CALCULATION SUMMARY FOR SINGLE-STORY CONFINED MASONRY HOUSE WITH CONCRETE ROOF

The following is a narrative of the structural design calculations that were performed for a 62m^2 single-story confined masonry house with a concrete roof. Text describing the conclusion reached for each calculation section is shown in italics. The full design calculations are available for those who would like more detail.

1.0 References

The design of the single-story confined masonry house is based on provisions from both US-based codes and Chinese design standards, as well as various recommendations from research on confined masonry structures. The primary references for the design are as follows (refer to 'Build Change Design Criteria and Material Properties' for additional references):

- National Standard of the People's Republic of China, Code for Design of Masonry Buildings (GB 50003)
- National Standard of the People's Republic of China, Code for Design of Concrete Buildings (GB 50010)
- National Standard of the People's Republic of China, Code for Structural Load Norms (GB 50009)
- National Standard of the People's Republic of China, Code for Seismic Design of Buildings (GB 50011)
- American Concrete Institute's Building Code Requirements for Masonry Structures (ACI 530-05)
- American Concrete Institute's Building Code Requirements for Structural Concrete (ACI 318)
- American Society of Civil Engineers' Minimum Design Loads (ASCE-7 2005)
- Seismic Rehabilitation of Existing Buildings (ASCE/SEI 41-06) and Evaluation of Earthquake Damaged Concrete and Masonry Wall Buildings, Basic Procedures Manual (FEMA 306)
- Masonry Structures Behavior and Design, by Drysdale, Hamid and Baker, Second Edition
- Design of Concrete Structures, A.H. Nilson, Twelfth Edition

The design of the masonry and concrete elements is based upon strength design principles.

2.0 House Description

The confined masonry house has the following characteristics:

- 7.8m x 7.5m x 3.2m high confined masonry house with a 150mm thick reinforced concrete flat roof
- 240x240 reinforced concrete tie columns, 240x200 reinforced concrete ring and plinth beams
- Full-brick wide walls connected to tie columns via horizontal bed joint reinforcement
- Approximately 5.5% shear wall density in each direction
- Unreinforced brick and stone foundation

Refer to the drawing set for additional details.

3.0 Materials

Material properties for the masonry wall, concrete, and reinforcing steel used in the calculations were selected from the Chinese standards. According to field surveys, the following material types are locally-available in Sichuan Province:

Bricks: MU10 type (10 MPa or 1450 psi compressive strength)
Mortar: M5 type (5 MPa or 725 psi compressive strength)
Concrete (tie columns, plinth beams): C20 type (f'c = 9.6 MPa or 1392 psi)
Concrete (roof slab, ring beams): C25 type (f'c = 11.9 MPa or 1726 psi)
Longitudinal Reinforcing Steel (12mm dia bars): HRB335 ribbed bars (fy = 300 MPa or 43.5 ksi)
Transverse Reinforcing Steel/Bed Joint Reinforcement (6mm dia stirrups): HPB235 smooth bars (fy = 210 MPa or 30.5 ksi)

Preliminary test results for the MU10 bricks demonstrate that the compressive strength of the bricks available in the field is higher than 10 MPa, with values ranging from 21 MPa to 53 MPa. A lower bound value of 15 MPa (2176 psi) was selected for use in the design due to the small sample set. This value may be increased if further testing demonstrates that a higher value is justified. Using the Unit Strength Method (Table 2105.2.2.1.1) of the International Building Code for Type N mortar (which is the closest mortar to M5 or 5 MPa mortar) and 15 MPa (2176 psi) brick, a design compressive strength (f'm) of 6.89 MPa (1000 psi) was obtained for the masonry/mortar matrix.

4.0 Loads

Dead, live, snow and wind loads were computed according to the Chinese standards.

The seismic design is based on a design basic ground motion acceleration of 0.3g which is an upgrade from the code-based design acceleration used prior to the Wenchuan earthquake. Using this acceleration, the seismic loads were computed two ways:

Method 1: using the methodology in ASCE-7 for the equivalent lateral force procedure based on assumed values for Ss and S1 corresponding to a ground acceleration of 0.3g

Method 2: using the base shear method and corresponding parameters according to GB50011

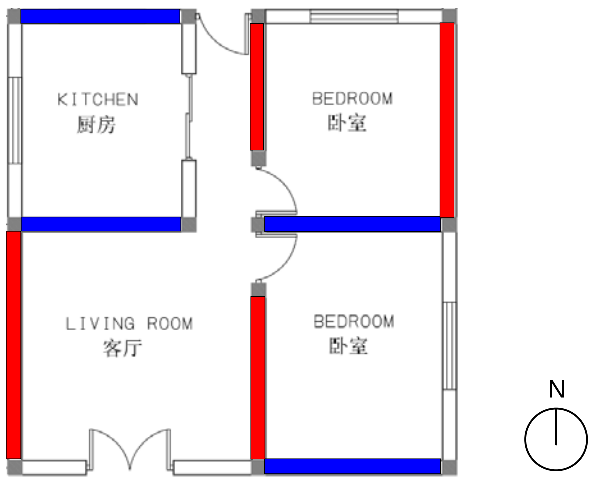
5.0 In-Plane Shear Force Calculations for Masonry Walls

For the in-plane shear calculations, both interior and exterior masonry walls were considered to be shear-resisting elements. The masonry walls were divided into 'shear walls' and 'non-shear walls'. 'Shear walls' were considered to be walls without openings with a maximum height/length ratio equal to 1.2 (ie 2.5m minimum length for a 3m tall wall). The north-south and east-west 'shear walls' are shown in red and blue respectively in the house layout below. 'Non-shear walls' were defined as walls that exceed this maximum ratio. Non-shear walls were not assumed to contribute to the lateral force-resisting system of the building.

For each shear wall, the in-plane seismic shear force demand was calculated based on a design ground acceleration of 0.3g and compared to its shear capacity. This procedure was performed two ways:

1 Demands based on ASCE-7 with R = 2, Capacities based on strut and tie formulas per FEMA 306
2 Demands based on GB50011, Capacities based on URM and CM formulas per GB50003

Each method is described below:



5.1 In-Plane Shear Calculations Based on US Codes

Demands: The shear force demands on the shear walls were calculated according to the equivalent lateral force procedure in ASCE-7. The base shear was calculated using the effective weight of the confined masonry walls and the concrete roof including the superimposed dead loads of the roofing draining and insulation materials. A Response Modification Coefficient (R factor) of 2 was selected based on judgment because ASCE-7 provides no guidance for confined masonry. However, it was decided that a confined masonry system has higher ductility than an unreinforced masonry system so an R factor greater than 1.5 was justified (future testing may help to justify an R factor greater than 2). The seismic loads were considered separately for the two primary directions of the building. The shear force in each shear wall was computed according to a rigid diaphragm assumption (due to the concrete roof) and the torsion was computed based on the maximum of that produced using the layout geometry or 5% accidental torsion.

Capacities: The in-plane shear capacity of each shear wall was computed using strut and tie principles on the premise that the primary lateral load path for the wall system following the formation of tensile cracks in the walls will be through strut and tie action. Although the confined masonry system is not equivalent to an infill frame system, Equation 8-10 of FEMA 306, which defines the shear capacity of an infill wall based on the compressive failure of the diagonal strut, was used to compute the post-tension cracking in-plane shear capacity of the confined masonry shear walls. This equation was used in combination with an assumed strength reduction factor of 0.8:

$$V_c = a * t_{inf} * f'_{m90} * \cos\theta$$

where f'_{m90} per FEMA 306 was assumed to be 50% of f'm, taken as 6.89 MPa or 1000 psi. In this equation, "t_{inf}" represents the width of the wall (240mm) and "a" represents the equivalent width of the compression strut which is computed according to FEMA 306 Equation 8-1 (or ASCE/SEI 41-06 Equation 7-7):

$$a = 0.175 * (\lambda_1 * h_{col})^{-0.4} * r_{inf}$$

where r_{inf} is the diagonal length of the compression strut and λ_1 is defined according to FEMA 306 Equation 8-2 (or ASCE/SEI 41-06 Equation 7-7) as:

$$\lambda_1 = [(E_{me} * t_{inf} * \sin 2\theta)/(4 * E_{fe} * I_{col} * h_{inf})]^{0.25}$$

For this house configuration, which has approximately 5.5% shear wall density in each direction, the calculations show that there is almost twice the necessary in-plane shear capacity in both the E-W and N-S directions. Therefore, it would be possible to add one additional wall opening in each primary direction provided that the symmetry of the shear walls is maintained.

For comparison only, the in-plane shear capacity of each shear wall was also computed according to the strength design equations for the shear capacity of unreinforced masonry in Section 3.24 of ACI 530 where the area of the wall was considered to be the area of the masonry wall neglecting the concrete tie columns. Also, the dead load of the concrete roof and the walls was considered as contributing to the shear capacity of the wall. The in-plane shear capacities were compared to increased seismic shear demands based on a reduced R factor of 1.5. *The computed capacities were compared to 2.5 times the shear demands in order to satisfy Section 3.1.3 of ACI 530, and the results showed that the shear capacity of each wall exceeded the amplified seismic shear demands.*

No ACI 530-based provisions for the shear capacity of reinforced masonry walls were considered because the masonry walls have neither vertical nor continuous horizontal reinforcement.

5.2 In-Plane Shear Calculations Based on Chinese Standards

Demands: The shear force demands on the shear walls were calculated according to the base shear method in GB50011. The base shear was calculated using the effective weight of the confined masonry walls, the concrete roof and 50% of the roof snow load, according to GB50011. Per GB50011, the seismic loads were considered for the two primary directions of the building separately, and the distribution of shear forces into the shear walls was based upon a rigid diaphragm assumption.

Capacities: The in-plane shear capacity of each wall was computed using the GB50003 equation (Eqn 10.2.3) for an unreinforced masonry wall which uses the design shear strength of the masonry-mortar matrix specified in the code as 16psi (0.11 MPa) for brick walls with M5 mortar.

GB50003 also provides an equation (Eqn 10.3.2) for the shear capacity of a masonry wall with reinforced concrete columns (which appears to refer to a confined masonry wall); however, the equation considers only the contribution of reinforced concrete columns “in the middle” of the wall, and because the defined shear walls in this design had no interior columns, this equation produced similar results to those based on the unreinforced masonry wall equation.

A comparison of the Chinese code-based shear demands to the wall capacities based on the Chinese code provisions for unreinforced masonry walls showed that the N-S and E-W shear walls do NOT have sufficient capacity to resist the seismic shear forces resulting from 0.3g ground acceleration.

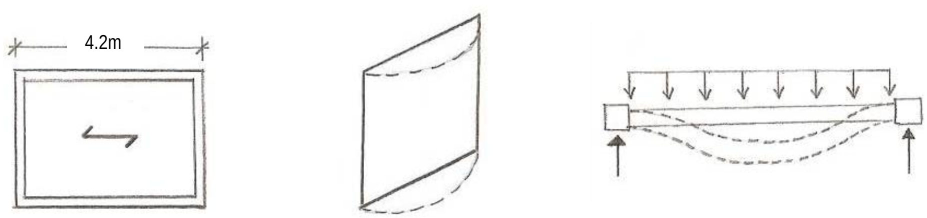
6.0 Out-of-Plane Behavior of Masonry Walls

The seismic surface pressure on the masonry walls was taken as the maximum of 0.4*Sds*Weight of the wall or 0.1*Weight of the wall per ASCE-7 and compared to 1.6*Wind Load specified in the Chinese code. The seismic pressure governed over the wind pressure and was used to compute out-of-plane bending demands.

The out of plane behavior of the shear walls was considered several ways, some more conservative than others. The flexural capacities were calculated using the flexural material properties for masonry with M5 mortar specified in GB50003 (horizontal span flexural strength f'tp = 33.4 psi or 0.23 MPa; vertical span flexural strength f'tn = 16.0 psi or 0.11 MPa) and a compressive strength of 690 psi or 4.76 MPa determined from Table 2105.2.2.1.1 of IBC 2006.

6.1 Horizontal Span Only

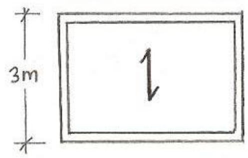
As a first, very conservative pass, the masonry wall was considered initially as an unreinforced fixed end beam spanning horizontally between tie columns spaced 4.2m apart (the longest spacing of tie columns for this house configuration). The wall was considered to have fixed ends due to the toothed connection between the tie columns and masonry. *The flexural capacity of the wall using the horizontal span flexural strength (f'tp) and a strength reduction factor of 0.6 was determined to be 80% of flexural demand at the ends, and therefore insufficient.*



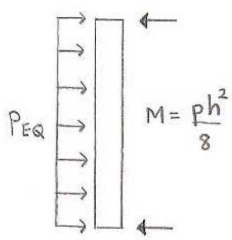
The same calculation was then performed including the capacity of the 6mm diameter U-shaped horizontal bed joint reinforcement spaced every 7 courses at the ends of the wall connecting it to the tie columns. *In this case, the flexural capacity of the wall ends (using a strength reduction factor of 0.9) was determined to be higher than the flexural demands, demonstrating that the wall has sufficient capacity to span horizontally. This is of course an extremely conservative calculation because in reality the wall will behave as a two-way span.*

6.2 Vertical Span Only

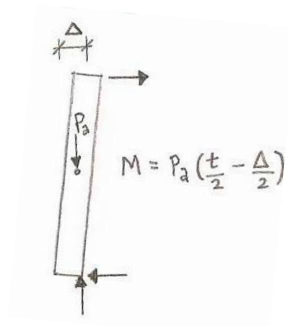
The masonry wall was then conservatively considered as a 3m unreinforced pinned end beam spanning vertically between the plinth beam and ring beam/roof. The wall was considered to have pinned ends in this direction because there is no toothing or reinforcement connecting the masonry to the concrete beams above and below. *The flexural capacity of the wall using the vertical span flexural strength (f'tn) and a strength reduction factor of 0.6 was determined to be 44% of the flexural demand at the middle of the wall, and therefore insufficient.* This calculation did not account for any dead load on the wall.



The flexural capacity was then recalculated (and increased) to include the beneficial effect of compression on the wall from 90% of the tributary concrete roof loads and 90% of the self weight of the wall at mid-height. The flexural demands were also recalculated (and reduced) to account for the effect of the restoring moment from the dead loads on the wall (as described in Section 7.3.2 of Drysdale). Because the roof is a rigid diaphragm, the moment of the ring beam (Δ) was considered to be negligible although the deflection of the wall itself in vertical bending was considered. *Based on this calculation, the wall was determined to have sufficient capacity to span vertically, although is also not representative of the actual two-way out-of-plane behavior of the wall.*

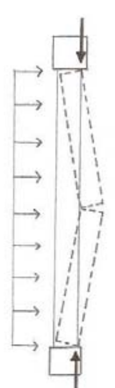


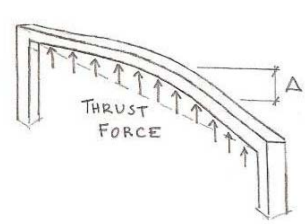
Out-of-Plane Bending Moment



Restoring Moment

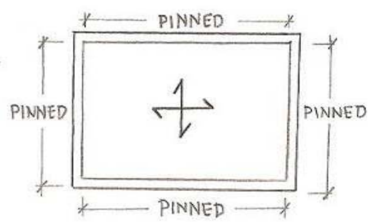
The effect of "arching" behavior in the vertical-span direction, as described in Section 7.5 of Drysdale, was also investigated. *It was determined that with the inclusion of the resistance of the weight of the concrete roof and the additional stiffness of the ring beam from the effective flange of roof slab, the thrust force from the arching effect would vertically deflect the ring beam approximately 3cm. It is likely that this deflection is low enough to allow for arching action to occur, although it is difficult to quantify how much could occur. If the concrete roof were replaced with a lightweight timber roof, it is unlikely that this beneficial arching effect would be possible.*





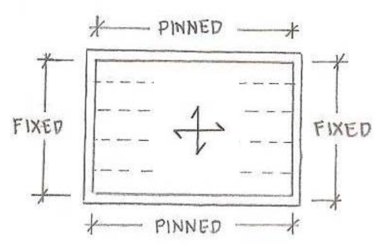
6.3 Two-Way Action

Each masonry shear wall was then considered as a two-way span using the Elastic Plate Analysis described in Section 7.4 of Drysdale. This calculation assumed that the top and bottom as well as the sides of each wall had pinned supports (although this is not fully accurate), and it used moment coefficients to determine the maximum permitted uniform pressure on the wall based on a sharing of the load in the horizontal and vertical directions. The calculation also disregarded the contribution of the horizontal bed joint reinforcement at the wall ends.



According to this analysis, all the walls except the 3.6m wide and 4.2m wide walls have sufficient capacity to work in two-way action under the ASCE-7 prescribed uniform seismic pressure. When the beneficial contribution of the compressive dead load on the walls is included, all walls including the longest walls have sufficient capacity.

Two-way action of each wall was also considered using the Cross Strips Method described in Section 7.4.4 of Drysdale. This calculation determines the out of plane capacity of each wall as the sum of the capacities of vertically-spanning strips and horizontally-spanning strips based on deflection compatibility at midspan. In this calculation, full end fixity of the horizontally-spanning strips was assumed due to the presence of horizontal bed joint reinforcement connection to the tie columns. The horizontal strip capacities at the fixed ends included the contribution from the horizontal bed joint reinforcement, although the midspans of the horizontal strips were considered unreinforced. The vertical strips were considered as simply-supported unreinforced spans.

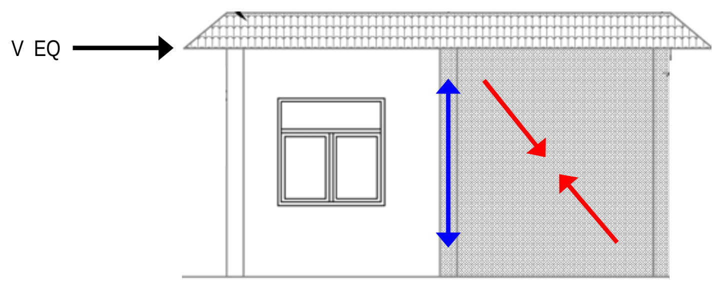


According to this analysis, all of the walls have twice the out-of-plane flexural capacity required. The wall lengths could be increased to up to 5m and maintain sufficient out-of-plane flexural capacity.

This calculation is likely the most accurate of the various calculations for out-of-plane behavior of the masonry walls as it accounts for two way action, different support conditions on the four sides of each wall and the presence of horizontal bed joint reinforcement. However, it is not completely correct in that the complications of deflection compatibility due to the differing support conditions on the sides versus the top and bottom are ignored, and also the beneficial effect of arching is disregarded.

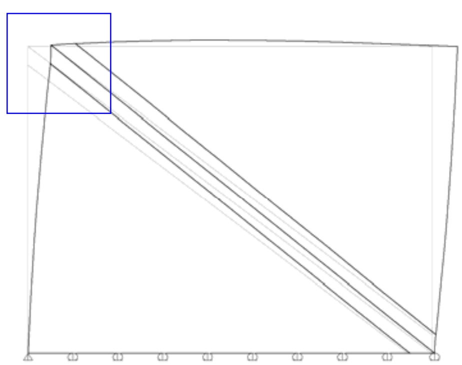
7.0 Design of Tie Columns

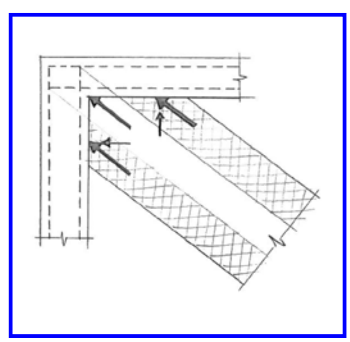
The tension demand in each tie column was calculated as the vertical component of the diagonal compression strut required to transfer the seismic shear in each wall to the ground. Therefore, the total shear load in each wall was assumed to act at the height of the roof even though the portion of the seismic shear from the wall itself would occur at a lower height. The ASCE-7-generated seismic shear forces based on an R factor of 2 were used rather than the values from the GB50011 method.



The highest tension demand, 99 kN or 22kips, in the tie columns associated with the longest N-S direction shear wall, was less than the tension capacity of the column based on four 12mm diameter HBR335 (300 MPa or 43.5ksi) steel bars.

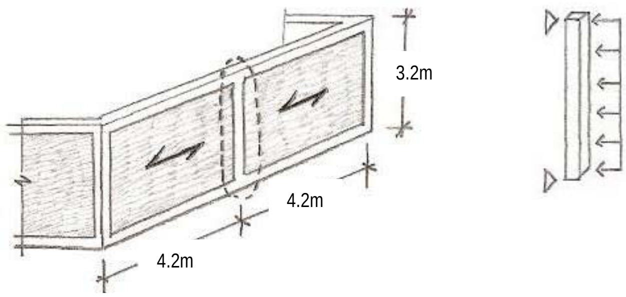
The longitudinal and transverse reinforcement in the tie columns was also checked for the moment and shear demands resulting from the transfer of the compression strut forces to the beam-column joints. These forces were approximately determined by dividing the calculated effective width of the compression strut into three zones and assigning one of the outer zones as a point load on the tie column located at a distance from the beam-column joint. The zone of force acting on the tie column and the distance of this force from the joint were determined based on the specific geometry of the shear wall. Only the shear wall with the highest shear forces (ie the longest N-S shear wall) was considered in this calculation.





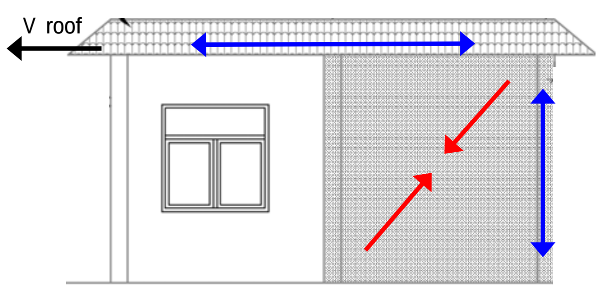
The bending moments induced in the tie column from the point load are small because the distance of the load from the face of the joint is small. *However, the shear forces on the tie column from this load are significant, and the current design which has 6mm diameter HPB235 steel stirrups spaced at 100mm on center at the tie column ends, is not sufficient for these forces. The stirrups would need to be either spaced at 50mm on center or a larger diameter.*

The tie columns were also checked for out-of-plane bending and shear resulting from a uniform seismic surface pressure acting on a 4.2m length of tributary masonry walls conservatively assumed to be spanning horizontally only (note: the tributary length used was larger than the largest in this house but may exist for other house configurations). In the calculation, the tie columns were assumed to be unbraced by perpendicular walls and spanning vertically between the ground and the ring beam as simply-supported beams. *The moment and shear demands resulting from this scenario are relatively low and do not govern the design.*



8.0 Design of Ring Beams

The ring beams were designed for the tension forces resulting from their function as collector elements to transfer seismic shear forces from the roof into the shear walls as well as for the tension forces resulting from their function as diaphragm chord elements (a smaller force). *Four 12mm diameter HRB335 bars are sufficient for these forces.*

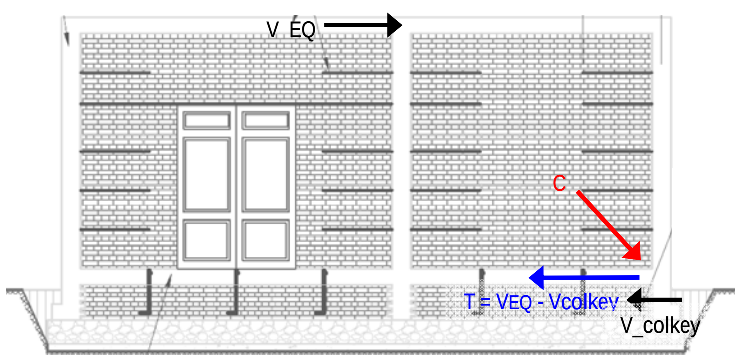


The ring beams were also checked in a similar manner to the tie columns for the shear and bending moment resulting from the transfer of the compression strut force to the beam-column joint for the shear wall with the highest seismic shear demand. The force transferred into the beam was assumed to be a point load acting at a distance from the beam-column joint determined from the specific configuration of the wall including the strut angle and the effective width of the compressive strut.

The flexural capacity of the ring beams is sufficient for the flexural demands. The shear capacity of the beams, based on 6mm diameter HPB235 steel stirrups spaced at 100mm, is also sufficient.

9.0 Design of Plinth Beam

There are no significant bending or shear forces on the plinth beam because it is sandwiched between the wall and the foundation and has sufficient load on it to prevent it from decoupling from the foundation. The plinth beam longitudinal reinforcement was selected to resist a tension force equal to the maximum seismic shear on a shear wall minus the shear capacity of the column key into the masonry foundation because the plinth beam must transfer and distribute this force to the other column keys and the evenly spaced anchor bars connecting the plinth beams to the foundation. *Four 12mm diameter HPB335 longitudinal bars are sufficient for this purpose. The plinth beam stirrups are assumed to be spaced at 150mm on center near the column joints and 200mm elsewhere as there is no significant shear in the plinth beam.*

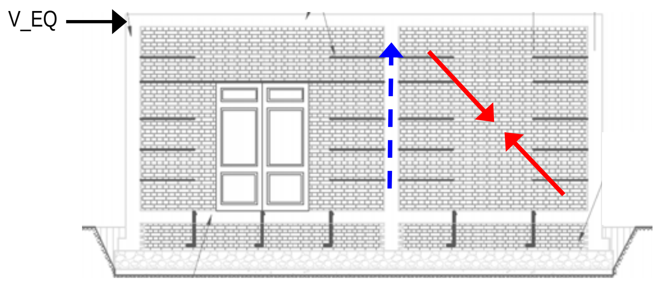


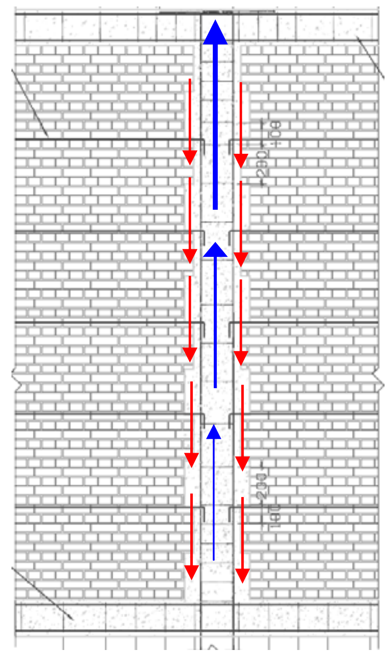
10.0 Design of Connections

The connections were designed according to the ASCE-7-generated seismic loads with an R factor of 2.

10.1 Uplift in Column-to-Foundation Connections

A calculation was performed to assess whether any tie column would be in tension at its base where it connects to the foundation, as this tensile force would need to be transferred to the foundation. In order to assess this, the shear transfer capacity of a tie column-wall interface was calculated as the combination of the shear capacity of the bricks for 50% of the interface (due to the toothing) plus the shear capacity of the horizontal bed joint reinforcement connecting the masonry walls to the tie columns. For each tie column, the maximum tension demand calculated in Section 7.0 was compared to the shear resistance provided by the columns' interfaces with the surrounding walls (either 1, 2 or 3 interfaces depending upon the column). For each column, it was determined that the shear transfer across these interfaces which would engage the dead load of the structure to resist the uplift was higher than the tension force in the column. This calculation was conservative in that it did not assume any additional resistance due to the connection between the ring beams and tie columns.

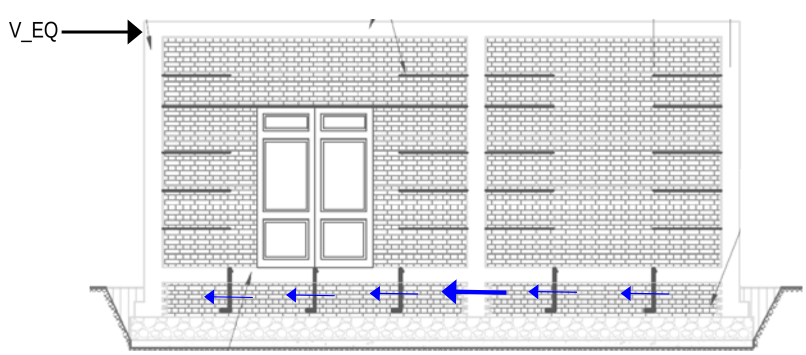




Based on this calculation, it is not likely that the column connection to the foundation would need to resist uplift forces. Nevertheless, the connection is toothed to provide some shear transfer between the concrete and brick, although no bed joint reinforcement is used for this purpose.

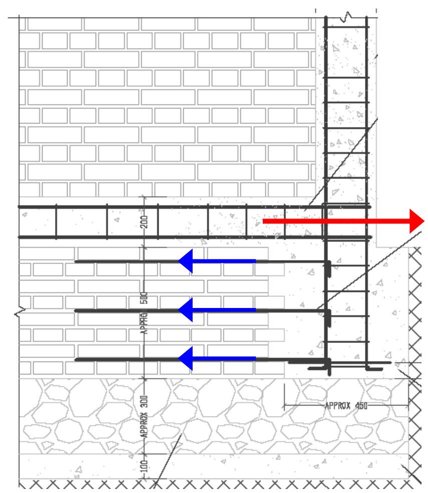
10.2 Shear Transfer to Masonry Foundation

The seismic shear force in each shear wall was compared to the total shear capacity of the interior column 'keys' into the foundation located in line with the shear wall to determine whether additional dowel bars between the plinth beam and foundation would be needed. *For most of the walls, the interior column 'keys' provided sufficient shear transfer; however several walls required additional shear transfer capacity so it was determined that 12mm diameter dowel bars would be used at 1.0m spacing under all walls.* These bars serve to distribute the shear forces more evenly into the foundation and also provide better out-of-plane connection between the plinth beam and the foundation.



10.3 Column Key Connection to Masonry Foundation at Corners

If a tie column is not located at the corner of the building, its 'key' has masonry on either side which allows a transfer of the shear force into the adjacent foundation through bearing. However, at the corners of the building and at the ends of walls, there is masonry on only one side of the column 'keys' and they must be able to transfer tension force across the interface with the adjacent masonry foundation. The tension demand across the interface is the shear capacity of the column key as this is the maximum load that can be transferred into the key and would need to be transferred back to the foundation. *It was determined that six layers of 6mm diameter HPB235 U-shaped horizontal bed joint reinforcement would be required within the height of the foundation to transfer this force. Because it is not feasible to fit more than three layers of horizontal bed joint reinforcement in the height of the foundation, the shear transfer at the exterior column keys is assumed to be reduced to equal the tensile capacity of the three layers of bars, and the plinth beam and the dowel bars to the foundation are checked for the additional loads that result from the reduced shear transfer.*



10.4 Tensile Connection Between Cross Walls to Prevent Rigid Overturning Failure of Wall

The tension capacity of the horizontal bed joint reinforcement between the tie columns and the masonry walls was checked against the tensile demands resulting from the out of plane bending of the walls assuming a 4.2m tributary length of wall spanning horizontally only to ensure that there is adequate connection between orthogonal walls. Even with the highly conservative assumption that the wall spans horizontally only and the neglecting of the direct transfer of force between orthogonal ring and plinth beams, the tension demands are significantly lower than the tension capacity; therefore rigid overturning of a wall is not a likely failure mechanism.

11.0 Design of Foundations

The foundations were checked for a combination of the ASCE-7-generated seismic loads and gravity loads. The maximum compressive stress resulting from 0.7 times the seismic loads plus 1.0 times the gravity loads was lower than the assumed allowable bearing pressure of 72 kN/m2 or 1.5ksf. Additionally, the foundation experienced no uplift from the combination of 0.7 times the seismic loads plus 0.6 times the gravity loads. The width of the foundations may need to increase based on the specific soil conditions at each site.

12.0 Design of Concrete Roof

The concrete roof was designed using a combination of Chinese and US code provisions. Per GB50010, the minimum allowable thickness of a floor slab without beams is 150mm. The 150mm concrete roof was designed as a two-way edge-supported slab according to the Coefficient Method (Method 3 from the 1963 ACI Code) described in Chapter 12 of Nilson.

Per GB50010, the maximum allowable reinforcement spacing is 200mm. The required area of positive flexural steel was governed by the ACI 318 requirements for temperature and shrinkage, and selected as 8mm diameter bars at 150mm spacing. The required area of negative flexural steel was governed by the flexural demands and selected to meet the minimum negative reinforcement requirement in GB50010 of 8mm diameter bars spaced at 200mm on center.