MASONRY MECHANICS
-ASPECTS REGARDING THE REDUCTION OF STRUCTURAL VULNERABILITIES IN EXISTING BUILDINGS WITH STRENGTH STRUCTURE OF NON-REINFORCED MASONRY (SINGLE) -

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The need to assess the seismic safety of masonry buildings is obvious and urgent because they constitute an important part of the existing built stock in Romania and, by their constructive nature, they present a high level of seismic vulnerability.

The 1992 census recorded, for residential buildings in Romania, the following categories of buildings with masonry walls:

☐ M2 - constructions with brick, stone walls or substitutes with reinforced concrete floors;

☐ M3 - constructions with brick, stone walls or substitutes with wooden floors;

Depending on these categories of materials, the houses in the cities with a masonry structure, related to the total number of houses, were recorded as follows (rounded values):

Houses with a masonry structure:

Material	Total houses	House in buildings P, P+1E	House in buildings P+2E
Total	4.000.000 (100%)	1.100.000 (27.5%)	2.900.000 (72.5%)
M2	900.000 (22.5%)	230.000 (6.0%)	670.000 (16.5%)
M3	500.000 (12.5%)	480.000 (12.0%)	20.000 (0.5%)

The percentages refer to the total number of houses (4,000,000)

From the table it follows that, from the point of view of the weight in the existing built housing stock, masonry buildings represent over 1/3 of the total.

Specific information needed to assess the safety of masonry constructions

General construction data

The information regarding the date (period) of the execution serves to identify the over time degradation premises of the quality of the built fund (including degradation due to non-seismic causes) and the available safety level of the buildings.

The data related to the year of construction are grouped, as a rule, into five major stages, each with certain characteristic elements, essential for assessing the level of vulnerability:

A. Initial seismic protection (by design).
B. The number and severity of earthquakes suffered from the date of construction until the time of expertise.

These stages are:

• Before 1944: buildings without initial seismic protection and which endured the 1940 earthquake

• Between 1945 ÷ 1960: buildings without initial seismic protection but which did not withstand the 1940 earthquake

• Between 1961 ÷ 1978(80): buildings with insufficient initial seismic protection, corresponding to norms P13-63 and P13-70 and the seismic zoning map from STAS 2963-63

• Between 1981-1991: buildings with satisfactory initial seismic protection corresponding to the P 100-78(81) standard and the seismic zoning map from STAS 11100/0-77.

After 1992: buildings with good initial seismic protection, ensured by Normative P 100-92 and the seismic zoning map therein.

The construction system with structural masonry walls is characteristic of the oldest religious or laic buildings existing in Romania.

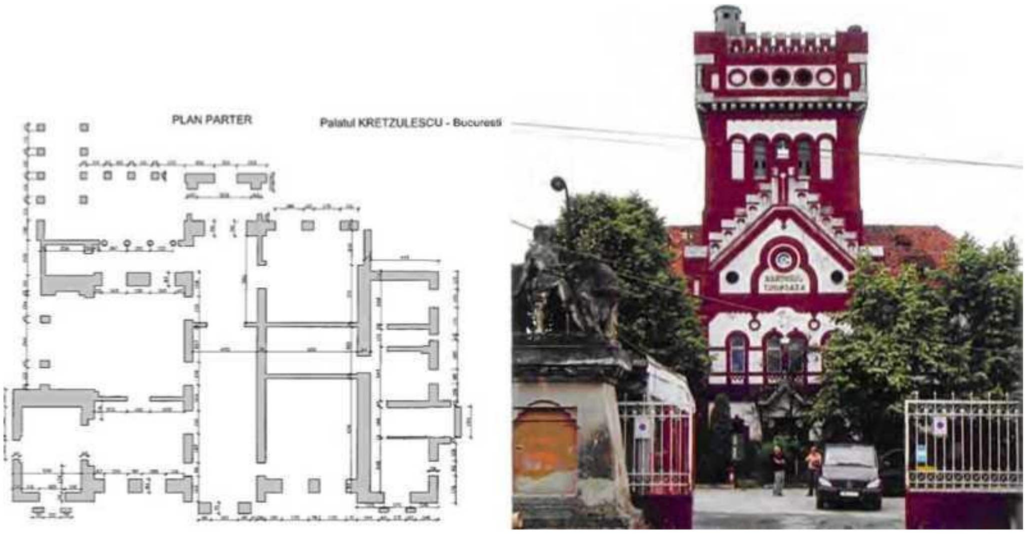
In the field of laic buildings, as a destination, buildings with structural masonry walls cover the entire range of residential, social-cultural and industrial functions.

In the existing built stock, several distinct categories of buildings with structural masonry walls can be identified that have common characteristics and as such have similar levels of vulnerability.

• Modest residential buildings, for one or two families, with a ground floor or a ground floor and a floor (also called cheap housing).

• Modest residential buildings with small commercial units on the ground floor (the historic center of the Capital, for example)

• Individual residential buildings for the wealthy (the so-called "palaces", for example, in Bucharest: Stirbey, Ghica, Cantacuzino, etc.).



Representative masonry buildings (a) Kreţulescu Palace, str, Ştirbei Vodă, Bucharest (b) Abattoir in Timisoara -1905

• Small and moderate-sized public buildings for administration, education, culture (schools from the "Spitu Haret" program, financial administration buildings, etc.).

• Monumental public buildings (for example, in Bucharest: the Court, the CEC headquarters, the Military Circle, etc.).

• Industrial buildings with small and moderate sizes.

From the point of view of architectural-structural compliance, buildings with structural masonry walls are characterized by a wide variety of shapes both in plan and in elevation.

The composition of the structures for this category of buildings generally fits into one of the following categories:

• Buildings with structural walls of simple (unreinforced) masonry and floors of:

- massive masonry arches;
- laminated metal profiles and brick arches;
- wooden beams;
- monolithic reinforced concrete;
- small prefabricated elements.

• Buildings with structural masonry walls with belts and reinforced concrete columns (confined masonry) with floors of:

- monolithic reinforced concrete;
- small prefabricated elements;
- large-sized prefabricated elements (semi-panels, panels, drawers with concrete overlay).Clasificarea dată mai sus poate fi asociată cu evoluţia în timp a tehnicilor şi materialelor de construcţii.

The composition of buildings with structural masonry walls made at the end of the 19th century and the beginning of the 20th century is exclusively based on the "gravitational" concept and is mainly characterized by:

- uneven placement of the walls in plan, which leads to pronounced dissymmetries;
- uneven composition of the walls in the vertical plane;
- door and window gaps that do not overlap vertically (on interior walls and, often, on facades as well);
- discontinuities in the flow of vertical forces (internal and external walls resting on beams - mostly due to subsequent interventions);
- reductions in strength and rigidity through vertical gaps (for chimneys or ventilation) or horizontal slits for pipes (in case of subsequent interventions);

Due to the reduced capacity to dissipate seismic energy, due to low tensile strength and reduced ductility, it is recommended that the use of structures with non-reinforced masonry walls (ZNA) be avoided.

The use of non-reinforced masonry structures (ZNA) for buildings of exposure class III and IV is not allowed, regardless of the number of levels above the embedment section (n_level), in seismic areas with $a\_g \geq 0.16g$.

Structures with unreinforced masonry walls with $n\_level > 2$ in seismic areas with $a\_g \leq 0.12g$ and with any height regime for areas with $a\_g \geq 0.16g$ regardless of the material used or the geometric and mechanical characteristics of the masonry elements, will be provided with pillars and reinforced concrete belts in the positions indicated below.

The maximum number of levels above the embedment section (n_level) for non-reinforced masonry buildings (ZNA) and the associated minimum constructive value of the density of the structural walls - interior and exterior - (p%), on each of the main directions, depending on the seismic acceleration (ag) are given in the following table:

n_{niv}	Design seismic acceleration					
	0,08g...0,12g		0,16g...0,20g		0,24g...0,32g	
	Burnt clay gr.1 şi 2	Burnt clay gr. 2S şi BCA	Burnt clay gr.1 şi 2	Burnt clay gr. 2S şi BCA	Burnt clay gr.1 şi 2	Burnt clay gr. 2S şi BCA
1(P)	>=4,0%	>=4,5%	>=4,5%	>=5,0%	>=5,0%	NA
2(P+1E)	>=4,5%	>=5,0%	>=5,0%	>=5,5%	NA	NA
3(P+2E)	>=5,0%	>=5,5%	NA	NA	NA	NA

NA – not accepted

The density of the structural walls of the masonry buildings, on each of the main directions of the building, is defined as the percentage of the total net area of the structural walls (A_(z,net)) on the respective direction, related to the floor area (A_pl) on the respective level :

$$p(\%) = 100\,\frac{A_{z,net}}{A_{pl}}$$

The values in the table above refer to the first level above the embedment section. For the following levels, a reduction in the density of the walls is accepted, keeping the conditions of regularity in elevation. In the case of buildings with a resistance structure made of patches of unreinforced masonry, the attic is considered level.

Structures with unreinforced masonry walls with fired clay elements groups 1,2 and 2S and BCA can be used in all seismic zones for:

- Constructions with a single level above the built-in section with the function of household annexes
- Temporary constructions with little usage date.

The issue of out-of-plane stability of walls subjected to seismic excitation is unusually not well highlighted in Eurocode 8, namely at the point where the seismic loading (action) is not clearly defined and the engineer will have to find his own way to verify safety, resorting for example to the seismic action defined for non-structural elements (as proposed by Tomazevic, 1999).

Nr. crt.	High vulnerability	Vulnerabilitate scăzută
1	Poor quality of materials (weak mortar, weak/brittle bricks), poor "internal connections" of masonry (multi-layer masonry without transverse connections)	Regular and robust units, close and closed connection of the units, the masonry behaves like a monolith through the entire thickness of the wall.
2	Very fragile walls, (out-of-plane instability)	Limiting the slenderness of the walls, preventing (restrictions) from yielding out of plan
3	Lack of effective connections between walls and between walls and horizontal structures, lack of structural redundancy	Good connections at the joint of the walls, the presence of braces and belts (rigid washers) at each floor (and roof) to favor the "rigid box effect", efficient floor-wall connections that reduce effort concentrations
4	The floors cannot ensure a diaphragm type behavior	Diaphragms with sufficient rigidity and strength to prevent out-of-plane wall vibrations, increase structural redundancy and favor internal force redistribution.
5	The presence of horizontal thrusts balanced by the out-of-plane resistance of structural walls (e.g. from roofs, from arches or arched type of structures)	horizontal thrusts are counteracted by thick walls/ or by suitable structural elements (floors, diaphragm plates) to form a balanced and "closed" system.
6	Excessive openings without intermediate supports between floors, walls located at large and irregular distances	Limited slab openings, shear walls regularly arranged in at least two orthogonal directions
7	Large structural and non-structural masses and low material strength	Masses and weights produce a low effort/resistance ratio
8	Structural irregularity in plan (torsion effects) and in elevation (inefficient load direction, effort concentrations)	Regular structure, sufficient torsional strength with regular direction of forces from top to foundation

The factors influencing the seismic vulnerability of masonry buildings [1]

Table 1.

It can be said that precise slenderness limitations, minimum thickness requirements and appropriate structural design and details (rigid diaphragms, effective floor-to-wall connection) can in most cases guarantee the prevention of out-of-plane failures; however, on the one hand, such thickness and slenderness limitations are nationally established parameters that can vary significantly from one country to another, and on the other hand, out-of-plane stability is also a problem for "secondary" seismic elements and non-structural partitions which cannot correspond to such limitations. Figure 1 shows some examples of local damage mechanisms.

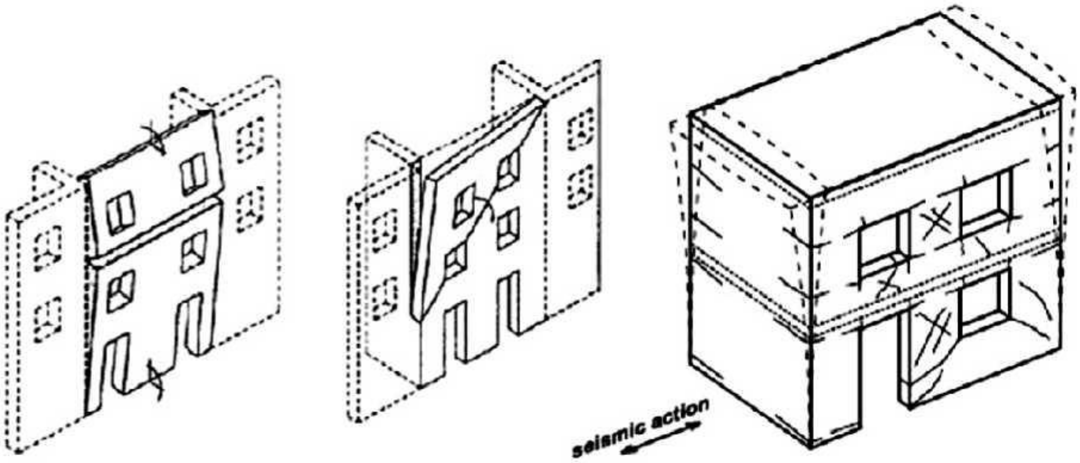


Fig. 1. Examples of the first "local" damage mechanisms (left, from D'Ayala & Speranza, 2003) and the global response mechanism (right)

Recently, practical and theoretical research has confirmed that the out-of-plane response and stability of walls subjected to seismic excitation, when ultimate conditions are considered, is more a problem of displacement effect on displacement capacity than a strength problem.
The problem is very complex, and it is necessary to evaluate:
- the seismic requirements regarding the walls considering the effects of the dynamic filtering of the building and the diaphragms and the dynamic response of the wall;
- the resistance of the walls compared to the out-of-plane forces and the relevant resistance mechanisms;
- the ability to move out of the plane of the walls;

Most of the research in the past has been mainly focused on shear strength which can come from three possible sources: vertical compression, apparent bending strength in one or two bending directions, thrust (or bowing). Considering the simplest way, the vertical bending condition is that in an unreinforced masonry wall (URM) the apparent bending resistance is given by the smallest value between the vertical stretching resistance of the horizontal joints and that of the bricks.

The realization of the crack, which has already developed under the service load, given for example by the eccentricity of the vertical loads, does not necessarily imply the collapse, and can be regarded as a limit stage of damage. In the post-cracking regime, the lateral resistance is given by the presence of vertical compression and could be sensitive to second-order geometric effects.

An appropriate safety assessment will be based on the main characteristics of the lateral response, namely the initial stiffness, the maximum force, the displacement at static instability (fig. 2) [2].

The behavior of the subsystem is close to the nonlinear elastic one, with moderate energy dissipation.

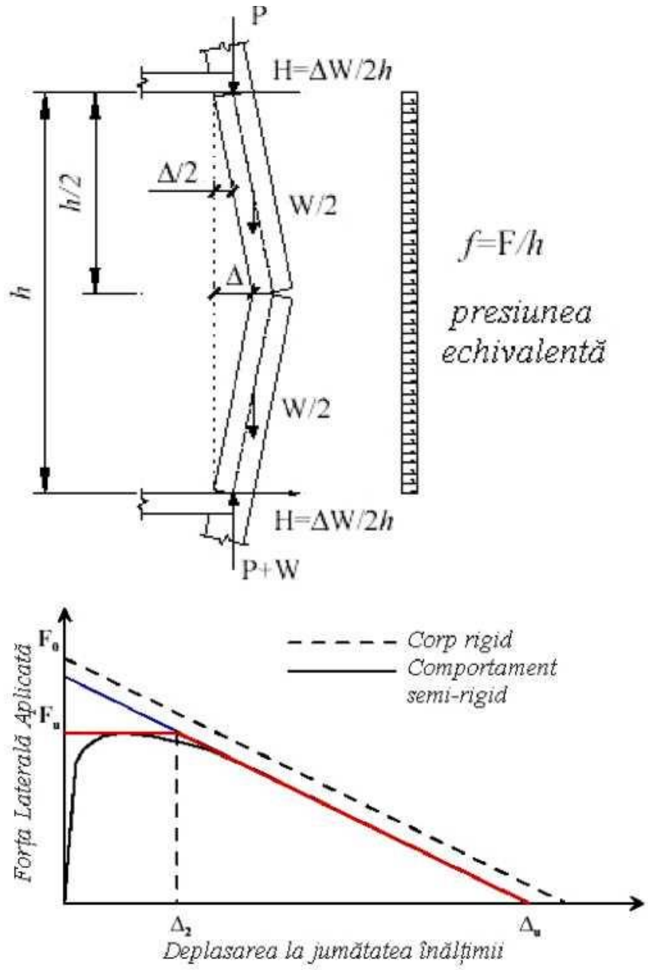


Fig. 2. Force diagram - post-cracking displacement of the URM wall during vertical bending.

STRUCTURAL DESIGN OF STORIED BUILDINGS WITH WALLS MASONRY BEARINGS

Principles of architectural and structural composition of current storied buildings

The composition of the structures of masonry buildings results, mainly, from the composition of the architectural plan, the design of buildings with structural masonry walls located in seismic zones involving the completion of an iterative process of "proposal-evaluation" in which the architect and the structural engineer.

Choosing the overall configuration of the building is the main attribution of the architect.

The design of the structure falls to the structural engineer, but it cannot be independent of the functional and plastic requirements formulated by the investor and the architect.

The preliminary architectural-structural design is also a design phase with a pre-dimensioning character, which precedes the verification by calculation of structural safety, and which conditions, among other things, the choice of the model and the method used for the calculation of the action of vertical and horizontal loads.

The architectural-structural preliminary design involves the following stages:

- Establishing the general shape of the building in plan and elevation.
- Preliminary design of the vertical superstructure (assembly of structural walls).
- Preliminary design of the floorboards.
- Preliminary infrastructure design.

In the preliminary architectural-structural design phase of masonry buildings, the plan shape and volume of the building, the distribution of spaces, the location and composition of the structural walls should be chosen in such a way that the seismic response of the building is favorable and can be determined by calculation with sufficient accuracy , using current (simple) models and methods.

For areas with design seismic acceleration ag ≥0.20g, it is recommended to choose plan and volumetric configurations that lead to buildings with structural regularity in plan and vertically.

Composition of the building in plan and elevation

It is recommended to adopt compact partitions, with geometric symmetry (given by the shape in plan) and with mechanical symmetry (resulting from the arrangement of the structural walls in plan) or with limited dissymmetry, which fall within the limits of figure 1.1.

The adoption of such forms is mandatory in the case of buildings with structural masonry walls founded directly on difficult terrain (PUCM, PSU).

The floor area will be kept constant at all levels of the building. Area reductions can be made, from one level to the immediately higher level, of about 10 ÷ 15%, provided that the drainage route

loads to the foundations should not be interrupted (for example, by resting a structural wall on the floor).
Buildings with structural masonry walls will be constructed in such a way as to create a spatial structure consisting of:

• vertical elements: structural walls arranged, at least, in two orthogonal directions;

• horizontal elements: floors which, as a rule, constitute a rigid diaphragm (washer) in the horizontal plane.

The connections between the floors and the structural walls that are made, depending on the type of masonry and non-reinforced masonry (ZNA): through reinforced concrete belts cast on all walls;

The rigidity of the structure will be approximately equal on the two main directions of the building; recommends that the difference between the respective stiffnesses not exceed 25%.

The strength and stiffness of the building will be kept approximately constant throughout the height of the building.

Any reductions in strength and stiffness must not exceed 20% and be achieved by reducing:

• the density of the walls;
• wall thicknesses;
• masonry resistance to compression.

Structural regularity criteria

The structural system will be simple, continuous and will have sufficient capacity for resistance and rigidity to ensure a direct and uninterrupted path of vertical and horizontal forces, up to the buried ground.

Masonry buildings are considered structurally regular in plan if:

• the plan form satisfies the following criteria:

- it is approximately symmetrical in relation to 2 orthogonal directions;
- it is compact, with regular contours and as few corners as possible;
- any withdrawals/protrusions in relation to the current contour of the floor do not exceed, each, the greater of the values: 10% of the area of the floor or 1/5 of the size of the respective side;

• the plan distribution of the structural walls does not lead to important dissymmetries of the lateral stiffness, of the resistance capacities and/or of the permanent loads in relation to the main directions of the building;

• the rigidity of the floors in the horizontal plane is high enough to ensure the compatibility of the lateral movements of the structural walls under the effect of horizontal forces;

• on the ground floor, on each of the main directions of the building, the distance between the center of gravity (CG) and the center of rigidity (CR) does not exceed 0.1 L where L is the size of the building in the direction perpendicular to the calculation direction.

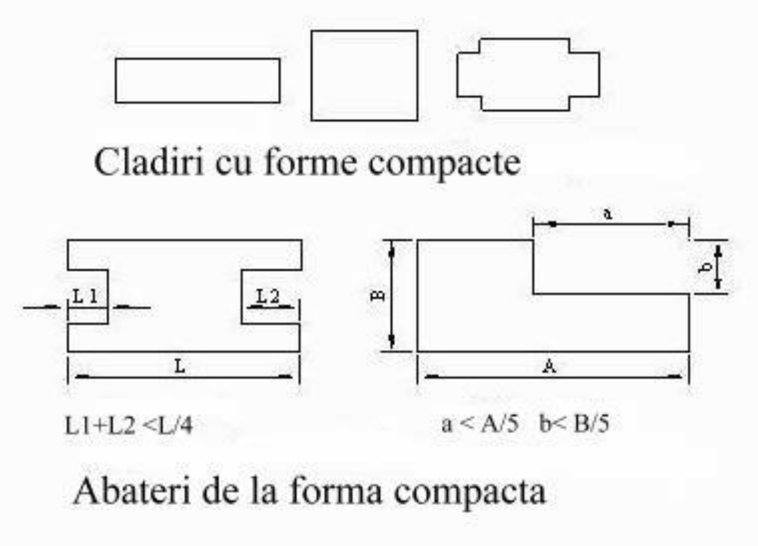


Figure 3 Conditions of structural regularity in the plan

Masonry buildings are considered structurally regular in elevation if:

• the heights of adjacent levels are equal or vary by no more than 20%;

• the structural walls have, in plan, the same dimensions at all above-ground levels or have variations that fall within the following limits:

- reducing the length of a wall compared to the lower level does not exceed 20%;
- the reduction of the total net areas of masonry on the upper levels, for buildings with level ≥3 does not exceed 20% of the area of the masonry on the ground floor;

• the building does not have "weak" levels (which have less rigidity and/or resistance capacity than those of the upper levels).

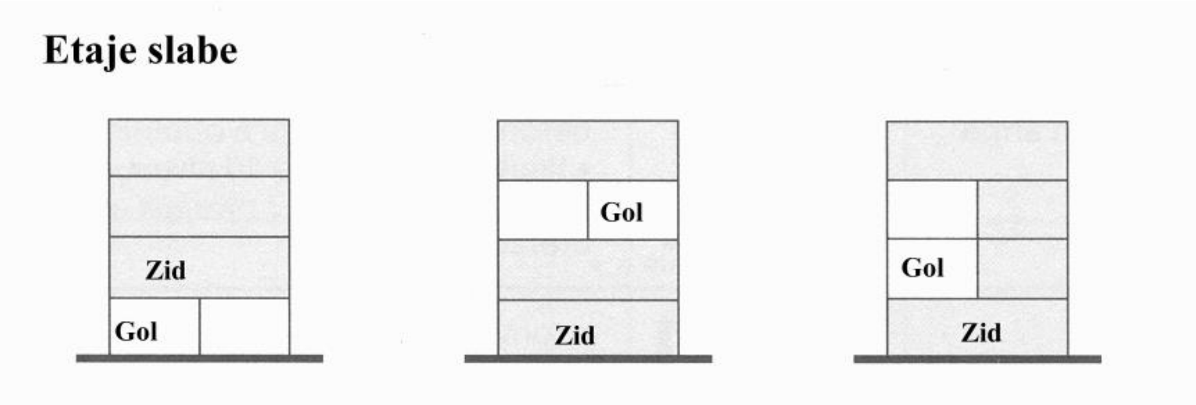


Figure 4 Buildings with "weak" levels (structural irregularity in elevation)

Buildings that do not meet these conditions are considered without structural regularity, as the case may be, in plan or in elevation.

For design (calculation and constructive detailing) in accordance with the provisions of Code CR6, buildings with structural masonry walls are classified into regularity groups as follows:

Classification of buildings with structural masonry walls into regularity groups

Table 1.1

Regularity group of the building	Regularity		
		Plan	Elevation
Regular buildings	1	Yes	Yes
	2	No	Yes
Irregular buildings	3	Yes	No
	4	No	No

Separation of the building into sections

Separation of the building into sections is necessary if:

• the length of the building exceeds the established maximum values;

• the shape in plan has irregularities that exceed the limits of figure 1.1.;

• the land on which the building is located has irregularities (stratification, consistency, local fillings, etc.).

It is recommended that the ratios of the main dimensions of the beams resulting from the fragmentation of the building with joints fall within the limits:

- height / width ≤ 1.5;
- length / width ≤ 4.0.

The separation joints between the adjacent buildings / sections are classified according to the structural role and the way they develop vertically in the building:

• complete joints, which cross both the superstructure and the infrastructure:
- settlement joints, which have the role of limiting the efforts in the structure due to the uniformity of the foundation land and/or the value of the building settlements in the case of foundation on difficult terrain;
• partial joints, which are made only in the superstructure:

- seismic joints, which mainly have the role of eliminating or reducing the negative effects of the overall torsion in the case of buildings with complex shapes in plan; in the case of buildings with a particularly long length, the seismic joints will also cross the foundations in order to avoid the effects of the non-synchronism of the seismic movement at the foundations located at relatively large distances;

- contraction-expansion joints, which have the role of limiting the efforts that may result from temperature variations or as an effect of phenomena specific to masonry/concrete.

The joints between the sections will be made by doubling the structural walls, they will be flat and they will completely separate both the structural and non-structural elements of the building.

The size of the free space between the construction elements of the adjacent sections is determined by calculation, according to the provisions of Code P100-1/2006, Chapter 4.

The closing of the free space between the sections will be done with materials or devices that do not prevent the relative movement of the adjacent sections, are impermeable to water and air, do not allow propagation

fire and are acceptable in terms of appearance. It is not allowed to close the joint with plaster.

Maximum dimensions of buildings

Maximum dimensions in plan

For masonry buildings founded on normal land, the maximum length of the sections will be 50.0 m.
For buildings founded on difficult foundations, the maximum length of the sections is established by the specific regulations, P 7-2000 and/or NP 001-2000.

Maximum dimensions in elevation

The maximum number of levels (nniv) above the embedment section and the associated minimum constructive value of the density of the structural walls (p%), are limited, according to Code P 100-1/2006, according to:

- design seismic acceleration at the site (ag);
- structural regularity/irregularity class defined in point 5.1.3.;
- importance class of the building;
- type/composition of masonry (ZNA, ZC, ZC+AR, ZIA);
- density of the structural walls p%;
- the type and group of masonry elements (1, 2, 2S), established according to Chap. 3

In the case of ZNA buildings, the attic is considered a "level", which is included in the total number admitted according to Code P 100-1/2006., even if they meet the above conditions.

In the case of reinforced masonry buildings (ZC, ZC+AR and ZIA) with an attic above the last current level, this is not included in the maximum number of levels (nniv) allowed according to Code P 100-1/2006, if the following conditions are met:

- the minimum constructive density of the walls is increased by 1.0%;
- the perimeter masonry walls do not exceed the average height of 1.25 m;
- partition walls are lightweight (plasterboard or similar);
- the wooden frame is designed so that there are no pushes in the perimeter walls;
- the masonry of the structural walls in the attic is confined with reinforced concrete pillars in continuation of those on the lower level;
- a reinforced concrete belt is provided at the upper part of the masonry walls of the attic.

If at least one of these conditions is not met, the attic will be considered "level" and the building will fit, in terms of the height and density of the structural walls, accordingly.

If on the floor above the last current level of the building there are annexes (dryers, washing rooms, etc.) that occupy less than 20% of the area of the current floor and whose height is not higher than its height, the respective rooms will be considered as an projection of the main building (they will not be considered as "level" within the limits indicated above).

Adoption of the structural system

The choice of the structural wall system

The choice of the structural wall system is made in such a way as to achieve, at the same time, the satisfaction of the following categories of requirements:

- functional, established by the investor: dimensions of free spaces, level height, type of circulations, etc.;

• of comfort;
• structural safety.

The density of the structural walls of the masonry buildings, on each of the main directions of the building, is defined by the percentage of the total net area of the masonry walls (Az,net) on the respective direction, related to the floor area (Apl) of the respective level

$$P(\%) = 100 \cdot \frac{A_{z,net}}{A_{pl}}$$

In the case of floors that unload in one direction (wooden floor, floor with metal beams, floor made of linear prefabricated elements of reinforced concrete) the walls parallel to the direction of the floor element are defined as "structural bracing walls" which mainly have the role structural to take over the horizontal forces in the respective direction.

The structural walls that make up a masonry structure are of two categories:

• insulated walls (uprights), connected to each other, at each level, only with the floor plate;
• coupled walls (with door and/or window gaps), consisting of uprights (slats) connected to each other, at the level of each floor, by reinforced concrete coupling beams.

Depending on the distance between the structural walls, the buildings can be designed structurally as follows.

Structures with walls though

Structures with solid walls (honeycomb system) are defined by the following geometric parameters:

• level height ≤ 3.20 m;
• maximum distances between walls, in the two main directions ≤ 5.00 m;
• the area of the cell formed by the walls on the two main directions A≤ 25.0 m2.

In this composition, as a rule, the positions in the building of the internal structural walls result from the architectural plan concept (separate the main rooms of the building).

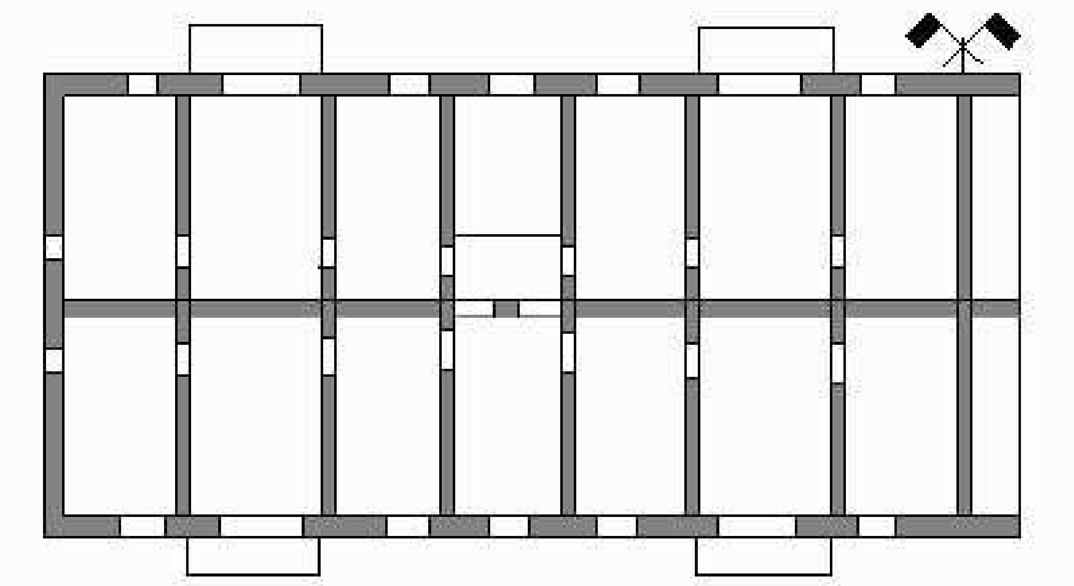


Figure 5 Structures with thin walls (honeycomb system)

If, at some level of a building with though walls, larger spaces are needed, locally, it is accepted to achieve this requirement by suppressing a structural wall at the respective level with the obligation to suppress this wall and at all higher levels so that avoid the formation of a "weak" floor. If, through this operation, the area of the structural walls on the respective direction is reduced by more than 20%, the building will be included in the class of buildings without vertical regularity.

The use of the wall system is recommended in the case of buildings founded on difficult terrain.

Thin-walled structures

Structures with rare walls (cellular system), are defined by the following geometric parameters:

• level height ≤ 4.00m;

• maximum distances between walls, in the two main directions ≤ 9.00 m;

• the area of the cell formed by the walls on the two main directions A≤ 75.0 m2.

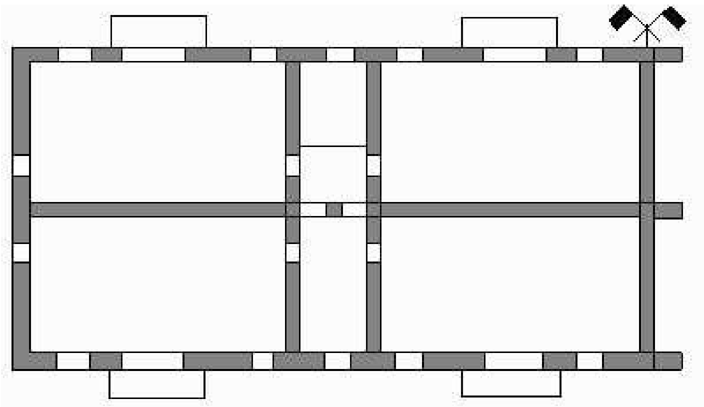


Figure 6 Thin-walled structures (cellular system)

In this arrangement, the internal structural walls are usually arranged at the border between the functional units (between apartments - at homes, between classrooms - at educational units, etc.), which eliminates, in most cases, their weakening with gaps pass.

Hall/hall type structures

"Hall/hall" type structures with small openings usually have the following overall geometric parameters:

- ground floor height regime;
- maximum distances between walls ≤ 18.0 m;
- level height ≤ 9.00 m.

A building defined as a "hall" type can have a reduced area, with a different type of structure distribution

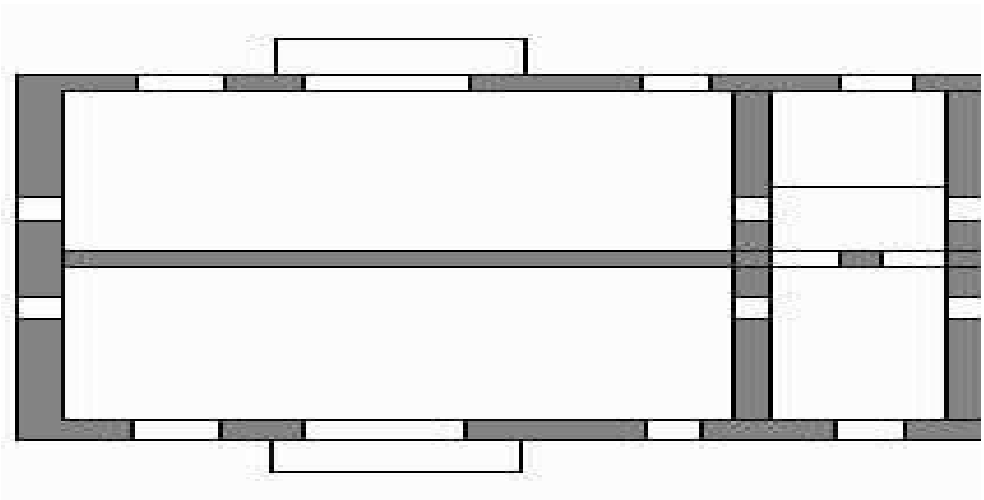


Figure 7 The structure with walls arranged in the "hall system"

- Simple/unreinforced masonry (ZNA): masonry that does not contain enough reinforcement to be considered reinforced masonry - such as confined masonry, confined masonry and reinforced in horizontal joints, masonry with reinforced heart.

Masonry elements

- Class I masonry element: masonry element for which the probability of not reaching the declared compression strength is ≤5%.
- Class II masonry element: masonry element that does not meet the reliability level of class I masonry elements.

Masonry walls

- Structural wall: wall designed to resist vertical and horizontal forces acting mainly in its plane.
- Stiffening wall: a wall placed perpendicular to another wall, with which it works to take over the vertical and horizontal forces and contributes to ensuring its stability; in the case of buildings with

floors that unload in one direction, walls parallel to the direction of the element, which are not directly loaded with vertical forces, but which take the horizontal forces acting in their plane, are also defined as bracing walls

- Non-structural wall: wall that is not part of the main structure of the construction; this type of wall can be removed, without prejudicing the integrity of the rest of the structure, but only after a specialized technical expertise.
- Infill wall: wall that is not part of the main structure but which, under certain conditions (detailed in Code P100-1/2006), contributes to the lateral rigidity of the construction and to the dissipation of seismic energy; the suppression during building operation or the creation of door/window gaps in a wall of this type can only be done on the basis of a specialized project, a calculation justification and with appropriate constructive measures.

When designing buildings with structural walls made of masonry, the choice of the type of masonry (composition of the masonry) for the structural walls will be made in compliance with the limit conditions depending on:

- the number of above-ground levels (nniv);
- the structural regularity of the building;
- group of masonry elements;
- design seismic acceleration at the site (ag);

as well as depending on the technological possibilities of execution.

The types used by bricklayers in the usual buildings, with a single-story or multi-story system, are those described below.

Unreinforced masonry (ZNA)

Due to the low capacity to dissipate seismic energy, due to low tensile and shear strength and low ductility, it is recommended that the use of reinforced masonry structures (RMS) be avoided.

Non-reinforced masonry structures (ZNA) with ceramic elements from groups 1, 2 and 2S can be used, under the established conditions, regarding the design seismic acceleration (ag), the number of levels (nniv) and the minimum constructive density of the structural walls (p %) in both directions, only if all the conditions below are met:

- the building falls into the category of "regular buildings with regularity in plan and elevation";
- the building falls into importance classes III or IV;
- the system of placing the walls is of the "even walls" type (honeycomb system);
- hetaj level height ≤ 3.00 m;
- the requirements for the construction of masonry and slabs are respected;
- the qualities of the materials used are those specified in Chapter 3.

ZNA structural walls design

Layout of the structural walls

The plan arrangement of the structural walls will be made as uniform as possible in relation to the main axes of the building, to avoid the unfavorable effects of the overall twisting. To ensure resistance

and torsional rigidity, it is recommended that the structural walls with high rigidity be placed as close as possible to the contour of the building.

For the same purpose, in the case of rectangular sections, it is recommended that the transversal structural walls at the ends of the sections be weakened as little as possible by gaps.

It will be ensured that, in the plan of the building, the walls with complex shapes with a single axis of symmetry (L,T) have the bases symmetrically placed with respect to the main axes of the building.

The thickness of the structural walls

The thickness of the structural walls will be established, through specialized calculations, to satisfy the following requirements:

- structural safety;
- thermal insulation/energy saving;
- soundproofing;

• fire protection.

The minimum thickness of the structural walls, regardless of the type of elements from which the building is made, will be 240 mm. From the point of view of structural safety, regardless of the calculation results, the ratio between floor height (het) and wall thickness (t) must satisfy the following minimum conditions:

• unreinforced masonry (ZNA) het/t ≤ 12;

• confined masonry (ZC) het /t ≤ 15.

Apart from this condition, the thickness of the walls subjected predominantly to the axial force must also meet the design requirements for mechanical resistance and stability.

If the dimensions chosen for the thickness of the walls in the preliminary design phase do not satisfy the structural safety requirements, one of the following measures can be adopted:

• changing the type / composition of masonry (for example, from ZNA to ZC or ZIA);

• increasing the thickness of the walls;

• the use of materials (masonry elements and/or mortar) with higher resistances.

The adoption of horizontal structural elements (floors).

When designing the slabs, it will be aimed at making them as rigid diaphragms in the horizontal plane, taking into account their role in terms of:

• collection of inertial forces and their transmission to the vertical elements of the structure;

• ensuring the cooperation of the vertical elements to take over the horizontal seismic forces:

- the distribution of the level seismic force between the structural walls in proportion to the translational stiffness of each one;
- the retransmission to the walls that have reserves of load-bearing capacity of the additional loads that result after the failure of the walls with insufficient resistance capacity;

• the possibility of adopting some simplified structural calculation models, having, as the case may be, only one or three degrees of freedom at each level.

The rigidity of the floors in the horizontal plane depends on:

• the constructive composition of the floor;

• the dimensions and positions of the large gaps in the floors.

The rigidity of the slabs in the horizontal plane will be superior to the lateral rigidity of the structural walls so that the deformability of the slabs does not significantly influence the distribution of the seismic force between
vertical structural elements.

In the case of slabs made of prefabricated elements, the joints will be designed so that the floor's behavior under horizontal forces is as close as possible to that of monolithic reinforced concrete slabs and the joints remain in the elastic stage of behavior for the stresses resulting from the action of the forces corresponding to the design earthquake multiplied by behavior coefficient "q".

Floor type

The floors of masonry buildings are classified, from the point of view of rigidity in the horizontal plane, into two categories:
• rigid floors in the horizontal plane;
• floors with insignificant rigidity in the horizontal plane.
In the conditions in which they are not significantly weakened by gaps, the slabs with the following constructive compositions are considered "rigid in the horizontal plane":
• monolithic reinforced concrete floors with a thickness of min. 130mm or from drawers with continuous concreting with a thickness of ≥ 60 mm, reinforced with concrete steel mesh with an area of ≥ 250mm2/m (for example, ≥ 5Φ8/m);
• slabs made of prefabricated reinforced concrete panels or semi-panels joined on the contour by welded metal parts, concrete steel loops and monolithic concrete;
• floors made of prefabricated elements of the "strip" type, with loops or connecting bars at the ends and with continuous over-concreting with a thickness of ≥ 60 mm, reinforced with concrete steel mesh with an area of ≥ 250 mm2/m (≥ 5Φ8/m).

The following categories of floors are considered to have insignificant rigidity, in the horizontal plane:
• floors made of prefabricated elements of the "strip" type with loops or connecting bars at the ends, without reinforced concreting or with unreinforced screed ≤ 30 mm thick;
• floors made of prefabricated concrete elements with small dimensions, or of ceramic blocks, with reinforced concreting;
• wooden floors.

The use of boards with insignificant rigidity in the horizontal plane (in particular, wooden boards) is allowed only under the following conditions:

- Slabs with insignificant rigidity in the horizontal plane are not accepted for kuag≥0.12g areas, with the exception mentioned below.
- Floors with insignificant rigidity in the horizontal plane can be used, only for:

☐ all floors of constructions with nlevel ≤ 3, from importance classes III and IV, in the seismic zone with ag=0.08g (except for the floor above the basement);

☐ the floor above the last level of the constructions with nniv ≤ 2, from importance class IV, located in the seismic zones with 0.12g≤ag≤0.16g.

The upper side of the slabs will, as a rule, have the same level on the entire surface of the construction.
Exceptionally, offsets of the upper face of the floor smaller than the current height of the belts (15÷20 cm) are accepted.

Conditions of use of the types of masonry

The maximum number of levels above the embedment section (nniv) of masonry buildings, for which the provisions of Code CR6-2006 apply, is limited depending on:

- design seismic acceleration at the site (ag);
- structural regularity/irregularity class;
- the importance and earthquake exposure class of the building;
- type/composition of masonry (ZNA, ZC, ZC+AR, ZIA);
- group of masonry elements (1, 2, 2S).

The provision in the design of the minimum constructive density of the structural walls (p%), according to the tables, does not ensure, in all cases, the satisfaction of the safety requirement and, for this reason, does not eliminate the designer's obligation to verify, by calculation, its fulfillment according to the provisions of the CR6-2006 Code.

Conditions of use for unreinforced masonry

Due to the low capacity to dissipate seismic energy, due to low tensile strength and reduced ductility, it is recommended that the use of unreinforced masonry structures be avoided.
Non-reinforced masonry structures (ZNA) with ceramic elements from groups 1, 2 and 2S can be used, under the conditions established in Code P 100-1/2006, regarding the design seismic acceleration (ag), the number of levels (nniv) and the minimum constructive density of the structural walls (p%) on both directions, only if all the conditions below are met:

• the building falls into the "regular buildings with regularity in plan and elevation" category, the position

1 from table 5.1;

• the building falls into importance classes III or IV;

• the system of placing the walls is of the "even walls" type (honeycomb system);

• hetaj level height ≤ 3.00 m;

• the requirements for the construction of masonry and slabs are respected;

• the qualities of the materials used are those stipulated in Code P100-1/2006.

The maximum number of levels above the embedment section (nniv) for buildings with structural walls of unreinforced masonry (ZNA) with fired clay elements of groups 1 and 2, and the minimum value the associated constructive factor of the density of the structural walls - interior+exterior - (p%), on each of the main directions, depending on the design seismic acceleration (ag), are given in table 2.

Table 2

nniv	Design seismic acceleration ag			
	0.08g	0.12g, 0.16g	0.20g	0.24g,0.28g,0.32g
1	≥4%	≥4%	≥5%	≥6%(*)
2	≥4%	≥6%(**)	NA	NA
3	≥5%	NA	NA	NA

(*) Only with mortar M10 and C10 (**) Only with mortar M10 and C10 for ag = 0.16g.

Note. In the case of ZNA buildings, the attic is considered a "level" which is included in the total number allowed according to table 8.1. even if

NA - nu accepted

Non-reinforced masonry structures (ZNA), with fired clay elements from group 2S, (e.g. POROTHERM type blocks) and BCA elements (GBN50) can only be used for residential buildings with a single level above the embedment section (nniv = 1), in the area with the design seismic acceleration ag= 0.08g, respecting the conditions from (2).

Non-reinforced masonry structures (ZNA) with fired clay elements from groups 1, 2 and 2S and with BCA elements (GBN50 and GBN35) can be used, regardless of the seismic zone, for:
- constructions with a single level above the embedment section, with the function of household annexes that house goods of low value and in which people's access is accidental;
- temporary constructions, with the expected duration of use less than three years (constructions for site organization, for example).

The short-term secant modulus of elasticity of unreinforced, simple masonry, ZNA (Ez), will be taken from the table, depending on the characteristic compressive strength of the masonry fk.

Values of the short-term secant modulus of elasticity of masonry (Ez)

Tab. 3

Calculation type (design situation)	Masonry with clay elements BURNT or concrete	Masonry with items from BCA
Establishing dynamic characteristics	1000 fk	850 fk
Deformations in ULS	500 fk	400 fk
Deformations in SLS (only for systems statically not determinated)	800 fk	650 fk

Transverse modulus of elasticity

The transverse modulus of elasticity, Gz, for unreinforced masonry, with masonry elements from all groups (1, 2, 2S), is determined with the relation:

$$G_Z = 0.4 \cdot E_z$$

where

- Ez - the short-term secant modulus of elasticity, with values corresponding to the respective design situation.

Durability of masonry

Masonry buildings will be designed so that they have the durability necessary to be used in accordance with the requirements and with the duration of operation established by the design theme, in the specific conditions of the surrounding environment.

The masonry mortar shall be durable enough to withstand, under the relevant exposure microclimate conditions, throughout the building's designed service life and shall not contain components that may have a detrimental effect on the properties or durability of the mortar or other materials with which it is placed in contact.
Connections between the core and footings of complex shaped walls as well as sections weakened by vertical slots shall be checked for vertical sliding forces.
The check is not necessary if the connection between the footing and the core of the wall satisfies the conditions below:

- **For unreinforced masonry (ZNA):**

• the walls on the two directions are executed simultaneously (completely woven);
• the connecting section between the walls is not weakened by vertical slits;
• at corners, intersections and branches, the minimum reinforcements established in **Code P 100-1/2006** are provided in the horizontal joints.

Specific provisions for constructions with structural walls of non-reinforced ZNA masonry:

In the case of buildings with floors made up of linear elements (which unload in one direction), regardless of the type of masonry (ZNA, ZC or ZIA), constructive measures will be provided for anchoring, at each floor, the external structural walls arranged parallel to the elements main floor.

At the level of each floor, regardless of the material from which it is made (reinforced concrete or wood), reinforced concrete belts will be provided in the plane of the walls. In the case of buildings with an attic or non-circulating bridge and with a wooden frame, belts will be provided at the top of all walls that exceed the level of the last floor.

The minimum height of the belts will be equal to the thickness of the floor plate for the interior walls and to its double for the walls on the building contour and from the stairwell. The width of the belts for the facade walls will be equal to the thickness of the wall but at least 25 cm, if the belt is withdrawn from the outer face of the wall to achieve its thermal insulation.

The longitudinal reinforcement of the belts will be done with at least four bars $\Phi \geq 10$ mm, ensuring a percentage of reinforcement $\geq 0.5\%$, with closed stirrups $\Phi \geq 6$ mm, arranged at a maximum distance of 15 cm in the current field and at a maximum distance of 10 cm on the the bending area of the longitudinal reinforcements.

The belts will form closed contours; the covering with concrete, the bracing and anchoring of the bars in the belts will be done using **STAS 10107/0-90** as a reference document. Reinforced concrete lintels will be provided over the door and window gaps, as a rule, tied with the belt at floor level.

The resistance capacity of the structural walls for forces in the plane:

The shear force associated with eccentric compression failure of an unreinforced masonry wall required by the design axial force Nd is calculated with the relation:

$$V_{f1} = \frac{N_d}{c_p \cdot \chi_p} \cdot \vartheta_d \cdot (1 - 1.15 \cdot \vartheta_d)$$

In wich:

$$\chi_p = \frac{H_p}{l_w}\ Factorul\ de\ forma\ al\ peretelui\ de\ zidarie$$

H_p= the height of the wall;
l_w= the length of the wall;
c_p= coefficient that depends on the fixing conditions at the ends of the wall;
- c_p = 2.0 for wall bracket (post);
- c_p = 1.0 for double wall recessed at the ends (slat);

$$\sigma_0 = \frac{N_d}{t \cdot l_w}\ the\ average\ unit\ compressive\ effortcorresponding\ to\ the\ axial\ force\ of\ design\ N\_d$$

where t= wall thickness; $\vartheta_d = \frac{\sigma_0}{f_d}$ where f_d= design compressive strength.

Among the data regarding the geometry of the masonry structures, the vertical continuity of the structural walls is of particular importance because:

- Defines structural regularity/irregularity;
- Constitutes a landmark regarding interventions over time on the structure (removal/addition of some walls);
- Provides information on the flow of loads towards the foundations and the areas/elements where effort concentrations occur;

For this reason, if the vertical discontinuity of the walls is found through the visual examination, it is recommended to draw up some plans to materialize these situations.

The behavior of the slabs is optimal when they are made as rigid and resistant diaphragms for the forces applied in their plane. These conditions are fulfilled, at the maximum level, by monolithic reinforced concrete slabs.

The following figure shows two cases in which the slabs do not comply with the condition of non-deformability in the horizontal plane

- mixed floors consisting of both reinforced concrete slabs and wooden beams and floors

cu goluri mari ($A_{Gol} > 0.5$ XY).

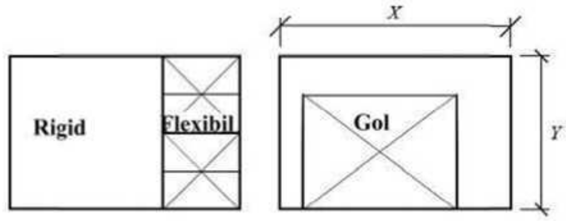


Fig. 8 Floors with insignificant stiffness in plan (examples of composition)

Calculation evaluation of the safety of masonry buildings

Safety against the effects of seismic action in the plane of the wall

Determination of the basic shear force for the building assembly for level 1 and 2 methodologies.

Although from the point of view of the general criteria, the modal calculation based on the response spectrum must be performed if the conditions are met for which the static calculation method with the statically equivalent lateral force is no longer acceptable, the expert must analyze the cases in which the adoption of this method is absolutely necessary. Among the analysis criteria should be considered:

- the level of seismic hazard at the location;
- importance and exposure class of the building
- the seismic performance level imposed by the design theme

The three calculation models mentioned in this paragraph are differentiated by the contribution of the coupling rules to the assumption of lateral forces.

Calculation of the resistance capacity for the seismic action in the plane of the walls

The design resistances of the masonry

Under the conditions of limited tests, in some countries that have important databases regarding the resistance of masonry in existing buildings, the specific technical regulations provide flat-rate, covering values of the main mechanical characteristics of the masonry. We continue to quote provisions from the most important regulations in the field

Provisions of Italian regulations

The Italian regulation includes a table of minimum and maximum values of the mechanical characteristics of masonry of various masonry compositions specific to existing buildings in Italy.

For example, in the case of masonry with solid bricks and weak lime mortar and irregular weave, the values from table 4 are given.

Mechanical characteristics of old masonry in Italy

Table 4

Mechanical characteristic	Notation	Value (N/mm2)	
		minimum	maximum
Average compressive strength	fm	1.80	2.80
Average shear strength	Ai	0.060	0.092
Longitudinal modulus of elasticity	E	1800	2400
Transverse modulus of elasticity	G	300	400

In order to take into account the quality of the masonry in the work, the calculation values can be determined by multiplying the values from the table with superunit factors

- < 1.5 - if the existence of a good quality mortar is found;
- < 1.3 - if the weaving is close to the correct one.

The design values of the average resistances are chosen according to the levels of knowledge as follows:

- KL1^ minimum values from tables
- KL2^ medium values from tables
- KL3 ^ values determined according to the number of samples tried:

\- for three samples, the values of resistances and modulus of elasticity are equal to the average of the test results; alternatively, the values of the elasticity modules are taken equal to the averages in the table;

\- for a single tested sample, if the resulting value is included in the interval in the table or is higher, the value of the average resistance is taken equal to the average of the values in the table; if the resulting value is less than the lower limit in the table, the mean value shall be taken equal to the experimental value.

The above provisions thus encourage the performance of a greater number of trials for the most accurate knowledge of the input data for the calculation.

The regulation allows local authorities to set values different from those in the tables based on the specific constructive data in the area.

US regulatory provisions

In the [ASCE/ACI/TMS] standard, flat values are given, considered lower limits for the main mechanical characteristics of masonry. The values are differentiated according to the state in which the masonry is found, established by visual examination:

- good, if the degradations do not exceed the insignificant or slight level;
- acceptable, if the degradations do not exceed the moderate level;
- weak for which degradations are heavy or extreme

The definition of degradation level is detailed in [FEMA 06,307,308].

The probable average values of the respective characteristics (N/mm2) are deduced from the fixed values by multiplying by the factor 1.3. The use of flat values is only allowed for linearly elastic calculations.

Mechanical characteristics of old masonry in the USA

Table5

Mechanical characteristic	Notation	Masonry condition		
		good	acceptable	weak
Compression strength	fme	8.2	5.5	2.7
Tensile strength from bending	fte	0.18	0.09	0.00
Shear resistance	Vme	0.25	0.18	0.12
Longitudinal modulus of elasticity	E	4500	3000	1500
Transverse modulus of elasticity	G_{me}	1800	1200	600

In the absence of tests, for the basic performance objectives, the values in the table are multiplied by the knowledge factor k = 0.75; idem in the case of higher performance objectives if there are only usual tests.

The high values of the average lump sum strengths of the existing masonry can be explained, first of all, by the high strength of the bricks used in the USA even from the first two decades of the 20th century.

Thus, a 1929 report, based on manufacturers' data, showed that 92% of US brick production had an average compressive strength of 50 N/mm2 (about 7,200 psi) [FEMA 274, sect.7.2.2].

Safety against seismic action perpendicular to the plane of the walls:

Damage to masonry walls under the effect of seismic action perpendicular to the plane of the wall is a phenomenon observed especially in the following situations:

- in buildings with floors supported by linear elements (metal or wooden beams);
- for filler panels for reinforced concrete or metal frames and, in particular, for double-layer walls with an inner hollow;
- to all categories of cantilevered masonry panels (major pediment/shield/tympanum elements and minor parapet-type elements);

The yielding occurs most often by going out of plane or overturning an entire wall or wall fragments.

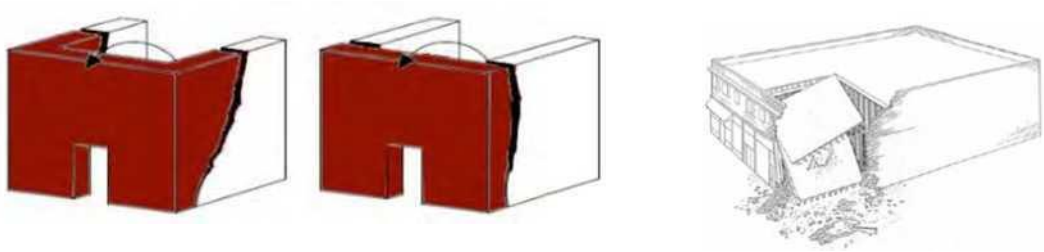


Fig. 9 The floor damage mechanism through seismic action perpendicular to the plane of the wall.

The behavior of unreinforced masonry to this request is particularly complex and represents an insufficiently known field of seismic design as also emphasized in the reference work [Paulay T., M.J.N. Priestley, Seismic design of reinforced concrete and masonry buildings John Wiley & Sons, 1992].

For example, the influences of the effective fixing conditions on the vertical sides and at the level of the slabs, the amplification effect given by the movement of the slab during the earthquake (especially in the case of slabs with insignificant stiffness in the plane), the mechanical characteristics of the masonry on the two directions, etc.

In the last decades, researches have been carried out in many countries to elucidate these aspects. Some of the first research in this field had as a practical aim the rational establishment of the limit ratio between height/thickness in the case of loading with inertial forces perpendicular to the plane given by the seismic action [ABK - A joint venture, Methodology for Mitigation of Seismic Hazards in Existing

Unreinforced Masonry Buildings: Wall Testing, Out-Of-Plane, Topical Report 04, El Segundo, California, 1981].

The introduction into design practice of more accurate calculation methods, for example, those based on rigid body rotation of wall fragments, was facilitated by more complex tests, on large scale models (1:2), carried out in the last years. [Doherty K., B. Rodolico, N.T.K. Lam, J.L. Wilson, M.C. Griffith, Displacement-based seismic analysis for out-of-plane bending of unreinforced masonry walls Earthquake Engineering and Structural Dynamics, 2002, Vol. 31, pp. 833- 850][Griffith M.C., N.T.K. Lam, J.L. Wilson, K. Doherty, Experimental investigation of unreinforced brick masonry walls in flexure, Journal of Structural Engineering, 2004, Vol. 130, No. 3, pp. 423-432.(2004)].

The most frequent situation of loading with forces perpendicular to the plane of the wall is given by the "direct" action of the earthquake that is exerted on all the walls in a building. This stress, resulting from the acceleration imparted to the wall during the earthquake, affects the external and internal walls in a building with consequences regarding both the ultimate limit state (damages leading to overturning) and the service limit state (cracking occurs more or less extensive).

The loss of stability (overturning) of structural masonry walls is a phenomenon that occurs, in particular, in the case of walls that are not properly anchored to the slabs or in the case of major masonry elements that work in the cantilever (heels, high pediments). These damages are typical for old buildings, with floors that have insignificant rigidity in the horizontal plane.

However, we must mention the fact that in older buildings where the load-bearing walls had significant thicknesses (2-3 bricks), the resistance module, proportional to the square of the wall thickness, ensured the wall a satisfactory resistance to relatively high seismic forces.

The breakdown schemes indicated in the following figure show the possible variants of the overturning mechanism in a building (partial or complete overturning of the pediment - figure 10 a). The highest probability of overturning exists in the case of walls that, in the case of slabs that discharge on two sides, are parallel to the main beams (figure 10 b)

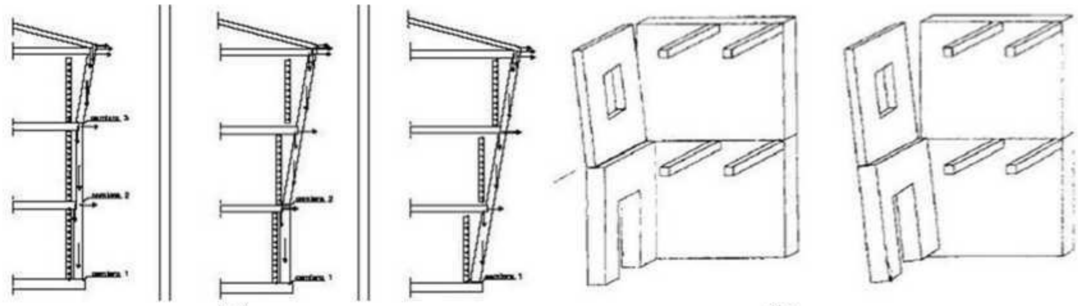


Fig.10 The overall damage mechanism through seismic action perpendicular to the plane of the wall

In the case of non-structural walls in new buildings, modern design regulations take into account the fact that seismic stress perpendicular to the plane can result in damage with different levels of severity, from superficial cracking of the walls to their complete destruction. These damages can have minor consequences, requiring only the restoration of the finishes, or more serious, going as far as injuring the people in the room. In rooms with special functions (operating rooms, for example) damage to non-structural walls can result in the interruption of activity.

Several studies mentioned in the literature highlight the existence of a phenomenon of "resonance" between the oscillations corresponding to the two categories of excitations (in the plane and perpendicular to the plane of the wall). This phenomenon can be quantified with sufficient precision only through nonlinear dynamic calculation procedures, which is impossible to achieve for current design practice. [Paulay, T., Priestley, M.J.N. Seismic design of Reinforced Concrete and Masonry Buildings, John Wiley& Sons, Inc, 1992]. For this reason, all technical regulations dissociate the two checks, establishing specific requirements and procedures for each individual situation.

Calculation of bending moments in walls subjected to loads perpendicular to the plane.

Under the effect of loads perpendicular to their plane, masonry walls are deformed, taking the form of a cylindrical or double-curved surface depending on:

- panel dimensions;
- the support conditions on the four sides;
- the position and shape of the gaps (if they exist).

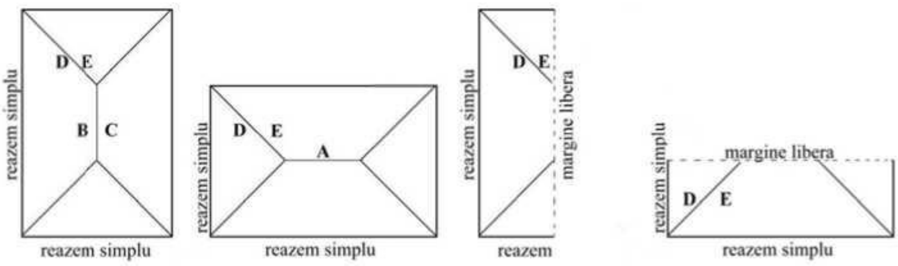


Fig. 11 Route of failure lines for walls loaded perpendicular to the plane (examples)

In the general case, the break occurs on routes that are dictated by these parameters and that go through the areas with the lowest resistance;

In detail, the pattern of the fracture lines identified above is shown in figure 12.

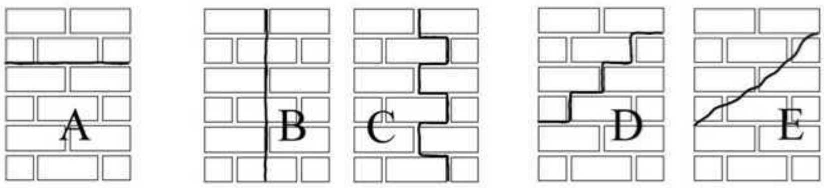


Fig.12. Details of the possible routes of the breaking lines

Figure 12 A corresponds to the fracture on a plane parallel to the horizontal joints and figures 12 B and 12 C correspond to the fracture on a plane perpendicular to the horizontal joints (these are the fundamental fracture mechanisms mentioned in SR EN 1996-1-1);

The details of breaking on inclined paths are shown in Figures 12 D and 12 E.

The breaks on the routes indicated in figures 12 C and 12 D are specific to masonry with weak mortars in relation to the elements, and the breaks in figures 12 B and 12 E are specific to masonry where the elements and mortar have similar strengths.

Walls that are supported only at the bottom and at the top and are free on the two vertical sides deform in the vertical plane along a cylindrical surface. Bending in the vertical plane under the action of forces perpendicular to the plane produces bending moments that give rise to unit tension/compression efforts perpendicular to the laying joints. In this case, the behavior of the walls is linearly elastic until the unit tensile strength of the mortar or its adhesion strength to the masonry elements is exceeded, when, as a rule, the deformations increase suddenly (brittle fracture);

The design value of these moments, for the unit length of the wall, is the largest of the values of the bending moments calculated using the theory of straight beams taking into account the respective bearing conditions:

• simple supports at both ends maximum moment at half height of floor: $M_w = (f_w \cdot h^2)/8$
• supports with total continuity at both ends → maximum moment in the sections at floor level: $M_w = (f_w \cdot h^2)/12$
• we rest with total continuity at one of the extremities and simply rest at the other extremity → maximum moment in the section with continuity: $M_w = (f_w \cdot h^2)/8$

A cylindrical deformation is also achieved if the wall is supported only laterally. This situation can be encountered in the case of a wall that is not fixed at the top and at the bottom it rests on a capillary break layer (the situation is very rarely encountered in the design of current buildings).

Examining the mode of failure in this case, however, has a methodological value because bending with a vertical plane of failure is a component of the real behavior of walls resting on all four sides.

In the case of walls resting on three or four sides, the deformed shape is a surface with double curvature (bending occurs both in the vertical and horizontal planes).
In this situation, the calculation of the bending moments presents difficulties that come from two main sources:

• the anisotropy of the masonry makes the strength and rigidity of the wall to be different in the two orthogonal directions (parallel to the laying joints and perpendicular to them);
• modeling of boundary conditions.

Due to the anisotropy of the masonry, the more accurate representation of the bending behavior perpendicular to the plane, implies the use of a more complex model.
In the case of non-structural walls, the boundary conditions can be:
• Recessed: non-structural wall connected by weaving with a structural wall of at least double thickness.
• With continuity: non-structural wall intersected by another perpendicular wall.
• With simple support: in this situation there is the lower edge of the wall (resting on the lower floor) and the upper edge (fixed to the upper floor); also the vertical edges of the infill masonry panels next to the columns / concrete walls.
• Free sides: the upper edge of the walls partially developed in height and

the side edges near the gaps (even if the gap does not develop over the entire height of the panel).

The calculation models for identifying the effects of seismic action perpendicular to the plane of the wall are established, for each damage mechanism of the wall panel, primarily depending on the constructive characteristics of the wall and in particular depending on the connection of the panels on the vertical edges:

1. The existence or absence of perpendicular walls, at both ends or at one end.
2. Wall geometry: the ratio between length (distance between perpendicular walls) and height (distance between floors).

The walls that are not fixed at the top, for example those of the sanitary groups or the corridors with overhead light, present the highest risk of overturning.

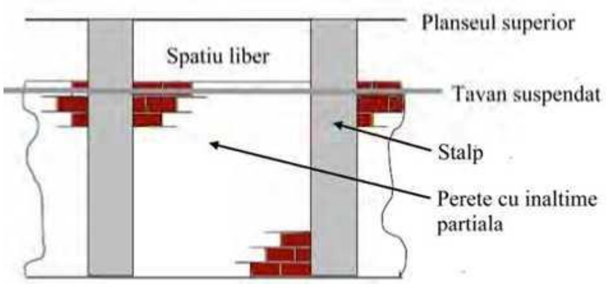


Fig.13. Free wall (not supported) at the top.

The verification by calculation of the stability and resistance of the walls to the seismic action perpendicular to the plane is established by various technical regulations depending on:
• the position of the panel on the height of the building;
• how to fix the panel to the main structure of the building
• the level of design seismic acceleration at the site

At the same time, depending on the fundamental concept of "seismic performance" established by each regulation, the seismic response of the walls to the action perpendicular to the plane is differentiated according to:

• the category of importance and exposure of the building or the area of the building where the wall is located;
• behavior required for "moderate" earthquake (SLS) or for "severe" earthquake (SLU):
- non-cracked
- cracked but repairable (requires/does not require replacement)
- cracked/cracked but stable in the project position

Overturning mechanisms under seismic action perpendicular to the plane of the wall depend primarily on:
• wall composition/areas weakened by voids (materials, slenderness, size of voids);

• the presence of some non-seismic stresses (thrusts given by vaults, beams, other similar elements);
• how to fix it to the floors or other structural walls;

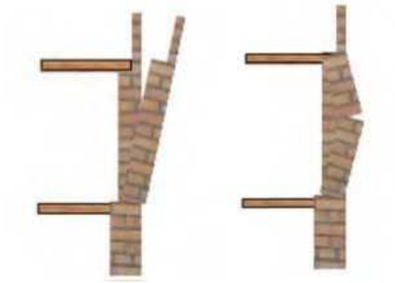


Fig.14. Breakage of the walls according to the clamping method (a) Clamping only at the base (b) Clamping at both ends

a) b)

For the ultimate limit state (ULS), the capable moment of the transverse section of the wall at failure in eccentric compression perpendicular to the plane can be calculated by accepting the rectangular compressive stress diagram, with the design value equal to 0.85 fd (neglecting the tensile strength of the masonry) . This equilibrium situation implies the acceptance of an advanced state of wall cracking. In the case of masonry with thin-walled fired clay elements (group 2S), this hypothesis is no longer fully applicable (in the areas marked in red in fig.15, it is possible that the width of the compressed area (0.1t) is greater than the thickness of the outer wall of the element for masonry!).

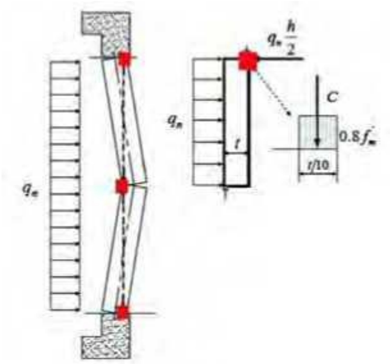


Figura 15. Ultimate limit state for a wall loaded perpendicular to the plane

For the evaluation of the capacity of a wall to the seismic action perpendicular to the plane in the ultimate limit state (ULS), the formation of fracture lines on paths compatible with the geometry and contour fixing conditions of the wall can be considered.

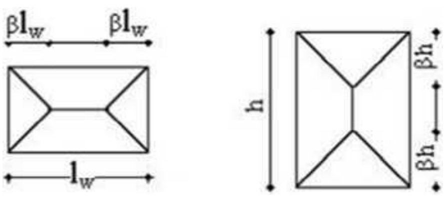


Figura 16 Break lines for normally loaded walls on plan

For the design of walls with irregular shapes or those with voids, a calculation based on a recognized method of determining bending moments in flat plates can be used, for example, the finite element method, or the analogy of fracture lines, taking into account, where appropriate, masonry anisotropy. For the application of the break line method taking into account the anisotropy of the masonry, no indications are given in the SR EN 1996-1-1 standard.

Establishing the risk class of constructions

The results of the checks represent the essential elements that underpin the final assessment regarding the state of safety against seismic actions. This is globally defined by vulnerability

of the construction, the evaluation report placing the examined construction in a vulnerability class associated with the design earthquake (risk class).

Seismic safety assessment and classification in seismic risk classes is based on 3 categories of conditions that are the subject of investigations and analyzes carried out during the assessment. To guide the final decision regarding the safety of the structure (including the construction's risk class) and the necessary intervention measures, the extent to which the 3 categories of conditions are met is quantified by means of 3 indicators. These are:

- the degree of fulfillment of the seismic composition conditions, denoted by R1, expresses the degree of fulfillment of the structural compliance conditions, of the composition of the structural elements and the constructive rules for structures that take the effect of the seismic action.
- the degree of structural damage, denoted by R2, which expresses the proportion of structural degradation produced by seismic action and other causes.
- the degree of seismic insurance, denoted by R3, represents the ratio between the capacity and the seismic structural requirement, expressed in terms of resistance in the case of using the level 1 and 2 methodologies or in terms of displacement in the case of using the level 3 methodology. This indicator is determined for ULS.

The R1 indicator takes values based on the score assigned to each category of composition conditions given in the list specific to the type of structure analyzed in the annex corresponding to the type of structural material used. There are 4 domains of the score achieved by the analyzed construction, associated with the 4 classes of seismic risk, within the limit of a maximum score R1 max = 100, corresponding to a construction that fully meets all categories of construction conditions. The 4 distinct ranges of R1 values are given in Table 6.

Table 6

R1 values associated with seismic risk classes

Seismic risk class			
I	II	III	IV
Values R1			
<30	30-60	61-95	96-100

The R2 indicator takes values based on the score assigned to the different categories of structural and non-structural degradation given in the list specific to the type of construction analyzed, from the annex corresponding to the structural material used.

And in the case of this indicator, 4 ranges of the score achieved by the analyzed construction are established, associated with the 4 classes of seismic risk, within the limit of a maximum score of R2 max =100, corresponding to a construction with integrity unaffected by degradation.

The 4 distinct ranges of R2 values are given in Table 7.

Table 7

R2 values associated with seismic risk classes

Seismic risk class			
I	II	III	IV
Values R2			
<40	40-70	71-95	96-100

The R3 indicator highlights the resistance and deformability capacity of the structure in relation to the seismic requirements. For guidance purposes, the classification of the construction into risk classes based on the R3 values is done (multiplying the obtained values by 100), according to table 8.

Table 8

R3 values associated with seismic risk classes

Seismic risk class			
I	II	III	IV
Values R3			
<35	36-65	66-90	91-100

The values of the three indicators, measures of the expected seismic performance of the construction, should only be considered indicative scores in the decision to place the construction in a certain seismic risk class. The fact that the value of a certain indicator (assuming that it is appreciated as the critical criterion out of all three, for the considered construction) falls within a certain range of values, associated with a certain risk class, does not automatically lead to the building being placed in that class.

The decision regarding the classification of the building in a certain risk class must be the result of a complex analysis of the set of conditions of different natures. The investigations carried out aim to identify the weak links of the structural system and the significant deficiencies of the non-structural elements. Once identified, these deficiencies must be prioritized from the point of view of the potential effects on the stability of the structure in the event of a strong earthquake attack and the risk of loss of human life and injury, or material damage.

In these evaluations, the expert must evaluate, first of all, the vital elements for structural safety against earthquakes that present major deficiencies and insufficient capacity in relation to the requirements of different natures, specify their weight in the structure as a whole and estimate the safety margin. Knowing the probable failure mechanism of a structure is essential for the correct assessment of both the potential seismic response of the construction and for the appropriate choice of the intervention solution.

The identification, even approximate, of the breaking mechanism is possible in a small number of cases for old constructions, which are also the most vulnerable. The reasons can be different: the absence of a well-defined structure for taking over the lateral forces, the lack of data to allow the evaluation of the behavior of the structure in the post-elastic field (for example, in reinforced concrete buildings, the data related to the anchoring and bracing lengths of the reinforcements, to the transverse reinforcement in critical areas), the uncontrollable risk of fragile breaks due to the action of the cutting force, etc. For this reason, the correct evaluation of the probable performance of the construction must be based on a comprehensive analysis and an engineering judgment of all the construction conditions, of the correlation between their effects, operations that demand high competence and special experience.

CAUSES THAT PRODUCE THE DETERIORATION OF MASONRY STRUCTURES:

Analyzing as a whole the resistance capacity of the masonry structures through the prism of the causes that can produce damage under loads or the appearance of defects, the following essential factors

are found:
• the way and extent in which the resistance capacity was initially ensured, during the design of the construction, through its conception, dimensioning and composition;
• the way in which the resistance capacity of the structure was actually realized during the execution (compliance with the project, materials, control of hidden works, etc.);
• the way in which the resistance capacity was preserved over time, taking into account the external influences and actions that took place along the way (seismic actions, subsidence or rotation of the foundations, structural changes, changes of destination, operating and maintenance conditions, etc.)

Among the factors listed above, the most important and which is dealt with further is the one related to external actions, due to one of the factors above.

We cannot neglect other causes that add to the effect produced by external actions, in many situations amplifying their effects, for example:
- in old buildings, previous seismic movements and in which the corresponding consolidations have not been executed;
- the degradation of materials over time under repeated or aggressive mechanical stress that led to a decrease in resistance;
- post-execution changes, through the disorderly practice of creating new gaps in the structural walls, changing the position of partition walls, changing the destination of some spaces, etc. ;
- filling some gaps in the walls without tying with tesserae, filling with fillings;
- water infiltrations, from various sources, at the foundation land, with the consequence of the displacement of the foundations.

Types of damage can be:
- rigid movements;
- deformations (structural configuration changes);
- cracks or cracks (relative displacements of some construction parts).

The causes that cause breakdowns can start:

- at the level of foundations;
- at the superstructure level.

Damages generated by foundations:

Damages caused by foundations are generated by their movements, in plan:
- horizontally;
- vertically;
- rotations;