 * 10^3 / 1 = 1221.64 \text{ kN}$	EC3-1-G 6.2.6.8

---

**1. Considerazioni introduttive**

**2. La normativa e la ricerca**

**3. What's next?**

**4. Conclusioni**



UNINA

## Sistemi innovativi



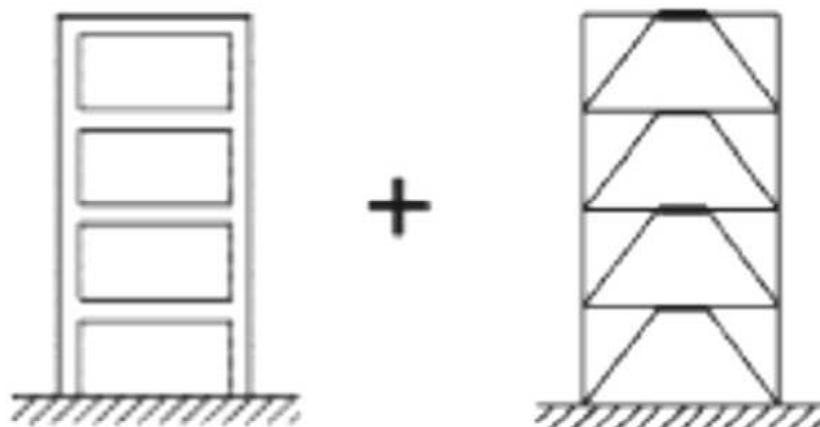
**DOG BONES**



**BRB FRAMES**



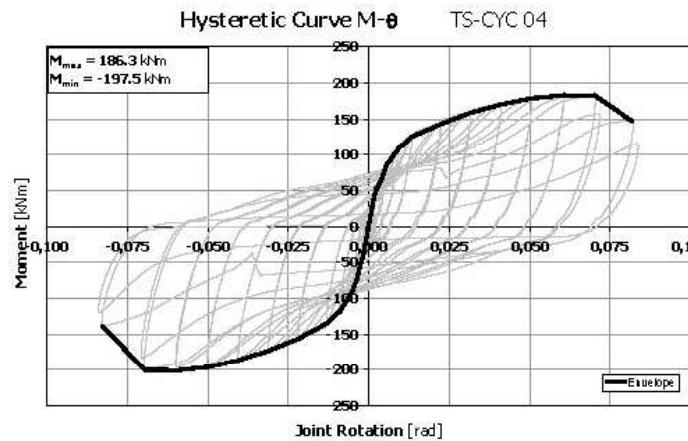
**FRAMES WITH METALLIC SHEAR PANELS**



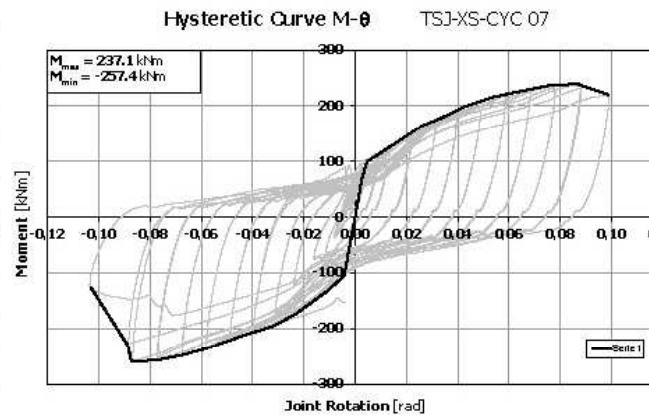
**DUAL MRFs AND EBFs**

## Sistemi innovativi

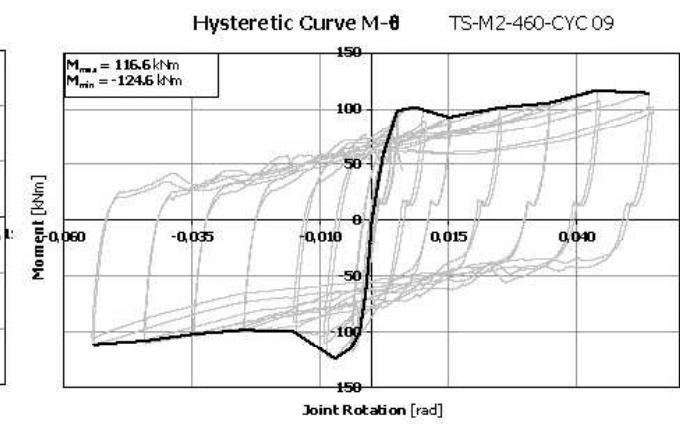
### T-stub Classico (TS-CYC 04)



### T-stub ad X (TS-XS-CYC 07)



### T-stub ad Attrito (TS-M2-460-CYC 09)



Fine Prova – Frattura piatto

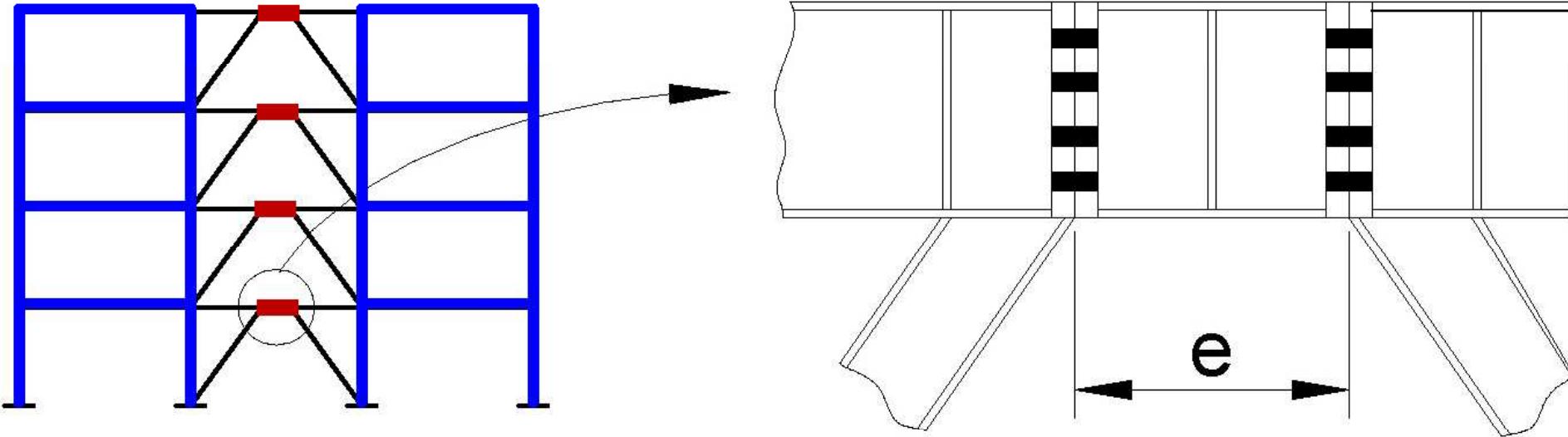


Fine Prova – Nessun danno



## Re-centring dual frames

- Structures easier to repair after the damage occurred as a result of an earthquake
  - plastic deformations constrained to replaceable dissipative members
  - re-centring capability of the structure provided by the elastic substructure
- Application: dual frames (EBF + MRF) with removable links



## DUAREM Project

### Objectives

- Experimental validation of the re-centring capability of dual structures with removable dissipative members
- Assess the overall seismic performance of dual eccentrically braced frames
- Analysis of the interaction between the steel frame and the reinforced concrete slab in the link region

### Partners

- Lead User: "Politehnica" University of Timisoara
- University of Naples "Federico II"
- University of Liege
- University of Ljubljana
- University of Coimbra



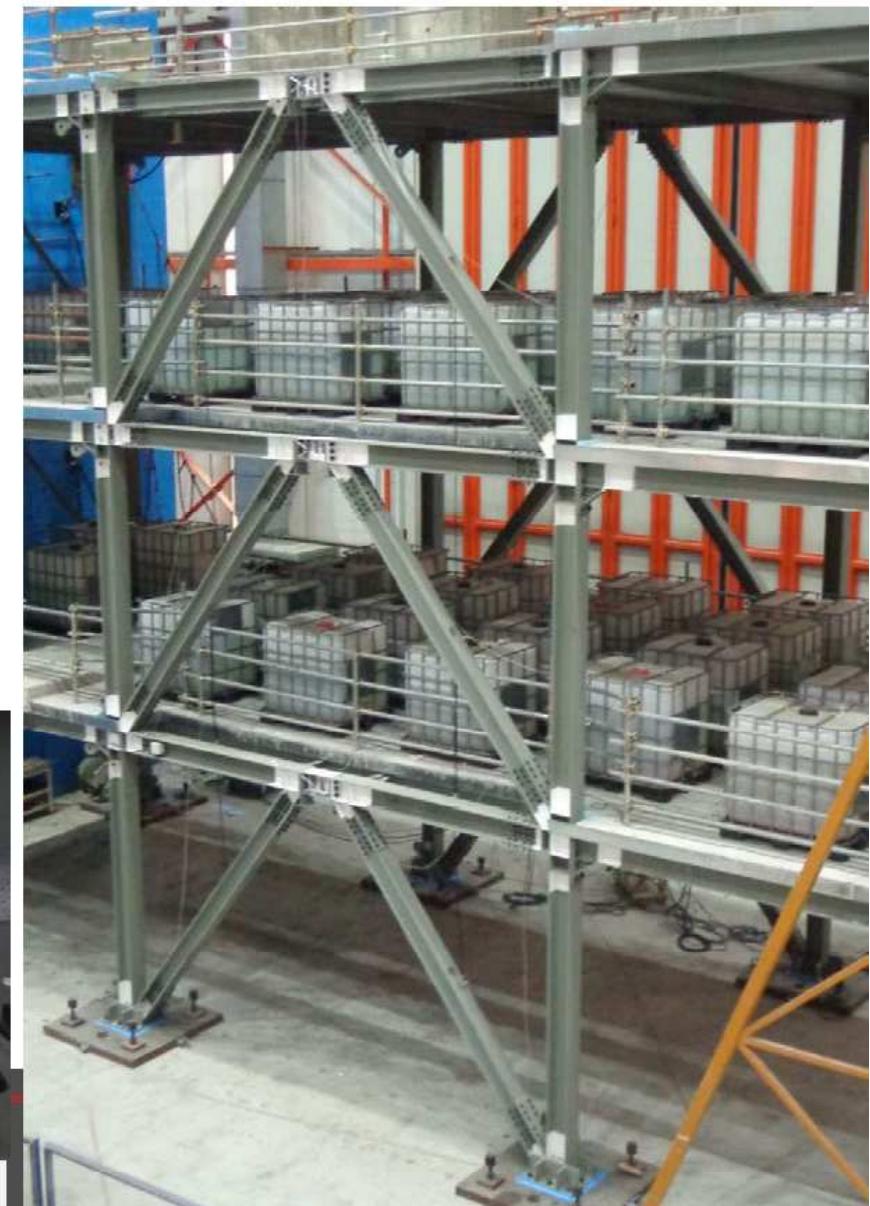
Concrete slab cast over the link



link disconnected from the concrete slab



Joint Research Centre  
European Laboratory for Structural Assessment



## DUAREM Project

### Test results

#### FO Earthquake

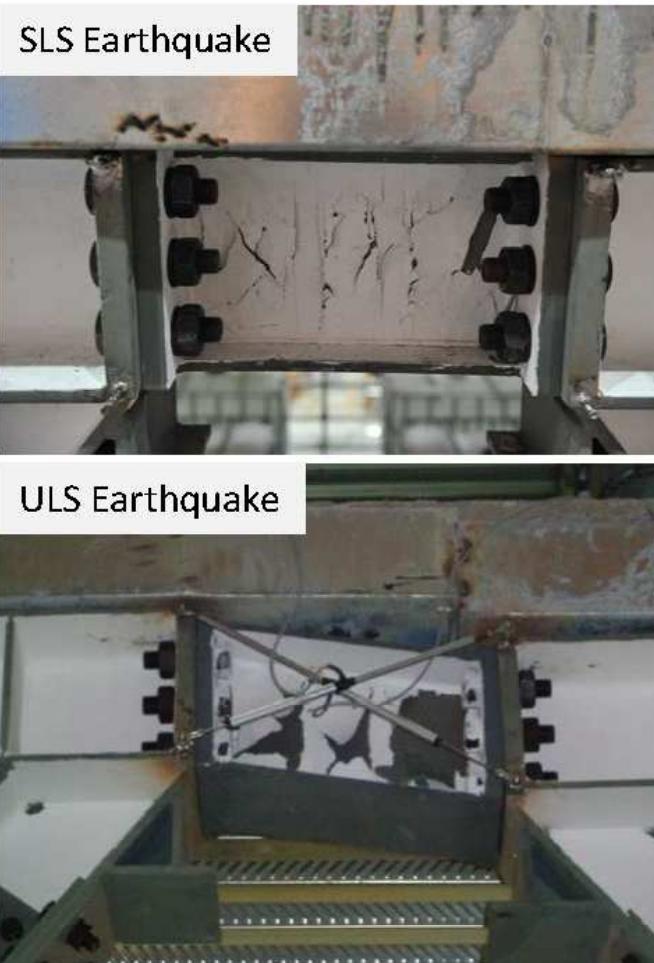
- PGA = 0.02g
- Elastic response

#### SLS Earthquake

- PGA = 0.191g
- Small plastic deformations in links ( $\approx 0.032$  rad)
- No yielding outside links
- Minor cracks in slab

#### ULS Earthquake

- PGA = 0.324g
- Moderate plastic deformations in links ( $\approx 0.059$  rad)
- No yielding outside links
- Moderate cracks in slab



### 3. What's next

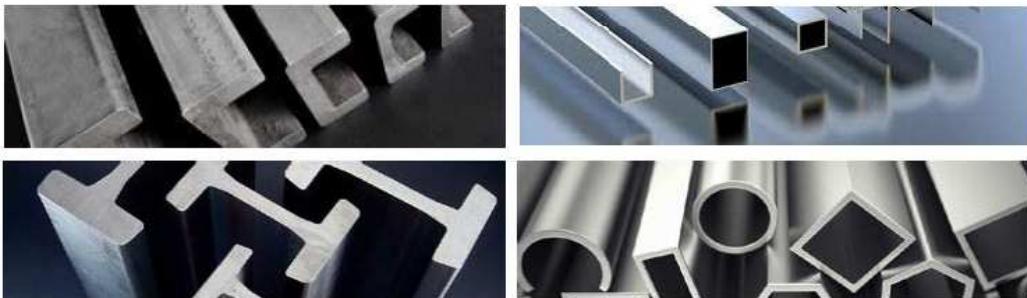
**Lightweight Steel-Framed Constructions** using Cold-Formed Steel (CFS) profiles are inherently lightweight



### 3. What's next

The lightweight nature of these systems is due to the use of **Cold-Formed Steel profiles**

#### Hot-rolled profiles



IPE



RHS



HE



SHS



UPE/UPN

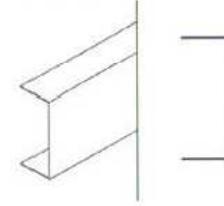
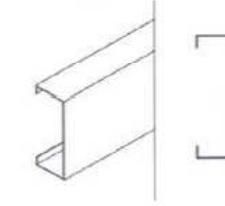
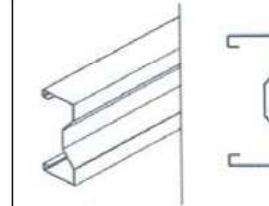
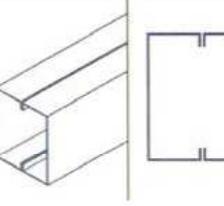
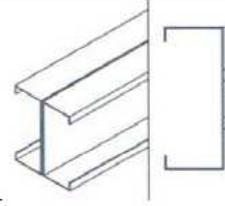
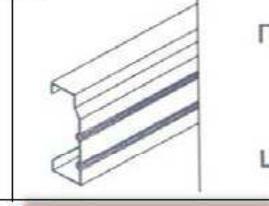


OHS

$4 \text{ mm} \leq t \leq 40 \div 45 \text{ mm}$

#### Cold-formed steel profiles



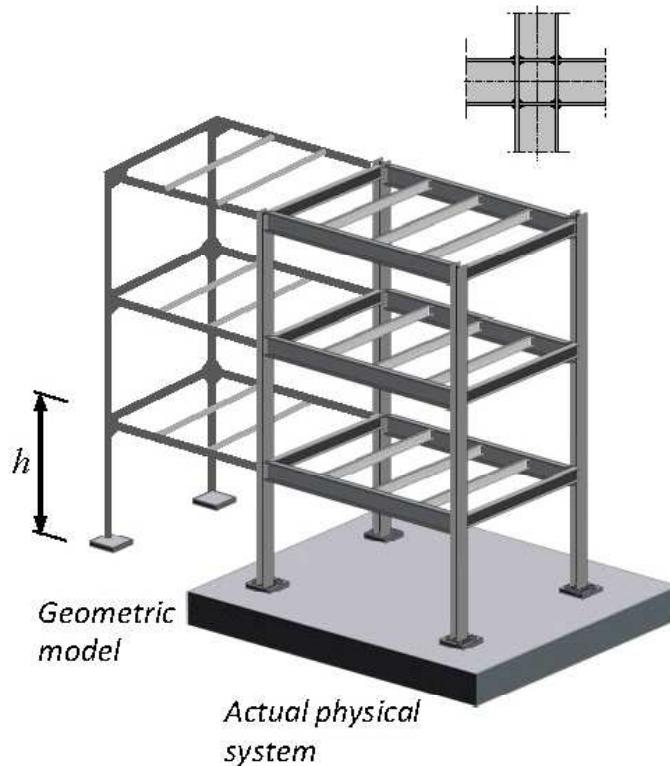
Unflipped channel section (U-section)	Lipped channel section (C-section)	Multibeam section
		
Box-section (obtained by double C-sections)	I-section (obtained by double C-sections)	SFS section (Steel framing system)
		

$0 \text{ mm} \leq t \leq 3 \text{ mm}$

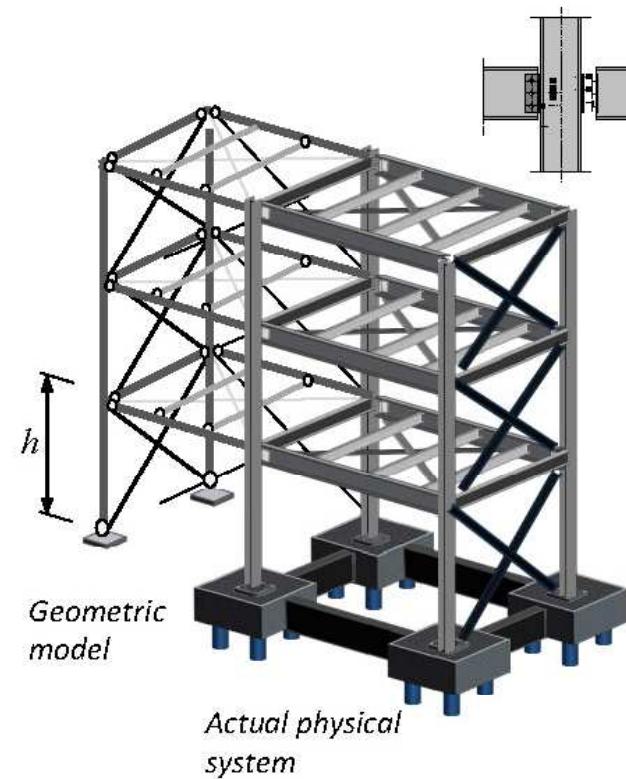
## Structural design concept

Since the Lightweight Steel-Framed Constructions are not traditional structural typologies, they need specific solutions that require a different structural design

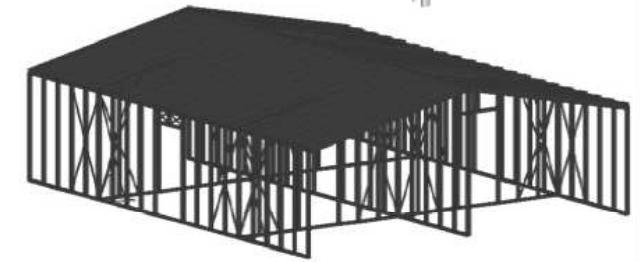
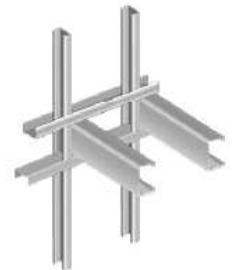
Moment-Resisting Frames (MRF)



Concentrically Braced Frame (CBF)



### Specific solutions for Lightweight Steel-Framed Construction



## Seismic design

In the last years, the application of Lightweight Steel-Framed Constructions has spread especially in non-seismic areas, but how they should be designed **in seismic areas?**



### 3. What's next



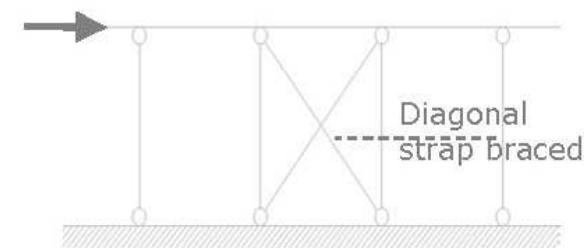
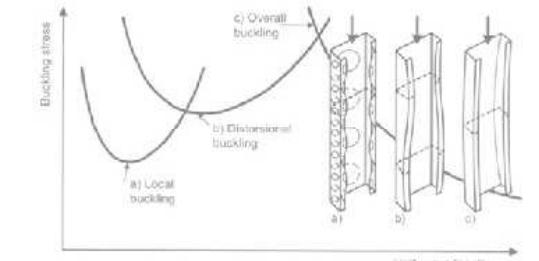
UNINA

The seismic design of Lightweight Steel-Framed Constructions under vertical and horizontal loads can be carried out using two approaches:

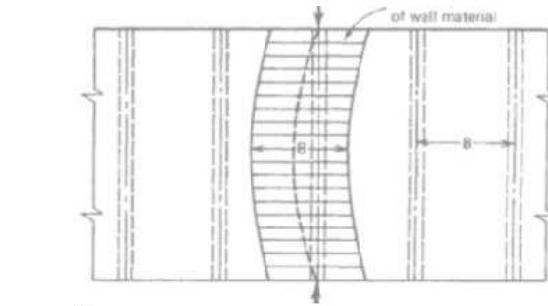
All-steel design



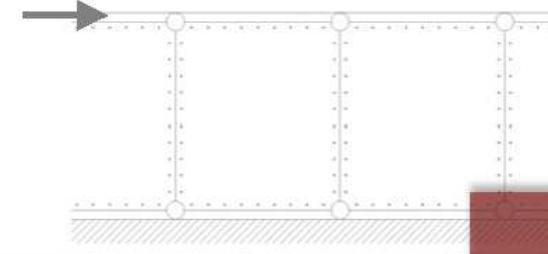
Sheathing-braced  
design



VERTICAL LOADS



HORIZONTAL LOADS



VERTICAL LOADS

HORIZONTAL LOADS

## 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



Eurocode 8 **does not provide specifications** applicable to Sheathing-braced structures

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

## Seismic design criteria according to North American codes



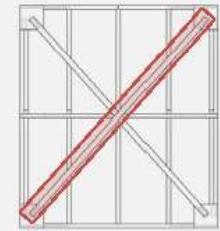
North American Codes (AISI S400 for USA, Mexico and Canada; ASCE 7 for USA and Mexico; NBCC for Canada) allow the **dissipative design approaches** according to the Capacity design.



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

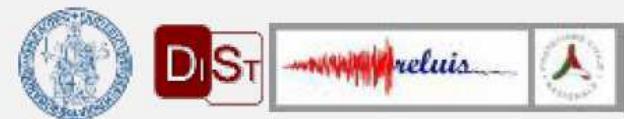
	ASCE 7	NBCC		
Behaviour factor	6.5 (bearing wall systems)	7.0 (building frame systems)	4.25 (shear walls with wood-based structural panel sheathing)	
Overstrength factor	2.5 (building frame systems)	3.0 (bearing wall systems)	1.33 (DFP and OSB panels)	1.45 (CSP wood panels)





#### Structural all-steel systems

- Italian national research project  
**ReLUIS-DPC 2010-2013**
- Italian national research project  
**Lamieredil Project 2012-2016**



#### Structural sheathing-braced systems

- Italian national research project  
**Prin 2001 - 2003**
- European research project  
**ELISSA Project 2013-2016**



#### Non-structural drywall building components

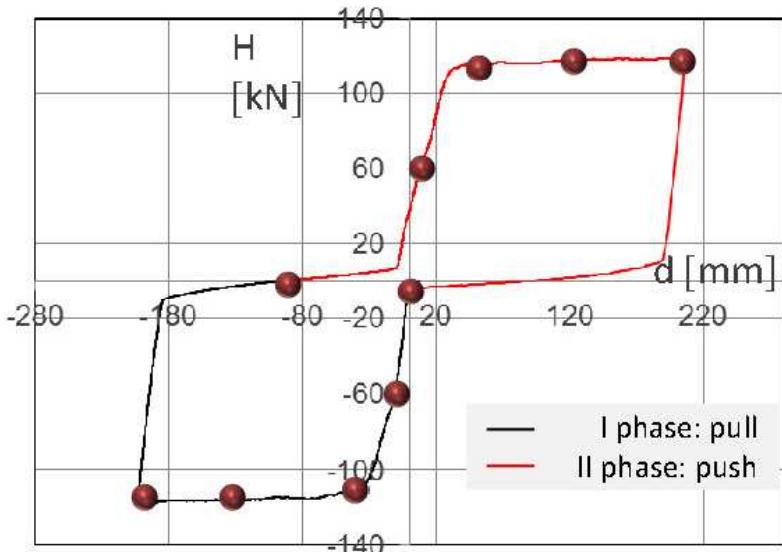
- Knauf Project 2012 - 2016



## Structural all-steel systems

### Experimental tests on full-scale shear walls: monotonic and cyclic tests

#### Monotonic tests



#### Cyclic tests

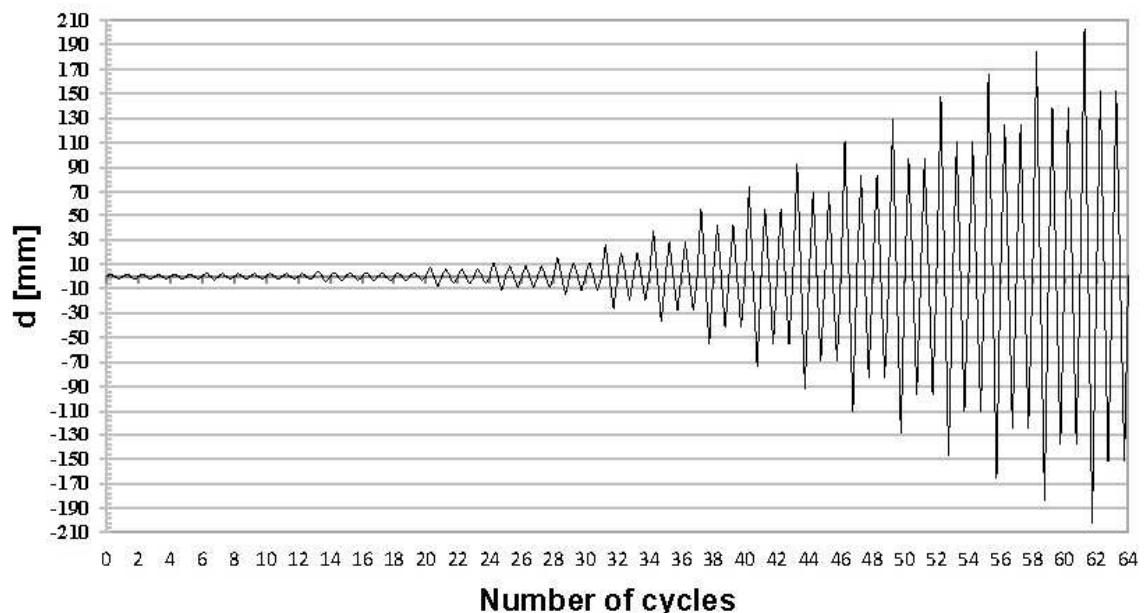
##### *CUREE ordinary ground motions reversed cyclic load protocol*

The cyclic loading test protocol consists of a series of stepwise increasing deformation cycles. The displacement amplitudes was defined starting from a *reference deformation*, that is:

$$\Delta = 2.667\Delta_y$$

where  $\Delta_y$  is the displacement at the conventional elastic limit evaluated in the monotonic tests on wall specimens.

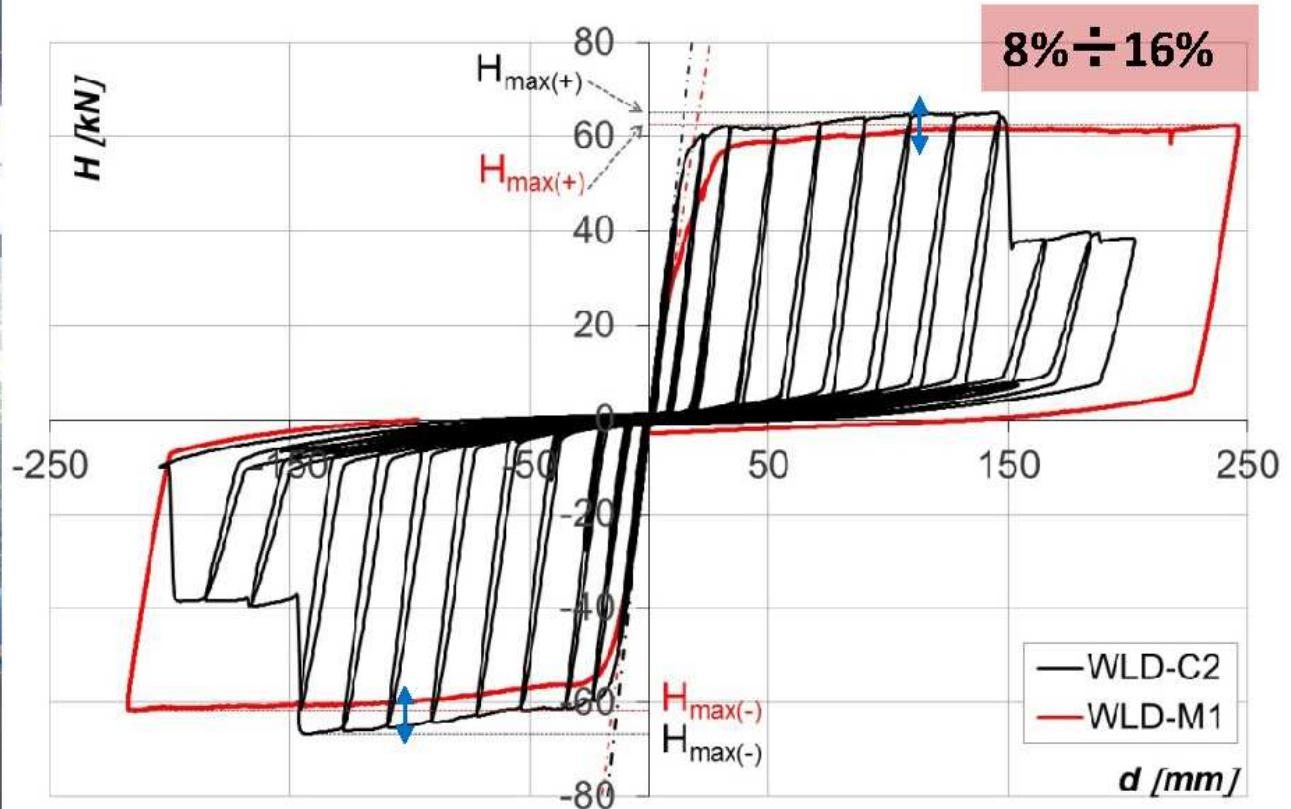
- [1] Krawinkler et al., *Development of a Testing Protocol for Woodframe Structures*, CUREE 2001
- [2] Velchev et al., *Evaluation of the AISI S213 seismic design procedures through testing of strap braced cold-formed steel walls, Thin walled structures*, 2010



## Structural all-steel systems

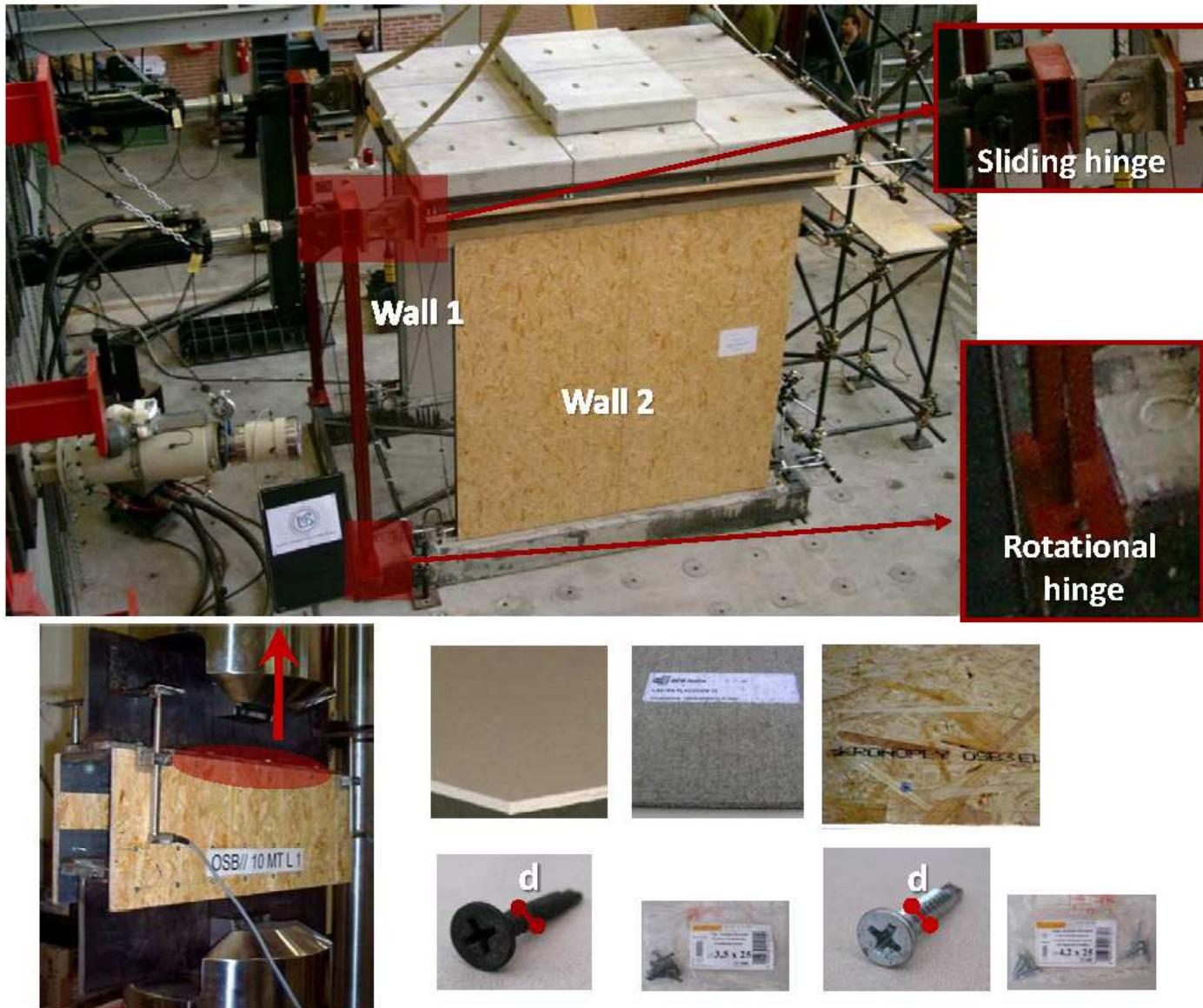
### Experimental tests on full-scale shear walls: monotonic and cyclic tests

The **comparison between the monotonic and cyclic wall tests** reveals that the average experimental strength and stiffness values registered under monotonic loads are lower than the one recorded in the cyclic tests with maximum variations of 8% and 16%, respectively.



## Structural sheathing-braced systems

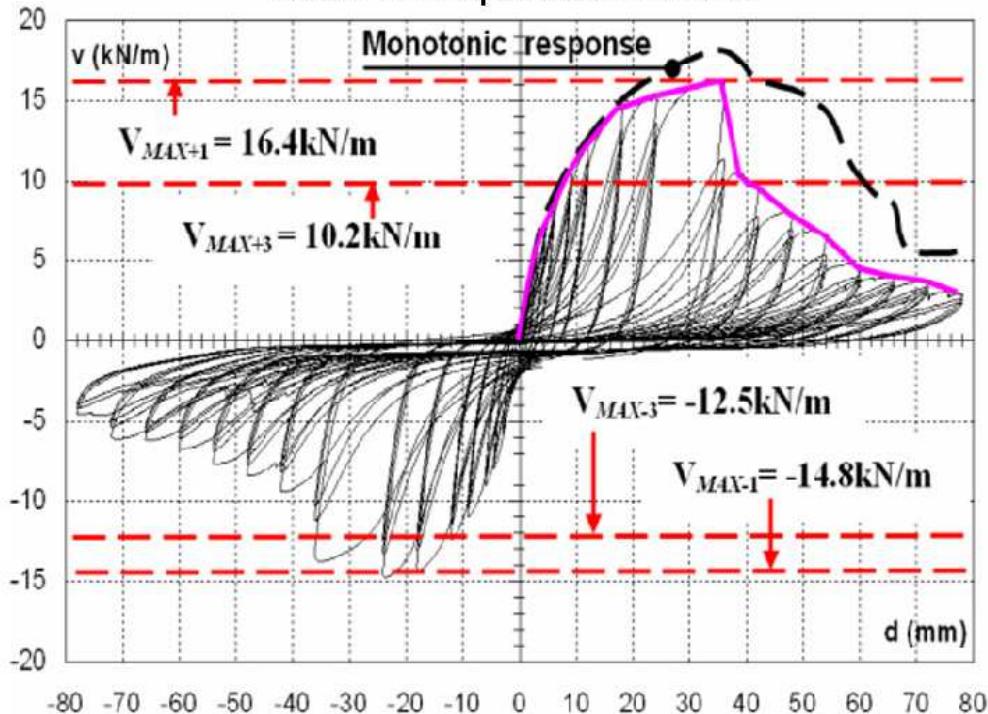
Experimental tests on full-scale shear walls: monotonic and cyclic tests



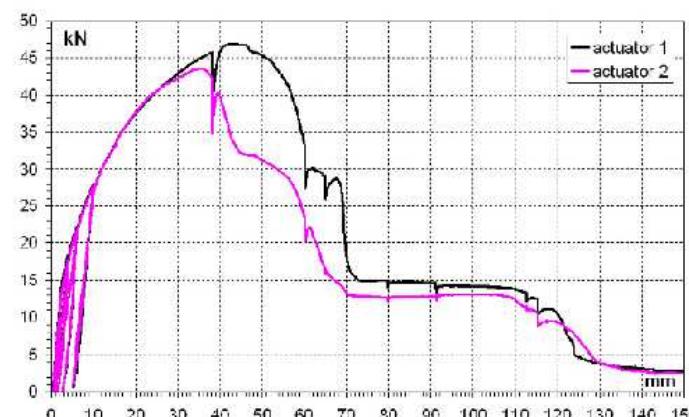
## Structural sheathing-braced systems

Experimental tests on full-scale shear walls: monotonic and cyclic tests

Shear vs. displacement curve



Deformed Wall specimen



Comparison between  
monotonic and cyclic curves



# ELISSA Research Project

Energy Efficient Lightweight –  
Sustainable – SAfe – Steel Construction



## PARTNERS



Takes the weight off your project.

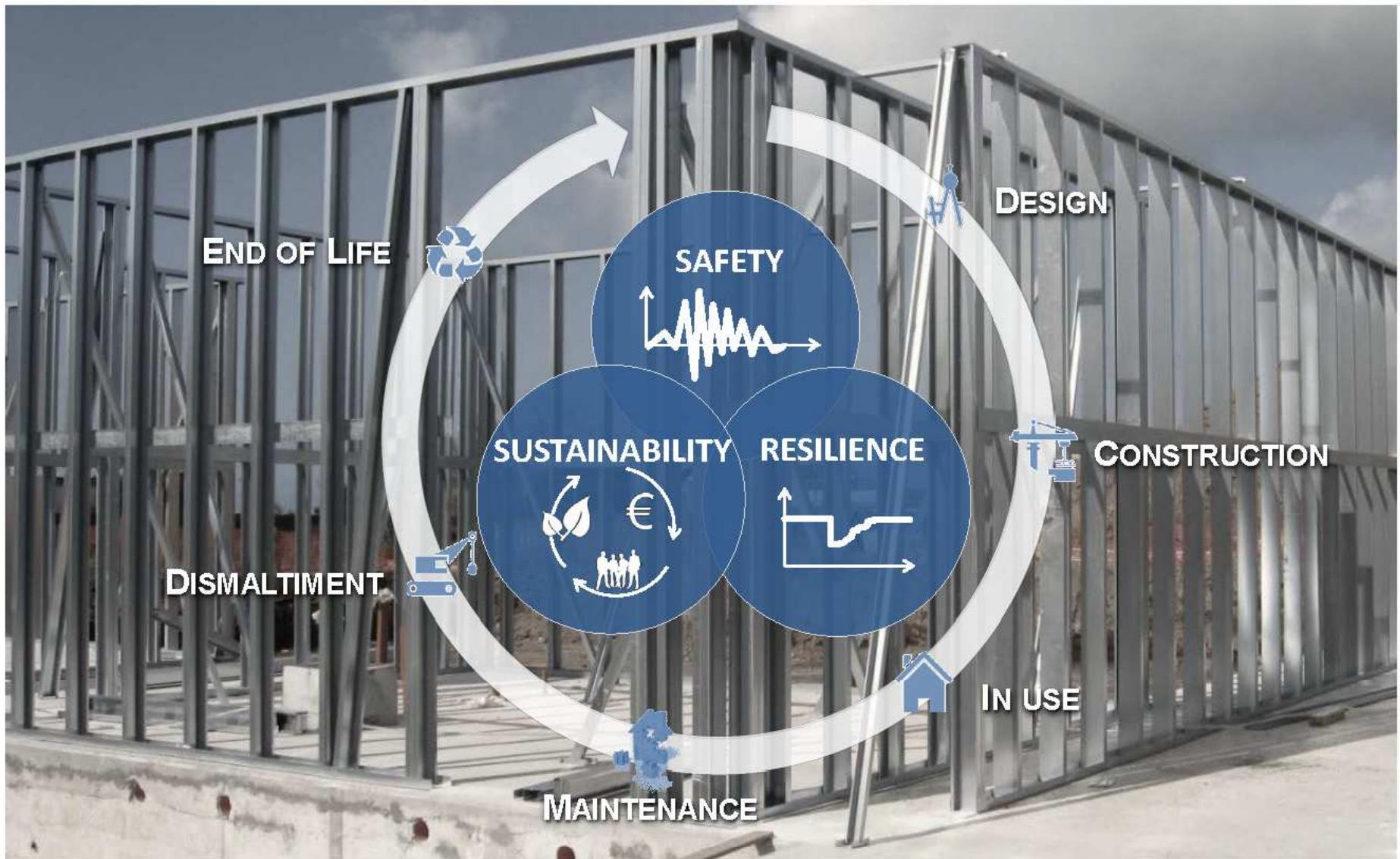


ZAE BAYERN



## The integrated approach

Since the new trends are going towards integrated solutions in terms of eco-efficiency, structural performance and economic aspects, the **Lightweight Steel-Framed Constructions**, if properly designed, could represent a competitive solution



## The basic structural system: The COCOON "Transformer"

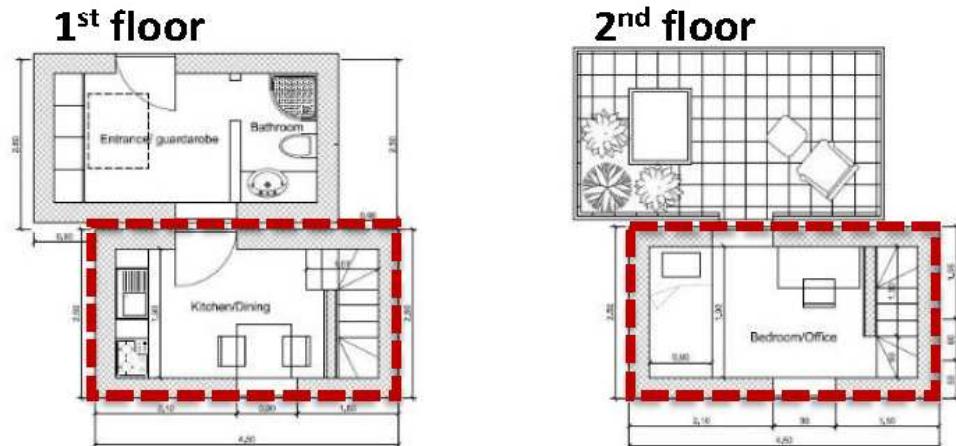
The system already obtained the European Technical Approval for static loads and the upgrading to withstand also seismic loads is one of the main objective of the ELISSA project.



## 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<sup>2</sup> + terrace**
- **Total height: 5.4 m**

## The Elissa Mock-up



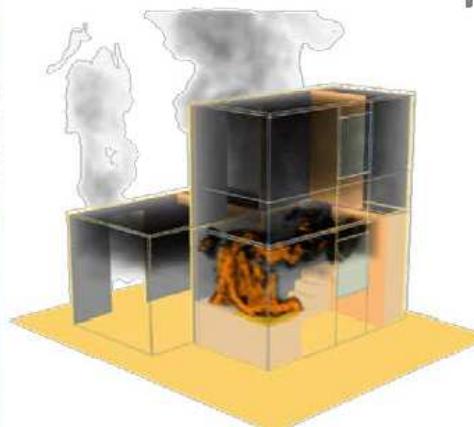
### ELISSA MOCK-UP data and design assumption

- 2 rectangular modules of plan dimensions 2.5 x 4.5 m, vertically jointed
- Two storeys building
- Total gross area: 22.5 m<sup>2</sup>
- Total height: 5.4 m
- Weight of the complete building (w/ finishing) : 102 kN (4.53 kN/m<sup>2</sup>)
- Weight of the structural part (w/o finishing): 46 kN (2.04 kN/m<sup>2</sup>)

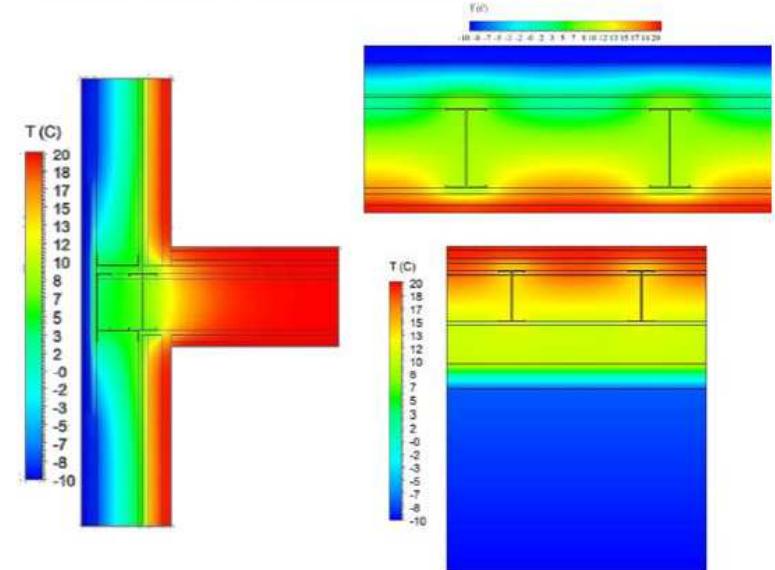
#### Nano-material



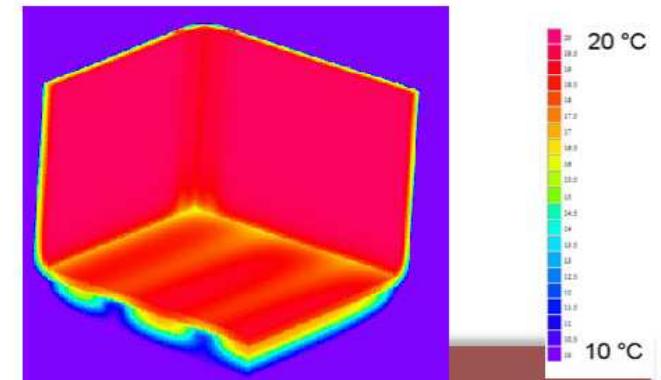
#### Fire Tests and analysis



#### Thermal Assessment



#### Hygrothermal analysis



## Structural design

### Dead load:

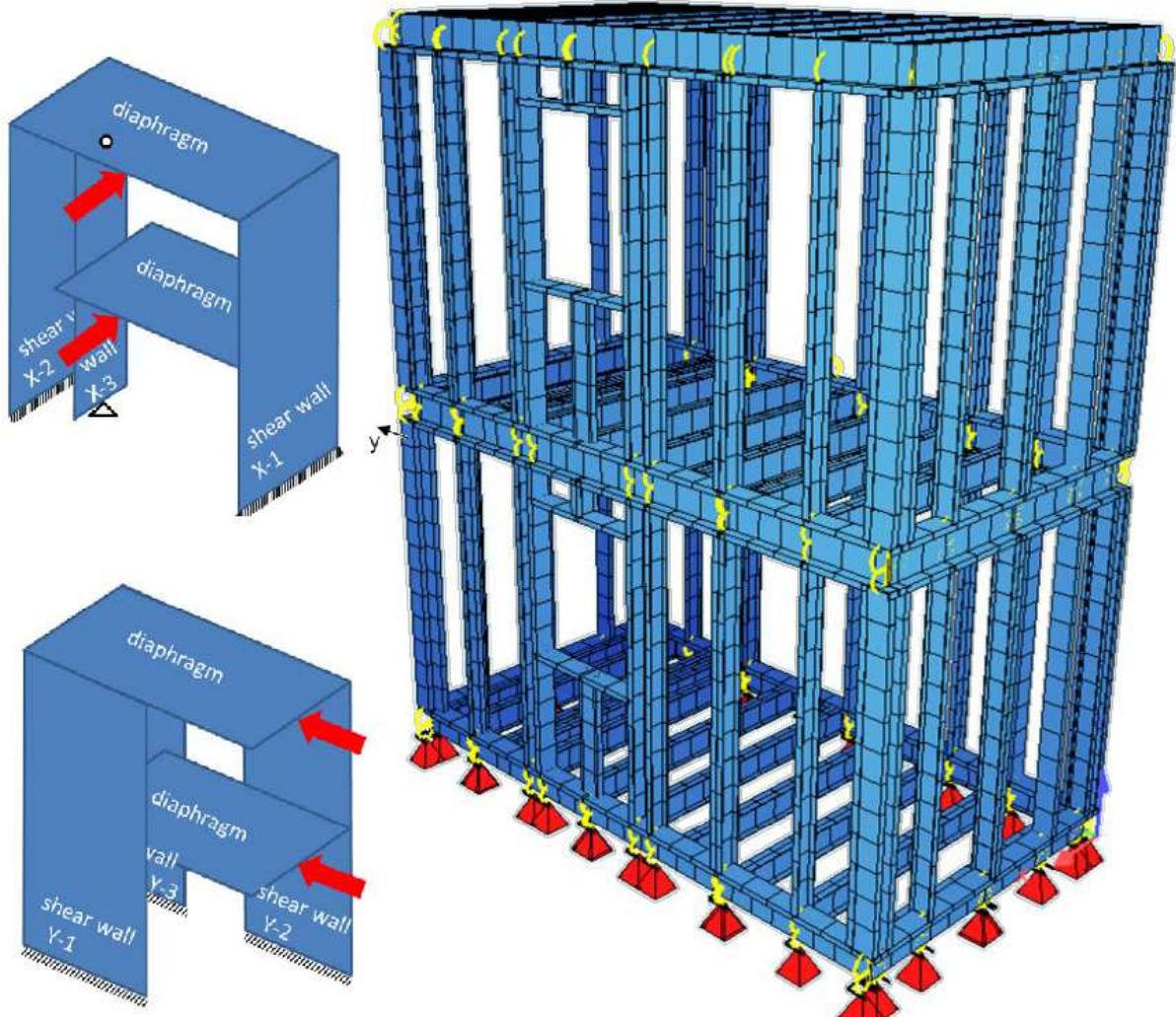
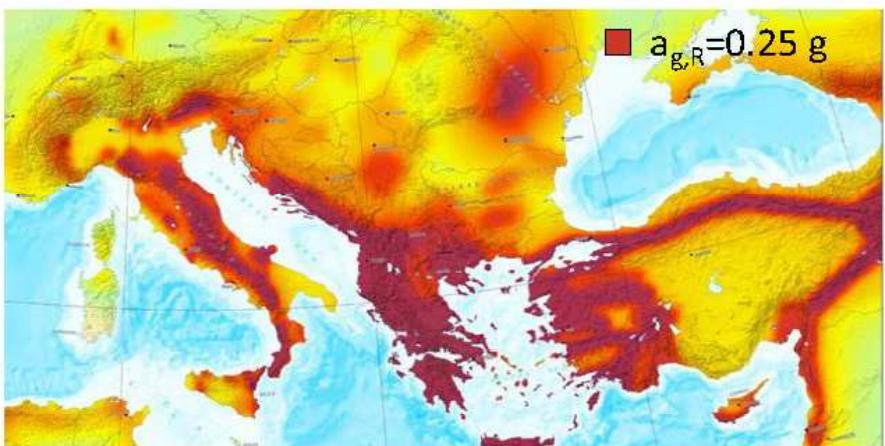
- Self weight of structure

### Imposed load:

- Residential building:  $q = 2.0 \text{ kN/m}^2$

### Seismic Input:

- PGA    ULS  $a_{g,R} = 0.25 \text{ g}$   
            SLS  $a_{g,R} = 0.10 \text{ g}$
- Spectrum: type 1, soil class C  
(medium-dense sand, gravel or stiff clay)



# Experimental program

## Micro-Scale

### Component tests

25 Panel-to-steel connections

15 Steel-to-steel connections



## Meso-Scale

### Sub-structures (wall) tests

4 wall tests



## Macro-Scale

### ELISSA mock-up

Shaking table tests



## Experimental program for shaking table tests

Elissa Mock-up configuration	Dynamic identification tests	Dynamic earthquake tests
Only Structure (Without finishing)	✓	-
Complete construction (With finishing)	✓	✓

*Seismic input was scaled with a factor from 5% to 120%*



## Dynamic earthquake tests - Input: 2009 L'Aquila Earthquake



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

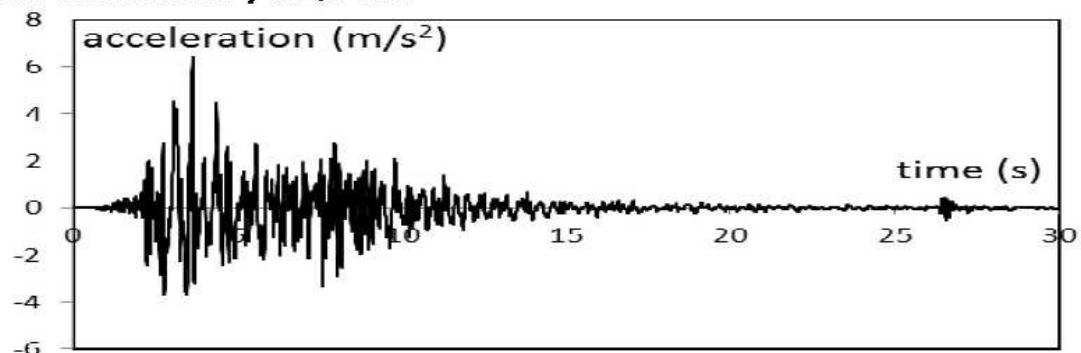


Damage caused by Aquila earthquake on traditional buildings

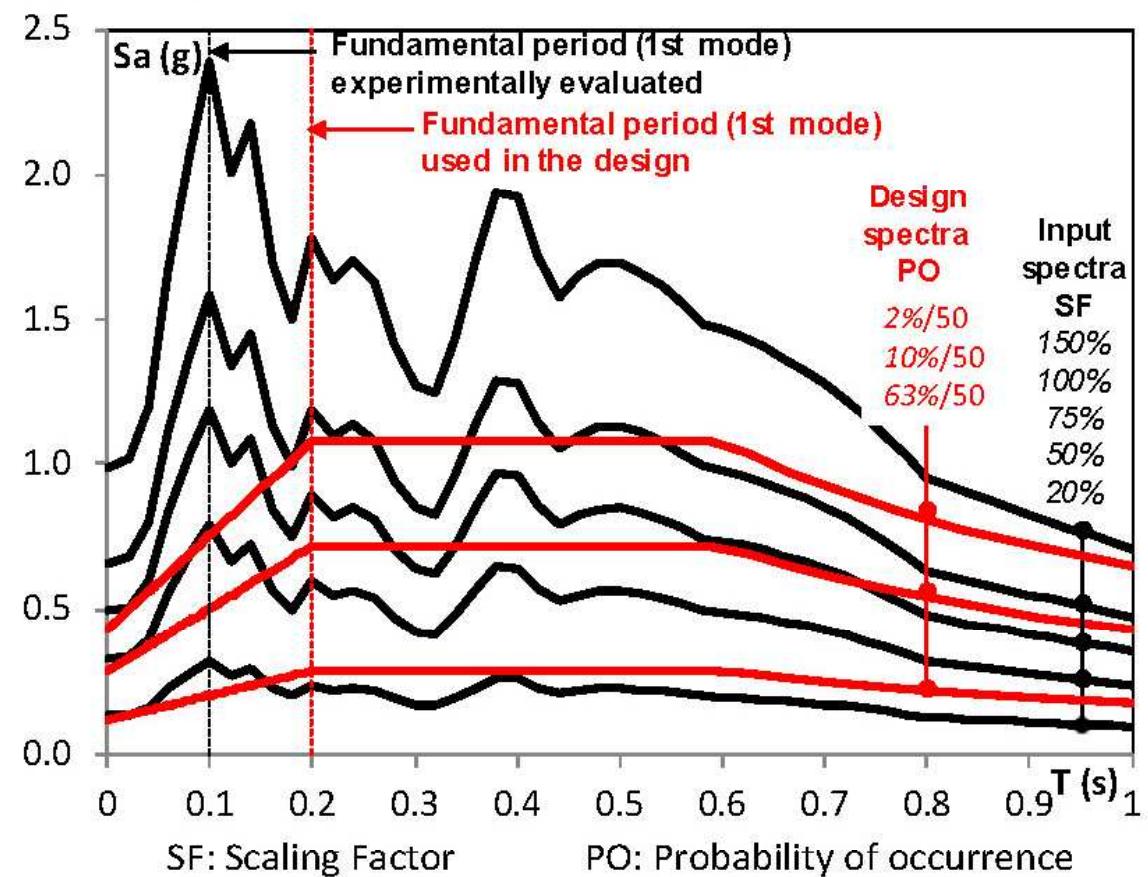
#### SELECTED GROUND MOTION

Event: L'Aquila - April 6<sup>th</sup>, 2009 3:33 a.m.  
Magnitude: Mw= 6.2  
Station: L'Aquila - Valle Aterno - Centro Valle  
Station code: AQV  
PGA: 6.44 m/s<sup>2</sup> (0.66 g)

Input time history AQV-EW



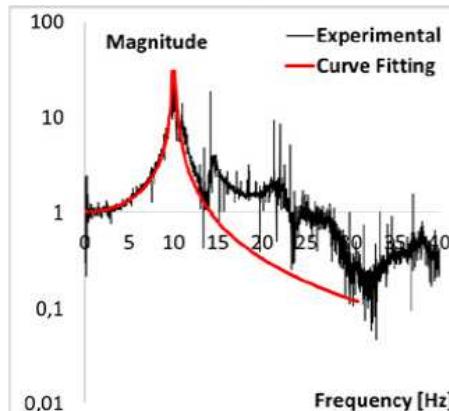
Response spectrum (Sa-T format)



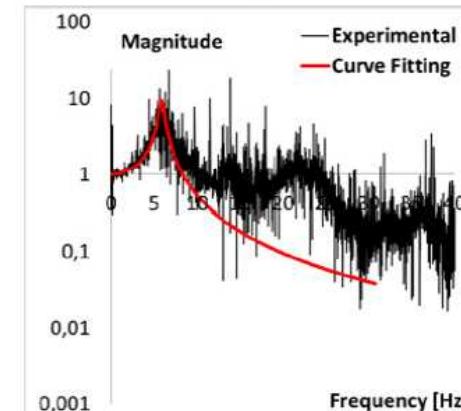
## Test results

- The **fundamental period** varied from 0.10s (w/ finishing before earthquake tests) to 0.16s (w/ finishing after earthquake tests)
- The **damping ratio** varied from 1.0% to 3.0% (w/o finishing materials before tests) and from 1% to 2% (w/ finishing materials before tests) and from 2% to 5% (w/ finishing after earthquake tests)
- The **maximum inter-storey drift** was very small (0.97% for first level and 0.58% for second level) and the **residual inter-storey drifts** were negligible (under 0.06%), evidencing a **very modest anelastic behaviour**.
- The **observed damage** was small in both structural parts and finishing materials
- Diaphragms** behaved as rigid in their plane (according to the ASCE 7 definition)

Before earthquake tests

 $T = 0.10s$ 

After earthquake tests

 $T = 0.16s$ 



MINISTRY OF DEFENCE

DiARC



## Building construction of foundation and primary stage school "BFS" Lago Patria – Naples, Italy (2009-2011)



### 3. What's next



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## REQUIREMENTS

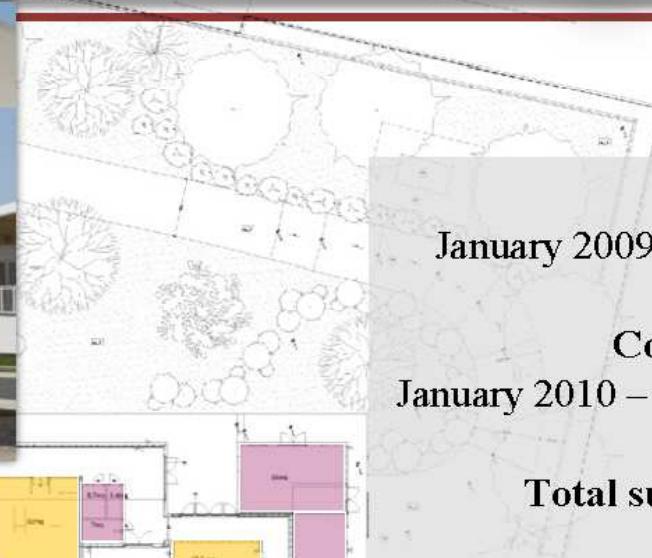
- BUILDING OF STRATEGICAL IMPORTANCE
- SHORT TIME
- HIGH PERFORMANCE IN TERMS OF:
  - SAFETY
  - DURABILITY
  - SEISMIC BEHAVIOUR
- ENVIRONMENTAL SUSTAINABILITY

FIRST IMPORTANT COLD-FORMED STEEL CONSTRUCTION WORK IN ITALY

### 3. What's next



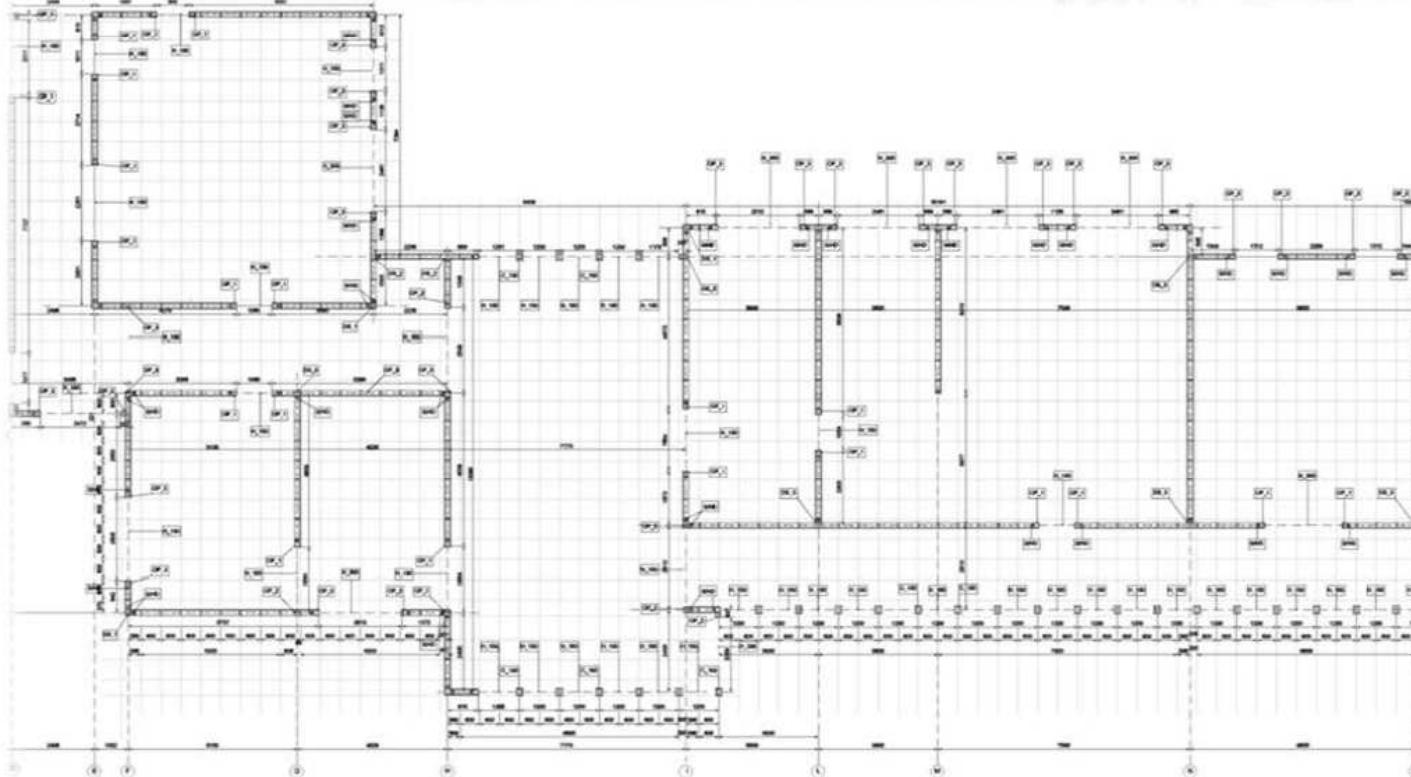
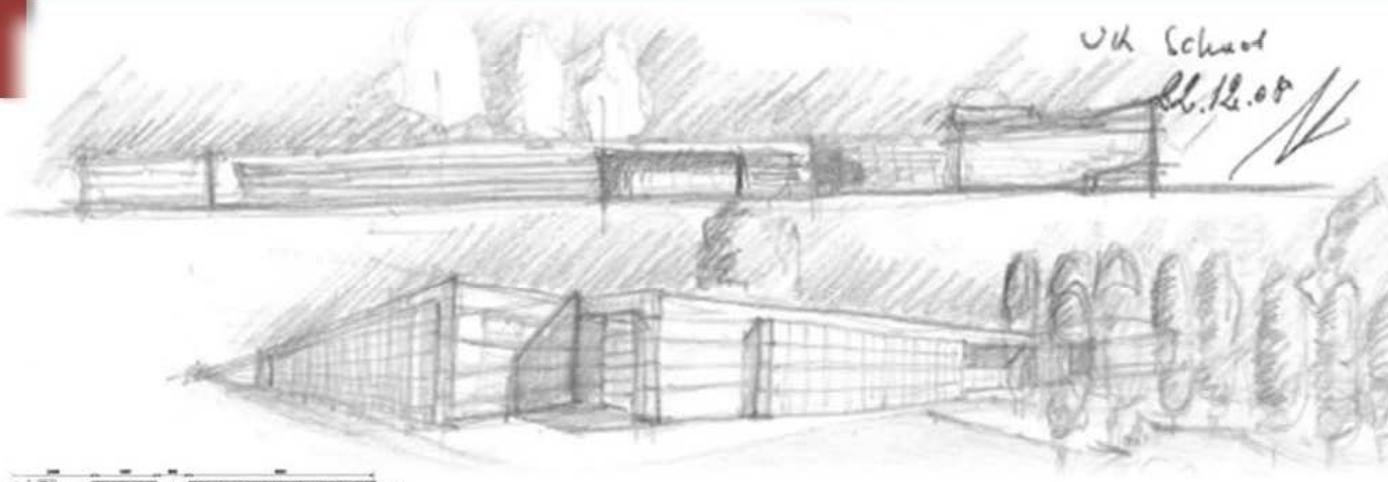
UNINA



### 3. What's next



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#### **Owner:**

Defense Estates UKNSU

#### **Contractor:**

COSAP

#### **Architects:**

Arch. Fiorenzo Petillo (*Team leader*)

Dr. Arch. Brigida Santangelo

Arch. Enza Terzigni

#### **Structures:**

Prof. Eng. Raffaele Landolfo

P.hD. Arch. Ornella Iuorio

P.hD. Eng. Luigi Fiorino

#### **Technology:**

Prof. Arch. Mario Losasso

Ass. Prof. Arch. S. Russo Ermolli

P.hD. Generosa Cacciapuoti

Arch. Antonio DAcunzi

#### **M&E:**

Eng. Roberto Romano

#### **Accounting:**

Arch. Alfonso Mauro

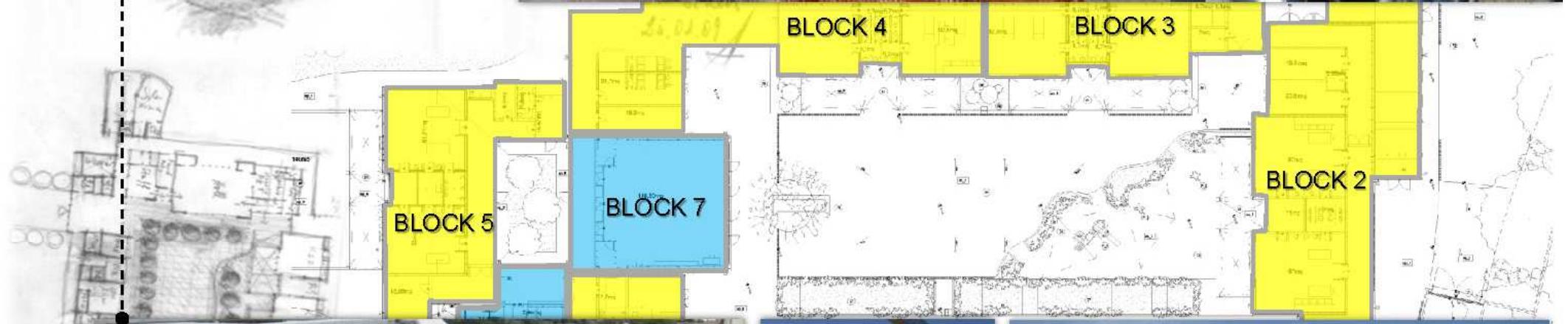
### 3. What's next



UNINA

Stick built construction-----

Cold formed steel frame

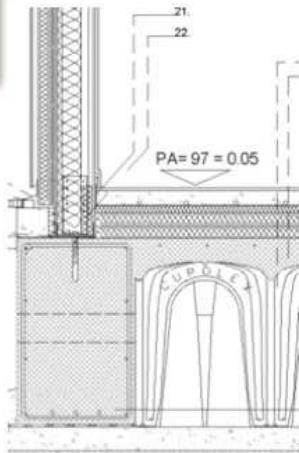


Cost

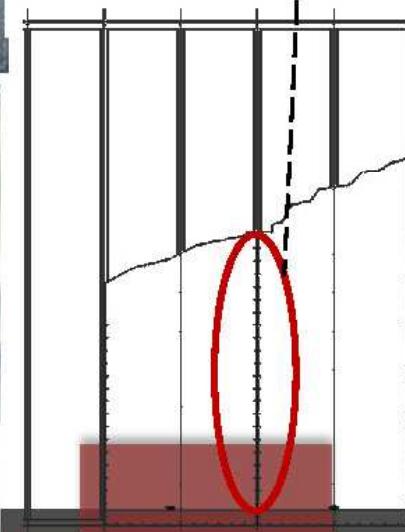
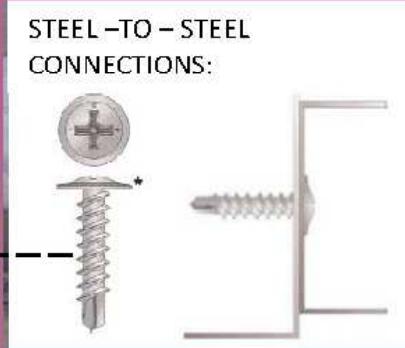
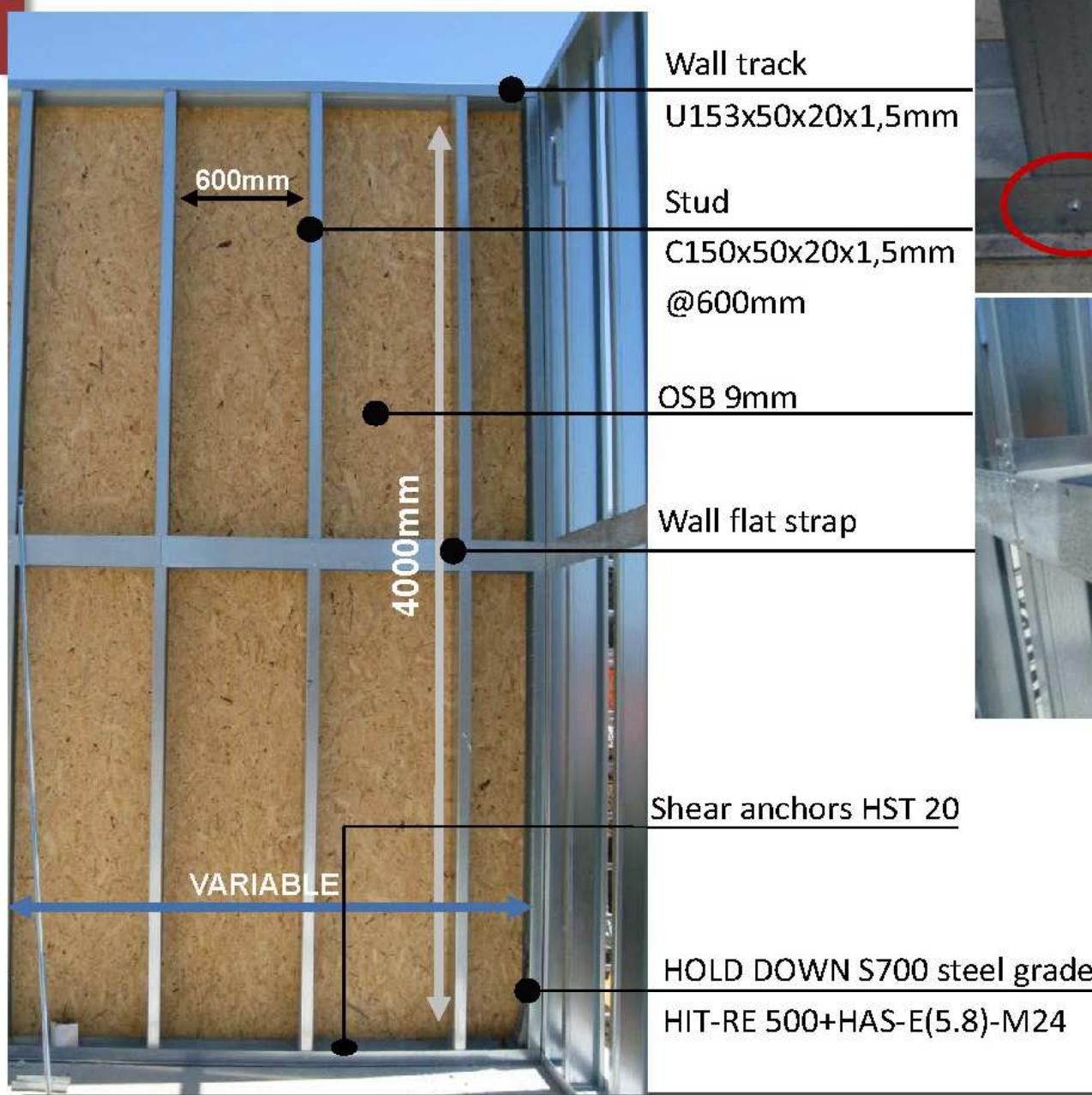
### 3. What's next



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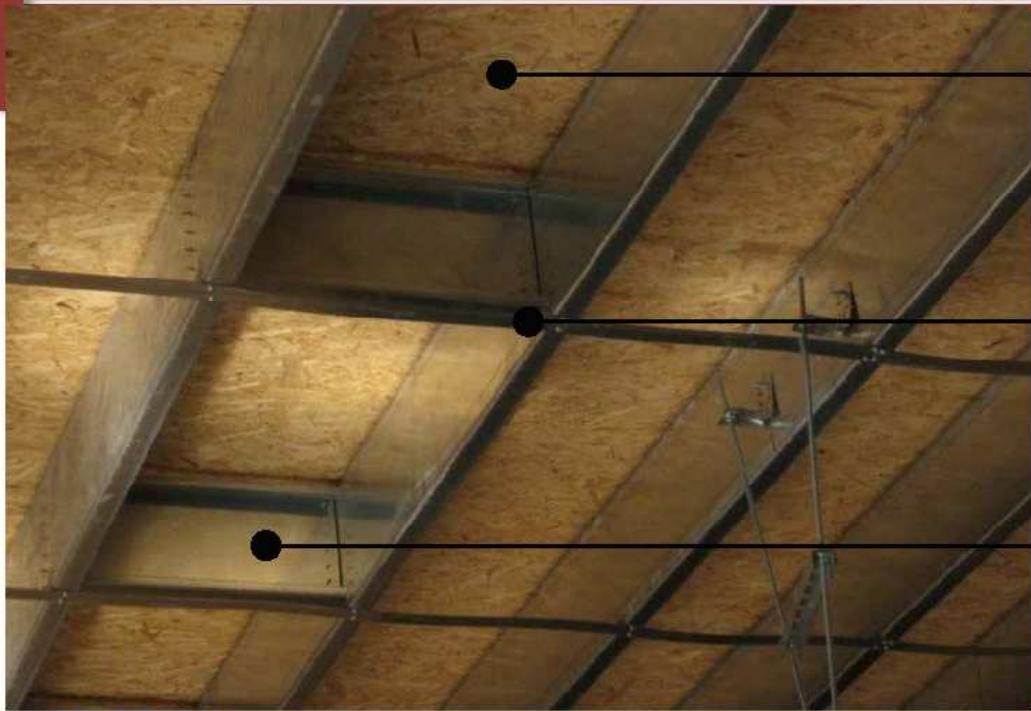
### 3. What's next



### 3. What's next



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OSB 18mm

Flat strap

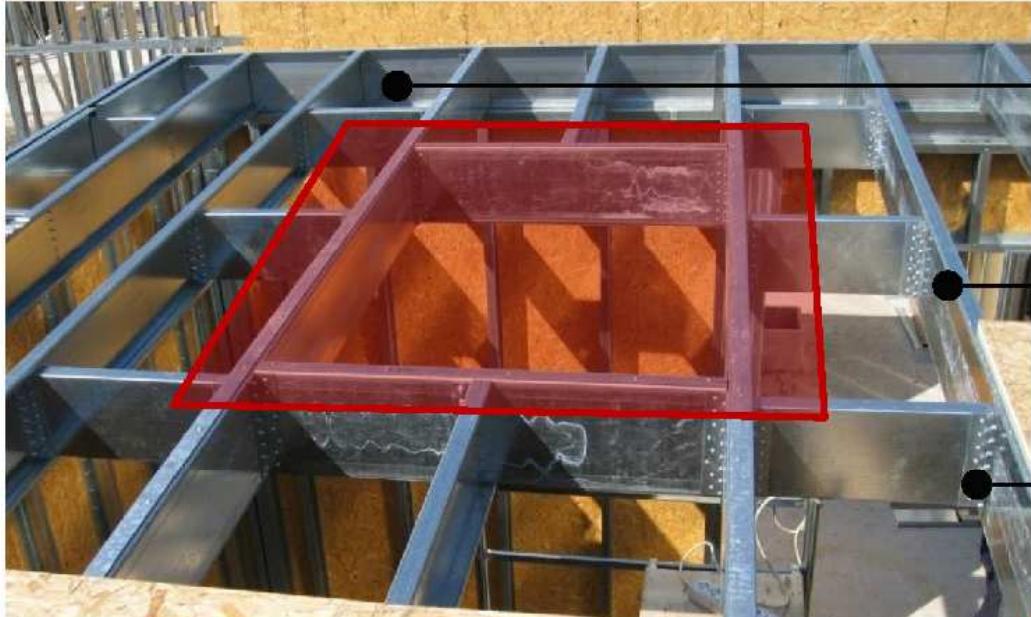
50x1,5mm

@ 1200 o 2400mm

Blocks

250x50x20x1,5

@ 1800 o 3000mm



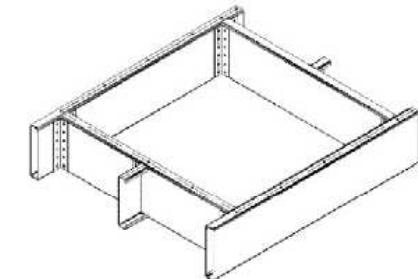
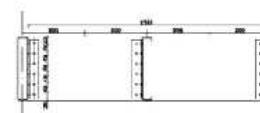
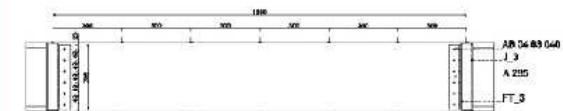
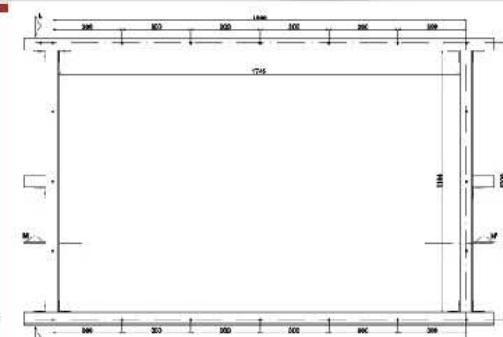
Floor track

U 303x50x1,5 - 3mm

Joist

C 300x50x20x1,5 - 3mm

Web stiffner



### 3. What's next



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Costruire in acciaio in zona sismica

Prof. R. Landolfo

### 3. What's next



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**Total surface area:**  
 $15.850 \text{ m}^2$

**Covered area by  
steel structures:**  
 $3.000 \text{ m}^2$

**TOTAL TONNAGE:**  
140 tons of steel



**Weight per square meter:**  
 $0.45 \text{ kN/m}^2$

**OSB 9 mm  
(WALLS):**  
 $10000 \text{ m}^2$

**OSB 18mm  
(FLOORS:  
 $3000 \text{ m}^2$**

## Experimental program

### Sub-structures (wall tests)

2 On-site wall tests (4.80 x 4.00 m)



### Component tests

50 sheathing-to frame connection tests

10 Hold-down device tests



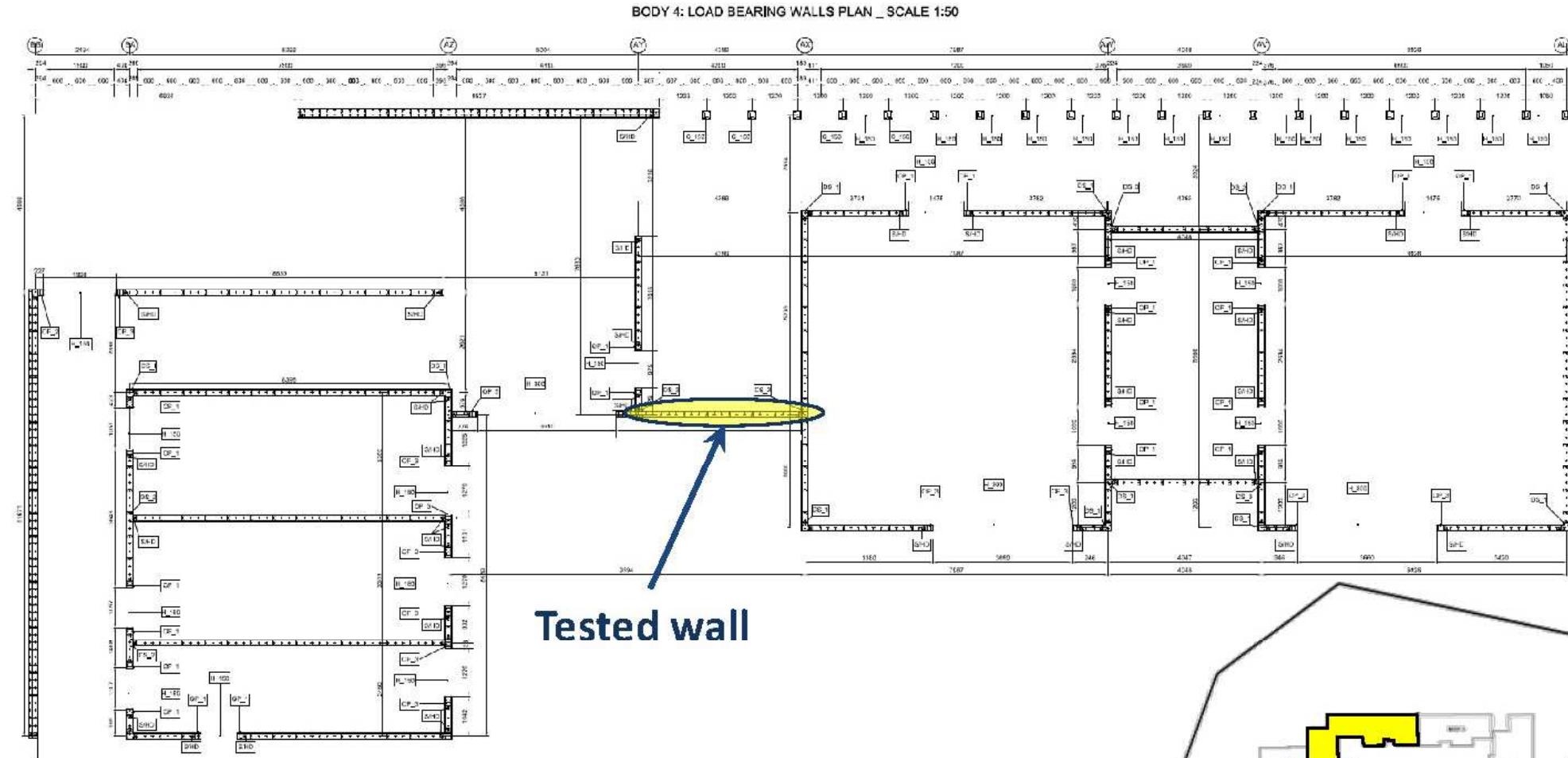
### Materials

20 OSB panel shear tests

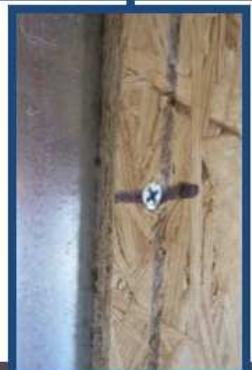
40 Self-drilling screw shear tests



## Experimental program: wall tests



## Experimental program: wall tests



### Wall configuration



**Wall dimension:** 4.80 x 4.00 m

#### Steel Framing:

Tracks U153x50x1.50  
Studs C150x50x15x1.50  
(S350GD+Z grade)

#### Sheathing:

OSB/3 panel 9 mm thick

#### Sheathing-to-frame connections:

Self-drilling screw diameter 4.2 mm  
bugle head (spacing: 100 mm at perimeter, 600 mm in field)

#### Frame-to-foundation connections:

Hold-down devices (specifically designed in S700 steel grade)  
connected to studs by 4 M16 bolts  
and to foundation by HIT RE 500 with  
M24 HAS adhesive-bonded anchors

#### Steel-to-steel connections:

Self-drilling screw diameter 4.8 mm  
lath head

## Experimental program: wall tests

**Test set-up**



**Loads distribution:**

2 coupled steel beams (RHS and IPE 500) set on wall top



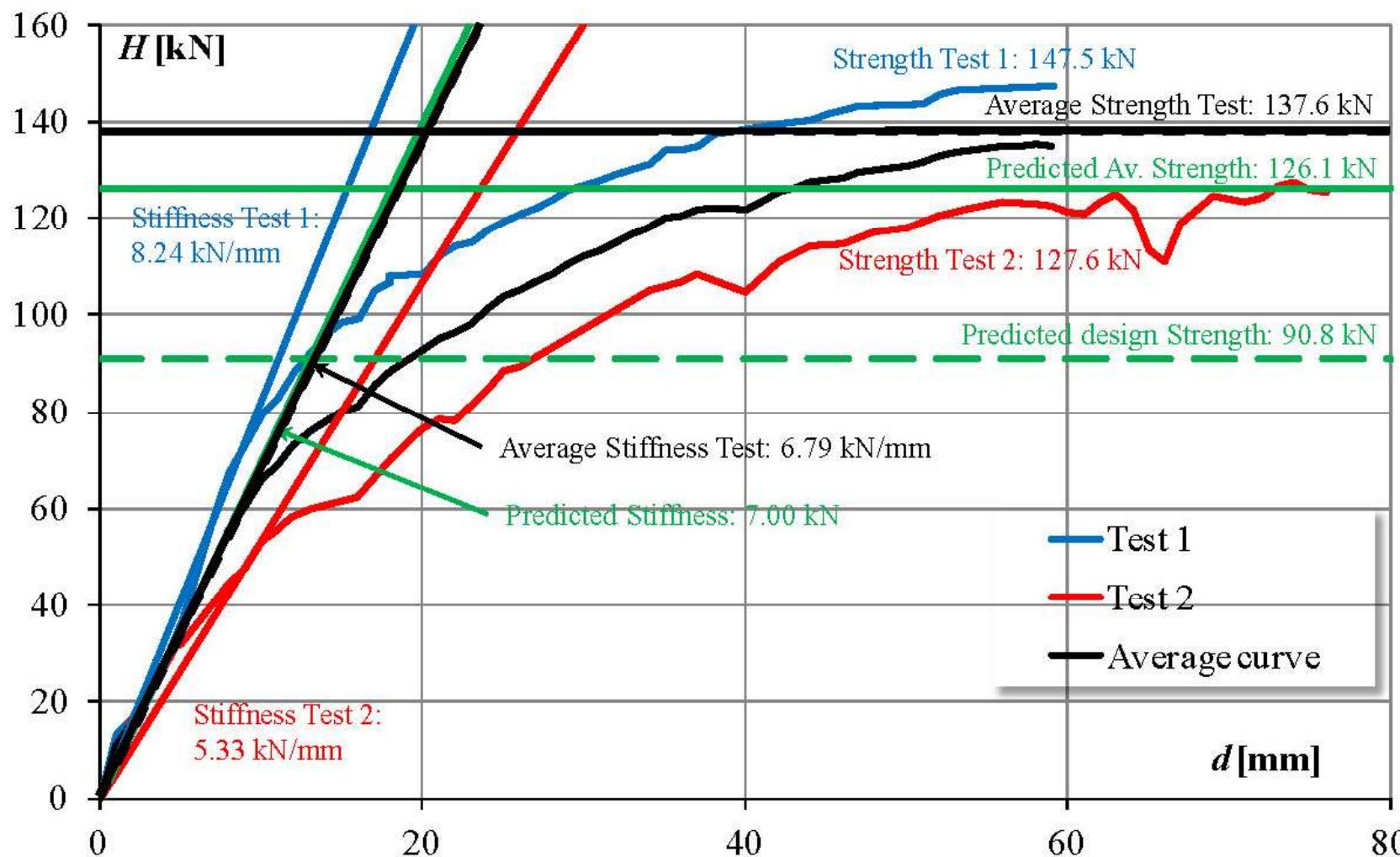
## Experimental program: wall tests

### Test Results – Deformated condition and collapse mechanism



## Experimental program: wall tests

### Test Results – Force vs. Displacement curve and comparison with design prediction



The average test strength is 9.1% higher than predicted average value and is 51.5% higher than the assumed design value. (Average safety factor = 1.52)

The average test stiffness is 3% lower than the predicted value.

### 3. What's next



---

**1. Considerazioni introduttive**

**2. La normativa e la ricerca**

**3. What's next?**

**4. Conclusioni**



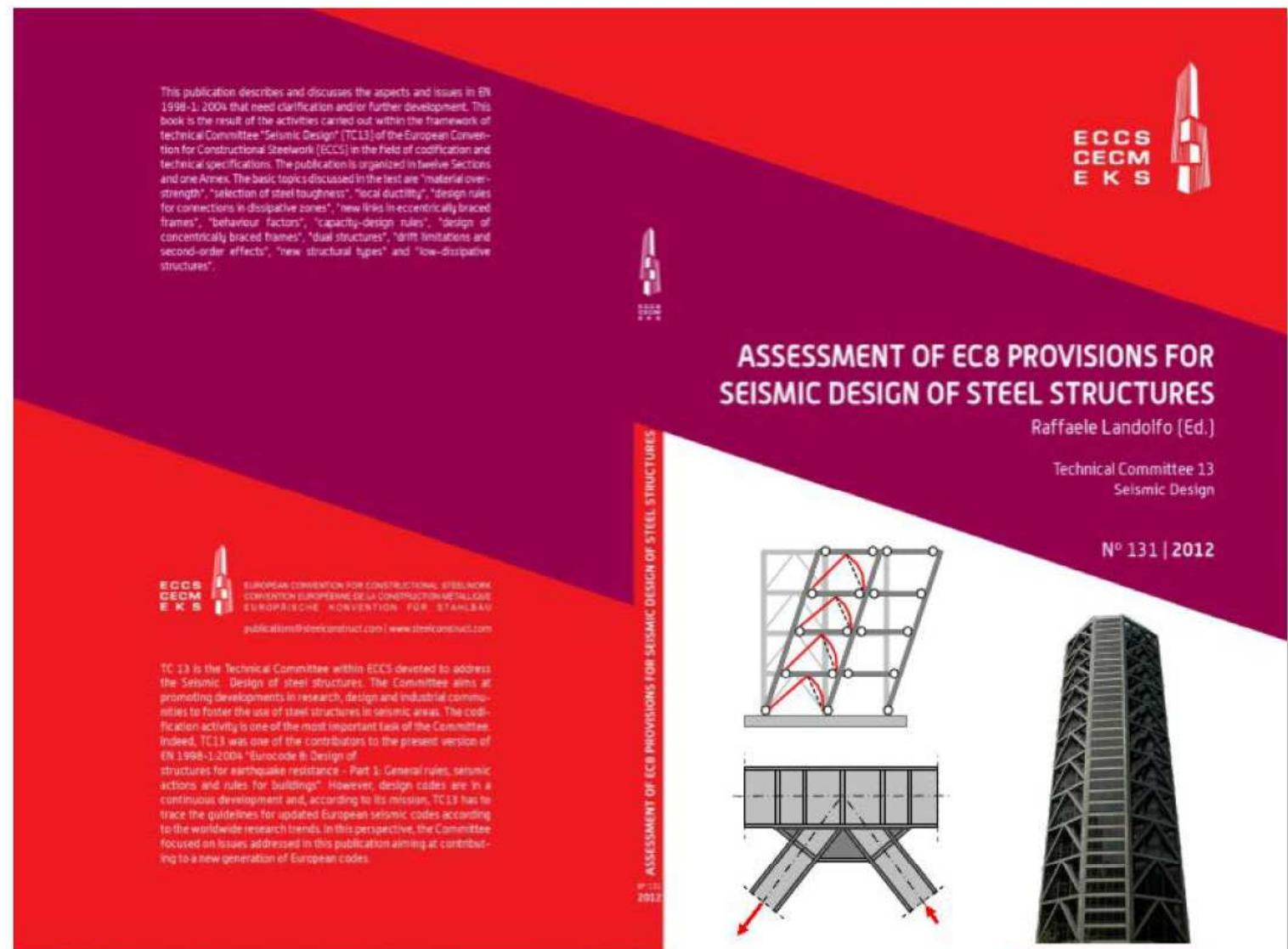
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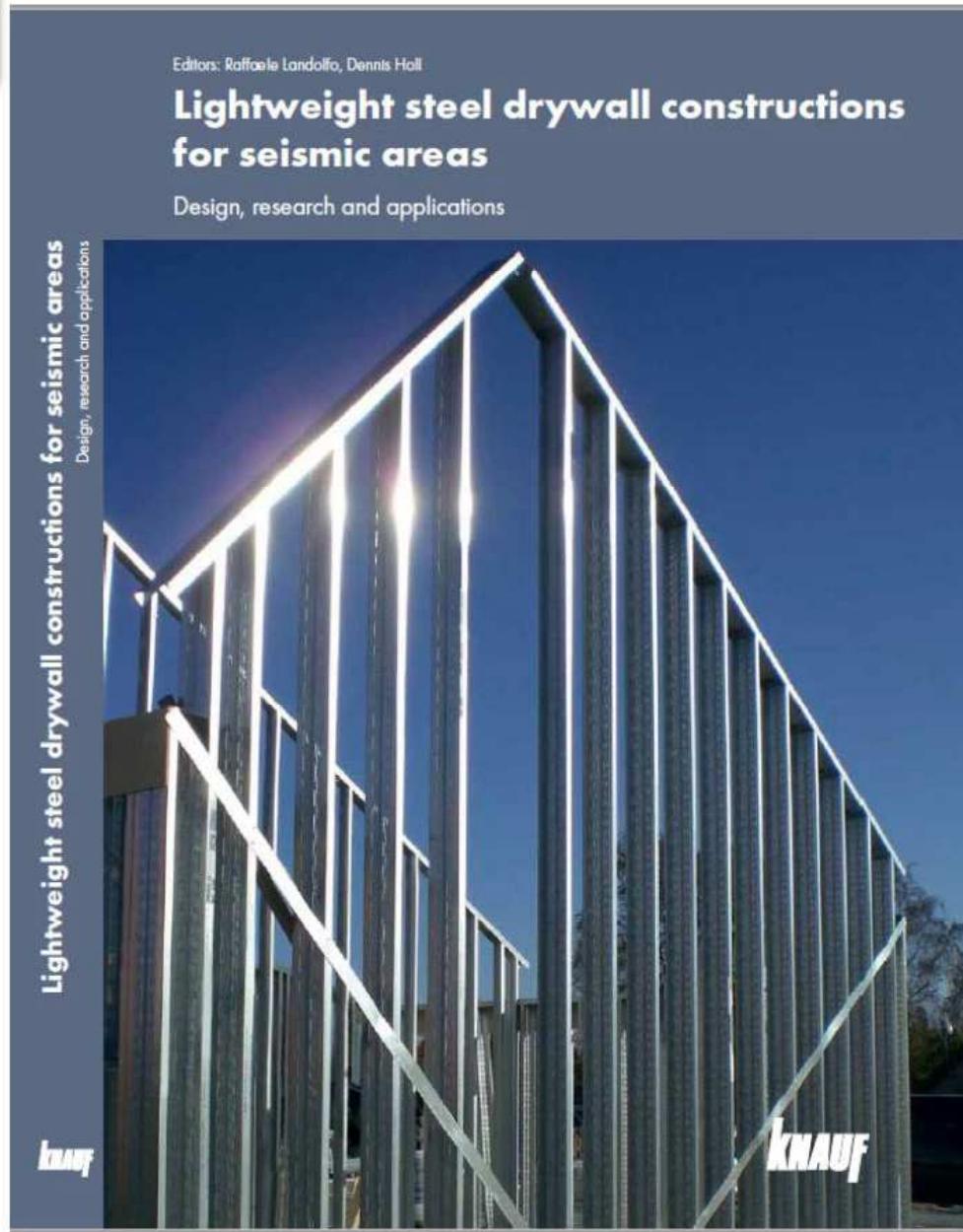
# European Convention for Constructional Steelwork

## Dissemination activities

TC13 Publication:

*Assessment of EC8  
Provisions for Seismic  
Design of Steel Structures*





# Lightweight steel drywall constructions for seismic areas

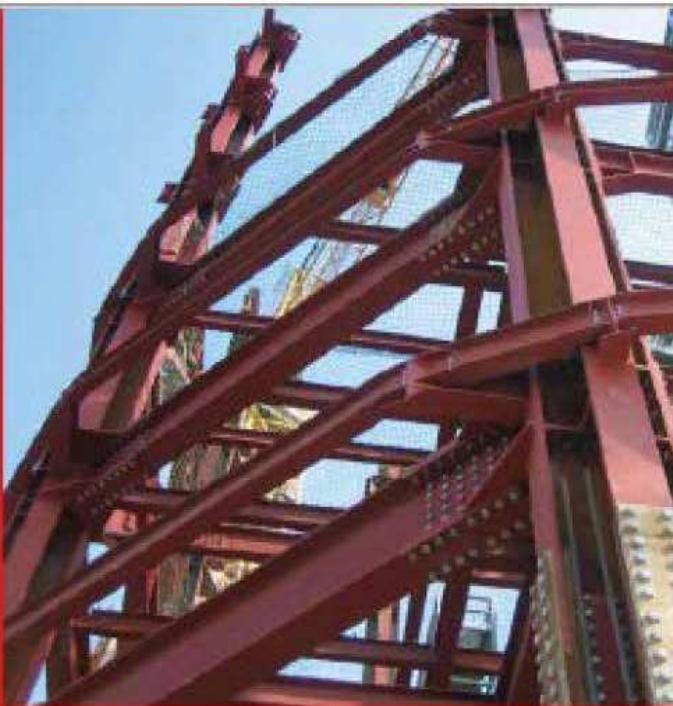
## Design, research and applications

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Georg KRÄME  
Vicenzo MACILLO  
Tatiana PALI

**knauf**





**Design of Steel Structures  
for Buildings in Seismic Areas**

Eurocode 8: Design of Structures  
for Earthquake Resistance  
Part 1: General Rules, Seismic Action  
and Rules for Buildings

Raffaele Landolfo  
Federico Mazzolani  
Dan Dubina  
Luis Simões da Silva  
Mario D'Aniello



ECCS Eurocode Design Manuals

## **DESIGN OF STEEL STRUCTURES FOR BUILDINGS IN SEISMIC AREAS**

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M. D'Aniello

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Grazie per l'attenzione



Prof. Raffaele Landolfo



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