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BUILDING CONSTRUCTION UNDER SEISMIC CONDITIONS IN THE BALKAN REGION

VOLUME 3

DESIGN AND CONSTRUCTION OF STONE AND BRICK-MASONRY BUILDINGS

BUILDING CONSTRUCTION
UNDER SEISMIC CONDITIONS
IN THE BALKAN REGION

UNDP/UNIDO PROJECT RER/79/015

VOLUME 3

DESIGN AND CONSTRUCTION OF STONE AND BRICK-MASONRY BUILDINGS

UNITED NATIONS INDUSTRIAL DEVELOPMENT ORGANIZATION
executing agency for the
UNITED NATIONS DEVELOPMENT PROGRAMME
Vienna, 1984

VOLUME 3

DESIGN AND CONSTRUCTION OF STONE AND BRICK-MASONRY BUILDINGS

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PREFACE

The Regional Project "Building Construction under Seismic Conditions in the Balkan Region", UNDP/UNIDO RER/79/015, has been carried out with the participation of the Governments of Bulgaria, Greece, Hungary, Rumania, Turkey and Yugoslavia, and with the United Nations Industrial Development Organization acting as Executing Agency for the United Nations Development Programme. Mr. J.G. Bouwkamp served as Chief Technical Advisor.

Within the framework of the Project, a set of seven Manuals has been produced, reflecting to a considerable extent the experience of the participating nations in earthquake resistant design and construction. These Manuals were developed by the National Delegates of the Project Working Groups, the Chief Technical Advisor and the Consultants.

The following Manuals have been prepared:

Volume 1 : Design and Construction of Seismic Resistant Reinforced Concrete Frame and Shear-Wall Buildings

Volume 2 : Design and Construction of Prefabricated Reinforced Concrete Building Systems

Volume 3 : Design and Construction of Stone and Brick-Masonry Buildings

Volume 4 : Post-Earthquake Damage Evaluation and Strength Assessment of Buildings under Seismic Conditions

Volume 5 : Repair and Strengthening of Reinforced Concrete, Stone and Brick-Masonry Buildings

Volume 6 : Repair and Strengthening of Historical Monuments and Buildings in Urban Nuclei

Volume 7 : Seismic Design Codes of the Balkan Region

ACKNOWLEDGEMENTS

The financial support of the United Nations and the Governments of Bulgaria, Greece, Hungary, Rumania, Turkey and Yugoslavia should hereby be acknowledged. Also the cooperation of the National Science Foundation of the USA is noted with appreciation.

The Project has been directed by the Project coordinating Committee. The membership of the Committee was as follows:

Bulgaria	G. Brankov
Greece	Th. Tassios and J. Sbokos
Hungary	F. Hunyadi
Rumania	V. Cristescu
Turkey	M. Erdik
Yugoslavia	N. Vukotic

From the United Nations the following individuals participated in the deliberations of the Coordinating Committee:

O.A. Nordstrand - UNDP, Athens, Greece, Resident Representative and UNDP Principal Project Representative,
E. Csorba - UNIDO, Vienna, Austria, Senior Industrial Development Officer, and
J.G. Bouwkamp - UNIDO, Thessaloniki, Greece, Chief Technical Advisor.

DISCLAIMER

The material contained in these Volumes includes detailed findings in earthquake engineering - particularly, objective evaluations of causes and effects in earthquake damage - and in the seismic and geologic characteristics of the physical environment.

The material reflects the interpretation and opinions of the authors and contributors and does not necessarily represent the viewpoint of the United Nations Industrial Development Organization, the United Nations Development Programme, the participating Governments and the National Science Foundation of the USA. The above mentioned Governments and Organizations - while providing for the presentation of these Volumes in the public interest and for their obvious informational value - assumes no responsibility for any views expressed therein.

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NOTE

This Manual is the third volume of the seven volumes developed under the UNDP/UNIDO Project RER/79/015 "Building Construction Under Seismic Conditions in the Balkan Region" and was prepared by the Project Working Group C on "Brick, Stone-Masonry and Adobe Buildings." It is aimed to provide engineers in the Balkan region, as well as in other earthquake-prone countries, with information pertinent to the analysis, design and construction of earthquake resistant masonry buildings.

In the Manual both qualitative and quantitative instructions for architectural and structural building configurations are given, materials and construction systems to be used so as to achieve a satisfactory aseismic behavior are specified and methods for analysis of masonry buildings and design of structural elements are provided. Additionally, three design examples are presented reflecting the existing practices in the Balkan countries.

The contents of this Manual are based on the National Reports of the participating countries and were developed in Working Group meetings together with the Project Chief Technical Advisor and the Consultant. The Working Group met three times, namely, in Istanbul (in March 1982 and December 1982) and in Thessaloniki (in March 1983). During the first meeting the scope of the Manual was defined and a general outline for the preparation of the National Reports was established. During the second meeting the National Reports were presented in a two-day joint seminar, together with Project Working Group F on "Assessment of Earthquake Resistance, Strengthening and Repair of Cultural and Historical Monuments in Urban Nuclei."

Subsequently, together with the Consultant, the members reviewed the reports and formulated the Manual contents. During their third meeting, a first draft of the Manual was reviewed and a final format was agreed upon.

The Working Group consisted of National delegates of the participant countries with Dr. Mufit Yorulmaz, Professor, Faculty of Engineering and Architecture, Istanbul Technical University, Istanbul, Turkey, serving as Convenor. Participating members of the Working Group were: Dipl. Eng. Elizabeth Vintzēleou, Assistant, National Technical University, Athens, Greece; Dr. Endre Dulacska, Structural Chief Engineer, Institute of Building Types Design, Budapest, Hungary; Florin-Ermil Dabija, Associate Professor, Institute of Constructions, Faculty of Civil Engineering, Bucarest, Rumania. Consultant of the Working Group was Mr. Miha Tomazevic, Senior Research Engineer, Institute for Testing and Research in Materials and Structures, Ljubljana, Yugoslavia.

Professor Jack G. Bouwkamp, University of California, Berkeley, California, USA, served as Project Chief Technical Advisor and participated in all Working Group discussions.

1. INTRODUCTION

Although masonry is used for construction of buildings since ancient times, our knowledge on its behaviour under seismic conditions is still not sufficient. Until recently, masonry was not considered to be a suitable material for building construction in seismic zones. This was mainly due to the consequences of earthquakes durign the last decades: most losses of life have occurred due to the collapse or heavy damage of masonry buildings.

However, the analysis of damage, suffered after earthquakes, as well as the research, carried out in many countries, have clearly shown the deficiencies of the collapsed and heavily damaged buildings, and indicated the appropriate technical measures to be taken in order to improve the behaviour of masonry buildings when subjected to earthquakes as well.

This Manual is based on the experiences of Balkan countries concerning masonry construction under seismic conditions as well as on the results of research work, carried out both in some Balkan countries and in other parts of the world. The aim of this Manual is to present the state of the art of masonry construction in the Balkan region, as well as to provide the designer with useful data for alternative design and construction of earthquake resistant masonry buildings by introducing reinforced masonry.

In the Manual, first the behaviour of masonry buildings during earthquakes is discussed. Observed damage patterns are analysed and consequently failure mechanisms and causes of failure are defined.

Then, the fundamental principles of architectural and structural concepts for desirable building configuration are presented, and some information about the constitutent materials of masonry walls is given. Masonry construction systems as used in the region are described, and reinforced masonry is briefly introduced as possible alternative.

In the following, design assumptions and procedures are explained. Seismic actions are defined and sectional capacities of wall elements are evaluated (limit state design method).

Finally, instructions are given for construction and design of other structural and non-structural elements, such as foundations, floors, tie-beams, roofs, partitions, and chimneys.

The Manual which reflects the present state of masonry construction in the Balkan region, but also shows some possibilities of improving masonry construction, does not represent a Code. Specific numbers, as given in the Manual, should therefore be only seen as general recommendations.

2. BEHAVIOUR OF BUILDINGS DURING RECENT EARTHQUAKES

2.1 Characterisation of Buildings

Masonry has been used for construction of buildings since ancient times. Nowadays, despite the extensive use of modern construction materials like reinforced concrete and steel, masonry buildings still represent a great majority of both residential and public buildings, constructed in the Balkan region.

According to the functional requirements, available materials, technical knowledge and traditional practices specific to different countries and different construction periods, a wide variety of different kinds of masonry buildings exist in the Balkans. Distinction can be made according to the materials used for construction (stone, adobe, brick, ceramic blocks), place of construction (urban or rural areas), period of construction (prior to World War One, between the two World Wars, post-war period, after the adoption of aseismic regulations), use of buildings (residential, public), and structural system.

Stone and adobe still are widely used as construction material for residential buildings in certain parts of the region. The same materials were also used in urban nuclei. The buildings of this type usually have a regular structural layout, with thick walls uniformly distributed in both directions, although not all are being load-bearing walls and tie-beams are often omitted. The number of stories does not exceed 2 in rural areas, and 3-4 in urban areas. Poor quality clayey sand or mud mortar are still used for construction. Floors are usually of timber joists, not anchored to the walls. Sometimes wooden floors are replaced by stone or brick vaults.

Brick masonry is used as a construction material for residential and public buildings in urban areas from the last half of the XIX. century onwards. The structural layout, especially in the case of public buildings, is frequently irregular, with many offsets and setbacks. Poor quality lime-sand mortar is often used for construction. Floors are usually of timber, but sometimes brick vaults supported by steel beams, or reinforced concrete slabs can be found. After the World War One, reinforced concrete tie beams were introduced, increasing the number of stories to 6-7, with story heights about 3-4 m.Mixed structural systems are found, using reinforced concrete columns as inner load bearing elements. In that case, the number of stories is relatively small (2-4), but the story heights reach 5-7 m.

During the post-war period the construction of apartment buildings, up to 6 stories high, having load bearing walls in the transversal direction only with prefabricated floor elements became very popular. Longitudinal walls have been weakened by many openings and were not considered to participate in the load-bearing system. Again not much attention has been paid to the quality of materials and construction.

Earthquake protection regulations, adopted during the 1950-60 period, introduced several measures to improve the earthquake resistance of masonry buildings. These regulations required the use of r.c. horizontal tie-beams and the uniform distribution of walls in both orthogonal directions.

Depending on the building height and seismic intensity these regulations further required the introduction of r.c. tie-columns. They also stipulated the quality requirements of materials.

2.2 Building Performance

Considering the response to earthquakes, adobe and stone masonry buildings suffered severe damage. Because of insufficient interlocking between intersecting walls and lack of anchorage between walls and floors, cracks between intersecting walls were often observed. In many instances separation of walls and even collapse out of the plane of the walls did occur. Also,despite the sometimes favourable structural layout,the quality of wall materials was often insufficient to prevent the walls from diagonal cracking, disintegration and ultimate collapse.

In case of old brick masonry buildings, the unfavourable effect of insufficient anchorage between walls and floors was also observed. Irregular structural layout in plan, large openings, lack of bearing walls in both orthogonal directions often caused severe damage or even collapse of many buildings. The unsatisfactory behaviour of brick masonry buildings often resulted from the poor quality of materials used for construction, especially mortar.

The behaviour of mixed structural type buildings was very poor, because the lateral load resisting system was concentraded in outer walls, often perforated by many openings. The same poor behaviour has been observed in buildings with load bearing walls in one direction only.

Masonry buildings, designed and constructed according to seismic codes, behaved more or less satisfactory. Cases of collapse were rare and heavy damage occurred rather unfrequently. New buildings, designed and constructed completely according to the requirements of aseismic regulations,including material requirements,suffered minor damages. However,where the requirements of aseismic protection were only met partly, and the quality of construction was inadequate, damages of various degrees occur. For instance the observations have shown, that the vertical reinforced concrete tie-columns prevented the buildings from collapse in many cases, even if the walls were seriously damaged because of the bad quality of materials and construction.

3. ANALYSIS OF DAMAGE PATTERN AND POSSIBLE CAUSES OF FAILURE

3.1 Classification of Damage Patterns

Of the great number of masonry buildings subjected to strong earthquakes many were severely damaged, some collapsed, however some survived earthquakes only slightly damaged or even undamaged.Obviously the observations of such kind alone cannot assess the potential earthquake resistance of masonry buildings in general. Therefore, an analysis of the observed damage pattern is essential.

Although the structural typology of masonry buildings varies in different countries such as buildings being constructed in traditional material like stone, adobe, brick, with no special provision against earthquakes and buildings designed to resist the earthquake forces, using modern materials like ceramic or concrete blocks, the observed types of damages allow classification into a common set of patterns.

The following typical patterns of damage of structural walls for different types of masonry buildings, subjected to earthquakes of different intensities, have been observed:

a. horizontal cracks between walls and floors (Fig.3.1);

b. vertical cracks at the joints or intersecting walls (Fig.3.2);

c. separation of peripheral walls, (Fig.3.3);

d. out of plane collapse of peripheral walls, (Fig.3.4 and 3.5);

e. cracks in spandrel walls (Fig.3.3);

f. diagonal cracks in wall piers (Fig.3.6, 3.7 and 3.8);

g. partial disintegration or failure of walls (Fig.3.9, 3.10 and 3.11);

h. partial or complete collapse of the building (Fig.3.12).

The analysis of damage patterns can clearly identify the weak and the good points of the different structural systems. On the base of the analysis of damage, the failure mechanism of wall elements as well as of the complete structural systems can be defined. However, quantitative data on the seismic action or on the seismic resistance of the buildings cannot be defined unless additional investigations to establish the material properties are performed.

3.2 Failure Mechanisms

When the building is subjected to earthquake motion, inertia forces, proportional to the masses of the structural system are induced. Since the ground motion is generally tridirectional both vertical and horizontal inertia forces will be acting on the structure, changing in time, and resulting in the tridimensional vibrations of the building. The structural elements which were carrying basically vertical loads before the earthquake, must carry horizontal loads as well, causing additional bending and shearing effects.

The observations of the behaviour of masonry buildings when subjected to earthquakes have shown that the vibrations of the building as a whole are strongly dependent on how the walls are interconnected and anchored at the floor and roof levels.

In the case of old masonry buildings where the timber joist are not anchored to the masonry (i.e. without ties of tie-beams) the individual walls tend to separate along their joints or intersections. Vertical cracks in the walls occur near the corners either in the side wall which experiences serious out of plane bending distortions or in the adjacent end wall when the tensile strength in that wall is insufficient to withstand the inertial forces of the side wall. Under those conditions the vibrations of the walls become uncoupled and peripheral walls might collapse (Fig.3.13-a and -b).

In the case where ties are placed or reinforced concrete tie beams are cast at the floor levels, the vibrations of the walls become synchronized (Fig.3. 13-c).However, in this case, the out of plane bending of walls takes place again, reducing the resistance of the building as a whole.

Obviously the behaviour of masonry buildings is improved when the walls are connected together by means of rigid reinforced concrete slabs with tie-beams on the top of the walls. In this case the vibrations of walls are synchronized (Fig. 3.13-d),and the out plane bending of walls is less significant, because the walls are almost rigidly supported on all four boundaries. The building behaves like a box and all the walls contribute to the resistance of the building.

On the basis of the analysis of typical damage patterns, the following can be concluded concerning the failure mechanisms of the wall as a structural element:

- the free standing wall is not stable to the out of plane forces (Fig.3.14-a). Usually the wall resists inertia forces by its own weight as well as by its flexural resistance. Unless the wall is vertically reinforced, the flexural resistance is to small to prevent the wall from failing out of plane after the horizontal tensile cracks occur at the bottom part of the wall.

- on the other hand, when the free standing wall is subjected to in plane forces, its resistance will be much greater, the wall becomes a "shear wall". There are several possibilities of failure of such a wall, which depend on its geometry (height to length ratio) and material characteristics: a.) horizontal sliding or pure shear failure occurs when a horizontal crack develops along the mortar joint (Fig.3.14-b); b.) diagonal tension or shear failure occurs when diagonal cracks either following the mortar joints or passing through the units develop in the wall (Fig.3.14-c); c.) flexural failure occurs when crushing of the compressed zones occurs after the horizontal, tensile cracks on the tensioned side have diminished the effective compressive cross-sectional area of the wall (Fig.3.14-d). Of course, failure mechanism could include various combinations of those described above.

Inertia forces induced in the structure due to ground accelerations tend to deform the building (Fig.3.15). The piers between the openings are more flexible than the portion of wall below or above the openings, so practically all deformations take place in the piers. In the sections below and above the openings the piers are carrying the maximum compressive or tensile stresses; whereas in the mid-height sections, they are carrying the maximum shear stresses. The magnitude of the stresses depends on the magnitude of the horizontal inertia force as well as on the magnitude of vertical load, carried by the wall.

If the tensile stresses become greater than the tensile strength of the wall, cracks develop, thereby reducing the effective compressive cross-sectional area of the wall. Depending on the geometry and material characteristics, the pier may subsequently fail either in shear due to diagonal tension or in flexure due to the vertical compression at the compressed corners, whichever comes first.

The idealized stress state in a pier is shown in Fig. 3.16, shere:

σ_o - compressive stress due to vertical load,

σ_M - compressive or tensile stress due to the overturning moment,

σ_f - compressive or tensile bending stress due to the bending of piers between openings,

τ - shear stress in the pier.

In the case of old multistory buildings with flexible wooden floors without tie beams provided at the floor levels, the spandrel beam between the two openings one above the other is a potential weak point of the structure (Fig. 3.17).

As it is the case of window pier, the spandrel beam is subjected to the combination of bending and shear, too. Often it fails in shear due to diagonal tension before the diagonal cracking of pier takes place, which results in a loss of support for the floor structure.

The cracking of the spandrel beam can be avoided by either a rigid reinforced concrete slab or reinforced concrete tie beams provided at the floor levels.

Simultaneously the inertia forces cause the out of plane bending of walls,and as a consenquence vertical bending cracks might develop in the critical sections of the wall.

Gable end masonry walls can be unstable. Unless properly anchored they behave like free standing walls. The strutting action of the roof purlins imposes additional forces that can contribute to failure.

3.3 Causes of Failures

As it has already been pointed out there are two fundamental factors which strongly influence the earthquake resistance of masonry buildings, namely

- structural layout, and
- quality of materials and construction.

Good behaviour of masonry buildings is guaranteed if walls are uniformly distributed in both orthogonal directions, there are no sudden changes in stiffness and the walls are tied together at the floor by means of rigid floors and r.c. tie-beams. Hovever, good quality of materials and construction methods are also necessary to guarantee good behaviour of masonry buildings.

3.3.1 Deficiencies in Structural Layout

Concerning the structural layout of masonry buildings the following main deficiencies were observed:

. Irregular distribution of walls in plan.This can result in torsional effects and overstressing the walls in critical zones. Buildings with load bearing walls in one direction only represent an extreme example of bad structural layout. Peripheral walls in the other direction, perforated often by many openings, can not resist earthquake forces. After the perforated walls failed in shear, the transversal walls behaved like free standing walls, failed in out-of-plane bending and the building collapsed (Fig.3.12).

. Nonuniform distribution of stiffness over the height of the building. This can result in severe damage of walls at the levels of sudden changes in stiffnesses.

. Lack of rigid floors with out ties or tie-beams. This can result in vertical cracks at the joints or intersections of walls due to the out-of-plane bending of walls. The walls separate one from another and may collapse even when the building is subjected to moderate earthquakes. Flexible floors may also cause cracking of spandrel beams.

3.3.2 Quality of Materials and Construction

Although sufficient for carrying the gravity loads, mechanical characteristics of materials used for construction of masonry buildings, especially of mortar, were not sufficient to resist additional bending and shearing effects, induced in the structural system by the earthquake.

Very low tensile strength which characterises the shear failure of wall element is especially evident in the case of old stone masonry and adobe buildings, where clayey sand and mud mortars were used for construction.

Poor quality of materials and construction was also one of the main reasons for structural damage of buildings, designed according to seismic codes. Inefficient quality control allowed that bricks, blocks and mortar with mechanical characteristics inferior to those specified were used for construction. The quality of construction was also found to be inadequate: excessive thickness of horizontal joints, vertical joints not being filled with mortar, bricks not being soaked into water before the construction, etc.

The influence of the quality of materials and construction on the resistance of masonry buildings is illustrated in Figs.3.18 and 3.19.

The two buildings were constructed at the same site separated by approximately 6 m. The rough construction works were finished at the time of the earthquake. The building shown in Fig.3.18, constructed with hollow modular clay blocks in cement-lime-sand mortar of good quality, survived the earthquake with no damage at all. The building in Fig.3.19, however, constructed with clay bricks and lime mortar of very poor quality, suffered severe damage.

3.3.3 Failure of Ground

During the recent earthquakes some cases were observed where the failure of ground was the main cause of the damage or collapse of masonry buildings. Two kinds of ground failure were the most frequent:

. sliding of slopes. This results in differential settlements or moving of foundations, and hence building separating or tilting of the building (Fig. 3.20);

. liquefaction of loose water-saturated sand. This results in settlements of foundations, and tilting or sinking of the building into the ground (Fig. 3.21).

Fig.3.1 - Kozjansko,1974 Earthquake (Yugoslavia): Typical cracks between walls and floor structure;rural stone-masonry dwelling house

Fig.3.2 - Magnessia,1980 Earthquake (Greece): Vertical cracks at the joints of walls; rural stone-masonry dwelling house

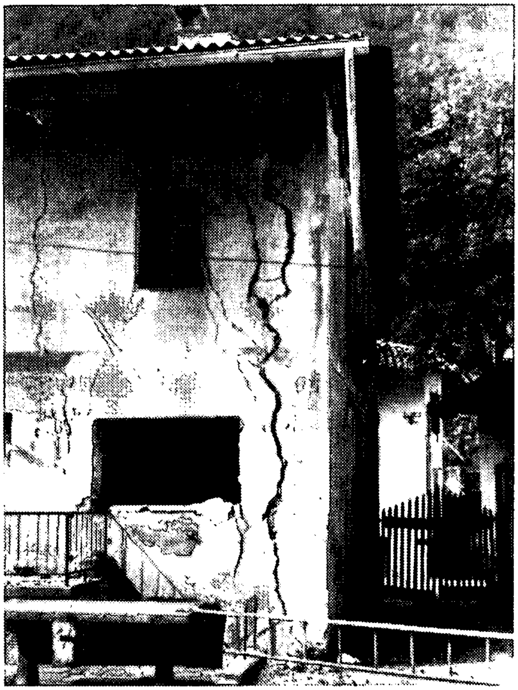

Fig.3.3 - Friuli,1976 Earthquake (Yugoslav territory):Separation of walls,cracks in spandrel beam; rural stone-masonry dwelling house

Fig.3.4 - Thessaloniki,1978 Earthquake (Greece): Overturning of a head wall; rural stone-masonry dwelling house

Fig.3.5 - Friuli,1979 Earthquake (Yugoslav territory): Overturning of a head wall; rural stone-masonry house

Fig.3.6 - Montenegro,1979 Earthquake (Yugoslavia):Typical diagonal cracks in window piers; brick masonry building

Fig.3.7 - Lice,1975 Earthquake (Turkey): Typical diagonal cracks in window piers; brick masonry public building

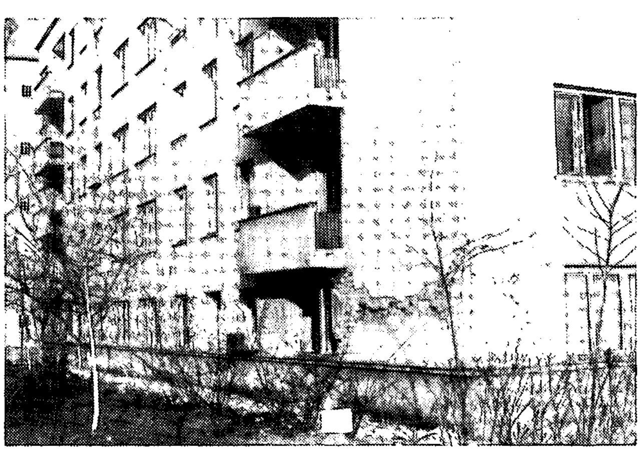

Fig.3.8 - Vrancea,1977 Earthquake (Romania):Typical diagonal cracks in window piers; brick masonry residential building

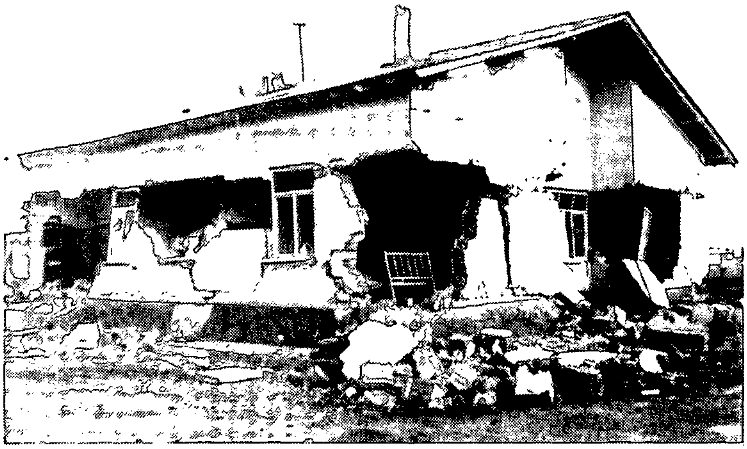

Fig.3.9 - Caldiran,1976 Earthquake (Turkey):Partial collapse of window piers and corner zone; brick masonry public building

Fig.3.10 - Corinth,1981 Earthquakes (Greece):Partial collapse of walls and corner zone;stone-masonry rural dwelling house

Fig.3.11 - Friuli,1976 Earthquake (Yugoslav territory):Disintegration of a stone-masonry wall;stone-masonry dwelling house

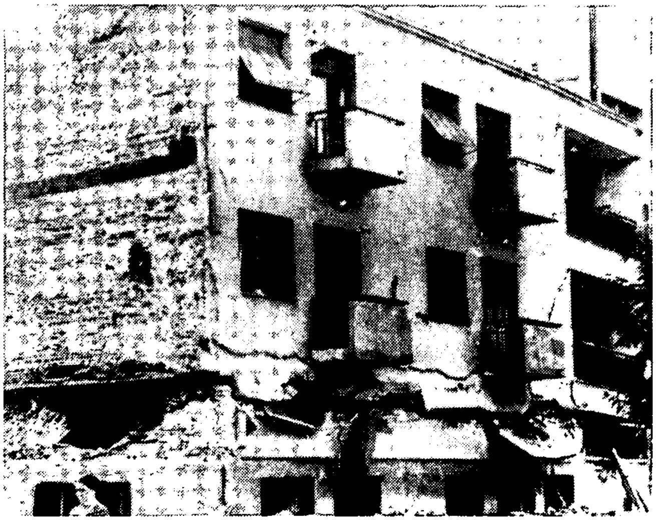

Fig.3.12 - Skopje,1963 Earthquake (Yugoslavia):Collapse of the building,having transversal load-bearing walls; brick masonry residential house

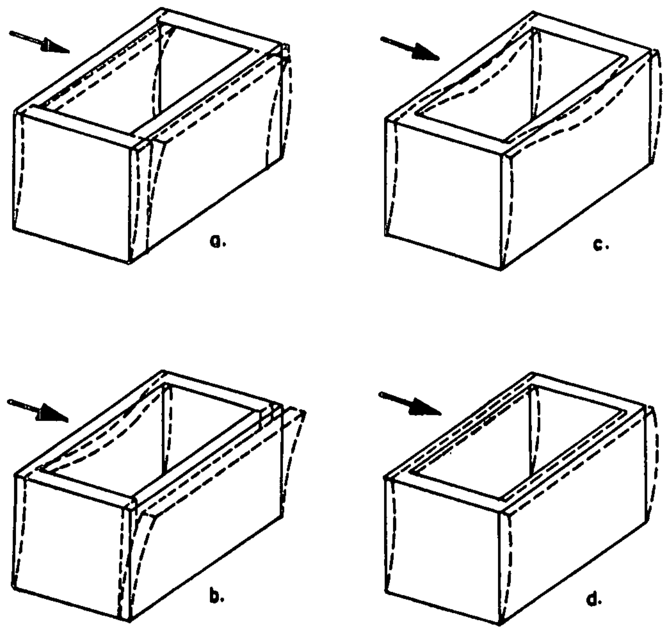

Fig.3.13 - Vibrations of masonry building during earthquake ground motion:

a. and b.: structural walls are not tied together;
c.: structural walls are tied together by means of tie-beams;
d.: structural walls are tied together by means of rigid floor slab.

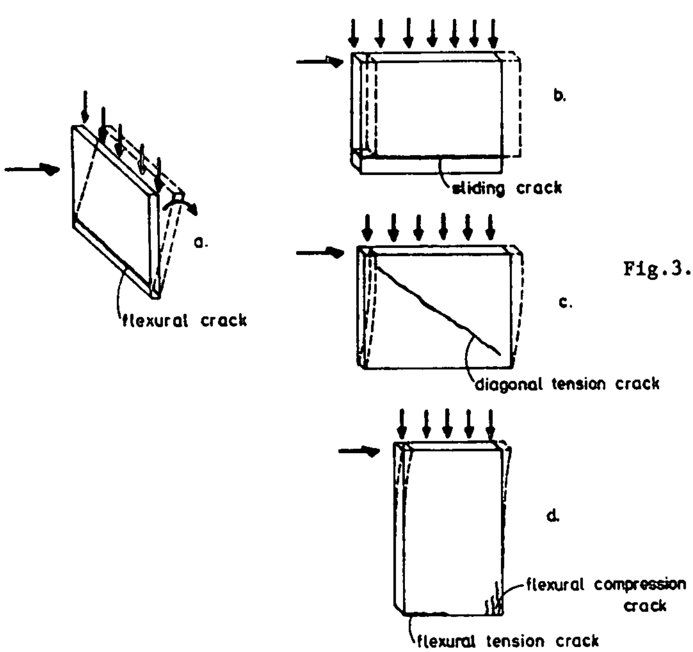

Fig.3.14 - Failure mechanisms of a free standing structural wall

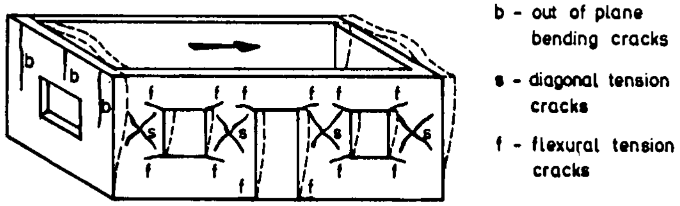

Fig. 3.15 - Failure mechanism of a masonry building

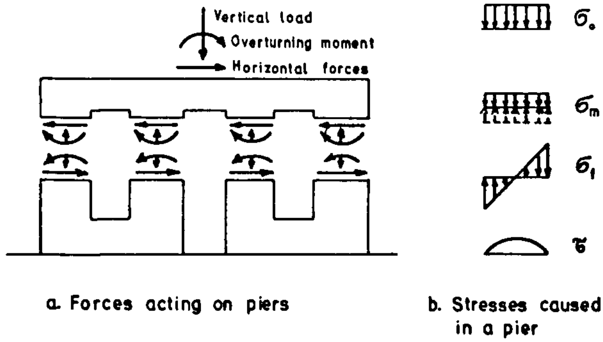

Fig. 3.16 - Idealized force and stress state in a masonry building when subjected to an earthquake

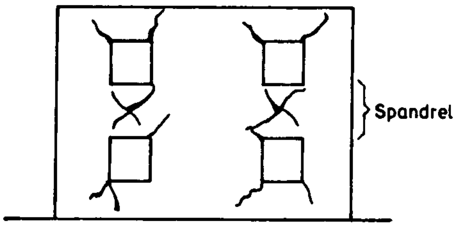

Fig. 3.17 - Failure mechanism of a spandrel wall with no tie-beams or rigid slab existing at the floor level

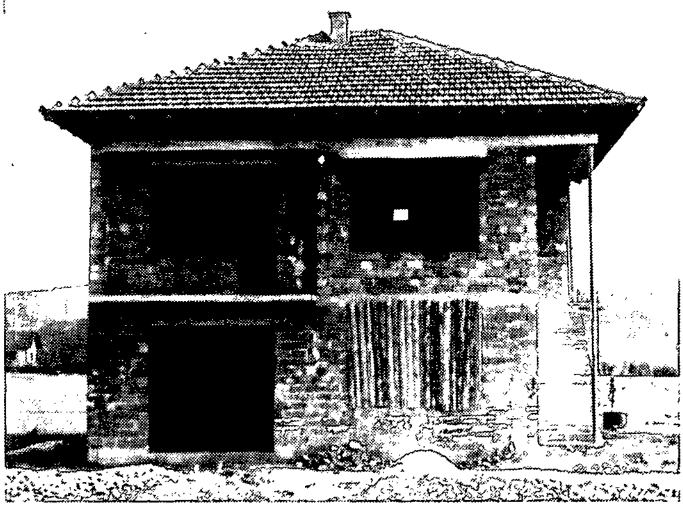

Fig.3.18 - Banja Luka,1969 Earthquake (Yugoslavia): The building constructed with hollow modular ceramic blocks in cement-lime-sand mortar of good quality suffered no damage

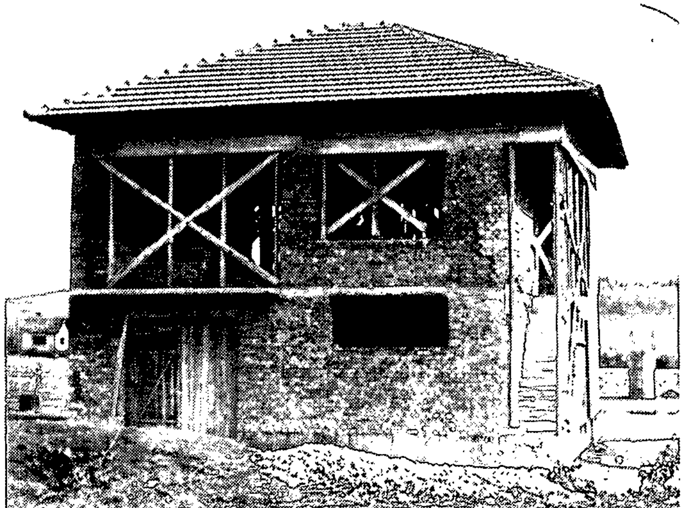

Fig.3.19 - Banja Luka,1969 Earthquake (Yugoslavia): The adjacent similar building, constructed with clay bricks and poor lime-sand mortar, suffered severe damage

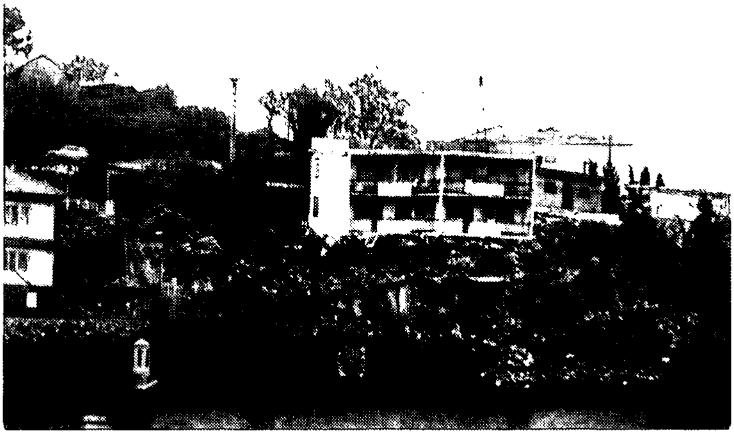

Fig.3.20 - Montenegro,1979 Earthquake (Yugoslavia): Tilting of the building due to slope sliding

Fig.3.21 - Montenegro,1979 Earthquake (Yugoslavia): Sinking of the building into the ground due to loose water-saturated sandy soil liquefaction

4. PRINCIPLES OF ARCHITECTURAL AND STRUCTURAL CONCEPTS FOR BUILDING CONFIGURATION

4.1 General

Masonry building represents a box-type structural system composed of vertical structural elements - walls and horizontal structural elements - floors. Vertical loads are transferred from the floors, acting as horizontal flexural members, to the bearing walls, and from the bearing walls, acting as vertical compression members, to the foundation system.

When subjected do earthquake motion, horizontal inertia forces are induced in the structural system. Again, horizontal inertia forces are transferred from the floor structures, acting as rigid horizontal diaphragms, into the bearing walls, causing shearing and bending effects, and from the bearing walls into the foundation system. Additionaly, due to the distributed mass of wall elements, distributed inertia forces are induced resulting in out of plane bending of walls.

The analysis of the behaviour of masonry buildings subjected to earthquakes has clearly shown the importance of building configuration. The buildings with regular structural layout, with the walls tied together at floor levels, have often performed rather well, even when not designed to resist earthquakes. They demonstrated that it is possible to achieve enough earthquake resistance by taking into account simple principles of architectural and structural planning, meeting the requirements for quality of materials and construction in the same time.

4.2 Building Configuration

The following general requirements should be taken into account when designing masonry buildings.

- the building as a whole or its various parts should be kept symmetrical about both principal axes. Lack of symmetry leads to torsional effects and hence to the concetrated damage in the critical zones. Symmetry is also desirable in location of openings (Fig.4.1-a);

- simple square or rectangular buildings in plan behave better when subjected to earthquake action as compared to those with many projections. Torsional effects due to the differences in ground motion are pronounced further in the case of narrow rectangular parts (Fig.4.1-b). Therefore it is desirable to limit the length of a part to 3.5 times its width. If longer lengths are required, the building should be divided into separate parts with the sufficient separation (Fig.4.1-c);

- separation of a large building into several parts may be required so as to obtain symmetry and rectangularity of each part. For preventing hammering effects between the adjacent parts, a sufficient separation between the parts must be provided. It is recommended that the width of separation should not be less than 30 mm, and 10 mm should be added for each story (or 3 mm) when the building height exceeds 9 m;

- the building should be as simple as possible. Ornamentations involving large cornices, vertical or horizontal cantilever projections, facia stones and the like are dangerous and undesirable from the seismic viewpoint. Where any ornamentation is used, it must be properly reinforced and anchored;

- the distribution of stiffnesses both in plan and over the building height should be as uniform as possible. Sudden changes in stiffness due to changes in plan of different stories result in severe concentration of damage in those zones (Fig.4.2);

- mixed structural siystems, combining masonry load bearing system with reinforced concrete load bearing system both in plan or over the height of the building, should be avoided unless a special analysis is carried out (Fig. 4.3). In the case when single reinforced concrete columns as load bearing elements cannot be avoided, masonry walls must be able to carry all seismic loading.

4.3 Plan Dimensions, Building Height and Number of Stories

Based on post-earthquake damage observations, materials and structural systems used for construction of masonry buildings, and considering the current technical knowledge and construction technology, the countries in this region have set limitations on plan dimensions and height of masonry buildings in their seismic Codes. These limitations have been defined in the seismic Codes in relation with regional seismicity, recognizing low (L), moderate (M) and high (H) seismicity.

Considerable research has been done recently or is under way in the Balkans, concerning the behaviour of masonry buildings and wall elements under seismic loading conditions. Special attention has been paid to the improvement of the strength and ductility of masonry walls by means of vertical and horizontal reinforcement. The results of the research indicate the possibility of relaxing the dimensional limitations noted above. However, pending the final conclusions of the work it is recommended, that the limitations, noted in the following sections,should be observed.

4.3.1 Plan Dimensions

In order to diminish the typical effects of temperature differences,shrinkage of reinforced concrete floors, and soil settlements, common to long buildings,but also to avoid unfavourable effects due to the differences of ground motion along the length of the buildings, the length of masonry building or its separated parts should be limited to 40 m in seismic zones of high intensity (H) and to 50 m in seismic zones of moderate (M) and low (L) intensity.

Furthermore the length to width ratio of masonry buildings should not be more than 3.5 in order to limit the damage potential of torsional effects when subjected to earthquake motion.

In the case of unfavourable soil conditions, further limitations should be observed, depending on the specific soil characteristics.

4.3.2 Height and Number of Stories

The height of the building and its number of stories should be limited according to the structural system and the seismic zone.

The height of the building is determined as the vertical distance from the ground level (or from the ground floor, if any) to the top face of the uppermost floor. If the ground floor is located at more than 1.5 m above the outside terrein, the basement should also be taken into account when determining the building height and number of stories.

Table 4.1 gives values for both the recommended maximum heights of such buildings and the maximum number of stories for different construction system and seismic zones.

Table 4.1 Recommended Maximum Building Height (H in m) and Number of Stories(n)

Construction system		Seismic zone		
		Low	Moderate	High
Adobe	H	3	3	3
	n	1	1	1
Plain masonry	H	9*	6	3
	n	3*	2	1
Confined and reinforced masonry	H	18	14	11
	n	5	4	3

*Except stone masonry, where the height is limited to maximum 6 m or two stories.

4.4 Distribution of Walls

In order to have a satisfactory performance when subjected to earthquake, the walls of masonry building must be uniformly distributed in both orthogonal directions, sufficient in number and strength to resist lateral earthquake loads (Fig.4.4). Walls must be firmly connected to the floors, which must be able to act as rigid horizontal diaphragms, distributing horizontal inertia forces onto the walls according to their stiffnesses.

From the viewpoint of the structural system, the walls of the masonry building can be defined as:

- structural walls, carrying their own weight together with the vertical and/or horizontal loads acting on the building;

- non-structural walls, having exclusively the function of partitioning the building space. Their own weight is transferred by means of floors to the structural walls.

The structural walls can be divided into two categories, namely:

- load-bearing walls, which carry both vertical load from the floor structures and their own weight and the horizontal load, and

- bracing walls, carrying their own weight and the horizontal load.

Considering the significance of the structural walls, these walls should have a minimum thickness of 0.40 m for adobe and stone-masonry buildings, and of 0.20 m for brick or block masonry buildings unless strength, stability and insulation conditions require more.

In order to obtain a satisfactory performance for different masonry system, the distances between the structural walls should be limited depending on zones. Indications are given in Table 4.2:

Table 4.2 Recommended Maximum Distances Between Structural Walls (in m)

Construction system	Seismic zone		
	Low	Moderate	High
Adobe and stone	4.5	4.5	4.5
Plain masonry	10	8	6
Confined and reinforced masonry	15	12	8

Using the above recommended values, the resulting structural layout, however, should be verified by calculation. Limiting factor may be the vertical load-bearing capacity and the out-of-plane bending capacity of these walls.

4.5 Openings

Experiences have indicated a strong effect of the size and the position of openings on the resistance of masonry buildings to earthquake loading. In the opening zones stress concentrations may occur under lateral earthquake loads, and may lead to cracking of wall element. Therefore, in order to improve the behaviour of masonry buildings when subjected to earthquakes, the following recommendations should be observed, as far as the location and size of the openings is concerned:

- openings should be located in those walls which are subjected to smaller intensity of vertical loads;

- openings should be located outside the zones of direct influence of concentrated loads at beam supports;

- openings should be located in the same position at each story level;

- openings should be located symmetrically with respect to the building configuration in plan, providing the uniform distribution of stiffnesses and strength in both orthogonal directions of the building;

- the top of the openings should be at the same horizontal level;

- openings should not interrupt floor tie-beams;

- the total length of openings in a structural wall should not exceed half the length of the wall;

- the total cross sectional area of walls in each story, oriented in the same direction, should not be less than 4% of the gross floor area.

The recommendations regarding the distribution and dimensions of openings in structural walls are presented in Fig.4.5. In the case that the actual dimensions of openings exceed the recommended value by 30%, reinforced concrete tie-beams should be provided at the bottom and atop of the opening, and be connected together with reinforced concrete tie-columns located on each side of the opening. Details are given in Chapter 6.3 and 10.2.

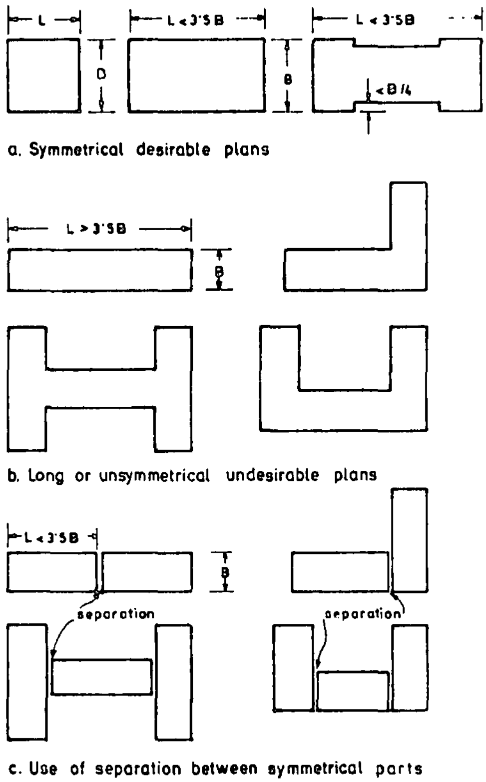

Fig.4.1 - General principles for building configuration in plan

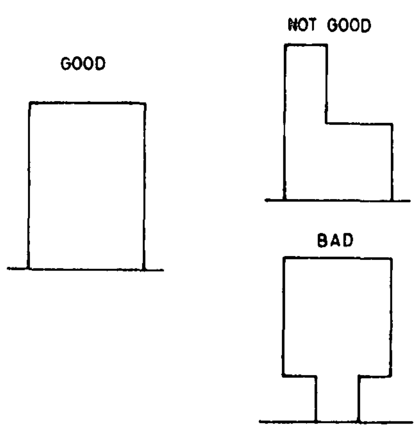

Fig.4.2 - General principles for building configuration in elevation

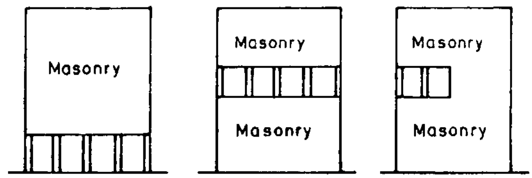

Fig.4.3 - Mixed structural systems should be avoided

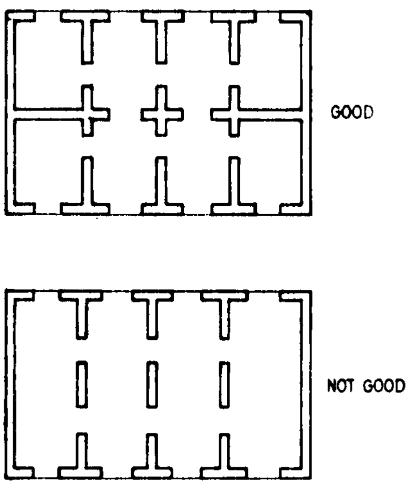

Fig.4.4 - Distribution of structural walls in plan

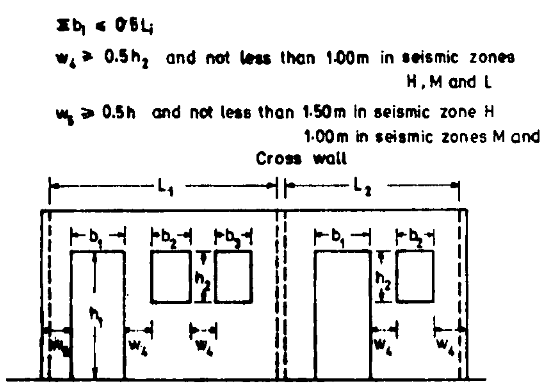

Fig.4.5 - Distribution and size of wall openings

5. MATERIALS

5.1 General

In addition to the structural layout, the quality of materials used for construction plays a significant role in the earthquake resistance of masonry building.

Masonry is a composite material which consists of several materials of different characteristics.

- masonry units,
- mortar and grout,
- reinforcing steel, and
- concrete,

which is required to act as a unique structural material. Of course not all materials are always encountered in every type of masonry structure.

Knowing the characteristics of the constituent materials, it is, however, not easy to predict the characteristics of the actual masonry. In this respect it is necessary, apart from controlling the quality of the constituent materials by means of standard tests, to investigate the behaviour of masonry structural elements by subjecting them to earthquake loading conditions. This is the only way to obtain sufficient information about the strength and ductility characteristics of masonry, necessary for the earthquake resistant design of masonry buildings.

A wide variety of masonry types are used for construction in the Balkan countries, from traditional types, like adobe and stone-masonry, to the modern ones, using brick or block masonry units.

The shape and the quality of modern units, such as bricks and blocks, are usually defined by national Standards or Codes, which differ from country to country, and limit the use of the units, depending on the seismic zone and units' types. Considering the variations in different national Standards, only some general requirements concerning the use of different units in earthquake prone areas, will be presented here.

5.2 Adobe

Adobe blocks (Fig.5.1) are usually made of clayey earth, consisting of 30-40% of clay and 70-60% of earth. Of the earthen part 40% should pass through 0.063 mm sieve, but the earth should not contain stones bigger than 30 mm in diameter. In case that earth contains more clay than recommended, sand, crushed brick or stone and slag should be added. Hay or straw, which is used as an additive, should be dry and not rotten, with fibre length of about 100-120 mm (7-10 kg of hay per each m^3 of earth is usually enough). Mixing water should be clean.

Characteristic compressive strength of adobe blocks is usually low and does not exceed the value of 1 MPa. However, by treating the adobe blocks with cement, gypsum or other stabilizers, the value of 5 MPa can be reached.

Adobe putty, used in manufacturing of adobe blocks, is used for bonding the adobe blocks.

5.3 Stone

Excavated stones and river bed stones having different petrographic characteristics (usually limestone) are the primary construction materials in stone masonry buildings. Irregular sized uncoursed or partly coursed stones are commonly used for construction of two layered walls, the bigger stones (up to .50 m in size) serve as the load-bearing portion of the wall, and the smaller stones are used as infill. Regular sized coursed stones are preferable, especially in the corner and wall intersection zones. A cement-lime-sand mortar of grade M 2.5 (characteristic compressive strength 2.5 MPa) should be used for construction of stone-masonry walls.

5.4 Bricks and Blocks

The units used for masonry construction can be made of either burned clay, concrete or light-weight concrete materials in different sizes and shapes, either solid or perforated, both partialy or complete. (Fig.5.2). As a rule, they must meet national Standards requirements.

When selecting the most suitable type of units for construction of the masonry building, one has to consider, apart from the load-bearing capacity, the following aspects also:

- adequate thermal and sound insulation capacity, especially in the case of external walls;

- reduction of the weight of the building (hence, reduction of seismic horizontal forces);

- economy of construction.

Masonry units can be either solid or perforated with vertical holes. To be considered as solid, units should have a net cross sectional area at least equal to 75% of the gross cross sectional area of the unit, and the thickness of at least 20 mm for both their face shells and cross webs.

If perforated or hollow units are used for construction of structural walls in seismic areas, their net cross sectional area should be at least equal to 45% of the gross cross sectional are of the unit, and the thickness of at least 15 mm for both face shells and cross webs.

Horizontally perforated units can only be used for construction of non-structural elements, such as partition walls.

The characteristc compressive strength of units to be used for construction of structural walls is recommended to be not less than B 5 (for both solid and perforated units).

5.5 Mortars

Mortars to be used for construction of structural masonry should be cement or cement-lime mortars with a minimum grade of M 2.5 (characteristic compressive strength 2.5 MPa). For construction of reinforced masonry, however, the use of cement mortar with minimum grade M 10 is recommended.

In order to obtain the appropriate mortar grade, the following rules for mortar mixes, expressed in volume parts, can be observed:

Table 5.1 - Recommended Mortar Mixes

Mortar grade	Characteristic compressive strength (MPa)	Proportion of mixes by volume		
		cement	lime	sand
M 2.5	2.5	1	3	9
M 5	5	1	2	6
M 10	10	1	0.5	5
M 20	20	1	-	3

Proportions are depending on the quality of cement and the quantity of water used for mixing. Other mixes may be used, when verified by laboratory testing and experience.

5.6 Reinforcing Steel

For vertical and horizontal reinforcement of walls and tie-columns or tie-beams, both plain bars with characteristic yield stress of approximately 240 MPa and deformed bars (with characteristic yield stress of approximately 400 MPa) can be used.

5.7 Concrete

For construction of reinforced concrete elements which are part of the masonry structural system (such as tie-beams and tie-columns), the minimum concrete grade C 15 (characteristic compressive strength of 15 MPa) is recommended.

5.8 Mechanical Characteristics of Masonry Walls

When designing masonry buildings the values of the mechanical characteristics of masonry, such as:

$f_{wc,k}$ - characteristic compressive strength

$f_{wt,k}$ - characteristic tensile strength,

E_w - elastic modulus,

G_w - shear modulus,

μ_w - ductility factor.

must be known.

As it has already been pointed out, it is not easily possible to determine the characteristics of the masonry from the characteristic values of the constituent materials used. The required values which are strongly depending on the size and shape of masonry units, as well as on the quality of mortar used, can only be obtained by means of laboratory testing.

The comparison of test results, obtained in different countries, has shown sometimes significant differences between the values obtained on specimens with rather similar characteristics (grade of units and mortar), but different shapes. In this respect the values, presented in Tables 5.2 and 5.3 which are based on Romanian and Yugoslav test results, should be considered for guidance only. In the case other units being used additional tests are needed to evaluate the characteristic compressive and tensile strength of masonry walls.

5.8.1 Characteristic Compressive and Tensile Strength of Masonry Walls

The values of the characteristic compressive strength of masonry walls, $f_{wc,k}$, obtained experimentally by subjecting the test specimens to vertical compression loading, which are recommended to be used for design of masonry buildings are given in Table 5.2.

The values of the characteristic tensile strength of masonry walls, $f_{wt,k}$, obtained experimentally by subjecting the test specimens to either diagonal compression or to horizontal shear with simultaneously acting compression loading, which are recommended for the design of masonry buildings, are given in Table 5.3

Values in parentheses are obtained in Yugoslavia, other values in Romania.

Table 5.2 Characteristic Compressive Strength of Masonry Walls $f_{wc,k}$ (in MPa)

Type of unit	Unit grade	Characteristic compressive strength			
		Mortar grade			
		20	10	5	2.5
Ceramic brick or hollow block, course depth less than 150 mm	20	(10.50)	4.40	3.60	2.80 (2.70)
	15	-	3.60	2.80	2.40
	10	-	2.80	2.40	2.00
Light weight ceramic block, course depth more than 150 mm	7.5	4.50	-	-	-
Light weight concrete block, course depth more than 150 mm	10	-	3.60	3.20	2.80
	7.5	-	3.00	2.70 (3.70)	2.40

Table 5.3 Characteristic Tensile Strength of Masonry Walls $f_{wt,k}$ (in MPa)

Type of unit	Unit grade	Characteristic tensile strength			
		Mortar grade			
		20	10	5	2.5
Ceramic brick or hollow block, course depth less than 150 mm	20	(0.46)	0.18	0.18	0.13 (0.17)
	15	-	0.18	0.18	0.13
	10	-	0.18	0.18	0.13
Light weight ceramic block, course depth more than 150 mm	7.5	(0.29)	-	-	-
Light weight concrete block, course depth more than 150 mm	10	-	0.16	0.16	0.11
	7.5	-	0.16 (0.32)	0.16 (0.26)	0.11 (0.19)

5.8.2 Elastic and Shear Moduli

The values of the elastic and shear moduli define the deformability characteristics of masonry. Accurate values must be known when determining the periods of vibrations as well as when distributing the story shear onto the structural walls, especially when different masonry types are used in the same story.

The values of the elastic modulus E_w and the shear modulus G_w should be obtained experimentally, elastic modulus is defined as the secant modulus of a vertical compression test of a masonry wall element.

The following relationship between the characteristic compressive strength of masonry wall $f_{wc,k}$, and the elastic modulus E_w may be used if experimentally obtained values for E_w are unavailable:

$$E_w = 1000\ f_{wc,k} \qquad \ldots (5.1)$$

As the experimental results have shown, the actual values of E_w may vary between:

$$500\ f_{wc,k} < E_w < 3000\ f_{wc,k}\ . \qquad \ldots (5.2)$$

Similarly, the values of the shear modulus G_w can be derived from the tensile strength of masonry wall $f_{wt,k}$. The following relationship was obtained:

$$G_w = 3000\ f_{wt,k}\ , \qquad \ldots (5.3)$$

where actual values for different types of masonry walls may vary between:

$$1000\ f_{wt,k} < G_w < 5000\ f_{wt,k} \qquad \dots (5.4)$$

for different types of masonry walls.

Experimental results indicate that the ratio between the characteristic tensile strength and the characteristic compressive strength does not vary much:

$$f_{wt,k} = 0.05\ f_{wc,k} \text{ to } 0.07\ f_{wc,k}$$

for different types of masonry walls. Taking this into consideration, the average ratio between elastic and shear moduli becomes a constant:

$$\frac{E_w}{G_w} = 6 \qquad \dots (5.5)$$

5.8.3 Ductility

Masonry, and especially plain masonry is a brittle structural material. However, as the test results have indicated, a certain degree of ductility can be observed when the masonry is subjected to a combination of vertical and cyclic horizontal loading, and a rather high possibility of energy dissipation (Fig. 5.3).

The envelopes of the experimentally obtained hysteresis loops, which represent the relationship between the horizontal force and horizontal deformations of the test specimens can be idealized in order to obtain strength and deformability characteristics (Fig.5.4).

Ductility factor is defined as the ratio between the deformation at failure and the deformation at the idealized elastic limit:

$$\mu_w = \frac{\delta_{max}}{\delta_o}, \qquad \dots (5.6)$$

where:

μ_w - ductility factor,

δ_{max} - maximum deformation,

δ_o - deformation at the idealized elastic limit.

Rather high values of ductility factors have been obtained experimentally for plain masonry elements of different types, failing in shear ($\mu = 2.0 - 5.0$), and even higher values for plain masonry elements, failing in flexure, or horizontally reinforced masonry elements, failing in shear.

However, when verifying the earthquake resistance of masonry building by using limit states methods, it is recommended that the following ductility factors for wall elements be taken into account:

- $\mu = 1.5$ for plain masonry, and

- $\mu = 2.0 - 3.0$ for confined and reinforced masonry.

The proposed values should be valid for both shear and flexural mode of failure of wall elements.

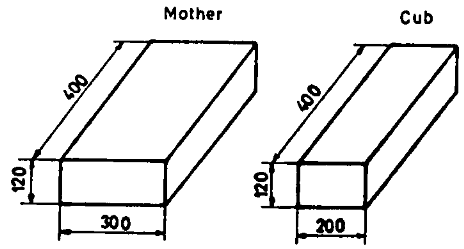

Fig.5.1 - Typical dimensions of adobe blocks

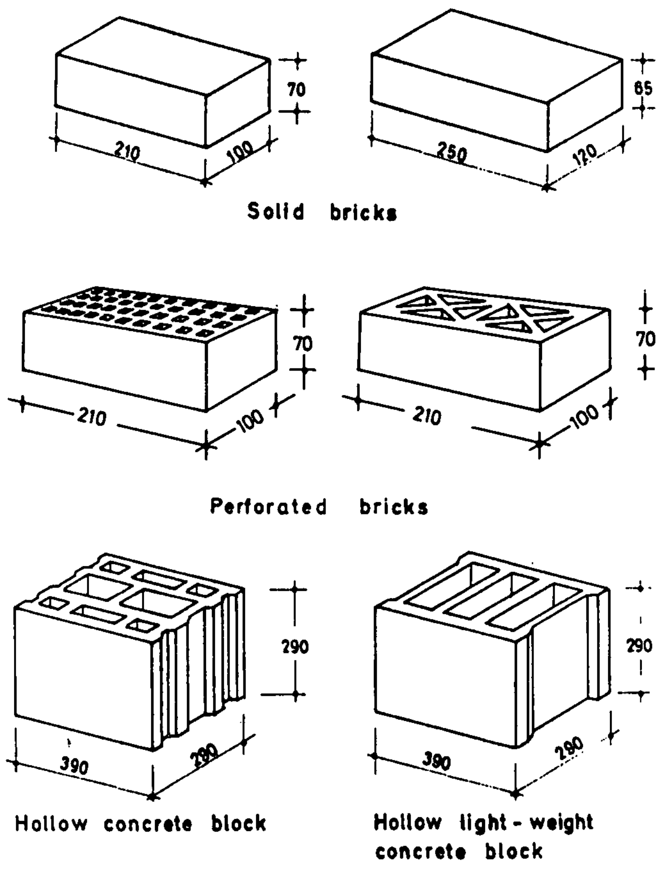

Fig.5.2 - Typical masonry units

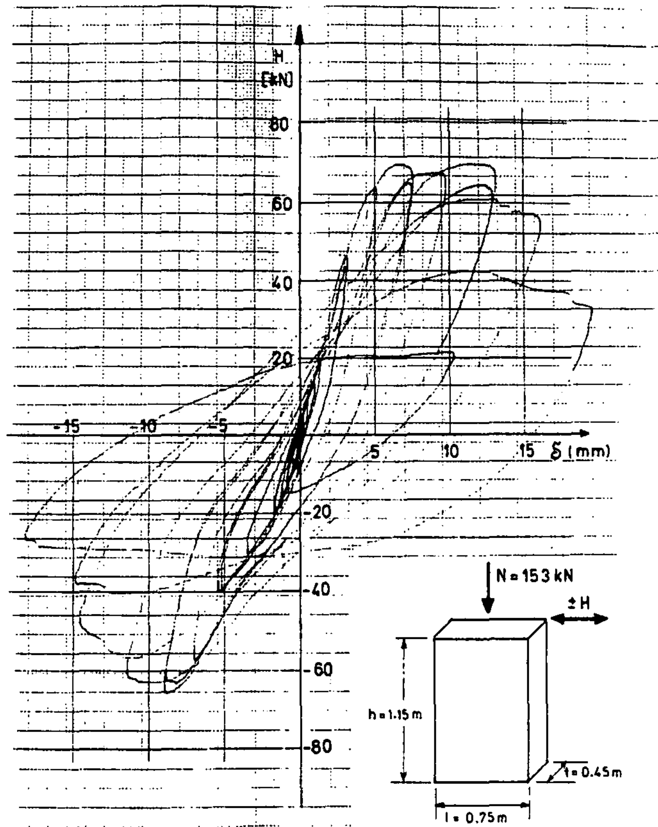

Fig. 5.3 - Typical hysteresis loops obtained by means of shear resistance test of unreinforced brick masonry wall [4]

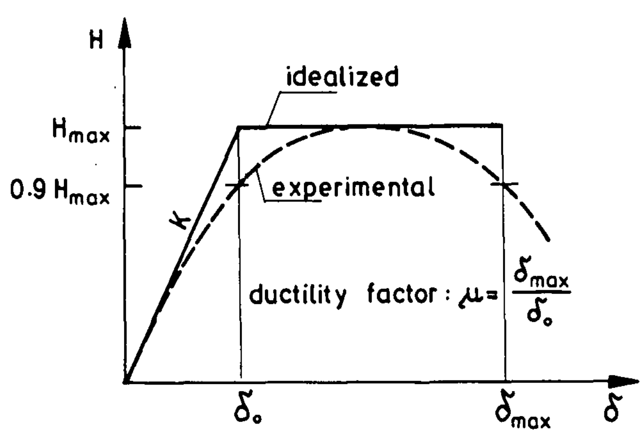

Fig. 5.4 - Idealization of the hysteresis envelope

6. CONSTRUCTION SYSTEMS

6.1 General

Different system of masonry construction are used for masonry buildings in the Balkan countries. According to the constituent materials and technology used in construction, the following three basic construction systems can distinguished:

- plain masonry,
- confined masonry, and
- reinforced masonry.

Because of the different structural configurations the behaviour of the three basic systems is quite different; plain masonry represents basically a brittle structural material, whereas confined and especially reinforced masonry represents a structural system of improved strength and ductility.

In the Balkan countries plain masonry was commonly used before adopting aseismic regulatins. Confined masonry, which was introduced in the sixties, became very popular because of the good behaviour observed during several subsequent earthquakes. On the other hand, reinforced masonry, representing the best masonry construction system in the earthquake loading, has practically not been used. Several attempts to introduce reinforced masonry have been made, without necessarily following procedures which would have resulted in a basically optimum design. Nevertheless, information on design and construction of reinforced masonry will also be presented here to illustrate its potential benefits.

Considerable research on the behaviour of reinforced masonry, subjected to simulated earthquake loadings has been performed throughout the world, and many principles for the construction and design of systems are known. However, not many data are available regarding the construction systems used in the Balkan region. In this respect, further research is needed to study the behaviour of national materials and the possible construction technologies. This very fact is also the reason why the height of buildings, even if constructed in reinforced masonry, has been limited. Unless new experimental data will be available, the use of masonry construction may well be limited to relatively low-size buildings.

6.2 Plain Masonry

6.2.1 Adobe

Adobe blocks and adobe putty, as described in Chapter 5.2 should be used for construction of adobe buildings (Fig.6.1). A minimum wall thickness of 0.40 m is required.

The height of the adobe wall should be limited to 2.70 m, and the maximum clear distance between the cross walls must no exceed 4.50 m. Four wooden tie-beams, as described in Chapter 10.2.2 should be provided in the wall, situated above the foundation, above and below the window and below the roof.

The openings should be located at least 1.00 m from the building corners and at least 0.50 m from the wall intersections. If the pier length between the openings is less than 0.60 m, then two 100 x 100 mm timber posts should be provided on both sides of the opening, well connected to the tie beams. The dimensions of openings should not exceed 1.00 x 2.10 m for doors and 0.90 x x 1.40 m for windows.

6.2.2 Stone-Masonry

For construction of stone masonry walls either irregular sized uncoursed and partly coursed stones or regular sized coursed stones are used.

In the case of irregular sized uncoursed or partly coursed stone, the minimum wall thickness of 0.40 m is required (Fig.6.2). If the walls are constructed as two layered walls, the outer layers of bigger stones serve as the load bearing part of the wall, with the inner infill of smaller pieces of stone. Connecting stones must be provided, at least one per each square meter of the vertical area of the wall. M 2.5 grade cement-lime mortar should be used for construction and care should be taken that all the voids between the stones are filled with mortar, especially the inner infill part of the wall.

Horizontal running bond is required for construction of stone-masonry walls, and vertical stacked bond is to be avoided. Regular sized coursed stones must be used at the corners and wall intersections, providing better connections in these, critical, zones (Fig.6.3).

When constructing the building in seismic zone H, the construction course must be leveled at each 1.0 m height (2.0 m in seismic zones L and M) and the reinforced concrete tie beams, as described in Chapter 10.2, must be provided, connecting the structural walls all around the building (Fig.6.4).

If regular sized coursed stones are used for construction, the same rules used for construction of brick and block masonry should be observed.

6.2.3 Brick and Block Masonry

Brick and block masonry walls should be constructed according to the rules of good mason workmanship as listed below:

- before the construction, masonry units must be soaked in water in order to prevent burning of the mortar, especially cement;

- masonry units must be laid by overlapping the units of the previous course for at least one quarter of the unit's length (Fig.6.5);

- vertical stacked bond is not allowed;

- the thickness of horizontal joints should be not more than 15 mm, and that of vertical joints not more than 15 mm, but not less than 10 mm.

As a rule, the same type of masonry units should be used for structural walls in the same story. The thickness of structural walls should not be less than 200 mm, unless stability and insulation criteria require more. It is recommended, that the thickness of individual walls is kept constant over the entire height of the building.

Bracing walls should be constructed simultaneously with load bearing walls. If the course depth of the two crossing walls is not identical, their connections should be carried out by means of built-in reinforced concrete tie-columns (Fig.6.6).

In order to improve the behaviour of plain masonry when subjected to earthquakes, reinforcing bars can be placed in horizontal mortar joints in the corners and wall intersection zones, using a cement mortar (Fig.6.7).

6.3 Confined Masonry

Confined masonry is a construction system, where reinforced concrete vertical tie-columns, located at regular intervals and connected together with reinforced concrete horizontal tie-beams, confine plain brick or block masonry structural walls, thus forming a framed masonry panel.

As the experimental investigations and the experiences obtained after the earthquakes have shown, providing reinforced concrete tie-columns results in:

- improvement of connection between the masonry upper structure and the foundation system;

- improvement of stability of slender masonry walls;

- improvement of strength and ductility of masonry panels;

- reduction in the risk of disintegration of masonry panels, damaged by the earthquake.

In order to be effective, reinforced concrete tie-columns - which are basically vertical non-load bearing structural elements-should be located at all corners and recesses of the building and at all crossings and joints of structural walls. Additionally, depending on the seismic zone such r.c. tie-columns should be located at all free extremities of structural walls, and at all opening sides, exceeding the recommended size.

The maximum distance between the two consecutive tie-columns should not exceed 5.0 m and they should be cast after the construction of all structural walls in the same story.

A typical distribution of reinforced concrete tie-columns in plan is shown in Fig. 6.8.

The following detailing provisious, as shown in Figs.6.9 and 6.10 should be observed:

- as a rule, the width of the tie-column should be equal to the thickness of wall, forming a minimum cross-sectional area of 0.20 x 0.20 m. In the case of external tie-columns, their width can be reduced in order to allow the thermal insulation layer be provided on the external face of the wall. Sometimes, special hollow blocks can be used for construction of tie-beams.

- minimum concrete grade C 15 (characteristic compressive strength 15 MPa) should be used for construction;

- minimum amount of longitudinal reinforcement should be equal to four 12 mm diameter plain bars (yield stress of approximately 240 MPa) or four 10 mm diameter deformed bars (yield stress of approximately 400 MPa).

- stirrups of 6 mm diameter plain bars at 200 mm intervals should be used for transversal reinforcement, more closely spaced at the splicing zones and above and below tie-beams.

6.4 Reinforced Masonry

6.4.1 Reinforced Masonry Forms

Reinforced masonry in principle offers the possibility to develop significant ductility in masonry wall elements and in the structures as a whole. In general, the following three forms of construction can be recognized:

- Confined reinforced masonry,
- Reinforced grouted masonry, and
- Reinforced hollow-unit masonry.

While masonry with a confinement of tie-columns and beams is widely used in the Balkan region, the use of reinforced masonry as a form of construction has been limited in general. While in Greece reinforced masonry is widely used in certain areas of high seismicity, the use in Bulgaria, Hungary, Rumania and Turkey is virtually nonexistent. On the other hand, in Yugoslavia reinforced masonry in one of the above forms is finding still limited although increasing application in residential buildings; Considering the benefits to be derived from using reinforced masonry a review of the three major forms of reinforced masonry is presented in the following sections.

Confined Reinforced Masonry

Confined reinforced masonry is a combination of confined masonry reinforced horizontaly with the reinforcement placed in the intermittent bed joints and anchored into the tie-columns. Different kinds of horizontal reinforcement can be used (Fig.6.11), such as plain or deformed bars in the form of stirrups,or plain cold drawn prefabricated units of two parallel bars, connected by means of trussed or transverse bars,spot-welded to the longitudinal bars.

For the construction of confined reinforced masonry either normal brick and/or block masonry units or specially formed units, with channels for placing the horizontal reinforcement, can be used.

The horizontal reinforcement must be well anchored to the tie-columns,which are located at both ends of the wall panel, as shown in Fig.6.12.

Reinforced Grouted Masonry

Reinforced grouted masonry consists of two wythes of masonry units (bricks or blocks), separated by a grouted cavity in which the vertical and horizontal reinforcement is placed (Fig.6.13).

The wythes are usually one unit thick (about 100 mm), and the cavity is 60-100 mm wide. The two wythes must be connected by means of stirrups or equivalent ties; there should be at least one 6 mm diameter bar stirrup provided for each 0.25 m^2 of the wall area. The masonry units should be laid in running or stretched bond, and vertical stack bond should be avoided.

The grout can be poured as the work progresses or after the masonry units have been laid. In this respect, two different construction procedures are defined below.

In the low-lift grouting procedure the vertical reinforcing bars are first placed into position. Then, the horizontal bars and stirrups are placed and grouted in as the laying of the masonry units progresses.

In the high-lift grouting procedure the mesh of vertical and horizontal bars is placed into position. Then, the masonry units are laid on each side of the reinforcement, and after the masonry is built a full story height, the cavity is filled with grout. At the time of construction both wythes should be kept at the same level, connected by means of stirrups (ties) placed at intervals that do not exceed 0.60 m horizontally and 0.40 m vertically. The stirrups should be laid in the bed joints and in the same vertical line to facilitate the vibrating of the grout pours. Cleanout openings of sufficient size and location should be provided to allow flushing away of mortar droppings and debris at the bottom of each pour.

All mortar droppings and overhangs should be removed from the foundations or other bearing surfaces and reinforcing. Grout should be transported from the mixer to the point of deposit in the grout gap as rapidly as possible by pumping and placing methods which will prevent segregation of the mix and will minimize grout splattter on reinforcing and masonry unit surfaces, which are immediately encased in the grout lift.

Contact surface of all foundations and floors that have to support masonry should be cleaned and roughened in order to provide a good bond between the grout fill and the concrete surfaces.

Reinforced Hollow Unit Masonry

Reinforced hollow unit masonry is usually made of a single wythe of hollow masonry units (bricks or blocks), reinforced vertically and horizontally with steel bars. At least the cores and voids containing reinforcement must be filled with grout as the work progresses (low-lift grouting procedure) (Fig.6.14).

When constructing the hollow unit reinforced masonry walls the vertical reinforcing bars are usually placed into position before laying the masonry units. Then, the horizontal bars can be placed in specially formed units or in standard units. Finally, vertical and horizontal bars are grouted as the work progresses.

6.4.2 Minimum Reinforcement

The reinforcement should be uniformly distributed in order to improve strength and ductility of masonry walls. The minimum reinforcement ratio depends on the quality of the masonry. However the experience shows that a minimum percentage of reinforcement, at least equal to 0.10% of the gross cross-sectional area of the wall should be placed both vertically and horizontally. The spacing between the adjacent bars should exceed neither 800 mm horizontallly nor 600 mm vertically. In order to ensure good bonding conditions between the reinforcing bars and the grout, the maximum bar diameter for reinforced grouted masonry should be limited to [7]:

$$d_b \leqq \frac{t_g}{4} \qquad \dots (6.1)$$

and for reinforced hollow unit masonry to:

$$d_b \leqq \frac{t_w}{10} \qquad \dots (6.2)$$

where:

d_b - bar diameter,

t_g - thickness of the grout core, and

t_w - overall wall thickness.

No more than one bar should be placed in any cell of hollow unit masonry.

6.4.3 Splicing and Anchorage

For vertical bars it is proposed [7] to provide an overlapping length, given by

$$l_d = 0.145\ f_y\ d_b \text{ (in mm)}, \qquad \dots (6.3)$$

but not less than 500 mm, where:

l_d - lap length (in mm)

f_y - yield limit of reinforcing steel (in MPa);

d_b - bar diameter (in mm).

Horizontal bars should be hooked around the extreme vertical bars, or a 90° vertical bend and extension parallel to the vertical reinforcement should be used. In no case should horizontal shear reinforcement be lapped in potential plastic hingening zones.

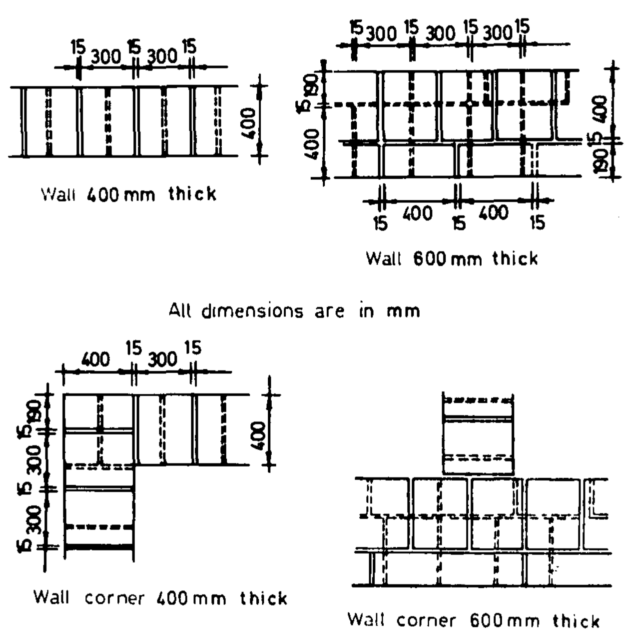

Fig.6.1 - Typical adobe construction

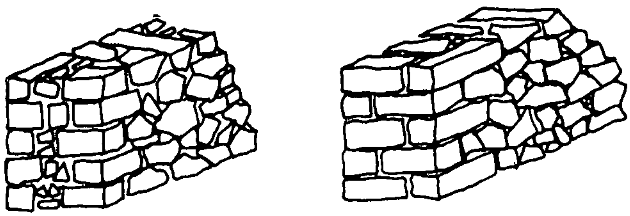

Fig.6.2 - Typical construction of stone-masonry walls

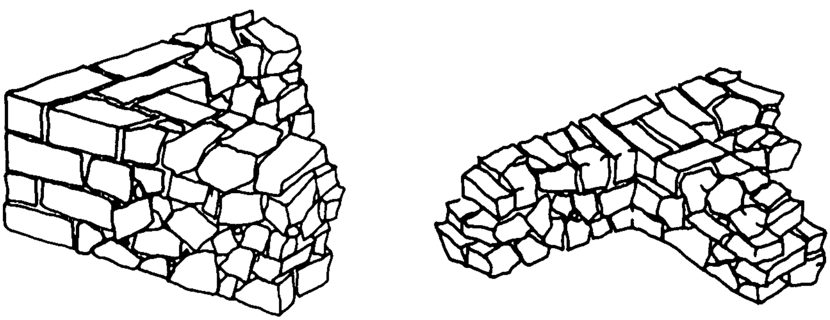

Fig.6.3 - Construction of corners and wall intersections

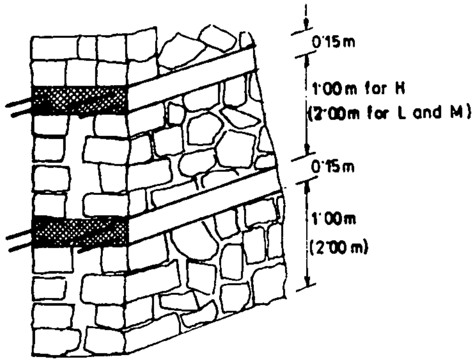

Fig.6.4 - Location of tie-beams

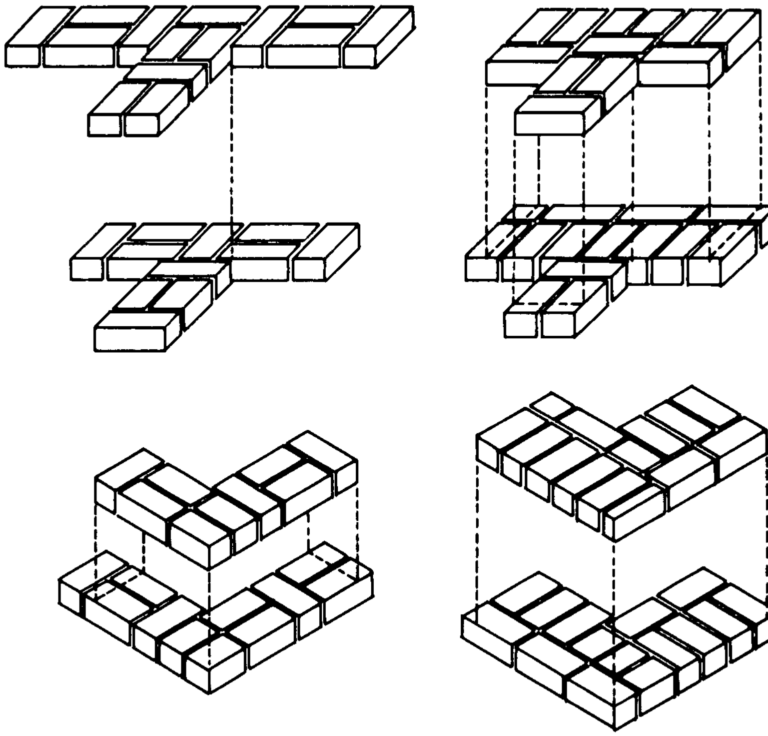

Fig.6.5 - Typical construction of brick masonry walls

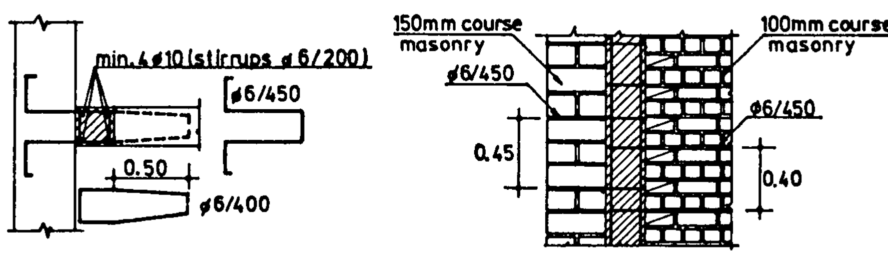

Fig.6.6 - Connection of structural walls with different course depth by means of tie-column

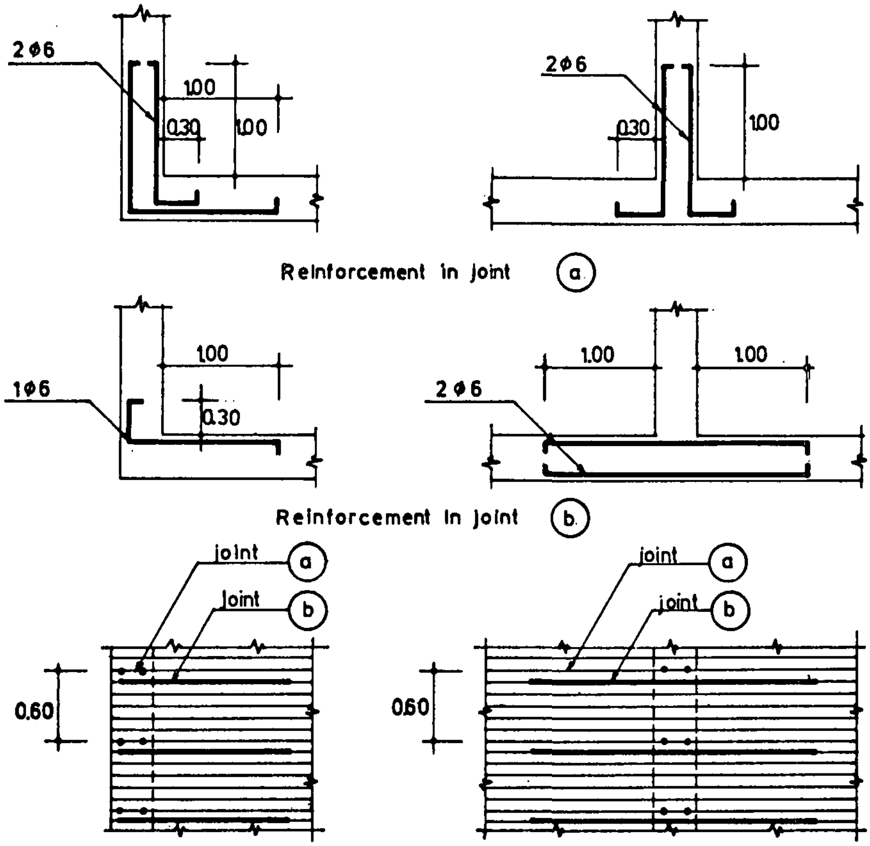

Fig.6.7 - Masonry reinforcement in horizontal joints

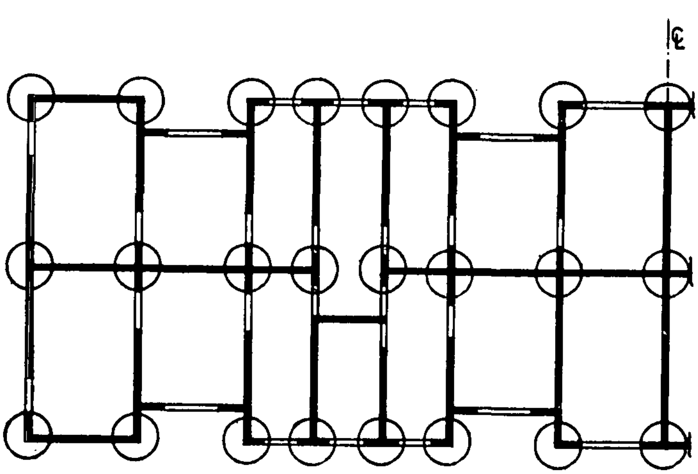

Fig.6.8 - Typical plan distribution of reinforced concrete tie-columns

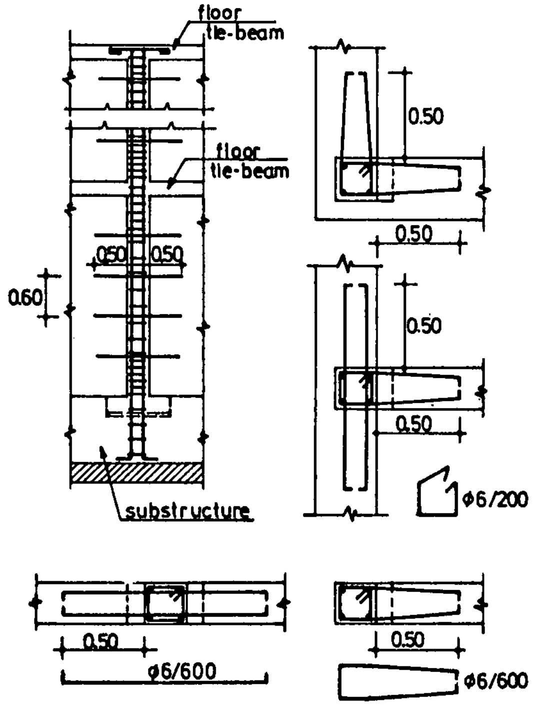

Fig.6.9 - Typical details of reinforcement of tie-columns

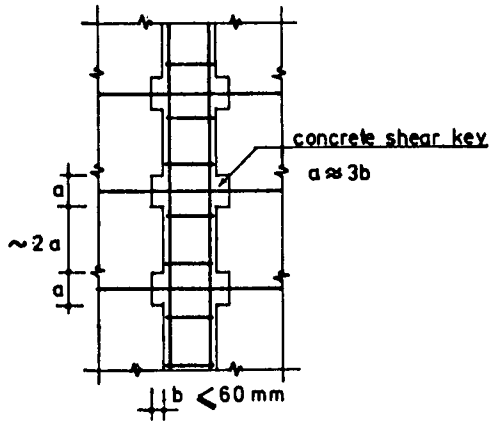

Fig.6.10 - Typical detail of reinforced concrete tie-column with shear key

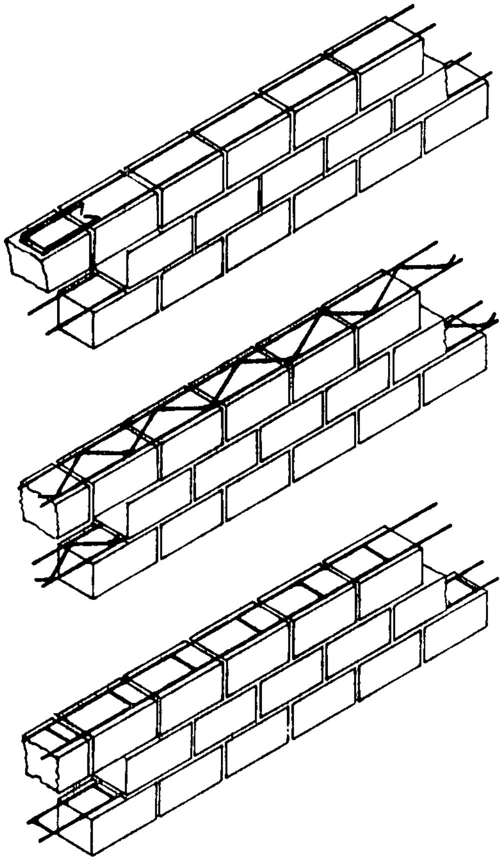

Fig.6.11 - Typical horizontal reinforcement forms

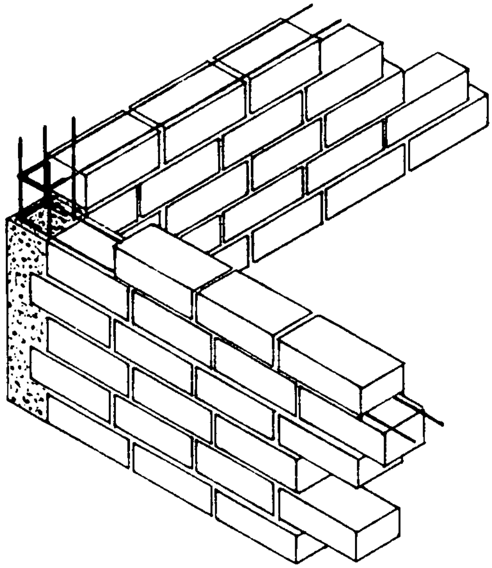

Fig.6.12 - Detail of anchoring the horizontal reinforcement into the tie-column

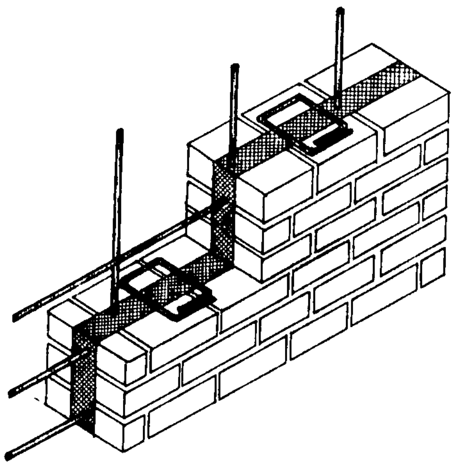

Fig.6.13 - Reinforced grouted masonry

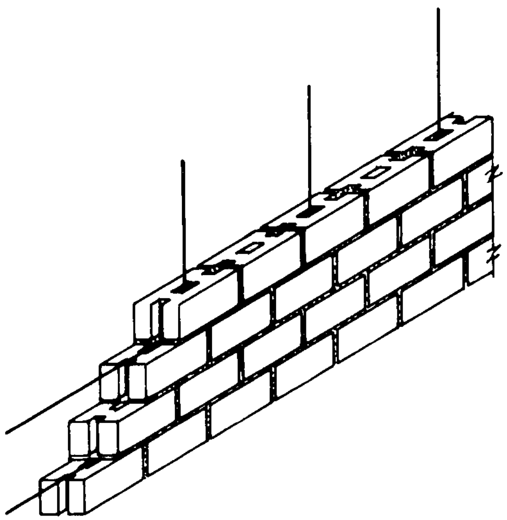

Fig.6.14 - Reinforced hollow unit masonry

7. DESIGN ASSUMPTIONS AND PROCEDURES

During the past earthquakes in the Balkan region, most of the loss of human lives occurred due to the collapse of traditionally constructed masonry buildings. In order to avoid future loss of life, the Balkan countries introduced different earthquake resistance provisions in their Codes, concerning the construction and design of masonry buildings in earthquake prone areas.

It is not easy to unify the requirements of different Codes as far as the values of different coefficients and the different methods for design and analysis of structural elements and buildings are concerned. However, some general assumptions and procedures concerning analysis and design of masonry buildings, subjected to earthquakes are given in this section.

7.1 Fundamental Principles for Earthquake Resistant Design and Analysis of Masonry Buildings

The construction of earthquake resistant buildings is governed by economical Considerations rather than technical issues. It is possible to construct buildings which will resist even the strongest expected earthquake with no damage, but the construction of such a building will be very expensive, and acceptable in only exceptional cases (hospitals, fire stations, etc).

The aim of the earthquake resistant design of buildings is to ensure the adequate, acceptable degree of earthquake protection, which means that:

- in the case of moderate intensity earthquakes, which may occur rather frequently during the building's life period, the building should not suffer structural damage;

- in the case of the maximum expected intensity earthquake in the region, the building should not suffer total or partial collapse, hence saving the human lives. The expected damage should be limited so that the building can be quickly repaired and restored to its normal functioning.

7.2 Seismic Actions

7.2.1 General

The dynamic effect of ground motion, acting on the building can be affected in different ways, according to desired level of accuracy and complexity of the analysis, as given below:

- by means of equivalent statical horizontal loads, independent of the dynamic characteristics of the building;

- by means of equivalent statical horizontal loads, depending on the dynamic and ductility characteristics of the building;

- by means of dynamic analysis using recorded or artificially created accelerograms.

When designing the building to resist earthquakes, both horizontal and vertical seismic action should be taken into account. The effect of vertical action, however, is not significant in case of structural walls, and needs usually be checked for large span beams and cantilever elements only.

7.2.2 Horizontal Seismic Actions

Base Shear

In the case of masonry buildings of limited height and plan dimensions, the horizontal seismic action is usually given in terms of "base shear", which represents the resultant of the inertia forces distributed over the height of the building, acting horizontally in both principal directions of the building.

The magnitude of the base shear is determined in accordance with the following formula:

$$V = C\ W, \qquad \ldots (7.1)$$

where:

V - base shear,

C - base shear coefficient,

W - weight of the building above the ground level, taking into account the total weight of the structure and secondary elements, as well as the prescribed portion of the floor live load and the effective snow load.

Base Shear Coefficient

The base shear coefficient is usually given in terms of several factors, which take into account regional seismicity, local soil counditions, importance of the building, and dynamic and structural characteristics of the building.

In order to develop a numerical value for the base shear coefficient different quantitative values for the factors mentioned above have been assigned by the various Balkan countries. This Manual does not give all of these values, but provides ranges for some of them to show representative values.

Regional seismicity. The regional seismicity is generally defined in terms of effective peak acceleration, and each country has selected values for use in design. Table 7.1 defines three general seismic zones and does give a range of effective peak accelerations for each zone. Moreover, Table 7.1 offers a correspondance with the MSK-64 intensity scale.

Table 7.1 - Values of Ground Accelerations According to the MSK-64 Intensity Scale

Seismic zone	Low	Moderate	High
Degree of seismic intensity according to MSK-64 intensity scale	9	8	7
Ground accelerations (a/g) according to MSK-64 intensity scale	0.20-0.40	0.10-0.20	0.05-0.10

Dynamic coefficient represents the dynamic amplification factor and is related to the natural period of the building and soil characteristics. Again, each country has adopted different idealized response spectra in its seismic Code.

The fundamental periods of normal height masonry buildings usually do not exceed 0.5 s, therefore the value of dynamic coefficient might be taken as a constant which is the maximum value given by the response spectrum.

Behaviour factor takes into account the ductility and energy dissipation capacity of the structural system. In the case of masonry buildings, distinction is made between the behaviour of plain masonry and confined or reinforced masonry systems.

Accordingly, the experimental investigations and post earthquake observations have shown, that the following values of the behaviour factors of masonry structural systems can be proposed:

- plain masonry: $K \doteq 1.0$,
- confined and reinforced masonry: $K = 1.5 - 2.5$.

7.2.3 Distribution of Seismic Forces Over the Building Height

In the case of masonry buildings the assumption of linear distribution of seismic inertia forces over the building height, corresponding to the fundamental mode shape, is usually taken into account.

The story seismic forces S_i are then determined as follows:

$$S_i = V \frac{H_i \, W_i}{\sum_{i=1}^{n} H_i \, W_i} , \qquad \ldots (7.2)$$

where:

S_i - seismic force, acting at i-th story level;

H_i - distance of the i-th story from the base of the structure;

W_i - weight of the i-th story;

n - number of stories.

7.2.4 Torsion

The structural analysis for seismic action is carried out, as a rule, by considering the seismic forces acting at the story mass centres, separately in each of the two principal directions of the structural system.

In the case where the position of the story mass centre does not coincide with the position of the story stiffness centre, torsion takes place.

When analyzing the earthquake resistance of masonry buildings, torsional effects are taken into account by considering the eccentricity between the story mass and stiffness centres, as given in the following formula:

$$e = e_1 \pm e_2 , \qquad \ldots (7.3)$$

where

e_1 - eccentricity of the story mass centre with respect to the story stiffness centre;

e_2 - conventional additional eccentricity, expressing the effect of the differences of the ground motion along the building length. Usually, this value is taken equal to 5% of the maximum building dimension in plan.

7.2.5 Vertical Seismic Actions

The vertical seismic forces are determined by multiplying the gravitional loads of the respective structural elements by the coefficient

$$C_v = \pm\, 0.7\; C, \qquad \ldots (7.4)$$

or similar, as defined in national Codes.

7.2.6 Seismic Loads for Non-Structural Elements

The total seismic load acting on a non-structural element may be determined by multiplying its own weight by the coefficient C, as given Table 7.2, depending on the seismic zone.

Table 7.2 - Values of Coefficinet C

Type of non-structural element	Direction of seismic action	L	M	H
Partition walls	Perpendicular to the plane of the element	0.08	0.15	0.30
Gable end walls or other cantilever walls	Perpendicular to the plane of the element	0.20	0.40	0.80
Ornamentations, chimneys	Any direction	0.30	0.60	1.20

7.3 Earthquake Resistance Analysis of Masonry Buildings

Either elastic or non-linear methods can be used for earthquake resistance analysis of masonry buildings. Mathematical models for calculation of the effect of lateral loading on masonry buildings are more or less the same in both cases. Non-linear methods, however, give the possibility to estimate the realistic behaviour of masonry buildings when subjected to earthquakes.

7.3.1 Mathematical Model

Masonry building represents a shear-wall structural system, so basic principles, hypotheses and mathematical models used in aseismic design of such kind of structural systems can be applied to masonry buildings, too.

However, due to specific mechanical characteristics of masonry, the mathematical models used for analysis of reinforced concrete structures can be simplified, especially having in mind the limited height of masonry buildings, and hence the prevalence of shearing effect of lateral earthquake loading on walls.

Masonry shear walls are usually considered to act as vertical cantilevers, fixed at their base and interconnected by means of floors, which are supposed to act as rigid horizontal diaphragms, distributing lateral load onto individual walls according to their stiffnesses.

This assumption, however, is valid only where simple cantilever shear walls located in the same plane are linked between them by flexible floor slabs, which ensure that the moment transfer between the walls is minimised (Fig. 7.1). In this case, openings within the wall must be kept small enough to ensure that the vertical cantilever action is not affected.

Masonry shear walls are most often pierced by window and door openings, as shown in Figs. 7.2 and 7.3. In this case, however, the assumption of shear walls acting as vertical cantilevers, in no more valid. Under lateral loading the weak point may be either the piers (Fig.7.2) or the spandrel beams (Fig. 7.3).

If the piers represent the weak point, which is the more common case, they are required to exhibit substantial ductility. Damage and hence plastic deformation are concentrated in the piers of generallly the lowest story. Consequently, the failure load (earthquake resistance of the building) will be attained when the flexural or shear strength of the lowest level piers is reached. Other structural elements will not have yielded at this stage.

If, on the other hand, the openings in masonry walls will be of such proportions that spandrels will be relatively weaker than piers, the behaviour will approximate coupled shear walls (Fig.7.3).

However, to satisfy the high ductility demand generated in the spandrel beams diagonal reinforcement is generallly required. Such a system is unsuitable for structural masonry, so rapid strength and stiffness degradation is likely to occur. This degradation results in an increase in wall moments: the walls finally behave as simple linked cantilevers and they must be designed accordingly.

7.3.2 Wall Stiffnesses

The horizontal earthquake forces are distributed onto the individual walls proportionally to their stiffnesses, assuming rigid horizontal diaphragm floor action.

The stiffness of wall element depends on its dimensions, its material characteristics (elastic and shear modulus) and the conditions of support at top and bottom.

For a pier or wall element, fixed at top and bottom, the initial, elastic stiffness is given by:

$$K = \frac{G}{1.2\ h} \frac{A}{1 + 0.83 \frac{G}{E} \left(\frac{h}{l}\right)^2} \qquad \ldots (7.5)$$

For a cantilever pier or wall element, the initial, elastic stiffness is given by:

$$K = \frac{G}{1.2\ h} \frac{A}{1 + 3.33 \frac{G}{E} \left(\frac{h}{l}\right)^2} , \qquad \ldots (7.6)$$

where:

K - initial, elastic stiffness of wall,

A - horizontal cross-sectional area of wall,

h - height of wall,

l - length of wall,

E - elastic modulus of wall,

G - shear modulus of wall.

7.3.3 Calculation Procedure

Elastic Method

The following general sequences of calculation are usually carried out in case of the elastic analysis:

- masses of the building, concentrated at floor levels are determined and vertical loads in walls are calculated;

- horizontal seismic forces or base shear acting on the building in each of the principal directions are calculated according to Code requirements;

- using appropriate mathemacital models, the stiffnesses of individual wall elements are evaluated;

- story shear is distributed onto the individual walls according to their stiffnesses, and sectional forces (bending moments, shear forces, and axial forces) are calculated;

- finally, sectional capacities are calculated and compared to the corresponding sectional forces.

Non-linear method

In this section a non-linear method for earthquake resistance analysis is briefly presented. Although this method has simplified and approximate character, both the elastic and non-linear part of the behaviour of masonry building, subjected to lateral earthquake loads, can be estimated.

Again, masses of the building, concentrated at floor levels and vertical loads in walls are calculated first. Then:

- strength and deformability characteristics of individual walls in the shape of idealized H-δ diagrams are calculated assuming shear wall with pier action (see Fig.7.2), i.e. individual walls in the story being fixed at both ends;

- position of story stiffness- and story mass-centres is determined and initial torsional rotation angle is calculated;

- story force-displacement diagram is calculated by means of step-wise increasing the displacement of story mass-centre, following the idealized H-δ diagrams of individual walls and correcting the torsional rotation angle, if necessary (Fig.7.4);

- earthquake resistance of building is calculated in terms of base shear coefficient, dividing the maximum calculated resistant force by the vertical loads of the building. The calculated value of base shear coefficient is compared to the required value as given in the Code.

Interpretation of results obtained by means of the above described method gives possibility to observe the behaviour of all the walls in the story when subjected to lateral load. The weak points in the structural system can be clearly seen and the necessary measures can be provided in order to improve strength and ductility characteristics of critical elements.

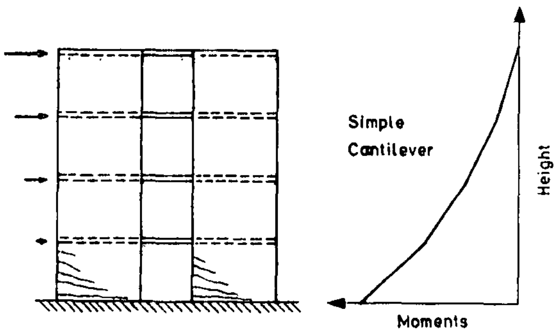

Fig.7.1 - Simple cantilever shear walls

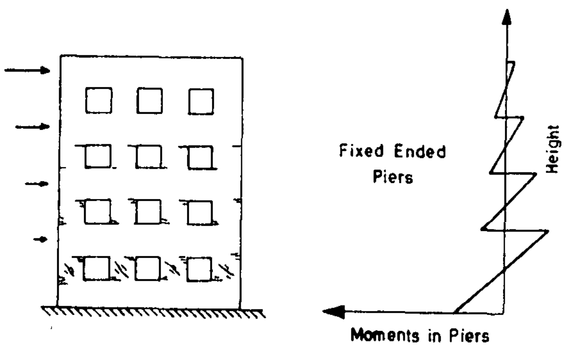

Fig.7.2 - Shear wall with pier action

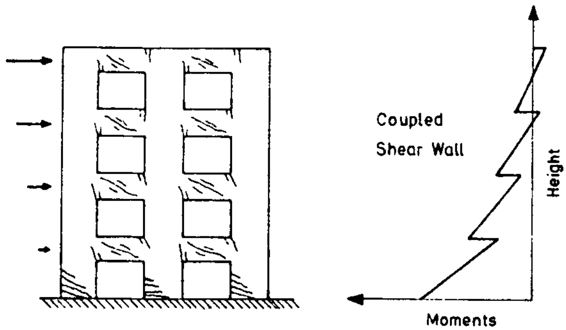

Fig.7.3 - Coupled shear wall

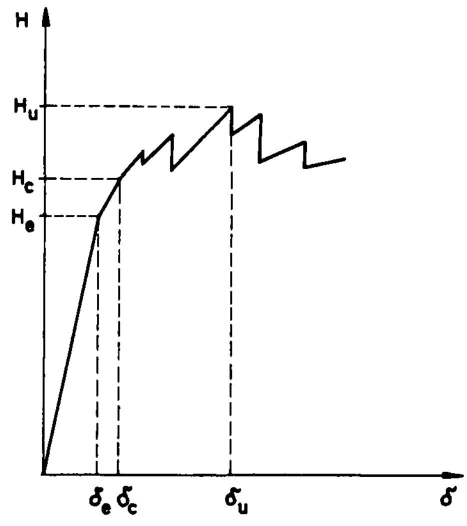

Fig.7.4 - Typical calculated story H-δ diagram

8. ANALYSIS AND DESIGN OF STRUCTURAL WALLS

Post earthquake observations and the experimental investigations, carried out in many countries have shown, that two modes of failure of structural walls can occur, depending on their geometry (height to length ratio) and mechanical characteristics of materials:

- flexural failure occurs when crushing of the compressed zones occurs after the horizontal, tensile cracks on the tensioned sides have diminished the effective cross-sectional area of the wall. Characteristic compressive strength of the wall is the governing parameter, which defines the flexural resistance of wall.

- shear failure, or diagonal tension failure occurs when diagonal cracks either following the mortar joints or passing through the units develop in the wall. Horizontal, tensile cracks may also occur before the shear failure takes place. Characteristic tensile strength of the wall is the governing parameter, which defines the shear resistance of wall.

In the following sections some formulae for the evaluation of the flexural and shear capacity of wall elements, constructed in different construction systems, will be briefly discussed.

8.1 Plain Masonry

8.1.1 Flexural Capacity

When calculating the flexural capacity of structural wall subjected to simultaneous vertical and lateral loading, it is usually assumed that the masonry cannot carry tensile stresses. In this case tension cracks will start at the tensioned sides, and will propagate until the wall fails in compression. At that instance (Fig.8.1) the actual shape of stress diagram can be idealized as rectangular stress block, the resultant of which equals the vertical load in wall.

Accordingly, the flexural capacity is given by:

$$M_u = \frac{\sigma_o\, t\, l^2}{2} \left(1 - \frac{\sigma_o}{f_{wc,k}}\right), \qquad \ldots (8.1)$$

where:

M_u - flexural capacity of wall,

σ_o - the average vertical compressive stress in the wall due to vertical load,

$f_{wc,k}$ - characteristic compressive strength of the wall,

t - thickness of the wall,

l - length of the wall,

h - height of the wall.

8.1.2 Shear Capacity

The relationship between the average ultimate shear stress in the wall at failure, which defines the shear capacity of the wall, and the average vertical stress in the wall due to vertical loading, can be expressed by means of the characteristic tensile strength of the wall in the following way (Fig.8.2):

$$\tau_u = \frac{f_{wt,k}}{\xi} \sqrt{\frac{\sigma_o}{f_{wt,k}} + 1} \;, \qquad \ldots (8.2)$$

The ultimate shear capacity of the wall can be obtained by multiplying the value of the average ultimate shear stress by the horizontal cross-sectional area of the wall:

$$V_u = A \frac{f_{wt,k}}{b} \sqrt{\frac{\sigma_o}{f_{wt,k}} + 1} \;, \qquad \ldots (8.3)$$

where:

V_u - ultimate shear capacity of the wall,

τ_u - the average ultimate shear stress in the wall,

σ_o - the average vertical compressive stress in the wall due to vertical load,

$f_{wt,k}$ - characteristic tensile strength of the wall,

A - horizontal cross sectional area of the wall,

ξ - shear stresses distribution coefficient.

The value of the shear stresses distribution coefficient "ξ" represents the ratio between the maximum and the average shear stress in the critical section. It is depending on the geometry of the wall (height to length ratio) as well as on the loading conditions(vertical load eccetricity to length ratio). The value of "ξ" varies from 1.5 in the case of slender walls (height to width ratio greater than 1.5) to 1.0 in the case of wide walls (height to ratio less than 1.0).

8.1.3 Interaction Diagrams

Since most walls are under a combined state of stress, failure theories are employed to determine the failure mode. Interaction diagrams can be prepared that express the relationship between the vertical load and the lateral resistance of walls in a nondimensional form. An example of such an interaction diagram, presenting both flexural and shear capacity of plain masonry walls is presented in Fig.8.3, for a given quality of masonry, which expresses the ratio between the characteristic tensile and the characteristic compressive strength.

8.2 Confined Masonry

Depending on the structural layout, tie-columns can be located either on one or both ends of the masonry wall or in its middle part, so generally speaking, all these cases should be taken into account when determining the resistance of the confined masonry wall to lateral loading. A possible approach will be explained for the case of tie-columns, located on both ends of the masonry wall, which represents the confined masonry panel.

8.2.1 Flexural Capacity

The flexural capacity of a confined masonry panel is defined when the ultimate compressive strain at the compressed side is reached with simultaneous yielding of the tensile reinforcement at the tensioned side (Fig.8.4).

The flexural capacity, expressed as the ultimate bending moment, can be obtained as:

$$M_u = N\, e_u, \qquad \dots (8.4)$$

where:

M_u - ultimate flexural capacity of the confined masonry panel,

N - vertical force acting on the panel,

e_u - ultimate eccentricity of the vertical force.

In the case of rectangular cross sections, the value of e_u is determined as:

$$e_u = \frac{0.85\ .\varphi\, f_{wc,k}}{N}\ t\ a\ (l_s - \frac{a}{2}) + A_{c,c}(n-1)\ (l_s - \frac{d}{2}) - x, \qquad \dots (8.5)$$

where

$$a = \frac{N + A_{s,t}\, f_{s,y}}{0.85\varphi\, t\ f_{cc,k}} - \frac{A_{c,c}\ (n-1)}{t}\ , \qquad \dots (8.6)$$

and

$$n = \frac{f_{wc,k}}{0.85\ f_{cc,k}}\ . \qquad \dots (8.7)$$

a - width of the equivalent compressive stress block,

n - equivalence coefficient for concrete compressive strength relative to the compressive strength of the masonry,

$f_{wc,k}$ - characteristic compressive strength of the masonry,

$f_{cc,k}$ - characteristic compressive strength of concrete,

$f_{s,y}$ - yield stress of the reinforcing steel,

$A_{c,c}$ - cross-sectional area of the compressed tie-column,

$A_{s,t}$ - cross-sectional area of the tensile reinforcement,

t - thickness of the confined wall,

d - depth of the tie-column,

l_p - length of the panel,

l_s - lever arm of the resultant tension force,

φ - buckling coefficient

8.2.2 Shear Capacity

The shear capacity of a confined masonry panel is defined when diagonal cracking occurs in the masonry panel due to the diagonal tension (Fig.8.5).

The shear capacity can be evaluated by using the following equation:

$$V_u = t\ l\ f_{wt,k} \left[\frac{h}{2\ l} + \sqrt{\left(\frac{h}{2\ l}\right)^2 + \frac{N}{A_e\ f_{wt,k}} + 1} \right] \quad \ldots (8.8)$$

where:

V_u - ultimate shear capacity of the confined masonry panel,

N - vertical force acting on the panel,

$f_{wt,k}$ - characteristic tensile strength of the masonry,

t - thickness of the confined wall,

l - length of the confined wall,

h - height of the confined wall,

A_e - equivalent cross-sectional area of the panel.

8.3 Reinforced Masonry

8.3.1 Flexural Capacity

The flexural capacity of a vertically reinforced masonry wall can be evaluated by following the same approach as that used for reinforced concrete (linear strain distribution across the section), as shown in Fig.8.6.

In the case of concentrated vertical reinforcement at both ends of the wall element, and subjected to simultaneously acting vertical and lateral loading, the flexural capacity, expressed as the ultimate bending moment, can be obtained as:

$$M_u = N \cdot e_u , \quad \ldots (8.9)$$

where:

M_u - ultimate flexural capacity of the reinforced wall,

N - vertical force, acting on the wall,

e_u - ultimate eccentricity of the vertical force.

In the case of equal amounts of reinforcement being placed at both ends of the wall, and assuming that yielding of reinforcement in both tension and compression takes place simultaneously, the following expression is obtained:

$$M_u = \frac{\sigma_o\ t\ l^2}{2}\ (1 - \frac{\sigma_o}{f_{wc,k}}) + 2\ A_s\ f_{s,y}\ (\frac{1}{2} - 1) \qquad \ldots (8.10)$$

where:

M_u - ultimate flexural resistance of reinforced wall,

σ_o - the average vertical compressive stress in the wall,

$f_{wc,k}$ - characteristic compressive strength of the wall,

$f_{s,y}$ - yield limit of the reinforcement,

A_s - cross sectional area of reinforcement,

t - thickness of the wall,

l - length of the wall,

h - height of the wall.

8.3.2 Shear Capacity

It might be supposed that uniformly distributed vertical reinforcement crossing potential 45^o failure planes would be equally as efficient in shear transfer as horizontal reinforcement, and thus flexural steel near the centre of the wall could not be utilized to carry the shear load. However, once cracking has initiated the shear force tends to displace the upper portion of the wall horizontally, past the lower portion, rather than open the crack perpendicular to the crack plane (Fig.8.7).

Under these conditions the horizontal steel resists the shear force by direct tension, but vertical steel resists the force by dowel action, which can only be mobilised if substantial horizontal displacement occur across the crack. Taking this into consideration, it can be shown, that in optimum conditions, the vertical steel can carry the shear, equal to:

$$V_v = 0.20\ d_b^2\ f_{s,y}\ , \qquad \ldots (8.11)$$

whereas the horizontal steel carries

$$V_h = \frac{\pi\ d_b^2}{4}\ f_{s,y}\ , \qquad \ldots (8.12)$$

d_b - diameter of reinforcing bar.

In other words, the same amount of steel, placed vertically, can carry 25% of shear, carried by horizontally placed steel.

Generally speaking, the shear capacity of a vertically and horizontally reinforced wall can be given as:

$$V_u = \phi_1\ \tau_u\ A + \phi_2\ A_{s,v}\ f_{s,y} + \phi_3\ A_{s,h}\ f_{s,y} \qquad \ldots (8.13)$$

where:

V_u - ultimate shear resistance of reinforced wall,

A - horizontal cross-sectional area of the wall,

$A_{s,v}$ - cross-sectional area of vertical reinforcement,

$A_{s,h}$ - cross-sectional area of horozontal reinforcement,

τ_u - average ultimate shear stress in the wall,

$f_{s,y}$ - yield limit of reinforcing steel,

ϕ_1, ϕ_2, ϕ_3 - capacity reduction factors, determining the contribution of respective components to the shear resistance of walls.

As the test results have shown, the contribution of the wall to the shear capacity, after the cracking takes place, can be neglected, and the effect of reinforcement only should be taken into account. For the case of height to length ratio of walls is greater than 1.0:

$$V_u = \phi_2 A_{s,v} f_{s,y} + \phi_3 A_{s,h} f_{s,y} \qquad \ldots (8.14)$$

The following values for capacity reduction factors should be used:

$\phi_2 = 0.20$, and

$\phi_3 = 0.90$.

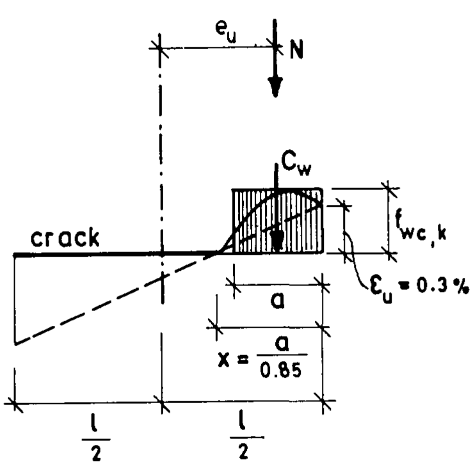

Fig.8.1 - Ultimate stress state at flexural failure of plain masonry wall

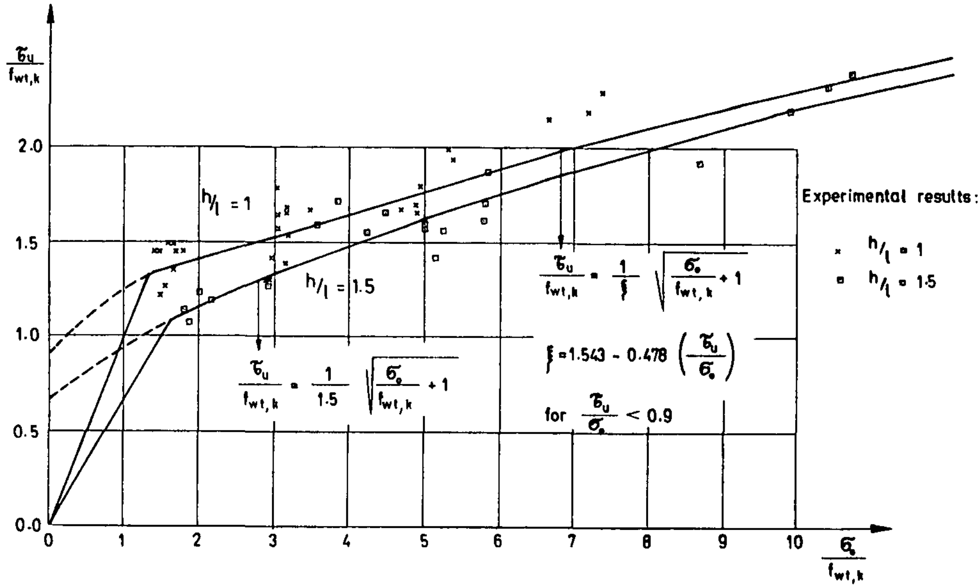

Fig. 8.2 - Ultimate shear strength of plain masonry walls in dependance of vertical load in nondimensional form

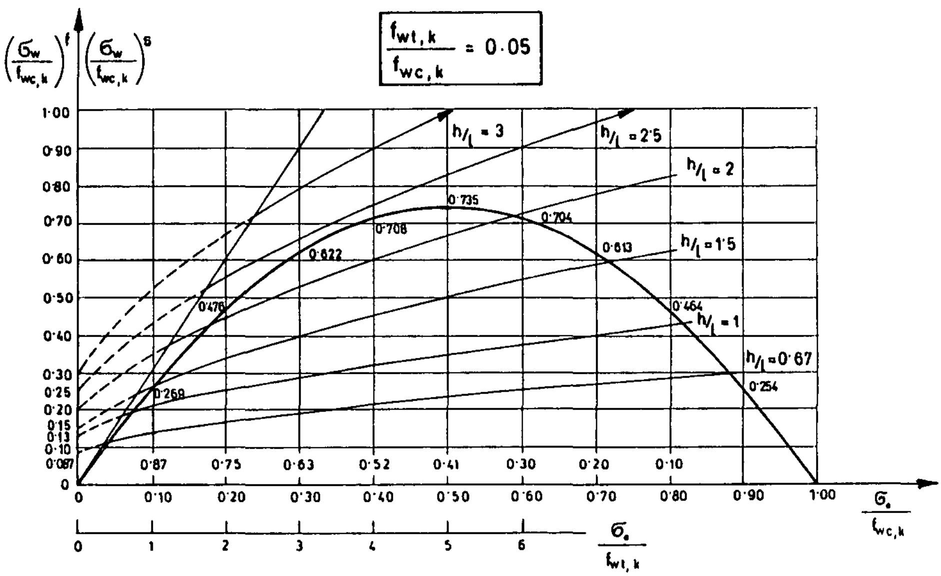

Fig. 8.3 - Typical lateral resistance versus vertical load interaction diagrams for plain masonry walls

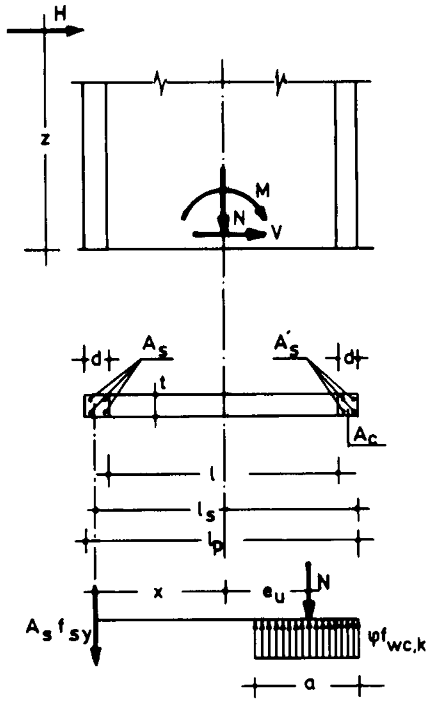

Fig. 8.4 - Ultimate stress state at flexural failure of confined masonry wall

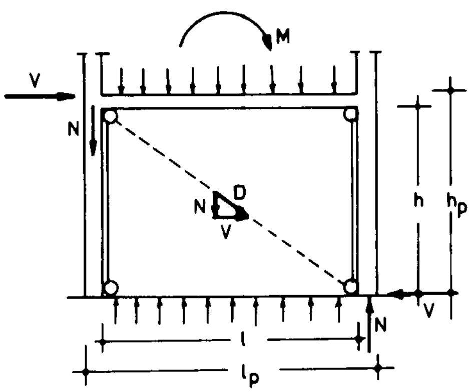

Fig. 8.5 - Mathematical model for calculation of shear capacity of confined masonry wall

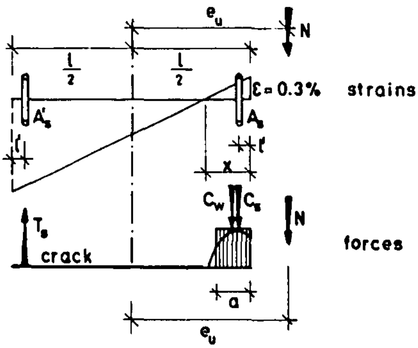

Fig. 8.6 - Ultimate stress state at flexural failure of reinforced masonry wall

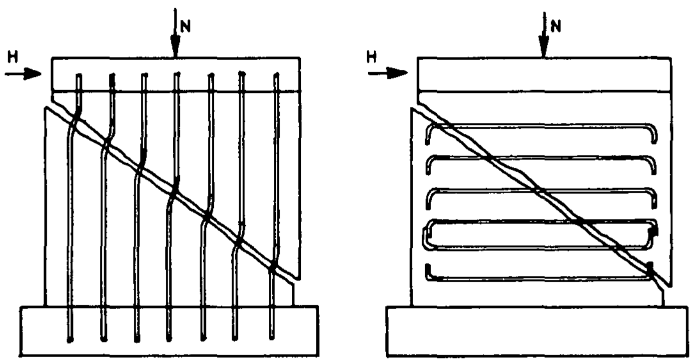

Fig. 8.7 - Mechanism of shear transfer in vertically or horizontally reinforced masonry wall

9. FOUNDATIONS

The foundation system transmits both vertical and horizontal forces from the structure to the ground.

In order to ensure the earthquake resistance of masonry building, its substructure (including foundations and or basement walls) should be able to distribute the forces uniformly to the ground, according to soil bearing capacity. Of course, care should be taken when selecting the construction site in order to avoid the phenomena of land-sliding and soil liquefaction.

The foundation system of masonry buildings should consist of continuous footings, designed according to the current practice. The depth of the footings should go below the level of deep freezing as well as below the level of shrinkage cracks in clayey soils. However, the depth of all footings of the building foundation system should preferably be at the same level. The footings are made of plain or reinforced concrete, using minimum concrete grade C 5. Specific provisions should be considered in the case of unfavourable soil conditions, high water-table level, or similar.

Basement walls can be constructed of various materials, depending on the number of stories, structural characteristics, and soil conditions:

- uncoursed rubble stone masonry or rubble concrete, can be used for buildings of maximum two stories high in seismic cone,of low intensity (L) and one story high in seismic zones; of moderate and high intensity (M and H);

- plain or reinforced concrete of minimum grade C 10;

- masonry, in the case of internal basement walls.

The thickness of basement walls should be at least equal to the thickness of walls in the ground floor. The walls must be designed to resist the out-of-plane earth pressure.

Openings in the basement walls should be located in the same position in plan as the openings in the upper structure, and should not reduce the horizontal cross-sectional area of the wall by more than 20%. The lintels should be made of cast-in-place reinforced concrete, and monolithically connected to the basement floor tie-beams.

It is recommended that tie beams be incorporated into the basement walls, on the top and bottom of the wall, if the height of the basement wall is greater than 1.50 m. The reinforcement of the tie-beams shall meet the requirements of Chapter 10.2.1 (Figs.9.1, 9.2 and 9.3).

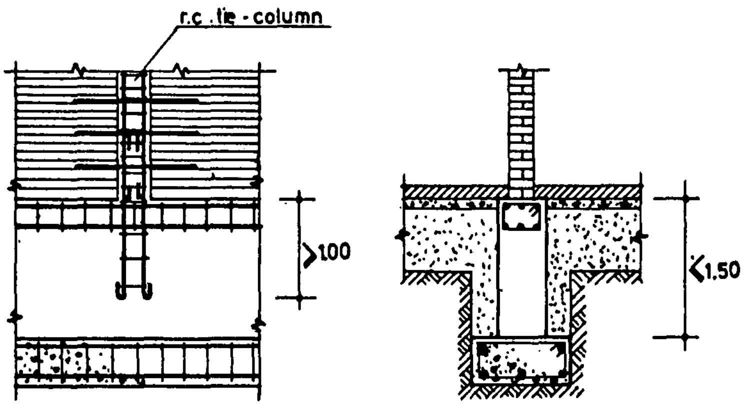

Fig.9.1 - Foundation system with single reinforced concrete tie-beam

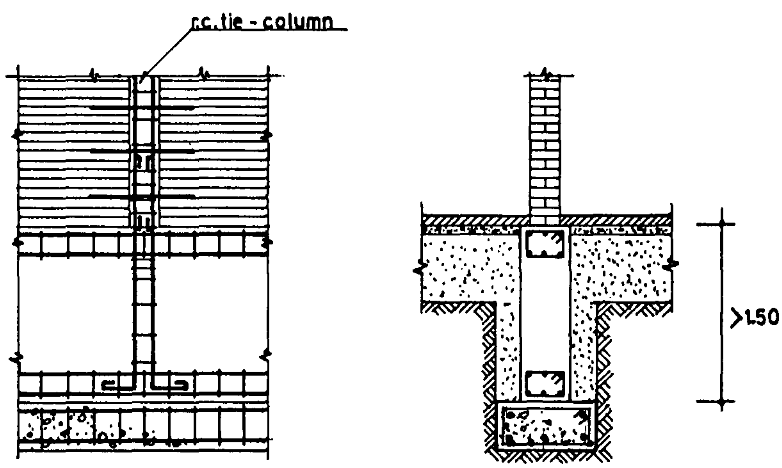

Fig.9. 2 - Foundation system with double reinforced concrete tie-beams - building with basement

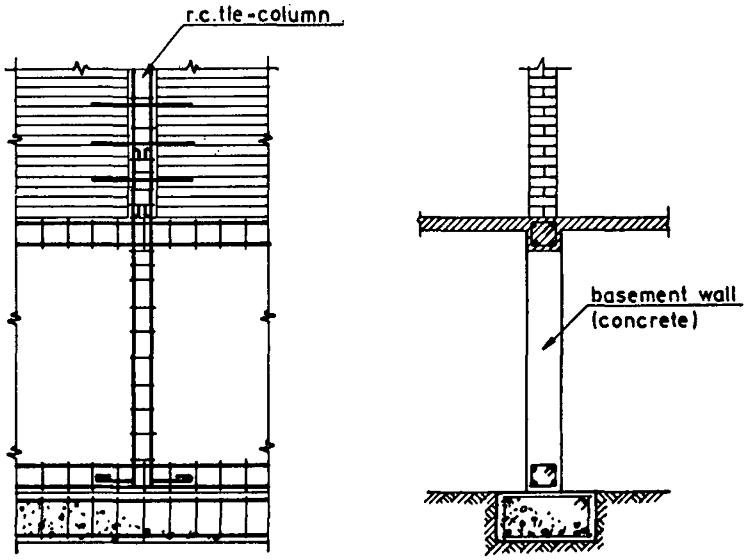

Fig.9.8 - Foundation system with double reinforced concrete tie-beams-building with basement

10. FLOORS, TIE BEAMS AND ROOFS

10.1 Floors

In order to ensure spatial interaction of structural walls and the more appropriate distribution of seismic forces onto the walls, floors should preferably be designed to act as rigid horizontal diaphragms, firmly connected to the walls at each floor level. In this respect, the following requirements should be taken into consideration.

- each floor should be situated in a single plane, avoiding sharp dislevelments;

- the rigid behaviour of horizontal diaphragm should not be altered by the presence of discontinuity, such as stairways. Large opening zones should be strengthened with special reinforcement or tie-beams;

- two-way slabs are preferable to one-way slabs, because they distribute vertical loads more uniformly onto the structural walls.

10.1.1 Concrete Floors

The following types of floors are recommended to be used in seismic regions:

a.) cast-in-place reinforced concrete floors cast simultaneously with reinforced concrete tie-beams;

b.) floors made of precast elements must be well connected to the tie-beam reinforcement. Reinforced concrete topping having minimum thickness of 40 mm should be provided, and should be cast simultaneously with reinforced concrete tie beams. A reinforcing mesh of 5 mm diameter bars at 200 mm intervals in both orthogonal directions has to be placed at the middepth of the topping;

c.) floors made of precast large panels with anchoring bars or loops, monolithically connected to tie beams.

10.1.2 Wooden Floors

In the case of stone-masonry and adobe buildings, constructed in rural areas, wooden floors are still widely used. Wooden floors constitute a flexible floor diaphragm and their use should be limited to one story buildings.

When wooden floor is used, timber joists have to be anchored into the tie-beams by means of well distributed steel anchors, as shown in Fig.10.1.It is recommended that the wooden floor be stiffened by means of planks nailed orthogonally to the first layer of planks, as shown in Fig.10.2.

10.1.3 Balconies and Overhangs

It is recommended that the span of cantilever structural elements, such as balconies and overhangs should be limited to:

- 1.20 m for cantilever slabs cast continuous with the floor slab (Fig.10.3);

- 0.50 m for cantilever slabs fixed into the tie beam, without continuity with the floor slab (Fig.10.4).

The stability of balconies and overhangs, as well as other horizontal cantilever elements, should be verified by taking into account the effect of vertical seismic loads.

10.1.4 Lintels

For vertical loads lintels function as beams, supporting the weight of the wall and floor above the openings.

Lintels can be made either of cast-in-place reinforced concrete, precast reinforced concrete, or reinforced masonry elements (Fig.10.5).

Depending on the distance from the top of the opening to the top of the above floor,cast-in-place lintels can be cast either independently or monolithically with the floor tie-beam. The latter solution offers better behaviour when subjected to earthquake.

The minimum bearing length of lintels should not be less than 250 mm on each end.

Where reinforced concrete tie-columns framing the openings are used, the reinforcement of lintels should be monolithically connected to them.

As a rule, the lintel width should be equal to the thickness of the wall.In the case of external walls, the lintel width can be reduced in order to accomodate the thermal insulation layer provided on the external face of the wall.

10.2 Tie-Beams

Tie-beams should be constructed on the top of all structural walls (i.e. load-bearing and bracing walls) at each story level, firmly connected to floors and roof.

Tie-beams represent a horizontal framing system which:

- transfers the horizontal shear induced by the earthquake from the floors to the structural walls;

- connects the structural walls;

- ensures the proper interaction between precast floor units, and increase the rigidty of horizontal diaphragms;

- when used with vertical tie-columns they frame the masonry walls, thereby improving their strength and ductility.

10.2.1 Reinforced Concrete Tie-Beams

The recommended structural details for reinforced concrete tie-beams are presented in Figs.10.6, 10.7, 10.8 and 10.9.

The following detailing provisions for tie-beams should be observed:

- as a rule, the width of the tie-beam should be equal to the thickness of wall. In the case of external walls, the width of the tie-beam can be reduced in order to accomodate the thermal insulation layer provided on the external face of the wall;

- the depth of the tie beam-should be at least equal to the thickness of the floor, and not less than 150 mm;

- minimum concrete grade C 15 (characteristic compressive strength 15 MPa) should be used for construction;

- the minimum amount of longitudinal reinforcement for tie-beams depends on the building dimensions and on the seismic zone. The minimum amount is shown in Table 10.1:

Table 10.1 - Recommended Longitudinal Reinforcement of Tie-Beams

Number of stories	Seismic zone		
	Low	Moderate	High
1	4 ϕ 10	4 ϕ 10	4 ϕ 12
2	4 ϕ 10	4 ϕ 10	4 ϕ 14
3	4 ϕ 10	4 ϕ 12	4 ϕ 16
4	4 ϕ 12	4 ϕ 14	-
5	4 ϕ 14	-	-

- stirrups of 6 mm diameter bars spaced at 250 mm invervals should be provided as a minimum.

Intermediate tie-beams should be constructed when the story height exceeds 4 m . The vertical distance between the two successive tie-beams should not be more than 4 m in seismic zone L and not more than 3 m in seismic zones M and H.

10.2.2 Wooden Tie-Beams

In adobe buildings, wooden tie-beams are commonly used to connect the walls. Wooden tie-beams should consist of 100 x 100 mm asphalt treated longitudinal members, placed along both faces of the wall, and connected together at 0.50 m intervals by means of transverse 50 x 100 mm members which are nailed to the longitudinal ones, as shown in Fig.10.10. The space between the longitudinal members should be filled with crushed stone.

Wooden tie-beams should be provided at four levels: above the foundation, above and below the windows, and under the roof, as shown in Fig.10.11.

10.3 Roofs

In order to transfer the inertia forces to the supporting walls, the roof system must be adequately braced in both orthogonal directions, and must be properly anchored to the reinforced concrete tie-beams which are constructed on top of the bearing walls (Fig.10.12).

Structural systems which exert lateral forces on the attic masonry elements should be avoided. If such situations cannot be avoided the attics should be anchored to the uppermost floor by means of reinforced concrete tie-columns.

Light roof structural systems are preferable to reduce the induced inertia forces. Where possible asbestos sheeting or tiles should be used instead of heavy roofing tiles.

Where the precast elements are used, reinforced concrete cast-in-place topping, with a minimum thickness of 40 mm must be provided which is reinforced by 5 mm diameter bars placed at 200 mm intervals in both directions. In such a case the ends of the precast elements must be embedded into the reinforced concrete tie-beam, along the complete perimeter of the roof.

In the case of adobe buildings, roofs should be as light as possible and should have at least 0.50 m eaves to the outer side of the building. Earth covered roofs, as shown in Fig.10.13 are only permitted in seismic zone L, provided that the earth cover is not thicker than 150 mm.

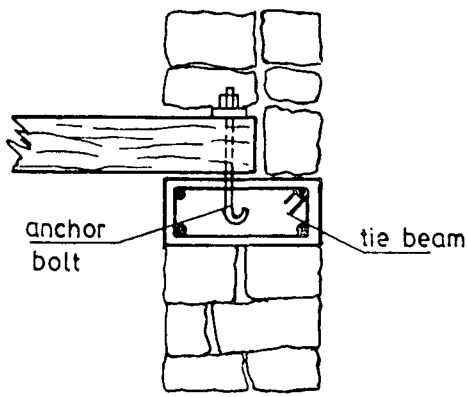

Fig.10.1 - Anchoring of timber joists into the reinforced concrete tie-beam

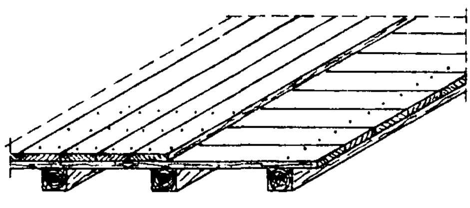

Fig.10.2 - Stiffening of wooden floor

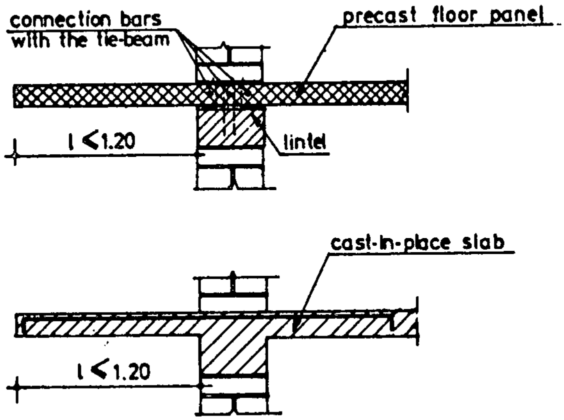

Fig.10.3 - Cantilever slab, having a continuity with the floor slab

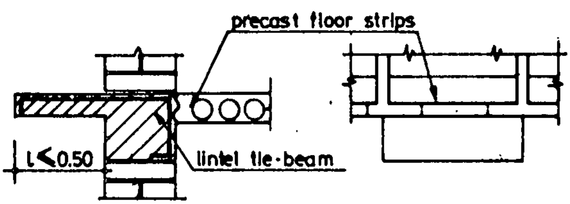

Fig.10.4 - Cantilever overhang,fixed into the tie-beam

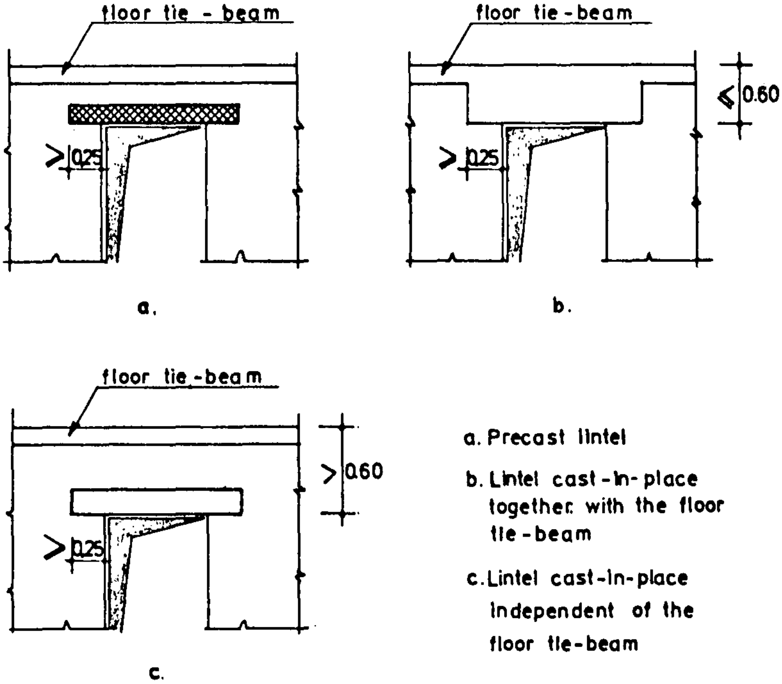

Fig.10.5 - Types of lintels

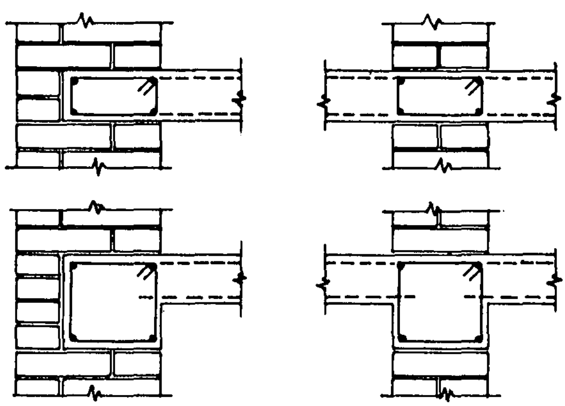

Fig.10.6 - Tie-beams in the case of cast-in-place reinforced concrete slabs

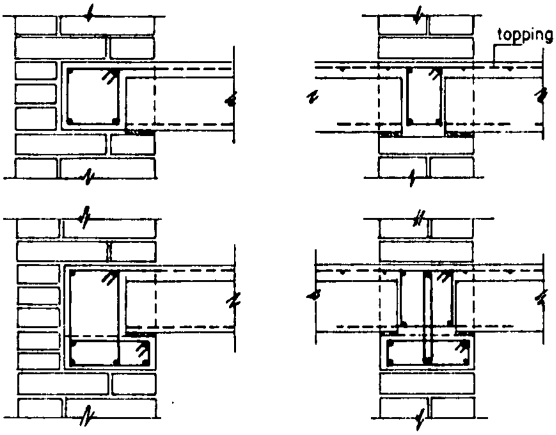

Fig.10.7 - Tie-beams in the case of prefabricated slabs with reinforced concrete topping

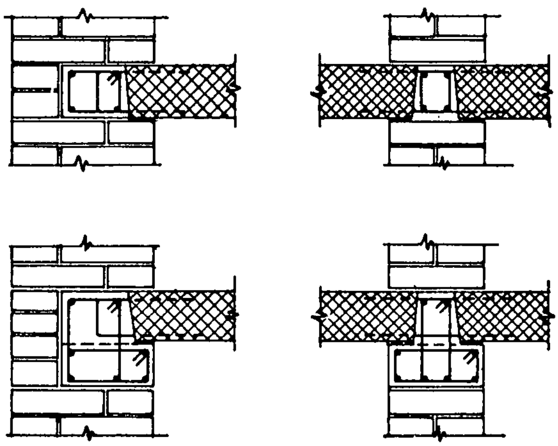

Fig.10.8 - Tie-beams in the case of precast large panel slabs

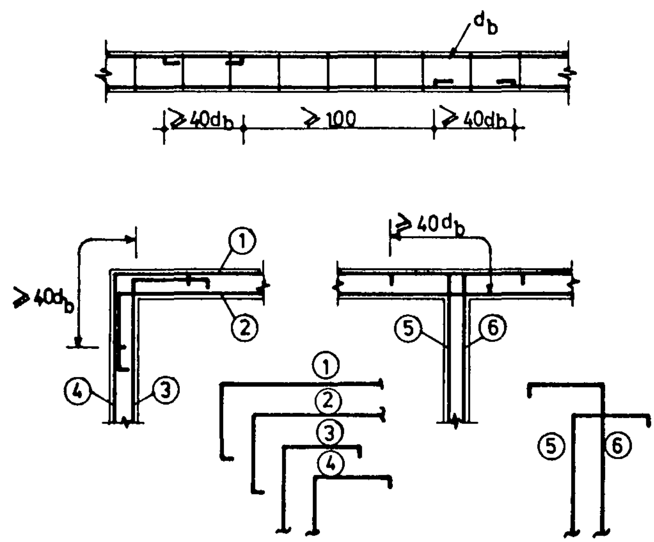

Fig.10.9 - Reinforced concrete tie-beams:
typical reinforcing detail

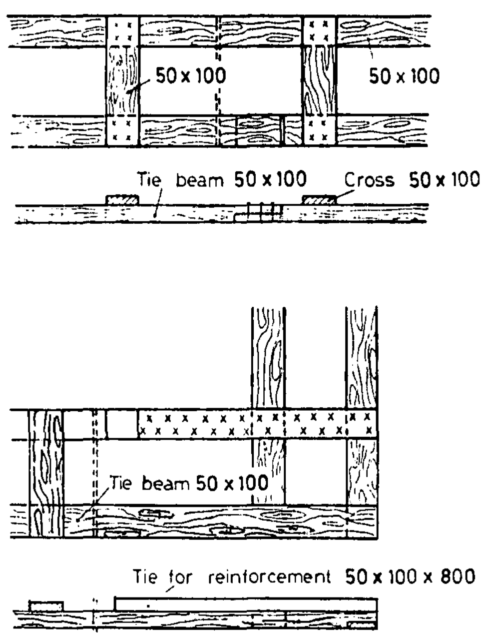

Fig.10.10 - Wooden tie-beam - details

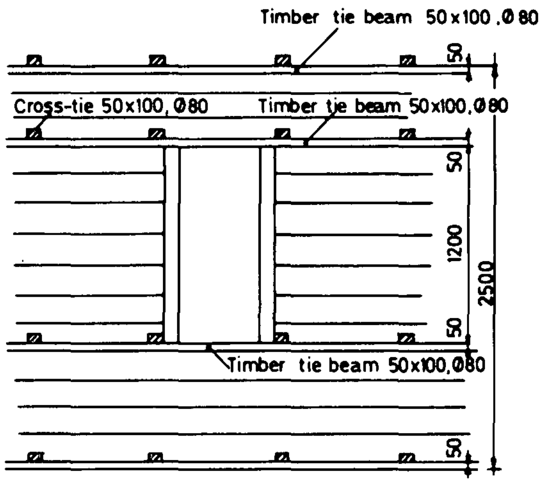

Fig. 10.11 - Location of wooden tie-beams in adobe buildings

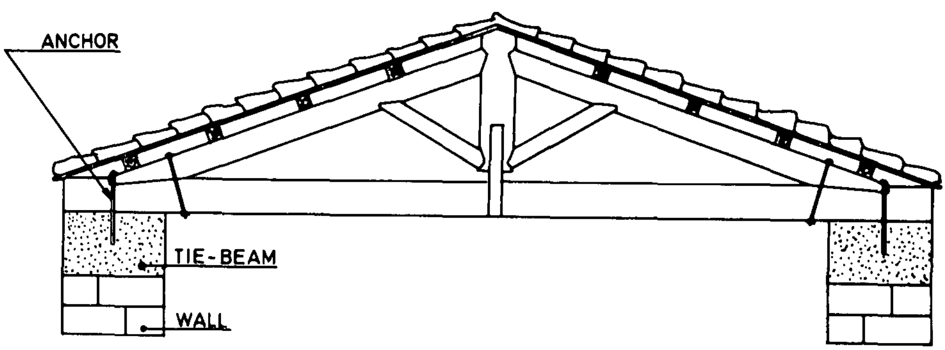

Fig. 10.12 - Typical timber roof structure

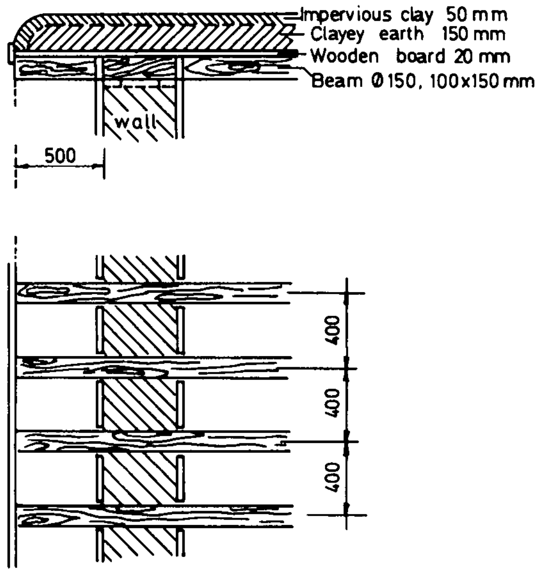

Fig. 10.13 - Typical roof for adobe buildings

11. NON-STRUCTURAL ELEMENTS

11.1 General

Failures or fall-downs of non-structural elements,such as ornamentations,chimneys, partition walls and the like, although being classified as "nonstructural damage", were the reason of many casualties and structural damage.

In this respect it is obvious that attention should be paid to adequate structural detailing and design of non-structural elements, too, in order to prevent:

- failure or fall-down of elements which could result in personal injury and damage to structural or non-structural elements;

- obstruction of passages used for emergency exit.

11.2 Partition Walls

Partition walls are usually made of bricks, hollow ceramic or light-weight concrete elements, and their thickness is less than 200 mm (usually about 100 mm).

Depending on their dimensions and the seismic zone, partition walls may be either unreinforced, or reinforced to prevent their out of plane instability. In the latter case, the reinforcing bars of 4-6 mm diameter are usually placed in the horizontal masonry joints at 400-600 mm intervals (Fig.11.1). Reinforced partition walls are recommended in seismic zone M and H.
Their out of plane stability, however, should be verified by calculation.

Partition walls are fixed between two consecutive floor slabs by means of cement mortar joints, whereas their vertical connections to the structural masonry walls or reinforced concrete tie-columns can be achieved either by bond or by anchors.

11.3 Gable Walls and Attics

Masonry gable end walls and attics higher than 0.50 m should be anchored to the uppermost floor tie-beams by means of reinforced concrete tie-columns. The columns should be located at maximum 3 m intervals for zones of high (H) and moderate (M) seismicity. For zones of low (L) seismicity, the distance can be increased, but should not exceed 6 m. Reinforced concrete tie-beams, which connect the tie-columns should be provided on the top of the wall.In case the height of gable end walls and attics exceeds 2.0 m (Fig.11.2), intermediate tie-beams should be added at intervals not exceeding 1.0 m.

11.4 Chimneys and Ventilation Stacks

Chimney or ventilation flues should be located so that the wall thickness is not reduced. In the case of masonry walls made of blocks, separate chimney flues should be constructed by using either brick units or precast concrete elements (Fig.11.3 and 11.4).

In seismic zones M and H, free standing chimneys and ventilation stacks should be constructed using cement mortar. Adequate anchoring into the top floor and reinforcement above the top floor level should be provided. For this purpose, a reinforced mortar jacket, as shown in Fig.11.5, is recommended.

The stability of the free standing chimneys and ventilation stacks should be verified by calculation. Additionally, special care should be paid to the fire protection.

11.5 Ornamentations

Ornamentations such as cornices, vertical or horizontal cantilever projections, facia stones, and the like must be reinforced with steel and properly anchored into the main structure of the building.

The adequacy of the anchoring must be verified by calculation.

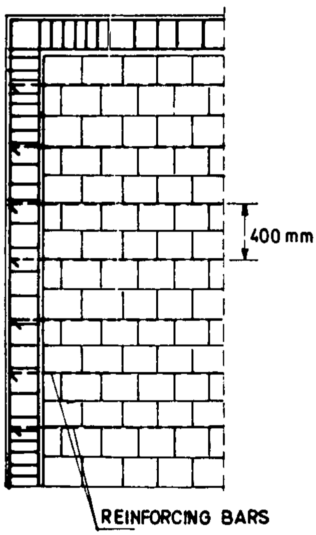

Fig. 11.1 - Reinforcing and anchoring of partition walls

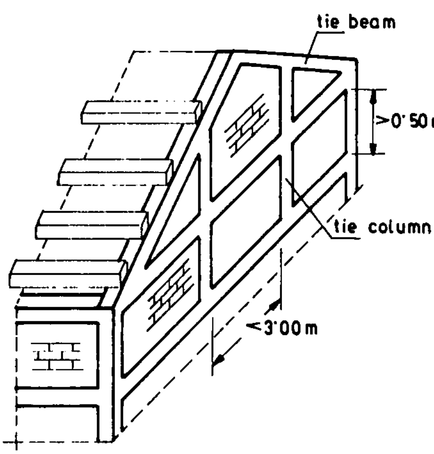

Fig. 11.2 - Anchoring of gable end walls and attics

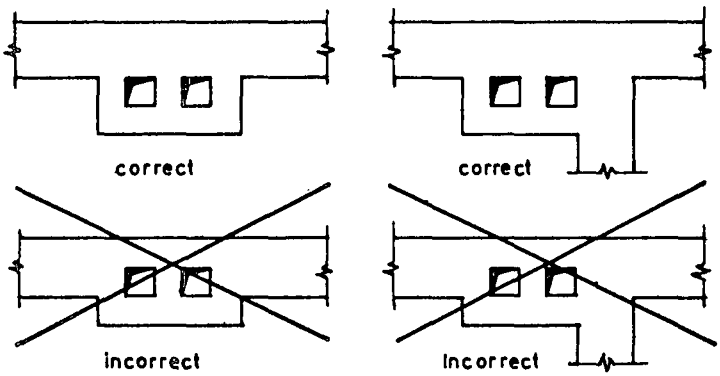

Fig.11.3 - Location of chimney flues in the case of brick masonry walls

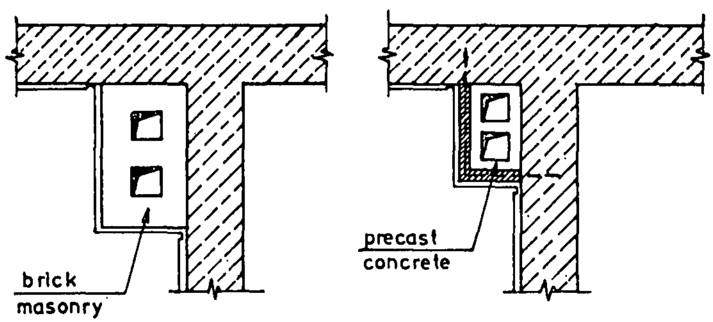

Fig.11.4 - Location of chimney flues in the case of block units masonry walls

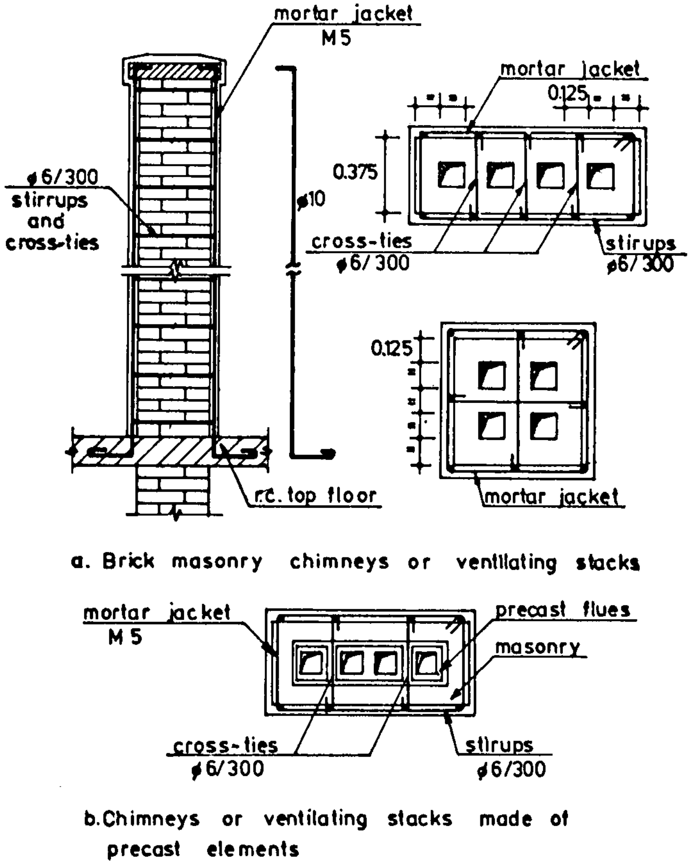

Fig.11.5 - Reinforcing details for free standing chimneys and ventilation stacks

12. REFERENCES

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13. DESIGN EXAMPLES

13.1 PLAIN MASONRY BUILDING
13.2 CONFINED MASONRY BUILDING
13.3 REINFORCED MASONRY BUILDING

13.1 REPRESENTATIVE DESIGN EXAMPLE - YUGOSLAVIA

Plain Masonry Building

by

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PLAIN MASONRY BUILDING

The example represents a three-storeyed plain masonry residential building, designed and constructed according to Yugoslav practice in low intensity seismic zone.

Dimensions of building

- Dimensions of building in plan: 15.97 m x 20.79 m;
- building height measured from ground level to the top of the second floor slab: 9.40 m;
- Story height: 2.80 m;

Structural description

- Walls: 290 mm or 190 mm thick, made of light-weight concrete blocks (block grade B 7.5, mortar grade M 2.5);
- Characteristic compressive strength of walls: $f_{wc,k} = 2.40$ Mpa;
- Characteristic tensile strength of walls: $f_{wt,k} = 0.16$ MPa;
- Design compressive strength of walls: $f_{wc,d} = 1.50$ MPa;
- Design tensile strength of walls: $f_{wt,d} = 0.10$ MPa;
- Foundations and basement walls: plain concrete C 15;
- Floor slabs: prefabricated ceramic elements with reinforced concrete topping, 40 mm thick. Reinforcing mesh of 5 mm diameter bars at 200 mm intervals in both orthogonal directions (yield stress 400 MPa) and concrete C 20 are used.
- Lintels and tie-beams: reinforcing steel with yield stress 240 MPa and concrete C 20 are used.
- Roof: wooden structure with asbestos tiles covering.

Loads

- Vertical loads:

Floor slabs:		$5.60\ kN/m^2$
Roof:		$0.80\ kN/m^2$
Live load:	roof (snow):	$1.25\ kN/m^2$
	floors:	$1.25/2 = 0.63\ kN/m^2$
	staircase:	$3.00/2 = 1.50\ kN/m^2$
	balconies:	$3.00/2 = 1.50\ kN/m^2$
Walls own weight:		$16.0\ kN/m^2$

- Seismic loads:
- Base shear coefficient:

The base shear coefficient K is determined according to Yugoslav Seismic Code from 1981:

$$K = K_o \cdot K_s \cdot K_d \cdot K_p = 1.0 \times 0.025 \times 1.0 \times 2.0 = 0.05,$$

where:

K_o = - building category coefficient (importance factor),

K_s = 0.025 - seismic intensity coefficient (degree of seismic intensity VII. according to MCS seismic intensity scale)

K_d = 1.0 - dynamic coefficient;

K_p = 2.0 - ductility and damping coefficient plain masonry).

- Ultimate base shear coefficient:

  Limit states are considered, so a global safety factor V = 1.5 (masonry structures) is taken into account, which results into an ultimate base shear coefficient:

$$K_u = VK = 1.5 \times 0.05 = 0.08.$$

Earthquake resistance analysis

- Non-linear method has been used for earthquake resistance analysis, assuming rigid horizontal diaphragm action and pier action of structural walls (fixed-ended walls).

- Vertical stresses in walls are calculated, assuming trapezoidal or triangular distribution of floor loads onto individual walls.

- Using the computer program SREMB (Seismic Resistance of Masonry Building), the earthquake resistance of the building in both orthogonal directions is calculated, following the three basic formulae:

- Flexural resistance of wall:

$$H_{u,f} = \frac{\sigma_o \, t \, l^2}{h} \left(1 - \frac{\sigma_o}{f_{wc,d}}\right) \qquad \ldots(1)$$

- Shear resistance of wall:

$$H_{u,s} = A \frac{f_{wt,d}}{\xi} \sqrt{\frac{\sigma_o}{f_{wt,d}} + 1} \qquad \ldots(2)$$

- Wall stiffness:

$$K = \frac{G}{1.2\,h} \frac{A}{1 + \frac{G}{E} \frac{1}{1.2} \left(\frac{h}{l}\right)^2}, \qquad \ldots(3)$$

where:

σ_o - average vertical compressive stress in wall due to vertical load,

$f_{wc,d}$ - design compressive strength of wall,

$f_{wt,d}$ - design tensile strength of wall,

G - shear modulus,

E - elastic modulus,

ξ - shear stresses distribution coefficient,

A - horizontal cross-sectional area of wall,

t - thickness of wall,

l - length of wall,

h - height of wall.

- Calculation procedure is a step-by-step procedure. First, the strength and deformability characteristics (idealized H-δ diagrams) of individual walls in the story under consideration are calculated. On the basis of these characteristics, the position of the story stiffness-centre is determined, and the possible rotation of the story due to torsional effects is calculated. Knowing the rotation, the displacements of the individual walls and the story mass-centre, as well as the corresponding story shear at the defined elastic limit can be evaluated.

In the following, the translational part of the relative story displacement is increased for a small step. On the basis of the increased value of the translational part of the relative story displacement, as well as on the basis of the rotation angle, calculated in the previous step of the calculation, new displacement of individual walls are obtained, following their idealized H-δ diagrams. The new position of the story stiffness-centre is calculated, and the rotation angle is corrected, if necessary.

The calculation is repeated and is terminated after the maximum value of the story shear has been obtained.

- Results of calculation are given in the Appendix. The following values of the calculated values of the ultimate base shear coefficients were obtained.

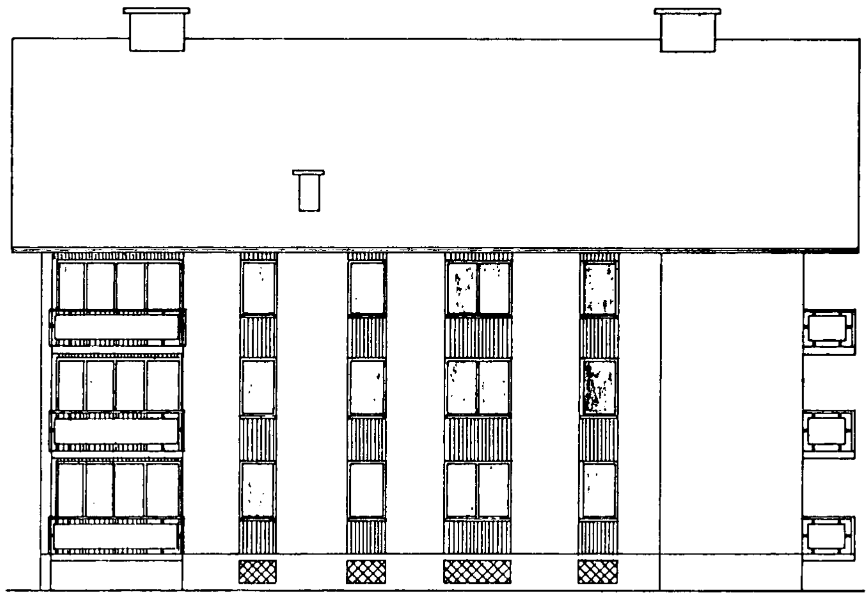
- in x - direction

$$VK_{u,col,x} = 0.254,$$

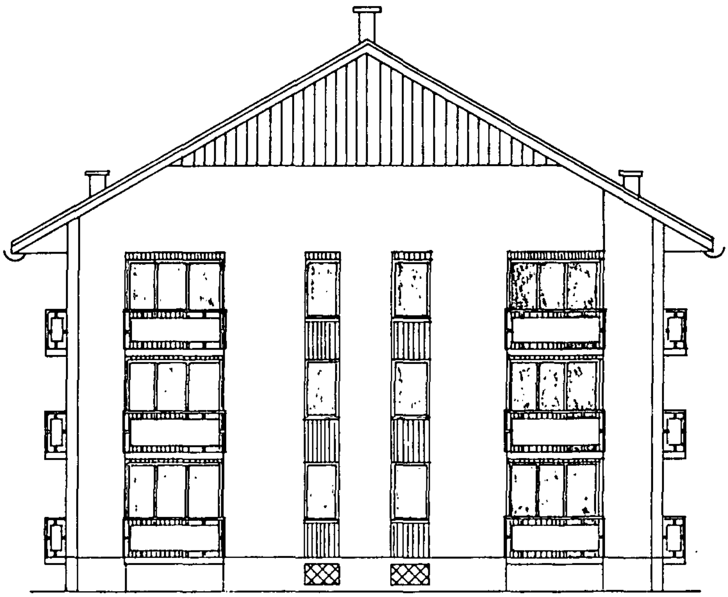
- in y - direction

$$VK_{u,col,y} = 0.229,$$

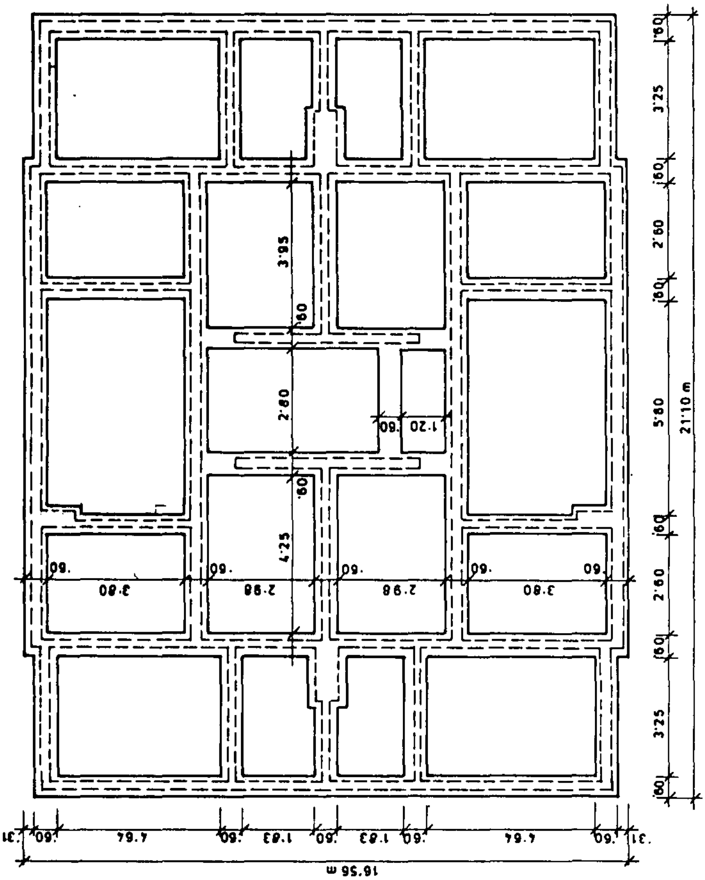
which satisfy the Code requirements.



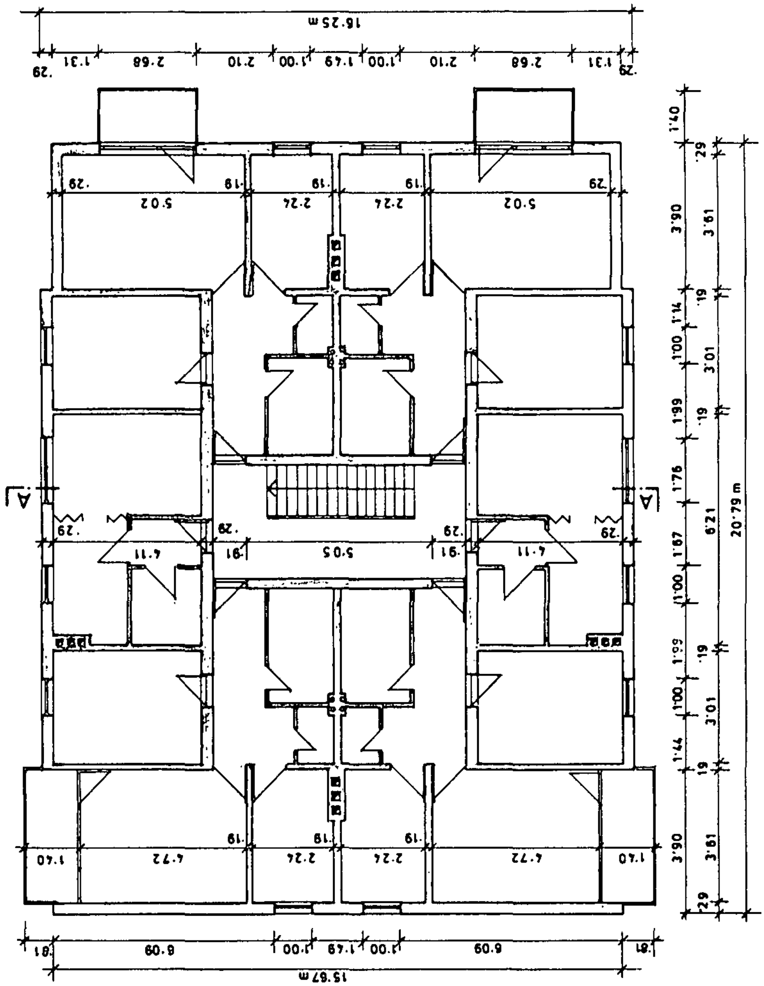
DESIGN EXAMPLE 1 - PLAIN MASONRY BUILDING
STREET FACE VIEW



DESIGN EXAMPLE 1 - PLAIN MASONRY BUILDING
SIDE FACE VIEW

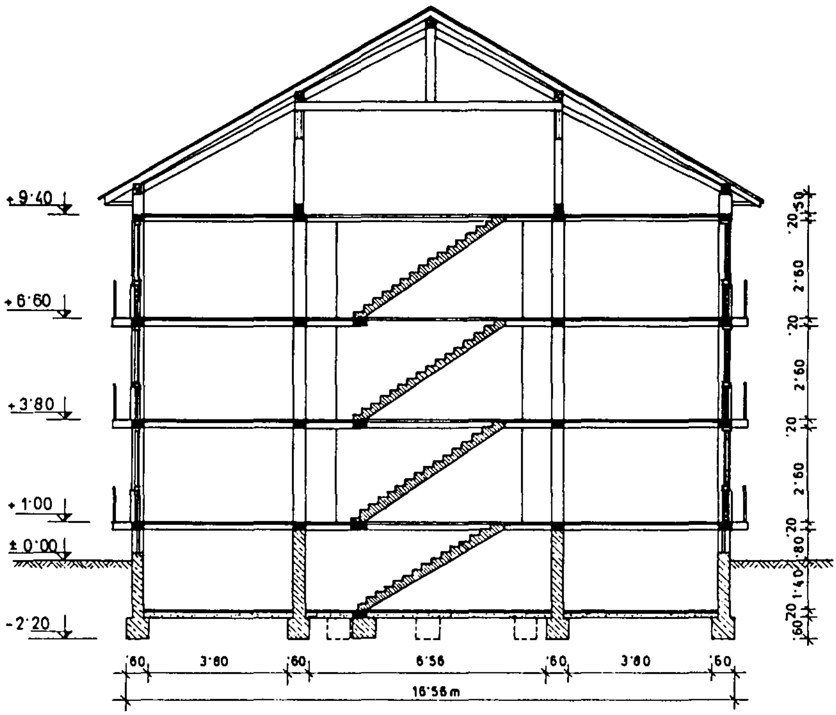


DESIGN EXAMPLE 1 - PLAIN MASONRY BUILDING
FOUNDATION PLAN

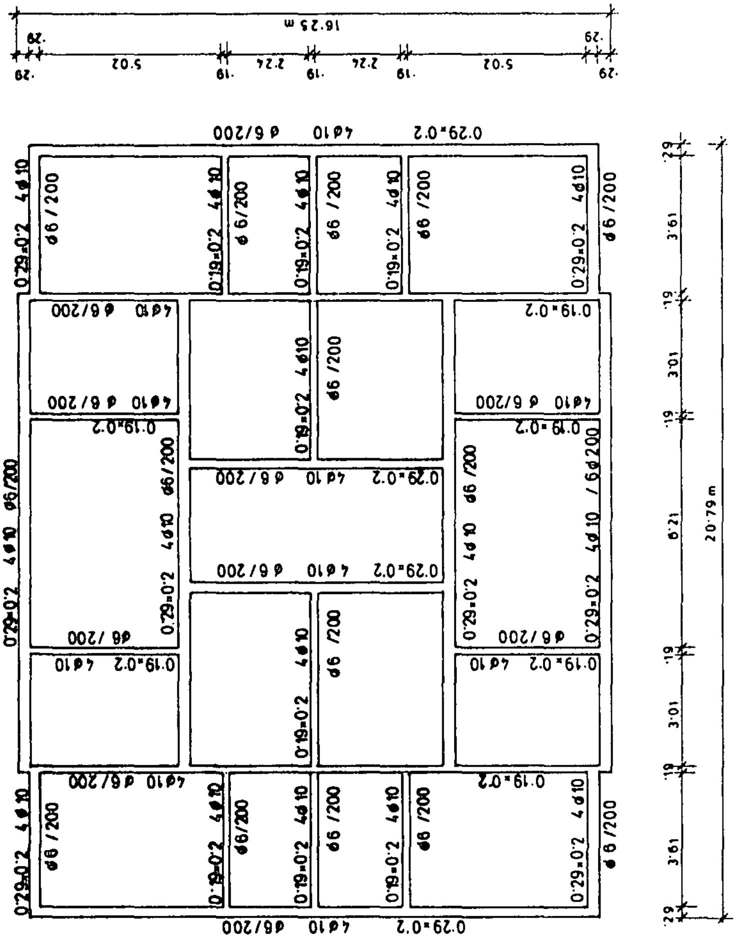


DESIGN EXAMPLE 1 - PLAIN MASONRY BUILDING

TYPICAL STORY PLAN

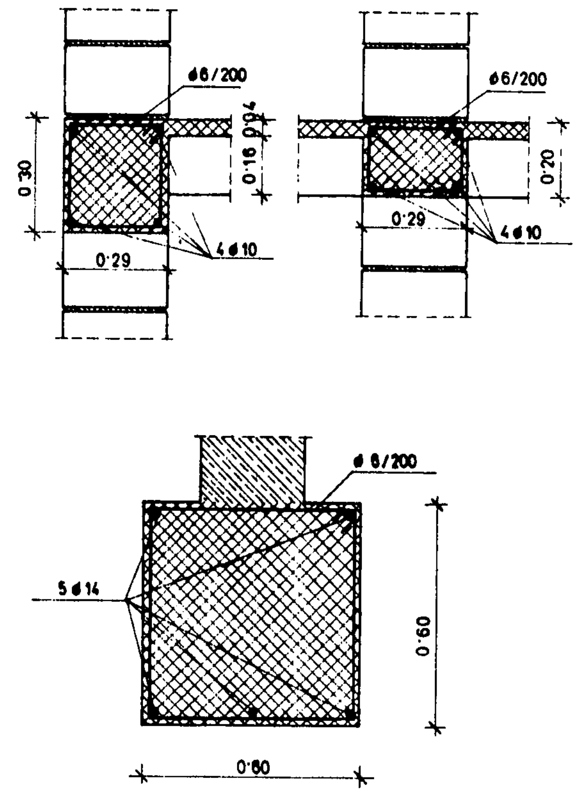


DESIGN EXAMPLE 1 - PLAIN MASONRY BUILDING
TRANSVERSAL SECTION A - A



DESIGN EXAMPLE 1 - PLAIN MASONRY BUILDING

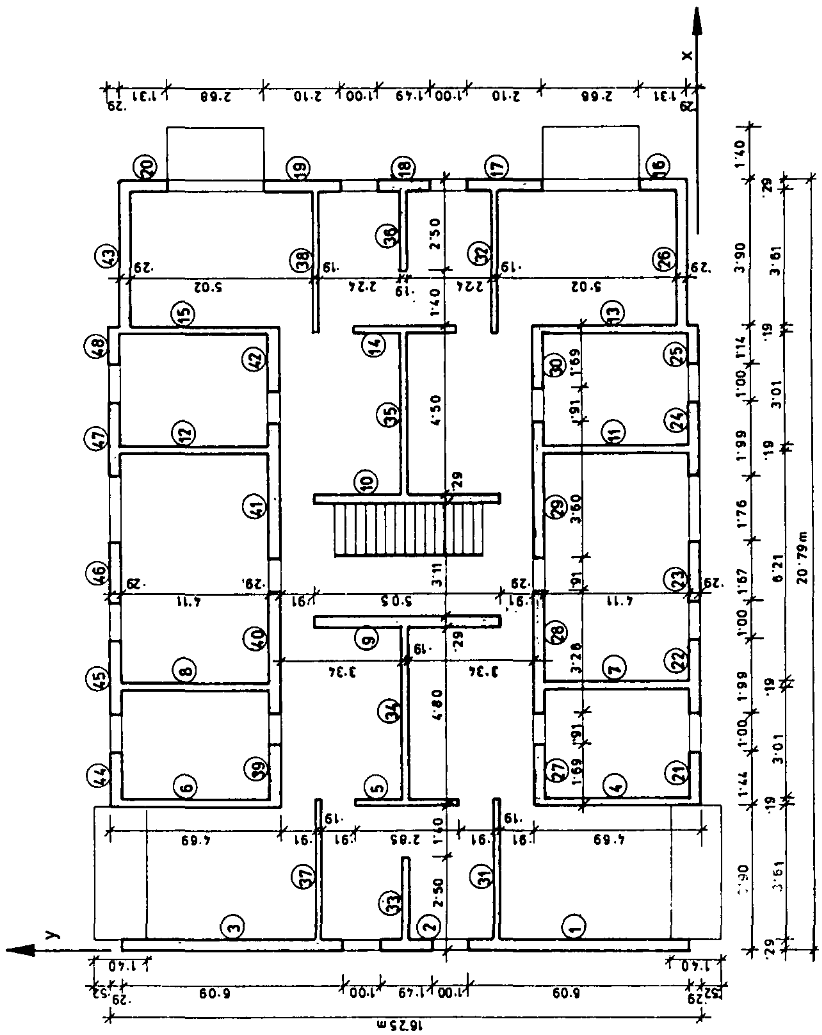
TIE - BEAMS AND LINTELS



DESIGN EXAMPLE 1 - PLAIN MASONRY BUILDING
REINFORCING DETAILS

APPENDIX

SEISMIC RESISTANCE ANALYSIS



DESIGN EXAMPLE 1 - PLAIN MASONRY BUILDING
COMPUTATION SCHEME

LIST OF SYMBOLS

INPUT VALUES

PARAMETERS OF STRUCTURAL LAYOUT

DX , DY	(M)	DIMENSIONS OF WALL IN X AND Y DIRECTION,RESPECTIVELY
X , Y	(M)	COORDINATES OF CROSS-SECTIONAL CENTER OF WALL
V	(M)	HEIGHT OF WALL
SIGMAO	(MPA)	AVERAGE VERTICAL COMPRESSIVE STRESS DUE TO VERICAL LOAD

MATERIAL CHARACTERISTICS OF WALLS

FC	(MPA)	CHARACTERISTIC COMPRESSIVE STRENGHT
FT	(MPA)	CHARACTERISTIC TENSILE STRENGHT
E	(MPA)	ELASTIC MODULUS
G	(MPA)	SHEAR MODULUS
DU		ULTIMATE DUCTILITY FACTOR

CALCULATED VALUES

PARAMETERS OF STORY RESISTANCE

HE,HC,HU	(MN)	STORY SHEAR AT THE ELASTIC,CRACK AND ULTIMATE LIMIT STATE,RESPECTIVELY
KE,KC,KU	(MN/M)	STORY STIFNESS AT THE ELASTIC,CRACK AND ULTIMATE LIMIT STATE,RESPECTIVELY
DE,DC,DU	(MM)	DISPLACEMENT OF THE STORY MASS CENTER AT THE ELASTIC,CRACK AND ULTIMATE LIMIT STATE,RESPECTIVELY
CE,CC,CU	(MN)	BASE SHEAR COEFFICIENT AT THE ELASTIC, CRACK AND ULTIMATE LIMIT STATE, RESPECTIVELY

PARAMETERS OF STORY RESISTANCE

H(I)	(MN)	STORY SHEAR AT THE I-TH STEP OF CALCULATION
K(I)	(MN/M)	STORY STIFNESS AT THE I-TH STEP OF CALCULATION
D(I)	(MM)	DISPLACEMENT OF THE STORY MASS CENTER AT THE I-TH STEP OF CALCULATION

STRENGHT AND DEFORMABILITY CHARACTERISTICS OF WALLS

HUF	(MN)	FLEXURAL CAPACITY
HUS	(MN)	SHEAR CAPACITY
KO	(MN/M)	ELASTIC STIFFNESS
DO	(MM)	DEFORMATION AT THE IDEALIZED ELASTIC LIMIT
H	(MN)	WALL SHEAR
K	(MN/M)	STIFFNESS
D	(MM)	DEFORMATION

PARAMETERS OF STRUCTURAL LAYOUT

WALL NO.	DX (M)	DY (M)	V (M)	X (M)	Y (M)	SIGMAU (MPA)
1	0.29	6.09	2.60	0.15	3.34	0.291
2	0.29	1.49	2.60	0.15	8.13	0.395
3	0.29	6.09	2.60	0.15	12.92	0.291
4	0.19	4.69	2.60	4.00	2.35	0.280
5	0.19	2.85	2.60	4.00	8.13	0.287
6	0.19	4.69	2.60	4.00	13.91	0.280
7	0.19	4.69	2.60	7.20	2.35	0.125
8	0.19	4.69	2.60	7.20	13.91	0.125
9	0.29	5.05	2.60	8.85	8.13	0.244
10	0.29	5.05	2.60	12.25	8.13	0.179
11	0.19	4.69	2.60	13.60	2.35	0.125
12	0.19	4.69	2.60	13.60	13.91	0.125
13	0.19	4.69	2.60	16.80	2.35	0.251
14	0.19	2.85	2.60	16.80	8.13	0.287
15	0.19	4.69	2.60	16.80	13.91	0.251
16	0.29	1.31	2.60	20.65	0.95	0.349
17	0.29	2.10	2.60	20.65	5.33	0.459
18	0.29	1.49	2.60	20.65	8.13	0.395
19	0.29	2.10	2.60	20.65	10.92	0.459
20	0.29	1.31	2.60	20.65	15.31	0.349
21	1.44	0.29	2.60	4.62	0.15	0.335
22	1.99	0.29	2.60	7.33	0.15	0.319
23	1.67	0.29	2.60	10.17	0.15	0.381
24	1.99	0.29	2.60	13.76	0.15	0.333
25	1.14	0.29	2.60	16.32	0.15	0.312
26	4.09	0.29	2.60	18.74	0.44	0.166
27	1.69	0.29	2.60	4.75	4.55	0.455
28	3.28	0.29	2.60	8.14	4.55	0.394
29	3.60	0.29	2.60	12.49	4.55	0.394
30	1.69	0.29	2.60	16.05	4.55	0.455
31	4.09	0.19	2.60	2.05	5.70	0.157
32	4.09	0.19	2.60	18.75	5.70	0.157
33	2.50	0.19	2.60	1.25	8.13	0.124
34	5.09	0.19	2.60	6.50	8.13	0.343
35	4.79	0.19	2.60	14.44	8.13	0.344
36	2.50	0.19	2.60	19.54	8.13	0.124
37	4.09	0.19	2.60	2.05	10.55	0.157
38	4.09	0.19	2.60	18.75	10.55	0.157
39	1.69	0.29	2.60	4.75	11.70	0.455
40	3.28	0.29	2.60	8.14	11.70	0.394
41	3.60	0.29	2.60	12.49	11.70	0.394
42	1.69	0.29	2.60	16.05	11.70	0.455
43	4.09	0.29	2.60	18.74	15.82	0.166
44	1.44	0.29	2.60	4.62	16.10	0.335
45	1.99	0.29	2.60	7.33	16.10	0.319
46	1.37	0.29	2.60	10.02	16.10	0.381
47	2.29	0.29	2.60	13.61	16.10	0.333
48	1.14	0.29	2.60	16.32	16.10	0.312

MATERIAL CHARACTERISTICS OF WALLS

WALL NO.	FC (MPA)	FT (MPA)	E (MPA)	G (MPA)	DU
1	1.50	0.100	4000.	300.	1.5
2	1.50	0.100	4000.	300.	1.5
3	1.50	0.100	4000.	300.	1.5
4	1.50	0.100	4000.	300.	1.5
5	1.50	0.100	4000.	300.	1.5
6	1.50	0.100	4000.	300.	1.5
7	1.50	0.100	4000.	300.	1.5
8	1.50	0.100	4000.	300.	1.5
9	1.50	0.100	4000.	300.	1.5
10	1.50	0.100	4000.	300.	1.5
11	1.50	0.100	4000.	300.	1.5
12	1.50	0.100	4000.	300.	1.5
13	1.50	0.100	4000.	300.	1.5
14	1.50	0.100	4000.	300.	1.5
15	1.50	0.100	4000.	300.	1.5
16	1.50	0.100	4000.	300.	1.5
17	1.50	0.100	4000.	300.	1.5
18	1.50	0.100	4000.	300.	1.5
19	1.50	0.100	4000.	300.	1.5
20	1.50	0.100	4000.	300.	1.5
21	1.50	0.100	4000.	300.	1.5
22	1.50	0.100	4000.	300.	1.5
23	1.50	0.100	4000.	300.	1.5
24	1.50	0.100	4000.	300.	1.5
25	1.50	0.100	4000.	300.	1.5
26	1.50	0.100	4000.	300.	1.5
27	1.50	0.100	4000.	300.	1.5
28	1.50	0.100	4000.	300.	1.5
29	1.50	0.100	4000.	300.	1.5
30	1.50	0.100	4000.	300.	1.5
31	1.50	0.100	4000.	300.	1.5
32	1.50	0.100	4000.	300.	1.5
33	1.50	0.100	4000.	300.	1.5
34	1.50	0.100	4000.	300.	1.5
35	1.50	0.100	4000.	300.	1.5
36	1.50	0.100	4000.	300.	1.5
37	1.50	0.100	4000.	300.	1.5
38	1.50	0.100	4000.	300.	1.5
39	1.50	0.100	4000.	300.	1.5
40	1.50	0.100	4000.	300.	1.5
41	1.50	0.100	4000.	300.	1.5
42	1.50	0.100	4000.	300.	1.5
43	1.50	0.100	4000.	300.	1.5
44	1.50	0.100	4000.	300.	1.5
45	1.50	0.100	4000.	300.	1.5
46	1.50	0.100	4000.	300.	1.5
47	1.50	0.100	4000.	300.	1.5
48	1.50	0.100	4000.	300.	1.5

```
STORY CHARACTERISTICS
---------------------

COORDINATES
OF THE STORY
MASS-CENTER       :      XM =  10.45 M    YM =    8.12 M

COORDINATES
OF THE STORY
STIFNESS-CENTER :      XK =   9.65 M    YK =    8.13 M

ECCENTRICITI
OF THE STORY
STIFNESS-CENTER :      EX =  -0.80 M    EY =    0.00 M

WEIGHT OF THE BUILDING
ABOVE THE CRITICAL LEVEL :      WTOT =   9.80 MN
```

```
**********************************
 STORY RESISTANCE IN X-DIRECTION
**********************************
```

STORY CHARACTERISTICS AT THE ELASTIC LIMIT STATE

X-DIRECTION

STORY SHEAR	HE =	1.912	MN
STIFNESS	KE =	1918.128	MN/M
DISPLACEMENT	DE =	0.997	MM
BASE SHEAR COEFFICIENT	VKE =	0.195	

STORY CHARACTERISTICS AT THE CRACK LIMIT STATE

X-DIRECTION

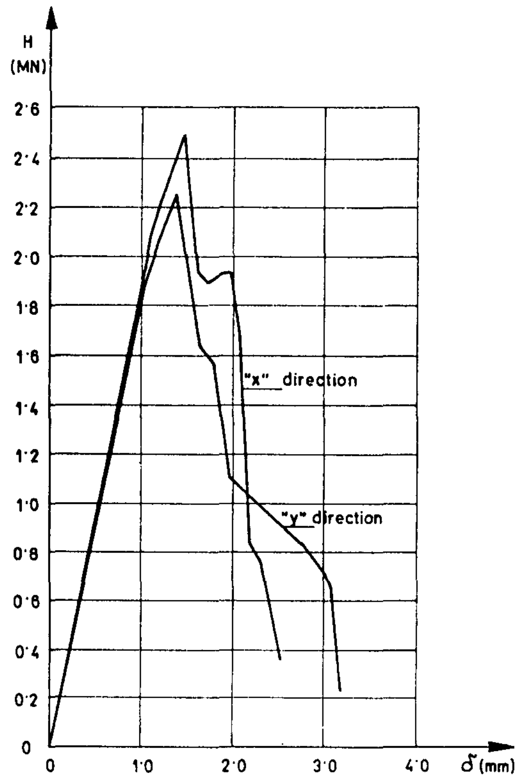
STORY SHEAR	HC =	2.209	MN
STIFNESS	KC =	1822.728	MN/M
DISPLACEMENT	DC =	1.212	MM
BASE SHEAR COEFFICIENT	VKC =	0.225	

STORY CHARACTERISTICS AT THE ULTIMATE LIMIT STATE

X-DIRECTION

STORY SHEAR	HU =	2.490	MN
STIFFNESS	KU =	1689.995	MN/M
DISPLACEMENT	DU =	1.473	MM
BASE SHEAR COEFFICIENT	VKU =	0.254	

```
 **********************************
  STORY RESISTANCE IN Y-DIRECTION
 **********************************


STORY CHARACTERISTICS  AT THE ELASTIC LIMIT STATE
--------------------------------------------------
                Y-DIRECTION


  STORY SHEAR               HE =       1.715 MN
  STIFNESS                  KE =    1873.670 MN/M
  DISPLACEMENT              DE =       0.923 MM
  BASE SHEAR COEFFICIENT VKE =       0.175

  STORY CHARACTERISTICS  AT THE CRACK LIMIT STATE
  -----------------------------------------------
                Y-DIRECTION


   STORY SHEAR               HC =       2.007 MN
   STIFNESS                  KC =    1804.905 MN/M
   DISPLACEMENT              DC =       1.123 MM
   BASE SHEAR COEFFICIENT VKC =       0.205

  STORY CHARACTERISTICS  AT THE ULTIMATE LIMIT STATE
  --------------------------------------------------
                Y-DIRECTION


     STORY SHEAR               HU =       2.248 MN
     STIFNESS                  KU =    1663.569 MN/M
     DISPLACEMENT              DU =       1.372 MM
     BASE SHEAR COEFFICIENT VKU =       0.229
```



DESIGN EXAMPLE 1 - PLAIN MASONRY BUILDING
CALCULATED STORY H - δ DIAGRAM

13.2. REPRESENTATIVE DESIGN EXAMPLE - RUMANIA

Confined Masonry Building

by

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and

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1. INTRODUCTION

This design example refers to a 4-story building whose structural system is made of confined masonry walls. As it has been shown, the confined masonry is defined as a construction system, where reinforced concrete tie-columns properly located and connected with reinforced concrete tie-beams confine plain brick or block masonry structural walls, thus forming framed masonry panels.

Confined masonry is currently used nowadays in Romania. The latest strong earthquake (March 4, 1977) showed that this construction system - properly designed and built - possessed a higher stiffness, strength and ductility than the plain masonry structures and, consequently, behaved far better than the latter ones.

The design example has been prepared according to the present Romanian technical regulations concerning aseismic design of buildings [1] , [2].

The building layout and details, as well as the structural analysis and computation procedures, are representative for the present design practice of masonry buildings in Romania.

The computations presented have been limited to a single masonry wall and only some typical drawings (architectural and structural) have been chosen to illustrate this design example.

2. NOTATION

a	length of the equivalent compression stress zone
A_c	area of the tie-column cross-section
A_{eqv}	equivalent area of the confined masonry cross-section
A_{sc}	area of the tie-column reinforcement, located within the compression zone of the cross-section
A_{st}	area of the tie-column reinforcement located within the tensile zone of the cross-section
$A_{s,tb}$	area of the tie-beam reinforcement
A_w	area of the masonry wall cross-section
$B';B''$	distance (bay) between successive parallel walls
d	distance from the tensile reinforcement up to the compressed edge of the cross-section
d_b	diameter of the reinforcement bars

e_u — eccentricity of the axial force in relation to the centroid of the cross-section equivalent area, corresponding to the limit state of strength

$f_{c,d}$ — design compression strength of the concrete

$f_{s,d}$ — design strength of the reinforcement

$f_{wc,d}$ — design compression strength of the masonry

$f_{wt,d}$ — design strength of the masonry in diagonal tension

G — permanent (dead) load

h_s — story height

h_w — height of the masonry panel between two succesive floor tie-beams

H — total height of the structure

I_{eqv} — equivalent moment of inertia of the confined masonry cross-section

k_s — coefficient of seismic intensity

ℓ — length of the wall cross-section

ℓ_w — length of the masonry panel between two successive tie-columns

M_d — design (required) bending moment due to gravity and seismic loads, determined by structural elastic analysis

M_u — capable bending moment of the wall subject to eccentric compression in its plane, corresponding to the limit state of strength

n — total number of stories

N — design axial force

Q — temporary (live) load

S_i — horizontal seismic load applied at the i-th story

t — thickness of the cross-section web

t_f — thickness of the cross-section flange

V — resultant of the horizontal seismic loads acting on the building

V — design (required) shear force due to the seismic loads, determined by structural elastic analysis

$V_{c,u}$ — capable shear force of the tie-column

$V_{w,cr}$ capable shear force corresponding to the limit state of cracking due to diagonal tension

$V_{w,dt}$ capable shear force corresponding to the failure due to diagonal tension

$V_{w,M}$ capable shear force related to the capable bending moment

$V_{w,s}$ capable shear force corresponding to the failure due to shearing of horizontal joint

V_{wu} capable shear force of the wall, corresponding to the limit state of strength

w_f effective width of the cross-section flange

$w_f'; w_f''$ width of that part of the cross-section flange situated on either side of the web

W resultant of the gravity loads

W_i gravity load at the i-th story

y_{st} distance from the tensile reinforcement up to the centroid of the equivalent cross-section area

Z lever arm of the horizontal seismic loads acting on that part of the building which is above the story for which computation is made

β dynamic coefficient, depending on the natural period of vibration of the building and on the nature of the foundation soil

ε equivalence coefficient between the actual structural system and a s.d.f. dynamic model

Ψ reduction coefficient for the seismic loads, which takes into account damping, ductility and the strength reserves

ϕ coefficient considering lateral stability of the wall

λ equivalence coefficient, expressing the ratio between the rigidity of concrete and that of masonry

λ_c equivalence coefficient, expressing the ratio between the compression strength of concrete and that of masonry

σ_o average compression stress

$\sigma_{w,dt}$ diagonal tensile stress

τ_o average shearing stress

3. DESCRIPTION OF THE BUILDING

3.1. Architecture

Destination: youth hostel.
Number of stories: 4 + partial basement (for heating and sanitary installations).
Plan dimensions: 32.00 x 13.40 m.
Story height: 2.72 m.
Total height: 11.45 m.
Partition walls (non-structural): cellular concrete masonry.
Roof: terrace.

Representative drawings: Fig.E2.1 through E2.5

3.2. Structure

Structural system: load-bearing confined masonry.
Degree of aseismic protection: 8.
Soil conditions: good, moderate water content.

Structural walls:
- exterior: vertically perforated ceramic blocks (365 x 180 x x 138 mm), grade B 10;
- interior: vertically perforated ceramic blocks (240 x 290 x x 138 mm), grade B 10;
- mortar grade M 5.

Reinforced concrete tie-columns:
- location: at all corners, all wall crossings and sides of certain openings, all over the entire building height;
- size: 250 x 250 mm;
- concrete grade C 20;
- reinforcement: 4Ø14 or 4Ø12 at the first story, 4Ø12 or 4Ø10 at the upper stories (hot rolled deformed steel bars PC 52), stirrups Ø8/100 (or 200) and projecting bars into adjacent masonry joints Ø6/600 (hot rolled plain steel bars OB 37).

Floors:
- precast large panels: 130 mm thickness, concrete grade C 25 (reinforced with cold drawn wire mesh and hot rolled deformed steel bars PC 52);
- cast-in-place slab, reinforced concrete grade C 20, above the partial basement;
- tie-beams: reinforced concrete grade C 25, on all exterior and interior structural walls.

Foundations:
- continuous footings: plain concrete grade C 5;
- pedestals and basement walls: plain concrete grade C 10;
- tie-beams: reinforced concrete grade C 10 at the bottom of pedestals and basement walls and grade C 20 at their top.

Representative drawings: Fig. E2.6 through E2.10.

4. COMPUTATION MODEL AND SEQUENCES OF ANALYSIS

4.1. Computation Model

The spatial structural system, as well as its vertical components (i.e. masonry walls), are considered vertical cantilevers fixed at the basis and subjected to the action of vertical (gravity) and horizontal (seismic) loads.

The masonry piers between openings are connected by lintels fixed elastically in the masonry.

The floors are considered as rigid horizontal diaphragms, able to carry and distribute the horizontal loads to the masonry walls in relation to their relative stiffness.

When computing the stiffness of masonry walls, all deformations produced by bending moments, shear force and axial forces are taken into account.

The distribution of horizontal seismic loads to the masonry walls, as well as the computation of their design sectional forces (i.e. bending moments, shear forces and axial forces), are performed by applying the basic principles and methods of structural elastic analysis, using either simplified but reliable procedures or more sophisticated electronic computer programs.

4.2. Sequences of Analysis

A typical elastic analysis of masonry structures subjected to seismic loads includes the following main sequences [2],[3]:
- establishing of the configuration and dimensions of the component masonry walls, both in plan and elevation, taking into account their spatial connection with perpendicular walls, the presence of openings, horizontal connections (lintels and spandrel beams), confining elements such as tie-columns etc;
- computation of geometrical and stiffness characteristics of the walls, for each of the principal directions of the structure;
- computation of the gravity loads for each wall and of the resulting axial force;
- computation of the horizontal seismic loads for the entire building and their distribution to the masonry walls, for each of the principal directions;
- computation of the design sectional forces resulting from the action of seismic loads;
- preliminary checking of the wall cross-sections to comply with ductility requirements;
- computation of the sectional strength capacity of the walls;
- verification of the wall cross-sections, by comparing their strength capacity with the corresponding design (required) sectional force and, subsequently, the proportioning of the reinforcement for tie-columns and tie-beams.

5. COMPUTATION EXAMPLE

5.1. Design Strength Values for Materials.

5.1.1. Masonry

$f_{wc,d} = 1.50$ MPa

$f_{wt,d} = 0.13$ MPa

5.1.2. Concrete

$f_{c,d} = 7.00$ MPa

$\lambda = 7.5$

5.1.3. Reinforcement

$f_{s,d} = 290$ MPa for hot rolled deformed bars PC 52

$f_{s,d} = 210$ MPa for hot rolled plain bars OB 37

5.2. Computation schemes

On the basis of the architectural plans of the building, the component masonry structural walls are established separately on transversal and longitudinal directions, taking into account the interaction existing between the walls oriented perpendicularly one against the other.

For computation a schematic layout of building masonry walls is shown in Fig. E2.11 and E2.12.
The design computations presented in detail in the following sub-chapters are limited to the transverse wall W_{T3} only.

5.3. Geometrical Characteristics of Walls' Cross-Sections.

The interaction existing between the structural walls is taken into account when considering I, T and L - shaped cross-sections, whose effective flange width is given by:

$$w_f = t + w_f' + w_f'' \tag{E2.1}$$

provided that the following conditions are observed:

$$w_f' \leqslant \frac{B' - t}{2} \; ; \; w_f'' \leqslant \frac{B'' - t}{2}$$

$$w_f' \text{ and } w_f'' \leqslant \frac{\ell}{2} \tag{E2.2}$$

$$w_f' \text{ and } w_f'' \leqslant \text{distance up to the first opening}$$

$$w_f' \text{ and } w_f'' \leqslant \frac{H}{10}$$

Accordingly, in the case of wall W_{T3} the effective width of the flanges as well as other characteristic dimensions of its cross-section are shown in Fig.E2.13.

The following values are computed:

$A_w = 2.5750\ m^2$
$A_c = 0.1250\ m^2$

$A_{eqv} = A_w + \lambda . A_c = 2.5750 + 7.5 \times 0.1250 = 3.51\ m^2$

$I_{eqv} = 13.41\ m^4$

5.4. Gravity Loads and Axial Forces

The floors' gravity loads acting on the masonry walls are determined on the basis of tributary floor areas, in relation to their layout and their supporting conditions.

The values of gravity loads are computed by applying the loading coefficients corresponding to the special group of actions (which includes the seismic action), namely:

$$W = 1.0G + 0.4Q \qquad \text{(E2.3)}$$

5.4.1. Design Values of Gravity Loads

Gravity loads uniformly distributed on the floors:

- top floor (terrace) 8.0 KN/m^2
- intermediate floors, rooms 6.0 KN/m^2
- intermediate floors, corridor 5.6 KN/m^2

Weight of structural masonry walls (plaster included):

- exterior walls 6.5 KN/m^2 wall area
- interior walls 4.9 KN/m^2 wall area

Total gravity loads acting on the wall W_{T3}:

- top story 325 KN
- intermediate story 265 KN

5.4.2. Design Axial Forces

The values of the design axial forces N of the wall W_{T3} are represented in Fig.E2.14.

5.5. Seismic Loads and Design Sectional Forces

5.5.1. Horizontal Seismic Loads

In the case of common-type buildings, the resultant of horizontal seismic loads is determined for the fundamental mode of vibration only, by using the following formula [1]:

$$V = k_s . \beta . \psi . \varepsilon . W \qquad \text{(E2.4)}$$

According to the code provisions, the following values have been used for the present design example:

$k_s = 0.20$ (degree of aseismic protection 8)
$\beta = 2$
$\psi = 0.30$
$\varepsilon = 0.75$

The resultant of the gravity loads of the entire building is:

$W = 25000$ KN

Consequently:

$$V = 0.20 \times 2 \times 0.30 \times 0.75 \times 25000 = 0.09 \times 25000 = 2250 \text{ KN}$$

5.5.2. Distribution of Horizontal Seismic Loads

The distribution of horizontal seismic loads over the structural masonry walls, as well as over their height, has been made by using an electronic computer program (CASE) largely applied in Romanian design practice. The influence of overall torsion of the building has been also included in this analysis.

The seismic loads acting on the wall W_{T3} are shown in Fig.E2.14.

5.5.3. Design (Required) Sectional Forces

The values of the design sectional forces, namely bending moments, shear forces and axial forces (the latter, in the case of walls with openings), resulting from the action of horizontal seismic loads, have been determined by using the same electronic computer program.

According to common practice in current design, two simultaneous analyses were made, using two different values of the concrete modulus of elasticity, for modelling the degrading stiffness of r.c. lintels during strong earthquakes.
Maximum values of bending moments and shear forces in the walls have resulted when considering lintels with reduced stiffness.

The values of design (required) sectional forces computed for the wall W_{T3} are also represented in Fig.E2.14.

5.6. Preliminary Checking of Ductility Requirements

To get satisfactory behavior of confined masonry walls, as far as ductility is concerned, the following requirements are necessary to comply with [2],[3]:

The average compression stress should be limited - as indicated by Eq.E2.5 - so that, under eccentric compression in the wall plane, failure in compression of masonry and concrete should not occur prior to the yielding of the tensile reinforcement.

$$\sigma_o = \frac{N}{A_{eqv}} \leqslant 0.5 f_{wc,d} \qquad \text{(E2.5)}$$

The reinforcement of the tie-beams shall have the minimum area given by Eq.E2.6, in order to avoid diagonal tensile failure of the masonry occuring prior to the yielding of the tensile column reinforcement, due to the eccentric compression in the wall plane.

$$A_{s,tb} \geqslant 0.3 \frac{N}{f_{s,d}} \cdot \frac{h_s}{Z} \qquad \text{(E2.6)}$$

When considering wall W_{T3}, by applying Eq.E2.5, for the 1-st story the result is:

$$\sigma_o = \frac{1120}{3.51} \times 10^{-3} = 0.32 \text{ MPa} < 0.5 \times 1.50$$

The results obtained by applying Eq.E2.6 at each story are given in Table E2.1.

Table E2.1: Necessary Reinforcement for Tie-Beams

Story	N (KN)	$f_{s,d}$ MPa	h_s (m)	Z (m)	$0.3 \frac{N}{f_{s,d}} \cdot \frac{h_s}{Z}$ (mm^2)
1	1120	290	2.72	10.55	299
2	855			6.83	352
3	590			4.66	356
4	325			2.72	336

The reinforcement actually provided for tie-beams is:

- at the 1-st and 2-nd floor

4ø12 + 2ø10 (PC 52) $A_{s,tb} = 609 \text{ mm}^2$

- at the 3-rd and 4-th floor

6ø10 (PC 52) $A_{s,tb} = 471 \text{ mm}^2$

5.7. Sectional Strength Capacity

5.7.1. Flexural Capacity for Eccentric Compression in the Wall Plane

For I-shaped masonry walls with tie-columns at both extremities of the cross-section, the capable bending moment corresponding to the limit state of strength is computed according to the following formula (Fig.E2.15):

$$M_u = N \cdot e_u \tag{E2.7}$$

The length of the equivalent compression stress zone is determined by the formula:

$$a = \frac{N + A_{st} \cdot f_{s,d}}{0.85\phi \cdot t \cdot f_{wc,d}} - \frac{A_c \cdot (\lambda_c - 1)}{t} - t_f \cdot (\frac{w_f}{t} - 1) \tag{E2.8}$$

and, subsequently, the eccentricity of the axial force used in Eq.E2.7 is computed by the following formula:

$$e_u = \frac{0.85\phi \cdot f_{wc,d}}{N} \cdot \left[t \cdot a \cdot (d - \frac{a}{2}) + A_c \cdot (\lambda_c - 1) \cdot (d - \frac{t_f}{2}) + \right.$$

$$+ t_f.(w_f - t).(d - \frac{t_f}{2}) \Big] - y_{st} \qquad (E2.9)$$

where:

$$\lambda_c = \frac{f_{c,d}}{0.85\ f_{wc,d}} \qquad (E2.10)$$

If the lateral stability of a wall is ensured by the presence of a flange, the coefficient ϕ should be taken equal to 1.

Due to the fact that the cross-section of wall W_{T3} is asymmetrical, the computations have to be performed for both senses of the seismic loads.

In the case of seismic loads acting from left to right (see Fig. E2.15), for the 1-st story the result is:

$$A_{st} = 452\ mm^2\ (4\phi 12\ PC\ 52)$$

$$\lambda_c = \frac{7.0}{0.85 \times 1.50} = 5.5$$

$$a = \frac{1120 \times 10^3 + 452 \times 290}{0.85 \times 1 \times 250 \times 1.50} - \frac{250 \times 250 \times (5.5 - 1)}{250} -$$

$$- 250 \times (\frac{2200}{250} - 1) = 850\ mm$$

$$e_u = \frac{0.85 \times 1 \times 1.50}{1120 \times 10^3} \times \Big[250 \times 850 \times (5825 - \frac{850}{2}) +$$

$$+ 250 \times 250 \times (5.5 - 1) \times (5825 - \frac{250}{2}) + 250 \times$$

$$\times (2200 - 250) \times (5825 - \frac{250}{2}) \Big] - 2650 = 3645\ mm$$

$$M_u = 1120 \times 3.645 = 4080\ KNm$$

and for the upper stories:

2-nd story	M_u = 3370 KNm
3-rd story	M_u = 2560 KNm
4-th story	M_u = 1640 KNm

In the case of seismic loads acting from right to left, the results are:

1-st story	M_u = 3880 KNm
2-nd story	M_u = 3230 KNm
3-rd story	M_u = 2330 KNm
4-th story	M_u = 1540 KNm

5.7.2. Shear Capacity Related to Flexural Capacity

The shear force related to the capable bending moment corresponding to the limit state of strength is computed after the following formula:

$$V_{w,M} = \frac{M_u}{Z} = \frac{M_u}{M}.V \qquad (E2.11)$$

Using the values of the capable bending moment previously computed and the design (required) values of the sectional forces shown in Fig.E2.15, the results obtained for wall W_{T3} by applying Eq.E2.11 are those shown below.

- seismic loads acting from left to right:

1-st story	$V_{w,M}$ = 386.7 KN
2-nd story	$V_{w,M}$ = 493.3 KN
3-rd story	$V_{w,M}$ = 548.6 KN
4-th story	$V_{w,M}$ = 601.7 KN

- seismic loads acting from right to left:

1-st story	$V_{w,M}$ = 367.7 KN
2-nd story	$V_{w,M}$ = 472.8 KN
3-rd story	$V_{w,M}$ = 499.3 KN
4-th story	$V_{w,M}$ = 565.0 KN

5.7.3. Shear Capacity Corresponding to Failure Due to Diagonal Tension

In the case of masonry walls with tie-columns at both extremities of the cross-section, the capable shear force corresponding to failure due to diagonal tension is computed after the following formula:

$$V_{w,dt} = (0.8A_{s,tb}.f_{s,d} + 0.2\frac{N}{A_{eqv}}.t.h_s).\frac{\ell}{h_s} \qquad (E2.12)$$

Introducing the reinforcement area actually provided for the tie-beams (see 5.6), the result obtained for wall W_{T3}, applying Eq.2.12 for the 1-st story, is:

$$V_{w,dt} = (0.8 \times 609 \times 290 + 0.2 \times \frac{1120 \times 10^3}{3.51 \times 10^6} \times 250 \times 2720) \times$$

$$\times \frac{6075}{2720} = 412400 \text{ N} \qquad = 412.4 \text{ KN}$$

and for the upper stories:

2-nd story	$V_{w,dt}$ = 388.8 KN
3-rd story	$V_{w,dt}$ = 295.3 KN
4-th story	$V_{w,dt}$ = 272.2 KN

5.7.4. Shear Capacity Corresponding to Failure Due to Shearing of Horizontal Joint

In the case of masonry walls with tie-columns at both extremities of the cross-section, the capable shear force corresponding to failure due to shearing along a horizontal joint is computed after the following formula:

$$V_{w,s} = 0.4(N + A_{st} \cdot f_{s,d}) + 0.8A_{sc} \cdot f_{s,d} \qquad (E2.13)$$

Because, in the case of wall W_{T3}, the tie-columns do not have identical reinforcement areas, computations are necessary for both senses of the seismic loads.

In case of <u>seismic loads acting from left to right</u>, for the first story the result is:

$A_{st} = 452\ mm^2$ (4Ø12 PC 52)
$A_{sc} = 616\ mm^2$ (4Ø14 PC 52)

$$V_{w,s} = 0.4(1120 \times 10^3 + 452 \times 290) + 0.8 \times 616 \times 290 = 643300\ N = 643.3\ KN$$

and for the upper stories:

2-nd story	$V_{w,s} = 537.3$ KN
3-rd story	$V_{w,s} = 393.3$ KN
4-th story	$V_{w,s} = 271.3$ KN

In case of <u>seismic loads acting from right to left</u>, the following results are obtained:

1-st story	$V_{w,s} = 624.3$ KN
2-nd story	$V_{w,s} = 518.3$ KN
3-rd story	$V_{w,s} = 393.3$ KN
4-th story	$V_{w,s} = 253.3$ KN

5.7.5. Shear Capacity Corresponding to the Limit State of Cracking Due to Diagonal Tension

The capable shear force of a confined masonry wall panel, corresponding to the limit state of cracking, due to diagonal tension, is computed using the following formula:

$$V_{w,cr} = t \cdot \ell_w \cdot f_{wt,d} \cdot \left[\frac{h_w}{2\ell_w} + \sqrt{\left(\frac{h_w}{2\ell_w}\right)^2 + \frac{N}{A_{eqv} \cdot f_{wt,d}} + 1} \right] \qquad (E2.14)$$

provided the following conditions are observed:
- checking of diagonal tensile stress:

$$\sigma_{wt} = -\frac{1}{2}\left(\sigma_o + \tau_o \cdot \frac{h_w}{\ell_w}\right) + \frac{1}{2}\sqrt{\left(\sigma_o + \tau_o \cdot \frac{h_w}{\ell_w}\right)^2 + 4\tau_o^2} \leqslant f_{wt,d} \qquad (E2.15)$$

- checking of shear capacity of the reinforced concrete tie-columns:

$$V_{c,u} \geqslant 0.25\ V_{w,cr} \qquad (E2.16)$$

If the latter condition is not observed, the capable shear force of the confined masonry panel should be re-evaluated in relation to the tie-column capable shear force, by applying the formula:

$$V_{w,cr} = 4\ V_{c,u} \qquad \text{(E2.17)}$$

Applying Eq.E2.14 for the 1-st story, the result is:

$$V_{w,cr} = 250 \times 5450 \times 0.13 \times \left[\frac{2380}{2 \times 5450} + \sqrt{\left(\frac{2380}{2 \times 5450}\right)^2 + \frac{1120 \times 10^3}{3.51 \times 10^6 \times 0.13} + 1} \right] =$$

$$= 370100\ \text{N} \qquad = 370.1\ \text{KN}$$

and for the upper stories:

2-nd story $V_{w,cr} = 341.4$ KN
3-rd story $V_{w,cr} = 309.7$ KN
4-th story $V_{w,cr} = 273.6$ KN

Checking of diagonal tensile stress, shows for the 1-st story:

$$\tau_o = \frac{1.5 \times 172.0 \times 10^3}{250 \times 5450} = 0.19\ \text{MPa}$$

$$\sigma_o = \frac{(1120 - 160) \times 10^3}{3.51 \times 10^6} = 0.27\ \text{MPa}$$

$$\sigma_o + \tau_o \cdot \frac{h_w}{\ell_w} = 0.27 + 0.19 \cdot \frac{2380}{5450} = 0.36\ \text{MPa}$$

$$\sigma_{wt} = -\frac{0.36}{2} + \frac{1}{2}\sqrt{0.36^2 + 0.19^2} = 0.082\ \text{MPa} < 0.13\ \text{MPa}$$

and for the upper stories:

2-nd story $\sigma_{wt} = 0.106$ MPa
3-rd story $\sigma_{wt} = 0.103$ MPa
4-th story $\sigma_{wt} = 0.085$ MPa

For tie-columns reinforced with 4Ø12 (PC 52) and with stirrups Ø8/100 (OB 37) - which is the case for the 1-st, 2-nd and 3-rd stories - the result is:

$$V_{c,u} = 75.0\ \text{KN}$$

As the condition expressed by Eq.E2.16 is not fulfilled, the capable shear force of the wall shall be limited to (Eq.E2.17):

$$V_{w,cr} = 4 \times 75.0 = 300.0\ \text{KN}$$

Similarly, for tie-columns reinforced with 4Ø10 (PC 52) and with stirrups Ø8/100 (OB 37) - which is the case for the 4-th story - the result is:

$$V_{c,u} = 67.5\ \text{KN}$$

hence:

$$V_{w,cr} = 4 \times 67.5 = 270.0 \text{ KN}$$

5.8. Verification

The verification of the cross-sections of the confined masonry walls is performed by applying the following formulae:

$$M_u \geqslant M_d \quad \text{(E2.18)}$$

$$V_{wu} \geqslant 1.5\ V_d \quad \text{(E2.19)}$$

The capable shear force should be considered in Eq.E2.19 as the minimum value among those computed by Eq.E2.11, E2.12 and E2.13.

Tables E2.2 and E2.3 summarize the values of the capable bending moments and, respectively, the shear forces previously computed, as well as the values of the design bending moments and shear forces.

The minimum value for each story, considered as the capable shear force $V_{w,u}$, is underlined in Table E2.3.

Tables E2.2 and E2.3 show that Eq. E2.18 and E2.19 are verified for wall W_{T3} at each story.

The shear capacity at the limit state of cracking due to diagonal tension, - in this case resulting from the shear capacity of the r.c. tie-column (see 5.7.5) - exceeds the corresponding values of the design (required) shear force at each story. Hence, when subjected to the conventional (code) seismic loads corresponding to degree 8 of aseismic protection, wall W_{T3} remains within the elastic range of behavior.

Table E2.2: Capable and Design Bending Moments.

Story	Capable Bending Moment M_u (KNm)		Design Bending Moment M_d (KNm)
	Sense of Seismic Loads		
	$\longrightarrow$	$\longleftarrow$	
1	4080	3880	1815
2	3370	3230	1347
3	2560	2330	811
4	1640	1540	337

Table E2.3: Capable and Design Shear Forces

<table>
<tr><th rowspan="3">Story</th><th colspan="5">Capable Shear Force (KN)</th><th rowspan="2">Factored Design Shear Force 1.5 V_d</th></tr>
<tr><th colspan="2">Related to Capable Bending Moment, V_{wM}
Sense of Seismic Loads</th><th rowspan="2">Failure in Diagonal Tension, $V_{w,dt}$</th><th colspan="2">Shearing of Horizontal Joint, $V_{w,s}$
Sense of Seismic Loads</th></tr>
<tr><th>→</th><th>←</th><th>→</th><th>←</th><th>(KN)</th></tr>
<tr><td>1</td><td>386.7</td><td>367.7</td><td>412.4</td><td>643.3</td><td>624.3</td><td>258</td></tr>
<tr><td>2</td><td>493.3</td><td>472.8</td><td>388.8</td><td>537.3</td><td>518.3</td><td>296</td></tr>
<tr><td>3</td><td>548.6</td><td>499.3</td><td>295.3</td><td>393.3</td><td>393.3</td><td>261</td></tr>
<tr><td>4</td><td>601.7</td><td>565.0</td><td>272.2</td><td>271.3</td><td>253.3</td><td>186</td></tr>
</table>

REFERENCES

[1] *** Romanian "Code for Aseismic Design of Residential, Public, Agro-Zootechnical and Industrial Buildings" (P100-81).

[2] *** Romanian "Code for Analysis and Layout of Masonry Structures" (P2-75) - Revised Edition 1982 (under print).

[3] *** "Masonry Building Construction in Romania" - Input Report Prepared for WG-C of UNDP-UNIDO Project RER/79/015, 1982.

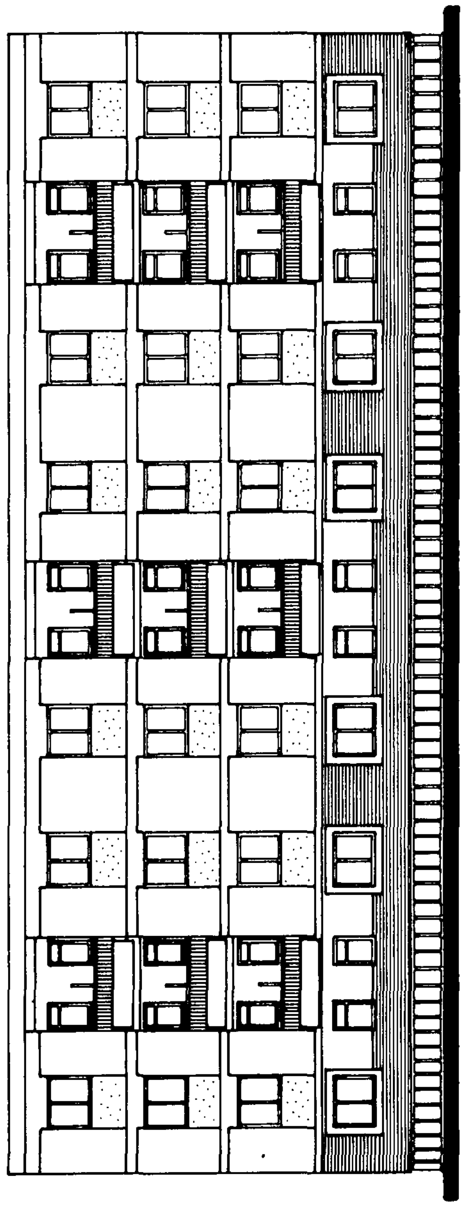

FIG. E2.1: STREET VIEW

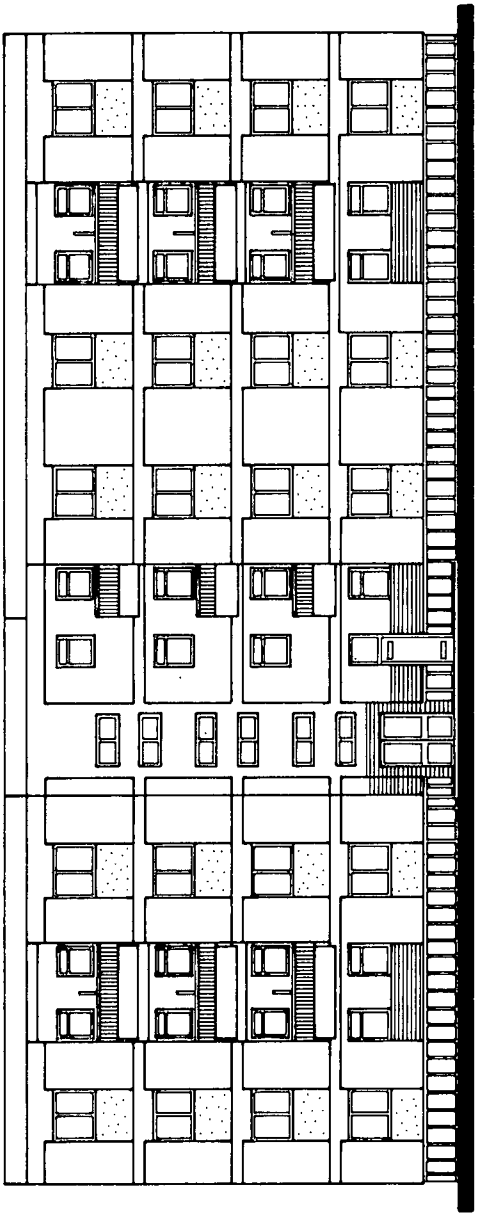

FIG. E2.2: BACK VIEW

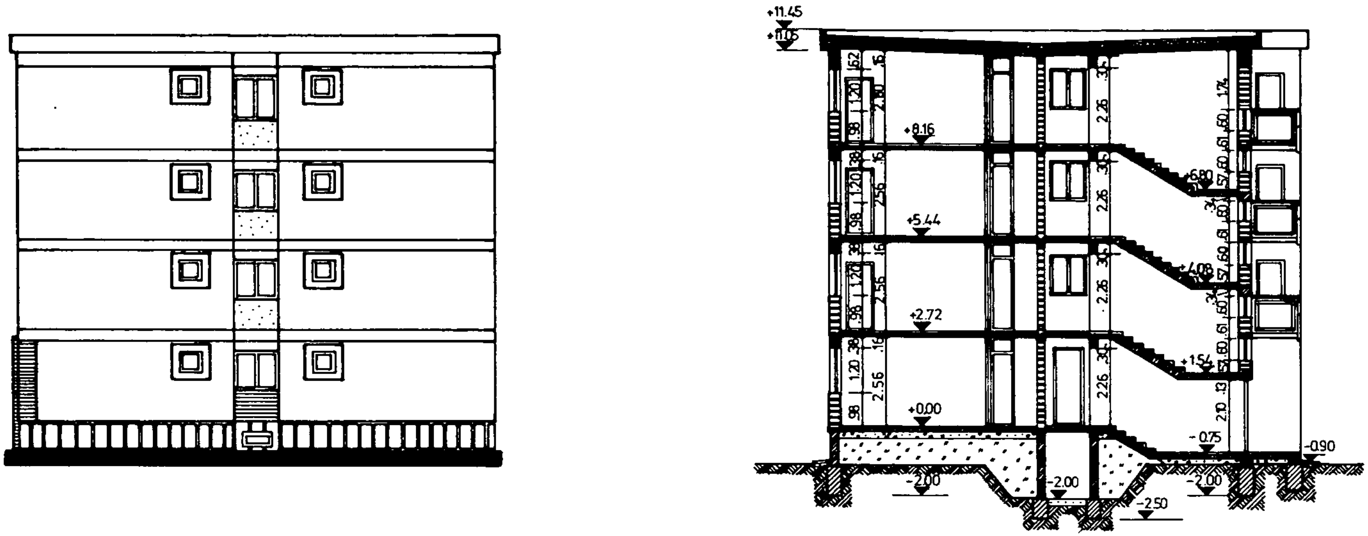

FIG. E2.3: SİDE VİEW AND TRANSVERSAL SECTİON

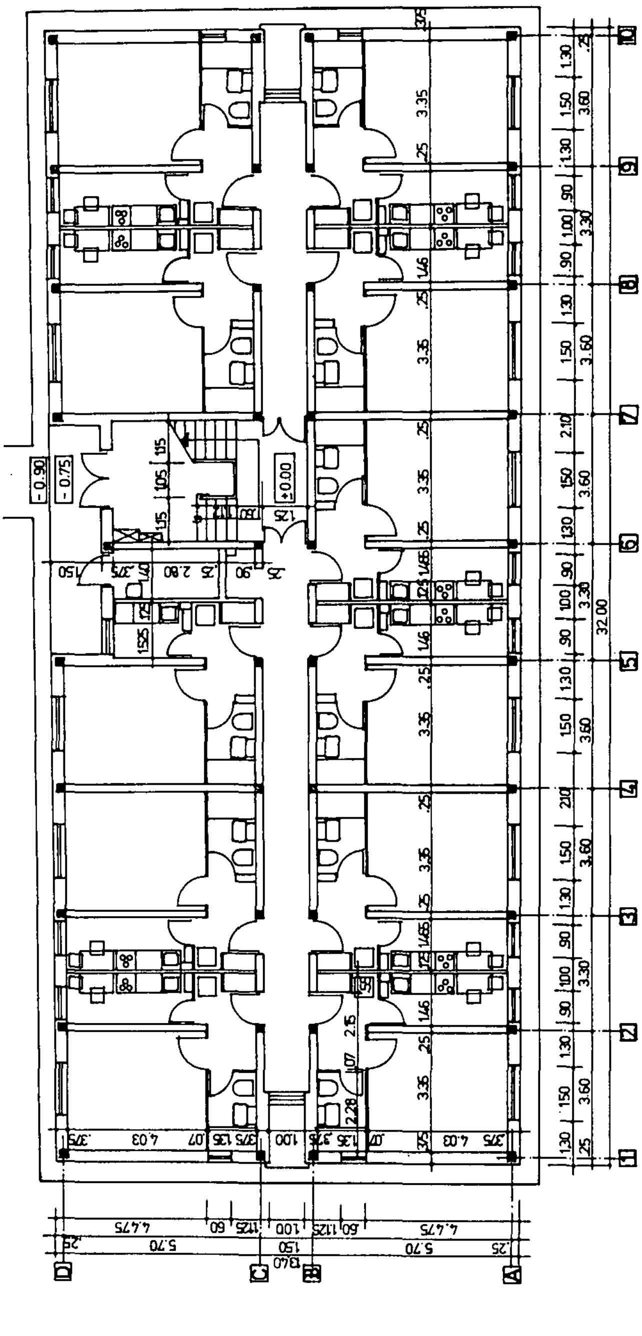

FIG. E2.4 : FIRST STORY PLAN

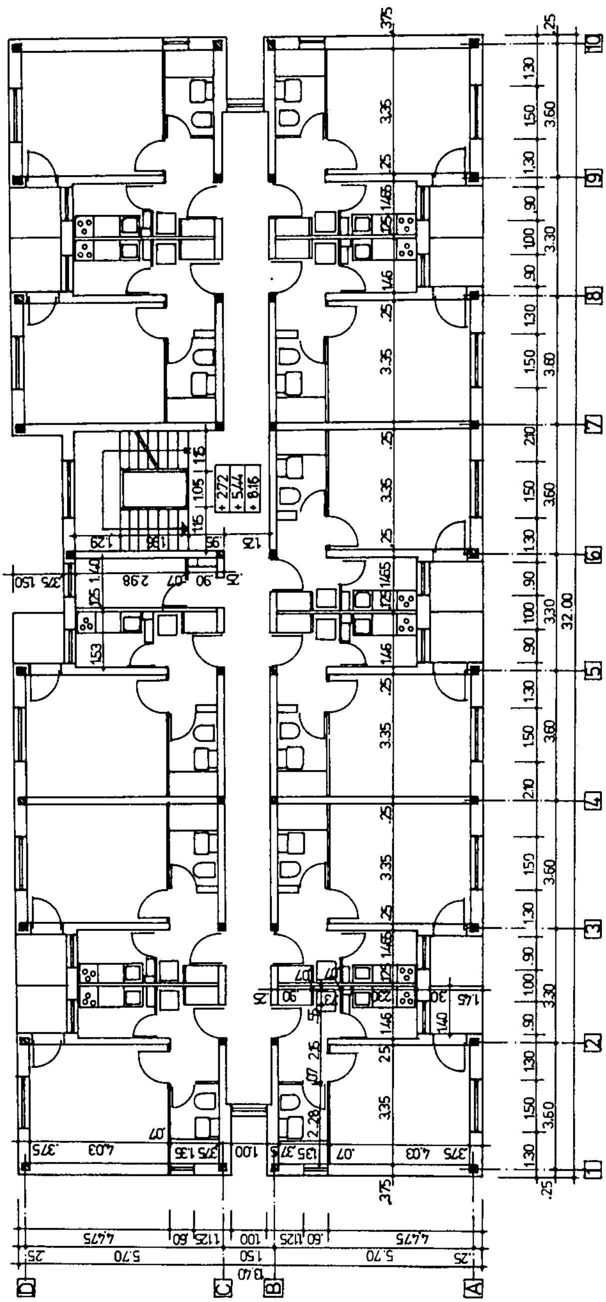

FIG. E2.5: 2-ND, 3-RD AND 4-TH STORY PLAN

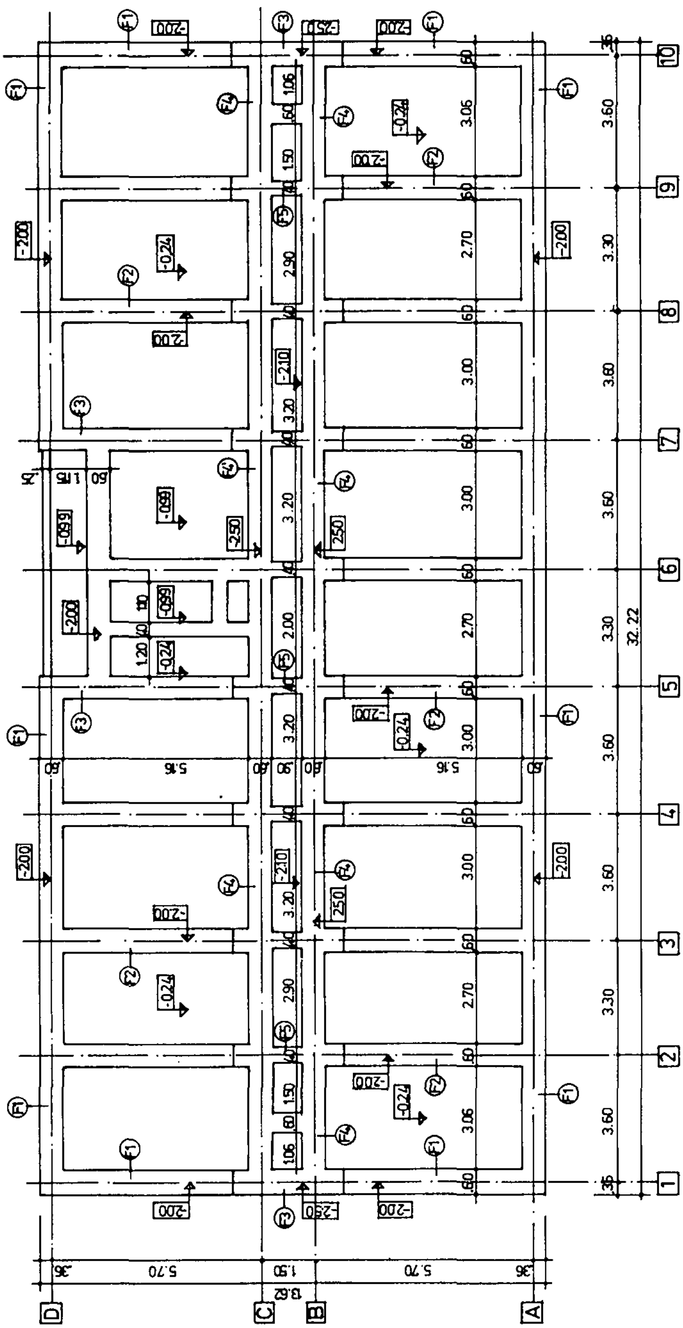

FIG. E2.6: FOUNDATION PLAN

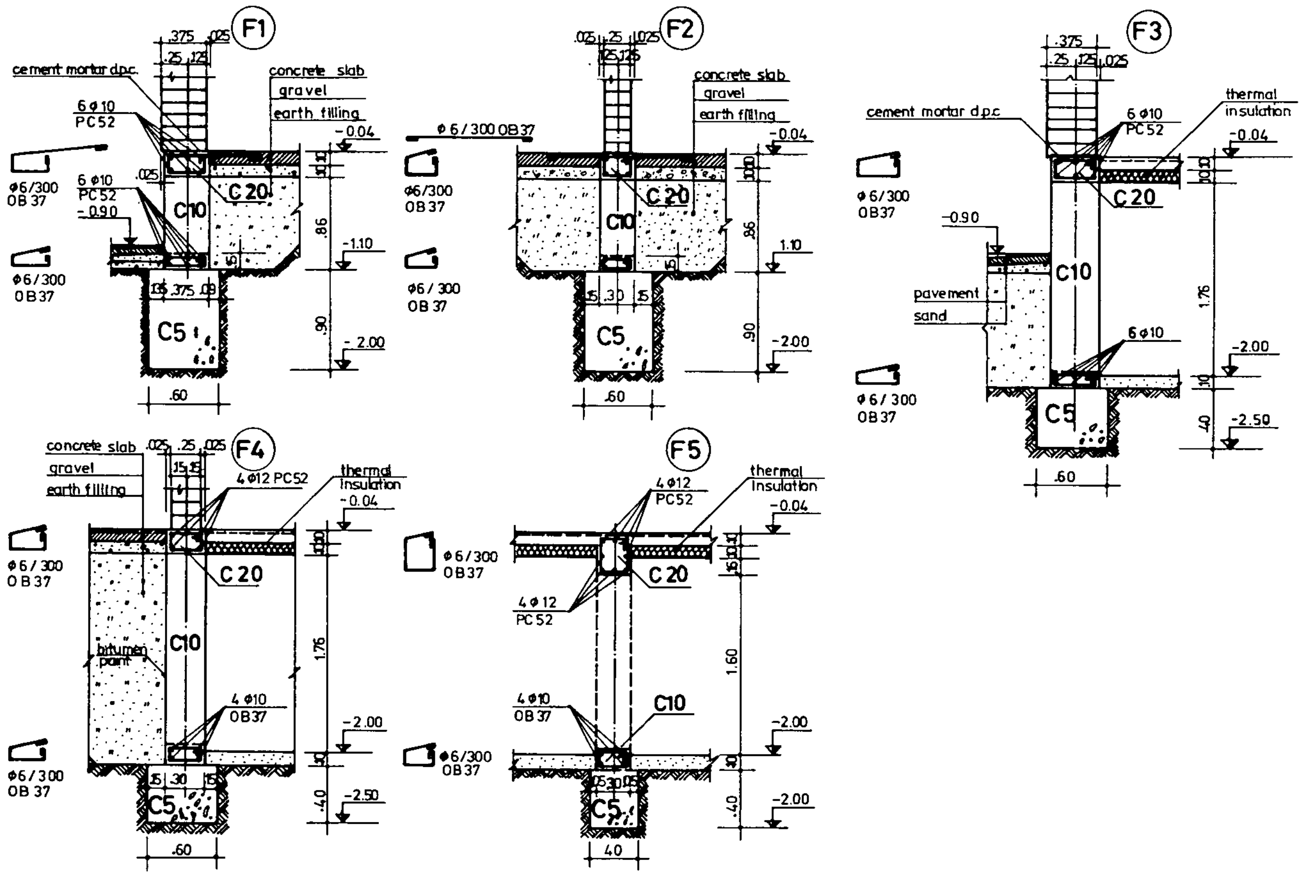

FIG. E2.7: TYPICAL DETAILS OF FOUNDATIONS

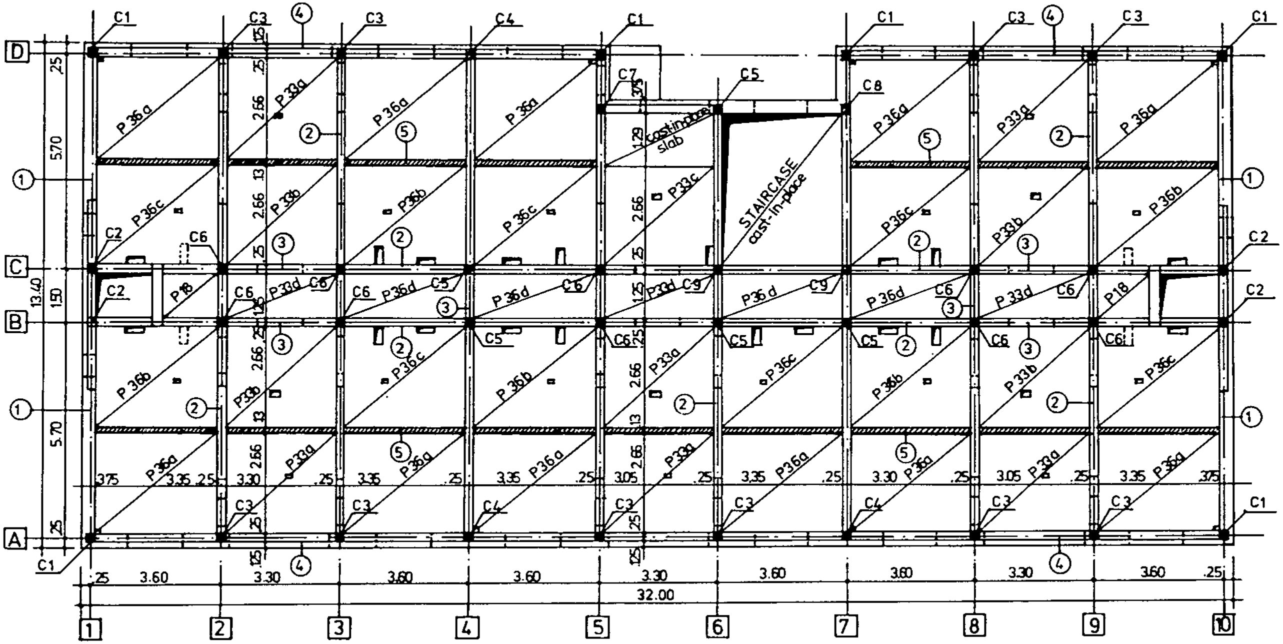

FIG. E2.8: TYPICAL FLOOR PLAN

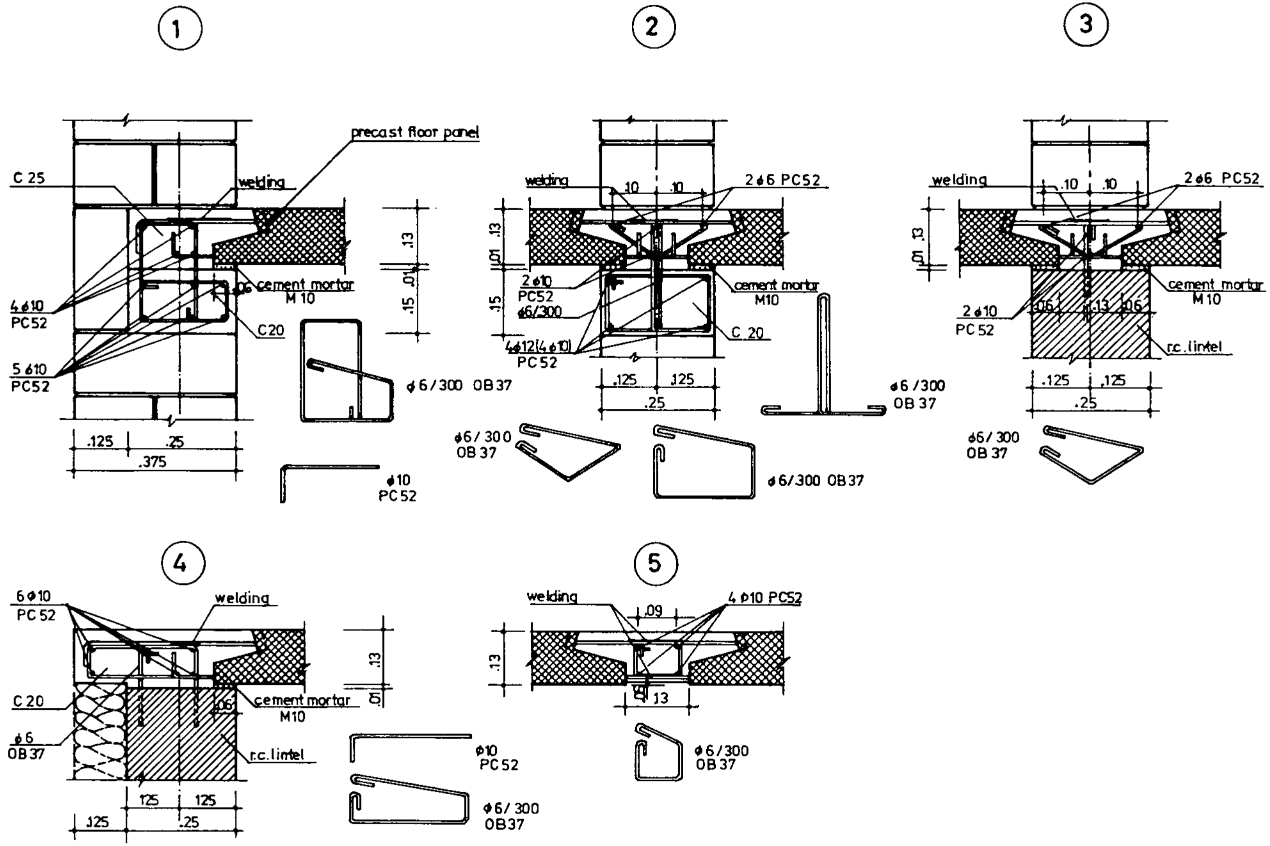

FIG.E. 2.9: TYPICAL DETAILS OF TIE-BEAMS

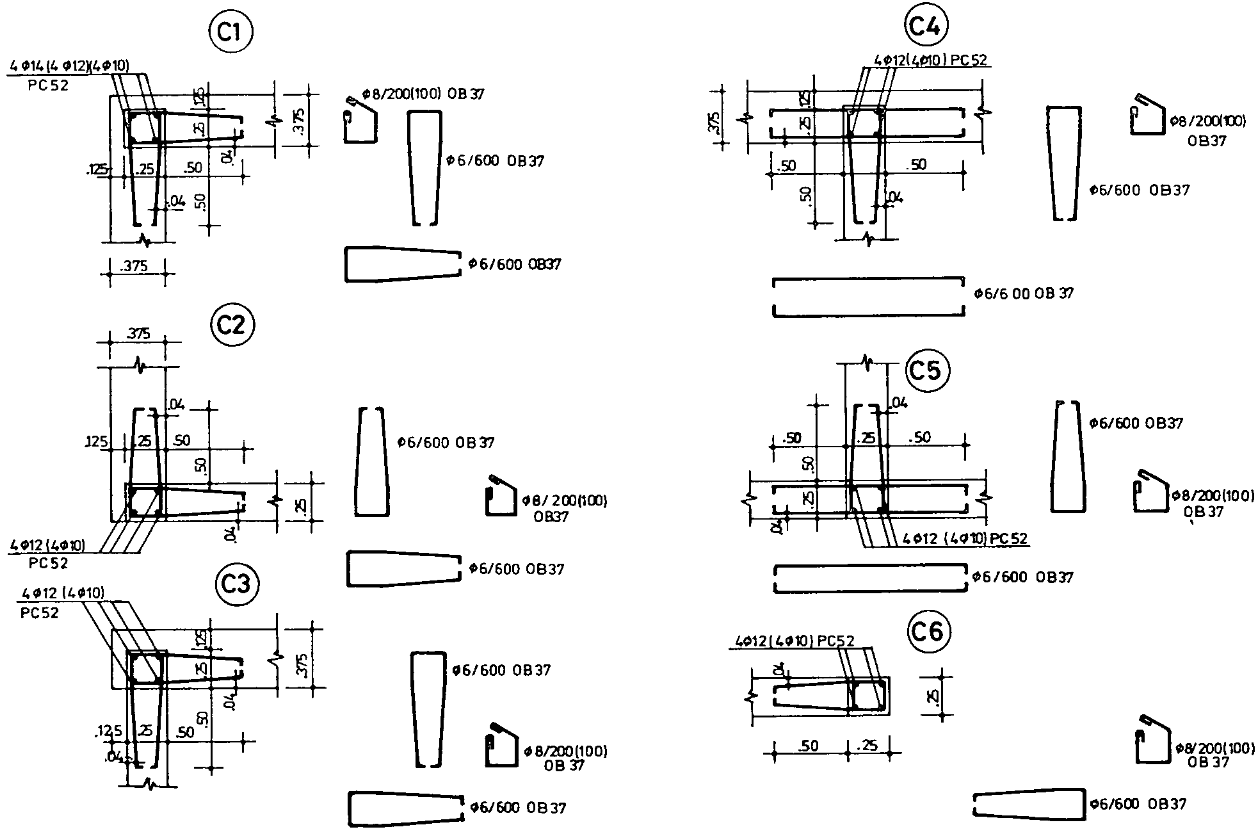

FIG E2.10: TYPICAL DETAILS OF TIE - COLUMNS

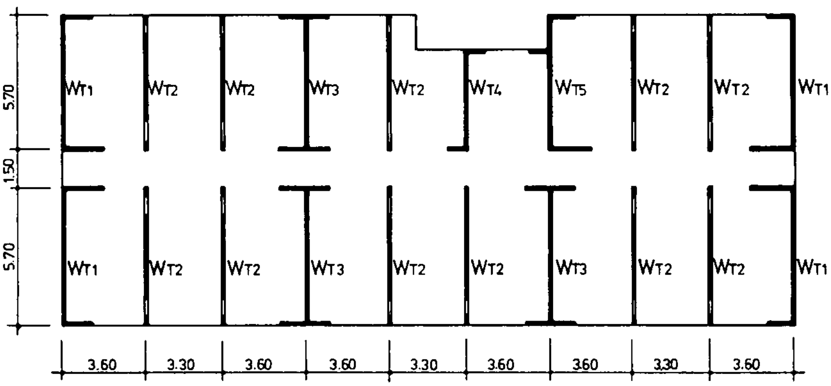

FIG. E2.11 : SCHEME OF STRUCTURAL MASONRY WALLS FOR COMPUTATION ON TRANSVERSAL DIRECTION

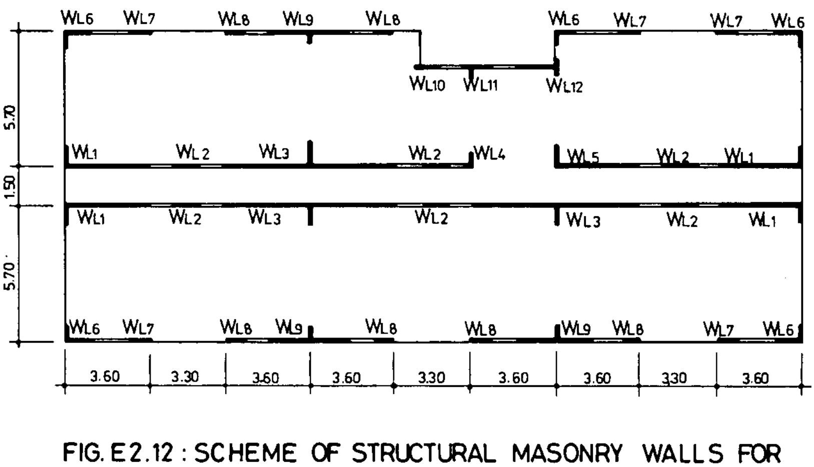

FIG. E2.12 : SCHEME OF STRUCTURAL MASONRY WALLS FOR COMPUTATION ON LONGITUDINAL DIRECTION

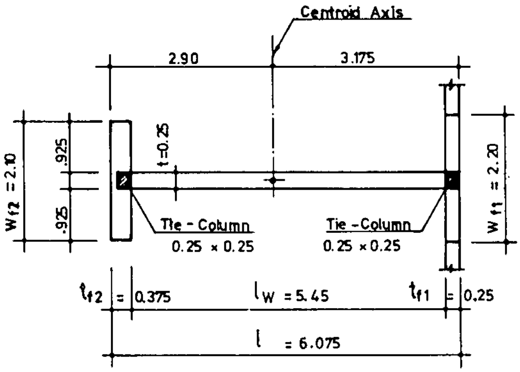

FIG. E2.13 : CROSS-SECTION OF CONFINED MASONRY WALL W_{T3}

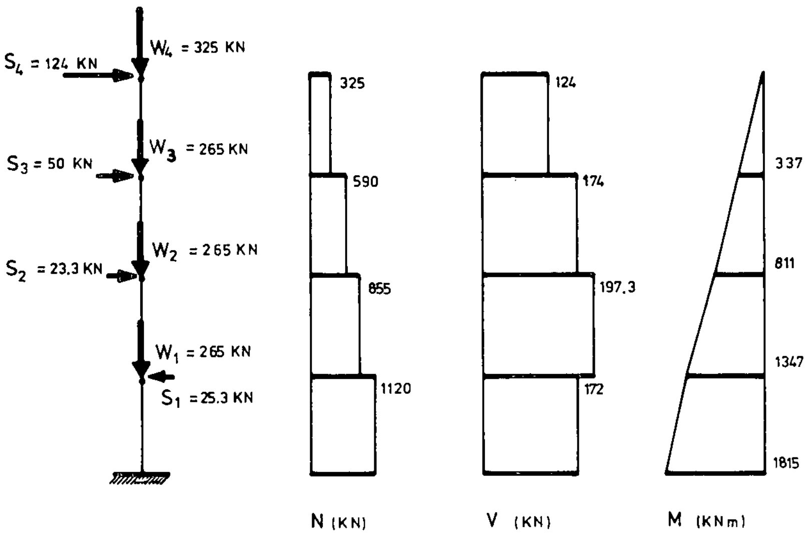

FIG. E2.14 : GRAVITY AND SEISMIC LOADS, DESIGN AXIAL FORCES, SHEAR FORCES AND BENDING MOMENTS FOR CONFINED MASONRY WALL W_{T3}

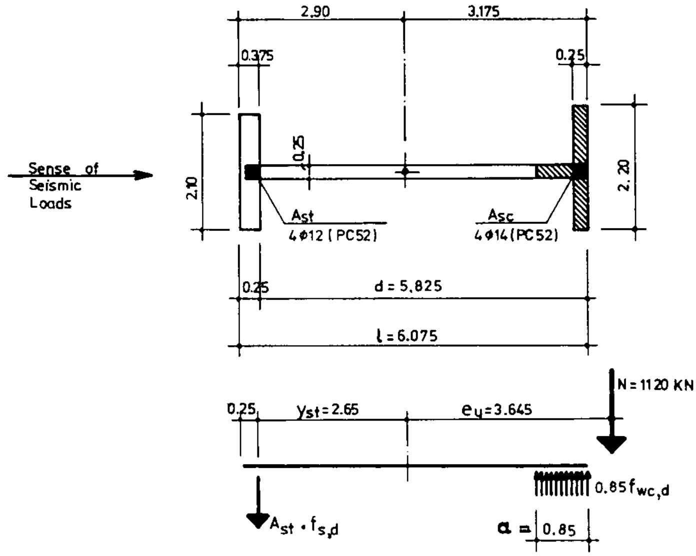

FIG. E 2.15 : LIMIT STATE OF STRENGTH UNDER ECCENTRIC COMPRESSION FOR CONFINED MASONRY WALL W_{T3}

13.3 REPRESENTATIVE DESIGN EXAMPLE - GREECE

Reinforced Masonry Building

by

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FOUR-STORY RESIDENTIAL BUILDING MADE OF REINFORCED MASONRY WALLS

- Floor area: 273.88 m^2
- Floor height: 2.8 m
- Wall thickness: 200 mm (made of vertically perforated bricks 200 x 300 x 150 mm)
  300 mm (made of vertically perforated bricks 300 x 400 x 200 mm)
- Characteristic compressive strength of structural units: $f_{bc,k} = 10.0$ MPa
- Characteristic compressive strength of lime-cement mortar: $f_{mc,k} = 10.0$ MPa
- Assumed characteristic compressive strength of walls (see CIB: "International Recommendations for Design and Erection of Masonry Buildings"): $f_{wc,k} = 5.3$ MPa
- Assumed partial safety factor γ_m (quality control): $\gamma_m = 2.5$
- Resulting design compressive strength of masonry: $f_{wc,d} = 2.12$ MPa
- Characteristic shear strength of walls determined according to the formula proposed by CIB: $f_{wv,k} = 0.3 + 0.4\ \sigma_0$
  where σ_0 : compressive stress due to vertical loads
- Reinforcing steel: Deformed bars are used. Yield stress = 420 MPa
- For R.C. slabs, tie-beams and lintels C20 is used
- For footings concrete with $f_{ck} = 10.0$ MPa is used
- Limit state design method has been applied
- Vertical loads
  i) Horizontal diaphragms: cast-in-place reinforced concrete slabs 120 mm thick (self weight: 2.4 KN/m^2)
  ii) Covering: Roof: 2.0 KN/m^2, other floors: 0.5 KN/m^2
  iii) Live loads: Roof and floors: 2.0 KN/m^2
  Staircase : 3.5 KN/m^2
  Cantilevers : 5.0 KN/m^2
  iv) Wind: 1.0 KN/m^2 of vertical surface
  v) Walls self weight: 3.6 KN/m^2 (walls 200 mm thick)
  5.1 KN/m^2 (walls 300 mm thick)
- For calculation of vertical loads acting on individual walls the floor areas are subdivided into triangles and trapeziums and the loads from these areas are allocated to the appropriate walls.
- The foundamental load combination for vertical loads is $S_d = 1.35\ G + 1.50\ P$
- Bending moments (due to vertical loads) at the fixed floor-wall connections and due to wind loads at the external walls have been considered
- The maximum stress due to vertical loads has been found equal to 0.77 MPa (design compressive strength = 2.12 MPa)
- Base shear coefficient
  The building is supposed to be built in a zone of high seismicity in good soil conditions.
  The base shear coefficient, C_d, is determined according to the CEB - Seismic Annex formula:

$$C_d = I \frac{a \times a_{max}}{g} \frac{1}{K} \beta$$

where I is the importance factor (taken equal to 1.0)
a is the spectral amplification factor (taken equal to 2.5)
a_{max} is the peak ground acceleration to be adopted for the seismic zone of interest (taken equal to 0.30 g)
K is the behaviour factor (equal to 2.5 for reinforced masonry)
β is a dynamic coefficient depending on the soil category and on the dynamic characteristics of the structure (taken equal to 1.0

due to the very low natural period of the structure. Estimated T_n = 0.18 sec - in the short direction and 0.11 sec - in the long direction)

Hence,

$$C_d = 1.0 \frac{2.50 \times 0.30g}{g} \frac{1}{2.50} 1.0 = 0.30$$

- The total horizontal seismic force is calculated according to the following formula:

  $V = C_d W$

  where,

  W is the total vertical load acting on the structure: $W = G + 0.5\ P$

- The total horizontal seismic force is distributed to the floor according to the formula:

  $$P_x = V \frac{W_x h_x}{\sum_{n=1}^{n} W_i h_i}$$

  where,

  P_x is the horizontal force at x-level

  W_x is the total vertical load at x-level

  h_x is the distance of x-level from foundation level

- The total horizontal seismic force, V, is equal to 3176 KN
- Horizontal seismic force at

4th floor	:	1247 KN
3rd floor	:	954 KN
2nd floor	:	636 KN
1st floor	:	339 KN

- Lateral load analysis:

  Horizontal diaphragms (cast-in-place R.C. slabs) are considered rigid in their plane. Consequently, horizontal seismic force is distributed to the walls proportionally to their stiffnesses.

  For estimation of the stiffnesses of walls the wide column frame analogy has been used.

  After the estimation of the wall stiffnesses, the seismic force has been distributed to them. Torsional effects, due to the fact that for earthquake acting along the long axis of the building the centre of masses does not coincide with the centre of stiffnesses, have been taken into account.

  This analysis has shown that for walls with a height to width ratio approximately equal to 1.0 the bending moment - values at the wall end are close to those which would result if the walls would be considered fixed at their ends into the horizontal diaphragms at each storey level (see Fig. 1a). On the contrary in case of squat walls with height to width ratio approx. equal to 0.50 (see Fig. 1b, c) the assumption of walls - cantilevers of a height equal to the building height seems to be more realistic.
- The horizontal reinforcement (placed in bed joints) has been determined assuming that the total shear force has to be resisted by reinforcement. The required reinforcement was in no case greater than the minimum percentage (0.15% of the horizontal gross cross sectional area) required for structural walls in zones of high seismicity. Hence, in all walls the minimum percentage has been placed (2∅8 at each bed joint).
- The vertical reinforcement (placed in the vertical perforations of the bricks) has been calculated assuming that the total tensile force (due to bending moments) is resisted by reinforcement. In no case the required vertical reinforcement was greater than the minimum required 0.15%. Hence, the typical vertical reinforcement is ∅10/150 mm.
- Reinforced concrete tie-beams are provided at all floor levels. Their

section is 300 x 300 mm. They have been considered as horizontal beams loaded by the seismic force corresponding to the wall on which they rest. Their longitudinal reinforcement is 4∅10 or 4∅14 or 4∅16, their transverse reinforcement being stirrups ∅6/200 or ∅8/200.

- Above the openings the tie-beams are 0.60 m high, being tie-beams and lintels at the same time.
- Foundation: continuous footings 0.80 m wide.
- As an example the design of the structural wall W5 (ground floor) is given in the following, so as to illustrate the method for design of reinforced masonry walls.

i) Vertical loads

From vertical load analysis, the vertical load acting on the wall has been determined: dead load = 86.88 KN/m
live load = 22.09 KN/m

The foundamental load combination for vertical load is

$$S_d = 1.35\ G + 1.50\ P$$

Hence, design vertical load $N_d = 1.35 \times 86.88 + 1.50 \times 22.09 = 150.42$ KN/m

Design vertical load resistance of the wall (per unit length)

$$R_{wc,d} = \beta\ t\ f_{wc,d}$$

where,
β = (capacity reduction factor due to buckling) = 1.0
t = effective wall thickness =0.30 m
$f_{wc,d}$=design compressive strength of the wall = 2.12 MPa
Hence,

$$R_{wc,d} = 0.30 \times 2120 = 636\ \text{KN/m} > 150.42\ \text{KN/m} = N_d$$

ii) Lateral loads

Shear force and bending moment due to earthquake have been determined,

V_d = 307.35 KN

M_d = 999.39 KNm

The load combination in case of earthquake is

$$S_d = G + 0.5\ P$$

Hence,
design vertical load $N_d = 86.88 + 0.50 \times 22.09 = 97.93$ KN/m or

$N_d = 4.30 \times 97.93 = 421.10$ KN (wall length = 4.30 m)

- Shear

The characteristic shear strength of the unreinforced wall is given by the following formula:

$$f_{wv,k} = 0.30 + 0.40\ \sigma_o$$

where σ_o is the normal stress due to vertical loads

Hence,

$$f_{wv,k} = 0.30 + 0.40 \frac{421.10}{4.30 \times 0.30 \times 10^3} = 0.43 \text{ MPa}$$

A γ_m factor equal to 2.50 has been assumed
Consequently the design shear strength of the wall will be

$$f_{wv,d} = f_{wv,k} : \gamma_m = 0.43 : 2.50 = 0.17 \text{ MPa}$$

Design shear resistance of the wall

$$R_{wv,d} = f_{wv,d} \times t \times l = 0.17 \times 300 \times 4300 \times 10^{-3} = 220.0 \text{ KN} < 307.85 \text{ KN} = V_d$$

Since the design shear resistance of the unreinforced wall is smaller than the design shear force, then reinforcement must be provided. This horizontal reinforcement has to resist the total shear force.

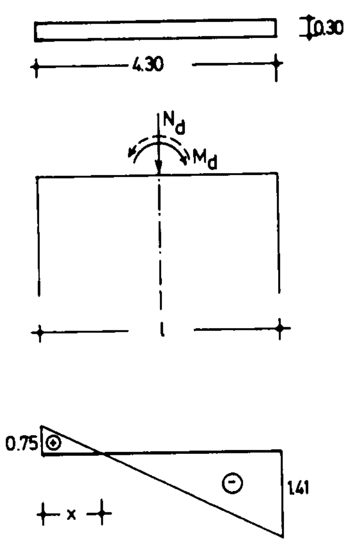
$$A_{s,h} = \frac{V_d\, s\, \gamma_s}{0.80\ l\ f_{sy}}$$

where,
$A_{s,h}$ = area of the horizontal reinforcement
s = bar spacing
γ_s = partial safety factor for steel (= 1.15)
l = wall length
f_{sy} = yield stress of steel

Hence: $A_{sh} = \frac{307850 \times 200 \times 1.15}{0.80 \times 4300 \times 420} = 49.01 \text{ mm}^2$

Minimum reinforcement: 0.15% of the gross cross sectional area of the wall (90 mm²/200 m or 2∅8/200)

- Bending



Design bending moment M_d = 999.39 KNm
Design vertical load N_d = 421.10 KN

Maximum compressive stress,

$$\sigma^- = -\frac{421.10}{4.3 \times 0.3 \times 10^3} - \frac{999.39 \times 6}{0.3 \times 4.30^2 \times 10^3} =$$

$$= -0.33 - 1.08 = -1.41 \text{ MPa} < 2.12 \text{ MPa}$$

Maximum tensile stress

$$\sigma^+ = -0.33 + 1.08 = 0.75 \text{ MPa}$$

$$x = \frac{0.75}{1.41 + 0.75} 4.30 = 1.49 \text{ m}$$

Tensile force (to be resisted by reinforcement)

$$= \frac{1}{2} \times 0.75 \times 1490 \times 300 = 167.63 \text{ KN}$$

Vertical reinforcement: $A_{sy} = \frac{167630}{420/1.15} =$

$= 459 \text{ mm}^2/1.49 \text{ m}$ or $308 \text{ mm}^2/\text{m}$

Minimum vertical reinforcement:

$$\min A_{sv} = 0.15 \times 10^{-2} \times 1000 \times 300 = 450 \text{ mm}^2/\text{m} \qquad (\varnothing 10/150)$$

Flexural resistance of the wall:

$$M_u = \frac{0.326 \times 300 \times 4300^2}{2} \left(1 - \frac{0.326}{2.12}\right) + 2 \times 450 \times 1.49 \times \frac{420}{1.15} \left(2150 - \frac{1490}{3}\right) =$$

$$= 1574.86 \text{ KNm} \; > 999.39 \text{ KNm} = M_d$$

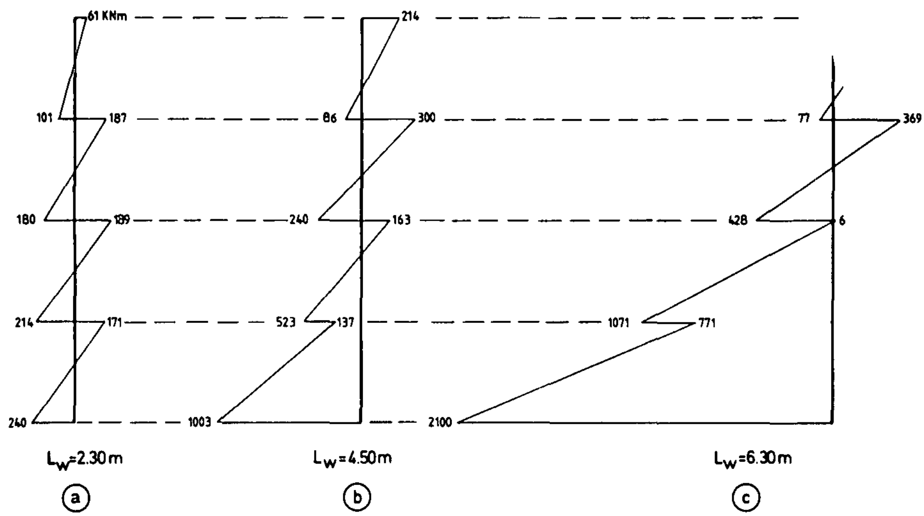
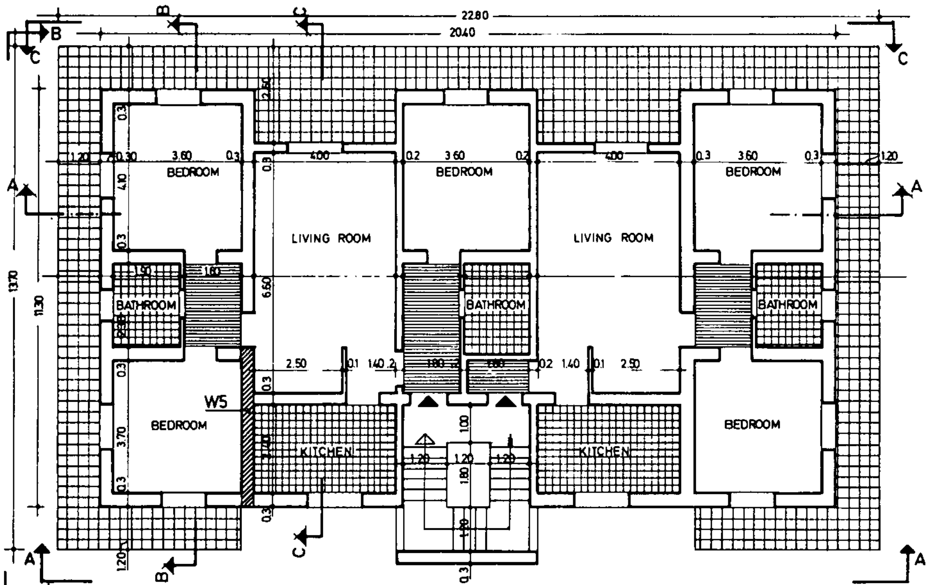


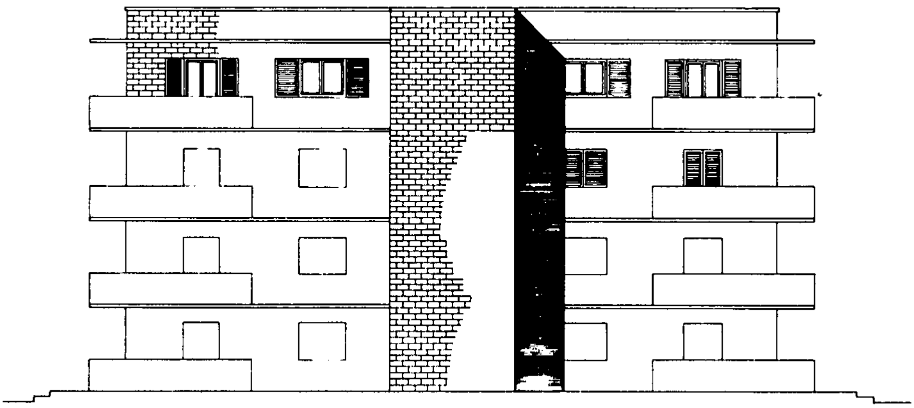
FIGURE 1

In the following a typical floor plan of the building (Figure 2), as well as longitudinal and transverse elevations (Figures 3 and 5, and Figure 4, respectively) are shown. Sections are presented in Figures 6, 7 and 8. Figures 9 and 10 show a typical floor and a typical floor tie-beam lay-out, respectively. Finally, details giving tie-beam reinforcement, wall reinforcement and a typical floor-wall connection, and a typical footing detail are presented in Figures 11, 12, and 13, respectively.



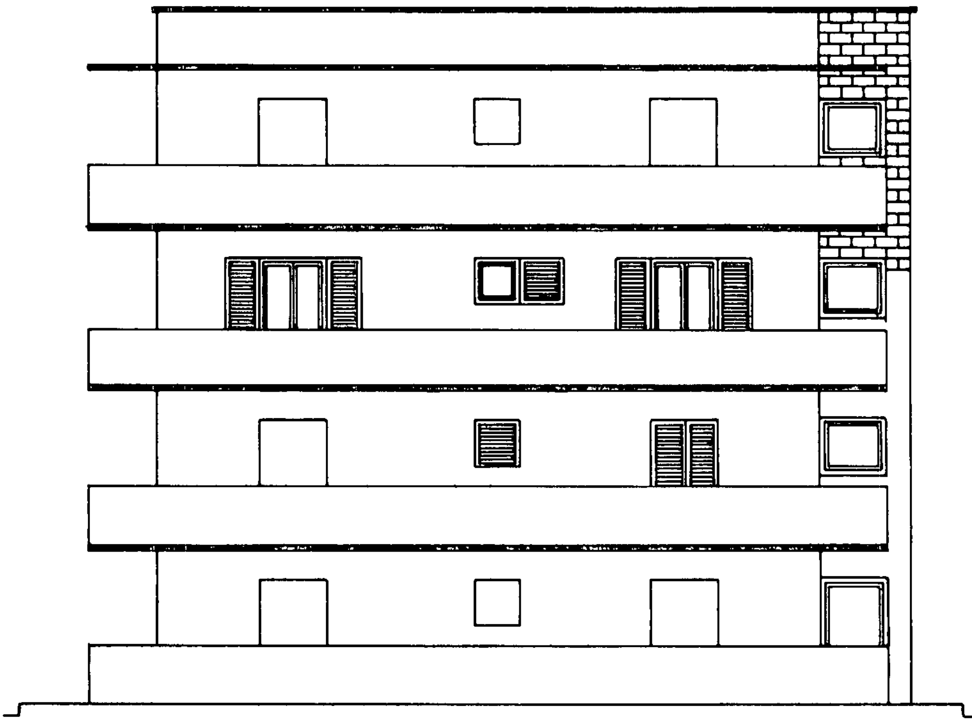
TYPICAL FLOOR

Fig. 2



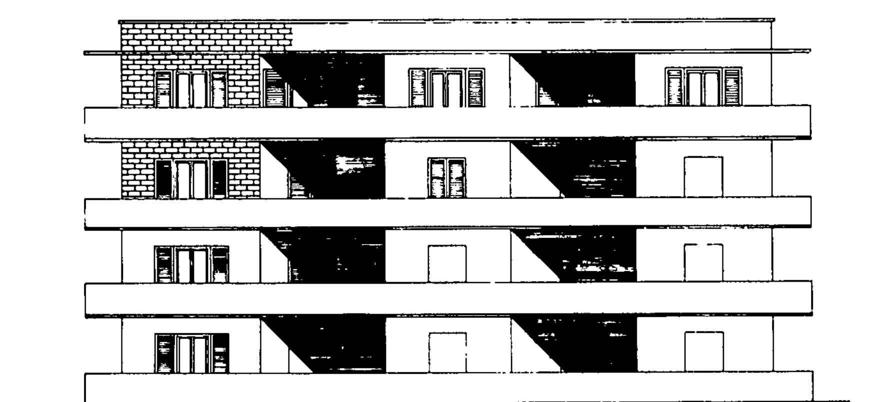
ELEVATION A-A

Fig. 3



ELEVATION B-B

Fig. 4



ELEVATION C-C

Fig. 5

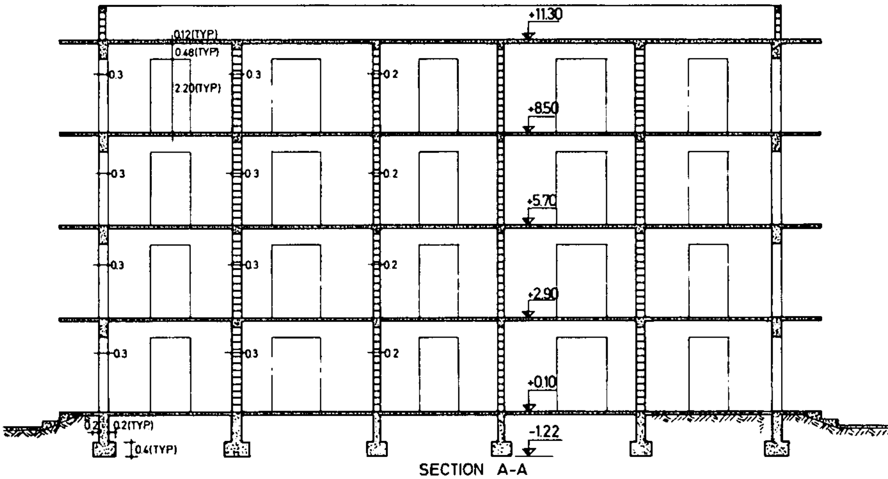

+11.30
0.12(TYP)
0.48(TYP)
2.20(TYP)
0.3
0.2
+8.50
+5.70
+2.90
+0.10
-1.22
0.2(TYP)
0.4(TYP)
SECTION A-A

Fig. 6

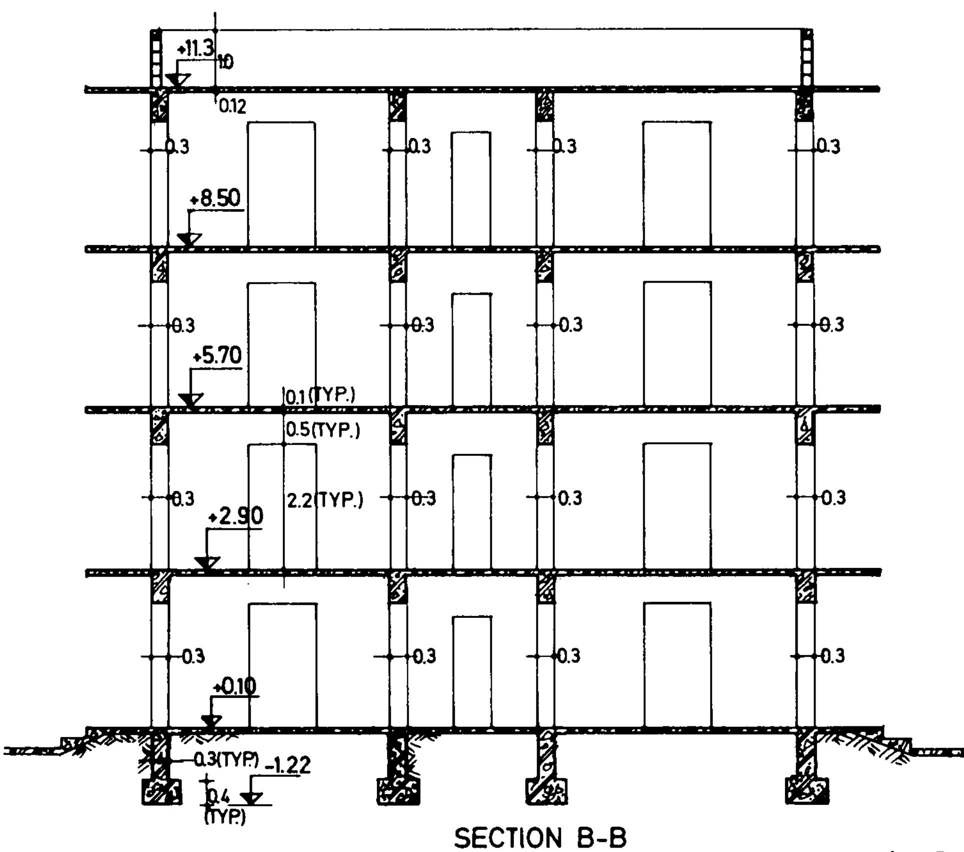
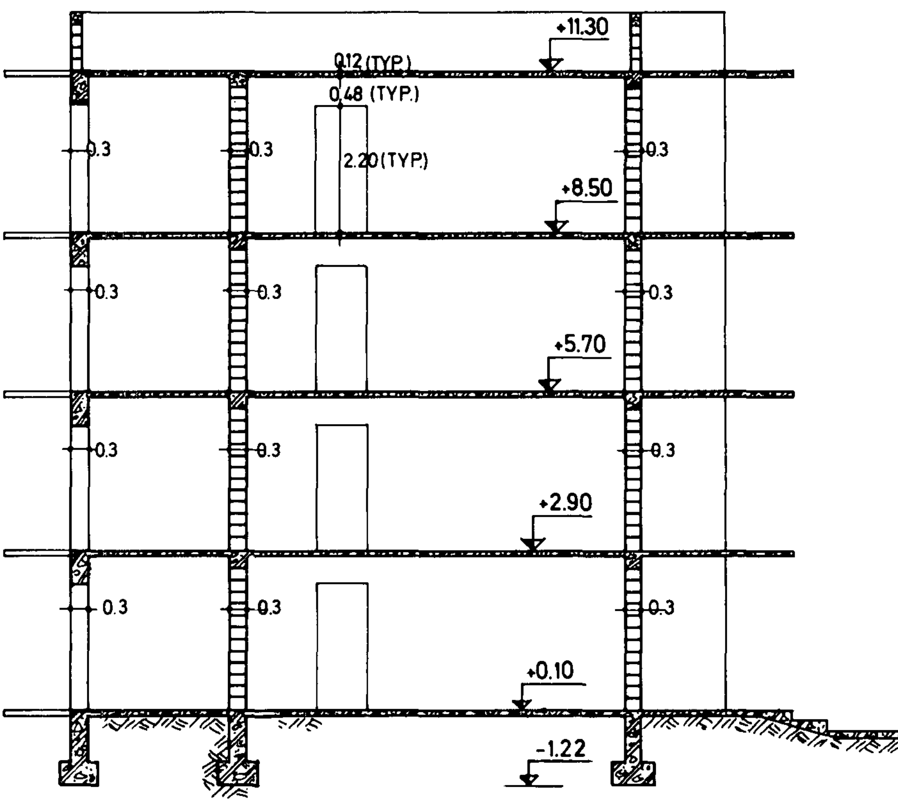

+11.3
1.0
0.12
0.3
+8.50
+5.70
0.1(TYP.)
0.5(TYP.)
2.2(TYP.)
+2.90
+0.10
0.3(TYP.)
-1.22
0.4
(TYP.)
SECTION B-B

Fig. 7



SECTION C-C

Fig. 8

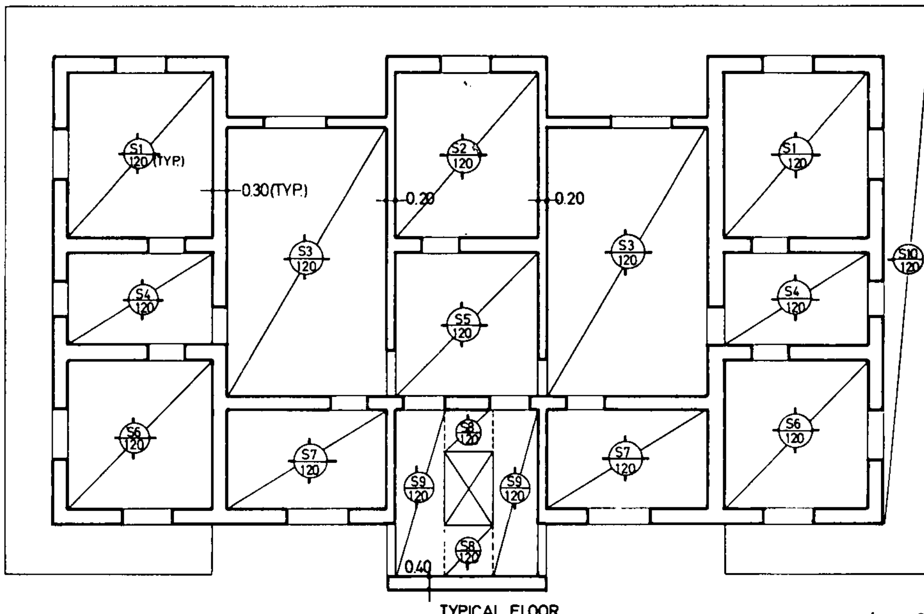

S1
120
(TYP.)
0.30(TYP.)
0.20
0.20
S2
S3
S4
S5
S6
S7
S8
S9
S10
0.40
TYPICAL FLOOR

Fig. 9

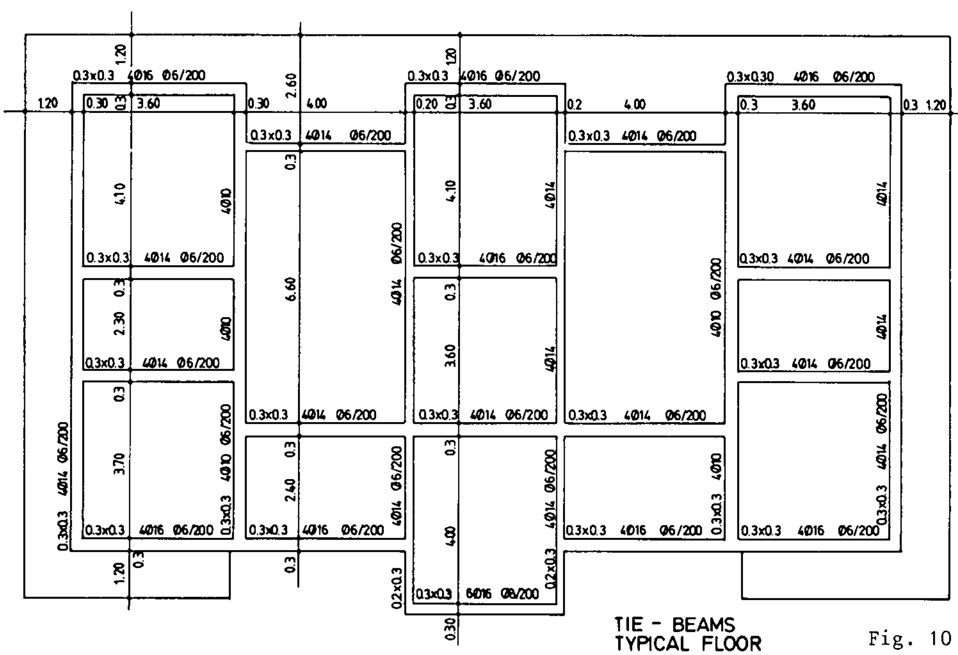

0.3x0.3 4Ø16 Ø6/200
0.3x0.3 4Ø14 Ø6/200
0.3x0.3 4Ø16 Ø6/200
0.3x0.3 6Ø16 Ø8/200
TIE - BEAMS
TYPICAL FLOOR

Fig. 10

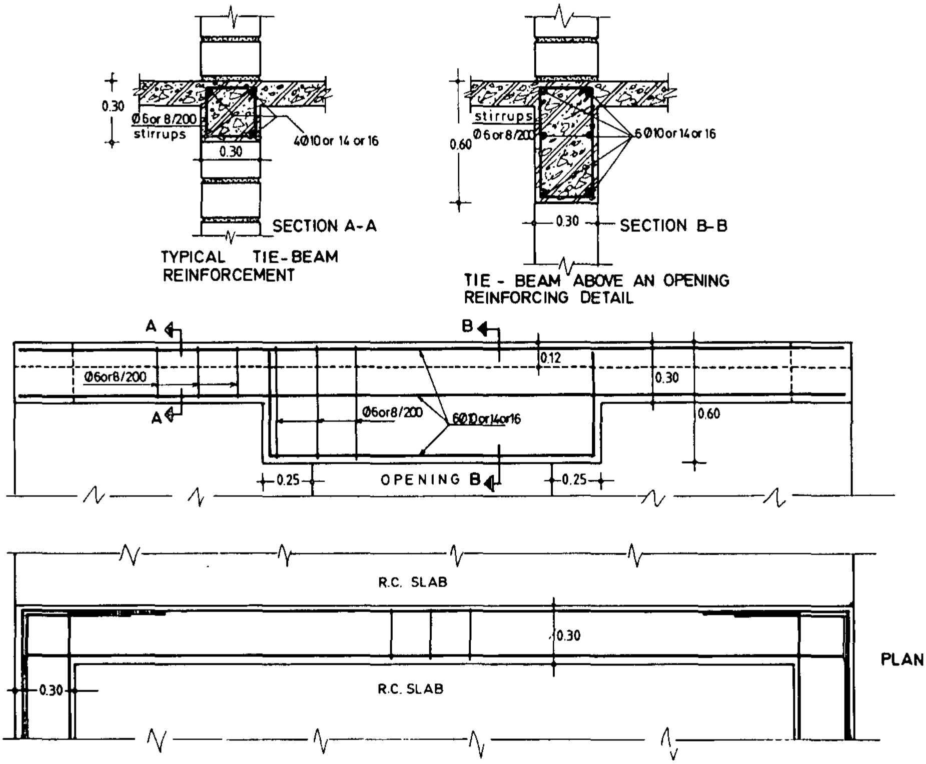
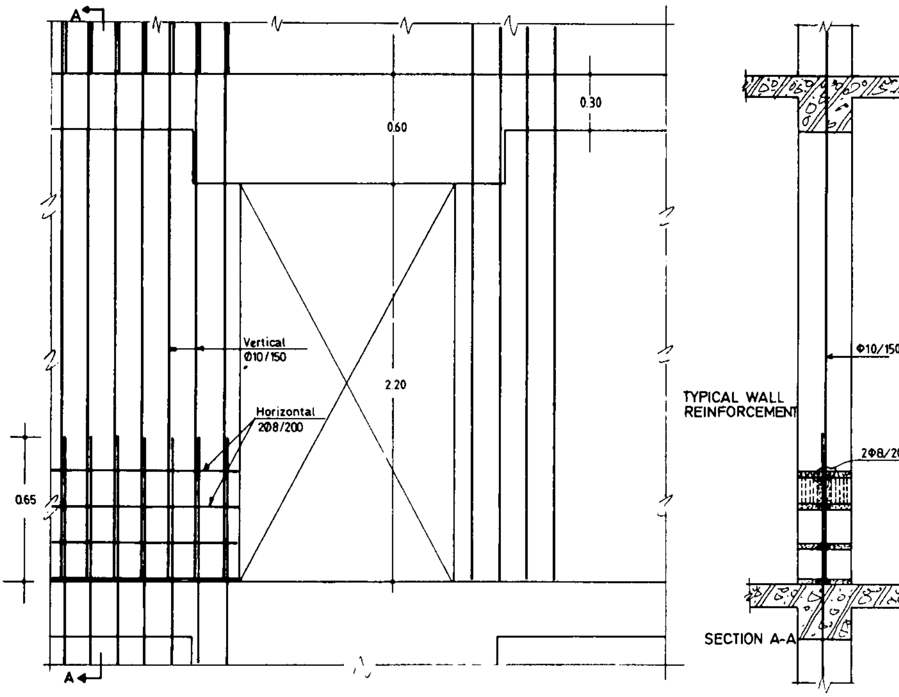
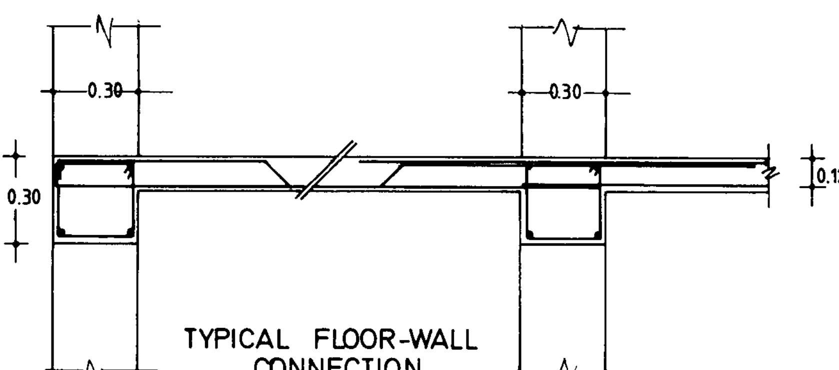

0.30
Ø6or 8/200
stirrups
4Ø10 or 14 or 16
0.30
SECTION A-A
TYPICAL TIE-BEAM
REINFORCEMENT
stirrups
Ø6 or 8/200
0.60
6Ø10 or 14 or 16
0.30
SECTION B-B
TIE - BEAM ABOVE AN OPENING
REINFORCING DETAIL
A
A
B
B
Ø6or8/200
0.12
0.30
Ø6or8/200
6Ø10or14or16
0.60
0.25
OPENING
0.25
R.C. SLAB
0.30
R.C. SLAB
0.30
PLAN

Fig. 11

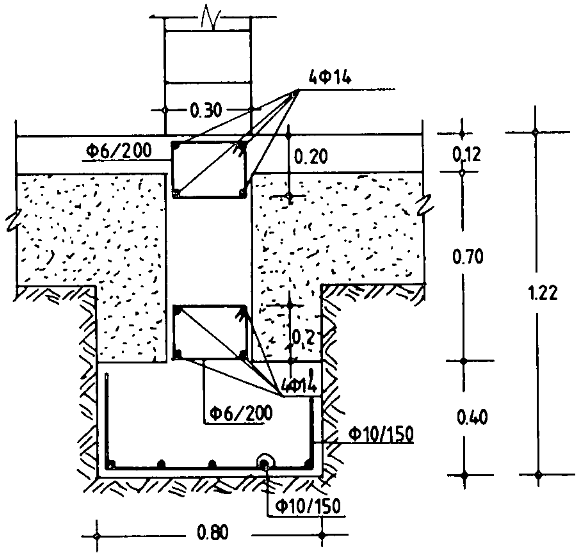


A
0.30
0.60
Vertical
Ø10/150
2.20
Horizontal
2Ø8/200
0.65
A
Φ10/150
TYPICAL WALL
REINFORCEMENT
2Φ8/200
SECTION A-A



0.30
0.30
0.30
0.12
TYPICAL FLOOR-WALL
CONNECTION

Fig. 12



TYPICAL FOOTING DETAIL

Fig. 13