Country Report WEFO-UNESCO

ROMANIA

# OUTLINE OF THE SEISMIC DESIGN OF WALL STRUCTURES IN ROMANIA (MASONRY AND REINFORCED CONCRETE)

DRAFT REPORT

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## 1. Seismicity of Romania

Romania is an earthquake prone country. Most of its territory (of app. 250000 $km^2$) is subjected to intermediatte depth epicenter earthquakes from Vrancea region. Other shallow epicenters are located in south-west (Banat region) or in the central part of the country (Fagaras region).

Vrancea earthquakes strongly affects more than 2/3 of the Romanian territory. Entire territory of the Republic of Moldavia and part of Ukraine and Bulgaria are also affected by these earthquakes.

The history of the Romanian major earthquakes have been synthesized in two catalogs by Radu and Constantin/Marza. According to the Radu catalog the most important earthquake events in the last century are:

- 1940, November 10 – 7.4 Gutenberg – Richter magnitude
- 1977, March 4 – 7.2 Gutenberg – Richter magnitude
- 1986. August 30 – 7.0 Gutenberg – Richter magnitude

According to Radu's catalog 2 earthquakes with a magnitude larger than 7,2 can be expected each century.

The earthquake of 1940 caused in between 500 and 1500 human losses. During the earthquake of 1977, 1570 people lost their life and 11321 have been wounded. 90% of the human losses were recorded in the capital city, Bucharest. Here, 31 buildings collapsed. 28 of them were designed and built before 1940 and 3 were design based on the seismic regulations developed in the '60s. All over country 32900 housing units have been severely damaged or collapsed and over 35000 families remained homeless. It is estimated that the earthquake of 1977 caused economic losses in excess of 2 billion dollars.

**Figure 1. Collapsed masonry building in Bucharest**

**2. History of Seismic Building Regulations in Romania:**

Four generations of earthquake resistance design codes can be identified in Romania.

1) Pre-code period
   i) Prior to 1945 – no code.

      However the severe damages for masonry and confined masonry structures in Bucharest caused by the 1940 Vrancea earthquake showed the importance of seismic design regulations.

   ii) 1941, Draft Instructions for earthquake resistant design of buildings and retrofitting of damaged buildings – enforced, but not compulsory
   iii) 1945, Instructions for earthquake resistant design of buildings – enforced, but not compulsory

2) Low-code period
   a) P13/1963, Code for seismic design of buildings and industrial facilities, enforced and compulsory
   b) P13/1970 (revision of P13/1963) Code for seismic design of buildings and industrial facilities, enforced and compulsory

3) Moderate-code period – the need for changes in the design codes was emphasized by the observations made after the earthquake of March 4, 1977 (Vrancea subcrustal earthquake)
   a) P100/1978 Code for seismic design of buildings and industrial facilities– enforced and compulsory;
   b) P100/1981 (revision of P100/1978) Code for seismic design of buildings and industrial facilities – enforced and compulsory;

4) Moderate to High-code period
   a) P100/1990 Earthquake resistant design code – enforced and compulsory;
   b) P100/1992 (revision of P100/1990) Earthquake resistant design code – enforced and compulsory;
   c) P100-1/2006 Seismic design code for buildings - enforced and compulsory, in line with Eurocode 8 Part 1 provisions.

## 3. Romanian data on building damage due to 1977 strong earthquake

According to the data published in *NBS* Special Publication 490, *Observation on the behavior of buildings in the Romanian earthquake of March 4, 1977*, Washington, based on statistics on casualties and property damage compiled as of the end of April 1977, the 1977 earthquake:

- destroyed or seriously damaged 33000 housing units in high-rise apartment flats and conventional type dwellings
- caused lesser damage to 182 000 other dwellings
- destroyed 374 kindergartens, nurseries, and schools and badly damaged 1,992 others
- destroyed 6 university buildings and damaged 60 others
- destroyed one orphanage and damaged 15 others
- destroyed 11 hospitals and damaged 2288 others hospitals and 220 polyclinics (health care centers)
- damaged almost 400 cultural institutions (theatre, museums, etc.)
- damaged 763 factories.

The following Romanian data on building damage are from *The Romanian Earthquake on March 4, 1977 – Balan St, Cristescu V, Cornea I – coordinators, 1982* and from *Report to the 8th European Conference on Earthquake Engineering, 1986 - Annex IV – Some data on vulnerability obtained in European countries* completed by Working Group *Vulnerability and Risk Analysis for individual structures and systems* of *European Association of Earthquake Engineering*.

Damage survey was performed on a sample of 18000 buildings after Vrancea earthquake of March 4, 1977. The survey performed on the basis of individual forms filled in on the site by students from Technical University of Civil Engineering of Bucharest. The site activity was accomplished in the period April 4-8 and April 25-28, 1977. Residential, schools and hotel buildings were included in the sample.

The Romanian results, based on the survey carried out in Bucharest after the earthquake of 1977, are expressed in terms of damage grade, *DG* histograms conditional upon intensity. The *DG* was quantified according to the adapted *MSK* methodology in Bucharest. The intensity was expressed in terms of *MSK* intensity related to the sub-sample of buildings corresponding to different spectral intervals. Results for Bucharest are given for eight classes of buildings:

*A1* – low quality material buildings

*A2* – pre-1940 masonry buildings with flexible floors

*A3* - post-1940 masonry buildings with flexible floors

*A4* – pre-1940 masonry buildings with rigid floors

*A5* - post-1940 masonry buildings with rigid floors

*A6* – RC frame structures

*A7* – high-rise buildings with RC structural walls closely spaced

*A8* – high-rise buildings with RC structural walls widely spaced

The data on building damage in Bucharest due to March 4, 1977 Vrancea earthquake are presented in the following tables. The building damage is expressed as the mean and standard deviation of the damage grade, *DG* considered for a given seismic intensity and for a given building typology.

**Table 1: Building damage data due to March 4, 1977 Vrancea earthquake**

| | *A1* | | | *A2* | | |
|---|---|---|---|---|---|---|
| Intensity | # of bldgs. | Mean *DG* | Stdev *DG* | # of bldgs. | Mean *DG* | Stdev *DG* |
| 6.5 | 804 | 1.25 | 0.71 | 1094 | 1.03 | 0.42 |
| 7 | 2697 | 1.49 | 0.84 | 2398 | 1.07 | 0.81 |
| 7.5 | 1278 | 1.88 | 0.92 | 2299 | 1.82 | 0.62 |
| 8 | 205 | 2.06 | 0.82 | 1641 | 2.12 | 0.61 |

**Table 2: Building damage data due to March 4, 1977 Vrancea earthquake (cont.)**

| | *A3* | | | *A4* | | |
|---|---|---|---|---|---|---|
| Intensity | # of bldgs. | Mean *DG* | Stdev *DG* | # of bldgs. | Mean *DG* | Stdev *DG* |
| 6.5 | 587 | 1.03 | 0.37 | 167 | 1.1 | 0.69 |
| 7 | 1293 | 1.21 | 0.49 | 604 | 1.27 | 0.56 |
| 7.5 | 581 | 1.4 | 0.7 | 500 | 1.31 | 0.62 |
| 8 | 89 | 1.93 | 1.07 | 417 | 1.74 | 0.59 |

**Table 3: Building damage data due to March 4, 1977 Vrancea earthquake (cont.)**

| | *A5* | | | *A6* | | |
|---|---|---|---|---|---|---|
| Intensity | # of bldgs. | Mean *DG* | Stdev *DG* | # of bldgs. | Mean *DG* | Stdev *DG* |
| 6.5 | 135 | 1.13 | 0.63 | 140 | 1.26 | 0.8 |
| 7 | 722 | 1.28 | 0.96 | 216 | 1.3 | 0.72 |
| 7.5 | 330 | 1.46 | 0.75 | 196 | 1.51 | 0.85 |
| 8 | 114 | 1.87 | 1.02 | 141 | 2.14 | 1.21 |

**Table 4: Building damage data due to March 4, 1977 Vrancea earthquake (cont.)**

| | *A7* | | | *A8* | | |
|---|---|---|---|---|---|---|
| Intensity | # of bldgs. | Mean *DG* | Stdev *DG* | # of bldgs. | Mean *DG* | Stdev *DG* |
| 6.5 | 31 | 1.62 | 0.67 | 32 | 1.79 | 0.79 |
| 7 | 64 | 1.79 | 0.79 | 191 | 2.03 | 0.74 |
| 7.5 | 131 | 2.13 | 0.81 | 82 | 2.14 | 0.78 |
| 8 | 37 | 2.86 | 0.82 | 45 | 2.47 | 0.84 |

**4. Romanian seismic design code P100-1/2006**

The current Romanian seismic design code have been issued in 2006. It follows the Eurocode 8 (EN1998-1:2004) format and concepts.

Romanian seismic design codes requires verifications at two limits states:

- Serviceability Limit State, SLS
- Ultimate Limit State, ULS

The main objectives of the seimic design at SLS is to limit the deterioration of the nonstructural elements and to avoid any damages in the structural members during non-severe earthquakes, having characterized by short mean return intervals. According to the code, this can be achieved by:

- Limiting the lateral displacement of the building under the SLS earthquake
- Asuring enough structural strength to obtain an essentially elastic structural response under the SLS earthquake

The main objectives for seismic design at ULS is to prevent the loss of human lives or severe wounding of the oupants and people in the proximity of the building during severe earhquakes. Therefore, it is necessary to limit the deterioration of the structural elements (to maintan the stability of the structure under gravity loads and to asure that the building is economically repairable after the earhquake) and to prevent the total damage of the nonstructural elements.

According to the code, this can be achieved by:

- Limiting the lateral displacement of the building under the ULS earthquake
- Asuring enough structural strength to limit the nonlinear lateral displacement demand under the ULS earthquake.
- Asuring a good ductile response of the structure

The structural design provision of P100-1/2006 are in accordance with the capacity design method concepts.

The mean return interval of the ground motion corresponding to the Ultimate Limit State is 100years. Design acceleration values, $a_g$, associated with this earthquake are situated between 0,08g and 0,32g.

Romanian deep Vrancea epicenter earthquakes are characterized by long predominant periods, $T_c$. According to P100-1-2006, $T_c$ values of 0,7s, 1,0s and 1,6s should be considered when the design acceleration spectrum is constructed.

Therefore, a dynamic amplification factor of 2,75 shall be selected for most of low to medium rise buildings (having the fundamental vibration periods smaller than the predominant period of the ground motion).

According to the Romanian Seismic Design Code for Buildings P100-1/2006 the following equation is to be used to determine the design seismic force for new buildings:

$$F_b = \gamma_I S_d(T_1) m \lambda$$

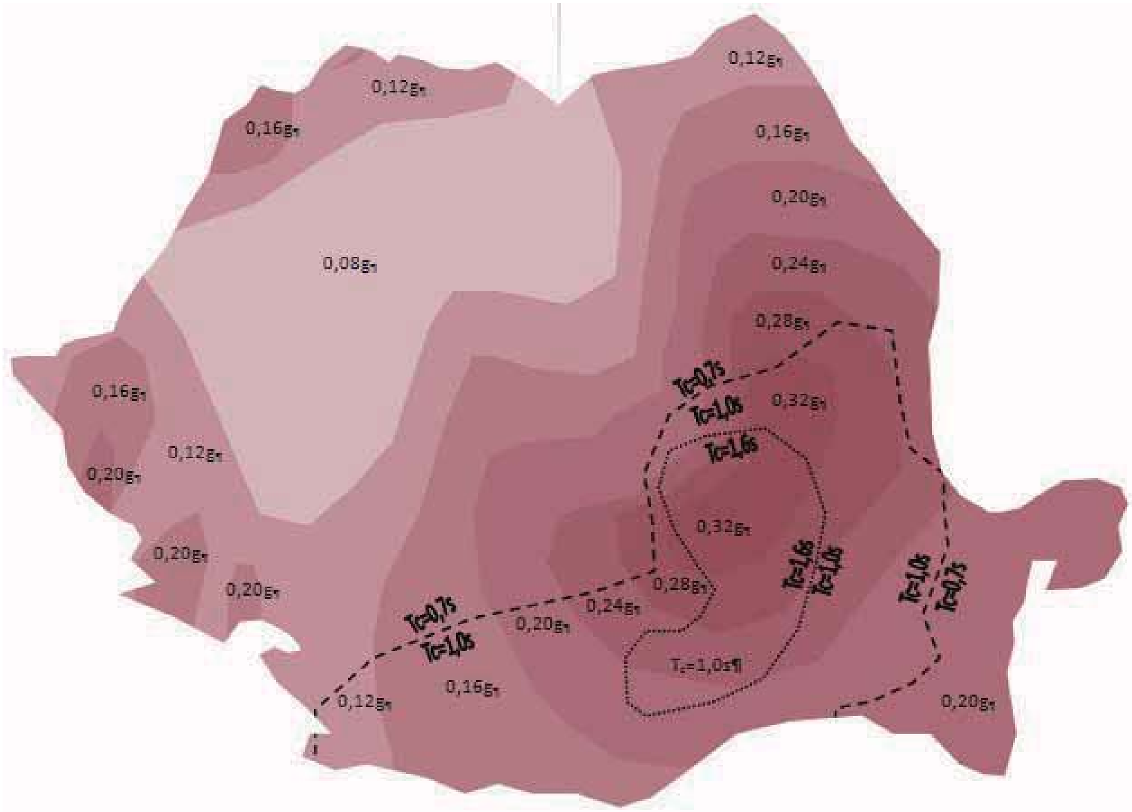


**Figure 2. Design acceleration values and predominant periods of the ground motions for Romanian teritory**

where,

$\gamma_I$ importance factor of the building ranging from 1 to 1,4 according the the importance class. For regular buildings $\gamma_I = 1$, while for temporary or low importance buildings, 0,8 can be considered.

m total mass of the building

$\lambda$ is the SDOF (single degree of freedom period)-MDOF (multi degree of freedom period) equivalence factor. This can be usually approximated to 0,85 for buildings with more than 1 bay and 1 story.

$S_d(T_1)$ design acceleration spectrum ordinate at the fundamental period of the building

$$S_d = a_g \frac{\beta(T)}{q}$$

$a_g$ horizontal acceleration of the ground with values ranging from 0,08 to 0,32g, depending on the seismic region as defined in the code

$\beta(T)$ value of the normalized dynamic amplification factor ranging from 2,75 to 1, depending on the structural dynamic characteristic and predominant period of the ground motion. This factor is estimated in the code considering a critical damping ratio $\xi_0 = 5\%$.

q is the behavior factor ranging from 1 to 6,75 which accounts mainly for the structural ductility and redundancy. Large (favorable) q factors are from 5..6,75 while small ones are between 1 and 2,5).

The factors in these equations have similar significance with those prescribed by EN1998-1.

## 5. Outline of the Romanian codes for seismic design of masonry wall structures

### 5.1. Relevant codes

1) CR6:2006 – Masonry structures design code

2) P100-1/2006 – Seismic design code for buildings

3) EN1996-1-1:2004 - Eurocode 6: Design of masonry structures, General rules for reinforced and unreinforced masonry structures.

4) EN1998-1:2004 - Design of structures for earthquake resistance - Part 1: General rules, seismic actions and rules for buildings

P100-1/2006 prescribes design rules for the following types of masonry structures:

- unreinforced masonry (URM)
- confined masonry (CM)
- confined masonry, reinforced in the horizontal joints (CHRM)
- reinforced masonry (RM)

The provisions of this code cannot be applied for masonry walls with brittle force displacement behavior (for which the ultimate strain is approximately equal with the strain at maximum strength).

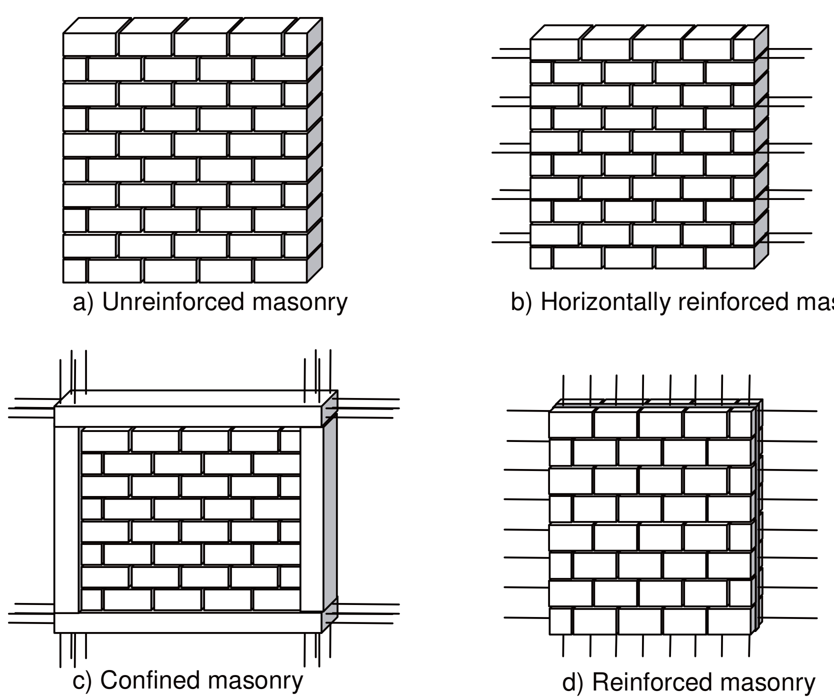


**Figure 3. Masonry types according to Romanian code**

## 5.2. Seismic load

For masonry structures the values of the behavior factors q shall be chosen based on the masonry type, structural regularity and expected over strength.

**Table 5: q factors for masonry buildings**

| Structural regularity | | Behavior factor q | | | |
|---|---|---|---|---|---|
| Plan | Elevation | URM | CM | CHRM | RM |
| Yes | Yes | 2.2 | 3.125 | 3.75 | 4.375 |
| No | Yes | 2.2 | 3.125 | 3.75 | 4.375 |
| Yes | No | 1.925 | 2.5 | 3.125 | 3.75 |
| No | No | 1.65 | 2.1875 | 2.5 | 3.125 |

The horizontal seismic load for masonry walls structures can be calculated considering a higher critical damping ration ξ=8%. This leads to a reduction of the acceleration design spectrum by:

$$\eta = \sqrt{\frac{10}{5+\xi}} = 0{,}88 \geq 0{,}55$$

$$S_e(T)_{\xi=8\%} = 0{,}88 S_e(T)_{\xi_0=5\%}$$

Considering all these, the seismic coefficient $c = \frac{\gamma_I S_d(T_1)\lambda}{g}$ for the city of Bucharest ($a_g$=0,24g, β=2,75, λ=1, $\gamma_I$=1, $\eta = 0{,}88$) yields to the following values:

**Table 6:**

| Structural regularity | | Seismic coefficient | | | |
|---|---|---|---|---|---|
| Plan | Elevation | URM | CM | CHRM | RM |
| Yes | Yes | 0.26 | 0.19 | 0.15 | 0.13 |
| No | Yes | 0.26 | 0.19 | 0.15 | 0.13 |
| Yes | No | 0.30 | 0.23 | 0.19 | 0.15 |
| No | No | 0.35 | 0.27 | 0.23 | 0.19 |

### 5.3. Design criteria:

According to the Romanian code the following structural characteristics should be checked:

- Walls Strength
- Structural Stiffness
- Stability

Ductility is to be achieved by implementing various detailing rules and by preventing brittle failure of the structural members. Should be noted here that, recognizing that masonry is a low ductility structural material, low values of the behavior factors q are prescribed by the design.

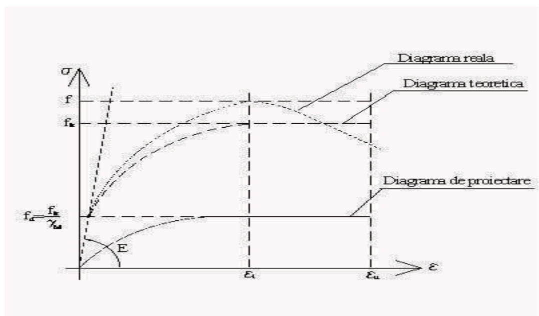


**Figure 4. Strain – stress relations for masonry**

### 5.4. General requirements

The strength in compression for structural masonry panels should be at least equal to $f_b$=7,5N/mm$^2$ for loads applied normal to the horizontal joint and $f_{bh}$=2,0N/mm$^2$ for loads applied parallel to the horizontal joint.

The story number, above the base section ($n_{niv}$), is limited based on the design acceleration value ($a_g$), the structural irregularities, the building importance class, the masonry type and the masonry units type.

For unreinforced masonry panels the story number is limited according to the following table based on the structural walls density in each direction and the design acceleration value.

**Table 7: Structural walls density according to the total stories number, $n_{niv}$, and design acceleration, $a_g$ for unreinforced masonry buildings**

| $n_{niv}$ | Design acceleration $a_g$ | | | |
|---|---|---|---|---|
| | 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 |

However it is recommended to avoid the use of unreinforced masonry due to the lack of ductility, low energy dissipation capacity and brittle failure type.

For reinforced or confined masonry structures the story number is limited according to the following table based on the structural walls density in each direction and the design acceleration value.

**Table 8: Structural walls density according to the total stories number, $n_{niv}$, and design acceleration, $a_g$ for confined and reinforced masonry buildings**

| $n_{niv}$ | Design acceleration $a_g$ | | | |
|---|---|---|---|---|
| | 0.08g,0.12g | 0.16g,0.20g | 0.24g | 0.28g,0.32g |
| 1 | ≥ 3% | ≥ 4% | ≥ 4% | ≥ 4% |
| 2 | ≥3% | ≥4% | ≥5% | ≥ 6% |
| 3 | ≥4% | ≥5% | ≥6% | NA |
| 4 | ≥4% | ≥6% | NA | NA |
| 5 | ≥5% | NA | NA | NA |

The necessary area of structural walls in each direction shall be established by structural design. These values shall not be less than those specified in table 1.3 and 1.4.

The structural masonry walls thickness, t, shall not be less than 240mm. Moreover, t shall be established according to the story height, $h_{et}$, as follows:

- for unreinforced masonry walls - $t > h_{et}/12$
- for confined or reinforced masonry $t > h_{et}/15$.

The ratio between the area of the openings (doors and windows) and the total area of a masonry structural wall shall be limited according to the following table, based on the seismic design acceleration, $a_g$, story number, $n_{niv}$, and location of the masonry wall.

**Table 9: Ratio of the openings according to the stories number, $n_{niv}$, design acceleration and location of the wall**

| Design acceleration $a_g$ | 0.08g | 0.12g, 0.16g | 0.20g, 0.24g | 0.28g, 0.32g |
|---|---|---|---|---|
| Exterior walls | $n_{niv} \le 3$<br>$\rho \le 1.5$ | $n_{niv} \le 3$<br>$\rho \le 1.25$ | $\rho \le 1.00$ | $\rho \le 0.80$ |
| | $n_{niv} = 4,5$<br>$\rho \le 1.25$ | $n_{niv} = 4$<br>$\rho \le 1.00$ | | |
| Interior walls | $n_{niv} \le 3$<br>$\rho \le 0.55$ | $n_{niv} \le 3$<br>$\rho \le 0.45$ | $\rho \le 0.35$ | $\rho \le 0.25$ |
| | $n_{niv} = 4,5$<br>$\rho \le 0.45$ | $n_{niv} = 4$<br>$\rho \le 0.35$ | | |

Undeformable slabs for horizontal loads are recommended in all situations. Deformable slabs are accepted only for buildings with less than 3 stories being placed in seismic regions with $a_g \leq 0,08$. The slab over the basement is not included and shall be undeformable, in any situation.

### 5.5. Strength requirements

For all structural members the following condition shall be met:

$$R > E$$

R resistance (bending, shear, compression)

E force under gravity and seismic loads

Regarding the bending capacity the following checking equation applies:

$$M_{Rd} \geq M_{Ed}$$

$M_{Rd}$ bending capacity of the masonry panel calculated considering the axial force $N_{Ed}$ resulted from the elastic analysis of the structure under gravity and seismic loading

$M_{Ed}$ design bending moment resulted from the elastic analysis of the structure under gravity and seismic loading

The shear capacity shall be larger than the design shear force corresponding to the bending strength of the masonry panel

$$V_{Rd} \geq 1.25 V_{Edu}$$

where,

$V_{Rd}$ shear strength of the masonry panel

$V_{Edu}$ the design shear forces corresponding to the bending strength of the masonry panel. For members with high bending over strength (bending capacity higher than q times the design bending moment, $M_{Rd} \geq qM_{Ed}$) the design shear force shall be limited to the value corresponding to an elastic response of the entire structure:

$$V_{Edu} \leq qV_{Ed}$$

where

q behavior factor

$V_{Ed}$ shear force resulting from elastic analysis of the building under gravity and seismic loads

### 5.5.1. Bending capacity of Unreinforced Masonry Walls:

To determine the bending capacity of URM walls the following assumptions shall be considered:

- Bernoulli hypothesis
- Tensile strength of masonry is to be neglected
- For the ultimate limit state, the compressive stress in the masonry are uniformly distributed in compressed area (rectangular distribution)

If the masonry wall is subjected only to forces from gravity loads the bending capacity shall be calculated as follows.

The compressed area of the masonry element shall be calculated considering a uniform compressive stress of $0.8f_{cd}$:

$$A_{zc} = \frac{N_{Sd}}{0.8 f_d}$$

Based on this area and on the configuration of the transversal section of the masonry wall, the distance between the center weight of the compressed area and the center weight of the transversal section, $y_{zc}$, can be calculated.

Using this distance the bending capacity can be found as:

$$M_{Rd} = N_{Sd} y_{zc}$$

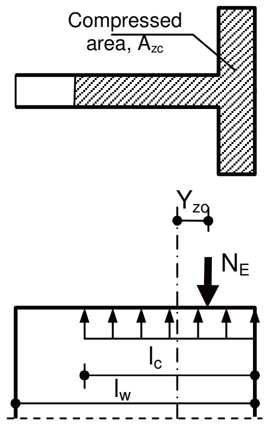


**Figure 5. Bending capacity under gravity loads for unreinforced masonry**

If the masonry wall is subjected to forces from seismic loads the bending moment shall be calculated under the condition that:

$y_{zc} < 1.2\ r_{sc}$

The significance of $r_{sc}$ is presented in the figure. This means that the eccentricity of the axial load is severely limited. However, cracking of the masonry on the tension side is allowed.

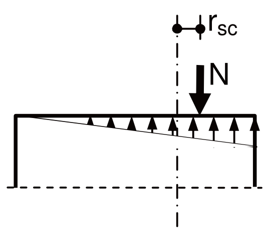


**Figure 6. Bending capacity under seismic loads for unreinforced masonry**

For example, for a rectangular section masonry wall the bending capacity can be calculated as follows:

$r_{sc}=l_w/6$

$y_{zc} < 0{,}2l_w$

$M_{Rd}=0{,}2l_wN_{Ed}$

### 5.5.2. Bending capacity of Confined Masonry Walls:

The bending capacity of confined masonry walls can be calculated based on Bernoulli hypothesis if the strain-stress relations are known for concrete, masonry and steel. The following assumptions can be made:

- Tensile strength of the concrete is neglected
- Tensile strength of the mortar in the horizontal joints is neglected
- Intermediate confining elements can be neglected
- Balanced failure strain distribution
    - The crushing strain is attained in the most compressed fiber (concrete or masonry)
    - The yielding strength is attained in the tensile reinforcement

Uniform stress distribution can be considered for compressed concrete and masonry.

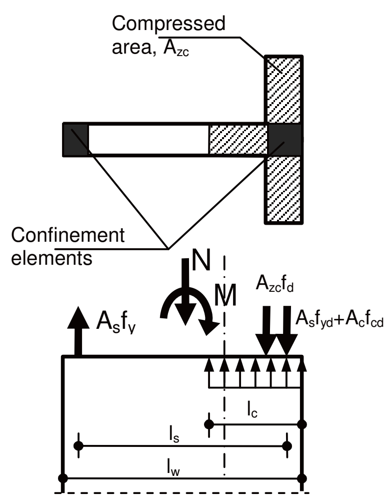


**Figure 7. Bending capacity for confined masory**

In a simplified approach the bending capacity of a confined masonry element can be calculated as follows:

$$M_{Rd} = M_{Rd(URM)} + M_{Rd\,(As)}$$

$M_{Rd(URM)}$ bending capacity of the equivalent unreinforced masonry element. The concrete confining elements are taken into account by considering an equivalent area of masonry $n_{equiv}$ times larger than the area of each confining element, $A_{ce}$:

$$n_{equiv} = 0.75\frac{f_{cd}}{f_d}$$

where,

$f_{cd}$ concrete compressive strength

$f_d$ masonry compressive strength

The total area of the masonry yields to:

$$A_{zi} = A_z + n_{equiv} A_{ce}$$

Considering this, the bending moment $M_{Rd(URM)}$ can be calculated as explained for the unreinforced masonry element using the following equations:

$$A_{zci} = \frac{N_{Ed}}{0.8 f_d}$$

$$M_{Rd(URM)} = y_{zci} N_{Ed}$$

Where,

$y_{zci}$ distance between the center weight of the equivalent compressed area and the center weight of the equivalent transversal section

The contribution of the reinforcement to the bending moment of the wall is accounted by:

$$M_{Rd(As)} = A_s f_{yd} l_s$$

Where,

$l_s$ distance between the centroids of the longitudinal reinforcement in the boundary confining elements

$A_s$ total area of longitudinal reinforcement in each boundary element

$f_{yd}$ yielding strength of the longitudinal reinforcement

To expect the yielding of the longitudinal tensile reinforcement prior to the failure of the masonry in the compressed area, the longitudinal reinforcement in the confining elements shall be limited to 50% of the area necessary to obtain balanced failure (simultaneous yielding of the tensile reinforcement and crushing of the compressed masonry).

### 5.5.3. Shear capacity of unreinforced masonry wall

The shear capacity of the unreinforced masonry elements shall be computed considering a uniform distribution of the shear stresses in the compressed area of the transversal section.

$$V_{Rd} = f_{vd} t l_c$$

Where:

$f_{vd}$ allowable shear stress

$t$ masonry panel web thickness

$l_c$ length of the compressed area as results from the bending analysis under the design forces $M_{Ed}$, $N_{Ed}$

$$f_{vd} = m_z \frac{f_{vk}}{\gamma_M}$$

$m_z = 0{,}75$ for masonry elements built with cement mortar

$\gamma_m = 2{,}2$ partial safety factor for the Ultimate Limit State for mortars prepared in quality controlled conditions ($\gamma_m = 2{,}5$ for mortars prepared on site)

$f_{vk}$ characteristic shear strength of masonry

$$f_{vk} = f_{vk,0} + 0{,}4\sigma_d \le 0{,}9(0{,}034 f_b + 0{,}14\sigma_d)$$

$f_{vk0}$ initial shear strength of masonry

$\sigma_d$ mean compressive stress in the masonry panel

$f_b$ standardized masonry strength

$$f_b = \delta f_{med}$$

**Table 10:**

| | Mean strength of the mortar $f_m$ (N/mm$^2$) | | |
|---|---|---|---|
| | M10 | M5, M2.5 | M1 |
| Ceramic tiles | 0.30 | 0.20 | 0.10 |
| Concrete tiles | 0.20 | 0.15 | 0.10 |
| AAC | --- | 0.15 | 0.10 |

**Table 11: Standardized shear strength of masonry, $f_b$, and transformation factor, δ, for different types of masonry unit available in Romania**

| Type of masonry unit | δ | $f_{med}$ (N/mm$^2$) | |
|---|---|---|---|
| | | 10 | 7.5 |
| Solid ceramic bricks - 240x115x63 mm | 0.81 | 8.1 | 6.1 |
| Hollow ceramic units 240x115x88 mm, 290x240x138 mm | 0.92 | 9.2 | 6.9 |
| Hollow ceramic units 240x115x88 - 240x115x138 mm | 1.12 | 11.2 | 8.4 |
| Hollow ceramic units 240x115x88 - 290x140x88 mm | 0.87 | 8.7 | 6.5 |
| Hollow ceramic units 240x115x88 - 290x140x138 mm<br>290x240x188 mm<br>Concrete units - 290x240x188 mm | 1.07 | 10.7 | 8.0 |

**Table 12: : Characteristic shear strength of masonry $f_{vk}$**

| $f_b$ N/mm$^2$ | Mortar type | Mean compressive stress $\sigma_d$ (N/mm$^2$) | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | | 0.1 | 0.2 | 0.3 | 0.4 | 0.5 | 0.6 | 0.7 | 0.8 | 0.9 | 1.0 |
| 10.0 | M10 | 0.340 | 0.368 | 0.382 | 0.396 | 0.410 | 0.424 | 0.438 | 0.452 | 0.466 | 0.480 |
| | M5/2.5 | 0.240 | 0.280 | 0.320 | 0.360 | 0.400 | | | | | |
| | M1 | 0.140 | 0.180 | 0.220 | 0.260 | 0.300 | 0.340 | 0.380 | 0.420 | 0.460 | |
| 7.5 | M10 | 0.269 | 0.283 | 0.297 | 0.311 | 0.325 | 0.339 | 0.353 | 0.367 | 0.381 | 0.395 |
| | M5/2.5 | 0.240 | 0.280 | | | | | | | | |
| | M1 | 0.140 | 0.180 | 0.220 | 0.260 | 0.300 | | | | | |
| 5.0 | M5/2.5 | 0.184 | 0.198 | 0.212 | 0.226 | 0.240 | 0.254 | 0.268 | 0.282 | 0.296 | 0.310 |
| | M1 | 0.140 | 0.180 | | | | | | | | |

### 5.5.4. Shear capacity of confined masonry walls

The shear capacity of confined masonry walls can be computed by:

$$V_{Rd} = V_{Rd1} + V_{Rd2}$$

$V_{Rd1}$ shear capacity of the masonry panel which can be calculated as previously presented

$V_{Rd2}$ contribution of the confining elements to the shear capacity of the panel

$$V_{Rd2} = 0{,}2A_{asc}f_{yd}$$

Where

$A_{asc}$ total area of reinforcement in the compressed boundary element

$f_{yd}$ yielding strength of steel

### 5.5.5. Shear capacity of confined and horizontally reinforced masonry walls

The shear capacity of confined and horizontally reinforced masonry walls can be computed as follows:

$$V_{Rd} = V_{Rd1} + V_{Rd2} + V_{Rd3}$$

Where,

$V_{Rd1}$ shear capacity of the masonry panel which can be calculated using eq. ()

$V_{Rd2}$ contribution of the confining elements to the shear capacity of the panel – can be computed as presented for confined masonry

$V_{Rd3}$ contribution of the horizontal reinforcement to the shear capacity of the panel calculated considering a shear crack inclination of 45°.

$$V_{Rd3} = 0.8l_w \frac{A_{sw}}{s} f_{ysd}$$

$A_{sw}$ total area of one layer of horizontal reinforcement

s spacing of the horizontal reinforcement

$l_w$ length of the web of the masonry panel

$f_{ysd}$ yielding strength of horizontal reinforcement

If the potential shear crack is likely to pass from one story to another ($l_w$>$h_{et}$) than $l_w$ shall be replaced with $h_{et.}$

**5.5.6. Out-of-plane failure bending capacity**

The out-of-plane bending capacity shall be calculated using the following eq.:

$$M_{Rxd1} = W_w f_{xd1}$$

$$M_{Rxd2} = W_w f_{xd2}$$

$$W_w = \frac{1000t^2}{6}$$

Where,

$W_w$ sectional modulus

t wall thickness

$f_{xd1}$, $f_{xd2}$ masonry design tensile strength for out-of-plane bending failure

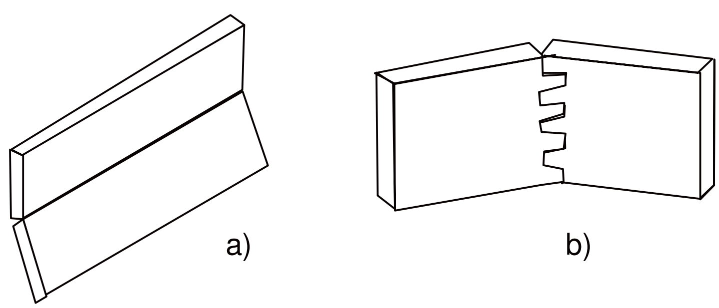


**Figure 8. Out of plane bending failure types: a)x1 type b) x2 type**

$$f_{xd1} = m_z \frac{f_{xk1}}{\gamma_M}$$

$$f_{xd2} = m_z \frac{f_{xk2}}{\gamma_M}$$

$\gamma_m$ partial safety factor (2.2..3.0 depending on the quality of the masonry work)

$m_z$ 0,75

**Table 13: : Masonry design tensile strength for out-of-plane bending failure**

| fxd (N/mm2) | Mean strength of mortar | | | |
|---|---|---|---|---|
| | M10*,M5 | | M2.5 | |
| Masonry units type: | $f_{xd1}$ | $f_{xd2}$ | $f_{xd1}$ | $f_{xd2}$ |
| Regular clay bricks | 0.072 | 0.144 | 0.054 | 0.108 |
| AAC | 0.024 | 0.048 | 0.020 | 0.039 |

**Table 14: : Masonry bending capacity for out-of-plane failure for a 250mm wall**

| Mxd (kNm) | Mean strength of mortar | | | |
|---|---|---|---|---|
| | M10*,M5 | | M2.5 | |
| Masonry units type: | $M_{xd1}$ | $M_{xd2}$ | $M_{xd1}$ | $M_{xd2}$ |
| Regular clay bricks | 0.75 | 1.50 | 0.56 | 1.13 |
| AAC | 0.25 | 0.50 | 0.20 | 0.41 |

# 6. Outline of the Romanian code for seismic design of reinforced concrete wall structures

## 6.1. Relevant codes

The relevant codes for the design of reinforced concrete wall structures in Romania are:

1) CR2-1-1:2005 – Reinforced concrete structural wall design code

2) P100-1/2006 – Seismic design code for buildings

3) EN1992-1-1:2004 - Eurocode 2: Design of concrete structures - Part 1-1: General rules and rules for buildings

4) EN1998-1:2004 - Design of structures for earthquake resistance - Part 1: General rules, seismic actions and rules for buildings

## 6.2. Seismic load

For reinforced concrete walls structures the following behavior factors shall be chosen according to the structural type and intended ductility class.

**Table 15: Behavior factor q for RC wall structures**

| Structural type | *q* | |
|---|---|---|
| | Ductility class, H | Ductility class, M |
| Coupled walls system, dual systems | $5\alpha_u/\alpha_1$ | $3{,}5\ \alpha_u/\alpha_1$ |
| Uncoupled walls system | $4\ \alpha_u/\alpha_1$ | 3,0 |

The ratio $\alpha_u/\alpha_1$ which describes the structural redundancy and overstrength can be chosen as follows:

- Structures with only two walls in each direction - $\alpha_u/\alpha_1 = 1.0$
- Structures with more than two walls in each direction - $\alpha_u/\alpha_1 = 1.15$
- Structures with coupled walls- $\alpha_u/\alpha_1 = 1.25$

For example, for a structure situated in Bucharest the following seismic coefficients can be obtained:

**Table 16: Seismic coeficients for a medium height wall structure situated in Bucharest**

| | Seismic coeficient, c ($a_g$=0,24, β=2,75, γ=1, λ=0,85) | |
|---|---|---|
| Structural type | Ductility class H | Ductility class H |
| Coupled walls | 0.09 | 0.13 |
| Uncoupled walls | 0.11 | 0.15 |

### 6.3. Structural analysis

The structural analysis is usually carried out using computer analysis software.

The cracked reinforced concrete stiffness of the structural members shall be taken into account when the calculation of the member forces is indented. To avoid the laborious calculation of the cracked stiffness for each element, the Romanian code provides simplified equations to roughly determine the stiffness value. The following chart gives the ratio between the cracked and untracked member stiffness for bending ($I_e/I_b$), shear ($A_{fe}/A_{fb}$) and axial force ($A_e/A_b$) as prescribed by the code. This ratio depends on the normalized axial stress acting on member: $\nu=N/A_c f_{cd}$.

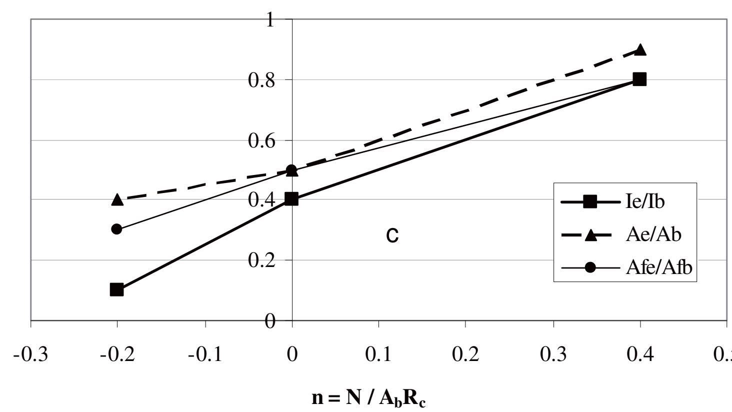


**Figure 9.**

For the coupling beams the following reduction factors are recommended:

- For orthogonally reinforced coupling beams

$I_e$ = 0,4 $I_b$

$A_{fe}$ = 0,4 $A_{bf}$

- For diagonally reinforced coupling beams

$I_e$ = 0,6 $I_b$

$A_{fe}$ = 0,6 $A_{bf}$

A repeated structural analysis process might be necessary in some situations (especially for coupled walls) since this stiffness reduction factors depend on the axial force. In the previous code the following stiffness reduction factor were recommended:

Uncoupled walls:

$I_e$ = 0,7 $I_b$

$A_e$ = 0,7 $A_b$

$A_{fe}$ = 0,5 $A_{fb}$

Coupled walls – the most compressed wall

$I_e$ = 0,8 $I_b$

$A_e$ = 1,0 $A_b$

$A_{fe}$ = 0,7 $A_{fb}$

Coupled walls – less compressed or tensioned wall

$I_e$ = 0,5 $I_b$

$A_e$ = 0,5 $A_b$

$A_{fe}$ = 0,35 $A_{fb}$

Coupled walls – coupling beam

$I_e$ ≈ 0,2-0,3 $I_b$

$A_{fe}$ = 0,3 $A_{bf}$ orthogonal reinforcement

$A_{fe}$ = 0,6 $A_{bf}$ diagonal reinforcement

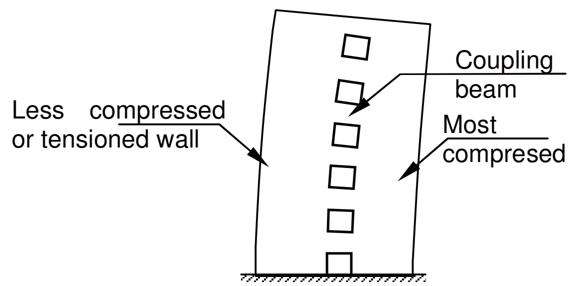


**Figure 10. Coupling walls.**

As stated above $I_b$, $A_b$ and $A_{fb}$ represents the moment of inertia, the area and the shear area of the gross concrete section. Same properties are denoted for the cracked section with $I_e$, $A_e$ and $A_{fe}$. It can be observed that for the tensioned or less compressed coupling walls smaller reduction factors are introduced to account for the small depth of the compressed area.

### 6.4. Design member forces

Capacity design method is used to design the reinforced concrete wall structures.

Therefore, the design member forces are derived from those directly obtained from the elastic structural analysis to assure the formation of the desired plastic mechanism.

In case of uncoupled wall structures, the plastic mechanism implies plastic hinging of the reinforced concrete walls at the base. In case of coupled walls, the plastic mechanism implies also the plastic hinging of all coupling beams at both ends.

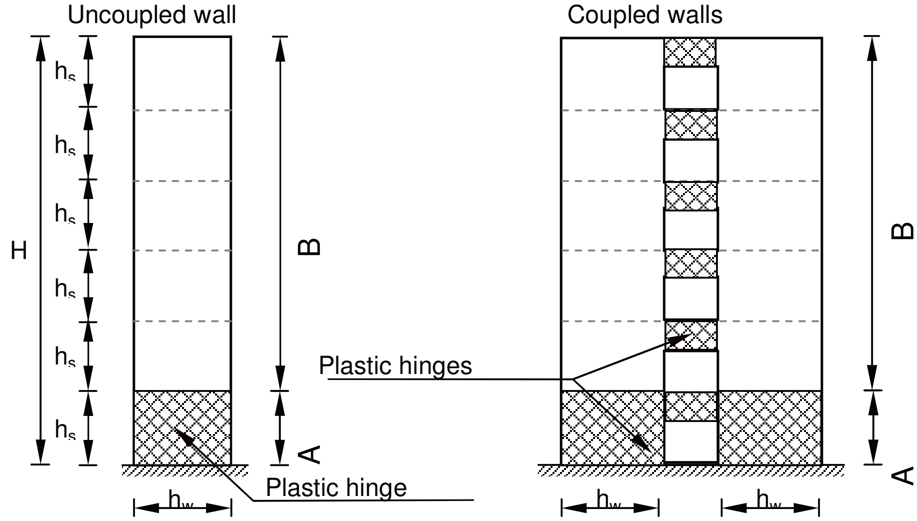


**Figure 11. Plastic hinge mechanism for RC wall structures**

### 6.4.1. Design bending moments:

The design bending moment at the base of the structural wall, $M_{Ed,0}$, is considered to be equal to the moment obtained from the structural analysis under gravity and seismic loads ( the subscript „0" shows that this is the moment at the base).

$$M_{Ed,0} = M'_{Ed,0}$$

Above the plastic hinge length the design moments, $M_{Ed}$, are derived from those obtained from the structural analysis under gravity and seismic loads using amplification factors to account for the overstrength of the plastic hinge:

$$M_{Ed} = \gamma_{Rd} \Omega M'_{Ed}$$

where

$\gamma_{Rd}$ takes into account the uncertainties in the estimation of the bending capacity in the plastic region $\gamma_{Rd} = 1{,}3$

$\Omega$ bending overstrength in the plastic hinge

$M'_{Ed}$ bending moment as resulted from the structural analysis under seismic and gravity loads

The plastic hinge bending overstrength factor, $\Omega$, can be computed by:

$$\Omega = \frac{M_{Rd,0}}{M_{Ed,0}}$$

where

$M_{Rd,0}$ bending capacity of the wall at the base (in the plastic region)

$M_{Ed,0}$ bending moment as resulted from the structural analysis under seismic loading

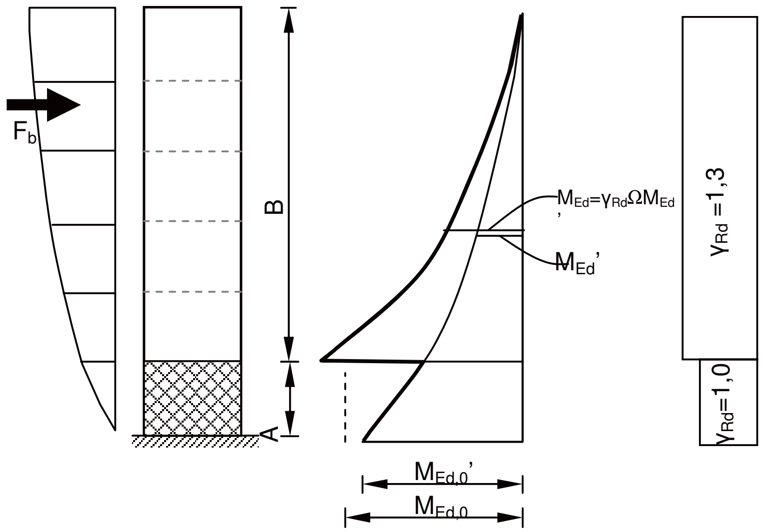


**Figure 12. Design bending moments for uncoupled walls**

The bending capacity of the uncoupled walls, $M_{Rd,0}$, shall be calculated by sectional analysis considering the axial load from structural analysis under gravity and seismic loading. Simplified analysis that takes into account only the longitudinal reinforcement from the extremities of the transversal section of the structural walls is not allowed.

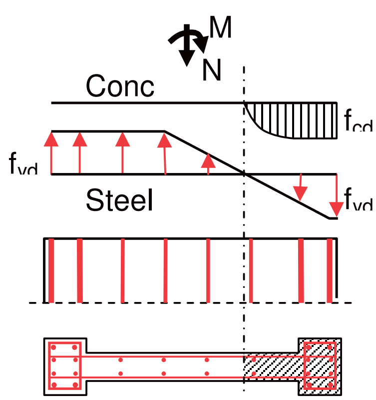


**Figure 13. Strain and stress distribution for concrete and steel**

According to the design code, the plastic hinge length can be calculated using the following equation:

$$l_p = 0,4h + 0,05H$$

where:

$h_w$ length of the transversal section of the wall

H total height of the wall

$l_p$ should be a multiply of the story height ($h_s$). If $l_p$, measured from the base of the wall, goes into a story more than 20% of its height then that story should be all included in the plastic hinge length, otherwise completely excluded.

Same equation can be used to determine the design bending moments for coupled walls as in case of uncoupled walls. However, the calculation of the plastic hinge bending overstrength requires additional comments. The overturning moments (demand or capacity) used to determine the ratio Ω shall be calculated by adding the bending moments at the base of each wall with the overturning moment balanced by the axial forces in the walls.

These forces represents only the efect of the lateral load and the associated overturning moment can be easily calculated by summation of the forces multiplied with the distance to a suitable located point.

The overturning moment as resulted from the structural analysis can be computed by:

$$M_{Ed,0}' = \sum_{j=1}^{n} M_{Ed,0}^{j}' + \sum_{j=1}^{n} N_{Ed,0}^{j}' L^{j}$$

Where,

n total number of walls in a coupled wall system

$M_{Ed,0}^{j}{}'$ bending moment at the base of wall „j” resulted from the structural analysis under design seismic loads

$N_{Ed,0}^{j}{}'$ axial load at the base of wall „j” resulted from the structural analysis under design seismic loads (solely)

$L^{j}$ distance from wall „j” to a suitable located point

Same concept can be applied to estimate the capable overturning moment (at the base of the wall) corresponding to the formation of the structural plastic mechanism:

$$M_{Rd,0} = \sum_{j=1}^{n} M_{Rd,0}^{j} + \sum_{j=1}^{n} N_{Ed,0}^{j} L^{j}$$

Where,

$M_{Rd,0}^{j}$ bending capacity at the base of wall „j”

$N_{Ed,0}^{j}$ axial force at the base of wall „j” corresponding to the formation of the structural plastic mechanism. This force accounts for the shear forces in the coupling beams associated with the plastic hinging.

$$N_{Ed,0}^{j} = \sum_{i=1}^{m} V_{Ed\,i}^{\;j}$$

Where,

m total number of coupling beams for wall „j”

$V_{Edi}^{j}$ shear force acting on wall „j" after hinging of coupling beam „i"

The calculation procedure for the overturning moments for a coupled wall system consisting of two walls linked together by coupling beams is presented in the following:

$$M_{Ed,0} = M_{Ed,0}^{1} + M_{Ed,0}^{2} + N_{Ed,0}^{1}L = M_{Ed,0}^{1} + M_{Ed,0}^{2} + N_{Ed,0}^{2}L$$

$$M_{Rd,0} = M_{Rd,0}^{1} + M_{Rd,0}^{2} + N_{Ed,0}^{1}L = M_{Rd,0}^{1} + M_{Rd,0}^{2} + N_{Ed,0}^{2}L$$

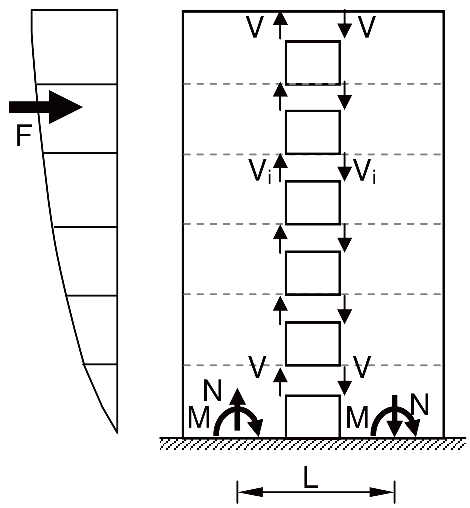


**Figure 14. Bending moments and axial forces for a coupled wall system**

Plastic hinges are expected to appear at both ends of the coupling beams so the design bending moments, $M_{Ed}$, is equal to the one resulting from the structural analysis unde design seismic loads, $M_{Ed}$':

$$M_{Ed} = M'_{Ed}$$

### 6.4.2. Design shear forces

In accordance with capacity design principles, the shear failure of the structural members is not allowed. Therefore, the design shear forces for a given structural member are calculated as the maximum shear forces that can develop in that particular member.

Regarding the RC structural walls (coupled or uncoupled) the following equation is prescribed by the Romanian code to determine the shear forces:

$$V_{Ed} = \gamma_{Rd} \Omega V_{Ed}^{'}$$

Where,

$\gamma_{Rd}$ factor that takes into account the uncertainties in the estimation of the bending capacity in the plastic region $\gamma_{Rd} = 1,2$

Ω plastic hinge bending overstrength factor

$V_{Ed}$' shear force obtained from the structural analysis under design seismic and gravity loads.

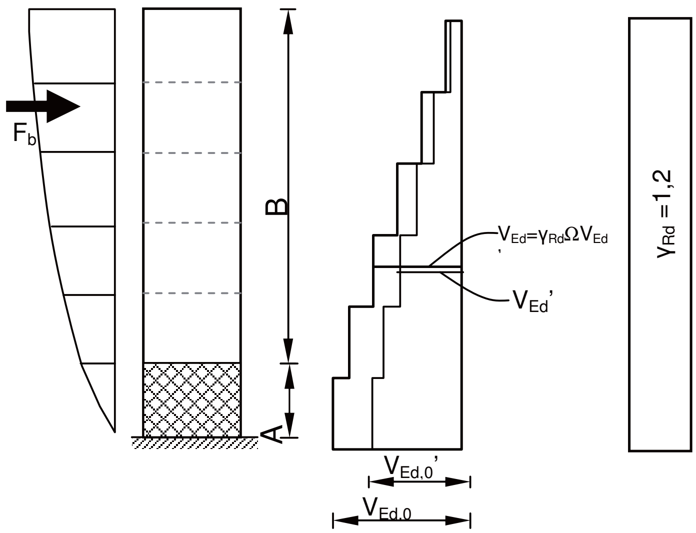


**Figure 15. Design shear force for structural walls**

The design shear forces for the coupling beams are those corresponding to the plastic hinging at both ends. Considering that the spans of the coupling beams is rather small, the amount of the shear force caused by the gravity loads can be neglected.

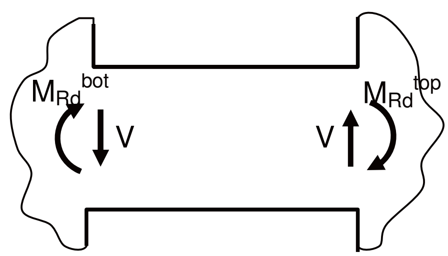


**Figure 16. Design shear force in the coupling beams**

$$V_{Ed}=1,25\frac{|M_{Rd}^{bot}|+|M_{Rd}^{top}|}{l_r}$$

Where,

$M_{Rd}^{bot}$, $M_{Rd}^{top}$ bending capacity a both ends of the coupling beams (slab reinforcements parallel to the direction of beam shall be accounted for)

$l_r$ clear span of the coupling beam

## 6.5. Reinforcement detailing rules

### 6.5.1. Coupling beams

In the following the main detailing rules provided by the code for coupling beams having the clear span longer than the height of the transversal section which are being reinforced with orthogonal reinforcement are presented

Considering that due to insignificant gravity load, the positive and negative bending moments at both ends are essentially equal the coupling beams are symmetrically reinforced in the longitudinal direction. The reinforcement can be calculated by:

$$A_{s1} = A_{s2} = \frac{M_{Ed}}{f_{yd}(d-a)}$$

Where

$A_{s1}$ and $A_{s2}$ longitudinal reinforcement area at the top and bottom part of the coupling beam

$M_{Ed}$ design bending moment

$f_{yd}$ design yield strength of the longitudinal reinforcement

d effective depth of the transversal section

a rebar concrete cover

Ductile deformed steel rebars should be used. The minimum diameter is D12. On the lateral sides of the transversal section supplementary longitudinal rebars should be placed to achieve a reinforcement ratio of 0,0025...0,004 depending on the beam aspect ratio and the seismic design acceleration.

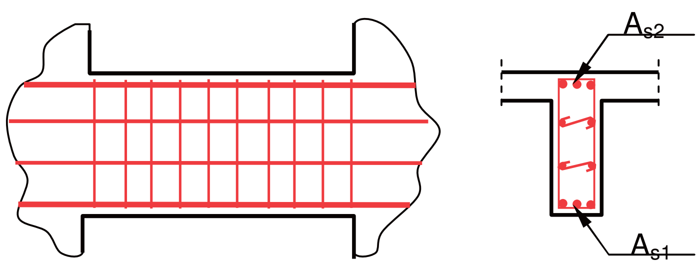


**Figure 17. Armarea riglelor de cuplare cu carcase ortogonale**

To prevent the shear failure of the beams the normalized shear stress should be, in any situation, limited at 2:

$$\upsilon = \frac{V_{Ed}}{bdf_{ctd}} \le 2 \qquad \text{or}$$

$$V_{Ed} \le 2bdf_{ctd}$$

Where:

$V_{Ed}$ design shear force

b width of the coupling beam

d effective depth of the coupling beam

$f_{ctd}$ design tensile strength of the concrete

If this condition is not complied then the concrete section has to be enlarged to allow for a smaller shear stress.

In case of coupling beams the contribution of the compressed concrete area to the shear strength is to be neglected. The stirrups should accommodate the entire shear force as follows:

$$V_{Ed} = \frac{h_w}{s_h} A_{sw} f_{ywd}$$

where,

$V_{Ed}$ design shear force

$h_w$ height of the beam transversal section

$s_h$ stirrups spacing

$A_{sw}$ stirrups area

$f_{ywd}$ design strength of shear reinforcement

Shear failure along a $45°$ inclined crack is considered.

The minimum diameter of the stirrups is 6mm. However, in common design situations D10 and D12 stirrups spaced at 100mm or less result.

The maximum stirrup spacing is given by:

$s_h < 8d$

$s_h < 150mm$

where d is the minimum diameter of the longitudinal bars

The minimum transversal reinforcement coefficient is 0,002.

$$\rho_e = \frac{A_{sw}}{s_h b} \leq 0{,}002$$

### 6.5.2. Walls

In common situations, the following types of rebars are used in RC walls:

- Bending reinforcement, $A_s$
- Horizontal reinforcement, $A_{sh}$
- Stirrups at the extremities of the transversal section, $A_{ss}$
- Vertical reinforcement along the web, $A_{sv}$

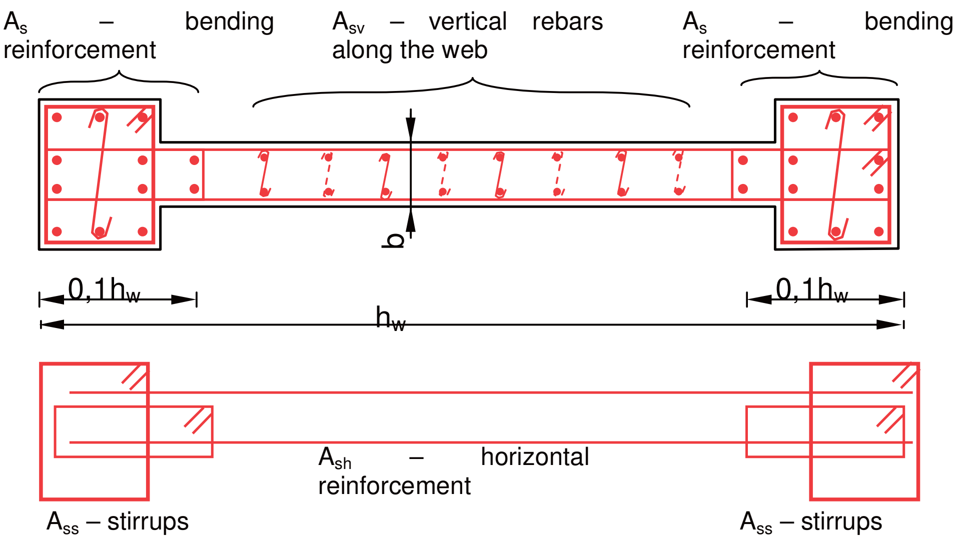


**Figure 18. Wall reinforcements**

The bending reinforcement, $A_s$, is designed to acommodate toghether with $A_{sv}$ the entire design bending moment in a particular section, $M_{Ed}$. Ductile deformed steel bars should be used. The minimum reinforcement ratio for each end of the transversal section is 0,6% for the plastic hinge region and 0,5% in rest. Minimum rebars diameter is 12mm. This can go up to D20 or higher if necessary. The distance between the vertical rebars should be less than 200mm to be able to achieve an effective confinement of concrete.

The horizontal shear reinforcement, $A_{sh}$, for long walls ($h_w$<H), can be determined by:

$V_{Ed}=V_c+V_s$

$V_c$ shear force transmitted through the compressed concrete area

$$V_c=\begin{cases} 0{,}3bd\sigma_0 \le 0{,}6bdf_{ctd} & \text{for the plastic hinge region} \\ bd(0{,}7f_{ctd}+0{,}2\sigma_0) & \text{in rest} \end{cases}$$

Where:

b width of the web

d effective height of the web

$f_{ctd}$ concrete design tensile strength

$\sigma_0$ average compression stress in the wall

$V_s$ shear force transmitted by the stirrups

$V_s = 0{,}8 A_{sh} f_{yd}$

$A_{sh}$ total area of horizontal reinforcement intersected by a 45°inclined crack.

$f_{ywd}$ design yield strength of the horizontal reinforcement

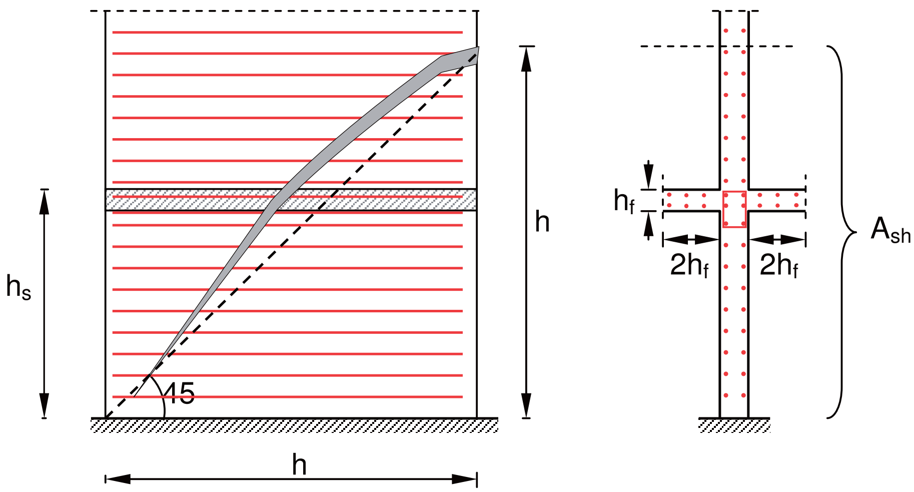


**Figure 19. Horizontal reinforcement area $A_{sh}$**

The following detailing rules shall be applied when the aria of the horizontal reinforcement is established:

The minimum diameter of the horizontal reinforcement: 8mm. Usually, D10 and D12 bars should be used.

The minimum reinforcement ratio is 0,0025 for the plastic hinge region and 0,002 in rest.

The maximum reinforcement spacing is 350mm but usually less than 200mm spacing is used.

The vertical reinforcement on the web of the transversal section, $A_{sv}$, is established based on a minimum ratio of 0,003 in the plastic hinge region and 0,0025 in rest. D8, D10 and D12 bars are usually used. In necessary, to improve the bending strengt of the wall, up to D20 diameters can be used. The maximum spacing of these vertical bars is 250mm.

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*~, Romanian code: Design code for reinforced concrete wall structures CR2-1-1.1/2005, 2005*

*~, Romanian code: Design code for masonry structures CR6/2006, 2006*

~, *Romanian code: Design of structures for earthquake resistance - Part 1: General rules, seismic actions and rules for buildings P100-1/2006, 2006*

# Annex 7

# Country Report

# **Turkey**

**Country Report WFEO-UNESCO**

Guidelines for low rise wall type structures

# TURKEY

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CONTENTS

# 1 Introduction.

## 1.1 General information[1]

Official name：Republic of Turkey

Area: 780,000 $km^2$

Population: 77,804,122*

Density: 92,6/$km^2$

Capital city: Ankara

Coordinates: 39°55’ N 32°50’ E

Area of capital city: 2516 $km^2$

Population of capital city: 3,763,591**

* 2010 estimation

** 2007 census

**Figure 1.** Location of Turkey

(Source: http://en.wikipedia.org/wiki/File:Turkey_(orthographic_projection).svg)

## 1.2 Geography

Turkey is a country located at a point where the 3 continents of the world (Asia, Africa and Europe) are closest to each other and where Asia and Europe meet. Asian Turkey (made up largely of Anatolia) including 97% of the country is separated from European Turkey by the Istanbul Bosporus, the Sea of Marmara, and the Dardanelles (which together form a water link between the Black Sea and the Aegean Sea). European Turkey (eastern Thrace or Rumelia in the Balkan peninsula) comprises 3% of the country [2].

Turkey's topographic structure shows clearly the country's high elevation in comparison to its neighbors, half of the land area being higher than 1000 meters and two thirds higher than 800 meters. In addition, mountain ranges extend in an east-west direction parallel to the north and south coasts (Fig. 2).


**Figure 2.** Topographic structure of Turkey

(Source: http://en.wikipedia.org/wiki/File:Turkey_topo.jpg)

## 1.3 Seismic sources of Turkey

Turkey is one of the most actively deforming regions in the world and has a long history of catastrophic earthquakes, as reminded by the recent August 17, 1999 Kocaeli (Magnitude = 7.4) and November 12, 1999 Düzce (Magnitude = 7.2) events [3].

The active tectonics of Turkey is resulted from the continental collision of the African and Eurasian plates and this framework of Turkey is outlined by three major structures: the two intracontinental transform faults, namely the dextral North Anatolian Fault Zone (NAFZ) and the sinistral East Anatolian Fault Zone (EAFZ), and the Aegean- Cyprian Arc (a convergent plate boundary where the African plate to the south is subducting beneath the Anatolian plate to the north). Furthermore, the sinistral Dead Sea fault zone also plays an important role (Fig. 3). The two strike-slip faults meet and form a continental triple junction in northeastern Turkey. The intervening Anatolian wedge of amalgamated crustal fragments is moving westward onto the eastern Mediterranean lithosphere, and this westward extrusion of Anatolia is accompanied by counterclockwise rotation. The continuum of deformation along the NAFZ and EAFZ and the westward extrusion of Anatolia have been accommodated through the internal deformation of Anatolia. Consequently, four distinct neotectonic provinces, each of which is characterized by unique structural elements and associated basin formation, have been generated: the East Anatolian contractional, the North Anatolian, the Central Anatolian "Ova," and the West Anatolian extensional provinces (Fig. 3) [3].

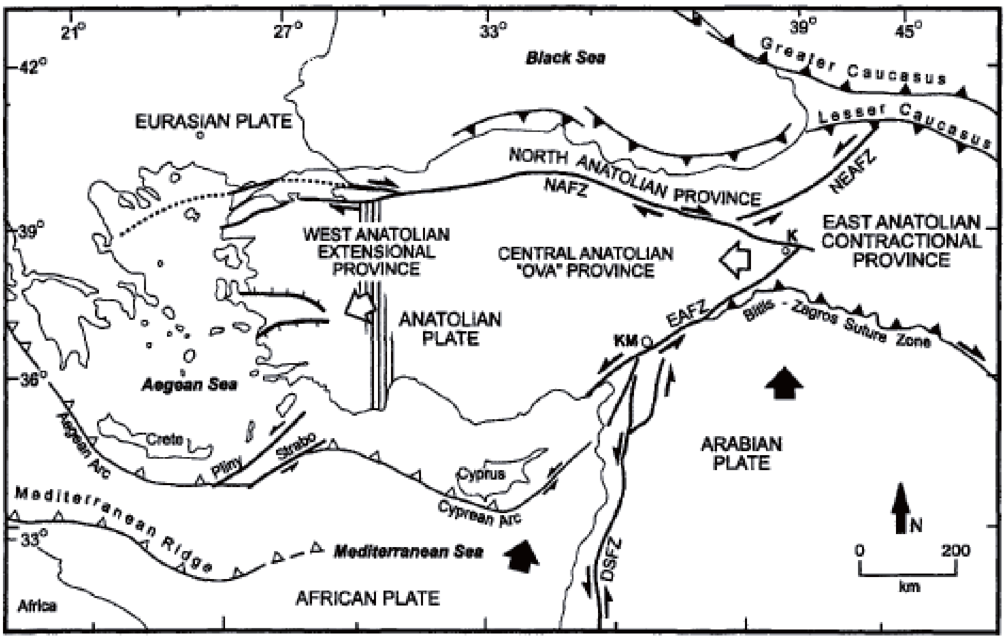


**Figure 3.** Active tectonic structures in Turkey [3]

While many catastrophic earthquakes hit different areas of Turkey in history, the first catastrophic natural disaster experienced by Republic of Turkey was the Erzincan Earthquake in 1939. The magnitude of the earthquake was 7.8 and caused a loss of more than 33.000 lives and destruction of 140.000 homes [4]. Recent experienced catastrophic earthquakes were the Kocaeli Earthquake (M=7.4, August 17, 1999) and the Duzce Earthquake (M=7.2 November 12, 1999) as mentioned before. The Kocaeli Earthquake caused death of more than 17.500 people and destruction about 66.500 buildings. The Duzce Earthquake resulted with loss of approximately 800 lives and destruction of 15.000 buildings [5].

# 2 History and details of regulations

## 2. 1 Seismic Regulation

### 2.1.1 Historical development of seismic regulation in Turkey

The first set of explicit legal provisions for earthquake resistant design was established in 1940 by the Ministry of Public Works, followed by another version in 1942 annexed with a seismic zone map. The seismic regulation was revised in 1944 within articles of Law No. 4623 [6]. Although the law stated that any building built in contravention of the requirements of the regulation would be demolished, this stipulation (and its future versions) did not clearly state which authority is to do the demolishment [6]. While two further updates of the regulation were made in 1949 and 1953 to reflect the amendments of the seismic zone map, there was no major change in the code [7]. By the foundation of the Ministry of Reconstruction and Resettlement in 1958, the disaster prevention policy was upgraded and the method to decide the base shear coefficient was revised in 1961 [8]. The next revisions in 1968 and 1975 brought important enhancements to the seismic design and introduced the international developments to the engineering society in Turkey. Ductility was first time mentioned at member and structural level in 1975 code. Capacity design principles were introduced by 1998 code together with important detailing issues for seismic design. The most recent code revised in 2007, particularly was a very important step towards displacement based design through the related requirements for seismic assessment of existing buildings and retrofitting. The evolution of the seismic design code is summarized in more detail below [4].

**1940 Seismic Regulation (Ministry of Public Works, 1940)**

This was the first seismic regulation in Turkey. Besides several rules related with construction, materials and workmanship, this code gave the fundamental base shear coefficient of 0.10 for calculation of lateral loads. In case of presence of wind loads (*W*), the design lateral load (*H*) is calculated by Eq. 1. In this equation, *G* and *P* are the sum of dead and live loads, respectively. No specific distribution of the lateral load was defined by this code [4].

$$H = 0.10(G + \frac{P}{2}) + \frac{W}{2} \qquad (1)$$

**1944 Seismic Regulation (Ministry of Public Works, 1944)**

In 1944, seismic zone map included two seismic zones, Zones I and II. The areas, which were not included in Zones I and II were considered to be safe in terms of seismicity. The fundamental base shear coefficient was 0.02-0.04 and 0.01-0.03 for Zones I and II, respectively [8]. The selection of the appropriate value in these ranges was the responsibility of the design engineer, and the selected value needed the approval of the inspecting authority [7]. Like in previous version, in this regulation, no consideration was given to the local geotechnical conditions and structural characteristics. Furthermore, the distribution of equivalent lateral loads through the height of the building was not defined as well [4].

**1961 Seismic Regulation (Ministry of Public Works and Housing, 1961)**

In 1960s and 1970s, based on very rapid industrialization and urbanization, the amount of constructions increased tremendously, particularly in cities such as Istanbul, Ankara and Izmir. Therefore, a great portion of the existing buildings constructed before the 1968 version of the Seismic Design Code are supposed to be constructed considering 1961 Seismic Regulation [9]. In this regulation parameters related with seismic zone, type of structural system and ground conditions were taken into account for determining the base shear coefficient. Upper limit of the base shear coefficient was 0.1. The distribution of the lateral loads was uniform along the height of the building. A qualitative recommendation was also present in this code on prevention of excessive irregularities in plan to minimize the potential effects of global torsion. According to this code, all parts of the building should resist seismic effect, which is to be calculated by Eq. 2 [4].

$$H = C(G + nP) + W / 2 \tag{2}$$

In this equation, $C$ and $n$ are the fundamental base shear and live load reduction factor, respectively. The live load reduction factor is given as 1.0 for densely populated buildings (theaters, hotels, factories, office buildings) and 0.5 for other ordinary buildings (residential buildings). Fundamental base shear coefficient is calculated by Eq. 3, where $C_o$ is a coefficient determined based on the height of the building, $n_1$ is a coefficient related with ground type (I, II and III) and type of structural system (reinforced concrete or steel), and $n_2$ is the seismic zone coefficient (Zones I and II). The potential values of $C_o$ and $n_1$ are summarized in Tables 2 and 3, respectively. It should be noted that the interaction between soil and structure was somehow taken into account through $n_1$ coefficient. The coefficient $n_2$ is to be taken 1.0 and 0.6 for seismic zones I and II, respectively [4].

$$C = C_o n_1 n_2 \tag{3}$$

**Table 2.** $C_o$ coefficient according to building height

| Height of building (m) | $C_o$ coefficient |
|---|---|
| <16 | 0.06 |
| 16-22 | 0.07 |
| 22-28 | 0.08 |
| 28-34 | 0.09 |
| 34-40 | 0.1 |
| >40 | +0.01 / 3 m |

It should be noted that when wind load (*W*) is higher than the design lateral load calculated by Eq. 2, design lateral load is considered to be equal to the wind load.

**Table 3.** $n_1$ coefficient according to building type and ground conditions

| Ground* | Reinforced | Steel |
|---|---|---|
| I | 0.8 | 0.6 |
| II | 0.9 | 0.8 |
| III | 1.0 | 1.0 |

*I: Rock, hard soil, II: medium soil, III: soft soil

This code permitted an increase of 50% in allowable stresses in case of design according to lateral loads.

The seismic zone map was revised in 1963 and number of seismic zones increased from two to four including the zone where seismic design is not required [8].

**1968 Seismic Regulation (Ministry of Public Works and Housing, 1968)**

This code brought significant enhancements to seismic resistant design such as:

- Consideration of dynamic characteristics of the building
- Introduction of building importance factor

- Inverse triangular distribution of lateral forces
- Consideration of increased shear forces due to global torsion of the building, when the eccentricity between centers of mass and rigidity exceeds 5% of the larger plan dimension of the building [4].

In this code, base shear force (*F*) to consider the effects of earthquakes is to be calculated by Eq. 4. In this equation, *W* is the total weight of the building to be considered for seismic analysis (Eq. 5) and *C* is the fundamental base shear coefficient to be calculated by Eq. 6. It should be noted that according to this code live load reduction factor *n* is 1.0 for buildings such as theaters, schools, stadiums, storage facilities, and 0.5 for health facilities, hotels, administrative or residential buildings [4].

$$F = CW \tag{4}$$

$$W = \sum W_i = \sum G_i + n_i P_i \qquad \text{(i: storey number)} \tag{5}$$

$$C = C_o \alpha \beta \gamma \tag{6}$$

In Eq. 6, $C_o$ is seismic zone factor (0.06, 0.04 and 0.02, for Zones I, II and III, respectively), $\alpha$ is the coefficient related with ground conditions (0.8, 1.0 and 1.2, for hard, medium and soft soil), $\beta$ is the building importance factor (1.5 for important or densely populated buildings such as communication buildings, hospitals, fire stations, museums, schools, stadiums, theaters, train stations, religious buildings, and 1.0 for ordinary buildings such as residential, office and industrial buildings, hotels, restaurants, etc.), and $\gamma$ is the dynamic coefficient to be calculated by Eqs. 7a or 7b based on the fundamental period of the building (T).

$$\gamma = 1 \qquad (T \le 0.5\ \text{s}) \tag{7a}$$

$$\gamma = \frac{0.5}{T} \ge 0.3 \qquad (T > 0.5\ \text{s}) \tag{7b}$$

A simple equation is also given in this code for calculation of fundamental period of the building (Eq. 8) to be used unless the period is not calculated based on a more sophisticated method. In this equation *H* and *D* are the height of the building (m) and plan dimension of the building (m) in the direction of the considered lateral load.

$$T = \frac{0.09H}{\sqrt{D}} \qquad \text{(second)} \tag{8}$$

According to this code, the lateral forces are to be distributed to floor levels along the height by using Eq. 9. In this equation, $F_i$, $W_i$ and $h_i$ are the lateral forces acting on the $i^{th}$ storey floor, weight of the $i^{th}$ storey and height of this story measured from the foundation level.

$$F_i = F \frac{W_i h_i}{\sum W_i h_i} \tag{9}$$

In this code and latter revisions, the effects of earthquake are taken into account separately without consideration of the wind load. Consequently, analysis against earthquake and wind loads are carried out separately and design is carried out according to the most unfavorable case.

In 1972, seismic zone map is divided into five zones, including the zone with no risk of earthquake [8]. This seismic zone map can be seen in Figure 4.

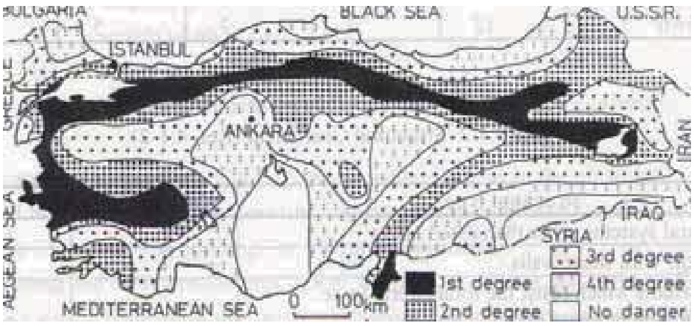


**Figure 4.** Seismic zone map of Turkey in 1944 Seismic Regulation [8]

**1975 Seismic Regulation (Ministry of Public Works and Housing, 1975) [4]**

1975 Seismic Regulation was valid for more than 20 years. This code was the first code, in which the term “ductility” was used explicitly. Furthermore, while determining the base shear force, structural ductility was considered first time implicitly according to the lateral load resisting system of the structure through the defined structure type coefficient. Other important improvements were:

- Inclusion of more detailed principles related with seismic resistant detailing

- Inclusion of ground dominant period into the equation given for determination of dynamic coefficient
- Inclusion of explicit definition of irregular buildings (the definition of irregularities were not sufficiently detailed)
- Inclusion of requirement of dynamic analysis for irregular or high-rise structures (*H*>75 m)
- Consideration of an additional eccentricity of 5% of the largest plan dimension of the building perpendicular to lateral load considered over existing eccentricity between mass and stiffness centers

In this code, base shear force to consider the effects of earthquakes is to be calculated by Eq. 10. In this equation *C, Co, K, S* and *I* are fundamental base shear coefficient, seismic zone coefficient (0.10, 0.08, 0.06 and 0.04, for Zones I, II, III and IV, respectively), structure type coefficient, dynamic coefficient and building importance factor, respectively.

$$C = C_o KSI \geq \frac{C_o}{2} \tag{10}$$

The definition of structure type coefficient is presented in Table 4 and dynamic coefficient (spectrum coefficient) is calculated by Eq. 11, where $T_o$ is the dominant period of the ground in second. It should be noted that dynamic coefficient should be considered as 1.0 for one and two storey structures and all masonry buildings.

**Table 4.** K-structure type coefficient

| **Structure type** | **K*** | |
|---|---|---|
| Ductile frames** | a) 0.60 | b) 0.80 |
| Non-ductile frames** | a) 1.20 | b) 1.50 |
| Steel frames with | a) 1.20 | b) 1.50 |
| Shear wall – ductile | a) 0.80 | b) 1.00 |
| Shear wall structures | 1.33 | |
| Masonry buildings | 1.50 | |
| Others | 1.00 | |

*the minimum value of K is 1.0 for one or two storey structures.

**a) reinforced concrete or reinforced masonry infill walls, b) unreinforced masonry infill walls, c) light weight or few infill walls, or prefabricated concrete infill walls.

*** the ductile frames should resist at least 25% of the lateral loads.

$$S = \frac{1}{\left|0.8 + T - T_o\right|} \leq 1.0 \tag{11}$$

For determination of the fundamental period of the building, in addition to Eq. 8, which was already given in 1968 regulation, another formula is given as well (Eq. 12). In Eq. 12, *N* is the number of stories. For the value of $T_o$, values ranging between 0.20 and 0.90 are given as a function of ground conditions (0.20 for ground type I - rock or very stiff soil, 0.90 for ground type IV - very soft soil, alluvial deposits).

$$T = (0.07 - 0.10)N \tag{12}$$

The definition of building importance factor, *I* was almost same as 1968 code (either 1.0 for ordinary structures, or 1.5 for important or densely populated structures).

The distribution of the base shear force along the height of the building was again according to first mode shape of the building (inverse triangular distribution) with an additional singular force to be acted on the top storey (Ft), to implicitly account for the effects of higher modes approximately, Eqs. 13 and 14. According to the code, when (*H/D*)≤3, Ft can be considered as zero.

$$F_i = (F - F_t)\frac{W_i h_i}{\sum W_i h_i} \tag{13}$$

$$F_t = 0.004F\left(\frac{H}{D}\right)^2 \leq 0.15F \tag{14}$$

It should be noted that while the building weight to be considered for calculation of base shear force was similar to 1968 code (Eq. 5), the values of live load reduction factor (n) were slightly revised. This value was 0.8 for storage type structures, 0.6 for schools, theaters, concert halls, shops, dormitories, and 0.3 for residential buildings, offices, hospitals, hotels)

In 1996, the seismic zone map was revised once more, Figure 5. This seismic zone map, while still having five zones as 1972 map, increased the areas in Zone

I significantly.

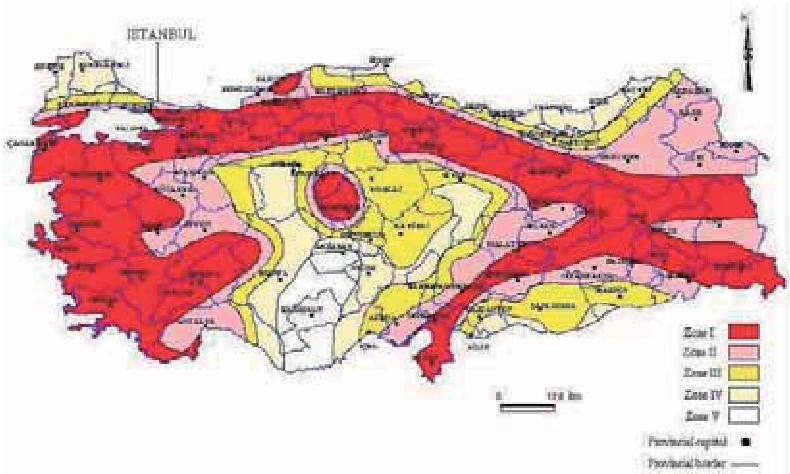


**Figure 5.** Seismic zone map of Turkey in 1972 Seismic Regulation [4]

**1998 Seismic Regulation (Ministry of Public Works and Housing, 1998) [10]**

After more than twenty years of publication of 1975 code, a revised version was published in 1998 just one year before the catastrophic earthquakes experienced in 1999. Therefore, the 1998 code and experienced earthquakes created a milestone in terms of earthquake resistant design and construction, as well as the demand of public for safe housing. Now, most engineers in Turkey believe that the buildings constructed after 1998-1999 are much safer against earthquakes with respect to older buildings [4].

The most important advances introduced through the 1998 code are:

- Explicit definition of design earthquake in terms of occurrence probability
- Explicit definition of acceptable structural performance against design earthquake
- Definition of an elastic design spectrum
- Definition of seismic load reduction factor determined based on the dynamic characteristics, ductility of the structural system and over-strength factor
- Quantitative definition of irregularities

The design earthquake considered in this code corresponds to an earthquake with the return period of 475 years for ordinary buildings (for building importance factor 1.0) and 2475 years for the most important buildings (for building importance factor 1.5). The probabilities of exceedence for these two cases are 10 and 2% in 50 years, respectively.

In this code, the spectral acceleration coefficient (*A(T)*), which corresponds to the division of elastic acceleration design spectrum by gravitational acceleration (g) for 5% damping ratio, is given by Eq. 15. In this equation $A_o$, *I* and *S(T)* are the effective gravitational acceleration coefficient (seismic zone coefficient), building importance factor and spectrum coefficient.

$$A(T) = A_o I S(T) \tag{15}$$

Effective gravitational acceleration coefficient ($A_o$) is to be taken as 0.40, 0.30, 0.20 and 0.10, for seismic zones I, II, III and IV, respectively, (Fig. 5). Building importance factor (*I*) is given with more details in this code with respect to previous versions, Table 5. Spectrum coefficient (*S(T)*) is determined through Eq. (16) as a function of fundamental period the building (*T*) and characteristic spectrum periods ($T_A$ and $T_B$), which are to be determined based on the ground type. The characteristic spectrum period values for different ground conditions are given in Table 6. In this table, it is apparent that Z1 represents strongest ground conditions, while Z4 corresponds to the weakest. The variation of spectrum coefficient with respect to fundamental period of the building is shown in Figure 6.

$$S(T) = 1 + 1.5\frac{T}{T_A} \tag{16a}$$

$$S(T) = 2.5 \tag{16b}$$

$$S(T) = 2.5\left(\frac{T_B}{T}\right)^{0.8} \tag{16c}$$

**Table 5**. I-building importance factor

| **Purpose of Occupancy or Type of Building** | **Importance Factor ( *I* )** |
|---|---|
| 1. Buildings to be utilized after the earthquake and buildings containing hazardous materials<br>a) Buildings required to be utilized immediately after the earthquake (Hospitals, dispensaries, health wards, fire fighting buildings and facilities, PTT and other telecommunication facilities, transportation stations and terminals, power generation and distribution facilities; | 1.5 |

| | |
|---|---|
| governorate, county and municipality administration buildings, first aid and emergency planning stations)<br>b) Buildings containing or storing toxic, explosive and flammable materials, etc. | |
| 2. Intensively and long-term occupied buildings and buildings preserving valuable goods<br>a) Schools, other educational buildings and facilities, dormitories and hostels, military barracks, prisons, etc.<br>b) Museums | 1.4 |
| 3. Intensively but short-term occupied buildings<br>Sport facilities, cinema, theatre and concert halls, etc. | 1.2 |
| 4. Other buildings<br>Buildings other than above defined buildings. (Residential and office<br>buildings, hotels, building-like industrial structures, etc.) | 1.0 |

**Table 6.** Characteristic spectrum period values

| Local Site Class | $T_A$ (second) | $T_B$ (second) |
|---|---|---|
| Z1 | 0.10 | 0.30 |
| Z2 | 0.15 | 0.40 |
| Z3 | 0.15 | 0.60 |
| Z4 | 0.20 | 0.90 |

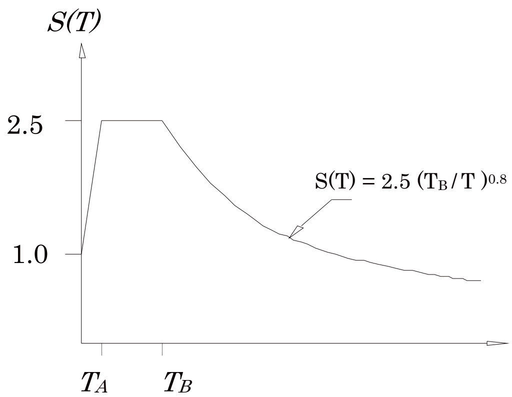


**Figure 6.** Determination of elastic spectrum coefficient

For consideration of inelastic deformation capacity of the structures, certain level of damage beyond elastic limits is allowed under design earthquake provided that the building does not collapse and life safety is provided. For utilizing such an assumption, the building should have a certain level of ductility. According to this code, the buildings can be designed considering two levels of ductility; normal or high. There are several rules, particularly in terms of capacity design principles, construction details and irregularities for classifying the structural systems as normal or high ductility. Based on the characteristics of the structural system and considering the ground characteristics, the lateral load calculated based on the elastic design spectrum can be reduced. Obviously, if the structural system posses the characteristics such that the system can be classified as high ductility system, the reduction in lateral loads is higher with respect to normal ductility structural systems. Consequently, the seismic load reduction factor (*Ra(T))* is to be determined by Eq. 17.

$$R_a(T) = 1.5 + (R - 1.5)\frac{T}{T_A} \qquad T \le TA \qquad (17a)$$

$$R_a(T) = R \qquad T > TA \qquad (17b)$$

In Eq. 17, *R* is the structural system coefficient, which can be determined through Table 7. It should be noted that the value of seismic load reduction factor does not represent the structural system ductility, but the structural system ductility and over-strength factor.

Finally, base shear force ($V_t$) can be calculated by Eq. 18, where *W* is the total weight of the building to be calculated in a similar method as in 1975 code.

$$V_t = W\frac{A(T)}{R_a(T)} \ge 0.10 A_o I W \qquad (18)$$

### 2.1.2 Current seismic regulation

**2007 Seismic Regulation (Ministry of Public Works and Housing, 2007) [11]**

Based on the demand of people and official institutions for earthquake safe environment after earthquakes experienced in 1999, many structures were investigated in terms of seismic safety and some of these were retrofitted. However, due to lack of official guidelines and standards about seismic safety assessment and retrofitting, in many cases non-standard, sometimes inappropriate approaches were being used by design engineers while analyzing or retrofitting the existing buildings. Therefore, the most recent version of the seismic design code published in 2007 mainly addressed the issues on seismic safety assessment of existing buildings and retrofitting comprehensively, with minor revisions for new buildings. With this version title of the code, which was "Regulation for structures in disaster areas" since 1961, was changed as "Regulation for buildings in seismic areas". Concordantly, other issues related with other disasters (such as flood and fire) and structures other than buildings (such as chimneys, silos) are taken out of the code [4].

The most important advances introduced through the 2007 code are:

- Inclusion of a new extensive chapter on seismic safety assessment and retrofitting
- Inclusion of a linear elastic method for seismic safety assessment considering the inelastic behavior in terms of approximate allowable demand/capacity ratios given according to the damage level
- Inclusion of performance based assessment principles for existing structures and retrofitting
- Inclusion of different design earthquakes and performance levels to be considered for different types of buildings

**Table 7.** Structural system coefficients (*R*)

| Building Structural System | Systems of Nominal Ductility Level | Systems of High Ductility Level |
|---|---|---|
| (1) Cast-In-Situ Reinforced Concrete Buildings | | |
| (1.1) Buildings in which seismic loads are fully resisted by frame | 4 | 8 |
| (1.2) Buildings in which seismic loads are fully resisted by coupled structural walls | 4 | 7 |
| (1.3) Buildings in which seismic loads are fully resisted by solid structural walls | 4 | 6 |
| (1.4) Buildings in which seismic loads are jointly resisted by frames and solid and/or coupled structural walls | 4 | 7 |

| | | |
|---|---|---|
| <u>(2) Prefabricated Reinforced Concrete Buildings</u> | | |
| (2.1) Buildings in which seismic loads are fully resisted by frames with connections capable of cyclic moment transfer | 3 | 6 |
| (2.2) Buildings in which seismic loads are fully resisted by single-storey hinged frames with fixed-in bases | — | 5 |
| (2.3) Buildings in which seismic loads are fully resisted by prefabricated solid structural walls | — | 4 |
| (2.4) Buildings in which seismic loads are jointly resisted by frames with connections capable of cyclic moment transfer and cast-in-situ solid and/or coupled structural walls | 3 | 5 |
| <u>(3) Structural Steel Buildings</u> | | |
| (3.1) Buildings in which seismic loads are fully resisted by frames | 5 | 8 |
| (3.2) Buildings in which seismic loads are fully resisted by single-storey hinged frames with fixed-in bases | 4 | 6 |
| (3.3) Buildings in which seismic loads are fully resisted by braced frames or cast-in-situ reinforced concrete structural walls | | |
| (a) Concentrically braced frames | 3 | — |
| (b) Eccentrically braced frames | — | 7 |
| (c) Reinforced concrete structural walls | 4 | 6 |
| (3.4) Buildings in which seismic loads are jointly resisted by frames and braced frames or cast-in-situ reinforced concrete structural walls | | |
| (a) Concentrically braced frames | 4 | — |
| (b) Eccentrically braced frames | — | 8 |
| (c) Reinforced concrete structural walls | 4 | 7 |

- Inclusion of single-mode and multi-mode push-over analysis for seismic safety assessment and retrofitting
- Inclusion of nonlinear time history analysis
- Inclusion of principles and details related with conventional retrofitting techniques (such as concrete jacketing, strengthening with steel, and shear wall addition) and retrofitting using innovative materials (such as fiber reinforced polymers).

## 2.2 Technical requirements for low rise wall type structures

### 2.2.1 General

The details of design and construction of low rise wall type masonry structures are given in 2007 seismic regulation [11] and technical requirements for design and construction methods masonry [12].

Seismic loads for masonry structures shall be determined according to 2007 seismic regulation with assumptions of $S(T_1)$=2.5 and $R_a(T_1)$=2 (Eq. 17). Seismic analysis is to be performed under the determined seismic loads and the shear and the normal stresses should be calculated. The obtained stresses are limited not to exceed the allowed material stress capacities.

The application of the 2007 seismic regulation and technical requirements are limited to the following conditions:

- Natural stone, fired brick, hollow brick (ratio of hollow area have to be satisfy regulation requirement), block brick, lime-sand stone, concrete briquette, gas concrete and adobe may be used as a load-bearing wall material.
- Natural stone shall only be used in basement and entrance floor bearing walls.
- Concrete walls shall only be used in basement.
- The maximum permitted number of stories for masonry structures is presented in Table 8 depending on seismic zones. In addition, it is permitted to add a roof story whose floor area shall not exceed 25 % of the foundation floor area of the structure.

**Table 8.** Maximum number of stories permitted [11]

| Seismic zone | Number of stories |
|---|---|
| 1 | 2 |
| 2,3 | 3 |
| 4 | 4 |

- Allowable story height of masonry structures shall not be more than 3.0 m. This limit shall be applied to the adobe structures as 2.80 m for normal stories and 2.40 m for basement.
- Load-bearing wall in plan shall be arranged, as much as possible, regularly and symmetric or nearly symmetric with respect to the main axes.
- In plan, load-bearing walls shall be constructed so as to be placed one over the other.

### 2.2.2 Allowable material stresses

Allowable compressive wall stress shall be determined with using methods explained below.

- Compressive strength of wall can be taken from laboratory tests of wall parts (having identical mechanical properties with actual wall). Allowable compressive wall stress shall be considered as 25 % of wall compressive strength obtained from laboratory tests.
- If compressive strengths of masonry material and mortar are known separately, allowable compressive wall stress shall be determined from Table 9 depending on these material compressive strengths.
- If laboratory compressive tests of wall parts are not available, allowable compressive wall stress can be calculated depending on compressive strength of masonry material units. The compressive strength of wall shall be assumed to be 50 % of the compressive strength of masonry material and the allowable compressive stress of wall shall be 25 % of the compressive strength of the wall.
- If mechanical characteristics of masonry constituents (unit and mortar) are not known, allowable compressive stress of wall can be taken from Table 10. In addition, in case of slender walls, allowable stress of the wall shall be decreased considering the slenderness effect as given in Table 11.

Allowable shear stress for masonry walls shall be calculated by Eq. 18, where $\tau_{em}$ is the allowable shear stress (MPa), $\tau_o$ is the allowable bond strength of the mortar (MPa), $\mu$ is the friction coefficient (which may be taken as 0.5) and $\sigma$ is the calculated normal stress (MPa). Allowable bond strength of the mortar shall be taken from Table 12 depending on the material type.

$$\tau_{em}=\tau_o+\mu\sigma \quad (18)$$

**Table 9.** Allowable compressive stresses depending on mortar and masonry [11]

| Average compressive strength of masonry material | Mortar compressive strength (MPa) | | | | |
|---|---|---|---|---|---|
| | A (15) | B (11) | C (5) | D (2) | E (0.5) |
| 25 | 1.8 | 1.4 | 1.2 | 1.0 | 0.8 |
| 16 | 1.4 | 1.2 | 1.0 | 0.8 | 0.7 |
| 11 | 1.0 | 0.9 | 0.8 | 0.7 | 0.6 |
| 7 | 0.8 | 0.7 | 0.7 | 0.6 | 0.5 |
| 5 | 0.6 | 0.5 | 0.5 | 0.4 | 0.4 |

**Table 10.** Allowable compressive stresses [11]

| Wall material | Allowable wall stresses (MPa) |
|---|---|
| Hollow (vertical) brick (hollow section ratio is less than 35 %, cement-lime mortar) | 1.0 |
| Hollow (vertical) brick (hollow section ratio is between 35 %-45 %, cement-lime mortar) | 0.8 |
| Hollow (vertical) brick (hollow section ratio is more than 45 %, cement-lime mortar) | 0.5 |
| Solid or clay brick (cement-lime based mortar) | 0.8 |
| Stone wall (cement-lime mortar) | 0.3 |
| Aerated concrete (adhesive) | 0.6 |
| Solid concrete briquette (cement based mortar) | 0.8 |

**Table 11.** Reduction coefficients of slenderness effects [11]

| Slenderness ratio | 6 | 8 | 10 | 12 | 14 | 16 | 18 | 20 | 22 | 24 |
|---|---|---|---|---|---|---|---|---|---|---|
| Reduction coefficient | 1 | 1 | 0.9 | 0.8 | 0.8 | 0.7 | 0.7 | 0.6 | 0.6 | 0.5 |

**Table 12.** Allowable bond strength of the mortar depending on masonry type [11]

| Wall material | Allowable bond strength of the mortar (MPa) |
|---|---|
| Hollow (vertical) brick (hollow section ratio is less than 35 %, cement-lime mortar) | 0.25 |
| Hollow (vertical) brick (hollow section ratio is more than 35 %, cement-lime mortar) | 0.12 |
| Solid or clay brick (cement-lime mortar) | 0.15 |
| Stone wall (cement-lime mortar) | 0.10 |
| Aerated concrete (adhesive) | 0.15 |
| Solid concrete briquette (cement mortar) | 0.20 |

### 2.2.3. Minimum wall thickness

The minimum thicknesses of the load-bearing walls (excluding plaster thicknesses) are listed in Table 13 depending on seismic zone, number of stories and masonry materials.

### 2.2.4. Geometrical requirements

The ratio of total length of masonry load-bearing walls in each of the orthogonal direction to gross floor area shall not to be less than **0.20 I** m/m$^2$ in which **I** is the building importance factor and can be taken from Table 5.

**Table 13.** Minimum wall thicknesses [11]

| Seismic Zone | Allowable stories | Natural stones (mm) | Concrete (mm) | Brick and aerated concrete | Others (mm) |
|---|---|---|---|---|---|
| 1, 2, 3, 4 | Basement | 500 | 250 | 1 | 200 |
| | Entrance | 500 | - | 1 | 200 |
| 1, 2, 3, 4 | Basement | 500 | 250 | 1.5 | 300 |
| | Entrance | 500 | - | 1 | 200 |
| | 1. Normal | - | - | 1 | 200 |
| 2, 3, 4 | Basement | 500 | 250 | 1.5 | 300 |
| | Entrance | 500 | - | 1.5 | 300 |
| | 1. Normal | - | - | 1 | 200 |
| | 2. Normal | - | - | 1 | 200 |
| 4 | Basement | 500 | 250 | 1.5 | 300 |
| | Entrance | 500 | - | 1.5 | 300 |
| | 1. Normal | - | - | 1.5 | 300 |
| | 2. Normal | - | - | 1 | 200 |
| | 3. Normal | - | - | 1 | 200 |

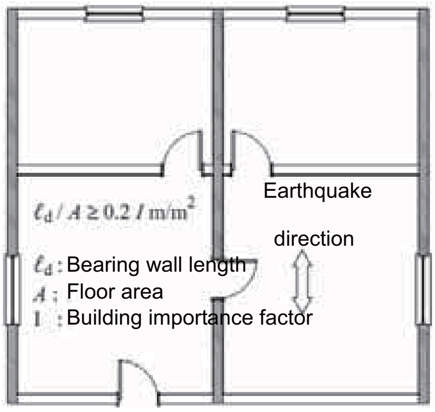


**Figure 6.** Total length of load-bearing wall in each direction [11]

The maximum length of unsupported load-bearing walls shall be 5.5 m in the first level seismic zone and 7.5 m for the other seismic zones. This length shall be applied for adobe walls as 4.5 m.

In the case the condition given above is not satisfied, reinforced concrete vertical bond beams shall be used with 4.0 m spacing to support bearing walls. On the other hand, the maximum wall length supported with reinforced concrete vertical bond beams shall not exceed 16.0 m Figure 7.

Window or door opening lengths in plan shall be less than 40 % of unsupported wall length. This ratio may be increased up to 20 % if reinforced concrete vertical bond beams are used along story height. However, this rule is not applicable for adobe wall structures.

In plan, each window and door opening shall not exceed 3.0 m invidually. For adobe structures, this limit shall be 1.00 m for doors and 0.90 m for windows. On the other hand, the vertical opening limits for adobe buildings shall be 1.90 m for door and 1.20 m for windows.

The minimum distance between door or window openings and building corners shall be 1.50 m in the first level seismic zone and 1.00 m for the other seismic zones. For adobe buildings, it shall be applied as 1.00 m for all seismic zones. In addition, in the case openings are supported with reinforced concrete vertical bond beams from two sides, solid wall distance may be reduced by 20 %.

Except building corners, distance between openings and interception of two perpendicular walls shall be more than 0.50 m. If openings are supported with reinforced concrete vertical bond beams, the distance between opening and interception of two perpendicular walls may be less than 0.50 m.

**2.2.5 Reinforced concrete member details**

**Lintels**

Each of seating lengths of window and door lintels on the walls shall not be less than 15% of lintel clear span nor 200 mm.

In adobe buildings, wooden lintel beams may be seated upside and downside of the windows. Minimum size of wooden lintel beams shall be 100 mm x 100 mm.

**Horizontal bond beams**

Reinforced concrete horizontal bond beams satisfying the following conditions shall be made at places where slabs, including stair landings, are supported by walls such that they shall be cast monolithically with the reinforced concrete slabs.

**(a)** Width of horizontal bond beams shall be equal to the width of wall, and their heights shall not be less than 200 mm.

**(b)** Concrete quality for bond beams shall be at least **C16** (cylinder compressive strength shall be more than 16 MPa). Longitudinal reinforcement shall be at least **6 Ø10** (six 10 mm diameter bars) on stone walls with three at the bottom and three at the top, whereas it shall be at least **4 Ø10** on other load-bearing walls with **Ø8** hoops with a maximum spacing of 250 mm. Longitudinal rebars shall be appropriately overlapped at the corners and intersections to achieve continuity (Fig. 8).

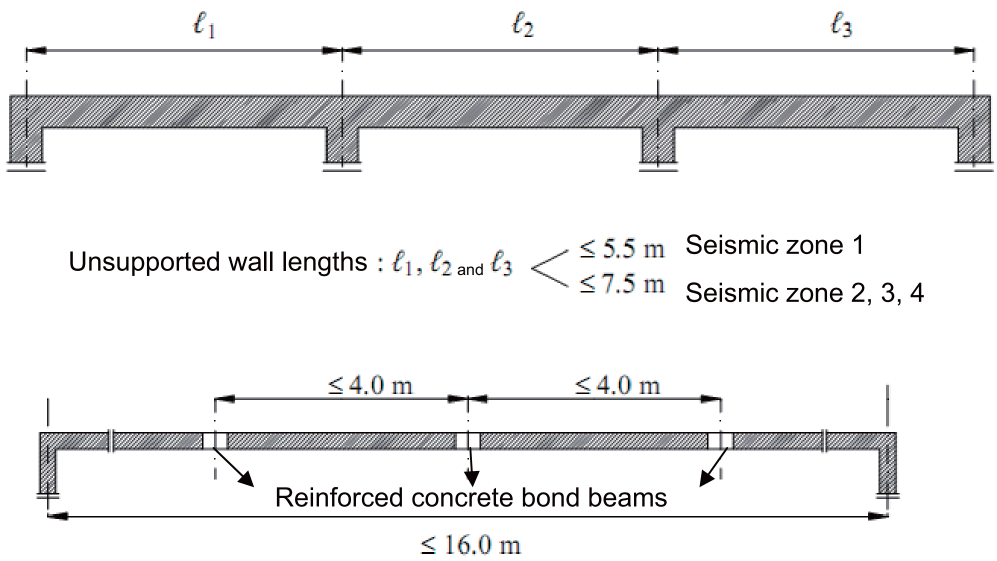


**Figure 7.** Maximum unsupported wall lengths [11]

**Vertical bond beams**

In order to enhance the earthquake resistance of masonry buildings, it shall be appropriate to construct reinforced concrete vertical bond beams in full storey height on the corners of buildings, along the vertical intersections of the load-bearing walls and on both sides of the door and window openings.

Vertical bond beams shall be constructed by reinforcing and concreting the section in between the formworks to be placed parallel to the walls, following the construction of walls on both sides (Fig. 9).

Cross section dimensions of vertical bond beams shall be equal to thicknesses of walls intersecting at corners and at the intersections of walls. In vertical bond beams to be constructed on both sides of window and door openings, cross section dimensions of the beam perpendicular to the wall shall not be less than the wall thickness, whereas the other cross section dimension shall not be less than 200 mm.

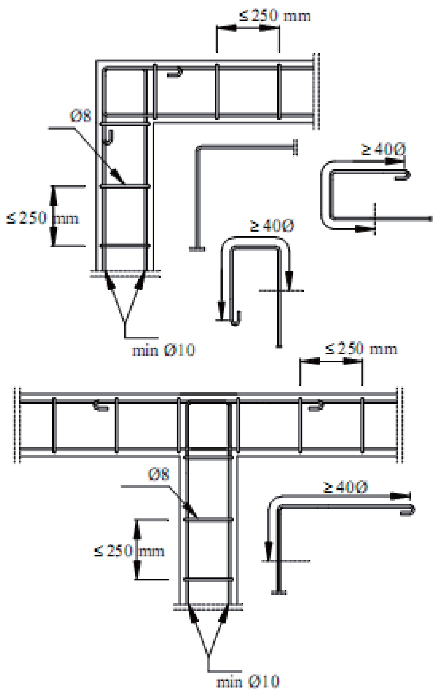


**Figure 8.** Reinforcement requirements for horizontal bond beams [11]

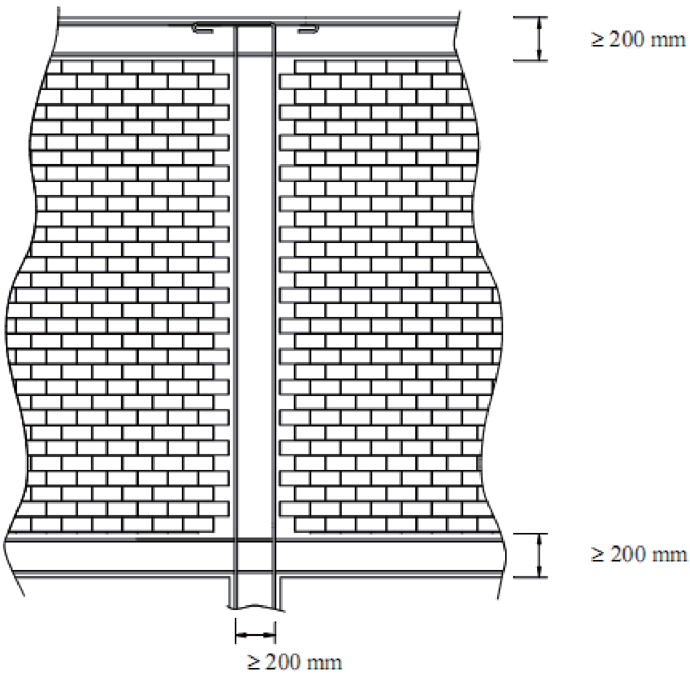


**Figure 9.** Details of vertical bond beams [11]

Concrete quality for vertical bond beams shall be at least **C16**. Longitudinal reinforcement shall be at least **6 Ø12** in stone walls with three at each wall face, whereas it shall be at least **4 Ø12** in other load-bearing walls with **Ø8** hoops with a maximum spacing of 200 mm. Longitudinal starter bars shall be provided at the foundation and at the intermediate floors, and development of the ends of rebars shall be provided to achieve continuity.

**Slabs**

Slabs of masonry buildings can be made as reinforced concrete plate and joist slab. Thickness of the reinforced concrete slabs shall not be less than 80 mm.

# 3 Outline of constructions

## 3.1 Type of constructions

In most parts of Turkey, the structural systems of new buildings are almost always reinforced concrete. According to the census of year 2000, in Istanbul, the structural systems of 76% of existing buildings are reinforced concrete frames with hollow clay brick walls, 22% of existing buildings are unreinforced masonry and 1.5% of the existing buildings are wooden [13]. The ratio of reinforced concrete buildings are increasing even more by demolishing and reconstruction of old existing buildings. This trend is also valid for most of other parts of Turkey, even for some remote villages. Therefore, considering the distinct characteristics of non-engineered existing unreinforced masonry buildings in some villages, it is decided that it is very difficult to obtain some general results for non-engineered wall type buildings in villages and cities. However, for understanding regional building characteristics of Turkey, four different areas were investigated (Istanbul, Denizli, Bitlis and Tokat). Most of the examined buildings are low rise and all of them are masonry. The locations of investigated buildings on Turkey map are given in Figure 10.


**Figure 10.** Investigated regions in Turkey

(map source: http://www.onrrent.com/turkeymap.jpg)

Istanbul, a city connecting Asia and Europe, was the capital of the Roman, Byzantine, and Ottoman Empires and is now the most important city in Turkey, from a cultural, financial, and social standpoint. It is located on the North Anatolian Fault, part of the Himalayan–Mediterranean Fault. Like other parts of Turkey, Istanbul has experienced a number of severe earthquakes in the past. Around the Marmara Sea, which forms the southern border of Istanbul, there have been 34 earthquakes with a surface wave magnitude of more than 7.0 in the last 2,000 years [13]. According to a 2004 investigation, the probability of a magnitude 7.0 or higher earthquake affecting Istanbul in the next 30 years is 41±14 per cent [13]. In this city, eight typical unreinforced fired brick masonry buildings are taken into the scope of the study. These buildings are located in three different parts of Istanbul, Yenikapi (3 buildings), Sirkeci (3 buildings) and Uskudar (2 buildings). While Yenikapi and Sirkeci are in Europe, Uskudar is in Asia (Figure 11). Both locations are in the first degree seismic zone and are among relatively old settlements of Istanbul at the coasts of Marmara Sea around Bosporus. The general appearances some of these buildings are presented in Figure 12.

Denizli is a city in Aegean region of Turkey. This city is important because of contribution to economy and employment of Turkey. It is located on Buyuk Menderes rift valley and it has not been experienced an earthquake of magnitude 7.0 or greater since 60 AD [14]. However, the earthquakes having magnitudes between 4.0 and 7.0 in past 100 years caused severe damages in the low rise buildings in Denizli. While the structural systems of new buildings in Denizli are reinforced concrete, the traditional Denizli houses are the low rise wall type structures (Fig. 13).

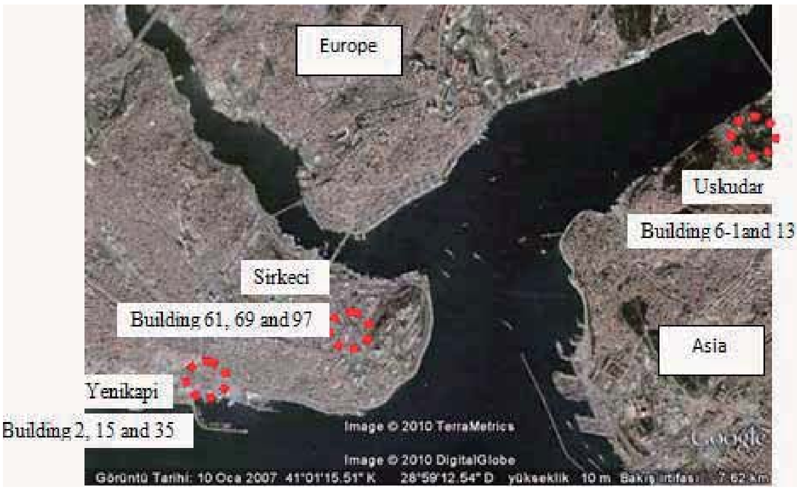


**Figure 11.** Locations of investigated buildings in Istanbul [15]

**Figure 12.** Examples of masonry buildings in Istanbul (Building 15, 69 and 13)

**Figure 13.** Denizli houses [16]

Tokat is a city from central Anatolian region of Turkey. The city is located on North Anatolian fault and a large part of the city is in the first degree earthquake zone. The investigated low rise buildings in Tokat are typical regional buildings because the new buildings are constructed as reinforced concrete. The typical Tokat houses are shown in Figure 14.

**Figure 14.** Traditional Tokat houses [17]

Bitlis is a city in southeastern Anatolian region of Turkey. The city is located on the south Anatolian fault and the region of Bitlis-Zağros overlapping belts [18]. Large part of the city is in the first degree earthquake zone. The regional buildings of Bitlis are ashlar stone buildings (Fig. 15).

**Figure 15.** Bitlis houses [19]

In Turkey, the mostly used wall materials in old constructions are adobe, fired clay brick and stones. In examined buildings from four different parts of Turkey, these materials are also encountered. Generally the used wall material are taken from the closest regional sources. Therefore, the buildings in the same region were generally constructed with same wall materials such as fired clay bricks in Istanbul or ashler stones in Bitlis buildings. In this study, investigated buildings are classified in three groups as adobe constructions, unconfined fired clay brick constructions and ashler stone constructions. Furthermore, the typical material and structural (technical requirements from code) characteristics of these buildings are presented with comparable tables.

### 3.1.1 Adobe constructions

One of the most encountered masonry materials in rural of Turkey are adobe. In this study, a typical Tokat regional adobe building was investigated in detail. Floor plan of this building is given in Figure 16. In the past, since most of these buildings were constructed by same regional masons and workers, these buildings have typical geometrical, material and structural characteristics. In Table 14, typical construction materials of different parts of these buildings are presented. As can be seen from this table, the structural load-bearing walls of these buildings were constructed as half timber-framed with adobe infill walls. An example of this type of walls is given in Figure 17. Non-bearing walls of these buildings are also adobe. It can be noted that the differences between the non-bearing and the bearing walls are thicknesses of walls (if thickness is more than 200 mm, the wall is load-bearing). In addition, the some geometrical and structural parameters considering regulation limits can be seen in Table 15. As it is seen in this table, these buildings generally have one or two stories and structural parameters including wall length to floor area ratio and average wall openings ratio satisfy the regulation limits. However, it should be emphasized that the maximum wall opening lengths exceed the permitted regulation limits ($l_{opening}$> 3 m).

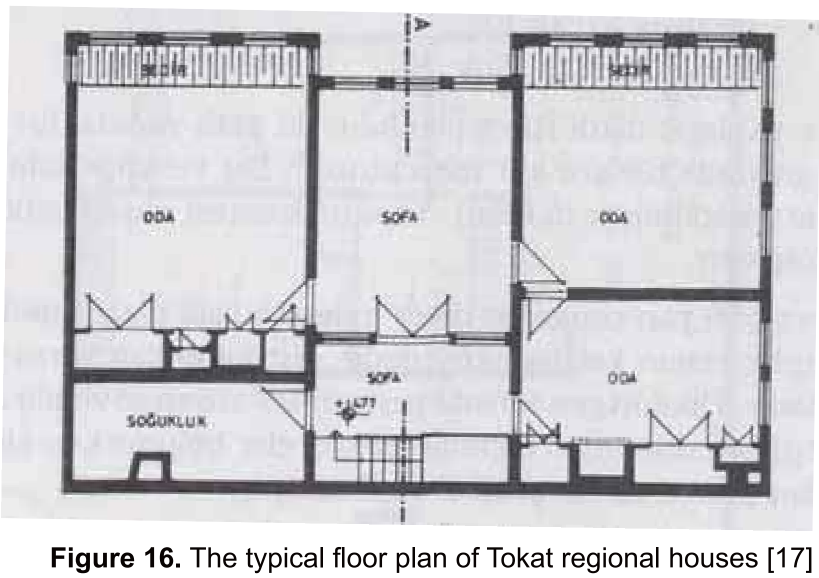


**Figure 16.** The typical floor plan of Tokat regional houses [17]

**Table 14.** Material characteristics of Tokat houses

| Structural load bearings | Non-bearing partitions | Roof structure materials | Slabs | Mortar | Staircases |
|---|---|---|---|---|---|
| Half timber framed with adobe infill walls | Adobe walls | Wooden beams + traditional tile roofs | Wooden flooring and wooden beams | Thatched mud | Wooden |

**Figure 17.** A typical half timber framed building [20]

**Table 15.** Geometrical and structural characteristics of Tokat houses

| Height of building | Minimum load bearing wall thickness | Wall length to floor area ratio ($m/m^2$) | | Average wall openings ratio | |
|---|---|---|---|---|---|
| | | x direction | y direction | x direction | y direction |
| 9 m | 200 mm | 0.24 | 0.22 | 0.32 | 0.26 |

### 3.1.2 Unconfined fired clay brick constructions

The most common masonry material of existing buildings in Istanbul is fired clay brick. In the current study, eight different buildings were investigated in different regions of Istanbul. A detailed floor plan of one of the examined buildings is given Figure 18. Moreover, the material, the geometrical and the structural characteristics of these buildings are presented in Table 16 in detail. It should be noted all of these buildings were constructed as unreinforced fired clay brick masonry and cement-lime mortars.

Today in Istanbul, while some of the examined buildings are quite tall, according to seismic regulation only 2 story high masonry buildings are permitted to be built. However, this permission is somehow only theoretical, since people do not prefer masonry construction in recent years in cities (maybe for last 30-40 years). Although average wall opening ratios of all examined buildings agree with regulations limits, some of the wall opening lengths exceed the 3 meters limit. In addition, the load-bearing wall length to floor area ratios do not satisfy the regulation in each direction. As it is seen from Table 16 for investigated buildings, generally this limit is satisfied only in one direction.

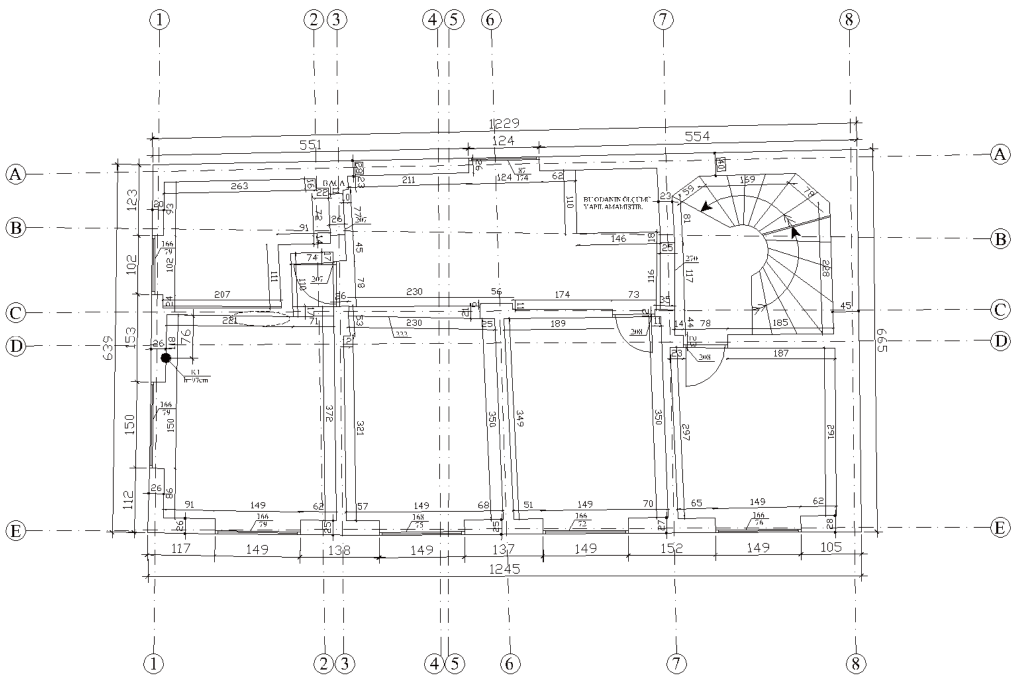


**Figure 18.** General plan view of one of the examined building in Istanbul

### 3.1.3 Stone constructions

Another type of wall structures in Turkey is stone constructions. The ashler stones and variable stone walls were used in some of traditional houses, historical monuments and mosques in the past. Today, it is hard to find a new

**Table 16.** Material and structural characteristics of Istanbul buildings

| Region | Building | Structural load bearings | Roof structure materials | Foundations | Height (m) | Minimum load bearing wall thickness (mm) | Wall length to floor area ratio ($m/m^2$) | | Average wall openings ratio |
|---|---|---|---|---|---|---|---|---|---|
| | | | | | | | x direction | y direction | |
| Yenikapi | 15 | Fired Brick Walls | Reinforced concrete slab | Strip stone masonry | 10.16 | 200 | 0.12 | 0.35 | 0.21 |
| | 35 | Fired brick and hollowed brick walls and reinforced concrete columns | Reinforced concrete slab | Strip stone masonry | 10.96 | 200 | 0.13 | 0.24 | 0.20 |
| | 2 | Fired Brick Walls | Reinforced concrete slab | Strip stone masonry | 11.60 | 200 | 0.21 | 0.42 | 0.19 |
| Sirkeci | 61 | Fired brick and hollowed brick walls | Reinforced concrete slab | Strip stone masonry | 16.42 | 200 | 0.11 | 0.30 | 0.21 |
| | 69 | Fired Brick Walls | Steel truss | Strip stone masonry | 15.07 | 230 | 0.13 | 0.22 | 0.18 |
| | 97 | Fired Brick Walls | Brick arch | Strip stone masonry | 16.40 | 200 | 0.12 | 0.25 | 0.27 |
| Uskudar | 6-1 | Fired Brick Walls | Reinforced concrete slab | Strip stone masonry | 2.35 | 200 | 0.32 | 0.35 | 0.18 |
| | 13 | Fired brick and hollowed brick walls | Reinforced concrete slab | Strip stone masonry | 4.75 | 200 | 0.14 | 0.28 | 0.35 |

construction with stone wall bearings (it can be only used in special buildings for aesthetical reasons).

The buildings old city of Bitlis was constructed with ashler stone materials [19] (Fig. 19). In addition, Denizli houses were constructed as variable stone wall type structures [16] (Fig. 20). The typical floor plans of these regional buildings are given in Figure 21 and 22, respectively. The detailed material characteristics of Bitlis and Denizli buildings are given in Table 17. It should be noted that the general mortar material used in stone walls is mud. In the buildings of Bitlis, these mud mortars were strengthened with thatch fibers. The geometrical and structural characteristics of these buildings are presented in Table 18. As can be seen from this table, the minimum wall thicknesses of Bitlis houses do not satisfy the regulation limit (for natural stones minimum thickness shall be more than 500 mm). Moreover, the average wall openings and total wall length to floor are ratios in each direction agree with the regulation.

**Figure 19.** Bitlis houses and ashler stone walls

(source:http://www.efsanevitopraklar.com/dosyalar/imajlar/imaj81/kultdeg0081.jpg)

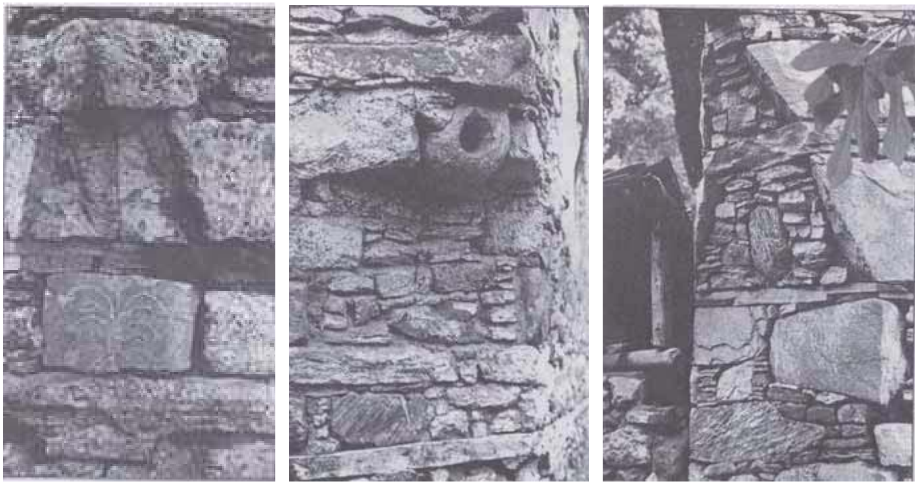

**Figure 20.** Variable stone wall in Denizli houses [16]

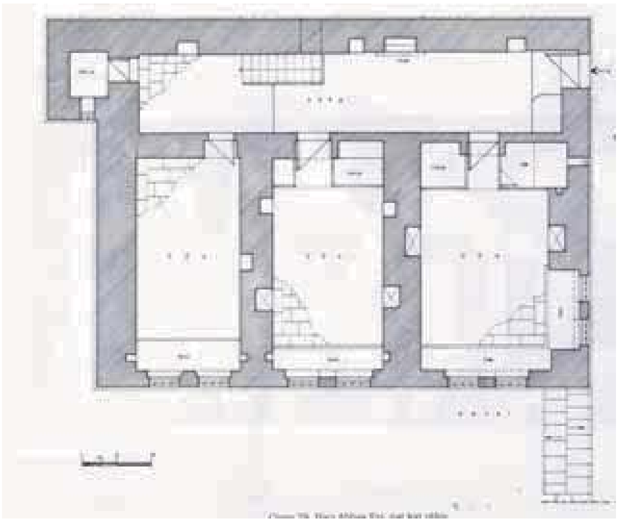

**Figure 21.** Typical floor plan of Bitlis houses [19]

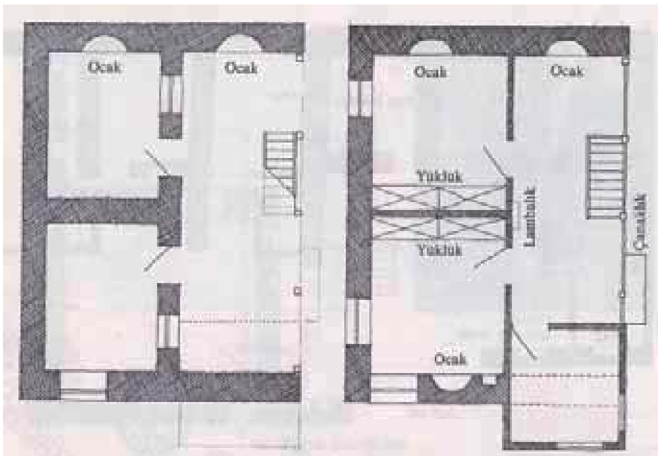


**Figure 22.** Typical floor plan of Denizli houses [16]

**Table 17.** Material characteristics of Denizli and Bitlis houses

| | Structural load bearings | Non-bearing partitions | Roof structure materials | Slabs | Mortar | Staircases |
|---|---|---|---|---|---|---|
| Denizli | Variable size stone walls | Stone and wooden walls | Wooden and wooden beams + tile roofs | Wooden beams + wooden flooring | Mud | Wooden |
| Bitlis | Uniform ashler stone walls | Stone walls or half timber frames with adobe | Wooden beams + soil roofs | Ashler stones or wooden flooring | Thatched mud | Wooden or stone |

**Table 18.** Structural characteristics of Denizli and Bitlis houses

| | Height of building | Minimum load bearing wall thickness | Wall length to floor area ratio (m/m$^2$) | | Average wall openings ratio | |
|---|---|---|---|---|---|---|
| | | | x direction | y direction | x direction | y direction |
| Denizli | ~7 m | 640 mm | 0.23 | 0.21 | 0.3 | 0.41 |
| Bitlis | 8.5 m | 400 mm | 0.37 | 0.33 | 0.3 | 0.25 |

### 3.2 Urban development law

Today in Turkey, reconstruction and planning laws details are given in following;

- Region plans are provided by state planning organization according to social economic developments, development potential of residential and trade and necessity of infrastructure distribution [21].
- In Turkey, development plans are consisted of regulatory development plan and application development plan. Officially district municipalities are responsible to comprise these two development plans in their boundary areas. After preparing of these development plans, they enter in force by approving of municipality council [21].
- Regulatory development plan is a document defining general usage of areas, general regions, intensity of population and constructions in the

next years, variable residential areas and their sizes, transportation systems and all expected problems in these issues. This document is consisted of map and prepared report in detail [21].

- Application development plan is a plan showing all details about city blocks, intensity of these blocks and their layouts (including permitted construction area and number of stories), road plans and compatible application stages of these roads depending on regulatory development plan [21].
- It should be noted that ministry of public works and settlement have authority to change development plans in some cases such as residential estates, urban transformation project and railway projects [21].

In construction stage, theoretically it is not possible to build a non-engineered construction today in Turkey. For engineering constructions, municipalities are also responsible about the building licenses authoritatively. In addition, some experienced and licensed independent engineering firms are technically responsible from the structural design and construction in-situ. Furthermore, chamber of civil engineering make an inspection on the design.

# 4 Earthquake experience of Turkey

## 4.1 History

While many catastrophic earthquakes hit different areas of Turkey in history, the first catastrophic natural disaster experienced by Republic of Turkey was the Erzincan Earthquake in 1939 (The magnitude of the earthquake was 7.8). In past 70 years, Turkey lived many different catastrophic earthquakes. The list of experienced earthquakes in Turkey (magnitude >5.0) from 1976 to present is given in Table 19 [5]. The additional statistics (number of deaths and injuries) in these earthquakes are also presented in this table.

**Table 19.** Turkey's earthquakes history [5]

| Place | Date | Mag. | Heavily damaged buildings | Moderately damaged buildings | Lightly damaged buildings | Deaths | Injuries |
|---|---|---|---|---|---|---|---|
| Denizli | 1976 | 5.0 | 887 | 2833 | 3887 | 4 | 28 |
| İzmir | 1977 | 4.8 | 11 | | | | |
| İzmir | 1977 | 5.5 | 40 | | | | |
| Balıkesir | 1979 | 4.9 | | | | | |

| | | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| Biga | 1983 | 6.1 | 85 | | | 3 | |
| Erzurum | 1983 | 6.8 | 3241 | 3007 | 4085 | 1155 | 1142 |
| Malatya | 1986 | 5.8 | 824 | 2539 | 4705 | 8 | 24 |
| Malatya | 1986 | 5.6 | 1174 | 313 | 458 | 1 | 20 |
| Erzincan | 1992 | 6.8 | 6702 | 9108 | 15384 | 653 | 3850 |
| İzmir | 1992 | 6.0 | | | | | |
| Dinar | 1995 | 5.9 | 4909 | 3276 | 6709 | 94 | 240 |
| Çorum | 1996 | 5.4 | 707 | 789 | 2080 | | 6 |
| Amasya | 1996 | 5.2 | | | | | |
| Antakya | 1997 | 5.5 | | 2709 | | | |
| Karlıova | 1998 | 5.0 | 69 | 79 | 878 | | |
| Adana | 1998 | 6.2 | 10675 | 20788 | 50663 | | |
| Kocaeli | 1999 | 7.4 | 66491 | | | 17408 | |
| Kocaeli | 1999 | 5.7 | | | | | |
| Marmara Adası | 1999 | 5.0 | | | | | |
| Düzce | 1999 | 7.2 | 15389 | 13548 | 13381 | 845 | 4948 |
| Denizli | 2000 | 5.2 | | | | | |
| Çankırı | 2000 | 5.9 | | | | | |
| Afyon | 2000 | 5.6 | 250 | | | 6 | 82 |
| Afyon | 2002 | 6.1 | 4401 | 1733 | 9785 | 42 | 325 |
| Tunceli | 2003 | 6.4 | 67 | 179 | 859 | 1 | |
| Bingöl | 2003 | 6.0 | 8142 | 4483 | 13277 | 184 | 515 |
| İzmir | 2003 | 5.6 | | | | | |
| Malatya | 2003 | 5.7 | | | | | |
| Denizli | 2003 | 5.3 | | | | | |
| Denizli | 2003 | 5.5 | | | | | |
| Denizli | 2003 | 5.0 | | | | | |

| | | | | | | | |
|---|---|---|---|---|---|---|---|
| Bingöl | 2004 | 5 | | | | | |
| Erzurum | 2004 | 5.1 | 1212 | | | 10 | |
| Erzurum | 2004 | 5.3 | | | | | |
| Muğla | 2004 | 5.1 | | | | | |
| Muğla | 2004 | 5.3 | | | | | |
| Elazığ | 2004 | 5.3 | | | | | |
| Muğla | 2004 | 5.1 | | | | | |
| Hakkari | 2005 | 5.4 | 82 | | | 3 | |
| Bingöl | 2005 | 5.6 | 760 | | | | |
| Bingöl | 2005 | 5.9 | | | | | |
| Izmir | .2005 | 5.8 | 96 | | | | |
| Izmir | 2005 | 5.5 | | | | | |
| Izmir | 2005 | 5.9 | 100 | | | | |
| Malatya | 2005 | 5.2 | | | | | |
| Bingöl | 2005 | 5.2 | | | | | |

### 4.2 Damages on wall type structures

In 1970, an earthquake with magnitude of 7.2 (Ms) on the Ricther scale occurred in Gediz which is a county of Kutahya (a city in Aegean region of Turkey). Before 1970, timber-framed building tradition was common in Gediz and newly built buildings were constructed with concrete skeleton and/or masonry systems including stone, brick or mud brick. According to official records, in that earthquake, 9452 buildings were damaged seriously. It caused loss of 1086 lives and 1260 injuries [22]. Most of damaged buildings were constructed as masonry or timber-framed with stone infill [22]. A damaged timber-framed building is given in Figure 23. It can be noted that there is less damage observed in the reinforced concrete buildings after that earthquake [22].

In 1976 Denizli experienced an earthquake with magnitude of 5.0 (Ms). As consequences of this earthquake, approximately 40 regional wall type buildings were collapsed and 1200 buildings were damaged heavily [23]. The typical failures of regional masonry buildings can be seen in Figure 24. It can be deduced from obtained reports about this earthquake, the pronounced reasons of damages are large openings in the walls and inadequate corner connections of masonry walls [23].

**Figure 23.** A damaged timber-framed building during Gediz earthquake [22]

**Figure 24.** Damages on regional buildings in Denizli [23]

In the Kocaeli earthquake of 17 August 1999 with 7.4 (Ms) magnitude, the number of seriously damaged or collapsed buildings was about 66.500, human loss was 17.500 and the number of wounded people was 32.000. The majority of building stock in the region before 1999 is the reinforced concrete or prefabricated systems since there is most important industrial region of Turkey. Therefore, while 99 % of the damaged buildings in that earthquake were constructed with reinforced concrete and prefabricated systems, minor part of damaged buildings was masonry, timber-framed and other types [22].

In 2004, a moderate earthquake with magnitude of 5.1 (Ms) was occurred in Dogubayazit (Agri) [24]. A majority of the buildings in the affected region was built as variable stone masonry construction. According to officials, 18 people were killed, 25 people were injured and 1000 building were damaged because of the earthquake [24]. Several damaged buildings can be seen in Figure 25, 26 and 27.

**Figure 25.** An heavily damaged building example from Dogubayazit [24]

**Figure 26.** A collapsed variable stone wall during Dogubayazit earthquake [24]

**Figure 27.** Damaged interior masonry walls in Dogubayazit [24]

Recent earthquake in Turkey is the Elazig-Kovancilar Earthquake (2010, Magnitude: 5.8). That earthquake caused loss of 40 people and thousands of homes. Most of these damaged or collapsed buildings were constructed with variable stone and adobe masonry walls. Typical damaged and collapsed buildings are given in Figure 28 and Figure 29 [25].

**Figure 28.** Damages on regional buildings in Elazig [25]

**Figure 29.** Damages on regional buildings in Elazig [25]

## 5 Current research outputs

In Turkey, scientific researches on masonry wall structures are generally conducted by universities. Some of conducted MSc and PhD theses about masonry wall structures are given below;

In 2007, a Master of Science thesis was conducted to find material behavior and for retrofitting of historical fired clay brick masonry. For this purpose, actual bricks, which are obtained from actual wall remains, were constructed and then retrofitted with FRP (fiber reinforced polymer). The dimensions of wall specimens were approximately 400x400x260 mm3. Two of the specimens were retrofitted with a plaster made of repair mortar, eight of the specimens were retrofitted with glass fiber polymers over the plaster of repair mortar and two specimens were tested as reference without any retrofit. Masonry wall specimens were tested under monotonic increasing or cyclic loading in diagonal [26]. Typical view of tested specimens are given in Figure 30.

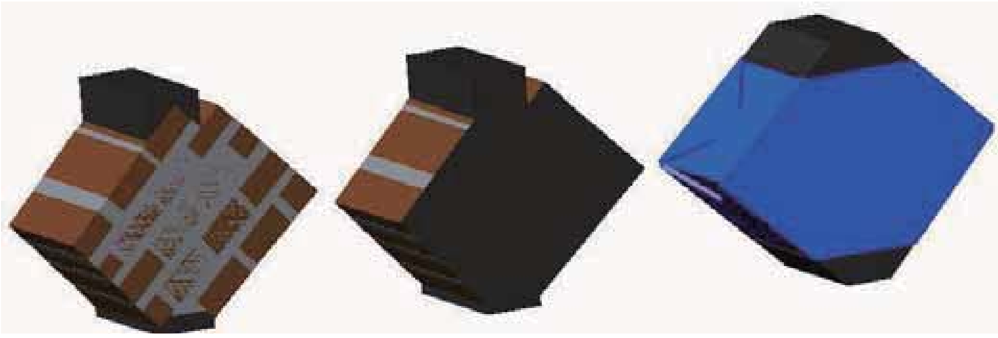

**Figure 30.** Diagonal fired clay brick wall specimens [26]

In 2010, a Doctorate thesis was published about fired clay brick wall behavior. In this study, a comprehensive experimental study conducted on historical masonry samples, which were obtained from a historical structure built in 19th century in İstanbul, is realized. In the frame of the thesis, mechanical characteristics of the historical masonry and its constituents are determined by testing different specimens in terms of size and composition under flexural tension, compression and shear loads. Several relations between the mechanical characteristics can be established based on the simple regression analysis [27]. Tested specimens are given in Figure 31, 32 and 33.

**Figure 31.** Core specimens [28]

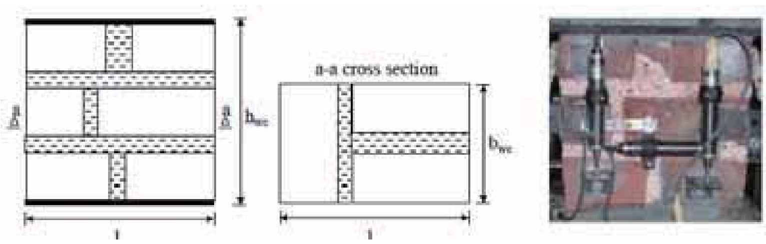


**Figure 32.** Wallet tests [28]

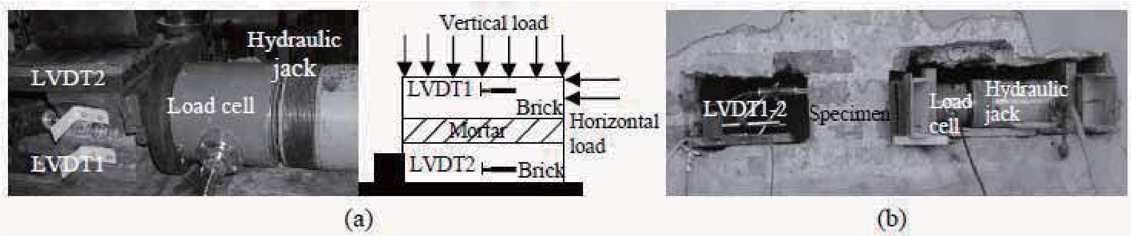


**Figure 33.** Shear tests (a) laboratory (b) in-situ [28]

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# Annex 8

# Country Report

# **U.S.A.**

**TABLE OF CONTENTS**

## LIST OF FIGURES

# 1. OUTLINE OF CODE FOR LOW-RISE WALL TYPE STRUCTURES

## 1.1 Introduction to Code Provisions for Low-rise Wall Type Structures in the USA

In contrast to many other countries, the United States has no national design codes, largely because the US Constitution does not specifically give the federal government the power to develop such a code. As a consequence, our US design codes come from a complicated process. Design provisions are developed by consensus technical committees; they are referenced by model codes; and they become law when they are adopted by local governmental jurisdictions.

Low-rise, wall-type structures in the United States of America (USA) are most commonly made of masonry (concrete or fired clay), which is generally reinforced. Adobe is used occasionally for residential structures only. A few low-rise commercial structures are made of reinforced concrete. In this section, code provisions for each of these are briefly reviewed; construction procedures are summarized; and earthquake performance is described. Subsections are organized according to each material type: adobe; unreinforced masonry; reinforced masonry; and reinforced concrete.

In the US code system, design requirements for masonry structures are provided in the Masonry Standards Joint Committee (MSJC) *Code* (MSJC 200a), and construction requirements are provided in the MSJC *Specification* (MSJC 2008b). Those requirements are referenced by the *International Building Code* (IBC) (IBC 2009), the dominant model code in the US (or another model code), and then become law when that model code is adopted by the governmental authority having jurisdiction over the geographic area where the building is located.

In the US code system, design requirements for concrete structures are provided in the American Concrete Institute (ACI) *Building Code Requirements for Concrete Structures* (ACI 318 2008), and construction requirements are provided in ACI 301 (*Specification for Concrete* (ACI 301 2009). Those requirements are referenced by the *International Building Code* (IBC) (IBC 2009), the dominant model code in the US (or another model code), and then become law when that model code is adopted by the governmental authority having jurisdiction over the geographic area where the building is located.

### 1.2 Code for Low-Rise Adobe Structures in the USA

Low-rise adobe structures are not addressed by the Masonry Standards Joint Committee (MSJC) *Code and Specification* (MSJC 2008a, 2008b), the source document for masonry design and construction in the USA. They are addressed by Section 2109 of the *International Building Code* (IBC 2009), which prescribes minimum strengths of the adobe material in compression and flexural tension, and also prescribes allowable stresses in compression. Adobe is used only for a small percentage of residential structures in the southwestern part of the US. It is not addressed further in this section.

### 1.3 Code for Low-Rise Unreinforced Masonry Structures in the USA

Design of unreinforced masonry structures in the USA is addressed by the MSJC *Code* (MSJC 2008a). Masonry is idealized as homogeneous, isotropic, and elastic. Design is permitted to be conducted by the allowable-stress approach or the strength approach. Because masonry is idealized as elastic in both approaches, they are in principle identical, and the factors of safety in the allowable-stress approach are adjusted so that both approaches result in essentially identical designs.

Unreinforced masonry is permitted to be used in walls loaded out-of-plane and in-plane. It is prohibited for beams and for columns.

In the MSJC *Code*, unreinforced masonry is defined according to design intent, rather than the simple absence of reinforcement. Prescriptive reinforcement may be required for seismic design, but it is not included in calculations of allowable or nominal capacity.

Allowable stresses and nominal resistances are based on a specified compressive strength of masonry. Compliance with that specified compressive strength is permitted to be verified by testing of masonry prisms (combinations of units and mortar), which are grouted (filled with fluid concrete) if the masonry that they represent is grouted. Compliance is also permitted to be verified without job-specific testing, using the compressive strength of the units as determined by the manufacturer and the prescribed proportions of the mortar and grout.

### 1.4 Code for Low-Rise Reinforced Masonry Structures in the USA

Design of reinforced masonry structures in the USA is addressed by the MSJC *Code* (MSJC 2008a). Design is permitted to be conducted by the allowable-stress approach or the strength approach.

In the allowable-stress approach, masonry is idealized as homogeneous, isotropic, and elastic. Steel reinforcement is idealized as elastic. Stresses in masonry and reinforcement are calculated using the principles of engineering mechanics for cracked, transformed sections. Nominal factors of safety are about 1.67 for failure modes governed by yield of reinforcement, about 2.5 for diagonal tension (shear) in masonry, and about 3.0 for compression in masonry.

In the strength approach, masonry is again idealized as homogeneous, isotropic, and elastic. Steel reinforcement is idealized as linear elastic, perfectly plastic. Nominal flexural capacity is calculated assuming plane sections, and compatibility of reinforcement and masonry. Tensile reinforcement is assumed to be yielded, and the nonlinear stress-strain behavior of masonry is idealized using an equivalent rectangular stress block. Nominal capacities, reduced by capacity-reduction factors (0.9 for flexure, 0.8 for shear), must equal or exceed the actions produced by factored design loads. Flexural reinforcement is limited so that the section is tension-controlled. Reinforced masonry is permitted to be used for all elements (walls loaded out-of-plane and in-plane, beams, and columns).

In the MSJC *Code*, reinforced masonry is defined according to design intent, rather than the simple presence of reinforcement. Prescriptive reinforcement may be required for seismic design, and is included in calculations of allowable or nominal capacity.

Allowable stresses and nominal resistances are based on a specified compressive strength of masonry. Compliance with that specified compressive strength is permitted to be verified by testing of masonry prisms (combinations of units and mortar), which are grouted (filled with fluid concrete) if the masonry that they represent is grouted. Compliance is also permitted to be verified without job-specific testing, using the compressive strength of the units as determined by the manufacturer and the prescribed proportions of the mortar and grout.

### 1.5 Code for Low-Rise Reinforced Concrete Structures in the USA

The design of reinforced concrete structures in the USA is addressed by ACI 318 (ACI 318 2008. Design is conducted by the strength approach.

The concrete is idealized as homogeneous, isotropic, and elastic. Steel reinforcement is idealized as linear elastic, perfectly plastic. Nominal flexural capacity is calculated assuming plane sections, and compatibility of reinforcement and masonry. Tensile reinforcement is assumed to be yielded, and the nonlinear stress-strain behavior of masonry is idealized using an equivalent rectangular stress block. Nominal capacities, reduced by capacity-reduction factors (0.9 for flexure, 0.75 for shear), must equal or exceed the actions produced by factored design loads. Flexural reinforcement is limited so that the section is tension-controlled; if not, a lower capacity-reduction factor must be used. Reinforced concrete is permitted to be used for all elements (walls loaded out-of-plane and in-plane, beams, and columns).

In ACI 318, reinforced concrete is defined according to design intent, rather than the simple presence of reinforcement. Prescriptive reinforcement may be required for seismic design, and is included in calculations of allowable or nominal capacity.

Nominal resistances are based on a specified compressive strength of concrete. Compliance with that specified compressive strength is required to be verified by testing of concrete cylinders.

### 1.6 Earthquake Loads for Low-Rise Structures in the USA

In the US code system, design earthquake loads are calculated according to Section 1613 of the 2009 *International Building Code* (IBC 2009). That section essentially references *ASCE 7* (*Minimum Design Loads for Buildings and Other Structures*), the latest edition of which was published in 2005 (ASCE7-05 2005). Seismic design criteria are given in Chapter 11 of that document. The seismic design provisions of ASCE 7-05 begin in Chapter 12, which prescribes basic requirements (including the requirement for continuous load paths) (Section 12.1); selection of structural systems (Section 12.2); diaphragm characteristics and other possible irregularities (Section 12.3); seismic load effects and combinations (Section 12.4); direction of loading (Section 12.5); analysis procedures (Section 12.6); modeling procedures (Section 12.7); and specific design approaches. Four procedures are prescribed: an equivalent lateral force procedure (Section 12.8); a modal response-spectrum analysis (Section 12.9); a simplified alternative procedure (Section 12.14); and a seismic response history procedure (Chapter 16). The equivalent lateral-force procedure is the most common, and is permitted in most situations.

## 2. OUTLINE OF CONSTRUCTION PRACTICE FOR LOW-RISE WALL TYPE STRUCTURES

### 2.1 Introduction to Construction Practice for Low-rise Wall Type Structures in the USA

In contrast to design codes (which have legal standing), construction regulations in the US are part of a civil contract. The designer is required to follow the legally adopted building code, such as the 2009 version of the *International Building Code* (IBC 2009). The 2009 IBC references the 2008 Masonry Standards Joint Committee (MSJC) *Code* (MSJC 2008a) and ACI 318-08 (ACI 318-08 2008). The MSJC *Code* in turn references the 2008 MSJC *Specification* (MSJC 2008b), which then becomes part of the construction documents for a masonry structure. Similarly, ACI 301 (*Specification*) becomes part of the construction documents for a concrete structure.

### 2.2 Construction of Low-Rise Adobe Structures in the USA

As noted previously, adobe construction is rarely used in the USA. Units are solid, and are unreinforced. Construction is governed by the quality-assurance requirements of the *International Building Code* (IBC 2009).

## 2.3 Construction of Low-Rise Unreinforced Masonry Structures in the USA

Low-rise unreinforced masonry in the USA is typically constructed with solid clay or concrete masonry units, laid with cementitious mortar.

## 2.4 Construction of Low-Rise Reinforced Masonry Structures in the USA

Low-rise reinforced masonry in the USA is typically constructed with hollow concrete masonry units, laid with cementitious mortar, and filled (partially or completely) with grout. Typical construction is shown in Figure 2.1.

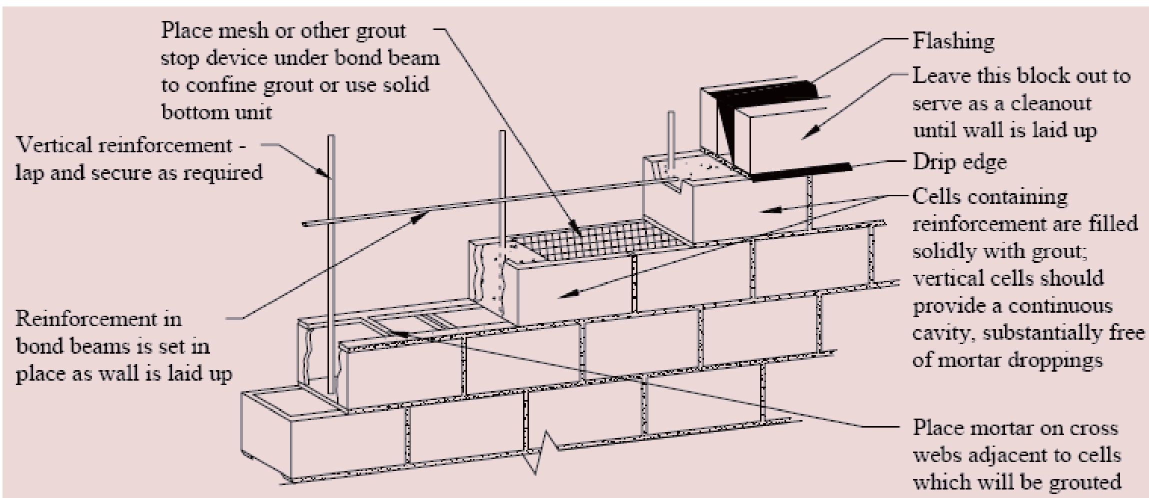


*Figure 2.1 Typical reinforced concrete masonry wall section*

## 2.5 Construction of Low-Rise Reinforced Concrete Structures in the USA

Low-rise reinforced concrete structures in the USA are typically constructed with multi-use metal formwork and prefabricated cages welded wire reinforcement. Other details are conventional.

# 3. EARTHQUAKE DAMAGE OF LOW-RISE WALL TYPE STRUCTURES

## 3.1 Introduction to Performance of Low-Rise Wall Type Structures in US Earthquakes

Performance of low-rise, wall-type structures in US earthquakes is generally distinguished by whether the structures are unreinforced or reinforced. Unreinforced masonry or concrete structures are not permitted in seismic regions of the US. For that reason, performance of unreinforced structures deals mainly with requirements for seismic rehabilitation. Low-rise, wall-type structures of reinforced masonry or concrete in the USA, designed according to current codes, have generally performed well in US earthquakes.

## 3.2 Performance of Low-Rise Adobe Structures in US Earthquakes

The earthquake performance of unreinforced adobe structures is known to be marginal at best. Because only a very few residential structures in the US are constructed of adobe, their performance is not discussed further here.

## 3.3 Performance of Low-Rise Unreinforced Masonry Structures in US Earthquakes

Because unreinforced masonry structures are not permitted in zones of high seismic risk in the US, our information on their performance comes from the performance of unretrofitted historic masonry. This is reviewed in the TMS report on the 1994 Northridge (Los Angeles, California, USA) earthquake (TMS Northridge 1994), and is summarized here.

For historical reasons that will be explained in this section, a summary of the historical performance of masonry in the United States can conveniently be divided into two periods: before the 1933 Long Beach Earthquake, and after that earthquake. In this section, that history is summarized, with emphasis on design implications.

The United States has several regions that have historically been recognized as having relatively high seismic risk. These include Alaska, Hawaii, California, parts of Montana and Idaho, the New Madrid area in southeast Missouri, and the Charleston, South Carolina area. This judgment is based on historical records of strong earthquakes there, throughout the past several centuries. Those early earthquakes did not cause significant damage to masonry buildings because few or no such buildings existed in seismic regions of the US until about the middle of the 1700's.

The Charleston earthquake of August 31, 1886, with an estimated Richter magnitude of 7.6, was felt from Cuba to New York, killed 110 people, and damaged 90% of the masonry buildings in Charleston (Bozorgnia and Bertero 2004).

The most destructive historical US earthquake was undoubtedly the San Francisco earthquake of April 18, 1906, which had an estimated Richter magnitude of about 8.0, and ruptured more than 400 km of the San Andreas Fault. The earthquake caused extensive damage to masonry buildings throughout the area, and San Francisco was almost completely destroyed by the combination of the earthquake and subsequent fire. Total damage was estimated at $500 million.

The earthquake that shook Long Beach, California on March 10, 1933, though having a Richter magnitude of only 6.3, caused 115 deaths and $40 million in property damage. While reinforced concrete buildings generally behaved well, unreinforced masonry buildings, including many school buildings, collapsed. The ensuing public outcry led to the passage, less than one month later, of the Field Act, which mandated earthquake-resistant design and construction for public schools in California, and prohibited the use of unreinforced masonry for such schools. Public opinion extended this prohibition to most other buildings as well.

When masonry construction was revived in California during the middle 1940's, it was required to comply with the new code provisions based on the reinforced concrete design practice of the time. The provisions required that minimum seismic lateral forces be considered in the design of masonry buildings, that tensile stresses in masonry be resisted by reinforcement, and that all masonry have minimum percentages of horizontal and vertical reinforcement. Those provisions led to the development of grouted, reinforced masonry constructed primarily of hollow concrete masonry units, which became the de facto standard for reinforced masonry in seismic regions of the United States up to the present.

On February 9, 1971, the San Fernando Valley (in the northwest portion of greater Los Angeles) was shaken by an earthquake that, although having a magnitude of only 6.7, produced extensive damage to modern buildings such as the new Olive View Hospital. It also produced extensive damage to unreinforced masonry. For example, the San Fernando Veterans Administration Hospital and complex, built in 1926, collapsed, causing 47 of the 58 deaths attributed to the earthquake. Failures in this earthquake of unreinforced masonry structures built before 1933 prompted the development of URM retrofitting ordinances.

Following the February 1971 San Fernando Earthquake, the City of Los Angeles, the Federal Government and the Structural Engineers Association of Southern California joined forces in a 10-year investigation. As a result of this investigation, in 1981 Los Angeles adopted an ordinance known as Division 68. Division 68 required seismic retrofitting of all unreinforced masonry bearing-wall buildings that were built, under construction, or for which a permit had been issued prior to October 6, 1933. The ordinance did not include one- or two-family dwellings or detached apartment houses comprising fewer than 5 dwelling units and used solely for residential purposes.

The 1985 edition of the *Los Angeles Building Code* revised Division 68 into Division 88, and included provisions for the testing and strengthening of mortar joints to meet minimum values for shear strength. Furthermore, Division 88 required that unreinforced masonry be positively anchored to floor and roof diaphragms with anchors spaced not more than 6 feet apart. It also imposed limitations on parapet height, based on wall thickness. Continuous inspection was also required on the retrofitting work. These retrofitting measures are shown schematically in Figure 3.1.

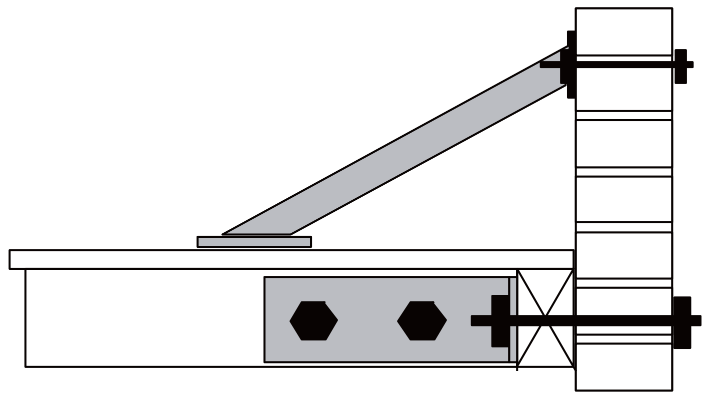

*Figure 3.1 Division 88 retrofitting requirements (Los Angeles, California, USA, 1988)*

At 5:04 p.m. on October 17, 1989, the San Francisco Bay area was shaken by an earthquake of Richter magnitude 7.1, whose epicenter was located about 10 miles northeast of Santa Cruz along a segment of the San Andreas Fault. Although damage to modern reinforced masonry buildings was generally low, many unretrofitted URM buildings experienced heavy damage. A large area of URM buildings in the Pacific Garden Mall in Santa Cruz collapsed. In the Marina District of San Francisco, a large region of unconsolidated fill, was the scene of many collapses of non-engineered houses and apartments with wooden frames and masonry veneer.

The Northridge earthquake, whose epicenter was located in the northwest part of the greater Los Angeles area, occurred at 4:31 a.m. on January 17, 1994. The earthquake had a moment magnitude of 6.7, and strong shaking lasted 15 to 20 seconds in the epicentral region. The following description is taken from The Masonry Society's report on the earthquake (TMS Northridge 1994).

The greater Los Angeles area contains tens of thousands of masonry structures, many of which were strongly shaken. In newer communities such as Northridge and Van Nuys (both in the epicentral region), the most common use of masonry by far was in one-story, reinforced masonry buildings, usually of fully grouted and reinforced hollow concrete block. Some multi-story, reinforced masonry bearing-wall structures were also found there, as well as steel or concrete frames with masonry veneer. In residential areas throughout Los Angeles, masonry site walls (landscaping walls) and brick chimneys were common. In older communities such as Hollywood, Santa Monica, and Pasadena (all 15 miles or more from the epicenter), unreinforced masonry (URM) structures, usually 2- or 3-story storefront buildings, are common. In accordance with the City of Los Angeles' Division 88 ordinance, most such structures have been retrofitted with parapet braces and floor-wall ties.

Unreinforced masonry buildings retrofitted in response to Division 88 requirements generally had parapet and wall damage, but did not collapse. Unreinforced masonry buildings without such retrofitting, in contrast, generally had more extensive damage, and some collapses.

### 3.4 Performance of Low-Rise Reinforced Masonry Structures in US Earthquakes

Performance of low-rise reinforced masonry structures in US earthquakes is reviewed in the TMS report on the 1994 Northridge (Los Angeles, California, USA) earthquake (TMS Northridge 1994), and is summarized here.

On February 9, 1971, the San Fernando Valley (in the northwest portion of greater Los Angeles) was shaken by an earthquake that, although having a magnitude of only 6.7, produced extensive damage to modern buildings. Failures in this earthquake of unreinforced masonry structures built before 1933 prompted the development of URM retrofitting ordinances, discussed previously in this section.

The Northridge earthquake, whose epicenter was located in the northwest part of the greater Los Angeles area, occurred at 4:31 a.m. on January 17, 1994. The earthquake had a moment magnitude of 6.7, and strong shaking lasted 15 to 20 seconds in the epicentral region. The following description is taken from The Masonry Society's report on the earthquake (TMS Northridge, 1994).

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Throughout Los Angeles were many reinforced masonry schools, post offices, fire stations, and police stations. Most of these buildings showed little apparent structural damage, and continued operating after the earthquake.

In the greater Los Angeles area, and particularly in the epicentral region, very little distress was shown by modern one-story reinforced masonry, or by multi-story, reinforced bearing-wall buildings. In some cases, however, masonry veneer was attached using connection details that were inadequate to resist the required inertial forces.

In general, masonry structures built since the 1950's that were engineered, grouted, reinforced, and inspected in accordance with then-current building codes, experienced little damage in the January 17, 1994 earthquake.

Since the 1940's, masonry structures in the western part of the US have generally been designed and constructed with minimum prescriptive requirements for reinforcement that are similar to those required in higher seismic design categories today. Such buildings have experienced little damage in US earthquakes.

### 3.5 Performance of Low-Rise Reinforced Concrete Structures in US Earthquakes

Low-rise, wall-type structures of reinforced concrete in the USA, designed according to current codes, have generally performed well in US earthquakes.

# 4. CURRENT RESEARCH OUTPUTS

## 4.1 Introduction to Current Research on Low-Rise Wall Type Structures in the USA

Because low-rise, wall-type structures designed and constructed in accordance with current requirements have performed well in recent US earthquakes, current research is devoted primarily to refinement of those existing requirements.

## 4.2 Current Research on Low-Rise Adobe Structures in the USA

Because adobe structures are used only rarely in the USA, there is no significant current structural research involving adobe in the USA.

## 4.3 Current Research on Low-Rise Unreinforced Masonry Structures in the USA

Because unreinforced masonry structures are prohibited in zones of significant seismic risk in the USA, there is no significant current structural research involving unreinforced masonry in the USA.

## 4.4 Current Research on Low-Rise Reinforced Masonry Structures in the USA

Current research on low-rise reinforced masonry structures is oriented toward the refinement of design and construction provisions. It is exemplified by the research noted in the 6$^{th}$ edition of The Masonry Society's *Masonry Designers Guide* (MDG 2010), and by the coordinated research program described in Klingner *et al.* (2010). Figure 4.1, taken from that reference, shows a full-scale reinforced concrete masonry building that was tested on a shaking table to levels of ground acceleration well in excess of that corresponding to collapse levels in US codes. It exhibited little distress, and its performance correlated well with the results of quasi-static testing and nonlinear analytical predictions.

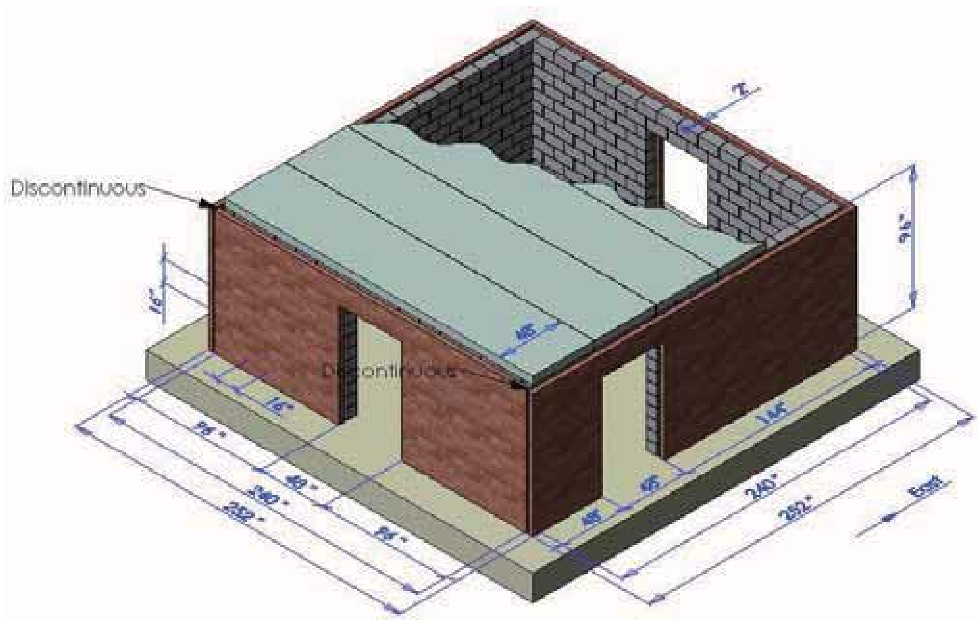


*Figure 4.1 Prototypical low-rise building with concrete masonry backing and clay masonry veneer (Klingner et al. 2010)*

## 4.5 Current Research on Low-Rise Concrete Structures in the USA

Because the behavior of low-rise reinforced concrete structures is well described by current design models, and because such structures have shown little distress in strong earthquakes, there is no significant current research involving low-rise concrete structures in the USA.

# 5. FUTURE WORKS

## 5.1 Introduction to Future Research on Low-Rise Wall Type Structures in the USA

Because low-rise, wall-type structures designed and constructed in accordance with current requirements have performed well in recent US earthquakes, future research will probably be devoted primarily to refinement of those existing requirements.

## 5.2 Future Research on Low-Rise Adobe Structures in the USA

Because adobe structures are used only rarely in the USA, no future structural research involving them is planned.

## 5.3 Future Research on Low-Rise Unreinforced Masonry Structures in the USA

Because unreinforced masonry structures are prohibited in zones of significant seismic risk in the USA, no future structural research involving them is planned.

## 5.4 Future Research on Low-Rise Reinforced Masonry Structures in the USA

Future research on low-rise reinforced masonry structures is oriented toward the refinement of design and construction provisions, principally displacement-based design. This research has just begun, and will be described in future publications.

## 5.5 Future Research on Low-Rise Concrete Structures in the USA

Because the behavior of low-rise reinforced concrete structures is well described by current design models, and because such structures have shown little distress in strong earthquakes, no significant future research is planned involving low-rise concrete structures in the USA.

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