GUIDELINES FOR
EARTHQUAKE RESISTANT
NON-ENGINEERED CONSTRUCTION

Anand S. ARYA
Teddy BOEN
Yuji ISHIYAMA
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](image)
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**

1: earthquake motion
2: spandrel wall
3: cracks
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).

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

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

**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.
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.
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>
<thead>
<tr>
<th>Mortar mix</th>
<th>Tensile strength (MPa)*</th>
<th>Shearing strength (MPa)*</th>
<th>Compressive strength (MPa)* corresponding to crushing strength of masonry unit</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cement</td>
<td>Sand</td>
<td>0.04</td>
<td>0.22</td>
</tr>
<tr>
<td>1 12</td>
<td>0.25</td>
<td>0.39</td>
<td>2.1</td>
</tr>
<tr>
<td>1 6</td>
<td>0.71</td>
<td>1.04</td>
<td>2.4</td>
</tr>
</tbody>
</table>

* 1 MPa = 1N/mm² = 10 kgf/cm²

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**

<table>
<thead>
<tr>
<th>Mortar mix or equivalent</th>
<th>Permissible stresses</th>
<th>Compressive strength (MPa)* of unit</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cement</td>
<td>Lime</td>
<td>Sand</td>
</tr>
<tr>
<td>1</td>
<td>–</td>
<td>6</td>
</tr>
<tr>
<td>1</td>
<td>1</td>
<td>6</td>
</tr>
<tr>
<td>1</td>
<td>–</td>
<td>3</td>
</tr>
</tbody>
</table>

*1 MPa = 1N/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 cementsand 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.
4. Masonry Buildings in Fired-Brick and Other Materials

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

<table>
<thead>
<tr>
<th>Slenderness ratio</th>
<th>Stress factor k for eccentricity ratio $e/t$</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>0</td>
</tr>
<tr>
<td>6</td>
<td>1.00</td>
</tr>
<tr>
<td>8</td>
<td>0.920</td>
</tr>
<tr>
<td>10</td>
<td>0.840</td>
</tr>
<tr>
<td>12</td>
<td>0.760</td>
</tr>
<tr>
<td>14</td>
<td>0.670</td>
</tr>
<tr>
<td>16</td>
<td>0.580</td>
</tr>
<tr>
<td>18</td>
<td>0.500</td>
</tr>
<tr>
<td>21</td>
<td>0.470</td>
</tr>
<tr>
<td>24</td>
<td>0.440</td>
</tr>
</tbody>
</table>

(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

<table>
<thead>
<tr>
<th>Category*</th>
<th>Proportion of cement-sand or cement-lime-sand**</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Cement-sand</td>
</tr>
<tr>
<td>I</td>
<td>“1 : 4” or richer</td>
</tr>
<tr>
<td>II</td>
<td>“1 : 5” or richer</td>
</tr>
<tr>
<td>III</td>
<td>“1 : 6” or richer</td>
</tr>
<tr>
<td>IV</td>
<td>“1 : 7” or richer</td>
</tr>
</tbody>
</table>

* 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 20 t and wall length between cross-walls not more than 40 t. If longer rooms are required, either the wall thickness is to be increased, or full height buttresses should be provided at 20 t 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.

Figure 4.8: Recommendations regarding openings in bearing walls

- C: cross wall
- D: door opening
- V: ventilation
- W: window opening

\[
\begin{align*}
\frac{b_1 + b_2 + b_3}{H_1} &\leq 0.5L_1 \text{ for one storey, } 0.42 \text{ for two storey, } 0.33 \text{ for three storey,} \\
\frac{b_6 + b_7}{H_2} &\leq 0.5L_2 \text{ for one storey, } 0.42 \text{ for two storey, } 0.33 \text{ for three storey,} \\
\frac{h_3}{H_1} &\geq 0.5h_1 \text{ but not less than } 0.6 \text{ m, } \\
\frac{h_5}{H_2} &\geq 0.25h_1 \text{ but not less than } 0.6 \text{ m,} \\
h_3 &\geq 0.6 \text{ m or } 0.5 (h_5 \text{ or } b_3 \text{ whichever is more}).
\end{align*}
\]
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).

**Figure 4.9: Strengthening of masonry walls around openings**

**a) Strengthening around openings**

**b) Section at A-A**

**c) Detail of B**

L : brick length
LB : lintel band
t : wall thickness (without finishing) ≥ L

<table>
<thead>
<tr>
<th>0.5 L</th>
<th>0.5 L</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td>0.75 L</td>
<td>L</td>
</tr>
</tbody>
</table>

Vertical and horizontal reinforcement around the opening should be minimum 8 mm in diameter.
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 roofeave 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).
Figure 4.11: Gable band and roof band in barrack type buildings

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

Notes:
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.

Table 4.5: Recommendation for steel in RC band

<table>
<thead>
<tr>
<th>Span (m)</th>
<th>Category I</th>
<th>Category I</th>
<th>Category III</th>
<th>Category IV</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Longitudinal steel in RC bands</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>No of bars</td>
<td>Dia of bars (mm)</td>
<td>No of bars</td>
<td>Dia of bars (mm)</td>
</tr>
<tr>
<td>5</td>
<td>2</td>
<td>12</td>
<td>2</td>
<td>10</td>
</tr>
<tr>
<td>6</td>
<td>2</td>
<td>16</td>
<td>2</td>
<td>12</td>
</tr>
<tr>
<td>7</td>
<td>2</td>
<td>16</td>
<td>2</td>
<td>16</td>
</tr>
<tr>
<td>8</td>
<td>4</td>
<td>12</td>
<td>2</td>
<td>16</td>
</tr>
<tr>
<td>9</td>
<td>4</td>
<td>16</td>
<td>4</td>
<td>12</td>
</tr>
</tbody>
</table>
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.

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 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.
Figure 4.13: Strengthening by dowel or wire fabric

(t1, t2: wall thickness, 1: cross links, 2: thick joint to receive dowels)

(a) [Diagram showing strengthening by dowel or wire fabric]

(b) [Diagram showing strengthening by dowel or wire fabric]

(c) [Diagram showing strengthening by dowel or wire fabric]

(d) [Diagram showing strengthening by dowel or wire fabric]

j: construction joint
m: wire mesh
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.

**Table 4.6: Recommendation for vertical steel at critical sections**

<table>
<thead>
<tr>
<th>No of Storey</th>
<th>Storeys</th>
<th>Diameter of mild steel single bar in mm at each critical section for Category*</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Category I</td>
</tr>
<tr>
<td>One</td>
<td></td>
<td>16</td>
</tr>
<tr>
<td>Two</td>
<td>Top Bottom</td>
<td>16</td>
</tr>
<tr>
<td></td>
<td></td>
<td>20</td>
</tr>
<tr>
<td>Three</td>
<td>Top Middle Bottom</td>
<td>16</td>
</tr>
<tr>
<td></td>
<td>Bottom</td>
<td>20</td>
</tr>
<tr>
<td>Four</td>
<td>Top Third Second Bottom</td>
<td>**</td>
</tr>
<tr>
<td></td>
<td>Bottom</td>
<td>**</td>
</tr>
<tr>
<td></td>
<td></td>
<td>16</td>
</tr>
</tbody>
</table>

* 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).

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.
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 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

Figure 4.14: Vertical reinforcement (V) in walls

<table>
<thead>
<tr>
<th>a) Corner junction for one (1) brick wall</th>
<th>b) Corner junction for one and half (1.1/2) brick wall</th>
<th>c) T-junction for one and half (1.1/2) brick wall</th>
</tr>
</thead>
<tbody>
<tr>
<td><img src="image_url" alt="Diagram" /></td>
<td><img src="image_url" alt="Diagram" /></td>
<td><img src="image_url" alt="Diagram" /></td>
</tr>
</tbody>
</table>

![Diagram](image_url)
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.

**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 microconcrete “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**

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### 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).

**Figure 4.17: Blocks for vertical bars**

- **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.
– **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.

**Figure 4.18: Confined brick masonry**

- Single storey house (Ache, Indonesia)
- Two storey house under construction (Jawa, Indonesia)
– **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 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.

**Figure 4.22: RC frame and confined masonry construction**

**a) RC frame construction**

1: Footing
2: Column
3: Infill wall panel
4: Beam

**b) Confined masonry construction**

1: Foundation
2: Plinth level tie beam
3: Tie column
4: Wall panel
5: Tie beam
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

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:

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

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 $\approx 40$ 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](image)

1: half wall height
2: tie column bars
3: brace 4.6 cm nailed to formwork
4: timber bracing
5: formwork half wall height
6: stake 4.6 cm

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.
Figure 4.24: Single storey confined masonry house

1: lintel 12x20 cm, also as horizontal reinforcement
2: horizontal bracing 8x12 cm
3: collar beam 12x20 cm
4: gable wall reinforcement
5: anchors, min. $\phi 10$ mm, min. length 40 d for each 6 layers of brick
6: column 12x12 cm
7: foundation beam 15x20 cm

Figure 4.25: Recommended minimum reinforcing bars

1: column reinforcing bar min. 4-$\phi 10$ mm
2: anchor min. $\phi 10$ mm, length > 40d every 6 layers of bricks
3: stirrup or hoop $\phi 8$ mm, distance < 150 mm
4: beam reinforcing bar min. 4-$\phi 10$ mm
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**

1: Plinth/ groundlevel beam
2: Steel bar from foundation to plinth beam
3: Brick pedestal
4: Plain cement concrete
5: One-brick flat layer
6: 150 mm thick sand filling
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.

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 halfdressed 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 intensifies 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.

- 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

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**Figure 5.2: Schematic cross section through a traditional stone house**

1: Stone wall with mud mortar,
2: Mud fill at roof and floor 150 to 300 mm thick,
3: Branches, reeds
4: Log beams,
5: Hammer crossed face,
6: Chip and mud filling,
7: Random rubble,
H: Wall height 3 to 4 m
t: Wall thickness 0.5 to 0.9 m
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.

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).
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 450x450 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 \( \phi \) 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).