EARTHQUAKE RESISTANCE OF LOADBEARING BRICKWORK STRUCTURES

Eng M D Wijeratne*, Dr M T R Jayasinghe **

* Director, STRAD Consultants.
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ABSTRACT: Unreinforced loadbearing brickwork is a cost effective alternative to reinforced concrete frame structures in low rise residential building such as two to three storey houses, hostel buildings and hotels. For its widespread use, it is necessary to show that such structures would be safe in earthquakes that can be expected. This is because historical records show that earthquakes have occurred around Sri Lanka from time to time. In this paper, an earthquake design method that can be used for loadbearing brickwork structures is highlighted. The values that can be used for design parameters are also given. An example is presented to show the calculation method and the performance of loadbearing brickwork in actual structures. The additional precautions that can be taken in loadbearing brickwork structures constructed in Sri Lanka are also highlighted.

INTRODUCTION

For residential buildings, brickwork out of hand moulded burnt bricks is widely used in Sri Lanka. The majority of these buildings are of single storey construction. However, in recent times, two storey residential buildings have gained popularity due to advantages such as saving in the land area required and the thermal comfort that can be offered on the ground floor during the day time.

In these residential buildings, the ground floor walls can be successfully constructed using unreinforced loadbearing brickwork (Jayasinghe, 1997b). When adequate quality control measures are exercised, it is possible to obtain characteristic compressive strengths in excess of 1.5 N/mm^2 by using 1:6 cement sand mortar (Jayasinghe, 1998). When loadbearing brickwork is used, the cost saving in a two storey residential building can be in the range of 10% of the total cost, which can be determined on the basis of cost figures given in Jayasinghe & Maharachchi (1998). Such cost savings may be the motivating factor for widespread use of loadbearing brickwork in Sri Lanka. Hence, it would be necessary to ensure that such structures will be reasonably safe under earthquakes since collapse of residential buildings can lead to loss of life and property.

Sri Lanka is generally considered as a country located away from earthquake prone zones. However, since 1819, a total of 88 earthquakes have occurred around Sri Lanka (Abayakoon, 1998). Eighteen of these earthquakes have recorded magnitudes between 5.0 and 6.0 on the open ended Ritcher scale. This indicates that the establishment of earthquake performance of loadbearing brickwork structures could be worthwhile.

OBJECTIVES

The main objectives of this study are:

1. to determine earthquake design techniques and design parameters that can be used to evaluate the performance of loadbearing brickwork structures,

2. to determine the earthquake performance of a properly designed loadbearing brickwork structure using above earthquake design techniques, and

3. to recommend good earthquake resistant construction practices that can be adopted for loadbearing brickwork buildings.

METHODOLOGY

In order to fulfill the above objectives, the following methodology was adopted for this study:

1. The performance of unreinforced brickwork structures in past earthquakes was reviewed by using the information available in literature.
2. The earthquake design methods that can be used for structures constructed in Sri Lanka were established along with suitable values for design parameters.
3. The guidelines on the structural forms that can be used with unreinforced loadbearing brickwork buildings were established.
4. A case study was used to show the performance of a properly designed loadbearing structure constructed with locally available bricks.
5. The construction practices that can further enhance the survival of loadbearing brickwork structures were established.

PERFORMANCE OF UNREINFORCED BRICKWORK STRUCTURES IN PAST EARTHQUAKES

According to Dowrick (1977), the seismic response of brickwork buildings is based largely on the field observations after actual earthquakes and on the inferences from static load testing. In the 1992 Erzincan earthquake in Turkey, which measured 6.8 on open ended Ritcher scale, a large number of properly built unreinforced brickwork low rise buildings have survived whereas adjacent buildings of reinforced concrete construction had been completely destroyed (Saaticioglu & Bruneau, 1993). These loadbearing brickwork buildings consisted of reinforced concrete floor slabs cast insitu on top of single leaf unreinforced brick walls.

Another notable feature of the buildings that survived this earthquake was the floor area and the subdivision of floor spaces. In many residential buildings, the floor area was about 70 m^2 and the area was sub divided into 2 or 3 rooms and other utility areas. This made the structures highly redundant. The high in-plane rigidity of the concrete floor also allowed a good distribution of seismically induced forces to walls as a function of their respective rigidity. This form of construction was also able to prevent out-of-plane failure of brick walls as well, due to adequate lateral resistance offered by partition walls and also due to low floor heights used in residential buildings.

Another notable observation was that the buildings, which used poor quality masonry materials such as hollow clay units as loadbearing materials, have suffered heavily. Double layer unreinforced masonry structures also have performed poorly in this earthquake which is generally attributed to increased height to thickness ratios used in double leaf construction. This was also observed in the 1991 earthquake in New Castle, Australia (Melchers & Page, 1992) where a large number of double leaf brick walls suffered extensive damage due to corrosion of metal ties and the poor quality of mortar used for construction. These observations indicate that quality single leaf brickwork with strong mortars and proper structural forms can be useful in resisting earthquakes.

EARTHQUAKE DESIGN METHOD FOR LOADBEARING BRICKWORK BUILDINGS

According to historical records (Abayakoon, 1998), earthquakes have occurred close to Sri Lanka from time to time although it is located well away from the known tectonic plate boundaries. These earthquakes can be categorised as intra-plate type. Some of these earthquakes have recorded 5.0-6.0 on open ended Ritcher scale. Thus, it would be advisable to ensure that loadbearing brickwork structures constructed with hand moulded burnt bricks can resist a minor earthquake without damage and be able to survive a moderate earthquake. The chances of a major earthquake occurring in intra-plate areas are somewhat remote unless historical records indicate a large number of earthquakes that could have resulted from localised effects like faults. Since Sri Lanka has experienced only a few earthquakes, the resistance to a major earthquake need not be a significant design criteria.

In contrast to most other loadings such as dead and live loading, earthquake loadings are induced in a structure because of the time varying motion of the ground. The loads actually induced in the structure depend on the distribution of mass throughout the structure. These induced loads are inertial loads which occur due to accelerations experienced by the structure.

For earthquake design, there are primarily two methods available for analysis, namely static analysis and dynamic analysis. Dynamic analysis is required for highly irregular structures. It should be noted that such structures have not performed well in past earthquakes (Woodside, 1995). Therefore, performing a dynamic analysis and designing for the corresponding forces is not a guarantee of satisfactory behaviour. For structures of regular plan, which are not subjected to twisting, a static analysis is generally sufficient. In static analysis, the effects of earthquake on the building are first determined by calculating the base shear force induced by the earthquake. Then, this force is distributed at each floor level on the basis of the vertical distribution of weight of the structure.

There are a number of methods available for determining the equivalent base shear such as those of Uniform Building Code (1985), (1988), Standards Australia (AS 1170/4, 1993) and National Building Code of Canada (1990). For large countries where sufficient number of past earthquake records are available, it is possible to divide the country into a number of earthquake zones and to define suitable acceleration coefficients to represent the intensity of earthquakes that can be expected. For countries where such acceleration coefficients have not been developed, it would be possible to design the buildings for the damage that could be expected from a probable earthquake. Such a method is given in UBC (1985) code. This method was used for this study.

Use of UBC (1985) Method for the Determination of Base Shear

In UBC (1985), the equivalent static base shear is calculated by considering the dynamic characteristics of the building expressed using the natural period of vibration of the building, the type of the structure, the seismic risk at a given location, the geology of the site, and the importance of the building, in addition to the building weight (Schueller, 1990). The equivalent base shear force is given by:

$$V = ZIKCSW \qquad \text{eq. 1}$$

where:

V = the total lateral force or shear at the base, which will be distributed appropriately at each floor level to determine the lateral response of the building.

Z = seismic probability zone factor which determines the probability of occurring an earthquake.

In zone zero where no seismic damage is expected, $Z = 0.125$
In zone one where minor seismic damage can be expected, $Z = 0.1875$
In zone two where moderate damage can be expected, $Z = 0.375$
In zone three where major damage can be expected, $Z = 0.75$
In zone four where the location is close to a major fault, $Z = 1.0$

I = occupancy importance factor which is 1.0 for residential buildings where a large number of people does not gather.

K = building type factor which has to be taken as 1.33 for loadbearing brick wall structures.

C = seismic coefficient which takes account of the dynamic characteristics of the building. It is given by the following equation where T is the fundamental natural period. This can be determined approximately by using $T = 0.1\ N$, where N is the number of stories. Alternatively, $T = 46/H$, where H is the total height of the structure, can be used.

$$C = 1/(15\ T^{1/2}) < 0.12 \quad \text{and} \qquad \text{eq. 2}$$

$$CS < 0.14 \qquad \text{eq. 3}$$

S = soil structure interaction factor. This factor takes account of the way that the vibrations can be amplified due to the response of the soil to earthquake vibrations.

For rocklike formations, or stiff soil conditions overlaying rock at a depth of less than $60\ m$, $S = 1.0$
For deep cohesionless or stiff clay soil conditions overlaying rock at a depth greater than $60\ m$, $S = 1.2$
For soft to medium stiff clay and sands $10\ m$ or more deep, or if the soil profile is unknown, $S = 1.5$

W = the total dead load and appropriate portions of the live loads.

Determination of a Suitable Soil Structure Interaction Factor

Since the soil profile at a site varies, an appropriate value for S should be considered. The value of S depends on the depth to the bed rock and the characteristics of soil above the bedrock. In certain locations of Sri Lanka, specially in hilly areas, there may be locations where the bedrock is exposed. When the bedrock is not exposed, the depth to the bedrock should be determined. A few borehole profiles collected from various areas in and around Colombo and few main towns in Sri Lanka are presented in Tables 1 and 2

Table 1: A sample of soil profiles in and around Colombo

Location					
Colombo 3	Kotahena	Borella	Kiribathgoda	Gampaha	Biyagama
0-3m loose to medium sand	0-2m silty sand	0-5m alluvial deposits	0-6m soft organic clay and peat	0-4m soft organic clay	0-2m alluvial deposits
3-14m stiff sand	2-6m fine sand	5-7m stiff sand	6-11m stiff soil	4-8m stiff clay	2-4m clayey sand
14-22m residual soil	6-8m fine sand with gravel	7-12m fine to coarse sand	hard rock	8-27m decom-posed rock	4-18m residual soil
22-23m decom-posed rock	8-9m decom-posed rock	12-20m decom-posed rock		hard rock	hard rock
hard rock	hard rock	hard rock			

Table 2: A sample of soil profiles in major towns

Location					
Panadura	Horana	Ratnapura	Galle	Matara	Monaragala
0-1m loose clayey sand	0-1m clayey coarse sand	0-4m silty sand	0-2m residual soil	0-1m loose gravel	0-2m soft clay
1-6m loose residual soil	1-6m clayey sand	4-6m soft silty sand	2-5m sand with sea shells	1-3m peat	2-11m decom-posed rock
hard rock	6-7m decom-posed rock	6-8m clayey sand with peat	5-10m clayey sand	3-7m silty sand	hard rock
	hard rock	8-9m decom-posed rock	10-17m medium sand	7-19m sand with sea shells	
		hard rock	hard rock	hard rock	

These soil profiles indicate that the depth to the bedrock can exceed 10 m in many locations in Sri Lanka. These boreholes were taken at low lying areas with poor soil conditions. In other areas, which are high ground not subjected to frequent flooding, laterite soil is generally available. This is due to tropical climates that prevail, which is ideally suitable for the formation of laterite soils. Laterite soils occur as a result of weathering of bed rock. The warm climate produces hot water during rainy seasons which removes soluble ions in the weathered soil leaving an iron and aluminum rich deposit of yellow to reddish in colour (Lilley & Robinson, 1995). The laterite soils found in Sri Lanka can be considered as clay with sand and gravel since the clay content can be as high as 30% - 50% (Perera, 1994). Thus, this soil can conservatively be assumed as soft to medium stiff soil. The thickness of this laterite clay layer can be more than 10m at many locations as usually indicated by drinking water wells. Drinking water wells excavated in laterite soils of depth in excess of 10 m is a common sight in many parts of Sri Lanka. Thus, the use of S = 1.5 could be appropriate for the building sites where the bedrock is not exposed.

Lateral Distribution of Base Shear

For low rise buildings having the same mass at each floor and a natural period of vibration less than 0.7 seconds, the base shear, *V*, will be distributed as shown in Figure 1. The lateral load, F_x, at any floor level *x* is given by:

$$F_x = \frac{Vh_x}{\sum_{i=1}^{n} h_i} \qquad \text{eq (4)}$$

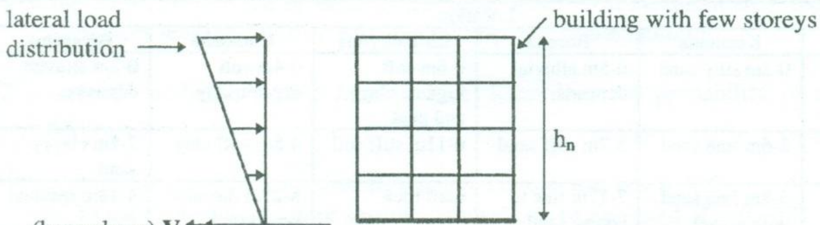

Figure 1: Equivalent lateral earthquake load distribution

For a building where the mass vary at different floor levels, the magnitude of the distributed forces, F_x, is given by:

$$F_x = \frac{VW_x h_x}{\sum_{i=1}^{n} W_i h_i} \qquad \text{eq (4)}$$

where

W_i, W_x = that portion of *W* which is located at or is assigned to level *i* or *x* respectively.

h_i, h_n, h_x = the height in metres above the base to level *i*, *n* or *x* respectively; the level *n* is the uppermost level in the main portion of the structure.

These equations can be used to determine the lateral loads due to earthquakes. Once the lateral loads are determined, it is possible to check the structure for its ability to resist these forces.

Since there is no guarantee that earthquake force will act through the centroid of the building, an eccentricity of 0.05 x the width of the building is usually considered for earthquake forces.

GUIDELINES ON STRUCTURAL FORMS

In loadbearing brickwork structures, all the walls in a particular direction participate in resisting earthquake loads. Therefore, it is important to ensure that the brick walls are arranged in such a way that the twisting of the structure is minimised. If this is not complied with, much higher earthquake induced forces can occur in certain walls leading to damage or progressive collapse.

The twisting of a structure can be minimised by ensuring that the shear centre of the structure in each direction coincides with the geometric centroid of the structure. When this does not happen, the external forces acts through the geometric centroid and the resistance of the structure occurs through the shear centre thus leading to twisting.

In order to keep the earthquake induced stresses as low as possible, it would be important to have long walls. Thus, the aspect ratio (height/width) of the structure should be kept to a minimum. For individual members also, the aspect ratio (height/length) should be kept as small as possible.

Another source of weakness in brickwork structures is openings. Provision of too many openings can lead to poor earthquake resistance in brick walls. When long brick walls resist earthquake loads, those would be resisting the flexural stresses arising due to in-plane loads. These stresses would be highest at the ends of the walls. Therefore, it would be advisable to avoid openings in such regions like those close to external corners.

It is suggested by Dowrick (1977) that the distance to an opening from an edge of a wall should be at least about the height of the opening. This indicates that door or window openings should not be located at the external corners of a loadbearing brickwork building. It is also advisable to ensure that the total area of openings in a wall does not exceed 1/3 of the total area of the wall. The openings such as windows and doors also should be separated as much as possible within the limits of those allowed by internal partitions.

DESIGN STUDY FOR A LOADBEARING BRICKWORK STRUCTURE

A design study was carried out for a two storey hostel building with the plan view and elevation shown in Figure A.1 given in Appendix A. In this building, all the ground floor cross walls are constructed with one and a half brick thick walls. All the other walls in ground floor and upper floor are one brick thick. The characteristic design strength of hand moulded burnt bricks available in Sri Lanka was taken as 1.5 N/mm^2 (Jayasinghe, 1998). For the earthquake design calculations, both minor and moderate earthquake intensities were considered. The wall thicknesses were based on a brick of length 200 mm, width 100 mm and height 50 mm. The corresponding wall thicknesses were 210 mm for one brick, and 310 mm for one and a half brick thick walls. These walls were added a thickness of 30 mm to allow for plaster on either side.

It is shown in the example given in Appendix A that for both minor and moderate earthquakes, the resultant stresses will not be tensile throughout the length of the walls. This is important since brickwork may develop cracking due to bond failures even at very low tensile stresses. It is also shown that the maximum design compressive stresses will be less than the compressive strength of 1.5 N/mm^2 for both minor and moderate earthquakes. This is also important to prevent crushing failure of brickwork. Thus, loadbearing brickwork structures with a lot of brick walls are likely to be safe in both minor and moderate earthquakes.

It should be noted that properly designed brickwork structures can have a lot of reserve strength also due to high factors of safety used against material strength. For dead and live loads, the partial factors of safety are 1.4 and 1.6 respectively. Thus, an average value of 1.5 could be used for loads. The partial factor of safety recommended for material strength is 3.5. The reserve strength of a loadbearing brickwork structure can thus be estimated approximately as 1.5 x 3.5 = 5.25. This could be a contributing factor to the high rate of survival of properly designed and constructed loadbearing structures in past earthquakes such as Erzincan, 1992 (Saaticioglu & Bruneau, 1993).

This indicates that the generous use of brick walls of sufficient width in a loadbearing structure with proper structural layouts would be sufficient to ensure the survival even in a moderate earthquake. However, sufficient precautions should have been taken against foundation failures that would occur in earthquakes specially in low lying areas with weak soil conditions. The calculations given in Appendix A do not take account of additional stresses that would arise due to excessive settlements associated with very soft alluvial deposits. In many past earthquakes such as Erzincan (Saaticioglu & Bruneau, 1993), the structures built in alluvial deposits have suffered heavier damage than other areas, which is attributed to the amplification of earthquake forces in very soft soils.

ADDITIONAL PRECAUTIONS FOR LOADBEARING BRICKWORK BUILDINGS

When subjected to earthquakes, there can be shear stresses in addition to in plane flexural stresses in loadbearing walls. These can give rise to tensile stresses that is generally in the diagonal directions. Since the direct tensile strength of brickwork made with hand moulded bricks is quite low, these stresses can cause severe cracking. It is shown by Jayasinghe (1997a) that the direct tensile strength can be only about 0.2 N/mm^2 for brickwork out of hand moulded bricks. Therefore, it is important to tie the structure horizontally. It is possible to achieve this by providing tie beams at the following levels:

1. A tie beam at plinth level - this will prevent the disintegration of foundation.
2. A tie beam at the sill level of windows - this will prevent any adverse shear stresses induced due to window openings from causing wide cracks. This tie beam should be continued to the internal partition walls to tie them properly to the external walls.
3. A tie beam at the first floor slab level or a insitu cast floor slab - this will connect all the walls together at the first floor slab level, thus leading to better distribution of earthquake induced lateral loads on to walls.
4. A tie beam at the window sill level at the upper floor windows - this will be required to prevent severe cracking at upper floor windows.
5. A tie beam connecting the lintels provided at upper floor window top levels - this will prevent any disintegration of walls that can occur at the upper floor walls.

Thus, there will be four additional tie beams to enhance the earthquake resistance, namely at ground floor plinth level, ground floor window sill level, upper floor window sill level and upper floor lintel level. It is shown by Jayasinghe & Maharachchi (1998) that such tie beams will incur an extra cost of about Rs 100/= per linear metre when used in loadbearing brick walls. In a house having about 100 m^2 per floor, the length of the tie beams at a particular level will be about 50 m. Thus, the extra cost of tie beams will be about Rs 20,000/=. If a two storey house of about 200 m^2 (100 m^2 each floor) would cost about Rs 1,000,000/=, the extra cost of providing tie beams will be about 2% of the total cost.

These tie beams also can perform the task of preventing any thermal cracks in loadbearing walls, thus providing multiple uses. It should also be noted that tie beams provided at plinth level and the ground floor window sill levels can improve the flexural capacity of the brick wall and foundation system considerably (Jayasinghe, 1997a). This will be useful in preventing any foundation failures that may arise due to settlement of soils, which is often seen after earthquakes.

In addition to these, it is advisable to ensure that upper floor gable walls are adequately supported by partition walls that continue up to the roof level. In past earthquakes, failures have occurred in unsupported gable walls due to lack of lateral restraint (Melchers & Page, 1992).

CONCLUSIONS

When a proper structural layout is selected, it would be possible to use hand moulded burnt clay bricks such as those available in Sri Lanka to construct safe multi-storey buildings that will have a good possibility of surviving even a moderate earthquake. It is important to ensure that the building will behave as a non-twisting structure as far as possible by arranging the walls in such a way so that shear centre coincide with the geometric centroid of the building. The door and window opening locations also should be carefully planned to avoid weaknesses. Large openings at the external corners should be avoided as a general practice. It is also possible to take some extra precautions such as providing tie beams that would minimise wide diagonal cracks in an earthquake. These tie beams also could be useful in minimising damages that would arise due to settlement of the foundations after an earthquake. It should be noted that the extra cost of earthquake resistant construction could be quite low, such as in the range of 2%.

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APPENDIX A

In order to indicate the earthquake resistance of a well proportioned loadbearing brickwork two storey building, the following example is presented.

Building geometry: The building has a floor to floor clear height of *2.7 m*. The thickness of the cast insitu one way spanning reinforced concrete floor slab is *0.125m.* This will give a total height of *5.525 m* to the *1st* floor ceiling. The *1st* floor cross walls are gable walls thus a purlin roof can be constructed for this building. The roof material is asbestos. The length of door and window openings is considered as 1.0 m. Each room is provided with a door opening to the central passage and a window on the external wall. The total length of walls in the longitudinal direction excluding openings is assumed as *18.4 m,* thus allowing *8.0 m* as openings. The density of masonry is assumed as *20.0* kN/m^2 and density of concrete is assumed as *24* kN/m^2.

Overall length	= *26.4 m*
Overall height	= *6.925 m*
Overall width	= *13.5 m*

Floor to floor height = *2.7 m*

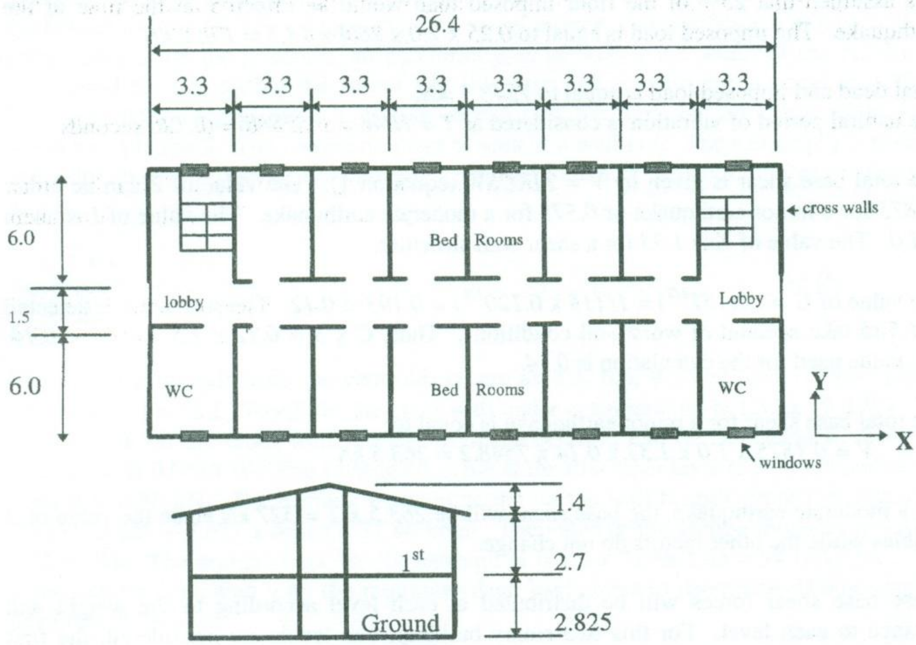

Figure A.1: Plan and elevation of the building used for the design example

Design calculations

Characteristic loads:

Roof: Dead load, G_k, with asbestos roof on plan	= *0.5 kN/m²*
Floors: Dead load, G_k, for *125 mm* slab	= *3.0*
Partitions	= *0.5*
Finishes, etc	= *0.5*
Services, etc	= *0.2*
Total	= *4.2 kN/m²*
Imposed load, Q_k, on roof	= *0.25 kN/m²*
Imposed load, Q_k, bed rooms	= *2.0 kN/m²*

For the calculation of dead loads, the *1st* floor gable wall is assumed to have an average height of *(2.7 + 4.1)/2 = 3.4 m.*

Total weight of the building:

Weight of cross walls = 2 x *9* x *6.0* x *[3.4* x *0.24 + 2.7* x *0.34]* x *20.0 = 3745.4 kN*

Weight of longitudinal walls = 2 x *4* x *18.4* x *0.240* x *2.7* x *20.0 = 1907.7 kN*

Weight of concrete slabs = *26.4* x *13.5* x *0.125* x *24 = 1069.2 kN*

Weight of partitions and finishes = *26.4* x *13.5* x *(0.5 + 0.5 + 0.2) = 427.7 kN*

Weight of the roof with 2.0 m eaves = *0.5* x *[(26.4 + 2.0)* x *(13.5 + 2.0)] = 220.1 kN*

It is assumed that *25%* of the floor imposed load would be effective at the time of the earthquake. The imposed load is equal to $0.25 \times 2.0 \times 26.4 \times 13.5 = 178.2\ kN$

Total dead and imposed load is equal to $7548.3\ kN$.
The natural period of vibration is considered as $T = H/46 = 5.525/46 = 0.120$ seconds

The total base shear is given by $V = ZIKCSW$ (equation 1). The value of Z can be either 0.1875 for a minor earthquake or 0.375 for a moderate earthquake. The value of I is taken as 1.0. The value of K is 1.33 for a shear wall structure.

The value of $C = 1/(15T^{1/2}) = 1/(15 \times 0.120^{1/2}) = 0.193 < 0.12$. The soil factor is selected as 1.5 to take account of worst soil conditions. Thus, $C \times S = 0.12 \times 1.5 = 0.18 < 0.14$. The value used for the calculation is 0.14.

The total base shear for a minor earthquake is equal to:

$$V = 0.1875 \times 1.0 \times 1.33 \times 0.14 \times 7548.2 = 263.5\ kN$$

For a moderate earthquake, the base shear will be $263.5 \times 2 = 527\ kN$ since the value of Z doubles while the other factors do not change.

These base shear forces will be distributed at each level according to the weight and distance to each level. For this two storey building, two levels are considered, the first floor level and the roof level. It is assumed that the dead weight from mid height of the floor below to the mid height of floor above is located at a floor level.

<u>Dead and imposed load lumped at the first floor level:</u>

Weight of cross walls = $2 \times 9 \times 6.0 \times (1.35 + 1.7) \times 0.240 \times 20.0 = 1581.1\ kN$
Weight of longitudinal walls = $4 \times 18.4 \times 0.240 \times 2.7 \times 20.0 = 953.8\ kN$
Weight of concrete slabs = $26.4 \times 13.5 \times 0.125 \times 24 = 1069.2\ kN$
Weight of partitions and finishes = $26.4 \times 13.5 \times (0.5 + 0.5 + 0.2) = 427.7\ kN$
Imposed load = $0.25 \times 2.0 \times 26.4 \times 13.5 = 178.2\ kN$
Total load = $4210\ kN$

<u>Dead load lumped at the roof level:</u>

Weight of cross walls = $2 \times 9 \times 6.0 \times 1.7 \times 0.240 \times 20.0 = 881.2\ kN$
Weight of longitudinal walls = $4 \times 18.4 \times 0.240 \times 1.35 \times 20.0 = 476.9$
Weight of roof = $0.5 \times [(26.4 + 2.0) \times (13.5 + 2.0)] = 220.1$
Total load = $1578.3\ kN$

The earthquake load acting at each level is calculated in the tabular form.

Height above base (h_x or h_i in m)	Total load (w_i kN)	$w_i h_i$ or $w_x h_x$	$w_x h_x / \Sigma\ w_i h_i$	$F_x = V\ w_x h_x / \Sigma\ w_i h_i$ (kN)
2.825	4210.0	11893.3	0.576	151.70
5.525	1578.3	8720.1	0.423	111.46
	$\Sigma\ w_i h_i =$	20613.4		

Thus, the earthquake force acting at the first floor level is 151.70 kN and that acting at the roof level is 111.46 kN. Since there is no guarantee that earthquake forces will act symmetrically on the structure, an eccentricity, e, of 0.05 x the width of the building is considered for the earthquake forces. In this case, the eccentricity is equal to 1.32 m. When the lateral forces are acting at an eccentricity, the force on individual walls, w_j, is given by equation 5. The second moment of area of a wall is I_j. The subscript j is used for individual walls. Subscript i has already been used to denote the floor numbers in equation 4.

$$w_{ij} = \frac{WI_j}{\sum I_j} + \frac{Wec_jI_j}{\sum I_jc_j^2} \qquad \text{eq (5)}$$

The distance to wall from the centroid, c_j, are as 3.3, 6.6, 9.9 and 13.2. Thus, ΣIc_j^2 = 1306.8 I. Σ I = 18 I. Wec_jI_j for the cross wall under consideration is 1.32 x 13.2 WI. The force acting on the cross wall furthest from the centroid can be calculated from eq (5). This value is 0.0688 W. The earthquake force at the first floor level is 9.38 kN and at roof level it is 6.88 kN. The bending moment acting on the wall furthest from the centroid is 10.43 x 2.825 + 7.677 x 5.525 = 71.88 kNm. The value of the section modulus is 0.310 x $6^2/2$ = 1.86. The design stress due the earthquake is 1.4 x 71.88/1.86 = 54.10 kN/m^2. This value can be compared with the minimum dead load stress to determine whether tension develops in the wall.

The minimum dead load stress can be calculated by considering the load transferred from the roof, slabs and the self weight of walls. It should be noted that for the edge wall, the width of the slab that transfers load is half the slab span.

0.9 x [0.5 x 3.3/2+ 4.2 x 3.3/2 + 3.4 x 0.24 x 20 + 2.7 x 0.34 x 20] / 0.31 = 123.2 kN/m^2

This is much higher than the earthquake induced tensile stress, thus the resultant stress block will be compressive. The maximum compressive stress due to dead and earthquake is given by $1.4\ G_k + 1.4\ Eq_k + 1.6\ Q_k$ = *(123.2 x 1.4) /0.9 + 54.10 + 1.6 x 2.0 x 3.3/(2 x 0.31) = 262.7 kN/m^2 = 0.2627 N/mm^2*. The strength required with a factor of safety of 3.5 for the material strength is *0.2627 x 3.5 = 0.919 N/mm^2*. This is much less than the allowable stress of *1.5 N/mm^2*. Similarly, the stresses in other walls also can be checked.

If the earthquake is moderate, the earthquake induced stress will be 2 x *54.10 = 108.20 kN/m^2*. The minimum stress due to dead load is still higher than the earthquake induced tensile stress, thus the resultant stress block will be compressive. The maximum compressive stress due to dead and earthquake is *316.8 kN/m^2 = 0.3168 N/mm^2*. The strength required in the wall is *1.11N/mm^2*, which is still less than the design strength of *1.5 N/mm^2*.