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HABILITATION THESIS

Rehabilitation of Existing Concrete and/or Masonry Structures in Seismic Regions. A Permanent Challenge for Civil Engineers

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Contents

ABSTRACT

The assessment of the protection level of constructions generally and particularly of reinforced concrete and/or masonry structures has become a constant preoccupation of all the specialists involved in design, execution and monitoring of construction.

The habilitation thesis presents research and case studies connected to the structural rehabilitation aspects as follows:

- **Introduction** to: durability problems; behaviour at seismic actions; repair and strengthening of existing structures.
- **Rehabilitation of existing concrete structures:** experimental research; case studies.
- **Rehabilitation of existing masonry structures:** experimental research; case studies.

The vulnerability of existing structures under seismic motions may be due to structural system weaknesses and specific detailing. Structural weaknesses are characterised by various irregularities and discontinuities or by general structural vulnerabilities. Specific detailing of existing structures is function of building materials: reinforced concrete; steel; masonry; wood. Reinforced concrete (RC) structures are characterised by common non-ductile detailing:

Regarding the rehabilitation solutions, for vertical irregularities the main solutions consist of: strengthening of existing structural elements and / or the structural system by increasing the strength, stiffness and ductility of the weak structural elements; stalling additional structural members. In the case of horizontal structural irregularities, the aim of rehabilitation is to decrease torsion effects and displacements as well as an increase of the strength with the respect to lateral actions. For irregularities of the geometric plan, the rehabilitation solution consists of the use of new walls and / or seismic joints. The rehabilitation solutions adopted in the case of deterioration of building component parts depend on the structural material.

EXISTING REINFORCED CONCRETE STRUCTURES are to be repaired and/or strengthened in cases when the general damage is limited, and demolished when the structural safety is greatly affected and the rehabilitation cost is very high.

Repairs are used for surface deterioration, cracks, damage resulting from casting defects and reinforcement corrosion. The methods used for repairs are: jacketing of damaged surfaces; infilling of cracks with usual mortar, epoxy resin or other polymers; replacement or strengthening of damaged reinforcement.

Strengthening of reinforced concrete structures takes into account the increase of strength, stiffness and ductility. In case of reinforced concrete framed structures, the increase in stiffness and ductility is to be achieved by jacketing of beams, columns and joints. The jacketing is performed by reinforced concrete, steel profiles, carbon fibres CFRP, etc. CFRP may be used for increasing ductility and slightly increased stiffness.

Sometimes it is necessary to transform the existing structure completely, especially for framed structures. In this case, special techniques are to be used: steel bracing of reinforced concrete structures; infilling of frame openings with reinforced masonry or reinforced concrete.

Experimental studies were performed on the RC jacketing strengthening method characterized by some important advantages: leads to a uniformly distributed increase in strength and stiffness of element (column); the durability of the original structural member is also improved; this strengthening procedure does not require specialized workers. . Different techniques for increasing the bond between the old (existing) and new (jacketing) concrete layers were studied and presented in the thesis.

Experimental studies were also performed for strengthening of reinforced concrete framed structures in seismic zones by using Carbon Fiber Reinforced Polymers (CFRP). The system's advantages as rehabilitation application at seismic resistant structures are: increase of load-carrying capacity; structural elements designed only for gravity loads will be able to withstand seismic loads; elements' mass remains, practically, the same; the technology is simple and rapid.

The reinforced concrete structures' rehabilitation case studies presented are: the Western University of Timisoara; tanks supporting structure; office building; the Palace Building; apartment house affected by a gas explosion; reinforced concrete silos; strengthening of an industrial building; strengthening of frame structure at the Timisoreana Brewery; strengthening of a block of flats.

EXISTING MASONRY STRUCTURES present some important vulnerability in seismic zones: the overall lateral stiffness values along the two main axes are different; lack of seismic joints to divide building parts having different dynamic characteristics; lack of reinforced concrete straps at each level; defects of wall connections at corners, crossings and ramifications as well as the presence of cracks; inadequate bearing capacity at normal forces on the walls. On the other hand, structural weakness is characterised by various irregularities and discontinuities or by general structural vulnerabilities: irregular distribution of stiffness at lateral displacements; strength discontinuities; mass irregularities; vertical load discontinuities.

Experimental research was performed in order to develop new solutions for rehabilitation of old masonry buildings located in seismic zones.

The methods of strengthening existing masonry structures with the use of traditional technology are various: erection of RC cores appropriate distance combined with straps at each level, masonry lining with reinforced concrete, masonry confinement with steel profiles, interlocking of masonry walls at corners, crossing and ramifications with RC elements and/or some steel profiles, adding new inner walls and/or some outside abutments.

The modern rehabilitation solution Near-Surface-Mounted Reinforcement (NSMR) implies that steel bars/rods mainly of CFRP are bonded in sawn grooves in the masonry or concrete cover. The use of this technology has a lot of advantages: no requirement for surface preparation work, installation time is minimal, no change of the existing structure dimensions, the cost compared with traditional methods is lower even thaw the material costs are higher.

The masonry structures' rehabilitation case studies presented are: rehabilitation of the Banatul Museum, Timisoara – classic solution; retrofitting of historic masonry structures – modern solution; structural rehabilitation of historical masonry buildings: rehabilitation of a tower structure by modern solutions.

SCIENTIFIC, PROFESSIONAL AND ACADEMIC ACHIEVEMENTS

The theoretical and experimental research activity was developed in different fields: reinforced concrete structures durability; design and redesign of reinforced concrete structures in seismic regions; rehabilitation of existing structures by classic and modern techniques by: carbon-fibre-reinforced-polymers CFRP, near-surface-mounted-reinforcement NMSR; structural design by testing; behaviour of structures subjected to seismic impact; high performance construction materials.

The research activity was performed within many projects as follows:

Grants / Projects

Director of Romanian National Research Grants:

- Redesign of reinforced concrete structures for rehabilitation, 2000-2001, Grant ANSTI;
- Rehabilitation of reinforced concrete structures by using composite fibres polymers, 2002-2004, Grant CNCSIS MEC;
- Modern solutions for strengthening of concrete and masonry structures, 2005-2007, Grant CNCSIS MEC.

Director of Romanian team of international research grants:

- Valorisation des additions minérales pour la production de bétons écologiques et durables, 2012-2014, Grant WBI – FRS-FNRS Belgium, Partners: University of Liege, Belgium and University Politehnica Timisoara, Romania

European Research Programmes Involved in:

- COPERNICUS project “Recycling of Fly Ash for Producing Building and Construction Materials Base on a New Mineral Binder System”, 1995 – 1997;
- COPERNICUS project “High Performance Materials Derived from Industrial Waste Gypsum”, 1997 – 1999.

Romanian National Research Grants Involved in:

- Analysis of platforms for industrial chimneys oh 350 m height, 1989;
- Disperse reinforced concrete with glass fibre, 1990 – 1992;
- Optimizing the detailing and reinforcing in the discontinuity regions of reinforced concrete elements by using the strut-and-tie models, 1992 – 1994;
- Behaviour of structural elements at fire action, 1995 – 1997;
- Optimization of design and detailing for reinforced concrete and composite steel-concrete structures, 1998 – 1999;
- Monitoring, assessment and redesign of existing reinforced concrete structures, 1999 – 2000.

Research and/or Design Projects for Industry:

- Design of reinforced concrete structures, monolithic and/or precast structures: hotel, hospital, industrial, office, shopping and/or apartment buildings – 10 projects;
- Design of composite steel – concrete structures: office building – 1 project;
- Design of steel structures: industrial and/or office buildings – 4 projects;
- Design of masonry structures: apartment buildings; houses – 15 projects.
- Rehabilitation by using carbon-fibre-reinforced-polymers CFRP solution of different reinforced concrete structures: industrial buildings, silos, apartment buildings, hotels, etc. – 10 projects;
- Rehabilitation by using reinforced concrete and/or steel profiles jacketing of different structures: concrete structures, masonry structures; industrial, office and/or apartment buildings, silos, etc. – 4 projects.

Publications (see Publication List):

The results of the research activity were published in different books and papers which could be summarized as follows:

- 5 international books;
- 2 national books;
- 3 manuals for students' lectures;
- 3 books for students' application projects;
- 20 papers published in ISI journals and proceedings;
- 28 papers published in different journals and proceedings – international databases.

Professional Activities

Organization of Conferences:

- WSEAS "World Scientific and Engineering Academy and Society" 11th International Conference "Sustainability in Science Engineering (SSE '09)", Timisoara, Romania, 2009.

Reviewing Activities:

- Reviewer, SEI Structural Engineering International, "Operations, Maintenance and Repair of Structures", Vol. 17, No. 4/2007, ISSN 1016-8664, Journal of IABSE "International Association for Bridge and Structural Engineering", Zurich, 2007;
- Reviewer, IABSE - International Association for Bridge and Structural Engineering - Symposium "Sustainable Infrastructure - Environment Friendly, Safe and Resource Efficient", ISBN 978-385748-121-5, Bangkok, 2009;
- Reviewer, SEI Structural Engineering International, "Fibre Reinforced Polymer Composites", Vol. 20, No. 4/2010, ISSN 1016-8664, Journal of IABSE "International Association for Bridge and Structural Engineering", Zurich, 2010.

Committees Member:

- IABSE "International Association for Bridge and Structural Engineering" – member of Working
  Commission 4 “Operation, Maintenance and Repair of Structures”;
- Scientific Committee member IABSE - International Association for Bridge and Structural Engineering - Symposium “Sustainable Infrastructure - Environment Friendly, Safe and Resource Efficient”, Bangkok, 2009.

Teaching Activities

- Politehnica University Timisoara, Civil Engineering Department, 1990 – present;
  1990 – 2001, Assistant Professor at:
    - “Reinforced and Prestressed Concrete Structures” laboratory, seminar and project works– Bachelor students;
    - “Computer Aided Design” seminar works – Bachelor students;
    - “Redesign of Existing Structures” project works – Master students;

  2001 – 2015, Lecturer at:
    - “Reinforced and Prestressed Concrete Structures” – Bachelor students;
    - “Redesign of Existing Structures” – Master students;
    - Diploma supervision – Bachelor and Master studies.

  2015 – present, Associate Professor at:
    - “Structural Concrete” – Bachelor students;
    - “Reinforced and Prestressed Concrete Structures” – Bachelor students;
    - “Redesign of Existing Structures” – Master students;
    - Diploma supervision – Bachelor and Master studies.
- University of Liege, Department ArGEnCo, 2010-2012. Lecturer at:
    - GCIV0097-1 “Steel and Concrete Structures I – Constructions métalliques et en béton I”
    - GCIV0099-1 “Steel and Concrete Structures II – Constructions métalliques et en béton II”

University Activities

- ERASMUS exchange agreement, 2006 – present: responsible for the program between Politehnica University of Timisoara, Civil Engineering Faculty – The New University of Lisbon, Applied Science Faculty;
- Politehnica University Timisoara, Civil Engineering Faculty, 2009 – 2011: Responsible for the Admission exam commission;
- Politehnica University Timisoara, Civil Engineering Department, 2004 – 2008: Responsible for the Research activities.
- Politehnica University Timisoara, Civil Engineering Department, 2015 – present: Head of Department.

SCIENTIFIC PAPERS AS BASIS FOR THE HABILITATION THESIS

The present thesis is based on the following scientific papers:

1. C. Bob, **DAN Sorin**, C. Badea, L. Iures, “Classic and Modern Solutions for Rehabilitation of Reinforced Concrete Structures”, Structures and Extreme Events, IABSE Symposium, Lisbon, Portugal, 2005, ISBN: 3-85748-112-9 **(INGENTA Proceedings)**;;
2. **DAN Sorin**, C. Bob, A. Gruin, “Analysis of Reinforced Concrete Existing Structures in Seismic Regions” – WSEAS International Conference “Engineering Mechanics, Structures, Engineering Geology (EMESEG ’08)”, Heraklion, Greece, 2008, pg. 96-103, ISBN 978-960-6766-88-6 **(ISI Proceedings)**;
3. **DAN Sorin**, C. Bob, A. Gruin, C. Badea, L. Iures, “Strengthening of Reinforced Concrete Framed Structures in Seismic Zones by Using CFRP” – WSEAS International Conference “Engineering Mechanics, Structures, Engineering Geology (EMESEG ’08)”, Heraklion, Greece, 2008, pg. 67-72, ISBN 978-960-6766-88-6 **(ISI Proceedings)**;
4. C. Enuica, C. Bob, **DAN Sorin**, C. Badea, A. Gruin, “Solutions for Bond Improving of reinforced Columns Jacketing” – 11th WSEAS International Conference “Sustainability in Science Engineering (SSE ’09)”, Timisoara, Romania, 2009, pg. 58-63, ISBN 978-960-474-080-2 **(ISI Proceedings)**;
5. **Dan Sorin**, C. Bob, L. Bob, A. Gruin, C. Badea, “Modern Solutions for Strengthening of Masonry Structures” – 11th WSEAS International Conference “Sustainability in Science Engineering (SSE ’09)”, Timisoara, Romania, 2009, pg. 64-69, ISBN 978-960-474-080-2 **(ISI Proceedings)**;
6. **DAN Sorin**, C. Bob, C. Badea, ”Impact of New Design Codes on Assessment and Redesign of Reinforced Concrete Existing Structures in Seismic Regions”, Large Structures and Infrastructures for Environmentally Constrained and Urbanised Areas, IABSE Symposium, Venice, Italy, 2010, ISBN: 978-385748-121-5 **(INGENTA Proceedings)**;
7. **DAN Sorin**, ”Energy Saving with Rehabilitation Solutions for Existing Structures”, 4th WSEAS International Conference on Energy Planning, Energy Saving, Environmental Education, EPESE'10, 2010, ISBN: 978-960-474-187-8 **(SCOPUS Proceedings)**;
8. **DAN Sorin**, L. Iures, C. Badea, “Impact of Structural Rehabilitation of Existing Buildings on Energy Saving in Constructions”, 13th SGEM -Multidisciplinary conference, Albena, Bulgaria, 2013, ISSN: 1314-2704 **(ISI Proceedings)**;
9. **DAN Sorin**, C. Bob, C. Badea, L. Iures, “Risk Assessment and Rehabilitation of Historical Masonry Buildings”, Proceedings of the 12th International Conference on Environment, Ecosystems and Development (EED 14), Brasov, Romania, 2014, ISBN: 978-960-474-385-8 **(SCOPUS Proceedings)**;
10. R. Chendes, C. Bob, C. Badea, C.E. Podoleanu, **DAN Sorin**, L. Iures, “Experimental Research on Recycled Concrete Fines”, Revista de Chimie, Bucuresti, 2016, 67/No. 9 **(ISI Journal)**;

ACHIEVEMENTS AND GOALS

The scientific research activity may be summarized as follows:

Main PhD activity

The PhD thesis **"Aspects Regarding Resistance Capacity of Existing Reinforced Concrete Structures at Different Service Life Duration"** original contributions of the author are:

- Presentation, by using multiple references, of the new and important aspects regarding the assessment and rehabilitation of reinforced concrete structures.
- Definition, by a personal concept, of the seismic earthquake ratio.
- Using at the analysis of the existing reinforced concrete structures of the real mechanical characteristics of component elements and their influence on the general state of efforts. Two procedures are proposed: the elasticity modulus modification procedure at different service life stages; the supplemental plastic hinges procedure placed in the damaged zones of the structure.
- Original approach of the issues regarding the time behaviour of reinforced concrete structures: Alkali-Aggregate Reaction into concrete; behaviour of existing structures to seismic impact.
- Structural assessment and strengthening of the Banatul Museum, Timisoara. An original solution was proposed: to ensure the structural safety by increasing the mechanical characteristics of interior columns.
- Structural assessment and strengthening of the “Timisoreana” Brewery. By application of the theoretical studies performed in the thesis, an economical strengthening solution was adopted to ensure the structural safety.

Post-Doctoral Activity

The research activity was developed in different fields as follows:

A. Assessment, redesign and rehabilitation of existing concrete and masonry structures aspects:

- theoretical and experimental assessment of reinforced concrete structures’ durability;
- monitoring, assessment and redesign of existing reinforced concrete structures;
- redesign of reinforced concrete structures for rehabilitation;
- rehabilitation of reinforced concrete structures by using composite fibres polymers;
- modern and efficient solutions for strengthening of concrete and masonry structures.

B. High performance construction materials:
- high performance construction materials derived from recycled materials;
- concrete with super-plasticizing additives;
- high performance concrete;
- innovative solution for optimizing the self-compacting concrete microstructure used in prefabricated elements;
- development of durable and ecological concrete by using minerals additions.

Future Post Habilitation Activities

In the field of rehabilitation of existing structures, the proposed future activities will be:

- new materials and technologies used for rehabilitation of concrete and/or masonry structures;
- complex analysis of old masonry structures;
- technical, economical and sustainable comparison between different classical and modern rehabilitation solutions;
- experimental research on possibilities of using carbon fibre reinforced polymers CFRP and other modern composites as reinforcement in structural concrete;
- using of modern strengthening solutions for reinforced concrete and/or masonry structures in industry projects and applications.
- life service assessment of existing infrastructure (bridges) exposed to different environmental conditions.

Other research fields to be involved in could be:

- new materials and technologies for structural concrete: i.e. ”textile carbon reinforced concrete”;
- sustainability of different rehabilitation solutions for reinforced concrete and/or masonry structures;
- robustness of reinforced concrete and/or masonry structures subjected to special or accidental actions;
- new sustainable materials obtained by cement replacing, in concrete and mortar, with different recycled materials from various sources.
- sustainable and environmental friendly solution for houses construction industry: i.e. earth made houses;
- contribution to structural concrete design codes revisions by theoretical and experimental research (i.e. design at shear forces).

The research achievements will be accomplished by co-operation with colleagues from own department, faculty and university, and different universities, research institutes and industry partners both from Romania and abroad.

The results of these researches should be disseminated at scientific conferences and journals both from Romania and abroad.

1. INTRODUCTION

The assessment of the protection level of constructions generally and particularly of reinforced concrete structures has become a constant preoccupation of all the specialists involved in design, execution and monitoring of construction. For achieving this goal it is necessary to estimate quantitatively two parameters: durability and safety, which are the principal components of construction quality [1].

The structure durability may be defined as the time period during which the construction preserves its own normal characteristics of function [2]. For pointing out the effect of all possible actions - ordinary loads and extreme loads - on the construction safety, the Figure 1.1 shows the hypothetical variation in time of bearing capacity of the structure.

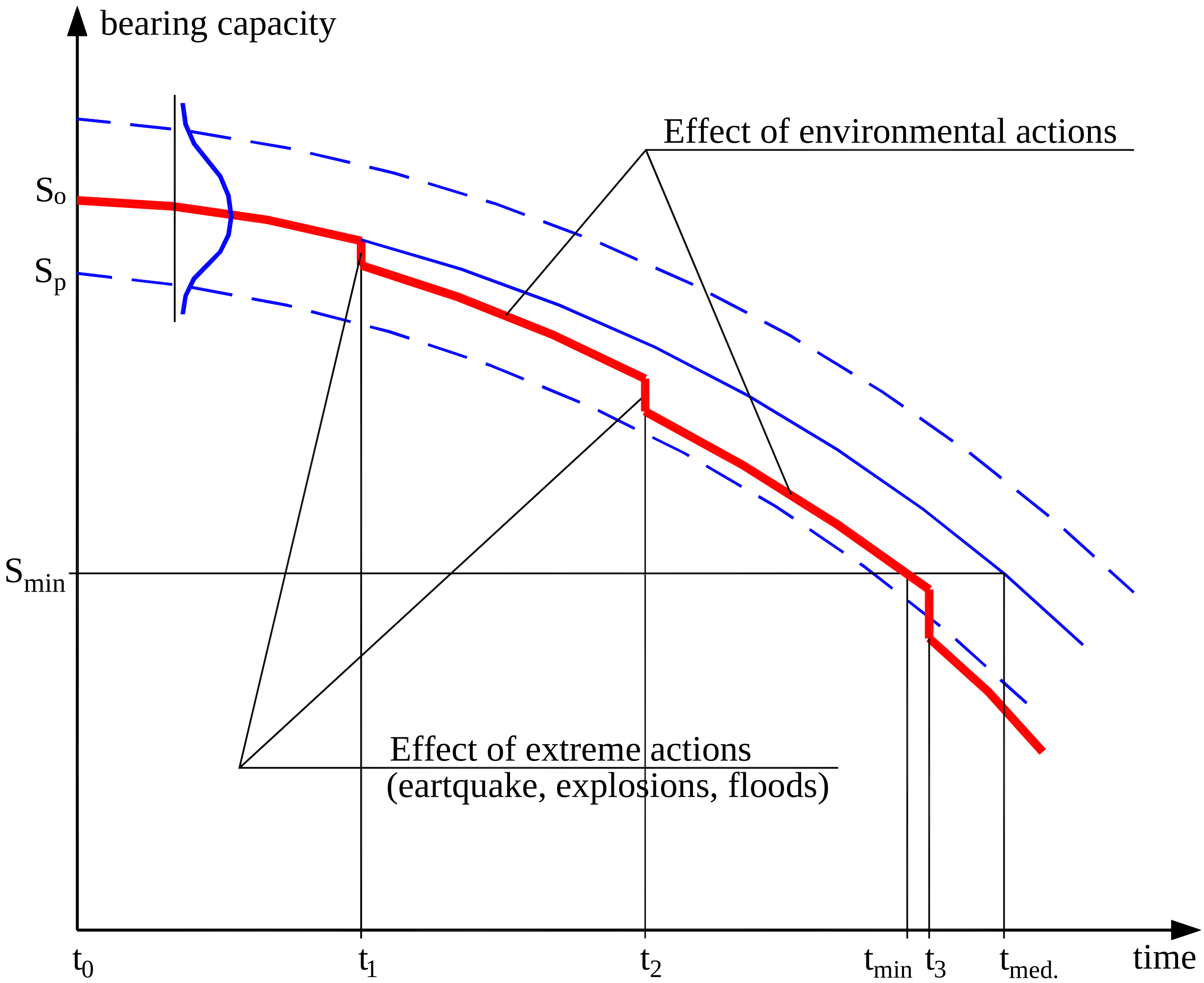


Figure 1.1. The effect of ordinary and extreme actions on the construction safety

This approach takes into account the permanent, variable and special actions and the environmental factors which covers more completely the effect of time:

Actions
- a) Ordinary actions
  - Dead loads
  - Variable loads
  - Actions from environmental conditions which produce:
    - Reinforcement corrosion
    - Fatigue of structural elements
    - Erosion
    - Specific factors: freeze-thaw cycles, alkali-aggregate reaction, etc.
- b) Special actions
  - Earthquakes
  - Explosions
  - Floods
  - Others

1.1. DURABILITY PROBLEMS

The structure's durability may be defined as the time period during which the construction preserves its own normal characteristics of function.

The service life is the time period during which a structure according to the design should meet some functional requirements (related to the strength and service) without unexpected costs for maintenance and repair.

The service life is largely depending on durability.

Nowadays the usual design service life for buildings of normal importance is 100 years, different from previous of 50 years.

Measures for preserving service life are function of environmental conditions, classified into **EXPOSURE CLASSES (X) OF CONCRETE TO ENVIRONMENTAL CONDITIONS** as follows:

0 – **Zero** risk
C – **C**arbonation
D – **D**eceiving salt
S – **S**eawater
F – **F**rost
A – **A**ggressive environment (chemical)
M – **M**echanical abrasion

Combinations of exposure classes could be (see Figure 1.2):

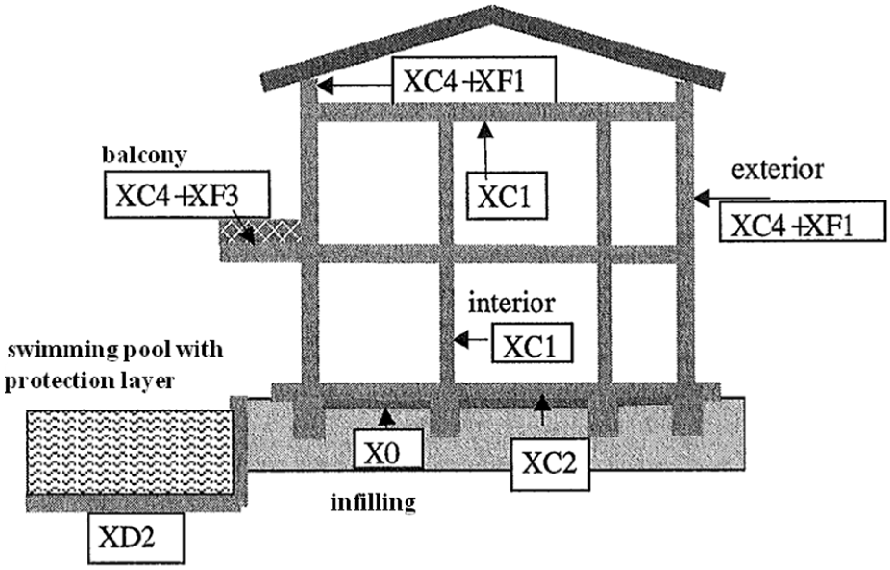


Figure 1.2. Concrete exposure classes [3]

Damages of concrete structures are influenced by the cracking of concrete. Types of cracks are:

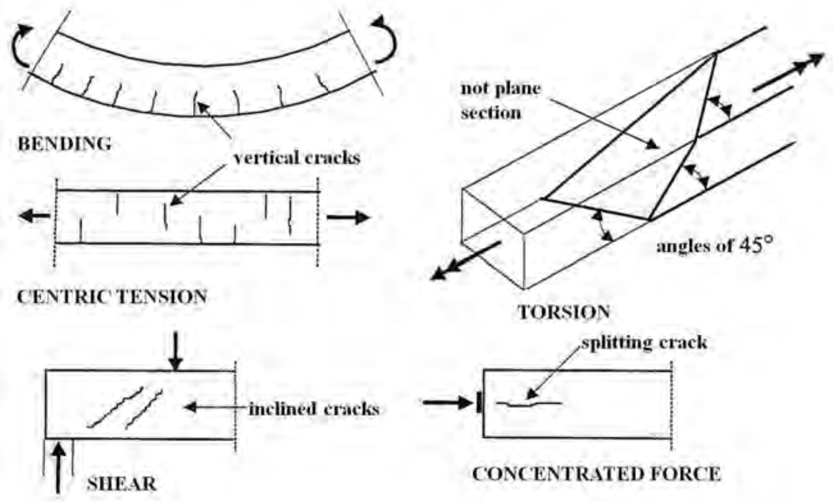


Figure 1.3. Cracks due to efforts from loads

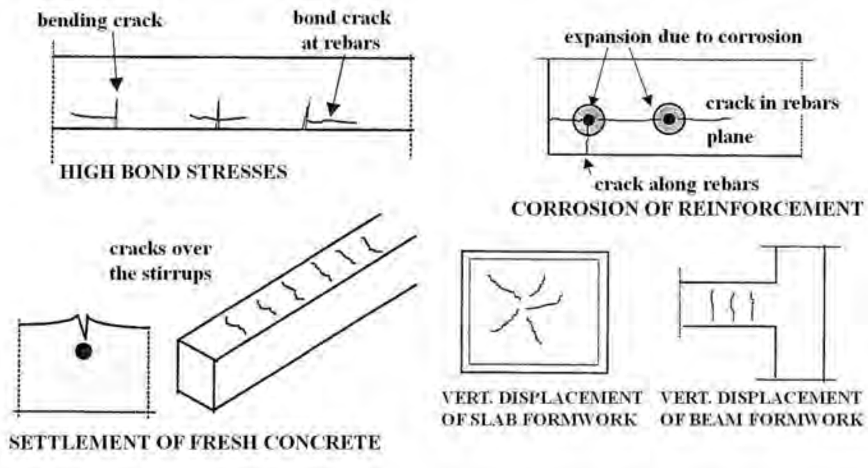


Figure 1.4. Cracks due to other factors

Damage of concrete structures could be due to:

- corrosion of reinforcement steel due to:
  - concrete carbonation
  - chloride ions (salts) penetration

Figure 1.5. Corrosion of columns' reinforcement

Figure 1.6. Corrosion of columns' reinforcement

Figure 1.7. Corrosion of slab reinforcement

Damage of concrete structures could be due to (continued):

- drying shrinkage
- plastic shrinkage
- plastic settlement and bleeding
- thermal cracking
- frost and frost scaling
- Alkali Silica Reaction (ASR) or Alkali Aggregate Reaction (AAR): alkali from cement react with some inappropriate aggregates

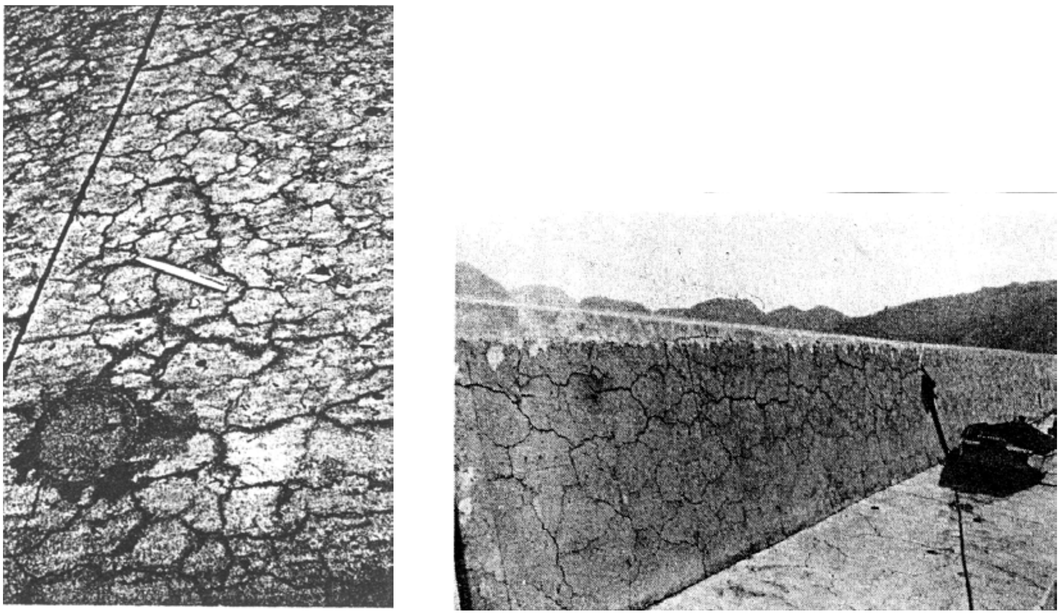


Figure 1.8. Damage due to Alkali Aggregate Reaction (AAR) [4]

Damage of concrete structures could be due to (continued):

- erosion of concrete
- acid attack
- fire
- discolouring and precipitation

CORROSION OF REINFORCEMENT – MECHANISM

Reinforced concrete structures that are subjected to environmental conditions are likely, after a certain period of exposure, to exhibit signs of distress as a result of initiation of reinforcement corrosion process.

The initial corrosion occurs mainly in two different ways: carbonation of the concrete surrounding the reinforcement and presence of chloride.

The principal correlation, which characterizes the reinforcement corrosion – an important part of concrete durability – is the depth of carbonation or chloride penetration and the time of CO_2 or/and Cl^- action.

Main factors influencing carbonation and chloride ingress are: carbonation dioxide and chloride concentration, environmental conditions, permeability properties and chemical reaction.

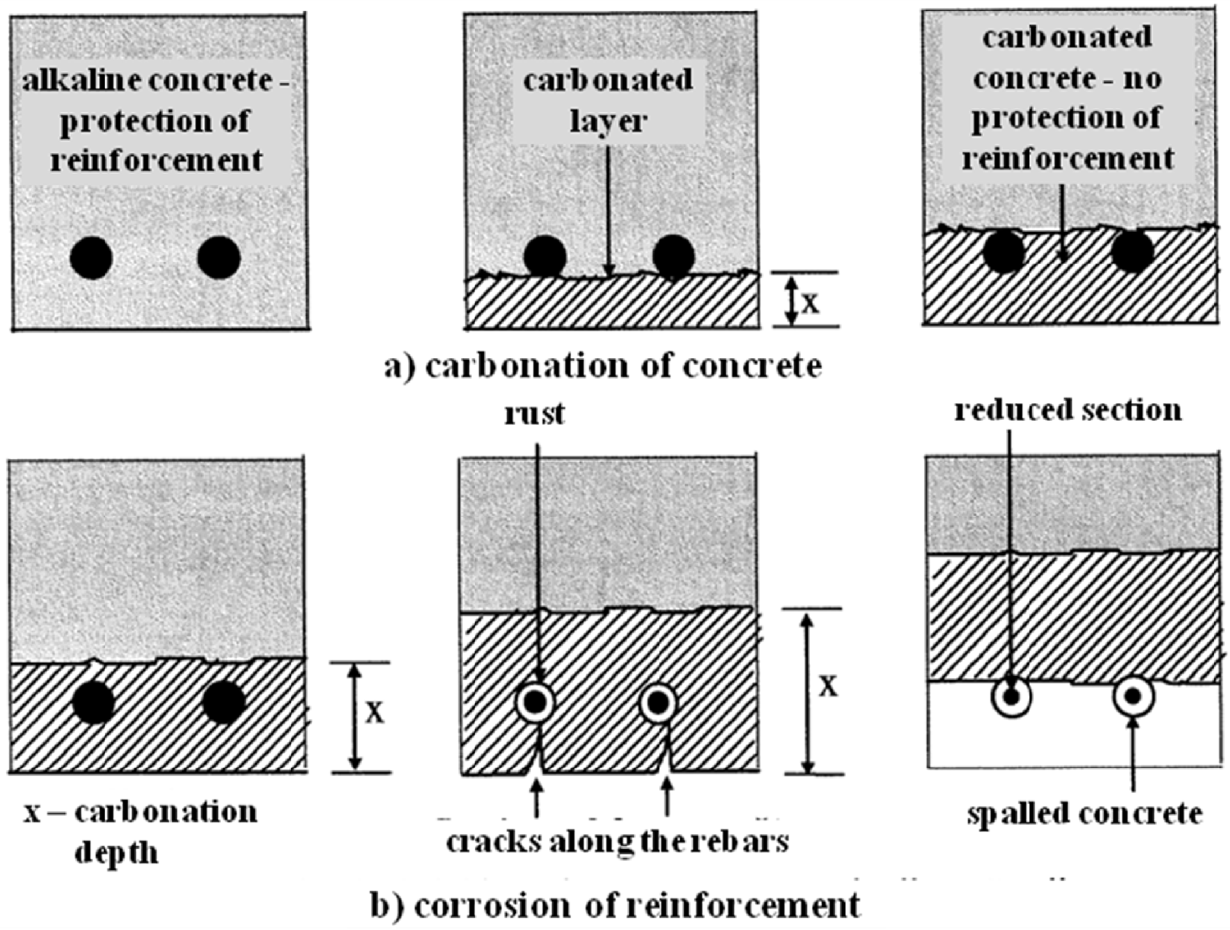


Figure 1.9. Corrosion of reinforcement – mechanism [5-7]

1.2. BEHAVIOUR AT SEISMIC ACTIONS

The vulnerability of existing structures under seismic motions may be due to structural system weaknesses and specific detailing [8-13]. Structural weaknesses are characterised by various irregularities and discontinuities or by general structural vulnerabilities:

1. Irregularities in the vertical direction of the buildings: irregular distributions of the stiffness at lateral displacement; strength discontinuities; mass irregularities; vertical load discontinuities.
2. Irregularities in the building layout: horizontal irregularities of masses, stiffness and strength, which all produce torsion effects; unfavourable plan layouts; slab discontinuities due to holes or weaknesses of the connections in some zones.
3. General structural vulnerabilities: the indirect transfer of strong forces by beam-on-beam supports or columns supported on beams; cantilever horizontal members with large spans and / or high loads; weak column / strong beam: eccentricities; finite service life due to deterioration of component parts of a building.

Specific detailing of existing structures is function of building materials: reinforced concrete; steel; masonry; wood.

RC structures are characterised by common non-ductile detailing [8-12]:

- inadequate column bending and shear capacity;
- inadequate beam shear resistance;
- inadequate joint shear resistance;
- inadequate quantities and anchorage of beam-positive reinforcement at the beam-column joint;
- inadequate confinement of the potentially plastic hinges of the columns and beams as well as of the boundary elements of RC frame-wall systems;
- inadequate reinforcement of the RC frame in the longitudinal direction of the building.

PRESENT-DAY SEISMIC DESIGN

The principles for seismic design are [14-18]:

Energy Balance Criterion

$E_S \leq E_R$

where: E_S - seismic induced energy
E_R - resistance energy without failure

Energy evaluation (Figure 1.10):

- elastic design at F_e (high) for structures without plastic deformation (reservoirs, water tanks, nuclear plants, etc.) ⇨ E_e
- plastic design at F_p (low) for most of the RC structures. Seismic energy is dissipated by plastic deformation ⇨ E_p

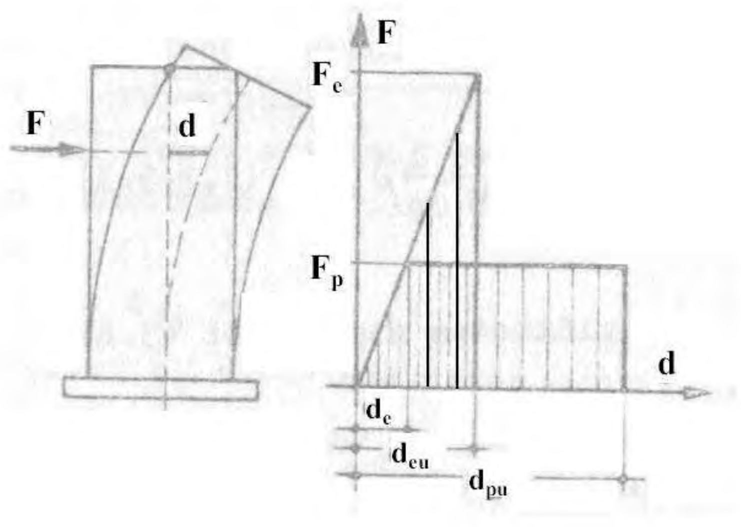


Figure 1.10

$E_e = E_p$

$$\frac{1}{2} \cdot F_e \cdot d_{eu} = \frac{1}{2} \cdot F_p \cdot d_e + F_p \cdot (d_{pu} - d_e)$$

$$\frac{1}{2} \cdot F_e \cdot d_{eu} = \frac{1}{2} \cdot F_p \cdot d_e \cdot \left[2 \cdot \frac{d_{pu}}{d_e} - 1\right]$$

with: - rigidity $K = \frac{F_p}{d_e} = \frac{F_e}{d_{eu}}$

- behaviour factor $q = \frac{F_e}{F_p} = \frac{d_{eu}}{d_e}$

- ductility factor $\Delta = \frac{d_{pu}}{d_e}$

⇨ $\boxed{q = \sqrt{2 \cdot \Delta - 1}}$

Finally:

D	6.6	8.5	13
q	3.5	4	5

Seismic Action (Eurocode 8)

Ex: $F_b = \gamma_1 \cdot a_g \cdot \frac{\beta(T_1)}{q} \cdot m \cdot \lambda$ - plastic design

where: F_b - base shear force; $S_d(T_1)$ - design spectrum;
g_1 - importance factor; m - total mass of the building;
λ - correction factor; a_g - design ground acceleration;
$\beta(T_1)$ - response spectrum;
***q* - <u>behaviour factor</u>**.

Deformation Criterion

$d_S \leq d_R$

where: d_S - seismic induced deformation;
d_R - deformation limit.

PROVISIONS FOR INCREASING STRUCTURAL DUCTILITY

Seismic design action *F(q)* takes into account the post-elastic deformation for seismic energy dissipation ⇨ structural ductility is necessary at bending with/without axial force (*M* + *N*), ensured by yielding of reinforcement in the possible plastic hinges.

The possibilities for increasing structural ductility are [19-21]:

Using Ductile Steel for Reinforcement

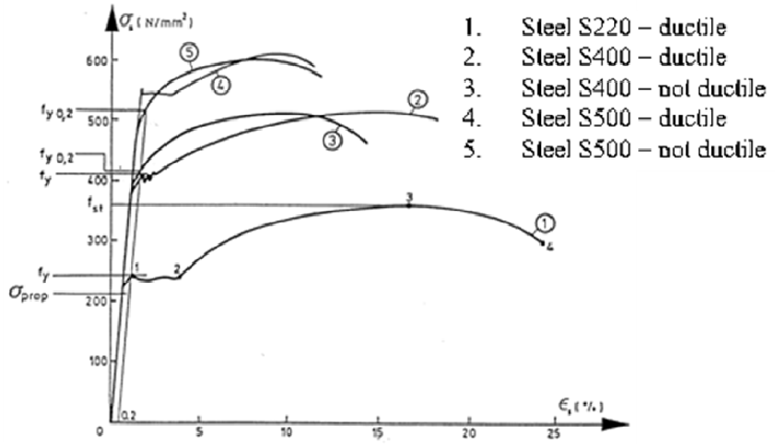


Figure 1.11.
Ductile vs. not ductile steel

Figure 1.12.
Behaviour and fracture of columns with ductile reinforcement

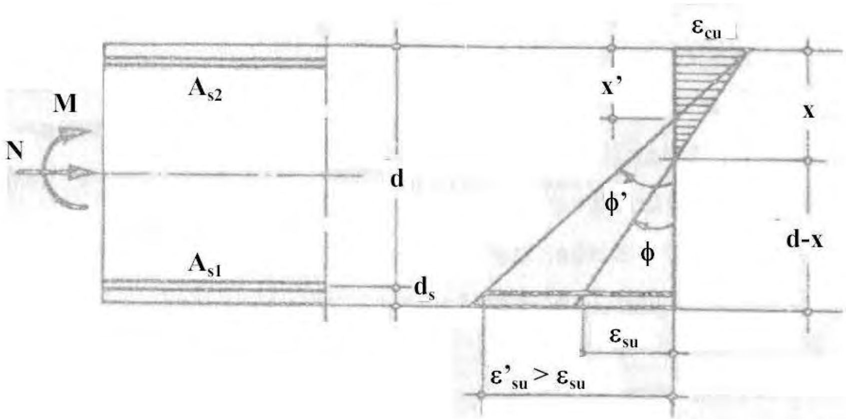


Figure 1.13. Strain diagram at bending with axial force (*M* + *N*)

For more ductile reinforcement:

$\varepsilon'_{su} > \varepsilon_{su}$ (strain) ⇨ x' < x (position of neutral axis)

⇨ $\phi' > \phi$ higher rotation capacity in the plastic hinge

⇔ higher ductility

Strengthening of Compressive Concrete

For avoiding failure of compressive concrete before yielding of tensile reinforcement:

- providing a minimum concrete class ≥ C16/20 (function of ductility class: high DCH or medium DCM)
- at beams → double reinforcement
- at columns → limits to axial force ⇨ failure at compression with <u>prevailing bending</u>
- confined concrete by - longitudinal reinforcement
  - transversal reinforcement (stirrups)

Strengthening of compressive concrete at beams by double reinforcement:

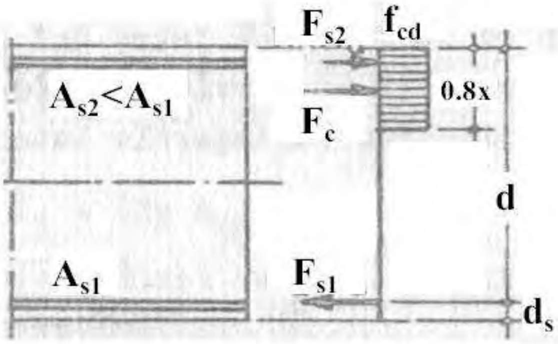


Figure 1.14. Stress diagram at bending for double reinforced beams

Equilibrium equation:
$\Sigma X = 0$
$F_c - (F_{s1} - F_{s2}) = 0$
$0.8 \cdot x \cdot b \cdot f_{cd} - (A_{s1} - A_{s2}) \cdot f_{yd} = 0$

$$x = \frac{(A_{s1} - A_{s2}) \cdot f_{yd}}{0.8 \cdot b \cdot f_{cd}}$$

- by using A_{s2} ⇨ $x \downarrow$ (decreased) ⇨ $\phi \uparrow$ (rotation increased)
⇔ higher ductility

At columns – limitation of axial force ⇨ failure at compression with prevailing bending:

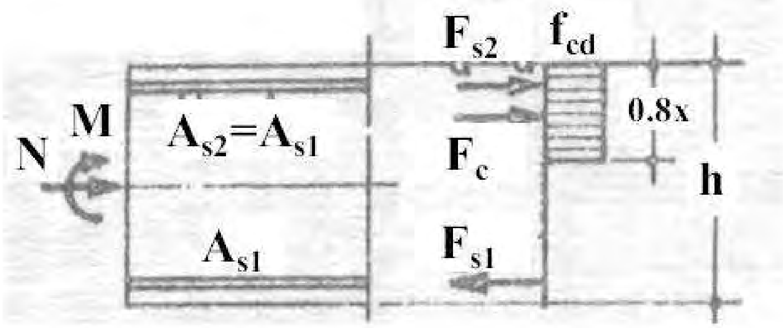


Figure 1.15. Stress diagram at bending with axial force ($M + N$) for columns

Equilibrium equation:
$\Sigma X = N$
$F_c - (F_{s1} - F_{s2}) = N$
$0.8 \cdot x \cdot b \cdot f_{cd} - (A_{s1} - A_{s2}) \cdot f_{yd} = N$

$$x = \frac{N}{0.8 \cdot b \cdot f_{cd}}$$ - for symmetric reinforcement $A_{s1} = A_{s2}$

- by design: $x \downarrow$ (decreased) ⇨ $\phi \uparrow$ (rotation increased)
⇔ higher ductility

Strengthening of compressive concrete by confinement:

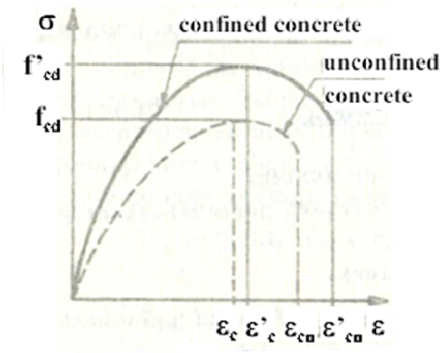


Figure 1.16.
Behaviour of confined concrete

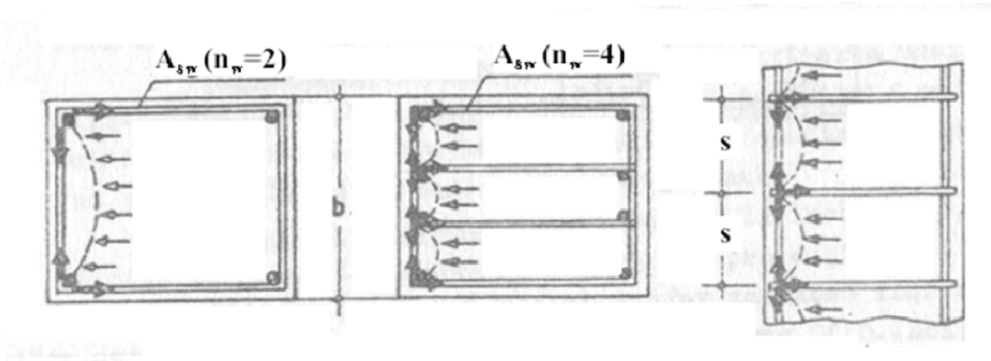


Figure 1.17. Confinement of concrete by reinforcement bars

Failure examples (from reports on earthquakes around the world [22-37]):

Figure 1.18. Lack of confinement at column head

Figure 1.19. Column failure in shear due to insufficient shear reinforcement and low quality of concrete

Figure 1.20. Excessive concrete cover, lowering design flexural capacity, and poor concrete quality

Figure 1.21. Insufficient shear reinforcement and low quality of concrete

Figure 1.22. Lack of transverse reinforcement in beam-column connection

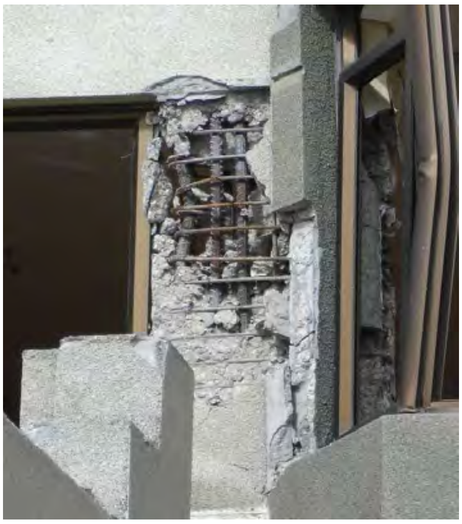


Figure 1.23. Buckling of main reinforcement at compression

Figure 1.24. Buckling of main reinforcement at compression

Figure 1.25. Slender columns, lack of bonding and absence of stirrups

Plastic Hinges in Beams Before in Columns:

- for local failure instead of general failure
- for lower overall horizontal deformation

The design bending moments for columns will be increased bending moments from static analysis:

$\Sigma M_{Rc} \geq \gamma_{Rd} \cdot \Sigma M_{Rb}$

where: $\gamma_{Rd} \geq 1{,}0$
b = beam
c = column
R = resistance

<u>Failure examples (from reports on earthquakes around the world [22-37])</u>:

Figure 1.26. Plastic hinges in columns

Figure 1.27. Plastic hinges in columns

Figure 1.28.
Column failure at top end: roof parapet activated a strong-beam-weak-column mechanism

Figure 1.29.
Hinges formed at bottom and top of columns

Figure 1.30.
Plastic hinges at column ends

Figure 1.31.
Roof columns failure

Figure 1.32.
Roof columns failure

Limitation of Shear Force

For avoiding <u>brittle</u> failure at shear before <u>ductile</u> failure at bending in plastic hinges:

- the design shear force will be the associated shear force to the bending moments from the plastic hinges for positive and negative directions of seismic loading;
- the plastic hinges should be taken to form at the ends of elements (beams or columns) corresponding to the maximum bending moments.

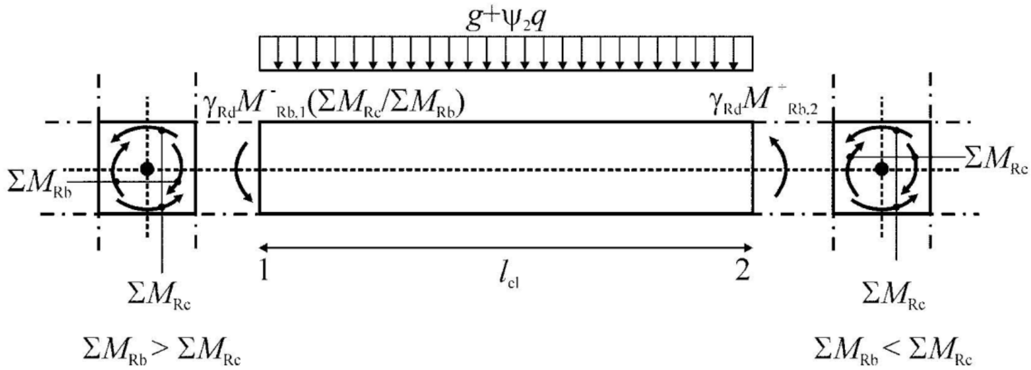


Figure 1.33. Design of beams [38]

Capacity design values of shear forces on beams:

$$V_{Ed} = \frac{(g + \psi_2 \cdot q) \cdot l_{el}}{2} + \frac{|M_{1d}| + |M_{2d}|}{l_{el}}$$

$$M_{i,d} = \gamma_{Rd} \cdot M_{Rb,i} \cdot \min\left(1, \frac{\Sigma M_{Rc}}{\Sigma M_{Rb}}\right)$$

Capacity design shear force on columns:

$$V_{Ed} = \frac{|M_{1d}| + |M_{2d}|}{l_{el}}$$

$$M_{i,d} = \gamma_{Rd} \cdot M_{Rc,i} \cdot \min\left(1, \frac{\Sigma M_{Rb}}{\Sigma M_{Rc}}\right)$$

where: $\gamma_{Rd} \geq 1{,}0$; *b* = beam;

c = column;

R = resistance

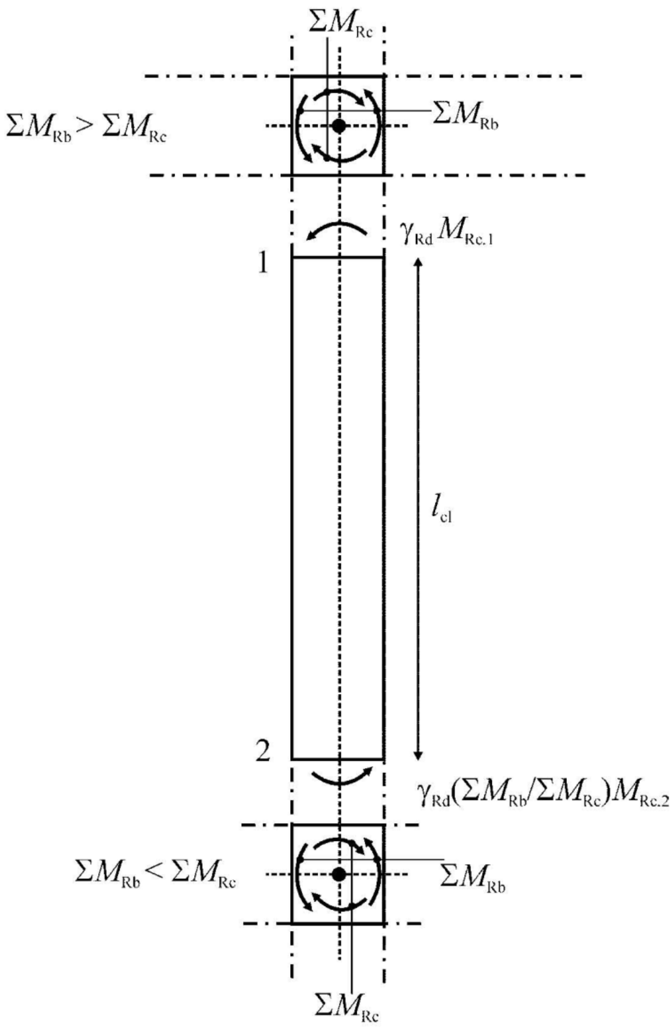


Figure 1.34. Design of columns [38]

Failure examples (from reports on earthquakes around the world [22-37]):

Figure 1.35. Detail of column failure in shear


Figure 1.36. Shear failure at ground floor column


Figure 1.37. Failure of column at top end at the second storey, due to insufficient shear reinforcement and activation of a strong-beam-weak-column mechanism

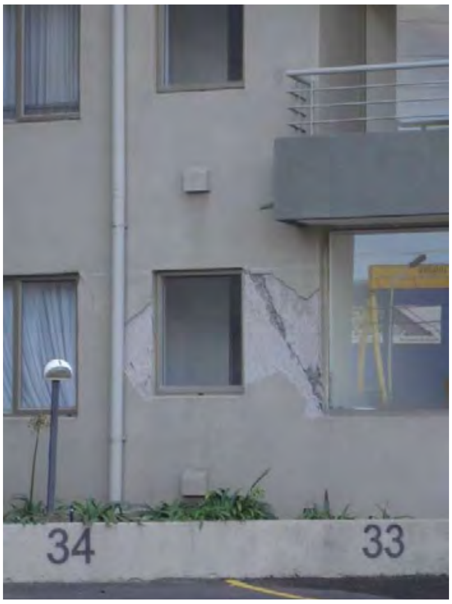


Figure 1.38. Cracking of shear walls

Figure 1.39. Detail of column failure in shear


Figure 1.40. Cracking of shear walls

Limitation of Interstorey Drift

<u>Displacement calculation</u>

$d_s = q \cdot d_e$

where: d_s - displacement induced by seismic action
q - behaviour factor
d_e - displacement determined by a linear analysis at seismic action

<u>Interstorey drift</u>

$d_r \leq 0.005 \cdot h$ - in case of brittle infilling walls
$d_r \leq 0.0075 \cdot h$ - in case of ductile infilling walls
$d_r \leq 0.010 \cdot h$ - in case of no infilling walls

where: $d_r = d_{s,i} - d_{s,i-1}$ - interstorey drift at storey i
h – storey height

Figure 1.41.
Permanent drift at the first floor of a reinforced concrete frame 4-storey structure under construction.
Ground floor drift may have been prevented by the infill masonry walls

GENERAL DETAILING

Seismic Gap

Is provided:
- between flexible and rigid buildings
- to divide buildings with special shapes
- between building' parts with different heights

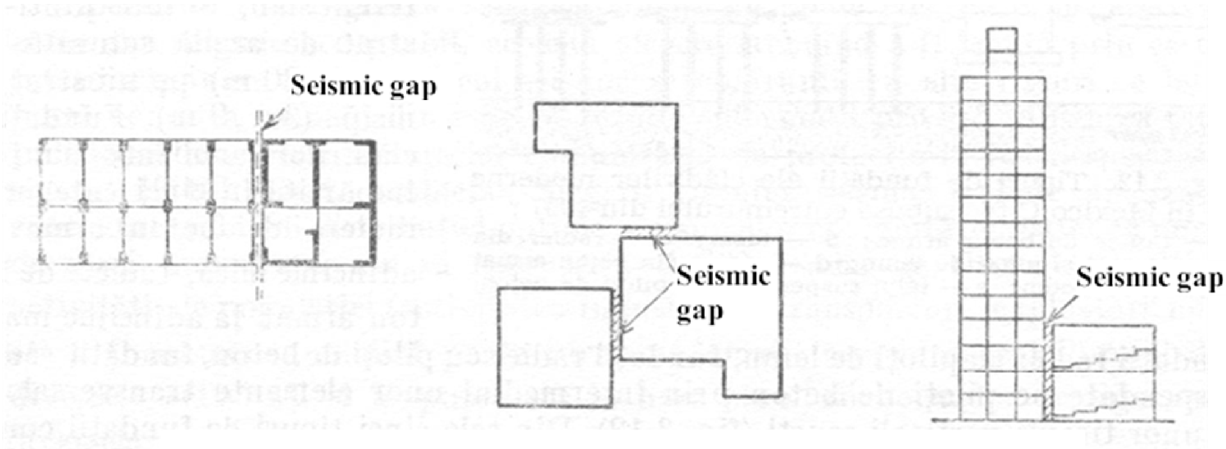


Figure 1.42. Seismic gap

Rules for providing the seismic gap:

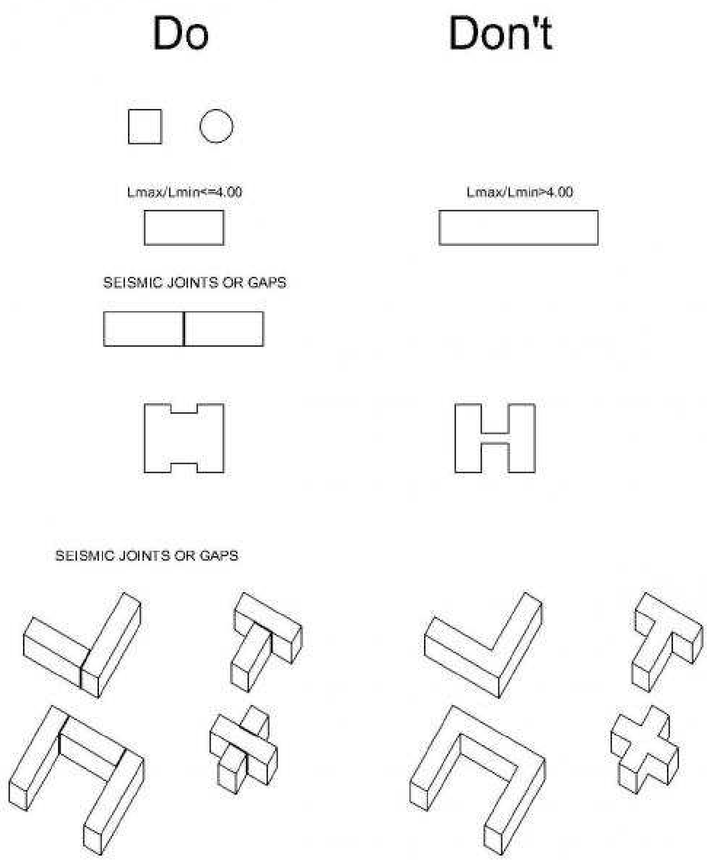


Figure 1.43. Desirable and undesirable plan shapes

Failure examples (from reports on earthquakes around the world [22-37]):

Figure 1.44. Bucharest, 1977: damage down the seismic gap line due to pounding [39]

Figure 1.45.
Different ground floor system – no seismic gap

Provisions for General Detailing

- Decreasing of self-weight.
- Uniform, symmetric, constant vertical variation of stiffness and masses. Avoiding soft-storeys.
- Decrease of torsional effects.
- Direct transmission of loads
  → column to column joint:

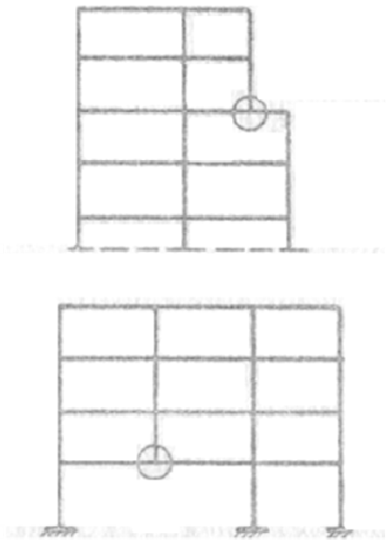


Figure 1.46. Undesirable column to beam joint

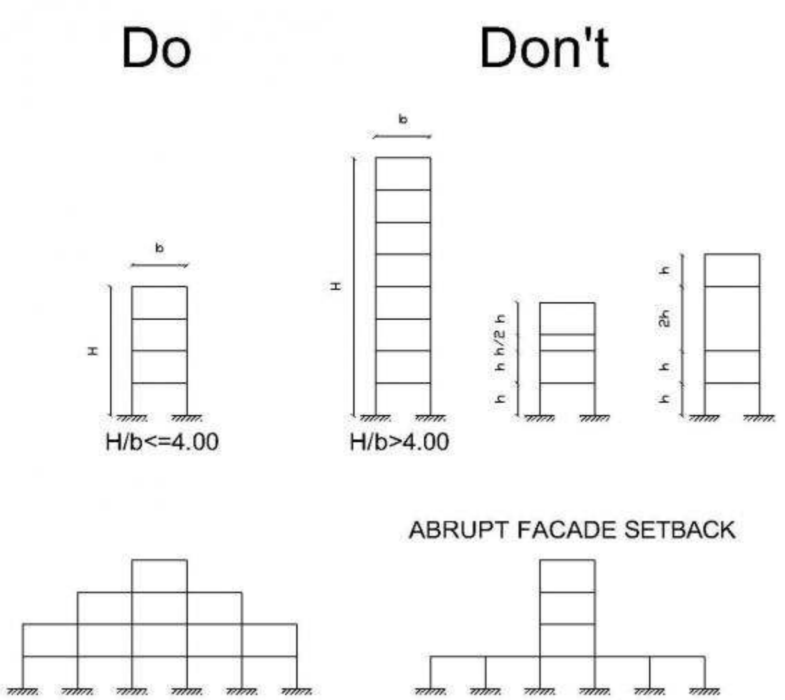


Figure 1.47.
Desirable and undesirable elevations

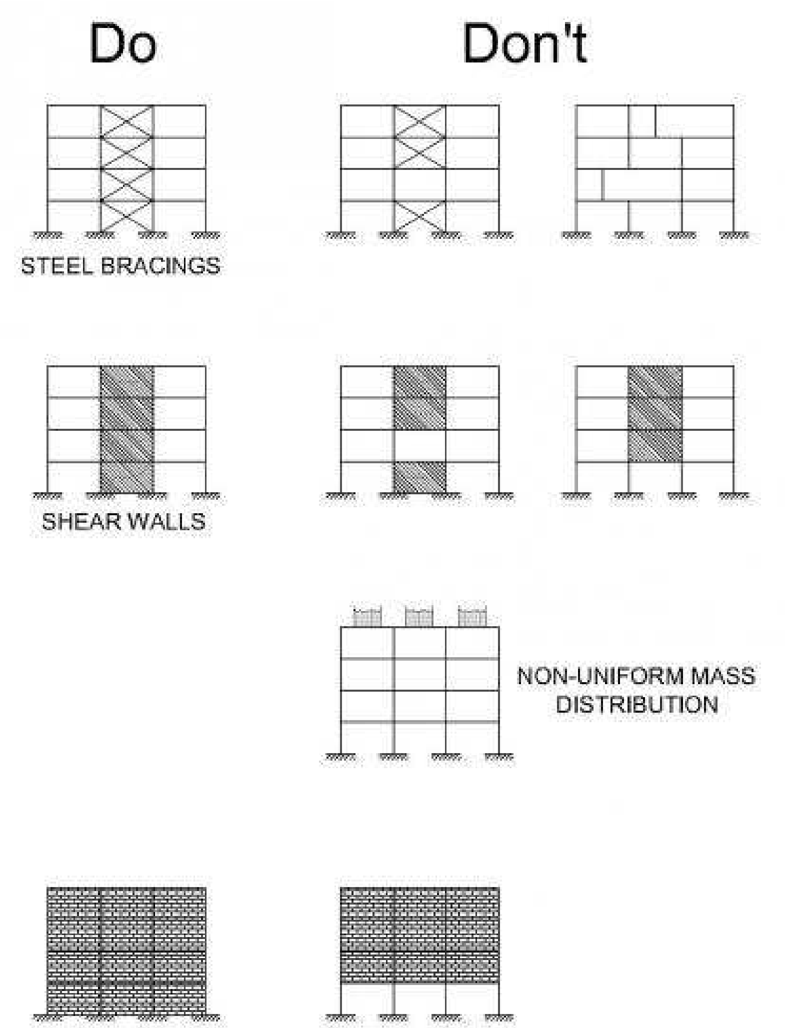


Figure 1.48.
Vertical uniformity and continuity in mass and stiffness

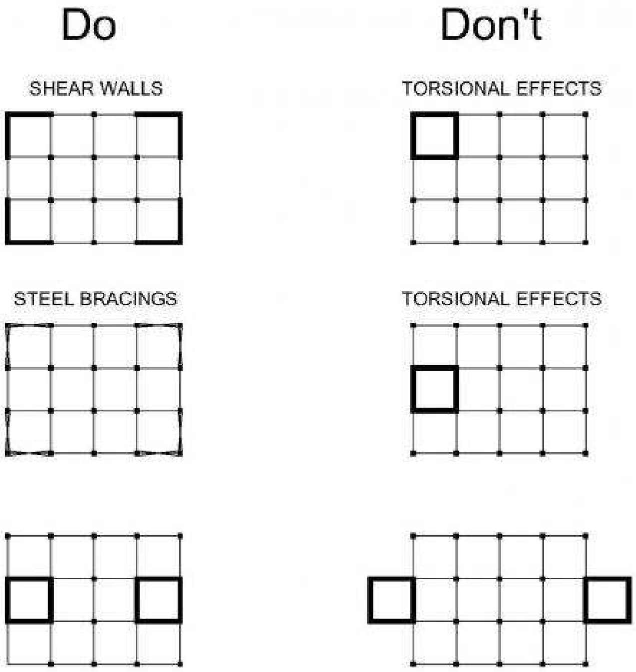


Figure 1.49.
Torsional effects

Failure examples (from reports on earthquakes around the world [22-37]):

Figure 1.50. Collapse of different soft storeys with changed stiffness

Figure 1.51. Soft ground storey collapse

Figure 1.52. Soft ground storey collapse

Figure 1.53.
Soft ground storey collapse

Figure 1.54.
Punching of flats slab by column due to vertical seismic action

Figure 1.55.
Restrained silos not damaged. Single silo buckling

1.3. REPAIR AND STRENGTHENING OF CONCRETE STRUCTURES

GENERAL SOLUTIONS FOR REHABILITATION

Regarding the rehabilitation solutions, for vertical irregularities the main solutions consist of:

- strengthening of existing structural elements and / or the structural system by increasing the strength stiffness and ductility of the weak structural elements;
- stalling additional structural members.

For both solutions it is necessary to avoid new stiffness discontinuities under lateral displacement. On the other hand, strengthening of vertical members at some levels may involve the rehabilitation of floors. In the case of horizontal structural irregularities, the aim of rehabilitation is to reduce the eccentricity between the centre of stiffness and the centre of mass: the result is a decreasing of torsion forces and displacements as well as an increase of the strength with the respect to lateral actions. The common solution is to use new symmetrical walls. For irregularities of the geometric plan, the rehabilitation solution consists of the use of new walls and / or seismic joints. The rehabilitation solutions for general structural vulnerabilities are presented below.

In the case of indirect transfer of strong forces and horizontal members with large span / high loads, the classical solution is to use additional columns (vertical or inclined) for transferring the strong forces to the existing (or new) foundations. For weak columns (compared with the adjacent beams) column strengthening is necessary. The rehabilitation solutions adopted in the case of deterioration of building component parts depend on the structural material.

Owing to structural vulnerabilities and/or torsion effects, elements of the system may be subjected to different displacements and some damages may result. Special rehabilitation systems may be used: adding abutments in directions of low stiffness; building of additional reinforced concrete walls.

SPECIFIC SOLUTIONS FOR STRENGTHENING OF REINFORCED CONCRETE STRUCTURES

Reinforced concrete structures are to be repaired and / or strengthened in cases when the general damage is limited [40-42], and demolished when the structural safety is greatly affected and the rehabilitation cost is very high.

Repairs are used for surface deterioration, cracks, damage resulting from casting defects and reinforcement corrosion. The methods used for repairs are: jacketing of damaged surfaces; infilling of cracks with usual mortar, epoxy resin or other polymers; replacement or strengthening of damaged reinforcement.

Strengthening of reinforced concrete structures takes into account the increase of strength, stiffness and ductility. In case of reinforced concrete framed structures, the increase in stiffness and ductility is to be achieved by jacketing of beams, columns and joints. The jacketing is performed by reinforced concrete, steel profiles, carbon fibres CFRP, etc. CFRP may be used for increasing ductility and slightly increased stiffness, see [43].

For reinforced concrete frame-wall structures the increase of bearing capacity is obtained by coating the core, the flange and the coupling beam.

Sometimes it is necessary to transform the existing structure completely, especially for framed structures. In this case, special techniques are to be used: steel bracing of reinforced concrete structures; infilling of frame openings with reinforced masonry or reinforced concrete.

Strengthening of Slabs

Strengthening of slabs could be done by classical solutions using reinforcement concrete. The possible solutions are [44-46]:

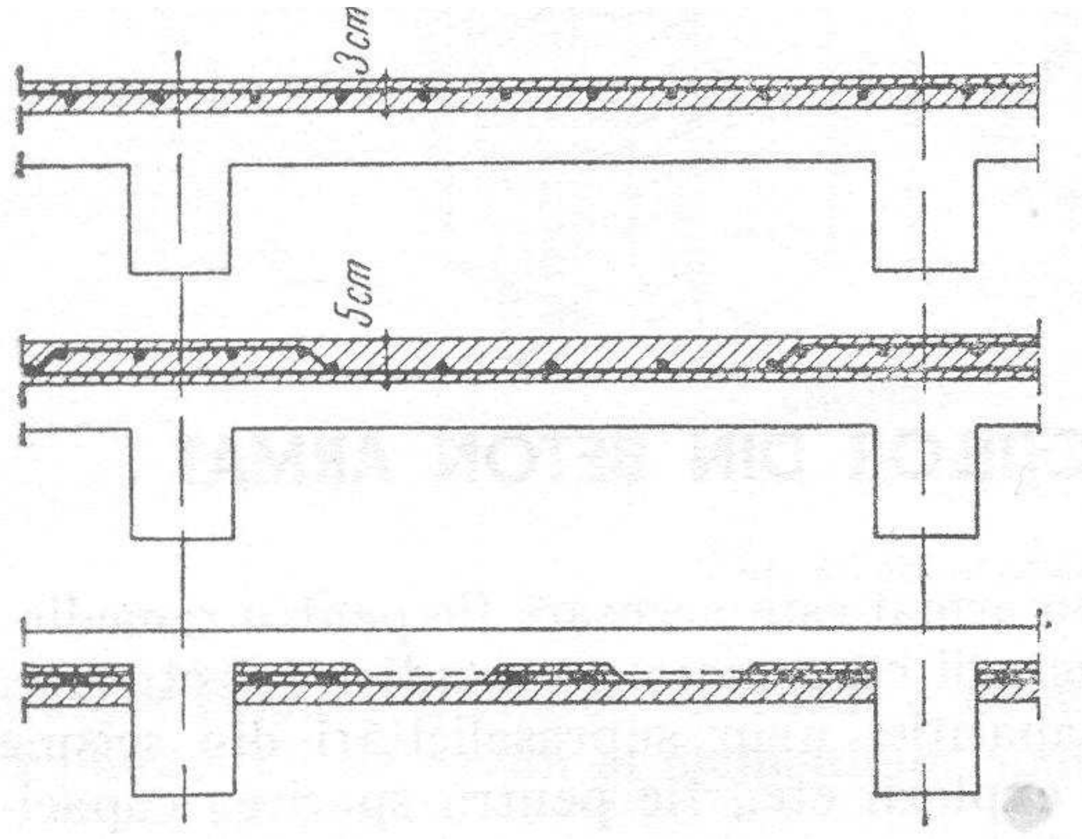


Figure 1.56. General strengthening of slab

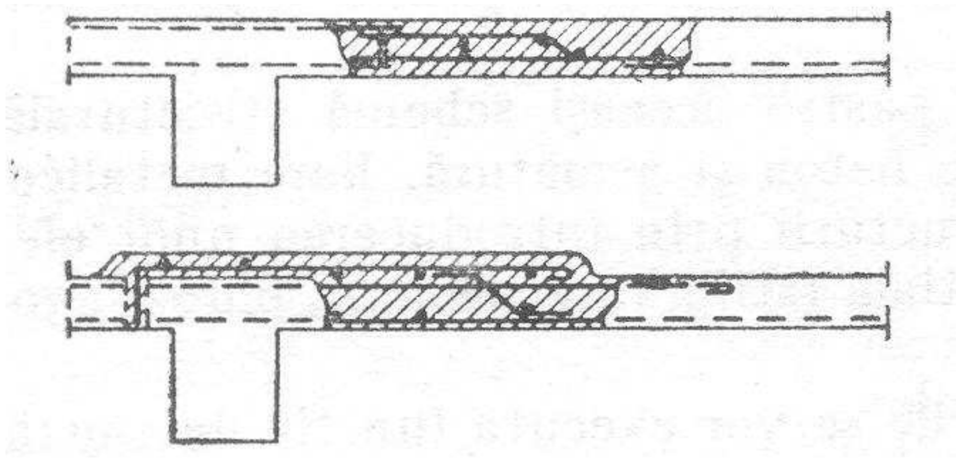


Figure 1.57. Local strengthening of slab

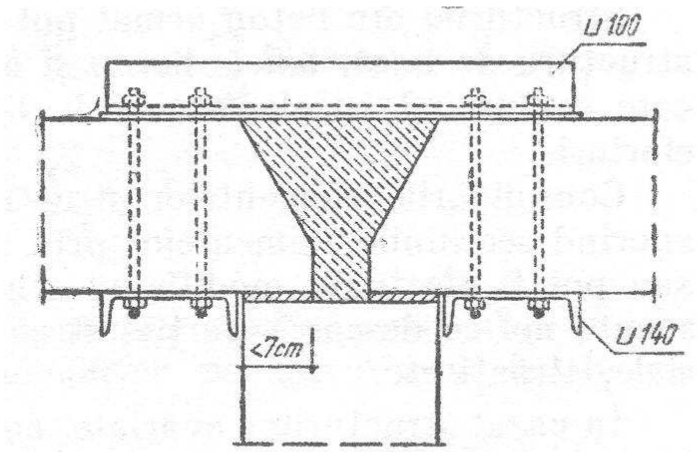


Figure 1.58. Strengthening of precast strips at supports

Strengthening of Beams

Strengthening of beams could be done by classical solutions using reinforcement concrete. The possible solutions are [44-46]:

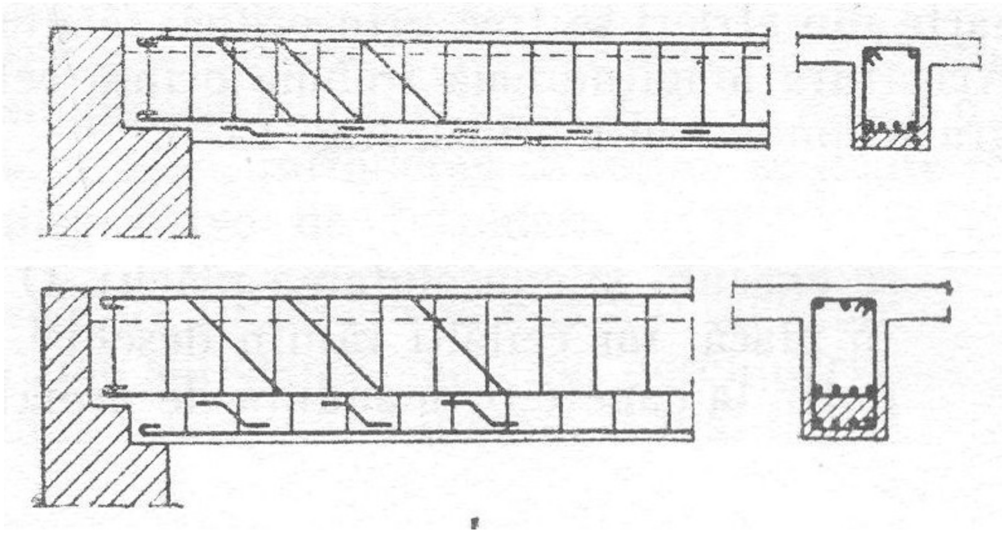


Figure 1.59. Strengthening by adding reinforced concrete at lower part

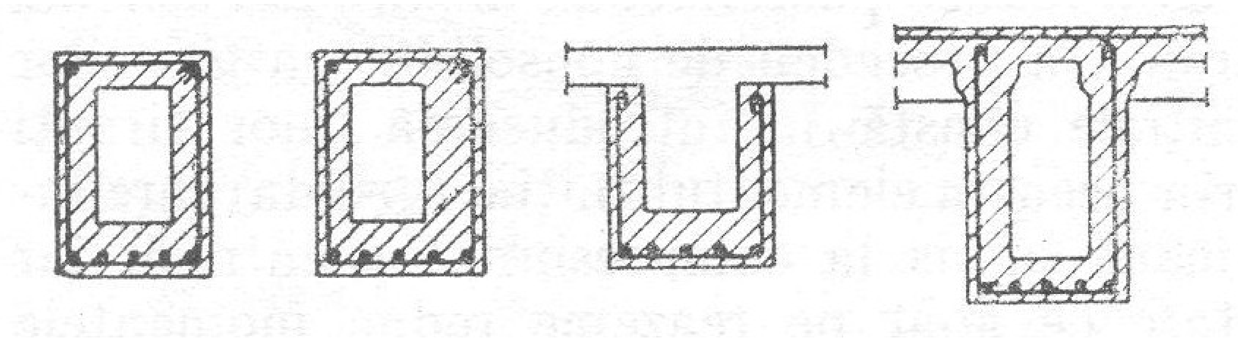


Figure 1.60. Strengthening by coating with reinforced concrete

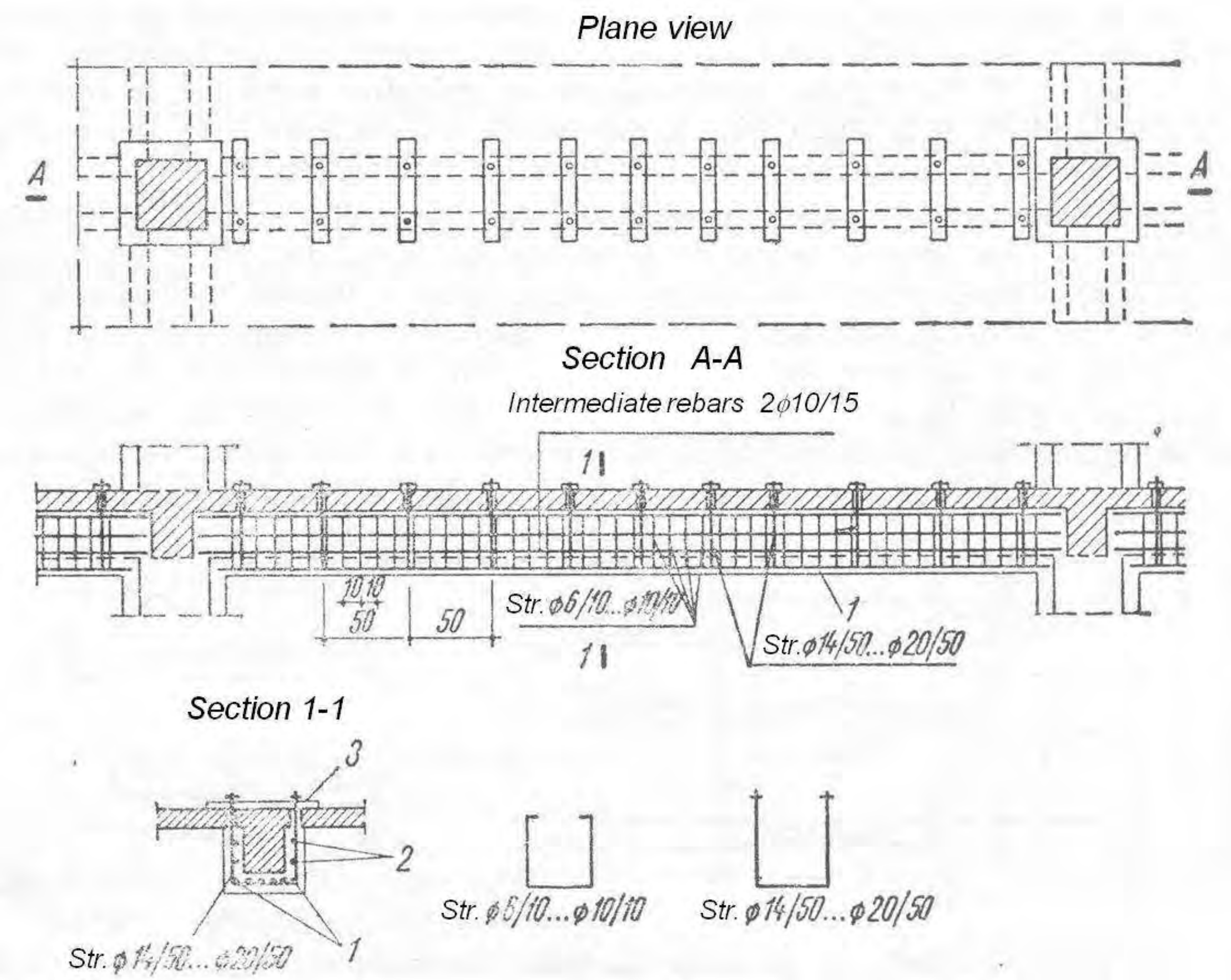


Figure 1.61. Strengthening of a frame beam

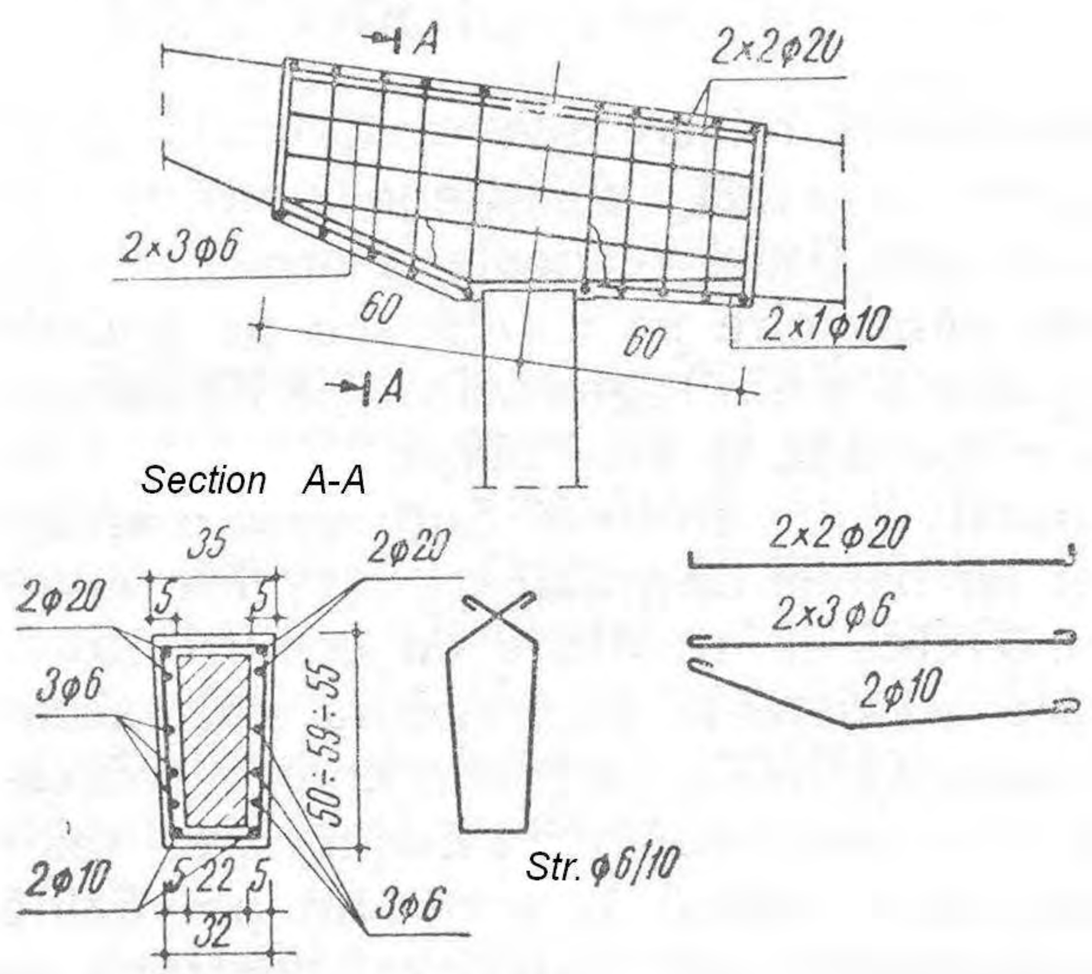


Figure 1.62. Local strengthening of beams at support

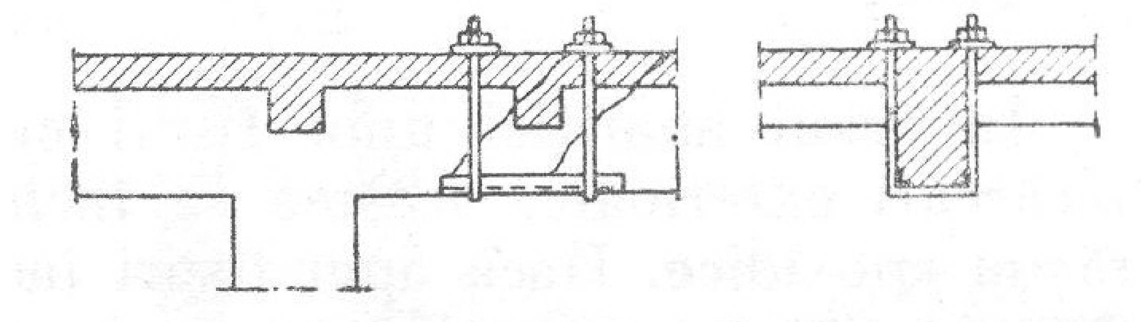


Figure 1.63. Local strengthening of beams with steel profiles

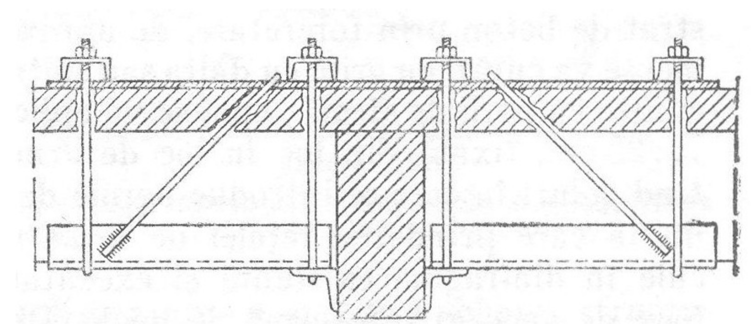


Figure 1.64. Local strengthening of beams at shear forces with steel profiles

Strengthening of Columns

Strengthening of columns could be done by classical solutions using reinforcement concrete [44-46]:

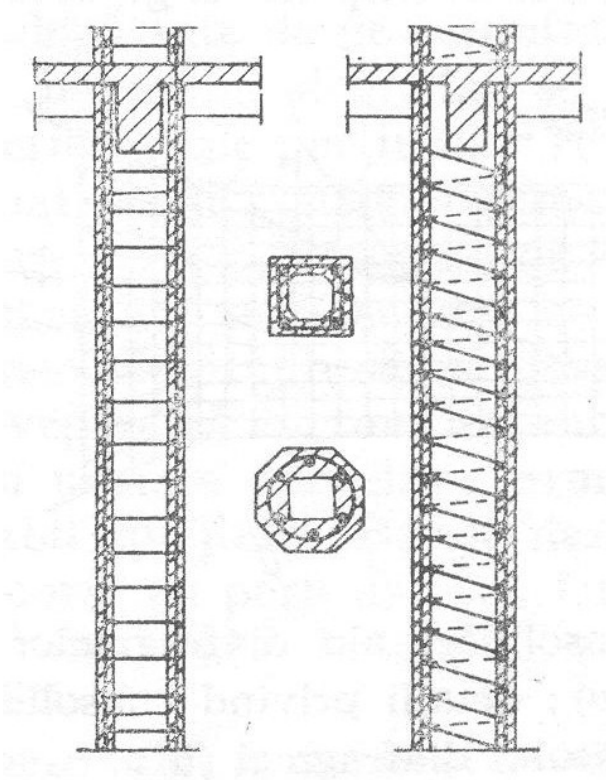
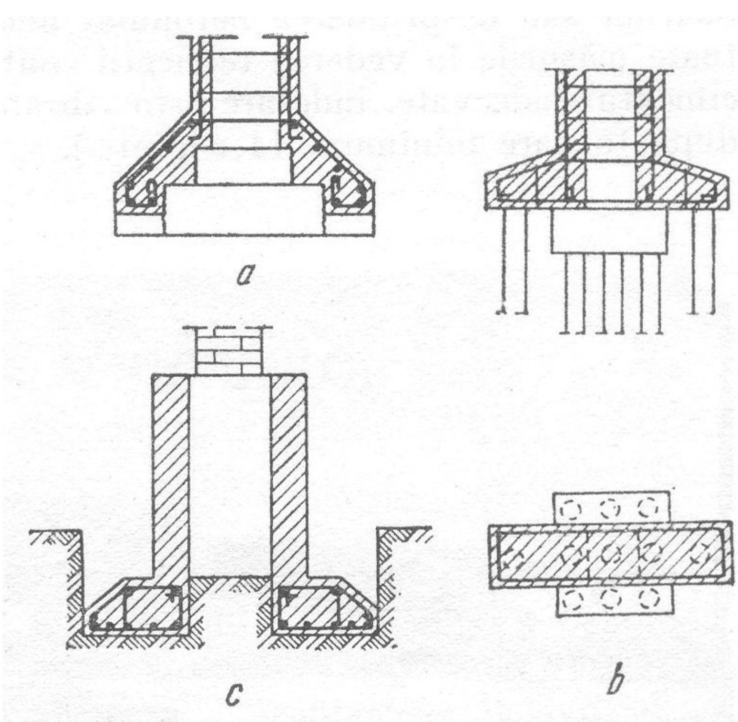


Figure 1.65. Strengthening of columns

Strengthening of Foundations

Strengthening of foundations could be done by classical solutions using reinforcement concrete [44-46]:



a – without extension of foundation horizontal dimensions
b – with extension of foundation horizontal dimensions
c – continuous foundation

Figure 1.66. Strengthening of foundations

2. REHABILITATION OF EXISTING CONCRETE STRUCTURES

2.1. EXPERIMENTAL RESEARCH

SOLUTIONS FOR BOND IMPROVING OF REINFORCED CONCRETE COLUMNS JACKETING

Structural strengthening represents an important aspect of the rehabilitation of existing RC structures. Some techniques for repairing and/or strengthening structures involve adding new concrete to an existing concrete substrate. One of the most commonly used strengthening techniques for structural elements is concrete jacketing [47-49].

The RC jacketing strengthening method is characterized by some important advantages:

- leads to a uniformly distributed increase in strength and stiffness of element (column);
- the durability of the original structural member is also improved;
- this strengthening procedure does not require specialized workers.

Different techniques for increasing the roughness of substrate surface are presented in literature [50-52]. Eduardo N.B.S. Julio et all [53] have been considered the following techniques: reference (1), surface prepared with steel brush (2), surface partially chipped (3), as in (3) plus water saturation 24 h prior concrete cast (4) and surface treated with sand-blasting (5). The values of the bond strength in tension, determined with the pull-off tests are: 1.92 MPa (N/mm^2) for surface prepared by procedure (2), 1.47 MPa for (3), 1.02 for (4) and 2.65 MPa for (5).

According to E.S. Julio, F. Branco and V.D. Silva [54-55] and other authors [56-60] the structural behaviour of a building rehabilitated by RC jacketing is highly influenced by applied technique and following aspects are to be considered: application of steel connectors – this should be considered only in the case of short RC columns to improve the level of strength and stiffness under cyclic loading; anchoring of the added longitudinal reinforcement – the steel bars can be efficiently anchored to the footing with a two-component epoxy resin.
The longitudinal reinforcement should be uniformly spread; added stirrups – half of the spacing of the original transverse reinforcement is recommended for the added stirrups to obtain a monolithic behaviour under cyclic loading; added concrete – a non shrinkage concrete should be adopted with characteristics of a self-compacting, high-strength and high-durability concrete.

The problem of pre-wetting the interface surface is controversial. The AASHTO-AGB-ARTBA Joint Committee recommends that the new concrete be cast on a dry concrete surface and on the other hand Canadian Standards Assoc. A 23 recommends wetting the old concrete surface for at least 24 h before the new concrete layer is cast.

The motivation for research and development into repairing, strengthening, and restoration of existing buildings in seismic zone is sustained by necessity to extend the life of structures. The masonry structures are the oldest and still very used type of buildings. The main target of the research represents the rehabilitation of old masonry buildings located in seismic zones.

EXPERIMENTAL PROGRAMME

The experimental programme focuses on quantifying the influence of different techniques for connecting between the two concrete layers: old concrete substrate and the added new concrete. Also, the influence of different old concrete quality was studied: a higher concrete class, as used in the present; a lower concrete class, as used in the past and encountered at old existing structures.

The test selected for the study was the pull-off test. The specimens were tested under compression using the standard procedure of cubes for compressive strength.
For the substrate concrete two classes were adopted – C 20/25 and C 16/20, since for the added concrete just one class of C 20/25 was adopted.

The adopted geometry for the pull-off specimens is presented in the Figure 2.1: a prism of 200x200x500 mm for the concrete substrate, reinforced with 8Φ12 mm PC52 longitudinal bars and stirrups Φ6/150 mm OB37; added concrete as RC jacketing of 100 mm width as illustrated in the other Figure 2.2 and a reinforcement similar to the inner prism.

Figure 2.1. RC columns – initial specimens

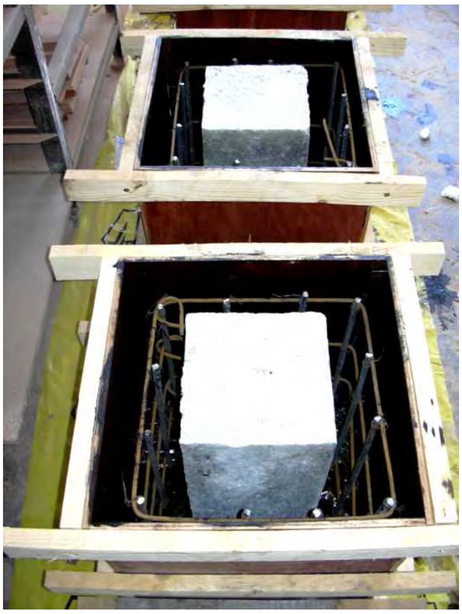


Figure 2.2. Initial RC columns and RC jacketing

The connection between the substrate (RC prism) and the added concrete (RC jacketing) were:

- three specimens without special technique for connection (concrete-to-concrete bond), Figure 2.2;
- one specimen with a bonding agent – a two-component epoxy resin, Figure 2.3;
- one specimen with steel connectors Φ10 mm PC52, anchored in the prism with a two-component epoxy resin, Figure 2.4;
- one specimen with special mechanical connectors M10/40/100 mm, anchored in the substrate into holes drilled in the prism (Φ12 mm), Figure 2.5.

Figure 2.3. Using of bonding epoxy resin agent

Figure 2.4. Using of chemical anchored connectors

Figure 2.5. Using of mechanical anchored connectors

For each variable, standard specimens (cubes) were casted to characterize the compressive strength of the concrete substrate and of the added concrete.

The samples of RC columns strengthened by jacketing after the pull-off tests are presented in Figure 2.6:

Figure 2.6. RC column samples after the pull-off tests

The cracking and fracture pattern at pull-off tests is similar to compression tests on prism specimen: either by vertical longitudinal cracking, Figure 2.7, or by inclined cracking, Figure 2.8.

Figure 2.7. Fracture by longitudinal cracking at pull-off test

Figure 2.8. Fracture by inclined cracking at pull-off test

Vertical cracking and pull-out of the prism is the normal failure, at which the bond strength was 1.92 N/mm^2. This value was used for further comparison of results. Inclined cracking appeared due to imperfections at casting (inclined inner prism). In this case the bond strength was 1.50 N/mm^2 smaller than at the normal failure. Consequently, this value was neglected.

The results obtained on the specimens without special techniques for connection, concrete-to-concrete bond, as well on the specimens with improved bond by using special techniques, are presented in Table 2.1.

Table 2.1. Pull-off test results

No.	Concrete-to-concrete bond		Improved Bond		Efficiency
	Bond strength τ_{ai} [N/mm^2]	$\frac{f_c^{prism}}{f_c^{jacketing}}$	Bonding solution	Bond strength τ_{af} [N/mm^2]	$(\tau_{af} - \tau_{ai}) / \tau_{ai} \times 100$ [%]
1*	1.30	$\frac{25.4}{27.5}$	epoxy resin bonding agent	1.38	6 %
2*	1.42	$\frac{25.4}{27.5}$	mechanical anchored connectors	1.70	23 %
3*	1.38	$\frac{25.4}{27.5}$	chemical anchored connectors	2.18	54 %
4**	1.38	$\frac{20.0}{36.8}$	chemical anchored connectors	1.92	39 %
Notes: - substrate concrete (inner prism): * C 20/25; ** C16/20 - added concrete (jacketing): C 20/25.					

Since the most efficient bonding solution has been the chemical anchored connectors, several tests using this type of bonding were performed on two different substrate concrete classes – C 16/20 and C 20/25. The influence of concrete class substrate (inner prism) is also illustrated in Figure 2.9:

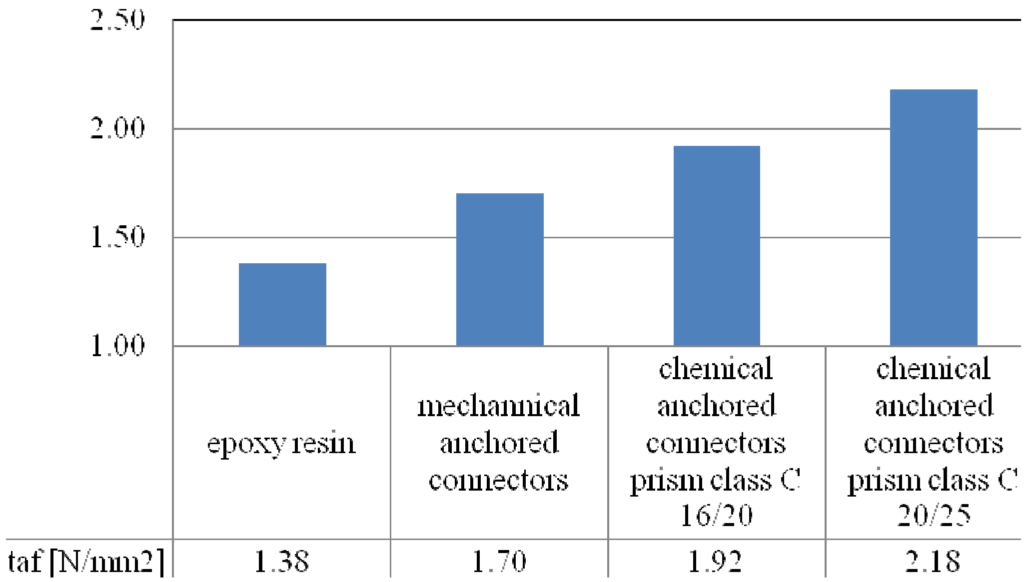


Figure 2.9. Influence of bonding solution on bond strength

For chemical anchored connectors the higher class of concrete of inner prism from C 16/20 to C 20/25 increased the bond strength with 15 % (from 1.92 N/mm^2 to 2.18 N/mm^2).

STRENGTHENING OF REINFORCED CONCRETE FRAMED STRUCTURES IN SEISMIC ZONES BY USING CARBON FIBER REINFORCED POLYMERS (CFRP)

Reinforced concrete (RC) structures are to be repaired and/or strengthened in cases when the general damage is limited, and demolished when the structural safety is greatly affected and the rehabilitation cost is very high.

Repairs are used for surface deterioration, cracks, casting defects and reinforcement corrosion. By repairing there is no increase in strength or stiffness in relation to the initial structure. The methods used for repairs are: covering of damaged surfaces; infilling of cracks with cement mortar, epoxy resin or other polymers; replacement or strengthening of damaged reinforcement.

Strengthening of reinforced concrete structures may take into account the increase of strength and/or stiffness and/or ductility. In case of RC framed structures, the increase in stiffness and ductility is possible to be achieved by jacketing beams, columns and joints [61]. The jacketing is performed by reinforced concrete, steel profiles, carbon fibers.

Sometimes it is necessary to transform the existing structure completely, especially for framed structures. In this case, special techniques are to be used: steel bracing; infilling of frame openings with reinforced masonry or reinforced concrete.

STRENGTHENING BY CARBON FIBER REINFORCED POLYMERS (CFRP)

CFRP systems are suitable for strengthening of RC structures due to their technical and economical advantages. Classic strengthening solutions may lead to some inconveniences, since these methods have been costly and disruptive to operation. A typical approach is the increasing of elements' dimensions with consequent mass increasing and leading to seismic problems. Furthermore, if reinforcement corrosion is present and its causes are not carefully removed the corrosion will continue.

CFRP systems' advantages as rehabilitation application at seismic resistant structures are: increase of load-carrying capacity; structural elements designed only for gravity loads will be able to withstand seismic loads; elements' mass remains, practically, the same; the technology is simple and rapid [62-63].

For strengthening of existing RC framed structures in seismic zones a very important target is to avoid the development of plastic hinges in columns. Results the necessity to increase columns' bending and shear resistance. A retrofit using CFRP vertical strips and horizontal wrap for columns means increasing of local ductility and deformation capacity as well as of entire bending capacity.

The strengthening of columns using CFRP vertical strips will increase the resistance capacity as well as the stiffness of the element. The increased flexural strength of column will force the plastic hinge to form at beam ends. On the other hand the increase of vertical elements stiffness will reduce the structural story drift under seismic motion.

The two effects of columns strengthening are responsible for increasing the horizontal load capacity and, finally, the structural dissipation energy. The advantages presented above are effective only if the shear capacity is also increasing and the debonding of vertical CFRP strips is eliminated by efficient systems.

The possibility of avoiding shear failure of column end (potential plastic hinge) may be solved by CFRP wrap confinement; results and increase of column shear strength, as well as ductility, and will transmit the plastic hinge at the beams.

The debonding of FRP strips, disposed along the column axis, in the form of peeling-off failure at the beam-column joint or column-foundation joint, it is necessary to be solved by different systems: a continuous fiber application in the longitudinal column direction where this possibility exists; by creating of some vertical gaps around the columns in which strips are anchored; by using special anchoring devices, such as steel plates and rods (El-Amoury and Ghobarah [64], Parese et all [65]). The verifying of the end anchorage can follows the model presented in the *fib* Bulletin 14 [66] which gives the maximum FRP force which can be anchored and the minimum anchorage length.

The analysis of RC structures members before and after strengthening using CFRP systems are also presented in the *fib* Bulletin 14 [66].

A quick overview of the experimental research and analytical studies performed by many authors on the effects of CFRP systems used at RC structures show up some interesting results:

- Test results (Mosallam [67]) on beam-column joints of RC frame structures show an important increase of strength, up to 53 % and ductility up to 42 %.
- Parvin and Granata [68] illustrated an increase in the moment capacity up to 37 % given by an analytical analysis (FEM) on exterior beam-column joints.
- The confining pressure of the FRP jackets on bridge columns with a circular cross section shows an increase of the lateral bending strength by 19-40 % (Sclick and Brena [69]).
- The results from an experimental study on a full scale RC structure illustrated an increase by 86-100 % of base shear force and about 100 % increase of lateral top-displacement capacity by using FRP strengthening (Della Corte, Borecchia and Mazzolani [70]).

EXPERIMENTAL RESEARCH

Detailing of experimental models

The experimental program focused on RC frames assumed as existing structures. Single span and single story frame (scale 1:2), see Figure 2.10 and 2.11, was designed and detailed according to the Romanian design codes from 1970 under which seismic design was inadequate: mainly, a lack of reinforcement to withstand the present-day seismic actions.

The materials used were: concrete C16/20 (f_{ck} = 16 N/mm^2; f_{cd} = 11 N/mm^2; ε_{cu} = 0,0035; E_c = 27 kN/mm^2); Romanian plain reinforcement bars OB37 (f_{yk} = 245 N/mm^2; f_{yd} = 210 N/mm^2; E_s = 210 kN/mm^2).

Figure 2.10. Reinforced concrete frame model

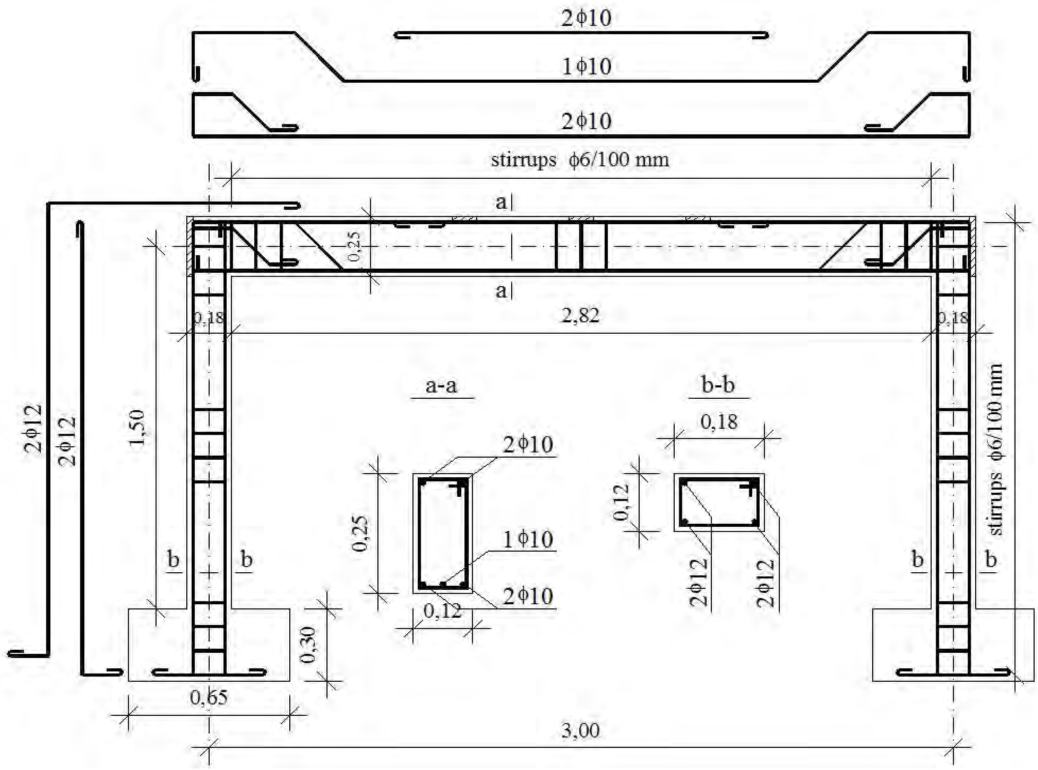


Figure 2.11. Reinforced concrete frame detailing

Finally, due to practical aspects, the manufacture of the experimental models was done as prefabricated frames, Figure 2.12 which were placed in cast on-site foundations, Figure 2.13.

Figure 2.12. Prefabricated frame

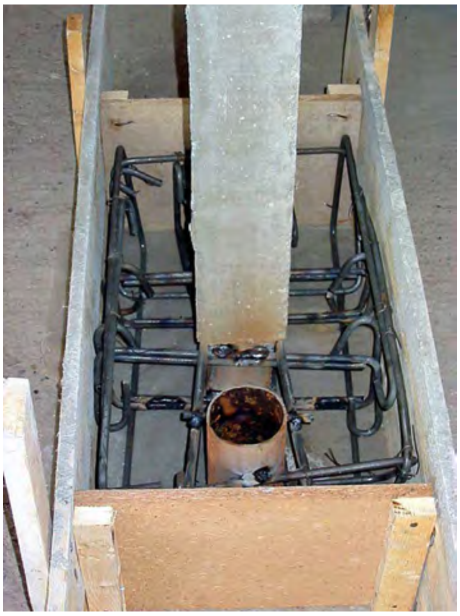


Figure 2.13. Cast on-site foundation.

These frames were loaded vertically with constant forces P and horizontally with variable alternant forces S^+ or S^- (seismic action), as presented in Figures 2.14 and 2.15. During the test were measured: load forces S; strain in reinforcement bars of columns and beams; horizontal and vertical displacements.

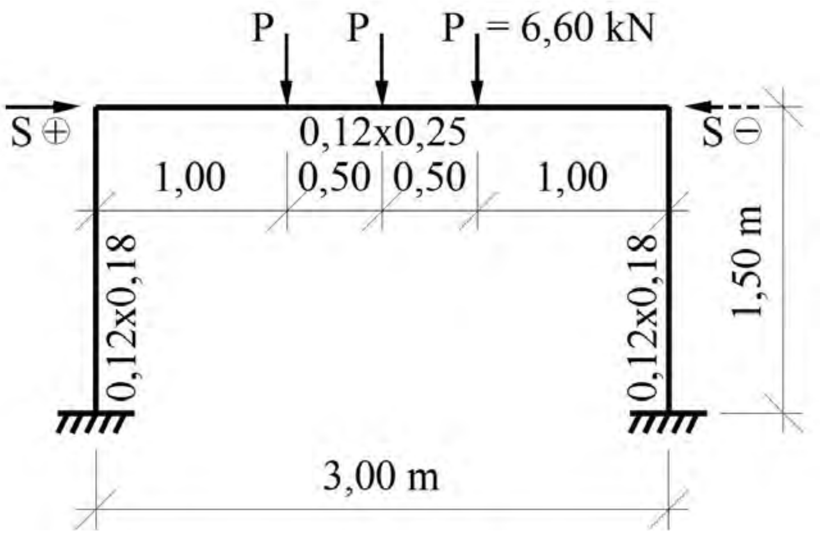


Figure 2.14. RC frame - loading outline

Figure 2.15. RC frame – experimental loading

All the tests were done in force controlled conditions, and performed as alternant horizontal cycles, first up to the service stage, and second up to the yielding of reinforcement and failure stage.

The RC structural design, according to the Romanian design codes from 1970, and the magnitude of applied forces were ensuring the failure mechanism, of non-strengthened RC frames, by plastic hinges at columns ends as in Figure 2.16. It is typical “strong beams-week columns” failure model.

Figure 2.16. Damage of columns

Then the RC frames were strengthened on both columns by using CFRP materials, as in Figure 2.17:

- longitudinal strips, on two sides, had a width b_f = 25 mm and a thickness t_f = 1,2 mm. The strips were anchored in foundations (on 100 mm depth) and at the top joints in different manners: glued anchorage – Figure 2.18.; wrap anchorage – Figure 2.19; mechanical anchorage – Figure 2.20;
- transversal confinement with a single layer of wrap closed jacket at both ends of the columns. The jackets had a width b_f = 300 mm and a thickness t_j = 0,12 mm.

Figure 2.17. RC frame + CFRP strengthening

Figure 2.18. Glued anchorage – Frame 1 (CADRU 1)

Figure 2.19. Wrap anchorage – Frame 2 (CADRU 2)

Figure 2.20. Mechanical anchorage – Frame 3 (CADRU 3)

CFRP materials characteristics used for strengthening are: E_f = 165 kN/mm^2 and ε_{fu} = 0,017 for longitudinal strips; E_j = 231 kN/mm^2 and ε_{ju} = 0,017 for transversal wraps. The bond of CFRP materials to the existing concrete layer was assured by specific epoxy adhesives.

Experimental results

During experimental tests the following parameters were measured: horizontal load S; horizontal displacement at the column-beam node; vertical displacement in the middle of the beam; strain in the longitudinal rebars at the ends of columns and beam; strain in the bottom longitudinal rebars in the middle of the beam.

From all the experimental data given by tests performed on frame structures the most significant, for the present study, are presented in the Table 2.2 and Figure 2.21:

Table 2.2. Experimental results

Model	State of structure	Horizontal load S [kN]	Top maximum displacement [mm]	Ratio $\frac{\text{strengthened}}{\text{non - strengthened}}$ for	
				Loads	Displacements
Frame 1	Non-strengthened	16 *	5,44	–	$\frac{0{,}71^*}{–}$
	CFRP strengthened	16 *	3,87		
		36	14,73		
		40 **	30,20		
Frame 2	Non-strengthened	16 *	4,60	1,06	$\frac{0{,}98^*}{2{,}00^{**}}$
		36 **	15,27		
	CFRP strengthened	16 *	4,50		
		36	15,27		
		38 **	30,70		
Frame 3	Non-strengthened	16 *	7,60	–	$\frac{0{,}72^*}{–}$
	CFRP strengthened	16 *	5,50		
		36 **	29,80		
* yielding stage of reinforcement; ** ultimate stage					

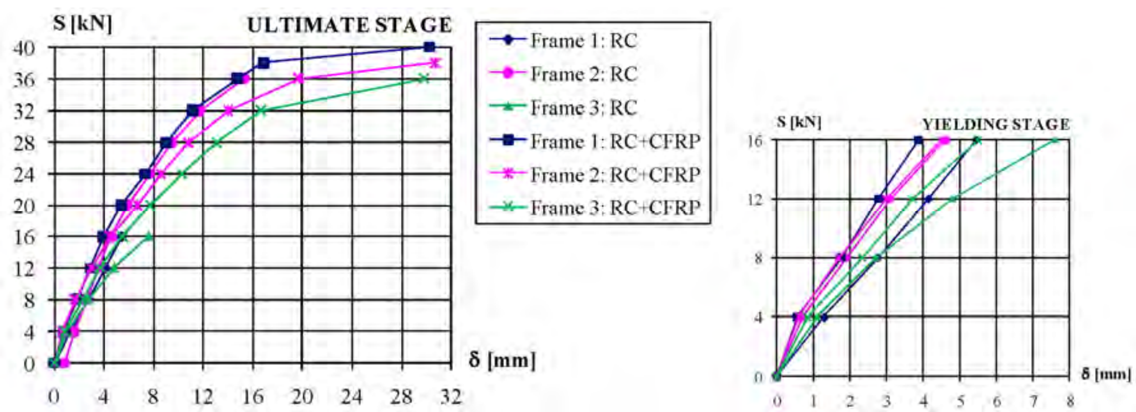


Figure 2.21. Top-displacement values for RC non-strengthened and CFRP strengthened frames.

The experimental program emphasized some important aspects regarding the behavior and failure of RC frames strengthened with CFRP:

- debonding of CFRP vertical strips from glued anchorage (Frame 1 – CADRU 1), see Figure 2.22, and wrap anchorage (Frame 2) due to tensile stress at the top joints;
- no debonding of CFRP strips from mechanical anchorage (Frame 3) at the top joints;
- debonding of CFRP vertical strips on inside face of the column (Frame 2 – CADRRU 2) due to compression stress which shows the lack of some necessary transversal stirrups to prevent strip buckling, see Figure 2.23;
- a pull-out tendency of ordinary concrete around the polymer mortar used for CFRP fixing into foundations, see Figure 2.24.

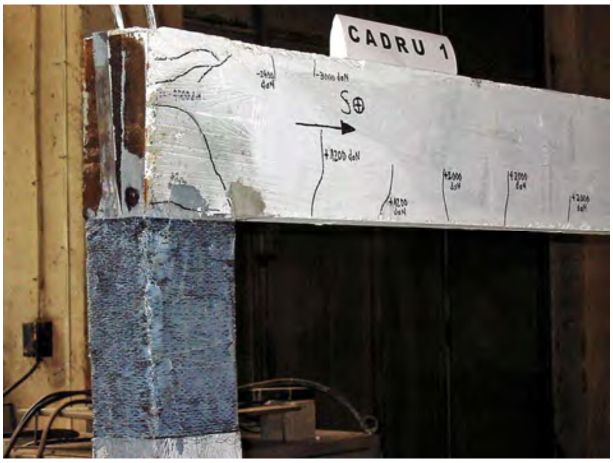


Figure 2.22. Debonding of CFRP vertical strip due to tensile stresses

Figure 2.23. Debonding of CFRP vertical strip due to compression stresses

Figure 2.24. Pull-out of concrete and CFRP from foundation

Theoretical values

The theoretical values are based on Eurocode 2: “Design of concrete structures – Part 1-1: general rules and rules for buildings” [71] for non-strengthened structure and on *fib* Bulletin 14 “Externally bonded FRP reinforcement for RC structures” [66], *fib* Bulletin 24 “State-of-art report: Seismic assessment and retrofit of reinforced concrete buildings” [72] and *fib* Bulletin 35 “Retrofitting of concrete structures by externally bonded FRPs” [73] for strengthened structure.

The resistance capacity was analyzed for columns subjected to flexure with compression axial force. The resistance capacity expressed by bending moment M_{Rd} was calculated for yielding stage and for ultimate stage.

For non-strengthened structure, the calculus of M_{Rd} was done for: rectangular cross-section 120x180 mm; symmetric reinforcement $A_{s1} = A_{s2}$ (2ϕ12 mm); yielding and ultimate strength of steel, since this has been an experimental test; actual magnitude of compression force N_{Ed}.

For strengthened structure, the calculus of M_{Rd} was done by using the following equations:

$$M_{Rd} = A_{s1} \cdot f_{yd} \cdot (d - 0,4 \cdot x) + A_f \cdot \sigma_f \cdot (h - 0,4 \cdot x) + A_{s2} \cdot E_s \cdot \varepsilon_{s2} \cdot (0,4 \cdot x - d_2) + N_{Ed}(h/2 - d_2)$$

where the neutral axis depth *x*, for flexure with compression axial force, is found by solving:

$$N_{Ed} = \psi \cdot b \cdot x \cdot f_{cd} + A_{s2} \cdot E_s \cdot \varepsilon_{s2} - A_{s1} \cdot f_{yd} - A_f \cdot \sigma_f$$

and the significant notations taken from the Figure 2.25.

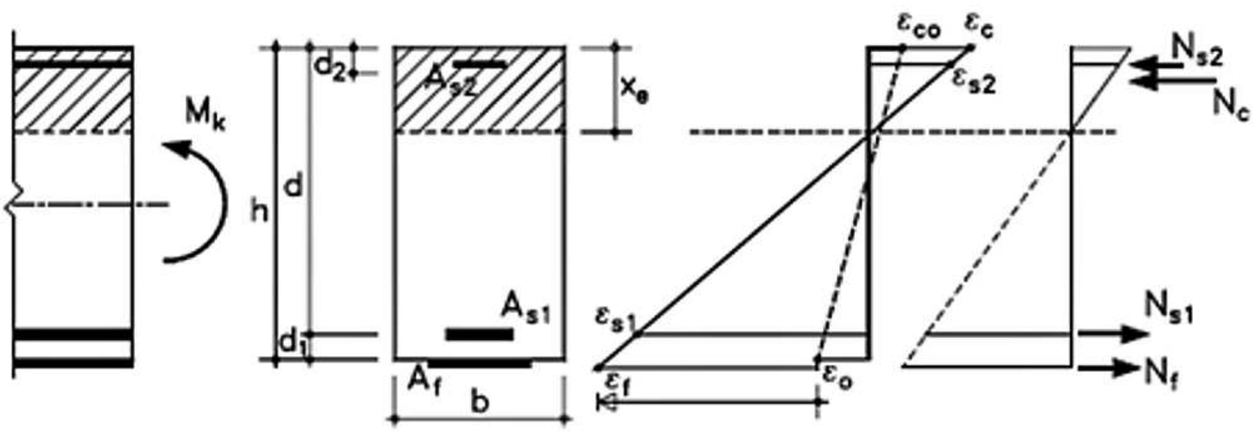


Figure 2.25. Linear elastic analysis of RC + CFRP strengthening cracked cross-section.

The value of stress in CFRP strips σ_f is:

- in case of steel yielding followed by concrete crushing

$$\sigma_f = E_f \cdot \varepsilon_f = E_f \cdot \left(\varepsilon_{cu} \cdot \frac{h - x}{x} - \varepsilon_0 \right)$$

- in case of end debonding of CFRP strips, the debonding strength (Monti)

$$\sigma_f = f_{fdd} = \sqrt{\frac{2 \cdot E_f \cdot \Gamma_{Fk}}{t_f}}$$

with $\Gamma_{Fk} = 0{,}03 \cdot \sqrt{f_{ck} \cdot f_{ctm}}$

Since the calculation was based on an experimental test, the previous Equations were using: reinforced concrete cross-section of the non-strengthened structure; CFRP strip area $A_f = b_f \cdot t_f = 30$ mm^2; yielding and ultimate strength of steel; characteristic and mean compression and tensile strengths of concrete; actual magnitude of compression force N_{Ed}; partial factors for loads and strengths equal to 1,0.

The top-displacements were calculated at the same stages for theoretical as well as for experimental approaches. The values of deflections were obtained by using a simplified calculation (*CEB-FIP Model Code* [74]) which gives reasonable accurate prediction at the service limit state SLS. The mean deflection is:

$$\delta = \delta_1 \cdot (1 - \zeta_b) + \delta_2 \cdot \zeta_b$$

where: δ_1 is the deflection in the un-cracked state and δ_2 for cracked state respectively;
ζ_b is a distribution coefficient which was taken as $0{,}3 \cdot l$ (l – column length) at each cracked end of the column.

The deflections δ_1 and δ_2 were obtained as follows:

$$\delta_1 = K \cdot \frac{S \cdot l^3}{E_c' \cdot I_1} \qquad \delta_2 = K \cdot \frac{S \cdot l^3}{E_c' \cdot I_2}$$

where: K = 1/12; S is the horizontal load for one column;
$E_c' = 0{,}8 \cdot E_c$;
I_1 is the inertia moment of un-cracked concrete cross-section;
I_2 is the inertia moment in the cracked state given by:

- for non-strengthened RC cross-section

$$I_2 = \frac{b \cdot x_o^3}{3} + (\alpha_s - 1) \cdot A_{s2} \cdot (x_o - d_2)^2 + \alpha_s \cdot A_{s1} \cdot (d - x_o)^2$$

and the neutral axis depth x_o can be solved from

$$\frac{b \cdot x_o^2}{2} + (\alpha_s - 1) \cdot A_{s2} \cdot (x_o - d_2) = \alpha_s \cdot A_{s1} \cdot (d - x_o)$$

- for CFRP strengthened cross-section, based on previous Figure 2.25 (Matthys – [73])

$$I_2 = \frac{b \cdot x_e^3}{3} + (\alpha_s - 1) \cdot A_{s2} \cdot (x_e - d_2)^2 + \alpha_s \cdot A_{s1} \cdot (d - x_e)^2 + \alpha_f \cdot A_f \cdot (h - x_e)^2$$

and the neutral axis depth x_e can be solved from

$$\frac{b \cdot x_e^2}{2} + (\alpha_s - 1) \cdot A_{s2} \cdot (x_e - d_2) = \alpha_s \cdot A_{s1} \cdot (d - x_e) + \alpha_f \cdot A_f \cdot (h - x_e)$$

where: $\alpha_s = E_s / E_c$; $\alpha_f = E_f / E_c$.

The depths of the neutral axis x and x_e are found by solving the first order moment equations for RC cross-section, respectively, CFRP strengthened cross-section.

Conclusions of experimental program

A comparison between experimental results of reinforced concrete framed structures and theoretical values are presented in the Table 2.3:

Table 2.3. Comparison between theoretical and experimental values

State of structure		Resistance capacity M_{Rd} [kNm]			Top-displacement δ [mm]		
		Theor.	Exp.	Exp./Theor.	Theor.	Exp.	Exp./Theor.
Non-strengthened structure	yielding stage	7,92	8,77	1,11	4,03	5,88	1,46
	ultimate stage	12,90	17,06	1,32	9,07	15,27	1,68
CFRP strengthened structure	yielding stage	10,34	8,77	0,85	3,06	4,62	1,51
	ultimate stage	13,86	17,89	1,29	7,65	21,41	2,80
Variation of strength and stiffness [%]	yielding stage	31 %	0 %	-	32 %	27 %	-
	ultimate stage	7 %	5 %	-	18 %	-29 %	-

From experimental data and theoretical values presented in previous tables it can be noticed:

- the values of the maximum horizontal loads were chosen differently for the two non-strengthened frames in order to vary the application level of strengthening: 16 kN (yielding stage of reinforcement) for Frame 1 and 3; 36 kN (ultimate stage) for Frame 2;

- the increase of the experimental bending moment at ultimate stage was only 5 % due to debonding of CFRP strips;

- the experimental values of top-displacements are higher than theoretical data. This fact can be explained by the tangent modulus of concrete E_c used instead of secant modulus;

- the increase of stiffness for the strengthened structure implies the smaller value of top-displacement at the yielding stage by 27 % for experimental framed structure and by 32 % for calculated displacement.

FINAL CONCLUSIONS

The experimental tests performed on RC framed structure emphasized some main aspects of the CFRP strengthening system:

- The slight increase of resistance capacity by 5 % at the ultimate stage and the decrease of top-displacement by 27 % at the service stage.

  Some results given by other authors, but in different conditions, are presented in the Table 2.4:

Table 2.4. The effect of CFRP strengthening on RC structures

Author(s)	The effects [%] on	
	strength	stiffness
Mosallam – 2000 [67]	52	42
Parvin and Granata – 2000 [68]	37*	-
Sclick and Brena – 2004 [69]	19 - 40	-
Della Corte et all – 2005 [70]	86 - 100	100
Nagy-Gyorgy et all – 2004 [75]	48 / 33*	- (31 - 69)
Dan and Bob – 2010	5 / 7*	27 / 32*
* Theoretical values		

- Peeling-off failure of CFRP strips at the top joint due to debonding without a proper anchorage.

- Debonding of CFRP strips in the compressed zone in case of proper transversal reinforcement absence.

The strengthening solutions based on CFRP systems have some important technical and economical advantages: facile strengthening technology and short refurbishment period; resistance to aggressive environments; safe behavior under seismic action; more sustainable solution.

2.2. CASE STUDIES

THE WESTERN UNIVERSITY OF TIMISOARA

The Western University has many buildings, among them the Main Building – Figure 2.26 [76], that is used as administrative part as well as classrooms for students, was built in 1962-1963:

Figure 2.26. The Western University of Timisoara – Main Building

The reinforced concrete structure consists of:

- transversal and longitudinal frames with eight storeys and two spans of 5.6 m and eleven bays of 3.8 m, see Figure 2.27;
- floors with girder mesh in two directions and a slab of 10 cm;
- foundation with a thick slab and deep beams in two directions.

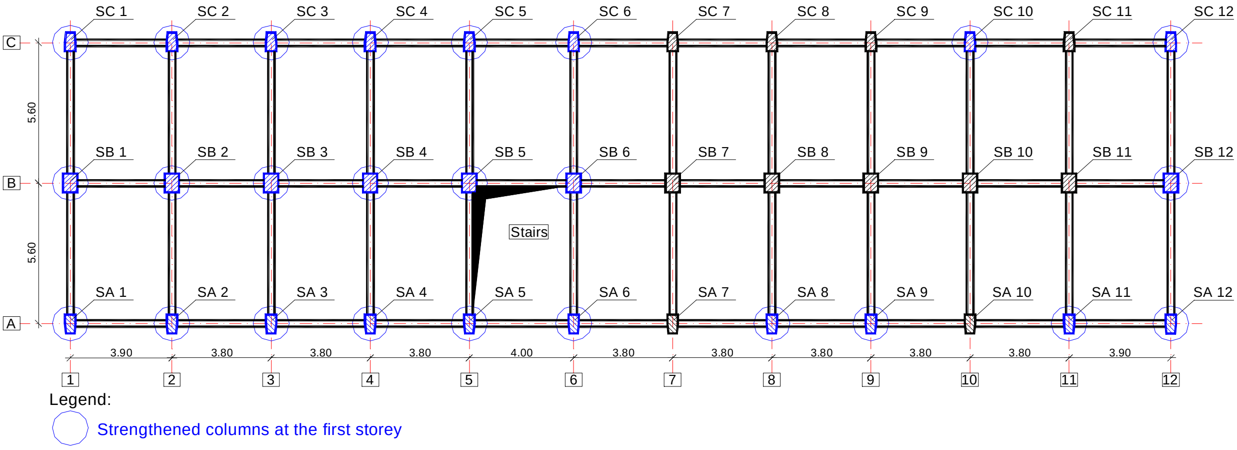


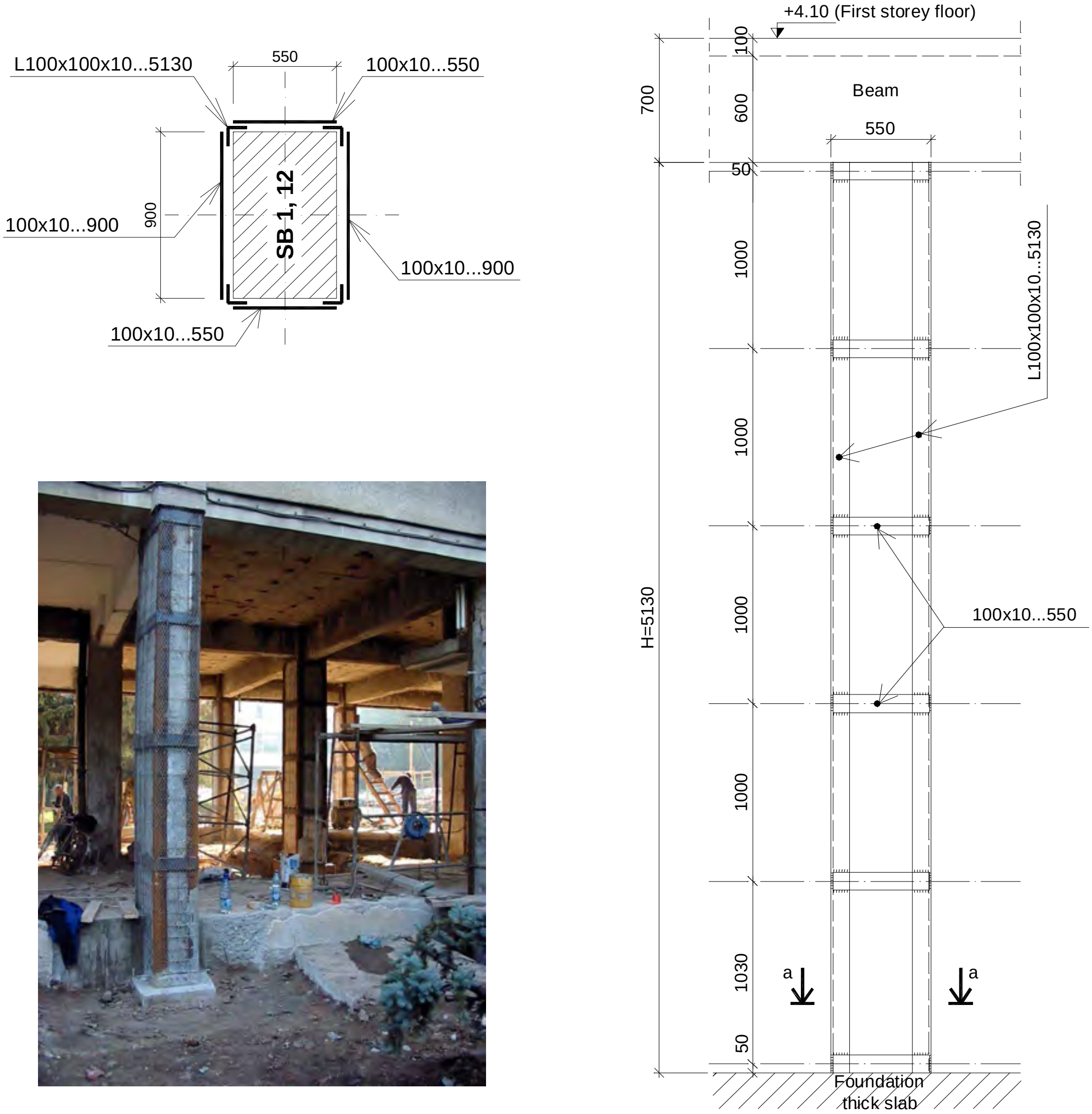
Figure 2.27. Framing plan – first storey

On examination of the building and from non-destructive measurements no important damages of the reinforced concrete structure were noticed. Some local damage due to incipient reinforcement corrosion was detected at the columns of the first storey.

The analysis of the structure has been performed at both combinations of actions: fundamental combinations and special combinations including seismic action at present-day level. From the analysis it was observed:

- weakness of reinforcement and insufficient anchorage of beam-positive reinforcement at the beam-column joint, especially in the longitudinal direction;
- the drift limitation conditions are not within the admissible limits at the first storey.

Rehabilitation solution (see Figure 2.28) consists in strengthening of the columns located at the first storey: 11 columns were consolidated in 1999 to prevent the local damages due to reinforcement corrosion; the next 25 columns shall be rehabilitated during this year for decreasing the lateral displacements (drift limitation conditions) and for a homogeneous columns stiffness at the first storey:



SA 1, 12; SC 1, 12

Figure 2.28. Rehabilitation solution of columns

TANKS SUPPORTING STRUCTURE

The assessed building was a reinforced concrete framed structure with two storeys supporting two tanks for raw material storage. The non-destructive testing was performed with the test hammer and pulse velocity measurement on the main elements. The concrete class given by the combined method was C8/10.

The main damages consisted of: concrete cover dislocated over large surfaces; high corrosion of stirrups and longitudinal reinforcement of columns (Figure 2.29) and beams.

In order to supply the columns corroded reinforcement, structural rehabilitation was performed using steel profiles (Figure 2.30). After rehabilitation the utility of the structure was changed for supporting new cooling equipment (Figure 2.31).

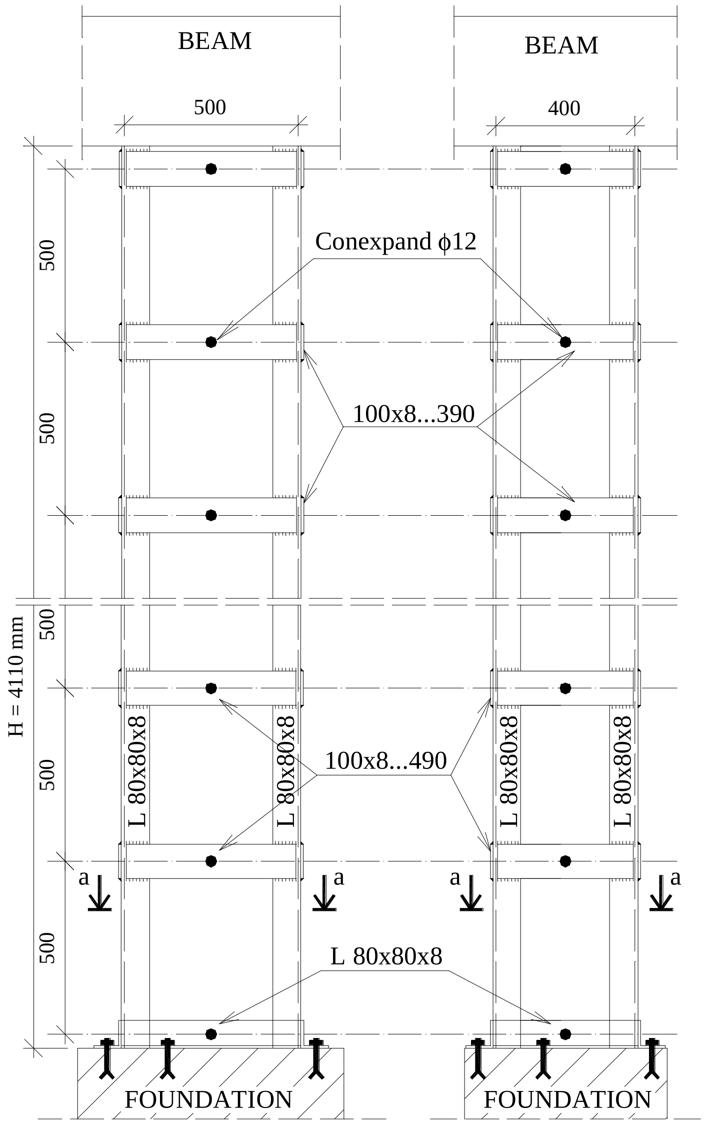


Figure 2.30. Rehabilitation solution for columns

Figure 2.29. Columns damages

Figure 2.31. Rehabilitated structure

OFFICE BUILDING

An office building, built 20 years ago, having an underground storey and five storeys was also assessed in order to construct another two extra-storeys. The vertical structural system is spatial reinforced concrete frame.

The assessment and the structural analysis at vertical and seismic actions emphasized the structural vulnerabilities: low concrete class in the columns; for the proposed situation, with two extra-storeys, the compression efforts in the columns were higher than the actual concrete strength.

The strengthening solutions for columns were designed according to the buildings damage, assessment and structural analysis, and chosen in order to obtain technical and economical advantages.

A general view of the office building and site aspects of columns strengthening on their entire height are presented:

Figure 2.32. Strengthening of columns for an office building

THE PALACE BUILDING

The "Palace" structure is a huge building (underground floor, ground floor - restaurant, 3 storeys – apartments and a timber roof), built before 1900's with a composite structure: masonry and reinforced concrete framed structure – Figure 2.33.

Figure 2.33. The "Palace" building

Figure 2.34. Longitudinal reinforced concrete frames

Initially it was an entire masonry structure, but later the ground floor was changed: some resistance brick walls were cut and two longitudinal reinforced concrete frames (Figure 2.34) were erected to sustain all the vertical loads. Due to this architectural operation the structure became more vulnerable at seismic actions: by the transversal direction main part of the ground floor became unstable at horizontal actions because the new reinforced concrete columns present hinge connection at both ends in masonry walls from the underground floor and first storey. Other vulnerabilities of the building consist of: overall lateral stiffness values along the two main axes are different; lack of seismic joints to divide building parts having different dynamic characteristics; lack of straps at each floor.

The building assessment emphasized some aspects: concrete quality is very variable in structural elements, having different classes (C8/10 - C16/20); some cracks in longitudinal beams; corrosion of the slab reinforcement; etc.

The results of the static and dynamic analysis for columns (performed by SAP2000N software) are presented in Table 2.5 for the non-rehabilitated structure. Two assumptions were taken into account: vertical structural members are not coupled by floor elements over the ground floor; vertical members are coupled by the floor elements (rigid floor).

Table 2.5. Static and dynamic analysis results

<table>
<tr><th>Type of structure</th><th>Analysis assumptions</th><th colspan="2">Efforts</th><th>Transversal direction</th><th>Longitudinal direction</th></tr>
<tr><td rowspan="5">Non-rehabilitated structure</td><td rowspan="3">Not coupled structure</td><td rowspan="2">M [kNm]</td><td>M_{nec}</td><td>700</td><td>700</td></tr>
<tr><td>M_{cap}</td><td>186</td><td>186</td></tr>
<tr><td colspan="2">$R = M_{cap}/M_{nec}$</td><td>0,27</td><td>0,27</td></tr>
<tr><td rowspan="2">Coupled structure</td><td colspan="2">M_{nec} [kNm]</td><td>127</td><td>700</td></tr>
<tr><td colspan="2">$R = M_{cap}/M_{nec}$</td><td>1,46</td><td>0,27</td></tr>
<tr><td rowspan="5">Rehabilitated structure</td><td rowspan="3">Not coupled structure</td><td rowspan="2">M [kNm]</td><td>M_{nec}</td><td>1048</td><td>1048</td></tr>
<tr><td>M_{cap}</td><td>1175</td><td>1175</td></tr>
<tr><td colspan="2">$R = M_{cap}/M_{nec}$</td><td>1,12</td><td>1,12</td></tr>
<tr><td rowspan="2">Coupled structure</td><td colspan="2">M_{nec} [kNm]</td><td>473</td><td>1048</td></tr>
<tr><td colspan="2">$R = M_{cap}/M_{nec}$</td><td>2,48</td><td>1,12</td></tr>
</table>

From data presented in the table a very important conclusion could be drawn: the ratios **R** between the actual values of ultimate bending moment *(M_{cap})* and the necessary bending moment *(M_{nec})*, given by the present-day seismic action level, were very low for columns, *0,27*. That meant that **the building was characterized by a high risk of collapse at seismic actions**. It resulted the necessity of structural rehabilitation.

In accordance to the structural analysis, the strengthening of the ground floor was chosen in order to obtain technical and economical advantages: safe behaviour at seismic actions; slight change of the overall structural stiffness; easy strengthening technology and short period of refurbishment (December 2004 - June 2005).

The strengthening was made on the following structural elements (Figures 2.35 and 2.36):

- strengthening by reinforced concrete coating (70 mm on each side) of masonry walls from the underground floor of the building;
- new reinforced concrete floor with embedded steel profiles (HEB 220) in two directions, which stands as beams for the new structure;

- strengthening of half from the existing columns (0,60x0,60 m coated by reinforced concrete to become 0,90x0,90 m) and erecting of new transversal reinforced concrete beams in order to create new transversal frames;
- strengthening by reinforced concrete coating of existing longitudinal beams;
- rehabilitation of some structural elements having corroded reinforcement as well as of some brick walls.

Figure 2.35. Strengthening of slab and columns

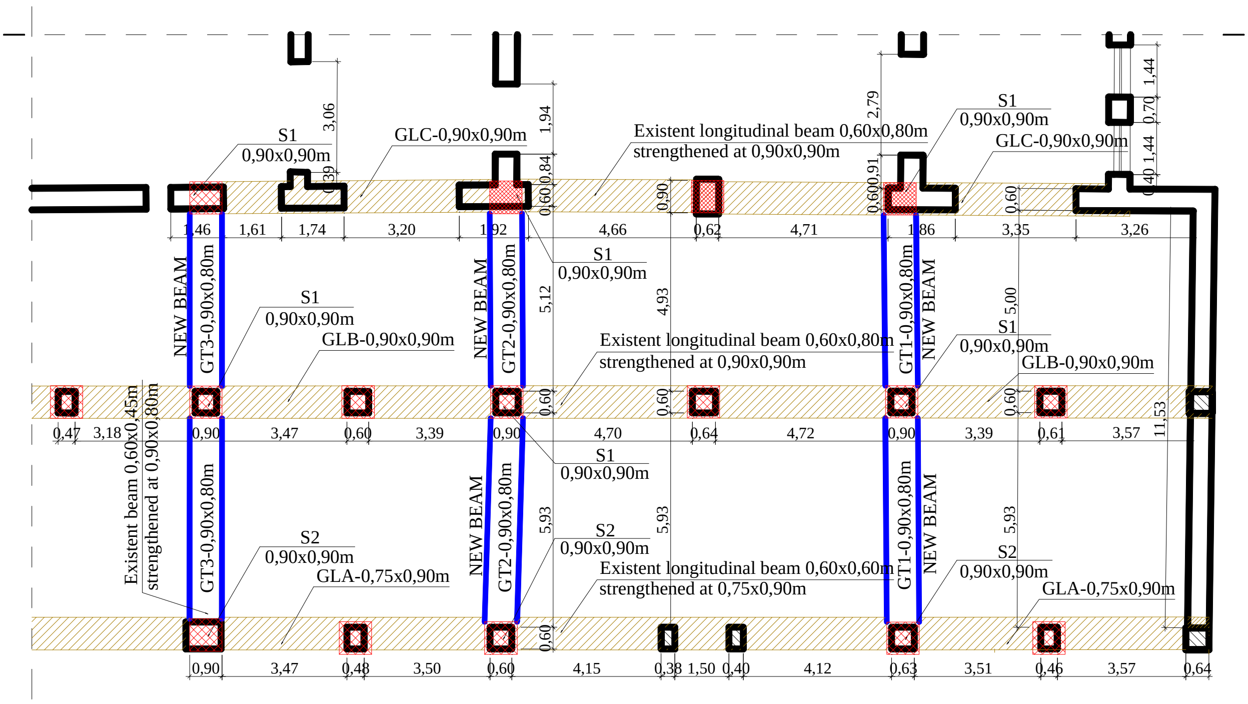


Figure 2.36. Ground floor rehabilitation

APARTMENT HOUSE AFFECTED BY A GAS EXPLOSION

Due to a gas explosion into a block of flats (Figure 2.37) in Timisoara severe damage was produced (Figure 2.38):

Figure 2.37. Block of flats

Figure 2.38. Fracture of reinforced concrete walls

This building is 5 stories with 100 bed-sitters and the explosion was located in the second flat at the 4th storey. The cause of explosion was the leaking and accumulation of propane gas from a gas recipient into the flat. The initial explosion totally destroyed the concrete walls, reinforced concrete floors, windows and door of the flat and caused serious damage of the others structural members.

The building was built in 1976. The building in plane dimensions are 43,55x14,75 m and it has a sub-basement and fire storey of 2,72 m high. The structural composition of the building consists of: vertical structure built of longitudinal and transversal reinforced (over the borders) concrete walls of 0,30 m width for concrete facades and 0,15 m of the interior walls; horizontal structure of 0,14 m width precast reinforced concrete floors.

Expected effects of explosions on buildings, personnel, etc. have been analysed by different authors [77-78] and their results used as follows.

The structural analysis takes into account three aspects: the non-destructive testing of main structural members, analysis of the elements subjected to blast pressure and analysis of the state of stresses in the hollow structure.

The non-destructive testing was performed with the test hammer and pulse velocity measurement on the main elements surrounding the directly affected flat. The concrete class, given by the combined method, was C12/15 and C16/20.

The structural elements, which have been affected directly by blast pressure and were seriously damaged, are analysed. These members are: concrete walls, 4,85x2,75x0,15 m which are considered as interior panels without reinforcement in the middle of the wall; reinforced concrete floors of dimensions 4,60x3,44x0,14 m and with all four edges discontinuous; window glass of 1,00x0,50x0,0015 m with all four edges discontinuous. The results concerning the relevant mechanical parameters of the structural members are presented in Table 2.6:

Table 2.6. Relevant values of structural members subjected to blast pressure

Parameter	Structural member		
	Concrete walls	Reinforced concrete bottom floor	Window glass
Cracking bending moment m_{cr} [kNm]	7,13	5,17	–
Maximum bending moment m_{max} [kNm]	7,13	11,88	2,25
Maximum static load P_{max} [kN/m^2]	21,00	12,80	0,81
Static deflection a_{st} [mm]	0,15	1,23	1,00
Natural period T [ms]	24,60	70,00	63,50
Ratio pulse time on natural period t_d/T	1,00	0,36	0,40
Ductility factor ρ	1,90	4,67	1,00
Peak load P_{so} [kN/m^2]	30,00	47,50	1,13
Dynamic coefficient $\mu = P_{so}/P_{max}$	1,43	3,70	1,39

The analysis of the hollow structure has been performed according to the Romanian code for seismic action at present-day levels in three stages: initial stage before the explosion; after explosion; finally after strengthening of the affected members. The dynamic analysis was performed by using Finite Element Method software. Some results of the analysis are presented in Table 2.7. The data refer to more significant values of the parameters (axial force *N*, shear force *T* and bending moment *M*) in the elements affected by the explosion. The ratio *R* between values of efforts after explosion (as well as after strengthening) and the initial values is also given in Table 2.7:

Table 2.7. Efforts in some structural elements

Element	Storey	Initial stage	After explosion		After strengthening	
		N [kN] *T* [kN] *M* [kNm]	*N* [kN] *T* [kN] *M* [kNm]	*R*	*N* [kN] *T* [kN] *M* [kNm]	*R*
Transverse walls	1	740	578	0,78	896	1,21
		106	111	1,05	160	1,51
		848	694	0,82	1189	1,40
	2	576	414	0,72	693	1,20
		106	124	1,17	151	1,42
		572	361	0,63	781	1,37
	3	452	290	0,64	550	1,22
		93	132	1,42	132	1,42
		444	180	0,40	609	1,37
	4	329	0	0	407	1,24
		74	0	0	101	1,36
		321	0	0	433	1,35
	5	164	0	0	203	1,24
		45	0	0	59	1,31
		122	0	0	160	1,31
Transverse coupling beams	4	0	0	–	0	–
		3,85	3,43	0,89	8,80	2,29
		2,20	1,86	0,85	4,81	2,19
	5	0	0	–	0	–
		1,94	1,42	0,73	4,05	2,09
		2,45	2,03	0,83	5,49	2,24

The replacements and strengthening have been made on the following structural elements:

- new reinforced concrete floors, with the same geometry and reinforcement characteristics as for the existing members, at levels 3, 4 and 5, total 5 elements;

- new reinforced concrete walls at levels 4 an 5 which represent 4 transversal walls (with the same width as of the corresponding walls with new shirts) and 4 longitudinal – lateral walls;
- local strengthening with reinforced concrete shirt (50 mm on each side), columns (0,60x0,20 m in the gangway and 0,30x0,20 m at facade) and longitudinal beams at all levels (0,20x0,30 m), vertical elements from ground floor to the roof, incorporating the new walls, on the one part of the gangway (Figure 2.39);
- rehabilitation of the cracked elements with carbon fibre reinforced polymer (CFRP) wrap for 10 transversal walls and 12 floors (Figure 2.40).

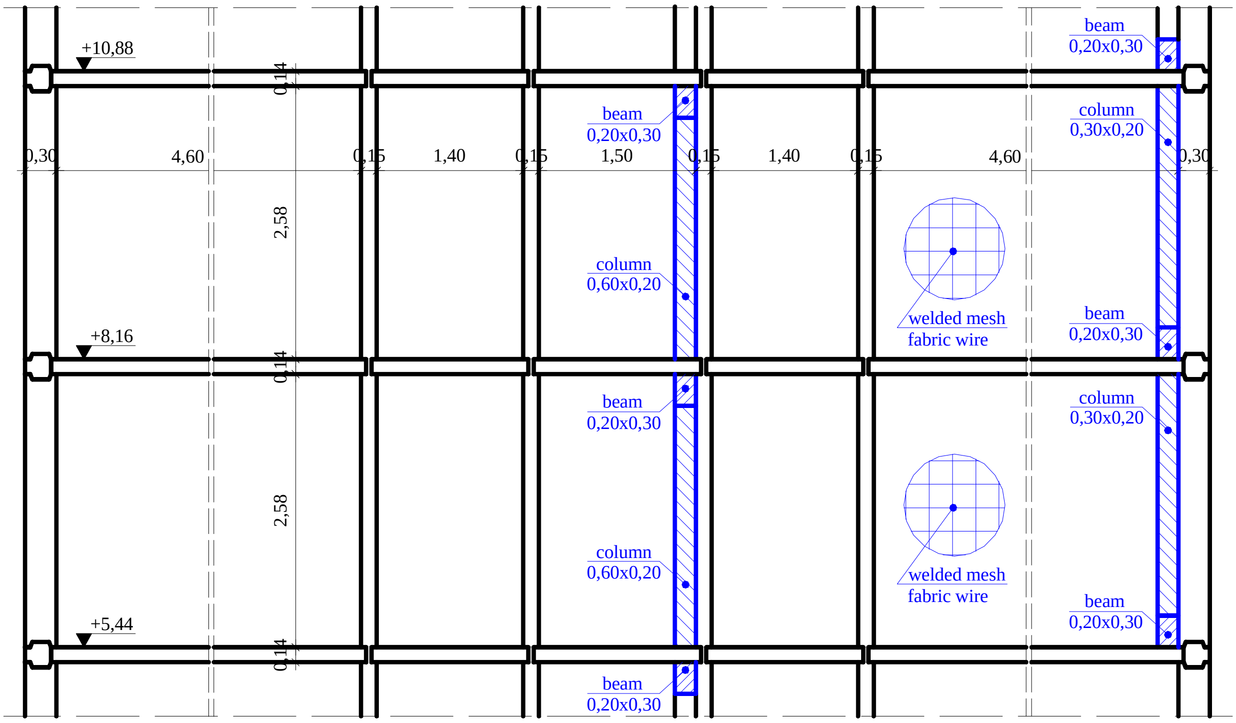


Figure 2.39. Reinforced concrete vertical rehabilitation solutions

Figure 2.40. CFRP structural rehabilitation of walls and floors

REINFORCED CONCRETE SILOS

The assessment and rehabilitation solutions for a group of silos owned by the SAB Miller Brewery Company “Timisoreana” are presented (Figure 2.41):

Figure 2.41. Reinforced concrete silos

The silos were built 40 years ago and stand 28 m high and 7,30 m in diameter. Initial silos inspection (1999) revealed large zones of circular cells with concrete cover dislocated and corrosion of circumferential steel reinforcement. Recent silos inspection and assessment (2004) emphasized other vulnerable parts: infrastructure and charging platform (gallery).

The silos infrastructure consists of foundation raft, discharge funnel and its supporting columns and beams.

The main damages are due to water infiltration and high humidity inside of each cell bottom part, which caused important corrosion of the columns and beams steel reinforcement (Figure 2.42).

The charging platform is composed of the following structural members: a composite steel-concrete slab as the floor over the cells; a frame-wall coupled system as vertical structure; a precast reinforced concrete floor with main and secondary beams as the building flat roof. The main damage of the charging platform structural members is located in reinforced concrete walls and consists of wide open cracks at the windows bottom level; the cracks width is about 1…2 mm and the length of 1,00 m. Such cracks were produced by the temperature action on the frame-wall coupled system and due to the higher stiffness of walls.

The silos cells are in similar stage as assessed in 1999 but presenting larger zones with dislocated concrete cover and higher corrosion of circumferential steel reinforcement.

The rehabilitation of the silos three parts was designed according to the building damage assessment and structural analysis and chosen in order to obtain technical and economical advantages.

Figure 2.42. Reinforcement corrosion of discharge funnel

The strengthening solution for the **silos circular cells** was the use of CFRP strips as near surface mounted reinforcement. The SikaWRAP HEX-230C strips were placed on the most stressed zone (+3,20…+13,20 m), outside of cells (Figure 2.43). This solution seemed to be more advantageous than the near surface mounted reinforcement of CFRP rods:

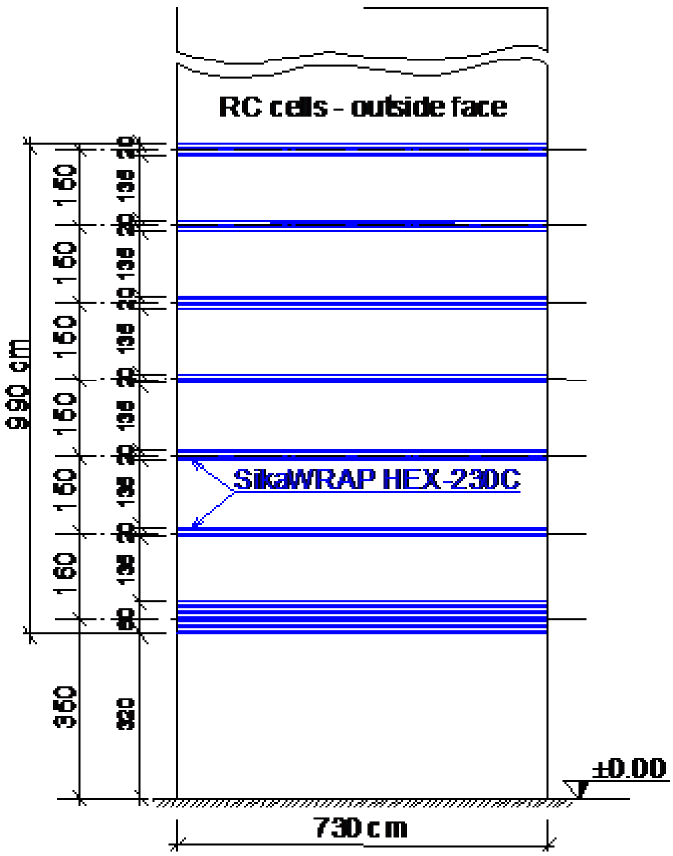


Figure 2.43. CFRP strengthening of cells walls

The **charging platform cracked walls** are to be strengthened by using carbon fibre reinforced polymers (CFRP) as illustrated in the Figure 2.44. The solution has the advantage of easy technology, short period of refurbishment and small rehabilitation cost. On the other hand, the buckling phenomenon of the SikaCARBODUR strips is not possible to show up.

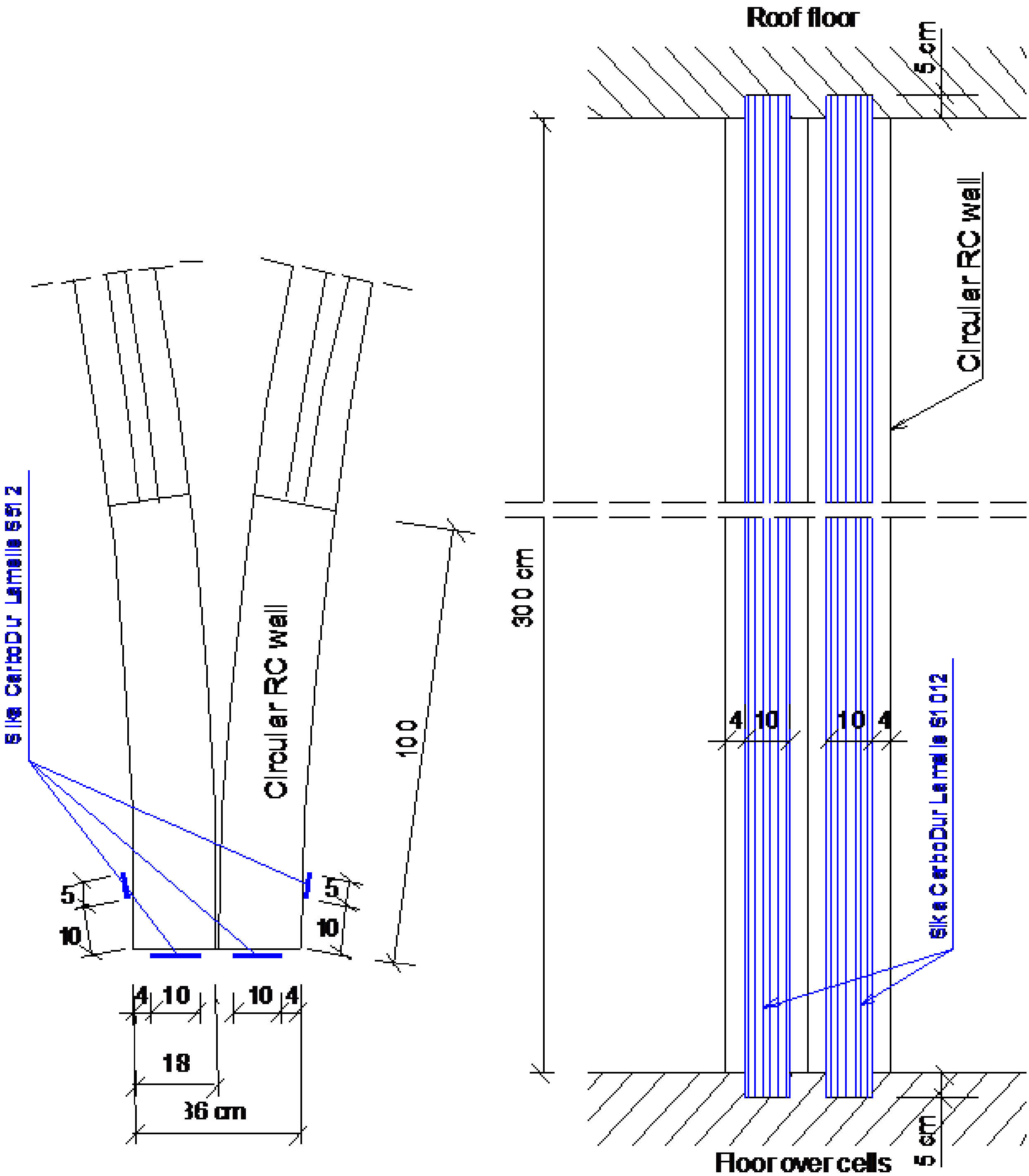


Figure 2.44. CFRP strengthening of gallery walls

The strengthening of **supporting columns for the discharge funnel** consists of steel profiles (Figure 2.45). This solution has a smaller cost than CFRP materials. On the other hand, steel profiles have a better buckling behaviour than CFRP strips.

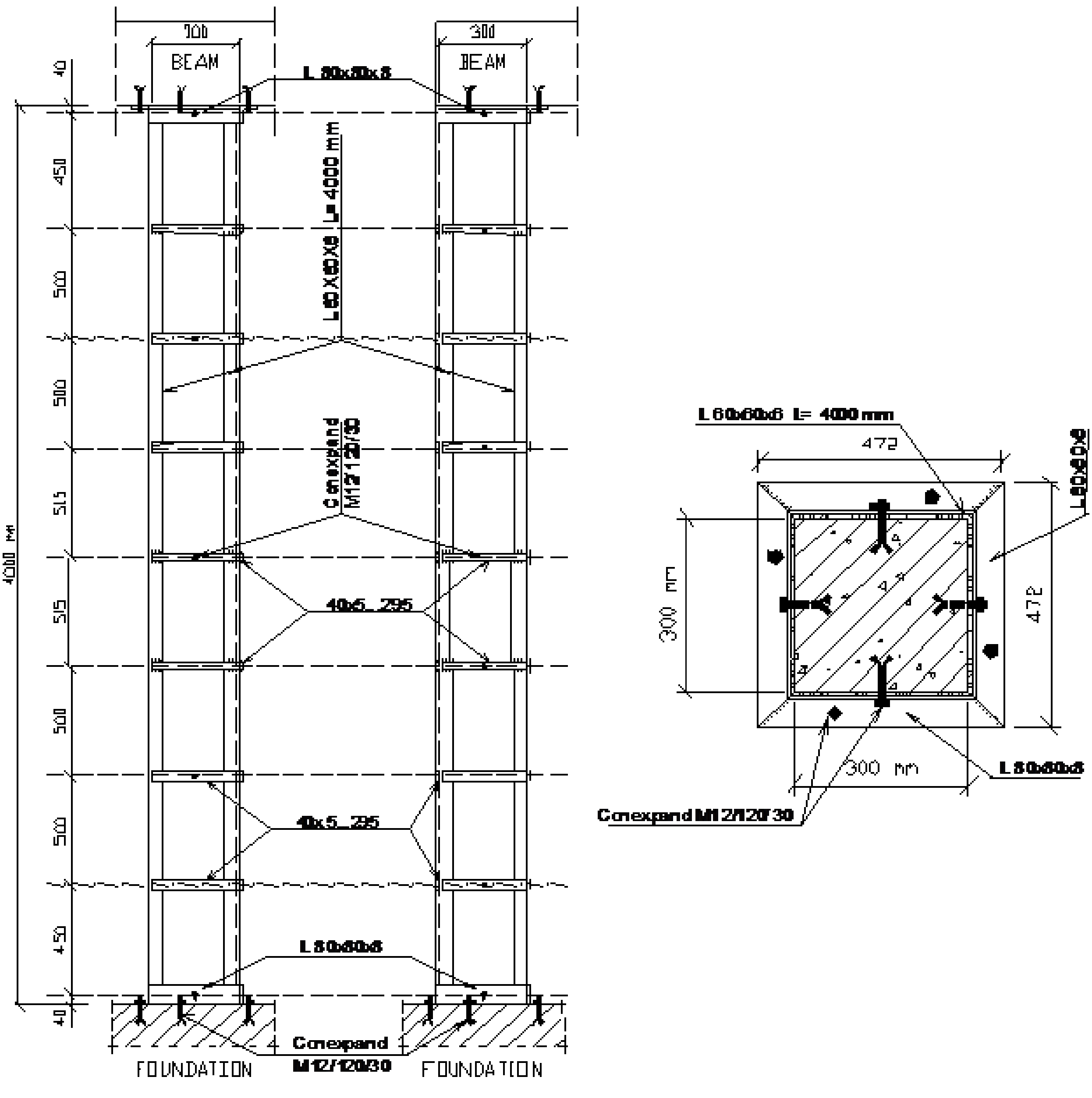


Figure 2.45. Strengthening of supporting columns for the discharge funnel

STRENGTHENING OF AN INDUSTRIAL BUILDING

The analysed strengthening solutions are applied on an existing four storeys building, GUBAN factory – Figure 2.46, erected in 1940 and located in a seismic zone – Timisoara, Romania. The structure consists of: vertical inner reinforced concrete frame and brick masonry resistance walls on outer perimeter; horizontal reinforced concrete floors with main and secondary beams. The owner requirement was to build two more storeys.

The factory has been assessed and rehabilitated.

Figure 2.46. Industrial building

The main problems comprised local damage of some structural elements and weakness of reinforcement of columns and beams at present-day magnitude of seismic action. Local damages were observed and assessed at slabs, main girders, secondary beams (Figure 2.47) and columns (Figure 2.48). The damage consisted of: concrete cover dislocated over a large surface; corrosion of many stirrups; deep corrosion of main reinforcement.

Figure 2.47. Damage of beams

Figure 2.48. Damage of columns

Weakness of reinforcement was deduced from the structural analysis. The initial analysis, done in 1940, was performed according to Romanian norms, under which seismic design was inadequate, owing to weakness in the structural system. On the other hand, weakness of shear reinforcement was deduced; inclined cracks were noticed at some main girders.

The rehabilitation of the reinforced concrete structure, performed in 2008, was adopted for both types of damages. The strengthening consisted in jacketing with reinforced concrete of columns and foundations:

Figure 2.49. Reinforced concrete jacketing of columns and foundations

For columns the previous experimental studied solution was adopted by using chemical anchored connectors:

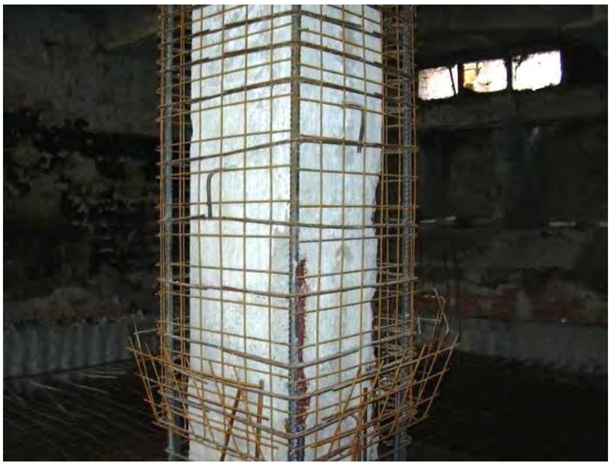


Figure 2.50. Using of chemical anchored connectors at columns

The strengthening solution adopted for beams and slabs was based on carbon fibre polymer composites (CFRP) as it is illustrated in Figure 2.51:

- longitudinal CFRP lamellas and transversal CFRP wrap for beams;
- CFRP wrap for slabs.

Figure 2.51. CFRP strengthening solution for beams

STRENGTHENING OF FRAME STRUCTURE AT THE TIMISOREANA BREWERY [79]

The brewery – Figure 2.52, a reinforced concrete framed structure with one section at five storeys and a tower of nine storeys, has been assessed and strengthened in two steps, 1999 and 2003. The brewery and the tower were built in 1961 and the extension in 1971.

Figure 2.52. Timisoreana Brewery

The industrial building vertical structure is a spatial frame as detailed in Figures 2.53-2.55. The foundation system consists of isolated reinforced concrete foundations under columns. The reinforced concrete monolithic floors are made of secondary and main beams and a one way reinforced slab.

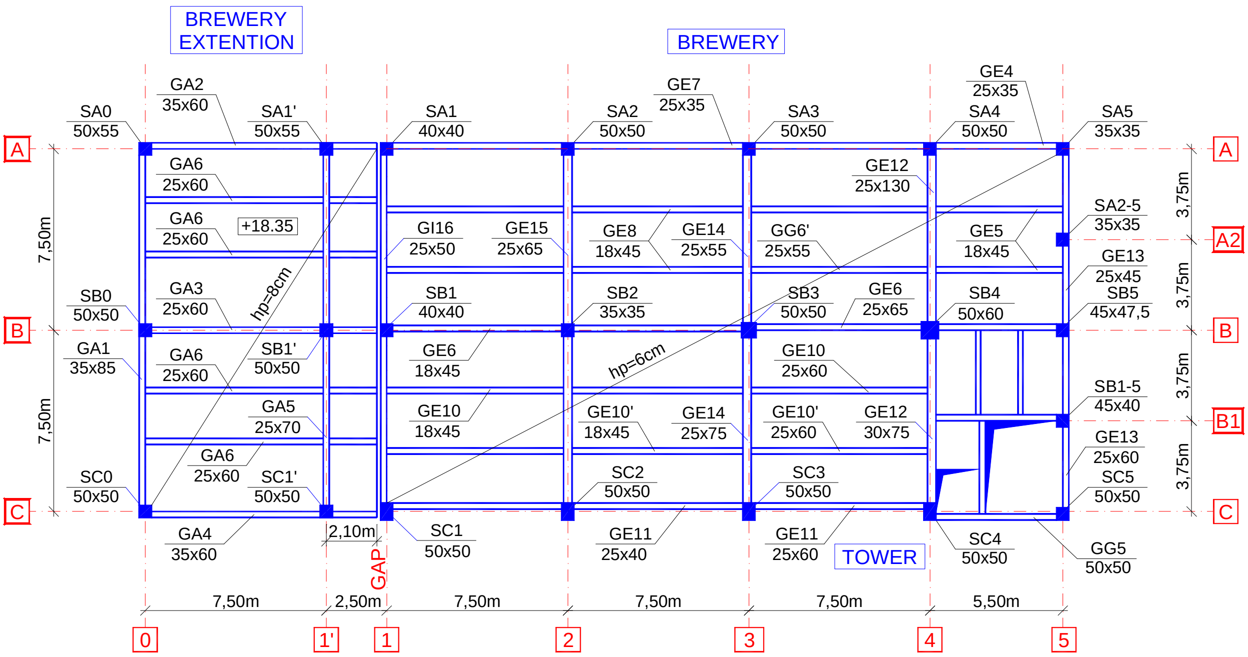


Figure 2.53. Framing plan, level +18.40 m

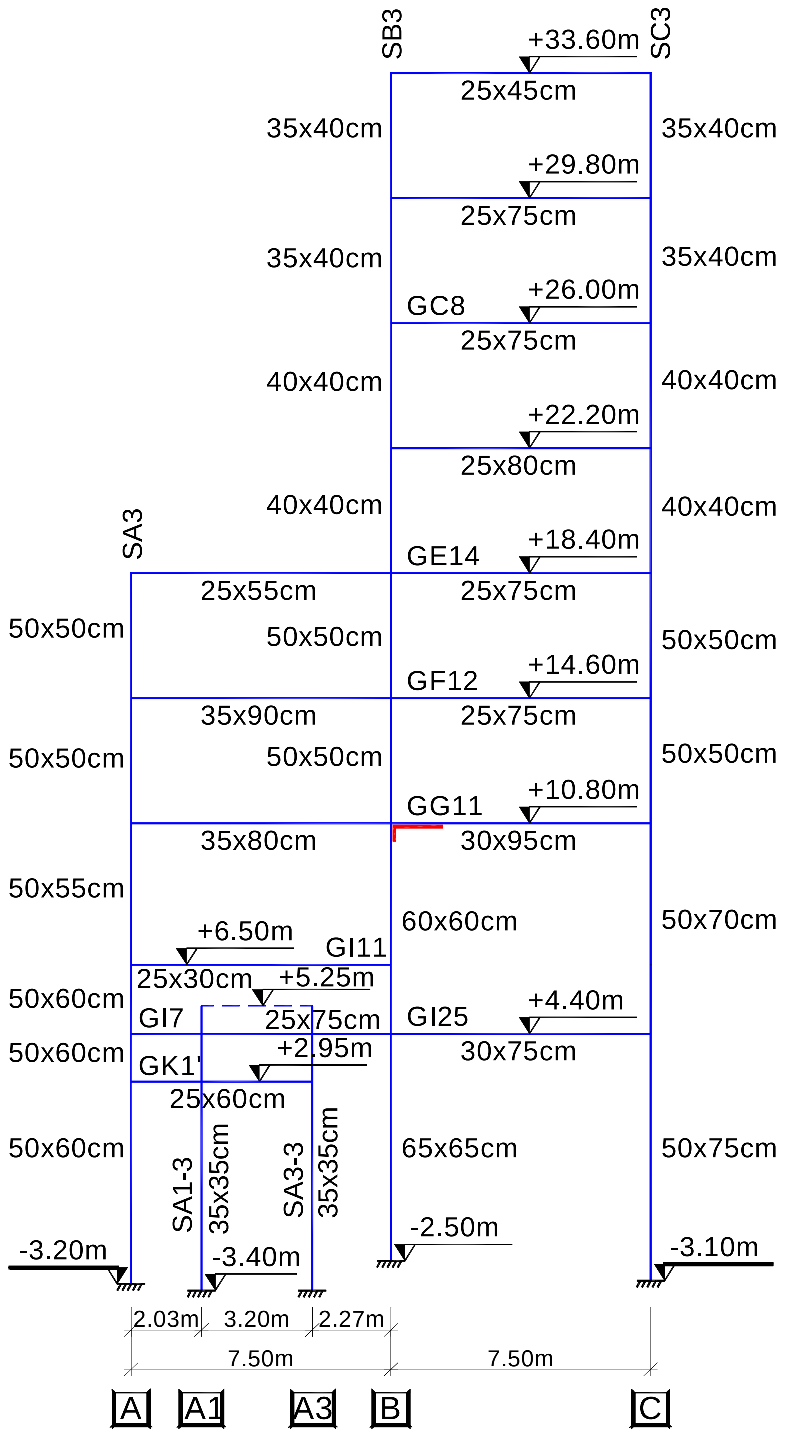


Figure 2.54. Transversal frame

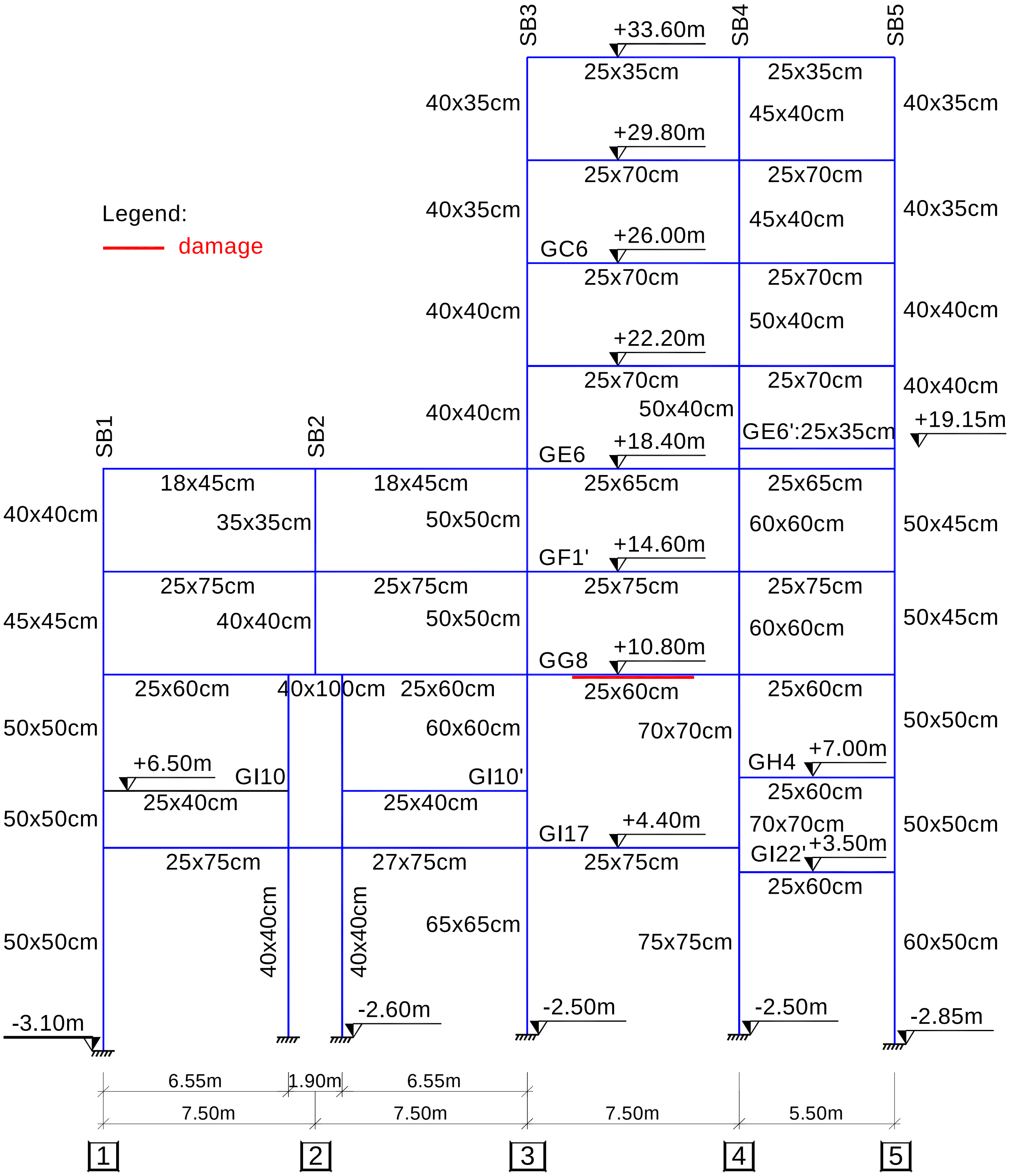


Figure 2.55. Longitudinal frame

SYMPTOMS THAT LED TO NEED OF STRENGTHENING AND ASSESSMENT OF IN-SITU CONDITIONS

Symptoms that Led to Need of Strengthening

The main problems comprised local damage of some structural elements and inadequate reinforcement of columns and beams at seismic actions. Local damages were noticed and assessed at slabs, main girders, secondary beams and columns. The damage consisted of: concrete carbonation; concrete cover spalled over a large surface; complete corrosion of many stirrups and deep corrosion of main reinforcement; some broken reinforcement. The damaged areas were located at the second floor (level +10,80 m), in the middle of the span for secondary beams, on potentially plastic hinge regions of the main girder and columns (Figures 2.56-2.59).

Figure 2.56. Damage of secondary beams

Figure 2.57. Damage of main beams

Figure 2.58. Damage of columns

Figure 2.59. Inclined cracks at main beams

Such damage was caused by the action of chloride ions (Cl^-) from salt solution, which was stored on the second floor as well as of CO_2, RH ≈ 80 % and temperatures over 40 ^oC.

At some main beams dangerous inclined cracks were also detected at the secondary beam to main beam connection due inadequate transversal reinforcement at shear force.

Inadequate longitudinal reinforcement was deduced from the structural analysis. The initial analysis, done in 1960, was performed according to Romanian codes [80], at low seismic design actions, owing to weakness in the structural system at present-day high seismic actions.

Assessment of In-situ Conditions

On the main structural elements non-destructive tests were performed: rebound test method as well as pulse velocity measurements. The average values are presented in Table 2.8. The mean compressive strength (f_{cm}) of the investigated elements was obtained by using the combined method: pulse velocity (v) - rebound index (n). The concrete class given by the combined method was: C8/10 – C16/20 at columns; C12/15 – C25/30 at main girders; C8/10 at secondary beams.

Table 2.8. Non-destructive analysis of concrete class

Element	n	v [m/s]	f_{cm} [N/mm^2]	Concrete class
Column SC2	38,4	3644	20,60	C12/15
Column SB2	45,1	3717	28,20	C16/20
Column SC1	39,7	3829	26,30	C16/20
Column SA2	47,8	3077	17,70	C8/10
Column SB3	41,9	3774	27,60	C16/20
Main beam GE14	36,9	3194	19,25	C12/15
Main beam GE15	48,4	3940	36,10	C25/30
Secondary beam GE10	41,1	3280	18,40	C8/10
Secondary beam GE8	36,2	3256	13,60	C8/10
Slab	34,2	2373	6,20	< C4/5

The results of the non-destructive analysis emphasized some important conclusions: the concrete class of columns is good for some of the elements; the concrete class in many beams and slabs is below the minimum necessary for reinforced concrete floors.

The specific service conditions of the structural elements (T = 40-60 ^{o}C, RH = 70–80 %, chloride ions present) during 42 years lead to some important damages.

Concrete carbonation and/or chloride ions penetration was checked by both procedures: theoretical analysis and experimental test. The theoretical values of the concrete carbonation/ion penetration were calculated according to [81-82] and are presented in Table 2.9. The experimental measurements were made by pH-test and the results are, also, illustrated in Table 2.9.

Table 2.9. Carbonation depth [mm]

Element	Theoretical	Experimental
Columns	33	5*-10*
Secondary beams	50	20*
Main beams	26	20-25
Note: * experimental measurements wereinfluenced by the periodical sanitation (with mortars) of the elements; new mortar layers had higher pH.		

The carbonation of cover concrete created the conditions for the reinforcement corrosion: 30–60 % of main reinforcement steel cross section was corroded at some elements like columns and beams (see Table 2.10). Reinforcement steel characteristics used in 1960 for the existing structural members are: Romanian ribbed bars PC52 (f_{yk} = 350 N/mm^2; f_{yd} = 300 N/mm^2) for longitudinal reinforcement and Romanian plain bars OB37 (f_{yk} = 245 N/mm^2; f_{yd} = 210 N/mm^2) for stirrups [83].

Table 2.10. Reinforcement corrosion of main bars in some structural elements

Storey	Element	Reinforcement characteristics					
		initial		measured		$\Delta\Phi = \frac{\Phi_0 - \Phi_r}{\Phi_0} \cdot 100$	$\Delta A = \frac{A_0 - A_r}{A_0} \cdot 100$
		Φ_0 [mm]	A_0 [mm^2]	Φ_r [mm]	A_r [mm^2]	[%]	[%]
III	Column	25	491	17	227	32	54
	Column	25	491	20	314	20	36
V	Column	25	491	20,5	330	18	33
	Column	25	491	21	346	16	30
	Main girder	22	380	20	314	9	17
	Main girder	22	380	19	283	14	26

ADOPTED SOLUTIONS FOR STRENGTHENING

The assessment performed in 1999 showed up local damages at slabs, main girders, secondary beams and columns. As previously presented, the damage consisted of: concrete carbonation; concrete cover spalled over a large surface; complete corrosion of many stirrups and deep corrosion of main reinforcement; some broken reinforcement. Also, inadequate longitudinal reinforcement was deduced from the structural analysis. The initial design, done in 1960, was performed according to Romanian codes in effect at that time with provisions of low seismic actions, owing to structural system weakness at present-day high seismic actions. The necessary rehabilitation of the reinforced concrete structure was adopted and performed for all types of damages. The main girders and secondary beams were strengthened by jacketing with reinforced concrete. The columns were strengthened for both local damage and inadequate reinforcement, by jacketing with reinforced concrete over two storeys. The existing foundation was jacketed over and around with reinforced concrete for secure fixing of the column new main reinforcement.

In 2003, due to continuous operation and subsequent damage of the structure, a new assessment was required. It was found that some beams and one column were characterised by inadequate longitudinal reinforcement (in the column) and shear reinforcement as well as corrosion of many stirrups at beams. The necessary strengthening was performed at beams and column characterised by inadequate longitudinal reinforcement (the column) and shear reinforcement as well as corrosion of many stirrups (five beams). The strengthening solution adopted was based on carbon fibre reinforced polymer CFRP composites.

STRUCTURAL ANALYSIS BEFORE AND AFTER REPAIR

According to the Romanian codes of actions [84], the structural analysis was performed in the fundamental combination of loads and in the special combination of loads by taking the seismic action into account at present-day magnitude [8]. The load characteristics are given in Table 2.11:

Table 2.11. Load cases and combinations

Load case		Dead load	Imposed load	Live load	Snow load	Wind load	Seismic load
Characteristic load		5,0 kN/m^2	1,0…10,0 kN/m^2	2,0 kN/m^2	0,7 kN/m^2	0,7 kN/m^2	a_g = 0,16g β_{max} = 2,5 q = 5,0
Load factor	Persistent and transient design situation	1,2	1,2	1,3	0,7	1,2	0,0
	Seismic design situation	1,0	1,0	0,8	0,3	0,0	1,0

According to the Romanian Code for seismic design P100-92 [8] as well as to the other norms, the design of structures to resist earthquake is based on the following design procedures and calculation methods:

- Common design procedures based on the following calculation methods: linear static with conventional forces distributed as inertia forces for linear static response; linear dynamic with accelerograms for modelling of seismic actions;

- Design procedure based on consideration of post-elastic deformation of structures with: non-linear static analysis and conventional forces distributed as inertia forces for seismic response; non-linear dynamic method with accelerograms for modelling of seismic action.

The assessment of the existing structures to the seismic action is estimated according to the Romanian Code by calculus of the earthquake capacity ratio R:

$$R = \frac{S_{cap}}{S_{nec}}$$

where: S_{cap} - seismic shear force capacity (seismic base shear force);
S_{nec} - conventional seismic load (seismic base shear force) calculated according to the Romanian Code P100-92 for seismic design action.

For the assessment of existing structures the previous general equation may be written for different sectional efforts and applied for individual structural members, as for instance:

$$R = \frac{M_{cap}}{M_{nec}} = \frac{M_{Rd}}{M_{Ed}}$$

where: M_{cap} or M_{Rd} - resistance bending moment;
M_{nec} or M_{Ed} - design bending moment calculated for the present-day level of actions.

The equivalence between the Romanian earthquake capacity ratio and the more common safety approach according to EN 1990 [84] is presented in Table 2.12:

Table 2.12. Safety factors of new and existing buildings

<table>
<tr><th colspan="3">Building class of importance</th><th>Earthquake capacity ratio R_{min}</th><th>Global safety coefficient C_0</th><th>Reliability index β</th><th>Failure probability P_f</th></tr>
<tr><td colspan="3">New buildings</td><td>1,00</td><td>2,250</td><td>4,75</td><td>$10^{-6} \ldots 10^{-7}$</td></tr>
<tr><td rowspan="4">Existing buildings of class:</td><td>I</td><td>Buildings of vital social importance</td><td>0,70</td><td>1,575</td><td>3,09</td><td>10^{-3}</td></tr>
<tr><td>II</td><td>Very important buildings</td><td>0,60</td><td>1,350</td><td>2,00</td><td>$2 \cdot 10^{-2}$</td></tr>
<tr><td>III</td><td>Normal importance buildings</td><td>0,50</td><td>1,125</td><td>1,28</td><td>10^{-1}</td></tr>
<tr><td>IV</td><td>Reduced importance buildings</td><td>0,50</td><td>1,125</td><td>1,28</td><td>10^{-1}</td></tr>
<tr><td colspan="7">Note: Values are given for normal distribution of actions and strengths and variation coefficient $C_v^r = C_v^a = 10\%$</td></tr>
</table>

The Timisoreana Brewery, an existing industrial building of normal importance (class III), has to satisfy the earthquake capacity ratio R_{min} = 0,5 corresponding to the failure probability $P_f = 10^{-1}$.

Advanced Structural Analysis

The authors used for design the proper procedures based on consideration of post-elastic deformation with non-linear analysis. These procedures were used for analysis and redesign of existing structures in seismic regions [85-86]. For the damage control of structural members at seismic design the authors proposed and used the stiffness modification procedure.

The stiffness modification procedure [85-86] is based on the influence of stiffness degree calculated as function of materials characteristics: elasticity modulus (E_s, E_c) and area (A_s, A_c) of reinforcing steel and concrete. For instance, according to the Romanian design code for reinforced concrete structures [83], at bending with / without axial force the stiffness is given by the formula:

$$K = \frac{E_s \cdot A_s \cdot \beta \cdot d^2}{1 \mp \frac{\xi - \bar{x}_s}{\bar{e}_0}}$$

where: E_s - elasticity modulus of reinforcing steel;
A_s - area of tension reinforcement;
d - effective depth of reinforced concrete cross section;
$\bar{e}_0 = e/d$ relative eccentricity of axial force N; $\bar{e}_0 = \infty$ for pure bending;
$\bar{x}_s = x_s/d$ where: x_s is the distance between reinforcement area A_s and centroid of the concrete cross section;
$\beta = \zeta(1-\xi)/\psi$ where: $\xi = x/d$ and x is the depth of neutral axis;
$\zeta = (d - x/2)/d = 1 - \xi/2$;
ψ given in Table 2.13, see [83]:

Table 2.13. Values of ψ, [83]

ν ratio between long term action and total action	reinforcement percentage [%]		
	0,2 – 0,5	0,5 – 0,8	> 0,8
$\nu \leq 0,5$	0,8	0,9	1,0
$\nu > 0,5$	0,9	1,0	1,0

Finite element method FEM analysis is used and there is possible to assign different values of stiffness K for each element. The procedure advantages arise from the opportunity to change the value of K at any time of reinforced concrete structure utilisation e.g. after a serious degradation of one or several structural members.

Structural Analysis Carried Out Before Repair

Inadequate longitudinal reinforcement was deduced from the structural analysis. The initial design, done in 1960, was performed according to Romanian codes, under which the buildings seismic design load had very low magnitude, owing to weakness in the structural system.

The actual structural analysis and assessment was performed at present-day level seismic action by finite element method FEM on the spatial structure presented:

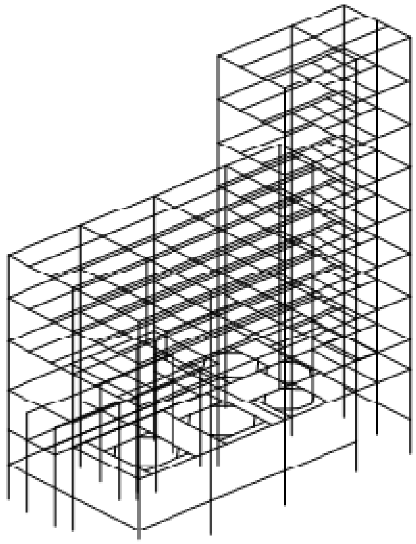


Figure 2.60. Spatial structure analysed by FEM

In order to quantify the influence of structural damage in structural analysis, the stiffness modification procedure was used. Due to reinforcement corrosion of transversal beam GG11 and longitudinal beam GG8 reduced values of stiffness *K*, as given by previous equations, were taken into account for the structural analysis.

The present structural analysis results quantified by the earthquake capacity ratio $R = M_{Rd} / M_{Ed}$ for the damaged structure are presented in Table 2.14:

Table 2.14. Analysis results for columns

Element	Transversal seismic action				Longitudinal seismic action			
	N_{Ed} [kN]	M_{Ed} [kNm]	M_{Rd} [kNm]	$R = \frac{M_{Rd}}{M_{Ed}}$	N_{Ed} [kN]	M_{Ed} [kNm]	M_{Rd} [kNm]	$R = \frac{M_{Rd}}{M_{Ed}}$
SA2 ground storey	1696	145	285	1,96	1460	136	209	1,53
SA3 ground storey	1439	218	332	1,52	1400	202	216	1,07
SB3 ground storey	3907	1991	1748	0,88	3539	1918	1737	0,91
SB4 ground storey	4615	762	342	0,44	4643	524	342	0,65
SA2* storey I	1139	195	195	1,00	1179	174	159	0,91
SA3* storey I	1042	262	223	0,85	1332	209	160	0,76
Note: * resistance capacity was calculated with the diminished area of the main reinforcement.								

From the structural analysis data presented it can be concluded:

- most of the actual values of earthquake capacity ratio $R > R_{min} = 0,50$;
- for column SB4 the value of $R = 0,44$;
- low values $R < 1$ were obtained for columns SB3, SA2 and SA3.

According to Romanian design codes for existing structures, in case that $R \leq R_{min} = 0,50$ for normal importance (class III) buildings, strengthening is necessary.

A special analysis was performed on the floor beams GE14 and GE15 of the 4th storey, where some inclined and dangerous cracks were present, due to: inadequate shear reinforcement (stirrups or/and inclined bars) near the force load (around the secondary beam) where the shear force has an important sensitive value.

Table 2.15. Static and dynamic analysis results for beams at shear forces

Storey	Element	Design shear force V_{Ed} [kN]		Resistance shear force V_{Rd} [kN]	$\left(\frac{V_{Rd}}{V_{Ed}}\right)_{min}$
		Persistent and transient design situation	Accidental design situation		
III	Transversal main beam	270	313	206	0,66
	Longitudinal main beam	130	180	125	0,69
IV	Secondary beam	154	0	121	0,79
VI	Transversal main beam	10	138	93	0,67
	Transversal main beam	262	281	172	0,61

Shear force resistance was calculated according to inclined cracks theory [83]. The fundamental condition for checking to shear forces at the ultimate limit state is $V_{Rd} / V_{Ed} \geq 1$. From the data presented in Table 2.15 it can be seen that all elements are vulnerable and a strengthening solution is necessary.

Structural Analysis Carried Out After Repair

Several strengthening solutions were proposed and analysed. The structural redesign was performed by finite element method on the spatial structure presented before, at present-day level actions.

Initial rehabilitation of the reinforced concrete structure, performed in 1999, was adopted for both types of damages and consisted of reinforced concrete jacketing of beams, columns and foundations. Due to the inadequate main reinforcement in columns SB3 and SB4 some strengthening solutions, by reinforced concrete jacketing, were analysed with the results of structural analysis quantified by the earthquake capacity ratio $R = M_{Rd} / M_{Ed}$ presented below:

Table 2.16. Redesign of strengthened structure: efficiency of different solutions

Element	Transv. seismic action			Long. seismic action		
	M_{Ed} [kNm]	M_{Rd} [kNm]	$R = \frac{M_{Rd}}{M_{Ed}}$	M_{Ed} [kNm]	M_{Rd} [kNm]	$R = \frac{M_{Rd}}{M_{Ed}}$
Strengthening solution A: - column SB3 from +4,40 m to +10,80 m - column SB4 from foundation to +10,80 m						
SB3 ground storey	376	356	0,95	335	278	0,83
SB4 ground storey	1585	1358	0,86	1193	1358	1,14
Strengthening solution B: - column SB3 from foundation to +10,80 m						
SB3 ground storey	2113	2214	0,91	1855	1984	1,07
SB4 ground storey	594	326	0,55	456	321	0,70

For both strengthening solutions $R > R_{min} = 0.50$ as necessary for existing buildings class III. Finally, due to economic reasons, the strengthening solution B, only of the column SB3, was chosen. The main girders and secondary beams were strengthened by coating with reinforced concrete. New longitudinal reinforcement bars and stirrups were located at the bottom of each beam in a new concrete layer of 15 cm depth. The column SB3 was strengthened for both local damage and inadequate reinforcement. The jacketing with reinforced concrete was used over two storeys and consists of 22,5 cm depth on all four sides. The existing foundation was jacketed over and around by 50 cm depth reinforced concrete for secure fixing of the column new main reinforcement.

At the assessment performed in 2003, due to continuous operation and subsequent damage of the structure, it was found that some beams and one column were characterised by inadequate longitudinal reinforcement (in the column) and shear reinforcement as well as corrosion of many stirrups at beams. The strengthening solution adopted was based on carbon fibre reinforced polymer (CFRP). Structural analysis carried out after repair shown the same results as before repair since no cross section dimensions changes were performed.

Codes of Design

The structural analysis was performed according to the Romanian codes of actions [87] by taking the seismic action into account at present-day magnitude [8]. The assessment of the existing structures to the seismic action is estimated according to the Romanian Code P100-92 [8] by calculus of the earthquake capacity ratio.

The initial rehabilitation of the reinforced concrete existing structure was performed by jacketing with reinforced concrete. The analysis before and after strengthening was done according to the Romanian Code for design and detailing of reinforced concrete structural members STAS 10107/0-90 [83].

The final strengthening was based on carbon fibre polymer composites (CFRP). The design and detailing of strengthening solutions were done according to *fib* Bulletin 14 [66], *fib* Bulletin 24 [72] and *fib* Bulletin 35 [73] for retrofitting of concrete structures by externally bonded CFRPs with emphasis on seismic applications. The cross section analysis after repair was done by using the SIKA Software for the design with Sika CarboDur Composite Strengthening Systems to increase Flexural, Shear and Confinement Strength of reinforced Concrete Structures based on the *fib* Bulletin 14 [66].

DETAILING

The rehabilitation of the reinforced concrete structure adopted and performed in 1999 for both types of damages consisted of jacketing with reinforced concrete of deteriorated beams, one column and its foundation (Figures 2.61-2.63).

The main girders and secondary beams were strengthened by coating with reinforced concrete. New 4ϕ25 mm reinforcement bars for each secondary beam and 6ϕ25 mm reinforcement bars for main girder were placed at 15 cm from the bottom side of the beams with new stirrups ϕ8/15 cm (see Figures 2.61-2.62). One column was strengthened for both local damage and inadequate existing reinforcement. The coating with reinforced concrete was used over two storeys and consists of 16ϕ28 mm longitudinal reinforcement bars, stirrups ϕ10/15 cm and 22,5 cm concrete depth on all four sides (see Figure 2.63). The existing foundation was jacketed over and around by 50 cm depth reinforced concrete for secure fixing of the column new main reinforcement. Reinforcement steel characteristics used for strengthening are: Romanian ribbed bars PC52 (f_{yk} = 350 N/mm^2; f_{yd} = 300 N/mm^2) for longitudinal reinforcement and Romanian plain bars OB37 (f_{yk} = 245 N/mm^2; f_{yd} = 210 N/mm^2) for stirrups. New concrete was class C20/25.

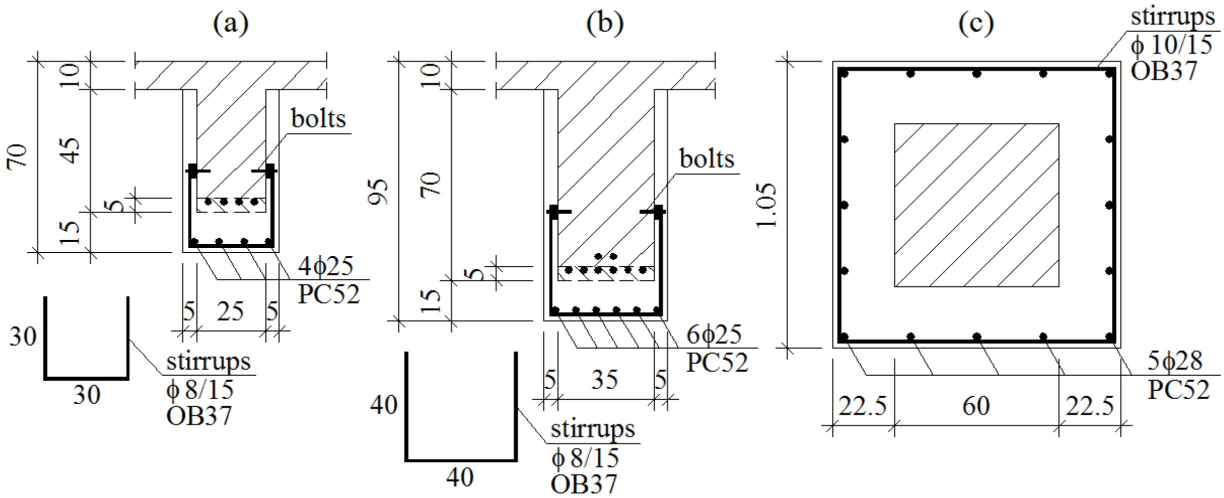


Figure 2.61. Reinforced concrete jacketing solutions:
(a) secondary beam; (b) main girder; (c) column

Figure 2.62. Reinforced concrete strengthening of main girder

Figure 2.63. Reinforced concrete strengthening of column

The new longitudinal reinforcement bars from beams were anchored at the ends by welding on steel plates fixed in the nodes by steel collars around the end nodes of the existing concrete structure. The new stirrups from beams were welded on longitudinal continuous steel plates fixed in the web of existing beams by using mechanic bolts. All these detailing aspects will be further illustrated as construction procedures.

The strengthening, performed in 2003, was used for some beams and one column characterised by inadequate flexural and shear reinforcement. The strengthening solution adopted was based on carbon fibre polymer composites (CFRP) as it is illustrated in the Figures 2.64-2.66.

The column was strengthened at the ground storey by longitudinal Sika Carbodur S1012 strips on each side of 10 cm width and 1,2 mm thickness.

The strips were placed in different position in the cross section in order to pass by the structures node. As shear strengthening a single layer of Sika wrap HEX 230C closed jacket was used on 1,20 m height at the ends of the column. The sheets had 60 cm width and 0,12 mm thickness.

The beams were strengthened at several stories by a longitudinal Sika Carbodur S1012 strip of 10 cm width and 1,2 mm thickness.

The strips were placed at the bottom side of the cross section as necessary from design. As shear strengthening, a single layer of Sika wrap HEX 230C open jacket was used on 1,20 m height at the ends of the beams.

The sheets had 60 cm width and 0,12 mm thickness. CFRP materials characteristics used for strengthening are: E_f = 165 kN/mm^2 and ε_{fu} = 0,017 for longitudinal strips; E_f = 231 kN/mm^2 and ε_{fu} = 0,017 for transversal wraps.

The bond of CFRP materials to the existing concrete layer was ensured by specific adhesives (Sikadur). The longitudinal CFRP strips for columns strengthening were anchored in holes of 20 cm depth performed into existing reinforced concrete foundation.

The longitudinal CFRP strips for beams strengthening started at the face of column-beam node as it were used as lower reinforcement in the beams span.

Ordinary protection of CFRP strengthening materials was ensured by a cement mortar layer. All these detailing aspects will be further illustrated as construction procedures.

Figure 2.64. CFRP strengthening of column

Figure 2.65. CFRP strengthening of main girder

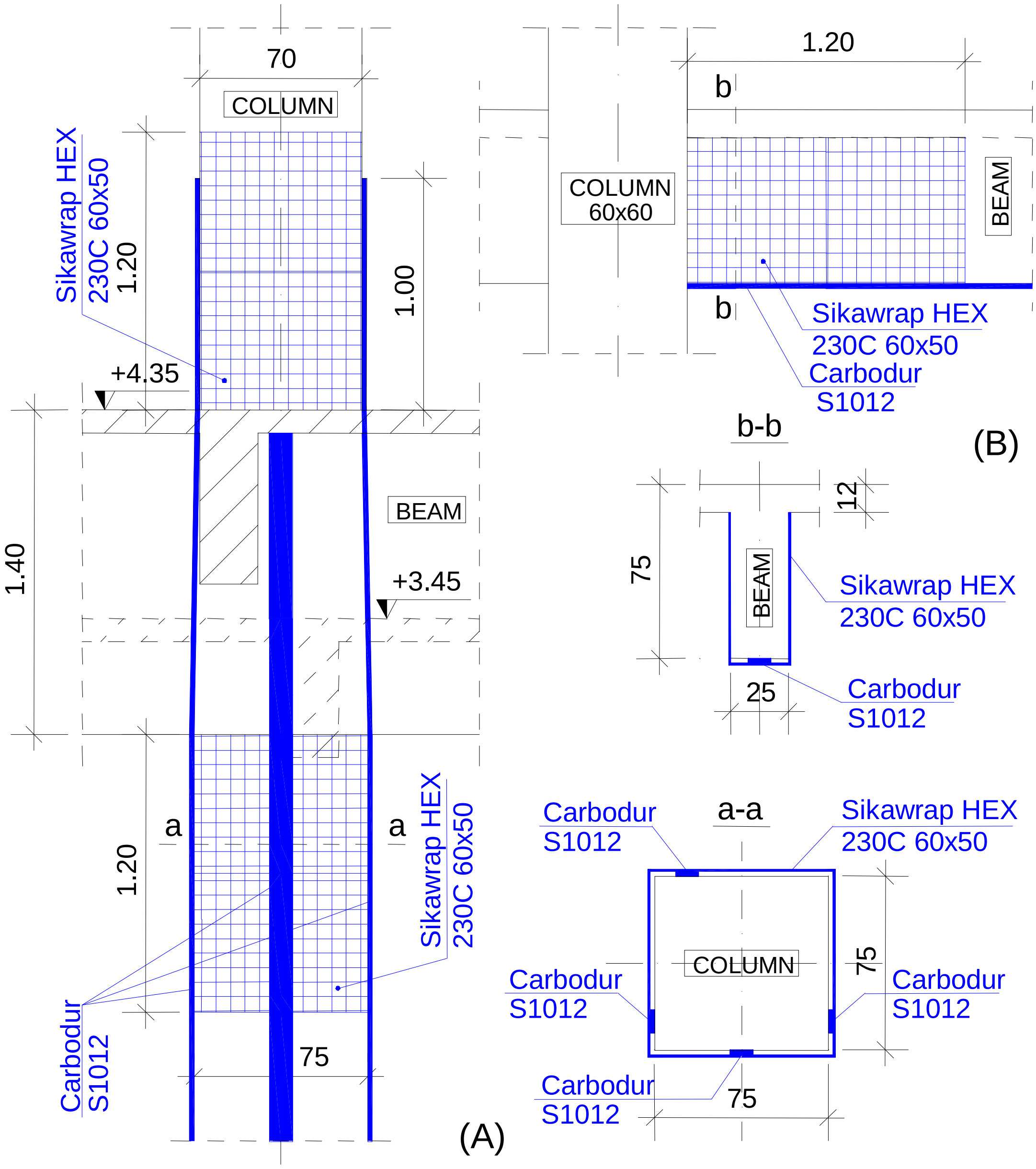


Figure 2.66. CFRP strengthening details for: (A) column; (B) main girder

CONSTRUCTION PROCEDURES

The first rehabilitation of the reinforced concrete structure was performed as follows. The beams were strengthened by coating with reinforced concrete at the bottom side for embedding the new longitudinal reinforcement bars and on the two lateral sides for embedding the new stirrups (Figures 2.67-2.68). The column was strengthened by coating with reinforced concrete on all four sides (Figure 2.69). The construction steps for reinforced concrete jacketing were:

- Spalling of damaged concrete cover and mechanical cleaning (sand blasting) of existing concrete substrate;
- Fixing of steel collars around the beam-column nodes by mechanical bolts and the steel plates for welded anchorages of longitudinal new reinforcement bars from beams (see Figure 2.67);
- Placing of longitudinal new reinforcement for beams (see Figure 2.68) and for columns with the secure fixing into foundation (see Figure 2.69);
- Placing of the new transversal stirrups for beams and columns. The stirrups from beams were welded on longitudinal continuous steel plates fixed in the in the web of existing beams by using mechanic bolts.
- Manufacture and placing of timber framework and shoring;
- Casting of concrete.

Figure 2.67. Anchorage detailing at beams end

Figure 2.68. Reinforcement detailing at beams

Figure 2.69. Reinforcement detailing at column-foundation joint (according to [88])

The second strengthening was performed as follows.

The column was strengthened with longitudinal CFRP strips on all four sides. The strips were placed in a different position in the cross section in order to pass by the structures node. As shear strengthening, a single layer of CFRP closed jacket sheet was used at the ends of the column. The beams were strengthened by a longitudinal CFRP strip placed at the bottom side of the cross section. As shear strengthening, a single layer of CFRP open jacket sheet was used at the ends of the beams. The construction steps for CFRP strengthening were:

- Mechanical cleaning (sand blasting) of existing concrete substrate and dust removal;
- Sealing of existing cracks and surface repair with Sika epoxy based mortars for obtaining a smooth plane application surface (see Figure 2.70);
- Mechanical rounding of concrete cross section corners at 20 mm radius for CFRP sheets application;
- Application of specific two components mixed adhesives on the concrete substrate. The used adhesives were Sikadur-30 for CFRP longitudinal strips and Sikadur-330 for CFRP sheets.

Figure 2.70. Surface repair and preparation before CFRP application

Figure 2.71. End anchorage of longitudinal CFRP strips for column

- Placing CFRP strips (see Figures 2.71-2.75) bonded by previous presented adhesives. The end anchorage of longitudinal CFRP strips for columns was ensured into holes of 20 cm depth performed into existing reinforced concrete foundation (see Figure 2.71). The longitudinal CFRP strips for beams started at the face of column-beam node as it was used as lower reinforcement in the beams span.
- Placing of CFRP sheets (see Figures 2.72-2.75) bonded by previous presented adhesives. The CFRP wrapping was applied: on four sides for column as closed jacket with a horizontal overlapping of 10 cm and on three sides for beams as open jacket.
- Protection of CFRP strengthening materials by a cement mortar layer.

Figure 2.72. CFRP application for column longitudinal strips

Figure 2.73. CFRP application for column transversal sheets

Figure 2.74. CFRP application for beams longitudinal strips

Figure 2.75. CFRP application for beams transversal sheets

FINAL STRENGTHENING SOLUTION PROPOSED

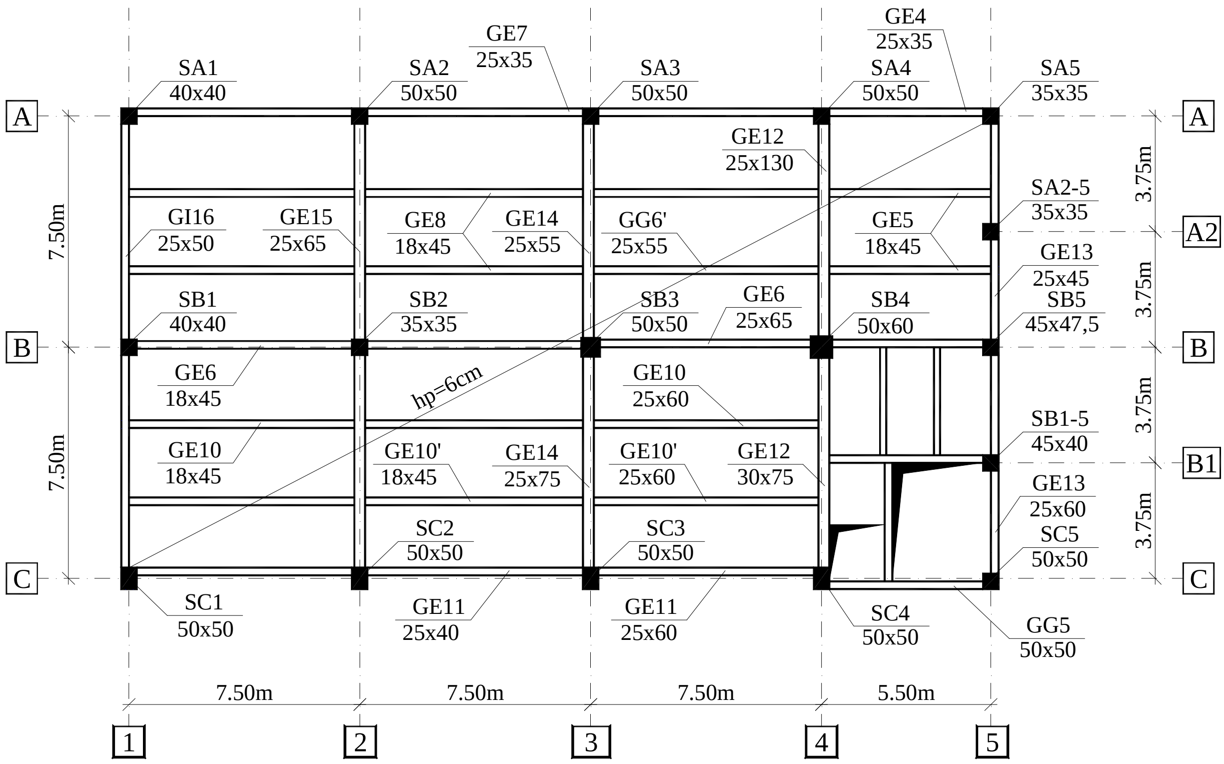


Figure 2.76a. EXISTENT STRUCTURE: Framing plan level +18,40 m

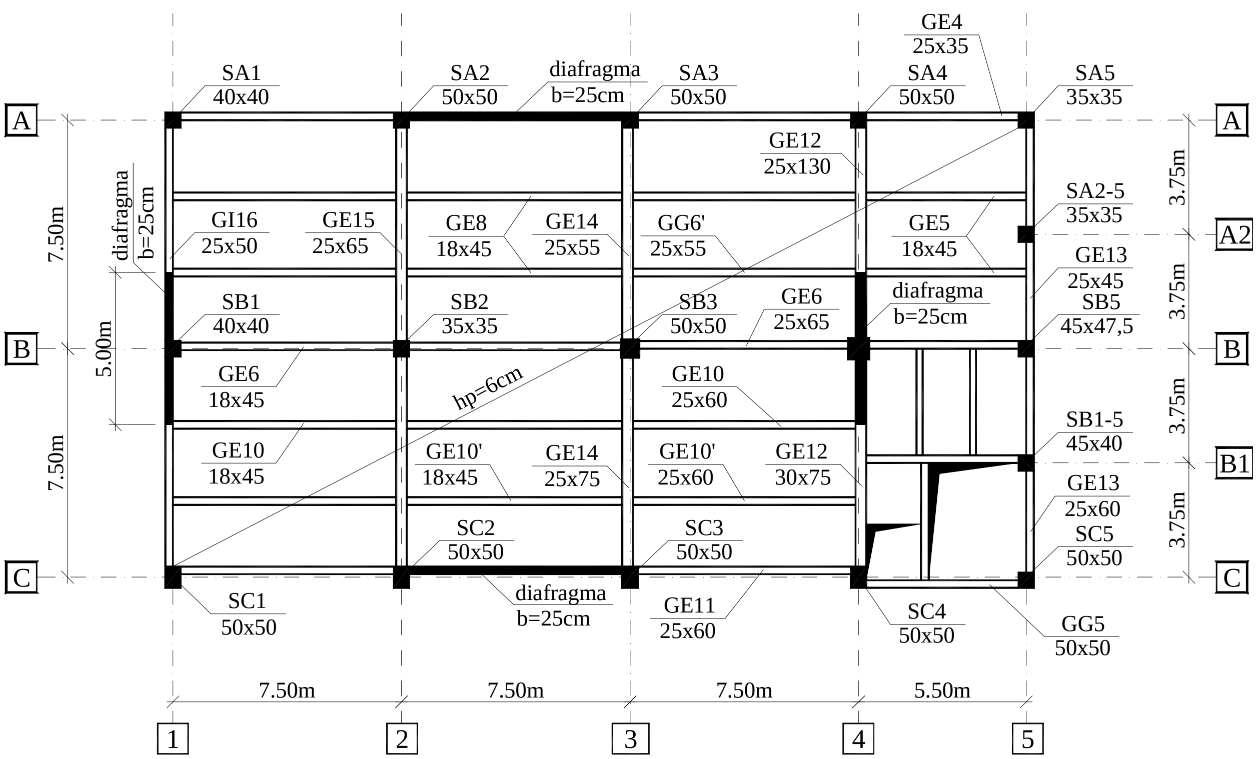


Figure 2.76b. PROPOSED STRUCTURE: Framing plan level +18,40 m

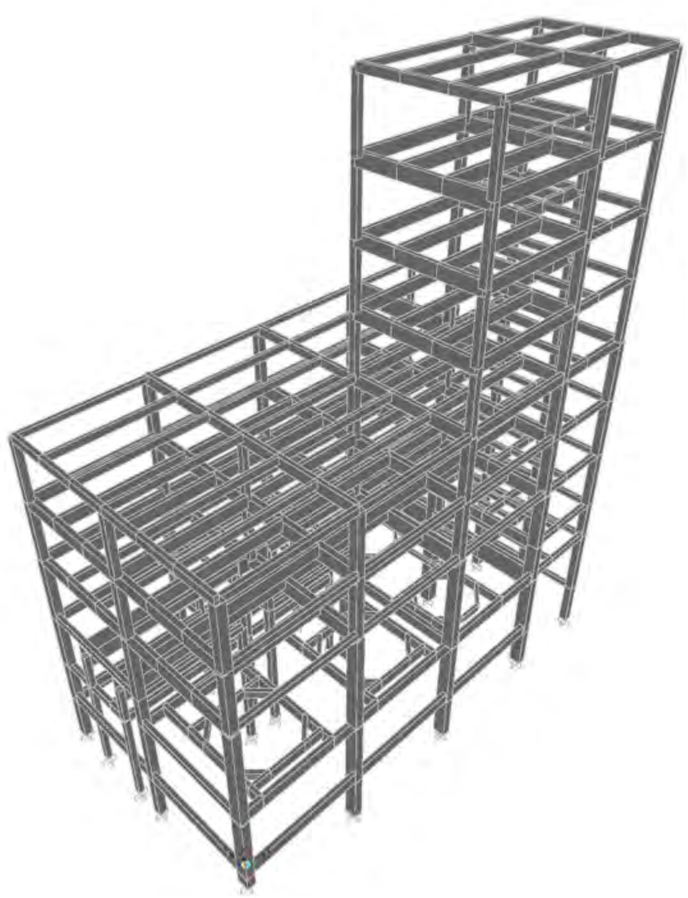


Figure 2.77a. EXISTENT STRUCTURE

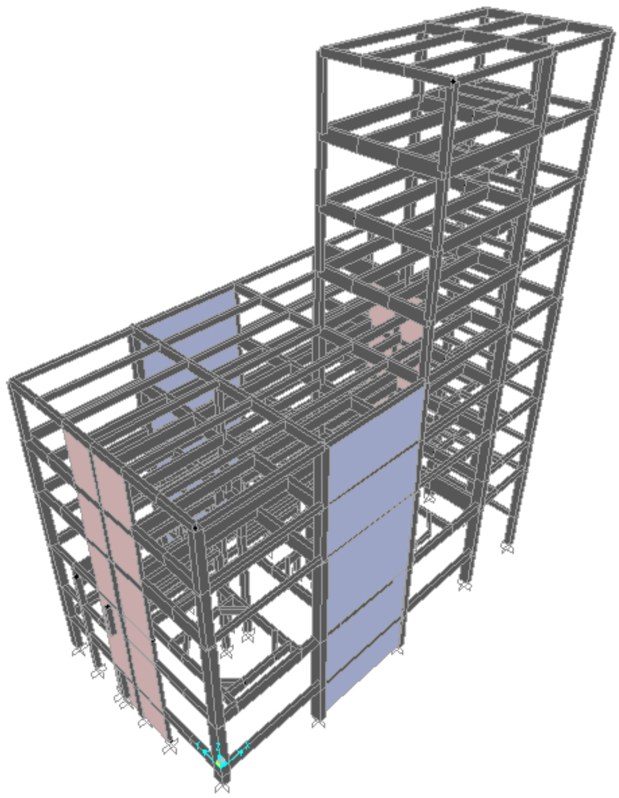


Figure 2.77b. PROPOSED STRUCTURE

DEMOLITION

Due to of general conditions of the RC framed structures with local damage caused by industrial exploitation and weakness of reinforcement of columns and beams due to design made before 1970, the company decided to build a new and modern location for industrial process. A demolition design and drawings were produced for a part of building and the other part remained in conservation stage (Figures 2.78-2.79).

Figure 2.78. Partial demolition phase of RC structure

Figure 2.79. RC structure after demolition

Main activities concerning the demolition are:

Stage 1: Choosing of the crane function of building dimension (20 m high and 20 m in diameter) and of elements weight of 60 kN. A necessary space around the building for the temporary deposit of elements was, also, established.

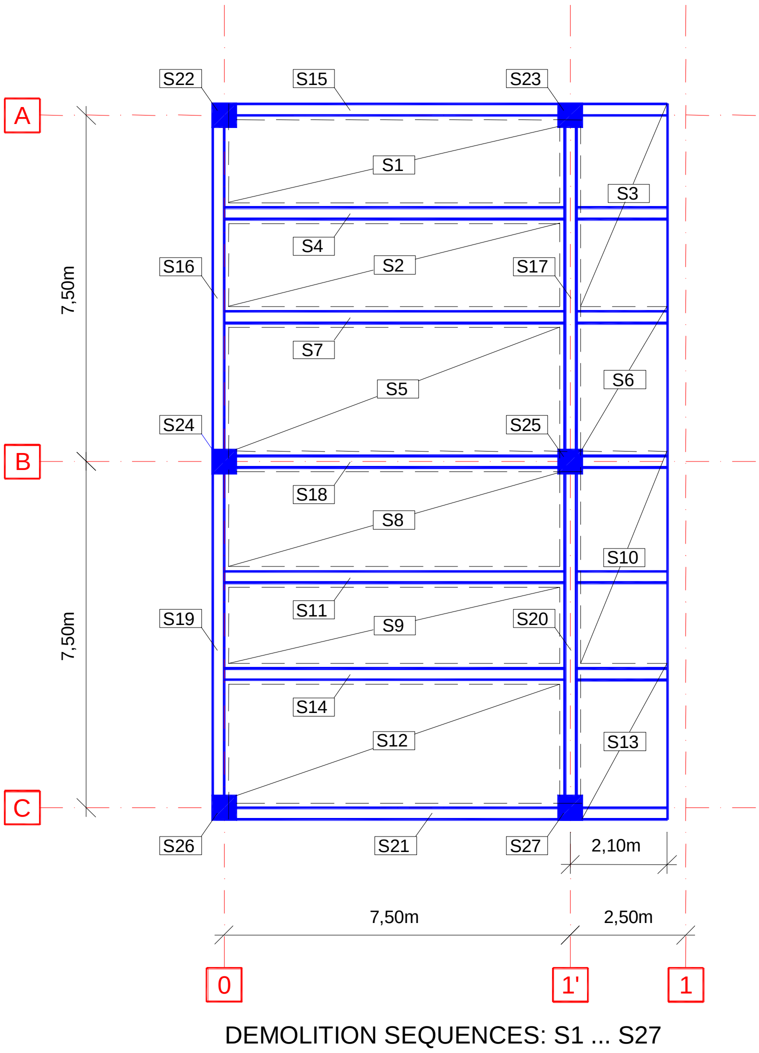
Stage 2: Removing of all furniture, instruments and machines.

Stage 3: De-connecting of power installations, gas, airing and water from all levels.

Stage 4: Opening of existing joint between the part of building which will be demolished and the remaining part; the minimum gap of joint will be 50 mm.

Erecting of a protection wall for the remaining part of the building.

Stage 5: This phase will refer to the demolition of both types of elements: structural and non-structural parts. The sequences of demolition of the 3rd floor are presented in Figures 2.80 and 2.81.



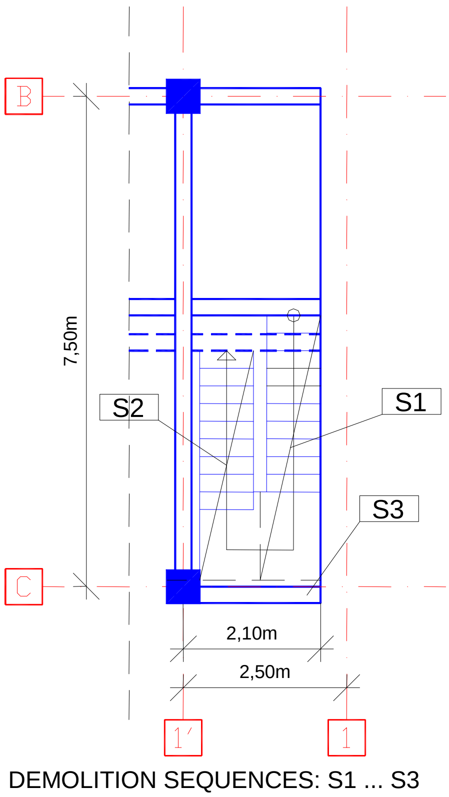


Figure 2.80. Demolition of 3rd floor.

Figure 2.81. Demolition of staircase at 2nd storey.

STRENGTHENING OF A BLOCK OF FLATS

The CFRP strengthening solution was applied on an existing building, a block of flats partially erected in 1990 and located in a seismic zone – Timisoara, Romania. The initial design was made for a six storeys reinforced concrete structure (one underground rigid storey; ground storey + four upper storeys) but only five storeys were initially built. The present owner requirement was to build two more storeys, resulting a seven storeys structure. Consequently, the structure was assessed and rehabilitated in 2008.

The reinforced concrete structure (Figure 2.82) consists of: rigid underground storey made of shear walls; monolithic spatial frame; horizontal precast floors supported by frame beams; isolated foundations under columns and continuous foundations under shear walls.

Figure 2.82. Block of flats

The assessment of in-situ conditions showed up the main problems of the reinforced concrete structure which consisted of inadequate concrete strength at some columns. The assessment of in-situ conditions was performed by non-destructive tests on the main structural elements using the combined method: pulse velocity measurements and rebound test method. The concrete compression strength given by the combined method was for columns: f_{co} = 9,5 N/mm^2 at ground storey; f_{co} = 6,5 N/mm^2 at first and second storey.

The structural analysis of the existing structure was performed, according to the Romanian code of actions, in the persistent and transient design situation of loads and in the accidental design situation of loads by taking the seismic action into account at present-day magnitude. The finite element method FEM analysis of the spatial framed structure was used. For the seismic action the response spectrum analysis was applied with the following characteristics: seismic acceleration a_g = 0,16g; dynamic coefficient β_{max} = 2,5; behaviour factor (takes into account structural ductility) q = 5,0.

The results of the structural analysis are presented in Table 2.17 for columns. The maximum design efforts M_{Ed} were given by the accidental design situation.

Table 2.17. Analysis results for columns

Columns	N_{Ed} [kN]	M_{Ed} [kNm]	M_{Rd} [kNm]	σ_{Ed} [N/mm^2]	Initial structure		Strengthened structure	
					f_{co} [N/mm^2]	$\frac{f_{co}}{\sigma_{Ed}}$	f_{cu} [N/mm^2]	$\frac{f_{cu}}{\sigma_{Ed}}$
Ground storey	1010	114	300	7,4	9,5	1,28	-	-
Storey I	820	95	130	12,1	6,5	*0,54*	12,5	1,03
Storey II	629	80	143	8,4	6,5	*0,77*	12,5	1,49

As noticed from the previous table the resistance capacity M_{Rd} > M_{Ed} for columns at flexure with compression axial force N_{Ed} , meaning that the reinforcement detailing of existing columns was adequate.

The design compression stress σ_{Ed} for concrete cross-section of columns at flexure with compression axial force presented some higher values than the concrete compression strength σ_{Ed} > f_{co} . This could lead to a very danger situation of concrete cross-section fracture.

The rehabilitation of the reinforced concrete structure, performed in 2008, was adopted in order to increase the reinforced concrete columns cross-section compression strength at first and second storey. The strengthening solution was chosen from the sustainability analysis of four possibilities. The strengthening of columns by CFRP (wrap confinement) is more sustainable in comparison to reinforced concrete jacketing due to: energy enclosed is about four times lower; manufacture time is about five times shorter.

The strengthening consisted in transversal confinement with a single layer of wrap closed jacket at both ends of the columns (Figure 2.83) were the maximum design efforts showed up. The columns have a square cross-section of 450x450 mm. Mechanical rounding of concrete cross section corners at 20 mm radius for CFRP wrap application was performed. The jackets had a width b_f = 600 m and a thickness t_j = 0,12 mm. CFRP materials characteristics used for strengthening are: E_j = 231 kN/mm^2 and ε_{ju} = 0.017. The bond of CFRP materials to the existing concrete layer was ensured by specific epoxy adhesives.

Figure 2.83. CFRP confinement of columns

The ultimate concrete compression strength f_{cu} was calculated based on Spoelstra&Monti [73] practical formulae for CFRP-confined concrete properties:

$$f_{cu} = f_{co} \cdot \left(0{,}2 + 3 \cdot \sqrt{\bar{f}_l}\right)$$

where the normalised value of the ultimate confinement pressure is:

$$\bar{f}_l = \frac{f_l}{f_{co}} = \frac{1}{f_{co}} 0{,}5 \cdot \rho_j \cdot E_j \cdot \varepsilon_{ju}$$

The volumetric ratio of transverse confinement CFRP reinforcement for square columns was ρ_j = 0,011. The ultimate compression pressure was f_l = 2,15 N/mm^2 and the ultimate concrete compression strength for CFRP-confined concrete f_{cu} = 1,92 · f_{co} = 12,5 N/mm^2.

The technical efficiency of CFRP confinement is shown in Table 2.17 and consisted in obtaining, for the strengthened concrete cross-section, values of ultimate concrete compression strength $f_{cu} > \sigma_{Ed}$ design compression stress.

3. REHABILITATION OF EXISTING MASONRY STRUCTURES

3.1. EXPERIMENTAL RESEARCH: MODERN SOLUTIONS FOR STRENGTHENING OF MASONRY STRUCTURES

The motivation for research and development into repairing, strengthening, and restoration of existing buildings in seismic zone is sustained by necessity to extend the life of structures. The masonry structures are the oldest and still very used type of buildings. The main target of the research represents the rehabilitation of old masonry buildings located in seismic zones.

Masonry structures present some important vulnerability in seismic zones: the overall lateral stiffness values along the two main axes are different; lack of seismic joints to divide building parts having different dynamic characteristics; lack of reinforced concrete straps at each level; defects of wall connections at corners, crossings and ramifications as well as the presence of cracks; inadequate bearing capacity at normal forces on the walls. On the other hand, structural weakness is characterised by various irregularities and discontinuities or by general structural vulnerabilities: irregular distribution of stiffness at lateral displacements; strength discontinuities; mass irregularities; vertical load discontinuities.

Masonry, made of bricks, stones or other blocks, has a high compressive strength but its main disadvantage is poor tensile strength due to masonry members will crack and fail even if they are subjected to relatively small loads.

The methods of strengthening existing masonry structures with the use of traditional technology are various: erection of RC cores appropriate distance combined with straps at each level, masonry lining with reinforced concrete, masonry confinement with steel profiles, interlocking of masonry walls at corners, crossing and ramifications with RC elements and/or some steel profiles, adding new inner walls and/or some outside abutments [89].

Near-surface-mounted reinforcement implies that steel bars/rods mainly of CFRP are bonded in sawn grooves in the masonry or concrete cover. The use of this technology has a lot of advantages: no requirement for surface preparation work, installation time is minimal, no change of the existing structure dimensions, the cost compared with traditional methods is lower even thaw the material costs are higher.

EXPERIMENTAL PROGRAMME

BOND STRENGTH TESTS

The bond between strengthening bars and substrate material like concrete, mortar, brick/stone masonry is an important factor in order to perform an efficient rehabilitation on structural members.

The aim of this research is to investigate the mechanism of bond between two types of bars and brick masonry element taking into account two parameters: type and diameter of strengthening bar.

The bar types used for the experimental program are: Romanian Profiled Steel Bar PC 52 which is a hot-rolled steel and Brutt Helical System – Brutt Saver which is a special bar which gives a high bond at a small cross-sectional area. Three diameters have been used for each type of bars. The geometrical and mechanical properties of the bars are presented in Table 3.1:

Table 3.1. Geometrical and mechanical properties of bars

Type of bar	Bar nominal diameter φ [mm]	Cross-sectional area A [mm^2]	Load at failure P_{max} [kN]	Ultimate strength f_u [N/mm^2]	Elongation at failure ε [%]
Profiled steel bar PC52	6	28.26	14.43	510	15.00
	8	50.24	25.50	507	15.00
	10	78.50	40.04	510	15.00
Brutt Helical System (BHS)	6	9.00	8.10	900	14.70
	8	11.00	9.95	905	7.25
	10	16.00	11.30	706	8.64

The effect of the near-surface technology can be explained by results of the experimental tests of adherence between brick masonry and steel bars. The test samples before the two bricks specimen are glued together are presented in Figure 3.1:

Figure 3.1. The samples before embedment

The embedment of bars into brick-mortar have been chosen to avoid the slipping of the bars during the test and are presented in Table 3.2:

Table 3.2. Data from pull-out test arrangement

Type of bar	Bar diameter φ [mm]	Embedment in brick-mortar system l_b [mm]	Pull-out load P_{po} [kN]	Bond strength τ_a [N/mm^2]	Ratio $\frac{P_{po}}{P_{max}}$
Profiled steel bar PC52	6	300	19.85*	3.510	1.38
	8	400	29.45**	2.930	1.15
	10	600	38.25**	2.030	0.96
Brutt Helical System (BHS)	6	300	7.87*	1.390	0.97
	8	400	8.25*	0.820	0.83
	10	600	12.15*	0.645	1.08

Notes: * failure in bar; **slipping of bar

The pull-out test arrangement is illustrated in Figure 3.2. The brick block dimensions are different in function of the bar embedment in brick-mortar system.

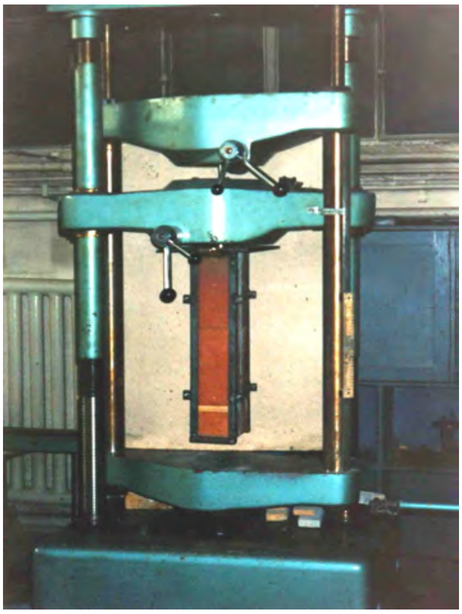


Figure 3.2. The pull-out test arrangement

The results of the experimental program are presented in Table 3.2. Pull-out load P_{po} is influenced by the type and diameter of bars. The smaller values obtained for Brut Helical System bars are in accordance with the cross-sectional area and it is explained, too, by the ratio pull-out load P_{po} – load at failure P_{max}.

Brutt Helical bars and Profiled Steel bar of ϕ6 mm, failed outside the embedment zone (Figure 3.3) and for the specimens of Profiled Steel with ϕ8 mm and ϕ10 mm the failure have been produced by sleeping of bars (Figure 3.4).

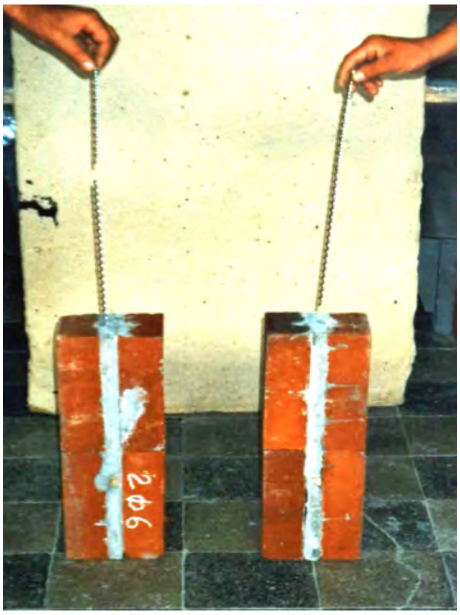


Figure 3.3. Brutt Helical Bars failure

Figure 3.4. Profiled steel bar slipping

The average bond strength τ_a was calculated using the formula:

$$\tau_a = \frac{P_{po}}{\pi \cdot d \cdot l_b}$$

The mortar used for bonded the bars into the sawn groove was a mortar with Powder HS which is characterized by the strength: 45.54 MPa at 3 days and 49.61 MPa at 7 days.

Technical efficiency of using the different types of bars for strengthening is illustrated in Figure 3.5 and [90] in which the maximum failure load at interface masonry/concrete-epoxy, $P_{\tau,max}$, is:

$$P_{\tau,max} = \tau_{av} \cdot p \cdot l_b$$

where: τ_{av} – the average bond strength of the bond masonry/concrete-epoxy interface (for a brick with the strength of 10 N/mm^2; τ_{av} = 1.4 N/mm^2);
p – groove cross-sectional perimeter;
l_b – embedment length.

The tensile strength P_{max} (load at failure) of the three types of bars are taken into account in function of experimental data or by calculation in accordance with the ultimate strength for CFRP.

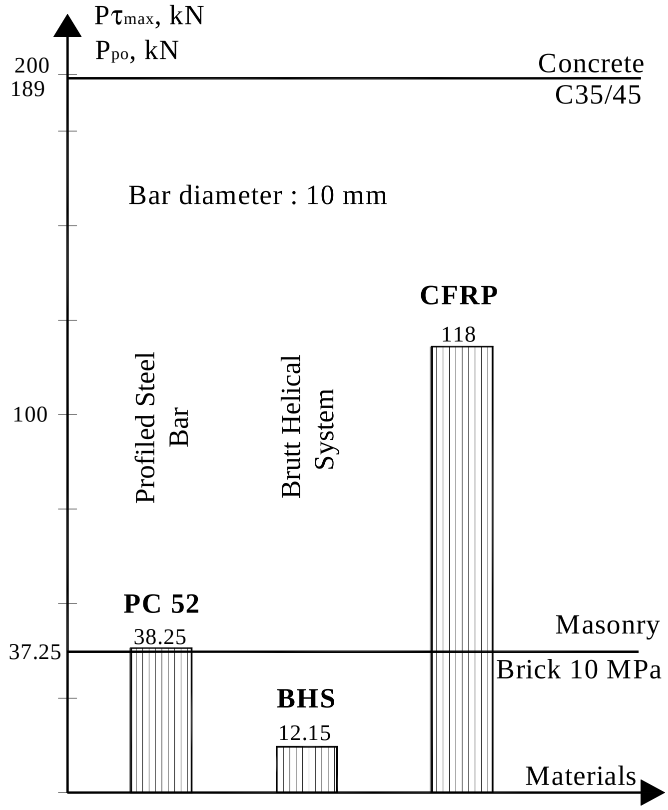


Figure 3.5. Technical efficiency of different types of bars

From the data illustrated above it can be noticed:

- the strengthening of masonry walls by using of CFRP rods seems to be inefficient due to of higher tensile strength P_{max} as compared with the maximum load at failure $P_{\tau,max}$ which can be supported by the brick-epoxy interface;
- the strengthening with Profiled Steel Bars as well as with Brutt Helical Systems, but with a higher diameter (12, 14 mm) are recommended for rehabilitation of masonry walls;
- the concrete structures can be well rehabilitated by using near-surface-mounted reinforcement with CFRP rods.

MASONRY WALL TESTS [91]

The experimental programme focused on strengthening of masonry walls by using different types of bars and CFRP.

Masonry brick walls using mortar M25 and having a window opening were designed and manufactured at characteristics shown in Figure 3.6 (scale 1:2).

Experimental tests were done according to the Figure 3.6. Vertical and horizontal loads were applied on the top RC strap. The horizontal load simulated a seismic action.

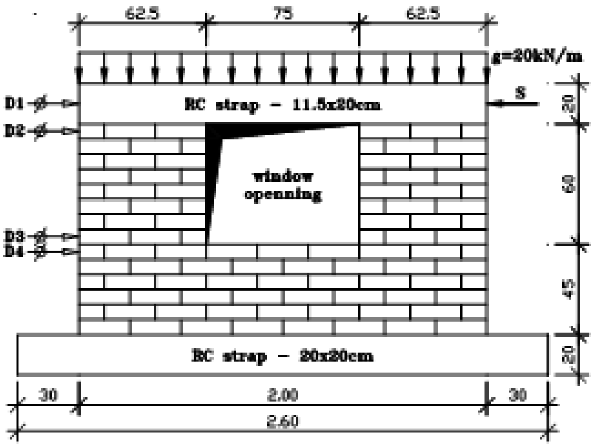


Figure 3.6. Experimental masonry walls

The first tests were done on the un-strengthened masonry wall until cracking and then strengthened (Figure 3.7) and rested.

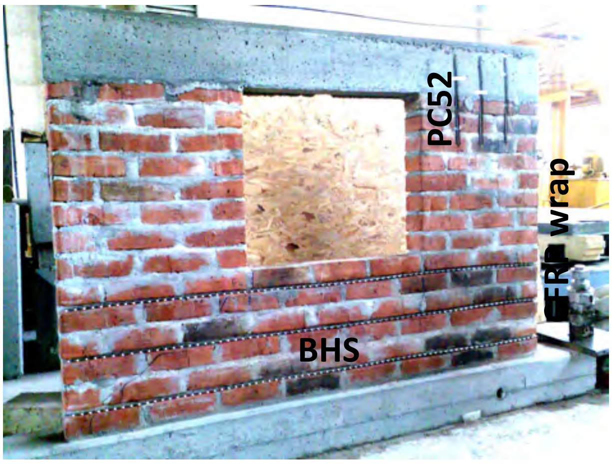


Figure 3.7. Masonry wall strengthening

During the tests were measured by data acquisition station the following experimental data: horizontal load magnitude; horizontal displacements at different heights.

By initial calculus, the failure bearing capacity of the experimental masonry wall was find out: at eccentric compression; at friction in the mortar layer; at main tension stresses.

During the tests the vertical load had a constant magnitude, while the horizontal load was increased up to the failure. The failure and the crack pattern for the un-strengthened masonry wall are presented in Figure 3.8:

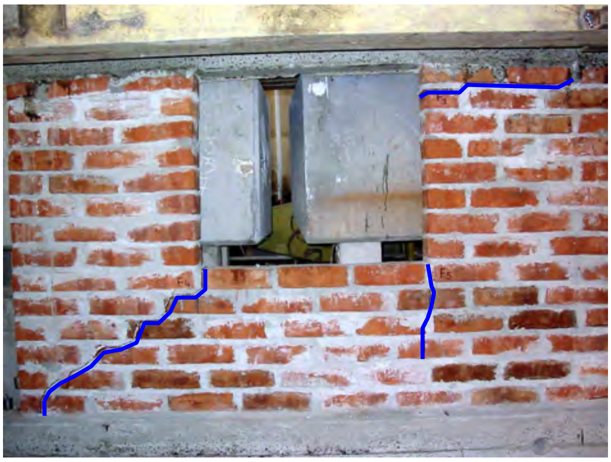


Figure 3.8. Un-strengthened masonry wall failure and crack pattern

The un-strengthened masonry wall failure was characterized by cracking in different ways:

- horizontal cracks at wall base – due to tension from eccentric compression;
- horizontal cracks at wall top – due to friction;
- inclined cracks starting at the window corner – due to main tension stresses.

The strengthening proposals were selected function of the cracking pattern. Subsequently, were applied: vertical CFRP wraps placed on the masonry wall exterior edge; vertical Romanian Steel Bar – PC52 as near-surface-mounted-reinforcement; horizontal high adherence steel bars Brutt Helical System – BHS as near-surface-mounted-reinforcement in the mortar layer.

These masonry walls once cracked were strengthened by the previous different modern and efficient techniques and rested in order to find out the bearing capacity of the rehabilitated walls. The failure and the crack pattern at failure for the strengthened masonry wall are presented in Figure 3.9:

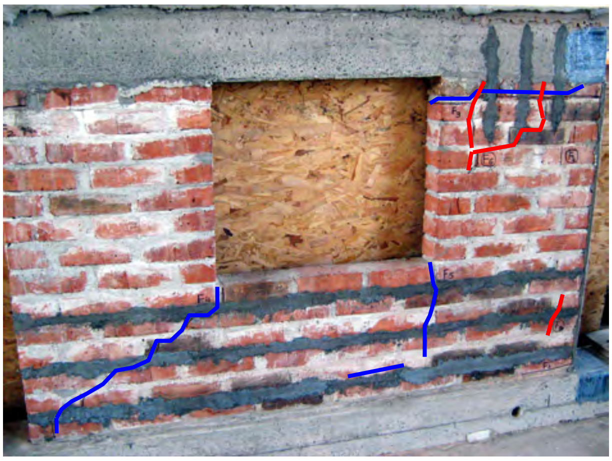


Figure 3.9. Strengthened masonry wall failure crack pattern

The experimental failure horizontal loads S as well as horizontal displacements at different heights D1 – D4 were measured. The results regarding horizontal top-displacements are presented in Figure 3.10, both for un-strengthened and strengthened masonry wall.

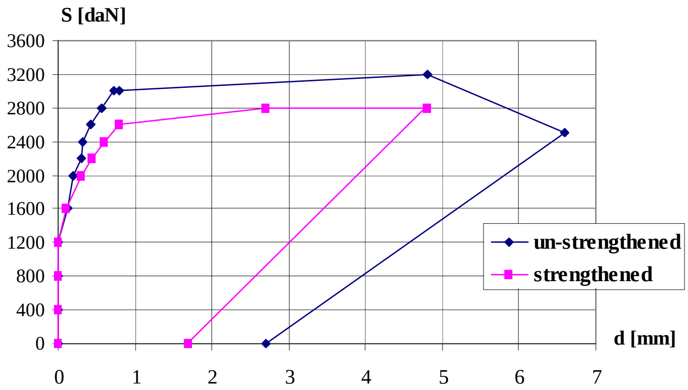


Figure 3.10. Horizontal displacements of un-strengthened and strengthened masonry wall

From the experimental data presented above an important conclusion may be drawn: by using the proposed modern strengthening techniques, the horizontal bearing capacity of the initial wall (fractured by testing and then strengthened) was recovered at 87.5 %, which shows the technical efficiency of the proposed rehabilitation solutions.

3.2. CASE STUDIES

REHABILITATION OF THE BANATUL MUSEUM, TIMISOARA – CLASSIC SOLUTION

The Banatul Museum (Figure 3.11) is one the most important historical building in Timisoara, Romania. The first site of the Huniade-Castle is mentioned in the XIV century. Bad soil conditions and soil water level had affected the castle building which was re-erected or rehabilitated many times: in the period 1720-1750 timber pilot foundations were used; in 1848 it was destroyed by fire and rebuilt in 1850-1856 in the present-day form.

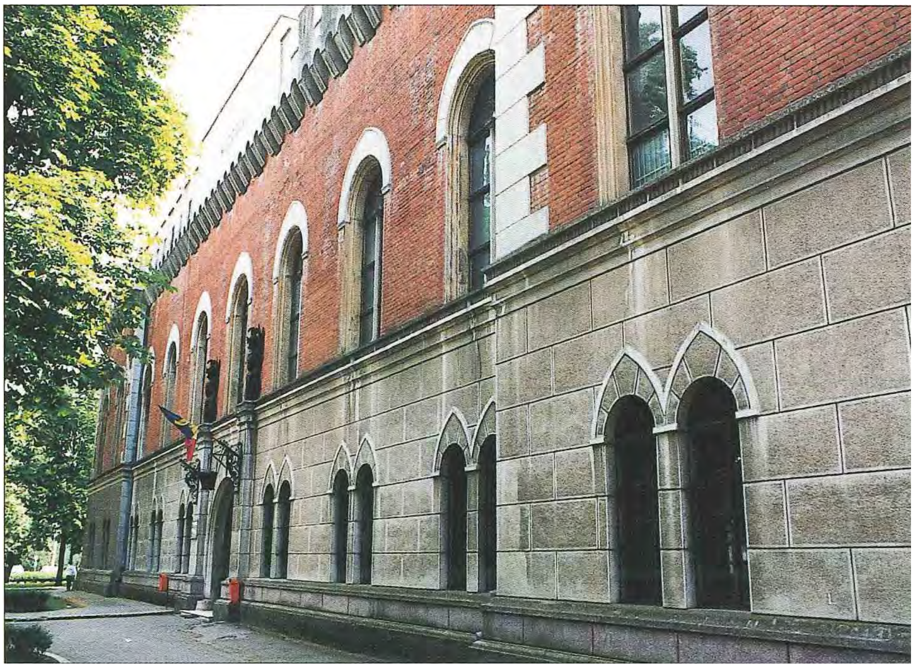


Figure 3.11. The Banatul Museum building

Some rehabilitation were performed during this century as in 1903-1906 by utilisation of bricks as sub-foundations for walls as well as stiffening of some pillar foundations by utilisation of reinforced concrete piles and reinforced concrete floors in 1956-1963 (Hall 2 - Figure 3.12).

The presence of eight reinforced concrete piles around each existing pillar foundation of the Hall 2 contributes to a stabilisation of the three columns. After 40 years, the settlement of Hall 2 columns was insignificant as compared to the settlement values of 24-39 mm under walls. The data concerning settlement were obtained by topographic surveying on markers displaced on the building in 1959. In 1980 there was performed a building maintenance, with filling up the cracks, but after a few years the cracks are still present in building structure.

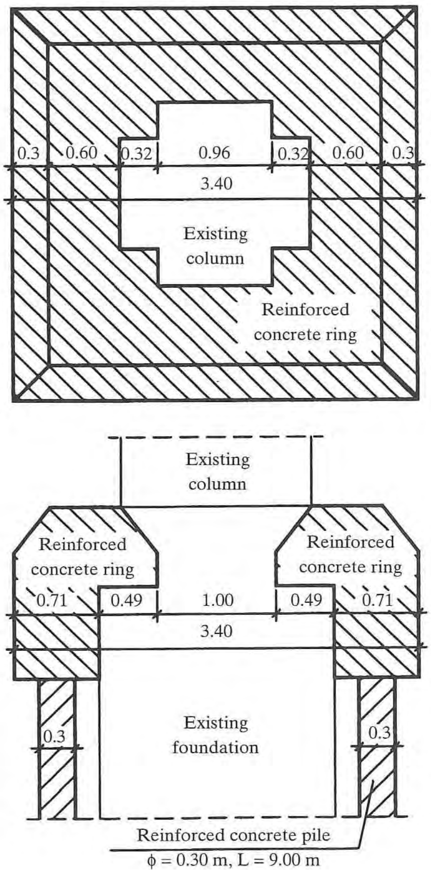


Figure 3.12. The strengthening of Hall 2 existing foundations

The structural composition of the building consists of:

- vertical members which were built of longitudinal and transversal masonry walls with different width 60-240 cm and stone/brick pillars of 98x98 cm (Hall 1) and of 160x160 cm (Hall 2) as presented in Figure 3.14;
- horizontal members consist of brick arches as domical vaults over the first and second level and wooden board over the third level.

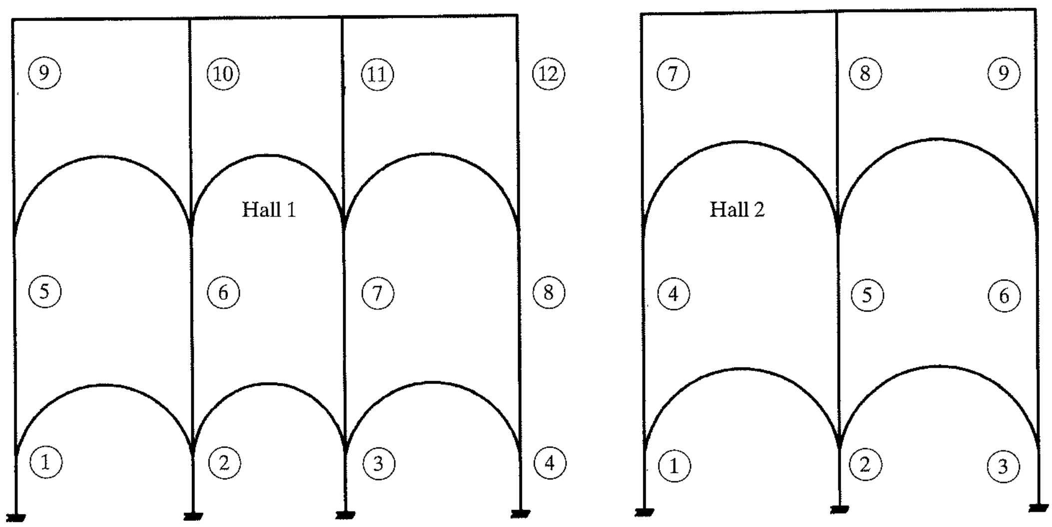


Figure 3.13. The structure of non-rehabilitated building

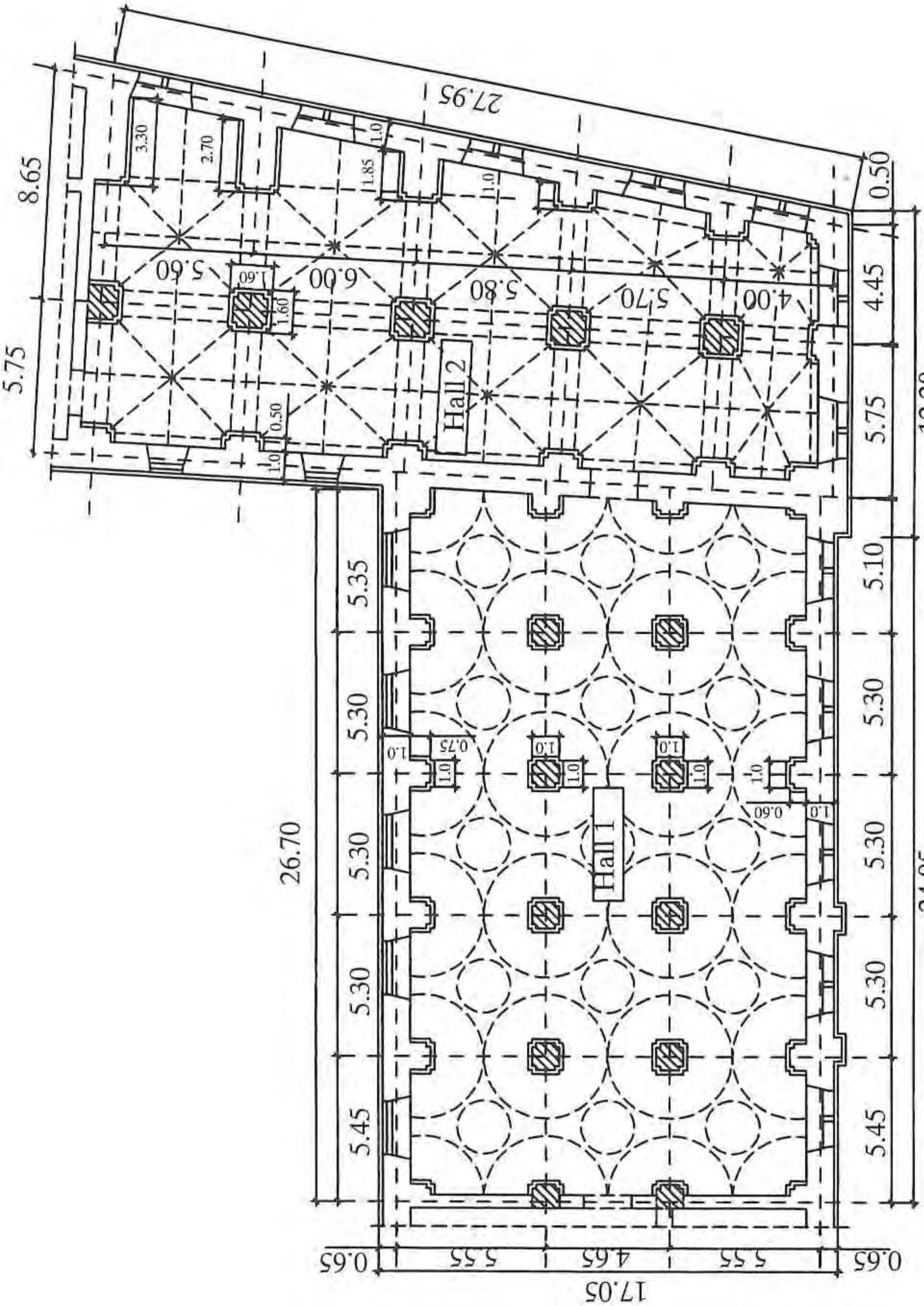


Figure 3.14. Horizontal section at second story through Hall 1 and Hall 2

The main damages of the actual structure of the building are located inside the Halls 1 & 2 and are characterised by cracks in the masonry walls: maximum width of 5 cm at upper part (Hall 1) as well as in the brick arches, especially in the vault head (5-8 mm). Such cracks were caused by both, earthquake actions and soil settlement due to bad soil, non-uniform foundations and soil water level.

STRUCTURAL ASSESSMENT

The analysis of the two structures (Hall 1 and 2) has been performed at both combinations of actions: fundamental combinations and special combinations including seismic action at present-day level. The schematic figure, as transversal framed structures, for analysis of the non-rehabilitated building is presented previously (Figure 3.13). Such framed structures seem to be reasonable for a masonry building due to the presence of cracks in the vault head and masonry walls which disturbs the spatial behaviour.

In Tables 3.3 and 3.4 partial results of the structures' analysis concerning Halls 1 and 2 are presented. It can be noticed: **the eccentric forces have to act out of the cross-sections of the members** which represent a very dangerous equilibrium state of the building during an earthquake. The conclusion is clear - **the building needs a rehabilitation**.

Table 3.3.
Efforts in the structural members of Hall 1 due to seismic combination of actions

Structural members		Non-rehabilitated structure (seismic action from left to right)			Rehabilitated structure								
					Transversal seismic action			Longitudinal seismic action			Diagonal seismic action		
		M [kNm]	N [kN]	e_0 [m]	M [kNm]	N [kN]	e_0 [m]	M [kNm]	N [kN]	e_0 [m]	M_x [kNm]	M_y [kNm]	N [kN]
lateral members	1	936	1138	**0,82**[1)]	449	869	0,51	357	947	0,38	320	258	898
	4	761	1050	0,72[2)]	462	1109	0,42	505	1068	0,47	328	362	892
	5	623	683	**0,91**[1)]	184	868	0,21	627	881	0,71	2	502	873
	8	583	989	0,58[2)]	214	832	0,26	752	930	0,81	28	547	909
	9	131	210	0,62[2)]	50	308	0,16	54	308	0,18	36	21	308
	12	216	227	**0,95**[1)]	101	306	0,33	54	283	0,19	73	11	302
central members	2	266	831	0,32[2)]	450	957	0,47	216	1095	0,20	321	149	964
	3	146	946	0,15[2)]	453	934	0,48	209	973	0,22	314	151	985
	6	268	446	**0,60**[1)]	309	586	0,53	79	441	0,18	223	65	453
	7	268	396	**0,68**[1)]	297	634	0,47	70	504	0,14	218	29	511
	10	132	220	**0,60**[1)]	118	318	0,37	129	314	0,41	86	91	315
	11	16	122	0,13[2)]	102	259	0,39	93	259	0,36	73	66	259
Note: [1)] The first order eccentricities exceed the cross section dimensions ($e_0 > h/2$) [2)] Maximum values are to be obtained for seismic action left to right					- For lateral members $e_0 < h/2$ - For central members: M_R = 522 kNm > M_S^{max} = 453 kNm								

Table 3.4.
Efforts in the structural members of Hall 2 due to special combination of actions

Structural members		Non-rehabilitated structure (seismic action from left to right)			Rehabilitated structure								
					Transversal seismic action			Longitudinal seismic action			Diagonal seismic action		
		M [kNm]	N [kN]	e_0 [m]	M [kNm]	N [kN]	e_0 [m]	M [kNm]	N [kN]	e_0 [m]	M_x [kNm]	M_y [kNm]	N [kN]
lateral members	1	471	1230	0,38	356	1986	0,18	716	1408	0,51	60	605	1309
	3	3090	2160	**1,43**[1)]	1653	2546	0,65	1263	2249	0,56	954	943	2314
	4	2522	1643	**1,53**[1)]	332	1519	0,22	947	567	1,67	120	726	602
	6	2879	1982	**1,45**[1)]	924	1434	0,64	779	568	1,37	711	321	1430
	7	768	484	**1,58**[1)]	49	270	0,18	625	263	2,38	30	410	183
	9	151	60	**2,52**[1)]	47	233	0,20	188	80	2,35	18	136	56
central members	2	598	1307	0,46	946	1032	0,92	190	1032	0,20	746	133	1032
	5	429	1092	0,39	118	748	0,16	151	722	0,21	62	133	723
	8	30	349	0,09	66	134	0,49	185	226	0,82	30	155	226
Note: [1)] The first order eccentricities exceed the cross section dimensions ($e_0 > h/2$)					- For all members $e_0 < h/2$								

The compressive stress in both brick arches and columns, did not exceed the limit value of the brick – mortar strength. The same conclusion was established for the stresses resulted from the fundamental combinations of the actions.

REHABILITATION SOLUTION

Two main parts of the rehabilitation were taken into account: the building structure and the foundation soil.

For the building structure some solutions have been analysed and finally an interesting and original idea was adopted: **to enlarge the structural stiffness of building by increasing the physical and mechanical properties of some component members of the structure.** The main steps of the structure rehabilitation are:

a) The strengthening of the stone / brick pillars which consists of: the propping of the floor; the cut-out of the sandstone on 5 cm around each column; the execution in each corner of the column of rectangular holes of approximately 25x25(20) cm; the mounting in the holes of two steel profiles (100x100x10 mm for Hall 1 and 120x120x12 mm for Hall 2) and concrete reinforcement (4ϕ16 mm with ϕ8/20 cm as stirrups); the interlocking of the steel profiles with wire back-tie; the concreting of the holes; the replacing of the sandstone plates which were well treated for durability increasing (Figures 3.15-3.16).

Some details of the strengthening of stone/brick columns are presented. The results of such strengthening are:

- the increase of the modulus of elasticity and consequently of the column stiffness by 7.5 times for Hall 1 (E_m = 1875 MPa for stone pillar and E_c = 35.000 MPa for composite member) and of 6.28 times for Hall 2;
- the brick confinement due to of the wire back-tie;
- a proper fixing into foundation which is also performed of steel profiles.

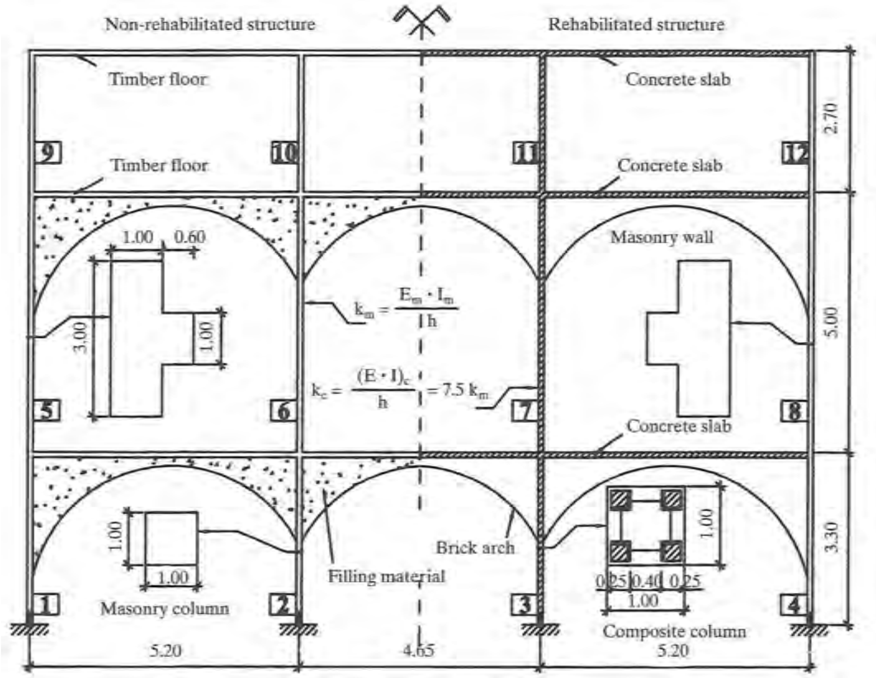


Figure 3.15. Non-rehabilitated and rehabilitated structure

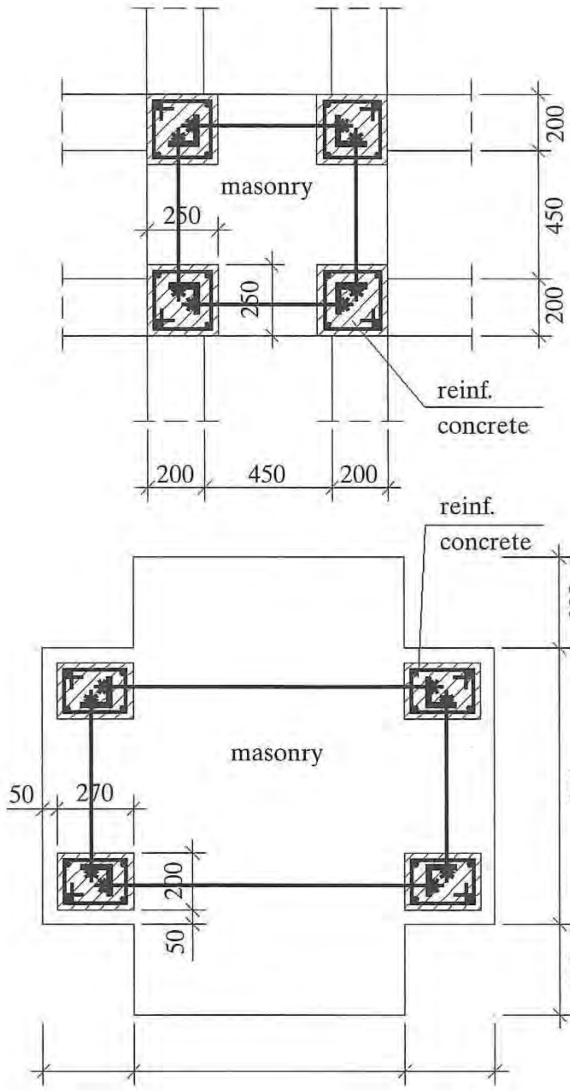


Figure 3.16. Details of the columns strengthening for Hall 1 and Hall 2

b) The horizontal strengthening is accomplished with reinforced concrete floors which were provided at each level over the brick arches (Hall 1). The pair of reinforced concrete beams, at each side of the pillar, were connected with the new composite columns; a short span – flat slab is supported on the pair of beams and on the lateral walls through a strap; the horizontal fixing between the lateral walls with the floor is realised with steel connectors (ϕ14/24 cm).

The rehabilitation of the foundations soil represents an important step for receiving a safety behaviour of the structure at different actions. The soil characteristics are to be increased by infilling a bentonite-cement slurry. The infilling material have to act as: soil stabilisation under and around foundations; to infill the holes and joints of the existing foundations. The injection with the bentonite-cement slurry is used between 1.5 and 4.5 m levels.

CONCLUSIONS

In case of rehabilitation of a historical building the final decision have to take into account the architectural requirements, among them, the geometrical proportions and the principal fronts. These requirements were fulfilled by the rehabilitation project.

The rehabilitation solution of the Banatul Museum building is based on the idea to enlarge the structural stiffness of building by increasing the physical and mechanical properties of some component members of the structure.

The advantages of the rehabilitation solutions are: no change of the architectural aspect and structure geometry; a safe behaviour to seismic action; an easy technology of refurbishment; economical advantages.

The rehabilitation works of the Banatul Museum of Timisoara have started in 2001 and are to be finished in 2002.

RETROFITTING OF HISTORIC MASONRY STRUCTURES – MODERN SOLUTION

At some churches, built in 19th century, arches and vaults presented damages due to the foundations settlement as well as earthquake actions.

The retrofitting strategy consisted in using classic strengthening of the perimeter walls with RC structural elements as well as providing confinement with FRP wrapping and near-surface-mounted-reinforcement.

The Oradea Church (Figure 3.17) was strengthened in 2004 by using near-surface-mounted-reinforcement technology with High Adherence Steel Bar PC52 and Brutt Helical System BHS. The system was applied outside the walls at upper and bottom part of the windows.

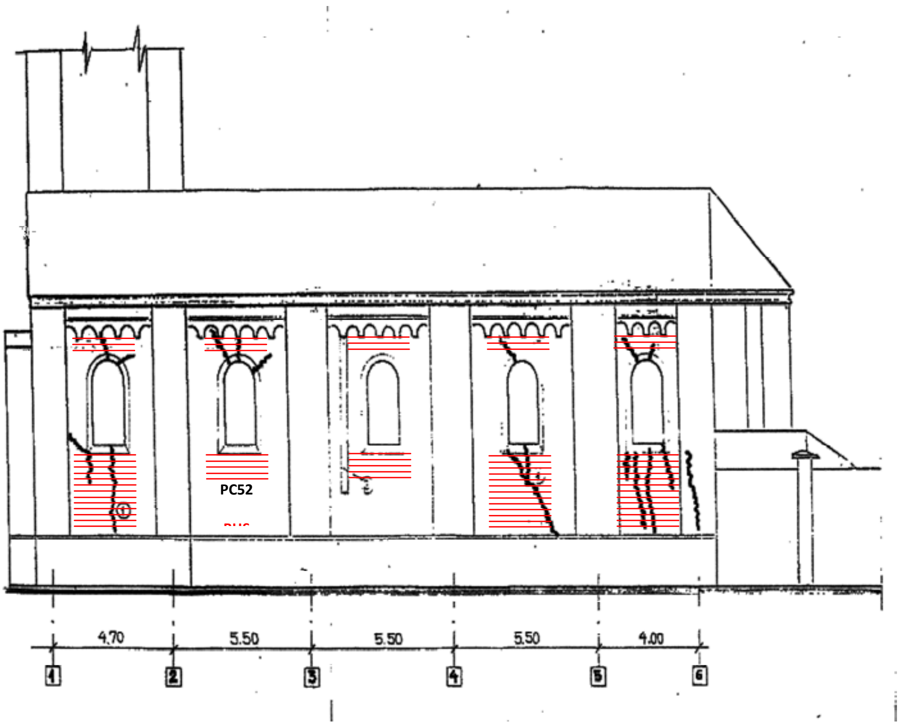


Figure 3.17. Existing masonry church in Oradea, Romania

STRUCTURAL REHABILITATION OF HISTORICAL MASONRY BUILDINGS: REHABILITATION OF A TOWER STRUCTURE BY MODERN SOLUTIONS

The old malting building, erected between 1857-1876 at the “Timisoreana” Brewery, is a five storeys masonry structure and a tower (Figure 3.18) composed of:

- walls of (140-50) cm thickness;
- inter-storey floors - brick masonry vaults supported by steel profiles;
- a tower, of about 14.00 m height and 2.80 m diameter, supported by an interior dome.

Figure 3.18. Old malting building

STRUCTURAL ASSESSMENT

The assessment of the structure was performed in 2007 according to the present-day Romanian codes for existing structures and codes for design loads magnitude.

The main structural damages are:

- vertical cracks in the tower masonry structure (Figure 3.19);
- corrosion of steel members: horizontal circular rings for confining the tower; profiles for supporting the floor masonry vaults.

Figure 3.19. Vertical cracks of masonry tower

The static and dynamic analysis at different actions showed up major structural vulnerability, mainly due to the period of design and erection (19^{th} century):

- the tower, about 14 m high, presents general instability at seismic actions: the total bending moment at tower base leads to an eccentricity $e_0 = 1.78\ m > D_{ext} / 2 = 1.50\ m$ where D_{ext} is the tower exterior diameter;
- in some zones of the tower masonry structure actual stresses, due to various loads, are greater than the tensile strength R_{ti} of masonry:
  - σ_{ef} = 0.93 daN/cm^2 > R_{ti} = 0.8 daN/cm^2 at the tower – dome crossing (50 cm width masonry);
  - σ_{ef} = 3.10 daN/cm^2 > R_{ti} = 0.8 daN/cm^2 at the tower base (20 cm width masonry);
- in the masonry dome, which supports the tower, the actual stresses by parallel direction are:
  - σ_θ = 0.85 daN/cm^2 > R_{ti} = 0.8 daN/cm^2 at the lower part of the dome;
  - σ_θ = 2.19 daN/cm^2 > R_{ti} = 0.8 daN/cm^2 at the upper part of the dome;
- temperature variations inside-outside the tower produce actual stresses σ_t = 1.0 daN/cm^2 > R_{ti} which causes the vertical cracking.

The structure, also, presents general and specific detailing lacks: no rigid floors at two storeys; no straps at all levels; the ratio between span and width of masonry shear wall is too large.

These major vulnerability classify the structure as having high risk of failure at present seismic code design magnitude.

STRENGTHENING SOLUTIONS

In order to preserve the old building as architectural monument and to reduce the seismic failure risk, the following strengthening solutions were designed:

- for general stability of masonry tower: vertical reinforcement bars (4 x 2ϕ28) embedded at the upper side of the tower in a RC beam and welded on steel profiles I 30 placed in the dome, at the tower base; vertical CFRP wrap (4 x 2 strips of 20 cm width) on the entire tower height (Figures 3.20-3.23);

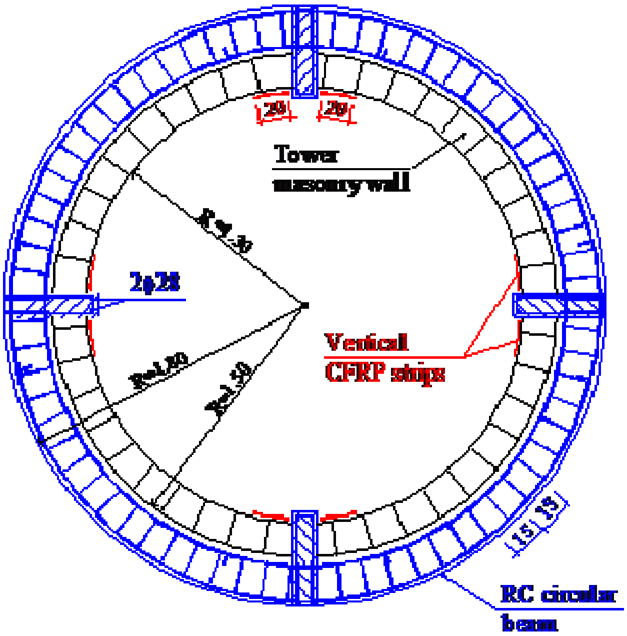


Figure 3.20. Tower strengthening at base section

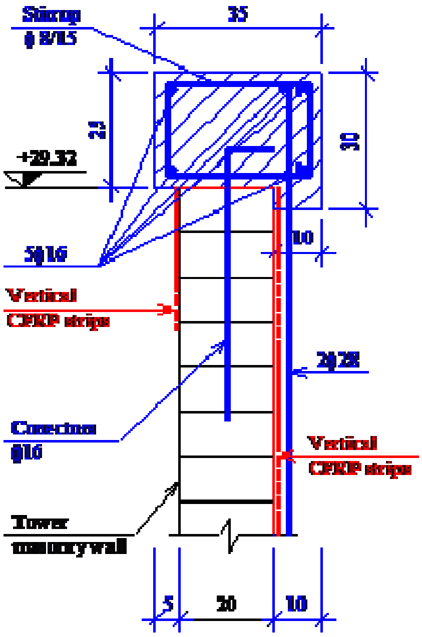


Figure 3.21. Tower strengthening at top section

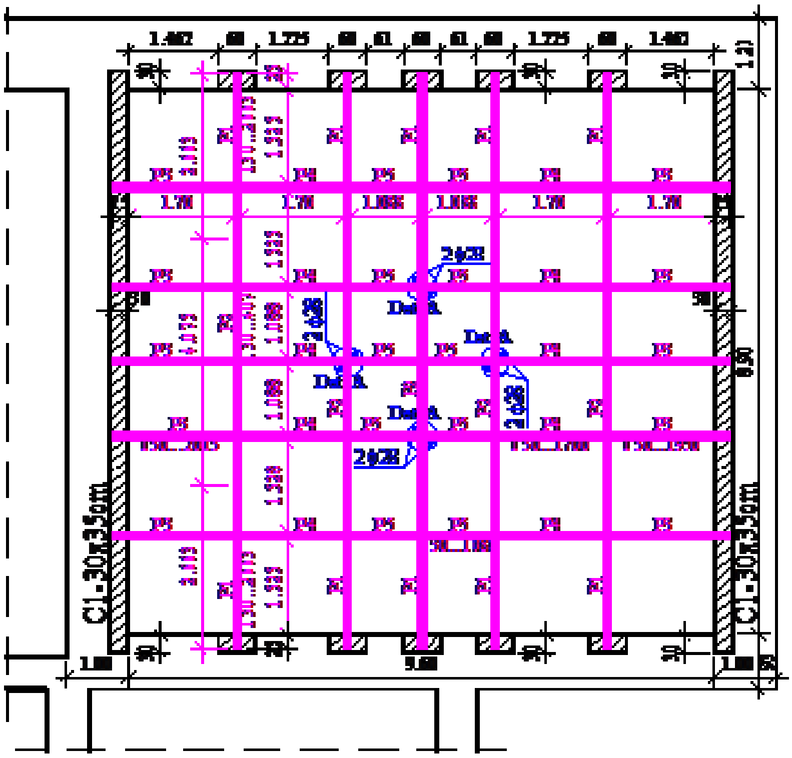


Figure 3.22. Steel beams network for tower stability

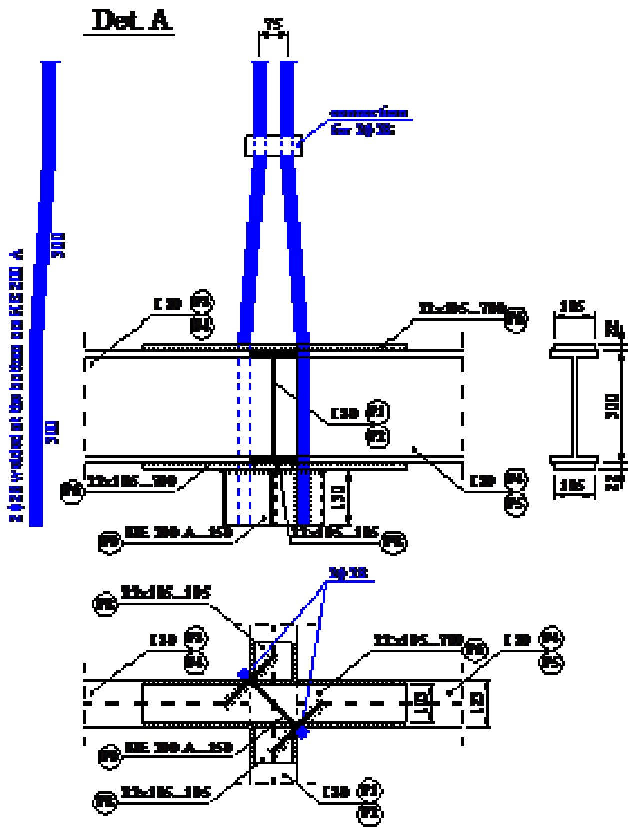


Figure 3.23. Detail of connection between vertical bars and base steel profiles

- in masonry structure, at zones with stresses greater that masonry tensile strength, were placed horizontal RC straps: at the tower – dome crossing; at the base of dome; at the level of steel profiles I 30 network for its embedding into vertical masonry structure;

- on the vertical cracked tower: corroded circular steel rings for confining the tower on outside face were replaced by horizontal CFRP strips (Figure 3.24).

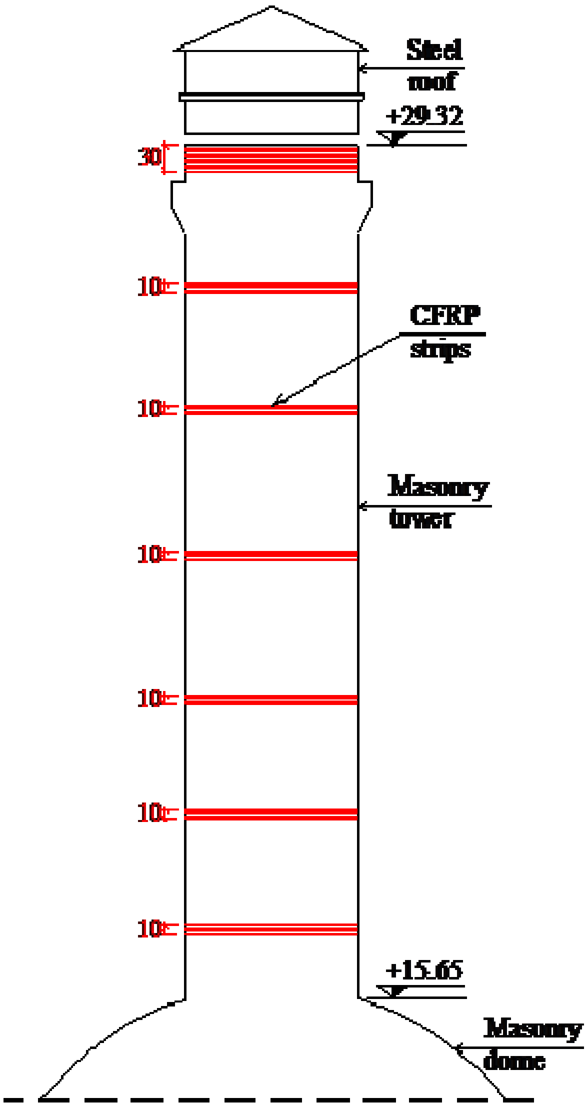


Figure 3.24. CFRP strips for tower confinement

CONSTRUCTION PROCEDURES

The old malting building at the "Timisoreana" Brewery was firstly repaired and rehabilitated. Than, in 2008, the masonry tower structure was strengthened.

In the dome, at the tower base was placed the steel beams network and embedded in the masonry walls by means of reinforced concrete straps (Figures 3.26-3.27).

Figure 3.25. Strengthening of tower structure

Figure 3.26. Steel beams network

Figure 3.27. RC straps for steel beams embedding

The general stability of masonry tower was ensured by vertical reinforcement bars and vertical CFRP wrap on the entire tower height (Figure 3.28).

Vertical reinforcement bars were embedded at the upper side of the tower in a reinforced concrete beam (Figures 3.29-3.30) and welded at the bottom side on steel profiles from the tower base.

Figure 3.28. Vertical rebars and CFRP strips

Figure 3.29. Reinforcing of the tower top beam

Figure 3.30. Reinforced concrete top beam

CONCLUSIONS

The assessment of the old malting building, erected between 1857-1876 at the "Timisoreana" Brewery, emphasized some structural damages and the static and dynamic analysis at different actions showed up major structural vulnerability, mainly due to the period of design and erection.

In order to preserve the old building as architectural monument and to reduce the seismic failure risk, the following strengthening solutions were designed and applied:

- for general stability of masonry tower: vertical reinforcement bars embedded at the upper side of the tower in a RC beam and welded on steel profiles placed in the dome, at the tower base; vertical CFRP wrap on the entire tower height;
- in masonry structure, at zones with stresses greater that masonry tensile strength, were placed horizontal RC straps: at the tower – dome crossing; at the base of dome that supports the tower; at the level of steel profiles for its embedding into vertical masonry structure;
- on the vertical cracked tower due to temperature variations: corroded circular steel rings for confining the tower on outside face were replaced by horizontal CFRP strips.

The strengthening solutions for rehabilitation of historic structure were selected in order to obtain technical and economical advantages: safe behaviour at seismic actions; slight change of overall structural stiffness; easy strengthening technology and short refurbishment period; low rehabilitation cost.

PROPOSAL FOR THE FUTURE ACADEMIC, SCIENTIFIC AND PROFESSIONAL CAREER DEVELOPMENT

The future academic, scientific and professional career development will be focused on the following targets:

- The future goal of the academic career will be the highest rank as Professor in the field of structural concrete and rehabilitation of existing structures, with lectures both at bachelor and master degrees, given in Romanian, English and German.

- Research on rehabilitation of existing structures, with specific targets: new materials ant technologies used for rehabilitation of concrete and/or masonry structures; complex analysis of old masonry structures; technical, economical and sustainable comparison between different classical and modern rehabilitation solutions; experimental research on possibilities of using carbon fibre reinforced polymers CFRP and other modern composites as reinforcement in structural concrete; using of modern strengthening solutions for reinforced concrete and/or masonry structures in industry projects and applications; life service assessment of existing infrastructure (bridges) exposed to different environmental conditions.

- Research on other fields which could be: new materials and technologies for structural concrete (i.e.”textile carbon reinforced concrete”); sustainability of different rehabilitation solutions for reinforced concrete and/or masonry structures; robustness of reinforced concrete and/or masonry structures subjected to special or accidental actions; new sustainable materials obtained by cement replacing, in concrete and mortar, with different recycled materials from various sources; sustainable and environmental friendly solution for houses construction industry (i.e. earth made houses); contribution to structural concrete design codes revisions by theoretical and experimental research (i.e. design at shear forces).

- The research achievements will be accomplished by co-operation with colleagues from own department, faculty and university, and different universities, research institutes and industry partners both from Romania and abroad.
  The results of these researches should be disseminated at scientific conferences and journals both from Romania and abroad.

- In the professional career another goal will be to continue at a managing position within the own department or faculty or university.

THE SCIENTIFIC RESEARCH ACTIVITY

The research activity was developed in different fields as follows:

- Assessment, redesign and rehabilitation of existing concrete and masonry structures aspects: theoretical and experimental assessment of reinforced concrete structures' durability; monitoring, assessment and redesign of existing reinforced concrete structures; redesign of reinforced concrete structures for rehabilitation; rehabilitation of reinforced concrete structures by using composite fibres polymers; modern and efficient solutions for strengthening of concrete and masonry structures.
- High performance construction materials: high performance construction materials derived from recycled materials; concrete with super-plasticizing additives; high performance concrete innovative solution for optimizing the self-compacting concrete microstructure used in prefabricated elements; development of durable and ecological concrete by using minerals additions.

The research activity was performed within many projects as follows:

Director of Romanian National Research Grants:

- <u>Redesign of reinforced concrete structures for rehabilitation</u>, 2000-2001, Grant ANSTI;
- <u>Rehabilitation of reinforced concrete structures by using composite fibres polymers</u>, 2002-2004, Grant CNCSIS MEC;
- <u>Modern solutions for strengthening of concrete and masonry structures</u>, 2005-2007, Grant CNCSIS MEC.

Director of Romanian team of international research grants:

- <u>Valorisation des additions minérales pour la production de bétons écologiques et durables</u>, 2012-2014, Grant WBI – FRS-FNRS Belgium, Partners: University of Liege, Belgium and University Politehnica Timisoara, Romania

European Research Programmes Involved in:

- COPERNICUS project "Recycling of Fly Ash for Producing Building and Construction Materials Base on a New Mineral Binder System", 1995 – 1997;
- COPERNICUS project "High Performance Materials Derived from Industrial Waste Gypsum", 1997 – 1999.

Romanian National Research Grants Involved in:

- Analysis of platforms for industrial chimneys oh 350 m height, 1989;
- Disperse reinforced concrete with glass fibre, 1990 – 1992;
- Optimizing the detailing and reinforcing in the discontinuity regions of reinforced concrete elements by using the strut-and-tie models, 1992 – 1994;
- Behaviour of structural elements at fire action, 1995 – 1997;
- Optimization of design and detailing for reinforced concrete and composite steel-concrete structures, 1998 – 1999;
- Monitoring, assessment and redesign of existing reinforced concrete structures, 1999 – 2000.

Research and/or Design Projects for Industry:

- Design of reinforced concrete structures, monolithic and/or precast structures: hotel, hospital, industrial, office, shopping and/or apartment buildings – 10 projects;
- Design of composite steel – concrete structures: office building – 1 project;
- Design of steel structures: industrial and/or office buildings – 4 projects;
- Design of masonry structures: apartment buildings; houses – 15 projects.
- Rehabilitation by using carbon-fibre-reinforced-polymers CFRP solution of different reinforced concrete structures: industrial buildings, silos, apartment buildings, hotels, etc. – 10 projects;
- Rehabilitation by using reinforced concrete and/or steel profiles jacketing of different structures: concrete structures, masonry structures; industrial, office and/or apartment buildings, silos, etc. – 4 projects.

The results of the research activity were published in different books and papers which could be summarized as follows:

- 5 international books;
- 2 national books;
- 3 manuals for students' lectures;
- 3 books for students' application projects;
- 20 papers published in ISI journals and proceedings;
- 28 papers published in different journals and proceedings – international databases.

THE ACADEMIC ACTIVITY AND PROFESSIONAL STAGES

Education

- Politehnica University of Timisoara, Civil Engineering Faculty 1985 – 1990, Diploma with Merit.

- Politehnica University of Timisoara - PhD degree in Civil Engineering "MAGNA CUM LAUDE", 2000, PhD thesis: "Aspects Regarding Resistance Capacity of Existing Reinforced Concrete Structures at Different Service Life Duration";

Additional Training, Research and Teaching Stages

- Research stage in 1994 at the Civil Engineering Department, "École Normale Supérieure" Cachan – Paris, France: R.C. structures durability - experimental research, finite element analysis;
- Research stage in 1996 at the Civil Engineering Department, Technical University Lisbon, Portugal: structural behaviour (linear and non-linear) of R.C. structures designed according to European Standards EUROCODE 2 and 8 at seismic impact;

- Training stage in 1997 at the Department of Civil Engineering, University of Nottingham, England: structural testing methods and experimental facilities;
- ***fib*** Course "Strengthening with Externally Bonded FRP Reinforcement – Behaviour, Design and Applications", Athens, Greece, 2003;
- Teaching stages in 2007 - 2010 at Civil Engineering Department, Technical University of Munich, Germany: structural concrete, reinforced and prestressed concrete structures;
- Research engineer in 2010-2012 at Department ArGEnCO, University of Liege, Belgium: robustness of reinforced concrete structures, lectures on reinforced concrete structures.

Professional Affiliations

- IABSE "International Association for Bridge and Structural Engineering" – member of Working Commission 4 "Operation, Maintenance and Repair of Structures";
- AICPS "Romanian Association of Civil Engineers Structural Designers";
- WSEAS "World Scientific and Engineering Academy and Society".

Employment

- Politehnica University of Timisoara, Civil Engineering Department, since 1990 to present;

  Since 2015 – Associate Professor for reinforced and prestressed concrete structures and redesign of existing structures;
- Research engineer and lecturer at University of Liege, Department ArGEnCO, 2010-2012;
- Consultant at structural engineering design office Proiecta-List-Invest Ltd, Timisoara, 2001-2010.

DEVELOPMENT OF THE UNIVERSITY CAREER

The future academic, scientific and professional career development will be focused on the following targets:

The future goal of the academic career will be the highest rank as Professor in the field of structural concrete and rehabilitation of existing structures, with lectures both at bachelor and master degrees, given in Romanian, English and German.

The lectures at bachelor degree will consist of: reinforced and prestressed concrete; concrete structures and special concrete structures.

The lectures at the master degree will consist of: redesign of existing structures; rehabilitation of concrete structures.

For industry requirements, new lectures will be proposed in co-operation with different partners from and/or outside university to fulfil to present-day continuous education necessity.

All these lectures should be kept up-to-date according to the design codes and research development by continuous editing and publishing of lectures books and practical design guides.

Young assistants should be employed and involved in the development of these lectures by teaching activities and doctoral research.

In the professional career another goal will be to continue at a managing position within the own department or faculty or university.

The development of scientific research career will be in several fields as rehabilitation of existing structures, new materials, design codes, sustainable development and recycling resources, with specific targets:

- new materials ant technologies used for rehabilitation of concrete and/or masonry structures;
- complex analysis of old masonry structures; technical, economical and sustainable comparison between different classical and modern rehabilitation solutions;
- experimental research on possibilities of using carbon fibre reinforced polymers CFRP and other modern composites as reinforcement in structural concrete; using of modern strengthening solutions for reinforced concrete and/or masonry structures in industry projects and applications;
- life service assessment of existing infrastructure (bridges) exposed to different environmental conditions.
- new materials and technologies for structural concrete (i.e.”textile carbon reinforced concrete”);

- sustainability of different rehabilitation solutions for reinforced concrete and/or masonry structures;
- robustness of reinforced concrete and/or masonry structures subjected to special or accidental actions;
- new sustainable materials obtained by cement replacing, in concrete and mortar, with different recycled materials from various sources;
- sustainable and environmental friendly solution for houses construction industry (i.e. earth made houses);
- contribution to structural concrete design codes revisions by theoretical and experimental research (i.e. design at shear forces).

- The research achievements will be accomplished by co-operation with colleagues from own department, faculty and university, and different universities, research institutes and industry partners both from Romania and abroad, like: INCD URBAN INCERC, Timisoara Branch; INCD EMC, Timisoara Branch; University of Liege, ArGEnCo Department; Technical University Munich, Massivbau Lehrstuhl; University Nova Lisbon, Civil Engineering Department.

In order to improve and develop the research activity, the specific tasks will be:

- involving more intensely in promoting the results of the scientific research performed, either as author or co-author at papers to be published in national and international journals with high impact factors;

- constant publication of papers in scientific journals indexed ISI and/ or BDI;

- have a sustained activity by taking part at scientific competitions, national and international by: closer collaboration with research institutions and other profile faculties in Romania on subjects specific to our research field. Develop a multidisciplinary research team, with various specialists from numerous institutions, able to respond more efficiently to the call for scientific competition; identify and promote several common research themes with other institutes and faculties, stating from the similar or complementary activities developed; identify and promote certain research themes in partnership with private institutions which have scientific research as activity object; identify and continuous initiate partnerships at institutional level, to take part at competitions financed by Governmental Institution or International Authorities; involve students in the research activity.

Synthesis of the personal development strategies

The future post habilitation research will be focused on the following proposed activities:

- Research on rehabilitation of existing structures, with specific targets: new materials ant technologies used for rehabilitation of concrete and/or masonry structures; complex analysis of old masonry structures; technical, economical and sustainable comparison between different classical and modern rehabilitation solutions; experimental research on possibilities of using carbon fibre reinforced polymers CFRP and other modern composites as reinforcement in structural concrete; using of modern strengthening solutions for reinforced concrete and/or masonry structures in industry projects and applications; life service assessment of existing infrastructure (bridges) exposed to different environmental conditions.

- Research on other fields which could be: new materials and technologies for structural concrete (i.e.”textile carbon reinforced concrete”); sustainability of different rehabilitation solutions for reinforced concrete and/or masonry structures; robustness of reinforced concrete and/or masonry structures subjected to special or accidental actions; new sustainable materials obtained by cement replacing, in concrete and mortar, with different recycled materials from various sources; sustainable and environmental friendly solution for houses construction industry (i.e. earth made houses); contribution to structural concrete design codes revisions by theoretical and experimental research (i.e. design at shear forces).

Specific future activities will be:

- be director of at least one national contract and a team member at an international one;
- continue improving of lecturing materials – at least one book to be published and also online support materials;
- continue the activity of publishing at least 4 papers in ISI journals with high impact factor in the research field; presenting the research results at different scientific events, national and international conferences: minimum two papers per year;
- continue to take part in different national and international scientific committees, by reviewing papers;
- continue to organize various national and international scientific events;
- be the co-author, with colleagues, of one lecture book;
- continue to a managing position.

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