

ADDIS ABABA UNIVERSITY  
SCHOOL OF GRADUATE STUDIES



FACULTY OF TECHNOLOGY  
DEPARTMENT OF CIVIL ENGINEERING

Design and Cost Comparison of Reinforced Masonry and  
Reinforced Concrete Building Structures

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A Thesis Submitted to School of Graduate Studies of Addis Ababa University in Partial  
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(Structures Major)

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Last, big thank you, goes to my parents and so I say, God bless you dear mother and father.

## **ABSTRACT**

Most building structures known to be constructed in Ethiopia and other many countries contain, by rule of thumb or knowledge, some proportion of masonry components. Despite this reality, the development of analysis and design techniques for masonry structures has lagged behind reinforced concrete. The lack of engineered guide lines, at governmental level, to the design of reinforced masonry and failure of local universities to deliver courses on masonry has played their shares in the ill-treatment of this potentially rich construction material-masonry.

Not few people perceive, as stone solely, the meaning of masonry and have clouded perceptions regarding the extendible use of this material as core component of a structure. This research work, therefore, attempts to set some insights on the actual meaning of masonry and the possible positive outcomes that can be enjoyed by making use of it in an engineered manner. To do so, this thesis introduces some of the available methods of design based on limit state design philosophy and so investigates some standard documents, in the captioned philosophy, like Ethiopian building code standard and the British code for design of masonry.

Based on techniques in the reviewed materials a G+4 building structure, consisting of reinforced masonry wall as a main structural component and located in a Zone 2 seismic region of Ethiopia, is modeled in 3-D by equivalent frame idealization, designed and its cost is compared with a similar reinforced concrete building. It is found that better economy, with overlapping advantages of sound insulation and short construction time are gained from the use of reinforced masonry as a main structural component of building structures with current and practical price ranges. Considering gradual cost increase in masonry units and decrease in cost of cement, starting from current prices of both materials, the break even point for the cost of construction of the designed building using both construction material types is found.

## NOTATIONS

$A$	Cross-sectional area of masonry ( $\text{mm}^2$ )
$A_s$	Area of compression reinforcement in the least compressed face ( $\text{mm}^2$ )
$A_s$	Cross-sectional area of Primary reinforcing steel ( $\text{mm}^2$ )
$A_{s1}$	Area of compression reinforcement in the most compressed face ( $\text{mm}^2$ )
$b$	Width of section (mm)
$\beta$	Capacity reduction factor for walls allowing for effects of slenderness and eccentricity
$c$	Lever arm factor
$d$	Effective depth (mm)
$d_1$	Depth from the surface to the reinforcement in the more compressed face (mm)
$d_2$	Depth of the centroid of the reinforcement from the least compressed face (mm)
$d_c$	Depth of masonry in compression
$e_a$	Additional eccentricity due to deflection in walls
$E_c$	Modulus of elasticity of concrete
$E_m$	Modulus of elasticity of masonry
$E_s$	Modulus of elasticity of Steel
$e_m$	The larger of $e_x$ or $e_t$
$e_t$	Total design eccentricity in the mid-height region of a wall
$e_x$	Eccentricity at top of a wall
$f'_{mb}$	Characteristic compressive strength of masonry ( $f_k$ in BS 5628)
$f_{sy}$	Characteristic tensile strength of reinforcement steel
$f_v$	Characteristic Shear Strength of Masonry
$f_y$	Characteristic tensile strength of reinforcing steel
$h_{ef}$	Effective height or length of wall or column
$K_h$	Compressive strength factor
$N$	Design vertical load (N)
$N_d$	Design vertical load resistance (in N)
$Q$	Moment of resistance factor
$S$	Spacing of shear reinforcement along the member
$S_r$	Slenderness ratio of a member

$t$	Overall thickness of a wall or column
$t_{ef}$	Effective thickness of a wall or column
$t_r$	Minimum thickness of the member
$V$	Shear force due to design loads
$v$	Shear stress due to design loads
$z$	Lever arm
$\gamma_f$	Partial safety factor for load
$\gamma_m$	Partial safety factor for material
$\gamma_{mm}$	Partial safety factor for compressive strength of masonry
$\gamma_{ms}$	Partial safety factor for shear strength of masonry
$\gamma_{mv}$	Partial safety factor for material in shear
$\rho$	$A_s/bd$

## **GLOSSARY**

### **Bed Joint**

Horizontal joint in masonry

### ***Cavity Wall***

Two similar or dissimilar wythes of masonry separated by an air space, and connected by metal ties. The cavity provides a space for insulation, and also a drainage path for any water entering the wall. To function properly, the cavity must be kept free of mortar bridges, droppings and flashing and weep holes must be correctly detailed and installed.

### ***Cellular Arrangement***

Wall arrangement in which both internal and external walls are load bearing and these walls form a cellular pattern in plan.

### ***Complex arrangement***

All kinds of hybrids between cellular and cross-wall arrangements are possible, and these are included under the heading 'complex'.

### ***Course***

Single layer of masonry units of uniform height, including the bed joint

***Cross wall Arrangement***

A wall arrangement in which the main bearing walls are at right angles to the longitudinal axis of the building. The floor slabs span between the main cross-walls, and longitudinal stability is achieved by means of corridor walls.

***Effective depth***

Depth, in members in bending, from the compression face to the centroid of the longitudinal tensile reinforcement or prestressing tendons

***Effective height or length***

The height or length of a wall, pier or column assumed for calculating the slenderness ratio

***Effective thickness***

The thickness of a wall, pier or column assumed for calculating the slenderness ratio

***Grouted masonry***

Masonry in which some or all of the cavities in the masonry or cores in hollow units, are filled with grout, but is not reinforced.

***Header***

Shorter face of a masonry unit showing on the face of a wall

***Masonry***

Assemblage of structural units laid in situ or constructed in prefabricated panels, in which the structural units are bonded and solidly put together with concrete and/or mortar so as to act compositely. Masonry may be reinforced or unreinforced

***Mortar***

Mix of one or more inorganic binders, aggregates, water, and sometimes additives and/or admixtures

***Nonloadbearing Wall***

These walls resist only in-plane loads from self-weight. They resist out-of-plane loads by out-of-plane flexure.

***Perpend joint***

Vertical cross joint that appears in the face of a wall between two masonry units

***Pier***

A member which forms an integral part of a wall, in the form of a thickened section placed at intervals along the wall

***Plain masonry***

Masonry that is neither grouted nor reinforced

***Quetta bond***

Masonry at least one and a half units thick in which vertical pockets containing reinforcement and mortar or concrete infill occur at intervals along its length

***Reinforced brickwork***

Brickwork that may be reinforced horizontally or vertically

***Reinforced masonry***

Masonry in which steel reinforcement is incorporated to enhance resistance to tensile, compressive or shear forces

***Single-Wythe Wall***

A single unit thick wall

***Slenderness ratio***

The ratio of the effective height or length to the effective thickness

***Special masonry***

Masonry which complies with the provisions of Chapter 4, 5 and 6 and which is tested during its construction in accordance with the requirements of Section 8.5 of EBCS-6.

***Structural units***

Bricks or blocks or square dressed natural stone

***Unreinforced masonry***

Masonry that is not reinforced

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## **1. INTRODUCTION**

### **1.1. Background**

As it is known, concrete- which is strong in compression and weak in tension- is commonly reinforced with steel to carry the tensile stresses. Similar principle can be applied to structural masonry to gain the benefit of extending its scope of application. Reinforced masonry is a construction system where steel reinforcement in the form of reinforcing bars or mesh is embedded in the mortar or placed in the holes and filled with concrete or grout. By reinforcing the masonry with steel reinforcement, the resistance to seismic loads and energy dissipation capacity can be improved significantly. In reinforced masonry, tension is developed in the masonry, but it is not considered to be effective in resisting design loads; reinforcement is assumed to resist all the tensile stresses.

In this thesis attention is given to the potentials of reinforced masonry structures, relative to reinforced concrete, not only because of the growing knowledge of the material and its composition, but also because of the architectural, performance and economic benefits that may prove superior to concrete. Also is the intention of this research to enforce the basic concepts involved with masonry structures and to inspire engineers to delve around this subject. And to assess the possible advantages that can be obtained from reinforced masonry as a major structural component in a building, by making analyses and designs according to limit state design philosophies.

### **1.2. Objective**

The general objective of this research is to access masonry building construction technology and possible economic benefits that can be derived from using reinforce masonry in lieu of reinforced concrete.

Specifically, the study is aimed at assessing the provisions of EBCS 6:1995 about reinforced masonry structures make analysis and design of sample buildings according to the philosophies of the captioned code and establish relative conclusions, by commenting on suitability, regarding the two construction alternatives. Based on the results, the study may also suggest future actions

to be taken, by any respective subject, to see into and further facilitate the use of the compared structural components appropriately.

Inductively, the thesis is aimed to inspire local universities so that their curriculum be revised and include subjects in areas of masonry as is being done to concrete.

### **1.3. What is in this thesis**

The present thesis paper investigates the modern concept of engineered reinforced masonry building construction so that benefits of better economy, durability, sound insulation, fire resistance, less construction time and low maintenance cost can be continued to be enjoyed in contemporary structures. The design provisions of this paper follow principles of limit state design, which is consistent with the design philosophy of EBCS and sample building structures of reinforced concrete are modeled, analyzed, designed and compared with those of reinforced masonry. Finally discussions, conclusions and recommendations are put forward in reference to the contents of the body of the thesis.

- Chapter one contains introduction where the background, the objective and the contents of the study are discussed.
- Chapter two addresses review of reinforced masonry construction technology commencing with the historical introduction to basic concepts behind the science.
- Chapter three reviews masonry construction provisions in Ethiopia and the Ethiopian Building code of practice for design of masonry structures.
- Chapter four discusses the design considerations and procedures of reinforced masonry and reinforced concrete building structures.
- Chapter five is devoted to cost evaluation and discussion of the designed buildings.
- In Chapter six, conclusions drawn from the study and proposed recommendations are presented.

The Appendix contains sample designs using reinforced masonry and bill of quantity for the reinforced concrete building.

## **2. REVIEW OF REINFORCED MASONRY CONSTRUCTION TECHNOLOGY**

### **2.1. Historical Development of Reinforced Masonry**

The use of reinforcement in masonry structures is not a new concept. The first recognized use of reinforced masonry was in 1813 when a British engineer Marc Brunel used reinforced brickwork to build a chimney for an industrial production plant near London. The development of reinforced masonry was halted in 1867, when the French gardener Joseph Monier introduced the concept of reinforced concrete. After that time, the use of reinforced masonry was put aside until the end of the First World War, when it was reintroduced in Canada, UK, USA and Australia. The more recent architect, Eladio Dieste – Argentina, built a lot of important structures in reinforced masonry and as such motivated the architects to use the full benefits of this construction technology. In the US, the most numerous examples of reinforced masonry construction are in California. These include such significant applications as warehouses, state buildings, in particular, high-rise hotel and apartment structures, low-rise commercial buildings and banks, churches, public and parochial schools. Today, the use of reinforced masonry is well developed in most western and Far East countries. Both in the US and Europe the adopted building codes have greatly increased the potential use.

Despite these historical precedents, the development of reinforced masonry has lagged far behind reinforced concrete. This is hardly surprising. There has been, until recently, relatively little research on the subject, hardly any technical papers, design guides, etc to assist the engineer, and practically no engineering student receives any instruction in the subject during his studies.

### **2.2. Methods of Reinforcing Masonry**

#### **2.2.1. Materials**

Fired bricks, concrete blocks and natural stone are used for the construction of masonry walls. In all cases the quality of masonry units should comply with the local national requirements with regard to materials and manufacture, dimensions and tolerances, mechanical strength, water absorption, frost resistance, soluble salts content, fire resistance, etc.

### 2.2.1.1. Clay masonry units (Bricks)

Clay masonry units primarily consist of clay, shale or similar naturally occurring earthy substances, water and additives. Most clay is composed mainly of silica and alumina of extremely small particle size formed by decomposition of rocks. Majority of the solid and hollow clay masonry units currently used in Ethiopia are produced by the “stiff-mud” process, also known as the “wire-cut” process. Clay masonry units that are acceptable for masonry construction are those complying with the requirements of ES C.D4.001. Material properties of clay masonry units which can affect their structural performance include: durability, initial rate of absorption, compressive strength and flexural strength. Some types of clay masonry units are shown in figure 1.

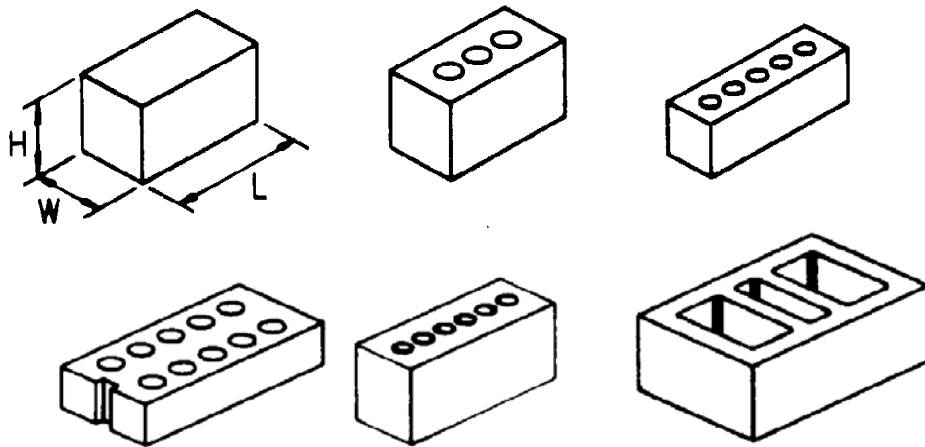
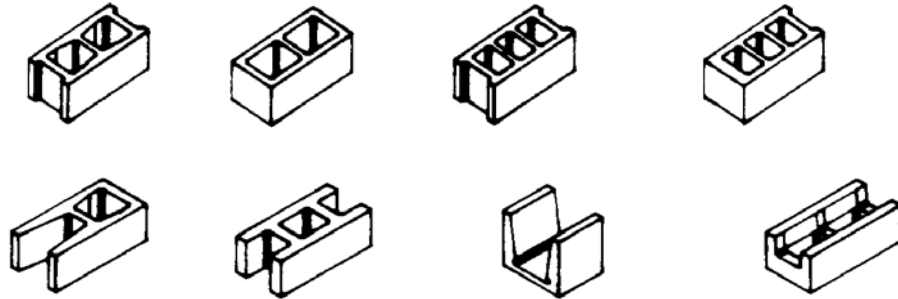


Figure 1: Clay Masonry Units

### 2.2.1.2. Concrete masonry units

Concrete masonry units are made from lightweight or normal weight aggregates, or both and primarily consist of Portland cement or blended cement, and water. Hydrated lime and/or pozzolans as well as air entraining agents may be used. Other ingredients that have been established as suitable for use in concrete such as coloring pigments, ground silica, etc., may also be used. There are three classes of masonry units; class A, class B, and class C. The structural requirements are the same for all classes. Concrete masonry units are cast using no-slump concrete. The mixture is placed into molds and vibrated under pressure for a specified time to

obtain compaction. The units are removed from the molds and may be cured under normal atmospheric conditions or by autoclaving (steam curing) [13]. Concrete masonry units are available in a wide variety of sizes and shapes as stipulated by ES C.D3.301 and some of these are shown in figure 2.



**Figure 2: Typical Concrete Masonry Units**

#### **2.2.1.3. Mortar**

Mortar, is a mixture of cementitious materials, aggregate and water. It serves to bond masonry units together to form a composite structural material. As such, mortar is a factor in the compressive, shear, and flexural strengths of the masonry assemblage. In addition, mortar compensates for dimensional and surface variations of masonry units, resists water and air penetration through masonry, maintains the sound and thermal characteristics of a wall and bonds to metal ties, anchors, and joint reinforcement so that they perform integrally with the masonry units [13]. Mortar may be specified in terms of proportions (by volume of Portland cement, hydrated lime, and aggregate) or in terms of properties (required compressive strength). The classification of mortar types given in EBCS 6 is based on volumetric proportions of Portland cement, lime and sand. This Ethiopian code designates the types of mortar as M1, M2, M3 and M4. Mortar type M3 and M4, being higher in strength, are recommended for use in reinforced masonry construction.

#### **2.2.1.4. Grout**

Grout, is a mixture of cementitious materials and aggregate to which sufficient water has been added to permit the grout to be readily poured into cavities created by masonry units without segregation of the materials. It bonds to the masonry units and to steel reinforcement, ties, and

anchors to form a unified composite structure. Grout is identified as fine or coarse depending on the maximum size of the aggregate used and in any case grout, for reinforced masonry as specified in EBCS 6, must possess a cement content not less than  $300 \text{ Kg/m}^3$  and is expected to attain minimum compressive strength of  $12 \text{ Mpa}$  as measured by a uniaxial compressive test in accordance with EBCS 6 provisions. Masonry units and grout interact in the same manner as unit-mortar interaction, that is, water is drawn from the grout into the masonry by suction. The final grout strength is a function of water content after suction [13].

#### **2.2.1.5. Reinforcing Steel**

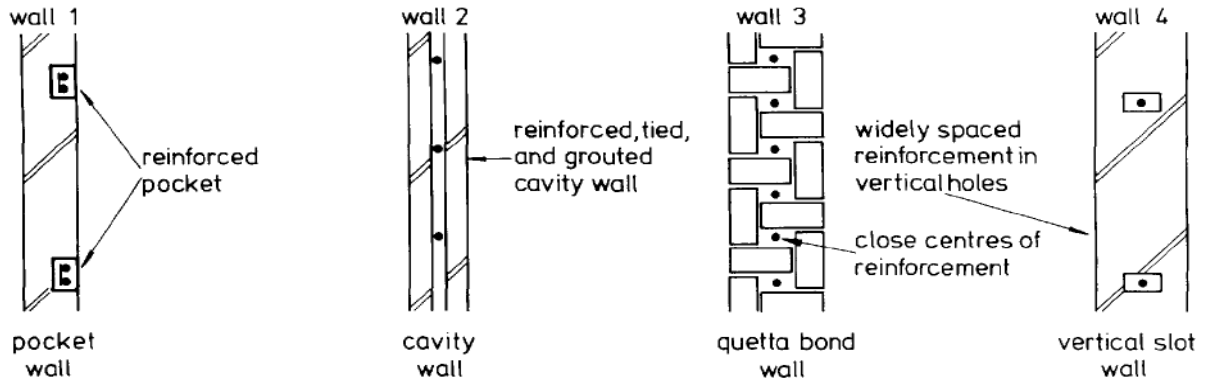
Masonry is reinforced with steel bars or joint reinforcement. Joint reinforcement, placed in mortar beds, is unique to masonry and is primarily used to resist internal forces due to shrinkage or thermally-induced movement [13]. Hot-rolled or cold-formed steel bars and fabric conforming to the relevant Ethiopian Standard can be used as reinforcement.

#### **2.2.2. Methods of Reinforcing Masonry**

Masonry, as stipulated previously has relatively low tensile strength. But this limitation of reduced field application is overcome by introducing reinforcing steel. Reinforcement may be introduced in one of the following ways: [2]

1. Placing Bars in Bed Joints or collar joints.
2. Embedding steel in pockets formed by suitable bonding patterns or using specially shaped units.
3. Placing steel in cavity formed by two leaves (or withes) of brickwork which is subsequently filled with small aggregate concrete or mortar. This is known as Grouted cavity construction.
4. Accommodating reinforcement in hollow block walls, provided that the design of the blocks permits the formation of continuous ducts for the reinforcing bars.

Figure 3 shows how masonry can be constructed with reinforcement incorporated in pockets in the face of the wall, in vertical holes inside the wall and in the voids of cavity construction).



**Figure 3: Methods of Reinforcing Masonry**

In the case of walls 2, 3 and 4, shuttering is not necessary and the void can be filled as the work proceeds. As far as simplicity is concerned, walls 2 and 4 are generally much easier and quicker to construct than walls 1 and 3 - wall 2 has the simplest bond. From the construction point of view, the Quetta bond wall is usually expensive and slow [1]. The simplest, most successful and economical method of filling voids is to fill as the work proceeds, and to make the masonry layers aware of the need for complete filling of the void to ensure adequate bond and protection against corrosion. As a good practice, complicated details should be avoided wherever possible, Shuttering should be reduced to a minimum, and Bonding should be kept simple [Curtin *et al*].

### 2.3. Design Consideration and Design of Reinforced Masonry Members

#### 2.3.1. Design Considerations

##### 2.3.1.1. General Requirements

In General Reinforced walls can be classified based on construction type, structural function or arrangement of walls. Referring to construction type walls may be categorized as: Single-Wythe Wall, Cavity Wall, Prefabricated Wall, Barrier Wall, and Masonry Veneer. Based on structural function masonry classification may be made as: Veneer Wall, Nonloadbearing Wall, Loadbearing Wall, and Shear Wall [6]. The great variety of possible wall arrangements in a masonry building makes it rather difficult to define distinct types of structure, but a rough classification might be made as: Cellular wall systems, Simple or double cross-wall systems and Complex arrangements [2]. The technical meaning of each of the above listed wall typologies is given in the “definitions of terms” section of this thesis.

The design and analysis of masonry building structures is an art as well as a science, requiring professional judgment as well as engineering principles. Decisions must be made about the type or types of masonry units to be used. Mortar, grout, reinforcement, and accessories must be selected. The structural configuration must be determined and serviceability considerations must be coordinated with structural requirements [6].

### 2.3.1.2. Limit State Design

At the design stage of a structure, it is not possible to predict accurately the loadings which will act on the structure during its planned life. Nor is it possible to define precisely the behavior of a structure under these loadings, or to predict with certainty the strength of the materials which combine to form the structure. It is necessary, therefore, for the engineer to introduce factors of safety when designing a structure in order to ensure that it will be satisfactory for its intended use. In determining the factors of safety to be used, it is necessary to first define the meaning of the word 'satisfactory' when related to the use of the structure [1].

Obviously, one criterion for the design of the building is that it should not collapse, but there are other conditions which need to be examined. The structure should be readily usable by the occupants, as well as visually acceptable. It should not excessively deflect or crack, and should be sufficiently durable to maintain its initial condition. There are other criteria which may be applicable to certain structures including, for example, resistance to fire, explosion, impact and vibration. [1]

Limit state design is an attempt to consider each item more closely so as to enable a more accurate factor of safety to be applied in the design, and depends upon the case being considered. This is achieved by breaking down the overall factor of safety used in the design into its various components, and then placing a specific factor – known as a 'partial factor of safety' - on that component for a given condition. It is thus possible to build up an overall, or 'global', factor of safety from these individual factors. Such load factors are prescribed by national documents such as EBCS and may be divided into two groups: the first group consists of the factors of safety to be applied to the materials and workmanship used in construction and the second group, the factors of safety to be applied to loads in the overall structure and the consequences of failure.

A Limit state may be one of complete failure (Ultimate Limit State or ULS) or it may define a condition of excessive deflection or cracking (Serviceability Limit State). Limit state approach

permits the definition of direct criteria for strength and serviceability taking into account the uncertainties of loading, strength and structural analysis as well as questions such as the consequences of failure.

The Application of the Limit State Design approach as exemplified by the Ethiopian building code standard is discussed in chapter three of this thesis under the subtopic that states about EBCS 6 provisions.

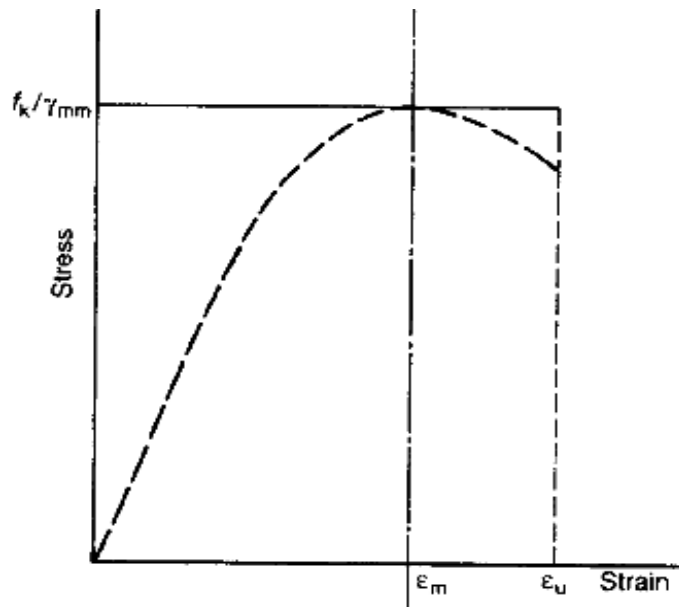
### 2.3.2. Design of Reinforced Masonry Beams

#### 2.3.2.1. Flexural Strength

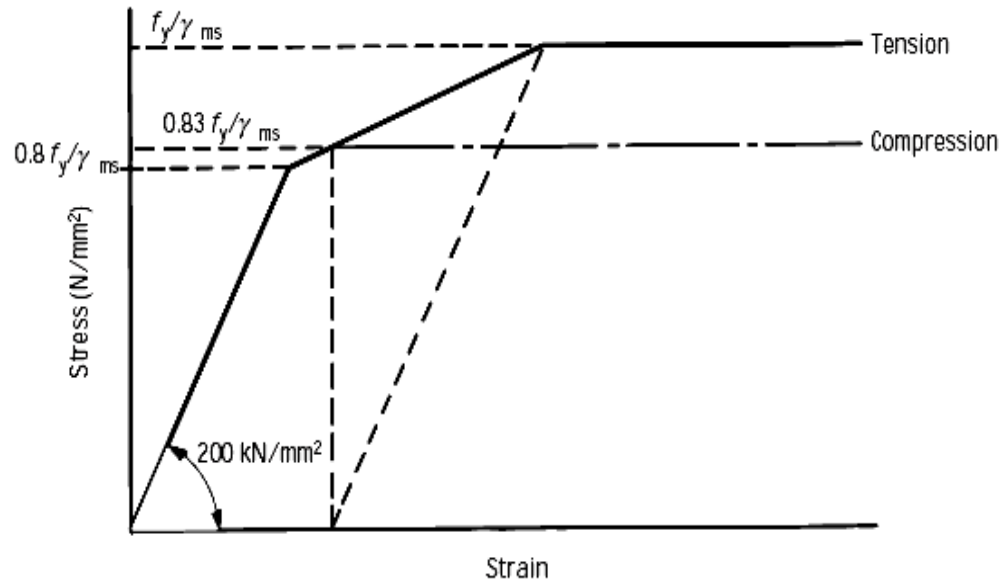
##### Assumptions: Stress-Strain Relationship

In order to develop design equations for elements subject to bending it is necessary to assume ideal stress-strain relationships for both the masonry and the reinforcement.

As far as the masonry is concerned the approximate parabolic distribution may be further simplified to a rectangular distribution in which the stress is assumed to be constant and equal to  $f_k / \gamma_{mm}$ , (Fig. 4a).



(a) Idealized Stress-Strain relationship for Masonry under compression



**(b) Idealized Stress-Strain relationship for Reinforcement**

**Figure 4: Stress-Strain Relationships**

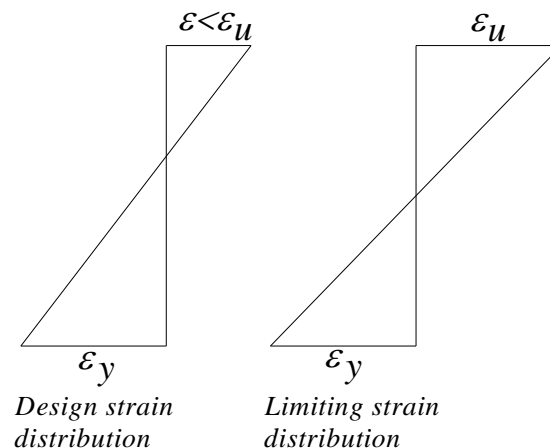
As far as the steel is concerned the relationship is assumed to be as shown in Fig. 4b where  $f_y$ , the characteristic tensile strength of the reinforcement, is assumed to be  $250 \text{ N/mm}^2$  for hot-rolled deformed mild steel and  $460 \text{ N/mm}^2$  for hot-rolled high-yield, cold-worked steel and stainless-steel bars.

#### **Additional assumptions and Limitations**

In addition to the idealization of the stress-strain relationships further assumptions are introduced as follows:

1. Plane sections remain plane after bending.
2. The tensile strength of the masonry is ignored.
3. The effective span of simply supported or continuous members is taken as the smaller of:
  - (i) The distance between support centers and,
  - (ii) The clear distance between supports plus the effective depth.
4. The effective span of cantilevers is taken as the smaller of:
  - (i) The distance between the end of the cantilever and the centre of its support and,
  - (ii) The distance between the end of the cantilever and the face of the support plus half its effective depth.

5. The ratio of span to effective depth is not less than 1.5 otherwise the beam would have to be designed as a deep beam and the basic equations would not be applicable.
6. The strains in both materials are directly proportional to the distances from the neutral axis.
7. The section is under-reinforced so that the strain in the reinforcement reaches the yield value  $\epsilon_y$  whilst the maximum strain in the masonry is still below the ultimate value  $\epsilon_u$ . (A limiting strain distribution can be defined in which the reinforcement is at  $\epsilon_y$  and the masonry at  $\epsilon_u$  (Fig. 5).
8. Although design is based on the ultimate limit state, recommendations are included in the codes of practice to ensure that the serviceability states of deflection and cracking are not reached. These recommendations are given as limiting ratios of span to effective depth.
9. To ensure lateral stability beams should be proportioned so that (i) for simply supported or continuous beams the distance between lateral restraints does not exceed the lesser of  $60b_c$  and  $250b_c^2/d$ , and (ii) for cantilevers the distance between the end and the support does not exceed the lesser of  $25b_c$  and  $100b_c^2/d$ , where  $b_c$  is the width of the compression face midway between restraints and  $d$  is the effective depth.



**Figure 5: Strain Distributions**

### Design equations

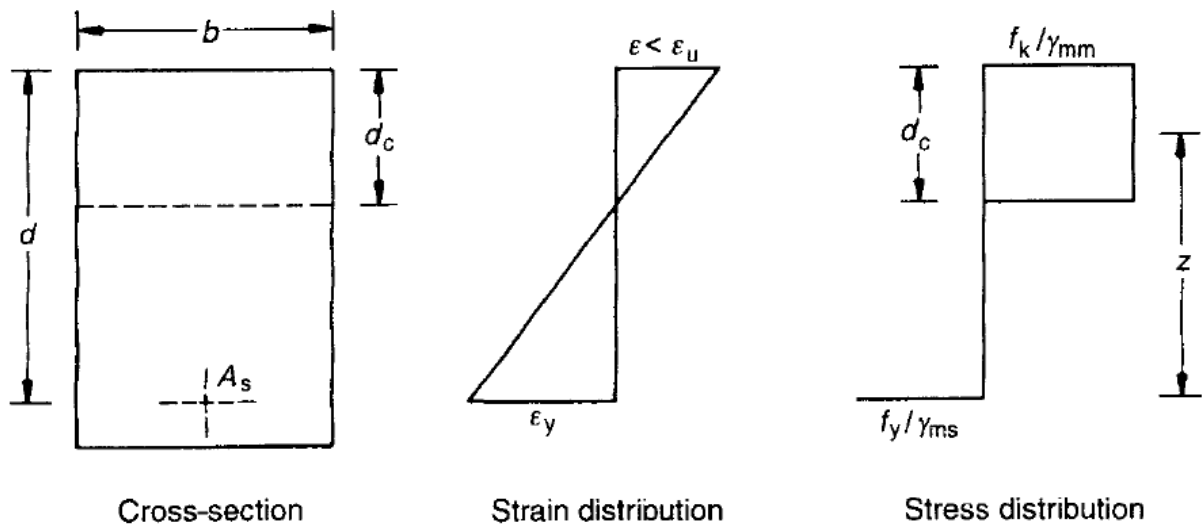
In figure 6 the strain distribution shows that the steel has reached yield strain and the maximum masonry strain is less than the ultimate value (assumption 7). Also the stress in the compressive

zone is constant at  $f_k/\gamma_{mm}$  (stress-strain relationship for masonry). Taking moments about the centroid of the compression block gives the design moment of resistance  $M_d$ .

$$M_d = \frac{A_s f_y z}{\gamma_{ms}} \leq \frac{0.4 f_k b d^2}{\gamma_{mm}} \dots [2] \quad (2.1)$$

$$\text{And } z = d \left( 1 - 0.5 \frac{A_s f_y \gamma_{mm}}{b d f_k \gamma_{ms}} \right) \leq 0.95 d \dots [2] \quad (2.2)$$

Since equations 2.1 and 2.2 both contain the variables  $A_s$  and  $z$  they must be solved using an iterative procedure to determine the area of steel for a given design moment.

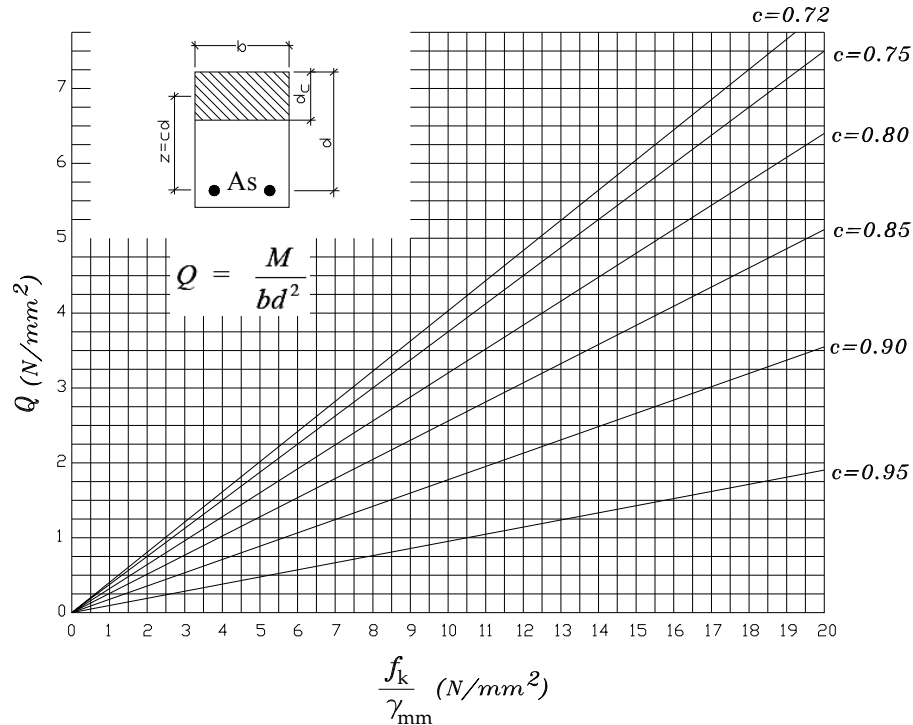


**Figure 6: Strain and Stress Distribution in Section**

An alternative solution to using equations given for bending moment of resistance is to use the design table given in BS 5628 as presented here as Figure 7. This design table gives values of moment of resistance factor,  $Q$ , for various values of  $f_k/\gamma_{mm}$  and lever-arm factor  $c=z/d$  and is used to determine the area of steel required.

$$M_d = Q * b * d^2 \text{ or } Q = \frac{M}{b d^2} \quad (2.3)$$

$$Q = 2 * c * (1 - c) * \frac{f_k}{\gamma_{mm}} \quad (2.4)$$



**Figure 7: Moment of Resistance Factor, Q**

### 2.3.2.2. Shear Strength of Reinforced Masonry Beams

As in reinforced concrete beams, shear transmission across a crack in a reinforced masonry beam can take place by one or more of the following mechanisms:

- Compression zone transmission resulting from the shear resistance of the masonry.
- 'Aggregate interlock' by frictional forces across the crack.
- 'Dowel effect' from the shear force developed by the reinforcing bars crossing the crack.

It is necessary to ensure that the shear stress, given by  $\frac{V}{bd}$ , in a beam does not exceed the design

shear strength of the material, given by  $\frac{f_v}{\gamma_{mv}}$ .

The Spacing of shear reinforcement whenever required can be computed from but not exceeding 0.75d:

$$\frac{A_{sv}}{S_v} \geq b \left[ v - \frac{f_v}{\gamma_{mv}} \right] \frac{\gamma_{ms}}{f_y} \dots [2] \quad (2.5)$$

Where  $v$  is shear stress due to design loads and not exceeding  $2.0/\gamma_{mv}$

### 2.3.2.3. Deflection of Reinforced Masonry Beams

The deflection of a reinforced masonry beam can be calculated in a similar way to that of a reinforced concrete beam with suitable adjustments for different material properties. Experiment has shown that the following moment-curvature relationship can be assumed:

$$\theta = \frac{M}{EI_u} + \frac{M - M_{cr}}{0.85EI_{cr}} \dots [2] \quad (2.6)$$

where  $M$  is the applied moment,  $EI_u$  is the flexural rigidity of the transformed uncracked section,  $EI_{cr}$  is the flexural rigidity of the transformed cracked section,  $M_{cr} = I_{cr} f_t / (H - d_c)$  is the cracking moment,  $f_t$  is the apparent flexural tensile strength of the masonry (or composite brick/concrete in a grouted cavity beam),  $H$  is the overall depth of the section and  $d_c$  is the neutral axis depth.

### 2.3.3. Design of Reinforced Masonry Columns

Elements such as columns, which are subjected to both vertical loading and bending, are classified as either short or slender and different equations are used for the design of the two classes. Additionally bending may be about one or two axes so that a number of cases can be identified. [2]

The following assumptions are used in design of Reinforced masonry columns in the Limit State Design philosophy.

1. Tensile Strength of masonry is ignored
2. Plane sections remain plane after bending
3. The strains in both materials are directly proportional to the distance from the neutral axis.
4. The effective height and thickness are taken as follows:

- Effective Height of a column with lateral supports in two directions:

$$h_{ef} = h \quad (2.7)$$

- Effective Height of a column with lateral supports in one direction:

$$h_{ef} = h, \text{ In lateral support direction} \quad (2.8)$$

$$h_{ef} = 2h, \text{ In direction in which support is not provided} \quad (2.9)$$

- Effective Height of Columns formed by adjacent openings in walls
  - Enhanced Resistance

$$h_{ef} = 0.75h + 0.25 * (\text{Height of the taller of the two openings}) \quad (2.10)$$

- Simple Resistance

$$h_{ef} = h \quad (2.11)$$

- Effective thickness of columns is taken as the actual thickness of the column.

5. The maximum Strain in the outermost compression fiber at failure is taken as 0.0035.

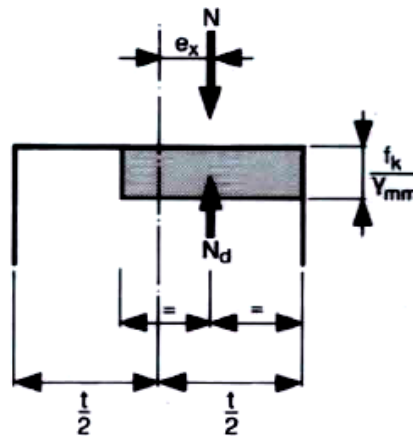
### 2.3.3.1. Short Columns under Uniaxial Bending

In the British code, BS5628, short columns are defined as those with a slenderness ratio of less than 12 and three different cases, outlined below, can be assumed in the design of short columns.

**Case a:**  $N < N_d$

Where: N = Design Axial Load

$$N_d = \text{Design Axial Load Resistance}; N_d = \frac{f_k * b * (t - 2e_x)}{\gamma_{mm}} \quad (2.12)$$



**Figure 8: Stress block-short column minimum reinforcement case**

$$e_x = \text{Resultant eccentricity}; e_x = \frac{M}{N} \quad (2.13)$$

If a short column falls in this category, nominal reinforcement is sufficient.

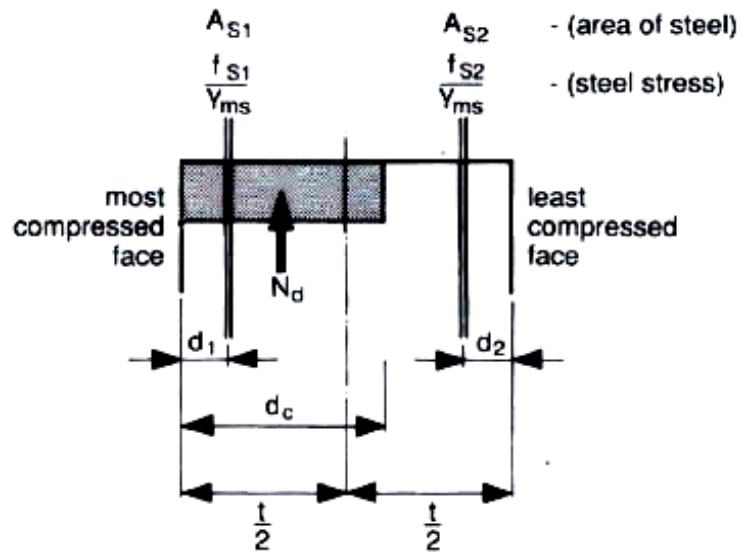
**Case b:**  $N > N_d$  (of case a)

The solution of equations 2.14 and 2.15 requires an assumption of the depth of the compressive stress block  $d_c$ , so that the steel stresses can be obtained. This can be done on the basis of the stress strain curves for reinforcement illustrated in Figure 9. Design is thus an iterative process

involving assumptions of  $d_c$ , and estimation of steel stresses which give load and moment capacities which exceed the design values.

$$N_d = \frac{f_k * b * d_c}{\gamma_{mm}} + \frac{0.83 * f_y * A_{s1}}{\gamma_{ms}} - \frac{f_k * A_{s2}}{\gamma_{ms}} \quad (2.14)$$

$$M_d = \frac{0.5 f_k * b * d_c (t - d_c)}{\gamma_{mm}} + \frac{0.83 * f_y * A_{s1} * (0.5 t_1 - d_1)}{\gamma_{ms}} + \frac{f_k * A_{s2} * (0.5 t - d_2)}{\gamma_{ms}} \quad (2.15)$$



**Figure 9: Stress block-short column design reinforcement case**

The values of  $N_d$  and  $M_d$  calculated from these equations must be greater than  $N$  and  $M$ , the applied Axial Load and Moment. Trial Sections and Areas of Reinforcement are first assumed and then  $f_{s2}$  is determined from an assumed value of  $d_c$  as follows.

It is always true that  $d_c > 2d_1$  is assumed and in addition:

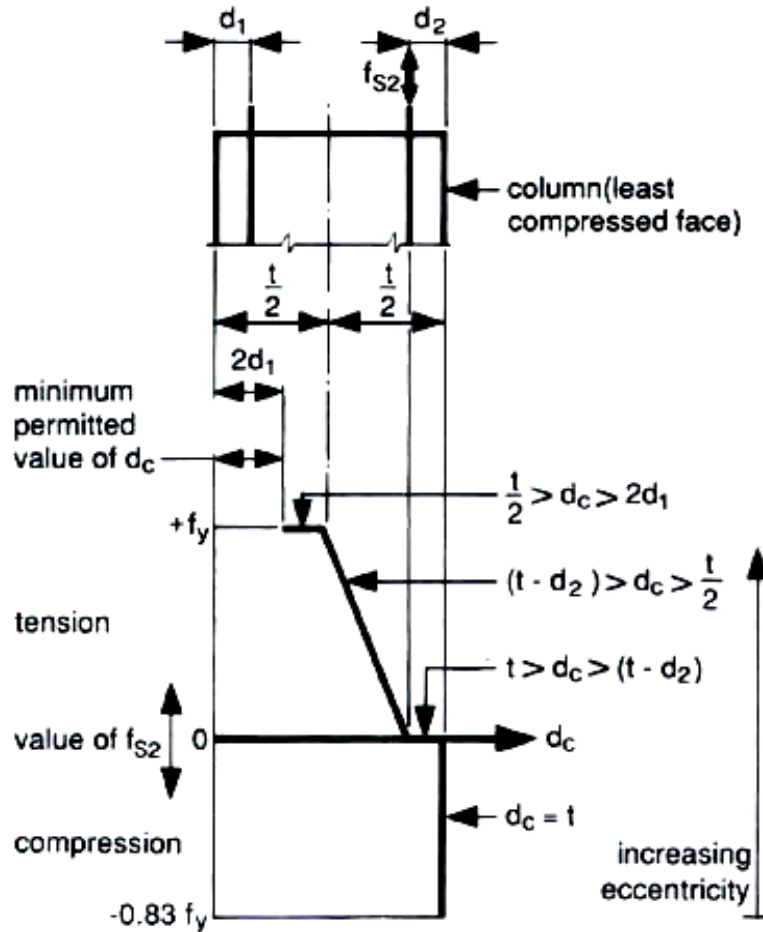
- If  $2d_1 \leq d_c \leq \frac{t}{2}$  is assumed take  $f_{s2}$  from equation 2.14

$$f_{s2} = f_y \quad (2.16)$$

- If  $\frac{t}{2} < d_c \leq (t - d_2)$  is assumed calculate  $f_{s2}$  from equation 2.15

$$f_{s2} = 2f_y \frac{(t - d_2 - d_c)}{(t - 2d_2)} \quad (2.17)$$

- If  $(t - d_2) < d_c \leq t$  is assumed take  $f_{s2}$  equal to zero.



**Figure 10: Variation of depth of compression block,  $d_c$  with stress in reinforcement in least compressed face of column,  $f'_{s2}$**

This method is tiresome and Interaction Diagrams, in which  $\frac{M}{bt^2 f_k}$  is plotted against  $\frac{N}{bt^2 f_k}$  for a range of values of  $f_k$ , and are available for different values of the ratio  $d/t$  and  $f_y$ , for more direct solution of the equations.

**Case c:** Is considered when the eccentricity  $\frac{M}{N} > (\frac{t}{2} - d_1)$ , and in this case the Axial load is ignored and the section is designed to resist an increased moment given by:

$$M_a = M + N\left(\frac{t}{2} - d_1\right) \quad (2.18)$$

### 2.3.3.2. Short columns under Biaxial Bending

BS 5628, gives a method to deal with the biaxial case of a symmetrically reinforced section (of dimension  $p$  by  $q$ ) by increasing the moment about one of the axes, the axis being that which has the greater ratio of the design moment about the axis to the column's overall dimension perpendicular to the axis, in accordance with the following formulae taking the design axial load resistance,  $N_{dz} = f_k A_m$ , for the complete section ( $A_m$ ) and ignoring all bending and the value for the bending moment coefficient  $\alpha$  is read from the table 2.1.

$$M'_x = M_x + \alpha \left(\frac{p}{q}\right) M_y \quad \text{For } \frac{M_x}{p} > \frac{M_y}{q} \quad (2.19)$$

$$M'_y = M_y + \alpha \left(\frac{q}{p}\right) M_x \quad \text{For } \frac{M_x}{p} < \frac{M_y}{q} \quad (2.20)$$

**Table 1: values of  $\alpha$  for biaxial Bending of Short columns**

$\frac{N}{N_{dz}}$	0	0.1	0.2	0.3	0.4	0.5	$\geq 0.6$
$\alpha$	1.00	0.88	0.77	0.65	0.53	0.42	0.3

### 2.3.3.3. Slender columns

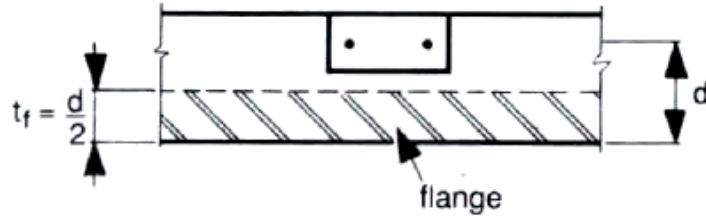
Slender columns as defined by BS 5628 are columns that have slenderness ratios between 12 and 27 and such columns may have additional horizontal deflections due to the vertical loading. The moment due to this additional eccentricity is given by equation 2.19 and uniaxial bending slender columns can be designed by using the method for short columns by allowing for the additional moment  $M_a$ .

$$M_a = \frac{N(h^2_{ef})}{2000t} \quad (2.21)$$

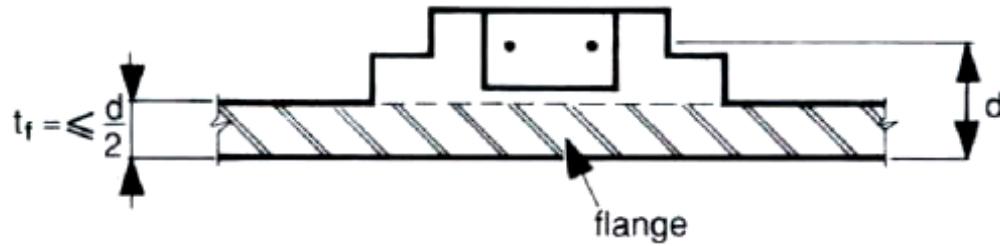
## 2.3.4. Design of Reinforced Masonry Walls

### 2.3.4.1. Walls in Flexure in which reinforcement is concentrated locally

These groups of walls include two cases, namely flanged members (with reinforcement concentrated locally in ribs or pockets) and reinforced block work where only intermittent holes are reinforced. In the former case, i.e. flanged members, the thickness of the flange is taken as in figure 11.



(a) Pocket within wall thickness

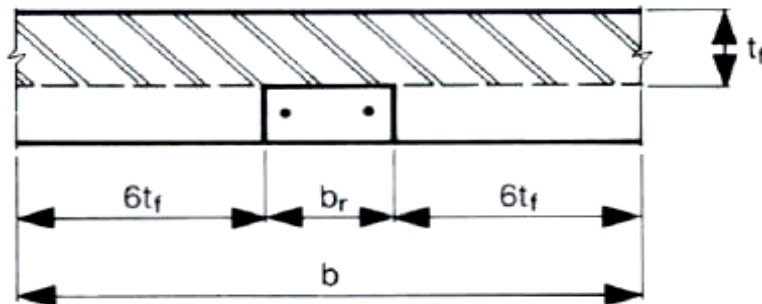


(b) pocket projects behind wall

**Figure 11: Pocket Reinforcement in walls**

The width of the flange (see Figure 12) is taken as the least of:

- Width of pocket or rib + 12\* flange thickness, or
- The spacing of pockets or ribs, or
- One third the height of the wall



$$b = \text{lesser of: i) } 12t_f + b_r, \text{ ii) spacing of pockets, iii) } \frac{\text{height of wall}}{3}$$

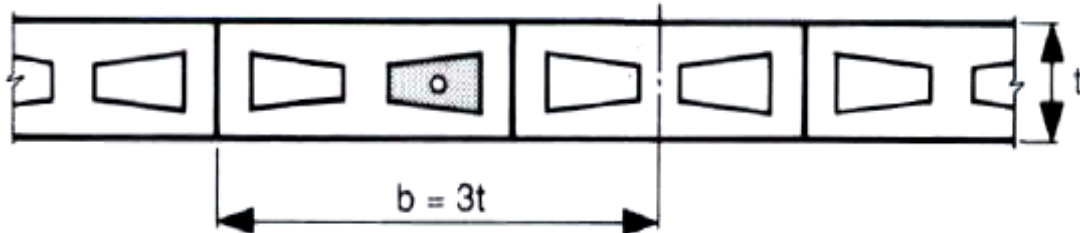
**Figure 12: Flange width of pocket type walls**

The design moment of resistance of the section,  $M_d$ , is obtained from the following formula as:

$$M_d = \frac{A_s f_y z}{\gamma_{ms}} \leq \frac{f_k}{\gamma_{mm}} b t_f (d - 0.5 t_f) \quad (2.22)$$

$$\text{And } z = d \left( 1 - 0.5 \frac{A_s f_y \gamma_{mm}}{b d f_k \gamma_{ms}} \right) \quad (2.23)$$

When the reinforcement in a section is concentrated locally such that the section cannot act as a flanged member, the reinforced section should be considered as having a width of three times the thickness of the block work as shown in figure 13.



**Figure 13: Effective width, reinforced hollow section**

The design shear stress,  $v$ , is calculated as the average shear stress on the section, i.e.  $v = \frac{V}{bd}$ , for flanged beams and locally reinforced hollow block work the area considered to resist shear for design purpose is the area given by  $b \cdot d$ .

If  $v$  is less than the characteristic shear strength of masonry divided by  $\gamma_{mv}$ . (i.e.  $\frac{f_v}{\gamma_{mv}}$ ), shear reinforcement is not generally needed. If  $\frac{f_v}{\gamma_{mv}}$  is exceeded links should be provided to resist

the excess shear stress such that  $\frac{A_{sv}}{S_v} \geq \frac{b(v - \frac{f_v}{\gamma_{mv}})\gamma_{ms}}{f_y}$  where  $A_{sv}$  is the cross sectional area of the shear reinforcement and  $S_v$  is its spacing.

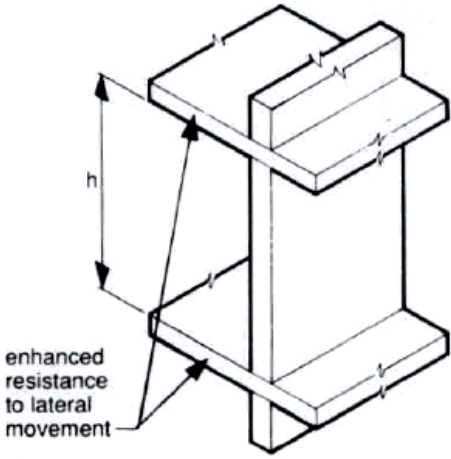
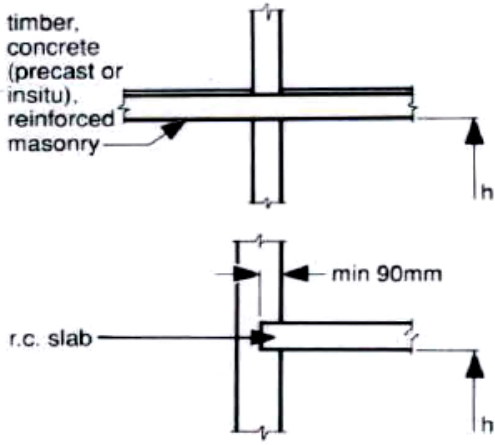
#### 2.3.4.2. Reinforced masonry walls subjected to a combination of Vertical loading and bending

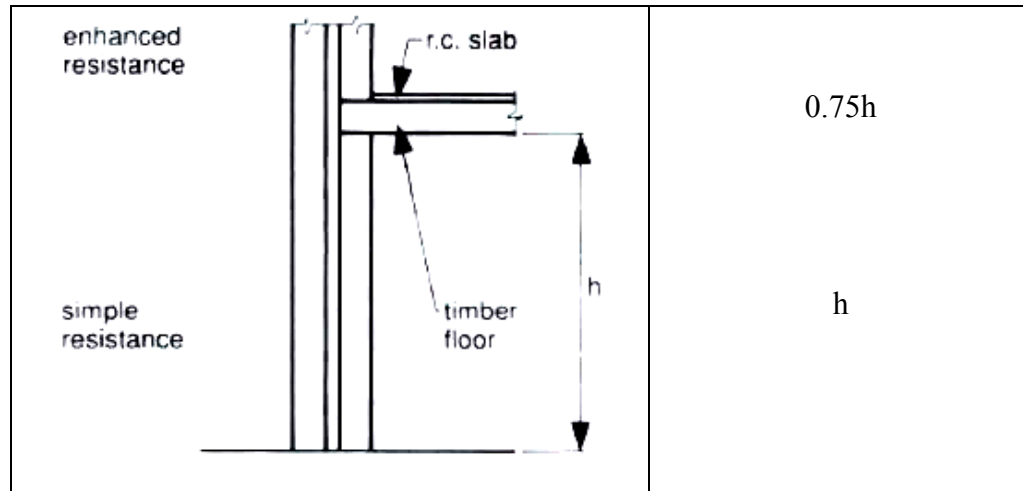
These groups include walls which carry substantial vertical loads and are required to resist bending moments arising from the eccentricity of the vertical loads or/and from lateral loading, such that the resultant eccentricity exceeds 0.05 times the member thickness in the direction of the eccentricity. The calculation of the value for eccentricity will be explained in subsequent sections of this thesis.

### i. Effective Heights of walls

The British standard relates the effective height to the degree of lateral resistance to movement provided by supports. This degree of lateral resistance may be distinguished between two types. The term enhanced resistance is intended to imply that there is some degree of rotational restraint at the end of the member. The conventional values of effective heights of walls recommended in BS 5628 are presented in table 2.

**Table 2: Effective Heights of walls**

Walls	$h_{ef}$
	0.75h
	0.75h



## ii. Effective thickness of walls

The effective thickness of single leaf wall is usually taken as the actual thickness. For a single leaf wall stiffened with piers as shown in figure 14a BS 5628 gives a factor  $K$  to get a wall of equivalent thickness shown in figure 14b and these values of  $K$  are presented in table 3.

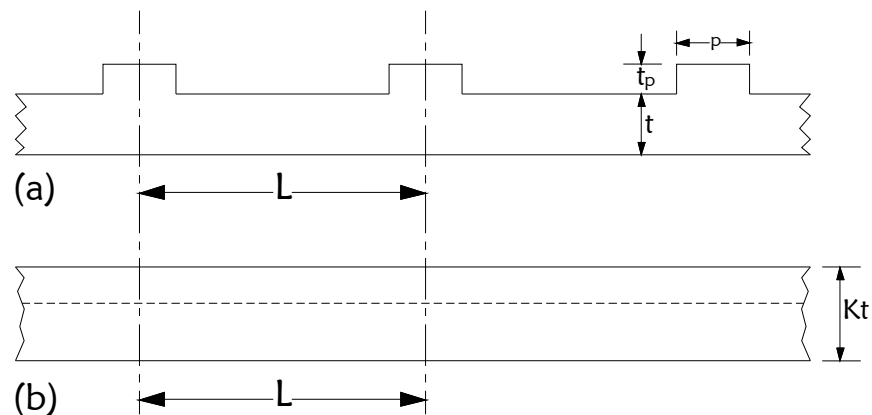


Figure 14: single leaf wall with piers

Table 3: Stiffness coefficient,  $K$ , values for effective thickness of walls stiffened by piers

$\frac{L}{P}$	$t_p / t$		
	1	2	3
6	1	1.4	2
10	1	1.2	1.4
20	1	1.0	1.0

The effective thickness for cavity walls is taken as the greater value of two thirds the sum of the actual thickness of the two leaves or the actual thickness of the thicker leaf. For the case of a cavity wall with piers (see figure 15) the factor  $K$  given in table 3 is referred and the effective thickness is taken as the greater value of  $\frac{2}{3}(t_1 + Kt_2)$ ,  $t_1$  and  $Kt_2$ .

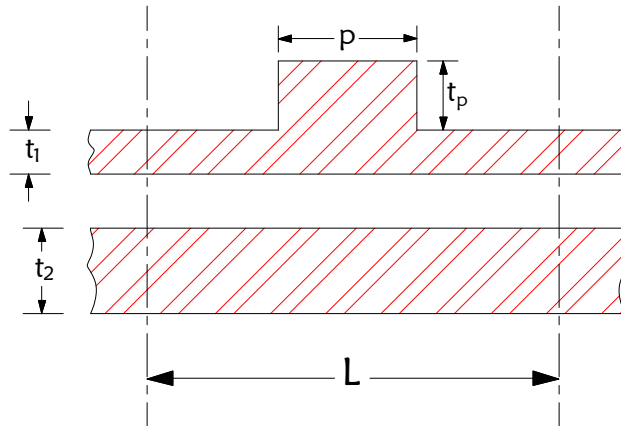


Figure 15: cavity wall with piers

### iii. Calculation of Eccentricity

BS 5628 gives a satisfactory approximate method to calculate the values of eccentricity,  $e_x$ , and this method, as compiled by *Hendry et.al*, is discussed in the following paragraphs.

- The load transmitted by a single floor is assumed to act at one-third of the depth of the bearing areas from the face of the wall (see Figure 16).
- For a continuous floor, the load from each side is assumed to act at one-sixth of the thickness of the appropriate face (see Figure 17).
- If the applied vertical load acts between the centroid of the two leaves of a cavity wall it should be replaced by statically equivalent axial loads in the two leaves.

An additional eccentricity,  $e_a$  that results from the lateral deflection related to slenderness is considered for walls with slenderness ratios of 6 and above. This additional eccentricity is given by an empirical relationship:

$$e_a = t[(1/2400)(h_{ef}/t)^2 - 0.015] \quad (2.24)$$

The total eccentricity is then computed from:

$$e_t = 0.6e_x + e_a \tag{2.25}$$

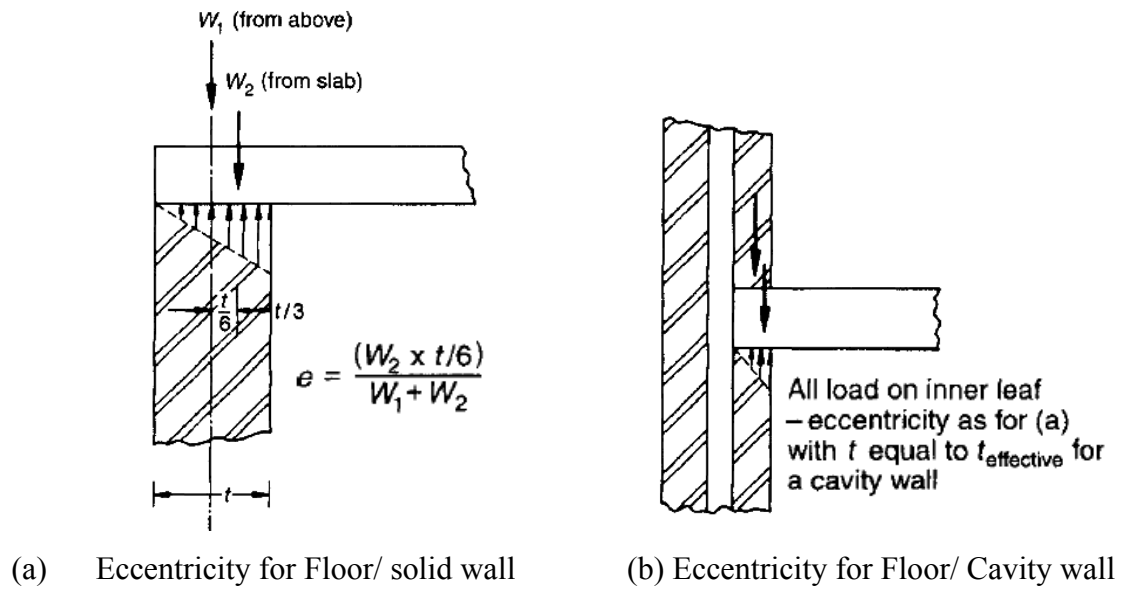


Figure 16: Load on one wall only

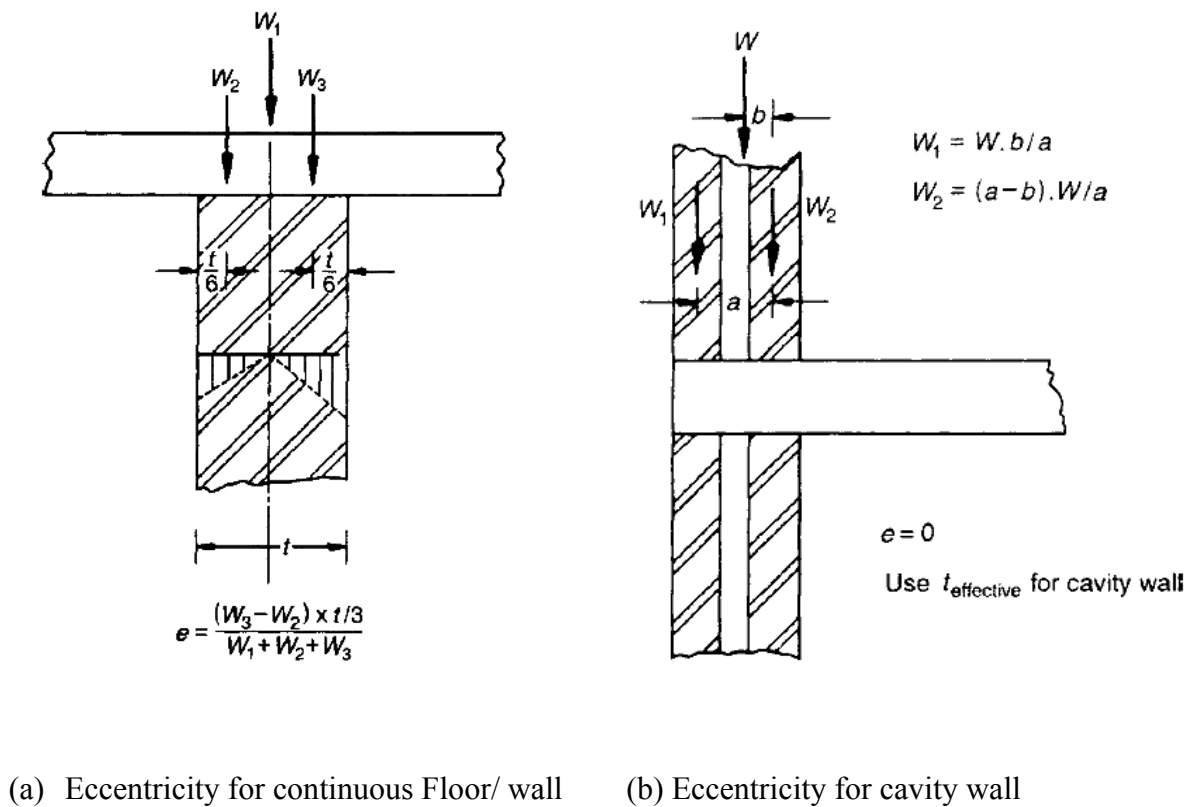
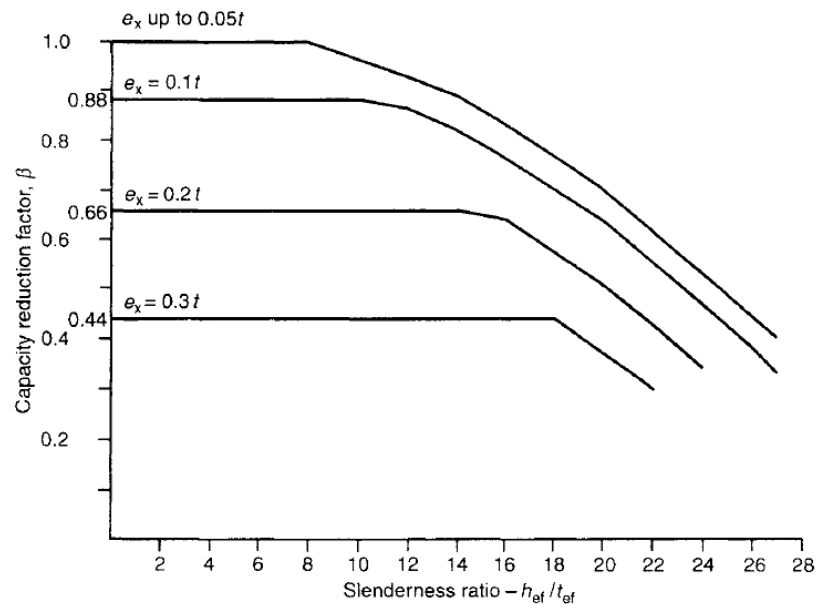


Figure 17: Continuous floor Loading

### 2.3.4.3. Vertical load Resistance

The design vertical load resistance per unit length of wall is given in BS5628 as  $\frac{\beta t f_k}{\gamma_m}$  where  $\gamma_m$  is the partial safety factor for the material and the value for the capacity reduction factor,  $\beta$  is obtained from figure 18.



**Figure 18: Capacity Reduction factor,  $\beta$**

The procedure for calculating the design vertical load resistance in BS 5628 can be summarized as follows:

1. Determine  $e_x$  at the top of the wall using the method illustrated in Figs 16 and 17.
2. Determine  $e_a$ , the additional eccentricity, using equation (2.24) and the total eccentricity  $e_t$  using equation (2.25).
3. If  $e_x > e_t$  then  $e_x$  governs the design. If  $e_t > e_x$  then  $e_t$  (the eccentricity at mid-height) governs.
4. Taking  $e_m$  to represent the larger value of  $e_x$  and  $e_t$ , then if  $e_m$  is  $\leq 0.05t$  the design load

resistance is given by  $\frac{\beta t f_k}{\gamma_m}$ , with  $\beta=1$ , and if  $e_m > 0.05t$  the design load resistance is given by

$$\frac{\beta t f_k}{\gamma_m} \text{ with } \beta = 1.1(1 - 2e_m/t)$$

The design vertical load resistance for cavity walls can be determined using the method described above if the vertical loading is first replaced by the statically equivalent axial load on each leaf.

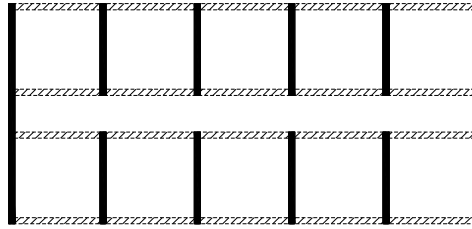
The effective thickness of the cavity wall or column is used for determining the slenderness ratio for each leaf of the cavity. [Hendry *et.al.*, 1997]

The design process for vertical loading is completed by equating the design vertical loading to the appropriate design vertical load resistance and using the resulting equation to determine the value of the characteristic compressive strength of the masonry  $f_k$ . The value of  $f_k$  obtained shall be modified to allow for the effects of small plan area and narrow masonry walls as the following recommendation by BS 5628:

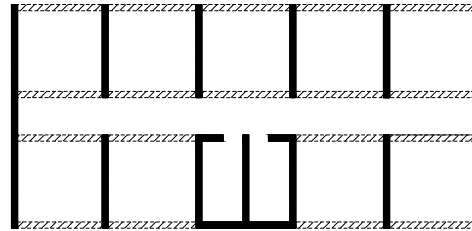
- If the horizontal cross-sectional area ( $A$ ) is less than  $0.2 \text{ m}^2$  then the value of  $f_k$  is divided by a factor  $(0.7+1.5A)$ .
- If the thickness of the wall is equal to the width of the masonry then the value of  $f_k$  is divided by 1.15.

### 2.3.5. Robustness and Stability

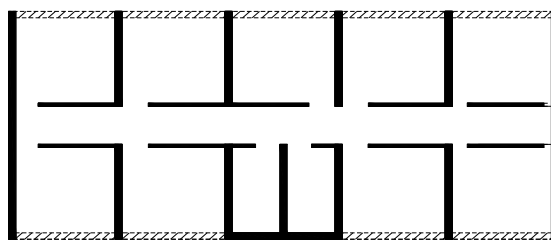
Considerable attention has been devoted in recent years to the necessity for ensuring the ‘robustness’ of buildings. This has arisen from a number of building failures in which, although the individual members have been adequate in terms of resisting their normal service loads, the building as a whole has still suffered severe damage from abnormal loading. It is impossible to quantify loads of this kind, and what is required is to construct buildings in such a way that an incident of this category does not result in catastrophic collapse, out of proportion to the initial forces. Meeting this requirement begins with the selection of wall layouts resistant to abnormal forces than others. This point is illustrated in figure 19a building consisting only of floor slabs and cross-walls (Fig. 19a) is obviously unstable and liable to collapse under the influence of small lateral forces acting parallel to its longer axis. This particular weakness could be removed by incorporating a lift shaft or stair well to provide resistance in the weak direction, as in figure 19b. However, the flank or gable walls are still vulnerable, for example to vehicle impact, and limited damage to this wall on the lowermost storey would result in the collapse of a large section of the building. [Hendry *et.al.*, 1997]



a) Cross walls without Longitudinal walls: Unstable



b) Cross Walls with Service Shaft: Normally Stable but Vulnerable to Accidental damage



— Structural Walls  
 // Non Structural Walls

c) Cross Walls with Longitudinal Walls and Service shaft: Robust construction

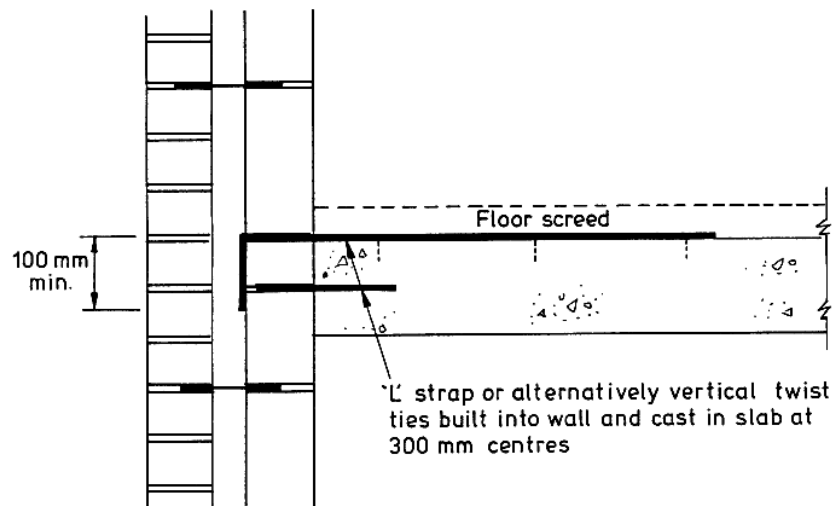
### Figure 19: Liability of a simple cross-wall structure to accidental damage

A building having a wall layout as in figure 19c on the other hand is clearly much more resistant to most kinds of disturbing forces, having a high degree of lateral stability, and is unlikely to suffer extensive damage from failure of any particular wall. Robustness is not, however, purely a matter of wall layout. Thus a floor system consisting of unconnected precast planks will be much less resistant to damage than one which has cast-in-situ concrete floors with two-way reinforcement. Similarly, the detailing of elements and their connections is of great importance. For example, adequate bearing of beams and slabs on walls is essential in a gravity structure to prevent possible failure not only from local over-stressing but also from relative movement between walls and other elements. Such movement could result from foundation settlement, thermal or moisture movements. An extreme case occurs in seismic areas where positive tying together of walls and floors is essential. The above discussion relates to multi-storey, load

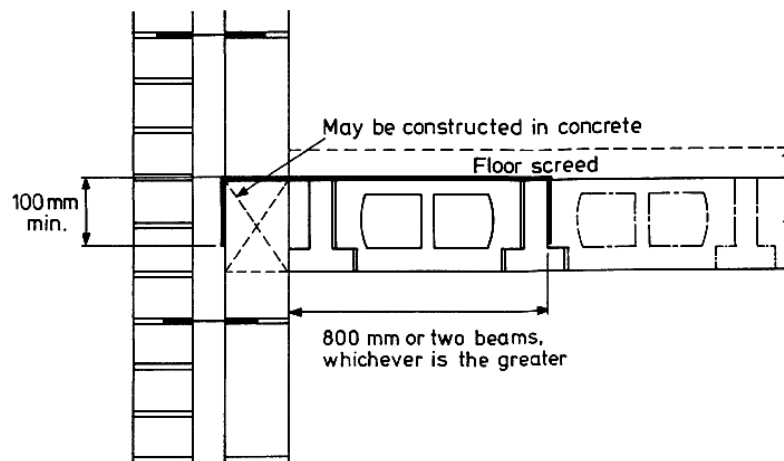
bearing masonry buildings, but similar considerations apply to low-rise buildings where there is the same requirement for essentially robust construction. [2]

Generally, to ensure a robust and stable design it will be necessary to consider the layout of structure on plan, returns at the ends of walls, interaction between intersecting walls and the interaction between masonry walls and the other parts of the structure.

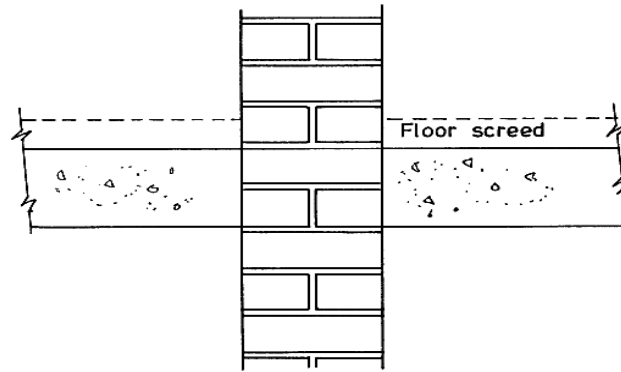
The British standard gives a way of satisfying stability and robustness by checking the safety of the building after a uniformly distributed lateral load equal to 1.5% of the total characteristic dead load above any level and connections that resist lateral movement as those shown in figure 20 are provided.



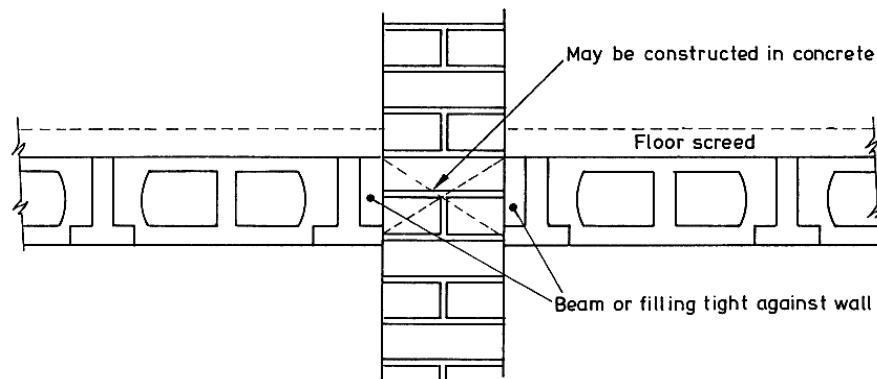
**(a) In-situ concrete floors abutting external cavity wall**



**(b) Beam and pot floors abutting external cavity wall**



(c) In-situ floors abutting internal wall



(d) Beam and pot floors abutting internal wall

Figure 20: Connections that resist lateral movement

## 2.4. Construction of Reinforced Masonry and Reinforced concrete buildings

### 2.4.1. Reinforced Masonry buildings

In the preceding sections a general description of reinforced masonry work has been given and it has been claimed that it is easier and simpler to construct than reinforced concrete work. It is now proposed to give, in as concise a manner as possible, the main points which require careful attention during construction, and to offer suggestions which are likely to be found of practical use in carrying out work.

The main requisites of good reinforced masonry work are:

- I. Good materials,
- II. Careful work.

### 2.4.1.1. Materials

a) Units (Bricks or concrete blocks): Only the best units complying with the usual 1<sup>st</sup> class specifications should be used. Hardness is a desirable quality, but brittleness is not, while anything approaching a smooth glaze on the surface, such as is sometimes observed in over-burnt bricks, is also undesirable as mortar will not adhere well to bricks with such a surface.

The following qualities are also desirable:

- Low absorption: If nothing else is obtainable and porous bricks and concrete blocks have to be used working stresses must be correspondingly reduced.
- Freedom from saltpetre (potassium nitrate or sodium nitrate). Bricks having too much saltpetre are unsuitable for reinforced masonry.

For reinforced masonry building construction EBCS-8 1995 specifies units to have not more than 50% by volume of formed holes; the minimum thickness of shells is 15mm and vertical webs in hollow and cellular units extend over the entire horizontal length of the unit.

b) Sand: should be clean, well graded and if possible sharp. Sharpness is not absolutely essential if the grading is good and the sand otherwise sound.

It should be free from organic and vegetable matter, and should also be as free from clay and mica as possible. The presence of mica in the sand is objectionable as a micacious sand requires a greater proportion of cement to produce a mortar equal in strength to mortar made from a sand free from mica and otherwise equally good, unfortunately, most sands are micacious and the only thing to be done is to determine by actual experiments the proportion in which it has to be mixed with cement to produce a suitable mortar.

Too fine sand should not be used as it requires extra cement to be added to produce good mortar.

c) Cement: The cement used must be fresh and shall comply in every way with the standard cement specification. Both Mugher and Messobo cement can be used to give excellent results. Any cement which shows signs of staleness should be rejected and samples of all cement used should be regularly tested.

d) Mortar: The mortar used should consist of cement and sand in proportions varying according to the quality of sand available. Only enough water should be added to make the

mortar of such a consistency that is easily workable, leaves the trowel clean, and can be readily packed round the reinforcement bars.

It will generally be found that 3:1 is a sufficiently rich mixture, but the best proportions should be found by experiment whenever possible.

All mortar used in work must first be thoroughly mixed dry and water should on no account be added except by the masons employed on the work and then only in small quantities. Too great stress cannot be laid on this point. If these precautions be ignored there is every likelihood that stale mortar will be used.

The most suitable method is to have the mortar mixed dry in some central position where this work can be easily supervised, and then have it distributed. If this method be adhered to, each mason need only add water in his iron pan and there is therefore no fear that the mortar will be partially set when used. Needless to say, this method can only be adopted when the sand is really dry. Only clean water should be used.

e) Steel reinforcement: Only the best mild steel should be used as reinforcement. A little rust on the reinforcement is desirable as it ensures good adhesion, but all loose and scaly rust should be removed prior to use. The ends of all rods should be bent into semi-circular hooks of a diameter at least six times the diameter of the rod itself with a short length of straight rod beyond the bend. As far as possible overlapping should be avoided by ordering rods of proper lengths, but where this cannot be done and overlapping has to be resorted to.

#### **2.4.1.2. Workmanship**

The following main points require careful attention:

a) When bricks are used they should be thoroughly soaked before being used. This hardly requires any comment; dry bricks are sure to suck moisture out of the mortar joints and thus interfere with setting.

b) Units are properly arranged as they are laid. "Where possible, the arrangement should be shown in drawings, but it may be laid down as a general principle that joint should be broken wherever possible as this gives increased strength.

c) Reinforcement is properly arranged. Before starting work all rods should be prepared and bent to the correct lengths and shapes shown in the drawings and where possible laid out in situ. If this is done difficulties will be anticipated and cutting and overlapping reduced to a minimum.

d) All joints are well filled and all reinforcement well surrounded by mortar. This requires careful attention as workmen unless watched are apt to scamp the work or grout the joints. Both faults are objectionable and apt to lead to trouble unless checked. All reinforcement must be thoroughly surrounded by mortar otherwise slipping and rusting may take place and adhesion, on which the strength of the structure depends, does not develop fully.

e) Make sure that the work after completion is properly looked after and watered. All work should be kept moist by means of wet straw, wet sand, or merely sprinkling water, for the first and part of the next day after finishing. It should then be profusely watered and kept wet until fully set.

#### **2.4.2. Qualitative Comparison**

Structural masonry walls perform various functions, which for a framed structure need to be provided for separately. The walls provide structure, sub-division of space, thermal and acoustic insulation, and fire and weather protection. In addition to these environmental and functional advantages, it offers the following:

##### **2.4.2.1. Design advantages**

- *Simplicity of detailing*: The architectural and structural detailing and layout are simple due to repetitive floor arrangement.
- *Freedom of architectural expression*: The architect is free to treat the exterior as he desires, because the internal and the corridor walls carry entire vertical and wind loading.
- *Foundation*: The loads from bearing walls are distributed rather than concentrated as happens below column footing resulting in the use of ground with low bearing capacity without special foundation. This advantage is lost if the ground floor is framed construction to give open area.
- *Versatility of Texture and Pattern*: There is unlimited choice of color, texture and patterns of masonry units.

- *Progressive collapse:* With the proper care and design, the chance of progressive collapse is minimal.
- *Savings in Energy:* Requires less energy during construction and occupancy of the building compared to frame construction.

#### **2.4.2.2. Environmental characteristic Advantages**

- *Sound absorbing* qualities of masonry are rated well above the effective levels recommended.
- The inherent *fire resistant characteristics* help in reducing the capacity and size requirements for cooling equipments.

#### **2.4.2.3. Construction Advantages**

The repetitive nature of the layouts in reinforced masonry buildings leads to a construction that does not require large scaffolding and when walls are finished, floors can be placed immediately which provide a working space for the next storey walls. The completion time is very short compared to the reinforced concrete framed buildings. The floor to floor height can be reduced compared to frame construction due to the elimination of beams.

#### **2.4.2.4. Cost advantages**

- According to A paper by Prof. Braj P. SINHA, The initial and maintenance costs are less than concrete and steel frame construction. As much as 10% saving in Europe over other forms, 7 to 9 % saving per square feet per floor over other structural forms, such as concrete in the USA and 38% on structure only over concrete frame in the UK have been reported.
- The absence of framing elements provides no projecting beams to form and this permits reduced floor to floor heights that lead to a significant height and cost reduction of the Reinforced masonry building.
- Use of precast slab permits the achievement of a degree of efficiency on on-site labor that is not available when building frames have to be constructed.
- By enclosing each floor as the work progress, craftsmen of other trades can install utilities and finish interiors while masons are working on additional stories above.

- Structural Masonry wall has a surface that can be left natural and no other surface treatment need be applied except in areas where furring is required for ducts.
- Occupancy of lower stories, by rental or not, is possible even before the above portion of the building is completed.

### **3. MASONRY CONSTRUCTION PROVISIONS IN ETHIOPIA**

#### **3.1. EBCS-6 Provisions**

##### **3.1.1. Ch.1 of EBCS-6: Materials for masonry construction**

This section of the code starts by defining masonry units, first in terms of relevant Ethiopian standards and then by categories which reflect quality control in manufacture. Masonry units are categorized as Bricks, Hollow concrete Blocks, solid concrete blocks, adobe, stabilized soil blocks, natural stone and cast stone.

Mortar general requirements for its component materials and classification based on the proportion of cement content follows and based on the proportion of their cement content mortars are classified as M1, M2, M3 and M4 and the details of the proportion are presented in table 1.3 of the code. Requirements are also set out for unit and mortar durability and for the properties of infill concrete.

##### **3.1.2. Ch. 2 of EBCS-6: Masonry construction**

This section states the general meaning and construction requirements to be achieved for solid, hollow concrete block, stone, stabilized soil block and cavity wall masonry construction.

##### **3.1.3. Ch. 3 of EBCS-6: General Design Principles of Masonry Wall**

As a scope, the code standard specifies requirements for the design and construction of masonry, both unreinforced and reinforced, using manufactured units of clay, concrete or units of square-dressed natural stone and random rubble masonry stone, laid in mortar based on limit state principles and in this section the design situations which have to be considered are defined.

In this section it is noted that the code is aimed for experienced structural engineers or similar appropriately qualified persons who understand structural requirements of masonry construction. And it is also noted that the code standard does not give specific requirements for prestressed masonry, prefabricated masonry panels, masonry in composite action with steel or concrete structural members. And upon use of alternate materials or design methods that are not referred to by this code it should be verified that these materials comply with requirements of appropriate Ethiopian code, and the methods of design and construction ensure as standard of strength and durability at least equal to that recommended in the code, If the material or methods to be used

are not referred to by the code or any other Ethiopian standard they should be proven by tests conducted under approved assembly and conditions that represent the actual building construction.

The last subsection of this chapter prescribes the structural requirements for masonry under construction, newly constructed masonry, masonry in earth quake zones, fire resistance, thermal and sound insulation, durability, moisture penetration prevention, differential movement, control joints, mortar joints, bond pattern, wall ties, bonding and tying for monolithic structural action, connections to structural supports and discusses matters of stability, stiffness and strength.

### 3.1.4. Ch. 4 of EBCS-6: General Requirements for Structural Design

This chapter of the code sets out, in its first quarter discussion, the general requirements for the structural design of unreinforced and reinforced masonry based on Limit state design concepts and specifies the provisions for safety, serviceability and proper functioning of the structure and its components. Actions which include loads and imposed deformations and design load combinations are obtained from EBCS-1 or other approved sources.

The characteristic strengths of masonry in compression (confined and unconfined), tension, shear, the characteristic lateral modulus of rupture of masonry units, compressive strength of grout, tensile strength of reinforcement and modulus of elasticity of reinforcement are defined as given below:

The characteristic compressive strength of masonry other than special masonry used in design is given by:

$$f'_m = K_h f'_{mb} \quad (2.26)$$

Where,  $f'_{mb}$ , the characteristic compressive strength of masonry, given in table form as a function of type of masonry unit, mortar composition and unconfined compressive strength ( $f'_{uc}$ ) of unit based on test data or equivalently computed from:

$$f'_{mb} = K_m \sqrt{f'_{uc}} \quad (2.27)$$

$K_h$  is compressive strength factor given as a function of masonry unit height to mortar bed joint.

The characteristic unconfined compressive strength of masonry units,  $f'_{uc}$ , may be determined by multiplying the values of characteristic compressive strength by aspect ratio factor,  $K_a$  as:

$$f'_{uc} = K_a f'_m \quad (2.28)$$

The characteristic Flexural Tensile strength of masonry,  $f'_{mt}$ , is specified by the code to take the values of:

$f'_{mt}$  have a value of less than or equal to 0.20 MPa for unreinforced masonry subjected to actions resulting from wind loads or similar forces of short term, transient nature, and acting on section of the masonry where the simultaneous minimum design compressive force and  $F_d$  is never a negative value (i.e. is never a net tensile force); a value less than or equal to 1.0 MPa and obtained from test for special masonry and a value of zero is used for all other cases.

Characteristic shear strength of masonry,  $f'_{ms}$ , values for various course and bed joint conditions are given and it is specified to attain a value of 0.35MPa for reinforced masonry. The characteristic lateral modulus of rupture of masonry units,  $f'_{ut}$ , is specified to have a value of 1.8MPa unless a higher value is obtained by tests. And the characteristic compressive strength of grout,  $f'_c$ , is determined from tests on specimens manufactured and tested at 28 days age in accordance with the requirements of concrete given in EBCS-2. And is specified to lie in the range given by:  $12\text{MPa} \leq f'_c \leq 1.3f'_{uc}$ .

The characteristic tensile strength of reinforcement steel,  $f_{yk}$ , is taken as specified in EBCS-2 and the modulus of elasticity of reinforcement,  $E_s$ , of 200GPa or a value derived from test is taken.

The second quarter of this chapter, discusses about reduction factors and recommends, in evaluating the strength of a member, its capacity be reduced by a factor called reduction factor,  $C$ , given for unreinforced and reinforced masonry members separately. The values given for reinforced masonry members are:  $C_r \leq 0.75$ , for transient out-of-plane forces and  $\leq 0.7$ , for all other forces

The third quarter of the chapter describes about cross-section dimensions (bedded thickness, effective depth of reinforced masonry members, effective width of compression faces and flanges, junction between reinforced and unreinforced masonry, structural end of a masonry member), cross-section areas (Bedded area, combined cross-sectional area, design cross-sectional area, grout area), section modulus, and second moment of area. The effective depth for reinforced masonry members is specified to be the distance from the extreme compressed fiber of the masonry to the resultant tensile force in the steel in that zone which will be tensile under ultimate

strength conditions. The combined cross-sectional area of reinforced masonry member is given to be the overall cross-sectional area, not including the depth of raking (if any) of the mortar joints and the design cross-sectional area,  $A_d$ , of the reinforced masonry member is taken as the combined cross-sectional area. The section modulus and second moment of area of a cross-section at mortar joint in a member are derived from the dimensions of the design cross-sectional area at the mortar joint.

Slenderness ratio,  $S_r$ , is defined as the ratio of the effective height to the effective thickness and is used in the design of masonry members for compression. It is assessed about each of the two principal axes of a cross-section of a member and is evaluated with different formulae for different sets of lateral support in the two axes and the presence of openings near the member under consideration.

The last quarter of the section discusses about the determination of design action effects and the design resistance effects for stability. It specifies that loads and other actions are determined from EBCS-1 and subdivision is made between stabilizing and destabilizing actions; the design action effects are calculated from the factored-destabilizing load components and are combined according to EBCS-1 provisions. The design resistance effect is calculated from 0.8 times the component of the un-factored-stabilizing actions and the masonry structure is designed for stability so that:

$$\text{Design Action Effect} \leq 0.8(\text{Design resistance effect})$$

The design for serviceability deals with designing the structure and its components with respect to limiting deflections and cracking and check adequacy of the degree of robustness of members and their connections. The minimum thickness required for robustness of isolated piers and walls is given in this section as:

$$t_r \geq \frac{H}{C_v k_t}; \text{ For isolated piers} \quad (2.29)$$

$$t_r \geq \text{smaller} \left[ \frac{L}{C_h} \text{ or } \left( \frac{H}{3C_v k_t} + \frac{L}{3C_h} \right) \right]; \text{ For walls without openings} \quad (2.30)$$

Where,  $C_v$  and  $C_h$  are robustness coefficients that depend on edge restraints and reinforcement availability and their values are given in table 4.5 of EBCS-6.

### 3.1.5. Ch.5 of EBCS-6: Design of Unreinforced Masonry

This chapter deals with particular requirements for the structural design of unreinforced masonry in addition to the general requirements discussed in the previous chapters. In this chapter unreinforced masonry is defined as masonry which do not contain any reinforcement or which may contain reinforcement that does not comply with the structural design requirements of reinforced masonry. Generally this section suggests that each unreinforced masonry member shall be designed for compressive forces and vertical bending, vertical bending from transient out-of-plane forces, horizontal bending from transient out-of-plane forces and shear forces as appropriate and sets out the factors to be considered for the above mentioned actions.

### 3.1.6. Ch. 6 of EBCS-6: Design of Reinforced Masonry

This section, as a scope, deals with the particular requirements for the structural design of masonry with steel reinforcement acting compositely and included to enhance the resistance to applied forces and gives provisions for the design of reinforced masonry members for axial tension, compression, bending, shear and compression and bending together as follows:

- i. Design for Compression: - the compressive load capacity,  $F_d$ , of a Reinforced masonry member is given by:

$$F_d \leq C_1 K (f'_m A_b + K_c \sqrt{\left[\frac{f'_c}{1.3}\right]} A_c + f_{sy} A_s) \quad (2.31)$$

$$f'_c \leq 1.3 * f'_{uc}$$

$$0.005 * A'_d \leq A_s \leq 0.04 * A_c$$

$K_c = 1.4$  for Hollow concrete masonry units of density greater than  $20 \text{ Kn/m}^3$  and  $1.2$  for all other masonry.

The main reinforcement in a compression member shall be located symmetrically and laterally restrained by ties not less than 6mm in diameter which are spaced at centers not exceeding the least cross-section of the member or 400mm which ever is lesser.

- ii. Design for Bending: - the design flexural capacity of a reinforced member is given by:

$$M_d \leq C_r f_{sy} A_{sd} d \left[ 1 - \frac{0.6 f_{sy} A_{sd}}{1.3 f'_m b d} \right] \quad (2.32)$$

$$\text{And } \frac{A_{sd}}{b d} \leq 0.29 \left[ \frac{1.3 f'_m}{f_{sy}} \right]$$

The main tensile reinforcement in a flexural member shall be spaced at centers not exceeding 800mm.

- iii. Design for Combined compression and Bending: - whenever a member is subjected to a combined action of compression and bending, the compressive load capacity is taken as 0.85 times that given for pure compression and the bending capacity is taken equal to the value for pure bending.
- iv. Design for Axial Tension: - the design axial tension capacity of a reinforced member is given by:  $F_{dt} \leq C_r f_{sy} A_s$
- v. Design for Shear: - this sub section of the code gives provisions for the shear capacity of piers and beams and in-plane and out-of-plane shear capacities for walls. The shear capacity of piers, beams and the out-of plane shear capacity of walls are calculated using the following formula:

$$V_d \leq C_r (f'_{ms} b_{wd} + f_{vs} A_{st} + f_{sy} \frac{A_{svd}}{S}) \text{ But } \leq 4 C_r f'_{ms} b_w d \quad (2.33)$$

Where:  $f_{vs} = 17.5 \text{ Mpa}$

$A_{st}$  = area of fully anchored longitudinal reinforcement in the tension zone of the cross-section under consideration, or  $0.02 b_{wd}$ , which ever is less.

$A_{sv}$  = cross-sectional area of shear reinforcement t per spacing interval with spacing s, in the direction of the span of the member not exceeding  $0.75D$  or  $500\text{mm}$  whichever is less.

S = spacing of shear reinforcement along the member.

The in-plane shear capacity of walls could be calculated as:

$$V_d \leq C_r (f_{vr} A_d + 0.8 f_{sy} A_s) \quad (2.34)$$

The last subsection of this chapter talks about reinforcement detailing, cover and protection and gives provisions of cove for reinforcements in grouted cavity wall construction and reinforcements embedded in mortar joints.

### 3.1.7. EBCS-8 provisions for masonry buildings

EBCS-8 1995 gives specific design provisions applicable to unreinforced, confined and reinforced masonry buildings in seismic areas. And relevant provisions to reinforced masonry buildings are discussed in this thesis.

Masonry buildings are classified depending on the masonry type used for the seismic resistant systems as unreinforced, confined and reinforced masonry systems.

In structural analysis of reinforced masonry buildings, the structural modeling of the building represents the stiffness properties of the whole system and the stiffness of structural elements shall be evaluated by taking account of flexural, shear and axial deformability as appropriate. In the modeling process, floor diaphragms may be considered rigid, without further verification, if they consist of reinforced concrete according to chapter 3 of EBCS-8 and their openings do not significantly affect the over all in-plane rigidity of the floors.

As a design and criteria and construction rules, reinforced masonry buildings are specified to be composed of floors and walls that are connected in every direction by steel ties or reinforced concrete ring beams. Shear walls must also be provided at least in two orthogonal directions fulfilling the geometric requirements of thickness not less than 240mm and effective height to thickness ratio not exceeding 15 as given in table 6.2 of EBCS-8 1995.

Horizontal reinforcement of diameter not less than 4mm shall be provided in the bed joints or suitable grooves in the units with a vertical spacing not exceeding 600mm. The reinforcements needed in lintels and parapets shall be accommodated by special units with recesses. And the minimum percentage of the horizontal reinforcement in the wall shall not be less than 0.05% of the wall cross-sectional area.

At every wall intersection and at both free edges of every wall element, the total vertical reinforcement area should not be less than  $400\text{mm}^2$ . If the spacing between such vertical reinforcements exceeds 4m, similar reinforcement provisions are provided at every 4 meters. The minimum percentage of the vertical reinforcement spread in a wall shall not be less than 0.1% of the walls gross area.

In the ultimate limit state verifications, the partial safety factors for masonry properties shall be taken equal to 2 and for reinforcing steel equal to 1. The value of the behavior factor  $\gamma$ , for reinforced masonry buildings is taken to be equal to 0.4.

EBCS-8 1995 specifies, for simple buildings, that no explicit safety verification against collapse is required because it is deemed verified if a building satisfies the following rules to classify masonry buildings as simple.

- The total numbers of storeys do not exceed 5 in zones 1, 2, 3 and 3 in zone 4.
- The plan of the building is approximately rectangular, allowing projections and recesses of 15% in each direction, with smaller to longer dimension ratio not less than 0.25.
- Minimum of two parallel shear walls, the length of each being greater than 30% of the building length in the direction of the wall. For buildings in zones 1 and 2 the wall length requirement can be verified by the cumulative length of shear walls in one axis separated by openings.
- The difference in mass and horizontal shear wall cross-section in two orthogonal axes between adjacent storeys does not exceed 20%.
- At every floor the horizontal shear wall cross section in two orthogonal directions given as a percentage of the total floor area above the level considered is not less than 2% for buildings in zones 1,2,3 and 4% in zone 4.

### **3.2. Comments to EBCS-6 documentation**

- The code gives section properties (size, compressive strength etc) to solid concrete blocks only but same specifications need to be given for bricks and other masonry units.
- The types of mortar given in EBCS-6 are M1, M2, M3, and M4 but in EBCS-8 under section 6.2.3 it is specified that the minimum mortar that can be used for reinforced masonry construction is type M10.
- Unlike the British and Euro code standards, Design aids are not provided.
- No specific guidance is given, for building as a function of number of storeys, for Accidental Damage considerations.
- Section 1.1.4.4 specifies to refer to section 1.1.4.6 for coloring but the so stated section does not exist.

- Section 3.4.8(1a) specifies to refer to section 2.6.12 but this does not exist.
- It has been specified in section 4.10.2.1(1) that masonry members and their connections shall have an adequate degree of robustness in their construction, and shall comply with the requirements of the non-existent section 2.5.10.2.2.
- Section 5.3.2(b) gives the basic compressive capacity of a cross section for two cases, one for a “testing done” and second case also gives for “testing done”; one of them has to be specified for “Testing not done”.
- Section 5.3.2 specifies to refer table 2.6.2 for reduction factor for slenderness and eccentricity for hollow units but there is no such table number in the code. This would have been correct if table 5.2 was specified.
- EBCS 6 in general considers masonry buildings as if wall is the only component. But, indeed, masonry beams and columns are also main components and treatment shall be given to them.

## 4. DESIGN OF REINFORCED MASONRY AND REINFORCED CONCRETE BUILDINGS

### 4.1. General Design Considerations

In design, it is assumed that for reinforced masonry structures the ultimate limit state will be critical. The design, therefore, is carried out using the partial safety factors appropriate to the ultimate limit state. Recommendations are given to ensure that the serviceability limit states of deflection and cracking are not reached.

When using the design relationships for the ultimate limit state, the design load should be taken as the sum of the products of the component characteristic loads, or for earth loads the nominal load, multiplied by the appropriate partial safety factor, as shown below. Where alternative values are shown, the case producing the more severe conditions is selected.

a. Dead and imposed load

Design dead load =  $0.9G_k$  or  $1.3G_k$

Design imposed load =  $1.6Q_k$

b. Dead and wind load

Design dead load =  $0.9G_k$  or  $1.3G_k$

Design wind load =  $1.6W_k$

In the particular case of freestanding walls and laterally loaded wall panels, whose removal would in no way affect the stability of the remaining structure,  $\gamma_f$  applied on the wind load may be taken as 1.2.

c. Dead, imposed and wind load

Design dead load =  $1.2G_k$

Design imposed load =  $1.2Q_k$

Design wind load =  $1.2W_k$

d. Dead, Imposed and Earthquake

Design dead load =  $0.75(1.3)G_k$

Design imposed load =  $0.75(1.6)Q_k$

Design earthquake load =  $A_d$

For all these cases:

$G_k$  is the characteristic dead load;

$Q_k$  is the characteristic imposed load;

$W_k$  is the characteristic wind load;

$A_k$  is the earthquake load;

In design, each of the load combinations (a) to (d) will be considered and that giving the most severe conditions adopted.

## **4.2. Design Procedures**

### **4.2.1. Reinforced Concrete Buildings**

#### **STEP 1: Data**

- Data on Use of the Building
- Data on Live Load, Wind Load, Earthquake Zone
- Data on Structural and non structural materials to be used in the building
- Location of the Building, Seismic zone
- Depth of Foundation below ground
- Type of Soil, Allowable bearing pressure
- Data on Material Properties

#### **STEP 2: Geometry of the Building**

- General Layout
- Storey Number
- Column Number
- Beam Number
- Preliminary size and dimension of Building elements

#### **STEP 3: Gravity Load Calculation**

- Member self weights (slab, beam, column, wall etc)
- Slab Load Calculations
- Frame Load Calculations
- Load frames with the above calculated loads

**STEP 4: Seismic Weight Calculations**

- Loads from Slab, parapet, walls, beams, columns of every storey
- Seismic Weight of the entire building =  $\sum_{i=1}^n (Storey)_{Weight}$
- Seismic weight of floors is lumped at respective floor levels and act at the center of mass of the floor.

**Table 4: Illustration Table for Seismic Weight Calculation for a Storey**

Description	Length (m)	Weight, W (kN/m)	Weight in kN	Distance X (m)	Distance Y (m)	$M_x = WX$	$M_y = WY$
Beam 1							
Beam n							
		$\sum_{i=1}^n W_{Beams}$				$\sum_{i=1}^n (WX)_{Beams}$	$\sum_{i=1}^n (WY)_{Beams}$

Description	Length (m)	Weight, W (kN/m)	Weight in kN	Distance X (m)	Distance Y (m)	$M_x = WX$	$M_y = WY$
Column 1							
Column n							
		$\sum_{i=1}^n W_{columns}$				$\sum_{i=1}^n (WX)_{columns}$	$\sum_{i=1}^n (WY)_{columns}$

Description	Length (m)	Weight, W (kN/m)	Weight in kN	Distance X (m)	Distance Y (m)	$M_x = WX$	$M_y = WY$
Wall 1							
Wall n							
		$\sum_{i=1}^n W_{walls}$				$\sum_{i=1}^n (WX)_{walls}$	$\sum_{i=1}^n (WY)_{walls}$

**Table 5: Illustration table for mass center calculation for a storey**

Description	Total Weight	Total WX (Mx)	Total WY (My)	MASS CENTER	
				$\bar{X}$	$\bar{Y}$
Beams	$\sum W_B$	$\sum WX_B$	$\sum WY_B$	$\frac{\sum M_x}{\sum W}$	$\frac{\sum M_y}{\sum W}$
Columns	$\sum W_C$	$\sum WX_C$	$\sum WY_C$		
Walls	$\sum W_W$	$\sum WX_W$	$\sum WY_W$		
Total	$\sum W$	$\sum WX = \sum M_x$	$\sum WY = \sum M_y$		

**STEP 5: Design Seismic Load, W, Calculation**

- Calculate the Fundamental Period

$$T_1 = C_1 * H^{3/4} \text{ (Allowed for buildings up to a height of 80 meters)}$$

$$C_1 = 0.075 \text{ for R.C. Moment Resisting Frames}$$

H = Height of the Building above Base

**STEP 6: Calculate Base Shear and distribute it to Different Floor Levels****Table 6: Illustration table for Base shear calculation**

Item to Calculate	Source to Calculate
Total Dead Load	$\sum W$ , From previous calculations
Bed Rock Acceleration Ratio, $\alpha_0$	$\alpha_0 = 0.05$ , from EBCS-8, 1995
Importance Factor	I, from EBCS-8, 1995
Ratio of Bed rock acceleration to gravity, $\alpha$	$\alpha = \alpha_0 * I$
Sub Soil Classification, S	S, From EBCS-8, 1995
Fundamental Period, $T_1$	$T_1$ , from Step 5
Elastic Response Factor, $\beta$	$\beta = 1.2S / T_1^{2/3} \leq 2.5$
Behavior factor, $\gamma$	$\gamma$ , from EBCS-8, 1995
Ordinate of the Design Spectrum, $S_d(T_1)$	$S_d(T_1) = \alpha\beta\gamma$
BASE SHEAR, $F_b$	$F_b = S_d(T_1) * \sum W$
$F_t$	$F_t = 0.07 * T_1 * F_b$

**Table 7: Illustration table for distribution of Base shear to different Storeys**

Storey	$W_i$	$h_i$	$W_i \cdot h_i$	Storey Shear, $Q_i$	$M_x$	$M_y$	$\bar{X}$	$\bar{Y}$
Roof				$Q_i = \frac{W_i h_i}{\sum W_i h_i} (F_b - F_i)$	$W_i \bar{X}_i$ From Table 5	$W_i \bar{Y}_i$ From table 5	$\bar{X}_i$ From table 5	$\bar{Y}_i$ From table 5
Ground								
	$\sum W_i$		$\sum W_i h_i$					

**STEP 7: Analyze by space frame**

- Model the space frame with software
- Load Gravity Loads
- Load earth quake loads accounting for accidental eccentricity of  $e_{Li} = \pm 0.05L_i$
- Include the basic Load cases shown in table 8

**Table 8: Basic Load Cases**

No.	Load Case	Direction
1	DL (Dead Load)	Down ward
2	IL (Imposed/ Live load)	Down ward
3	EXTP (+ve torsion)	+X: EQ acting to the LEFT of C.G.
4	EXTN (-ve torsion)	+X: EQ acting to the RIGHT of C.G.
5	EYTP (+ve torsion)	+Y: EQ acting ABOVE C.G.
6	EYTN (-ve torsion)	+Y: EQ acting BELOW C.G.

- Include the Load combinations given in table 9

**Table 9: Load Combinations**

No.	Combination Name	Combination Description
1	COMB1	1.3DL+1.6IL
2	COMB2	0.75(COMB1)+EXTP
3	COMB3	0.75(COMB1)-EXTP
4	COMB4	0.75(COMB1)+EXTN

5	COMB5	0.75(COMB1)-EXTN
6	COMB6	0.75(COMB1)+EYTP
7	COMB7	0.75(COMB1)-EYTP
8	COMB8	0.75(COMB1)+EYTN
9	COMB9	0.75(COMB1)-EYTN
10	COMB10	ENV(COMB1, COMB2, ..., COMB9)

**STEP 8:** Storey Drift Calculation and Second Order (P-Δ) Effect consideration

– EBCS-8, 1995 clause 2.3.4 (displacement analysis) states that the displacement induced by the design seismic action shall be calculated on the basis of the elastic deformation of the structural system by means of the simplified formula given in equation 2.33.

$$d_s = \frac{d_e}{\gamma_d} \quad (2.35)$$

Where  $d_s$  is the displacement of a point of the structural system induced by the design seismic action,  $\gamma_d$  is the displacement behavior factor equal to  $\gamma$ , and  $d_e$  is the displacement of same point as determined by a linear analysis based on the design spectrum.

– The Design Interstorey Drift,  $d_r$  is then calculated from the following formula:

$$d_r = (d_s)_{top} - (d_s)_{bottom} \quad (2.36)$$

– EBCS-8, 1995 clause 2.4.3.2 specifies that  $d_r \leq 0.015h$  for buildings having non-structural elements fixed in a way as not to interfere with structural deformations.

– EBCS-8, 1995 clause 2.4.2.2 states that second order effect, (P-Δ), need not be considered when the Interstorey drift sensitivity coefficient,  $\theta$ , at all storeys satisfy the condition given in equation 2.35.

$$\theta = \frac{P_{tot} * d_r}{V_{tot} * h} \leq 0.10 \quad (2.37)$$

Where:  $\theta$  is Interstorey Drift-Sensitivity coefficient

$P_{tot}$  is the total gravity load at and above the  $i^{\text{th}}$  storey considered

$d_r$  is the Design Interstorey Drift

$V_{\text{tot}}$  is the total Seismic storey shear

H is the Interstorey height

- If  $0.1 < \theta \leq 0.2$ , the second order effect can be approximately taken into account by increasing the relevant seismic action effect by a factor equal to  $\frac{1}{1-\theta}$ .
- Similar to the Above formula, but slightly different, given in EBCS-2, 1995 clause 4.4.4.2 states that beam and column type frames in a building structure with beams connecting each column at each storey level may be classified as Non-Sway for a given load case, when first order theory is used, the horizontal displacement in each storey due to the design loads (both vertical and horizontal) plus the initial sway imperfection satisfy the following:

$$Q_{si} = \frac{N\delta}{HL} \quad (2.38)$$

Where:  $\delta$  is Horizontal displacement at the top of the storey relative to the bottom of the storey /same as Interstorey drift/, L is the Interstorey height = h, H is the total Horizontal reaction at the bottom of the storey (total lateral force acting in the storey) and N is the total Vertical reaction at the bottom of the storey (the sum of Axial loads on all columns in the  $i^{\text{th}}$  storey).

The Criteria for Classification is then:

$$Q_{si} \leq 0.1, \text{ NON-SWAY column}$$

$$Q_{si} > 0.1, \text{ SWAY column}$$

**Table 10: Storey Drift Calculation**

Storey	Displacement (mm)	Storey Drift, $d_r$
Top storey	a	(a-b)
⋮	b ...	(c-b) ....
First storey	n	n-0
Footing Top	0	0

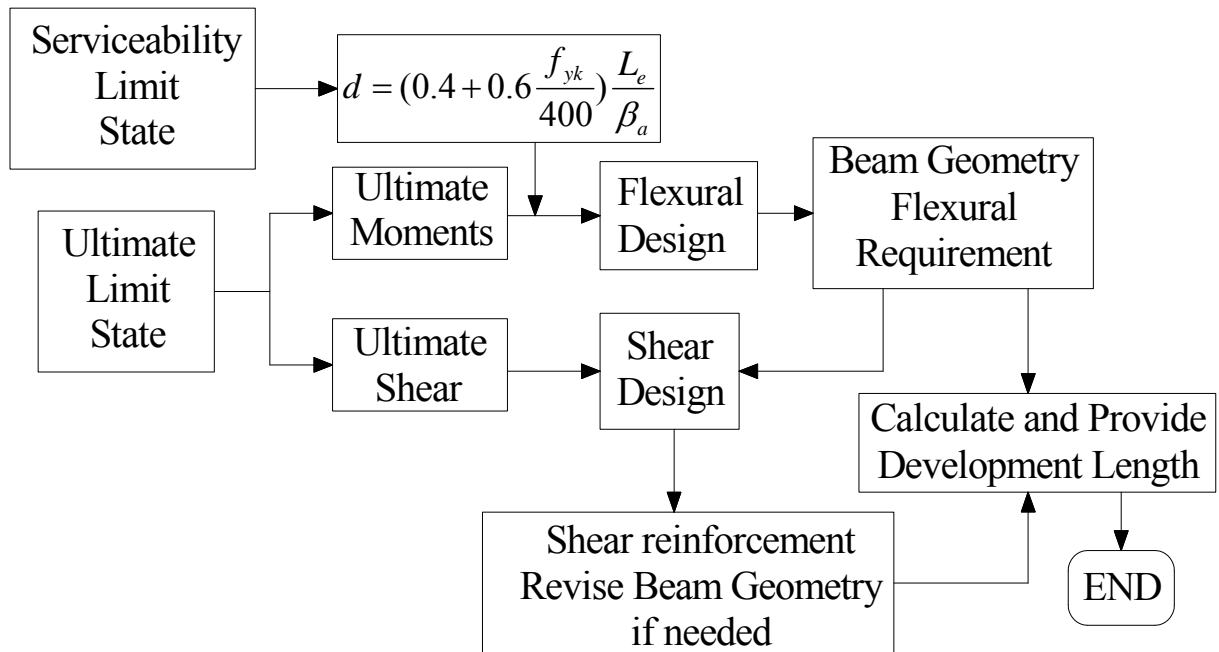
Note that for all storeys, the storey drift,  $d_r \leq 0.015h$

**Table 11: Illustration Table for Stability Indices of Different storeys**

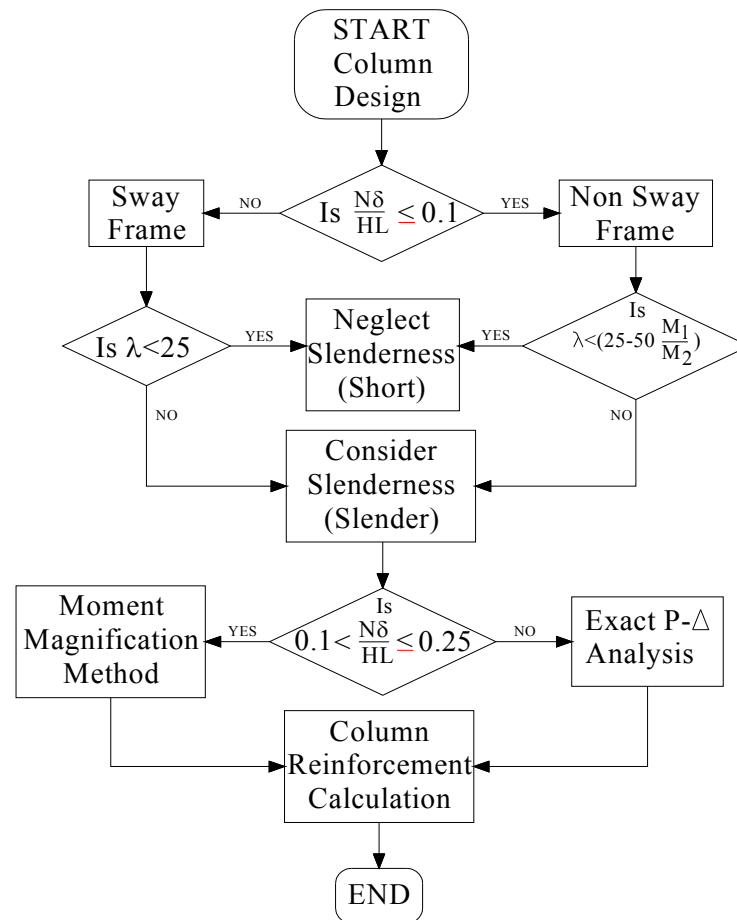
Storey	Storey Seismic Weight $W_i$ (kN)	Axial Load $N = \sum W_i$ (kN)	Storey Drift $\delta$ (mm)	Lateral Load $H = V_{tot}$ (kN)	Interstorey Height $h_s$ (mm)	$Q_{si} = \frac{N\delta}{HL}$	Classification
Top	From column 2 of table 7	Sum down $W_i$ from left column	From column 3 of table 10	From column 5 of table 7	From column 3 of table 7	Calculate from other columns	If $Q_{si} \leq 0.1$ , Non Sway
⋮							If $Q_{si} > 0.1$ , Sway
Ground							

**STEP 9: Design of Beams**

In typical beams the preliminary behavior is flexure. Beam sections, generally, are designed for flexure first and then checked for shear strength. Over all design scheme of beams is shown in the following flow chart.



**Flow chart for beam design**

**STEP 10: Design of Columns****Flow chart for column Design****4.2.2. Reinforced Masonry Buildings****STEP 1: Given Data Analysis****A. Loading Data**

Roof – Roof Dead Load

– Roof Live Load

Floor(s) – Floor Dead Load

– Floor Live Load

Wall Weight – Wall Dead Load

B. Calculate Seismic Weight

Roof Dead Load	}	Seismic weight on Roof
Wall Dead Load of Upper storey		
Floor Dead Load, Live Load	}	Seismic weight on Floors
Wall Load of Upper floors		

**STEP 2:** Calculate the Base shear

Seismic Zone	}	$V_b$ , Base shear
Occupancy, Importance factor		
Response reduction factor		
Height of the Building		

**STEP 3:** Determine Design Lateral Forces at Each storey level

**Table 12: Design Lateral Forces at Each Storey Level**

Storey	$W_i$ , (kN)	$h_i$ , (m)	$W_i h_i$ ( $\times 10^{-3}$ kN-m)	$\frac{W_i h_i}{\sum W_i h_i} V_b$ , (kN)	Lateral Force at Level i (kN)
1				a	a
:				b...	a+b...
n				n	a+b+...+n
	$\Sigma$		$\Sigma$		$\Sigma$

**STEP 4:** Distribution of Shear Among different Walls at all floor levels

- Calculate the location of the Center of Gravity
- Calculate the stiffness of Piers and walls.
- Locate the Center of Rigidity, Calculate static eccentricity as the difference between the center of mass and center of rigidity and the Torsional moment developed due to eccentricity as  $M_T = V_b * \text{Eccentricity}$
- Evaluate the combined shear forces carried by each wall (i.e. translational shear  $V$ , and torsional shear  $V_m$ )

**Note for Step 4 (B):** the Stiffness of piers and walls may be computed by the following formulas. (*IITK-GSDMA Guidelines for Structural use of Reinforced Masonry*)

$$k_F = \frac{E_m t}{\left[ \left( \frac{H}{L} \right)^3 + 3 \frac{H}{L} \right]}, \text{ for Fixed Wall or Pier} \tag{2.39}$$

$$k_C = \frac{E_m t}{\left[ 4 \left( \frac{H}{L} \right)^3 + 3 \frac{H}{L} \right]}, \text{ for Cantilever Wall} \tag{2.40}$$

$E_m = 550 * f_m$ , where  $f_m$  is the grade of masonry in Mpa.

**Table 13: Calculation of stiffness of piers and walls**

Pier No.	Length L (m)	H(m)	H/L	$K_i$ (mN/m)
1				
⋮				
n				
Wall 1				
⋮				
Wall n				

For several piers connected at their tops, stiffness of the wall is determined as:

$$k = \sum_{i=1}^n k_i = k_1 + k_2 + \dots + k_n \tag{2.41}$$

**Table 14: Distribution of translational and torsional shears among different Walls at a Floor Level**

Wall	Stiffness, $K_i$ (mN/m)	$D_y$ (m)	$K_i D_y$	$K_i D_y^2$	$V = \frac{K_i}{\sum K_i} V_b$ (kN)	$V_m = \frac{K_i D_y}{\sum K_i D_y^2} M_T$ (kN)	$V_T = V_m + V$ (kN)
	Σ			Σ			

**Note:** - The same is done when load is in y-direction by replacing x for y.

**STEP 5:** Distribute the Lateral shear force carried by each wall among piers within a wall

Assuming the floor to be rigid, lateral forces will be distributed in proportion to the pier stiffness.

**Table 15: Distribution of Lateral force among piers within a wall**

Pier No.	$K_i$ (kN/m)	$\frac{K_i}{\sum K_i}$	$V_{ei} = \frac{K_i}{\sum K_i} V_b$
⋮	⋮	⋮	⋮
	$\Sigma$	$\Sigma$	$\Sigma$

**STEP 6:** Distribution of Overturning Moment to Piers as Axial force

A. Locate of the Neutral axis

**Table 16: Calculation of Location of the Neutral axis**

Pier No.	$Y_{i_1}$ (m)	$A_{i_1}$ (m <sup>2</sup> )	$A_i * Y_i$ (m <sup>3</sup> )	$Y_{NA}$
⋮	⋮	⋮	⋮	$Y_{NA} = \frac{\sum A_i * Y_i}{\sum A_i}$
		$\Sigma$	$\Sigma$	

B. Calculate the distance of the Center line of the piers from the Neutral Axis

C. Distribute the Over Turning Moment to Piers

**Table 17: Distribution of the Over Turning Moment to Piers**

Pier No.	$A_i$ (m <sup>2</sup> )	$Y_{bi}$ (m)	$A_i * Y_{bi}^2$ (m <sup>4</sup> )	$(I_{gi}) = \frac{tL^3}{12}$	$(I_{NAi}) = (A_i Y_{bi}^2) + (I_{g_i})$	$A_i * Y_i$ (m <sup>3</sup> )	$(DF_m)_i$	$P_{ei}$
⋮	⋮	⋮	⋮	⋮	⋮	⋮	$\frac{A_i Y_{bi}}{\sum I_{NA}}$	$(DF_m)_i * M_T$
	$\Sigma$		$\Sigma$	$\Sigma$	$\Sigma$	$\Sigma$	$\Sigma$	$\Sigma$

**STEP 7:** Distribution of Direct Axial Compression to Piers within a wall

A. Axial forces due to gravity loads are distributed in proportion to the tributary wall length and the distribution is made for dead loads and live loads separately.

B. Summarize the forces in different piers due to different loads as in the following table

**Table 18: Summary of Forces in Piers due to different Loads**

Pier No.	Pier Height, $h_i$	$(P_D)_i$ (kN)	$(P_L)_i$ (kN)	$(P_E)_i$ (kN)	$(V_E)_i$ (kN)	$(M_E)_i$ (kN -m)
⋮	⋮	From A of step 7	From A of step 7	From table 16	From table 14	$\frac{(V_e)_i * h_i}{2}$
		$\Sigma$	$\Sigma$	$\Sigma$	$\Sigma$	

**STEP 8: Load Combinations**

Within the combinations allowed by EBCS-2, 1995; all load cases, Dead Load (DL), Live Load (LL) and Earthquake Load (EL), are taken into account and the combinations that govern in that particular load combination and are combined with envelope as follows.

**Table 19: Load Combinations**

No.	Combination Name	Combination Description
1	COMB1	1.3DL+1.6IL
2	COMB2	0.75(COMB1)+EXTP
3	COMB3	0.75(COMB1)-EXTP
4	COMB4	0.75(COMB1)+EXTN
5	COMB5	0.75(COMB1)-EXTN
6	COMB6	0.75(COMB1)+EYTP
7	COMB7	0.75(COMB1)-EYTP
8	COMB8	0.75(COMB1)+EYTN
9	COMB9	0.75(COMB1)-EYTN
10	COMB10	ENV(COMB1, COMB2, ..., COMB9)

**Table 20: Loads on Piers under Different Load combinations**

Pier No.	$(P_{comb1})_i$	$(P_{comb2})_i$	$(P_{comb3})_i$	$(M_{comb1})_i$	$(M_{comb2})_i$
⋮	⋮				

**STEP 10: Design Reinforced Masonry Elements According to Section 2.3 of this thesis or any method equivalent.**

## 5. COST EVALUATION AND COMPARISON OF BUILDINGS

### 5.1. General Evaluation scheme

For the purpose of cost evaluation the following tasks are first carried out.

- Design of a sample G+4 apartment building constructed mainly of reinforced masonry walls.
- Document obtained from Addis Ababa Housing Development project office for the design of similar building, constructed mainly, with reinforced concrete frame system is taken as it is for comparison with the masonry building.
- Quantity of the major works in both buildings is calculated.
- The slab system is assumed to be rigid enough to transfer lateral forces to respective structural elements and in both cases, the same type of slab- irrespective of what kind- is assumed and so the cost of slab is not considered in both.
- Similarly, the roof system and the façade walls are taken to be identical and their cost ignored in the comparison.

The costs of construction applied on both buildings, as current unit prices of construction, are referred from the actual market in Addis Ababa city.

In this thesis, comparison is made for buildings of similar size and shape but with different construction technologies, namely reinforced concrete and masonry.

The cost comparison is continued, until a break even point is reached, by altering the costs of cement and brick units – which are the main components that affect cost- in the two construction technologies.

The cost of cement and brick are altered beginning from their current prices of 250 Birr per quintal and 0.85 birr per unit respectively.

The price of cement is reduced up to a practically feasible limit and is taken in this thesis as 91Birr per quintal. And that of bricks is set not to exceed 10 Birr per unit for a similar reason.

The plan views and geometrical details of the buildings and their main structural components used in design are shown in Figure 21 and Figure 22.

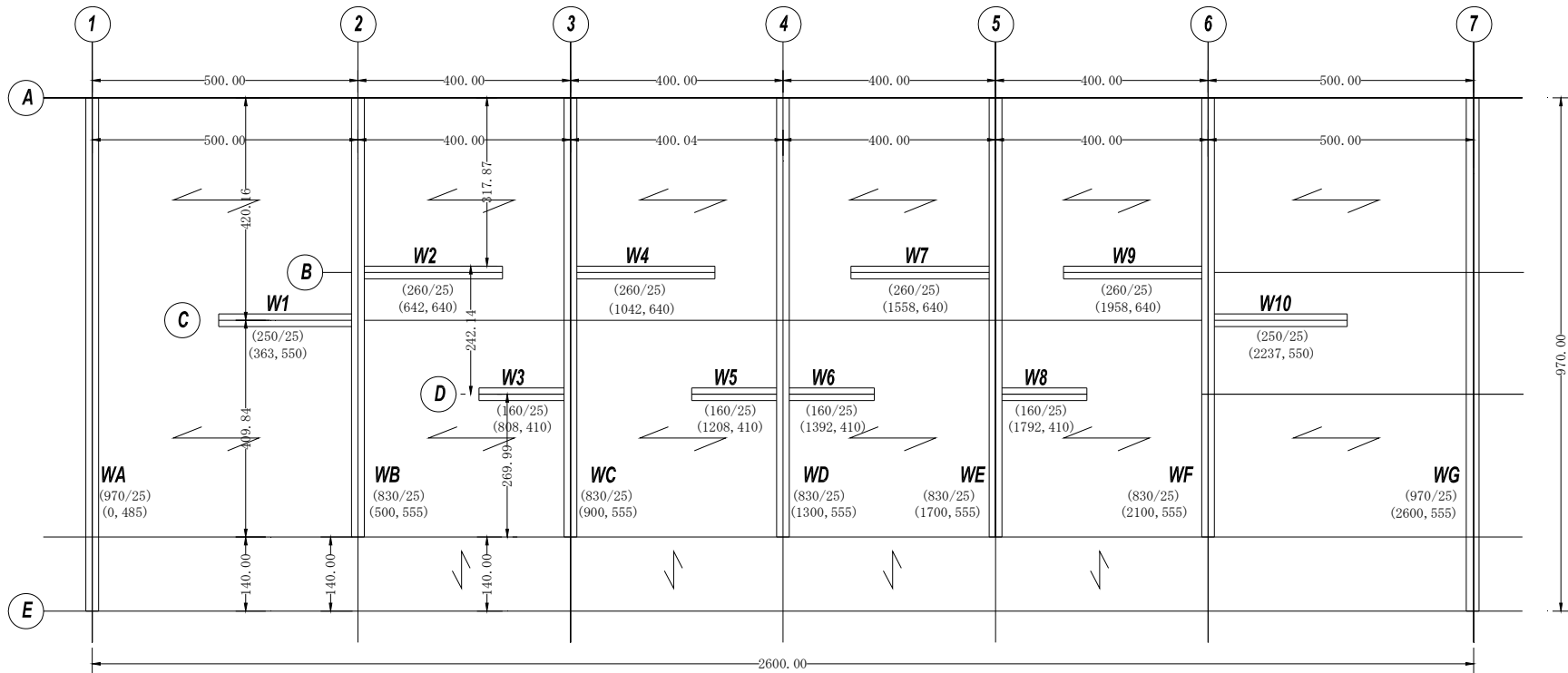


Figure 21: Reinforced Masonry Wall Building Typical floor Plan

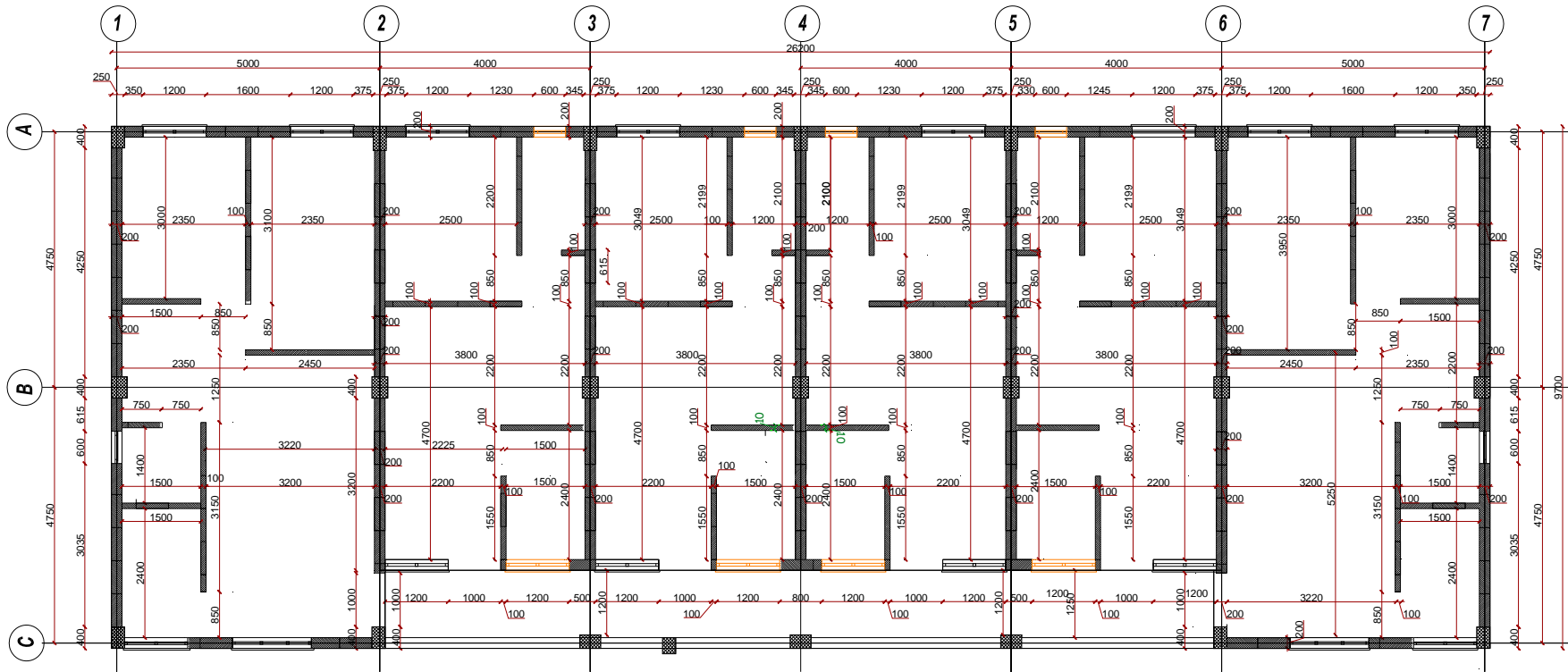


Figure 22: Reinforced Concrete Building Typical Floor Plan

## 5.2. Cost evaluation of Reinforced Concrete and Reinforced Masonry Buildings

The quantities and respective unit costs for the reinforced concrete building is given in the Bill of quantities table attached as appendix B and the summary of the costs is presented in Table 21.

**Table 21: Cost Evaluation Tables**

**a) Trial #1: 10% reduction in cost of cement and 10% increase in cost of brick units**

No.	Cost of Cement per Quintal	Cost of C-25 Concrete per cubic meter	Cost of Lean Concrete per square meter	Total RC Building Cost	Cost of Bricks per unit	Cost of 25cm Brick wall	Cost of Mortar per cubic meter	Total RM Building Cost
1	91.00	949.40	28.96	842,046.27	2.43	522.05	579.11	571,170.99
2	100.00	994.07	30.25	848917.74	2.20	491.94	604.99	569,912.50
3	110.00	1043.69	31.69	856552.71	2.00	466.51	633.74	570,779.88
4	120.00	1093.32	33.12	864187.68	1.82	443.95	662.50	572,462.91
5	131.00	1147.90	34.71	872586.15	1.66	424.63	694.13	575,864.00
6	144.00	1212.42	36.58	882511.61	1.51	407.43	731.51	584,373.77
7	158.00	1281.89	38.59	893200.57	1.37	392.01	771.77	588,401.63
8	173.00	1356.33	40.75	904653.03	1.24	378.37	814.90	596,635.78
9	190.00	1440.69	43.19	917632.48	1.13	368.31	863.78	607,490.38
10	208.00	1530.02	45.78	931375.43	1.03	360.03	915.54	619,655.20
11	228.00	1629.27	48.65	946645.37	0.94	353.15	973.05	633,828.72
12	250.00	1738.44	51.82	963442.30	0.85	348.39	1036.31	650,214.86

**b) Trial #2: 10% reduction in cost of cement and 20% increase in cost of brick units**

No.	Cost of Cement per Quintal	Cost of C-25 Concrete per cubic meter	Cost of Lean Concrete per square meter	Total RC Building Cost	Cost of Bricks per unit	Cost of 25cm Brick wall	Cost of Mortar per cubic meter	Total RM Building Cost
1	91.00	949.40	28.96	842,046.27	6.32	1082.54	579.11	729,635.55
2	100.00	994.07	30.25	848917.74	5.26	933.78	604.99	694,829.30
3	110.00	1043.69	31.69	856552.71	4.39	810.66	633.74	668,080.07
4	120.00	1093.32	33.12	864187.68	3.65	708.63	662.50	647,293.05
5	131.00	1147.90	34.71	872586.15	3.05	624.51	694.13	632,376.91
6	144.00	1212.42	36.58	882511.61	2.54	555.73	731.51	626,302.04
7	158.00	1281.89	38.59	893200.57	2.12	499.48	771.77	618,787.77
8	173.00	1356.33	40.75	904653.03	1.76	453.75	814.90	617,947.26
9	190.00	1440.69	43.19	917632.48	1.47	417.18	863.78	621,307.60
10	208.00	1530.02	45.78	931375.43	1.22	387.43	915.54	627,403.93
11	228.00	1629.27	48.65	946645.37	1.02	365.41	973.05	637,295.26
12	250.00	1738.44	51.82	963442.30	0.85	348.39	1036.31	650,214.86

c) Trial #3: 10% reduction in cost of cement and 25% increase in cost of brick units\*

No.	Cost of Cement per Quintal	Cost of C-25 Concrete per cubic meter	Cost of Lean Concrete per square meter	Total RC Building Cost	Cost of Bricks per unit	Cost of 25cm Brick wall	Cost of Mortar per cubic meter	Total RM Building Cost
1	91.00	949.40	28.96	<b>842,046.27</b>	9.90	1598.92	579.11	<b>875,627.09</b>
2	100.00	994.07	30.25	<b>848917.74</b>	7.92	1316.51	604.99	<b>803,036.97</b>
3	110.00	1043.69	31.69	856552.71	6.33	1091.54	633.74	747,491.58
4	120.00	1093.32	33.12	864187.68	5.07	912.25	662.50	704,860.05
5	131.00	1147.90	34.71	872586.15	4.05	769.83	694.13	673,462.02
6	144.00	1212.42	36.58	882511.61	3.24	657.34	731.51	655,029.71
7	158.00	1281.89	38.59	893200.57	2.59	568.57	771.77	638,319.57
8	173.00	1356.33	40.75	904653.03	2.08	498.85	814.90	630,697.41
9	190.00	1440.69	43.19	917632.48	1.66	446.83	863.78	629,690.52
10	208.00	1530.02	45.78	931375.43	1.33	403.03	915.54	631,813.58
11	228.00	1629.27	48.65	946645.37	1.06	371.54	973.05	639,028.53
12	250.00	1738.44	51.82	963442.30	0.85	348.39	1036.31	650,214.86

\*It is shown, by bold texts, in the table that even break point is obtained between those costs.

The graphs of cost of concrete and brick units verses total cost of building are attached in appendix B and the even break point can be seen in the graph for the third trial.

### 5.3. Result and Discussion

- The shape and size of the selected building satisfies the criteria set by Ethiopian code of practice for simplicity of buildings.
- Reinforced masonry construction is recommended for repetitive type building structures. And the selected building satisfies this condition.
- The storey numbers is set to G+4 to comply with the EBCS-8, 1995 specification, that recommend to have 5 stories in seismic zone 2 regions.
- The steel grade used is 420Mpa and this is taken to be similar with the already specified steel of reinforced masonry building.
- The result of the design, done and submitted to the housing agency, by consultants for the T-18 type condominium is taken without modification.
- Building components that are similarly used for the two types of construction are purposely left out from the comparison because the relative cost difference will be zero.

- The left out components include the slab, the roof, the façade walls with doors and windows, finishing works, sanitary installation, electrical installation and common partitions. The structural walls are modeled as stick elements using SAP2000 and designed, with analysis result from SAP2000, using a licensed and verified design software called GaLa Reinforcement.
- The foundations are designed, first, by proportioning bottom area to allow the soil to support the vertical load safely and then check is made for bending and wide beam shear.
- The study shows a cost reduction result, when the proposed building is constructed using reinforced masonry as a main structural element as compared to the reinforced concrete option. This fashion continues up to the break even point which is obtained when the cost of cement is 94.60 Birr per quintal and the cost of brick unit is 9.10 Birr. If the cost of brick units goes up from 9.10 Birr, masonry option will no more be cost effective and the reinforced concrete alternative becomes economical choice.
- The reduction in cost of cement is assumed not be less than 91 Birr per quintal for practical reasons. And three trials of cost comparison are performed, as shown in Appendix B, and break even was obtained when the cost reduction scheme of cement is 10% and cost increment of unit bricks is 25% with total building cost of 844,794.62 Birr.
- In the first trail a 10% cost reduction of cement per quintal is assumed and 10% cost increment in brick unit. The result was that it failed to produce break even in the practical cost ranges. In the second trail the cost of bricks is assumed to increase with 20% and it showed a sign of approaching the graph for concrete building but still the cost didn't break even. In the third trail the cost of bricks is assumed to increase with 25 % and it successfully produced a break even point as can be seen in the graph attached with appendix B.

## 6. CONCLUSIONS AND RECOMMENDATIONS

### 6.1. Conclusions

From the study the following points are concluded.

- Use of masonry in engineered way magnifies the scope of using masonry units from just retaining structures to main structural component including overlapping benefits of partitioning, load resisting, sound insulation, and better aesthetics.
- By reinforcing masonry cost of construction of building structures can be reduced, even below the accepted low cost house system of our country.
- Obtaining a cost effective construction system with reinforced masonry can be extrapolated to a case of obtaining a better seismic resistant masonry building structure with the cost involved for concrete buildings.

### 6.2. Recommendations

The following are recommendations from the Study;

- It is cost effective, with the current market price, using reinforced Masonry for simple and regular building structures like that of condominium houses than reinforced concrete.
- The advantages of less cement requirement in masonry construction should exhaustively be utilized in regions that suffer from shortage of cement.
- Regulatory bodies should ask consultants to come up with alternative designs using masonry.
- Stake holders in the educational institutions should delve for further advancement in the knowledge and practice of reinforced masonry and curriculums be devised in such a manner that masonry is treated, as one construction material, at college or university levels.
- Budgets need to be allocated, preparation of more refined and practical code of standard accomplished and chances opened so that engineers, who stick to concrete, join and see the possible potentials of masonry usage for building construction as a main structural element.

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## **APPENDICES**

### **A. Design Samples on Reinforced Masonry Building**

### **B. Cost Comparison and Break Even Point**

## **DECLARATION**

I, the undersigned, declare that this thesis is my original work and has not been presented for a degree in any other university. All sources of materials used for the thesis have been duly acknowledged.

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