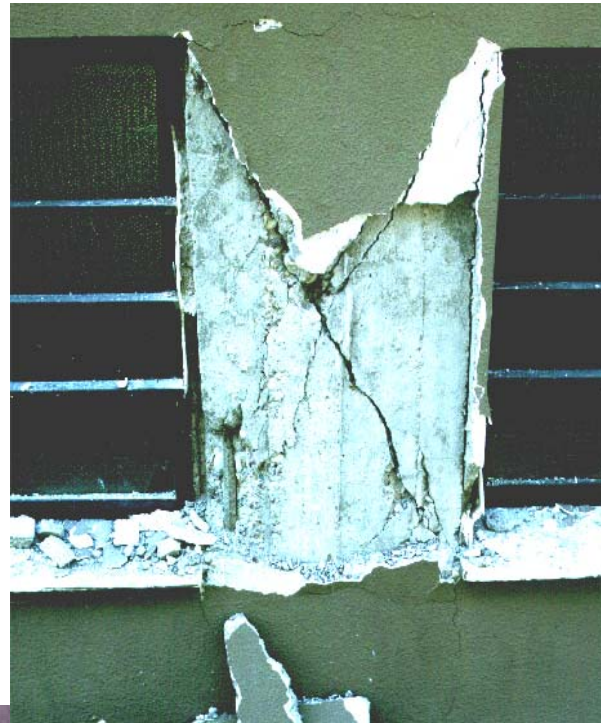
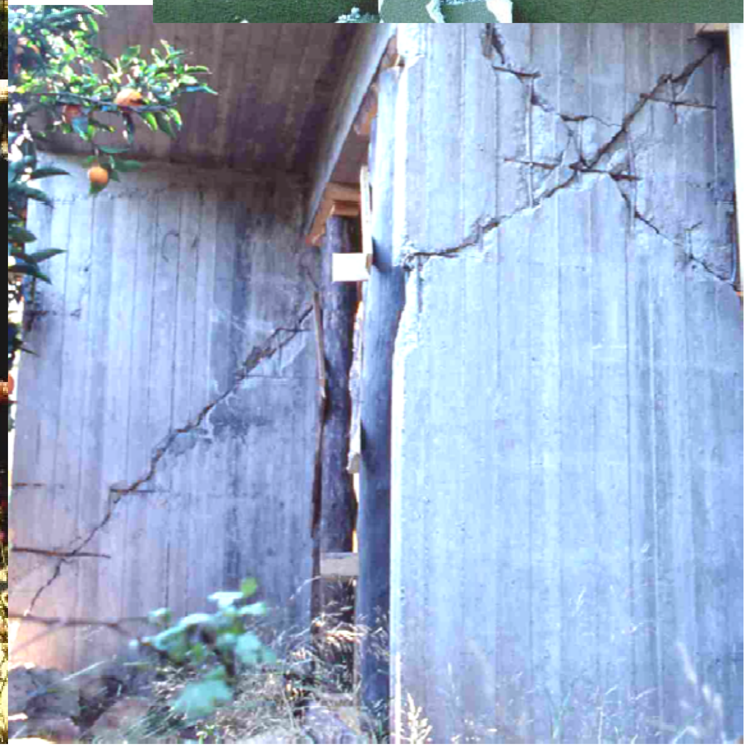
Cyclic Shear

Shear failures of columns or walls

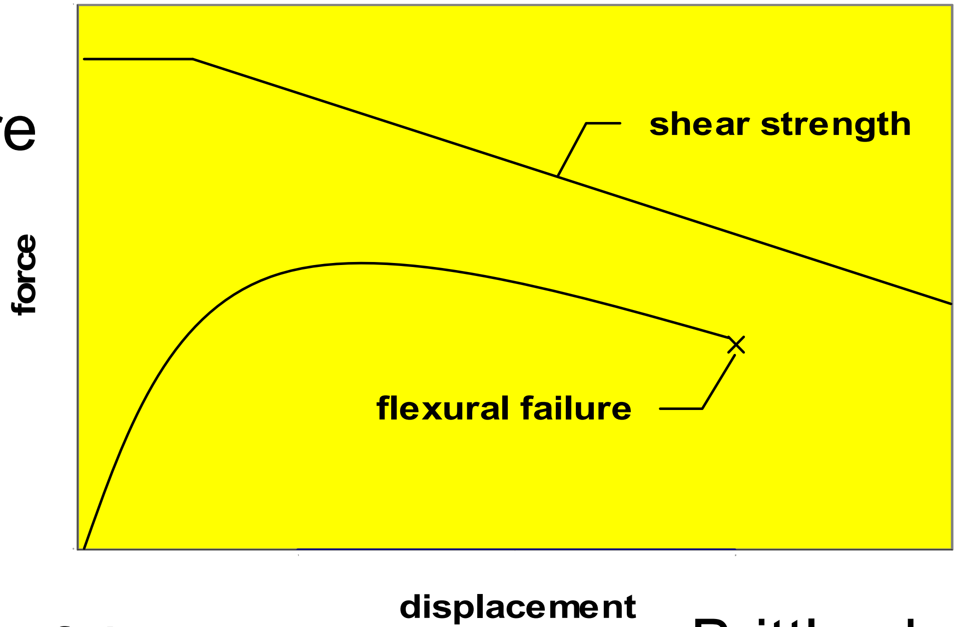




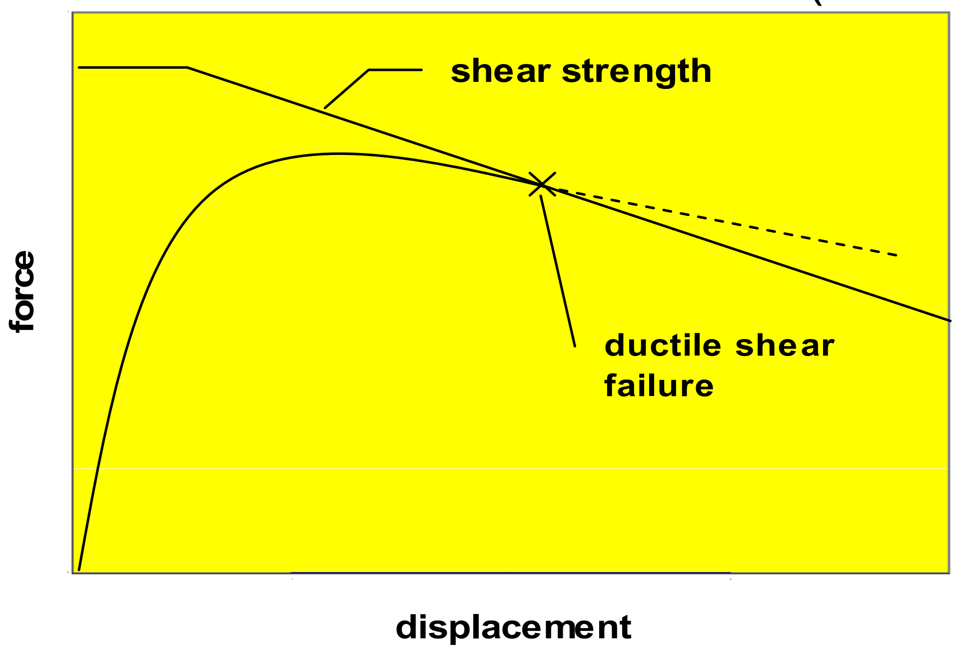
Shear failures of columns or walls (top two, in plastic hinge region)

Shear strength decay during loading

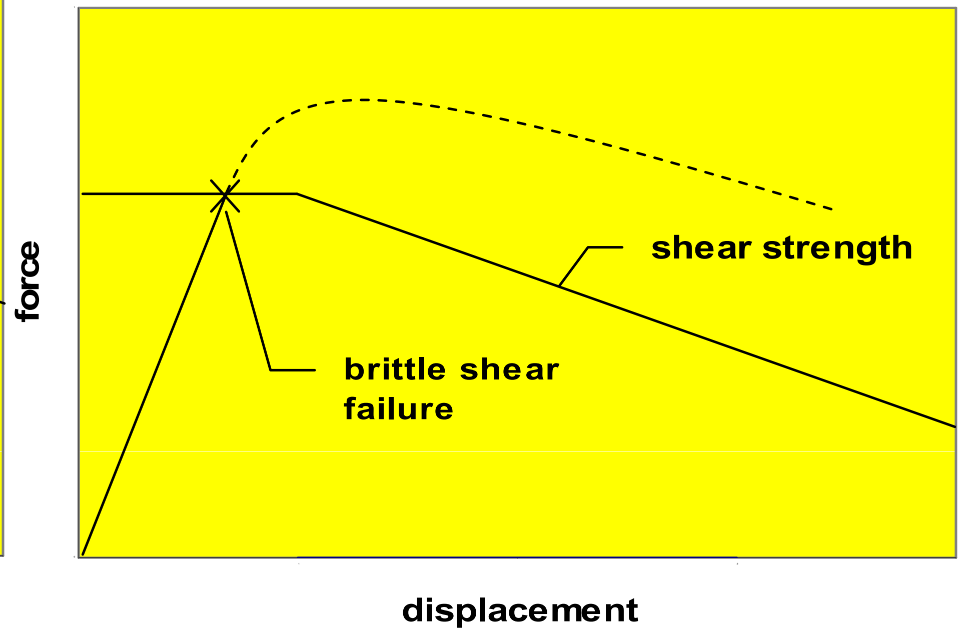
Flexural failure



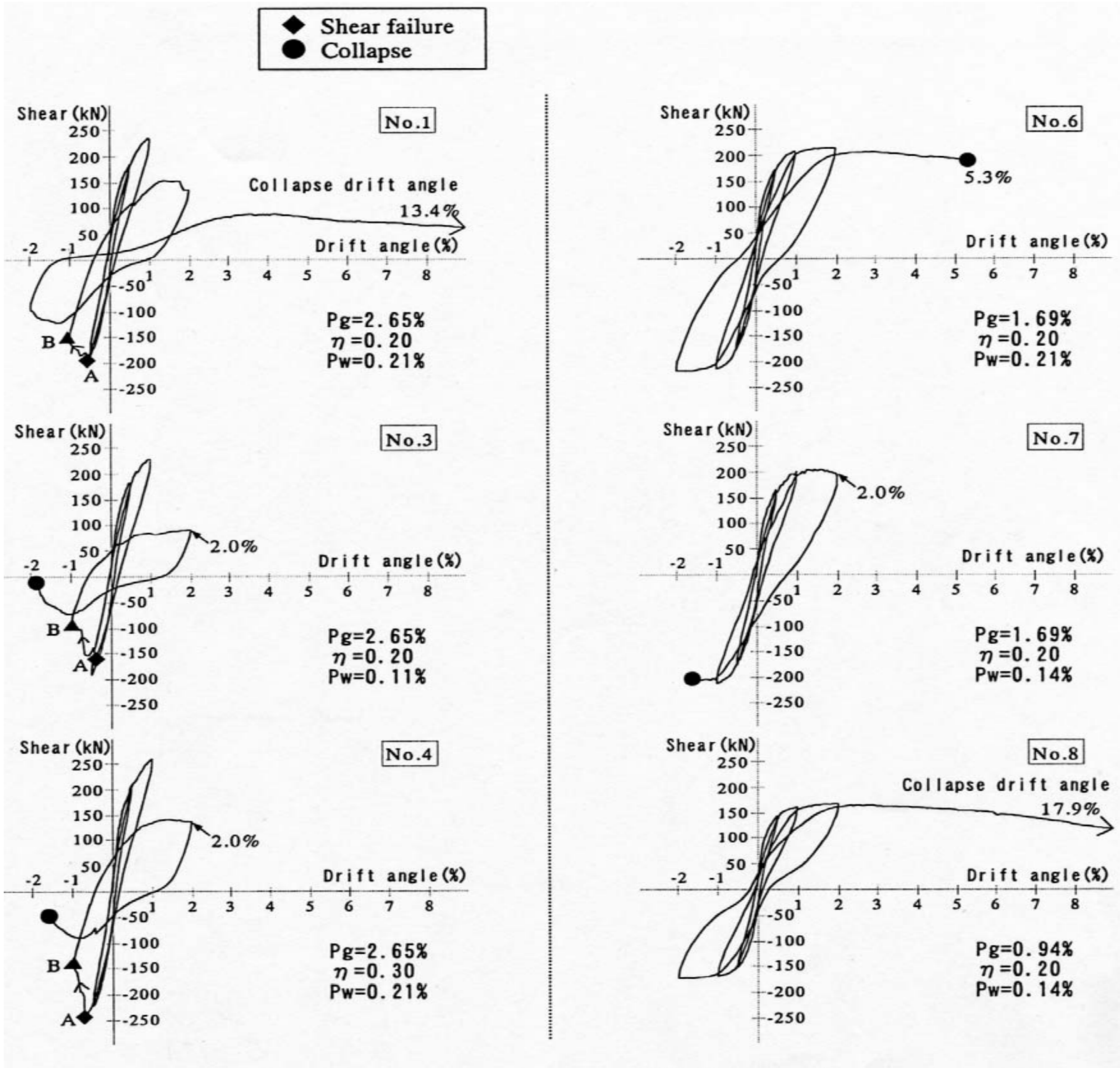
"Ductile" shear failure



Brittle shear failure
(described by monotonic shear strength model)



Brittle vs ductile behavior in cyclic shear

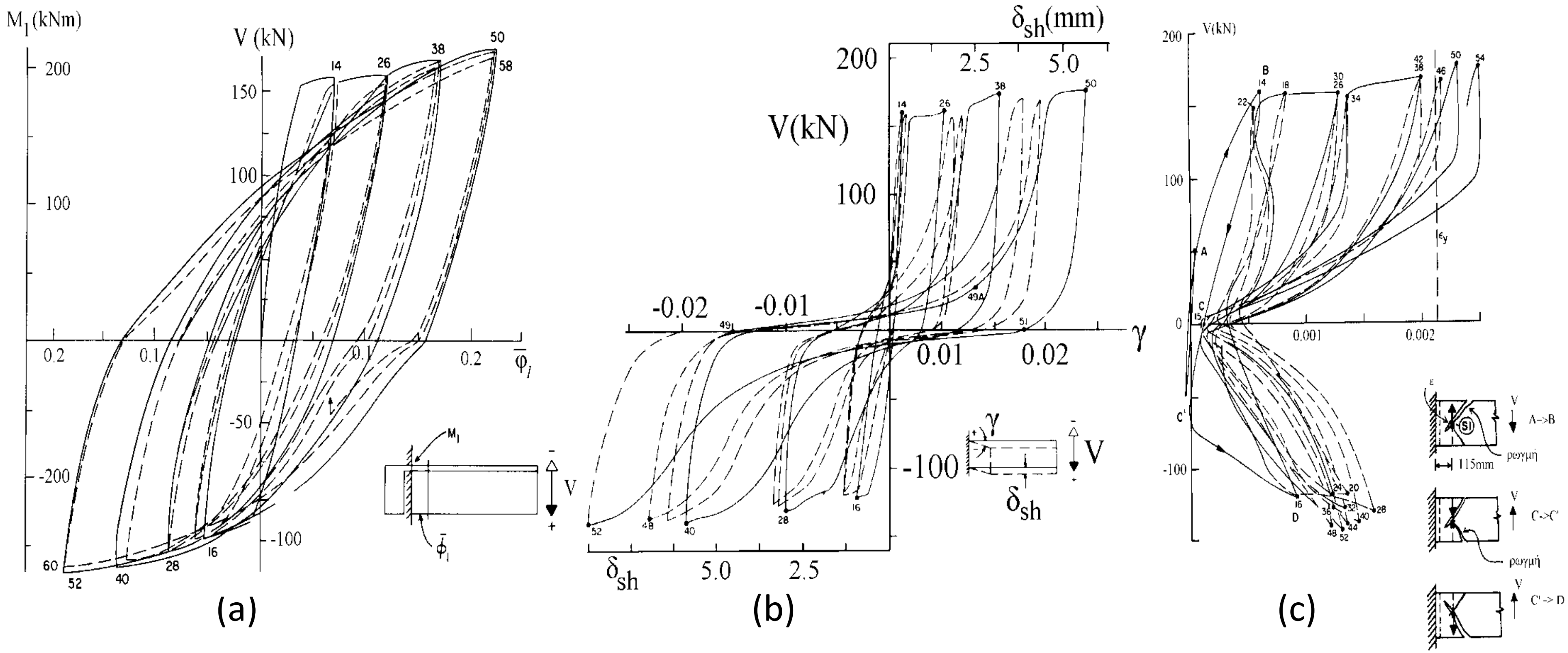


(a) Shear mode

(b) Flexure-Shear mode
(No.8: Flexure mode)

Shear force-chord rotation behavior: (a) brittle shear; (b) “ductile shear” or flexural behaviour

Effect of cyclic inelastic deformations after flexural yielding on shear behavior

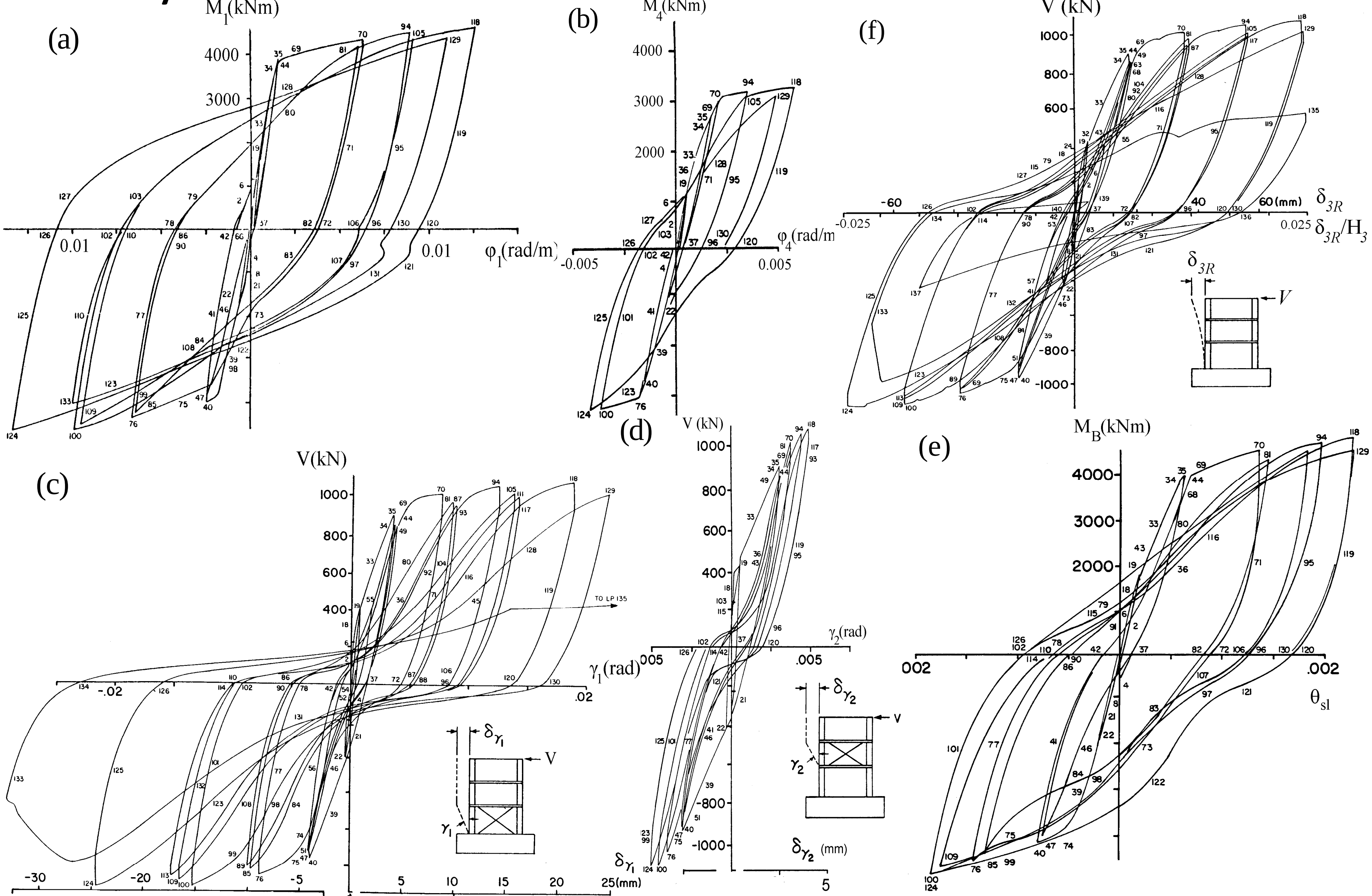


(a) (b) (c)

(a) *M*-*φ* loops next to end section; (b) *V*-*γ* loops in plastic hinge region; (c) loops of shear force (V) - stirrup strain W

Witness in (b) and (c) that shear strains & stirrup strains start increasing after flexural yielding. With little increase of the peak shear force, the stirrups go from almost zero stress to yielding. *M*-*φ* loops are very stable

Effect of cyclic inelastic flexural deformations on shear behavior -cont'd



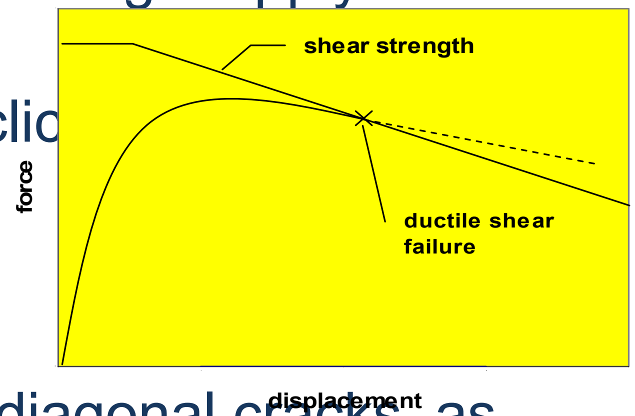
(a), (b): *M*-*φ* next to base of 1st & 2nd story; (c), (d): *V*-*γ* over 1st & 2nd story; (f) base shear v top deflection (e) base moment v fixed-end rotation due to bar pull-out from footing . Shear strains start increasing at the 1st story after yielding there, but not in the 2nd story; when failure due to shear is approaching, flexural deformations decrease!

Cyclic shear strength degradation

— Shear resistance degrades with cyclic loading: RC member that yields in flexure may ultimately fail in shear.

— Provisions of concrete design codes for shear strength apply to monotonic loading;

— Seismic codes (e.g. EC8) may reduce V_R if cyclic ductility demands are high.



Degradation mechanisms :

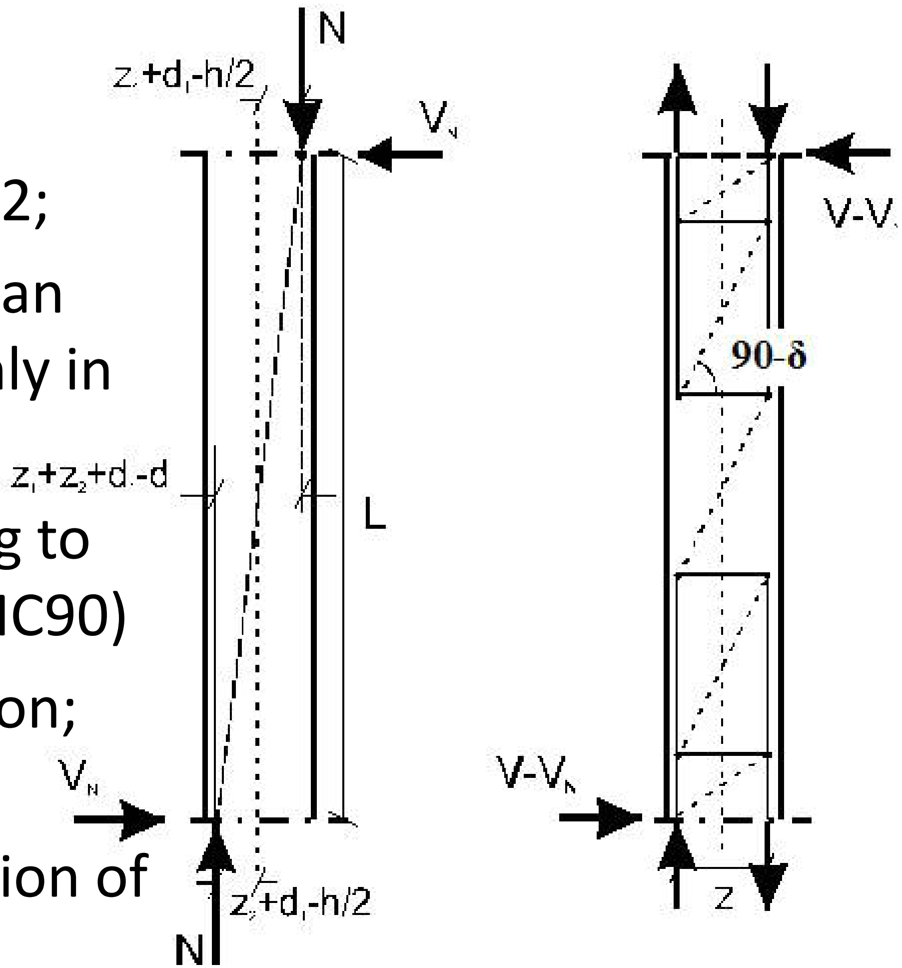
- Gradual reduction of aggregate interlock along diagonal cracks, as interfaces become smoother with cycling of the loading.
- Degradation of dowel action (also due to accumulation of inelastic strains in the longitudinal reinforcement).
- Development of flexural cracks throughout the depth of the member → reduction of contribution of compression zone to shear resistance.
- Bond slippage & accumulation of inelastic strains in shear reinforcement → aggregate interlock reduced as diagonal cracks gradually open up.
- Softening of concrete in diagonal compression due to accumulation of transverse tensile strains.

Monotonic shear resistance models in *fib* MC2010 or prEN 1992-1-1:2018 - comparison with cyclic test results

fib MC2010, Eurocode 2 (2018): Resistance in shear tension: $V_R=V_{R,s}+V_{R,c}(+V_{R,N})\leq V_{R,max}$ (resistance in shear compression)

- $V_{R,s}$: shear reinforcement contribution :
  $V_{R,s}=\rho_w b_w z f_{yw}\cot\theta$
- $V_{R,c}$: concrete contribution;
  - =0 in MC2010 Levels I & II and EC2;
  - MC2010 Level III: $V_{R,c}>0$ (other than that, the MC2010 Levels differ only in the values of θ_{min} and $V_{R,max}$).
- $V_{R,N}$: axial load contribution according to MC90 (V_N in the figure, taken from MC90)
  - MC2010 & EC2: no explicit mention; they deal only with beams
  - $V_{R,N}$ is implicit in EC2 as contribution of inclined chords to shear:

$$V_{R,N}=\frac{z}{L}N$$



Level II approximation in MC2010 & 2018 Eurocode 2

- minimum angle of compressive stress field: $\boxed{\theta_{\min} = 20^0 + 10000\varepsilon_x}$
- ε_x: longitudinal strain at section mid-depth (50% of elastic strain in tension chord due to section's M, N, V)

$$\varepsilon_x = \frac{1}{2A_{st}E_s}\left(\frac{M}{z} + V + \frac{N}{2}\right) \geq 0$$

- Contribution of concrete: $\boxed{V_{R,c} = 0}$
- Resistance to shear compression:

$$\boxed{V_{R,\max} = k_\varepsilon \eta_{fc} f_c b_w \cdot z \frac{\cot\theta}{1+\cot^2\theta}}$$

with:

$$k_\varepsilon = \frac{1}{1.2 + 55(\varepsilon_x + (\varepsilon_x + 0.002)\cot^2\theta)} \leq 0.65$$

(**2018 Eurocode 2**: 80 instead of 55, 0.001 instead of 0.002)

$$\eta_{fc} = \left(\frac{40}{f_c(MPa)}\right)^{1/3} \leq 1$$

(**in 2018 EC2**; in MC2010 the numerator is 30)

- **Squat members** (angle of compr. stress field θ<β=atan(h/L_s)):

(in 2018 EC2, denoting k_ε as ν**):**

$$V_R = b_w \cdot z \cdot k_\varepsilon \eta_{fc} f_c \frac{\cot\theta - \cot\beta}{1+\cot^2\theta} + \frac{A_{sw}}{s} z \cdot \sigma_{sw} \cdot \cot\beta \leq b_w \cdot z \cdot k_\varepsilon \eta_{fc} f_c \frac{\cot\theta}{1+\cot^2\theta}$$

with:

$$\sigma_{sw} = E_s\left(\cot^2\theta(\varepsilon_x + 0.001) - 0.001\right) \leq f_{yw}$$

Level III approximation in *fib* MC2010

- As in Level II approximation, except that:
- Contribution of concrete:

$$V_{Rc,d} = k_v \frac{\min(\sqrt{f_{ck}};8MPa)}{\gamma_c} b_w z$$

where:

$$k_v = \frac{1}{1+1500\varepsilon_x}\left(1-\frac{V_{Ed}}{V_{Rd,\max}(\theta_{\min})}\right) \geq 0$$

(**Note:** if the angle of the compressive field at which $V_{Rd,s}$+$V_{Rd,c}$ is equal to $V_{Rd,max}$ is less than the minimum value, the parenthesis has negative value and $V_{Rd,c}$ is zero.

$V_{Rd,c}$ is greater than zero only when θ = θ_{min}).

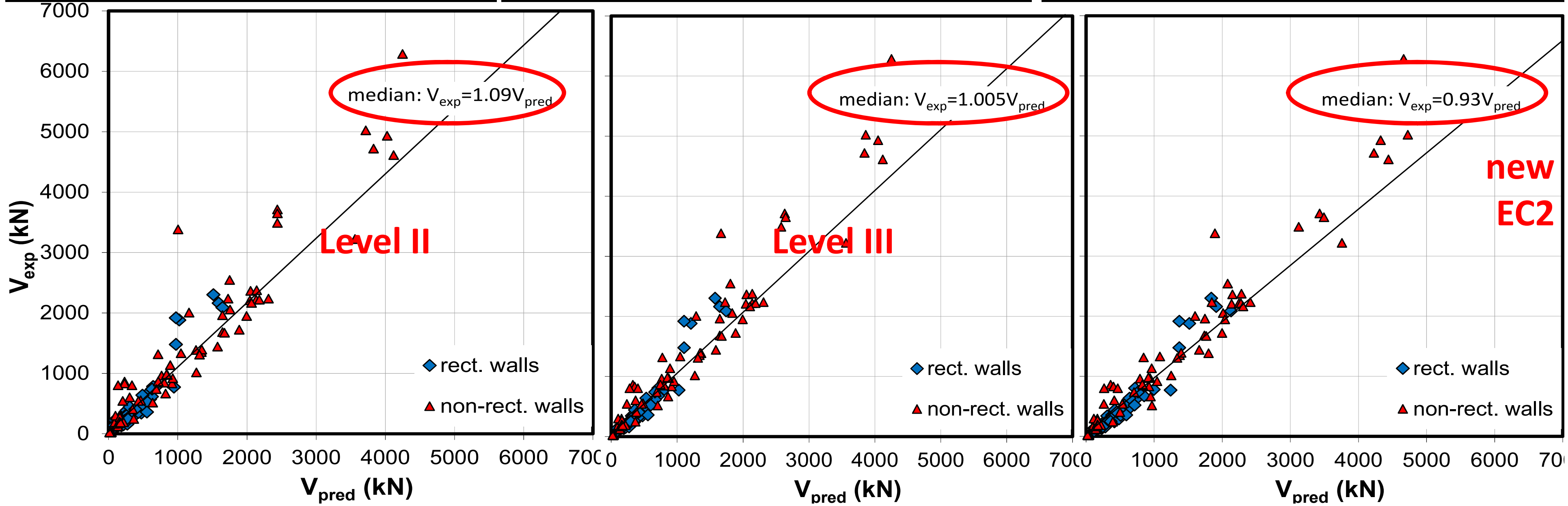
MC2010 Level II or III approach & new EC2 vs cyclic tests

- 50 rect., 99 non-rect. walls; <u>brittle shear</u> failure **before flexural yielding**
- For angle θ that gives $V_{R,max}=V_{R,s}(+V_{R,c})$ & ε_x at end section with $M=V_R L$

Level II	Rect. walls	Non-rect. walls	All
median	1.11	1.09	1.09
CoV (%)	32	61	55
No. of tests	50	99	149

Level III	Rect. walls	Non-rect. walls	All
median	0.92	1.05	1.00
CoV (%)	25	42.5	39
No. of tests	50	99	149

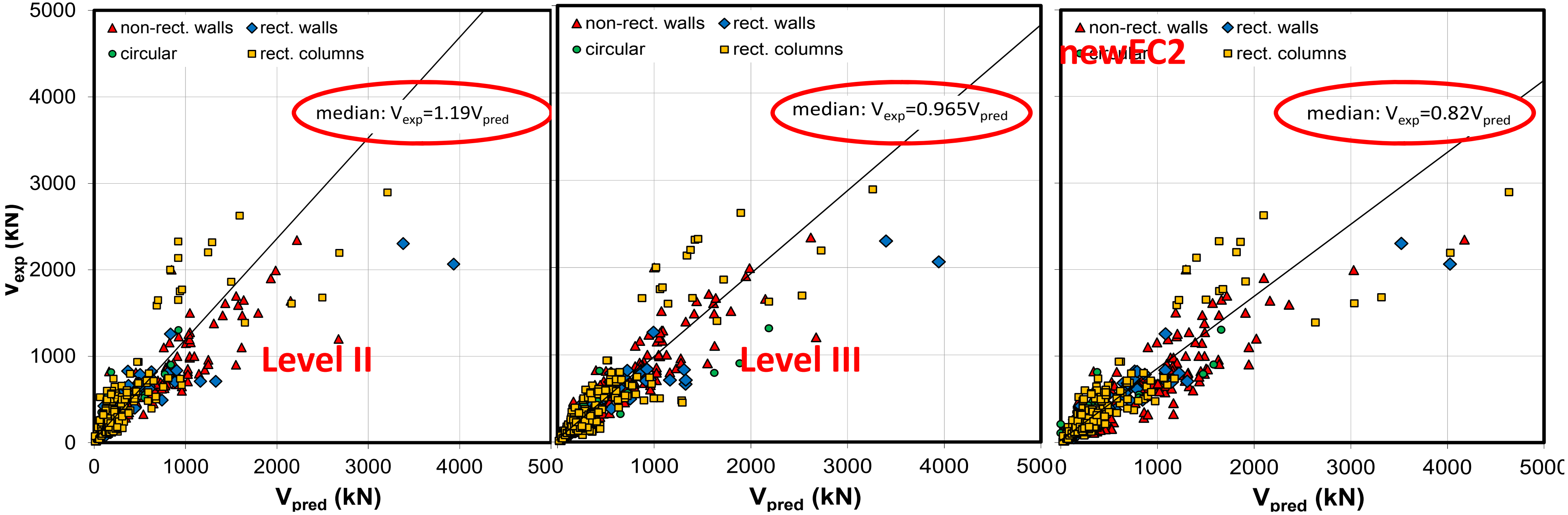
EC2	Rect. walls	Non-rect. walls	All
median	0.8	0.99	0.93
CoV (%)	25	42.5	41
No. of tests	50	99	149



MC2010 Level II or III & new EC2 – test-to-model ratio

- **571 cyclic** tests: “ductile” shear failure **after flexural yielding**.

Level II	rect. colu-mns	rect. walls	non-rect. walls	circ-ular	All	Level III	rect. colu-mns	rect. walls	non-rect. walls	circ-ular	All	new EC2	rect. colu-mns	rect. walls	non-rect. walls	circ-ular	All
median	1.28	1.14	0.98	1.52	1.19	median	0.96	0.99	0.90	1.07	0.97	median	0.88	0.79	0.71	0.99	0.82
CoV %	57	53	51	44	56	CoV %	36	26	38	29.5	35	CoV %	42	37	50	35	44
# of tests	290	60	152	69	571	# of tests	290	60	152	69	571	# of tests	290	60	152	69	571



Conclusions: MC2010 Level II or III approach & new EC2

- Cyclic resistance in brittle shear **before flexural yielding:**
  - Decent average agreement of **Level III** approach with test results;
  - Under- or over-prediction by ~8% when using Level II or new EC2, respectively;
- Cyclic resistance in “ductile” shear **after flexural yielding:**
  - Decent average agreement of **Level III** approach with test results;
  - Under- or over-prediction by ~18% with Level II or new EC2, respectively;

  But:
  - **Lack-of-fit**, of all three approaches with respect to several variables
    - Marked under- or over-prediction at low, or high values, respectively, of:
      - Ductility ratio, μ
      - Longitudinal strain at mid-depth, ε_x
      - Angle of inclination of compression field, θ
      - Transverse reinforcement ratio, ρ_w
    - Some over- or under-prediction at low, or high values, respectively, of:
      - Axial load ratio, N/A_cf_c

The semi-empirical models in Eurocode 8 (EN 1998-3:2005 and prEN 1998-3:2018)

Resistance of flexural plastic hinge against shear-tension failure in cyclic loading (after flexural yielding)

- Shear resistance in plastic hinge after flexural yielding, as <u>controlled by stirrups</u>

For rectangular or circular columns, rectangular or non-rectangular walls, or box -section

It uses a 45° truss and adopts a linear decay of $V_{R,c}$ & $V_{R,s}$ with cyclic plastic rotation ductility ratio $\mu_\theta^{pl}=(\theta-\theta_y)/\theta_y>0$, with a maximum reduction of 25%

$$V_R = V_{R,N} + \left(1 - 0.05\min\left(5;\, \mu_\theta^{pl}\right)\right)\left(V_{R,c} + V_{R,s}\right)$$

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

$$V_{R,c} = 0.16\max(0.5;100\rho_{tot})\left(1 - 0.16\min\left(5;\frac{L_s}{h}\right)\right)\sqrt{f_c(MPa)}\,A_c$$

h: section depth

x : neutral axis depth at yielding

ρ_{tot}: total longitudinal steel ratio

L_s: shear-span (M/V-ratio)

Non-circular sections:

$V_{R,s}=\rho_w b_w z f_{yw}$ (b_w: web width, z: internal lever arm; ρ_w: shear steel ratio); $A_c= b_w d$

Circular sections:

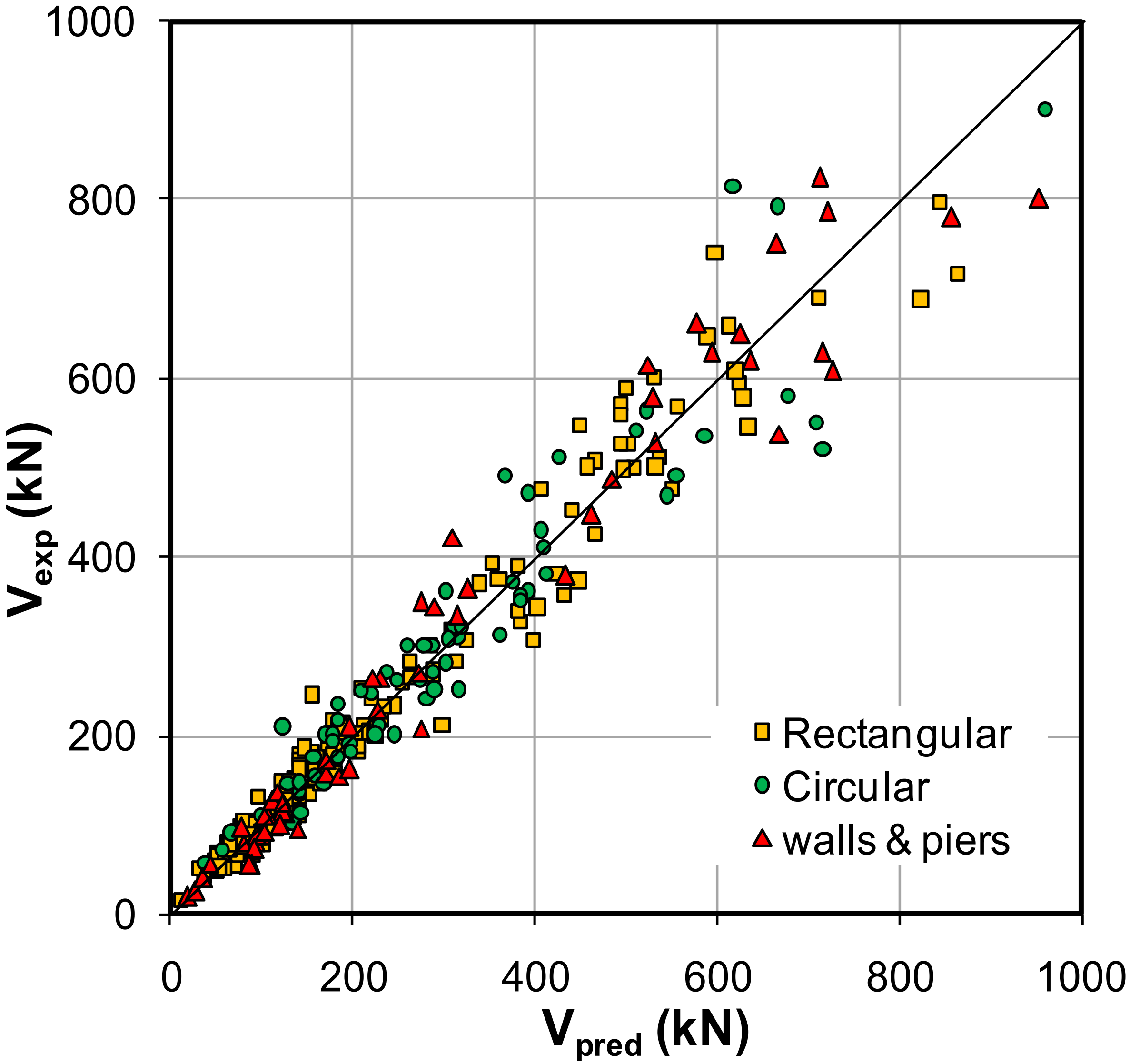
$V_{R,s}=(\pi/4)\rho_w D z f_{yw}$ (z: internal lever arm =); $A_c=(\pi/4)D_o^2$ (D_o: diameter of confined core)

Test-to-prediction: Shear-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≥L_s/h>1.0,

Median test-to-prediction-ratio: 1.00; CoV=17%



Resistance of walls or short columns against shear-compression failure in cyclic loading

Purely empirical models:

- **Walls** before flexural yielding ($\mu_\theta^{pl} = 0$) or after (cyclic $\mu_\theta^{pl} > 0$)

$$V_R =$$

$$0.85\left(1-0.06\min\left(5,\ \mu_\theta^{pl}\right)\right)\left(1+1.8\min\left(0.15,\frac{N}{A_c f_c}\right)\right)\left(1+\frac{\max(1.75,\ 100\rho_{tot})}{4}\right)\left(1-0.2\min\left(2,\ \frac{L_s}{h}\right)\right)\sqrt{f_c(MPa)}\,b_w z$$

- **Short columns** ($L_s/h \leq 2$) after flexural yielding (cyclic $\mu_\theta^{pl} > 0$)

$$V_R =$$

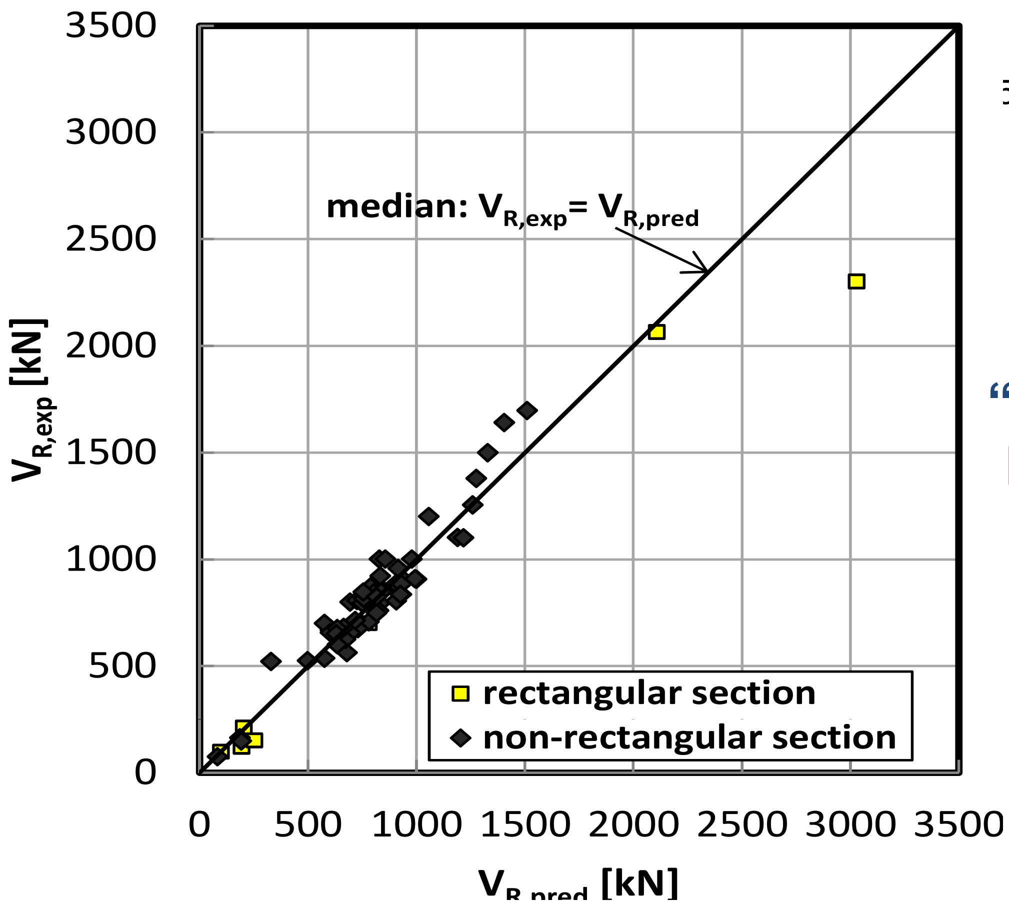
$$\frac{4}{7}\left(1-0.02\min\left(5,\ \mu_\theta^{pl}\right)\right)\left(1+1.35\frac{N}{A_c f_c}\right)\left(1+0.45\cdot 100\rho_{tot}\right)\sqrt{\min\left(f_c,\ 40MPa\right)}\,b_w z\sin 2\delta$$

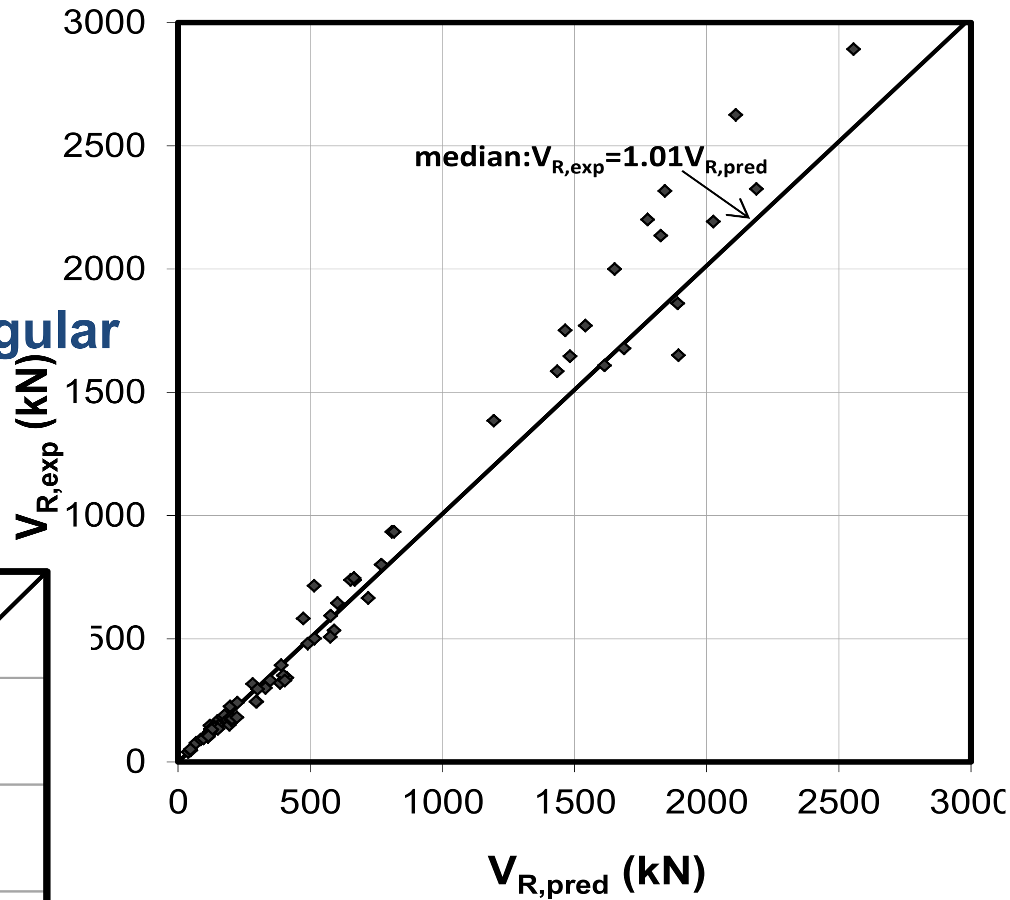
δ: angle between axis and diagonal of column ($\tan\delta = 0.5h/L_s$)

Test-to-prediction: walls or short columns in shear-compression

Walls: 7 rectangular, 55 non-rectangular

Median test-to-prediction ratio: 1.0, CoV=14%





"Short" columns ($L_s/h \leq 2$) ~ 90 tests

Median test-to-prediction ratio: 1.0, CoV=12%

Shear resistance of "squat" walls - Physical model (for $0.25 \le L_s/h \le 1.2$):

$$V_{R,squat,1} = V_c + V_s$$

- **Concrete contribution:**

$$V_c = \underbrace{\left(1+150\rho\right)\left(1-0.725\frac{L_s}{h}\right)}\underbrace{\left(\frac{2}{3}A_c f_{ct}\sqrt{1+\frac{N}{A_c f_{ct}}}\right)}$$

➢ First two terms: empirical corrections for the tension reinforcement ratio, ρ, and the shear-span-ratio.

➢ Last term: shear force which, alongside the axial load N, exhausts, in the principal stress direction, the concrete tensile strength ($f_{ct} = 0.3 f_c^{2/3}$ per *fib* MC2010) in the uncracked section.

- **Contribution of web steel:**

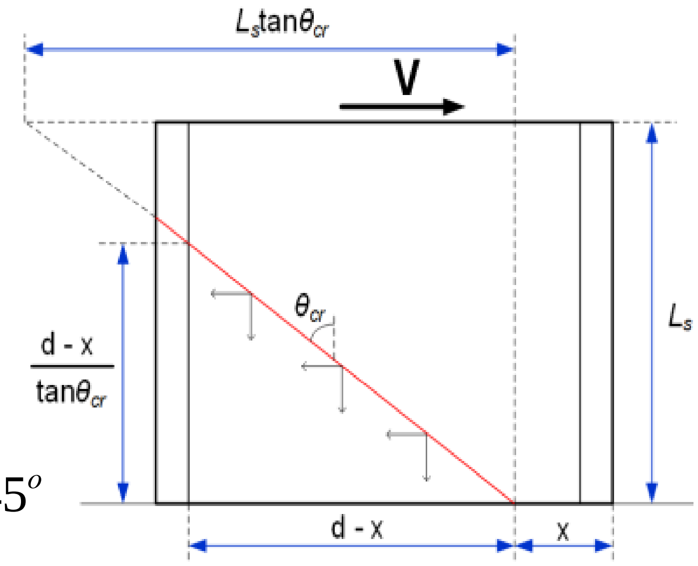
$$V_s = \min\left\{\begin{array}{c} \rho_h b_w \min\left(\left(d-x\right)/\tan\theta_{cr}, L_s\right) f_{yh} \\ \left(\rho_v b_w \min\left(L_s \tan\theta_{cr}, d-x\right) f_{yv} + A_s f_y\right)/\tan\theta_{cr} \end{array}\right\}$$

➢ max horizontal force intercepted by crack;

➢ cap due to vertical web bars crossed by crack.

- **Angle of main diagonal crack, fit to tests:**

$$\theta_{cr}\left(^{o}\right) = 60 - 15\frac{L_s}{h} \ge 45^{o}$$



Shear resistance of "squat" walls - Empirical model
(for $L_s/h \le 1.2$ - modification of Gulec & Whittaker model)

Rectangular walls:

$$V_{R,squat,2} = \frac{0.035 A_{c,eff} f_c + 0.32 A_{sv} f_{yv} + 0.18 A_{sh} f_{yh} + 0.17 A_s f_y + 0.2N}{\sqrt{L_s / h}} \le 1.3 \sqrt{f_c} A_{c,eff}$$

Non-rectangular or barbelled walls:

$$V_{R,squat,2} = \frac{0.04 A_{c,eff} f_c + 0.225 A_{sv} f_{yv} + 0.1 A_{sh} f_{yh} + 0.3 A_s f_y + 0.25N}{\sqrt{L_s / h}} \le 1.3 \sqrt{f_c} A_{c,eff}$$

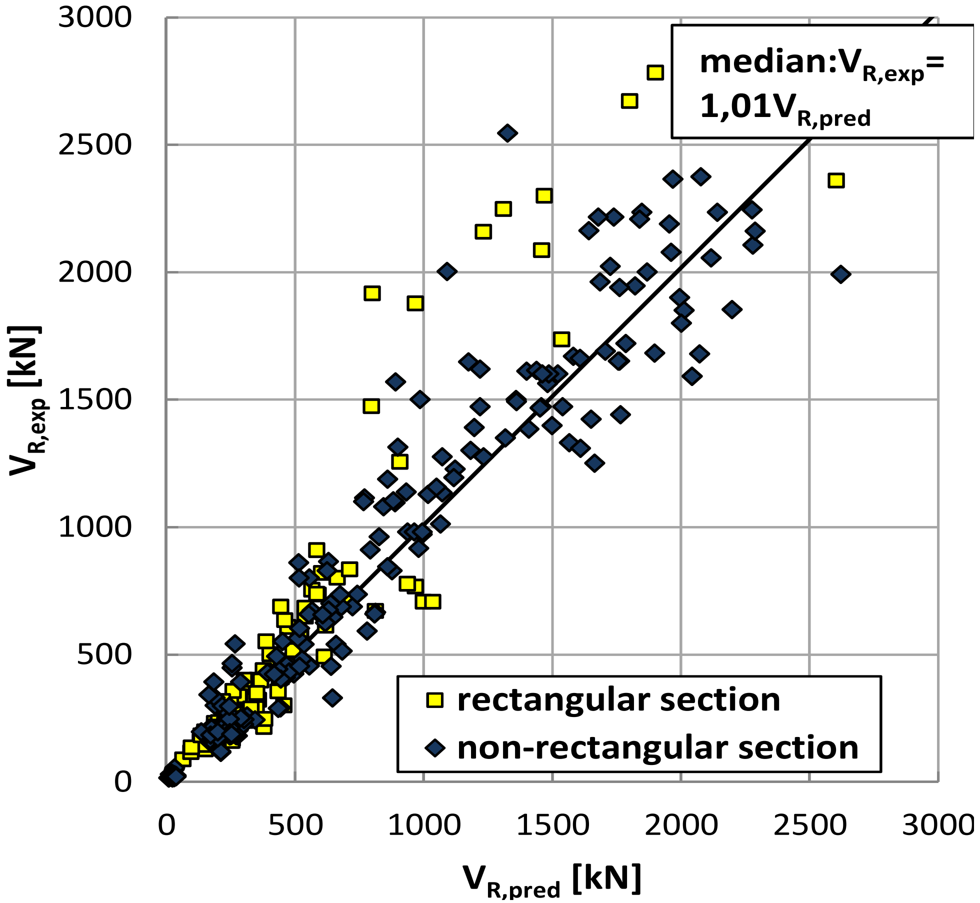
- Rectangular or barbelled walls: A_{eff} = total section area;
- All other sections: A_{eff} = (Area of web) + ∑(flange thickness)x(wall shear span - web thickness)/2.

Test-to-prediction: Squat walls

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

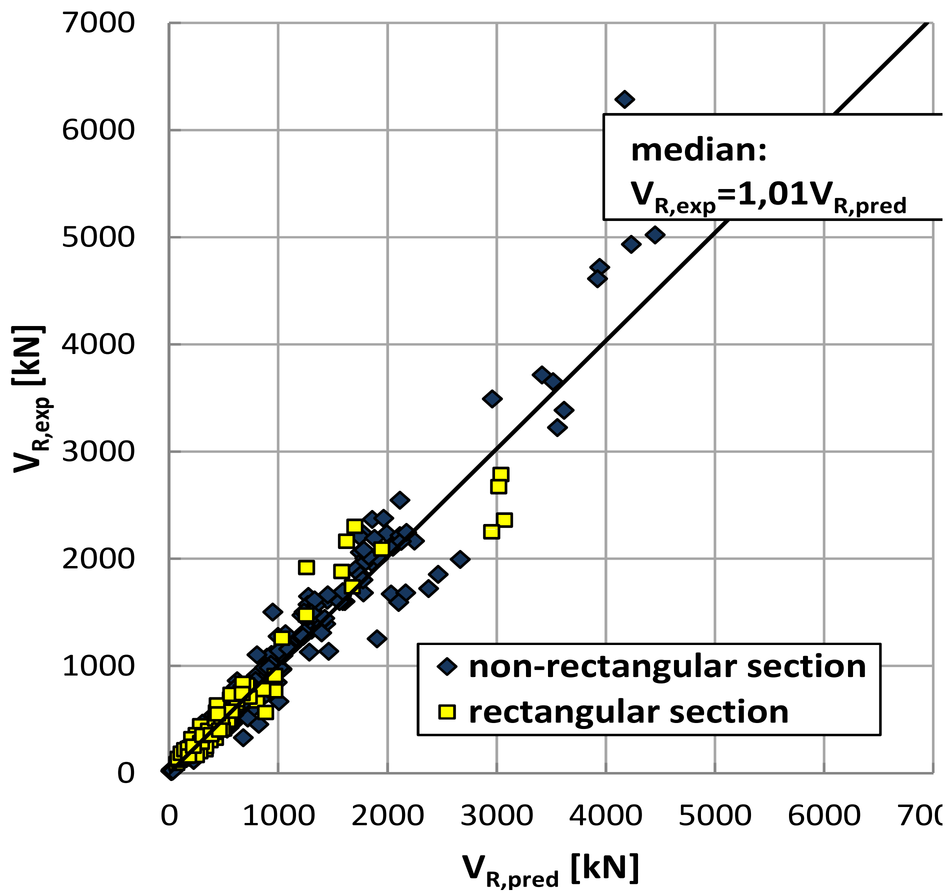
Physical model

CoV=29%



Empirical model

CoV=22%



Sliding shear resistance of RC walls

Modified *fib* MC2010:

$$V_{\mathrm{R,SLS,fibMC2010,mod}} =$$

$$\left(1-0.025\mu_\theta^{\mathrm{pl}}\right)\min\left(\kappa_1\sum A_s f_y\left(\mu\sin\phi+\cos\phi\right)+\mu N+\kappa_2\sum A_s\sqrt{f_c f_y}\sin\phi;\ \beta\min\left(1;\left(\frac{30}{f_c(MPa)}\right)^{\frac{1}{3}}\right)f_c A_c\right)$$

- $\mu_\theta^{\mathrm{pl}}=\mu_\theta$-1: plastic chord rotation ductility factor (new term),
- sum extends over all bars crossing at angle φ to the horizontal the section controlling the yield moment; A_s area of such bars;
- μ: friction coefficient,
- β : compress. strength reduction due to transverse tensile strain
- κ_1, κ_2: coefficients for clamping & dowel action of the reinforcement,
- Monotonic values for smooth interfaces: μ = 0.6, κ_1 = 0.5, κ_2 = 0.7x1.6=1.1, β = 0.11,
- **Only modification of MC2010: First term (reduction due to μ_θ^{pl} > 0)**

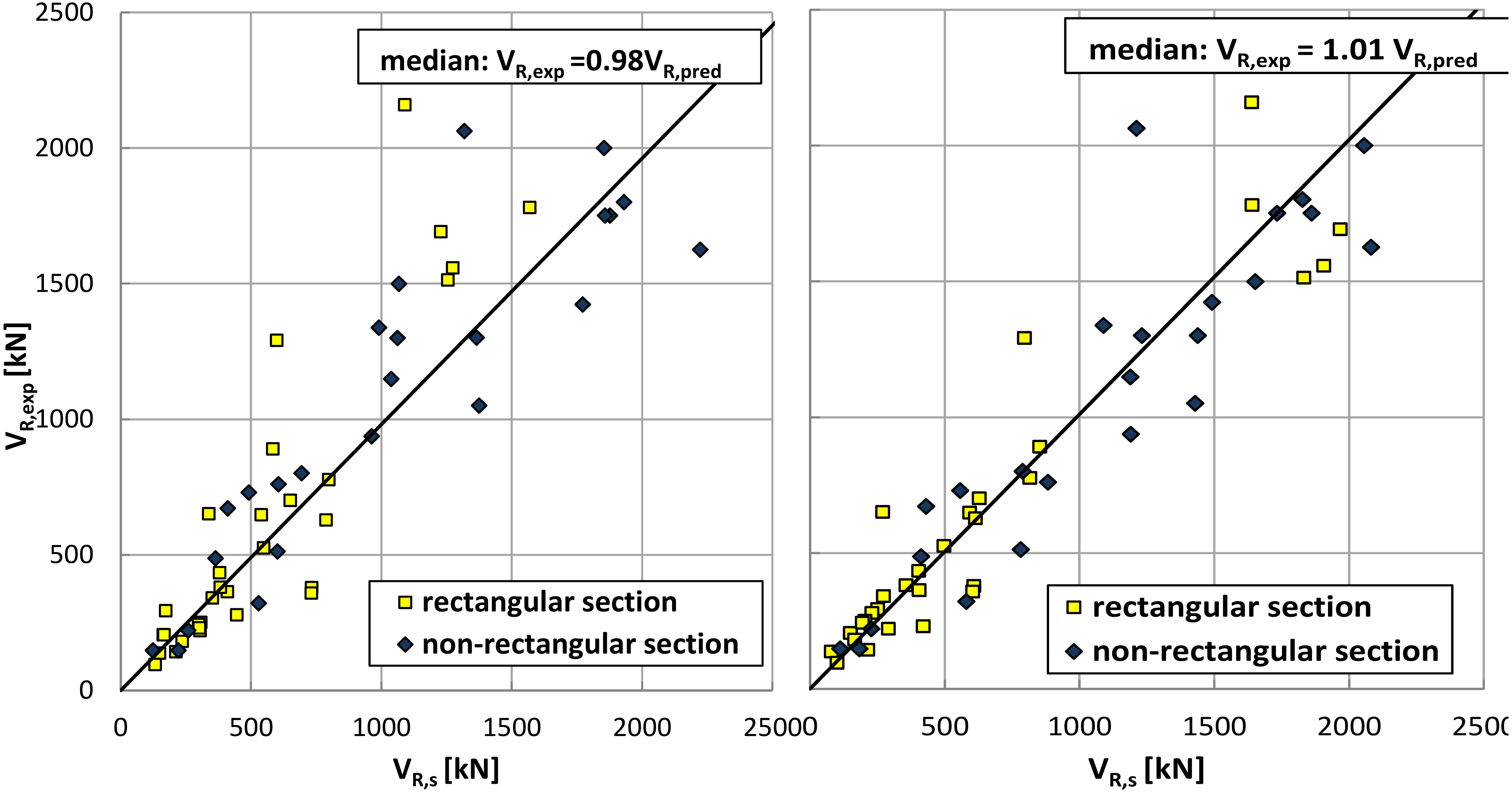
Modified Eurocode 8 (2004):

$$V_{\mathrm{R,SLS,EC8,mod}} =$$

$$\min\left(\mu\left[\left(\sum A_{\mathrm{sv}}f_{\mathrm{yv}}+N\right)\xi+\frac{M_{\mathrm{y}}}{z}\right];0.3f_{\mathrm{c}}A_{\mathrm{compr}}\right)+\sum A_{\mathrm{sv}}\min\left(1.6\sqrt{f_{\mathrm{c}}f_{\mathrm{yv}}};\ f_{\mathrm{yv}}/\sqrt{3}\right)+\sum A_{\mathrm{si}}f_{\mathrm{y}}\cos\phi$$

- A_{sv}: cross-sectional area of vertical web bars,
- A_{si}: area of bars crossing critical section at angle φ to the horizontal,
- A_{compr}: area of compression zone of section,
- z: internal lever arm of section.
- ξ: neutral axis depth normalized to section length, obtained from $\mu_\theta=\theta/\theta_y$ by interpolation

Test-to-prediction: 30 rect. & 25 non-rect. walls in sliding shear



fib MC2010-based, CoV=31%

Eurocode 8-based, CoV=34%

Proposed modification of monotonic shear resistance models in *fib* MC2010 or Eurocode 2 (2018), for use in cyclic loading and Eurocode 8 (2019)

Modifications to Level II or III Approach in *fib* MC2010

$$V_{Rs,d} = \min(\omega_{wd};0.1) b_w z f_{cd} \cot\theta \qquad V_{Rd,\max} = k_\varepsilon \eta_{fc} \min(f_{cd};100MPa) b_w z \frac{\cot\theta}{1+\cot^2\theta}$$

$$k_\varepsilon = \frac{1}{1.75(1.2+55\epsilon_1)}, \eta_{fc} = \left(\frac{30\,MPa}{f_c}\right)^{\frac{1}{3}} \leq 1, \epsilon_1 = \epsilon_x + (\epsilon_x + 0.002)\cot^2\theta$$

or $k_\varepsilon = \dfrac{0.6}{1.2+80\epsilon_1}, \eta_{fc} = \left(\dfrac{40\,MPa}{f_c}\right)^{\frac{1}{3}} \leq 1, \epsilon_1 = \epsilon_x + (\epsilon_x + 0.001)\cot^2\theta$

- **Level II approximation:** $\theta_{min}=20^o \leq \theta \leq \theta_{max}=45^o$

$$V_{Rd} = V_{Rd,s} + V_{Rd.N} \leq V_{Rd.\max} \qquad V_{Rd,N} = \frac{h-x}{L} N_{Ed}$$

- **Level III approximation:** $\theta_{min}=21.8^o \leq \theta \leq \theta_{max}=45^o$

$$V_{Rd} = V_{Rd,s} + V_{Rd.c} + V_{Rd.N} \leq V_{Rd.\max}$$

- **Squat members** (angle of compress. stress field $\theta<\beta=\text{atan}(h/L_s)$)

$$V_{R,sd} = V_{Rc,d} + V_{RN.d} + k_\varepsilon \eta_{fc} f_{cd} b_w z \frac{\cot\theta - \cot\beta}{1+\cot^2\theta} + \frac{A_{sw}}{s} z\sigma_{swd} \cot\beta$$

$$\sigma_{swd} = E_s\left(\cot^2\theta(\varepsilon_x + 0.001) - 0.001\right) \leq f_{ywd}$$

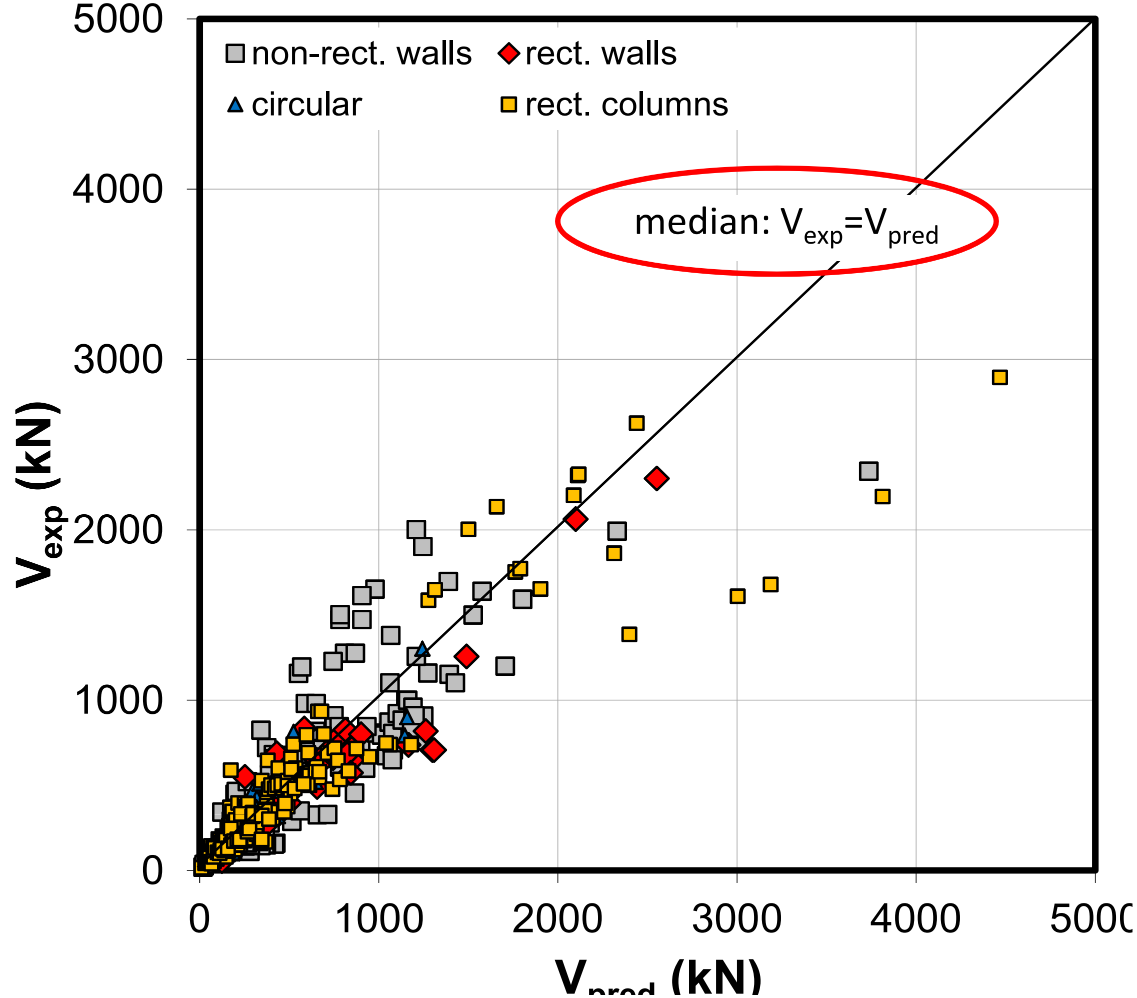
Calculation of ε_x in the plastic hinge

- Calculation of ε_x (inelastic longitudinal strain at section mid-depth) from the curvature of the end section, φ, and the neutral axis depth, x: $\boldsymbol{\varepsilon_x=\varphi(0.5h-x)\ (\leq 0.02)}$
- For given chord-rotation ductility factor, $\mu=\theta/\theta_y$, the curvature, φ, and the neutral axis depth, x, are interpolated between the corresponding values at yielding and (cyclic) ultimate flexural deformation (from section analysis, using the strain criteria for steel and - unconfined or confined - concrete at yielding and ultimate), on the basis of the chord rotations at yielding and (cyclic) ultimate flexural conditions, θ_y and θ_u, determined from the (semi)-empirical formulae.
- As μ increases, ε_x increases, and shear resistance drops.
- So, implicitly, one obtains the dependence of post-yield shear resistance on μ.

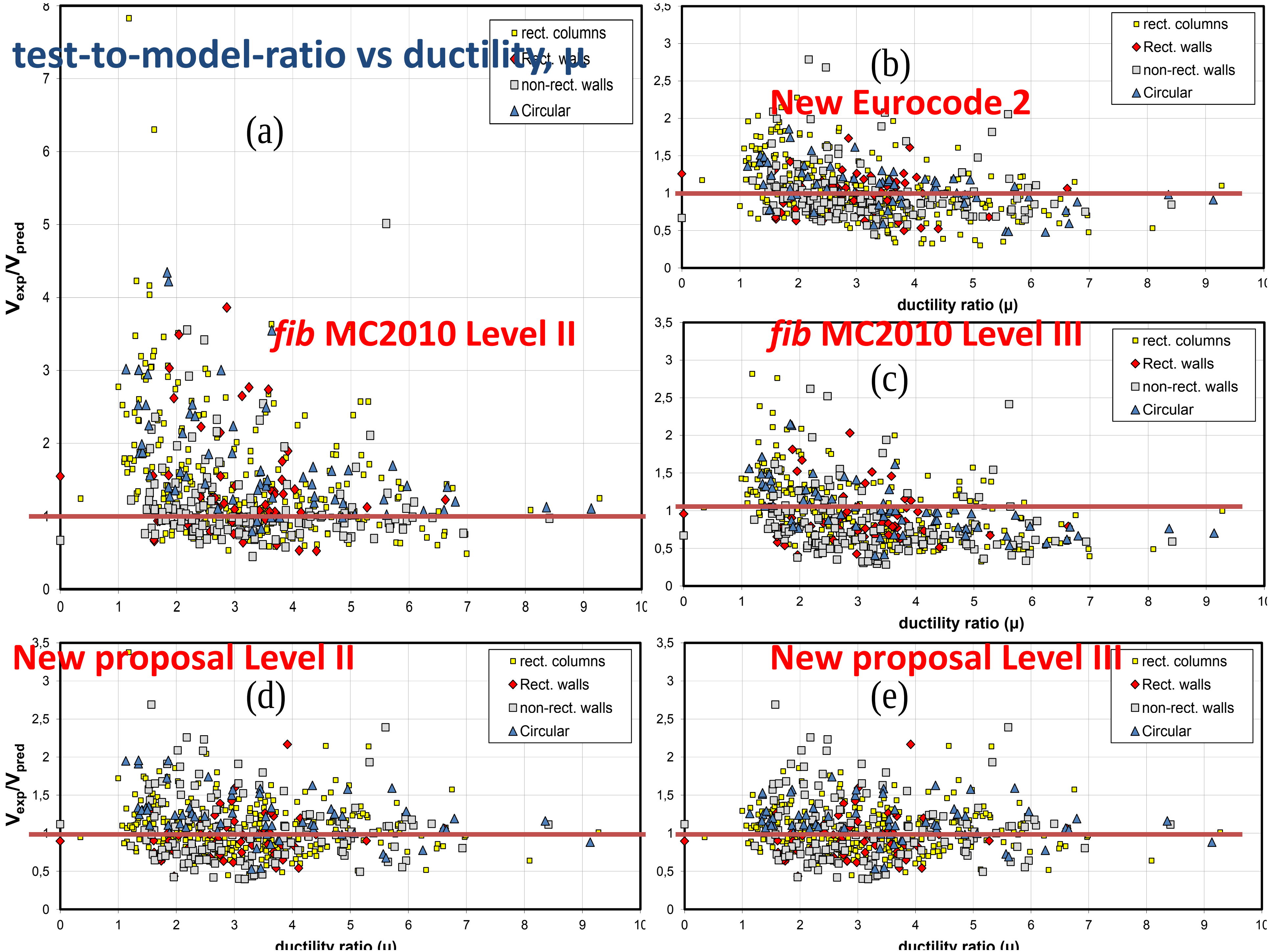
Test-to-(new)model-ratio. Shear failure after flexural yielding

Level II	Rect. columns	Rect. walls	Non-rect. walls	Circular	All
median	1.0	0.9	0.93	1.22	1.00
CoV (%)	31	31	43	26	32
No. of tests	290	60	152	69	571

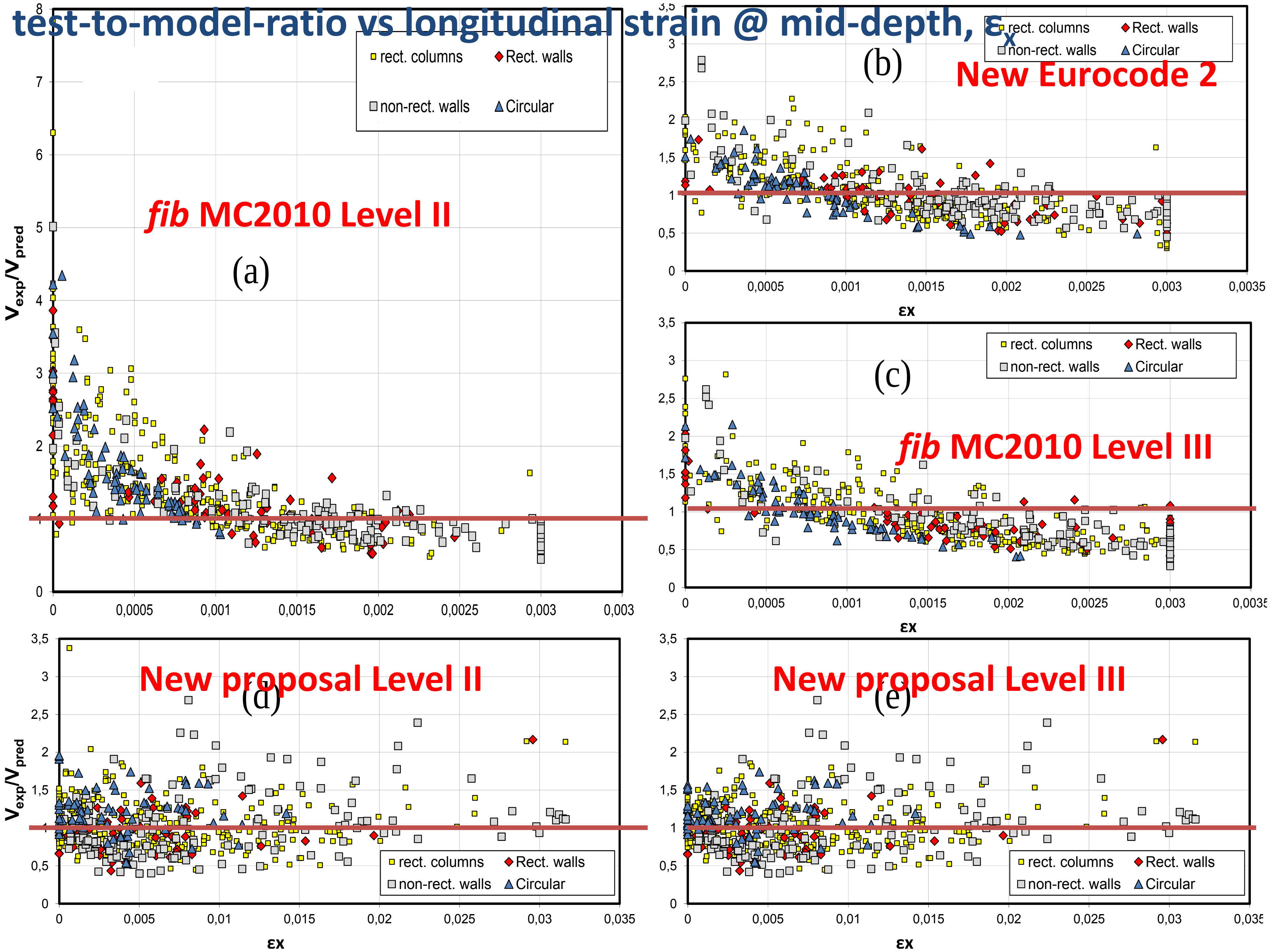
Level III	Rect. columns	Rect. walls	Non-rect. walls	Circular	All
median	1.0	0.9	0.93	1.15	1.00
CoV (%)	27	33	43	21	32
No. of tests	290	60	152	69	571

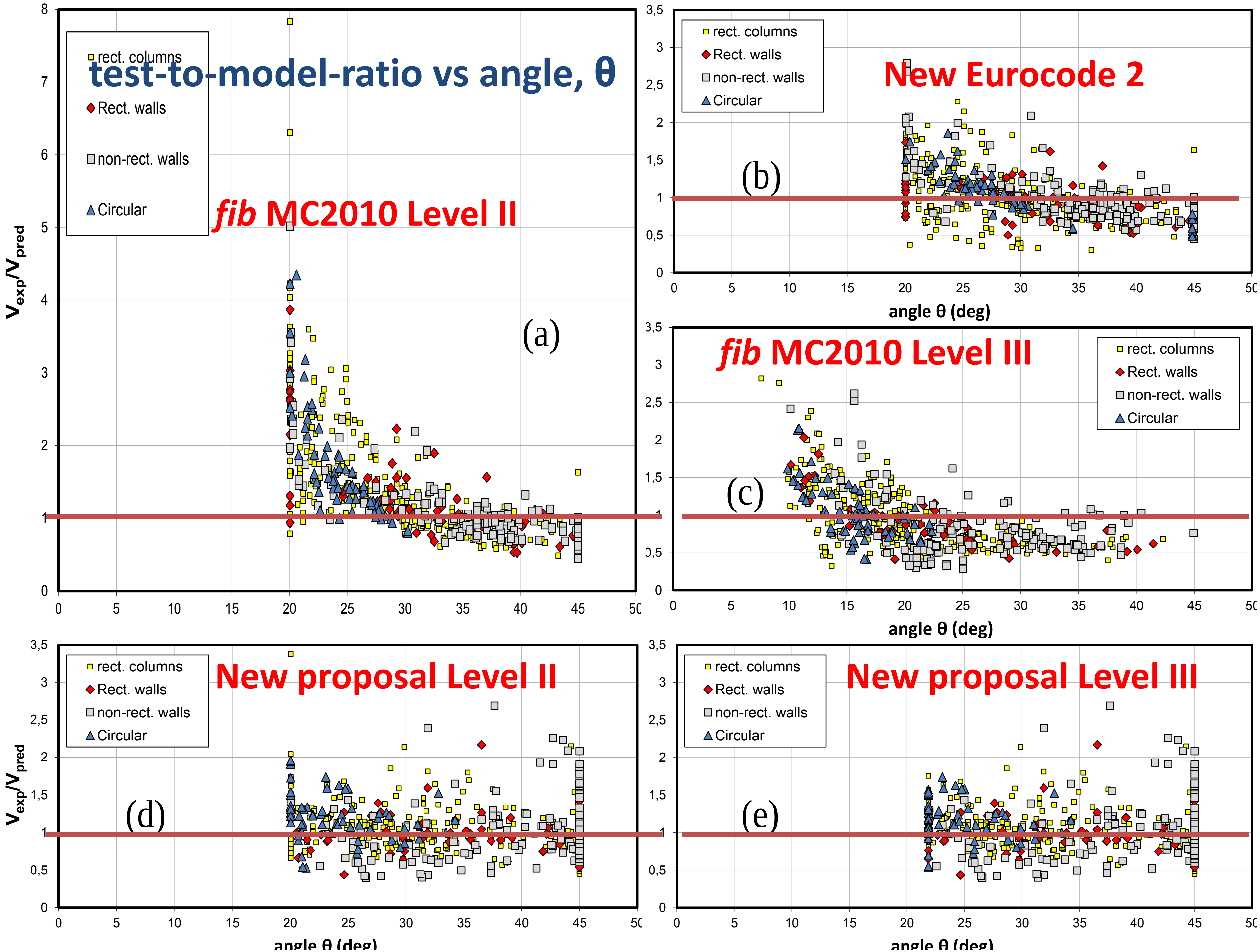


test-to-model-ratio vs ductility, μ

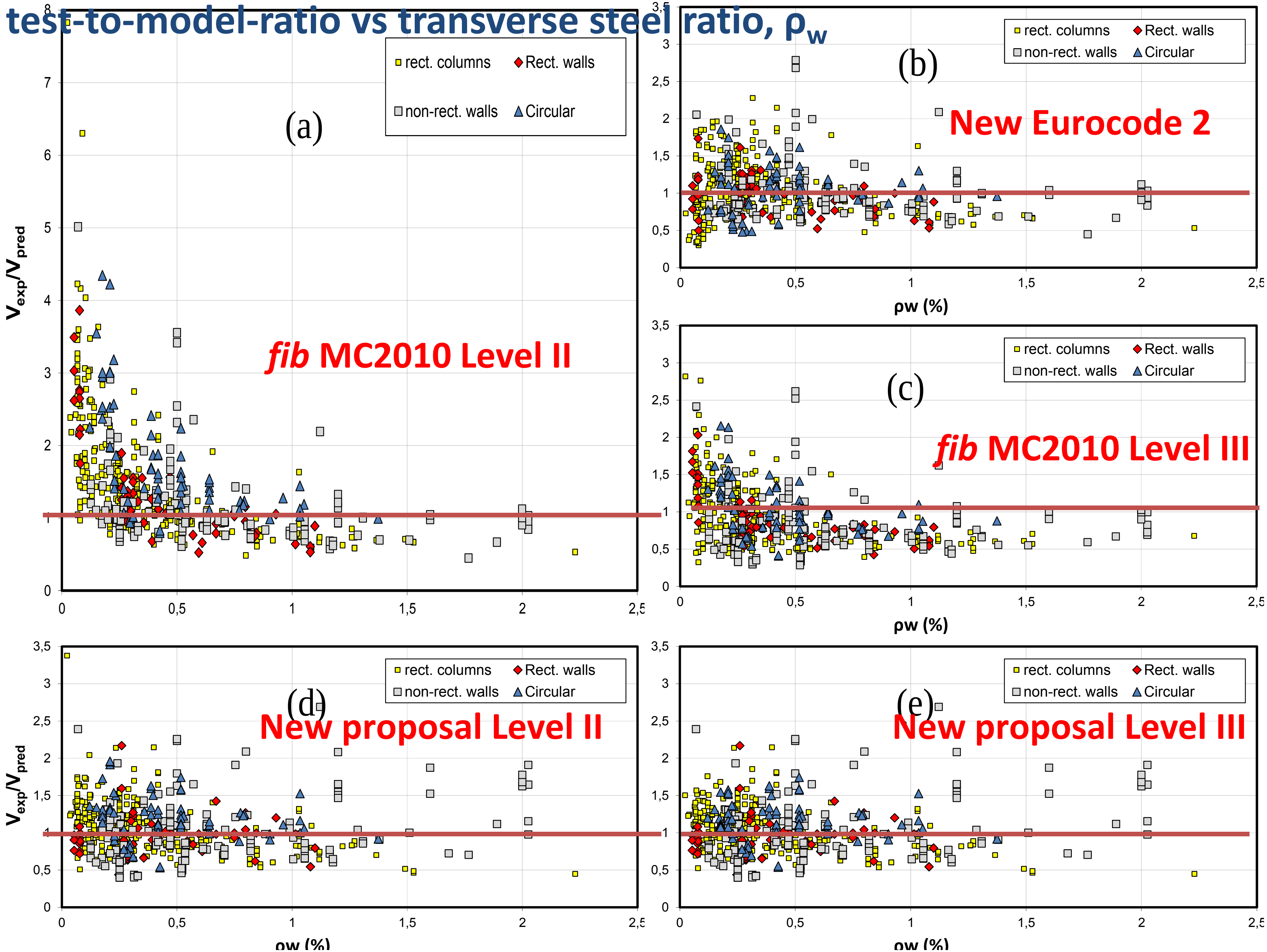


test-to-model-ratio vs longitudinal strain @ mid-depth, ε_x

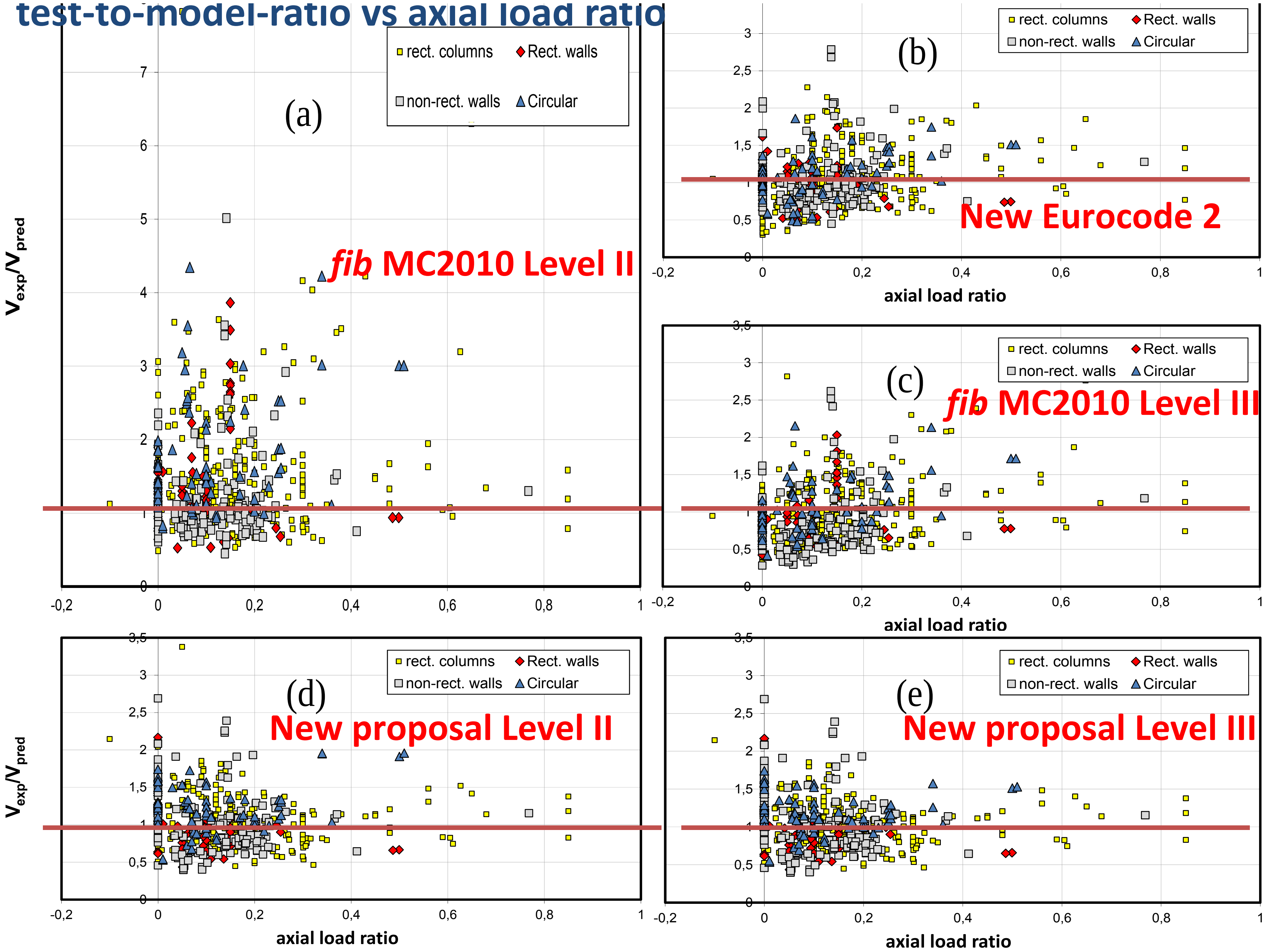



test-to-model-ratio vs angle, θ
fib MC2010 Level II
(a)
New Eurocode 2
(b)
fib MC2010 Level III
(c)
New proposal Level II
(d)
New proposal Level III
(e)
rect. columns
Rect. walls
non-rect. walls
Circular
Vexp/Vpred
angle θ (deg)

test-to-model-ratio vs transverse steel ratio, ρ_w



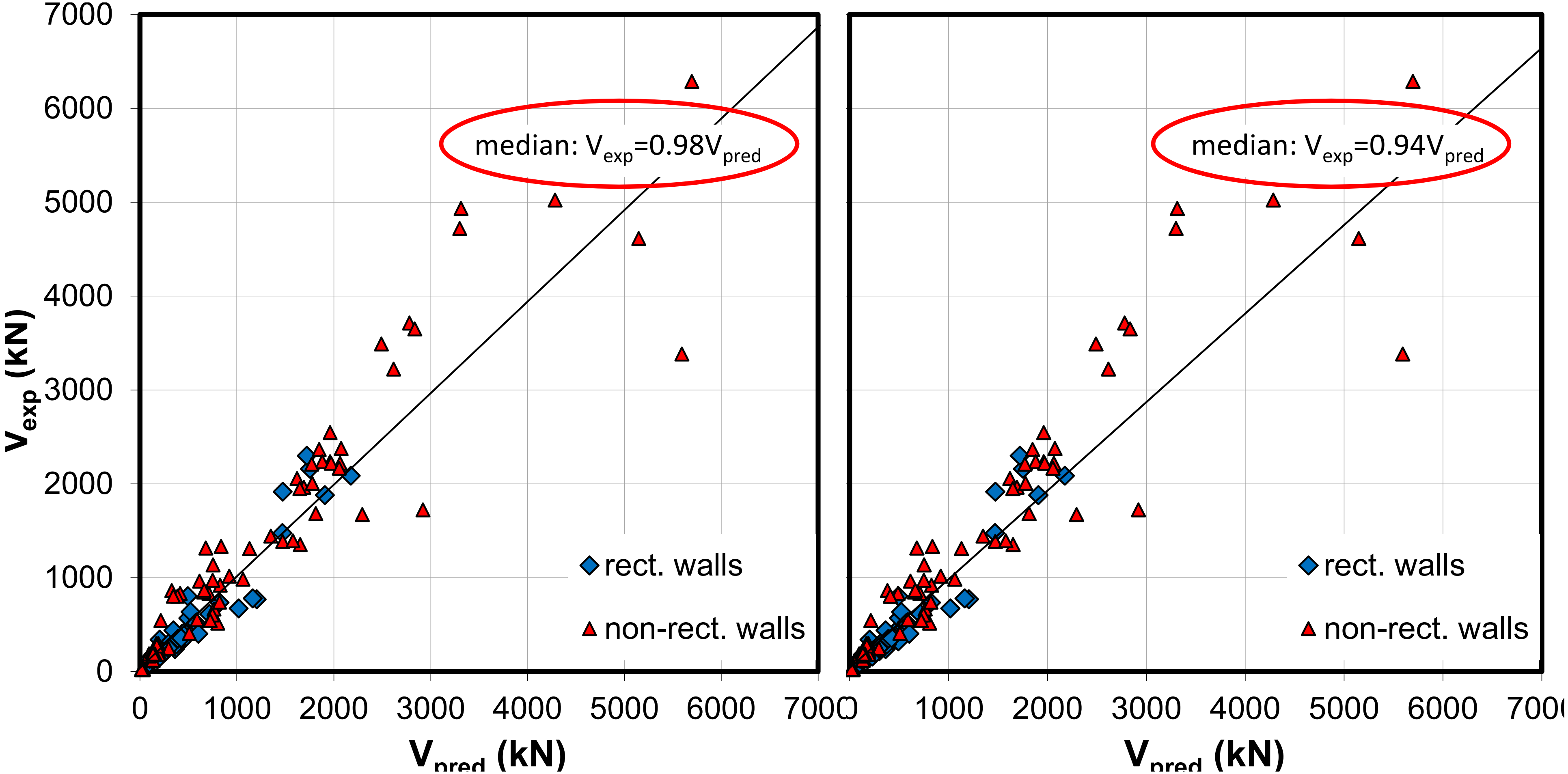
test-to-model-ratio vs axial load ratio



Test-to-(new)model-ratio. Brittle shear failure before flexural yielding

Level II	Rect. walls	Non-rect. walls	All
median	0.90	1.10	0.98
CoV (%)	25	38	35
No. of tests	50	99	149

Level III	Rect. walls	Non-rect. walls	All
median	0.97	1.10	0.94
CoV (%)	25	35	34
No. of tests	50	99	149



Conclusion: Proposed new Level II or III approach for cyclic shear

- **Key element:** inelastic longitudinal strain at mid-depth, ε_x, estimated from curvature and neutral axis depth in the inelastic range (or from chord rotation at member end, by interpolation between yielding and ultimate conditions).
- Cyclic resistance in brittle shear **before flexural yielding:**
  - Worse average agreement with test results than by means of ***fib*** MC2010 Level III approach for monotonic shear;
  - Better mean agreement with tests than via ***fib*** MC2010 Level II for monotonic shear.
- Cyclic resistance in "ductile" shear **after flexural yielding:**
  - Minor change with respect to ***fib*** MC2010 Level II or III approach;
  - Much better agreement with test results, than with ***fib*** MC2010 approaches, both on average and regarding scatter or lack-of-fit.