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CONSTRUCŢII. ARHITECTURĂ

ASSESSMENT AND STRENGTHENING OF EXISTING *RC* FRAMED STRUCTURES

BY

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Existing structures built before the 1970s are gravity load designed with inadequate lateral load resistance because earlier codes specified lower levels of seismic loads and many of these structures are still in service beyond their design life. On the other hand, some deterioration of component parts of buildings is encountered in old structures due to the actions of different hazard factors.

Theoretical aspects on the risk assessment of the reinforced concrete structures are presented. The earthquake capacity ratio is analysed for reinforced concrete framed structure. More attention is paid to the seismic shear force capacity and some new procedures are introduced to estimate the earthquake capacity of existing structures.

The assessment, rehabilitation and redesign were performed on an industrial *RC* framed structure. The main problems comprised local damage due to industrial exploitation of some structural elements (concrete cover dislocated over a large surface, complete corrosion of many stirrups, deep corrosion of main reinforcement, some broken reinforcement) and weak reinforcement of columns and beams under seismic action. The redesign and rehabilitation of the *RC* structure was performed for both types of damages by using reinforced concrete jacketing solutions.

1. Characteristics of Existing Structures

The vulnerability of existing structures may be due to structural system weaknesses and specific detailing. Structural weaknesses are characterized by various irregularities and discontinuities or by general structural vulnerabilities namely:

1. Irregularities in the vertical direction of the buildings: irregular distributions of the stiffness at lateral displacement; strength discontinuities; mass irregularities.

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 some structural elements.

Also *RC* structures are characterized by common non-ductile detailing:

a) inadequate beam and column bending and shear capacity;

b) inadequate confinement of the potentially plastic hinges of the columns and beams as well as of the boundary elements of *RC* frame-wall systems;

c) inadequate reinforcement of the *RC* frame in the building longitudinal direction.

2. Assessment and Analysis of Existing *RC* Structures

According to the Romanian Code for seismic design P100-92 [1] as well as to the other norms, the design of structures to resist earthquake is based on the next design procedures and calculation methods:

(i) 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.

(ii) 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 calculating the earthquake capacity ratio

$$R = \frac{S_{\text{cap}}}{S_{\text{nec}}}, \tag{1}$$

where: S_{cap} is the 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 at the present-day level of seismic action.

The effect of different actions, ordinary and special (Fig. 1), on the structural safety and S_{cap} is presented in Fig. 2 for the service live of a structure.

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

Fig. 1.– Types of actions.

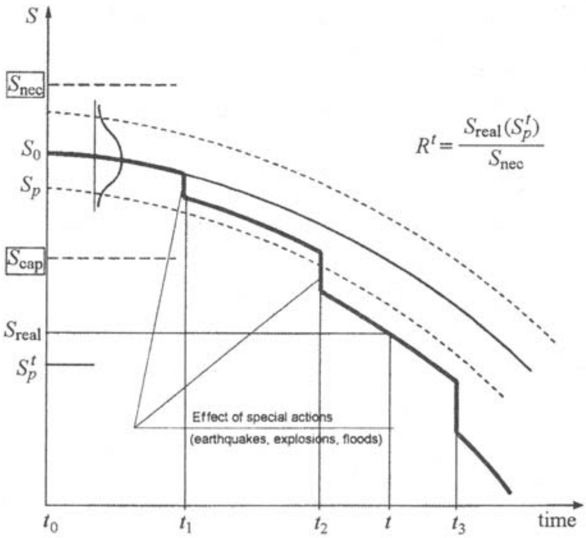

Fig. 2.– Effect of different actions on structural safety.

The authors have used for design the 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.

For the damage control of structural members at seismic design the authors proposed and used the following methods:

a) the plastic hinges procedure;

b) the stiffness modification procedure.

2.1. Plastic Hinges Procedure

The procedure consists in the analysis of a modified static scheme of the *RC* framed structure.

Plastic hinges are to be introduced in the cross sections with corroded reinforcement and strength degradation. Simplified, mechanical hinges with applied bending moments loads equal to the real cross section bearing capacity are introduced. A new static scheme is obtained and analysed by using certain computer programs.

A case study at seismic actions was performed on a *RC* framed structure presented in Fig. 3.

Plastic hinges were introduced (Fig. 4) in the cross sections at $L/4$ of beams with assumed corroded reinforcement at 2/3, 1/3 and 0 from initial reinforcement area, A_a.

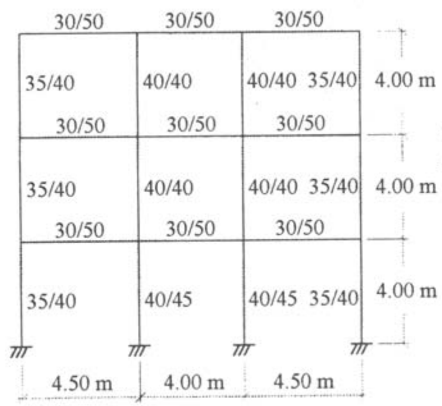

Fig. 3.– *RC* framed structure.

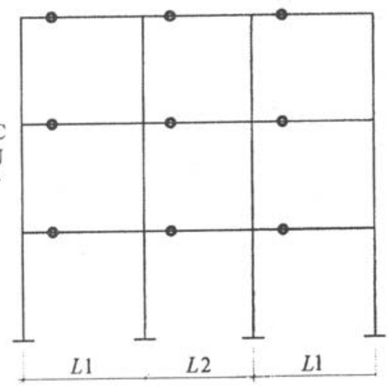

Fig. 4.– Position of plastic hinges.

The influence of beam reinforcement corrosion and plastic hinges on the values of earthquake capacity ratio, R, calculated for beams and columns is presented in Fig. 5.

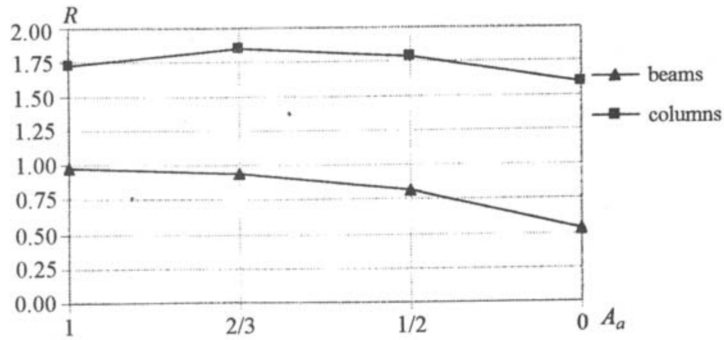

Fig. 5.– The influence of beams reinforcement corrosion progression on R value.

2.2. Stiffness Modification Procedure

The procedure is based on the influence of stiffness degree calculated as function of materials characteristics: elasticity modulus (E_a, E_b) and area (A_a, A_b) of reinforcement and concrete. For instance, at bending with/without axial force the stiffness is given by the formula

$$K = \frac{E_a A_a \beta h_0^2}{1 \mp (\bar{\xi} - \bar{x}_A)/\bar{e}_0}, \tag{2}$$

where: E_a represents the elasticity modulus of reinforcement; A_a – area of tension reinforcement; $h_0(d)$ – effective depth of reinforced concrete cross section; $\overline{e}_0 = e_{oc}/h_0(d)$ – relative eccentricity of axial force, N; negative sign for compression of N; $\overline{e}_0 = \infty$ for pure bending; $\overline{x}_A = x_A/h_0$, where x_A is the distance between reinforcement A_a and centroid of the cross section; $\beta = \zeta(1-\xi)/\psi$, where $\xi = x/h_0$, $\zeta = z/h_0 \cong 1 - \xi/3$ and ψ is given in Table 1.

Table 1
Values of ψ

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	

Finite element method is used and there is possible to use 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 RC structure utilisation, *e.g.* after a serious degradation of one or several structural members.

The study at seismic actions was performed on the RC framed structure represented in Fig. 3. Modified stiffness, K_m, at values 2/3 and 1/2 from initial stiffness K, were taken into account for beams and columns according to Fig. 6.

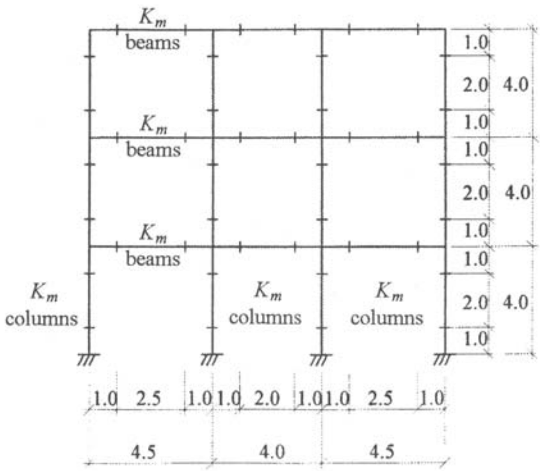

Fig. 6.– Modified stiffness of beams and columns.

In Fig. 7 the influence of stiffness, K, modification on the earthquake capacity ratio, R, is represented.

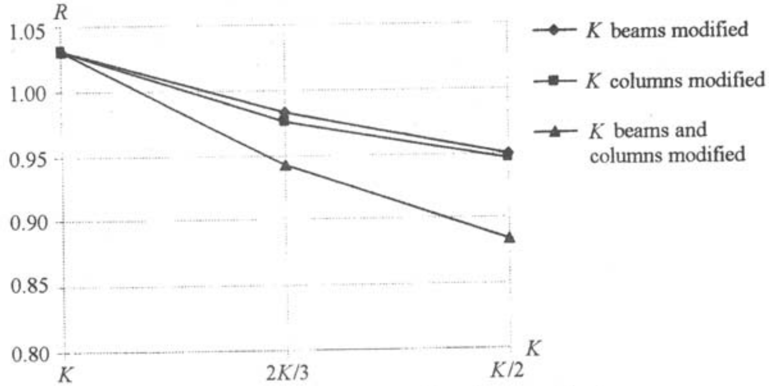

Fig. 7.– The influence of stiffness modification on R values.

3. Assessment and Strengthening Example

The Timişoreana Brewery, a *RC* framed structure having five storeys and a tower of nine storeys (Fig. 8), has been assessed and rehabilitated.

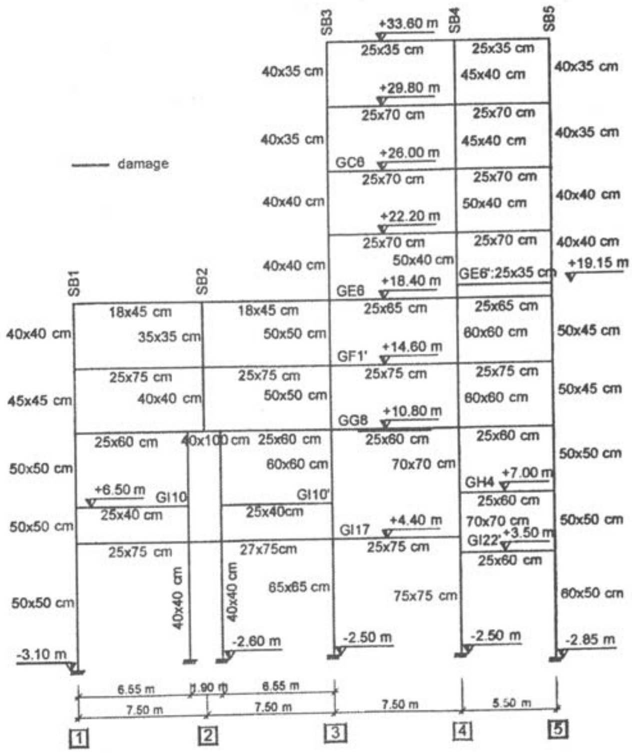

Fig. 8.– Longitudinal frame of Timişoreana Brewery.

The main problems comprised local damage of some structural elements (Fig. 9) and weak reinforcement of columns and beams.

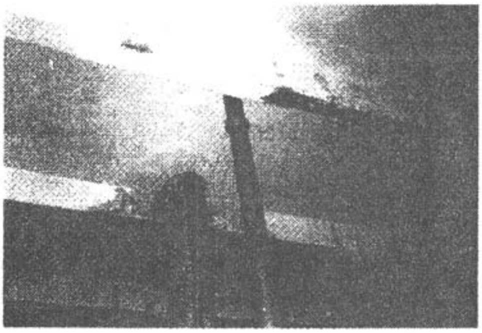

Fig. 9.– Damage of secondary beams.

Local damages were observed and assessed at slabs, main girders, secondary beams and columns. The damage consisted of: concrete cover dislocated over a large surface; complete corrosion of many stirrups and deep corrosion of main reinforcement; some broken reinforcement. The damaged areas were located in the middle of the span for secondary beams (Fig. 9) and on potentially plastic hinge regions of the main girders at level +10.80 m, where dangerous inclined cracks were also detected.

Such damages were caused by the action of chloride ions (Cl^-) from salt solution which was stored on the floors as well as of CO_2, RH$\approx$ 80% and $T > 40°C$.

Weakness of reinforcement was deduced from the structural analysis. The initial analysis, performed in 1960, was realized 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 observed at some main girders.

In order to quantify the influence of structural damage in seismic assessment the stiffness modification procedure was used. Due to reinforcement corrosion of transversal and longitudinal beams reduced values of stiffness, K, were taken into account for the structural analysis. The results of earthquake capacity ratio, R, for initial not-damaged structure and the damaged structure and different failure assumptions are presented in Table 2.

Table 2
Earthquake Capacity Ratio, R

Initial structure		Damaged structure	
Assuming failure of beams	Assuming failure of columns	Assuming failure of beams	Assuming failure of columns
Transversal frame			
0.423	0.388	0.417	0.388
Longitudinal frame			
0.208	0.184	0.184	0.182

From the performed analysis results it can be noticed the decrease of values R for the damaged structure, which shows the importance of analysing real structures. Also, the stiffness modification procedure proves to be useful.

According to Romanian design codes for existing structures, in case when $R \le R_{\min} = 0.50$ for common buildings, strengthening is necessary.

Several strengthening solutions were proposed and analysed. The structural re-design was performed by finite element method on the spatial structure at present-day level seismic action.

Due to the weakness of the main reinforcement in columns SB3 and SB4, especially, in the building longitudinal direction the strengthening solutions, by *RC* coating, were analysed. The obtained results are given in Table 3.

Table 3
Redesign of Strengthened Structure: Efficiency of Different Solutions

Transversal seismic action	Longitudinal seismic action
Strengthening solution A: a) column SB3 from +4.40 m to +10.80 m b) column SB4 from foundation to +10.80 m	
$R = 0.86$	$R = 0.83$
Strengthening solution B: a) column SB3 from foundation to +10.80 m	
$R = 0.55$	$R = 0.70$

In both strengthening cases $R > R_{\min} = 0.50$. Finally, due to economic reasons, was chosen the strengthening solution B of column SB3.

Rehabilitation of the *RC* structure was performed for both types of damages. The main girder and secondary beams were strengthened by coating with reinforced concrete. New 4 ∅ 25 mm reinforcement were placed at the bottom of each secondary beam; 6 ∅ 25 mm reinforcement were placed at 15 cm from the lower side of the main girder with stirrups at 15 cm (Figs. 10 A and 10 B).

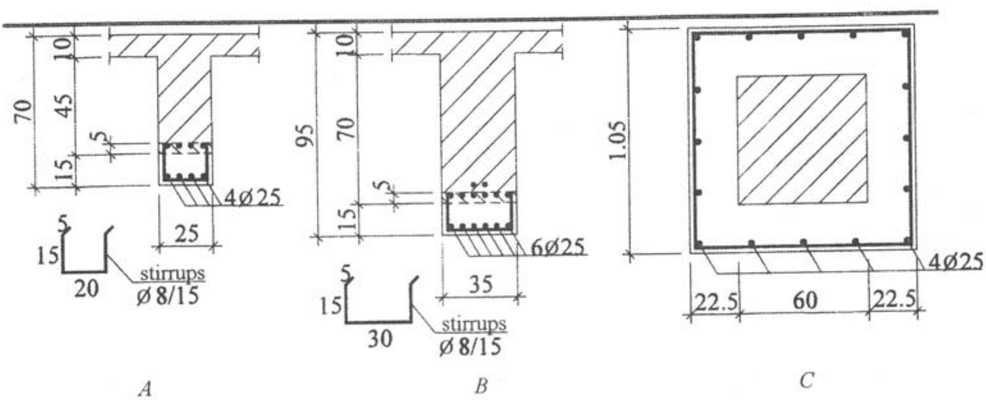

Fig. 10.– Strengthening solutions of *RC* structure:
A – secondary beam; B – main girder; C – column.

The column SB3 was strengthened for both local damage and weakness of reinforcement. The coating with reinforced concrete was used over two storeys and consists of 16 ∅ 28 mm reinforcement and 22.5 cm concrete depth on all four sides (Fig. 10 C). A new and appropriate foundation, over and around the existing one, was erected for secure fixing of the column reinforcement.

4. Conclusions

The main ideas which emerge from this study, are summarized below:

1. Theoretical considerations on the risk assessment of the reinforced concrete structures were presented. The earthquake capacity ratio ($R = S_{\text{cap}}/S_{\text{nec}}$) is analysed for reinforced concrete framed structure. More attention is paid to the seismic shear force capacity, S_{cap}, and some new procedures are introduced to estimate the earthquake capacity of existing structures.

2. For the damage control of structural members in seismic design the authors proposed and used the following methods:

a) *The plastic hinges procedure*. Consists in the analysis of a modified static scheme of the RC framed structure with plastic hinges introduced in the cross sections with corroded reinforcement and strength degradation.

b) *The stiffness modification procedure*. Is based on the emphasizing of stiffness calculated as function of materials characteristics: elasticity modulus and area of reinforcement and concrete. Finite element method is used and there is possible to use different values of stiffness, K, for each element. The procedure shows the opportunity of changing the value of K at any time of RC structure utilization or degradation.

3. The assessment, rehabilitation and redesign were performed on an industrial RC framed structure Timişoreana Brewery. The main problems comprised local damage due to industrial exploitation of some structural elements (concrete cover dislocated over a large surface, complete corrosion of many stirrups, deep corrosion of main reinforcement, some broken reinforcement) and weak reinforcement of columns and beams built according to inadequate norms for seismic action.

The redesing and rehabilitation of the RC structure was performed for the both types of damages by using reinforced concrete jacketing solutions.

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EVALUAREA ŞI CONSOLIDAREA STRUCTURILOR ÎN CADRE DIN BETON ARMAT

(Rezumat)

Structurile de beton armat existente în număr mare sunt, multe dintre ele, proiectate corespunzător la acţiuni gravitaţionale dar având o capacitate portantă insuficientă la acţiuni orizontale de tip seismic. Numeroase construcţii existente au depăşit durata de exploatare proiectată. Pe de altă parte au apărut şi degradări ale elementelor structurale datorate diferiţilor factori.

În prima parte a lucrării sunt prezentate aspecte teoretice privind evaluarea riscului seismic prezentat de structurile existente în cadre din beton armat. Se propun şi se detaliază noi procedee de evaluare a capacităţii portante la diferite acţiuni a structurilor existente.

În partea a doua a lucrării se prezintă evaluarea capacităţii portante, consolidarea şi reproiectarea unei construcţii industriale cu structura în cadre din beton armat. Construcţia a prezentat probleme de deteriorare locală a unor elemente structurale datorită tehnologiilor industriale (beton exfoliat, armături longitudinale şi etrieri corodaţi sau rupţi) şi probleme de capacitate portantă insuficientă la acţiuni seismice. În final, s-a realizat reproiectarea şi consolidarea structurală a construcţiei avariate.