Seminar

Institut Universitaire de Technologie (IUT) de Cergy-Pontoise
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A SIMPLIFIED PROCEDURE FOR THE SEISMIC DESIGN OF STRUCTURES EQUIPPED WITH FLUID-VISCOUS DAMPERS

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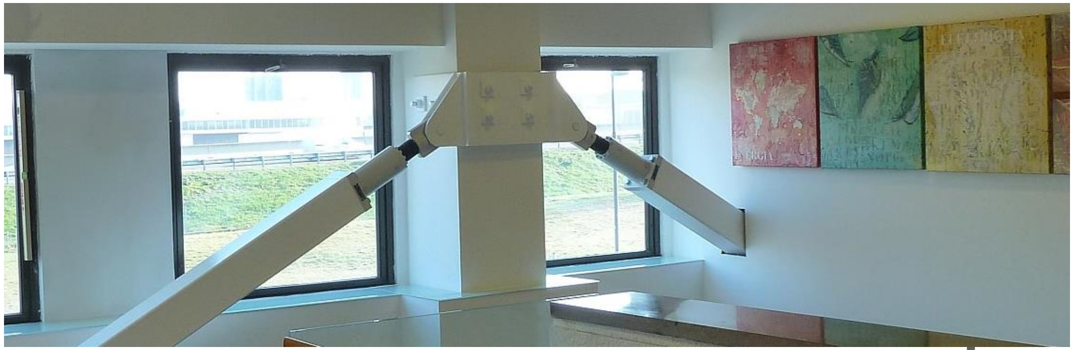


Presentation outline

- Background
- Theoretical basis
- The "five-step procedure" (2010)
- The "direct five-step procedure" (2016-2018)
- Applicative example
- Conclusions and future developments

MPD vs SPD systems

OBJECTIVES

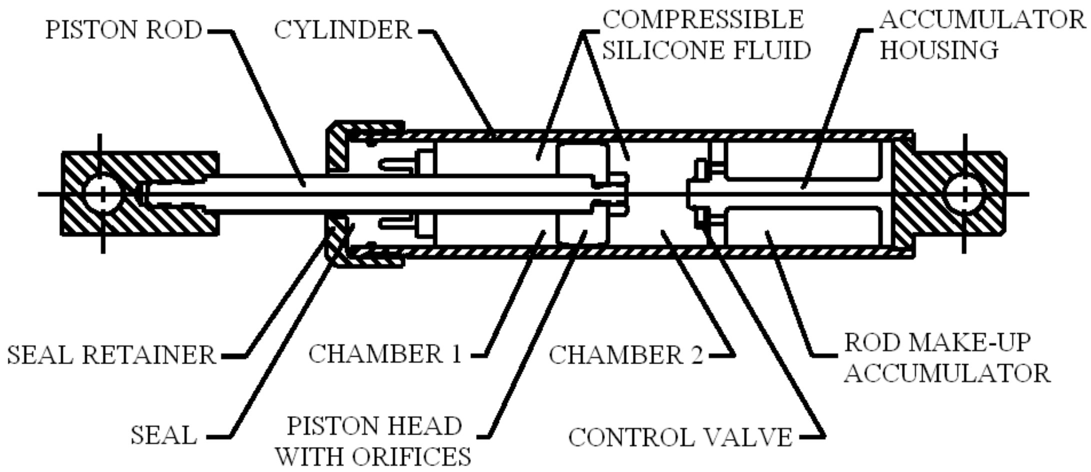
To develop an easy method for preliminary quick design of structures equipped with fluid-viscous dampers (for the wide diffusion of their use)

To give fully-analytical tools to the professional engineers for the control of the results of non-linear TH analyses

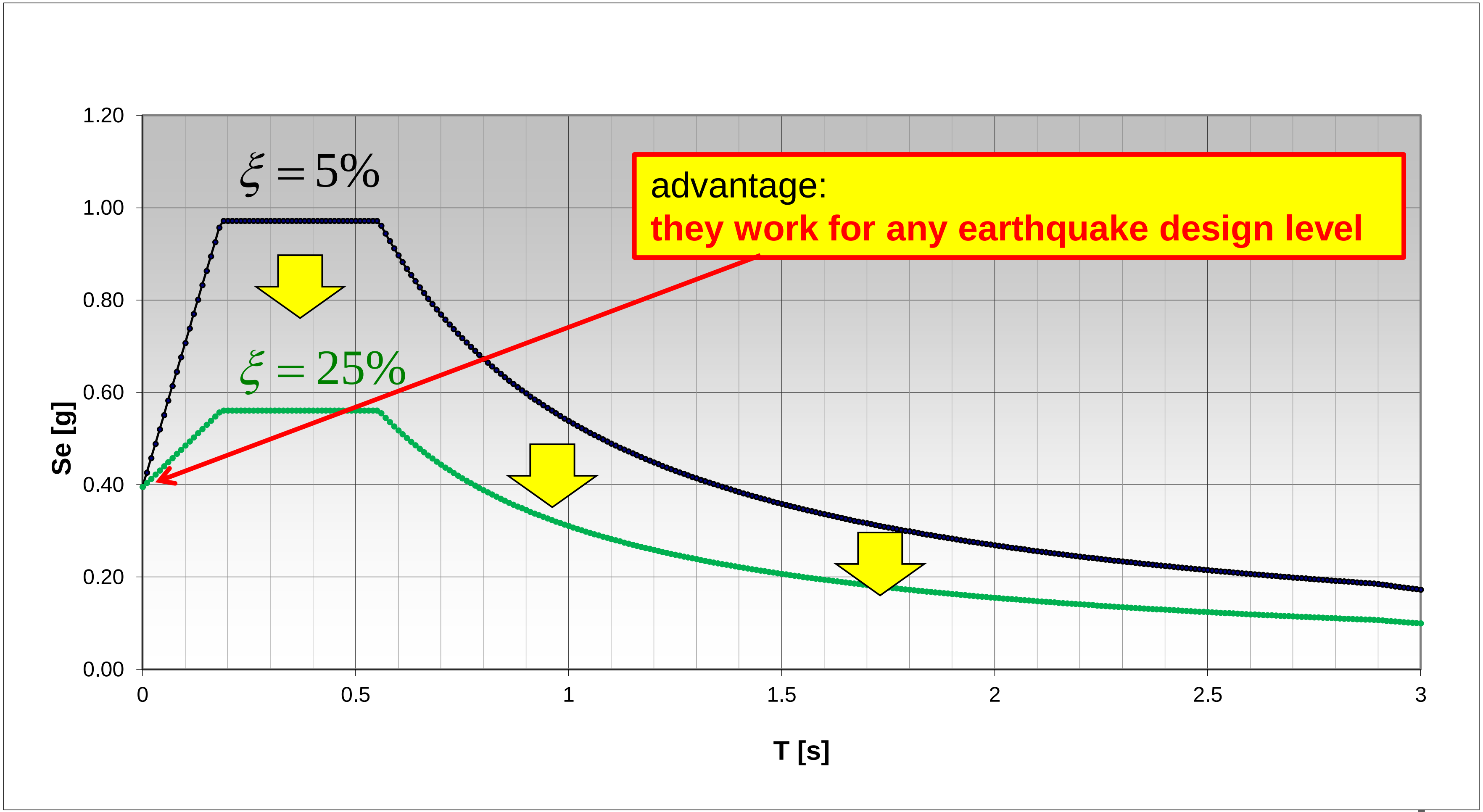
Background

Fluid-Viscous Dampers

energy is dissipated in the guise of **heat** through the passage of a **viscous silicone fluid** (stable w.r.t. temperature) across the piston head with orifices

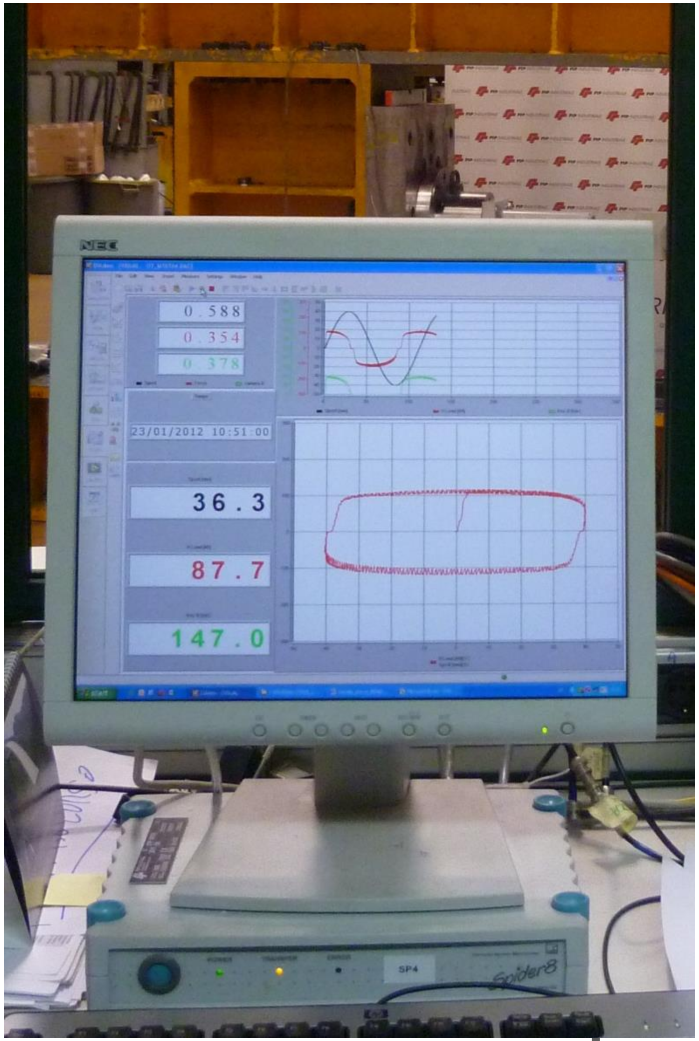


Benefits provided by viscous dampers



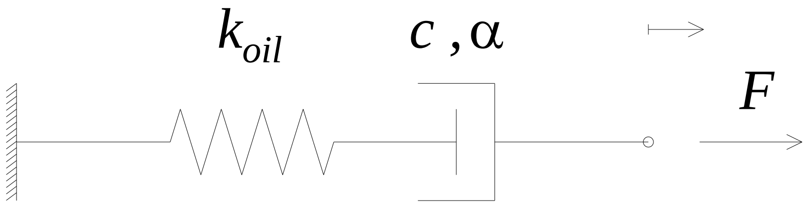
Viscous dampers

constant velocity tests



Viscous dampers

Maxwell model:

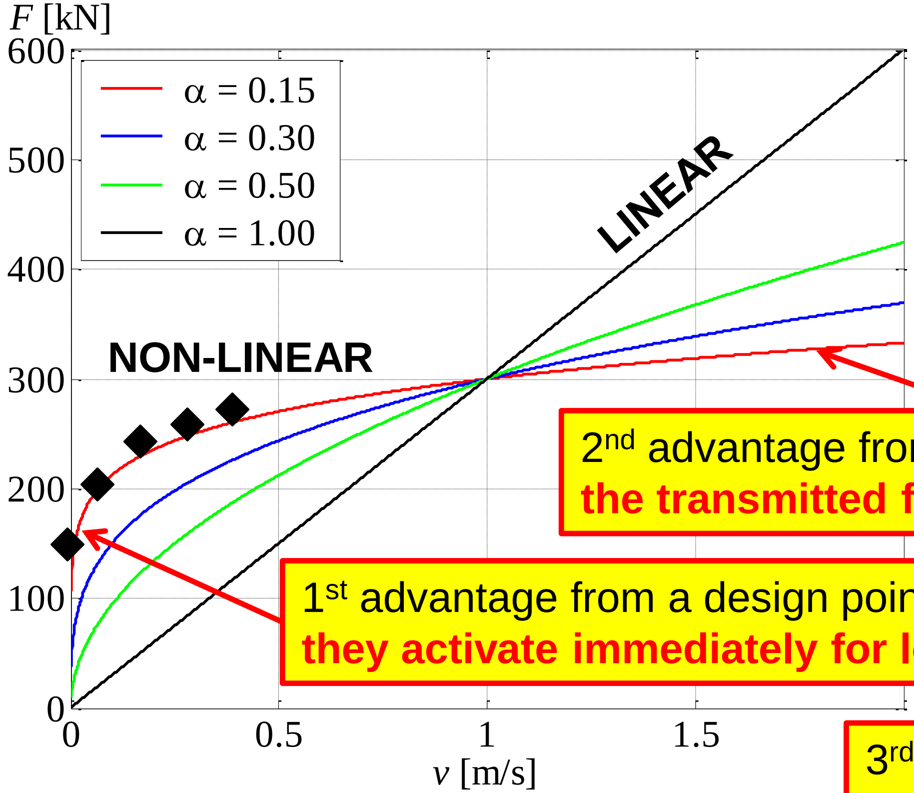


$$F = k_{oil} \cdot x = c \cdot v^{\alpha}$$

$\alpha \cong 0.15$

$$k_{oil} = \frac{F_{max}}{5\%\ x_{max}}$$

half piston stroke



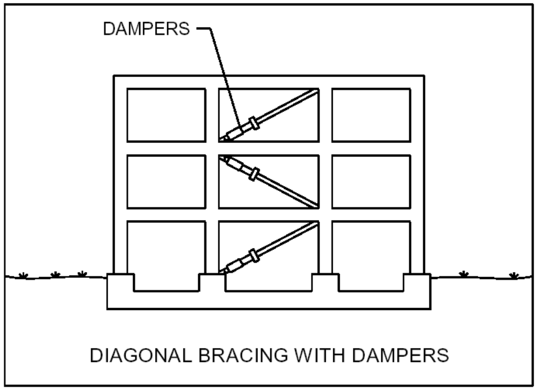
2nd advantage from a design point-of-view:
the transmitted force is limited!

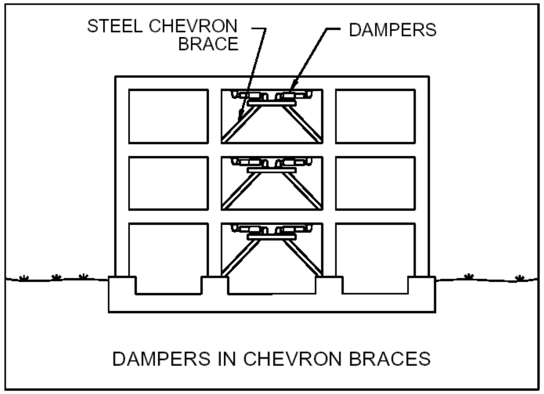
1st advantage from a design point-of-view:
they activate immediately for low velocities

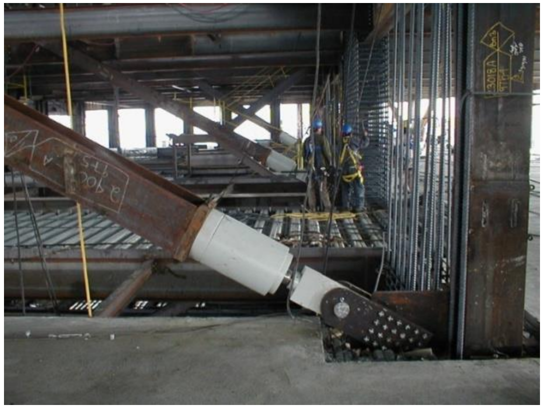
3rd advantage from a design point-of-view:
they don't reduce the period of vibration

Effectiveness

- The **effectiveness** of fluid viscous dissipative devices in reducing the seismic demand on the structural elements has been demonstrated by a number of research works and real applications since the 1980s.

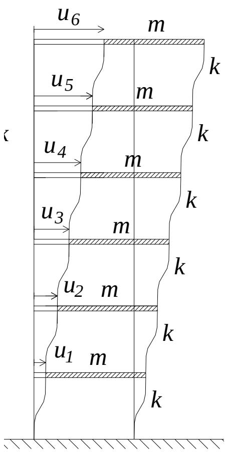




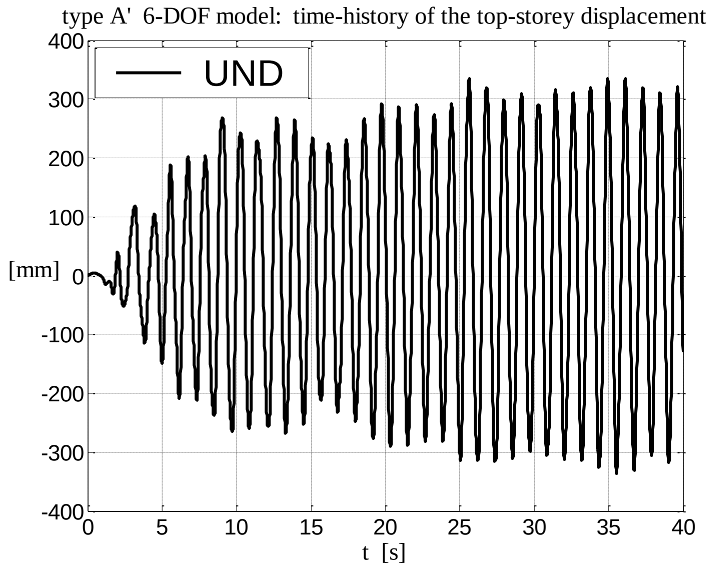


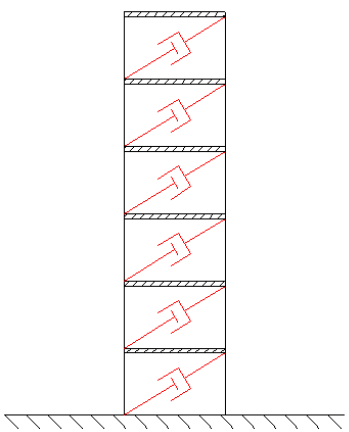
Seismic Dampers installed at the Hotel Woodland, in Woodland, California
Force = 100,000 lbs., Stroke = +/- 2 inches
Production = 16 pieces

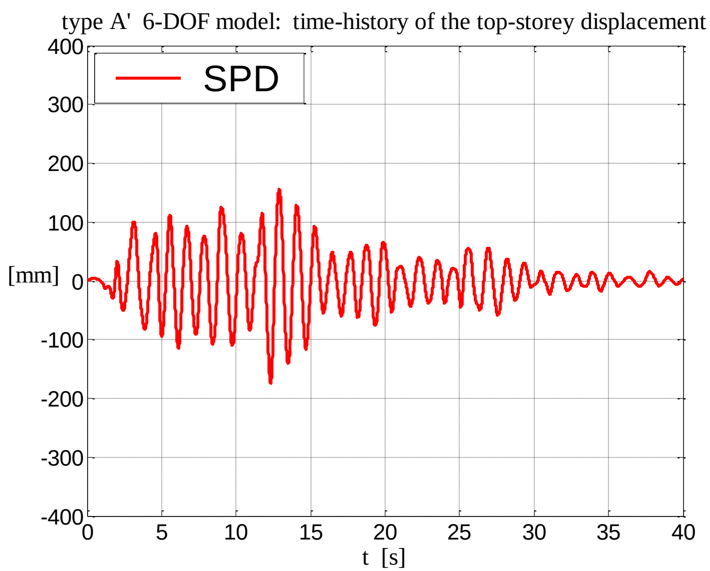
Effectiveness



$T_1 = 1.17$s







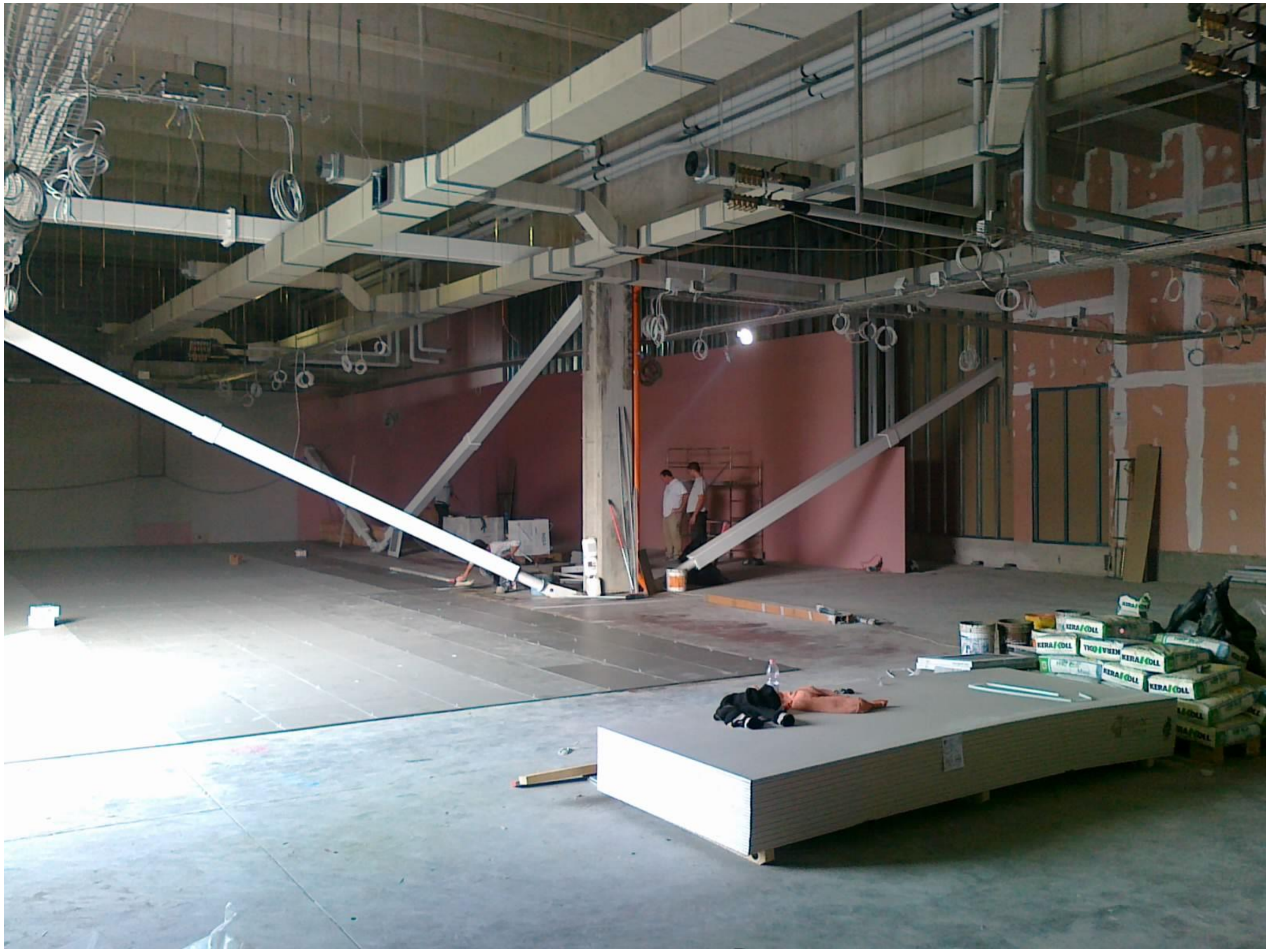
top-storey displacement response

El Centro 1940

scaled to

PGA = 0.3*g*

Real application



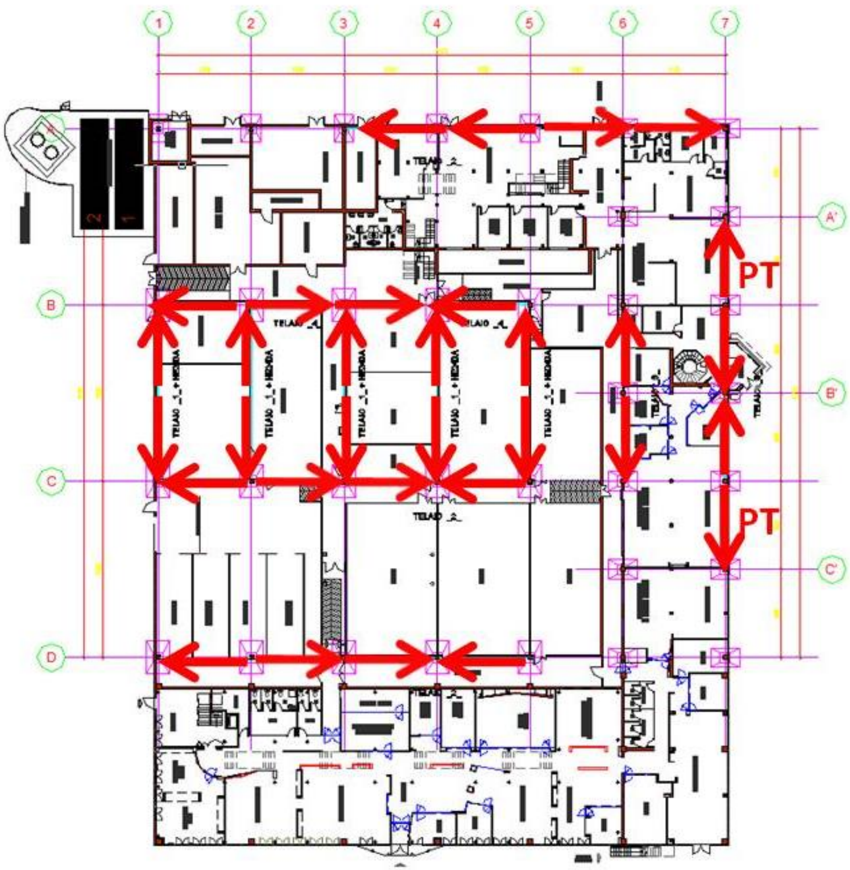
HERA building, Imola (BO)

dampers manufactured by FIP, Padova, Italy

2010-2011

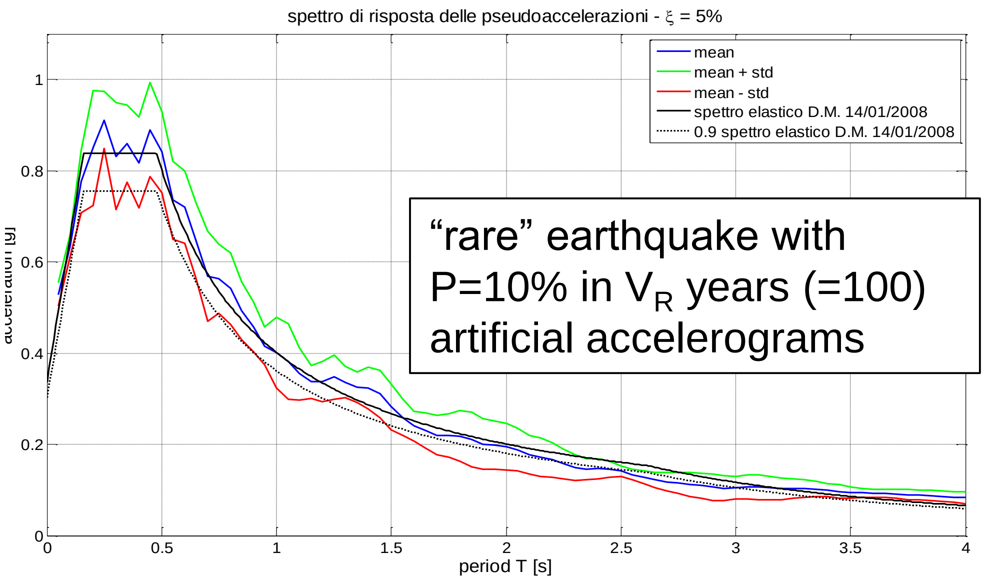
pictures courtesy of Ing. Franco Baroni, Studio Ceccoli & Associati, Bologna

Real application



HERA building, Imola (BO)

n. 32 dampers manufactured by FIP, Padova, Italy



"rare" earthquake with $P=10\%$ in V_R years (=100) artificial accelerograms

Real application

HERA building, Imola (BO)

dampers manufactured by FIP, Padova, Italy

2010-2011

pictures courtesy of Ing. Franco Baroni, Studio Ceccoli & Associati, Bologna

Real application

HERA building, Imola (BO)

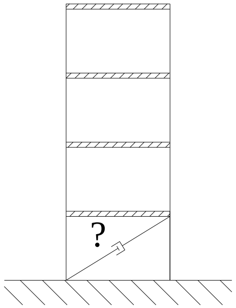
dampers manufactured by FIP, Padova, Italy

2010-2011

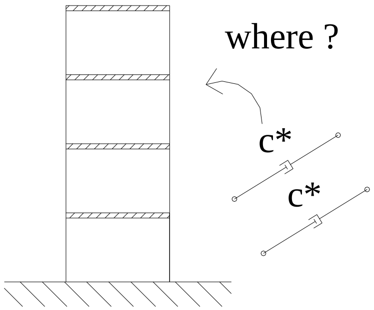
pictures courtesy of Ing. Franco Baroni, Studio Ceccoli & Associati, Bologna

In the scientific literature:

- **Sophisticated numerical algorithms for dampers optimization** (Takewaki 1997, 2000 and 2009, Shukla and Datta 1999, Lopez Garcia 2001, Singh and Moreschi 2002, Levy and Lavan 2006, ...)



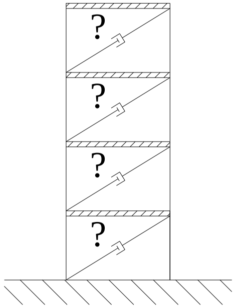
- **Computational expertise and time** (beyond the typical availabilities of the designers) are needed



- **Numerical results** which do not provide physical insight into the matter.

- The issue of developing **simple/analytical methods** in order to size and locate the viscous dampers is still open.

(like equivalent static analysis vs. non-linear time-history analysis)

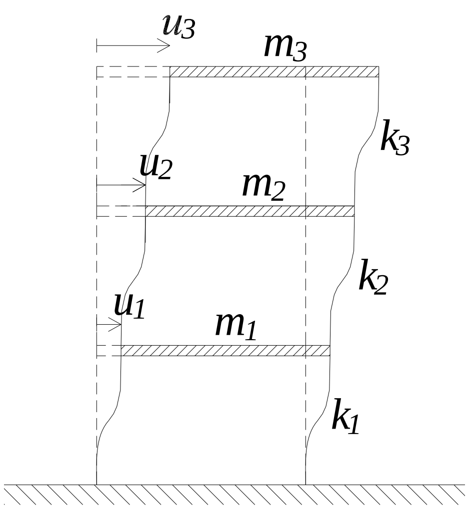


Design procedures

- **report NCEER-92-0032 (Constantinou and Symans 1992)**
- **report MCEER-00-0010 (Ramirez et al. 2000)**
- **ASCE 7 (2005) Chapter 18**, which is grounded on the MCEER-00-0010 approach and on the works by Ramirez et al. (2002a and b, and 2003) and by Whittaker et al. (2003), contains systematic procedures for design and analysis of building with damping systems (***use of the residual mode approach***).
- **Lopez-Garcia 2001** developed a simple **algorithm** for optimal damper configuration (placement and properties) in MDOF structures, assuming a constant inter-storey height and a straight-line first modal shape.
- **Christopoulos and Filiatrault (2006)** suggested a design approach for estimating the damping coefficients of added viscous dampers consisting in a ***trial and error procedure*****.**
- **Silvestri et al. 2010** ⇨ **"five-step procedure"**

Theoretical basis

Insight into the Rayleigh damping



$$\underset{\sim}{\mathbf{m}}\ddot{\underline{\mathbf{u}}} + \underset{\sim}{\mathbf{c}}\dot{\underline{\mathbf{u}}} + \underset{\sim}{\mathbf{k}}\underline{\mathbf{u}} = -\underset{\sim}{\mathbf{m}}\underline{\mathbf{1}} \cdot \ddot{u}_g(t)$$

$\underline{\mathbf{u}}$ physical coordinates vector

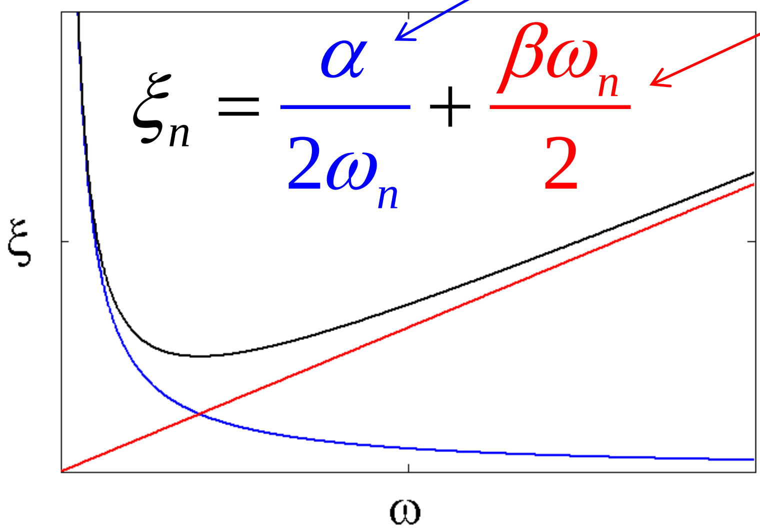
$\underset{\sim}{\mathbf{m}}$ mass matrix

$\underset{\sim}{\mathbf{k}}$ stiffness matrix

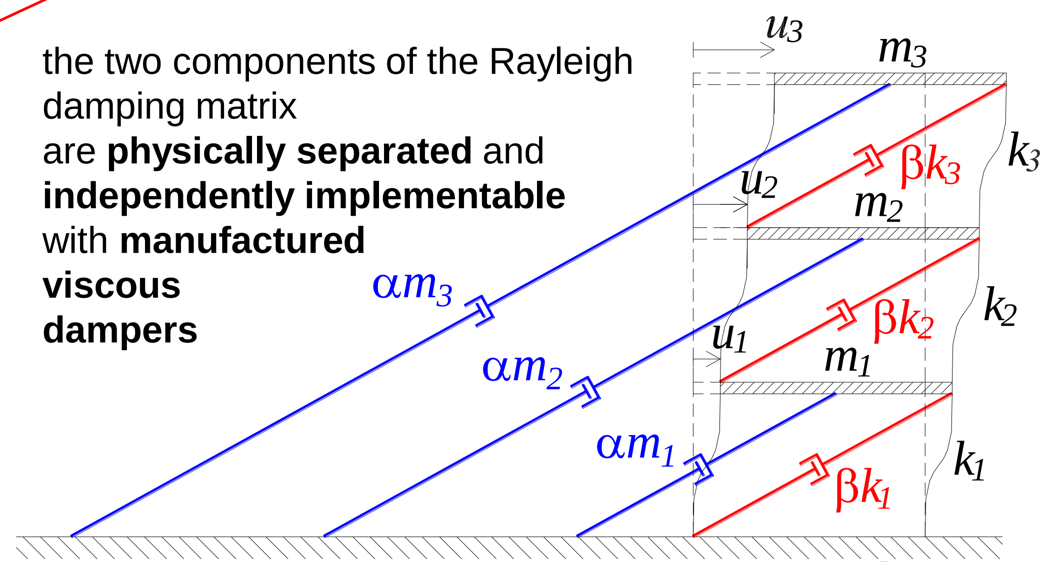
$$\underset{\sim}{\mathbf{c}} = \alpha\underset{\sim}{\mathbf{m}} + \beta\underset{\sim}{\mathbf{k}}$$

$$\underset{\sim}{\mathbf{c}} = \alpha\begin{bmatrix} m_1 & 0 & 0 \\ 0 & m_2 & 0 \\ 0 & 0 & m_3 \end{bmatrix} + \beta\begin{bmatrix} k_1+k_2 & -k_2 & 0 \\ -k_2 & k_2+k_3 & -k_3 \\ 0 & -k_3 & k_3 \end{bmatrix}$$

$$\xi_n = \frac{\alpha}{2\omega_n} + \frac{\beta\omega_n}{2}$$



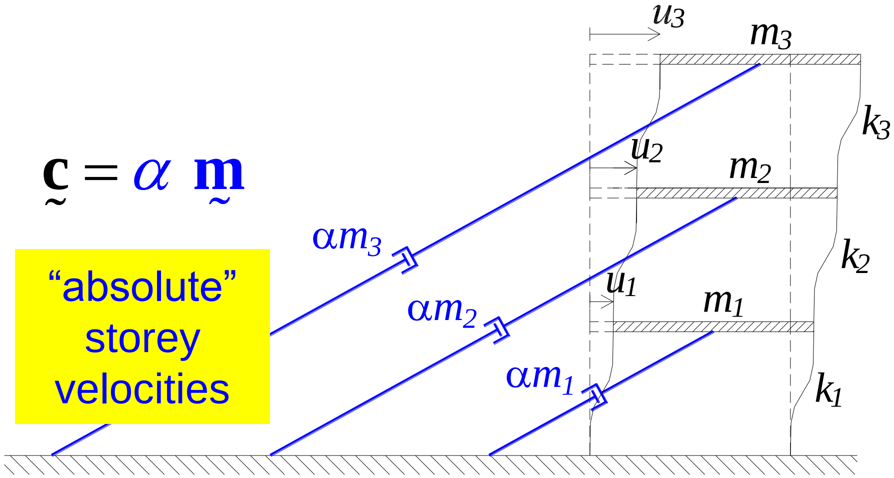
the two components of the Rayleigh damping matrix are **physically separated** and **independently implementable** with **manufactured viscous dampers**



MPD and SPD systems

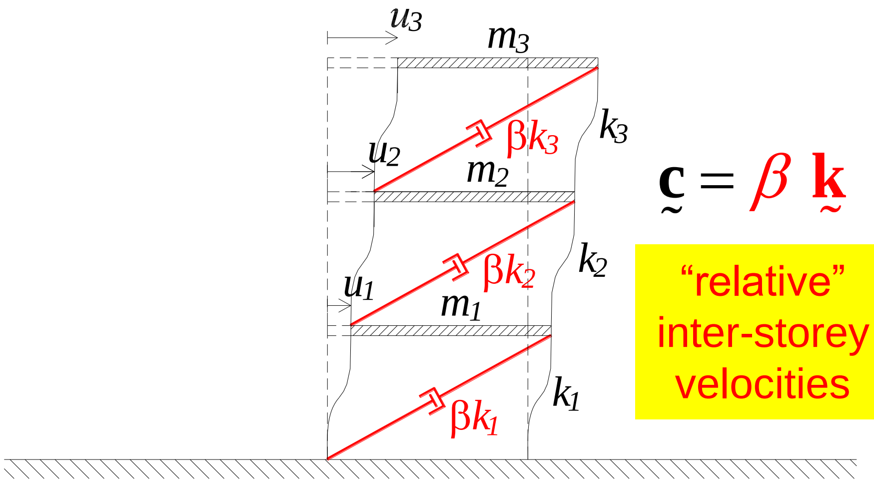
Mass Proportional Damping system

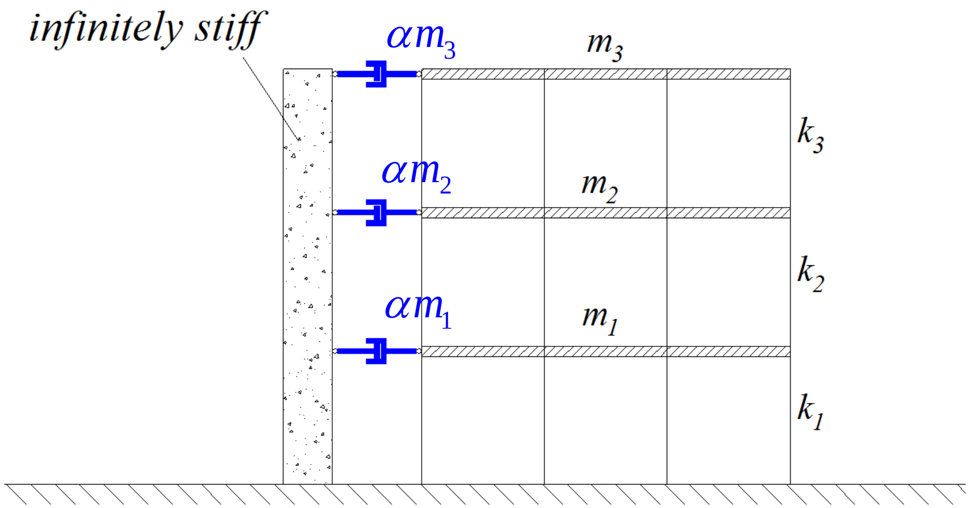
$$\underset{\sim}{\mathbf{c}} = \alpha \; \underset{\sim}{\mathbf{m}}$$



Stiffness Proportional Damping system

$$\underset{\sim}{\mathbf{c}} = \beta \; \underset{\sim}{\mathbf{k}}$$

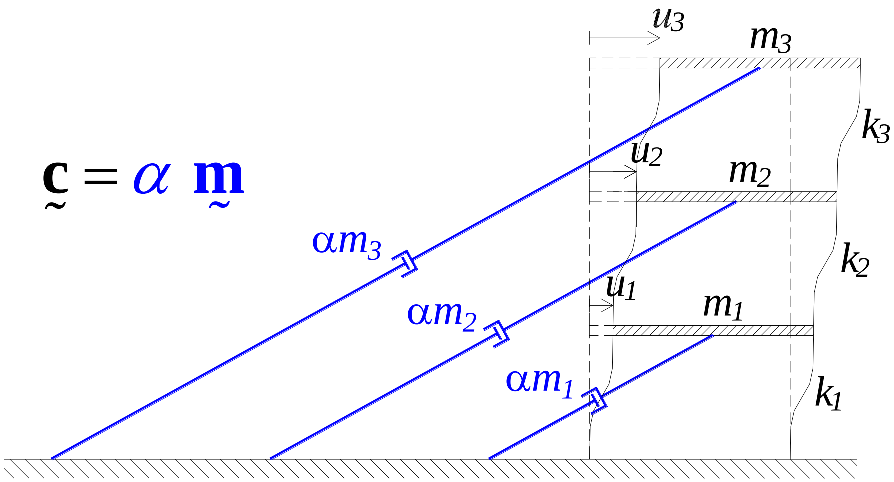


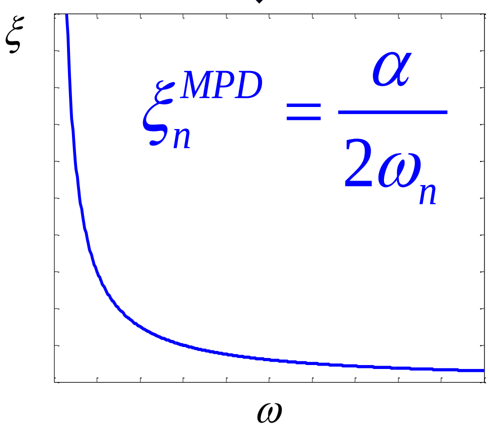


Properties of MPD and SPD systems (1)

Mass Proportional Damping system

$$\underset{\sim}{c} = \alpha \; \underset{\sim}{m}$$

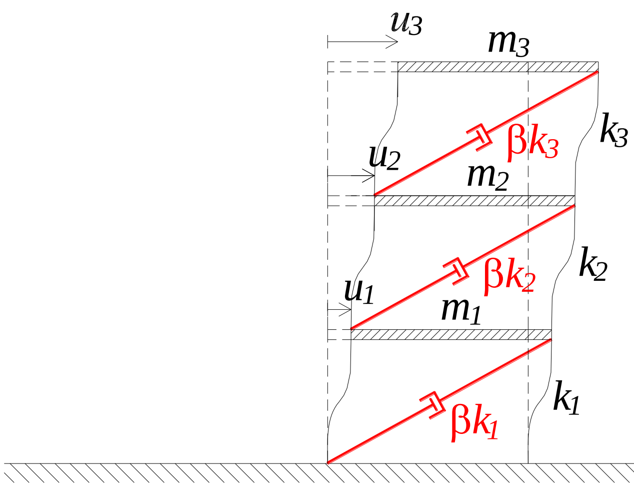


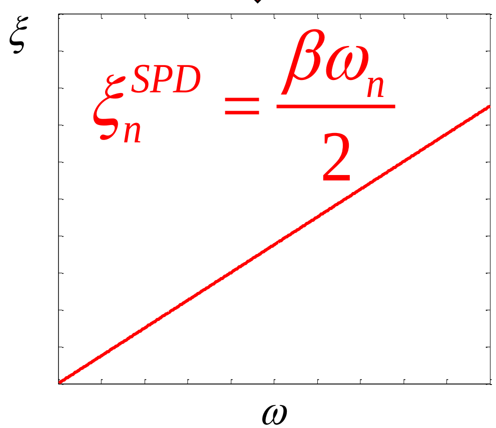


$$\xi_n^{MPD} = \frac{\alpha}{2\omega_n}$$

Stiffness Proportional Damping system

$$\underset{\sim}{c} = \beta \; \underset{\sim}{k}$$





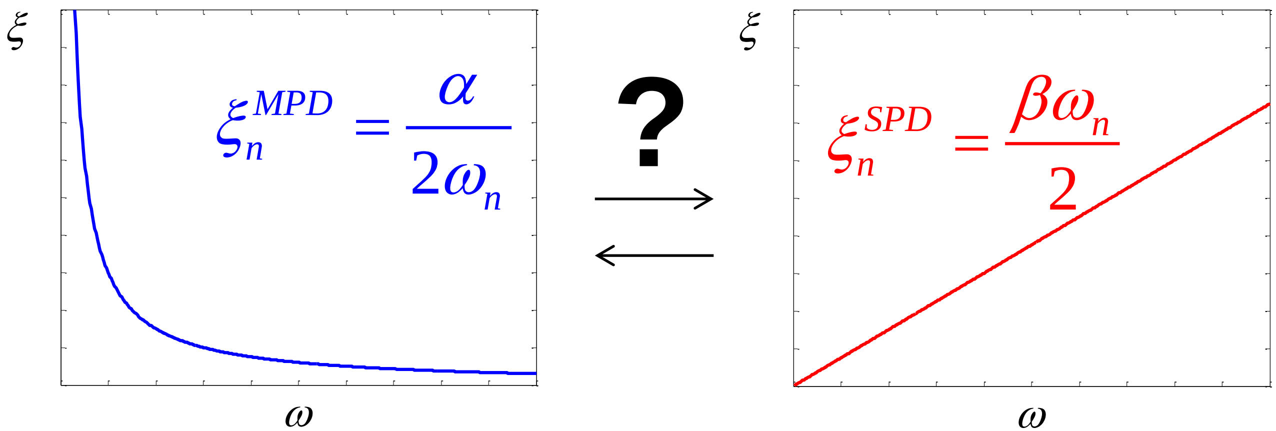
$$\xi_n^{SPD} = \frac{\beta\omega_n}{2}$$

Properties of MPD and SPD systems (2)

HP:

The two systems have equal total damping:

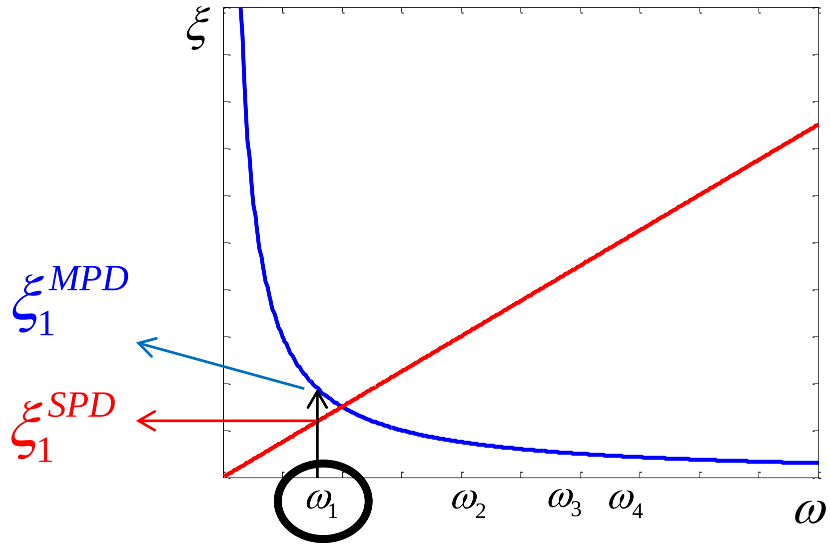
$$c_{tot} = \sum c_i$$



QUESTION:

How the **first** modal damping ratios of the SPD system and the “corresponding” MPD system are related to each other?

$$\xi_1^{MPD} \;\overset{?}{\geq}\; \leq \;\xi_1^{SPD}$$

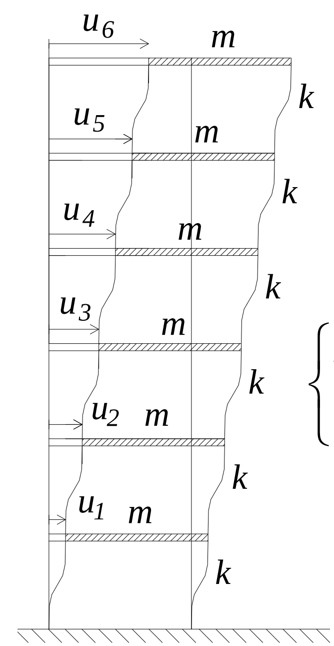


Properties of MPD and SPD systems (3)

From basic dynamics:

$$\frac{\xi_i^{SPD}}{\xi_i^{MPD}} = \frac{\frac{\beta}{2}\omega_i}{\frac{\alpha}{2\omega_i}} = \frac{\beta}{\alpha}\omega_i^2 = \dots$$

HP1) structures characterised by constant values of lateral stiffness *k* and storey mass *m*



$$\begin{cases} k = \text{cost} \\ m = \text{cost} \end{cases}$$

HP2) equal "total cost" constraint

$$\left.\begin{aligned} \bar{\alpha} &= \frac{\bar{c}}{\sum_{i=1}^{N} m_i} = \frac{\bar{c}}{Nm} \\ \bar{\beta} &= \frac{\bar{c}}{\sum_{i=1}^{N} k_i} = \frac{\bar{c}}{Nk} \end{aligned}\right\} \quad \bar{\alpha} m = \bar{\beta} k \quad \Rightarrow \quad \frac{\bar{\beta}}{\bar{\alpha}} = \frac{m}{k} = \frac{1}{\omega_0^2} \qquad \left(\omega_0 = \sqrt{\frac{k}{m}}\right)$$

$$\frac{\xi_i^{SPD}}{\xi_i^{MPD}} = \frac{\frac{\bar{\beta}}{2}\omega_i}{\frac{\bar{\alpha}}{2\omega_i}} = \frac{\bar{\beta}}{\bar{\alpha}}\omega_i^2 = \frac{\omega_i^2}{\omega_0^2} = \dots$$

$$\frac{\xi_i^{SPD}}{\xi_i^{MPD}} = \frac{\frac{\bar{\beta}}{2}\omega_i}{\frac{\bar{\alpha}}{2\omega_i}} = \frac{\bar{\beta}}{\bar{\alpha}}\omega_i^2 = \frac{\omega_i^2}{\omega_0^2} = \Omega_i^2 = \Lambda_i$$

Properties of MPD and SPD systems (4)

$$\frac{\xi_i^{SPD}}{\xi_i^{MPD}} = \frac{\dfrac{\bar{\beta}}{2}\omega_i}{\dfrac{\bar{\alpha}}{2\omega_i}} = \frac{\bar{\beta}}{\bar{\alpha}}\omega_i^2 = \frac{\omega_i^2}{\omega_0^2} = \Omega_i^2 = \Lambda_i$$

→ eigenproblem

The matrix $\underset{\sim}{\mathbf{A}}_N = \dfrac{1}{\omega_0^2}\underset{\sim}{\mathbf{k}}_N \underset{\sim}{\mathbf{m}}_N^{-1} = \begin{bmatrix} 2 & -1 & 0 & \ldots & & & 0 \\ -1 & 2 & -1 & 0 & \ldots & & 0 \\ 0 & -1 & 2 & \ldots & \ldots & & \ldots \\ \ldots & 0 & \ldots & \ldots & & & \ldots \\ & & \ldots & & \ldots & -1 & 0 \\ & & & & -1 & 2 & -1 \\ 0 & \ldots & & \ldots & 0 & -1 & 1 \end{bmatrix}$ has eigenvalues Λ_i

if $\Lambda_i < 1 \Rightarrow \xi_i^{SPD} < \xi_i^{MPD}$

→ The *i*-th damping ratio of the SPD system is less then the "corresponding" one of the MPD system (the two systems with same total damping)

Properties of MPD and SPD systems (5)

- It can be demonstrated:

$$\frac{\xi_1^{SPD}}{\xi_1^{MPD}} = \Lambda_1 < 1$$

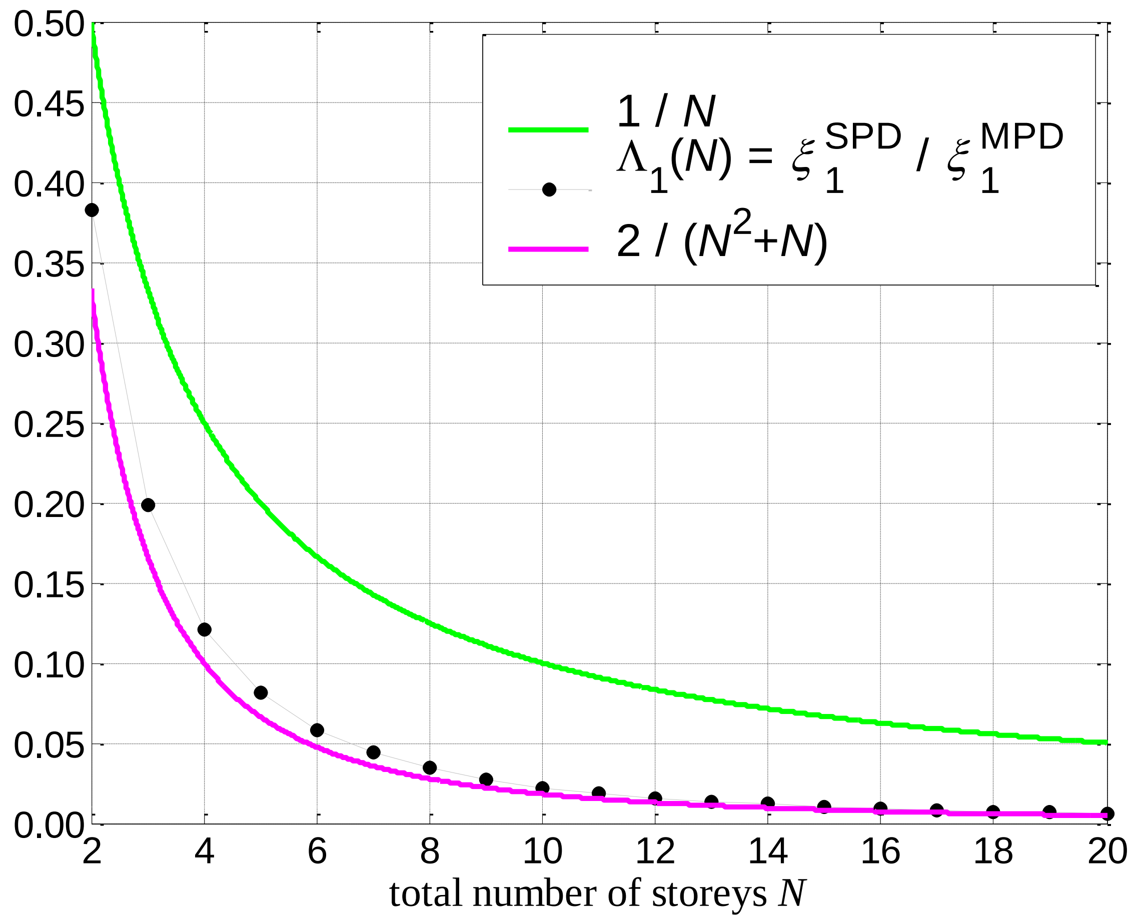
- Upper bound:

$$\frac{\xi_1^{SPD}}{\xi_1^{MPD}} < \frac{1}{N}$$

- Approximation:

$$\frac{\xi_1^{SPD}}{\xi_1^{MPD}} \cong \frac{2}{N(N+1)}$$

*



Trombetti & Silvestri 2006 (JSV)

Properties of MPD and SPD systems (6)

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On the modal damping ratios of shear-type structures equipped with Rayleigh damping systems

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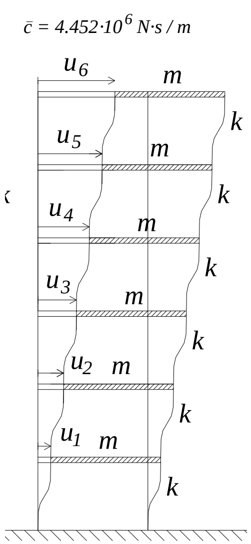
Abstract

The effects of added manufactured viscous dampers upon shear-type structures are analytically investigated here for the class of Rayleigh damping systems. The definitions of mass proportional damping (MPD) and stiffness proportional damping (SPD) systems are briefly recalled and their physical counterpart is derived. From basic physics, a detailed mathematical demonstration that the first modal damping ratio of a structure equipped with the MPD system is always larger than the first modal damping ratio of a structure equipped with the SPD system is provided here. All results are derived for the class of structures characterised by constant values of lateral stiffness and storey mass, under the equal "total size" constraint. The paper also provides closed form demonstrations of other properties of modal damping

Seismic response of MPD and SPD systems

6-DOF

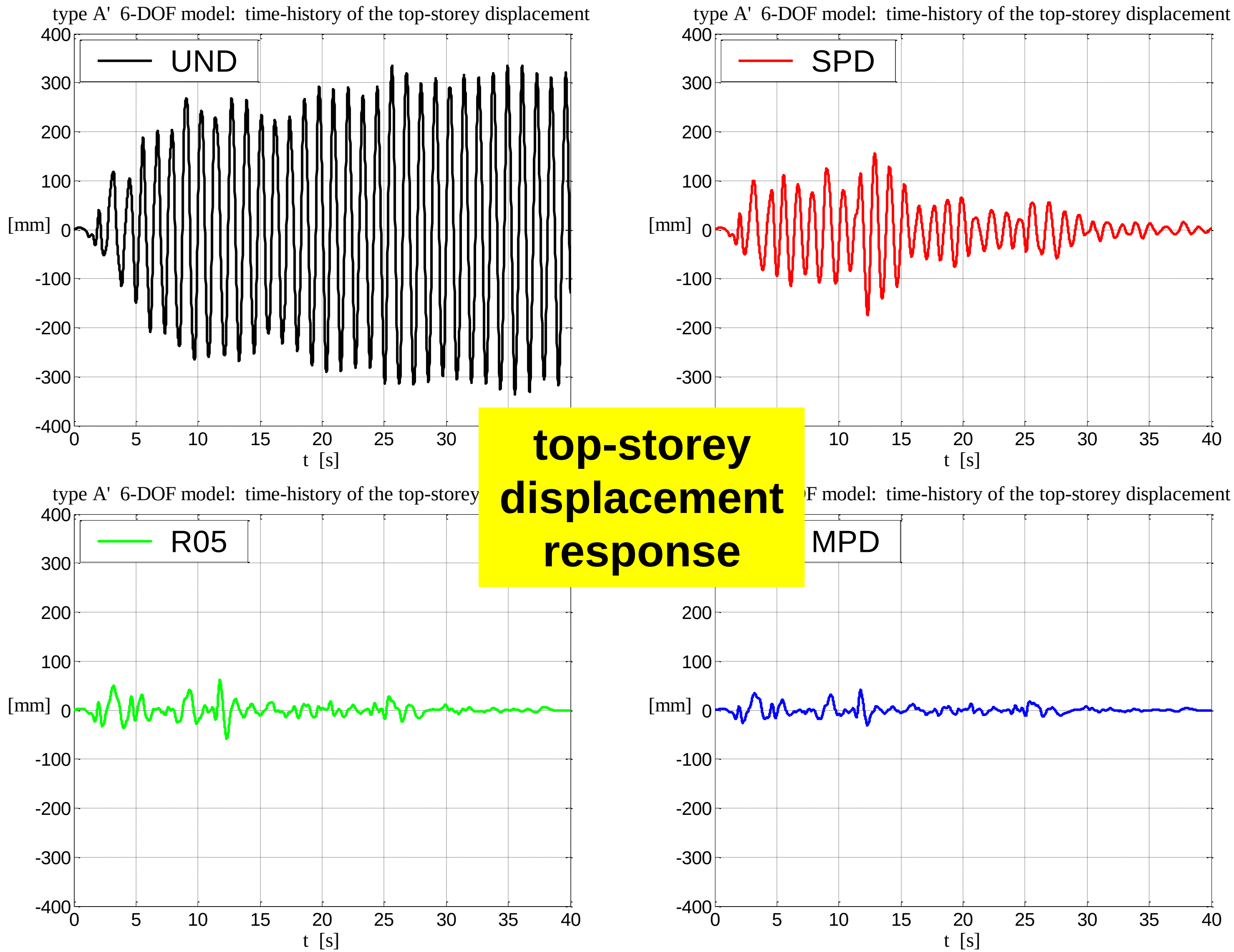


$T_1 = 1.17$s

El Centro 1940

scaled to

PGA = 0.3*g*

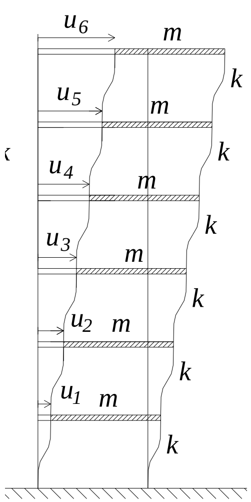


top-storey displacement response

Seismic response of MPD and SPD systems

6-DOF

$\bar{c} = 4.452 \cdot 10^6 \; N \cdot s / m$



$T_1 = 1.17\text{s}$

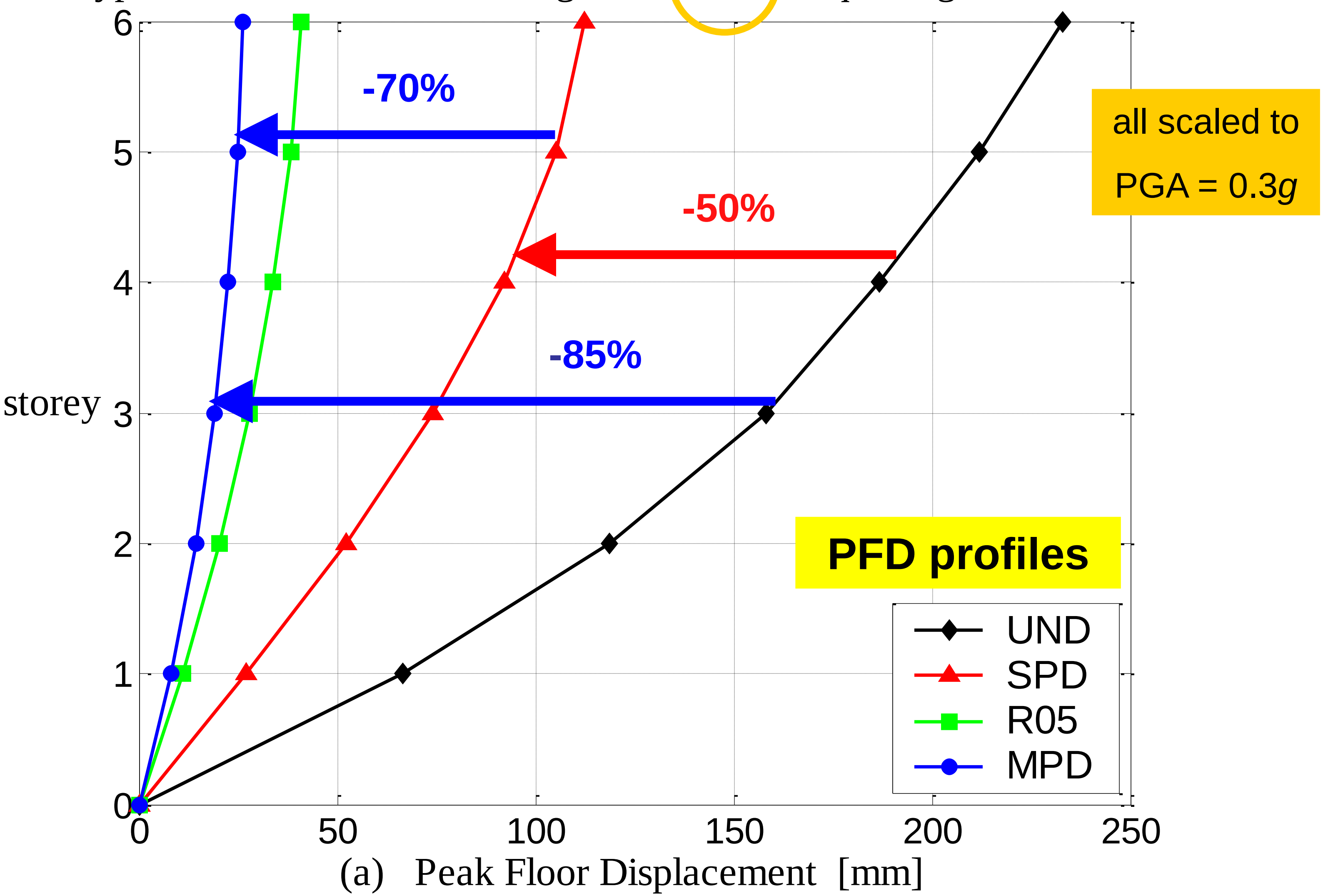
$\xi_1^{MPD} = 86\%$

$\xi_1^{SPD} = 5\%$

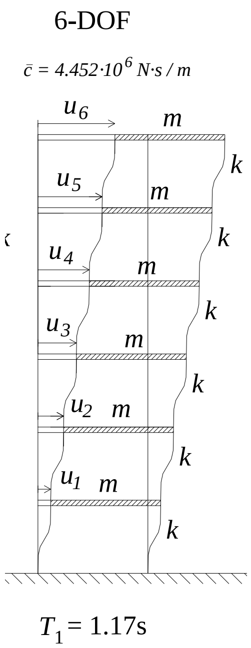
$\xi_2^{MPD} = 29\%$

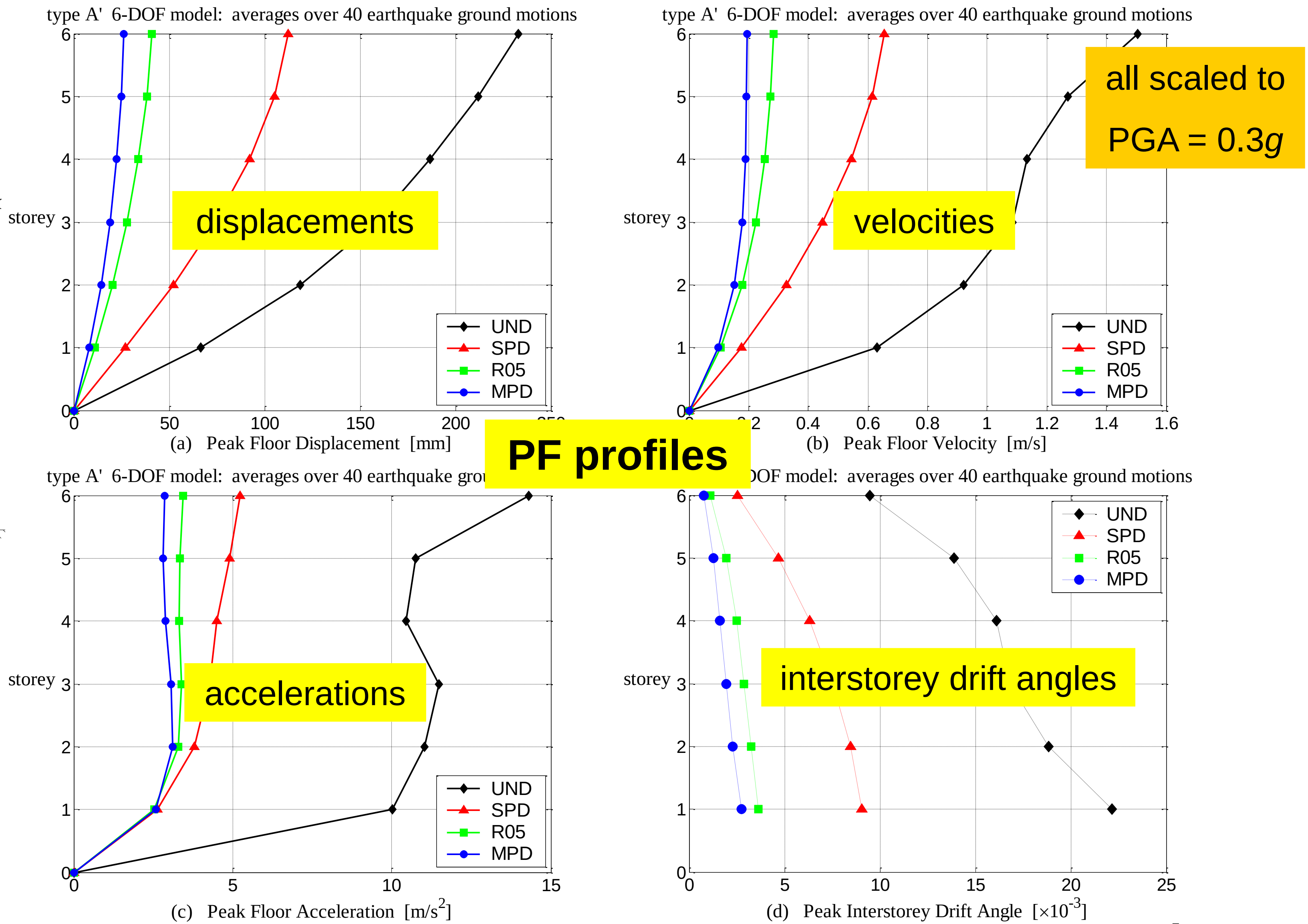
$\xi_2^{SPD} = 15\%$

type A' 6-DOF model: averages over 40 earthquake ground motions



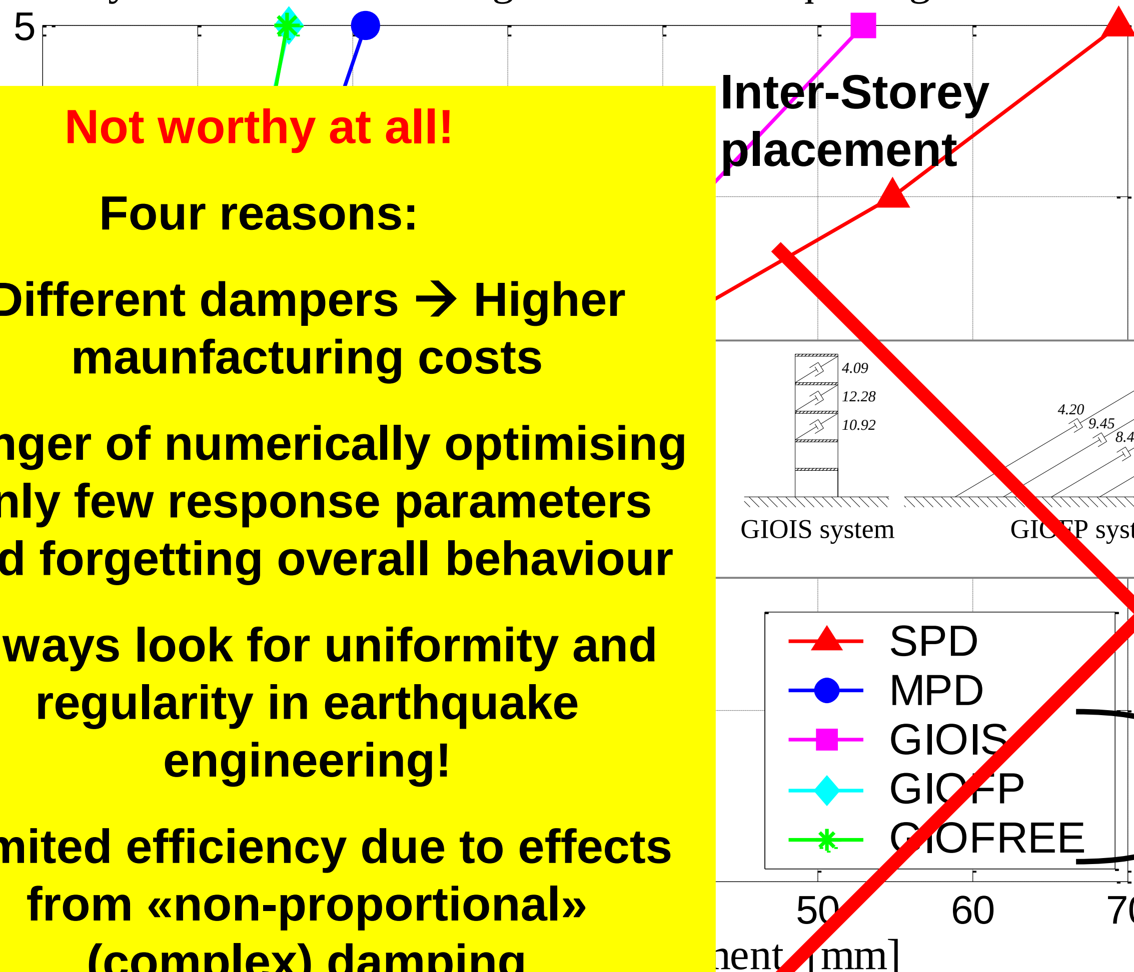
Seismic response of MPD and SPD systems

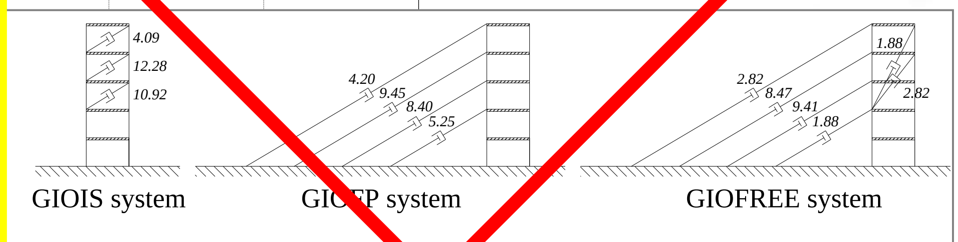


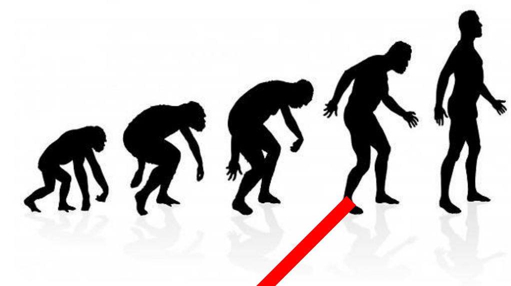


Seismic response of Genetically Identified Optimal (GIO) systems

5-storey r.c. structure: averages over 40 earthquake ground motions







Not worthy at all!

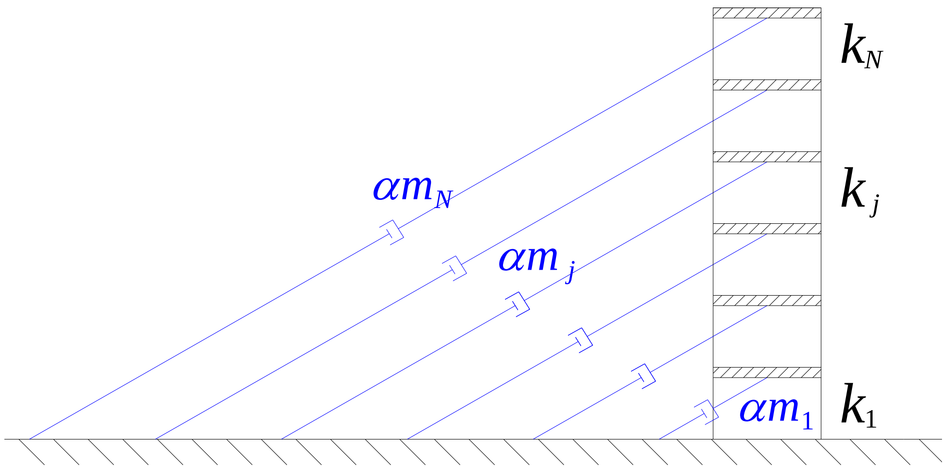
Four reasons:

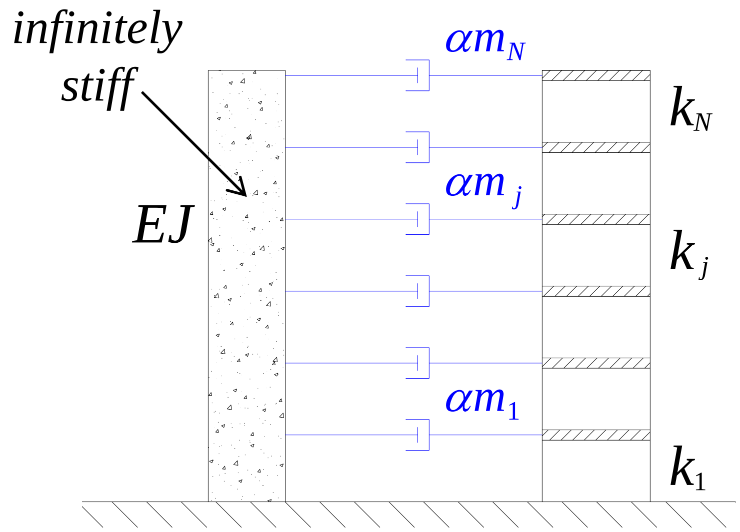
1) **Different dampers → Higher maunfacturing costs**
2) **Danger of numerically optimising only few response parameters and forgetting overall behaviour**
3) **Always look for uniformity and regularity in earthquake engineering!**
4) **Limited efficiency due to effects from «non-proportional» (complex) damping**

Objective function:

average of the standard deviations of the interstory drift angles

Implementation of MPD systems





<u>DIRECT</u> IMPLEMENTATION

Use of long buckling-resistant braces (unbonded braces, buckling-restrained-braces BRB, mega-braces, prestressed steel cables, ...)

<u>INDIRECT</u> IMPLEMENTATION

Dampers placed between the structure and a very stiff vertical lateral-resistant element

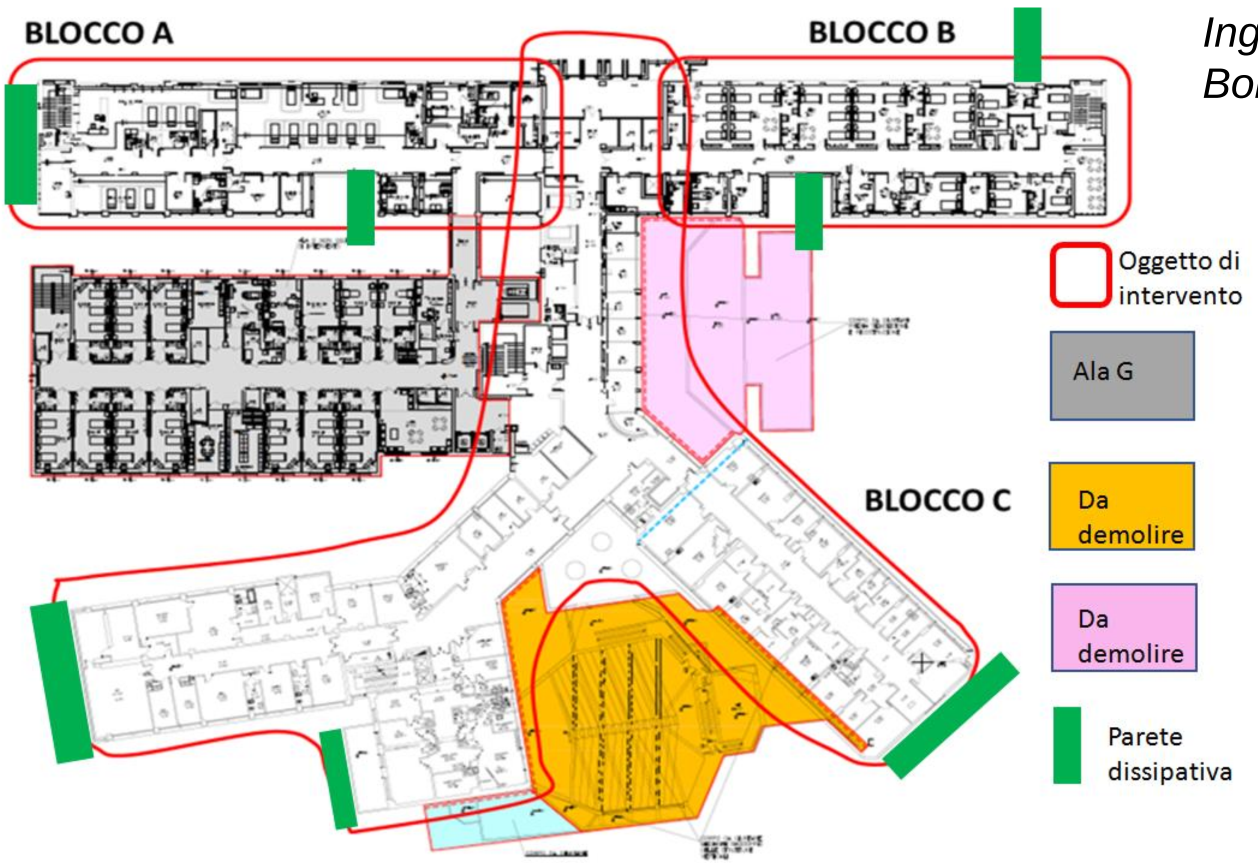
Indirect Implementation of MPD systems

S.Orsola hospital, Bologna
Pavilion n.5

Indirect Implementation of MPD systems

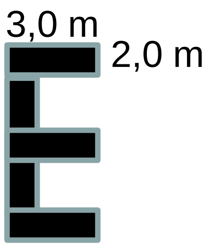
S.Orsola hospital, Bologna
Pavilion n.5

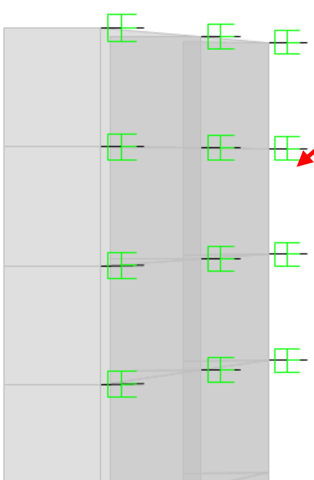
2020-2021
executive design by:
Prof. Ing. Tomaso Trombetti
Ing. Friedrich Drollmann
Bologna

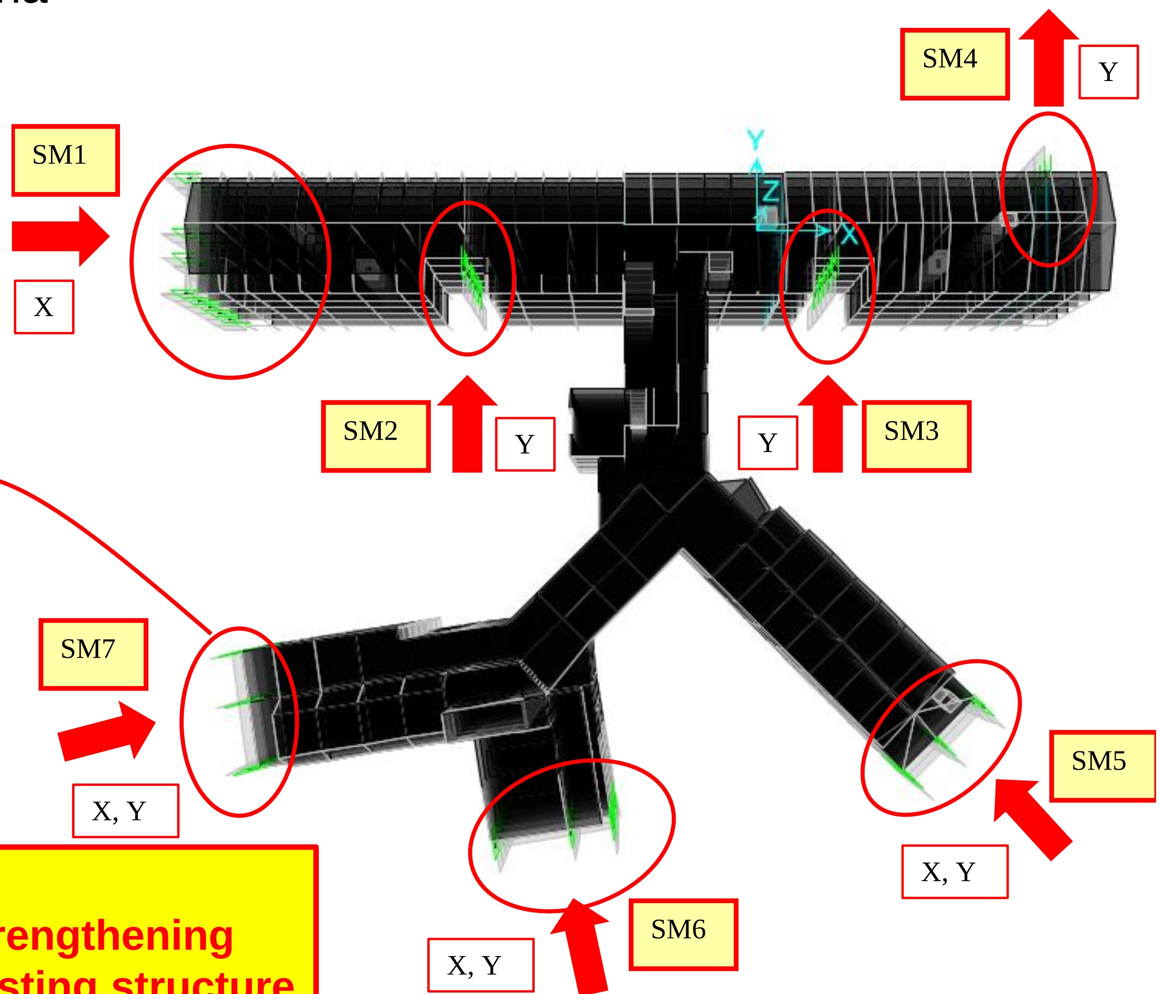


Indirect Implementation of MPD systems

S.Orsola hospital, Bologna
Pavilion n.5



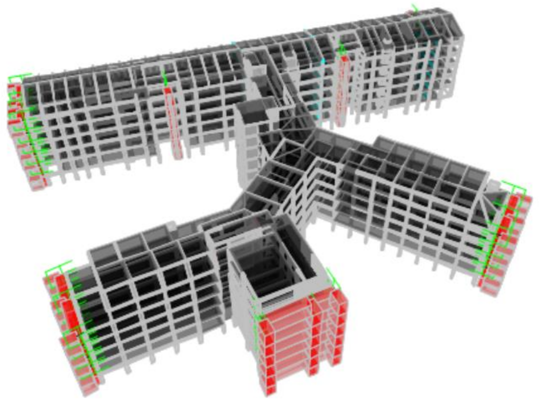




advantage:
non-invasive seismic strengthening (from outside) of the existing structure

Indirect Implementation of MPD systems

S.Orsola hospital, Bologna
Pavilion n.5



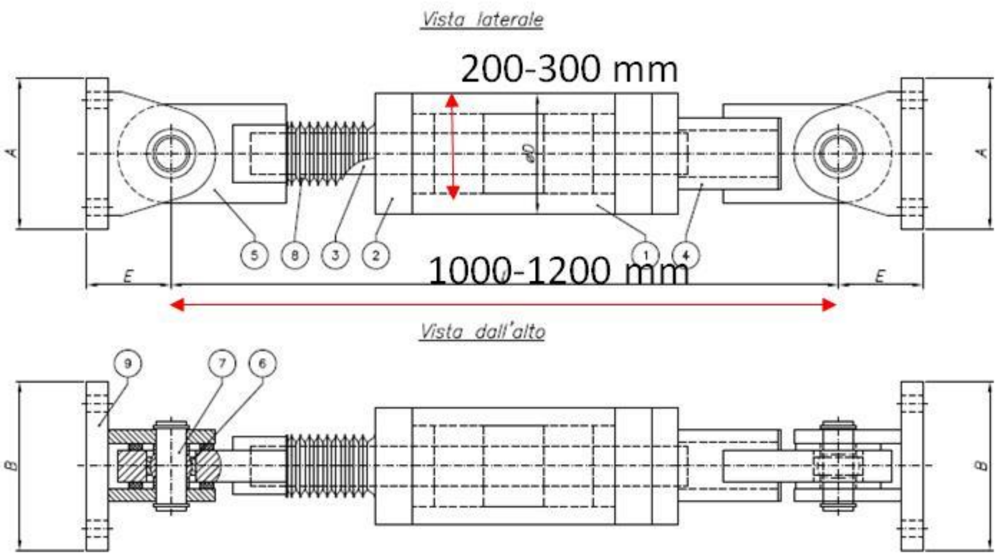
Caratteristiche di progetto

		OTP 68/100	OTP 90/120	OTP 120/150
Quantità		21	12	44
Costante di smorzamento	C	800 kN/(m/s)$^{\alpha}$	1000 kN/(m/s)$^{\alpha}$	1250 kN/(m/s)$^{\alpha}$
Velocità massima	v	0,322 m/s	0,465 m/s	0,762 m/s
Esponente di smorzamento	α	0,15	0,15	0,15
Forza di progetto	F_d	680 kN	900 kN	1200 kN
Spostamento sismico (SLV)	d_{bd}	±23 mm	±29 mm	±36 mm
Scorrimento totale	d_{Ed}	±50 mm	±60 mm	±75 mm
Lunghezza perno–perno	L	1000 +/−50	1110 +/−60	1200 +/−75
Diametro cilindro	øD	ø225		
Dimensione struttura	A	310		
Dimensione struttura	B	310		
Dimensione struttura	E	165		

tot. 77

NOTE
1. Viti/barre di ancoraggio (min classe 8.8 ISO898) non sono comprese
2. La geometria indicata consente il montaggio del dispositivo e la rotazi
3. La tolleranza di installazione consentita sulla lunghezza è ±5 mm.

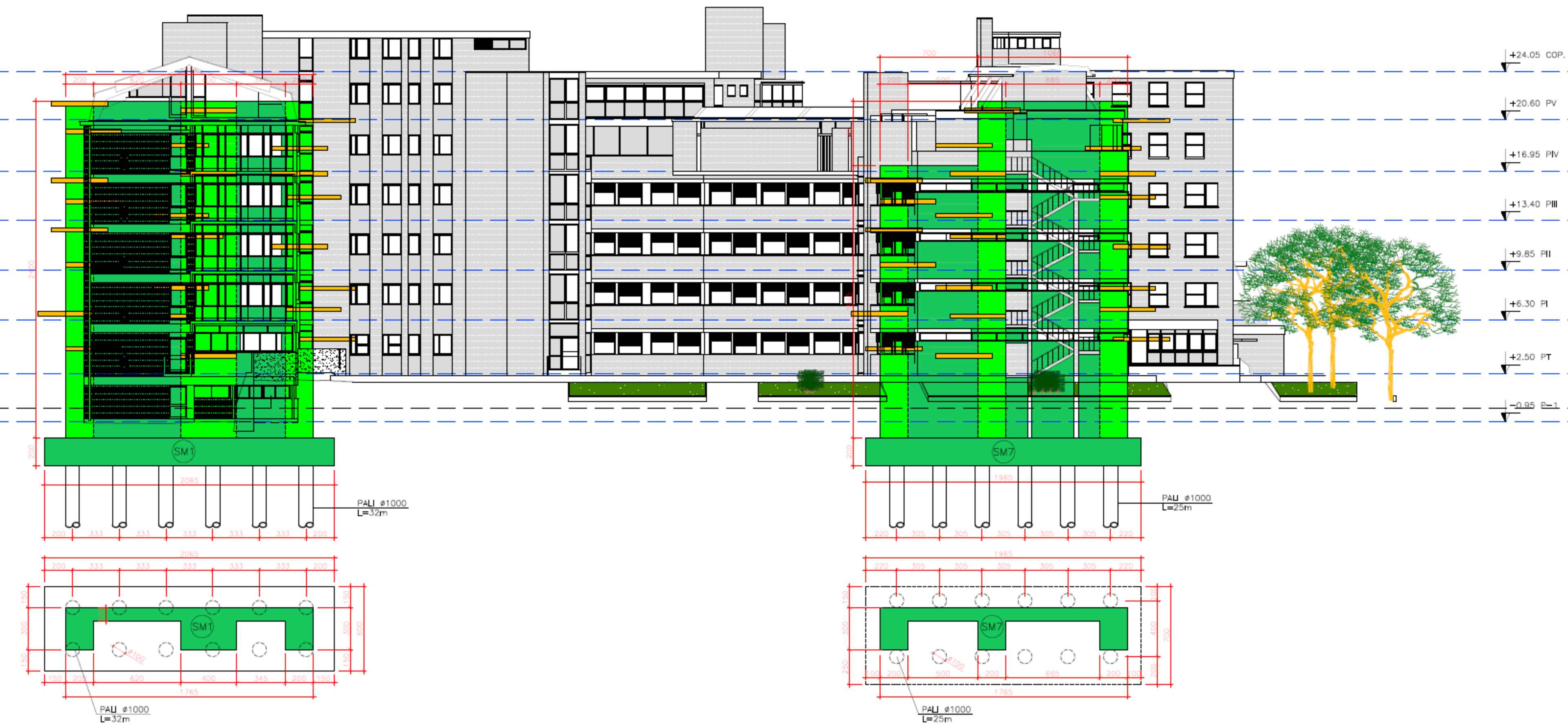
PROTEZIONE DALLA CORROSIONE
Superficie esterne:
– strato epossi–ammino ciclo–alifatico: 280–350 µm, RAL7035



Indirect Implementation of MPD systems

S.Orsola hospital, Bologna
Pavilion n.5

PROSPETTO SUD - EST



Indirect Implementation of MPD systems

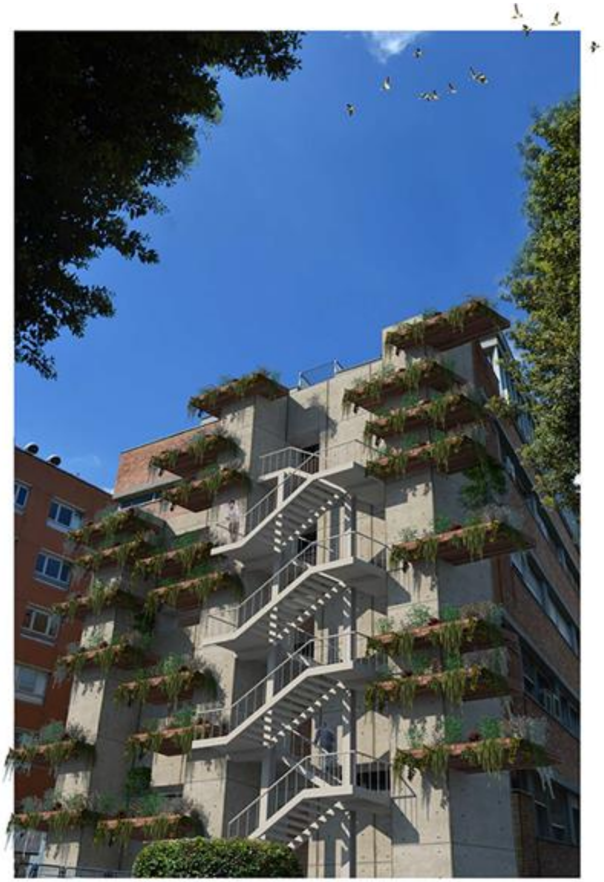
S.Orsola hospital, Bologna
Pavilion n.5

PROSPETTO SUD - EST

Indirect Implementation of MPD systems

S.Orsola hospital, Bologna
Pavilion n.5

VISTA 1



VISTA 2

VISTA 3

Properties of MPD and SPD systems (7)

$$\underset{\sim}{\mathbf{c}} = \alpha \ \underset{\sim}{\mathbf{m}}$$

$$\xi_1^{MPD} = \frac{\alpha}{2\omega_1} \qquad \alpha = 2 \cdot \xi_1^{MPD} \cdot \omega_1$$

$$\underset{\sim}{\mathbf{c}} = 2 \cdot \xi_1^{MPD} \cdot \omega_1 \cdot \underset{\sim}{\mathbf{m}}$$

$$\boxed{c_{storey,j} = 2 \cdot \xi_1^{MPD} \cdot \omega_1 \cdot m_j}$$

m_j is easy to be calculated

$$\boxed{c_{tot,MPD} = 2 \cdot \xi_1^{MPD} \cdot \omega_1 \cdot m_{tot}}$$

$$\underset{\sim}{\mathbf{c}} = \beta \ \underset{\sim}{\mathbf{k}}$$

$$\xi_1^{SPD} = \frac{\beta\omega_1}{2} \qquad \beta = \frac{2 \cdot \xi_1^{SPD}}{\omega_1}$$

$$\underset{\sim}{\mathbf{c}} = \frac{2 \cdot \xi_1^{SPD}}{\omega_1} \cdot \underset{\sim}{\mathbf{k}}$$

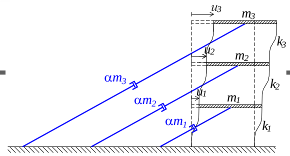
$$\boxed{c_{storey,j} = \frac{2 \cdot \xi_1^{SPD}}{\omega_1} \cdot k_j}$$

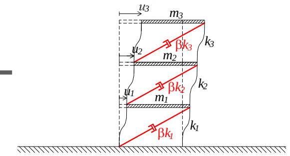
k_j is not so immediate ...

$$\boxed{c_{tot,SPD} = \ldots}$$

?

Properties of MPD and SPD systems (8)





$$c_{tot,MPD} = 2 \cdot \xi_1^{MPD} \cdot \omega_1 \cdot m_{tot}$$

$$\xi_1^{MPD} = \frac{c_{tot}}{2 \cdot \omega_1 \cdot m_{tot}}$$

$$c_{tot,SPD} = \xi_1^{SPD} \cdot \omega_1 \cdot m_{tot} \cdot N(N+1)$$

$$\xi_1^{SPD} = \frac{c_{tot}}{\cancel{2} \cdot \omega_1 \cdot m_{tot}} \cdot \frac{\cancel{2}}{N(N+1)}$$

$$\xi_1^{SPD} \frac{N(N+1)}{2} = \frac{c_{tot}}{2 \cdot \omega_1 \cdot m_{tot}}$$

*

$$\frac{\xi_1^{SPD}}{\xi_1^{MPD}} \cong \frac{2}{N(N+1)}$$

$$\xi_1^{MPD} \cong \xi_1^{SPD} \frac{N(N+1)}{2}$$

Fundamental design formulas

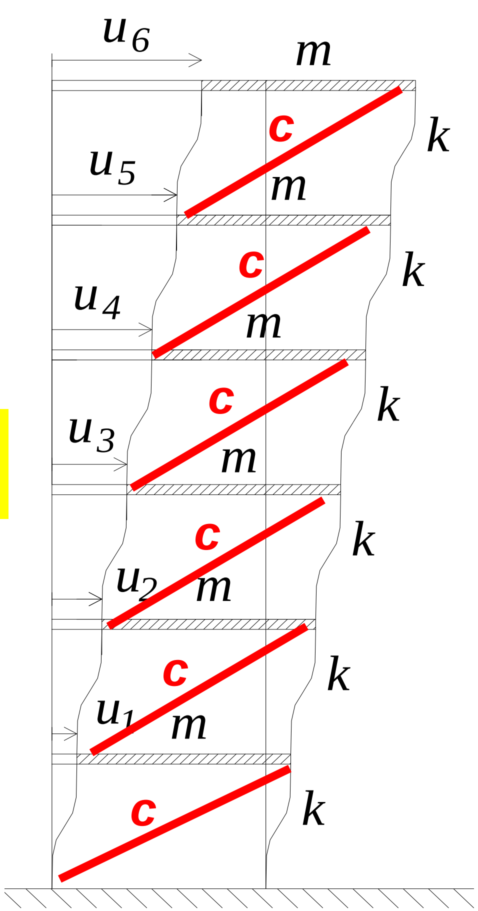
If a target damping ratio is looked for: $\xi_1^{SPD} = \bar{\xi}$

the **fundamental results** are:

$$c_{tot,SPD} = \bar{\xi} \cdot \omega_1 \cdot m_{tot} \cdot N(N+1)$$

$$c_{storey,SPD} = \frac{c_{tot,SPD}}{N} = \bar{\xi} \cdot \omega_1 \cdot m_{tot} \cdot (N+1)$$

**

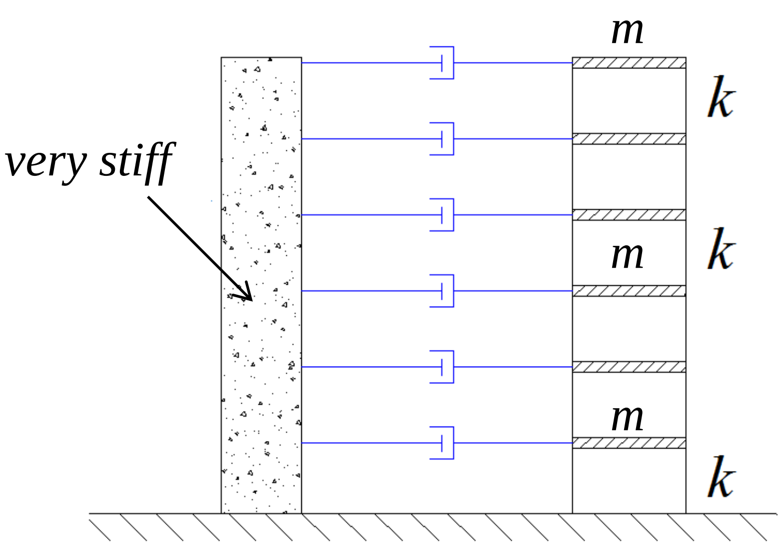


The above equations allow to size the damping coefficients of each damper of an inter-storey dampers system, in order to get a target damping ratio $\bar{\xi}$, **by simply knowing:**

- **the total mass m_{tot}**
- **the fundamental period of vibration T_1 ($\omega_1 = \frac{2\pi}{T_1}$)**
- **the total number of storeys *N***

Fundamental design formulas

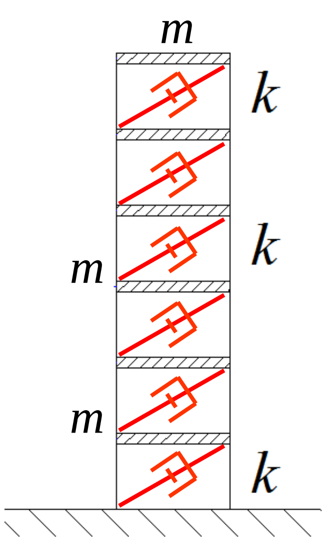
MPD system



$$c_{tot,MPD} = 2 \cdot \bar{\xi} \cdot \omega_1 \cdot m_{tot}$$

$$c_{storey,MPD} = \frac{2 \cdot \bar{\xi} \cdot \omega_1 \cdot m_{tot}}{N}$$

SPD system



$$c_{tot,SPD} = \bar{\xi} \cdot \omega_1 \cdot m_{tot} \cdot N(N+1)$$

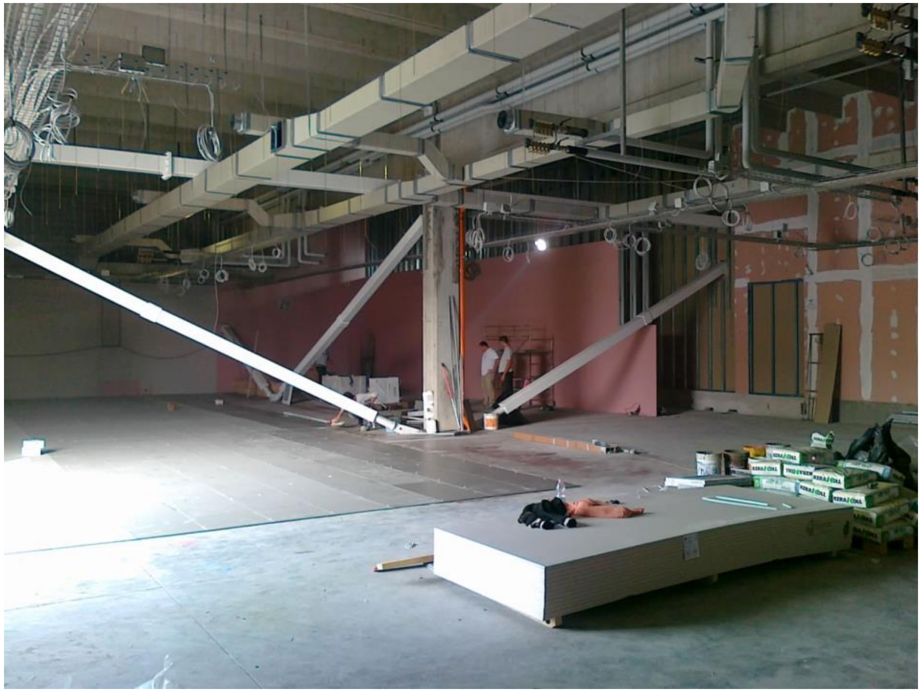
$$c_{storey,SPD} = \bar{\xi} \cdot \omega_1 \cdot m_{tot} \cdot (N+1)$$

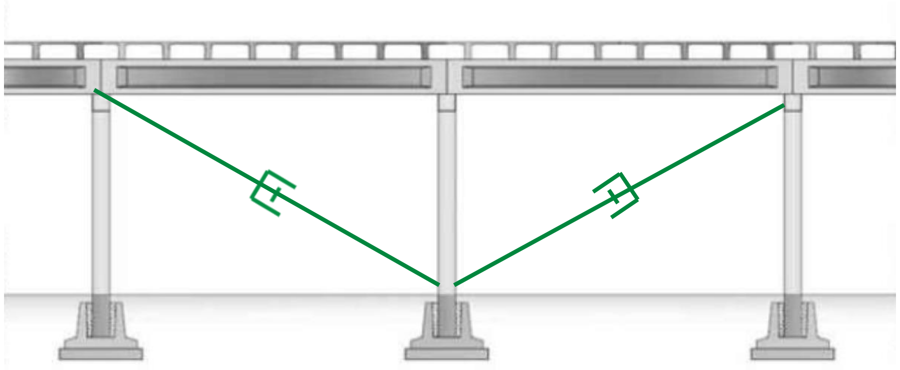
**

One-storey buildings: SDOF systems

MPD system ≡ SPD system

$$c_{tot,MPD} = c_{tot,SPD} = 2 \cdot \bar{\xi} \cdot \omega_1 \cdot m_{tot}$$





HERA building, Imola (BO)

The five-step procedure (2010)

Design strategy

- The design philosophy is **to limit the structural damages** under severe earthquakes.

- The structural elements (columns and beams) should remain **in the elastic phase**.

- Let's keep the **ductility resources** of columns and beams **as an additional property** to withstand very severe and unexpected earthquakes.

The five-step procedure for inter-storey dampers placement

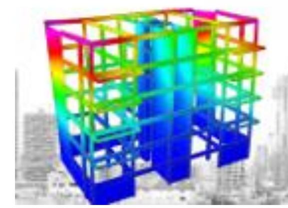
DESIGN PROCESS

STEP 1: performance objectives ⇨ $\bar{\eta} \rightarrow \bar{\xi}$ $\quad \bar{\eta} = \sqrt{\frac{10}{5+\bar{\xi}}}$

STEP 2: linear dampers ⇨ $\bar{\xi} \rightarrow \overline{c_L}$ $\quad \overline{c_L} = \bar{\xi} \cdot \omega_1 \cdot m_{tot} \cdot \left(\frac{N+1}{n}\right)$

STEP 3: linear TH analyses ⇨ structural response

FEM



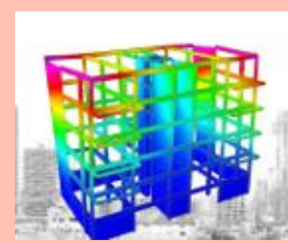
⇨ v_{max}

STEP 4: non-linear dampers ⇨ $\overline{c_L} \rightarrow \begin{cases} \overline{c_{NL}} \\ \bar{\alpha} \\ \overline{k_{oil}} \end{cases}$ $\quad \overline{c_{NL}} = \overline{c_L} \cdot (\chi \cdot v_{max})^{1-\bar{\alpha}}$

VERIFICATION

STEP 5: non-linear TH analyses ⇨ structural response

FEM



Silvestri et al. 2010 (JEE)

The five-step procedure for inter-storey dampers placement

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A Five-Step Procedure for the Dimensioning of Viscous Dampers to Be Inserted in Building Structures

STEFANO SILVESTRI, GIADA GASPARINI,
and TOMASO TROMBETTI

Department DISTART, University of Bologna, Bologna, Italy

Viscous dampers have widely proved their effectiveness in mitigating the effects of the seismic action upon building structures. In view of the large impact that use of such dissipative devices is already having and would most likely have soon in earthquake engineering applications, this article presents a practical procedure for the seismic design of building structures equipped with viscous dampers, which aims at providing practical tools for an easy identification of the mechanical characteristics of the manufactured viscous dampers which allow to achieve target levels of performances. Selected numerical applications are developed with reference to simple, but yet relevant, cases.

Keywords Added Viscous Dampers; Seismic Design; Design Procedure; Nonlinear Modeling; Damping Ratio

Starting point

$\bar{\eta}$

$$\bar{\eta} = \frac{V_{base,\xi}}{V_{base,\xi=5\%}}$$

$$\bar{\eta} = \frac{M_{Ed,\xi}}{M_{Ed,\xi=5\%}}$$

$$\bar{\eta} = \frac{\delta_{top-storey,\xi}}{\delta_{top-storey,\xi=5\%}}$$

EXAMPLE:
If the bending moment action at the base of a column is $M_{Ed,\xi=5\%}$ = 800 kNm and if the bending moment resistance is M_{Rd} = 400 KNm
then
we want that dampers lead to $M_{Ed,\xi}$ = 400 kNm

$$\bar{\eta} = \frac{400}{800} = 0.5$$

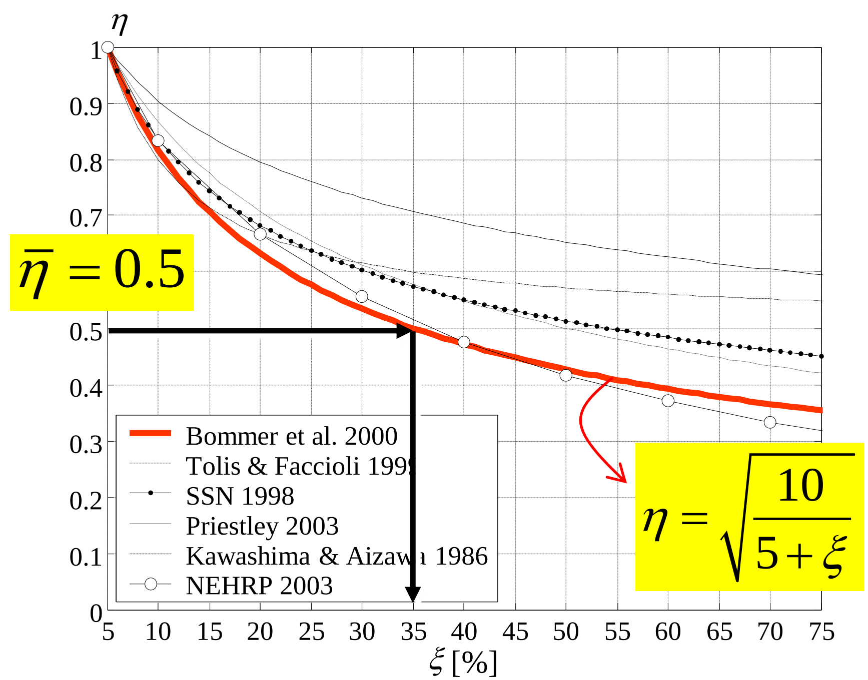
$$\bar{\eta} = \frac{\text{target seismic demand (i.e. capacity/strength or acceptable action or drift)}}{\text{actual seismic demand with no dampers}}$$

Step 1

$\bar{\eta} \longrightarrow \bar{\xi}$

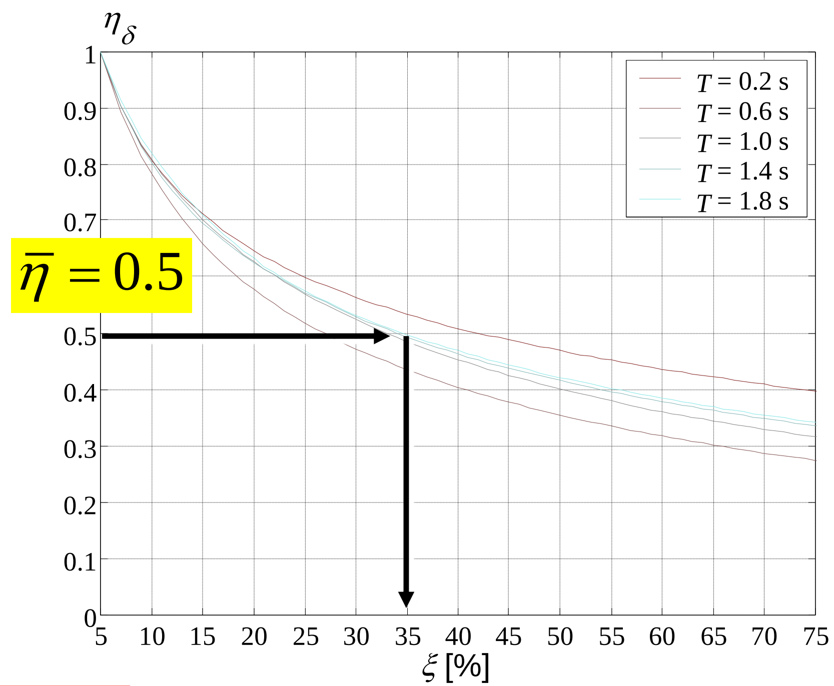
Some available formulations to relate $\bar{\eta} \longrightarrow \bar{\xi}$

Cardone, Dolce, Rivelli (ANIDIS 2007)



$\bar{\xi} = 35\%$

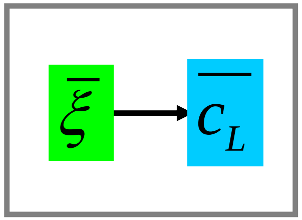
mean SDOF response from T-H analyses



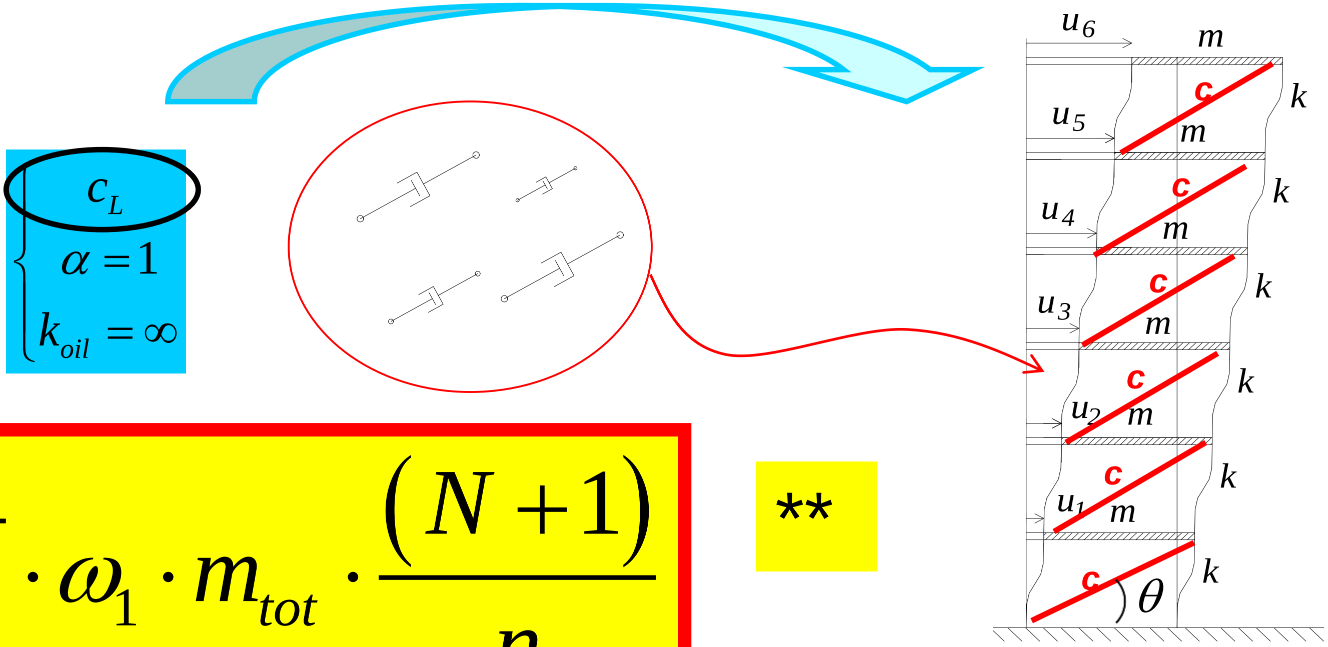
$\bar{\xi} = 35\%$

$$\bar{\xi} = \frac{10}{\bar{\eta}^2} - 5$$

Step 2



Preliminary sizing of linear viscous damper using analytical formula **



$$c_L = \bar{\xi} \cdot \omega_1 \cdot m_{tot} \cdot \frac{(N+1)}{n}$$

**

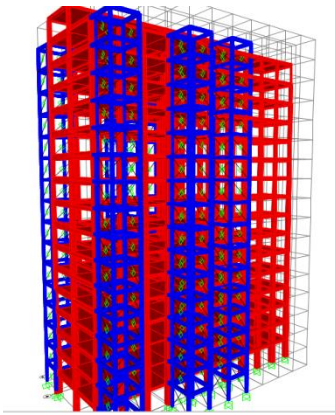
$$c_{L,\text{inclined}} = c_{L,\text{horizontal}} \cdot \left(\frac{1}{\cos^2 \theta} \right)$$

n = number of dampers at a given storey in a given direction

Step 3

FEM
simulations

After linear viscous dampers are dimensioned



$$c_L = \overline{c_L} \quad , \quad \alpha = 1 \quad , \quad k_{oil} = \infty$$

Linear TH dynamic analyses are necessary in order to:

1. Verify by means of **snap-back tests** that the actual damping properties of the model are in line with the expected ones (e.g. **target damping ratio is achieved**) $\bar{\xi}$

2. Calculate the **maximum working velocities of the linear dampers** v_{max}

3. Calculate **the maximum damper piston-strokes** x_{max}

Step 4

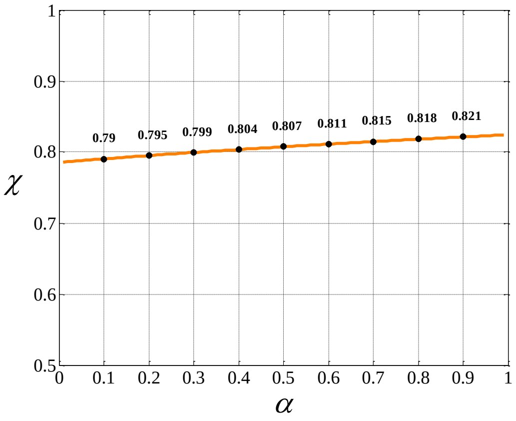
$$\overline{c_L} \rightarrow \begin{cases} \overline{c_{NL}} \\ \overline{\alpha} \\ \overline{k_{oil}} \end{cases}$$

ENERGETIC APPROACH:

equal energy dissipated over a full cycle of harmonic motion

$$E_{d,L} = E_{d,NL} \longrightarrow$$

$$\begin{aligned}\overline{c_{NL}} &= \overline{c_L} \cdot \left(v_{\max}\right)^{1-\overline{\alpha}} \cdot \chi = \\ &= \overline{c_L} \cdot \left(\chi \cdot v_{\max}\right)^{1-\overline{\alpha}} \cdot \chi^{\overline{\alpha}} \\ &\cong \overline{c_L} \cdot \left(\chi \cdot v_{\max}\right)^{1-\overline{\alpha}} \cdot 0.8^{0.15} = \overline{c_L} \cdot \left(\chi \cdot v_{\max}\right)^{1-\overline{\alpha}}\end{aligned}$$

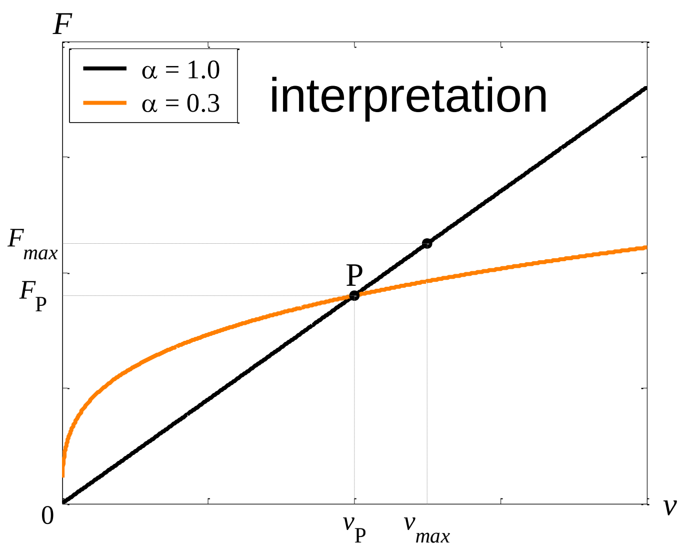


$$\chi = \left(\frac{\sqrt{\pi}}{2} \frac{\Gamma\left(\frac{\overline{\alpha}+3}{2}\right)}{\Gamma\left(\frac{\overline{\alpha}+2}{2}\right)} \right)^{\frac{1}{1-\overline{\alpha}}} \cong 0.8 \quad , \quad \forall \alpha$$

REFERENCE POINT P:

$$v_P = 0.8 \cdot v_{\max}$$

$$F_P = 0.8 \cdot \overline{c_L} \cdot v_{\max}$$

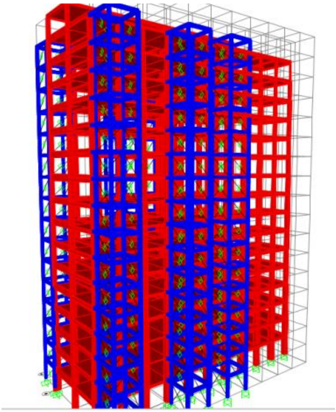


$$\overline{\alpha} = 0.15$$

$$\overline{k_{oil}} \geq 10 \cdot \frac{F_{\max}}{x_{\max}} \cong 10 \cdot \overline{c_L} \cdot \omega_1$$

Step 5

FEM
simulations



After non-linear viscous dampers are dimensioned

$$c_{NL} = \overline{c_{NL}} \quad , \quad \alpha = \overline{\alpha} \quad , \quad k_{oil} = \overline{k_{oil}} \geq 10 \cdot \overline{c_L} \cdot \omega_1$$

Non-linear TH dynamic analyses are necessary in order to:

1. Verify the effectiveness of the non-linear dampers in reducing the global structural response (e.g. **target damping ratio is achieved, maximum forces in the structural elements are acceptable**)

2. Evaluate the **maximum damper forces in the non-linear dampers**

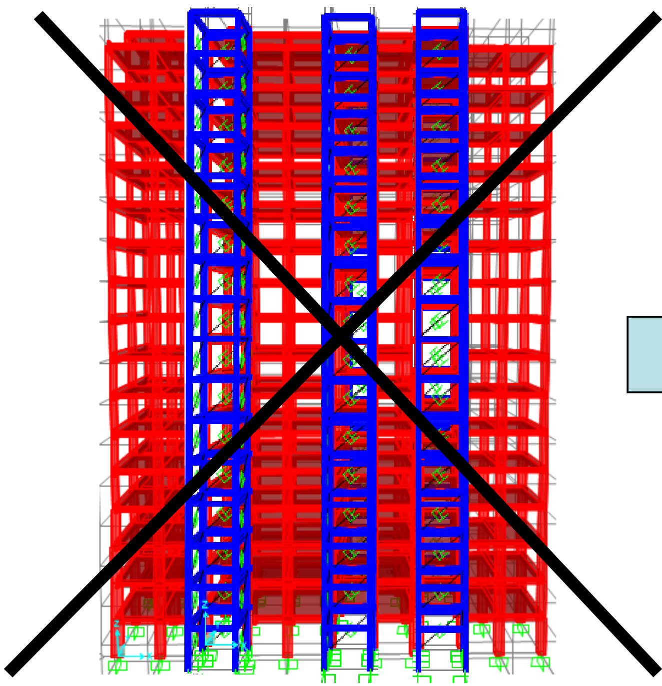
3. Evaluate **the maximum strokes in the non-linear dampers**

The direct five-step procedure (2016-2018)

A step forward:
a <u>direct</u> five-step procedure

The challenge:

Can we directly design (at least PRELIMINARY SIZING) the viscous dampers and the frames without performing TH dynamic analyses?



The five-step procedure for inter-storey dampers placement

DESIGN PROCESS

STEP 1: performance objectives ⇨ $\bar{\eta} \rightarrow \bar{\xi}$

$$\bar{\eta} = \sqrt{\frac{10}{5+\bar{\xi}}}$$

STEP 2: linear dampers ⇨ $\bar{\xi} \rightarrow \overline{c_L}$

$$\overline{c_L} = \bar{\xi} \cdot \omega_1 \cdot m_{tot} \cdot \left(\frac{N+1}{n} \right)$$

STEP 3: linear TH analyses ⇨ structural response

⇨ v_{max}

STEP 4: non-linear dampers ⇨ $\overline{c_L} \rightarrow \begin{cases} \overline{c_{NL}} \\ \bar{\alpha} \\ \overline{k_{oil}} \end{cases}$

$$\overline{c_{NL}} = \overline{c_L} \cdot (\chi \cdot v_{max})^{1-\bar{\alpha}}$$

VERIFICATION

STEP 5: non-linear TH analyses ⇨ structural response

Silvestri et al. 2010 (JEE)

The five-step procedure for inter-storey dampers placement

DESIGN PROCESS

STEP 1: performance objectives ⇨ $\bar{\eta} \rightarrow \bar{\xi}$ $\quad \bar{\eta} = \sqrt{\dfrac{10}{5+\bar{\xi}}}$

STEP 2: linear dampers ⇨ $\bar{\xi} \rightarrow \overline{c_L}$ $\quad \overline{c_L} = \bar{\xi} \cdot \omega_1 \cdot m_{tot} \cdot \left(\dfrac{N+1}{n}\right)$

STEP 3: linear TH analyses ⇨ structural response

1. Estimation of inter-storey velocities

STEP 4: non-linear dampers ⇨ $\overline{c_L} \rightarrow \begin{cases} \overline{c_{NL}} \\ \bar{\alpha} \\ \overline{k_{oil}} \end{cases}$ $\quad \overline{c_{NL}} = \overline{c_L} \cdot (\chi \cdot v_{max})^{1-\bar{\alpha}}$

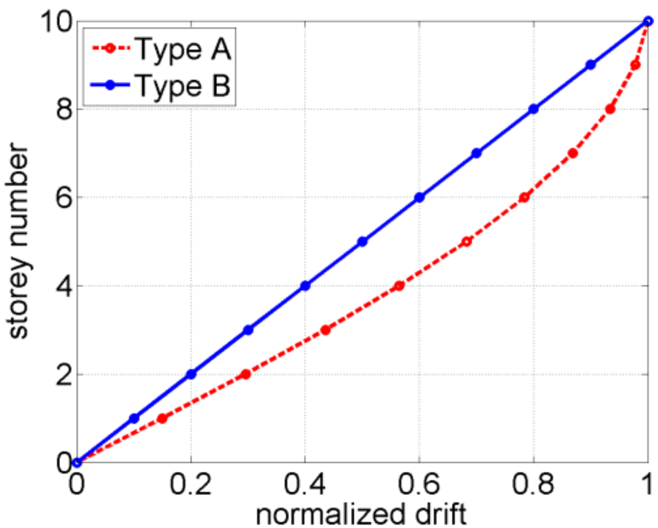
VERIFICATION

STEP 5: non-linear TH analyses ⇨ structural response

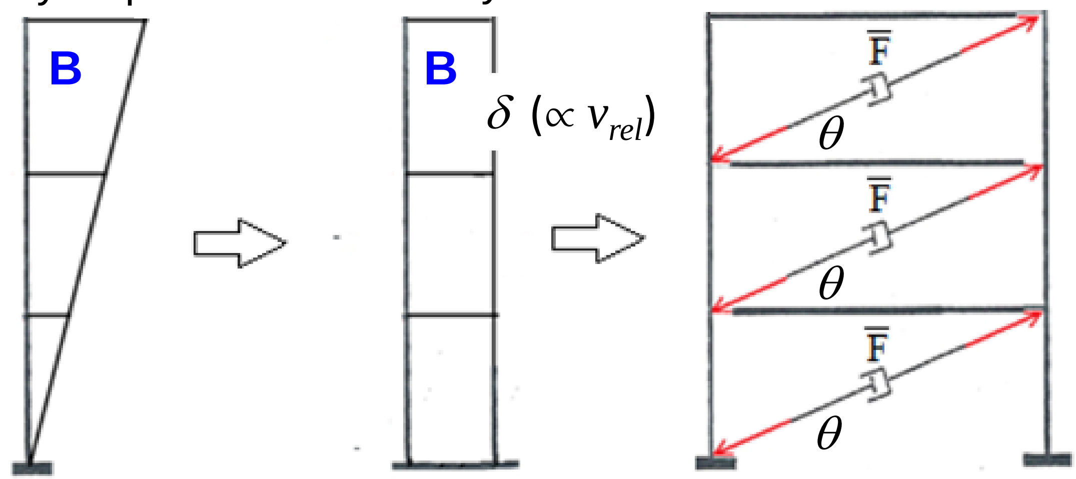
2. Equivalent Static Analysis (ESA)

1. Inter-storey velocities and damper forces

Analytical Estimations Based On First Mode Response



storey displ. interstorey drift



B

$$\delta_{max} = \frac{S_{e,\xi}(T_1)}{\omega_1^2} \cdot \frac{2}{(N+1)}$$

$$v_{max} = \frac{S_{e,\xi}(T_1)}{\omega_1} \cdot \frac{2}{(N+1)} \cdot \cos\theta$$

$$F_{D\max} = \frac{2 \cdot \xi \cdot m_{tot} \cdot S_{e,\xi}(T_1)}{\cos\theta}$$

A

$$\delta_{max} = \frac{S_{e,\xi}(T_1)}{\omega_1^2} \frac{12N}{(2+5N+5N^2)}$$

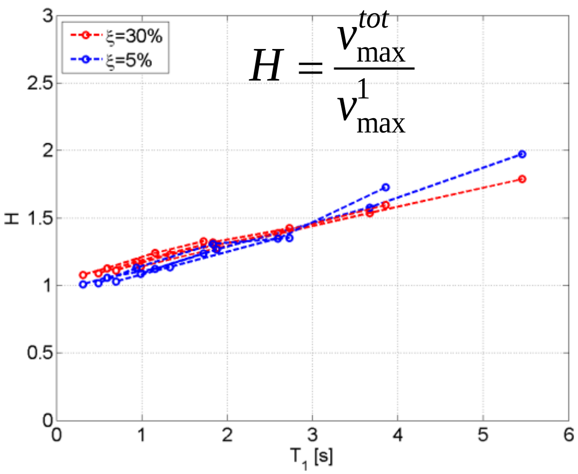
$$v_{max} = \frac{S_{e,\xi}(T_1)}{\omega_1} \cdot \frac{12N}{(2+5N+5N^2)} \cdot \cos\theta$$

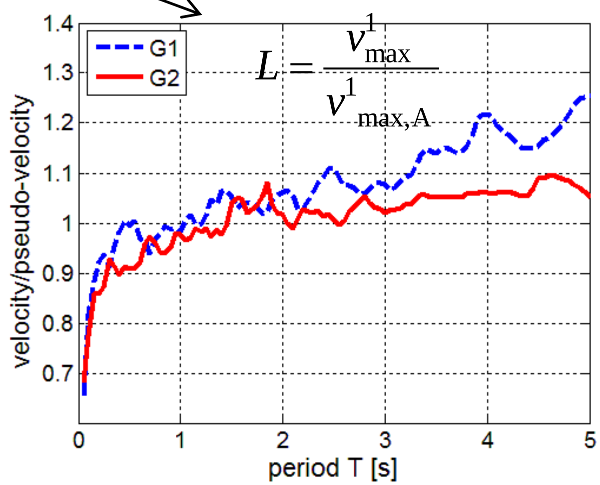
...

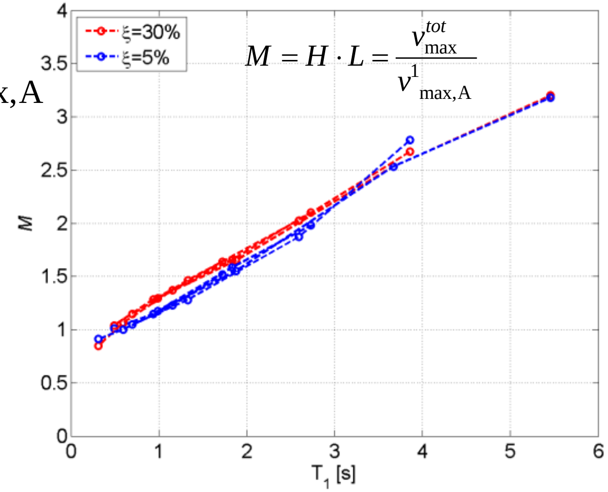
Palermo et al. 2016 (BEE)

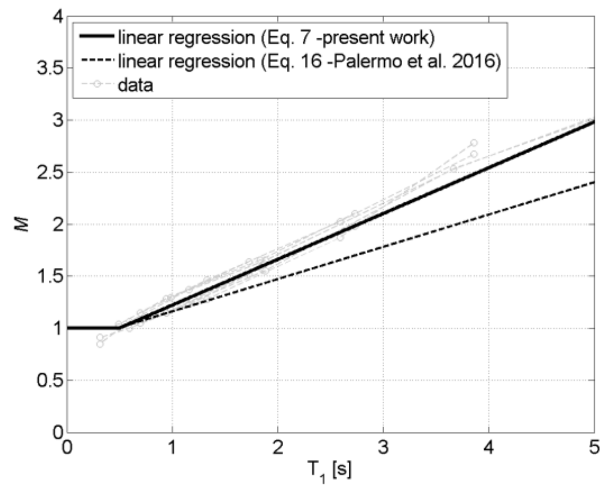
1. Higher modes contribution

$$v_{\max}^{tot} = \left(\frac{v_{\max}^{tot}}{v_{\max}^{1}}\right) \cdot \left(\frac{v_{\max}^{1}}{v_{\max,A}^{1}}\right) \cdot v_{\max,A}^{1} = H \cdot L \cdot v_{\max,A}^{1} = M \cdot v_{\max,A}^{1}$$







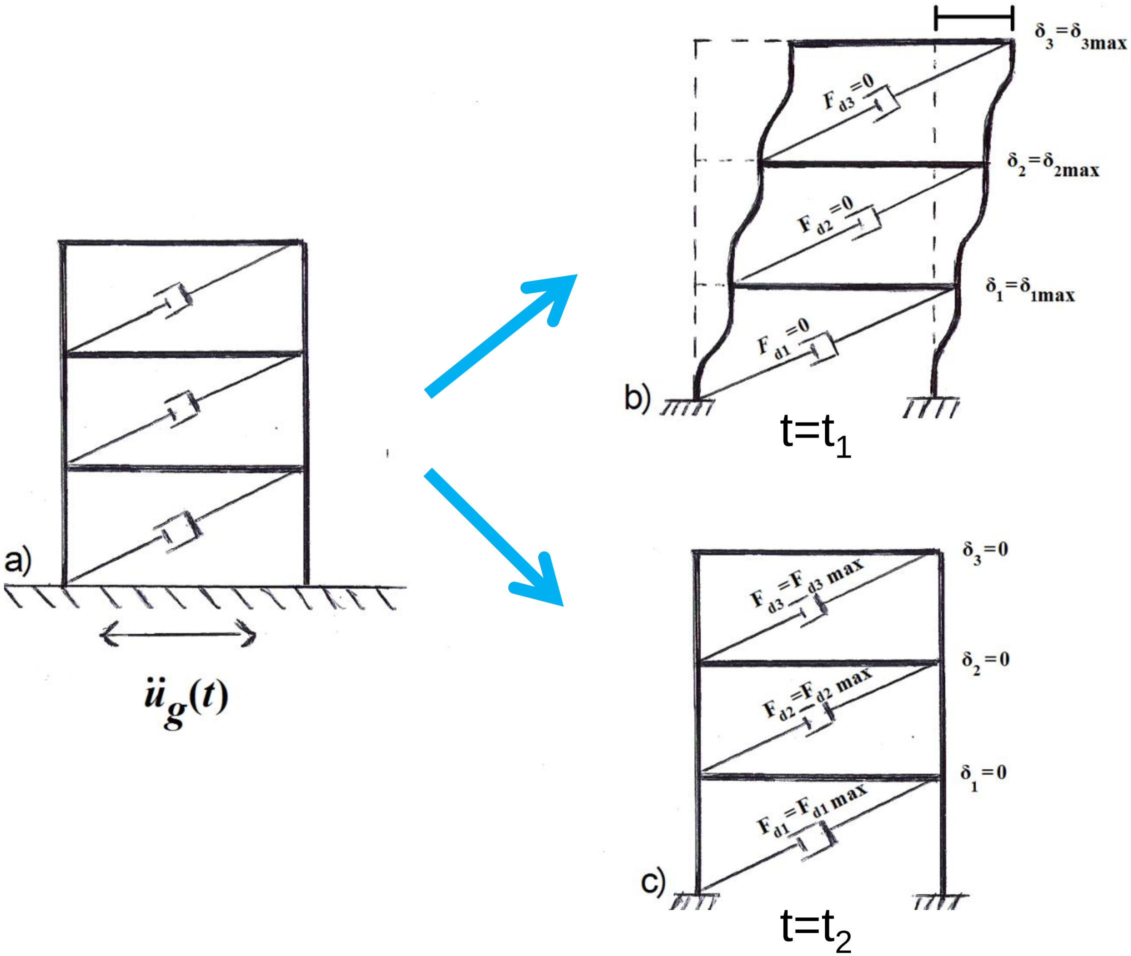


$$v_{\max,A}^{tot} = \begin{cases} \dfrac{12N}{(2+5N+5N^2)} \cdot \dfrac{S_a(T_1)}{\omega_1} & \text{for} \quad T_1 \le 0.5s \\ (0.44 \cdot T_1 + 0.78) \cdot \dfrac{12N}{(2+5N+5N^2)} \cdot \dfrac{S_a(T_1)}{\omega_1} & \text{for} \quad 0.5 < T_1 \le 5.0s \end{cases}$$

correction factor *M*

Palermo et al. 2017 (SDEE)

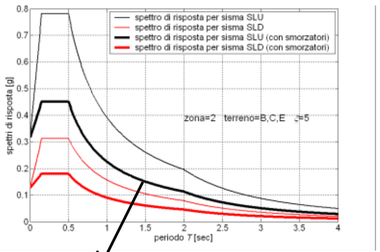
2. Equivalent Static Analysis for damped structures



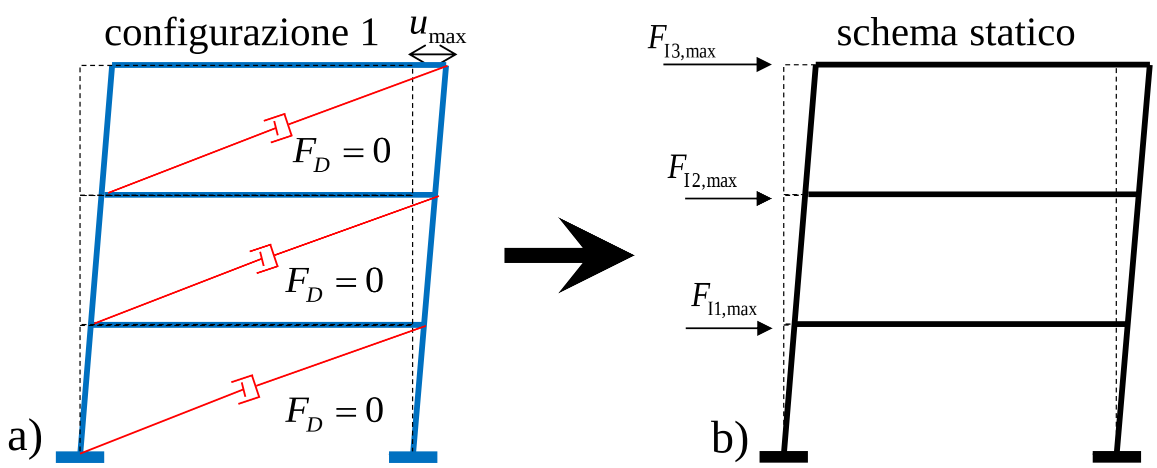
Maximum Lateral Displacement Configuration (ESA1)

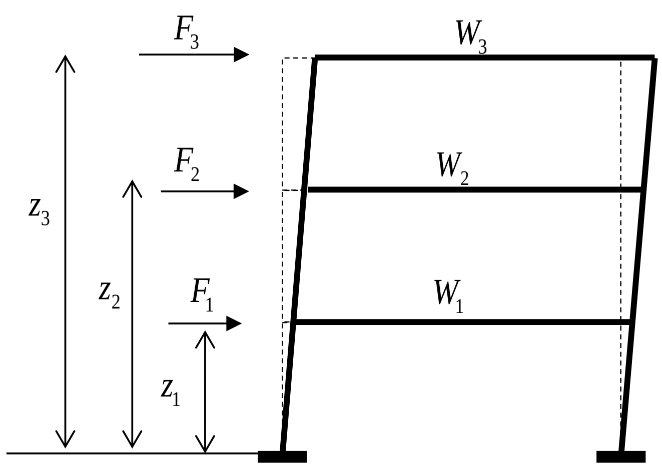
Maximum Velocity (Max Damper Force) Configuration (ESA2)

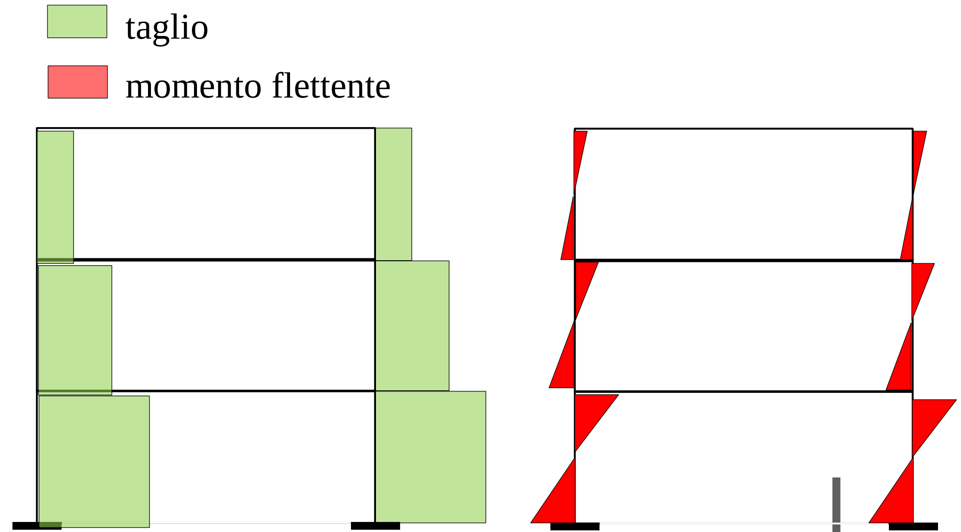
How to perform
Equivalent Static Analysis ESA1



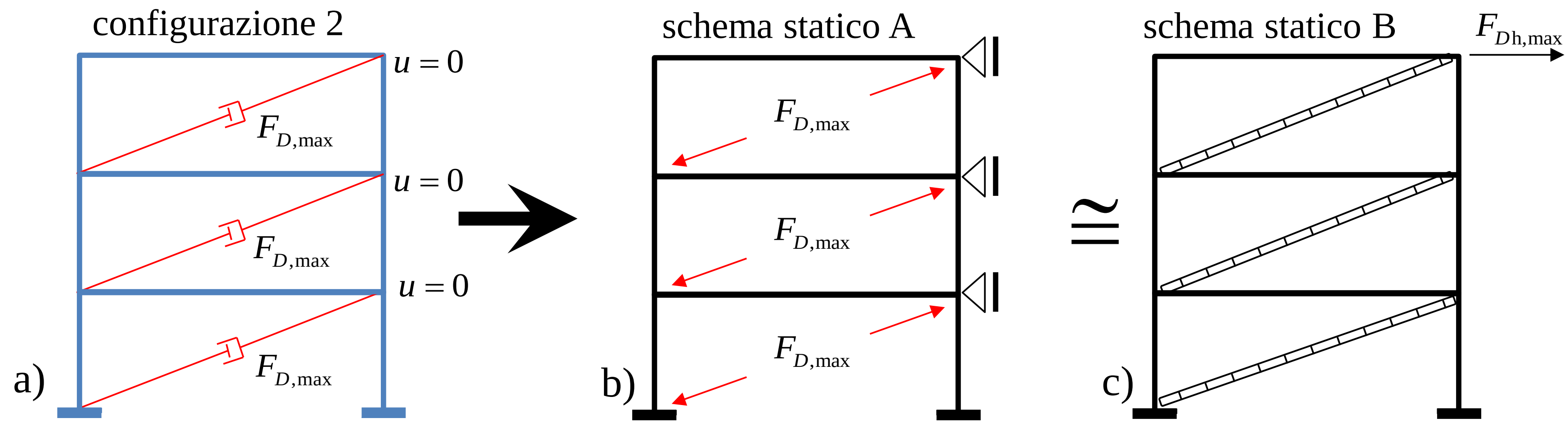
$$F_i = S_{e,\xi} \cdot \frac{W_i}{\sum_{i=1,2,\dots,N} W_i \cdot z_i}$$

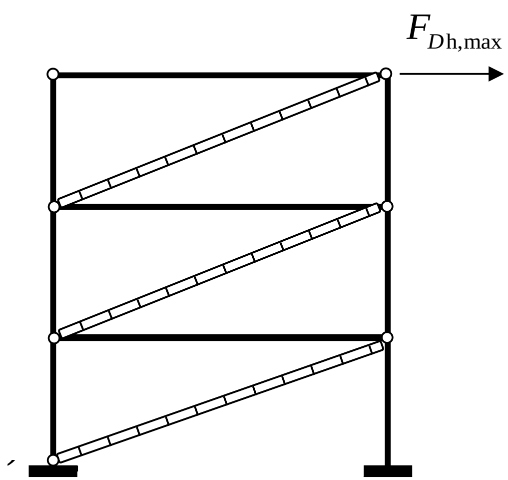


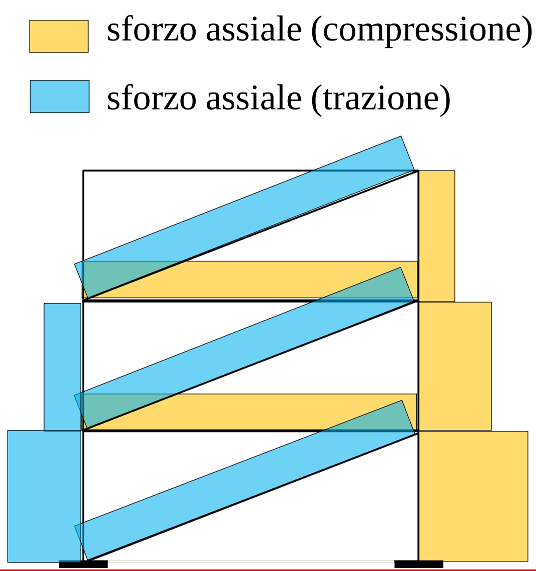


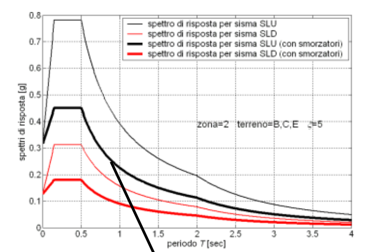


How to perform
Equivalent Static Analysis ESA2









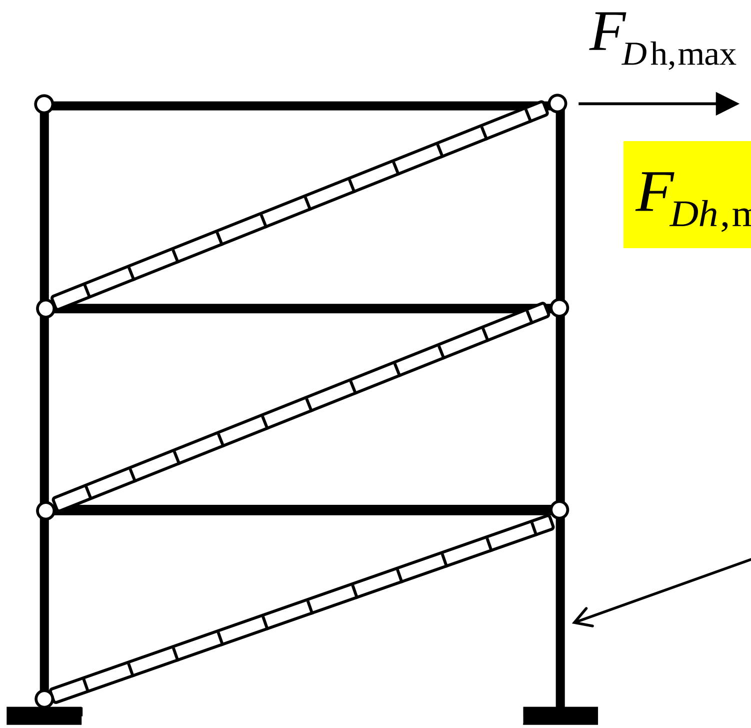
COLUMNS AXIAL FORCES DUE TO DAMPERS

$$P_{i,\max} = (N - i + 1) F_{d,v,\max} =$$

$$= (N - i + 1) \cdot 0.8^{1-\alpha} \cdot \frac{2 \cdot \xi \cdot W_{tot} \cdot S_{e,\xi}}{n} \tan\theta$$

How to perform Equivalent Static Analysis ESA2

ESA2

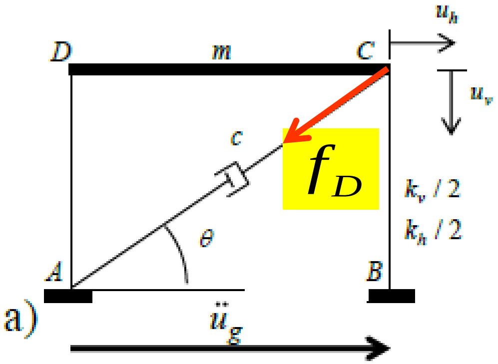


$$F_{Dh,\max,struttura} = 0.8^{1-\bar{\alpha}} \cdot 2 \cdot \bar{\xi}_{visc} \cdot m_{tot} \cdot S_e\left(T_1, \bar{\eta}\right)$$

objective:

N_{column}

2. The rationale behind ESA: the damped frame



dynamic equations

$$\begin{cases} f_{Ih} + f_{Dh} + f_{Sh} = 0 \\ f_{Iv} + f_{Dv} + f_{Sv} = 0 \end{cases}$$

two coupled equations (due to the damper)

$$\begin{cases} m\ddot{u}_h + c_h\dot{u}_h + c_{hv}\dot{u}_v + k_h u_h = -m\ddot{u}_g(t) \\ m\ddot{u}_v + c_{hv}\dot{u}_h + c_v\dot{u}_v + k_v u_v = 0 \end{cases}$$

$k_v \gg k_h$

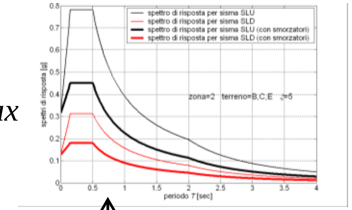
$\dot{u}_v$ can be neglected in the first equation

first equation (only in the horizontal d.o.f.)

$$m\ddot{u}_h + c_h\dot{u}_h + k_h u_h = -m\ddot{u}_g(t)$$

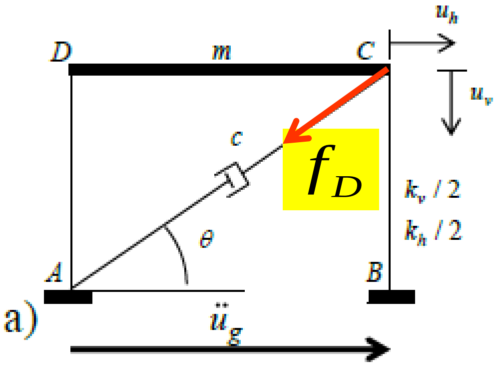
SDOF

$u_{h,max}$



$T_{(horizontal)}$

2. The rationale behind ESA: the damped frame



$u_h(t)$ or $u_{h,max}$ is obtained
and the damper force is obtained as:

$$c_h \dot{u}_h(t) = f_{Dh}(t) = f_D(t) \cdot \cos\theta$$

$$f_D(t) = \frac{c_h \dot{u}_h(t)}{\cos\theta}$$

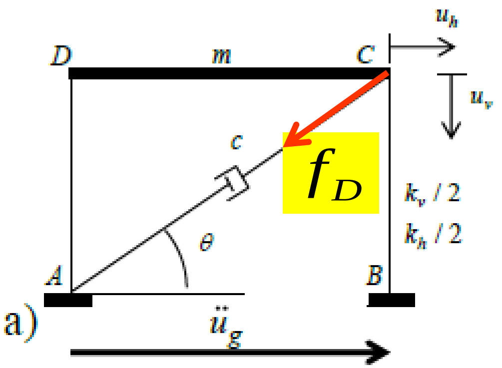
second equation

$$m\ddot{u}_v + c_v \dot{u}_v + k_v u_v = -c_{hv} \dot{u}_h = -f_{Dv}(t) = -f_D(t) \cdot \sin\theta$$

$$m\ddot{u}_v + c_v \dot{u}_v + k_v u_v = -f_{Dv}(t)$$

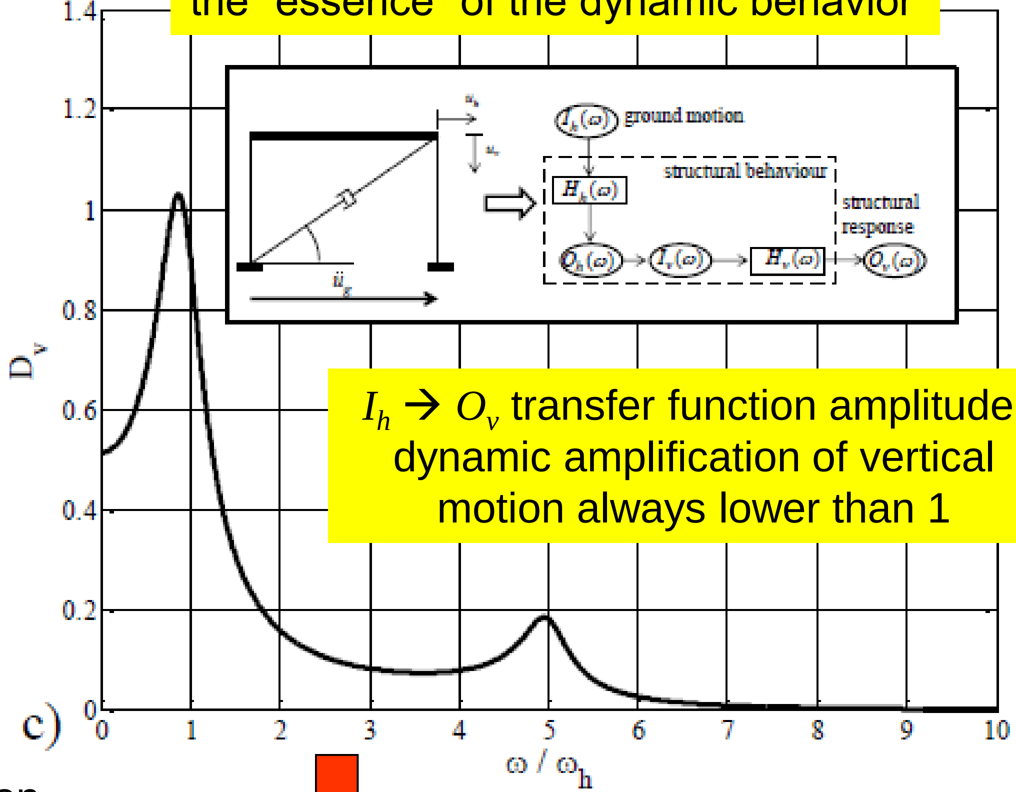
the input for the vertical d.o.f. is given by the damper force (coupled response)

2. The rationale behind ESA: the damped frame



$k_v \gg k_h$

the "essence" of the dynamic behavior



$I_h \rightarrow O_v$ transfer function amplitude: dynamic amplification of vertical motion always lower than 1

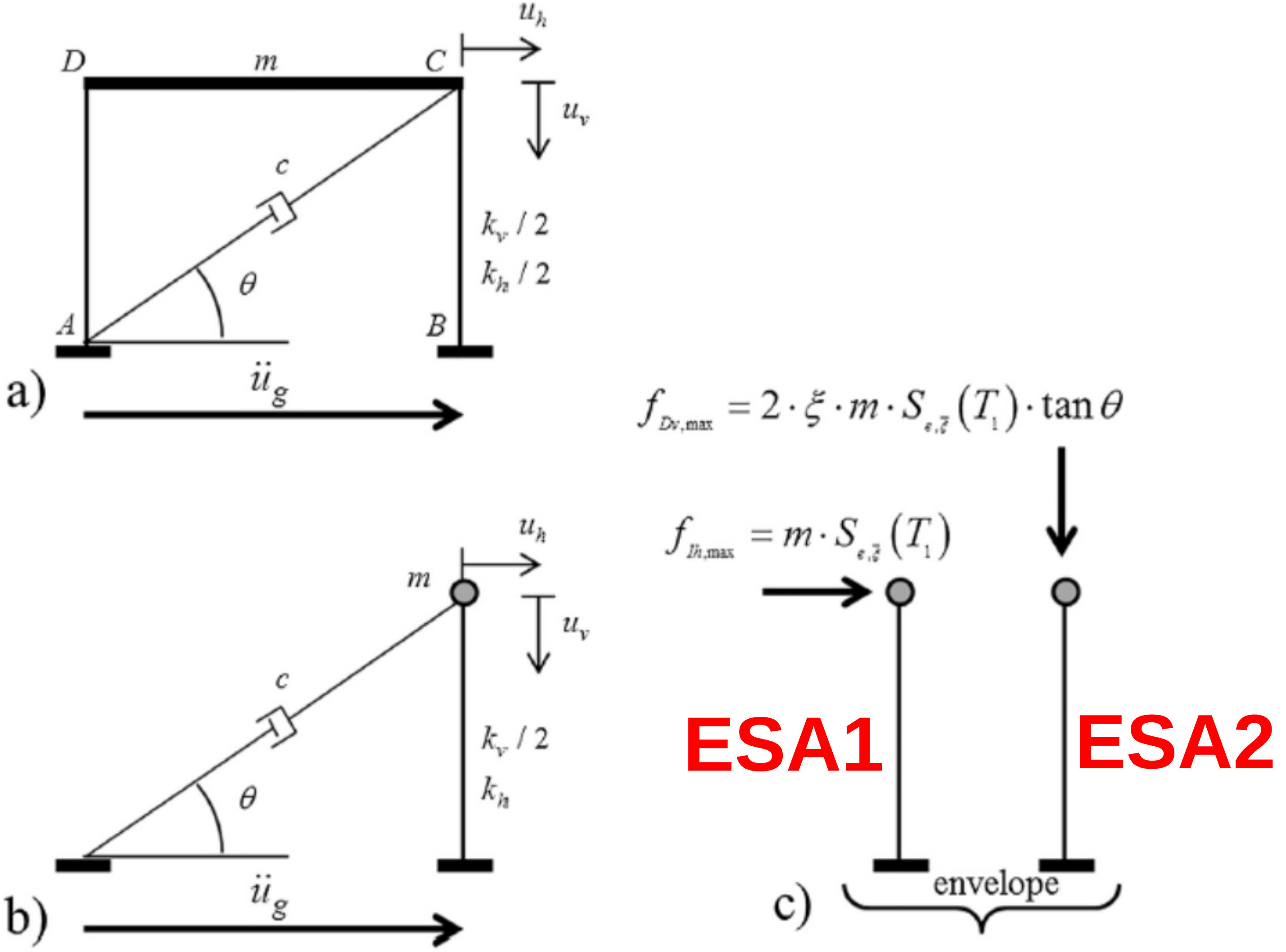
the second dynamic equation can be treated as a static equation

$$m\ddot{u}_v + c\dot{u}_v + k_v u_v = -f_{Dv}(t)$$

axial force in the column

$$k_v u_{v,\max} = -f_{Dv,\max}$$

2. The rationale behind ESA: the damped frame

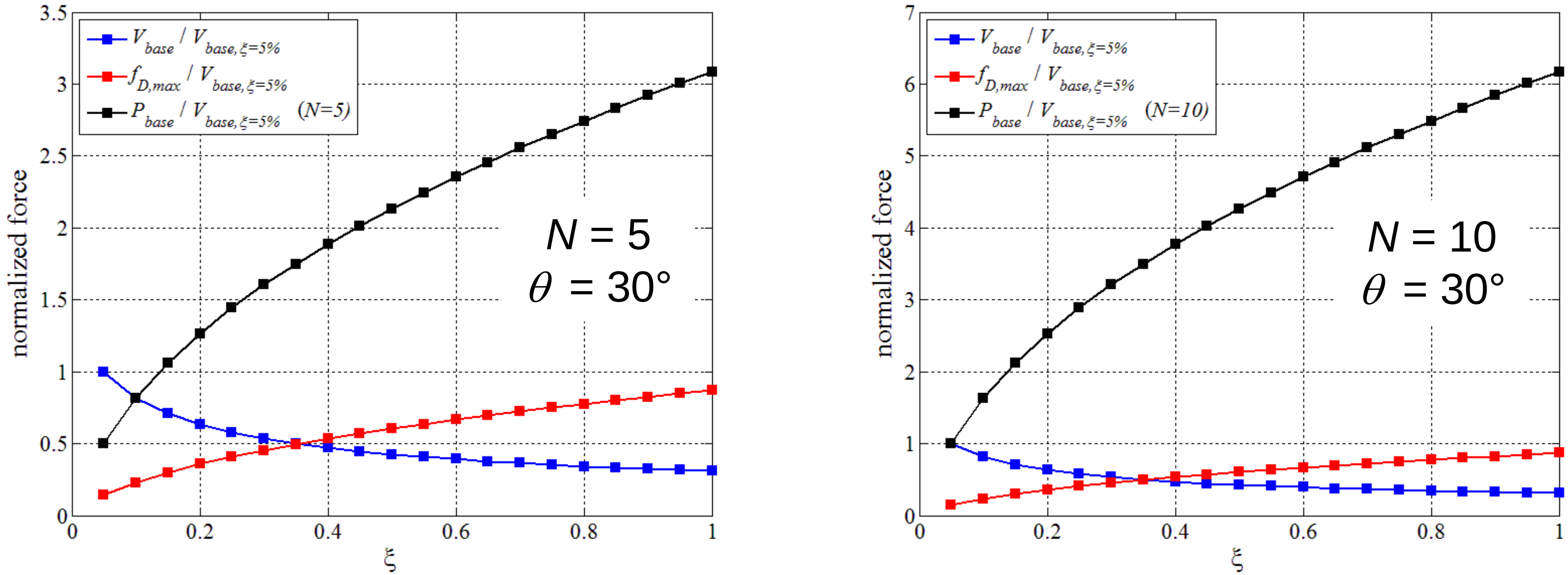


$$f_{Dv,\max} = 2 \cdot \xi \cdot m \cdot S_{e,\bar{\xi}}(T_1) \cdot \tan\theta$$

$$f_{Ih,\max} = m \cdot S_{e,\bar{\xi}}(T_1)$$

Palermo et al. 2018 (ES)

Axial forces in the columns due to dampers



REMEMBER: For tall buildings the axial forces due to dampers may become even larger than the axial forces due to static vertical loads

The direct five-step procedure for inter-storey dampers placement

DESIGN PROCESS

STEP 1: performance objectives ⇨ $\bar{\eta} \rightarrow \bar{\xi}$

$$\bar{\eta} = \sqrt{\frac{10}{5+\bar{\xi}}}$$

STEP 2: linear dan

$$v_{\max} = \frac{S_{e,\xi}(T_1)}{\omega_1} \cdot \frac{2}{(N+1)} \cdot \cos\theta$$

$$c_L = \bar{\xi} \cdot \omega_1 \cdot m_{tot} \cdot \left(\frac{N+1}{n}\right)$$

$$v_{\max} = \frac{S_{e,\xi}(T_1)}{\omega_1} \cdot \frac{2}{(N+1)} \cdot \cos\theta$$

STEP 3: ~~linear TH~~ response

analytical estimations

$$F_{D\max} = \frac{2 \cdot \xi \cdot m_{tot} \cdot S_{e,\xi}(T_1)}{\cos\theta}$$

$$F_{D\max} = \frac{2 \cdot \xi \cdot m_{tot} \cdot S_{e,\xi}(T_1)}{\cos\theta}$$

STEP 4: non-linear

$$c_{NL} = c_L \cdot (\chi \cdot v_{\max})^{1-\alpha}$$

k_{oil}

VERIFICATION

STEP 5:

$$P_{i,\max} = (N-i+1) \cdot 0.8^{1-\alpha} \cdot \frac{2 \cdot \xi \cdot m_{tot} \cdot S_{e,\xi}}{n} \cdot \tan\theta$$

or

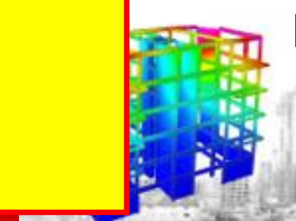
*ESA*2 procedure

$) \cdot 0.8^{1-\alpha} \cdot \frac{2 \cdot \xi \cdot m_{tot} \cdot S_{e,\xi}}{n} \cdot \tan\theta$

or

*ESA*2 procedure

FEM



“Direct five-step procedure”

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ELSEVIER

A “direct five-step procedure” for the preliminary seismic design of buildings with added viscous dampers

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A B S T R A C T

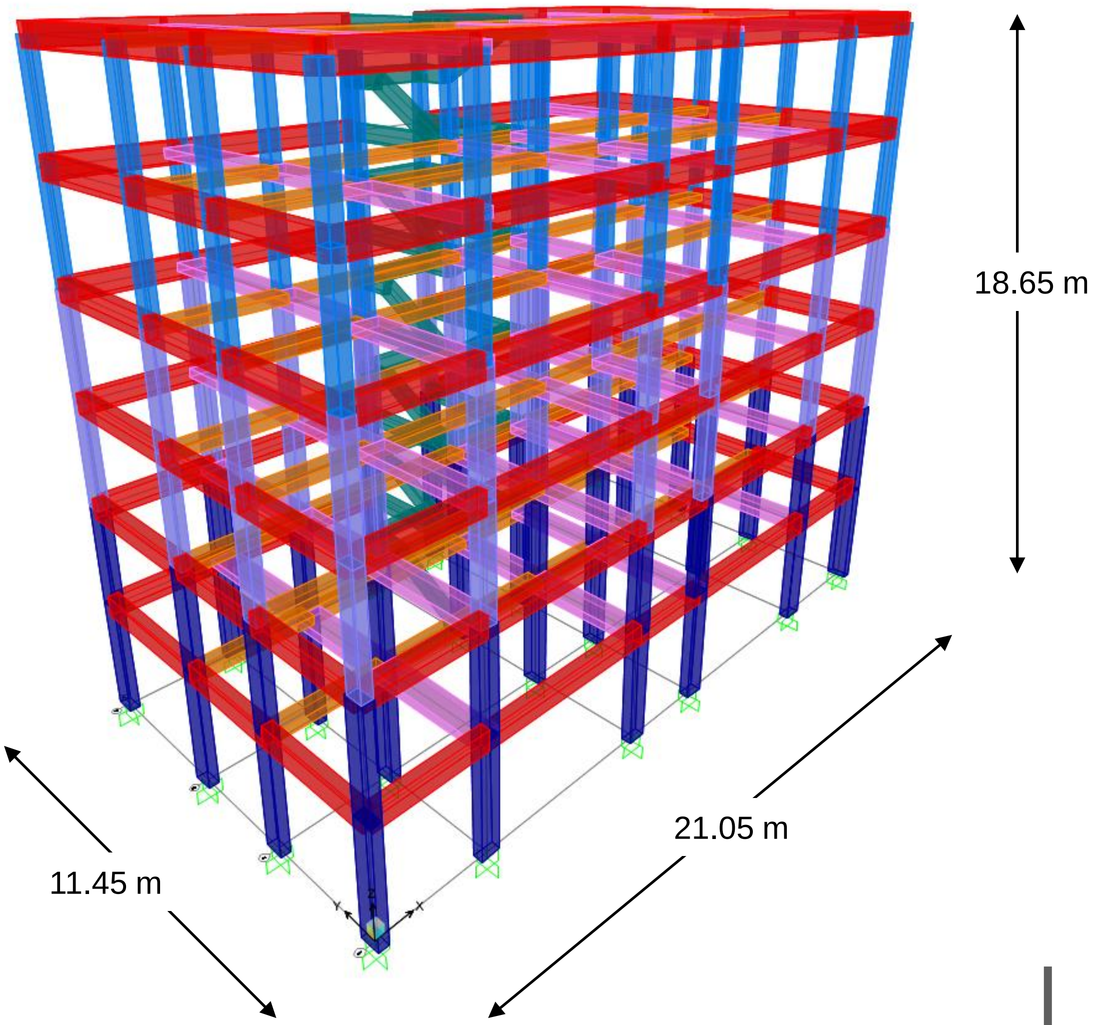
In the present work a direct procedure for the preliminary seismic design of building structures with added dampers is described which represents the simplification of the so-called “five-step procedure” originally developed in 2010 by some of the authors. The procedure is applicable to yielding frame structures with a generic along-the-height distribution of inter-storey viscous dampers. It is aimed at guiding the structural engineer through the sizing of both viscous dampers and structural elements making use of an equivalent static analysis approach. First, the peak structural response under earthquake excitation is reduced by imposing an overall reduction factor accounting for both the ductility demand and the viscous damping provided by the added dampers. Second, linear damping coefficients are calculated in order to reduce the structural response according to the selected target damping ratio. Then, analytical formulas allow the estimation of peak velocities and forces in the dissipative devices, and an energy criterion is used to identify the non-linear mechanical characteristics of the actual manufactured viscous dampers. Finally, the internal actions in the structural elements are estimated through the envelope of two equivalent static analyses (ESA). At this initial stage of the research, the procedure appears suitable for the preliminary design phase, while correction factors for the higher modes contributions need to be applied to improve its accuracy, especially for high-rise buildings. A numerical verification of the final behaviour of the system by means of non-linear time-history analyses is recommended. An applicative example is finally developed to highlight the soundness of the procedure.

Applicative example

The case study

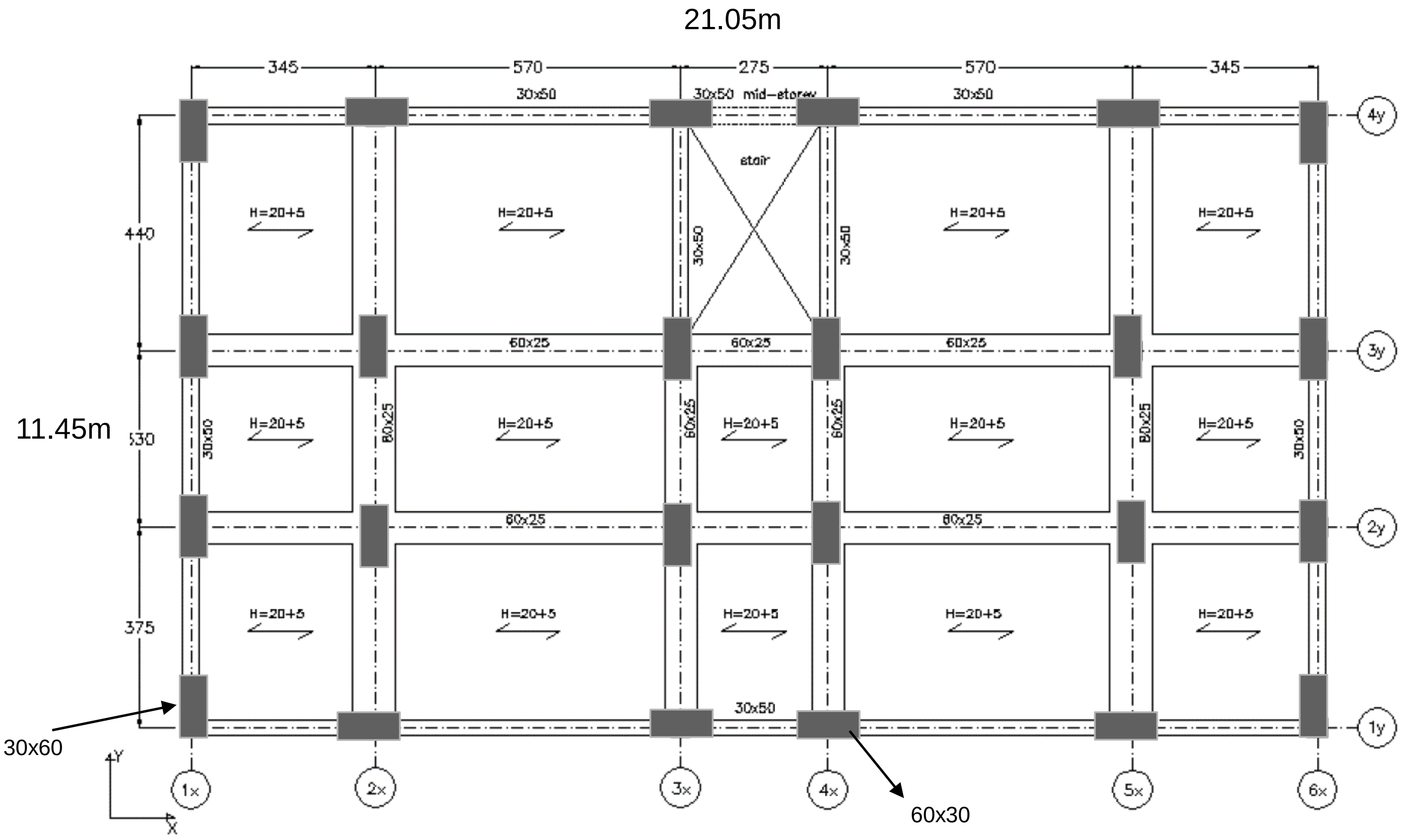
NEW DESIGN

Reinforced-Concrete
6-storey building
supposed
to be located in L'Aquila
(Central Italy)



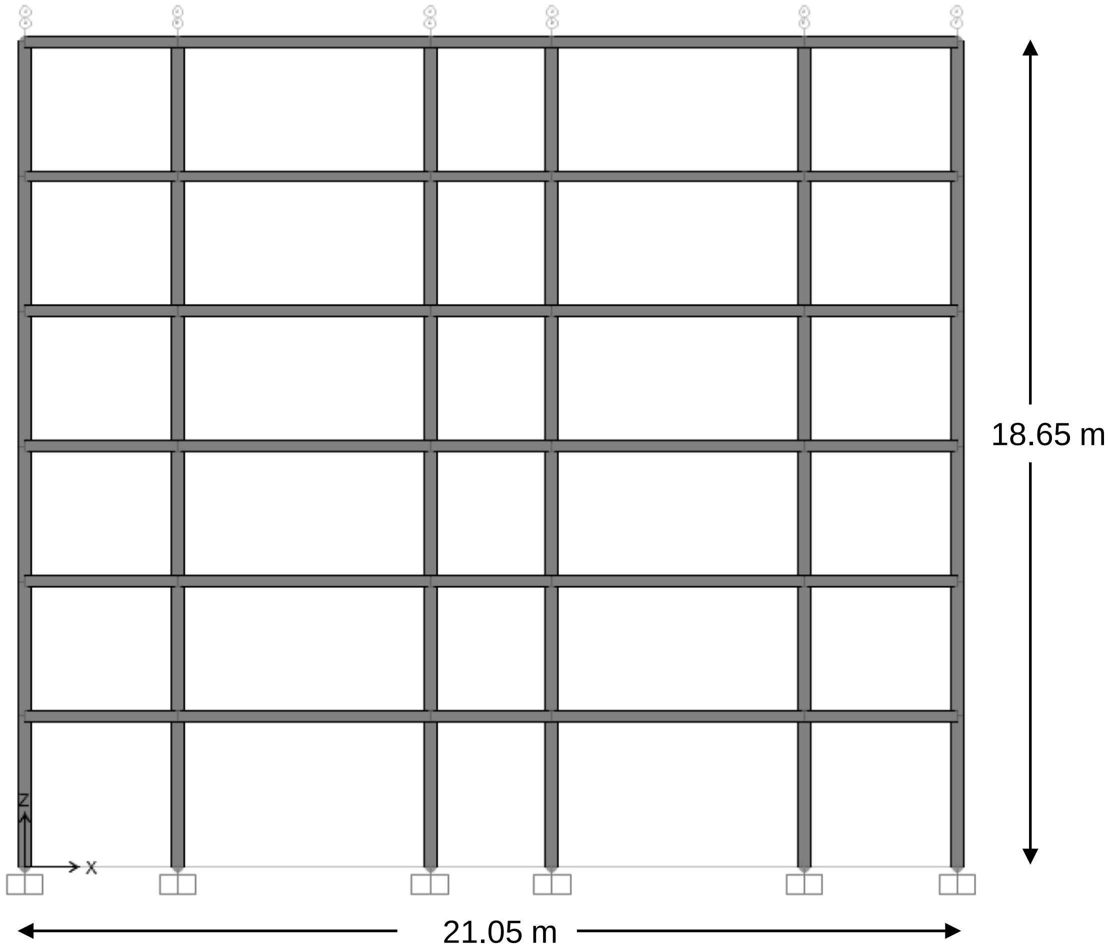
The case study

Ground floor:



The case study

Frame along X direction:



The case study

Load analysis:

Loads	*From floor 1 to floor 5*	*Floor 6 (attic + roof)*
Dead	To be considered floor by floor	To be considered floor by floor
Permanent G1	3,0 [kN/m2]	3,0 [kN/m2]
Permanent G2	2,5 [kN/m2]	1,8 [kN/m2]
Imposed Loads Q	2,0 [kN/m2]	2,0 [kN/m2]
TOTAL in static conditions + Dead	7,5 [kN/m2]	6,8 [kN/m2]
TOTAL in seismic conditions + Dead	6,05 [kN/m2]	5,35 [kN/m2]

$$A_{floor} = 21.05\ \text{m} \cdot 11.45\ \text{m} = 241\ \text{m}^2$$

$$W_{floor} = p_{seismic,\,floor} \cdot A_{floor} = 6.05\ \frac{\text{kN}}{\text{m}^2} \cdot 241\ \text{m}^2 = 1458\ \text{kN}$$

$$W_{roof} = p_{seismic,roof} \cdot A_{floor} = 5.35\ \frac{\text{kN}}{\text{m}^2} \cdot 241\ \text{m}^2 = 1289\ \text{kN}$$

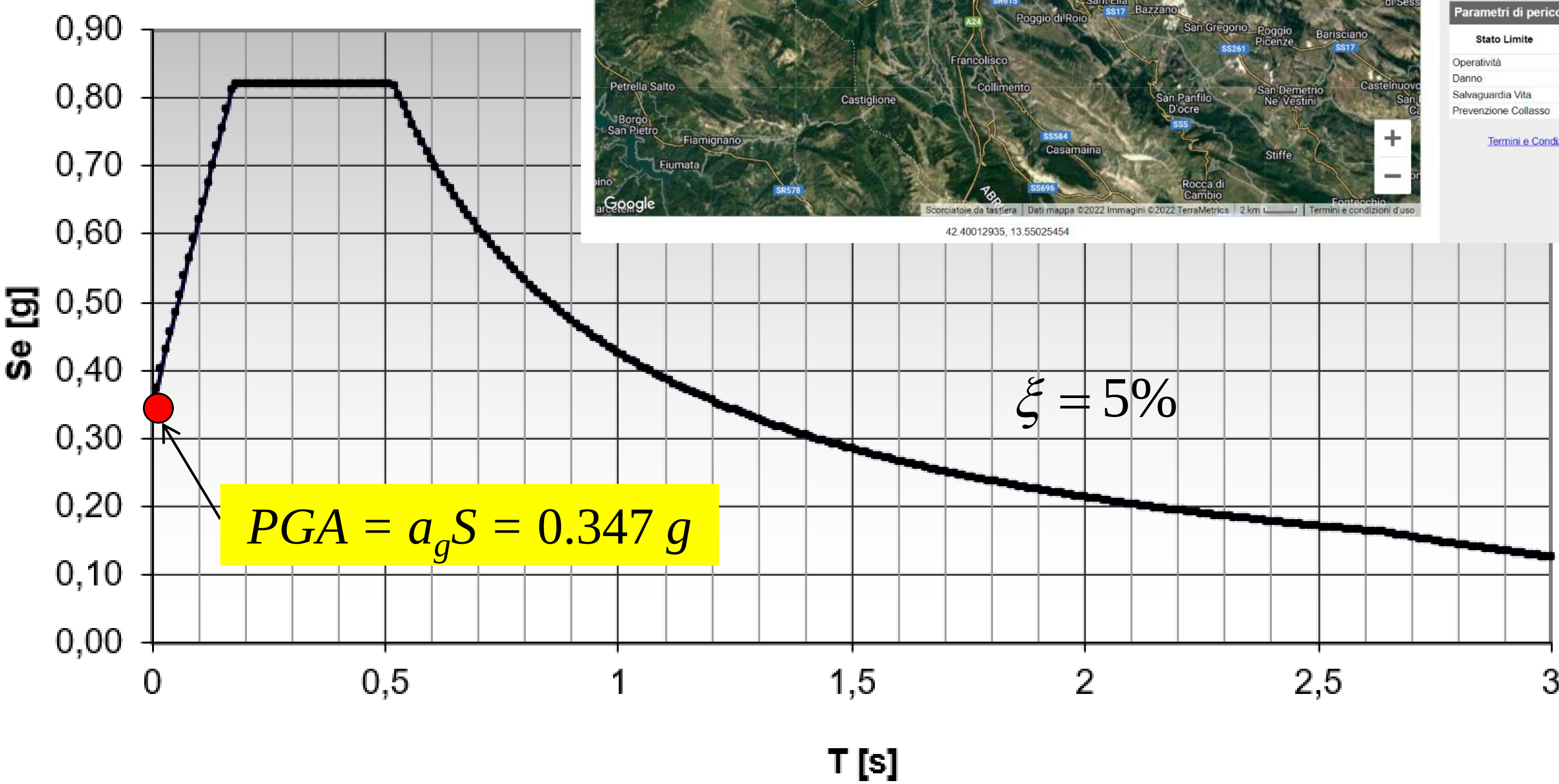
$$W_{perimeter\ walls} = 455\ \text{kN}$$

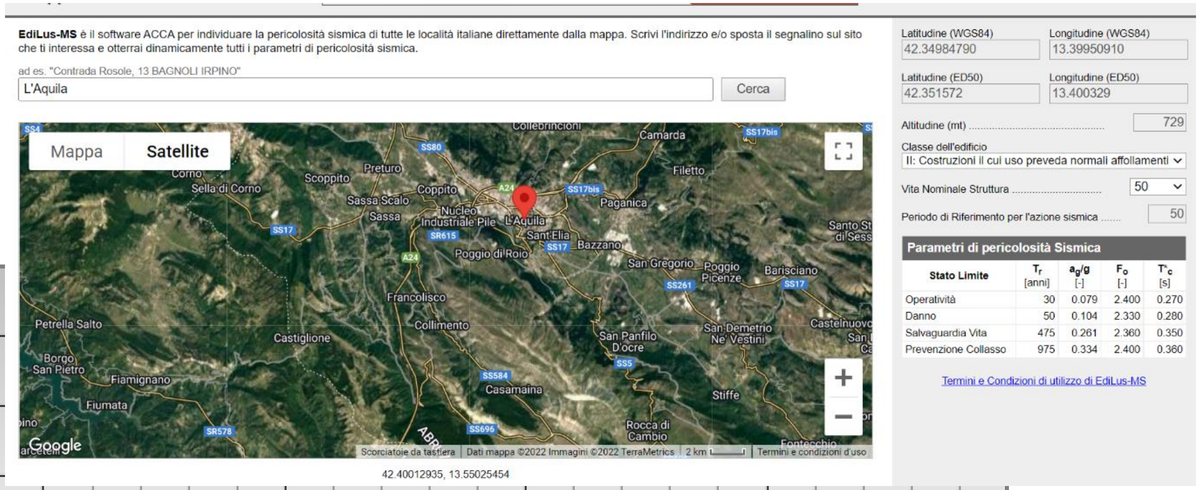
$$W_{beams\ and\ columns} = (V_{columns} + V_{beams}) \cdot \gamma_{cls} \simeq\ 1000\ \text{kN}$$

total weight in seismic conditions
W_{tot} = 16600 kN

The case study

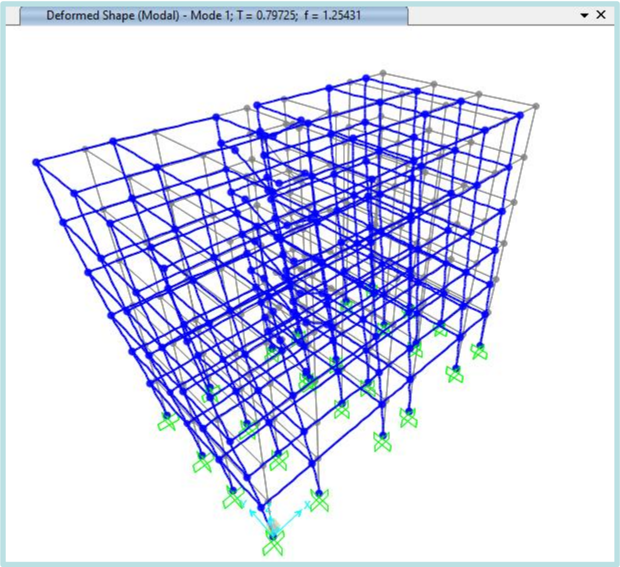
The horizontal pseudo-acceleration **elastic** response **spectrum:**

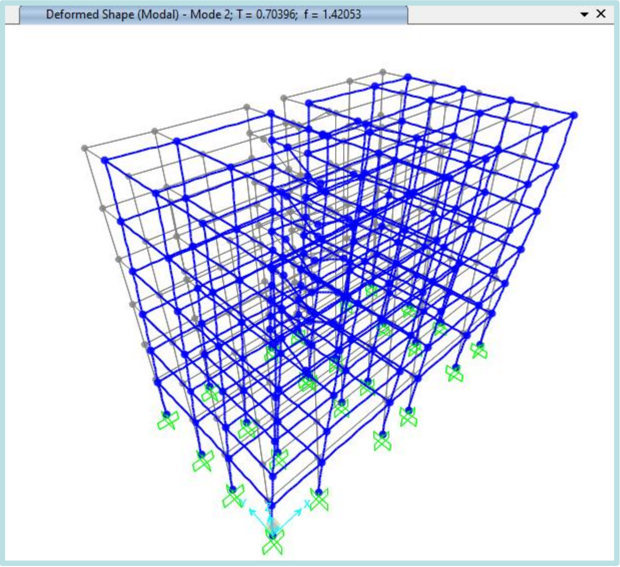


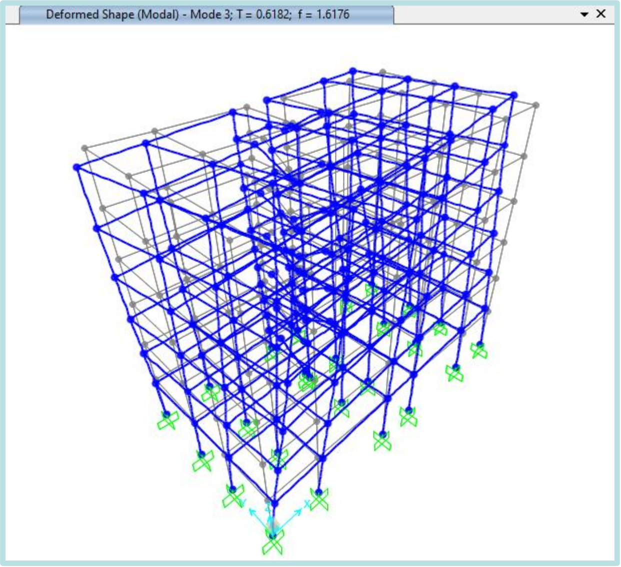


The case study

Modes of vibration:



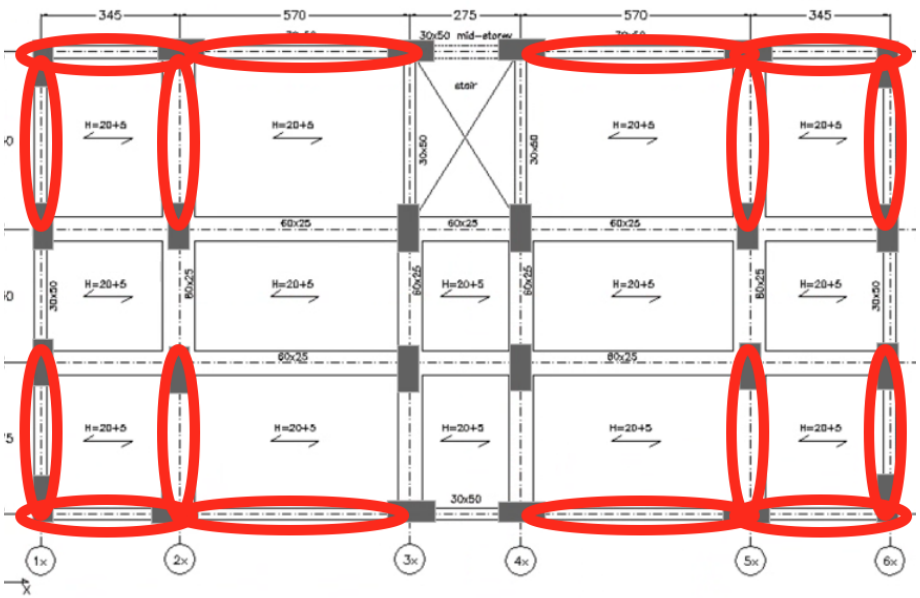


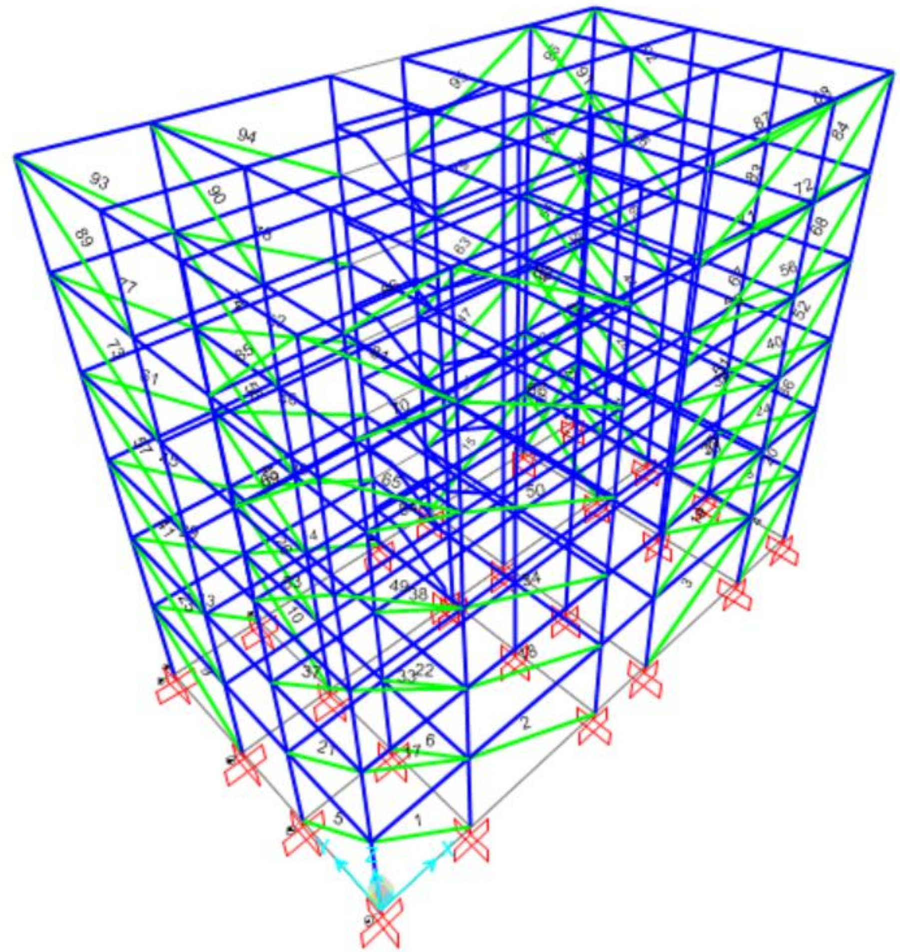


Mode 1	Mode 2	Mode 3
Translational along X	Translational along Y	Rotational
T = 0.797 s	T = 0.704 s	T = 0.618 s
$M_{\%,UX}$ = 81.4 %	$M_{\%,UY}$ = 80.3 %	$M_{\%,RZ}$ = 80.8 %

The case study

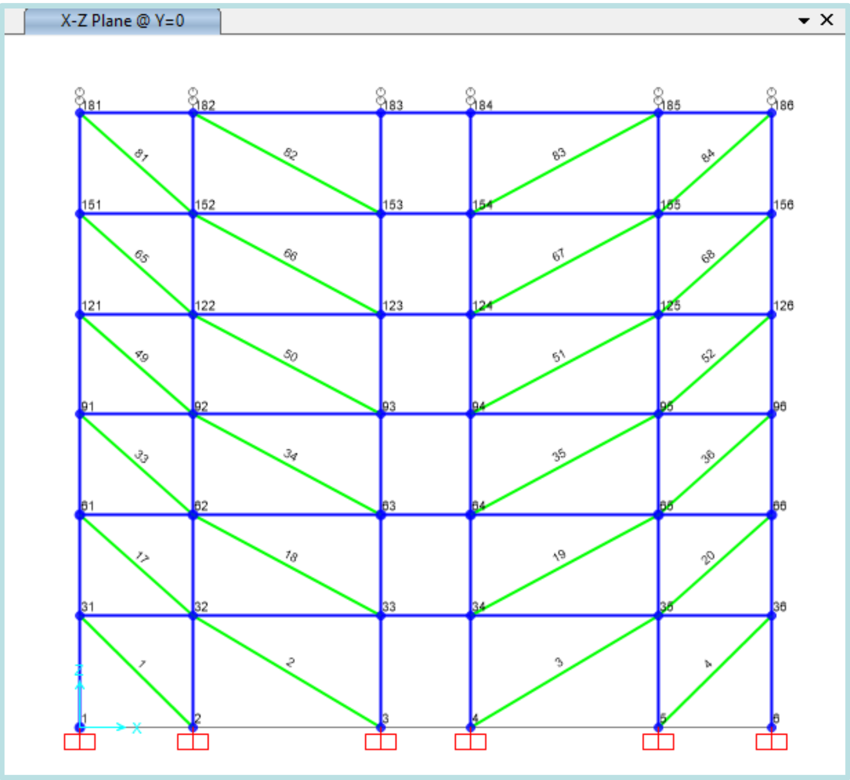
Dampers location:

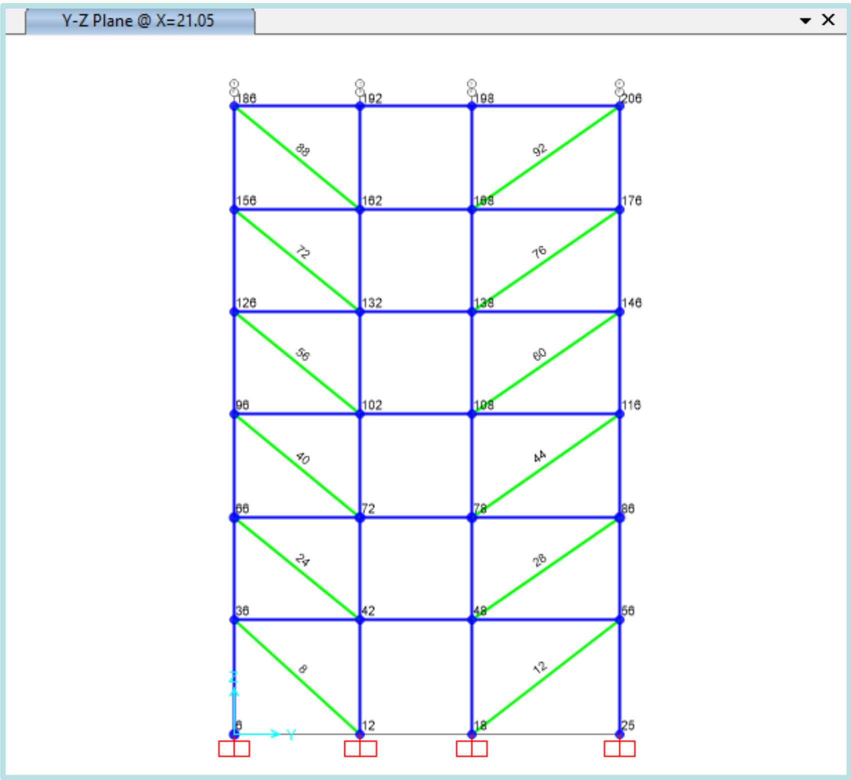




The case study

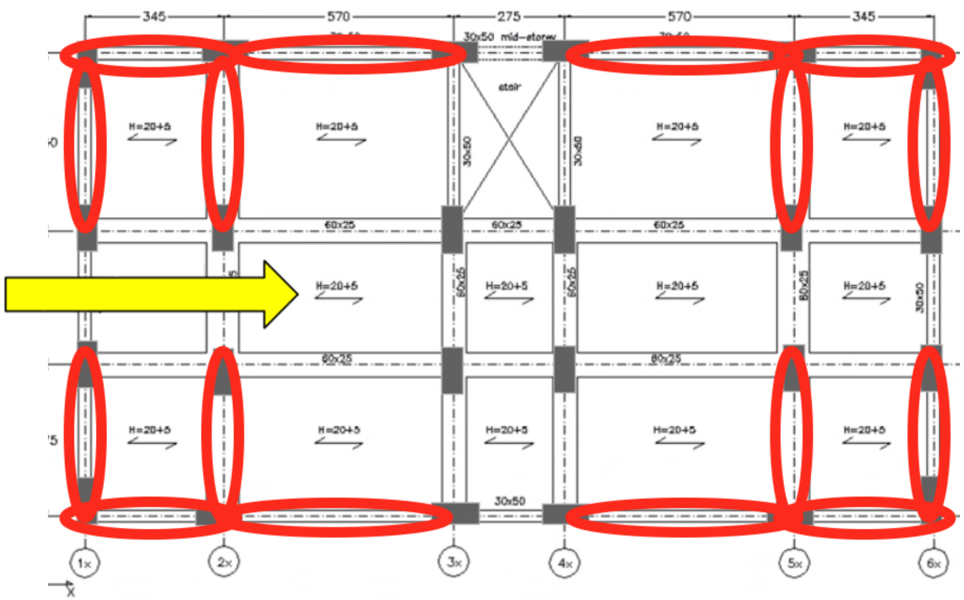
Dampers location:





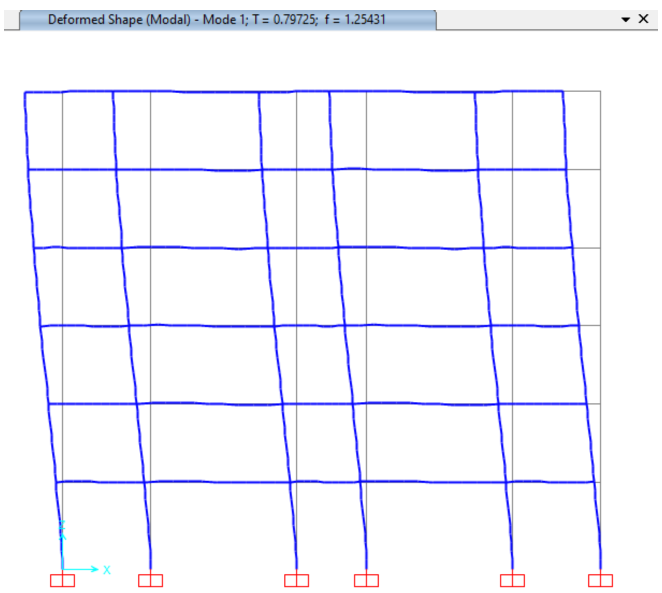
The direct five-step procedure applied to the case study

seismic action
along the X-direction



First mode
along the X-direction

$$T_{1,x} = 0.797\ s$$



Application - Step 1

STEP 1

through dampers

Target damping ratio: $\bar{\xi}_{tot} = \xi_{\text{intr}} + \bar{\xi}_{visc} = 5\% + ? = ?$

Corresponding response reduction factor: $\bar{\eta} = \sqrt{\dfrac{10}{5+\bar{\xi}_{tot}}} = \sqrt{\dfrac{10}{5+?}} = ?$

Fundamental period along the considered (longitudinal) direction: $T_1 = 0.797s$

How can we define the size of the needed dampers in a «preliminary quick design / dimensioning phase»?

Application - Step 1

$\bar{\eta}$

$$\bar{\eta} = \frac{M_{Ed,\xi}}{M_{Ed,\xi=5\%}} = \frac{Demand\ reduced\ by\ dampers}{Demand} = \frac{Capacity}{Demand}$$

Elastic Demand:

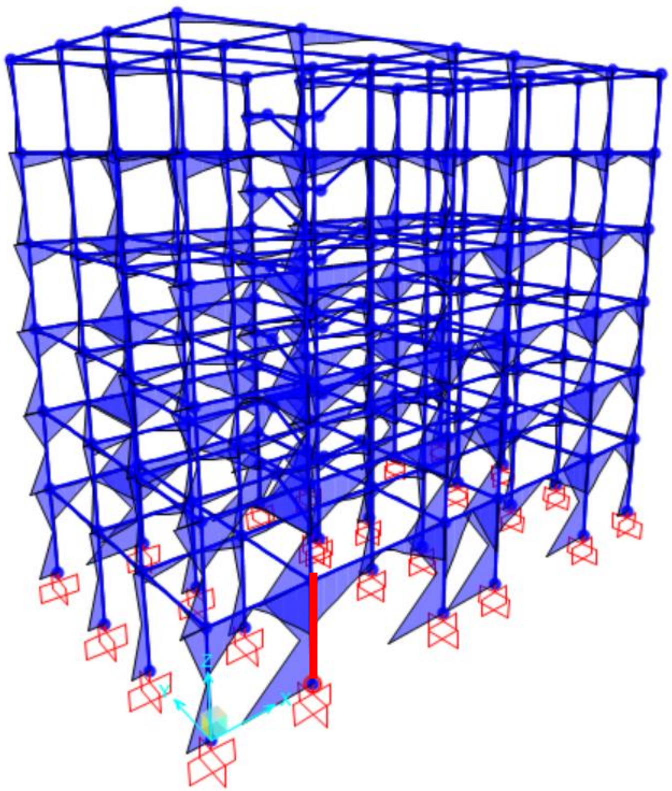
The bending moment generated by an elastic response spectrum analysis at the base of the **most stressed ground floor column** is:

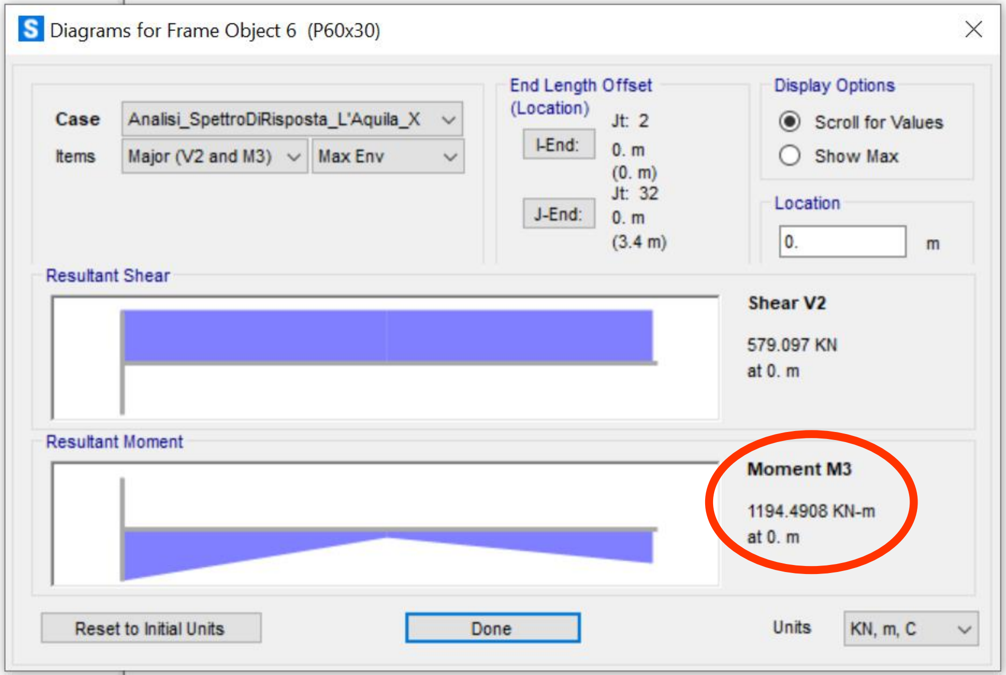
$$M_{Ed,\xi=5\%} = 1194\ \text{kNm}$$

Application - Step 1

$\bar{\eta}$

$$\bar{\eta} = \frac{M_{Ed,\xi}}{M_{Ed,\xi=5\%}} = \frac{Demand\ \ reduced\ \ by\ \ dampers}{Demand} = \frac{Capacity}{Demand}$$





Application - Step 1

$\bar{\eta}$

$$\bar{\eta} = \frac{M_{Ed,\xi}}{M_{Ed,\xi=5\%}} = \frac{Demand\ \ reduced\ \ by\ \ dampers}{Demand} = \frac{Capacity}{Demand}$$

Capacity:

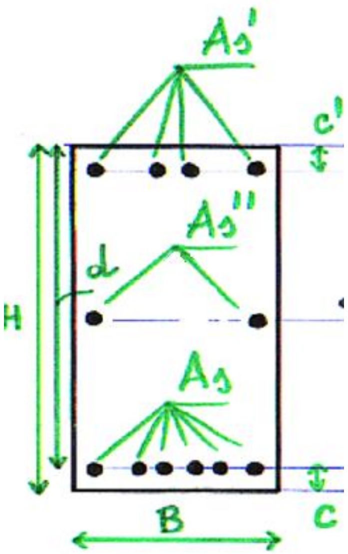
The yielding bending moment of the base cross-section of the **most stressed ground floor column** is, for a reasonable amount of reinforcement bars and for the axial force corresponding to the seismic condition (*N* = 686 kN) is:

$$M_{Rd} = 510 \text{ kNm}$$

Application - Step 1

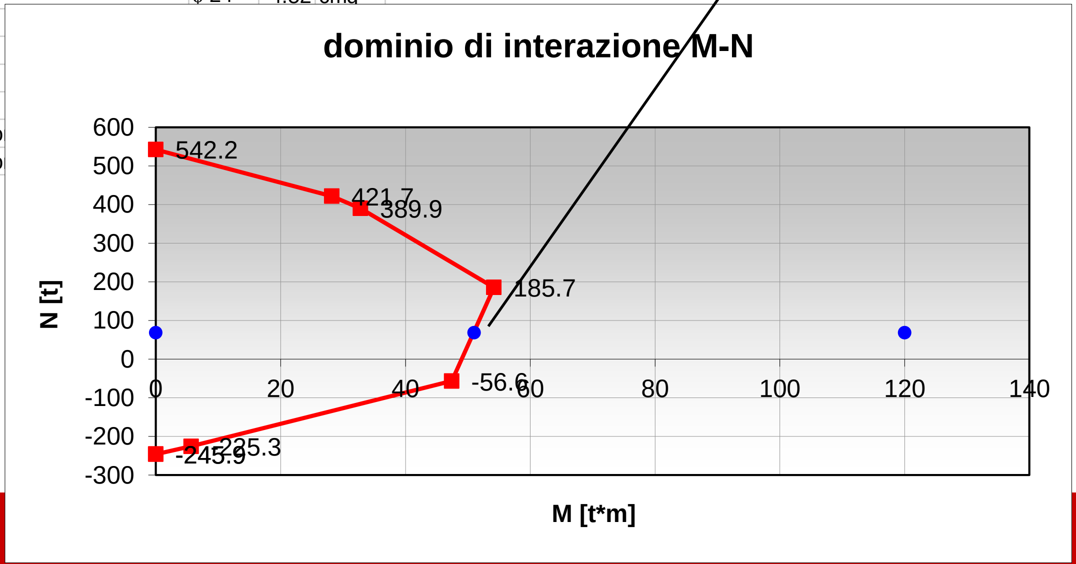
$\bar{\eta}$

$$\bar{\eta} = \frac{M_{Ed,\xi}}{M_{Ed,\xi=5\%}} = \frac{Demand\ \ reduced\ \ by\ \ dampers}{Demand} = \frac{Capacity}{Demand}$$



DATI DELLA SEZIONE			(in rosso i dati di input)			
B =	30	cm	base	ϕ 6 =	0.28	cmq
H =	60	cm	altezza	ϕ 8 =	0.50	cmq
c =	4	cm	copriferro ferro teso	ϕ 10 =	0.79	cmq
c' =	4	cm	copriferro ferro compresso	ϕ 12 =	1.13	cmq
d = H-c =	56	cm	altezza utile	ϕ 14 =	1.54	cmq
As =	15.71	cmq	area ferro teso	ϕ 16 =	2.01	cmq
As' =	15.71	cmq	area ferro compresso	ϕ 18 =	2.54	cmq
As'' =	31.42	cmq	area ferro centrale	ϕ 20 =	3.14	cmq
Astot =	62.83	cmq	area totale	ϕ 22 =	3.80	cmq
fyk =	4500	kg/cmq		ϕ 24 =	4.52	cmq
Es =	2100000	kg/cmq				
Rck =	350	kg/cmq				
fyd =	3913	kg/cmq				
fcd =	165	kg/cmq				
εsu =	6.75E-02		deformazio			
εse =	1.86E-03		deformazio			

M [t*m]	N [t]
0	68.6
120	68.6
51	68.6



Application - Step 1

$\bar{\eta}$

$$\bar{\eta} = \frac{M_{Ed,\xi}}{M_{Ed,\xi=5\%}} = \frac{Demand\ reduced\ by\ dampers}{Demand} = \frac{Capacity}{Demand}$$

$$\bar{\eta} = \frac{M_{Ed,\xi}}{M_{Ed,\xi=5\%}} = \frac{M_{Rd}}{M_{Ed,\xi=5\%}} = \frac{510\ \text{kNm}}{1194\ \text{kNm}} = 0.43$$

Application - Step 1

$\bar{\eta}$

$$\bar{\eta} = \frac{M_{Ed,\xi}}{M_{Ed,\xi=5\%}} = \frac{M_{Rd}}{M_{Ed,\xi=5\%}} = \frac{510\ \text{kNm}}{1194\ \text{kNm}} = 0.43$$

Now, two possibilities corresponding to two different strategies:

- **To rely on the behaviour factor** (dissipation due to damage in the structural elements)
  ⇒ lower immediate costs, but lower performances (and also possible higher future costs in the event of a strong ground motion)

- **To introduce dampers** (dissipation with no damage in the structural elements)
  ⇒ higher immediate costs, but higher performances (and, most likely, no future costs in the event of a strong ground motion). Also, plastic resources (which are anyway there) are saved for a very very strong earthquake

Application - Step 1

$\bar{\eta}$

$$\bar{\eta} = \frac{M_{Ed,\xi}}{M_{Ed,\xi=5\%}} = \frac{M_{Rd}}{M_{Ed,\xi=5\%}} = \frac{510 \text{ kNm}}{1194 \text{ kNm}} = 0.43$$

Let's go for **dampers**.

Let's assume: $\bar{\eta} = 0.50$

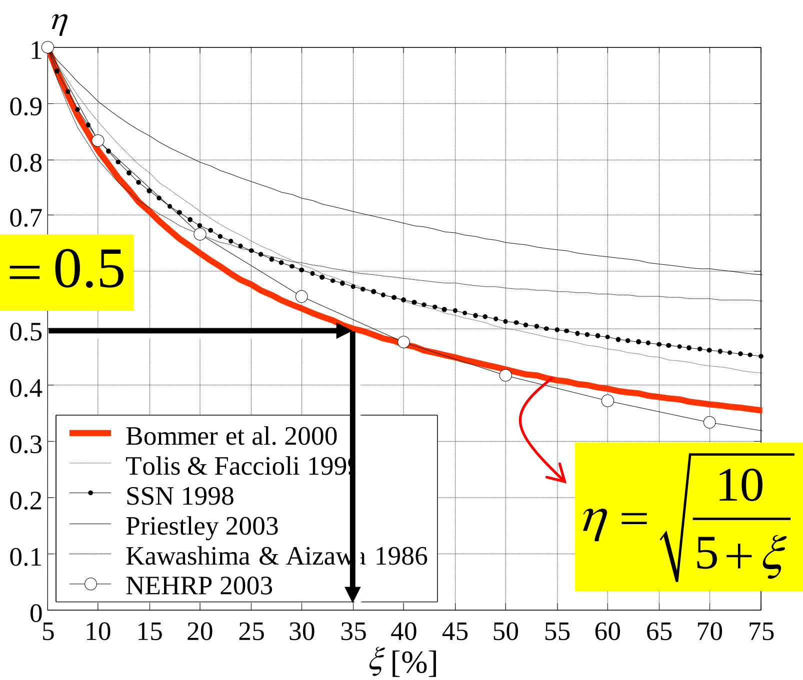
even if it could be not enough …

Application - Step 1

$\bar{\eta} \longrightarrow \bar{\xi}$

Some available formulations to relate $\bar{\eta} \longrightarrow \bar{\xi}$

Cardone, Dolce, Rivelli (ANIDIS 2007)

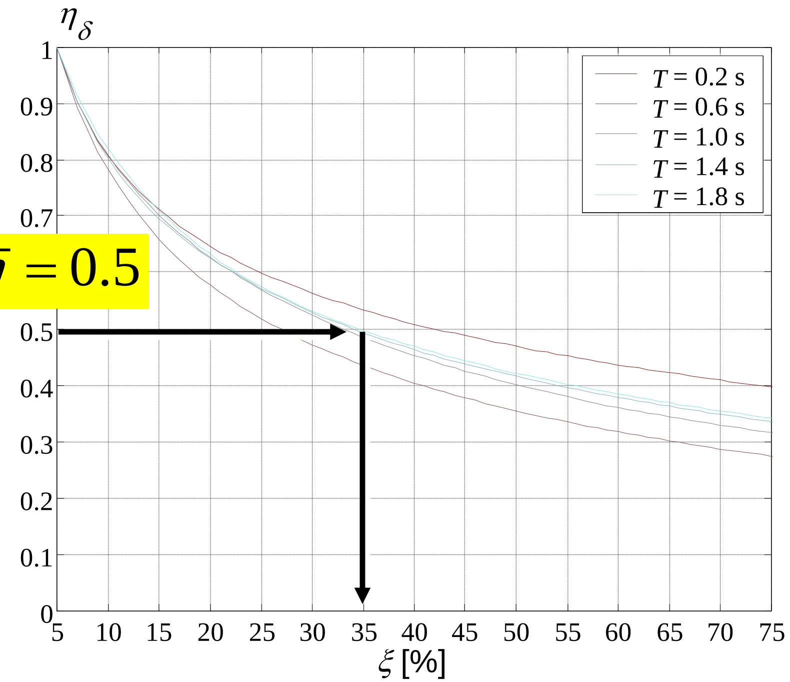


$\bar{\eta} = 0.5$

$$\eta = \sqrt{\frac{10}{5+\xi}}$$

$\bar{\xi} = 35\%$

mean SDOF response from T-H analyses



$\bar{\eta} = 0.5$

$\bar{\xi} = 35\%$

$$\bar{\xi} = \frac{10}{\bar{\eta}^2} - 5 = \frac{10}{0.5^2} - 5 = 35$$

Application - Step 1

STEP 1

through dampers

Assumed target damping ratio: $\bar{\xi}_{tot} = \xi_{intr} + \bar{\xi}_{visc} = 5\% + 30\% = 35\%$

Corresponding response reduction factor: $\bar{\eta} = \sqrt{\frac{10}{5+\bar{\xi}_{tot}}} = \sqrt{\frac{10}{5+35}} = 0.50$

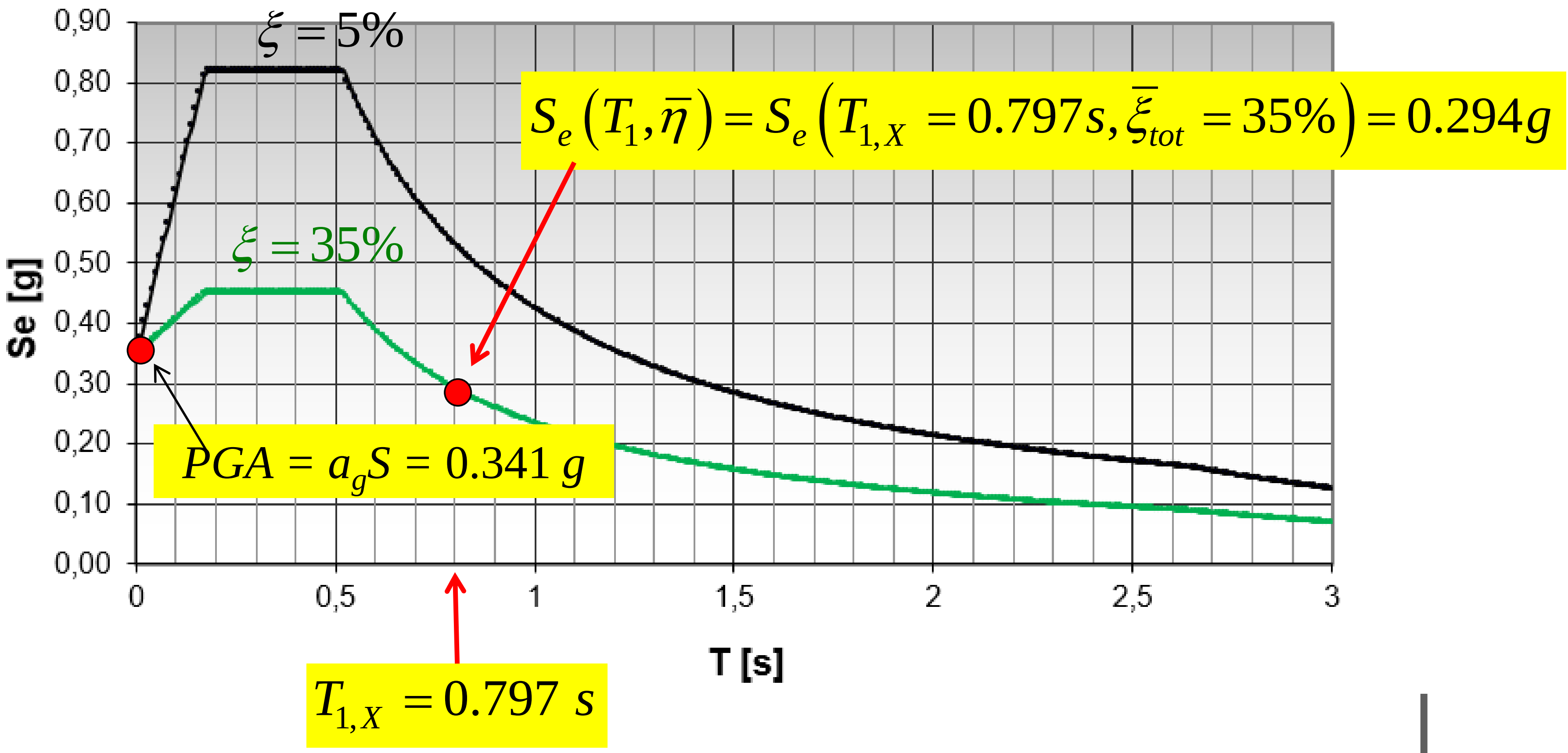
Fundamental period along the considered (longitudinal) direction: $T_1 = 0.797s$

Spectral acceleration:

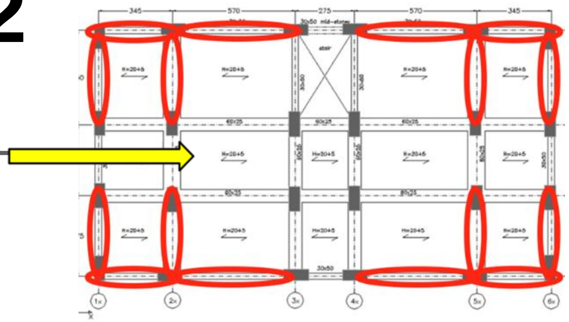
$$S_e(T_1, \bar{\eta}) = a_g \cdot S \cdot \bar{\eta} \cdot F_o \cdot \left(\frac{T_c}{T_1}\right) =$$

$$= 0.261g \cdot 1.333 \cdot 0.50 \cdot 2.360 \cdot \frac{0.520}{0.797} \cong 0.294g$$

Application – Step 1



Application - Step 2



STEP 2

Number of dampers per floor placed along the X-direction: $n = 8$

Average damper inclination with respect to the horizontal line: $\theta = 36°$

Linear damping coefficient:

$$\overline{c_L} = \overline{\xi}_{visc} \cdot \omega_1 \cdot \frac{W_{tot}}{g} \cdot \left(\frac{N+1}{n} \right) \frac{1}{\cos^2 \theta} =$$

$$= 0.30 \cdot \frac{2\pi}{0.797\text{s}} \cdot \frac{16600\ \text{kN}}{9.81\ \frac{\text{m}}{\text{s}^2}} \cdot \left(\frac{6+1}{8} \right) \cdot \frac{1}{\cos^2 36°} \cong 5384 \frac{\text{kN} \cdot \text{s}}{\text{m}}$$

Application - Step 3

STEP 3

Peak damper **velocity** estimation for the equivalent linear damper:

$$v_{\max} = \frac{S_e\left(T_1, \bar{\eta}\right)}{\omega_1} \cdot \frac{2}{N+1} \cdot \cos\theta =$$

$$= \frac{0.294 \cdot 9.81 \ \frac{\mathrm{m}}{\mathrm{s}^2}}{\left(\frac{2\pi}{0.797\mathrm{s}}\right)} \cdot \frac{2}{6+1} \cdot \cos 36° \cong 0.084 \frac{\mathrm{m}}{\mathrm{s}}$$

Application - Step 3

STEP 3

Peak damper **force** estimation for the single equivalent linear damper:

$$F_{L,\max} = 2 \cdot \overline{\xi}_{visc} \cdot \frac{W}{g} \cdot \frac{S_e\left(T_1, \overline{\eta}\right)}{n \cdot \cos\theta} =$$

$$= 2 \cdot 0.30 \cdot \frac{16600\ \text{kN}}{g} \cdot \frac{0.294 g}{8 \cdot \cos 36°} \cong 454\ \text{kN}$$

Application - Step 3

STEP 3

Peak damper **stroke** estimation for the equivalent linear damper:

$$s_{\max} = \frac{S_e\left(T_1, \overline{\eta}\right)}{\omega_1^{\ 2}} \cdot \frac{2}{N+1} \cdot \cos\theta =$$

$$= \frac{0.294 \cdot 981 \ \dfrac{\text{cm}}{\text{s}^2}}{\left(\dfrac{2\pi}{0.797\text{s}}\right)^2} \cdot \frac{2}{6+1} \cdot \cos 36° \cong 1.07 \ \text{cm}$$

Application - Step 4

STEP 4

α-exponent of the commercial damper: $\alpha = 0.15$

Non-linear damping coefficient of the commercial damper:

$$\overline{c_{NL}} = \overline{c_L} \cdot \left(0.8 \cdot v_{\max}\right)^{1-\bar{\alpha}} =$$

$$= 5384 \frac{\mathrm{kN} \cdot \mathrm{s}}{\mathrm{m}} \cdot \left(0.8 \cdot 0.084 \frac{\mathrm{m}}{s}\right)^{1-0.15} \cong 544 \frac{\mathrm{kN} \cdot \mathrm{s}^{0.15}}{\mathrm{m}^{0.15}}$$

Minimum axial stiffness of the device (non-linear damper + supporting brace):

$$\overline{k_{axial}} \geq 10 \cdot \overline{c_L} \cdot \omega_1 =$$

$$= 10 \cdot 5384 \frac{\mathrm{kN} \cdot \mathrm{s}}{\mathrm{m}} \cdot \frac{2\pi}{0.797\mathrm{s}} = 4.24 \cdot 10^5 \frac{\mathrm{kN}}{\mathrm{m}} \longrightarrow 4.5 \cdot 10^5 \frac{\mathrm{kN}}{\mathrm{m}}$$

Application - Step 4

STEP 4

Peak damper **force** estimation for the single “non-linear” damper:

$$F_{NL,\max} = 0.8^{1-\bar{\alpha}} \cdot F_{L,\max} =$$
$$= 0.8^{1-0.15} \cdot 454 \text{ kN} \cong 375 \text{ kN}$$

Application - Step 5

STEP 5

Peak **axial force** in the base column (i=1):

$$P_{\mathrm{i,max}} = (N - i + 1) \cdot 0.8^{1-\bar{\alpha}} \cdot \frac{2 \cdot \bar{\xi}_{visc} \cdot m_{tot} \cdot S_e(T_1, \bar{\eta})}{n} \cdot \tan\theta$$

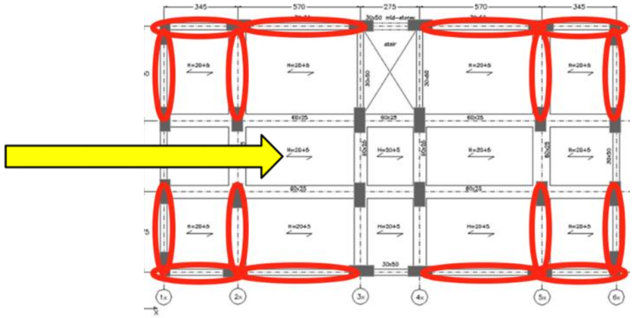
$$i = 1$$

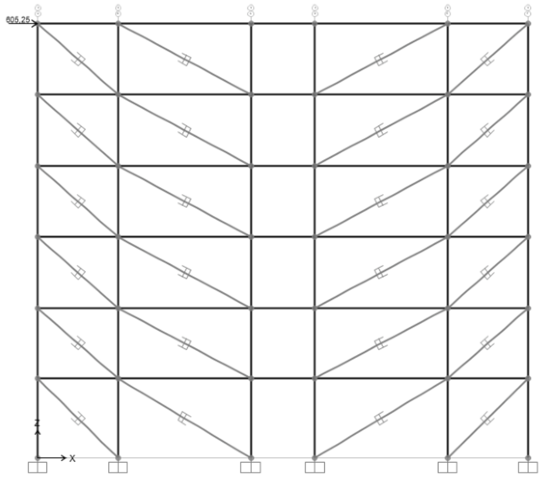
$$P_{base,\mathrm{max}} = P_{1,\mathrm{max}} = (6 - 1 + 1) \cdot 0.8^{1-0.15} \cdot \frac{2 \cdot 0.30 \cdot \frac{16600\ \mathrm{kN}}{g} \cdot 0.294g}{8} \cdot \tan 36° = 1332\ \mathrm{kN}$$

Application - Step 5

STEP 5

ESA2





$$F_{D,h,\max,struttura} = 0.8^{1-\bar{\alpha}} \cdot 2 \cdot \bar{\xi}_{visc} \cdot m_{tot} \cdot S_e\left(T_1, \bar{\eta}\right)$$

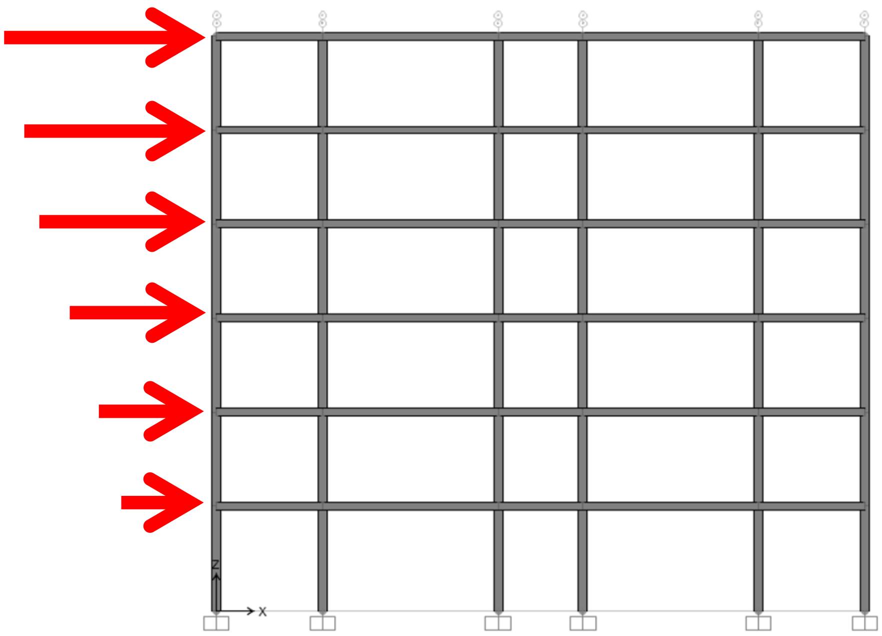
$$F_{D,h,\max,struttura} = 0.8^{1-0.15} \cdot 2 \cdot 0.30 \cdot \frac{16600\ \text{kN}}{g} \cdot 0.294g = 2421\ \text{kN}$$

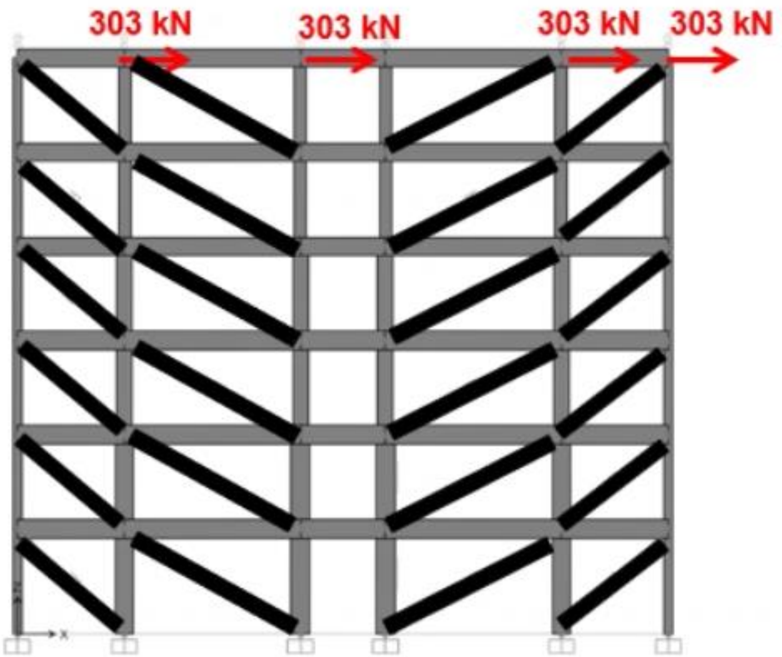
$n = 8$

$$F_{D,h,\max,specchiatura} = 0.8^{1-\bar{\alpha}} \cdot \frac{2 \cdot \bar{\xi}_{visc} \cdot m_{tot} \cdot S_e\left(T_1, \bar{\eta}\right)}{n}$$

$$F_{D,h,\max,specchiatura} = 0.8^{1-0.15} \cdot \frac{2 \cdot 0.30 \cdot \frac{16600\ \text{kN}}{g} \cdot 0.294g}{8} = 303\ \text{kN}$$

Application - Step 5

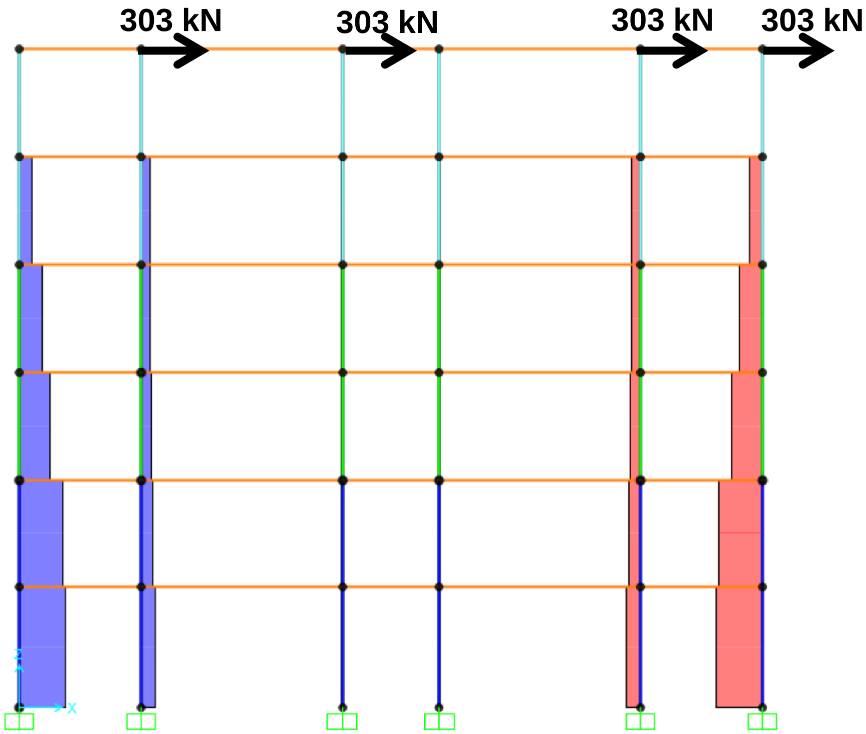


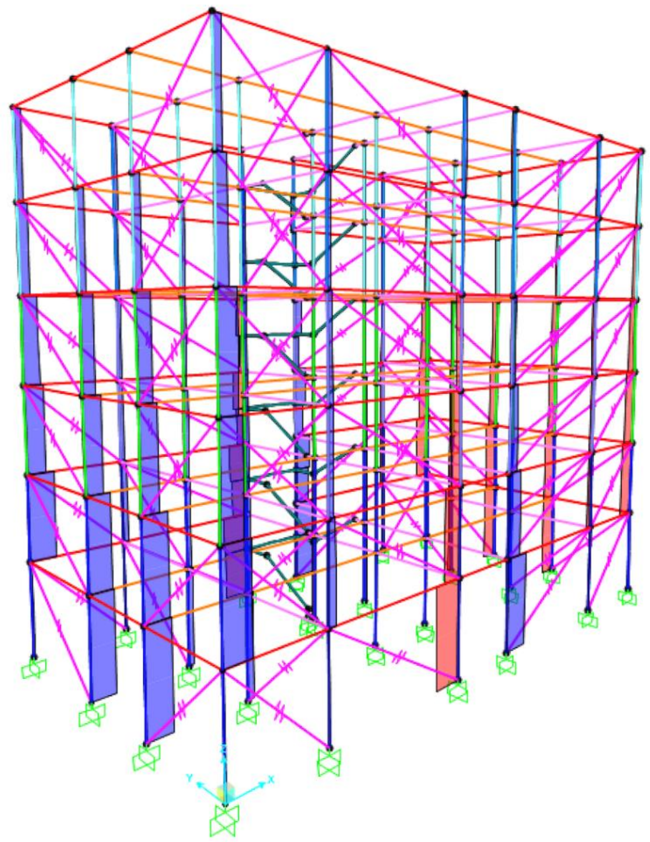


ESA1

ESA2

Application - Step 5





ESA2

axial forces in the columns

XLS file of the procedure

		dir. long. (x)	dir. trasv. (y)	unità di misura
STEP 1				
smorzamento intrinseco	ξ intr =	0,05	0,05	
smorzamento viscoso	ξ visc =	0,30	0,30	
smorzamento totale di target	ξ tot =	0,35	0,35	
fattore riduzione risposta di target	η =	0,500	0,500	

STEP 2				
numero totale piani	N =	6	6	
peso totale struttura	Wtot =	16600	16600	kN
periodo fondamentale struttura	T1 =	0,797	0,704	s
pulsazione fondamentale struttura	ω1 =	7,88	8,93	rad/s
numero di smorzatori per piano	n =	8	8	
inclinazione smorzatori	θ =	36	38	°
coefficiente smorzamento lineare	cL =	5384	6461	kNs/m
rigidezza assiale	kaxial =	infinita	infinita	

XLS file of the procedure

STEP 3				
accelerazione spettrale	Sa(T1) =	0,294	0,333	g
coefficiente correttivo	M =	1,00	1,00	
velocità massima smorzatori lineari	vmax =	0,084	0,082	m/s
forza massima smorzatori lineari	FLmax =	454	529	kN
corsa massima pistone	smax =	1,07	0,92	cm

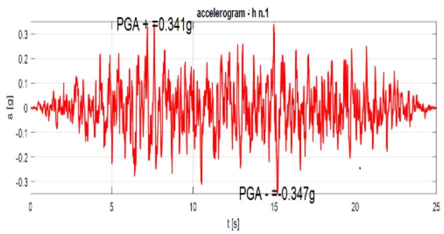
STEP 4				
esponente	α =	0,15	0,15	
coefficiente smorzamento non-lineare	cNL =	544	637	kN (s/m)^a
forza massima smorzatori non-lineari	FNLmax =	375	437	kN
rigidezza assiale minima	kaxial >	424327	576634	kN/m

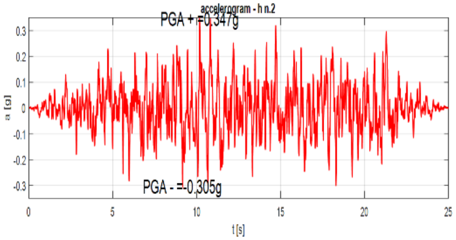
XLS file of the procedure

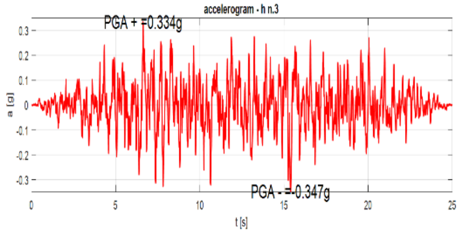
STEP 5				
ESA1				
forza totale	Fh =	4877	5523	kN
ESA2				
forza struttura	Fstructure =	2421	2741	kN
numero telai con smorzatori	n frames =	2	4	
forza telaio	Fframe =	1210	685	kN
numero specchiature con smorzatori nel telaio	n bays =	4	2	
forza specchiatura (singola reticolare)	Fbay =	303	343	kN
sforzo normale max nelle colonne				
	P1,max =	1332	1631	kN
	P2,max =	1110	1359	kN
	P3,max =	888	1088	kN
sforzo normale max alla base singola colonna	Pbase =	1332	1631	kN

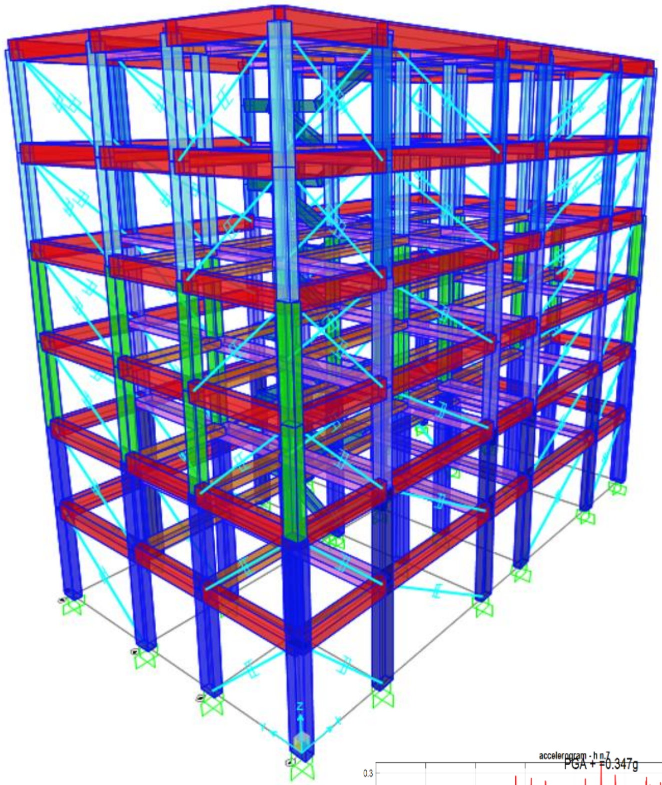
TH verification

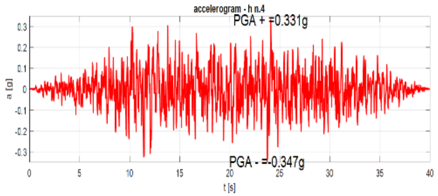
7 artificial accelerograms which are consistent with the elastic spectrum:

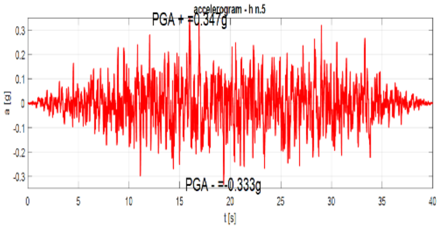


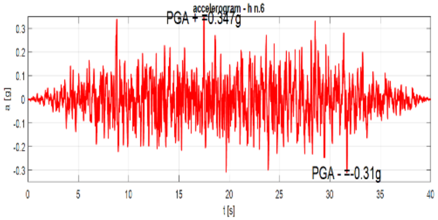


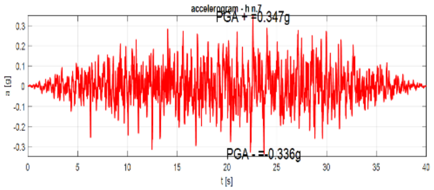




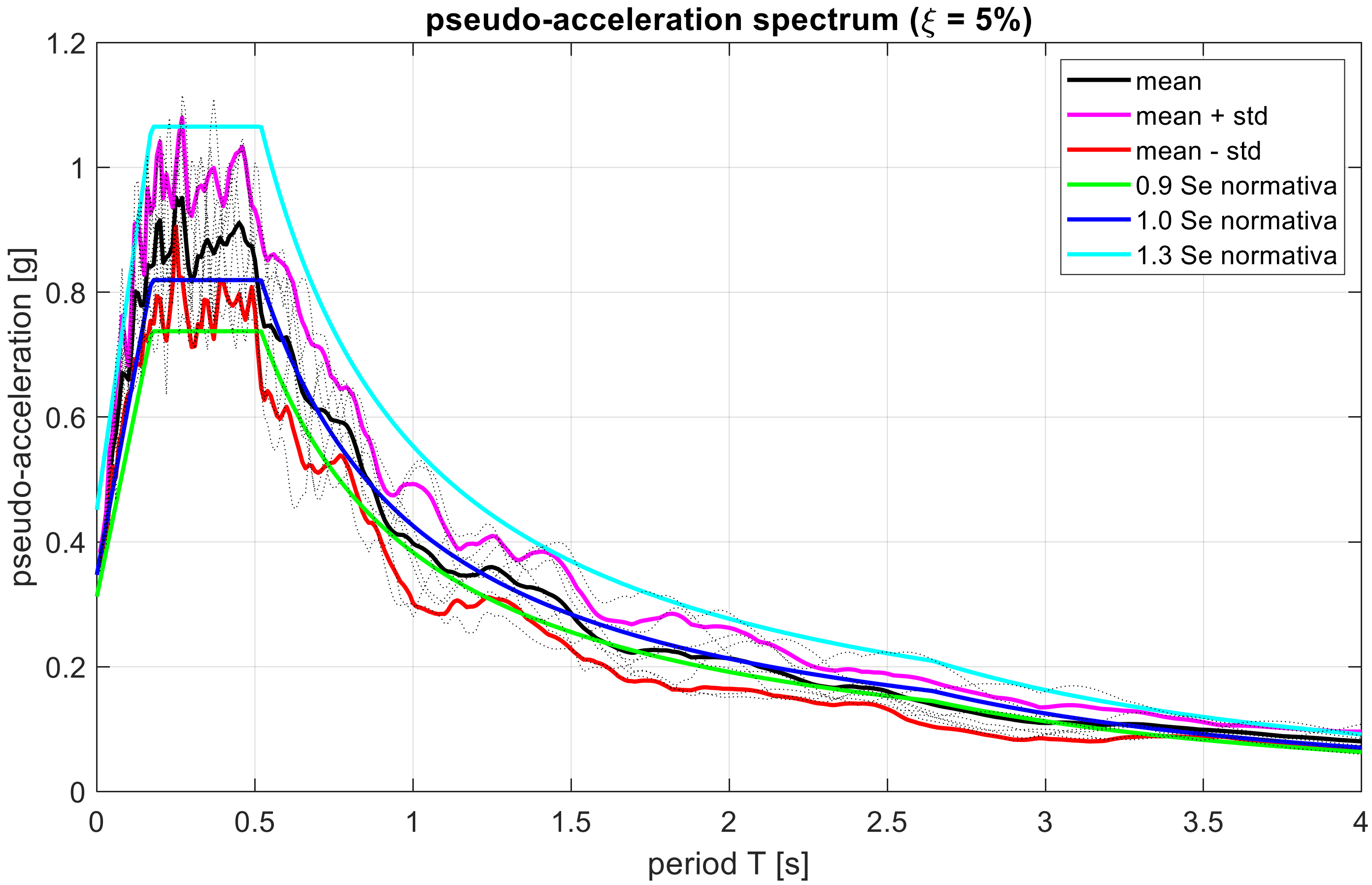






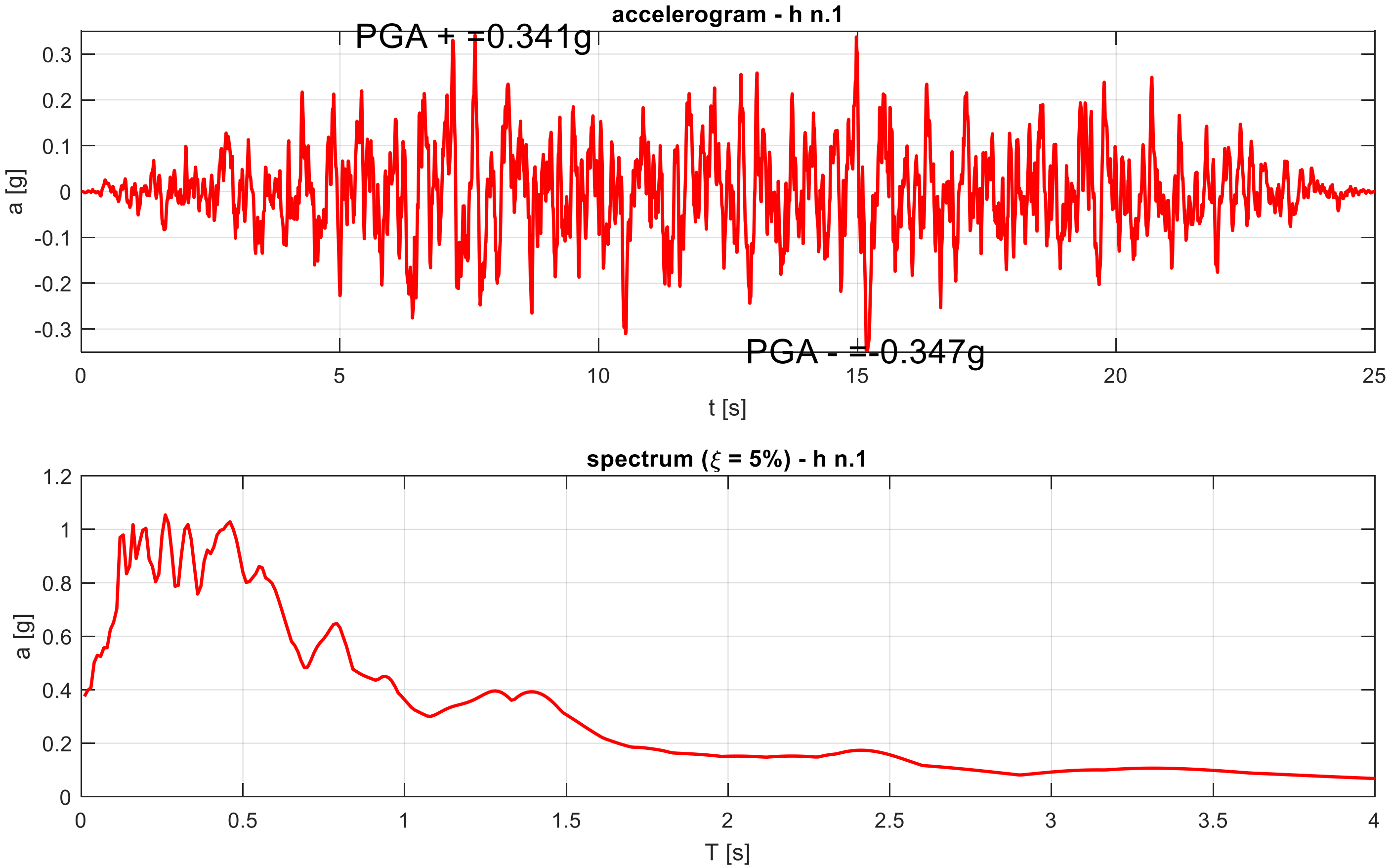


TH verification



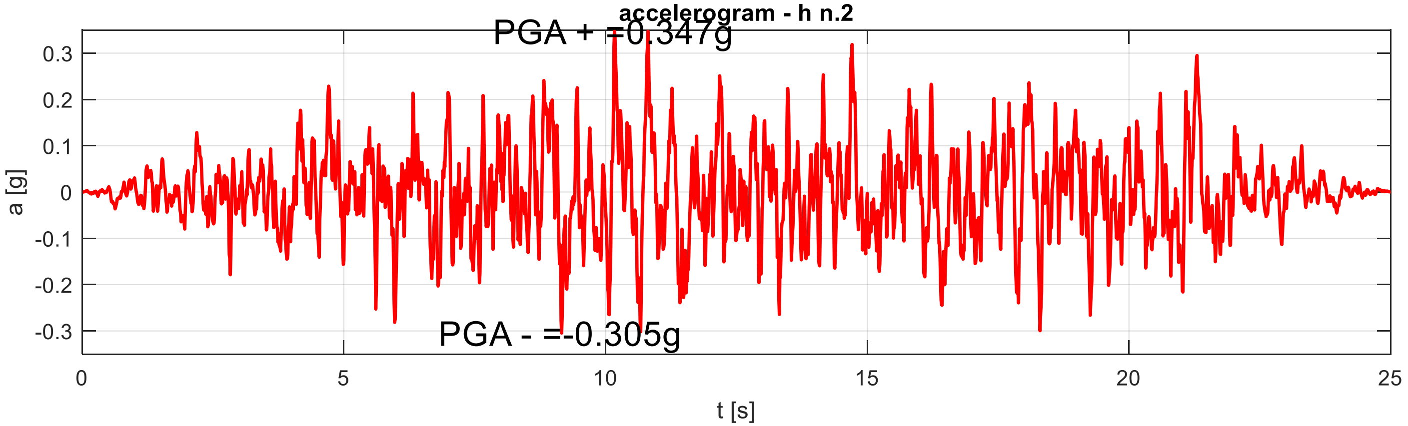
TH verification

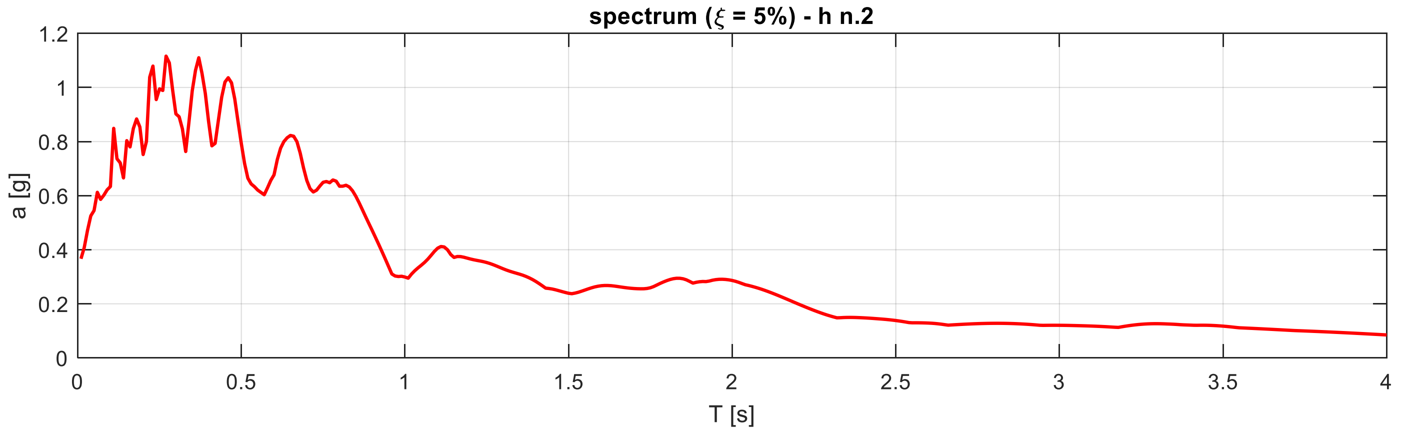
Ex: ground motion n.1



TH verification

Ex: ground motion n.2





TH verification

(comparison between models)

Model DL

$$c_L = 5384 \frac{\mathrm{kN \cdot s}}{\mathrm{m}}$$

$$\alpha = 1$$

$$K = \infty$$

Model DNL kinf

(with stiffness of dampers equal to infinity)

$$c_{NL} = 544 \frac{\mathrm{kN \cdot s^{\alpha}}}{\mathrm{m^{\alpha}}}$$

$$\alpha = 0.15$$

$$K = \infty$$

Model DNL

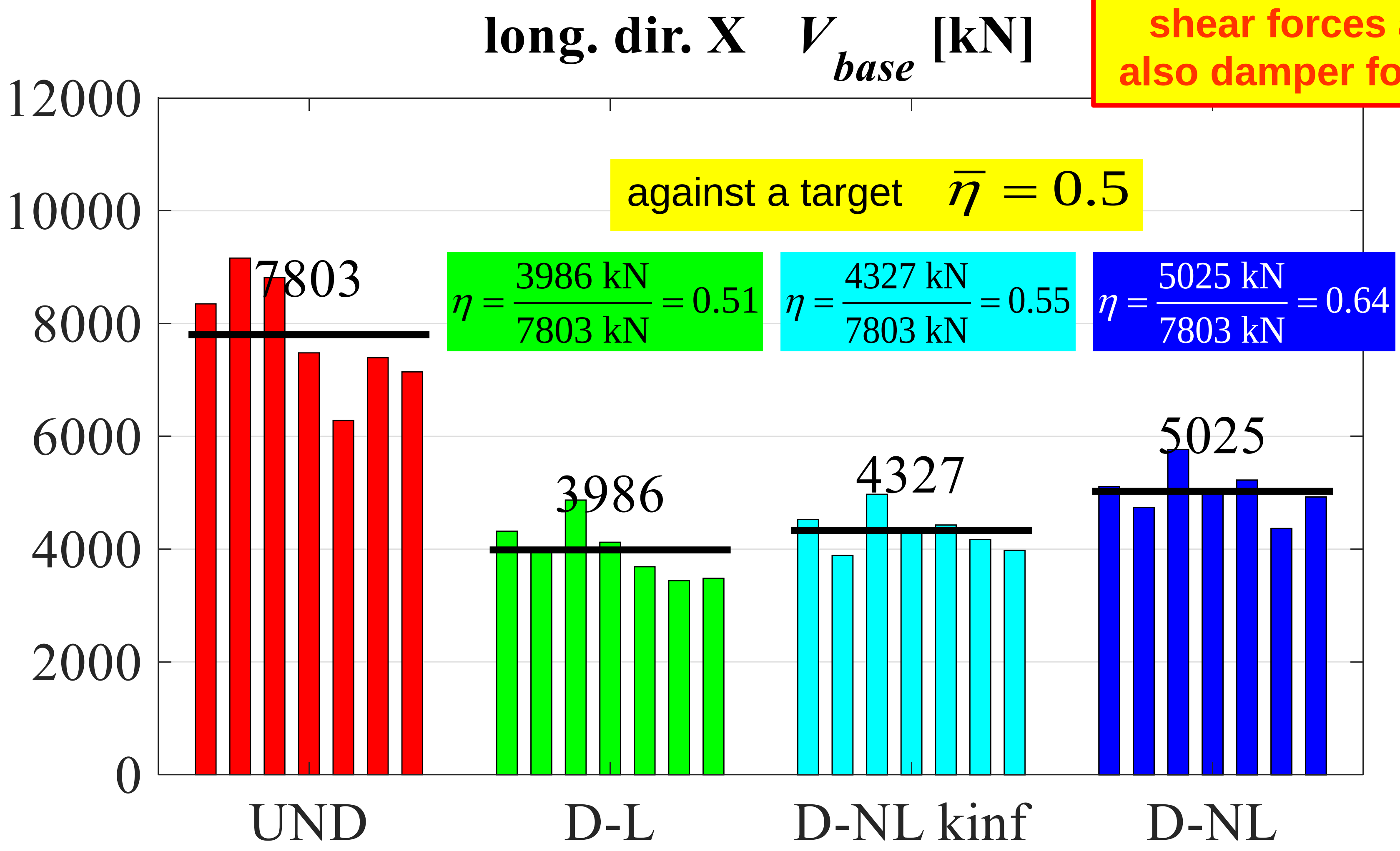
$$c_{NL} = 544 \frac{\mathrm{kN \cdot s^{\alpha}}}{\mathrm{m^{\alpha}}}$$

$$\alpha = 0.15$$

$$K = 4.5 \cdot 10^5 \frac{kN}{m}$$

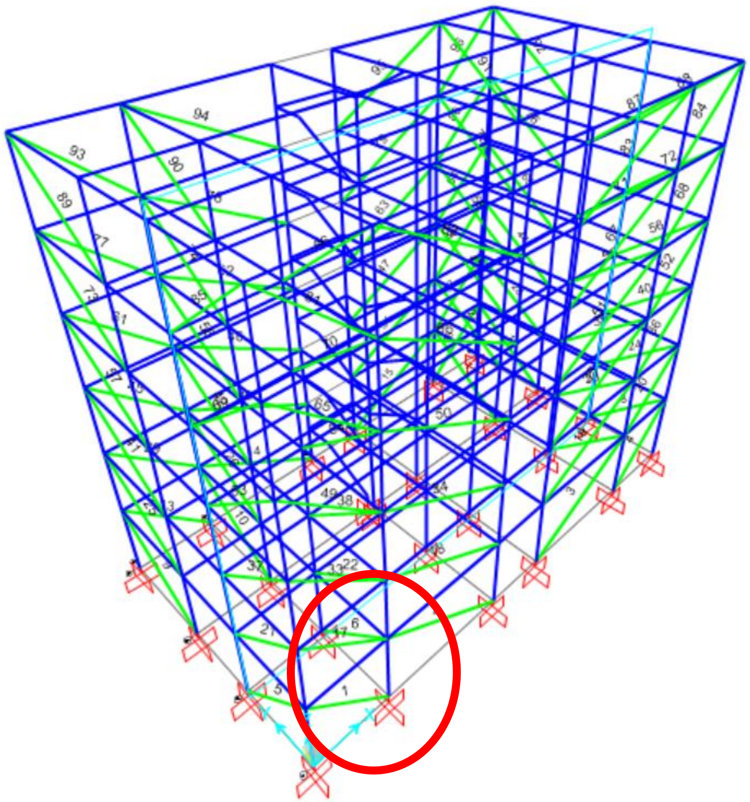
TH verification

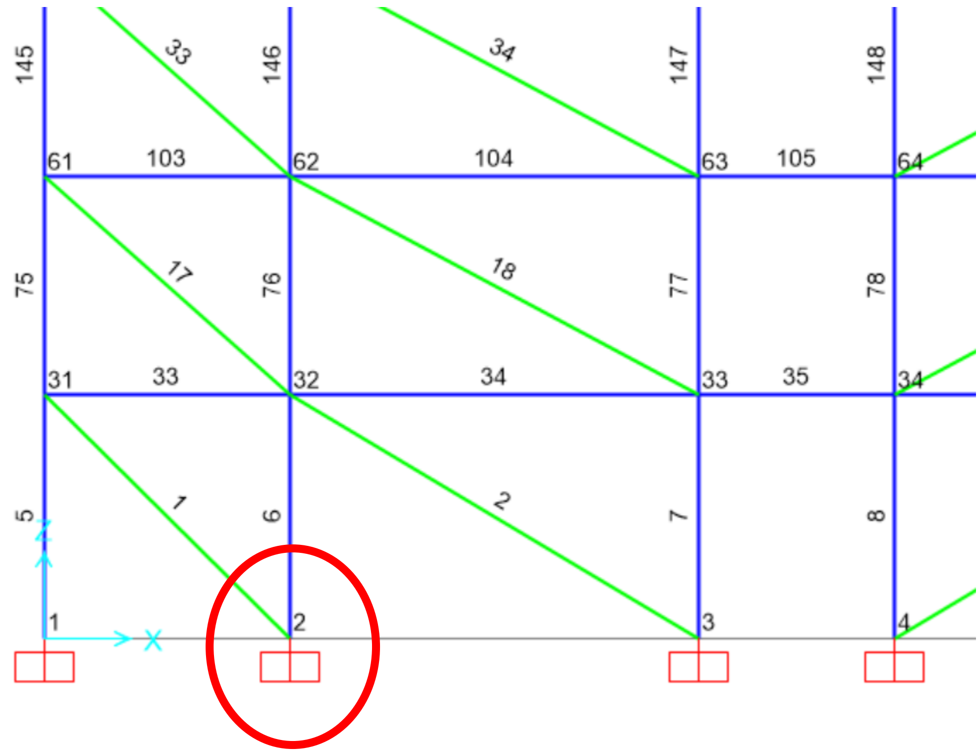
!!!
Total base shear at the foundation level (including column shear forces and also damper forces)



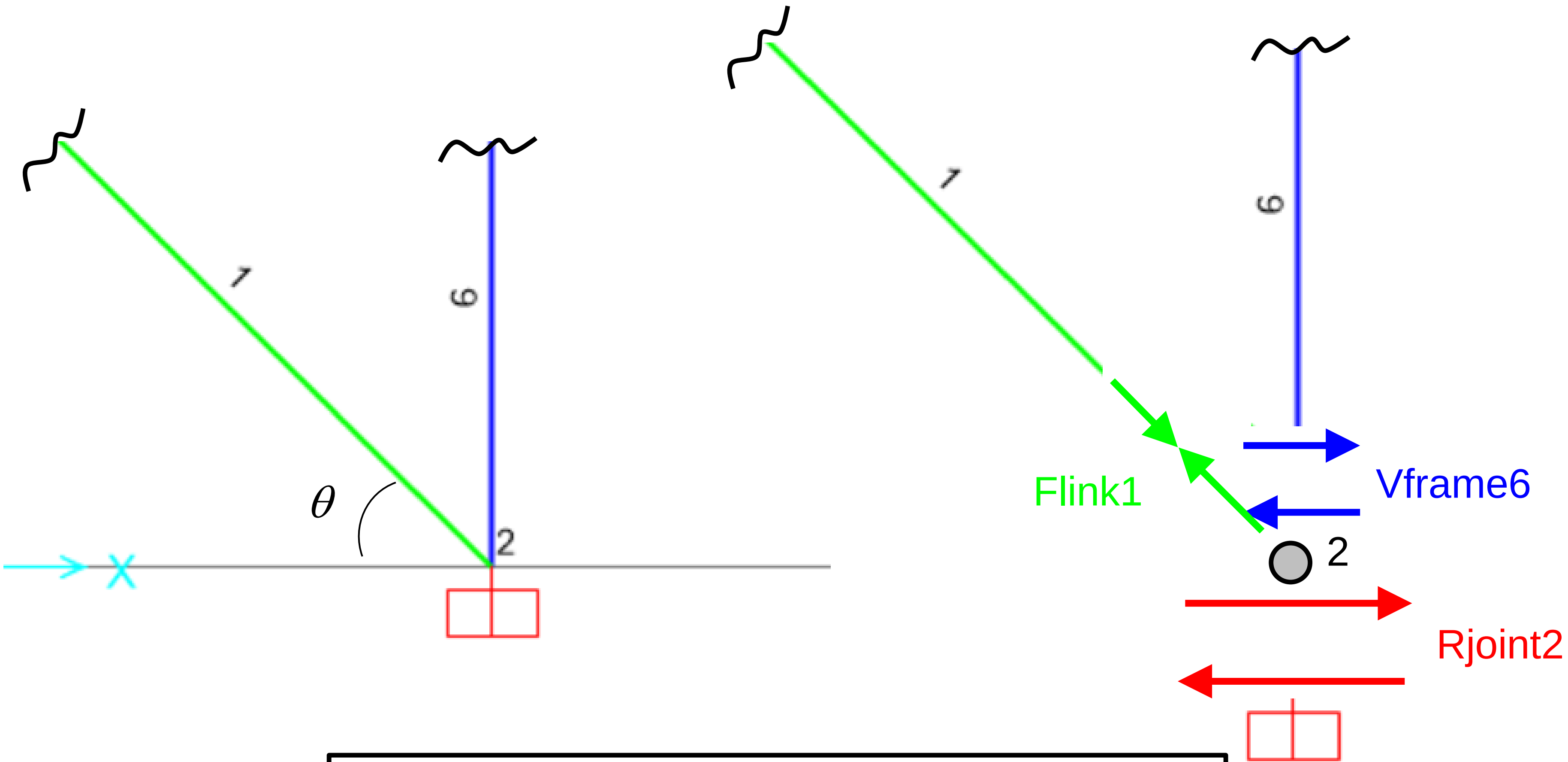
TH verification

Model D-NL





TH verification

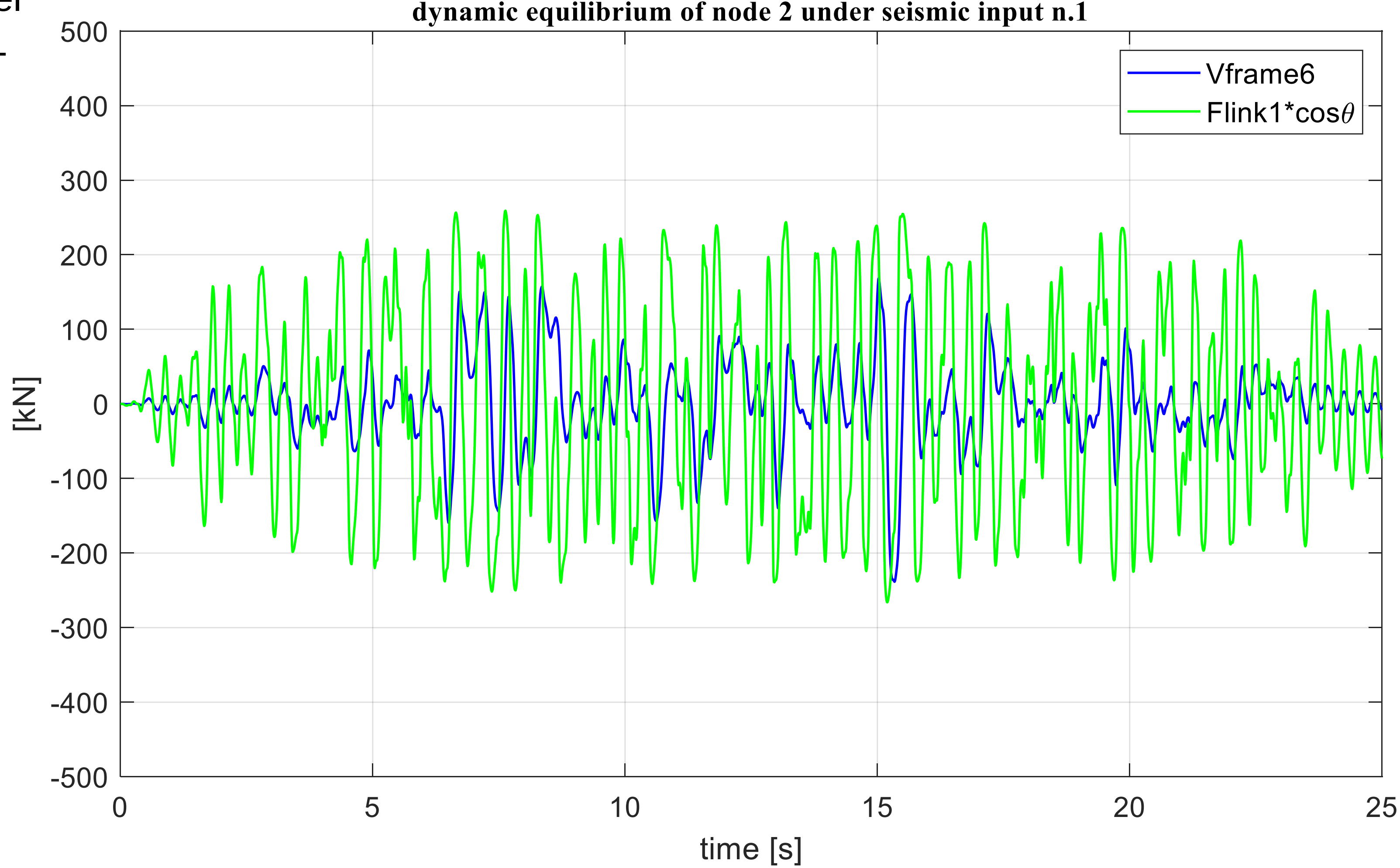


$$R_{\text{joint2}} = V_{\text{frame6}} + F_{\text{link1}} \cdot \cos\theta$$

TH verification

$$R_{joint2} = V_{frame6} + F_{link1} \cdot \cos\theta$$

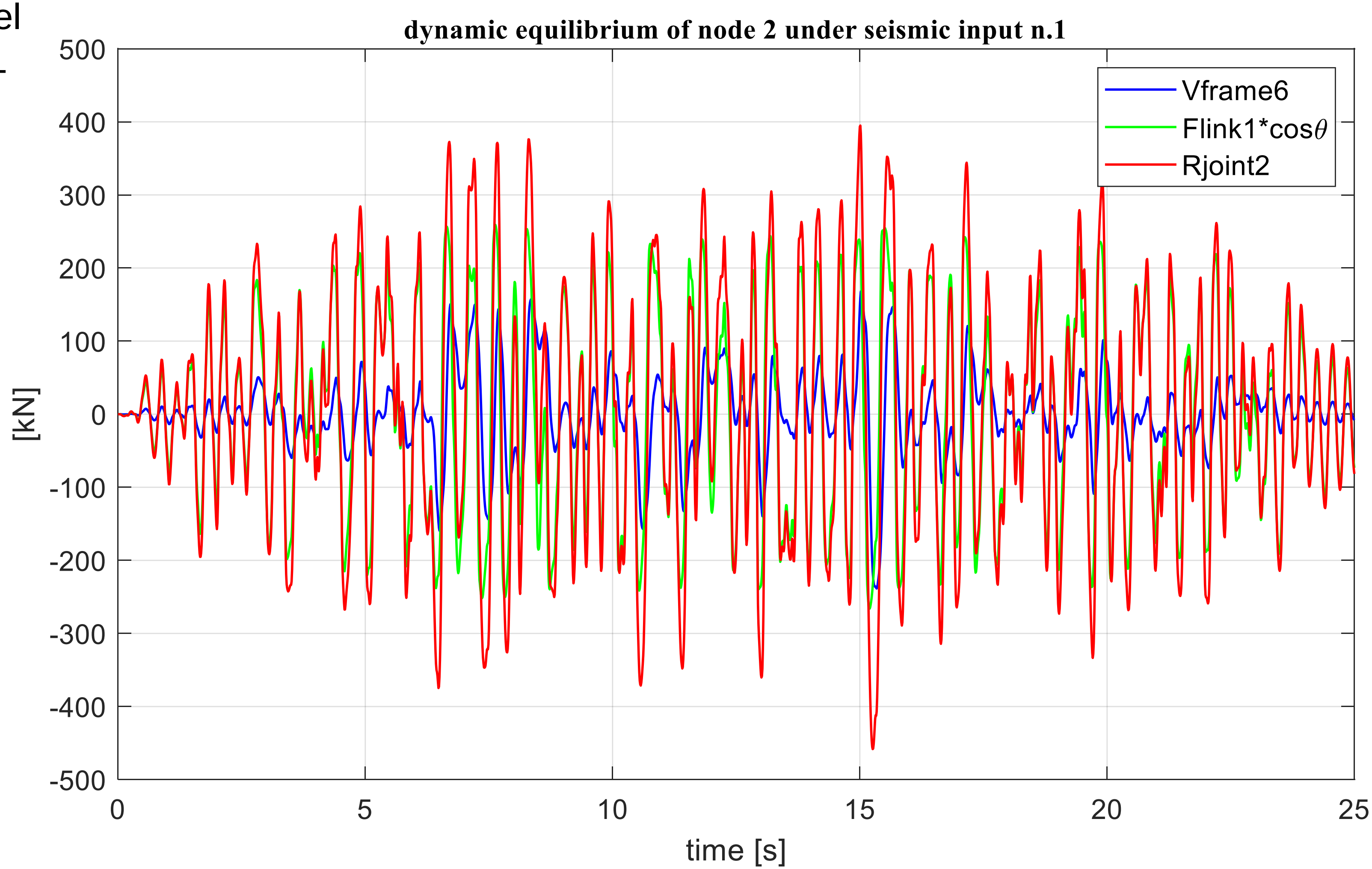
Model D-NL



TH verification

$$R_{joint2} = V_{frame6} + F_{link1} \cdot \cos\theta$$

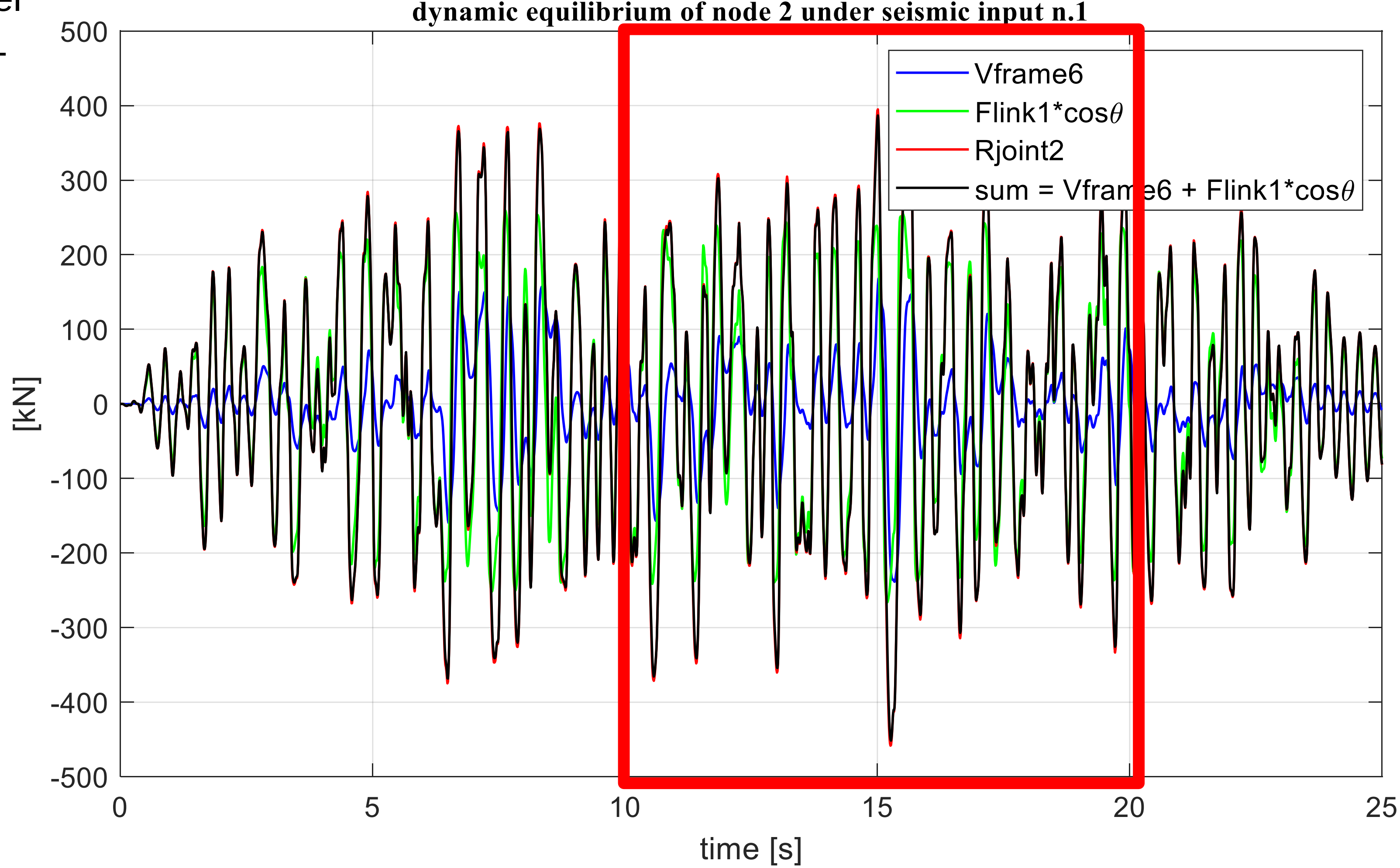
Model
D-NL



TH verification

$$R_{joint2} = V_{frame6} + F_{link1} \cdot \cos\theta$$

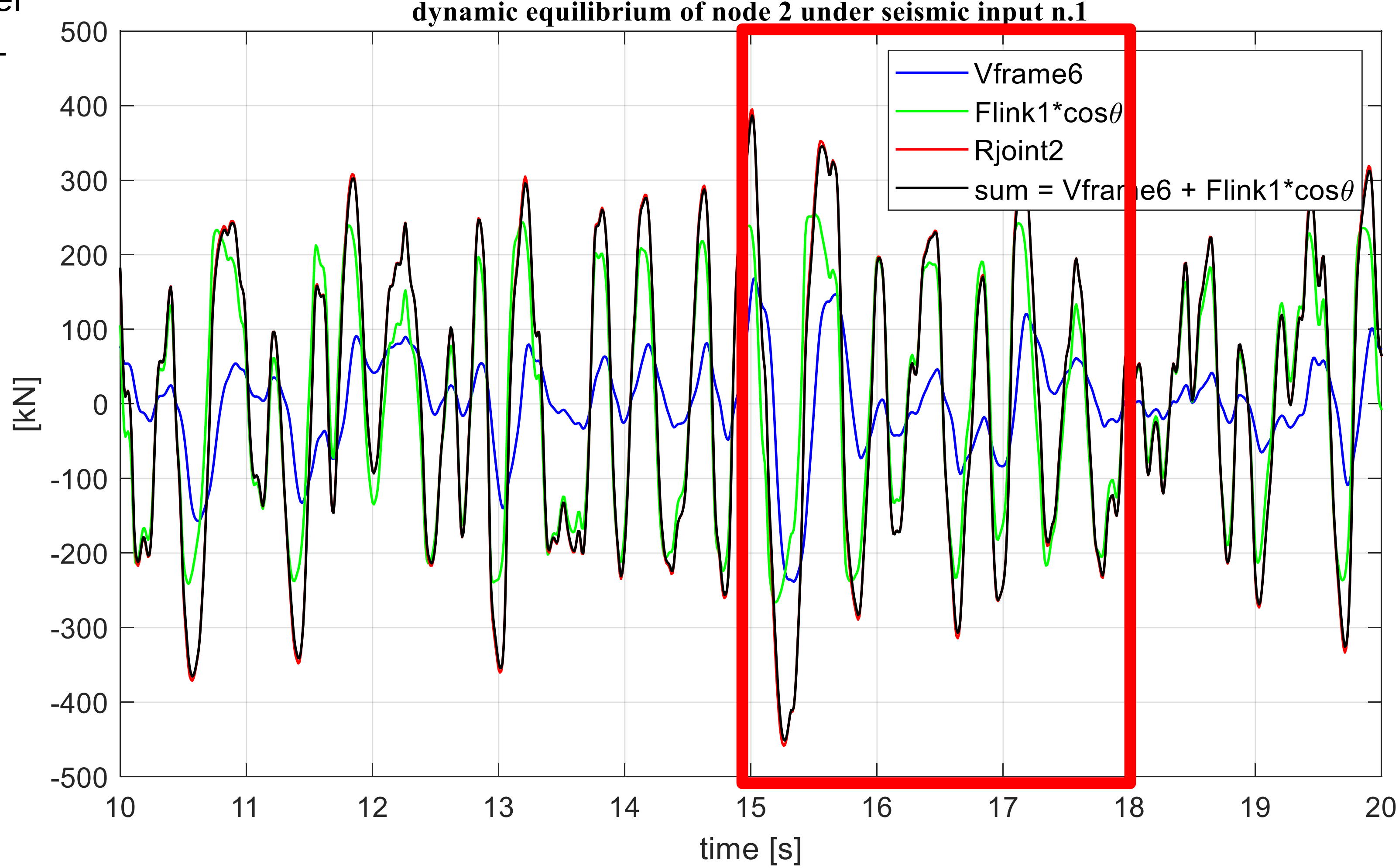
Model D-NL



TH verification

$$R_{joint2} = V_{frame6} + F_{link1} \cdot \cos\theta$$

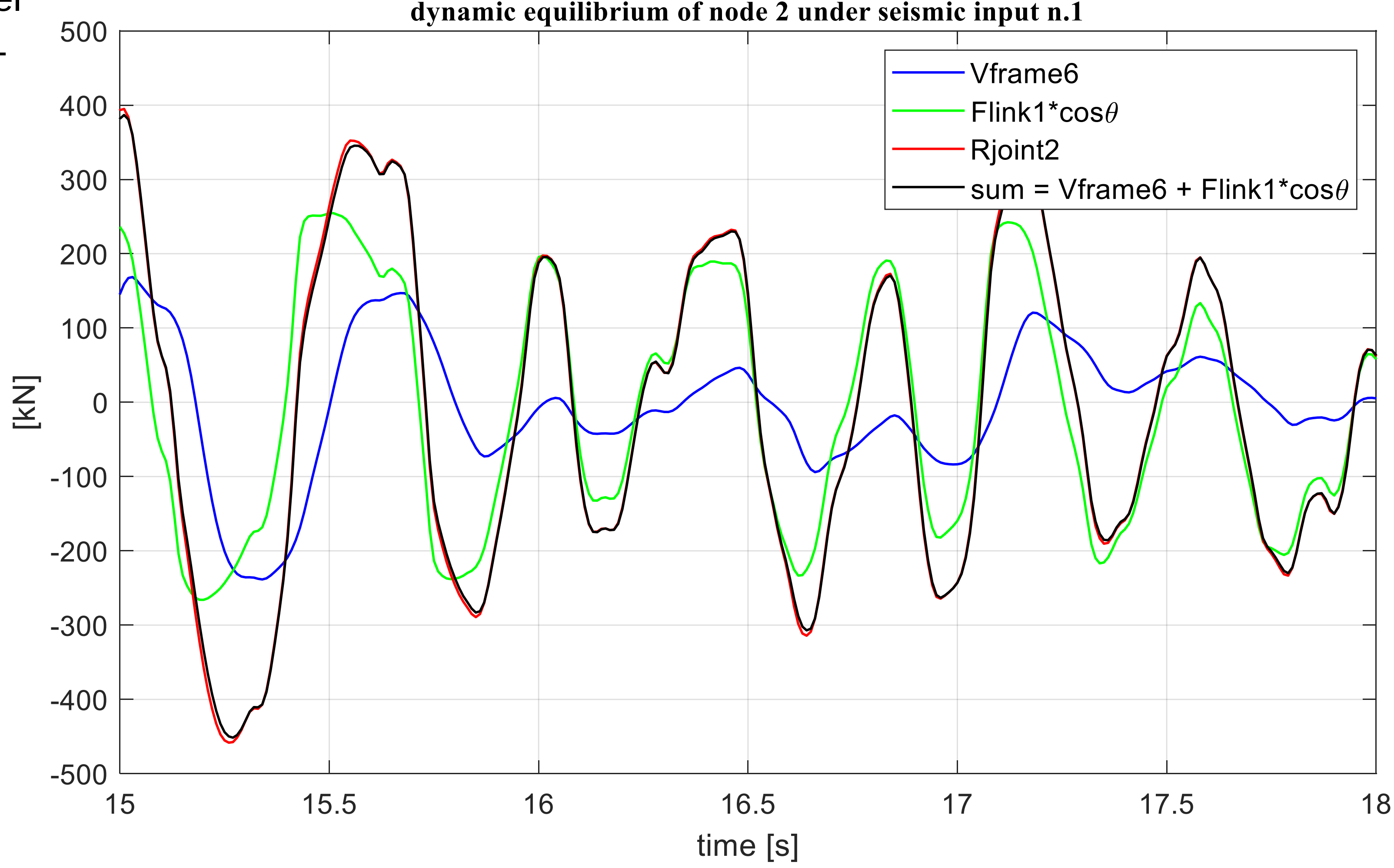
Model
D-NL



TH verification

$$R_{joint2} = V_{frame6} + F_{link1} \cdot \cos\theta$$

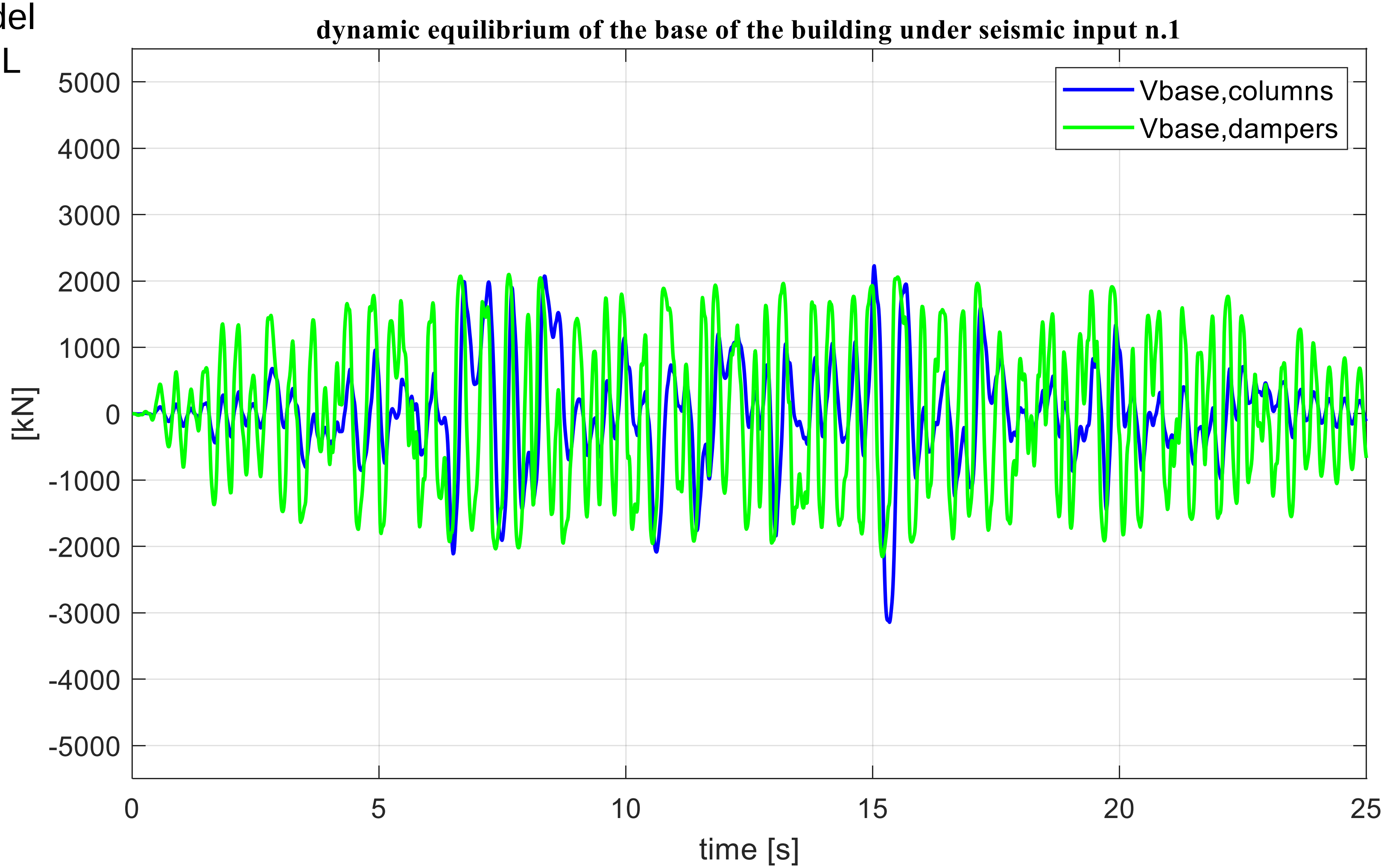
Model
D-NL



TH verification

The same occurs for all ground floor joints

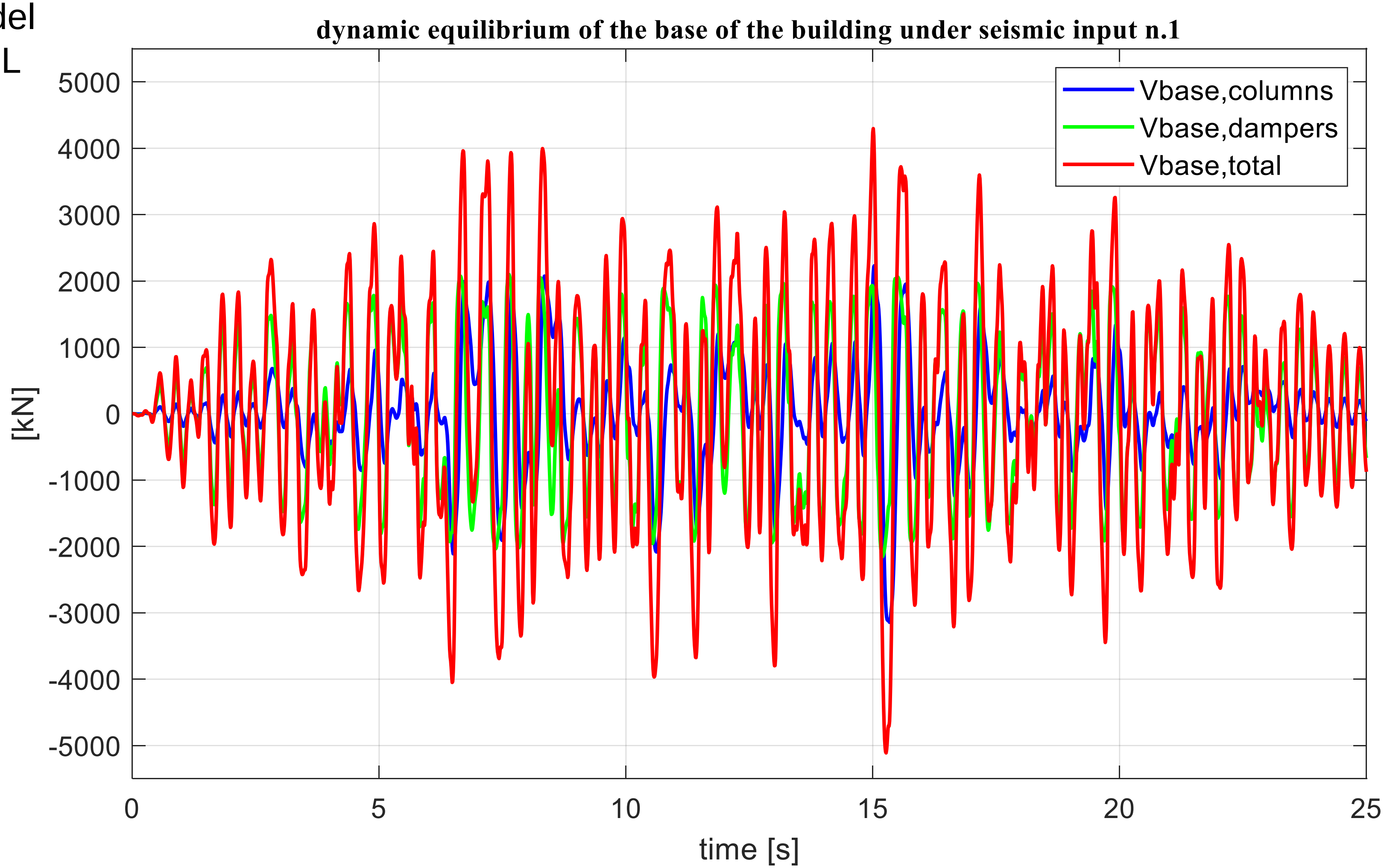
Model D-NL



TH verification

The same occurs for all ground floor joints

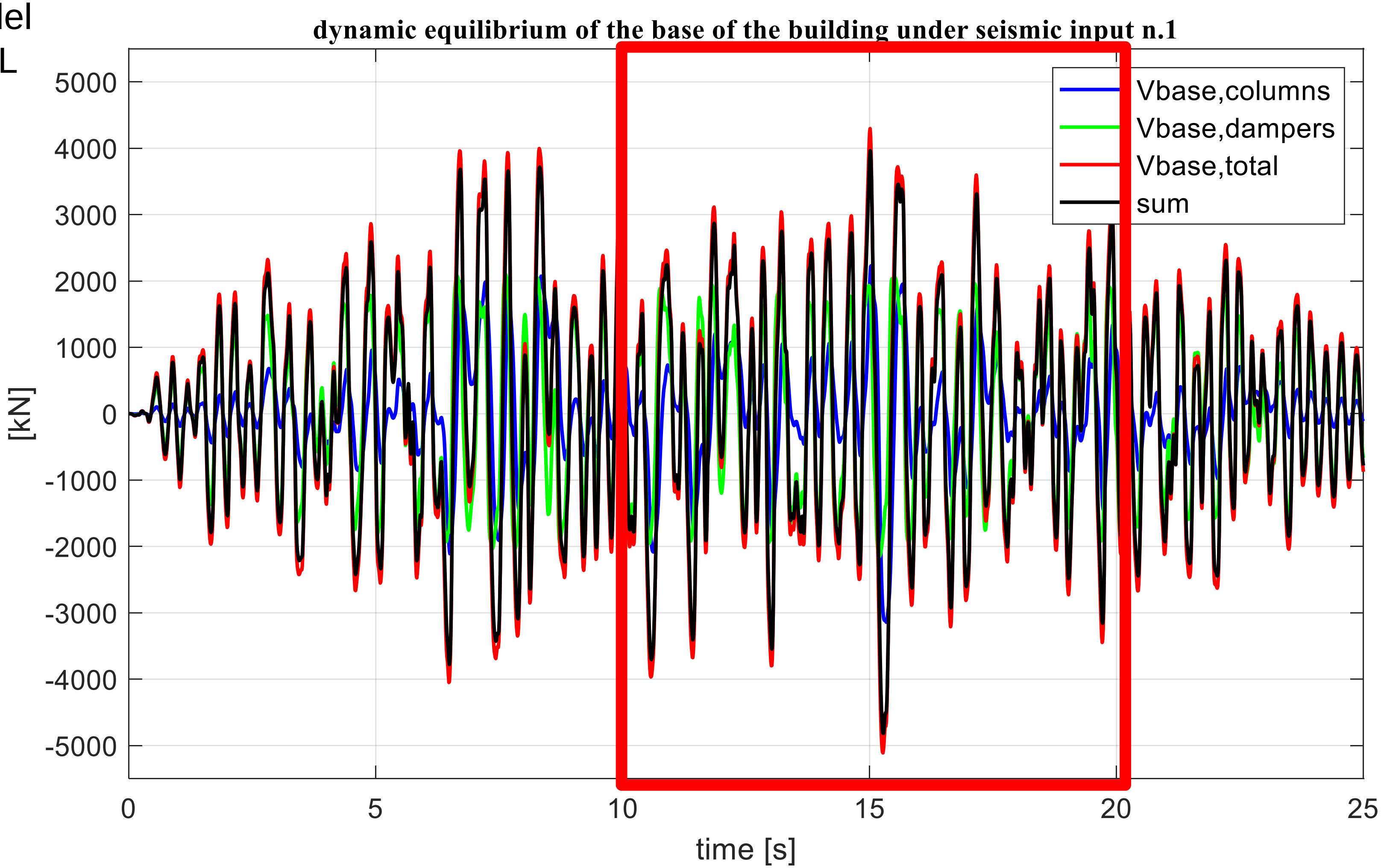
Model D-NL



TH verification

The same occurs for all ground floor joints

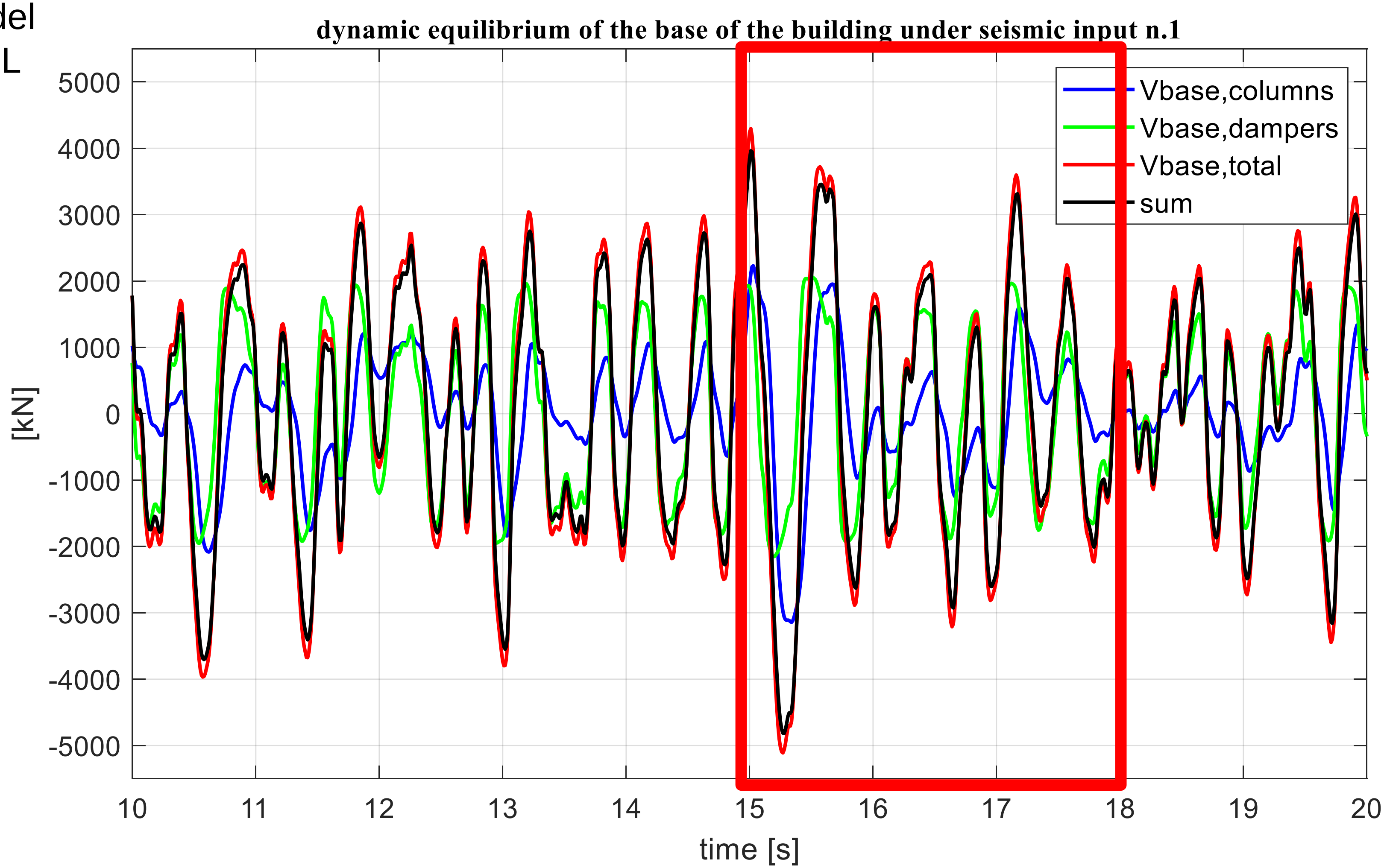
Model D-NL



TH verification

The same occurs for all ground floor joints

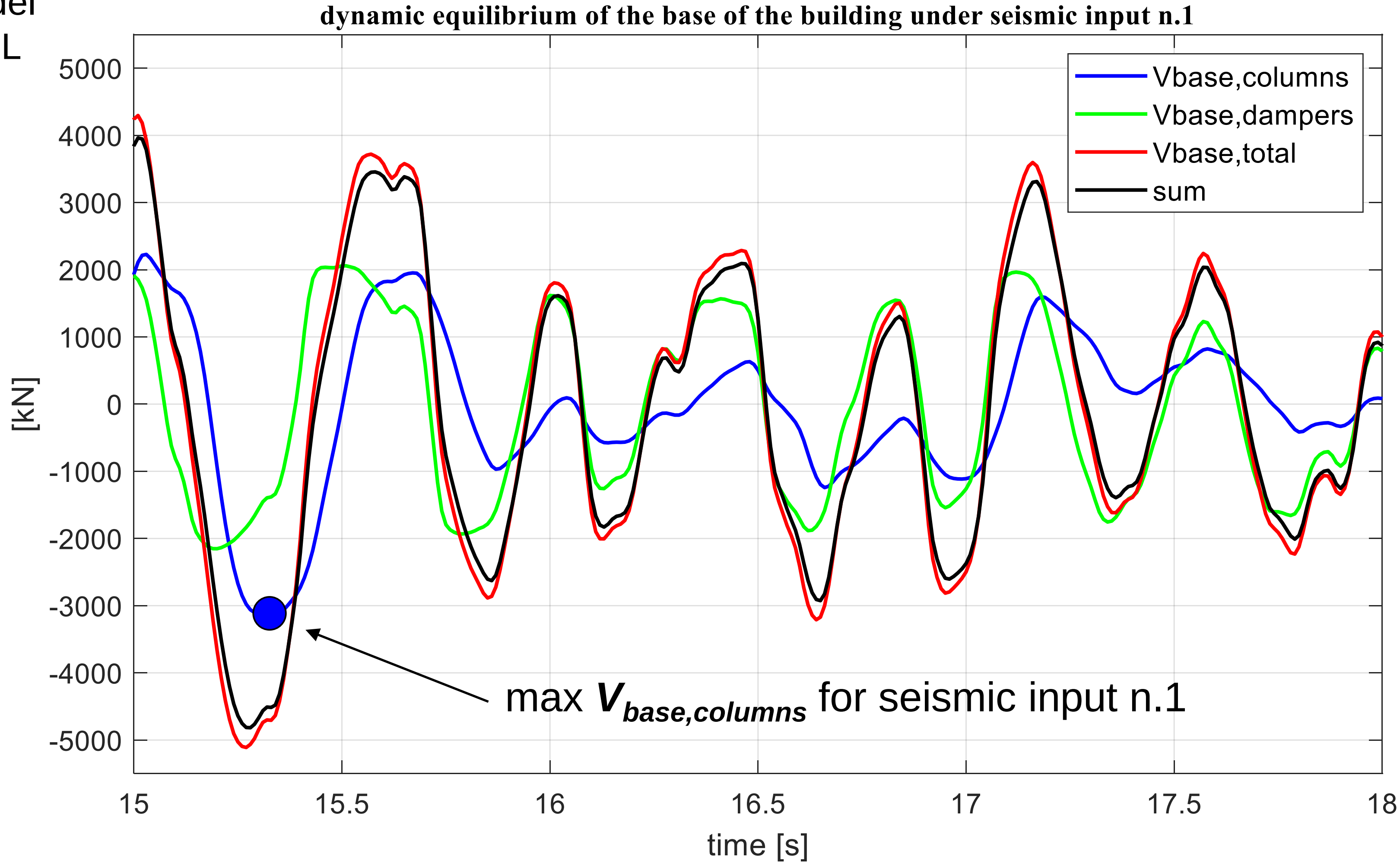
Model D-NL



TH verification

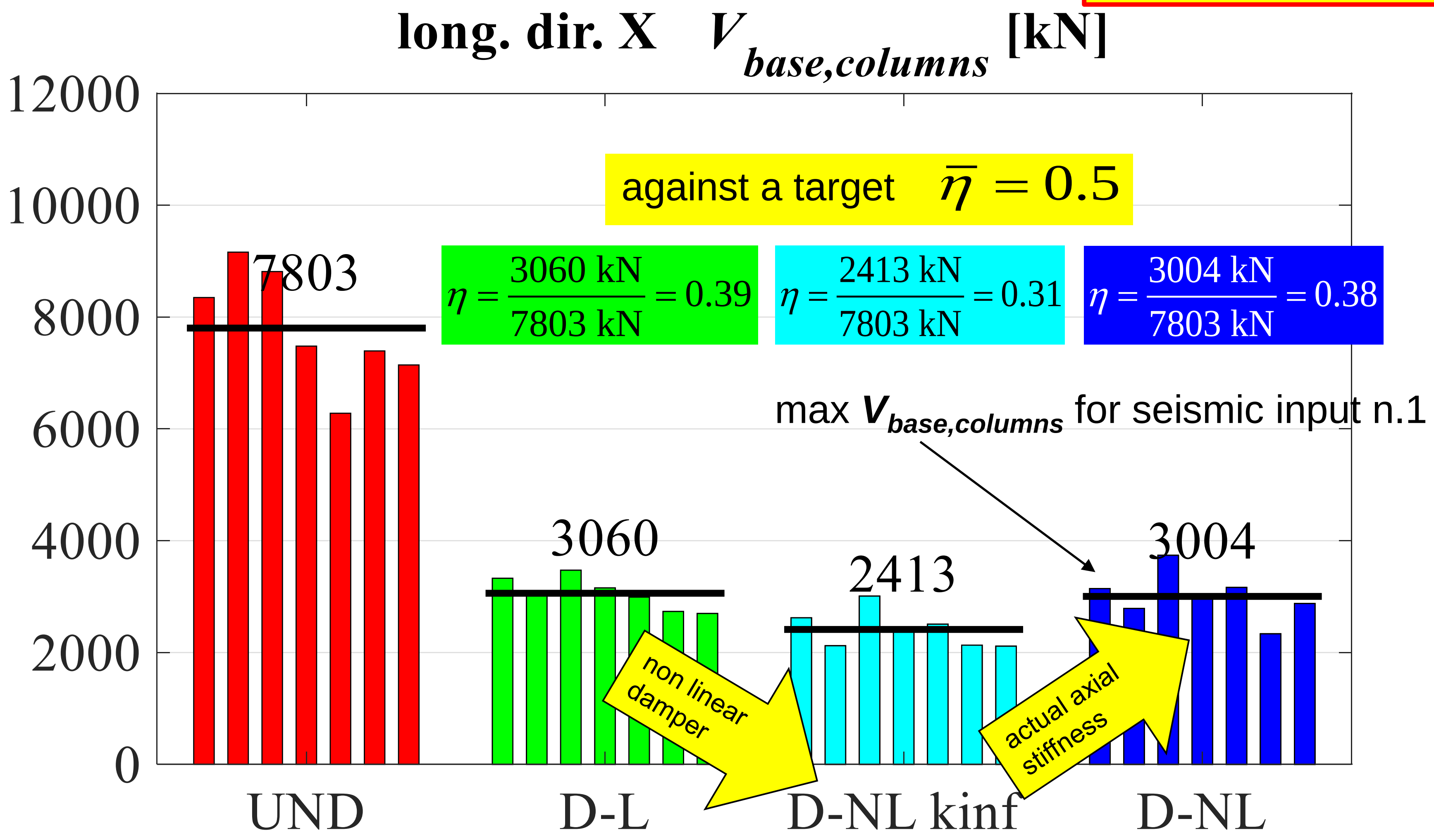
The same occurs for all ground floor joints

Model D-NL



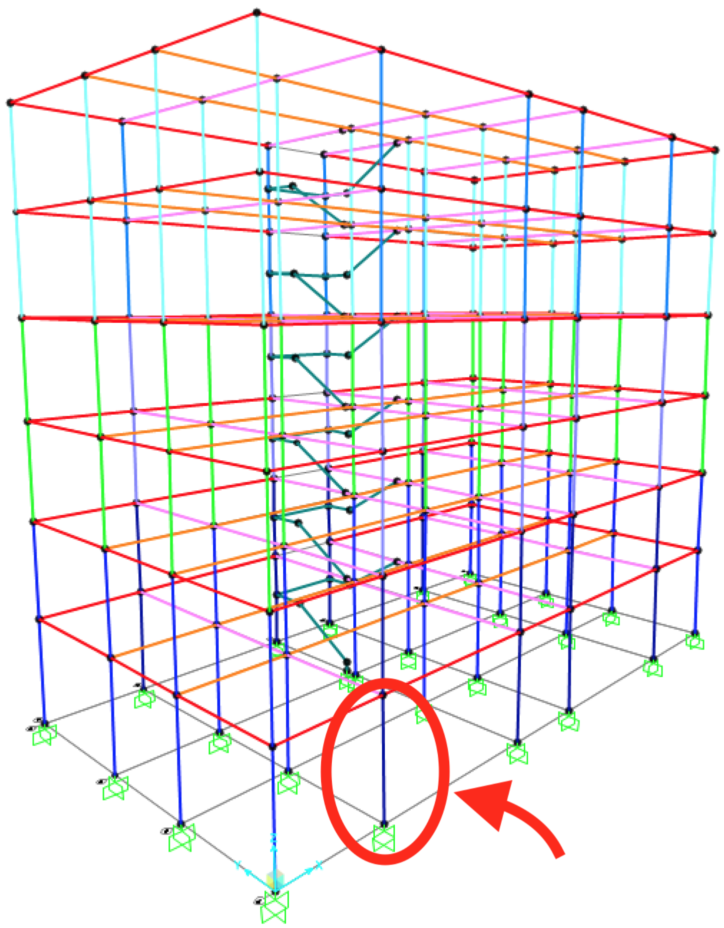
TH verification

!!!
Sum of base shear forces in the columns



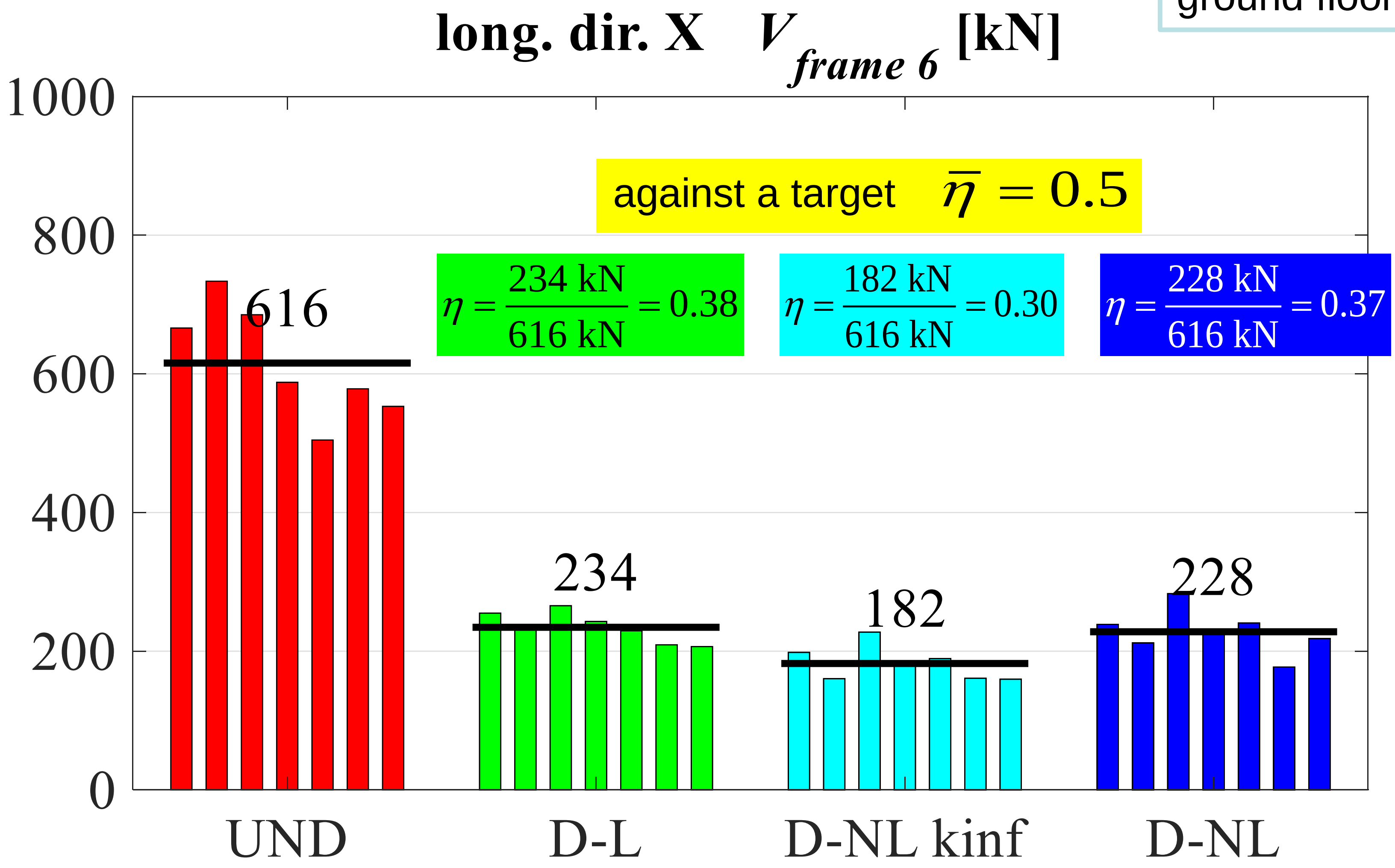
TH verification

FOCUS on the most stressed column at the ground floor



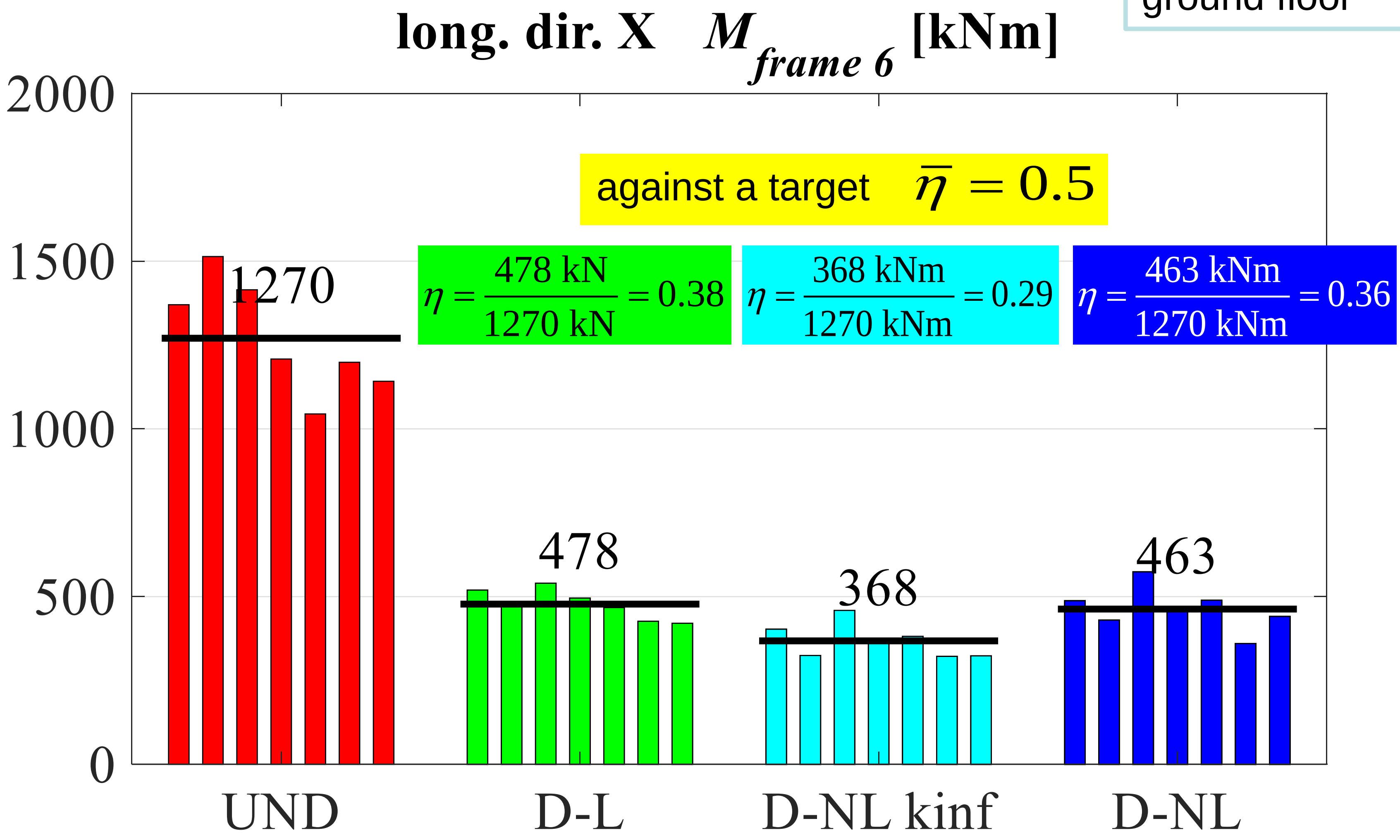
TH verification

FOCUS on the most stressed column at the ground floor

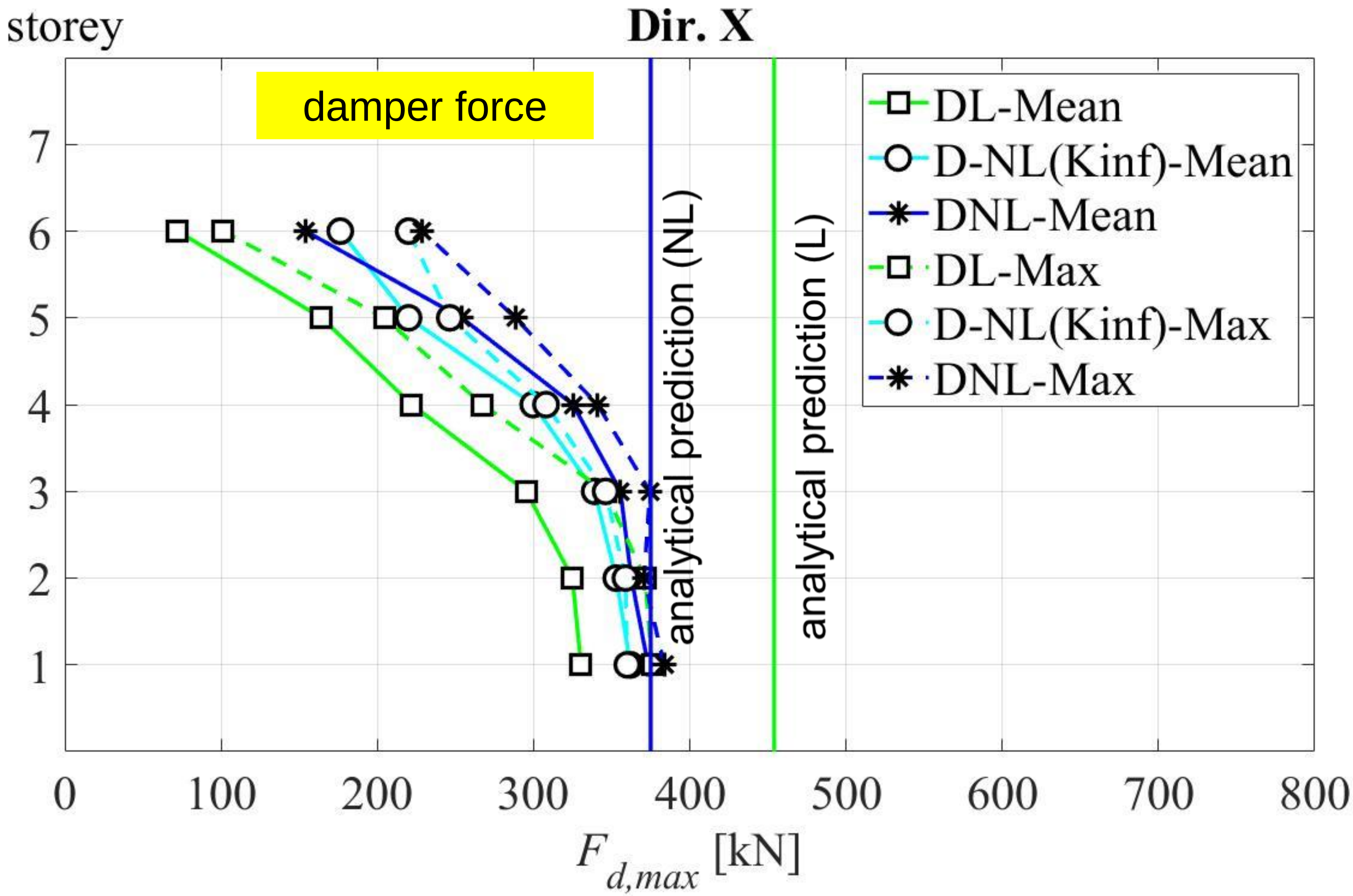


TH verification

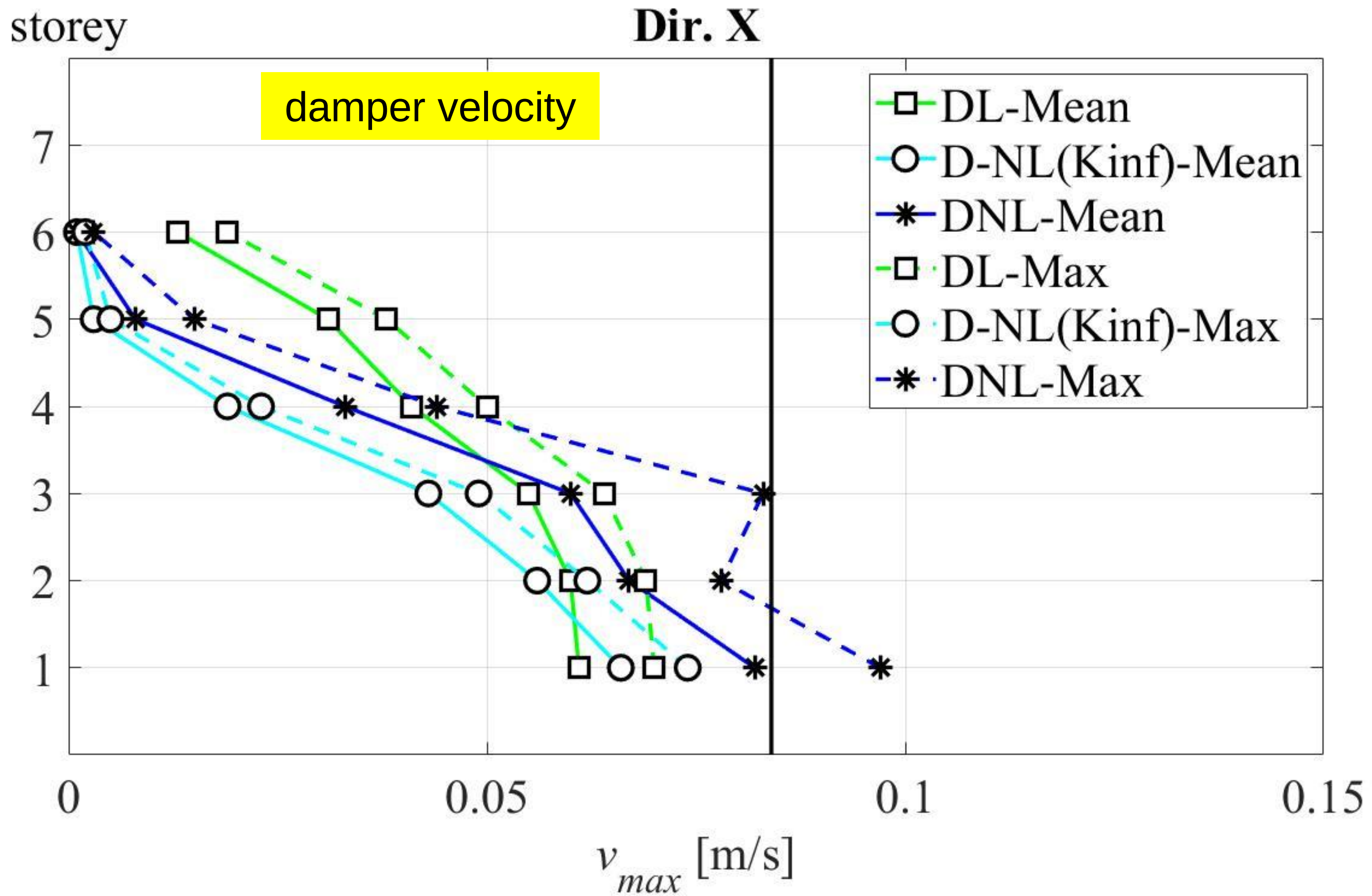
FOCUS on the most stressed column at the ground floor



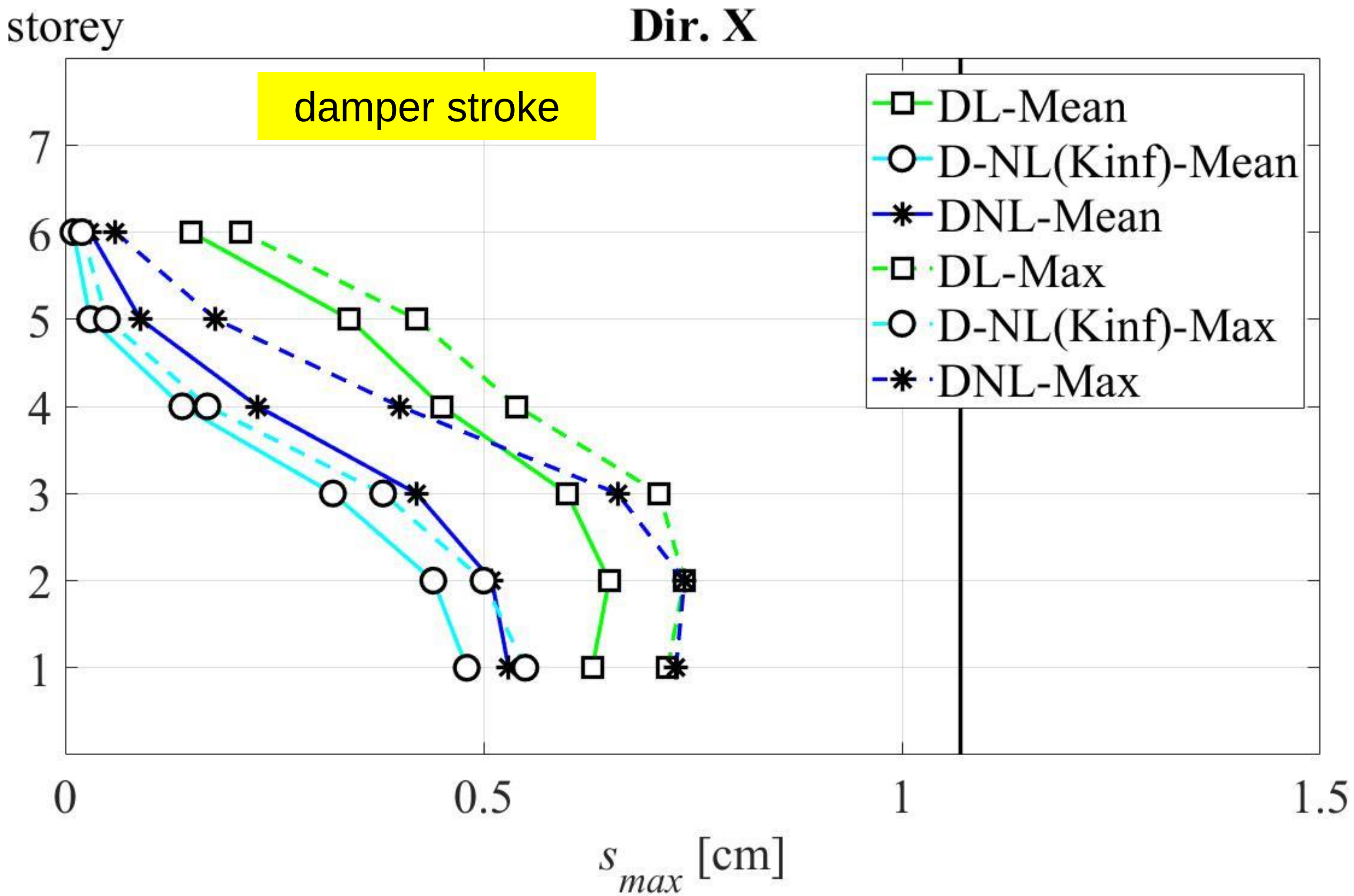
TH verification



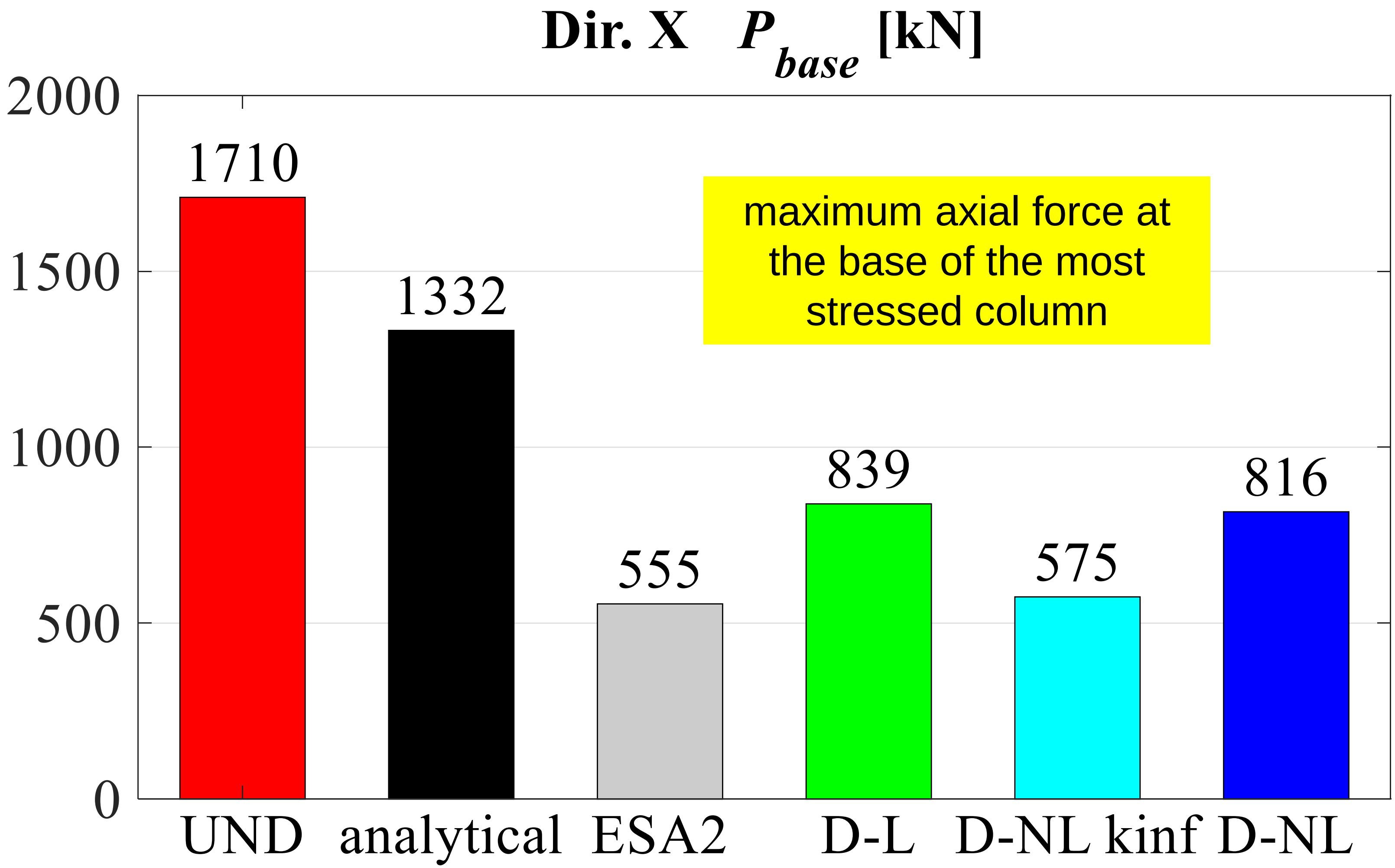
TH verification



TH verification



TH verification



“Italian” reference paper

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DOI 10.7414/PS.8.3.9-24 - http://dx.medra.org/10.7414/PS.8.3.9-24

Ricerca

Un metodo semplificato per il dimensionamento e l’analisi di strutture equipaggiate con smorzatori viscosi

A simplified method for dimensioning and analyzing equipped structures with viscous dampers

Michele Palermo[1], Stefano Silvestri[1], Giada Gasparini[1], Tomaso Trombetti[1] ■

Sommario
La presente memoria presenta un metodo diretto per il dimensionamento di strutture a telaio dotate di smorzatori viscosi che permette: (1) di dimensionare la taglia degli smorzatori viscosi da inserire nella struttura in modo da soddisfare un determinato obiettivo prestazionale; (2) di stimare le massime sollecitazioni negli elementi strutturali attraverso l’inviluppo di due analisi statiche equivalenti.

General Conclusions

- **MPD vs SPD systems**

- A **direct (fully analytical) procedure** for the seismic design of building structures with added viscous dampers is presented.

- It represents the **step forward** of the “five-step procedure” (2010).

- It aims at providing **practical tools** for a direct identification of the mechanical characteristics of the manufactured viscous dampers which allow **to achieve target levels of performances**.

- The procedure seems to be **conservative**.

- **In any case, a numerical verification of the final behaviour** of the system by means of non-linear time-history analyses **is recommended**.

Future developments

- In its current version, the procedure is applicable to **regular multi-storey frame structures** which are characterized by a period of vibration lower than 1.5 s.

- At this stage of the research, the procedure is suitable:
  - for the **preliminary design phase**
  - for the **control of the order of magnitude of FEM results**,

  since correction factors for the higher modes contributions are necessary to improve its accuracy, especially for high-rise buildings.

- **Extension to existing buildings** to account for the hysteretic dissipation / ductility capacity of the structural elements

Thank you for your kind attention!

Thanks to :

- DPC-RELUIS 2014–2018 Grant - Research line 6:
  “Seismic isolation and dissipation”
- DPC-RELUIS 2019-2021 Grant - WP15:
  “Contribution to codes for isolated and dissipative structures”
- DPC-RELUIS 2022-2024 Grant - WP15:
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  Ing. Gilberto Dallavalle (Studio Ceccoli, Bologna)
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  - Michele Palermo, Luca Landi
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- My past and current Graduation Thesis students at UNIBO:
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  - Simone Bernardelli
  - Paolo Catti
  - Luca D’Alonzo
  - Camilla Rita Valente

Questions?

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Extra slides

Direct-five step procedure for existing buildings

“Direct five-step procedure” per edifici esistenti

- Per gli edifici esistenti progettati per soli carichi verticali (spesso caratterizzati da indici di sicurezza sismica C/D attorno a 0.20-0.30), in generale, l’introduzione di un sistema di smorzatori viscosi di interpiano (tipo **SPD**) potrebbe essere non sufficiente per un “*adeguamento*” sismico completo in grado di mantenere gli elementi strutturali in campo elastico (visto che la riduzione max della domanda è attorno al 50%)

- Per arrivare ad un “*adeguamento*” e per non fermarsi solo ad un “*miglioramento*”, potrebbe risultare utile affidarsi quindi in parte alla duttilità disponibile (dissipazioni isteretiche associate a danneggiamento elementi strutturali)

- Ma la normativa consente di accoppiare le due dissipazioni solo con Analisi Non Lineari Dinamiche ed impone il confronto con Analisi Modale con Spettro di Risposta:

7.3.4.1 ANALISI NON LINEARE DINAMICA

L’analisi non lineare dinamica consiste nel calcolo della risposta sismica della struttura mediante integrazione delle equazioni del moto, utilizzando un modello non lineare della struttura e le storie temporali del moto del terreno definite al § 3.2.3.6. Essa ha lo scopo di valutare il comportamento dinamico della struttura in campo non lineare, consentendo il confronto tra duttilità richiesta e duttilità disponibile allo *SLC* e le relative verifiche, nonché di verificare l’integrità degli elementi strutturali nei confronti di possibili comportamenti fragili.

L’analisi non lineare dinamica deve essere confrontata con un’analisi modale con spettro di risposta di progetto, al fine di controllare le differenze in termini di sollecitazioni globali alla base della struttura.

Nel caso delle costruzioni con isolamento alla base l’analisi dinamica non lineare è obbligatoria quando il sistema d’isolamento non può essere rappresentato da un modello lineare equivalente, come stabilito nel § 7.10.5.2. Gli effetti torsionali sul sistema d’isolamento sono valutati come precisato nel § 7.10.5.3.1, adottando valori delle rigidezze equivalenti coerenti con gli spostamenti risultanti dall’analisi. In proposito si può fare riferimento a documenti di comprovata validità.

"Direct five-step procedure" per edifici esistenti

$$\eta = \frac{\text{risp. plast. smorz.}}{\text{risp. elast. non smorz.}} = \left(\frac{\text{risp. plast. smorz.}}{\text{risp. elast. smorz.}}\right) \cdot \left(\frac{\text{risp. elast. smorz.}}{\text{risp. elast. non smorz.}}\right) =$$

$$HP)\ \frac{\text{risp. plast. smorz.}}{\text{risp. elast. smorz.}} = \frac{\text{risp. plast. non smorz.}}{\text{risp. elast. non smorz.}} = \eta_q = \frac{1}{q} \rightarrow \eta_q$$

$$\eta_\xi = \sqrt{\frac{10}{5+\xi}}$$

PROPOSTA: $\eta = \eta_\xi \cdot \eta_q$

Fattore di riduzione totale (**da applicare allo spettro elastico (ξ=5% e q=1)**) = Livello di progetto per gli elementi strutturali

Aliquota del fattore di riduzione dovuta agli effetti dissipativi degli **smorzatori viscosi**

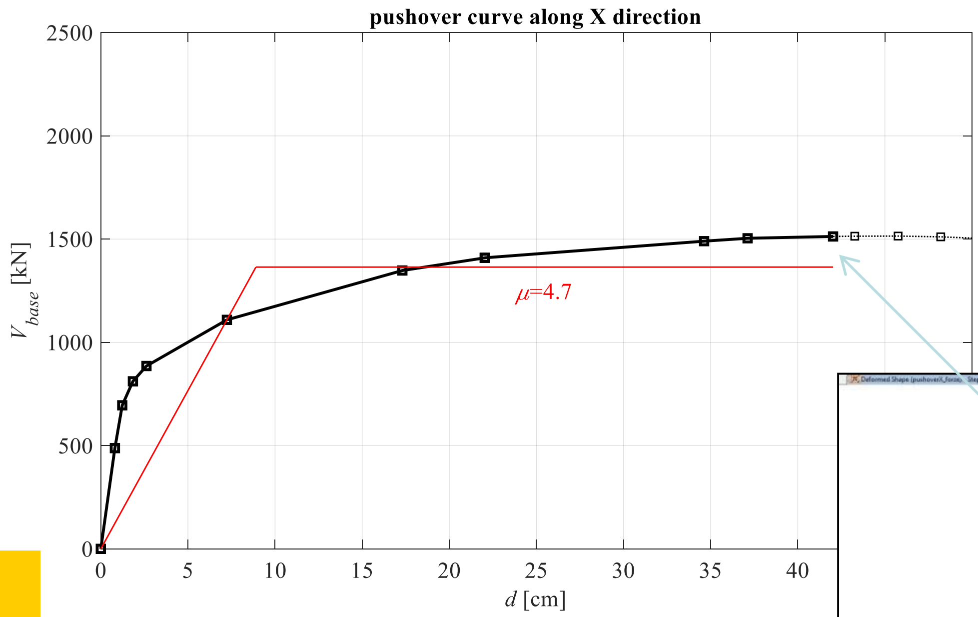
Aliquota del fattore di riduzione dovuta alla **duttilità disponibile**

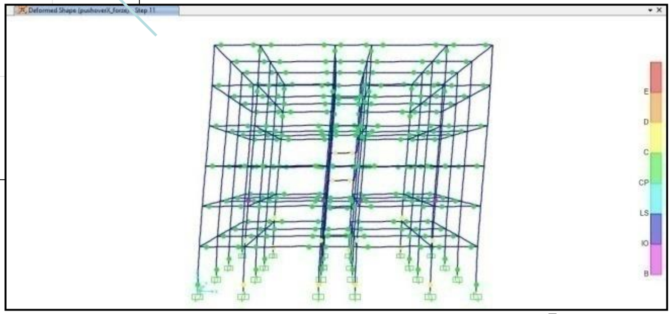
	Edifici Nuovi	Edifici Esistenti
η	**Da definire** sulla base dei dati di progetto e del tasso di lavoro che si vuole attribuire agli elementi strutturali.	**Definito** sulla base delle **armature** e delle **dimensioni geometriche** degli elementi strutturali (es. M_{Rd}) e l'azione sismica di progetto (es. M_{Ed}).
η_q	**Da definire** sulla base della tipologia di struttura e i valori del fattore di comportamento *q* dati dati nella Norma.	**Definito** sulla base della reale duttilità della struttura, identificabile mediante la sua **curva di capacità.**
η_ξ	**Da definire** per **progettare il sistema di dissipazione**.	**Da definire** per **progettare il sistema di dissipazione**.

"Direct five-step procedure" per edifici esistenti

ad esempio: flessione nei pilastri o taglio alla base

$$\bar{\eta} = \frac{\text{target seismic demand (i.e. capacity/strength)}}{\text{actual seismic (elastic) demand with no dampers}} = \frac{\overline{M_{Ed,\xi}}(=M_{Rd})}{M_{Ed,\xi=5\%}} = 0.25$$





$$\bar{\eta} = \zeta_E = \frac{C}{D}$$

$$\mu_{disp} = 4.7$$

↓

$$HP) \quad q = \mu$$

$$q_{\max,disp} > q_{NTC,esistenti}$$

$$q_{NTC,esistenti} = 1.5 \div 3.0$$

“Direct five-step procedure” per edifici esistenti

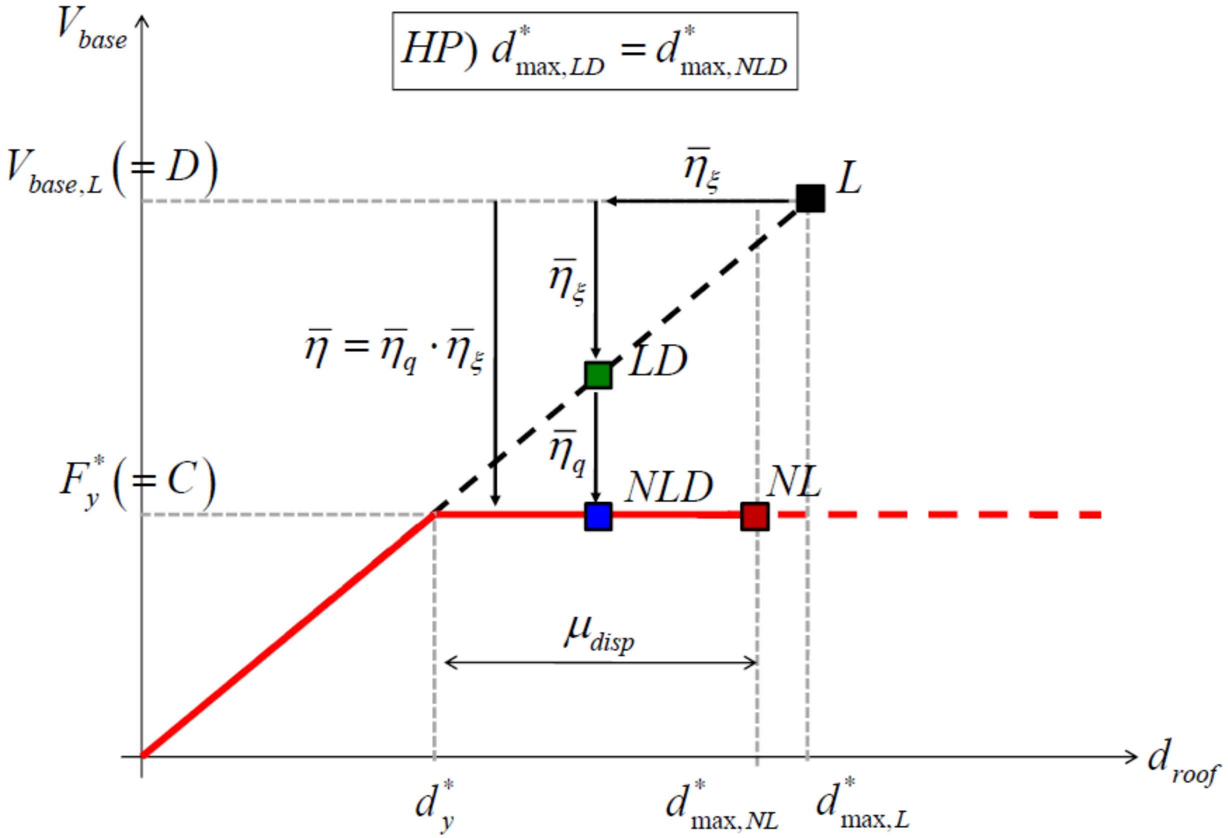


Figura I – Illustrazione dell'obiettivo prestazionale, rappresentato dal punto di performance identificato dal quadratino blu (NLD). NL = risposta non-lineare della struttura esistente allo stato di fatto. NLD = risposta non-lineare della struttura esistente con smorzatori. L = risposta della struttura lineare equivalente. LD = risposta della struttura lineare equivalente con smorzatori.

"Direct five-step procedure" per edifici esistenti

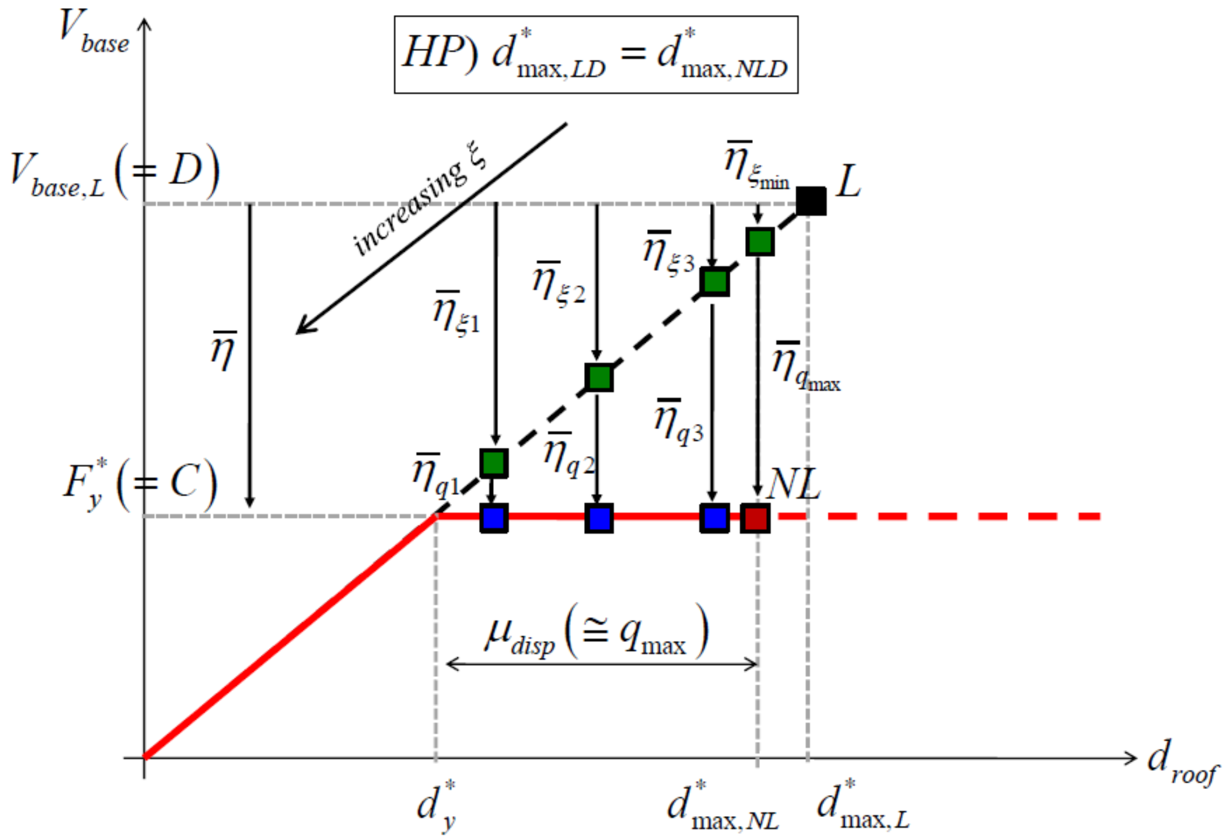


Figura II – Illustrazione delle strategie progettuali, basate su una ponderata ripartizione del fattore di riduzione della risposta sismica tra quotaparte dovuta a dissipazione viscosa e quotaparte dovuta a dissipazione isteretica.

"Direct five-step procedure" per edifici esistenti

Possibilità a disposizione del progettista:

$$\eta = \eta_q \cdot \eta_\xi$$

$$\eta = \frac{1}{q} \cdot \sqrt{\frac{10}{5+\xi}}$$

	η		η_q		η_ξ
$\xi = 15\% \rightarrow q = 2.9$	0.25	=	0.35	·	0.71
$\xi = 25\% \rightarrow q = 2.3$	0.25	=	0.43	·	0.58
$< q_{\max,disp}$	0.25	=	...	·	...

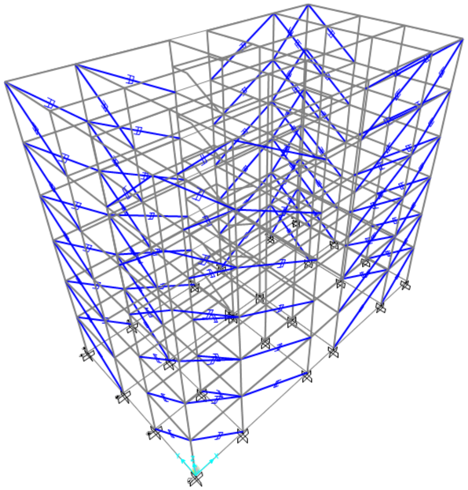
“Direct five-step procedure” per edifici esistenti

$$q = 2.3 \quad + \quad \xi = 25\%$$

$$\xi = \xi_{\text{intr}} + \xi_{visc}$$

$$= 5\% + 20\%$$

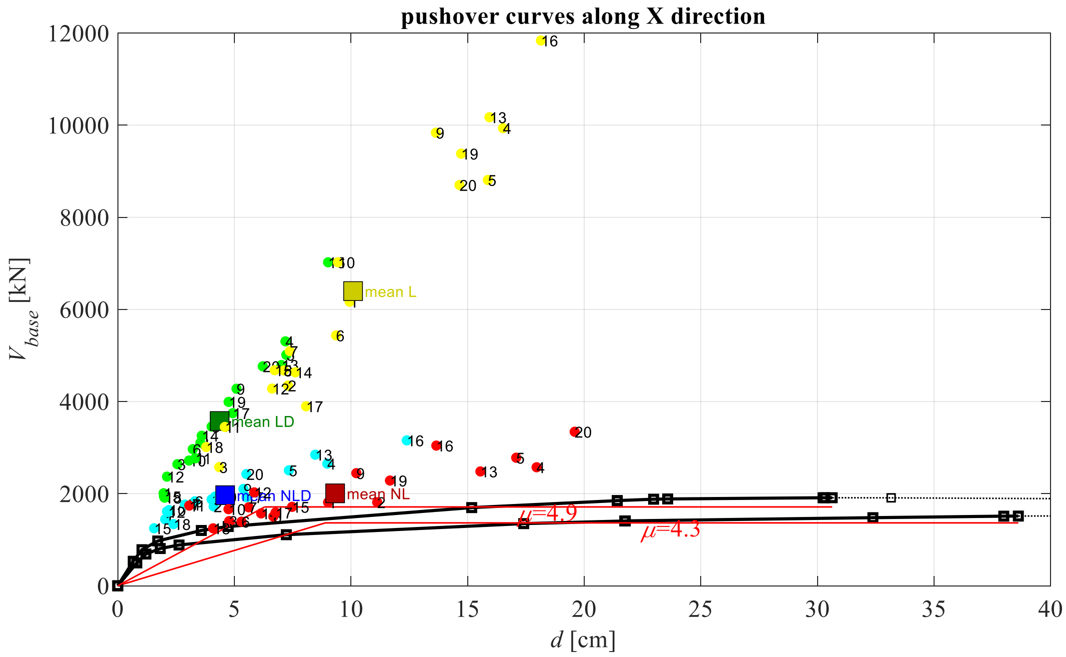
progettazione smorzatori viscosi con Direct Five-Step Procedure



		dir. long. (x)	dir. trasv. (y)	unità di misura
STEP 1				
smorzamento intrinseco	ξ intr =	0.05	0.05	
smorzamento viscoso	ξ visc =	0.20	0.20	
smorzamento totale	ξ tot =	0.25	0.25	
fattore riduzione risposta	η =	0.577	0.577	
STEP 2				
numero totale piani	N =	6	6	
peso totale struttura	Wtot =	16006	16006	kN
periodo fondamentale struttura	T1 =	0.795	0.693	s
pulsazione fondamentale struttura	ω1 =	7.90	9.06	rad/s
numero di smorzatori per piano	n =	8	8	
inclinazione smorzatori	θ =	43	39	°
coefficiente smorzamento lineare	cL =	4218	4284	kNs/m
rigidezza assiale	kaxial =	infinita	infinita	
STEP 3				
accelerazione spettrale	Sa(T1) =	0.244	0.280	g
coefficiente correttivo	M =	1.00	1.00	
velocità massima smorzatori lineari	vmax =	0.063	0.067	m/s
forza massima smorzatori lineari	FLmax =	267	288	kN
corsa massima pistone	smax =	0.80	0.74	cm
STEP 4				
esponente	α =	0.15	0.15	
coefficiente smorzamento non-lineare	cNL =	334	357	kN (s/m)^α
forza massima smorzatori non-lineari	FNLmax =	221	238	kN
rigidezza assiale minima	kaxial >	333216	388233	kN/m
STEP 5				
ESA1				
forza totale	Fh =	3904	4478	kN
ESA2				
forza struttura	Fstructure =	1292	1482	kN
numero telai con smorzatori	n frames =	2	4	
forza telaio	Fframe =	646	370	kN
numero specchiature con smorzatori nel telaio	n bays =	4	2	
forza specchiatura (singola reticolare)	Fbay =	161	185	kN
sforzo normale max nelle colonne				
	P1,max =	903	900	kN
	P2,max =	753	750	kN
	P3,max =	602	600	kN
sforzo normale max alla base singola colonna	Pbase =	903	900	kN

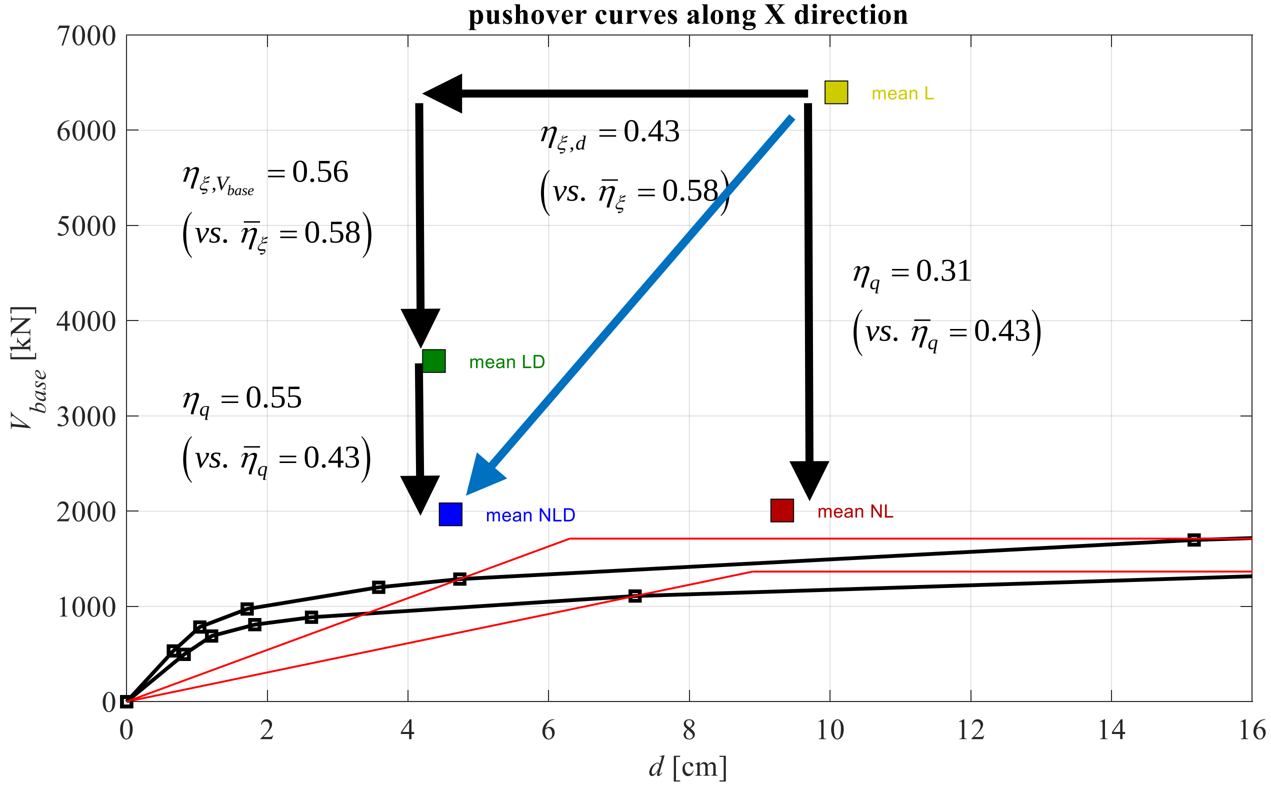
“Direct five-step procedure” per edifici esistenti

Verifica dell’efficacia della procedura:



“Direct five-step procedure” per edifici esistenti

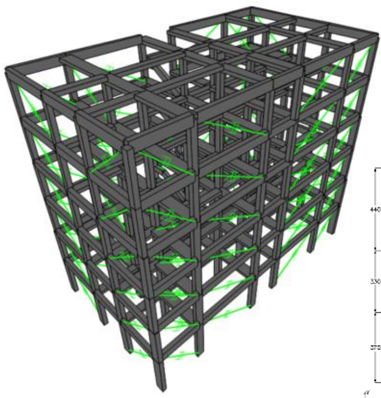
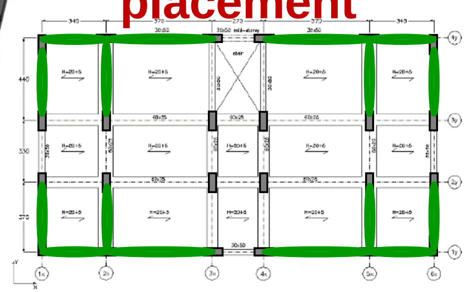
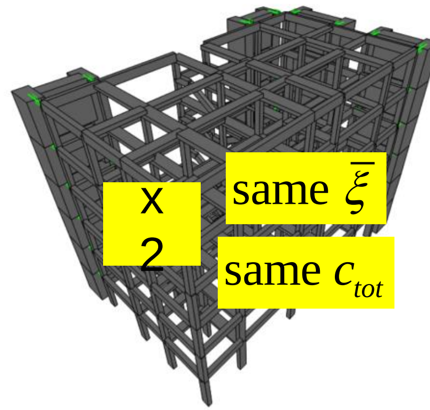
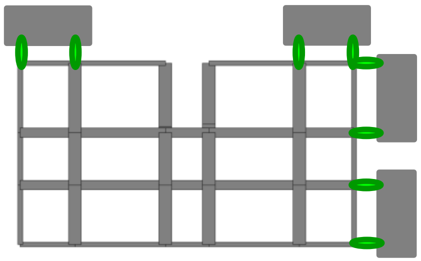
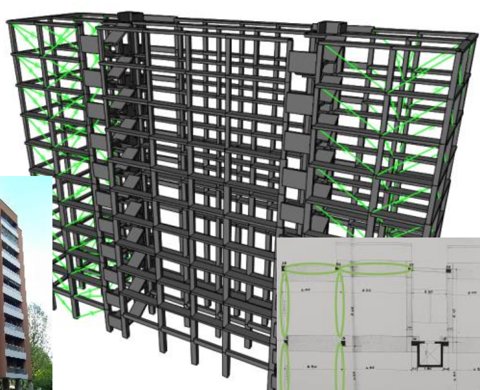
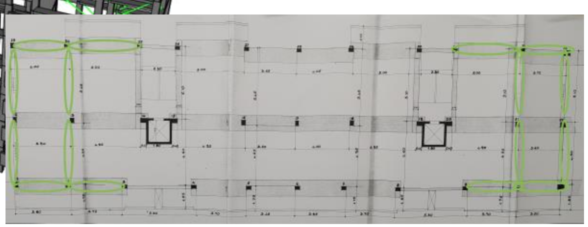
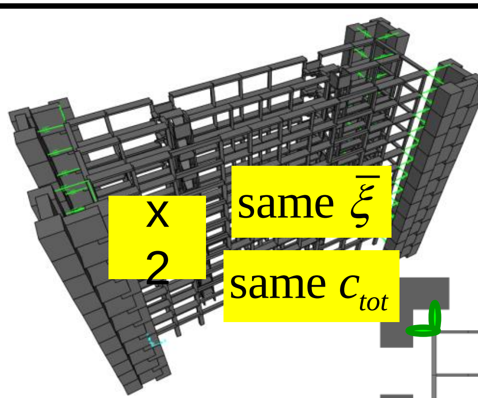
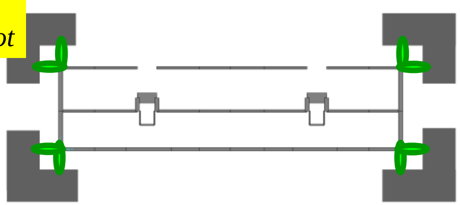
Verifica dell’efficacia della procedura:



Other applications
MPD and SPD

(RELUIS project 2022-2023)

CASE-STUDIES

	Interstorey viscous dampers	Dissipative links connected to external stiff towers
6-storey new building	 **SPD placement** 	 **MPD tower** 
11-storey existing building	  **SPD placement** 	 **MPD tower** 

THE «DIRECT FIVE-STEP PROCEDURE» (2018)

STEP 1: performance

$\bar{\eta} \rightarrow \bar{\xi}$

$\bar{\eta} = \sqrt{\frac{10}{5+\bar{\xi}}}$

$$c_{NL} = \bar{\xi}_{\text{visc}} \cdot \frac{2\pi}{T_1} \cdot \frac{W}{g} \cdot \left(\frac{N+1}{n}\right) \cdot \frac{1}{\cos^2\theta} \cdot \left(0.8 \cdot \frac{S_e\left(T_1, \bar{\eta}_\xi\right)}{2\pi/T_1} \cdot \frac{2}{N+1} \cdot \cos\theta\right)^{1-\alpha}$$

$$k_{axial} \geq 10 \cdot \bar{\xi}_{\text{visc}} \cdot \left(\frac{2\pi}{T_1}\right)^2 \cdot \frac{W}{g} \cdot \left(\frac{N+1}{n}\right) \cdot \frac{1}{\cos^2\theta}$$

VERIFICATION

STEP 5: non-linear TH analyses

structural response

2. Equivalent Static Analysis (ESA)

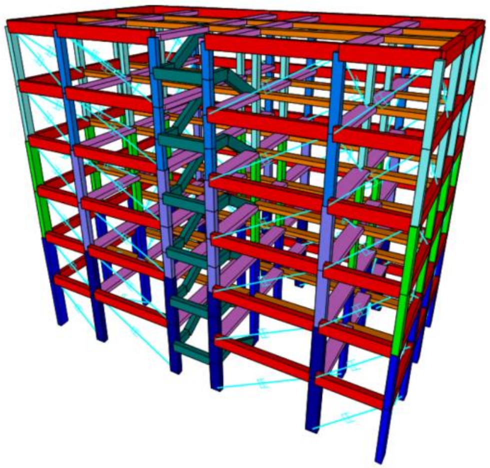
CASE 1A

6-storey new RC frame building + interstorey viscous dampers

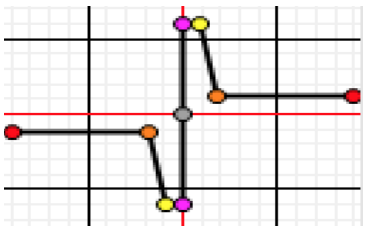
THE 6-STOREY STRUCTURE

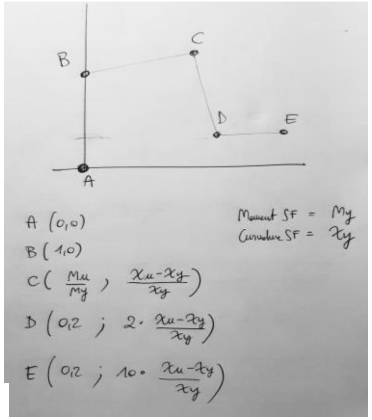
bars in the columns → guarantee at least 1% of the area of the concrete section

bars in the beams → guarantee at least 0.15% of the area of the concrete section, both in the tension area and in the compressed area, and in any case able to carry the maximum bending moments induced by a distribution of static vertical loads corresponding to the rare SLS combination



Flexural Plastic Hinges





Cerniere Platiche Trave						
	My	Xy	Mu	Xu	Mu/My	(Xu-Xy)/Xy
Trave a Ginocchio	133,82	0,00519	138,25	0,0671	1,03	11,9
Trave Perimetrale	253,96	0,00437	267,78	0,0597	1,05	12,7
Trave in spessore X	74,60	0,01201	73,92	0,0773	1	5,4
Trave in spessore Y	180,95	0,01310	178,76	0,0642	1	3,9

Cerniere Plastiche Pilastri M3																
Telaio 1	My	Xy	Mu	Xu	Ned	Mu/My	(Xu-Xy)/Xy	Telaio 2	My	Xy	Mu	Xu	Ned	Mu/My	(Xu-Xy)/Xy
P1	399,2	0,006231	426,1	0,016863	428,73	1,1	1,7	P1	423,2	0,005244	448,5	0,013086	552,4	1,1	1,5
P2	448,1	0,004176	468,6	0,010536	686,05	1,0	1,5	P2	442,5	0,004437	487,5	0,008487	851,55	1,1	0,9
P3	444,6	0,004146	465,9	0,010842	666,68	1,0	1,6	P3	446,5	0,004375	483,6	0,008891	812,9	1,1	1,0
P4	444,6	0,004146	465,9	0,010842	666,52	1,0	1,6	P4	446,5	0,004375	483,6	0,008891	812,13	1,1	1,0
P5	448,1	0,004176	468,6	0,010536	686,47	1,0	1,5	P5	442,5	0,004437	487,5	0,008487	851,6	1,1	0,9
P6	399,2	0,006231	426,1	0,016863	429,74	1,1	1,7	P6	423,2	0,005244	448,5	0,013086	552,65	1,1	1,5
P7	384,2	0,006994	410,6	0,020458	353,47	1,1	1,9	P7	403,9	0,006016	430,7	0,015959	452,99	1,1	1,7
P8	424,3	0,005202	449,5	0,01294	558,6	1,1	1,5	P8	451,1	0,004201	470,7	0,010294	702,15	1,0	1,5
P9	422,2	0,005277	447,7	0,0132	547,63	1,1	1,5	P9	445,5	0,004154	466,6	0,010763	671,58	1,0	1,6
P10	422,2	0,005277	447,7	0,0132	547,59	1,1	1,5	P10	445,5	0,004154	466,6	0,010763	671,79	1,0	1,6
P11	424,3	0,005202	449,5	0,01294	558,9	1,1	1,5	P11	451,1	0,004201	470,7	0,010294	702,16	1,0	1,5

The non-linear (elasto-brittle) **shear behaviour** of the structural elements is **not modelled.**

It is therefore implicitly **assumed that the shear strength of all the structural elements has been adequately increased by means of structural reinforcement interventions** (e.g. bands with fibre-reinforced polymeric materials) aimed at:

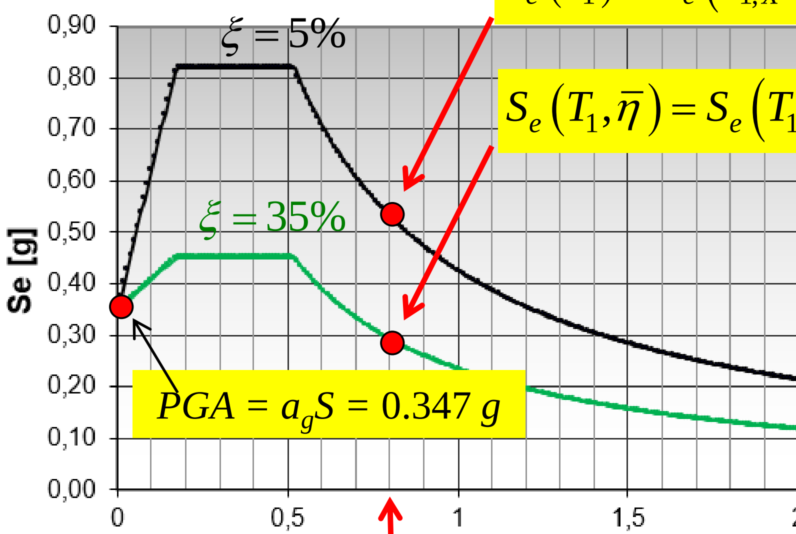
(i) guaranteeing a shear strength higher than the shear force corresponding to the formation of bending plastic hinges (bending capacity suitably increased with overstrength factors), according to the hierarchy of resistances

(ii) increasing the ductile capacity of the cross-section

SEISMIC INPUT

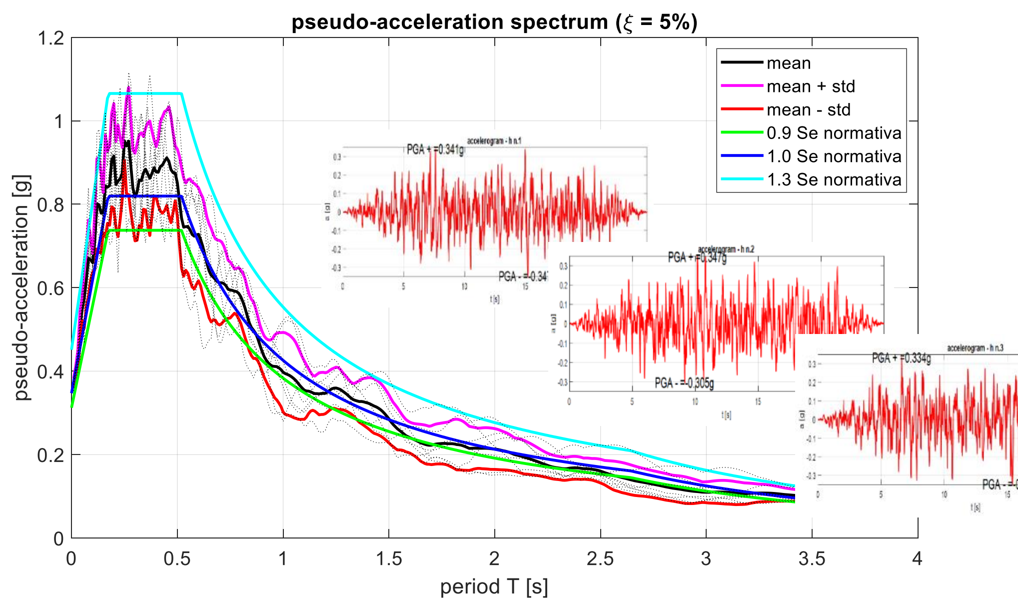
$$S_e(T_1) = S_e(T_{1,X} = 0.797s, \xi = 5\%) = 0.534g$$

$$S_e(T_1, \bar{\eta}) = S_e(T_{1,X} = 0.797s, \bar{\xi}_{tot} = 35\%) = 0.294g$$

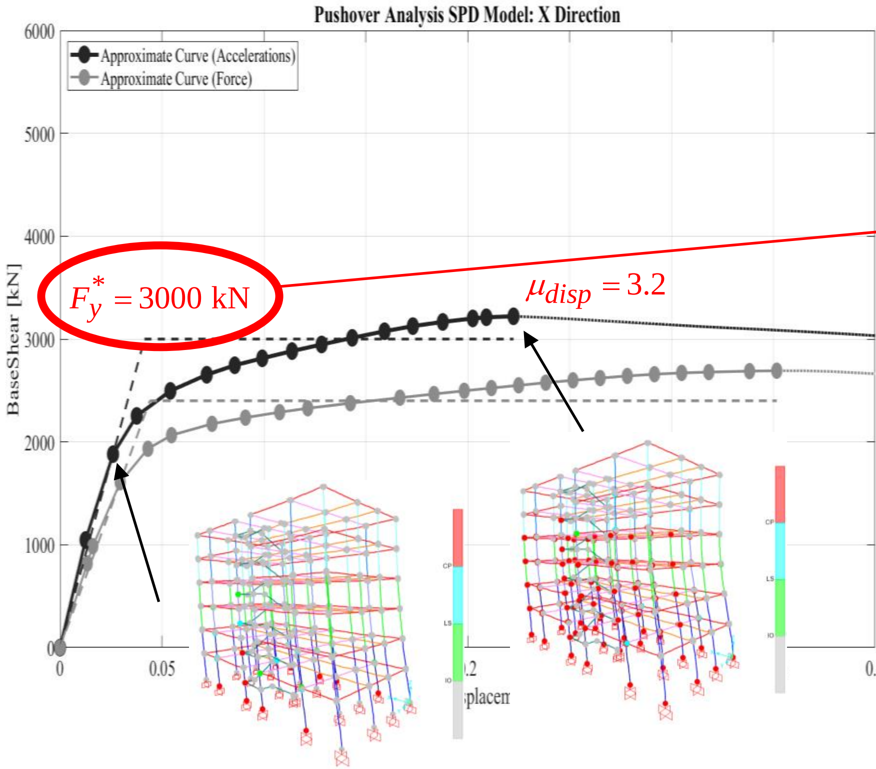


$$PGA = a_g S = 0.347\ g$$

$$T_{1,X} = 0.797\ s$$



STARTING POINT: CAPACITY-DEMAND RATIO (X Direction)

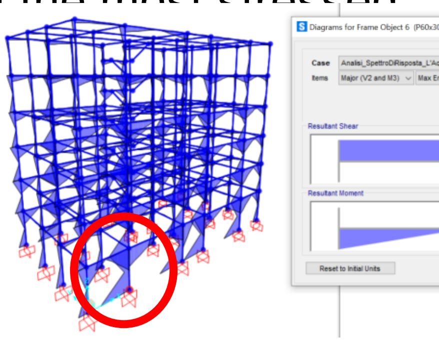


- at the global response level of the entire structure in terms of base shear - top displacement curve

$$V_{base} \cong \lambda \cdot \frac{W_{tot}}{g} \cdot S_a(T_1) = 0.85 \cdot \frac{16600 \text{ kN}}{g} \cdot 0.534g \cong 7500 \text{ kN}$$

$$\left.\frac{C}{D}\right|_{global} \cong \frac{3000}{7500} \cong 0.4$$

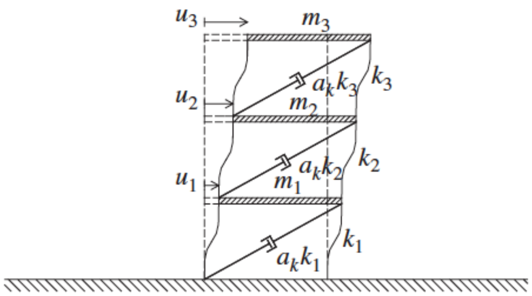
- at the local response level (e.g., bending moment, shear force) of the most stressed structural element (e.g. beam).

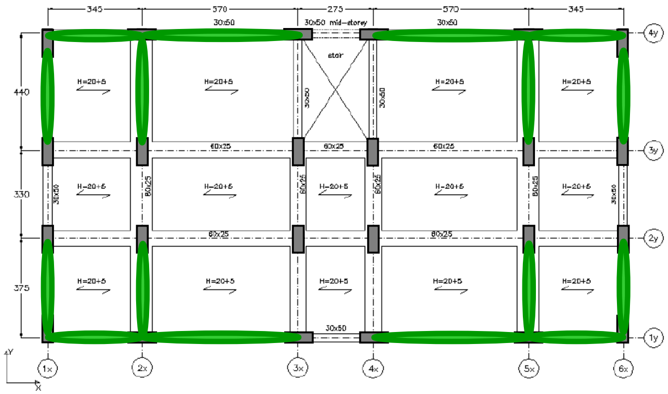


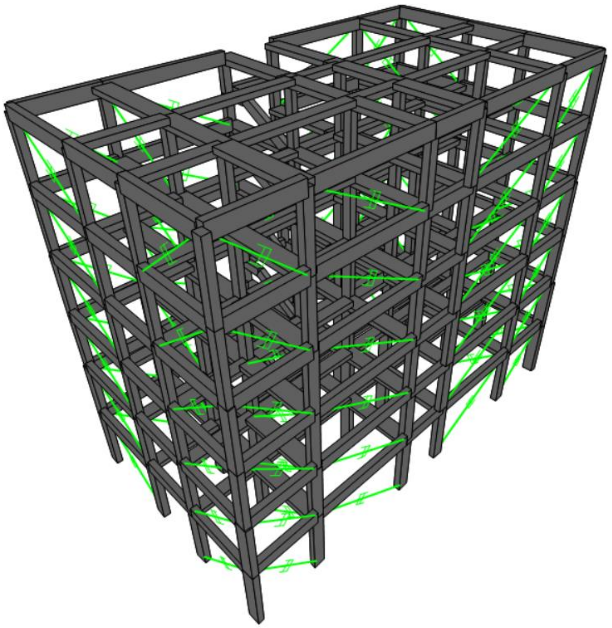
$$\left.\frac{C}{D}\right|_{local} = \frac{M_{Rd}}{M_{Ed}} \cong 0.25$$

CASE 1A

6-storey new RC frame building + interstorey viscous dampers







18.65 m

DESIGN STRATEGIES

The design strategy is based on the dimensioning of our dampers in order to achieve the target. Moreover, is also possible to taking into account the weighted coupling of hysteretic dissipation (ductility available in the structural elements). The starting point is represented by:

$$\left.\frac{C}{D}\right|_{global} \cong \frac{3000}{7500} \cong 0.4$$

$$\left.\frac{C}{D}\right|_{local} = \frac{M_{Rd}}{M_{Ed}} \cong 0.25$$

$$\bar{\eta} = \zeta_E = \frac{C}{D} = 0.5$$

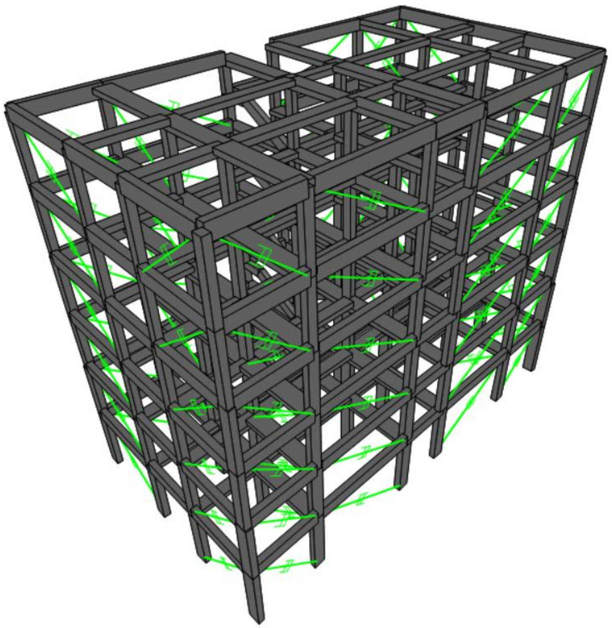
Assuming three different target demands, the reduction factors and the target damping ratios are evaluated, and three viscous dampers systems are obtained:

$$\xi_{tot} = 35\% \rightarrow \bar{\xi}_{visc} = 30\% \rightarrow c_{NL} = 544\ \text{kN} \cdot (\text{s/m})^{0.15}$$

$$\xi_{tot} = 25\% \rightarrow \bar{\xi}_{visc} = 20\% \rightarrow c_{NL} = 378\ \text{kN} \cdot (\text{s/m})^{0.15}$$

$$\xi_{tot} = 15\% \rightarrow \bar{\xi}_{visc} = 10\% \rightarrow c_{NL} = 224\ \text{kN} \cdot (\text{s/m})^{0.15}$$

DESIGN OF FLUID VISOUS DAMPERS: APPLICATION OF DIRECT FIVE STEP PROCEDURE FOR $\xi_{visc} = 30\%$



$$\xi_{tot} = \xi_{intr} + \xi_{visc} = 5\% + 30\%$$

		dir. long. (x)	dir. trasv. (y)	unità di misura
STEP 1				
smorzamento intrinseco	ξ intr =	0,05	0,05	
smorzamento viscoso	ξ visc =	0,30	0,30	
smorzamento totale di target	ξ tot =	0,35	0,35	
fattore riduzione risposta di target	η =	0,500	0,500	
STEP 2				
numero totale piani	N =	6	6	
peso totale struttura	Wtot =	16600	16600	kN
periodo fondamentale struttura	T1 =	0,797	0,704	s
pulsazione fondamentale struttura	ω1 =	7,88	8,93	rad/s
numero di smorzatori per piano	n =	8	8	
inclinazione smorzatori	θ =	36	38	°
coefficiente smorzamento lineare	cL =	5384	6461	kNs/m
rigidezza assiale	kaxial =	infinita	infinita	
STEP 3				
accelerazione spettrale	Sa(T1) =	0,294	0,333	g
coefficiente correttivo	M =	1,00	1,00	
velocità massima smorzatori lineari	vmax =	0,084	0,082	m/s
forza massima smorzatori lineari	FLmax =	454	529	kN
corsa massima pistone	smax =	1,07	0,92	cm
STEP 4				
esponente	α =	0,15	0,15	
coefficiente smorzamento non-lineare	cNL =	544	637	kN (s/m)^a
forza massima smorzatori non-lineari	FNLmax =	375	437	kN
rigidezza assiale minima	kaxial >	424327	576634	kN/m
STEP 5				
ESA1				
forza totale	Fh =	4877	5523	kN
ESA2				
forza struttura	Fstructure =	2421	2741	kN
numero telai con smorzatori	n frames =	2	4	
forza telaio	Fframe =	1210	685	kN
numero specchiature con smorzatori nel telaio	n bays =	4	2	
forza specchiatura (singola reticolare)	Fbay =	303	343	kN
sforzo normale max nelle colonne				
	P1,max =	1332	1631	kN
	P2,max =	1110	1359	kN
	P3,max =	888	1088	kN
sforzo normale max alla base singola colonna	Pbase =	1332	1631	kN

MODELS ANALYZED

LS = bare structure (without dampers) modeled as linear

NLS = bare structure (without dampers) modeled as non-linear with flexural plastic hinges

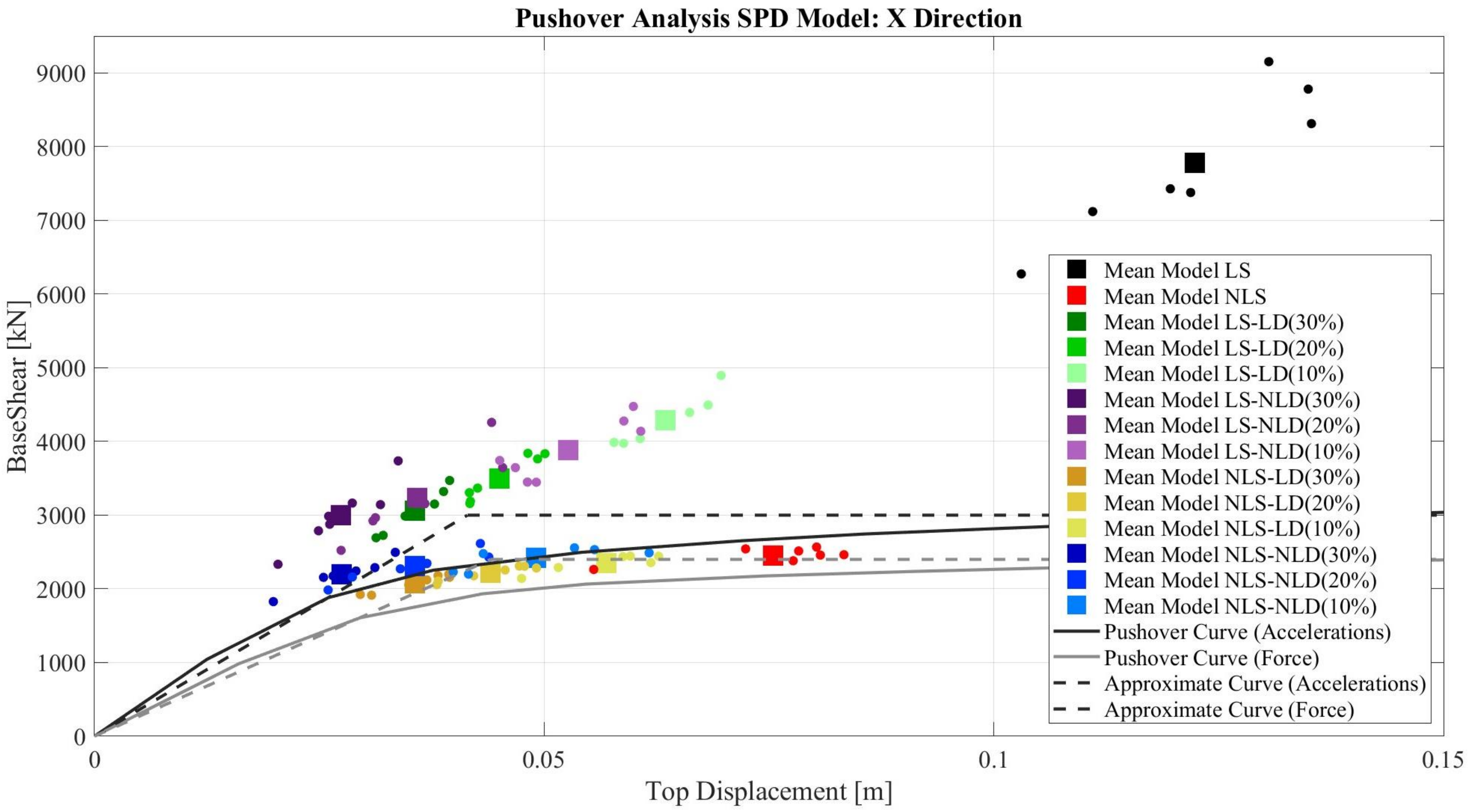
LS_LD = linear structure (modeled without the flexural plastic hinges) with linear dampers (designed with the direct five-step procedure)

LS_NLD = linear structure (modeled without the flexural plastic hinges) with non-linear viscous dampers (designed with the direct five-step procedure)

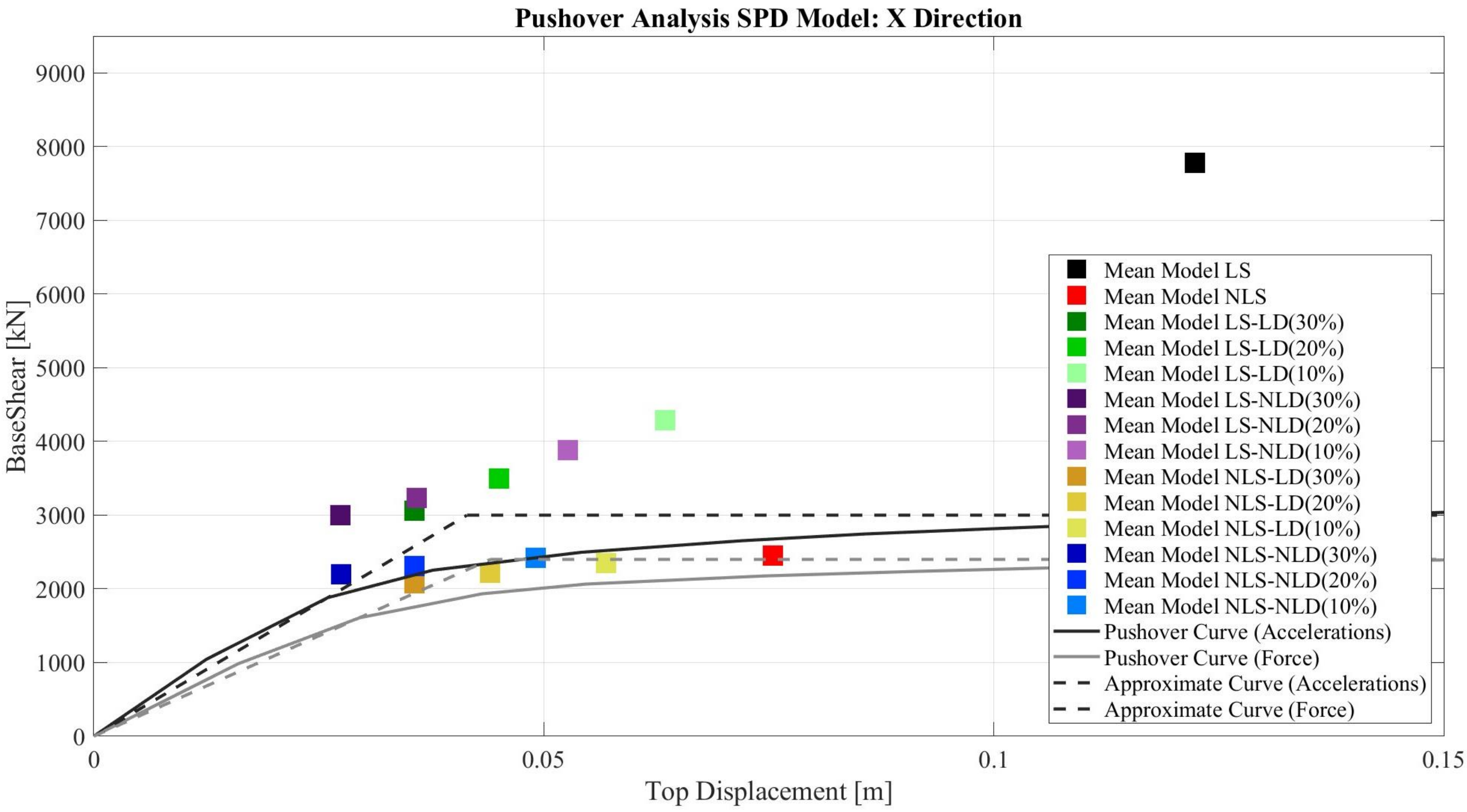
NLS_LD = non linear structure (modeled with the flexural plastic hinges) with linear dampers (designed with the direct five-step procedure)

NLS_NLD = non linear structure (modeled with the flexural plastic hinges) with non-linear viscous dampers (designed with the direct five-step procedure)

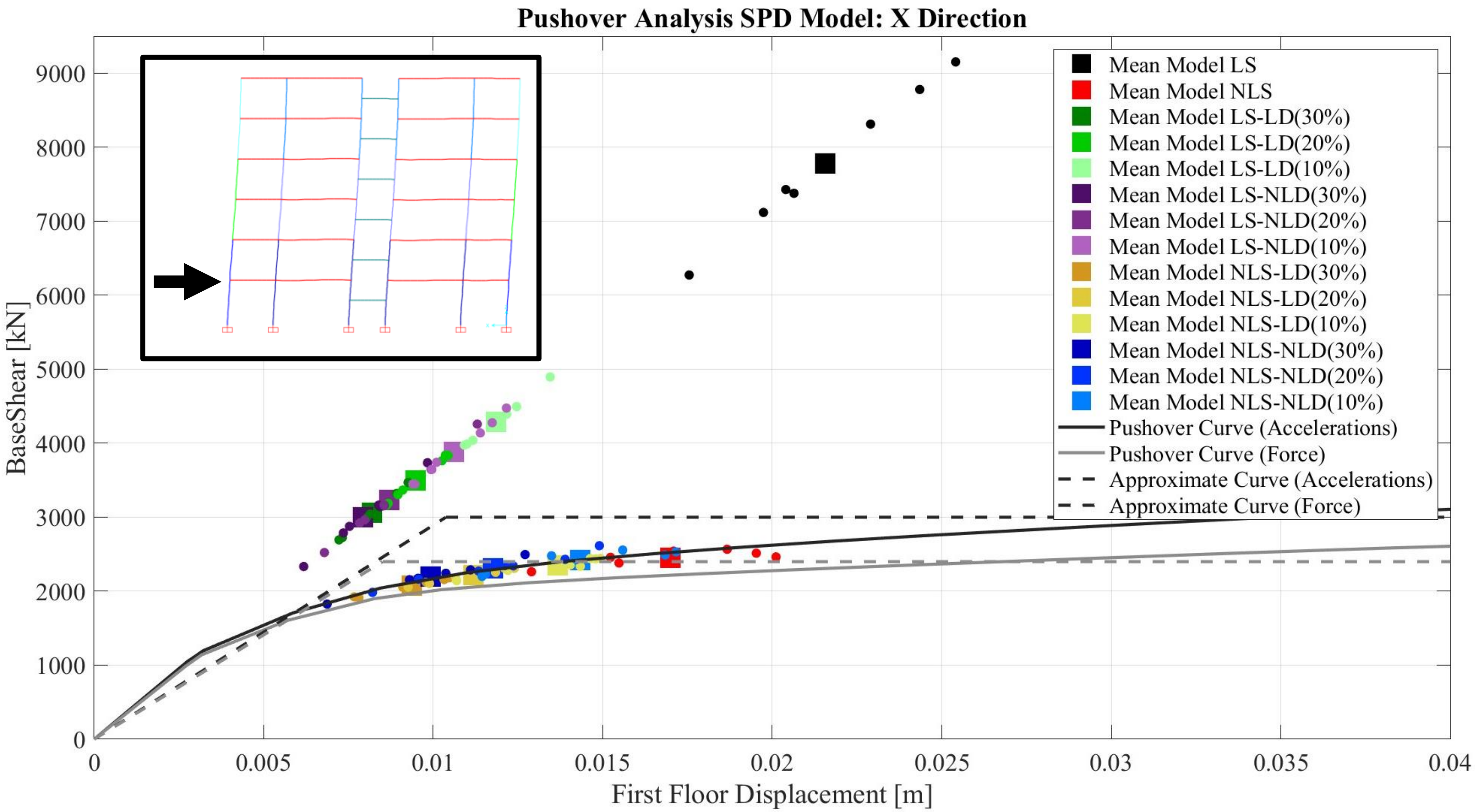
TIME-HISTORY RESULTS (X Direction)



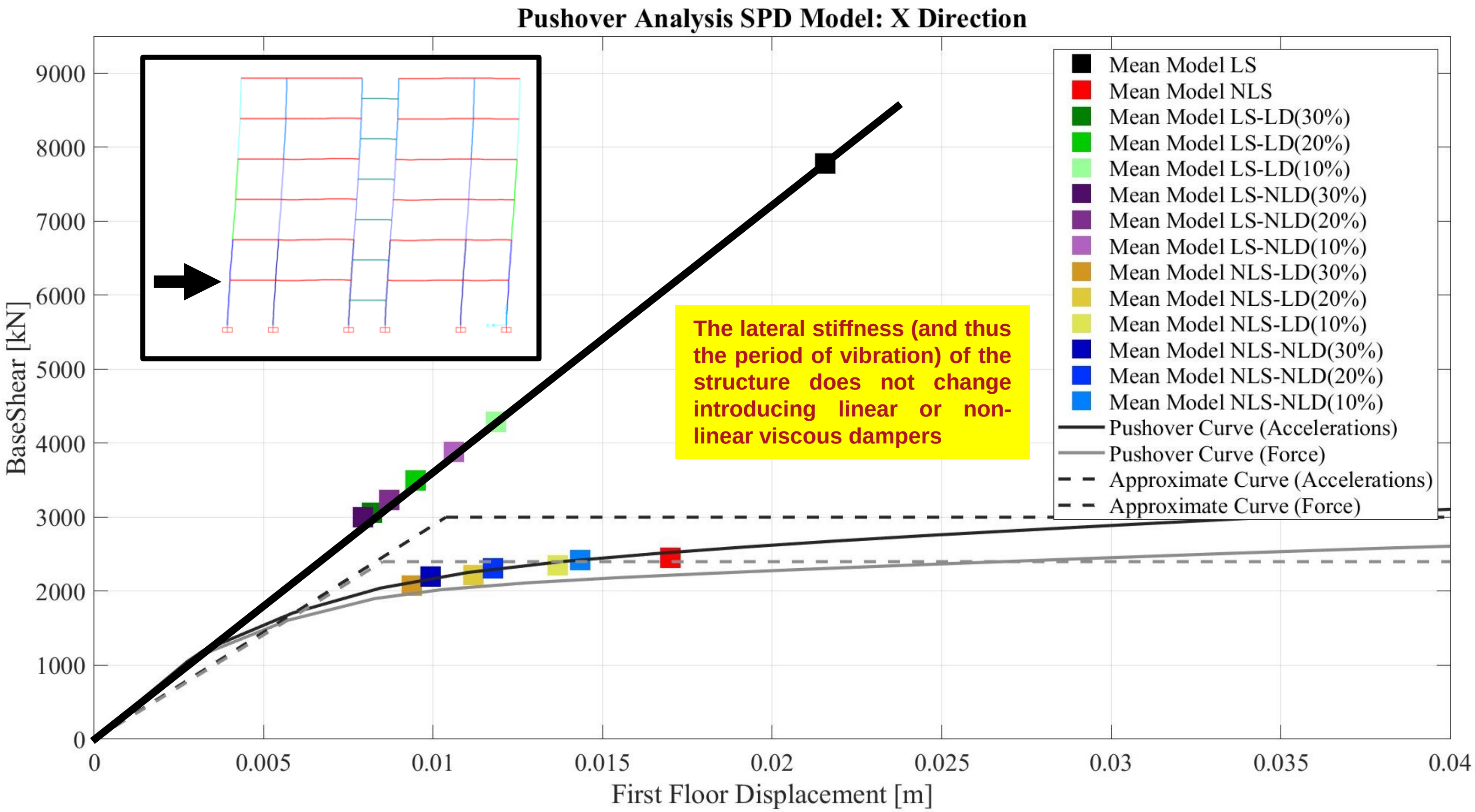
TIME-HISTORY RESULTS (X Direction)



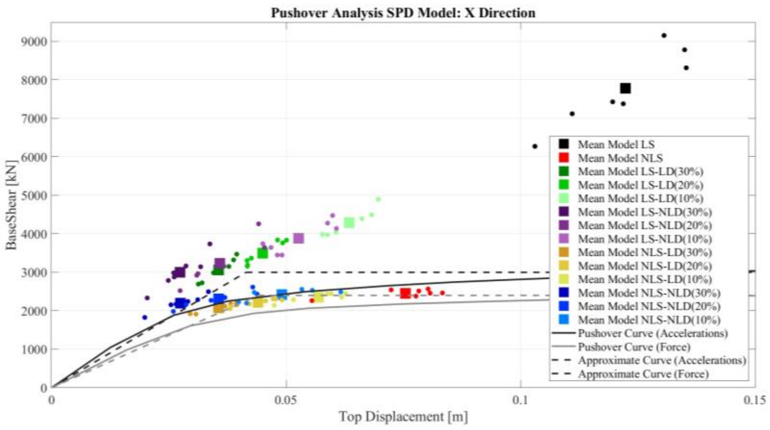
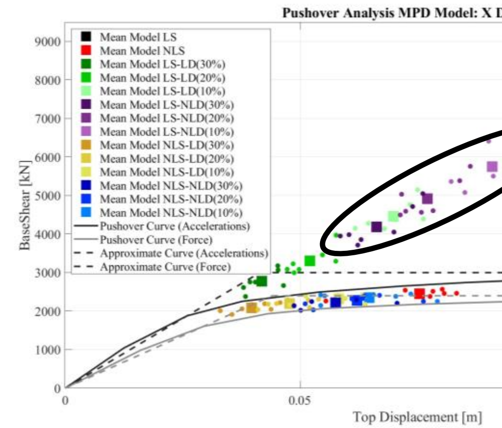
TIME-HISTORY RESULTS (X Direction)



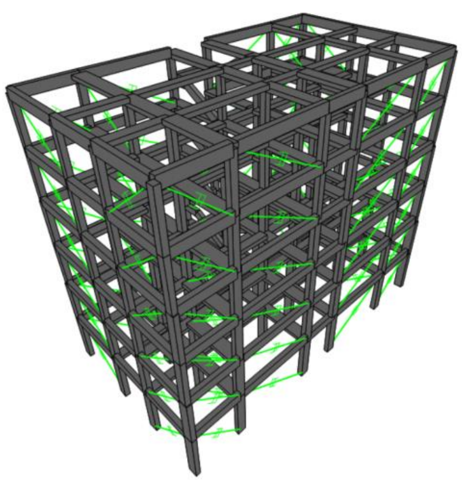
TIME-HISTORY RESULTS (X Direction)



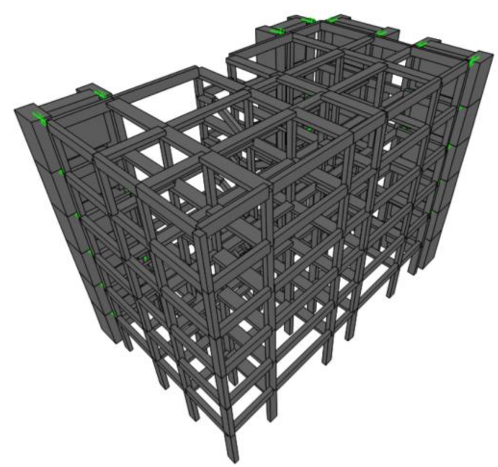
COMPARISON SPD-MPD with same target damping ratio

	Interstorey viscous dampers	Dissipative links connected to external stiff towers
6-storey new building		

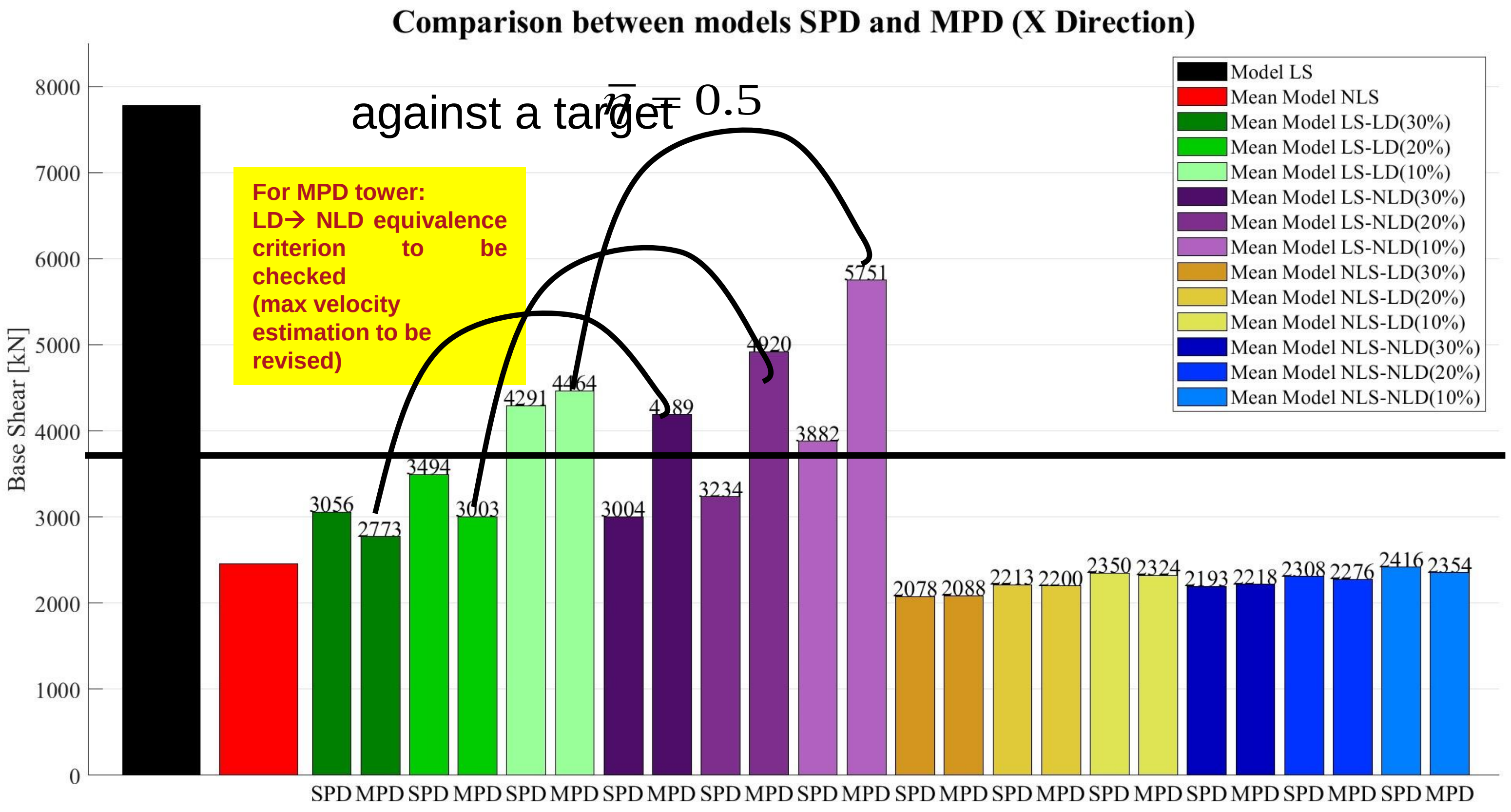
For MPD tower: LD→ NLD equivalence criterion to be checked (max velocity estimation to be revised)



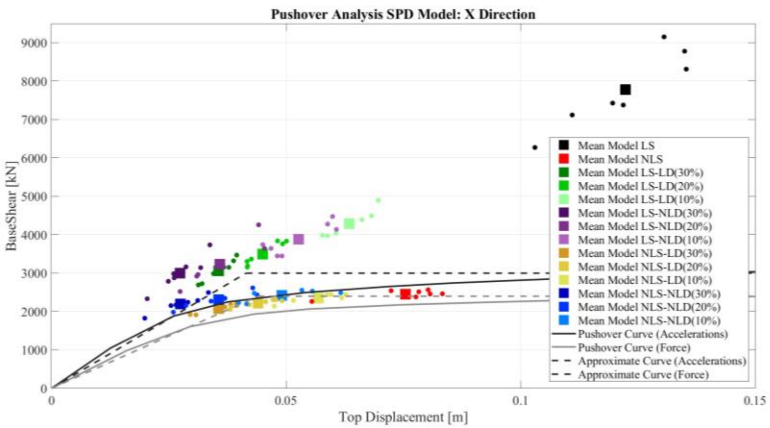
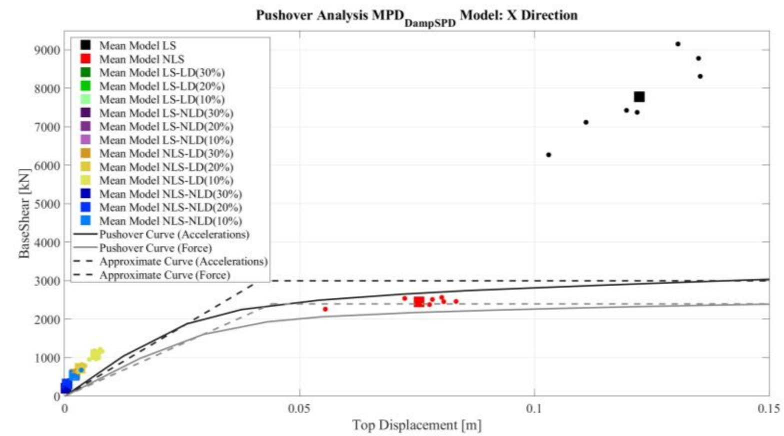
same $\bar{\xi}$

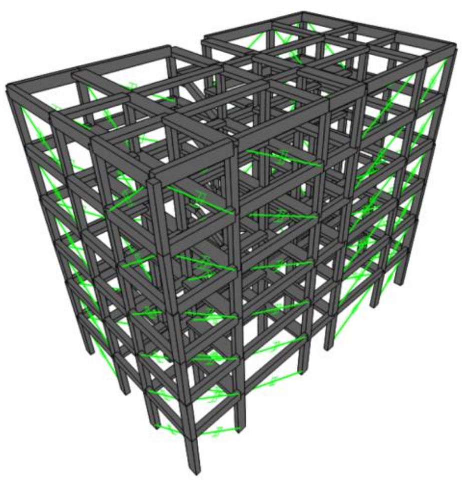


COMPARISON SPD-MPD with same target damping ratio

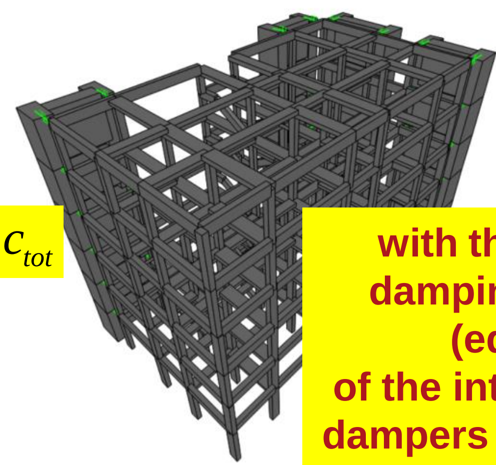


COMPARISON SPD-MPD with same total c

	Interstorey viscous dampers	Dissipative links connected to external stiff towers
6-storey new building		

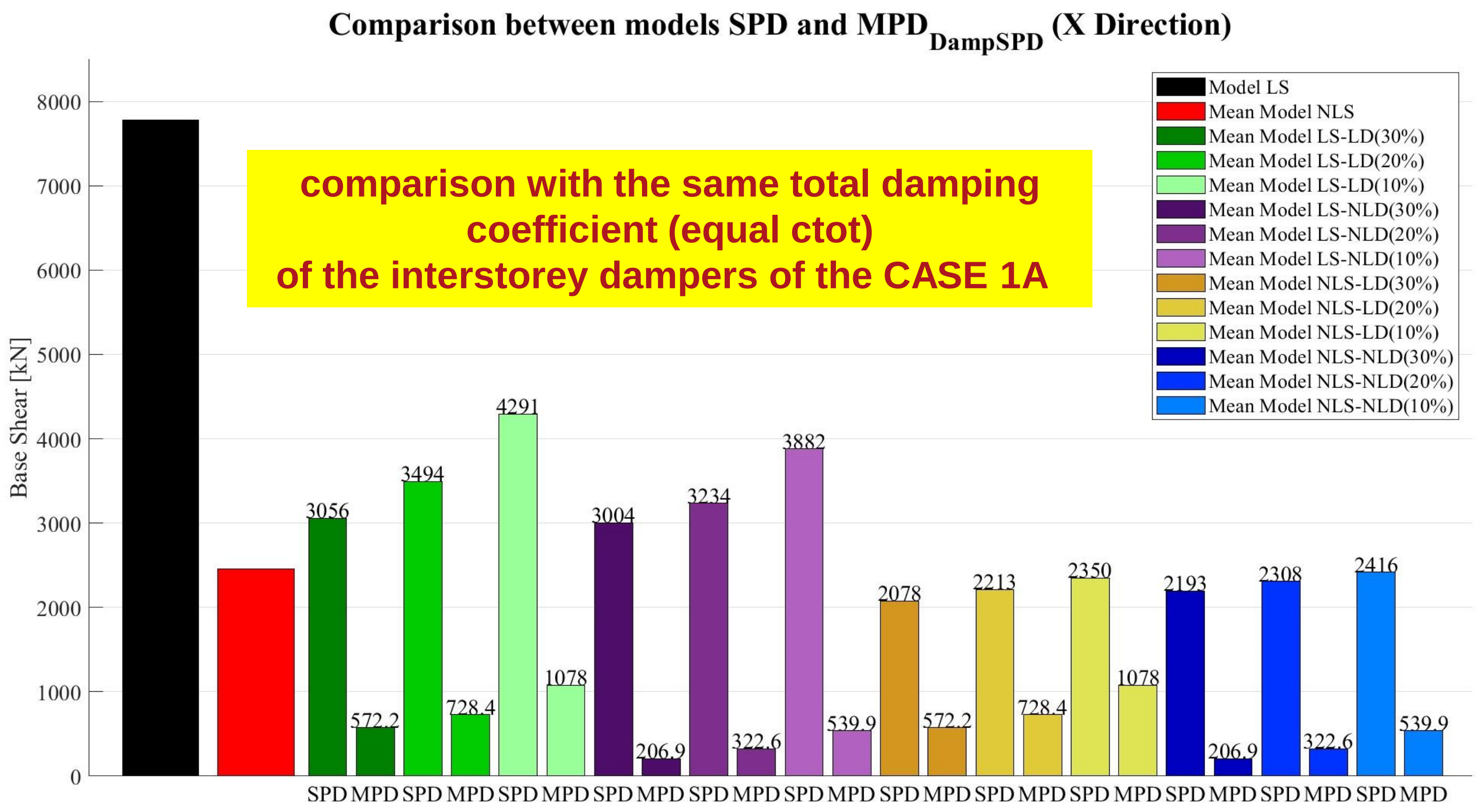


same c_{tot}



with the same total damping coefficient (equal ctot) of the interstorey dampers of the CASE 1A

COMPARISON SPD-MPD with same total c



comparison with the same total damping coefficient (equal ctot) of the interstorey dampers of the CASE 1A

CASE 2

11-storey existing RC frame building

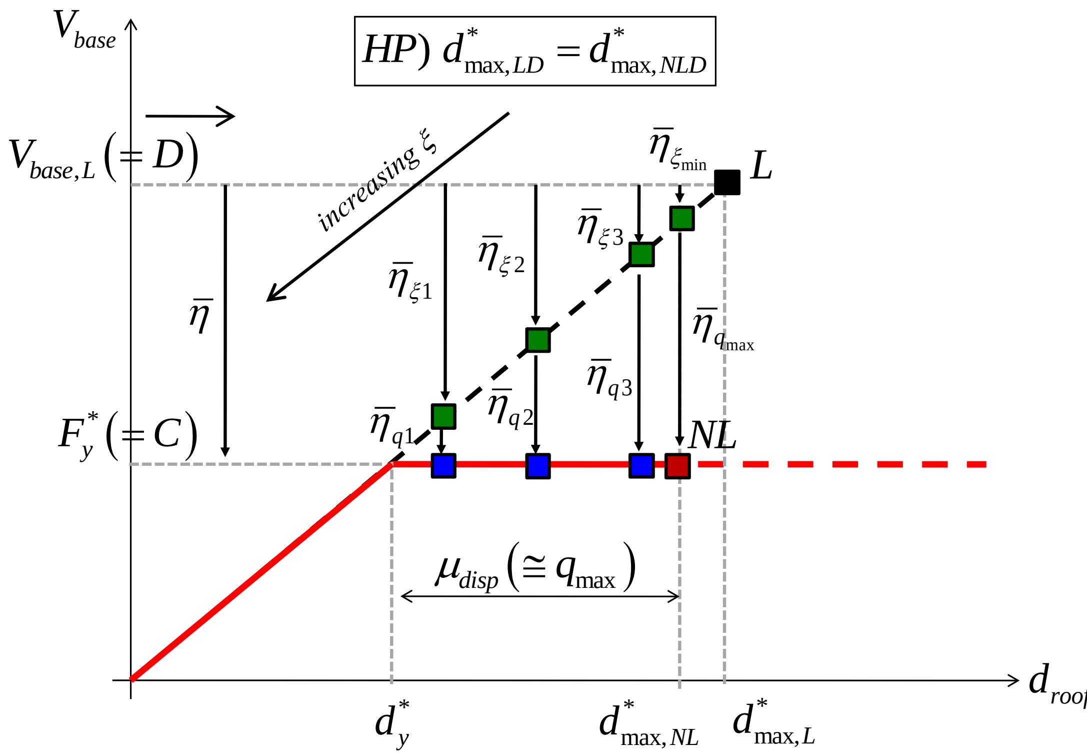
DIRECT FIVE-STEP PROCEDURE FOR EXISTING BUILDINGS: REVISION OF STEP 1

several strategies:

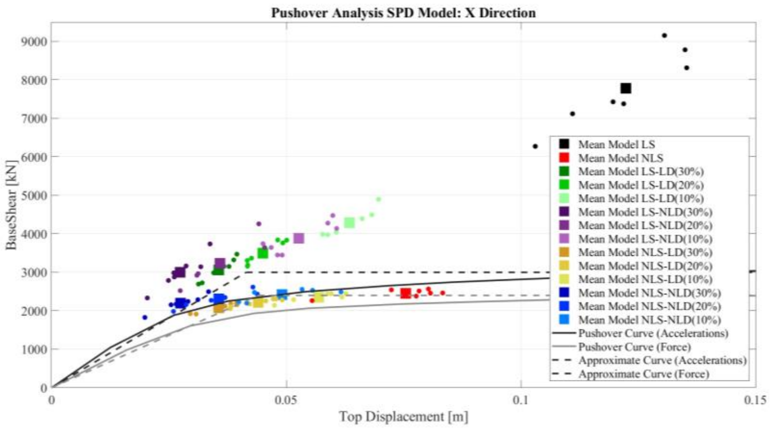
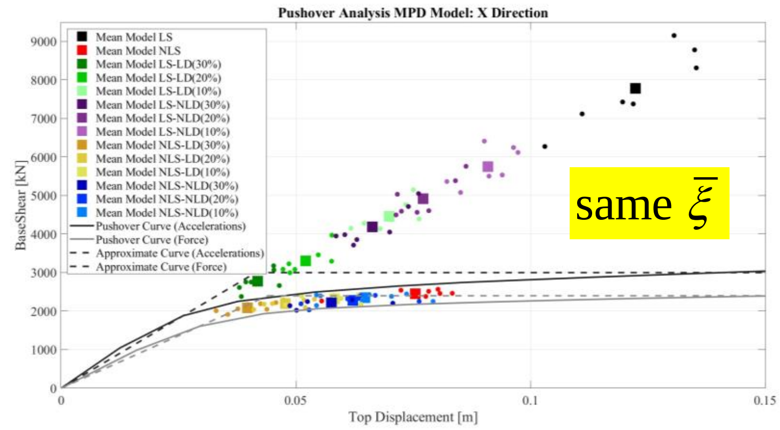
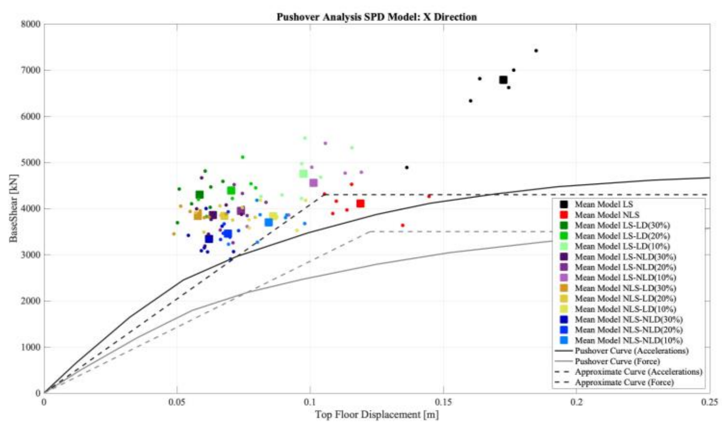
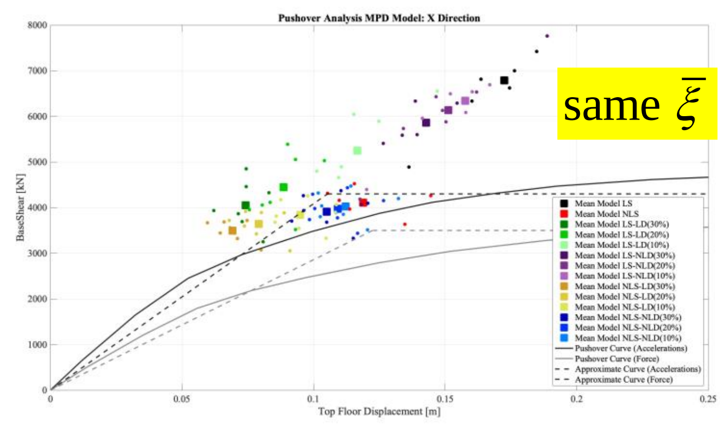
$$\bar{\eta} = \bar{\eta}_{q1} \cdot \bar{\eta}_{\xi 1}$$

$$\bar{\eta} = \bar{\eta}_{q2} \cdot \bar{\eta}_{\xi 2}$$

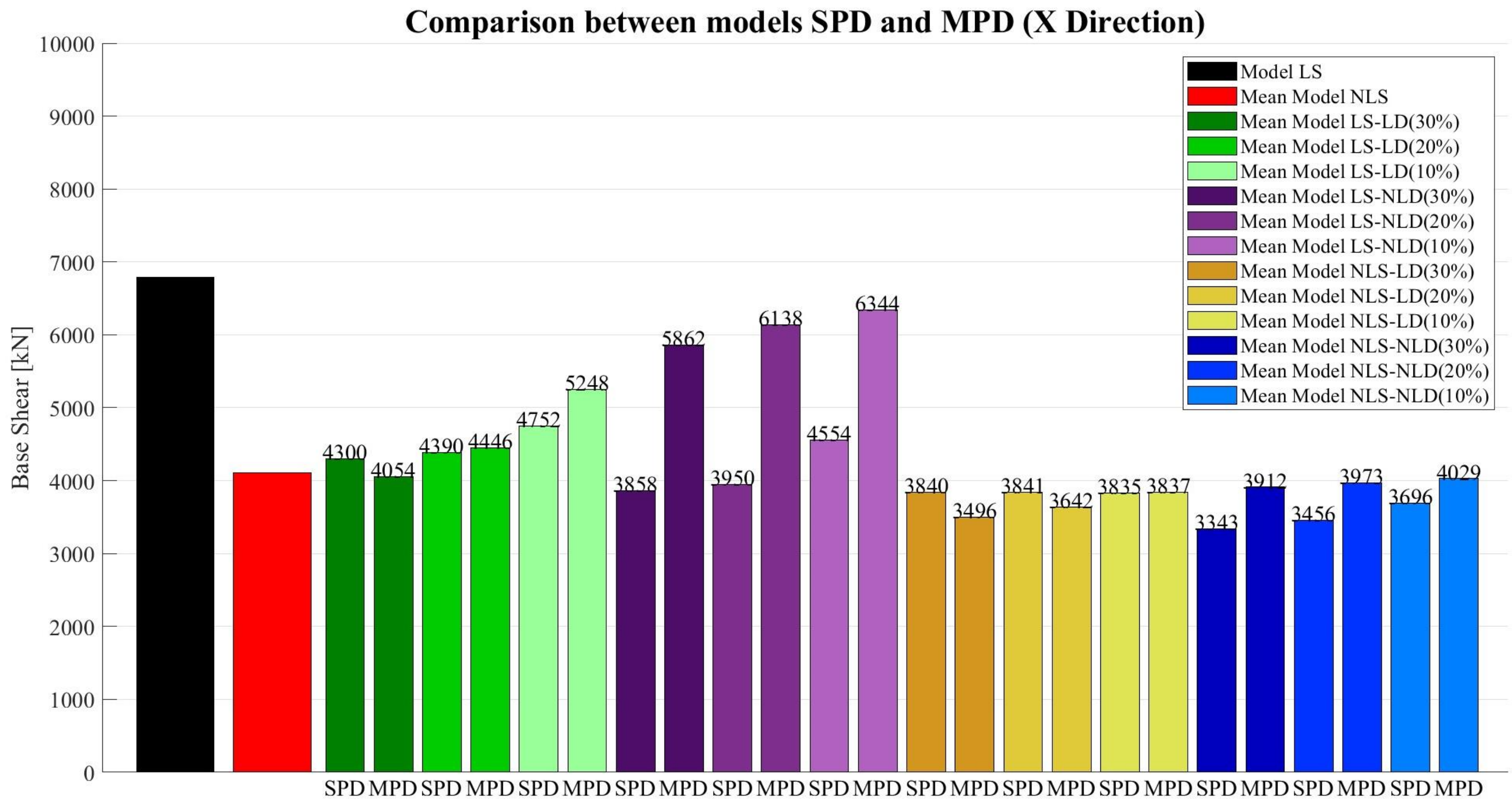
$$\bar{\eta} = \bar{\eta}_{q3} \cdot \bar{\eta}_{\xi 3}$$



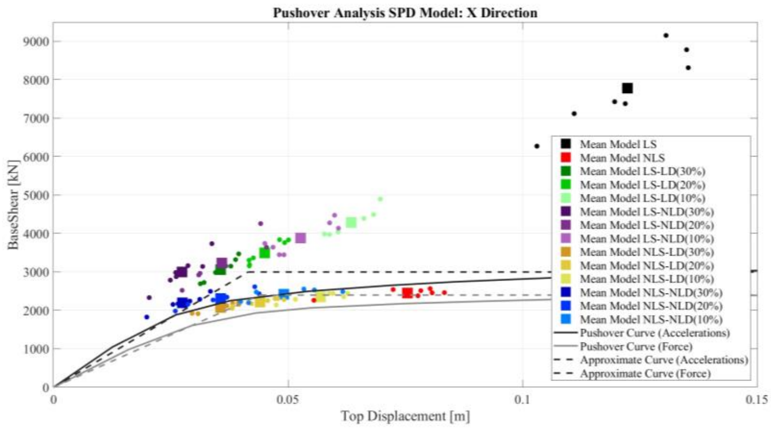
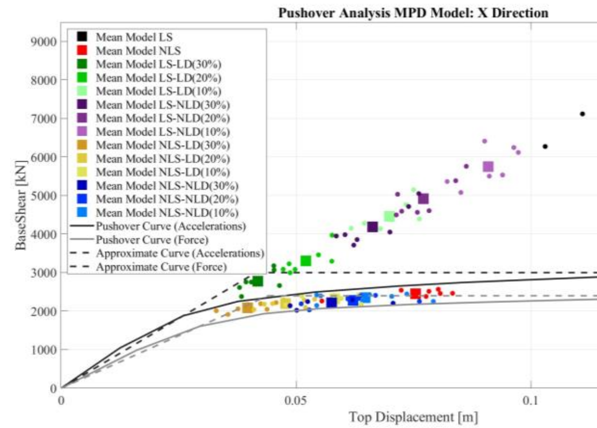
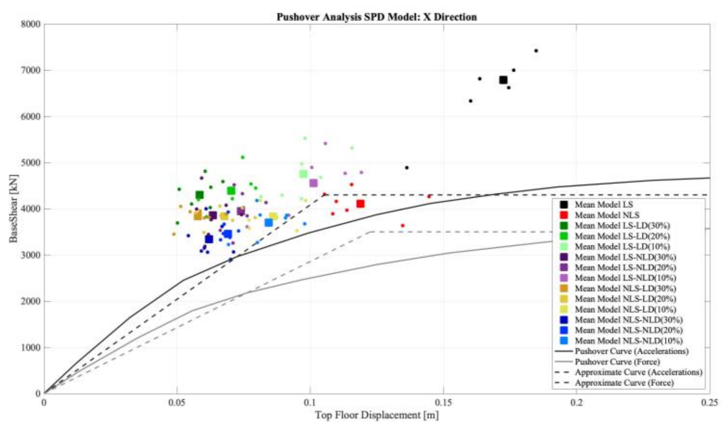
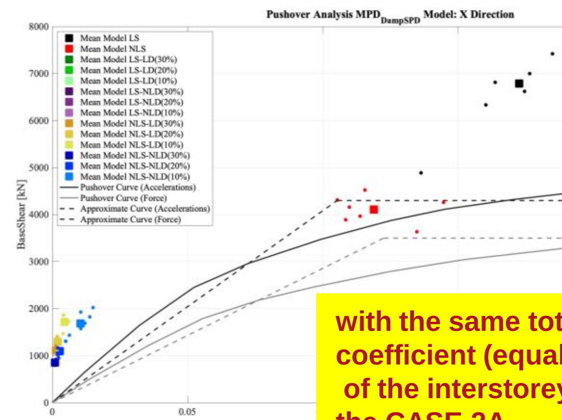
COMPARISON SPD-MPD with same target damping ratio

	Interstorey viscous dampers	Dissipative links connected to external stiff towers
6-storey new building		 same $\bar{\xi}$
11-storey existing building 		 same $\bar{\xi}$

COMPARISON SPD-MPD with same target damping ratio

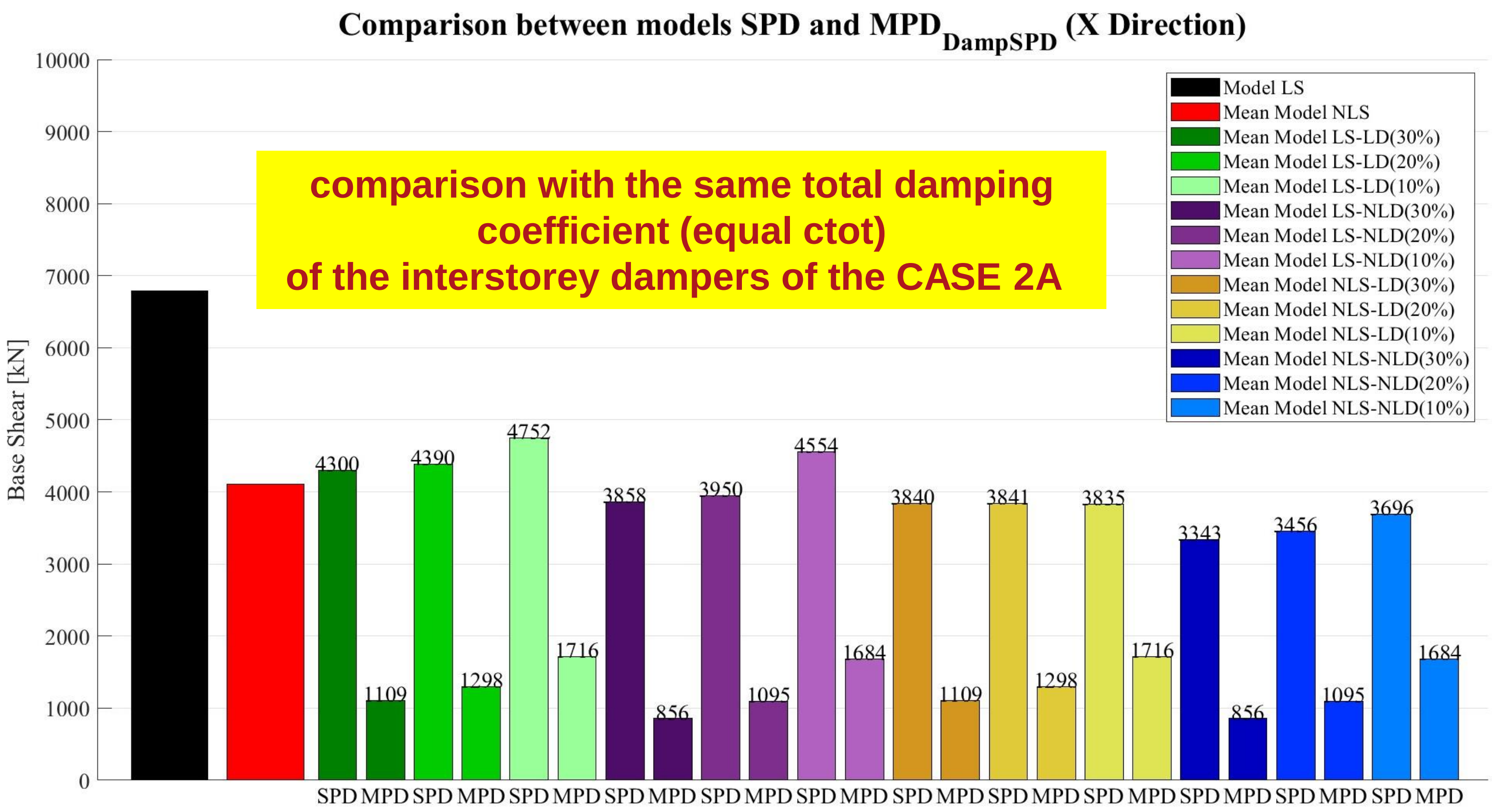


COMPARISON SPD-MPD with same total c

	Interstorey viscous dampers	Dissipative links connected to external stiff towers
6-storey new building		 same c_{tot}
11-storey existing building 		 same c_{tot}

with the same total damping coefficient (equal ctot) of the interstorey dampers of the CASE 2A

COMPARISON SPD-MPD with same total c



Thank you!

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