European Commission

EN 1998-3: Seismic assessment and retrofitting of existing buildings

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ELABORATION OF MAPS FOR CLIMATIC AND SEISMIC ACTIONS FOR STRUCTURAL DESIGN IN THE BALKAN REGION

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- In seismic regions, existing substandard buildings:
  Largest threat to human life & property.

- From cost-benefit point of view:
  Unless triggered by earthquake, change in use, etc., seismic retrofitting normally not worthwhile.

- Obstacle to upgrading, in addition to economic factors:
  - standards & guidelines are few and untried;
  - technical difficulty of design of retrofitting;
  - long disruption of occupancy and use of facility

- Problem technically more challenging for concrete than for masonry buildings:
  - Diversity: wider typology, continuous evolution of codes;
  - Short history of exposure to seismic hazard

PART I:

OVERVIEW OF EN1998-3:2005

Introduction
Seismic Assessment in EC8-Part 3
(including comments from application experience)
Seismic Retrofitting per EC8-Part 3 (Annex A)
An evolution plan for EC8-Part 3

Part 3 of EC8 (EN 1998-3:2005): Seismic assessment and retrofitting of buildings

- The only part in the whole set of 58 EN-Eurocodes that deals with existing structures
- 1st standard in Europe on seismic assessment and retrofitting of buildings – No experience in European practice w/ codified seismic assessment and retrofitting.

➢ Part 3 of EC8 is an experiment. Still unknown how it works in practice.

STRUCTURE OF EN 1998-3

- Normative part: General rules on:
  - Performance requirements & criteria (LSs),
  - Analysis methods & applicability conditions,
  - Format of verifications,
  - Information for assessment & implications, etc.

- All material-specific aspects:

  In 3 Informative (nonbinding) Annexes:
  - Concrete structures
  - Steel or composite structures
  - Masonry buildings

PERFORMANCE-BASED APPROACH:

- Assessment & Retrofitting for different Performance Levels ("Limit States") under different Seismic Hazard levels
- "Limit States" (Performance Levels)

  Damage Limitation (: Immediate Occupancy)

  Significant Damage (: Life Safety)

  Near Collapse

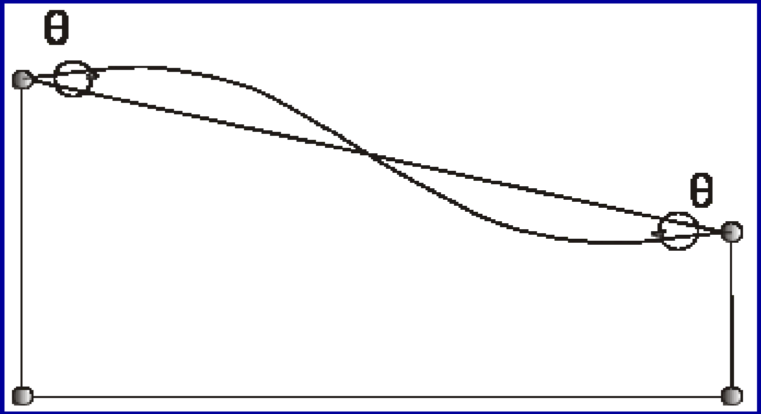
➢ Flexibility for countries, owners, designers:

- How many/which Limit States will be met for what Hazard Level:
  - to be decided by country, or
  - (if country doesn't decide in Nat. Annex) by owner/designer
- Hazard Levels: NDPs - No recommendation given

  Noted that Basic Objective for ordinary new buildings is:
  - Damage Limitation: Occasional EQ (225yrs??)
  - Significant Damage: Rare EQ (475yrs)
  - Near Collapse: Very rare EQ (2475yrs)

Fully displacement-based approach:

- Capacity-demand-comparisons for checking ductile elements (existing, retrofitted or new): in terms of deformations.
- Deformation measure: Chord rotation at member end

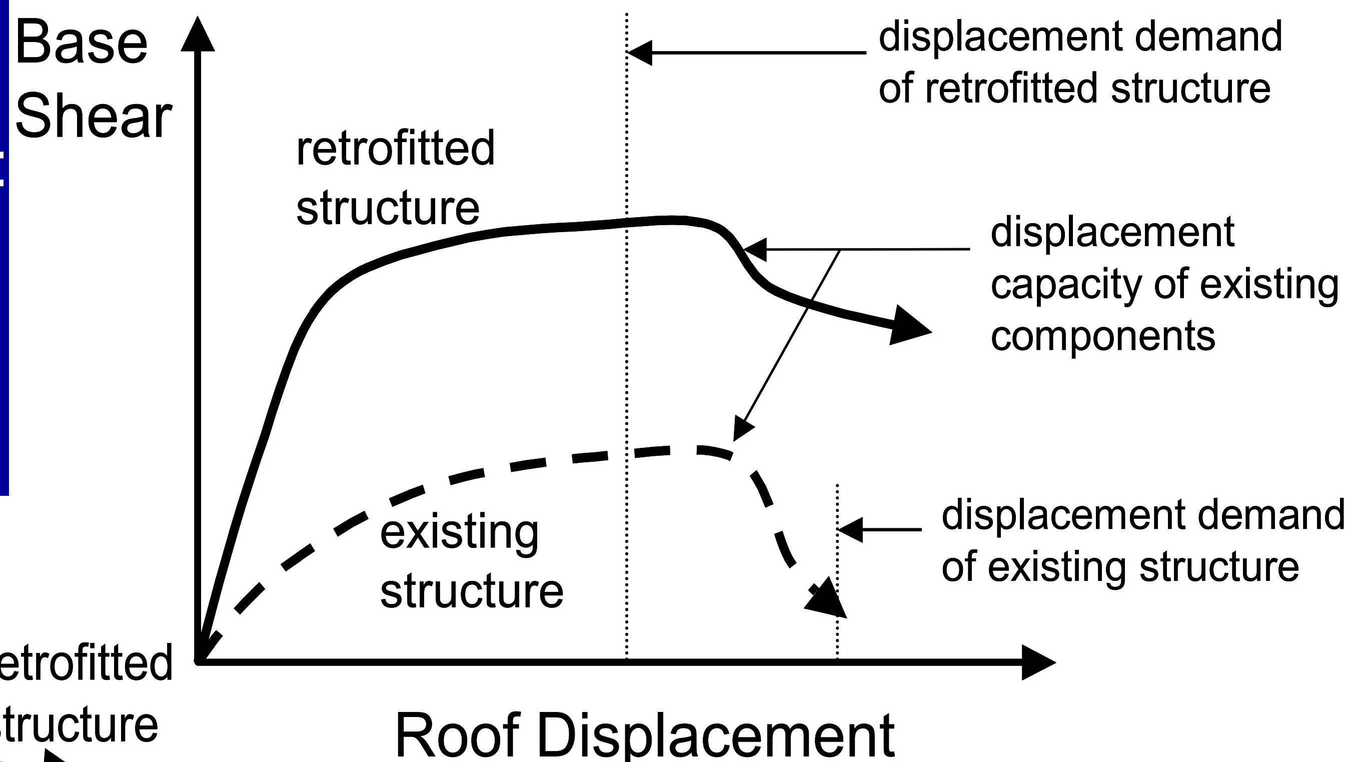


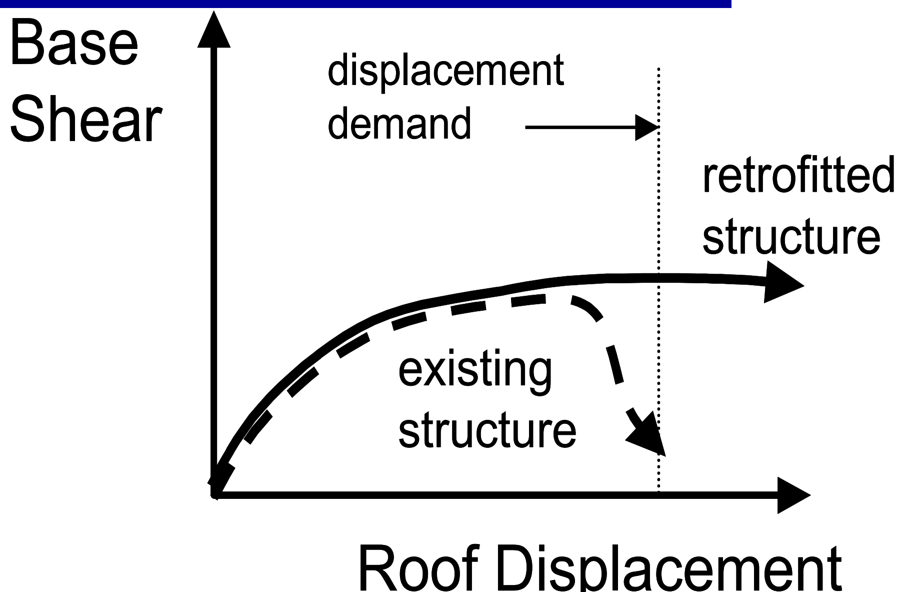
- Retrofit aims at reducing deformation demands on existing members below their capacities

  (global stiffening by addition of new elements easier than local modification of existing members to increase their deformation capacities).
- More cost-effective assessment & retrofitting

Displacement-Based Retrofitting Strategies

Increase global stiffness to reduce deformation demand on components:





Increase deformation capacity of components

Seismic Assessment in EC8-Part 3

Detailed assessment of individual building

- Guides selection of retrofit strategy & extent of intervention:
  - *Deficiencies in few scattered elements*:
  - local modification of elements
  - *Deficiencies in one part of the structure:*
  - possible irregularity (weak story, unbalanced structure, etc.) to be removed (by adding new elements, strengthening or even weakening existing members, etc.)
  - *Generalized deficiency:*
  - add new elements (eg walls) to increase stiffness & reduce deformation demands;
  - or upgrade most (if not all) elements (costly, inconvenient)

Information for the Assessment

1. "limited knowledge":
   - Only for linear analysis;
   - "Confidence factor", equal to 1.35, corrects mean material strengths from in-situ tests etc. (division or multiplication, whatever is less favorable).
2. "normal knowledge":
   - For linear or nonlinear analysis;
   - "Confidence factor", equal to 1.2, corrects mean material strengths from in-situ tests etc. (as above).
3. "full knowledge":
   - For linear or nonlinear analysis;
   - Mean material strengths from in-situ tests etc. used w/o "confidence factor".

1. “limited knowledge”:
   - Structural geometry from:
     - ✓ original drawings & in-situ spot checks; or
     - ✓ full campaign of in-situ measurements, if original drawings not available.
   - Default assumptions for materials, verified with 1 sample /floor /type of member.
   - Reinforcement by simulation of original design (checks in ~20%/type of member).
2. “normal knowledge”:
   - Structural geometry & reinforcement from:
     - ✓ original drawings & in-situ checks in ~20% of members / type of member; or
     - ✓ full imeasurements & rebars exposed in 50%/ member type, for no drawings.
   - Materials from:
     - ✓ original specifications, verified in-situ w/ 1 sample /floor / type of member; or
     - ✓ 2 samples / floor / type of member.
3. “full knowledge”:
   - Structural geometry & reinforcement from:
     - ✓ original drawings & in-situ checks in ≥20% of members / type of member; or
     - ✓ full in-situ measurements, bars exposed in ≥80% of members/ member type
   - Materials from:
     - ✓ original test reports, verified w/ 1 sample /floor/type of member; or
     - ✓ 3 samples / floor / type of member.

“Ductile” vs. “Brittle” elements

- Ductile elements (in RC: columns, beams, walls in bending):
- Verification on the basis of deformations (regardless of analysis procedure).

- Brittle elements (in RC: columns, beams, walls, joints in shear):
- Verification on the basis of forces.

"Primary" & "Secondary" elements

- Engineer may designate elements as "primary" or "secondary", depending on which ones he relies upon for lateral stiffness & resistance:
- Criteria for their acceptable deformation or force limits: less strict than for primary elements.

Analysis methods for deformation demands in ductile elements

- 4 types of analysis for deformation demands, all with seismic action defined by 5%-damped elastic spectrum:
  1. Linear static (equivalent lateral forces);
  2. Linear dynamic (modal response spectrum);
  3. Nonlinear static ("pushover") – Reference method;
  4. Nonlinear dynamic (time-histories: ≥ 3, ≥ 7 for mean results).

For 1 & 2: Equal displacement rule, w/o correction coefficients.

For 3: N2-method (target displacement: Equal displacement rule w/ correction due to short-T only).

For 3: If higher-modes important ($T>4T_c$, or T>2sec): "Modal pushover" or nonlinear dynamic analysis.

For 3 & 4: Simple nonlinear member models encouraged;

- More important than sophistication of model: ability to represent effective stiffness up to yielding, to capture dominant periods.

Applicability of linear analysis

Under seismic action (hazard level) of interest:

- Uniform distribution of inelasticity:

  DCR: Ratio of (elastic) moment demand to capacity (~member displacement ductility ratio).

  Criterion:

  Ratio of Max. to Min. value of DCR over all ductile members that go inelastic (ends of strong elements framing into joint excluded)

  < limit value, between 2 and 3 (NDP; recommended value: 2.5).

  (Fairly restrictive; linear analysis only for buildings w/ fairly uniform distribution of overstrengths).

If

(a) criterion above is satisfied,

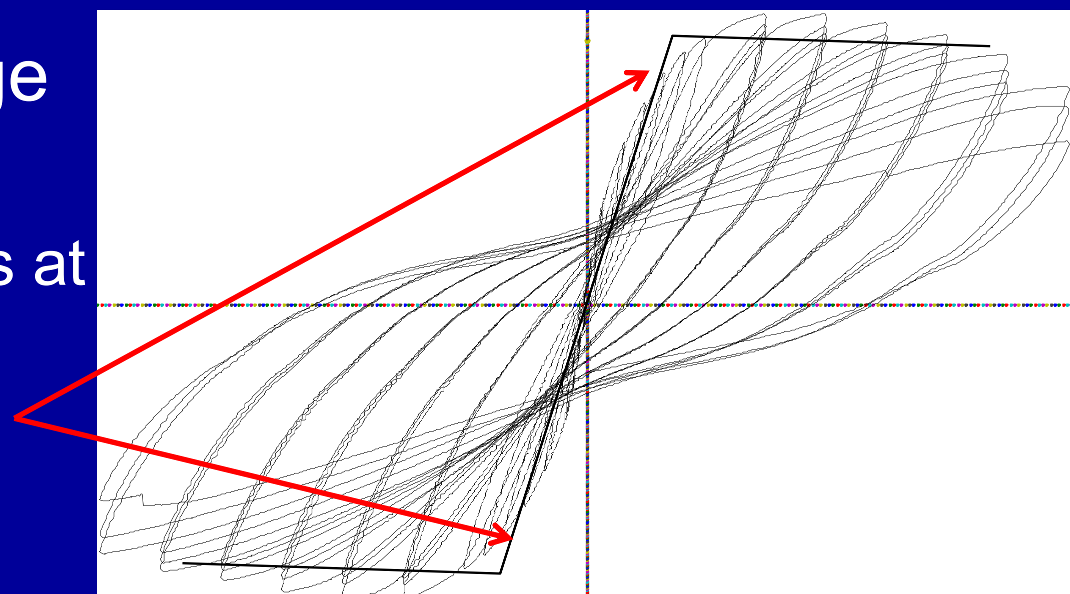
(b) building is heightwise regular &

(c) higher-modes are unimportant ($T<4T_c$, $T<2$sec), then:

- Linear static analysis w/ triangular distribution of lateral forces

Effective elastic stiffness, EI (in linear or nonlinear analysis)

- In Part 1of EC8 (for design of new buildings):
  - EI = secant stiffness to yield point;
  - RC: EI = 50% of uncracked gross-section stiffness.
- 50% of uncracked gross-section stiffness:
  - OK in force-based design of new structures (safe-sided for forces);
  - Not OK in displacement-based assessment (unsafe for displacement demands).
- More realistic, esp. in damage limitation check, $\theta_E \leq \theta_y$:
  - $EI = M_y L_s / 3\theta_y$: secant stiffness at yielding of both ends in skew-symmetric bending



RC member verification criteria

Limit State (LS):	Damage Limitation	Significant Damage (SD)	Near Collapse (NC)	
Member:			linear analysis	nonlinear analysis
ductile primary	$\theta_E \leq \theta_y$	$\theta_E \leq 0.75\theta_{u,m-\sigma}$	$\theta_E \leq \theta_{u,m-\sigma}$	
ductile secondary		$\theta_E \leq 0.75\theta_{um}$	$\theta_E \leq \theta_{um}$	
brittle primary	Check only if NC LS not checked. Then use NC criteria w/ V_E (or $V_{E,CD}$ for SD LS w/ linear analysis)		$V_{E,CD} \leq V_{Rd,EC2}$, $\leq V_{Rd,EC8}/1.15$	$V_E \leq V_{Rd,EC2}$, $\leq V_{Rd,EC8}/1.15$
brittle secondary			$V_{E,CD} \leq V_{Rm,EC2}$, $\leq V_{Rm,EC8}$	$V_E \leq V_{Rm,EC2}$, $\leq V_{Rm,EC8}$

θ_E, V_E: chord-rotation & shear force demand from analysis; $V_{E,CD}$: from capacity design;
θ_y: chord-rotation at yielding; θ_{um}: expected value of ultimate chord rotation;
$\theta_{u,m-\sigma}$: mean-minus-sigma ult. chord rotation $=\theta_{um}/1.5$, or $=\theta_y+\theta^{pl}_{um}/1.8$;
V_{Rd}, V_{Rm}: shear resistance, w/ or w/o material safety & confidence factors;
$V_{R,EC2}$: monotonic shear resistance; $V_{R,EC8}$: cyclic shear resistance after flexural yielding.

If analysis is linear: Shear force $V_{E,CD}$ from equilibrium, under end moments consistent with plastic hinging there or (in beams or columns) around joint

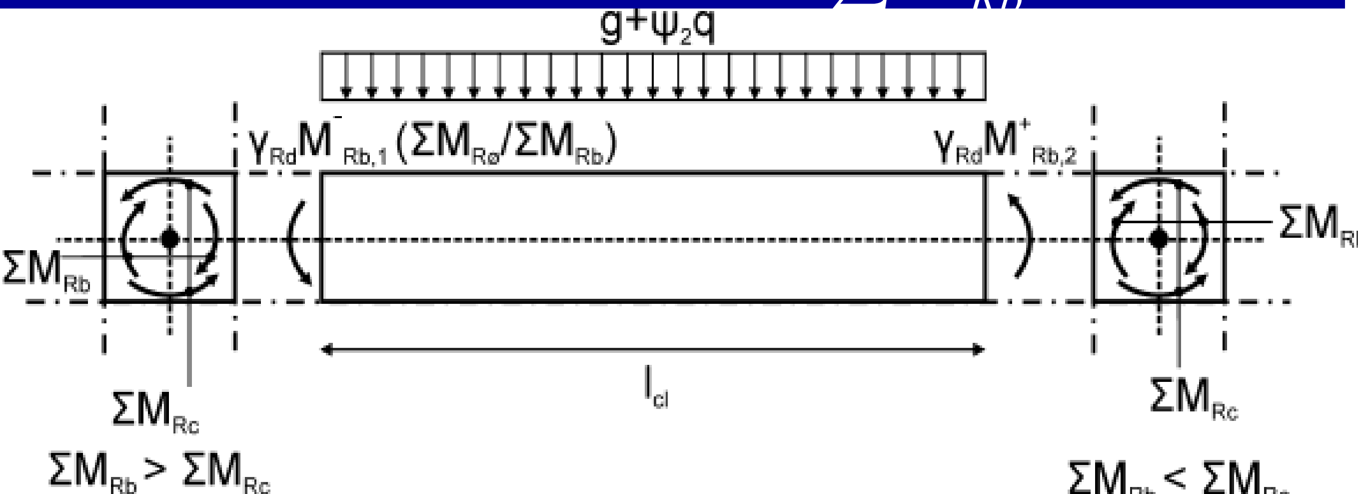
Walls:

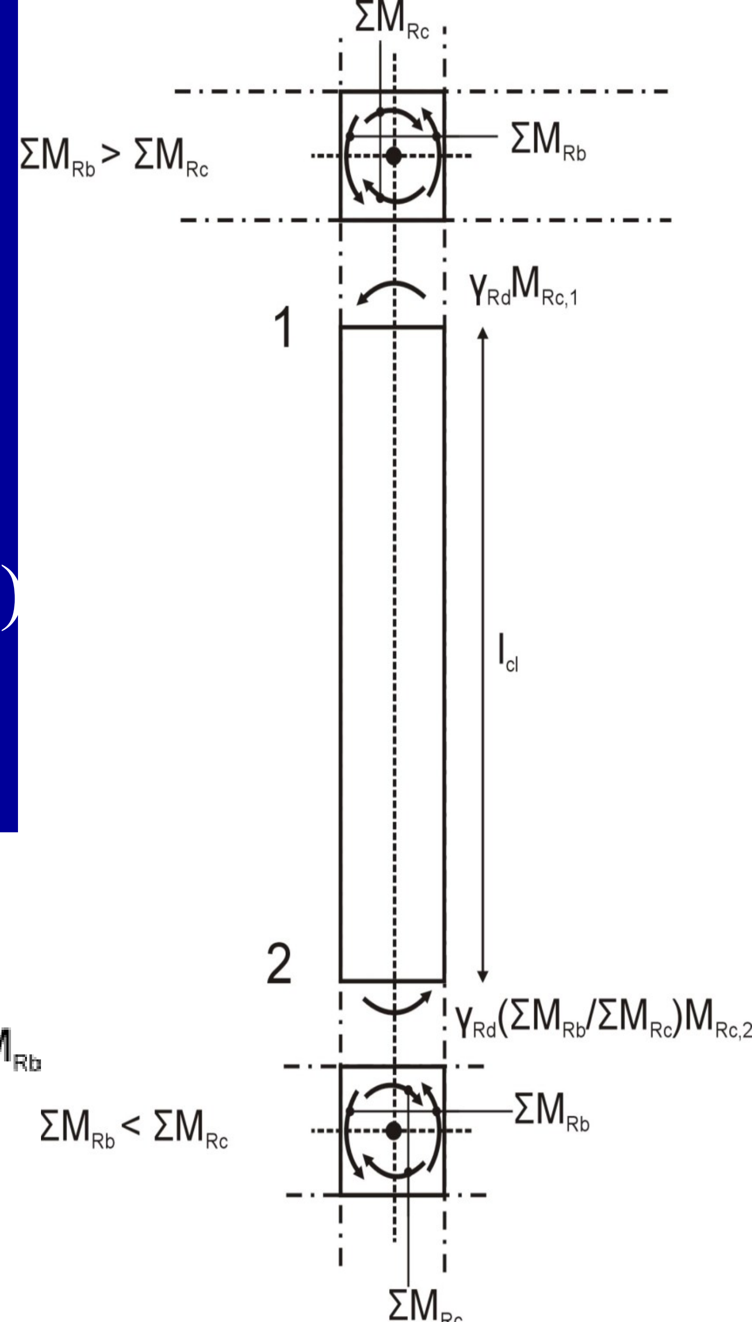
$$V_{Sd} = \frac{M_{Rw}}{M_{EW}} V_E$$ (M_{RW}, M_{EW} : at the base)

Columns:

$$M_{i,d} = M_{Rc,i} \min\left(1, \frac{\sum M_{Rb}}{\sum M_{Rc}}\right)$$

Beams: $$M_{i,d} = M_{Rb,i} \min\left(1, \frac{\sum M_{Rc}}{\sum M_{Rb}}\right)$$

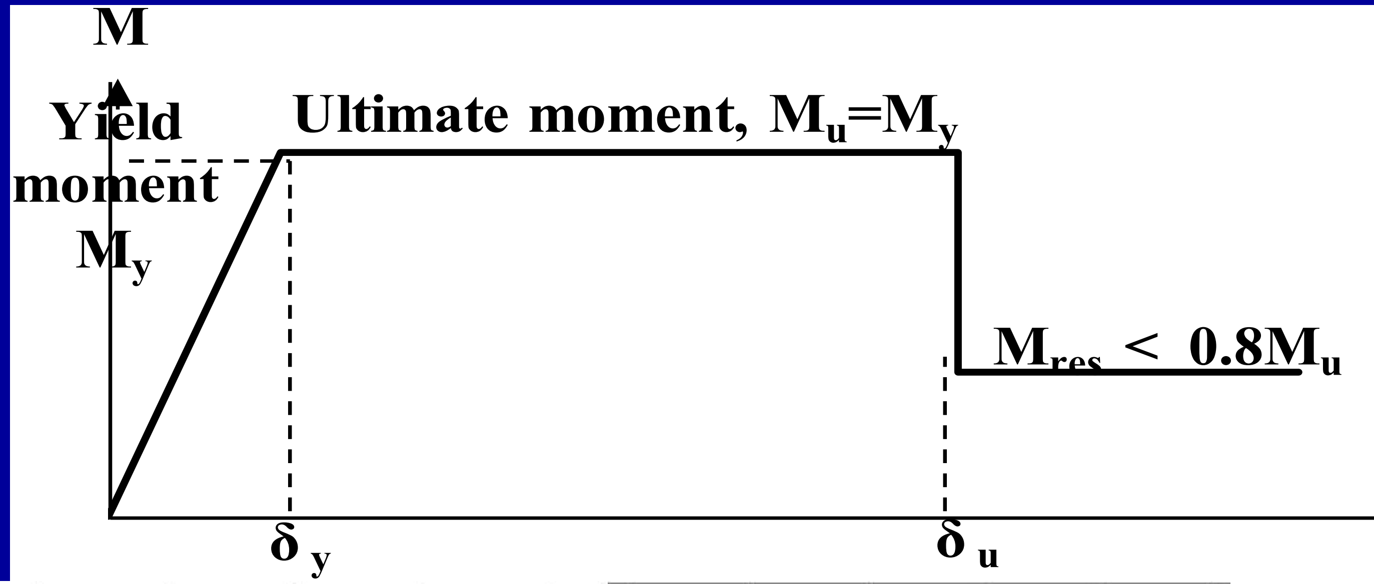


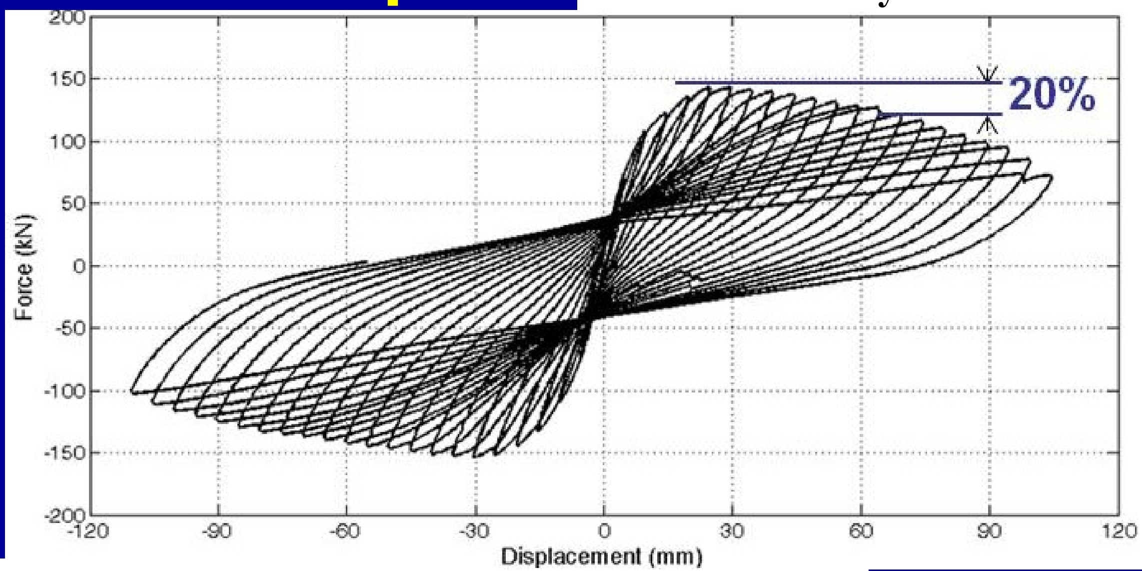


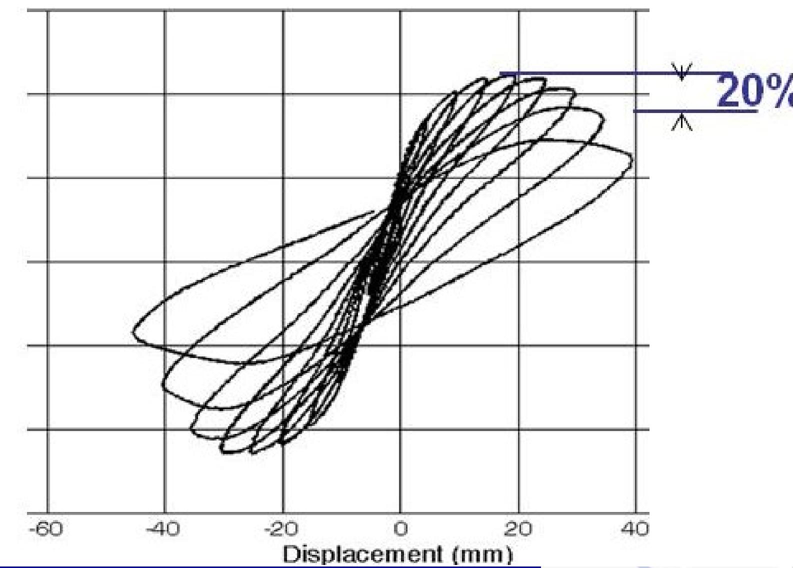
Practical expressions for resistance, stiffness & ultimate deformation of RC members

Effective elastic stiffness: secant-to-yield-point

Idealized skeleton curve - envelope to hysteresis loops







Chord-rotation at RC member yielding

Beams, rect. columns:

Walls:

- ϕ_y: yield curvature (via 1st principles, adapted to median M_y);
- $L_s = M/V$: shear span at member end (~L/2);
- z~0.9d: tension shift (= 0 if member not diagonally cracked by shear at flexural yielding: M_y/L_s);
- h : section depth;
- f_y, f_c: MPa;
- d_b: bar diameter;
- Last term: Due to bar slip from anchorage zone beyond member end (omitted if such slippage not possible)

Seismically-detailed RC members w/ rect. web

Expected value of ultimate chord rotation (20% drop in resistance)

$$\theta_{um} = \alpha_{st}\left(1 - \frac{3a_{wall}}{8}\right)\left(0.3^{\nu}\right.$$

or:

$$\theta_{um} = \theta_{y} + \alpha_{st,pl}\left(1 - 0.4a_{wall}\right)\left(0.25^{\nu}\right)$$

α_{st}: 0.016 for hot-rolled ductile steel or heat-treated (tempcore); 0.01 for brittle cold-worked steel;

$\alpha_{st,pl}$: 0.0145 for hot-rolled ductile steel or heat-treated (tempcore); 0.0075 for brittle cold-worked steel;

α_{wall}: 1 for shear walls;

ω, ω': mechanical ratio of tension (including web) & compression steel;

ν: N/bhf_c (b: width of compression zone; N>0 for compression);

L_s/h : M/Vh: shear span ratio;

α : confinement effectiveness factor :

$$\alpha = \left(1 - \frac{s_h}{2b_c}\right)\left(1 - \frac{s_h}{2h_c}\right)\left(1 - \frac{\sum b_i^2}{6b_c h_c}\right)$$

ρ_{sx}: $A_{sh}/b_w s_h$: transverse steel ratio // direction (x) of loading;

ρ_d: ratio of diagonal reinforcement.

Non-seismically detailed members w/o lap splices - cyclic loading

Plastic part, $\theta^{pl}_{um} = \theta_{um} - \theta_y$, of ult. chord rotation multiplied by 0.825.

RC members w/ or w/o seismic detailing, w/ ribbed bars lap-spliced over l_o in plastic hinge region

- Compression reinforcement counts as double.
- For yield properties M_y, ϕ_y , θ_y:

  f_y of tension steel multiplied x $l_o/l_{oy,min}$ <u>if $l_o<l_{oy,min}=(0.3f_y/\sqrt{f_c})d_b$</u>
- For ultimate chord rotation $\theta_{um}=\theta_y+\theta^{pl}_{um}$:

  θ^{pl}_{um} x $l_o/l_{ou,min}$ <u>if $l_o<l_{ou,min}=d_b f_y/[(1.05+14.5\alpha_{rs}\omega_{sx})\sqrt{f_c}]$,</u>
  - f_y, f_c in MPa, $\omega_{sx}=\rho_{sx}f_{yw}/f_c$: mech. transverse steel ratio // loading,
  - $\alpha_{rs}=(1-s_h/2b_o)(1-s_h/2b_o)n_{restr}/n_{tot}$ (n_{restr}/n_{tot} restrained-to-total lap-spliced bars).

Cyclic shear resistance of RC members (reduction w/ cyclic displacements)

- Shear resistance after flexural yielding, as controlled by stirrups

(linear degradation of both V_c and V_w with displacement ductility demand $\mu_{\Delta}^{pl}=(\theta-\theta_y)/\theta_y$)

$$V_R = \frac{h-x}{2L_s}\min(N, 0.55A_c f_c) + \left(1-0.05\min(5, \mu_{\Delta}^{pl})\right)\left[0.16\max(0.5, 100\rho_{tot})\left(1-0.16\min\left(5, \frac{L_s}{h}\right)\right)\sqrt{f_c}A_c + V_w\right]$$

V_w: contribution of web reinf. = $\rho_w b_w z f_{yw}$ (b_w: web width, z: internal lever arm; ρ_w: web reinf. ratio)

ρ_{tot}: total longitudinal reinforcement ratio

h: section depth

x : depth of compression zone

$A_c = b_w d$

- Shear resistance as controlled by web crushing (diagonal compression)
  - **Walls**, before flexural yielding ($\mu_{\Delta}^{pl} = 0$), or after flexural yielding (cyclic $\mu_{\Delta}^{pl} > 0$):

$$V_R = 0.85\left(1-0.06\min\left(5, \mu_{\Delta}^{pl}\right)\right)\left(1+1.8\min\left(0.15, \frac{N}{A_c f_c}\right)\right)\left(1+0.25\max(1.75, 100\rho_{tot})\right)\left(1-0.2\min\left(2, \frac{L_s}{h}\right)\right)\sqrt{f_c}\,b_w z$$

  - **Squat columns** ($L_s/h \le 2$) after flexural yielding (cyclic $\mu_{\Delta}^{pl} > 0$):

$$V_R = \frac{4}{7}\left(1-0.02\min\left(5, \mu_{\Delta}^{pl}\right)\right)\left(1+1.35\frac{N}{A_c f_c}\right)\left(1+0.45\cdot 100\rho_{tot}\right)\sqrt{\min\left(f_c, 40\right)}\,b_w z \sin 2\delta$$

δ: angle between axis and diagonal of column ($\tan\delta = h/2L_s$)

Summary: EC8-Assessment approach

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- Estimation of displacement/deformation demands independent from deformation capacities: deformation demands and capacities estimated and compared at the member level (chord rotations).
- Analysis for the estimation of displacement/deformation demands may be simple (even linear, if inelasticity is uniformly distributed within structure);
  - basis for estimation of displacement/deformation demands: the equal displacement rule (except in nonlinear dynamic analysis);
  - simple member models are encouraged;
  - more important than the sophistication of the model is the ability to reproduce the effective stiffness to yielding, in order to capture the dominant periods of vibration.
- Simple, yet fairly accurate semi-empirical models for the estimation of member deformations at yielding and ultimate (as controlled by flexure, shear or lap-splices)
- (Note: models updated and their scope extended since 2005).

Critisism from experience of applying EC8 Part 3

- It addresses ony concrete buildings; masonry ones poorly covered;
- “Confidence factor” (function of “Knowledge Level”) modifying mean material properties:

  Poor representation of uncertainties;
- Performance requirements are expressed for the structure as a whole, the but compliance criteria are given for individual members only:

  Need criteria for the whole structure.
- Non-structural elements are not addressed.

Seismic Retrofitting per EC8-Part 3 (Annex A)

Concrete Jackets

Concrete Jackets (continued/anchored in joint) per EC8

Calculation assumptions:

- Full composite action of jacket & old concrete assumed (jacketed member: monolithic"), even for minimal shear connection at interface (roughened interface, steel dowels epoxied into old concrete: useful but not essential);
- f_c of "monolithic member"= that of the jacket (avoid large differences in old & new f_c)
- Axial load considered to act on full, composite section;
- Longitudinal reinforcement of jacketed column: mainly that of the jacket. Vertical bars of old column considered at actual location between tension & compression bars of composite member (~ "web" longitudinal reinforcement), with its own f_y;
- Only the transverse reinforcement of the jacket considered for confinement;
- For shear resistance, the old transverse reinforcement taken into account only in walls, if anchored in the (new) boundary elements.

Then:

✓ M_R & M_y of jacketed member: ~100% of
✓ Pre-yield (elastic) stiffness of jacketed member ~95% of
✓ Shear resistance of jacketed member: ~90% of
✓ Flexure-controlled ultimate deformation θ_u: ~100% of

those of "monolithic member" calculated w/ assumptions above.

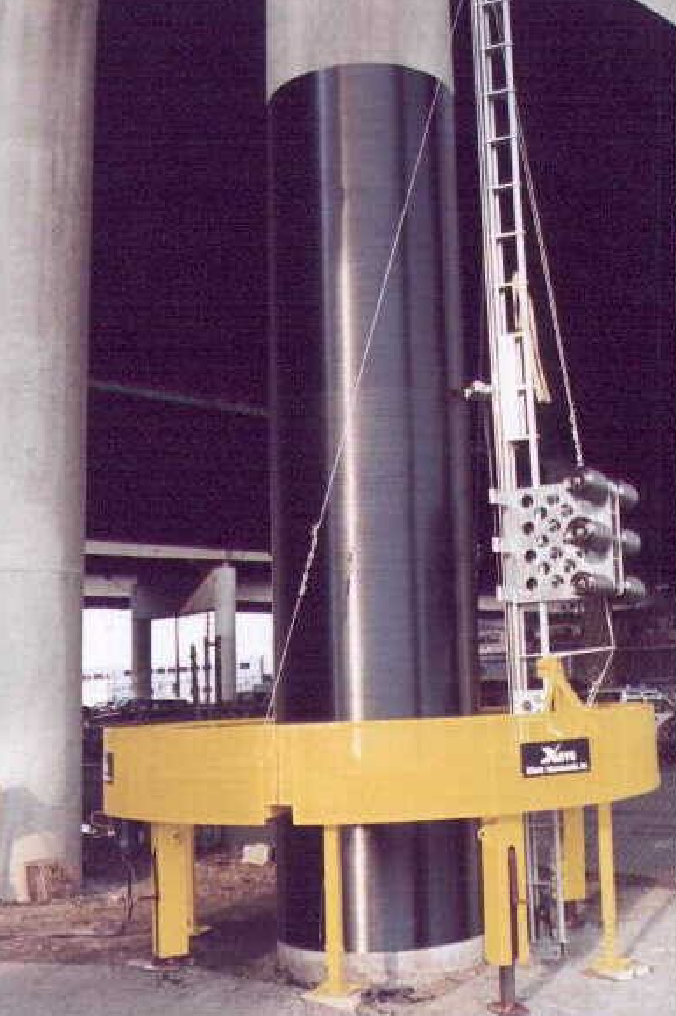
Steel Jackets

Steel Jackets (not continued/anchored in joint) per EC8-3

Jacket stops ahead of joint (several mm gap to joint face)

- Flexural resistance, pre-yield (elastic) stiffness & flexure-controlled ultimate deformation of RC member : not enhanced by jacket (flexural deformation capacity ~same as in "old" member inside jacket, w/o effect of confinement);
- 50% of shear resistance of steel jacket, $V_j=A_j f_{yj} h$, can be relied upon for shear resistance of retrofitted member (suppression of shear failure before or after flexural yielding);
- Lap-splice clamping effected via friction mechanism at jacket-member interface, if jacket extends to ~1.5 times splice length and is bolt-anchored to member at end of splice region & ~1/3 its height from joint face (anchor bolts at third-point of side)

FRP Jackets



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FRP Jackets (not continued/anchored in joint) per **Eurocode 8**

Rectangular sections, continuous long. bars (no lap splices)

- M_R & M_y, pre-yield (elastic) stiffness EI_{eff} of RC member: not significantly enhanced by FRP jacket (increase neglected);
- Flexure-controlled ultimate deformation, θ_u: confinement factor due to stirrups enhanced due to FRP confinement by $\alpha\rho_f f_{f,e}/f_c$
  - $\rho_f=2t_f/b_w$: FRP ratio;
  - $f_{f,e}$: FRP effective strength:

    $$f_{f,e} = \min\left(f_{u,f}, \varepsilon_{u,f}E_f\right)\left(1-0.7\min\left(f_{u,f}, \varepsilon_{u,f}E_f\right)\frac{\rho_f}{f_c}\right)$$

    where:

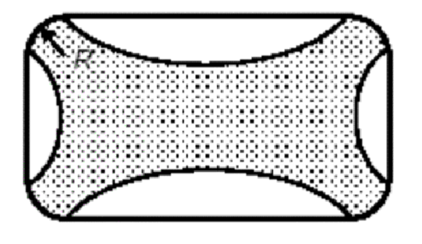
    $f_{u,f}$, E_f : FRP tensile strength & Modulus;

    $\varepsilon_{u,f}$: FRP limit strain; CFRP, AFRP: $\varepsilon_{u,f}$=0.015; GFRP: $\varepsilon_{u,f}$=0.02; polyacetal FRP: $\varepsilon_{u,f}$= 0.032;
  - confinement effectiveness:

    $$\alpha = \left(1-\frac{(h-2R)^2+(b-2R)^2}{3bh}\right)$$

    b, h: sides of X-section;

    R: radius at corner



FRP Jackets (not continued/anchored in joint) **per Eurocode 8**

Rectangular section, ribbed longitudinal bars lap-spliced **over l_o** in plastic hinge:

- Compression reinforcement counts as double.
- For yield properties M_y, ϕ_y , θ_y:

  f_y of tension steel multiplied x $l_o/l_{oy,min}$ if $l_o<l_{oy,min}=(0.2f_y/\sqrt{f_c})d_b$

- For ultimate chord rotation $\theta_{um}=\theta_y+\theta^{pl}_{um}$:

  θ^{pl}_{um} calculated on the basis of confinement by the stirrups alone, multiplied x $l_o/l_{ou,min}$ if $l_o<l_{ou,min}=d_b f_y/[(1.05+14.5\alpha_{rs}\rho_f f_{f,e}/f_c)\sqrt{f_c}]$,

  – f_c in MPa, $\rho_f=2t_f/b_w$: FRP ratio, $f_{f,e}$: effective FRP strength in MPa,

  – $\alpha_{rs}=4/n_{tot}$ (n_{tot} : total lap-spliced bars, only the 4 corner ones restrained).

FRP Jackets per Eurocode 8 (cont'd)

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- Shear resistance of FRP-jacketed member:

$$V_R = \frac{h-x}{2L_s}\min\left(N, 0.55A_c f_c\right) +$$

$$\left(1-0.05\min\left(5, \mu_\theta^{pl}\right)\right)\left[0.16\max(0.5, 100\rho_{tot})\left(1-0.16\min\left(5, \frac{L_s}{h}\right)\right)\sqrt{f_c}A_c + V_w\right] + V_f$$

$$V_f = \min(\varepsilon_{u,f}E_{u,f}, f_{u,f})\rho_f b_w z/2$$

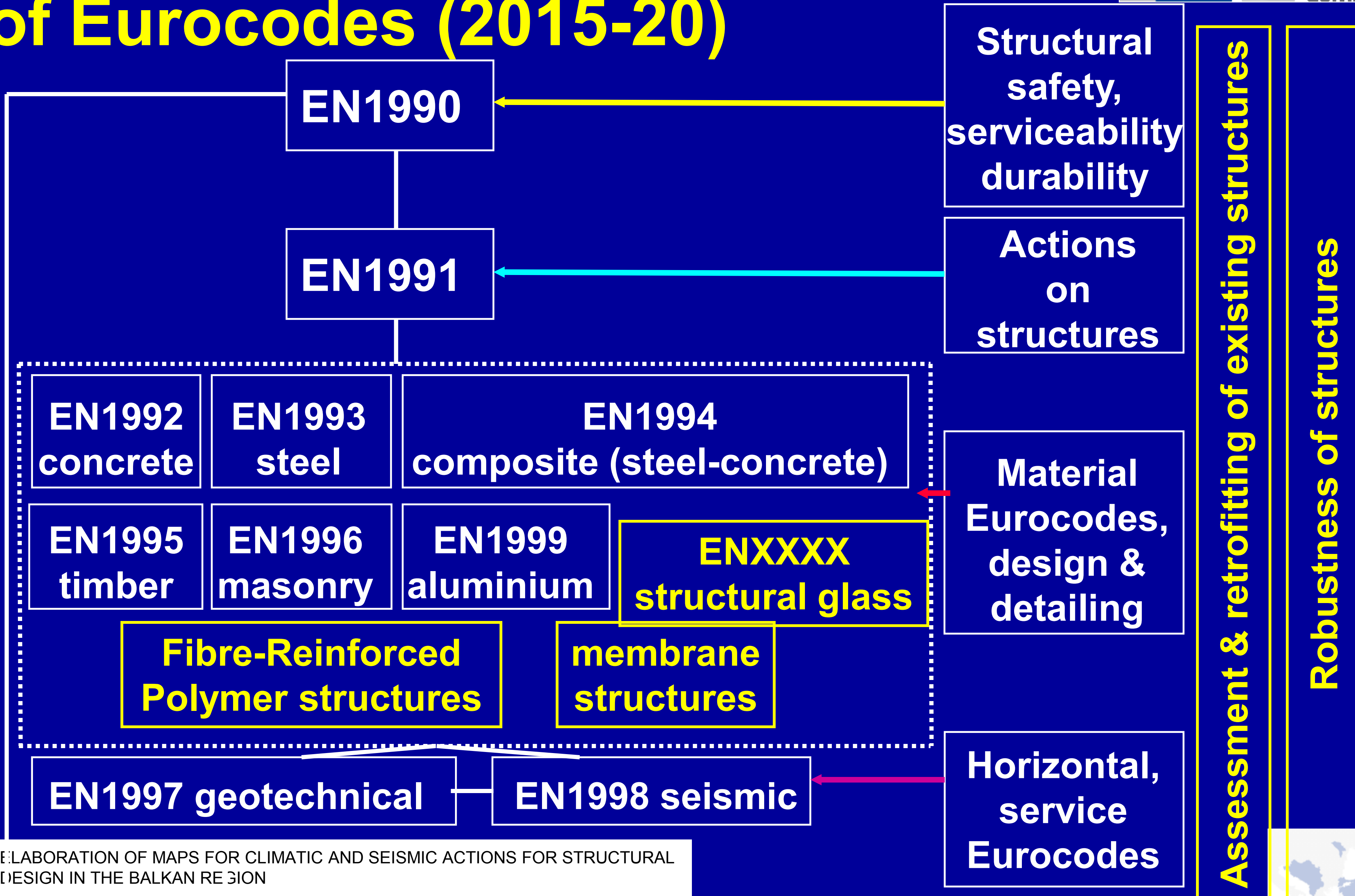
contributes to member shear resistance in diagonal tension

- ρ_f :FRP ratio, $\rho_f = 2t_f/b_w$;
- $f_{u,f}$:FRP tensile strength;
- z : internal lever arm.

- Total shear resistance of retrofitted member as controlled by diagonal tension, should not exceed shear resistance of old RC member as

The evolution plan for EC8-Part 3

Evolution to the 2nd generation of Eurocodes (2015-20)



New title “Seismic retrofitting of structures”

1. Buildings

- EN1998-3:2005: Criticisms:
  - It addresses concrete buildings; masonry ones poorly covered;
  - “Confidence factor” (function of “Knowledge Level”) modifying mean material properties: poor representation of uncertainties;
  - Performance requirements expressed for the structure as a whole, but compliance criteria refer to members only;
  - Non-structural elements not addressed;
  - Shear failure of concrete & masonry elements needs revisiting.
- Plan for the 2nd generation EN1998-3 (2015-19):
  - ➢ Revisit “Confidence factors” and reliability format;
  - ➢ Criteria for ultimate condition of the structural system;
  - ➢ Drastically strengthen/improve provisions for masonry buildings;
  - ➢ Advance/update nonlinear (static) analysis, as reference method

2. Bridges

- EN1998-3:2005:

  Not covered in scope.

- 2nd generation EN1998-3:
  - Cover seismic assessment & retrofitting of:
    - concrete & steel/composite bridges;
    - bridge foundations and bearings.
  - Seismic isolation or dissipation devices: effective for retrofitting.

PART II:

FIRST BUILDING RETROFITTED TO EN1998-3:2005 TESTED BY NEAR-DESIGN-LEVEL EARTHQUAKE


ΔΗΜΟΤΙΚΟ ΘΕΑΤΡΟ ΑΡΓΟΣΤΟΛΙΟΥ «Ο ΚΕΦΑΛΟΣ»


Retrofitting triggered by corrosion of
rebars in exterior columns

- RC frame constructed in 1979-80
- Building completed later.
- Plaster rendering was not applied till 1991-92.
- Concrete exposed to salt-laden coastal environment for >10 yrs
- Mean concrete strength: 22.5 MPa



Framing plan: Roof level

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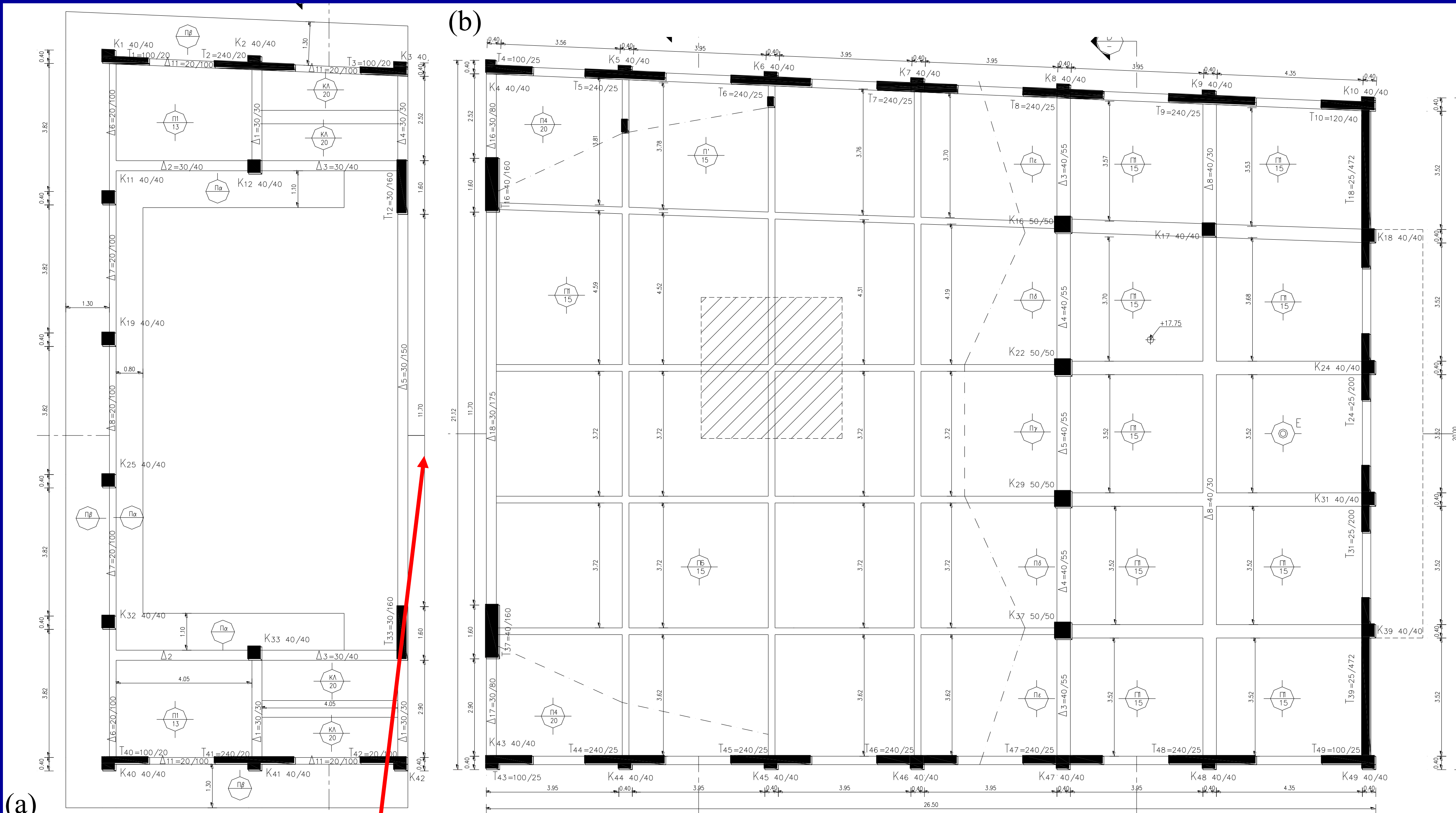
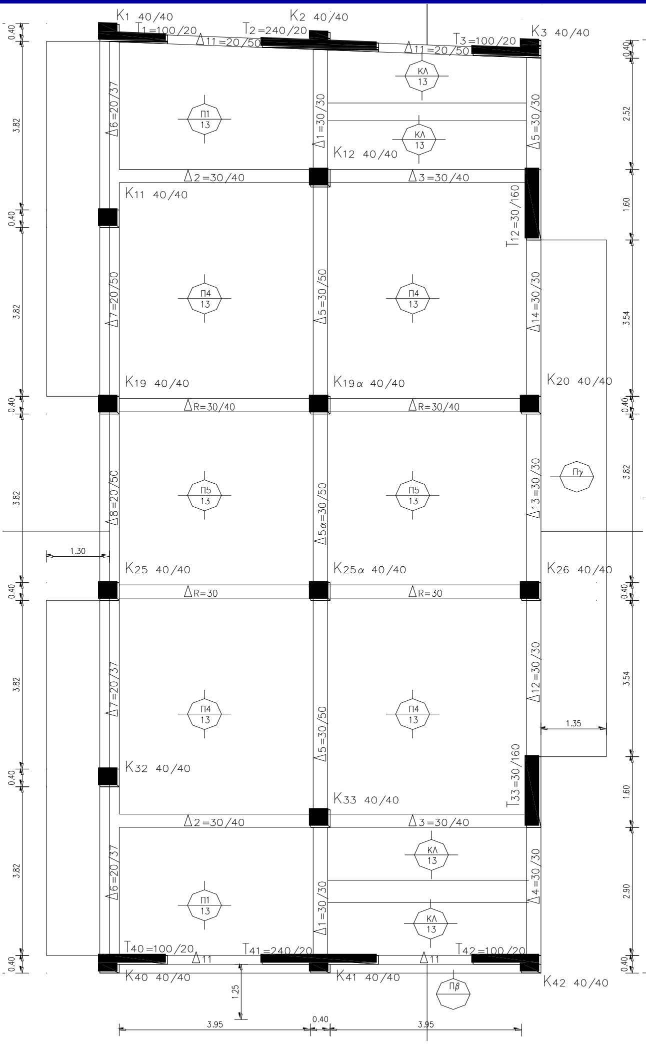
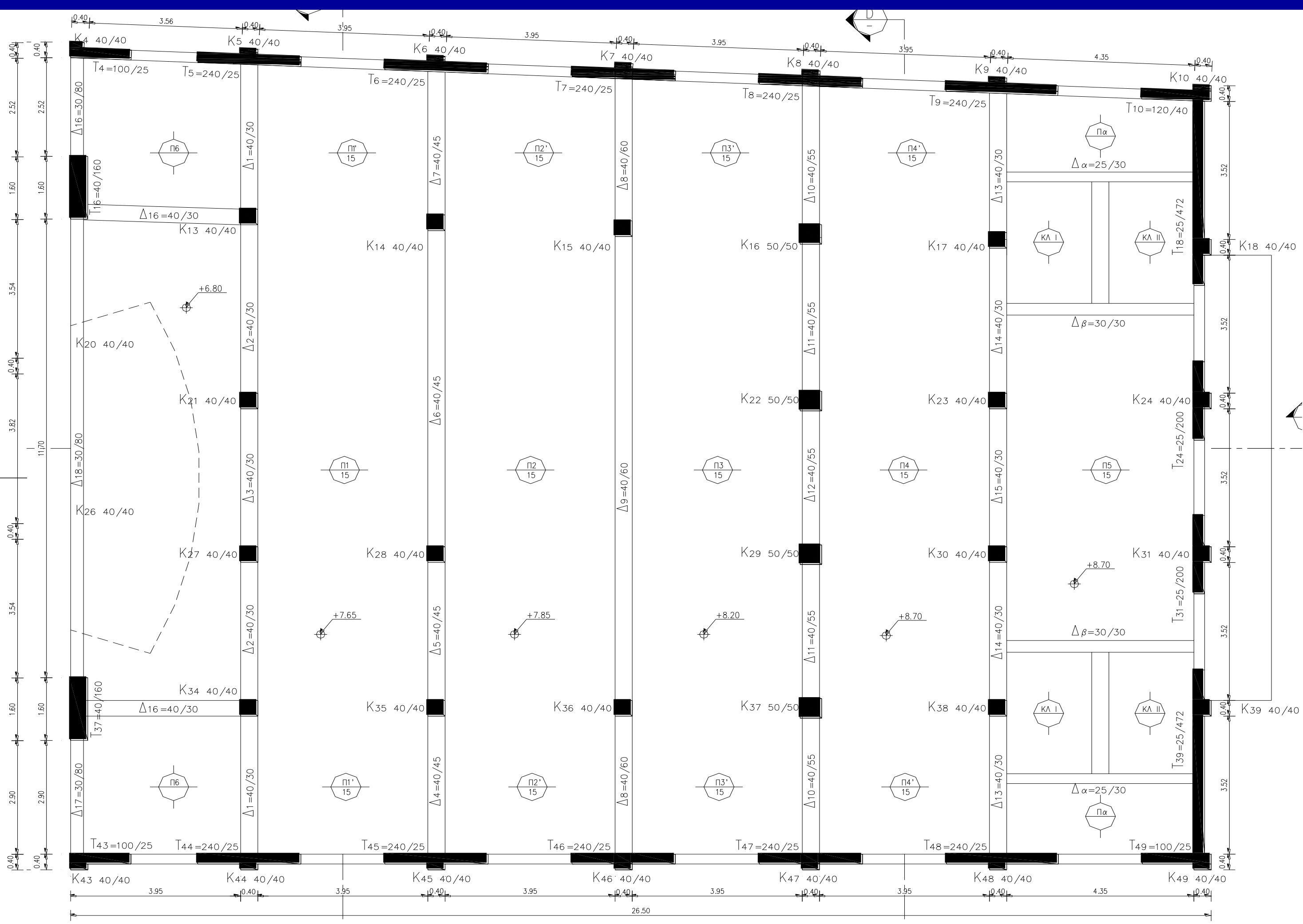


Narrow expansion joint separates building in two independent parts (Stage” & “House”), both irregular in plan and elevation

Framing plan: Ground floor



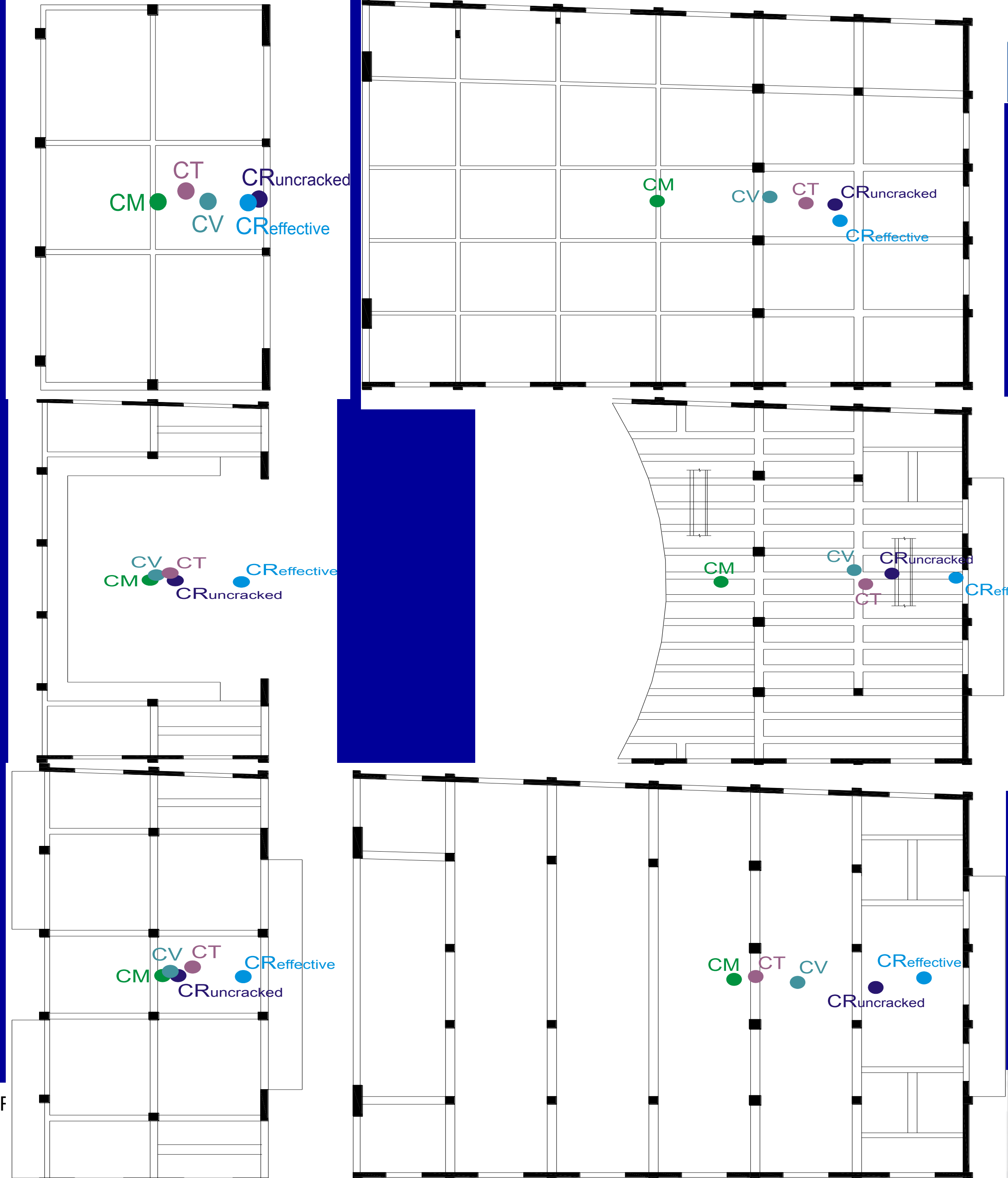
(c)



(d)

Eccentricities between Centre of Mass (CM) & Centres of Rigidity (CR) or Strength (CV), or Twist (CT) in both parts of the building →

Torsional seismic response & pounding of the two parts at the expansion joint

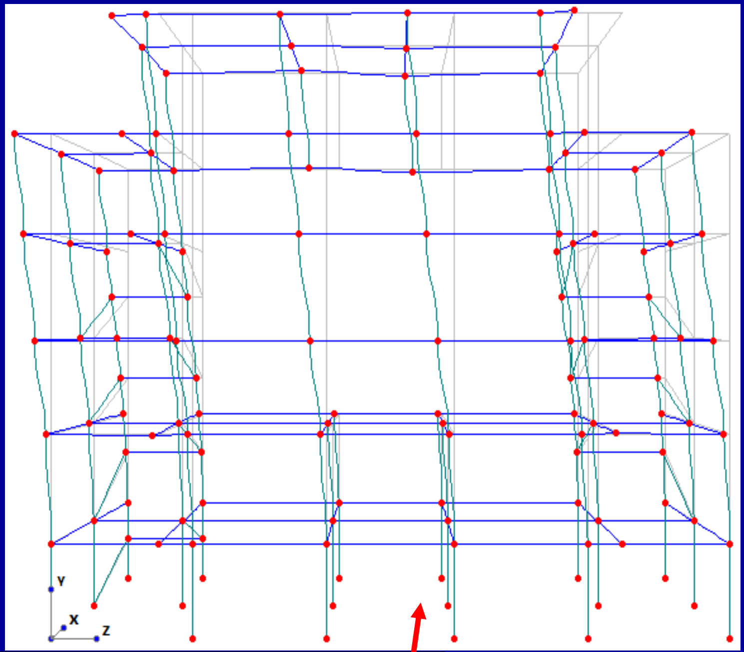


2004-05: Seismic evaluation of building per EC8

- **<u>Nonlinear dynamic</u> analysis under 56 bidirectional ground motions conforming to Eurocode 8 Soil C spectrum:**
  - Vertical members fixed at foundation level
  - finite size of beam-column joints
  - P-Δ effects
  - Members:
    1. Point-hinge model;
    2. Takeda model (bilinear envelope, no strength degradation);
    3. Elastic stiffness $EI = M_y L_s / 3\theta_y$: secant-to-yielding;
    4. Flexure-controlled ultimate chord rotation (mean capacity);
    5. Shear resistance reduced due to post-yield cyclic deformations.

    3.-5. w/ modifications due to:
    - poor detailing of unretrofitted members (including short lap-splicing);
- Performance evaluated in terms of chord rotation or shear force Demand-to-Capacity (damage) Ratio:
  - At "ultimate" (: resistance < 80% of peak resistance): Demand-to-Capacity (damage) Ratio = 1.

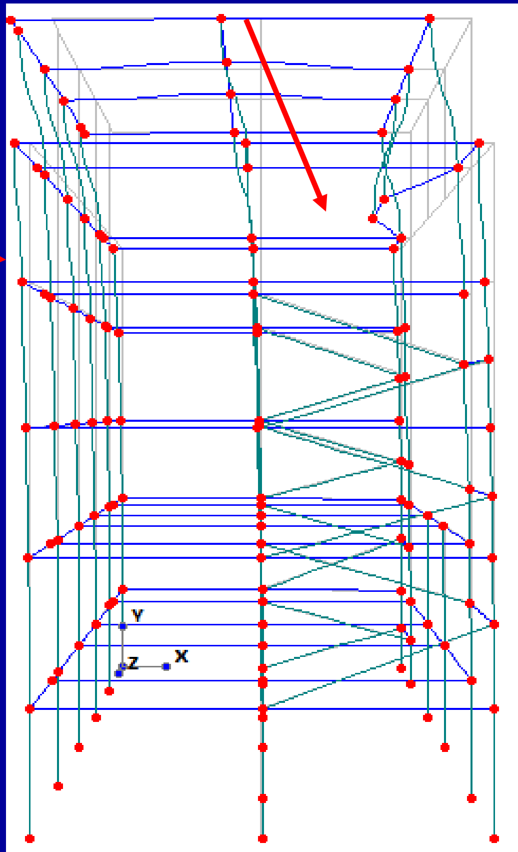
Total mass large: Building flexible.



Roof beam supporting floating penthouse columns on the Stage side: sensitive

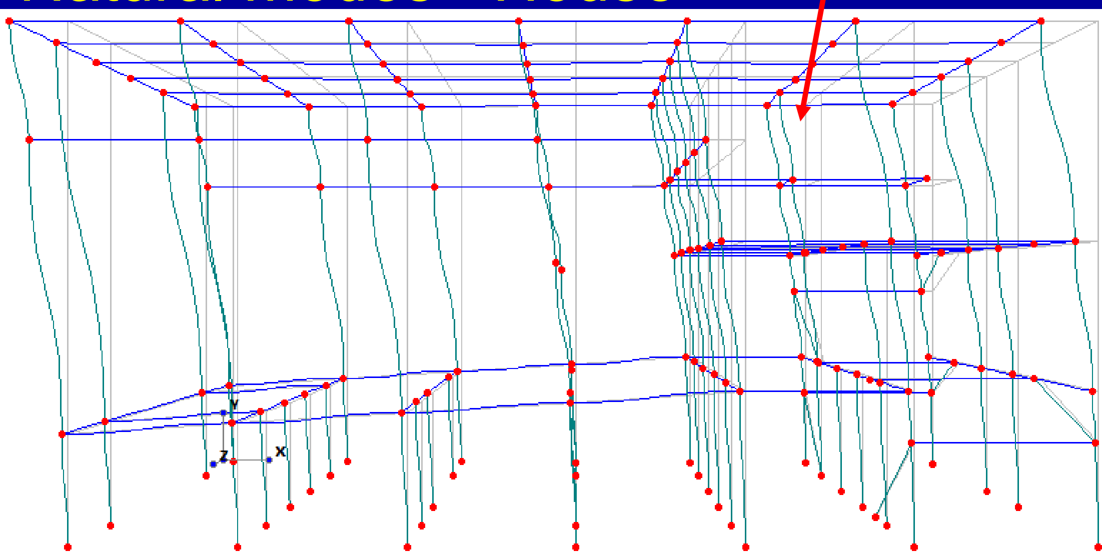
Natural modes - Stage:

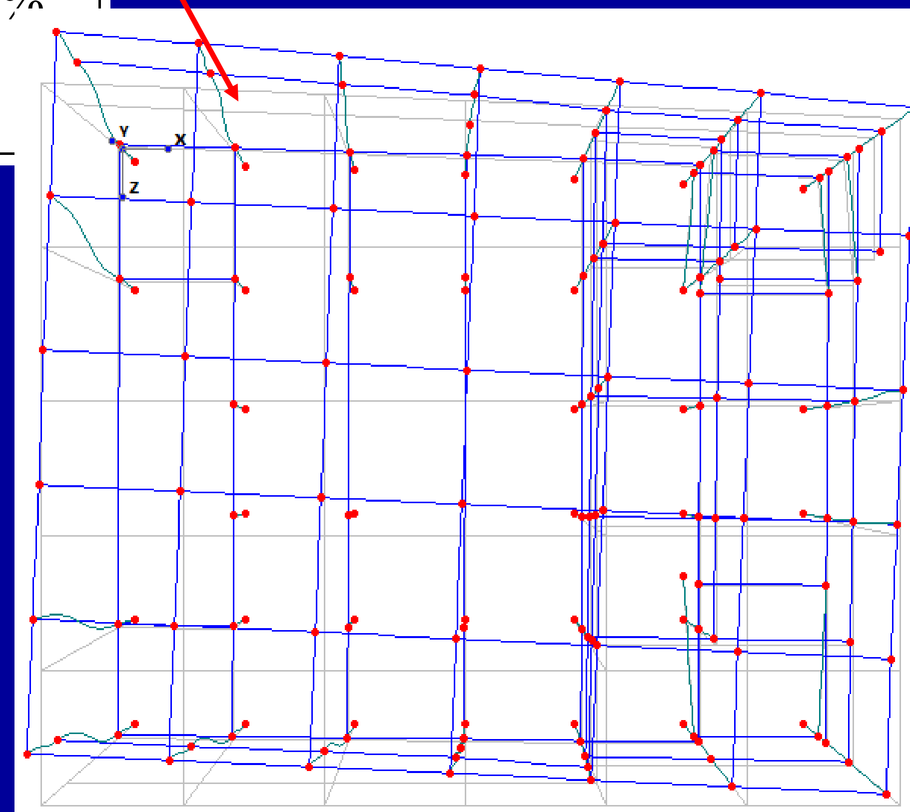
Stage	Period (s)		Mass X	Mass Z
1	1.05	1st-Z	0.1%	66.8%
2	0.84	1st-X (penthouse)	36.1%	0.1%
3	0.56	1st-X+penthouse	8%	-
4	0.51	Torsional	-	0.2%
5	0.38	2nd-X (penthouse)	24%	-
6	0.35	Z+twist	-	3.9%
7	0.27	Z+twist	-	12.6%
8	0.19	3rd-Z	0.4%	3.5%
9	0.18	3rd-X	6.8%	0.4%
10	0.17	Local mode	2.7%	-
11	0.135	4th-X	10.2%	0.2%
12	0.13	4th-Z	-	2.9%



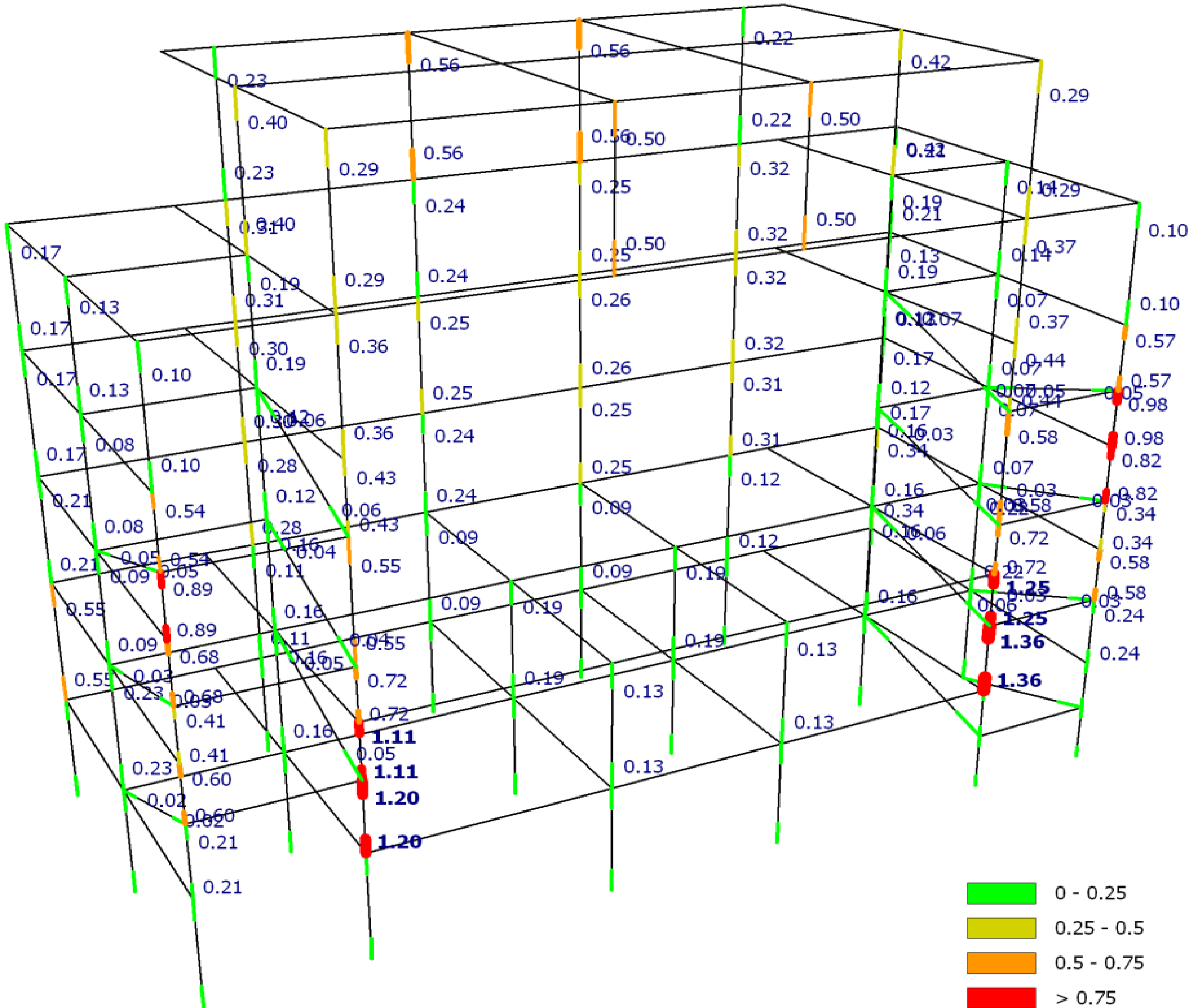
House	Period (s)		Mass X	Mass Z
1	1.08	1st-Z+twist	-	45.4%
2	0.97	1st-X	65.3%	
3	0.63	torsional	-	21%
4	0.26	torsion+local	0.1%	11.4%
5	0.215	local-X	6.9%	-
6	0.21	local-X	4.7%	-
7	0.21	local-X	6.8%	-
8	0.18	3rd-Z	-	9.5%
9	0.11	3rd-X	7.6%	-
10	0.08	4th-X	3.9%	-

Natural modes - House

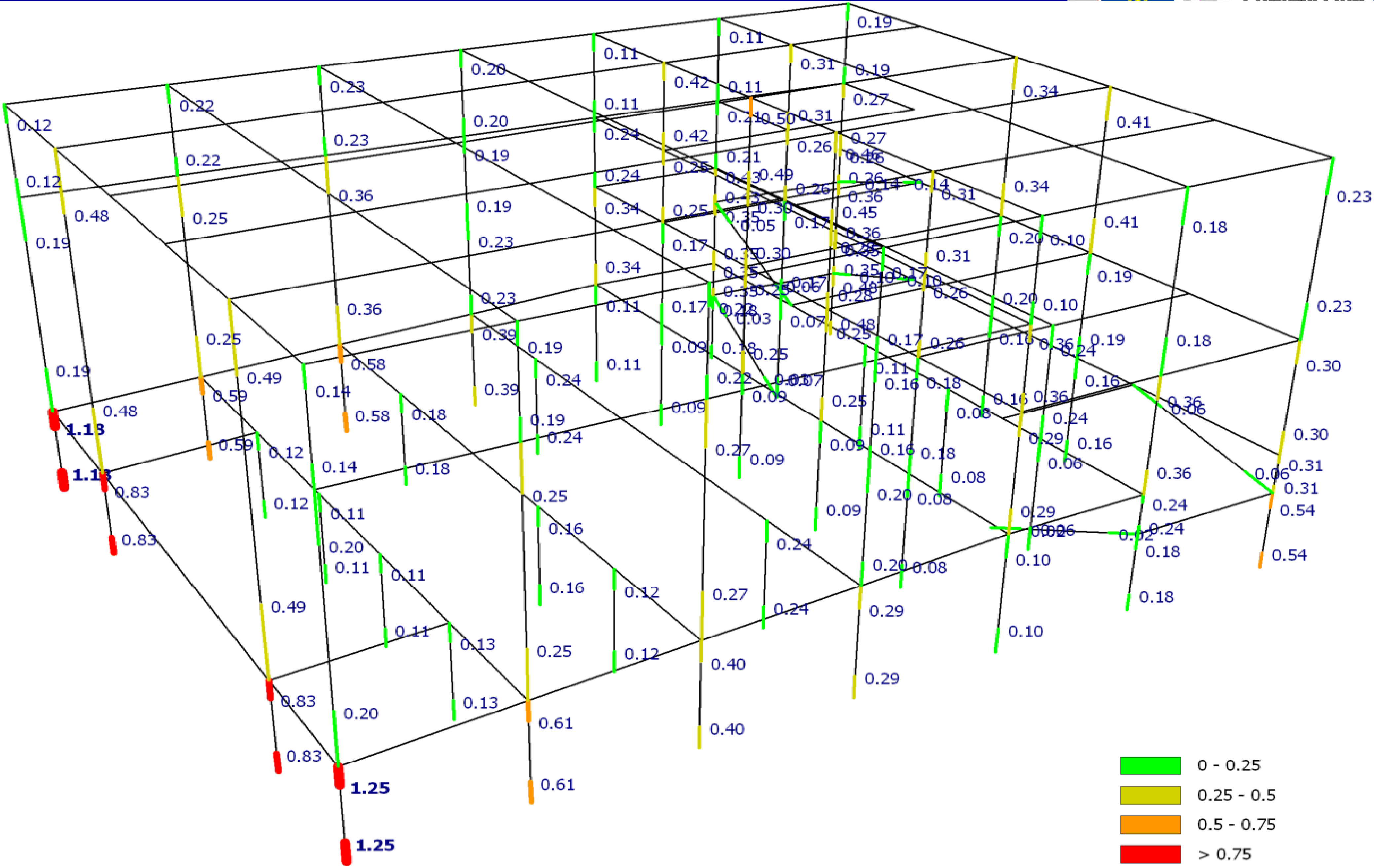




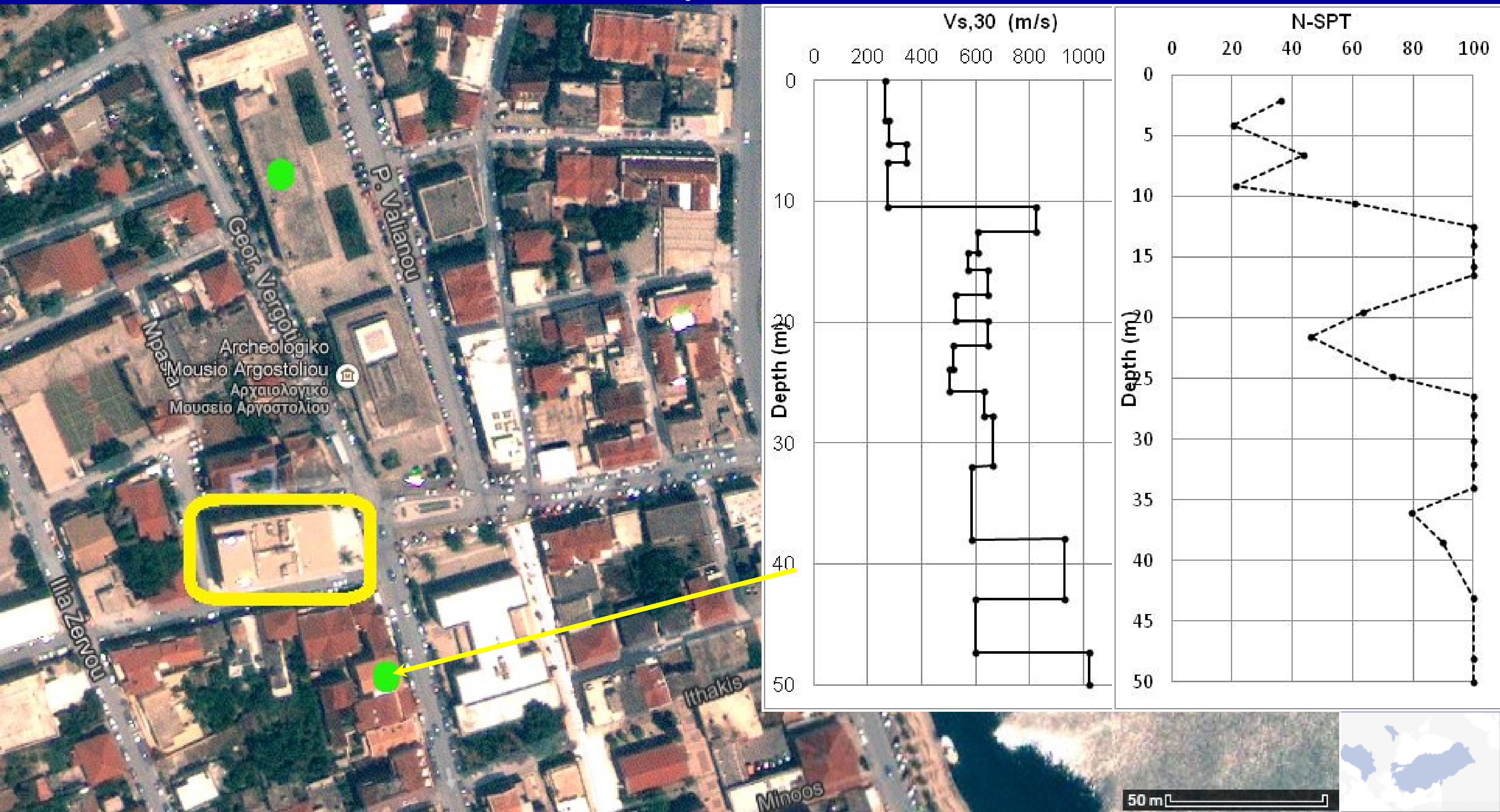
Demand-capacity ratios in **shear** of “Stage” for earthquake with Peak Ground Acceleration **PGA=0.1g**



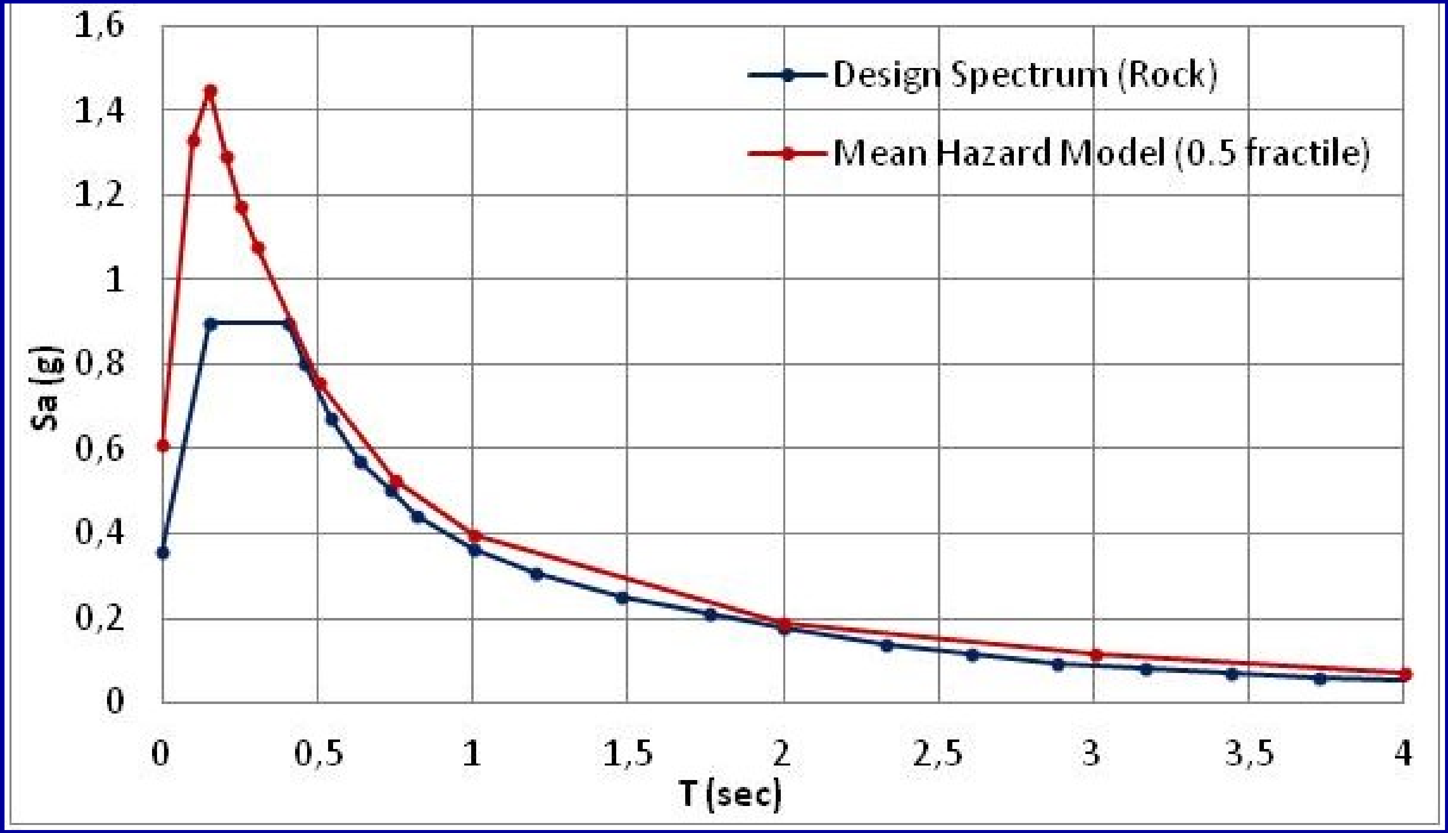
Shear demand-capacity-ratio for PGA=0.1g



- Geotechnical exploration with SPT and Cross-Hole measurements 40 m from the theater.
- Stiff pre-consolidated clay, with mean shear wave velocity in top 30 m: 430 m/sec, SPT N : 71 → Soil B per EC8.



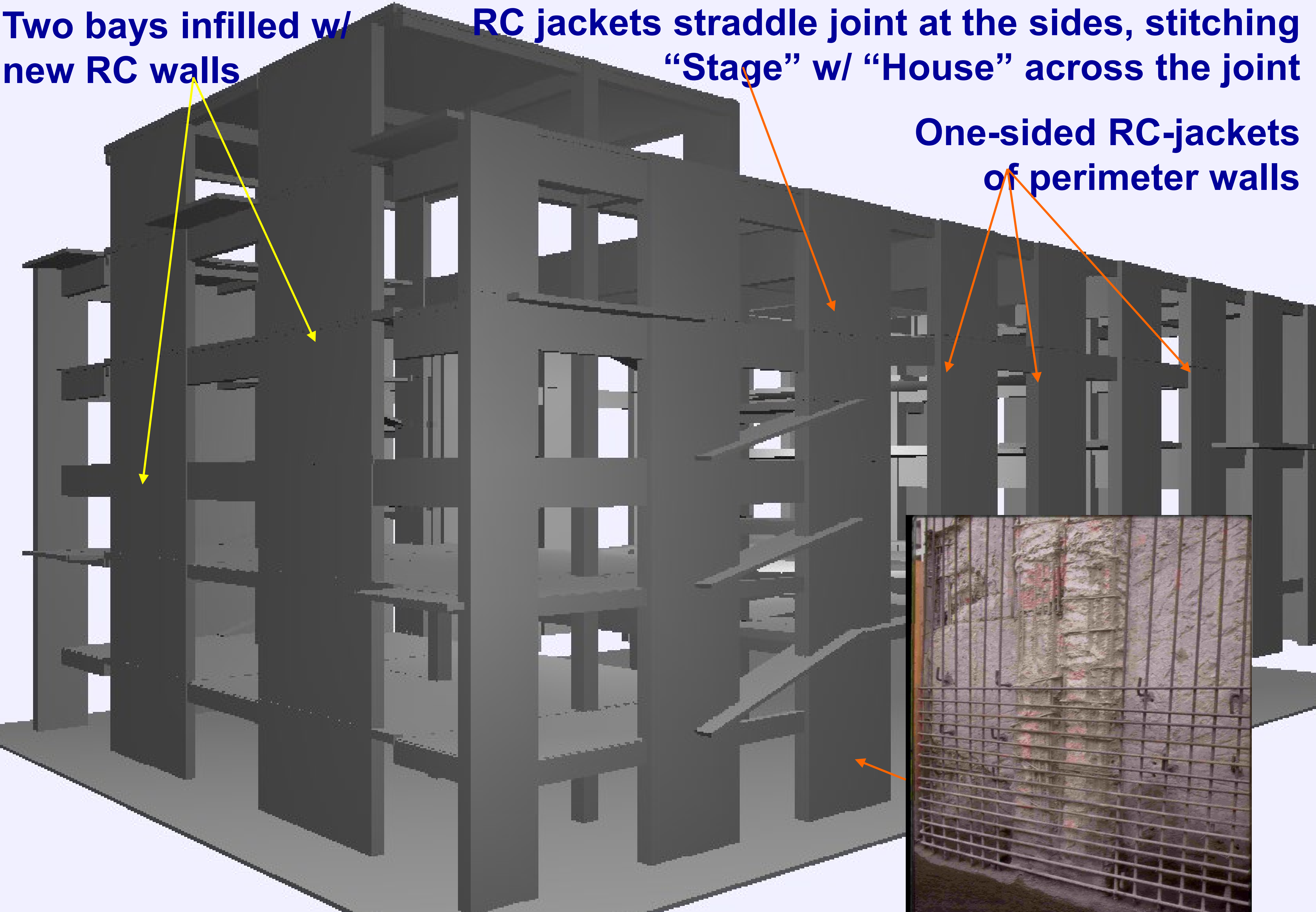
Seismic hazard at the site:



- Response spectrum at rock (soil A) with 10% probability of been exceeded in 50yrs (FP7 project SHARE: Seismic Hazard Map of Europe)
- Over the period-range of interest, SHARE spectrum agrees well with design spectrum, which has PGA on rock 0.36g.

July 2005: Design of retrofitting per EN1998-3:2005 complete (first real-life application)

- Interventions only to the exterior (facades);
- Address rebar corrosion in perimeter elements;
- No change in façade;
- Minimal alteration of two lateral sides;
- No disruption of use/operation of the facility during retrofitting works;
- Minimal intervention to foundation;
- In the end: Few interior interventions with FRP.
- **Retrofitting works completed in July 2007**



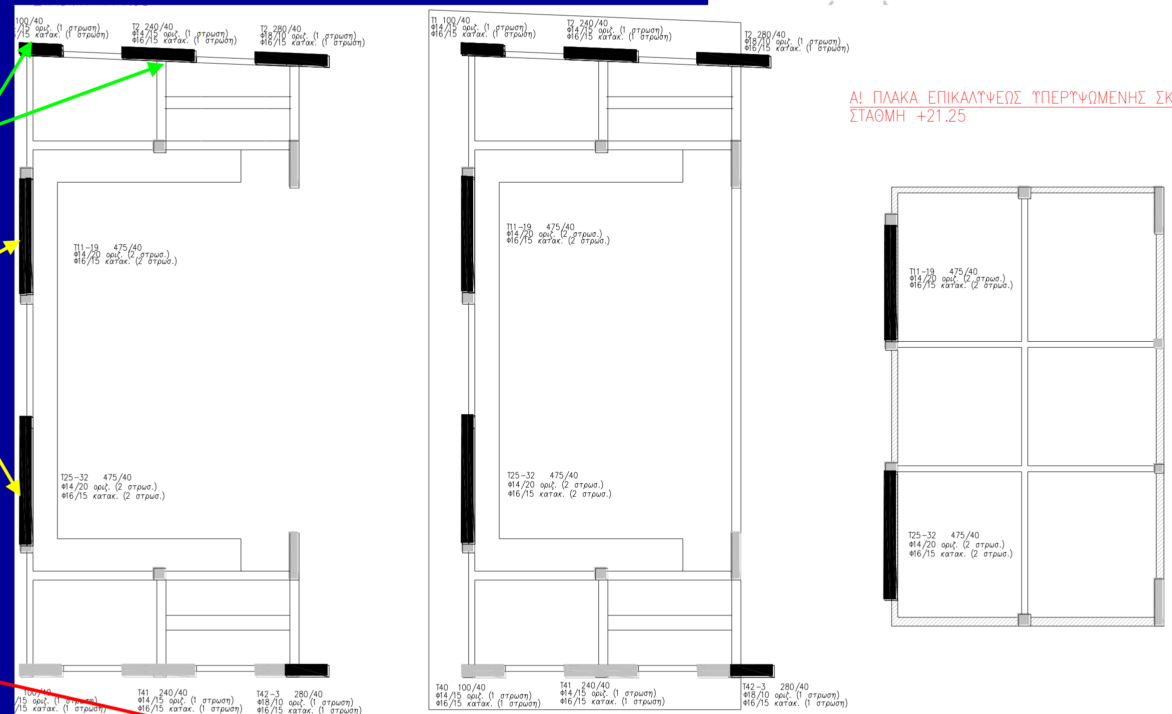
Two bays infilled w/ new RC walls
RC jackets straddle joint at the sides, stitching "Stage" w/ "House" across the joint
One-sided RC-jackets of perimeter walls

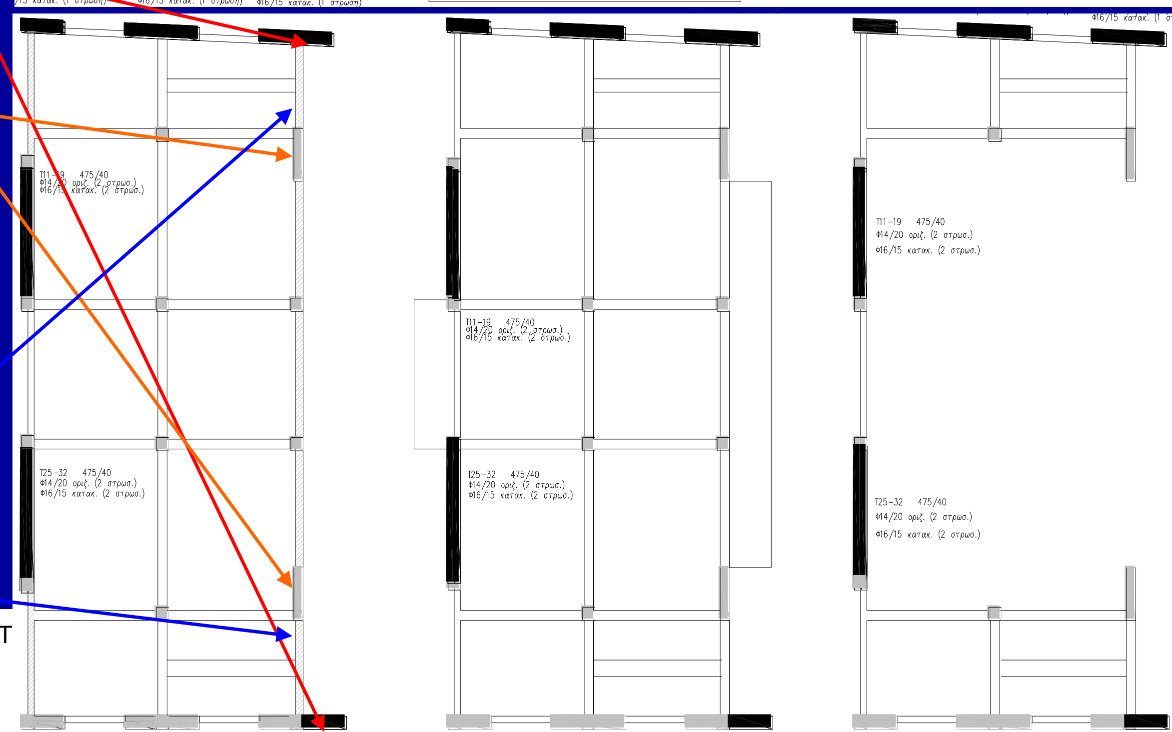
Strengthening of "Stage" part

1. RC-jacketing of perimeter walls (also due to bar corrosion).

2. Two bays infilled w/ new RC walls, from foundation to rooftop.

3. "Stage" stitched together w/ "House" across the joint into single structural unit (to prevent torsional response & pounding) via RC jackets straddling joint at the two sides, steel rods connecting interior walls across the joint & RC belt straddling joint at the roof.

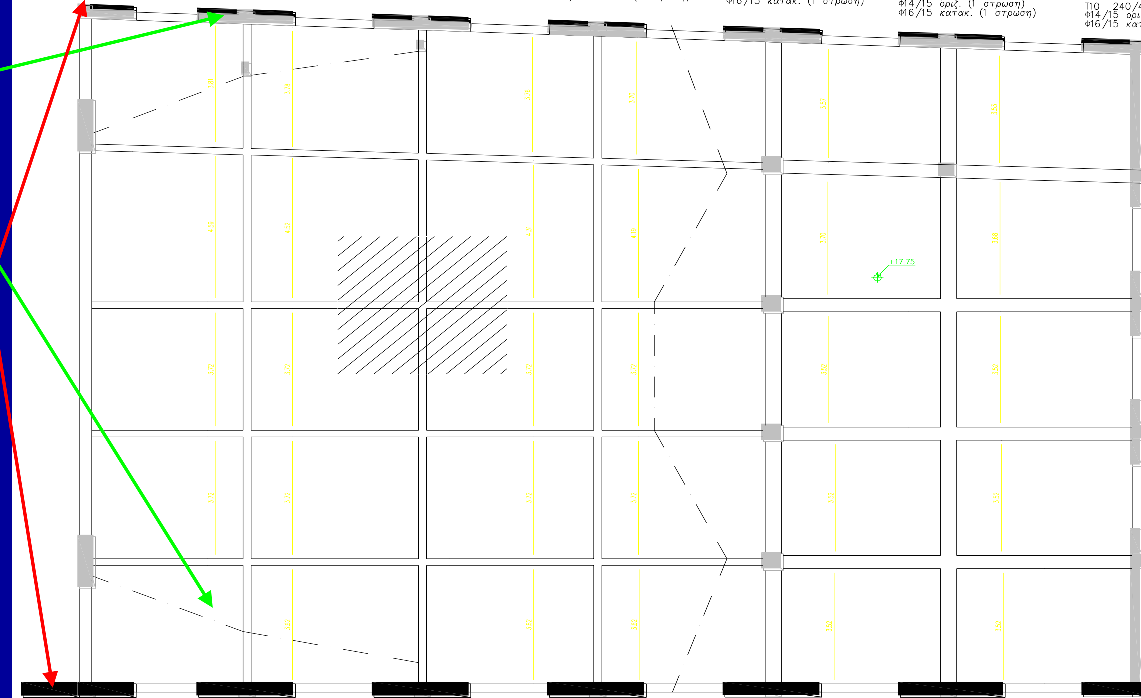


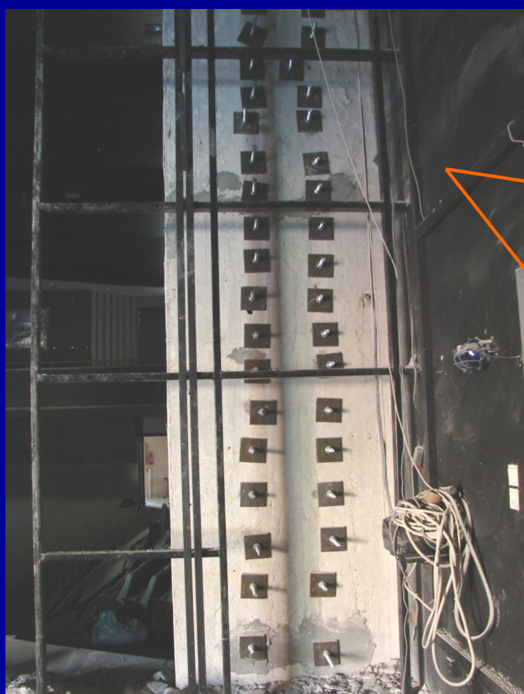


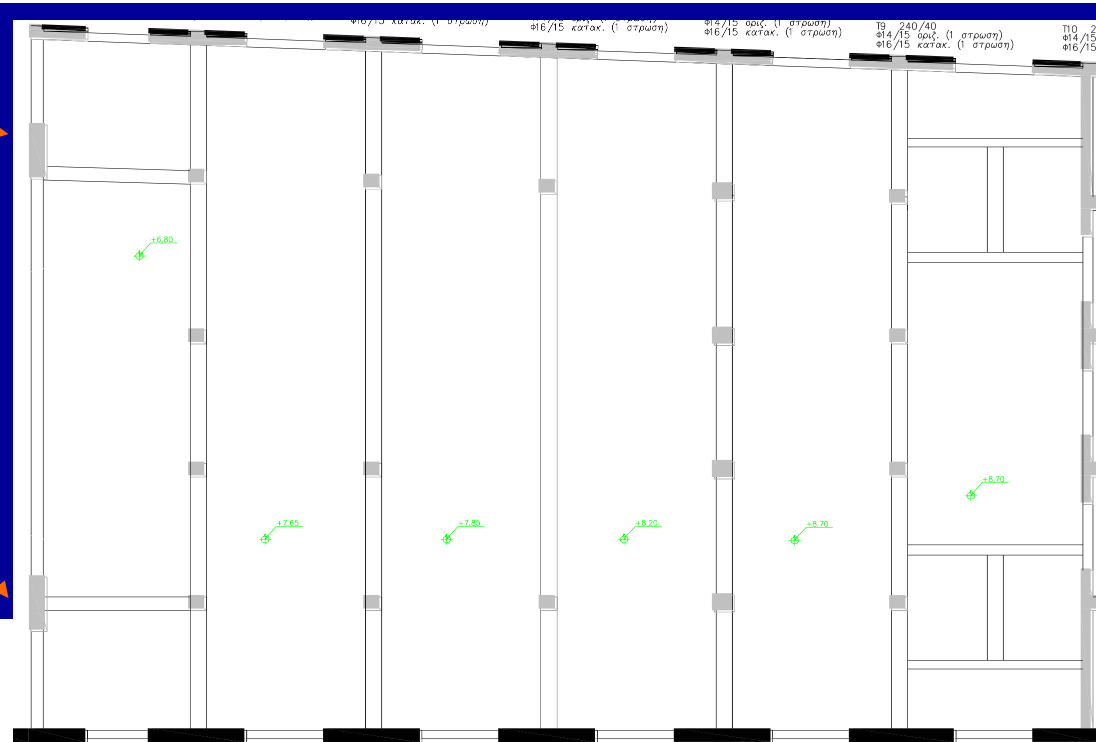
Strengthening of “House” part

1. RC-jacketing of perimeter walls (also due to bar corrosion).

2. “House” stitched together w/ “Stage” part across the joint into single structural unit, via RC jackets straddling joint at the two sides, RC belt straddling joint at the roof & steel rods connecting interior RC walls across the joint.

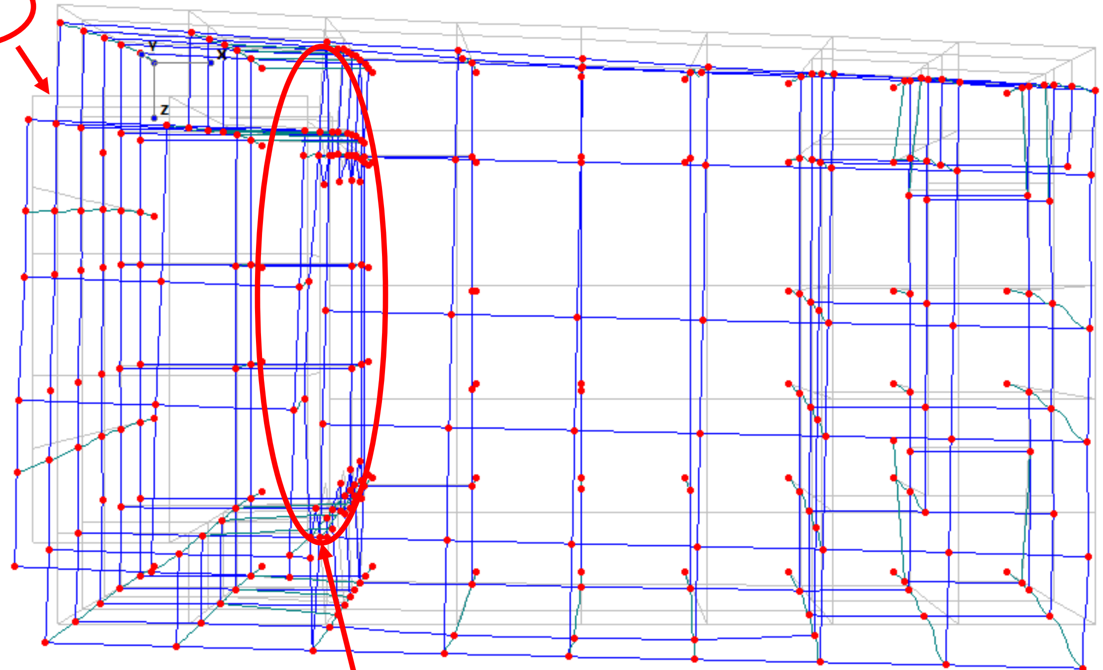


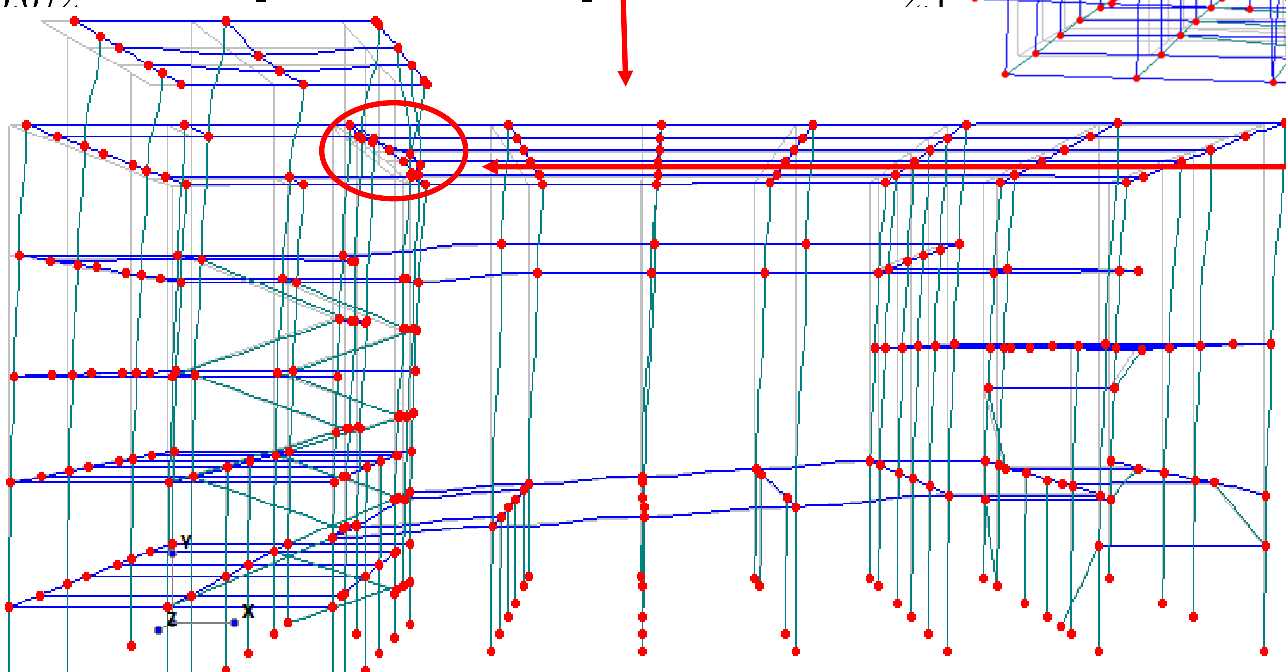




Retrofitted building: Horizontal normal modes

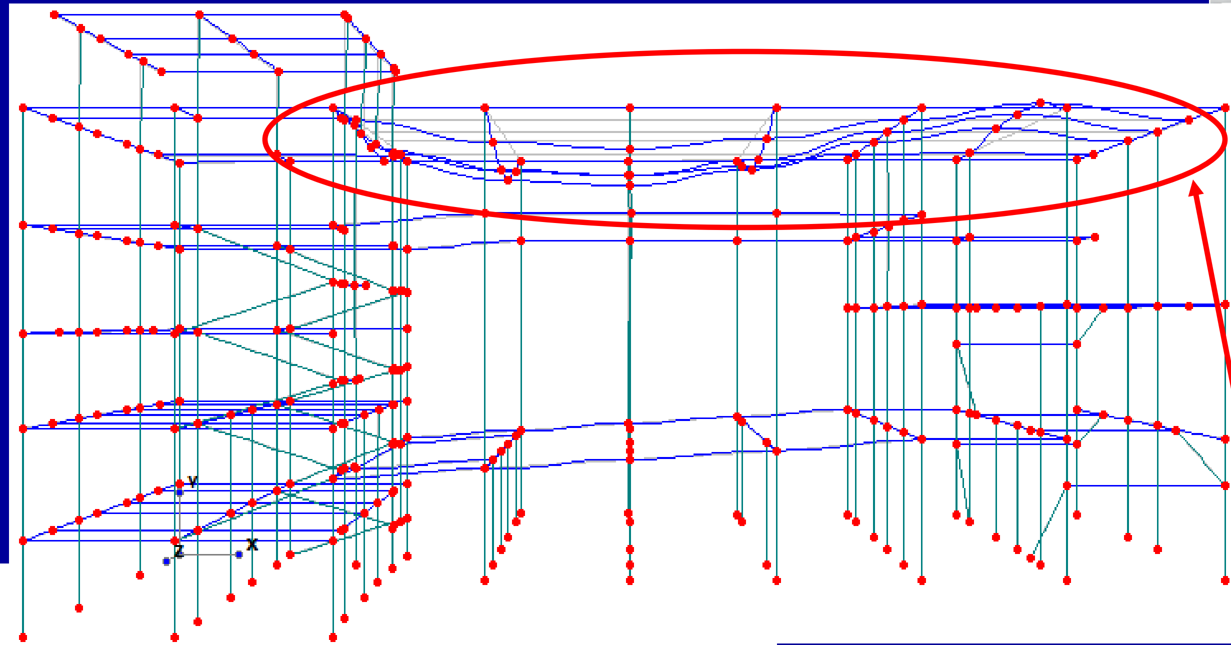
Period (sec)		Participating mass, % of total: EW (long) direction	Participating mass, % of total: NS (short) direction
0.76	translation+torsion	10.2	54.5
0.745	-	49.7	11
0.52	Torsion	-	2.8
0.48	penthouse	4.2	-
0.33	penthouse	2.8	-
0.24	-	-	8.1
0.17	-	15.1	-
0.16	-	-	1.9
0.15	-	-	4.1
0.14	-	-	3.3
0.09	-	3.2	
0.075	-	2.6-	-
0.072	-	-	2.1

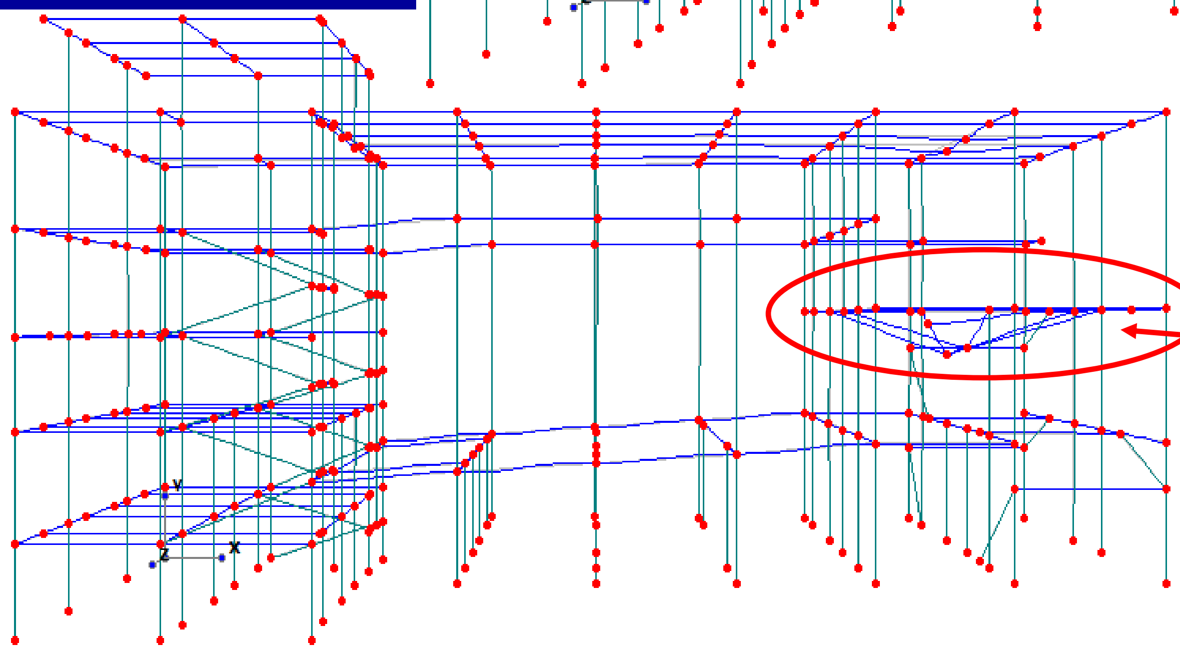




Roof beam supporting floating penthouse columns on the Stage side: still sensitive

Vertical modes

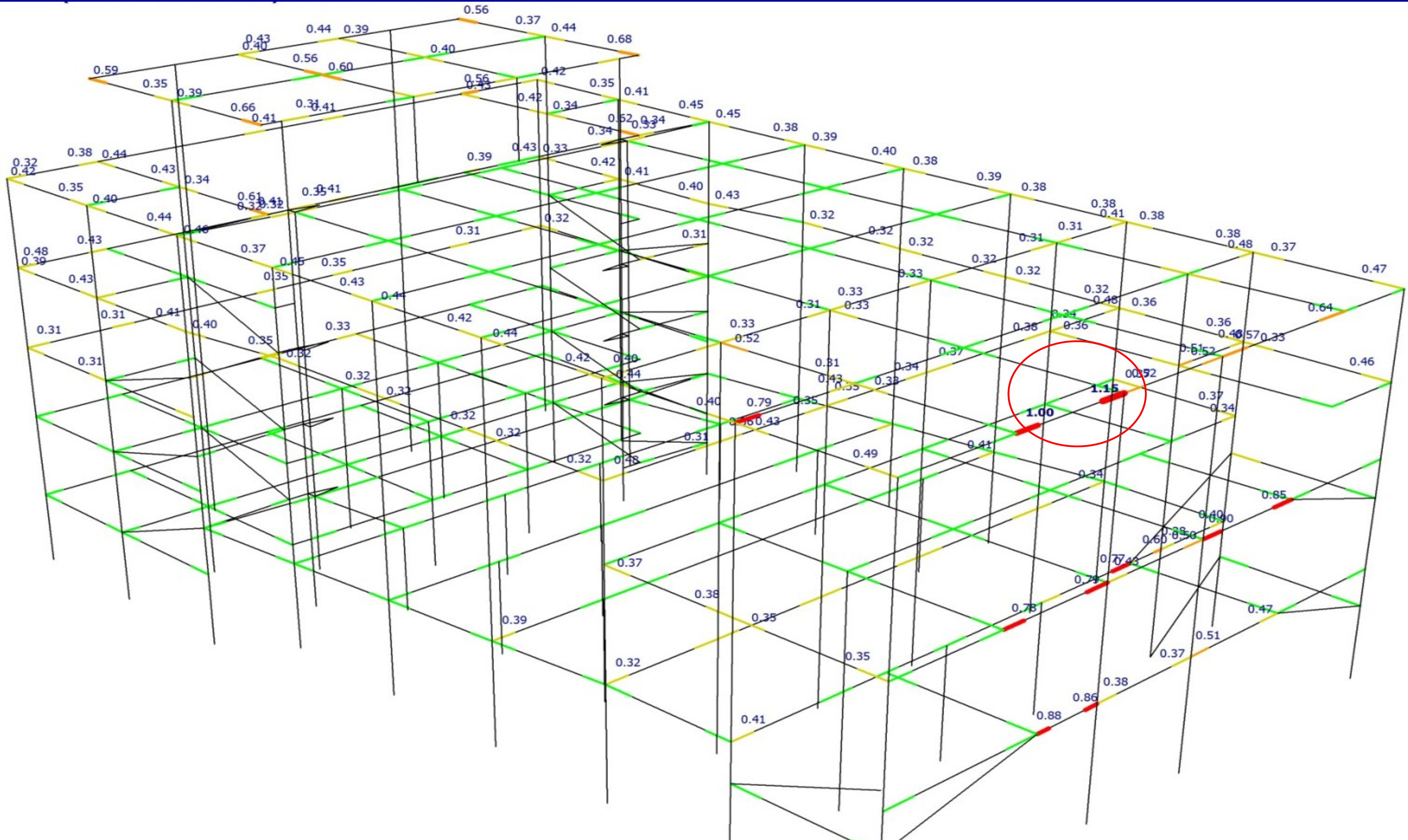




Mode	Period (sec)	Vertical mass
6th	0.39	6.42%
8th	0.30	2.87%
10th	0.27	2.09%
13th	0.23	1.08%
16th	0.20	2.66%

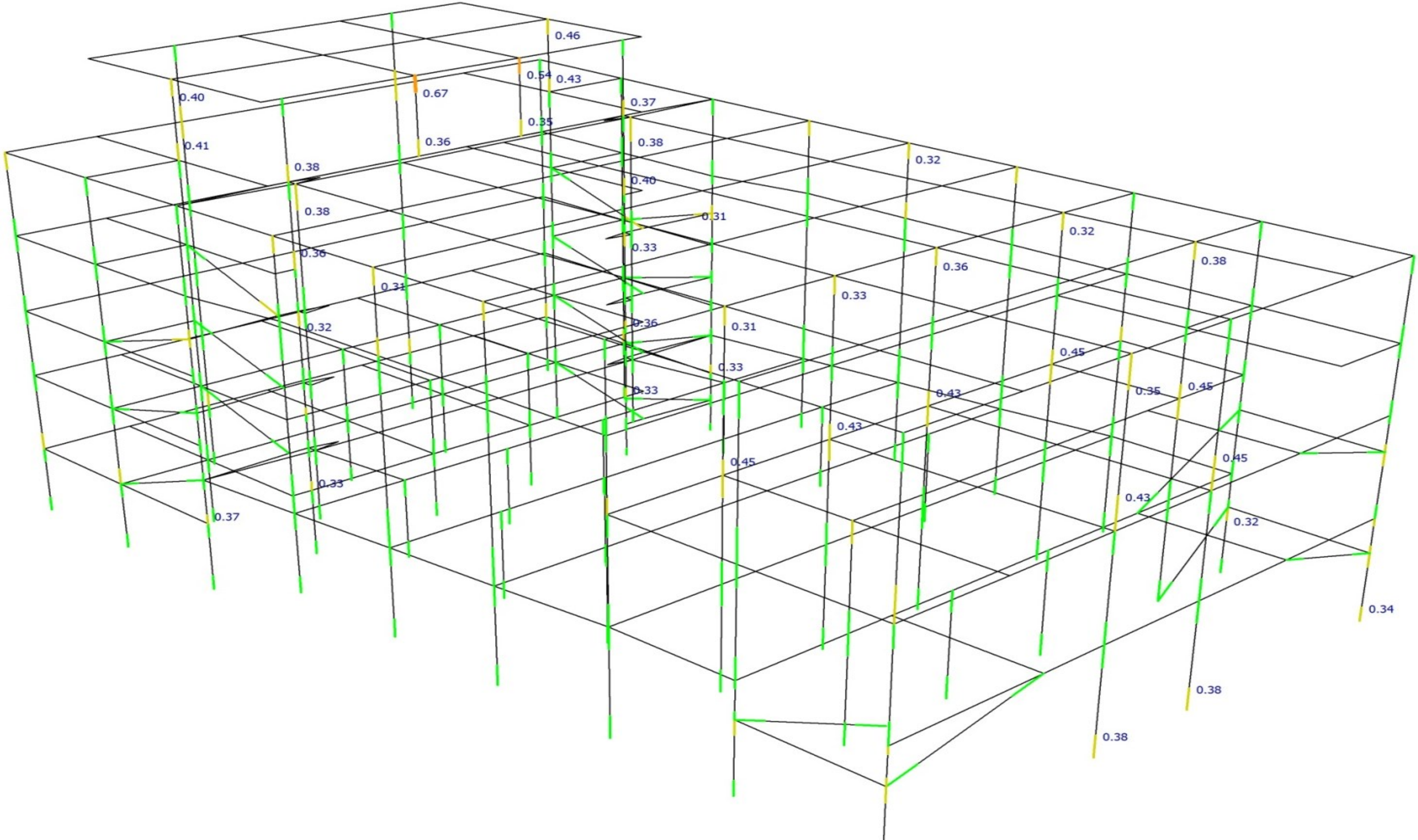
Vertical vibration of House roof & balcony

Demand-capacity ratios in **flexure** of **beams;** retrofitted building for 56 bidirectional ground motions at **PGA=0.36g**



Demand-capacity ratios in **flexure** of **vertical members;** in retrofitted building for 56 bidirectional ground motions with **PGA=0.36g**

European Commission

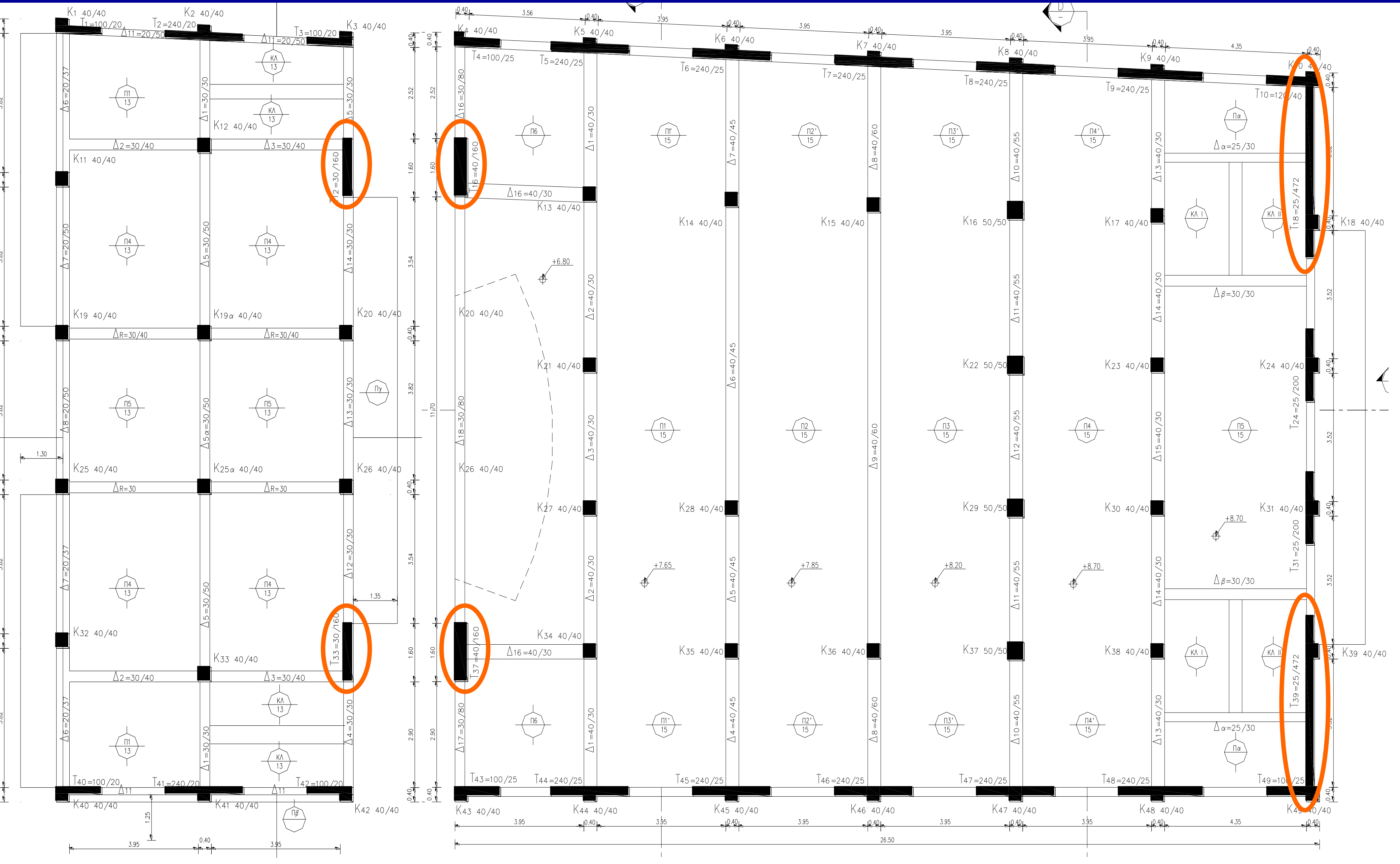


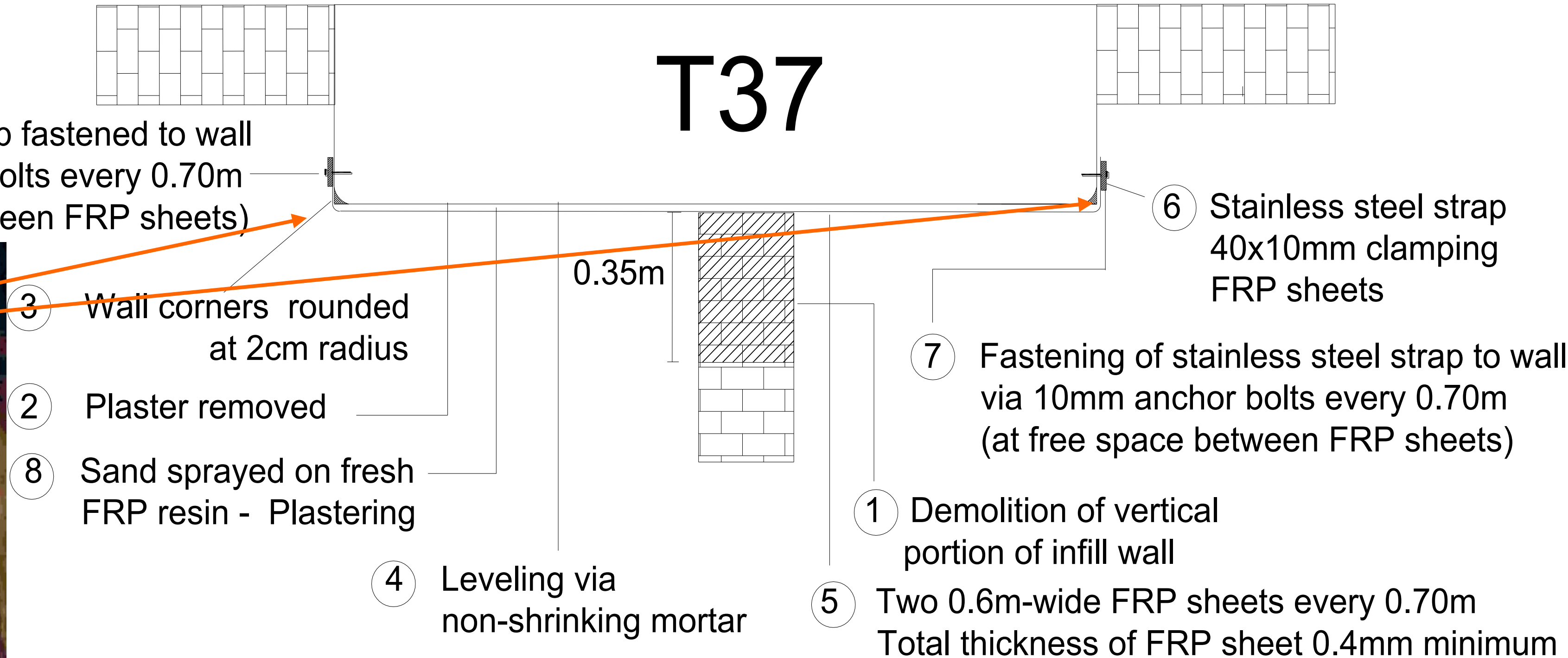
Conversion of demand-to-expected-capacity ratios to Limit States of Part 3 of Eurocode 8

Failure mode	Member	Limit State in Part 3 of Eurocode 8: Significant Damage	Limit State in Part 3 of Eurocode 8: Near Collapse	Expected capacity - ultimate condition
flexure	primary	2.0×values in Figures	1.5×values in Figures	Values in Figures
flexure	secondary	4/3×values in Figures	Values in Figures	Values in Figures
shear	primary	(1.32 to 1.72)×values in Figures	(1.32 to 1.72)×values in Figures	Values in Figures
shear	secondary	Values in Figures	Values in Figures	Values in Figures

One-sided CFRPs at shear-deficient walls (RC jackets unfeasible)

European Commission

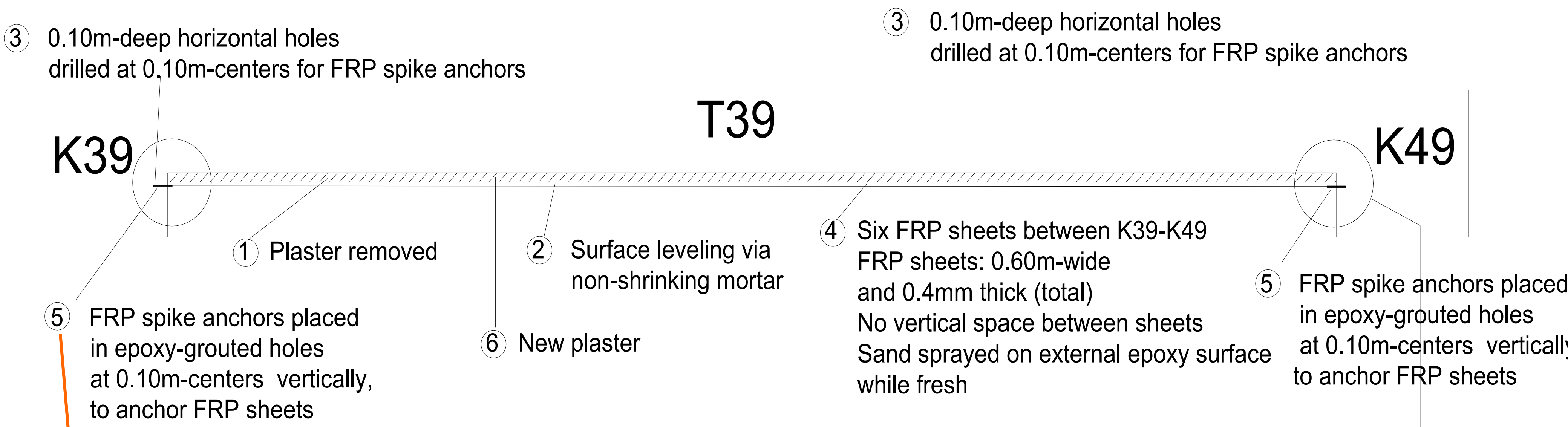


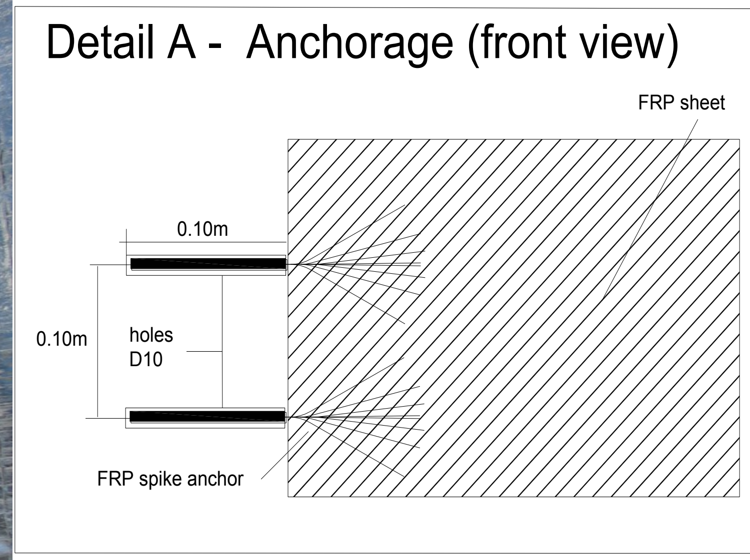


Four 1.6m-wide interior walls strengthened in shear on one side with CFRP –

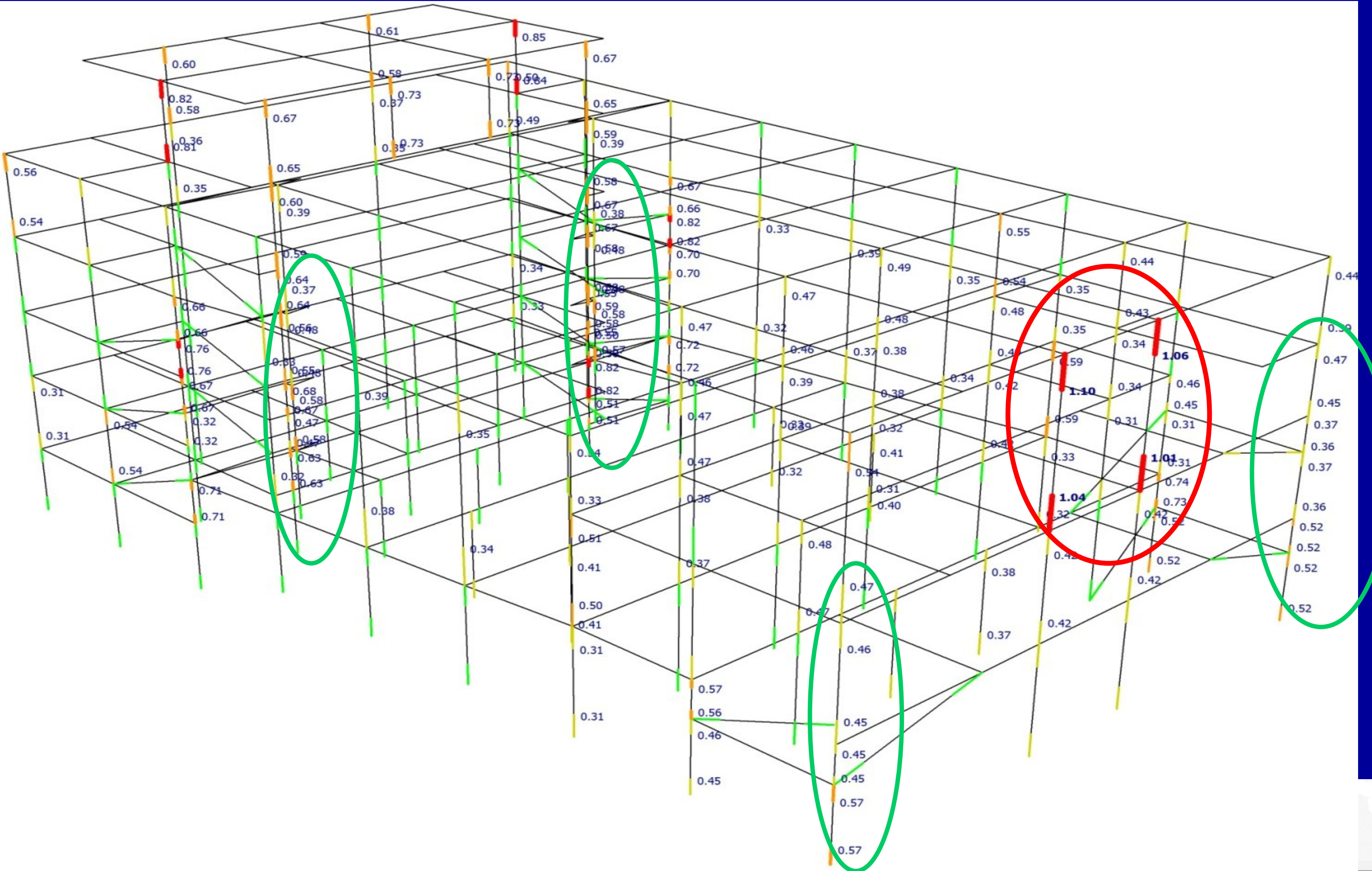
Total thickness of Carbon fiber sheets: 0.4-0.5mm

Two 3.5m façade walls strengthened in shear on one side with CFRP - Carbon fibre sheets of total thickness 0.4mm





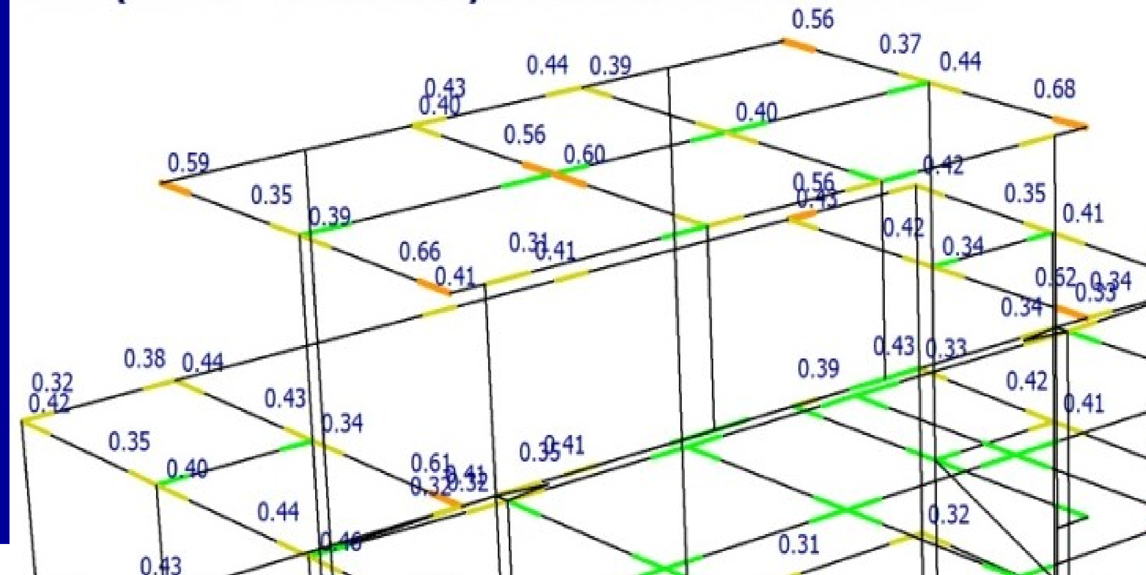
Demand-capacity ratios in **shear** – **vertical members**. CFRP retrofitted building, 56 bidirectional motions with **PGA=0.36g**

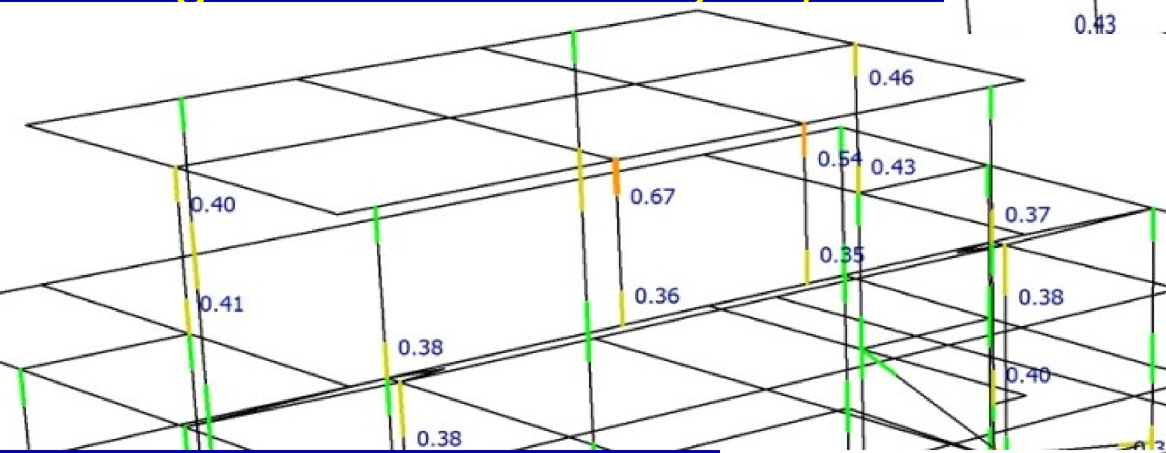


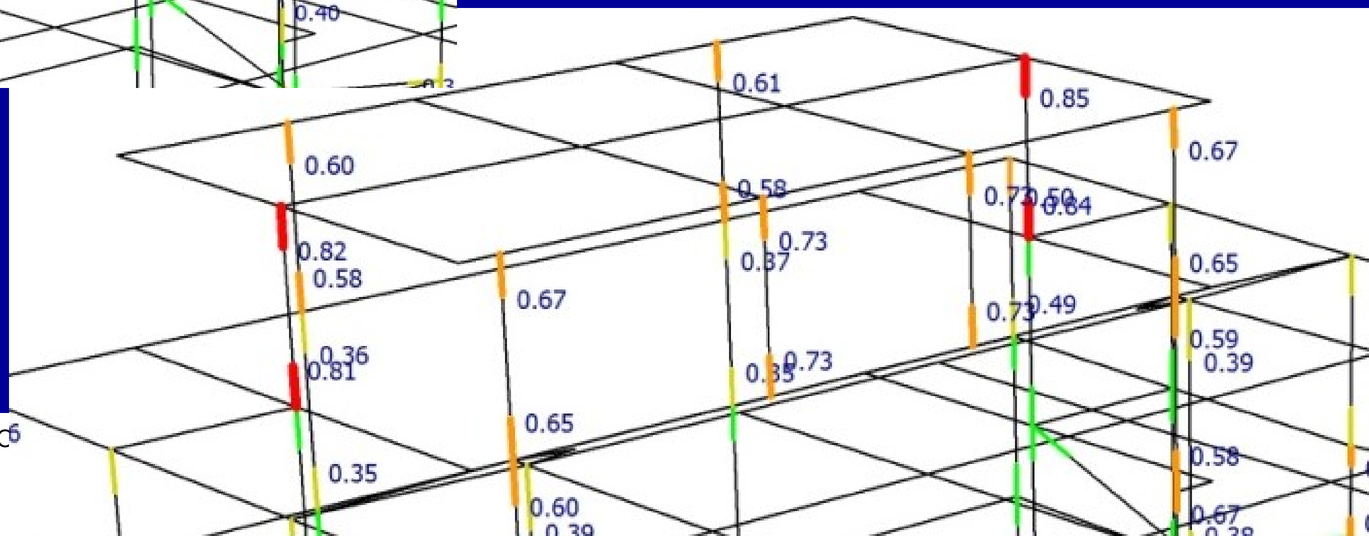
Budget (euros)

- Total cost, including all essential auxiliary works (removal/replacement of wall & floor finishings, etc) and 19% VAT €**240,000**
- (plus €70,000 to re-paint entire exterior & interior surfaces affected by the intervention)
- At €20/m^3 of building's volume: upgrade of seismic resistance by order-of-magnitude in terms of PGA

Vulnerable penthouse elements cannot be retrofitted without jeopardizing supporting beams (:vital for the building) → Kept as built, protected by masonry infills around the penthouse (which were ignored in the analysis)





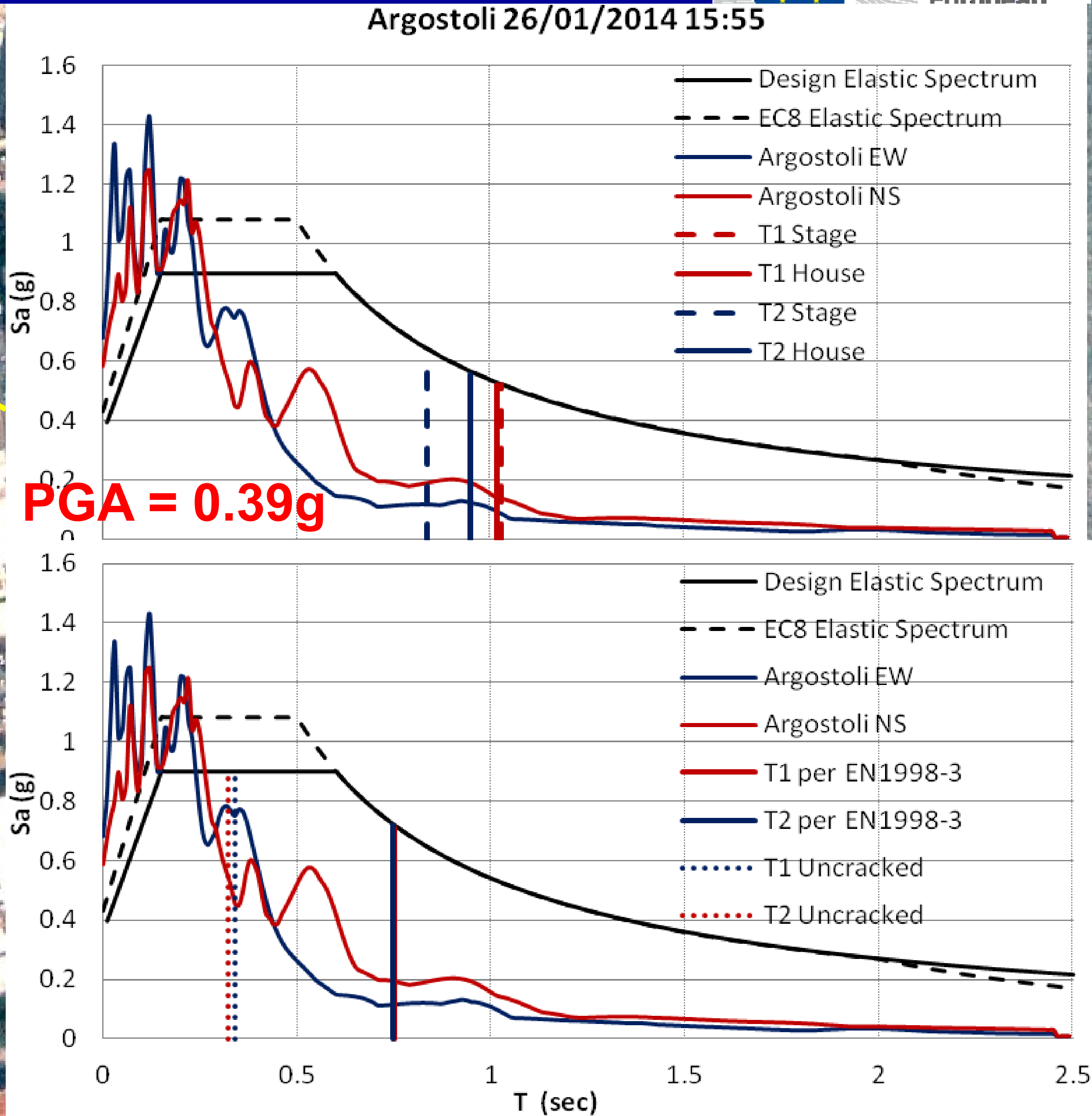


6.1M Earthquake

26-01-2014

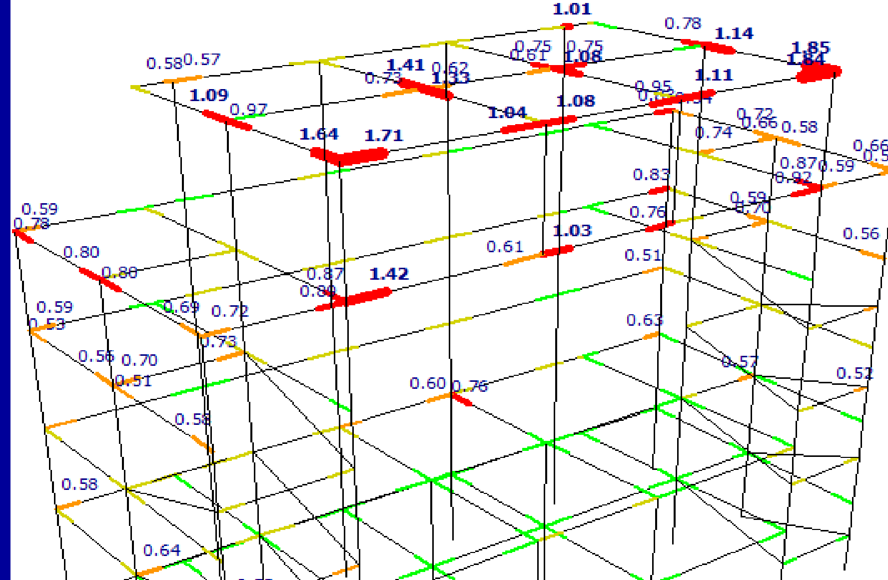
IC ACTIONS

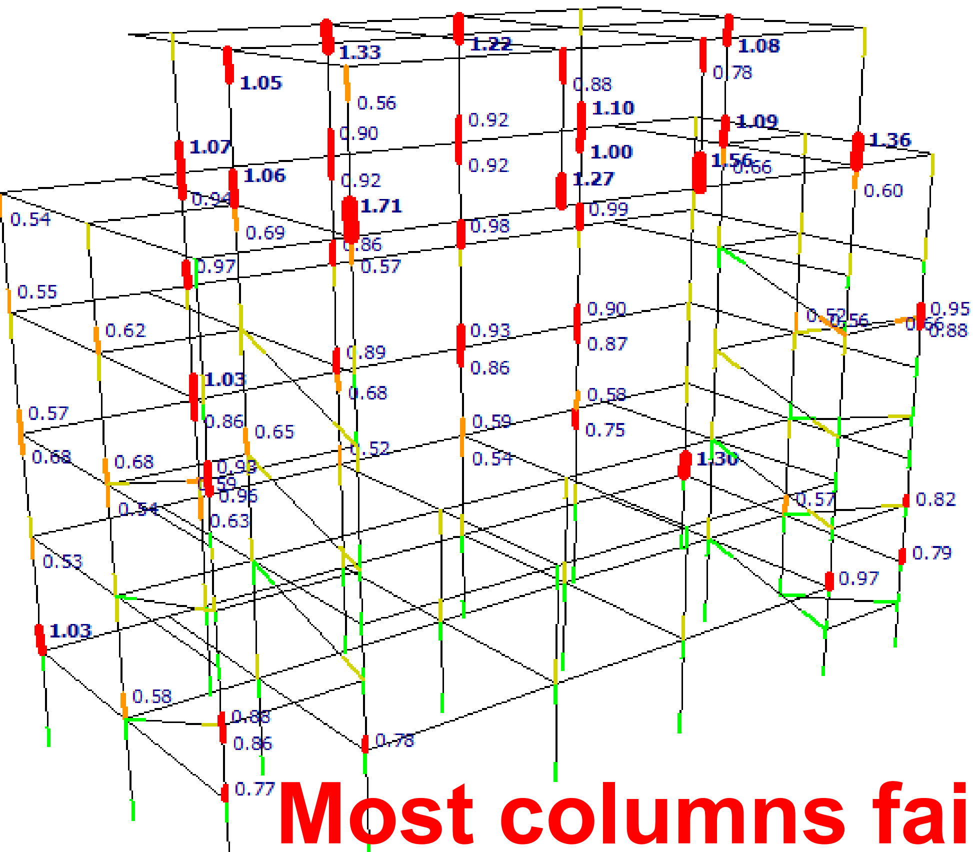
6.1M Earthquake 26/1/2014 recorded 90m from site

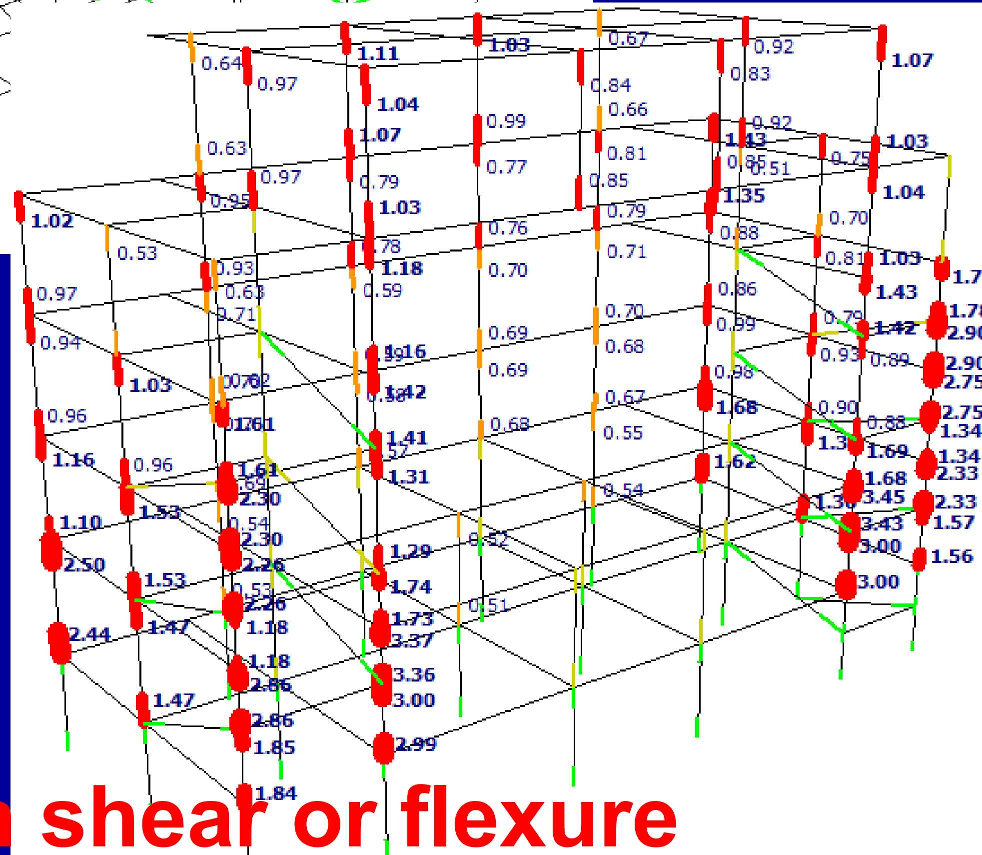


As-built Stage, if it were subjected to 26-01-2014 earthquake

European Commission

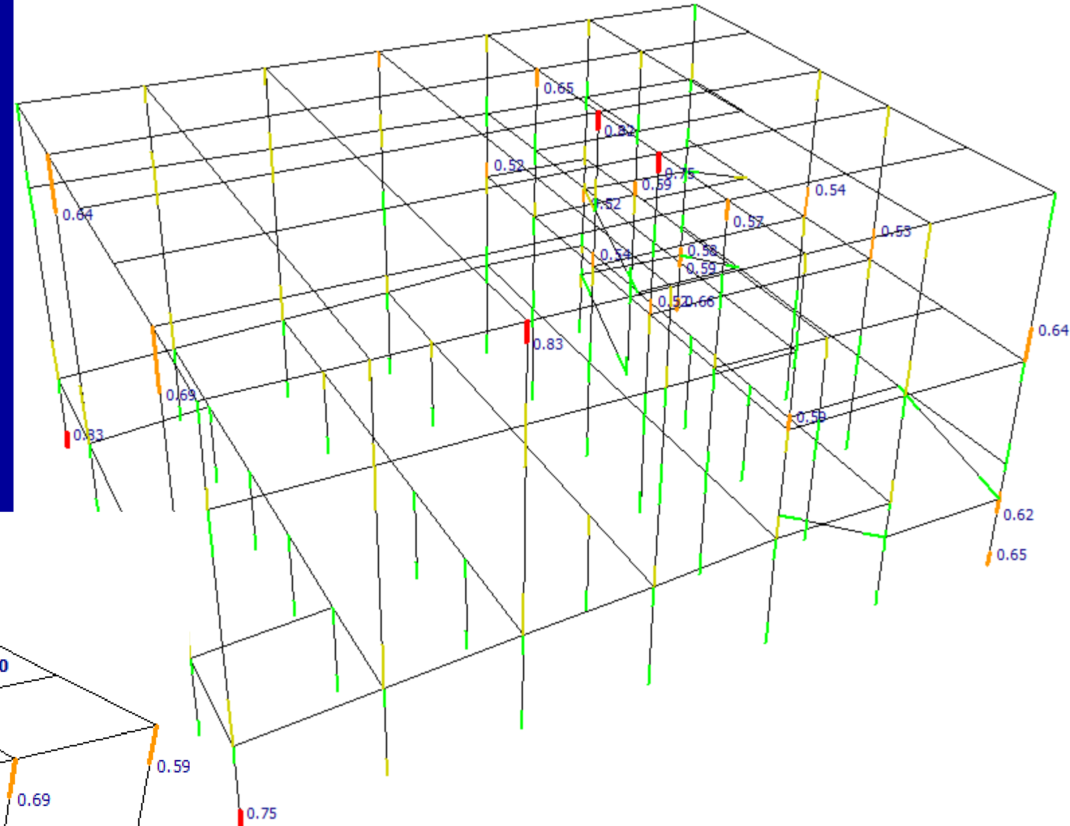


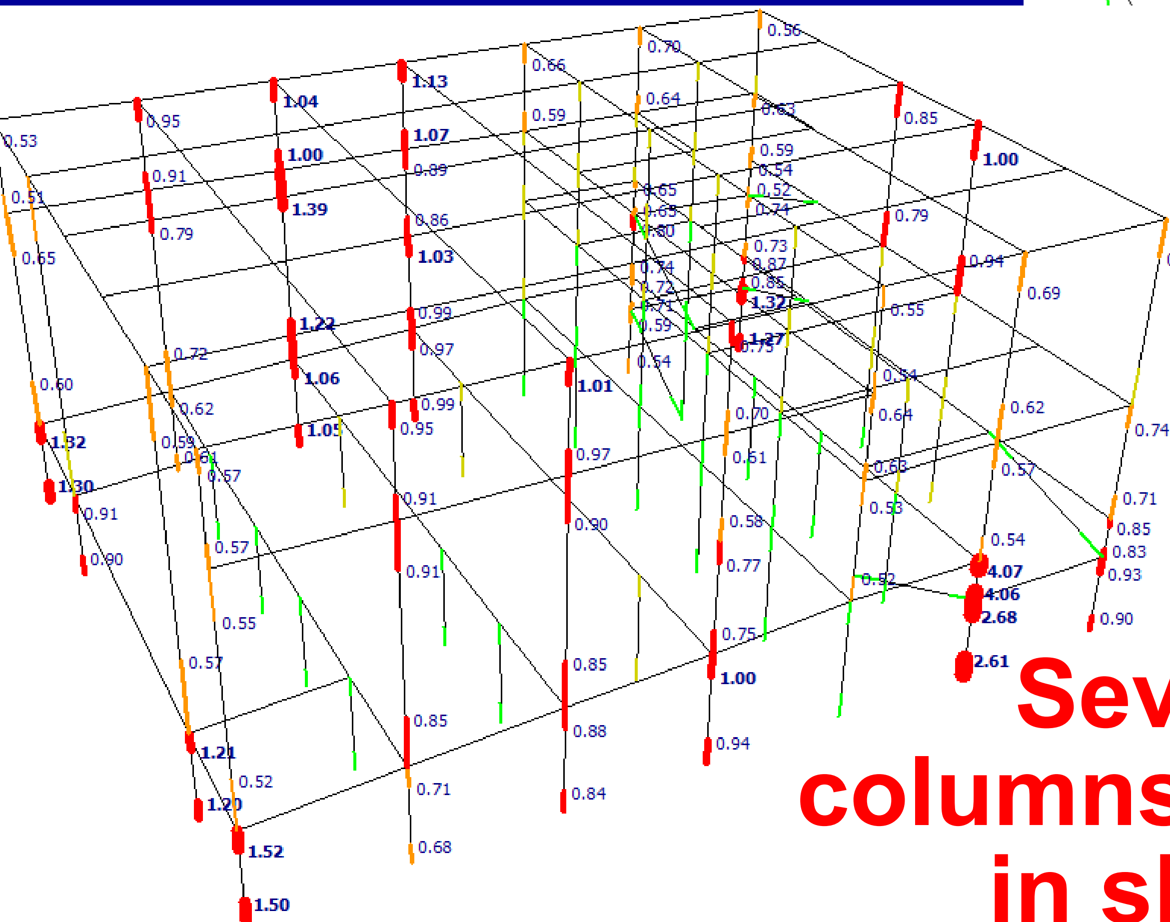




Most columns fail in shear or flexure

As-built House, if it were subjected to 26-01-2014 earthquake

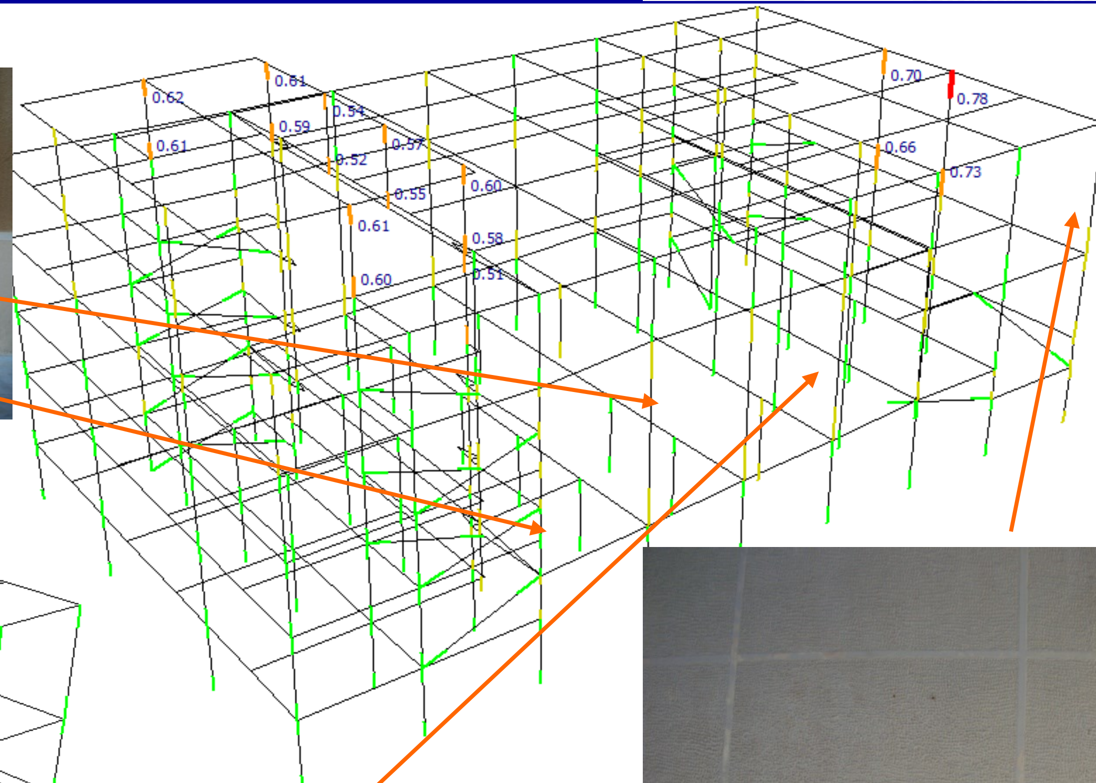


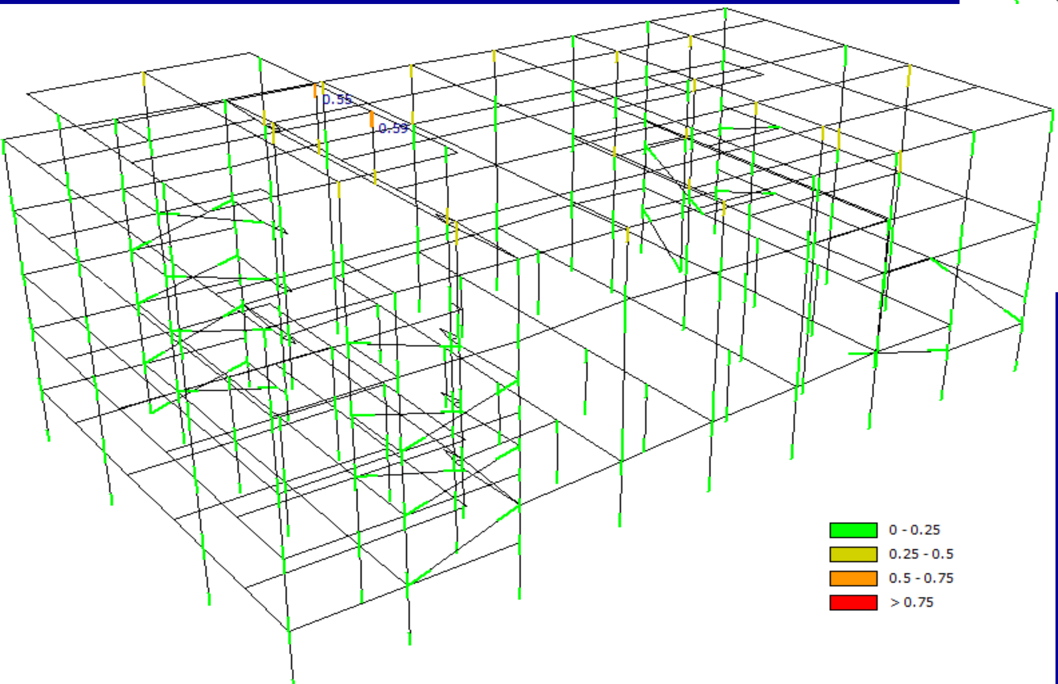


Several columns fail in shear

Retrofitted building, 26-01-14 earthquake

Undamaged

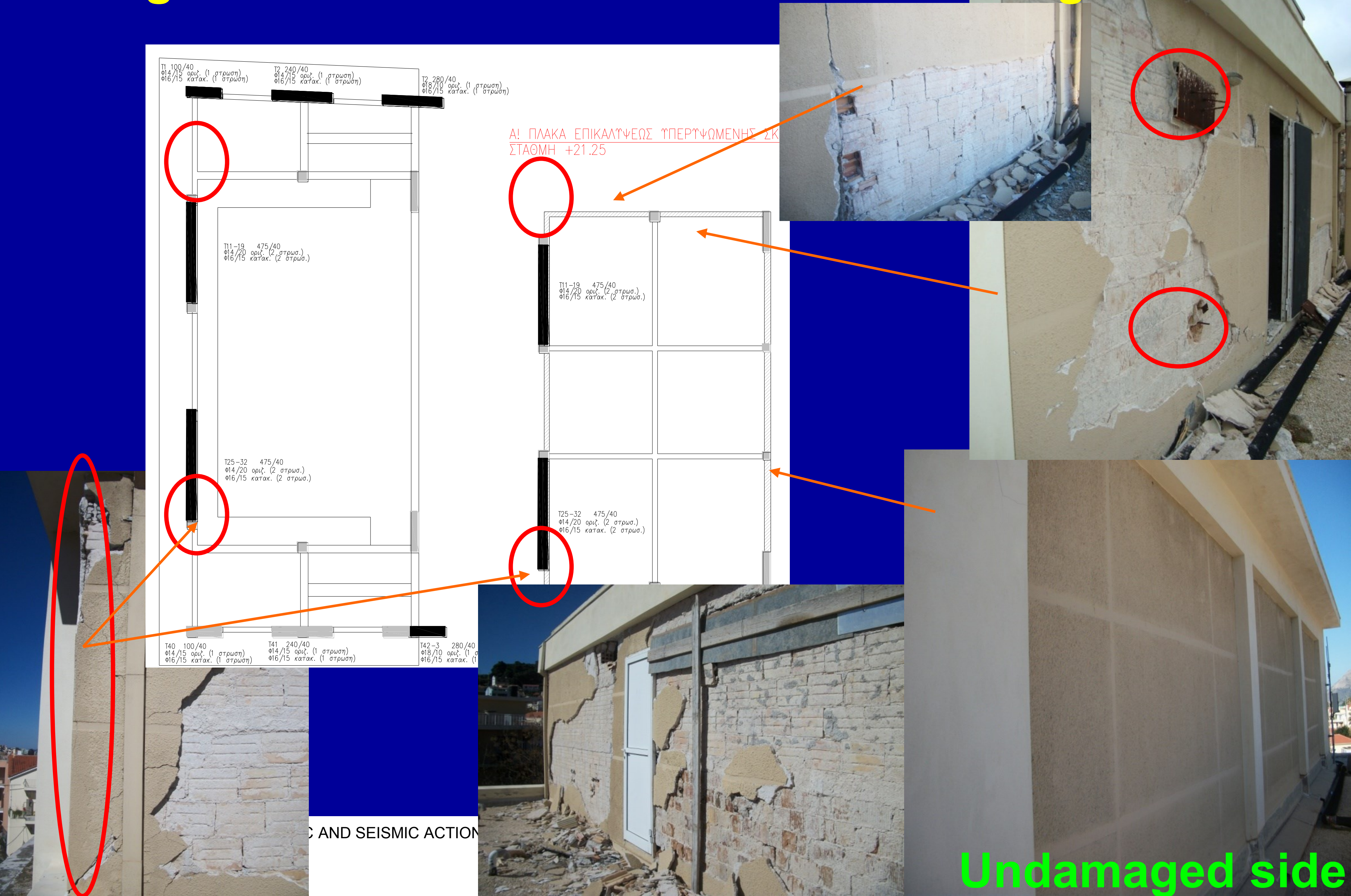




Masonry infills on two opposite sides of penthouse (supporting the grid, the fly gallery, the drapery etc of the stage): seriously damaged.

Those on two other sides: undamaged

Damaged infills: no column at one vertical edge



Conclusion: Performance, 26-01-14 EQ

- Earthquake exceeded design level in PGA & spectral values in the range of higher modes; approached design levels in the range of fundamental periods of uncracked building, but much below them at fundamental period(s) for secant-to-yield-point stiffness.
- The retrofitted structure performed well, as foreseen in the analysis and the design of the retrofitting.
- Masonry infills around the penthouse performed according to the design of the retrofitting, shielding the columns of the penthouse from damage (especially in the EW direction in which the penthouse was more flexible & the ground motion stronger).
- The infill walls of the penthouse were damaged because:
  - Stage grid, fly gallery etc were suspended from these walls; &
  - At one of their two vertical edges there was no column.

Thank you!