University of Naples Federico II

Department of Structures for Engineering and Architecture

Seismic behavior of lightweight structures in steel

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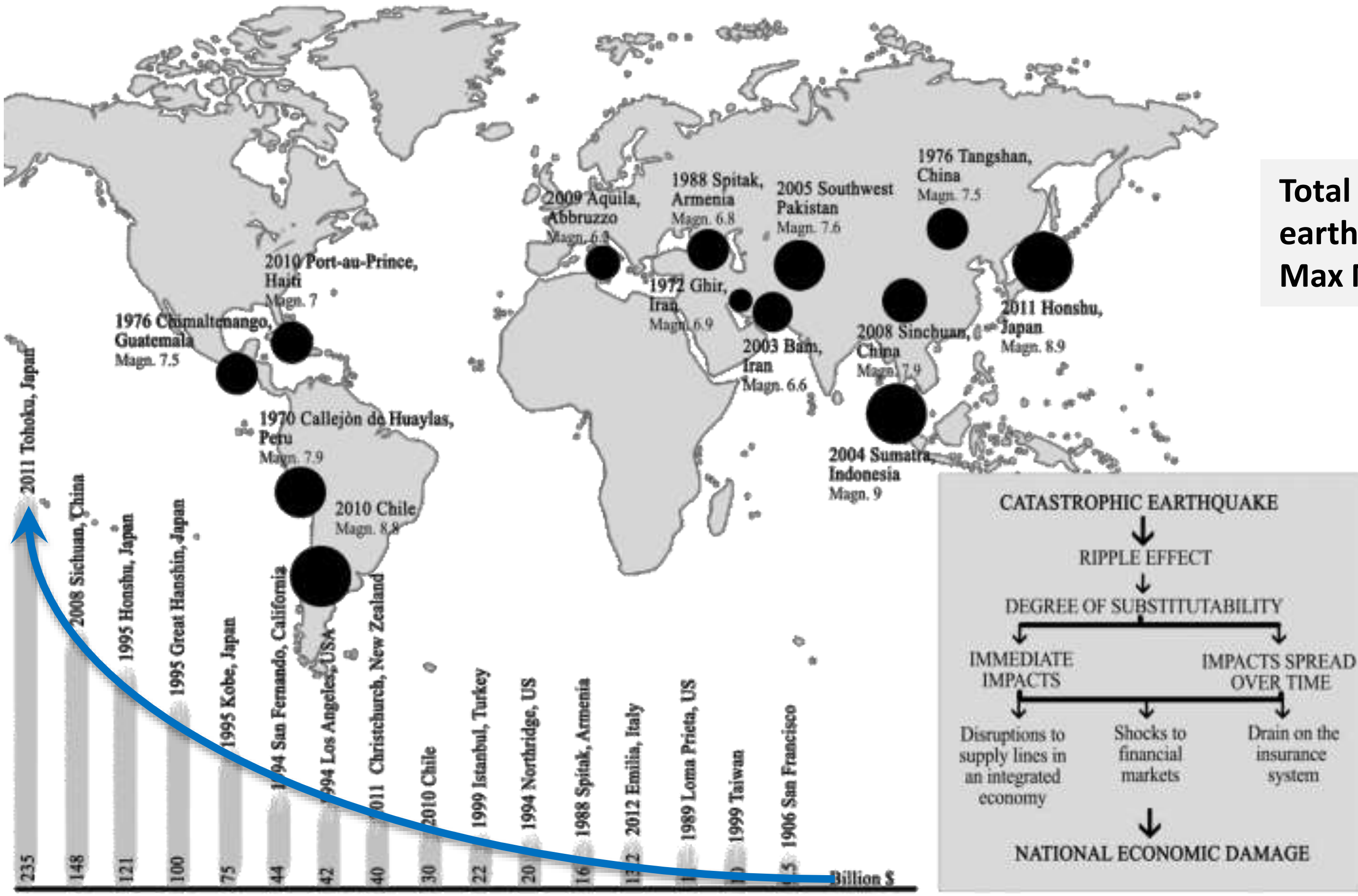
Why to defend against earthquakes?

Earthquakes caused approximately **2.5 million of deaths** and over **2.9 trillion US dollars** damage since 1900



Total number of earthquakes (M>4.5): 318
Max M: 6.3

(Source: http://www.bbc.com)

How to defend against earthquakes?

SEISMIC RISK

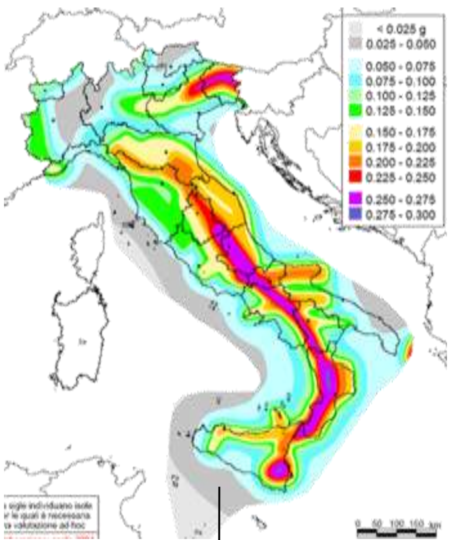
- **Hazard**

- **Exposure**

- **Vulnerability**

MITIGATION OF SEISMIC RISK

Evaluation of the seismic hazard



Seismic classification

Reduction of the sesmic vulnerabilities

Seismic design of new structures

Seismic refurbishment of existing structures

Steel is an optimal solution…

“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

Why steel structures have a good behavior under seismic actions?

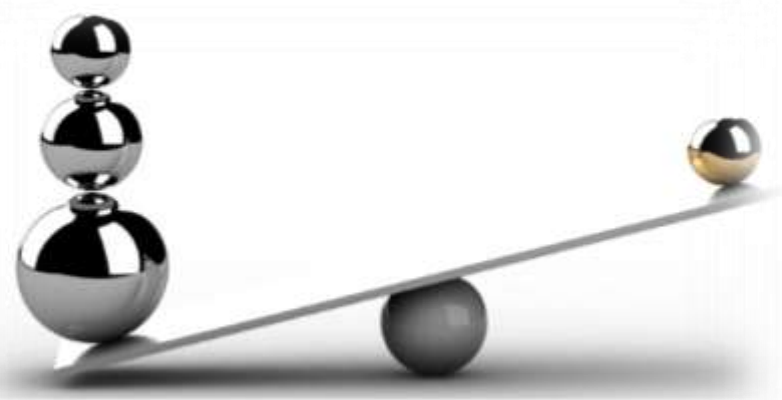
Tokyo, March 2011

Peculiarities of steel structures:

Resistance

Ductility

Lightness



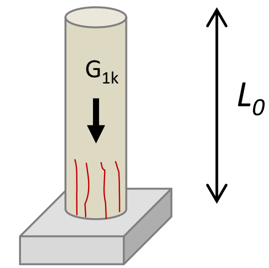
Steel structures — VS — Other structures

Resistance

The lightweight of steel structures is due to the high structural efficiency offered by steel material

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

Specific strength or strength-to-weight ratio of materials (m)

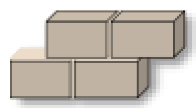


free breaking length or self-support length

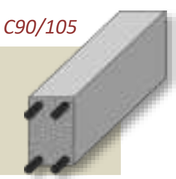
N.B. L_0 is the maximum length of a vertical column of the material (assuming a fixed cross-section) that could suspend its own weight when supported only at the top.

CONSTRUCTION MATERIALS

- **300** — Masorny — *M15; f_{bk}40*

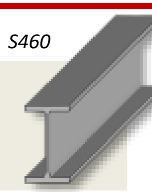


- **1000** / *2500* — Ordinary reinforced-concrete NSC (*C40/50*) — High-strength reinforced concrete HPC (*C90/105*)



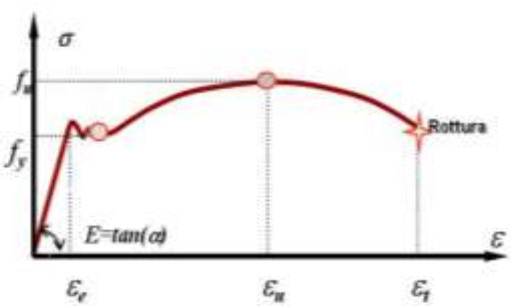
- **3800** — Wood — *GL36h*

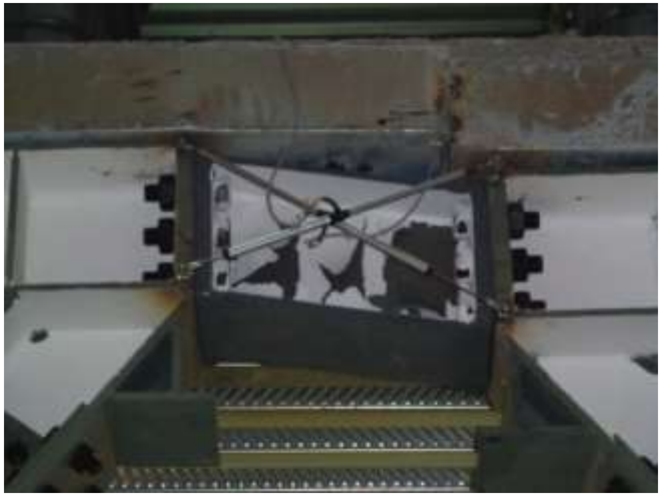
- **5500** / *11000* — Steel — *S460* / *S960*

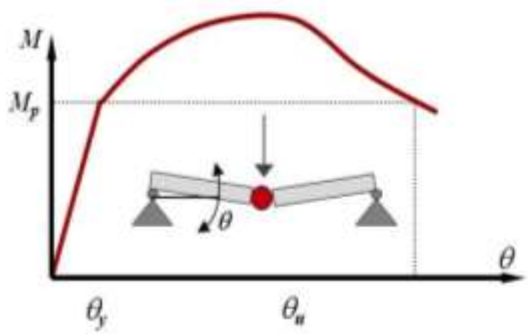


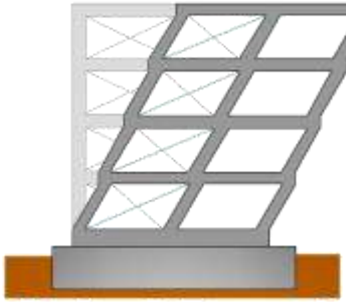
Ductility

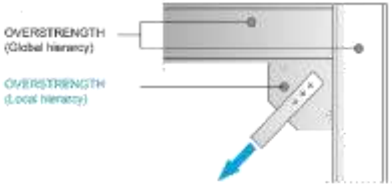
Material ductility | **Local ductility** | **System ductility**

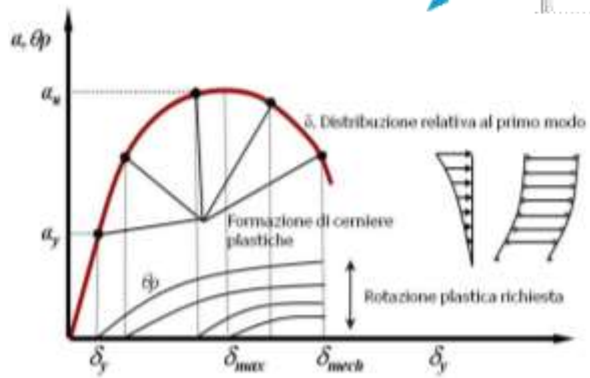




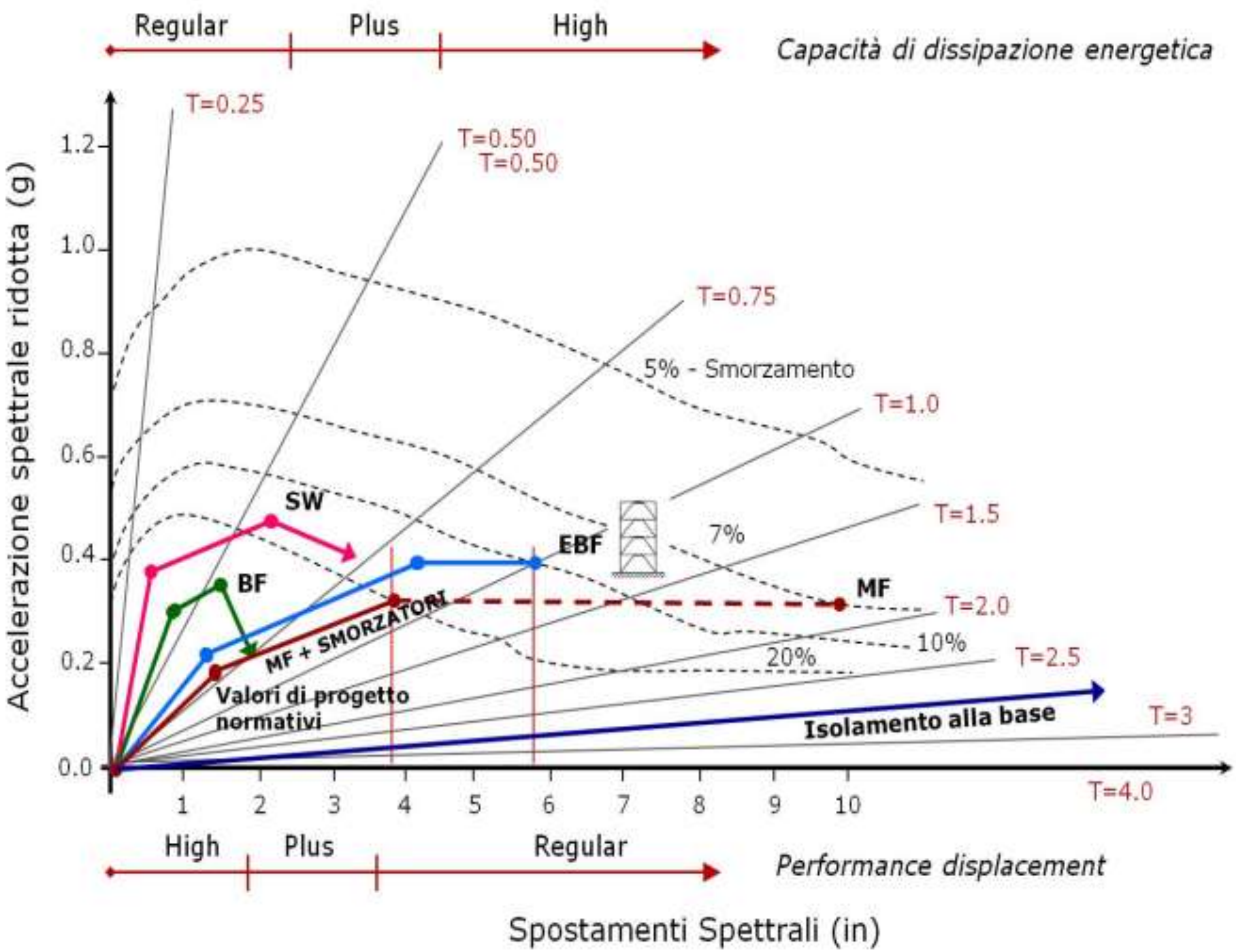




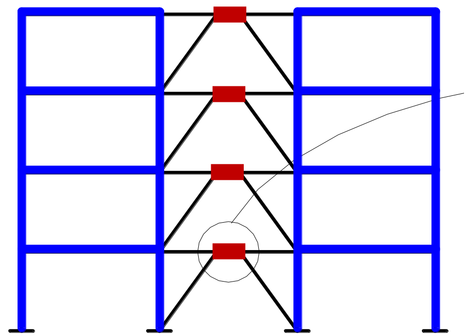


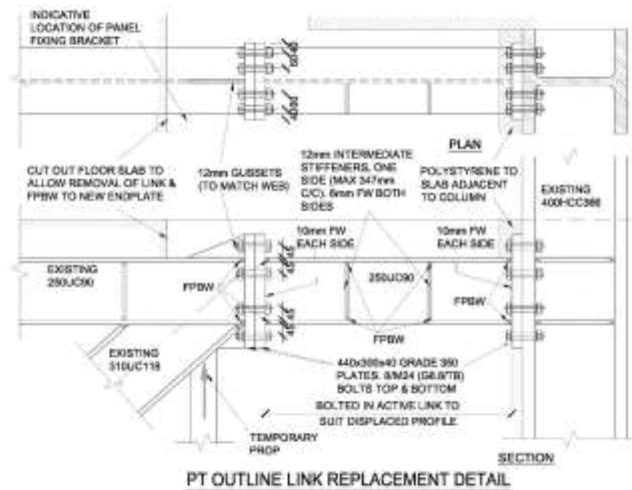


Ductility


Regular
Plus
High
Capacità di dissipazione energetica
T=0.25
T=0.50
T=0.50
T=0.75
5% - Smorzamento
T=1.0
T=1.5
7%
T=2.0
10%
20%
T=2.5
T=3
T=4.0
SW
BF
EBF
MF
MF + SMORZATORI
Valori di progetto normativi
Isolamento alla base
Accelerazione spettrale ridotta (g)
0.0
0.2
0.4
0.6
0.8
1.0
1.2
1
2
3
4
5
6
7
8
9
10
High
Plus
Regular
Performance displacement
Spostamenti Spettrali (in)

Damage control and reparability





Pacific Tower, Christchurch

Removable link

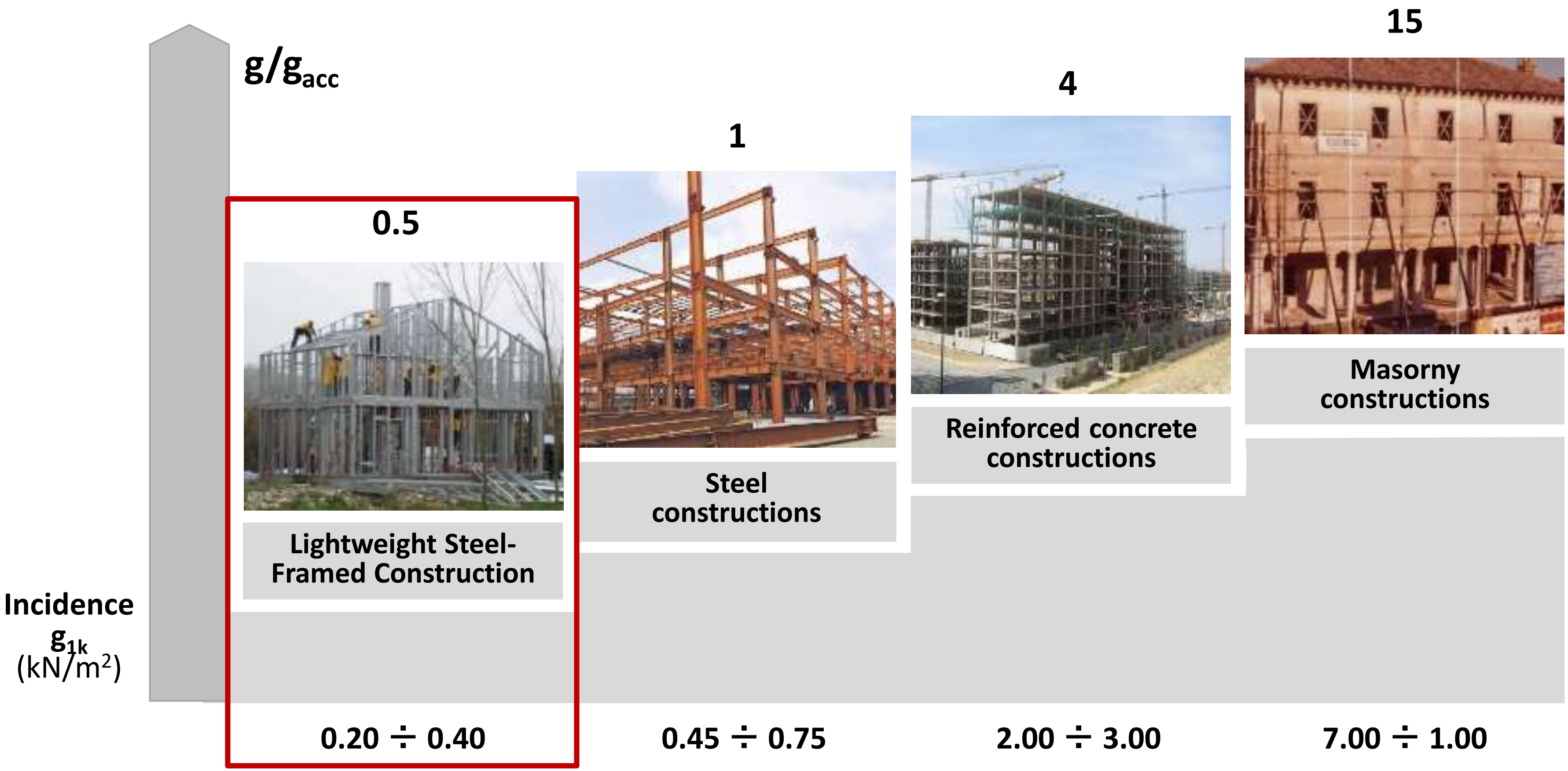
Source:
G Charles Clifton, Gregory A MacRae, Lessons from the Field; Steel Structure Performance in Earthquakes in New Zealand from 2010 to 2016. *Key Engineering Materials 763:61-71*

Lightness

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

Lightness

Comparison between the weight incidences per unit area for different structural skeletons

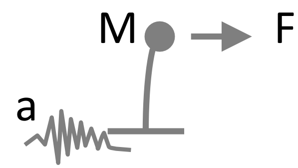


Seismic safety

The inherent lightweight of Lightweight Steel-Framed Constructions should be adopted as design strategy for ensuring good seismic performances of these systems.

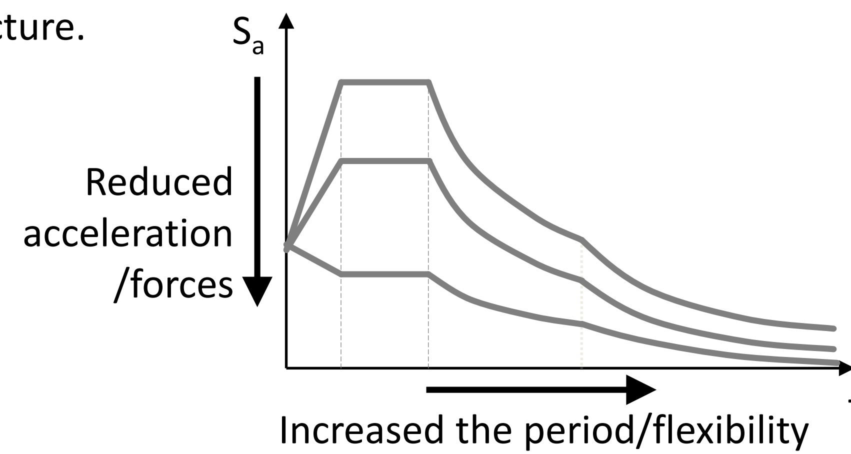
1. Lower weight → Reduced acceleration /forces

Lightweight Steel Constructions with less mass are an advantage in seismic design. Since earthquake forces are inertia force due to accelerating mass: the lower is the mass, the lower is the seismic design forces.



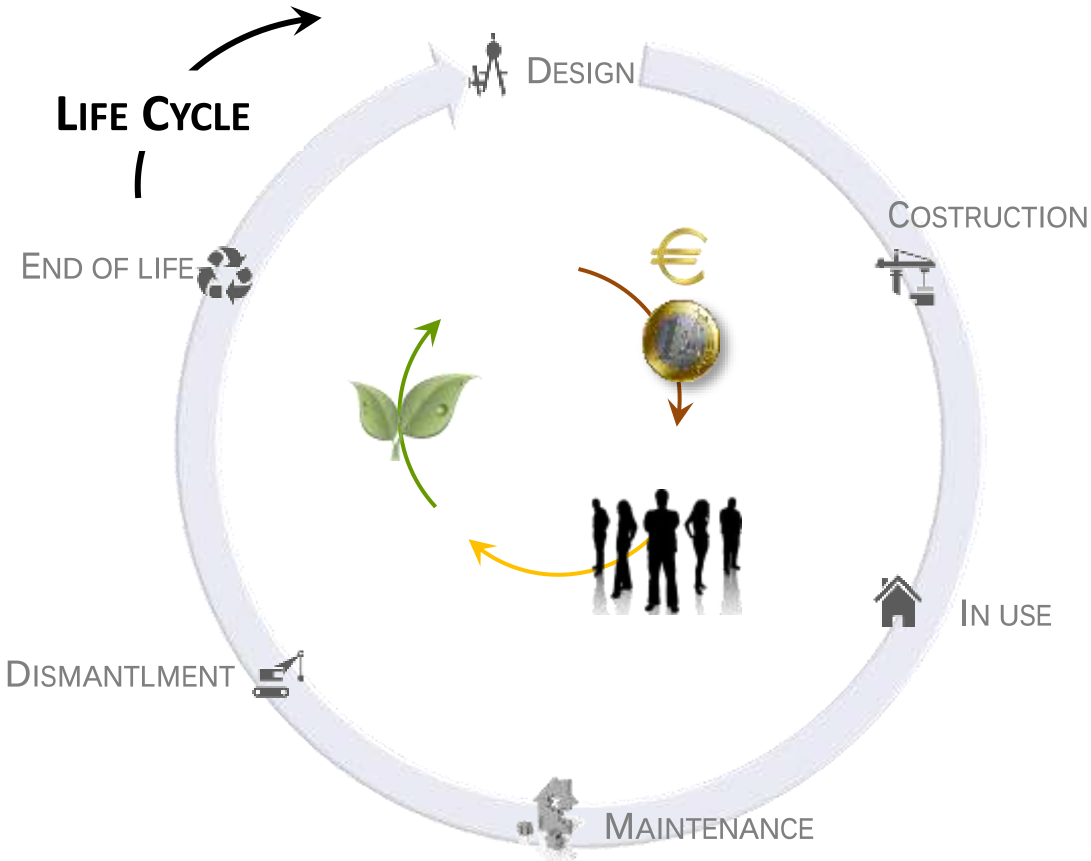
2. Flexibility → Increased the period

Steel structures are generally more flexible than other types of structure.



Structural sustainability

The challenge of sustainability of structures is to maximize the **mechanical, durability, economic and environmental performance** of a structure, during the **whole life-cycle**, reducing, at the same time, the adverse impacts played on planet, people and economy.



Multi-performance based approach

1 Multi-performance

- Enhanced safety and reliability
- Reduced environmental impacts
- Optimized life-cycle costs

2 Life-cycle oriented

The basic requirements shall be achieved during the whole life-cycle of the construction

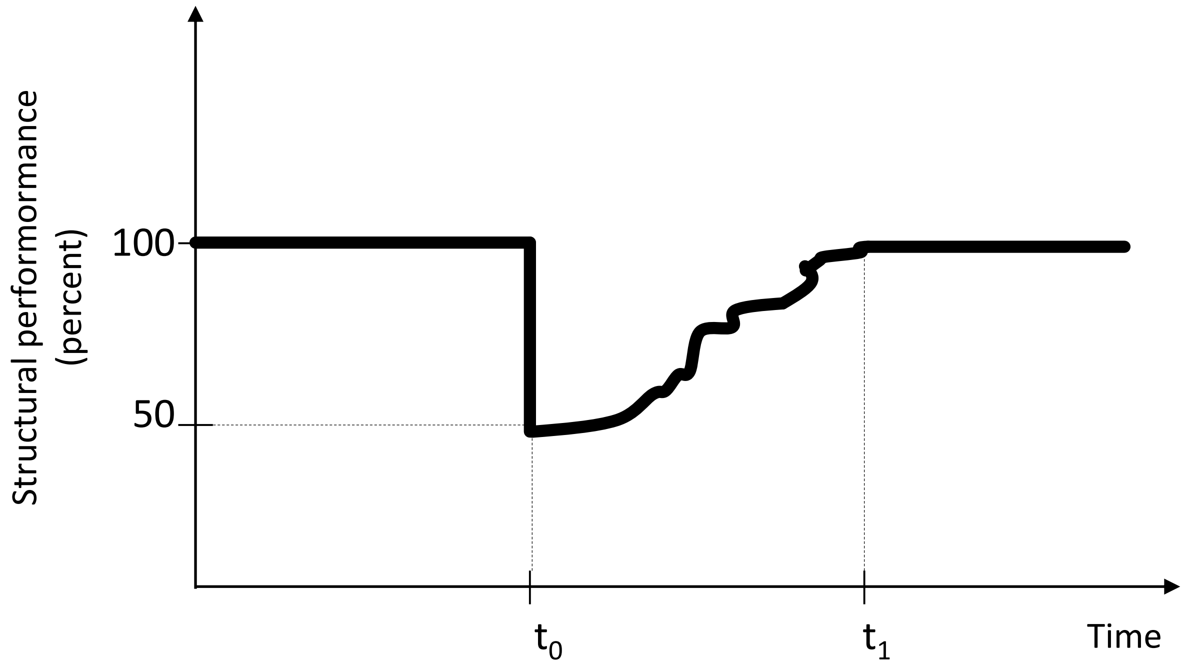
3 Based on quantitative methodologies

Performance requirements shall be verified according to quantitative methodologies

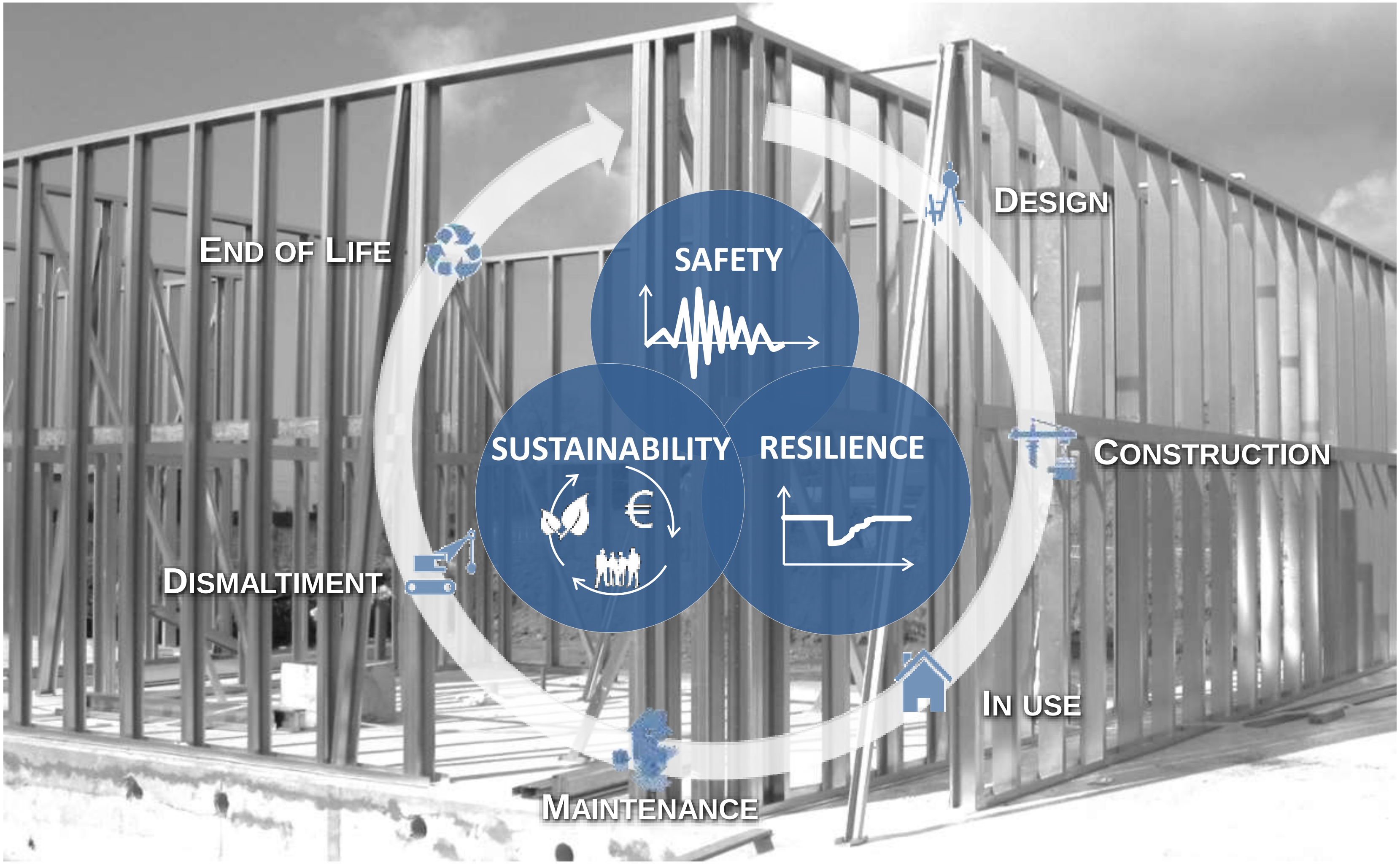
Structural resilience

A resilient structure is one that shows:

- Reduced failure probabilities,
- Reduced consequences from failures, in terms of lives lost, damage, and negative economic and social consequences,
- Reduced time to recovery (restoration of a specific system or set of systems to their “normal” level of functional performance)



Since the new trends are going towards integrated solutions in terms of eco-efficiency, structural performance and economic aspects, the Lightweight Steel-Framed Constructions represent the **optimal solution for both structural and non-structural applications.**



Potentialities

Comfort and design 

Safety and Durability 

Lower cost 

Eco-efficiency 

Classification

STICK-BUILT CONSTRUCTIONS

PANELIZED CONSTRUCTIONS

MODULAR CONSTRUCTIONS

Prefabrication level

Stick-Built Constructions

Stick-built constructions are obtained by assembling on site, a modest number of members (studs, joists and tracks) and sheathing panels, which are connected together by mechanical fasteners (screws and nails).

Project data

Location:
Rzeskov, Romania

Architects:
SAM IMPEX S.R.L.

Client:
ArcelorMittal Foundation & Habitat for Humanit

Structural project:
COBIM & ArcelorMittal R&D Liège

Typology:
Residential-Extension of a RC building

Casa Buna, Comanesti, Romania (2008)

Panelized Constructions

Panelized constructions are made of two-dimensional elements (wall and floor sub-frames and roof trusses), which are prefabricated in shop. Thermal insulation and some of the lining and finishing materials may also be applied to the steel sub-frame to form panels and to reduce execution times. This system is particularly indicate to build houses characterized by repetitive elements.

Project data

Location:
Lillie Road, London

Architects:
Feilden Clegg Bradley Architects

Client:
Peabody Trust

Structural project:
Michael Barclay Partnership

Typology:
Residential-Multi-story building


Beaufort Court, Lillie Road, London (2003)

Modular Constructions

Modular constructions use pre-engineered modular units, made out by the assembling of frames completed of any finishing (e.g. doors, windows, and any finishing material) in the shop and by the vertical and horizontal addition of the units on site.

Project data

Location:
Lecco, Italy

Architects:
Ar.De.A., Montanelli

Client:
Politecnico di Milano

Typology:
University and Campus-Multi-storey building

Campus Point POLIM, Lecco, 2007

Main advantages in using Lightweight Steel-Framed constructions

Technological issues: fastening systems

Mechanical fasteners

SCREWS

NAILS

BOLTS

BLIND RIVETS

SPECIAL MECHANICAL FASTENERS

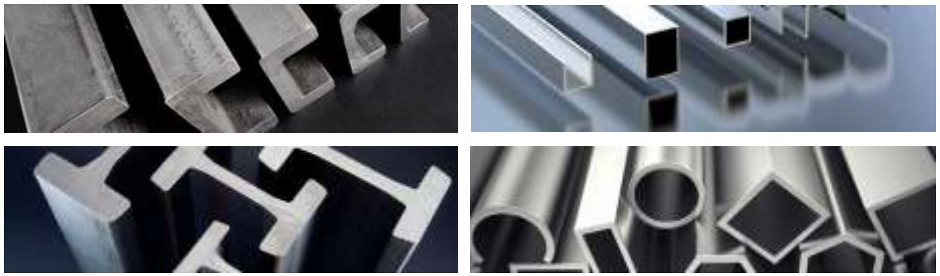
Welding

Adhesives

Anchors

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

Hot-rolled profiles




HE


RHS


IPE

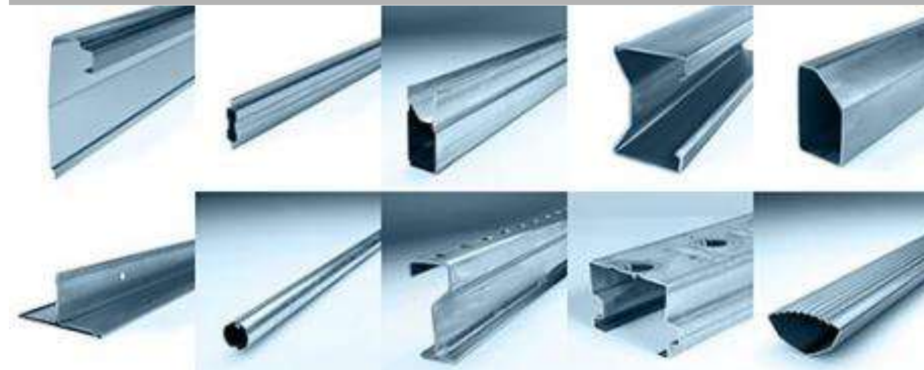

SHS

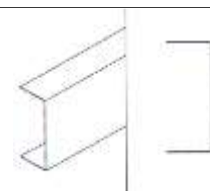
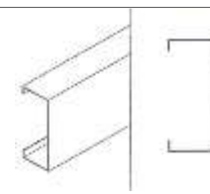
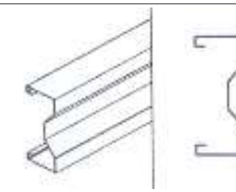
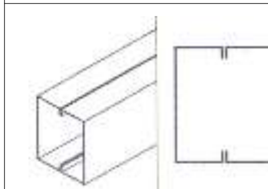
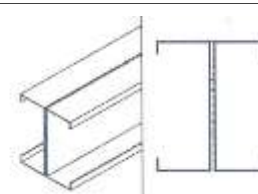

UPE/UPN


OHS

4 mm ≤ t ≤ 40 ÷ 45 mm

Cold-formed steel profiles



Unlipped 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 mm < t ≤ 3 mm

Technological issues: cold-forming manufacturing methods

Cold formed steel profiles are obtained from relatively thin steel sheet, that can be subjected to corrosion even in inland areas, and it corrodes rapidly in salt air.
Therefore, CFS members typically utilizes hot-dip galvanized steel

Table 2.1
Zinc Coating Weights (Mass) / Thickness

Coating Designation	Minimum Requirement Total Both Sides		Thickness Nominal per Side	
	(oz/ft^2)	(g/m^2)	(mils)	(microns)
Zinc (Galvanized)				
G40/Z120	0.40	120	0.34	8.5
G60/Z180	0.60	180	0.51	12.7
G90/Z275	0.90	275	0.77	19.4
55% Aluminum-Zinc				
AZ50/AZM150	0.50	150	0.80	20.0

Table 2.2
Minimum Coating Weight Requirements

Framing Member Designation	Zinc (Galvanized)	55% Aluminum-Zinc
Structural	G60/Z180	AZ50/AZM150
Non-Structural	G40/Z120	AZ50/AZM150

CFSEI (202) 785-2022

Technological issues: cold-forming manufacturing methods

1. Cold-rolling

The shape is obtained from a strip which is formed gradually, by feeding it continuously through successive pairs of rolls which act as male and female dies.

Main advantages of continuous process:

- high production capacity;
- ability to maintain fine surface finishes during roll forming operations (particularly important where pre-galvanized steel or steel pre-coated with plastic are utilized).

2. Press braking or folding

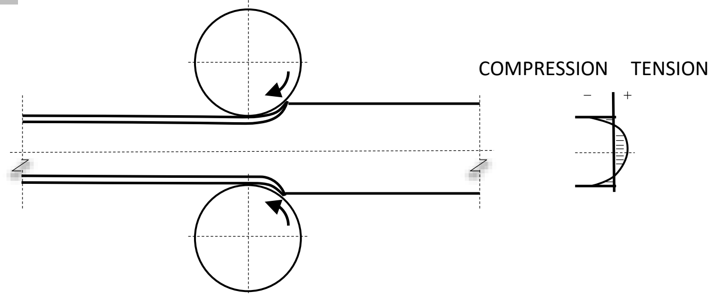
Short lengths of strip are fed into the brake and bent or pressed round shaped dies to form the final shape. Usually each bend is formed separately and the complexity of shape is limited to that into which the die can fit.

Main advantages of discontinuous process:

- more convenient for small series of sections with length ≤ 6 m
- for sections having relatively simple configurations.

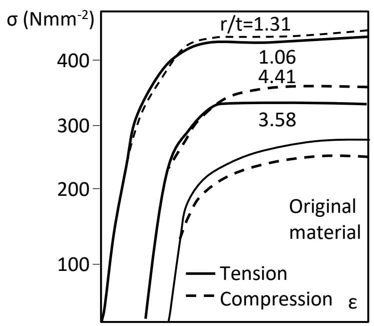
Technological issues: effects of cold-forming process

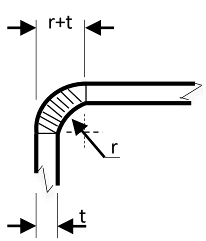
1. Residual stresses



Cold-rolling produces **mechanical residual stresses** which vary across the sheet thickness. The outer fibres tend to elongate, while the centre tends to remain undeformed.
The effect of such a distribution **can be neglected in most cases**.

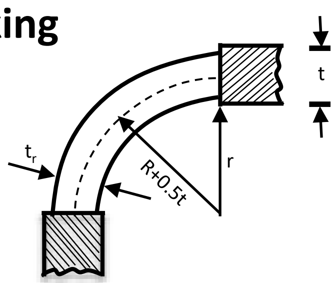
2. Strain-hardening





The cold-forming process produces an **increase in the elastic limit of the material** compared to its original value and its increment is proportional to the severity of folding, expressed as the ratio between fillet radius r and sheet thickness t.

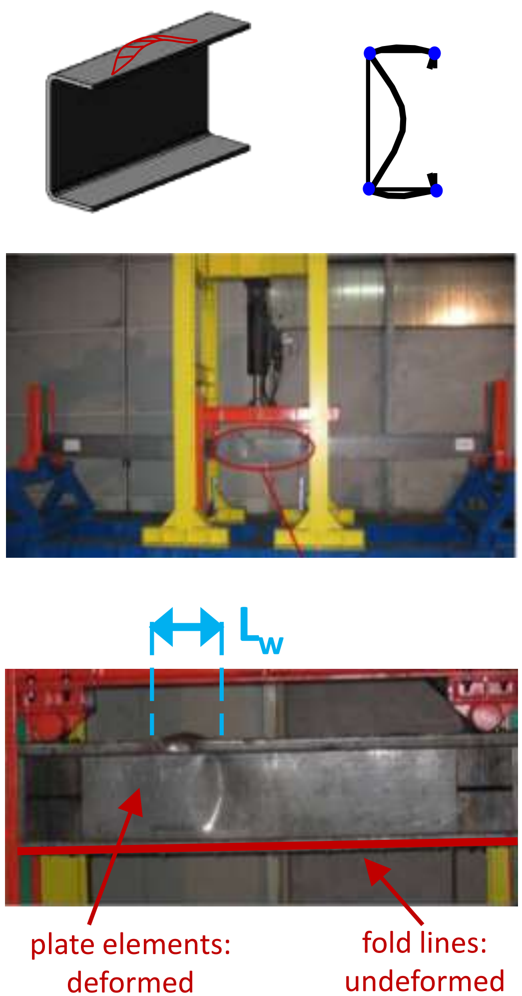
3. Necking



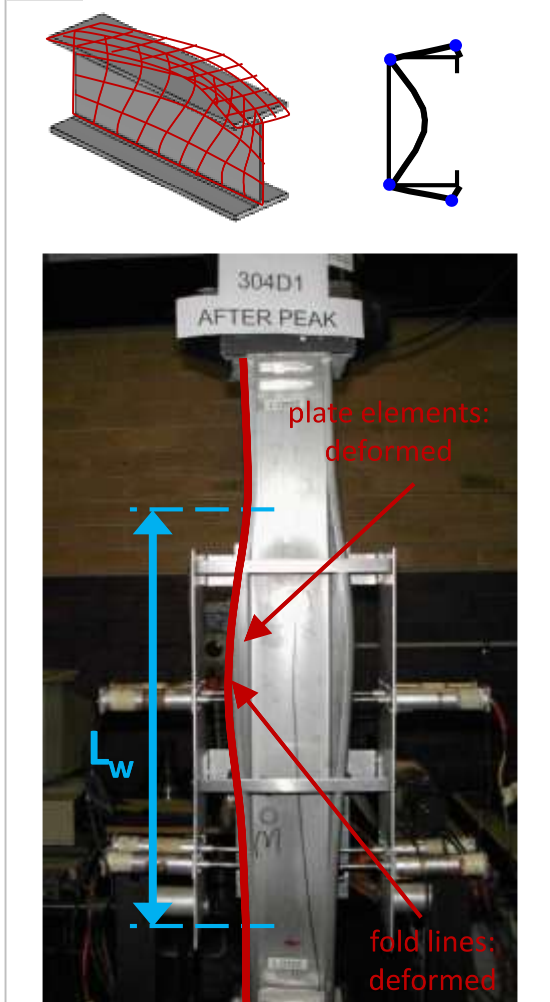
Forming corners of small radius can have the effect of **producing "thinning" of the corners** and this can have an effect on the section properties, but this effect is generally small since the corners usually are just a small proportion of the overall cross-sectional area.

Effects of the high lightweight of CFS profiles: instability phenomena

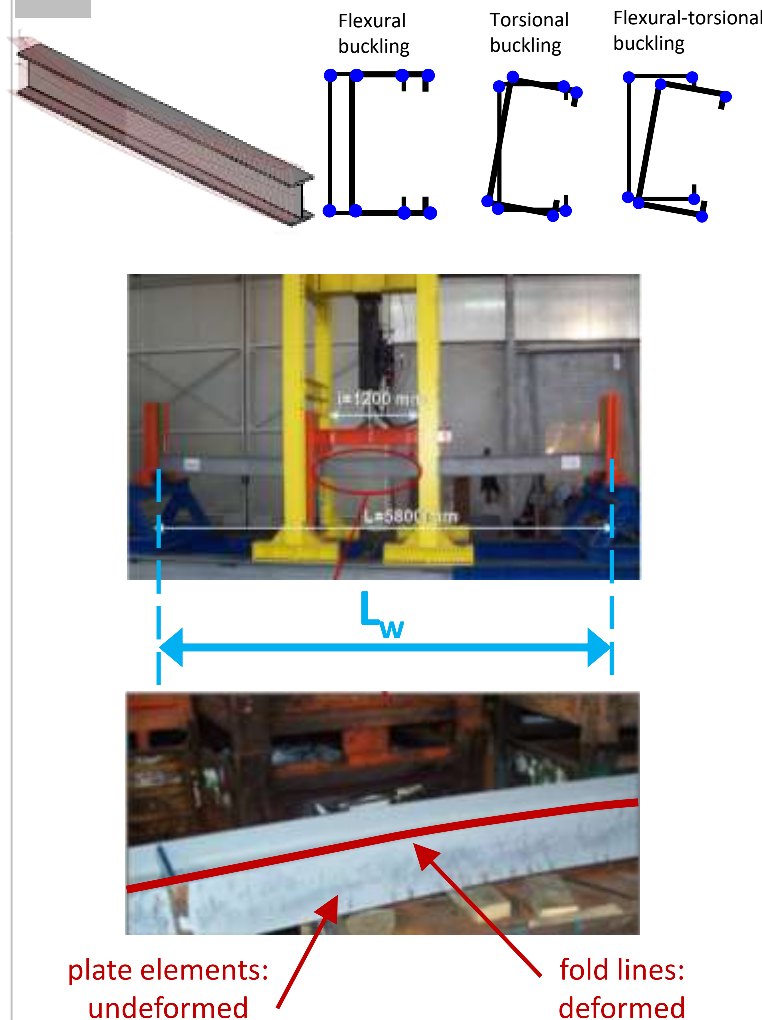
1. Local buckling



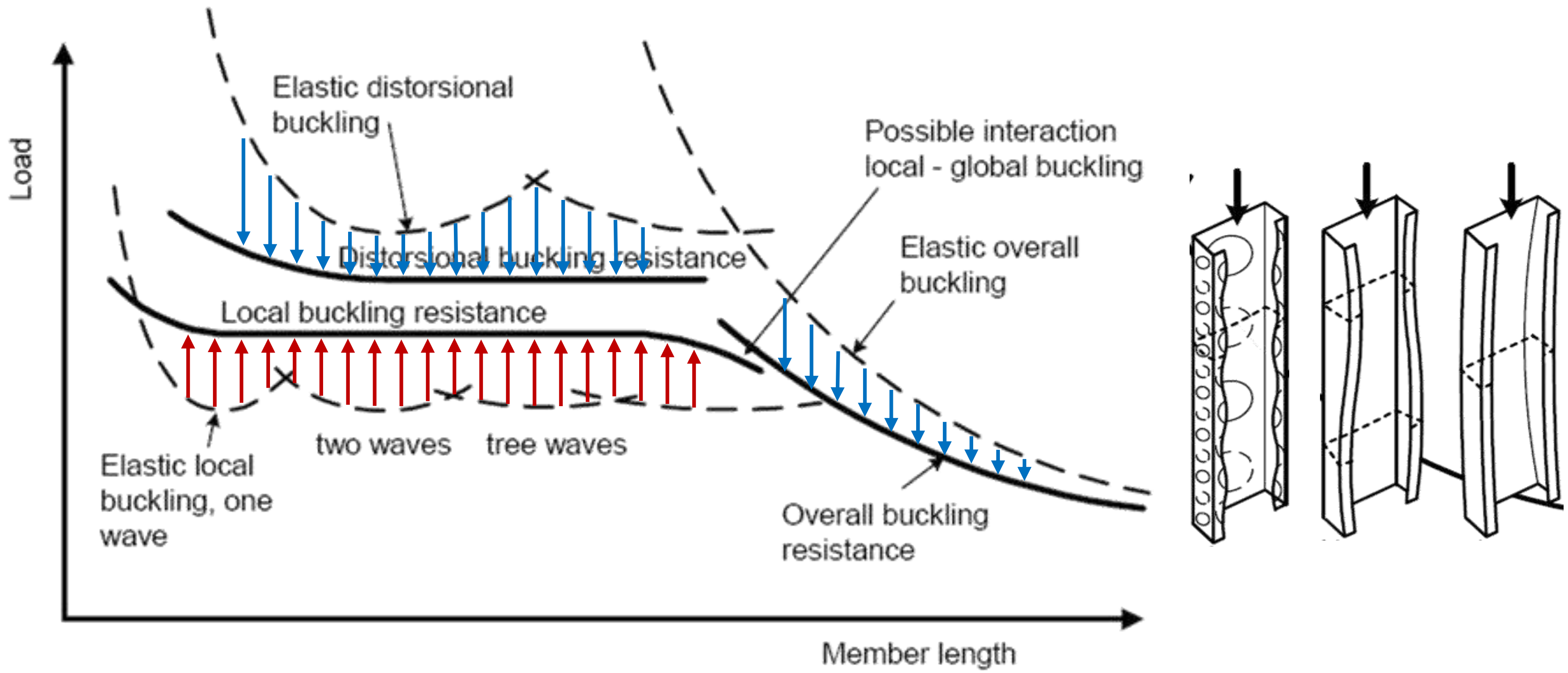
2. Distortional buckling



3. Global buckling

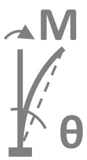


Effects of the high lightweight of CFS profiles: instability phenomena



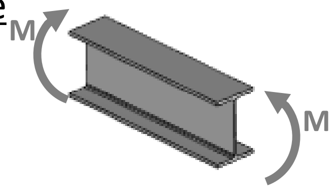
The influence of local buckling: the need of specific calculation methods

The European code classifies the cross-sections according to their capability to reach the following limit states:



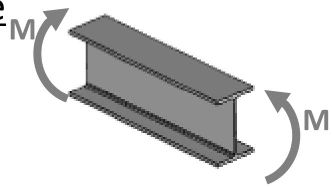
Class 1: ductile cross-sections
collapse limit state

Class 2: compact cross-sections
plastic limit state



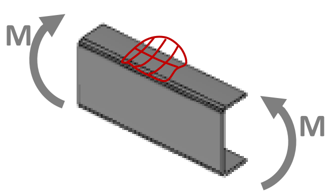
$$M_{c,Rd} = \frac{W_{pl} \cdot f_y}{\gamma_{M0}}$$

Class 3: semi-compact cross-sections
elastic limit state

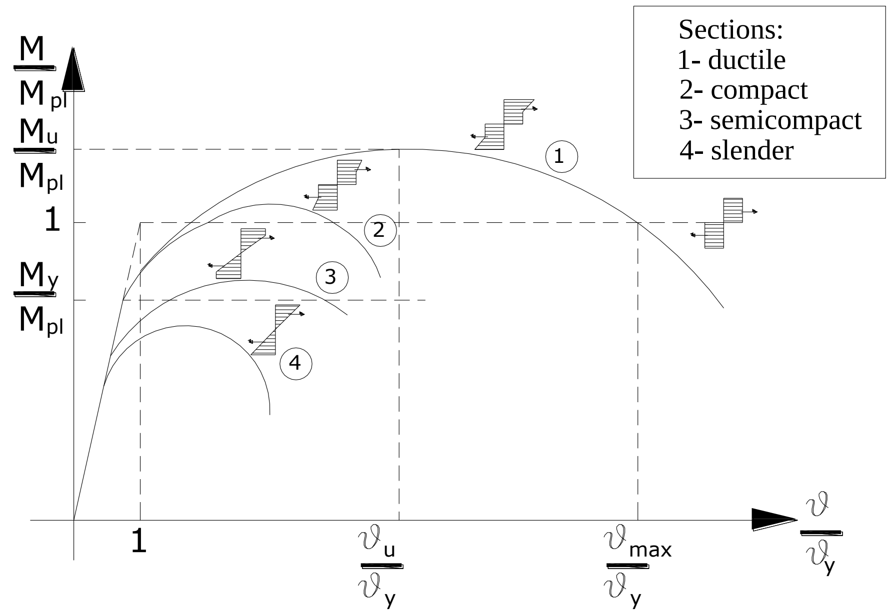


$$M_{c,Rd} = \frac{W_{el} \cdot f_y}{\gamma_{M0}}$$

Class 4: slender cross-sections
elastic buckling limit state

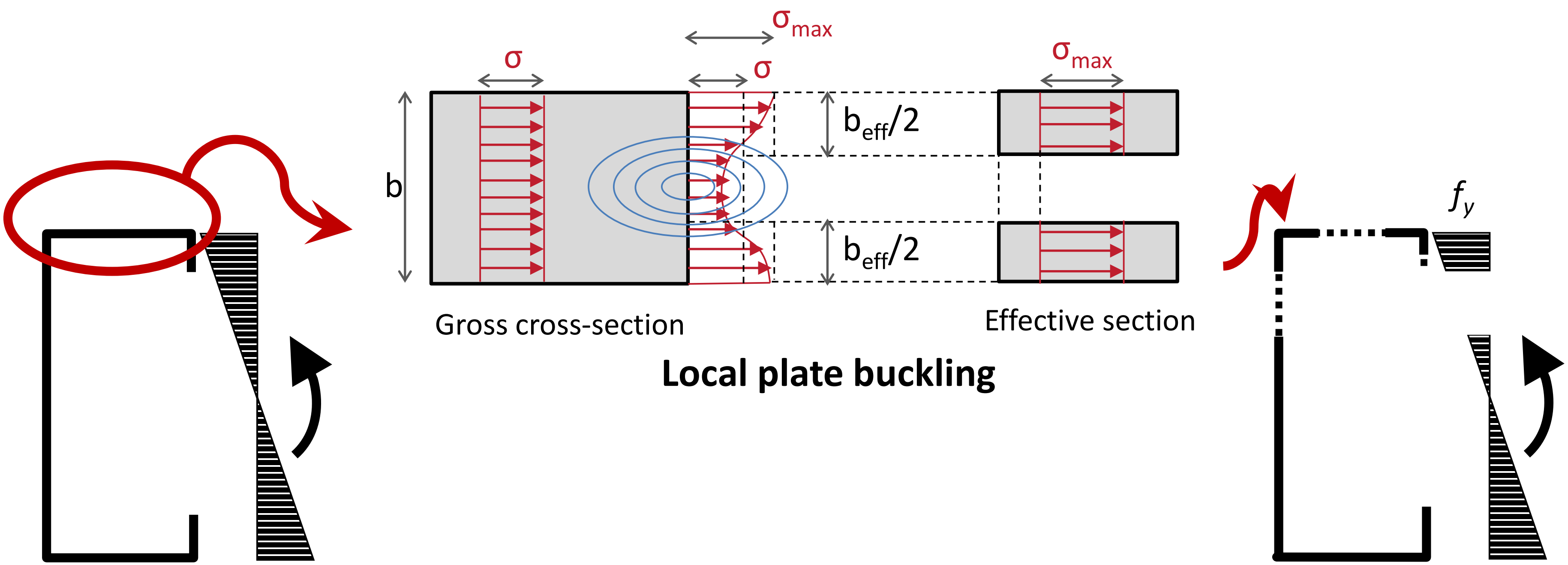


$$M_{c,Rd} = \frac{W_{eff} \cdot f_y}{\gamma_{M0}}$$



The influence of local buckling: the need of specific calculation methods

The local plate buckling leads to reductions in the effectiveness of the plates that comprise a cross-section. The Effective Width Method takes into account of the reduction from the gross cross-section to the effective cross-section

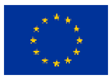


Gross Section

Effective Section

$$b_{eff} = \rho(\bar{\lambda}_p)b$$

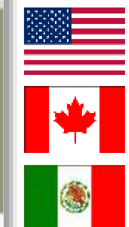
Reference codes for CFS profiles	Reference seismic codes for structural applications

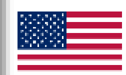
- **CEN (2006), EN 1993-1-3,** *Eurocode 3: Design of steel structures – Part 1-3: General rules - Supplementary rules for cold formed thin gauge members and sheeting.* European Committee for Standardization, Bruxelles, 2006.


- **CEN (2006), EN 1993-1-5,** *Eurocode 3: Design of steel structures - Part 1-5: Plated structural elements.* European Committee for Standardization, Bruxelles, 2006.


- **CEN (2003), EN 1998-1,** *Eurocode 8: Design of structures for earthquake resistance – Part 1: General rules, seismic actions and rules for buildings.* European Committee for Standardization, Bruxelles, 2003.

- **AISI (2015), AISI S400-15,** *North American Standard for Seismic Design of Cold-Formed Steel Structural Systems.* AISI (American Iron and Steel Institute), Washington, DC, 2016.

- **ASCE/SEI (2010), ASCE 7-10,** *Minimum design loads for buildings and other structures.* American Society of Civil Engineers, Reston, Virginia, 2010.

CONTENTS

1. Introduction

2. Seismic design of LWS Constructions

3. Researches at the University of Naples Federico II

4. Recent applications in italy

5. Concluding Remarks

In the last years, the application of Lightweight Steel-Framed Constructions has spread for **both structural and nonstructural applications especially in non-seismic areas.**

How they should be properly designed in seismic areas?

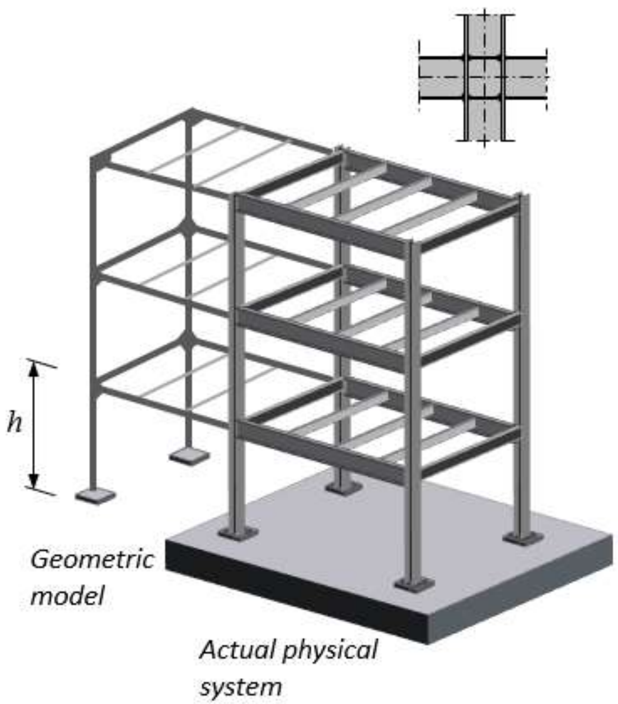
2. Seismic design of LWS Constructions

Structural design concept

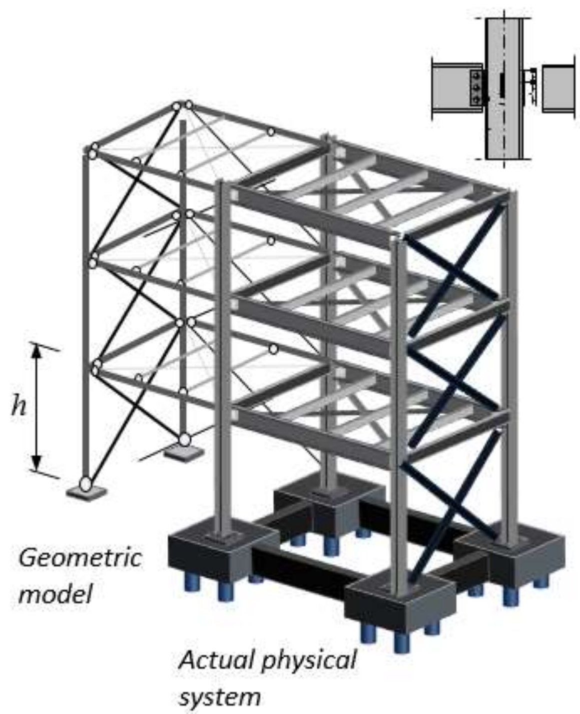
Since the Lightweight Steel-Framed Constructions are not traditional structural typologies, they need specific solutions that require a different structural design

Traditional typologies for single or multi-storey steel buildings

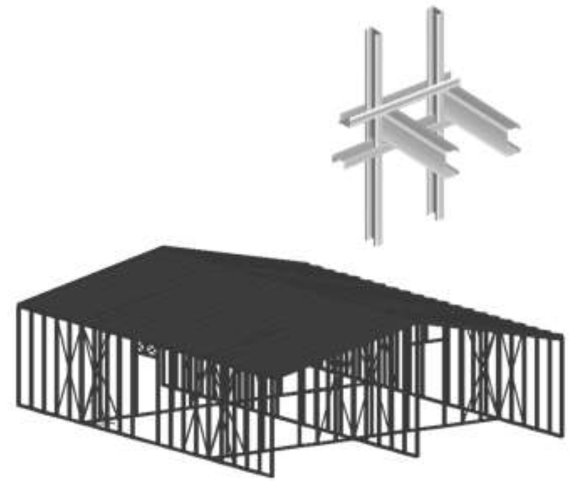
Moment-Resisting Frames (MRF)



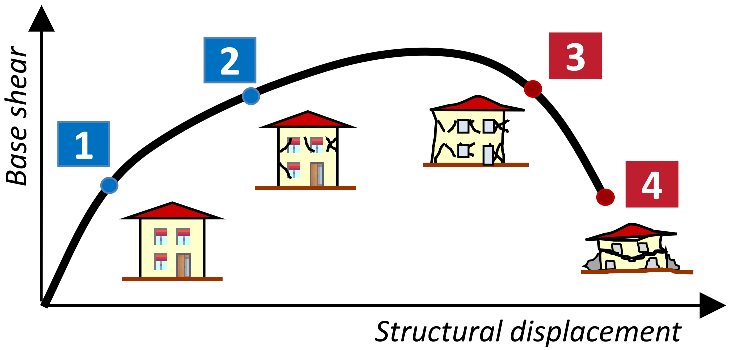
Concentrically Braced Frame (CBF)



Specific solutions for Lightweight Steel-Framed Construction



"Performance-Based Design" approach



Serviceability Limit States (SLS)

1. Fully operational:
Only minor structural or non-structural damage occurred. The building retains its original stiffness and strength. Non-structural components operate and the building is available for continuous service. The risk of life threatening injury is negligible.

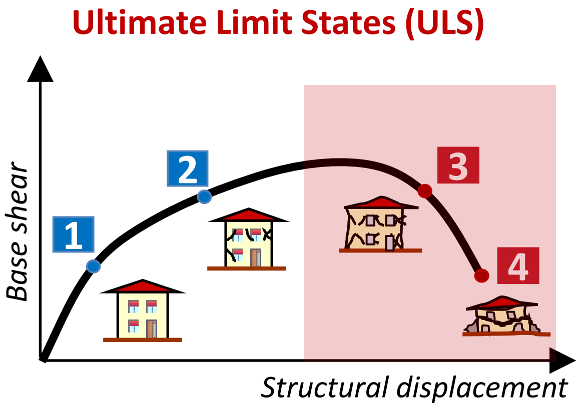
2. Operational:
Only minor structural damage occurred. The building structure retains nearly its original stiffness and strength. Non-structural components are secured and most of them would function. The risk of life threatening injury is very low. The service interruption is less than 3 days.

Ultimate Limit States (ULS)

3. Life Safety:
Significant structural and non-structural damage occurred. The lateral strength has still a margin against collapse. Non-structural components are secure, but cannot operate. The building may not be safe for occupancy until repaired. The risk of life threatening injury is low. The service interruption is less than 3 months.

4. Near Collapse:
Substantial damage occurred. The building has lost most of its original stiffness and strength, having a very little margin against collapse. Non-structural components may become dislodged and present a falling hazard. In many cases the repair is not practical.

Structural LWS systems

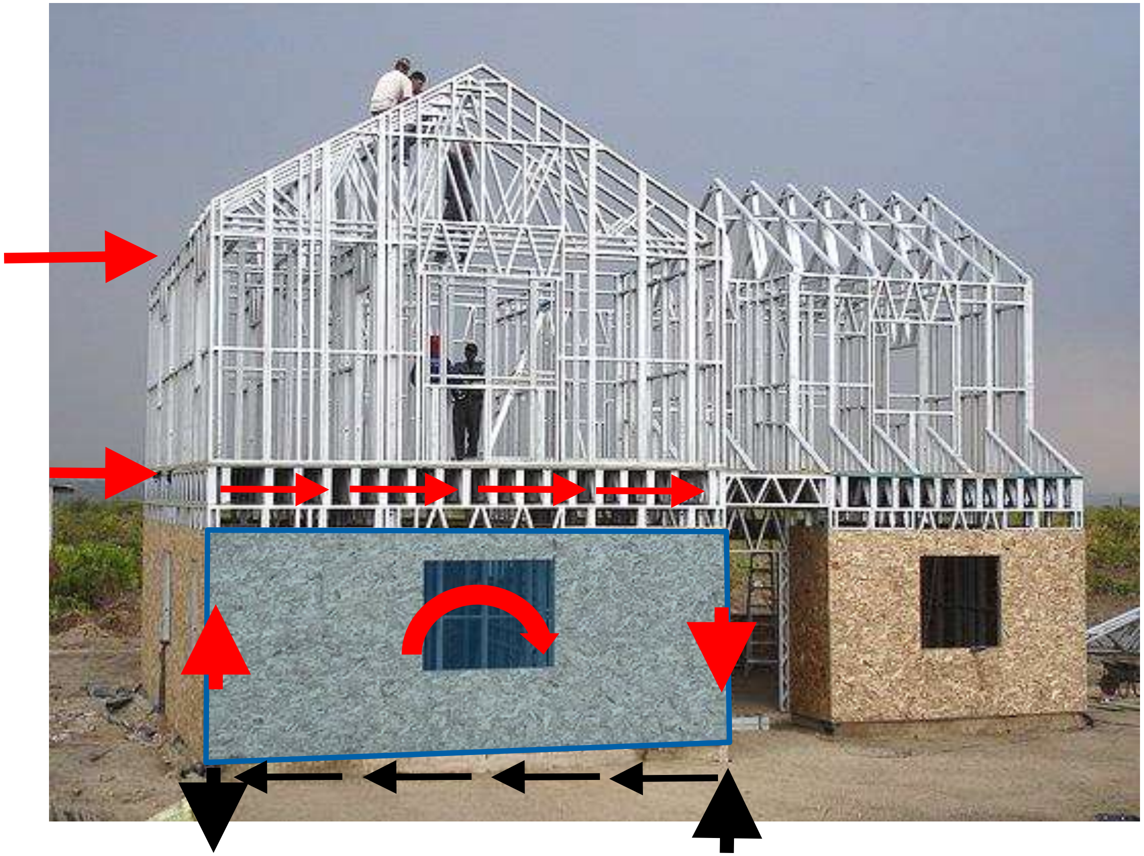


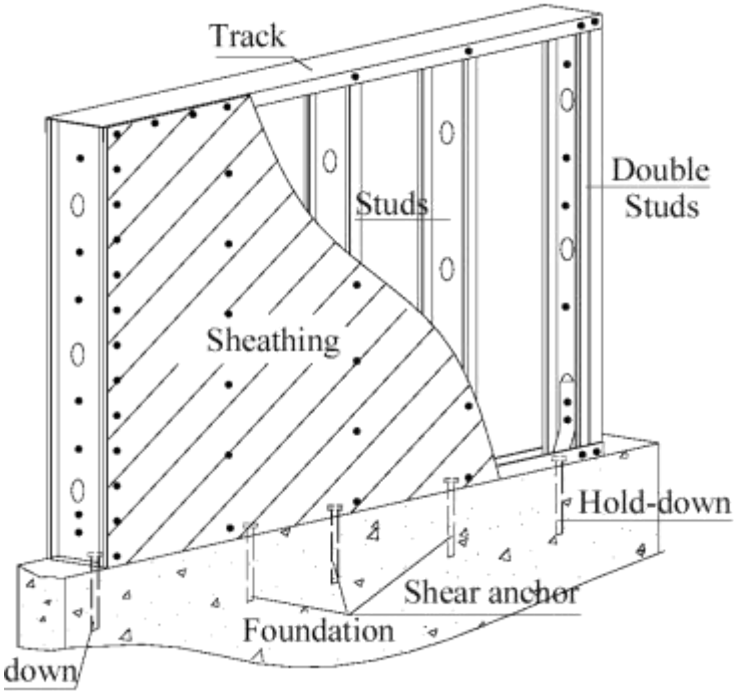
3. Life Safety

4. Near Collapse

The load-bearing structural units under vertical and horizontal loads are the **Shear walls**

Seismic lateral response



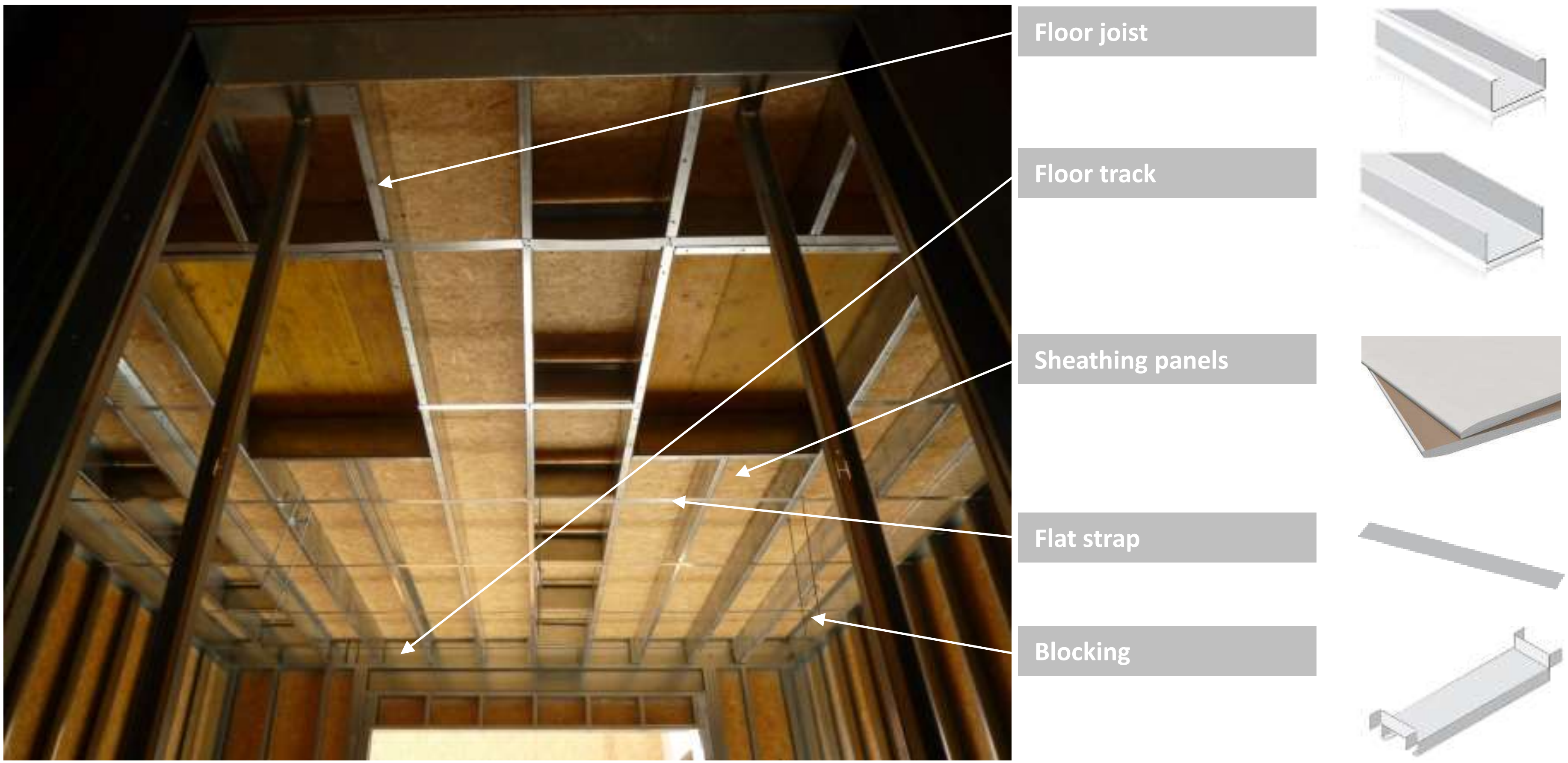


Main structural subsystems: wall framing

Load bearing walls are made of **studs**. The studs are fastened at each end to **wall tracks**. At mid wall height, **straps** can be connected to both flanges of the studs and **blocking profiles** can be introduced at wall ends. Walls are completed by **sheathing panels** (gypsum or wood –based panels).

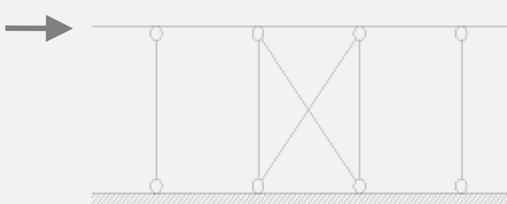
Main structural subsystems: floor framing

Floors are made of **joists**, located in line with the studs, and fastened at each end to **floor tracks**. At the joist ends, **bearing stiffeners** are used to strengthen the joists against web crippling. The top flange of joists is laterally braced by the **floor sheathing**, while the bottom flange can be braced by **straps** and **blocking.**

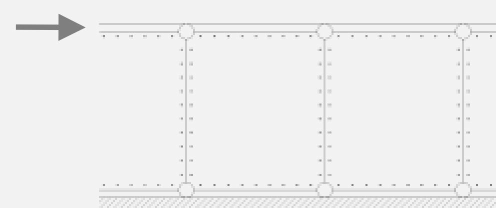


The design of Lightweight steel drywall constructions can be carried out using two approaches:

ALL-STEEL DESIGN



SHEATHING-BRACED DESIGN



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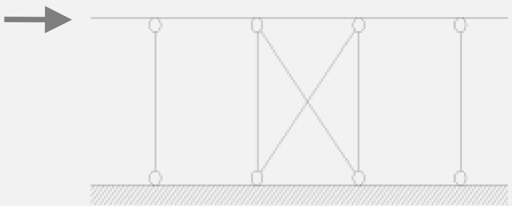
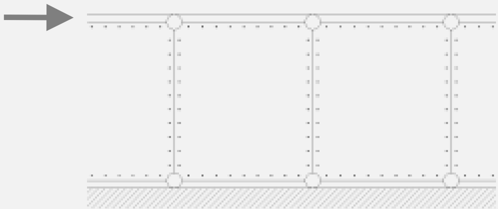


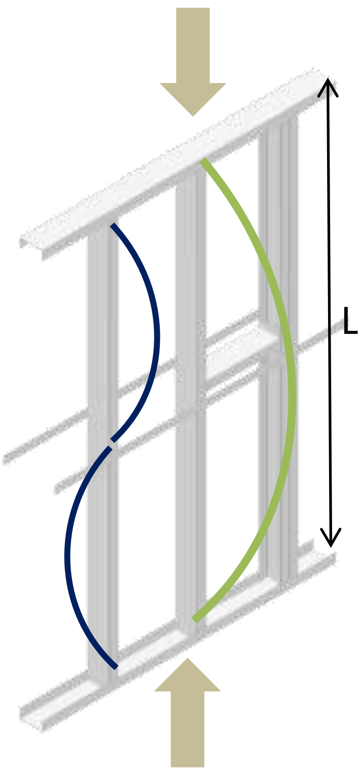
SHEATHING-BRACED DESIGN



Design under vertical loads

The all-steel approach does not consider the presence of sheathing panels and the generic profile is assumed as isolated (free-standing), by neglecting the interaction between the profile itself and the sheathing.

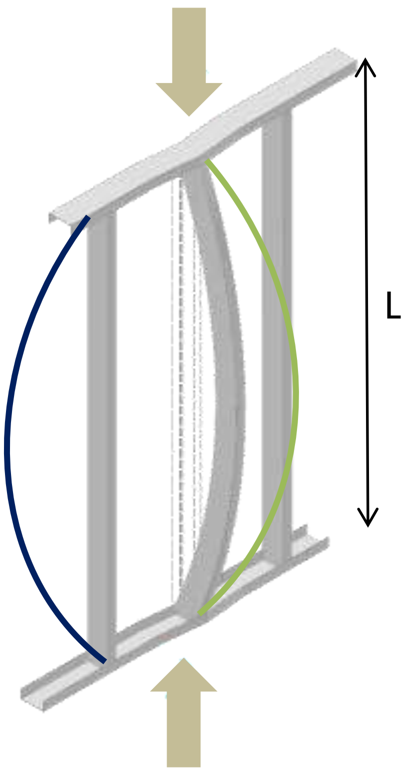
Case 1: wall with straps and blocking at mid wall height



In-plane:
$L_b = 0.5\ L$

Out-of-plane:
$L_b = L$

Case 2: wall without straps and blocking at mid wall height



In-plane:
$L_b = L$

Out-of-plane:
$L_b = L$

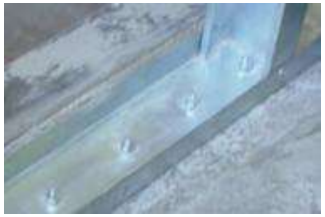
Global buckling of an "isolated" stud

L_b: buckling length, L: wall height

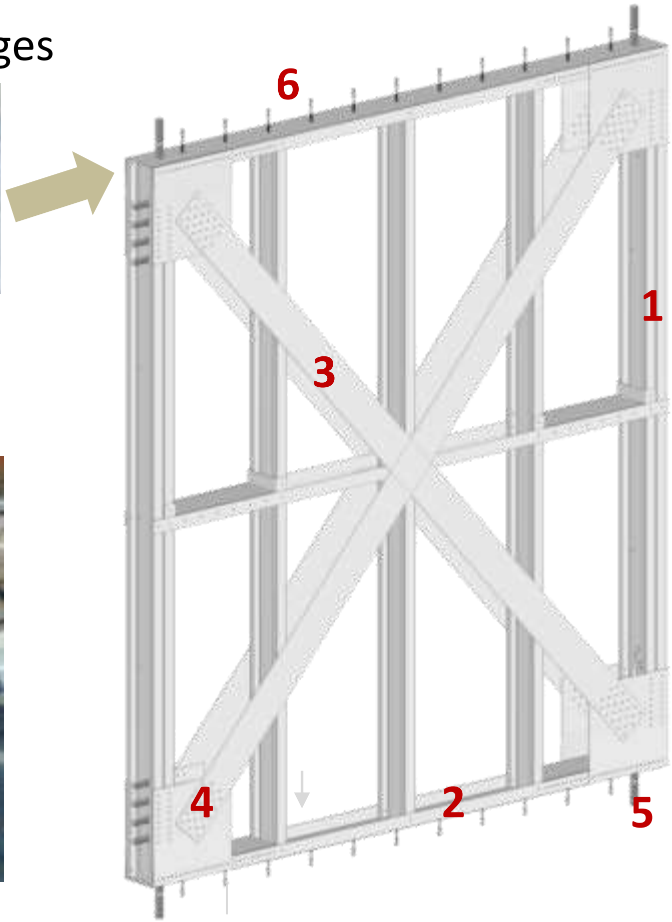
Design under horizontal loads

In the case of all-steel approach under horizontal actions, the in-plane resistance and stiffness are assured by **X-bracings** and steel straps are generally used to obtain the diagonal elements in walls, floors and roofs.

6 Shear anchorages



5 Tension anchorages



3 Diagonal straps

<u>Main wall structural components:</u>

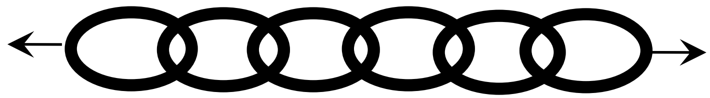
1. Chord (end) studs
2. Wall tracks
3. **Diagonal straps**
4. Diagonal connections
5. Tension anchorages
6. Shear anchorages

4 Diagonal joints

Design under horizontal loads

Evaluation of resistance contributions

The design lateral resistance of CFS diagonal strap-braced walls can be evaluated as the strength associated to the weakest of the possible failure mechanisms for each wall components. Therefore, the design lateral wall resistance (H_c) can be written as follows:



$$H_c = \min(H_{c,d}; H_{c,c}; H_{c,g}; H_{c,s}; H_{c,t}; H_{c,a})$$

1. Tension failure of diagonal strap braces

2. Failure of diagonal connections

3. Failure of the gusset plates

4. Studs failure

5. Track failure

6. Frame-to-foundations anchors failure

Design under horizontal loads

Evaluation of deformation contributions

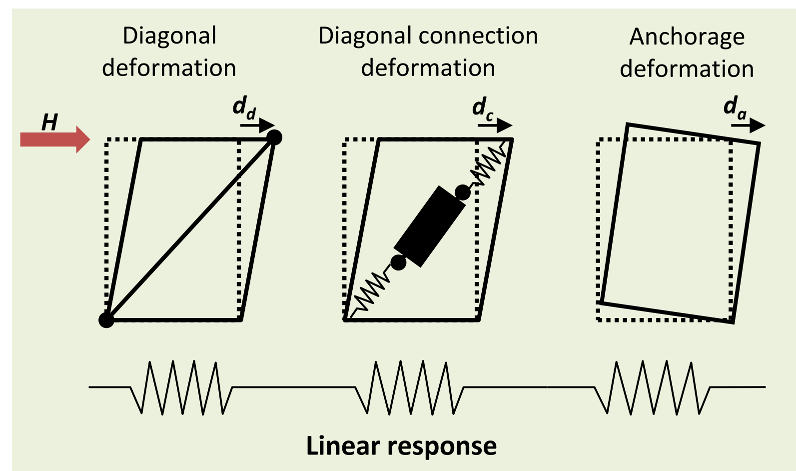
The lateral displacement (d) at the wall top under horizontal loads can be evaluated by taking into account the contributions due to main wall structural components, such as diagonals in tension (d_d), connections between frame and diagonal braces (d_c) and the anchorages between frame and foundations (d_a).

$$d = d_d + d_c + d_a$$

1. Diagonal deformation

3. Anchorage deformation

2. Diagonal connection deformation



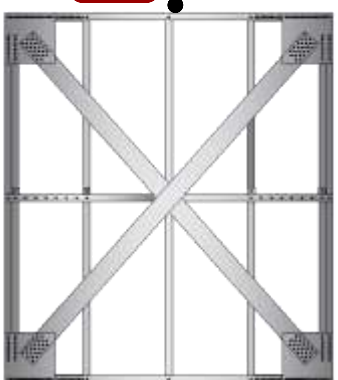
Design under horizontal loads

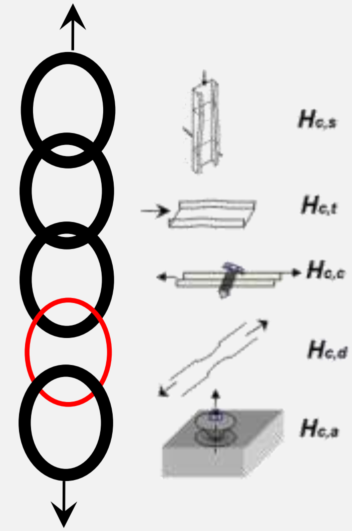
Dissipative systems

CAPACITY DESIGN

For diagonal strap braced walls the most ductile mechanisms is the **yielding of the diagonal strap**.
The non-dissipative members must be designed with an adequate overstrenght.

q=2.5 Dissipative Design (AISI S400)



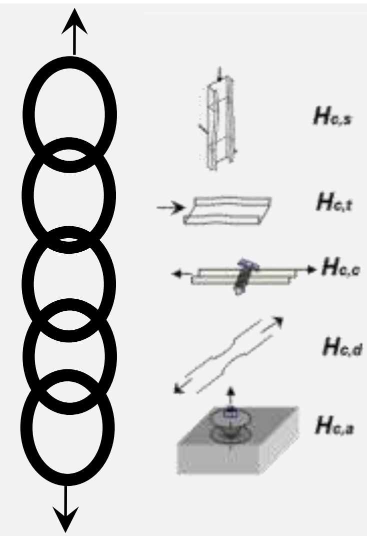


Iper-resistant systems

ELASTIC DESIGN

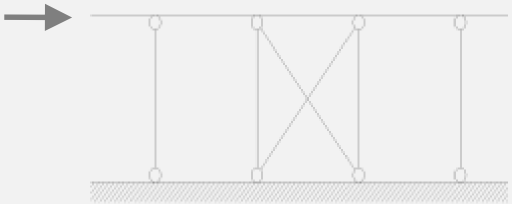
Designed to remain in the elastic range without any structural damage.
No dissipative mechanism is promoted.

q=1 Elastic Design

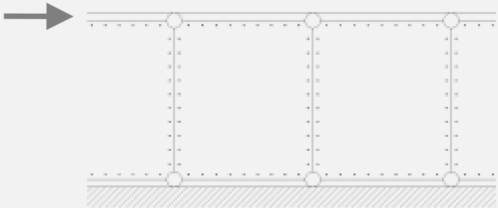


The design of Lightweight steel drywall constructions can be carried out using two approaches:

ALL-STEEL DESIGN



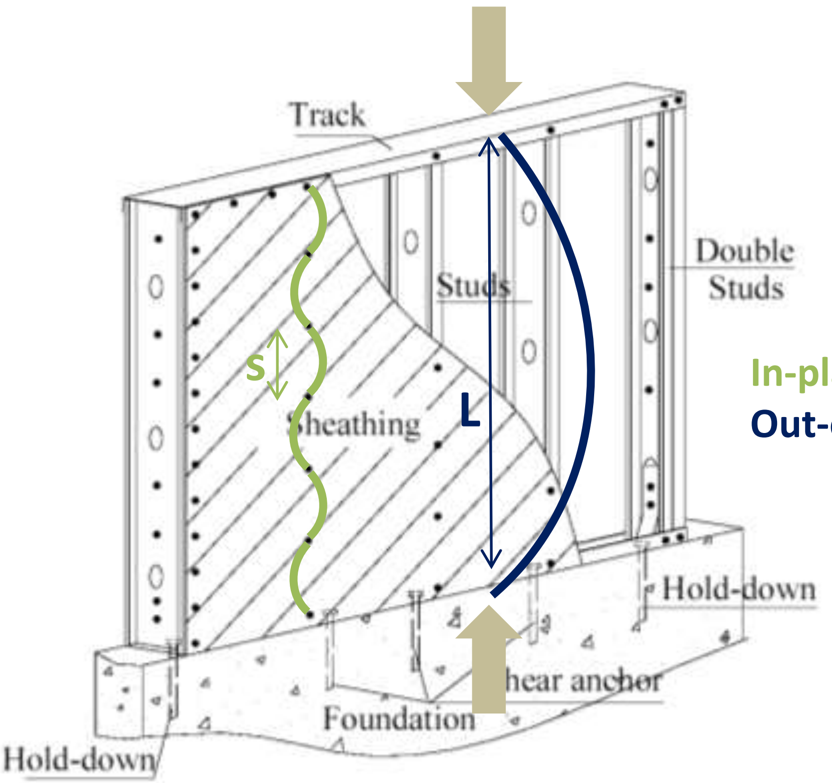
SHEATHING-BRACED DESIGN



Design under vertical loads

The sheathing-braced approach calculates the load bearing capacity of member taking into account the presence of sheathing panels. This is possible when the sheathing has adequate strength and stiffness and it is effectively connected to steel profiles.

Example: design of studs

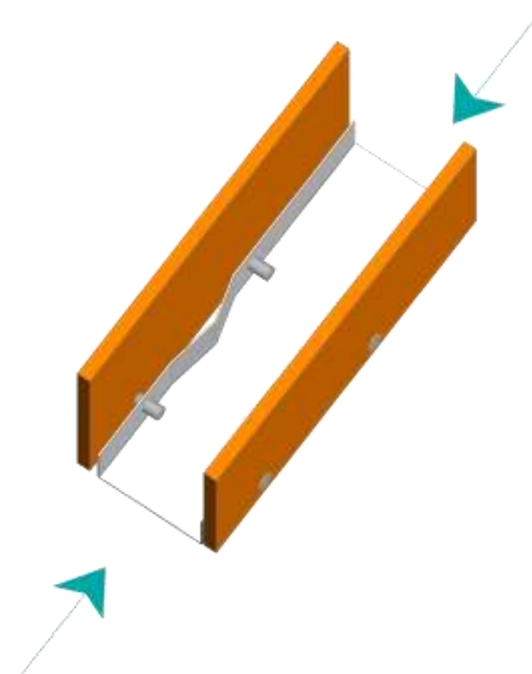


In-plane*: $L_b = 2\ s$

Out-of-plane: $L_b = L$

L_b: buckling length, L: wall height, s: fasteners spacing

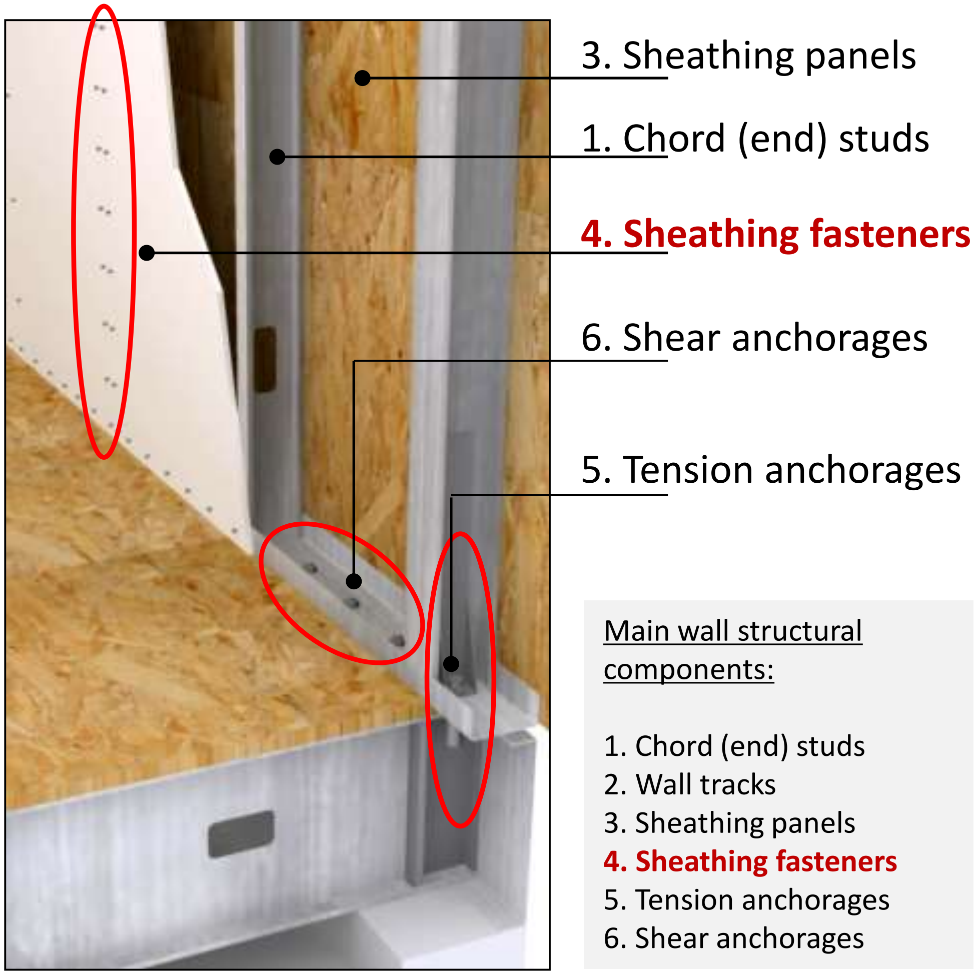
Local buckling of "sheathed" studs



*See: AISI S211 "North American Standard for Cold-Formed Steel Framing—Wall Stud Design

Design under horizontal loads

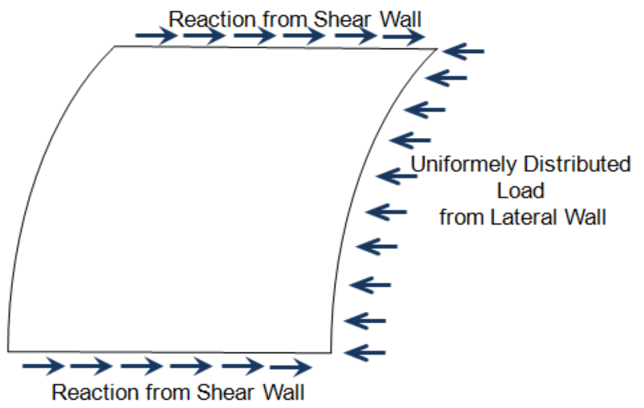
In the case of sheathing-braced approach under horizontal actions, the in-plane resistance and stiffness are assured by the interaction between sheathing panels and CFS frame.



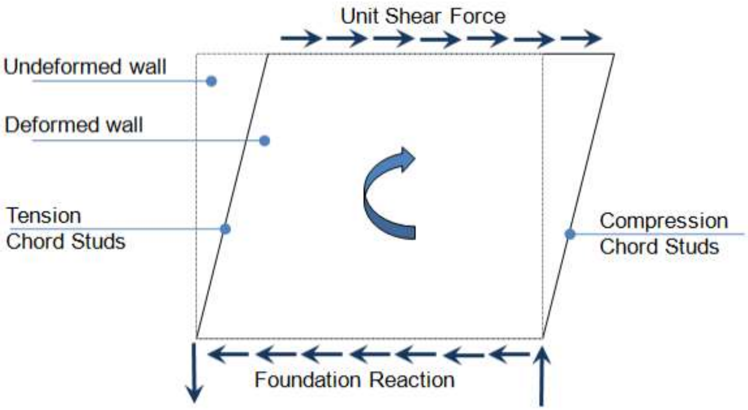
Main wall structural components:

1. Chord (end) studs
2. Wall tracks
3. Sheathing panels
4. **Sheathing fasteners**
5. Tension anchorages
6. Shear anchorages

Structural scheme of floor diaphragms



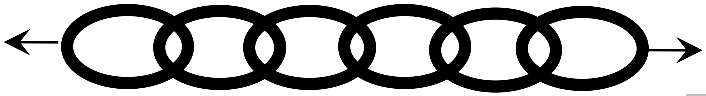
Structural scheme of walls diaphragms



Design under horizontal loads

Evaluation of resistance contributions

Also the evaluation of wall lateral strength (H_c) can be can be obtained by the wall lateral strength associated to the failure of sheathing-to-frame connections ($H_{c,f}$), panels ($H_{c,p}$), chord studs ($H_{c,s}$), tracks ($H_{c,t}$), and frame-to-foundation anchors ($H_{c,a}$) as follows:



$$H_c = \min(H_{c,f}; H_{c,p}; H_{c,s}; H_{c,t}; H_{c,a})$$

1. Failure of sheathing-to-frame connections

2. Failure of panels

3. Failure of chord studs

4. Failure of tracks

5. Failure of frame-to-foundation anchors

Generally, the failure mechanism of sheathing-to-frame connections is the most ductile one.

Design under horizontal loads

Evaluation of deformation contributions

The lateral displacement (*d*) at the wall top under horizontal loads can be evaluated by taking into account the contributions due to main wall structural components, such as sheathing-to-frame connections (d_f); sheathing boards (d_p); steel frame (d_s), and frame-to-foundation anchors (d_a):

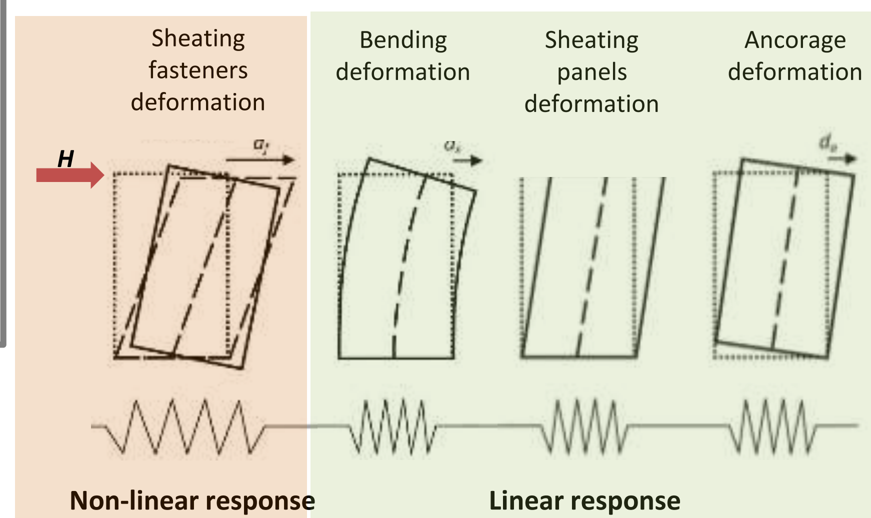
$$d = d_f + d_p + d_s + d_a$$

1. Sheathing-to-frame connections

2. Sheathing boards deformation

3. Steel frame deformation

4. Frame-to-foundation anchors deformation



Design under horizontal loads

Dissipative systems

Iper-resistant systems

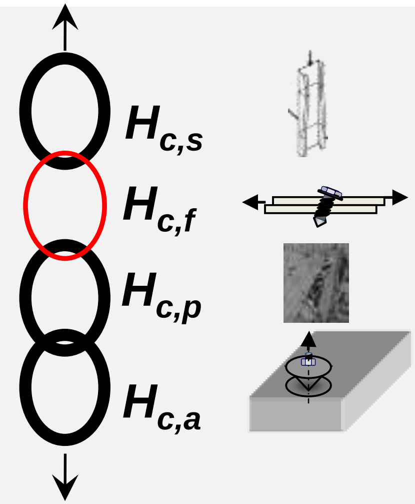
CAPACITY DESIGN

The failure mechanism of sheathing to frame connections is the most ductile and, in order to follow the capacity design criteria, panels, anchors and steel framing have to be oversized

$$H_{c,f}>H_{c,s};\ H_{c,f}>H_{c,t};\ H_{c,f}>H_{c,a};\ H_{c,f}>H_{c,p}$$

$$\mathbf{H = H_{c,f}}$$

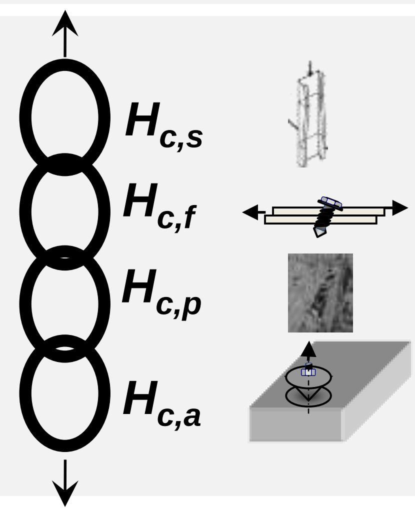
q=4.25 Dissipative Design
(AISI S400- For Canada / Wood panels)



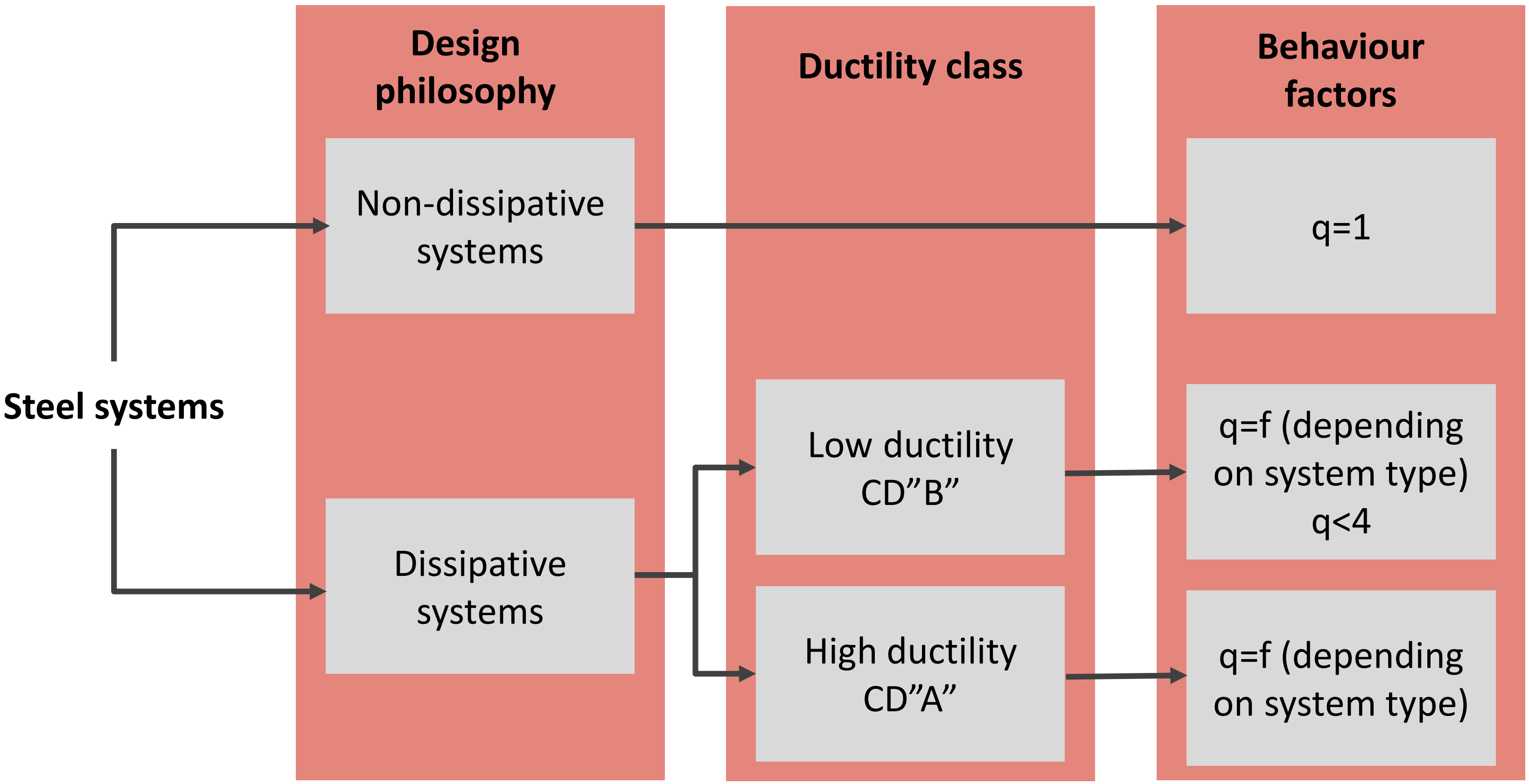
ELASTIC DESIGN

Designed to remain in the elastic range without any structural damage.
No dissipative mechanism is promoted.

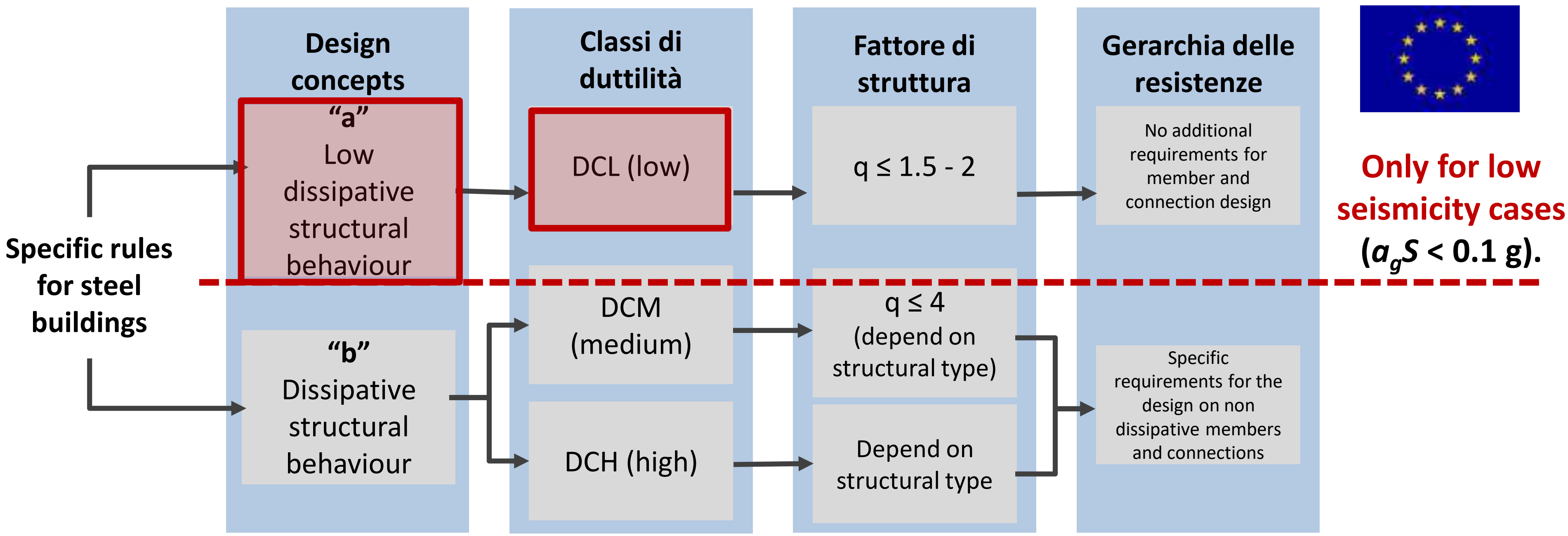
q=1 Elastic Design



Italian previsions for steel systems



European previsions for steel systems



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.

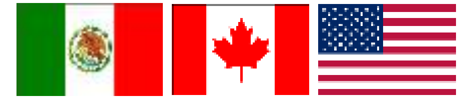
However, this approach may be restrictive, since the lightness of these systems makes them a good solution also for high seismicity zones

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	

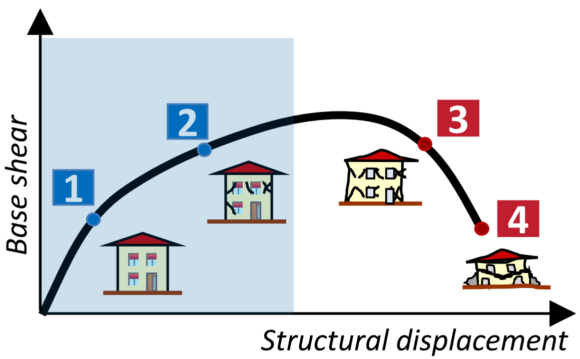
Sheathing- braced structure

Sheathing connections act as the energy-dissipating elements

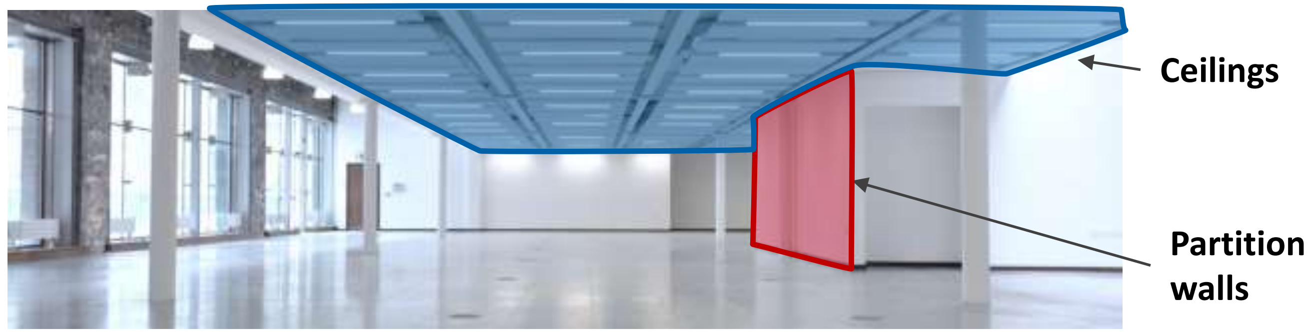
	ASCE 7		NBCC	
Behaviour factor	6.5 (bearing wall systems)	7.0 (building frame systems)	4.25 (shear walls with wood-based structural panel sheathing	
Overstrength factor	2.5 (building frame systems)	3.0 (bearing wall systems)	1.33 (DFP and OSB panels)	1.45 (CSP wood panels)

Nonstructural LWS drywall architectural components

Serviceability Limit States (SLS)

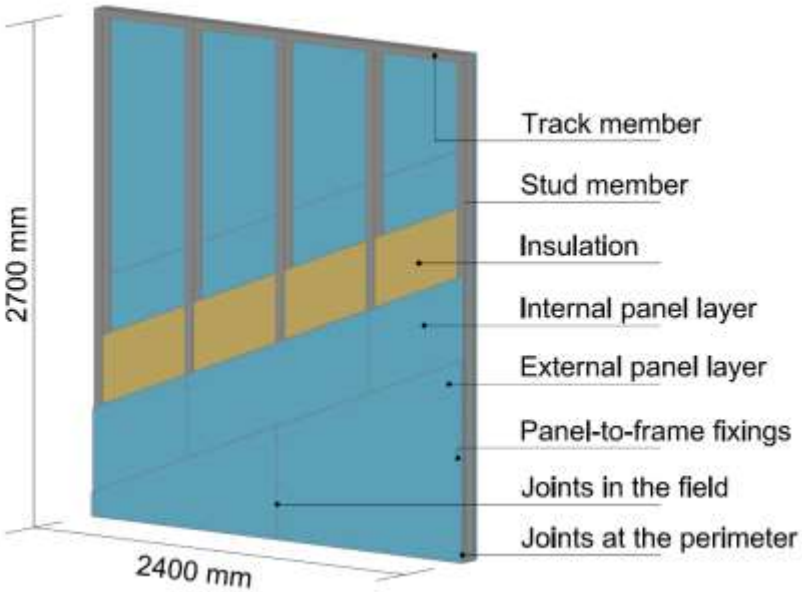


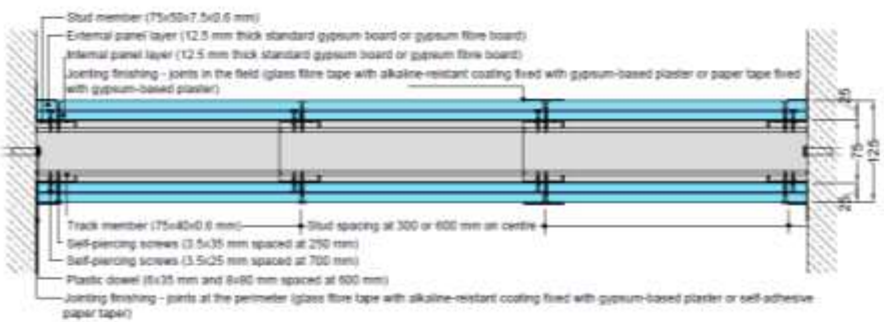
1. **Fully operational**
2. **Operational**



Nonstructural LWS drywall architectural components

<u>Interior partitions</u>





Lightweight steel profiles

Sheathing panels

Steel-to-steel connection: Punching

Screwed panel-to-frame connections

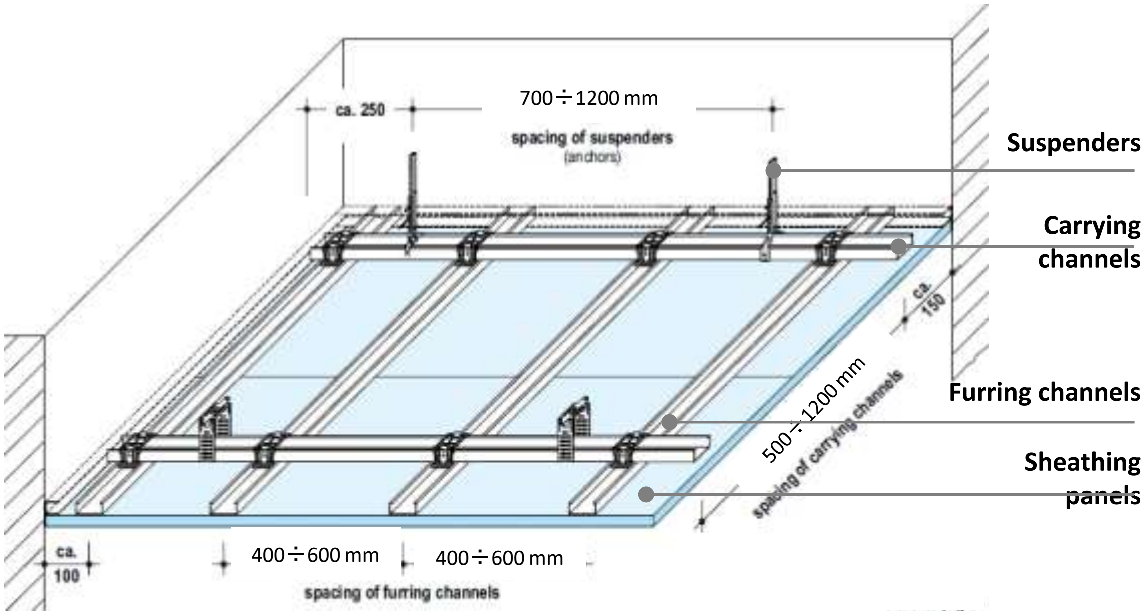
Wall-to-structure connections

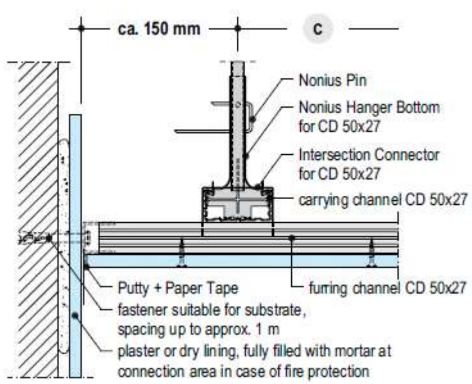
Steel dowel

Plastic dowel

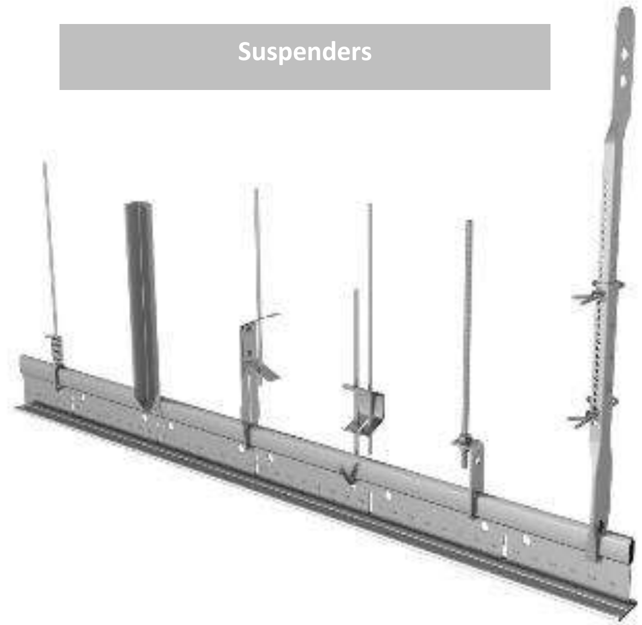
Nonstructural LWS drywall architectural components

Suspended ceilings





Suspenders



Sheathing panels

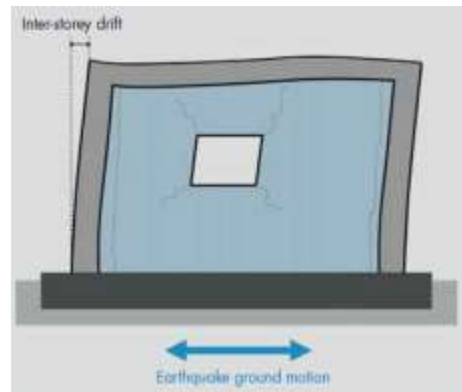
Seismic classification

The seismic response of non-structural architectural components is affected mainly by their sensitivity to structure response parameters and they are distinguished in:

- **deformation-sensitive components**
- **acceleration-sensitive components**
- **deformation-and-acceleration-sensitive components**

Architectural components	Acceleration-sensitive	Deformation -sensitive
Interior partitions	S	**P**
Suspended continuous ceilings	**P**	
Suspended acoustic lay-in tile (modular) ceilings	S	**P**
P: Primary response; S: secondary response		

DEFORMATION-SENSITIVE
partitions



ACCELERATION-SENSITIVE
ceilings
partitions

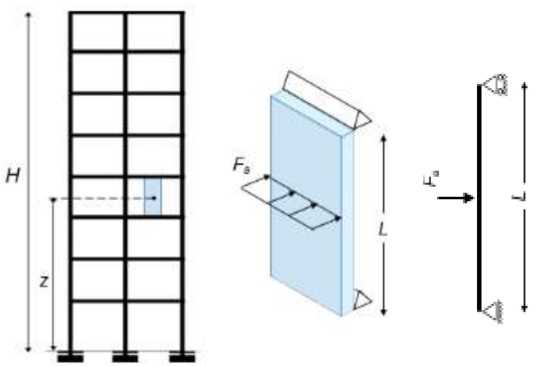

Interior partitions


Suspended continuous ceiling


Suspended acoustic lay-in tile (modular) ceiling

Seismic code prescriptions

Verification of acceleration-sensitive components according to EN 1998 Part 1-1 Section 4.3.5



Verification

Demand → F_a

Capacity → F_{Rd}

$$\frac{F_a}{F_{Rd}} \le 1$$

Design resisting force

Acting (deign) seismic force

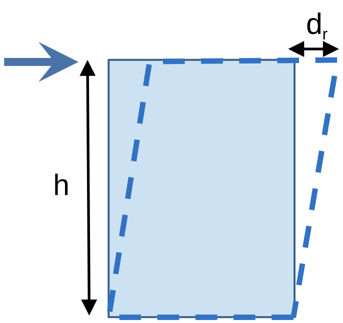
$$F_a = (S_a \cdot W_a \cdot \gamma_a)/q_a$$

Acting maximum acceleration

$$S_a = \alpha \cdot S \cdot \left[\frac{3 \cdot (1 + z/H)}{1 + (1 - T_a/T_1)^2} - 0.5\right] \ge \alpha \cdot S$$

Fundamental vibration period

Verification of deformation-sensitive components according to EN 1998 Part 1-1 Section 4.3.3



Limitation of the inter-storey drift ratio

Demand → $\frac{d_r \cdot \nu}{h} \le$

0.5%	for buildings having non-structural components made of brittle materials and attached to the structure
0.75%	for buildings having ductile non-structural components
1.0%	for buildings having ductile non-structural components fixed in a way so as not to interfere with structural deformations

← *Capacity*

CONTENTS

1. Introduction
2. Seismic design of LWS Constructions
3. Researches at the University of Naples Federico II
4. Recent applications in italy
5. Concluding Remarks

Motivations

1. **Lightweight Steel-framed Constructions** can be a very competitive solution for thanks to their lightness, which allows satisfactory structural/seismic performances for both structural and nonstructural applications

2. Nowadays, **European Code** does not cover properly the seismic design of Lightweight Steel-framed Constructions

3. The ongoing researches at the University of Naples "Federico II" have the main aim to bridge the gap between **the code specifications and the application** of Lightweight Steel-framed Constructions in seismic areas

4. The use of Lightweight Steel-framed Constructions is still limited. This can be explained by the **reduced technical knowledge, especially in the case of application in seismic areas.**

Ongoing researches



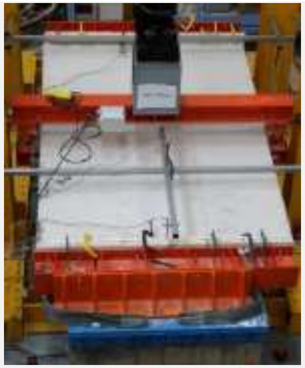
Structural all-steel systems

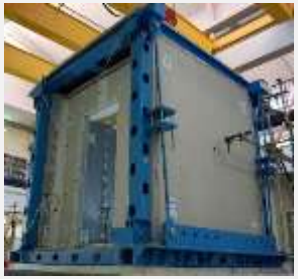
- Italian national research project
  ReLUIS-DPC, Line 1, years 2010-2013
- National research project
  Lamieredil-UNINA Project, years 2014-2017



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

3. Researches at the University of Naples "Federico II"



Structural all-steel systems

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



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Drywall non-structural building components

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

3. Researches at the University of Naples “Federico II”

Research objectives

1 The main goal of the study was the experimental characterization of the seismic response of **low dissipative CFS all-steel strap-braced structures**.

2 The research goals were the evaluation of the **seismic response** and the survey of the **observed damages.**

3 An experimental activity on materials, products, components, single seismic resistant systems and whole structures.

Seismic response evaluation and optimization of structural all-steel systems

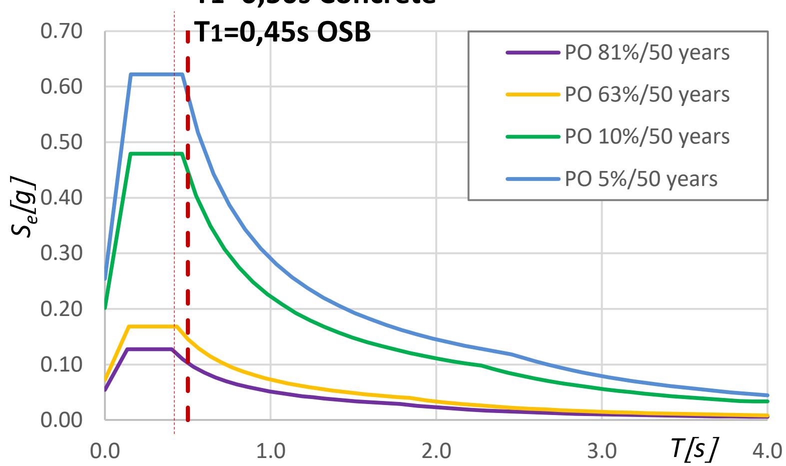
The Lamieredil project is a research funded by a manufacturer of cold-formed metal framing, the Lamieredil S.p.A. Company, Italy. The main goal of the study was the experimental characterization of the seismic response of low dissipative CFS all-steel strap-braced stud structures, obtained through an experimental activity on materials, products, components, single seismic resistant systems and whole structures.

UNIVERSITY OF NAPLES FEDERICO II

Department of Structures for Engineering and Architecture

Lamieredil SpA
il metallo prende forma

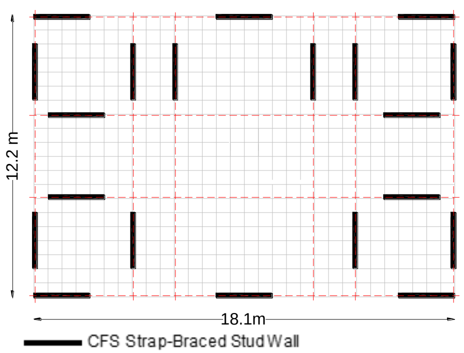
Case study and design assumptions



PO: Probability of occurrence

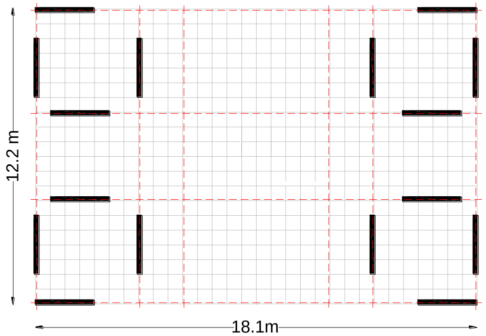
PO	a_g [g]	PGA [g]	Sa (0.50s) [g] CONCRETE	Sa (0.45s) [g] OSB
81%/50	0.05	0.05	0.10	0.11
63%/50	0.06	0.07	0.15	0.16
10%/50	0.17	0.20	0.55	0.49
5%/50	0.21	0.25	0.58	0.65

Type 1 building (Concrete solution)



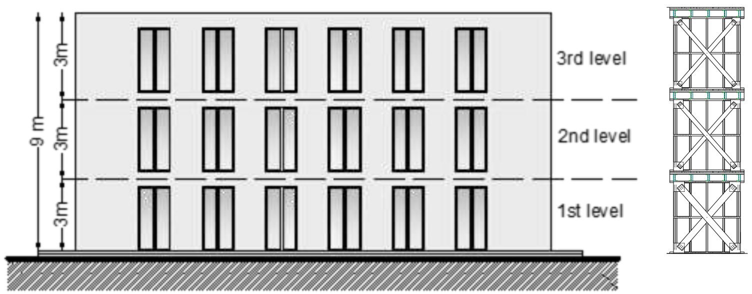
10 CFS strap-braced stud walls

Type 2 building (OSB solution)



8 CFS strap-braced stud walls

ELASTIC DESIGN APPROACH
(q=1)



The studied building, with rectangular plan, covered an area of 220 m^2 and three storeys with a storey height of 3.00 m.

3. Researches at the University of Naples "Federico II"

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

3. Researches at the University of Naples "Federico II"

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

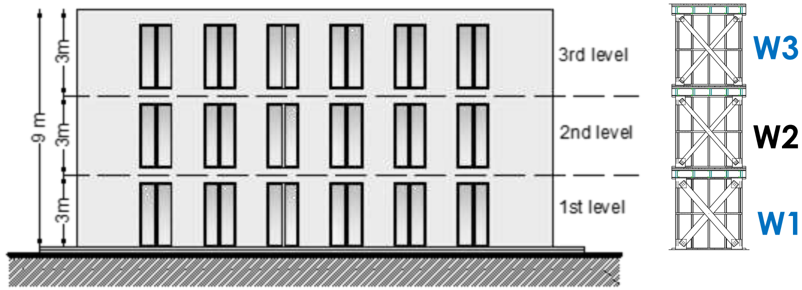
Total tests: 6

Tests on hold-down devices

Total tests: 4

3. Researches at the University of Naples "Federico II"

Elastic design of the tested walls

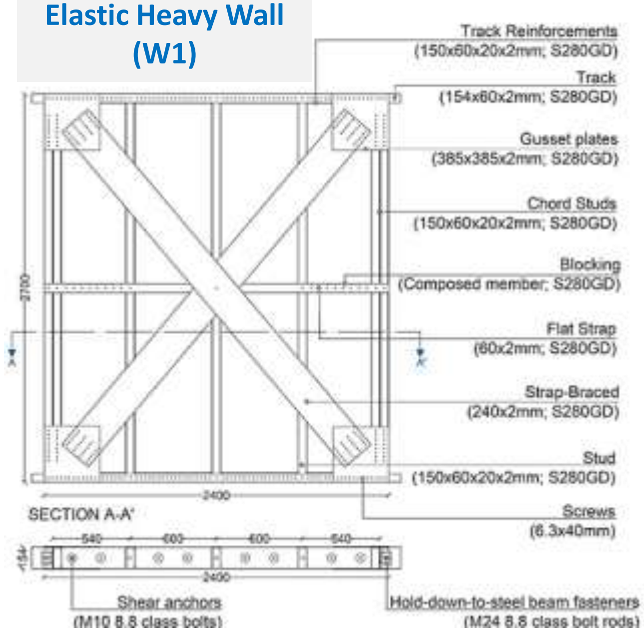


The experimental campaign investigated the walls representative of the **first (W1) and third (W3) levels** of the case study building.

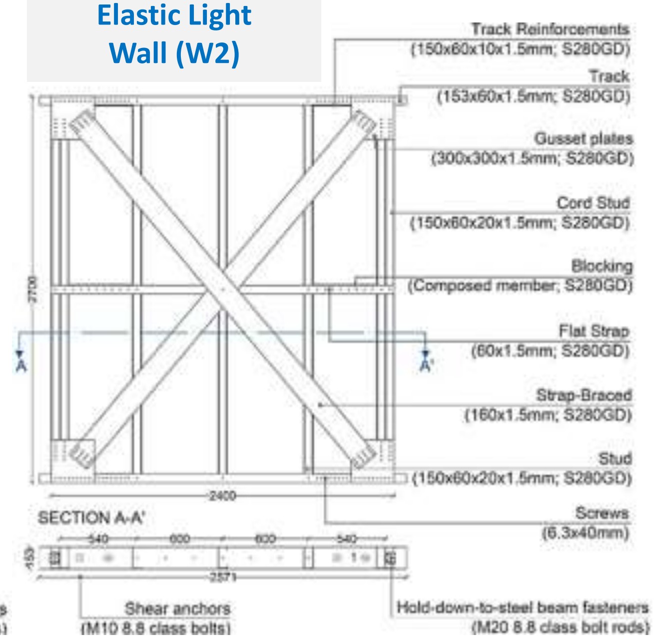
ELASTIC DESIGN APPROACH
(q=1)

Wall configuration	W1	W2
Storey	First	Third
Seismic action on single wall (H_d) [kN]	121	54
Design lateral wall resistance (H_c) [kN]	152	71
Lateral wall stiffness (k) [kN/mm]	6.8	4.2
Predicted collapse mechanism	Local buckling of tracks	Diagonal net area failure

Elastic Heavy Wall (W1)

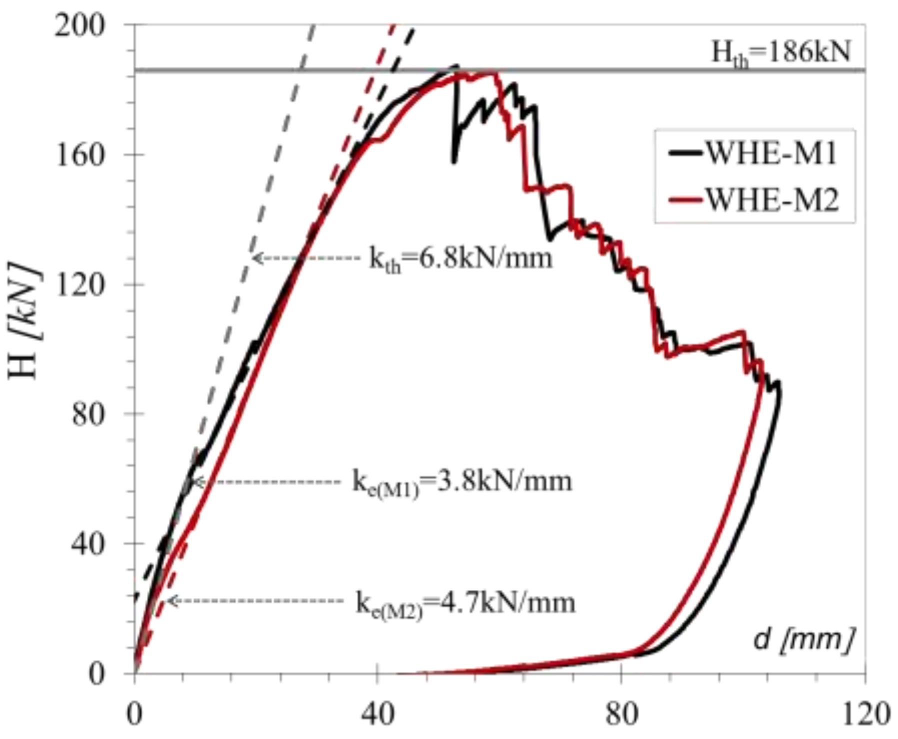


Elastic Light Wall (W2)



In-plane monotonic tests on wall specimens

Test results



2 monotonic tests

Failure modes

W1-M1

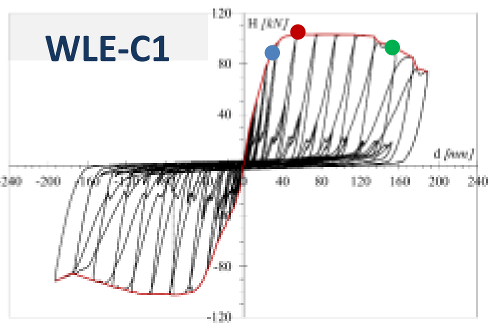
W1-M2

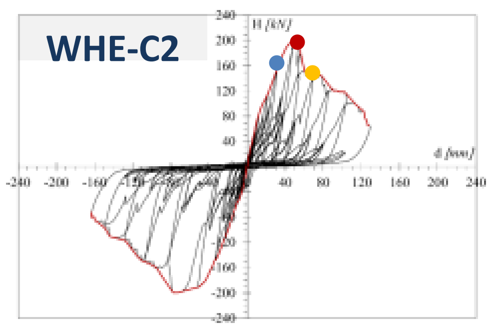
Type	H_y (kN)	H_p (kN)	d_y (mm)	d_p (mm)	k_e (kN/mm)	FM
WHE-M1	160.2	187.1	36.6	52.9	3.8	GT
WHE-M2	164.1	185.7	38.9	58.7	4.7	GT

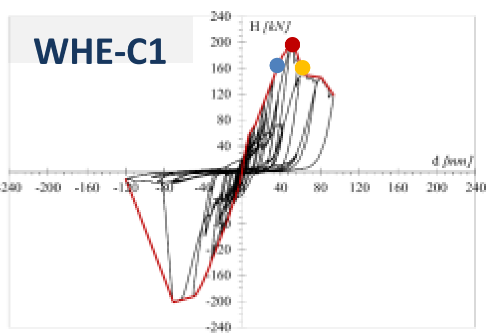
FM: Failure mode; GT: failure of gusset-to-track connection

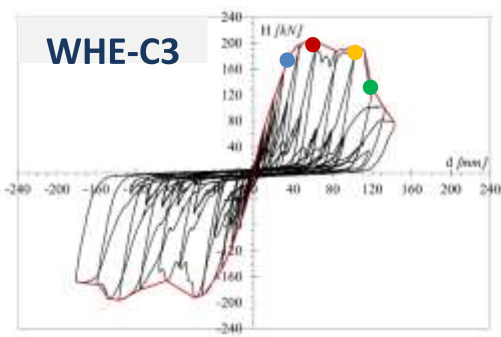
In-plane quasi-static reversed cyclic tests on wall specimens

Test results









4 cyclic tests

Failure modes


Local buckling of the tracks


Crushing of the stud ends


Gusset-to-track connection failure

Plastic hinges in chord studs

From the case study to the 1:3 reduced scale specimens for shake table tests

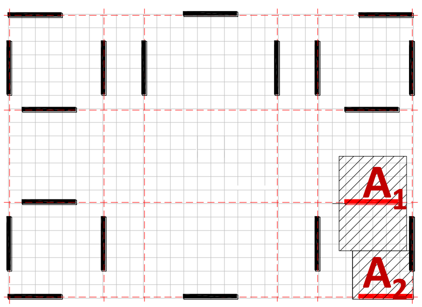
The reduced-scale specimens were defined starting from a part of the full-scale case study (i.e. A_1=12.8 m^2 and A_2=6.10 m^2)

Two different specimens:

- **prototype with composite steel-concrete floors;**
- **prototype with wood-based (OSB panels) sheathing-braced floors.**

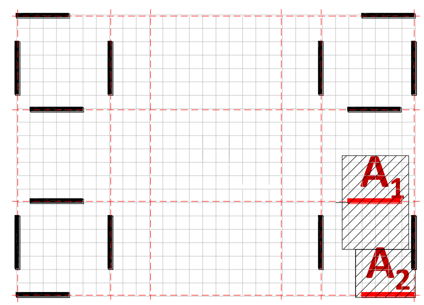
Both specimens have nominally identical CFS strap-braced stud walls.

Type 1 building (concrete solution)



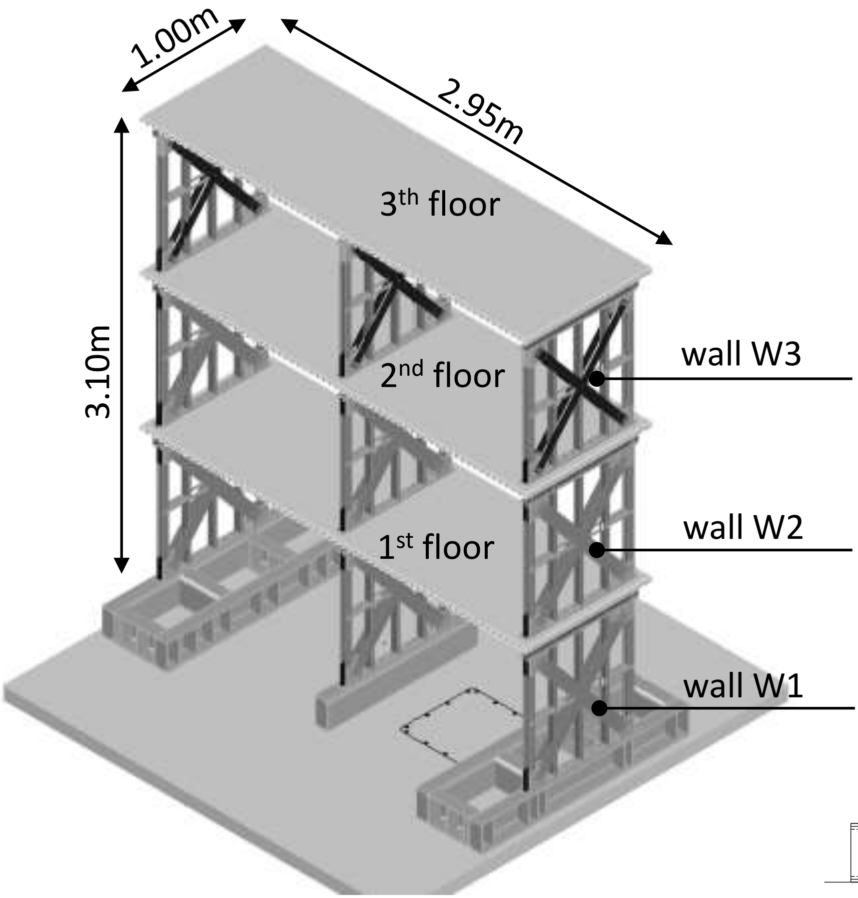
Prototype with composite steel-concrete floors

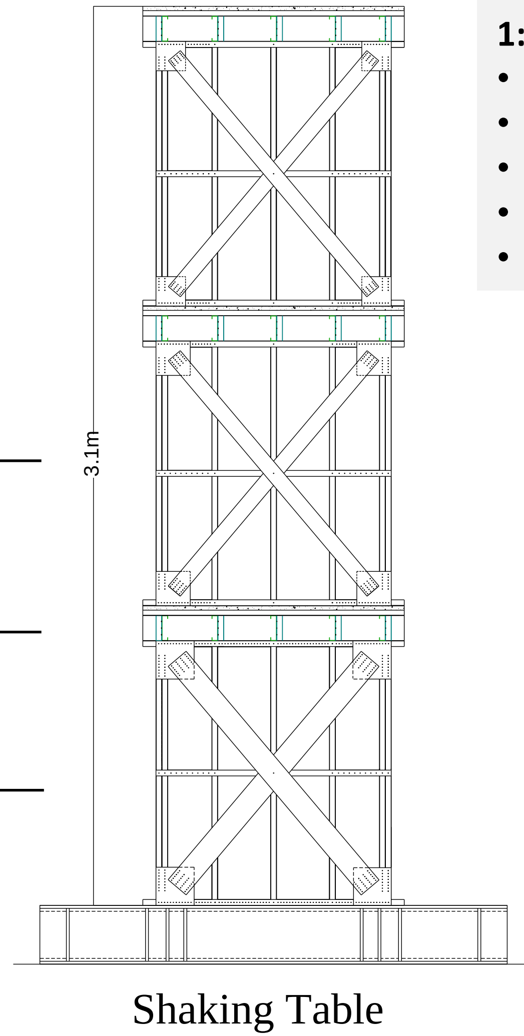
Type 2 building (OSB solution)



Prototype with wood-based (OSB panels) sheathing braced floors

1:3 reduced scale CFS three-storeys strap-braced stud structures

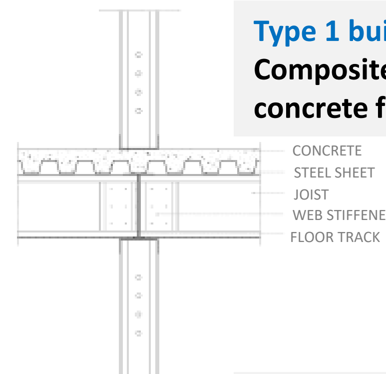




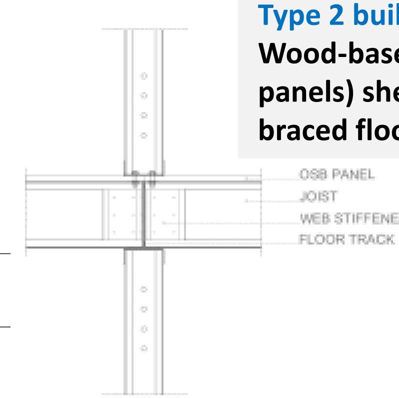
1:3 REDUCED SCALE SPECIMEN:

- Three storeys building
- Two bays
- Total area: 3 x 2.8 m^2= 8.4 m^2
- Storey height: 1.03 m
- Total height: 3.10 m

Type 1 building:
Composite steel-concrete floor



Type 2 building:
Wood-based (OSB panels) sheathing-braced floors



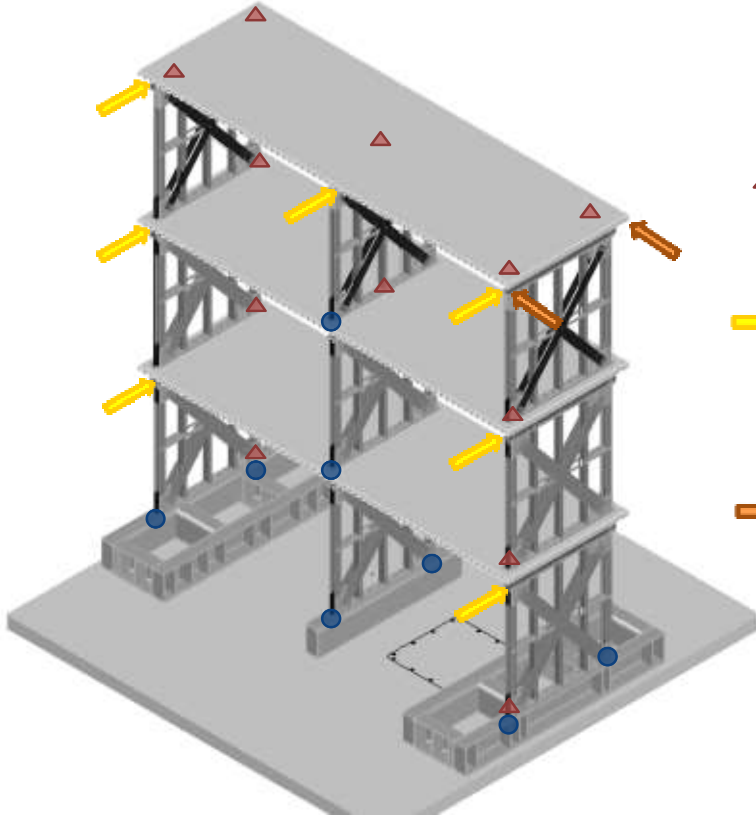
Experimental program for shake table tests

1:3 reduced scale CFS three-storeys strap-braced stud structures	**Dynamic identification tests**	**Dynamic earthquake tests**
Prototype with composite steel-concrete floors	8 tests (0,036g)	7 tests (9 – 150 % Scaling Factor)
Prototype with wood-based (OSB panels) sheathing-braced floors	8 tests (0,028 g)	7 tests (9 – 150 % Scaling Factor)


Prototype with composite steel-concrete floors


Prototype with wood-based (OSB panels) sheathing braced floors

Instrumentation



- Load cells (8)
- Triaxial accelerometers (12)
- Laser sensors for horizontal displacement measurement (7)
- Laser sensors for out-of-plane displacement measurement (2)

Max number of sensors: 12 accelerometers + 9 laser distance meters

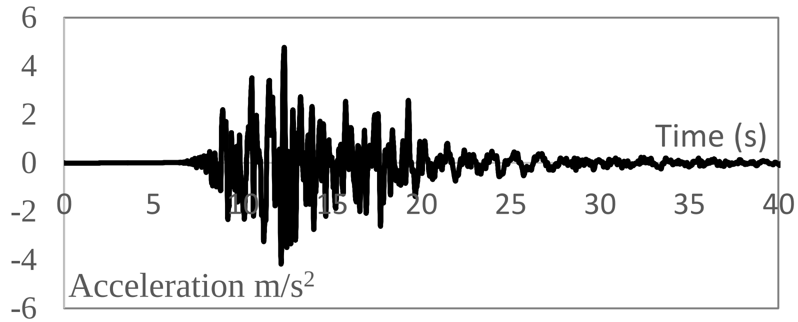
Load cells

Triaxial accelerometers

Laser distance meters

Dynamic earthquake tests - Input: 2016 Norcia Earthquake

Input time history NRC-EW

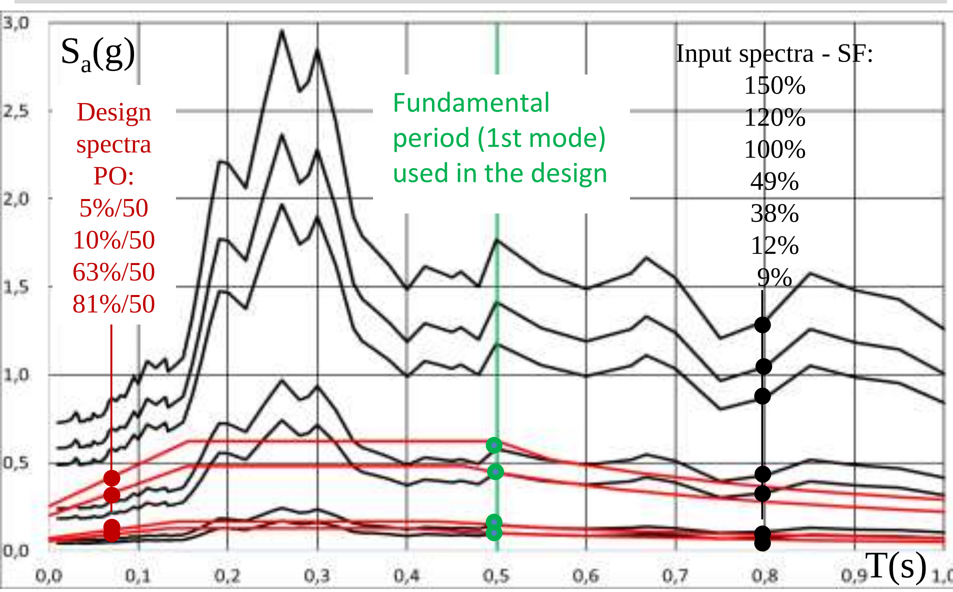


SELECTED GROUND MOTION

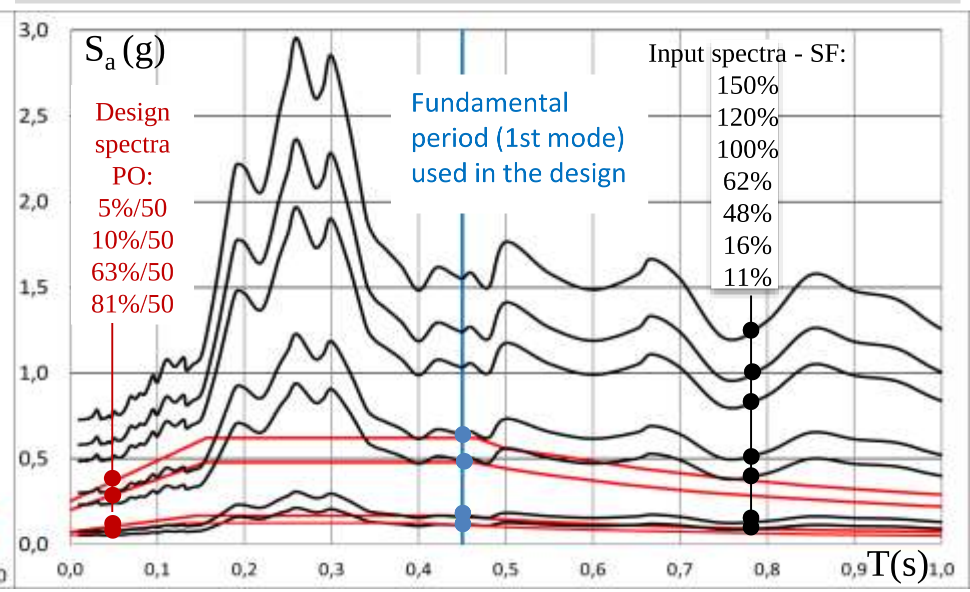
Event:	*Norcia – October 30th, 2016 6:40 a.m.*
Magnitude:	Mw= 6.5
Station:	*Norcia*
Station code:	*NRC*
PGA:	*4.76 m/s²* (0,49 g)

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

Concrete solution



OSB solution



SF: Scaling Factor; PO: Probability of occurrence

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%***

3. Researches at the University of Naples “Federico II”

Main outcomes

1 Under quasi static cyclic tests single seismic resisting systems (walls) exhibited typical response of structural elements designed **without capacity design criteria**

2 Results of shake table tests on whole seismic resisting structures showed that the **global response was almost linear** for both mockups for all scaling factors (from 9% to 150%), with maximum inter-storey drifts recorded at the 3rd level (3.62% for Concrete solution and 2.44 % for Wood solution)

3 The **observed damages** were strap yielding and bolt loosening for both Concrete and Wood solutions, whereas local buckling of chord studs occurred only for mockup with wood-based floors

4 **Floors** behaved as rigid in their plane according to the ASCE 7 definition for both solutions

5 A **numerical model** for CFS structures which describes the dynamic behaviour of structures in both linear and non linear fields was developed. The modelling of **hold down connections**, which can significantly influence the structural response, should certainly be further investigated

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 **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

Research objectives

1 The ELISSA Research Project was devoted to the development and demonstration of **nano-enhanced prefabricated lightweight Cold-Formed Steel (CFS) skeleton/dry wall constructions** with improved thermal, seismic and fire performance, resulting from the inherent thermal, antiseismic and fire spread prevention properties. In this framework, the ELISSA Project is configured as an integrated research project.

2 From the structural point of view, the research was focused on the **seismic response** of the proposed structural solution, in which the lateral force resisting system is based on CFS floors and walls sheathed with gypsum-based panels (sheathing-braced solution).

3 Additional research goals were the evaluation of the influence of **box-building behaviour** and **nonstructural components** on the seismic response of a whole building.

ELISSA Project

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

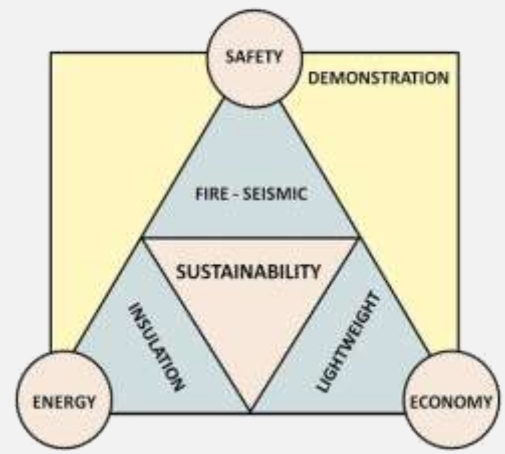
ELISSA Research Project

Energy Efficient LIghtweight – Sustainable – SAfe – Steel Construction

PARTNERS

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.

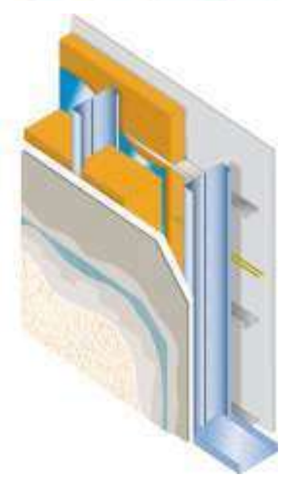


The reference structural system: The COCOON “Transformer”

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

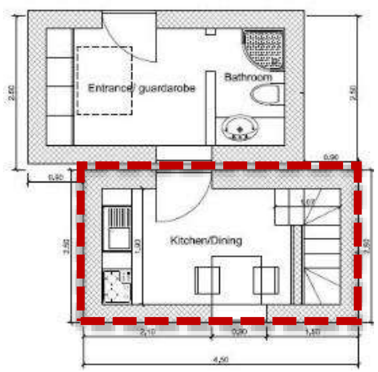
Evaluation of the seismic response of sheathed CFS buildings by means experimental tests on connections, walls and 3D mock-up.



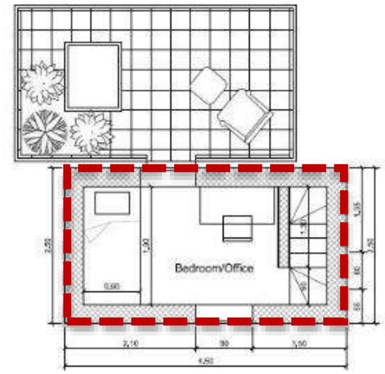
The case study: The “ELISSA house”

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).

1st floor



2nd floor



“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

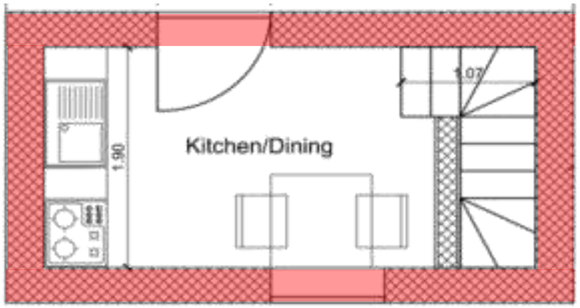
ELISSA MOCK-UP data

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

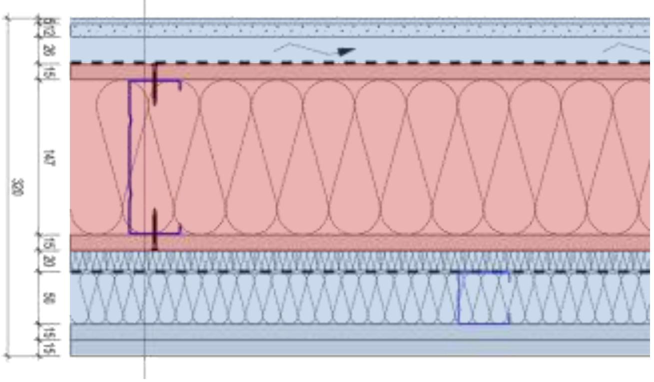
- Two storeys building
- Total gross area: 22.5 m^2
- Total height: 5.4 m
- Weight of the complete building (w/ finishing) : 102 kN (**4.53 kN/m²**)
- Weight of the structural part (w/o finishing): 46 kN (**2.04 kN/m²**)

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

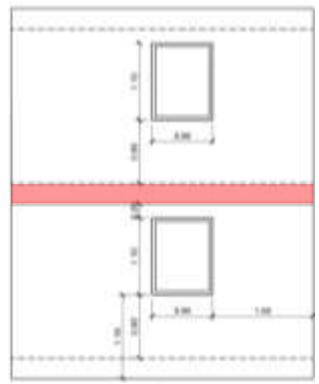


Floor/roof

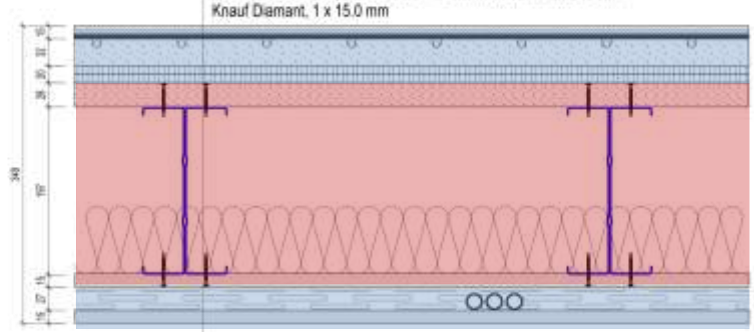
Structural elements

Non-structural elements

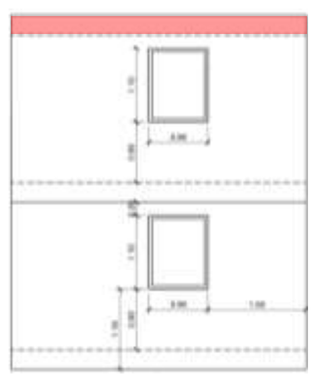
Floor



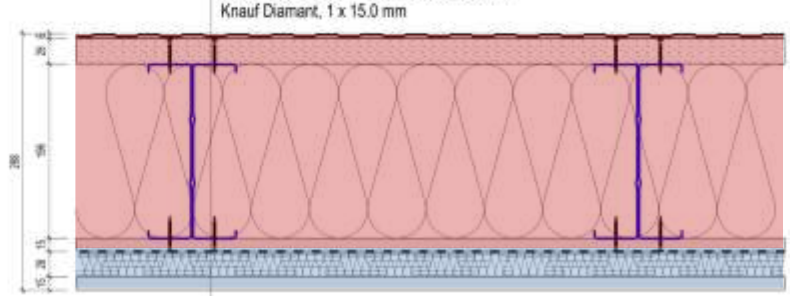
- Floor covering
- Floor heating / cooling system Knauf GIFAfloor Klima, 32 mm
- Impact sound insulation Knauf WF, 2 x 10 mm
- Load panels Knauf GIFAfloor, 28 mm
- Structure Cocoon DT 2xC197/50/2.0 mm, centered at 500 mm
- Knauf Insulation mineral wool, max. 180 mm
- **Knauf Diamant, 1 x 15.0 mm**
-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



Design assumptions and structural design

Loads [EN 1991]

Roof: 2.00 kN/m² (snow);
Floors: 2.00 kN/m² (live);
Wind: 0.85 kN/m²

Seismic action [EN 1998]

Medium to high intensity seismic area:

•Hazard level: 10% in 50 years probability of exceedance

- reference PGA of 0.29g
- design elastic spectral acceleration (Sa,e,d) equal to **0.72g**

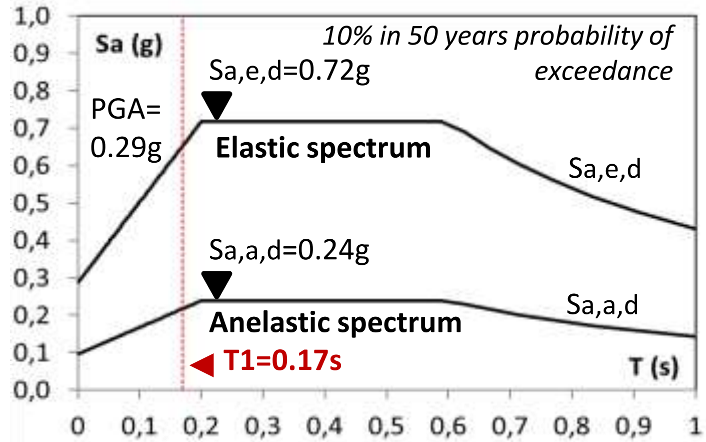
•Hazard level: 2% in 50 years probability of exceedance

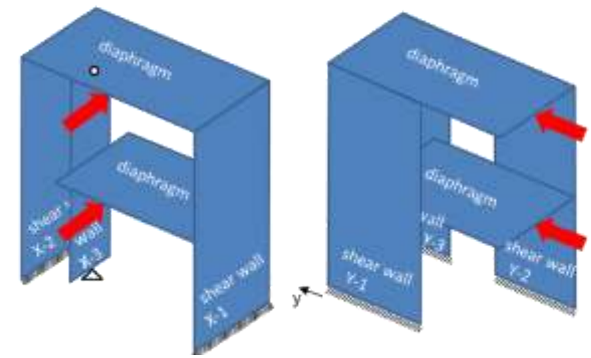
- reference PGA of 0.44g
- design elastic spectral acceleration (Sa,e,d) equal to 1.08g

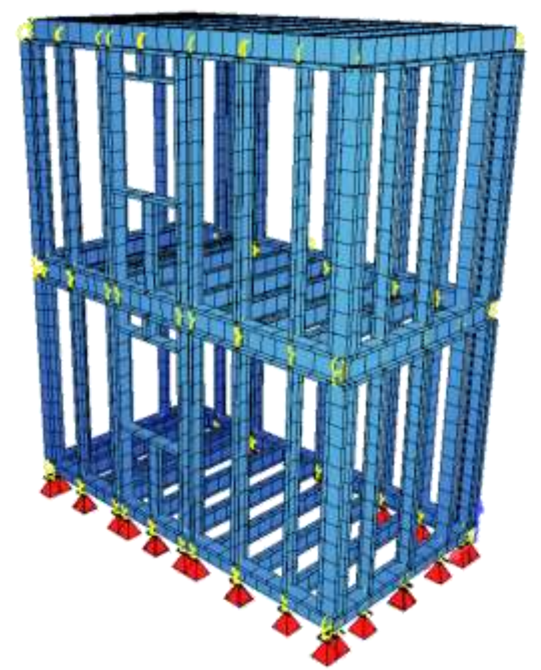
Behaviour factor q: 3.0

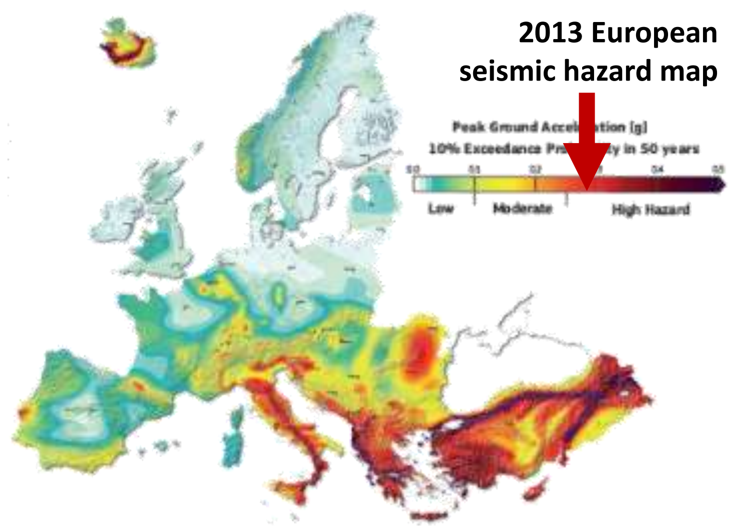
Overstrength factor Ω: 1.8

Elastic (Sa,e,d) and anelastic (Sa,a,d) design response spectra







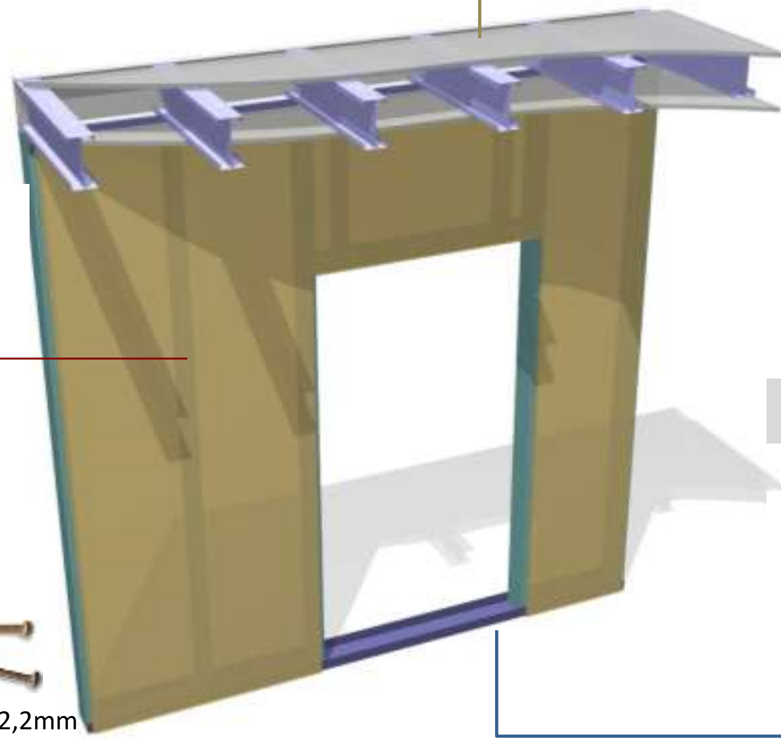


2013 European seismic hazard map

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 prototypes (w/ and w/o finishing)
	Total no. of tests	***81***

Micro-Scale tests: shear tests on connections

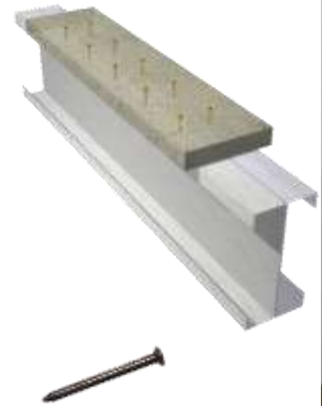


Panel-to-steel connections for walls

Total tests: 11

Balistic nails 2,2mm

Panel-to-steel connections for floors



Balistic nails 3,4mm

Total tests: 7

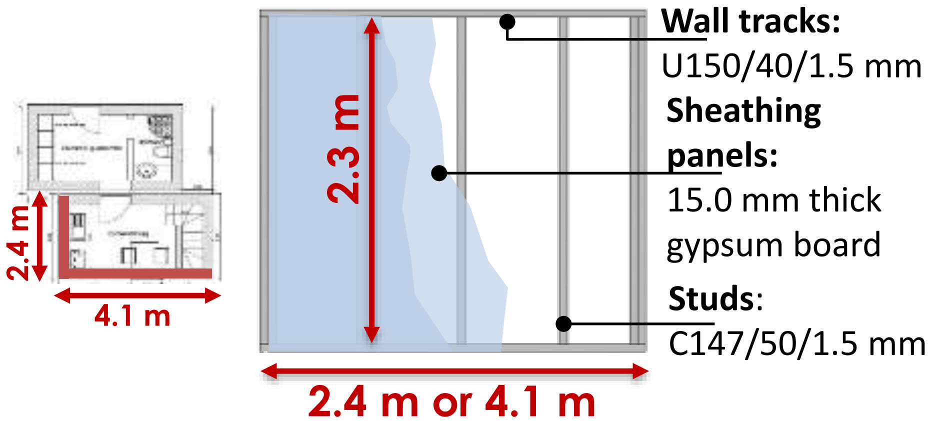
Steel-to-steel connections

Clinching 8mm

Total tests: 15

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

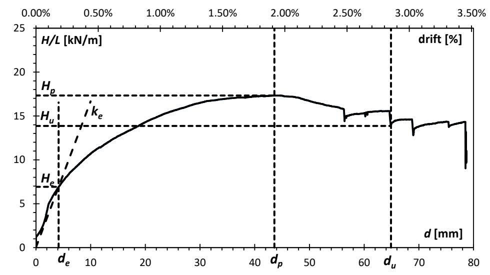
Specimen typologies and test program

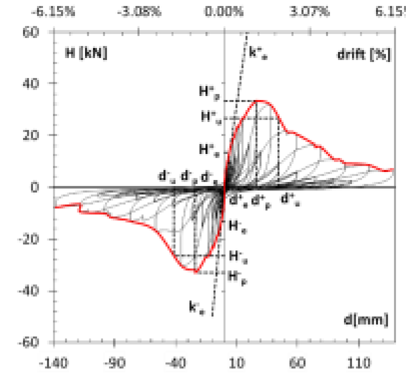


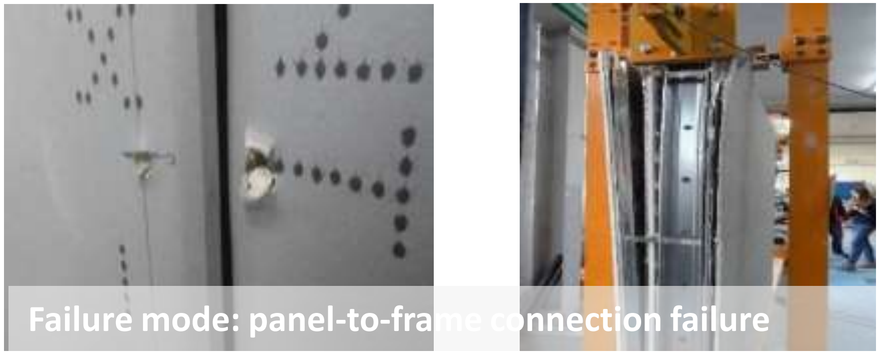
Label	Geometry	Finishing	Load type	No. tests
WS_2400_M	2.4 m x 2.3 m [A]	NO	Monotonic	1
WS_2400_C	2.4 m x 2.3 m [A]	NO	Cyclic	1
WS_4100_C	4.1 m x 2.3 m [B]	NO	Cyclic	1
WF_2400_C	2.4 m x 2.3 m [A]	YES	Cyclic	1

1 monotonic test and 3 cyclic tests

Experimental results







Experimental program for shake table tests

Elissa Mock-up configuration	Dynamic identification tests	Dynamic earthquake tests
Only Structure (Without finishing)	5 tests (0,05 – 0,10 g)	-
Complete construction (With finishing)	11 tests (0,05 – 0,10 g)	28 tests (5 – 150 % Scaling Factor)

Bare structure (without finishing)


Exterior wall panels nailing


Walls and floors lifting


Whole bare structure

Complete structure (with finishing)

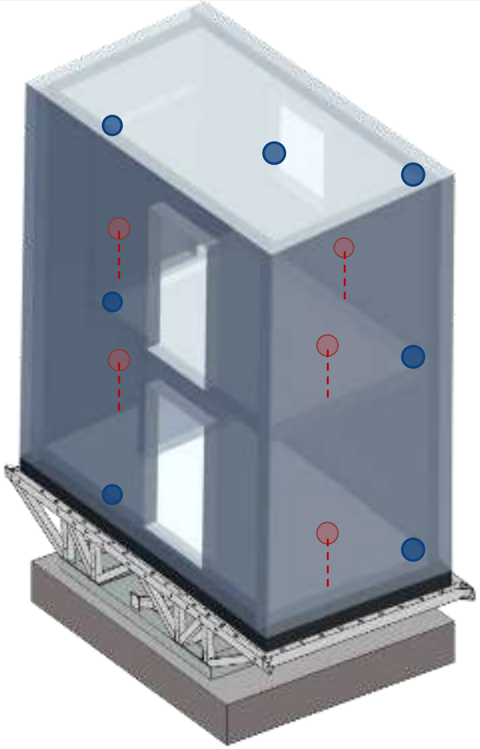

Exterior wall panels fixing


Interior wall panels fixing


Whole complete structure

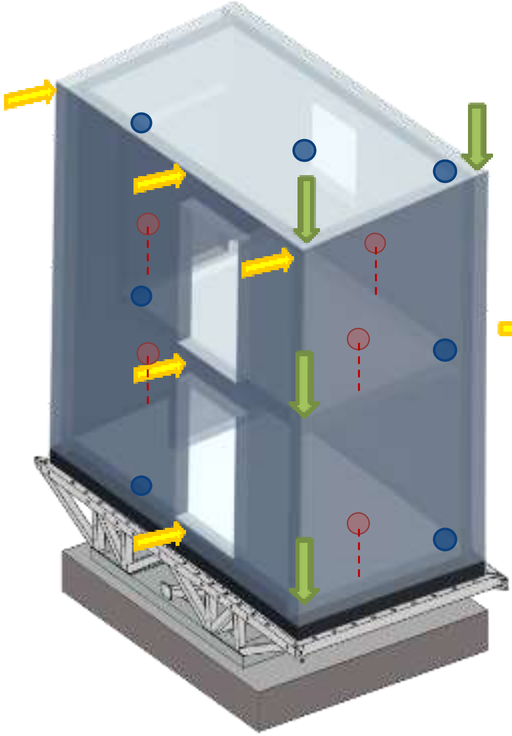
Instrumentation

Bare structure (without finishing)



- Triaxial accelerometers on walls (5)
- Triaxial accelerometers on floors (7)

Complete structure (with finishing)



- Triaxial accelerometers on walls (5)
- Triaxial accelerometers on floors (7)
- Laser sensors for horizontal displacement measurement (5)
- Laser sensors for vertical displacement measurement (4)

Max number of sensors: 12 accelerometers + 9 laser distance meters

Triaxial accelerometers

Laser distance meters

Dynamic earthquake tests - Input: 2009 L'Aquila Earthquake

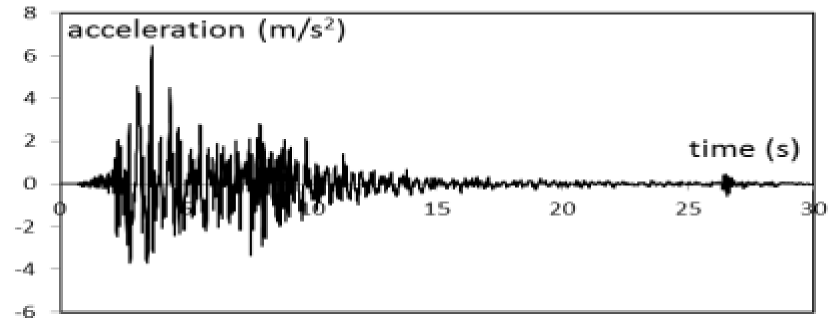
Mercalli Intensity (effects): 8-9
Richter magnitude (energy): 5.8

Damage caused by Aquila earthquake on traditional buildings

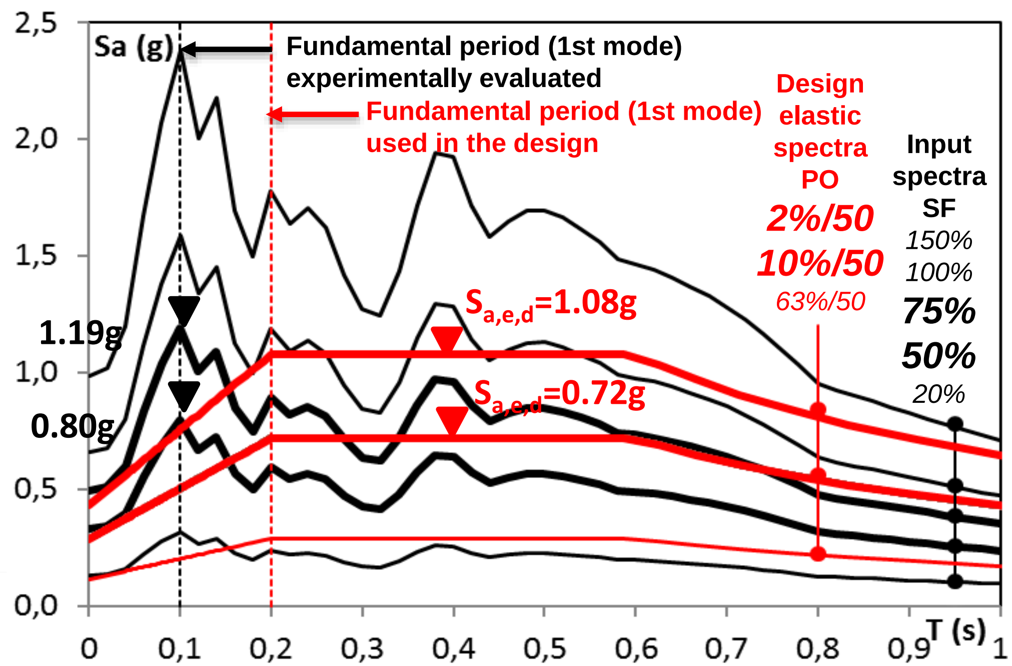
SELECTED GROUND MOTION

Event: *L'Aquila - April 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

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%***

Main outcomes

1 Results of shake table tests on the whole building showed that the maximum inter-storey drift was very small and the residual inter-storey drifts were negligible, evidencing a **very modest inelastic behaviour and a very small damage** in both structural parts and finishing materials for all scaling factors (from 5% to 150%).

2 The seismic response of the whole building were significantly altered by the **box*-behaviour** and **presence of nonstructural systems**, which caused a significant decreasing of the fundamental vibration period and a very high overstrength.

* methodologies for the prediction of the overall response of CFS buildings that are able to take into account the box building behaviour and the strengthen function of the non-structural materials should certainly be further investigated

3 Numerical models were developed using OpenSees Software and **both structural and non-structural parts** were modelled. A good agreement in terms of fundamental period and inter-storey drift time history was found.



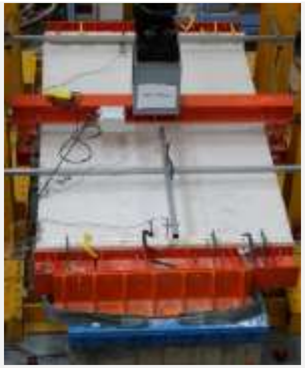
Structural all-steel systems

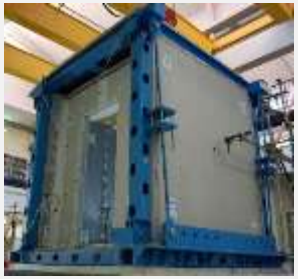
- Italian national research project
  ReLUIS-DPC, Line 1, years 2010-2013
- National research project
  Lamieredil-UNINA Project, years 2014-2017



Structural sheathing-braced systems

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

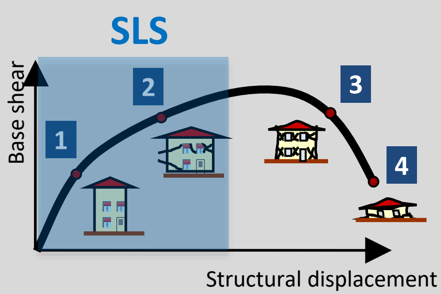



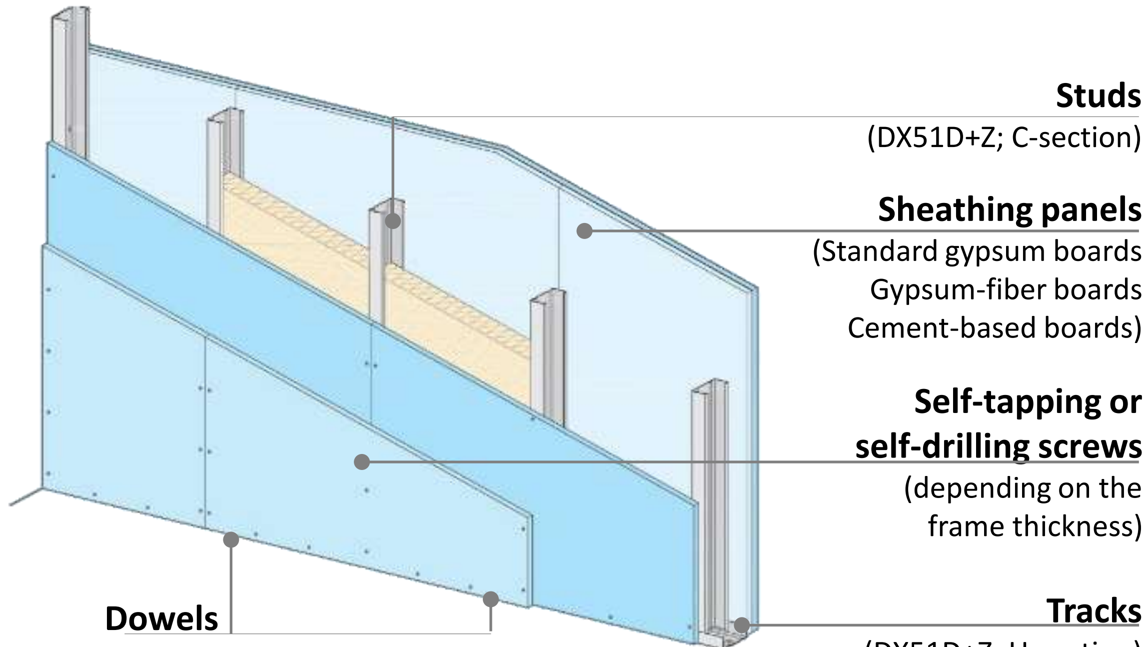


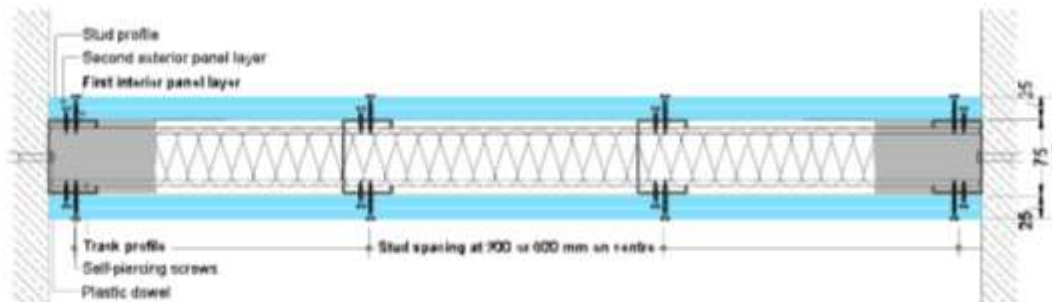
Drywall non-structural building components

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

Serviceability Limit States







Research objectives

1 Last earthquakes occurred in the most advanced countries demonstrated the **vulnerability** of nonstructural elements to relatively low seismic intensity levels and showed that their damage or collapse might have severe consequences in terms of **economic, social and human life losses,** even in the case in which no damage occurred in structural elements.

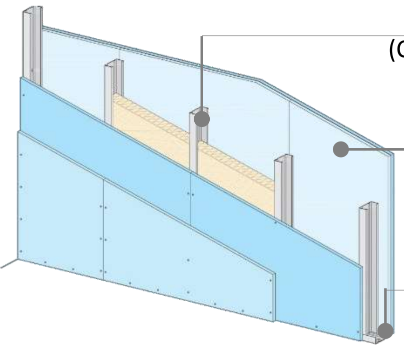
2 Lightweight steel drywall products can represent a **valid alternative** to traditional systems for nonstructural architectonic applications in seismic areas. In fact, lightweight steel drywall products can guarantee a very **good seismic behaviour with respect to damage limit states**, mainly thanks to their lightness and the possibility to easily improve their seismic response by means of relatively simple constructional details.

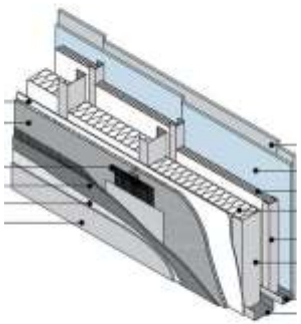
3 The Knauf Project was devoted to the **experimental structural characterization** of the seismic behaviour of nonstructural architectonic building systems, consisting of partition walls, façade walls and suspended ceilings, and the interaction between them and structural elements by means of specific connections (**basic and enhanced anti-seismic solutions**). The goals were to evaluate the **seismic fragility** and to define the **observed damages** of non-structural components.

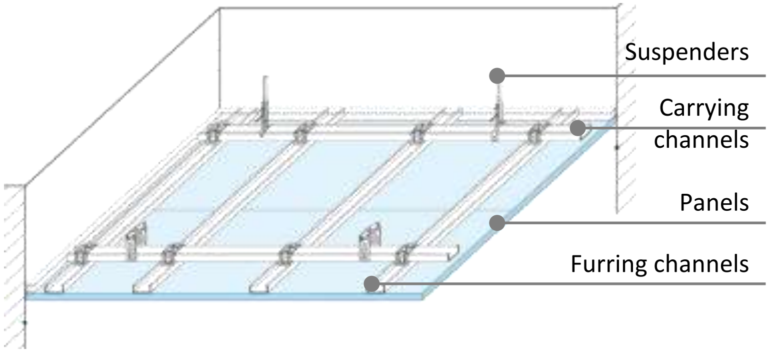
Seismic response evaluation of non-structural lightweight steel drywall building components

The research project is devoted to investigate the seismic performance of lightweight steel gypsum-sheathed interiror partition walls, exterior façade walls and suspended continuous ceilings and the interaction between them and other structural elements.

Interior partition walls	Exterior façade walls	Suspended continuous ceilings







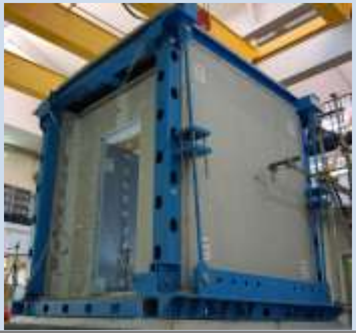
General experimental program

Test type		no. tests
Material and component tests	Steel material	12
	Self-tapping and self-drilling screws	42
	Sheathing panels	30
	Panel-to-steel connections	60
Wall tests	In-plane quasi-static reversed cyclic tests	12
	Out-of-plane monotonic tests	22
	Out-of-plane dynamic identification tests	11
Shake table tests	Dynamic identification and earthquake tests	83 + 75 tests on 4 protoypes
	Total no. of tests	*349*









Tests on materials, components and connections

Since the response of lightweight steel gypsum board partition walls is strongly influenced by the local response of the different materials composing these systems, a large number of **tests on materials** and **components** was carried out in order to characterize their mechanical properties.

Tensile tests on steel material

Total tests: 12

Shear tests on self-tapping and self-drilling screws

Total tests: 42

Bending tests on sheathing panels

Total tests: 30

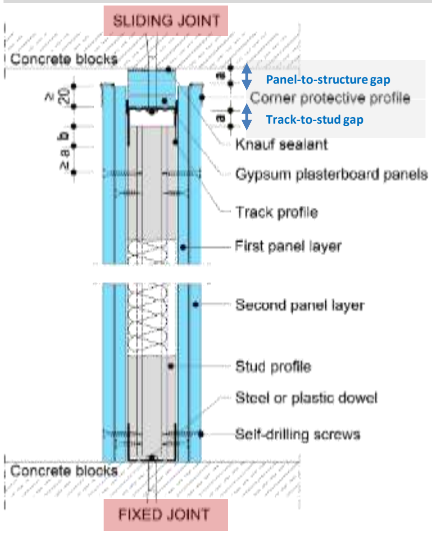
Shear tests on screwed panel-to-steel connections

Total tests: 60

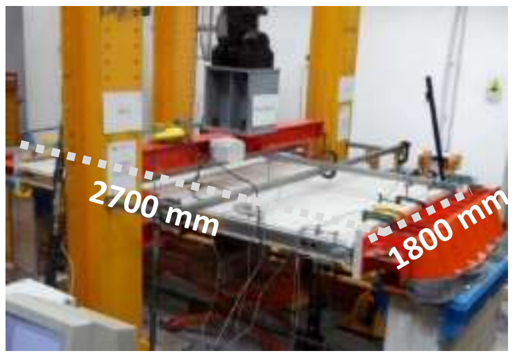
Out-of-plane quasi-static monotonic and dynamic identification tests

Experimental assessment of the out-of-plane seismic response of indoor partition walls for evaluating the **wall resistance** and the **fundamental vibration period.**

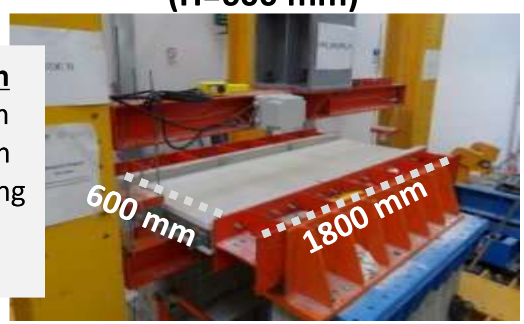
Specimen typologies



Conventional partition (H=2700 mm)



Non-conventional partition (H=600 mm)



Parameters under investigation

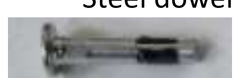
- **wall height:** 600 or 2700 mm
- **stud spacing:** 300 or 600 mm
- **joint type-gap:** fixed/sliding (a=0 mm/20mm/30 mm)
- **dowel type:** plastic or steel

a= 0

a= 20

a= 30

Steel dowel



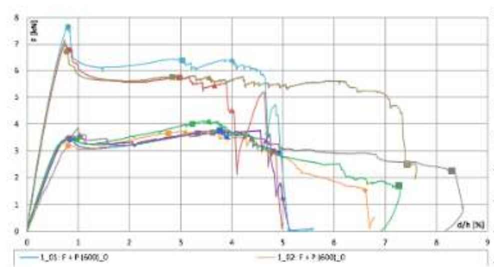
Plastic dowel

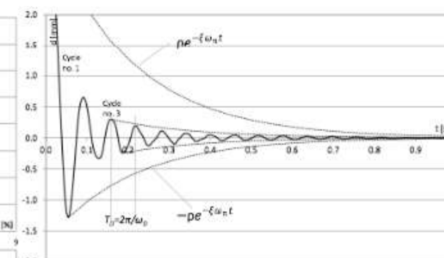
Test program and main results

Conventional partition

MONOTONIC (QUASI-STATIC) TESTS (No. 14)
for evaluating the **wall resistance (F_{Rd})**
(Collapse phenomena were to the wall framing local buckling)

STEP-RELAXATION (DYNAMIC) TESTS (No. 11)
for evaluating the **fundamental vibration period** (T_a)





Non-conventional partition

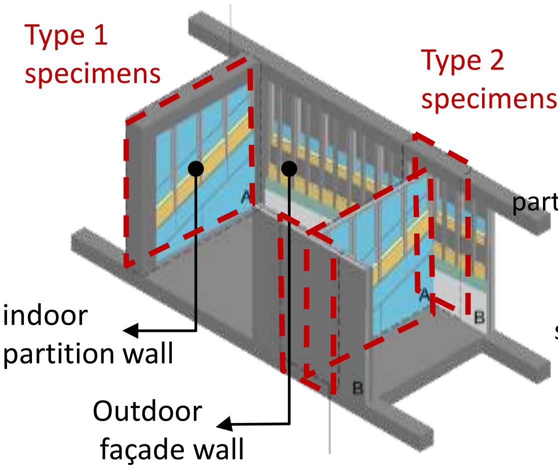
MONOTONIC (QUASI-STATIC) TESTS (No. 8)
for evaluating the behaviour of joints between partition walls and reinforced concrete surrounding structures.
(Collapse phenomena were related to the joint collapse)

Total out-of-plane monotonic tests: 22
Total dynamic identification tests: 11

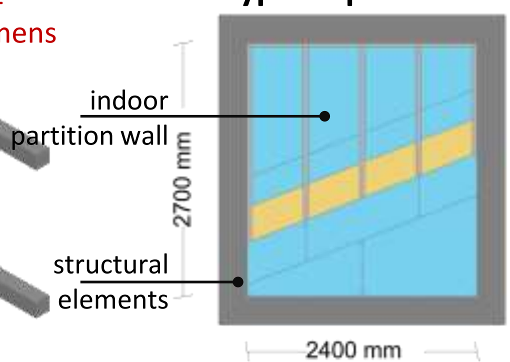
In-plane quasi-static reversed cyclic tests on partition walls

Experimental assessment of the in-plane seismic response of the interior partition walls, also considering the interaction with exterior façade walls, and the **related damage levels** in accordance with the inter-storey drift limits defined by the European code.

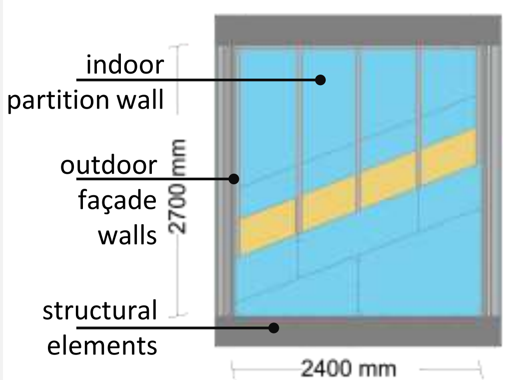
Specimen typologies



Type 1 specimen



Type 2 specimen



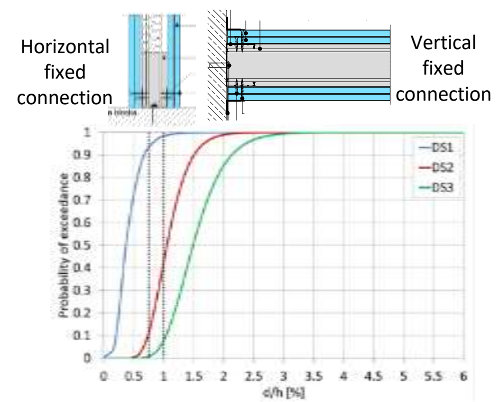
Total tests: 12

Parameters under investigation

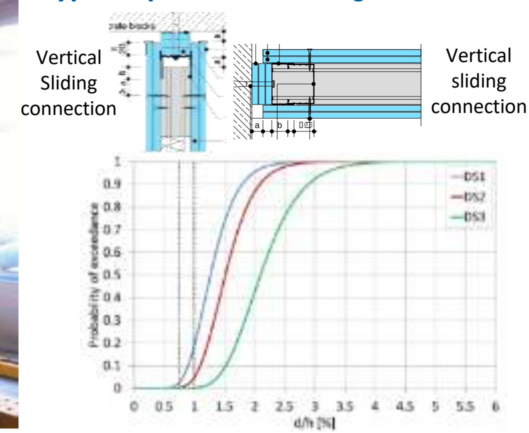
- **Type of horizontal and vertical connections to structure:** Fixed/sliding
- **Stud spacing:** 300 or 600 mm
- **Type of sheathing panel :** standard gypsum/gypsum fibre boards
- **Type of jointing finishing:** glass fibre or paper tape and self-adhesive paper tape

Main results-fragility curves

Type 1 specimens-*Fixed connections*



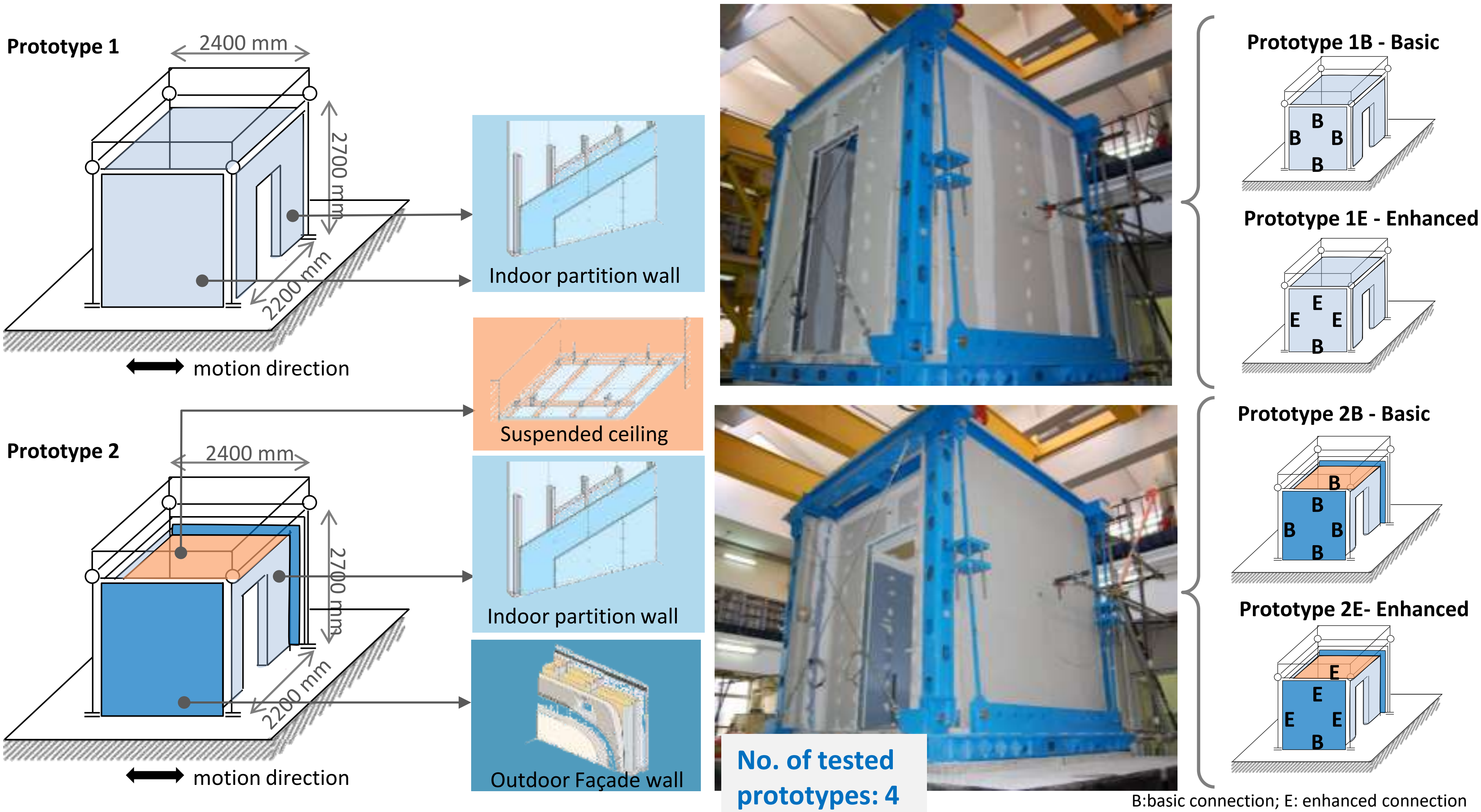
Type 1 specimens-*Sliding connections*



Shake table tests on partition walls, façade walls and suspended continuous ceilings

Assessment of the seismic behavior under dynamic loading conditions of four prototypes made of **different non-structural components differently connected** between them and to the structural systems.

Tested prototypes

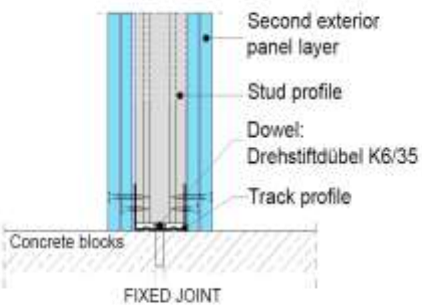


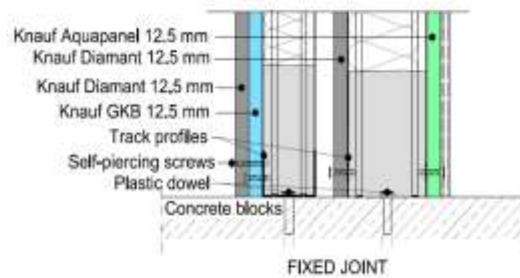
Shake table tests on partition walls, façade walls and suspended continuous ceilings

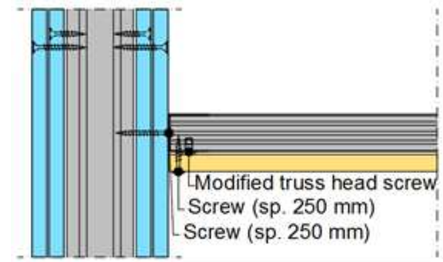
Connection typologies

Interior Partition Wall | Exterior Façade Wall | Suspended Ceiling

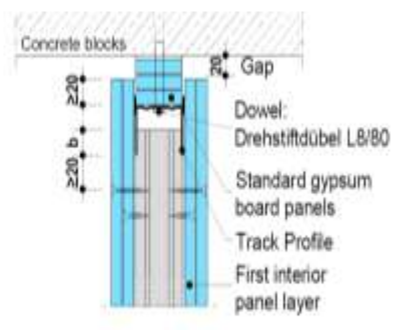
BASIC SOLUTION
FIXED CONNECTIONS

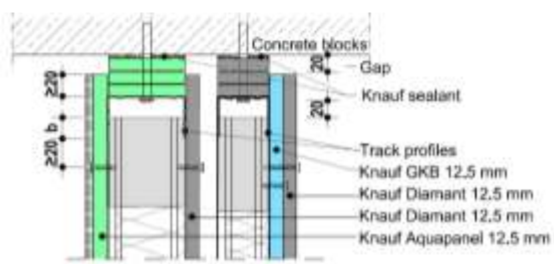


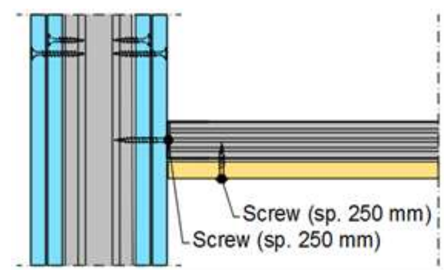




ENHANCED SOLUTIONS
SLIDING CONNECTIONS







Experimental program for shaking table tests

Tested prototypes	Dynamic identification tests	Dynamic earthquake tests
Bare structure	7 tests (10– 30 % Scaling Factor)	5 tests (10 – 55 % Scaling Factor)
Prototypes 1B – 1E	48 tests (5– 20% Scaling Factor)	43 tests (5 – 100% Scaling Factor)
Prototypes 2B – 2E	35 tests (5– 10% Scaling Factor)	32 tests (5 – 120 % Scaling Factor)

Bare structure

Bare Steel Frame

Prototype 1

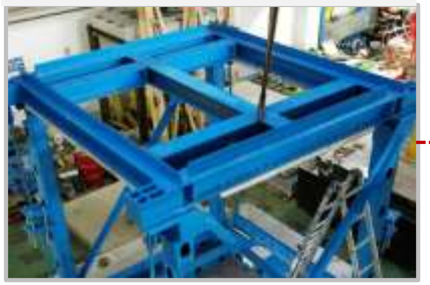
Only Interior Partition Walls

Prototype 2

Interior partition walls +
Exterior façade walls +
Suspended ceiling

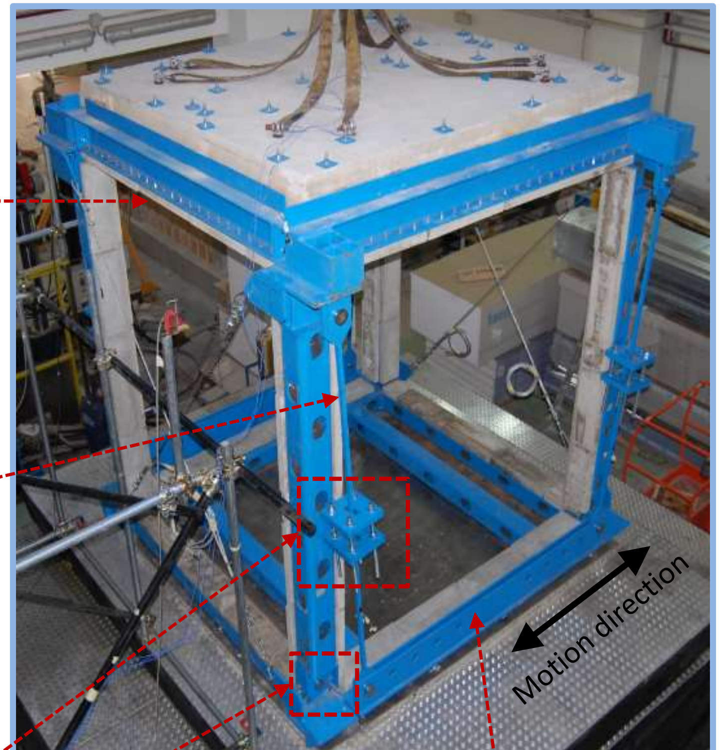
Test Set Up – Bare structure

Top beam grid



Diagonals

Pre-loading device



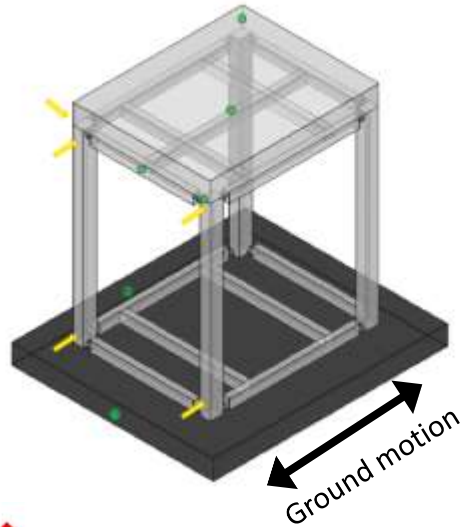
Column hinge

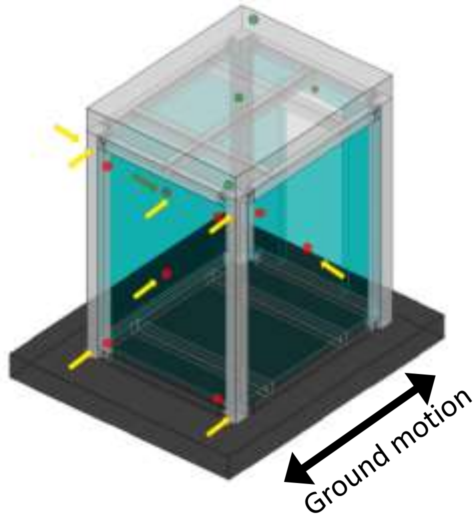
Bottom beam grid

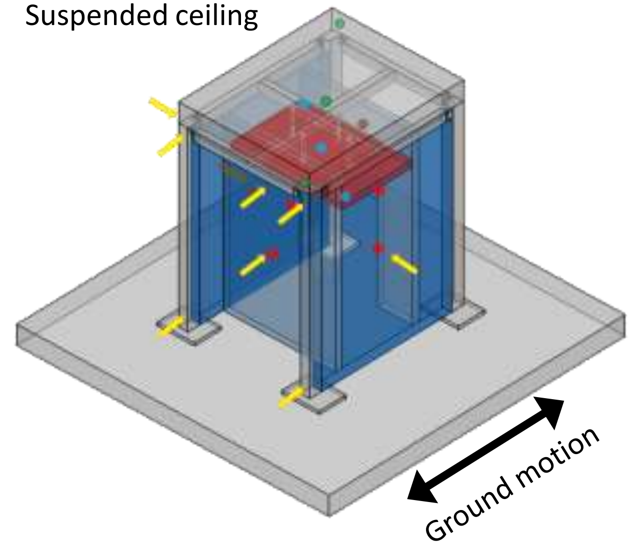
COLUMNS	***Cross section***	***TUB 200x200x16***
	Height	2800 m
	Steel grade	***S355JR***
BEAMS	***Cross section***	***HE200B***
	Length	from 2900 to 1050 mm
	Steel grade	***S355JR***
CONCRETE BALLAST	Resistance class	C25/30
	Area	6,5 m^2
	Thickness	200 mm
	Weight	3340 kg
BASE BEAM Type 1	***Cross section***	***TUB 260x180x16***
	Length	2.50 m
	Steel grade	***S355JR***
BASE BEAM Type 2	***Cross section***	***TUB 180x180x10***
	Length	2.24 m
	Steel grade	***S355JR***
DIAGONALS	***Cross section***	***24x26***
	Length	2.87 m
	Steel grade	***RAEX 450***
STEEL GRADE	S355JR	RAEX 450
σ_y [Mpa]	355	1250
σ_u [Mpa]	600	1450

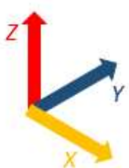
Instrumentation

Bare structure	Prototype 1	Prototype 2
Bare Steel Frame	Only interior partition walls	Interior partition walls + Exterior façade walls + Suspended ceiling









Displacement Laser Sensor

Triaxial Accelerometer on set up

Triaxial Accelerometer on walls

Triaxial Accelerometer on ceilings

Max number of sensors: 12 accelerometers + 9 laser distance meters

Displacement Laser Sensor

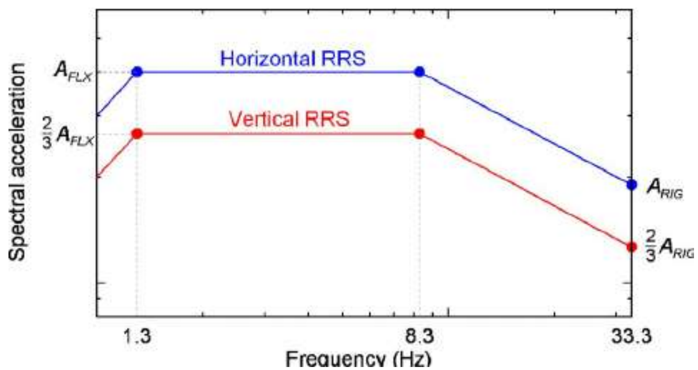
Triaxial Accelerometer

Loading protocol – Test Input

ICBO-AC156 code

The **time history** was **artificially defined** in order to match the Required Response Spectrum (RRS), provided by the **ICBO-AC156 code**. The input accelerogram spectrum developed from the selected time history shall be in the range from **90** percent and **130** percent of RRS and the matching procedure is valid for frequency range from **1.3 to 33.3 Hz**.

Required Response Spectrum normalized for the component

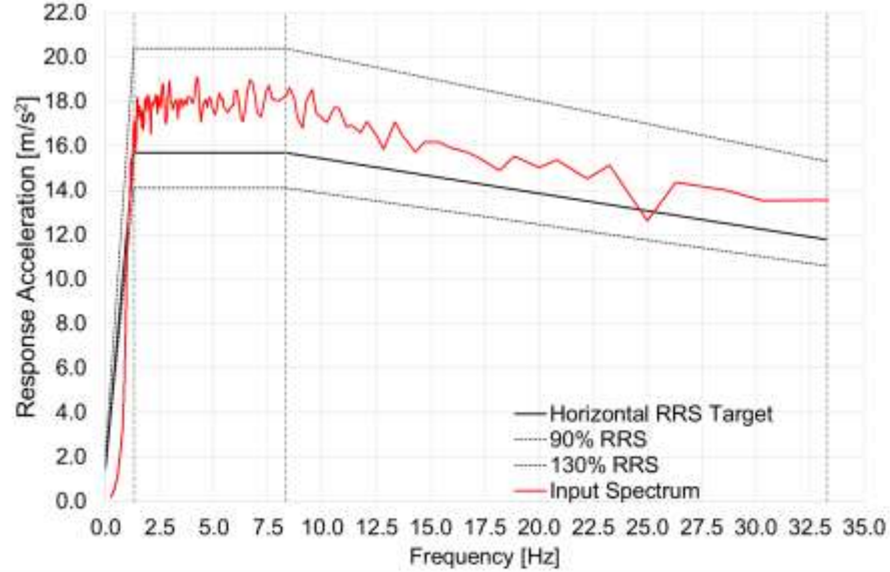


Assumptions

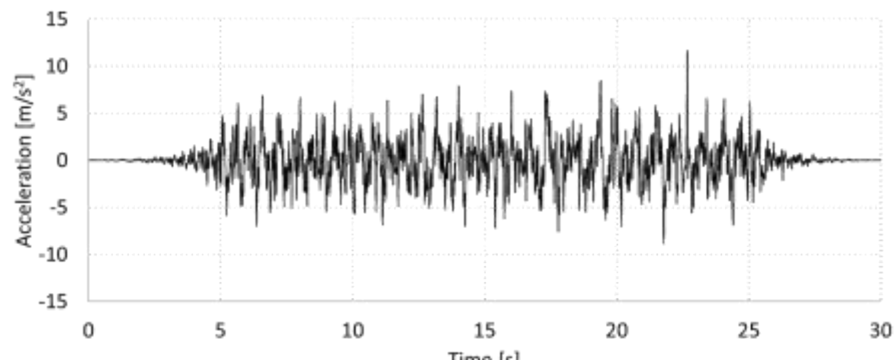
Spectral acceleration at short periods (S_{DS}): **1.0 g**
Scaling factor: from **5%** to **120%**

Input

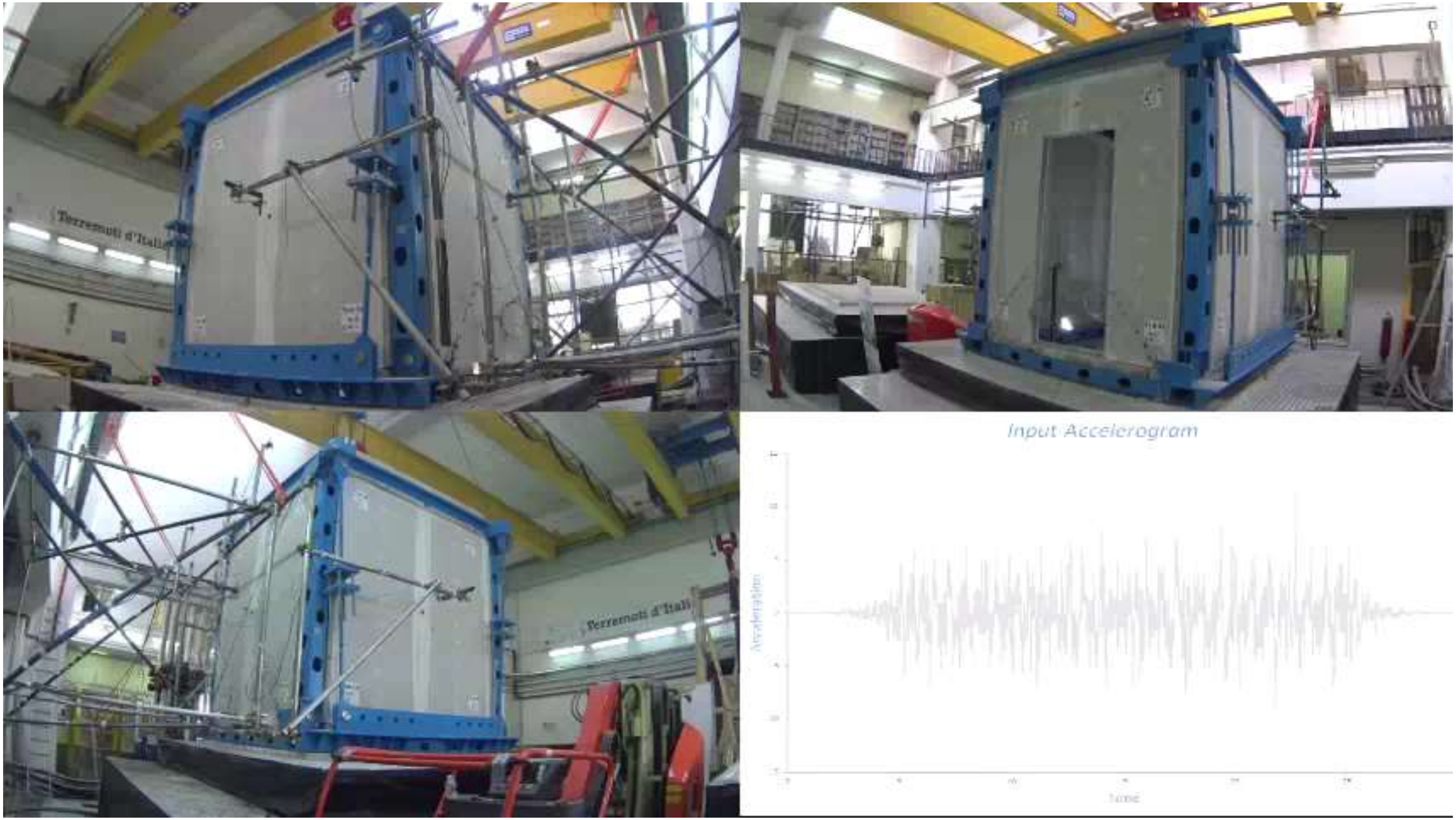
Required Response spectrum target and Input Spectrum



Input time history – Unidirectional



Earthquake test on shake table of Prototype 1



*Videos recorded during the Earthquake tests with scaling factor of **85%** on Prototype 1B-II- max drift reached 3.20%*

Observed damage phenomea

Interior partition walls


1. Drop of gypsum dust

3. Detachment of joint paper


5. Cracks in the panels


4. Detachment between walls and structural elements

Exterior façade walls


2. Drop of basecoat dust


6. Corner crushing of panels

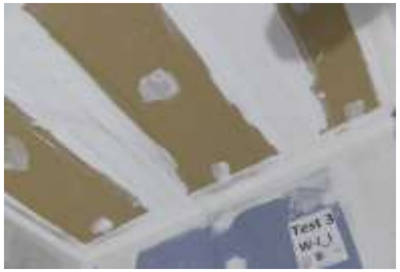

8. Collapse of panel-to-frame fixings

9. Rupture of panel portions

10. Out-of-plane collapse of panel


7. Crushing of exterior façade wall corner

Suspended ceiling



Very low damage observed for suspended ceiling

Fragility curves

1 Definition of damage limit-states (DS)

DS1 – superficial damage, it requires minimum repair with plaster, tape and paint

DS2 – local damage of sheathing panels and/or steel frame components, it required the removal and replacement of elements (sheathing panels and/or local repair of steel frame components)

DS3 – severe damage, it requires the replacement of part or whole component

2 DS-damage correlation

The observed damages were associated to the damage limit states depending on the required level of repair

Observed damage phenomena	DS1	DS2	DS3
1. Drop of gypsum and/or plaster dust	•		
2. Detachment of joint tape	•		
3. Detachment between walls and surrounding structural elements		•	
4. Crack in panels		•	
5. Corner crushing of panels		•	
6 Collapse of panel-to-frame fixings		•	
7. Rupture of panel portions			•
8. Out-of-plane collapse of panels			•

3 DS - Drift--Damage correlation

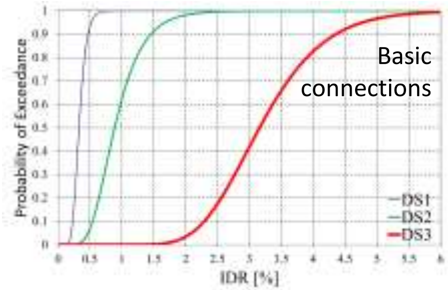
The drift levels triggered the damage limit states were recorded for each specimen and correlated to the damage limit states

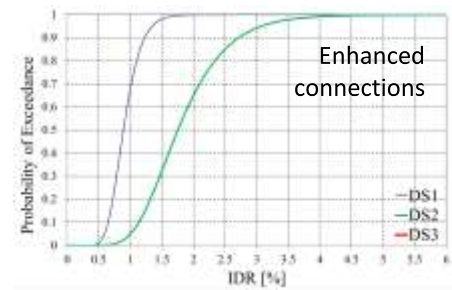
Damage limit states	Interior Partition Walls		
	1_B-I	1_B-II	1_E
	Drift [%]	Drift [%]	Drift [%]
DS1	0.32%	0.28%	0.89%
DS2	0.66%	1.19%	1.39%
DS3	3.12%	3.20%	-

Damage limit states	Exterior Façade Walls	
	2_B	2_E
	Drift [%]	Drift [%]
DS1	0.31	1.11
DS2	1.17	2.44
DS3	3.74	4.54

4 Fragility curves

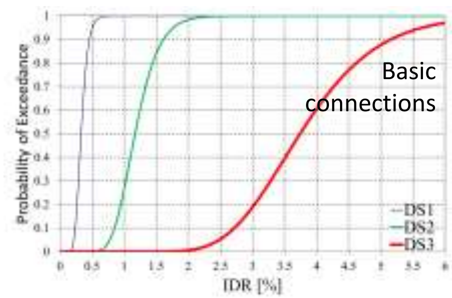
Interior Partition walls

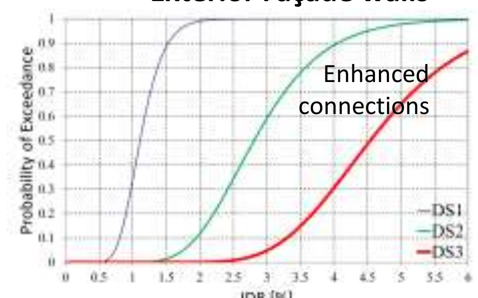




Interior Partition Walls showed an higher seismic fragility than Exterior Façade walls for each DS

Exterior Façade walls





Basic solutions showed an higher fragility than Enhanced solutions for each DS

Main outcomes

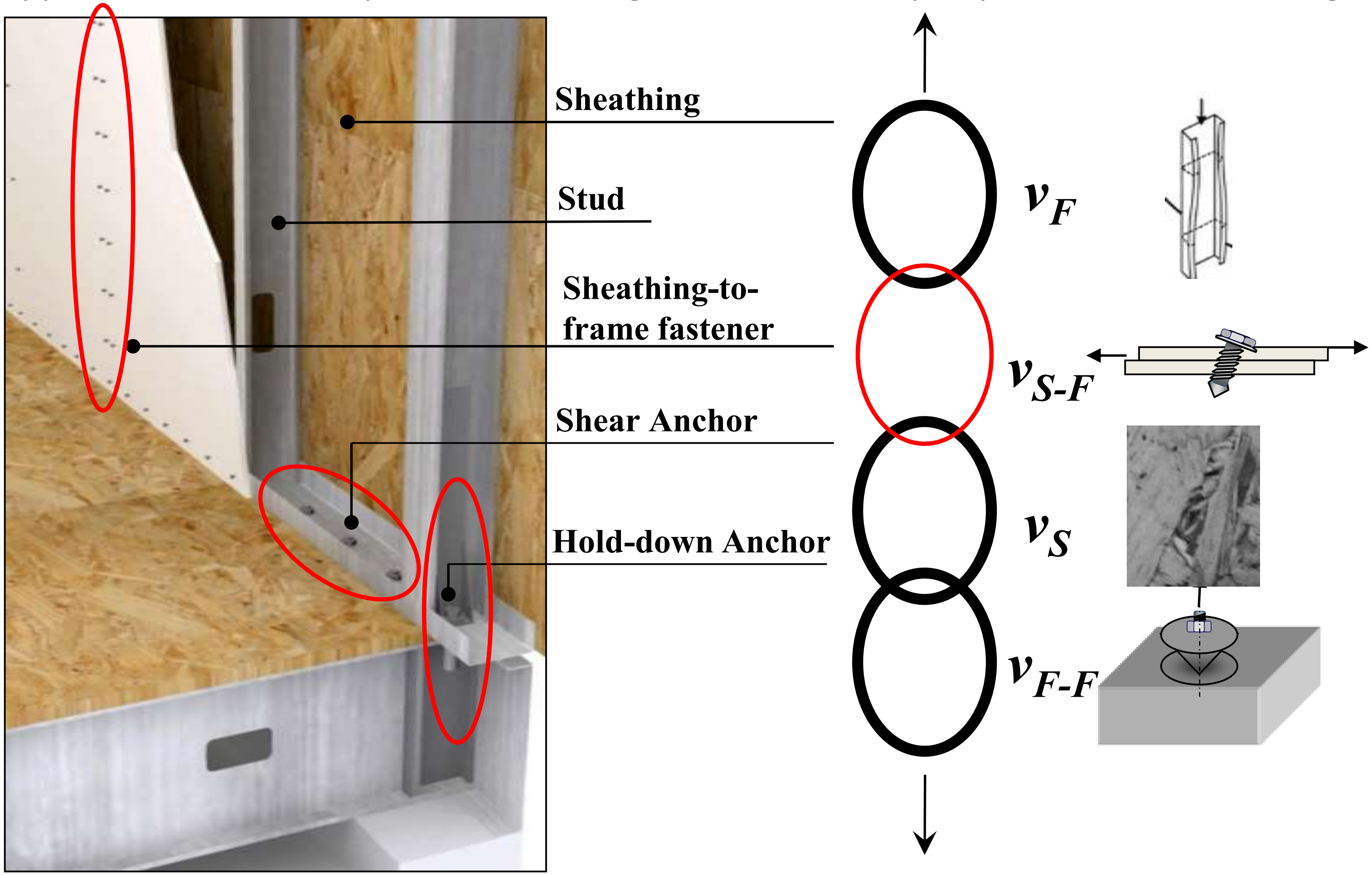
1 **The vulnerability** of lightweight steel drywall systems to relatively low seismic intensity levels is confirmed, but they can guarantee a very **good seismic behaviour with respect to damage limit states.**

2 About the in-plane behaviour of nonstructural walls, the main findings of experimental tests showed that if no specifications are given on the connections between walls and surrounding elements (**Basic solutions**), an inter-storey drift of **0.75%** can be considered an adequate limit for damage limit states related with limited level of damage and required repair action (serviceability limit states), whereas if anti-seismic connections (**Enhanced solutions**) are used (i.e. sliding-connection) an acceptable limit of the inter-storey drift for serviceability limit states can be assumed equal to **1.00%.**

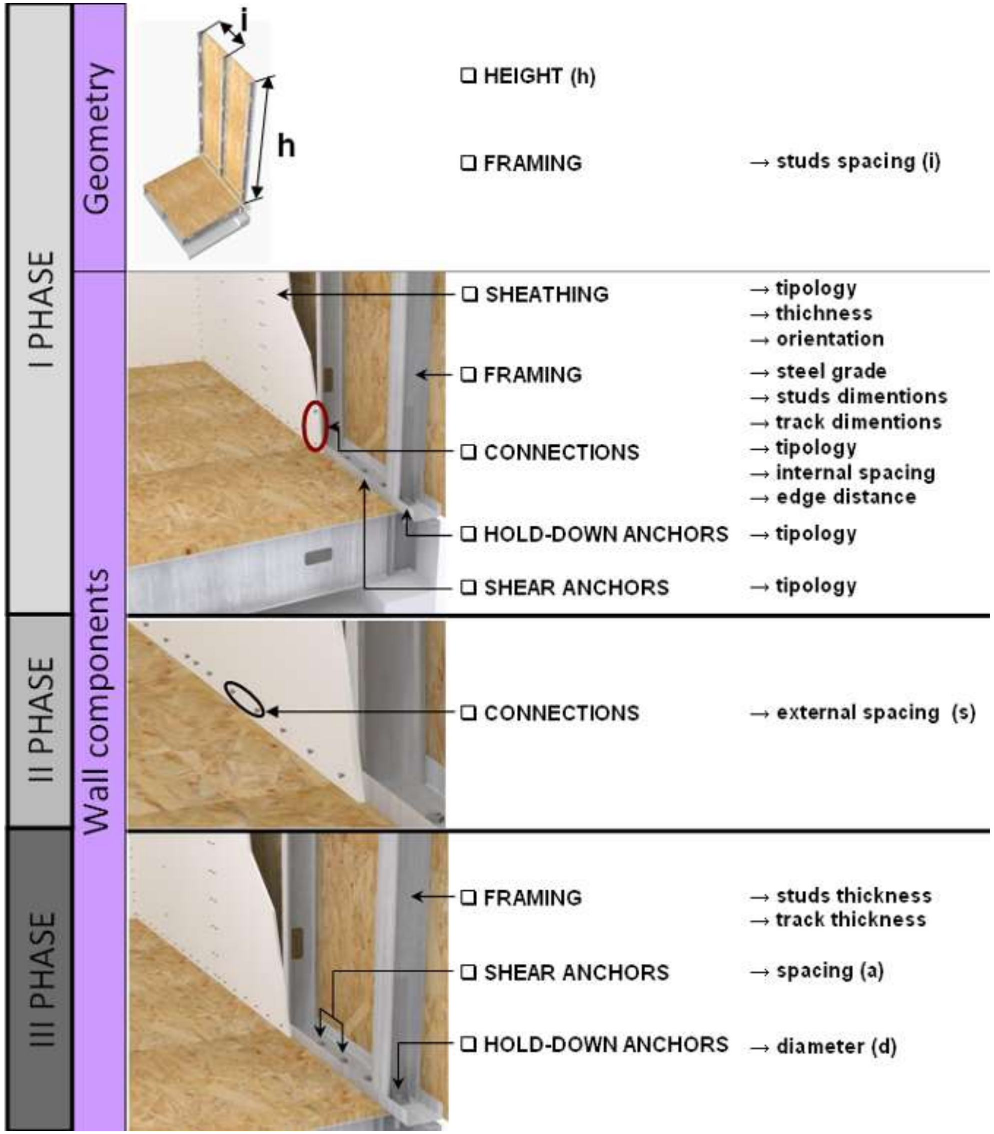
3 About the out-of-plane response, the main findings of experimental tests showed that nonstructural walls exhibited a very good seismic behaviour due to their high "dynamic" stiffness (**low dynamic amplification**) and low weight (**low seismic mass**). In addition, the seismic response was not affected by the connections between walls and surrounding elements (i.e., Basic solutions or Enhanced solutions)

Development of a seismic design procedure

A seismic design procedure refers to CFS walls sheathed with wood-based or gypsum-based panels, has been developed to propose a design tool that can be readily adopted by designers for the seismic design
The proposed approach the wall components are designed in such a way to promote the sheathing fastener failure.



Development of a seismic design procedure



PHASE 1:
the "assigned" design parameters does not depend on the seismic design, but usually derives from architectural and technological choices and design for vertical loads.

PHASE 2:
Only the assessment of the sheathing fasteners exterior spacing (s) directly derives from seismic analysis results.

PHASE 3:
The definition of stud thickness, hold-down anchor diameter and shear anchor spacing is carried out only on the basis of "capacity design" criteria.

Development of a seismic design procedure: Design Chart

I PHASE

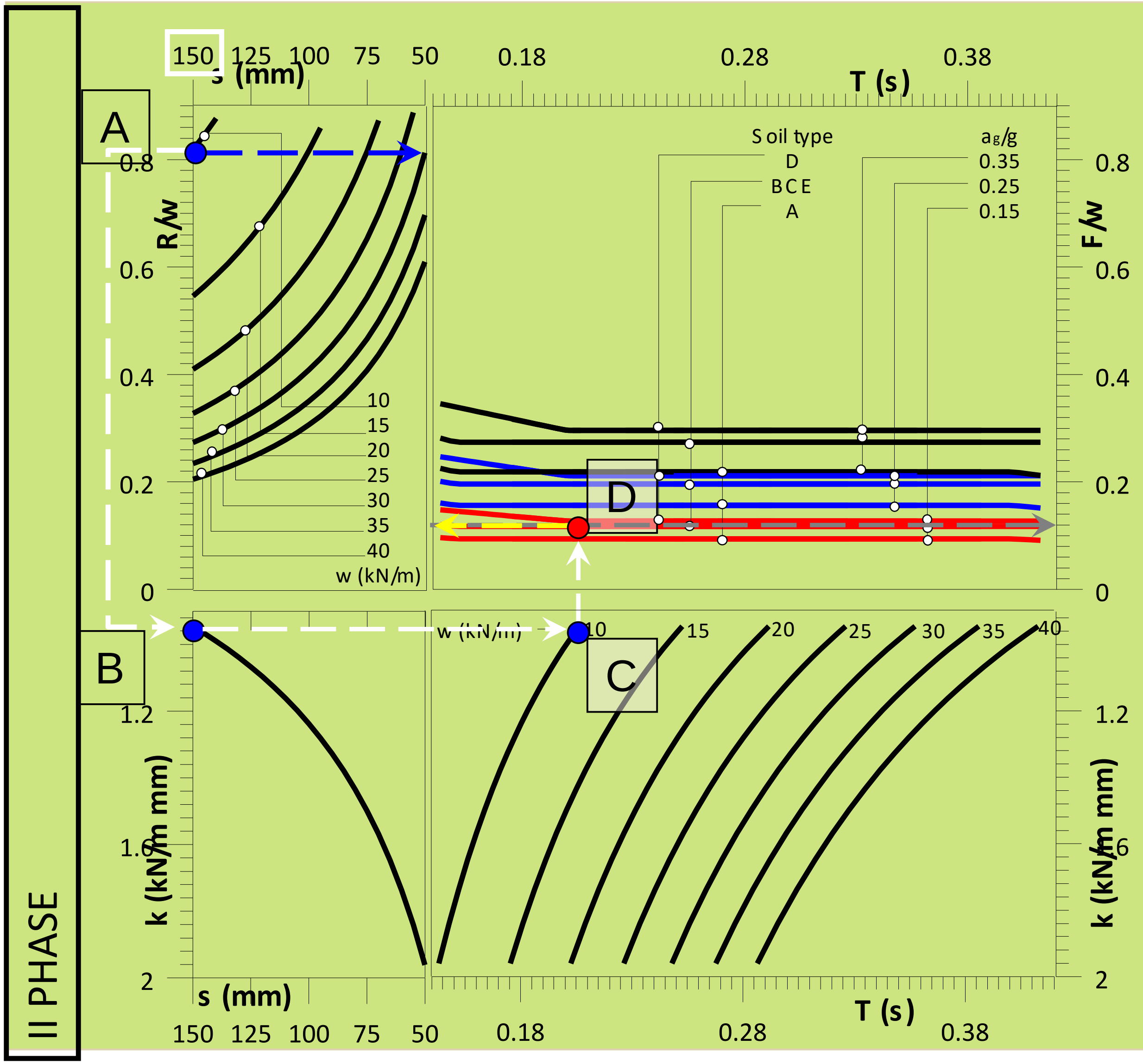
Geometry

II PHASE

Screw connection spacing

III PHASE

Frame
Shear anchors
Tension anchors



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2. Seismic design of LWS Constructions

3. Researches at the University of Naples Federico II

4. Recent applications in italy

5. Concluding Remarks

Residential building / Pordenone

The construction is a one-family house of about 210 m^2.

Project data

Location:
Pordenone, Italy

Client:
Private

Designers:
Francesco Mariuzzo

Builder:
Impresa edile Andrea dall'Acqua (Treviso)

Residential building, Pordenone, Italy

Residential building / Verona

The construction is a one-family house of about 250 m^2.

Project data

Location:
Verona, Italy

Client:
Private

Designers:
Innokasa s.r.l. (Verona)

Builder:
Innokasa s.r.l. (Verona)

Residential building, Verona, Italy

Residential building / Monza Brianza

The construction is a one-family house of about 250 m^2. The structure is a stick built construction.

Project data

Location:
Monza Brianza, Italy

Client:
Private

Designers:
Cogi s.r.l. (Monza)

Builder:
Cogi s.r.l. (Monza)

Typology:
CFS solution

Residential building, Verona, Italy

"CasaLow" residential building / Bologna

Crevalcore suffered great losses during the magnitude 5.9 Emilia Earthquake in 2012. Designed especially for the fast and realizable recovery of the infrastructure, a project called "CasaLow" was introduced. Node analysis and a special foundation were the basis for a lightweight steel structure according to the Italian building code. The motivation was to combine renewable energy, an innovative envelope as well as comfort and earthquake safety for single family houses at affordable prices.

Project data

Location:
Bologna, Italy

Year:
2013

Client:
Private

Designers:
Studio=2A (Tivoli)

Builder:
Nuova Rinnova P. Testi (Cesena)


Residential building, CasaLow, Bologna, Italy

The "San Giacomo community" building / L'Aquila

The San Giacomo community building is one of the works financed by the Italian Caritas after L'Aquila earthquake in April 2009. This construction is realized with CFS panels and organized in 2 buildings: a two floors rectangular building for dining hall and bedrooms and a one floor octagonal building as a multi-purpose hall.

Project data
Location:
L'Aquila, Italy

Year:
2010

Client:
Italian Caritas

Designers:
Studio Pericoli (Roma)

Structural project:
Ing. Conflitti (Roma)


The San Giacomo community building. L'Aquila, Italy

Residential building / Catanzaro

Three-storey residential building with a gross floor area of about 280 m2. The lateral seismic resisting system was obtained by using "all-steel" solution. In particular, cold-formed steel stud walls braced with diagonal straps were used to counter the horizontal seismic actions.

Project data

Location:
Catanzaro, Italy

Client:
Private

Designers:
Michele Condino (Catanzaro)

Builder:
Condino Engineering (Catanzaro)

Residential building, Catanzaro, Italy

Residential building / Catanzaro

The construction is a one-family house of about 250 m2. The structure is a X-braced stick built construction made by 1.5 and 2 mm thick lightweight steel profiles.

Project data

Location:
Catanzaro, Italy

Year:
2011

Client:
Private

Designers:
Studio Boccafurri (Catanzaro)

Builder:
Condino Engineering (Catanzaro)

Detached house, Catanzaro, Italy

Single-storey residential building / Venticano

Project data

Location:
Località Campoceraso, Venticano (AV)

Year:
2017

Client:
Private

Designers:
Ufficio Tecnico G.G. Costruzioni s.r.l. (Avellino)

Builder:
GG Costruzioni (Avellino)

Structural typology:
Steel solution

Number of storey:
1

Area:
280 mq

Single-storey residential building / Mirabella Eclano

Project data of the vertical extension

Year:
2017

Client:
Private

Designers:
Ufficio Tecnico G.G. Costruzioni s.r.l. (Avellino)

Builder:
GG Costruzioni (Avellino)

Structural typology:
X-bracing solution

Number of storey:
1

Area:
150 mq

MINISTRY OF DEFENCE

Building construction of foundation and primary stage school "BFS"

Lago Patria – Naples, Italy (2009-2011)

classbasesKS2
classbasesKS2
classbasesKS1
classbasesKS1
changing rooms
under 2's room
plant room
group rooms
hall
over 2's room
office
classbasesFS2
teacher's room
community room
music room
classbasesFS1
interview room
waiting room
foyer
secretary room
medical room
deputy head
head
computer
art
science
food tech.
library

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

An emblematic application

Design
January 2009 – July 2009

Construction:
January 2010 – March 2011

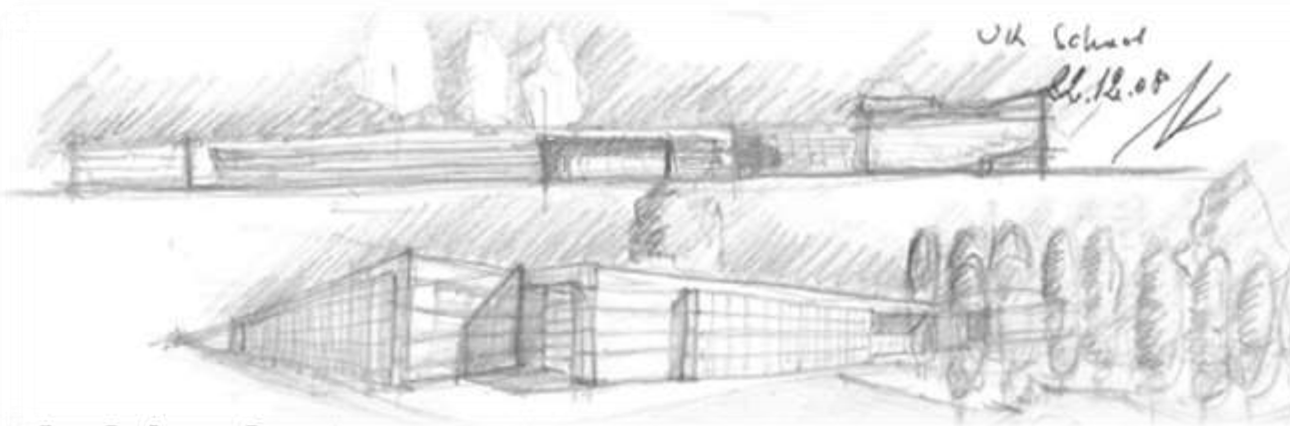
Total surface area:
15.850 m^2

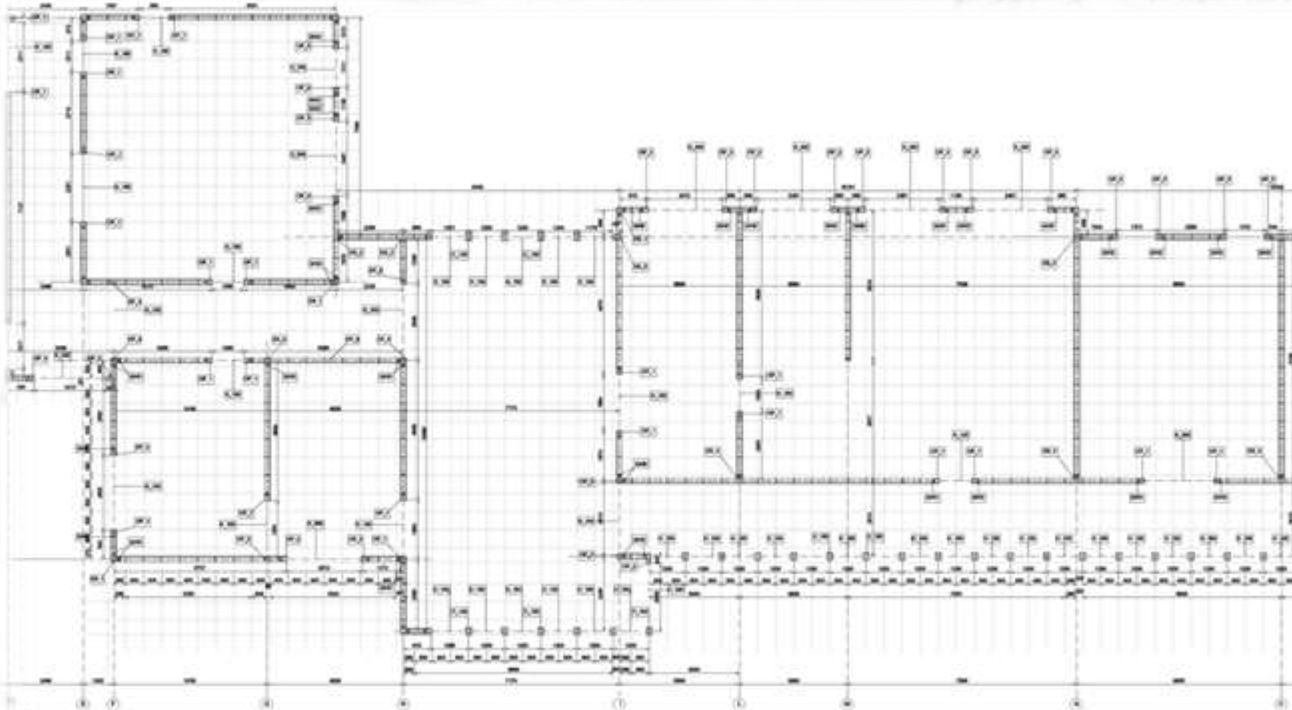
Covered area by steel structures: 3.000 m^2

Internal courtyards:
1.900 m^2

Roads and parking area:
3.700 m^2

An emblematic application





Owner:
Defense Estates UKNSU

Contractor:
COSAP

Architects:
Arch. Fiorenzo Petillo (*Team leader)*
Dr. Arch. Brigida Santangelo
Arch. Enza Terzigni

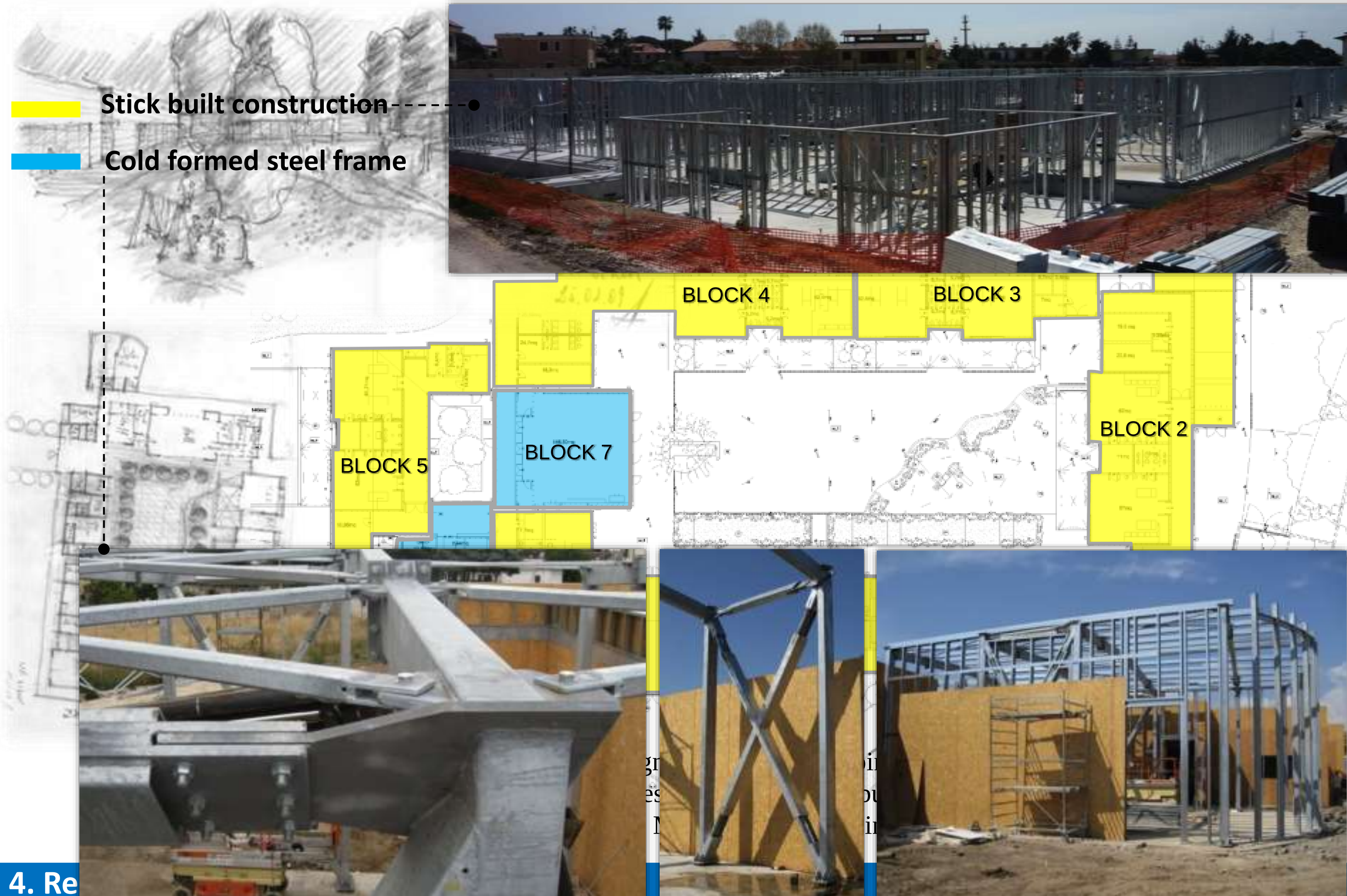
Structures:
Prof. Eng. Raffaele Landolfo
P.hD. Arch. Ornella Iuorio
P.hD. Eng. Luigi Fiorino

Technology:
Prof. Arch. Mario Losasso
Ass. Prof. Arch. S. Russo Ermolli
P.hD. Generosa Cacciapuoti
Arch. Antonio DAcunzi

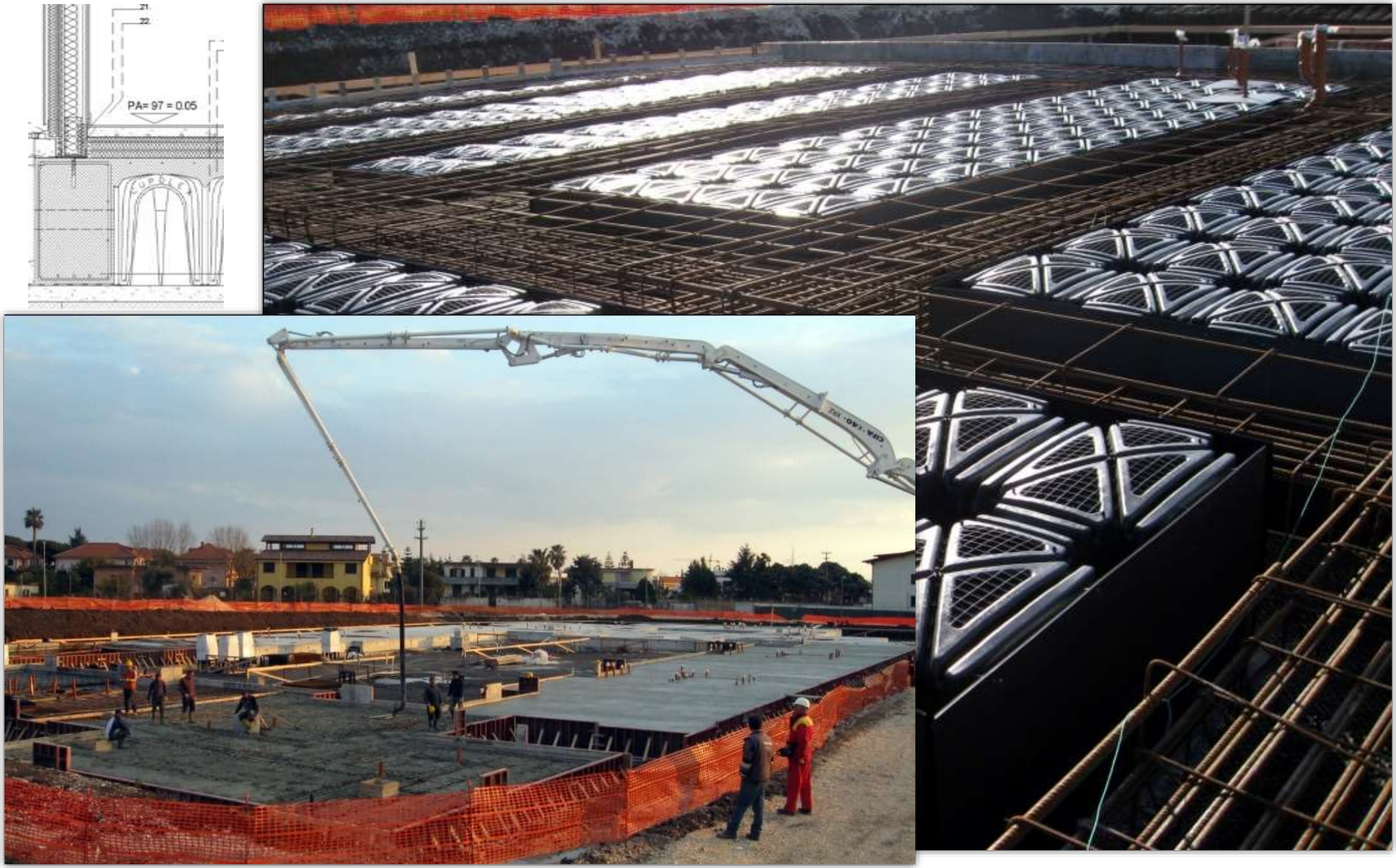
M&E:
Eng. Roberto Romano

Accounting:
Arch. Alfonso Mauro

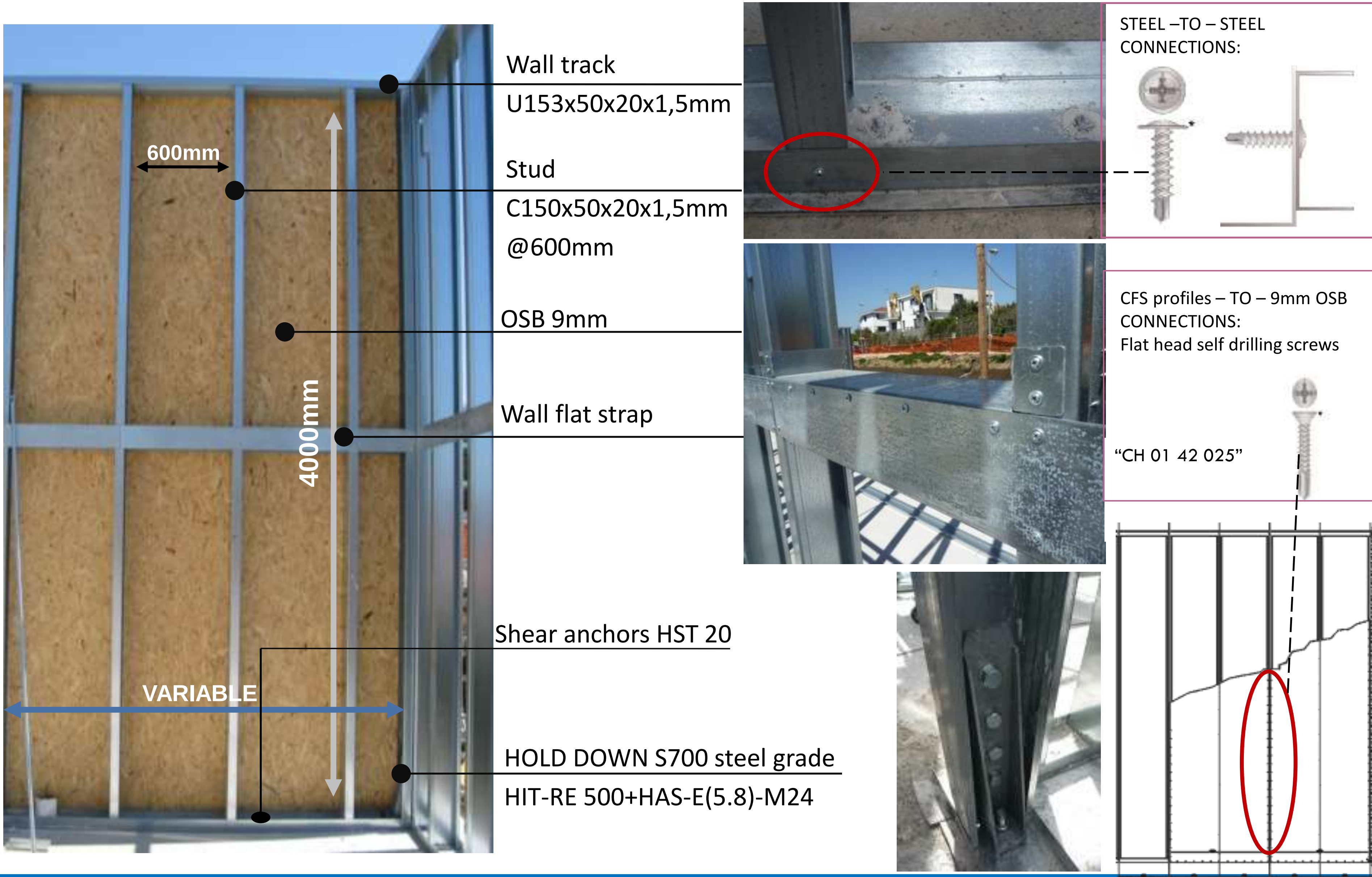
An emblematic application



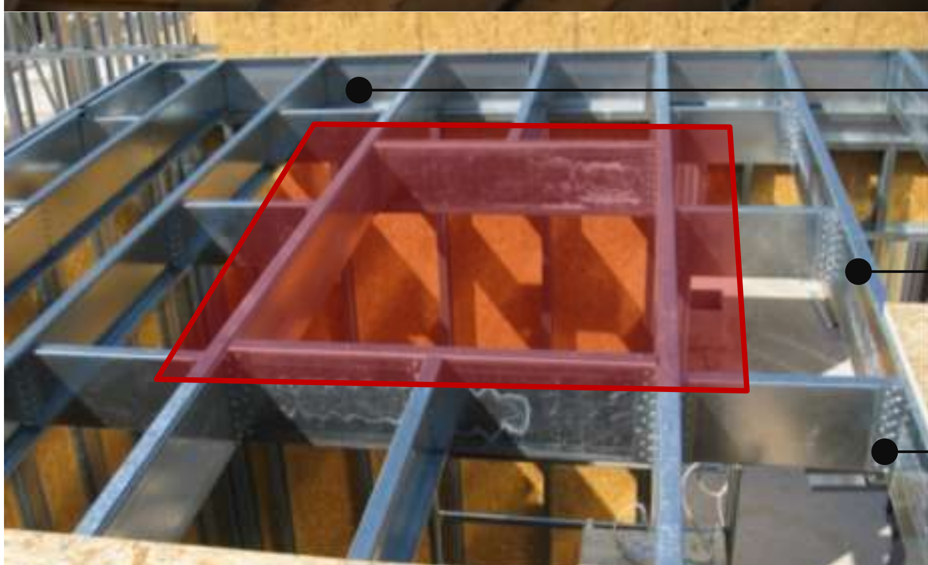
4. Re

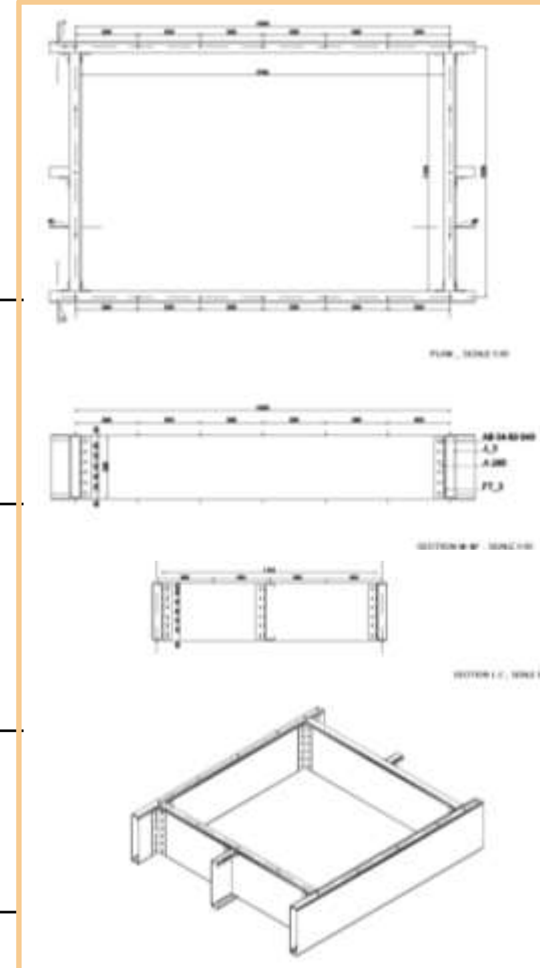

PA= 97 = 0.05

An emblematic application



4. Recent applications in Italy



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



An emblematic application

An emblematic application

An emblematic application

Total surface area:
15.850 m^2

Covered area by steel structures:
3.000 m^2

TOTAL TONNAGE:
140 tons of steel

Weight per square meter:
0.45 kN/m^2

OSB 9 mm (WALLS):
10000 m^2

OSB 18mm (FLOORS:
3000 m^2

An emblematic application

Reference codes for the design of structural systems

Design:

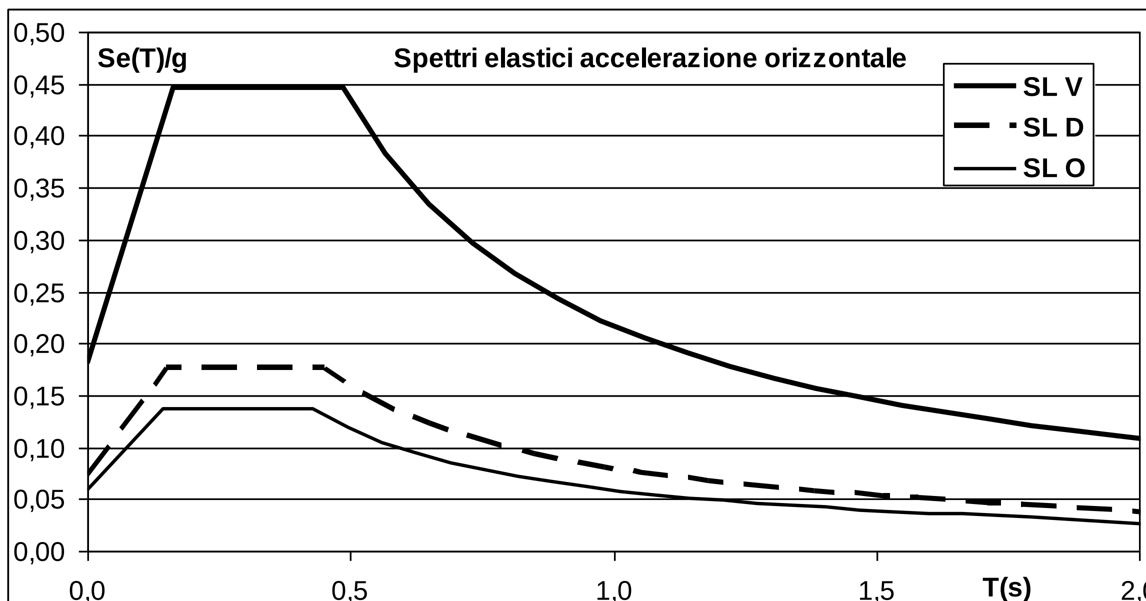
D.M. 2008, Norme Tecniche per le Costruzioni, Decreto Ministeriale 14/01/2008, Ministero delle infrastrutture, Roma, 2008.

Verifications:

CEN, EN 1993-1-1, Eurocode 3: Design of steel structures – Part 1-1: General rules and rules for buildings. *CEN, European Committee for Standardization*, Bruxelles, 2006.

CEN, EN 1993-1-3 - Eurocode 3 - Design of steel structures - Part 1-3: General rules - Supplementary rules for cold-formed members and sheeting, *CEN, European Committee for Standardization*, Bruxelles, 2006.

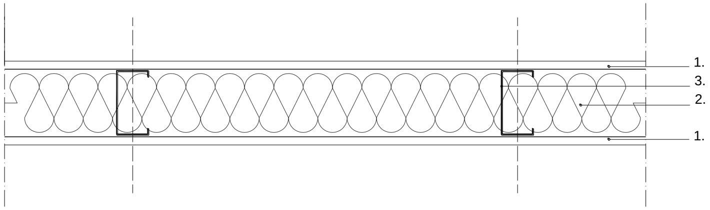
The seismic loads have been defined in agreement D.M. 14 (2008). The Spectra have been defined accounting a Reference Life equal to 75 years



Spectra for the different Limit States

Non-structural building components

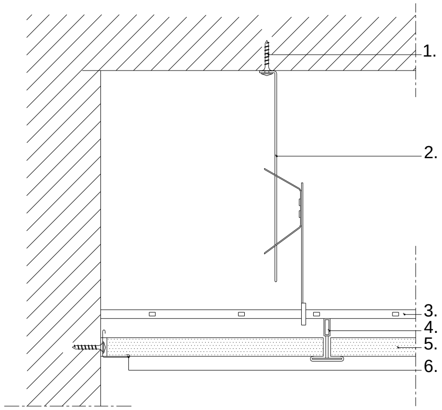
Interior partition wall



1. Fiber-cement board t=12,5mm (prod. Knauf Aquapanel indoor)
2. Isulation panel rock wool 60kg/m^3 t=100mm
3. Stud in CFS@600mm t=100mm

Total tickness= 130mm

Modular ceiling



1. Self-piercing screw
2. Suspender
3. Currying channel
4. Furring channel
5. Sheathing panel
6. Corner profile

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

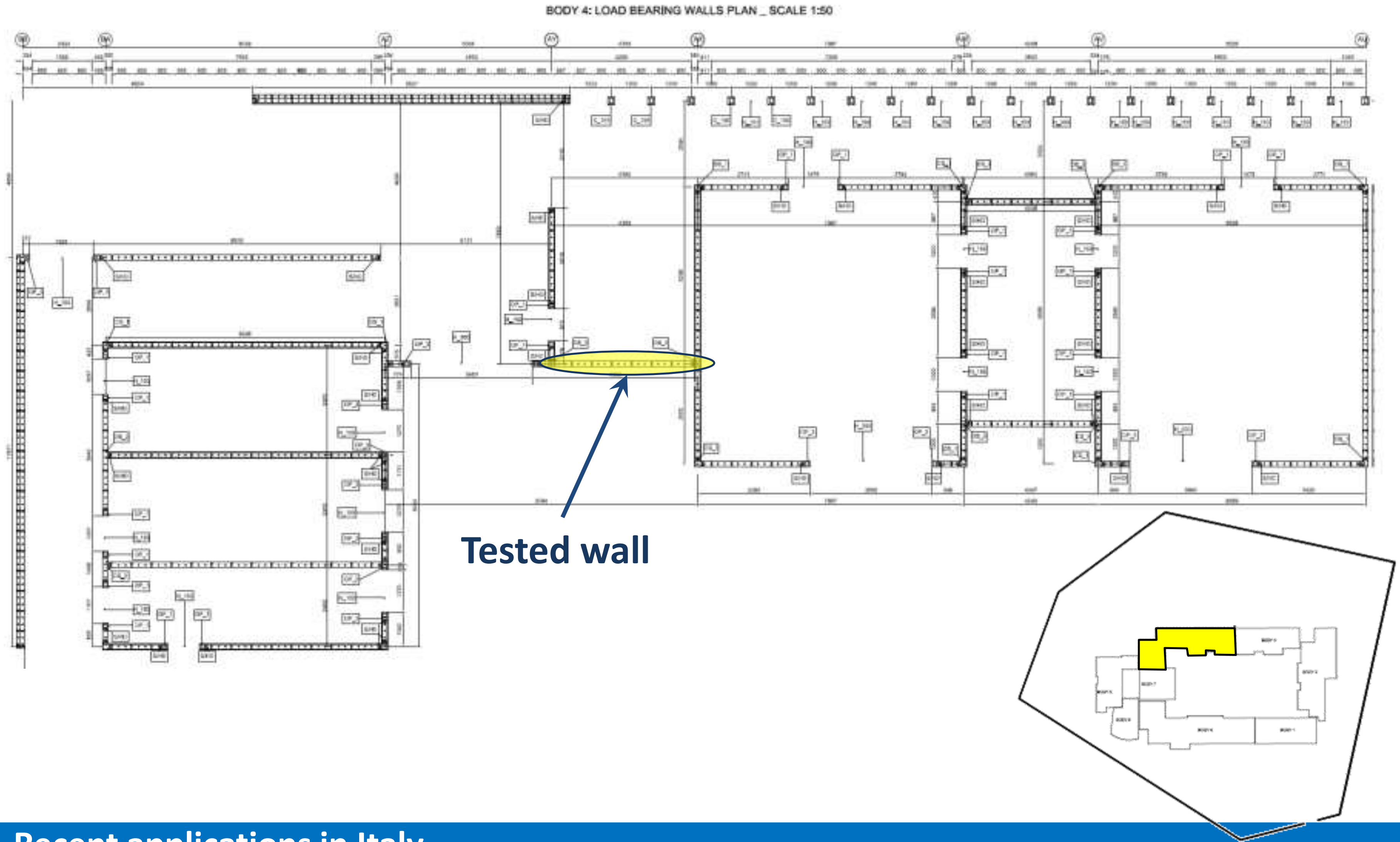
10 Hold-down device tests

Materials

20 OSB panel shear tests

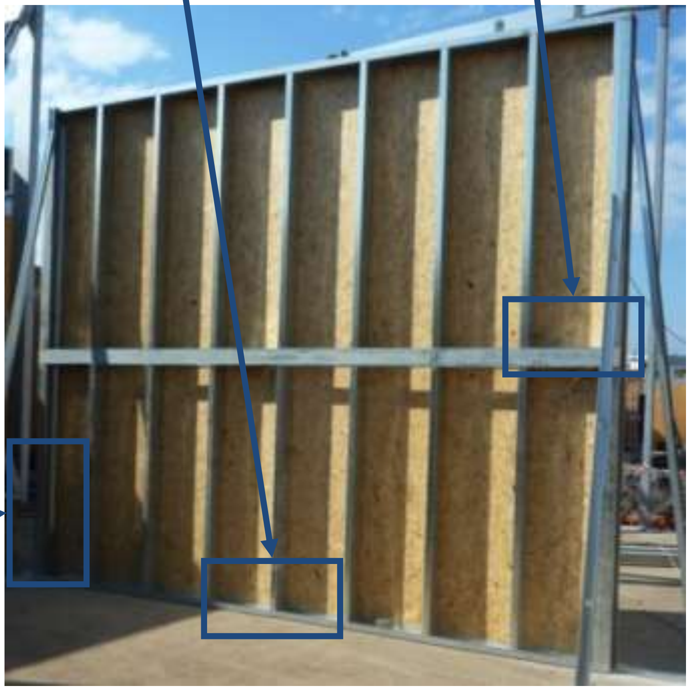
40 Self-drilling screw shear tests

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

Loads

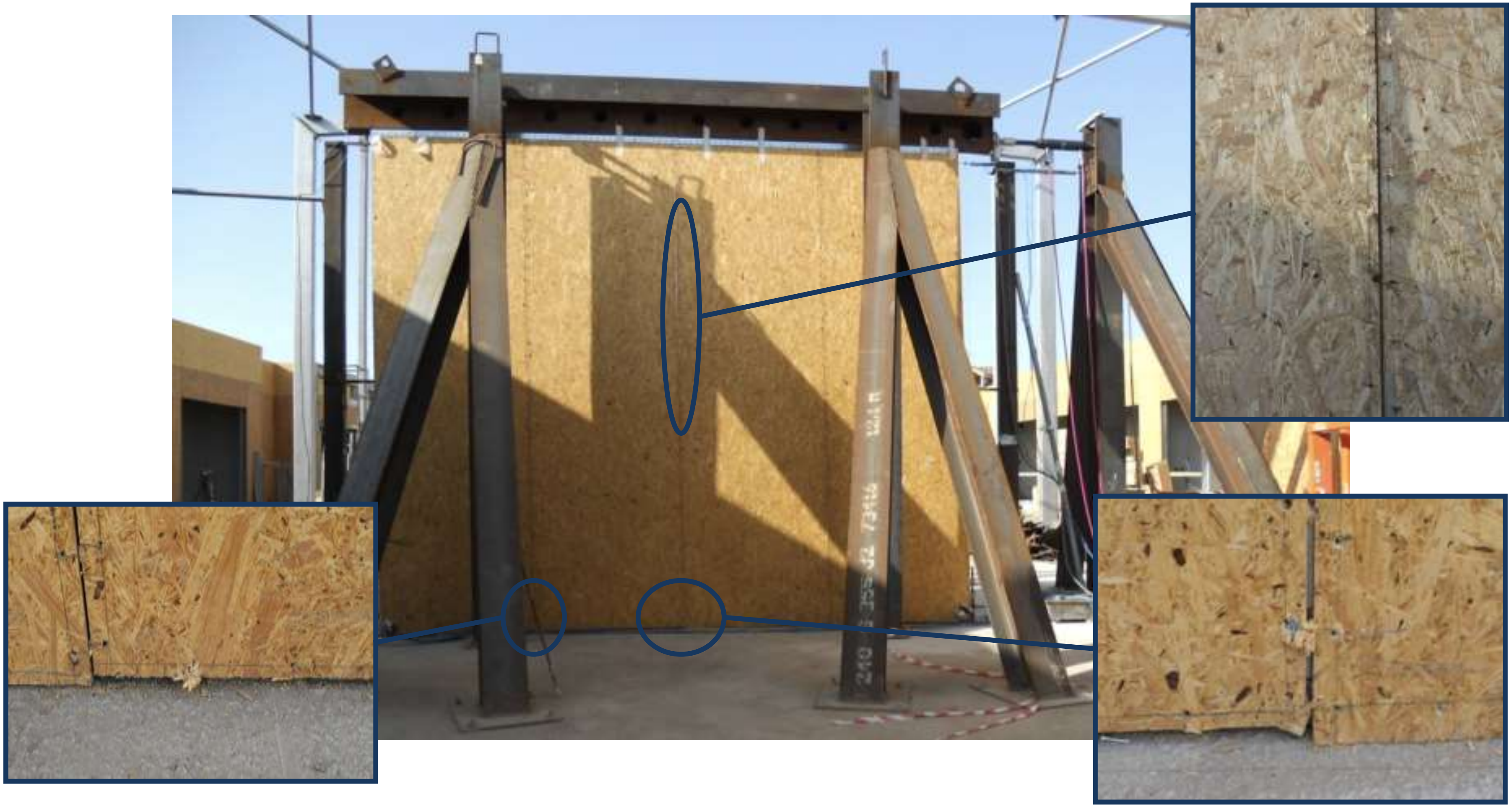
Horizontal loads:
Double effect jack COD25N260 by Europress (range of displacement: 260 mm; maximum force equal: 232 kN for pushing and 121 kN for pulling)

Vertical loads:
First test: 5.92 kN/m
Second test 10.20 kN/m
(Applied by concrete blocks)

Loading protocol:
First phase: cyclic up to 9 and 13 mm for first and second test, respectively
Second phase: monotonic up to collapse

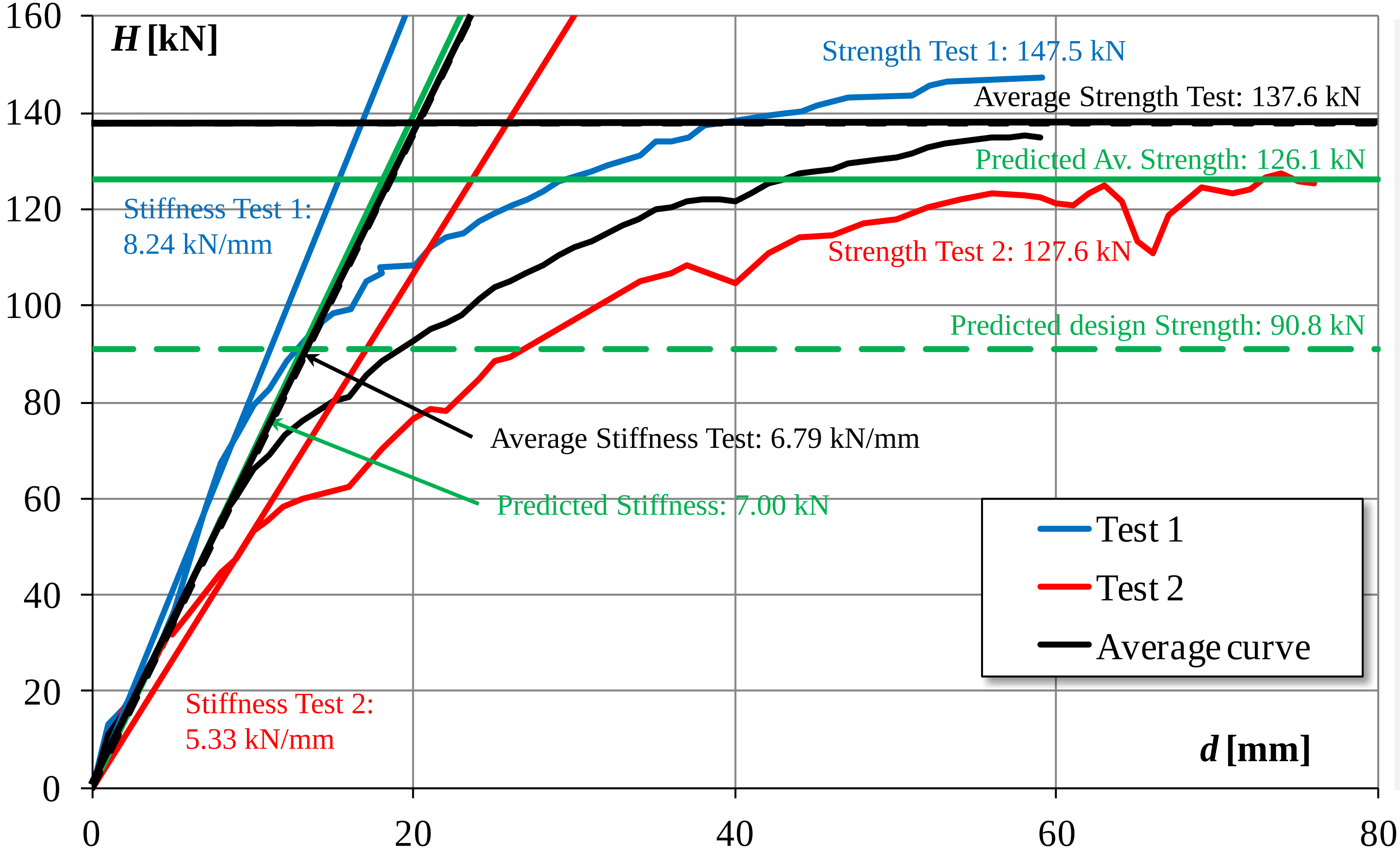
Experimental program: wall tests

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.

Fire test following earthquakes on a full-scale six-storeys building realized with sheathed CFS structure
(Input: 1994 Northridge Earthquake)

Fire test following earthquakes on a full-scale six-storeys building realized with sheathed CFS structure (with steel sheets adherent to gypsum-based panels)

- 6 storeys
- 10.4 m x 6.2 m (plan dimensions)
- Large High-Performance Outdoor Shake Table (LHPOST) available at the University of California, San Diego (UCSD)
- Input: 1994 Northridge Earthquake
- Station 150% Canoga Park
- **Max horizontal P.G.A. = 0.91g**
- **Max acceleration amplification= 4.2 (3.8g)**
- **Max drift = 1.70%**

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3. Researches at the University of Naples Federico II

4. Recent applications in Italy

5. Concluding Remarks

1 The Lightweight Steel-Framed Constructions are **very competitive** in the contemporary markets for the lightness, high structural efficiency, high structural performance, dry realization, recyclable nature

2 High levels of **prefabrication, safety, durability and sustainability** are spreading these construction systems all over the world.

3 Furthermore, **high seismic performance** even in case of strong ground motions characterize these systems.

4 The current trend of the construction market leads toward **integrated solutions** that must satisfy multiple requirements in terms of eco-efficiency, structural performance, without neglecting the economic aspects.

5 The actual lack in specific design codes, mainly for the applications in seismic area, requires the **development of new research in the field**.

6 The **research and applications developed at the University of Naples Federico II** demonstrated the competiveness of Lightweight Steel-Framed Constructions in seismic zone.

Next developments

Ongoing revision process of European Standard

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ANNEX Z (informative). Examples of adequate connection design for steel and composite

ANNEX Y (normative). Data for pushover analysis for steel and composite structures

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Seismic behavior of lightweight structures in steel

Prof. Eng. ***Raffaele Landolfo***

Thanks for your kind attention