

(Final Draft for Revision)

**GUIDELINES FOR
EARTHQUAKE RESISTANT
NON-ENGINEERED CONSTRUCTION**

Anand S. ARYA, Teddy BOEN and Yuji ISHIYAMA

February 2012

International Association for Earthquake Engineering (IAEE),
United Nations Educational, Scientific and Cultural Organization (UNESCO)
and
International Institute of Seismology and Earthquake Engineering (IISEE)

Preface

The first edition of the book was published by the International Association for Earthquake Engineering (IAEE) in 1986. It consisted in a revised and amplified version of the original document, “Basic Concepts of Seismic Codes, Vol. 1, Part II, Non-Engineered Construction”, published also by IAEE in 1980. The revision resulted from the work of ad-hoc committee, integrated by Anand S. Arya, Chairman (India), Teddy Boen (Indonesia), Yuji Ishiyama (Japan), A. I. Martemianov (USSR), Roberto Meli (Mexico), Charles Scawthorn (USA), Julio N. Vargas (Peru) and Ye Yaoxian (China). published in October 1986. The Preface of the Guidelines had stated the following important points:

The non-engineered buildings considered in the book are those which are spontaneously and informally constructed in various countries in the traditional manner without any or little intervention by qualified architects and engineers in their design. Such buildings involve field stone, fired brick, concrete blocks, adobe or rammed earth, wood or a combination of these traditional locally available materials in their construction. Cement and lime are sometimes used for the mortars. Reinforced concrete lintels and floor and roof slabs and beams are also being increasingly used. In some cases, use of reinforced concrete columns and beams is also made, particularly for shopping centres and school buildings but here also a post-beam type simple concept is frequently adopted in a non-engineered manner without consideration of the stability of the system under horizontal seismic forces. This book aims at explaining the lessons learned from the seismic performance of such buildings during past earthquakes and the basic concepts involved in their aseismic design and construction, restoration and strengthening, and laying down guidelines for their safe planning and construction.

The safety of non-engineered buildings from the fury of earthquakes is a subject of highest priority in view of the fact that in the moderate to severe seismic zones of the world more than 90 percent of the population is still living and working in such buildings, and that most losses of lives during earthquakes have occurred due to their collapse. The risk to life is further increasing due to rising population, particularly in the developing countries, poverty of the people, scarcity of modern building materials, viz. cement and steel, lack of awareness and necessary skills. In view of these factors, the committee felt that:

- a) revolutionary change in the construction pattern is not feasible nor practical;
- b) the use of local materials will continue to be made with only a minimum amount of cement and steel where absolutely necessary, and
- c) very simple modifications to traditional building systems need to be recommended which can be easily understood and adopted by the local artisans.

The material included in the book should be useful to people at various levels concerned with earthquake disaster mitigation through the construction of safe housing for and by the

masses. For making this information more easily available to their people, the local governments, societies and institutions are encouraged to prepare its translations without changing its contents including cover pages in the national and local languages. For this purpose no royalty is to be paid and only due acknowledgement is to be given to this book and the International Association for Earthquake Engineering.

Since its publication, the Guideline has been used extensively by being reproduced locally or nationally and translated into Hindi in India, and Spanish by PAHO for use in Central, and South American Countries.

This revised edition essentially retains the Guidelines in the original form except for some minor editorial changes and modifications in the data tables. A few building damage photographs from recent earthquakes have been included for illustration. A major addition is Confined Masonry in Chapter 4 and Appendices in Chapter 10 giving the MSK Intensity Scale as related to buildings, a table for assessment of seismic safety of a masonry building, and examples of posters on brick and wooden buildings.

Acknowledgement The revision for 2012 edition was financially supported by the UNESCO IPRED (International Platform for Reducing Earthquake Disasters) programme and peer-reviewed by its members. The contributions for the revision from experts: Dr. Tatsuo Narafu, Japan, Dr. Osamu Joh, Japan, and Mr. Andrew Charleson, New Zealand are hereby acknowledged.

February 2012

Anand S. Arya

Teddy Boen

Yuji Ishiyama



Meeting for 2012 edition
in Singapore, March 2011



Meeting for 1986 edition during 8WCEE
in San Francisco, July 1984

If you have questions, suggestions, or comments on this draft, please e-mail to
Anand S. ARYA, asarun3155@gmail.com
Teddy BOEN, tedboen@cbn.net.id
Yuji ISHIYAMA, to-yuji@nifty.com

Contents

Preface	ii
1 The Problem, Objective and Scope	1
1.1 The Problem	1
1.2 Socio-Economic Considerations in Seismic Safety	1
1.3 Objective and Scope	2
2 Structural Performance during Earthquakes	3
2.1 Introduction	3
2.2 Earthquake Effects	4
2.2.1 Ground shaking	4
2.2.2 Ground failure	4
2.2.3 Tsunamis	5
2.2.4 Fire	6
2.3 Ground Shaking Effects on Structures	7
2.3.1 Inertia forces	7
2.3.2 Seismic load	7
2.3.3 Factors affecting the seismic coefficient	9
2.3.4 Nature of seismic stresses	9
2.3.5 Important parameters in seismic design	10
2.4 Effect of site conditions on building damage	10
2.5 Other Factors Affecting Damage	11
2.5.1 Building configuration	11
2.5.2 Opening size	11
2.5.3 Stiffness distribution	11
2.5.4 Ductility	11
2.5.5 Strength of Buildings	12
2.5.6 Foundations	12
2.5.7 Construction quality	12
2.6 Failure Mechanisms of Structures	12
2.6.1 Free-standing masonry wall	12
2.6.2 Wall enclosure without roof	13
2.6.3 Roof on two walls	15
2.6.4 Roof on wall enclosure	15
2.6.5 Roofs and floors	16

2.6.6	Long buildings with roof trusses	17
2.6.7	Shear wall with openings	17
2.7	Earthquake Damage Categories	18
3	General Concepts of Earthquake Resistant Design	20
3.1	Introduction	20
3.2	Categories of Buildings	21
3.2.1	Seismic zones	21
3.2.2	Importance of building	22
3.2.3	Classification of foundation soil	22
3.2.4	Combination of parameters	22
3.3	General Planning and Design Aspects	23
3.3.1	Plan of building	23
3.3.2	Choice of site	25
3.3.3	Structural design	25
3.3.4	Fire resistance	25
3.4	Structural Framing	27
3.5	Requirements of Structural Safety	27
3.6	Concepts of Ductility, Deformability and Robustness	28
3.6.1	Ductility	28
3.6.2	Deformability	28
3.6.3	Robustness	28
3.7	Concept of Isolation	29
3.8	Foundations	29
3.8.1	Hard and medium soils	30
3.8.2	Soft soil	30
4	Masonry Buildings in Fired-Brick and Other Materials	31
4.1	Introduction	31
4.2	Typical Damage and Failure of Masonry Buildings	31
4.2.1	Non-structural damage	31
4.2.2	Damage and failure of bearing walls	32
4.2.3	Failure of ground	34
4.2.4	Failure of roofs and floors	34
4.2.5	Causes of damage in masonry buildings	35
4.3	Typical Strengths of Masonry	35
4.4	General Construction Aspects	38
4.4.1	Mortar	38
4.4.2	Wall enclosure	39
4.4.3	Openings in walls	39
4.4.4	Masonry bond	40
4.5	Horizontal Reinforcement in Walls	41
4.5.1	Horizontal bands or ring beams	41
4.5.2	Cross-sections of bands or ring beams	41
4.5.3	Dowels at corners and junctions	41

4.6	Vertical Reinforcement in Walls	43
4.7	Framing of Thin Load Bearing Walls	44
4.8	Reinforcing Details for Hollow Block Masonry	45
4.8.1	Horizontal bands	45
4.8.2	Vertical reinforcement	46
4.9	Confined Masonry	48
4.9.1	Understanding confined masonry construction	48
4.9.2	Difference between confined masonry and RC frame construction	50
4.9.3	Guide to earthquake-resistant confined masonry construction	50
4.9.4	Construction details of confined masonry	52
4.9.5	Concluding remarks	54
4.10	Foundation and Plinth	54
5	Stone Buildings	57
5.1	Introduction	57
5.2	Typical Damage and Failure of Stone Buildings	57
5.3	Typical Structural Properties	59
5.4	General Construction Aspects	59
5.4.1	Overall dimensions	59
5.4.2	Mortar	59
5.4.3	Openings in walls	60
5.4.4	Masonry bond	60
5.4.5	Horizontal reinforcing of walls	60
5.4.6	Vertical reinforcing of walls	61
5.4.7	Foundations	62
6	Wooden Buildings	64
6.1	Introduction	64
6.2	Typical Damage and Failure	64
6.3	Typical Characteristics of Wood	66
6.4	Typical Structural Properties	68
6.5	The Building Plan	69
6.6	Stud Wall Construction	69
6.7	Brick Nogged Timber Frame	71
6.8	Joints in Wood Frames	72
6.9	Foundations	73
7	Earthen Buildings	75
7.1	Introduction	75
7.2	Typical Damage and Collapse of Earthen Buildings	75
7.3	Classification of Walls and Material Properties	75
7.3.1	Classification of earthen construction	76
7.3.2	Suitability of soil	77
7.3.3	Strength test of adobe	78
7.4	Construction of Walls	78

7.4.1	Hand-moulded layered construction	79
7.4.2	Adobe or block construction	79
7.4.3	Tapial or pise construction	80
7.4.4	Earthen construction with wood or cane structure	81
7.5	General Recommendations for Seismic Areas	82
7.5.1	Walls	82
7.5.2	Foundations	83
7.5.3	Roofing	85
7.6	Seismic Strengthening Features	86
7.6.1	Collar beam or horizontal band	86
7.6.2	Pillasters and buttresses	86
7.6.3	Vertical reinforcement in walls	87
7.6.4	Diagonal bracing	88
7.7	Plastering and Painting	88
7.8	Summary of Desirable Features	89
7.9	Working Stresses	90
7.9.1	Unit compressive strength	90
7.9.2	Compressive strength of earthen construction	90
7.9.3	Shear strength of earthen construction	91
7.9.4	Permissible tensile strength of masonry for loads perpendicular to its plane (f_a)	91
8	Non Engineered Reinforced Concrete Buildings	92
8.1	Introduction	92
8.2	Typical Damage and Collapse of RC Buildings	92
8.3	Care in Concrete Construction	95
8.4	Material Properties	98
8.4.1	Concrete	98
8.4.2	Reinforcing steel	99
8.5	Critical Zones for Ductility	99
8.6	Detailing of Beams	100
8.7	Detailing of Columns	100
8.8	Connections	102
8.9	Illustrative Sketches	102
9	Repair, Restoration and Strengthening of Buildings	105
9.1	Introduction	105
9.2	Repair, Restoration and Strengthening Concepts	106
9.2.1	Repairs	106
9.2.2	Restoration	106
9.2.3	Strengthening of existing buildings	107
9.3	Repair Materials	108
9.3.1	Shotcrete	108
9.3.2	Polymer resins	108
9.3.3	Polymerised mortar	108

9.3.4	Quick-setting cement mortar	108
9.3.5	Mechanical anchors	109
9.4	Retrofitting Steps	109
9.5	Techniques to Restore Original Strength	109
9.5.1	Small cracks	110
9.5.2	Wide cracks and crushed concrete	111
9.5.3	Fractured, excessively yielded and buckled reinforcement	112
9.5.4	Fractured wooden members and joints	113
9.6	Retrofitting of Roofs or Floors	113
9.7	Substitution or Strengthening of Slabs	115
9.8	Improving Structural Layout and Strengthening of Walls	118
9.8.1	Inserting new walls	118
9.8.2	Strengthening existing walls	119
9.8.3	External binding	123
9.8.4	Other points	123
9.9	Strengthening RC Members	125
9.10	Strengthening of Foundations	128
10	Appendices	130
10.1	MSK Intensity Scale (as related to buildings)	130
10.2	Assessment of Seismic Safety for Masonry Buildings	132
10.3	Examples of Posters	132
Index		136

Chapter 1

The Problem, Objective and Scope

1.1 The Problem

Most of the loss of life in past earthquakes has occurred due to the collapse of buildings, constructed in traditional materials like stone, brick, adobe and wood, which were not initially engineered to be earthquake resistant. In view of the continued use of such buildings in most countries of the world, it is essential to introduce earthquake resistance features in their construction.

1.2 Socio-Economic Considerations in Seismic Safety

From the results of studies on the performance of buildings during past earthquakes, it appears that:

- a) certain building types should entirely be ruled out in seismic zones having probable seismic intensity of VIII or more on the Modified Mercalli or the MSK Intensity Scales (see Sec.10.1 in Appendices). This would include earthen houses, random rubble masonry as well as brickwork in clay mud mortar, and the like;
- b) rich mortars involving cement and lime should be used in fired brick and coursed stone masonry; and
- c) substantial steel reinforcement should be introduced in the walls in both (horizontal and vertical) directions.

But there are a number of socio-economic constraints such as the following which do not permit the adoption of high level of safety in the buildings for the masses:

- 1) lack of concern about seismic safety due to the infrequent occurrence of earthquakes;
- 2) lack of awareness that buildings could be made earthquake resistant at a small additional cost only, hence lack of motivation;
- 3) lack of financial resources for meeting earthquake resistance requirements in building construction;
- 4) other normal priorities on financial aspects of the daily life of the people;
- 5) scarcity of cement, steel as well as timber in developing countries in general; and

- 6) lack of skill in seismic design and construction techniques and the unorganised nature of the building sector.

Such considerations therefore compel the continued use of seismically unsuitable construction practices.

While theoretically, if appropriate resources and building materials are made available, it may be possible to construct buildings which can withstand the effects of earthquakes without **any** appreciable damage, practically it is not feasible to do so due to the costs involved. From the safety view point, the safety of human lives is the primary concern and the functioning of the buildings has lower priority except the buildings required for community activities such as schools, assembly halls, places of worship, and cinema halls, etc., and those required for an emergency, such as, buildings for hospitals, power stations, communication, fire fighting and the like. The safety aims would therefore be met, if a building is designed and constructed in such a way that even in the event of the probable maximum earthquake intensity in the region,

- i) an ordinary building should not suffer total or partial collapse;
- ii) it should not suffer such irreparable damage which would require demolishing and re-building;
- iii) it may sustain such damage which could be repaired quickly and the building put back to its usual functioning; and
- iv) the damage to an important building should even be less so that the functioning of the activities during a post-emergency period may continue unhampered and the community buildings may be used as temporary shelters for the adversely affected people.

The present state of research indicates that fortunately the above structural safety can be achieved by adopting appropriate design and construction details involving only **small** extra expenditure which should be within the economic means of people in most countries.

1.3 Objective and Scope

The objective of this book is to deal with the basic concepts involved in achieving appropriate earthquake resistance of such buildings as stated above, collectively called “Non-Engineered Buildings”; to include suitable illustrations to explain the important points, and to present such data which could be used to proportion the critical strengthening elements. The term non-engineered building is defined as buildings which are spontaneously and informally constructed in the traditional manner without intervention by qualified architects and engineers in their design, but may follow a set of recommendations derived from observed behaviour of such buildings during past earthquakes and trained engineering judgement. Specifically, such buildings include load bearing masonry wall buildings, stud-wall and brick-nogged constructions in wood, and composite constructions using combinations of load bearing walls and piers in masonry, tie columns and tie-beams of reinforced concrete, or steel, wood, and the like.

Reinforced masonry, reinforced concrete or steel frame buildings, tall buildings using various types of structural systems, and major industrial buildings, etc., are excluded from consideration although some of the principles stated herein will equally apply to these constructions.

Chapter 2

Structural Performance during Earthquakes

2.1 Introduction

Earthquakes are natural hazards under which disasters are mainly caused by damage to or collapse of buildings and other man-made structures. Experience has shown that for new construction, establishing earthquake resistant regulations and their implementation is the critical safeguard against earthquake-induced damage. As regards existing structures, it is necessary to evaluate and strengthen them before an earthquake based on evaluation criteria.

Earthquake damage depends on many parameters, including earthquake ground motion characteristics (intensity, duration and frequency content of ground motion), soil characteristics (topography, geologic and soil conditions), building characteristics, and quality of construction, etc. Building design must be such as to ensure that the building has adequate strength, high ductility, and will remain as one integral unit, even while subjected to very large ground motions.

Social and other factors are also important, such as density of population, time of day of the earthquake occurrence and community preparedness for the possibility of such an event.

Up to now we could do little to diminish direct earthquake effects. However we can do much to reduce risks and thereby reduce disasters provided we design and build or strengthen the buildings so as to minimize losses based on the knowledge of the earthquake performance of different building types during an earthquake.

Observation of the structural performance of buildings during an earthquake can clearly identify the strong and weak aspects of designs, as well as the desirable qualities of materials and techniques of construction, and site selection. The study of damage therefore provides an important step in the evolution of strengthening measures for different types of buildings.

This Chapter discusses the earthquake performance of structures, earthquake intensity, ground shaking effects on structures, site condition effects on building damage, other factors affecting damage, failure mechanisms of structures, earthquake damage and damage categories. Typical patterns of damage for specific types of construction are discussed in the respective chapters.

2.2 Earthquake Effects

There are four basic causes of earthquake-induced damage: ground shaking, ground failure, tsunamis and fire. However, the main cause of damage is the **ground shaking**. Others (ground failure, tsunamis and fire) are secondary disasters. Therefore, the main priority is making earthquake resistant structures.

2.2.1 Ground shaking

As mentioned above, the principal cause of earthquake-induced damage is ground shaking (see Fig. 2.1). As the earth vibrates, all buildings on the ground surface respond to that vibration in varying degrees. Earthquake induced accelerations, velocities and displacements can damage or destroy a building unless it has been designed and constructed or strengthened to be earthquake resistant. Therefore, the effect of ground shaking on buildings is a principal area of consideration in the design of earthquake resistant buildings. Seismic design loads are extremely difficult to determine due to the random nature of earthquake motions. However, experiences from past strong earthquakes have shown that reasonable and prudent practices can keep a building safe during an earthquake.



2009 West Java Earthquake, Indonesia



2009 West Sumatra Earthquake, Indonesia

Figure 2.1: Damage caused by ground shaking

2.2.2 Ground failure

Earthquake-induced ground failure has been observed in the form of ground rupture along the fault zone, landslides, settlement and soil liquefaction. Ground rupture along a fault zone may be very limited or may extend over hundreds of kilometers. Ground displacement along the fault may be horizontal, vertical or both, and can be measured in centimeters or even metres. Obviously, a building directly astride such a rupture will be severely damaged or collapsed.

While a landslide can destroy a building, settlement may only damage it (see Fig. 2.2).

Soil liquefaction can occur in low density saturated sands of relatively uniform grain size. The phenomenon of liquefaction is particularly important for dams, bridges, underground pipelines, and buildings standing on such ground.



1978 Izu-Ohshima-kinkai Earthquake, Japan



1993 Kushiro-oki Earthquake, Japan



2005 Nias Earthquake, Indonesia



2007 West Sumatra Earthquake, Indonesia

Figure 2.2: Damage caused by ground failure

2.2.3 Tsunamis

Tsunamis or seismic sea waves are generally produced by a sudden movement of the ocean floor. As the water waves approach land, their velocity decreases and their height (run-up height) increases to 5 to 8 m, or even more. Obviously, tsunamis can be devastating for buildings built in coastal areas (see Fig. 2.3). A huge tsunami occurred in Aceh, Indonesia, on December 26, 2004, killing more than 200,000 people.

The conditions for a tsunami to occur are as follows:

- under-sea earthquake – reverse or normal fault,
- shallow earthquake, and
- Magnitude greater than 6.5

The tsunami destruction is greater if the beach is slightly sloped.



1993 Hokkaido-nansei-oki Earthquake, Japan
(Tsunami run-up height reached 15 m average and 31 m max. and all houses were swept away.)



1992 Flores Earthquake, Indonesia



2004 Ache Earthquake, Indonesia



2011 Great East Japan Earthquake

Figure 2.3: Damage caused by tsunamis

2.2.4 Fire

When a fire following an earthquake starts, it becomes difficult to extinguish it, since a strong earthquake is often accompanied by the loss of water supply and traffic jams. Therefore,

an earthquake-induced fire adds to the damage to buildings directly due to earthquakes (see Fig. 2.4). In the case of the 1923 Great Kanto Earthquake, Japan, 50 % of Tokyo and 70 % of the total number of houses were burnt, and more than 100,000 people were killed by the fire.



1995 Hyogo-ken-nanbu Earthquake, Japan



2009 West Sumatra Earthquake, Indonesia

Figure 2.4: Damage caused by fire

2.3 Ground Shaking Effects on Structures

2.3.1 Inertia forces

A building sits on the ground as shown in Fig. 2.5. As the base of the building moves, the superstructure including its contents tends to shake and vibrate in a very irregular manner due to the inertia of the masses.

When the ground motion suddenly moves the base of the building to the left, the building moves to the right relative to the base (see, Fig. 2.5), as if it was being pushed to the right by an unseen force which we call “**Inertia Force**”. Actually, there is no push at all but, because of its mass, the building resists any motion. Every part of the building has to sustain inertia forces. The process is much more complex than the simple explanation above because the ground moves simultaneously in two horizontal and vertical directions during an earthquake.

2.3.2 Seismic load

The resultant lateral force or seismic load is represented by the force F in Fig. 2.6. It is distinctly different from dead, live, snow, wind and impact loads. The horizontal ground motion action is similar to the effect of a horizontal force acting on the building, hence the term “Seismic Load” or “Lateral Load” is used. As the base of the building moves in an extremely complicated manner, inertia forces are created throughout the mass of the building and its contents. It is these reversible forces that cause the building to move and sustain damage or collapse.

An additional and uplift vertical load effect is caused on slabs, beams, cantilevers and columns due to vertical vibrations, which may cause damage. Being reversible, at certain instants of time the effective load is increased, at others it is decreased.

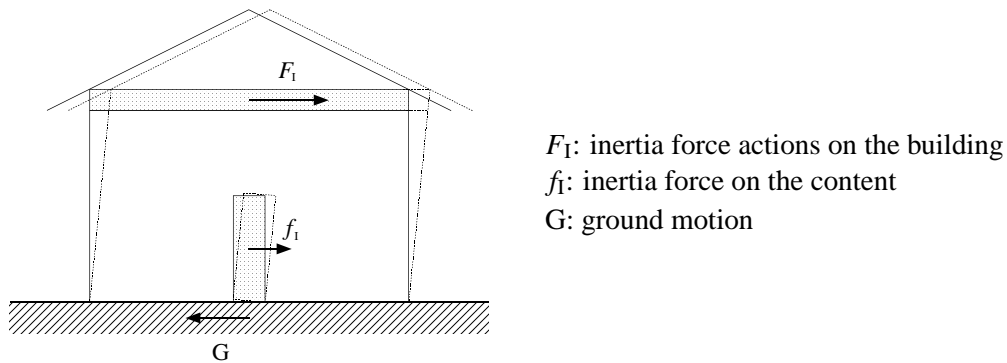


Figure 2.5: Inertia forces caused by the earthquake ground motion

Earthquake loads are dynamic and impossible to predict precisely in advance since every earthquake exhibits different characteristics. The following equivalent lateral force F , that is used for seismic design, is expressed as the product of the mass of the structure m and the acceleration a , or the seismic coefficient k and the weight of the structure W :

$$F = m a = k W \quad (2.1)$$

W is the total weight of the super-structure of a building including its contents. The inertia forces are proportional to the mass (or weight) of the building and only building elements or contents that possess mass will give rise to seismic force on the building. Therefore, the lighter the material, the smaller will be the seismic force.

The seismic coefficient k is the function of various factors and it is usually determined as follows:

$$k = A_g S D I \quad (2.2)$$

where, A_g , S , D and I are the factors affecting seismic coefficient, which are explained in the following section.

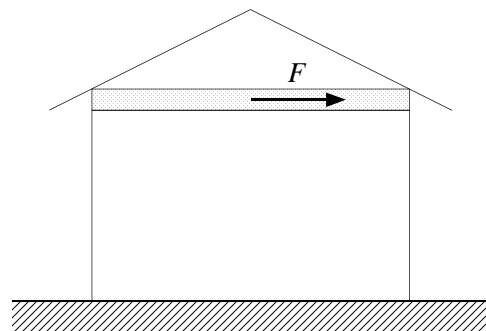


Figure 2.6: Equivalent seismic force F

2.3.3 Factors affecting the seismic coefficient

The normalized design acceleration A_g is the design ground acceleration divided by the acceleration due to gravity. It may be expressed as the product of the seismic zoning factor Z and the normalized standard ground acceleration a_g of a seismic region (zone) of a country. The seismic zoning factor Z depends upon the relative ground intensity of the earthquake, and it is usually plotted on maps in terms of seismic intensity isoseismal lines or maximum acceleration contours. Obviously, the higher the intensity or acceleration, the larger will be the seismic force.

The normalized design response spectrum S depends mainly upon the fundamental natural period of vibration of a building T and the soil profile (see Fig. 2.7).

The structural factor D is a factor depending on the ductility and damping of the structure. The larger the ductility of the structure, the more energy the structure can absorb and smaller the value of D . Damping is the energy dissipation property of the building; the larger the damping, the smaller the value of D .

The occupancy importance factor or hazard factor I depends upon the usage of the building. The higher the importance or larger the hazard caused by the failure of the building, the greater the value of the factor I .

In some recent building codes, the term “occupancy importance” factor has changed to “risk” factor. The term “occupancy” as used by older building codes relates primarily to issues associated with fire and life safety protection, as opposed to the risks associated with structural failure. The term “Risk Category” was adopted in place of the older Occupancy Category to distinguish between these two considerations.

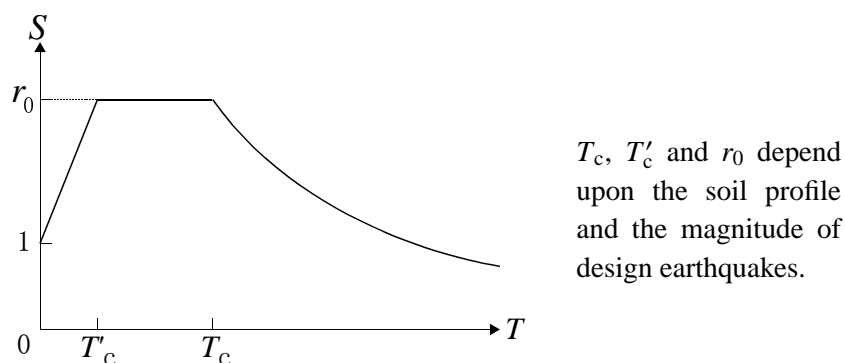


Figure 2.7: Normalized design response spectrum S

2.3.4 Nature of seismic stresses

Horizontal seismic forces are reversible in direction. Structural elements such as walls, beams and columns that were bearing only vertical loads before the earthquake, have now to carry horizontal bending and shearing effects as well. When the bending tension due to an earthquake exceeds the vertical compression, net tensile stress will occur. If the building material is weak in tension such as brick or stone masonry, cracking occurs which reduces the effective area for resisting bending moment, as shown in Fig. 2.8. It follows that the building material strength in tension and shear is important for earthquake resistance.

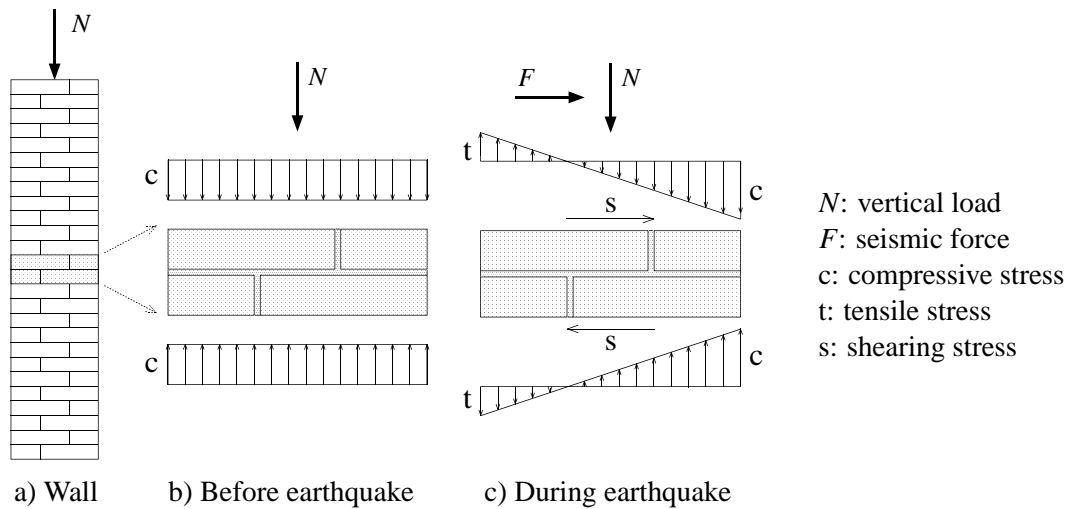


Figure 2.8: Stress condition in a wall element

2.3.5 Important parameters in seismic design

The following properties and parameters are most important from the point of view of seismic design.

- a) Building material properties
 - Strength in compression, tension and shear, including dynamic effects
 - Unit weight (density)
 - Modulus of elasticity
- b) Dynamic characteristics of the building system, including periods, modes of vibration and damping.
- c) Load-deflection characteristics of building components.

2.4 Effect of site conditions on building damage

Past earthquakes show that site conditions significantly affect building damage. Earthquake studies have almost invariably shown that the intensity of a shock is directly related to the type of soil layers supporting a building. Structures built on solid rock and firm soil frequently perform far better than buildings on soft ground. This was dramatically demonstrated in the 1985 Mexico Earthquake, where the damage on soft soils in Mexico City, at an epicentral distance of 400 km, was substantially higher than at closer locations.

From studies of the 1957 Mexico Earthquake, it was already known that the damage on the soft soils in the center of the city could be 5 to 50 times higher than on firmer soils in the surrounding area. Another example occurred in the 1976 Tangshan Earthquake, China in which 50% of the buildings on deep soil sites collapsed, while only 12% of the buildings on the rock subsoil near the mountain areas totally collapsed. Rigid masonry buildings resting on rock may on the contrary show more severe damage than when built on soft soil during a near earthquake, as in the 1967 Koyana Earthquake, India and the 1980 North Yemen Earthquake.

Lessons learned from recent earthquakes show that the topography of a building site can also have an effect on damage. Buildings on sites with flat and even topography are usually less damaged during an earthquake than buildings on ridges, in narrow valleys, and on steep slopes.

2.5 Other Factors Affecting Damage

The extent of damage to a building depends on the strength, ductility, and integrity of a building, and the stiffness of the ground beneath it in a given intensity of the earthquake motions.

Almost any building can be designed to avoid earthquake collapse provided its site is suitable. Buildings suffer damage during an earthquake primarily because horizontal forces are exerted on a structure that is often meant to contend only with vertical stresses. The principal factors that influence damage to buildings and other built structures are listed below:

2.5.1 Building configuration

An important feature is regularity and symmetry in the overall shape of a building. A building shaped like a box, rectangular both in plan and elevation, is inherently stronger than one L-shaped or U-shaped, such as a building with wings. An irregularly shaped building will twist as it shakes, increasing the damage.

2.5.2 Opening size

In general, openings in walls of a building tend to weaken the walls, and the fewer the openings the less the damage suffered during an earthquake. If it is necessary to have large openings or if an open first floor is desired, then special provisions should be made to ensure structural integrity.

2.5.3 Stiffness distribution

The horizontal stiffness of a building up its height should be uniform. Changes in the structural system of a building from one floor to the next will increase the potential for damage, and should be avoided. Columns or shear walls should run continuously from foundation to the roof, without interruptions or changes in material.

2.5.4 Ductility

Ductility is the ability of the building to bend, sway, and deform by large amounts without collapse. The opposite condition is brittleness, arising both from the use of materials that are inherently brittle and from the wrong design of structures using otherwise ductile materials. Brittle materials break under overload; some examples are adobe, brick and concrete blocks. It is not surprising that most damage during past earthquakes was to unreinforced masonry structures constructed of brittle materials, poorly tied together. The addition of steel reinforcement can add ductility to brittle materials. Concrete, for example, can be made ductile by proper use of reinforcing steel and closely spaced steel ties, e.g. hoops in columns and stirrups in beams.

2.5.5 Strength of Buildings

Structures of buildings should have adequate strength to resist earthquake shaking and particularly the “rocking” effect. “Rocking” can occur in rigid normal houses. Therefore, all buildings components, i.e. foundations, columns, beams, walls, roof trusses, roofing, etc. must be tied to each other, so that when shaken by earthquakes, each building will act as one integral unit (see Fig. 2.9).

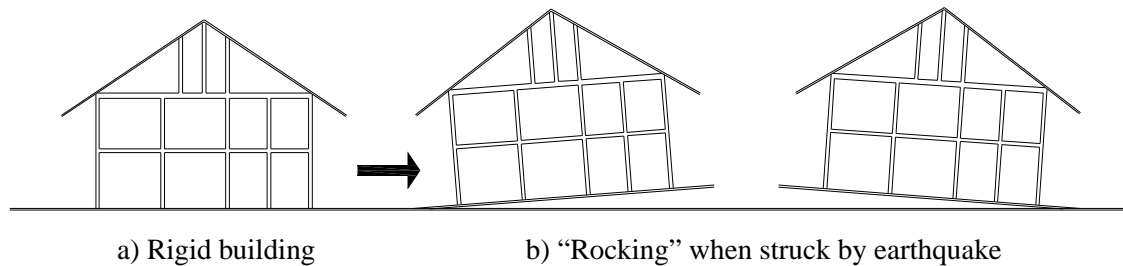


Figure 2.9: Illustration of “rocking” buildings

2.5.6 Foundations

Buildings which are structurally strong against earthquakes sometimes fail due to inadequate foundation design. Tilting, cracking and failure of superstructures may result from soil liquefaction and differential settlements of footings.

Certain types of foundations are more susceptible to damage than others. For example, isolated footings of columns are likely to be subjected to differential settlement particularly where the supporting ground consists of different or soft types of soil. Mixed types of foundations such as piles and footings within the same building may also lead to damage due to differential settlement.

Very shallow foundations deteriorate because of weathering, and when exposed to freezing and thawing in cold climate regions.

2.5.7 Construction quality

In many instances the failure of buildings in an earthquake has been attributed to poor quality of construction, substandard materials, poor workmanship, e.g., inadequate skill in bonding, absence of “through stones” or bonding units, and improper and inadequate construction.

2.6 Failure Mechanisms of Structures

2.6.1 Free-standing masonry wall

Consider the free-standing masonry walls shown in Fig. 2.10. In Fig. 2.10 a), the ground motion is acting transverse to a free-standing wall. The out-of-plane inertia force acting on the mass of the wall tends to overturn it. The seismic resistance of the wall is by virtue of its weight and

tensile strength of mortar. It is obviously very small. This wall will collapse by overturning under the ground motion (see Figs. 2.11 and 2.12).

The free-standing wall fixed on the ground in Fig. 2.10 b) is subjected to ground motion in its own plane. In this case, the wall will offer much greater resistance because of its large depth in the direction of the force and the plane of bending. Such a wall is termed a shear wall. The damage modes of an unreinforced shear wall depend on the height-to-length ratio or aspect ratio of the wall. A wall with large aspect ratio (see Fig. 2.10 b) will generally develop a horizontal crack at the bottom due to bending tension and then slide due to shearing. A wall with moderate aspect ratio and bounding frame cracks diagonally due to shearing as shown in Fig. 2.10 c).

A wall with small aspect ratio, on the other hand, may develop diagonal tension cracks at both sides and horizontal cracks at the middle as shown in Fig. 2.10 d).

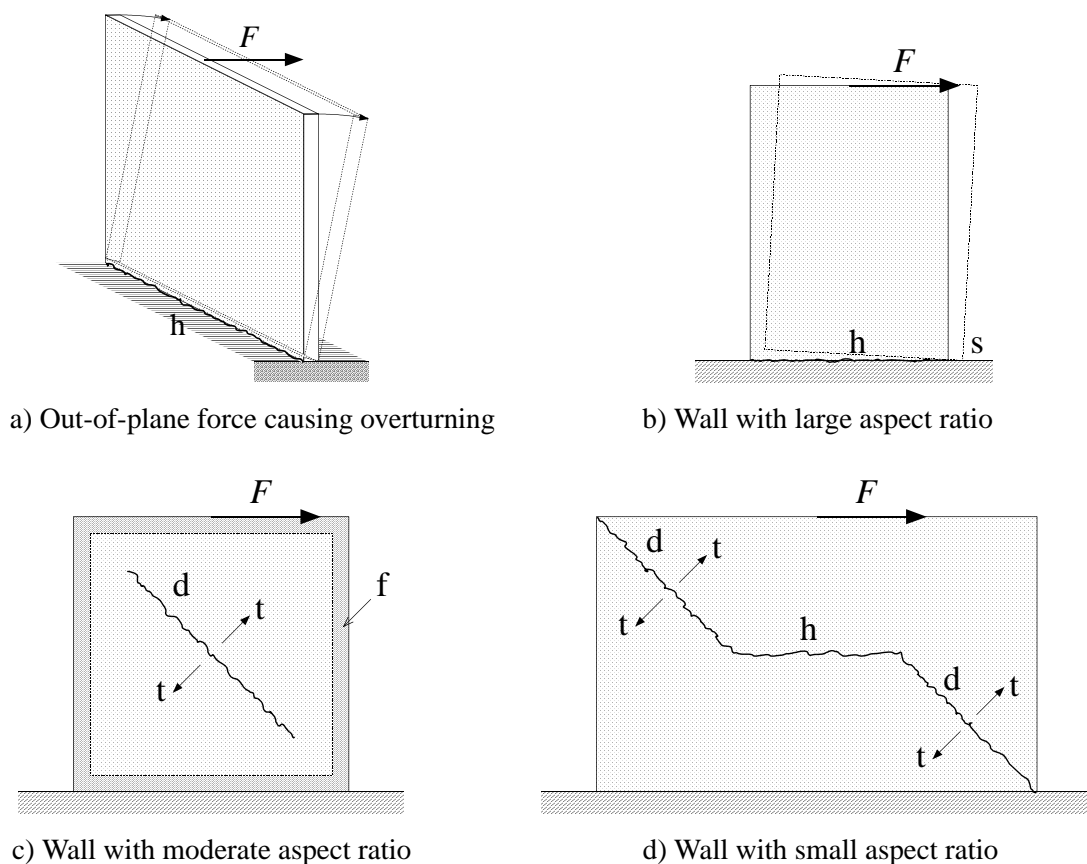


Figure 2.10: Failure mechanism of free-standing walls
 (F : earthquake force, d : diagonal cracking, f : frame,
 h : horizontal cracking, s : sliding, t : tensile stress)

2.6.2 Wall enclosure without roof

Now consider the combination of walls A and B as an enclosure shown in Fig. 2.13. For the earthquake force F as shown, walls B act as shear walls and, besides taking their own inertia, they offer resistance against the collapse of walls A as well. As a result walls A now act as

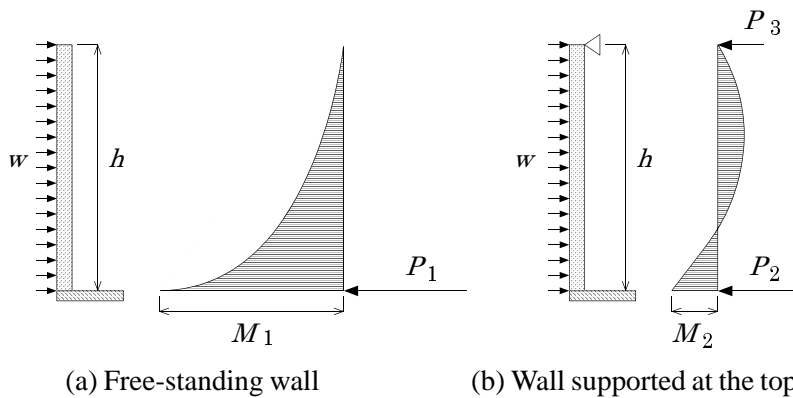


1994 Liwa Earthquake, Indonesia



2006 Central Java Earthquake, Indonesia

Figure 2.11: Out-of-plane failure of brick masonry wall



The free-standing wall (a) may overturn because of the bending moment M_1 at the base. The wall (b), supported at the top may not overturn nor be damaged because the top support reduces the bending moment at the base, i.e. $M_2 = \frac{1}{4}M_1$.

Figure 2.12: Out-of-plane bending moment of a wall fixed at its base

vertical slabs supported on two vertical sides and the bottom plinth. The walls A are subjected to the inertia force acting on their own mass. Near the vertical edges, the wall will carry reversible bending moments in the horizontal plane for which the masonry has little strength. Consequently cracking and separation of the walls may occur along these edges as shown in the figure.

It can be seen that in the action of walls B as shear walls, the walls A will act as flanges connected to the walls B acting as web. Thus if the connection between walls A and B is not lost due to a lack of bonding action, the building will tend to act as a box and its resistance to horizontal loads will be much larger than that of walls B acting separately. Most unreinforced masonry enclosures, however, have very weak vertical joints between walls meeting at right angles due to the construction procedure involving toothed joints that are generally not properly filled with mortar. Consequently the corners fail and lead to collapse of the walls. It may also be easily imagined that the longer the walls in plan, the smaller will be the support to them from the cross walls and the lesser will be the box effect.

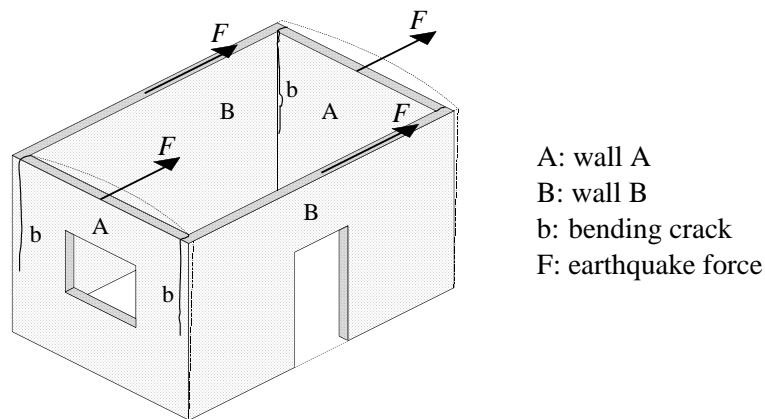


Figure 2.13: Failure mechanism of wall enclosure without roof

2.6.3 Roof on two walls

In Fig. 2.14 the roof slab is shown to be resting on two parallel walls B and the earthquake force is acting in the plane of the walls. Assuming that there is enough adhesion between the slab and the walls, the slab will transfer its inertia force at the top of walls B, causing shearing and overturning action in them. To be able to transfer its inertia force to the two side walls, the slab must have enough strength in bending in the horizontal plane. This action of the slab is known as diaphragm action. Reinforced concrete or reinforced brick slabs have enough strength to inherently and act as rigid diaphragms. However, other types of roofs or floors such as timber or reinforced concrete joists with brick tile covering will be very flexible. Any joists have to be connected together and fixed to the walls suitably so that they are able to transfer their inertia force to the walls. At the same time, the walls B must have enough strength as shear walls to withstand the force from the roof and their own inertia forces. Obviously, the structure shown in Fig. 2.14, when subjected to ground motion perpendicular to its plane will collapse very easily because walls B have little bending resistance in the direction perpendicular to their length. In long barrack-type buildings without intermediate walls, the end walls will be too far away to offer much support to the long walls, and the situation will be similar to the one just mentioned above.

2.6.4 Roof on wall enclosure

Now consider a complete wall enclosure with a roof on the top subjected to an earthquake force acting along X-axis as shown in Fig. 2.15. If the roof is rigid and acts as a horizontal diaphragm, its inertia will be distributed to the four walls in proportion to their stiffness. The inertia of roof will almost entirely go to walls B since their stiffness is much greater than that of the walls A in the X direction. In this case, the walls A will be restrained by the roof at the top and horizontal bending of walls A will be reduced. On the other hand, if the roof is flexible the roof inertia will go to the wall on which it is supported and the support provided to the tops of walls A will be small. Again the enclosure will act as a box for resisting the lateral loads, this action decreasing in value as the plan dimensions of an enclosure increase.

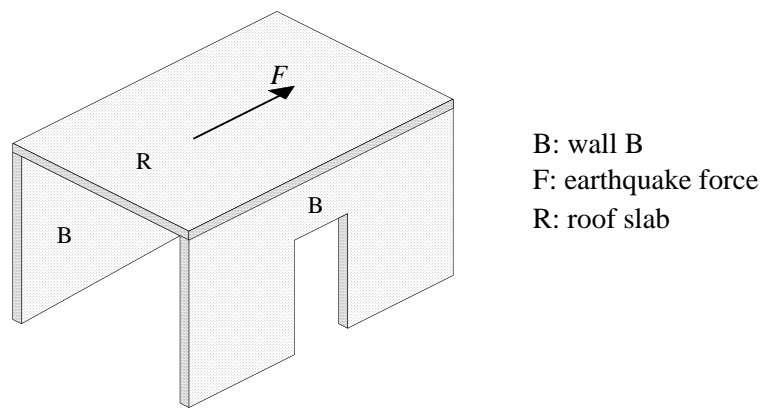


Figure 2.14: Roof on two walls

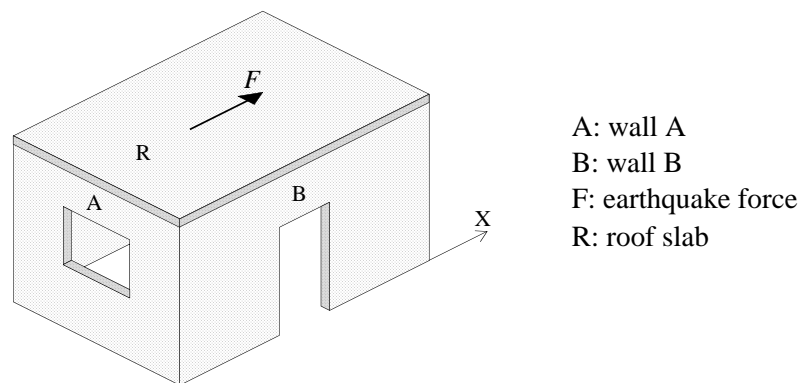


Figure 2.15: Roof on wall enclosure

2.6.5 Roofs and floors

Earthquake-induced inertia force can be distributed to the vertical structural elements in proportion to their stiffness, provided the roofs and floors are rigid and act as horizontal diaphragms. Otherwise, the roof and floor inertia will only go to the vertical elements on which they are supported. Therefore, the stiffness and integrity of roofs and floors are important for earthquake resistance.

Roofs and floors, which are rigid and flat and are bonded or tied to the masonry, have a positive effect on the wall. Slab or slab and beam construction directly cast over the walls or jack arch floors or roofs provided with horizontal ties and laid over the masonry walls through good quality mortar fall into this category. Others that simply rest on the masonry walls will offer resistance to relative motion only through friction, which may or may not be adequate depending on the earthquake intensity. In the case of a floor consisting of timber joists placed at center to center spacing of 200 to 250 mm with brick tiles placed directly over the joists and covered with clayey earth, the brick tiles have no binding effect on the joists. Therefore, relative displacement of the joists is quite likely to occur during an earthquake. This could easily bring down the tiles, damaging property and causing injury to people. Similar behaviour may be visualized with the floor consisting of precast reinforced concrete elements not adequately tied

together. In this case, relative displacement of the supporting walls could bring down the slabs.

2.6.6 Long buildings with roof trusses

Consider a long building with single span roof trusses as shown in Fig. 2.16. The trusses rest on the walls A. The walls B are gabled to receive the purlins of the end bays. Assuming that the ground motion is along the X-axis, the inertia forces will be transmitted from sheeting to purlins, to trusses and from trusses to walls A.

The end purlins will transmit some force directly to gable ends. Under the seismic force the trusses may slide on the walls unless anchored into them by bolts. Also, wall A, which does not get much support from the walls B in this case, may overturn unless made strong enough in vertical bending as a cantilever, or other suitable arrangement such as adding horizontal bracings between the trusses is made to transmit the force horizontally to end walls B.

When the ground motion is along the Y direction, walls A will be in a position to act as shear walls and all forces may be transmitted to them only if roof bracing is provided. If this is not the case, the purlins act as ties and struts and transfer the inertia force of roof to the gable ends.

As a result the gable ends may fail. Gable triangles are very weak when loaded perpendicular to their plane and may fail even in small earthquakes. Also, if there is insufficient bracing between the roof trusses, they may overturn when their supporting walls are intact.

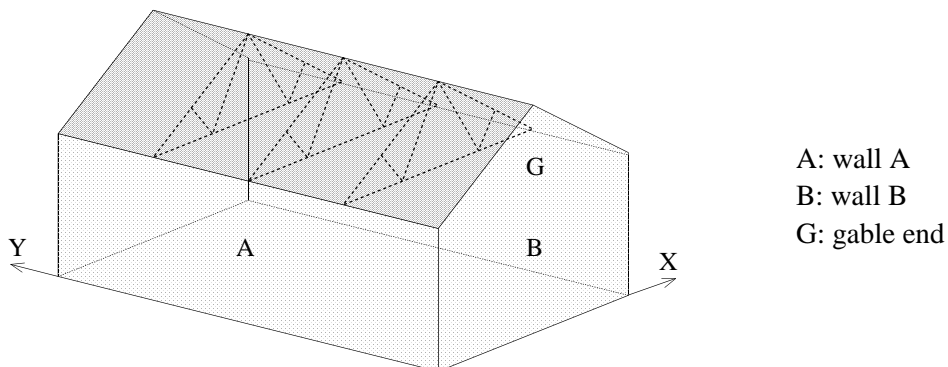


Figure 2.16: A long building with roof trusses

2.6.7 Shear wall with openings

Shear walls are the main lateral earthquake resistant elements in many buildings. For understanding their action, consider a shear wall with three openings shown in Fig. 2.17. Obviously, the piers between the openings are more flexible than the portion of wall below (sill masonry) or above (spandrel masonry) the openings. The deflected form under horizontal seismic force is also sketched in the figure as dashed lines.

The sections at the level of the top and bottom of opening are found to be the worst stressed in tension as well as in compression and those in the piers carry the maximum shears. Under reversed direction of horizontal loading the sections carrying tensile and compressive stresses

are also reversed. Thus it is seen that tension occurs in the jambs of openings and at the corners of the walls (see Fig. 2.18).

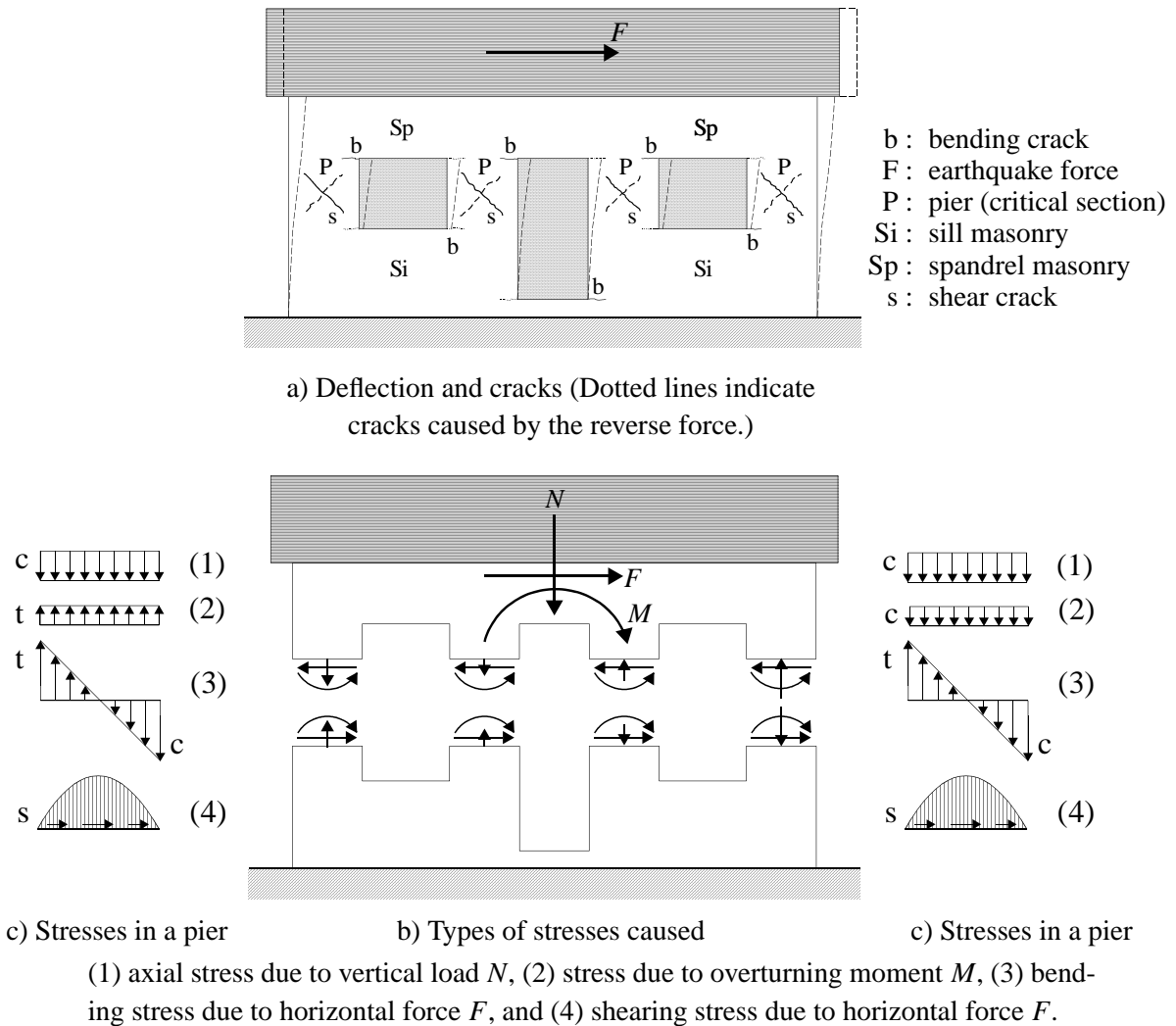


Figure 2.17: Cracks and stresses of a shear wall with openings

2.7 Earthquake Damage Categories

In this section, an outline of damage categories is described in Table 2.1 on the basis of past earthquake experience. In the table the appropriate post-earthquake action for each category of damage is also suggested.



1990 Majalrnga Earthquake, Indonesia



1994 Liwa earthquake, Indonesia

Figure 2.18: Damage at corners of openings

Table 2.1: Categories of damage

Damage category		Extent of damage in general	Suggested post-earthquake actions
0	No damage	No damage.	No action required.
I	Slight non-structural damage	Thin cracks in plaster, falling of plaster bits, in limited parts.	Building need not be vacated. Only architectural repairs needed.
II	Slight structural damage	Small cracks in walls, falling of plaster in large bits over large areas; damage to non-structural parts like chimneys, projecting cornices, etc. The load carrying capacity of the structure is not reduced appreciably.	Building need not be vacated. Architectural repairs required to achieve durability.
III	Moderate structural damage	Large and deep cracks in walls, widespread cracking of walls, columns, piers and tilting or falling of chimneys. The load carrying capacity of structure is partially reduced.	Building needs to be vacated and reoccupied after restoration and strengthening. Structural restoration and seismic strengthening are necessary after which architectural treatment may be carried out.
IV	Severe structural damage	Gaps occur in walls; inner or outer walls collapse; failure of ties to separate parts of buildings. Approximately 50 % of the main structural elements fail. The building is dangerous.	Building has to be vacated. Either the building has to be demolished or extensive restoration and strengthening work has to be carried out before reoccupation.
V	Collapse	A large part or whole of the building collapses.	Clearing the site and reconstruction.

Chapter 3

General Concepts of Earthquake Resistant Design

3.1 Introduction

Experience in past earthquakes has demonstrated that many common buildings and typical methods of construction lack basic resistance to earthquake forces. In most cases this resistance can be achieved by following simple and inexpensive principles of good building construction practice. Adherence to these simple rules will not prevent all damage in moderate or large earthquakes, but life threatening collapses should be prevented, and damage limited to repairable proportions. These principles fall into several broad categories:

- a) Planning and layout of the building involving consideration of the location of rooms and walls, openings such as doors and windows, the number of storeys, etc. At this stage, site and foundation aspects should also be considered.
- b) Layout and general design of the structural framing system with special attention to providing lateral resistance, and
- c) Consideration of highly loaded and critical sections with provision of reinforcement as required.

Chapter 2 has provided an overview of structural action, mechanism of damage and modes of failure of buildings. From these studies, certain general principles have emerged:

- 1) Structures should not be brittle or collapse suddenly. Rather, they should be tough, able to deflect or deform a considerable amount.
- 2) Resisting elements, such as bracing or shear walls, must be provided evenly throughout the building, in both directions side-to-side, as well as top to bottom.
- 3) All elements, such as walls and the roof, should be tied together so as to act as an integrated unit during earthquake shaking, transferring forces across connections and preventing separation.
- 4) The building must be well connected to the foundation and the earth. Wet, soft soils should be avoided, and the foundation must be well tied together, as well as tied to the walls. Where soft soils cannot be avoided, special strengthening must be provided.

- 5) Care must be taken that all materials used are of good quality, and are protected from rain, sun, insects and other weakening actions, so that their strength lasts.
- 6) Unreinforced earth and masonry have no reliable strength in tension, and are brittle in compression. Generally, they must be suitably reinforced by steel or wood.

These principles will be discussed and illustrated in this Chapter.

3.2 Categories of Buildings

For categorising buildings with the purpose of achieving seismic resistance at economical cost, three parameters turn out to be significant:

- a) Seismic intensity zone which depends on where the building is located,
- b) How important the building is, and
- c) How stiff is the foundation soil.

A combination of these parameters will determine the extent of appropriate seismic strength of the building.

3.2.1 Seismic zones

In many countries, macro level seismic zones are defined on the basis of the relative seismic hazard of the region or the expected seismic intensity scale (see Table 3.1 and MSK Intensity Scale in Sec. 10.1). Recently, design seismic intensity (acceleration) is expressed based on a reference probability of exceedance in 50 years or a reference earthquake return period[†].

Table 3.1: Seismic zones

Zone	Possible risk	I_{MSK}
A	Widespread collapse and destruction	IX or greater
B	Collapse and heavy damage	VIII likely
C	Damage	VII likely
D	Minor damage	VI maximum

I_{MSK} : MSK Intensity scale (see Sec. 10.1).

The extent of special earthquake strength is greatest in Zone A and, for reasons of economy, can be decreased in Zone C, with relatively little special strength in Zone D. However, since the principles stated in Sec.3.1, are sound principles for building in general (not just for earthquake), they should always be followed.

[†]The reference probability is 10% to 2%, or the reference return period is approximately 500 to 2500 years.

3.2.2 Importance of building

The importance of a building should be a factor in grading it for earthquake strength purposes, and the following buildings are classified into two groups:

- IMPORTANT - Hospitals, clinics, communication buildings, fire and police stations, water supply facilities, cinemas, theatres and meeting halls, schools, dormitories, cultural treasures such as museums, monuments and temples, etc.
- ORDINARY - Houses, hostels, offices, warehouses, factories, etc.

3.2.3 Classification of foundation soil

Four soil profile types are considered here as shown in Table 3.2.

Buildings can be constructed on hard, medium and soft soils, but it is dangerous to build them on weak soils.

Weak soils must be avoided or compacted to improve them so as to qualify at least as medium or soft.

Table 3.2: Classification of soil

Soil profile	Ground Characteristics			N_a * (t/m ²)
	Description	V_s (m/s)	N_s	
Hard	Rock, deposits of very dense sand, gravel, or very stiff clay, etc.	> 400	> 50	50
Medium	Deposits of dense or medium-dense sand, gravel, or stiff clay, etc.	400 ~ 200	50 ~ 15	20
Soft	Deposits of loose-to-medium cohesionless soil, or soft-to-firm cohesive soil, etc.	< 200	< 15	5
Weak	Very soft soil that is liable to large differential settlement, or liquefaction during an earthquake	—	—	—

V_s : shear wave velocity, N_s : standard penetration test blow count

* Soil investigations should be carried out to establish the appropriate allowable bearing capacity N_a . (1 t/m² ≈ 10 kN/m²)

3.2.4 Combination of parameters

For defining the categories of buildings for seismic strength purposes, four categories I to IV are defined in Table 3.3 in which category I requires maximum strength and category IV the least. The general planning and design principles are, however, equally applicable to all categories.

Table 3.3: Category of buildings for seismic strength purposes

Building category	Combination for the category		
	seismic zone*	building**	soil profile***
I	A	important	soft
II	A	important	medium to hard
	B	important	soft
III	A	ordinary	medium to hard
	B	Important	medium to hard
	C	important	soft
IV	B	ordinary	medium to hard
	C	important	medium to hard
	C	ordinary	soft

* Seismic zones A, B and C are defined in Sec.3.2.1

** Importance of buildings are defined in Sec.3.2.2

*** Soil profiles are defined in Sec.3.2.3. Weak soil liable to compaction and liquefaction is not covered here.

3.3 General Planning and Design Aspects

3.3.1 Plan of building

- a) Symmetry: The building as a whole or its various blocks should be kept symmetrical about both X and Y axes. Asymmetry leads to torsion during earthquakes and is dangerous (see Fig. 3.1). Symmetry, as far as possible, is also desirable in the placing and sizing of door and window openings.

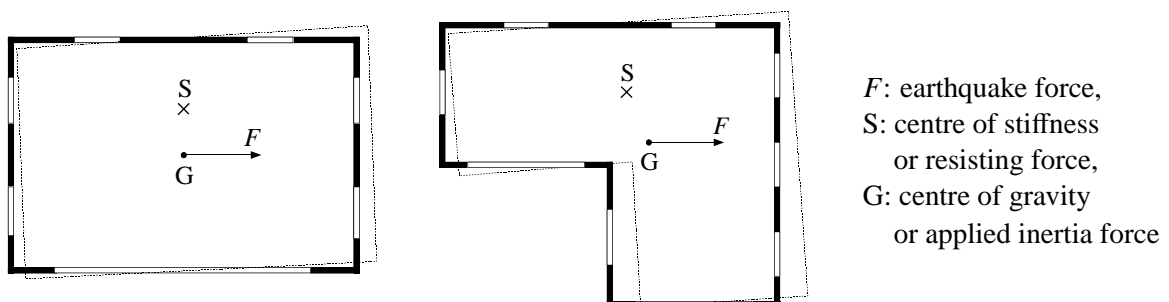


Figure 3.1: Torsion or twisting of unsymmetrical plans

- b) Regularity: Simple rectangular shapes (see Fig. 3.2 a) behave better in an earthquake than shapes with projections (see Fig. 3.2 b). Torsional effects of ground motion are pronounced in long narrow rectangular blocks. Therefore, it is desirable to restrict the length

of a block to three times its width. If longer lengths are required two separate blocks with sufficient separation between should be provided (see Fig. 3.2 c).

- c) Separation of Blocks: Separation of a large building into several blocks may be required so as to obtain symmetry and regularity of each block.

For preventing hammering or pounding damage between blocks a physical separation of 30 to 40 mm throughout the height above the plinth level will be adequate as well as practical for up to 3 storey buildings (see Fig. 3.2 c).

The separation section can be treated just like expansion joint or it may be filled or covered with a weak material which would easily crush and crumble during earthquake shaking. Such separation is more practical in larger buildings since it is less convenient in small buildings.

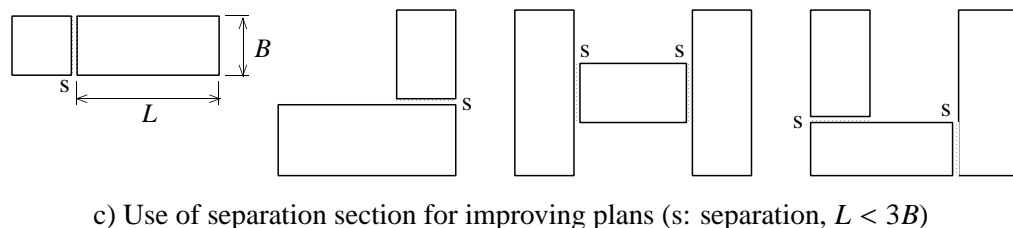
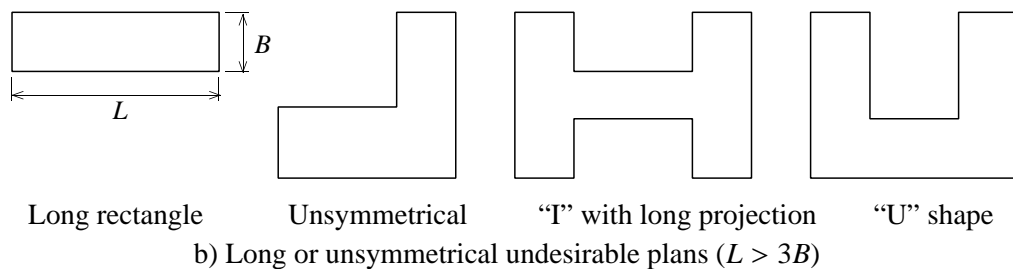
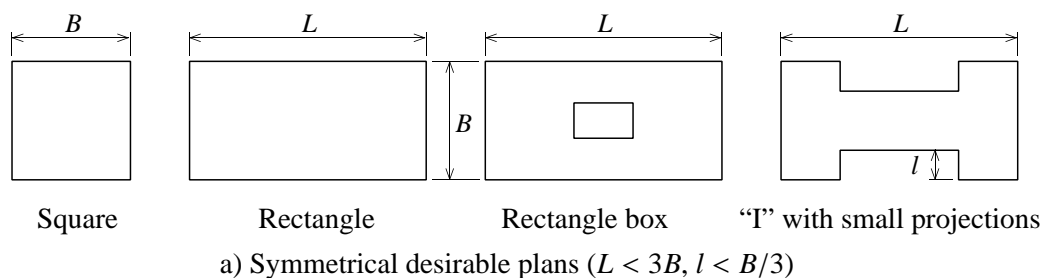


Figure 3.2: Plan of building blocks

- d) Simplicity: Ornamentation involving large cornices, vertical or horizontal cantilever projections, fascia stones and the like are dangerous and undesirable from a seismic viewpoint. Simplicity is the best approach.

Where ornamentation is insisted upon, it must be reinforced with steel, which should be properly embedded or tied into the main structure of the building.

Note) If designed, a seismic coefficient about 5 times the coefficient used for designing the main structure should be used for vertical cantilever ornamentation.

- e) **Enclosed Area:** A small building enclosure with properly interconnected walls acts like a rigid box, since long walls are supported by transverse walls to prevent out-of-plane failure of the walls and to increase their lateral strength.

Structurally it is advisable to have separately enclosed rooms rather than one long room (see Fig. 3.3). For masonry walls of thickness t and wall spacing of a , the ratio of $a/t = 40$ should be the upper limit between the cross walls for mortars of cement : sand = 1 : 6 or richer, and less for poor mortars. For larger panels or thinner walls, framing elements should be introduced as shown in Fig. 3.3 c).

- f) **Separate Buildings for Different Functions:** In view of the difference in importance of hospitals, schools, assembly halls, residences, communication and security buildings, etc., it may be economical to plan separate blocks for different functions so as to lessen the cost in providing earthquake strength.

3.3.2 Choice of site

The choice of site for a building from the seismic point of view is mainly concerned with the stability of the ground. The following ground conditions are important:

- a) **Stability of Slope:** Hillside slopes liable to slide during an earthquake should be avoided and only stable slopes should be chosen to locate a building. Also it is preferable to have several blocks on terraces than have one large block with footings at different elevations. A site subject to the danger of rock falls has to be avoided.
- b) **Very Loose Sands or Sensitive Clays:** These two types of soils are liable to lose their original structure and thereby undergo compaction during an earthquake. This can result in large unequal (uneven or differential) settlements and damage a building. If loose cohesionless soils are saturated with water, they are apt to lose their shear resistance altogether during shaking and liquefy.

Although such soils can be compacted, for small buildings the operation is usually too costly and these soils are better avoided. For large building complexes, such as housing developments, new towns, etc., this factor should be thoroughly investigated and appropriate action taken.

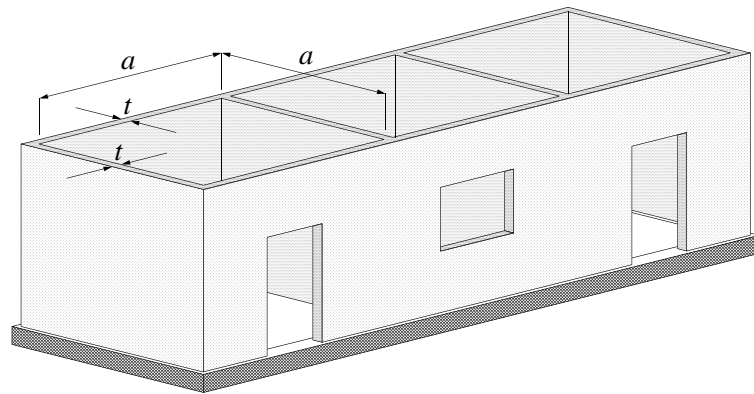
Therefore a site with sufficient bearing capacity and free from the above defects should be chosen and its drainage condition should be improved so that no water accumulates and saturates the ground close to the footing level.

3.3.3 Structural design

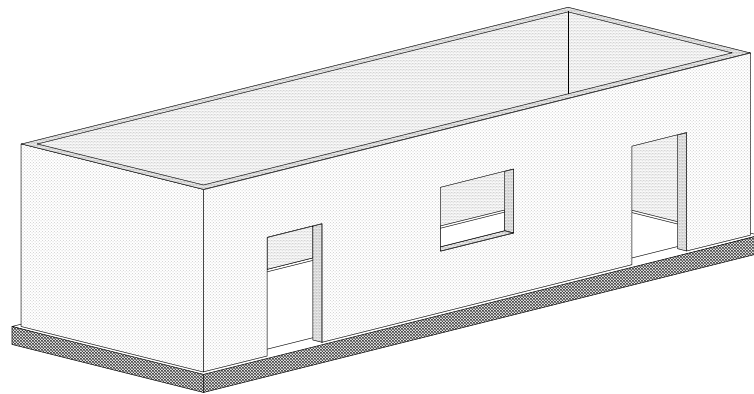
Ductility (defined in Sec.3.6) is the most desirable quality for good earthquake performance and can be incorporated to some extent in otherwise brittle masonry construction by introduction of steel reinforcing bars at critical sections as indicated in Chapters 4 and 5.

3.3.4 Fire resistance

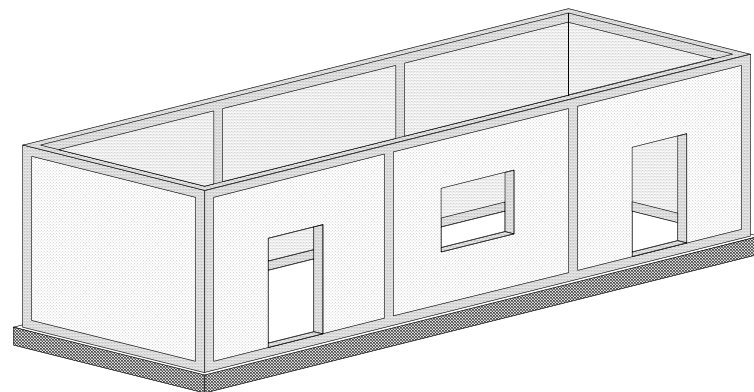
It is not unusual during earthquakes that due to snapping of electrical fittings short circuiting takes place and gas pipes develop leaks and catch fire. Fire could also be started due to kerosene



a) Many crosswalls, small boxes, seismically strong
 (Wall length and thickness ratios should be $a/t \leq 40$, or framing be used as c) below.)



b) No crosswalls, large box, seismically weak



c) Wall with framing elements (usually reinforced concrete)

Figure 3.3: Enclosed areas forming box units

lamps and kitchen fires. Post-earthquake fire damage can be more serious than the earthquake damage due to ground shaking. Buildings should therefore preferably be constructed of fire resistant materials.

3.4 Structural Framing

There are basically two types structural framing possible to withstand gravity and seismic load, viz. bearing wall construction and framed construction. Framed construction may consist of:

- a) Light framing members with diagonal bracing such as wood frames (see Chapter 6) or infill walls for lateral load resistance (see Fig. 3.3 c), or
- b) Substantial rigidly jointed beams and columns capable of resisting the lateral loads by themselves, i.e. moment resisting frame.

The latter is usually required for large column free spaces, such as assembly halls.

Framed construction can be used for a greater number of storeys compared to bearing wall construction. The strength and ductility can be better controlled in framed construction through design. The strength of the framed construction is not affected by the size and number of openings. Such frames fall in the category of engineered construction, hence outside the scope of this book.

3.5 Requirements of Structural Safety

As a result of the discussion of structural action and mechanisms of failure of Chapter 2, the following main requirements of structural safety of buildings can be arrived at:

- a) A free standing wall must be designed to be safe as a vertical cantilever.
This requirement will be difficult to achieve in unreinforced masonry in Zone A. Therefore all partitions inside the buildings must be held on the sides as well as the top. Parapets of category I and II buildings must be reinforced and connected to the main structural slabs or frames.
- b) Horizontal reinforcement in walls is required for transferring their own out-of-plane inertia load horizontally to shear walls.
- c) Walls must be effectively tied together to avoid separation at vertical joints due to ground shaking.
- d) Shear walls must be present along both X and Y axes of a building.
- e) A shear wall must be capable of resisting all horizontal forces due to its own mass and inertia force transmitted to it.
- f) Roof or floor elements must be tied together and be capable of exhibiting diaphragm action.
- g) Trusses must be anchored to the supporting walls and have some bracing arrangement for transferring their inertia force to the end walls.

The strength measures necessary to meet these safety requirements are presented in the following Chapters for various building types. In view of the low seismicity of Zone D, no strength measures from seismic considerations are considered necessary except an emphasis on good quality of construction. The following recommendations are therefore intended for Zones A, B and C. For this purpose certain categories of construction in a number of situations were defined in Table 3.3.

3.6 Concepts of Ductility, Deformability and Robustness

Desirable properties of earthquake-resistant design include ductility, deformability and robustness. Ductility and deformability are interrelated concepts signifying the ability of a structure to sustain large deformations without collapse. Robustness refers to the ability of a structure to undergo substantial damage, without partial or total collapse. This is desirable because it means that structures can absorb more damage, and because it permits the deformations to be observed for repairs or evacuation to proceed, prior to collapse. In this sense, a warning is received and lives are saved.

3.6.1 Ductility

Formally, ductility refers to the ratio of the displacement of a building just prior to ultimate displacement or collapse to the displacement at first damage or yield. Some materials are inherently ductile, such as steel. Other materials are not ductile (that is, brittle), such as cast iron, plain masonry, adobe or concrete. They break suddenly, without warning. Brittle materials can be made ductile, usually by the addition of modest amounts of more ductile materials, such as wood elements in adobe construction, or steel reinforcing in masonry and concrete construction.

For ductile materials to achieve a ductile effect in the overall behaviour of the component, they must be proportioned and placed so that they act in tension or bending and are subjected to yielding. Thus, a necessary requirement for good earthquake-resistant design is to have sufficient ductile material at points of tensile stress.

3.6.2 Deformability

Deformability is a less formal term referring to the ability of a structure to displace or deform substantial amounts without collapsing. Besides inherently relying on the ductility of materials and components, deformability requires that structures be well-proportioned, regular and well tied together so that excessive stress concentrations are avoided and forces are capable of being transmitted from one component to another even through large deformations.

Ductility is a term applied to both material and structures, while deformability is applicable only to structures.

Even when ductile materials are present in sufficient amounts in structural components such as beams and walls, overall structural deformability requires that geometrical and material instability be avoided. Components must have proper aspect ratios (that is, not be too high), must be adequately connected to resisting elements (for example sufficient wall ties for a masonry wall, tying it to floors, roof and shear walls), and must be well tied together (for example positive connection at beam seats, so that deformations do not permit a beam to simply fall off a post) so as to permit large deformations and dynamic motions to occur without sudden collapse.

3.6.3 Robustness

Robustness is also a desirable quality for construction, and refers to the ability of a structure to undergo substantial damage, without partial or total collapse.

A key to good robustness is redundancy, or provision of several supports for key structural members, such as ridge beams, and avoidance of central columns or walls supporting excessively large portions of a building. A key to achieving good robustness is to always ask the question, “if this beam or column, wall connection, foundation, etc. fails, what is the consequence?” If the consequence is total collapse of the structure, additional supports or alternative structural layouts should be examined, or an additional factor of safety be provided for such critical members or connections.

3.7 Concept of Isolation

The foregoing discussion of earthquake-resistant design has emphasized the traditional approach of resisting the forces an earthquake imposes on a structure. An alternative approach which is presently emerging is to avoid these forces, by isolation of the structure from the ground motions which actually impose the forces on the structure.

This is termed base-isolation. For simple buildings, base-friction isolation may be achieved by reducing the coefficient of friction between the structure and its foundation, or by placing a flexible connection between the structure and its foundation (see Fig. 3.4).

For reduction of friction between the structure and its foundation, one suggested technique is to place two layers of good quality plastic membrane between the structure and its foundation, so that the plastic layers may slide over each other.

Flexible connections between the structure and its foundation are difficult to achieve on a permanent basis. One technique that has been used for generations has been to build a house on short posts resting on large stones, so that under earthquake motions, the posts are effectively pin-connected at the top and bottom and the structure can rock to and fro somewhat. This has the advantage of substantially reducing the lateral forces, effectively isolating the structure from high frequency motions. Unfortunately, traditional applications of this technique usually do not account for occasional large displacements of this pin-connected mechanism. Collapse results when lateral displacements reach a certain point due to rare very large earthquakes or unusually large low-frequency content in the ground motion. A solution to this problem would be provision of a plinth slightly below the level of the top of the posts, so that when the posts rock too far, the structure only drops a centimeter or so.

3.8 Foundations

For the purpose of making a building truly earthquake resistant, it will be necessary to choose an appropriate foundation type for it. Since loads from typical low rise buildings are light, providing the required bearing area are not usually a problem. The depth of footing in the soil should go below the zone of deep freezing in cold regions and below the level of shrinkage cracks in clayey soils. For choosing the type of footing from an earthquake perspective, the soils may be grouped as “Hard”, “Medium” and “Soft” (see Sec.3.2.3), avoiding the weak soil unless compacted and brought to Soft or Medium condition.



Figure 3.4: “Traditional” isolation (Sukabumi, Indonesia)

(This may not prevent the house falling off its piers in a moderate and large earthquake.)

3.8.1 Hard and medium soils

In hard or medium soil conditions, any type of footing (individual or strip type) can be used. It should of course have a firm base of lime or cement concrete with requisite width over which the construction of the footing may start. It is desirable to connect the individual reinforced concrete column footings in Zone A by means of reinforced concrete beams just below plinth level intersecting at right angles.

3.8.2 Soft soil

In soft soil, it is desirable to use a plinth band in all walls and where necessary to connect the individual column footings by means of plinth beams as suggested above. Continuous reinforced concrete footings are considered to be most effective from earthquake considerations as well as to avoid differential settlements under normal vertical loads. Details of plinth band and continuous reinforced concrete footings are presented in Chapters 4 and 9, respectively. These should ordinarily be provided continuously under all the walls. Continuous footings should be reinforced both in the top and bottom faces, the width of the footing should be wide enough to make the contact pressures uniform, and the depth of footing should be below the lowest level of weathering (and freezing in cold regions).

Chapter 4

Masonry Buildings in Fired-Brick and Other Materials

4.1 Introduction

Buildings in fired bricks, solid concrete blocks and hollow concrete or mortar blocks are dealt with in this chapter. The general principles and most details of earthquake resistant design and construction of brick-buildings are applicable to those using other rectangular masonry units such as solid blocks of mortar, concrete, or stabilized soil, or hollow blocks of mortar, or concrete having adequate compressive strength.

4.2 Typical Damage and Failure of Masonry Buildings

The creation of tensile and shearing stresses in walls of masonry buildings is the primary cause of different types of damage. The typical damage and modes of failure are briefly described below:

4.2.1 Non-structural damage

Non-structural damage excludes damage to the building structure. Such damage occurs frequently even under moderate intensities of earthquakes as follows:

- 1) Cracking and overturning of masonry parapets, roof chimneys, large cantilever cornices and balconies.
- 2) Falling of plaster from walls and ceiling particularly where it was loose.
- 3) Cracking and overturning of partition walls, infill walls and cladding walls from the inside of frames. (Though not usually accounted for in calculations, this type of damage reduces the lateral strength of a building.)
- 4) Cracking and falling of ceilings.
- 5) Cracking of glass panes.
- 6) Falling of loosely placed objects, overturning of cupboards, etc.

4.2.2 Damage and failure of bearing walls

- 1) Failure due to racking shear is characterized by diagonal cracks mainly due to diagonal tension. Such failure may be either through the pattern of joints or diagonally through masonry units. These cracks usually initiate at the corner of openings and sometimes at the centre of a wall segment. This kind of failure can cause partial or complete collapse of the structure (see Fig. 4.1).

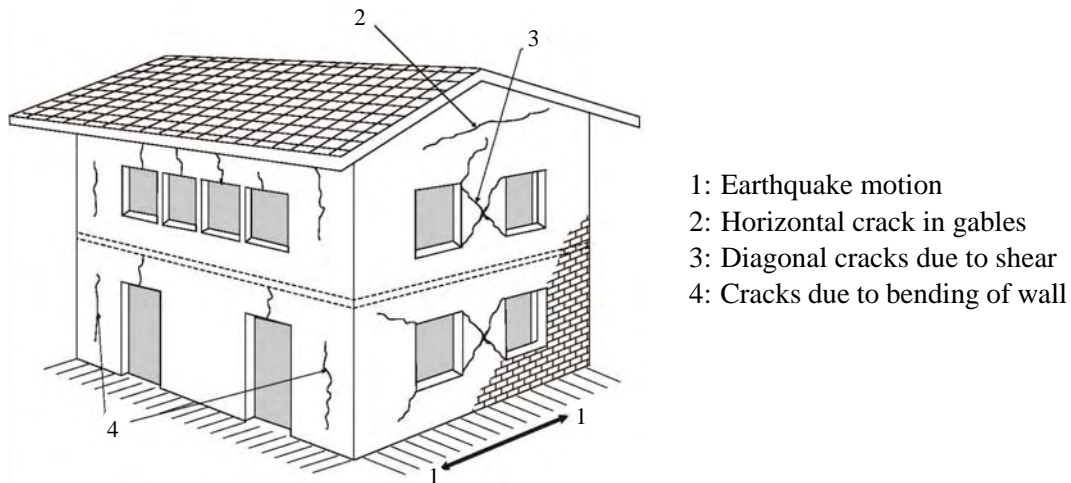


Figure 4.1: Cracking in bearing walls due to bending and shear

- 2) A wall can fail as a bending member loaded by seismic inertia forces on the mass of the wall itself in a direction transverse to the plane of the wall. Tension cracks occur vertically at the centre, ends or corners of the walls. The longer the wall and longer the openings, more prominent is the damage (see Fig. 4.1). Since earthquake effects occur along both axes of a building simultaneously, bending and shearing effects occur often together and the two modes of failures are often combined. Failure in the piers occur due to the combined action of flexure and shear.
- 3) Unreinforced gable end masonry walls are very unstable and the pushing action of purlins imposes additional force to cause their failure. Horizontal bending tension cracks develop in the gables.
- 4) The deep beam between two openings one above the other is a weak point of the wall under lateral inplane forces. Cracking in this zone occurs before diagonal cracking of piers unless the piers are quite narrow (see Fig. 4.2). In order to prevent it and to enable the full distribution of shear among all piers, either a rigid slab or RC band must exist between them.
- 5) Walls can be damaged due to the seismic force from the roof, which can cause the formation of tension cracks and separation of supporting walls (see Fig. 4.3). This mode of failure is characteristic of massive flat roofs (or floors) supported by joists, which in turn are supported by bearing walls, but without proper connection with them. Also, if the connection to the foundations is not adequate, walls crack there and slide. This may cause failure of plumbing pipes.

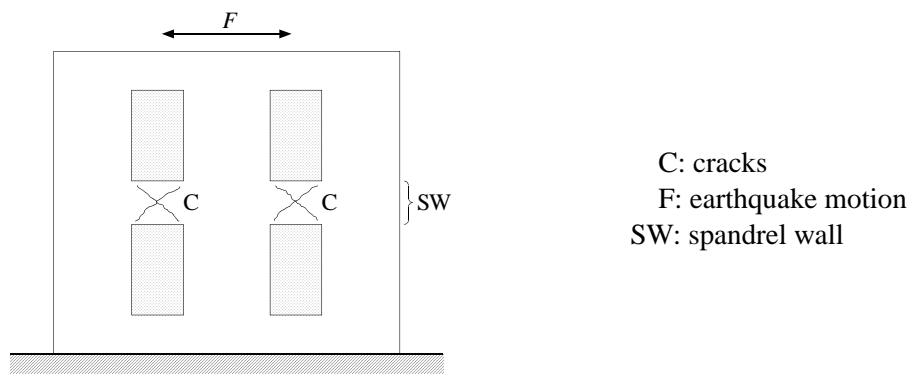


Figure 4.2: Cracking of spandrel wall between opening

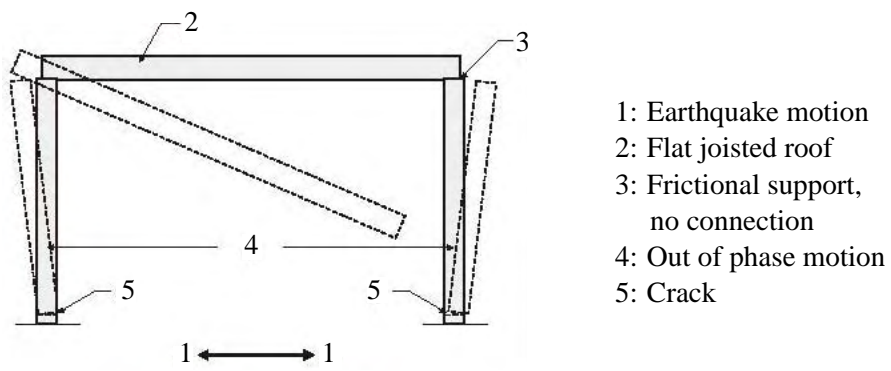


Figure 4.3: Fall of roof because of inadequate connection between roof and wall



Figure 4.4: Damaged brick masonry
(2006 Central Java Earthquake, Indonesia)

6) Failure due to torsion and warping: The damage in an unsymmetrical building occurs due to torsion and warping in an earthquake (see Fig. 3.1). This mode of failure causes

excessive cracking due to shear in all walls. Larger damage occurs near the corners of the building.

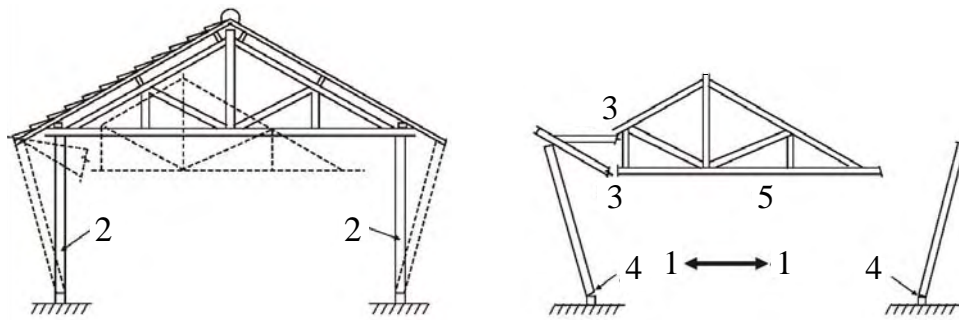
- 7) Arches across openings in walls are often badly cracked since the arches tend to lose their end thrust under in-plane shaking of walls.
- 8) Under severe prolonged intense ground motions, the following happens:
 - cracks become wider and masonry units become loose.
 - partial collapse and gaps in walls occur due to falling of loose masonry units, particularly at the location of piers.
 - falling of spandrel masonry due to collapse of piers.
 - falling of gable masonry due to out of plane cantilever action.
 - walls get separated at corners and intermediate T-junctions and fall outwards.
 - roof collapse, either partial or full.
 - certain types of roofs may slide off the tops of walls and the roof beams fall down.
 - masonry arches across wall openings as well as those used for roof collapse completely.

4.2.3 Failure of ground

- 1) Inadequate depth of foundations: Shallow foundations deteriorate as a result of weathering and consequently become weak for earthquake resistance.
- 2) Differential settlement of foundations: During severe ground shaking, liquefaction of loose water-saturated sands and differential compaction of weak loose soils occur which lead to excessive cracking and tilting of buildings which may even collapse completely.
- 3) Sliding of slopes: Earthquakes cause sliding failures in man-made as well as natural hill slopes and any building resting on such a slope have a danger of complete disastrous disintegration.

4.2.4 Failure of roofs and floors

- 1) Dislodging of roofing material: Improperly tied roofing material is dislodged due to inertia forces acting on the roof. This mode of failure is typical of sloping roofs, particularly when slates, clay, tiles etc. are used as roofing material.
Brittle material like asbestos cement may be broken if the trusses and sheeting purlins are not properly braced together.
- 2) Weak roof-to-support connection is the cause of separation of roof trusses from supports, although complete roof collapse mostly occurs due to collapse of the supporting structure. The rupture of bottom chord of roof truss may cause a complete collapse of a truss as well as that of walls (see Fig. 4.5).
- 3) Heavy roofs as used in rural areas with large thickness of earth over round timbers cause large inertia forces on top of walls and may lead to complete collapse in severe earthquakes.
- 4) Lean-to roofs easily cause instability in the lower supporting walls or piers and collapse easily due to lack of ties (see Fig. 7.1).



(1: Earthquake motion, 2: Wall or column, 3: Rupture of tie and rafter, 4: Crack in wall or rupture of column, 5: Collapse of truss)

Figure 4.5: Failure due to rupture of bottom chord of roof truss

4.2.5 Causes of damage in masonry buildings

The following are the main weaknesses in unreinforced masonry construction and other reasons for the extensive seismic damage of such buildings (see Fig 4.6):

- Heavy and stiff buildings, attracting large seismic inertia forces.
- Very low tensile strength, particularly with poor mortars.
- Low shear strength, particularly with poor mortars.
- Brittle behaviour in tension as well as compression.
- Weak connections between walls.
- Stress concentration at corners of windows and doors.
- Overall unsymmetry in plan and elevation of building.
- Unsymmetry due to imbalance in the sizes and positions of openings in the walls.
- Defects in construction such as use of substandard materials, unfilled joints between bricks, walls not-plumb, improper bonding between walls at corners and T junctions.

4.3 Typical Strengths of Masonry

The crushing strength of masonry walls depends on many factors such as the following:

- 1) Crushing strength of the masonry unit.
- 2) Mix of the mortar used and age at which tested. The mortar used for different walls varies in quality as well as strength. It is generally described on the basis of the main binding material such as cement or lime mortar, cement lime composite mortar, lime-pozzolana or hydraulic lime mortar. Clay mud mortar is also used in many countries, particular in rural areas.
- 3) Slenderness ratio of the wall. That is, the lesser of the ratio of effective height and effective length of the wall to its thickness. The larger the slenderness ratio, the smaller the strength.



Figure 4.6: Damaged unreinforced masonry house (Muzafarabad, Pakistan)
(2005 Northern Pakistan Earthquake)

- 4) Eccentricity of the vertical load on the wall. The larger the eccentricity, the smaller the strength.
- 5) Percentage of openings in the wall. The larger the openings, the smaller the strength. The tensile and shearing strengths of masonry mainly depend upon the bond or adhesion at the contact surface between the masonry unit and the mortar and, in general, their values are only a small percentage of the crushing strength. A mortar richer in cement or lime content, the higher is the percentage of tensile and shearing strength in relation to the crushing strength. Tests carried out on brick-couplets using hand-made bricks in cement mortar give the typical values as shown in Table 4.1. (The values in Tables 4.1, 4.2 and 4.3 may be used as default values where more precise values are not available or nationally specified values are not available.)

Table 4.1: Typical strengths of masonry

Mortar mix		Tensile strength (MPa)*	Shearing strength (MPa)*	Compressive strength (MPa)* corresponding to crushing strength of masonry unit			
Cement	Sand			3.5	7.0	10.5	14.0
1	12	0.04	0.22	1.5	2.4	3.3	3.9
1	6	0.25	0.39	2.1	3.3	5.1	6.0
1	3	0.71	1.04	2.4	4.2	6.3	7.5

* 1 MPa = 1 N/mm² \approx 10 kgf/cm²

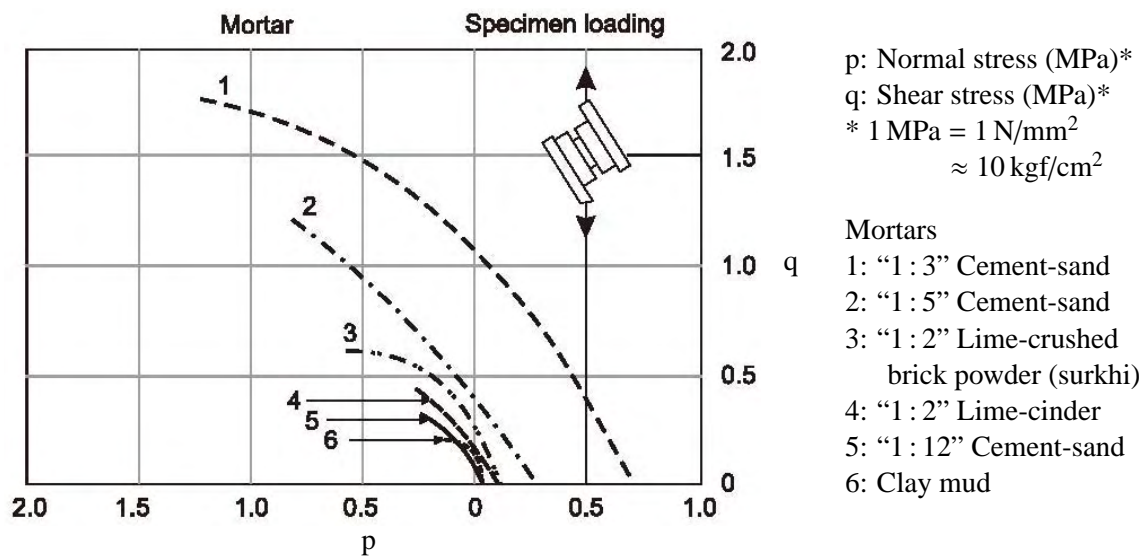


Figure 4.7: Combined stress couplet test results

Brick couplet tests under combined tension-shear and compression-shear stresses show that the shearing strength decreases when acting with tension and increases when acting with compression. Fig. 4.7 shows the combined strengths.

The tensile strength of masonry is not generally relied upon for design purposes under normal loads and the area subjected to tension is assumed cracked. Under seismic conditions, it is recommended that the permissible tensile and shear stresses on the area of horizontal mortar bed joint in masonry may be adopted as given in Table 4.2.

Table 4.2: Typical permissible stresses

Mortar mix or equivalent			Permissible stresses		Compression strength (MPa)* of unit			
			Tension (MPa)*	Shear (MPa)*	3.5	7.0	10.5	14.0
Cement	Lime	Sand						
1	-	6	0.05	0.08	0.35	0.55	0.85	1.00
1	1	6	0.13	0.20	0.35	0.70	1.00	1.10
1	-	3	0.13	0.20	0.35	0.70	1.05	1.25

* 1 MPa = 1 N/mm² ≈ 10 kgf/cm²

The modulus of elasticity of masonry depends upon the density and stiffness of the masonry units, besides the mortar mix. For brickwork, the values are of the order 2 000 MPa for cement-sand mortar in “1 : 6” proportion. The mass density of masonry mainly depends on the type of masonry unit. For example brickwork has a mass density of about 1 900 kg/m³ and dressed stone masonry 2 400 kg/m³.

The slenderness ratio of the wall is taken as the lesser of h/t and ℓ/t where h = effective height of the wall, ℓ = its effective length and t = its thickness. The allowable stresses in Table 4.2 must be modified for eccentricity of vertical loading due to its position and seismic moment

and the slenderness ratio multiplying factors given in Table 4.3. The effective height h may be taken as a factor times the actual height of wall between floors, the factor being 0.75 when floors are rigid diaphragms and 1.00 for flexible roofs; it is 2.0 for parapets.

The effective length ℓ will be a fraction of actual length between lateral supports, the factor being 0.8 for wall continuous with cross walls or buttresses at both ends, 1.0 for continuous at one end and supported on the other and 1.5 for continuous at one and free at the other.

Table 4.3: Stress factor for slenderness ratio and eccentricity of loading

Slenderness ratio	Stress factor k for eccentricity ratio e/t						
	0	0.04	0.10	0.20	0.30	0.33	0.50
6	1.000	1.000	1.000	0.996	0.984	0.980	0.970
8	0.920	0.920	0.920	0.916	0.880	0.870	0.850
10	0.840	0.835	0.830	0.810	0.770	0.760	0.730
12	0.760	0.750	0.740	0.705	0.664	0.650	0.600
14	0.670	0.660	0.640	0.604	0.556	0.540	0.480
16	0.580	0.565	0.545	0.500	0.440	0.420	0.350
18	0.500	0.480	0.450	0.396	0.324	0.300	0.230
21	0.470	0.448	0.420	0.354	0.276	0.250	0.170
24	0.440	0.415	0.380	0.310	0.220	0.190	0.110

(i) Linear-interpolation may be used. (ii) Values for $e/t = 0.5$ are for interpolation only.

4.4 General Construction Aspects

4.4.1 Mortar

Since tensile and shear strength are important properties for seismic resistance of masonry walls, use of mud or very lean mortars is unsuitable. A mortar mix “cement : sand” equal to “1 : 6” by volume or equivalent in strength should be the minimum. Appropriate mixes for various categories of construction are recommended in Table 4.4. Use of a rich mortar in narrow piers between openings is desirable even if a lean mix is used for walls in general.

Table 4.4: Recommended mortar mixes

Category*	Proportion of cement-sand or cement-lime-sand**	
	Cement-sand	Cement-lime-sand
I	“1 : 4” or richer	“1 : $\frac{1}{2}$: 4 $\frac{1}{2}$ ” or richer
II	“1 : 5” or richer	“1 : 1 : 6” or richer
III	“1 : 6” or richer	“1 : 2 : 9” or richer
IV	“1 : 7” or richer	“1 : 3 : 12” or richer

* Category of construction is defined in Table 3.3.

** In this case some pozzolanic material like Trass (Indonesia) and Surkhi (burnt brick fine powder in India) may be used with lime as per local practice.

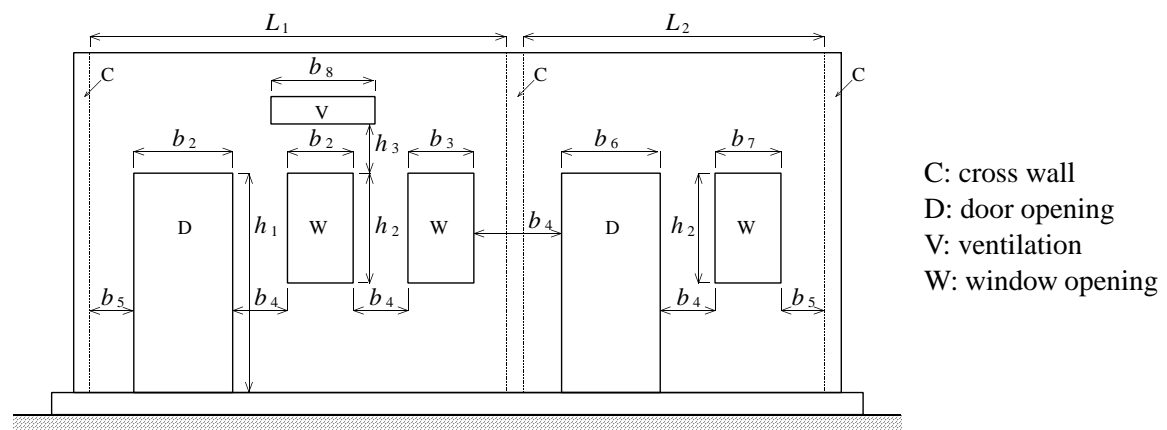
4.4.2 Wall enclosure

In load bearing wall construction, the wall thickness ' t ' should not be less than 190 mm, wall height not more than $20t$ and wall length between cross-walls not more than $40t$. If longer rooms are required, either the wall thickness is to be increased, or full height buttresses should be provided at $20t$ or less apart. The minimum dimensions of the buttress shall be thickness and top depth equal to t , and bottom depth equal to one sixth the wall height.

4.4.3 Openings in walls

Studies carried out on the effect of openings on the strength of walls indicate that openings should be small in size and centrally located. The following are the guidelines on the size and position of openings (see Fig. 4.8):

- 1) Openings to be located away from the inside corner by a clear distance equal to at least $1/4$ of the height of openings but not less than 0.6 m.
- 2) The total length of openings not to exceed 50 % of the length of the wall between consecutive cross walls in single-storey construction, 42 % in two-storey construction and 33 % in three storey buildings.
- 3) The horizontal distance (pier width) between two openings to be not less than half the height of the shorter opening, but not less than 0.6 m.
- 4) The vertical distance from an opening to an opening directly above it not to be less than 0.6 m nor less than $1/2$ of the width of the smaller opening.
- 5) When the openings do not comply with requirements 1) to 4), they should either be boxed around in reinforced concrete or reinforcing bars provided at the jambs through the masonry (see Fig. 4.9).



$b_1 + b_2 + b_3 \leq 0.5L_1$ for one storey, 0.42 for two storey, 0.33 for three storey,
 $b_6 + b_7 \leq 0.5L_2$ for one storey, 0.42 for two storey, 0.33 for three storey,
 $b_4 \geq 0.5h_2$ but not less than 0.6 m, $b_5 \geq 0.25h_1$ but not less than 0.6 m,
 $h_3 \geq 0.6$ m or $0.5(b_2 \text{ or } b_8 \text{ whichever is more})$.

Figure 4.8: Recommendations regarding openings in bearing walls

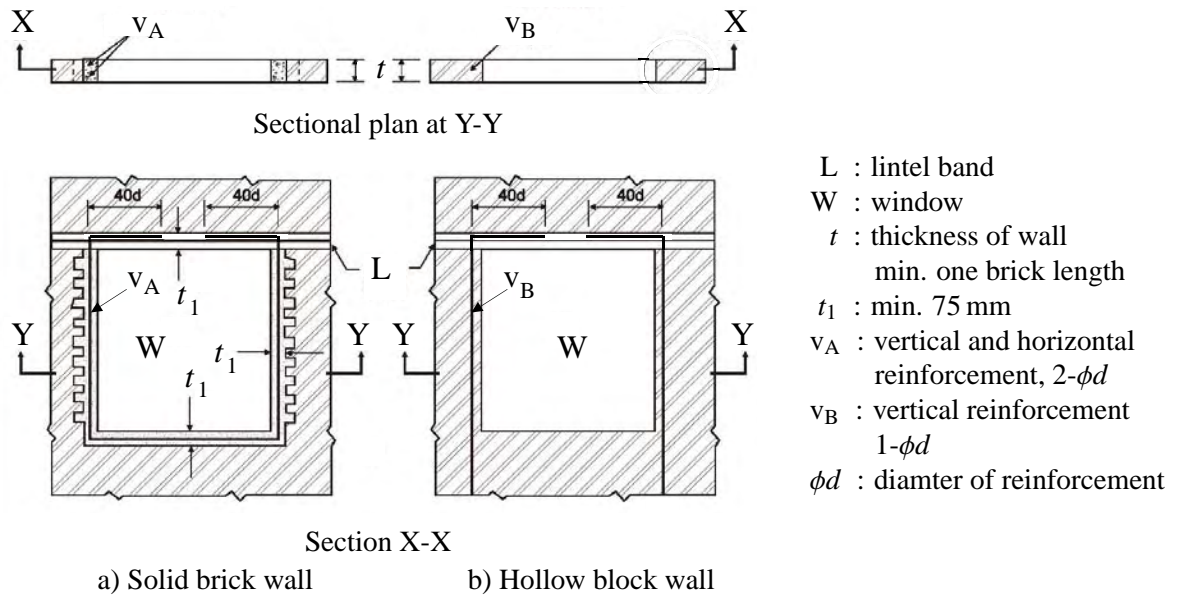


Figure 4.9: Strengthening of masonry walls around openings

4.4.4 Masonry bond

For achieving the full strength of masonry, the usual bonds specified for masonry should be followed so that the vertical joints are broken properly from course to course.

Vertical joint between perpendicular walls

For convenience of construction, builders prefer to make a toothed joint which is many times left hollow and weak. To obtain full bond it is necessary to make a sloping (stepped) joint by making the corners first to a height of 0.6 m and then building the wall in between them. Otherwise, the toothed joint should be made in both the walls alternately in lifts of about 45 cm (see Fig. 4.10).

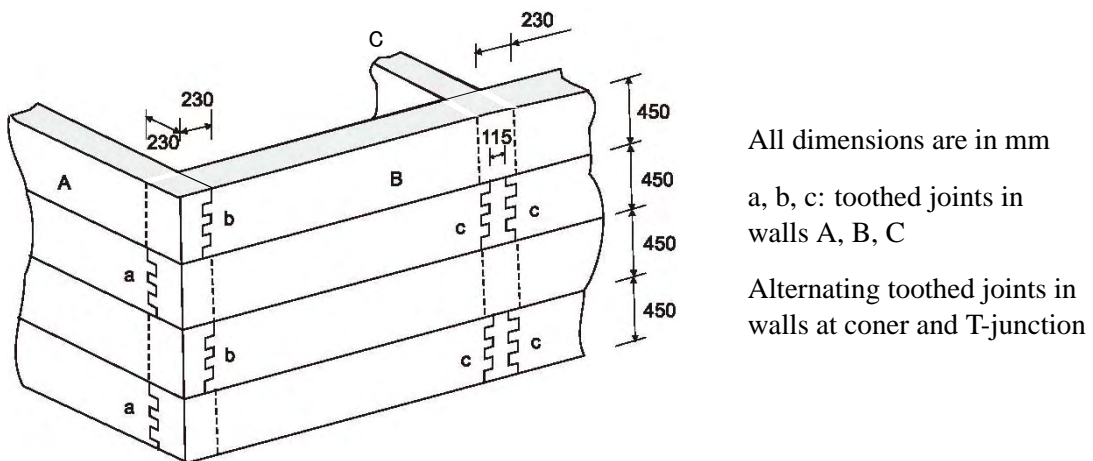


Figure 4.10: A typical detail of masonry

4.5 Horizontal Reinforcement in Walls

Horizontal reinforcing of walls is required for increasing horizontal bending strength against plate action for out of plane inertia load and for tying perpendicular walls together. In partition walls, horizontal reinforcement helps preventing shrinkage and temperature cracks. The following reinforcing arrangements are necessary.

4.5.1 Horizontal bands or ring beams

The most important horizontal reinforcing is through reinforced concrete bands provided continuously through all load bearing longitudinal and transverse walls at plinth, lintel, and roof-eave levels, as well as at top of gables according to requirements as stated hereunder:

- 1) Plinth band: This should be provided where the soil is soft or uneven in its properties as happens in hill areas. It also serves as damp proof course. This band is not too critical.
- 2) Lintel band: This is the most important band and is incorporated in all door and window lintels. Its reinforcement should be extra to the lintel band steel. It must be provided in all storeys as per Table 4.5.
- 3) Roof band: This band is required at eaves level of pitched roofs (see Fig. 4.11) and also below or level with suspended floors which consist of joists and flooring elements, so as to properly integrate them at ends and fix them into the walls.
- 4) Gable band: Masonry gable ends must have the triangular portion of masonry enclosed in a band, the horizontal part will be continuous with the eaves level band on longitudinal walls (see Fig. 4.11).

4.5.2 Cross-sections of bands or ring beams

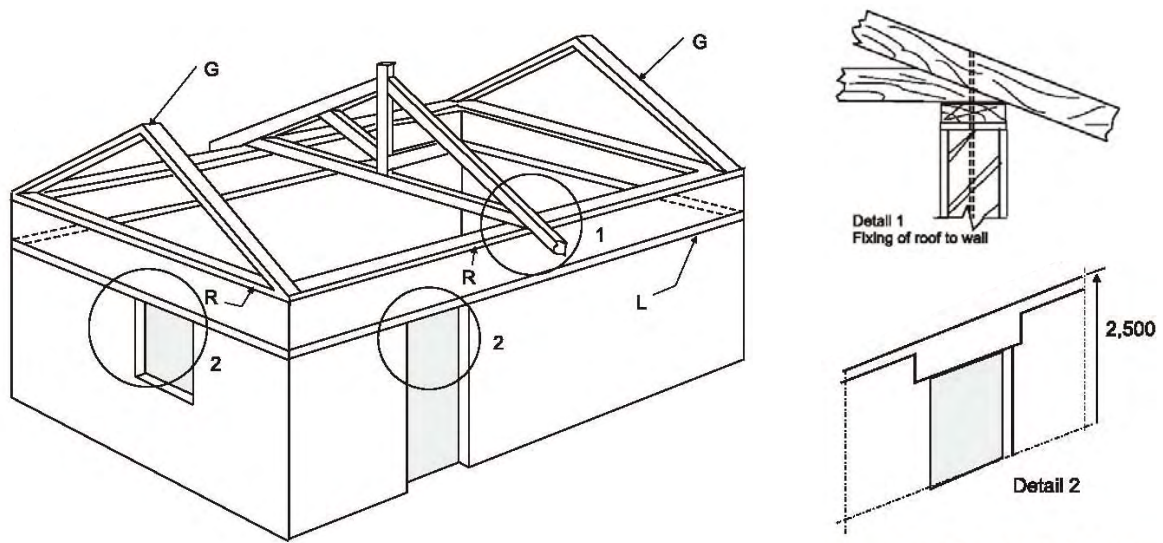
The reinforcement and dimensions of these bands are as follows for wall spans up to 9 m between cross walls or buttresses. For longer spans, the size of band must be calculated.

A band consists of two (or four) longitudinal steel bars with links or stirrups embedded in 75 mm (or 50 mm) thick concrete (see Fig. 4.12). The thickness of band may be made equal to or a multiple of masonry unit and its width should equal the thickness of wall. The steel bars are located close to the wall faces with 25 mm cover and full continuity is provided at corners and junctions. The minimum size of the band and amount of reinforcing will depend upon the unsupported length of wall between cross walls and the effective seismic coefficient based on seismic zone, the importance of the building, type of soil and number of storeys of the building.

Appropriate steel and concrete specifications are recommended for various buildings in Table 4.5. Bands are to be located at the critical levels of the building, namely plinth, lintel, roof and gables according to requirements (see Sec. 4.5.1).

4.5.3 Dowels at corners and junctions

As a supplement to the bands described in Fig. 4.12 a), steel dowel bars may be used at corners and T-junctions to guarantee the box action of walls. Dowels (see Fig. 4.13) are placed in every



Notes: L: Lintel band, R: Roof band, G: Gable band,

1. As an alternative to the gable masonry, a truss or open gable may be used and opening covered with light sheeting.
2. If the wall height up to eaves level is less than or equal to 2.5 m, the lintel band may be omitted and the lintels integrated with the eave level band as shown at detail 2.

Figure 4.11: Gable band and roof band in barrack type buildings

Table 4.5: Recommendation for steel in RC band

Longitudinal steel in RC bands								
Span (m)	Category I		Category I		Category III		Category IV	
	No of bars	Dia of bars (mm)	No of bars	Dia of bars (mm)	No of bars	Dia of bars (mm)	No of bars	Dia of bars (mm)
5	2	12	2	10	2	10	2	10
6	2	16	2	12	2	10	2	10
7	2	16	2	16	2	12	2	10
8	4	12	2	16	2	16	2	12
9	4	16	4	12	2	16	2	12

- i) The width of the RC band is assumed to be the thickness of wall. Wall thickness shall be 200 mm minimum. A cover of 25 mm from face of wall to be maintained. For thicker walls, the quantity of steel need not be increased. (For thinner walls, see Sec. 4.7).
- ii) The vertical thickness of the RC band is to be kept to a minimum of 75 mm where two longitudinal bars are specified and 150 mm where four longitudinal bars are specified.
- iii) Concrete mix to be “1 : 2 : 4” by volume or having 15 MPa cube crushing strength at 28 days.
- iv) The longitudinal bars shall be held in position by steel links or stirrups 6 mm diameter spaced at 150 mm apart (see. Fig. 4.12 a).
- v) Bar diameters are for mild-steel. For high strength deformed bars equivalent diameters may be used.

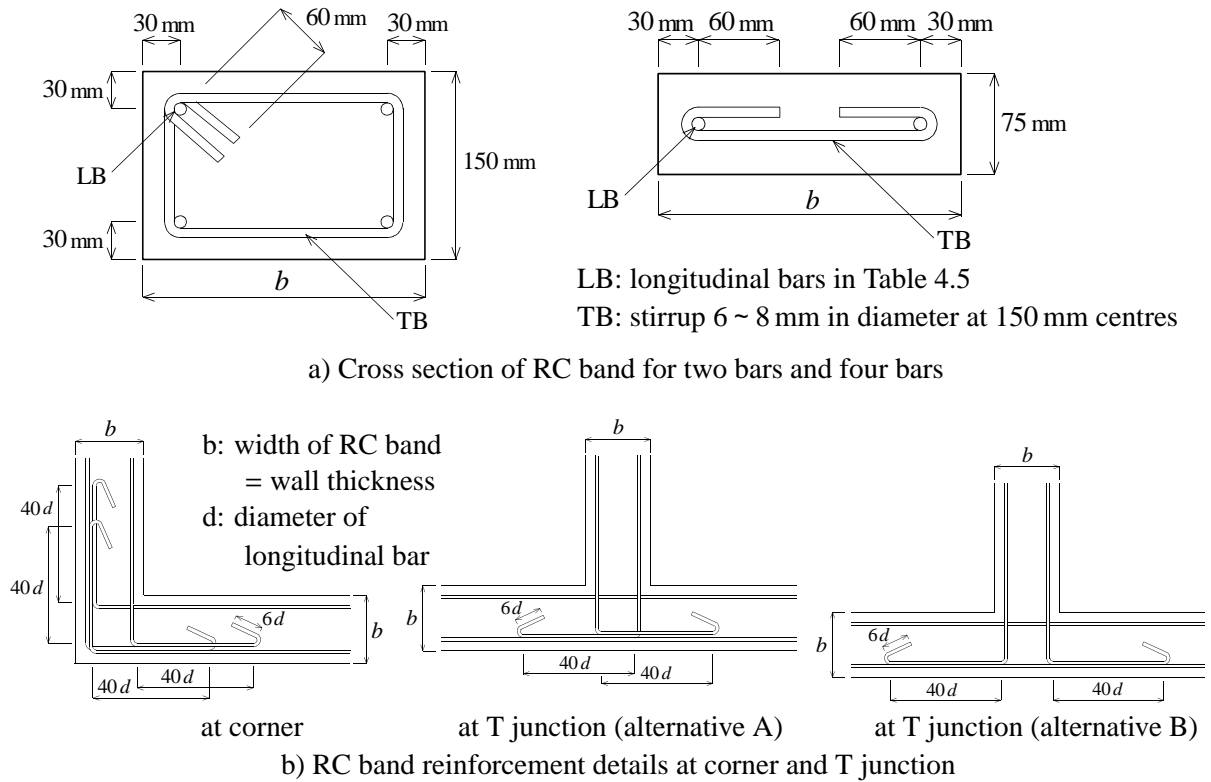


Figure 4.12: Reinforcement in RC band

fourth course or at about 0.5 m intervals and taken into the walls a sufficient length so as to provide full bond strength. Wooden dowels can also be used instead of steel. The dowels do not serve to reinforce the walls in horizontal bending except near the junctions.

4.6 Vertical Reinforcement in Walls

The need for vertical reinforcing of shear walls at critical sections was established in Sec. 2.6.7. The critical sections were the jambs of openings and the corners of walls. The amount of vertical reinforcing steel will depend upon several factors like the number of storeys, storey heights, the effective seismic coefficient based on seismic zone, the importance of building and soil type. Values are given in Table 4.6. The steel bars are to be installed at the critical sections at the corners of walls and jambs of doors, taken from the foundation concrete and covered with cement concrete in cavities made around them during masonry construction. This concrete mix should be “1 : 2 : 4” by volume or richer. Typical arrangements of placing the vertical steel in the brick work are shown in Fig. 4.14.

The jamb steel is shown in Fig. 4.9. The jamb steel of window openings will be easiest to provide in box form around them. The vertical steel of openings may be stopped by embedding it into the lintel band but the vertical steel at corners and junctions of walls must be taken into the floor and roof slabs or roof band.

The total arrangement of providing reinforcing steel in masonry wall construction is schematically shown in Fig. 4.15.

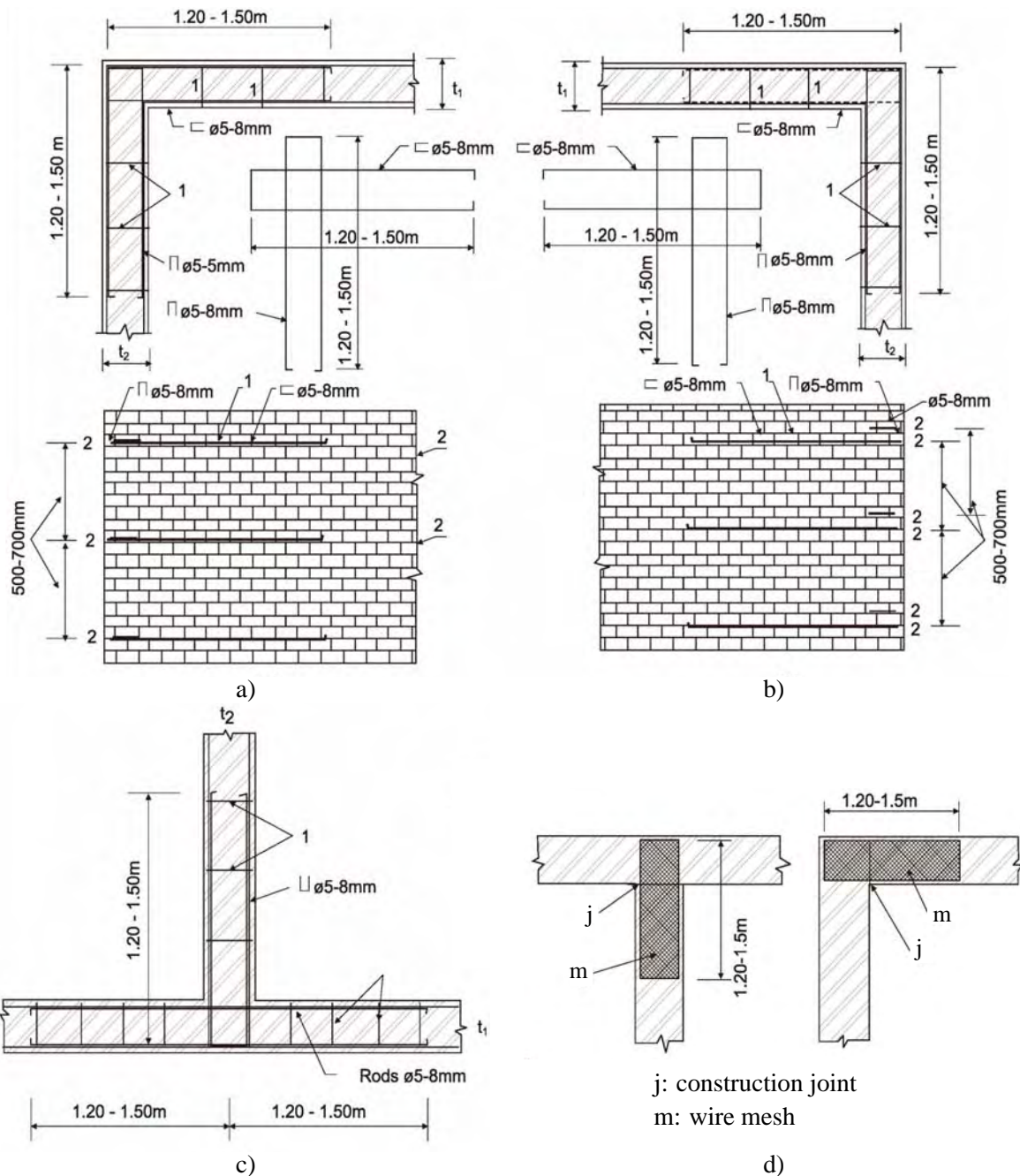


Figure 4.13: Strengthening by dowel or wire fabric
 (t_1 , t_2 : wall thickness, 1: cross links, 2: thick joint to receive dowels)

4.7 Framing of Thin Load Bearing Walls

If load-bearing walls are made thinner than 200 mm, say 150 mm inclusive of plastering on both sides, reinforced concrete framing columns and collar beams are necessary which are constructed to have full bond with the walls. Columns are to be located at all corners and junctions of walls and at not more than 1.5 m apart but so located as to frame up the doors

Table 4.6: Recommendation for vertical steel at critical sections

No of Storeys	Storey	Diameter of mild steel single bar in mm at each critical section for Category*			
		Category I	Category II	Category III	Category IV
One		16	12	12	Nil
Two	Top	16	12	12	Nil
	Bottom	20	16	16	Nil
Three	Top	16	12	12	Nil
	Middle	20	16	12	Nil
	Bottom	20	16	16	Nil
Four	Top			12	12
	Third	**	**	12	12
	Second			12	12
	Bottom			16	12

* Category of construction is defined in Table 3.3. Equivalent area of twisted grip bars or a number of mild steel bars could be used but the diameter should not be less than 12 mm.

** Four storey load bearing wall construction may not be used for categories I and II buildings (see Fig. 3.3).

and windows. Horizontal bands or ring beams are located at all floors and roof as well as lintel levels of the openings. The sequence of construction is first to build the wall up to 4 to 6 courses in height leaving toothed gaps (tooth projection being about 40 mm only) for the columns and secondly to pour "1:2:4" concrete to fill the columns against the walls using wood-forms only for two sides. The column steel should be accurately held in position. The band concrete should be cast on the masonry wall directly so as to develop full bond with it. Such construction is limited to only two storeys maximum in view of its limited vertical load carrying capacity. The horizontal length of walls between cross walls is restricted to 7 m and the storey height to 3 m (see Sec. 4.9 for details of Confined Masonry).

4.8 Reinforcing Details for Hollow Block Masonry

The following details are recommended in placing the horizontal and vertical steel in hollow block masonry using cement-sand or cement concrete blocks.

4.8.1 Horizontal bands

U-shaped blocks may be best used for construction of the horizontal bands at various levels of the storeys as per seismic requirements, as shown in Fig. 4.16.

The amount of horizontal reinforcement is to be taken as 25 % more than that given in Table 4.5 and provided by using four bars and 6 mm diameter stirrups. Other continuity details shall be followed as shown in Fig. 4.12.

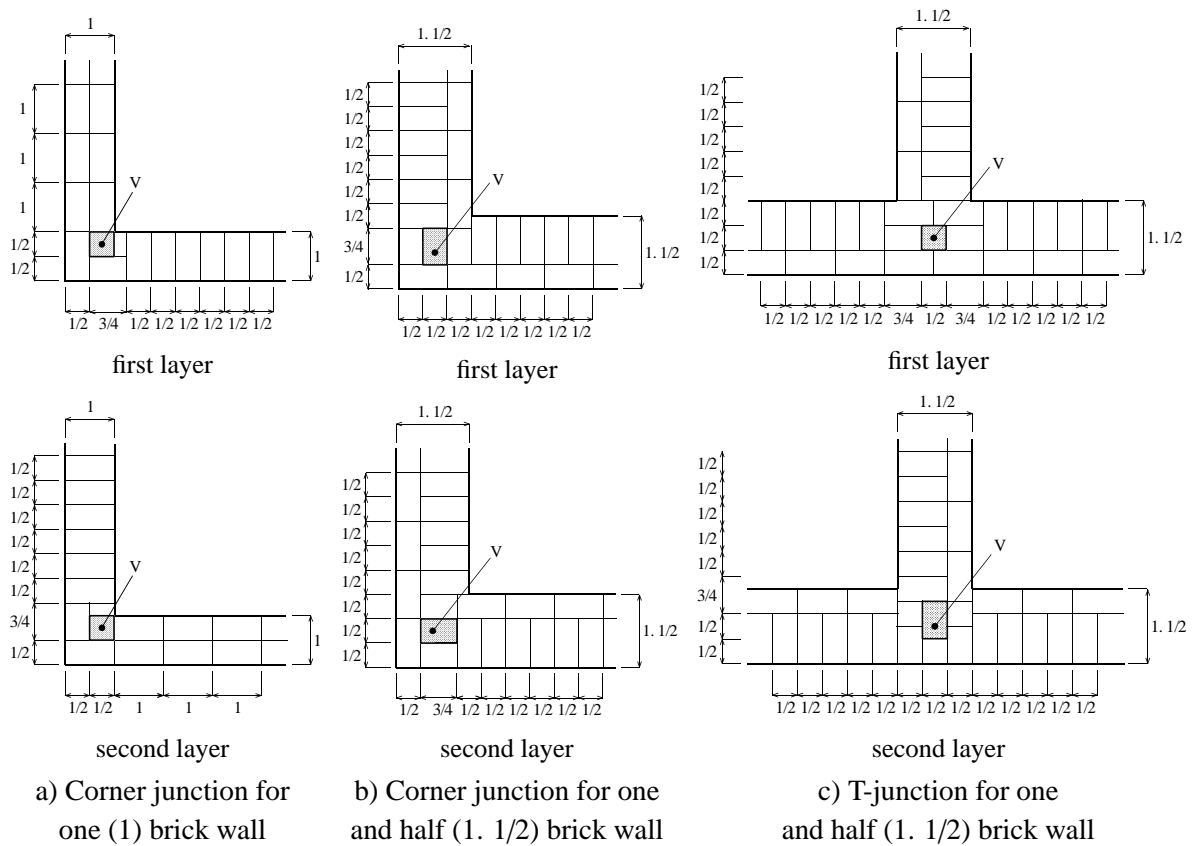


Figure 4.14: Vertical reinforcement (V) in walls

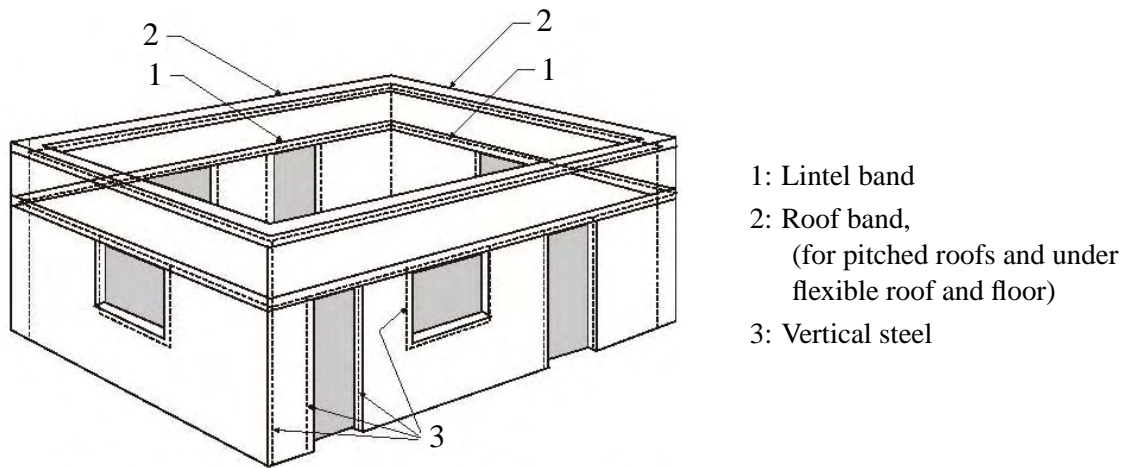


Figure 4.15: Overall arrangement of reinforcing masonry buildings

4.8.2 Vertical reinforcement

The vertical bars as specified in Table 4.6 may conveniently be located inside the cavities of the hollow blocks, one bar per cavity. Where more than one bar is planned, these can be located in

two or three consecutive cavities. The cavities containing bars are to be filled by using micro-concrete “1 : 2 : 3” or cement-coarse sand mortar “1 : 3” and properly rodded for compaction.

A practical difficulty is faced in threading the bars through the hollow blocks since the bars have to be set in footings and have to be kept standing vertically while lifting the blocks whole storey heights, while threading the bar into the cavity and lowering it down to the bedding level. To avoid lifting blocks too high, the bars are made shorter and overlapped with upper portions of bars. This wastes steel and the bond strength in small cavities remains doubtful. For solving this problem, two alternatives may be used (1) use of three sided blocks as shown in Fig. 4.17 or (2) splicing of bars. But vertical bars should not preferably be spliced for single storey buildings. In taller buildings, the splicing of vertical bars where found necessary as shown in Fig. 4.17 done by overlapping by a distance of 50 d and wrapped using binding wire.

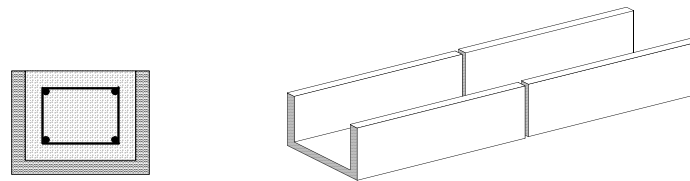
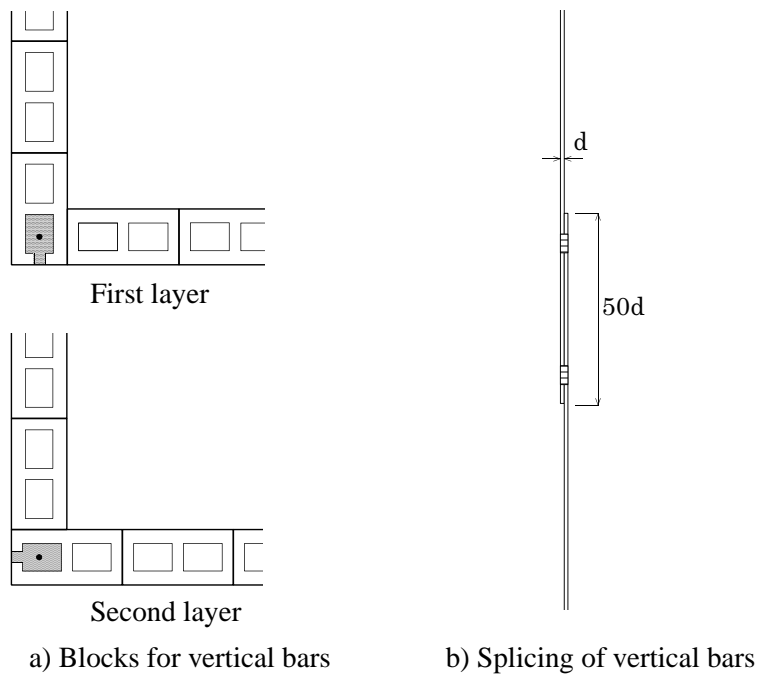


Figure 4.16: U-blocks for horizontal bands



a) Blocks for vertical bars b) Splicing of vertical bars

Figure 4.17: Blocks for vertical bars

4.9 Confined Masonry

4.9.1 Understanding confined masonry construction

Confined masonry construction is a building technology that offers an alternative to “Minimally reinforced masonry” with “RC Bands and Vertical Bars” as per Sec. 4.5 and Sec. 4.6 in this Chapter and “RC Frame Construction”. It consists of masonry walls (made either of clay brick or concrete block units) and horizontal and vertical RC “confining members” built on all four sides of the masonry wall panels. Vertical members are called “tie-columns” or “practical columns” and though they resemble columns in RC frame construction they are of much smaller cross-section. Horizontal elements, called “tie-beams”, resemble beams in RC frame construction, but also of much smaller section. It must be understood that the confining elements are not beams and columns in the way these are used in RC frames. Rather they function as horizontal and vertical ties or bands for resisting tensile stresses and may better be termed as such (see Fig. 4.18).



Single storey house
(Ache, Indonesia)



Two storey house under construction
(Jawa, Indonesia)

Figure 4.18: Confined brick masonry

The structural components of a building using confined masonry walls are as follows (see Figs. 4.19, 4.20 and 4.21):

- **Masonry walls** are load bearing elements, and transmit the gravity loading from the slab(s) and walls above down to the foundation. The “confined” walls also work as bracing panels acting due to the confining tie elements which enable the walls to resist the horizontal earthquake forces.
- **Confining elements** (horizontal and vertical tie elements) provide the necessary tensile strength and ductility to the masonry wall panels and protect them from disintegration in the major earthquakes.
- **Floor and roof slabs** transmit both vertical gravity and lateral loads to the confined masonry walls. In an earthquake the slabs behave like rigid horizontal diaphragms.

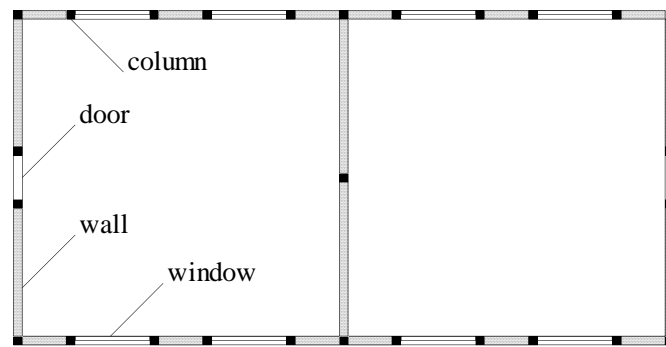


Figure 4.19: Typical confined masonry building plan

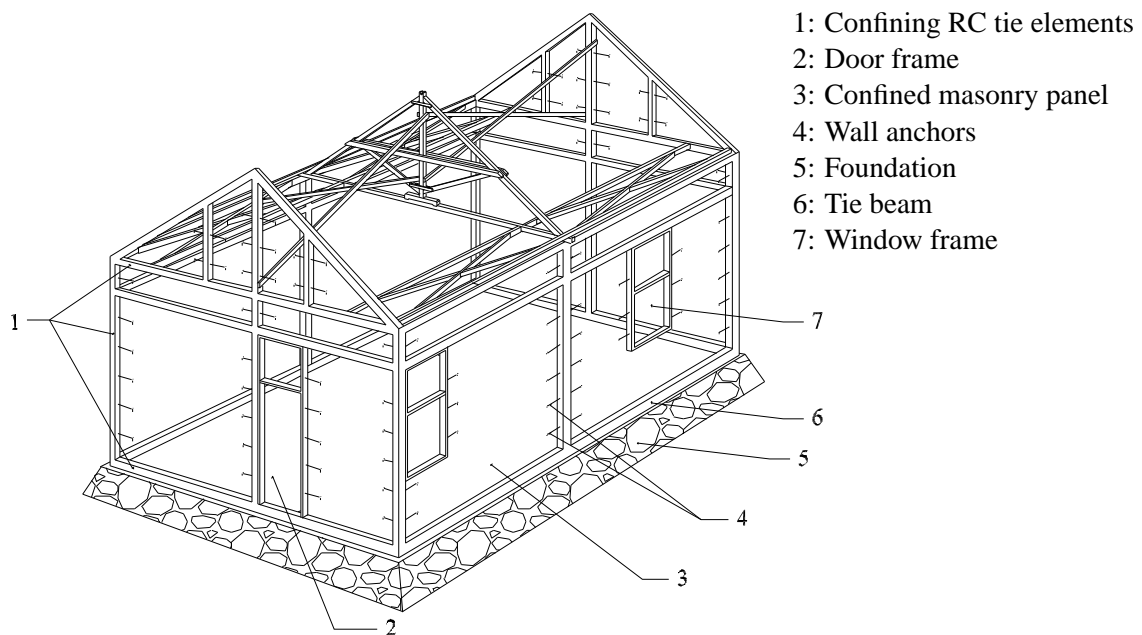


Figure 4.20: Typical single storey confined masonry building

- **Roof truss and ceilings**, for single-storey buildings, transmit both vertical gravity and lateral loads through the roof truss supports. In an earthquake the horizontal braced ceiling framing behaves like a horizontal diaphragm.
- **Plinth bands or tie-beams** transmit the vertical and horizontal loads from the walls down to the foundation. It also protects the ground floor walls from settlement in soft soil conditions.
- **Foundations** transmit the loads from the structure to the ground.

Confined masonry walls can be constructed using different types of masonry units, such as burnt clay bricks, concrete blocks of hollow or solid types or dressed rectangularised stones. Masonry wall construction should follow all details in Sec. 4.4 except those to be modified here. In confined masonry, the reinforcement is concentrated in vertical and horizontal confining

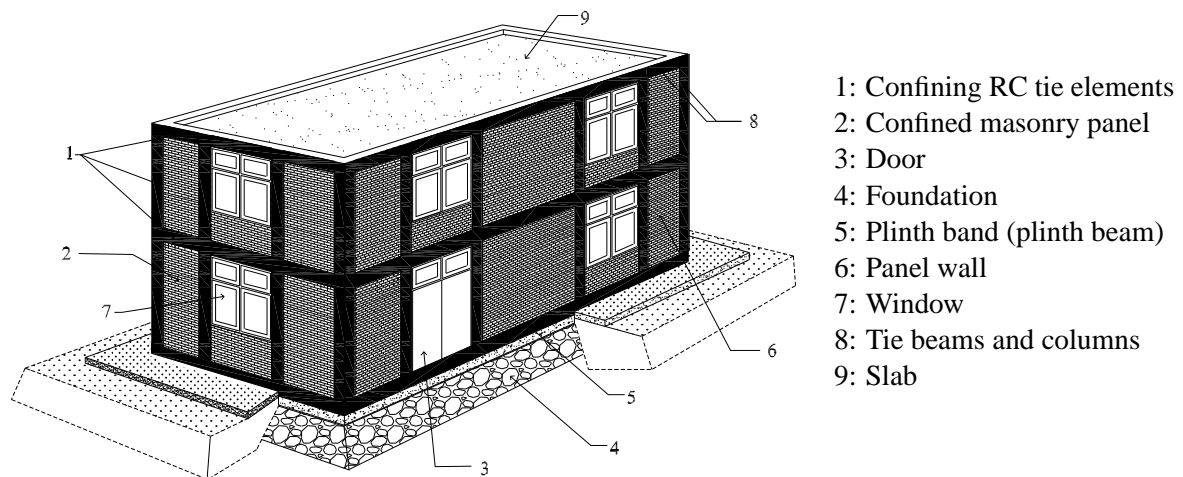


Figure 4.21: Typical 2 storey confined masonry building

elements whereas the masonry walls are usually free of reinforcement, but may be connected with the confining elements using steel dowels.

4.9.2 Difference between confined masonry and RC frame construction

The appearance of finished confined masonry construction and a RC frame construction with masonry infills may look alike but in terms of load carrying schemes they are very different. The differences are related to the construction sequence, as well as to the manner in which these structures resist gravity and lateral loads (see Fig. 4.22). Whereas in RC Frames, the RC columns and beams carry the vertical gravity as well as the lateral loads from earthquakes or wind storms unaided by the masonry infills, in the case of confined masonry buildings, the wall panels are the main load carrying elements (both vertical and horizontal) aided by the confining elements for resisting tensile forces.

Note) Because RC frame members mainly resist seismic forces by bending and shear actions, their cross-sectional areas are expected to be far larger than the tie members of confined masonry construction.

4.9.3 Guide to earthquake-resistant confined masonry construction

The satisfactory earthquake performance of confined masonry is due to the joint action of masonry walls and the reinforced concrete confining elements. Properly designed and built, confined masonry buildings are expected to exhibit good performance even in the Maximum Considered Earthquake (MCE). Moderate cracking in the elements is likely, but the collapse of a building will be highly improbable. Depending on the crushing strength of the masonry units as per requirements of country's masonry code, confined masonry buildings may be constructed up to five storeys in height for various Building Categories defined in Table 3.3 as suggested below:

Categories I and II: up to 4 storeys

Categories III and IV: up to 5 storeys

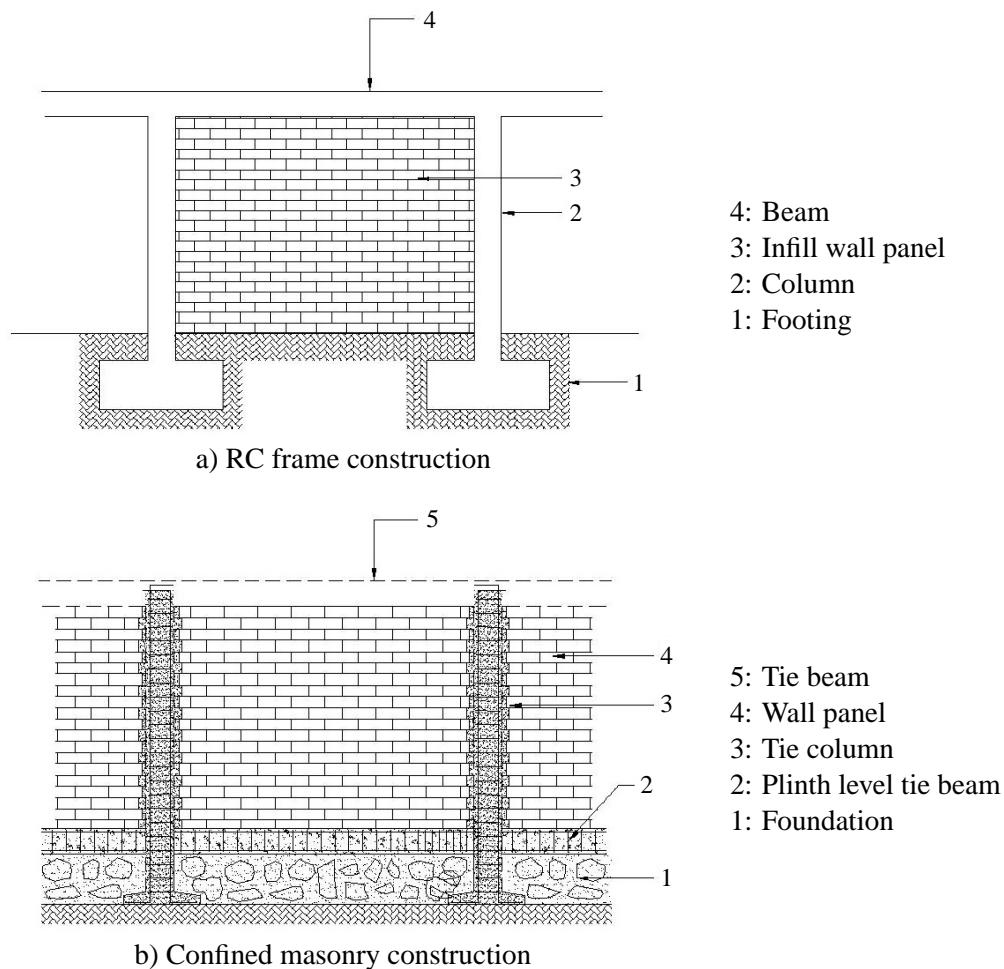


Figure 4.22: RC frame and confined masonry construction

1) Building Configuration

The architectural configuration concepts as highlighted in Sec. 3.3 are necessary in confined masonry construction.

2) Confining Elements

The tie-beams should be placed at plinth and every floor level. Vertical spacing of tie-beams should not exceed 3 m. The tie-columns should be placed at a maximum spacing of 4 m in 200 mm or thicker walls and 3 m in 100-114 mm thick walls, as well as at the following locations:

- a) at the corners of rooms and all wall-to-wall intersections,
- b) at the free end of a wall,
- c) at the jambs of doors/windows of 900 mm or wider openings.

3) Walls

The wall thickness may be kept 100 or 114 mm in the case of one to two storey high residential buildings. But for all important buildings as defined in Sec. 3.2.2, and those of more than two storeys height, the thickness should be 200 or 230 mm or larger as required by the country code, and the mortar shall be as per Sec. 4.4.1. At least two fully confined panels should be provided in each direction of the building. The earthquake performance of a confined masonry building depends on the shear resistance of masonry walls. Therefore, it is essential to provide an adequate number of confined walls in each direction. The walls should be placed preferably at the periphery so as to minimize torsion of the building in an earthquake.

The following shear F_s may be permitted on the area of mortar bed joint under seismic loads:

$$F_s = 1.33\left(f_s + \frac{f_d}{6}\right) \leq 0.70 \text{ N/mm}^2 \quad (4.1)$$

where $f_s = 0.1$ for “1 : 5” cement- sand mortar; 0.15 for “1 : 4” and 0.2 for “1 : 3” mortar mix, and, f_d = the actual compressive stress on the bed joint due to dead loads.

4) Wall Density

The wall density in a storey is defined as the total solid cross sectional areas of all confined wall panels in the storey in one direction divided by the sum of the roof and floor plan areas of all floors above the storey under consideration. Wall density of at least 1.0% in each of two orthogonal directions is required to ensure good earthquake performance of confined masonry residential buildings in Seismic Zone A. To achieve adequate earthquake performance in the lower Seismic Zones, wall density of at least 0.8% in Zone B and 0.6% in Zone C should be achieved.

For important buildings, the minimum wall area should be increased to 1.2 times that of the residential buildings.

4.9.4 Construction details of confined masonry

1) Construction of walls

The aim is to use good quality masonry units and mortar, as well as good quality workmanship.

- Minimum wall thickness is 100 mm. Wall panel height to thickness ratio should not exceed 30.
- Toothed edges should be left on each side of the wall; the tooth projection may be kept about ≤ 40 mm to achieve full concrete filling in the teeth space. Use of horizontal dowels instead or in addition to teething can be made at the wall-to-column interface.
- Concrete is to be poured in the tie-columns upon completion of desirable wall height.

Note) The construction sequence is opposite to RC frames where the beams and columns are cast before placing the masonry infill. This sequence can be adopted for confined masonry only if adequate anchor bars are provided between RC columns and infill walls (see Fig. 4.24).

- Bricks or other masonry units must be wetted before casting of concrete.

- Formwork support must be provided on two sides of the wall (see Fig. 4.23). The concrete needs to be vibrated to fill the teeth space thoroughly.

2) Construction of the column and beam confining elements

A single-storey confined masonry building is schematically shown in Fig. 4.24 and the reinforcing bars are in Fig. 4.25. Details for RC frames are in Chapter 8.

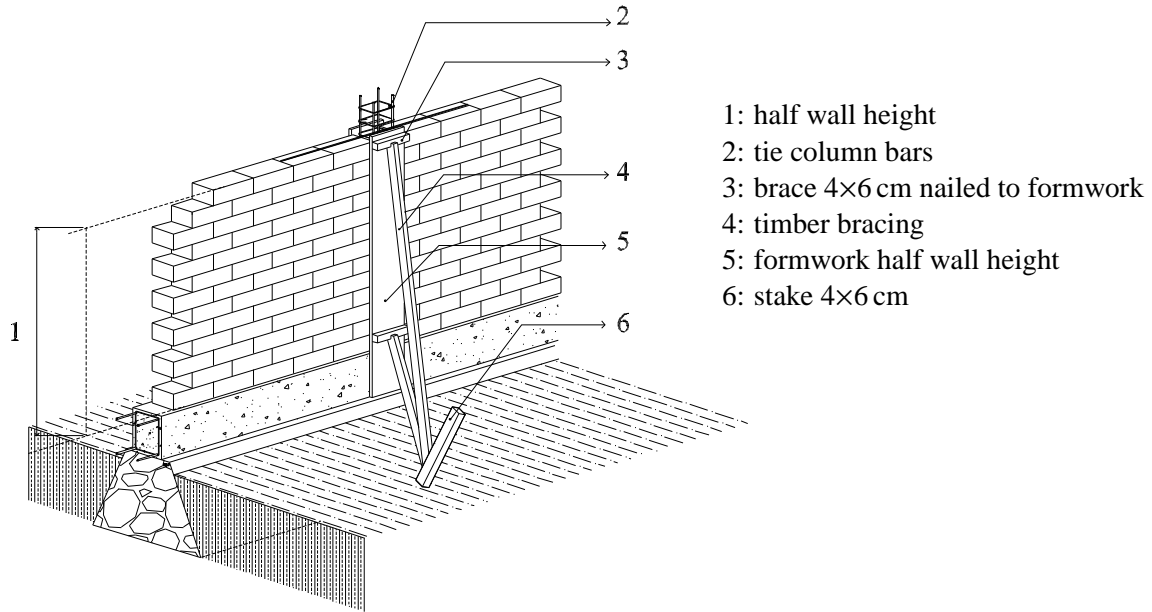


Figure 4.23: Framework for tie columns

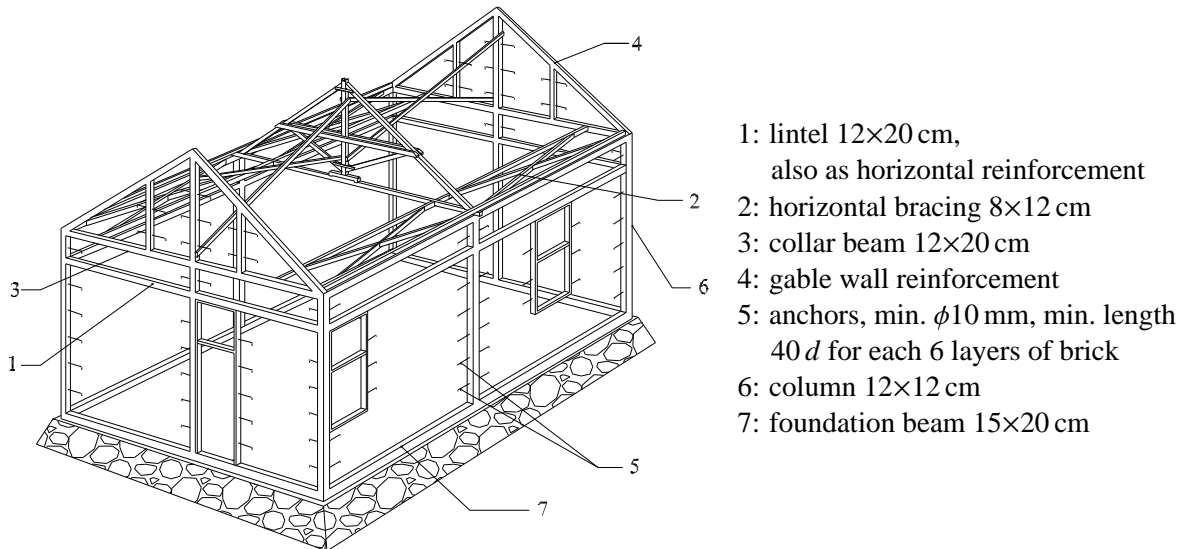


Figure 4.24: Single storey confined masonry house

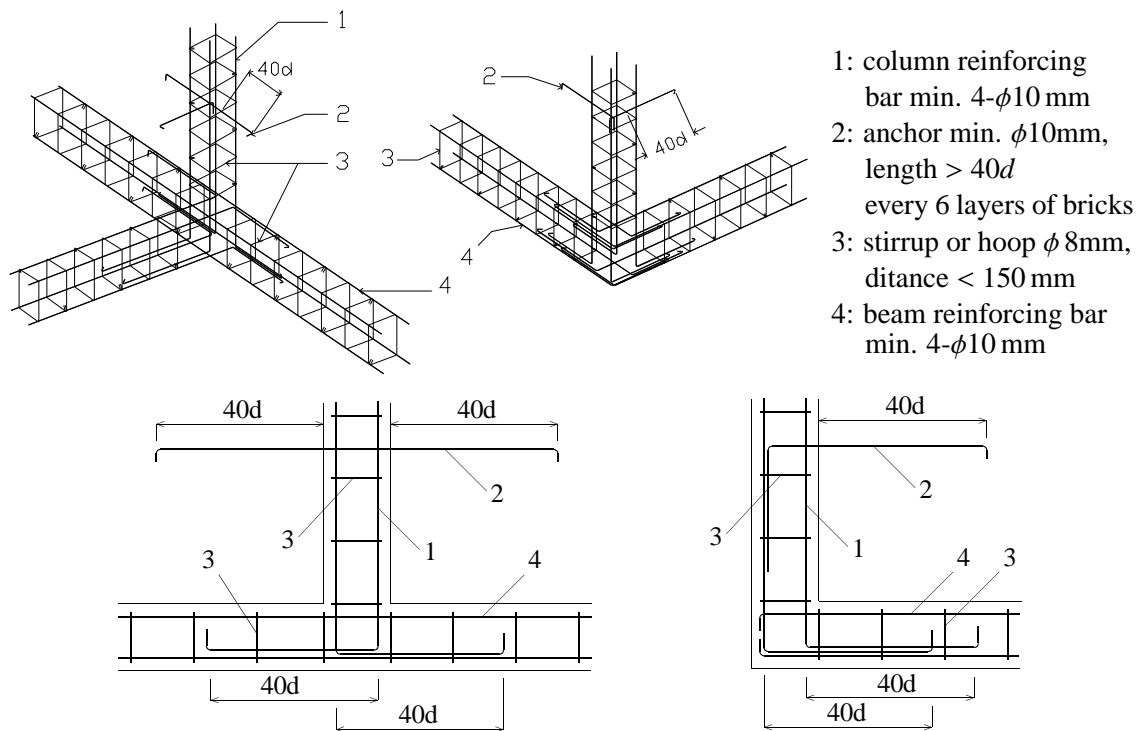


Figure 4.25: Recommended minimum reinforcing bars

4.9.5 Concluding remarks

An important question arises: How to choose between the reinforcing methods of masonry buildings detailed in Sec. 4.5 and Sec. 4.6 and confined masonry construction? Essentially both reinforcing techniques rely on the joint action of masonry and the reinforcement acting together for resisting the lateral forces of earthquake or wind. Therefore, both the techniques result into adequate performance even under high earthquake intensities.

In contrast to the technique of Sec. 4.5 and Sec. 4.6, confined masonry construction requires more skilled supervision and control on the process of joining the wall masonry with the tie columns. Also confined masonry may be costlier than the techniques of reinforcing masonry walls as per Sec. 4.5 and Sec. 4.6. The confined masonry construction will give the appearance of reinforced concrete frame construction inspiring more confidence.

4.10 Foundation and Plinth

Foundations are critical to provide safety to houses under flood and earthquake conditions.

- i) A detailed soil exploration should be carried out to determine the soil profile at least down to a depth of 3 m below ground level. The underground water table should be noted and the bearing capacity of soil at suitable depths should be assessed to arrive at a correct foundation system and design details.

- ii) In case of cohesive (clayey, silty clayey or clayey silty) soils, square brick pedestal foundation (see Fig.4.26) may be used with foundation depth of 1.5 m below ground level using a safe bearing capacity of 7 to 9 t/m² (70 to 90 kN/m²). However, for deeper scouring depths at any particular location foundation depth may need to be increased below silty clay soil.

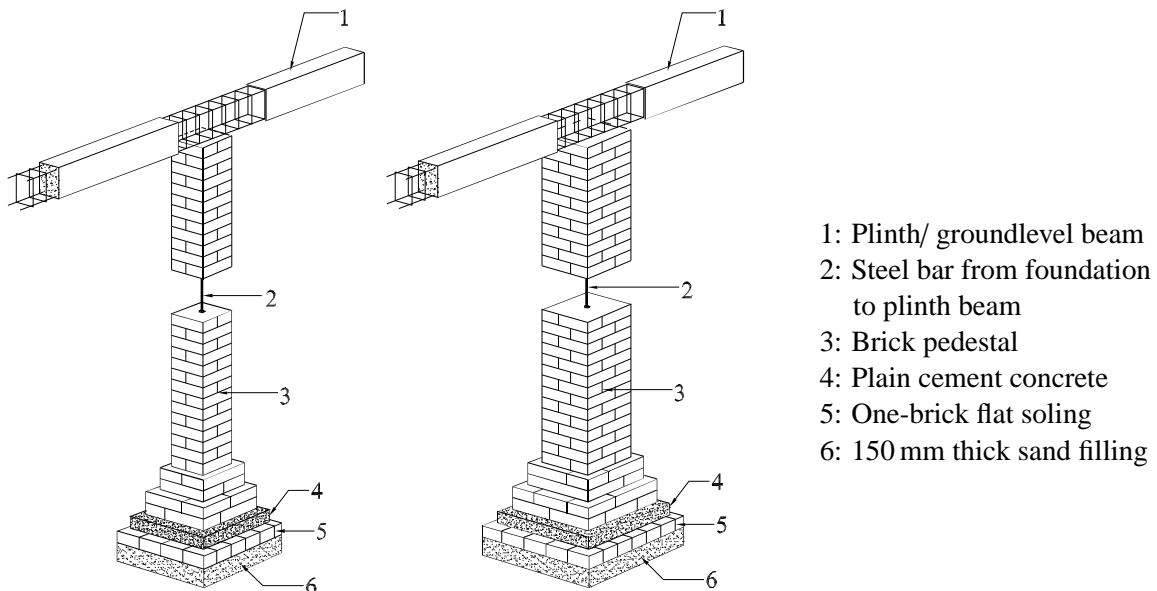


Figure 4.26: Brick pedestal foundation

- iii) Where stiff soil at a depth of about 0.6 m below ground level will not be eroded either under flowing flood water or subject to liquefaction, wall foundations of a plinth level RC band and vertical bars at each corner of rooms if required for the seismic zone can be used (see Fig. 4.27).
- iv) Where non-cohesive, soft alluvial soils may be saturated during floods or by a high water table, with possibility of scour, a minimum depth of 1.5 m below ground level is recommended for the pedestal footings. For deeper scour or liquefaction during earthquake, pile foundations are recommended. A deep RC pile foundation with an appropriate concrete bulbs may be used. In such a situation, a depth of 3 to 8 m may be required, based on the liquefaction potential of the soil strata. A minimum depth of 3 m for single or two storey houses should be adopted. In case the pile foundations are required, they should be designed by qualified engineers.
- v) Foundation and plinth masonry should be constructed using backed or concrete blocks. The pedestal or pile foundations require a reinforced concrete beam at plinth level (or at ground level) to support the superstructure. Reinforcement from the piles and piers shall be anchored into the plinth beam as per Fig.4.27.
- vi) The distance between two pedestal or pile foundations shall not be more than 1.5 m.
- vii) A plinth beam to tie the pedestals or piles shall be designed based on superstructure loads and distance between pedestals or piles.

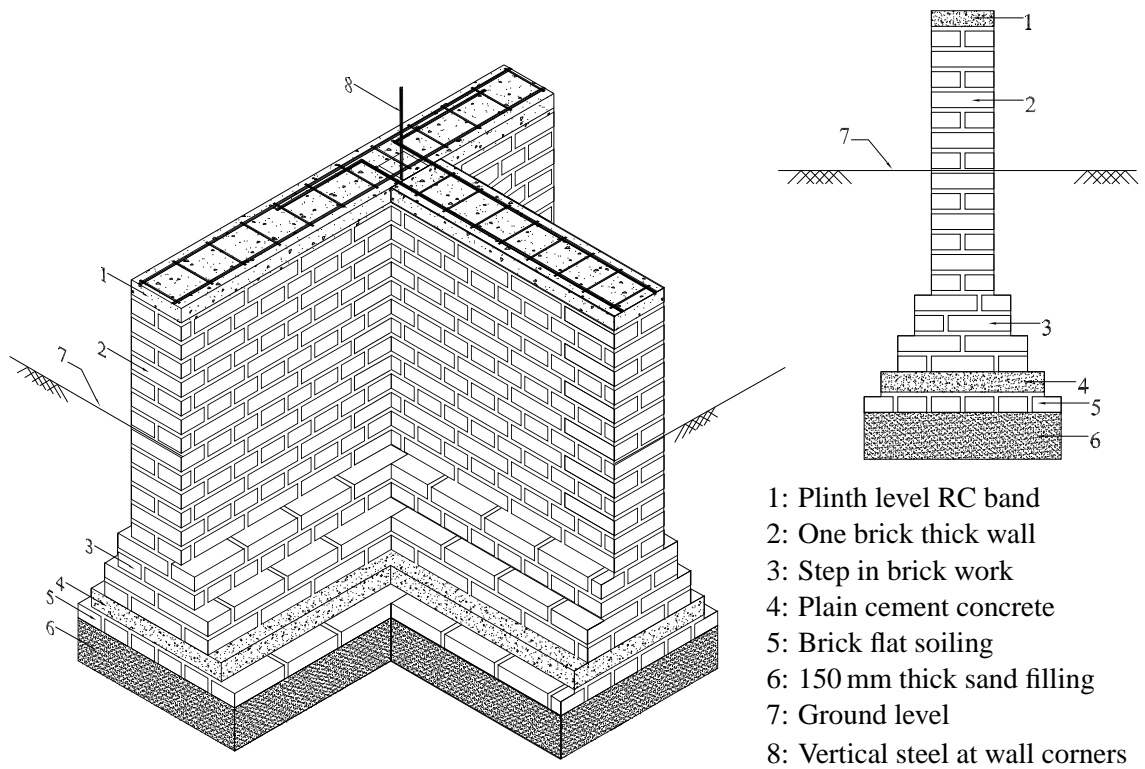


Figure 4.27: Strip footing for wall

- viii) Whenever a tie beam connecting pile/post/pedestal foundations is provided at plinth level, a toe wall between the pedestal/post/piles should be constructed to hold the earth filling of the plinth. This toe wall can be constructed with lean cement mortar of “1:8” 25 cm below ground.

Chapter 5

Stone Buildings

5.1 Introduction

Stone buildings using fully dressed rectangular shape stone units, or cast solid blocks consisting of large stone pieces in cement concrete mix “1 : 3 : 6” may be built according to the confined masonry details given in Chapter 4. These details also apply to the random-rubble and half-dressed stone buildings except where details in this chapter are provided.



Figure 5.1: Example of stone buildings (Nepal)

5.2 Typical Damage and Failure of Stone Buildings

Random rubble and half dressed stone buildings (see Fig. 5.2), have suffered extensive damage and complete collapse during past earthquakes having intensities of MSK VII and more.

The main ways in which such buildings are damaged:

- Separation of walls at corners and T-junctions takes place even more easily than in brick buildings due to poorer connection between the walls.

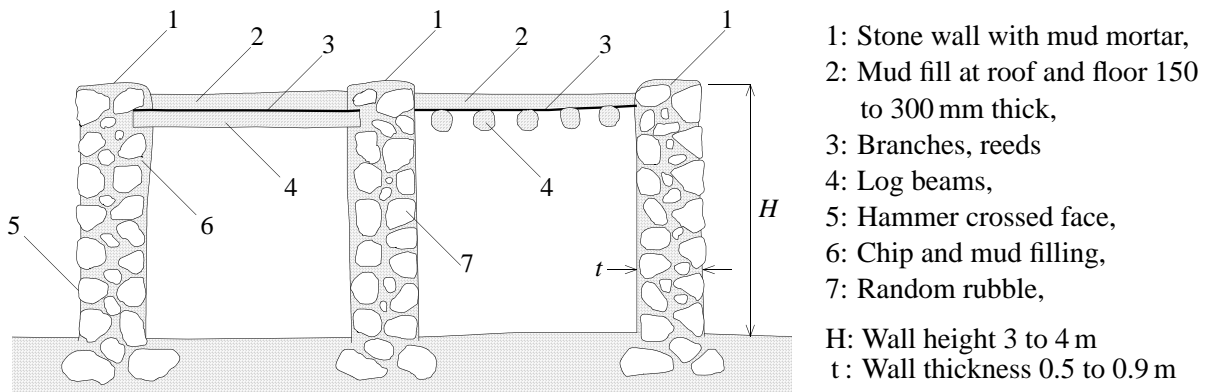


Figure 5.2: Schematic cross section through a traditional stone house

- Delamination and bulging of walls. That is, vertical separation of internal and external wythes (see Figs. 5.3 and 5.4 left). This occurs due mainly to the absence of “through” or bond stones and weak mortar filling between the wythes. In half-dressed stone masonry, the surface stones are pyramidal in shape having more or less an edge contact one over each other, so the stones have an unstable equilibrium and easily disturbed under shaking.
- Crumbling and collapsing of bulged wythes after delamination under the heavy weight of roofs/floors leading to collapse of the roof along with walls, or causing large gaps in walls.
- Outward overturning of stone walls after separation at corners due to inertia loads of roofs and floors and their own inertia when the roofs are incapable of acting as horizontal diaphragms. This is common when the roof consists of round poles, reed matting and clay covering.

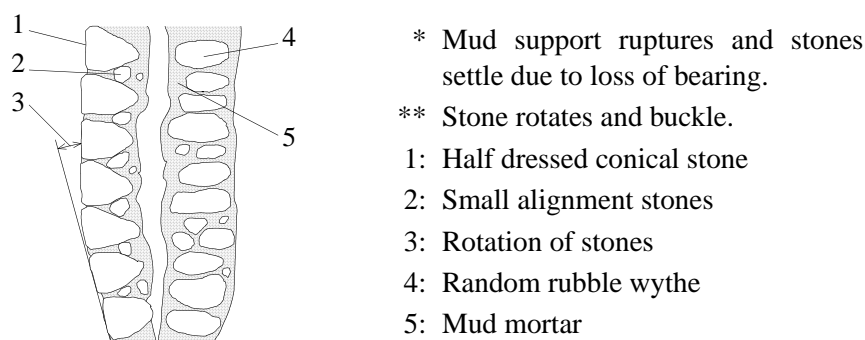


Figure 5.3: Delaminated wall with buckled wythes

Frequently, such stone houses under MSK VII or higher intensities, are completely shattered, and the walls reduced to only heaps of rubble. People are often killed. Such buildings, without the seismic improvements suggested below are dangerous, particularly in seismic zones defined by Zones A and B in Sec. 3.2.1 (Figs. 5.4 right and 4.6).

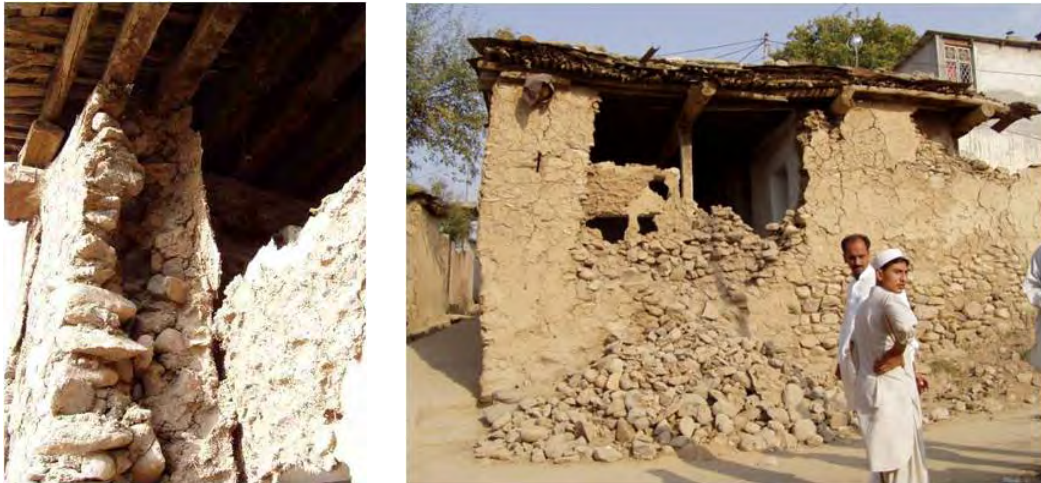


Figure 5.4: Damage to stone masonry
(2005 Northern Pakistan Earthquake)

5.3 Typical Structural Properties

Test data on the strength characteristics of random rubble and half-dressed stone masonry are unavailable. It is, however known from experience that its compressive strength even while using clay mud as mortar can support three storeys but its tensile strength is near to zero. Shear strength will only be due to frictional resistance.

5.4 General Construction Aspects

5.4.1 Overall dimensions

- The height of the construction is restricted to one storey of category I and II buildings and two storeys of categories III and IV buildings (see Table 3.3). Where light sheeted roof is used, an attic floor may also be used.
- The height of a storey should be less than 3.5 m.
- The wall thickness should be as small as feasible, say 350 to 450 mm.
- The unsupported length of a wall between cross walls is limited to 7 m.
- For longer walls, buttresses may be used at intermediate points not farther apart than 3 m. The size of buttress may be kept as: thickness = top width = t and base width = $h/6$, where t = thickness of wall and h = actual wall height.

5.4.2 Mortar

- Clay mud mortar should be avoided as far as possible.
- Mortars as specified in Table 4.4 can be used for stone walls.

5.4.3 Openings in walls

- Openings should be as small and as centrally located as practicable.
- The recommended opening limitations are shown in Fig. 5.5.
- Ventilators, where used, should be 450×450 mm or smaller.

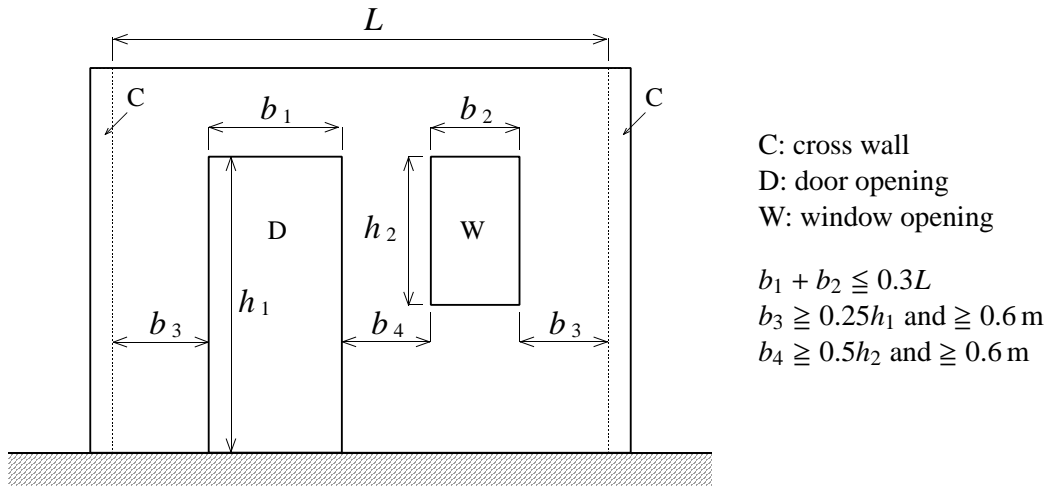


Figure 5.5: Recommended openings in rubble masonry bearing walls

5.4.4 Masonry bond

- Random rubble masonry construction should be laid in courses not more than 0.6 m high.
- “Through” stones of full length equal to the wall thickness should be used in every 0.6 m lift at not more than 1.2 m apart horizontally. If full length stones are not available, stones in pairs, each of about 3/4 of the wall thickness may be used in place of one full length stone so as to provide an overlap between them (see Fig. 5.6).
- In place of “through” stones, bonding elements of steel bars 8 to 10 mm ϕ in S-shape or bent as a hooked link may be used with a cover of 25 mm from each face of the wall (see Fig. 5.6).
- Alternatively, wood lengths of 38×38 mm cross-section or equivalent may be used as the “through” stones. The wood should be well preserved through seasoning and chemical treatment so as to be durable against weathering action and insect attack (see Fig. 5.6).
- Use of long stones should also be made at corners and junction of walls to break the vertical joint and provide bonding between perpendicular walls.

5.4.5 Horizontal reinforcing of walls

All the horizontal reinforcing recommended for brick buildings in Secs. 4.5.1, 4.5.2 and 4.5.3 may be used for random rubble construction as well.

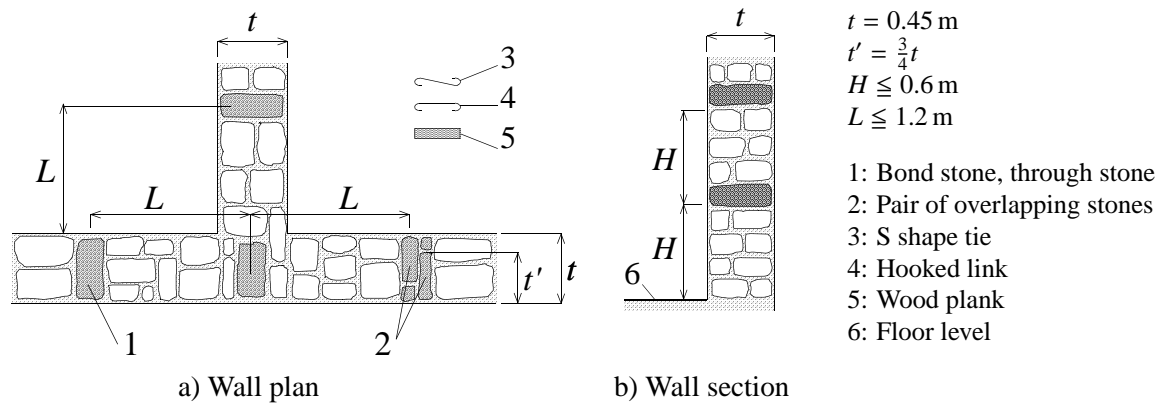
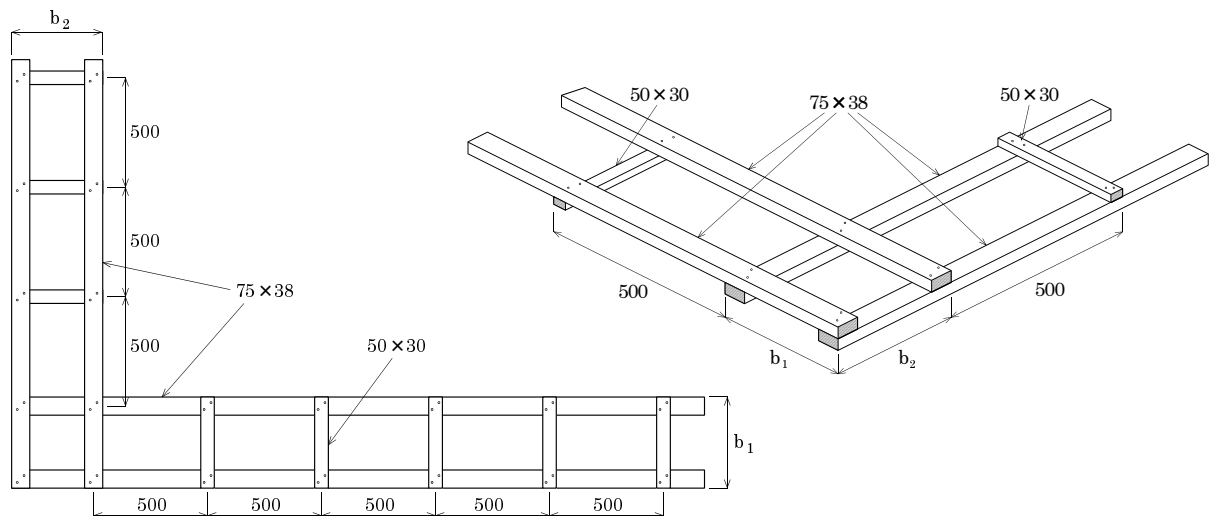


Figure 5.6: “Through” stones or “bond” elements
(If bond stones are not available, then provide wood planks or steel bars 8mm diameter in the stone wall.)

As an alternative to steel reinforcing bars, wooden planks of rectangular section, effectively spliced longitudinally and held by lateral members in lattice form may be used where timber is available and more economical. Recommended sections are shown in Fig. 5.7 and Fig. 5.8.



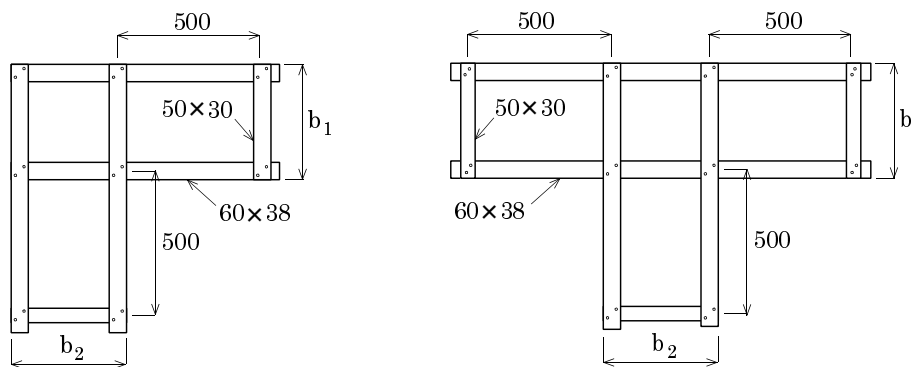
b_1, b_2 : wall thickness, All dimensions are in mm.

Figure 5.7: Lintel level wooden band on all load bearing walls

5.4.6 Vertical reinforcing of walls

The amount of vertical steel in masonry walls required to be provided at the corners and T-junctions of walls and at jambs of openings is shown in Table 5.1.

Buildings of Category IV need not have vertical steel at all. For providing a vertical bar in stone masonry a casing pipe is recommended around which the masonry is built to heights of 0.6 m (see Fig. 5.9). The pipe is kept loose by rotating it during masonry construction. Then the



b_1, b_2 : wall thickness, All dimensions are in mm.

- a) Wood dowel at corner of wall b) Wood dowel at T-junctions at about 900 above floor

Figure 5.8: Detail of wood reinforcing at corners and T-junctions

Table 5.1: Recommended vertical steel at critical sections

Number of Storeys	Diameter (mm) of mild steel single bar at each critical section for category*		
	Category I	Category II	Category III
One	20	16	14
Two	**	**	16

* Category of construction is defined in Table 3.3. Equivalent area of twisted grip bars or a number of mild steel bars could be used alternatively, but the diameter should not be less than 12 mm.

** Two storey buildings with load bearing stone masonry of random rubble or half-dressed stone type are not recommended in Categories I and II.

casing pipe is raised and the cavity below is filled with “1 : 2 : 4” concrete mix and rodded to compact it. The concrete will not only provide the bond between the bar and the masonry but will also protect the bar from corrosion.

The jamb steel may be taken from the footing up to the lintel band and anchored into it. The corner steel must be taken from the footing up to the roof slab or roof band and anchored into it.

5.4.7 Foundations

Foundation of stone masonry building is covered by the considerations as given earlier under Sec.4.10. (See also Sec. 2.5.6 and Sec. 3.8.) A section of the stone foundation for stone walls is shown in Fig.5.10.

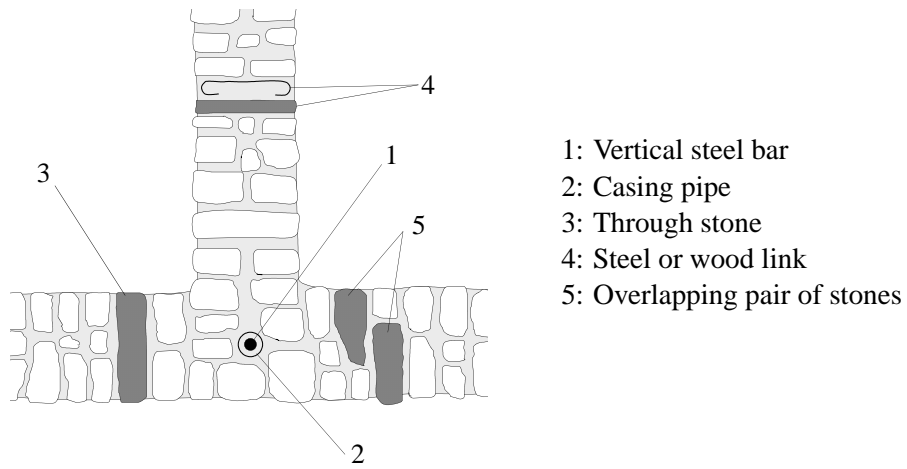


Figure 5.9: Vertical steel in rubble masonry

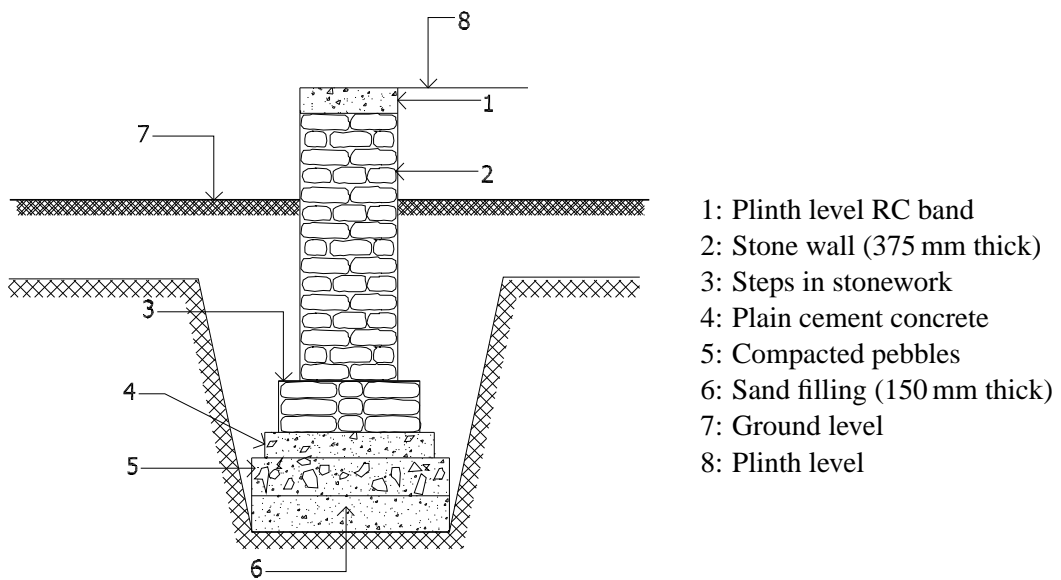


Figure 5.10: Open foundation stone wall footing

Chapter 6

Wooden Buildings

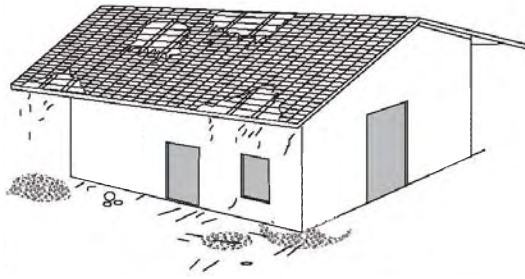
6.1 Introduction

Wood has a high strength per unit weight and is very suitable for earthquake resistant construction. However heavy cladding walls impose high lateral loads on a frame beyond its structural capacity. Although seismically suitable, use of timber is declining in building construction even where it used to be the prevalent material on account of vanishing forests due to population pressure. The situation in many countries of the world has in fact become rather alarming on account of the ecological imbalance. Hence use of timber must be restricted in building construction for seismic strengthening weaker construction such as adobe and masonry. Timber is suitable in those areas where it is still abundantly available as a renewable resource.

6.2 Typical Damage and Failure

The typical features of earthquake damage to wooden buildings are as follows:

- 1) Roof tiles slide down during earthquakes if they are not properly fastened to the roof and injure people (see Fig. 6.1).
- 2) The failure of the joints connecting columns and beams frequently occurs. As the inclination of the building increases, its restoring force against distortion decreases due to the structural deterioration and the roof weight, and finally leads to the complete collapse of the building (see Figs. 6.2 and 6.3).
- 3) In the case of two storey buildings, the first storey usually suffers more severe damage than the second storey. Often the first storey collapses while the second story is undamaged (see Figs. 6.4 and 6.5).
- 4) Damage is considerably influenced by the ground conditions. In general, the softer the subsoil, the more severe the damage to the building. The damage due to differential settlements of foundations is also observed for buildings on soft ground. Damage due to liquefaction of the subsoil occurs to buildings on saturated soft sand.
- 5) Sliding of the building as a whole is sometimes seen when there are no anchor bolts connecting the sill to the foundation (see Fig. 6.6). Damage to superstructure is also observed when the foundation cannot resist the lateral force caused by earthquake motion.



(Damaged wooden house in 1978 Miyagi-ken-
oki Earthquake, Japan)

Figure 6.1: Falling roof tiles

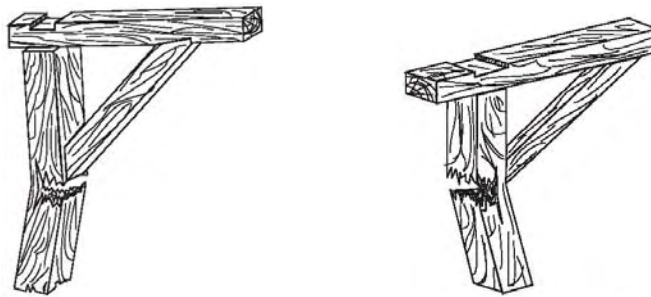


Figure 6.2: Rupture of columns at the connection of knee brace and column

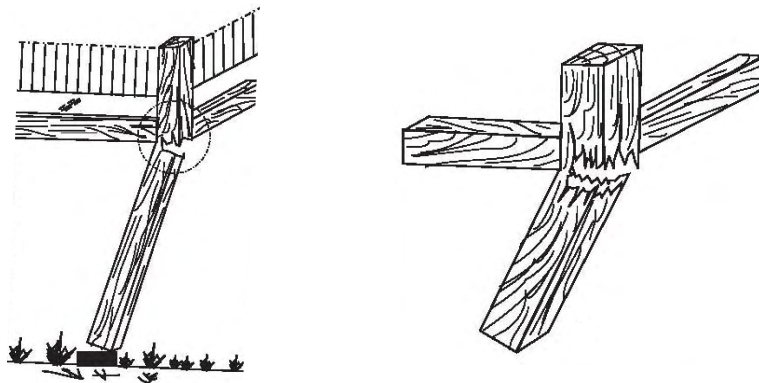


Figure 6.3: Rupture of column due to notching at the connection of beam and column

- 6) Other types of damage in wooden buildings are failure of wooden gable frames (see Fig. 6.7), and failure due to rupture of bottom chords of roof truss (see Fig. 6.8).
- 7) Severe destruction of wooden buildings has been due to fire resulting from electrical short-circuiting or kitchen fires during earthquake shaking and causing conflagration. Precautions against fire are most important for wooden buildings.

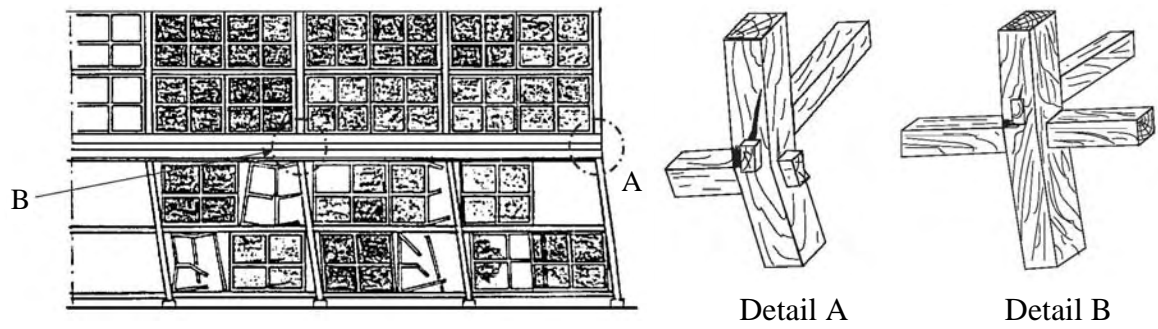


Figure 6.4: Damage to a building with no diagonal bracing



Figure 6.5: Damage to wooden houses (1995 Kobe Earthquake, Japan)

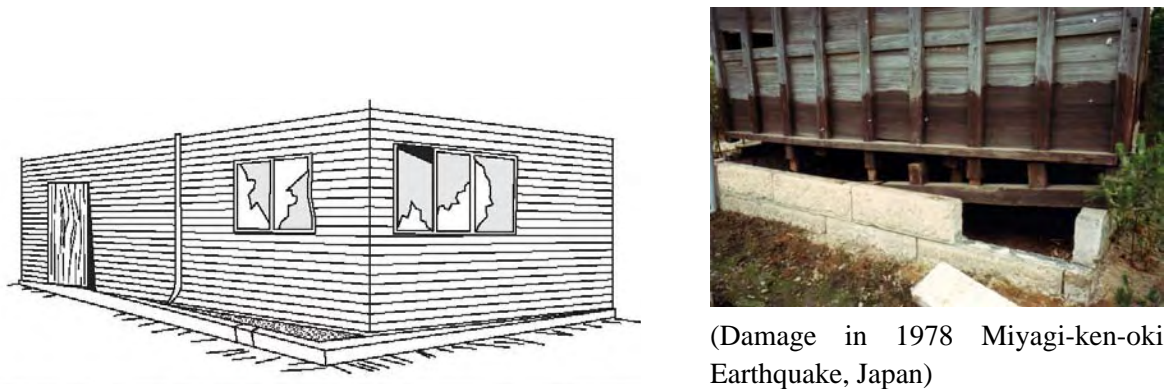


Figure 6.6: Sliding due to insufficient connections between sill and footing

6.3 Typical Characteristics of Wood

Though wood has higher strength per unit weight than most other construction materials, it has the following peculiarities that are not seen in other materials.

- 1) It is a non-homogeneous and anisotropic material showing different characteristics not only in different directions but also in tension and compression.

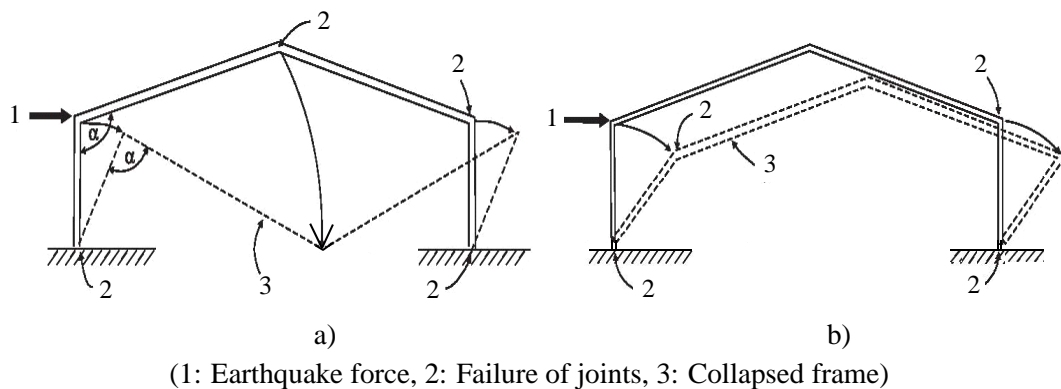


Figure 6.7: Failure modes of gable frame

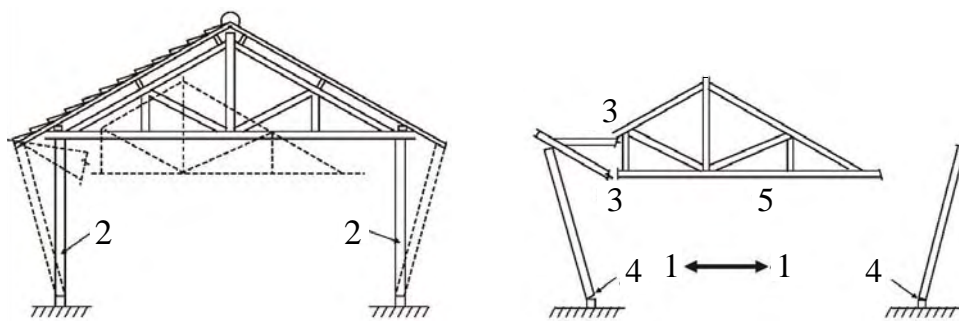


Figure 6.8: Failure due to rupture of bottom chord of roof truss

- 2) Shrinkage of wood on drying is relatively large. Joints loosen easily due to contraction in the direction perpendicular to fibres. Therefore dry wood shall be used with the moisture content less than 20 %.
- 3) The elastic modulus is small. Consequently, members are apt to show large deformations.
- 4) A notable creep phenomenon occurs under permanent vertical loads. This is important especially in snowy areas.
- 5) Large deformation occurs due to compressive force perpendicular to fibers. This influences the amount of deformation of horizontal members and chord members of built-up members.
- 6) The defects and notches of wood influence greatly its strength and stiffness. Consequently it is necessary to select and to arrange structural members considering their structural properties.
- 7) Wood can decay from repeated changes of moisture. Therefore seasoned wood should be used in construction.
- 8) Preservative treatment is necessary to avoid premature rotting and insect attack.
- 9) Wood is a combustible material. Precautions must be taken to minimize the danger of fire.

- 10) Lengths more than 3.5 m long and large size timbers are difficult to obtain. This leads to splicing through connectors or gluing.

In view of its lightness, very easy workability like cutting and nailing and safe transportability, timber makes an excellent material for post-earthquake relief and rehabilitation construction.

6.4 Typical Structural Properties

There are many varieties of timbers in use in various countries. It is therefore not practicable to present their strength properties here that depend on a number of factors as follows:

- 1) Wood species
- 2) Direction of loading relative to the grain of wood
- 3) Defects like knots, checks, cracks, splits, shakes and waness
- 4) Moisture content or seasoning
- 5) Type of wood, such as sapwood, pith and wood from dead trees
- 6) Location of use, viz. inside protected, outside, alternate wetting and drying.

The permissible stresses must be determined taking all these factors into account. Table 6.1 gives typical basic stresses for timbers placed in three groups A, B and C classified on the basis of their stiffness. It is reasonable to increase the normal permissible stress by a factor of 1.33 to 1.5 when earthquake stresses are superimposed.

Table 6.1: Basic permissible stresses for timber group*

Types of stress		Location	Permissible stress (MPa)		
			Group A	Group B	Group C
i)	Bending and tension along grain	Inside	18	12	8
		Outside	15	10	7
		Wet	12	8	6
ii)	Shear in beams	All	1.2	0.9	0.6
	Shear along grain	All	1.7	1.3	0.9
iii)	Compression parallel to grain	Inside	12	7	6
		Outside	11	6	6
		Wet	9	6	5
iv)	Compression perpendicular to grain	Inside	6	2.2	2.2
		Outside	5	1.8	1.7
		Wet	4	1.5	1.4

* Based on IS:883-1970, Indian Standard for use of structural timbers in buildings

Note: Group A, B and C are classified according to Young's Modulus of elasticity as Group A for more than 12,600 MPa, Group B for more than 9,800 to 12,600 MPa and Group C for 5,600 to 9,800 Mpa.

6.5 The Building Plan

The plan of the building should be surrounded and divided by bearing wall lines. The maximum spacing of bearing wall lines is 6 m. The maximum width of openings in the bearing wall lines is 4 m and the opening is at least 0.5 m away from the corner. Adjacent openings should be at least 0.5 m apart (see Fig. 6.9).

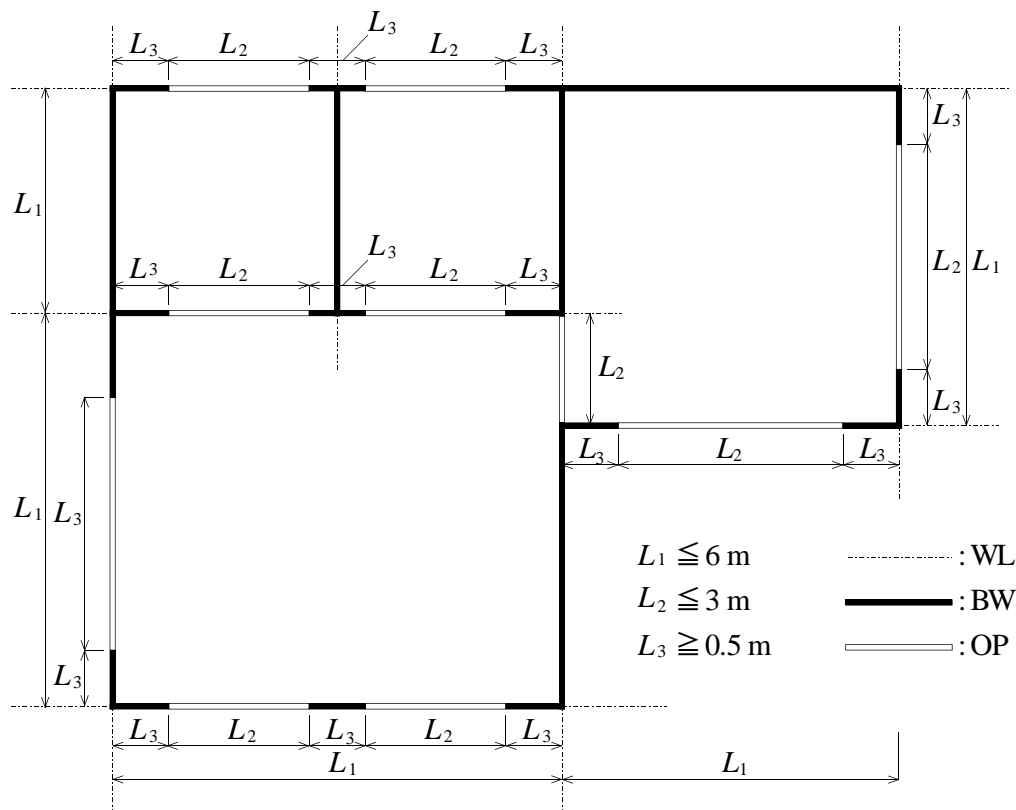


Figure 6.9: Floor plan divided by bearing wall lines
(WL: bearing wall line, BW: bearing wall, OP: opening)

All bearing wall lines of the lower storey should be supported by continuous foundations, through sills, or the columns should rest on pedestals (for details see Sec. 6.9). All bearing wall lines of the upper storey should be supported by the bearing wall lines of the lower storey. The bearing walls may have a stud wall or brick nogged type construction as detailed in Secs. 6.6 and 6.7, respectively. The height of the building will be limited to two storeys plus attic.

6.6 Stud Wall Construction

Stud-wall construction consists of timber studs and corner posts framed into sills, top plates and wall plates. Horizontal diagonal braces are used to stiffen the frame against lateral loads due to earthquake and wind. The wall covering may consist of matting made from bamboo, reeds, and timber boarding or the like. Typical details of stud walls are shown in Fig. 6.10.

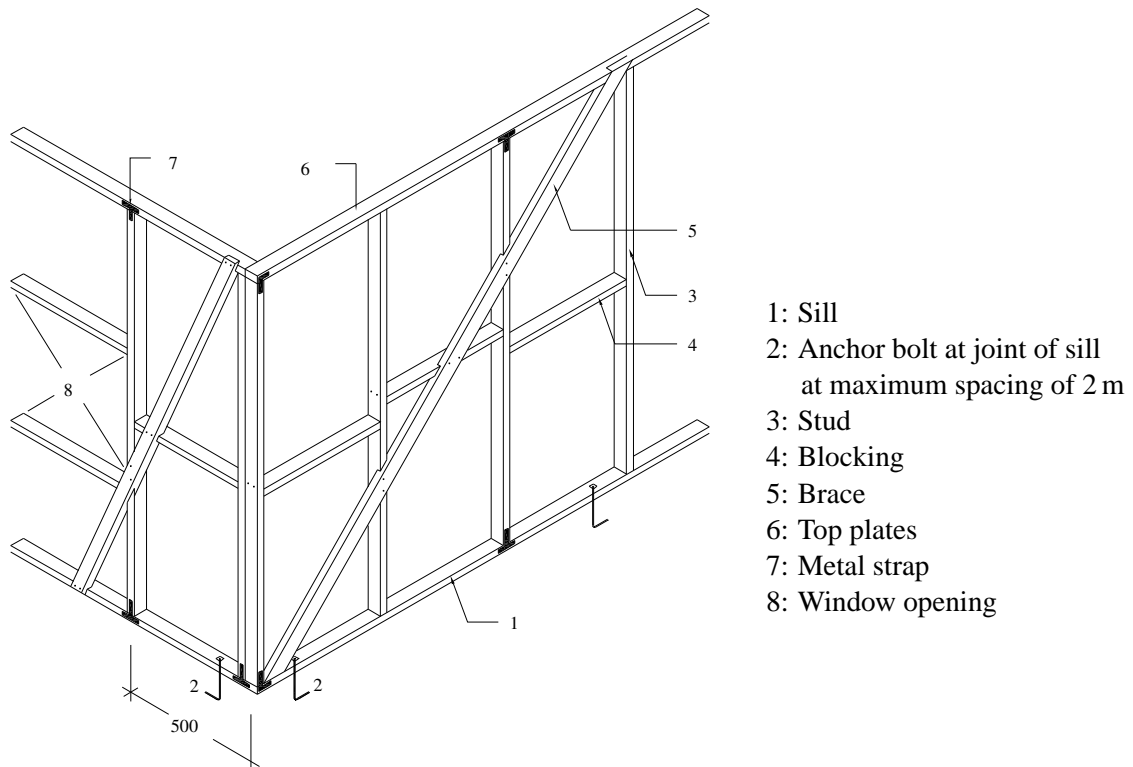


Figure 6.10: Stud-wall construction

If the sheathing boards or sheet linings are properly nailed to the timber frame, the diagonal bracing may be omitted. The diagonal bracing is usually framed into the verticals. Other details are given below:

a. Sill

The dimension of the sill is 40×90 mm, 90×90 mm or larger. The sill is connected to the foundation by anchor bolts whose minimum diameter is 12 mm and length 350 mm. The anchor bolts are installed at both sides of joints of sills and at the maximum spacing is 2 m.

b. Studs

The minimum nominal dimension of studs is 40×90 mm. The maximum spacings of these studs are shown in Table 6.2. If 90×90 mm studs are used the spacing may be doubled, but the interior or exterior linings might need to be thicker. Storey height should not be more than 2.7 m.

c. Top plates

The tops of studs are connected to top plates whose dimension is not less than the dimension of the stud.

d. Bearing walls

Wall framing consisting of sills, studs and top plates should have diagonal braces, or sheathing boards so that the framing acts as a shear or bracing wall. In case no sheathing boards are attached, all studs should be connected to the adjacent studs by horizontal blockings at least every 1.5 m in height. The maximum spacing of 40 × 90 mm stud is shown in Table 6.2.

The minimum size of braces is shown in Table 6.3. The brace is fastened at both ends and at middle portion by more than two nails whose minimum length is 50 mm into each of the framing members. Sheathing boards are connected to the framing members by nails whose minimum length is 50 mm with a maximum spacing of 150 mm at the ends of the board and 300 mm at other parts.

Table 6.2: Maximum spacing (m) of 40 × 90 mm stud

Group of timber*	Single storey or upper floor of double storey building		Lower floor of double storey building	
	Exterior wall	Interior wall	Exterior wall	Interior wall
A	1.0		0.5	
B, C	1.0	0.8	0.5	

* Group of timber defined in Table 6.1

Table 6.3: Minimum size (mm×mm) of diagonal braces

Category*	Group of timber**	Single storey or upper floor of double storey building		Lower floor of double storey building	
		Exterior wall	Interior wall	Exterior wall	Interior wall
I, II	A	20 × 60		20 × 90	
	B, C	20 × 60		20 × 90	
III, IV	A, B, C	20 × 60		20 × 60	

* Categories of construction are defined in Table 3.3

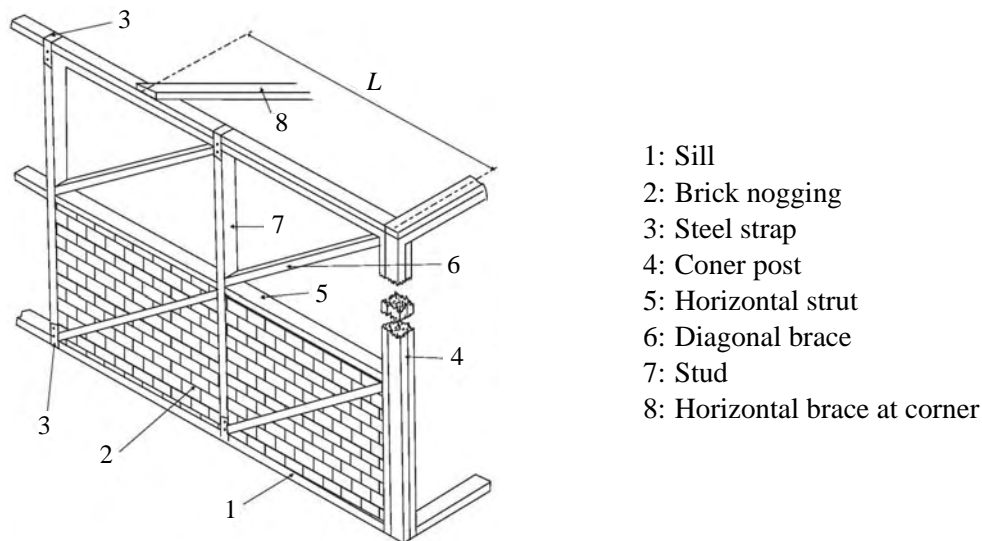
** Group of timber defined in Table 6.1

6.7 Brick Nogged Timber Frame

Brick nogged timber frames consist of studs, columns, sills, wall plates, horizontal nogging members framed into each other. Diagonal braces may also be framed with the studs or nailed or bolted on the faces. The space between framing members is filled with tight fitting brick or dressed stone masonry in stretcher bond.

Typical details of brick nogged timber frame construction are shown in Fig. 6.11. The studs in brick nogged bearing walls should have minimum finished sizes as specified in Table 6.4. The size of diagonal braces should be the same as in Table 6.3. The horizontal framing members

in brick construction should be spaced not more than 1.0 m apart. Their minimum sizes are recommended in Table 6.5.



- 1: Sill
- 2: Brick nogging
- 3: Steel strap
- 4: Coner post
- 5: Horizontal strut
- 6: Diagonal brace
- 7: Stud
- 8: Horizontal brace at corner

Figure 6.11: Brick nogged timber frame

Table 6.4: Minimum size (mm × mm) of studs in brick nogged timber frame construction

Spacing	Group of timber*	Single storey or upper floor of double storey building		Lower floor of double storey building	
		Exterior wall	Interior wall	Exterior wall	Interior wall
1.0 m	A	50 × 100		50 × 100	70 × 100
	B, C	50 × 100		70 × 100	90 × 100
1.5 m	A	50 × 100	70 × 100	70 × 100	80 × 100
	B, C	70 × 100	80 × 100	80 × 100	100 × 100

* Group of timber defined in Table 6.1

Table 6.5: Minimum size of horizontal members

Spacing of studs (m)	Size (mm × mm)
1.5	70 × 100
1.0	50 × 100
0.5	25 × 100

6.8 Joints in Wood Frames

The joints of structural members should be firmly connected by nails or bolts. The use of metal straps is strongly recommended at structurally important joints such as those of studs/columns with sill or wall plates and with horizontal nogging members at the ends of every bearing wall.

6.9 Foundations

The superstructure should be supported by concrete or masonry footings as shown in Fig. 6.12. Openings for ventilation need be provided in continuous foundations (see Fig. 6.12). Some reinforcement as shown is also preferable in very soft soil areas and in areas where liquefaction is expected. On hard or medium soil, isolated footings or boulders can also be used under the wood columns as shown in Fig. 6.13.

Usually some sort of diagonal bracing is used to transfer earthquake inertia forces from ground floor level to foundation level.

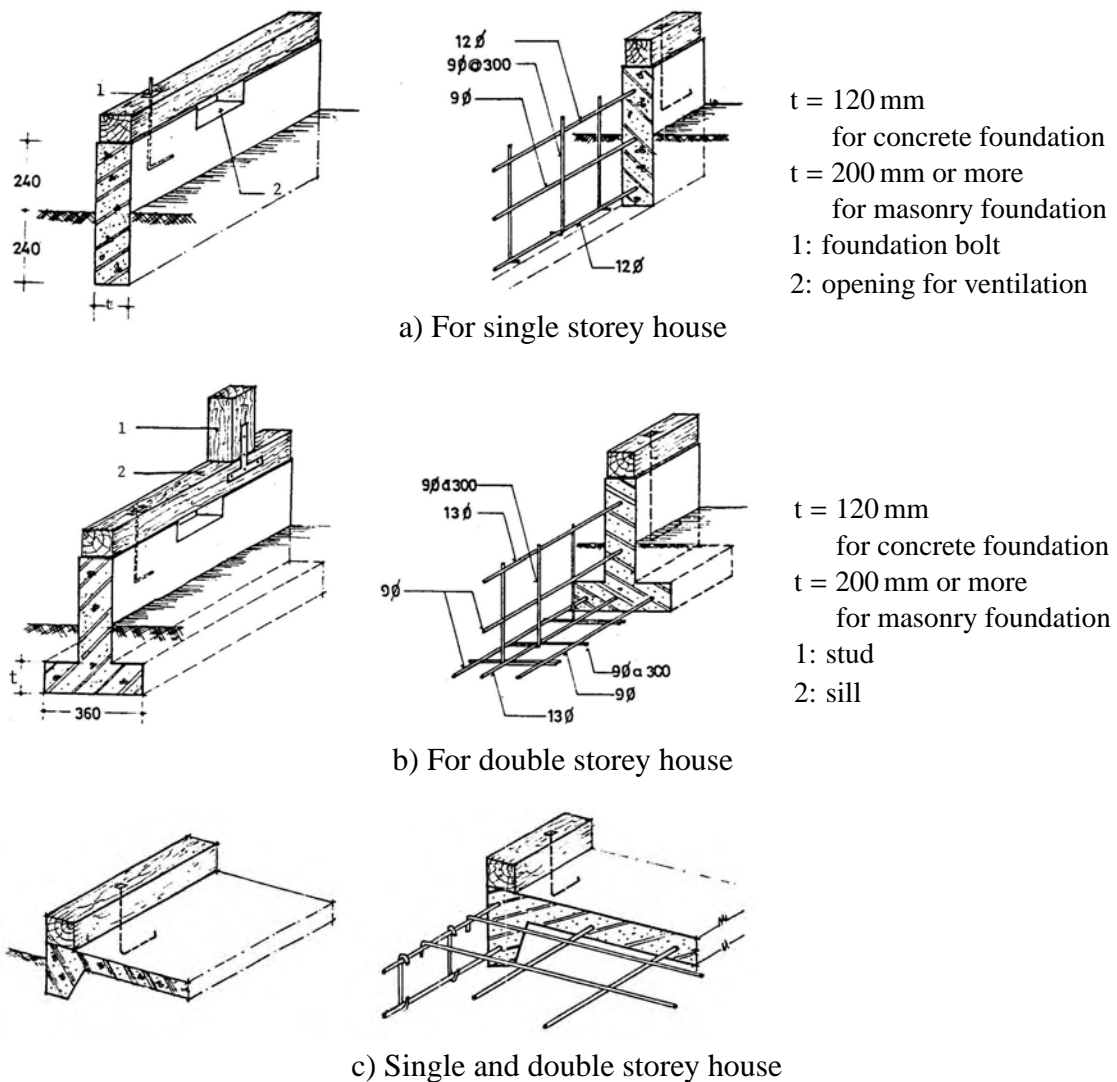


Figure 6.12: Foundations for wooden buildings with or without reinforcement

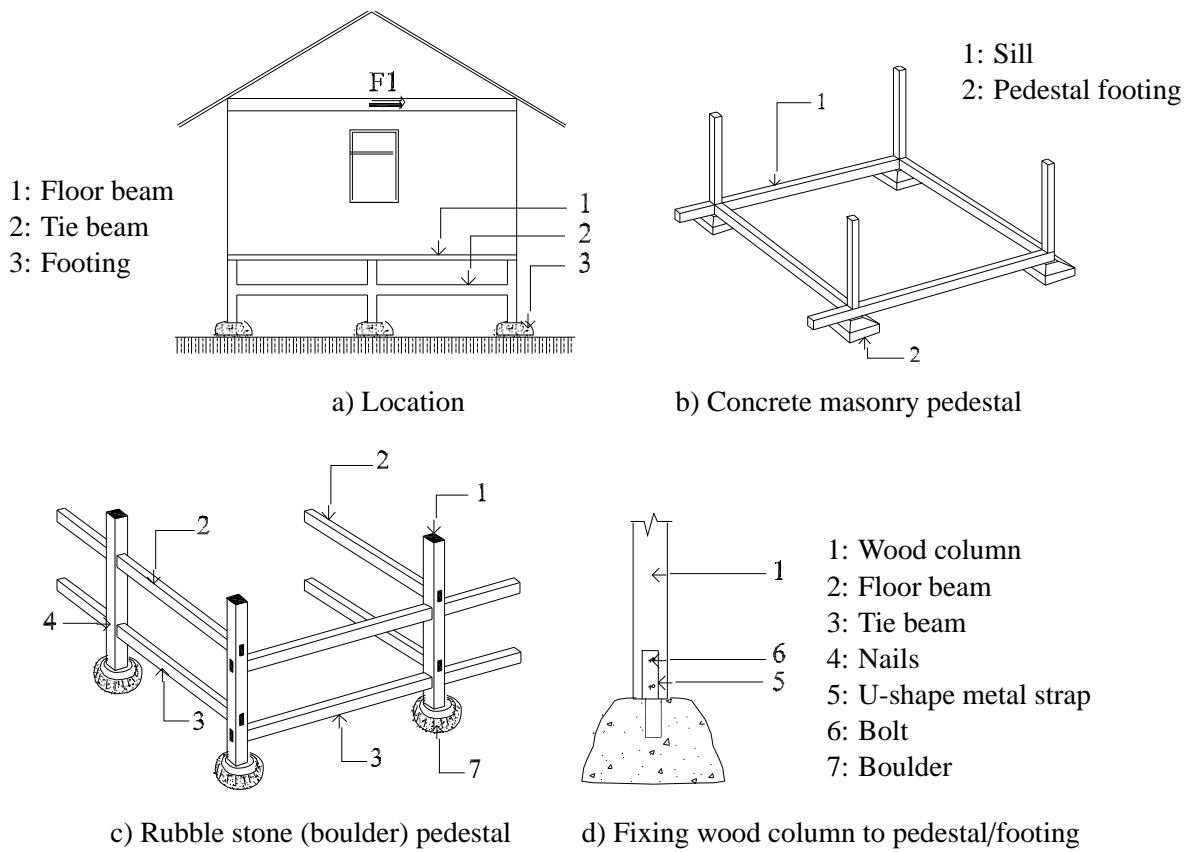


Figure 6.13: Wooden column footings

(Details of a), b) and c) are sometimes adopted in rural areas and superstructures did shift during earthquakes but the houses did not collapse.)

Chapter 7

Earthen Buildings

7.1 Introduction

Earthen construction has been, is and will continue to be a reality. Recent statistics show that the percentage of earthen housing in the next 15 years will be higher than 50 % of the total.

Even though this material has clear advantages of costs, aesthetics, acoustics and heat insulation and low energy consumption, it also has some disadvantages such as being weak under earthquake forces and water action. However, technology developed to date has allowed some reduction of its disadvantages.

Earthen construction is, in general, spontaneous and a great difficulty is experienced in the dissemination of knowledge about its adequate use.

The recommendations presented herein are applicable to earthen construction in general, but they are especially oriented to popular housing, aiming to enhance the quality of spontaneous, informal construction which cause the greatest loss of life and damage during seismic events.

Therefore, this chapter does not include solutions involving the use of stabilizers (cement, lime, asphalt, etc.) to improve the strength or durability. Also, to enhance the dynamic behaviour of the structure economically, minimum use of the expensive materials (concrete, steel, wood, etc.) has been indicated.

7.2 Typical Damage and Collapse of Earthen Buildings

Earthquake experience shows that earthen buildings may crack at MSK Intensity VI, wide cracks and even partial collapse may occur at MSK VII and collapses are widespread under MSK VIII. Damage is always much more severe in two storey buildings than in one storey buildings. Single storey houses with flat roofs constructed in clay have been found to be undamaged in Intensity VIII zone, whereas at the same location two storey houses were completely ruined. Some typical types of damage are sketched in Fig. 7.1. The main causes of failure of earthen buildings in earthquakes are graphically summarised in Fig. 7.2.

7.3 Classification of Walls and Material Properties

In earthen construction, the walls are the basic elements and can be classified as follows:

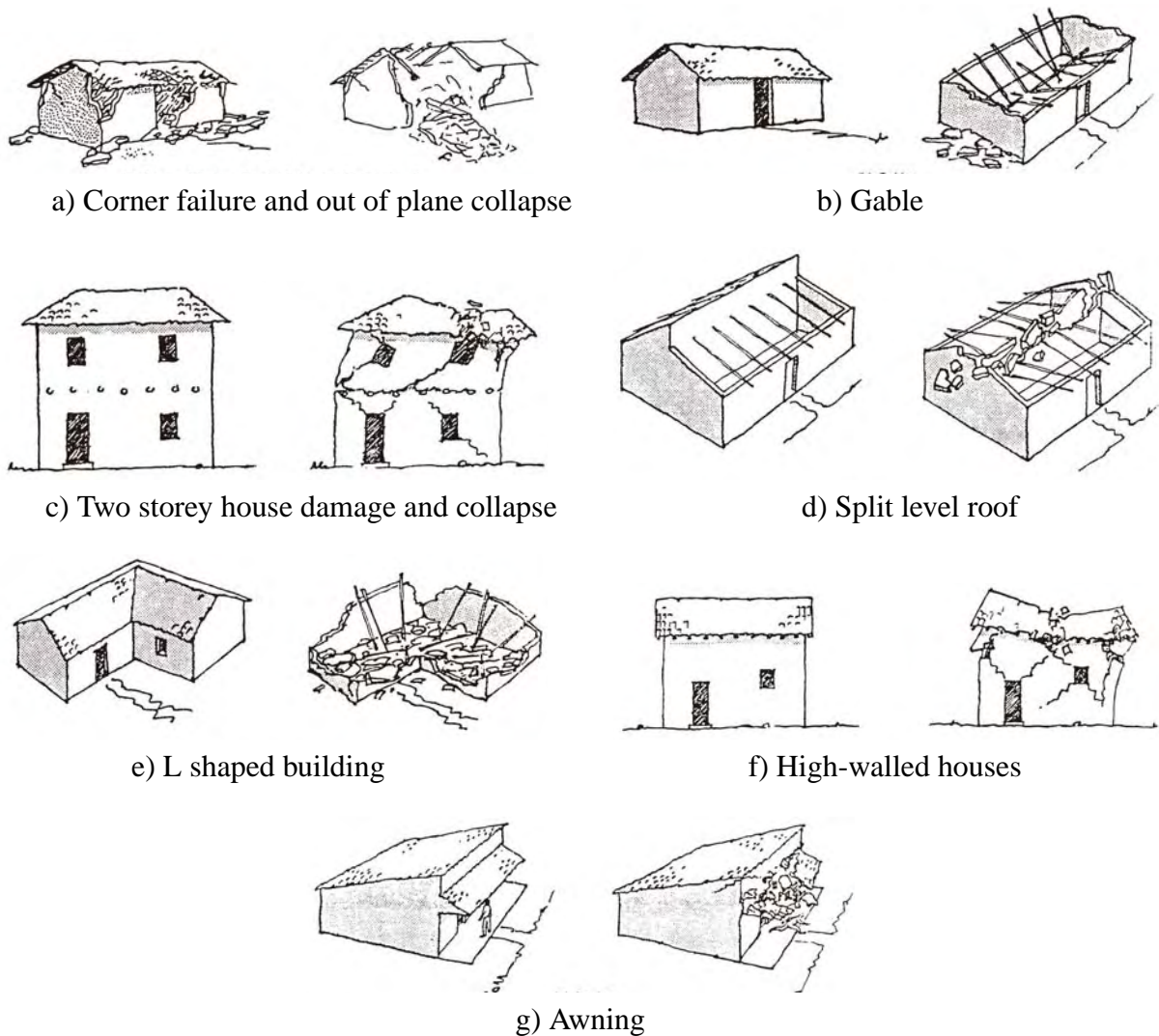


Figure 7.1: Typical damage to earthen buildings

7.3.1 Classification of earthen construction

- a) Hand-formed by layers
 - a.1) Simple forming
 - a.2) Earth balls, thrown and moulded as the wall
- b) Adobe or blocks
 - b.1) Cut from hardened soil
 - b.2) Formed in mould
 - b.3) Moulded and compacted
- c) Tapial or pise (rammed earth)
 - c.1) Compacted by hand blows
 - c.2) Mechanized or vibrating compaction
- d) Wood or cane structure, with wood or cane mesh frameworks plastered with mud

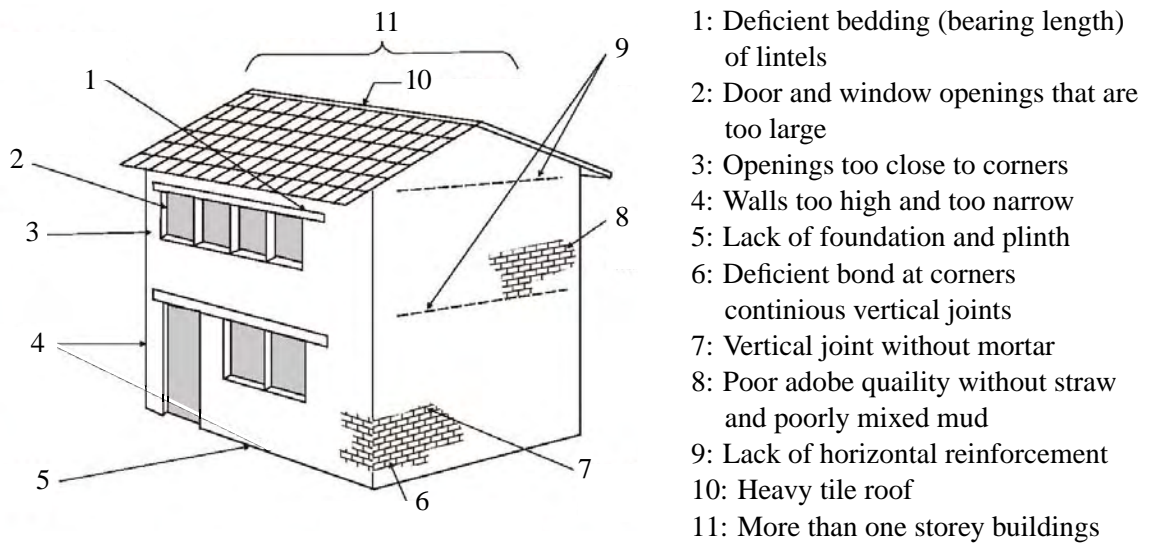


Figure 7.2: Graphic summary of causes of failure

- d.1) Continuous
- d.2) Pre-fabricated panels

Whereas systems a), b) and c) depend for stability on the strength of earthen walls, system d) behaves like a wood frame and its construction will be dealt with separately.

7.3.2 Suitability of soil

The quality of materials, particularly the clay content of the soil vary for the type of construction. In general the following qualitative tests are sufficient for determining the suitability of a soil for earthen construction:

a. Dry strength test

Five or six small balls of soil of approximately 20 mm in diameter are made. Once they are dry (after 48 hours), each ball is crushed between the forefinger and the thumb (see Fig. 7.3). If they are strong enough that none of them breaks, the soil has enough clay to be used in adobe construction, provided that some control over the micro-fissures caused by the drying process is exercised.

If some of the balls break, the soil is not considered to be adequate, because it does not have enough clay and should be discarded.

b. Fissuring control test

At least eight sandwich units are manufactured with mortars made with mixtures in different proportions of soil and coarse sand. It is recommended that the proportion of soil to coarse sand

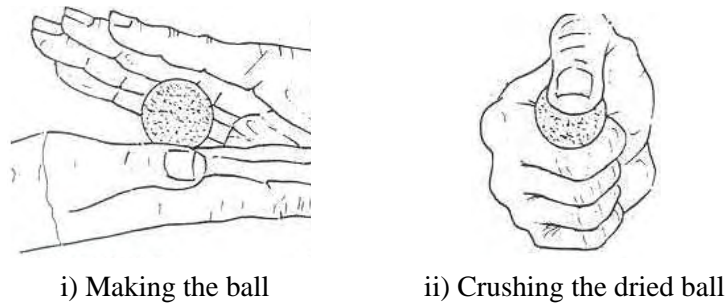


Figure 7.3: Dry ball strength test for soil

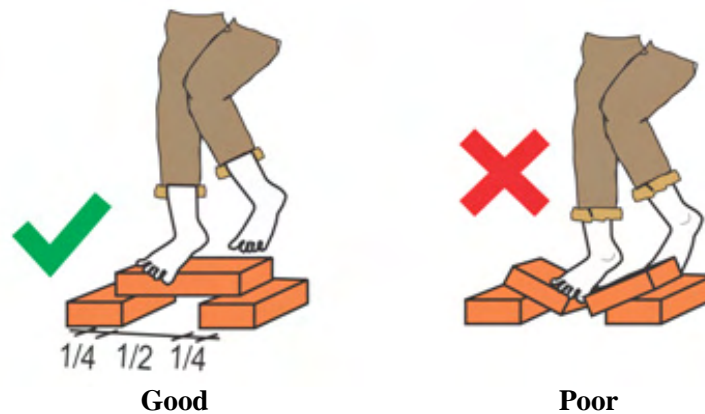


Figure 7.4: Field testing of adobe strength

vary between “1 : 0” and “1 : 3” in volume. The sandwich having the least content of coarse sand which, when opened after 48 hours, does not show visible fissures in the mortar, will indicate the most adequate proportion of soil/sand for adobe constructions, giving the highest strength.

7.3.3 Strength test of adobe

The strength of adobe can be qualitatively ascertained as follows: After 4 weeks of sun drying the adobe should be strong enough to support in bending the weight of a person (see Fig. 7.4). If it breaks, more clay and fibrous material is to be added. The compressive strength may be determined by testing 100 mm cubes of clay after completely drying them. A minimum value of 1.2 N/mm^2 (12 kg/cm^2) is desirable.

7.4 Construction of Walls

In general, the strength of walls is a function of clay content, its activation by humidity (promoted by wetting or compaction procedures), and the control of fissuring.

The positive effect of a traditional practice, namely “sleeping” the mud (storing it at least for one day) before the fabrication of adobe bricks or mortar has been confirmed. It seems that this procedure allows for a better dispersion and thus a more uniform action of the clay particles.

If the soils are clayey, stronger construction can be built, provided an adequate technology is used to control the typical fissures caused by drying from the high moisture content. The most economical and simple form to control such fissures is by adding coarse sand to diminish the clay contraction or by adding dry straw to the mud to control the micro-fissures.

In general, there are no “recommended” mixing ratios for soil to be used in earthen construction. The different percentages of clay, lime, fine sand and coarse sand will be defined by the most abundantly available nearby soil, its clay content (see dry strength test), the type of construction required according to the classification, and the amount of coarse sand needed to control or avoid the visible fissures and attain a monolithic behaviour.

In conclusion:

- 1) Soils with low clay content should not be used (see dry strength test).
- 2) Coarse sand and straw need to be added to avoid fissures and to control them.

7.4.1 Hand-moulded layered construction

This is the most primitive and weakest type of construction because of the low percentage of moisture employed to make the hand-moulding and the poor level of compaction attained. For these reasons, all the clay is not activated, either by moisture, or by compaction.

Even though a small amount of moisture is used (depending on the soil), some horizontal and also vertical fissures normally appear. These should be controlled by adding straw yet attain reasonable workability of the mixture. If this is not possible, coarse sand could be used as an additive. In an experiment achieve the disappearance of visible fissures (try with increasing proportions and wait a few days to check the results). An excess of coarse sand will inevitably reduce the wall strength.

Generally, it is necessary to moisten the area of the lower layer which will be in contact with the mud, in order to avoid sudden drying of the contact zone, which produces the fissures.

7.4.2 Adobe or block construction

In the case of cut as well as moulded blocks, the strongest units are from plastic or clay soils. However, the block strength plays a secondary role in masonry since the joints between blocks become critical. The blocks used should be well dried in order to avoid future shrinkage. Blocks are made in different sizes in various countries. The dimensions of blocks or the way these are placed do not have a serious effect on their strength.

Traditional practices obtain an adequate block without significant fissures, either by mixing sandy and clayey soils or by looking after the block so it dries without restraint, thus eliminating the fissures. The soil to be used should be verified with the dry strength test (see Fig. 7.3) to ensure minimum strength. The mud used to fill the space between blocks (called “mortar”) requires special attention.

To guarantee the bond between blocks and mortar, the micro-fissures of mortar should be avoided. The conditions of the mortar drying are critical because of the fact that the mortar gets in contact with blocks which readily absorb moisture and also they restrict the drying contraction. This produces the above-mentioned micro-fissures, which consequently weaken the masonry.

The joint mud should normally be the same as that used to manufacture the block. If it is found to fissure, some straw (nearly “1 : 1” by volume) should be added to the mortar until an acceptable degree of workability is attained. Some coarse sand could also be added, the adequate proportion being given by the fissuring control test (see Sec. 7.3.2 b).

For clayey soils, the adobe blocks should be moistened for a few minutes before placing them. Also moisten the previous layer of blocks before placing the joint mortar. For sandy soils, only to moisten the preceding layer of blocks.

In order to minimize the damage (see Fig. 7.5), the usual good principles of bonds in masonry should be adopted for construction of adobe walls, i.e.

- 1) All courses should be laid level.
- 2) Vertical joints should be broken between two consecutive courses by overlap of adobe blocks and must be carefully filled with mortar.
- 3) Right angle joints between walls should be made in such manner that the walls are properly joined together and a continuous vertical joint is avoided.



Figure 7.5: Damaged adobe (2007 Pisco Earthquake, Peru)

7.4.3 Tapial or pise construction

Tapial or pise is rammed earth construction in which moistened soil is poured into wooden forms and compacted to achieve the desired density (see Fig. 7.6).

Whilst adobe construction acquires its strength by activation of the clay through moisture contained in the soil, tapial construction relies upon compaction, using small percentages of moisture in the soil.

High strength is obtained by moisture and compaction when clay is present. There are however, practical limitations on restricting the moisture, such as the feasibility to pound and compact the soil, the excessive deformation occurring when the forms are removed and the fissuring problem.

The use of low moisture content (such as the optimum or lower in the Proctor test) and the control of the amount of clay by adding coarse sand are required to control shrinkage fissures.

If the amount of coarse sand is excessive, the strength diminishes dangerously. It is recommended to make wall tests with increasing percentages of sand, until fissuring is reasonably under control.

The compaction or number of blows applied to the wall is a function of the weight and shape of the tool used for this purpose. Higher strengths are obtained under higher compaction, but there is a point at which this is not true any more. Normal compaction is recommended and this will be the one under which no mud remains stuck to the form when the form is removed.

Fifty strokes per 0.1 m^2 of wall area, applied with a mallet of about 8 to 10 kg mass is the recommended practice. The required height for the blocks varies between 0.5 and 0.8 m, but it is very important that the compacted layers in the blocks do not exceed 0.1 m each.

The best way to ensure a monolithic structure of tapial walls is to pour a sufficient quantity of water at the sub joints at every 0.1 m. Likewise, between the tapial layers, every 0.5 to 0.8 m, it is necessary to pour plenty of water on the layer before compacting further material. The placing of straw between the tapial layers is not necessary.

The use of excessive amounts of straw in the mud mixture, more than “1 : 1/4” in volume is self defeating, because it causes a strength reduction.



a) Tapial house under construction



b) Damaged Tapial house

Figure 7.6: Tapial under construction and damaged tapial (1990 Rioja Earthquake, Peru)

7.4.4 Earthen construction with wood or cane structure

Earthen construction using a structural framework of wood or cane is shown in Fig. 7.7. It consist of vertical posts and horizontal blocking members of wood or cane or bamboo. The panels are filled with cane or bamboo, or some kind of reed matting and are then plastered over both sides with mud. The construction can be done in the rudimentary way, building element by element or by using prefabricated panels.

The behaviour of this type of construction can be very good, as long as the following fundamental rules are observed:

- Good connections between the wood or cane elements to ensure an integral behaviour of the structure. The connections are normally fixed with nails. Their number and dimensions should be enough without splitting the elements. The connections can be also tied with wires, ropes, leather straps, etc.

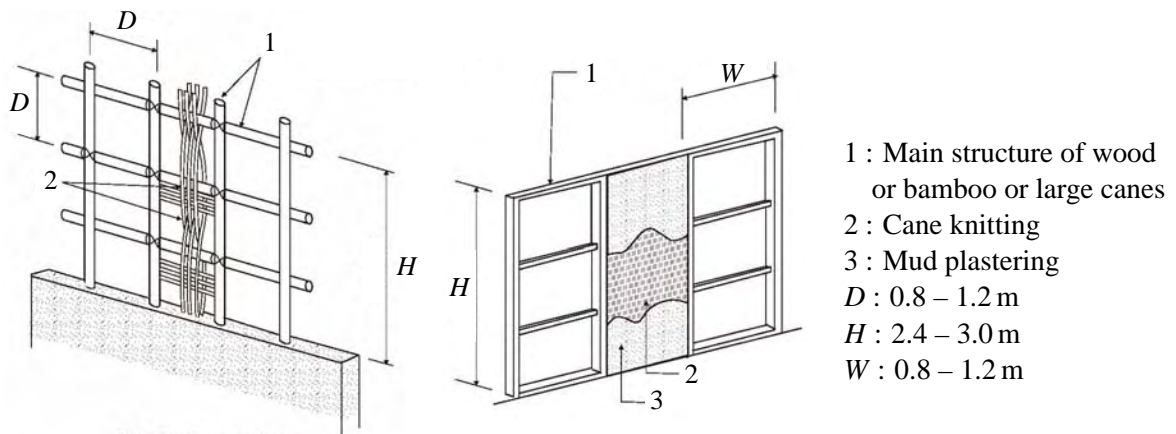


Figure 7.7: Earthen construction with wood or cane structures

- Preserve the wood or cane elements by charring the surface or painting by coal tar, especially where embedded in the foundation, which should preferably be of concrete, stone or bricks laid with cement, lime or gypsum mortar.
- The panel filling material should consist of wood or cane mesh, over which a layer of mud and straw (“1 : 1” in volume) is placed on each face in the form of plaster. Very often, the meshes knit around the structure.
- In houses built as a continuous system as well as in those made with pre-fabricated panels, an upper ring beam should be placed. Its purposes are to:
 - 1) Ensure the integral behaviour of all walls, and
 - 2) Distribute the roofing load evenly.

Only after fixing this upper ring beam and the roof (after completing the nailing), the mud filling must be placed. This will avoid fissuring caused by the nailing operation.

In the case of pre-fabricated panels, the frames could have very small and economical sections 25×50 or 25×75 mm. The connection between panels is made through nails, but the wood or cane knit mesh over which the mud filling is placed, can be fixed without the use of nails.

7.5 General Recommendations for Seismic Areas

7.5.1 Walls

- 1) The height of an adobe building should be restricted to one storey plus attic only in seismic zones A and B and to two storeys in zone C.
- 2) The length of a wall, between two consecutive (cross) walls at right angles to it, should not be greater than 10 times the wall thickness “ t ” nor greater than $64 t^2/h$ where h is the height of the wall.

- 3) When a longer wall is required, the walls should be strengthened by intermediate vertical buttresses (see Fig. 7.8 a).
- 4) The height of wall should not be greater than 8 times its thickness.
- 5) The width of an opening should not be greater than 1.2 m.
- 6) The distance between an outside corner and the opening should be not less than 1.2 m.
- 7) The sum of the widths of openings in a wall should not exceed 1/3 of the total wall length in seismic zone A, 2/5 in zones B and C.
- 8) The bearing length (embedding) of lintels on each side of an opening should not be less than 0.5 m. An adequate configuration is shown in Fig. 7.9 for an adobe and tapial house.
- 9) Hand-formed walls can preferably be made tapering upwards keeping the minimum thickness 0.3 m at top and increasing it with a batter of “1 : 12” at bottom (see Fig. 7.8 b).
- 10) Providing outside pilasters at all corners and junctions of walls will increase the seismic stability of the buildings significantly (see Fig. 7.10).

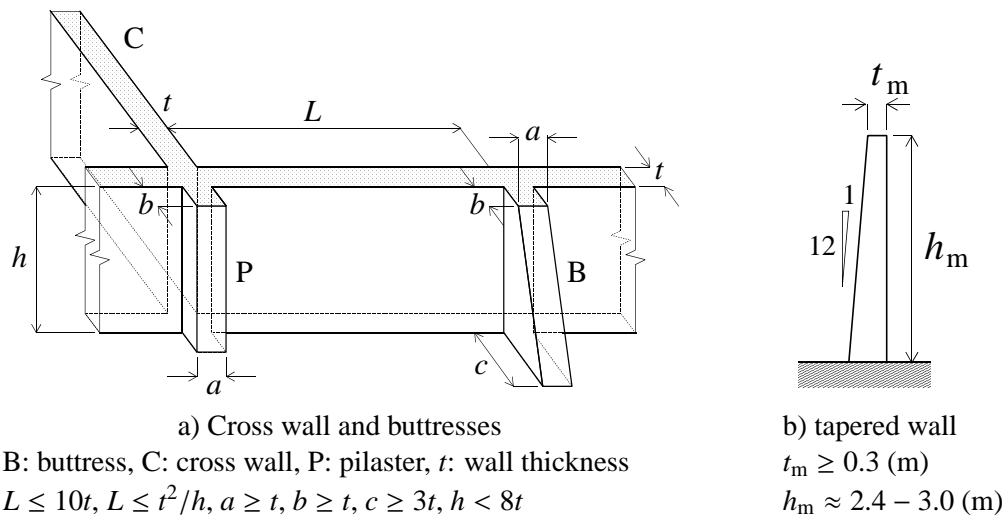
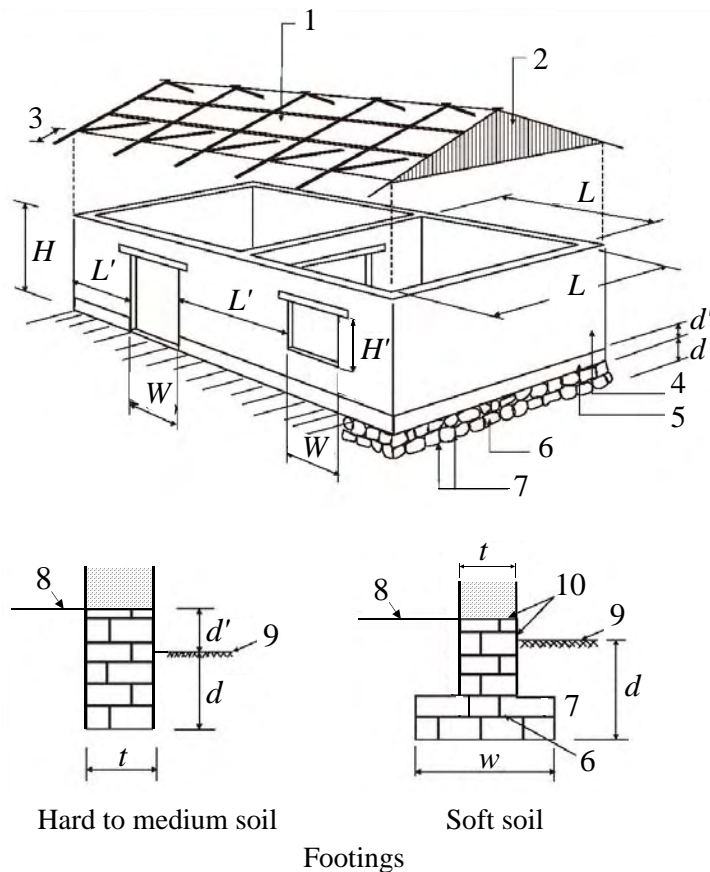


Figure 7.8: Wall dimensions

7.5.2 Foundations

- 1) The brittleness and reduced strength of adobe construction restricts the possible locations of these buildings to areas associated with firm subsoils. Sandy loose soils, poorly compacted clays and fill materials should generally be avoided due to their settlements during seismic vibrations. Also, soils with high water table should be avoided. These recommendations are particularly important for seismic zones A and B.
- 2) The width of strip footings of the walls may be kept as follows:
 - One storey on firm soil – Equal to wall thickness
 - 1.5 or 2 storeys on firm soil – 1.5 times the wall thickness
 - One storey on soft soil – 1.5 times the wall thickness
 - 1.5 or 2 storeys on soft soil – 2 times the wall thickness



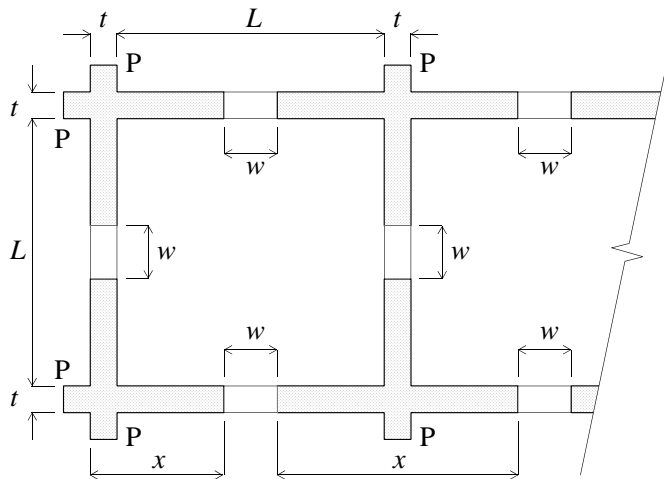
Recommended

- Single storey construction
- Roughly square rooms
- Symmetric distribution of walls
- Small openings
- Use of pilasters

- 1: Light roof
- 2: Light gable roof (matting or boarding)
- 3: Rain protection overhanging about 0.5 m
- 4: Stable plaster
- 5: Plinth height for flood protection
- 6: Stable foundation
- 7: Good mortar preferably non-clay
- 8: Floor level
- 9: Ground level
- 10: Water proof layer

- $H \leq 8t$
- $L \leq 10t$
- $L' \geq 1200 \text{ mm}$
- $W \leq 1200 \text{ mm}$
- $d \geq 400 \text{ mm}$
- $d' \geq 300 \text{ mm}$
- $t = \text{wall thickness, } t > h/8, t > L/10$
- $w \geq 600 \text{ mm}$

Figure 7.9: Adequate configuration



- P: pilaster
- L: distance between two pilasters
- t: wall thickness
- w: opening width for door, window or ventilation
- x: wall length
- $L \leq 10t$
- $w \leq 1.2(m)$
- $x \geq 1.2(m)$

Figure 7.10: Pilasters at corners

The depth of foundation below ground level should be at least 0.4 m.

- 3) The footing should preferably be built using stone, fired brick using cement or lime mortar. Alternatively it may be made in lean cement concrete with plums (boulders) (cement

: sand : gravel : stones as “1 : 4 : 6 : 10”) or without plums as “1 : 5 : 10”. Lime could be used in place of cement in the ratio lime : sand : gravel as “1 : 4 : 8”.

- 4) **Plinth masonry:** The wall above the foundation up to plinth level should preferably be constructed using stone or burnt bricks laid in cement or lime mortar. Clay mud mortar may be used only as a last resort. The height of plinth should be above the flood water line or a minimum of 0.3 m above ground level. It is preferable to use a waterproofing layer in the form of waterproof mud (see Sec. 7.7.3) or thick polyethylene sheet at the plinth level before starting the construction of superstructure wall. If adobe itself is used, the outside face of plinth should be protected against damage by water by suitable facia or plaster. A water drain should be made slightly away from the wall.

7.5.3 Roofing

Roofs have two main parts: structure and cover. Roofing structure must be light, well connected and adequately tied to the walls.

- 1) The roof covering should preferably be of light material, like sheeting of any type.
- 2) If thatch is used for roof covering, it should be made waterproof and fire resistant by applying water mud plaster (see Sec. 7.7.3).
- 3) The roof beams, rafters or trusses should rest on longitudinal wooden elements for distributing the load on the adobe (see Fig. 7.11). If wood is not used two top courses of burnt bricks may be laid instead of adobe for supporting the roof structures.
- 4) The slopes and the roof over-hang depends on local climatic conditions. In zones subjected to rain and snow, wall protection must be ensured by projecting the roof about 0.5 m beyond the walls (see Fig. 7.11).
- 5) The roof beams or rafters should be located to avoid their position above door or window lintels. Otherwise, the lintel should be reinforced by an additional lumber (see Fig. 7.12).

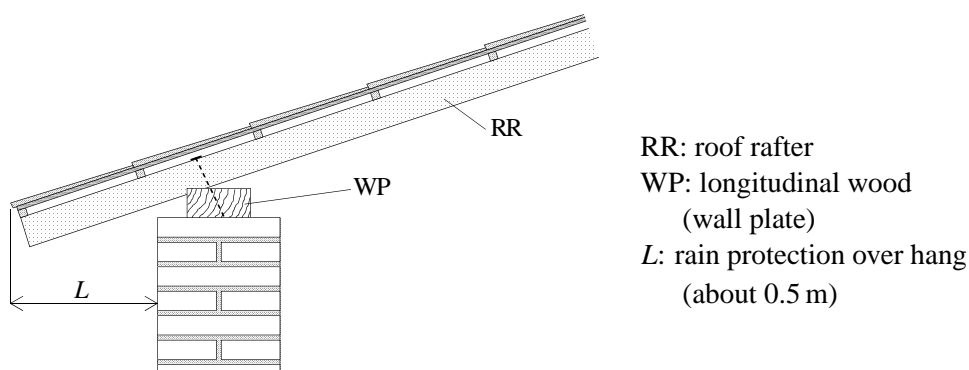


Figure 7.11: Use of longitudinal wood under roof rafters

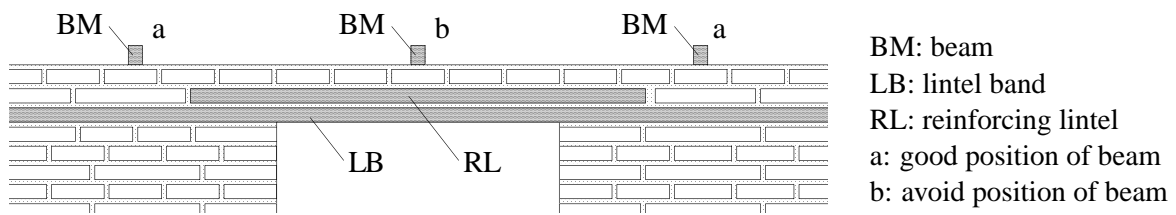


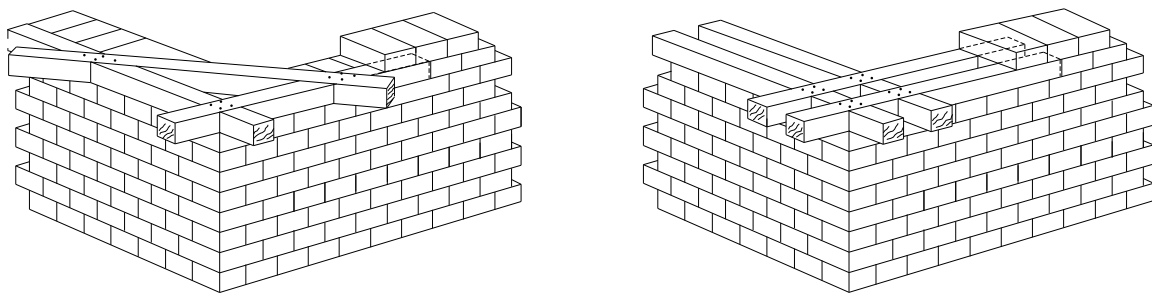
Figure 7.12: Reinforcing lintel under floor beam

7.6 Seismic Strengthening Features

7.6.1 Collar beam or horizontal band

Two horizontal continuous reinforcing and binding beam or bands should be placed, one coinciding with lintels of doors and window openings and the other just below the roof in all walls, in all seismic zones for constructions of types a), b) and c) described in Sec. 7.3.1. Proper connection of ties placed at right angles at the corners and junctions of walls should be insured. Where the height of wall is no more than 2.5 m, use of the lintel band can be avoided but the lintels should be connected to the roof band as shown in Fig. 7.14. The band could consist of:

- 1) Unfinished rough cut lumber in single pieces provided with diagonal members for bracing at corners (see Fig. 7.13 a).
- 2) Unfinished rough cut or sawn (50 × 100 mm in section) lumbers two pieces in parallel with halved joints at corners and junctions of walls placed in parallel (see Fig. 7.13 b).



a) Rough cut lumber in single plate with corner diagonal b) Rough cut lumbers in parallel

Figure 7.13: Collar band in walls at lintel level

7.6.2 Pillasters and buttresses

Where pilasters or buttresses are used, as recommended in Sec. 7.5.1 10) at T-junctions, the collar beam should cover the buttresses as well as shown in Fig. 7.14. Diagonal struts at corners will further stiffen the collar beam.

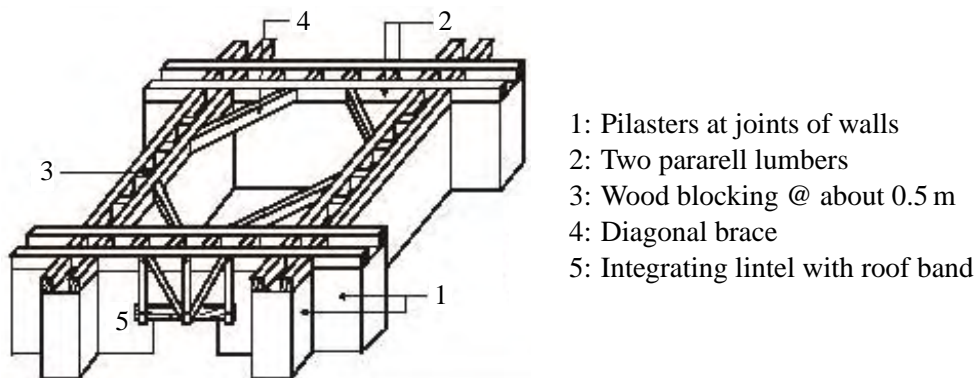
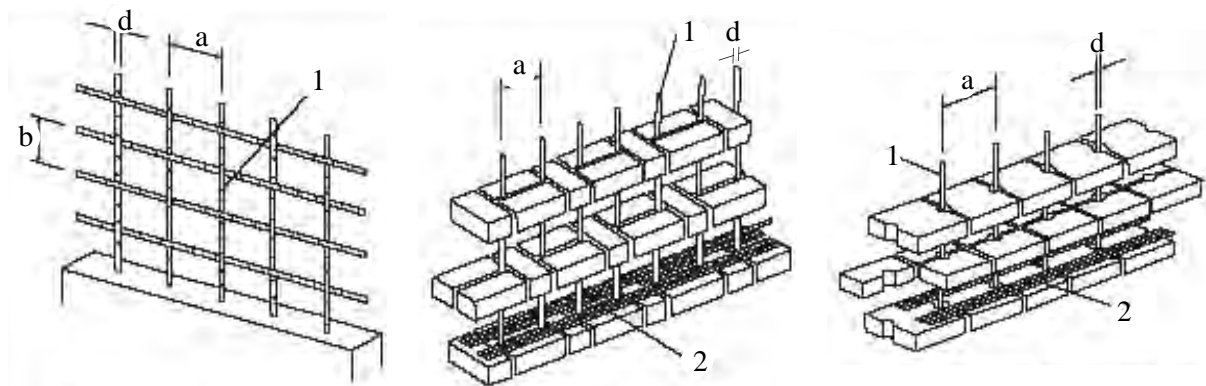


Figure 7.14: Roof band on pilastered walls

7.6.3 Vertical reinforcement in walls

a. In mesh form of bamboo or cane

In seismic zone A, a mesh form of reinforcing is preferable. Here the whole walls are reinforced by a mesh of canes or bamboos as shown in Fig. 7.15, along with the collar beams, which may in this case be made from canes, or bamboos themselves. The vertical canes must be tied to the horizontal canes as well as the collar beam at lintel level and the roof beam at eave level.



a) Pattern of mesh in walls construction a) and c) in Sec. 7.3.1

b) Pattern in adobe walls

c) Pattern in adobe walls (note the preformed notches)

(1: cane or bamboo of diameter about 20 mm, 2: crushed cane or split bamboo every four layers of adobe, a, b: spacing of cane 0.4 m maximum, d: diameter of cane or bamboo about 20 mm)

Figure 7.15: Reinforcement of earthen walls

b. With collar beams or bands

For seismic zones A and B, in addition to the collar beams recommended in Secs. 7.6.1 and 7.6.2, provisions of vertical reinforcement in earthen walls types a), b) and c) in Sec. 7.3.1 is necessary. It can be avoided in zone C.

The most effective vertical reinforcement is in the form of wooden posts, bamboo or cane located at corners and junctions of walls. It should start at foundation level and be continued through and tied to the lintel and roof bands by binding wire, fishing line or rope etc.

7.6.4 Diagonal bracing

In case of earthen construction type d) in Sec. 7.3.1, to achieve adequate seismic resistance in zones A and B provide diagonal bracing members in the planes of walls as well as horizontally at the top level of walls as shown in Fig. 7.16. Use canes or bamboos nailed to the framing members at the ends and intermediate points of intersection.

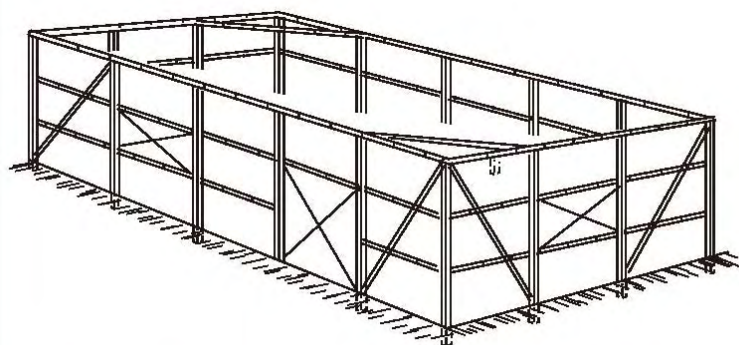


Figure 7.16: Diagonal bracing of wood-structures of earthen construction

7.7 Plastering and Painting

The purpose of plastering and painting is to give protection and durability to walls, in addition to obvious aesthetic reasons.

- 1) Plastering based on natural additives can be formed in two layers. The first, about 12 to 15 mm thick, is a mixture of mud and straw of “1 : 1” in volume, plus a natural additive like cow dung to increase the moisture resistance of the mud. It prevents the occurrence of fissures during the drying process. This natural additive helps to withstand the shrinkage tensions of the restrained drying process. The second and last layer is made with fine mud which when dried, should be rubbed with small, hard, rounded pebbles.
- 2) A technology which consists of plastering walls with a mud stucco stabilized with cactus is described below:
 - a) Prepare the cactus stabilizer by soaking cactus chopped pieces until the soft (inside) part dissolves completely leaving the skin only as residue. The obtained product is characterized by gluey consistency, green color and strong smell of decomposed organic matter.
 - b) Remove dust from the wall surface.

- c) Apply the stucco in two layers, a first layer of 12 mm thick and a second very thin layer (approximately 3 mm thick). The first layer contains straw, and coarse sand in amounts that allow an adequate workability. The second layer contains straw in small pieces (approximately 50 mm long) and should not contain coarse sand. The second layer covers the cracks of the first layer and provides a surface adequate to be polished. Both layers are mixed with cactus stabilizer (water is not used).
- d) Rub the stucco surface with a coarse stone (granitic). Thereafter, moisten the surface with the stabilizer and polish it with a smooth stone (basaltic stone).
- e) Paint the finished surface with the cactus stabilizer.
- 3) To obtain a truly waterproof mud plaster, bitumen may be used in the following way where this material is feasible to use: cut-back is prepared by mixing bitumen 80/100 grade, kerosene oil and paraffin wax in the ratio "100 : 20 : 1". For 1.8 kg cut-back, 1.5 kg bitumen is melted with 15 grams of wax and this mixture is poured in a container having 300 millilitre kerosene oil with constant stirring until all ingredients are mixed. This mixture can now be mixed with 0.03 m³ of mud mortar to make it both water repellent as well as fire protect the thatch.

The exterior of walls may then be suitably painted using a water-insoluble paint or a wash with water solutions of lime or cement or gypsum and plant extracts.

7.8 Summary of Desirable Features

The desirable features for earthquake resistance of earthen houses are briefly illustrated in Fig. 7.17.

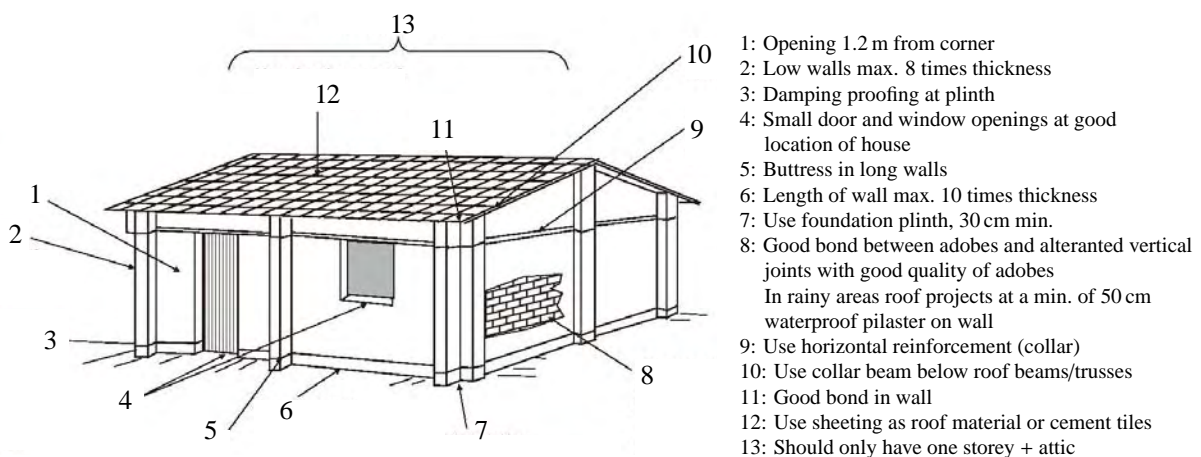


Figure 7.17: Good features of earthquake resistant construction

7.9 Working Stresses

7.9.1 Unit compressive strength

The compressive strength of a masonry unit is an index of its quality and not of the masonry.

It is determined by testing cubes of approximately 100 mm. The compressive strength f_0 is the value exceeded by 80% of the number of specimens tested.

The minimum number of specimens is six (6) and they should be completely dry at the time of testing. The minimum value of f_0 is 1.2 N/mm².

7.9.2 Compressive strength of earthen construction

The compressive strength of earthen construction may be determined by:

- a) The prism test with materials and technology used in the field. The prisms are composed by the number of full adobes necessary to obtain a height/thickness ratio of three.

The minimum number of adobes is four and the joint thickness less than 20 mm (see Fig. 7.18).

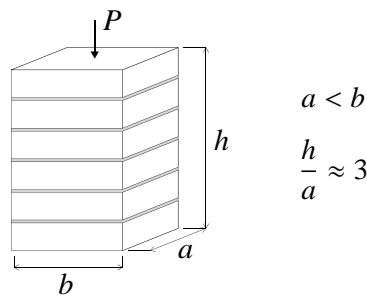


Figure 7.18: Axial compression test

Special care should be observed to keep the specimens vertical. They should be tested 30 day after construction or when the mortar is completely dry. The minimum number of prisms is three.

The permissible compressive stress f_m in wall is:

$$f_m = 0.4Rf'_m \quad (7.1)$$

where:

R = Reduction factor due to wall slenderness.

R can be obtained from Table 4.3 but not greater than 0.75.

f'_m = Ultimate compressive stress of prism. Two of every three prisms should have greater values than the required compressive strength.

Alternatively the following expression can be used:

$$f_m = 0.2f'_m \quad (7.2)$$

- b) If no prism test is conducted, the permissible compressive stress can be taken as

$$f_m = 0.2 \text{ N/mm}^2 \quad (7.3)$$

The permissible crushing stress is: $1.25 f_m$

7.9.3 Shear strength of earthen construction

The shear strength of earthen construction can be determined by:

- a) A diagonal compression test with materials and technology to be used in the field (see Fig. 7.19).

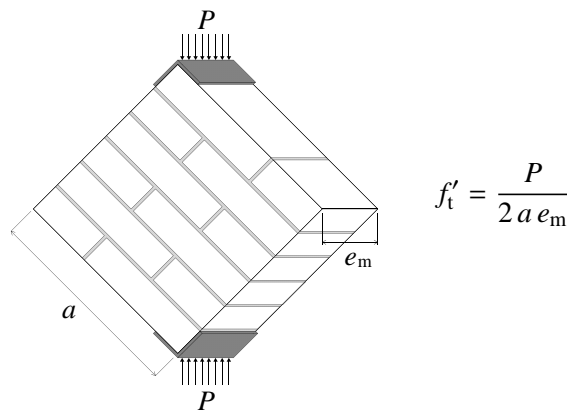


Figure 7.19: Diagonal compression test

A minimum of three specimens should be tested. The permissible strength v_m of wall is obtained from :

$$v_m = 0.4 f'_t \quad (7.4)$$

Where: f'_t = ultimate strength of specimen tested. Two of every three specimens require values that exceed f'_t .

- b) When no tests are conducted, the following value for the shear strength may be used

$$v_m = 0.025 \text{ N/mm}^2 \quad (7.5)$$

7.9.4 Permissible tensile strength of masonry for loads perpendicular to its plane (f_a)

$$f_a = 0.04 \text{ N/mm}^2 \quad (7.6)$$

Chapter 8

Non Engineered Reinforced Concrete Buildings

8.1 Introduction

With the spread of reinforced concrete (RC) construction to semi-urban and rural areas in various countries, buildings are often constructed using reinforced concrete columns and beams, without proper engineering design, only based on the experience of local masons and small contractors. Use of isolated columns in parallel with load bearing walls for supporting long internal beams or those in verandahs and porches is becoming quite common. In most cases, such construction suffers from deficiencies from the seismic view point, since no consideration is given for the effect of seismic lateral loads and the connection details are usually such that no moment carrying capacity can be relied upon. Beams simply rest on top of columns and are mostly held in position through friction.

The other serious deficiency is in concrete quality in respect of mixing, compacting and curing. The aim of this chapter is to provide working guidelines for such low-rise (up to three storeys) small buildings in RC frame construction in which columns are supposed to resist vertical as well as horizontal seismic loads, and the infill walls are assumed to be neither load bearing nor taking part in the lateral resistance of the building. Large halls for gymnasias, assembly halls, etc., having a floor area more than 60 m² or beam spans greater than 7 m, must be designed by an engineer.

Unless designed by engineers, RC frames can not be expected to perform well in damaging earthquakes. RC shear walls are a far more reliable system for resisting earthquake forces.

8.2 Typical Damage and Collapse of RC Buildings

The following types of damage are quite common in these types of reinforced concrete (RC) buildings:

(a) Sliding of roofs off supports

Where beams simply rest on walls or columns they are bound to slide when the earthquake intensity exceeds their frictional resistance. They move off the support and fall, particularly if

the bearing length is small.

(b) Falling of infill walls

Infill panel walls between reinforced concrete columns overturn, if not connected to the frames.

(c) Crushing of column ends and virtual hinging

During severe shaking, column ends are subjected to heavy eccentric compressive stresses under which the concrete gets crushed and spalls off from the outer surfaces. In repeated cycles the damage progresses inwards, thus the effective sections get very much reduced. Eventually both column ends start behaving as pins and the whole framework collapses like a mechanism (see Fig. 8.1). This type of damage may lead to complete collapse of the storey (see Fig. 8.2).

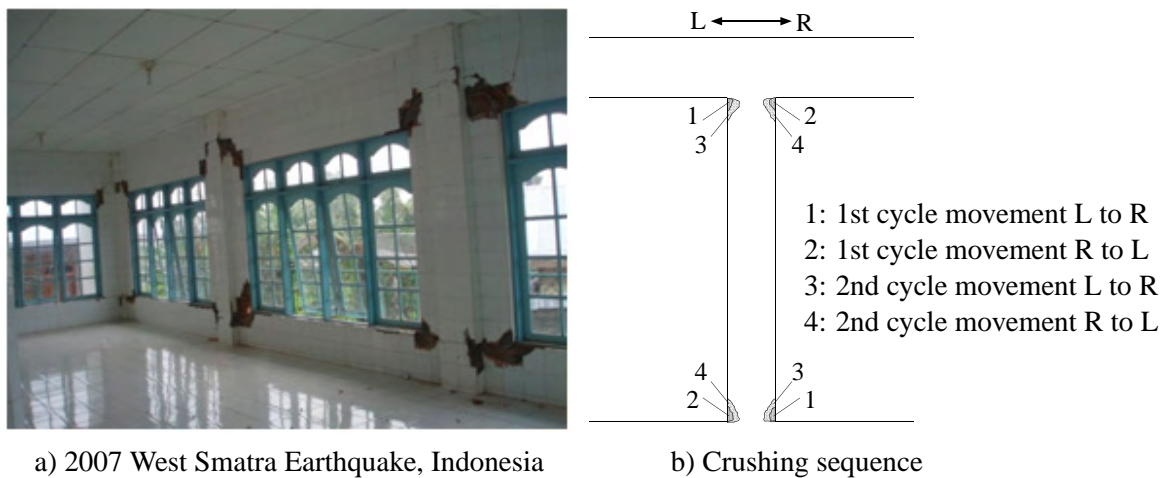


Figure 8.1: Crushing of concrete at ends of column



Figure 8.2: Collapsed RC building
(2005 Northern Pakistan Earthquake)

(d) Short column effect

When infill walls with wide openings are placed between columns, the portions of the columns, that deform under lateral seismic loads become very short as compared to their normal height. Such short columns are much stiffer than other columns and attract much larger shear forces. Severe diagonal tensions occur which may lead to failure of the column (see Figs. 8.3 and 8.4).

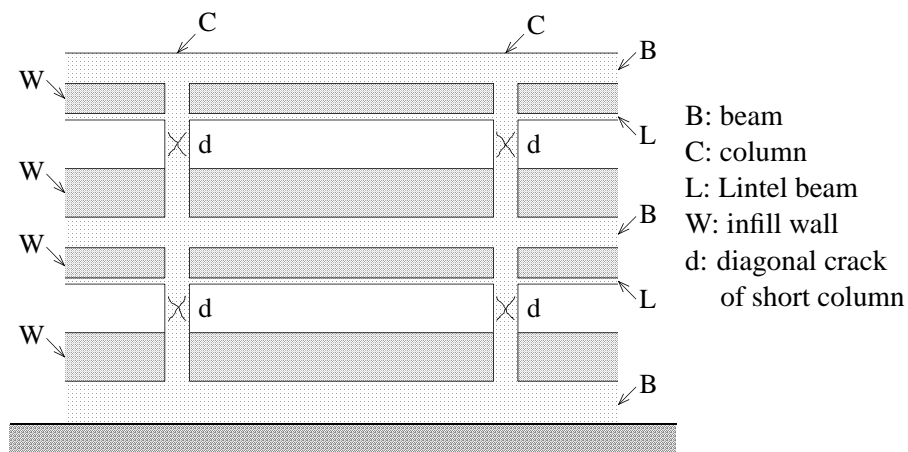


Figure 8.3: Shear failure of short columns



Figure 8.4: Diagonal shear cracks of a short column (1995 Kobe Earthquake, Japan)

(e) Diagonal cracking in columns

Columns are subjected to diagonal cracking due to seismic shears caused under severe ground shaking. If twisting of the building also occurs, the cracks may take a spiral form reducing the load capacity of the columns severely.

(f) Diagonal cracking of column-beam joint

Often diagonal cracking occurs through the junction of the column with the beam. This seriously impairs the strength of the frame.

(g) Pulling out of reinforcing bars

Where the anchor length of column bars or overlaps between the longitudinal bars are not adequate for developing full tensile strength of the bar, they are pulled out due to tensions caused in the column under severe reversal of stresses.

(h) Collapse of gable frames

Reinforced concrete gable or pitched portal frames, often used for school workshops, gymnasias and assembly halls, or cinema halls, have a tendency of spreading out with no secondary resistance available once a joint fails. These are often found to fail and collapse (as was shown in Fig. 6.7 for wooden gable frames), unless very carefully designed and detailed.

(i) Foundation sinking and tilting

Sinking or tilting of foundations of columns due to seismic shaking occurs in loose soft soils and can lead to severe cracking of the superstructure and even collapse.

8.3 Care in Concrete Construction

In reinforced concrete work, the most important requirement is good quality of concrete. This is not usually achieved in non-engineered construction. Here simple guidelines are given for making concrete of adequate strength.

(a) Measuring materials

In non-engineered reinforced concrete construction the proportions of the concrete mix are usually kept “1 : 2 : 3” or “1 : 2 : 4” by volume of “cement : sand : aggregate”. The aggregate may be in the form of river shingle, or crushed stone, of maximum 20 mm size. A 50 kg cement sack has a nominal volume of 0.0317 m³. It is best to make the concrete mixture using whole bags of cement. For measuring sand and aggregate, a wooden box with handles having a volume equal to one sack of cement will be most accurate as well as convenient to use. The measurements of such a box are shown in Fig. 8.5. Such a box can also be made of steel sheets. A simple method to measure the proportion of cement, sand and gravel using a pail is shown in Fig. 8.6.

(b) Mixing materials

Where mixing is done manually without using a power driven mixer, it should be done on an impervious platform, say, using iron sheets or concrete floor. For making a mix of 1 : 2 : 4, four boxes of aggregate should first be measured and flattened on the platform, then two boxes of sand should be spread on the aggregate and finally one full sack of cement opened on top. The

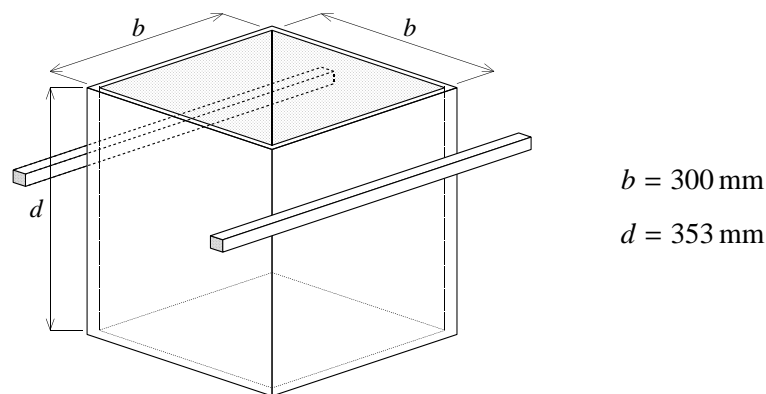


Figure 8.5: Measuring box for 50 kg cement sack

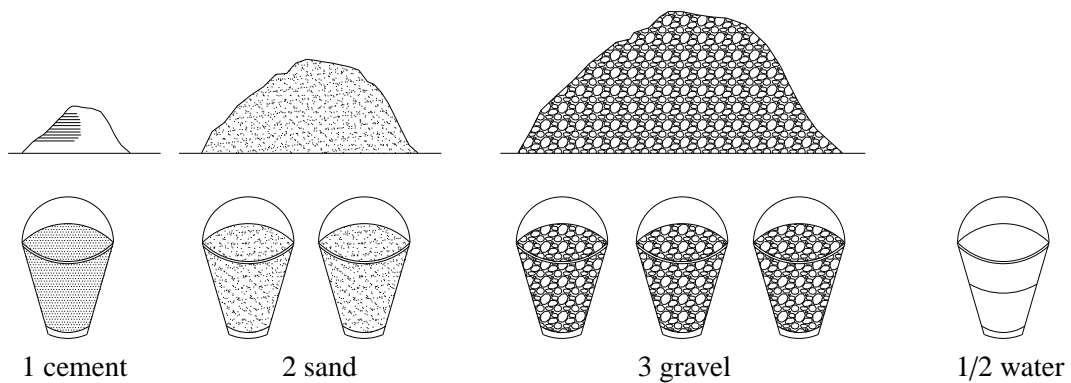


Figure 8.6: Simple measurement of concrete mix “1 cement : 2 sand : 3 gravel”

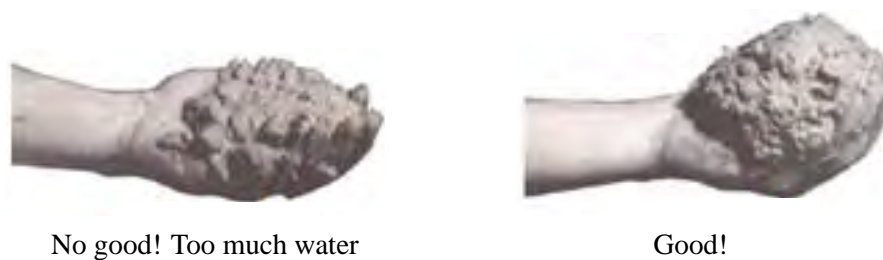


Figure 8.7: Test of concrete mix consistency by making a soft ball of concrete mix in hand

material should first be mixed thoroughly in dry state so as to obtain uniform colour and then water added. The quantity of water should be enough to make a soft ball of the mixed concrete in hand. A slightly wetter mix is better for hand compaction and a drier mix where a vibrator is used for compaction.

(c) Formwork

The quality of not only the concrete surface but also the strength of concrete depends on the surface of the formwork and its imperviousness to the leakage of water and cement through the

joints. Wooden formwork with a well-formed surface and tight joints between planks should be used. Use of water resistant plywood for the skin of the formwork gives a very good surface to the concrete.

(d) Placing of reinforcement

While placing reinforcing bars, the following points must be taken care of:

- Minimum clear cover to the reinforcement: 15 mm to the bars in slabs, 25 mm to bars in beams and columns. In large columns, say 450 mm thick, the cover should be 40 mm. For achieving proper cover, a simple and effective method is to make cement mortar brickets of required size and install them between the bars and formwork (see Fig. 8.8). Tying to bars with thin binding wire will ensure the proper cover.
- Tying longitudinal bars to transverse bars and stirrups and links at each crossing with binding wire.
- Minimum overlap in bars: 45 times the diameter of the bar for plain mild steel and 50 times the diameter for high strength deformed bar. The overlapping portion should preferably be wound with binding wire.
- Shape of links and stirrups: The ends of bars should be hooked by bending through 180° in mild-steel bars and 135° in deformed bars (see Fig. 8.9).

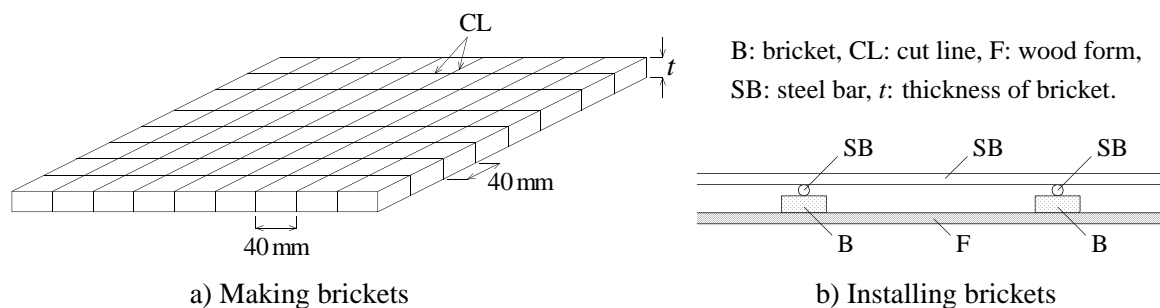


Figure 8.8: Use of cement brickets for cover

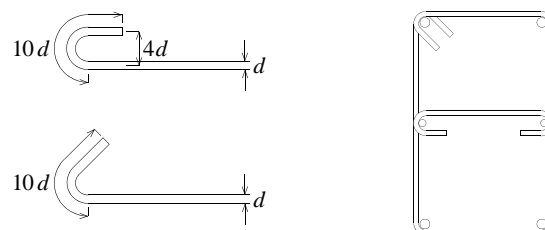


Figure 8.9: Hooks at the end of bars

(e) Casting and compacting concrete

Concrete should normally be cast in one continuous operation so as to avoid undesirable joints. Mixed concrete should not be allowed to stay on the platform for more than 45 minutes and must be placed in the forms and compacted continually. Hand compaction must be done by rodding through the freshly placed concrete. Simply leveling the surface with trowels will leave voids in the mass. Lack of compaction results in a large reduction in concrete strength, hence utmost attention should be given to compaction. Good results from rodding will be obtained by using 16 mm diameter rods about 0.5 m long.

(f) Curing of concrete

Concrete requires water-curing for a minimum of 14 days so as to gain its full strength and to achieve maximum density and impermeability of the outer few millimetres of cover concrete. This is important for durability. Concrete slabs can be kept under ponding water by making earthen barriers all around the edges. Columns should be kept covered with wet empty gunny bags or hessian material. Keeping the side forms intact on beam webs will prevent the evaporation of water from the concrete and help in curing. Covering any concrete surface with polythene sheets after wetting the surface will help retaining the moisture.

(g) Construction joints

Where a joint is to be made, the surface of the concrete shall be thoroughly roughened, cleaned and all laitance removed. The surface shall be thoroughly wetted, and covered with a coat of neat cement slurry immediately before placing the new concrete. Construction joints in floors shall be located near the middle of the spans of slabs, beams or girders, unless a beam intersects a girder at this point, in which case the joints in the girders shall be offset a distance equal to twice the width of the beam.

8.4 Material Properties

8.4.1 Concrete

Concrete must have the desired strength for the required use. The strength is defined on the basis of 28 days age cube or cylinder crushing strength. For use in buildings, the cube strength F_c of between 15 to 20 N/mm² is adequate. The concrete mix is accordingly designed but ordinarily “cement : sand : coarse aggregate” mix proportion is taken as “1 : 2 : 4” or “1 : 1½ : 3” by volume.

The mass density of RC is about 2400 kg/m³ and modulus of elasticity is variously related to the concrete strength. Each country has its own standards for allowable stresses and load factors which should be referred to. A factor of safety of about 3 is used in determining the allowable stress in axial compression relative to the 28-day cube strength. Under seismic condition these allowable values may be increased by 33⅓% and the load factors may be decreased by 25% unless specified otherwise in national standards.

The tensile strength of concrete is only about one-tenth of its compressive strength. Diagonal tension caused by seismic shear forces, if not thoroughly protected by well designed stirrups or ties, can lead to wide cracking and failure.

Concrete is a brittle material and weak against impact shock and vibrations. Ductility is imparted to it by the reinforcing steel only. Its compressive strength as well as strain capacity can be greatly increased by using closely-spaced stirrups or hoops or ties or spiral reinforcement. This is an important method of improving the earthquake resistance of reinforced concrete columns and frames.

8.4.2 Reinforcing steel

The reinforcement may be any of the following:

- a) Mild steel and medium tensile steel bars.
- b) High strength deformed steel bars

All reinforcement should be free from loose rust and coats of paints, oil, mud or any other substance which may destroy or reduce bond with concrete.

In case structural calculation is required, engineers may use the modulus of elasticity 200 kN/mm^2 and the yield strength as the minimum yield stress or 0.2 % specified proof stress unless specified otherwise in national standards.

8.5 Critical Zones for Ductility

The critical zones in reinforced concrete frames, where ductility of sections and confinement of concrete by closely-spaced stirrups or hoops or spiral is required, are shown shaded in Fig. 8.10 and explained below:

- 1) Ends of beams up to a length of twice the depth of the beam where large negative moments and shears develop are likely locations for plastic hinges. Here shear and moment reversal is possible under large seismic forces.
- 2) Ends of columns where maximum moments develop due to lateral forces. The length of such zones is about $1/6$ of the clear height of the column between floors or twice the dimension of the column section in the plane of the frame. Values of maximum column moments closely approaching plastic moment capacity can be expected and these moments are likely to undergo full reversal. High lateral shears can be developed based on moments of opposite sign at the column ends and these shears can undergo full reversal.
- 3) Joint regions between beams and columns undergo very high local shears. Full reversal is likely with diagonal cracking and local deformation causing significant rotation at joint increasing the lateral displacement of frame and weakening the frame.

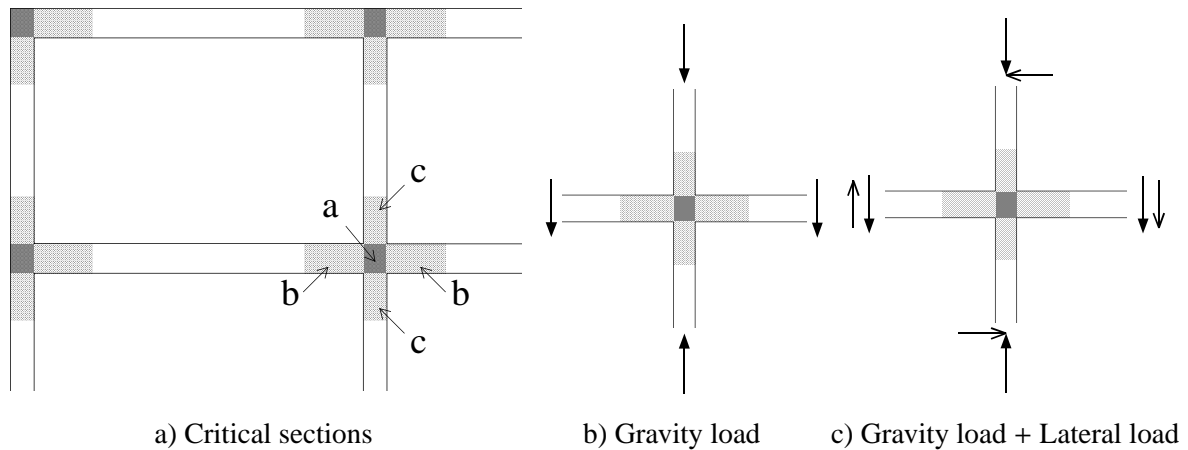


Figure 8.10: Critical sections in a RC frame

(a: joint between beam and column, b: beam end, c: column end)

8.6 Detailing of Beams

(a) Longitudinal steel

Beams should be reinforced on both top and bottom faces throughout. Where reinforcement is required by calculation, the percentage should correspond to ductile behaviour. The recommended limits on steel area are shown in Table 8.1. Minimum steel should consist of two bars (top and bottom) of 12 mm diameter in case of mild steel (MS) and 10 mm diameter when high strength deformed bars (HSD) are used (see Fig. 8.11). All longitudinal steel should preferably be deformed to avoid sliding through joint regions.

(b) Splicing of steel

All longitudinal bars should be anchored or spliced for full strength development. All splices should be contained within at least two stirrups at each end of the splice so as to avoid spalling of cover concrete (see Fig. 8.11).

(c) Transverse steel stirrups

The ultimate shear strength of the beam should be designed to be more than its ultimate flexural strength (see Fig. 8.11). Vertical shear stirrups should be closely spaced at not more than $1/4$ of effective depth (h) in end $2h$ -length of spans of the beams and where the longitudinal bars are spliced. In the remaining length spacing should not exceed $h/2$.

8.7 Detailing of Columns

(a) Column section

Since earthquake forces act in all directions, square section columns are better than rectangular.

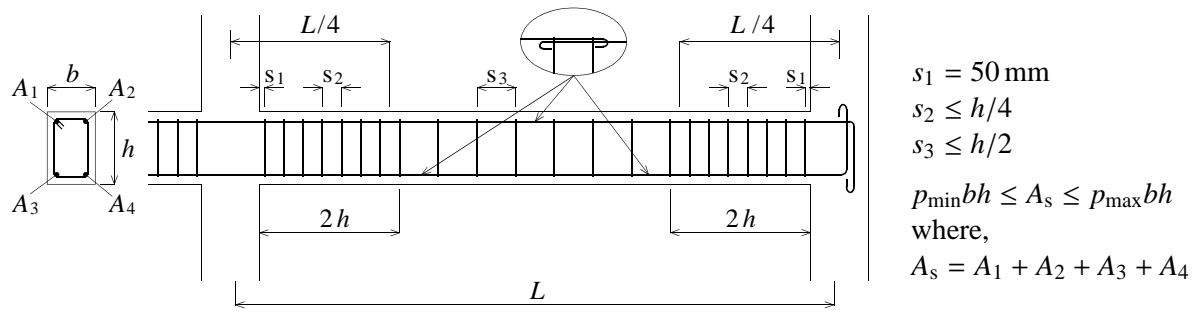
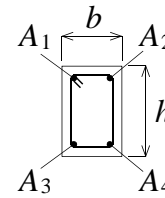


Figure 8.11: Detailing of beam reinforcement

Table 8.1: Recommended limits on steel area ratio in beam

Concrete	Steel	p_{max}	p_{min}
1 : 2 : 4 ($F_c = 15$ MPa)	MS ($F_y = 250$ MPa)	0.011	0.0035
	HSD ($F_y = 415$ MPa)	0.007	0.0048
1 : 1½ : 3 ($F_c = 20$ MPa)	MS ($F_y = 250$ MPa)	0.015	0.0048
	HSD ($F_y = 415$ MPa)	0.009	0.0029



Notes) F_c : 28 days crushing strength of 150 mm cubes,
 F_y : yield strength of reinforcement,
 MS: mild steel, HSD: high-strength deformed bars,
 $p_{min}bh \leq A_s \leq p_{max}A_sbh$, where $A_s = A_1 + A_2 + A_3 + A_4$.

(b) Longitudinal steel

Vertical reinforcement should be distributed on all faces of columns. Use of 8 vertical bars is preferable to four bars of equivalent area. The minimum diameter of bars is 12 mm.

(c) Lateral reinforcement

Concrete confined within spirals is stronger as well as much more ductile as compared with plain concrete or concrete containing widely spaced hoops (lateral ties).

The behaviour of columns can be much improved by using the hoops with adequate anchorage at ends in the form of suitable hooks as shown in Fig. 8.9 and Fig. 8.12 at close spacing.

Over a length of about 450 mm from the ends of columns, a spacing of hoops not more than 100 mm must be adopted for achieving ductility there.

(d) Corner column

The corner columns of buildings are stressed more than any other column due to biaxial bending and must therefore have steel distributed on all faces and closely spaced hoops.

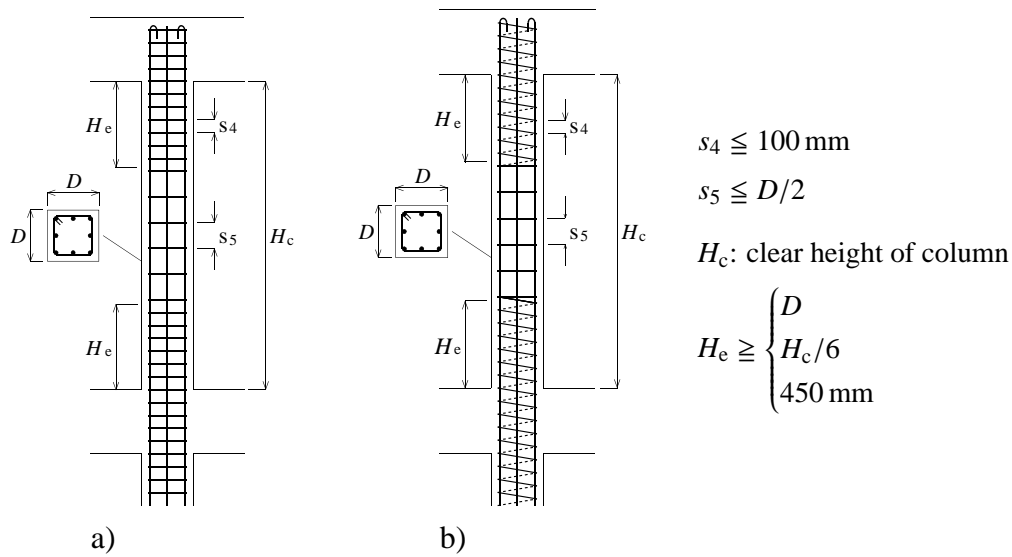


Figure 8.12: Column reinforcement

8.8 Connections

The beam and column bars must be well anchored in the compression zone so as to achieve their full strength.

It is always necessary to place closely spaced hoops in the column throughout the height of the joint to avoid shear failure.

8.9 Illustrative Sketches

Fig. 8.13 to Fig. 8.19 give typical recommended details of connections in earthquake resistant frames.

The notations are as follows.

b : beam width	$L_e \geq 2h$ and $4L - D/2$
D : column depth or width	$s_1 \leq 50 \text{ mm}$
d : bar diameter of reinforcement	$s_2 \leq 24d, h/4, 8d$ and 300 mm
h : beam height	$s_3 \leq h/2$ and 150 mm
H_c : clear height of column	$s_4 \leq D/4, 6d$ and 100 mm
$H_e \geq D, H_c/6$ and 450 mm	$s_5 \leq 6d$ and 150 mm
L : span of beam	$s_6 \leq h/2$ and 300 mm

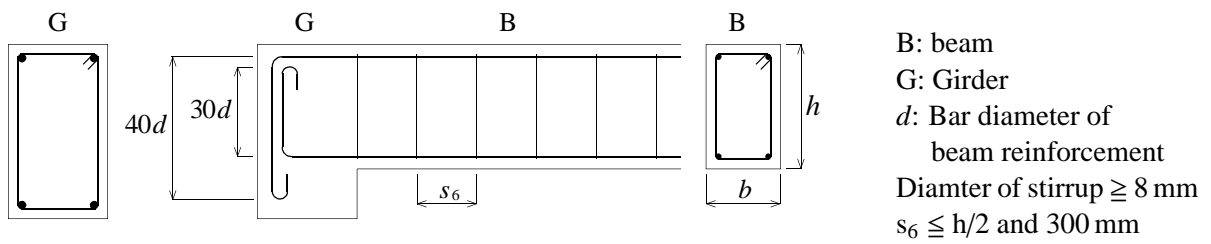


Figure 8.13: Connection between beam and girder

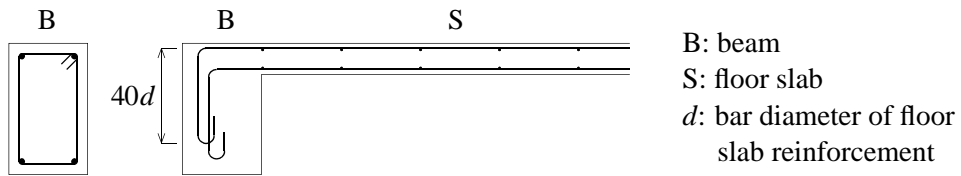


Figure 8.14: Connection between floor and beam

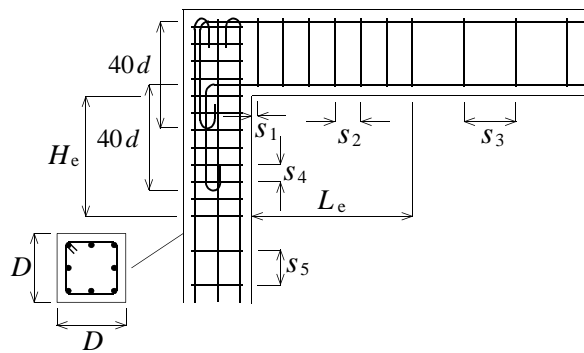


Figure 8.15: Connection between roof beam and exterior column

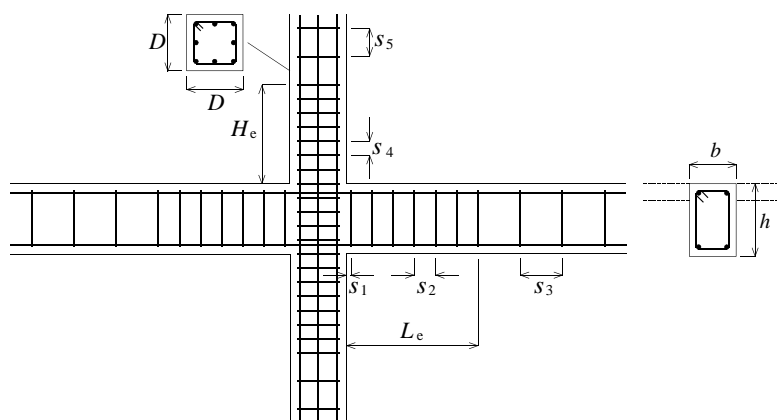


Figure 8.16: Connection between floor beam and interior column

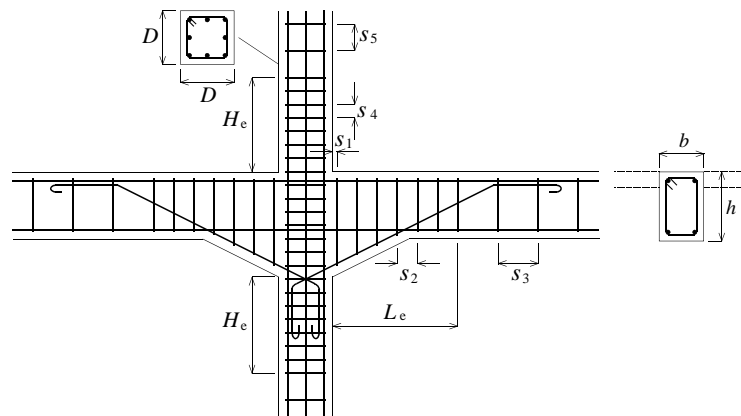


Figure 8.17: Interior joint between haunched beam and column

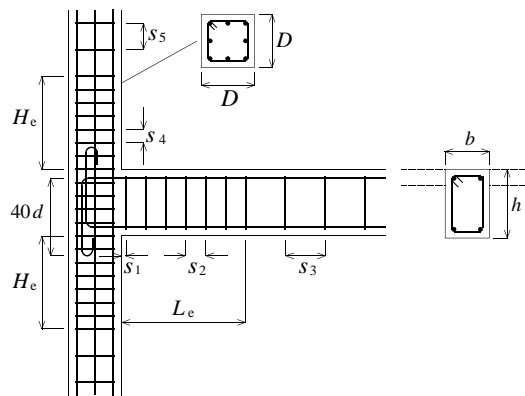


Figure 8.18: Connection between floor beam and exterior column

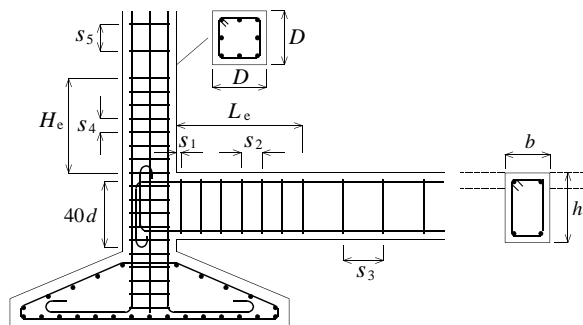


Figure 8.19: Column footing and foundation/plinth beam

Chapter 9

Repair, Restoration and Strengthening of Buildings

9.1 Introduction

The need to improve the ability of an existing building to withstand seismic forces arises usually from the evidence of damage and poor behaviour during recent earthquakes. It can arise also from calculations or by comparisons with similar buildings that have been damaged in other places. While in the first case the owner can be rather easily convinced to take measures to improve the strength of his or her building, in the second case dwellers that have much more stringent day-to-day needs are usually reluctant to invest money in the improvement of seismic safety. The problems of repairs, restoration and seismic strengthening of buildings are briefly stated below:

- 1) Before the occurrence of the probable earthquake, the required strengthening of seismically weak buildings is determined by a survey and analysis of the structures.
- 2) Just after a damaging earthquake, temporary supports and emergency repairs are carried out so that precariously standing buildings will not collapse during aftershocks and the less damaged ones can be quickly brought back into use.
- 3) The real repair and strengthening problems are faced at the stage after the earthquake when things start settling down. At this stage distinction has to be made in the type of action required, i.e. repairs, restoration and strengthening, since the cost, time and skill required of each may be quite different.

The decision as to whether a given building needs to be strengthened and to what degree, must be based on calculations that show if the levels of safety demanded by present codes and recommendations are met. Difficulties in establishing actual strength arise from the considerable uncertainties related with material properties and with the amount of strength deterioration due to age or to damage suffered from previous earthquakes. Thus, decisions are frequently based on conservative assumptions about actual strength.

The method of repair and strengthening naturally depends very largely on the structural scheme and materials used for the construction of the building in the first instance, the technology that is feasible to adopt quickly and on the funds that can be assigned to the task, usually

very limited. Some methods like “splints and bandages”, “wire mesh with gunite”, “epoxy (polymer) injection,” etc., have already been tried and applied in a few countries for repairing as well as strengthening earthquake damaged buildings. These as well as other possible methods are discussed in this chapter.

9.2 Repair, Restoration and Strengthening Concepts

9.2.1 Repairs

The main purpose of repairs is so that all services start working and the functioning of building is resumed. Repair does not pretend to improve the structural strength of the building and can be very deceptive for meeting the strength requirements of the next earthquake. Actions include the following:

- 1) Patching up of defects such as cracks and fall of plaster.
- 2) Repairing doors, windows, replacement of glass panes.
- 3) Checking and repairing electric wiring.
- 4) Checking and repairing gas pipes, water pipes and plumbing services.
- 5) Re-building non-structural walls, chimneys, boundary walls, etc.
- 6) Re-plastering of walls as required.
- 7) Rearranging dislocated roofing tiles.
- 8) Relaying cracked flooring at ground level.
- 9) Redecoration - whitewashing, painting, etc.

The architectural repairs as stated above do not restore the original structural strength of cracked walls or columns and may sometimes be dangerous. The redecorated elements hide all the weaknesses and the building will suffer even more severe damage if shaken again by an equal shock since the original energy absorbing capacity will not be available.

9.2.2 Restoration

This is the restoration of the building strength before the damage occurred. This type of action must be undertaken when there is evidence the original strength provides in an adequate level of safety.

The main purpose of restoration is to carry out structural repairs to load bearing elements. It may involve cutting portions of the elements and rebuilding them or simply adding more structural material so that the original strength is more or less restored. The process may involve inserting temporary supports, underpinning, etc. Some of the approaches are stated below:

- 1) Removal of portions of cracked masonry walls and piers and rebuilding them in richer mortar. Use of non-shrink mortar is recommended.
- 2) Addition of reinforcing mesh on both faces of cracked walls, holding it to the walls through spikes or bolts and then plastering it. Several alternatives have been used.

- 3) Injecting epoxy-like (polymer) material, which is strong in tension, into the cracks in walls, columns, beams, etc.

Where structural repairs are considered necessary, these should be carried out prior to or simultaneously with the architectural repairs so that total planning of work can be done in a coordinated manner and wastage is avoided.

9.2.3 Strengthening of existing buildings

The seismic behaviour of old existing buildings is affected by their original structural inadequacies, material degradation due to time, and alterations carried out during use over the years such as making new openings, addition of new parts inducing asymmetry in plan and elevation, etc.

The possibility of demolition and rebuilding with new earthquake resistant buildings is generally not feasible due to historical, social and economical reasons. The complete replacement of the buildings in a given area will also lead to destroying a number of social and human links. Therefore seismic strengthening of existing damaged or undamaged buildings is a definite requirement in same areas.

Strengthening improves the original strength when the evaluation of a building indicates that the strength available before the damage was insufficient and restoration alone will not be adequate in future earthquakes.

The extent of the modifications must be determined by the general principles and design methods stated in earlier chapters, and should not be limited to increasing the strength of members that have been damaged, but should consider the overall behaviour of the structure. For this reason strengthening schemes should be designed and supervised by experienced structural engineers. Commonly, strengthening procedures should aim at one or more of the following objectives:

- 1) Increasing the horizontal strength in one or both X and Y directions, by reinforcement or by increasing wall thickness/areas or the number of walls and columns.
- 2) Unifying the structure by providing a proper connection between its resisting elements, in such a way that inertia forces generated by the vibration of the building can be transmitted to the members that have the ability to resist them. Important aspects are the connections between roofs or floors and walls, between intersecting walls and between walls and foundations.
- 3) Eliminating features that are sources of weakness or that produce concentrations of stress in some members. Asymmetrical plan distribution of resisting members, abrupt changes of stiffness from one floor to the other, concentration of large masses, large openings in walls without a proper peripheral reinforcement are examples of defect of this kind.
- 4) Avoiding the possibility of brittle modes of failure by proper reinforcement and connection of resisting members. Since the cost of strengthening may go as high as 50 to 60% of the cost of rebuilding, the justification of such strengthening must be fully considered. The extent of modification must be found using the principles of strengthening discussed in previous chapters and in accordance with the local factors applicable to each building.

9.3 Repair Materials

The most common materials for damage repair work are cement and steel. In some situations non-shrink cement or an admixture like aluminium powder in ordinary portland cement is admissible. Steel may be required in many forms, like bolts, rods, angles, channels, expanded metal and welded wire fabric. Wood and bamboo are the most common material for providing temporary supports and scaffolding etc., and will be required in the form of rounds, sleepers, planks, etc.

Besides the above, special materials and techniques are available for best results in the repair and strengthening operations. A quality assurance is needed to guarantee sound results. They are described below:

9.3.1 Shotcrete

Shotcrete is a method of applying a combination of sand and portland cement (concrete) which is mixed pneumatically and conveyed in dry state to the nozzle of a pressure gun. Here water is mixed and hydration takes place just prior to expulsion. The material bonds well to properly prepared surface of masonry and steel. In application to curved or irregular surfaces, its high strength after application and good physical characteristics, make for an ideal means to achieve added structural capability in walls and other elements. There are some restrictions of clearance, thickness, direction of application, etc.

9.3.2 Polymer resins

Polymer resins are excellent binding agents with high tensile strength with compositions that can be changed as per requirements. Polymer components are mixed just prior to application. The product is of low viscosity and can be injected in small cracks. The higher viscosity epoxy resin can be used for surface coating or filling larger cracks or holes. The polymer strength is dependent upon the temperature of curing and method of application.

9.3.3 Polymerised mortar

For larger void spaces, it is possible to combine polymers of either low viscosity or higher viscosity with sand aggregate to form polymer mortar. Polymer mortar mixture has higher compressive strength, higher tensile strength and a lower modulus of elasticity than Portland cement concrete. Thus the mortar is not stiff enough to replacing reinforced concrete.

9.3.4 Quick-setting cement mortar

This material was originally developed for the use as a repair material for reinforced concrete floors adjacent to steel blast furnaces. It is a non-hydrous magnesium phosphate cement with two components, a liquid and a dry, which can be mixed in a manner similar to portland cement concrete.

9.3.5 Mechanical anchors

Mechanical anchors employ wedging action to provide anchorage. Some anchors provide both shear and tension resistance. Such anchors are manufactured to give required strengths. Alternatively, chemical anchors bonded in drilled holes with polymer adhesives can be used.

9.4 Retrofitting Steps

(See the flowchart in Fig. 9.1.)

- 1) Determine as accurately as possible how the building will behave when shaken by an earthquake:
 - a) check the building
 - b) check building material quality
 - c) list all components of the building that are damaged
- 2) Perform a structural analysis for the building to get an idea of the causes of damage and determine the load paths when shaken by the earthquake.
- 3) Determine the causes of damage of components; caused by shear, compression, tension, flexure, anchoring, etc.
- 4) As soon as the type of damage can be identified, repair and restoration of the components can be done separately in order that the original strength of the components can be restored.
- 5) If results of analysis indicate that the building with restored components can withstand the maximum expected earthquake for that area based on the latest code, then there is no need to strengthen.
- 6) However, if the building with restored components was not designed or designed for a lower than the maximum expected earthquake specified by the latest code, then the building needs to be strengthened.
 - a) For strengthening, the restored building must be analyzed to identify which components must be strengthened.
 - b) If cost for strengthening the building to its original function is not feasible, one option that can be chosen is to change the building function with less stringent requirement. Therefore, the cost will be reduced.

9.5 Techniques to Restore Original Strength

While considering restoration work, it is important to realize that even fine cracks in load bearing members which are unreinforced, like masonry and plain concrete reduce their resistance significantly. Therefore all cracks must be located and marked carefully and the critical ones fully repaired either by injecting strong cement or chemical grout or by providing external bandages. The techniques are described below along with other restoration measures.

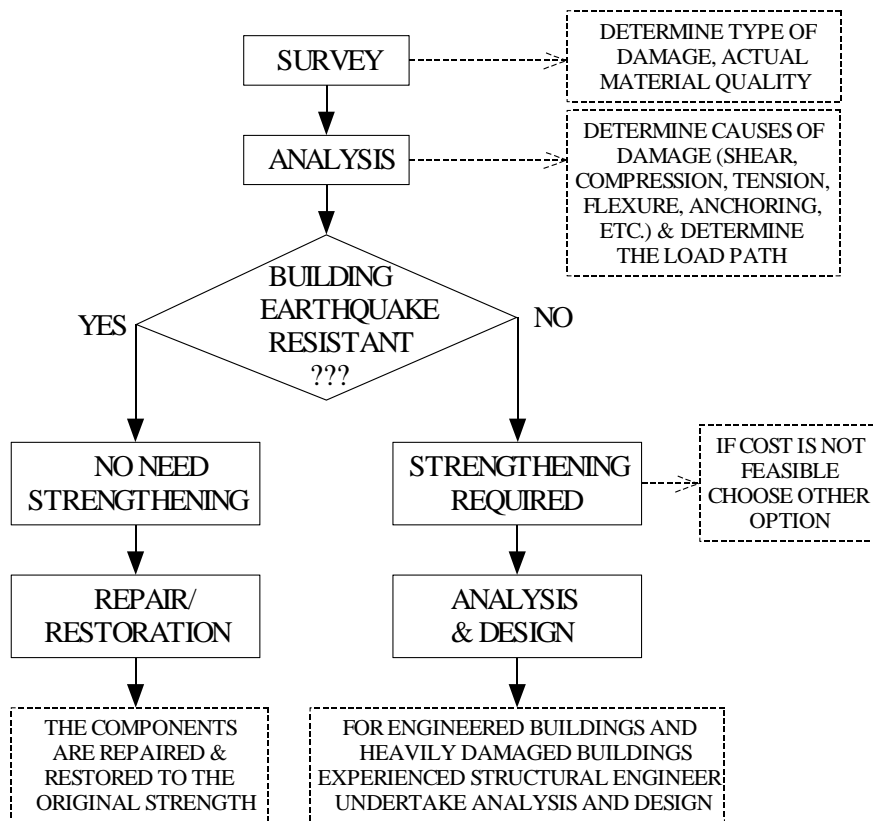


Figure 9.1: Flowchart for retrofitting

9.5.1 Small cracks

If the cracks are reasonably small (opening width = 0.75 mm), the technique to restore the original tensile strength of the cracked element is by pressure injection of epoxy-like material. The procedure is as follows (see Figs. 9.2 and 9.3).

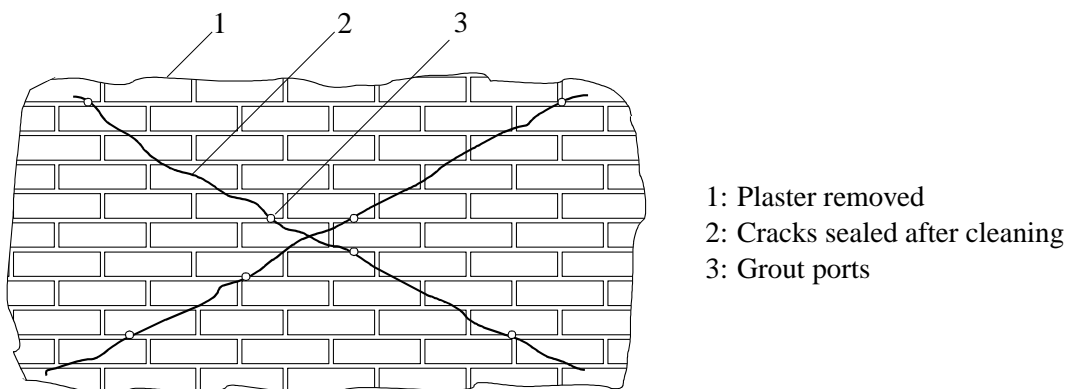


Figure 9.2: Strengthening existing masonry (A)

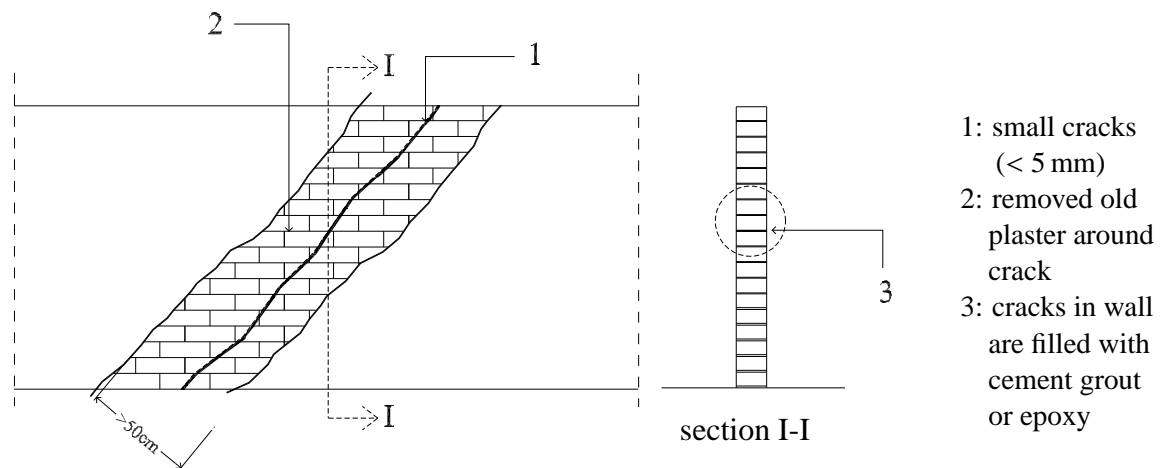


Figure 9.3: Strengthening existing masonry (B)
(Grout injection in cracks)

The external surfaces are cleaned of non-structural materials and plastic injection ports are placed along the surface of the cracks on both sides of the member and are secured in place with a polymer sealant. The centre to centre spacing of these ports is approximately equal to the thickness of the element. After the sealant has cured, a low viscosity epoxy resin is injected into one port at a time, beginning at the lowest part of the crack in case it is vertical or at one end of the crack in case it is horizontal.

The resin is injected until it is seen flowing from the opposite sides of the member at the corresponding port or from the next higher port on the same side of member. The injection port should be closed at this stage and injection equipment moved to the next port and so on.

The smaller the crack, higher is the pressure or more closely spaced should be the ports so as to obtain complete penetration of the epoxy material throughout the depth and width of member. Larger cracks will permit larger port spacing, depending upon width of the member. This technique is appropriate for all types of structural elements, i.e. beams, columns, walls and floor units in masonry as well as concrete structures. In the case of loss of bond between reinforcing bar and concrete, if the concrete adjacent to the bar has been pulverised to a very fine powder, this powder will dam the epoxy from saturating the region. So it should be cleaned properly by air or water pressure prior to injection of epoxy.

9.5.2 Wide cracks and crushed concrete

For cracks wider than about 6 mm or for regions in which the concrete or masonry has crushed, a treatment other than injection is indicated. The following procedure can be adopted (see Fig. 9.4).

- 1) The loose material is removed and replaced with any of the materials mentioned earlier, i.e., expansive cement mortar, quick setting cement.
- 2) Where found necessary, additional shear or flexural reinforcement is provided in the region of repairs. This reinforcement must be covered by mortar to give further strength as well as protection.

- 3) In areas of very severe damage, replacement of the member or portion of the member can be carried out as discussed later.
- 4) In the case of damage to walls and floor diaphragms, steel mesh can be provided on the outside of the surface and nailed or bolted to the wall. Then it can be covered with plaster or micro-concrete.

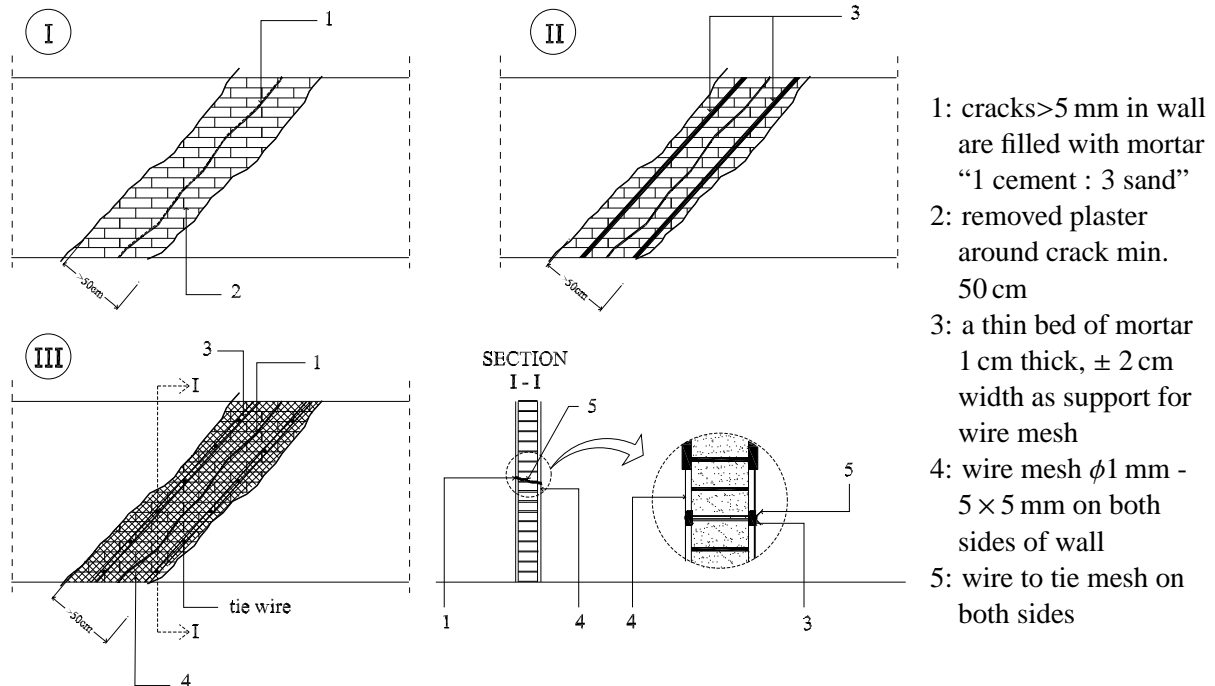


Figure 9.4: Strengthening existing masonry (C)
(Wire mesh and cement plaster)

9.5.3 Fractured, excessively yielded and buckled reinforcement

In the case of a severely damaged reinforced concrete member, it is possible that the reinforcement has buckled or elongated, or excessive yielding may have occurred. This element can be repaired by replacing the old portion of steel with new steel using butt welding or lap welding. Ensure the quality of the welding is checked by a qualified welder.

Splicing by overlapping is risky. If repair has to be made without removal of the existing steel, the best approach depends upon the space available in the original member. Additional stirrup ties are to be added in the damaged portion before concreting so as to confine the concrete and enclose the longitudinal bars to prevent their buckling in the future.

In some cases it may be necessary to anchor additional steel into existing concrete. A common technique for providing the anchorage uses the following procedure:

A hole larger than the bar is drilled. The hole is filled with epoxy, expanding cement, or other high strength grouting material. The bar is pushed into place and held there until the grout has set. This method is prone to workmanship problems so on-site tests are required to confirm quality.

Retrofitting of columns damaged at their tops (See Fig. 9.5.)

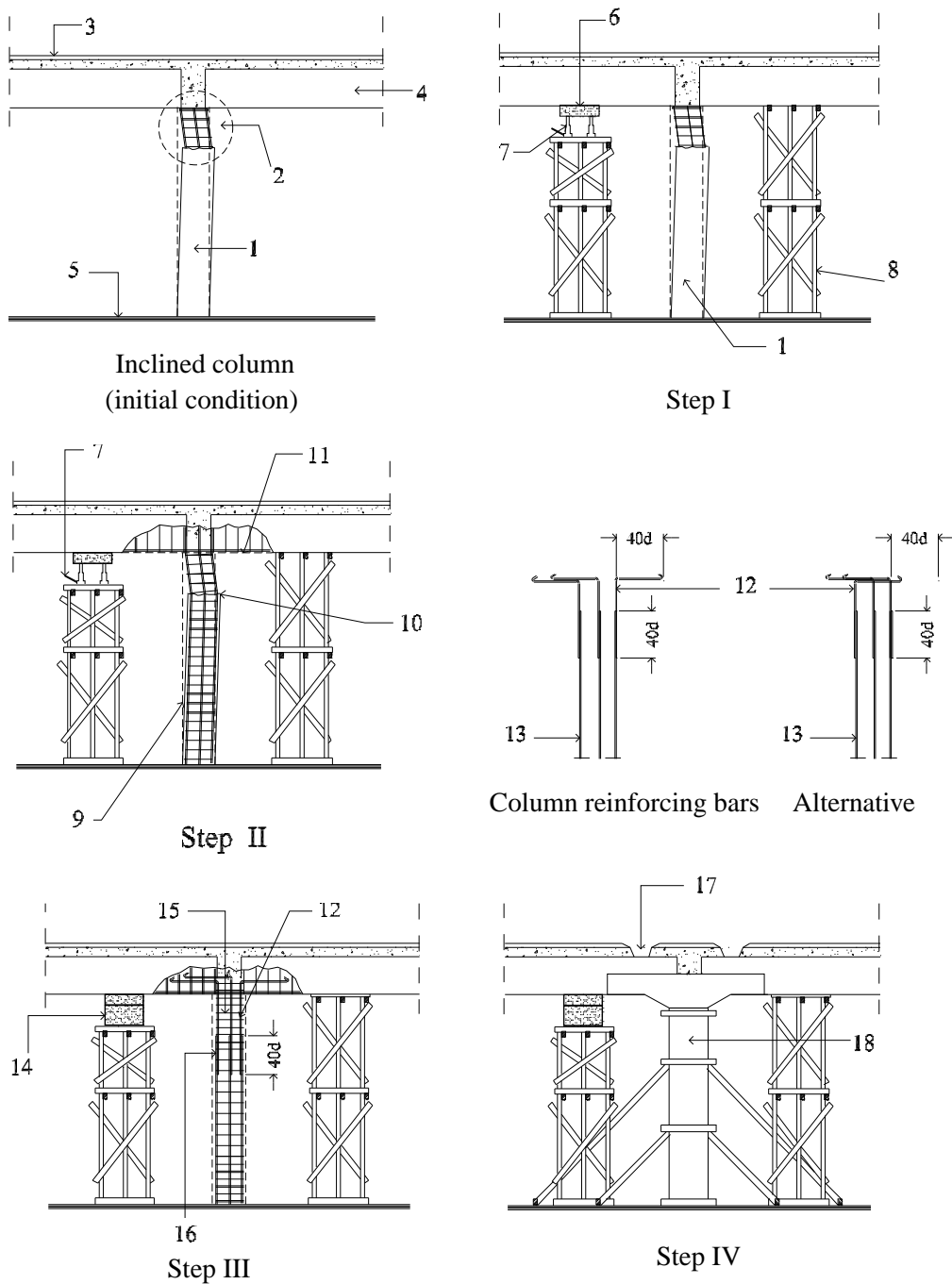
- 1) Support the beams around the column to be retrofitted.
- 2) The slanted beam must be supported and jacked-up to restore the level (Step I).
Note: if jacking is difficult, the deformed column reinforcement can be cut first.
- 3) Remove the concrete of the damaged column.
- 4) Cut the deformed reinforcing bars (Step II).
- 5) Jack-up the beam until the desired level.
- 6) Remove the concrete of the lower part of the beam for improving the joint reinforcement detail (Step II).
- 7) After the structure is horizontal at the desired level, the jacks are replaced with timber or concrete blocks (Step III).
- 8) Install the new column reinforcement and splice with the existing one. the length of splice between the existing and the new reinforcement is minimum $40d$ (Step III).
Re-arrange and add the column hoops.
- 9) Construct the formwork of 9 mm thick plywood, the formwork must be shaped like a cone (Step IV).
- 10) If there is not enough space, concreting can be done by drilling a hole in the slab (Step IV).
- 11) Place the concrete as soon as the formwork is ready.
- 12) After 24 hours, the formwork can be removed and the protruding concrete part (due to the cone shape) can be chipped.

9.5.4 Fractured wooden members and joints

Since wood is an easily workable material, it is relatively easy to restore the strength of wooden members, beams, columns, struts and ties by splicing additional material. Weathered or rotten wood should first be removed. Nails, wood screws or steel bolts will be most convenient as connectors. It will be advisable to use steel straps to cover all such splices and joints so as to keep them tight and stiff.

9.6 Retrofitting of Roofs or Floors

- 1) Slates and roofing tiles are brittle and easily dislodged. Where possible they should be replaced with corrugated iron or sheeting.
- 2) False ceilings of brittle material are dangerous. Non-brittle material like hesian cloth, bamboo matting, or other light materials should be used.
- 3) Roof truss frames should be braced by welding or clamping suitable diagonal bracing members in the vertical as well as horizontal planes.
- 4) Anchors of roof trusses to supporting walls should be improved and the roof thrust on walls should be eliminated. Figs. 9.6 and 9.7 illustrate one of the methods.



1: inclined column to be retrofitted, 2: column crushed (deformed), 3: concrete slab not horizontal, 4: beam, 5: floor, 6: timber blocks as footing, 7: jack to restore the level/to adjust the desired level, 8: timber frames scaffolding, 9: remove the concrete of the inclined column, 10: cut the deformed reinforcing bars, 11: chip lower part of beam to anchor column reinforcement, 12: new main column reinforcement, anchor to the beam, 13: existing reinforcing bars, 14: timber block to replace the jack, 15: re-arrange and add if necessary the column stirrups, 16: welded, 17: holes in slabs to pour concrete, 18: formwork

Figure 9.5: Inclined column and its retrofitting

- 5) Where the roof or floor consists of prefabricated units like RC rectangular, T or channel units or wooden poles and joists carrying brick tiles, tying together of such units is necessary. Timber elements could be connected to diagonal planks nailed to them and spiked to an all round wooden frame at the ends. RC elements may either have 40 mm cast-in-situ concrete topping with 6 mm ϕ bars 150 mm c/c both ways or a horizontal cast-in-situ RC ring beam all round into which the ends of RC elements are embedded. Fig. 9.8 shows one such detail.
- 6) Roofs or floors consisting of steel joists and flat or segmental arches must have horizontal ties holding the joists horizontally in each arch span so as to prevent the spreading of joists. If such ties do not exist, these should be installed by welding or clamping.

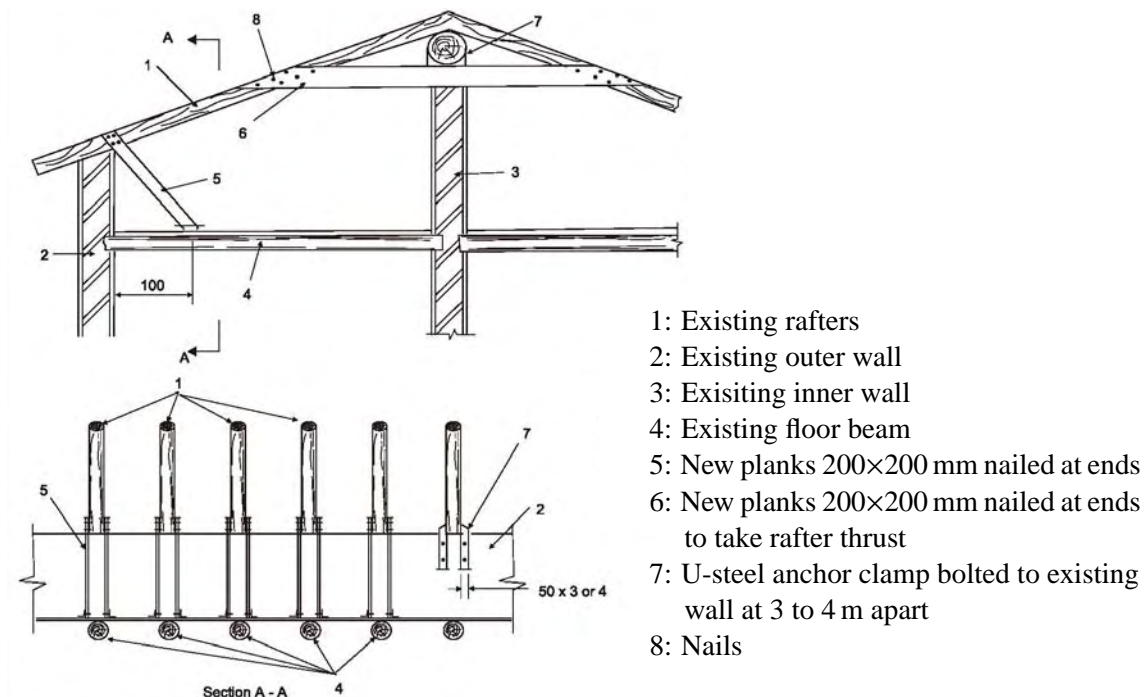


Figure 9.6: Roof modification to reduce thrust of walls

9.7 Substitution or Strengthening of Slabs

(a) Insertion of a new slab

A rigid slab inserted into existing walls plays an important role in the resisting mechanism of the building keeping the walls together and distributing seismic forces among the walls.

The slab has to be properly connected to the walls through appropriate keys. Fig. 9.8 shows typical arrangement to be adopted while in Fig. 9.9 some details are shown.

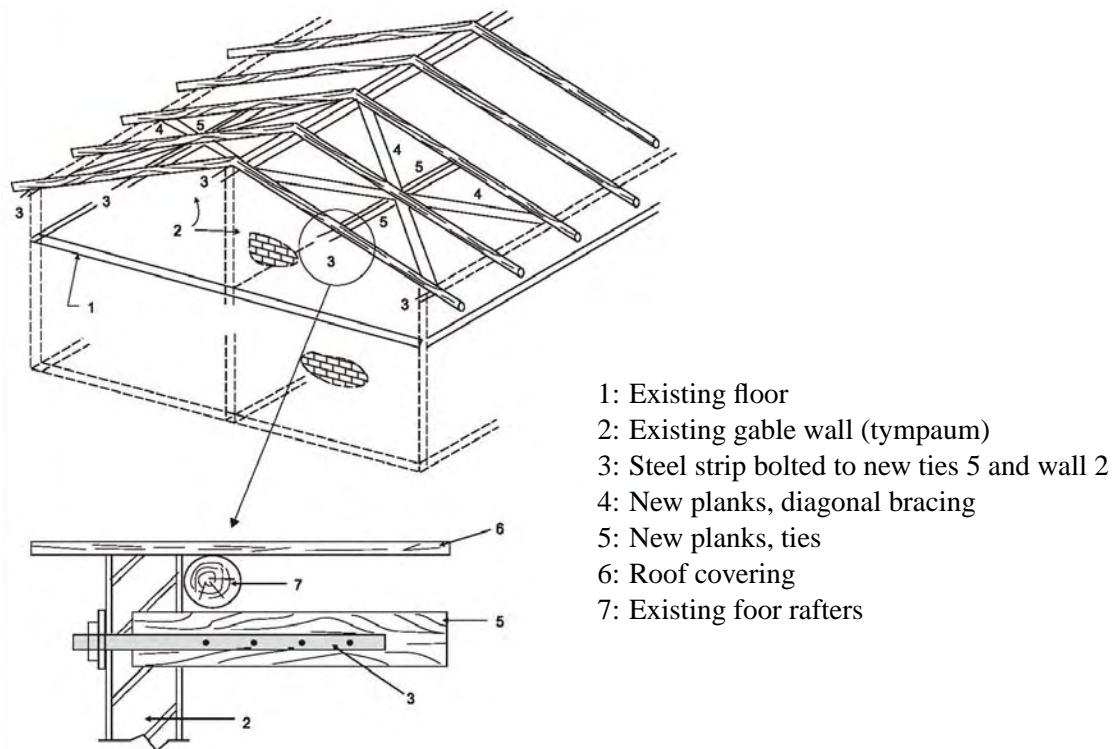


Figure 9.7: Details of new roof bracing

(b) Existing wooden floors

In the case in which the existing slab is not removed the following actions have to be undertaken:

Stiffening of the floor

This can be achieved either by planks nailed perpendicularly to the existing ones (see Fig. 9.10a) or by placing a RC thin slab over the old one (see Fig. ??b).

In this case steel mesh is nailed to the wooden floor and connected to the walls by a number of distributed steel anchors. These can be hammered into the interstices of the wall and local hand cement grouting has to be applied for seating.

Connection of the slab to the walls

A proper link can be obtained by means of the devices shown in Figs. 9.11 and 9.12.

They consist of flat steel bars nailed to the wooden supporting beams and to the wooden floor. Holes drilled in the walls to anchor the bars have to be infilled with cement. If steel mesh has been used, the connection can be made as shown in Fig. 9.9, i.e., inserting a small RC band into the existing walls. The band has to be keyed at least every 3 m.

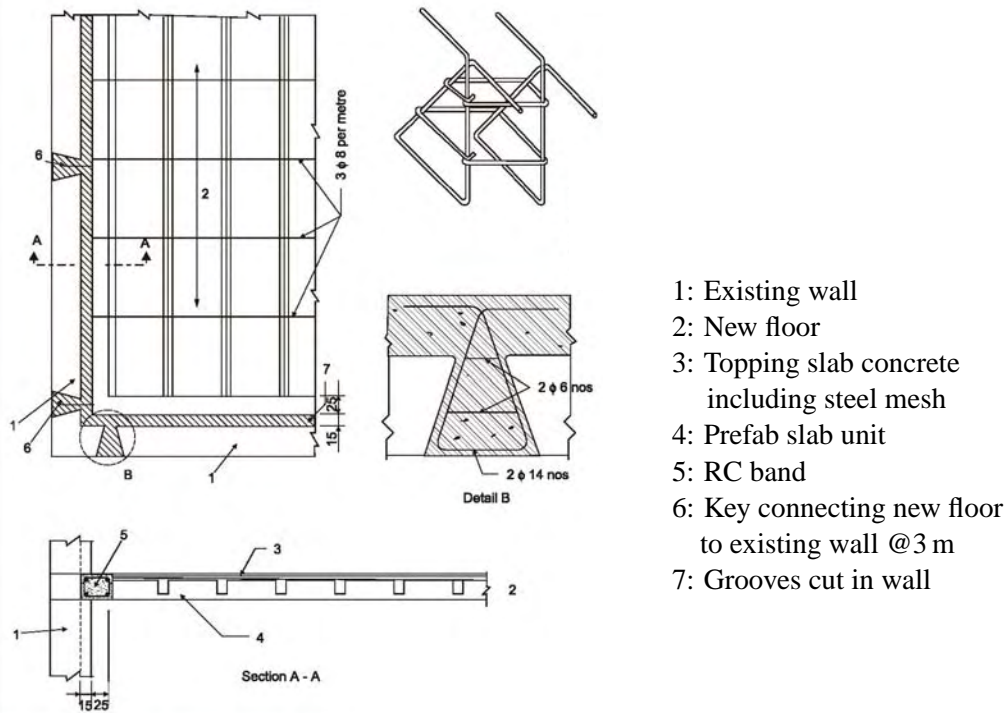


Figure 9.8: Stiffening of an existing floor and tying it into the walls

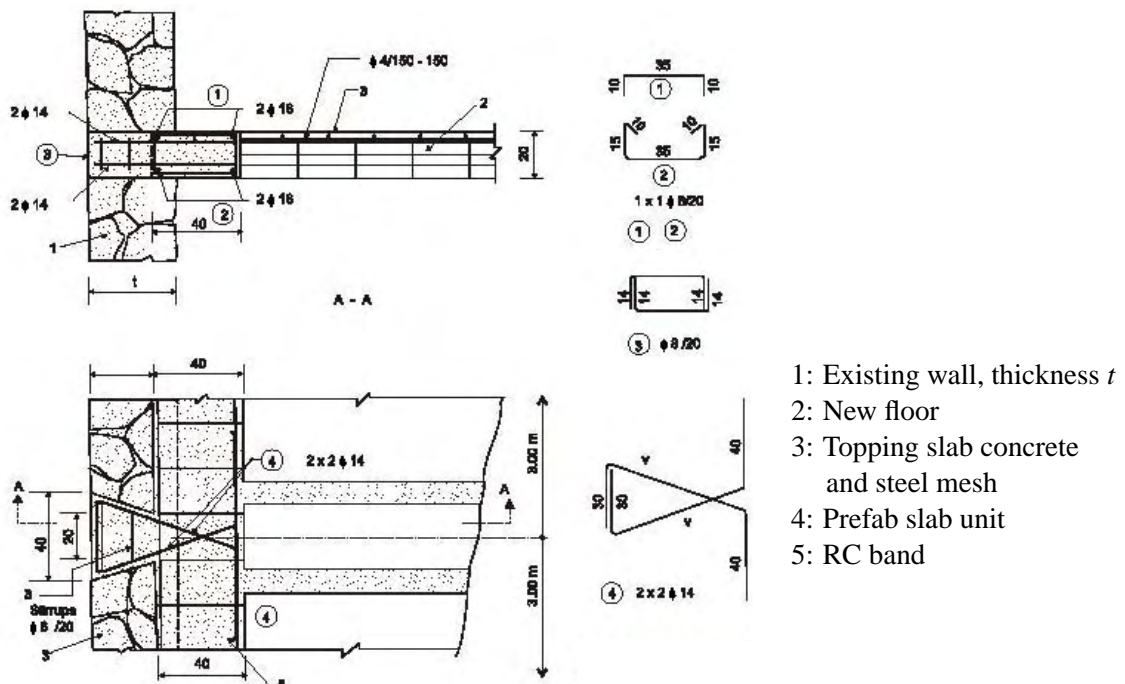


Figure 9.9: Details of inserted slab

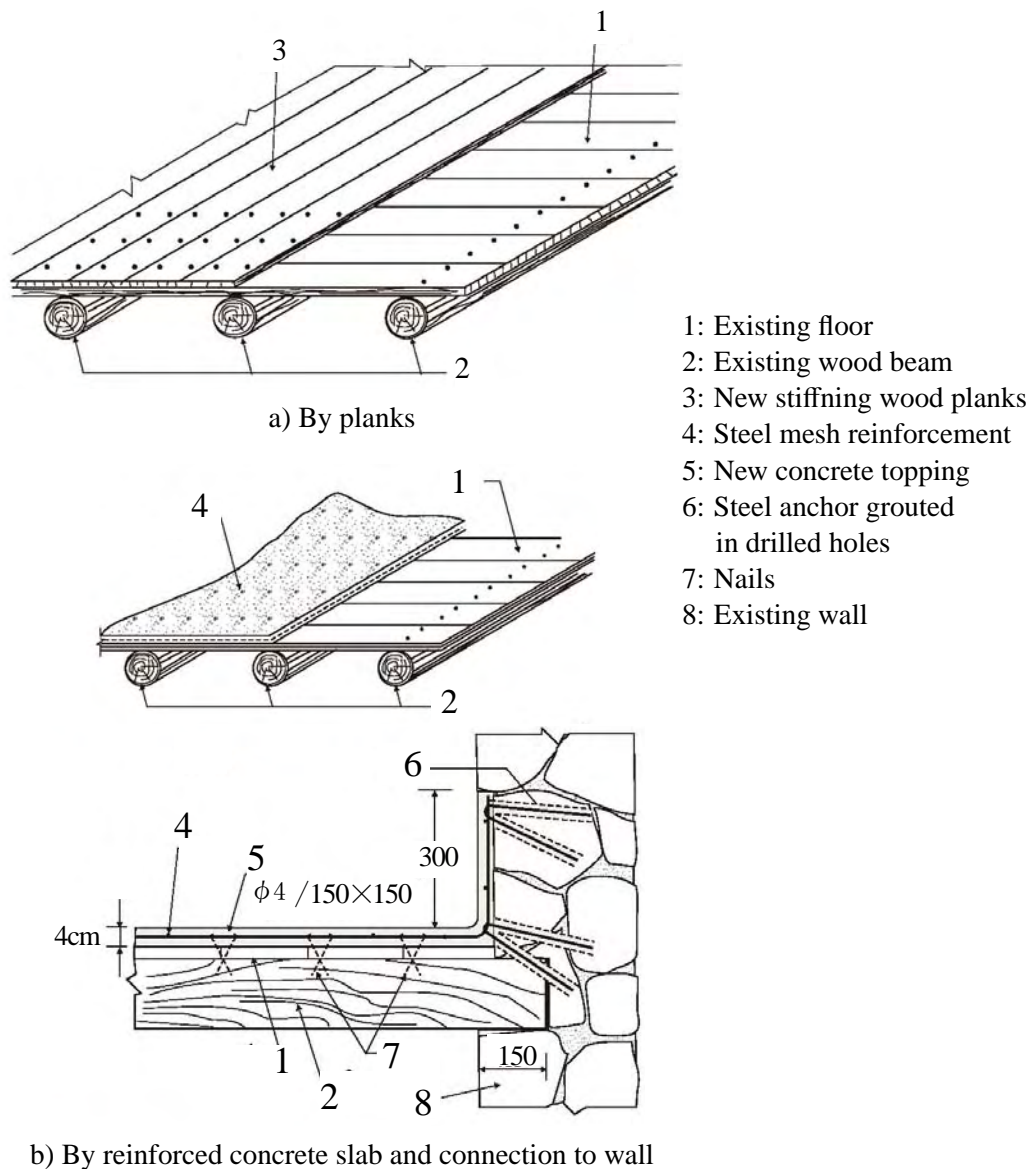


Figure 9.10: Stiffening of wooden floor

9.8 Improving Structural Layout and Strengthening of Walls

9.8.1 Inserting new walls

Where existing buildings show asymmetries which may produce dangerous torsional effects during earthquakes, the center of masses can be made coincident with the center of stiffness by separating parts of buildings, thus achieving individual symmetric units and/or, inserting new vertical resisting elements such as new masonry or reinforced concrete walls either internally as shear walls, or externally as buttresses. Insertion of cross walls will be necessary for providing transverse supports to longitudinal walls of long barrack-type buildings used for various purposes such as schools and dormitories.

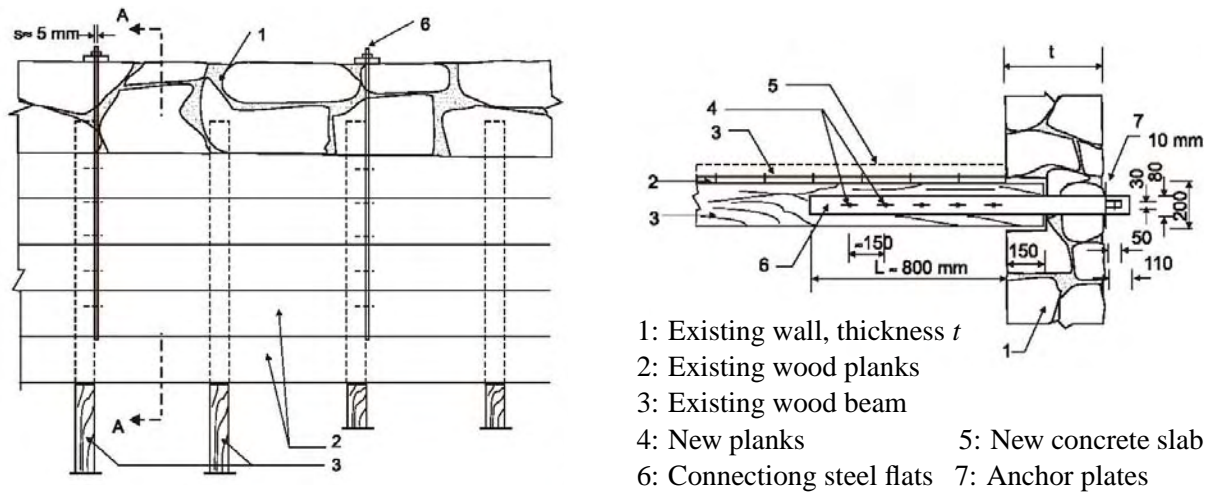


Figure 9.11: Connection of floor to wall

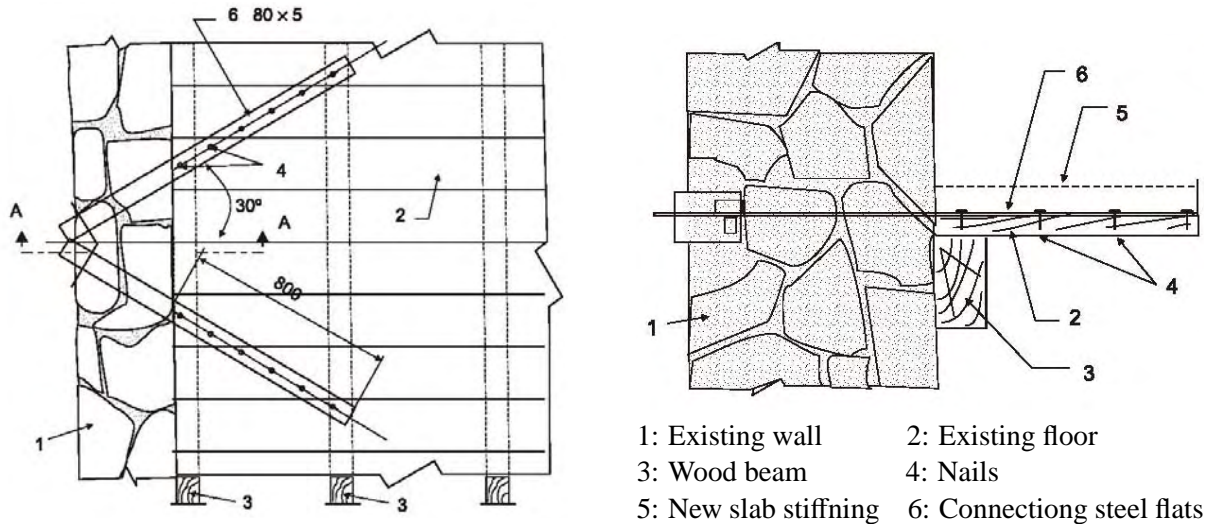
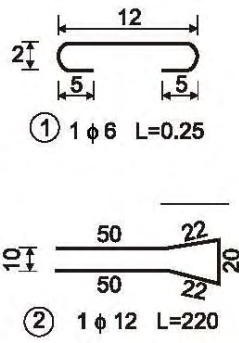
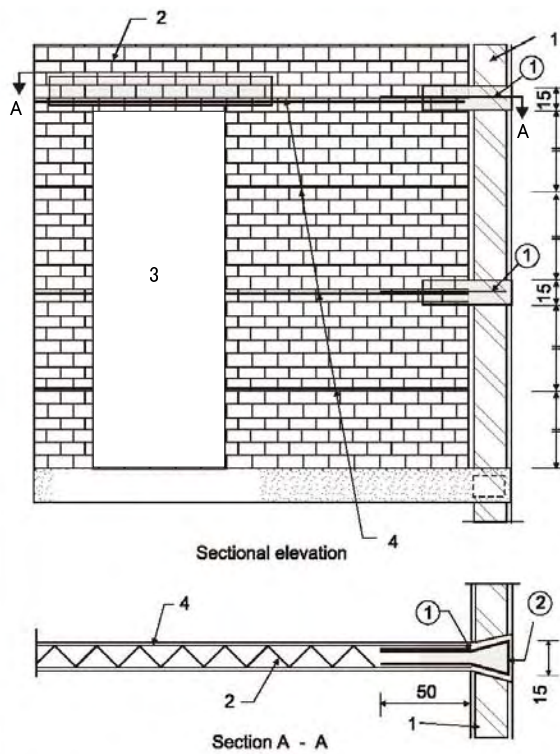


Figure 9.12: Connection of floor with wall

The main problem with such modifications is the connection of new walls with old walls. Figs. 9.13 and 9.14 show two examples of connection of new walls to existing ones. The first case refers to a T-junction, the second figure to a corner junction. In both cases the link to the old walls is performed by means of a number of keys made in the old walls. Steel is inserted in them and local cement infilling is made. In the second case however the connection can be achieved by a number of steel bars inserted in short drilled holes which substitute as keys.

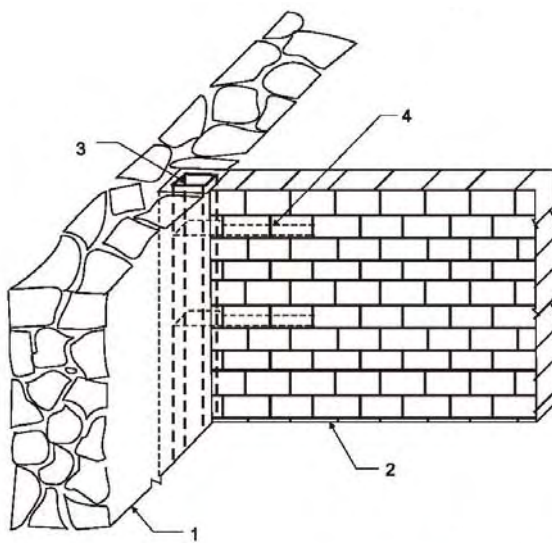
9.8.2 Strengthening existing walls

The lateral strength of buildings can be improved by increasing the strength and stiffness of existing individual walls whether they are cracked or uncracked. This can be achieved by (a)



- 1: Existing wall
- 2: New wall
- 3: Door opening
- 4: Horizontal reinforcement
(example of truss system shown)

a) Connection of new brick walls and old stone walls (T-junction)



- 1: Existing old wall
- 2: New wall
- 3: Concrete column for bonding
- 4: Connecting ties of steel,
every four courses

b) Connection of new brick wall to existing stone wall

Figure 9.13: Connection of new and old masonry walls

grouting, (b) addition of vertical reinforced concrete covering on both sides of the wall, (c) splicing walls and (d) pre-stressing walls.

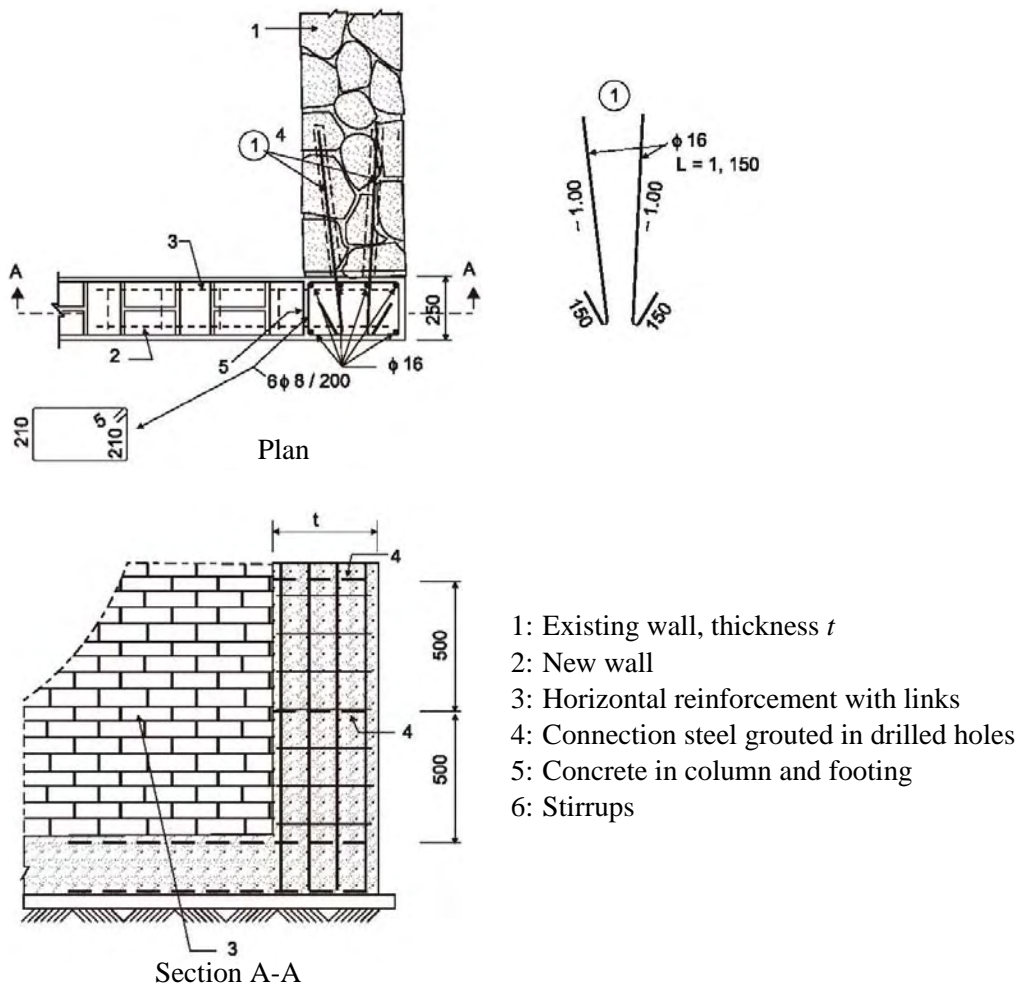


Figure 9.14: Connection of new and old walls (corner junction)

(a) Grouting

A number of holes are drilled in the wall (2 to 4 m²). First water is injected in order to wash hole surfaces inside and to improve cohesion between the grouted mixture and the wall elements. Secondly a cement water mixture (“1 : 1”) is grouted at low pressure (0.1 to 0.25 MPa or 1 to 2.5 kgf/mm²) in the holes starting from the lower holes and going up.

Alternatively, polymeric mortars may be used for grouting. The increase of shear strength which can be achieved in this way is considerable. However grouting cannot be relied on as far as the improving or connection between orthogonal walls is concerned. Note that pressure needed for grouting can be obtained by gravity flow from super-elevated tanks.

(b) Strengthening with wire mesh

One layout of steel mesh (welded wire fabric approximately 50 × 50 mm) is placed on each side of the wall. They are connected by steel crossties 500 to 750 mm apart (see Fig. 9.15). A 20 to 40 mm thick cement mortar or micro-concrete layer is then applied on the mesh thus giving

rise to two interconnected vertical plates. This system can also be used to improve connection of orthogonal walls.

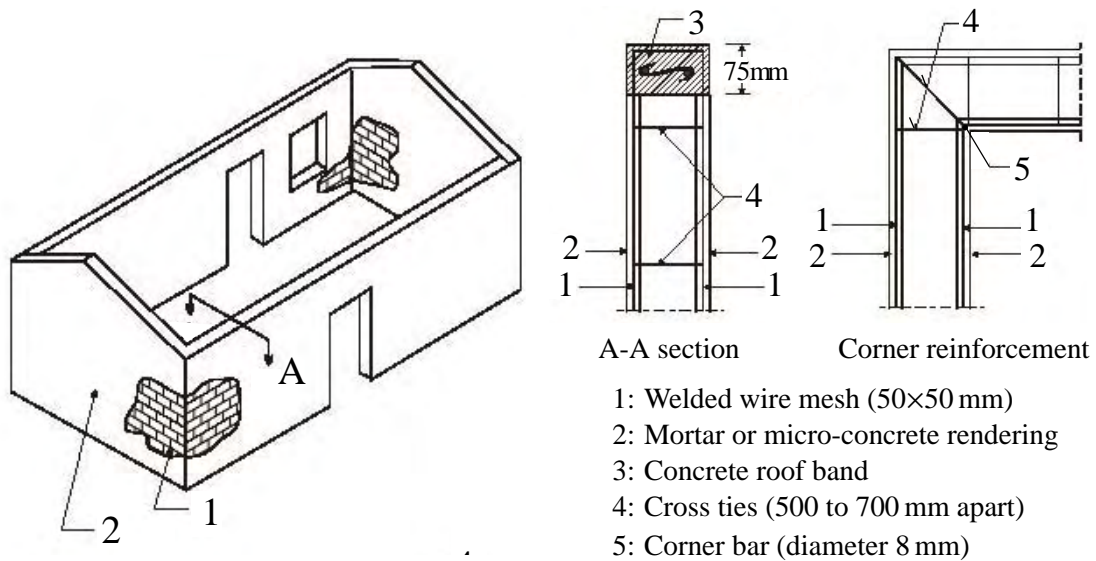


Figure 9.15: Strengthening with wire mesh and mortar

(c) Improving connections between existing stone walls

In stone buildings of historic importance consisting of fully dressed stone masonry in good mortar, effective stitching of perpendicular walls can be done by drilling inclined holes through them, inserting steel rods and injecting cement grout (see Fig. 9.16).

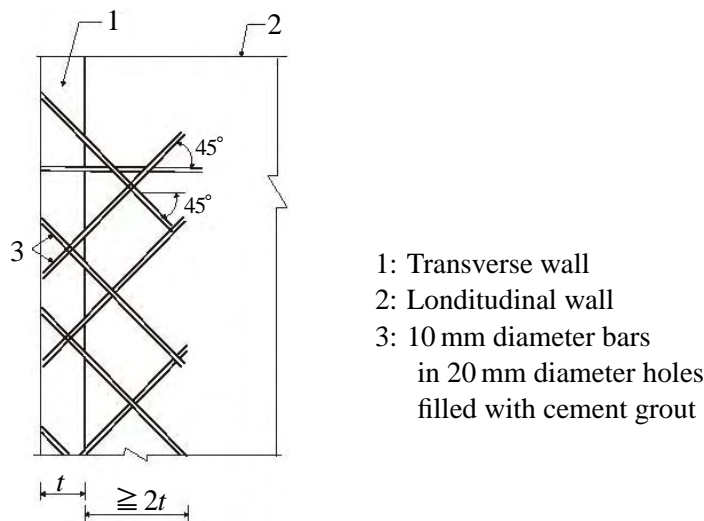


Figure 9.16: Stitching transverse walls with inclined bars

(d) Prestressing

A horizontal compression state induced by horizontal tendons can be used to increase the shear strength of walls. Moreover this will also improve the connections of orthogonal walls (see Fig. 9.17). The easiest way to achieve the precompression is to place a steel rod on each side of the wall and stress them by turnbuckles. Note that beneficial effects can be obtained by slight horizontal prestressing (about 0.1 MPa or 1 kgf/mm²) on the vertical section of the wall. Prestressing is also useful to strengthen spandrel beams between two rows of openings where no rigid slab exists.

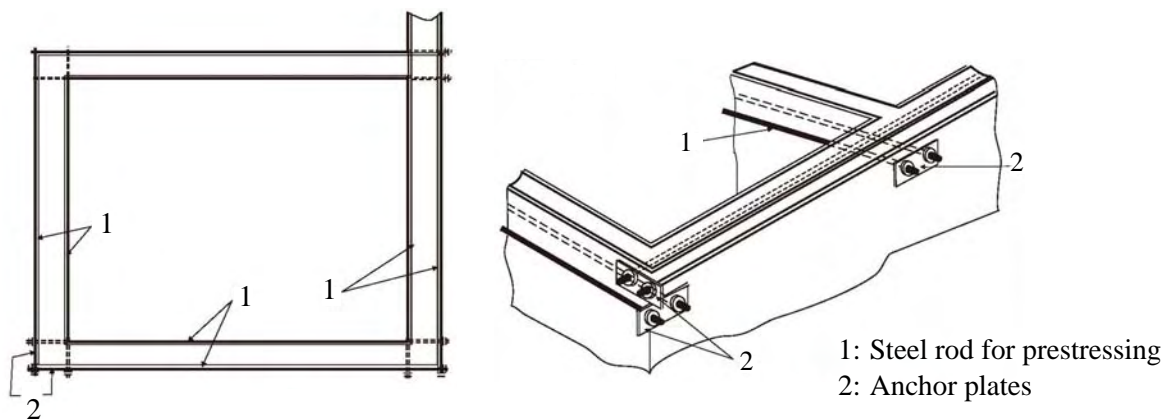


Figure 9.17: Strengthening of walls by prestressing

9.8.3 External binding

Opposite parallel walls can be held to internal cross walls by prestressing bars as illustrated in Fig. 9.17. The anchoring is done against horizontal steel channels instead of small steel plates. The steel channels running from one cross wall to the other will hold the walls together and improve the integral box-like action of the walls.

The technique of covering the wall with steel mesh and mortar or micro-concrete can be used only on the outside surface of external walls but maintaining continuity of steel at the corners. This strengthens the walls as well as bind them together. As a variation and for economy in the use of materials, the covering may be in the form of vertical splints between openings and horizontal bandages over spandrel walls at suitable number of points only (see Fig. 9.18).

9.8.4 Other points

- 1) **Masonry arches** If walls have large arched openings in them, it is necessary to install tie rods across them at springing levels or slightly above it by drilling holes on both sides and grouting steel rods in them (see Fig. 9.19 a). Alternatively, a lintel consisting of steel channels or I-shapes, could be inserted just above the arch to take the load and relieve the arch as shown at Fig. 9.19 b). In jack-arch roofs, flat iron bars or rods may be provided to connect the bottom flanges of I-beams, connected by bolting or welding.

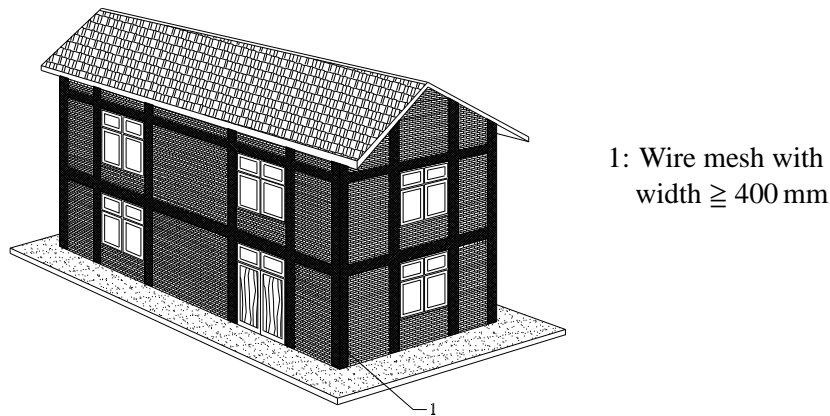


Figure 9.18: Splint and bandage strengthening technique

- 2) Random rubble masonry walls are most vulnerable to complete collapse and can be strengthened by internal impregnation by rich cement mortar grout in the ratio of “1:1” as explained in Sec. 9.8.2 (a) or better still, covered with steel mesh and mortar as in Sec. 9.8.2 (b). Damaged portions of the wall, if any, should be reconstructed using richer mortar.
- 3) For bracing the longitudinal walls of long barrack type buildings, a portal type framework can be inserted transverse to the walls and connected to them. Alternatively, masonry buttresses or, pillasters may be added externally as shown in Fig. 9.20.
- 4) In framed buildings, the lateral resistance can be improved by inserting knee braces or full diagonal braces or inserting infill walls.

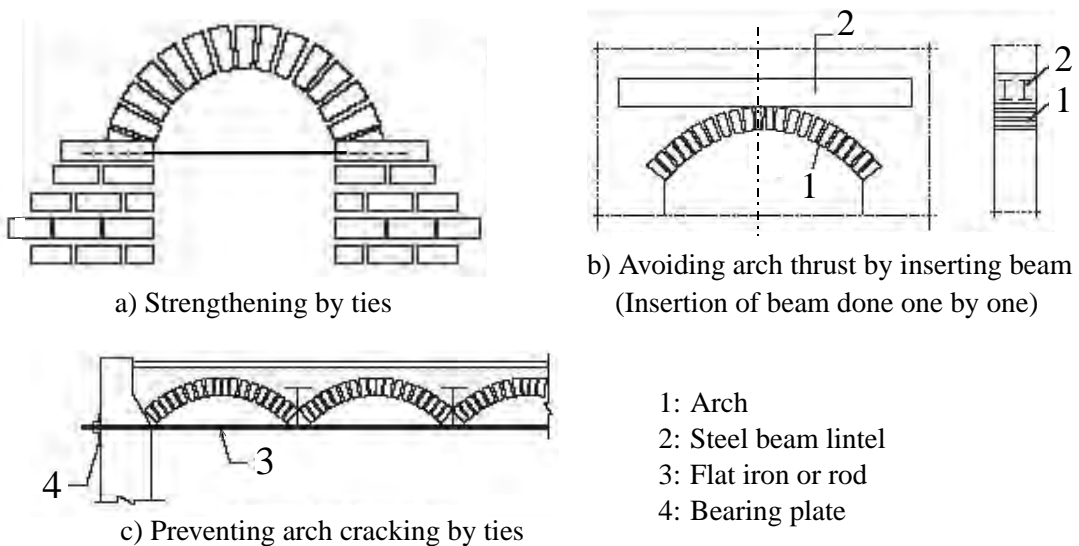


Figure 9.19: Strengthening arches in masonry

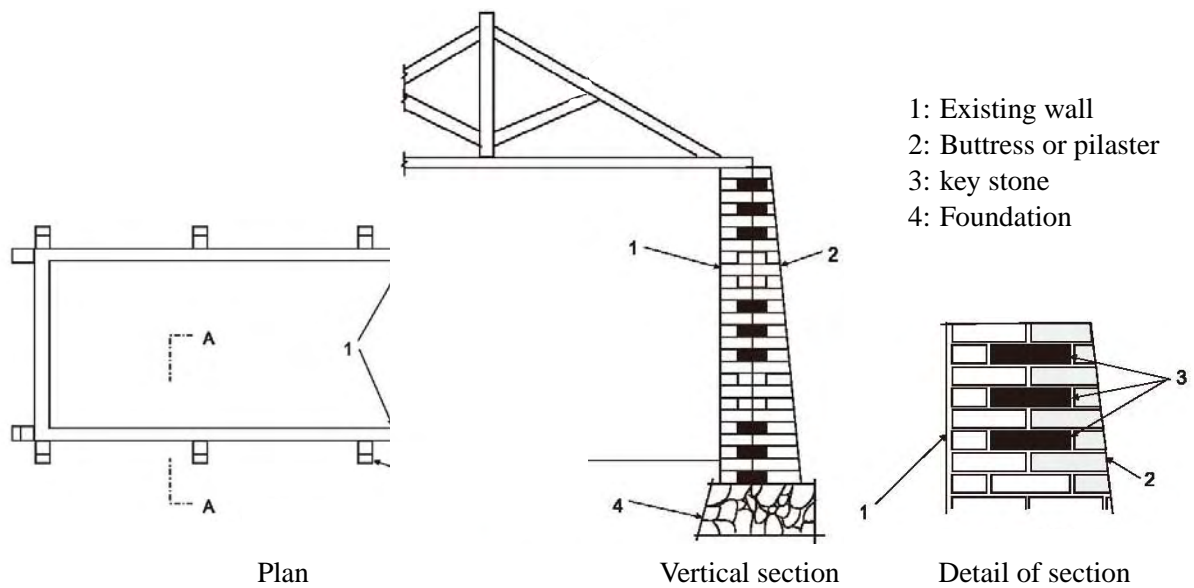


Figure 9.20: Strengthening of long walls by buttresses

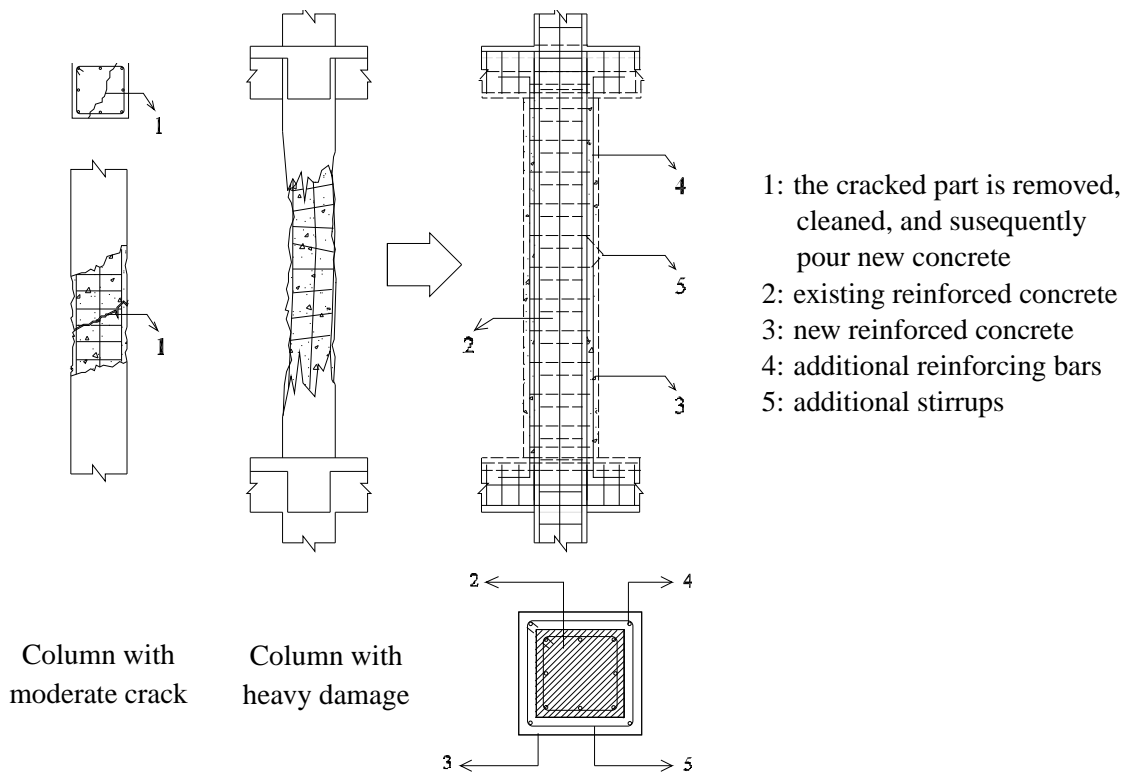
9.9 Strengthening RC Members

The strengthening of reinforced concrete (RC) members is a task that should be carried out by a structural engineer according to calculations. Here only a few suggestions are included to illustrate the ways in which the strengthening could be done.

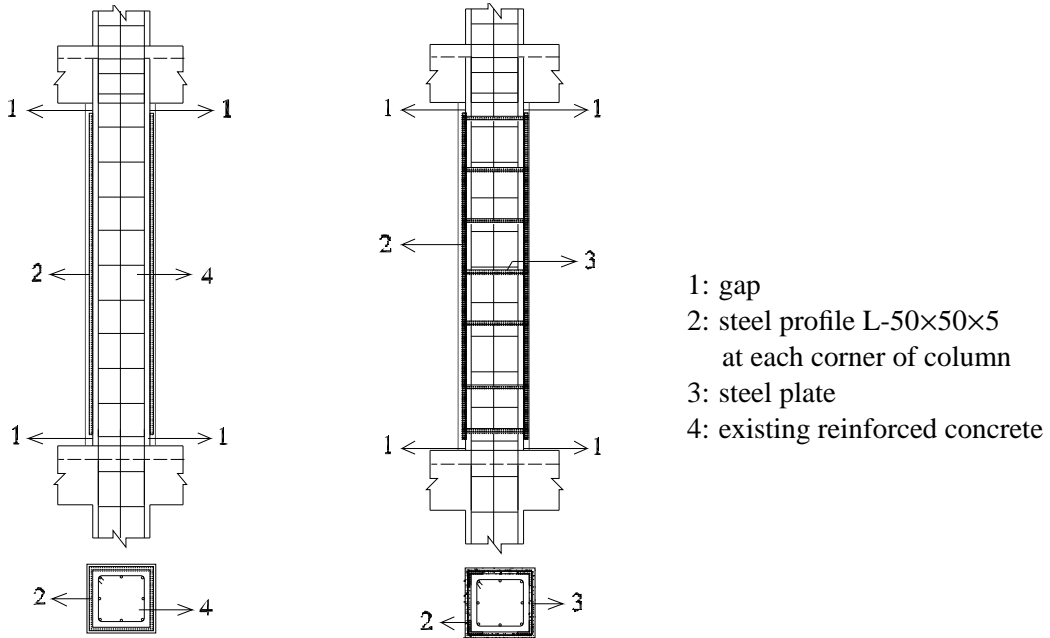
- 1) RC columns can best be strengthened by jacketing. By providing an additional cage of longitudinal and lateral tie reinforcement around the columns and casting a concrete ring (see Fig. 9.21), the desired strength and ductility can be built-up.
- 2) Jacketing a reinforced concrete beam can also be done in the above manner. For holding the stirrup in this case, holes will have to be drilled through the slab (see Figs. 9.22 and 9.23).
- 3) A similar technique could be used for strengthening RC shear walls.
- 4) Inadequate sections of RC column and beams can also be strengthened by removing the cover to old steel, welding new steel to old steel and replacing the cover.

When adding new concrete to old concrete the original surface should be roughened, groves made in the appropriate direction for providing shear transfer. The ends of the additional steel are to be anchored in the adjacent beams or columns as the case may be.

- 5) RC beams can also be strengthened by applying prestress to them so that opposite moments are caused to those applied. The wires will run on both sides of the web outside and anchored against the end of the beam through a steel plate.

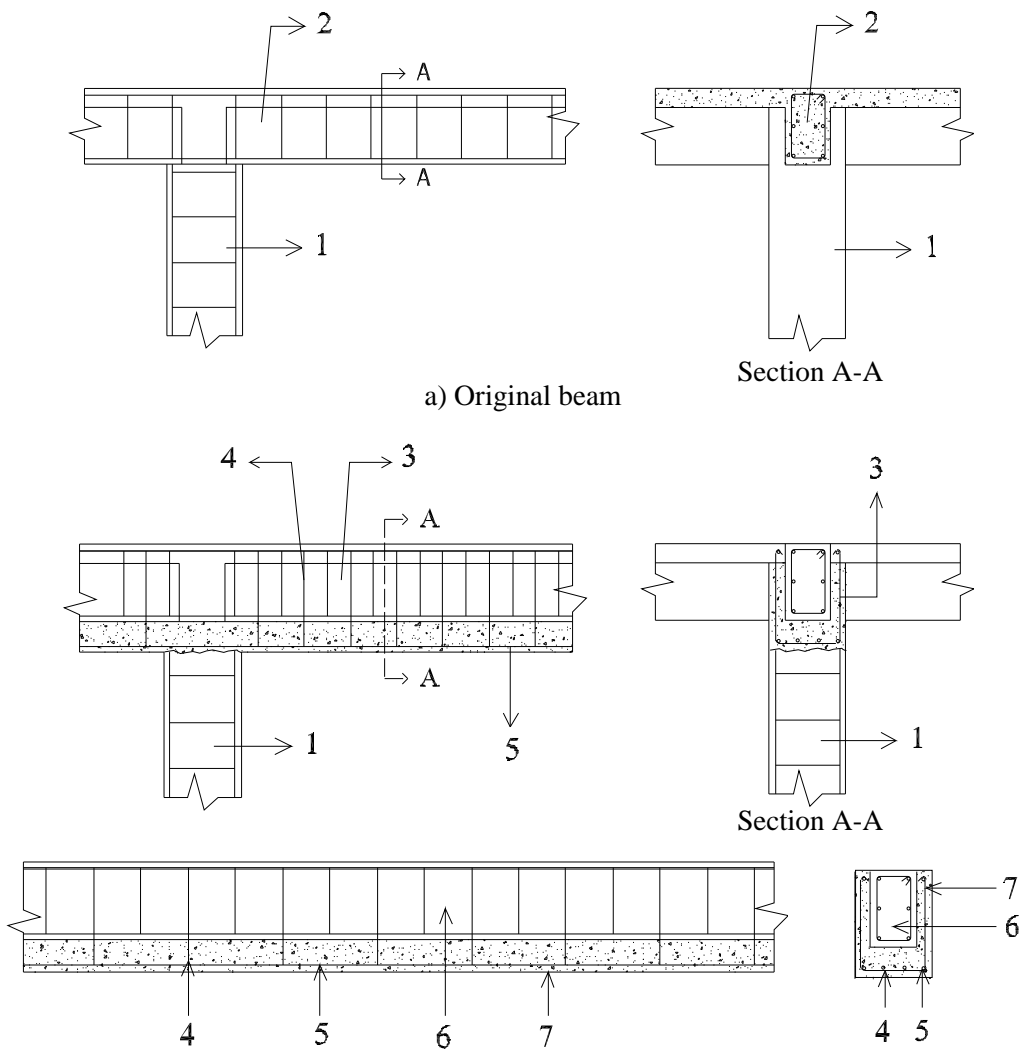


a) Strengthening of damaged column adding flexural and shear strength



b) Adding steel profile to increase shear strength without increasing flexural capacity

Figure 9.21: Strengthening RC columns



b) Increasing flexural and shear strength by adding reinforcing bars and stirrups

1: column, 2: beam with added reinforcing bars, 3: increased beam dimension with additional reinforcement, 4: additional stirrups, 5: additional reinforcing bars, 6: existing reinforced concrete, 7: new concrete

Figure 9.22: Increasing flexural and shear strength of existing beams

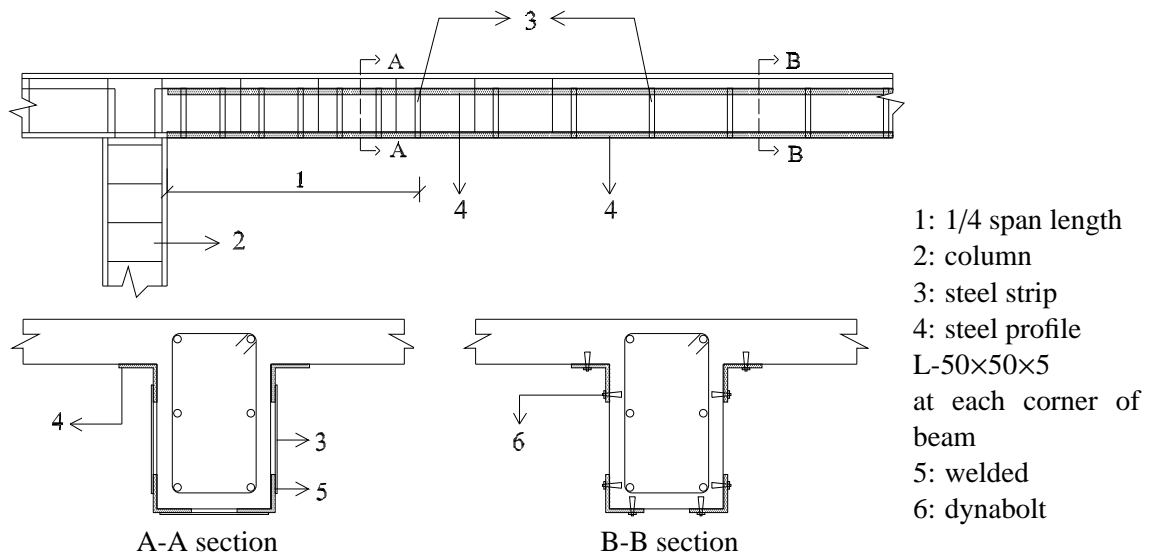


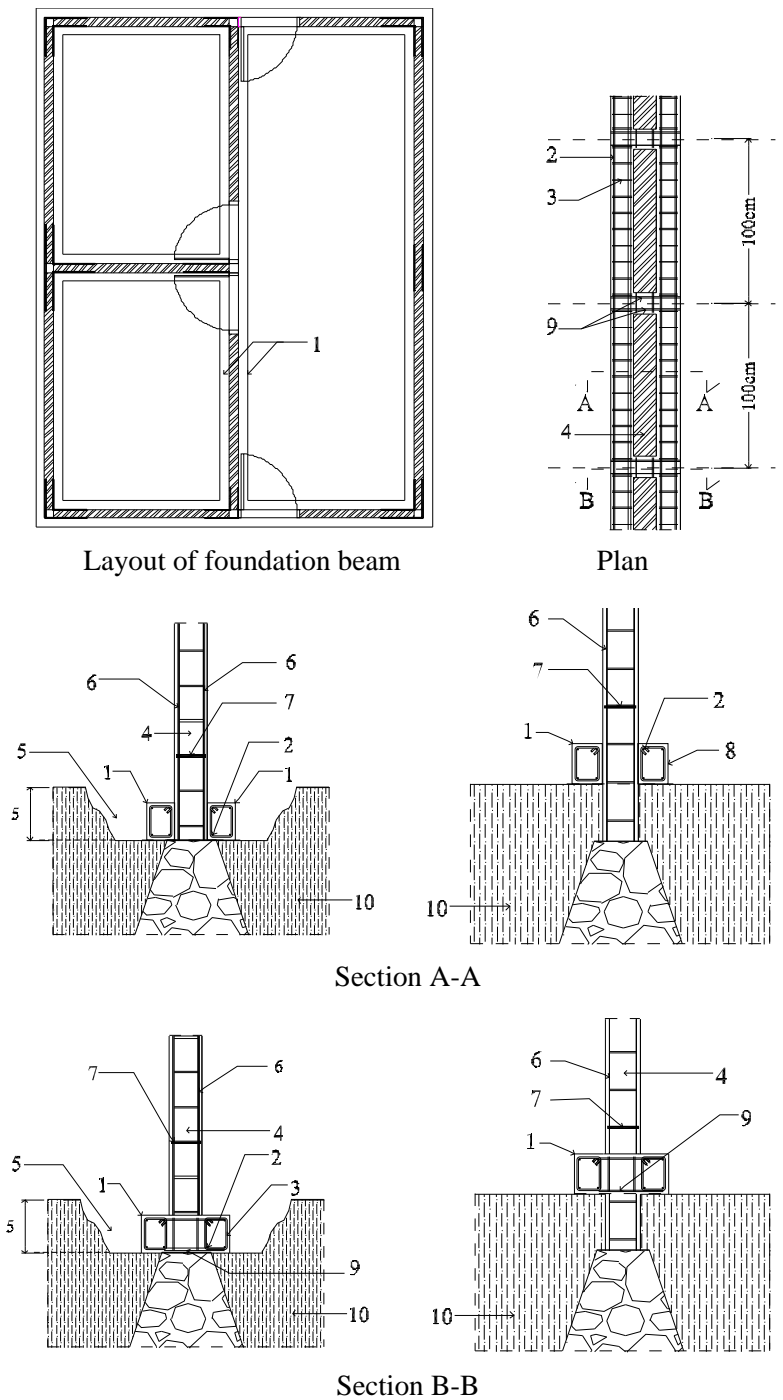
Figure 9.23: Increasing shear strength of existing beams

9.10 Strengthening of Foundations

Seismic strengthening of foundations before or after the earthquake is the most involved task since it may require careful underpinning operations. Some alternatives are given below for preliminary consideration of the strengthening scheme.

- 1) Introducing new load bearing members to the foundations to relieve the already overloaded members. Jacking operations may be needed in this process.
- 2) Improving the drainage of the area to prevent saturation of foundation soil to obviate any problems of liquefaction which may occur because of poor drainage.
- 3) Providing apron around the building to prevent soaking of foundation directly and draining off the water.
- 4) Adding elements in the form of reinforced concrete strips attached to the existing foundation part of the building. These will also bind the various wall footings and can be provided on both sides of the wall (see Fig. 9.24). To avoid digging the floor inside the building, the extra width could be provided only on the outside of external walls. The extra width can also be provided above the existing footing or at the level of the existing footing. The reinforced concrete strips and the walls have to be linked by a number of keys, inserted into the existing footing.

Note: To avoid disturbance to the integrity of the existing wall during the foundation strengthening process, proper investigation and design are called for.



1: foundation beam 15×20 cm, 2: reinforcing bars 4- ϕ 10 mm, 3: stirrups ϕ 8@150 mm, 4: wall, 5: foundation evacuation, depth min. 300 mm, 6: wire mesh min. ϕ 1 mm - 5×5 mm, 7: wire to tie wire mesh, 8: at door opening, the beam is staggered, 9: reinforcing bars 4- ϕ 10 mm every 100 cm, 10: ground

Figure 9.24: Improving a foundation by inserting lateral concrete beams

Chapter 10

Appendices

10.1 MSK Intensity Scale (as related to buildings)

The following definitions are used in the scale:

(a) Type of structures (buildings)

Structure A: Buildings in field-stone, rural structures, unburnt brick houses, clay houses.

Structure B: Ordinary brick buildings, buildings of the large block and prefabricated type, half timbered structures, buildings in natural hewn stone.

Structure C: Reinforced buildings, well built wooden structures.

(b) Definition of quantity

Single, few: About 5%

Many: About 50%

Most: About 75%

(c) Classification of damage to buildings

Grade 1 Slight damage: Fine cracks in plaster; fall of small pieces of plaster

Grade 2 Moderate damage: Small cracks in walls; fall of fairly large pieces of plaster, pantiles slip off; cracks in chimneys; parts of chimney fall down.

Grade 3 Heavy damage: Large and deep cracks in walls; fall of chimneys.

Grade 4 Destruction: Gaps in walls; parts of building may collapse; separate parts of the building lose their cohesion; and inner walls collapse.

Grade 5 Total damage: Total collapse of building.

(d) Intensity scale

I Not noticeable

II Scarcely noticeable (very slight)

III Weak, partially observed only

IV Largely observed

V Awakening

VI Frightening

Damage of Grade 1 is sustained in single buildings of Type B and in many of Type A. Damage in few buildings of Type A is of Grade 2.

VII Damage of buildings

In many buildings of Type C damage of Grade 1 is caused; in many buildings of Type B damage is of Grade 2. Most buildings of Type A suffer damage of Grade 3, few of Grade 4. In single instances landslips of roadway on steep slopes; cracks in roads; seams of pipelines damaged; cracks in stone walls.

VIII Destruction of buildings

Most buildings of Type C suffer damage of Grade 2, and few of Grade 3.

Most buildings of Type B suffer damage of Grade 3, and most buildings of Type A suffer damage of Grade 4.

Many buildings of Type C suffer damage of Grade 4. Occasional breaking of pipe seams. Memorials and monuments move and twist.

Tombstones overturn.

Stone walls collapse.

IX General damage to buildings

Many buildings of Type C suffer damage of Grade 3, and a few of Grade 4.

Many buildings of Type B show damage of Grade 4, and a few of Grade 5.

Many buildings of Type A suffer damage of Grade 5.

Monuments and columns fall.

Considerable damage to reservoirs; underground pipes partly broken.

In individual cases railway lines are bent and roadway damaged.

X General destruction of buildings

Many buildings of Type C suffer damage of Grade 4, and a few of Grade 5.

Many buildings of Type B show damage of Grade 5; most of Type A have destruction of Grade 5; critical damage to dams and dykes and severe damage to bridges.

Railway lines are bent slightly.

Underground pipes are broken or bent.

Road paving and asphalt show waves.

XI Destruction

Severe damage even to well built buildings, bridges, water dams and railway lines; highways become useless; underground pipes destroyed.

XII Landscape changes

Practically all structures above and below ground are greatly damaged or destroyed.

10.2 Assessment of Seismic Safety for Masonry Buildings

Seismic safety for masonry buildings can be assessed using Table 10.1.

If the building does not comply with the requirements, in order to save the building from collapse or severe damage, actions can be taken according to the number of the first column in the table as follows.

- 1 Extra upper stories may be removed.
- 2 For weaker building units, providing RC or steel columns may be considered.
- 3 For thinner walls pilasters of RC may be designed.
- 4 For weaker mortar case, providing RC columns & beams may be considered.

Note) In cases 2, 3 & 4 above, strengthening the walls by ferro-cement plating or by FRP may be an option.

- 5, 6 Provide buttresses at regular intervals for strengthening the wall.
- 7 a) Consider closing one of the windows or reduce the width of window openings by adding additional thickness of piers.
b), c) strengthening the piers by ferro-cement plating.
- 8 b) Use RC screed with peripheral RC beam in place of RC ceiling band.
c) Introduce cross ties with diagonal bracing underneath the floor.
d) Provide cross ties welded to the steel joists along with diagonal bracing.
- 9 b), c) Use diagonal bracing in the plane of the ties and the principal rafters in every fourth panel.
- 10 a) In case the plinth is high say 0.9 m or higher provide seismic belt.
b) to f) Provide seismic belt on the walls at the level where seismic band was required.
- 11 a) to d) Provide seismic belt or a vertical bars at the corners of the walls in the replacement of the required vertical reinforcement as per the code.
e), f) Strengthen the jambs with seismic belts.

10.3 Examples of Posters

These posters were developed for the reconstruction of houses in Aceh Province, Indonesia after the 2004 Sumatra Earthquake which created the tsunami, resulting in the death of more than 200,000 people.

Table 10.1: Assessment of seismic safety for a masonry building

	Data of the building	Requirements are complying ?	Tick if	
			Yes	No
1	Number of stories $n =$	$n \leq 4$		
2	Wall unit: BB / CB (solid) / CB (hollow) Comp. strength $f_c =$ (kg/cm ²)	$f_c \geq 50$ kg/cm ² $f_c \geq 35$ kg/cm ²		
3	Thickness of bearing wall External wall = (mm) Internal wall = (mm)	BB = 230 mm CB = 200 mm		
4	Mortar used =	C : S = 1 : 6 or richer		
5	Longest wall in room = (m)	BB \leq 8 m, CB \leq 7 m		
6	Height of wall (floor to ceiling) = (m)	BB \leq 3.45 m CB \leq 3.0 m		
7	Door/window openings (see Fig. 4.8 on p.39) a) Max. opening ratio = b) Min. pier width = (mm) b) Min. end corner distance = (m)	1 story: 0.50, 2 story: 0.42 3 or 4 story: 0.33 0.6 m 0.45 m		
8	Floor type a) RC or RB slab b) Precast elements c) Wood joist with tiles d) Jack arches	with RC screed with bracing with ties & bracing		
9	Roof type a) Flat like floor b) Sloping trussed c) Sloping raftered	with bracing with ties & bracing		
10	Seismic bands a) at plinth b) at lintel level c) at window sill level d) at ceiling level eave level e) at gable ends f) at ridge top	Required Required Required for 3 or 4 story Required for 8b), c) & 9b), c) Required for 9b), c) Required for 9b), c)		
11	Vertical bars a) at external corners b) at external T-junctions c) at internal corners d) at internal T-junctions e) at jambs of door f) at jambs of windows	Required for a) to f)		

Note : BB = burnt brick, CB = concrete block, C : S = cement : sand (volumn ratio),
RC = reinforced concrete, RB = reinforced brick

MINIMUM REQUIREMENT FOR EARTHQUAKE RESISTANT MASONRY BUILDING WITH REINFORCED CONCRETE STRUCTURES with MASONRY GABLE WALL

Roof Truss Details:

- Purlin 6/12 cm
- Wooden pin min ϕ 10 mm
- Iron sheet 4.40 mm or timber 20.100 mm
- Bolts min ϕ 10 mm

Wind Bracing: 6x12 cm

Gable Wall:

- Anchor min ϕ 10 mm length > 40 cm every 6 layers of brick
- Stirrups ϕ 8 mm spacing < 15 cm
- Reinforced concrete column 12x12 cm
- Reinforced concrete beam 12x20 cm

Foundation and Lintel Details:

- Lintel above window/door frame 12x20 cm
- Anchor min ϕ 10 mm length > 40 cm every 6 layers of brick
- Stirrups min ϕ 8 mm space < 15 cm
- Reinforcing min ϕ 10 mm
- Foundation beam 15x20 cm
- Mortar joint thickness 0,8-1,5 cm
- Concrete mix
- Rubble stone foundation

Reinforcement Details:

- Reinforcement min ϕ 10 mm
- Stirrups min ϕ 8 mm spacing < 15 cm
- Concrete cover 2,5 cm from center of main reinforcement bar.

Practical column	Practical beam	Foundation beam
12 cm x 12 cm	12 cm x 20 cm	15 cm x 20 cm
4 ϕ 10 mm	4 ϕ 10 mm	4 ϕ 10 mm
Min ϕ 8 mm spacing < 15 cm		

Mortar mix:

- 1 cement
- 4 sand
- Mix well with adequate water

Concrete mix:

- 1 cement
- 2 sand
- 3 gravel
- Minimum strength 150 kg/cm²

Gravel gradation: should be correct. Do not use excessive water.

Bricks soaked in water min 10 minutes before layed

Pay attention to:

1. Good quality building materials
2. Good workmanship
3. Seismic joint detailing
4. Wood should be treated for anti termite & fungus

Logos: USAID, YIPD, and other project partners.

Figure 10.1: Poster for Half Brick Confined Masonry

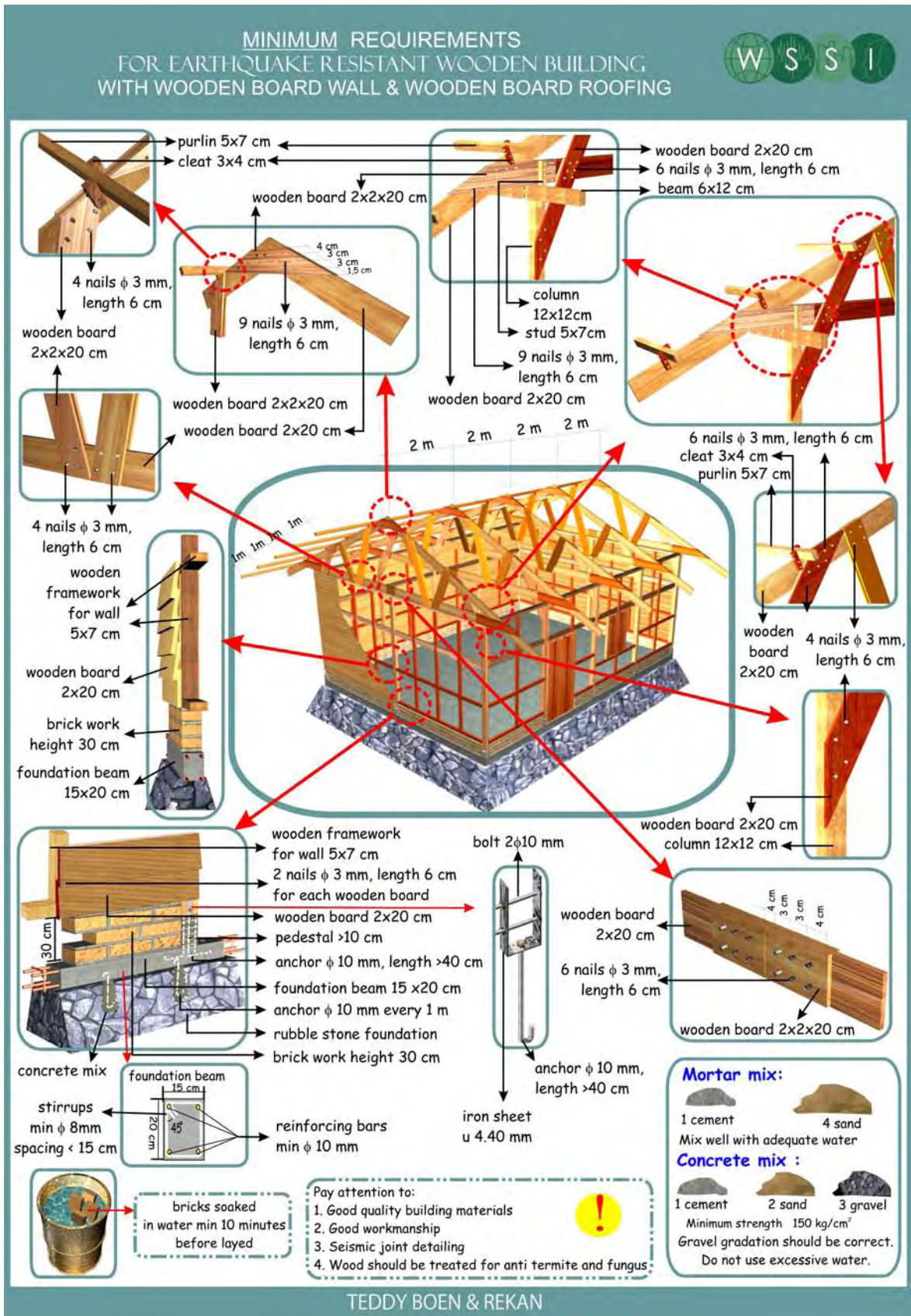


Figure 10.2: Poster for Wooden Buildings

Index

- adobe, 76
- aspect ratio, 13
- base-isolation, 29
- bearing wall construction, 27
- bond stone, 58
- building configuration, 11
- chemical anchor, 109
- classification of soil, 22
- compressive strength, 90
- confined masonry, 48
- confining element, 48
- construction joint, 98
- construction quality, 12
- damage category, 18
- deformability, 28
- design acceleration, 9
- design response spectrum, 9
- diagonal compression test, 91
- diagonal cracking, 94
- diaphragm action, 15
- differential settlement, 12, 34
- dry strength test, 77
- ductility, 11, 28
- enclosed area, 25
- equivalent lateral force, 8
- fire, 6
- fire resistance, 25
- fissuring control test, 77
- foundation, 12
- framed construction, 27
- free-standing masonry wall, 12
- gable band, 41
- gable triangle, 17
- ground failure, 4
- ground rupture, 4
- ground shaking, 4
- grouting, 121
- hammering, 24
- hard soil, 30
- hazard factor, 9
- horizontal band, 41, 45, 86
- importance of building, 22
- inertia force, 7
- integral unit, 3
- isolation, 29
- landslide, 4
- lintel band, 41, 86
- liquefaction, 5, 64
- masonry wall, 48
- mechanical anchor, 109
- medium soil, 30
- moment resisting frame, 27
- mortar, 38, 59
- MSK intensity scale, 130
- non-engineered building, 2
- occupancy category, 9
- occupancy importance, 9
- occupancy importance factor, 9
- opening size, 11
- openings in walls, 39, 60
- out-of-plane force, 12
- permissible compressive stress, 90
- permissible tensile strength, 91
- pilaster, 84
- rise, 76, 80
- plastering, 88
- plinth band, 41, 49

- polymer mortar, 108
- polymer resin, 108
- pounding, 24
- practical-column, 48
- prestressing, 123
- prism test, 90

- quick-setting cement mortar, 108

- rammed earth, 76
- regularity, 23
- repair, 106
- restoration, 106
- risk category, 9
- robustness, 28
- rocking, 12
- roof band, 41
- run-up height, 5

- seismic coefficient, 8
- seismic load, 7
- seismic sea wave, 5
- seismic zone, 21
- seismic zoning factor, 9
- separation, 24
- shear strength, 91
- shear wall, 13
- short column, 94
- shotcrete, 108
- sill masonry, 17
- simplicity, 24
- site condition, 10
- soft soil, 30
- spandrel masonry, 17
- stiffness distribution, 11
- strengthening, 107
- structural factor, 9
- structural framing, 27
- symmetry, 23

- tapial, 76, 80
- through stone, 12, 58, 60
- tie-beam, 48, 49
- tie-column, 48
- toothed joint, 40
- tsunami, 5

- vertical reinforcement, 46
- wall density, 52
- water table, 83