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DESIGN AND CONSTRUCTION OF MASONRY BUILDINGS IN SEISMIC AREAS

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Introduction

In the past severe earthquakes all over the world, the masonry buildings have generally been damaged the most because of their heavy weight, small or no tensile strength, small shearing resistance, lack of proper bonding between longitudinal and cross walls and poor-workmanship. Yet such buildings are being put up in the conventional way for reasons of climatic suitability, cheapness and local availability of materials, widespread knowledge of methods of construction etc. It is quite apparent that it will not be possible to do away with this kind of construction in the seismic areas particularly in the developing countries. Therefore finding effective methods of strenthening such buildings is of paramount importance so that the danger to life and property during future earthquakes is minimised. An important requirement of such methods is that they should be cheap so that the additional cost of strengthening is not more than what one would like to spend on insurance.

For closer examination a building is dissected into various components to establish where each component lacks strength against earthquake forces and suggest means to strengthen them. The discussion here is based on brick buildings in India not taller than four storeys. The conclusions can easily be interpreted for similar construction in other countries. It is assumed that the bricks are not weaker than the building mortar in tensile, shear and compressive strengths.

Structural Action

During ground motion, an inertia force acts on the mass of the building due to which the buildings tend to remain stationary while the ground moves. If the building is rigidly fixed into ground, the inertia forces will cause horizontal shears in the building the magnitude of which will be a function of the ground motion and the stiffness and damping characteristics of the building. However, if relative motion is possible between the ground and the building, the building may slide to and fro and the forces generated, during the earthquake, which would tend to shear the building, would be small. This type of action may be visualised if the ground under a railway wagon is imagined to shake along the direction of rails. It has clearly been demonstrated in the 1930 Dhubri (Assam) earthquake and the 1934 Bihar-Nepal earthquake that where a relative displacement was possible between the ground and the superstructure, the building suffered less than other similar buildings fixed into the ground.

As a result of the shaking, the roof tends to separate from the supports, the roof covering tends to be dislodged, walls tend to tear apart and if unable to do so the walls tend to shear off diagonally in the direction of motion. If filler walls are used within steel, concrete or timber framing, they may fall out of the frame bodily unless properly tied to the framing members. Thus on the whole it is indicated that a necessary condition for earthquake resistance is that the various parts of the building should be adequately tied together.

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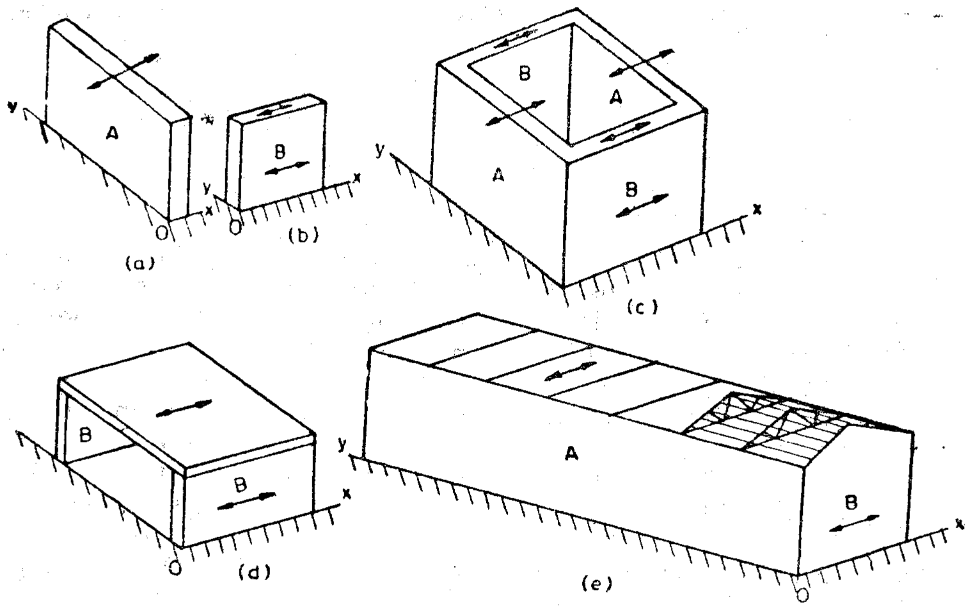

Fig. 1

Consider the structural elements and structures shown in Figure 1. In (a), wall A is free standing and the ground motion is acting transverse to it. The force acting on the mass of the wall tends to overturn it. The resistance of the wall is obviously very small. In (b), the free standing wall B is subjected to ground motion in its own plane. It is clear that in this case, because of its large depth in the plane of bending, the wall will offer great resistance. Such a wall is termed as shear well. It may be noted that for ground motion parallel to y-axis, wall A will act as shear wall and wall B will topple over. Now consider the combination of walls A and B as shown in (c). For the x-direction of motion as shown, walls B act as shear walls and besides themselves they offer resistance against the collapse of walls A as well. Walls A now act as vertical slabs supported on two vertical sides and bottom and subjected to the inertia force on their own mass. Near the vertical edges, the wall will carry bending moments in the horizontal plane for which the masonry has little strength. Consequently cracking and separation of the walls may occur. If however a horizontal bending member (like a timber beam or reinforced concrete or reinforced brick runner) were inserted at a suitable level in wall A and continued in wall B, this will take care of the bending tensions in the horizontal plane. So far as vertical bending is concerned, the wall gets aprecompression due to self weight and can be made to take care of bending tensions The situation will be the same for walls B for ground motion along y-axis. Thus the bending member will be required in walls B also. Therefore a horizontal runner, strong in bending, is required alround the enclosure. Such a runner is termed as band and depending upon at which level it is used, it is called **roof band, lintel band, gable band or plinth band.**

In Figure 1(d), a roof slab is resting on walls B and earthquake is along the x-axis. Assuming that there is enough adhesion between the slab and the walls, the slab transfers its inertia force at the top of walls B, causing shearing and overturning actions in them. To be able to transfer its force on to the walls, the slab must have enough strength in bending in the horizontal plane. Whereas R. C. or R. B. slabs shall possess such strength inherently, other types of roofs or floors such as timber joists with timber plank or brick tile covering will have to be connected together and fixed to the walls suitably so that they are able to transfer their inertia force to the walls This point is discussed in greater detail later. After this load transfer, the walls B must have enough strength as shear walls to withstand the applied inertia force and its own inertia force It is quite clear that the structure shown at (d), when subjected to ground motion along y-axis, will collapse very easily because walls B have very little bending resistance in the y-direction. Hence bracing arrangement, either by shear walls or by diagonal braces, must be provided in the y-direction.

Lastly, consider the barrack type structure carrying roof trusses as shown at (e). The trusses rest on walls A and the walls B are gabled to receive the purlins of the end bays. The ground motion is along x-axis. The inertia forces will be transmitted from sheeting to purlins, trusses and from trusses to walls A. The end purlins will transmit some force directly to gable ends. In order that the structure does not collapse, the following arrangements must be made :

(i) the trusses must be anchored into the walls by holding down bolts,

(ii) walls A, which do not get much support from the walls B in this case, must be made strong enough in vertical bending as cantilever, or

(iii) some suitable arrangement must be made to transmit the force horizontally to end walls B which are well suited to resist the force by shear wall action.

Strengthening of walls A in vertical direction may be done by providing adequate pillasters under the roof trusses. For horizontal transfer of forces, a band may be provided at top of wall to which the roof trusses are conneted, or diagonal bracing may be provided in plan at the level of main ties of the trusses which should extend from one gable end to the other and be suitably connected to the end walls B. The inertia force of top half height of walls A may be assumed to be resisted by such a band or bracing system.

Now if ground motion is along y-direction, walls A will be in a positon to act as shear walls and all forces may be transmitted to them. In this case, the purlins act as ties and struts and transfer the inertia force of roof to the gable ends. A sloping band is required at the top of gable ends to transfer this inertia force from gable ends to the walls A where this band joins roof band at the top of walls A.

The above discussion leads to the following requirements for structural safety of masonry buildings during ground motions:

1. A free standing wall must be designed as vertical cantilever.

2. A shear wall must be capable of resisting all horizontal forces due to its own mass and those transmitted to it.

3. The roof or floor elements must be tied together and be capable of transferring their inertia force to shear walls by bending in horizontal plane.

4. The trusses must be anchored to supporting walls and have an arrangement of transfering their inertia force to end walls.

5. The shear walls must be present along both the axes of the building.

6. The walls must be effectively tied together to avoid separation at vertical joints due to shaking.

Horizonthal bands may be provided for tying the walls together and transferring the inertia load horizontally to the shear walls.

Design of Bands

As stated above, the bands are required for tying the walls together and imparting horizontal bending strength to them. The forces, which the bands are subjected to, are usually indeterminate. In most cases arbitrary dimensions are adopted. An approximate method is suggested here for arriving at minimum dimensions of the band at any level. This is explained by the following numerical example.

Example 1

Figure 2 shows a single room building with its roof removed. The walls are 20 cm thick in modular bricks built in 1 : 6 cement sand mortar. The roof, which is other than R. C. or R. B. slab, weighs 600kg/m^2. The bands are required for earthquake coefficient of 0.16.

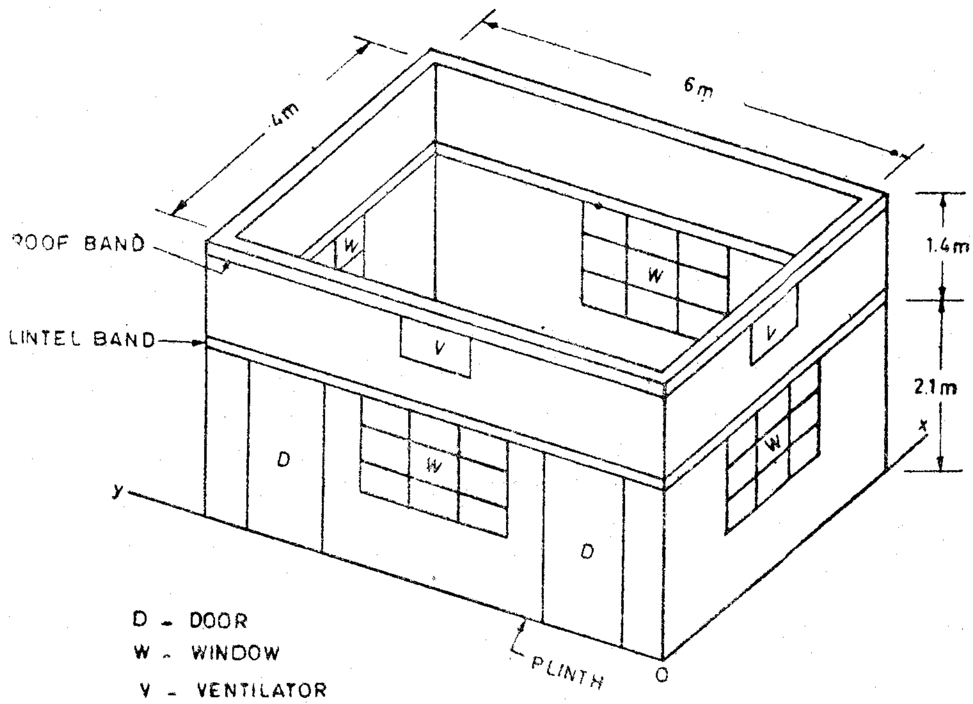

Fig. 2

Considering the door and window openings, two bands will be suitable, one at lintel and the other at roof level. A band may be necessary at plinth level also under certain conditions. Let the earthquake force act along x-axis. The lintel band divides the long wall in two portions, the lower portion spanning between plinth and lintel band and the upper portion spanning between lintel band and roof band.

Taking weight of masonry 1920 kg/m^3, weight of 20 cm wall is 384 kg/m^2.

Checking Vertical Bending of Wall

Vertical span is 2.1 m with flat ends.

Moment $= 0.16 \times 384 \times 2.1^2/12 = 22.6$ kgm/m

$$\text{Stress} = \frac{22.50 \times 100}{100 \times 20^2/6} = \pm\, 0.338 \text{ kg/cm}^2$$

Weight of wall and roof per m approximately

$$= 384 \times 1.4 + \frac{600 \times 6\ 4 \times 4.4}{2\,(6.2 + 4 \cdot 2)}$$

$$= 1350 \text{ kg.}$$

$$\text{Stress} = \frac{1350}{100 \times 20} = 0.675 \text{ kg/cm}^2$$

Combined stresses $= 0.675 \pm 0.338 = 1.013, 0.337$ kg/cm^2 (compressive)

Therefore the wall is safe.

Lintel Band

Neglecting openings, horizontal load on lintel band

$$q_h = 0.16 \times \frac{21 + 1.4}{2} \times 384 = 107 \text{ kg/m}$$

Assuming continuity of band at corners, bending moment in band

$$M = 107 \times 6.2^2/10 = 411 \text{ kgm}$$

$$F = 107 \times 6/2 = 321 \text{ kg}$$

Taking the following stresses for design,

allowable bending compression in concrete	50 kg/cm²
allowable shearing stress in concrete	5 kg/cm²
allowable tensile stress in steel	1400 kg/cm²
modular ratio	18

permissible increase in stress due to earthquake $33\frac{1}{3}\%$ and taking the compression steel into account (because reinforcement is required on both faces due to reversible nature of force) or using steel beam theory, the area of steel on each face

$$A_t = \frac{41100}{1.333 \times 1400 \times 15} = 1.471 \text{ cm}^2$$

The thickness of the band may be determined from the consideration of diagonal tension. Limiting the shear stress to the allowable value, thickness

$$b = \frac{321}{1.333 \times 5 \times 0.867 \times 17.5} = 3.17$$

A minimum thickness of 7.5 cm may be adopted. One bar 14mm dia may be used on each face. Links consisting of 6 mm dia with hooks at both ends may be used @ 15 cm/cc to keep longitudinal bars in position. Figure 3 show the reinforcement detail near a corner.

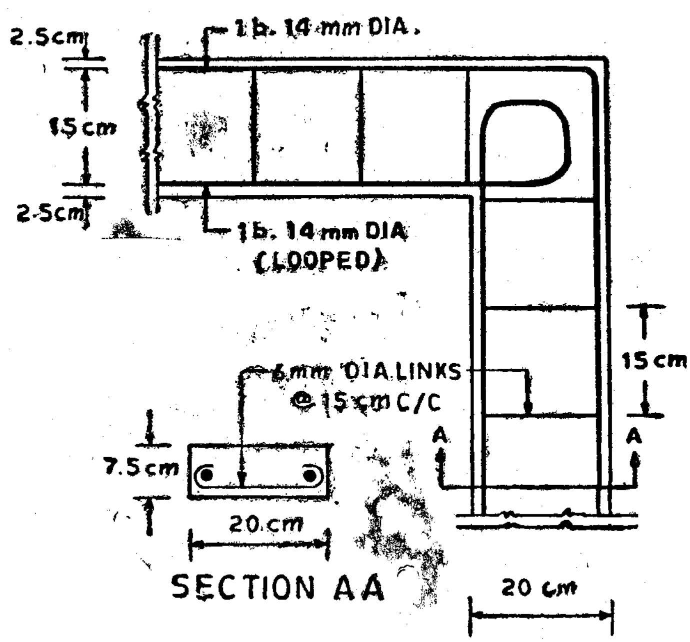

Fig. 3

As an alternative to the reinforced concrete, reinforced brickwork may be used. In the present case, the steel requirement is the same. Two 10 mm dia bars may be located in two consecutive motar courses on each face. The motar must be 1:3 cement sand mortar, its thickness being such as to provide at least 6mm cover to the bar, say 25 mm in the present case. Figure 4 shows the cross section and the arrangement of bending in plan.

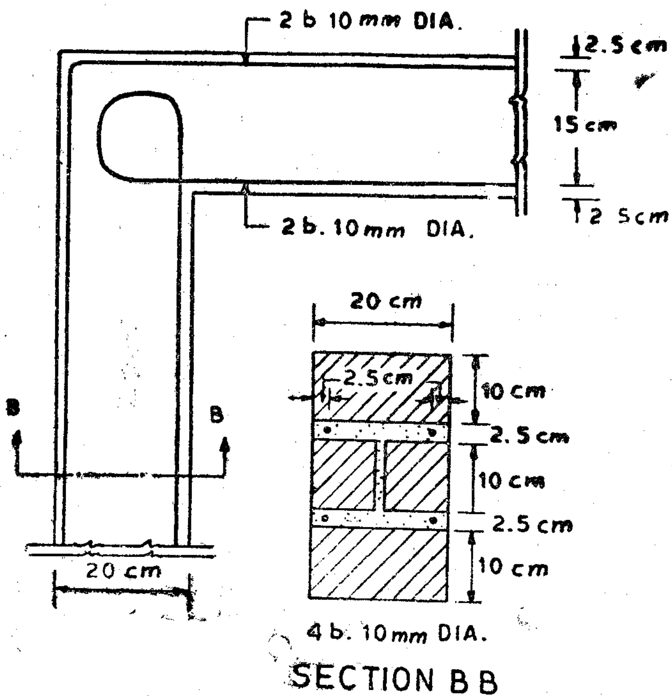

Fig. 4

Roof Band

The inertia force due to load as coming on the roof band depends on the arrangement of roofing elements. Let us assume that the roofing elements are connected with the long walls. Inertia load on roof band

$$q_h = 0.16\,(600 \times 4.4/2 + 384 \times 1.4/2) = 254 \text{ kg/m}$$

As for lintel band the section of 7.5 cm $\times$ 20 cm may still be maintained. In this case two bars 22 mm dia, one on each face, will be found to be sufficient.

Example 2

A barrack, 15 m long between end walls and 6 m wide carries A C. sheet roofing over timber trusses. The roof weighs 100 kg/m^2 of horizontal area. The walls are 30 cm thick and 4 m tall above plinth, the top level of door and window openings being 2.5 m. The strengthening is required against an earthquake coeffecient of 0.107.

Anchorage

Holding down bolts as required for wind shall suffice.

Roof Band

Span	$= 15 + 0.30$	$= 15.3$ m
Load from roof	$= 0.107 \times 100 \times 6.6/2$	$= 35.4$ kg/m
Load from wall	$= 0.107 \times 1920 \times 0.30\,(4.0-2.5)/2$	$= 46.1$ kg/m
Total load	$= 81.5$ kg/m	
M	$= \dfrac{81.5 \times 15.3^2}{10} = 1905$ kg/m	
F	$= 81.5 \times 15/2 = 611$ kg	

Proceeding as in Example 1, a 10 cm $\times$ 30 cm band in M 150 concrete is sufficient when reinforced with 4 bars 16 mm dia, one at each corner and 6 mm diameter stirrups 25 cm apart.

Horizontal Bracing

Alternative to the roof band, horizontal bracing system may be used. Let the trusses be 3 m apart and the main tie be divided into 3 panel lengths of 2.1 m each. Let a horizontal truss be provided as shown in Figure 5, which will be able to take the horizontal force from roof as well as both the longitudinal walls. In large halls such trusses may be provided along both the long walls.

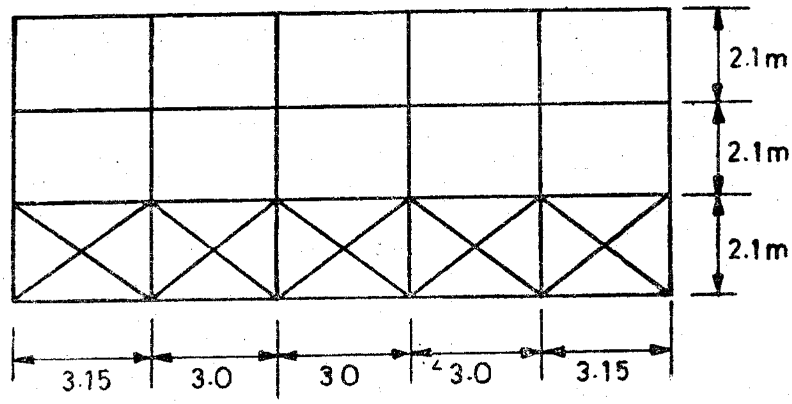

Fig. 5

Panel point load $= (35.4 + 2 \times 46.1) \times 3 = 383$ kg.

Maximum shear in end panel $= 383 \times 2 = 766$ kg.

Maximum force in tension diagonal assuming the compression diagonal to buckle

$$= \frac{766}{2.1}\sqrt{3.15^2+2.1^2} = 1380 \text{ kg.}$$

Use may be made of 12 mm dia. rods.

Max. force in chord member $= (1/2.1)(766 \times 6.15 - 383 \times 3.0)$

$= 1700$

For timber trusses, the members may be of wood and for steel trusses, of steel, designed for a force of 1700 kg as usual.

Maximum force in end members $= 766 + 383/4$

$= 862$ kg.

The members at end should normally be R.C. or R.B. so as to continue around the walls as a binding member in the shear walls.

Lintel band may be designed as in Example 1.

Free Standing Walls

Free standing walls, like compound walls, may be designed in two ways. If the mortar has doubtful or no tensile strength, the design must be made for no-overturning. It may be shown that with a factor of safety of 1.5, the minimum thickness of such a wall b is given by

$$b = 1.5\,C.h \tag{1}$$

where C is the seismic coefficient and h is the height of the wall.

But when the mortar used is believed to have a known tensile strength, the minimum thickness of wall may be arrived at by using a factor of safety of 2.0. For brick work in 1:6 cement mortar, tensile stress of 1.0 kg/cm^2 may be permitted for design against earthquake forces. It can be derived that in this case minimum value of b will be given by

$$b = \frac{3C\,w\,h^2}{f + wh} \tag{2}$$

where w is the unit weight of masonry and f is the allowable tension.

Example 3

Given h = 160 cm, C = 0.12, w = 0.00192 kg/cm^3, f = zero or 1.0 kg/cm^2, the thickness of free standing wall is required.

For no tensile strength, using Eq. (1)

$$b = 1.5 \times 0.12 \times 160 = 28.8 \text{ cm, say 30 cm.}$$

For given tensile strength, using Eq. (2)

$$b = \frac{3 \times 0.12 \times .00192 \times 160^2}{1.0 + .00192 \times 160}$$

$$= 18 \text{ cm, say 20 cm.}$$

Partition Walls

Non-bearing partitions should also be checked for stability against earthquakes because their collapse may result in damage to property and life. They should in general be built into the bearing walls and be stayed against deflection at top as far as possible. If the top has to be kept free, horizontal steel bars may be inserted below the top course, one on each face and anchored into the bearing walls at ends. This would act as a sort of band. The stability of wall may then be checked in bending as a vertical slab.

Roofs and Floors

The more common types of roofs and floors used in conjunction with masonry are the following :

i) Roof trusses,
ii) Jack arches,
iii) Slab or slab and beams (reinforced concrete, reinforced brickwork of cast in situ construction)
iv) Joists (timber, reinforced concrete, presstressed concrete, precast slab units, stone slabs)

A general distinction is to be made among them. Those which are flat and are bonded or tied to the masonry, have a binding effect on the walls and do not require extra tying arrangement in the form of a band for the walls at roof or floor level. The examples are 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 mortar like 1 : 6 cement sand mortar or equivalent in tensile strength. Others which simply rest on the masonry walls will offer resistance to relative motion through friction only which may not be relied upon. In such cases, other positive means are to be taken to tie the roofing and flooring elements together, to fix them to the walls and also tying the walls together.

To illustrate the point, let us consider the case of a timber joints floor of conventional construction in which the joints about 7.5 cm wide by 10 cm deep are placed across spans of about 3 m at a centre to centre spacing of 25 to 30 cm. These are then covered by timber planks nailed to the joints or by brick tiles placed directly over the joists. Clayee earth is then laid for water proofing and the roof or floor may finally be finished as desired. When timber planks are used, they act as effective ties to bind the joists together. But brick tiles have no such binding effect. In fact during an earthquake a small relative displacement of the joists will be enough to bring down the tiles damaging life and property in the room. In such cases the joists must be tied togetherat fix spacing apart so that the tiles above them may not be dislodged. This may be achieved by blocking the space between the joints by means of timber blocks having the same depth as the joists and fixed to them by nailing. Besides blocking at the ends, cross-bridging planks may be nailed to the joists at mid-span.

In order to fix the so formed joist-grill to the walls, a timber runner may be used on the wall under the grill to which the joints may be spiked. The timber runners will also be capable of tying the walls together if they are sufficiently large in section and full length pieces are used in all walls and jointed together firmly at their junctions. In that case, the timer runners may be built into the walls. If separate tying arrangement is desired for the walls an R.B. or R.C. band may be provided between the wall and roof or floor. The design of such a band has been illustrated in Example 1 before.

In the case of other kinds of elements, suitable details may be worked out for connecting the units together and fixing them to the band. For example for precast R.C. planks iron straps and bolts, welding the protruding reinforcement, laying cast-in-situ concrete, etc. are some of the devices that may be used for the purpose.

Design of Shear Walls

As has been stated earlier, shear walls are the main members for transferring all earthquake forces to the foundations. Tests have indicated that the factors determining their strength are too many and varied. The greatest source of error and uncertainty is the workmanship. Any effort to use the theory of elasticity for analyzing the stresses in brick or stone masonry therefore prove futile. Hence, the simplest available approach for their analysis should suffice. One such procedure for shear walls is explained below :

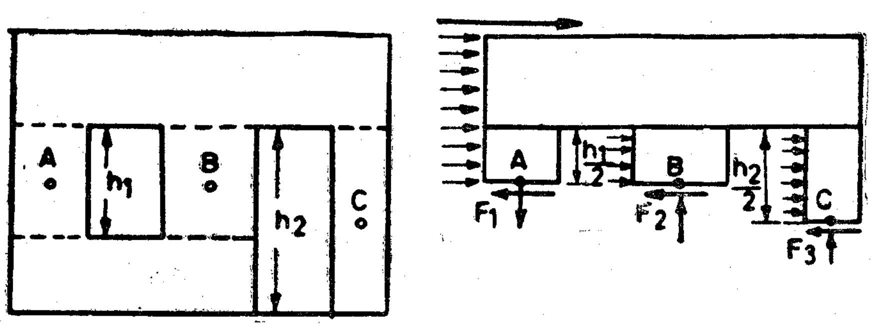

Fig. 6

Let masonry wall have openings as shown in Figure 6 and carry horizontal inertia forces due to the weight of roof and wall as indicated. For analyzing the stresses in such walls, it is assumed that the rotational deformations of the portions above and below the openings are much smaller than those of the piers between the openings and are neglected. Points of contraflexure are assumed at the mid-points of piers and shears are assumed to be shared among the piers such that their tops deflect by the same amount.

For calculating the deflections, the piers are assumed to be restrained against rotation at the ends but as free to deflect; and shear deformations are also considered. Thus if the horizontal shear in a pier is H, the deflection at its top is given by

$$\triangle = \frac{H\,h^3}{12\,EI} + \frac{1.2\,H\,h}{GA} \tag{3}$$

when h is the height of the pier, I its moment of inertia about the axis of bending, E the modulus of elasticity, G the modulus of rigidity and 1.2 is a factor required in the calculation of shear deflection of a member having a rectangular section. The shear stiffness of a pier may be defined 'as the load required to deflect its top by unit deflection'. Hence the shear stiffness is given by

$$S = \frac{H}{\Delta} = \frac{1}{\frac{h^3}{12\,EI} + \frac{1.2\,h}{GA}} = \frac{\frac{12\,EI}{h^3}}{1 + \left(14.4\,\frac{EI}{GA\,h^2}\right)} \tag{4}$$

If the thickness of the wall is b and the width of pier is d, then expressing I and A in their terms and taking $G = E/2$ for masonry,

$$S = \frac{12\,EI}{h^3} \cdot \frac{1}{1 + 2.4\,(d/h)^2} \tag{5}$$

The total horizontal shear in the piers may be distributed between them in the ratios of their shear stiffnesses. Let the shears produced be F_1, F_2 etc. Then the piers carry shears F, and bending moments $\frac{Fh}{2}$ at their top and bottom sections. The resulting values of maximum shear stress, q, and bending stress, P_b in any pier will be given by

$$q = \frac{1.5\,F}{bd} \tag{6}$$

$$P_b = \frac{3Fh}{bd^2} \tag{7}$$

The total weight of the structure above the top of piers W may be assumed to produce uniform direct stress P_0 in them. Thus

$$P_0 = W/\Sigma A \tag{8}$$

where ΣA represents the sum of areas of all the piers in the plan of the building at any level.

The overturning forces are determined as follows:

Let the areas of the piers be A_1, A_2 and A_3 (Fig. 7) and the combined centre of gravity of the three areas be at G. Assuming the stress in the piers to be proportional to the distance from G, the forces in the three piers are ;—

$-K\,x_1\,A_1$, $-K\,x_2\,A_2$ and $K\,x_3\,A_3$, where K is the coefficient of proportionality, and x_1, x_2 and x_3 are the distances of the points considered (in the three piers) from G. If M is the moment of the horizontal forces about A, we get,

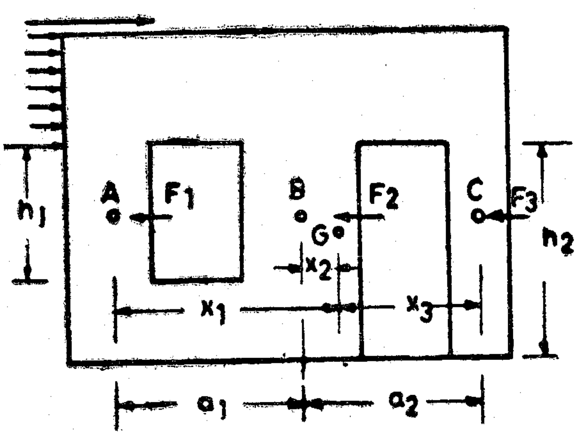

Fig. 7

$$K = \frac{M + F_3\left(\frac{h_2 - h_1}{2}\right)}{A_1 x_1^2 + A_2 x_2^2 + A_3 x_3^2} \tag{9}$$

Knowing the constant of proportionality K the stresses due to overturning, p_t, may be found as follows :

$$\begin{aligned} &- Kx_1 \text{ in pier A} \\ &- Kx_2 \text{ in pier B} \\ &+ Kx_3 \text{ in pier C} \end{aligned} \tag{12}$$

Combining all the stresses, the maximum and minimum stresses on the cross section of each pier can be determined. That is, the resultant total stress p is given by

$$p = p_b + p_o + p_t \tag{11}$$

which may be positive or negative depending upon the relative magnitudes and signs of the component stresses. Such calculations are to be made along each axis of the building for reversible earthquake force. If the stress becomes tensile, vertical steel reinforcement may be designed according to the usual theory of reinforced concrete for combined direct and banding stresses. In any case the compressive stress must remain within the safe compressive stress of the type of brickwork. For values of allowable stresses and other specifications reference may be made to IS:1905 1961. For designing the sections under direct and bending stresses combined, the charts given in Figure 8 may directly be used. The modular ratio of steel to brickwork may be assumed 40 for this purpose.

Conclusion

It may be concluded that if the tying and reinforcing arrangements as explained and illustrated in the text are adopted, a masonry building will act as one unit for resisting inertia forces caused by ground motion and have sufficient strength to withstand them. The provision of reinforcement is expected to impart some ductility to the otherwise brittle structure which will further help it to absorb energy and delay its collapse.

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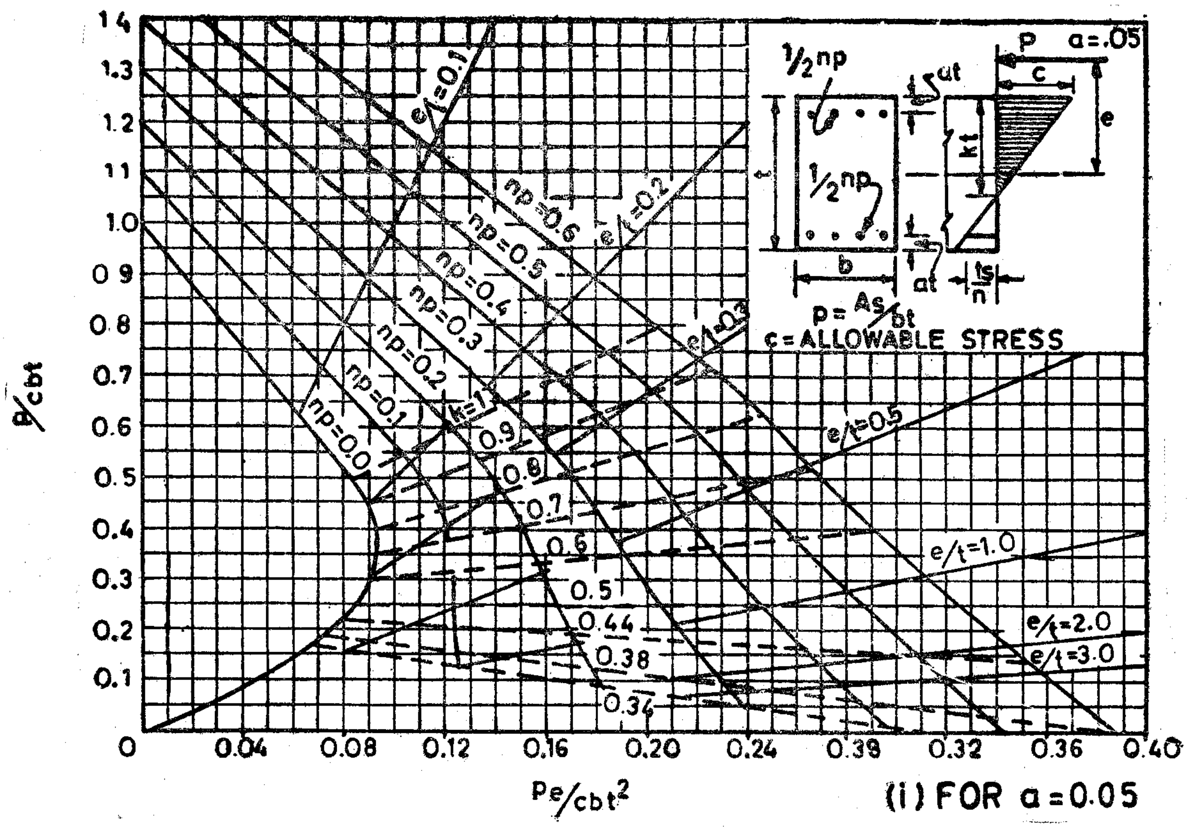


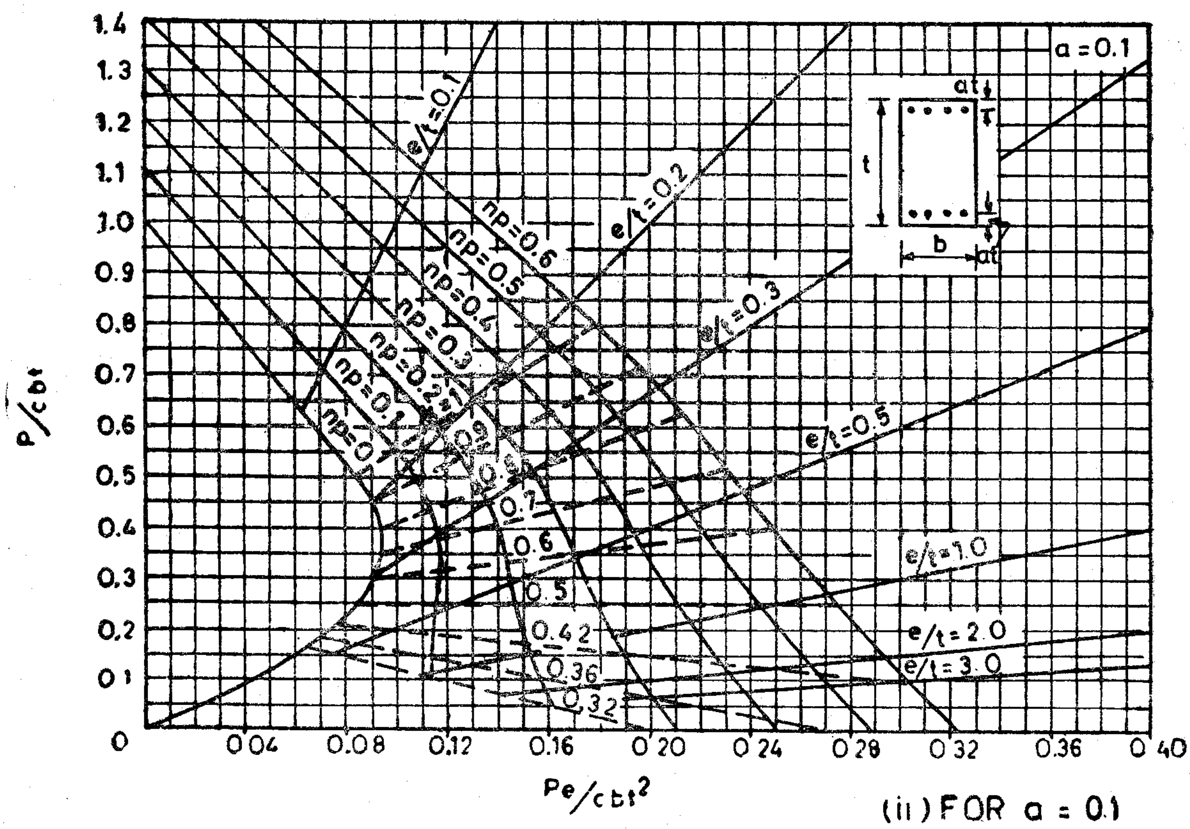
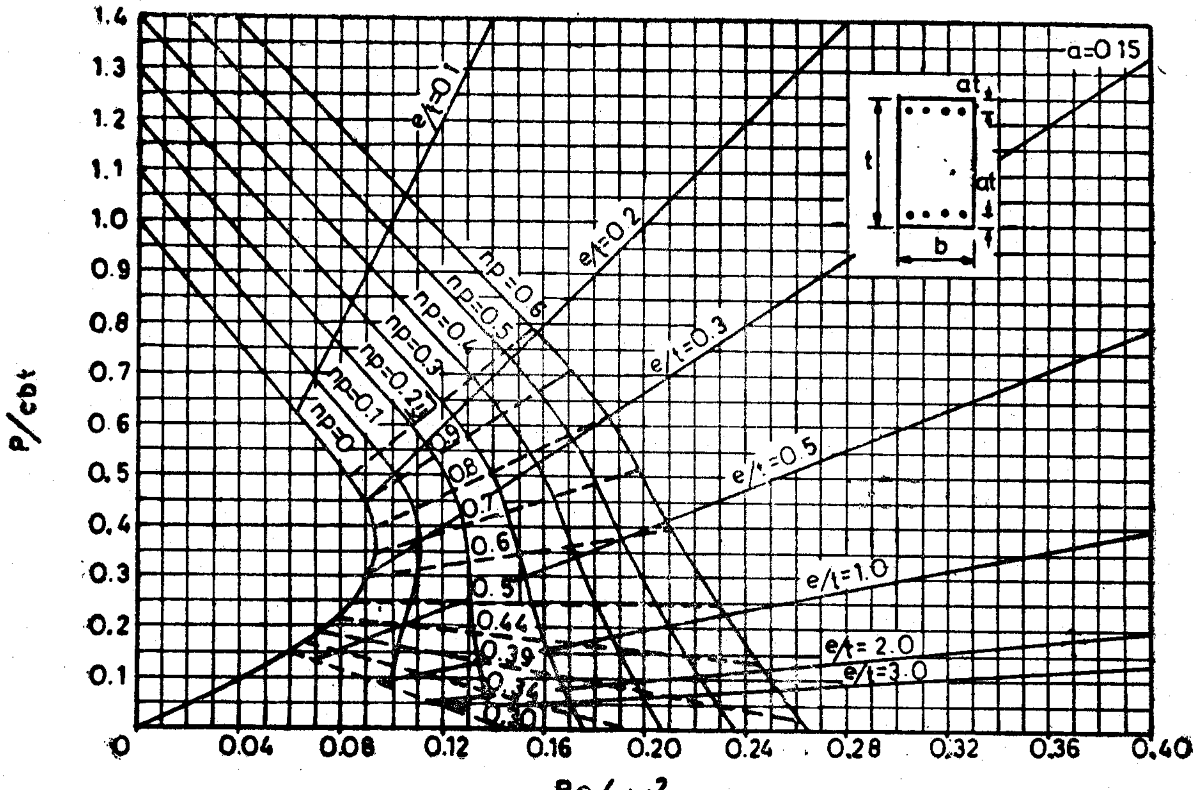
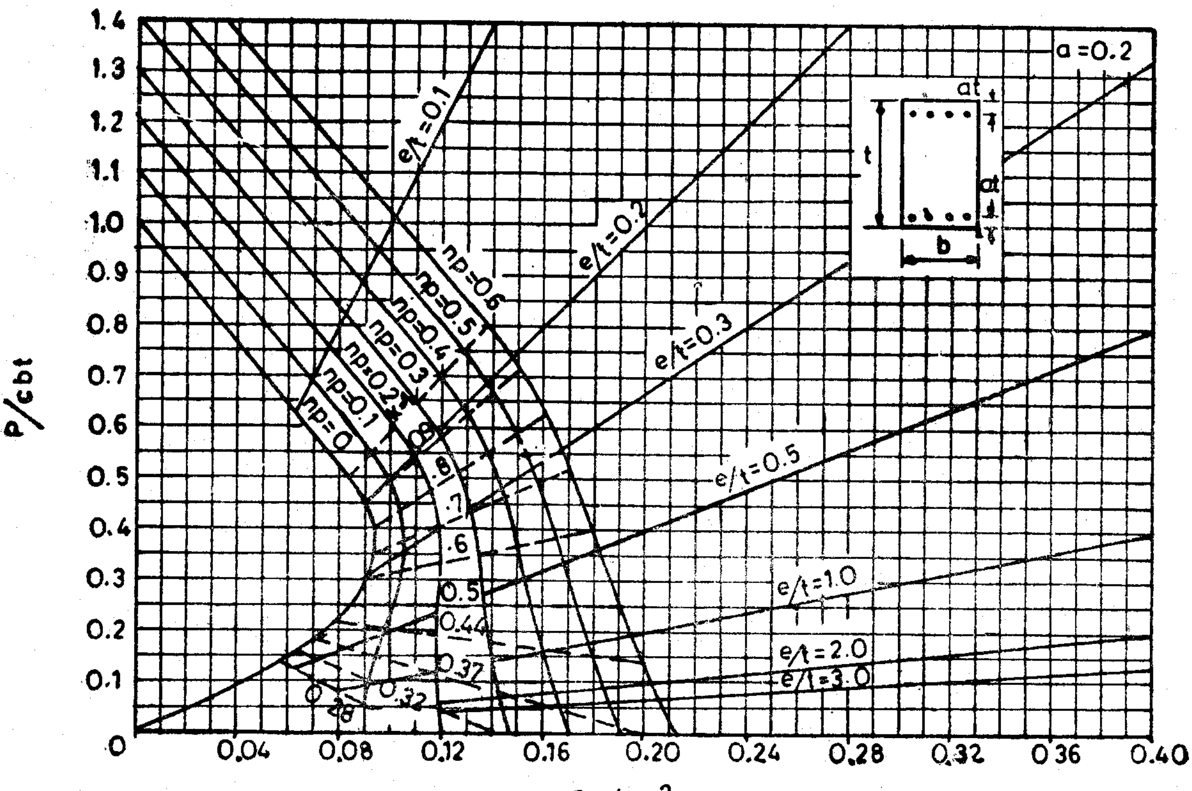


Fig. 8 (Continued)



(iii) FOR a=0.15



(iv) FOR a=0.2

Fig. 8