EPFL

Conventional seismic design and capacity design of structures

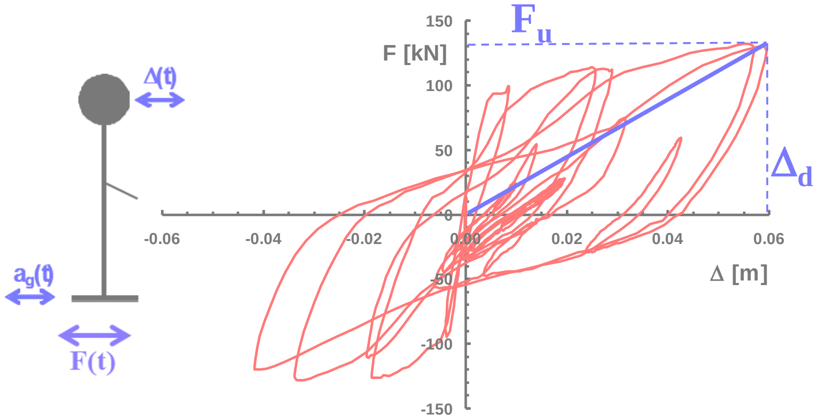
Seismic Engineering

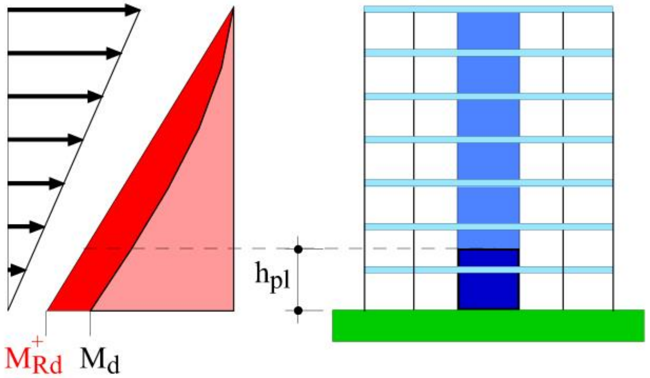
Dr. Igor Tomić

Course objectives

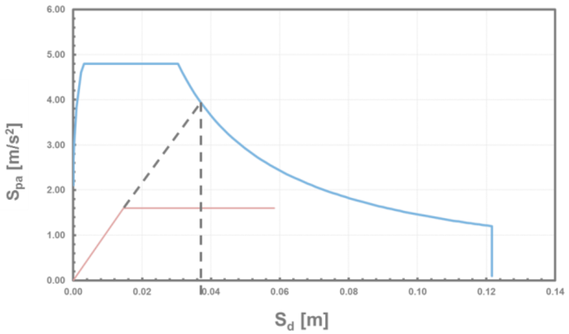
Know typical failure modes of structures during earthquakes.

Know how to estimate the peak forces and displacements of structures subjected to earthquakes.





Know how to design new buildings



Know the basic elements of a displacement-based evaluation of existing structures.

Content

- Seismic design
  - Conventional design vs. Capacity design
  - Principal idea of capacity design
- Capacity design of RC wall buildings
  - Failure modes of RC walls
  - Behaviour of concrete and steel
  - Capacity design of RC wall buildings -> next course

Seismic design

Seismic design objectives of SIA-codes

- Protection of persons
- Limitation of excessive damage
- Ensure functionality of important construction works for frequent earthquakes (BWK III)

Seismic design checks according to SIA 261

- Verification of structural safety for all construction work classes (BWK)
- Verification of serviceability for construction work class III for return period 475 yrs (γ_f=1.0
-)

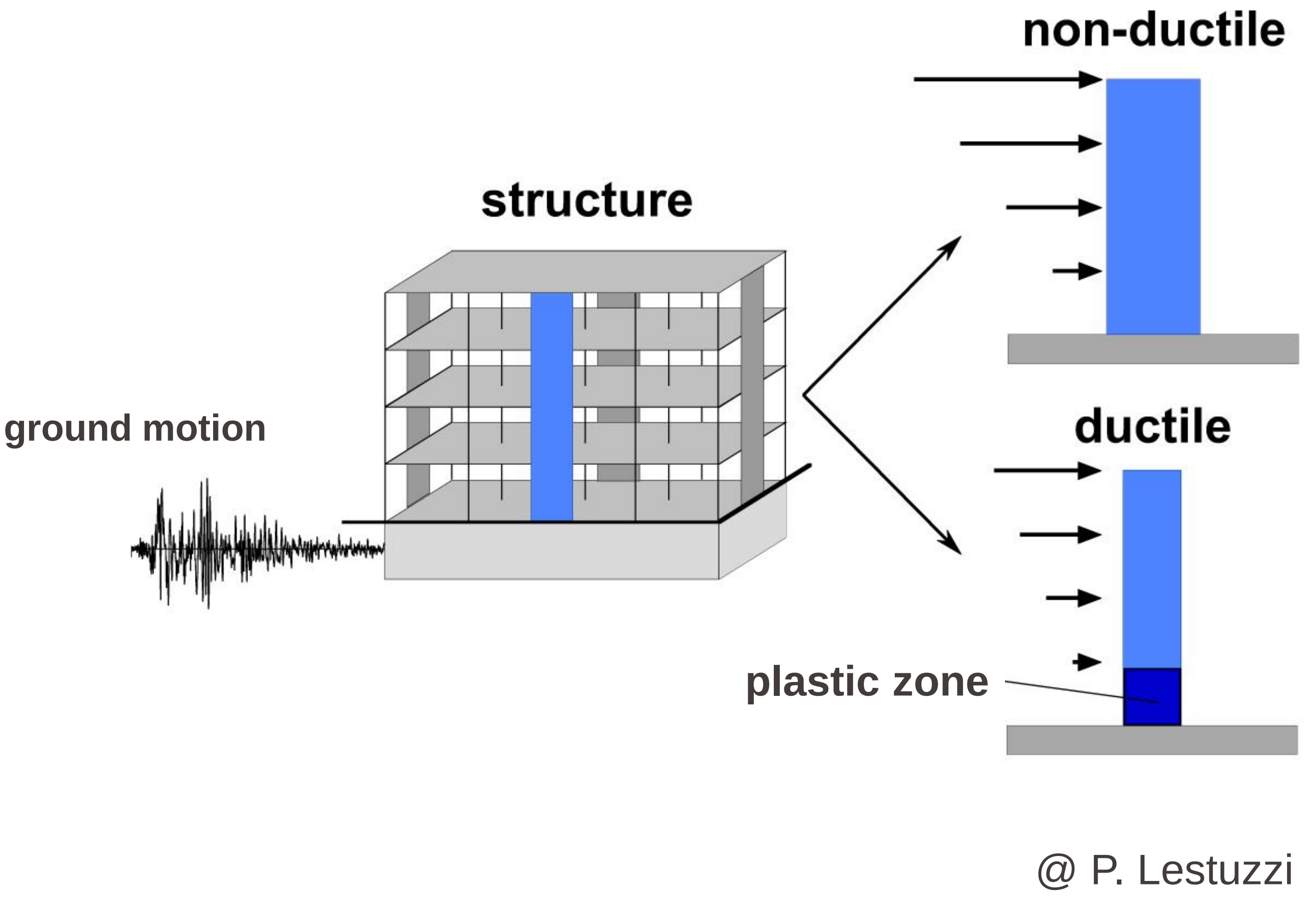
BWK	Verification of structural safety		Verification of serviceability	
	Return period of design earthquake	Corresponding PGA (for rock)	Return period of design earthquake	Corresponding PGA (for rock)
I	475 years	$\gamma_f a_{gd}=1.0a_{gd}$	Not required	
II	~600-800 years	$\gamma_f a_{gd}=1.2a_{gd}$	Not required	
III	~1000-1250 years	$\gamma_f a_{gd}=1.5a_{gd}$	475 years	a_{gd}

PGA= Peak ground acceleration

a_{gd}= PGA on rock for 475 year return period

Seismic design

Two different design approaches in SIA 261



- Design for larger horizontal forces (smaller q-factors)
- No extra detailing requirements

- Design for smaller horizontal forces (larger q-factors)
- Extra detailing requirements because local ductility demands are larger than for conventional design

Seismic design

Conventional design (non-ductile design)

- Non-ductile behaviour of the structure assumed
- Design approach as for «traditional» loads

Behaviour factors for non-ductile RC wall buildings

Behaviour factor	Reinforcement class			Prestressed steel
	A	B	C	
q	1.5	2	2	1.5

Capacity design (ductile design)

- Ductile behaviour of the structure assumed
- Special design requirements to ensure that the assumed mechanism forms → «Capacity design»
- Special detailing requirements to ensure a sufficiently large local ductility capacity
- RC walls: $h_w / l_w \geq 2.0$

$$-N / A_c f_{cd} \leq 0.4$$

Behaviour factors for ductile RC wall buildings

Behav. factor	Reinforcement class			Prestressed steel
	A	B	C	
q	-	3	4	Special investigation required

Conventional design

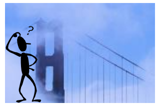
- Conventional design for seismic effects
- Design as for other actions (gravity forces, wind, …)
- Use q-factors that correspond to a non-ductile behaviour, e.g., for RC wall buildings:

Reinforcement class	Non-ductile
B500A	q=1.5
B500B, B700B	q=2.0
B500C	q=2.0

- These q- factors account for the overstrength and for reinforcement classes B and C for a small inelastic displacement capacity.
- 3 steps of conventional design of RC walls
  - Flexural design
  - Shear design
  - Detailing (stabilisation of longitudinal reinforcement following the rules for «Compression members» in SIA 262, 5.5.4)

One question…

Connect to ttpoll.eu
Session ID: cape5



A conventionally-designed structure (q=2) subjected to a design intensity ground motion will behave as follows:

A. It will form a mechanism (with a small ductility demand), but we do not know which one. It might collapse.

B. It will form a mechanism (with a small ductility demand), but we do not know which one. It will not collapse.

C. It will start forming a mechanism and then fail in shear.

D. It will start forming a mechanism and then fail in flexure.

E. It will start forming a shear mechanism but not fail.

F. It will start forming a flexural mechanism but not fail.

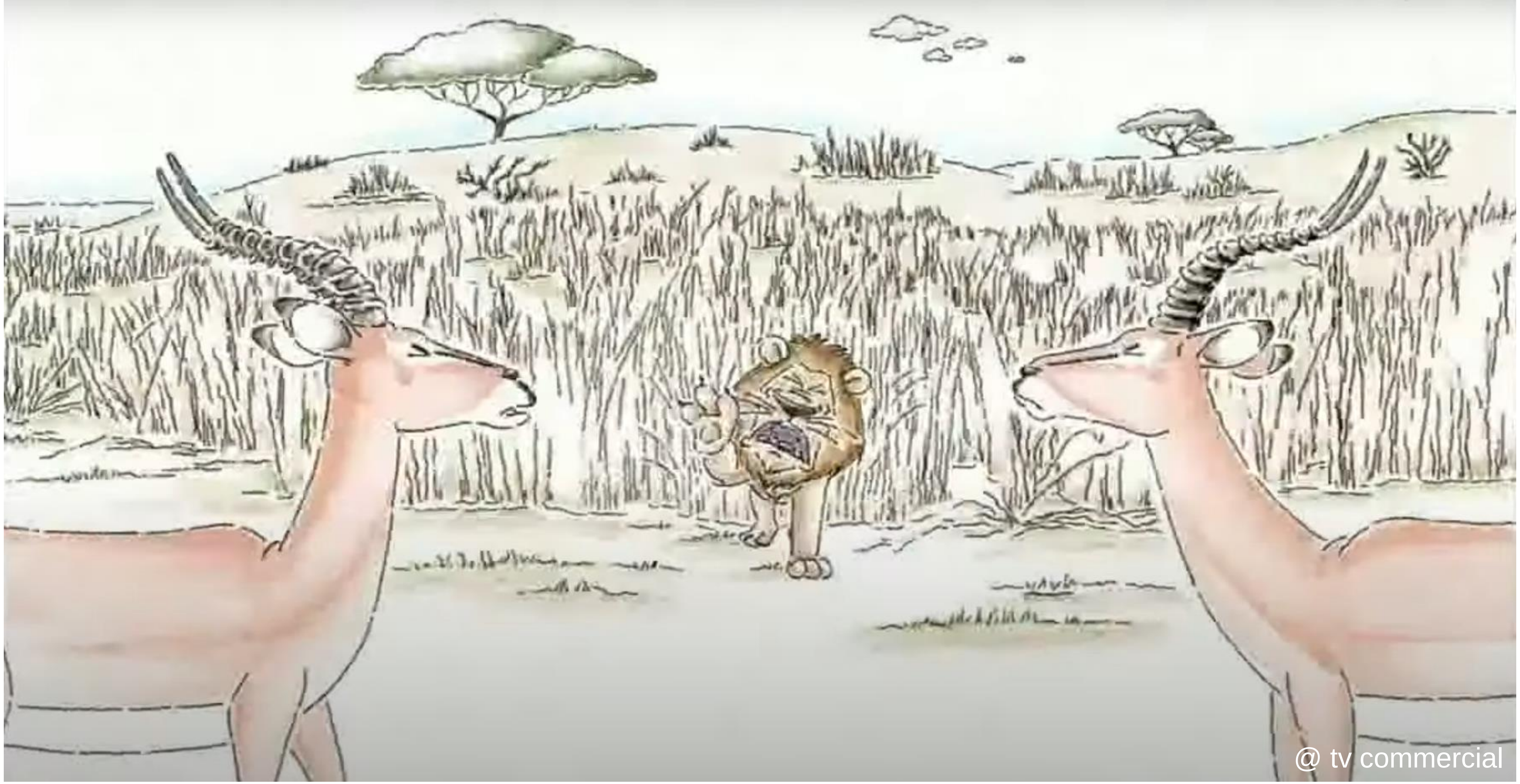
Capacity design

Idea: The engineer chooses the mechanism that should form

→ The engineer imposes on the structure where it can plastify and where it must not.

→ Establish a clear hierachy of strengths.

Capacity design



Capacity design

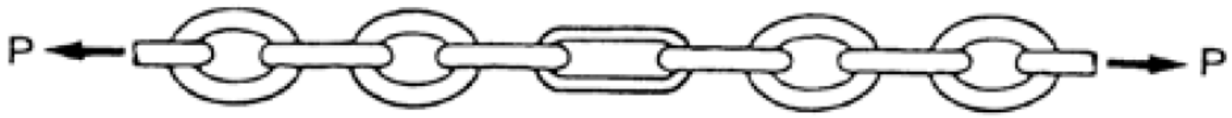
Hierachy of strength in a structure

- Make the most ductile element the weakest

→ This element will plastify.

- The other elements have to remain elastic even if the element that plastifies develops its overstrength.

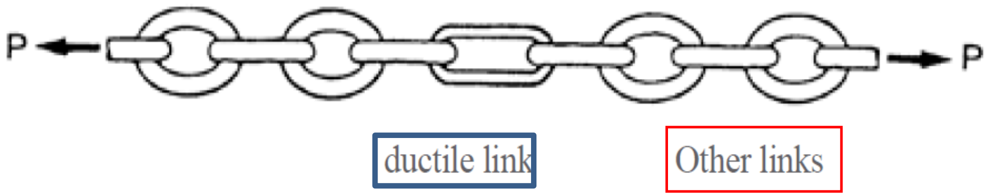
→ The design force for these other elements do not depend on the seismic demand but on the resistance of the element that plastifies.



	ductile link	Other links
Calculated action effect: ——>	E_{di}	E_{dj}
Required resistance: ———>	$R_{di} > E_{di}$	$\gamma(R_{di} / E_{di})\, E_{dj}$ ($\gamma = 1{,}2$)

One question…

Which of the pairs of force-displacement capacity curves leads to a capacity-designed system?

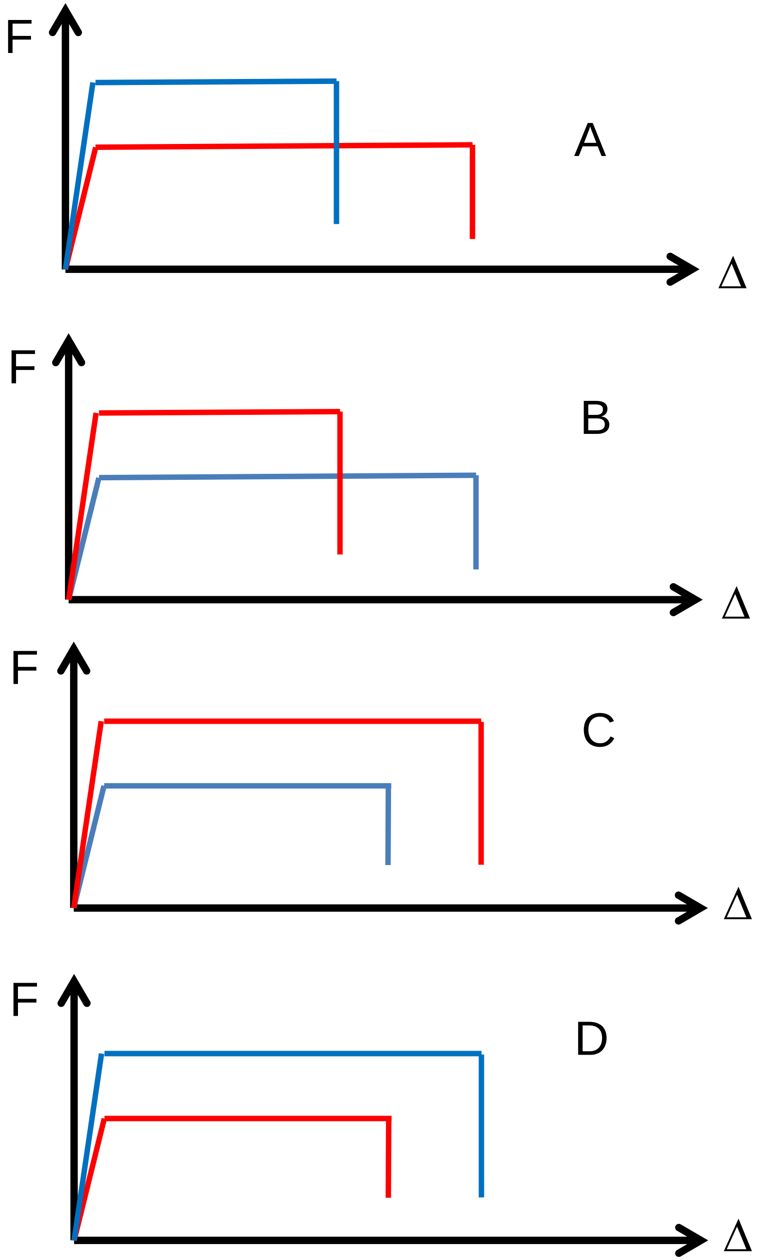


A. Curves of schema A

B. Curves of schema B

C. Curves of schema C

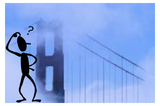
D. Curves of schema D



Dr. Igor Tomić

One question...

Connect to ttpoll.eu
Session ID: cape5



If the system shown on the right is a capacity-designed system, which of the following statements is <u>not</u> correct?

A. Increasing the force capacity of the “other links” does not influence the force or displacement capacity of the system.

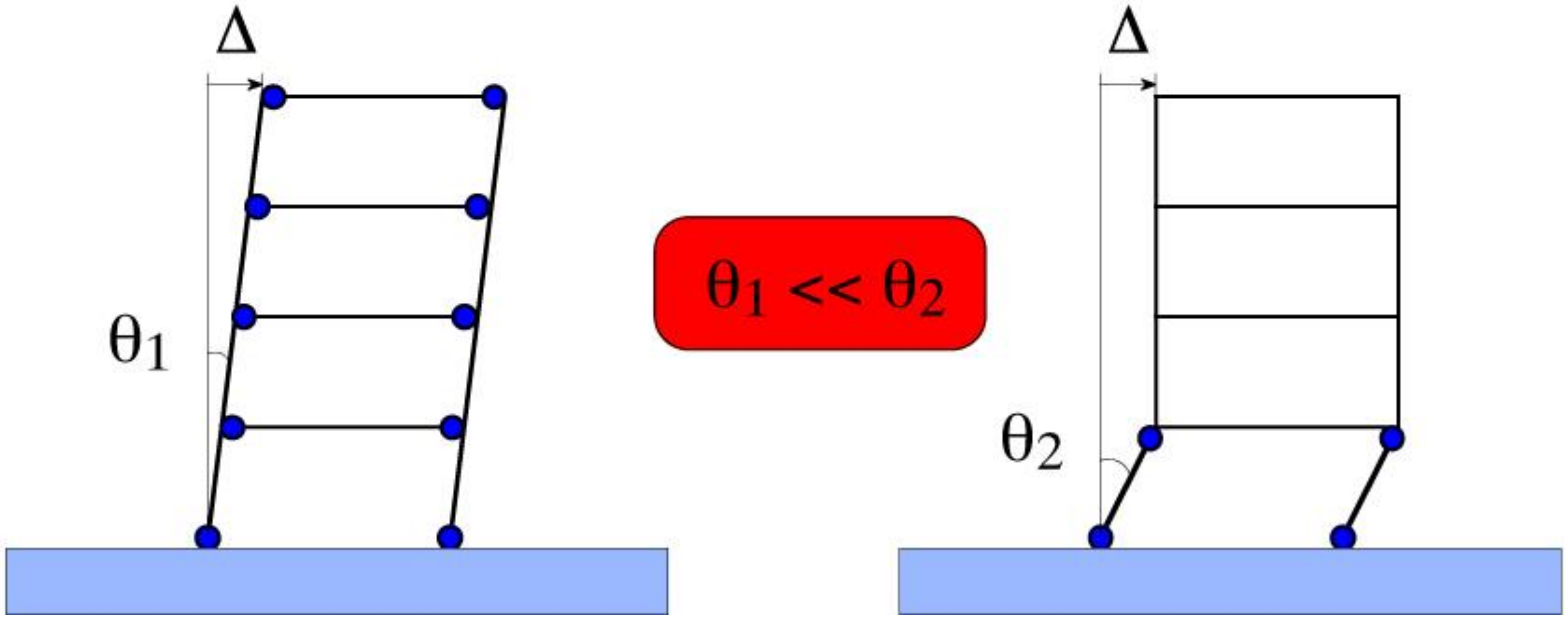
B. Increasing the deformation capacity of the “other links” increases the displacement capacity of the system.

C. Increasing the force capacity of the ductile link significantly reduces the displacement capacity of the system.

D. Increasing the displacement capacity of the ductile link increases the displacement capacity of the system.

Capacity design

Frame buildings: Choice of the plastic mechanism



Good

Bad

Objective: Choose the plastic mechanism that minimises the local deformations (curvature, rotation) for the same global displacement Δ.

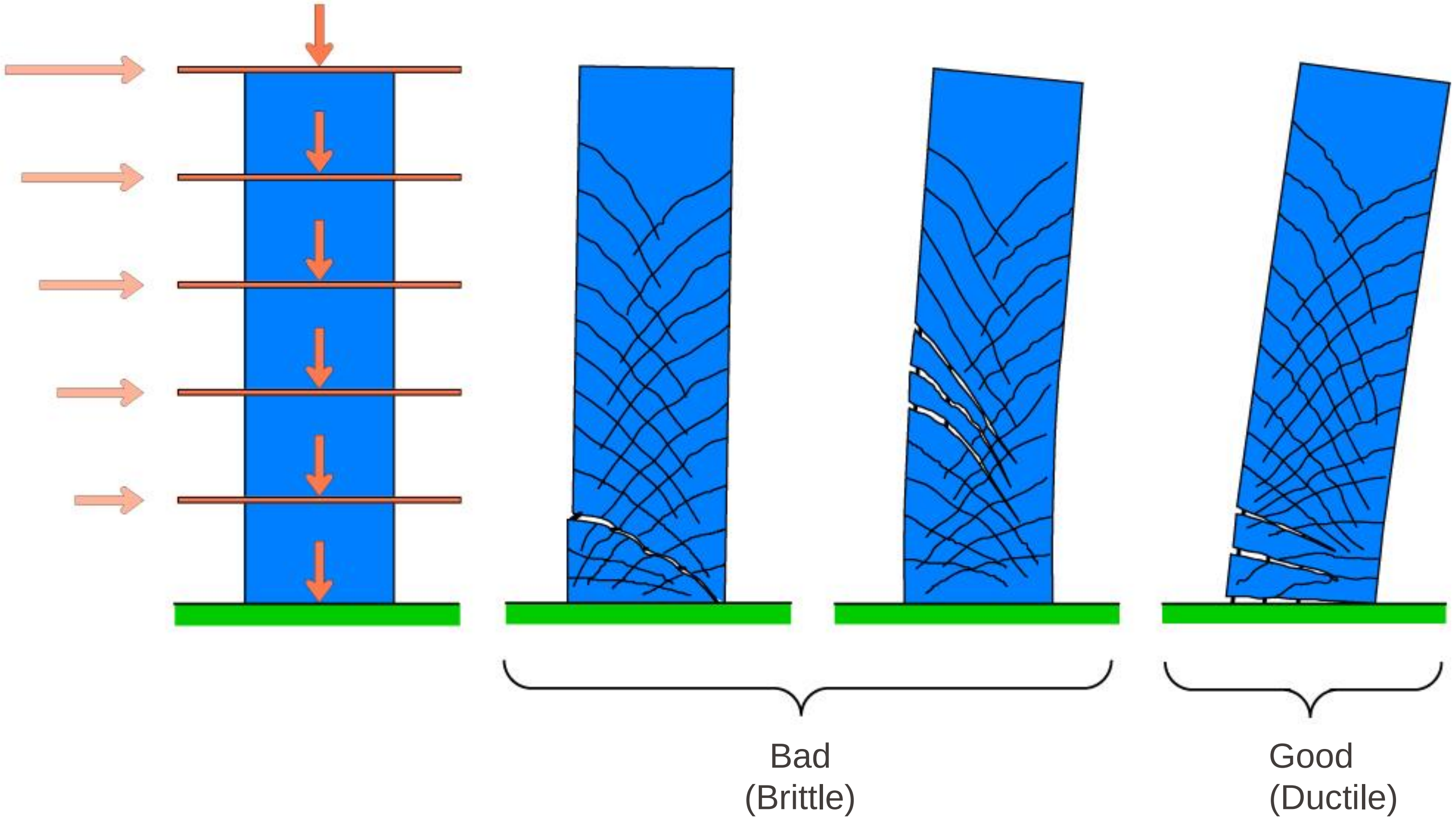
Failure of a structure is linked to local deformations. Failure occurs if:

Local deformation demand > Local deformation capacity

Dr. Igor Tomić

Capacity design

RC wall buildings: Choice of the plastic mechanism



Capacity design

- **Idea behind capacity design**

- Mechanism: Choose a favourable mechanism

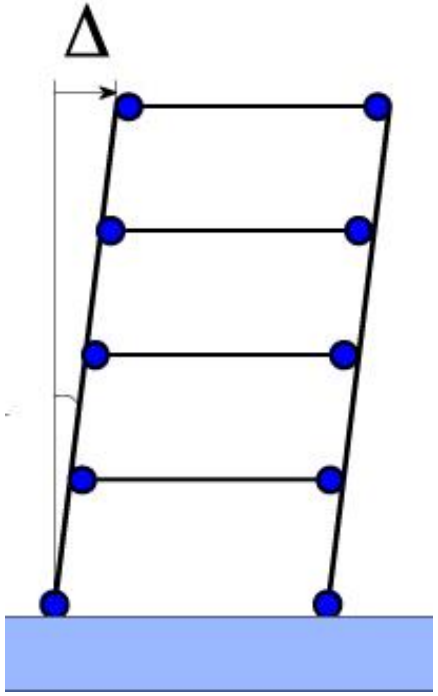
  = a mechanism that allows the structure to reach large displacement ductilities

  → Identify the zones that should plastify («plastic zones»)

- Deformation capacity of plastic zones: Design these zones in such a way that brittle failure modes are avoided and that the deformation capacity of these zones is sufficiently large (→ good detailing of plastic zones required).
- Hierachy of strengths: The others zones must be designed in a way that they remain elastic even if the plastic zones develop their effective strength (larger than the design strength!)
- Capacity design is the principal idea of ductile design in all modern seismic design codes.
- Head behind capacity design: Prof. Tom Paulay

One question...

If a capacity-designed frame is subjected to a ground motion that is larger than the design ground motion – how will the frame behave?



A. It will form a mechanism, but we do not know which one. It might collapse.

B. It will form a mechanism, but we do not know which one. It will not collapse.

C. It will form a mechanism and we know which one. It might collapse.

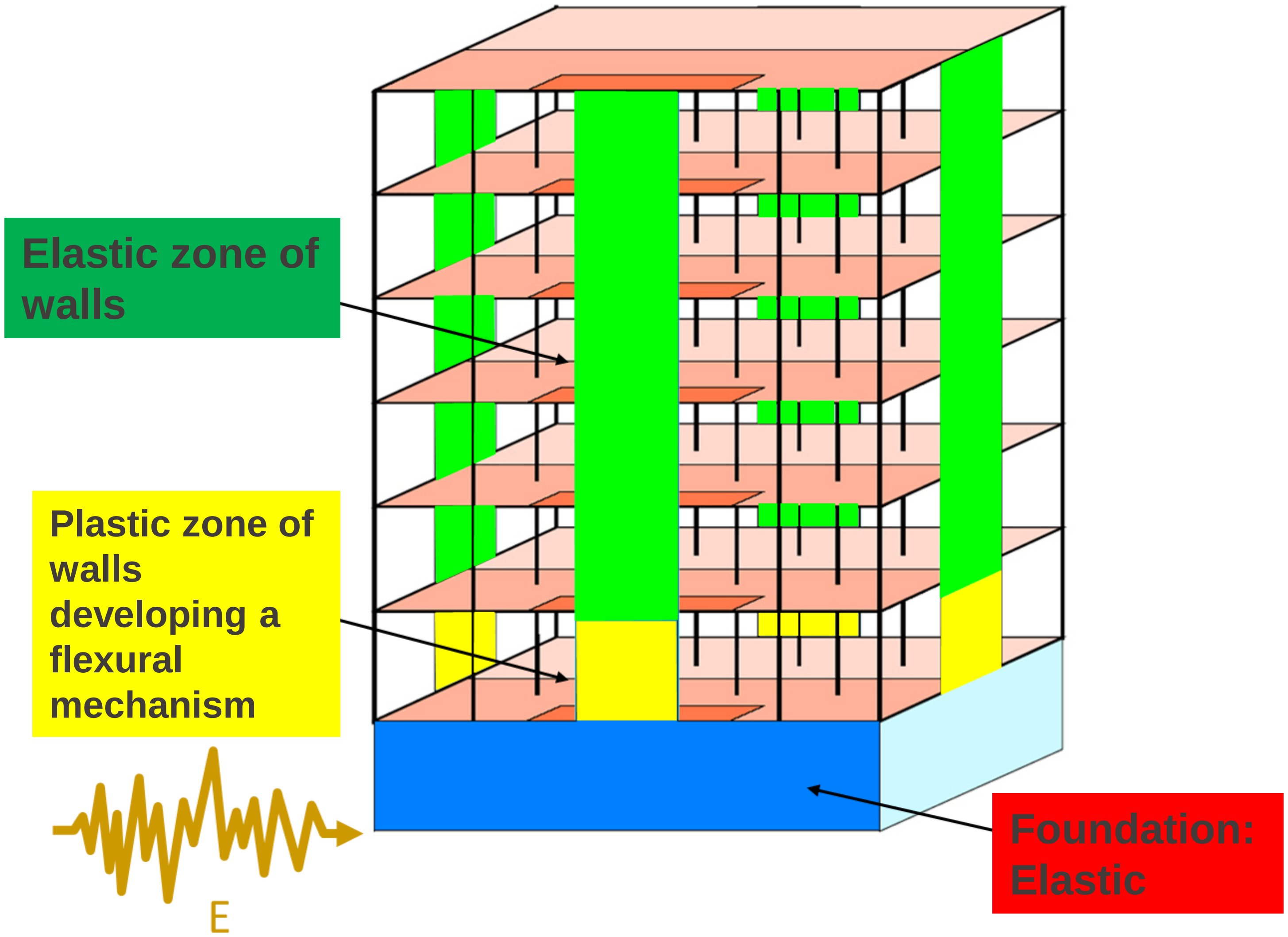
D. It will form a mechanism and we know which one. It will not collapse.

Capacity design

Seismic behaviour	
Conventional design	**Capacity design**
• For a ground motion corresponding to the design intensity: Very limited plastification. • If intensity larger than design intensity: No control over which zones plastify. • The zones that plastify were not designed for it. • The behaviour during an earthquake with an intensity larger than the design intensity is not known.	Even if the intensity of shaking is larger than the design intensity: • The mechanism is known. • Only those zones plastify that were designed for it. • The local ductility demands will be somewhat larger than anticipated during design (but we also design with a safety margin...).
Limited safety against collapse	**High safety against collapse**

Capacity design

Capacity design of a building with slender RC walls



Content

- Seismic design
  - Conventional design vs. Capacity design
  - Principal idea of capacity design
- Capacity design of RC wall buildings
  - Failure modes of RC walls
  - Behaviour of concrete and steel
  - Capacity design of RC wall buildings → Next course

Failure mechanisms of RC walls

Ductile failure mechanisms:

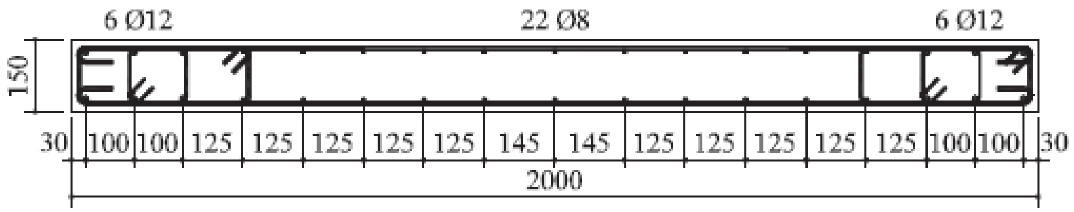
- Flexural failure due to rupture of the longitudinal bars at large plastic strains
- Flexural failure due to crushing of the concrete after the longitudinal bars yielded in tension

Brittle failure mechanisms:

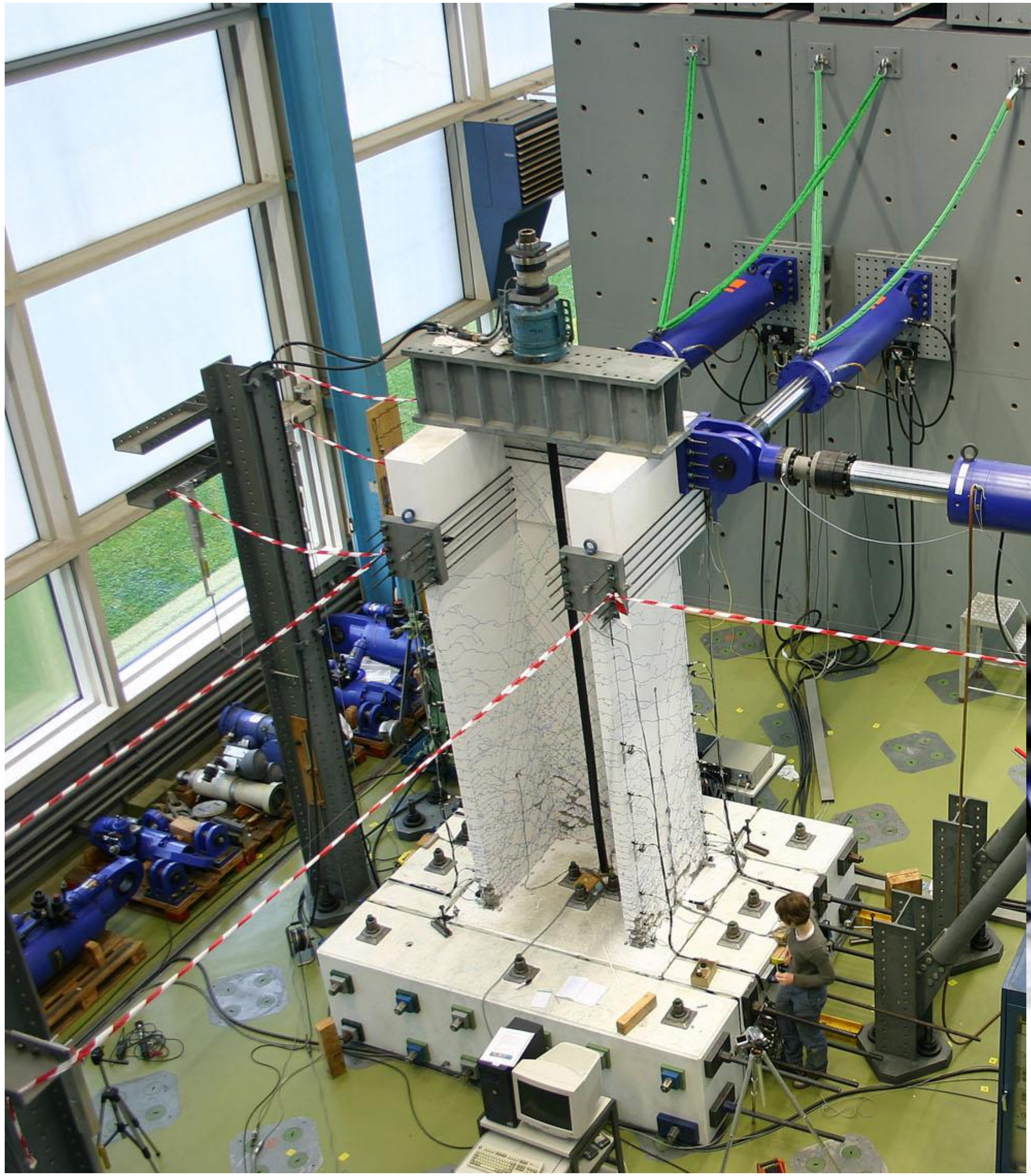
- Flexural failure due to rupture of the longitudinal bars at small plastic strains
- Flexural failure due to crushing of the concrete before the longitudinal bars yielded in tension
- Shear failure due to rupture of horizontal reinforcement
- Shear failure due to crushing of the compression strut
- Failure of the lap splices or the anchorage of the longitudinal bars.

Failure mechanisms of RC walls

Cross section of a RC wall - Terminology



Failure mechanisms of RC walls

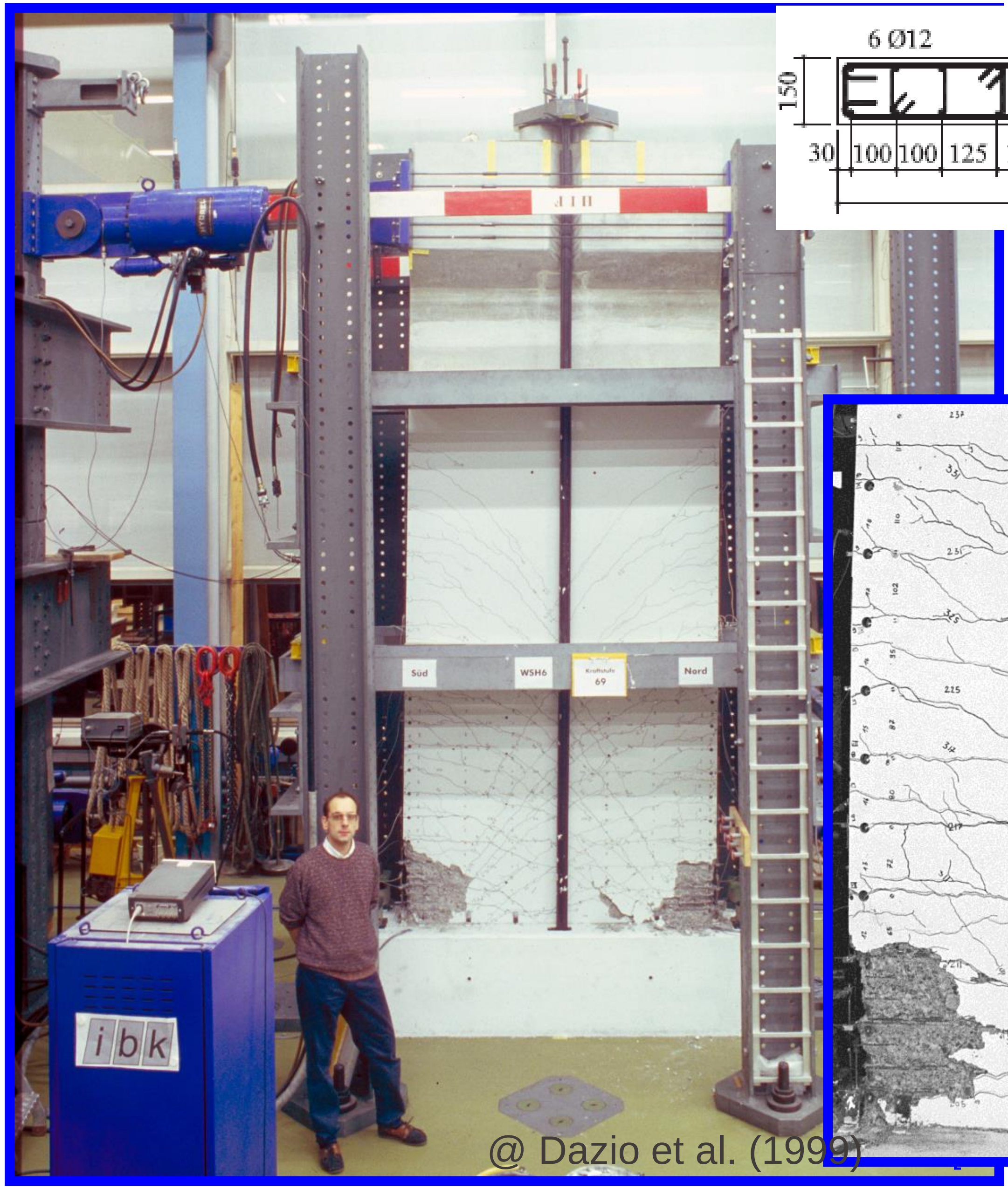


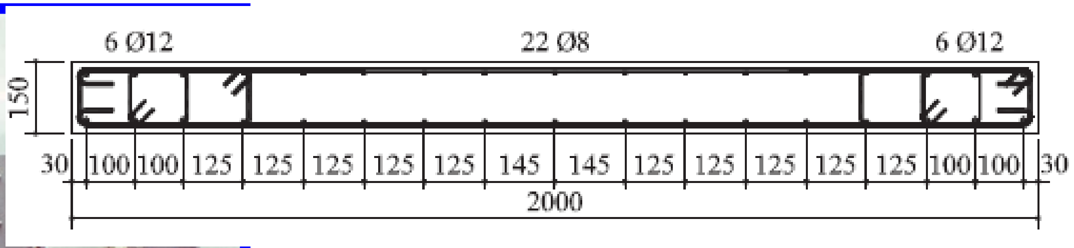
Ductile mechanism

Flexural failure due to rupture of the longitudinal bars at large plastic strains

@ Beyer et al. (2008)

Failure mechanisms of RC walls





Ductile mechanism

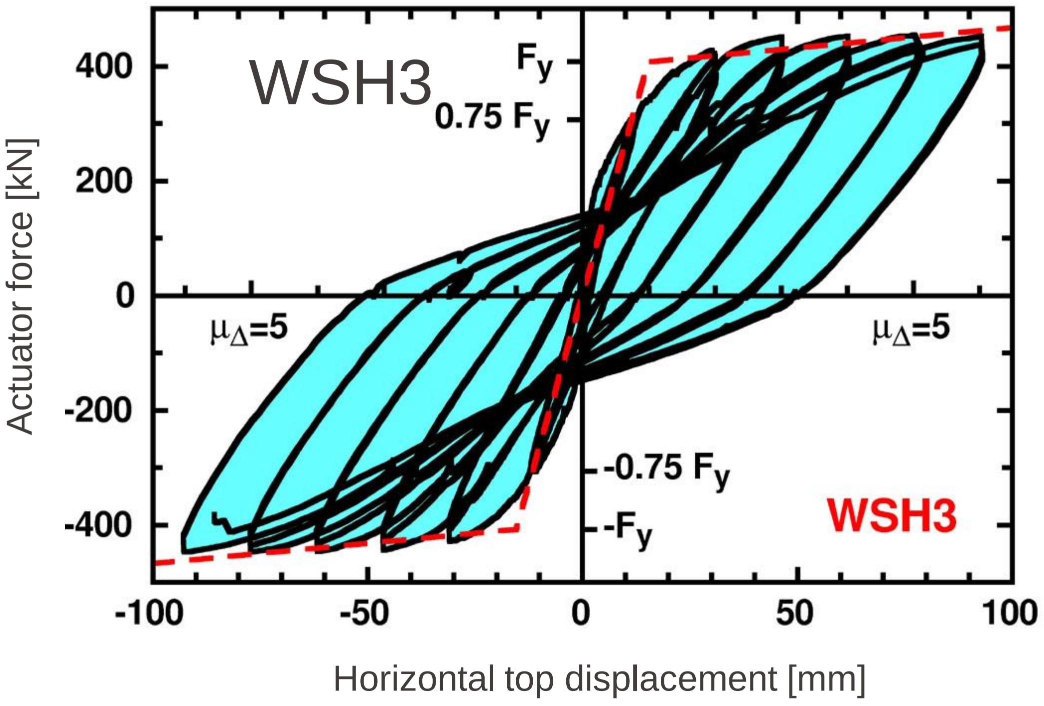
Flexural failure due to crushing of the concrete after the longitudinal bars yielded in tension

@ Dazio et al. (1999)

Dr. Igor Tomić

Failure mechanisms of RC walls

Example: Force-displacement relationship of a ductile walls



@ Dazio et al. (1999)

Failure mechanisms of RC walls

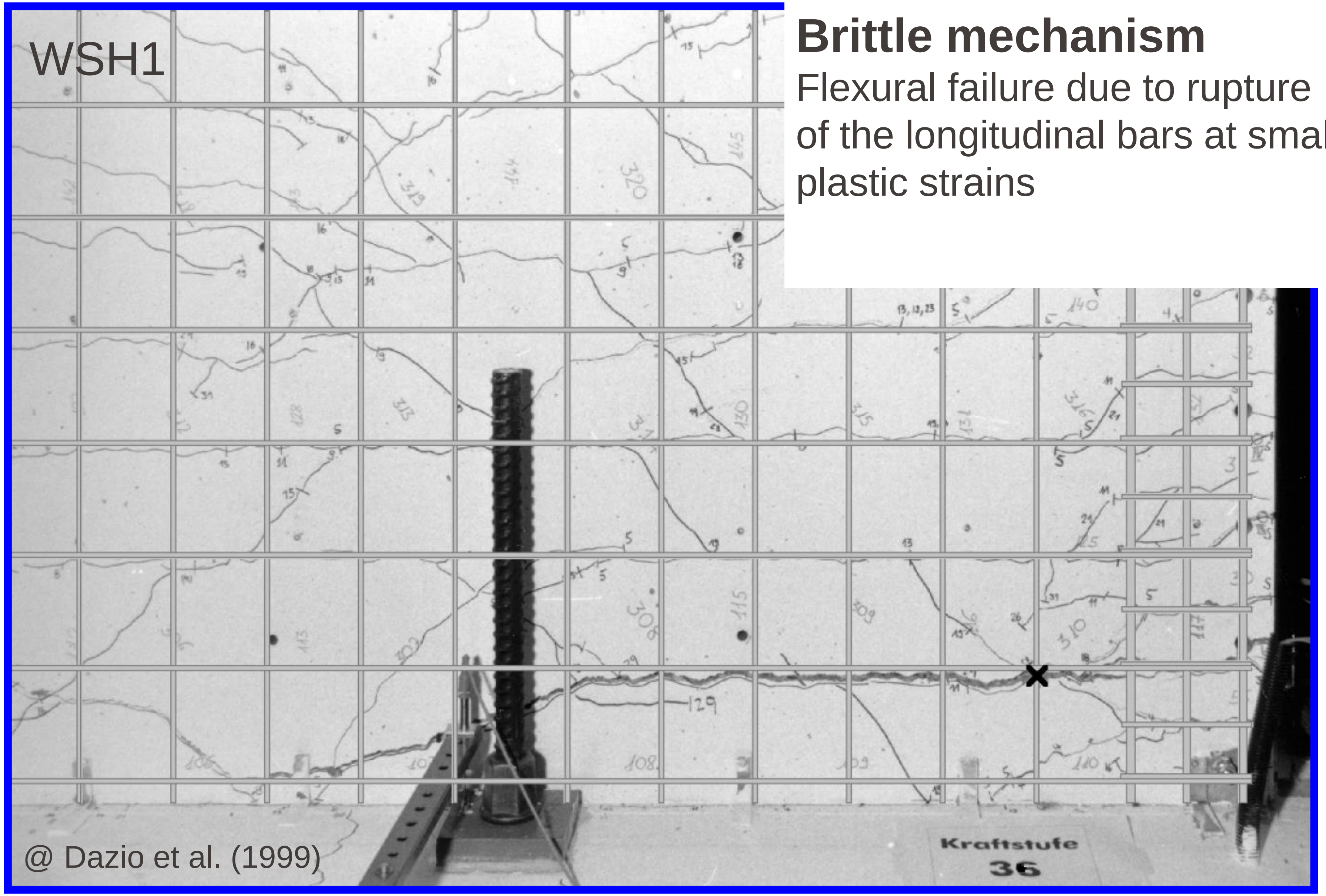
Ductiles failure mechanisms:

- Flexural failure due to rupture of the longitudinal bars at large plastic strains
- Flexural failure due to crushing of the concrete after the longitudinal bars yielded in tension

Brittle failure mechanisms:

- Flexural failure due to rupture of the longitudinal bars at small plastic strains
- Flexural failure due to crushing of the concrete before the longitudinal bars yielded in tension
- Shear failure due to rupture of horizontal reinforcement
- Shear failure due to crushing of the compression strut
- Failure of the lap splices or the anchorage of the longitudinal bars.

Failure mechanisms of RC walls



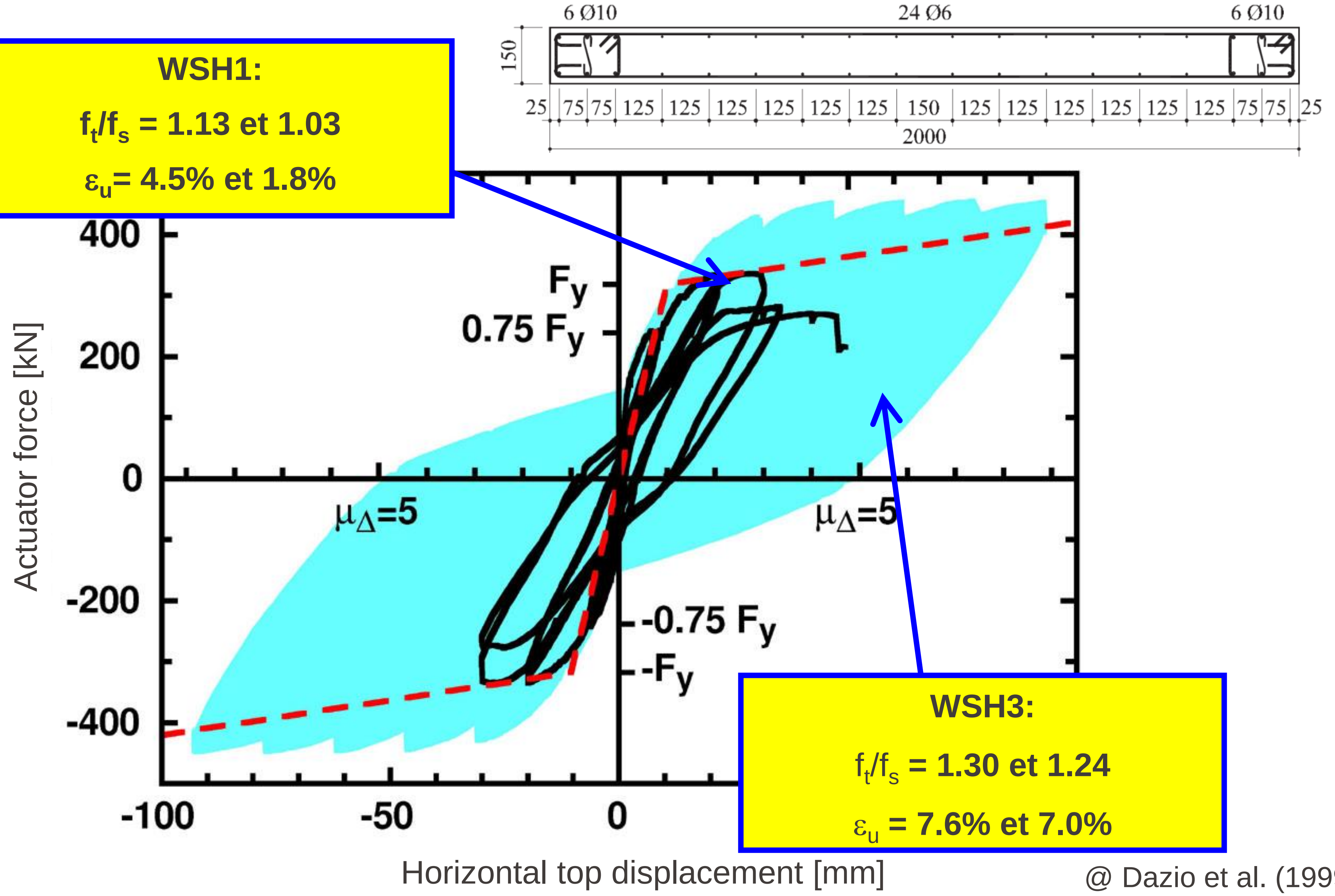
Brittle mechanism

Flexural failure due to rupture of the longitudinal bars at small plastic strains

WSH1:

f_t/f_s = 1.13 et 1.03

ε_u= 4.5% et 1.8%



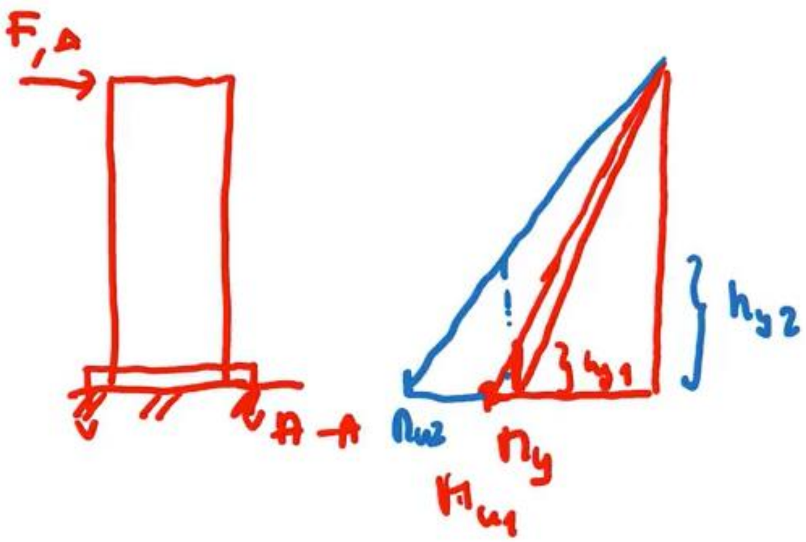
WSH3:

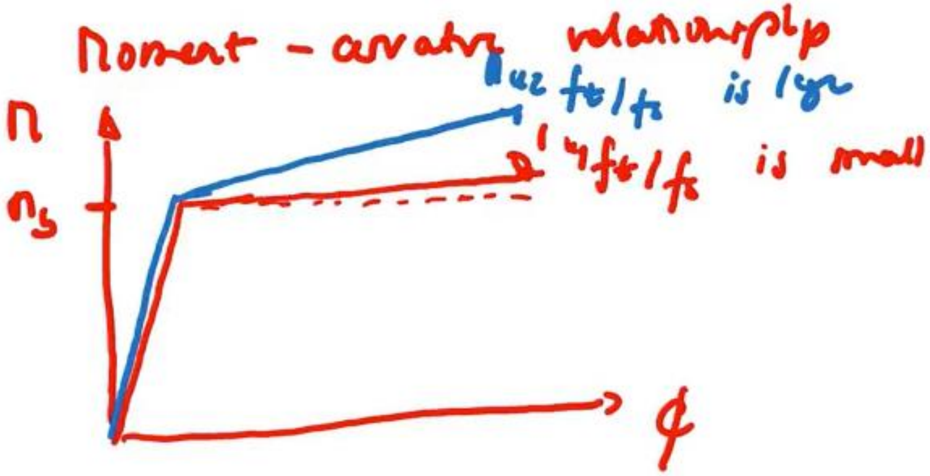
f_t/f_s = 1.30 et 1.24

ε_u = 7.6% et 7.0%

@ Dazio et al. (1999)

Failure mechanisms of RC walls





h_{y1}, h_{y2} – heights on which longit. bars will yield
for f_t/f_s small / large
(neglecting any tension shift effects)

Failure mechanisms of RC walls

Ductility classes of reinforcement bars (SIA 262)

Acier d'armature passive	B500A	B500B	B500C	B700B	Fractile [1]
Produit	Torches, treillis soudés	Barres, torches, treillis soudés	Barres, torches, treillis soudés	Barres, torches, treillis soudés	
Classe de ductilité	A	B	C	B	
Limite d'écoulement f_{sk} [N/mm^2] [2] [3]	500	500	500	700	5%
Rapport $(f_t/f_s)_k$	≥ 1,05 [4]	≥ 1,08	≥ 1,15 ≤ 1,35	≥ 1,08	10%
Allongement sous charge ultime ε_{uk} [%]	≥ 2,5 [4]	≥ 5,0	≥ 7,5	≥ 5,0	10%
Essai de fatigue Contrainte supérieure [N/mm^2] Amplitude de charge [N/mm^2]	300 Barres, torches : 150 [5]; treillis soudés : 100				10%
Essai de cisaillement pour les treillis, force [kN]	$A_s \cdot 150$ N/mm^2				5%
Ecart maximal par rapport aux dimensions nominales [%]	± 4,5 pour Ø > 8 mm ± 6,0 pour Ø ≤ 8 mm				
Surface	nervurée à ailettes				
Surface projetée relative des nervures f_R [–] 5 mm < Ø ≤ 6 mm 6,5 mm < Ø ≤ 12 mm Ø > 12 mm	0,035 0,040 0,056				

[1] Fractiles pour un seuil de confiance de 90%
[2] La valeur maximale déterminée par des essais ne doit pas être supérieure à 1,3 f_{sk}
[3] Les aciers d'armature passive avec des résistances plus hautes sont déclarés dans le Registre des aciers d'armature passive conformes aux normes
[4] Pour les barres Ø < 6 mm il faut : $(f_t/f_s)_k \geq 1{,}03$ et $\varepsilon_{uk} \geq 2{,}0\%$
[5] Pour les barres 20 mm < Ø ≤ 40 mm : 135 N/mm^2

Failure mechanisms of RC walls

Concepcion, 152 Castellon (Chile)

Failure mechanisms of RC walls

Brittle mechanism
Flexural failure due to crushing of the concrete before the longitudinal bars yielded in tension

RC building with different bracing systems for the two directions: Frames in the short direction and walls in the long direction

Via Svizzera
@ A. Dazio

Failure mechanisms of RC walls

Shear failure: Diagonal crack and rupture of the horizontal reinforcement

Via Svizzera

Failure mechanisms of RC walls

Shear failure: Crushing of the compression diagonal

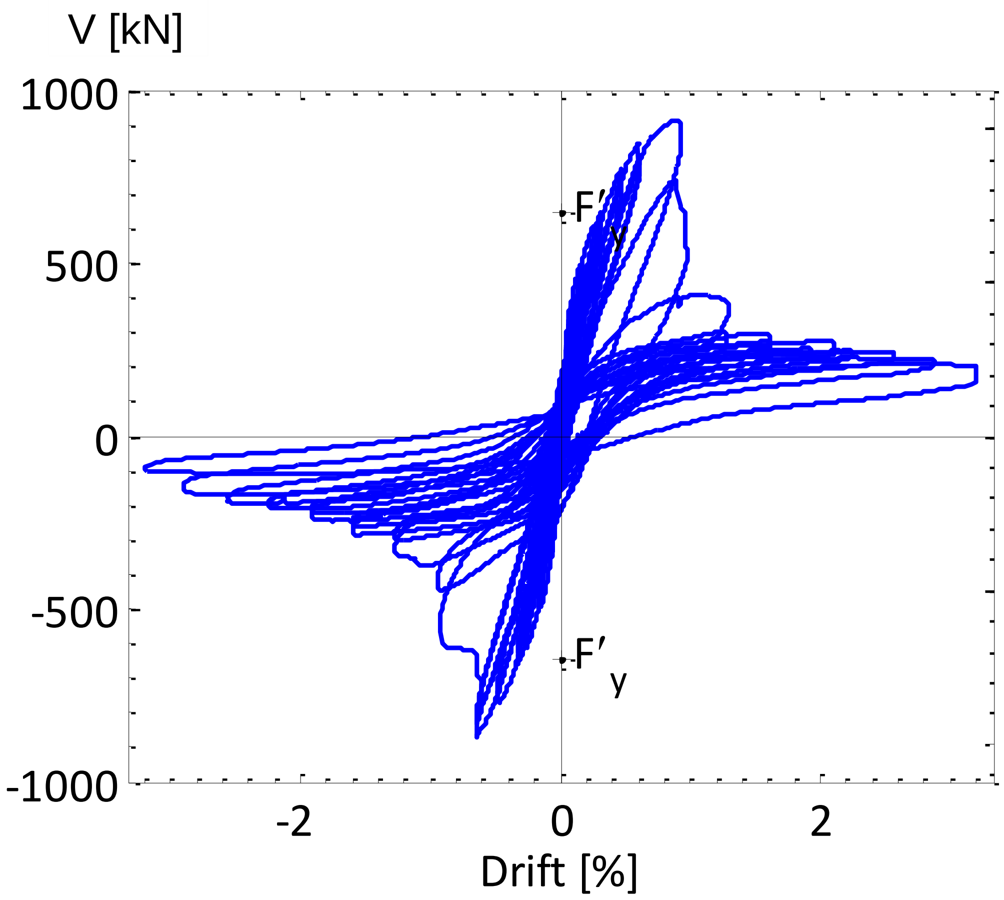
Concepcion, 152 Castellon (Chile)

Failure mechanisms of RC walls

Failure mechanisms of RC walls

Lap splice failure in plastic zone

Axial load capacity remains often intact!



@ Hannewald, Bimschas et Dazio (2013)

Failure mechanisms of RC walls

Stress-strain relationships of concrete and steel

Behaviour of concrete and steel

Concrete

- Unconfined concrete
- Confined concrete

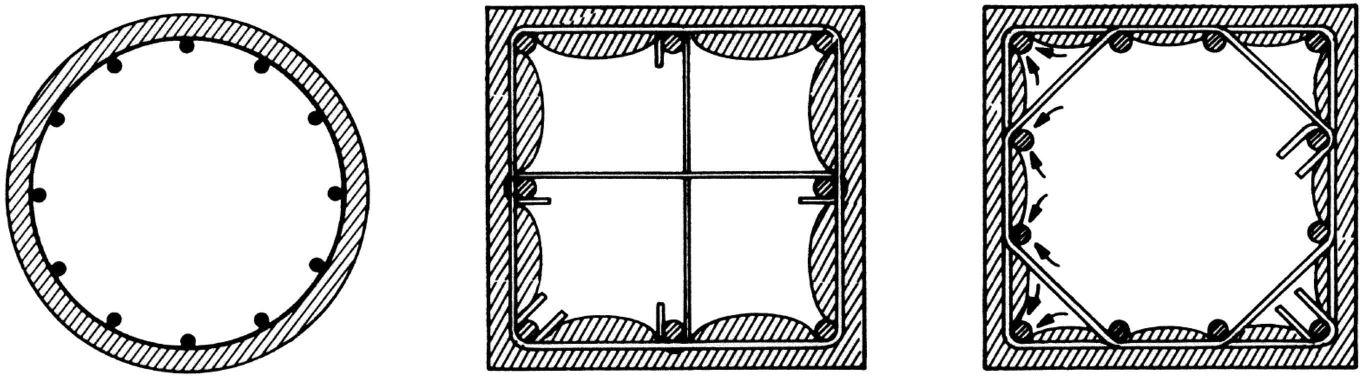
Reinforcement bars

- Monotonic and cyclic response
- Buckling and fracture of reinforcement bars under cyclic loading

Stress-strain relationships of concrete and steel

Dr. Igor Tomić

Concrete



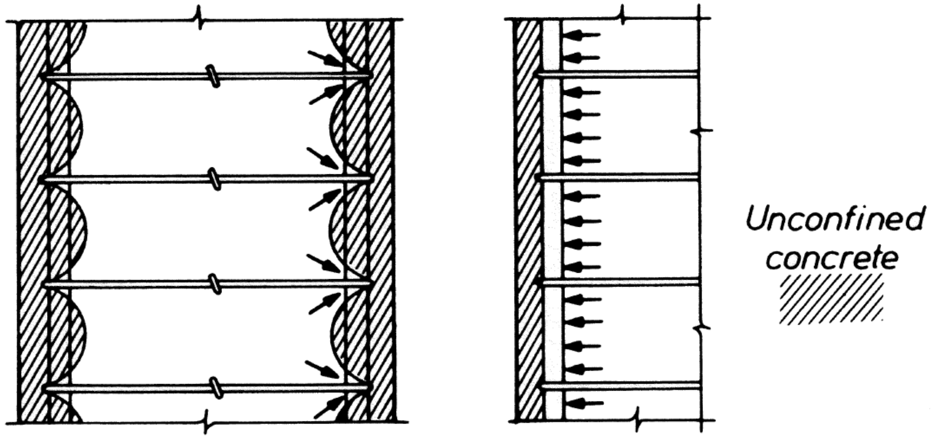
(a) Circular hoops or spiral

(b) Rectangular hoops with cross ties

(c) Overlapping rectangular hoops

Unconfined concrete
(p. ex.: Cover concrete)

Confined concrete



(d) Confinement by transverse bars

(e) Confinement by longitudinal bars

@ Paulay and Priestley (1992)

Concrete: Stress-strain relationship

Confined and unconfined concrete subjected to monotonic loads

Mechanics behind confinement

- Concrete expands in the direction orthogonal to the applied compression stress
- Pushes against confining reinforcement

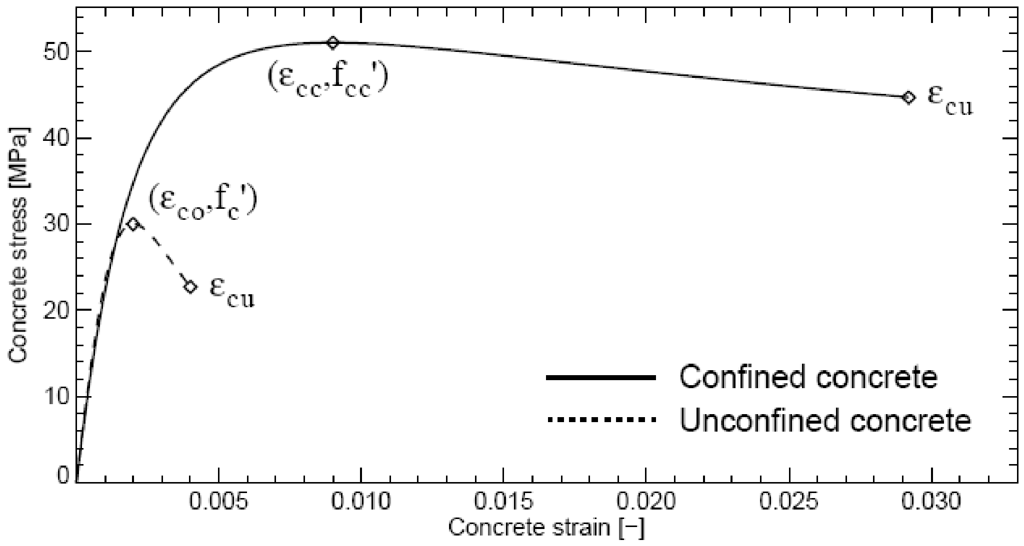
→ Confining reinforcement restrains lateral expansion of concrete

→ Triaxial compression state

→ Strength and deformation capacity of the concrete increase

- Models for estimating f_{cc}', ε_{cc}' et ε_{cu} (Paulay and Priestley (1992), Priestley et al. (1996))

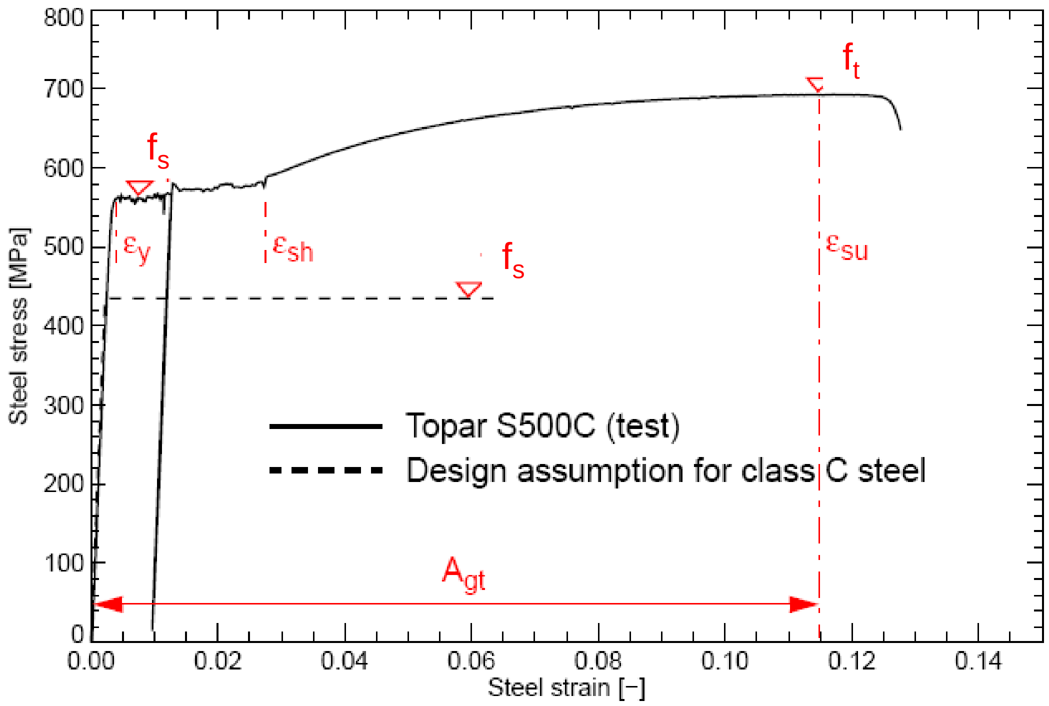
- The tensile strength of concrete is typically neglected.

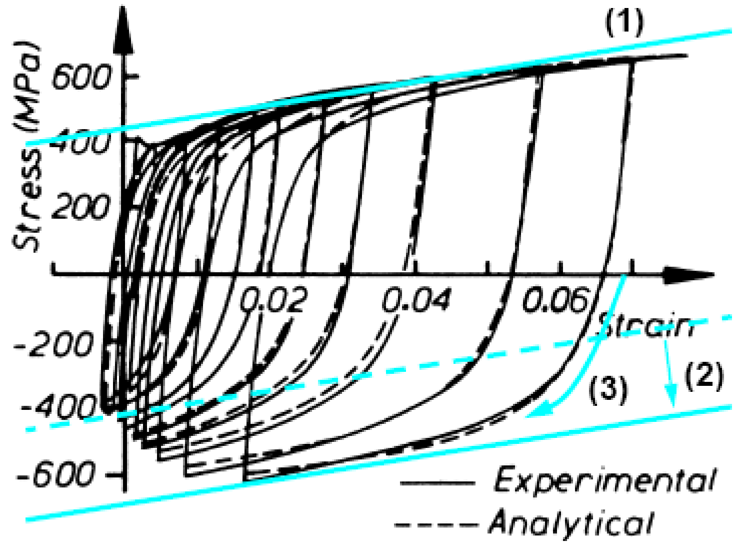


Reinforcement: Stress-strain relationship

2. Reinforcement steel

a) Stress-strain relationship





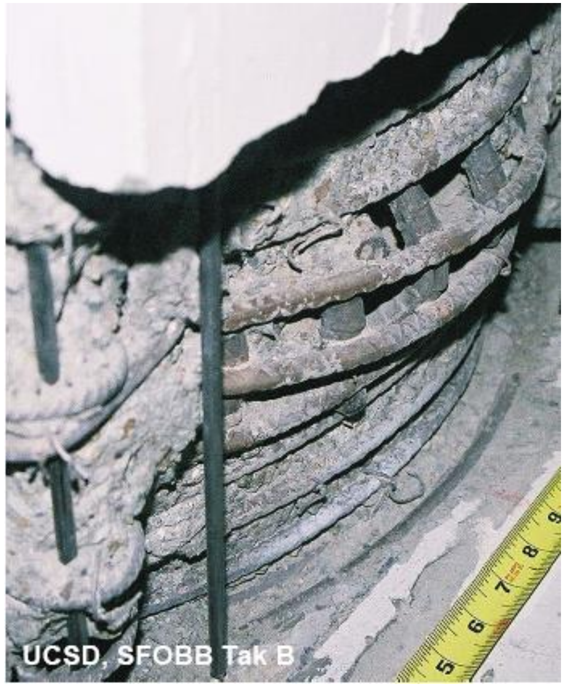
Important properties for the seismic behaviour:
- Strain A_{gt} at maximum strength
- Ratio f_t/f_s

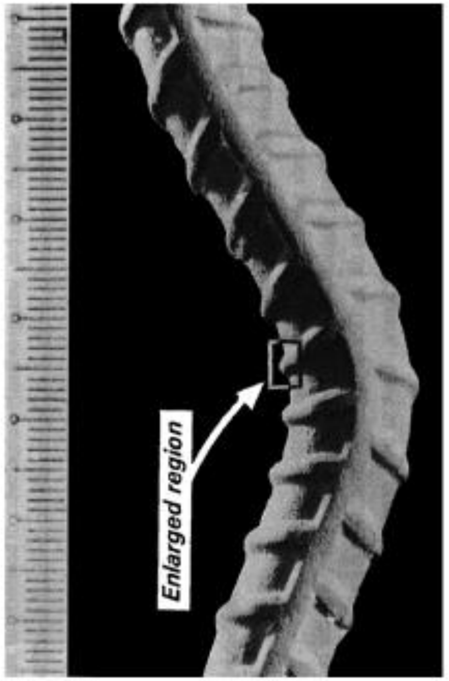
Dr. Igor Tomić

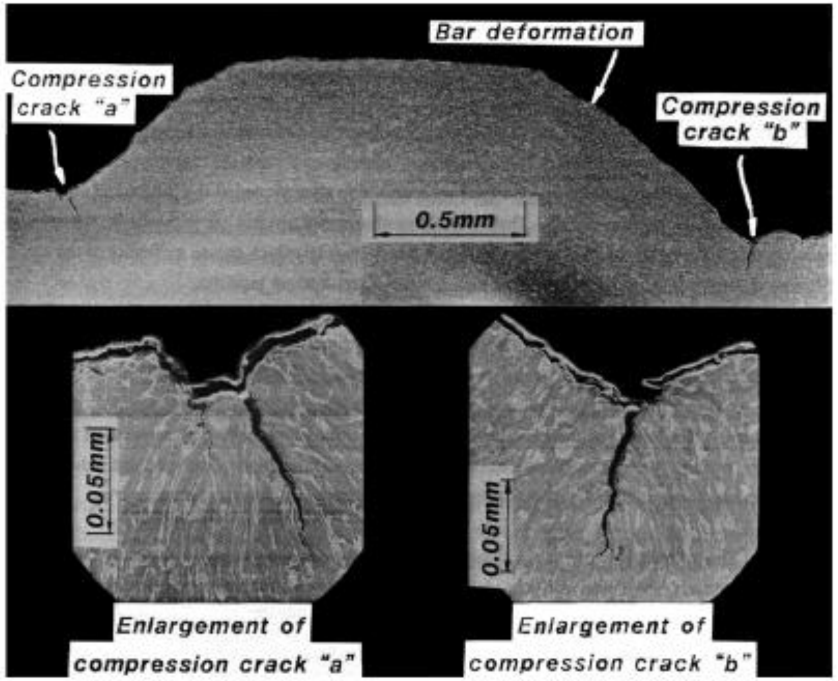
Reinforcement: Stress-strain relationship

Dr. Igor Tomić

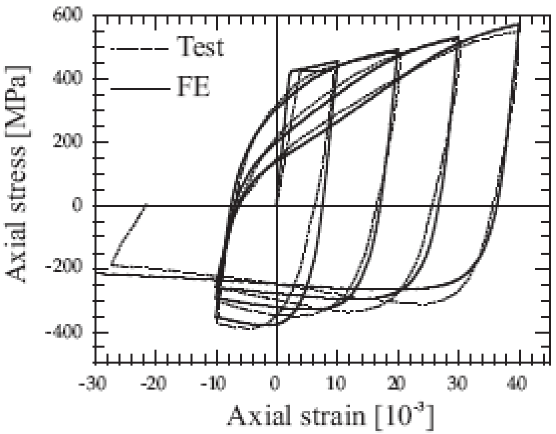
b) Buckling of reinforcing bars







@ Restrepo-Posada (1993)



Rupture mechanism:

- Development of micro-cracks on the compressed side of the bar
- If load is reversed → bar in tension → mirco-cracks propagate and bar fractures

References

Key references:

- Dazio, A. (2005) “Capacité portante des constructions en béton”, SIA Documentation D 0211 “Vérification de la sécurité parasismique des bâtiments existants”. (sur le page web du cours)
- Dazio, A. (2009) , Cours notes for post-graduate course at the University of Stellenbosch.
- Hannewald, P., Bimschas, M., Dazio, A. (2013) “Quasi-static cyclic tests on RC bridge piers with detailing deficiencies,” IBK Report, ETH Zürich, Switzerland.
- Lestuzzi, P. (2011), Cours de génie parasismique, EPFL.
- Arcelor Mittal, Earthquake Resistant Steel Structures, Luxembourg http://www.szs.ch/user_content/editor/files/Downloads_Erdbebensicherheit/earthquake%20resistant%20steel%20structures.pdf
- FEMA 451 , NEHRP recommended provisions: Seismic design of steel structures, Design Examples, Instructional Material Complementing FEMA 451 http://www-classes.usc.edu/architecture/structures/seismic/files/Steel%20Seismic%20Design.pdf

Further references:

- Beyer, K., Dazio, A. and Priestley, M.J.N. (2008) “Quasi-static cyclic tests of two U-shaped reinforced concrete walls,” Journal of Earthquake Engineering 12(7): 1023-1053.
- Bentz E. (2001) “Response-2000 User Manual”. Version 1.1. Department of Civil Engineering, University of Toronto. Canada.
- Dazio, A., Wenk, T. and Bachmann, H. (1999) “Versuche an Stahlbetontragwänden unter zyklisch-statischer Einwirkung”, IBK-Report No 239, Birkhäuser, Zürich, Switzerland.
- Paulay T., Priestley M.J.N. (1992) “Seismic Design of Reinforced Concrete and Masonry Buildings”. John Wiley & Sons, New York, United States.
- Priestley, M.J.N., Seible, F. and Calvi, G.M. (1996) “Seismic design and retrofit of bridges”, John Wiley and Sons, New York, United States.
- Restrepo-Posada J.I. (1993) “Seismic Behaviour of Connections Between Precast Concrete Elements”. Report 93-3. Department of Civil Engineering, University of Canterbury, Christchurch, New Zealand.
- Département Prévention et Intervention (2022) Directive Cantonale sur la sécurité parasismique des bâtiments, Fribourg