STRUCTURAL DESIGN

Earthquake Engineering

PART C: Assessment and Retrofitting of Existing Structures

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Pisa, March 2015
CONTENT

• Introduction
• Performance Levels or Damage Levels
• Elements‘ Behaviour
• Documentation
• Methods of Analysis
• Seismic Strengthening Strategies - Methods of Strengthening the Whole Structure
• 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
<table>
<thead>
<tr>
<th>Code</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>EN1998-1</td>
<td>General rules, seismic actions and rules for buildings</td>
</tr>
<tr>
<td>EN1998-2</td>
<td>Bridges</td>
</tr>
<tr>
<td>EN1998-3</td>
<td>Assessment and retrofitting of buildings</td>
</tr>
<tr>
<td>EN1998-4</td>
<td>Silos, tanks and pipelines</td>
</tr>
<tr>
<td>EN1998-5</td>
<td>Foundations, retaining structures and geotechnical aspects</td>
</tr>
<tr>
<td>EN1998-6</td>
<td>Towers, masts and chimneys</td>
</tr>
</tbody>
</table>
CODE ENVIRONMENT

EUROPE

1983

- CEB Bul. No. 162, “Assessment of Concrete Structures and Design Procedures for Upgrading (Redesign)”.

1995


1996

1999

2000

- ATC 40, “Seismic Evaluation and Retrofit of Concrete Buildings”.

- FEMA 356, “Prestandard and Commentary for the Seismic Rehabilitation of Buildings”.

2003

- fib Bul.No24, “Seismic Assessment and Retrofit of Reinforced Concrete Buildings”.

2005


2006

- GCSI, “Greek Code of Structural Interventions”.

2007

- ASCE/SEI 41, ASCE Standards Seismic Rehabilitation of Existing Buildings.

2008

- ASCE/SEI 41, Supplement 1, 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 $\sim 1/3$
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 is 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 \( M_{Rd} = 150 \text{ KNm} < M_{sd} = 200 \text{ KNm} \)

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

Points \((v_i, \delta_i)\)

Capacity curve

Performance Levels
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**

<table>
<thead>
<tr>
<th>Occurrence probability in 50 years</th>
<th>Collapse prevention (CP)</th>
<th>Life safety (LS)</th>
<th>Immediately occupancy (IO)</th>
</tr>
</thead>
<tbody>
<tr>
<td>2% Return period 2475 years</td>
<td>$CP_{2%}$</td>
<td>$LS_{2%}$</td>
<td>$DL_{2%}$</td>
</tr>
<tr>
<td>10% Return period 475 years</td>
<td>$CP_{10%}$</td>
<td>$LS_{10%}$</td>
<td>$DL_{10%}$</td>
</tr>
<tr>
<td>20% Return period 225 years</td>
<td>$CP_{20%}$</td>
<td>$LS_{20%}$</td>
<td>$DL_{20%}$</td>
</tr>
<tr>
<td>50% Return period 70 years</td>
<td>$CP_{50%}$</td>
<td>$LS_{50%}$</td>
<td>$DL_{50%}$</td>
</tr>
</tbody>
</table>

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

<table>
<thead>
<tr>
<th>Seismic activity probability of exceedance in the conventional design life of 50 years</th>
<th>Minimal Damage (Immediate Occupancy)</th>
<th>Severe Damage (Life Safety)</th>
<th>Collapse Prevention</th>
</tr>
</thead>
</table>
| **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

\[ S_d \leq R_d \]

decomposition demand deformation capacity

**Brittle**
Shear controlled

\[ S_d \leq R_d \]

strength demand strength capacity

**Seismically Primary**

“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

**Seismically Secondary**
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

Beams and columns

Walls of rectangular, T- or barbell section

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

The value of the plastic part of the chord rotation capacity of concrete elements under cyclic loading
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, \( \phi_{sd}, \phi_{Rd} \)

(G.S.I. Code)

A Level (IO)

\( \phi_{Rd} = \theta_y \)

B Level (LS)

\( \phi_{Rd} = \frac{1}{\gamma_{Rd}} \frac{\theta_y + \theta_u}{2} \)  \quad “primary” elements  \quad \gamma_{Rd} = 1.8

\( \phi_{Rd} = \frac{\theta_u}{\gamma_{Rd}} \)  \quad “secondary” elements  \quad \gamma_{Rd} = 1.8

C Level (NC)

\( \phi_{Rd} = \frac{\theta_u}{\gamma_{Rd}} \)  \quad \gamma_{Rd} = 1.8  \quad for “primary” elements

\( \gamma_{Rd} = 1.0 \)  \quad for “secondary” elements
ELEMENT’S SHEAR CAPACITY

Beams and Columns

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

\[ V_w = \rho_w b_w f_{yw} \]

rectangular web cross section

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

circular cross section

Shear Walls

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

Short Columns \((LV/h)\leq2\)

\[ V_{R,max} = \frac{4}{7} \left(1 - 0,02 \min(5; \mu_{p}^{pl})\right) \left(1 + 1,35 \frac{N}{A_c f_c}\right) \left(1 + 0,45(100\rho_{tot})\right) \sqrt{\min(40/f_c) b_w z \sin 2\delta} \]
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
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
- Normal Knowledge
- Limited Knowledge
- Inadequate: May allowed only for secondary elements
### Knowledge Levels and Confidence Factors

<table>
<thead>
<tr>
<th>Knowledge Level</th>
<th>Geometry</th>
<th>Details</th>
<th>Materials</th>
<th>Analysis</th>
<th>CF</th>
</tr>
</thead>
<tbody>
<tr>
<td>KL₁: Limited Knowledge</td>
<td>From original outline construction drawings with sample visual survey or from full survey</td>
<td>Simulated design in accordance with relevant practice and from limited in-situ inspection</td>
<td>Default values in accordance with standards of the time of construction and from limited in-situ testing</td>
<td>LF-MRS</td>
<td>CF&lt;sub&gt;KL₁&lt;/sub&gt; = 1.35</td>
</tr>
<tr>
<td>KL₂: Normal Knowledge</td>
<td>From original detailed construction drawings with limited in-situ inspection or from extended in-situ inspection</td>
<td>From incomplete original detailed construction drawings with limited in-situ inspection or from extended in-situ inspection</td>
<td>From original design specifications with limited in-situ testing or from extended in-situ testing</td>
<td>All</td>
<td>CF&lt;sub&gt;KL₂&lt;/sub&gt; = 1.20</td>
</tr>
<tr>
<td>KL₃: Full Knowledge</td>
<td>From original detailed construction drawings with limited in-situ inspection or from comprehensive in-situ inspection</td>
<td>From original test reports with limited in-situ testing or from comprehensive in-situ testing</td>
<td>From original test reports with limited in-situ testing or from comprehensive in-situ testing</td>
<td>All</td>
<td>CF&lt;sub&gt;KL₃&lt;/sub&gt; = 1.00</td>
</tr>
</tbody>
</table>
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 KL₂
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.)

<table>
<thead>
<tr>
<th>ORIGINAL DESIGN DRAWINGS</th>
<th>DATA ORIGIN</th>
<th>NOTES</th>
<th>DATA</th>
</tr>
</thead>
<tbody>
<tr>
<td>Exist Do not exist</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1</td>
<td>Data that is derived from a drawing of the original design that is proved to have been applied without modification</td>
<td>(1)</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>2</td>
<td>Data that is derived from a drawing of the original design that has been applied with few modifications</td>
<td>(2)</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>3</td>
<td>Data that is derived from a reference (e.g. legend in a drawing of the original design)</td>
<td>(3)</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>4</td>
<td>Data that has been determined and/or measured and/or surveyed reliably</td>
<td>(4)</td>
<td></td>
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<tr>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>5</td>
<td>Data that has been determined by an indirect but sufficiently reliable manner</td>
<td>(5)</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>6</td>
<td>Data that has been reasonably assumed using the Engineer’s judgment</td>
<td>(6)</td>
<td></td>
</tr>
</tbody>
</table>

- **TYPE AND GEOMETRY OF FOUNDATION OR SUPERSTRUCTURE**
  - KL1
  - KL2
  - KL3

- **THICKNESS, WEIGHT etc. OF INFILL WALLS, CLADDING, COVERING, etc.**
  - KL1
  - KL2
  - KL3

- **REINFORCEMENT LAYOUT AND DETAILING**
  - KL1
  - KL2
  - KL3
METHODS OF ANALYSIS
In Redesign other analysis methods are required

Elastic analysis methods currently in use (for new buildings) have a reliability under specific conditions to make sure new buildings to be met.

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

Points \((v_i, \delta_i)\)

(Base shear)

Capacity curve

Performance Levels

Light  Significant  Heavily  damage
\[ \Phi_\delta = \frac{T^2}{4\pi^2} \Phi_d g \]

acceptable demand curve

code elastic spectrum

\[ V = \alpha \Phi_d W \]

\[ \delta = \beta \Phi_\delta \]

<table>
<thead>
<tr>
<th>n</th>
<th>(\alpha)</th>
<th>(\beta)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>1</td>
<td>1</td>
</tr>
<tr>
<td>2</td>
<td>0.90</td>
<td>1.20</td>
</tr>
<tr>
<td>5</td>
<td>0.80</td>
<td>1.35</td>
</tr>
</tbody>
</table>

demand curves

elastic spectrum

inelastic spectrum
SAFETY VERIFICATION
Checking a Structure’s Capacity

- **Demand Curve** (Required Seismic Capacity)
  - Sufficient for Level A
  - Sufficient for Level B
  - Sufficient for Level C
  - Insufficient

- Safe Behaviour
- Unsafe behaviour
Seismic Strengthening Strategies
Methods of Strengthening the Whole Structure
SEISMIC STRENGTHENING STRATEGIES

- (a) Initial capacity
- (b1) Retrofitting local weakness and enhancement of ductility
- (c) Enhancing strength and ductility
- (d) Enhancing strength and stiffness
- (b2) As (b1) plus some strength increase
- (s) Required seismic capacity

Base Shear

Displacement

Safe Design

Unsafe Design
SEISMIC STRENGTHENING METHODS

Strength

Strength & Ductility

Ductility

Add New Walls
(a) Infill walls
(b) Externally attached to the structural system (specific design)

Steel or Concrete Bracing

Adding RC Wing Walls

Jackets
(a) of RC
(b) of steel elements
(c) of composite materials

Strength & Stiffness
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 $\rho_{\text{vertical}} = \rho_{\text{horizontal}} = 0.005$

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)
Addition of new external walls

Schematic arrangement of connections between existing building and new wall
Addition of a bracing system
Temporary support and stiffening of the damaged soft floor
COMPOSITE ELEMENTS
### 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 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

\[ S_d \leq R_d \]

\[ V_{\text{interface}}^{S_d} \leq V_{\text{interface}}^{R_d} \]

Interface Shear Force \( \leq \) Interface Shear Resistance
INTERFACE SHEAR FORCES:

\[ V_{interface} = F_{AB} - F_{CD} \]

(a) strengthening in the tensile zone  (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:

Mechanisms

- Friction and Adhesion
- Dowel Action
- Clamping Action
- Welded Connectors
UNREINFORCED INTERFACES

Concrete-to-concrete adhesion

Roughened interface concrete-to-concrete friction

(CEB Bul. No. 162, 1983)

GRECO, 2012
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

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 ($\sigma_c$) is Mobilized at the Interface
6. Frictional Resistance is Activated

(Tassios and Vintzeleou, 1987)
Frictional resistance

\[
\frac{s_f}{s_{fu}} \leq 0.5 \rightarrow \left( \frac{\tau_f}{\tau_{fud}} \right) = 1.14 \sqrt[3]{\frac{s_f}{s_{fu}}} \\
\frac{s_f}{s_{fu}} > 0.5 \rightarrow \left( \frac{\tau_f}{\tau_{fud}} \right) = 0.81 + 0.19 \frac{s_f}{s_{fu}}
\]

\[\tau_{fud} = 0.4 (f_{cd}^2 \ast (\sigma_{cd} + \rho_d f_{yd}))^{1/3}\]

Reinforced Interfaces

(GRECO, 2012)
Reinforced Interfaces

Dowel action
Shear Resistance
for Dowel Action as a function of the interface slip

\[ s_d = 0.1d_u + 1.80d_u \left( \frac{V_{sd}}{V_{ud}} \right)^4 - 0.5 \left( \frac{V_{sd}}{V_{ud}} \right)^3 \]

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

\[ V_{ud} = 1.3d_b^2 \sqrt{f_c f_y} \]
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:

\[
\varepsilon_{sb} = \frac{s/\sqrt{2}}{\sqrt{2}h_s} = \frac{s}{2h_s} \quad \text{and} \quad \sigma_{sb} = E_s \frac{s}{2h_s} \leq f_{yb}
\]

- Force is Transferred between Reinforcements:

\[
T_s = A_{sb} \times E_s \left(\frac{s}{\sqrt{2}h_s}\right) \leq T_{sy} = \sqrt{2}A_{sb} f_{yb}
\]
Mechanism is mobilized for very small Slippage

\[ T_{sy} = \sqrt{2} A_{sb} f_{yb} \]
Superposition of shear resistance mechanisms

\[ V_{tot} = \beta_D V_d + \beta_f V_f \]
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

Action effect vs. Deformation

\[ F_y,\mu \quad F_y,\varepsilon \]

\[ \delta_{y,\mu} \quad \delta_{y,\varepsilon} \]

\[ \delta_{u,\varepsilon} \quad \delta_{u,\mu} \]

\[ K_\mu \quad K_\varepsilon \]

\[ K_r = \frac{F_y,\varepsilon}{F_y,\mu} \]

\[ y_0 \quad y_{0,\mu} \quad y_{0,\varepsilon} \]

\[ \delta_{u,\varepsilon} \quad \delta_{u,\mu} \]
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:
  \[ k_{\delta_y} = \frac{\text{the displacement at yield of the strengthened element}}{\text{the displacement at yield of the monolithic element}} \]
  \[ k_{\delta_y} = \frac{\text{the ultimate displacement of the strengthened element}}{\text{the ultimate displacement of the monolithic element}} \]

\[(EI)_{\text{strengthened}} = k_k (EI)_M \]
\[ R_{\text{strengthened}} = k_r R_M \]
\[ \delta_{i,\text{strengthened}} = k_{\delta_i} \delta_{i,M} \]
Addition of a new concrete layer to the top of a cantilever slab
Monolithic Factors

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

**For slabs:**

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

**For concrete jackets:**

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

**For other elements:**

\[ k_k = 0,80 \quad k_r = 0,85 \quad k_{\theta y} = 1,25 \quad k_{\theta u} = 0,75 \]
Monolithic Factors

Influence of Interface Connecting Conditions in Case of Concrete Jackets

For $\mu = 1.4$

$k_k = 0.80$ and $k_r = 0.94$

$k_k = 0.70$ and $k_r = 0.80$

$k_k = 0.80$ and $k_r = 0.90$

(EC8, Part 1.4)

(G.C.S.I.)