Questioni legate alla sicurezza nell'evoluzione dell'Eurocodice 8

Paolo Franchin

Università degli studi di Roma La Sapienza

Leader del SC8.T6 e in precedenza membro del SC8.T3 (Revisione degli Eurocodici)

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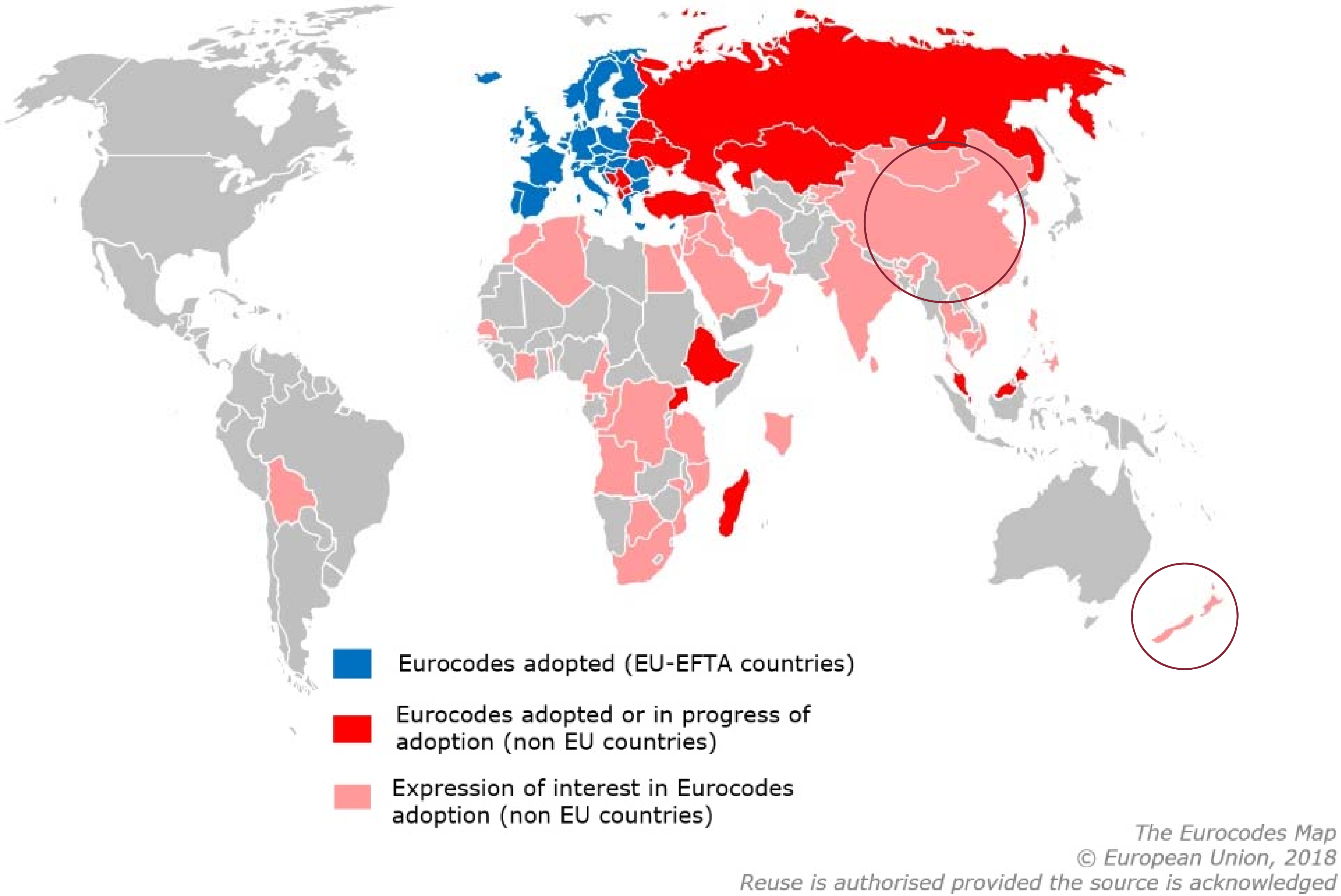
Eurocodes

Market for engineering services 65B €/year – Half a million engineers affected



Eurocodes revision: towards the 2nd generation

CEN Mandate 515 (M515) to NEN (Dutch NSB)

- 10 Eurocodes to revise (EN1990 to 1999)
- 1 Eurocode to introduce (Structural glass)
- A total of more than 60 parts (about 5000 pages, to be reduced)
- 4 Phases
  - Phase 1 concluded
  - Phase 2 concluding
  - Phase 3 started June 2018
  - Phase 4 just started
- Probable release not earlier than 2022
- Expected use over the following 15 years
- NEN administrative management
- CEN/TC250 technical management
  - Project teams work for the relevant TC250 sub-committee
  - Revision of EN1998 is under responisibility of TC250/SC8

Eurocodes revision: goals

M515

- Reduction of NDPs
  - 1° gen. ENs: a few 1000s parameters left to Members' decision (JRC data base)
  - Deviations from recommended values in reality are a minority
  - Legitimate NDPs: only those related to safety (safety = Member decision)
  - Non legitimate NDPs
    - Nothing related to physics can have cross-border differences
    - Alternative methods cannot be NDPs (result of inability to reach consensus back then at the time of drafting)
- Increase Ease-of-use
  - Overall reduction of text size, mainly through elimination of duplications
  - Elimination of inconsistencies
  - Elimination of alternative procedures, unless they are consistent and have different (clearly specified) scopes of application
    - e.g. a simplified procedure that is more conservative and works for a subset of cases
  - Consistent table of contents across different Ens and parts
  - Adoption of an electronic format (XML)
- Extension of scope
  - Often for new structural types

Eurocodes revision: EN1998

M515

- Phase 1
  - PT1 Labbè, …
  - PT3 Kappos, Franchin, …
- Phase 2
  - PT2
  - PT4
- Phase 3
  - PT5
  - PT6 Franchin, Labbè,…

EN1998

- Part 1 General and (New) Buildings
  - General (Safety, Design of new buildings – material-independent clauses)
  - Material-dependent clauses
- Part 2 Bridges (new)
- Part 3 Buildings (Existing: assesment & retrofit)
- Part 4 Silos, tanks, pipelines
- Part 5 Geotechnical aspects
- Part 6 Towers, masts and chimneys

Safety related matters (this presentation):

- Partial factors format introduced in Part 3
- Quantitative definition of resistance at significant damage/life safety
- Need for a single shear strength model for assessment and design

Partial factors in Part 3

Partial factors in Part 3 (Seismic assessment)

Partial factors on model parameters *vs* global partial factor on resistance

EN1998-3:2005 (1st generation):

- Verification: $E_d \leq R_d = R(p_i/\gamma_i)/\gamma_{el}$
  - Action effect E_d from analysis of a model: **mean** material properties μ
    - Ductile: from analysis
    - Brittle: from analysis, capped by capacity design value computed with $\mu \times CF$
  - Resistance R_d
    - Ductile (flexure with or w/o axial): μ/CF
    - Brittle (shear): $\mu/(CF \times \gamma_m)$

Comments:

- Confidence Factor depends on Knowledge Level
  - KL value is unique, over the structure
  - KL depends on Geometry, Details and Materials, but affects only Materials
  - Inadequate link between gathered information and verifications
- Four different values of material properties $(\mu, \frac{\mu}{CF}, \frac{\mu}{CF\gamma_m}, \mu \times CF)$
- Fractile of R uncontrolled and inconsistent across verifications

Partial factors in Part 3 (Seismic assessment)

Partial factors on model parameters *vs* global partial factor on resistance

prEN1998-3:2018 (2nd generation):

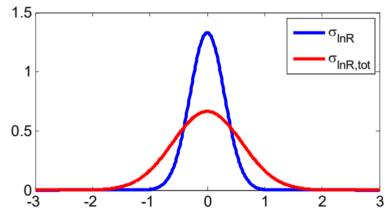
- Confidence factor and single KL disposed of
  - Knowledge does not increase homogeneously in G, M and D
    - Three distinct KLs have been introduced KLG, KLD, KLM
  - Non-critical/low stress areas: penalizing lower knowledge non influential
    - KLG, KLD and KLM can vary within the structure
    - Achieve higher KL only in important areas (conditional on preliminary analysis)
  - Uncertainty: not just material properties, but geometry and details
    - Now all linked to the verification inequality
- Material properties: μ now used for both model and resistance (or demand on brittle mechanisms)
  - Ease of use, lower chance of errors
  - Consistent shear demand evaluation between linear and nonlinear analyses
- Verification: $\gamma_{Sd} E_k \leq R_d = \dfrac{\hat{R}}{\gamma_{Rd}}$

Partial factors in Part 3 (Seismic assessment)

Partial factors on model parameters *vs* global partial factor on resistance

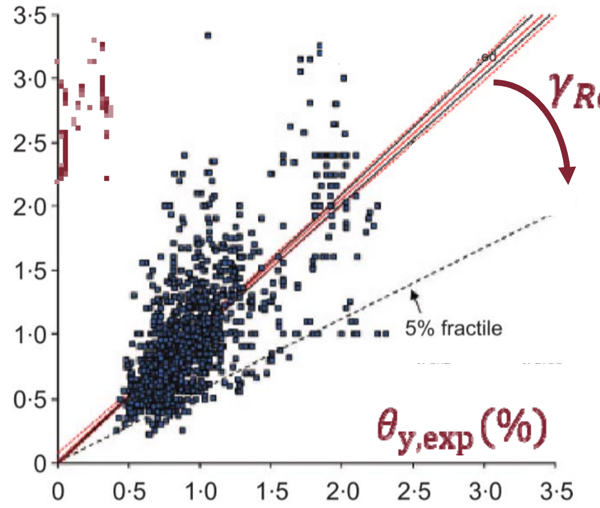
In general, resistance models can be put in the form: $R = R(\boldsymbol{x})\epsilon_R$

- $R(\boldsymbol{x})$ predicts median resistance (unbiased model)
  - $\boldsymbol{x}$: collects input variables (both random and deterministic)
- ϵ_R: model error (uncertainty 'orthogonal' to $\boldsymbol{x}$)
  - unit median
  - σ_{lnR}, typical values: 0.2-0.3 for strength, 0.3-0.7 for deformation
- Distribution of R: function of joint distribution of $\boldsymbol{x}$ and of that of ϵ_R
  - Total log-standard deviation: $\sigma_{lnR,tot}$ function of σ_{lnR} and covariance of $\boldsymbol{x}$
- If R assumed LN with median $R(\boldsymbol{x})$
  a simple expression relates R_k and $\sigma_{lnR,tot}$:



$$R_k = e^{\mu_{\ln R} + \kappa\sigma_{\ln R,tot}} = e^{\mu_{\ln R}} e^{\kappa\sigma_{\ln R,tot}} = \frac{\bar{R}}{\gamma_{Rd}}$$

$$\gamma_{Rd} = \exp(-\kappa\sigma_{\ln R,tot})$$



Partial factors in Part 3 (Seismic assessment)

Evaluation of the new partial factor

The partial factor γ_{Rd} is known once $\sigma_{\ln R,tot}$ is known

$$\ln R = \ln \hat{R}(\mathbf{x}) + \tilde{\varepsilon}_R \cong \ln \hat{R}(\hat{\mathbf{x}}) + \sum_i \left.\frac{\partial \ln \hat{R}(\mathbf{x})}{\partial \ln x_i}\right|_{\hat{\mathbf{x}}} \left(\ln x_i - \mu_{\ln x_i}\right) + \tilde{\varepsilon}_R =$$

$$= \ln \hat{R}(\hat{\mathbf{x}}) + \sum_i \frac{1}{\hat{R}(\hat{\mathbf{x}})} \left.\frac{\partial \hat{R}(\mathbf{x})}{\partial \ln x_i}\right|_{\hat{\mathbf{x}}} \left(\ln x_i - \mu_{\ln x_i}\right) + \tilde{\varepsilon}_R =$$

$$= \underbrace{\ln \hat{R}(\hat{\mathbf{x}})}_{\substack{\text{median } R \text{ evaluted} \\ \text{in the median of } \mathbf{x}}} + \underbrace{\sum_i \underbrace{\frac{\hat{x}_i}{\hat{R}(\hat{\mathbf{x}})}}_{\substack{\text{correction} \\ \text{for the logarithm}}} \underbrace{\left.\frac{\partial \hat{R}(\mathbf{x})}{\partial x_i}\right|_{\hat{\mathbf{x}}}}_{\substack{\text{sensitivity of} \\ \text{median R to xi}}} \underbrace{\tilde{\varepsilon}_i}_{\substack{\text{deviation} \\ \text{of xi from} \\ \text{its median}}}}_{\substack{\text{(linear) effect of deviations} \\ \text{of } \mathbf{x} \text{ from its median}}} + \underbrace{\tilde{\varepsilon}_R}_{\substack{\text{deviation from} \\ \text{the median R} \\ \text{evaluated in} \\ \text{median } \mathbf{x}}} =$$

$$= \ln \hat{R}(\hat{\mathbf{x}}) + \sum_i c_i \tilde{\varepsilon}_i + \tilde{\varepsilon}_R \rightarrow \sigma_{\ln R,tot} = \sqrt{\sigma^2_{\ln R} + \sum_i c_i^2 s^2_{\ln Xi}}$$

$$\sigma_{\ln R,tot,KL} = \sqrt{\sigma^2_{\ln R} + \sum_i c_i^2 \left(CF_i s_{\ln Xi}\right)^2} \rightarrow \gamma_{Rd,KL} = \exp\left(-\kappa \sqrt{\sigma^2_{\ln R} + \sum_i c_i CF_i s^2_{\ln Xi}}\right)$$

Final format. Linearization validated through MC simulation

Franchin. 2018. "A general model of resistance partial factors for seismic assessment and retrofit." 16th European Conference on Earthquake Engineering, Thessaloniki.

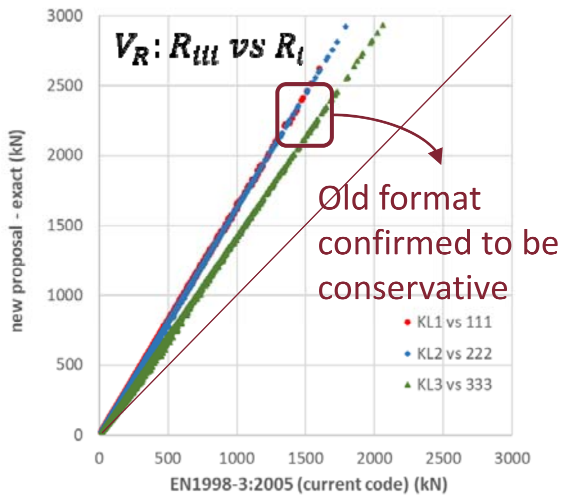
Partial factors in Part 3 (Seismic assessment)

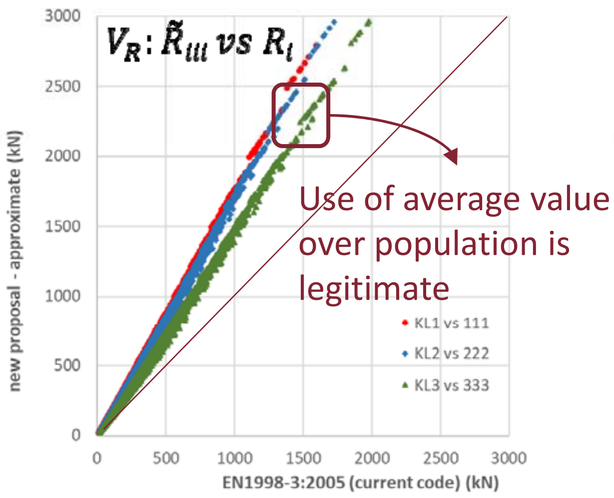
Evaluation of the new partial factor

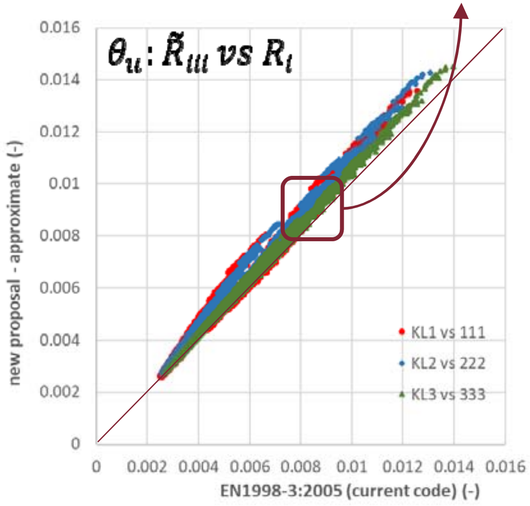
Evaluation of $\gamma_{Rd,KL} = \exp\left(-\kappa\sqrt{\sigma^2_{\ln R} + \sum_i c_i CF_i s^2_{\ln X_i}}\right)$. Two options:

- Let the user do it for each resistance model and each member…
- Compute values for all resistance models over a large population of structural members and provide average values tabulated as a function of KLG, KLM, KLD
- Calibration example: shear strength of RC columns
  - Model (presented later) depends on f_c, f_y, h, b_w, L_V, ρ_w and ν → 3888 cases considered, values computed:
    - R_i = EN1998-3:2005 resistance (i.e. with γ_c, γ_s and γ_{el})
    - $\hat{R}$ = Median resistance (i.e. w/o partial factors)
    - R_{ijk} = exact resistance, i.e. with case-specific γ_{Rd}
    - $\tilde{R}_{ijk}$ = approximate resistance, i.e. with average γ_{Rd} over all 3888 cases

New format gives same safety as old one, but fractile is known (16%)



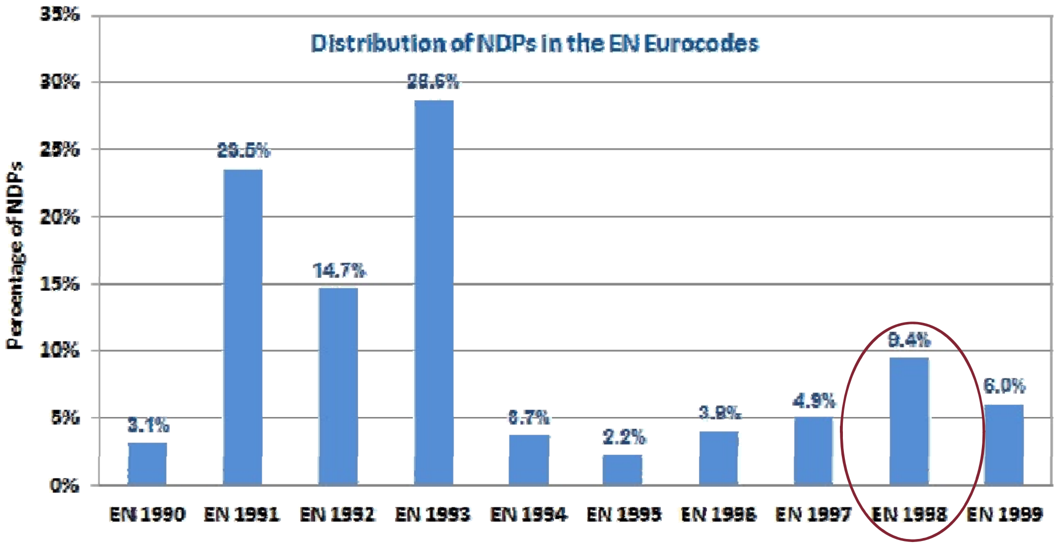




Partial factors in Part 3 (Seismic assessment)

How to make it an NDP

- Partial factors are legitimate NDPs
- If a partial factor is introduced for each resistance model, number of NDPs will increase
- NDPs are not really a criticality for EN1998, but increasing NDPs is contrary to M515



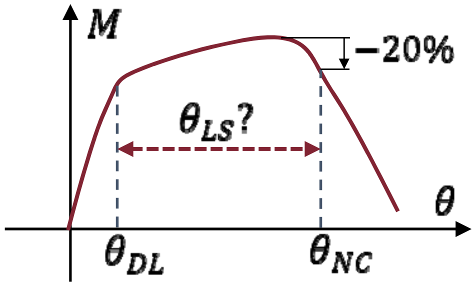
- A way to introduce a single NDP has been devised, by making the fractile *k* of resistance, for all resistance models, the NDP. An approximate equation allows to pass from one fractile of resistance to another:

$$\gamma_{Rd2} = (\gamma_{Rd1})^{\kappa_2/\kappa_1}$$

k=16%

Deformation thresholds at life safety limit state

Definition of resistance at significant damage LS



- EN1998-1 defines four LSs:
  - Two SLS: Operational and **Damage**
  - Two ULS: Life safety and **Near collapse**
- Assessment (EN1998-3) is carried out with ref.to Near collapse
  - Near collapse, like Damage (e.g. yield): clear physical meaning
- Design is carried out with reference to Life safety (proxy for NC)
  - Life safety: quantitative definition is less clear-cut

- Problem 1: cross-approach consistency (values of q vs values of θ)
  - Force-based approach (mainly used in design)
    - linear analysis with a design spectrum = elastic acceleration spectrum reduced by a factor q function of overstrength and ductility
  - Displacement-based approach (mainly used in assessment, but allowed for design)
    - Inelastic displacement spectrum with (preferably) nonlinear analysis methods
  - Displacement thresholds should be consistent with values of q:
    - Same safety of force-based and displacement-based designed structures
    - Same safety of existing and new structures

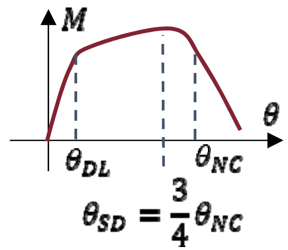
Adeguamento *vs* miglioramento:
ma l'adeguamento è adeguato?

- Problem 2: cross-material consistency (values of q for RC, steel, timber,...)
  - Whatever the approach (force- or displacement-based), same safety should be guaranteed for different structural materials

Definition of resistance at significant damage LS

Key changes/Topics currently under discussion with SC8

- Problem 1: deformation thresholds are not consistent with values of q
  - Current Part 3 defines $\theta_{SD} = {}^{3}/_{4}\,\theta_{NC}$:
    - Ductile (new) structures: too much damage (not consistent with q).
    - Brittle (existing) structures: may be lower than θ_y



- Possible alternatives:
  - Smaller fraction of NC but larger than yield: $\theta_{SD} = {}^{2}/_{3}\,\theta_{NC} > \theta_y$
    - Even ${}^{2}/_{3}\,\theta_{NC}$ for ductile structures may be too much damage…
  - In between yield and ultimate: $\theta_{SD} = \theta_y + \alpha(\theta_u - \theta_y)$ e.g. with $\alpha = 0.3 \div 0.5$
  - Multiple of yield (i.e. ductility=damage) $\theta_{SD} = \mu_{SD}\theta_y$ e.g. with $\mu_{SD} = 3$
    - μ_{SD} would depend on the material, e.g. may be 3 or 4 for RC, and lower for masonry

Consensus between SC8.T6 and SC8.T2

- Need a model for the elastic limit deformation θ_y for all materials

- The issue (whether the format is accepted and, then, what values of α should be adopted) will be discussed in the next SC8 meeting in Ljubljana

Definition of resistance at significant damage LS

(Global) Collapse risk is not uniform across structural materials

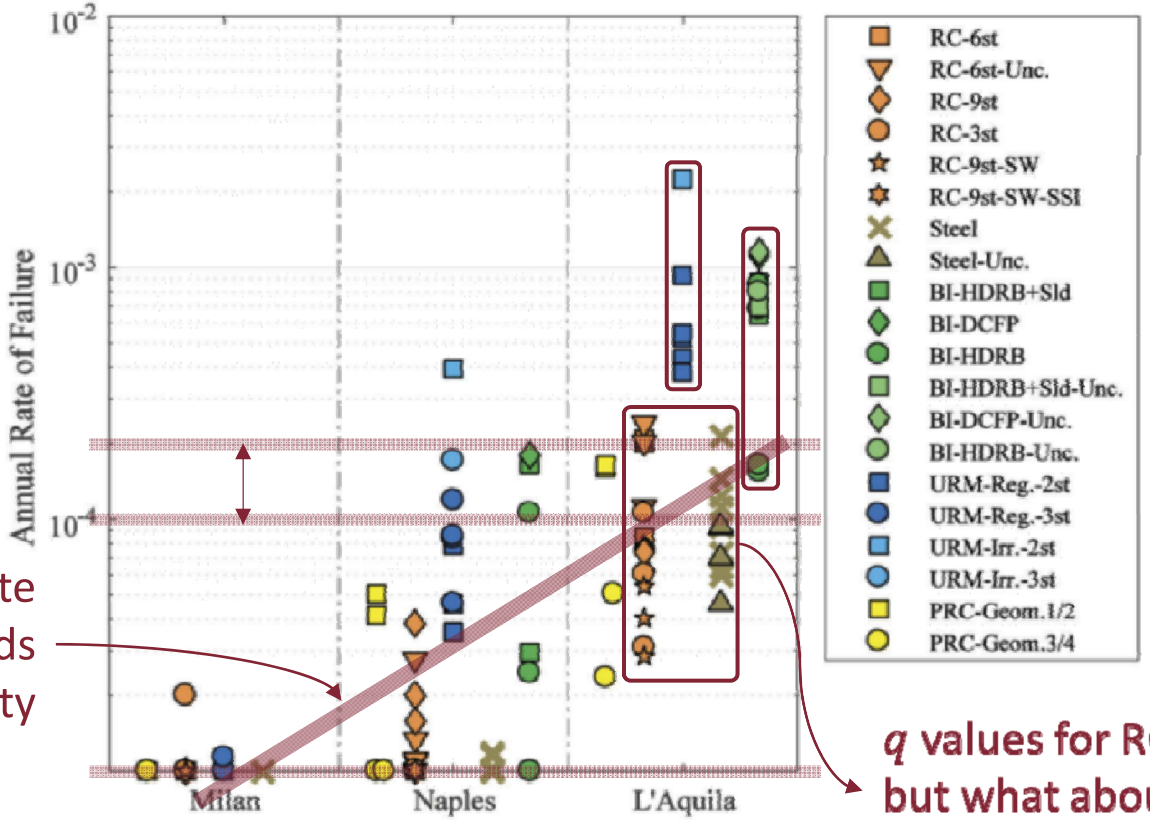


Figure 11. Global collapse failure rates (soil C-type).

Results from the RINTC project 2016-2018 https://goo.gl/KX5qks

q values for RC and Steel may be ok, but what about other materials?

(In any case q= 3.5 DC low for RC...)

- CC2, permanent and variable loads: target reliability is $10^{-5}/year$.
- Seismic action: target reliabitliy not declared in EN1998.
- Values under discussion (Annex F of new Part 1) between $10^{-4}/year$ and $2 \times 10^{-4}/year$
- These issues will be also discussed in the next SC8 meeting in Ljubljana

Shear strength model

Need for a single shear strength model

Shear design

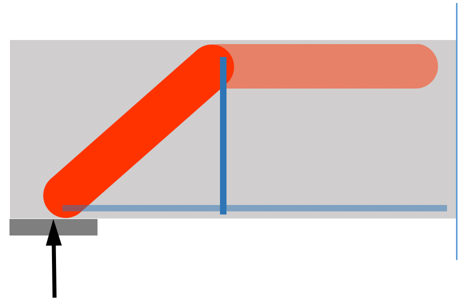
- Shear design of RC bridge piers was criticized as conservative
- Shear strength is actually an issue beyond RC piers. Currently:
  - EN1998-1 adopts formulas from EN1992
  - EN1998-3 has its own "seismic" formulas
- Non-seismic shear strength models:
  - Target non-yielding members subjected to non cyclic loads→beams, slabs
  - Systematically underpredict strength of members in seismic situations
- Seismic shear strength models:
  - Target members deforming cyclically in inelastic range and mostly subjected to axial load →columns, walls
- Again, a problem of consistency
  - Possible solutions:
    1. Adopt *fib* Model code models and their modification for seismic situations. This would bring everything under the same "theoretical umbrella"
    2. Adopt also for design (EN1998-1) the model used for assessment (in EN1998-3). It would be inconsistent with EN1992, but correct.

Need for a single shear strength model

Shear strength according to *fib* MC2010:

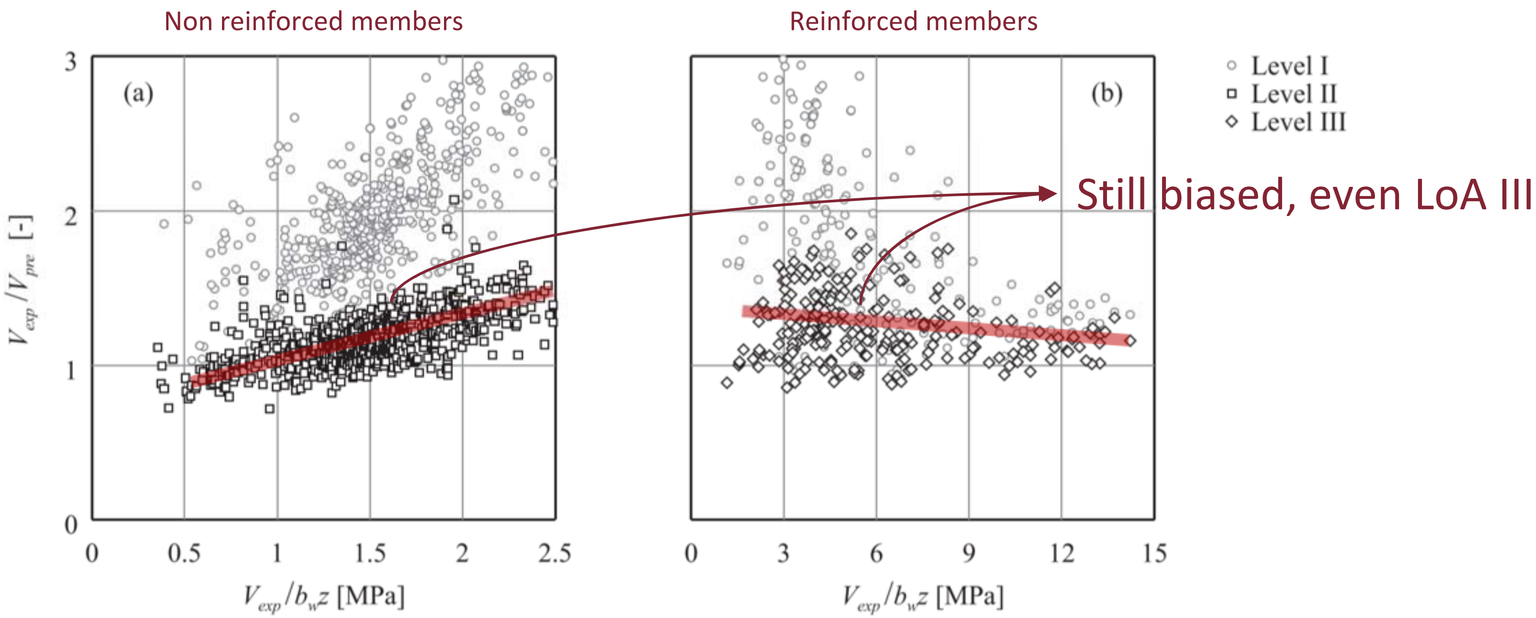
$$V_R = \min \begin{cases} V_{R,s}(\theta(\epsilon)) + \sout{V_{R,c}(\theta(\epsilon))} \\ V_{R,max}(\theta(\epsilon)) \end{cases}$$

shear – tension failure
shear – compression failure



Level of approximation (LoA) approach:

- Higher level, more refined model is formulated first
- Lower level models derived from higher level one with conservative simplifications
- Scope of application different: low LoA for prelim. design, high LoA for assessment

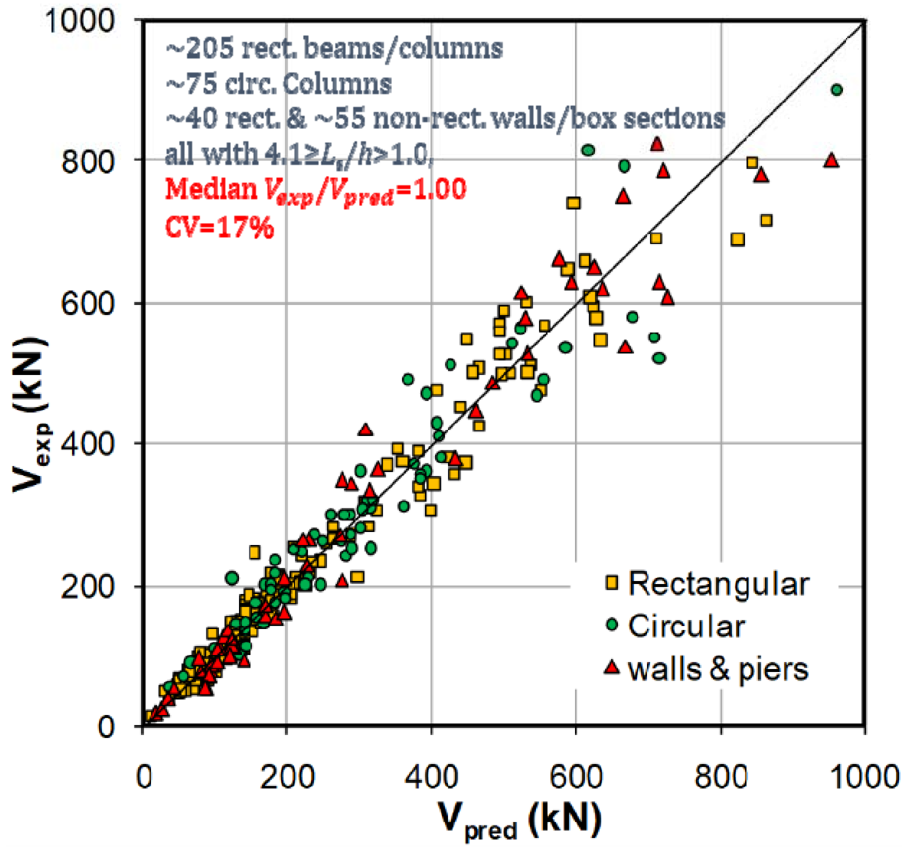


Sigrist, Bentz, Fernández Ruiz, Foster. 2013. “Background to the Fib Model Code 2010 Shear Provisions - Part I: Beams and Slabs.” Structural Concrete 14 (3): 195–203.

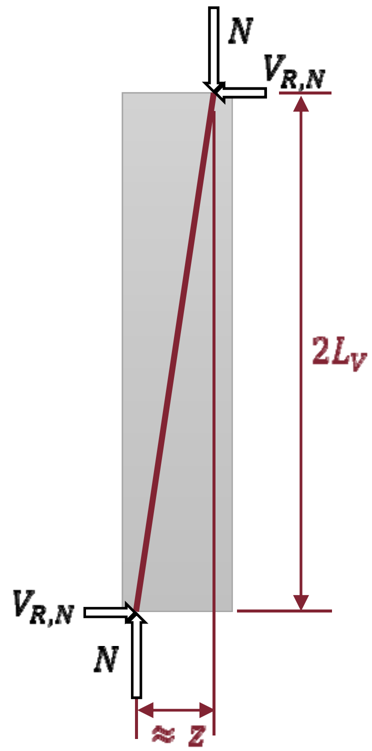
Need for a single shear strength model

Semi-empirical models in EN 1998-3:2005 and in prEN 1998-3:2018

$$V_R = (V_{R,s} + V_{R,c})\left(1 - 0.005\min\left(5; \mu_\theta^{pl}\right)\right) + V_{R,N} \le V_{R,max}$$



$$V_{R,N} 2L_V = Nz \rightarrow V_{Rd,N} = \frac{Nz}{2L_V}$$



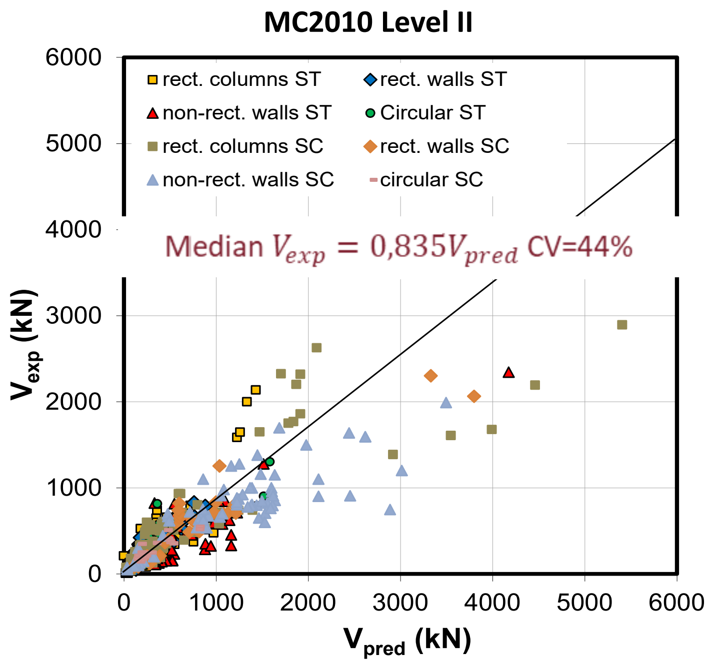
Biskinis, Roupakias, Fardis (2004) Degradation of shear strength of RC members with inelastic cyclic displacements ACI Struct. J. 101(6): 773-783
Grammatikou, Biskinis, Fardis (2015) Strength, deformation capacity and failure modes of RC walls under cyclic loading Bull. Earthq. Eng. 13: 3277-3300

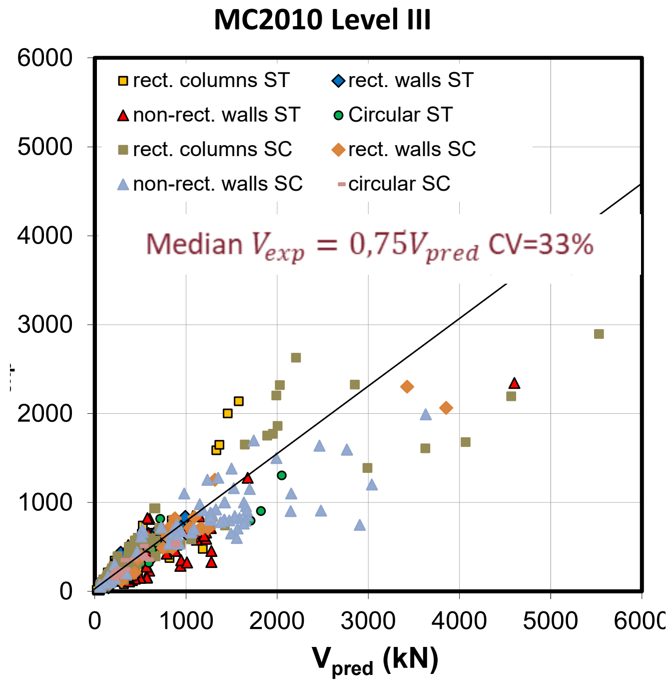
Courtesy of Michael Fardis

Need for a single shear strength model

fib MC2010 shear strength model: poor 'seismic' performance

$$V_R = V_{R,s} + V_{R,c} \leq V_{R,max}$$





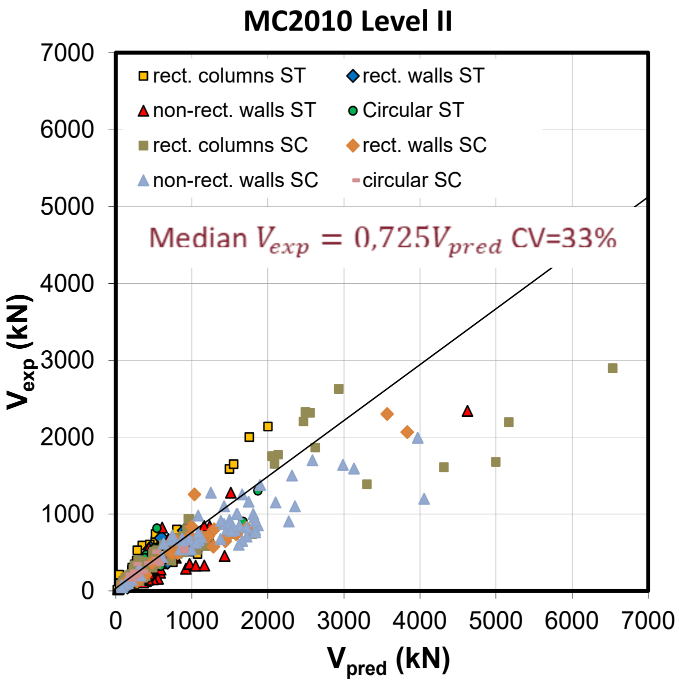
- Before flexural yielding:
  - LoA II gives almost unbiased estimates of cyclic shear resistance, albeit with considerable scatter
  - LoA III reduces scatter but is biased in the unsafe direction $V_{pred} > V_{exp}$
- After flexural yielding (plastic hinge):
  - Cyclic shear resistance is seriously overestimated at both LoA (especially LoA II)

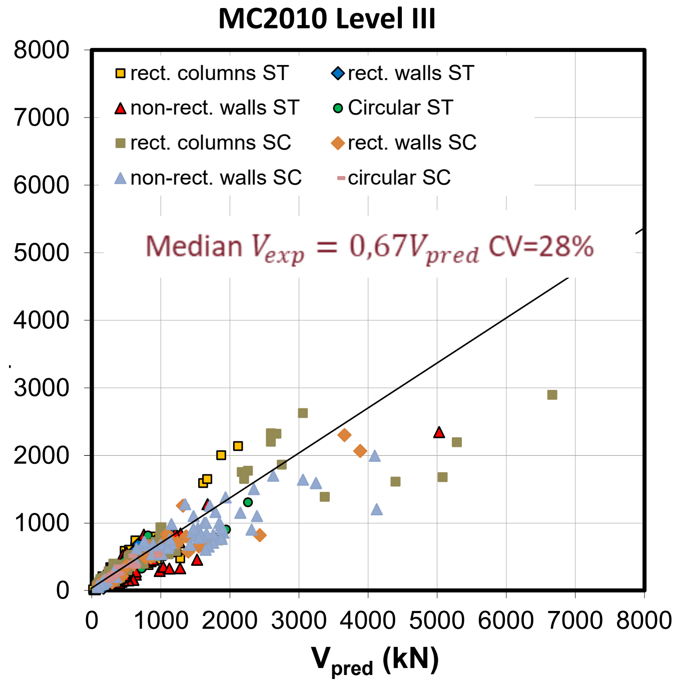
Courtesy of Michael Fardis

Need for a single shear strength model

***fib* MC2010 shear strength model: poor 'seismic' performance**

$$V_R = V_{R,s} + V_{R,c} + V_{R,N} \leq V_{R,max}$$





- If the contribution of the axial load to shear resistance is included (as in *fib* MC1990):
  - Scatter decreases but bias increases.
- In general, improvement in accuracy through sophistication, increases the bias, but improves the scatter.
- Better shear resistance models needed for the plastic hinge

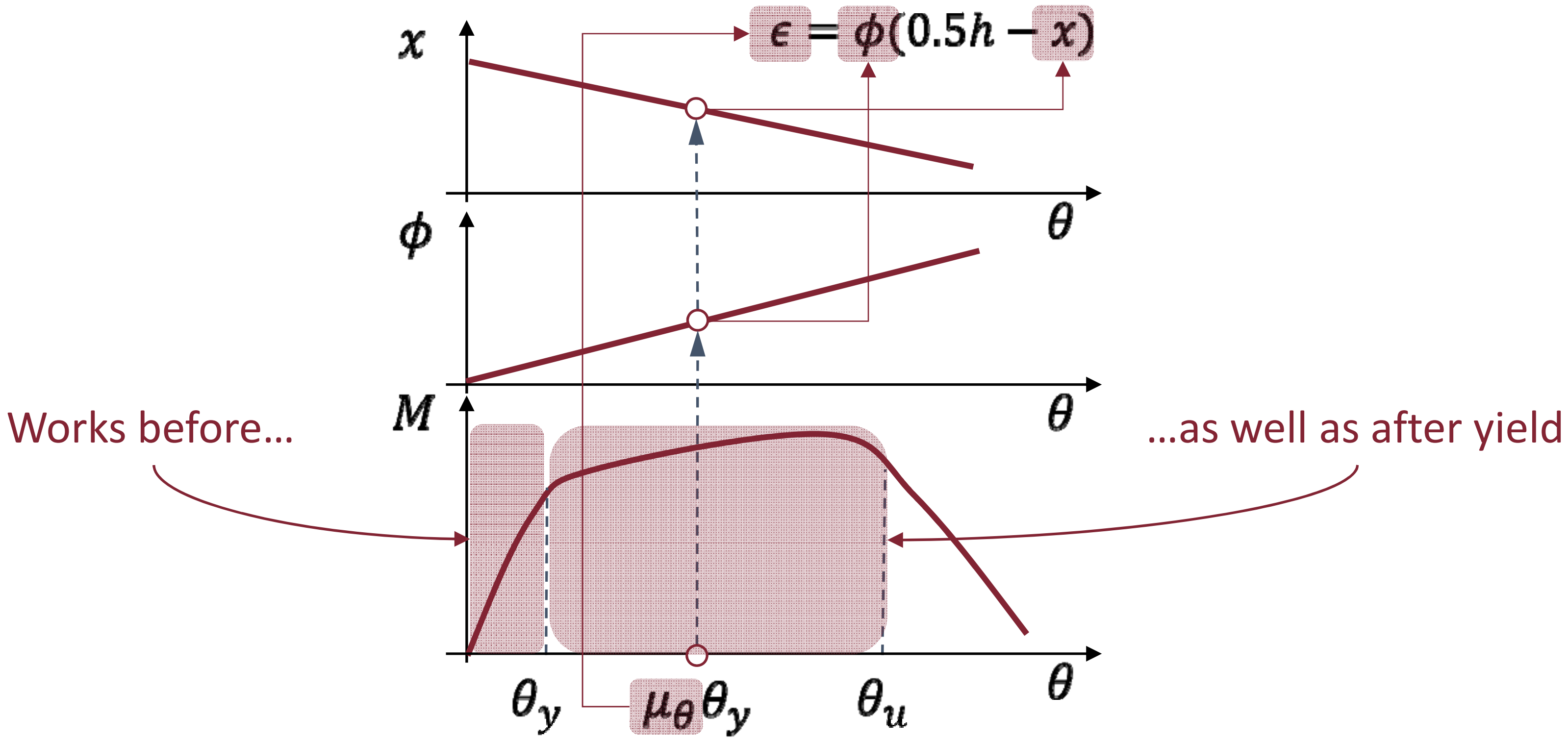
Courtesy of Michael Fardis

Need for a single shear strength model

New model as modification of *fib* MC2010/prEN1992-1-1:2018 shear strength model

$V_R = V_{R,s}(\epsilon) + V_{R,c}(\epsilon) + V_{R,N} \leq V_{R,max}(\epsilon)$ where $\epsilon = \phi(0.5h - x)$

i.e. ϵ replaces μ: as μ increases, ϵ increases, and shear resistance drops, thus, implicitly, one obtains the dependence of post-yield shear resistance on μ



$$V_R = (V_{R,s} + V_{R,c})\left(1 - 0.005 \min\left(5; \mu_\theta^{pl}\right)\right) + V_{R,N} \leq V_{R,max}$$

Need for a single shear strength model

New model as modification of *fib* MC2010/prEN1992-1-1:2018 shear strength model

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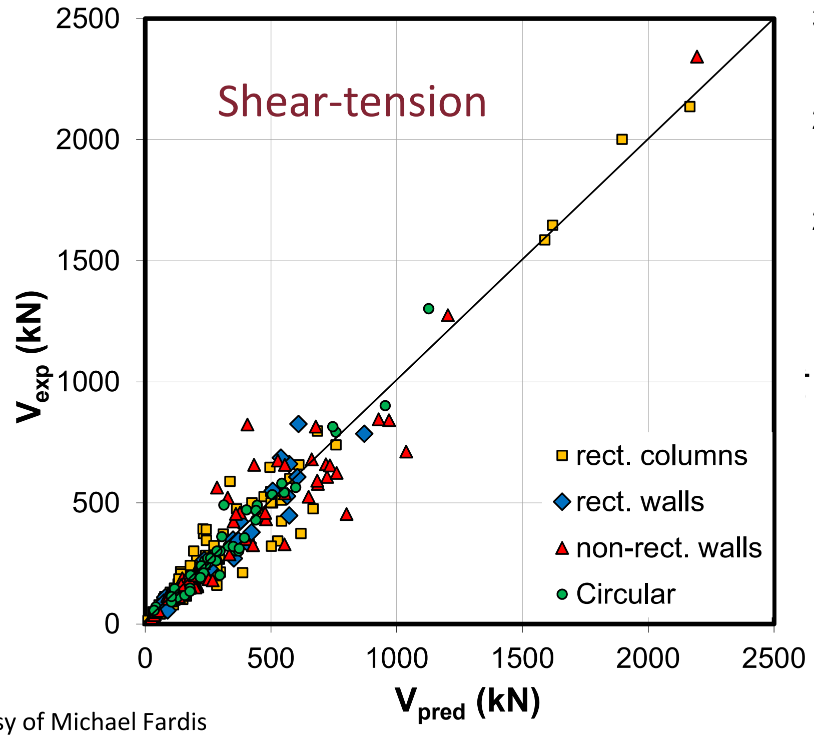
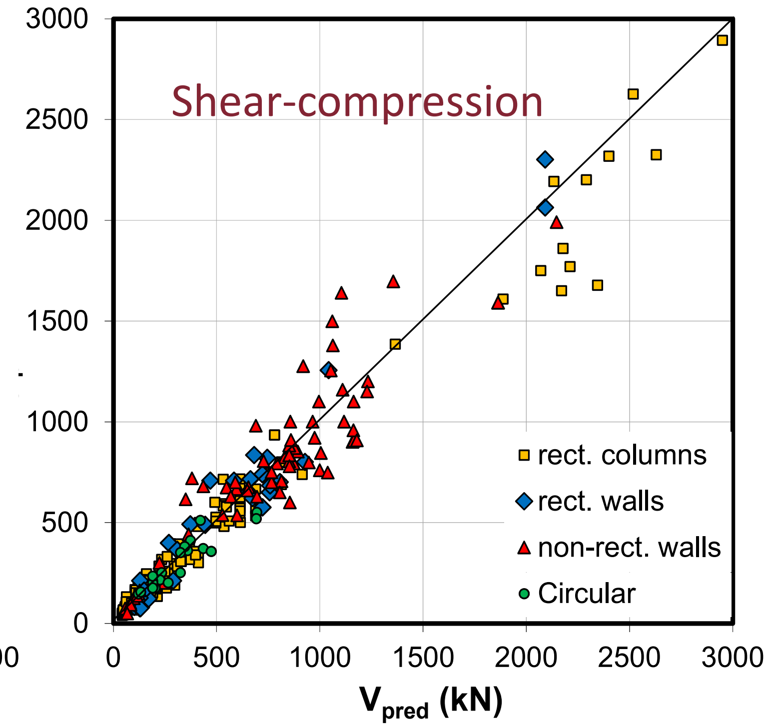
	Rect. columns	Rect. walls	Non-rect. walls	Circular	All
median	1,00	1,00	1,00	1,01	1,00
C.o.V. (%)	24,5	18,9	25,1	19,3	23,3
No. of tests	121	24	70	43	258

	Rect. columns	Rect. walls	Non-rect. walls	Circular	All
median	1,00	1,00	1,00	1,00	1,00
C.o.V. (%)	22,1	24,8	22,4	15,8	22,1
No. of tests	171	36	70	26	303





Courtesy of Michael Fardis

Conclusions

Conclusions

- A general formulation for partial factors on the resistance side has been introduced in Part 3 for seismic assessment
  - It provides means to change safety according to national choices in a consistent manner with a single NDP for all resistance models
  - It allows to control the actual lower fractile of resistance used in verifications

- Consistency between force-based design and displacement-based design requires re-definition of deformation thresholds at life-safety LS
  - These should be lower than what is currently stated in EN1998-3:2005
  - Values of q should also be revisited for cross-material consistency in safety

- Shear strength predicted according to prEN1992-1-1:2018 for concrete members is unconservative for seismic situations
  - Solution A: use the prEN1998-3 model also for design
  - Solution B: introduce a new 'seismic' model to be used for both design and assessment, which shares the theoretical basis with the model in prEN1992-1-1:2018