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Study on seismic design characteristics of existing buildings in Bucharest, Romania

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STUDY ON SEISMIC DESIGN CHARACTERISTICS OF EXISTING BUILDINGS IN BUCHAREST, ROMANIA

1. INTRODUCTION

In order to start any retrofitting programme for a peculiar country it is important to understand the peculiarities of the building and the codes of practice that were used during the design process. It is obviously that the codes were changed along the history and that the building differs according to the design period. This study contains the evolution of the base shear coefficient, the most important factor in seismic design, the evolution of the reinforced concrete codes of practice and the evolution of the masonry structures codes of practice.

Because the information is nothing without examples, we attached to this study the planes of four building built under different codes of practice and built of different material such us reinforced concrete or masonry.

2. EVOLUTION OF THE BASE SHEAR COEFFICIENT

Along the last half-century in Romania there were in force several codes for earthquake resistant design of buildings that contained different formula for the computation of the base shear force and for the computation of the base shear coefficient. This chapter contains a short description of the formulas involved in the seismic design according to each code of practice.

2.1. CODE P13 – 63

2.1.1. Seismic horizontal load

The total seismic horizontal loading acting on a building (the base shear) can be determined by the following eq.:

$$S = cQ$$

where:

„c” is the seismic coefficient:

$$c = K_s \beta \varepsilon \psi$$

that should be greater than 0.02.

2.1.2. Coefficients used to determine the seismic coefficient

1. K_s is a coefficient which accounts for the design seismic factor. Can be determined from the following table:

Table 1: The values of Ks according to building design seismic factor

Building design seismic factor	K_s
7	0.025
8	0.050
9	0.100

2.1.3. Building design seismicity factors

The design seismicity factor should be established based on:

- site seismicity factor
- importance of the building

The site seismicity factor can be determined from the Romania seismic map given by STAS 36840-63 “Seismic intensity factors”.

The design seismicity factor can be determined using the following table, based on the site seismic intensity factor and the building importance class.

Table 2: Classification of buildings according to importance class and the design seismicity factor

Importance class	***Definition***	***The design seismicity factor determined for a site seismicity factor of:***		
		7*	***8***	***9***
I	*Monumental buildings, high importance buildings*	*8*	*9*	*9*
II	*All buildings except classes I, III,IV, V*	*7*	*8*	*9*
III	*Single level industrial buildings, energetic facilities of local interest, etc.*	*7*	*7*	*8*
IV	*Single level buildings used for housing, administrative or commercial purposes.*	*7*	*7*	*8*
V	*Facilities which don't present any hazard to human life in case of collapse,*	*7*	*7*	*8*

2. β is the dynamic coefficient. It can be determined based on the vibration period of the associated single degree of freedom system and the soil condition, as follows:

- for soils with the maximum allowable stress $\sigma \geq 2kg/cm^2$, β can be determined by the following eq.: $0.6 \leq \beta = \frac{0.9}{T} \leq 3$ (figure 1)
- for soils with de maximum allowable stress $\sigma < 2kg/cm^2$, β can be determined using the previous equation multiplied with 1.25. However it should be limited to the value of 3 ($\beta \leq 3$).
- for very soft and humid soils β can be determined by the same eq. multiplying with 1.5. However it should be limited to the value of 3 ($\beta \leq 3$).

* Bucharest had a site seimicity factor of 7 (MSK-64 intensity scale was used.)

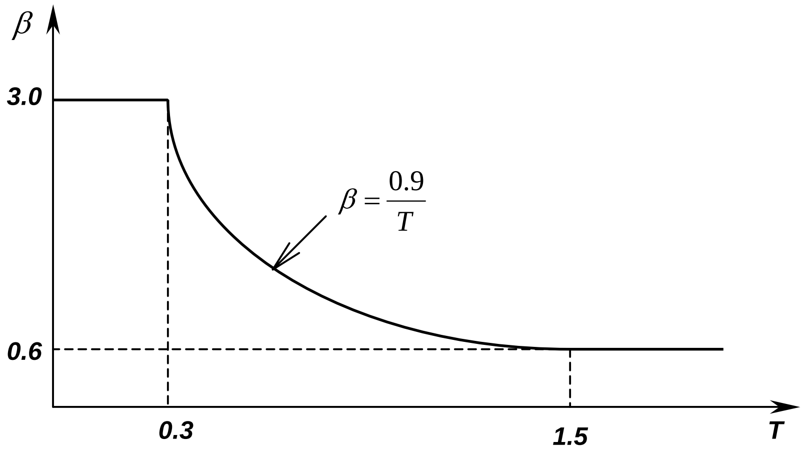


Figure 1: Amplification spectra

Observations:

If available, mIcrozonation maps should be used to determine the soil category for a specific site.

3. ε is a coefficient that makes the equivalence between the single degree of freedom system and the real multi-degree one. It can be determined as follows:

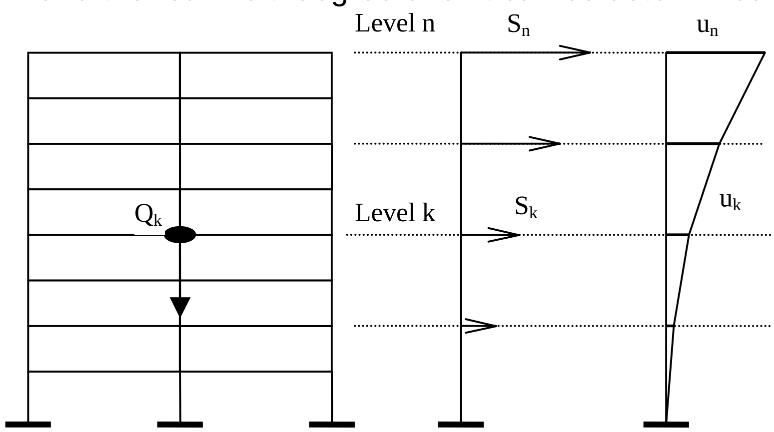


Figure 2: Distribution of seismic force

$$\varepsilon = \frac{\left[\sum_{1}^{n} Q_k u_k\right]^2}{\left[\sum_{1}^{n} Q_k\right]\left[\sum_{1}^{n} Q_k u_k^2\right]} = \frac{\left[\sum_{1}^{n} Q_k \chi_k\right]^2}{\left[\sum_{1}^{n} Q_k\right]\left[\sum_{1}^{n} Q_k \chi_k^2\right]}$$ where:

- u_k is the horizontal displacement of the structure at level k in the direction of the seismic forces, for the considered vibration mode.

- $\chi_k = \frac{u_k}{u_n}$ is the ratio between the structural displacement at level k and the structural displacement at top

4. ψ is a coefficient that accounts for the structural and material types . It introduces the influence of friction damping and can be determined as follows:

- For reinforced concrete moment resisting frames and for reinforced concrete frames pinned at top $\psi = 1.2$;

- For very tall, flexible buildings such as independent chimneys, elevated water tanks, antennas: $\psi = 1.5$

- For all other buildings $\psi = 1.0$

The total gravity load $Q = \sum_1^n Q_k$ is computed based on the gravity load at each "k" floor Q_k" Floor loads can be determined by multiplying the standard gravity loads with the following coefficients.

Table 3: Load combination for seismic design

No.	Load type		Loading coefficient
1	**Permanent loads**		1.0
2	**Snow load**		0.8
3	**Temporary loads**	**a)** For tanks, warehouses or elevators	1.0
		b) For residential, administrative or office buildings	0.8
		c) For industrial buildings	0.8

Observations:

For all other cases the loading coefficients should be taken 0.8.

The total seismic load is $S = \sum_1^n S_k$. It represents the sum of the seismic loads at each floor "k", namely S_k. S_k can be determined as follows:

- the total seismic load is distributed to each floor using the following eq.:

$$S_k = S \frac{Q_k u_k}{\sum_1^n Q_k u_k} = S \frac{Q_k \chi_k}{\sum_1^n Q_k \chi_k}$$

- or, S_k can be directly determined by:

$S_k = c_k Q_k$ where:

$c_k = K_s \beta \psi \eta_k$

and,

$$\eta_k = u_k \frac{\sum_1^n Q_k u_k}{\sum_1^n Q_k u_k^2} = \chi_k \frac{\sum_1^n Q_k \chi_k}{\sum_1^n Q_k \chi_k^2},$$

ε is related with η_k by: $\varepsilon = \frac{\sum_1^n Q_k \eta_k}{Q}$

The total seismic load should be greater than:

$S = \sum_1^n S_k \geq 0.02Q$.

Observations:

For current buildings the seismic loads can be computed based only on the building fundamental mode of vibration.

For tall buildings, such as chimneys, elevated water tanks, towers or antennas, multistory buildings with more than 10 stories or all other buildings with ε<0.6 (previously defined), the seismic loads should be determined based on the first three vibration modes. In these cases the sectional efforts should be computed independently for each vibration mode and then they should be superimposed using:

$N = \sqrt{N_a^2 + 0.5(N_b^2 + N_c^2)}$ where

N_a, N_b and N_c are the sectional efforts associated to each considered vibration mode; N_a is the biggest value of all three.

2.2. CODE P13 – 70

2.2.1. The determination of the horizontal seismic loads

The seismic horizontal loads that act on the building structure at the k level, corresponding to the vibration mode r, are determined using the equation:

$$S_k = k_s \beta_r \psi \eta_{kr} \cdot Q_k$$

1) k_s is the coefficient which takes into account the influence of the site seismicity and the importance of the building. The values of this coefficient are presented in the table bellow. The k_s coefficient represents the ratio between the seismic design acceleration and gravitational acceleration.

The seismic design acceleration depends on the seismic intensity defined for the site (STAS 2923) and also on the importance of the building.

The classification of the buildings in importance classes takes into account the social and economic importance of the building.

Table 4: The values of the k_s coefficient according to importance class and seismic intensity

Importance class	Buildings types	k_s			
		6	7[†]	8	9
I	Very important buildings: monumental buildings, museums, buildings that should remain functional in case of earthquake (hospitals, administrative buildings, etc.), buildings which are very important for the national economy.	0.03	0.05	0.08	0.12
II	All the buildings except those from the importance classes I, III, IV.	--	0.03	0.05	0.08
III	Buildings with one story: residential, administrative and commercial buildings which are not so important. Industrial buildings with one story. Agricultural buildings.	--	0.02	0.03	0.05
IV	Buildings with small importance: buildings used like shelters in agriculture and temporary buildings.	--	--	--	0.03

2) β_r is the coefficient which introduces the influences of the natural vibration period of the building T_r and the foundation soil:

- for normal soil conditions: $\beta_r = \dfrac{0.8}{T_r}$

[†] Bucharest had a site seimicity factor of 7 (MSK-64 intensity scale was used.)

$0.6 \le \beta_r \le 2.0$, with T_r measured in seconds;

- for hard soil the values of the coefficient β_r given in the preceding paragraph should be reduced by 20% except for the masonry and precast reinforced concrete structures.
- For clay soils and in case of high underground water level the coefficient β_r should be increased by 50% without exceeding the limit value $\beta_r = 2.5$.

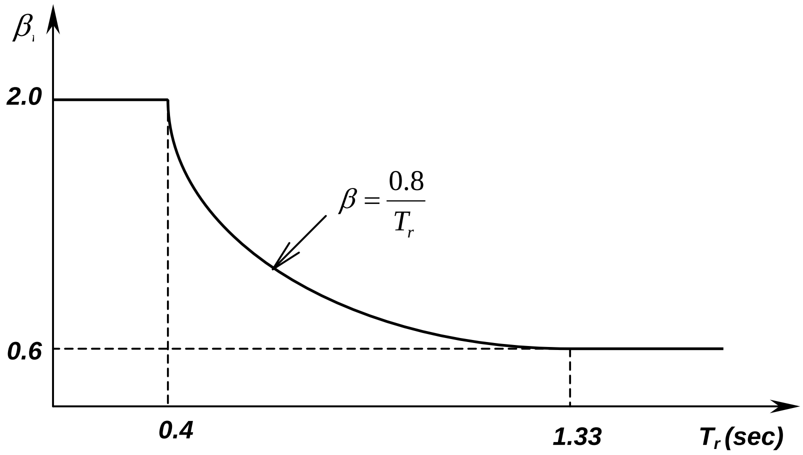


Figure 3: Amplification spectra

Observations:

The effect of the foundation rocking on the calculation of the natural vibration period will be determined:

- For buildings with shear walls the surface inside the exterior foundation should be consider as the foundation surface. For buildings with reinforced concrete should be taken into account the surfaces of the insulated foundations under the columns.
- The settlement coefficient for the foundation rocking expressed in daN/cm^3 is limited to minimum value: $C_\theta \ge 50\frac{\sigma_{at}}{B}$, σ_{at} represents the allowable stress for the foundation soil (daN/cm^3) and B represents the horizontal dimension of the foundation in the vibration direction determined considering the information from the preceding paragraph.

3) ψ is the coefficient which introduces the influences of the damping properties and the ductility of the structure.

Table 5: The values of ψ coefficient according to structure type

Structure type	ψ
RC frame structures	**1.0**
RC walls structures	**1.2**
Masonry structures	**1.3**
Very flexible structures	**1.8**
Elevated water tanks	**2.0**

For all other types of structures that cannot be found in the table the value of the coefficient will be $\psi = 0.2$.

4) η_{kr} is the coefficient which introduces the influence of the vibration mode and is determined by:

$$\eta_{kr} = u_{kr} \frac{\sum_{j=1}^{n} Q_j u_{jr}}{\sum_{j=1}^{n} Q_j u_{jr}^2}$$

u_{kr} is the eigenvalue for the degree of freedom (k) for the modal shape of the vibration mode (r).

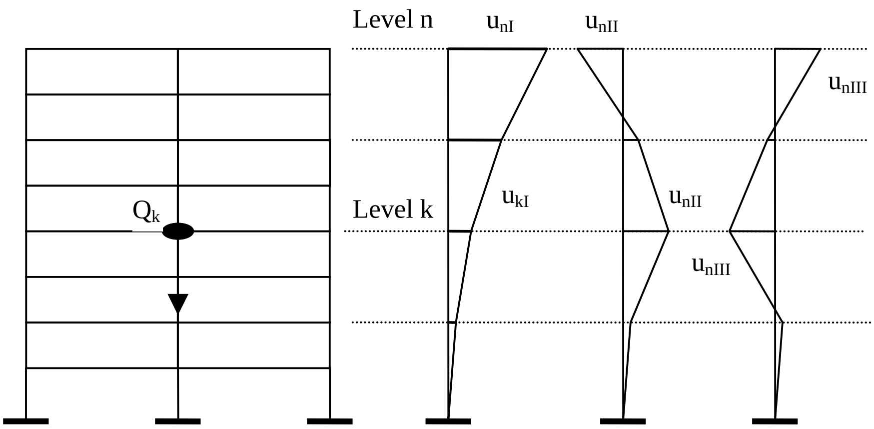


Figure 4: First three vibration modes

5) Q_k is the resultant load equal to the sum of the gravitational loads for level k which are determined using the code permanent and temporary loads. The temporary loads should be multiplied by a coefficient equal with 0.8.

Observations:

The seismic horizontal loads S_{kI}, corresponding to the first (fundamental) vibration mode, should be determined tacking into account the equation:

$$S_I = \sum_{k=1}^{n} S_{kI} \geq 0.02Q$$

S_I is the base shear corresponding to the first vibration mode and Q is the resultant force of the gravitational loads used in the determination of the seismic loads:

$$Q = \sum_{k=1}^{n} Q_k$$

The base shear force corresponding to the vibration mode r can be determined using the equation:

$$S_r = c_r Q$$

$$c_r = k_s \beta_r \psi \varepsilon_r$$

ε_r is the equivalence coefficient

$$\varepsilon = \frac{\left[\sum_{j=1}^{n} Q_j u_{jr}\right]^2}{\left[\sum_{j=1}^{n} Q_j\right]\left[\sum_{j=1}^{n} Q_j u_{jr}^2\right]} = \frac{\sum_{j=1}^{n} Q_j \eta_{jr}}{Q}$$

S_{kr} is the seismic horizontal load for level k

$$S_{kr} = S_r \frac{Q_k u_{kr}}{\sum_{j=1}^{n} Q_j u_j}$$

It is allowed to determine the seismic horizontal loads for current structural types tacking into account only the fundamental vibration mode.

The horizontal seismic loads for high structures with the natural period greater then 1.0 s or total height grater then 40m should be calculated tacking into account the contribution of the first three vibration modes. The stresses should be established separately for each vibration mode (r) and the total stress *N* in one section will be determined using the equation:

$$N = \sqrt{\sum_{r=1}^{3} N_r^2}$$

N_r is the stress in the considered section corresponding to the vibration mode (r).

2.3. CODE P100 – 78

2.3.1. General equations for establishing the horizontal seismic forces

The horizontal seismic forces that act on the structure should be determined for each vibration mode. The forces at the kth floor (on the direction of the degree of freedom), corresponding to the rth vibration mode, are established with the relation:

$$S_{kr} = c_{kr} G_k \quad ; \qquad \text{where:}$$

$$c_{kr} = k_s \beta_r \psi \eta_{kr}$$

1) c_{kr} is the seismic coefficient for the kth floor in the rth vibration mode;

2) k_s is the seismic coefficient corresponding to the attributed rank of seismic protection of the construction. It represents the ratio between the maximum ground acceleration (that corresponds to the construction's rank of seismic protection) and the acceleration of gravity. The values of k_s are given in the table below.

Table 6: The values of K_s according to the rank of seismic protection

The values of the coefficient K_s							
The rank of seismic protection of the construction	6	6 ½	7	7 ½	8[‡]	8 ½	9
K_s	0.07	0.09	0.12	0.16	0.20	0.26	0.32

3) β_r is the dynamic coefficient that corresponds to the rth vibration mode. It depends on the period of vibration T_r (for the considered vibration mode) and the nature of the foundation ground as follows:

- foundation grounds with normal rigidity: $\beta_r = \dfrac{3}{T_r}$**, (**T_r in seconds) and limited between $0.75 \le \beta_r \le 2.0$;
- for rigid foundation grounds: rocky lands, layers of stable deposits of sand, coarse gravel or high consistency clays (Tertiary layers or older) etc., the values of the β_r should be reduced with 20%, but respecting the above given limits;
- for foundation grounds with reduced rigidity: layers of reduced consistency clay (the consistency index $I_c < 0.5$ with or without insertions of sand or other non-cohesive formations, sand in refined state, (ramming rank $D < 0.33$), loess with high humidity ($w > 20\%$) or for sandy or clayish land with high level of underground water (depth < 5 m), the values of β_r should be increased by 30%, but without exceeding the limit value $\beta_r = 2.5$.

[‡] Bucharest had a rank of seismic protection equal to 8 (MSK-64 intensity scale was used.)

4) ψ is the reduction coefficient of the seismic effects and it takes into account the structural ductility, the stresses redistribution capacity, the contribution of strength reserve not considered in the analysis (especially from the interaction with non-structural elements) and the vibration damping effects. The values of ψ coefficient are given in the table below.

Table 7: The values of ψ coefficient

No.	Construction type and constructive system	Coefficient ψ
1	Buildings with a rigid structure (masonry or reinforced concrete cast-in-place or prefabricated shear walls) or a semi–rigid structure (shear walls interacting with frames):	
	- up to 5 stories (including the ground level) and the total height less or equal to 15 m	**0.30**
	- higher than 5 stories and the total height over 15 m	**0.25**
2	Multi-story frame buildings:	
	- with one span	**0.25**
	- with more spans	**0.20**
3	Industrial one-story plants and other related constructions:	
	- with one span	**0.20**
	- with more spans	**0.15**
4	Silos and other special constructions with a rigid structure	**0.25**
5	Very flexible tall constructions like chimneys and independent towers	**0.35**
6	Elevated water tanks	**0.35**

Remarks:

1. The values of the ψ coefficient for the multi-story frame buildings are established considering that the ductility conditions for the structural elements are fulfilled.
2. For steel structures, the ψ values from the table should be reduced with 20%.

5) η_{kr} is the coefficient of distribution of seismic forces corresponding to kth floor in the rth vibration mode and it can be obtained with the following relation:

$$\eta_{kr} = u_{kr} \frac{\sum_{1}^{n} G_k u_{kr}}{\sum_{1}^{n} G_k u_{kr}^2} \quad ; \quad \text{where:}$$

u_{kr} is the component of the rth eigenvector corresponding to kth degree of freedom or in other words is the component of the rth shape mode at the kth floor. The shape modes are obtained according to dynamics of structures.

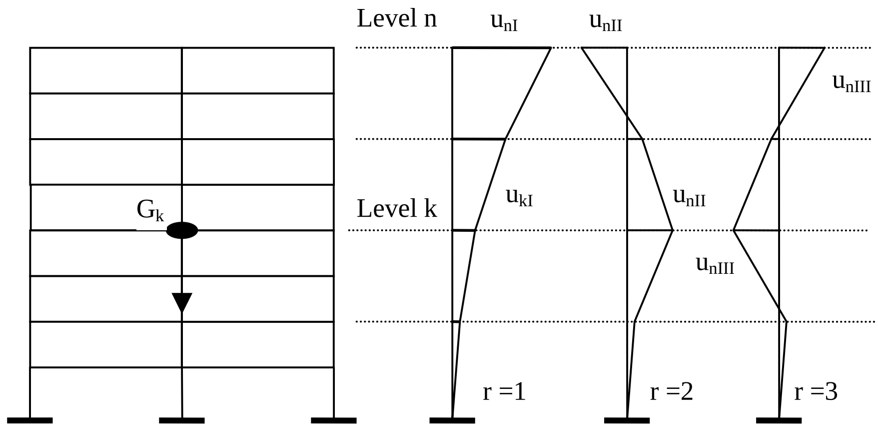


Figure 5: First three vibration modes

6) G_k represents the resultant of gravity loads at the kth floor.

7) The resultant of horizontal seismic forces (the base shear force) can be calculated either by summing all the level seismic forces S_{kr} , or directly with the relation:

$$S_r = K_s \cdot \beta_r \cdot \psi \cdot \varepsilon_r \cdot G \qquad \textbf{;} \qquad \text{where:}$$

$$G = \sum_1^n G_k \qquad \textbf{, and}$$

ε_r is the coefficient of equivalence between the real system (with "n" degrees of freedom) and a system with a single degree of freedom corresponding to the rth vibration mode. It is determined with the equation:

$$\varepsilon = \frac{\left[\sum_1^n G_k u_{kr}\right]^2}{\left[\sum_1^n G_k\right]\left[\sum_1^n G_k u_{kr}^2\right]} = \frac{\sum_1^n G_k \eta_{kr}}{G}$$

Remarks:

The efforts in the structural elements are separately calculated for each vibration mode. Usually, it is enough to consider only the first three oscillation modes. The total design effort "N" in a section of a structural element is obtained with the relation:

$$N = \sqrt{\sum_1^3 N_r^2} \qquad ; \qquad \text{where:}$$

N_r is the effort in the considered section corresponding to the rth vibration mode.

2.3.2. Establishing the horizontal seismic forces for the current types of buildings

The current types of buildings are those constructions that have a fundamental vibration period less than 1.5 sec., as for example: multi-story residential and socio-cultural buildings with less than 10 stories and a maximum height above the ground of 30 m; industrial and agricultural one-story halls and multi-story industrial constructions with at most 6 stories and a maximum height of 25 m above the ground. The resultant S of horizontal seismic forces is determined, considering the fundamental vibration mode, with the expression:

$$S = c \cdot G \quad ; \quad \text{where:}$$

$c = K_s \cdot \beta \cdot \psi \cdot \varepsilon$ is called global seismic coefficient (of the base shear force); the coefficients β and ε correspond to the fundamental vibration mode.

It is admitted to directly determine the horizontal seismic forces using the global seismic coefficients given the following table.

Table 8: Global seismic coefficient

No.	Construction type and constructive system	β	ε	ψ	$c/k_s = \beta\varepsilon\psi$
1	Buildings with a rigid structure (masonry or reinforced concrete cast-in-place or prefabricated shear walls) or a semi–rigid structure (shear walls interacting with frames):				
	- up to 5 stories (including the ground level) and the total height less or equal to 15 m	**2**	**0.75**	**0.30**	**0.45**
	- higher than 5 stories and the total height over 15 m	**2**	**0.75**	**0.25**	**0.38**
2	Multi-story frame buildings:				
	- with one span	**2**	**0.80**	**0.25**	**0.40**
	- with more spans	**2**	**0.80**	**0.20**	**0.32**
3	Industrial one-story plants and other related constructions:				
	- with one span	**2**	**1.00**	**0.20**	**0.40**
	- with more spans	**2**	**1.00**	**0.15**	**0.30**

The resultant S of the horizontal seismic forces is distributed on the construction height with the relation:

$$S_k = S\frac{G_k u_k}{\sum_1^n G_k u_k} \approx S\frac{G_k h_k}{\sum_1^n G_k h_k} \quad ; \quad \text{where:}$$

S_k is the horizontal seismic force at the kth level (floor). The fundamental shape mode of the structure is approximated through a linear variation.

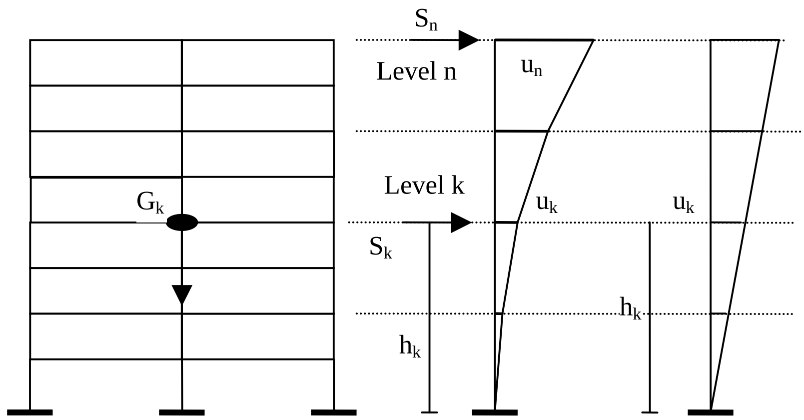


Figure 6: Linear approximation of the fundamental vibration mode

2.4. CODE P100 – 81

2.4.1. General relations for establishing the seismic horizontal forces acting on a structure

The seismic horizontal forces that act on a building are established for each eigenmode. The force that acts at the level k (on the direction of the degree of freedom) in the r[th] eigenmode is calculated with the relation:

$$S_{kr} = c_{kr} G_k \quad \text{where:}$$

$$c_{kr} = k_s \beta_r \psi \eta_{kr} \quad \text{where:}$$

1) c_{kr} is the seismic coefficient for level k and for the r[th] eigenmode;

2) k_s seismic intensity coefficient corresponding to the seismic protection level. It means the ratio between the peak ground acceleration of the seismic motion corresponding to the seismic zone where the building is situated and the gravity acceleration. The values for k_s are given in the next table.

Table 9: Values for the K_s coefficient

Values for the K_s coefficient							
Seismic protection level of the building	6	$6\frac{1}{2}$	7	$7\frac{1}{2}$	8 [§]	$8\frac{1}{2}$	9
K_s	0.07	0.09	0.12	0.16	0.20	0.26	0.32

3) β_r dynamic amplification coefficient in the r[th] eigenmode of the building. It is calculated depending on the vibration period of the structure T_r (for the considered eigenmode) and on the nature of the ground type as follows:

[§] Bucharest had a 8 seismic protection level (MSK-64 intensity scale was used.)

- ground type with normal stiffness, $\beta_r = \frac{3}{T_r}$**,** obeying the requirement $0.75 \le \beta_r \le 2.0$, where T_r is measured in seconds;
- for stiff ground: rock, deposits of very dense sand, gravel or very stiff clay (tertiary layers or older) etc., the values of the coefficient β_r are reduced by 20%, obeying the previous inequality;
- for ground with reduced stiffness: deposits of layers of low consistency clay (the consistency index $I_c < 0.5$) with or without intrusions of sand or other cohesionless soils, loose sand (density index $D < 0.33$), high-humidity loess ($w > 20\%$), as well as in the case of ground with high level of the underground water (depth < 5 m), the values of the coefficient β_r are increased by 30%, but it must be less than $\beta_r = 2.5$.

4) ψ is the coefficient for reduction of seismic effects taking into account the structural ductility, the stress re-distribution capacity, the contribution of strength reserve not considered in the analysis from the co-operation among of the structure with non-structural elements and the vibration damping effects. The values of the ψ coefficient are given in the next table.

Table 10: The values of the ψ coefficient

No.	Type of building and structural system	Coefficient ψ
1	Buildings with rigid structure (masonry structural walls or reinforced concrete structural walls, monolithically cast-in-situ or precast) or with semi – rigid one (structural walls co-operating with frames):	
	- up to ground floor + 4 floors ($\le$ 5 floors) and having the total height less than 15 m	**0.30**
	- more than ground floor + 4 floors ($>$ 5 floors) and having the total height superior to 15 m	**0.25**
2	Multi-story buildings with frame structure:	
	- having a single span	**0.25**
	- having more spans	**0.20**
3	Single story industrial buildings and other hall-type buildings:	
	- having a single span	**0.20**
	- having more spans	**0.15**
4	Silos, including the ones with cells supported on columns and other special constructions with rigid structure	**0.25**
5	Very flexible tall constructions like chimneys, independent towers	**0.35**
6	Water tanks	**0.35**

Remarks:

1. The values of the ψ coefficient for multi-storey buildings with frame structure are established considering that the structure fulfills the requirements for ductile behavior.

2. For steel structures, the values of ψ given in the table are reduced by 20%.

3. For the constructions at point 5 and 6 the values of the ψ coefficient are given considering that the provisions of present-day codes apply, according to which in the special combinations that contain the seismic action, the wind load is considered with a reduced intensity.

5) η_{kr} is the coefficient that describes the distribution of the seismic force at the level „k" in the „r" eigenmode, which is determined with the next relation:

$$\eta_{kr} = u_{kr} \frac{\sum_{k=1}^{n} G_k u_{kr}}{\sum_{k=1}^{n} G_k u_{kr}^2} \qquad \text{where}$$

u_{kr} is the component of the (r) modal shape on the (k) degree of freedom. The modal shapes are determined by means of the methods of dynamics of structures.

6) The resultant of the horizontal seismic forces (the base shear force) can be calculated either by summation of the level horizontal seismic forces S_{kr}, or directly with the relation:

$$S_r = K_s \cdot \beta_r \cdot \psi \cdot \varepsilon_r \cdot G \qquad \text{where:}$$

$G = \sum_{1}^{n} G_k$ **,** and the coefficient of equivalence ε_r between the actual system and the one with a single degree of freedom corresponding to the r vibration mode is calculated with the relation:

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

Remarks:

The efforts in the structure are determined separately for each vibration mode r. Usually, taking into account the first three vibration modes is sufficient. The total design effort N in a cross-section of an element of the structure is calculated with the relation:

$$N = \sqrt{\sum_{1}^{3} N_r^2} \text{ where}$$

N_r is the effort in the considered cross-section corresponding to the r vibration mode.

2.4.2. Calculation of the horizontal seismic forces for current buildings structures

By current type structures one can understand those structures whose fundamental natural period is less than 1.5s, that is multi-storey buildings for dwelling or for socio-cultural purposes, having less than 9 floors and the height less than 30m above the ground, industrial halls and agricultural ones with a single storey and multi-story industrial buildings up to 6 floors and with the maximum height of 25m above the earth.

The resultant S of the horizontal seismic forces is calculated, considering the fundamental eigenmode, with the relation:

$$S = c \cdot G \qquad \text{where:}$$

$c = K_s \cdot \beta \cdot \psi \cdot \varepsilon$ is called global seismic coefficient (of the base shear force); β and ε coefficients correspond to the fundamental eigenmode.

The simplified calculation of the horizontal seismic forces is accepted, by means of the values given in the next table for the global seismic coefficient c.

Table 11: Global seismic coefficient

No.	Type of building and structural system	β	ε	ψ	$c/k_s = \beta\varepsilon\psi$
1	Buildings with rigid structure (masonry structural walls or reinforced concrete structural walls, monolithically cast-in-situ or precast) or with semi – rigid one (structural walls co-operating with frames):				
	- up to ground floor + 4 floors (≤ 5 floors) and having the total height less than 15 m	**2**	**0.75**	**0.30**	**0.45**
	- more than ground floor + 4 floors (> 5 floors) and having the total height superior to 15 m	**2**	**0.75**	**0.25**	**0.38**
2	Multi-story buildings with frame structure:				
	- having a single span	**2**	**0.80**	**0.25**	**0.40**
	- having more spans	**2**	**0.80**	**0.20**	**0.32**
3	Single story industrial buildings and other hall-type buildings:				
	- having a single span	**2**	**1.00**	**0.20**	**0.40**
	- having more spans	**2**	**1.00**	**0.15**	**0.30**

The distribution of the resultant S of the horizontal seismic forces on the height of the building is calculated with the relation:

$$S_k = S\frac{G_k u_k}{\sum_1^n G_k u_k} \approx S\frac{G_k h_k}{\sum_1^n G_k h_k} \quad \text{where:}$$

S_k is the horizontal seismic force at the k floor. The fundamental mode shape is approximated by an inclined straight line.

2.5. CODE P100 – 92

2.5.1. Basic relations for horizontal seismic loads computation for structures

The relations further provided will be 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”-th vibration mode, is determined using further relations:

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

1) c_r is the overall seismic coefficient, corresponding to the „r”-th 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. Based on the performance criteria the buildings are splitted into four performance classes.

Table 12: Classes of importance for buildings

Classes of importance			
I	II	III	IV
1.4	1.2	1.0	0.8

Table 13: 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:***
	- hospitals ambulance services, fire-stations; - administration buildings of great importance in organizing the emergency post-earthquake activity; - communication buildings of national or country level importance; - power plants belonging to the national system - buildings hosting museums of national importance.
Class II	***Very important buildings requiring a limitation of damage, keeping in view its potential consequences:***
	- other health protection buildings; - schools, crèches, kinder gardens, boarding-houses for children handicapped, old people; - high occupancy buildings: sports and performance halls, churches, important shopping centers; - buildings sheltering special artistic, historic, scientific values; - industrial buildings and plants where fire or toxic gases leakage may occur; - industrial buildings with equipment of high economic importance; - stores for highly necessary products to be used in case of emergency.
Class III	***Normal importance buildings (not falling into classes I, II, or IV)***
	- residential buildings, hotels, boarding-houses (except those in class II) - common industrial and agrozootechnical buildings.
Class IV	***Reduced importance buildings***
	- agrozootechnical structures of reduced importance (e.g. greenhouses, various single story buildings for animal and poultry breeding. a.s.o.); - one or two story residential buildings; - other civil and industrial buildings sheltering goods of low importance and reduced staff.

Remarks:

The above classification of the buildings on the importance classes is not the same as that provided by STAS 10100/75.

The types of buildings that are not explicitly mentioned in the table, will fall into the four classes, based on a technical argumentation provided by the design theme;

In certain cases, some components of the buildings, including installations and equipment, may fall into different classes of importance than the other parts of the buildings

The decision on the classes of importance the buildings fall into, will be made by the owner, who will be advised by the technical authorities in charge.

4) The "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" coefficient are supplied in the next table, according to the seismic zones described on the code map.

Table 14: The k_s coefficient

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

5) The amplification factor ("β_r") is determined according to the natural oscillation periods ("T_r") of the building and to the local seismic conditions, characterized by corner periods ("T_c") using the following relations:

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

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

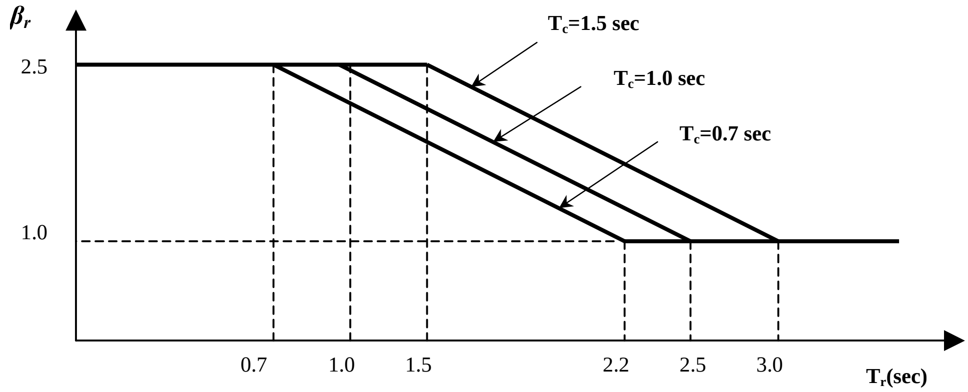


Figure 7: Amplification spectra

6) The coefficient of reduction of seismic forces ("y") due to the structural ductility, to the redistribution of the efforts and to the damping effects is provided in the following table:

[**] Bucharest belongs to seismic zone C.

Table 15: The values of ψ coefficient according to structural type

Structure type	ψ coefficient
A. 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*
7 Silos:...	*0.25*
B. 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*

Remarks:

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

$$\varepsilon = \frac{\left[\sum_{k=1}^{n} G_k u_{kr}\right]^2}{G\sum_{k=1}^{n} G_k u_{kr}^2}$$ 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 will be determined using structural dynamics methods.

8) The seismic load, acting at the „k"-th level on the DOF direction, corresponding to the „r"-th vibration mode, is determined using the following relation:

$$S_{kr} = c_{kr} G_k$$ where:

- S_{kr} is the seismic load acting at the level „k" on the direction of the „r"-th dynamic degree of freedom.

- $c_{kr} = \alpha k_s \beta_r \psi \eta_{kr}$ where:

$$\eta_{kr} = u_{kr} \frac{\sum_{k=1}^{n} G_k u_{kr}}{\sum_{k=1}^{n} G_k u_{kr}^2}$$

Remark:

The values S_{kr} may be obtained by the distribution of the resultant (base shear force) „S_r"of the seismic loads by the relation

$$S_{kr} = S_r \frac{G_k u_{kr}}{\sum_{k=1}^{n} G_k u_{kr}}$$

9) The internal forces, displacements, a.s.o., are calculated separately for each natural vibration mode. If the oscillations analysis can be made independently, it is usually sufficient to take into account the first three natural vibration modes.

For the case of spectral analysis of the vertical plane vibrations, the overall stress („N") at a certain member section is determined using the relation:

$$N = \sqrt{\sum_{1}^{3} N_r^2}$$ where

N_r is the section force, corresponding to vibration mode „r".

Remark:

For crane-bridge working inside a one-story building braced in the longitudinal direction by ductile frames, special measures will be applied for an eventual bracing system collapse. Therefore, the structure without bracing system will be designed for a seismic coefficient over than $0.3k_s \geq 0.05$.

The designed energy dissipating mechanism has to induce plastic deformations at the columns bottom, as well as under the covering beams; the structure should be stiffened by bracings placed above the covering beams.

2.6. Conclusions:

2.6.1. Capacity Curves for Representative Building Types in Bucharest

2.6.1.1. Description of Model Building Types

Table 16 lists the model building types most widely used in Bucharest.

Table 16: Model Building Types

No.	Label	Description	HEIGHT			
			Range		Typical	
			Name	Stories	Stories	Meters
1	RC1L	Concrete Moment Frame	Low-Rise	1 - 3	2	5.7
2	RC1M		Mid-Rise	4 - 7	6	17.1
3	RC1H		High-Rise	8+	10	28.5
4	RC2L	Concrete Shear Walls	Low-Rise	1 - 3	2	5.7
5	RC2M		Mid-Rise	4 - 7	6	17.1
6	RC2H		High-Rise	8+	10	28.5

2.6.1.2. Capacity Curves

A building capacity curve (also known as a push-over curve) is a plot of a building's lateral load resistance as a function of a characteristic lateral displacement (i.e., a force-deflection plot). It is derived from a plot of static-equivalent base shear versus building (e.g., roof) displacement. In order to facilitate direct comparison with earthquake demand (i.e. overlaying the capacity curve with a response spectrum), the force (base shear) axis is converted to spectral acceleration and the displacement axis is converted to spectral displacement.

The building capacity curves developed are based on engineering design parameters and judgment. Three control points that define model building capacity describe each curve:

- Design Capacity
- Yield Capacity
- Ultimate Capacity

Design capacity represents the nominal building strength required by current model seismic code provisions or an estimate of the nominal strength for buildings not designed for earthquake loads.

Yield capacity represents the true lateral strength of the building considering redundancies in design, conservatism in code requirements and true (rather than nominal) strength of materials. Ultimate capacity represents the maximum strength of the building when the global structural system has reached a fully plastic state. Ultimate capacity implicitly accounts for loss of strength due to shear failure of brittle elements. Typically, buildings are assumed capable of deforming beyond their ultimate point without loss of stability, but their structural system provides no additional resistance to lateral earthquake force.

Up to the yield point, the building capacity curve is assumed to be linear with stiffness based on an estimate of the true period of the building. The true period is typically longer than the code-specified period of the building due to flexing of diaphragms of short, stiff buildings, flexural cracking of elements of concrete and masonry structures, flexibility of foundations and other factors observed to affect building stiffness. From the yield point to the ultimate point, the capacity curve transitions in slope from an essentially elastic state to a fully plastic state. The capacity curve is assumed to remain plastic past the ultimate point. An example building capacity curve is shown in Figure 8.

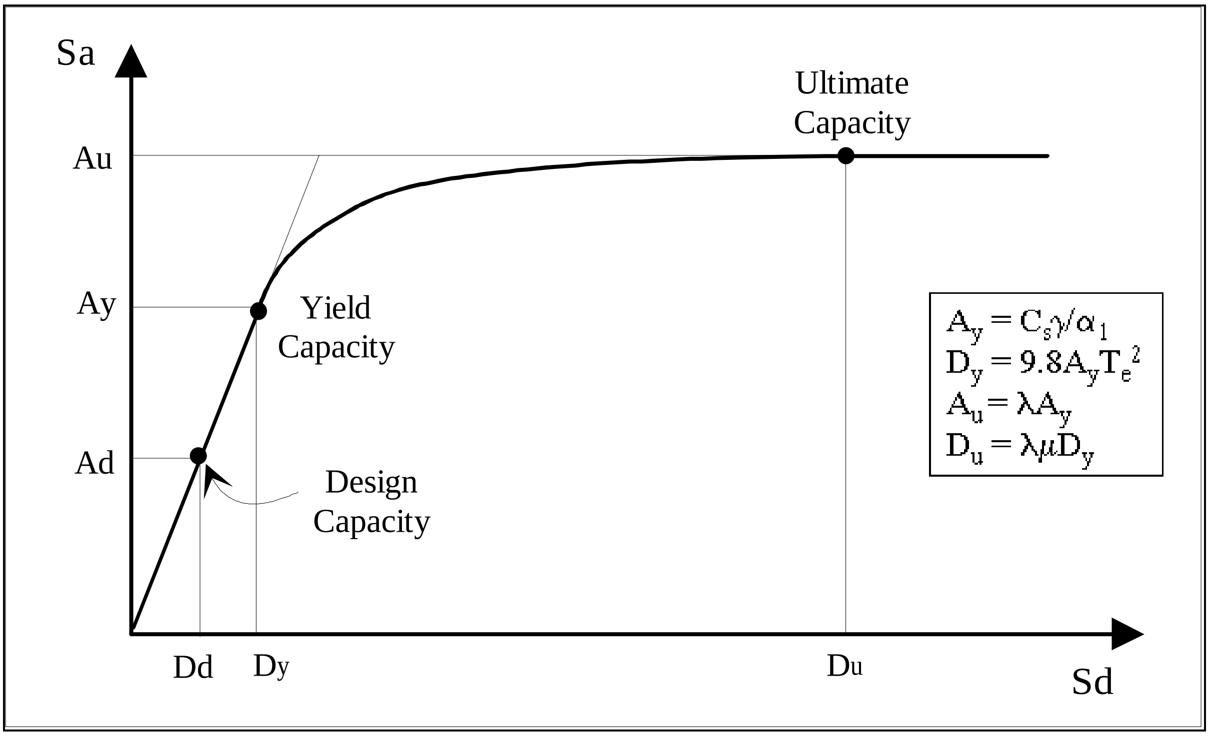


Figure 8: Example of Building Capacity Curve

The building capacity curves are constructed based on estimates of engineering properties that affect the design, yield and ultimate capacities of each model building type. These properties are defined by the following parameters, Figure 8:

- C_s design strength coefficient (fraction of building's weight),
- T_e true "elastic" fundamental-mode period of building (seconds),
- α_1 fraction of building weight effective in pushover mode,
- α_2 fraction of building height at location of pushover mode displacement,
- γ "overstrength" factor relating "true" yield strength to design strength,
- λ "overstrength" factor relating ultimate strength to yield strength, and
- μ "ductility" factor relating ultimate displacement to λ times the yield displacement (i.e., assumed point of significant yielding of the structure)

The design strength, C_s is based on the lateral-force design requirements of seismic codes. These requirements are a function of the building's seismic zone location and other factors including type of lateral-force-resisting system and building period.

2.6.1.3. Major developments in four generations of seismic codes

Pre-code period (prior to 1941/45):

1941 Draft Instructions
1945 Instructions

- Development of first national seismic code after the 1940 earthquake: lateral seismic force was 5% of building

Low-code period (1963-1977):

P13-63

- Control period of response spectra T_C=0.3 s
- Maximum dynamic amplification DAF = max SA/PGA = 3
  SA - structure absolute acceleration response spectra.

P13-70

- Control period of response spectra T_C=0.4 s
- Max. dynamic amplification DAF = 2
- Some ductility rules for RC frames.

Moderate-code period (1978 - 1990):

P100-78
P100-81

- Development of the first seismic code based on the unique strong ground motion recorded in soft soil of Bucharest during March 4, 1977 event: PGA = 0.2g and the long predominant period of ground vibration was T_p =1.6s.
- Control period of response spectra T_C=1.5 s
- Maximum dynamic amplification DAF = 2
- Ductility rules for RC shear wall & frame structures.

Moderate to High-code period (1990-present):

P100-90
P100-92

- Control period of response spectra T_C=1.5 s
- Maximum dynamic amplification DAF = 2.5
- Advanced ductility rules for RC shear wall and frame structures and for steel structures.

It is emphasized that after the 1977 event, new ductility rules for RC structures were imported from *US* practice and incorporated into Romanian seismic codes, *P100.* According to the *EUROCODE 8* requirements the rules were significantly improved after 1989.

Table 17 summarizes design capacity for each building type and design level. Building period, T_e, push-over mode parameters α_1 and α_2, the ratio of yield to design strength, γ, and the ratio of ultimate to yield strength, λ, are assumed to be independent of design level. Values of these parameters are summarized in Table 18&Table 19 for each building type. Values of the "ductility" factor, μ, are given in Table 20 for each building type and design level. The values are given only for medium and high-rise RC buildings considered to be representative for Bucharest.

Table 17: Code Building Capacity Parameters - Design Strength (C_s)

Building Type	Seismic Design Level (Percentage of Building Weight)				
	1941-1962	1963-1969	1970-1977	1978-1989	1990-2002
RC1M	5	3.21	3.4	8	10
RC1H	5	2.25	2.4	8	10
RC2M	5	7.71	7.2	10	12.5
RC2H	5	4.15	4.4	10	12.5

Table 18: Code Building Capacity Parameters - Period (T_e), Pushover Mode Response Factors (α_1, α_2) and Overstrength Ratios (γ, λ) - 1941-1977

Building Type	Height to Roof (m)	Period, T_e (Sec)	Modal Factors		Overstrength Ratios	
			Weight, α_1	Height, α_2	Yield, γ	Ultimate, λ
RC1M	17.1	0.7	0.85	0.6	1.4	1.5
RC1H	28.5	1	0.75	0.6	1.4	1.5
RC2M	17.1	0.4	0.9	0.7	1.4	1.5
RC2H	28.5	0.7	0.8	0.7	1.4	1.5

Table 19: Code Building Capacity Parameters - Period (T_e), Pushover Mode Response Factors (α_1, α_2) and Overstrength Ratios (γ, λ) - 1978-2002

Building Type	Height to Roof (m)	Period, T_e (Sec)	Modal Factors		Overstrength Ratios	
			Weight, α_1	Height, α_2	Yield, γ	Ultimate, λ
RC1M	17.1	0.6	0.85	0.6	1.5	2
RC1H	28.5	0.85	0.75	0.6	1.5	2
RC2M	17.1	0.35	0.9	0.7	1.5	2
RC2H	28.5	0.6	0.8	0.7	1.5	2

Table 20: Code Building Capacity Parameter - Ductility (μ)

Building Type	Seismic Design Level (Percentage of Building Weight)				
	1941-1962	1963-1969	1970-1977	1978-1989	1990-2002
RC1M	2	3	3	5	5
RC1H	2	3	3	5	5
RC2M	2	3	3	4	4
RC2H	2	3	3	4	4

Building capacity curves are assumed to have a range of possible properties that are lognormal distributed. Capacity curves described by the values of parameters given in Table 17, Table 18 Table 19and Table 20 represent median (best) estimates of building capacity.

Table 21, Table 22, Table 23, Table 24 and Table 25 summarize yield capacity and ultimate capacity control points.

Table 21: Code Building Capacity Curves - 1941-1962

Building	Yield Capacity Point		Ultimate Capacity Point	
Type	D_y (cm)	A_y (g)	D_u (cm)	A_u (g)
RC1M	1.00	0.082	3.01	0.124
RC1H	2.32	0.093	6.96	0.140
RC2M	0.31	0.078	0.93	0.117
RC2H	1.07	0.088	3.20	0.131

Table 22: Code Building Capacity Curves - 1963-1969

Building	Yield Capacity Point		Ultimate Capacity Point	
Type	D_y (cm)	A_y (g)	D_u (cm)	A_u (g)
RC1M	0.64	0.053	2.90	0.079
RC1H	1.04	0.042	4.70	0.063
RC2M	0.48	0.120	2.15	0.180
RC2H	0.88	0.073	3.98	0.109

Table 23: Code Building Capacity Curves - 1970-1977

Building	Yield Capacity Point		Ultimate Capacity Point	
Type	D_y (cm)	A_y (g)	D_u (cm)	A_u (g)
RC1M	0.68	0.056	3.07	0.084
RC1H	1.11	0.045	5.01	0.067
RC2M	0.45	0.112	2.00	0.168
RC2H	0.94	0.077	4.22	0.116

Table 24: Code Building Capacity Curves - 1978-1989

Building	Yield Capacity Point		Ultimate Capacity Point	
Type	D_y (cm)	A_y (g)	D_u (cm)	A_u (g)
RC1M	1.26	0.141	12.63	0.282
RC1H	2.87	0.160	28.73	0.320
RC2M	0.51	0.167	4.06	0.333
RC2H	1.68	0.188	13.42	0.375

Table 25: Code Building Capacity Curves - 1990-2002

Building	Yield Capacity Point		Ultimate Capacity Point	
Type	D_y (cm)	A_y (g)	D_u (cm)	A_u (g)
RC1M	1.58	0.176	15.79	0.353
RC1H	3.59	0.200	35.91	0.400
RC2M	0.63	0.208	5.07	0.417
RC2H	2.10	0.234	16.77	0.469

The values of yielding and ultimate displacement and acceleration given in Table 21 to Table 25 are represented in Figure 9, Figure 10, Figure 11 and Figure 12 for building types *RC1H* and *RC2H*.

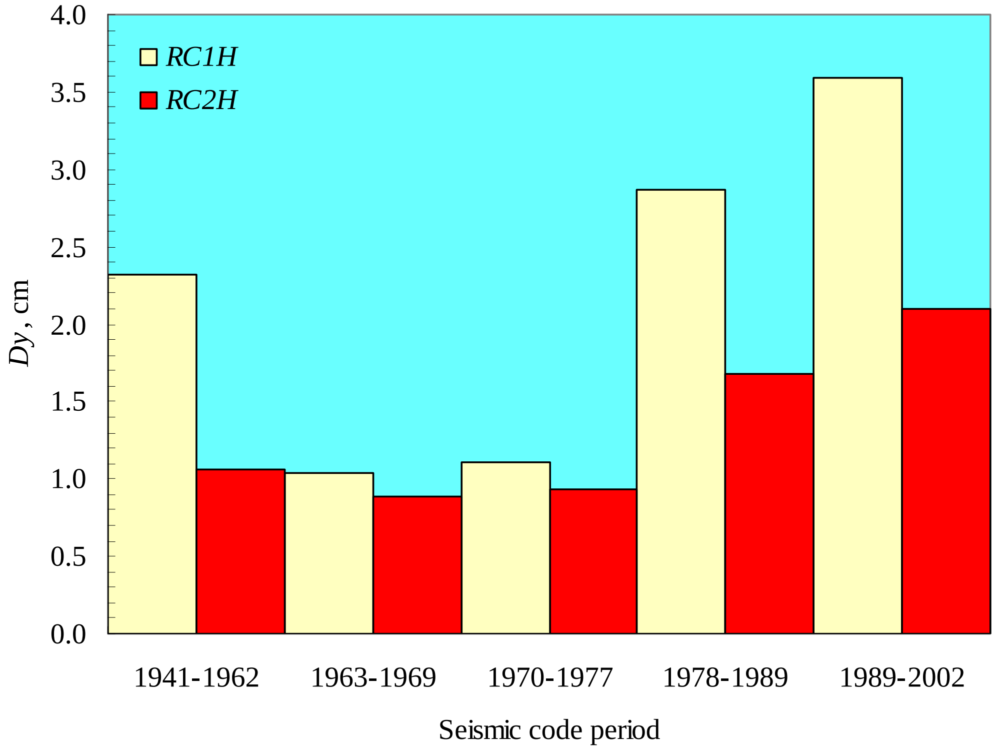


Figure 9: Yielding displacement according to seismic code period

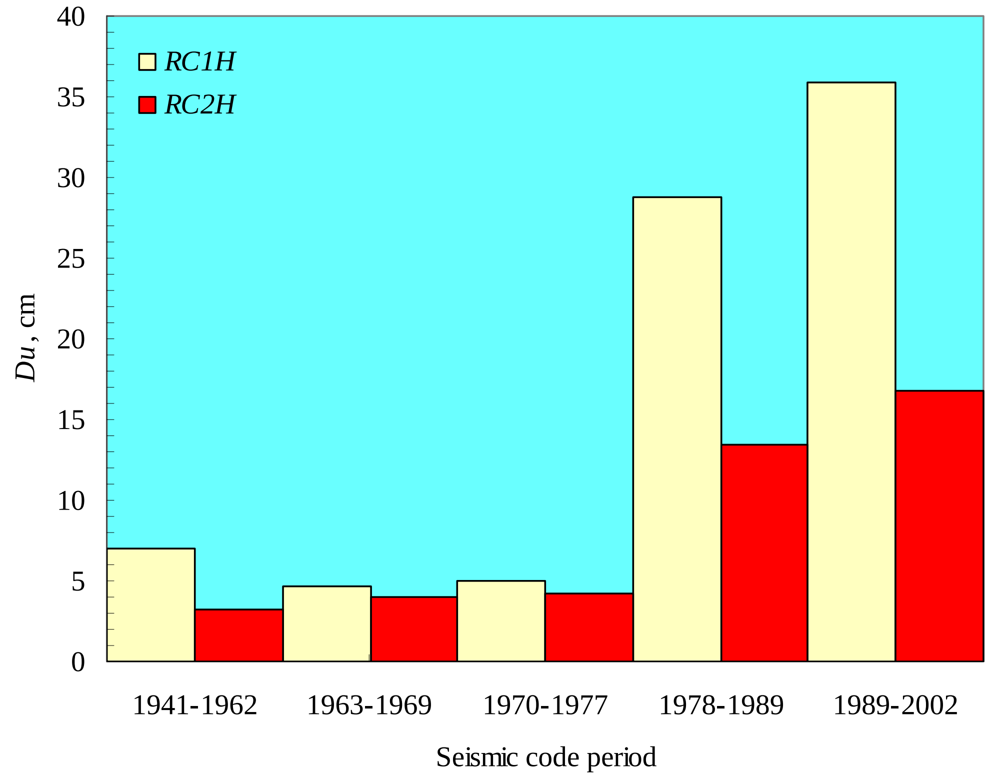


Figure 10: Ultimate displacement according to seismic code period

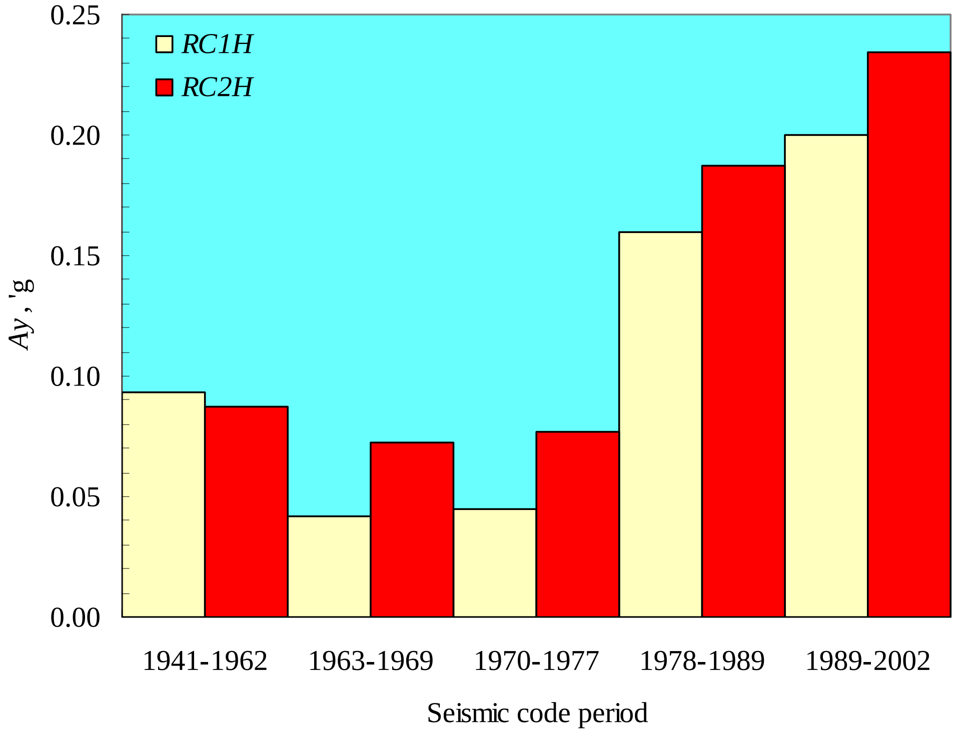


Figure 11: Yielding acceleration according to seismic code period

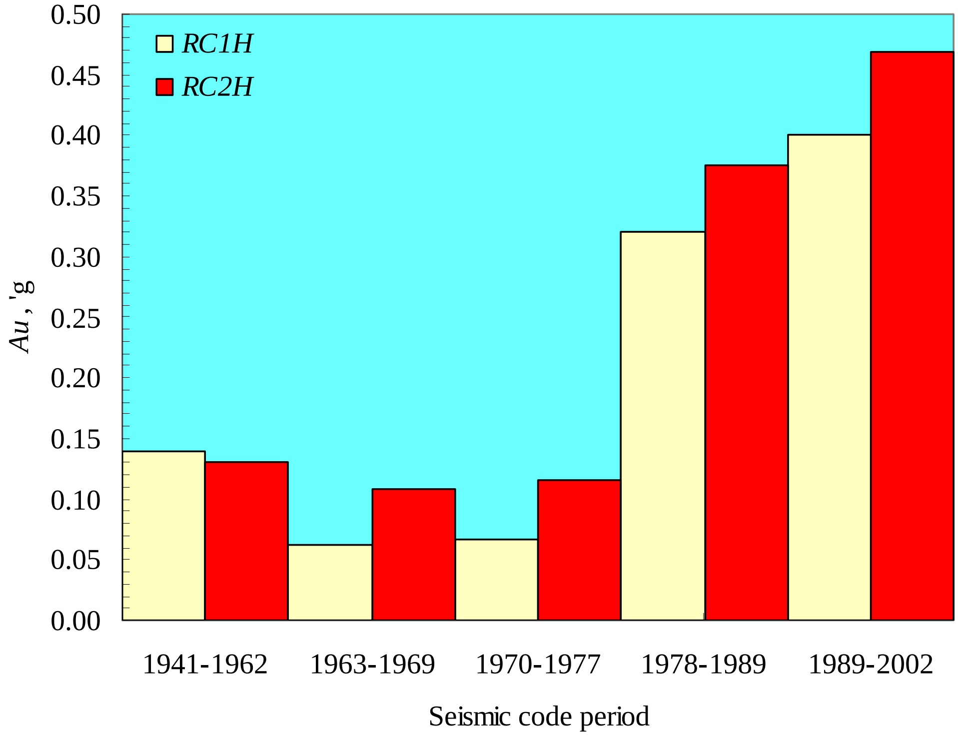


Figure 12: Ultimate acceleration according to seismic code period

3. EVOLUTION OF THE REINFORCED CONCRETE DESIGN CODES

3.1. German Recommendations (1950)

The design codes for reinforced concrete structures appeared in Romania since 1932. These recommendations were not enforced as design codes but designers used them as technical regulations. The first rules that were used in Romania were translated from the German book “Beton Kalender”. The most known translation belongs to the professor M. D. Hangan (1932)[1].

The principles of the method are presented in the following paragraphs.

3.1.1. Reinforced concrete beam subjected to pure bending: Generalities [2]

Under small loads the beam is in the behavior stage 1, defined by the fact that whole concrete section is active in taken the exterior efforts. The concrete from the tension zone take the great part of the tension stresses. The tension reinforcement is as small acted as the width of the beam increases. The greatest tension stresses in the concrete are smaller than the concrete tensile ultimate stresses, leading to no cracks in the beam.

As the exterior loads on the beam are raising, the tensile stresses in the concrete are greater than the concrete ultimate tensile stress and cracks appears in the region of the largest bending moment. The cracks are perpendicular to the direction of principal stress. The beam is in behavior stage II meaning that the tensile stresses are taken by the tension reinforcement while the tension concrete zone has practically no static influence on the behavior of the beam.

The further increasing in the exterior loads leads to reaching of the ultimate strain in the tensile reinforcement and then to beam failure.

In the behavior stage I the neutral axis is in the lowest position while in the behavior stage III the neutral axis is in upper position.

The computation models generally used are assuming the behavior stage II – with cracked tension concrete zone and are based on the following simplified assumptions:

a) the plane section remain plane after the bending

b) Hook’s low “the stresses are proportional to strains” is applied to steel as well as to concrete from compressed zone. The elasticity modules used are:

$$\begin{cases} E_a = 2100000 kg/cm^2 \\ E_b = 140000 kg/cm^2 \\ n = E_a / E_b = 15. \end{cases}$$

c) The tension stresses are taken only by the tension reinforcement. The tensile zone of the concrete is considered as not-subjected to stresses and the concrete situated under the neutral axis is considered as unsolicited.

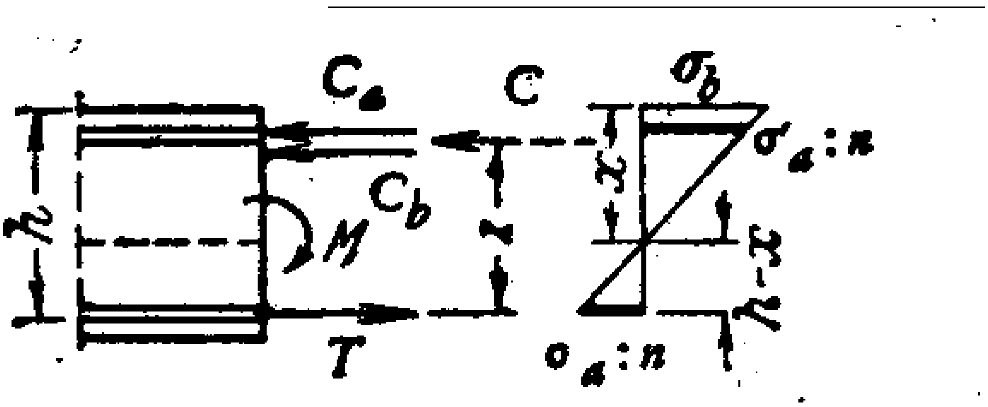


Figure 13: Simple reinforced concrete section

In order to establish the stresses the following equations are used (see Figure 13):

From hypothesis a and b it results:

$$\frac{1}{n}\cdot\sigma_a : \sigma_b = (h-x) : x \qquad \frac{1}{n}\cdot\sigma'_a : \sigma_b = (x-h') : x$$

$$\sigma_a = n\sigma_{b\cdot}\cdot\frac{h-x}{x} \qquad \sigma'_a = n\sigma_{b\cdot}\cdot\frac{x-h'}{x}$$

$$x = h\cdot\frac{n\sigma_b}{n\sigma_b+\sigma_a} = \alpha h$$

If we denote by Cb the sum of all the compression stresses in the concrete, Ca the sum of the compression stresses of the steel reinforcement the resultant of Cb and Ca is C. Let's denote by T the sum of all tensile stresses in the reinforcement. In order to have an equilibrium in the section we can write:

Cb+Ca-T=0 or C=T

The forces C and T represents the force couple with internal level arm z. This couple should be in equilibrium with the exterior bending moment M: so

M=Cz=Tz

The above equations are sufficient to determine the stresses σ_b, σ_a and σ'_a when the external bending moment M and beam dimensions are known.

3.1.2. Reinforced concrete beam subjected to pure bending: dimensioning

We will consider only the case of the rectangular shape beam.

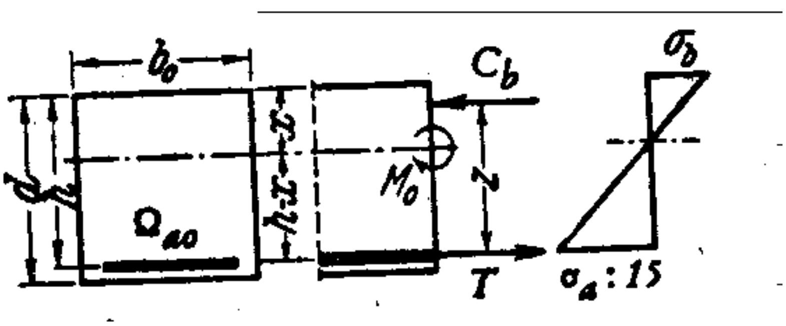


Figure 14: Simple reinforced concrete section

If we choose the stresses σ_b and σ_a the neutral axis position results from the equation $x = \alpha h$. The internal level arm (for triangular distribution of compression stresses) of the interior forces is:

$$z = h - \frac{1}{3} \cdot x = \beta \cdot h,$$

and the sum of the compression stresses:

$$C_b = \frac{1}{2} \cdot \sigma_b x b_o = b_o h \cdot \frac{\alpha \sigma_b}{2}$$

The sum of the tensile stresses is: $T = \Omega_{ao} \cdot \sigma_a$

Because C_b =T we get: $\Omega_{ao} = \frac{b_o h}{k}$ where $k = \frac{2\sigma_a}{\alpha \sigma_b}$

From the moment equation we can obtain:

$$M_o = C_b \cdot z = b_o h \cdot \frac{\alpha \sigma_b}{2} \cdot \beta h \quad \text{or}$$

$$M_o = \frac{b_o h^2}{r^2} \quad \text{where } r^2 = \frac{2}{\alpha \beta \sigma_b}$$

Effective height is $h = r\sqrt{\frac{M_o}{b_o}}$

3.1.3. Reinforced concrete columns subjected to bending with axial force: Generalities

Lets consider the case of an exterior axial force N acting in the axis of the column and a bending moment M_m with respect to the same column axis. We can replace the two efforts by only one exterior force N applied eccentrically at the distance $e_m = M_m$:N. e_m is considered from the column axis to the compressed zone of the section if it is compression and to the tension zone of the section if it is a tensile force. The eccentrically force N with the eccentricity e_m should be in equilibrium with the resultant of the interior forces R_i, so N should be equally and opposite sign to R_i.

If consider the values of Mm and N we can have to distinct cases

a) if N is a compression or tension force, the whole section will be acting in the behavior stage 1 with the whole section compressed so that there will be no cracks. If the exterior force N is N is a tension force the active section will be only the part in between the two reinforcement layers. Because Ω_a and Ω'_a are tensioned, N should remain in equilibrium with the two tension forces.

This method can be applied as long as $e_m \leq \frac{c}{2}$. The concrete is in the behavior stage II.

In the German codes of practice translated into Romanian the computation of the columns subjected to bending with axial force in case I is made by the aid of the tables.

Let's consider the case of a concrete column subjected to bending and axial force with small eccentricity.

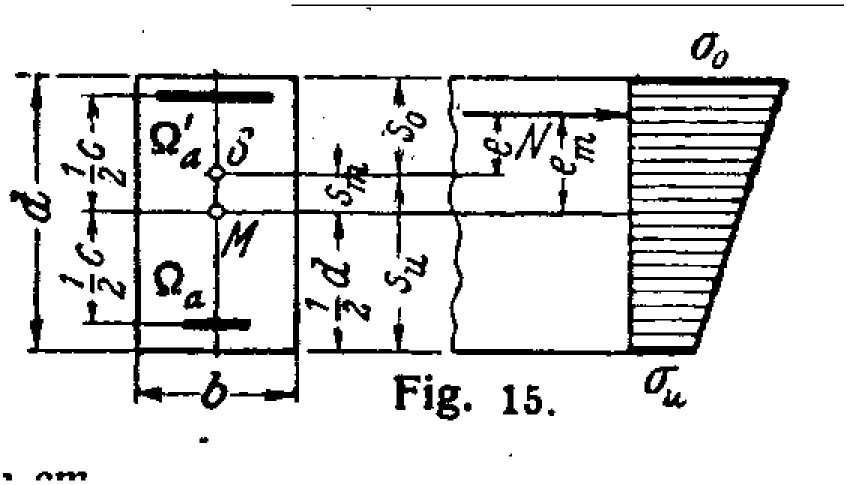


Figure 15: Concrete section subjected bending and axial force with small eccentricity

b =is the width of the concrete section

d =is the height of the concrete section

Ω_a $=\alpha bd$- is the tension reinforcement in cm^2

Ω'_a $=\alpha' bd$- is the compressed reinforcement in cm^2

Ω_I $=bd(1+15\alpha+15\ \alpha')=$ bd:γ is the ideal concrete section in cm^2

W_o $=bd^2{:}\delta_o$ is the resistance modulus for σ_o

W_u $=bd^2{:}\delta_o$ is the resistance modulus for σ_u

e_m =the eccentricity of the axial for

s_m $=\beta d$=the displacement of the gravity center from the middle section

In order to compute the extreme fiber stresses, σ_o and σ_u we have to compute first the quantities:

$$\psi_o = \gamma - \beta\delta_o \quad \psi_u = \gamma + \beta\delta_u \quad \gamma = \frac{1}{1+15\alpha+15\alpha'}$$

and we obtain for the case of asymmetric reinforcement.

$$\sigma_o = -\frac{N}{bd}\left(\psi_o + \frac{e_m\delta_o}{d}\right) \qquad \sigma_u = -\frac{N}{bd}\left(\psi_u - \frac{e_m\delta_u}{d}\right)$$

or in the case of symmetrical reinforcement:

σ_o respective $\sigma_u = -\frac{N}{bd}\left(\psi \pm \frac{e_m\delta}{d}\right)$.

The above equations should be used only when $\sigma_u \leq -\frac{1}{2}\sigma_o$. The maximum value of the eccentricity for this limit case is $e_{max}=\varepsilon d$.

The asymmetric reinforcement pattern is recommended if the resultant of the exterior forces is always acting on the same side of the section. For economy reasons it is recommended to use a 0.3% reinforcement ratio for the tension side so that α=0.003.

The symmetric reinforcement ($\Omega'_a=\Omega_a$) if the resultant of the exterior forces may fall on one side or the other side of the section.

b) The case of the column subjected to axial force and bending moment with great eccentricity.

$$e_m = M_m : N$$

This case is applicable when the exterior longitudinal forces is acting outside the central core of the section, for compressed rectangular sections when $e_m \geq \varepsilon d$ (ε given in tables with values between 0.310 and 0.448 depending on the reinforcement percentage) and when $\varepsilon \geq \frac{c}{2}$ for tensioned rectangular sections:

Sign rule: the bending moments are taken without sign

the axial force N is positive when it is a compression force

the axial force N is negative when it is tension force, so that e_m and e_u will be positive for compression and negative for tension.

The position of the axial force will be in the compressed zone for the compression exterior effort and in tensioned zone for the tensile exterior effort.

$$e_m = M_m : N \qquad e_u = e_m + u$$

The equilibrium equations:

$$C_a + C_b - T - N = 0$$

$$Ne_u + C_a c - C_b \upsilon = 0$$

We will call “replacement bending moment” the always-positive bending moment of the exterior force N with respect to the tension zone reinforcement.

$$M_u = Ne_u = N(e_m + u) = M_m + N_u$$

$$M_u = C_a c + C_b \upsilon$$

We can observe that the compressed zone of member subjected to bending and axial compression behaves like it is subjected to pure bending with a bending moment equal to M_u. The general dimensioning method for members subjected to axial compression with great eccentricity e_m is to consider the member under pure bending, to compute the reinforcement area and the to reduce the reinforcement amount with $N : \sigma_a$ so that the tensile reinforcement will be:

$$\Omega_a = \Omega_{au} - \frac{N}{\sigma_a}$$

According to the previous equation we have to reduce the reinforcement amount for compression case and to increase it for tension case, according to the sign of the exterior force N.

c) the case of the rectangular section used totally.

We choose the width b_o and the stresses σ_b and σ_a and we are interested in the effective height h and reinforcement amount Ωa.

Because in the expression for Mu the effective height it should be introduced by approximation with $h = r\sqrt{M_u : b_o}$. Usually Mu should be modified and then the value of *h* is recalculated and the reinforcement area determined by the formula:

$\Omega_a = \frac{b_o h}{k} - \frac{N}{\sigma_a}$. The coefficients k and r are given in tables.

d) Rectangular section, simple reinforced partially used:

We choose the dimensions b_o, h and steel stress and we are interested in the compression strength and steel reinforcement amount.

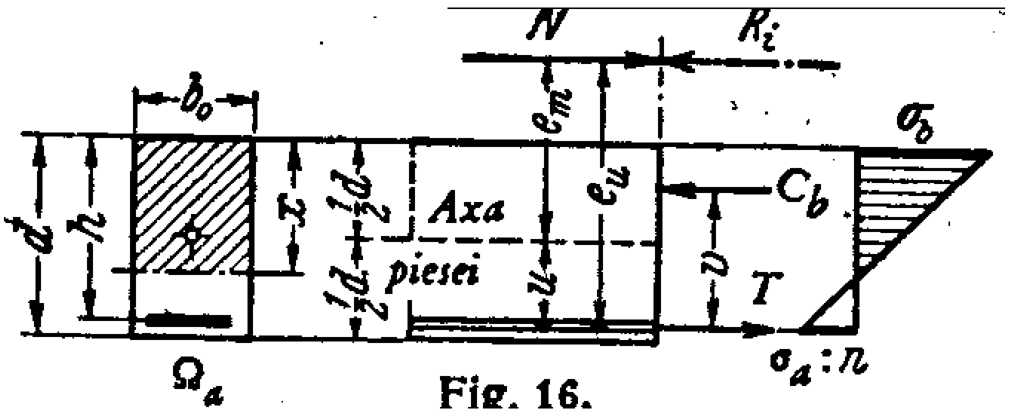


Figure 16: Rectangular section, single reinforced partially used

If $M_u < (b_o h^2 : r^2)$ the compression stress cannot be fully used, so we have to find $r^2 = b_o h^2 : M_u$ and then choose from tables σ_b and k corresponding to r^2 in order to apply the equation $\Omega_a = \frac{b_o h}{k} - \frac{N}{\sigma_a}$.

e) Double reinforced rectangular section:

We choose the dimensions b_o, g, c and allowable stresses σ_b and σ_a and we are interested in the reinforcement amount Ω_a and Ω'_a.

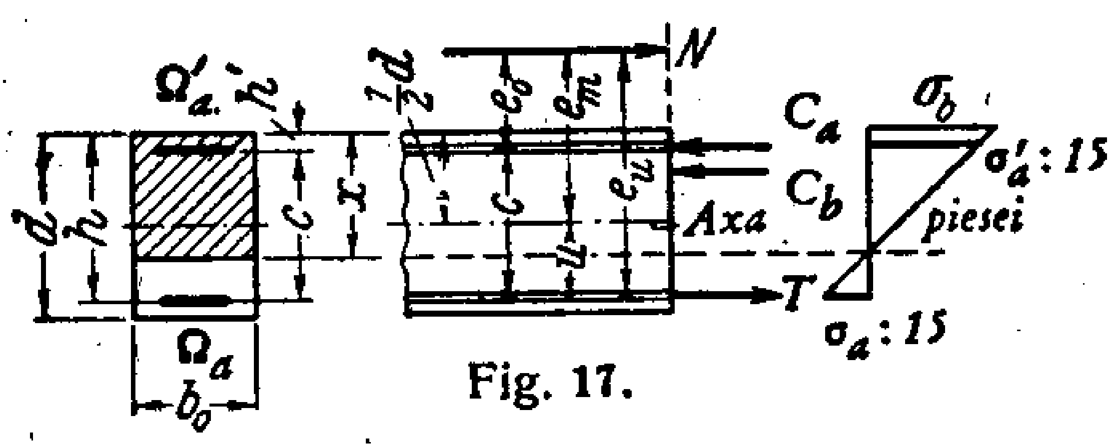


Figure 17: Double reinforced rectangular section

Usually the whole allowable stresses σ_b and σ_a are used. The compressed reinforcement is needed only when $M_u > \frac{b_o h^2}{r^2}$. In order to solve the problem we have to find in table the values for k, r^2, and σ_a' corresponding to σ_b and σ_a.

The unbalanced bending moment: $\Delta M = M_u - \frac{b_o h^2}{r^2}$

The tension reinforcement amount: $\Omega_a = \frac{b_o h}{k} + \frac{\Delta M}{c\sigma_a} - \frac{N}{\sigma_a}$

The compressed reinforcement amount: $\Omega'_a = \frac{\Delta M}{c\sigma'_a}$.

3.1.4. Shear stresses:

In any beam subjected to bending will appear, in the plane of neutral axis and in planes parallel to it sliding (shear) stresses. If we consider a prism like in the next figure for the horizontal plane as for the vertical plane there is force $a\tau_o$. The resultant of these forces has the value $a\tau_o\sqrt{2}$ and the direction of the 45^o. The inclined stresses should be taken into account for the sliding assurance, without concrete aid, by means of stirrups or bent-up bars or both.

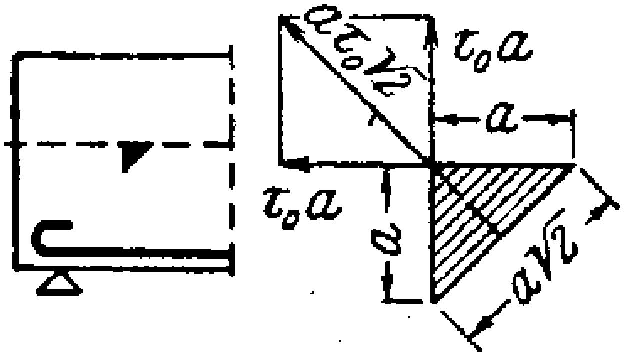


Figure 18: Principal stresses

Dimensioning of the reinforcement for shear efforts:

The inclined tension T45 is the resultant of the tensions in the 45^o inclined surfaces of total length $s = a \cdot \cos 45^o$. Because $\sigma_s = \tau_o$ we have:

$$T_{45} = sb_o\sigma_s = a \cdot \cos 45^0 b_o \tau_o .$$

The sliding stress is: $H = ab_o\tau_o$ so $T_{45} = H \cdot \cos 45^o$.

The relation between the reinforcement area and the shear force is:

$$\frac{T_{45}}{\sigma_a} = \frac{H \cos 45^o}{\sigma_a} = \Omega_{a45} + \Omega_s \cdot \cos 45$$

$$H = \sqrt{2}\sigma_a \Omega_{a45} + \sigma_a \Omega_s = L_{45} + L_s$$

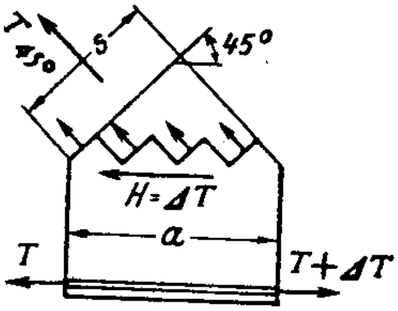


Figure 19: Shear force decomposition

The bent-up bars undertake the L_{45} force of the sliding total force while the other part is taken by the stirrups (L_s).

*I*n order to simplify the computation in the Romanian version of the German recommendations there are some tables.

$$L_{45} = 1.414\sigma_a \cdot \Omega_{a45}$$

$$L_S = \sigma_a \Omega_S = 2f\sigma_a$$

The bent-up bars undertake on the length ν the following efforts:

The bending moment: $\Delta M_{45} = L_{45} \cdot z$

The shear force: $V_{45} = \dfrac{L_{45} \cdot z}{\upsilon}$

The shear stress: $\tau_{45} = \dfrac{L_{45}}{b_o \upsilon}$

And the stirrups with two braces, with a spacing equal to e and area 2*f* on the length ν.

The bending moment: $\Delta M_s = L_s \cdot z$

The shear force: $Vs = \dfrac{L_s \cdot z}{\upsilon}$

The shear stress: $\tau_s = \dfrac{L_s}{b_o \upsilon} = \dfrac{2f\sigma_a}{b_o e}$

The distribution of the stirrups and bent-up bars should be made in such a way that they undertake in any position along the beam the same efforts.

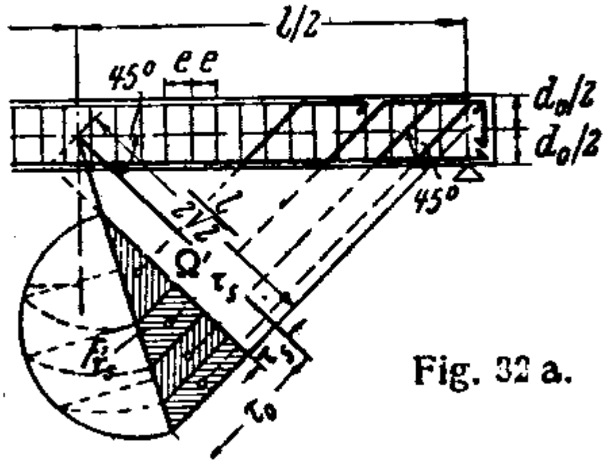


Figure 20: Reinforcement detailing provision

3.2. STAS 1546-50 [3]

The first Romanian reinforced concrete design standard appeared in 1950. It was influenced by the Russian standards and in Romanian is known as a standard with "unique safety coefficient". It contained the characteristics of the material used, (concrete and steel), basic computation principles with the unique safety coefficients, constructive provisions and computation methods.

The concrete was characterized by the concrete mark that was the mean compression strength in kg/cm^2 of concrete cubes with edge of 20cm at the age of 28 days.

Table 26: The design values for different concrete

ACTION	CONCRETE MARK									
	90	**110**	**140**	**170**	**200**	**250**	**300**	**400**	**500**	**600**
Axial compression	72	88	108	125	145	175	200	260	310	350
Axial tension	10	11	13	15	17	20	23	27	31	35
Compression by bending	90	110	135	155	180	220	250	325	390	440

The steel-yielding limit:

For tensioned or compressed reinforcement made of OL38:……2850 kg/cm^2

For tensioned or compressed reinforcement made of OL52:……3500 kg/cm^2.

Basic design rules:

The values for safety coefficients used are given in the following table:

Table 27: The values for safety coefficients

LOADS	$\frac{Q_u}{Q_g}$	FAILURE MODES		
		The concrete reach the compression allowable stress or the reinforcement reach the yielding limit		The concrete reach its tensile allowable stresses
		Columns, arches supports	Other elements	
		c		c_1
Fundamental loads	<2 >2	2,0 2,2	1,8 2,0	2,2 2,4
Fundamental and accidental loads	<2 >2	1,8 2,0	1,6 1,8	2,0 2,2
Fundamental accidental and extraordinary loads	any	1,6	1,5	1,8

3.2.1. The computation of the rectangular section elements under pure bending:

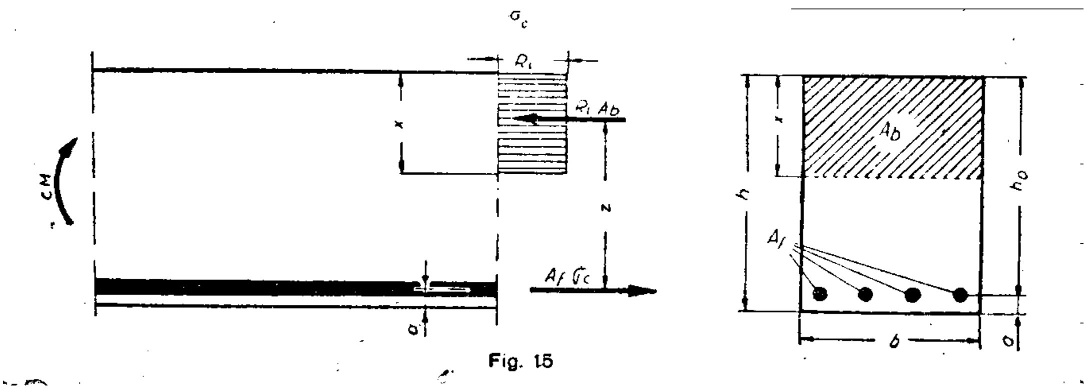


Figure 21: Simple rectangular section under pure bending

$$cM = R_i bx\left(h - \frac{x}{2}\right)$$

$$A_f \sigma_c = R_i bx$$

with the limitation of the height of the compressed zone

$x \leq 0.5h_o$

and of the maximum reinforcement percentage:

$$\mu \leq 0.5\frac{R_i}{\sigma_c}$$

3.2.2. The computation to shear force of the concrete beams:

In the case of the beams, no matter the computation results if there is no compressed reinforcement, we have to provide stirrups at the maximum spacing ¾ h or 50 cm (h-beam height). For the case of the beams with compressed reinforcement taken into consideration in the computation process, the maximum stirrups spacing will be 15 times the compressed reinforcement diameter. The stirrups will be closed and we have to provide supplementary reinforcement so that at least one from two reinforcement bars to be in the stirrups corner.

The shear stress of the stirrups can be computed by the formula:

$$\sigma_{etr.} = \frac{A_e \sigma_c n}{abc}$$

A_e - stirrup brace area

n - number of stirrup braces (sections)

a - stirrup spacing

b - beam width or web width for T shape beam.

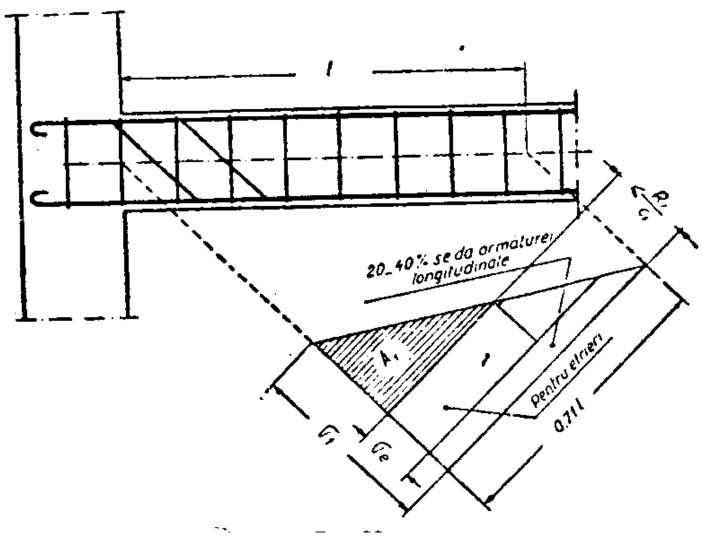


Figure 22: Computation to shear force

3.2.3. Computation of the eccentric compressed members:

According to Romanian standard from 1950 we may have two cases:

Case I of eccentric compression:

$x \le 0.55 h_o$

$$cNe = R_i bx\left(h_o - \frac{x}{2}\right) + A'_f \sigma_c (h - a')$$

$$cN = R_i bx - (A_f - A'_f)\sigma_c$$

with the restriction $x \ge 2a'$

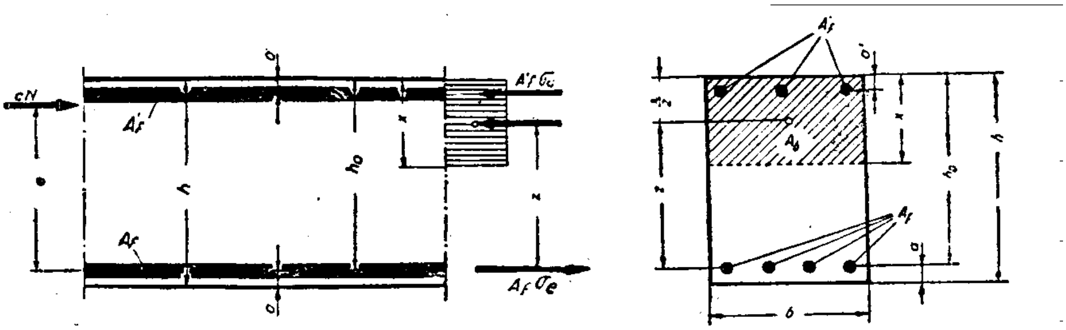


Figure 23: Eccentric compressed reinforced concrete member

Case II

$$x \geq 0.55 h_o$$

$$cNe = 0.5 R_{pr} b h_o^2 + A'_f \sigma_c (h_o - a')$$

$$cNe' = 0.5 R_{pr} b h'^2_o + A_f \sigma_c (h_o - a)$$

3.3. STAS 10107-0/90 [4]

The actual code of practice for reinforced concrete is in force since 1990. Before this it was another version (P8000) with few minor differences. The main difference in STAS 10107-90 is the fact that it deals with concrete class and no with concrete mark. The mark is the medium strength of the concrete while the class is the strength with an non-exceedance probability of 5%.

Table 28: Concrete characteristics according to concrete class

Concrete class	**Bc7.5**	**Bc10**	**Bc15**	**Bc20**	**Bc25**	**Bc30**	**Bc35**	**Bc40**	**Bc40**	**Bc50**
Tensile strength (N/mm2)	0.50	0.60	0.80	0.95	1.10	1.25	1.35	1.45	1.65	1.85
Design strength (N/mm2)	4.7	6.5	9.5	12.5	15.0	18.0	20.5	22.5	26.5	31.5
Elasticity modulus (kN/mm2)	14	21	24	27	30	32.5	34.5	36	38	40

Table 29: Reinforcement characteristics according to steel type

Steel Type	Diameter (mm)	Design strength (N/mm^2)
PC60	6….40	350
PC52	6….28	300
	32…40	290
OB37	6….40	210

The main requirements for reinforced concrete columns detailing are:

For longitudinal reinforcement:

- For efficient confinement in end regions, column bars should be reasonably closely spaced around the periphery. Bars should not be farther apart than 200mm center to center;
- Minimum bar diameters – 12mm for PC60 or PC52;
- Maximal recommended diameters – 28mm;
- Minimum distance between bar - 50mm;
- Maximum distance between bars axes – 250mm;
- 4 cornered bars reinforcement is admitted just for columns with $b \le 350mm$
- Reinforcement ratio should be less than 2.5%.

For transversal reinforcement:

- Maximum distance between stirrups $a_e = 15d \le 200mm$ (d – is the minimal diameter of longitudinal reinforcement
- Maximum distances between stirrups along the height of the column on the hinge region should be considered $\begin{cases} a_e \le 6d \\ a_e \le h/5 \\ a_e \ge 100mm \end{cases}$
- l_p The height of hinge region should be considered the maximum value between $\begin{cases} l_p \ge H_{column}/6 \\ l_p \ge h \\ l_p \ge 600mm \end{cases}$
- Minimal diameter of stirrups is considered $1/4$ from the maximal diameter of longitudinal reinforcement but no less than 8mm.

A short time after the apparition of the standard a guide book [8] was published having the same authors as the standard. This book is very used in common design because it has algorithms for every important problems that may appear.

The most important algorithms are inserted below with the following notations:

b	section width
h	section height
A_a	tensioned reinforcement area
A_a’	compressed reinforcement area
A_e	stirrup brace reinforcement area
a_e	stirrups spacing
R_c	concrete compressive design strength
R_a	reinforcement design strength
R_t	concrete tensile design strength
M_{cap}	section capable bending moment
N	axial force in column
Q	shear force in beam/column.

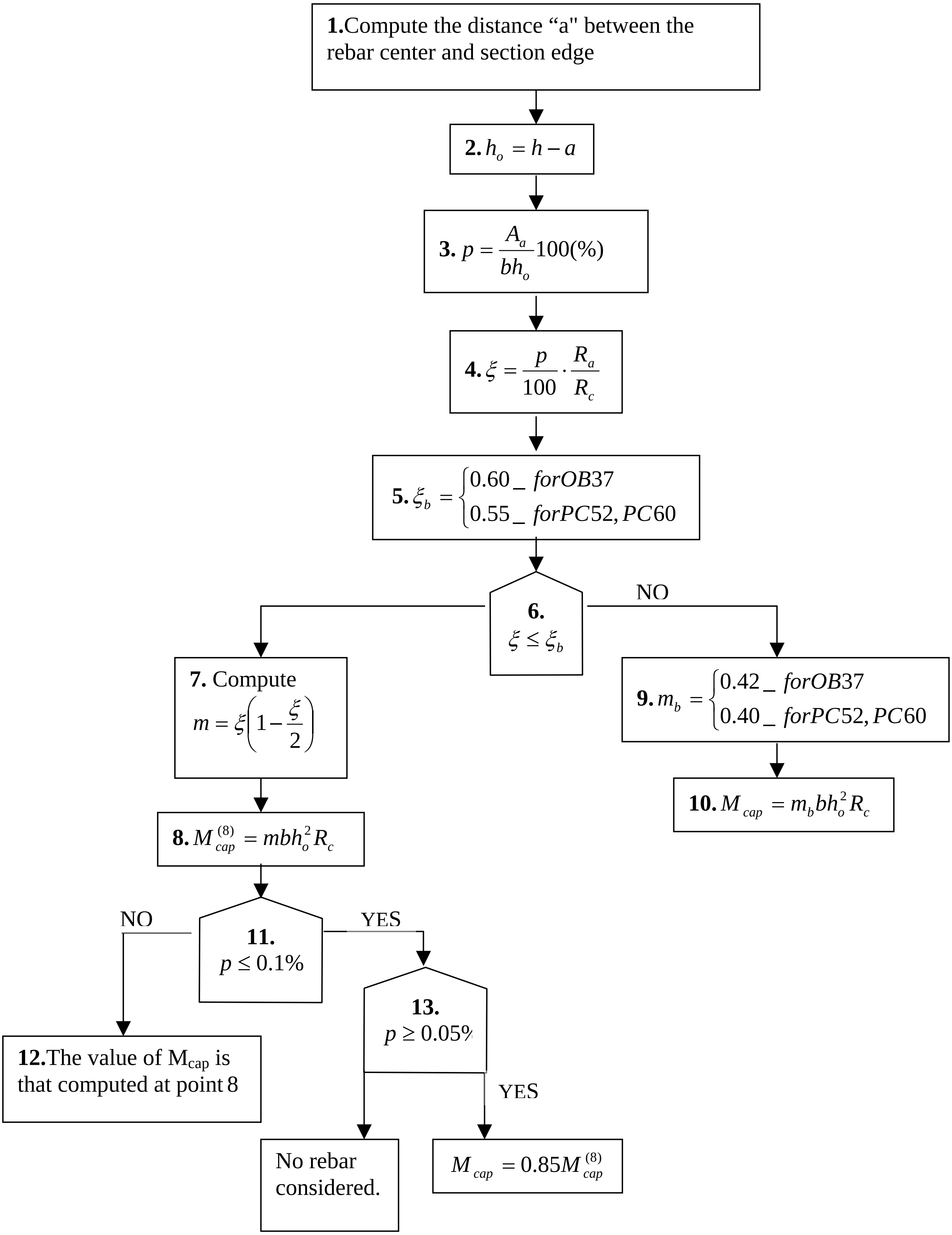


Figure 24: Computation of the capable bending moment for a simple reinforced concrete section under pure bending. We know b, h, A_a, R_c, R_a and we are interested in M_{cap}

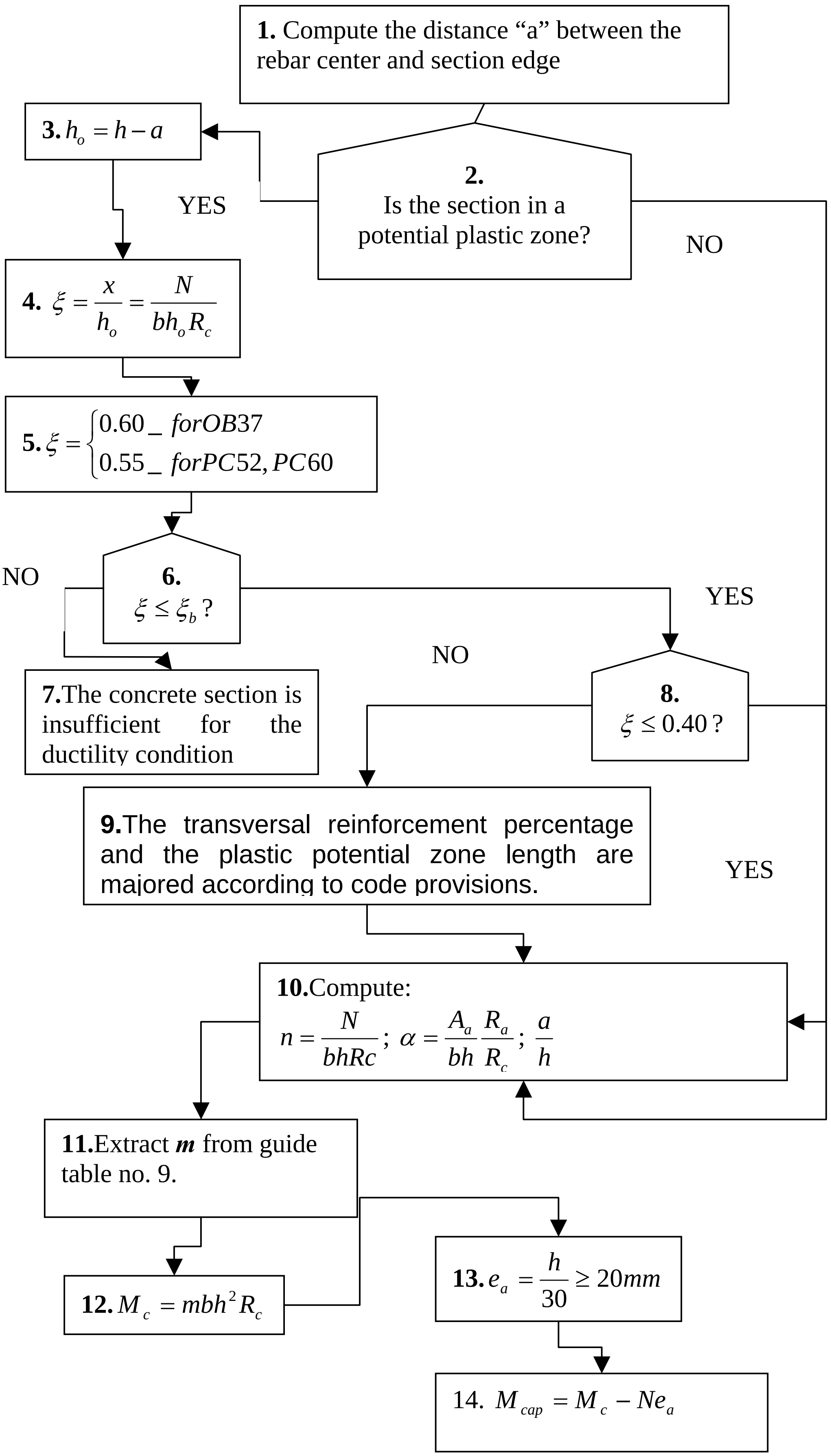
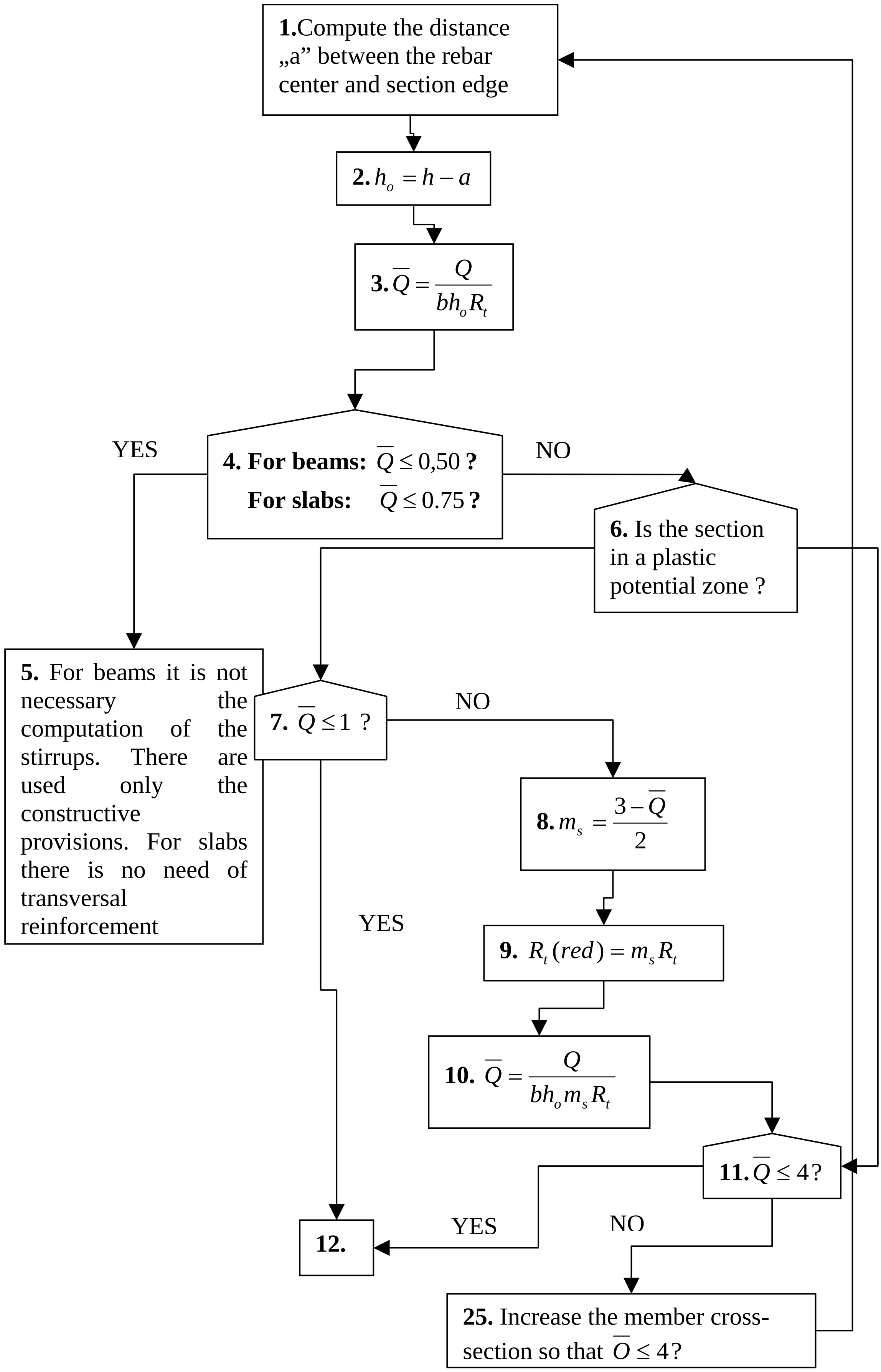


Figure 25: Computation of the bending capacity for a column section with eccentricity. We know b,h,Rc,Ra,N,Aa=Aa’ and we are interested in Mcap



1.Compute the distance „a" between the rebar center and section edge
2. $h_o = h - a$
3. $\overline{Q} = \frac{Q}{bh_o R_t}$
YES
4. For beams: $\overline{Q} \le 0{,}50$? For slabs: $\overline{Q} \le 0.75$?
NO
6. Is the section in a plastic potential zone ?
5. For beams it is not necessary the computation of the stirrups. There are used only the constructive provisions. For slabs there is no need of transversal reinforcement
7. $\overline{Q} \le 1$?
NO
8. $m_s = \frac{3-\overline{Q}}{2}$
YES
9. $R_t(red) = m_s R_t$
10. $\overline{Q} = \frac{Q}{bh_o m_s R_t}$
11. $\overline{Q} \le 4$?
YES
NO
12.
25. Increase the member cross-section so that $\overline{O} \le 4$?

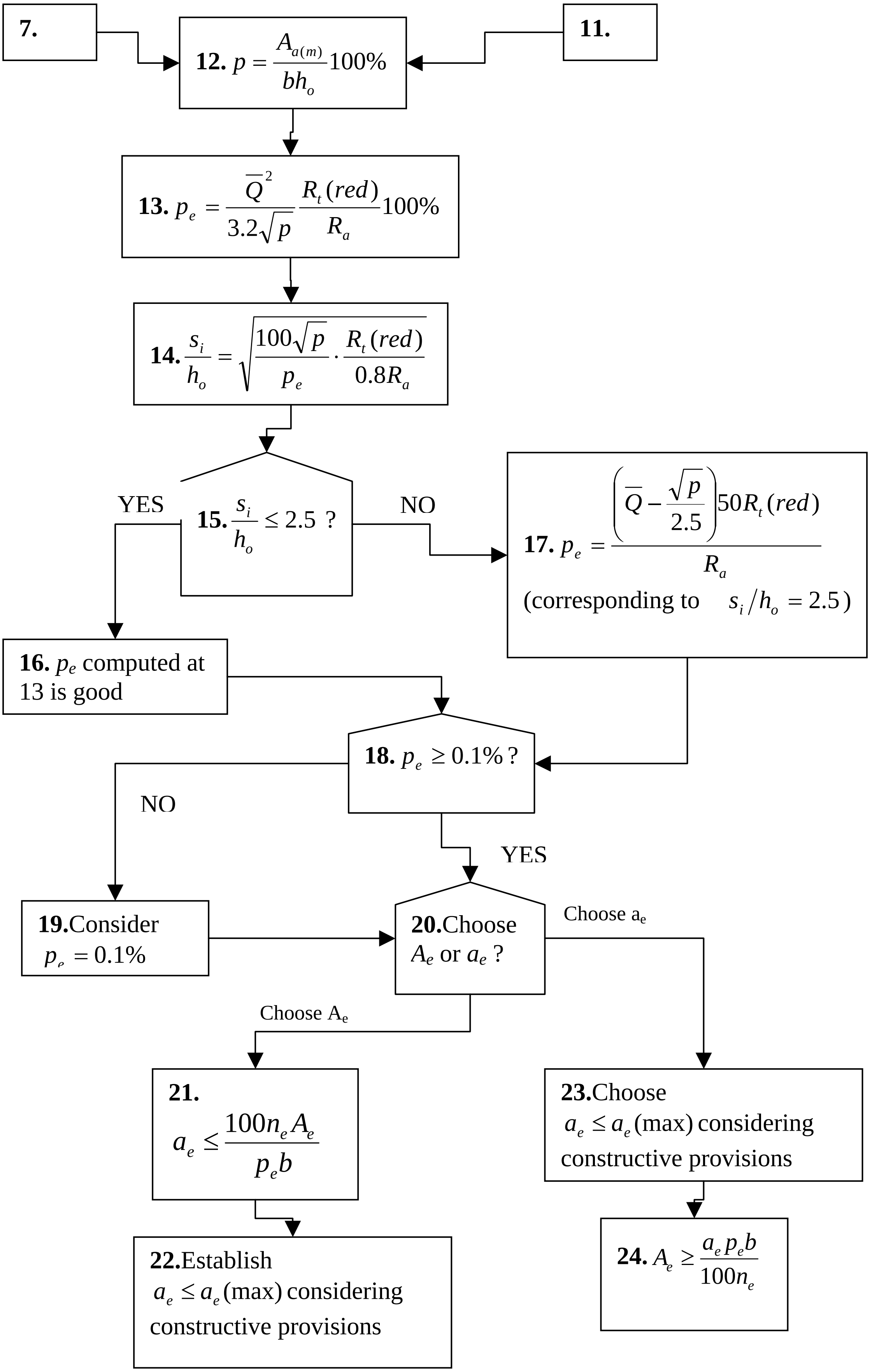
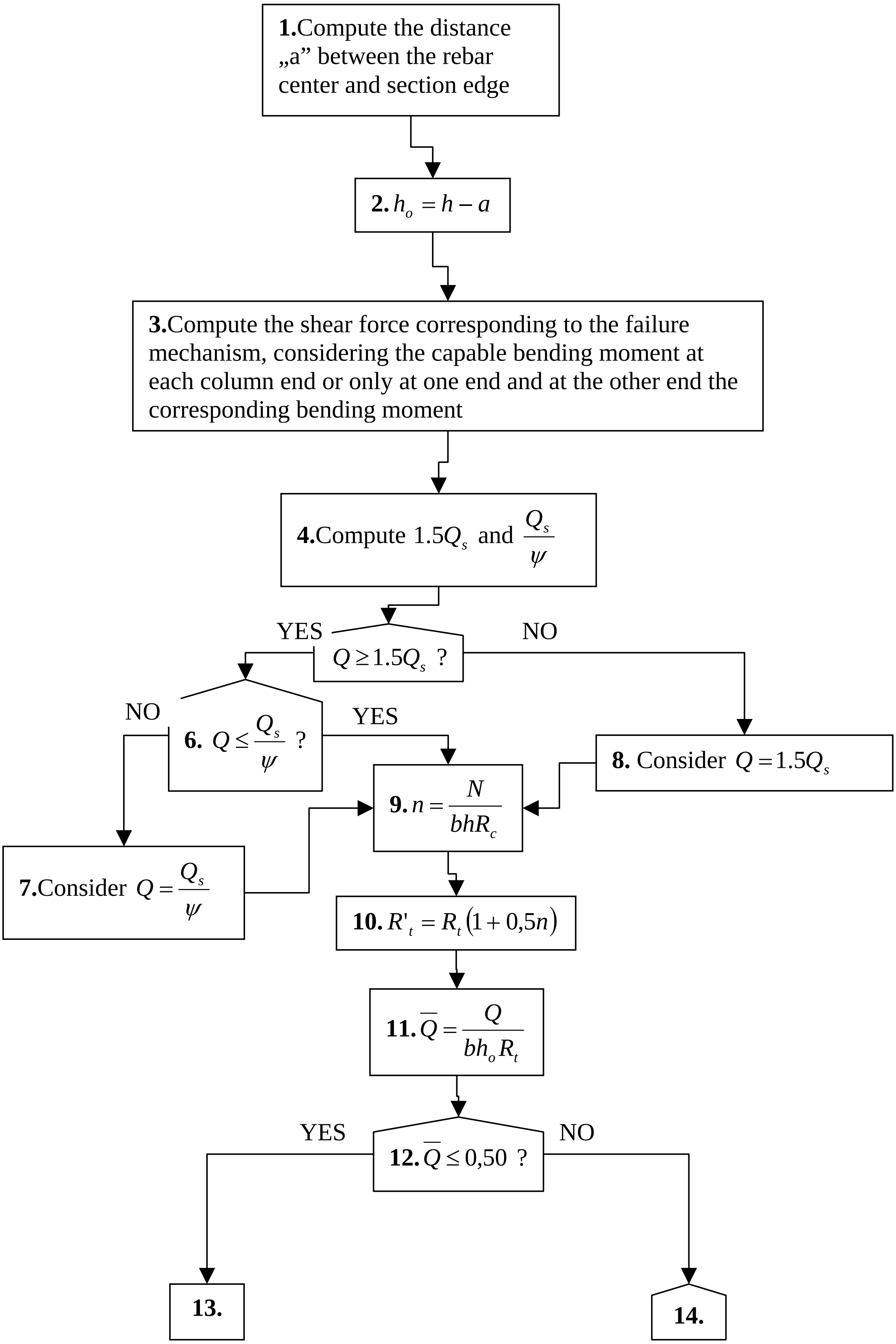


Figure 26: Computation of the stirrups for an element without axial force: We know b,h,Rt,Ra,Q,Aa and we are interested in Ae, ae.


1.Compute the distance „a” between the rebar center and section edge
2. $h_o = h - a$
3.Compute the shear force corresponding to the failure mechanism, considering the capable bending moment at each column end or only at one end and at the other end the corresponding bending moment
4.Compute $1.5Q_s$ and $\frac{Q_s}{\psi}$
YES
NO
$Q \geq 1.5Q_s$?
NO
6. $Q \leq \frac{Q_s}{\psi}$?
YES
8. Consider $Q = 1.5Q_s$
9. $n = \frac{N}{bhR_c}$
7.Consider $Q = \frac{Q_s}{\psi}$
10. $R'_t = R_t(1 + 0{,}5n)$
11. $\overline{Q} = \frac{Q}{bh_o R_t}$
YES
NO
12. $\overline{Q} \leq 0{,}50$?
13.
14.

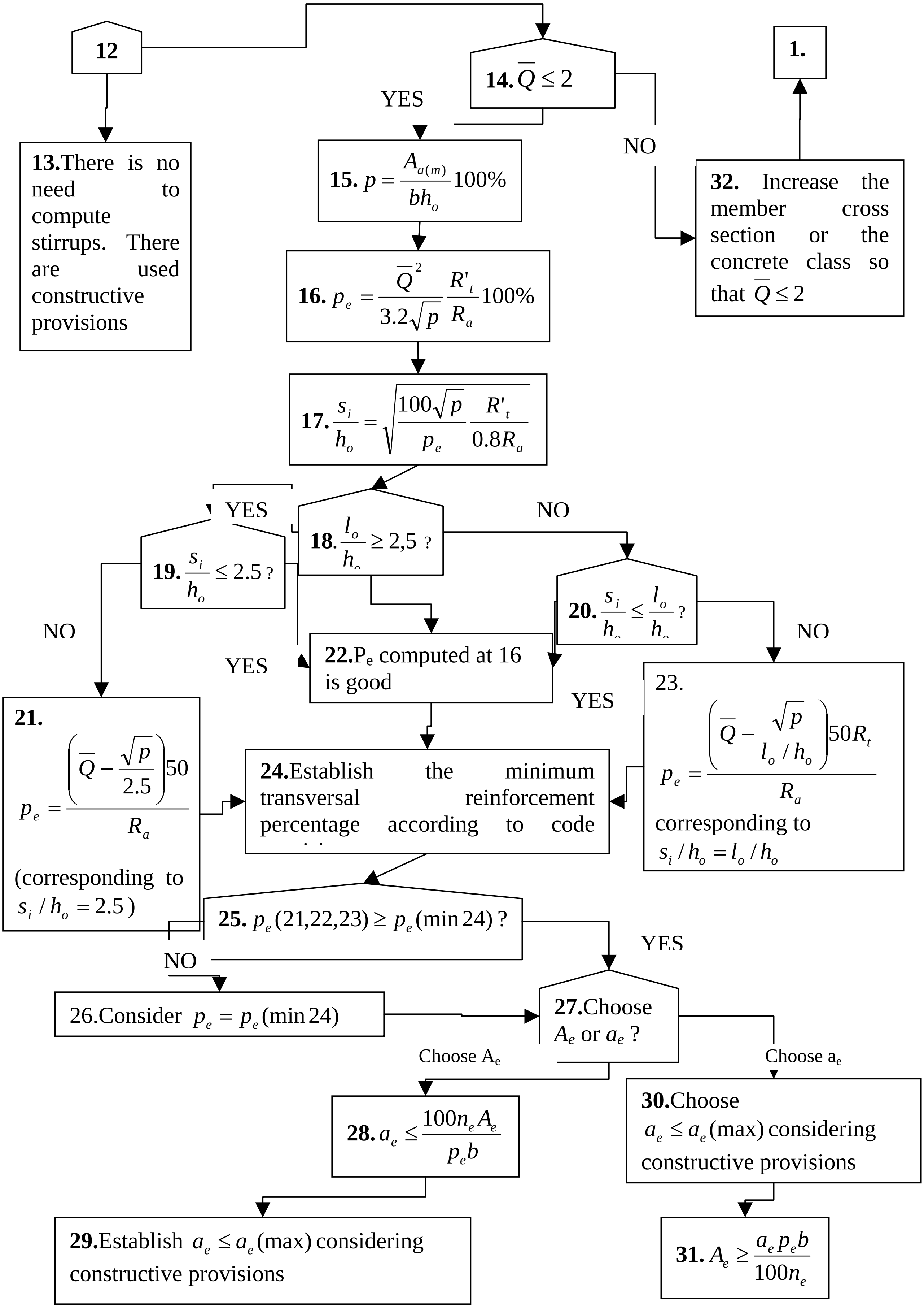


Figure 27: Computation of the stirrups for an element with axial force: We know b,h,Rt,Ra,N,Q,Aa and we are interested in Ae, ae.

4. THE EVOLUTION OF THE MASONRY STRUCTURES DESIGN CODES

The only recommendations that existed before 1975 were in the form of a table with the masonry wall thickness for different types buildings, assumption based on a maximum allowable soil pressure of 2.5kg/cm^2.

Table 30: Masonry wall width according to storey and other considerations [5]

STOREY NAME	RESIDENTIAL BUILDINGS				INDUSTRIAL BUILDINGS			
	Exterior walls with openings supporting floor beams	Interior walls with openings supporting floor beams	High wall without openings but supporting floor beams	Staircase walls	Exterior walls with openings supporting floor beams	Interior walls with openings supporting floor beams	High wall without openings but supporting floor beams	Staircase walls
Basement	0.84	0.56	0.70	0.42	0.98	0.70	0.84	0.56
Ground floor	0.70	0.56	0.56	0.42	0.84	0.56	0.70	0.42
1st story	0.56	0.42	0.56	0.42	0.70	0.56	0.56	0.42
2nd story	0.56	0.42	0.42	0.28	0.56	0.42	0.56	0.42
3rd story	0.42	0.42	0.42	0.28	0.56	0.42	0.42	0.28
4th story	0.42	0.42	0.42	0.28	0.42	0.42	0.42	0.28
Attic	0.42		0.28	0.28	0.42		0.28	0.28

The actual code of practice for masonry structure design is in force from 1985 [7]and is a revision of the code published in 1975[6].

The assumptions for the computations are:

- The floors are considered infinitely rigid in order to be able to consider the loads redistribution, after an element or other elements have reach the ultimate bearing capacity and works in post elastic range.
- By load redistribution the elements with strength reserves are also loaded
- The differences between the real behavior and design behavior are corrected by a working conditions coefficient
- The immersed reinforced concrete elements works together with masonry and also affects its strength by a working condition m_z=0.85 base of the materials different stiffness.
- For walls with openings for doors and windows the favorable spandrel effect is taken into account supposing the section with zero bending moment due to horizontal loads in the middle of the spandrel.

Modeling:

Each element out of the spatial structure is considered a vertical cantilever, fixed at the bottom, load with horizontal forces, and connected at each floor.

The plane dimensions of the structural masonry walls are established on each principal dimension, taking into consideration the active width for T and L section flange.

The horizontal connection can be pinned connection (for slabs without spandrel beam) ar fixed connection for spandrel beam.

The basic relation:

The computation for capacity design of the structure is made with the aid of the following formula:

$$\eta \sum_{k=n}^{k=j} s_k \leq m \sum_{i=1}^{i=t} T_{cij} \quad \text{where:}$$

S_k horizontal seismic force al story k;

T_{cij} minimum shear force capacity of structural wall I at story j; $(\min(T_{CM}, T_{cf}, T_{cp}))$

η load coefficient that takes into account the torsion effect

m working condition coefficient according to the following table

n number of stories

t total number of structural walls on the design considered direction

Table 31: The values of the working coefficient m

SEISMIC ACTION	SLAB TYPE	RATIO $\frac{B(width)}{L(length)}$	m COEFFICIENT
Longitudinal direction	Cast in place, precast	$B \leq L$	0,8
Transversal direction	Cast in place, or equivalently precast	$> 0{,}20$	0,8
	Panels connected, overcast in place strips	$\leq 0{,}20$	0,8
	Precast panels made of narrow strips	$\leq 0{,}20$	0,65
		0.25	0,67
		0,30	0,68
		0,35	0,70
		0,40	0,72
		0,45	0,73
		0,50	0,75

The shear capacity for eccentric compression for masonry structural walls without considering the spandrel beam effect is:

- For simple masonry:

$$T_{CM} = \frac{M_c}{Z} = \frac{Ne_o}{Z} = \frac{1.25RS_c}{Z}$$

T_{CM} shear capacity of the masonry structural wall "i" at story k.

M_C bending capacity of the structural masonry structural wall at the considered story

N maximum axial load for the masonry structural wall at the considered story

Z vertical distance from the design section to the application point of the horizontal seismic force above considered story

R masonry compression strength

S_c static moment of the compressed zone with respect to the axis that cross the section centroid.

$$S_c = A_c e_o$$

$$A_c = \frac{N}{1{,}25R}$$

- For confined masonry with concrete columns

$$T_{CM} = \frac{Mc}{Z} = \frac{1}{Z}\left(R_b S_b + RS_z - NY_N\right) \text{ where:}$$

R_b concrete compressive strength

S_b static moment of the concrete compressed zone with respect to the centroid of tensioned reinforcement

S_z the same, for the compressed masonry

Y_N distance between the centroid of the tensioned reinforcement to the axis that passes through the application point of the gravity force N.

$$A_c = A_z + A_b$$

$$A_z = \frac{N + A_a R_a - A_b R_b}{R}\,;\, S_z = A_z Y_z\,;\, S_b = A_b Y_b$$

The shear capacity considering sliding of the masonry in horizontal joints is computed with the aid of the following formula:

- For rectangular sections with simple masonry:

  If $e_o = \frac{T_{CM} N}{N} \leq \frac{1}{6} l$ then $T_{ef} = \frac{b \cdot l}{1.5}\left(R_f + 0.7 f\sigma_o\right)$

  If $e_o = \frac{T_{CM} N}{N} > \frac{1}{6} l$ then $T_{ef} = \frac{0.7 fN}{1.5}$ where:

$b \cdot l$ wall section area

R_f shear design masonry strength

f friction coefficient

σ_o mean compression stress $\sigma_o = \frac{N}{A}$

- For rectangular sections with reinforced masonry:

  $T_{ef} = 0{,}5 f \left(N + A_a R_a\right) + \left(\sum A_{ai} - A_a\right) R_a$ where:

  A_{ai} sum of the vertical bars area from all the columns in the considered wall

  A_a sum of the vertical bars area from the tensioned side column

  R_a reinforcement design strength

  The computation is done considering the both cases, reinforced and simple masonry and from these cases the biggest value should be considered.

The shear capacity of the structural masonry wall considering tensile stresses in the wall:

- For rectangular sections with simple masonry

$$T_{cp} = \frac{R_p \cdot b \cdot l}{\mu_i} \sqrt{1 + 0{,}8\Phi \frac{\sigma_o}{R_p}}$$

  R_p the design tensile strength of the masonry obtained by multiplying the standard value with a coefficient $m = 1.2$ when it is subjected to seismic action

  Φ the multiplying coefficient that depends on relative eccentricity l / e_o .

Table 32: The values of Φ according to relative eccentricity

RELATIVE ECCENTRICITY $\frac{l}{e_o}$	Φ
≥ 6	1
$6 \geq \frac{l}{e_o} \geq 4$	$2\left(\frac{l}{4e_o} - 1\right)$
≤ 4	0

where $\frac{l}{e_o} = \frac{N}{T_{CP} \cdot Z}$

- For reinforced masonry when the tensile stress is taken by the masonry wall we have to replace the mean axial stress σ_o with the formula:

$$\sigma_o = \frac{N}{A_z + \frac{R_b}{R} A_b}$$ and μ_i with

$\mu_i = \frac{S_{id}}{I_{id}} \cdot l_i$ where S_{id} and I_{id} are the characteristics of the equivalent masonry section.

- For reinforced masonry when the tensile stresses are not taken by the masonry wall because it is cracked and the whole tensile stress should be taken by the girder beam reinforcement and joint reinforcement the formulas are:

For girder beam and joint reinforcement:

$$T_{CP} = \frac{2A_{ac}R_a}{\mu_i} \cdot \frac{l_i}{h_{et}} > T_{CP}$$ for simple masonry.

For column reinforcement:

$$T_{CP} = \frac{2\sum A_{as}R_a}{\mu_i} \frac{l_i}{\sum l_1} > T_{CP}$$ for simple masonry where:

A_{ac} the reinforcement area from girder beam designed to undertake only seismic action and the total reinforcement area from masonry joints.

$\sum A_{as}$ the reinforcement area from column from the interior of the wall (not end column reinforcement)

$\sum l_i$ sum of the afferent walls length corresponding to column

5. SHORT DESCRIPTION OF THE ATTACHED BUILDINGS PLANS:

5.1. Reinforced concrete structures

5.1.1. RC moment resisting frame structure built in 1963: RC S1

This building was efected in the year 1963 in Constanta. This building has been considered because at the design time there were no differences in design for buildings build in different regions. The only rule used for designing to earthquake action was to consider a base shear coefficient equal to 0.05. The destination of the building was a hospital and nowadays it is still functioning as a hospital. The Romanian engineers are trying at this moment to retrofit this building. The most important problem that can be seen if you analyze the plans is that of the short anchorage length and the good point for this building is the regular shape of it. The masonry partition walls are anchored using 6 mm diameter reinforcement.

5.1.2. RC moment resisting frame structure RC S2

This building was designed in the year 2000 according to the provisions of the Romanian seismic code of practice P100/92 and Romanian reinforced concrete design standard 10107/0-90. At the ground floor the building can be enclosed by a rectangle with dimensions 17,20x14,20. The elevation of the building is made of six levels, namely basement, ground floor and other four stories. The story height is situated in between 2,775 to 4,75 with the most common value of 3,15m. The space planning of the building allows it to be used as a first destination as office and apartment building.

The structural system is made of reinforced concrete frames, on the two orthogonal directions. The partition walls were not considered as structural walls. The slabs and the stairs are made of cast in place reinforced concrete.

The foundation system is made of a reinforced mat foundation and of shallow continuous foundation under basement extension. The mat foundation is cast in place over a layer of 15 cm thickness of compacted gravel and of 10 cm of plain concrete slab. The basement walls are made of cast in place reinforced concrete. The allowable soil pressure given in the geotechnical study has the values of 250kPa for short action loads.

The exterior (closure) walls 25cm width are made of bricks with vertical holes of while the interior (partition) walls has 12.5 or 25 cm width.

The designer guaranties the maximum story drift of 3.5‰.

5.2. Masonry structures

5.2.1. Masonry structure - M S1

The building is the headquarter of the Ministry of Agriculture. It was built in between 1884 and 1887. It is a great building with great spans and higher story heights. The plane shape is a U-shape, one of the side wing extending behind the central zone.

The plane dimensions of these zones are of 47.00x17.80m for central zone, 50.00x13.00m for one wing and 65.00x16.00 for the other wing.

The height regime is basement, ground floor, two stories and attic. The different story heights that varies from 3.50m at the basement to5.5m (maximum story height) for the ground floor.

The structural system is made of masonry structural walls, with floor made of steel beams and masonry domes or concrete at inferior profile flange and ribbon-iron ties.

Figure 28: View of the main façade of the M S1 building, “Central” wing

Figure 29: View of the main façade of the M S1 building, “Slaninc” wing

Figure 30: View of the main façade of the M S1 building, "Bratianu" wing

5.2.2. Masonry structure - M S2

This building will be built this year in Bucharest. It is a residential one family building and it is design according to the actual codes of practice. The elevation of the building is made of four stories, namely partial basement, ground floor and two stories. The plans that we provide you are not the final plans because it might be few minor changes because of the architectural solution. The partition walls and enclosure walls are connected with the main structural elements according to code provisions. The story height is around 3 meters. The slabs are made of reinforced concrete. The basement walls are made of cast in place reinforced concrete. The foundation system is made of continuous foundation under partial basement and under ground floor walls.

Because the designer does not want to take chances, and because the Romanian masonry structures design codes is in force since 1985 when the capacity design was not well known, the design choose to increase the column section so that it becomes a strong column, weak beam system.

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Figure 31: Rendered image of the M S2 building (main façade)

Figure 32: Rendered image of the M S2 building (side view)

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