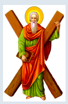
Seismic Assessment and Retrofitting of Existing Buildings

Under the Eurocodes Framework

Assessment and Retrofitting Framework



➤ Prof. Stephanos E. Dritsos
Department of Civil Engineering, University of Patras

VIENNA - BOKU, October 2012

WEAKNESSES OF EXISTINHG OLD STRUCTURES

(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 ~ 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

PERFORMANCE REQUIREMENTS

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

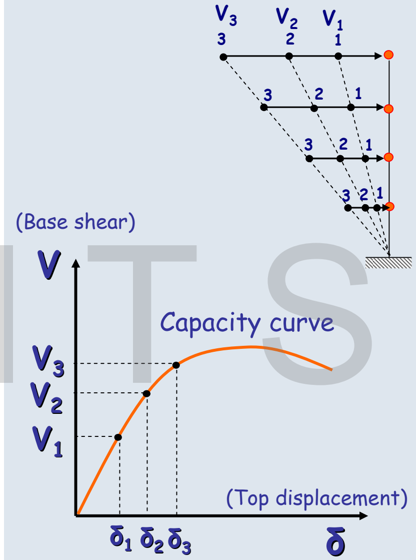
The cost of repair may be high. The cost of reconstruction should be examined as an alternative solution

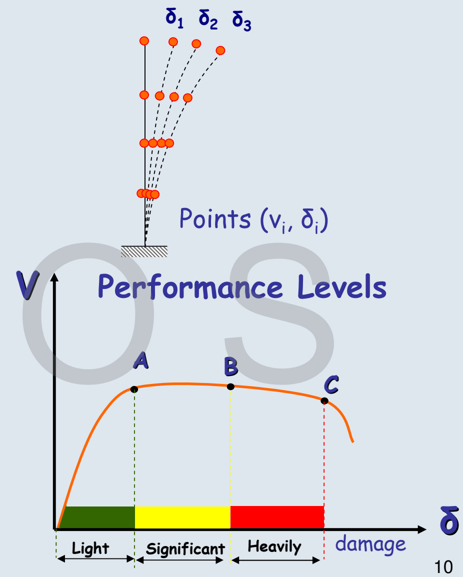
Level C: **Collapse Prevention (CP) or Near Collapse (NP)**

- 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	$CP_{50\%}$	$LS_{50\%}$	$DL_{50\%}$

Usual design of new buildings

Design of important structures (remain functional during earthquake)

Minimum acceptable seismic action level
Instead, do nothing due to economic, cultural, aesthetic and functional reasons

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

ELEMENT CLASSIFICATION

Ductile

Flexure controlled

$$S_d \leq R_d$$

deformation demand — deformation capacity

Brittle

Shear controlled

$$S_d \leq R_d$$

strength demand — strength capacity

Primary Seismic

Secondary Seismic

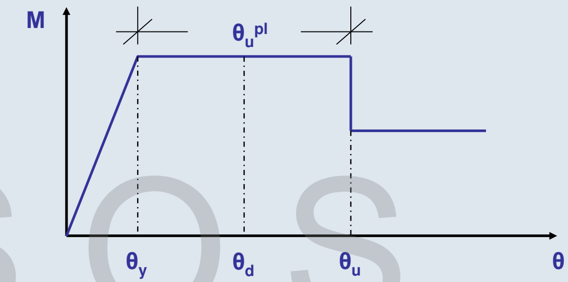
"Secondary" seismic element

- 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

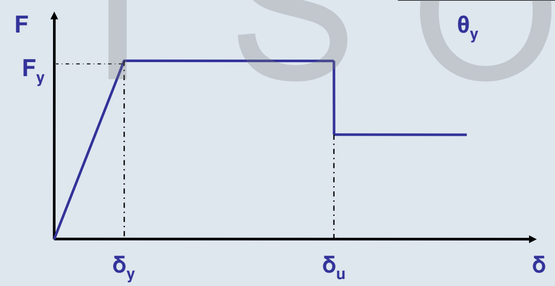
REINFORCED CONCRTETE STRUCTURES

Element's Capacity Curve

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



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



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

Elements'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 SHEAR CAPACITY

Beams and Columns

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

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

rectangular web cross section

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

circular cross section

Shear Walls

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

Short Columns $(L_V/h) \leq 2$

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

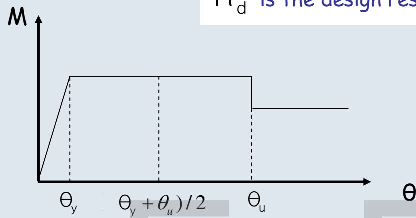
ELEMENT'S SAFETY VERIFICATION

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}

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) $\quad \Theta_{Rd} = \dfrac{\theta_u}{\gamma_{Rd}}$ $\quad \gamma_{Rd} = 1{,}8$ for "primary" elements

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

METHODS OF ANALYSIS

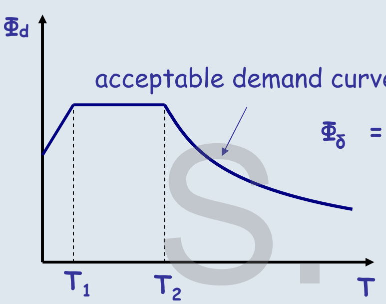
- Lateral force analysis (linear)
- Modal response spectrum analysis (linear)
- Non-linear static (pushover) analysis
- Non-linear time history dynamic analysis
- q-factor approach

q=3.0 – 3.5 for new design $\quad (R_d = q.\ R_{R,el})$

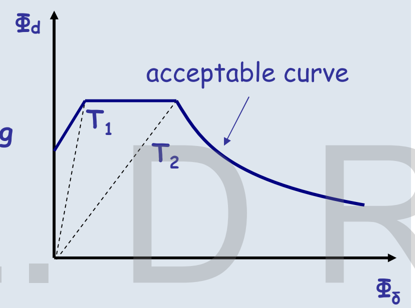
Proposed q factor values for existing structures

Applied codes of design (and construction)	Beneficial participation of infill walls (throughout the building)	Negative participation or absence of infill walls
1995 <… **New seismic code**	**3.00**	**2.30**
1985 < …< 1995 **Revised seismic code**	**2.30**	**1.80**
…<1985 **Old seismic code**	**1.80**	**1.30**

CAPACITY DEMAND



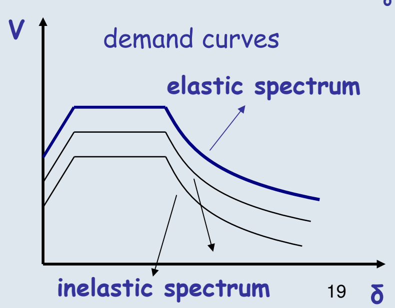
$$\Phi_\delta = \frac{T^2}{4\pi^2}\Phi_d\, g$$



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

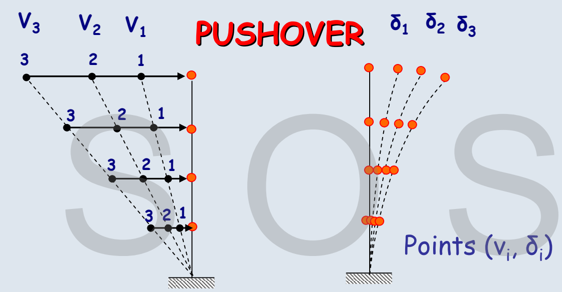
$\delta = \beta\, \Phi_\delta$

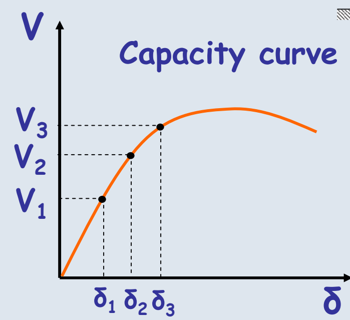
n	α	β
1	1	1
2	0.90	1.20
5	0.80	1.35

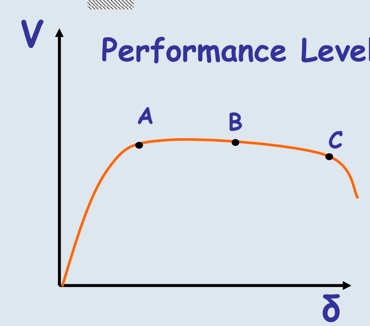


THE BASIS OF PUSHOVER ANALYSIS

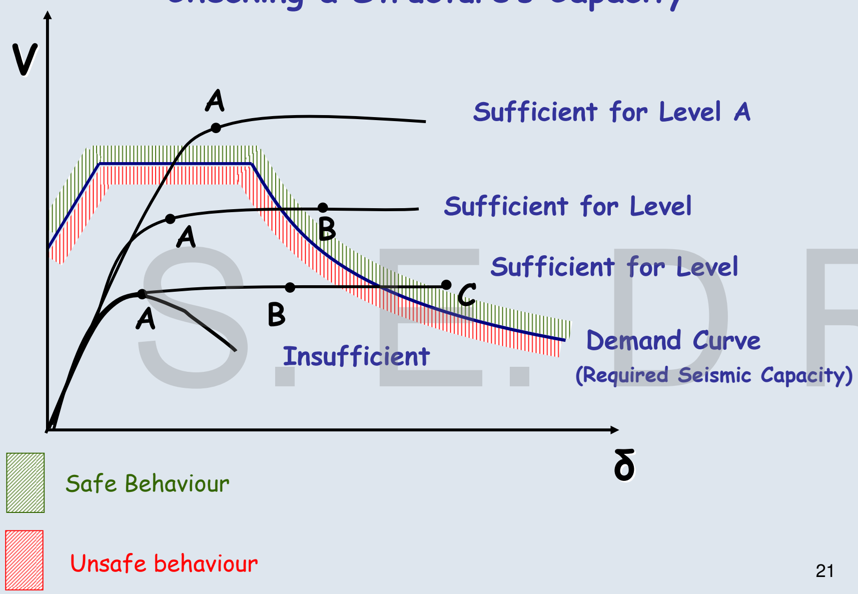
Graduate pushing (static horizontal loading) of structure up to failure



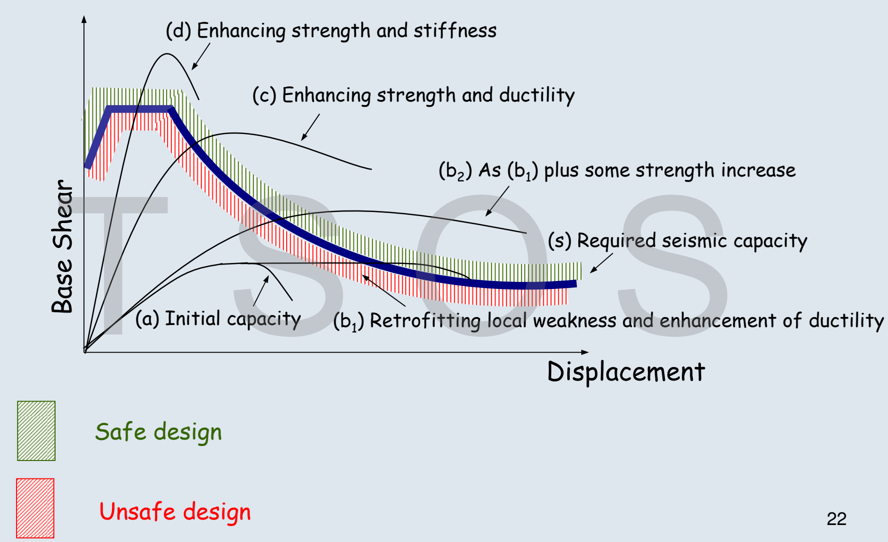




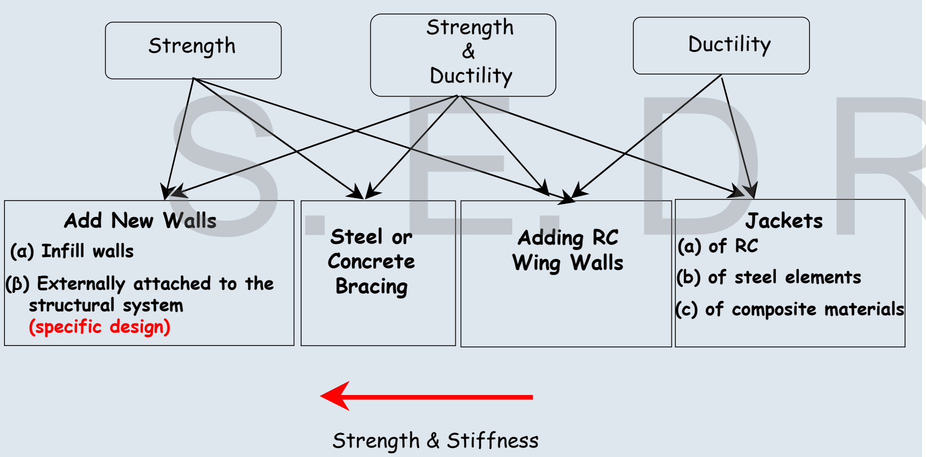
SAFETY VERIFICATION
Checking a Structure's Capacity



SEISMIC STREGHTENING STRATEGIES



SEISMIC STRENGHTENIG METHODS



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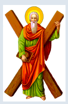
Seismic Assessment and Retrofitting of Existing Buildings

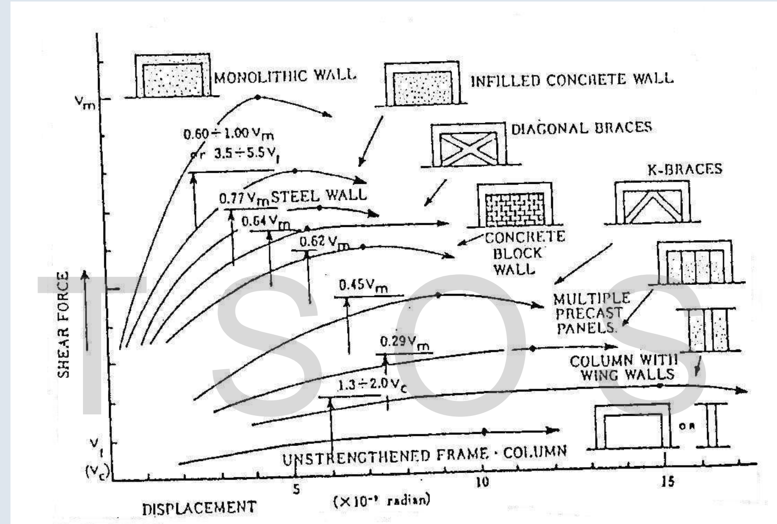
Under the Eurocodes Framework

Strengthening Methods and Techniques

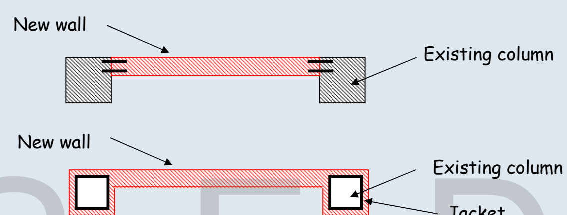


➤ Prof. Stephanos E. Dritsos

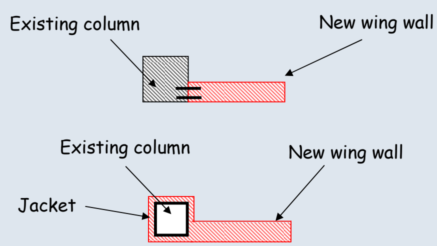
Department of Civil Engineering, University of Patras



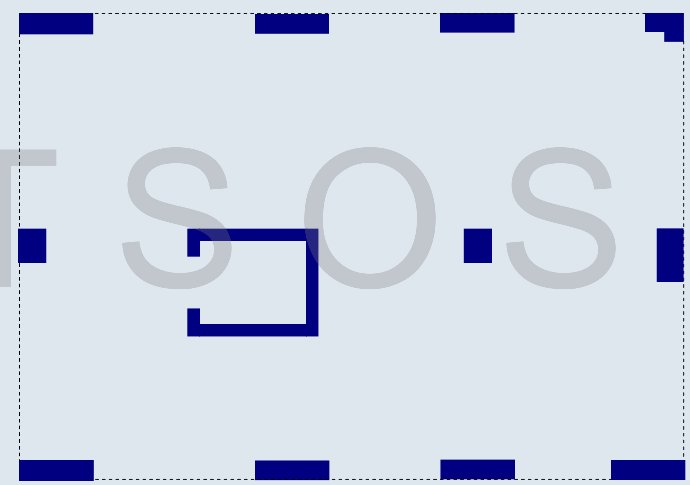
The relative effectiveness of strengthening



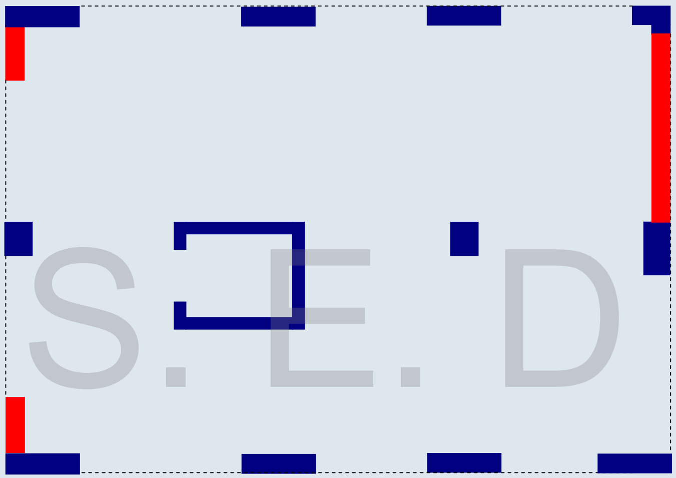
Infilling new shear walls



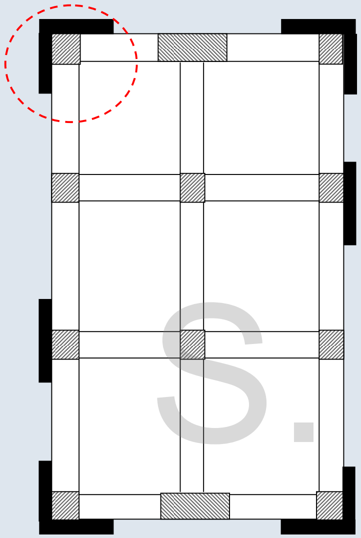
Addition of new wing walls

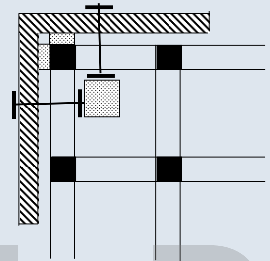


Existing vertical element configuration (PLAN)



Strengthening proposal

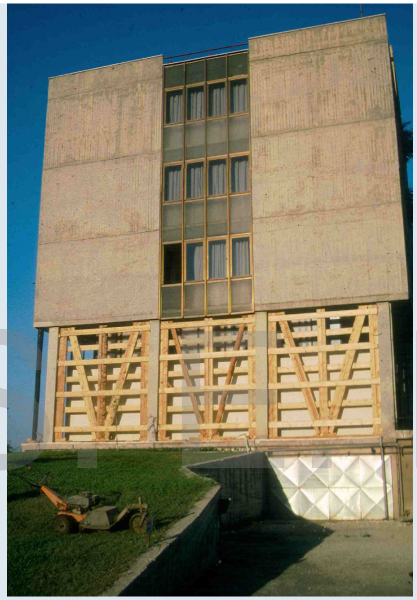




Schematic arrangement of connections between existing building and new wall

Addition of new external walls

Addition of a bracing system



Temporary support and stiffening of the damaged soft floor

13

Concrete jacketing in practice

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Reinforced concrete jackets

15

16

Total jacket

Roughening and the use of dowels

Bar buckling due to stirrup end opening

Jacket bar fracture

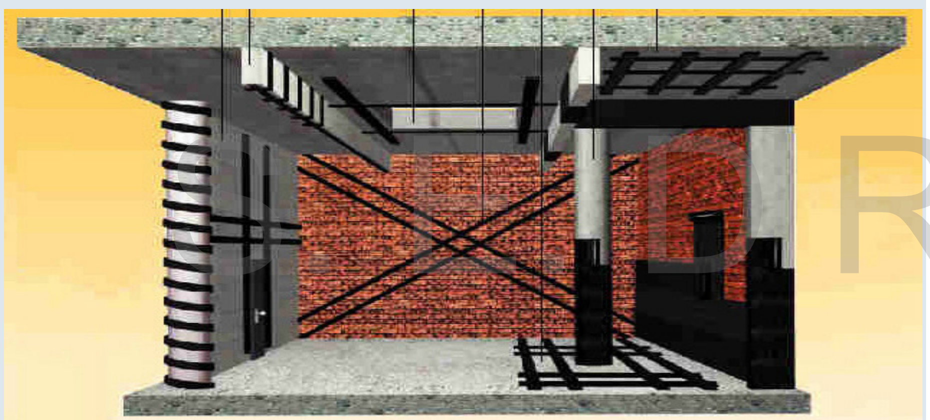
Welding of jacket's stirrup ends

Column repair

Construction of a steel cage around a column

Steel cage

Steel cage (temporary support)



FRP strengthening

JOINT STRENTHENING

JOINT STRENTHENING

Addition of steel plates

JOINT STRENTHENING

Addition of FRP
CEA, Sacley

JOINT STRENTHENING

Addition of FRP
CEA, Sacley

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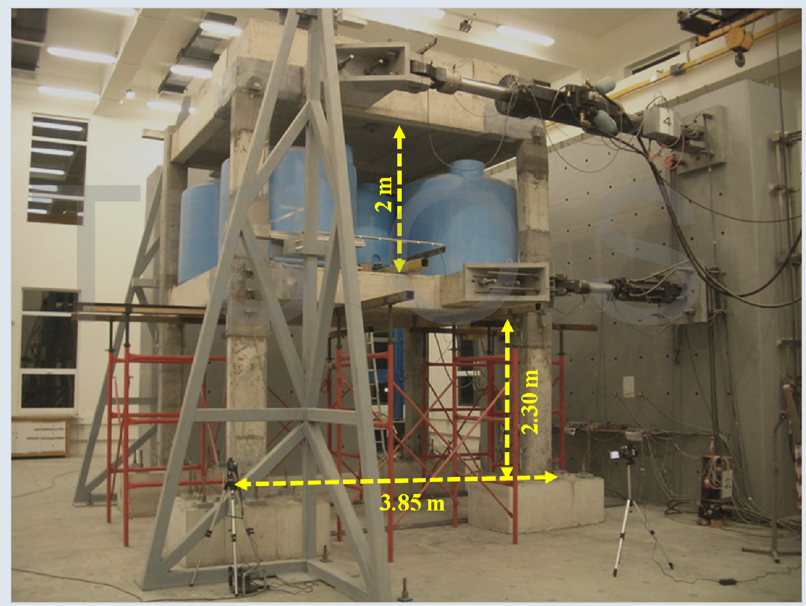

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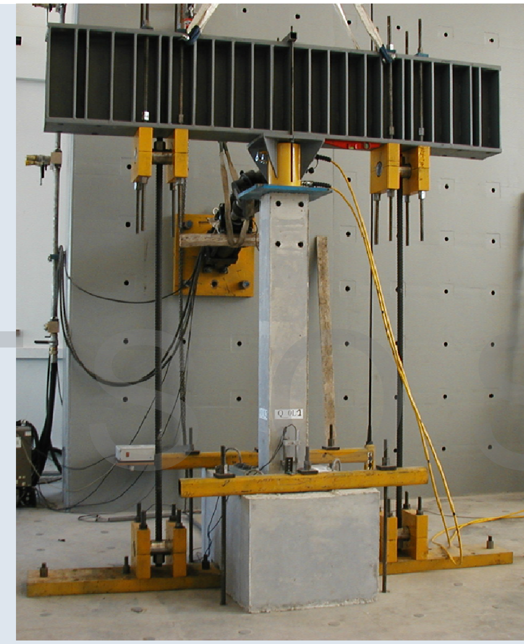

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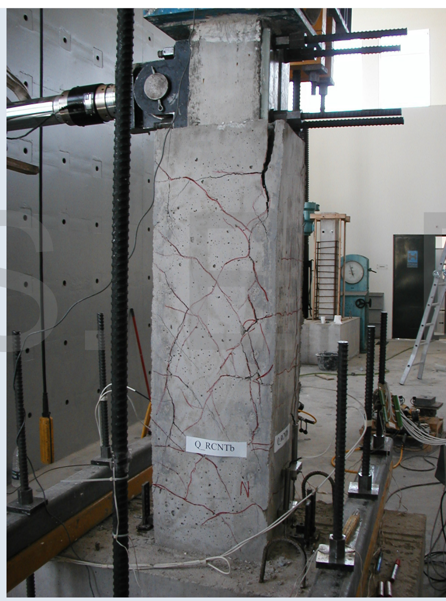

University of Patras, Structural Lab
Damage to the specimen with poured concrete, smooth interface without dowels

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S. DRITSOS

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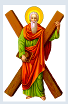
Under the Eurocodes Framework

Structural Design of Interventions



Prof. Stephanos E. Dritsos

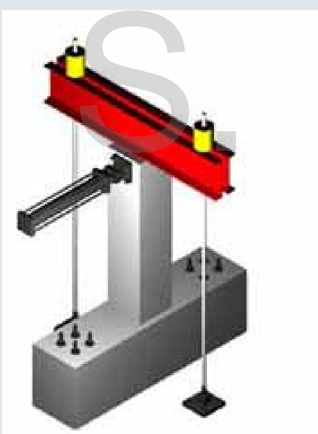
Department of Civil Engineering, University of Patras

VIENNA - BOKU, October 2012

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)

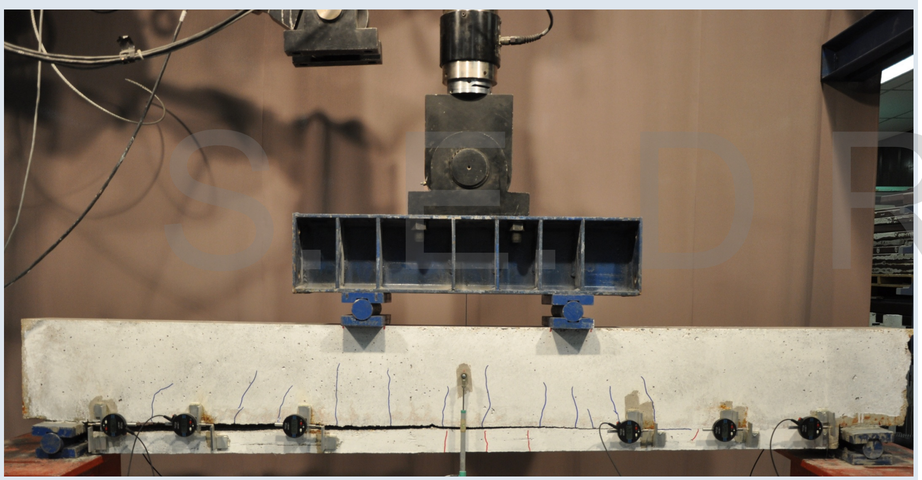


Damage to a specimen with shotcrete and dowels

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

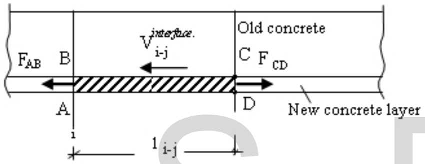
CONTROL OF A SUFFICIENT CONNECTION BETWEEN CONTACT SURFACES

$$S_d \leq R_d$$

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

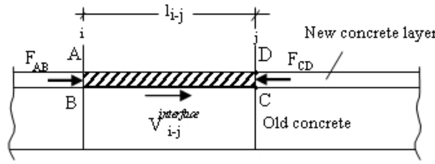
Interface Shear Force $\leq$ Interface Shear Resistance

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



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

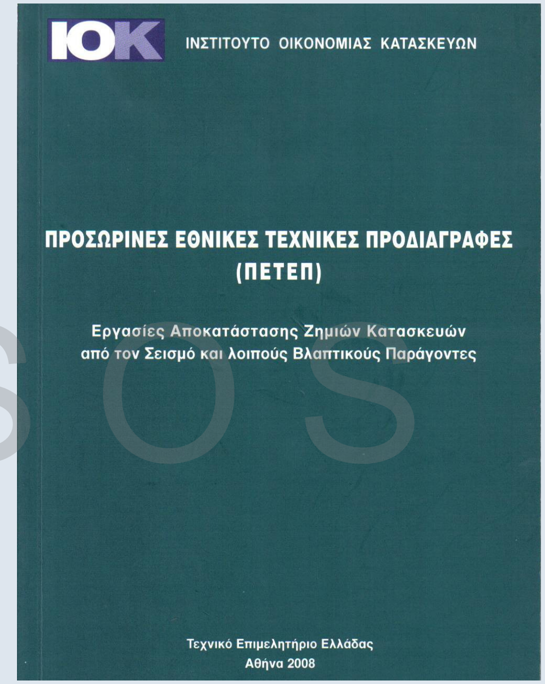
(a) strengthening in the tensile zone



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

(b) strengthening in the compressive zone

Technological guidelines for repairs and strengthening:

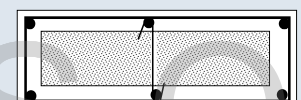


Roughening by sandblasting

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

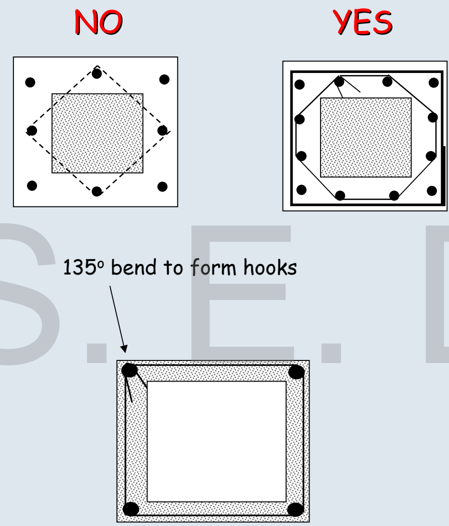
Concrete jacketing in practice

Total jacket



Inserting intermediate links in sections with a high aspect ratio

Inserting intermediate stirrups in square sections



Bar buckling due to stirrup ends opening

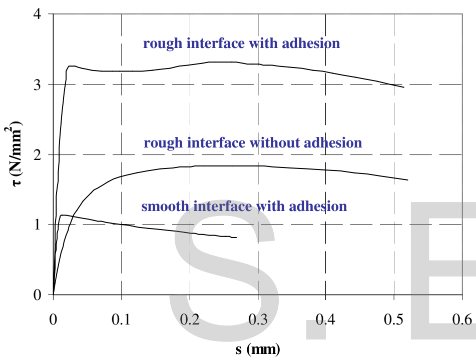
Welding of jacket's stirrup ends

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

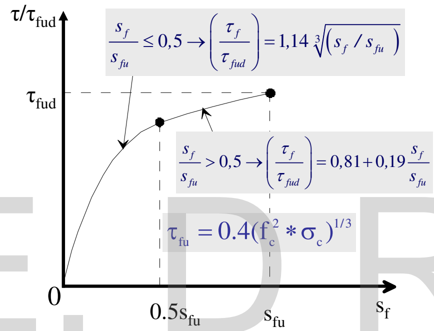
Mechanisms

- 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

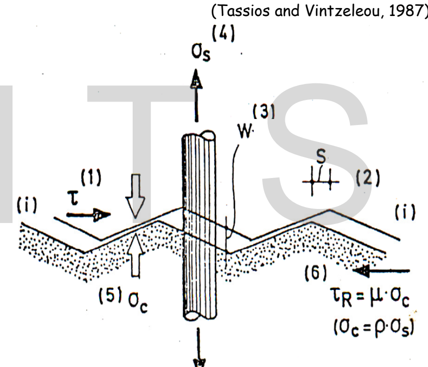
REINFORCED INTERFACES

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)

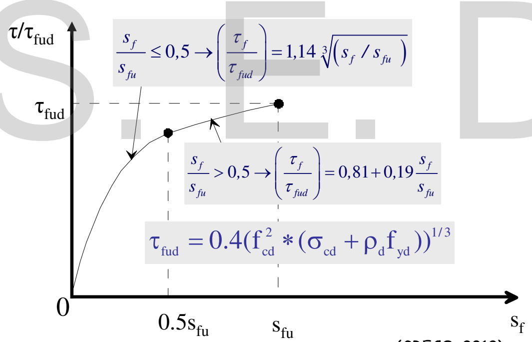
(4) Tensile Strength is Activated in the Steel Bar

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

(6) Frictional Resistance is Activated

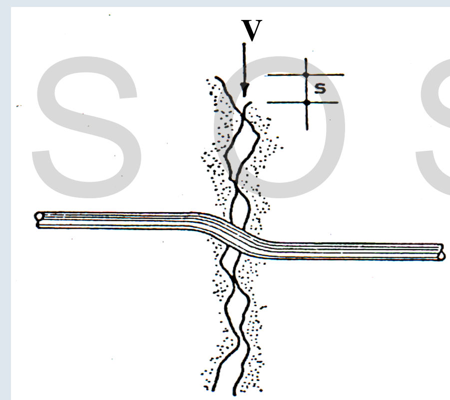
Reinforced Interfaces

Frictional resistance



(GRECO, 2012)

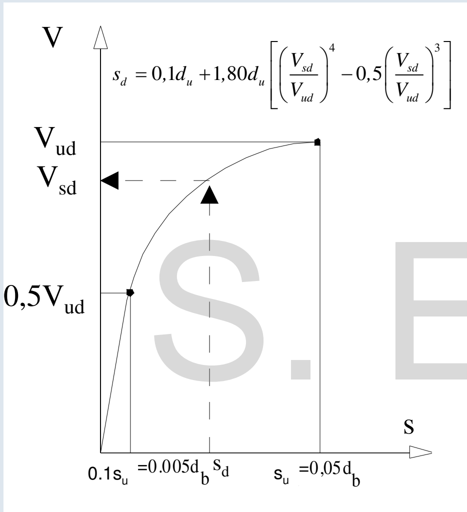
Reinforced Interfaces

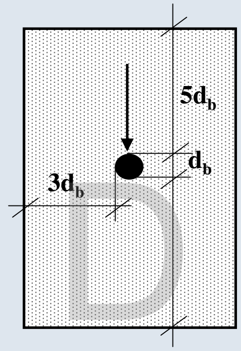


Dowel action

Shear Resistance

for Dowel Action as a function of the interface slip



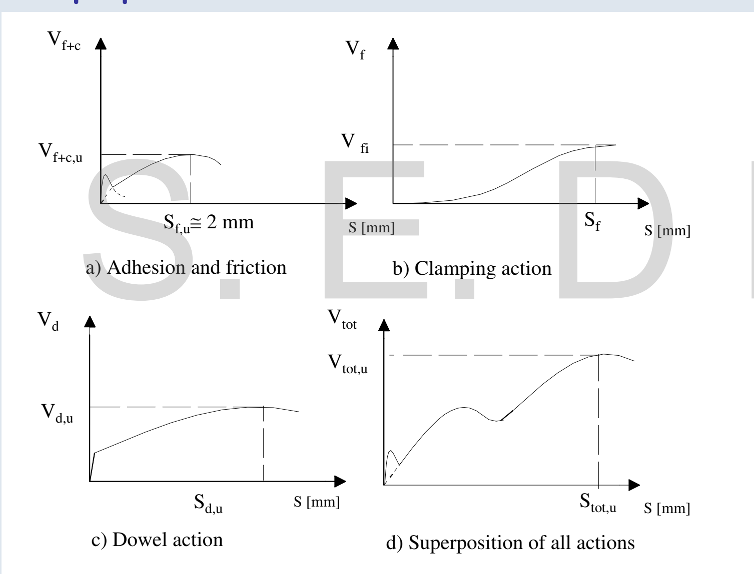


A minimum concrete cover is necessary for full activation of dowel action

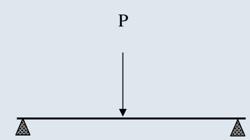
Use of steel dowels and roughening the surface of an original column

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

Superposition of shear resistance mechanisms

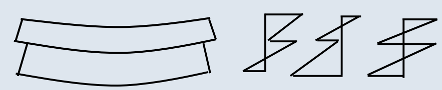


$$V_{tot} = \beta_D V_d + \beta_f V_f$$

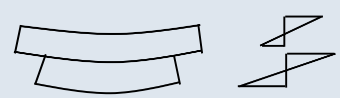


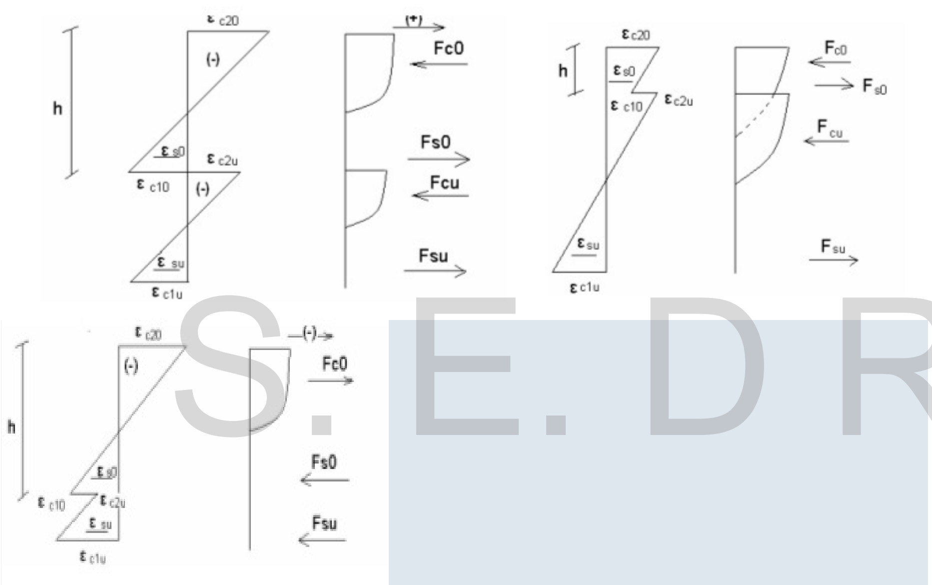
Full interaction

Partial interaction



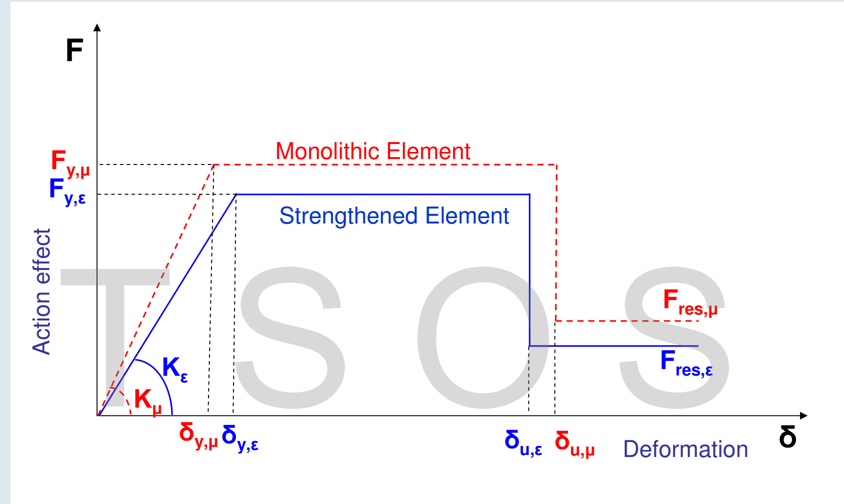
Independent action





Possible strain and stress distributions

CAPACITY CURVES



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

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:**

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

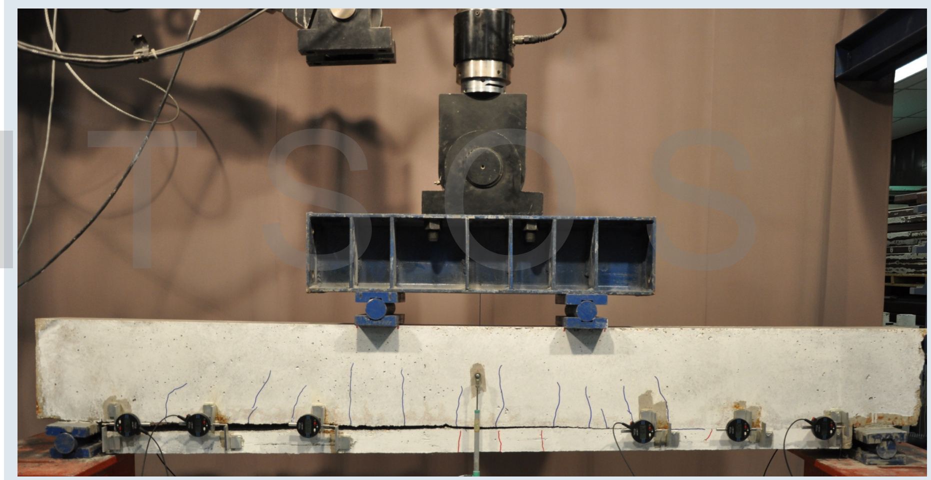
- **For the Displacement:**

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

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

$$(EI)_{strengthened} = k_k\,(EI)_M$$
$$R_{strengthened} = k_r\,R_M$$
$$\delta_{i,strengthened} = k_{\delta i}\,\delta_{i,M}$$

Beam strengthened with a new concrete layer



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

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

Addition of new concrete layers

Capacity assessment

- Considering slip at the interface
- Approximations using monolithic behaviour factors

For slabs:

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

For other elements:

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

Reinforced concrete jackets

Reinforced Interfaces

Bent down bars welded to the old and new reinforcement

Compressive side interface resistance of a jacket

$$F_{cm} = \underbrace{4u_o \mu f_{ctm}}_{\text{Friction}} + \underbrace{10 n_b \frac{A_{sb}}{h_s}}_{\text{Welded bent down bars}} + \underbrace{n_D F_{uD}}_{\text{Dowels}}$$

Minimum jacket stirrups

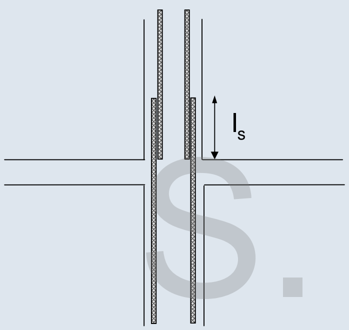
According to EC-2 and EC-8

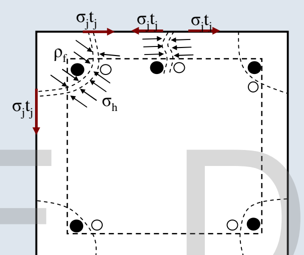
and $\frac{A_{sw}}{\alpha_{sw}} \geq \frac{t \cdot f_{ctm}}{f_{ywd}}$, i.e. $\alpha_{sw} \leq 0.8 \left(\frac{f_{ywd}}{f_{ctm}} \right) \cdot \frac{d_h^2}{t}$

Monolithic factors (approximate method)

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

Restoration of insufficient lap splice lengths





$$T_{req.} = (1-\lambda_s)\ A_b\ f_s$$

$$T = \mu \left(\rho_f\ l_s \right) \sigma_h \rightarrow \sigma_{h,req.} = \frac{(1-\lambda_s)\ A_b\ f_s}{\mu\ \rho_f\ l_s}$$

$$\sigma_j t_j = \sigma_h\ B \qquad \text{where:} \quad \beta = {\rho_f}/{B}$$

$$\left(A_j / s \right)_{req.} = \frac{(1-\lambda_s)\ A_b\ f_s}{\beta\ \mu\ l_s\ \sigma_j}$$

$$\left(\frac{A_j}{s_w} \right)_{req.} = \frac{12}{(s_d : s_u)} \left(\frac{f_{sy}^3}{f_u\ f_c^2} \right) \left(\frac{d_s^2}{a_N \ell_s} \right)^3 (a_N)$$

$$(A_j / s)_{req.} = 1.3 \left[k_1 \left(\frac{f_{sy}}{f_c} \frac{d_s}{\ell_s} \right) - 0.4 \frac{c}{d_s} - 0.30 \right]^2 \frac{f_c^2\, d_s^2}{k_2 E_j f_{ctm}}$$

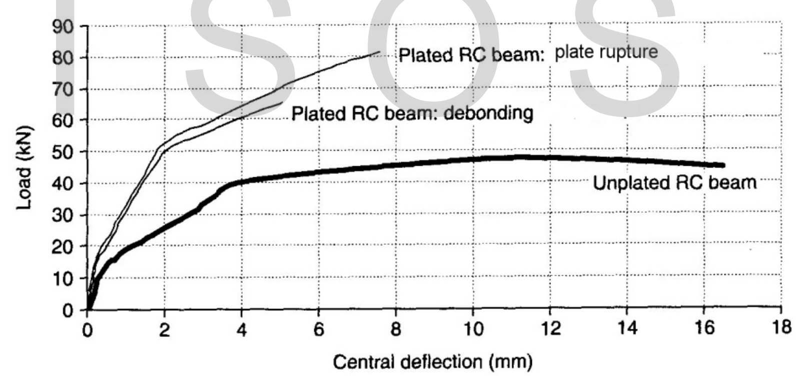
For jackets $\quad t_j = \left(A_j / s \right)_{req.}$

Flexural strengthening

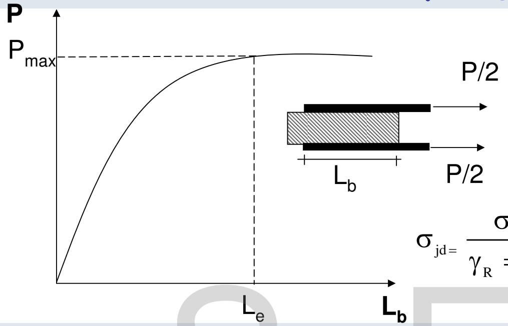
Debonding of glued FRP laminates

Spalling of the concrete cover at the edge of the FRP laminate

Experimental results for strengthened beams



Check for Debonding



$$L_e = \sqrt{\frac{E_j\, t_j}{2 f_{ctm}}}$$

$$P_{max} = k\, f_{ctm}\, b_j\, L_e$$

$$\sigma_{jd} = \frac{\sigma_{crit}}{\gamma_R = 1,2} = \frac{P_{max}}{1,2\, b_j t_j} \cong 1,15\sqrt{\frac{E_j\, f_{ctm}}{2\, t_j}}$$

As an example, let us consider a beam of C16/20 concrete strengthened on the tensile side with a carbon FRP of thickness t_j = 1 mm and width $b_j = 1/2b_w$.

$$f_{ctm} \cong 0.3\, f_{ck}^{2/3} = 0.316^{2/3} = 1.92\ \text{MPa}$$ and

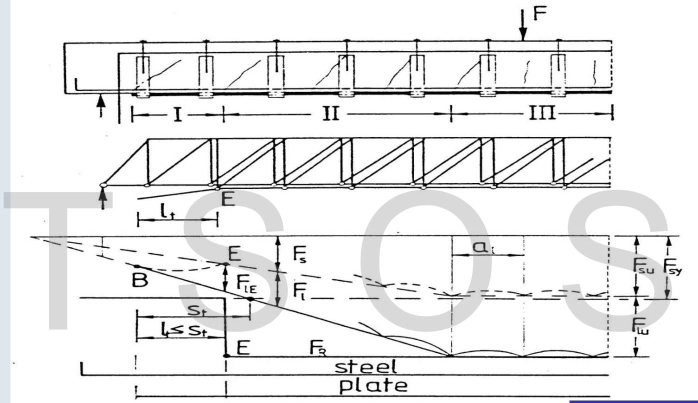
$$\sigma_{j,crit} = 1.15\sqrt{\frac{200 \times 1.92 \times 10^3}{2}} = 504\ \text{MPa}$$

- This technique is useful when a new opening is cut in a slab or a wall

$$t_j \uparrow \Rightarrow \sigma_{j,crit} \downarrow$$

Check for Spalling

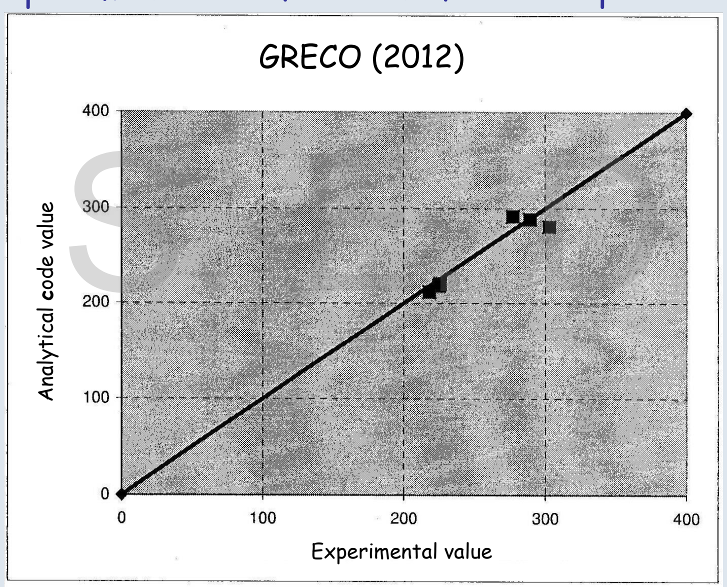
$V_{sd,end} \le V_{cd,end}$ $\quad$ $M_{sd,end} \le 0.67 M_{Rd,end}$



Rostasy, 1997

$$V_{sdj} = \frac{A_j\, \sigma_{jd}}{A_{so}\, f_{ydo} + A_j \sigma_{jd}} V_{sd,end.}$$

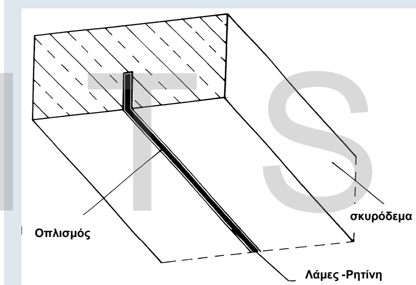
Experimental verification of code expression

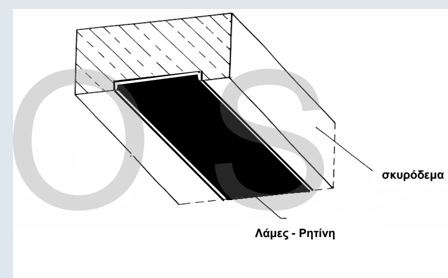


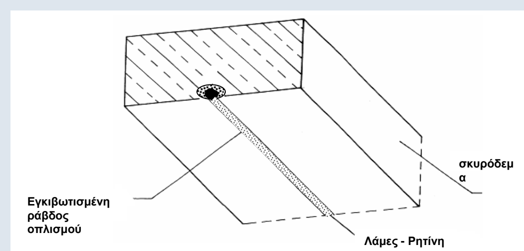
Mitolidis, PhD Thesis, Aristotle University, 2009

Flexural strengthening by placing reinforcement in a "groove" (near surface mounting (NSM))

(Not covered by GRECO)







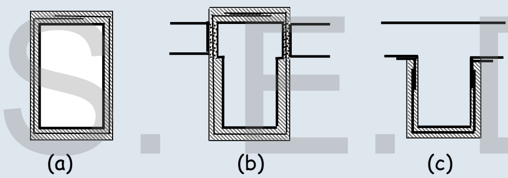
Increasing Shear Capacity

Increasing oblique compressive strength ($V_{sd}>V_{Rd2}$)

- With confinement

$$f_{ck,c}=(1{,}125+1{,}25a\omega_w)f_{ck}$$

- By adding new layers of concrete
  - closed jacket (recommended)
  - three sided jacket

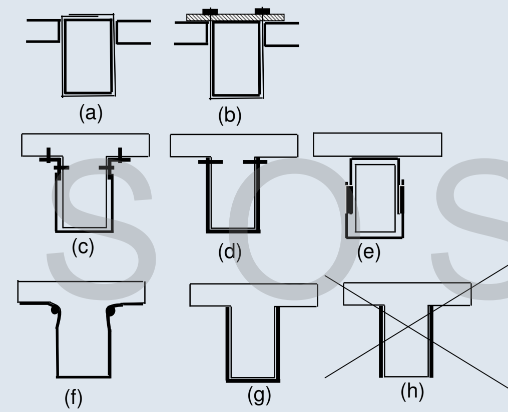


Illustrative ways of strengthening against shear failure:
(a), (b) Closed strengthening, (c) Open strengthening

$$V_{sd}\leq\frac{1}{\gamma_{Rd}}\left(V_{Rd,r}+V_{RM}\right)$$

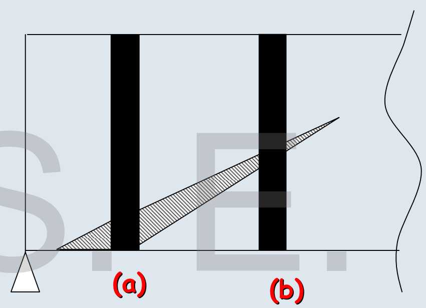
Increasing shear reinforcement ($V_{sd}>V_{Rd3}$)

- With additional concrete layers
- With additional steel plates or FRP laminates



Illustrative ways of strengthening against shear failure :
(a), (b) “closed” strengthening, (c), (d),(e),(f) “open” strengthening with anchorage
& (g) “open” strengthening exceptionally accepted under specific conditions

Closed Shear Strengthening using FRPs



- The stress in the fibres depends on the width of the crack where the fibres bridge the crack
- There is no stress redistribution
- Fibres in position (a) would fail before fibres in position (b) have any significant stress

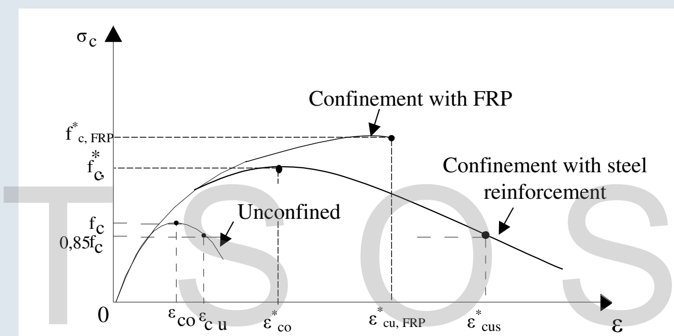
➡ Mean value of fibre strength ≈ ½ ultimate strength of the fibres ➡ k_v = 0,5

Confinement by Steel Cage

FRP Strengthening

Stress concentrations at corners of FRP sheets leads to failure

Requirement to round column's corners to minimise stress concentrations



Steel reinforcement confinement $\varepsilon^*_{cu} = 0{,}0035 + 0{,}1\alpha\omega_w$

Confinement using carbon FRP $\varepsilon^*_{cu} = 0{,}0035\ \ (f^*_c : f_c)^2$

Confinement using glass FRP $\varepsilon^*_{cu} = 0{,}007\ \ (f^*_c : f_c)^2$

where $f^*_c = \left(1{,}125 + 1{,}25\alpha\omega_w\right) f_c$

Procedure to determine required confinement when an increase in ductility is desired for a specific q factor

- Calculate the required behaviour index $q_\mu = q/q_o$
  (q_o is the overstrength value according to EC8)
- Calculate the required ductility index in terms of displacements:

$$\mu_d = \begin{cases} q_p & \text{when } T > T_2 \\ 1 + T_2/T(q_p - 1) & \text{when } T < T_2 \end{cases}$$

- Calculate the required $\mu_{1/r}$ value as follows:
  $(\mu_d - 1)/(\mu_{1/r} - 1) = 3$
- Calculate the required maximum concrete compressive strain:

$$\varepsilon^*_{cu} = 2{,}2 \cdot \mu_{1/r} \cdot \varepsilon_{sy} \cdot v$$

- The confined volumetric mechanical ratio ω_w is determined as follows:

Steel reinforcement confinement: $\varepsilon^*_{cu} = 0{,}0035 + 0{,}1 \cdot \alpha \cdot \omega_w$

Carbon FRP confinement: $\varepsilon^*_{cu} = 0{,}0035\left(f^*_c : f_c\right)^2$

Glass FRP confinement: $\varepsilon^*_{cu} = 0{,}007\left(f^*_c : f_c\right)^2$ with $f^*_c = (1{,}125 + 1{,}25 \cdot a \cdot \omega_w) f_c$

STRENGTHENING OF BEAM-COLUMN JOINT

By the addition of diagonal steel sections

STRENGTHENING OF BEAM-COLUMN JOINT

By epoxy injection

CEA, Sacley

S. E. DRITSOS

STRENGTHENING OF BEAM-COLUMN JOINT

By the addition of steel plates

STRENGTHENING OF BEAM-COLUMN JOINTS

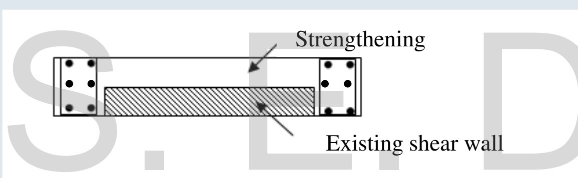
By gluing FRP laminates

University of Patras, Structural Lab

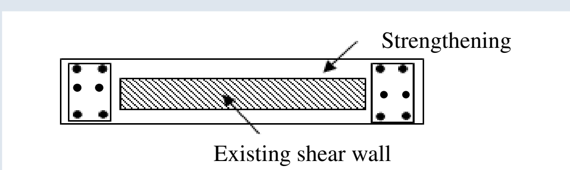
CEA, Sacley

Strengthening of Shear Walls

- Either by the the addition of columns at the ends of the shear wall or by one sided strengthening with an additional layer and end columns



- Full jacket strengthening (recommended)



Adding Simple Infill

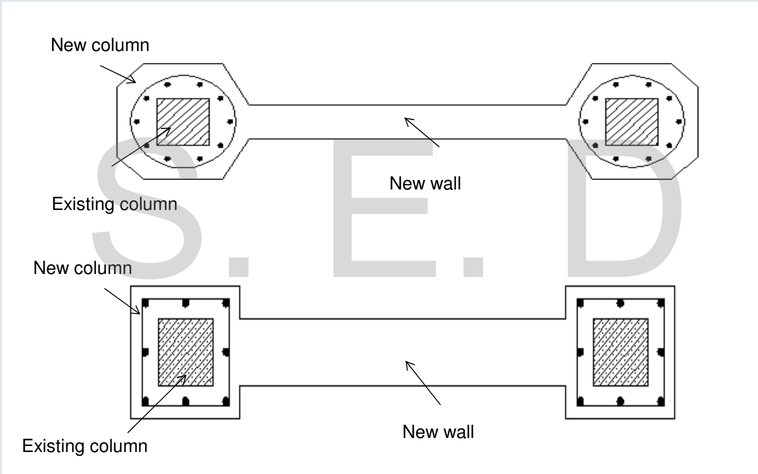
- Addition of walls from: a) Unreinforced or reinforced concrete (cast in situ or prefabricated)
  b) Unreinforced or reinforced masonry
- No specific requirement to connect infill to the existing frame
- Modelling of infills by diagonal strut
- Low ductility of infill. Recommended $\mu \leq 1,5$

WARNING

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

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



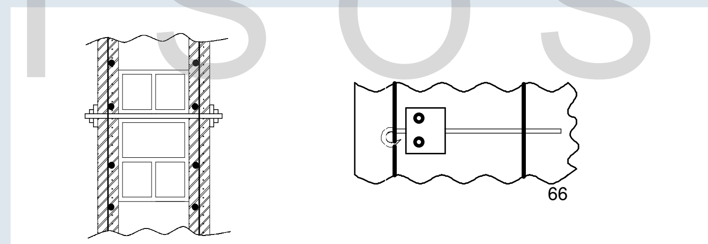
Strengthening of existing masonry infills

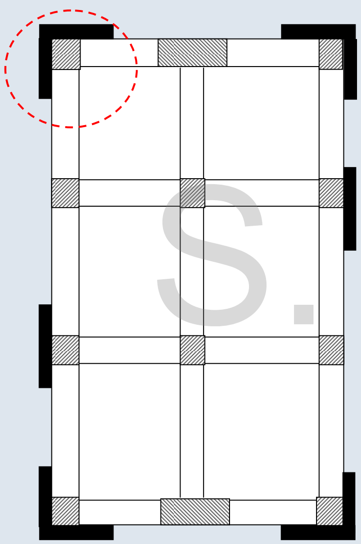
- Reinforced shotcrete concrete layers applied to both sides of the wall
  - Minimum concrete thickness 50 mm
  - Minimum reinforcement ratio $\rho_{vertical} = \rho_{horizontal} = 0{,}005$

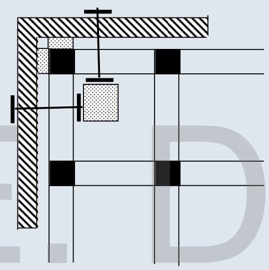
Essential to positively connect both sides by bolting through the wall

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

All new construction must be suitably connected to the existing foundation







Schematic arrangement of connections between existing building and new wall

Addition of new external walls

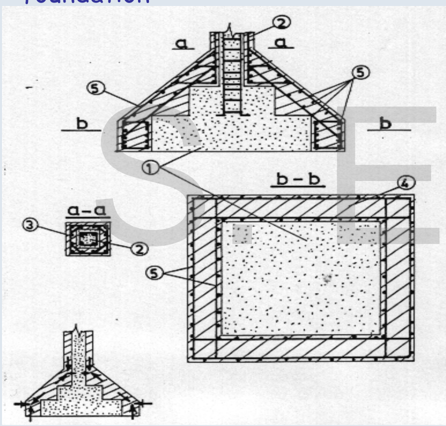
Addition of a bracing system

STRENGTHENING OF FOUNDATIONS

- When insufficient bearing surface in contact with the soil
- When insufficient foundation height

Increase the dimensions of the foundation usually in combination with other possible strengthening procedures

Cannot strengthen structures without strengthening the foundation



$$\sum A_{sw} \geq \frac{P_n \tan\alpha}{f_{ywd}}$$

Indicative strengthening by adding a jacket to the foundation and associated column

This presentation has been uploaded to:

www.episkeves.civil.upatras.gr

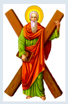
It is entitled "VIENNA - BOKU 2012"

Click on **"PRESENTATIONS"** on the side menu to find it

Seismic Assessment and Retrofitting of Existing Buildings

Under the Eurocodes Framework

Earthquake Damages



➢ Prof. Stephanos E. Dritsos

Department of Civil Engineering, University of Patras

VIENNA - BOKU, October 2012

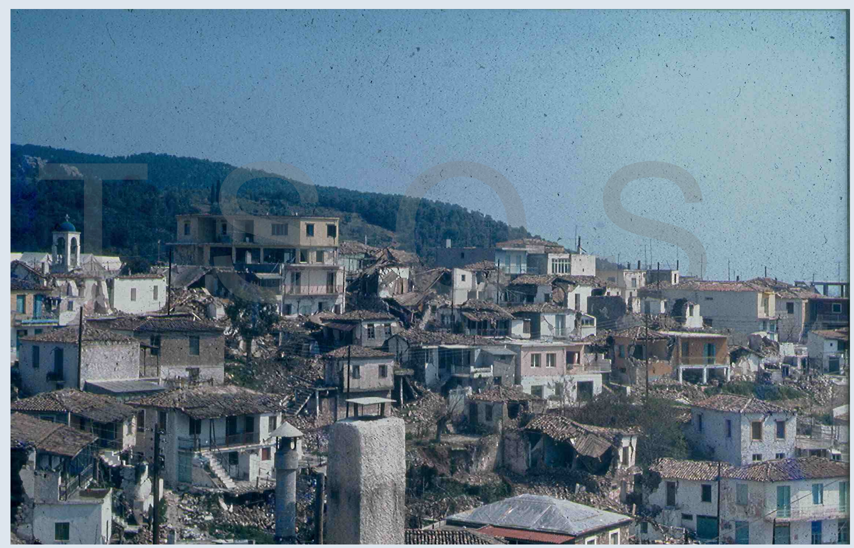
http://www.boston.com/bigpicture/2010/01/earthquake_in_haiti.html

Haiti

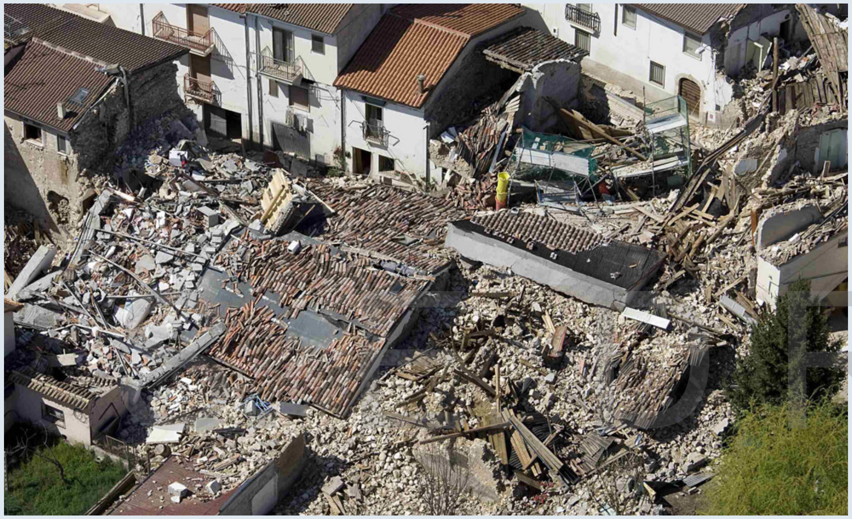
A view of a damaged neighbourhood in the Canape-Vert, Port-au-Prince, Haiti area on January 13, 2010, the day after the earthquake

http:www.boston.com/bigpicture/2009/04/the_laquilaa_earthquake.html11

L' Aquila



1981 Alkyonides earthquakes 6.7R and 6.4R



http:www.boston.com/bigpicture/2009/04/the_laquilaa_earthquake.html11

L' Aquila

http:www.boston.com/bigpicture/2009/04/the_laquilaa_earthquake.html11

L' Aquila

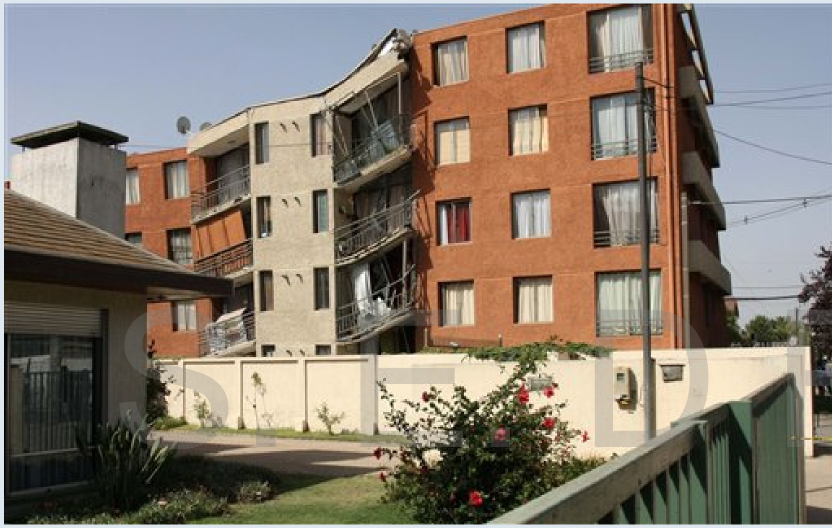
http://www.boston.com/bigpicture/2010/01/earthquake_in_haiti.html

Haiti

http://www.boston.com/bigpicture/2010/01/earthquake_in_haiti.html

Haiti

Crisscross cracking



http://www.cbsnews.com/2300-202_162-10002626-32.html?tag=page

Chile

Full collapse of the ground floor

1999, Parnitha (Athens) 5.9R

Full collapse of the ground floor

1999, Parnitha (Athens) 5.9R

Full collapse of the ground floor

http://www.reluis.it/doc/pdf/Rapport_fotografico_V1.2.pdf

L' Aquila

Diagonal cracks in masonry walls of the building's second level

G. Manfredi, M. Dolce (eds), The state of Earthquake Engineering Research in Italy: the ReLUIS-DPC 2005-2008 Project, 469-480, © 2009 Doppiavoce, Napoli, Italy

L' Aquila

Examples of damage to masonry infills in RC multi-storey buildings

1995 Aigio 6.1R

1995 Aigio 6.1R

Note the holes opened in slab for emergency team access

1995 Aigio 6.1R

Pancake collapse, the most fatalities

17

http://www.boston.com/bigpicture/2010/01/earthquake_in_haiti.html

Haiti

People look at what remains of a six storey communications building on January 13, 2010 in Port-au-Prince, Haiti

18

1999, Parnitha (Athens) 5.9R

Pancake collapse, the most fatalities

19

1981 Alkyonides earthquakes 6.7R and 6.4R

Partial collapse due to failure of columns

20

Inadequate column bar lap spice lengths 21

Detail of inadequate column bar lap spice lengths 22

Two identical buildings, one collapsed, one minor damage 23

1986 Kalamata 6.2R

Two identical buildings, one the ground floor collapsed, the other no damage 24

Soft Story – Strong Beams Weak Columns – Joint Failure

Soft storey

http://www.reluis.it/doc/pdf/Rapport_fotografico_V1.2.pdf

L' Aquila

Soft storey mechanism in a three storey RC building

The first storey is characterized by openings (entrance and garages) unlike the other storeys.

The displacement demand was concentrated at the ground floor level.

Soft storey effect

Soft storey effect

Note no damage to the 100 year old building in background due to the building's natural frequency being out of phase with the frequency at the maximum density of seismic acceleration

Connection failure

Connection failure

Bad quality concrete

Absence of stirrups

33

Absence of stirrups in critical zone and lap splice region

34

1995 Aigio 6.1R

Column shear failure

35

1986 Kalamata Earthquake, 6.2R and 5.6R

Note the inadequate diameter and spacing of stirrups
Column shear failure

36

Inadequate stirrups

37

High compression and inadequate stirrups

Plastic Hinge at Column End

38

High compression and inadequate stirrups

Plastic hinge at column end

39

First level damage

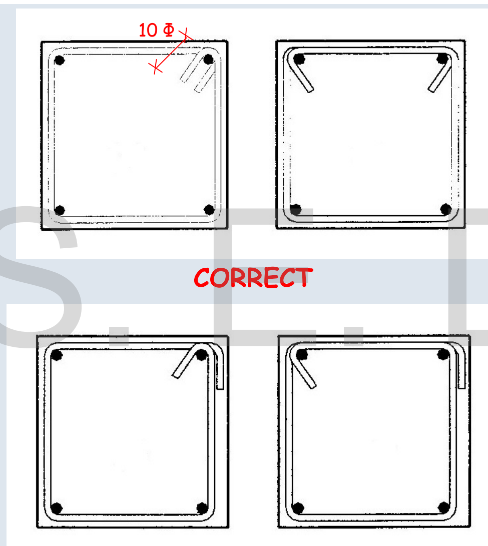
Lack of concrete cover causing spalling in column corner

40

S.E. DRITSOS

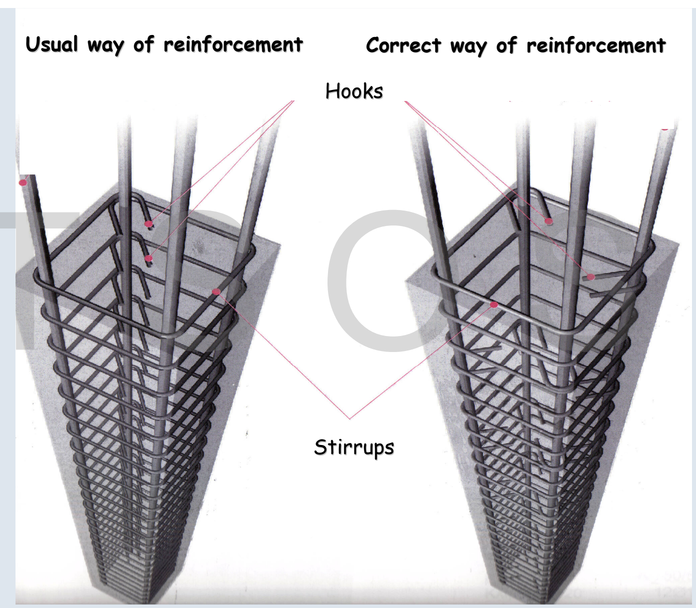
Opening of stirrups due to inadequate hooks at ends

Stirrups with inadequate hook angles



INCORRECT

135 degree hook angle at both ends of stirrups is correct



Types of stirrups

Hooks should be sequenced in alternative corners

Reinforcement congestion

1986 Kalamata 6.2R

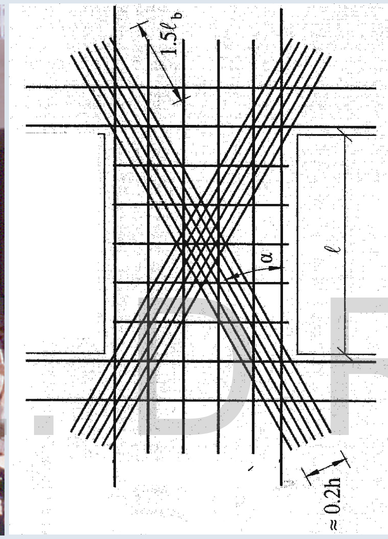
1995 Aigio 6.1R

High vulnerability of short columns

1999, Parnitha (Athens) 5.9R

Short column effect

Masonry infills reduce the effective column height and create a short column effect



Special bidiagonal reinforcement required in short columns

http://www.reluis.it/doc/pdf/Rapporto_fotografico_V1.2.pdf

L' Aquila

External joint damage

Absence of stirrups in joint results in buckling of column longitudinal bars. The concrete cover spalling is due to the anchorage of longitudinal bars in horizontal beam.

Lack of stirrups in critical joint region

Lack of stirrups in critical joint region

Lack of stirrups in critical joint region

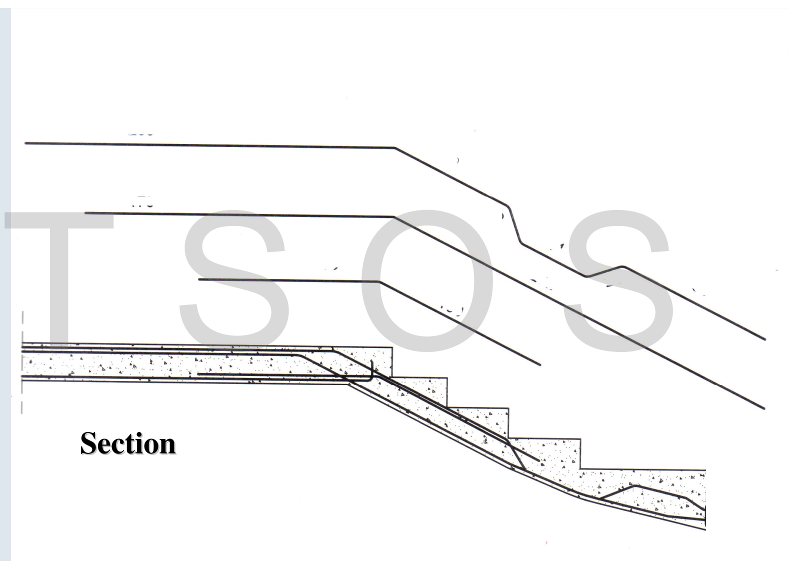
53

Lack of stirrups in critical joint region

54

Incorrect reinforcement detailing in stairs

55



Correct reinforcement detailing in stairs

56

S. E. DRITSOS

Construction joint required to separate wall and balcony

57

Warning: Supervision does not end with last concrete pour

Damage by electrician as no specific path for cables

58

Water drainage pipe in column

Not allowed by codes

59

1999 Kocaeli (Turkey) 7.4 R.

Buckling of railway tracks crossing the fault line

60

1999 Kocaeli (Turkey) 7.4 R.

Building tilted due to soil liquefaction (Erdik, 2000)

1999 Kocaeli (Turkey) 7.4 R.

Due to liquefaction, the buildings sank into the ground and the displaced soil heaved (Erdik, 2000)

1999 Kocaeli (Turkey) 7.4 R.

Displacements in excess of the bearing width leading to the collapse of the bridge (Erdik, 2000)

If you are on the limit,
strong shutters may save your life!!!

Fatalities can also occur without damage to the building

Mind to get under the table when you feel shaking from an earthquake

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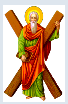
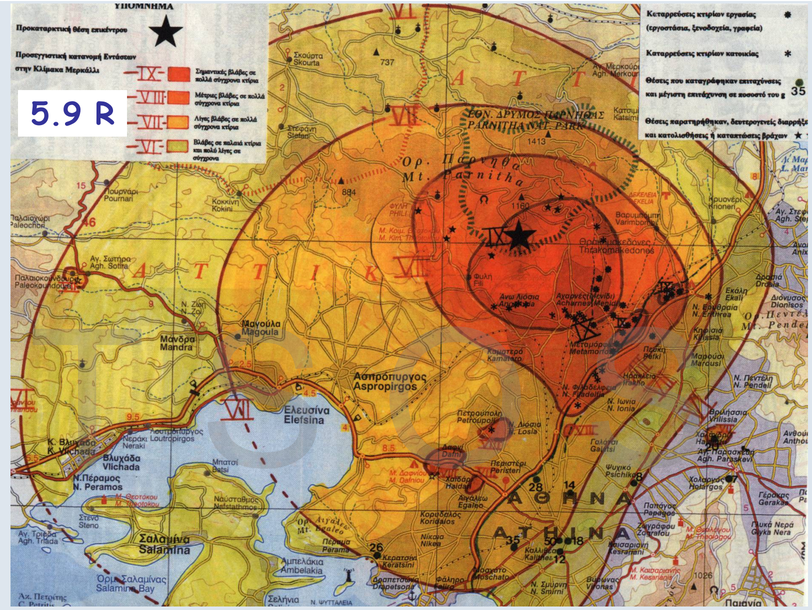
Seismic Assessment and Retrofitting of Existing Buildings

Under the Eurocodes Framework

1999 Parnitha (Athens) Earthquake
Post-Earthquake Survey

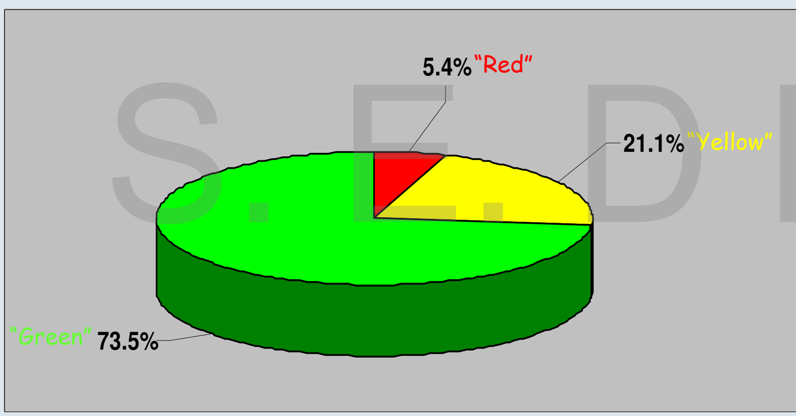


➢ Prof. Stephanos E. Dritsos
Department of Civil Engineering, University of Patras

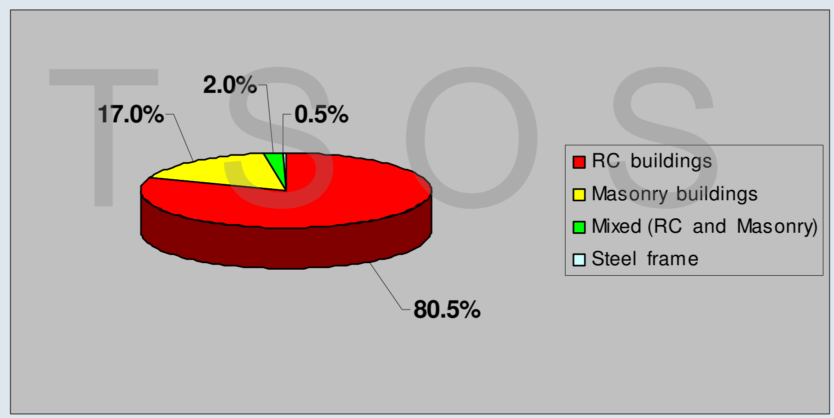


Recorded acceleration up to 0.35g (25 kilometres far from the epicentre)
Estimated acceleration at the epicentre 0.7-1.0g

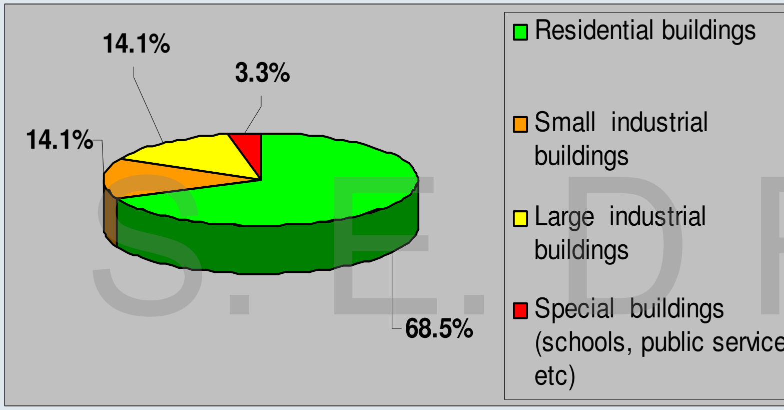
Buildings Inspected
915 in total



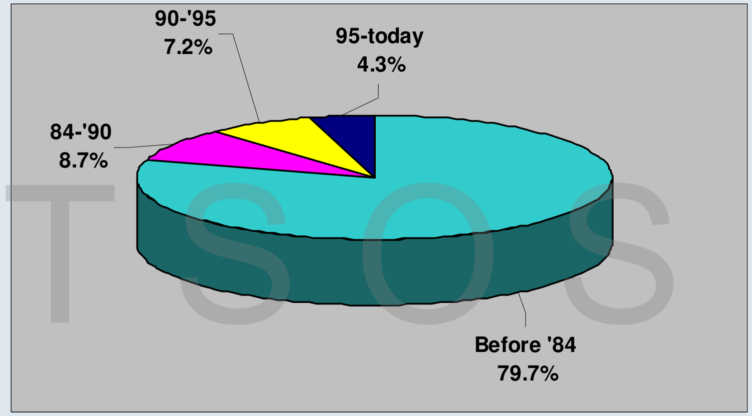
Building classification regarding construction material



Percentage of buildings regarding their type of use



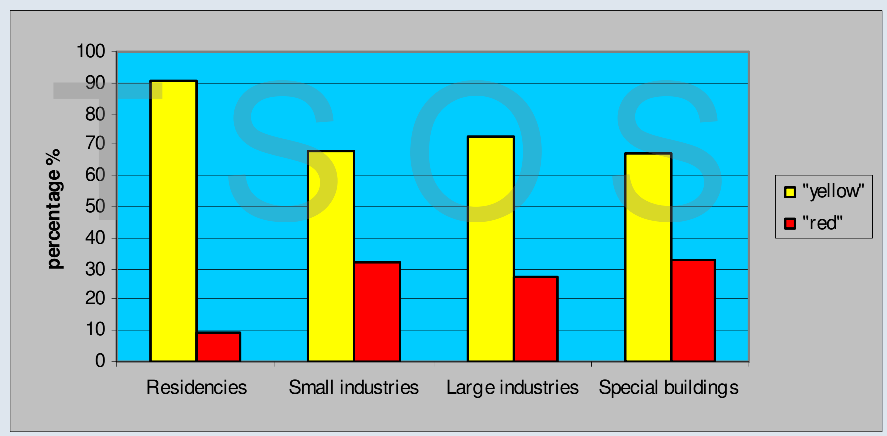
Year of construction of RC damaged buildings



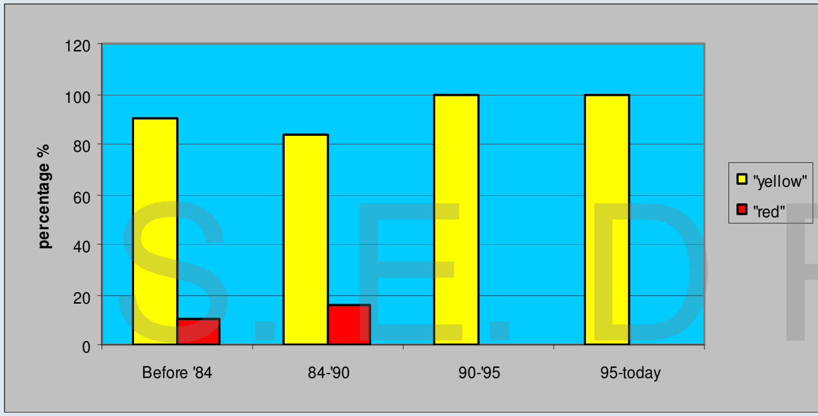
Definition of usability characterization

GREEN	The original seismic capacity of the building has not been decreased. The building is immediately usable and entry is permitted without restriction.
YELLOW	The seismic capacity of the building has deteriorated and repair measures should be taken. However, the vertical load carrying system of the building does not appear to have suffered. Thus the building is not expected to collapse suddenly. Usage is temporarily permitted under special restrictions.
RED	The building is unsafe and entry is prohibited. Lateral and vertical load carrying capacities of the building have both been substantially reduced. The building may be subject to sudden collapse and must be considered as dangerous. Decision for demolition or repair/strengthening will be made on the basis of a more thorough examination.

Usability classification of RC damaged buildings with respect to their type of use

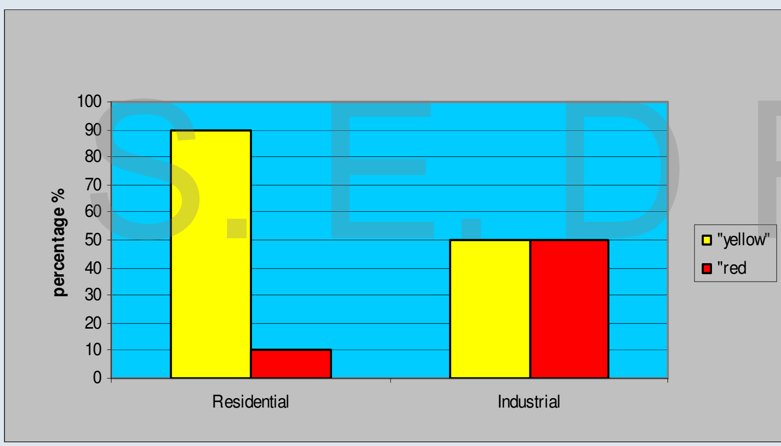


Usability classification related to the year of construction

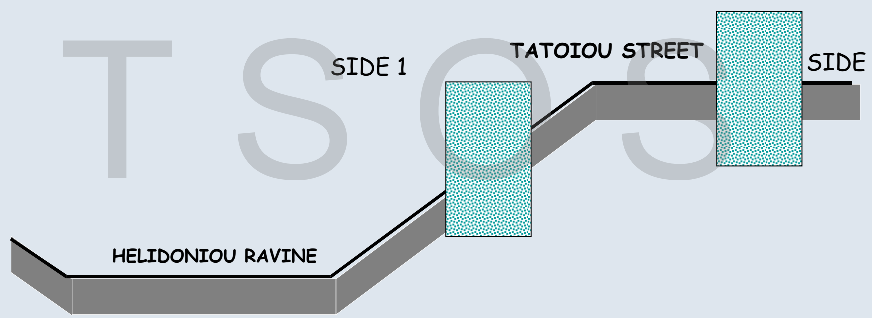


Usability characterization of buildings with stiffness irregularity in plan

Usability characterization of buildings with geometric irregularity in plan

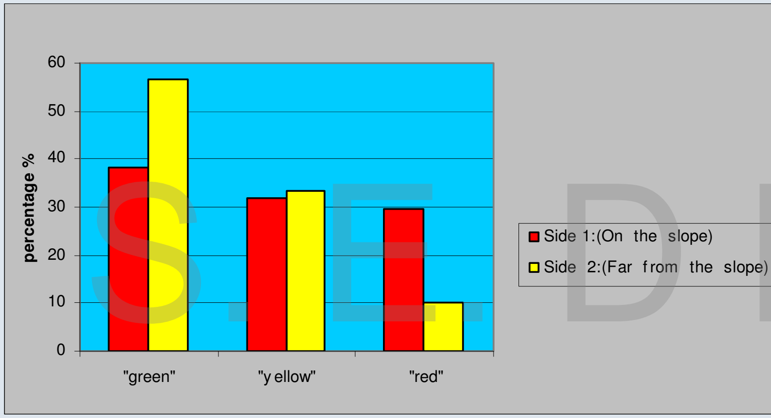


SLOPE INFLUENCE



Section Across Tatoiou Street

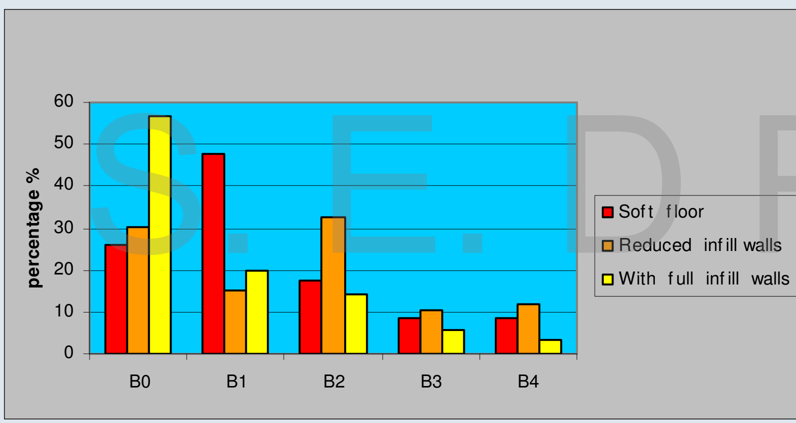
Usability characterization of buildings regarding the distance from the slope



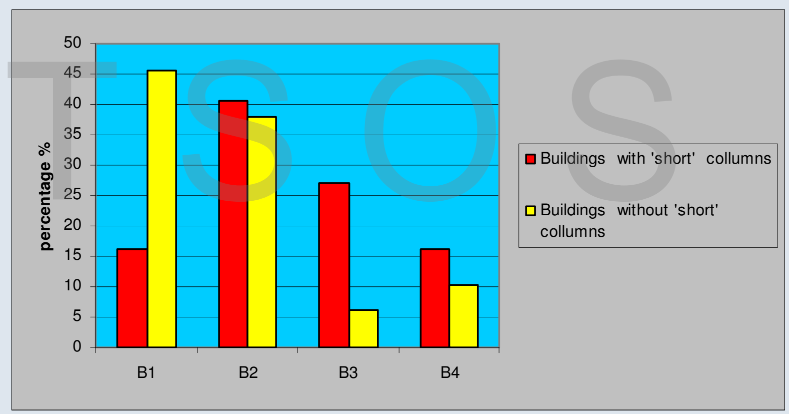
Definition of damage level

Bo	Level of damage in structures without damage
B1	Level of damage in structures with hairline cracks in structural elements
B2	Level of damage in structures with severe cracking in structural elements and/or slight buckling of reinforcement
B3	Level of damage in structures with buckling of reinforcement and permanent displacement of structural elements
B4	Level of damage in structures where structural elements have collapsed

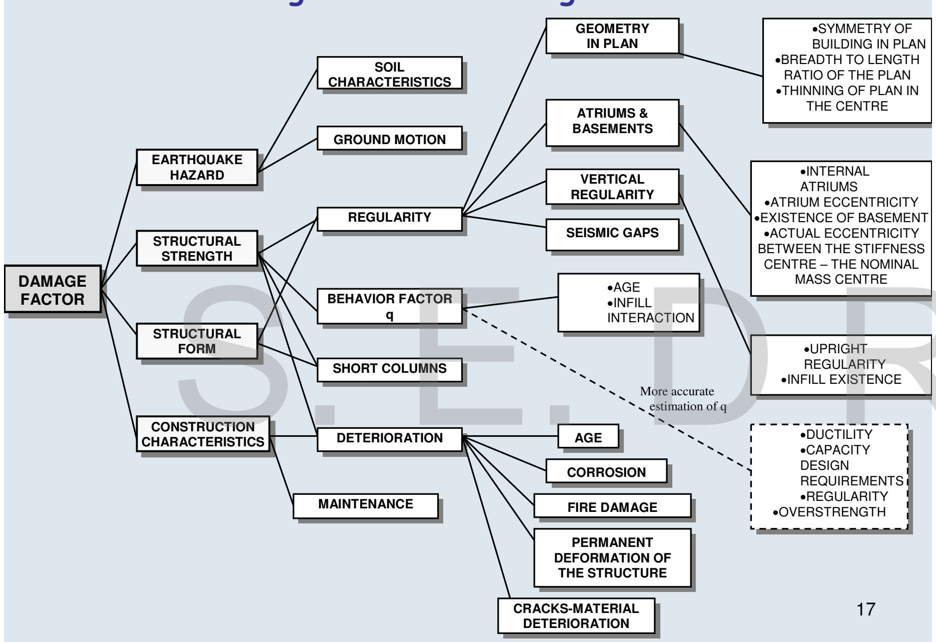
Distribution of damage related to the ground floor type

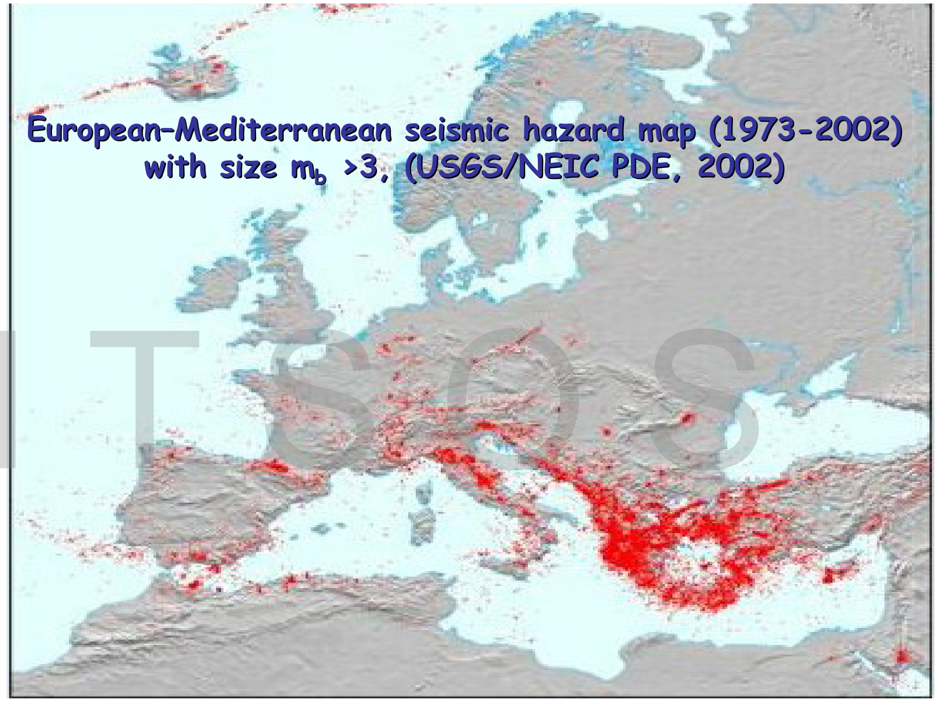


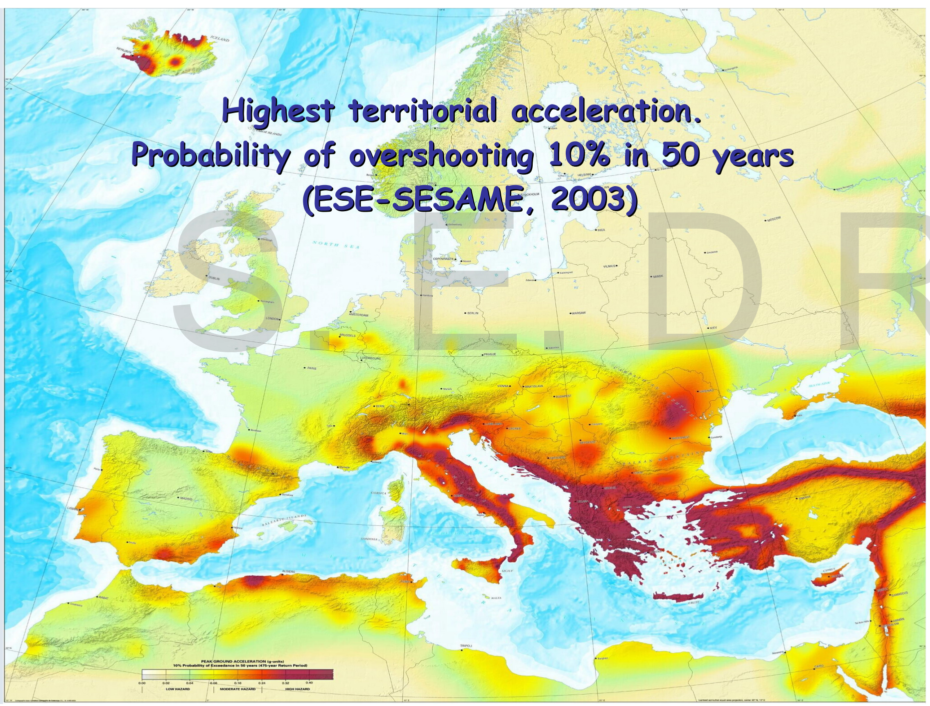
Influence of "short" columns on the level of damage



Damage assessment diagram







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