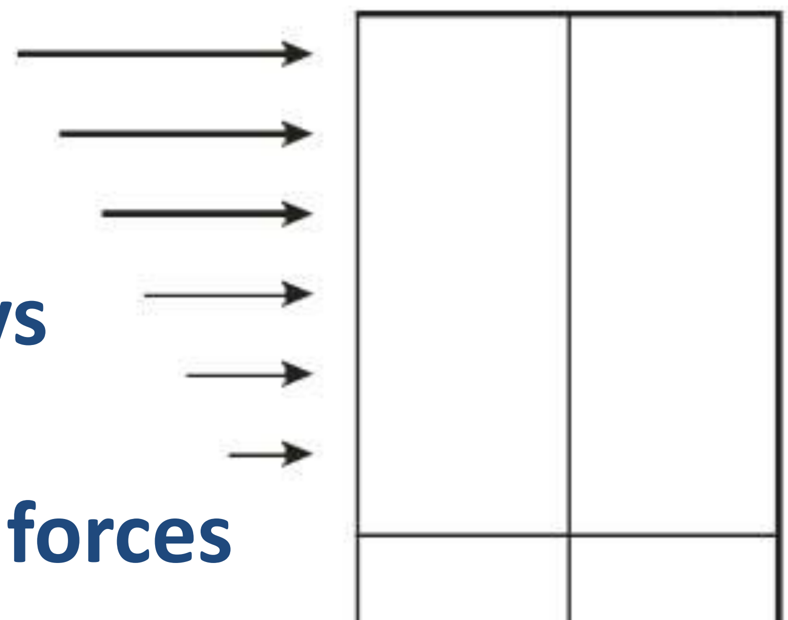
16th European Conference of Earthquake Engineering
Thessaloniki, June 18-21, 2018

From force-based to displacement-based seismic design and beyond

Michael N. Fardis
University of Patras, Greece

Structures move or collapse sideways in earthquakes

Early-days belief:



Structures can avoid sideways collapse in earthquakes, if designed to resist horizontal forces

How strong should these forces be?

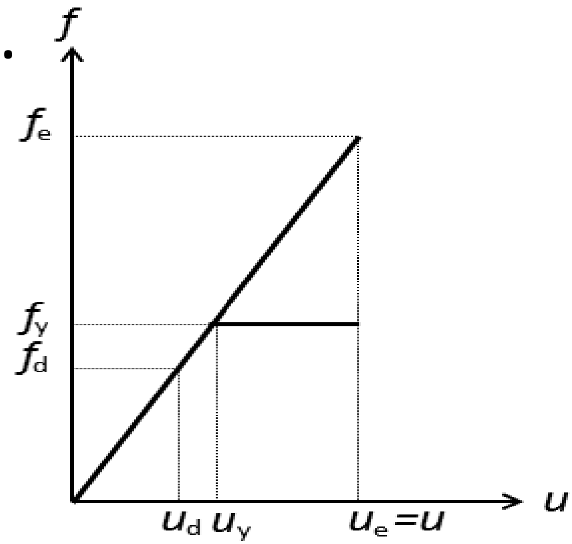
- Earthquake-induced accelerations→ forces (% of a structure's weight).
- Early ground acceleration measurements: Peak values ~ 0.1 - 0.2g.
- Later measurements: much higher accelerations → lateral forces 50% to 100% of the weight!
- Unfeasible to design structures for such a lateral force resistance.
- Early conclusion: keep magnitude of forces low – rationalize choice:
- No need of structure to stay elastic under design earthquake→ design for a fraction, R or q, of the force it would had felt, had it stayed elastic
- R or q: force-reduction or behaviour factor, with (arbitrarily chosen) value: 3 to 10

Present-day: Force-based design for ductility

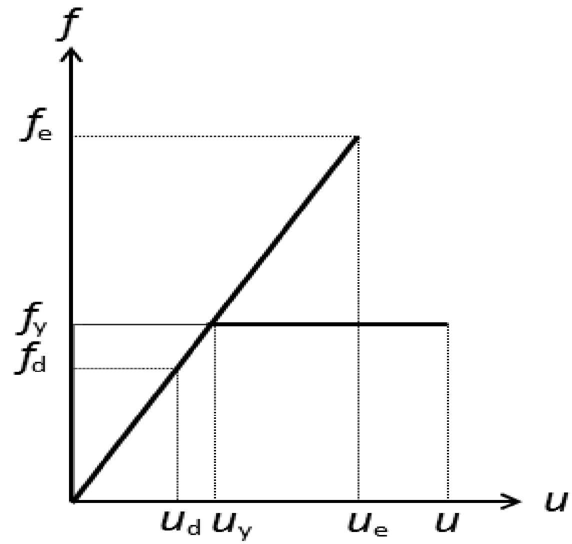
- Linear-elastic analysis (often linear dynamic analysis of sophisticated computer model in 3D) for lateral forces due to an earthquake R- or q-times (i.e., ~3-10-times) less than the design earthquake.
- Design calculations apply only up to 1/R of the design earthquake.
- **Rationalization**: It suffices to replace dimensioning for the full design earthquake with detailing of the structure to sustain through ductility inelastic deformations ~R-times those due to its elastic design forces.
- **Basis**: “Equal-displacement rule”.

"Equal-displacement rule"

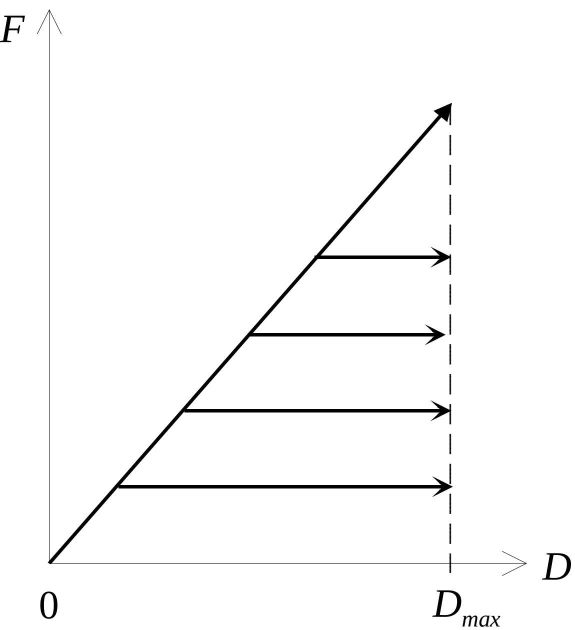
Empirical observation that earthquakes induce inelastic displacements ~equal to those induced had the structure remained elastic.



It applies

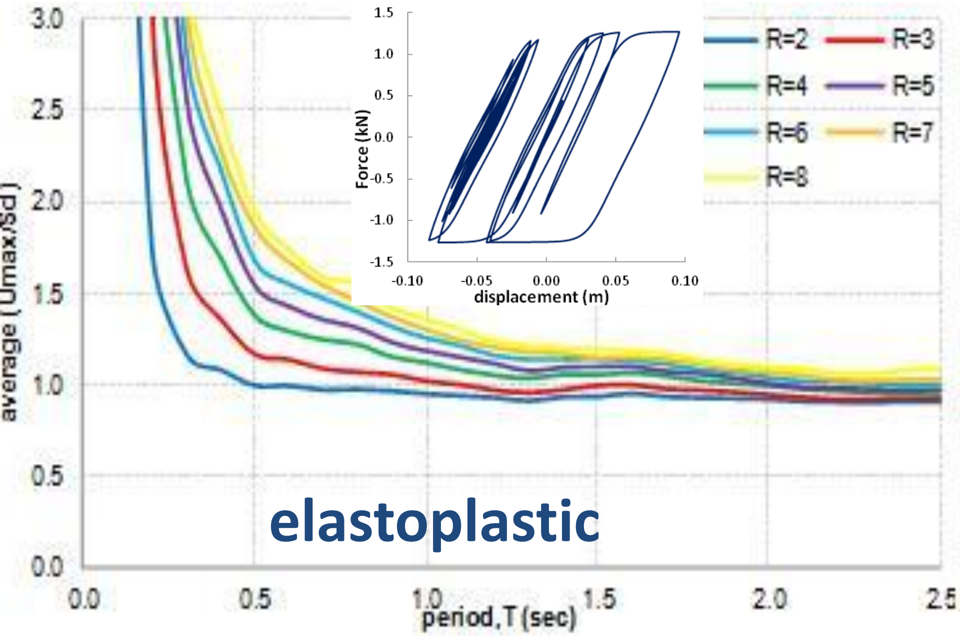


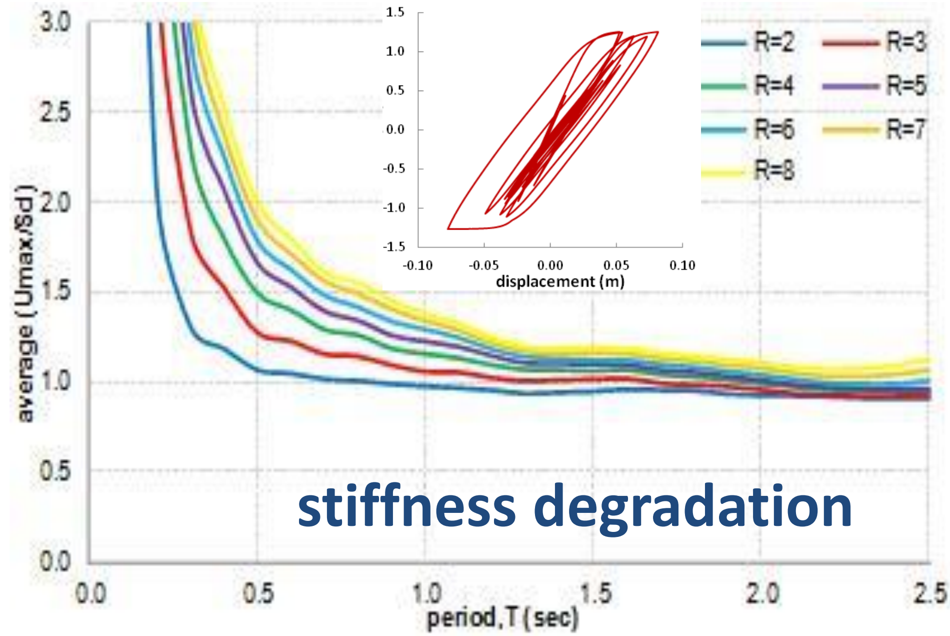
It does not apply

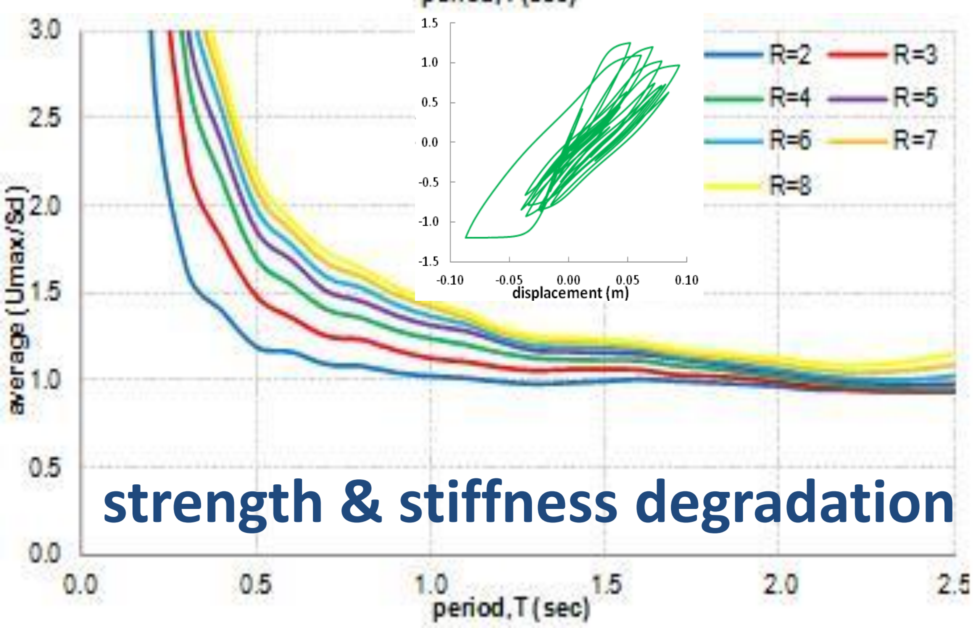


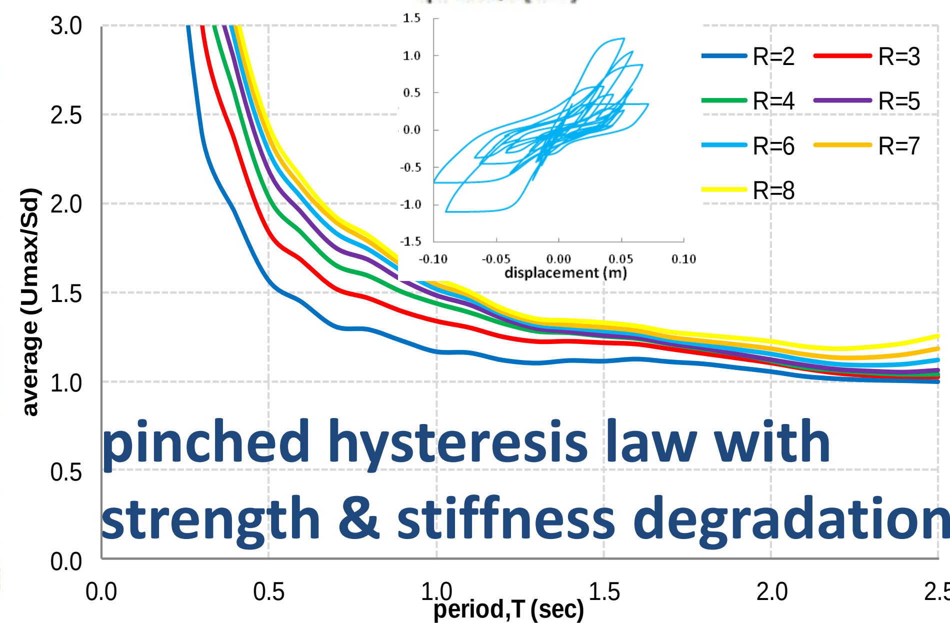
If it applies: earthquake induces the same peak displacement, no matter the lateral force resistance

Peak-inelastic-to-peak-elastic-displacement









Design checks ("Verification format"):

- **Force-based design (FBD):**

Internal force or moment demand < force or moment resistance

- **Displacement-based design (DBD):**

Deformation (eg, chord rotation) demand < Cyclic deformation capacity

Force-based seismic design

Pros:

- Force-based loadings: familiar to designers.
- Solid basis: Equilibrium (if met, we are not too far off)
- Easy to combine analysis results with those due to gravity.
- Lessons from earthquakes: calibration of R-values.

Cons:

- Performance under the design earthquake: ~Unknown.
- No physical basis: Earthquakes don't produce forces on structures; they generate displacements and impart energy. Forces: the off-spring of displacements, not the cause; they sum up to the structure's lateral resistance, no matter the earthquake.
- Lateral forces don't bring down the structure; lateral displacements do, acting with the gravity loads (P-Δ).

Displacement-based design (DBD)

- **Concept:**

  Moehle JP (1992) Displacement-based design of RC structures subjected to earthquakes *Earthq. Spectra* **8**(3): 403-428.
- **Priestley MJN (1993)** Myths and fallacies in earthquake engineering - conflicts between design and reality *T. Paulay Symp.: Recent developments in lateral force transfer in buildings* La Jolla, CA:
  - **“Direct” DBD:** Displacements estimated iteratively with Shibata’s “Substitute Structure”, which has the secant stiffness to the peak response point (a step up in displacement) and the associated damping (a step down). In the end, design is force-based: displacement demands are converted to forces for member proportioning.
- **US approach (FEMA, ASCE):**
  - Displacement demands by “coefficient method”: Elastic estimates times (up to four) coefficients, accounting for special features of the motion or the system.

Displacement-Based Assessment in Eurocode 8 (2005) & Displacement-Based Design in Model Code 2010 of *fib* (Intern. Association Structural Concrete)

- **Displacement measure: Chord rotations at member ends.**
- **Members dimensioned for non-seismic loadings – re-dimensioned/ detailed so that their chord-rotation capacities match seismic demands from (5%-damped elastic) analysis with secant-to-yield-point stiffness**
  - Fardis MN, Panagiotakos TB (1997) Displacement-based design of RC buildings: Proposed approach and application, in "*Seismic Design Methodologies for the Next Generation of Codes*" (P Fajfar, H Krawinkler, eds.), Balkema, 195-206.
  - Panagiotakos TB, Fardis MN (1998) Deformation-controlled seismic design of RC structures, *Proc. 11th European Conf. Earthq. Eng.* Paris.
  - Panagiotakos TB, Fardis MN (1999a) Deformation-controlled earthquake resistant design of RC buildings *J. Earthq. Eng.* **3**: 495-518
  - Panagiotakos TB, Fardis MN (2001) A displacement-based seismic design procedure of RC buildings and comparison with EC8 *Earthq. Eng. Struct. Dyn.* **30**: 1439-1462.
  - Bardakis VG, Fardis MN (2011) A displacement-based seismic design procedure for concrete bridges having deck integral with the piers *Bull. Earthq. Eng*. **9**: 537-560

Displacement-Based Assessment in Eurocode 8 (2005) & Displacement-Based Design in Model Code 2010 of *fib* (Intern. Association of Structural Concrete) (*cont'd*)

- **Member chord-rotation demands from linear-elastic analysis with 5% damping, unmodified by "coefficients"**
  – If applicability conditions are not met: nonlinear static (pushover) or dynamic (response history) analysis.

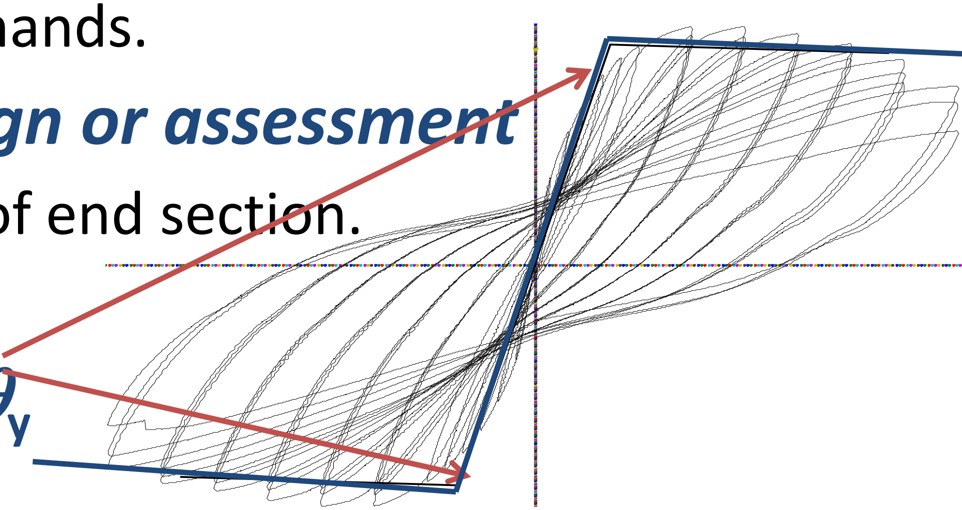
ϑ: rotation with respect to chord connecting two ends at displaced position

- Economou SN, Fardis MN, Harisis A (1993) Linear elastic v nonlinear dynamic seismic response analysis of RC buildings *EURODYN '93,* Trondheim, 63-70
- Panagiotakos TB, Fardis MN (1999) Estimation of inelastic deformation demands in multistory RC buildings *Earthq. Eng. Struct. Dyn.* **28**: 501-528
- Kosmopoulos A, Fardis MN (2007) Estimation of inelastic seismic deformations in asymmetric multistory RC buildings *Earthq. Eng. Struct. Dyn.* **36**: 1209-1234
- Bardakis VG, Fardis MN (2011) Nonlinear dynamic v elastic analysis for seismic deformation demands in concrete bridges having deck integral with the piers. *Bull. Earthq. Eng.* **9**: 519-536

Elastic stiffness:

It controls the natural periods of the elastic structure and the apparent periods of the nonlinear response

- ***For seismic design of new buildings:***
  - *EI*=50% of uncracked section stiffness overestimates by ~2 realistic secant-to-yield-point stiffness;
- overestimates force demands (safe-sided in force-based design);
- underestimates displacement demands.
- ***For displacement-based design or assessment***
  - *EI*= Secant stiffness to yield point of end section.

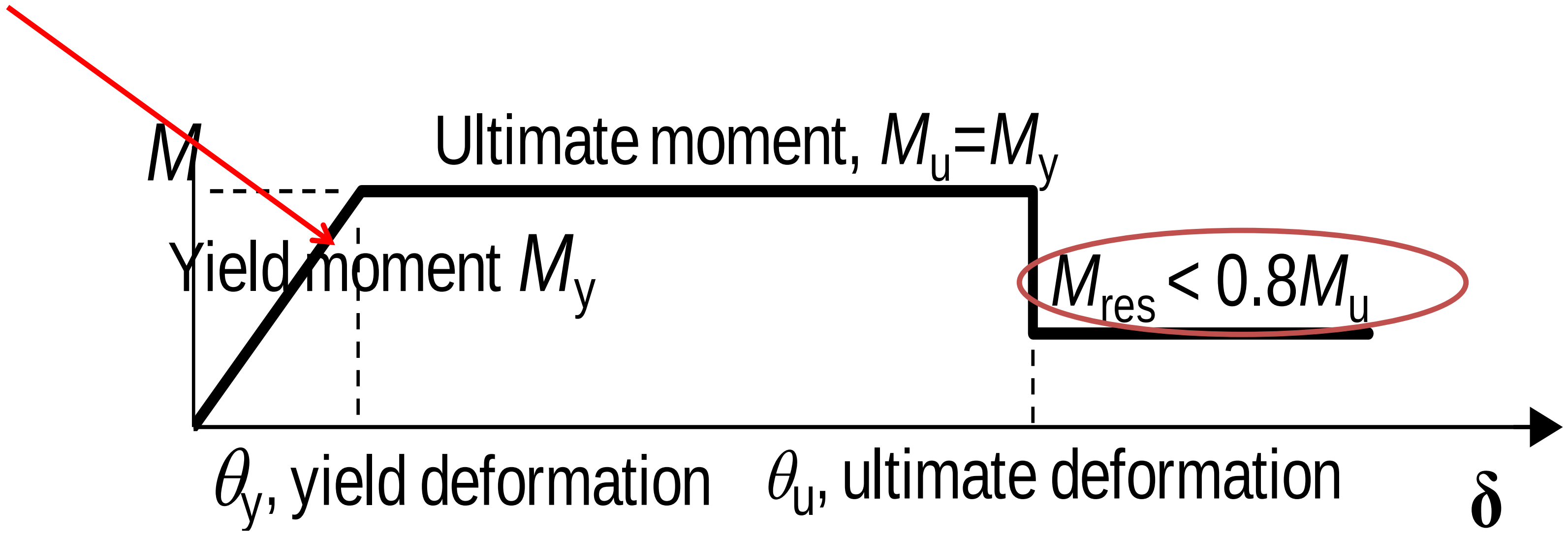


$$EI = M_y L_s / 3\theta_y$$

- Effective stiffness of shear span L_s
- $L_s = M/V$ (~$L_{cl}/2$ in beams/columns, ~$H_w/2$ in cantilever walls),
- M_y, ϑ_y: moment & chord rotation at yielding;
- Average *EI* of two member ends in positive or negative bending.

Parameters of idealized envelope to cyclic moment-deformation behaviour of RC members

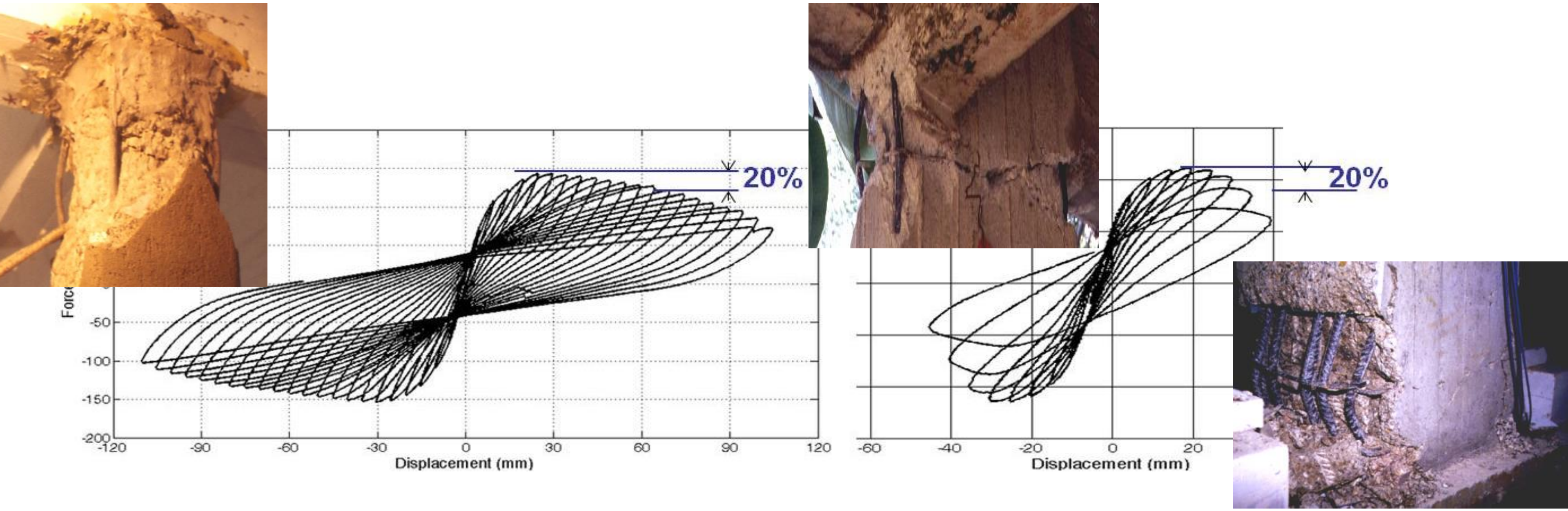
Effective elastic stiffness:
secant-to-yield-point:



member deformation: chord-rotation
section deformation: curvature

“Ultimate” member deformation

- Local failure of material (even loss of a bar) ≠ member failure.
- A plastic hinge is taken to fail by accumulation of local failures during cycling of deformations, until it loses ~20% of its moment resistance.



- Deformation measures used in the verifications should reflect the behaviour of the plastic hinge as a whole.
- Appropriate measure for the plastic hinge:

plastic part of chord rotation at a member end, ϑ_{pl}

(including post-yield part of fixed-end-rotation, ϑ_{slip}, due to slippage of longitudinal bars from their anchorage beyond the member end).

Displacement-Based Assessment in Eurocode 8 (2005) & Displacement-Based Design in Model Code 2010 of *fib* (Intern. Association of Structural Concrete) (*cont'd*)

- **Member chord-rotation capacity** (from member geometry & materials
  - ❖ **at yielding** (to limit damage & allow immediate re-use);
  - ❖ **at "ultimate"** conditions, conventionally identified with >20% drop in moment resistance (to prevent serious damage & casualties).
  - – Panagiotakos TB, Fardis MN (2001) Deformations of RC members at yielding and ultimate. *ACI Struct. J.* **98**(2): 135-148.
  - – Biskinis D, Fardis MN (2007) Effect of lap splices on flexural resistance and cyclic deformation capacity of RC members *Beton- Stahlbetonbau, Sond. Englisch* **102**
  - – Biskinis D, Fardis MN (2010a) Deformations at flexural yielding of members with continuous or lap-spliced bars. *Struct. Concr.* **11**(3): 127-138.
  - – Biskinis D, Fardis MN (2010b) Flexure-controlled ultimate deformations of members with continuous or lap-spliced bars. *Struct. Concr.* **11**(2): 93-108.
- **Member cyclic shear resistance after flexural yielding**.
  - – Biskinis D, Roupakias GK, Fardis MN (2004) Degradation of shear strength of RC members with inelastic cyclic displacements *ACI Struct. J.* **101**(6): 773-783

Next generation of codes (and models): Displacement-Based Design, Assessment or Retrofitting in Eurocode 8 (2020) & Model Code 2020 of *fib*

- **Member chord-rotation at yielding and at "ultimate" conditions**
  - Grammatikou S, Biskinis D, Fardis MN (2016) Ultimate strain criteria for RC members in monotonic or cyclic flexure *ASCE J. Struct. Eng.* **142**(9)
  - Grammatikou S, Biskinis D, Fardis MN (2018) Effect of load cycling, FRP jackets and lap-splicing of longitudinal bars on the effective stiffness and the ultimate deformation of flexure-controlled RC members *ASCE J Struct Eng* **144**(6) 04017195
  - Grammatikou S, Biskinis D, Fardis MN (2018) Flexural rotation capacity models fitted to test results using different statistical approaches *Struct. Concrete* **19**(2) 608-624
  - Grammatikou S, Biskinis D, Fardis MN (2018) Models for the flexure-controlled strength, stiffness and cyclic deformation capacity of concrete columns with smooth bars, including lap-splicing and FRP jackets *Bull. Earthq. Eng*. **16**(1)
- **Cyclic shear resistance of walls after flexural yielding.**
  - Grammatikou S, Biskinis D, Fardis MN (2015) Strength, deformation capacity and failure modes of RC walls under cyclic loading *Bull. Earthq. Eng.* **13**: 3277-3300

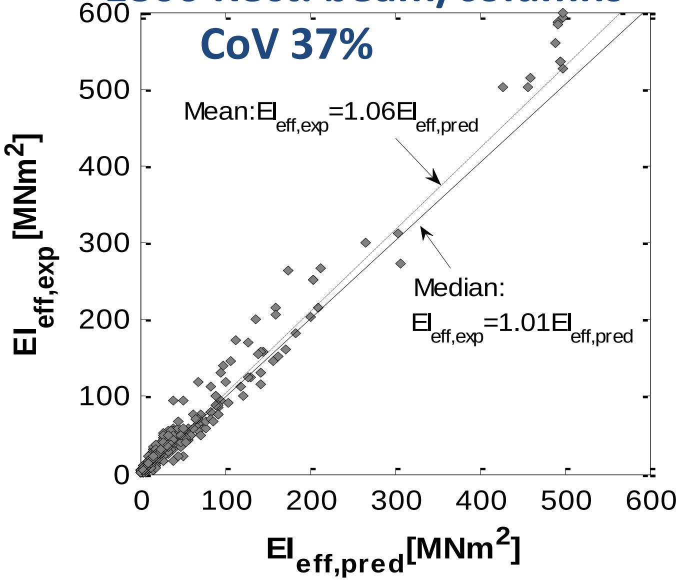
New models, from database of ~4200 tests

- Seamless portfolio of **physical** models for the stiffness and the flexure-controlled cyclic deformation capacity of RC members:
  - "conforming" to design codes or not, with continuous or lap-spliced **ribbed** (deformed) bars, with or without fiber-reinforced polymer (FRP) wraps;
  - "non-conforming", with continuous or lap-spliced **smooth** (plain) bars (with hooked or straight ends), with or without FRP wraps.
- Portfolio of **empirical** models for the flexure-controlled cyclic deformation capacity of the above types of members, but only for sections consisting of one or more rectangular parts.
- Models for the cyclic shear resistance as controlled by:
  - Yielding of transverse reinforcement in a flexural plastic hinge;
  - Web diagonal compression in walls or short columns;
  - Yielding of longitudinal & transverse web reinforcement in squat walls
  - Shear sliding at the base of walls after flexural yielding;
- Models for the unloading stiffness and – through it – energy dissipation in cyclic loading.

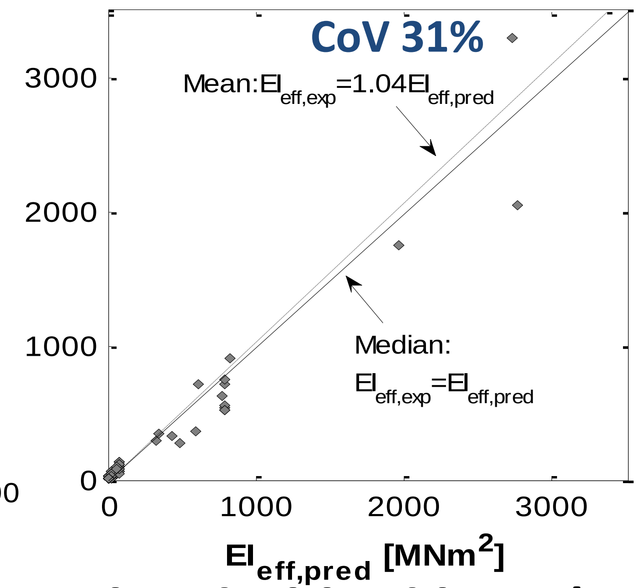
Test-vs-prediction & their ratio - secant-to-yield-point stiffness

Members with continuous ribbed bars (~2700 tests)

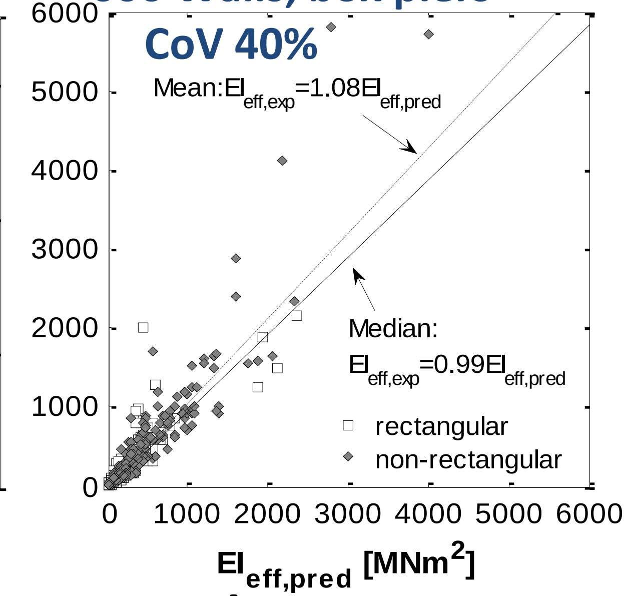
~1800 Rect. beam/columns



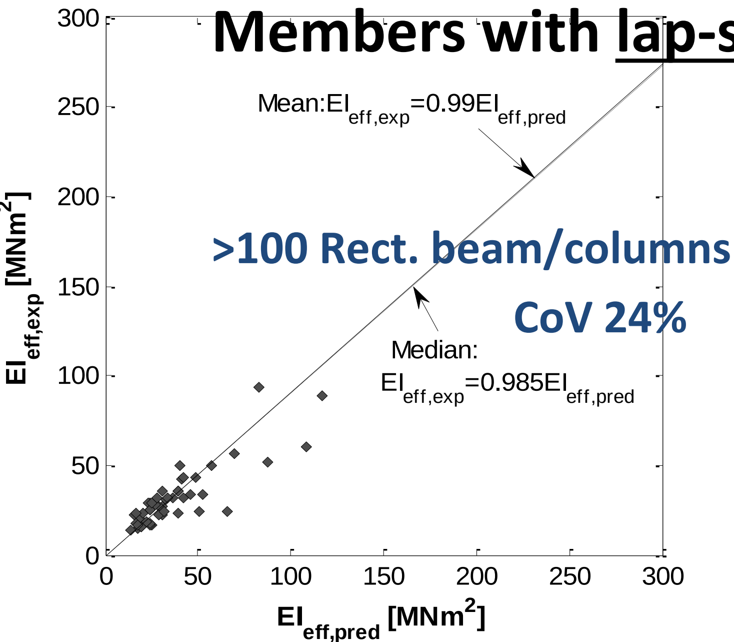
~300 Circular Columns

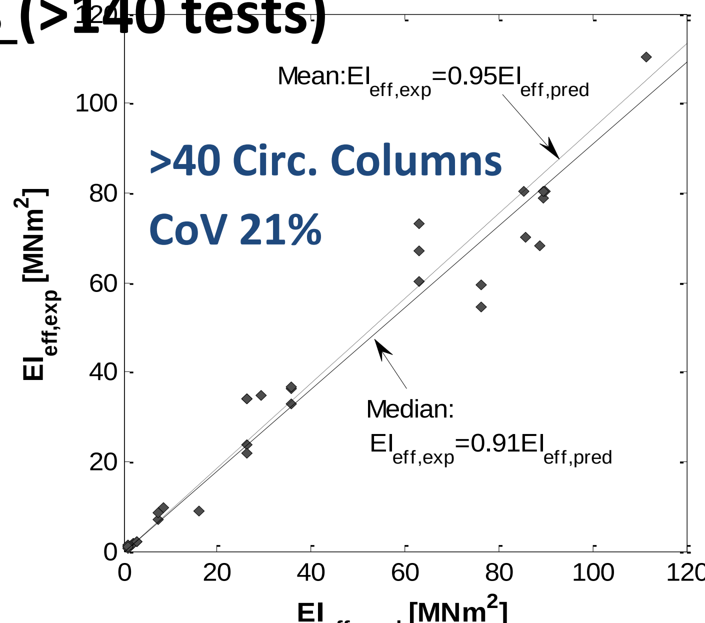


~600 Walls, box piers



Members with lap-spliced ribbed bars (>140 tests)

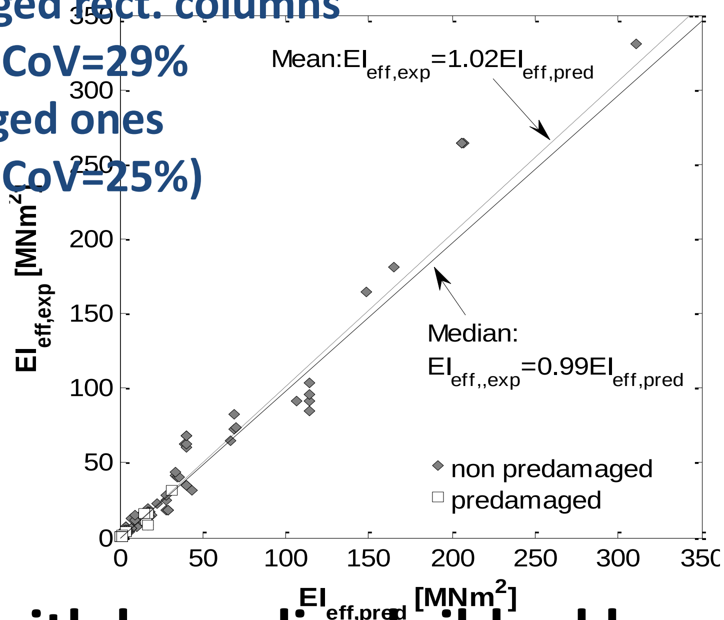




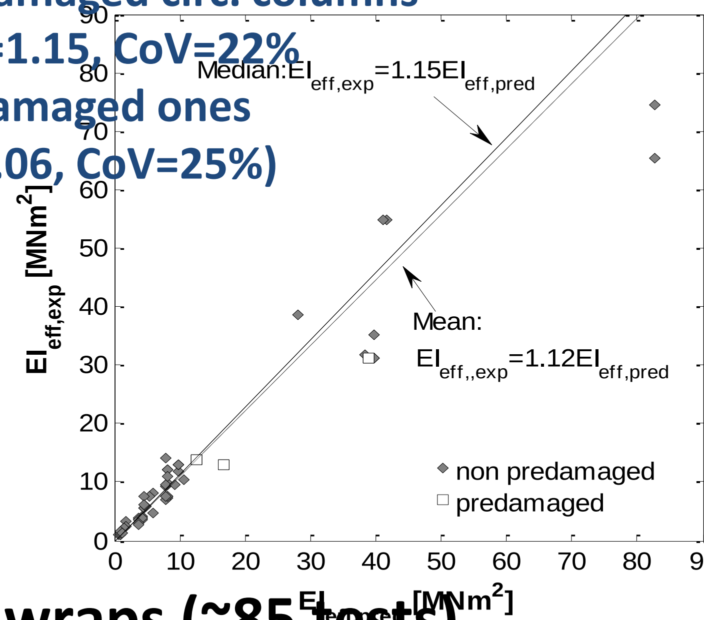
Test-vs-prediction & their ratio - secant-to-yield-point stiffness *(cont'd)*

Members with continuous ribbed bars & FRP wraps (~240 tests)

~160 undamaged rect. columns
median=0.99, CoV=29%
(22 pre-damaged ones
median=0.68, CoV=25%)

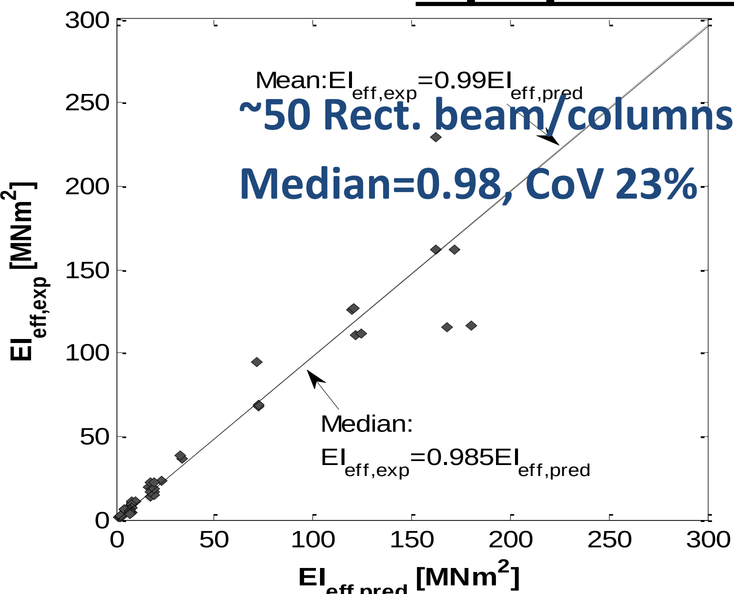


~50 undamaged circ. columns
median=1.15, CoV=22%
(5 pre-damaged ones
mean=1.06, CoV=25%)

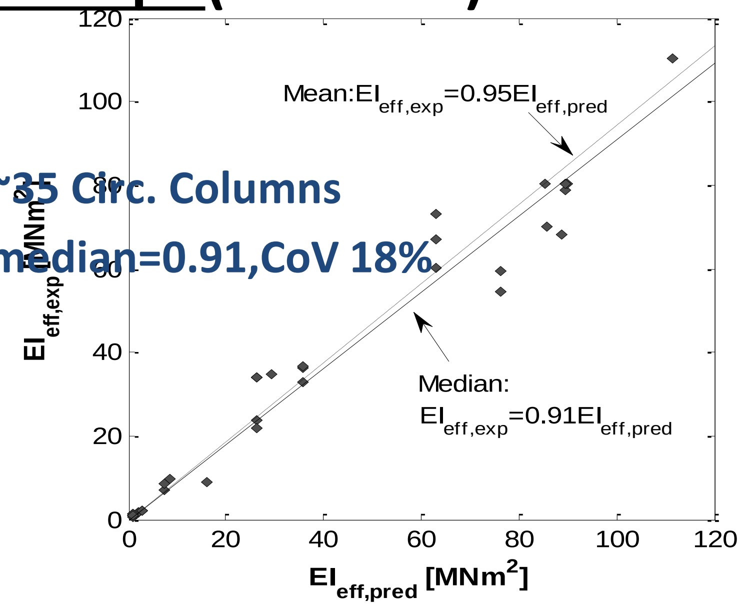


Members with lap-spliced ribbed bars & FRP wraps (~85 tests)

~50 Rect. beam/columns
Median=0.98, CoV 23%

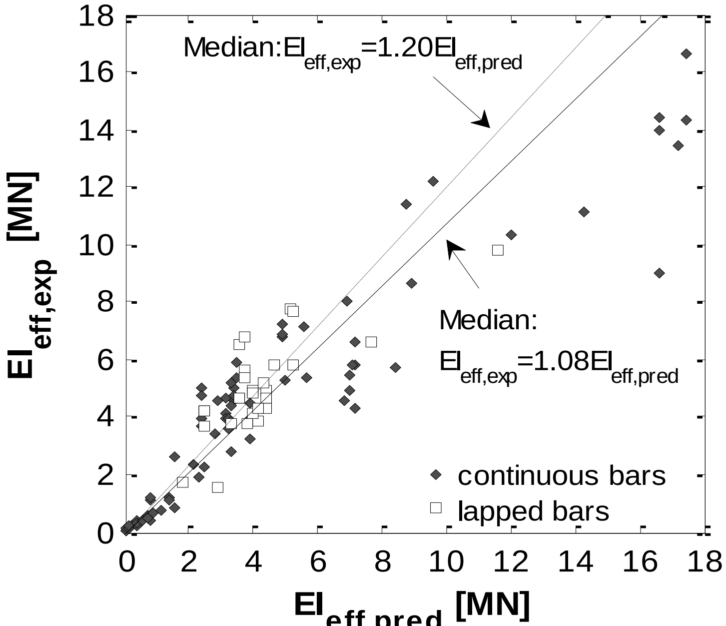


~35 Circ. Columns
median=0.91,CoV 18%



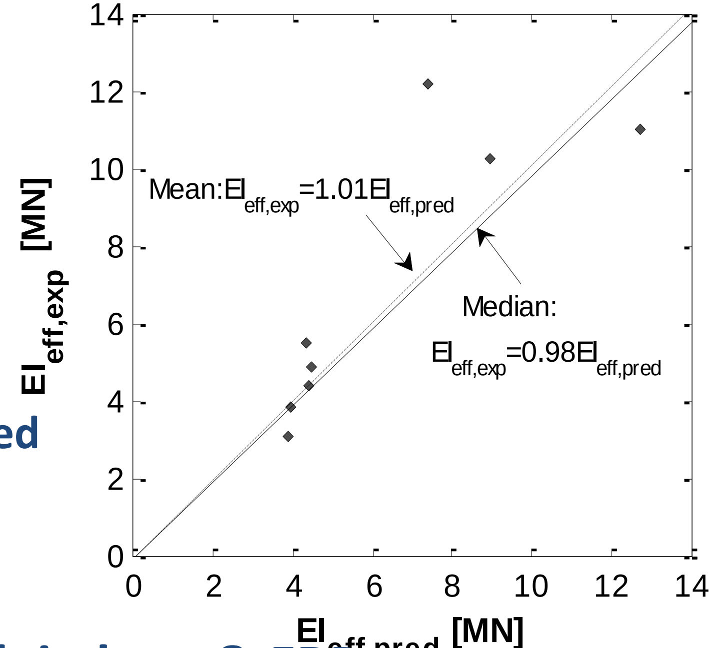
Test-vs-prediction & their ratio - secant-to-yield-point stiffness *(cont'd)*

Rect. columns with continuous or lap-spliced smooth bars (~125 tests)

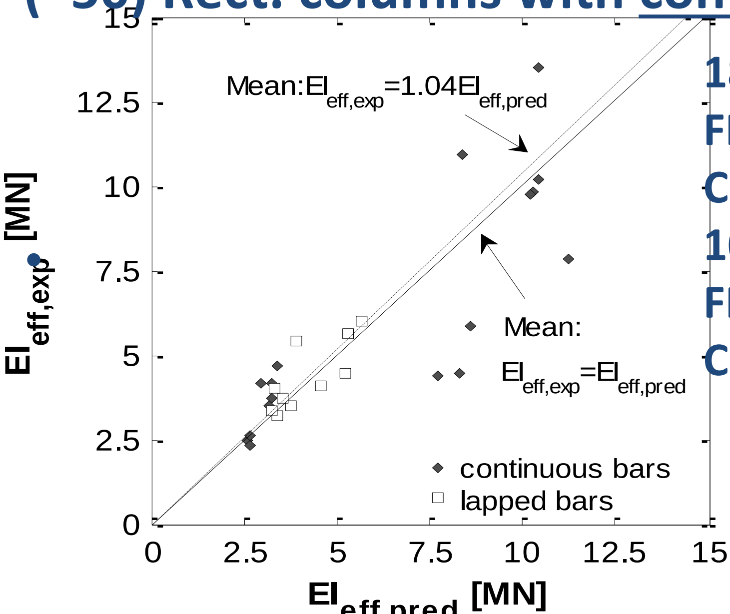


~85 cantilevers w/ cont. bars CoV 36%
~30 cantilevers w/ hooked laps CoV 25%

10 doubly fixed w/ cont. bars CoV 29%

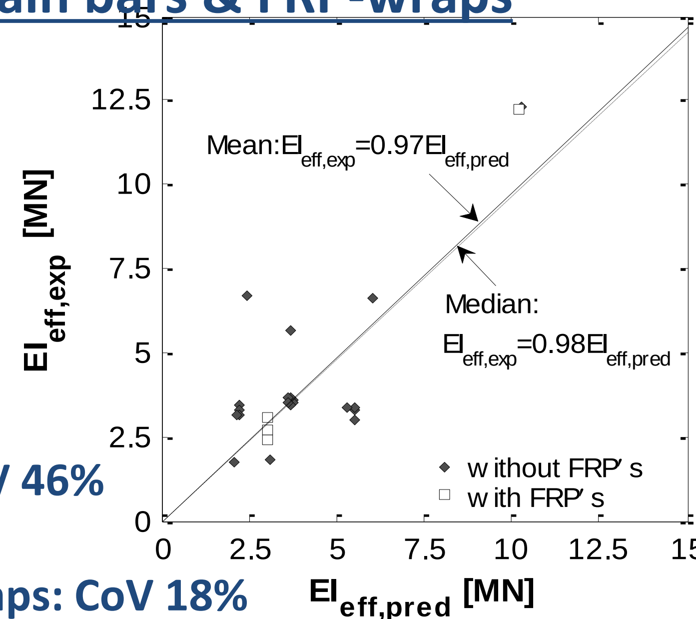


(~50) Rect. columns with continuous or lapped plain bars & FRP-wraps



18 cantilevers w/ FRP & cont. bars CoV 26%
10 cantilevers w/ FRPs & hooked laps CoV 16%

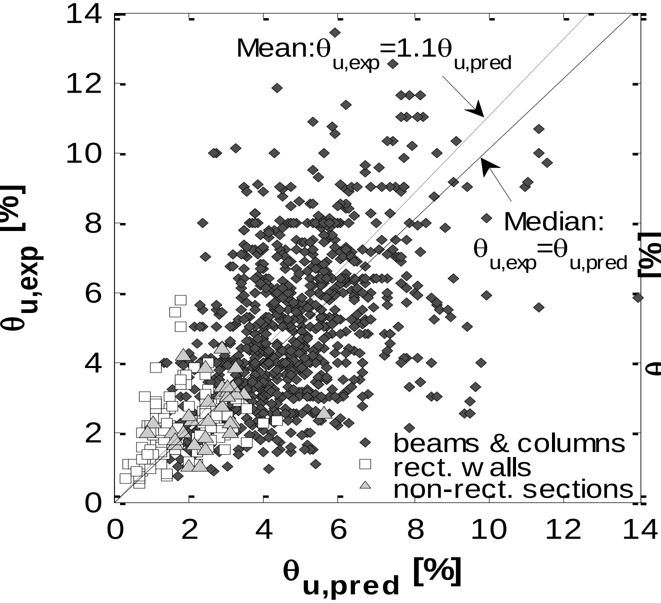
20 cantilevers straight laps CoV 46%
4 cantilevers w/ FRP & straight laps: CoV 18%



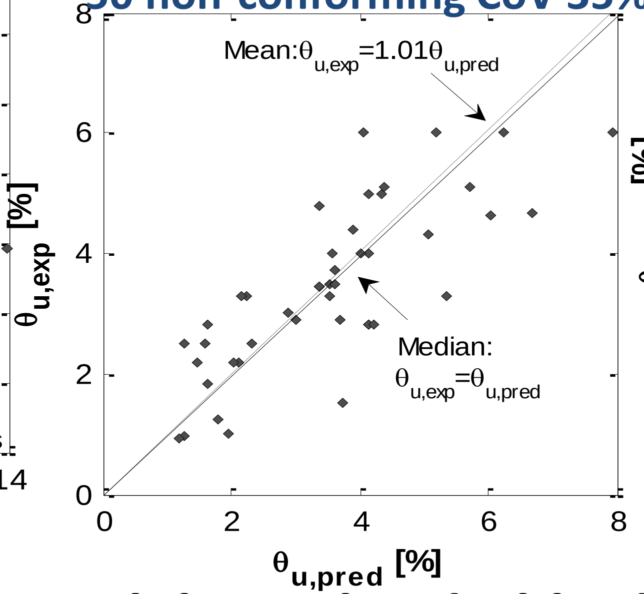
Test-vs-predicted ultimate chord-rotation & their ratio – physical model

Members with continuous ribbed bars

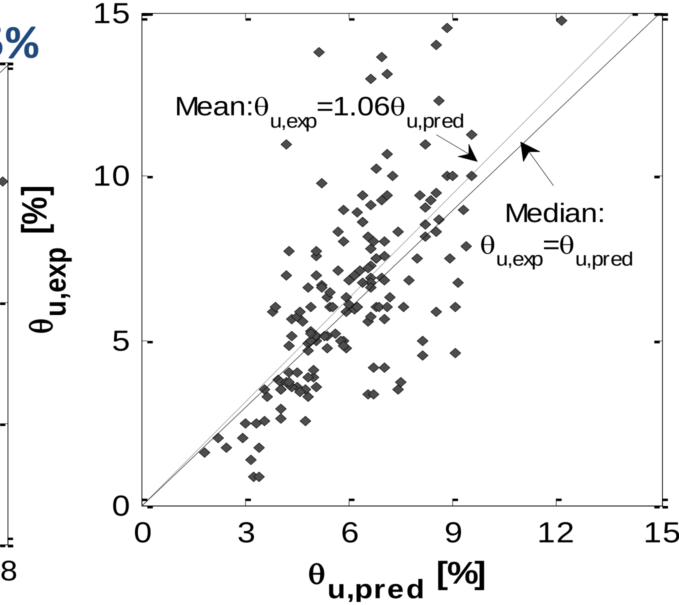
~1200 conforming, non-circular CoV 45%



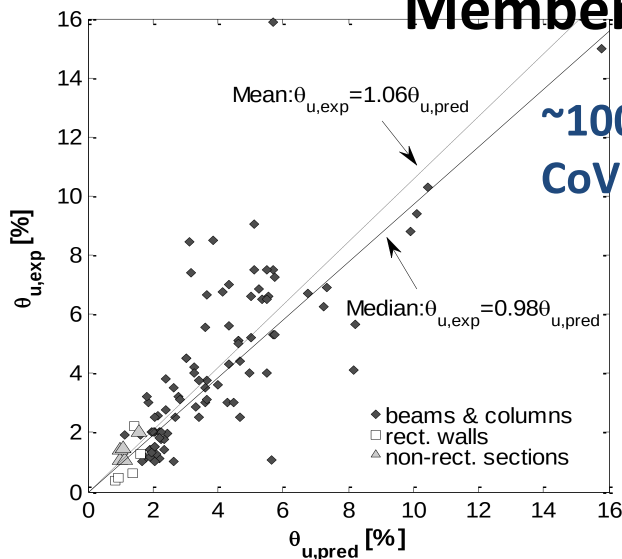
~50 non-conforming CoV 35%



~150 Circ. columns CoV 35%

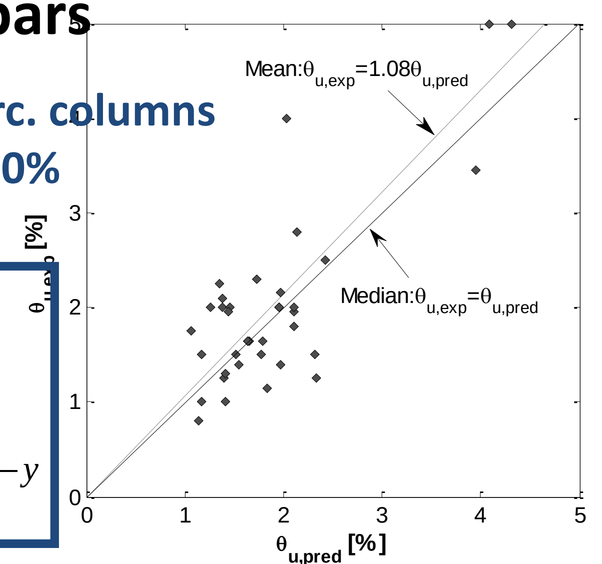


Members with lap-spliced ribbed bars



~100 rect. columns CoV=42%

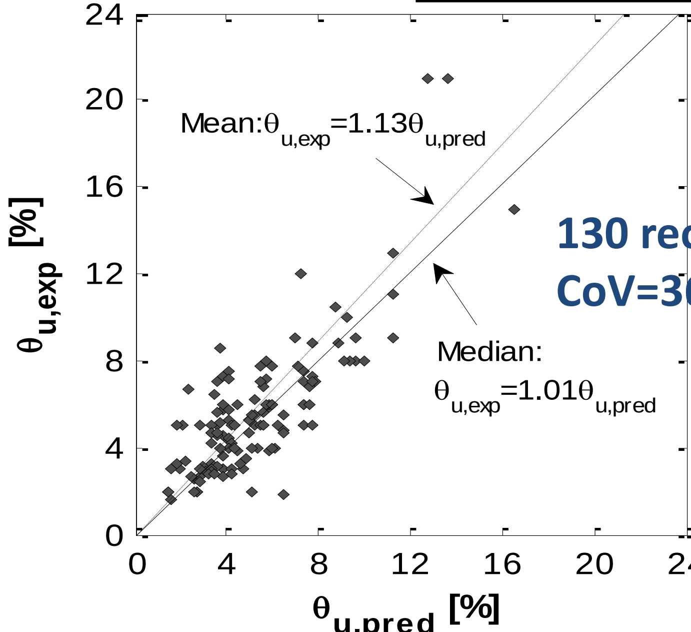
~50 circ. columns CoV=30%



$$\theta_u^{pl} = (\varphi_u - \varphi_y)L_{pl}\left(1-\frac{L_{pl}}{2L_s}\right)+\Delta\theta_{slip,u-y}$$

Test-vs-predicted ultimate chord-rotation & their ratio – physical model *(cont'd)*

Members with continuous ribbed bars and FRP wrapping

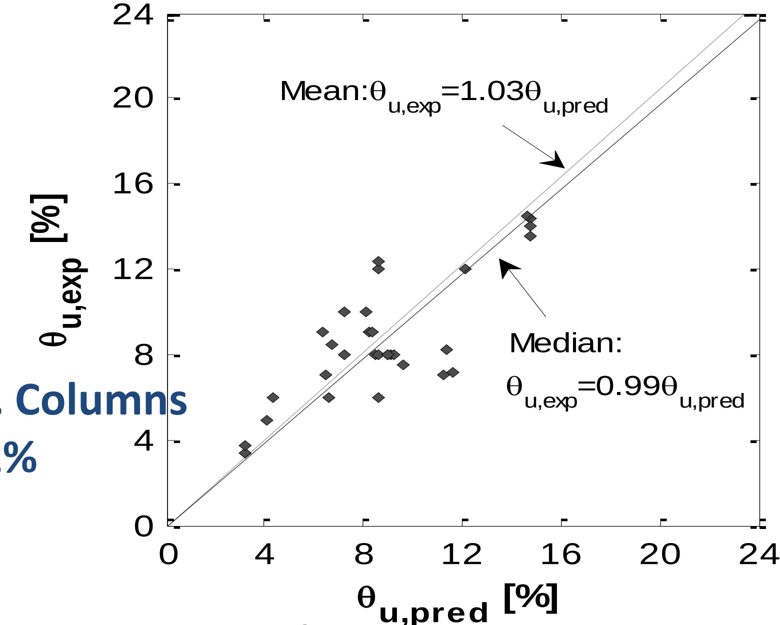


130 rect. Columns

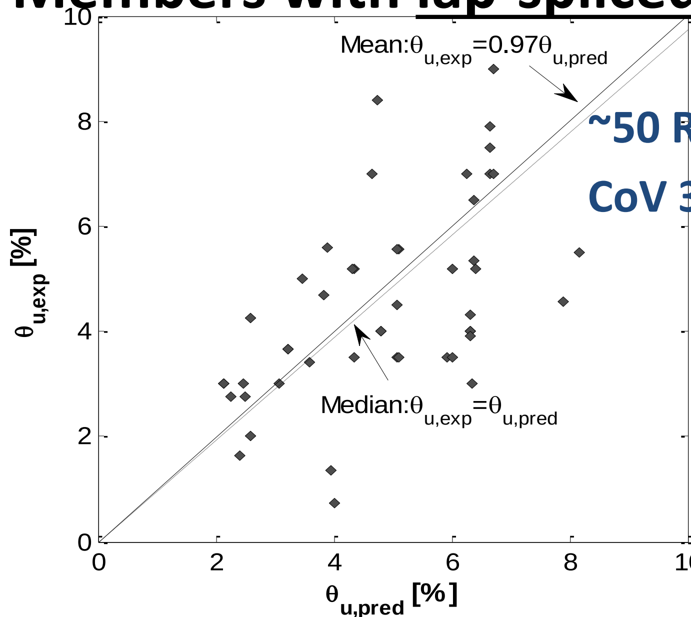
CoV=36%

~30 circ. Columns

CoV=22%



Members with lap-spliced ribbed bars and FRP-wrapping

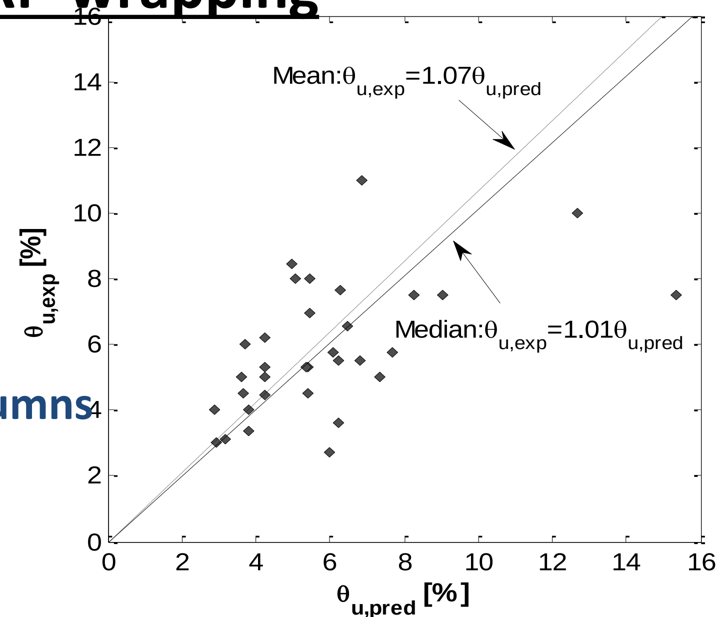


~50 Rect. beam/columns

CoV 35%

35 circ. columns

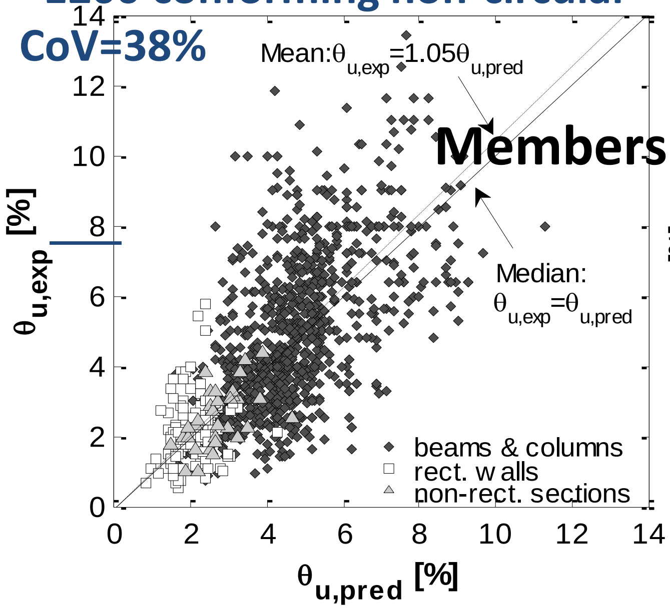
CoV 31%



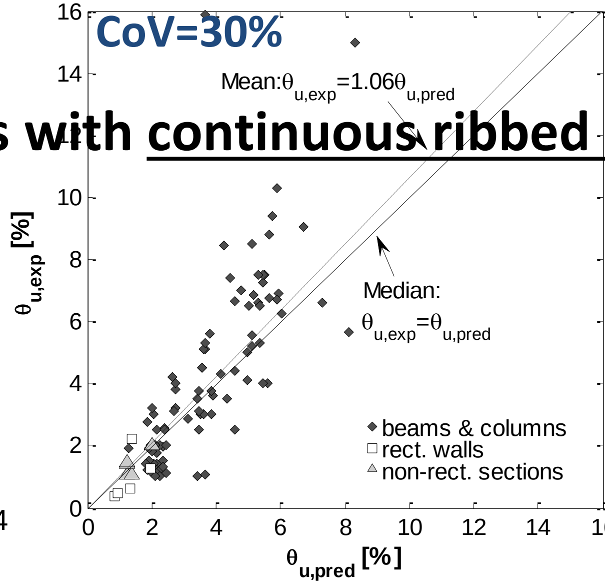
Test-vs-predicted ultimate chord-rotation & their ratio– empirical model

Members with continuous ribbed bars

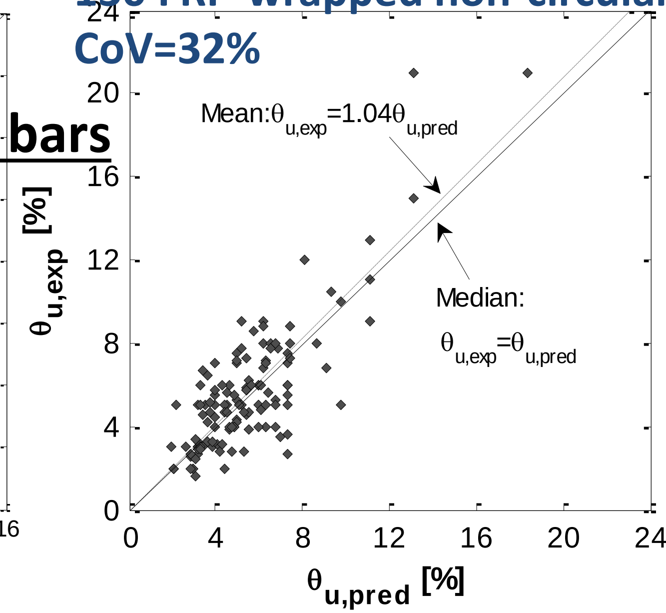
~1200 conforming non-circular
CoV=38%



~50 non-conforming
CoV=30%

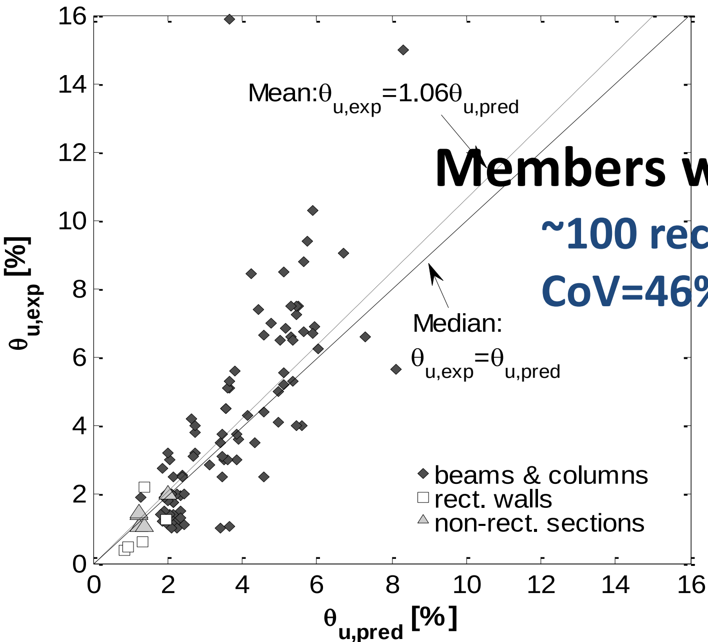


130 FRP-wrapped non-circular
CoV=32%

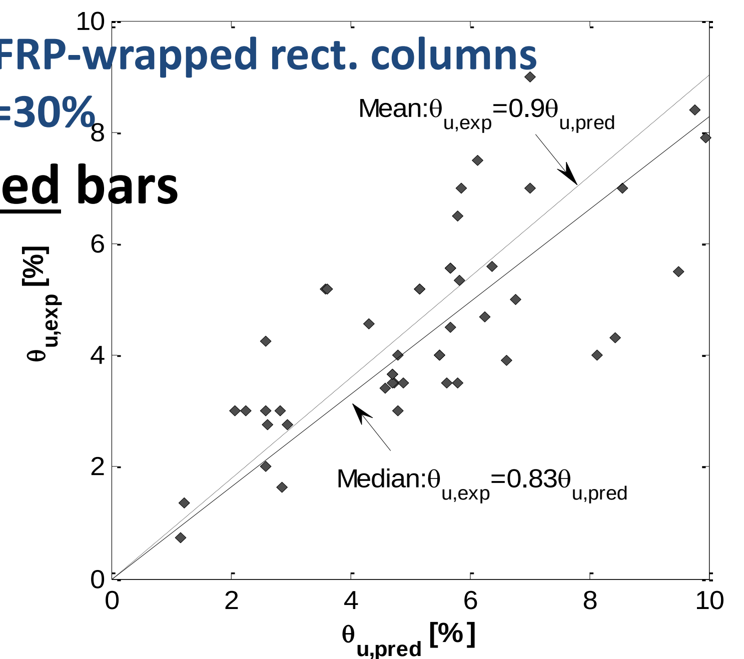


Members with lap-spliced ribbed bars

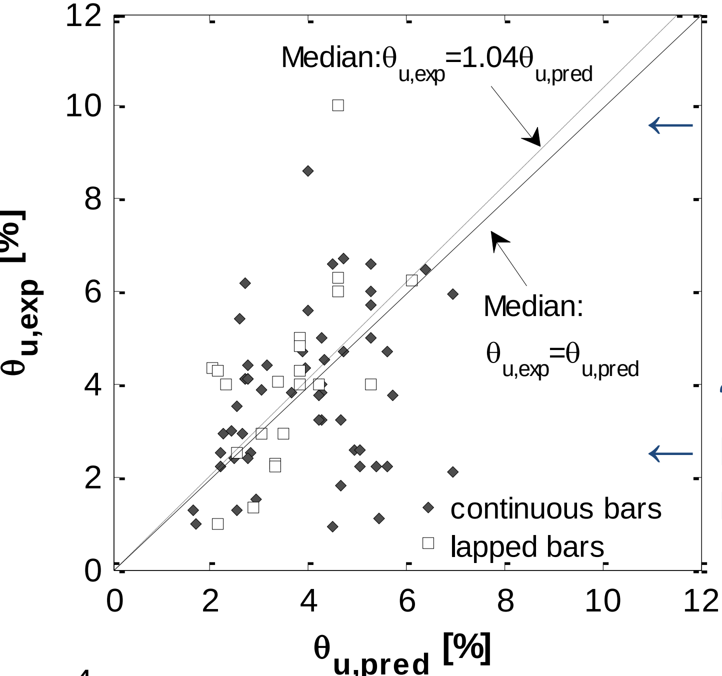
~100 rect. columns
CoV=46%



~50 FRP-wrapped rect. columns
CoV=30%



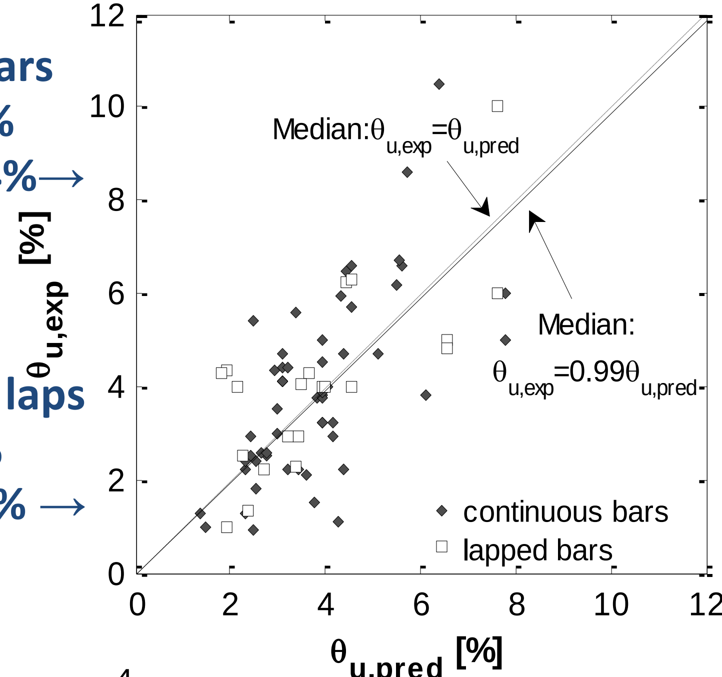
Test-vs-predicted ultimate chord-rotation of members with smooth bars & their ratio – physical v empirical model



~55 cantilevers, cont. bars

← **Physical model CoV 46%**

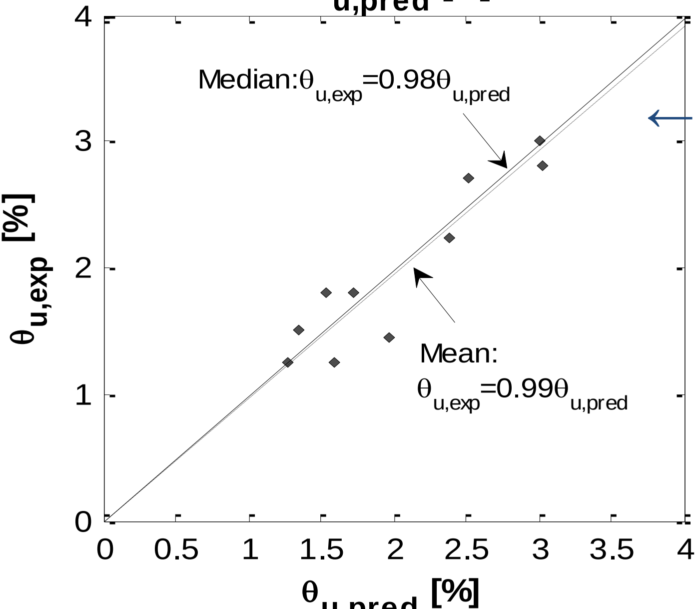
Empirical model CoV 34% →



~20 cantilevers, hooked laps

← **Physical model CoV 43%**

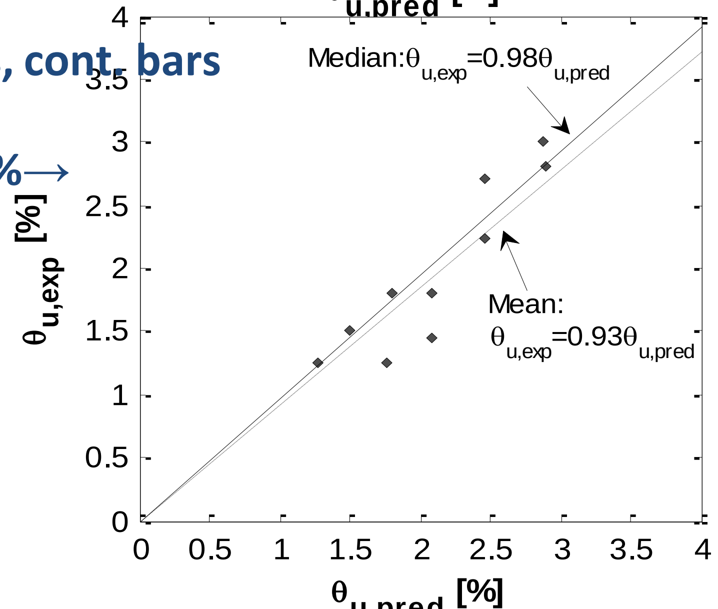
Empirical model CoV 45% →



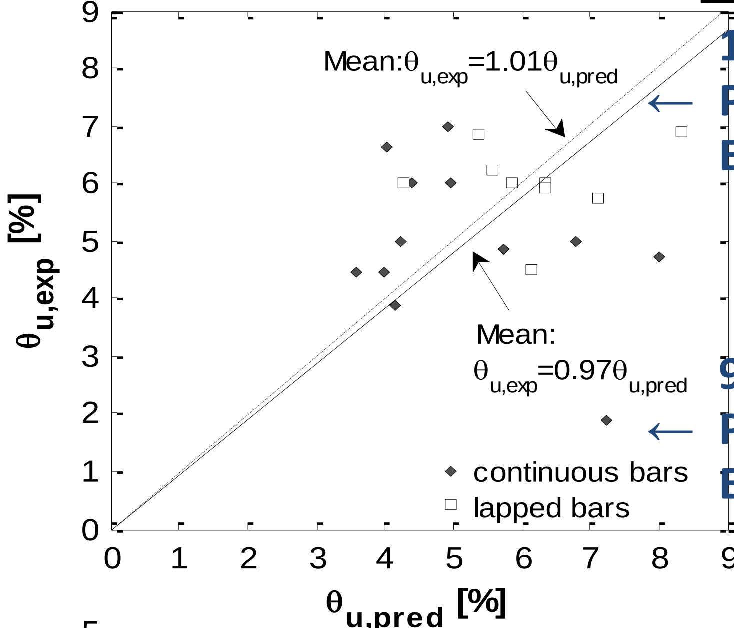
10 doubly fixed columns, cont. bars

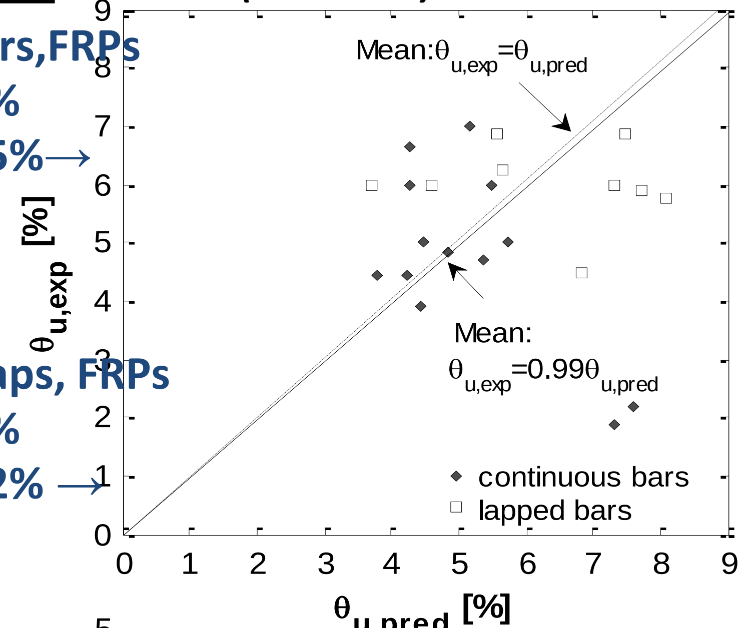
← **Physical model CoV 14%**

Empirical model CoV 14% →



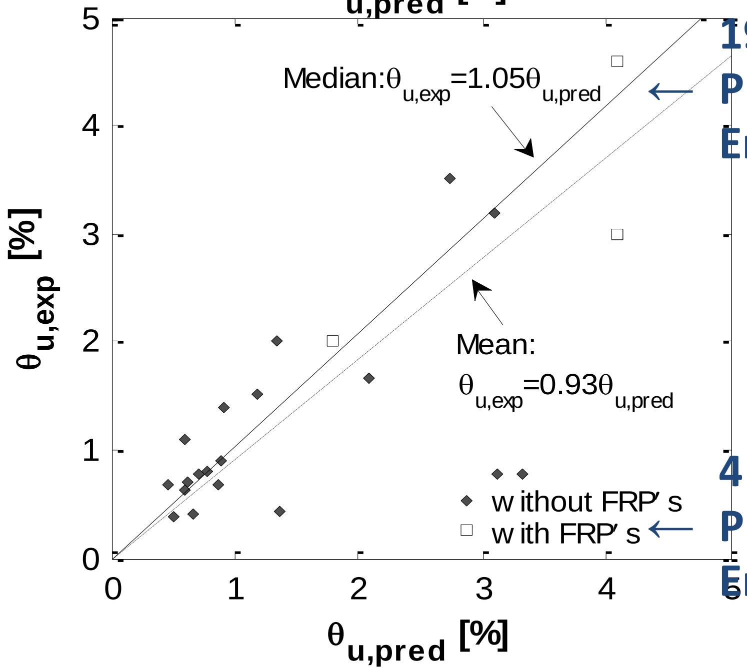
Test-vs-predicted ultimate chord-rotation of members with smooth bars & their ratio – physical v empirical model *(cont'd)*

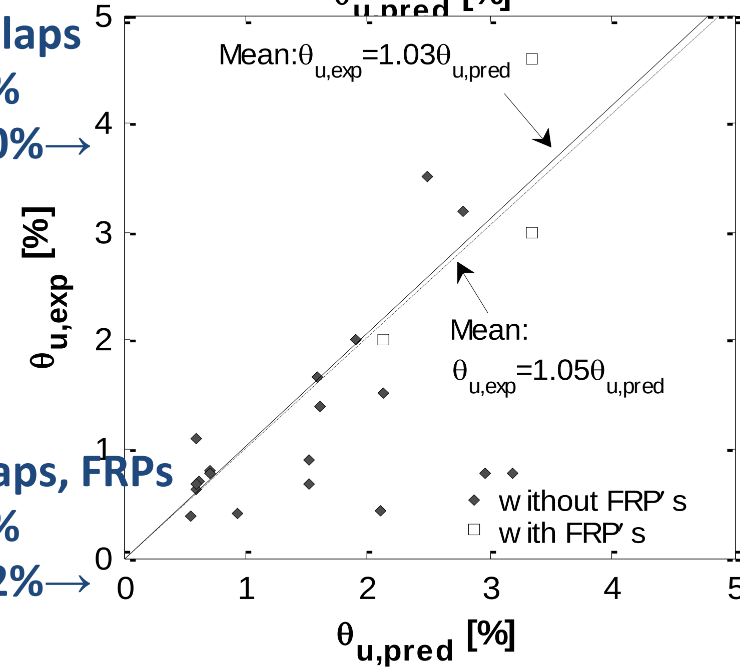




14 cantilevers, cont. bars,FRPs
← Physical model CoV 45%
Empirical model CoV 35% →

9 cantilevers, hooked laps, FRPs
← Physical model CoV 22%
Empirical model CoV 32% →





19 cantilevers, straight laps
← Physical model CoV 45%
Empirical model CoV 50% →

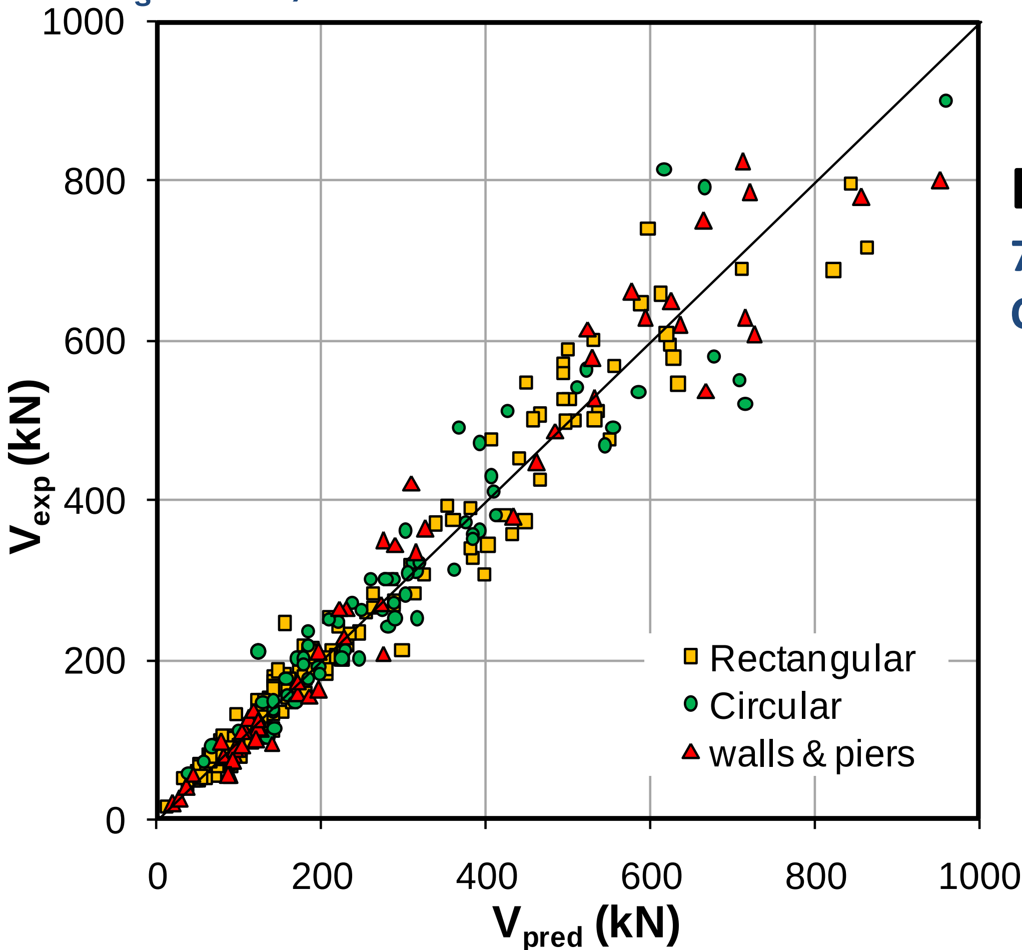
4 cantilevers, straight laps, FRPs
← Physical model CoV 24%
Empirical model CoV 22% →

Cyclic shear resistance (after yielding)

Test-vs-prediction & their ratio

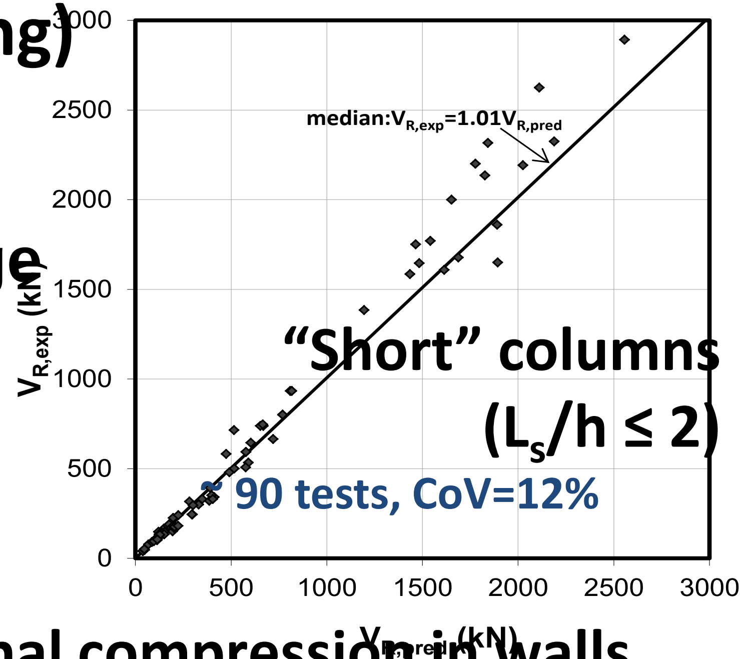
Diagonal tension failure of plastic hinge

~205 rect. beams/columns, ~75 circ. columns ~40 rect. & ~55 non-rect. walls or box sections, all with $4.1 \geq L_s/h > 1.0$, CoV=17%



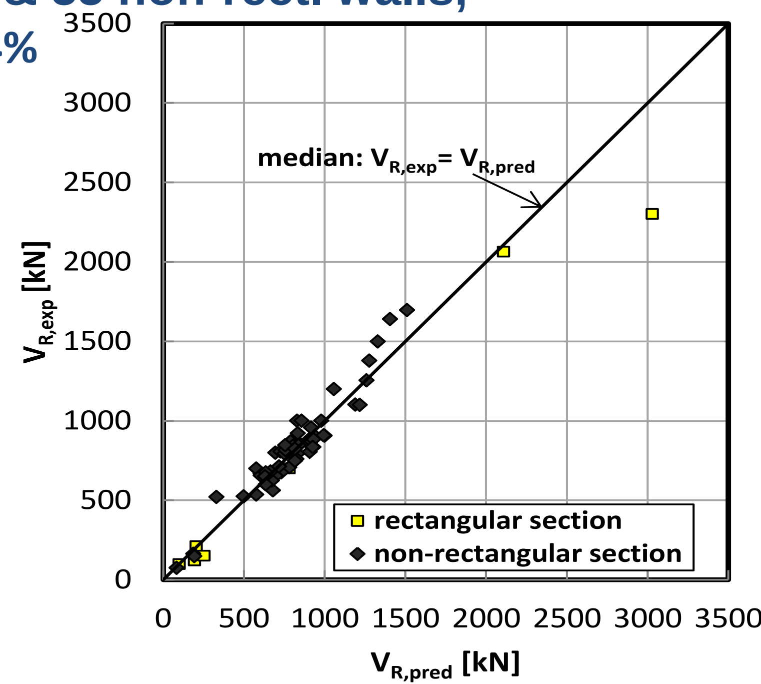
"Short" columns ($L_s/h \leq 2$)

~90 tests, CoV=12%



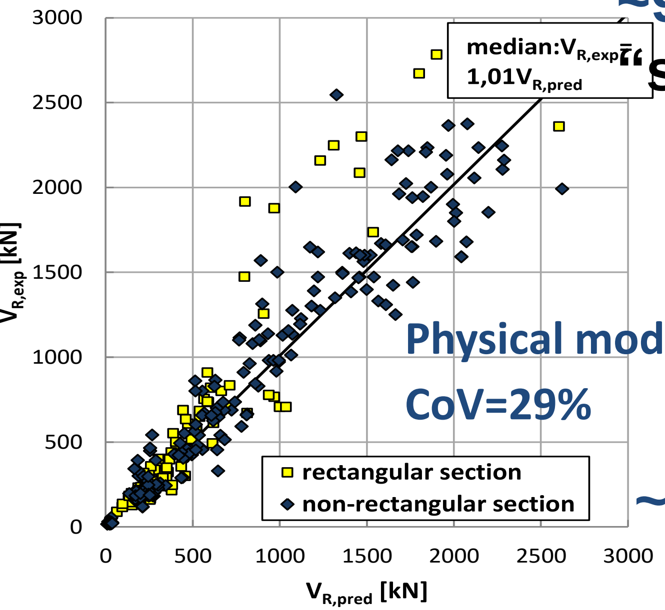
Diagonal compression in walls

7 rect. & 55 non-rect. walls, CoV=14%

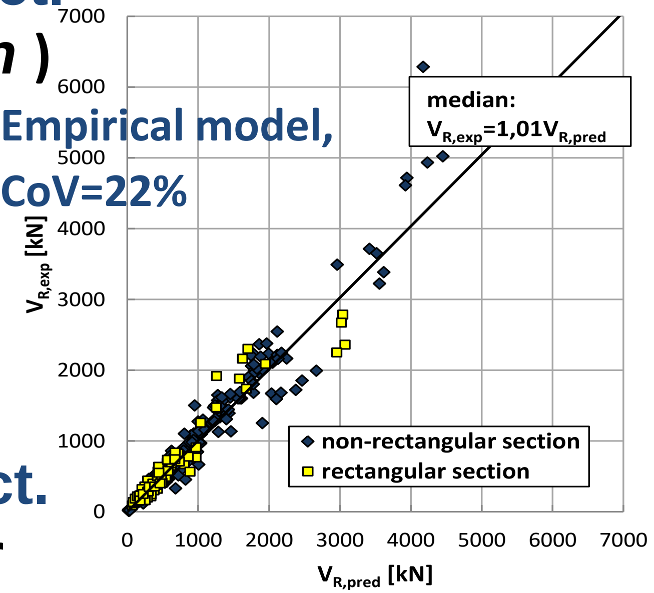


Cyclic shear resistance after flexural yielding (*cont'd*)

~95 rect. & ~235 non-rect. "squat" walls (1.2 ≥ L_s/h)



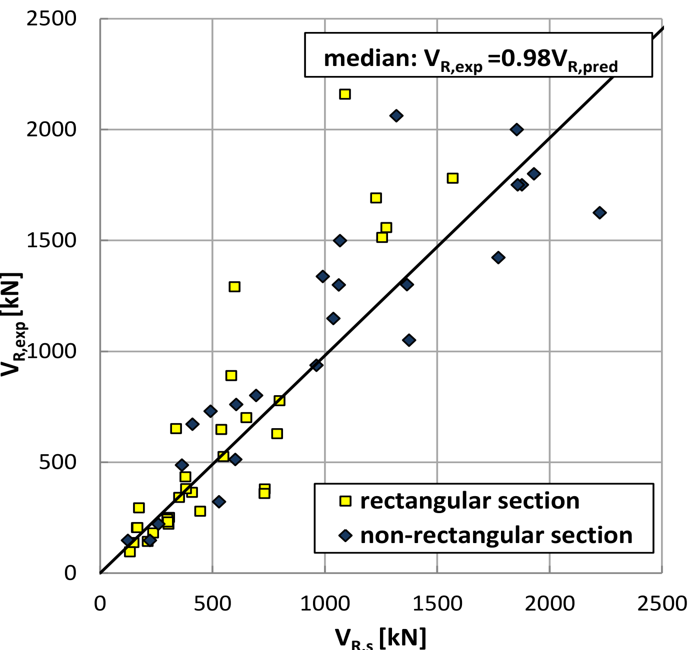
Physical model, CoV=29%



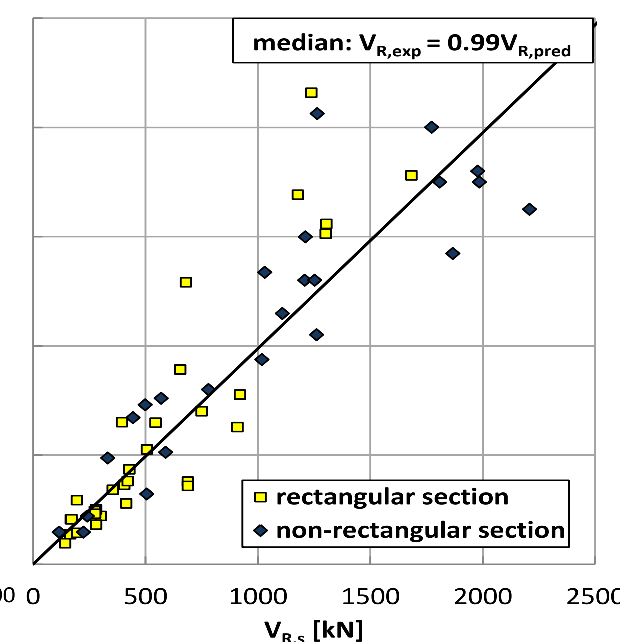
Empirical model, CoV=22%

~30 rect. & ~25 non-rect. walls in sliding shear

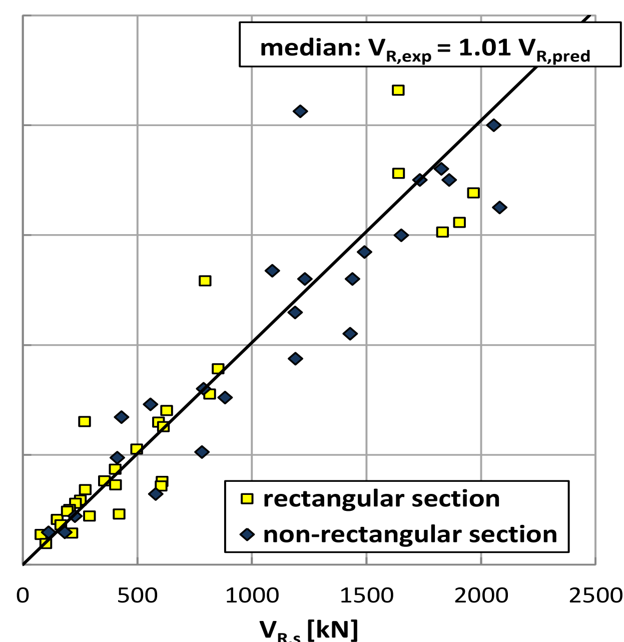
fib MC2010-based, CoV=34%



ACI318-based, CoV=32%

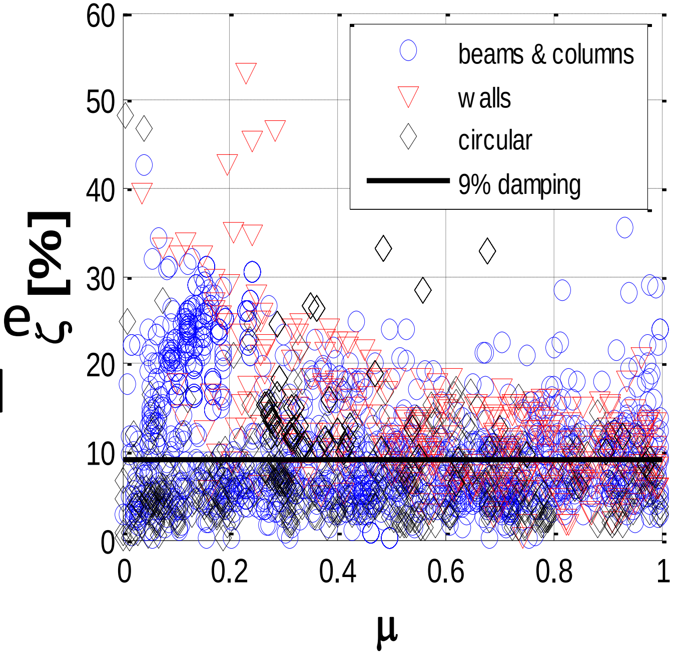


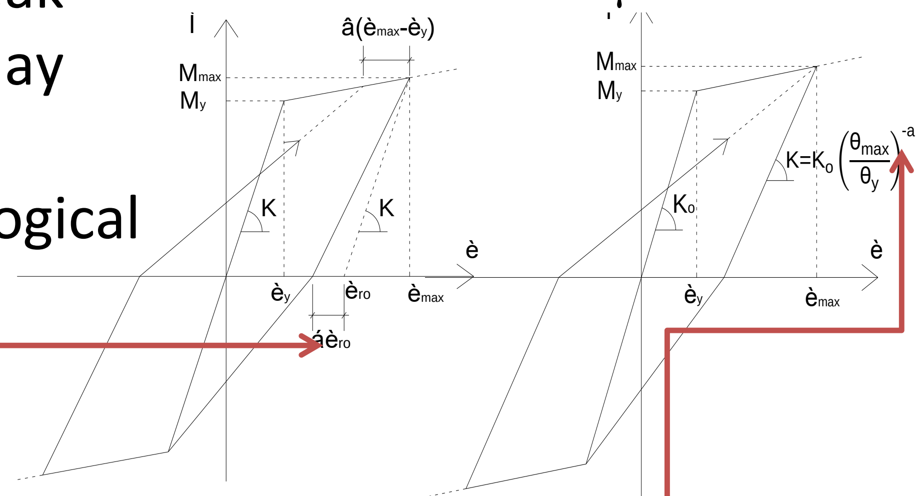
Eurocode 8-based, CoV=31%



Energy dissipation

- If the behaviour is taken as linear-elastic till yielding (so that the natural periods are controlled by the secant-to-yield-point stiffness) viscous damping in the elastic range should reflect the energy dissipation in small pre-yield cycles.
- The amount of hysteretic energy dissipated in post-yield cycles to peak displacement ductility demand μ may be reproduced by choosing the unloading stiffness of phenomenological hysteresis laws.





Modified Takeda model: *α* parameter for *β=0*

If viscous ζ=9%: $\alpha = 0.63 - 0.017 L_s / h + 0.12 N / A_c f_c$

If viscous ζ=5%: $\alpha = 0.47 - 0.01 L_s / h + 0.14 N / A_c f_c$

If viscous ζ=0%: $\alpha = 0.3 - 0.003 L_s / h + 0.12 N / A_c f_c$

Modified Clough model: *a* parameter

with ζ=9%: $a = 0.70 - 0.013 L_s / h + 0.07 N / A_c f_c$

with ζ=5%: $a = 0.56 - 0.0075 L_s / h + 0.1 N / A_c f_c$

with ζ=0%: $a = 0.38 - 0.0025 L_s / h + 0.1 N / A_c f_c$

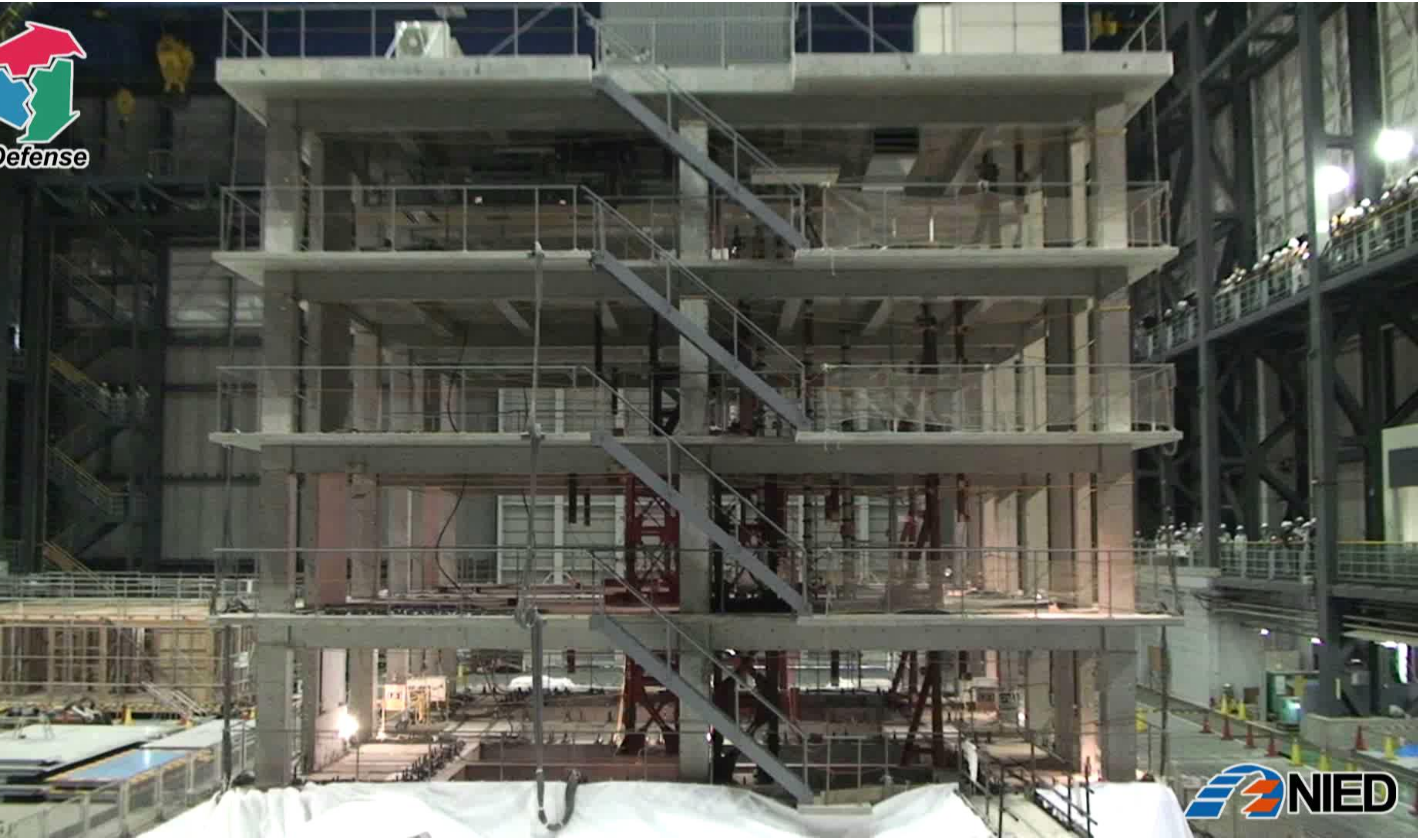
Application:
E-Defence shake-table test

Full-scale 4-storey reinforced concrete building:

- moment frames in one direction;
- shear walls coupled with frames in the other.

2-directional shake-table tests:

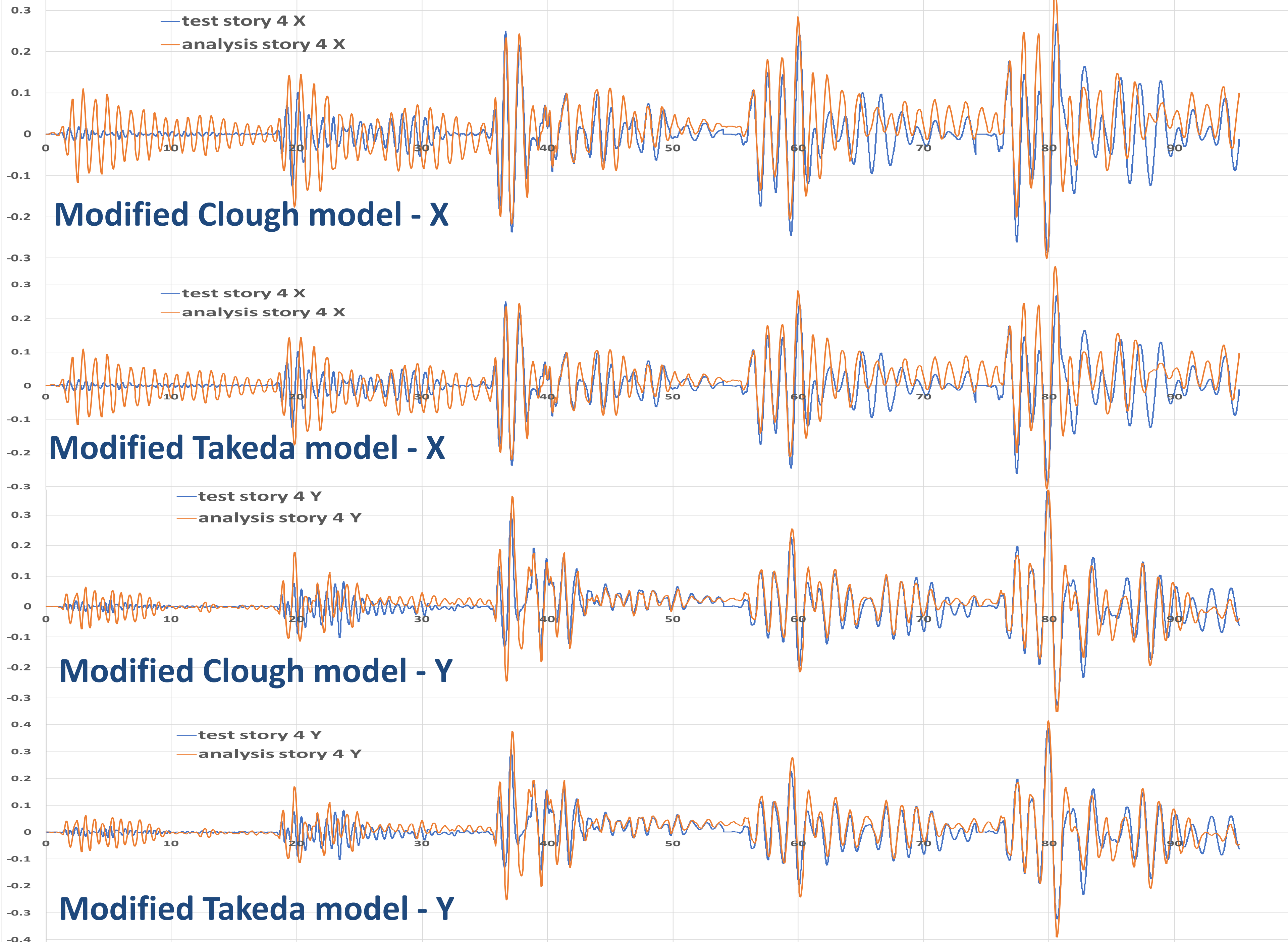
- Kobe (1995) JMA records:
  - scale-factor of 0.25 (PGAs: 0.16g, 0.27g)
  - scale-factor of 0.5 (PGAs: 0.36g, 0.47g);
  - scale-factor of 1.0 (PGAs: 0.79g, 1.07g);
- JR-Takatori records:
  - scale-factor of 0.4 (PGAs: 0.31g, 0.34g);
  - scale-factor of 0.6 (PGAs: 0.46g, 0.55g).


Defense
NIED

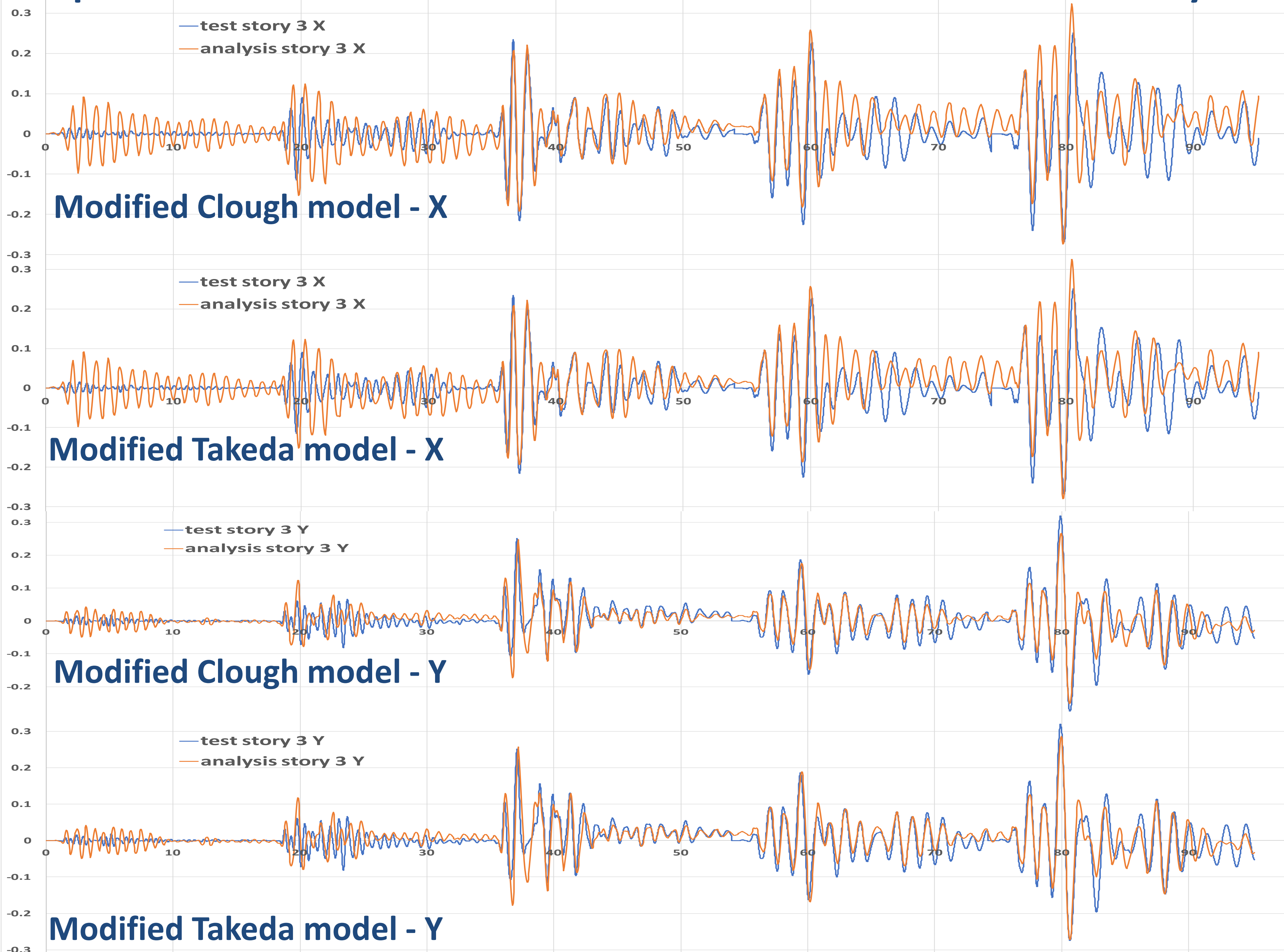
Nonlinear response-history analysis & performance evaluation using Eurocode 8 rules for member models

- Columns with P-Δ effects, fixed at foundation.
- Beam-column joints: of finite size, but rigid.
- Members:
  1. Point-hinge model, without biaxial or axial-flexural coupling;
  2. Modified Takeda or Clough hysteresis (bilinear envelope, no strength decay) with mass- & initial-stiffness-proportional Raleigh damping, taken as 5% at the two lowest periods.
  3. $EI = M_y L_s / 3\theta_y$: secant at yielding in skew-symmetric bending;
  4. Loss of resistance after ultimate deformation: ignored.
- Five shake-table motions applied in a row, without zeroing the residual drift after each test.
- Performance evaluated via chord rotation demand-to-capacity (damage) ratio, with “capacity” taken according to Eurocode 8.

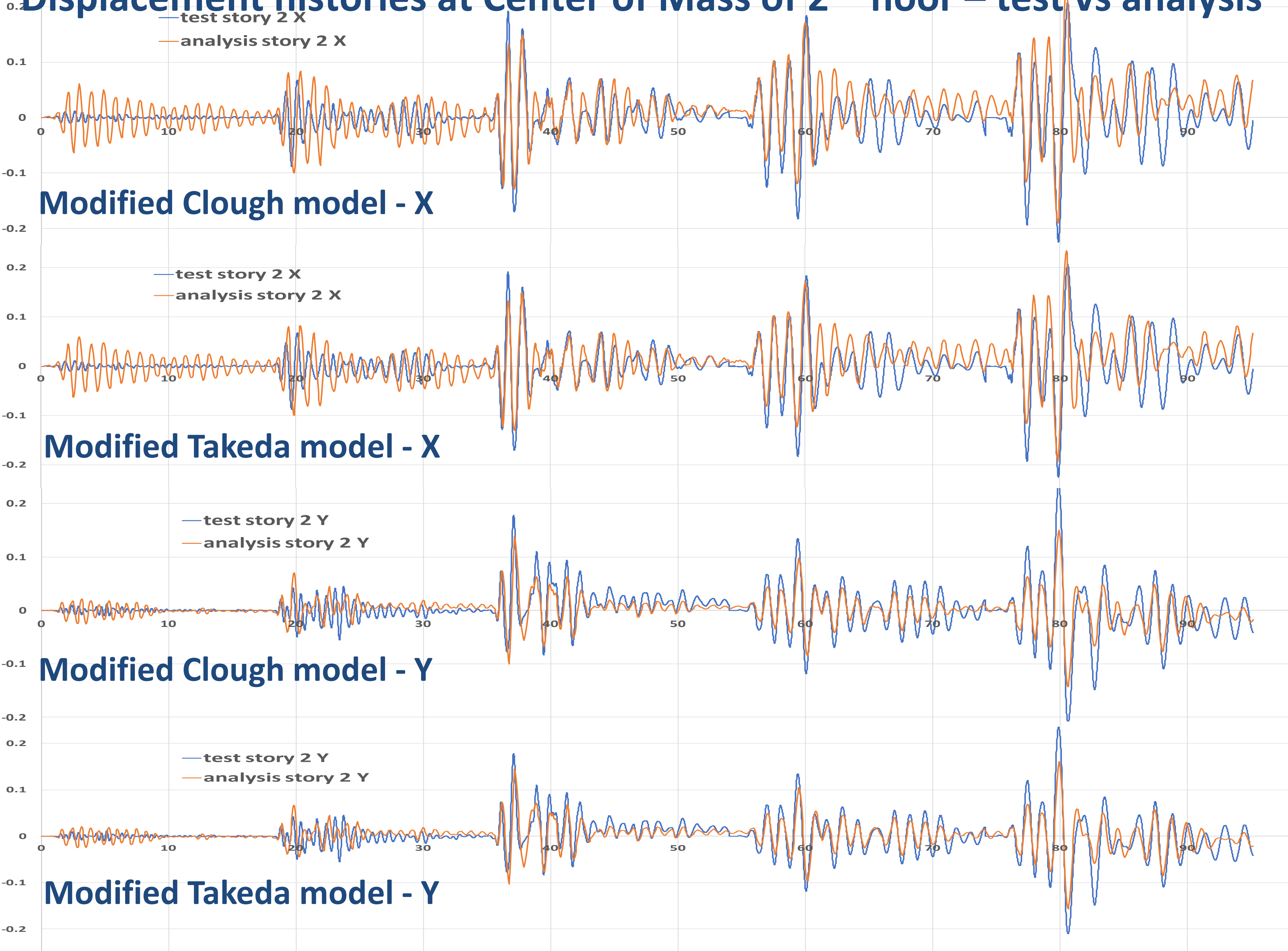
Displacement histories at Center of Mass of 4th floor – test vs analysis



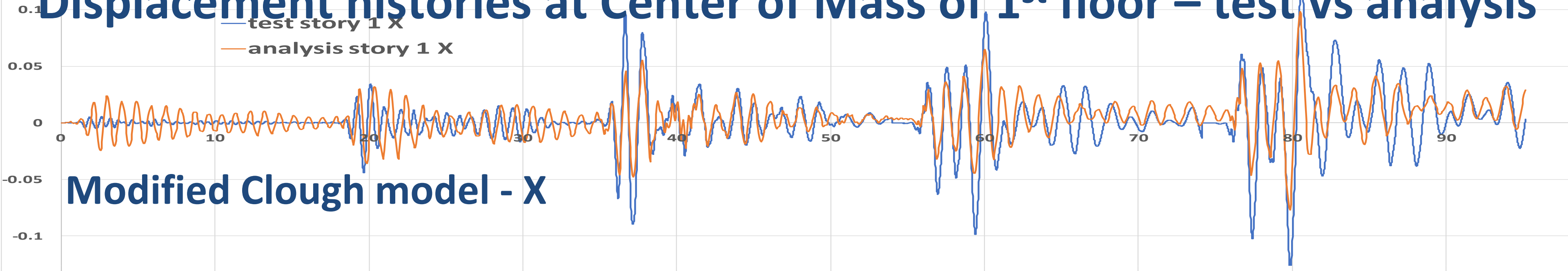
Displacement histories at Center of Mass of 3rd floor – test vs analysis

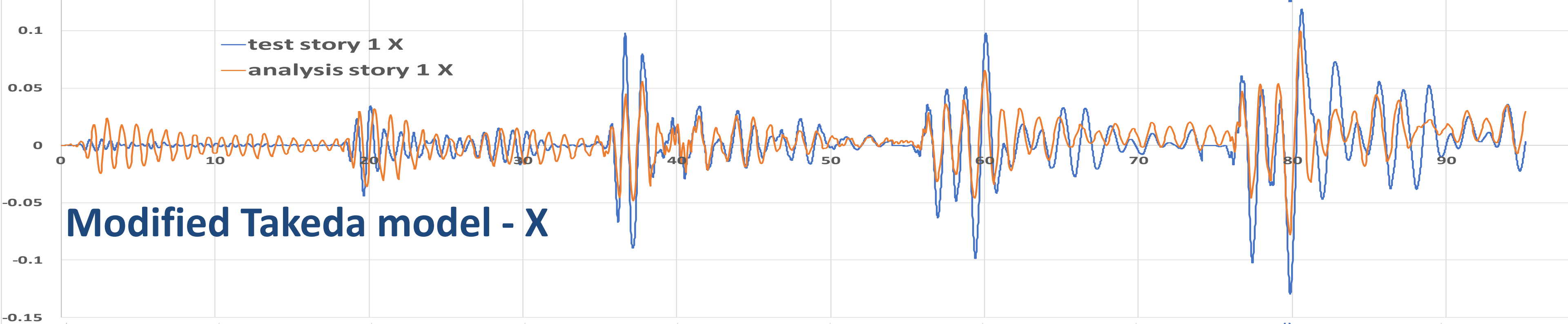


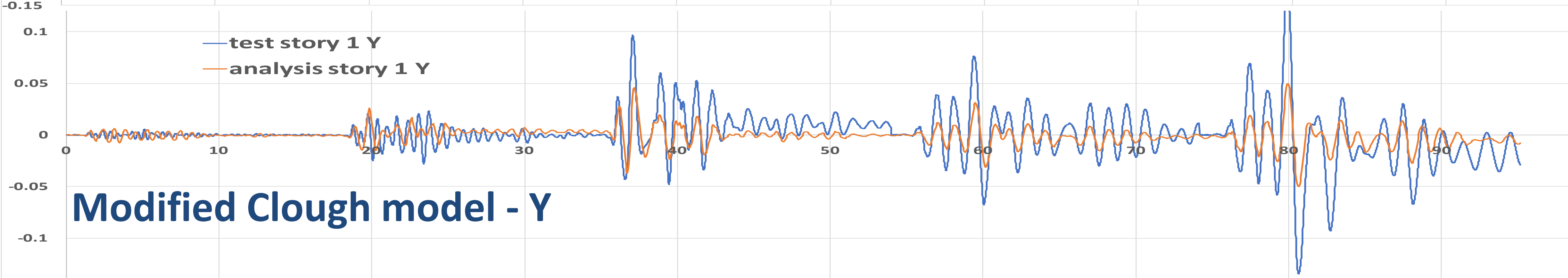
Displacement histories at Center of Mass of 2nd floor – test vs analysis

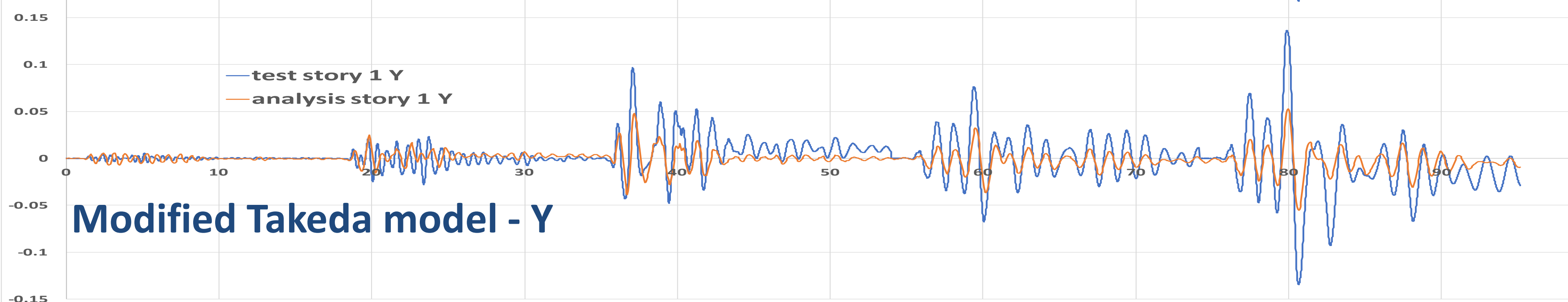


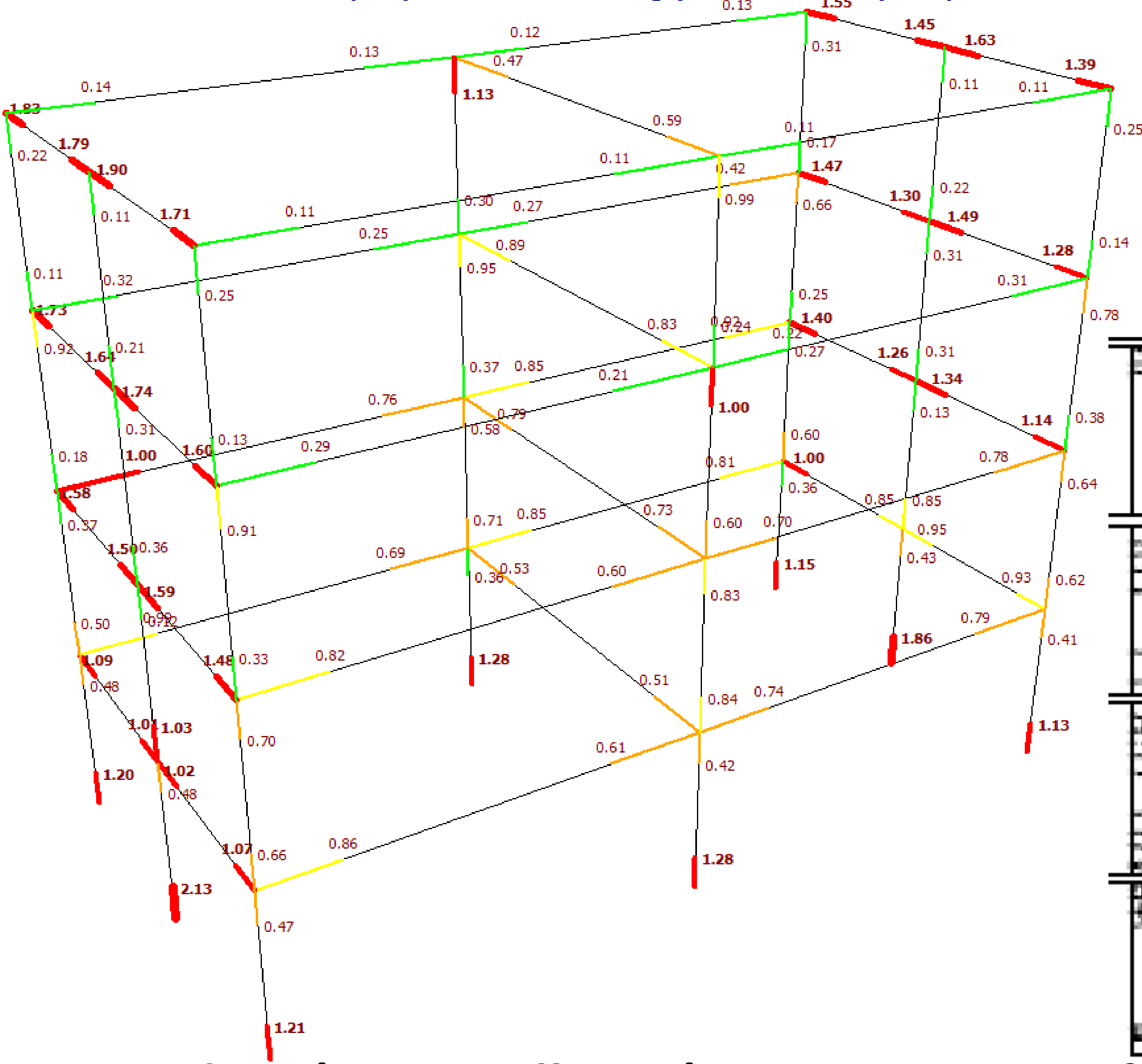
Displacement histories at Center of Mass of 1st floor – test vs analysis

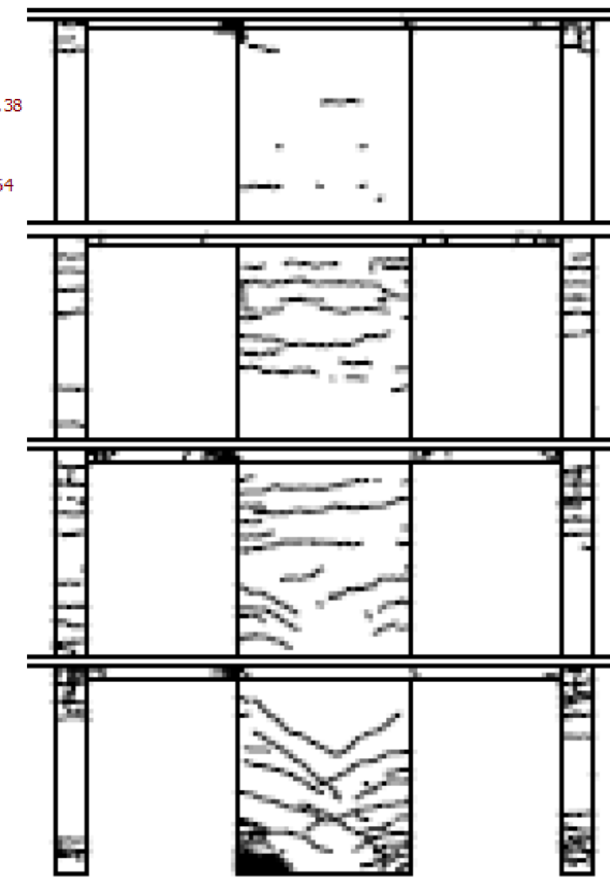




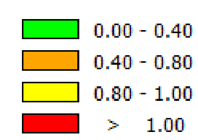


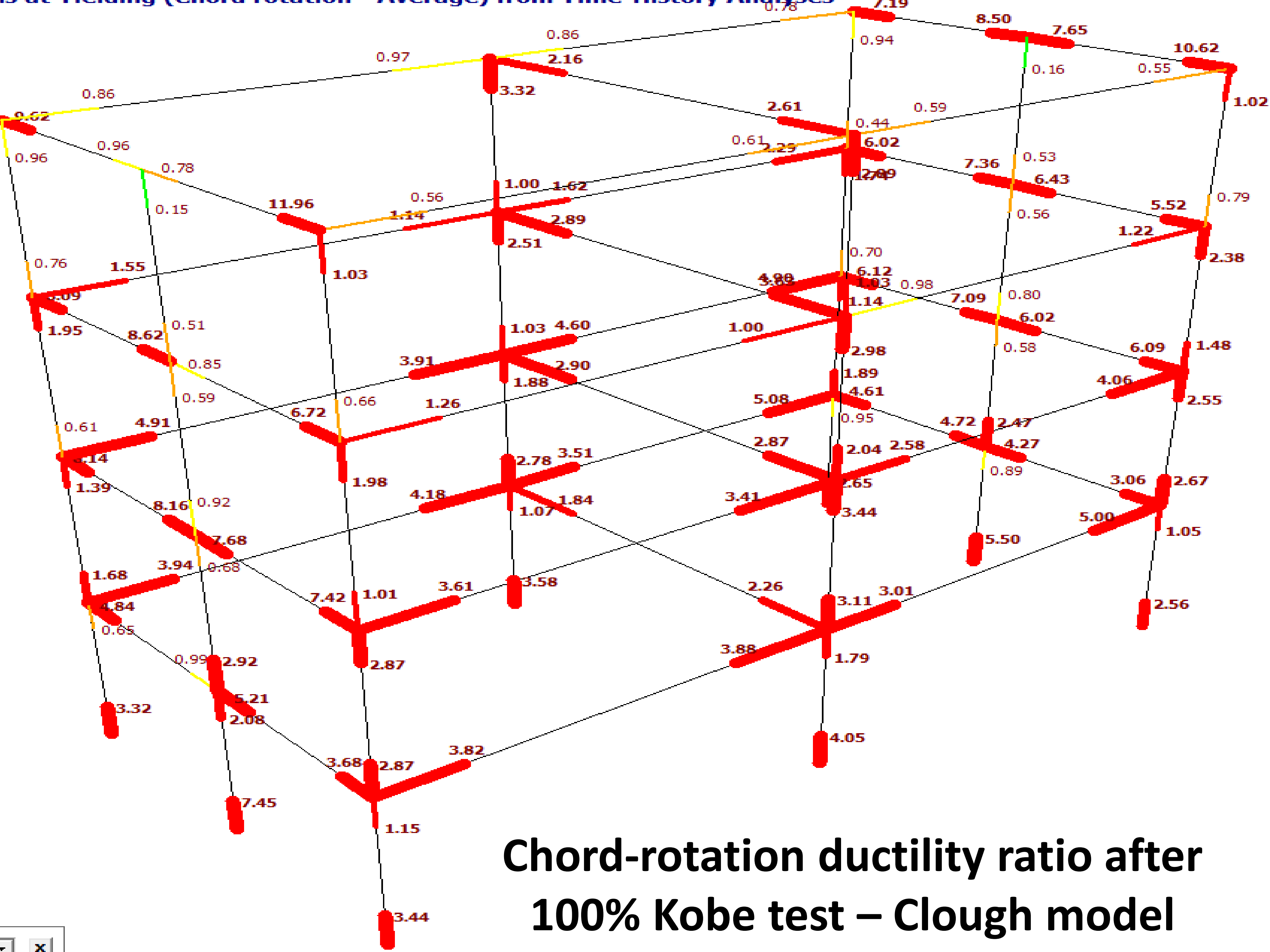






Demand-to-(Near Collapse) Capacity ratio after 100% Kobe – Clough model





Chord-rotation ductility ratio after 100% Kobe test – Clough model

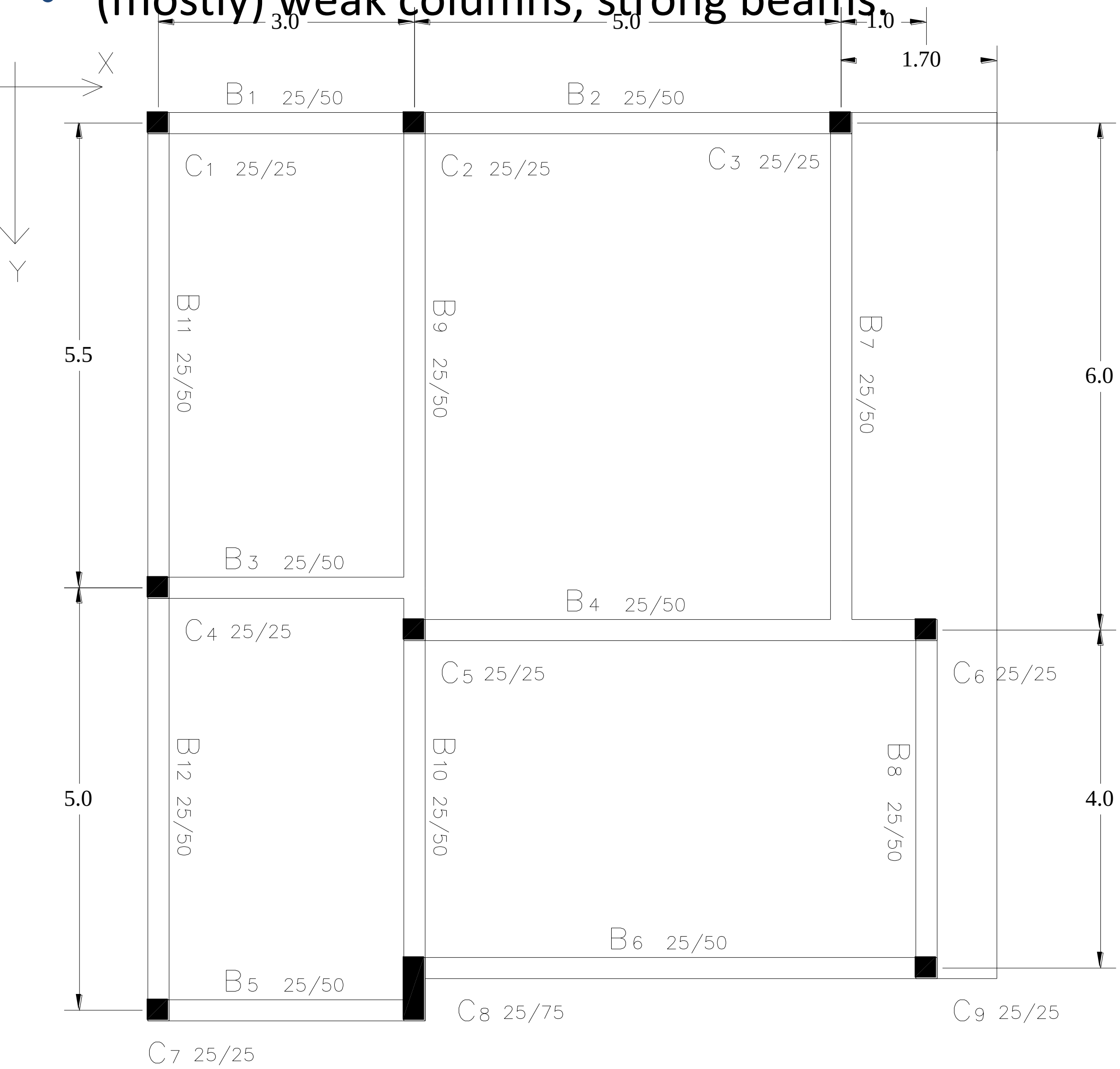
Conclusions from simulation of E-defence tests

- The effective stiffness of members according to the current rules in Eurocode 8 reproduced well the dominant periods and the displacement waveforms under high intensity shaking (that causes moderate to heavy damage), but led to overestimation of both under "serviceability" motions
- Simple nonlinear models of the type allowed in Eurocode 8 gave reasonable estimates of the displacement response, but with some over-estimation at upper floors and under-estimation at the lower ones (especially in the direction of the walls, as shear-sliding at the base is ignored).
- The extent and location of damage is well predicted.

Simulation of 2-directional SPEAR building tests

Torsionally imbalanced Greek building of the '60s; no engineered earthquake-resistance

- eccentric beam-column connections;
- plain/hooked bars lap-spliced at floor levels;
- (mostly) weak columns, strong beams.



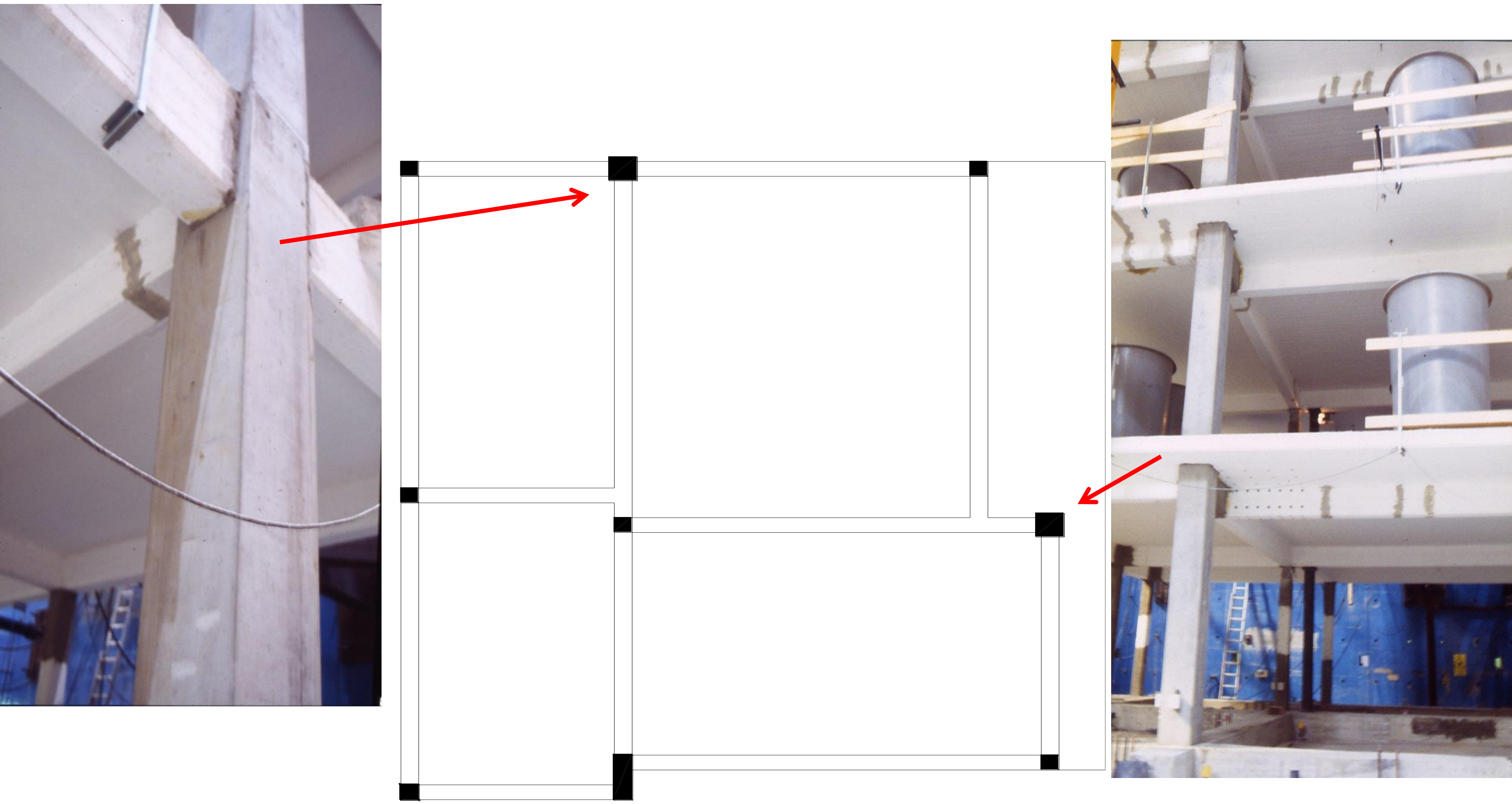
Unretrofitted building: Pseudodynamic tests at PGA 0.15g & 0.2g

Fiber-Reinforced Polymer (FRP) retrofitting. Test at PGA 0.2g

- Ends of 0.25 m-square columns wrapped in uni-directional Glass FRP over 0.6 m from face of joint.
- Full-height wrapping of 0.25x0.75 m column in bi-directional Glass FRP for confinement & shear.
- Bi-directional Glass FRP applied on exterior faces of corner joints

RC-jacketing of two columns. Tests at PGA 0.2g & 0.3g

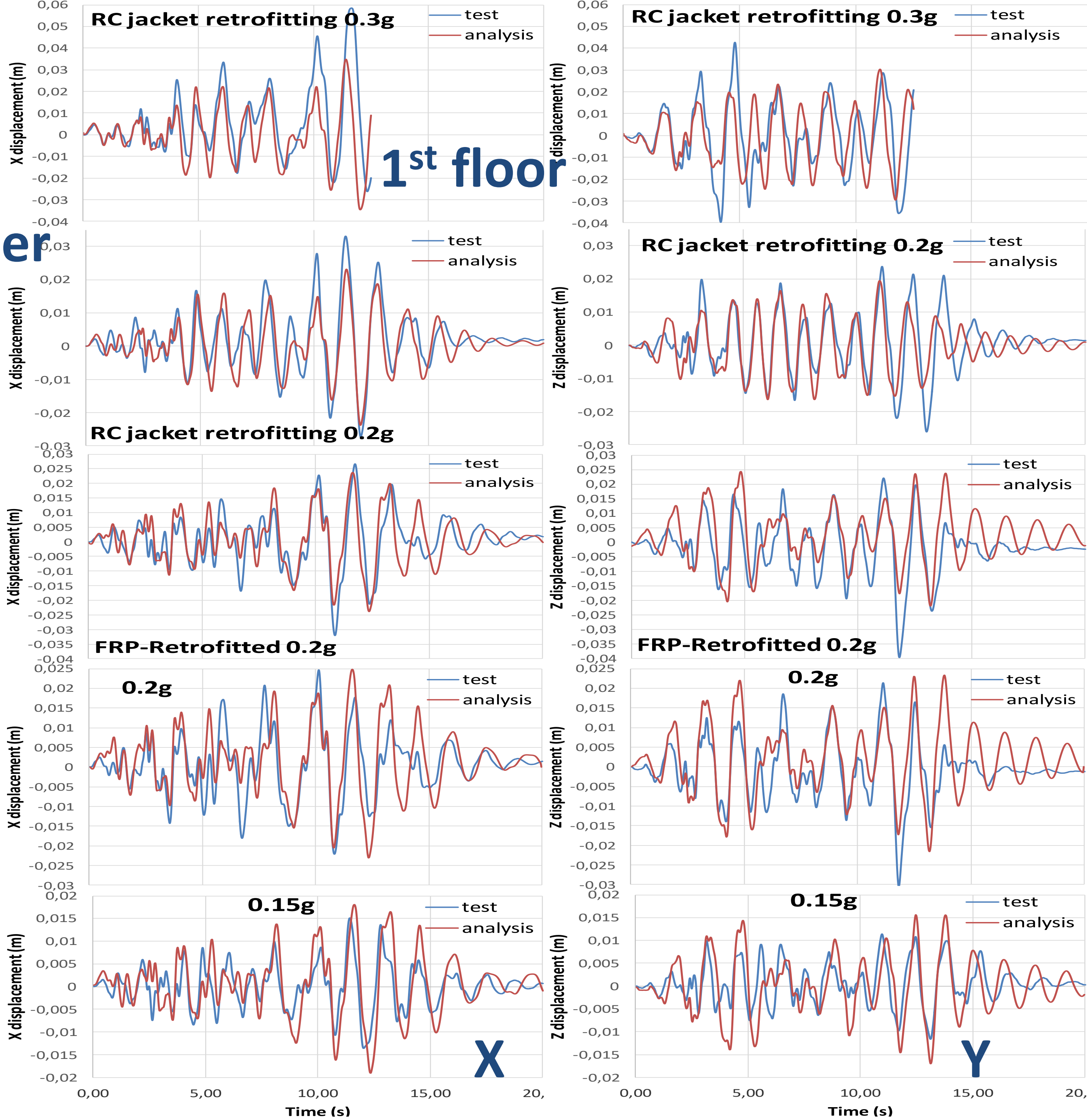
- FRP wrapping of all columns removed.
- RC jacketing of central columns on two adjacent flexible sides from 0.25 m- to 0.4 m-square, w/ eight 16 mm-dia. bars & 10 mm perimeter ties @ 100 mm centres.

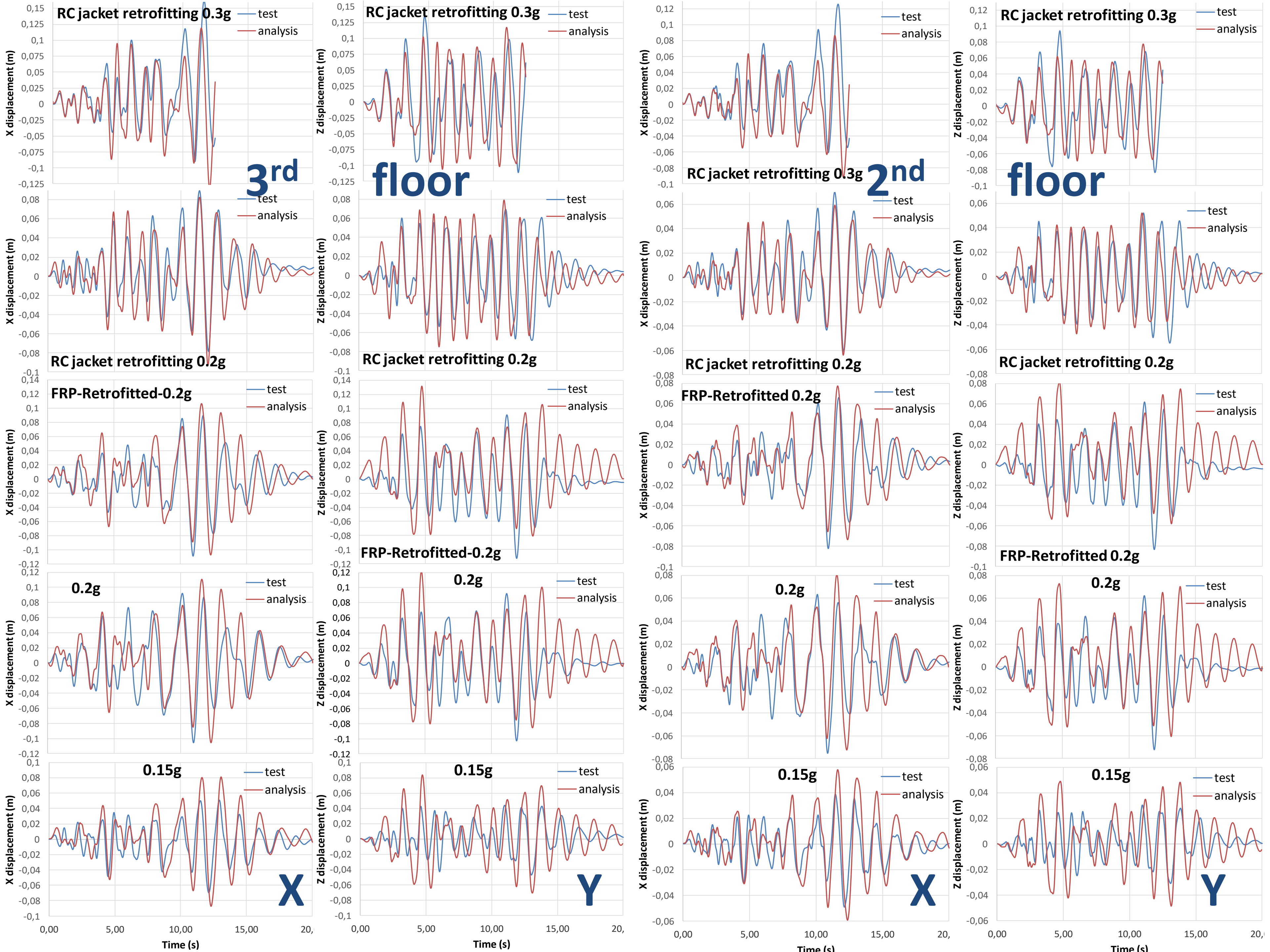


Nonlinear response-history analysis & performance evaluation using new Eurocode 8 rules for member models

- Columns with P-Δ effects, fixed at foundation
- Finite size, but rigid beam-column joints.
- Members:
  1. Point-hinge model, without biaxial or axial-flexural coupling;
  2. Modified Takeda hysteresis (bilinear envelope, no strength decay) with mass- & initial-stiffness-proportional Raleigh damping of 5%.
  3. $EI = M_y L_s / 3\theta_y$: secant at yielding in skew-symmetric bending;
  4. Unretrofitted columns: smooth hooked bars lap-spliced at floor level:
     - **Adaptation of Strut & Tie models derived from cantilever or doubly-fixed specimens to geometry of multistorey building**
  5. Effects of FRP-wrapping and RC jacketing of columns considered.
- Performance evaluated via chord rotation demand-to-capacity ratio:
  - At “ultimate deformation” (: resistance < 80% of yield moment) Demand-to-capacity (damage) ratio = 1.
- **Amplitude of torsional response underpredicted (due to ignoring biaxial effects in columns): Focus on translations at floor CM**

Displacement histories at Center of Mass

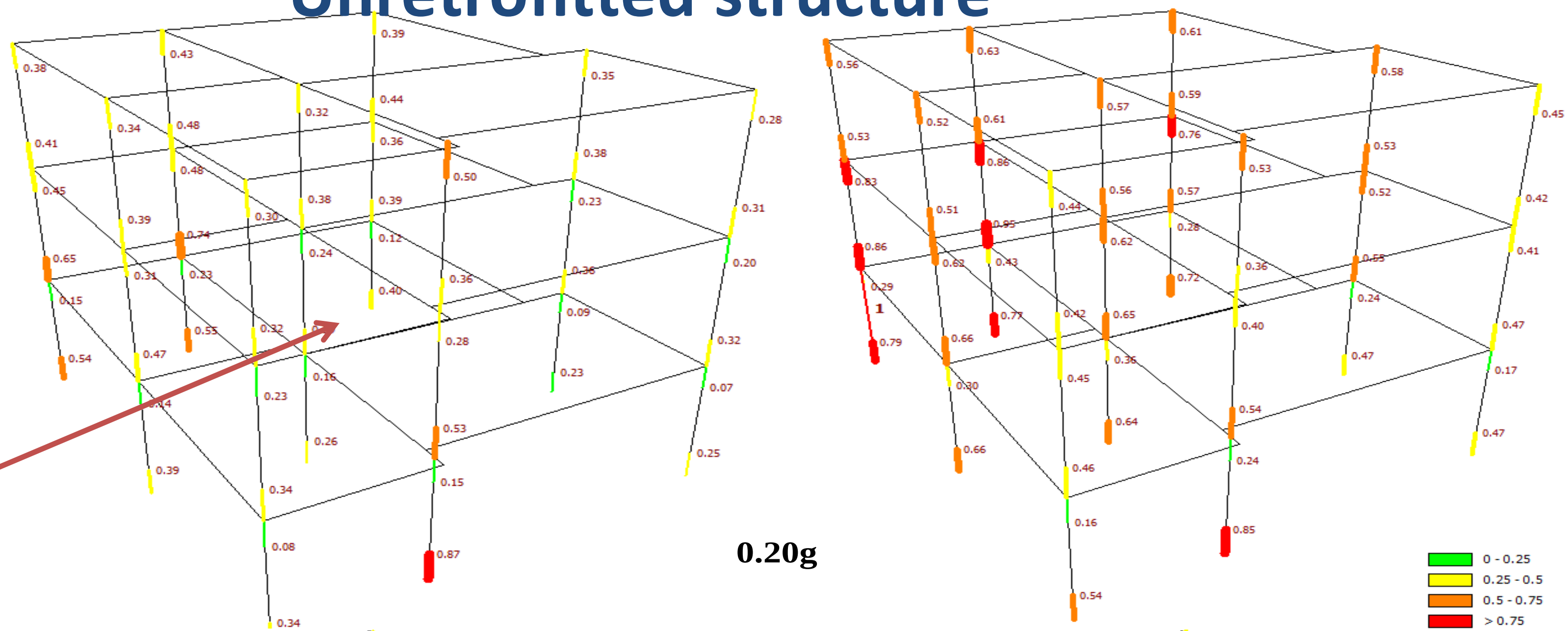


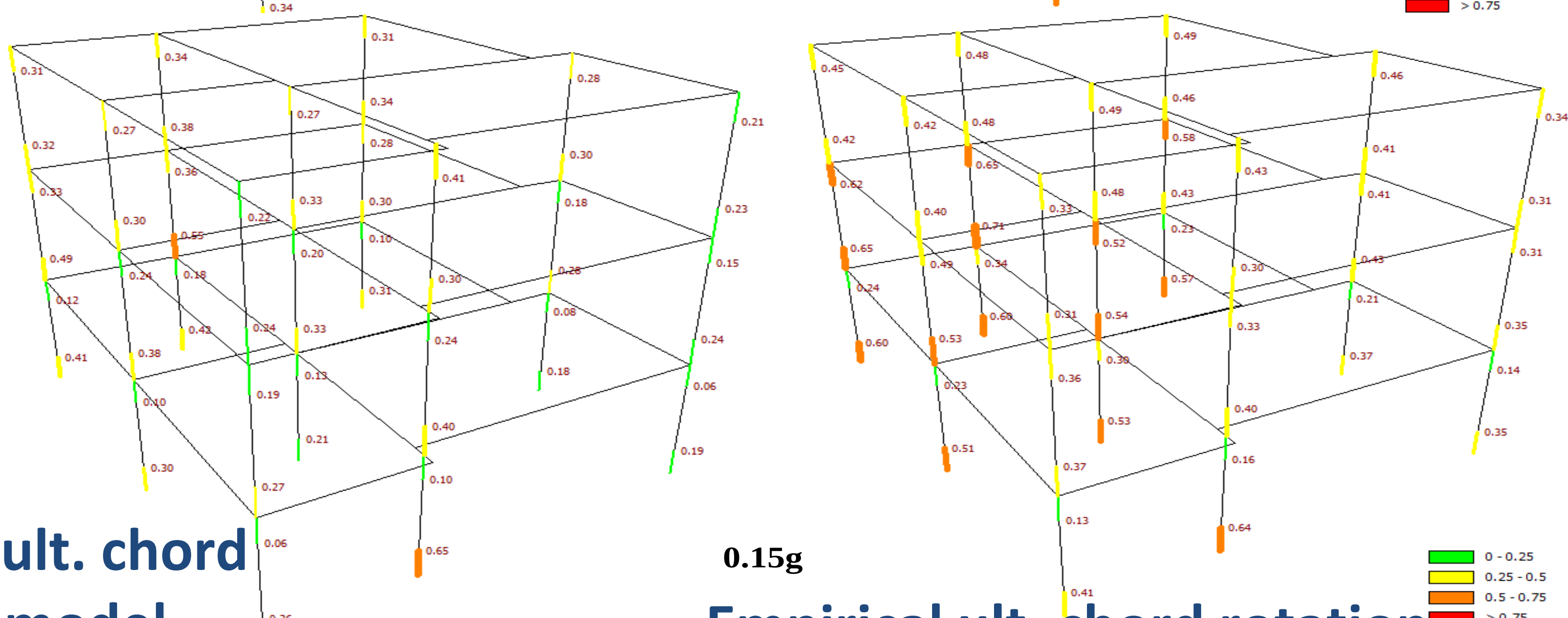


3rd floor
2nd floor
RC jacket retrofitting 0.3g
RC jacket retrofitting 0.2g
FRP-Retrofitted-0.2g
FRP-Retrofitted 0.2g
0.2g
0.15g
test
analysis
X displacement (m)
Z displacement (m)
Time (s)
X
Y

Chord-rotation-demand-to-ultimate-chord-rotation-ratio

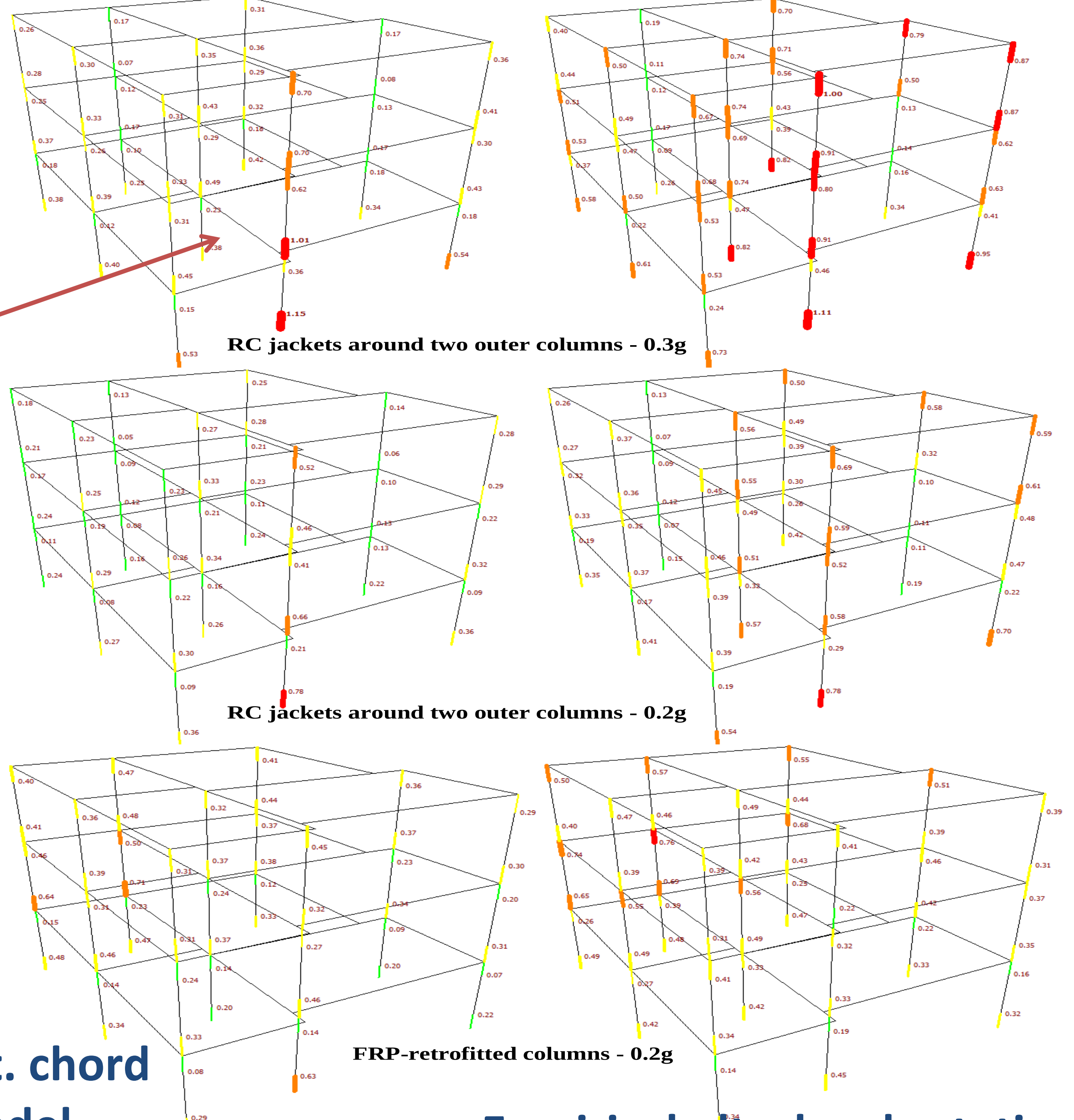
Unretrofitted structure





Physical ult. chord rotation model

Empirical ult. chord rotation



Chord-rotation-demand-to-ultimate-chord-rotation-ratio: Retrofitted structure

Physical ult. chord rotation model

Empirical ult. chord rotation

Conclusions of Case Study of SPEAR test building

- Estimation of effective stiffness of members with smooth bars lap-spliced at floor levels validated by the good agreement of predominant periods of computed and recorded displacement waveforms.
- Extent and location of damage agree better with the physical model of ultimate chord rotation than with the empirical one.

Promising alternative:
Energy-based seismic design (EBD)

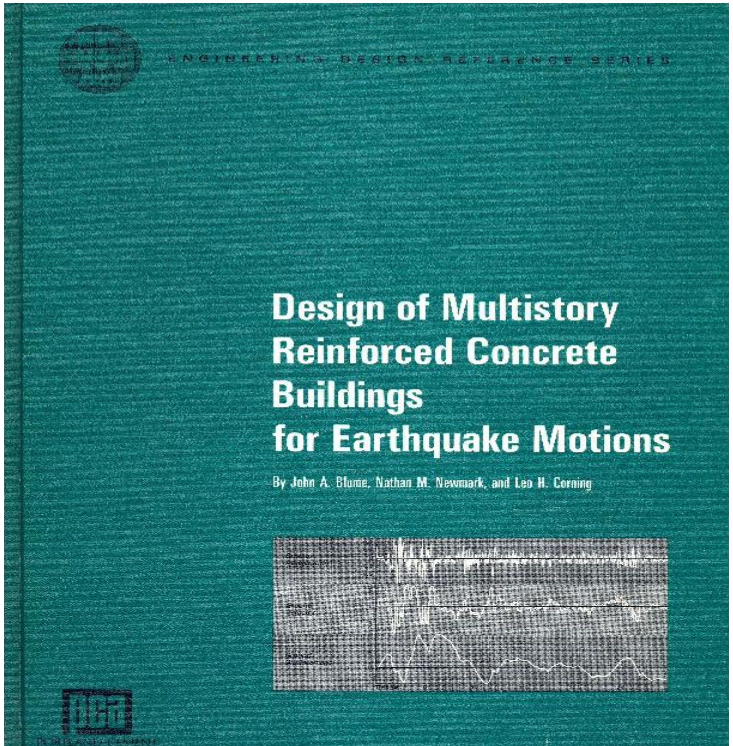
Energy demand < Energy capacity

Pros of EBD

- Energy balance (or conservation): a law of nature, as solid, familiar to engineers and easy to apply as equilibrium.
- Input energy from an earthquake per unit mass essentially depends only on a structure's fundamental period, no matter the viscous damping ratio, the inelastic action (ductility factor) or the number of degrees of freedom - the equivalent of the "equal displacement rule“ (but what happens in 3D cases?).
- Forces, displacements: vectors, with components considered separately in design. 3D seismic response better summarized by a scalar, such as energy.
- Energy demand embodies more damage-related-information than peak displacements (number of cycles, duration).
- The energy capacity of concrete elements with large stiffness/ strength contrast between the two lateral directions is much more balanced between these directions than resistance and ductility.
- The evolution of the components of energy can flag failure to converge or instability in nonlinear response-history analysis.

Early history of EBD

- Seismic energy and its potential first mentioned:
  - Housner GW (1956) Limit design of structures to resist earthquakes. *Proc. 1st World Conf. Earthq. Eng.* Berkeley, CA.
- Very important (but forgotten) ideas concerning energy capacity:
  - Blume JA, Newmark NM, Corning LH (1961) Design of multistory reinforced concrete buildings for earthquake motions. *Portland Cement Association*



The history of EBD *(cont'd)*

- Seminal publications drew attention to seismic energy 25-30 yrs later:
  - Zahrah TF, Hall WJ (1984) Earthquake energy absorption in SDOF structures *ASCE J. Struct. Eng.* **110**(8)
  - Akiyama H (1988) Earthquake-resistant design based on the energy concept *9th World Conf. Earthq. Eng.* Tokyo-Kyoto.
  - Uang CM, Bertero VV (1990) Evaluation of seismic energy in structures *Earthq. Eng. Struct. Dyn.* **19**
- EBD considered, along with DBD, as the promising approach(es) for Performance-based Seismic Design, in:
  - SEAOC (1995) *Performance Based Seismic Engineering of Buildings* VISION 2000 Committee, Sacramento, CA
- Boom of publications for ~20 to 25 years.
- Then effort run out of steam and research output reduced to a trickle
- No impact on codes.
- EBD was eclipsed by (its junior by 35 years) DBD.

EBD: State-of-the-Art and challenges

- State-of-the-Art satisfactory only concerning seismic energy input:
  - Shape and dependence of seismic energy input spectra on parameters: fully understood and described.
  - Attenuation equations of seismic energy input with distance from the source: established.
- The distribution of energy input in the structure (height- & plan-wise) and its breakdown into kinetic, stored as deformation energy – recoverable or not – and dissipated in viscous and hysteretic ways: well studied ; some hurdles remain:
  - Dependence of energy input on period(s) in coupled 3D cases?
  - Global Rayleigh-type viscous damping produces fictitious forces and misleading predictions of inelastic response. Replace with elemental damping, preferably of the hysteretic type alone?
  - Potential energy of weights supported on rocking vertical elements: important component of the energy balance – yet presently ignored.
- The energy capacity of the structure is the most challenging aspect; it remains a terra incognita, essentially not addressed so far.

EBD: needs, potential and prospects

- Achievements concerning the seismic input energy and the progress so far regarding the demand side, will be wasted and an opportunity for a new road to performance-based design will be missed, unless:
  - A concerted effort is undertaken on the analysis side to:
    - resolve the issue of modeling energy dissipation; and
    - find an easy way to account for the variation in the potential energy of weights supported on large rocking elements, such as concrete walls of large length (a geometrically nonlinear problem).
  - The capacity of various types of elements to dissipate energy by hysteresis and to safely store deformation energy is quantified in terms of their geometric features and material properties.
  - Energy-based design procedures are devised and applied on a pilot basis, leading to a new, energy-based, conceptual design thinking.
- Goal: infiltration of codes of practice and seismic design standards
- Europe is the most promising region for that, as it is more daring and its academics (still) have strong influence on codes.