Seismic Assessment and Retrofitting of Existing Structures following Eurocode8-Part3 and the Greek Code

Stephanos E. Dritsos, Prof. Emeritus

University of Patras, Greece

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- Elements‘ Behaviour
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- Methods of Analysis
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- Composite Elements

INTRODUCTION

EUROCODES
European Standard (EN) for the Design

EN 1990 Eurocode 0:
Basis of Structural Design

EN 1991 Eurocode 1:
Actions on structures

EN 1992 Eurocode 2:
Design of concrete structures

EN 1993 Eurocode 3:
Design of steel structures

EN 1994 Eurocode 4:
Design of composite steel and concrete structures

EN 1995 Eurocode 5:
Design of timber structures

EN 1996 Eurocode 6:
Design of masonry structures

EN 1997 Eurocode 7:
Geotechnical design

EN 1998 Eurocode 8:
Design of structures for earthquake resistance

EN 1999 Eurocode 9:
Design of aluminium structures

Eurocode 8- Design of structures for earthquake resistance

1: EN1998-1	General rules, seismic actions and rules for buildings
2: EN1998-2	Bridges
3: EN1998-3	Assessment and retrofitting of buildings
4: EN1998-4	Silos, tanks and pipelines
5: EN1998-5	Foundations, retaining structures and geotechnical aspects
6: EN1998-6	Towers, masts and chimneys

CODE ENVIRONMENT

EUROPE

U.S.A.

1983 — **CEB Bul. No. 162**, "Assessment of Concrete Structures and Design Procedures for Upgrading (Redesign)".

1995 — **EC 8-Part 1.4,** "Eurocode 8: Design Provisions for Earthquake Resistance of Structures: Part 1-4: Strengthening and Repair of Buildings"

1996 — **ATC 40.** "Seismic Evaluation and Retrofit of Concrete Buildings".

2000 — **FEMA 356.** "Prestandard and Commentary for the Seismic Rehabilitation of Buildings".

2003 — **fib Bul.No24**, "Seismic Assessment and Retrofit of Reinforced Concrete Buildings".

2005 — **EC 8-Part3,** "Eurocode 8: Design of Structures for Earthquake Resistance. Part 3: Assessment and Retrofitting of Buildings". Draft No 5.

2006 — **GCSI**, "Greek Code of Structural Interventions".

2007 — **ASCE/SEI 41**, ASCE Standards Seismic Rehabilitationof Existing Buildings.

2008 — **ASCE/SEI 41**, Supplement1, Update ASCE/SEI 41.

2012 — **GCSI,** Draft

WEAKNESSES OF EXISTING OLD STRUCTURES UNDER SEISMIC ACTIONS

(a) The structural system of many old buildings was designed with architectural excesses. Lack of regularity (geometry, strength or stiffness) in plan or in elevation.

(b) A number of approximations and simplifications were adopted in the analysis. Computers were not in use, 3D analysis was impossible, 2D rarely used. Beams and columns were considered independent elements.

(c) Critical matters concerning the behaviour of structures under earthquake actions were ignored.

- Ductility
- Capacity design
- Inadequate code provisions for detailing of concrete elements (minimum stirrups,lower limit for compressive reinforcement, upper limit for tensile reinforcement)

(d) Design for seismic actions much lower than that now accepted for new structures.

ESTIMATED SEISMIC CAPACITY OF CONCRETE BUILDINGS:
OLD/NEW $\simeq$ 1/3

QUESTIONS

- Which structures have the priority to be strengthened and how to identify them?

- Is it possible (or is it worth) strengthening these structures and to what extent? Is this preferable when compared to the demolition and reconstruction solution?

- What resources (materials, methods, techniques) are available to intervene and under what standards are they to be applied?

- Which is the best method of intervention in a specific structure?

- Which is the design framework to assess the seismic capacity of an existing structure and document choices for retrofitting or strengthening?

- What are the quality control procedures for intervention works?

REDESIGN → A MUCH MORE COMPLICATED ISSUE THAN THE DESIGN OF NEW STRUCTURES

- Limited knowledge, poorly documented for the subject
- Lack of codes or other regulations
- The configuration of the structural system of an existing structure may not be permitted. However it exists
- High uncertainty in the basic data of the initial phase of documentation. Hidden errors or faults
- Use of new materials which are still under investigation!
- Low (or negative) qualifications or experience of workmanship

Why we need a new design framework in addition to the existing one for new structures?

Existing Structures:

(a) Reflect the state of knowledge at the time of their construction

(b) May contain hidden gross errors

(c) May have been stressed in previous earthquakes (or other accidental actions) with unknown effects

→ Structural assessment and redesign of an existing structure due to a structural intervention are subjected to a different degree of uncertainty than the design of a new structure → **Different material and structural safety factors** are required

→ **Different analysis procedures** may be necessary depending on the completeness and reliability of available data

Usually, analytical procedures (or software) used for the design of new structures are not suitable to assess existing structures. New structures designed according to new codes necessarily fulfil specific code requirements for being analysed acceptably with conventional analytical procedures, e.g. linear elastic analysis

THREE MAIN OBJECTIVES

- Assess the seismic capacity of an existing structure
- Decide the necessary intervention work
- Design the intervention work

ASSESSMENT PROCEDURE

1st stage

Document the existing structure

2nd stage

Assessment of the (seismic) capacity of the structure

3rd stage

Decide if structural intervention required

4th stage

Design the structural intervention

5th stage

Construct the intervention work

Design in progress

PERFORMANCE LEVELS
OR
DAMAGE LEVELS

What is failure?

Action effects > Resistance

- **Distinguishing elements for "Ductile" and "Brittle"**

Brittle: Verified in terms of forces (known as M, N, V)
Ductile: Verified in terms of deformation

Let $M_{Rd} = 150\,KNm < M_{sd} = 200\,KNm$

In a study of a new building this is never accepted

However in an existing building this is very possible to occur

Questions: What level of damage will there be?
What are the consequences?
Is this acceptable?

Damage Levels

Performance Levels or Limit States (LS)

LS Level A Limitation Damage (DL)

➡ **Immediate Occupancy** (other Codes e.g. FEMA): Minimal damage, elements have not substantially yielded

LS Level B of Significant Damage (SD)

➡ **Life Safety** (other Codes e.g. FEMA): Building with serious damage accepted as the design of new buildings

LS Level C of Near Collapse (NC)

➡ **Collapse prevention** (other Codes e.g. FEMA): Extensive and serious or severe damage, building is very close to collapse

PERFORMANCE LEVELS

Acceptable **Performance** Levels or **Level of Protection** (e.g. **State of Damage**) of the Structure:

Level A: Immediately Occupancy (IO) or Damage Limitation (DL)

- Very light damage
- Structural elements retain their strength and stiffness
- No permanent drifts
- No significant cracking of infill walls
- Damage could be economically repaired

Level B: Life Safety (LS) or Significant Damage (SD)

- Significant damage to the structural system however retention of some lateral strength and stiffness
- Vertical elements capable of sustaining vertical loads
- Infill walls severally damaged
- Moderate permanent drifts exist
- The structure can sustain moderate aftershocks
- The cost of repair may be high. The cost of reconstruction should be examined as an alternative solution

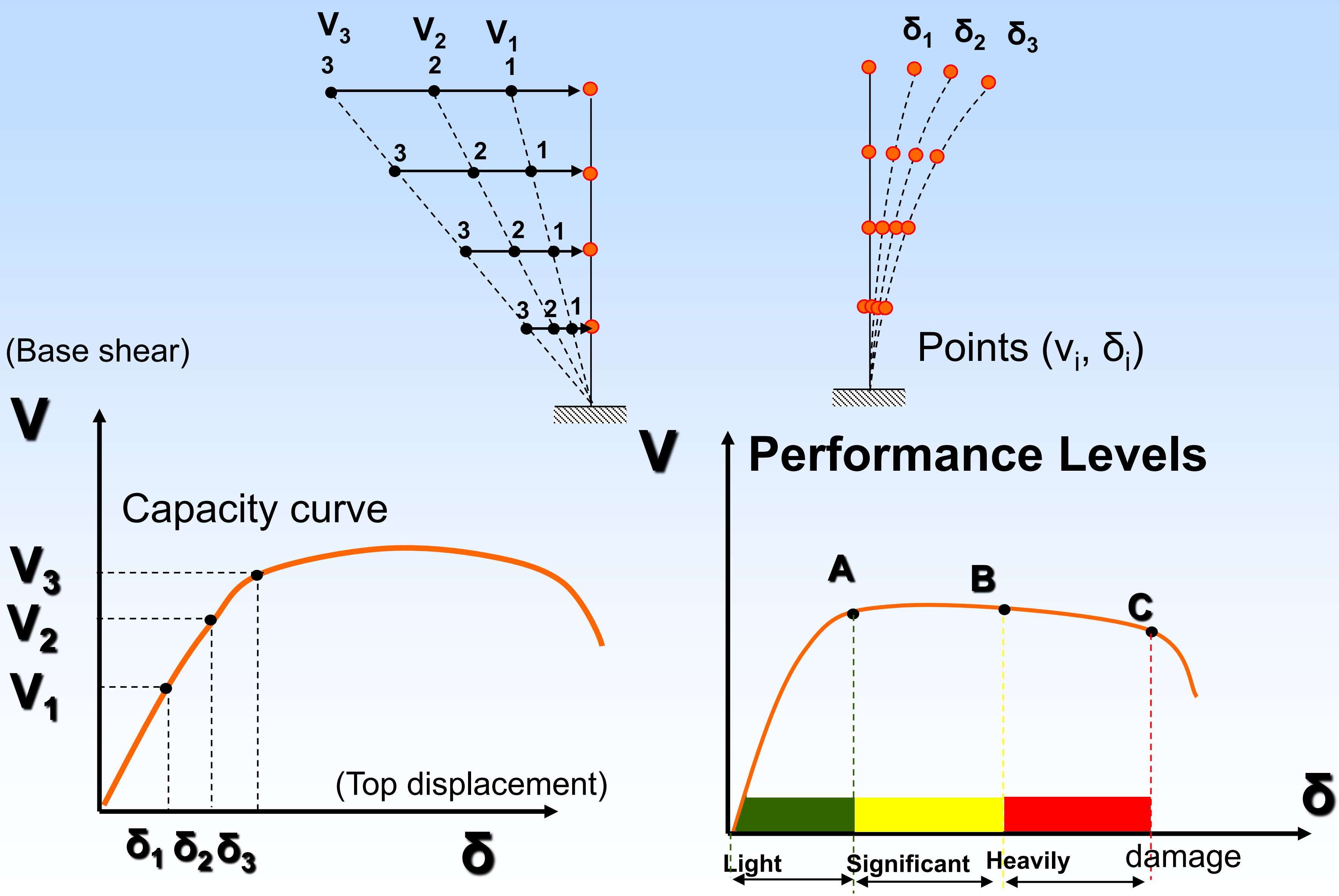
PERFORMANCE LEVELS

Level C: Collapse Prevention (CP) or Near Collapse (NP)

- Structure heavily damaged with low lateral strength and stiffness
- Vertical elements capable of sustaining vertical loads
- Most non-structural components have collapsed
- Large permanent drifts
- Structure is near collapse and possibly cannot survive a moderate aftershock
- Uneconomical to repair. Reconstruction the most probable solution

PERFORMANCE LEVELS

Gradual pushing (static horizontal loading) of structure up to failure



SEISMIC ACTIONS

What is the design seismic action?
Which return period should be selected for the seismic action?
Should this be the same as for new structures?

Design Levels

Occurrence probability in 50 years	Collapse prevention (CP)	Life safety (LS)	Immediately occupancy (IO)
2% Return period 2475 years	$CP_{2\%}$	$LS_{2\%}$	$DL_{2\%}$
10% Return period 475 years	$CP_{10\%}$	$LS_{10\%}$	$DL_{10\%}$
20% Return period 225 years	$CP_{20\%}$	$LS_{20\%}$	$DL_{20\%}$
50% Return period 70 years	$CP_{50\%}$	$LS_{50\%}$	$DL_{50\%}$

- (green) Usual design of new buildings
- (yellow) Design of important structures (remain functional during earthquake)
- (red) Minimum acceptable seismic action level
  Instead, do nothing due to economic, cultural, aesthetic and functional reasons

Performance Levels according to the Greek Code of Structural Interventions (Greek.C.S.I.)

Seismic activity probability of exceedance in the conventional design life of 50 years	Minimal Damage (Immediate Occupancy)	Severe Damage (Life Safety)	Collapse Prevention
10% (Seismic actions according to EK8-1)	**A1**	**B1**	**Γ1**
50% (Seismic actions = 0.6 x EK8-1)	**A2**	**B2**	**Γ2**

The public authority defines when the 50% probability is not permitted

ELEMENT’S BEHAVIOUR

ELEMENT BEHAVIOR

Ductile

Flexure controlled

$$S_d \leq R_d$$

deformation demand — deformation capacity

Brittle

Shear controlled

$$S_d \leq R_d$$

strength demand — strength capacity

Seismically Primary

Seismically Secondary

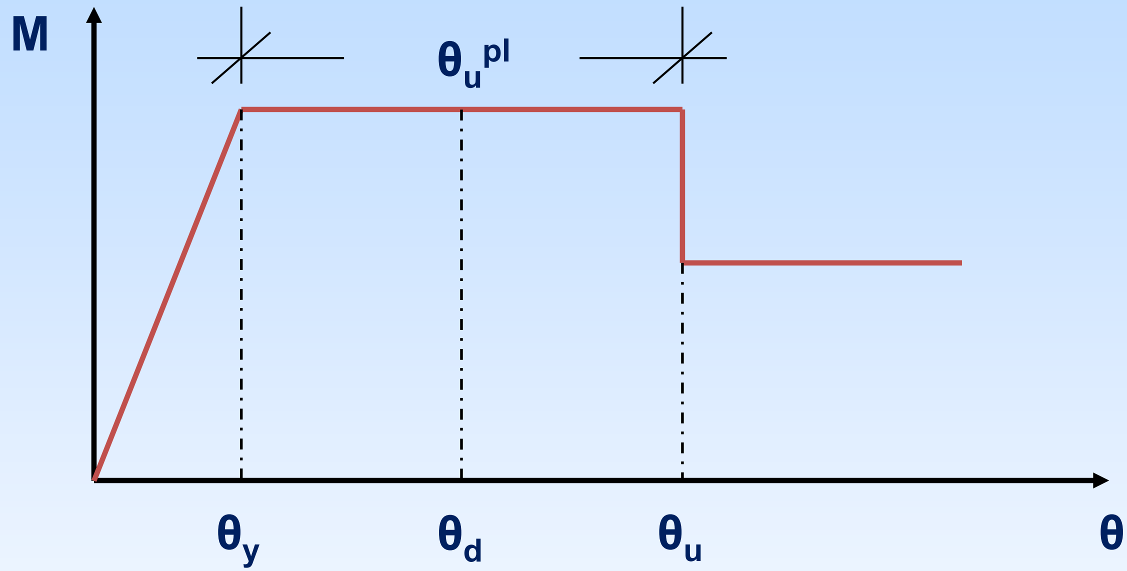
"Secondary" seismic element

- More damage is acceptable for the same Performance Level
- Considered not participating in the seismic action resisting system. Strength and stiffness are neglected
- Able to support gravity loads when subjected to seismic displacements

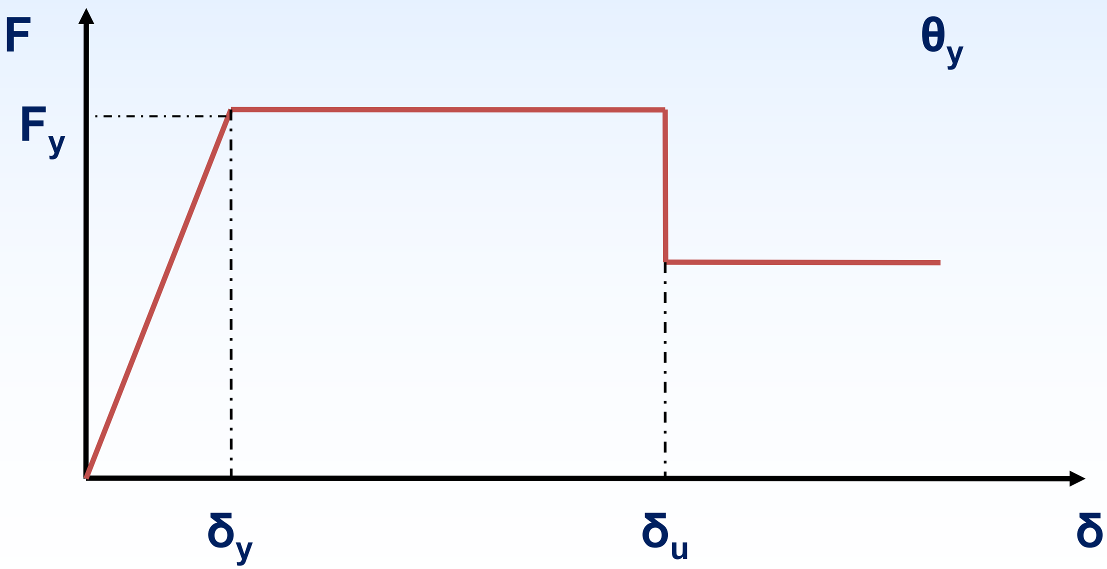
REINFORCED CONCRTETE STRUCTURES

Element's Capacity Curve

$$m = \frac{\theta_d}{\theta_y}$$



$$K = EI_{ef} = \frac{M_y \cdot L_s}{3\theta_y}$$



$$K = \frac{F_y}{\delta_y}$$

Element's Capacity

Chord rotation at yielding of a concrete element

$$\theta_y = (1/r)_y \frac{L_s + a_V z}{3} + 0{,}0014\left(1 + 1{,}5\frac{h}{L_s}\right) + \frac{(1/r)_y d_b f_y}{8\sqrt{f_c}}$$

Beams and columns

$$\theta_y = (1/r)_y \frac{L_s + a_V z}{3} + 0{,}0013 + \frac{(1/r)_y d_b f_y}{8\sqrt{f_c}}$$

Walls of rectangular, T- or barbell section

The value of the total chord rotation capacity of concrete elements under cyclic loading

$$\theta_{um} = 0{,}016 \cdot (0{,}3^{\nu})\left[\frac{\max(0{,}01;\ \omega')}{\max(0{,}01;\ \omega)} f_c\right]^{0.225} (\alpha_s)^{0{,}35}\ 25^{\left(\alpha\rho_s \frac{f_{yw}}{f_c}\right)} (1{,}25^{100\,\rho_d})$$

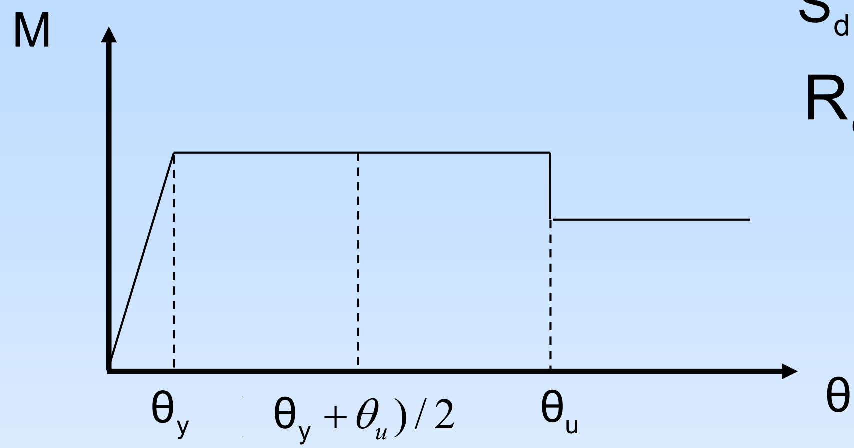
The value of the plastic part of the chord rotation capacity of concrete elements under cyclic loading

$$\theta_{um}^{pl} = \theta_u - \theta_y = 0{,}0145\,(0{,}25^{\nu})\left[\frac{\max(0{,}01;\omega')}{\max(0{,}01;\omega)}\right]^{0{,}3} (f_c)^{0{,}2} (\alpha_s)^{0{,}35}\ 25^{\left(\alpha\rho_s \frac{f_{yw}}{f_c}\right)} \left(1{,}275^{100\rho_d}\right)$$

ELEMENT'S SAFETY VERIFICATION

Inequality of Safety

$$S_d \leq R_d$$



S_d is the design action effect

R_d is the design resistance

For brittle components/mechanisms (e.g. shear) S_d, R_d **concern forces**

For ductile components/mechanisms (e.g. flexural) S_d, R_d **concern deformations,** θ_{sd}, θ_{Rd}

(G.S.I. Code)

A Level (IO) $\quad \theta_{Rd} = \theta_y$

B Level (LS) $\quad \theta_{Rd} = \dfrac{1}{\gamma_{Rd}} \dfrac{\theta_y + \theta_u}{2}$ "primary" elements $\gamma_{Rd} = 1,8$

$\theta_{Rd} = \dfrac{\theta_u}{\gamma_{Rd}}$ "secondary" elements $\gamma_{Rd} = 1,8$

C Level (NC) $\quad \theta_{Rd} = \dfrac{\theta_u}{\gamma_{Rd}}$

$\gamma_{Rd} = 1,8$ for "primary" elements

$\gamma_{Rd} = 1,0$ for "secondary" elements

ELEMENT'S SHEAR CAPACITY

Beams and Columns

$$V_R = \frac{h-x}{2L_s}\min(N;\ 0{,}55A_c f_c) + \left(1-0{,}05\min\left(5,\ \mu_\theta^{pl}\right)\right)\left[0{,}16\max(0{,}5;\ 100\rho_{tot})(1-0{,}16\min(5;\ \alpha_s))\sqrt{f_c}A_c + V_w\right]$$

$$V_w = \rho_w b_w z f_{yw}$$

rectangular web cross section

$$V_w = \frac{\pi}{2}\frac{A_{sw}}{s} f_{yw}(D-2c)$$

circular cross section

Shear Walls

$$V_{R,max} = 0{,}85\left(1-0{,}06\min\left(5;\ \mu_\theta^{pl}\right)\right)\left(1+1{,}8\min\left(0{,}15;\frac{N}{A_c f_c}\right)\right)(1+0{,}25\max(1{,}75;100\rho_{tot}))(1-0{,}2\min(2;a_s))\sqrt{f_c}\,b_w z$$

Short Columns (*L*V/*h*)≤2

$$V_{R,max} = 4/7\left(1-0{,}02\min\left(5;\ \mu_\theta^{pl}\right)\right)\left(1+1{,}35\frac{N}{A_c f_c}\right)(1+0{,}45(100\rho_{tot}))\sqrt{\min(40;f_c)}\,b_w z\sin 2\delta$$

DOCUMENTATION

ASSESSMENT PROCEDURE

1st stage

Document the existing structure

2nd stage

Assessment of the (seismic) capacity of the structure

3rd stage

Decide if structural intervention required

4th stage

Design the structural intervention

5th stage

Construct the intervention work

Design in progress

Documentation of an Existing Structure

- Strength of materials
- Reinforcement
- Geometry (including foundation)
- Actual loads
- Past damage or “wear and tear” or defects

→ Knowledge Levels (KL)

→ Confidence factors (Other safety factors for existing materials and elements)

→ New safety factors for new materials

Knowledge Levels (KL)

- Full Knowledge → KL3
- Normal Knowledge → KL2
- Limited Knowledge → KL1
- Inadequate: May allowed only for secondary elements

DOCUMENTATION

Knowledge Levels and Confidence Factors

KL_1: Limited Knowledge

KL_2: Normal Knowledge

KL_3: Full Knowledge

Knowledge Level	Geometry	Details	Materials	Analysis	CF
KL1	From original outline construction drawings with sample **visual** survey ***or*** from **full** survey	Simulated design in accordance with relevant practice ***and*** from **limited** *in-situ* inspection	Default values in accordance with standards of the time of construction ***and*** from **limited** *in-situ* testing	LF-MRS	CF_{KL1} = 1.35
KL2		From incomplete original detailed construction drawings with **limited** *in-situ* inspection ***or*** from **extended** in-situ inspection	From original design specifications with **limited** *in-situ* testing ***or*** from **extended** *in-situ* testing	All	CF_{KL2} = 1.20
KL3		From original detailed construction drawings with **limited** *in-situ* inspection ***or*** from **comprehensive** *in-situ* inspection	From original test reports with **limited** *in-situ* testing ***or*** from **comprehensive** *in-situ* testing	All	CF_{KL3} = 1.00

Knowledge Levels (KL) for Materials Data

Concrete (G.C.S.I.)

- Assessment methods f_c:
  - Combination of indirect (non-destructive) methods.
  - Calibrate with destructive methods involving taking samples (e.g. cores).
  - Pay attention to correct correlation between destructive and non-destructive methods.
  - Final use of calibrated non-destructive methods throughout the structure
- Required number of specimens
  - Not all together, i.e. spread out over all floors and all components
  - At least 3 cores per alike component per two floors, definitely for the "critical" floor level
- Additional methods (acoustic or Schmidt Hammer or extrusion or rivet for $f_c < 15$ MPa)
  - Full knowledge/storey: 45% vertical elements/25% horizontal elements
  - Normal knowledge/storey: 30% vertical elements/25% horizontal elements
  - Limited knowledge/storey: 15% vertical elements/7.5% horizontal elements

Steel

Visual identification and classification is allowed. In this case, the KL is considered KL_2

Knowledge Levels for Details Data

- **Data Sources:**

1. Data from the original study plans that has proof of implementation
2. Data from the original study plans which has been implemented with a few modifications identified during the investigation
3. Data from a reference statement (legend) in the original study plan
4. Data that has been established and/or measured and/or acquired reliably
5. Data that has been determined indirectly
6. Data that has been reasonably obtained from engineering judgement

Knowledge Levels for Details Data (G.C.S.I.)

ORIGINAL DESIGN DRAWINGS		DATA ORIGIN		NOTES	DATA								
					TYPE AND GEOMETRY OF FOUNDATION OR SUPERSTRUCTURE			THICKNESS, WEIGHT etc. OF INFILL WALLS, CLADDING, COVERING, etc.			REINFORCEMENT LAYOUT AND DETAILING		
Exist	Do not exist				KL1	KL2	KL3	KL1	KL2	KL3	KL1	KL2	KL3
✓		1	Data that is derived from a drawing of the original design that is proved to have been applied without modification	(1)			✓			✓			✓
✓		2	Data that is derived from a drawing of the original design that has been applied with few modifications	(2)			✓			✓		✓	
✓		3	Data that is derived from a reference (e.g. legend in a drawing of the original design)	(3)	✓			✓			✓		
	✓	4	Data that has been determined and/or measured and/or surveyed reliably	(4)		✓			✓			✓	
	✓	5	Data that has been determined by an indirect but sufficiently reliable manner	(5)	✓			✓			✓		
	✓	6	Data that has been reasonably assumed using the Engineer's judgment	(6)	✓	✓		✓	✓		✓	✓	

METHODS OF ANALYSIS

METHODS OF ANALYSIS

In Redesign other analysis methods are required

Elastic analysis methods, currently in use (for new buildings), can provide realiable results, since specific code conditions are followed in design.

In most cases, these conditions are not met in the old buildings.

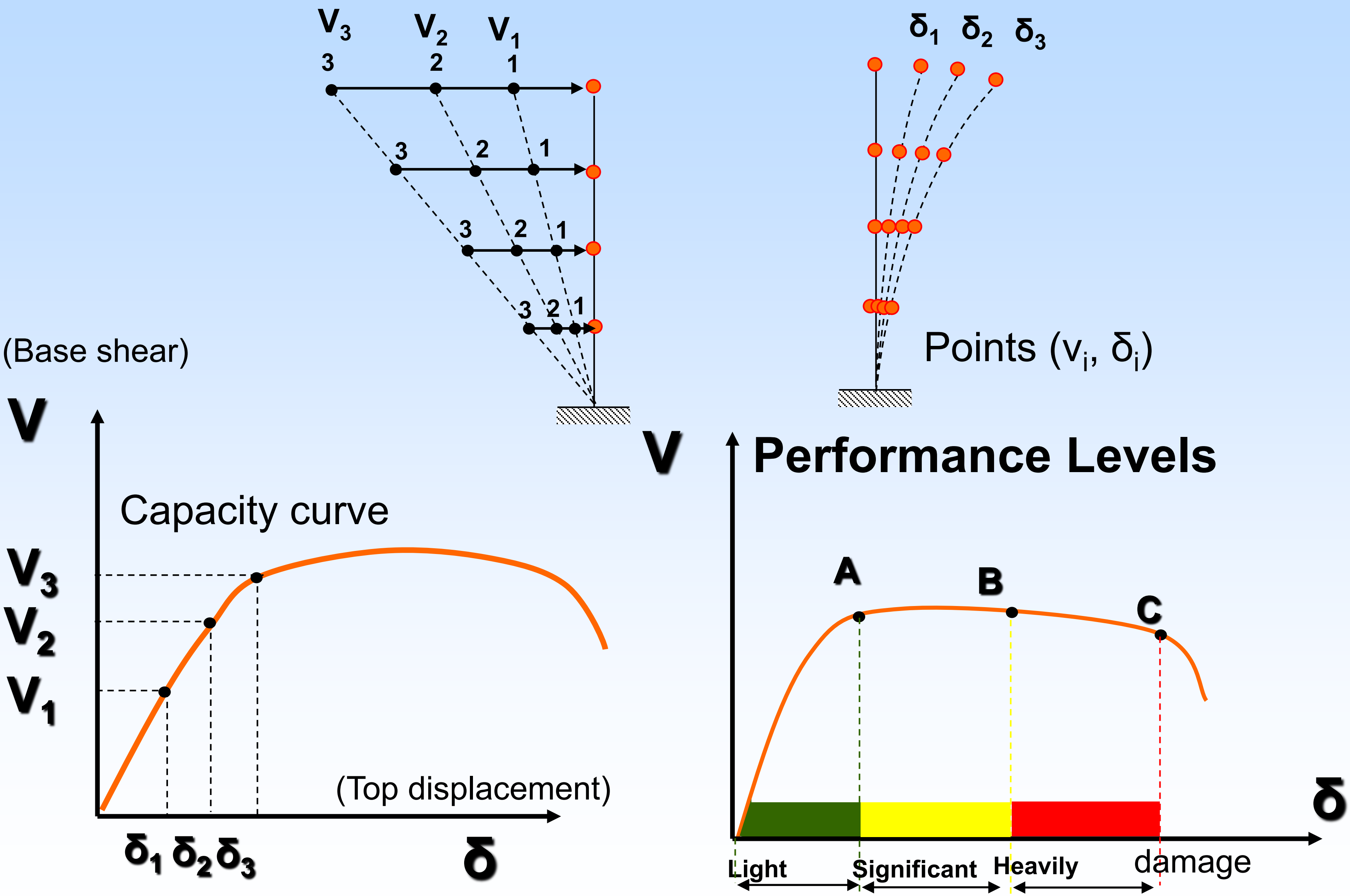
METHODS OF ANALYSIS

Same as those used for design new construction (EC8-Part 1)

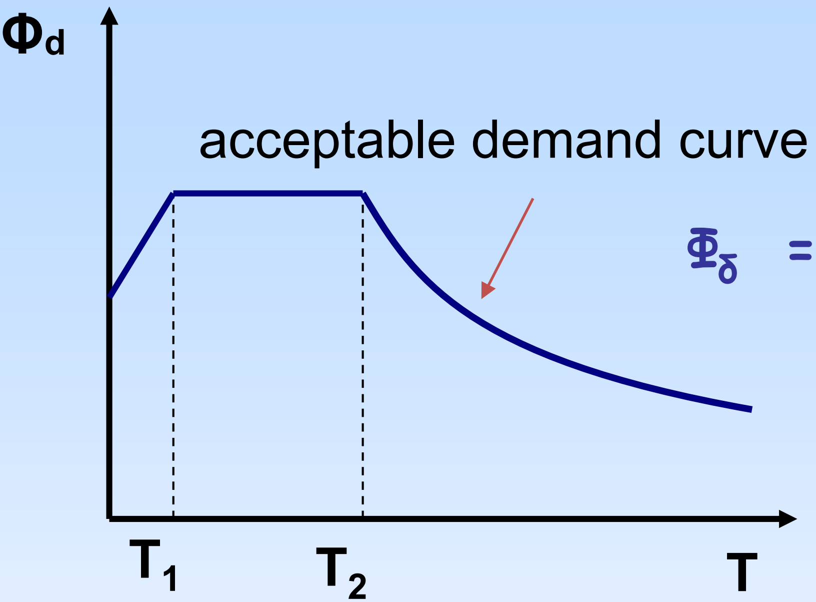
- Lateral force analysis (linear)
- Modal response spectrum analysis (linear)
- Non-linear static (pushover) analysis
- Non-linear time history dynamic analysis
- q-factor approach

PERFORMANCE LEVELS

Gradual pushing (static horizontal loading) of structure up to failure

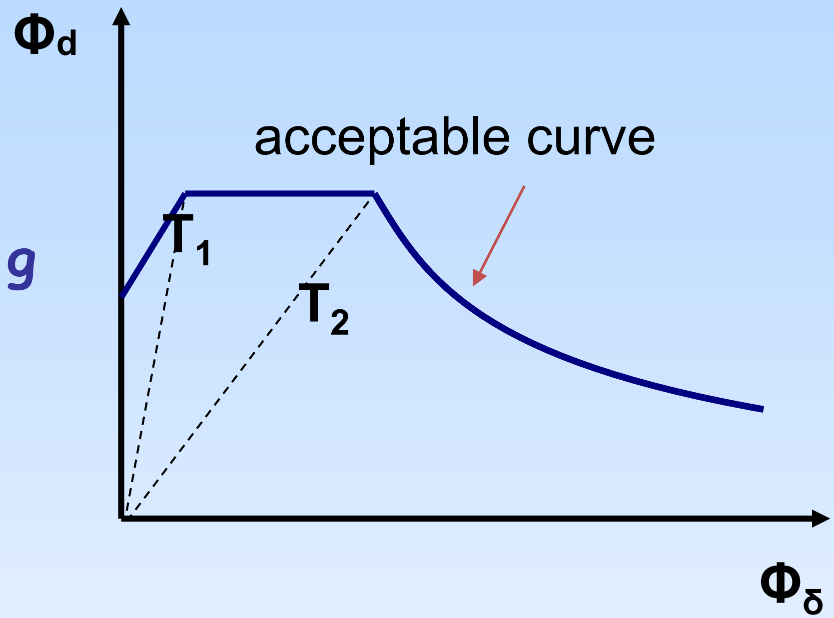


CAPACITY DEMAND



code elastic spectrum

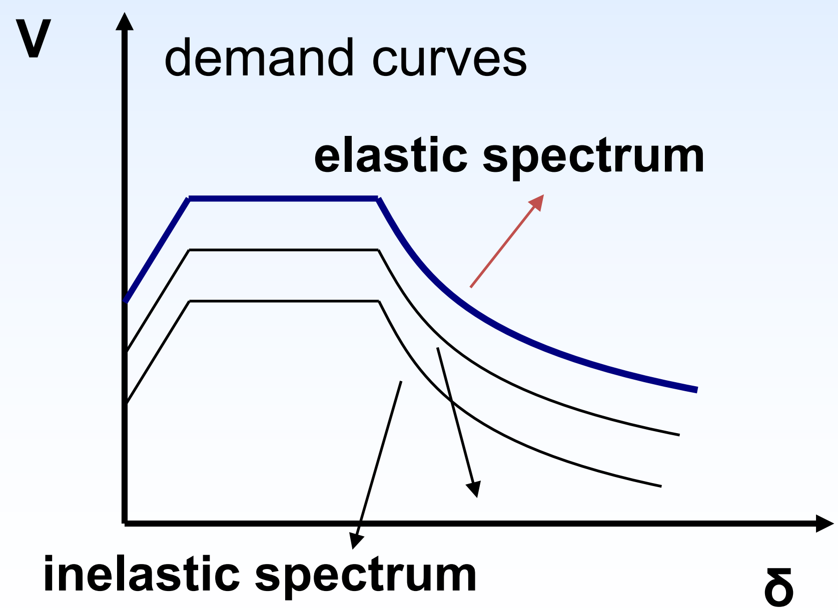
$$\Phi_\delta = \frac{T^2}{4\pi^2}\,\Phi_d\, g$$



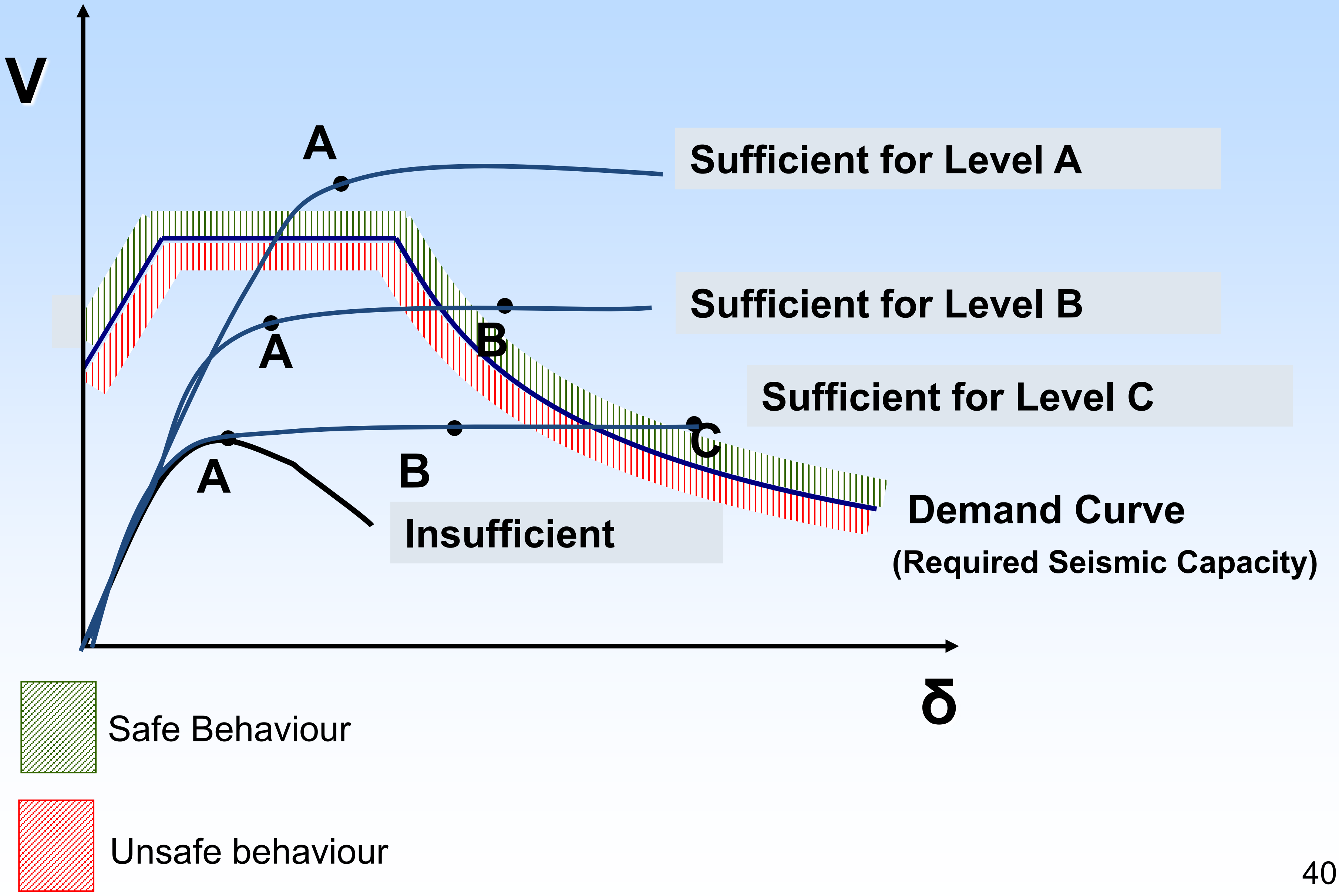
$$V = \alpha\ \Phi_d\ W$$

$$\delta = \beta\ \Phi_\delta$$

n	α	β
1	1	1
2	0.90	1.20
5	0.80	1.35

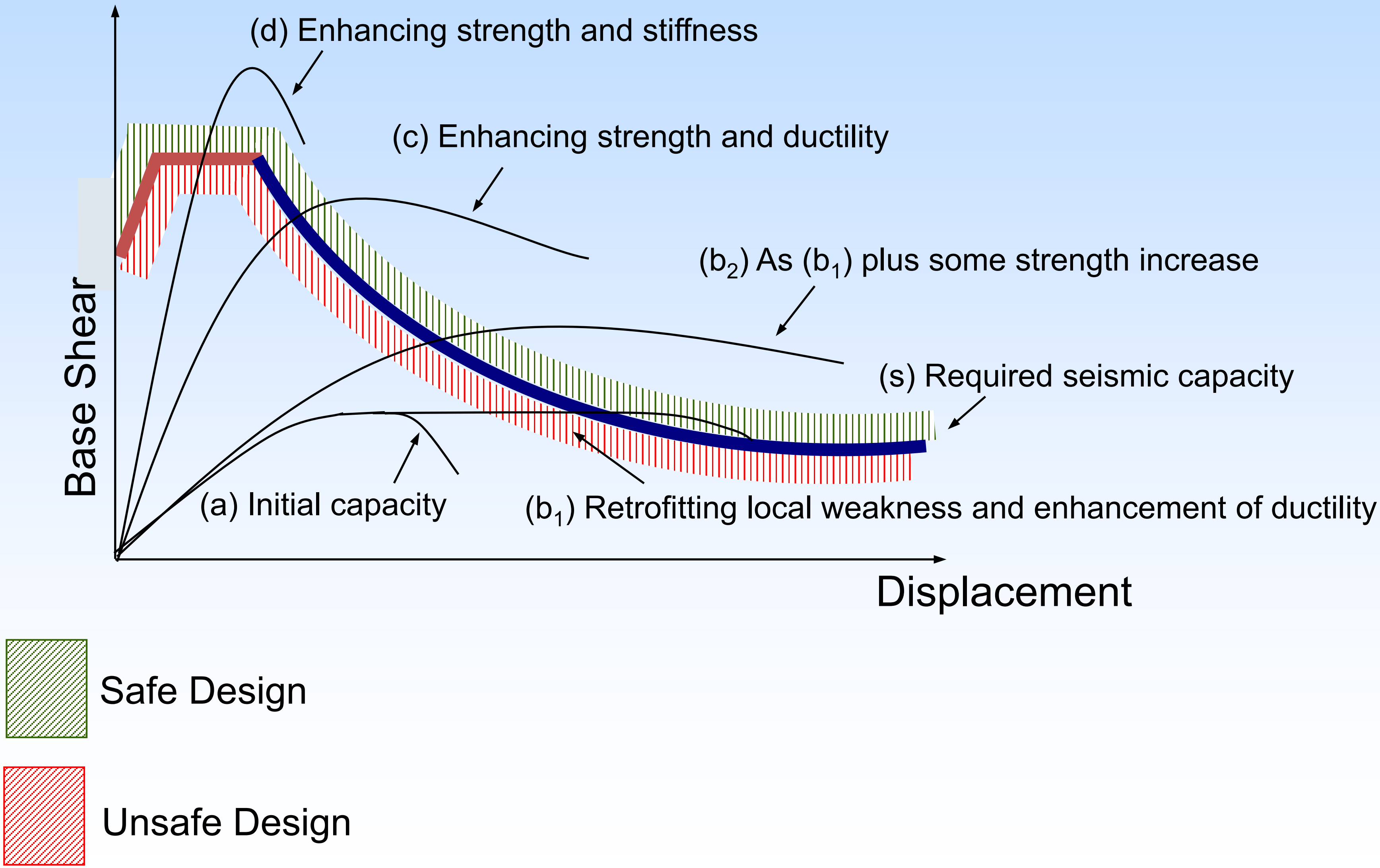


SAFETY VERIFICATION
Checking a Structure's Capacity

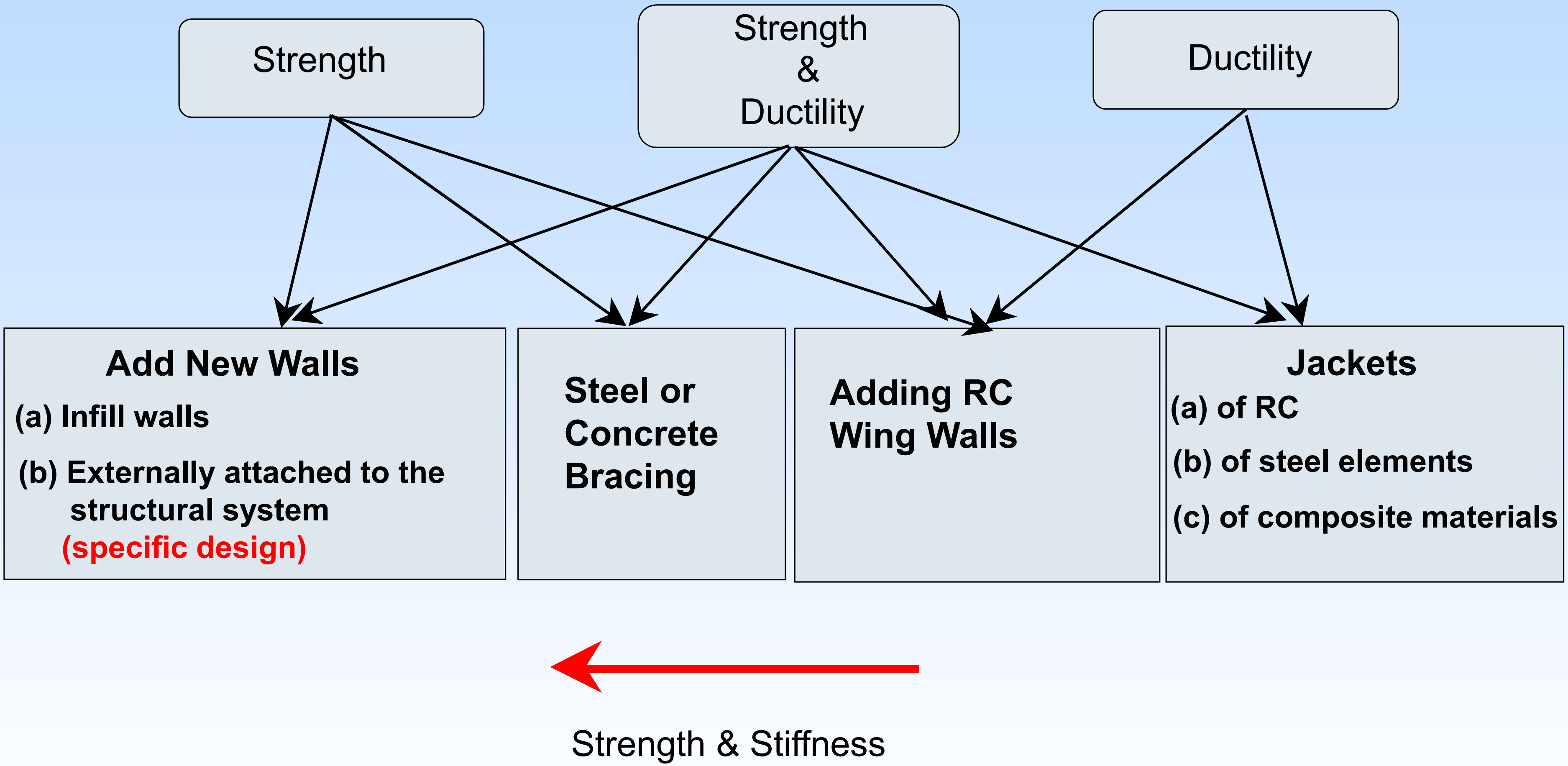


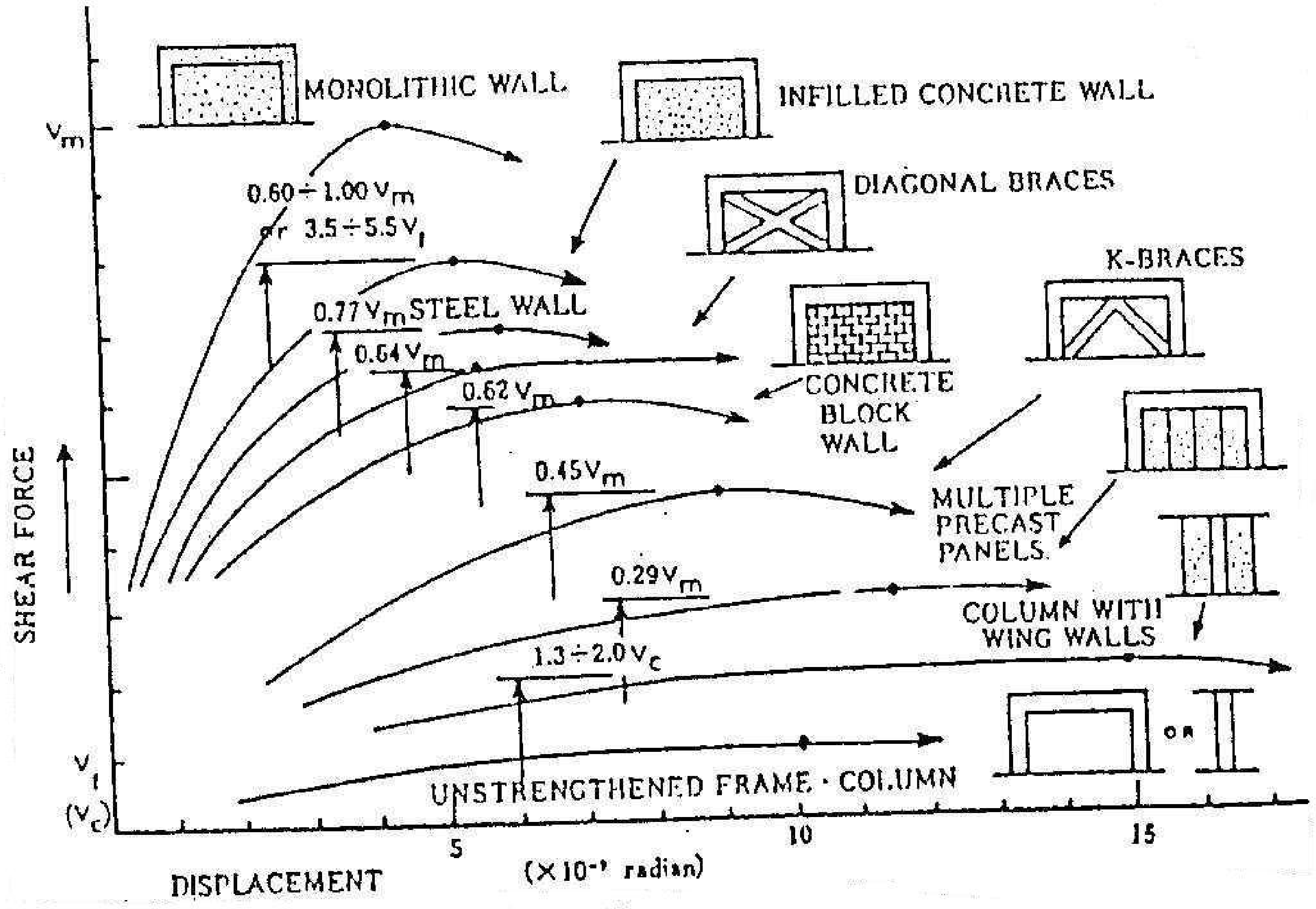
Seismic Strengthening Strategies
Methods of Strengthening the Whole Structure

SEISMIC STREGHTENING STRATEGIES



SEISMIC STRENGHTENIG METHODS





The relative effectiveness of strengthening

Adding Simple Infill

- Addition of walls from: a) Unreinforced or reinforced concrete
  (cast in situ or prefabricated)
  b) Unreinforced or reinforced masonry
- No specific requirement to connect infill to the existing frame
- Modelling of infills by diagonal strut
- Low ductility of infill. Recommended $m \leq 1{,}5$

WARNING

Additional shear forces are induced in the columns and beams of the frame

Strengthening of existing masonry infills

- Reinforced shotcrete concrete layers applied to both sides of the wall

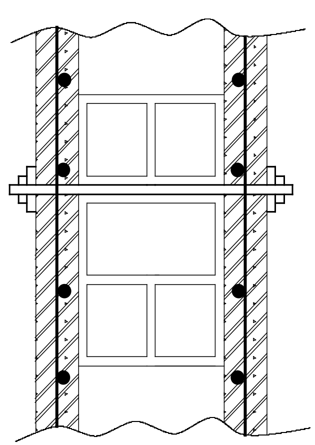
  Minimum concrete thickness 50 mm

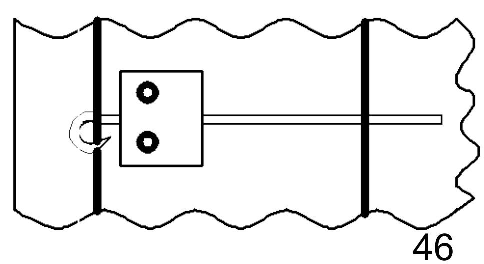
  Minimum reinforcement ratio $\rho_{vertical} = \rho_{horizontal} = 0,005$

  Essential to positively connect both sides by bolting through the wall

  No need to connect to existing frame as it is an infill

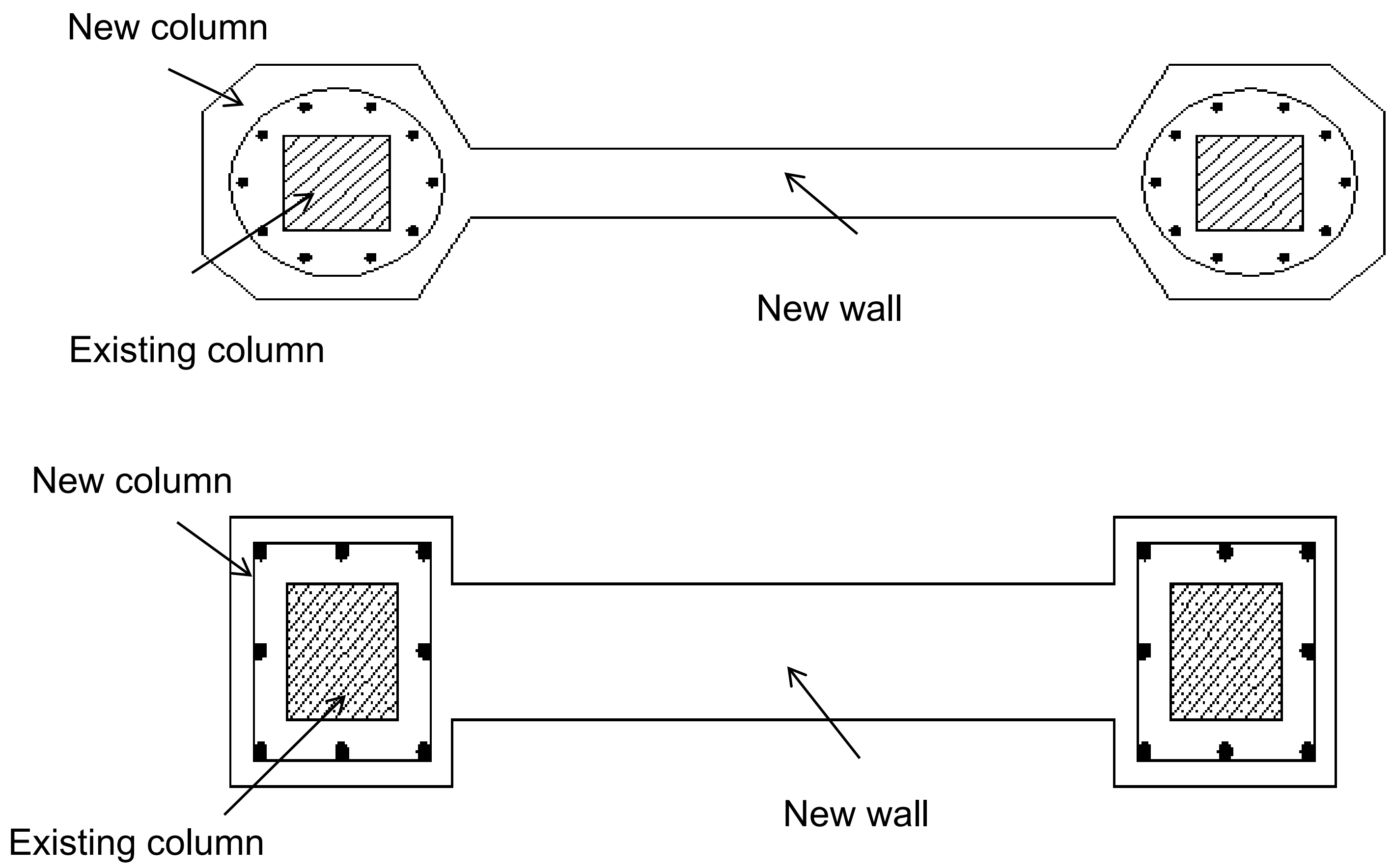
  All new construction must be suitably connected to the existing foundation

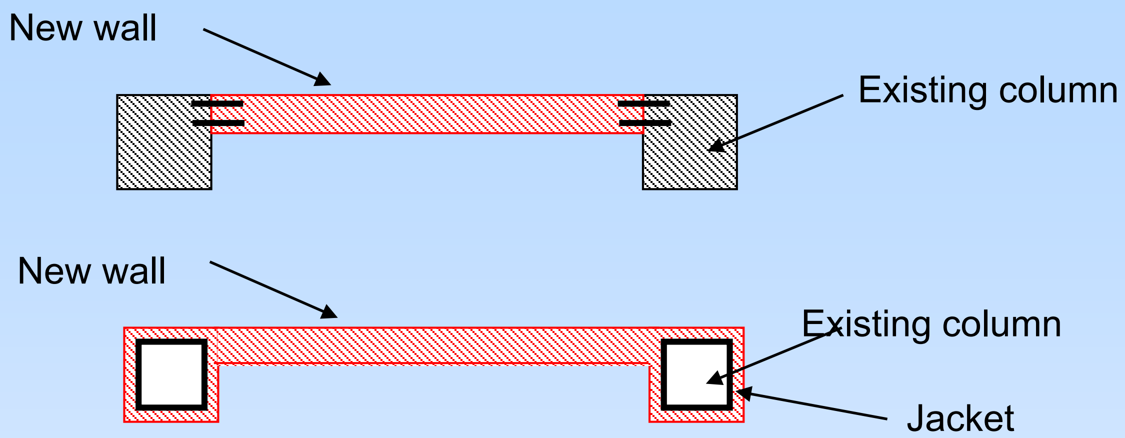




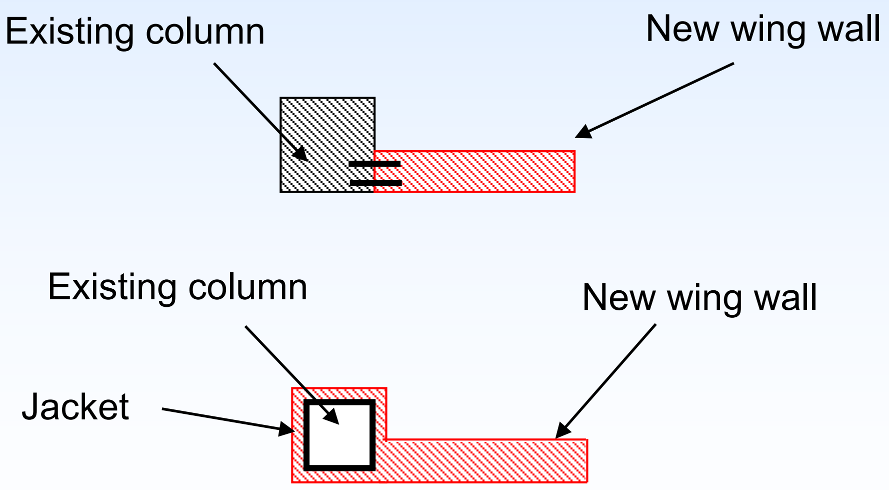
Frame Encasement

Reinforced walls are constructed from one column to another enclosing the frame (including the beam) with jackets placed around the columns. Note, all new construction must be suitably connected to the existing foundation

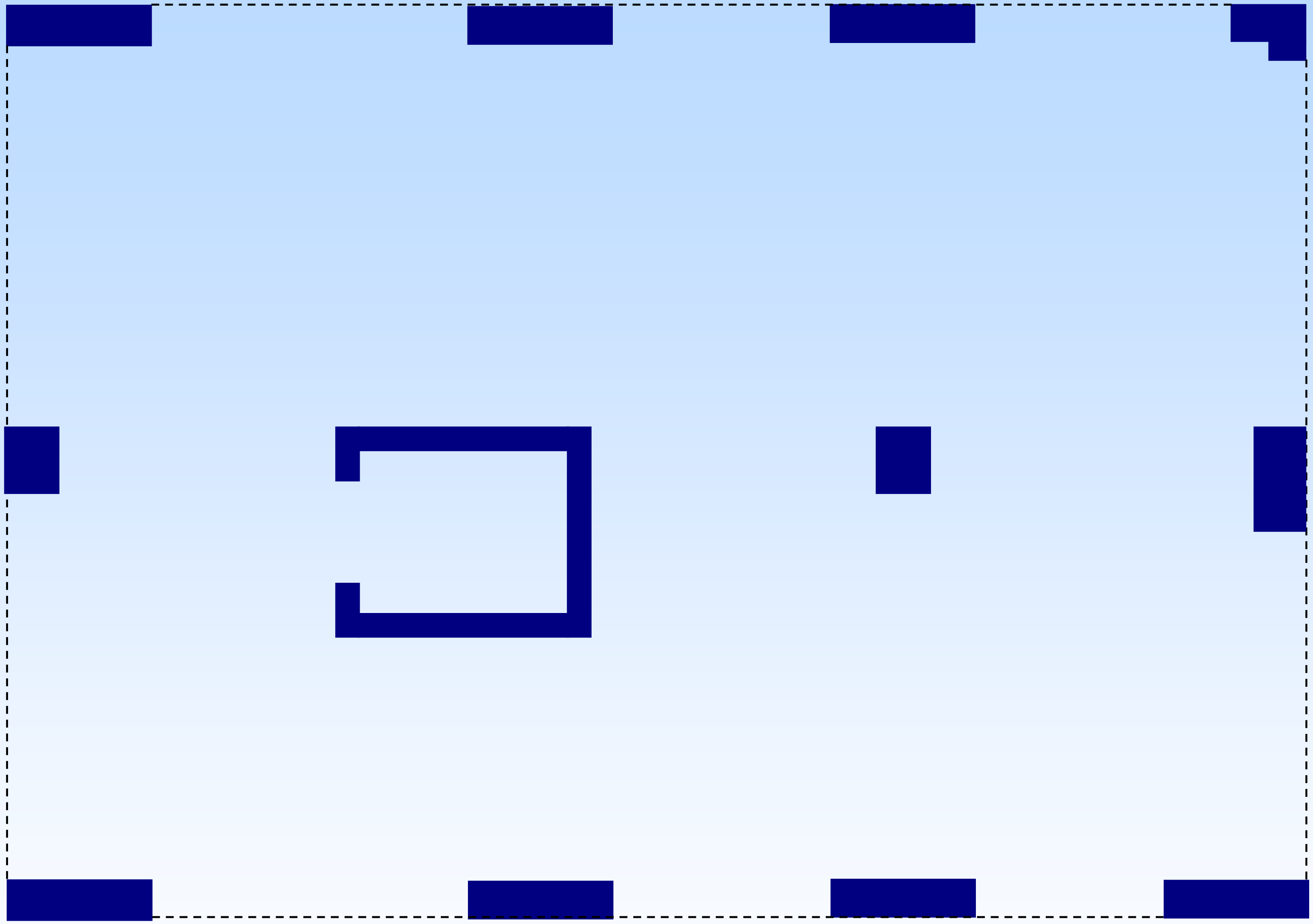




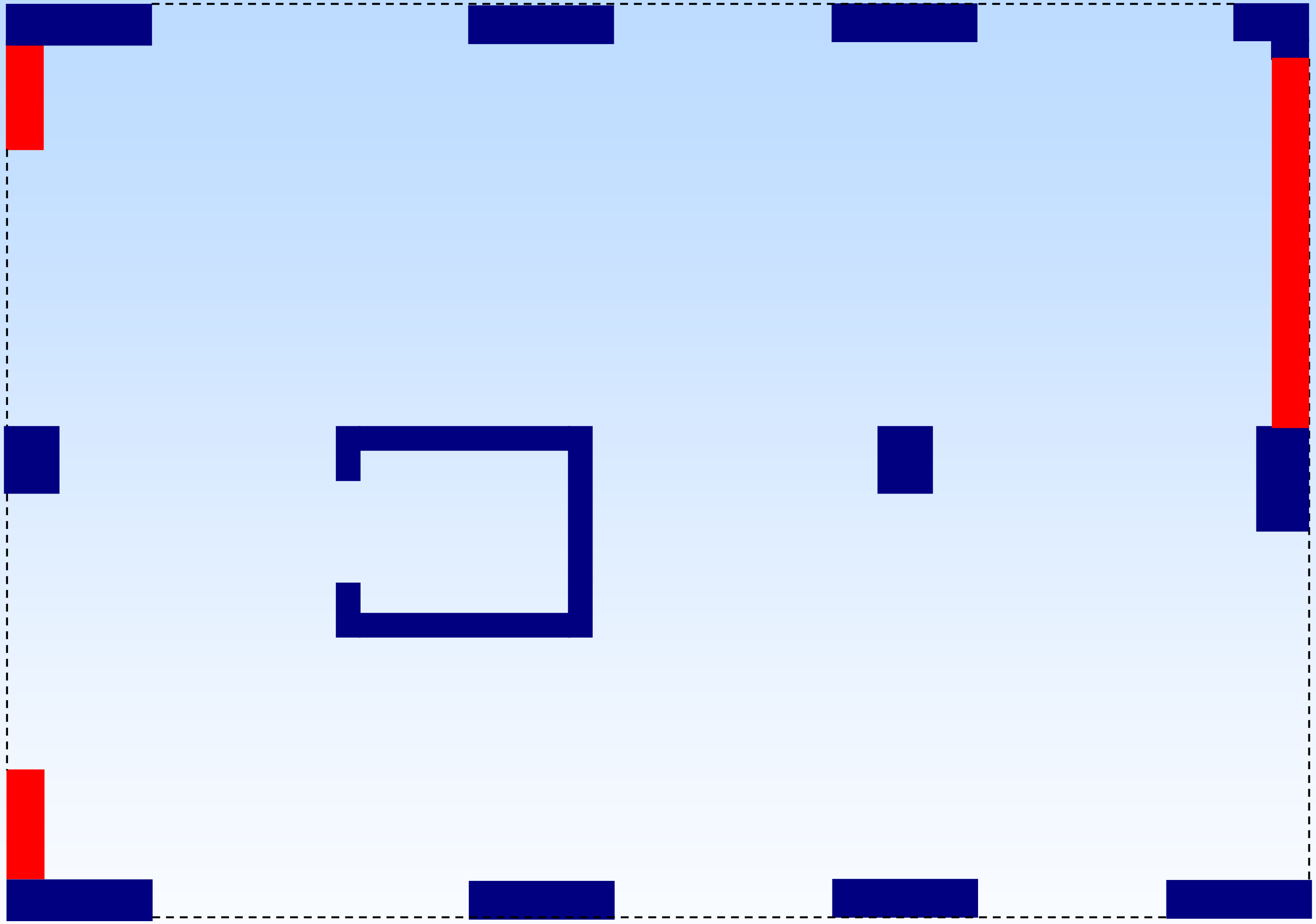
Infilling new shear walls



Addition of new wing walls

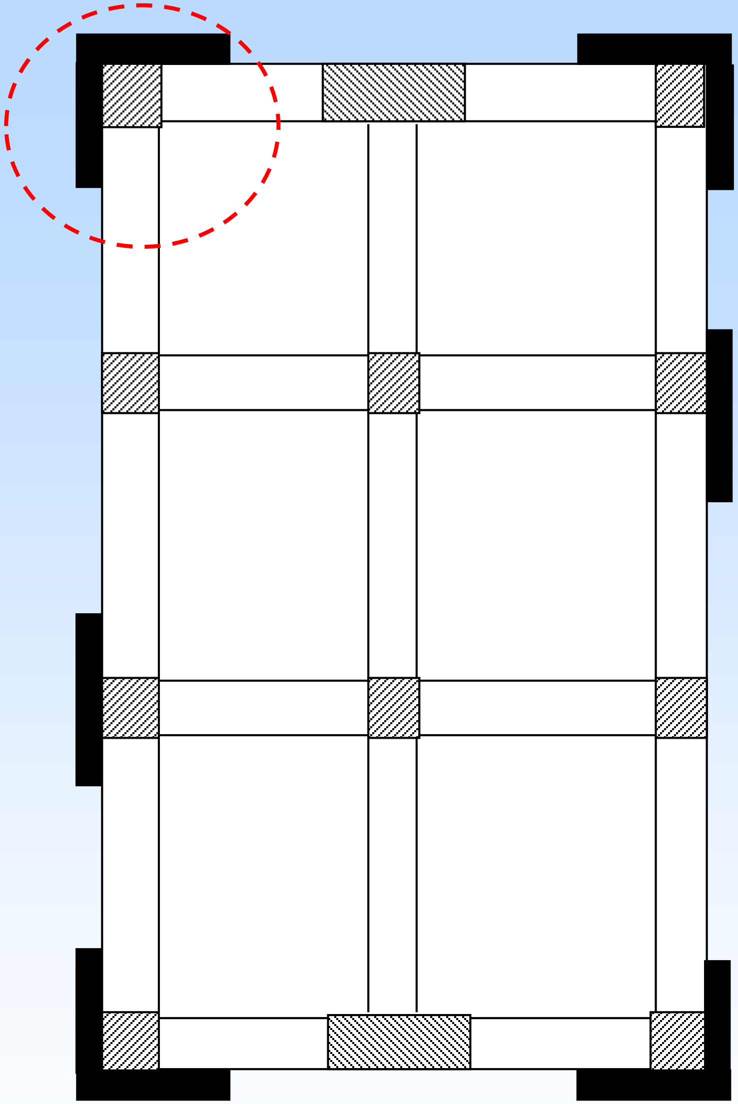


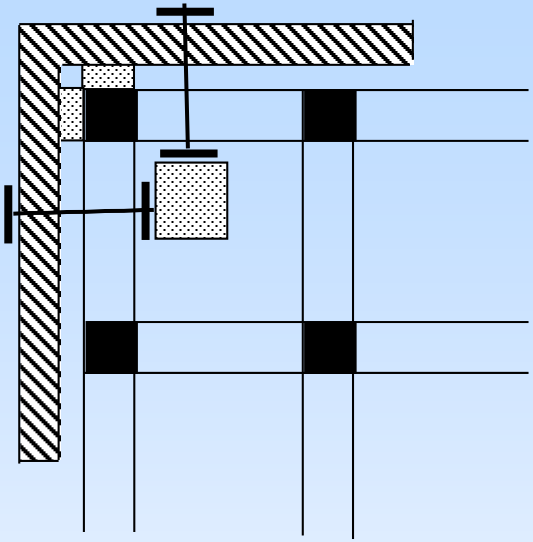
Existing vertical element configuration (PLAN)



Strengthening proposal


18 11 '96



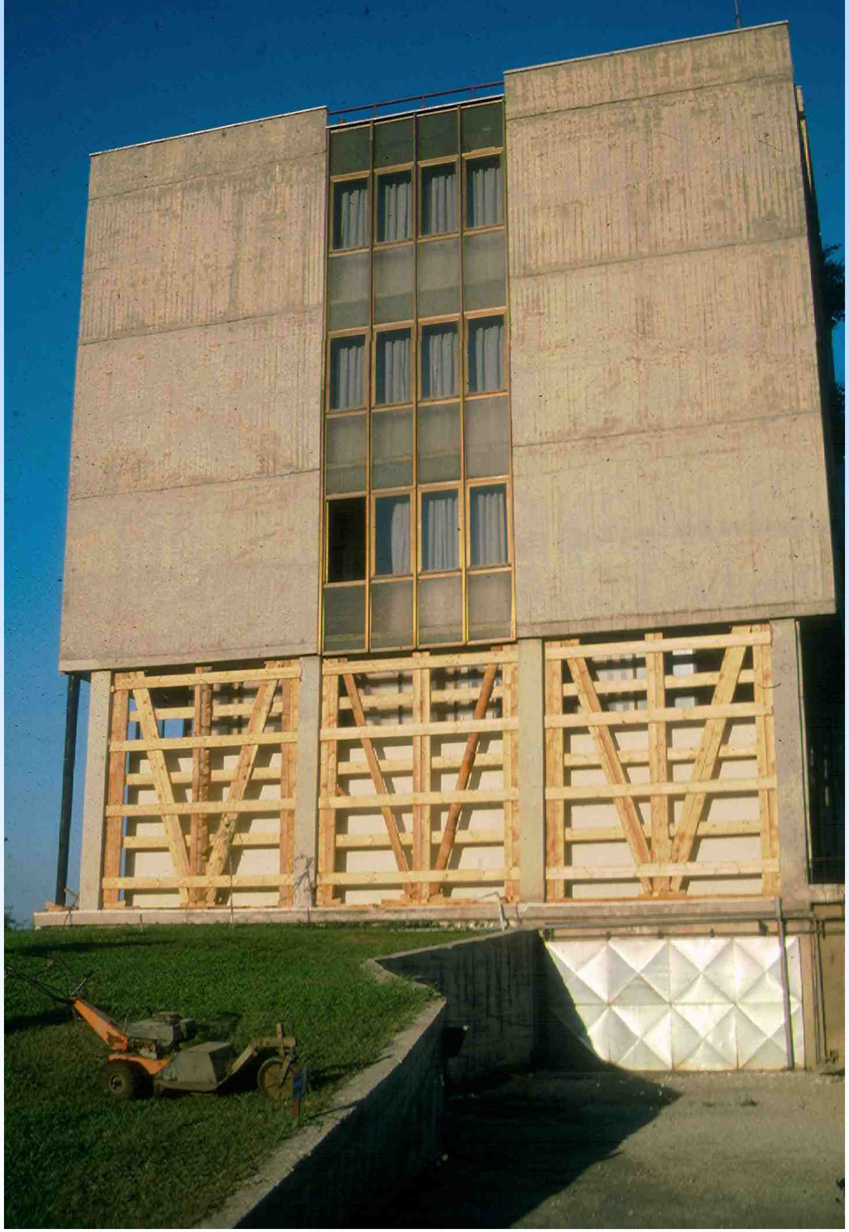


Schematic arrangement of connections between existing building and new wall

Addition of new external walls

Addition of a bracing system


ΣΥΝΕΤΑΙΡΙΣΤΙΚΗ Ε.Π.Ε.
IR Montabert
Robit
DYNO
P&V
P&V



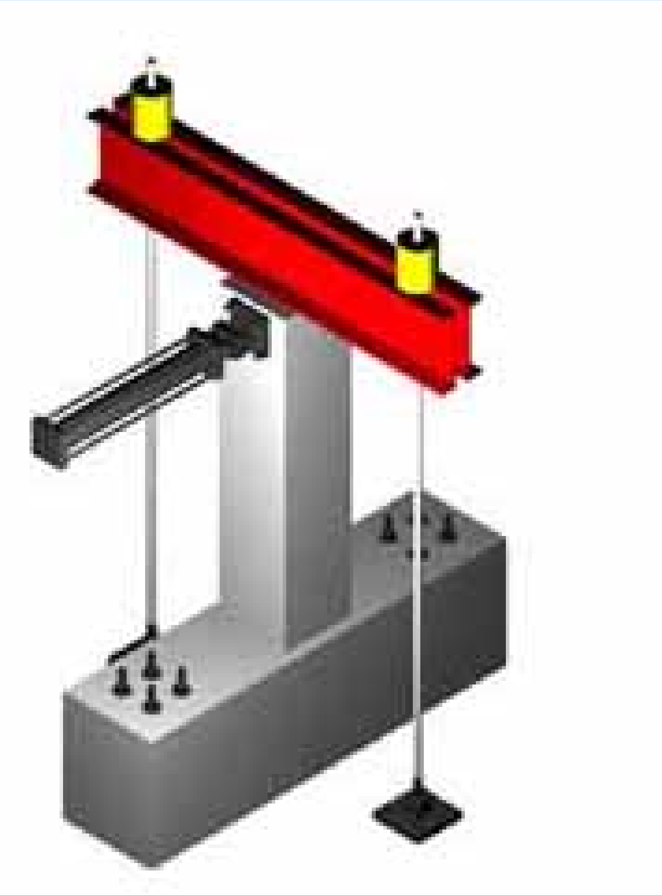
Temporary support and stiffening of the damaged soft floor

COMPOSITE ELEMENTS

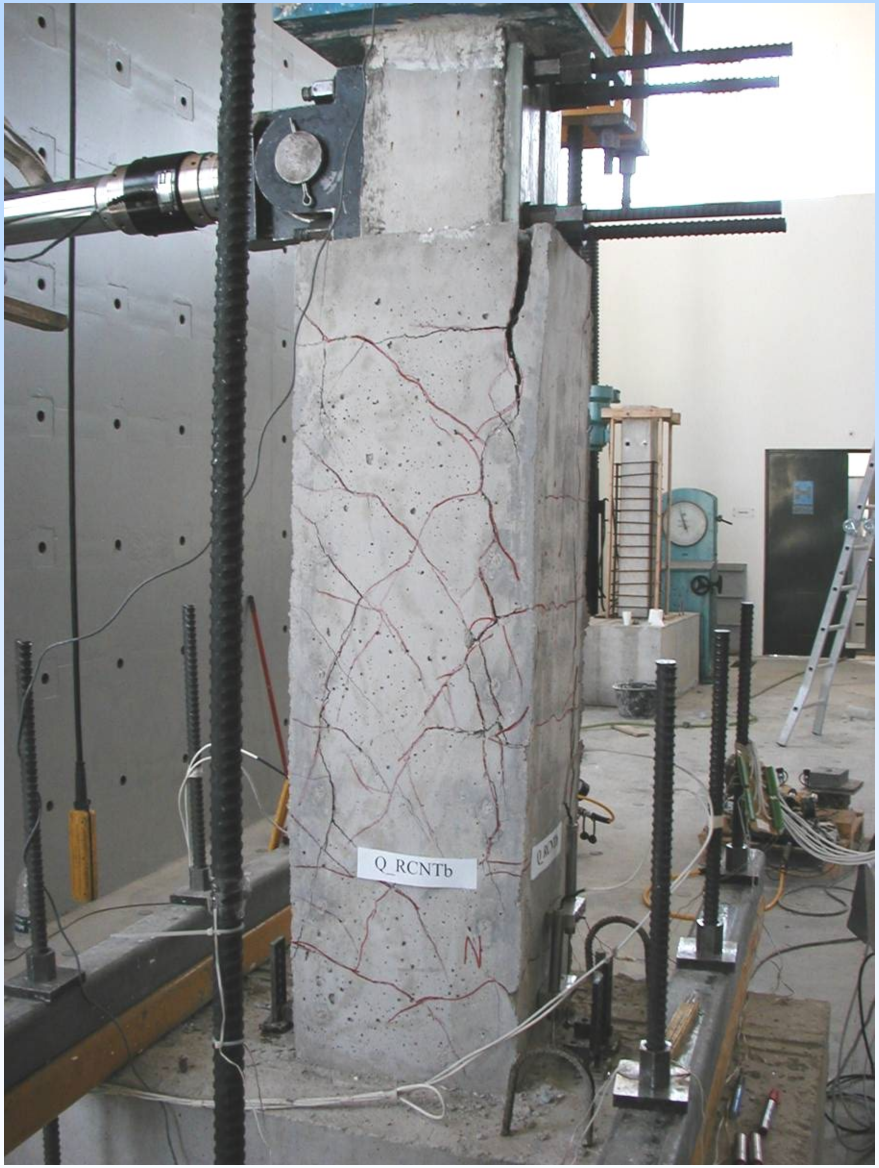
STRUCTURAL DESIGN OF INTERVENTIONS
Greek Retrofitting Code (GRECO) Ch. 8

	Concrete	Steel	FRP
8.1 General requirements			
▪Interface verification	■	■	■
8.2 Interventions for critical regions of linear structural elements			
▪Interventions with a capacity objective against flexure with axial force	■	■	■
▪Interventions with the objective of increasing the shear capacity	■	■	■
▪Interventions with the objective of increasing local ductility	■	■	■
▪Interventions with the objective of increasing the stiffness	■		
8.3 Interventions for joints of frames			
▪Inadequacy due to diagonal compression in the joint	■		
▪Inadequacy of joint reinforcement	■	■	■
8.4 Interventions for shear walls			
▪Interventions with a capacity objective against flexure with axial force	■		
▪Interventions with the objective of increasing the shear capacity	■	■	■
▪Interventions with the objective of increasing the ductility	■		
▪Interventions with the objective of increasing the stiffness	■		
8.5 Frame encasement			
▪Addition of simple “infill”	■	■	
▪Converting frames to to shear walls	■		
▪Strengthening of existing masonry infill	■		
▪Addition of bracing, conversion of frames to vertical trusses		■	
8.6 Construction of new lateral shear walls			
▪Stirrups	■	■	
▪Foundations for new shear walls	■	■	
▪Diaphragms	■	■	
8.7 Interventions for foundation elements	■		

EXPERIMENTAL WORK (UNIVERSITY OF PATRAS)



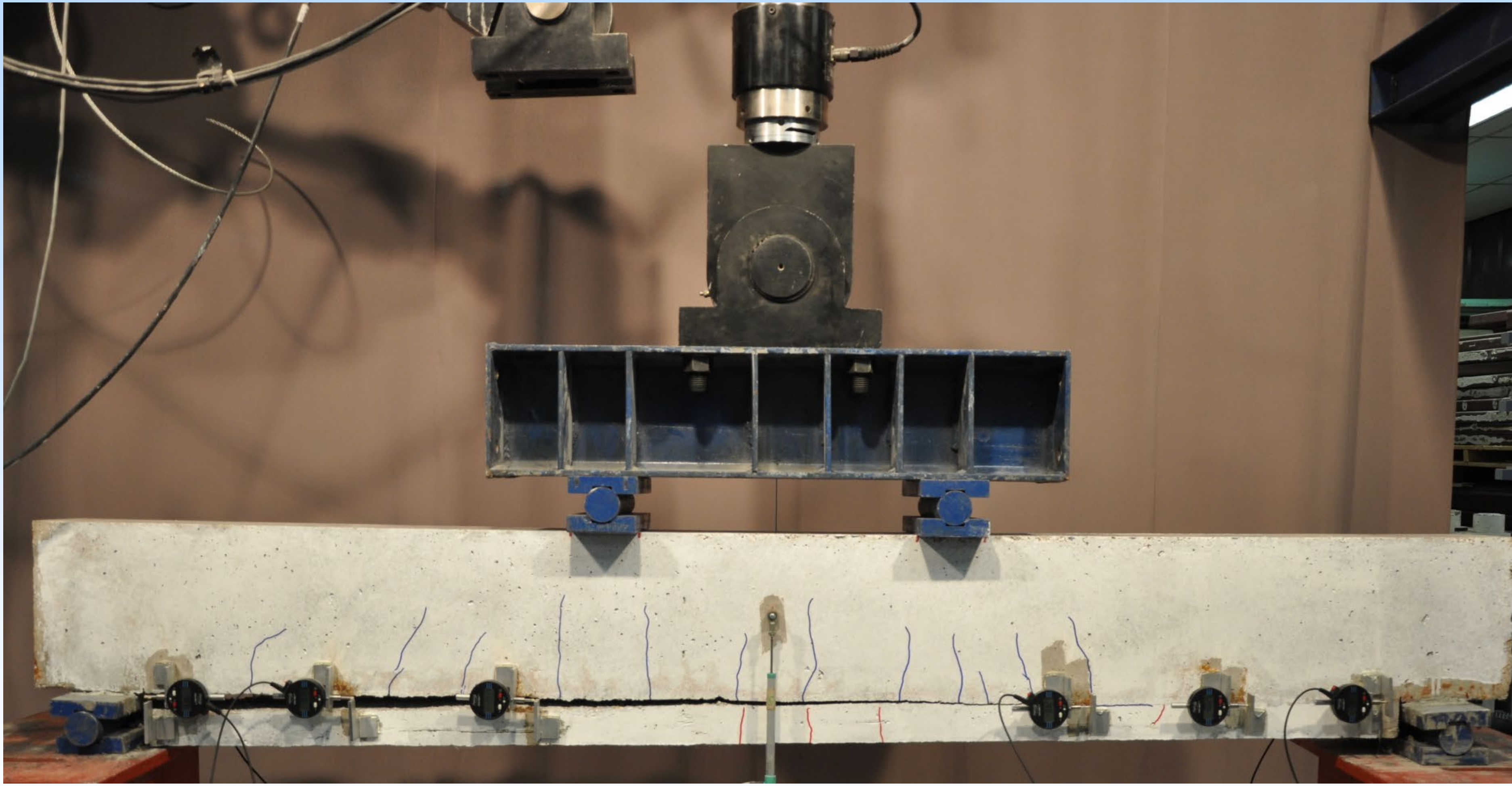
Damage to a specimen with shotcrete and dowels



Damage to a specimen with poured concrete, smooth interface without dowels

Addition of a new concrete layer to the top of a cantilever slab

Beam strengthened with a new concrete layer



Interface failure due to inadequate anchorage of the new bars at the supports

BASIC DESIGN CONSIDERATION

Repaired/Strengthened Element

↓

Multi – Phased Element

↓

Composite Element

↓

Influence of Interface Connection

DESIGN FRAMEWORK

Into the existing framework for new constructions Supplemented by:

- Control of Sufficient Connection Between Contact Surfaces

- Determination of Strength and Deformation Capacity of the Strengthened Element

  - **As a Composite** Element (Multi-Phased Element)

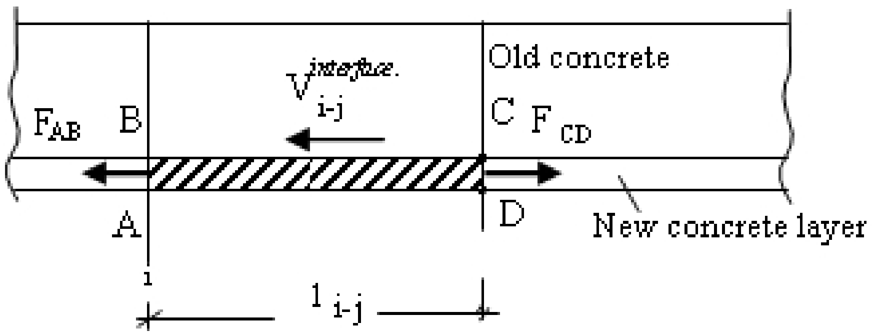
CONTROL OF A SUFFICIENT CONNECTION BETWEEN CONTACT SURFACES

$$S_d \leq R_d$$

$$V_{Sd}^{interface} \leq V_{Rd}^{interface}$$

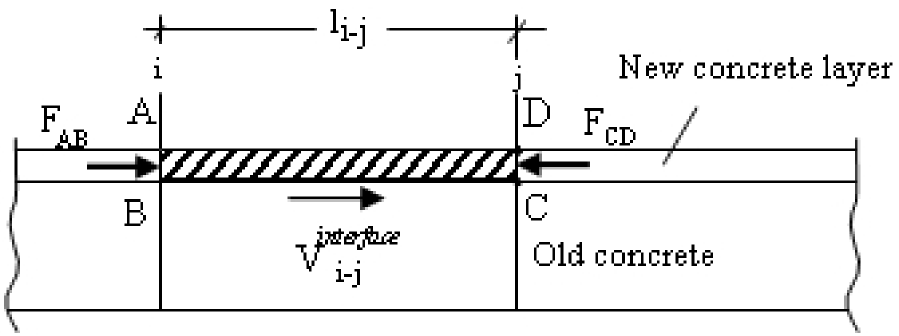
Interface Shear Force $\leq$ **Interface Shear Resistance**

INTERFACE SHEAR FORCES: $V_{sd}^{interface}$



$$V_{i-j}^{interface} = F_{AB} - F_{CD}$$

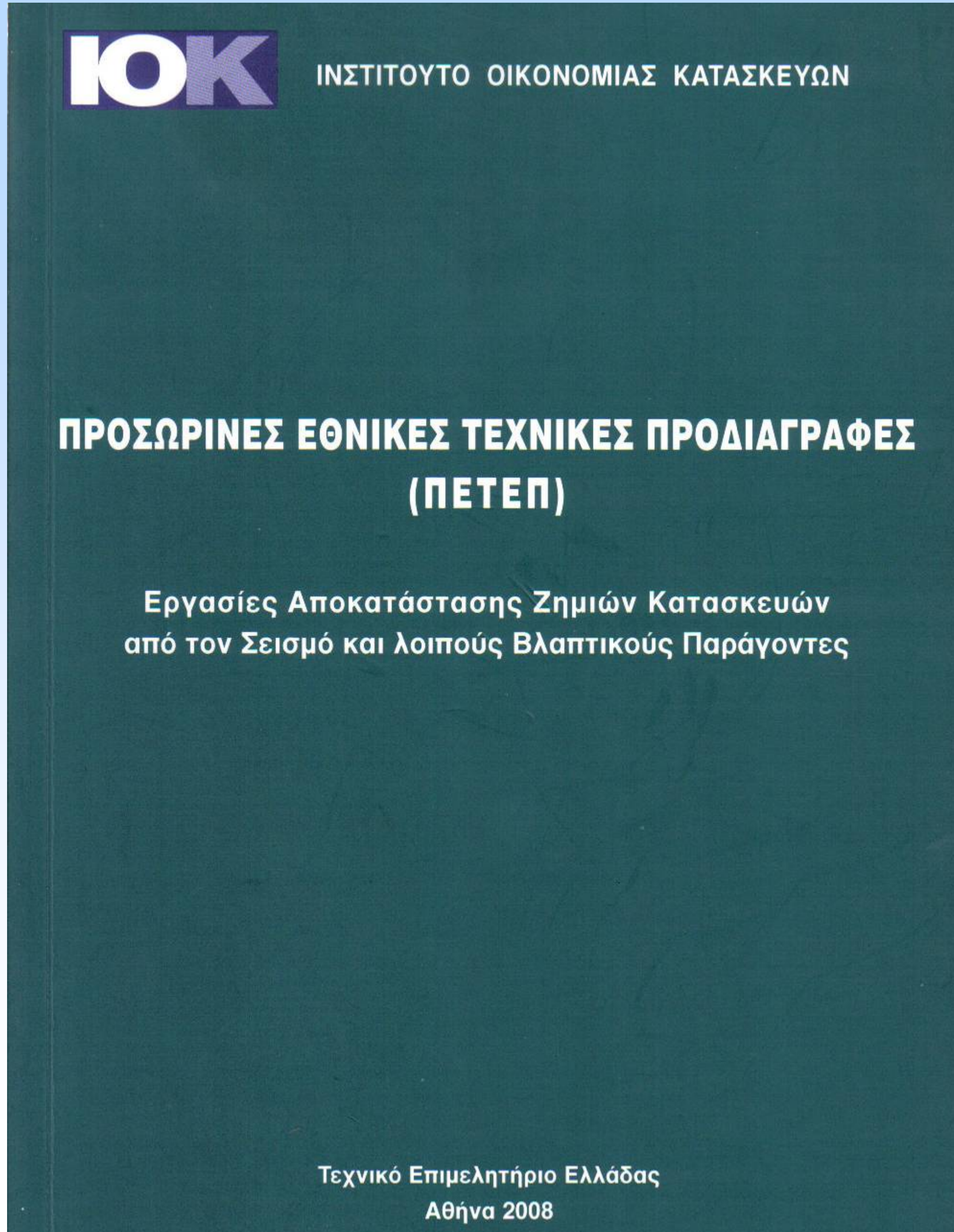
(a) strengthening in the tensile zone



$$V_{i-j}^{interface} = F_{AB} - F_{CD}$$

(b) strengthening in the compressive zone

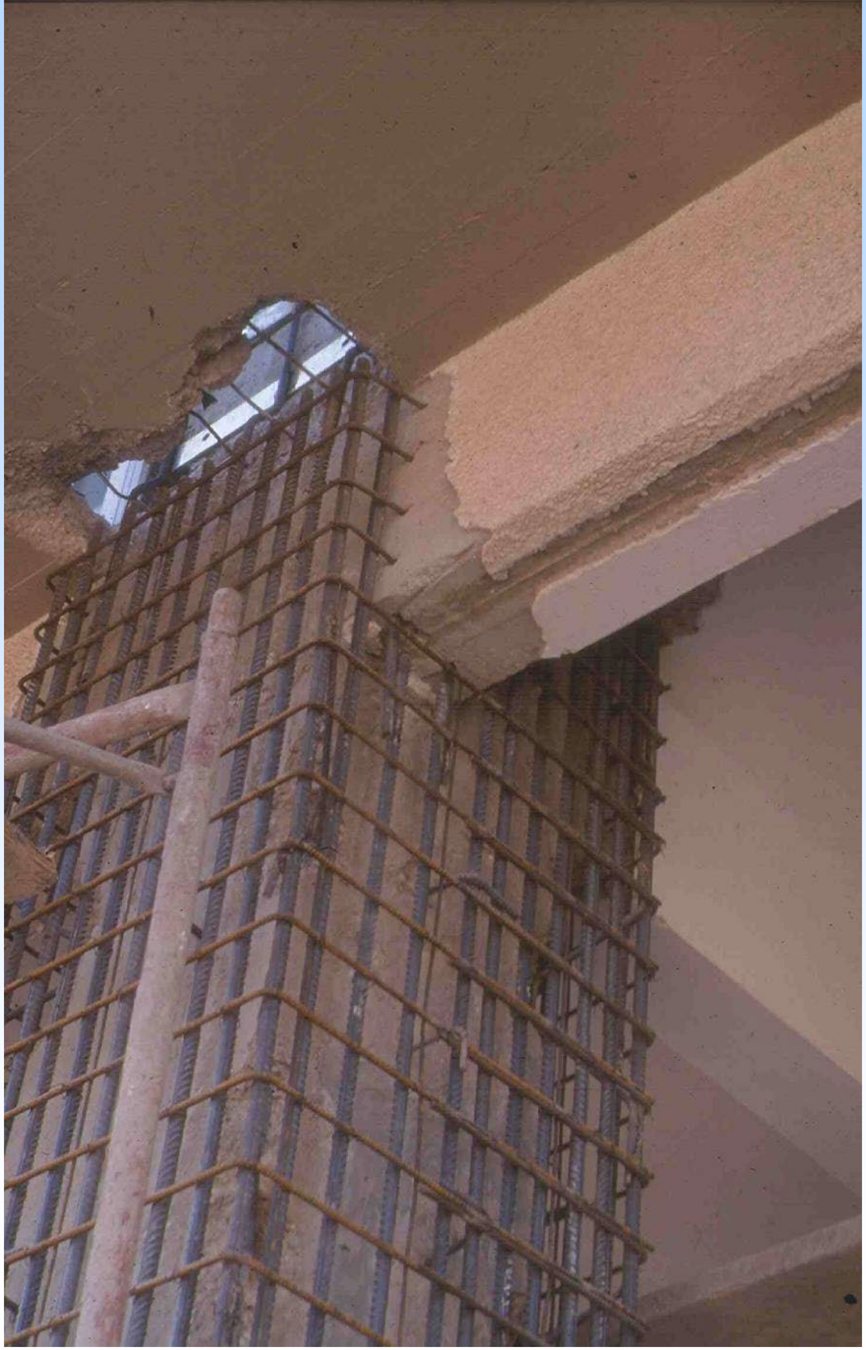
Technological guidelines for repairs and strengthening:



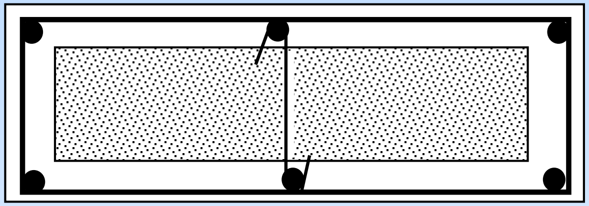
Roughening by sandblasting

Use of a scabbler to improve frictional resistance by removing the exterior weak skin of the concrete to expose the aggregate

Concrete jacketing in practice



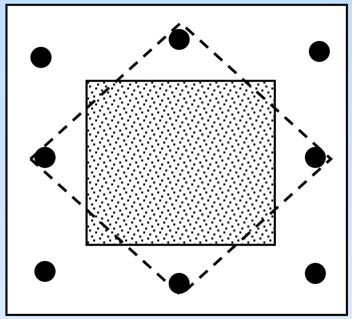
Total jacket



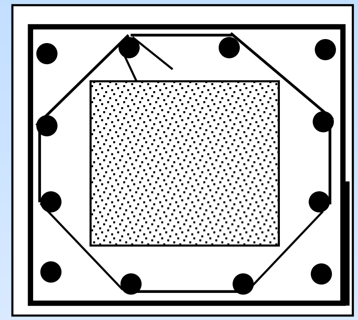
Inserting intermediate links in sections with a high aspect ratio

Inserting intermediate stirrups in square sections

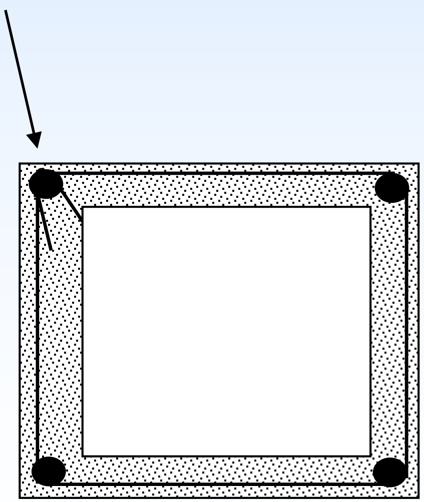
NO



YES



135° bend to form hooks



Bar buckling due to stirrup ends opening

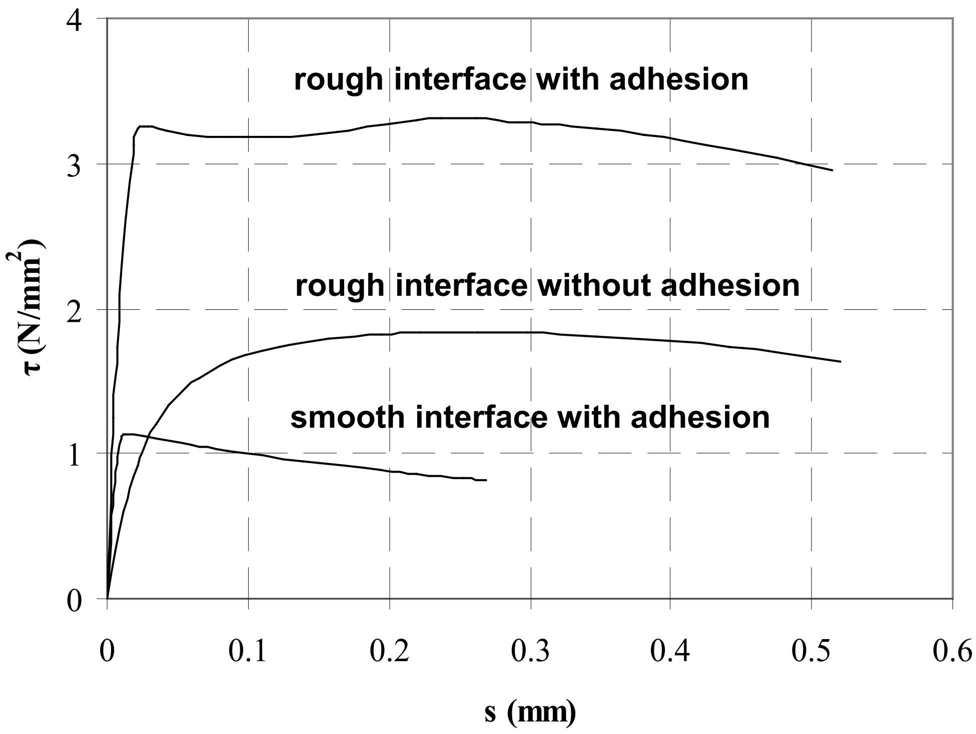
Welding of jacket's stirrup ends

INTERFACE SHEAR RESISTANCE: $V_{Rd}^{interface}$

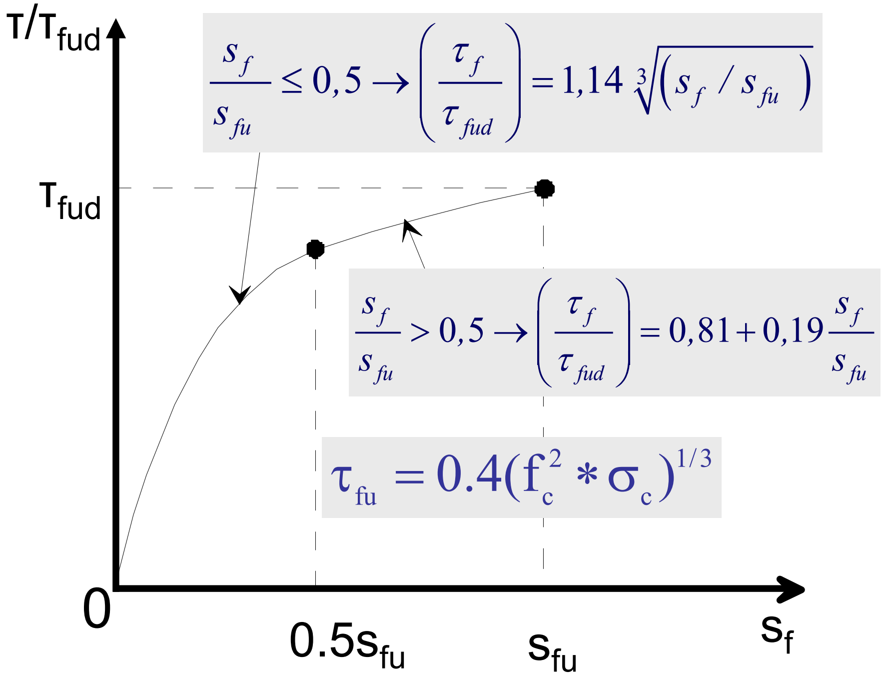
Mechanisms

- **Friction and Adhesion**
- **Dowel Action**
- **Clamping Action**
- **Welded Connectors**

UNREINFORCED INTERFACES



(CEB Bul. No. 162, 1983)



(GRECO, 2012)

Concrete-to-concrete adhesion

Roughened interface concrete-to-concrete friction

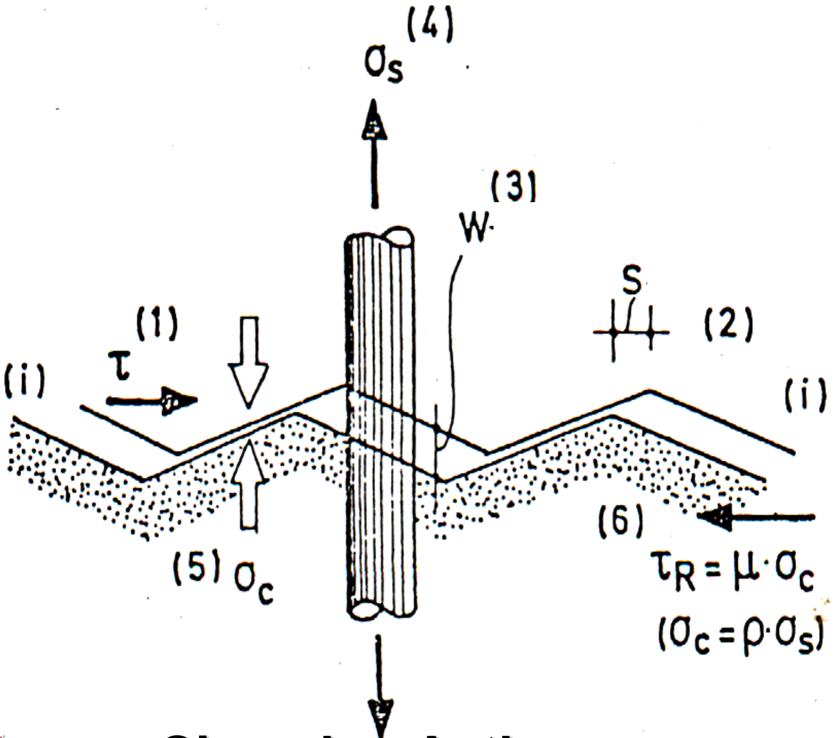
REINFORCED INTERFACES

Additional Friction

When a Steel Bar Crosses an Interface, a Clamping Action May Occur if:

- Surface of Existing Concrete has been Roughened
- The Steel Bar is Adequately Anchored

(Tassios and Vintzeleou, 1987)



Clamping Action

(1) When Shear Stress is Applied

(2) Slip Occurs

(3) Contact Surface Opens (one surface rides up over the other due to roughness)

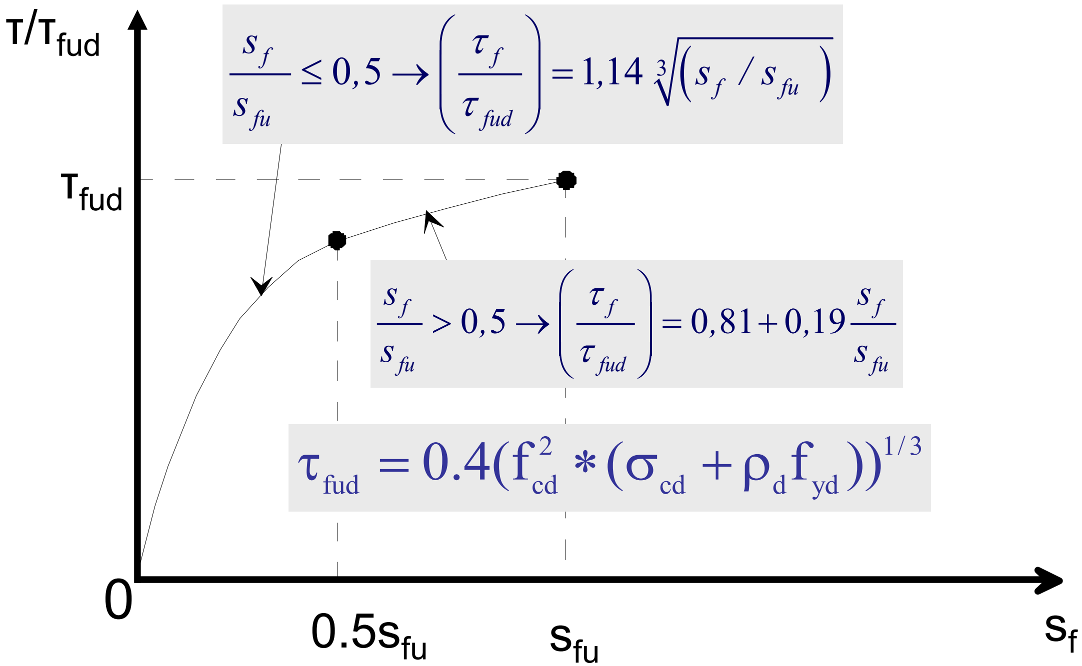
(4) Tensile Strength is Activated in the Steel Bar

(5) Compression Stress (σ_c) is Mobilized at the Interface

(6) Frictional Resistance is Activated

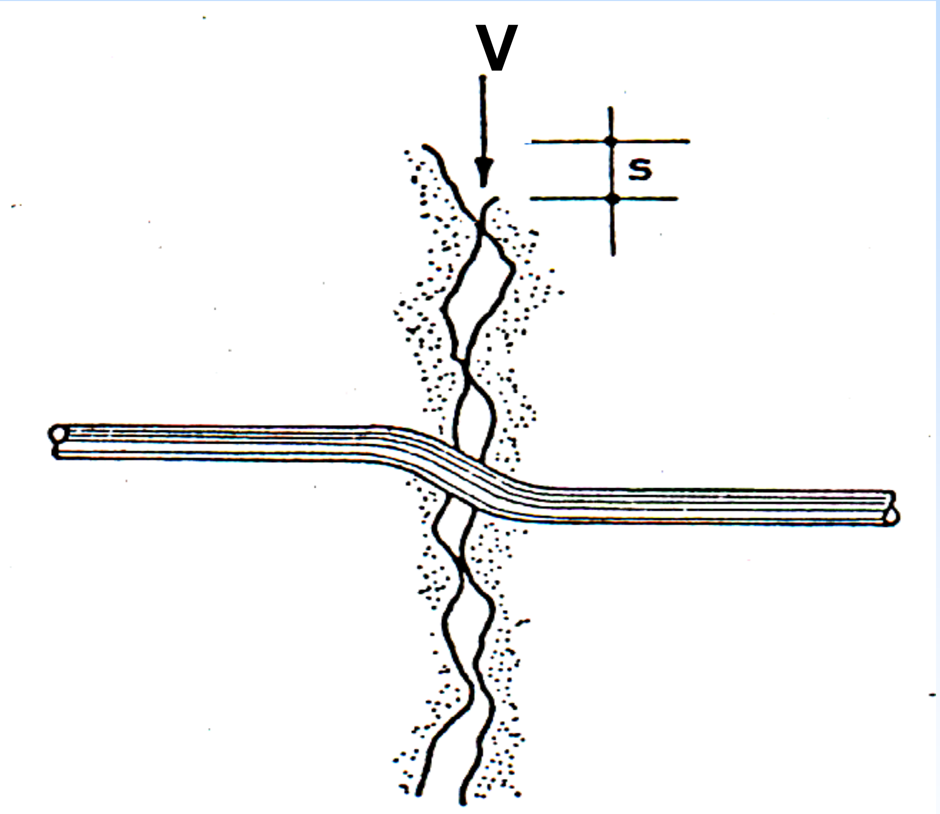
Reinforced Interfaces

Frictional resistance



(GRECO, 2012)

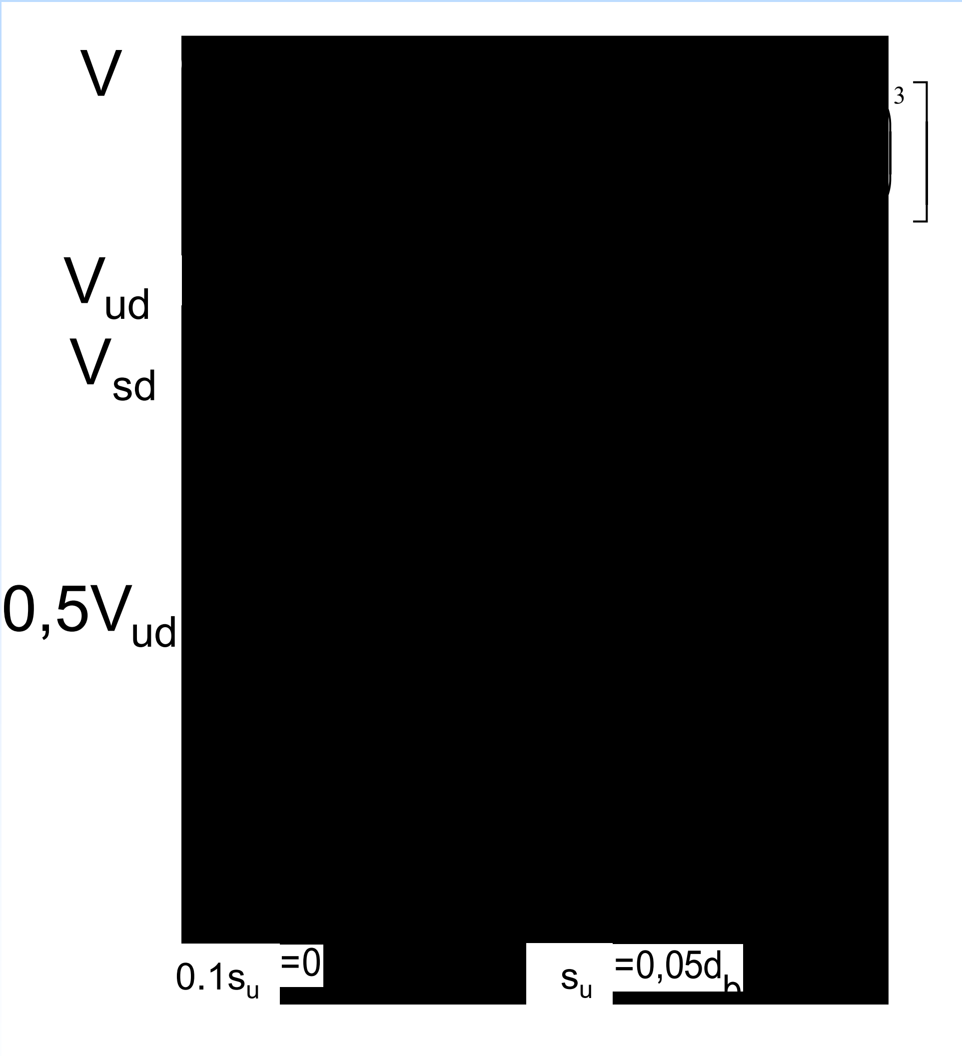
Reinforced Interfaces



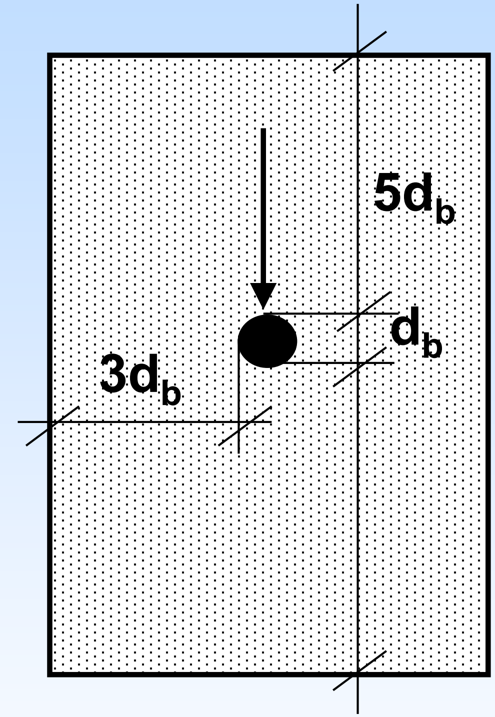
Dowel action

Shear Resistance

for Dowel Action as a function of the interface slip



$$V_{ud} = 1.3\, d_b^2 \sqrt{f_c\, f_y}$$



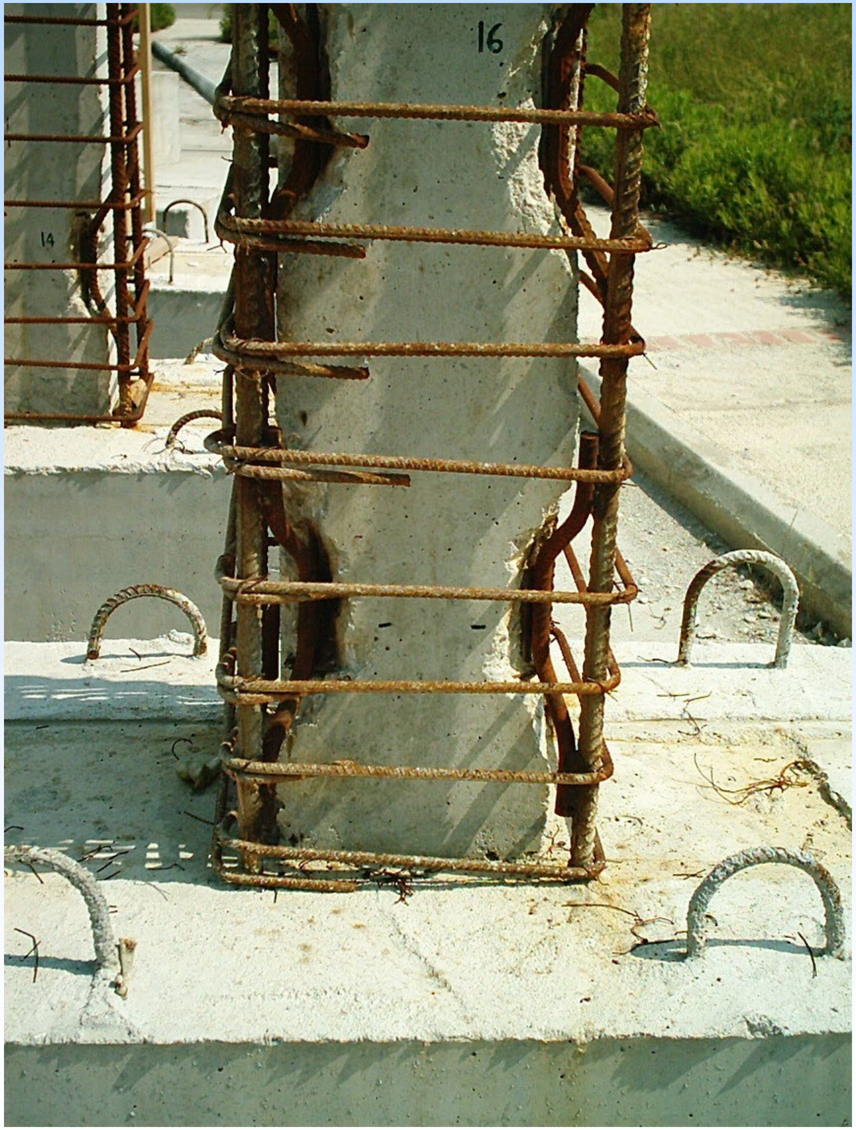
A minimum concrete cover is necessary for full activation of dowel action

Use of steel dowels and roughening the surface of an original column

- Most popular in practice to achieve a sufficient connection at the interface

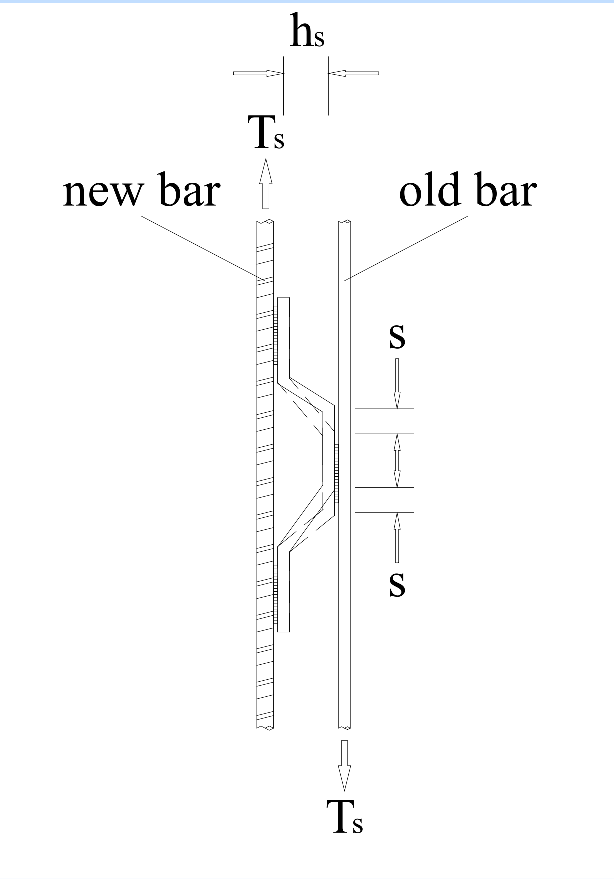
Reinforced Interfaces

Bent Connecting Steel Bars



Bent Bar Model

(Tassios, 2004)



When s occur at the interface one leg of the bent bar is elongated by $s/\sqrt{2}$ the other is shortened

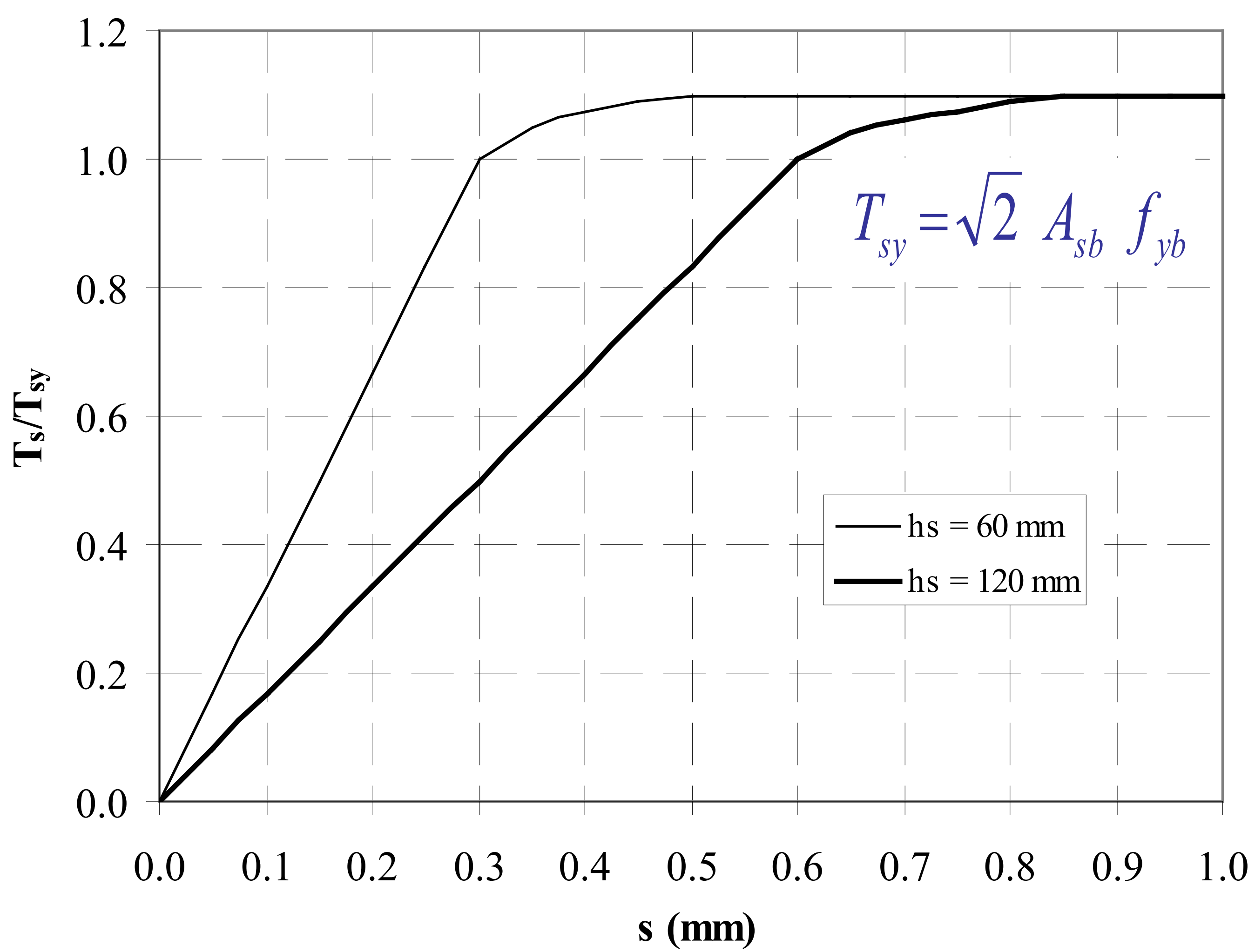
➡Tensile and Compressive Leg Stresses are mobilized:

$$\varepsilon_{sb} = \frac{s/\sqrt{2}}{\sqrt{2}h_s} = \frac{s}{2h_s}$$ and $$\sigma_{sb} = E_s \frac{s}{2h_s} \leq f_{yb}$$

➡Force is Transferred between Reinforcements:

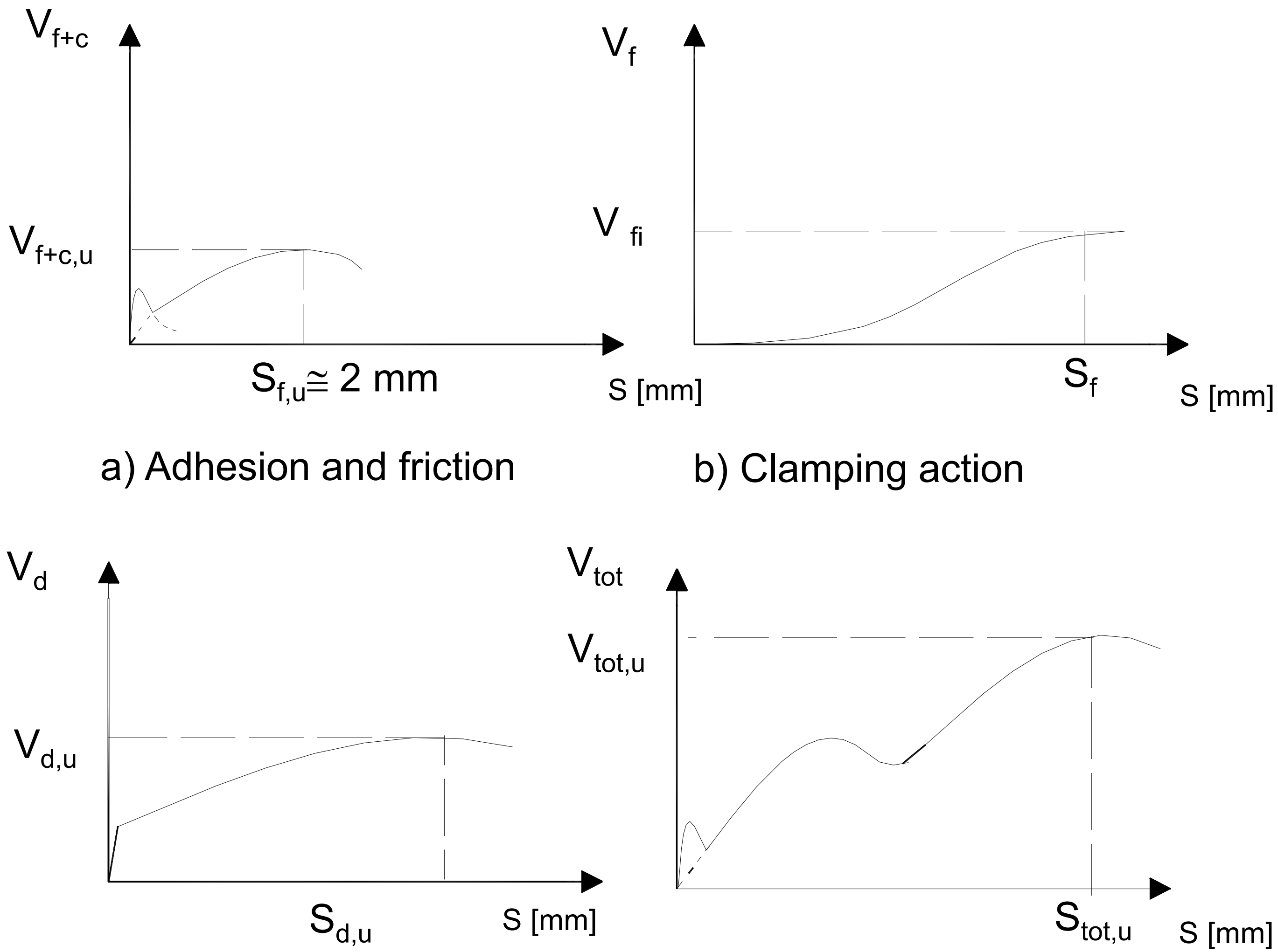
$$T_s = A_{sb} * E_s (s/\sqrt{2}h_s) \leq T_{sy} = \sqrt{2}A_{sb}f_{yb}$$

Force Transferred – Interface Slippage



Mechanism is mobilized for very small Slippage

Superposition of shear resistance mechanisms



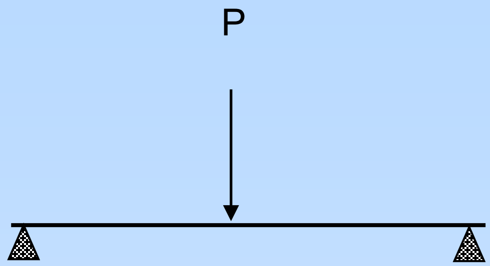
a) Adhesion and friction

b) Clamping action

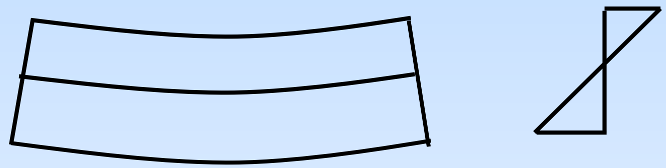
c) Dowel action

d) Superposition of all actions

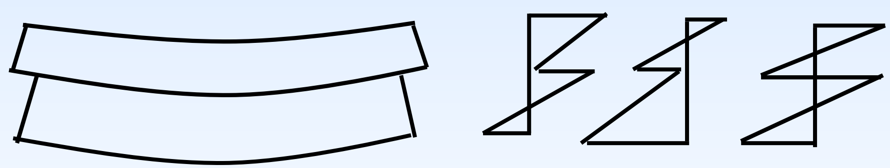
$$V_{tot} = \beta_D V_d + \beta_f V_f$$



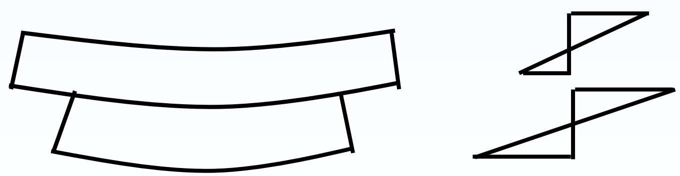
Full interaction



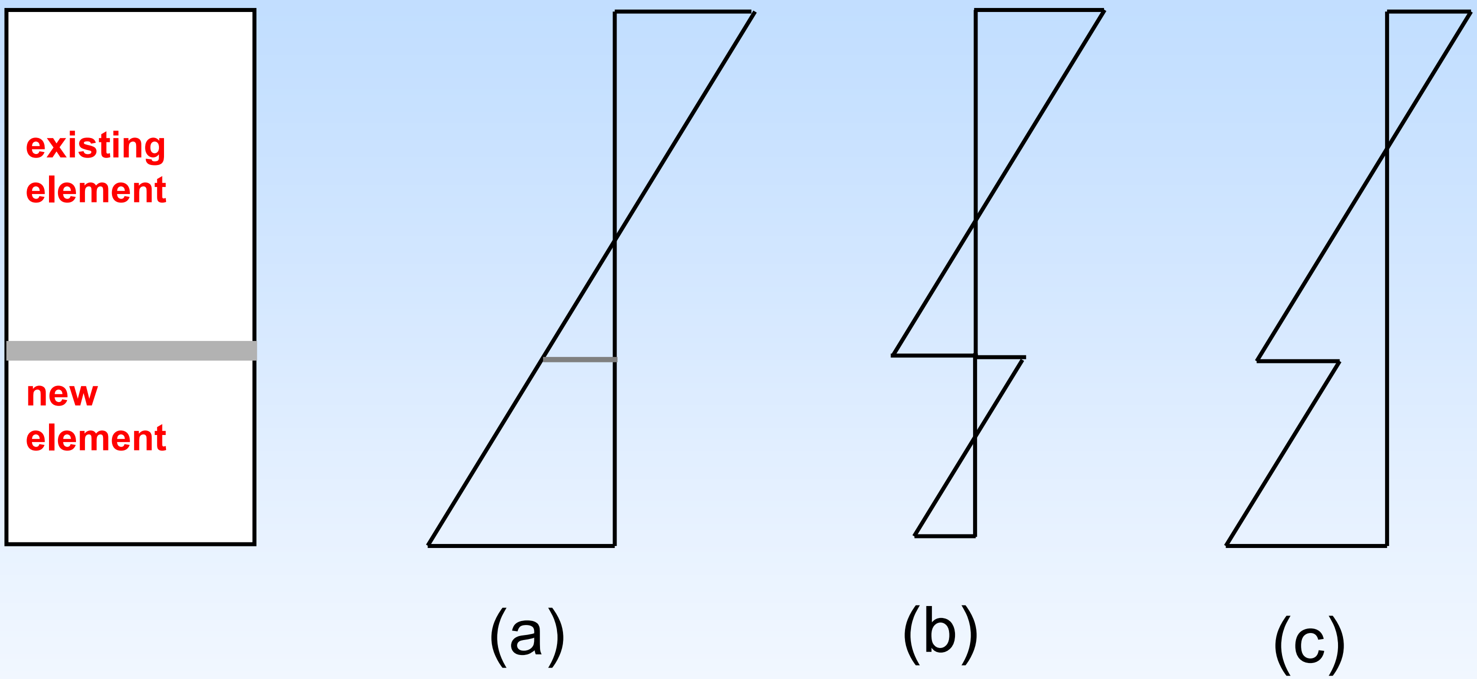
Partial interaction



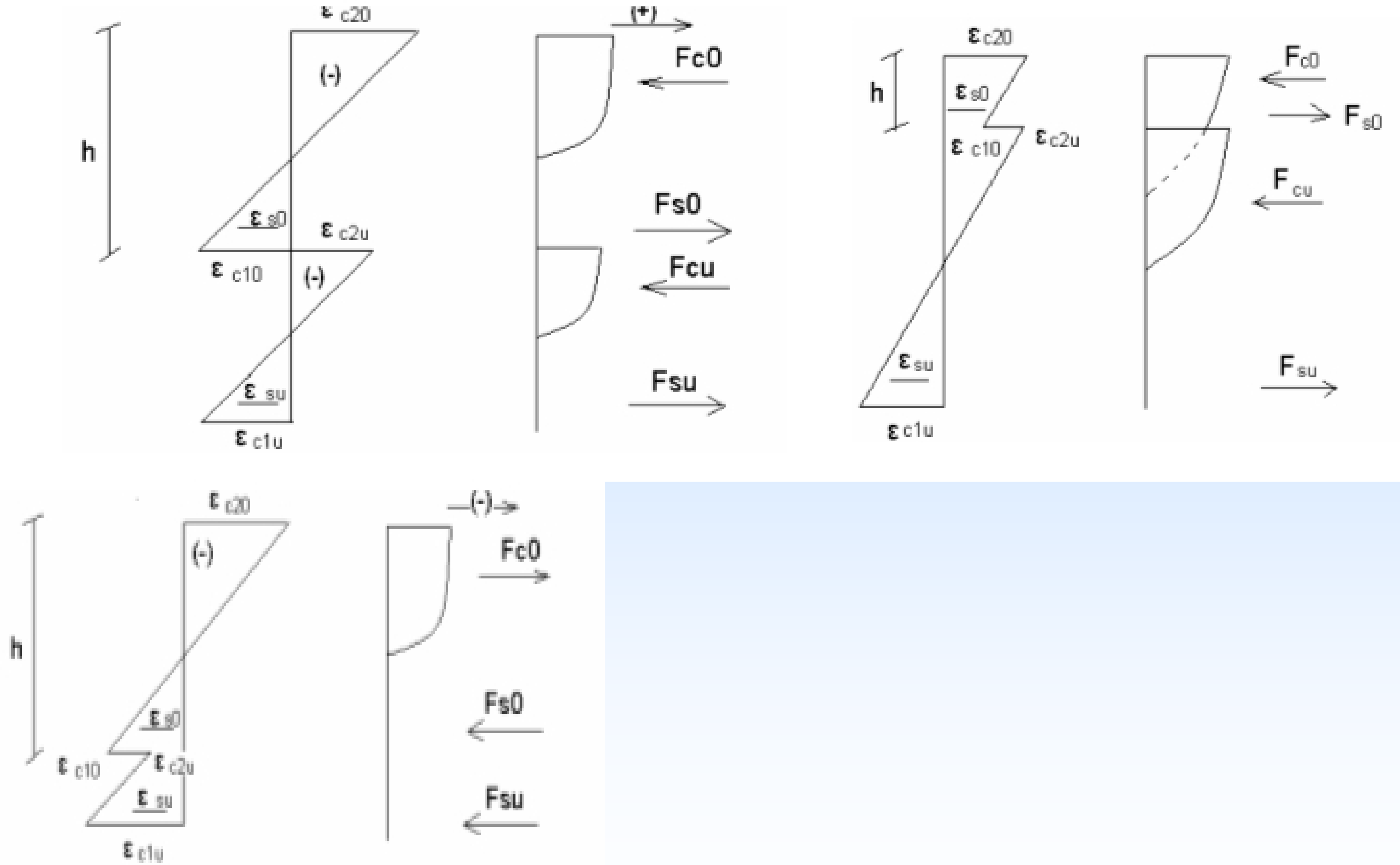
Independent action



CAPACITY OF MULTI-PHASED ELEMENT

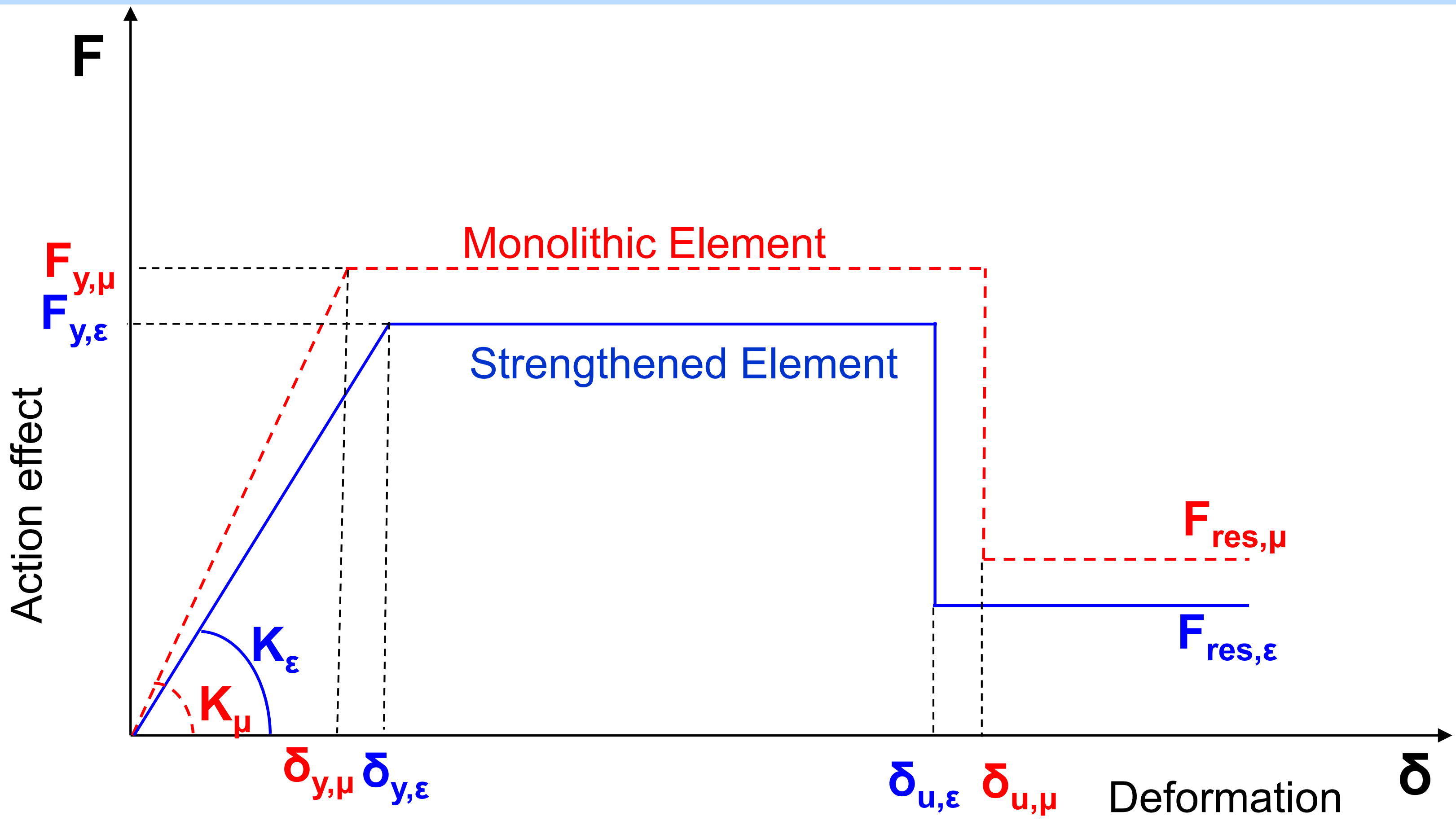


Distribution of Strain With Height of Cross Section



Possible strain and stress distributions

CAPACITY CURVES



$$K_K = \frac{K_\varepsilon}{K_\mu} \qquad K_r = \frac{F_{y,\varepsilon}}{F_{y,\mu}} \qquad K_{\delta y} = \frac{\delta_{y,\varepsilon}}{\delta_{y,\mu}} \qquad K_{\delta u} = \frac{\delta_{u,\varepsilon}}{\delta_{u,\mu}}$$

MONOLITHIC BEHAVIOUR FACTORS

- **For the Stiffness:**

$$k_k = \frac{\text{the stiffness of the strengthened element}}{\text{the stiffness of the monolithic element}}$$

- **For the Resistance:**

$$k_r = \frac{\text{the strength of the strengthened element}}{\text{the strength of the monolithic element}}$$

- **For the Displacement:**

$$k_{\delta y} = \frac{\text{the displacement at yield of the strengthened element}}{\text{the displacement at yield of the monolithic element}}$$

$$k_{\delta y} = \frac{\text{the ultimate displacement of the strengthened element}}{\text{the ultimate displacement of the monolithic element}}$$

$$(EI)_{strengthened} = k_k \, (EI)_M$$

$$R_{strengthened} = k_r \, R_M$$

$$\delta_{i,strengthened} = k_{\delta i} \, \delta_{i,M}$$

Addition of a new concrete layer
to the top of a cantilever slab

Monolithic Factors

- Approximations according to G.C.S.I.

For slabs:

$k_k = 0{,}85$ $k_r = 0{,}95$ $k_{\theta y} = 1{,}15$ $k_{\theta u} = 0{,}85$

For concrete jackets:

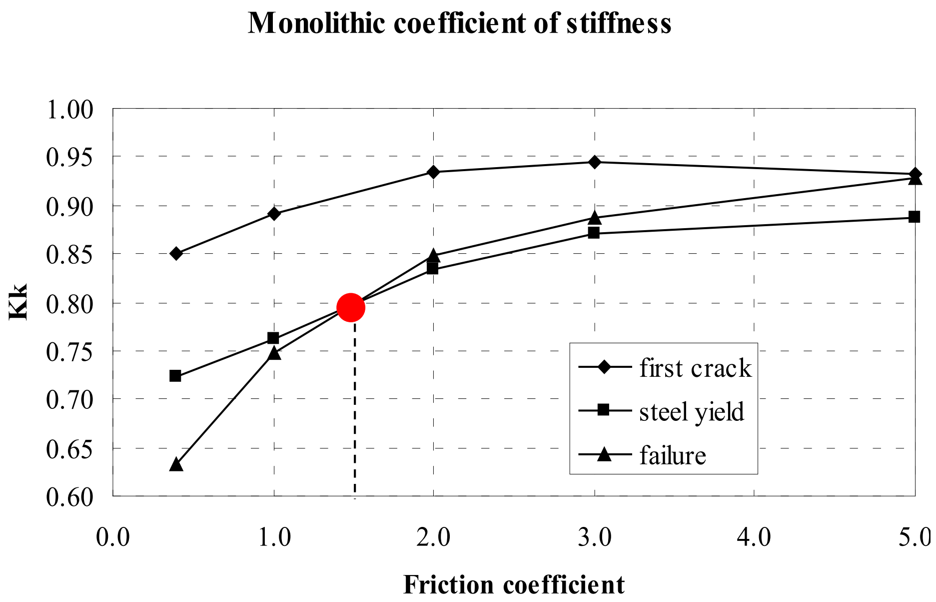
$k_k = 0{,}80$ $k_r = 0{,}90$ $k_{\theta y} = 1{,}25$ $k_{\theta u} = 0{,}80$

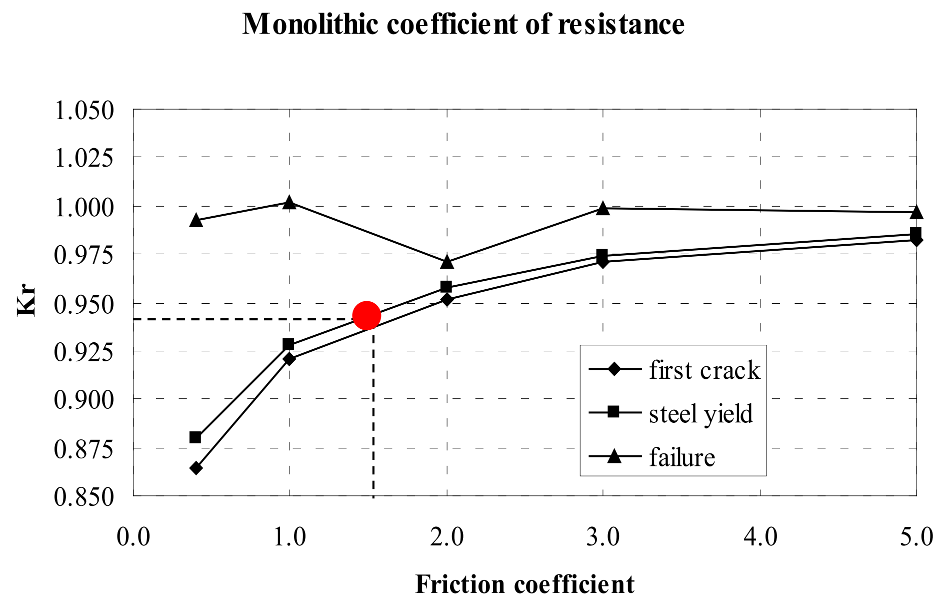
For other elements:

$k_k = 0{,}80$ $k_r = 0{,}85$ $k_{\theta y} = 1{,}25$ $k_{\theta u} = 0{,}75$

Monolithic Factors

Influence of Interface Connecting Conditions in Case of Concrete Jackets





For μ=1.4 **k_k = 0.80 and k_r = 0.94**

k_k = 0.70 and k_r = 0.80 **(EC8, Part 1.4)**

k_k = 0.80 and k_r = 0.90 **(G.C.S.I.)**

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