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Study on seismic performance of existing buildings in Romania

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1. Introduction

Romania is one of the earthquake prone countries in the Eastern Europe. The first generation of Romanian seismic design code was issued in 1963, as P13/63. The buildings build before the first issue of the seismic design code and also the buildings designed before 1978, when P100-78 earthquake resistant design code was prepared and enforced, should be evaluated and most of them retrofitted in order to comply with the current code provisions.

The vulnerable buildings in Romania can be ranked in two major categories:

1. Mid rise and high rise buildings built prior to 1945
2. Mid rise and high rise buildings built after 1945 and prior to March 4, 1977 earthquake

The two categories of vulnerable buildings have very distinctive structural features and consequently the approach for improvement of retrofitting techniques and methods must be different.

Some explanations for each building categories follows.

1. *Mid rise and high rise buildings built prior to 1945*

No seismic design is incorporated in the structural system of these buildings. The layout of the buildings and of the structural systems is different from one building to the other.

There are two building typologies in this category:

a. *Unreinforced masonry buildings with wooden slabs (most vulnerable) or with RC/masonry vaults slabs*

   The main vulnerability of this building typology comes from the lack of any reinforcement to improve the behavior of masonry walls to lateral loads. The situation is worsening if the slabs are made of wood. The main failure mechanisms include in plane shear failure of the walls and/or out of plane failure of the masonry walls. In 1977 earthquake, out of 28 buildings built before 1940 collapsed in Bucharest 5 were unreinforced masonry buildings.

   The main drawbacks of this building typology are:

   - Lack of shear strength of structural walls
   - Lack of capacity of horizontal structural system to transfer lateral loads

- Lack of proper connections between the masonry walls in different directions and between the masonry walls and the slabs.

The usual retrofitting solution consists in the provision of RC jackets on one side (or both sides) of masonry walls and provision of RC slabs after demolishing the wooden slabs or provision of RC slabs cast atop the masonry vaults slabs. The new vertical RC elements run along the height of the building and new foundations are provided for them.

b. *RC columns and beams*

The RC columns and beams are not providing earthquake resistant frames. This is because of lack of continuity of structural system in vertical and horizontal directions and because the joints are not able to transfer the lateral loads. The concrete is of very poor quality and strength (<10 MPa) and the gravity axial loads in the lower columns are very high. In 1977 earthquake, out of 28 buildings built before 1940 collapsed in Bucharest 23 were in this building typology.

The main drawbacks of this building typology are:

- Lack of strength (low capacity in shear and bending)
- Lack of stiffness for the overall structural system
- Lack of ductility.

The most common retrofitting solution consists in provision of RC jackets for columns and for adjacent beams and provision of new RC shear walls. The new vertical RC elements run along the height of the building and new foundations are provided for them.

The main disadvantages of the previously presented retrofitting solutions such as:

- Long period of construction
- Major disturbance for the residents of the buildings.

2. *Mid rise and high rise buildings built after 1945 and prior to March 4, 1977 earthquake*

Some earthquake resistant design was used for the development of these buildings after 1963 and some ductility rules were enforced and used after 1970. The main building typologies within this category are:

a. *RC frames*
b. *RC shear walls*
c. *RC soft story buildings (more precisely, soft and weak groundfloor buildings)*

a. *RC frames*

The major vulnerabilities of this building typology come from:

- Insufficient shear capacity in columns
- Insufficient overall stiffness
- Insufficient ductility

The usual retrofitting solution consists in jacketing of RC vertical elements and, sometimes provision of RC shear walls.

b. *RC shear walls*

The major vulnerabilities of this building typology come from:

- Insufficient shear capacity
- Insufficient bending capacity (not so often)

The usual retrofitting solution consists in jacketing of RC vertical elements.

c. *RC soft and weak groundfloor buildings*

This building typology consists of a dual structural system in the vertical direction, i.e. RC columns in the groundfloor and RC shear walls in the upper stories. The major vulnerabilities of this building typology come from:

- Concentration of most of seismic lateral displacement in the groundfloor
- Concentration of most of seismic induced energy in the groundfloor
- Insufficient ductility for RC columns in the groundfloor
- Insufficient shear capacity for both RC columns and RC upper shear walls
- Insufficient overall stiffness.

One of the new buildings collapsed in Bucharest during March 4, 1977 earthquake was of this typology.

The usual retrofitting solution for this building typology consists in continuation of upper RC shear walls in the groundfloor and sometimes in the provision of new shear walls along the height of the building.

For mid rise and high rise buildings built after 1945 and prior to March 4, 1977 earthquake, new/alternative retrofitting techniques are adequate since the existing structural system is more regular and more engineered, the quality of the concrete is good and the seismic capacity is definitely higher than of the buildings built before 1940. The structural system of the buildings built after 1945 and prior to March 4, 1977 earthquake is appropriate to be integrated with the new retrofitting systems such as steel bracings, exterior mega frames, seismic dampers and so on. This category of buildings gives the opportunity to the designer to implement with fewer disturbances and in a short period of time the modern retrofitting techniques.

Content of the Report

Chapter 2 of this study summarizes the principle and procedures of seismic evaluation methods of existing buildings in Romania according to chapter 11 of P100-92 code.

In chapter 3 some evaluation methodologies, proposed to be used in Romania, are presented.

Chapter 4 presents the current retrofitting techniques used in Romania for existing vulnerable buildings. The most widely used retrofitting techniques consist in inserting RC elements in the existing structures or members jacketing as it will be detailed in Chapter 4.

Chapter 5 presents a case study of the evaluation process of one existing building and also three solutions that may be used to retrofit that structure.

2. Current enforced procedure for buildings evaluation

2.1. General background

The existing buildings are subject, according to the legal provisions in force, to surveys aiming to assess the level of protection against seismic actions. According to the regulations in force, only certified expert engineers (“technical experts”) assess the level of protection of the existing buildings. The technical expert is responsible for the way in which the protection level was evaluated, as well as for the proposed decision of intervention. The responsibility for the carrying out of the interventions falls upon the owner(s) of the building.

By their nature, the operations of assessing the protection level of the existing buildings, of establishing and carrying out the intervention works require higher levels of education, experience, professional and moral integrity of the engineer who carries out such operations, than those necessary for the current design of new buildings. Working in the field of the “building pathology”, the expert engineer has to face completely different, unconventional situations generated by the necessity to correctly understand the notions related to the conception, design, maintenance, preservation, response to seismic motions or other kinds of actions (gravity actions, soil settlements, temperature variations, corrosion etc.) of the building.

The enforced provisions concern the investigations to be carried out only between seismic events.

2.2. Enforced evaluation methods

The enforced evaluation methods employed in P100/92 in order to asses the level of protection against seismic actions of the existing buildings may be classified as follows:

- E_1 – qualitative assessment method;
- E_2 – analytical assessment methods (based on calculations); they are divided into three categories:
- E_{2a} – current calculation methods,
- E_{2b} – static post-elastic calculation methods,
- E_{2c} – dynamic post-elastic calculation methods

As a rule, the combined application of the E_1 qualitative method and E_2 current calculation method is compulsory.

When the strength and ductility characteristics of the materials or of the foundation soil of existing buildings are not available, or if it is necessary to identify the areas where discontinuities, degradation or uncontrolled bonds have occurred, the expert may decide whether or not to apply non-destructive tests.

2.2.1. Qualitative evaluation method

The qualitative evaluation is based on the inspection of the building as a whole and in detail and on the examination of:

- the structural and architectural design of the building;
- the plotting of the building, its members and details considered significant for the assessment of the protection level, whenever the original design is not available or when the building of the structure fails to comply with the design, or when the building has suffered structural transformations during its service life without any purpose-made documentation;
- the designs and documentation on which previous interventions were based, as well as other information on the history of the structure;
- information on the building behavior during previous seismic events;
- considerations related to the norms on which the design of the building was based or, if necessary, related to the date practices compared with the present specifications in force;
- the plotting of the possible damage and deterioration;
- the building as a whole and in detail.

The object of the qualitative assessment is the structural system as well as the non-structural internal or external members that are likely to cause accidents during seismic events (partition walls, parapets, ornaments, blind walls, chimneys, etc.).

2.2.2. Quantitative evaluation method

The quantitative evaluation method aims at:

- determining the conventional capacity to bear the seismic loads – ”S_{cap}” – of the inspected building;
- identifying highly vulnerable members/areas of the structure;

- checking up the compliance with the criteria of *ductility* and *brittle crush prevention*;
- determining the structure's stiffness to lateral displacement.

2.2.3. Level of seismic protection and seismic risk classes

The value of the *nominal level of protection against seismic actions* – "R" – is determined on the basis of the conventional capacity to bear the seismic load "S_{cap}"; "R" is given by the capacity to demand ratio:

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

The conventional capacity to bear the seismic load "S_{cap}" is the value of the seismic load that leads, together with the gravity loads, to the achievement of the resistance capacity in the critical sections (areas) of the structure. The critical sections (areas) shall be indicated by the expert on the basis of engineering judgment.

In order to determine the value of "$S_{required}$" in case of existing buildings, the values of seismic force reduction factor "Ψ" is established by the technical expert on the basis of the analysis of the characteristics of the building.

Based on the sectional stress under conventional capacity to bear the seismic load "S_{cap}" the fulfillment of the criteria of ductility and brittle crush prevention shall be checked up using the procedures for new constructions.

The fulfillment of the conditions of *stiffness to lateral displacement* shall be checked up by comparing the structure displacements under the conventional capacity to bear the seismic load "S_{cap}" with the allowable displacements for new constructions. This checking has an informative character, aiming to quantify the general performance of the structure to seismic events.

The entire survey activity is synthesized by the ranking of the building into classes of seismic risk taking into account the seismic zone where the building is located and the following criteria regarding the type of structure, the behavior of the building in operation and under seismic actions:

- the category of the structural system;
- the general conformity of the building from the point of view of the expected seismic response;
- the nominal level of protection against seismic actions (index R)

- the presence of weak zones, from the point of view of the resistance capacity in relation to the requirements, in the structural members playing a major role in taking over the seismic strain;
- the probable nature of the yield of the main structural members that are vital for the stability of the building: ductile, with limited ductility, brittle;
- the method of solving the constructive details of sections (for instance: the cross reinforcement with cross-ties in the potential plastic zones, reinforcement bars' anchorage, their splicing, etc.);
- the age of the building (the year of erection);
- the number of significant earthquakes to which the building was subject;
- the structural damage experienced after earthquakes;
- the condition of non-structural members;
- the height and the mass of the building, etc.

Four classes of seismic risk are established as regards the seismic risk, i.e. the possible effects of certain earthquakes, characteristic for the site, on the existing buildings on that site:

Seismic Risk Class I, RsI, comprising buildings with high risk of collapse in case of earthquakes with intensities corresponding to the design seismic zones (design seismic rank);

Seismic Risk Class II, RsII, comprising buildings for which the risk of collapse is low, but for which major structural damage is expected on the occurrence of the design seismic rank;

Seismic Risk Class III, RsIII, comprising buildings which are expected to suffer structural damage which does not significantly affect the structural safety, but the damage of their non-structural members can be significant;

Seismic Risk Class IV, RsIV, comprising buildings for which the expected seismic response is the same with that of the new buildings, designed on the basis of the design codes in force.

2.2.4. Annex: seismic design force according to P100/92

In order to assure a better understanding of the seismic evaluation method we provide here the formula for computing the design shear force for a new building.

The relations further provided are used in determining the equivalent static loads, used in engineering analysis, required by the common design method. These loads take into account, in a simplified and implicit way, the effects of dynamic behavior and post-elastic deformation phenomena.

The horizontal seismic loads, acting on a structure, are determined for each natural mode of vibration. If the natural vibrations occur in one plane, the resultant of horizontal seismic loads (base shear force), corresponding to the ground motion direction and to the „r"-the vibration mode, is determined using further relations:

$$S_r = c_r G \text{ where: } c_r = \alpha \cdot k_s \cdot \beta_r \cdot \psi \cdot \varepsilon_r \text{ where:}$$

1) c_r, the overall seismic coefficient, corresponding to the „r"-the vibration mode;

2) G **,** resultant of gravity loads for the whole building;

3) α , importance coefficient of the building according to the classes of importance. This factor differentiates the protection level of the building depending on the importance classes, Table1. Based on the performance criteria the buildings are divided into four importance classes, Table 2.

Table 1: Importance coeficient for classes of importance for buildings

Classes of importance			
I	II	III	IV
1.4	1.2	1.0	0.8

Table 2: Classification of buildings according to importance classes

Class I	Buildings of vital social importance, whose functionality during and immediately after earthquakes should be fully granted:
Class II	Very important buildings requiring a limitation of damage, keeping in view its potential consequences:
Class III	Normal importance buildings (not falling into classes I, II, or IV)
Class IV	Reduced importance buildings

4) "k_s" coefficient represents the ratio between the peak ground acceleration of the seismic motion (with an average recurrence period of about 50 years), corresponding to the seismic zone, and the gravity acceleration. The "k_s" coefficients are supplied in Table 3, according to the seismic zones described on the code map.

Table 3: k_s coefficient

Seismic zone	k_s
A	0.32
B	0.25
C[1]	0.20
D	0.16
E	0.12
F	0.08

5) amplification factor, Figure 1 ("βr") is determined according to the natural oscillation periods ("Tr") of the building and to the local seismic conditions characterized by corner periods ("Tc") using the following relations:

$$\beta_r = 2.5 \quad \text{for} \quad T_r \le T_c$$

$$\beta_r = 2.5 - (T_r - T_c) \ge 1 \quad \text{for} \quad T_r > T_c$$

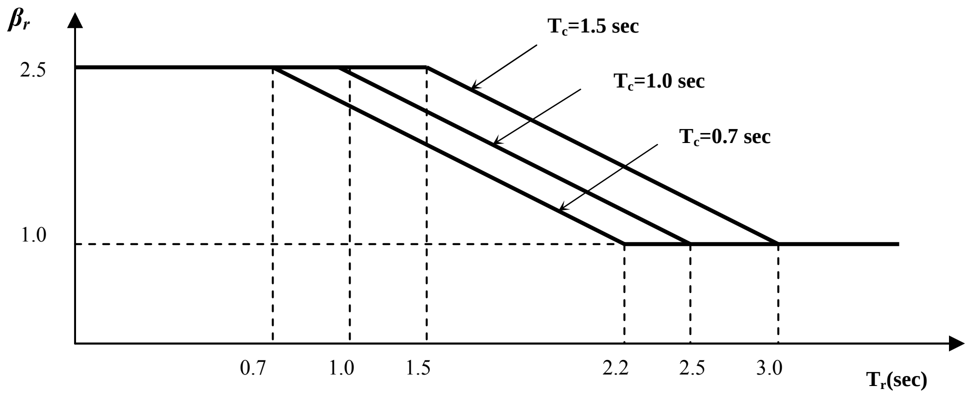

Figure 1: Amplification spectra

6) coefficient of reduction of seismic forces ("ψ") due to the structural ductility, to the redistribution of the efforts and to the damping effects, other than structural is provided in the following Table 4.

7) The equivalence coefficient between the actual system and the system with one dynamic degree of freedom for the vibration mode „r", „ε_r " is determined using the following relation;

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[1] Bucharest belongs to seismic zone C.

$$\varepsilon = \frac{\left[\sum_{k=1}^{n} G_k u_{kr}\right]^2}{G\sum_{k=1}^{n} G_k u_{kr}^2} \quad \text{where:}$$

- u_{kr} the „r"-th eigenvector component corresponding to the „k"-th freedom degree;

- G_k the resultant of gravity loads at ‚k" level $G = \sum_{k=1}^{n} G_k$;

The „u_{kr}" eigenvectors, as well as the „T_r" natural periods are determined using structural dynamics methods.

Table 4: The values of ψ coefficient according to structural type

Structure type	ψ coefficient
Reinforced concrete structures	
1 Multistory frame structures:	
- with infill walls designed as structural members...........................	0.25
- the infill walls are not considered structural members..................	0.20
2 Industrial halls and other one story structures:	
- with stiff beam-columns joints..	0.15
- with hinged joints...	0.20
3 Buildings with structural walls...	0.25
4 Structures made of walls, columns and flat-slabs (no beams)............	0.30
6 Elevated tanks..	0.35
Masonry structures	
1 Structures made of masonry structural walls with reinforced concrete boundary elements (spandrel beams and columns:.................	0.25
2 Structures with plain masonry structural walls.................................	0.30

2.3. Retrofitting solutions

When substantiating and deciding the intervention, as a rule, two solutions shall be brought forward: a minimal one and a maximal one.

The minimal solution will focus on preventing the collapse of the building as well as other phenomena that can cause serious injuries or human life loss in case an earthquake occurs that has the same characteristics as the one considered in the seismic design according to the Code P100-92. For this purpose, the intervention will provide the adoption of all necessary measures that can prevent the total or partial collapse of the building, as well as the raising of the nominal level of building protection against seismic actions.

Table 5: Recommended minimum values of the minimal level of protection against seismic actions Rmin for the existing buildings

Importance class of the building	I	II	III	IV
R_{min}	0,70	0,60	0,50	0,50

The retrofitting method shall be substantiated by means of general calculation in relation to the way the structural system withstands and transfers the loads to the ground.

The compatibility of the associated bearing capacity (bending moment-axial force, bending moment-shear force) shall be checked, as well as the correlation of the strength capacity of the superstructure, the infrastructure and the foundation soil.

The retrofitting methods shall be applied and the addition of new structural members shall be done such as to prevent any structural sensitivity (non-uniform distribution of the stiffness along the horizontal and vertical axes, floor overloading, etc.).

Sudden modifications of the longitudinal sections of the structural members shall be avoided; they could lead to concentrations of unfavorable internal stresses, such as, for instance, unfavorable changes of the ratio “bending moment/shear force” or “short column” effects in the reinforced concrete structures.

This solution is described in the survey report and detailed in sketches and drawings. The expert engineer (who drew up the survey report and the decision of intervention) certifies the fact that the intervention works design fully complies with the concept in the intervention decision. The expert engineer is, finally, responsible for the entire development of the process of assessment/survey/ choosing the intervention solution/drawing up the intervention works design including the execution details

The minimal intervention measure concerns the prevention of the total or partial collapse of the existing buildings, rather than ensuring functionality similar to that of new buildings.

The maximal measures are those that ensure – under the specific technical and economic conditions of a particular building – the enhancement of the protection level to attain values similar to or even higher than those provided for the new buildings.

2.4. Conclusions

The current enforced evaluation procedure gives a lot of decision freedom to the evaluation engineer (technical expert). Although it presents the basic idea of analytical evaluation procedures they are not at all detailed in establishing S_{cap} and $S_{required}$. In the next chapter some evaluation procedures are proposed.

3. Evaluation methods proposed to be used in Romania

In Romania the evaluation of the building is usually made under the provisions of the code P100/92. For detailed evaluation procedures Technical University of Civil Engineering of Bucharest, Reinforced Concrete Department prepared a draft version of “Methodologies for the seismic assessment of buildings”. This chapter will present the main features of this document.

3.1. Performance Objectives

The performance objective is determined by the level of structural and nonstructural performance of the analyzed construction for a certain level of seismic hazard. The performance levels describe the expected performance of the building, in terms of damage level, economic losses and interruption of its functioning, given a certain level of seismic hazard.

Association of the performance level with a certain earthquake characterized by a certain mean recurrence interval is made function of category of importance and the building exposure to seismic hazard:

- 1st category, corresponds to those buildings whose activity must not be interrupted for the duration of the earthquake or immediately following it.

In this category there are buildings that ensure essential functions during and after the earthquake.

- 2nd category, generally corresponds to very important buildings requiring a damage limitation
- 3rd category, corresponds to current type of constructions
- 4th category, corresponds to constructions of small importance and/or low degree of occupation.

Seismic hazard level depends on the location of the building with respect to the seismic source, on the geological features of the region where the building is situated as well as on the level of hazard selected for the seismic motion within the performance objective.

The current methodologies have in view 3 levels of performance or limit states, namely, Figure 2:

1. The immediate occupancy performance level, associated to the limit state of service (IO).
2. The life safety performance level (LS).
3. The collapse prevention performance level, associated to the ultimate limit state (CP).

For the common buildings it is considered that the life safety limit state covers also the collapse prevention limit state while the immediate occupancy limit state is mandatory only for 1st category of buildings.

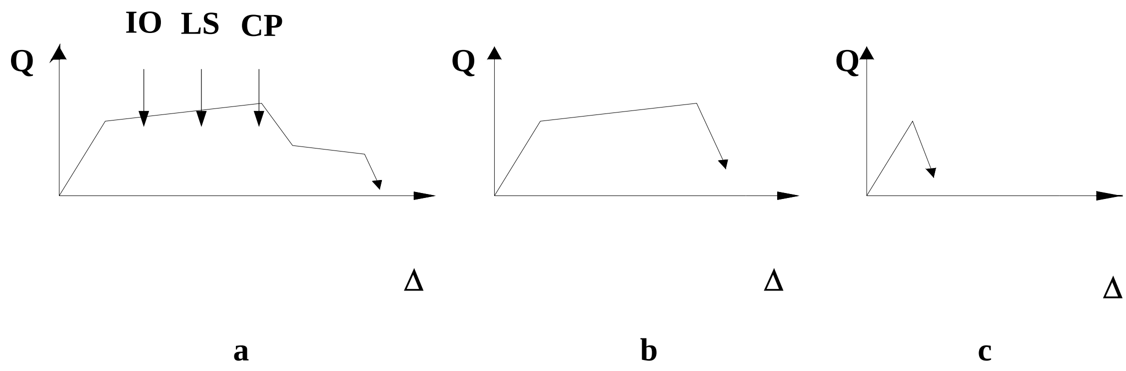


Figure 2: Performance levels for ductile and brittle structures

Figure 2: The three limit states and building behavior (ductile/brittle)

The current document classifies the assessment methodologies into 3 categories:

- level 1 methodologies (simplified methodologies);
- level 2 methodologies (usual methodologies for common buildings types);
- level 3 methodologies (methodologies used for complex or very important buildings and which use advanced methods of investigation).

3.2. Level 1 methodology

The level 1 methodology is applied to the following building types:

• regular buildings having reinforced concrete frames, with or without infill masonry walls up to 3 story high, located in seismic areas from D to F (low and average seismic activity);

- buildings with non-reinforced masonry structural walls with concrete slabs up to 3 story high or with masonry walls reinforced with reinforced concrete cores located in seismic areas – D ÷ F;
- buildings with reinforced concrete structural walls, made by cast in place or precasted concrete panels, up to 5 story high, located in any seismic area;
- any type of building located in the F seismic area for any performance level, as well as for any buildings with any characteristics located in the E seismic area, for the performance level of life safety.

These conditions of applying the level 1 methodology refer only to categories 3 and 4 of importance and exposure to the seismic hazard. Level 1 assessment methodology may be optionally used for more complex buildings as well or buildings located in a high seismic area, with the aim of getting preliminary information.

3.2.1. Principle of the method

Level 1 assessment methodology entails the checking of certain well-conformed conditions, grouped into several lists of requirements, structural or non-structural, more developed or less extensive, function of the requirement system and the level of the performance pursue.

The verification relations (compliance criteria) are expressed in forces. In order to be compared with the capable force values, the values of the forces produced by the conventional seismic force are reduced by coefficients, whose values depend on behavior of structural elements, ductile or non-ductile. Since the checking of the structure is done element by element, this approach has the advantage that a structure is not defined by means of only one reduction coefficient (ψ in the case of the Code P100/92), but by means of different reduction coefficients for the elements of the structure, in accordance with the deformation capacity of the element.

In the case of level 1 assessment, this checking is applied only to elements vital for the stability of the building (columns, walls), by using several values of the ψ reduction factor considered appropriate by the certified designer. The values of the ψ reduction factor are established based on the performance level and element type for every element of the structural system.

3.2.2. Evaluation of equivalent lateral seismic forces

The evaluation force in a horizontal direction of the building is computed by means of the following relation:

$$S = k_s \, \beta \, G$$

where:

S = the equivalent lateral force (the pseudo-lateral force);

k_S = the coefficient corresponding to the seismic area intensity from P100/92 (PGA/g);

β = the coefficient of dynamic amplification, according to P100/92;

G = the weight of the entire building. We use the values of the loads for the special load combination.

The story shear forces

In the case of multistory buildings, the conventional lateral force is distributed on the height of the building on the basis of the approximate relation:

$$S_j = \frac{n+j}{n+1} \frac{G_j}{G} S$$

where:

S_j = the j storey shear force;

n = the total number of storey above ground level;

j = the number of storey, up to the level under consideration;

G_j = the total of gravitational loads for the storey above level j;

G = the total weight of the building;

S = the conventional seismic force;

Fundamental period of vibration

The fundamental period of vibration of the building in the considered direction needed for the establishment of the values of the β spectral factors is usually calculated on the basis of the following expression :

$$T = k_T \, H^{3/4}$$

where:

T = fundamental period in seconds;

k_T = coefficient which has the values 0,07 for structure made of reinforced concrete frames and 0,045 for structures with reinforced concrete walls and masonry walls.

For reinforced concrete frame structures with up to 12 stories the approximate assessment of the fundamental period can be made alternatively with the following relation:

$$T = 0{,}10\ n$$

where n is the number of storey above the base.

3.2.3. Computation of the shear stresses in vertical elements

The average unit stress τ_m in the columns of the reinforced concrete frames is determined by the expression:

$$\tau_m = \frac{\psi}{q}\left(\frac{n_s}{n_s - n_c}\right)\frac{S_j}{A_{Ss}}$$

where:

n_S = the total number of columns;

n_C = the number of frames in the direction of the calculation;

A_{SS} = the sum of the areas of the sections of all columns at the level under consideration;

Sj = the shear force at the level under consideration, j;

ψ = modification (behavior) factor for the frame structures,

ψ is taken to have the value 0,4 for the LS performance level and 0,55 for the IO performance level.

The average value τ_m in the structural walls is calculated with the relation:

$$\tau_m = \psi \ \frac{S_j}{A_{Sp}}$$

where:

Sj = the storey shear force;

A_{Sp} = the sum of the net areas (subtracting the openings) of the sections of structural walls oriented on the direction of calculation;

ψ = behavior factor, Table 6.

Table 6: Behavior factor values for walls structures (ψ)

Type of wall	Performance level	
	LS	IO
Reinforced concrete	0,25	0,5
Unreinforced masonry	0,50	It is not the case
Masonry reinforced with RC elements	0,33	0,5

3.2.4. Verification conditions

The verification conditions express the conditions that a structure should respect in order to be considered safe.

The verification implies checking of the conditions regarding the configuration of the structure and conditions for the structural system resistant to lateral forces. One of the most important checking for the first level evaluation is the existence of horizontal load carrying system in the structure. If the evaluation engineer cannot find the horizontal load carrying system he usually proceeds to the retrofitting solutions rather than making further analyses.

The average tangential stress in the concrete, determined by previous procedures, should be lower than 0,8 N/mm^2 for the LS and the IO performance objectives.

3.3. Level 2 evaluation methodologies

Level 2 evaluation methodologies are usually applied to all buildings where level 1 assessment methodology cannot be applied. This methodology exclusively uses methods of linear, static and dynamic analysis.

3.3.1. Principle of the method

The effects of the earthquake are approximated by a set of conventional forces (pseudo-forces) applied to the building. The size of the lateral forces is established so that the deformation obtained as a consequence of the linear analysis of the structure to the action of lateral forces approximates the deformation imposed on the structure by the design earthquake.

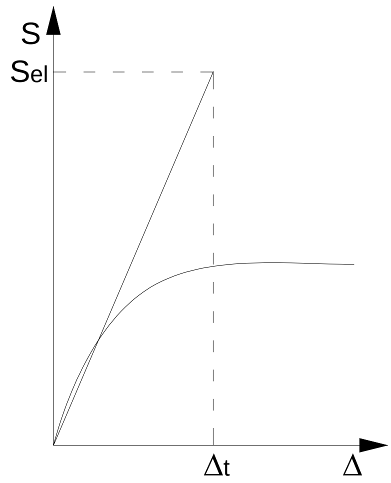


Figure 3: The elastic evaluation of seismic force for the building

The verification relation depends on the ductile or brittle failure of the structural element considered with different types of forces (M, N, Q).

Because in the case of ductile failure, the analysis relations indirectly express deformation conditions, the capable forces must be computed on the basis of the most likely values of strength, so on the basis of the mean values.

In the case of brittle failure, reaching the strength capacity cannot be admitted. The checking consists of the comparison of the force resulting under the action of gravitational and lateral forces, associated to the yielding of ductile structural elements of the structure, with the minimum element strength. These types of members are called “controlled by forces”.

3.3.2. Linear static analysis

The value of the basic shear force is determined according to the relation:

$$S = C_1 C_2 \alpha k_s \beta_r \varepsilon_r G$$

The α,.k_s, β_r, ε_r, coefficients are determined in accordance with P100/92 code:

α - importance factor of the building (0.8; 1; 1.2; 1.4)

k_s - ratio between peak horizontal ground acceleration (PGA) and gravitational acceleration

β_r - dynamic amplification factor

ε_r - equivalence coefficient between real structure and the SDOF structure

C_1 - coefficient that takes into account the amplifying of local deformation in the non-linear domain compared to the deformation obtained from the elastic analysis.

This takes into account the large extent of the degradation on certain levels of the structure and for regular buildings we suggest the value 1.25.

C_2 - coefficient which takes into account that in the domain T<Tc (Tc, corner period) inelastic displacements are superior to elastic ones. In the T>Tc domain inelastic displacements may be approximated by means of the displacement of elastic systems. The value of the C_2 coefficient is suggested along the following relation:

$$C_2 = \begin{cases} 2.0 \ \ for\ T < T_c/3 \\ \text{linear interpolation for the } T_c/3 - T_c domain \\ 1 \ \ for\ T > T_c \end{cases}$$

3.3.3. Linear dynamic analysis

The linear dynamic analysis is recommended in the cases of tall buildings (no. of stories > 10) or in the case of those buildings that have pronounced irregularities of the mass, stiffness or geometry.

In the case of modal analysis, the response of the structure is determined on the basis of elastic spectra of acceleration multiplied by C_1 and C_2 coefficients, respectively. The response forces and displacements will be computed as the square root from the sum of the squares of individual spectral response, or by using CQC (complete quadratic combination) if the period of the modes in the same direction differs by less than 25%.

It is advisable for the time-history linear dynamic analysis to be used in the case of tall buildings whose predominant period exceeds the corner period. In this case, the values of the forces will be multiplied only by the C_1 coefficient.

3.3.4. Verification relations

The structural members are deformation-controlled or force-controlled elements according to Table 7.

Table 7. Failure types and associated sectional forces

Structural elements		Loading	
		Deformation-controlled	Force-controlled
Reinforced	Beams	M	Q
	Columns	M	Q,N

concrete frame structures	Joints	-	Q
Reinforced concrete walls		M	N, Q

Deformation controlled structural members

The member forces for deformation-controlled elements is determined according to the relation:

$$N_C = N_G \pm N_S$$

N_C - total generalized force

N_G - generalized force due to the gravitational forces from the special load combination

N_S - generalized force resulting from the application of the seismic force defined in previous paragraphs.

The verification relation is:

$$N_{cap} \geq \psi N_C$$

where

N_{cap} - strength of the structural element. In the case of the forces that trigger ductile behavior, the strength capacity is computed on the basis of the mean strength of the materials;

N_C - generalized force;

Ψ - reduction coefficient owing to the ductile behavior of the element under the action of the force to be considered, Table 8.

Table **8**: The values of the reduction coefficient Ψ

Structural element	LS	IO
Beams		
Ductile behaviour[1)]		
$(p-p')/p_{max}{}^{2)} \leq 0;\ Q \leq 0.5bh_0R_t$	0.125	0.35
$(p-p')/p_{max}{}^{2)} \leq 0;\ Q \leq 2bh_0R_t$	0.25	0.40
$(p-p')/p_{max}{}^{2)} \geq 0.5;\ Q \leq 0.5bh_0R_t$	0.25	0.40
$(p-p')/p_{max}{}^{2)} \geq 0.5;\ Q \leq 2bh_0R_t$	0.35	0.60
Non-ductile behavior	0.40	0.65
Columns		
Ductile behaviour[1)]		
$n^{3)} \leq 0.1$	0.15	0.35
$n^{3)} \geq 0.4$	0.35	0.65
Non-ductile behavior		
$n^{3)} \leq 0.1$	0.35	0.65
$n^{3)} \geq 0.1$	0.5	0.65
Structural walls		
Ductile behavior		
$\xi^{4)} \leq 0.1$	0.2	0.35
$\xi^{4)} \geq 0.4$	0.35	0.65
Non-ductile behavior		
$\xi^{4)} \leq 0.1$	0.35	0.5
$\xi^{4)} \geq 0.4$	0.5	0.65
Structural walls yielding by means of shear force	0.4	0.65

Notes:

[1)] The ductile behavior means that the beam, the column, the structural wall fulfill all the reinforcement detailing conditions for the designing of new buildings, specific for these types of structures. Furthermore, all the conditions regarding the developing of a favorable energy dissipation mechanism must be met;

[2)] p - the reinforcing percentage

p' the compressed reinforcing percentage

p_{max} - the maximum reinforcing percentage corresponding to the balance point

[3] n - the adimensional axial force

[4] ξ - the adimensional height of the compressed area

Force-controlled members

The member forces that lead to a brittle failure of the structural elements must be associated with the structural failure mechanism. For the computation of the shear forces and the axial forces capacity for the structure with reinforced concrete frames or with structural walls the use of relations for the design of new buildings described in specific codes is recommended.

$$N_{cap}^{min} \geq N_C$$

N_{cap}^{min} - the minimum strength of the structural element. In the case of forces that trigger brittle failure it is recommended to compute the strength on the basis of the design strength of the materials.

N_C - the force determined according to previous paragraph

Checking of the deformation for the IO performance level

$$\Delta_{OI} = \frac{\Delta_s}{C_1 C_2 C} \leq \Delta_{OI}^{adm}$$

where:

Δ_{OI} - interstory drift at the level corresponding to the conditions of immediate occupation;

Δ_s - interstory drift at the level computed upon the seismic force

C - reduction factor which takes into account the fact that the earthquake associated to this performance objective has a mean recurrence interval smaller than that corresponding to level LS;

Δ_{OI}^{adm} - allowable interstory drift;

$\Delta_{OI}^{adm} = 0.035$ Hstory, if the partitions are rigidly connected to the structure;

$\Delta_{OI}^{adm} = 0.1$Hstory if the partition deforms independently of the structure.

In the case when the partitions are attached to the structure, the stiffness of the structural elements corresponding to the non-cracked sections is used in order to take into account their contribution to the stiffness of the building on the whole. If the infill walls deforms independently, the stiffness is taken half of the one corresponding to the uncracked concrete sections.

Checking of the deformation for the LS performance level

$$\Delta_{LS} = \Delta_s \leq \Delta_{LS}^{adm}$$

where:

Δ_{LS} - interstorey drift corresponding to LS limit state;

Δ_{LS}^{adm} - allowable drift; Δ_{LS}^{adm} = 0,025Hstory, with the aim of avoiding total breaking of non-structural elements, thus reducing human life lost risk and limiting structural damage.

It is admitted to take half of the computed values of the uncracked concrete sections, for all elements.

3.4. Level 3 evaluation methodology

Although according to the proposed methodologies it is not compulsory to be applied it is sometimes preferred by the evaluation engineers because it predicts the failure mechanism of the structure and because of the accuracy of the results.

3.4.1. Principle of the method

Level 3 evaluation methodology uses computation methods that take into account the inelastic behavior of structural elements. This method is recommended in those cases where level 2 evaluation method has application restrictions and/or when a more precise analysis of the seismic performance of the structure is needed.

It is necessary that in the case of a level 3 evaluation method the original design of the analyzed building should be available, due to the need of a much more precise information of the execution details.

Two analysis methods may be used, namely:

- the method based on non-linear static analysis (push-over).
- the method based on non-linear dynamic analysis (time-history).

3.4.2. Method based on non-linear static analysis

For the application of this method the base shear – lateral displacement curve of the reference point is needed. Any level of the building may be chosen as a reference point of the structure, but the mass centre at top story of the structure is usually chosen. The curve is obtained by performing a non-linear static analysis with the support of adequate software using fixed distributions of the horizontal forces applied to the structure.

The target displacement, in case the assessment is performed for the life safety performance level, is set by using approximant non-elastic displacement spectra function of the structural period and the strength of the structure.

For the comparison with the seismic requirements by using the response spectra of the seismic response, the values of the base shear – top displacement curve are changed using a MDOF to SDOF equivalence.

The checking is performed in terms of rotations for ductile behavior elements and in terms of forces for brittle behavior elements at the point of target displacement.

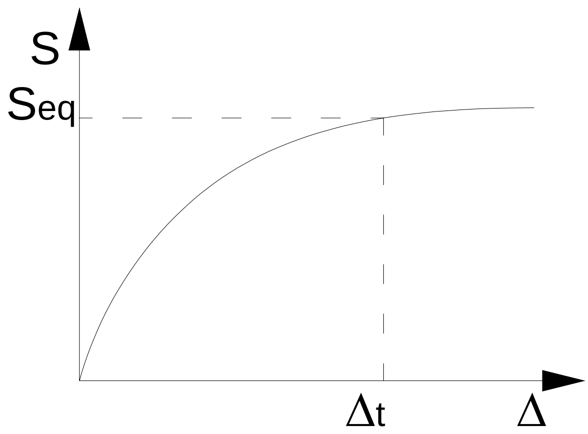


Figure 4: The Push-over curve of a building

3.4.3. Method based on the non-linear dynamic analysis

The method consists of performing a time-history analysis in the non-linear domain. The seismic action is directly applied, by means of accelerograms, at the base of the structure. The use of at least 3 significant accelerograms consistent with the site conditions of the building is needed.

Although it is the most rigorous method from the theoretical point of view, this approach has important limitations due especially to the large amount of time required for the analysis in the case of big structures, as well as the high level of technical knowledge needed for design engineers. Since the available structural analysis programs perform only a plain analysis, the method can not be applied without additional modifications unless the buildings dominant vibration modes correspond to the translations in the two main directions of the structure.

This assessment method is suitable only for the buildings with a ductile behavior.

3.4.4. Verification relations

Forces producing ductile failure

The maximum deformation of the element resulting from the non-linear static analysis should not exceed the limit deformation admitted for the performance level considered:

$\theta_{adm} \geq \theta_c$

where:

θ_{adm} - allowed plastic rotation admitted of the plastic hinge, see Table 9;

θc - plastic rotation resulted from the non-linear analysis.

Table **9**: The values of the admitted plastic rotation θ_{adm} (radians)

Structural element	LS	OI
Beams		
Ductile behaviour[1)]		
$(p-p')/p_{bal}{}^{2)} \leq 0$; $Q \leq 0.5bh_0R_t$	0.025	0.005
$(p-p')/p_{bal} \leq 0$; $Q \leq 2bh_0R_t$	0.015	0.005
$(p-p')/p_{bal} \geq 0.5$; $Q \leq 0.5bh_0R_t$	0.02	0.005
$(p-p')/p_{bal} \geq 0.5$; $Q \leq 2bh_0R_t$	0.01	0.005
Non ductile behavior	0.01	0.005
Columns		
Ductile behaviour[1)]		
$n^{3)} \leq 0.1$	0.02	0.005
$n^{3)} \geq 0.4$	0.015	0.005
Non ductile behavior		
$n^{3)} \leq 0.1$	0.01	0.005

$n^{3)} \geq 0.4$	0.005	0.000
Structural walls Ductile behavior $\xi^{4)} \leq 0.1$ $\xi^{4)} \geq 0.4$ Non ductile behavior $\xi^{4)} \leq 0.1$ $\xi^{4)} \geq 0.4$	 0.015 0.01 0.008 0.005	 0.005 0.003 0.002 0.001

Notes:

1) The ductile behavior means that the beam, the column, the structural wall fulfill all the reinforcement detailing conditions for the designing of new buildings, specific for these types of structures. Furthermore, all the conditions regarding the developing of a favorable energy dissipation mechanism must be met.

2) p - the reinforcing percentage

p' the compressed reinforcing percentage

p_{max} - the maximum reinforcing percentage corresponding to the balance point

3) n - the adimensional axial force

4) ξ - the adimensional height of the compressed area

Forces producing non-ductile failure

In the case brittle failure the minimum strength of the elements should not be lower than the forces produced by the seismic action superposed over the gravitational ones.

$$N_{cap}^{min} \geq N_C$$

$\mathbf{N_{cap}^{min}}$ - minimum strength of the structural element. In the case of forces producing non-ductile failure it is advisable to compute the capacity using the design stress of the material;

N_C - design force resulting from the non-linear static analysis.

4. Current retrofitting techniques used in Romania

The retrofitting of the reinforced concrete structures usually has the objective of increasing the strength, the stiffness and/or the capacity of post-elastic deformation of the existing structural elements (collar beams, columns, joints) or the transformation of the entire structural system.

The improvement of the performances of structural elements in frames is usually accomplished through the jacketing of columns, of the beams and/or of the joints. The jacketing technique of the columns and of the existing beams in a reinforced concrete mixture may be applied in order to enhance the stiffness and ductility, with or without an increase of the bending resistance and/or of the shear force.

4.1. Retrofitting of reinforced concrete frame structures

4.1.1. Interventions that do not involve the alteration of the structural system

The enhancement of the structural performances of the reinforced concrete frames might be obtained through interventions that do not involve an essential change in the characteristics of the initial building. These interventions mainly rely on jacketing techniques of the frames units, which pursue the enhancement in the resistance, stiffness and/or the capacity of post-elastic deformation of the existing structural elements.

The technique of jacketing the existent columns and beams with reinforced concrete represents an advantageous solution, depending on the needs, for increasing either stiffness only or both resistance and stiffness, and it is frequently used for the retrofitting of the damaged buildings in Romania. Localized interventions in areas that exhibit degradations of the elements are considered repairs if their scope is limited to the restoration of the initial situation without implying an enhancement of the level of performances.

The interventions may aim at increasing the strength of the elements in terms of shear capacity, bending moment or axial force, an increase in stiffness or in the post-elastic deformation capacity.

Structural deficiencies of the reinforcing detailing of concrete sections, having negative effects on the performance of the frames (insufficient shear reinforcing, lack of overlapping length, joints without shear reinforcement, etc.) usually require generalized interventions based on the technique of jacketing the elements.

The intervention solution by jacketing of the frame elements increases also resistance and stiffness of the substructure with smaller foundations compared to the techniques of structural walls additions. The jacketing of the elements of reinforced concrete frames may seriously affect the non-structural elements of the building

The technique of jacketing the existent columns and beams with reinforced concrete represents an advantageous solution, depending on the needs, for enhancing either stiffness only or both resistance and stiffness, and it is frequently used for the retrofitting of the damaged buildings in Romania. Localized interventions in areas that exhibit degradations of the elements are considered repairs if their scope is limited to the restoration of the initial situation without implying an enhancement of the level of performances.

Increasing shear capacity

Increasing the capacity in shear is many times necessary for the columns, beams or joints of the existent buildings. The capacity in shear is increased mainly through jacketing of the reinforced concrete elements.

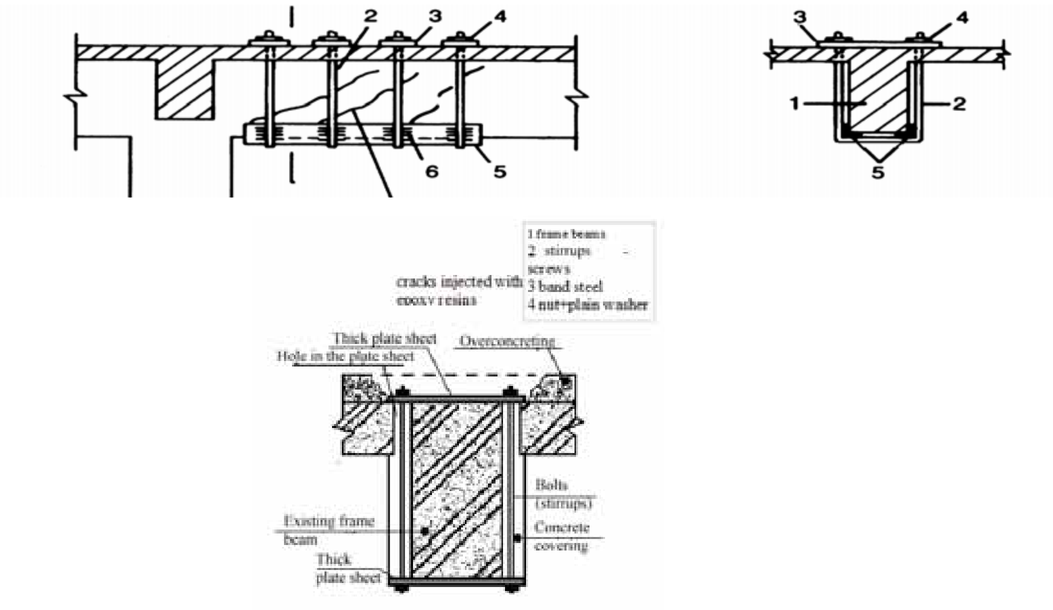


Figure 5: Local intervention details with regard to the capacity in shear of the frame reinforced concrete beams

Jacketing, and mainly the reinforced concrete jacketing, leads at the same time to the increase in rigidity and deformability and, in some cases, also to increasing the capacity in bending and shear.

In the situation when the concrete from the existing beam does not play an important part, it is also necessary to jacket the beams. Generally, in these cases, it is also necessary to increase the capacity in bending, Figure 5.

Increasing the capable shear force in the columns can be achieved also through reinforced concrete jacketing, as presented in Figure 6.

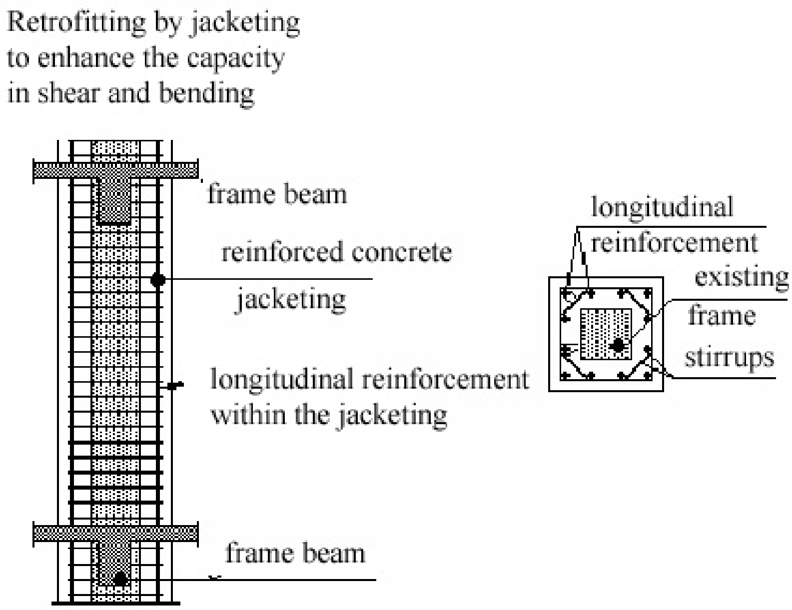


Figure 6: Jacketing of the columns

Increasing the capacity in bending

The intervention technique used in most of the cases consists of jacketing the reinforced concrete frame members, this solution ensuring a good cooperation with the existent element and the possibility to achieve the continuity of the reinforcements. The capacity in bending and in shear and in axial force can be controlled depending on the thickness of the jacketing concrete layer, of the quantity and of the position of the longitudinal and transversal reinforcements.

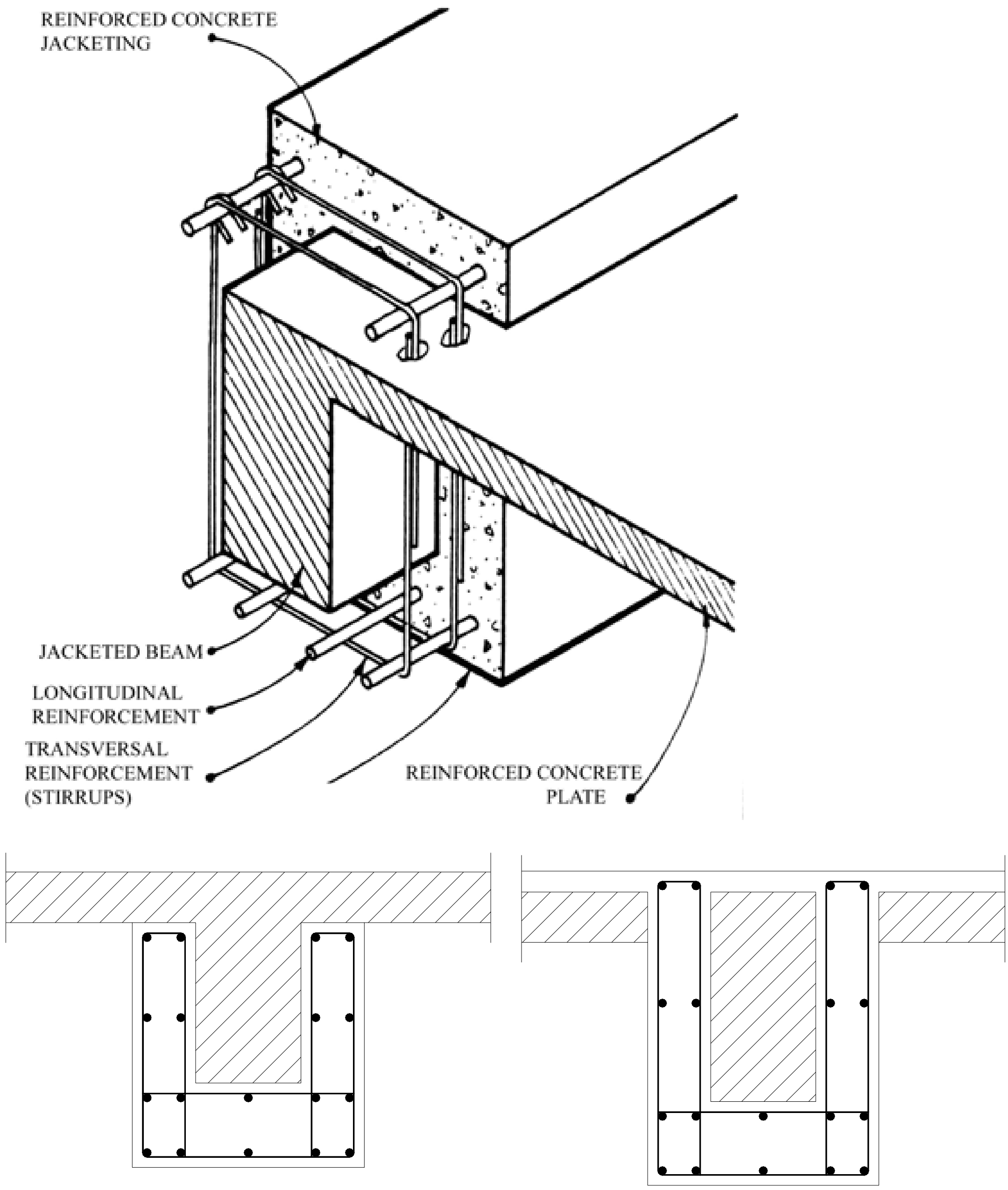
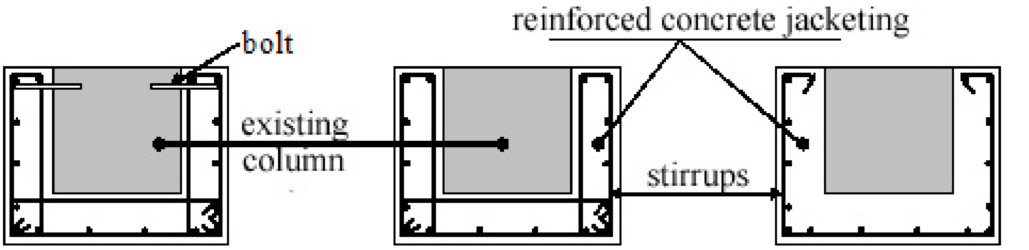


Figure 7: Beam jacketing for increasing flexural and shear capacity

In the situation when the jacketing of the columns cannot be done on all their sides (as in the case of some columns situated on the facade of the building), the details of performing the jacketing consider the bonding failure of the sleeve from the existent column. For this purpose, connectors with expanding heads may be used or connectors glued with epoxy resins that are introduced in the holes drilled in the existent concrete, Figure 8.



a. Favourable behaviour b. Acceptable solution c. Incorrect solution

Figure 8: Column jacketing on three sides

Interventions on beam-column joints

The main deficiencies that frame joints can exhibit refer to their low resistance in shear or to the inadequate anchoring of longitudinal reinforcements consisting of the elements that bind together in a joint.

In the case of the retrofitting by concrete jacketing, lateral reinforcement on the vertical line of the joint involves one of the following measures to be taken, Figure 9:

- drilling holes through the beams, fixing hoop and injecting the voids;
- breaking of the heads of the beams, by adequately supporting them, execution of the reinforcing of the joints and concreting.

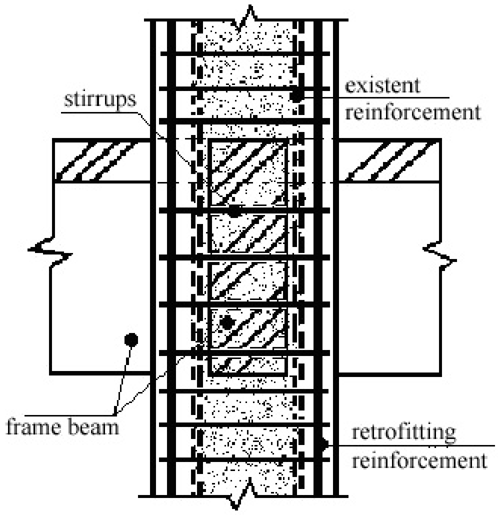


Figure 9: Beam-column joint RC jacketing

4.1.2. Interventions that involve the transformation of the reinforced concrete structural frames

Introduction of infill structural walls

Filling in the empty spaces of the frames with reinforced masonry walls or reinforced concrete leads to a significant increase of the stiffness and strength of the structural system. The second solution involves the attachment of the wall near the beam of the frame, and the columns are coated thus becoming the bulbs of the new wall. The solution can be applied also without coating of the columns and/or of the beams, if the elements of the frame can take over the efforts which result from the association with the introduced panels, including the local compression consequences.

If the reinforced concrete frames are completely filled, the specific deformation of the frame can be inhibited through the stiffness of the infill wall. Stiff infill walls (made of reinforced concrete or of reinforced masonry) will mainly resist to the lateral forces in the same proportion like the structural walls, while the un-retrofitted frames will have a relatively low level of loading. The reinforced concrete frames filled with less rigid walls (un-reinforced masonry) will tend to resist to the lateral forces as systems with compressed diagonals that are formed in the filling.

The existent frames can be filled with different types of masonry: solid brick, perforated brick etc. These types of fillings can be reinforced, partially reinforced or un-reinforced. At the same time, the filling may be strengthened with coatings of concrete mortar reinforced with wire meshes. These meshes will not be able to prevent the breaking of the filling, but can prevent the unfolding of different parts from the filling.

The possible situations of intervention on framed structures through building filling walls are presented in Figure 10.

The forces between the elements of the existent frame and the concrete or reinforced masonry infill panel can be taken over by connection elements that are made according to Figure 11. Such measures are necessary especially in those cases when the filling panel is made of reinforced concrete or of a good reinforced masonry, and in this case the filling frame acts overall as a structural wall.

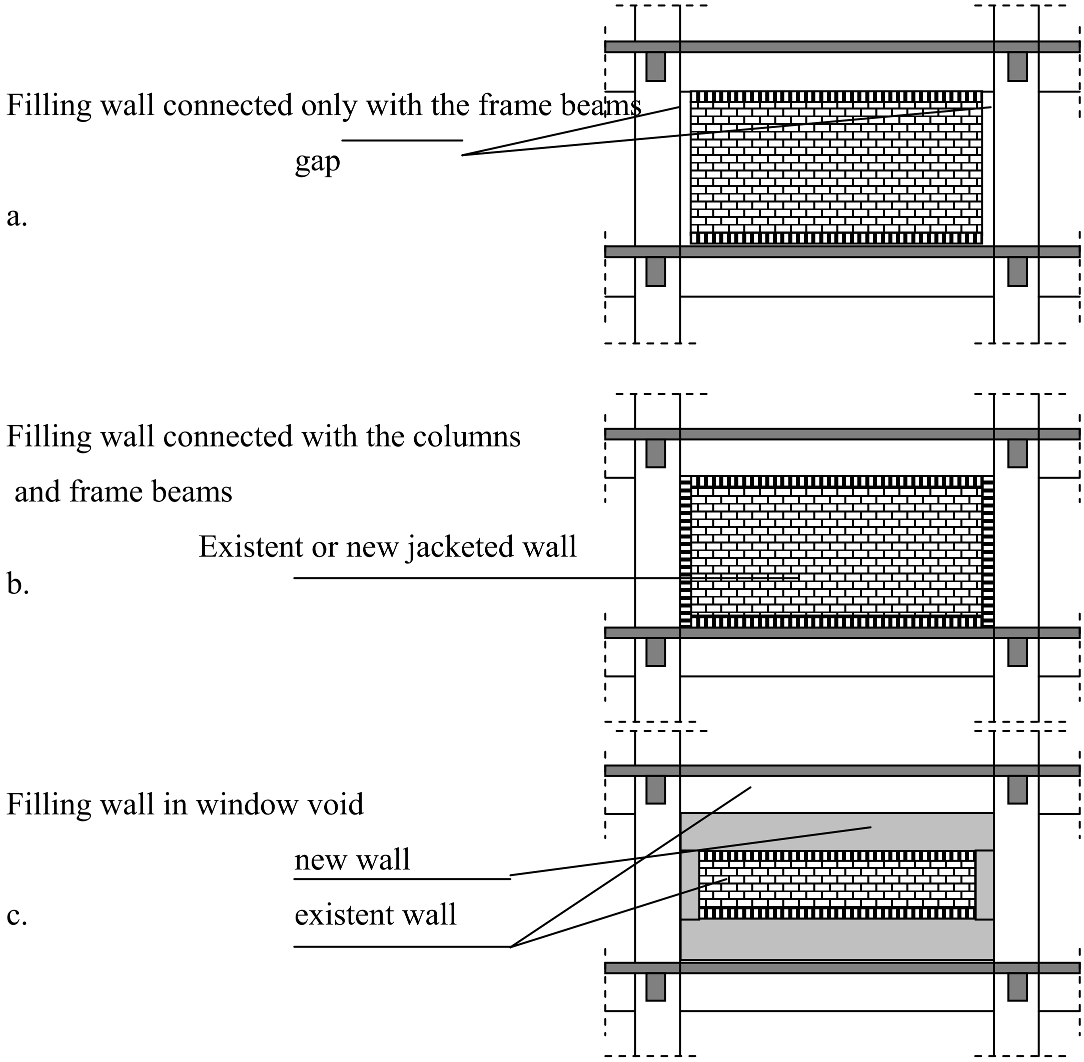


Figure 10: Filling of the frame openings

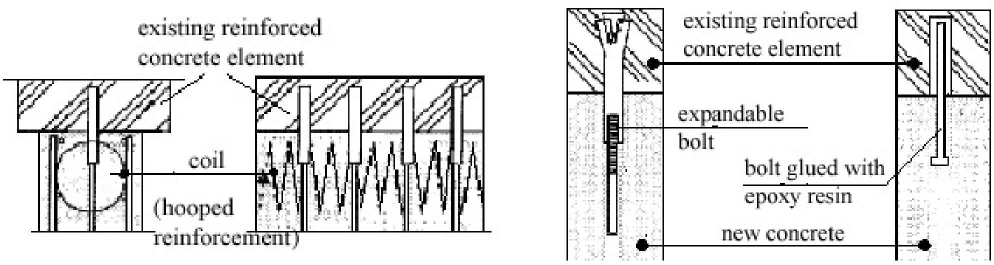


Figure 11: Connecting the concrete wall with the frame units

Introduction of new structural reinforced concrete walls

The addition of reinforced concrete structural walls to the initial framed structure triggers an important increase of both the strength and the stiffness to lateral drifts. In this way, the cooperation of the existent frames with the reinforced concrete structural walls may give the ensemble a specific behavior similar to dual structures or to a the stiff-walled structure, Figure 12.

The structural reinforced concrete walls may be placed both on the exterior, Figure 13 and/or the interior side of the building. The making of interior structural walls involves some difficulties (such as the change in functioning or of the existent equipments) and higher manufacturing costs.

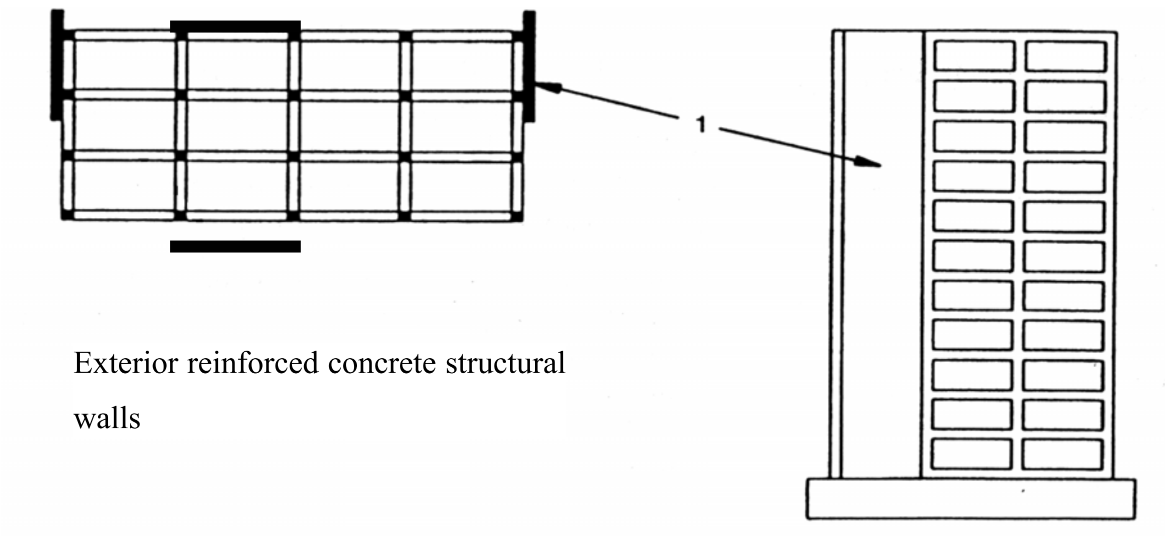


Figure 12: Adding shear walls to an existing RC frame structure

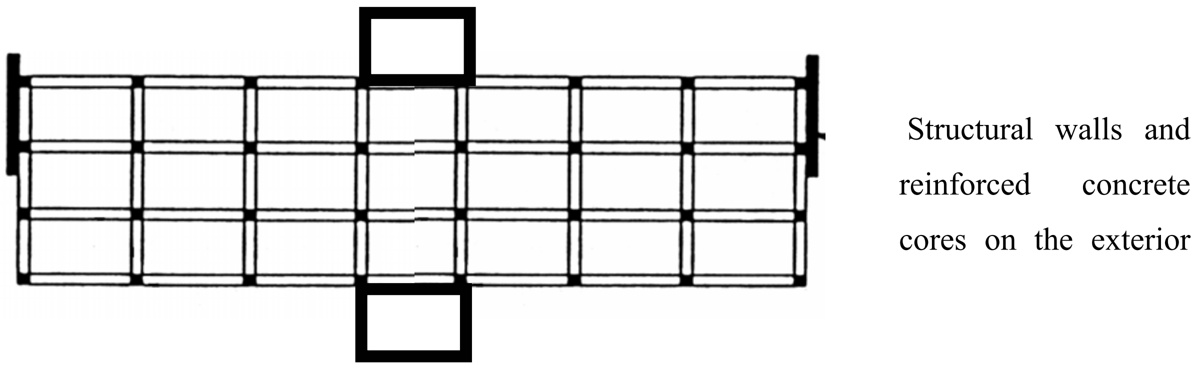


Figure 13: Reinforced concrete structural walls, situated on the exterior part of the building

Figure 14 presents the structural walls that are made by placing the core outside the plane of the beams and attaching it to the coating of the columns, which act as wall bulbs. This solution allows for the continuity of the vertical reinforcements and it is useful when a substantial enhancement of the stiffness capacity of the lateral force is required.

By wedging the new elements among the already existent units of the frame during the process of deformation under seismic action, we can ensure the engagement of the gravitational forces of the structure.

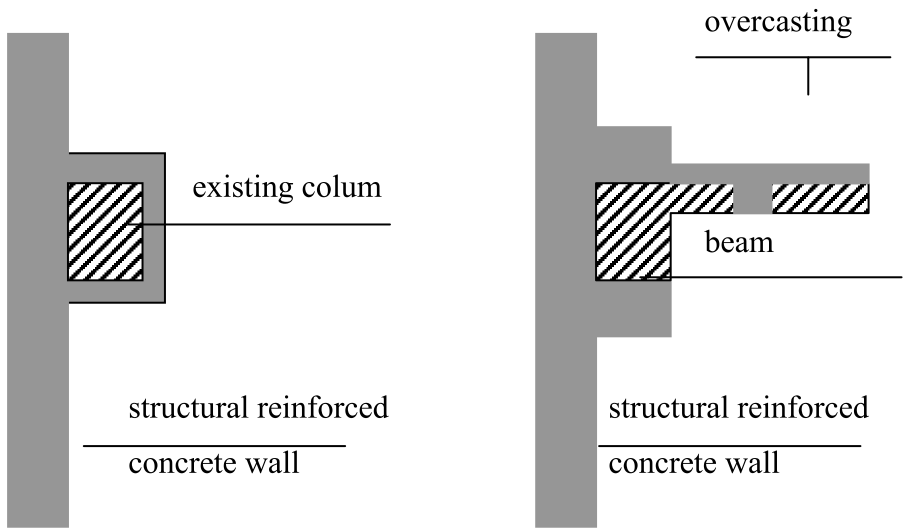


Figure 14: The connection between the existing frame and new added shear wall

In the case of a solution similar to the one described in Figure 13, when the cores are attached to the exterior side of the building, it is necessary to tie the cores to the main part of the structure (drifting reinforcements, collectors etc.) that can ensure the entrainment of the building mass by the cores.

4.2. Interventions on reinforced concrete wall structures

4.2.1. Interventions that do not involve the alteration of the structural system

The retrofitting of structures with reinforced concrete walls usually has the following targets:

- Increasing the shear capacity of the walls;
- Increasing the bending capacity;
- Increasing the post-elastic deformation capacity of the walls;
- Increasing the capacity in shear of the connecting beams;

- Increasing the capacity in bending of the connecting beams;
- Increasing the post-elastic deformation capacity of the connecting beams;

The available techniques for fulfilling the above-mentioned targets are:

- Wall thickening by adding several layers of reinforced concrete (for instance, through injection of concrete) which work together with the existent wall ;
- Thickening the connecting beam by adding several layers of reinforced concrete which are anchored in the existing beam ;
- Building reinforced concrete bulbs which are anchored on the cross-sections of the wall;
- Re-building the connecting beam with increased concrete sections and reinforcement.

Retrofitting the reinforced concrete walls with the technique of thickening the reinforced concrete cross-section favors the simultaneous achievement of several different targets, through adequate dimensioning of the longitudinal and transversal reinforcement and of the thickness of new layer of concrete.

Increasing the shear and flexural capacity of the shear walls

The retrofitting of the reinforced concrete wall structures with the purpose of having an increased strength in shear for the walls and increased flexural capacity can be achieved through:

- Wall thickening by adding several layers of reinforced concrete which work together with the existent wall, Figure 15;

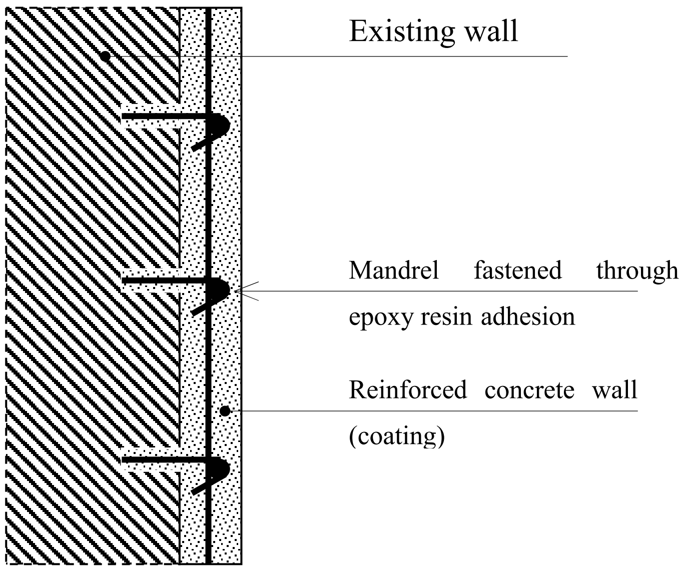


Figure 15. Jacketing of reinforced concrete walls

Figure 15: Wall retrofitting by using thickening process of the reinforced concrete cross-section

- Filling in the empty spaces such as doors or windows with reinforced concrete.

Increasing the of post-elastic deformation capacity of the walls

The retrofitting of the reinforced concrete walls' structures that targets the increase of the post-elastic deformation capacity can be achieved through:

- Wall thickening by adding several layers of reinforced concrete;
- Placing bulbs in the compressed areas of the cross-section, igure 16.

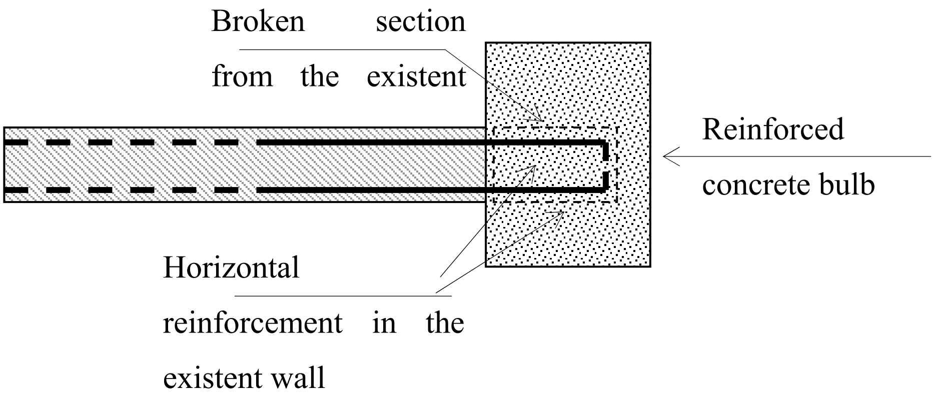


Figure 16: Wall retrofitting by using bulbs

Generally, connecting the bulb with the wall is done with horizontal reinforcements. If these are not sufficient and if it is intended not to break the wall on too long distances, there can be added bolts attached with epoxy resin in holes from the existing wall. The provision of the bulbs also implies interventions on the foundations.

4.2.2. Interventions that involve the transformation of the reinforced concrete structural walls

The interventions that mean the transformation of the structure with reinforced concrete walls consisted of:

- Introduction of new structural reinforced concrete walls in the existing system; the procedure is similar to inserting structural wall to reinforced concrete frames;
- Filling in the empty spaces such as doors or windows, Figure 17 and Figure 18.

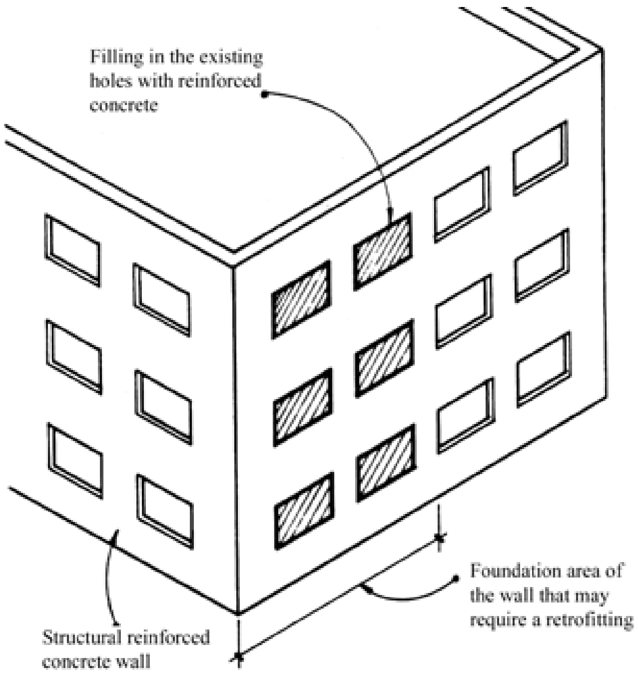


Figure 17: Transformation of structural reinforced concrete wall by filling the empty spaces such as windows; interventions on masonry structures

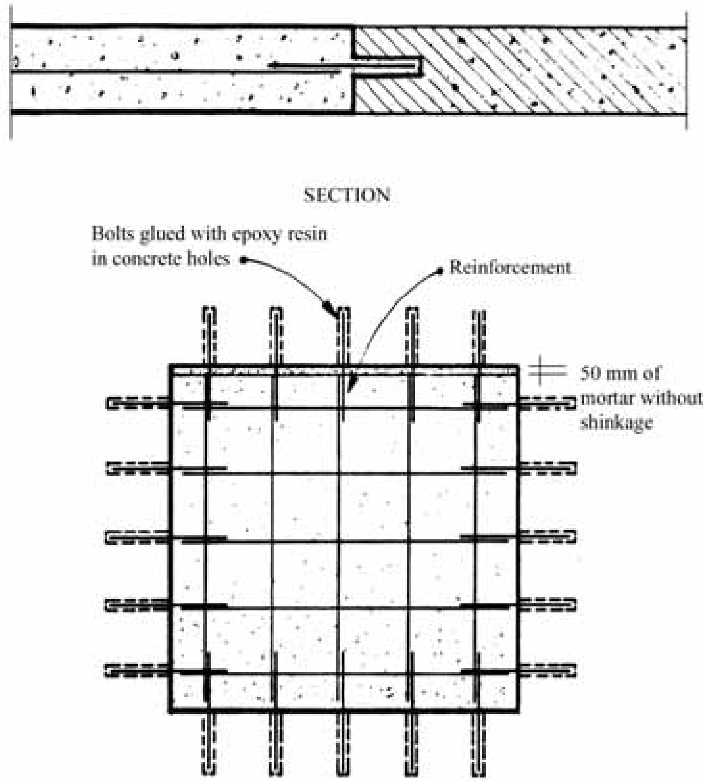


Figure 18: Details regarding the process of filling in the empty spaces of the structural walls with reinforced concrete

4.3. Interventions for masonry structures

Unreinforced masonry was used as material for structural walls in pre ‘60s buildings. The main deficiencies of the structures made of un-reinforced masonry walls are:

- Low capacity in shear;
- Low resistance capacity to out of plane forces;
- Weak connections between the perpendicular oriented walls.

Interventions that are used in Romania to remove deficiencies of structures made of unreinforced masonry are:

- Introduction of reinforced concrete elements at appropriate distances;
- Coating masonry walls with reinforced concrete;
- Interlocking the walls arranged on these two directions, using reinforced concrete elements that are capable to take over the efforts of tension and compression;
- Introduction of additional interior walls.

Interventions that involve the increase of masonry walls shear capacity

Failure of the masonry walls can occur in two situations: inclined cracking or joints’ failure. The capacity of unconfined masonry walls can be increased using the following measures:

1. Masonry confinement through adding vertical reinforced concrete components, situated at distances that take into consideration the level height, Figure 19.
2. Local reinforcement of the masonry around the larger empty spaces through the introduction of a consistent reinforced concrete frame, adequately anchored in masonry walls has also the ability to increase the resistance capacity of the masonry in the panel, Figure 20

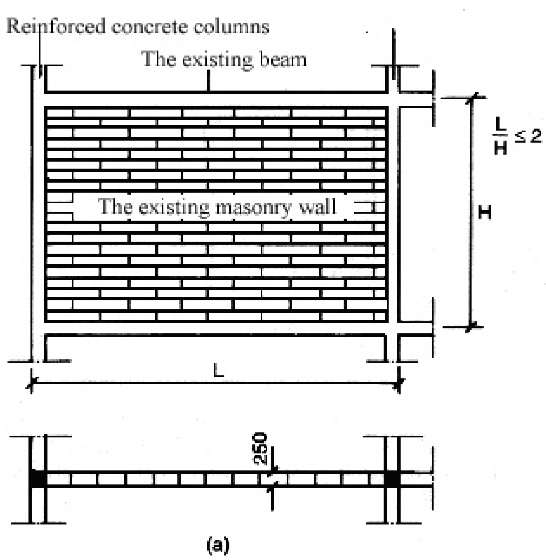


Figure 19: Masonry retrofitting by inserting concrete elements

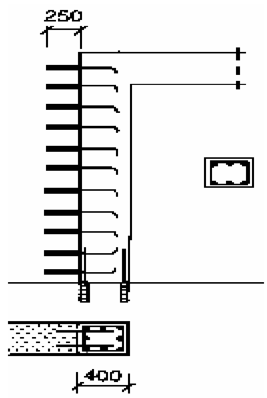


Figure 20: Masonry anchoring to the RC frame

4.4. Interventions on slabs

The interventions on slabs considered to be horizontal diaphragms can have the following targets:

- Increasing the capacity in shear;
- Increasing the capacity in bending;
- Increasing the strength to failure at the interconnecting areas between vertical elements and floors (the transfer of shear force between vertical elements and diaphragm);
- Increasing the strength of the slab to local loadings (in case of suspended loadings);
- Increasing the strength of the slab in the areas with local weakening (areas that contains openings).

Increasing the strength of the slabs to shear force is usually achieved by overconcreting the floors. The connection between overconcreting and the existent slab can be achieved by using one of the techniques presented in Figure 21. The overconcreting is reinforced with reinforcement meshes and with local, concentrated reinforcements.

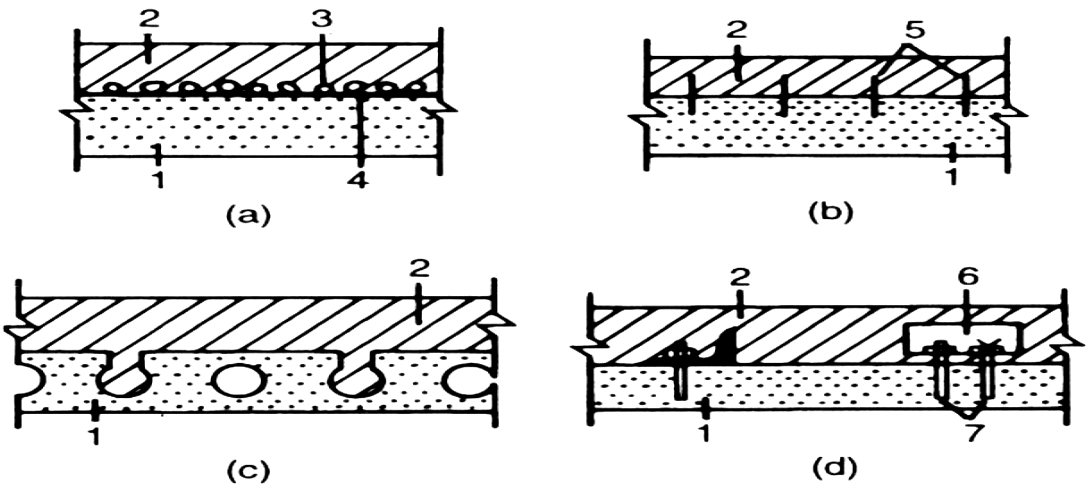


Figure 21: Slab overconcreting
1 – existing floor; 2 – overconcreting; 3 – gravel; 4 – epoxy adhesive; 5 – bolts attached with epoxy resins; 6 – connectors (metallic profile – angle); 7 – expandable bolts

Increasing the capacity in bending is achieved through the introduction or increase of the tensile reinforcement which plays the part of tie rope (truss). Additional necessary reinforcements can be placed as bars included in over concreting (if the intervention involves overconcreting), Figure 22.
The weakening of the diaphragm caused by the large openings usually imposes interventions on the overall floor, Figure 23.

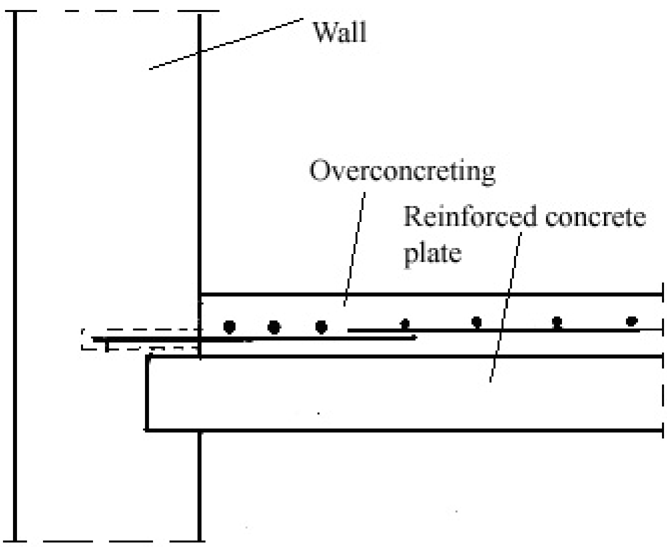


Figure 22: Slab overconcreting with aditional flexural reinforcement

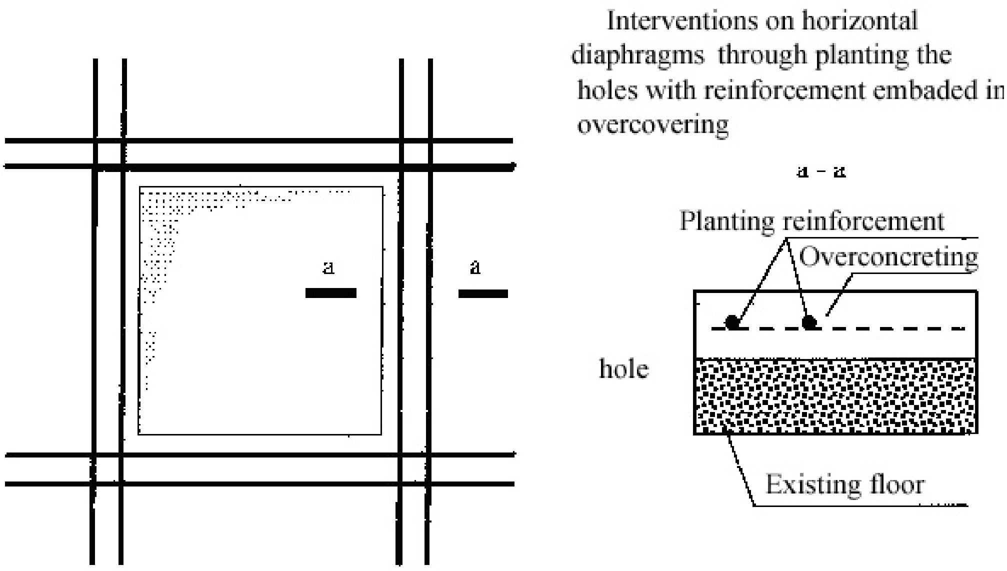


Figure 23: Interventions on horizontal membranes through planting the openings

5. Case study of building evaluation and retrofitting

In this chapter is shown a case study of current Romanian evaluation and retrofitting technique. The study was conducted based on the design project of a hospital designed and built in 1960. Given the design procedures at that time it was considered necessary to evaluate the seismic capacity.

5.1. Short description of the analyzed building

The hospital is in use since 1963. The building is “Y” shaped. It consists out of 10 blocks of approximately 40m, separated by narrow gaps. Only block 2 evaluation and retrofitting is discussed hereinafter.

The building has 1 underground floor, and 10 upper stories. The structural system consists of moment resisting reinforced concrete frames aligned on two orthogonal directions. In transversal direction the frames have three spans of 5.80, 2.60 and 3.50, respectively. In longitudinal direction the spans are uniform at 3.60m, Figure 24, Figure 25 and Figure 26.

The columns and beams sections have been designed only to accommodate the gravity loads, based on the code available at that time, STAS 1546-56. The columns and beams sections are gradually reduced over the building height. The main façade columns have rectangular sections of 80cm height in transversal direction.

The floor slabs are made of reinforced concrete. They are reinforced in both directions in the outer spans and on a single direction in the central span.

The underground floor is enclosed by a weakly reinforced concrete wall and a general mat.

5.2. Building behavior during past earthquakes

The hospital staff reported no major damage of the structural and nonstructural elements during the past earthquakes. It is worth mentioning that from the construction time the building was hit by several Vrancea epicentered earthquakes. However, the building response to these earthquakes can be considered satisfactory. The reasons of this behavior will be explained in the following.

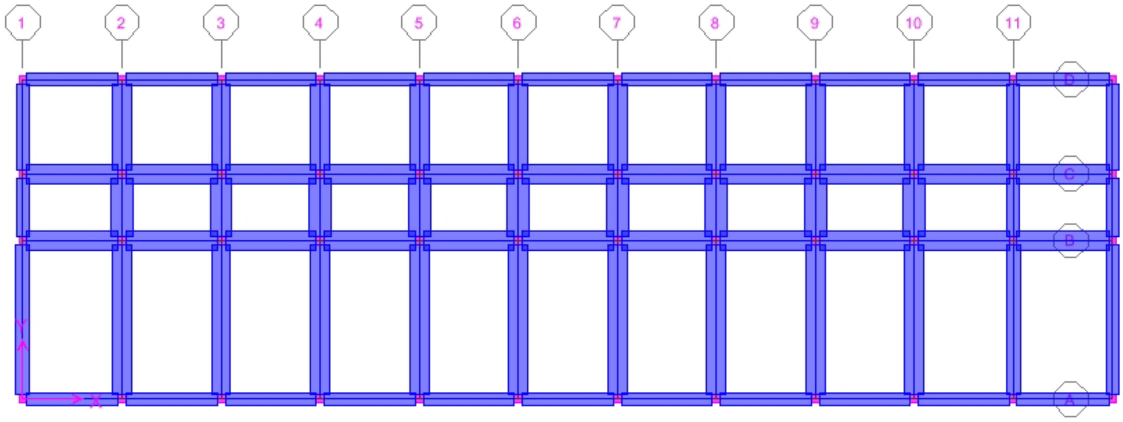


Figure 24: Plan view

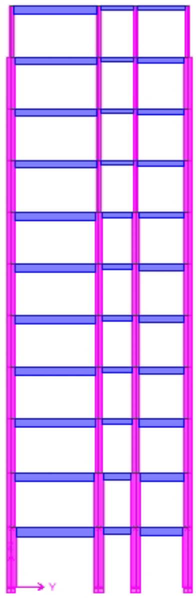


Figure 25: Transversal section

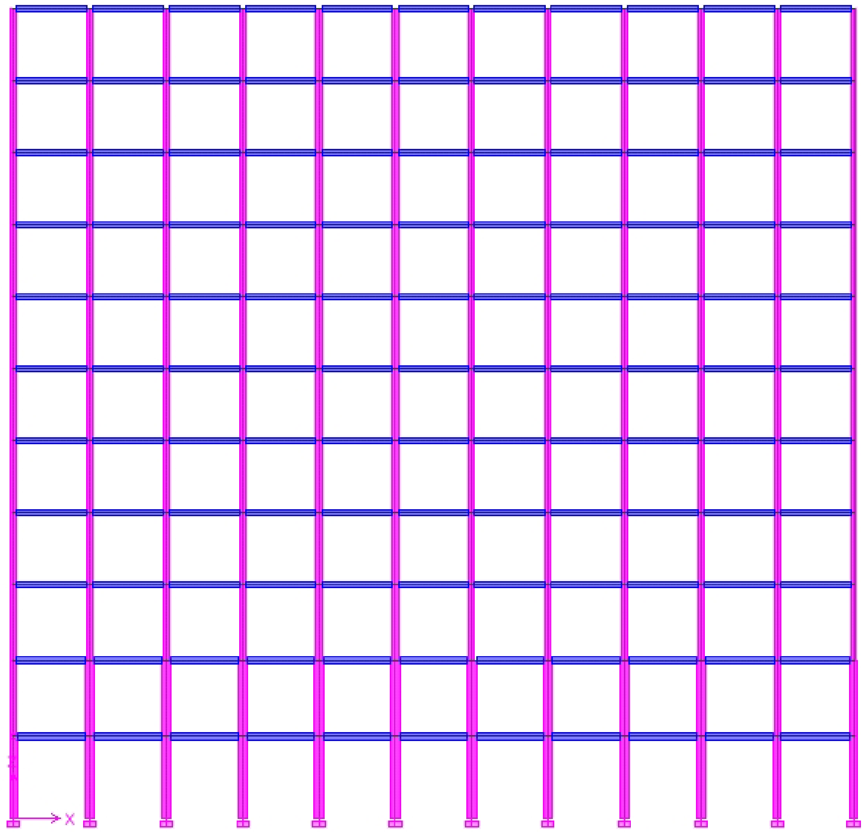


Figure 26: Longitudinal Section

5.3. Seismic evaluation based on the Romanian P100/92 seismic design code

The required seismic force is:

S = cG

c = α k_s β ε ψ

c – seismic coefficient

α – building importance factor 1.4

k_s – seismic intensity factor, depends on the site seismicity 0.12

β – dynamic amplification factor (Tc=0.7), Figure 27

$\beta_r = 2.5$ for $T_r \leq T_c$

$\beta_r = 2.5 - (T_r - T_c) \geq 1$ for $T_r > T_c$

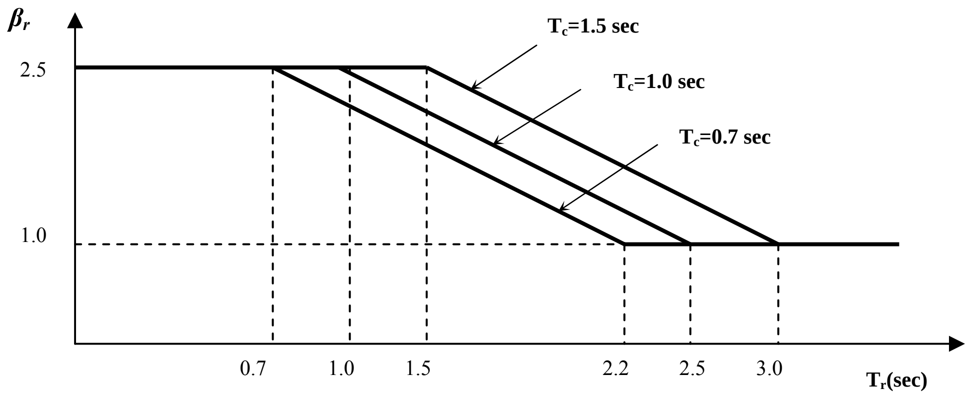


Figure 27: Amplification spectra

ψ – reduction (behavior) factor (0.2)

G – building weight

5.4. Checking of existing structure lateral stiffness

After the modal analysis the following values have been obtained:

• the fundamental vibration period in the longitudinal direction

T_{1x}= 1.75 s

• the fundamental vibration period in the transversal direction

T_{1y}= 1.29 s

The global seismic coefficient in the longitudinal direction:

c= α ks β ε ψ where: β= 2.5-T+Tc

β= 1.45

c= 1.4 * 0.12 * 1.45 * 0.75 * 0.2

= 3.65%

The global seismic coefficient in the transversal direction:

c= α ks β ε ψ where: β= 2.5-T+Tc

β= 1.91

c= 1.4 * 0.12 * 1.91 * 0.75 * 0.2

= 4.81%

Table 10: Drift control for existing structure

Storey	Direction	Level	Δ X	Δ/Ψ*1000	Δ Y	Δ/Ψ*1000
11	SEISMX / SEISMY	38.26	0.000637	3.2	0.000907	4.5
10		34.86	0.001031	5.2	0.001104	5.5
9		31.46	0.00133	6.7	0.001244	6.2
8		28.06	0.001556	7.8	0.001277	6.4
7		24.66	0.001758	8.8	0.001231	6.2
6		21.26	0.001792	9.0	0.001197	6.0
5		17.86	0.001861	9.3	0.001174	5.9
4		14.46	0.00194	9.7	0.001126	5.6
3		11.06	0.001763	8.8	0.001028	5.1
2		7.46	0.001039	5.2	0.000769	3.8
1		3.86	0	0.0	0.000401	2.0

5.5. Retrofitting solutions

5.5.1. Retrofitting by reinforced concrete jacketing on the central span

(Checking of the building lateral stiffness)

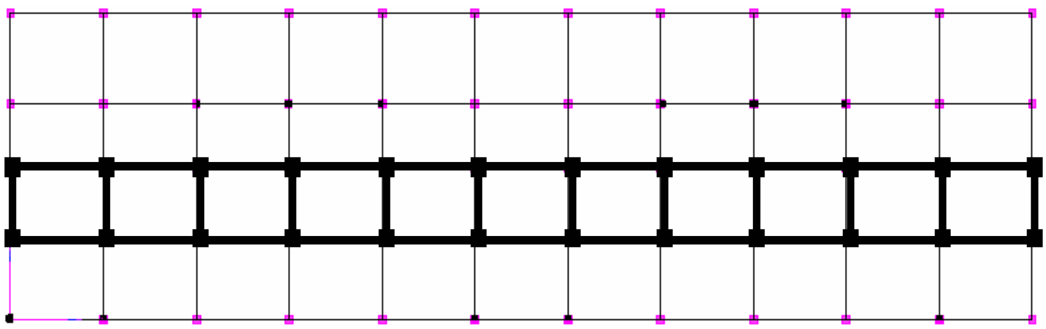


Figure 28: Plan view: RC jacketing

After the modal analysis the following values have been obtained:

• fundamental vibration period in the transversal direction

$T_{1x}=$ 0.78 s

• fundamental vibration period in the longitudinal direction

$T_{1y}=$ 0.97 s

The global seismic coefficient in the transversal direction:

$c = \alpha\, k_s\, \beta\, \varepsilon\, \psi$ where $\beta = 2.5 - T + T_c$

$\beta = 2.42$

$c = 1.4 * 0.12 * 2.42 * 0.75 * 0.2$

$= 6.10\%$

The global seismic coefficient in the longitudinal direction:

$c = \alpha\, k_s\, \beta\, \varepsilon\, \psi$ where $\beta = 2.5 - T + T_c$

$\beta = 2.23$

$c = 1.4 * 0.12 * 2.23 * 0.75 * 0.2$

$= 5.62\%$

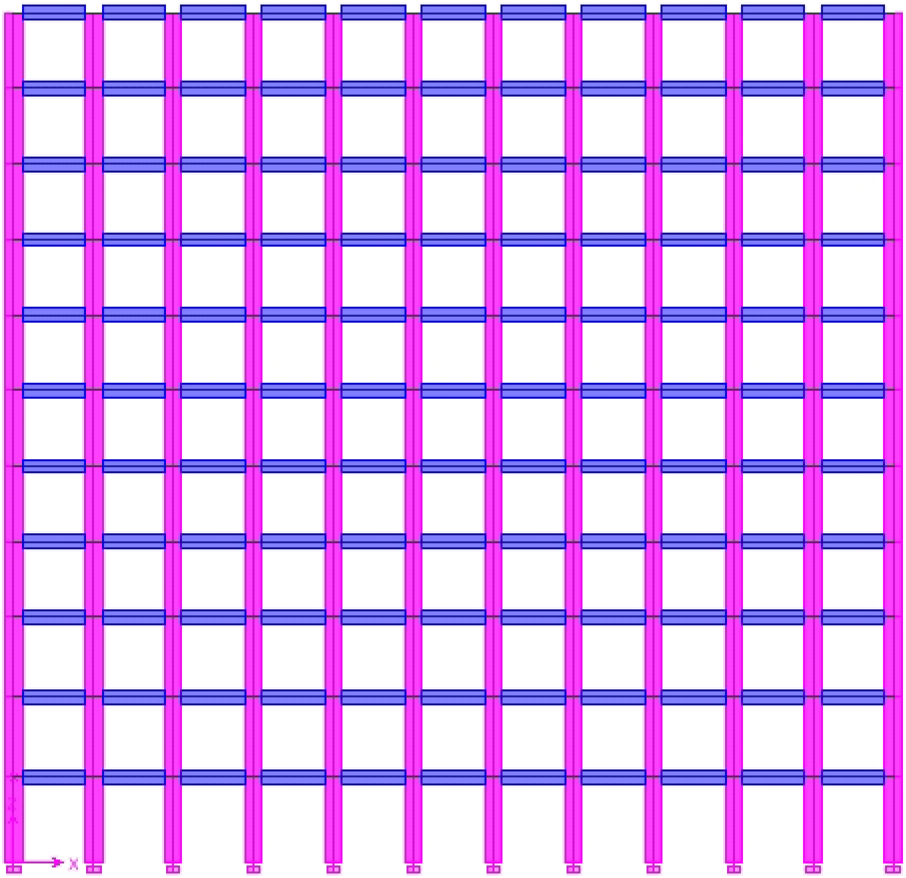


Figure 29: Transversal section: RC jacketing

Table 11: Drift control for reinforced concrete jacketing:
Columns 75x105 Long. beams 60x45 transv. Beams 55x75

Storey	Loading	Level	Δ X	Δ/Ψ*1000	Δ Y	Δ/Ψ*1000
11	SEISMX / SEISMY	38.26	0.000146	0.7	0.000715	3.6
10		34.86	0.000233	1.2	0.000731	3.7
9		31.46	0.000326	1.6	0.000773	3.9
8		28.06	0.000409	2.0	0.000792	4.0
7		24.66	0.000483	2.4	0.000791	4.0
6		21.26	0.000544	2.7	0.000777	3.9
5		17.86	0.000595	3.0	0.000753	3.8
4		14.46	0.000636	3.2	0.000702	3.5
3		11.06	0.000662	3.3	0.000632	3.2
2		7.46	0.000486	2.4	0.000497	2.5
1		3.86	0.000016	0.1	0.000283	1.4

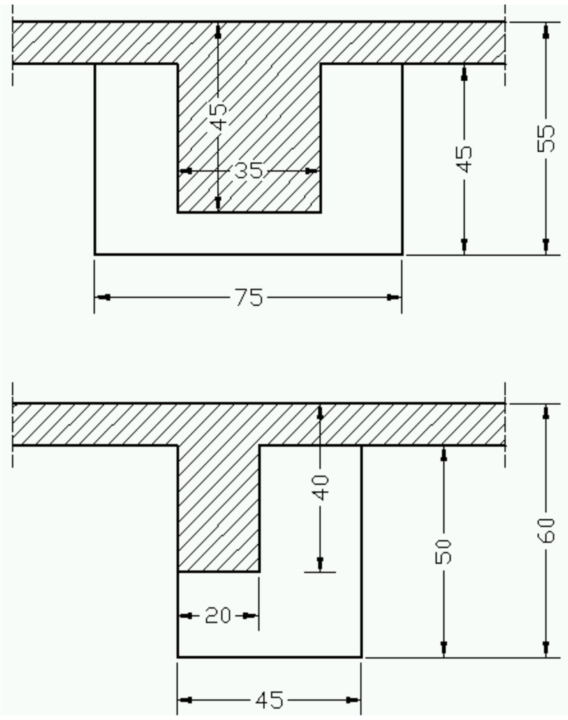


Figure 30: Beam jacketing retrofitting solutions

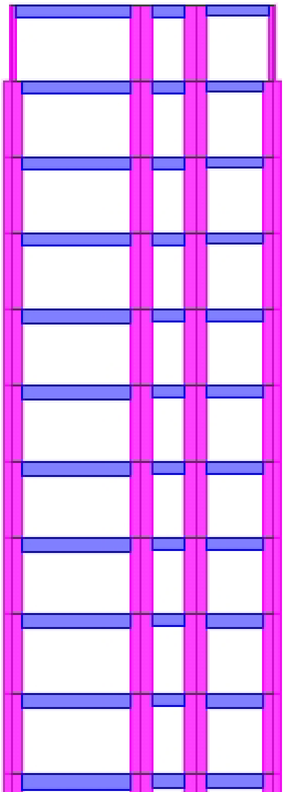


Figure 31: Transversal view: RC Jacketing

5.5.2. Retrofitting by introducing reinforced concrete shear walls

(Checking of the building lateral stiffness)

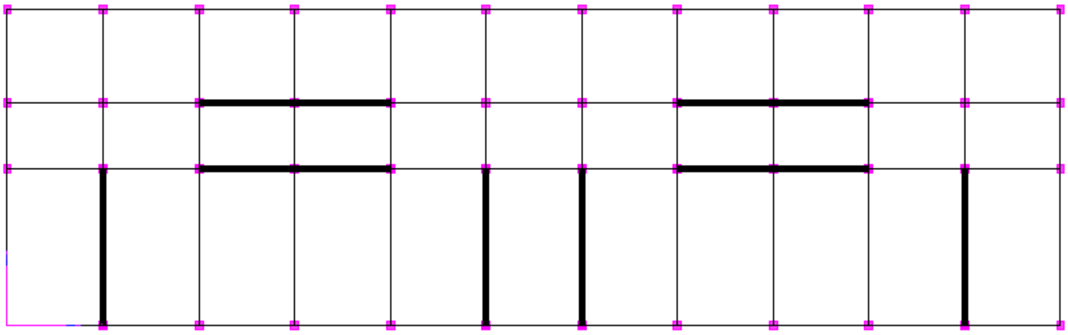


Figure 32: Plan view: RC walls

The modal analysis resulted as following:

- fundamental vibration in the transversal direction

T_{1x}= 0.79 s

- fundamental vibration in the longitudinal direction

T_{1y}= 0.83 s

Global seismic coefficient in the transversal direction:

c= α ks β ε ψ where β= 2.5-T+T_c

β= 2.41

c= 1.4 * 0.12 * 2.41 * 0.75 * 0.2

= 5.70%

Global seismic coefficient in the longitudinal direction:

c= α ks β ε ψ where: β= 2.5-T+T_c

β= 2.37

c= 1.4 * 0.12 * 2.37 * 0.75 * 0.2

= 5.84%

Table 12: Drift control for reinforced concrete shear wall

Story	Load	Level	Δ X	Δ/Ψ*1000	Δ Y	Δ/Y*1000
11	SEISMX / SEISMY	38.26	0.000637	3.2	0.000718	3.6
10		34.86	0.000637	3.2	0.000729	3.6
9		31.46	0.000658	3.3	0.00074	3.7
8		28.06	0.000654	3.3	0.000729	3.6
7		24.66	0.000653	3.3	0.000711	3.6
6		21.26	0.000636	3.2	0.000679	3.4
5		17.86	0.000587	2.9	0.000636	3.2
4		14.46	0.000521	2.6	0.000566	2.8
3		11.06	0.000431	2.2	0.000455	2.3
2		7.46	0.000291	1.5	0.000321	1.6
1		3.86	0.000005	0.0	0.000161	0.8

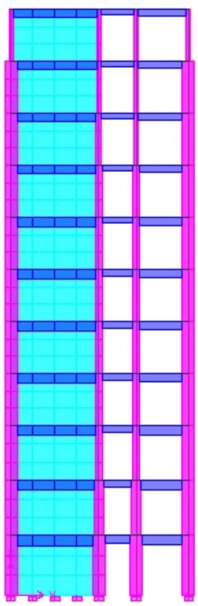


Figure 33: Transversal view: RC walls

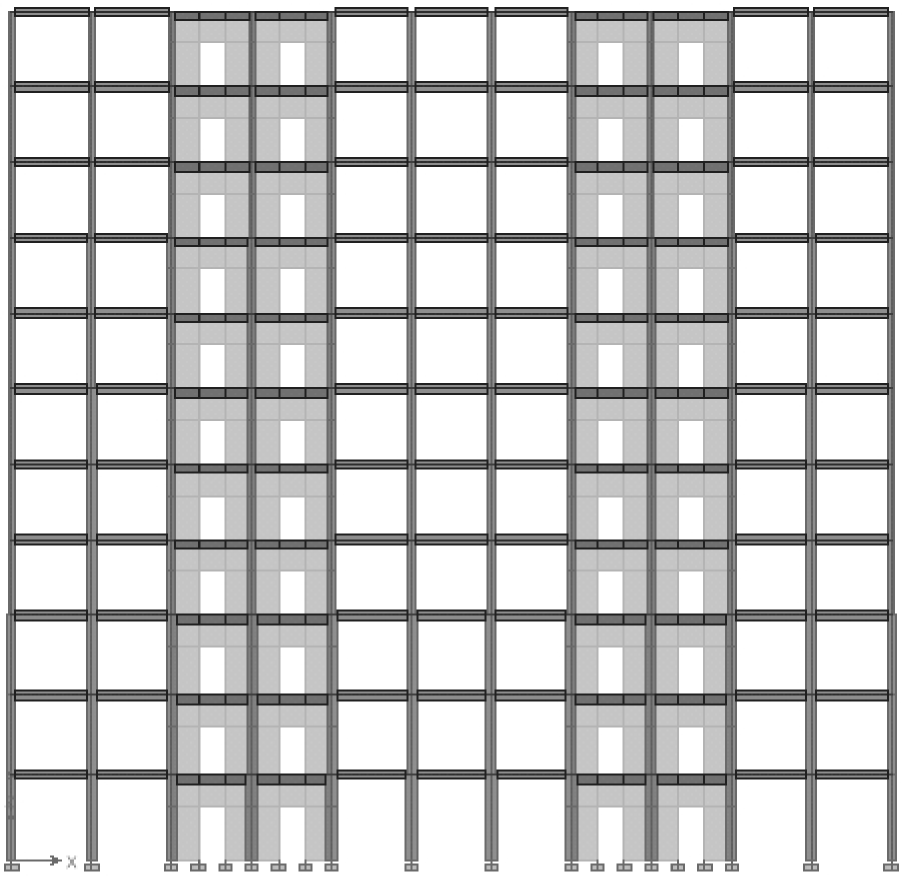


Figure 34: Longitudinal view: RC walls

5.5.3. Retrofitting by introducing steel braces

(Checking of the building lateral stiffness)

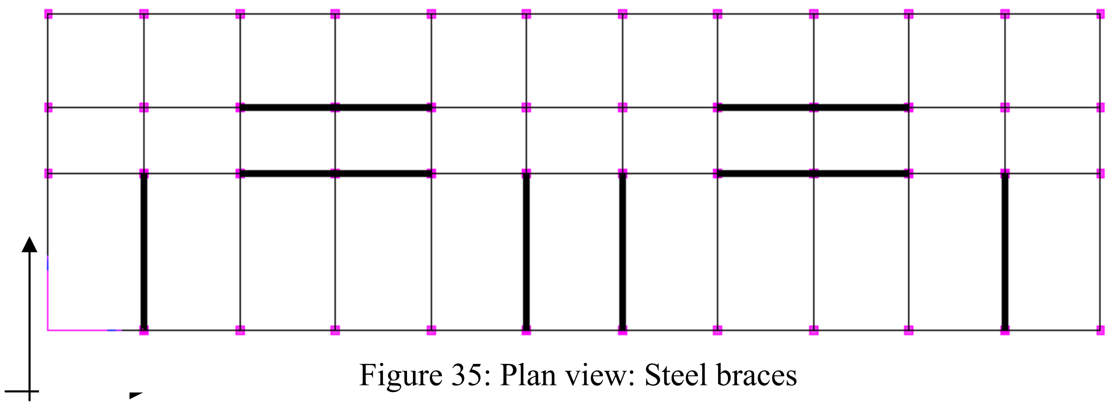


Figure 35: Plan view: Steel braces

After the modal analysis the following results have been obtained:

• fundamental vibration period in the transversal direction

T_{1x}= 1.01 s

• fundamental vibration period in the longitudinal direction

T_{1y}= 1.02 s

The global seismic coefficient in the transversal direction:

c= α ks β ε ψ Where: β= 2.5-T+T_c

β= 2.19

c= 1.4 * 0.12 * 2.19 * 0.75 * 0.2

= 5.52%

The global seismic coefficient in the longitudinal direction:

c= α ks β ε ψ Where: β= 2.5-T+T_c

c= 1.4 * 0.12 * 2.18 * 0.75 * 0.2

= 5.49%

Table 13: Drift control for steel bracing retrofitting

Storey	Load	Level	Δ X	Δ/Ψ*1000	Δ Y	Δ/Ψ*1000
11	SEISMX / SEISMY	38.26	0.000623	3.1	0.000806	4.0
10		34.86	0.000722	3.6	0.000858	4.3
9		31.46	0.000804	4.0	0.00091	4.6
8		28.06	0.000861	4.3	0.000922	4.6
7		24.66	0.000899	4.5	0.000905	4.5
6		21.26	0.000896	4.5	0.000877	4.4
5		17.86	0.000873	4.4	0.000839	4.2
4		14.46	0.000823	4.1	0.000776	3.9
3		11.06	0.000756	3.8	0.000659	3.3
2		7.46	0.000541	2.7	0.000506	2.5
1		3.86	0	0.0	0.000284	1.4

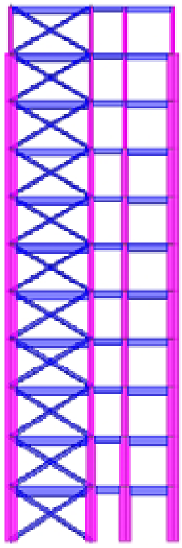


Figure 36: Longitudinal view: RC walls

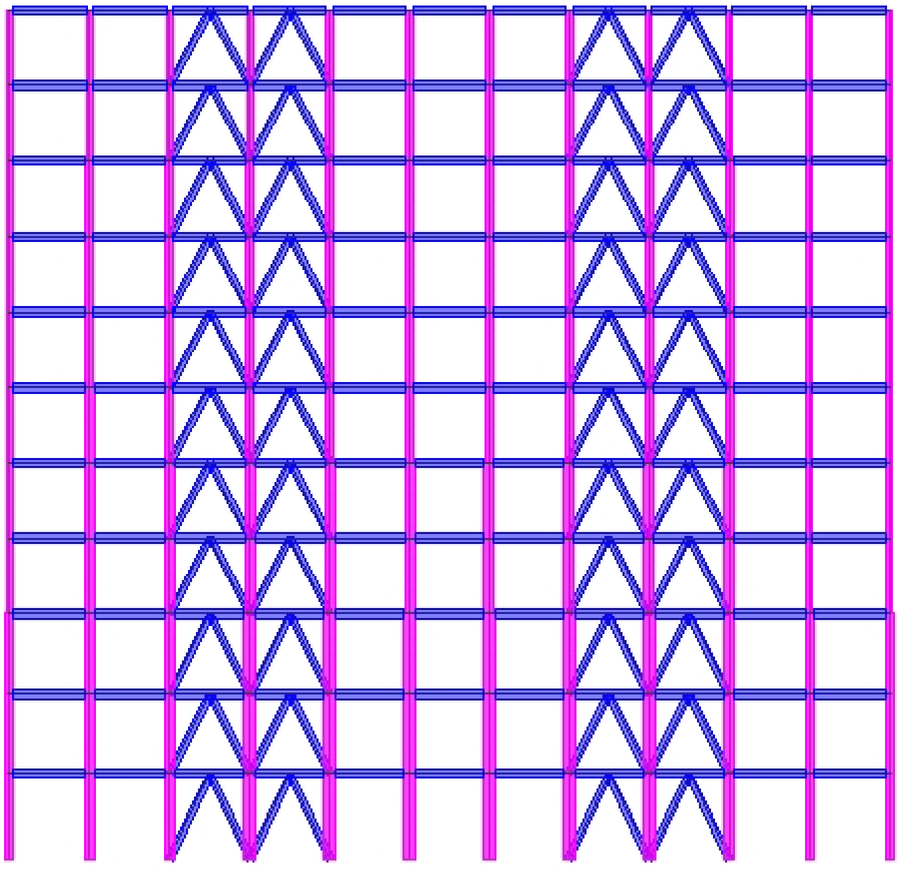


Figure 37: Transversal view: V braces

5.6. Comments on the selected solutions

The seismic evaluation of the existing building underlined the following structural deficiencies:

- lack of strength for beams and columns
- lack of ductility for beams and columns
- a reduced lateral stiffness of the RC frame
- detailing deficiencies of the reinforcement – insufficient anchorage or splices, low percentage of transversal reinforcement .

Therefore the retrofitting strategy was selected to accommodate some of the following requirements:

- increase the lateral stiffness of the structure.
- increase the strength of some beams and columns
- increase the overall structural strength
- increase the ductility of the structural members

Three methods of retrofitting have been proposed and commented hereinafter.

5.6.1. Retrofitting by beams and columns RC jacketing

First solution consists of RC jacketing of

- internal columns
- transversal across the central span
- longitudinal beams along the central span

Basically a strong RC frame was intended for the central span. Such a solution meet the functionality requirements for a hospital, given that most of the working space has not been affected.

The jacketing width have selected based also on the functionality requirements:

- columns with constant section of 105x75 cm
- transversal beams of 75x55
- longitudinal beams of 45x60

This procedure was used to mitigate the following deficiencies:

- strength
- stiffness
- ductility
- reinforcement detailing

5.6.2. Retrofitting by RC walls

Second solution consists of introduction of RC wall as infilled panels in some of the transversal and longitudinal frames. The connection between the infilled panels and the existing frames can be made by chemical anchors.

The new structural system can be considered a dual system since both the RC shear walls and RC frames will resist together the seismic forces. In the longitudinal walls door openings had to be introduced in accordance with the functionality of the building.

In this case special care has to be paid to the anchorage of the reinforcement. Local retrofitting work has to be used to improve the behavior of the anchorages and the reinforcement in the splices regions.

5.6.3. Retrofitting by steel bracing

The third solution consists of introducing steel braces in the frames openings. To meet the functionality requirements two types of braces have been selected:

- for the transversal direction "X" shaped braces since no doors are needed
- for the longitudinal direction "V" shaped braces to accommodate doors.

The connection between the braces and the existing RC frames is made through a layer of non-shrinkage mortar and by using steel headed anchors on the steel braces and chemical anchors in the reinforced concrete elements.

5.7. CONCLUSIONS

This study showed that all the proposed solutions are feasible for the selected structure. Advantages and disadvantages can be concluded from the previous paragraph.

The authors opinion is that RC jacketing of the frames members in the central span (solution 1) is the best option of three presented here. Installing new RC shear walls or steel braces requires a major intervention in the underground floor and in the foundation system. Also, the anchorage deficiencies are difficult to be solved with local interventions.

However, any retrofitting strategy should take into account the functionality requirements, the overall intervention costs, the availability of the retrofitting materials and the technical know-how for design and construction.

6. References

1. *Romanian Code for earthquake resistant design of buildings - P100/92*, MLPAT, 1992, 1996
2. *Methodologies for the Seismic Evaluation of Existing Buildings (Draft Version)*, Technical University of Civil Engineering Bucharest, Reinforced Concrete Department, 2001
3. *Guidelines For Seismic Rehabilitation (Draft Version)*, Technical University of Civil Engineering of Bucharest, Reinforced Concrete Department, 2001