

APPENDIX 3: STRUCTURAL DESIGN NARRATIVE FOR CONFINED MASONRY HOUSING IN HAITI

**CALCULATION REPORT FOR CONFINED MASONRY HOUSING
(narrative only, structural calculations not included¹)**

**Build Change Post-Earthquake Housing Reconstruction Technical Assistance
Program, Haiti**

Prepared for

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¹ For the complete document, including appendices of drawings, go to www.buildchange.org/USAIDPrimers.html .

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1.0 INTRODUCTION

The following is a narrative of the structural design calculations that were performed for a variety of single-story and two-story confined masonry house configurations with both lightweight timber-framed and concrete flat slab roofs to arrive at a set of design rules and guidelines for use in the construction of confined masonry permanent housing in Haiti.

This narrative provides a summary of the design of the primary gravity and lateral load carrying systems including the masonry bearing walls, confining elements and their reinforcement and roof systems. Also included is additional information regarding the design of the foundations, porches and stairs.

1.1 General Methodology

Confined masonry is not a recognized engineered structural system in most building codes. Therefore, these designs referenced US-based design standards and specifications for reinforced concrete and masonry while drawing from current and past research and testing results on confined masonry systems. The designs also relied upon the provisions for confined masonry provided in the Mexican Building Code. Existing prescriptive confined masonry guidelines, including those recently developed by the Haitian Ministry of Public Works (MTPTC) were also consulted but not used directly as the basis of the design.

Two general house categories were considered in the calculations and a set of design and construction guidelines was developed for each:

- Single-story house with lightweight timber frame roof
- Single-story house with concrete roof OR two-story house with either a timber frame or concrete roof

For each house category, the structural load demands in elements were checked for numerous floor plan configurations complying with the design guidelines developed for that house category, and the elements were sized for the greatest demands resulting from the most severe configuration. Therefore, all confining elements for the single-story house with lightweight roof have the same reinforcement and detailing requirements, and all confining elements for the single-story house with concrete roof or two-story house have the same reinforcement and detailing requirements. This approach was taken to minimize the complexity of the guidelines and to allow a simple, easy-to-follow and repeatable standard.

Where possible, an effort was also made to unify the guidelines for the single-story house with concrete roof / two-story house with the design guidelines developed for the same system by the MTPTC in order to minimize inconsistency between two design standards. For example, the MTPTC detailing guidelines for ring beams and tie column reinforcement were followed and Grade 60 rather than Grade 40 steel was used for confinement in this system. The guidelines for the single-story house with lightweight roof deviate from the MTPTC guidelines in order to reduce the cost of this type of house to the greatest extent possible.

2.0 DESIGN CRITERIA

This section provides a summary of the codes and references consulted as well as the material properties and loading criteria used in the design of the confined masonry house systems and development of building construction guidelines.

2.1 References

The design of the one- and two-story confined masonry houses references provisions from US-based codes, the Mexican Building Code, the International Building Code, the Association of Caribbean States' Building Code, as well as various recommendations from research on confined masonry structures. The references consulted in the design include the following:

2.1.1 Loads

- Pan American Health Organization (PAHO) Wind Speed Maps for the Caribbean for Application with the Wind Load Provisions of ASCE 7, 2008
- United States Geological Survey (USGS) Documentation for Initial Seismic Hazard Maps for Haiti, 2010
- United States Geological Survey (USGS) Worldwide Seismic "Design Maps" Web Application, Beta version, 2010
- American Society of Civil Engineers' Minimum Design Loads for Buildings and Other Structures, SEI/ASCE 7-05, 2005
- International Code Council's International Building Code (IBC), 2009

2.1.2 Confined Masonry and Masonry Design

- Mexican Building Code's "Complementary Technical Norms for Design and Construction of Masonry Structures"
- American Concrete Institute's Building Code Requirements for Masonry Structures (ACI 530-05)
- The Masonry Society's Building Code Requirements for Masonry Structures (TMS 602-08), 2008
- ASCE/SEI 41-06 Seismic Rehabilitation of Buildings, which includes strut-and-tie provisions for infill walls
- "Out of Plane Resistance of Concrete Masonry Infilled Panels" by Dawe and Seah, 1988
- "Arching of Masonry Infilled Frames: Comparison of Analytical Methods" by Flanagan and Bennett, 1999
- "Behavior of Confined Masonry Shear Walls with Large Openings" by Yanez et al, 2004
- "Seismic Design Guide for Confined Masonry Buildings - Draft" by the Confined Masonry Network, 2010
- "Seismic Design Guide for Masonry Buildings" by Anderson and Brzev, Canadian Concrete Masonry Producers Association, 2009

2.1.3 Concrete Design

- American Concrete Institute's Building Code Requirements for Structural Concrete (ACI 318)

2.1.4 Wood Design

- American Forest and Paper Association Wood Frame Construction Manual for One and Two-Family Dwellings (WFCM-01), 2001
- American Forest and Paper Association National Design Specification for Wood Construction with 2005 Supplement (NDS-05), 2005
- American Forest and Paper Association Special Design Provisions for Wind and Seismic (ANSI/AF&PA SDPWS-08), 2008

2.1.6 Other References Consulted

- Association of Caribbean States' Model Building Code for Wind Loads, 2003
- Association of Caribbean States' Model Building Code for Seismic Loads, 2003
- Caribbean Uniform Building Code (CUBiC), 1985
- European Standard for Earthquake Resistant Design of Structures (Eurocode 8), 2003
- Small Building Code of Trinidad and Tobago 2000
- Organization of Eastern Caribbean States Building Guidelines, 1999
- Minimum Building Standards and Environmental Guidelines for Housing, Safer Housing and Retrofit Program, St Lucia National Research and Development Foundation, 2003
- Architecture for Humanity Rebuilding 101 Manual, 2010
- "Seismic Behavior of Confined Masonry Walls" by Tomazevic and Klemenc, 1997
- "Verification of Seismic Resistance of Confined Masonry Buildings" by Tomazevic and Klemenc, 1997
- "Effect of Vertical and Horizontal Wall Reinforcement on Seismic Behavior of Confined Masonry Walls" by Yoshimura et al, 1996
- "Design of Confined Masonry Walls Under Lateral Loading" by Bariola and Delgado, 1996
- "Simplified Method for Seismic Analysis of Masonry Shear-Wall Buildings" by Tena-Colunga and Cano-Licona, ASCE Journal of Structural Engineering, May 2010
- "Use of Nonlinear Static Analysis for the Displacement-based Assessment of Confined Masonry Buildings" by Teran-Gilmore et al, 2010
- "Simplified Drift-Based Fragility Assessment of Confined Masonry Buildings" by Ruiz-Garcia et al, 2010
- "User's Guide to NZS 4230:2004, Design of Reinforced Concrete Masonry Structures", New Zealand Concrete Masonry Association Inc, 2004
- Various prescriptive confined masonry guidelines including those developed by the following organizations or individuals: City University, Marcial Blondet, Tom Schacher, and Build Change (for programs in Indonesia and China)

2.2 Materials

Material properties for the masonry wall, concrete, reinforcing steel and timber used in the calculations were selected from the US and international codes based on the properties of

materials commonly available in Haiti. According to field surveys, the following material types are locally available in Haiti:

2.2.1 Concrete Block

- "Bloc 15" type, dimensions are 40 cm x 20 cm x 15 cm
- "Bloc 20" type, dimensions are 40 cm x 20 cm x 20 cm
- "Bloc 10" type, dimensions are 40 cm x 20 cm x 10 cm
- Density: 2400 kg/m³
- Compressive strengths considered: 4.8 MPa (700 psi), 6.9 MPa (1,000 psi), 11.7 MPa (1700 psi)
- Modulus of elasticity of the masonry/mortar matrix: 2,700 MPa – 6,500 MPa (392,000 psi – 942,300 psi)

2.2.2 Concrete Block Masonry

- Mortar Type M assumed (17 MPa or 2500 psi compressive strength)
- Compressive strength (f'_m) (net area): 3.86 MPa (560 psi), 5.52 MPa (800 psi), 9.28 MPa (1,346 psi) for the three block strengths listed above based upon Unit Strength Method (Table 2105.2.2.1.1) of IBC 2009
- Tensile strength: 431 kPa (63 psi) vertical, 862 kPa (125 psi) horizontal based on modulus of rupture strength for Strength Design listed in ACI 530 Table 31.18.2.1

2.2.3 Grout

- Density: 2400 kg/m³
- Compressive strength assumed: 13.8 MPa or 2000 psi (ASTM C476)
- Modulus of elasticity: 17,575 MPa

2.2.4 Plaster

- Density: 2,400 kg/m³
- Compressive strength assumed: 17 MPa or 2500 psi
- Modulus of elasticity: 19,650 MPa

2.2.5 Concrete

- Density: 2,400 kg/m³
- Design compressive strength (f'_c) for confining elements and roof: 17 MPa (2,500 psi) although mix proportions specified may provide higher actual strength
- Design compressive strength (f'_c) for foundations: 15 MPa or 2,200 psi
- Modulus of elasticity: 19650 MPa

2.2.6 Steel Reinforcement

- Grade 40 bars ($f_y = 276$ MPa or 40 ksi) or Grade 60 bars ($f_y = 414$ MPa or 60 ksi)
- All bars are #4 or smaller
- All reinforcement is ribbed

2.2.7 Timber

- Visually Graded Southern Pine No. 2
- Density: 550 kg/m³ (specific gravity based on weight and volume when oven-dry)
- Design Bending Stress (F_b): 10.3 MPa (1,500 psi)
- Design Tension Stress (Parallel to Grain) (F_t): 5.7MPa (825 psi)
- Design Shear Stress (Parallel to Grain) (F_v): 1.2 MPa (175 psi)
- Design Compression Stress (Perpendicular to Grain) (F_{c+}): 3.9 MPa (565 psi)
- Design Compression Stress (Parallel to Grain) (F_c): 11.3 MPa (1,650 psi)
- Modulus of Elasticity (E): 4,000 MPa (580,000 psi)

2.3 Loads

2.3.1 Dead Loads

Dead loads include the self weight of building materials including concrete block masonry walls, reinforced concrete confining elements, timber frame roof with CGI panels, and foundations.

These calculations make the following assumptions regarding the dead loads of the structure in order to calculate gravity and seismic loads:

- Confined masonry walls: 2.25 kPa (47.0 psf) estimated, including confining elements, bed joints and grouted cells (additional load due to plaster also considered in cases where plaster is used)
- Concrete flat slab floor/roof with waterproofing: 4.0 kPa (84 psf)
- Timber frame roof with CGI panels: 1.05 kPa (22 psf)

2.3.2 Live Loads

A live load of 1.0 kPa (20 psf) is assumed on timber frame roofs and 2.5 kPa (50 psf) is assumed on flat concrete slab floors/roofs.

2.3.3 Wind Loads

Wind pressures for the main structural system and roof cladding were calculated using the Simplified Procedure (Method 1) of ASCE 7-05 in combination with the Basic Wind Speed of 119 mph provided by the *Pan American Health Organization (PAHO) Wind Speed Maps for the Caribbean for Application with the Wind Load Provisions of ASCE 7, 2008*. Exposure Category C was assumed as well as an Importance Factor of 1.0. The maximum wind pressure calculated for any region of a surface was taken as the governing load.

- Lateral Wind Pressure on Walls: 1.87 kPa (39 psf) maximum
- Wind Pressure (down) on Gable/Hip Roof Structure: 0.14 kPa (3 psf) maximum
- Wind Pressure (up) on Gable/Hip Roof Structure: 1.72 kPa (36 psf) maximum
- Wind Pressure (down) on Roof Cladding: 0.67 kPa (14 psf) maximum
- Wind Pressure (up) on Roof Cladding: 1.91 kPa (40 psf) maximum
- Wind Pressure (up) on Roof Cladding corners: 3.01 kPa (63 psf maximum)

2.3.4 Seismic Loads

The seismic design loads were determined using the short period spectral acceleration (Ss) data provided in the 2010 USGS Worldwide Seismic “Design Maps” Web Application in combination with the equivalent lateral force procedure of ASCE 7-05. The seismic loads correspond to a 2% probability of exceedance in 50 years.

Two sets of seismic criteria were considered in the designs. The first set (Orange Zone), corresponding approximately to a peak ground acceleration of 0.6g, covers Port-au-Prince, where most of the reconstruction is likely to occur. The second set (Red Zone), corresponding approximately to a peak ground acceleration of 1.0g, covers the more severe seismic hazard areas to the north and west of Port-au-Prince, which for the most part are outside of the zones affected by the January 2010 earthquake.

Orange Zone Seismic Criteria

- Mapped MCE Short Period Spectral Response Acceleration (Ss): 1.58g (corresponds approximately to a peak ground acceleration of 0.6g for 2% in 50 years)
- Site Class D ($F_a = 1.0$)
- Short Period Design Spectral Response Acceleration (Sds): 1.05g

Red Zone Seismic Criteria

- Mapped MCE Short Period Spectral Response Acceleration (Ss): 2.51g (corresponds approximately to a peak ground acceleration of 1.0g for 2% in 50 years)
- Site Class D ($F_a = 1.0$)
- Short Period Design Spectral Response Acceleration (Sds): 1.67g

The following map shown in Figure 2.1, which was developed using data gathered in 2010 by USGS, indicates regions of Haiti where either the Orange Zone or Red Zone seismic criteria apply. The Red Zone covers all regions of Haiti including those with the highest anticipated ground motions. The Orange Zone covers a majority of Haiti, including most areas in the earthquake-affected zone around Port-au-Prince. The zones shown in yellow on the map have lower expected ground motions. These zones were not a focus of the design because they are not in the earthquake-affected zone.

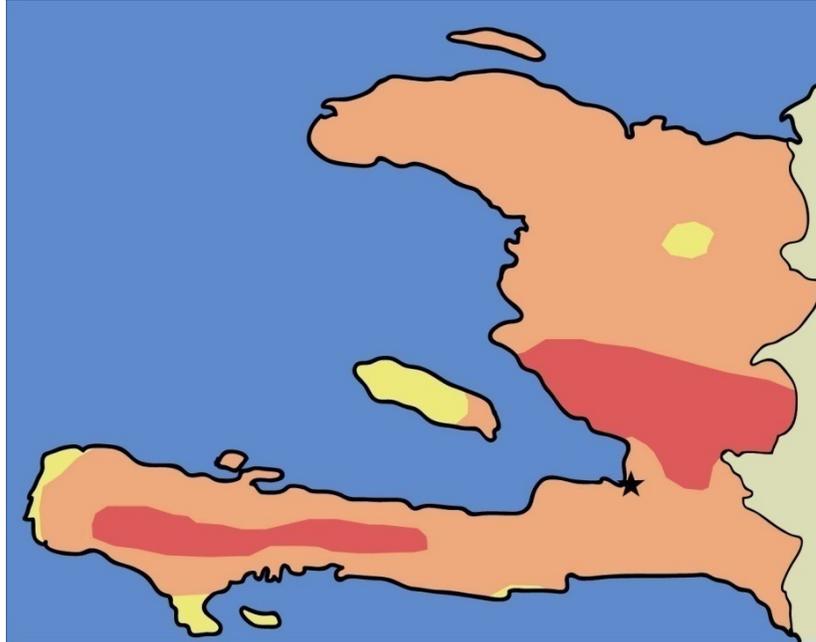


Figure 2.1 Map of Haiti showing severity of seismic demands by region based upon 2010 USGS data (Sds = 0.5g for yellow zones, Sds = 1.05g for orange zones, Sds = 1.67g for red zones)

The Response Modification Coefficient (R Factor) that was used to arrive at seismic demands on the structure varied between 2.5 and 3.0 depending upon the strength of concrete block assumed in the design. Where concrete block strengths were specified ranging from 6.9 MPa (1000 psi) to 11.7 MPa (1700 psi) (ie strengths which are generally consistent with accepted design standards and test data) an R-factor of 3.0 was used, based upon the recommendation of the Confined Masonry Network's "Draft Seismic Design Guide for Confined Masonry Buildings". Where a lower concrete block strength of 4.8MPa (700 psi) was assumed, a more conservative R-factor of 2.5 was selected.

3.0 DESIGN METHODOLOGY

Due to the significant variation in strength of concrete blocks available in Haiti as well as the large range of ground motions expected in Haiti based upon current USGS maps, we approached the design by coupling the use of a specific concrete block strength with an anticipated seismic performance for one set of design and construction guidelines (and house layouts) for the single-story case and one set for the two-story case.

The primary benefit of this approach is that the use of one set of design and construction guidelines should simplify its implementation and reduce misunderstanding, thereby allowing it to be more widespread, understood, and, ideally, replicated. This approach also allows for an understanding of the improvement in the seismic performance that could be achieved if the concrete block manufacturing industry in Haiti is more tightly controlled. The present poor quality of most blocks in Haiti makes it near-impossible to design for the highest anticipated ground motions in Haiti; however, it is reasonable to assume that the quality control standards for the production of these blocks may improve over time,

enabling an increased seismic performance for an identical house configuration built with better blocks.

On this basis, the following cases were considered:

Cases Considered for Single Story with Lightweight Roof

| CONCRETE BLOCK STRENGTH | SEISMIC DESIGN CRITERIA |
|-------------------------|---|
| 4.8 MPa (700 psi) min | Permitted in all zones (Sds = 1.05g to 1.67g) |

Cases Considered for Single Story with Concrete Roof/Two Story

| CONCRETE BLOCK STRENGTH | SEISMIC DESIGN CRITERIA |
|--------------------------|---|
| 4.8 MPa (700 psi) | Not permitted |
| 6.9 MPa (1,000 psi) min | Permitted in orange and yellow zones only (Sds = 1.05g) |
| 11.7 MPa (1,700 psi) min | Permitted in all zones (Sds = 1.67g) |

Using the above criteria, one set of design and construction guidelines and proposed house layouts was developed for the single-story house with a lightweight timber frame roof, and a second set of design and construction guidelines and proposed house layouts was developed for the single-story house with concrete roof / two-story house.

The guidelines also include provisions for vertical and horizontal expansion. The additional house layouts in the single-story house with timber frame roof provide examples of options for horizontal expansion. Vertical expansion is permitted only in the single-story house with a concrete roof. The single-story house with a timber frame roof is not designed for the additional load of a second story.

The following sections describe the calculation methodology used to arrive at the set of design and construction guidelines and proposed house layouts. The design of all masonry and concrete elements is based upon strength design principles. The design of timber frame roof and determination of foundation dimensions are based upon allowable stress design principles.

The in-plane shear behavior of the confined masonry walls is the governing factor in the house layouts and design and construction guidelines due to the generally low concrete block masonry compressive strength. Out-of-plane behavior of the confined masonry walls is less of a factor due to the relatively low wall height-to-thickness ratio resulting from the 15cm-wide concrete block.

3.1 In-Plane Shear Design

The in-plane design of the confined masonry walls was based upon the provisions of the Mexican masonry code for in-plane shear capacity of confined masonry systems. For a variety of house layouts coupled with a variety of concrete block strengths and seismic loads, the in-

plane seismic shear force demand was calculated for each wall and compared to its capacity. The set of guidelines were developed based upon an evaluation of this data set.

For these calculations, all masonry walls, including those with openings, were considered to be shear-resisting elements and assumed to be designed and detailed according to the design guidelines. In these calculations, the wall lengths were divided into individual 'shear walls' at wall ends, corner, windows or doors. The guidelines for tie column placement and reinforcement requirements around openings were developed to always result in individual 'shear walls' with tie columns at their extreme ends and at most one window opening centered on a wall panel between tie columns. This permitted direct application of the in-plane shear capacity provisions of the Mexican masonry code as well as the recommendations of Yanez et al, which tested only individual confined masonry panels with a single large opening. The following figures demonstrate the subdivision of a longer wall length into individual shear walls for the purposes of the calculation:



Figure 3.1 Examples of subdivision of walls into individual shear walls (each bounded by tie columns with at most one opening) for in-plane shear calculations

3.1.1 In-Plane Shear Demand

The shear force demands on the shear walls were calculated according to the equivalent lateral force procedure in ASCE 7-05. The base shear was calculated using the effective weight of the confined masonry walls and the floor/roof system. No Redundancy Factor was assumed in calculating the seismic shear demands.

For the lightweight roof cases, a flexible diaphragm assumption (ie tributary area) was used to distribute the seismic load of the roof to each wall, in addition to its own seismic load. Seismic load due to orthogonal walls was also distributed to shear walls based on tributary area, assuming they are spanning approximately horizontally between the perpendicular shear walls.

For the concrete roof cases, a rigid diaphragm assumption was used to distribute all seismic loads from walls, floor and roof to the shear walls. For this case, the reduction in wall stiffness due to openings and its influence on shear distribution was accounted for by using finite element models built in SAP2000 for the primary house configurations to determine the rigidity of each wall (the increased relative stiffness of short wall lengths due to the high tie column-to-wall area ratio was neglected). A 5% accidental torsion was also considered per ASCE 7-05.

3.1.2 In-Plane Shear Capacity

The in-plane shear capacities were calculated according to Mexico's "Complementary Technical Norms for Design and Construction of Masonry Structures" using the following equation (where P represents the axial loading on the wall, v_m represents the shear strength of the masonry-mortar matrix based upon gross area, A represents the gross area of the confined masonry wall, and R is a reduction factor):

$$V_{mR} = R_F (0.5 v_m * A_T + 0.3P) < 1.5 F_R v_m * A_T$$

The value v was calculated from the compressive strength of the masonry-mortar matrix according to $0.25 * \text{sqrt}(f'm)$ [MPa]. The Mexican code places a limit on v of 0.6 MPa.

Using this methodology it was determined that the single-story house with a lightweight timber roof constructed from 700 psi concrete block has sufficient in-plane shear capacity for the most severe seismic criteria of $S_{ds} = 1.67g$:

Table 3.1 Shear stress demand vs capacity for single-story house

| Wall Label | Shear Stress Demand [kPa] | Shear Stress Capacity [kPa] |
|------------|---------------------------|-----------------------------|
| A | 73.66 | 137.32 |
| B | 128.96 | 139.78 |
| C | 104.79 | 138.83 |
| D | 104.79 | 138.83 |
| E | 110.05 | 137.42 |
| F | 68.02 | 137.42 |
| 1 | 97.91 | 137.04 |
| 2 | 100.40 | 138.72 |
| 3 | 107.57 | 138.91 |
| 4 | 100.40 | 138.72 |
| 5 | 89.76 | 137.87 |
| 6 | 74.94 | 137.73 |

Using this methodology it was determined that the two-story house constructed from 1000 psi concrete block has sufficient in-plane shear capacity to resist a short period design spectral acceleration of 1.05g. A higher block strength of 1700 psi is required for the more severe seismic criteria of $S_{ds} = 1.67g$:

Table 3.2 Shear stress demand vs capacity for two-story house with concrete roof

| Wall Label | Shear Stress Demand [kPa] With $S_{ds} = 1.67g$ | Shear Stress Capacity [kPa] With 1700 psi blocks |
|------------|--|---|
| A | 206.42 | 229.08 |
| B | 206.42 | 229.08 |
| C | 198.15 | 229.08 |
| D | 179.72 | 228.41 |
| 1 | 224.93 | 230.79 |
| 2 | 224.93 | 230.79 |
| 3 | 214.73 | 236.16 |
| 4 | 214.73 | 236.16 |

| | | |
|---|--------|--------|
| 5 | 214.73 | 236.16 |
| 6 | 214.73 | 236.16 |
| 7 | 224.93 | 225.41 |
| 8 | 224.93 | 225.41 |

3.1.3 Influence of Openings

As described in Section 3.1.1, the influence of openings on the stiffness of the walls was considered in determining the distribution of shear demands to the walls where required. Additionally, the openings were considered in determining the in-plane shear capacity of the walls. Based upon research by Yanez et al which tested the behavior of individual confined masonry shear walls with large openings, it was assumed that the shear capacity of a wall with window opening is proportional its net transverse area.

Therefore, the full shear capacity of each shear wall was calculated according to the provisions of the Mexican masonry code, and for walls with window openings this capacity was reduced proportionally. According to the same research by Yanez et al, the effect of door openings is not well-represented by this methodology; therefore, these calculations assume that each door opening is confined on either side by a tie column, and each masonry wall on either side of the door is considered to be an independent shear wall.

Two cases were developed for the reinforcement requirements around window openings:

Case A: Where a single window is centered on a confined masonry panel and it is the only opening in the shear wall, it is permitted to be reinforced according to the provisions of Section 5.1.3 of the Mexican masonry code and the recommendations found in the research by Yanez et al. Per the Mexican code, horizontal steel bars anchored in the walls may be used as reinforcement at the lower edge of an opening if the bars are designed to withstand a tension force of 29kN. The horizontal reinforcement below these window openings has been designed for this load. Vertical reinforcing at the edges of these window openings was designed based on testing configurations in Yanez et al's research, which used one 10mm diameter bar in a continuous, fully grouted cell of the masonry block on either side of the window opening.

Case B: When window openings are positioned in series along a single wall, every other window must be reinforced with full tie columns and sill beams in order to ensure that the individual shear wall piers created by the openings are bounded by tie columns at their extremities. The following sketch demonstrates the reason for this requirement:

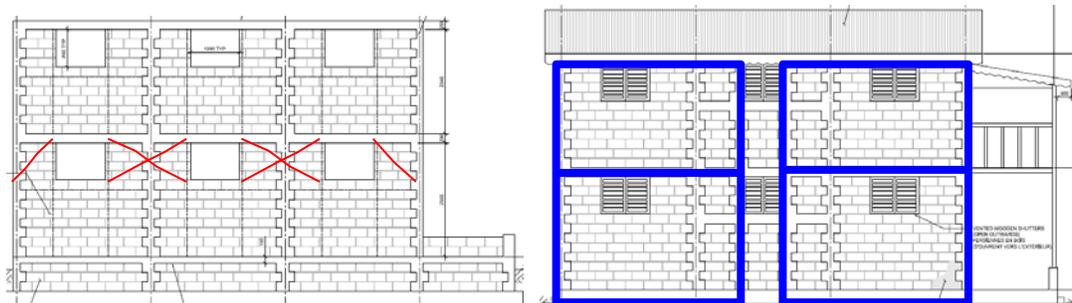


Figure 3.2 Window openings in series without tie columns may result in cracking of the unconfined piers created by the openings. Placing tie columns at every other window in series allows subdivision of walls into individual shear walls (with a maximum of one opening) bounded by confining elements

More details regarding the design of tie columns and wall reinforcement can be found in Section 3.4.

3.1.4 Spacing of Orthogonal Walls

In performing the above in-plane calculations for a variety of configurations for single-story and two-story houses, it was determined that there must be a limit on the spacing of orthogonal walls in order to limit the in-plane shear force distributed to each wall. For the single-story house with a lightweight roof, the maximum spacing of orthogonal walls set at 4m, although the limit in the guidelines was conservatively set at 3.5m. For the single-story house with a concrete roof (or two-story house), the maximum spacing of orthogonal walls cannot be greater than 3.5m.

3.1.5 Influence of Plaster Finish

The influence of adding a 1cm plaster finish to the walls was investigated. The addition of plaster increases the in-plane shear capacity of the walls because the thickness of the wall and therefore A is increased; however, it also increases the seismic demands due to the increased weight. Overall, there is a net benefit to adding plaster (on the order of 5%); however, the guidelines and house layouts were developed conservatively assuming that plaster would not be applied to the walls, although this practice is recommended.

3.2 Axial and In-Plane Wall Flexural Design

The combined axial and in-plane flexural strength of each confined masonry shear wall was calculated according to the confined masonry provisions of the Mexican masonry code and compared to the axial and flexural demands on each wall due to gravity plus seismic loads. This design check was particularly relevant for the two-story house design.

3.2.1 In-Plane Flexural Demands

To determine in-plane flexural demands on each wall, seismic forces were distributed to each wall using the C_v factors of the equivalent lateral force method of ASCE-7. For the lightweight roof case, orthogonal walls were assumed to span horizontally between shear walls; therefore the seismic load from orthogonal walls was assumed to act at the mid-height of the walls in the calculation of C_v factors. For the concrete roof case, the orthogonal walls were assumed to span vertically and horizontally. Therefore the seismic load from orthogonal walls was distributed accordingly in the calculation of C_v factors.

3.2.2 In-Plane Flexural Capacity

The in-plane flexural capacities were calculated according to Mexico's "Complementary Technical Norms for Design and Construction of Masonry Structures" using the following equation:

$$M_R = F_R * M_o + 0.3 * P_u * d \quad \text{where } 0 < P_u < P_R/3$$

$$= [(1.5 * F_R * M_o) + (0.15 * P_R * d)] * [1 - P_u/P_R] \quad \text{where } P_u > P_R / 3$$

where $M_o = A_s * f_y * d'$ and d' is the distance between centroids of tie column steel and where P_R , the compressive strength of the wall, is equal to

$$F_R * F_E * (f'm + 0.4) * A_T$$

The in-plane flexural demands and axial loads on the single story house with lightweight roof were not significant. However, for the two-story case, this equation governed the longitudinal steel requirements of the tie columns.

3.3 Out-of-Plane Design

The seismic surface pressure on the masonry walls was taken as the maximum of $0.4 * S_d * \text{Weight of the wall}$ or $0.1 * \text{Weight of the wall}$ per ASCE-7 and compared to the design wind pressures on the walls. The seismic pressure governed over the wind pressure and was used to compute out-of-plane bending demands.

The out-of-plane capacity of the confined masonry walls was calculated using the equations provided in Flanagan and Bennett based on research by Dawe and Seah with an assumed strength reduction factor of 0.75. For the lightweight roof case, the following equations were used, assuming a three-side supported wall which is not supported at the roof:

$$q_{ult} = 800 * f'm^{0.75} * t^2 * \alpha / L^{2.5}$$

$$\alpha = 1/H * E * I_c * H^2 < 50$$

where "t" represents the full width of the masonry block walls, "H" is the wall panel height, "L" is the wall panel length, and "Ic" is the tie column moment of inertia.

Using this equation, the out-of-plane capacity of a 4m wide by 2.7m tall wall (which is the maximum wall size permitted by the guidelines due to in-plane shear considerations) is 80 kPa (1,670 psf). This capacity is well above the anticipated out-of-plane demands.

For the concrete roof case, the following equations were used, assuming a four-side supported wall:

$$q_{ult} = 800 * f'm^{0.75} * t^2 * \alpha / ((\alpha / L^{2.5}) + (\beta / H^{2.5}))$$

$$\alpha = 1/H * E * I_c * H^2 \leq 50$$

$$\beta = 1/L * E * I_b * L^2 \leq 50$$

where “t” represents the full width of the masonry block walls, “H” is the wall panel height, “L” is the wall panel length, and “I_c” is the tie column moment of inertia, and “I_b” is the ring beam moment of inertia.

Using this equation, the out-of-plane capacity of a 3 m wide by 2.7 m tall wall (which is the maximum wall size permitted by the guidelines due to in-plane shear considerations) is more than 80 kPa, also well-above the anticipated out-of-plane demands.

3.4 Design of Confining Elements

3.4.1 Tie Columns

The longitudinal reinforcement in the tie columns was designed for the governing flexural or axial demands resulting from the following checks:

- 1 Tension due to overturning in the wall resulting from the in-plane flexural strength provisions of the Mexican masonry code (see Section 3.2.2)
- 2 Tension due to strut and tie action (see Section 3.4.1.1)
- 3 Flexure due to strut and tie action (see Section 3.4.1.1)
- 4 Flexure in tie column acting as a vertical beam for out-of-plane support of masonry walls (see Section 3.4.1.2)
- 5 ACI 318 minimum longitudinal steel requirement (1% A_g)

The single-story design was governed by case #3 above. The two-story design was governed by case #1 above.

The transverse reinforcement in the tie columns was designed for the governing shear demands resulting from the following checks:

- 1 Shear due to strut and tie action (see Section 3.4.1.1)
- 2 Shear in tie column acting as a vertical beam for out-of-plane support of masonry walls (see Section 3.4.1.2)

Both the single-story and two-story designs were governed by case #1 above. It should be noted that the maximum spacing requirement in ACI 318 of $d/2$ for shear reinforcement in the confining elements is not satisfied. The small dimension of the confining elements makes application of this requirement impractical.

3.4.1.1 Demands Due to Strut and Tie Action

Although strut-and-tie action was not considered to be the dominant lateral force transfer mechanism for in-plane shear in the confined masonry systems, this behavior was considered in the detailing of reinforcement for the tie columns and ring beams, particularly at their joints.

Although the confined masonry system is not equivalent to an infill frame system, Equation 8-10 of FEMA 306 (or ASCE/SEI 41-06), which defines the shear capacity of an infill wall based on the compressive failure of the diagonal strut, was used to compute the maximum capacity of a diagonal compression strut through each confined masonry wall, and this capacity was used as the maximum demand on the surrounding

confining elements. The following equation was used in combination with an assumed strength reduction factor of 0.75 to compute the horizontal component of the compression strut:

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

where f'_{m90} per FEMA 306 was assumed to be 50% of f'_m . In this equation, “ t_{inf} ” represents the thickness of the compression strut which was assumed to be two-times the wall side wall thickness of the concrete block (ie 5.4cm) on the basis that these side walls are the only continuous parts of the concrete masonry wall. In this equation, “ 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:

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

In this equation, “ t_{inf} ” is assumed to be the full width of the concrete block rather than the reduced value used above.

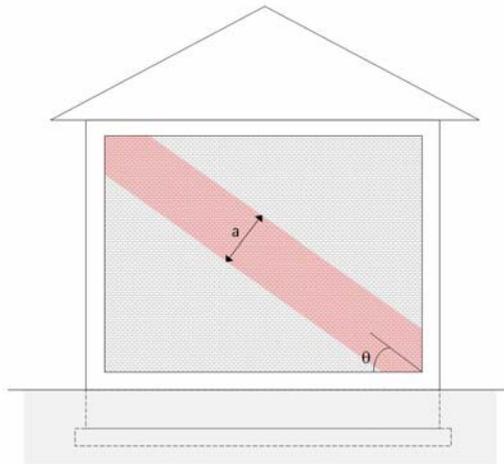


Figure 3.3 Diagonal compression strut in confined masonry wall

Longitudinal reinforcing in the tie column was sized for tension in the tie column resulting from strut-and-tie behavior of the confined masonry shear wall. A minimum of four bars were used to facilitate the placing of the reinforcement. This reinforcing steel was then checked against ACI criteria for minimum longitudinal steel ratio.

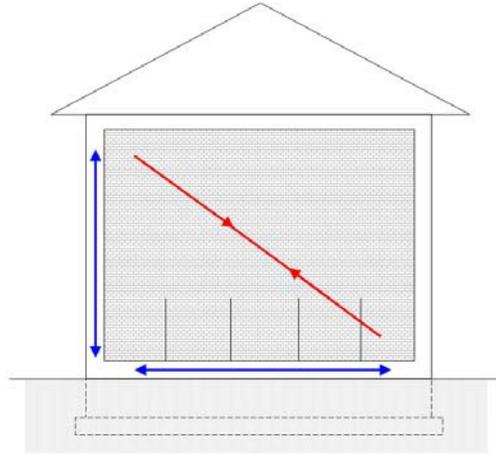


Figure 3.4 Tension in confining elements (blue) resulting from compression strut in masonry wall (red)

Longitudinal reinforcement in the tie column was also checked for the flexural demands resulting from the concentrated loading of the compression strut in the masonry wall near the joints between the tie-columns and ring beams. Figure 3.5 shows the distributed load (ie the total compression strut capacity divided by the length of contact between the strut and confining elements) applied to the tie column to determine the flexural demand. In this calculation, the tie column was assumed to be fixed at both top and bottom. The shear reinforcement in the tie columns near the top and bottom joints was also sized for the shear force demands resulting from this loading condition.

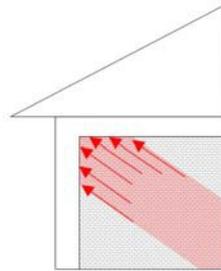


Figure 3.5 Concentrated loading on tie column-ring beam joint from compression strut

3.4.1.2 Demands Due to Out of Plane Action

The flexural and shear reinforcement in the tie columns were also checked for the demands on a tie column resulting from the condition where it is not braced by a masonry wall along both axes and therefore must function as a vertical beam to support one side of a masonry wall panel.

For the single-story house with lightweight roof, in this calculation, the maximum length of wall panel without orthogonal walls bracing both ends was 1.3m and the wall was assumed to span out-of-plane horizontally, resulting in a tributary width of wall of 0.65m loading the tie column.

For the two-story house, in this calculation the maximum length of wall panel without orthogonal walls bracing both ends was 2.6m and the wall was conservatively assumed to span horizontally only, resulting in a tributary width of wall of 1.3m loading the tie column.

In both cases, the tie column was conservatively assumed to be pinned at top and bottom.

3.4.2 Ring Beam

The longitudinal reinforcement in the ring beams was designed for the governing flexural or axial demands resulting from the following checks:

- 1 Tension due to strut and tie action (see Section 3.4.2.1)
- 2 Flexure due to strut and tie action (see Section 3.4.2.1)
- 3 Flexure in ring beam for out-of-plane support of masonry walls for the flexible diaphragm case only (see Section 3.4.2.2)
- 4 Tension due to diaphragm action of concrete roof (chord and drag forces) (see Section 3.4.2.3)
- 5 Flexure due to effect of openings (ie load transfer, coupling etc) (see Section 3.4.2.4)

The single-story design was governed by case #2 above. The two-story design was governed by case #2 above.

The transverse reinforcement in the ring beams was designed for the governing shear demands resulting from the following checks:

- 1 Shear due to strut and tie action (see Section 3.4.2.1)
- 2 Shear in ring beam acting for out-of-plane support of masonry walls for the flexible diaphragm case only (see Section 3.4.2.2)
- 3 Shear due to effect of openings (ie load transfer, coupling, etc) (see Section 3.4.2.4)

Both the single-story and two-story designs were governed by case #1 above. It should be noted that the maximum spacing requirement in ACI 318 of $d/2$ for shear reinforcement in the confining elements is not satisfied. The small dimension of the confining elements makes application of this requirement impractical.

3.4.2.1 Demands Due to Strut and Tie Action

A similar methodology to that used for the tie column detailing was used to determine axial, flexural and shear force demands on the ring beams as a result of strut-and-tie action of the confined masonry system. The ring beam reinforcement was sized for a tension force equal to the horizontal component of the compression strut capacity as well as the flexural and shear forces at the joints resulting from the concentrated loading of the compression strut. In these calculations, the ring beam was assumed to be fixed at both ends.

Refer to Section 3.4.1.1 for additional information regarding this calculation methodology.

3.4.2.2 Demands Due to Out of Plane Action (Flexible Diaphragm Only)

The flexural and shear reinforcement in the ring beams was also checked for the demands resulting from the condition where a wooden roof is used. In this scenario, the flexible roof diaphragm cannot provide continuous bracing to the top of the confined masonry walls for out of plane loading and the ring beam at the top of the wall must act as a horizontal beam to transfer out of plane loads to the orthogonal shear walls.

In this calculation, the ring beam was conservatively assumed to be pinned at both ends. The loading on the beam was based upon a tributary width of wall of 1.35m, assuming conservatively that the 2.7m tall masonry wall spans vertically between the ground and the ring beam.

3.4.2.3 Demands Due to Roof Diaphragm (Rigid Diaphragm Only)

Longitudinal reinforcing was also checked against the demand due to the development of chord forces in the beams perpendicular to the direction of seismic force, which did not govern the sizing of the reinforcing in any configuration.

3.4.2.4 Demands Due to Openings

The longitudinal and shear reinforcement in the ring beams was also checked for the shear and bending forces over door and window openings due to coupling beam behavior and due to the vertical concentrated loading from the timber truss or concrete roof slab. The coupling beam forces were determined using two-dimensional analytical structural models of the shear wall and ring beam with the openings accurately proportioned. These forces were found not to govern the design of the ring beam reinforcement. The reactions due to the roof loading were found with simple hand calculations and were also found not to govern the design of the ring beam.

3.4.3 Plinth Beam

The cast-in-place reinforced concrete plinth beam and longitudinal reinforcement was designed for the tension resulting from the strut-and-tie behavior of the confined masonry shear wall. Again, a minimum of four bars were used to facilitate uniform placement of the longitudinal steel. Due to the anchorage of the tie column in the footing below, there is no interaction between the tie column and the plinth beam and therefore no additional plinth beam reinforcing due to tie column influence was required.

It should be noted that the spacing of shear reinforcement in the plinth beam does not comply with the maximum spacing requirements of ACI 318.

3.4.4 Wall-Column Interface

Strut-and-tie shear wall behavior does not rely on the transfer of shear between the tie column/masonry wall interface. The Mexican masonry code calculations, however, rely on this transfer of force, and therefore the wall/column interfaces were checked for their ability to transfer these loads.

To confirm that no additional shear reinforcement was necessary, the confined masonry shear walls were checked to confirm that no uplift forces resulted at the base of the tie columns that would need to be transferred to the plinth beam through shear. There was found to be no net tension in the tie columns at the plinth beams in any configuration. This was determined by calculating the transfer of load between the tie column and the masonry wall over their interface due to the shear capacity of the unreinforced concrete tothing pattern.

3.5 Reinforcement Detailing

In most reinforced elements, the design of the reinforcing steel was in accordance with the provisions of ACI 318 for ratios, placement, cover, splice lengths, and development lengths. However, in certain instances, ACI recommendations were not met for reasons of economy and practicality. For example, due to the small dimensions of the confining elements, the maximum spacing requirements for transverse reinforcement in confining elements and the reinforcement cover requirements were not always met. A minimum of 25mm (1in) of cover was provided for all reinforced elements, with most non-tie column elements having the full 1.5in of cover.

3.6 Foundation Design

The foundation system was designed as an unreinforced concrete strip footing supporting reinforced concrete columns at the tie column locations and supporting unreinforced concrete block masonry foundation walls between tie columns.

Soil bearing pressures due to axial loads and overturning were checked locally per individual wall as well as globally as if the building had a continuous mat foundation for the worst case configuration permitted by the design guidelines for the one and two story houses. Allowable Stress Design (ASD) load combinations were used for these calculations, and the ground floor was set at 0.8m above the ground level, as the most conservative case. The bearing pressures were checked to ensure no uplift on any portion of the footing for the global check; however, the factor of safety against overturning was found to be less than 1.5. For overturning calculations, the portion of the reduced foundation below the porch area was not considered to participate in overturning resistance.

While the single story house with timber roof footing calculations were based on an allowable bearing capacity of 50 kN/m², the calculations for the two-story house assumed an allowable soil bearing capacity of 70kN/m² with a 1/3 increase permitted for seismic loads. While the design criteria had specified using an allowable bearing capacity of 50 kN/m², it was determined that this assumption would result in a footing width greater than 1m for the two-story house designed for Sds=1.67g. Because the MPTC guidelines specify a footing width of 70cm for poor soil conditions, we felt that a 1m footing width was sufficiently conservative and the 50kN/m² allowable bearing capacity criterion was overly restrictive for the two-story case.

The depth of the unreinforced concrete footings was determined based on shear demand at the critical section. The depth was set such that the shear capacity of the concrete was adequate to resist the shear force without requiring any reinforcement.

The foundation design assumed that in-plane wall shear forces were transferred from

the confined masonry superstructure to the foundation system through a combination of shear in the lower portion of the tie columns that extend below the plinth beam into the footing and shear in the unreinforced masonry foundation wall (transferred from the plinth beam through friction) (Figure 3.6). A check was made to ensure that for each wall the shear demand remaining after the shear capacity of the tie columns extending below the plinth beam was subtracted out was less than the shear capacity of the unreinforced masonry foundation wall per ACI 530.

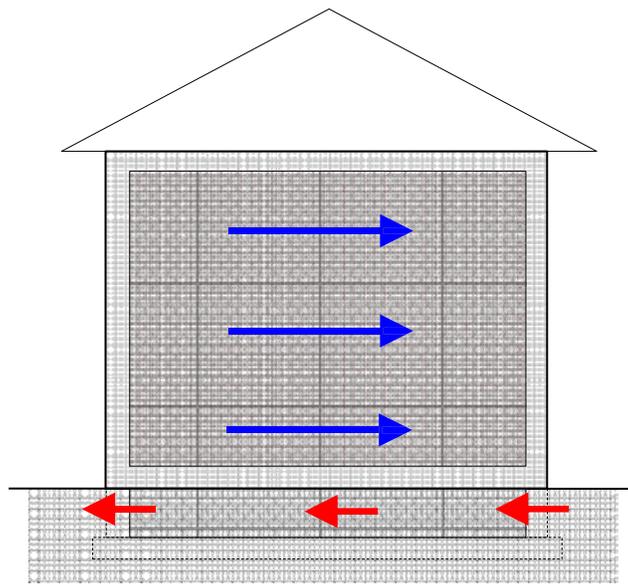


Figure 3.6 Shear transfer from confined masonry walls to foundation (capacity is based upon a combination of shear resistance of reinforced concrete tie columns and shear resistance of unreinforced masonry wall)

Lateral stability of the foundation system was checked to ensure a factor of safety against sliding of 1.5. This calculation was based on assumed coefficients of soil / concrete friction plus passive bearing at the ends of walls parallel to the direction of ground motion being considered. A coefficient of friction between the sand underlying the footing and the footing itself was assumed to be 0.5. Passive pressure was assumed to be 300lb/sqft, or

approximately 15kPa. This calculation was used to set the minimum depth below grade required for the foundation system.

Foundation walls, which could extend above grade up to 80cm in flood zones, were also checked for out of plane bending due to active soil pressure. Out-of-plane bending calculations were based on the assumption that the foundation walls perpendicular to the direction of ground motion restrain the slab and retain the interior soil build-up.

Out-of-plane calculations were carried out using the same methodology outlined in Section 3.2. Out-of-plane forces were calculated for the governing case in which the ground level slab is elevated 0.8m above the surrounding grade and found to be lower than the out-of-plane capacity of the foundation walls.

3.7 Roof Design

3.7.1 Timber Frame Roof

The timber frame roofs for one- and two-story options were designed according to the American Forest and Paper Association's National Design Specification for Wood Construction with 2005 Supplement (NDS 2005) using assumed superimposed dead loads and live loads as well as the region-specific wind loads as defined in PAHO's wind speed report which includes the effects of hurricanes. Gable roof frames were designed as simple roof trusses spaced at a maximum of 0.5 meters on centers for spans of 3-3.5m without an intermediate support and a maximum of 1.0m on centers for spans of 3-3.5m with an intermediate support. Roof truss members are 2x4 timbers and have 2x4 roof purlins running perpendicular to their top chord.

Roof truss configuration is determined by span. For the maximum loading condition for up to 3.5 meter spans, trusses with a single vertical member and two interior diagonals can be used. For spans greater than 3.5 m, two additional verticals and two additional diagonals are added to the truss configurations. Roof slope is kept constant at 25 degrees. Light-gauge corrugated metal roof deck spans between purlins and provides a continuous roof surface.

Each truss member was checked per NDS 2005 requirements for axial load, bending, and the interaction of axial and bending. Truss connections, including plywood gusset plates and sheet steel hurricane straps, were designed according to NDS 2005 specifications. Lag screw connections securing the metal roof decking to the 2x4 purlins were designed per the specifications and were typically governed by wind and uplift loads, as was much of the wood truss roof design. Detailed calculations for the wood roof system can be found in Appendix A.

3.7.2 Concrete Roof

The concrete roof and floor slabs are assumed to be two-way ribbed (or coffered) slabs created using void forms of 10 cm thick confined masonry block as is common practice in Haiti. The total depth of the ribbed two-way system is assumed to be 20 cm, consisting of 10 cm deep ribs and 10cm continuous slab above, creating a span to depth ratio that, for the longest wall-to-wall spans of 3.5m, is within ACI's recommendation for minimum slab depths.

Concrete floor and roof slabs were designed as two-way beam systems supporting slabs between beams. Beam reinforcing was designed for shear and bending, and reinforcing was designed to span between adjacent beams. Slab reinforcement was checked for diaphragm action.

For the most part, the design of the reinforcing steel was in accordance with the provisions of ACI 318 for ratios, placement, and splice lengths. However, in certain instances, ACI recommendations were not met for reasons of economy and practicality. For example, due to the small dimensions of the roof joists, the maximum spacing requirements for transverse reinforcement in confining elements and the reinforcement cover requirements were not always met.

3.8 Stair Design

It is recommended that two-story houses be built with either wooden stairs or prefabricated metal spiral stairs located on the exterior of the building. However, properly detailed reinforced concrete stairs are also permitted. Reinforced concrete stairs were designed and detailed for the particular rise and run resulting from the two-story configuration shown in the drawing set. Due to the highly specific nature of stair design, it was not practical to generate a set of guidelines to cover all possible stair geometries.

It should be noted that the reinforced concrete stair design was based upon the specification of a cold joint between the base of the stair and its foundation as well as between the edge of the stair and the adjacent wall. The purpose of this requirement is to limit the seismic load transfer from the building into the stairs, which are typically a stiffer lateral load path and therefore tend to take seismic load. Therefore, the in-plane seismic load calculations described in Section 3.1 for the two-story rigid diaphragm case conservatively assumed that the full seismic load of the stairs is transferred to the shear walls of the building in determining the center of mass of the building but not the center of rigidity.

For reasons of economy and practicality, in certain instances, ACI recommendations for reinforcement detailing were not met in the stair design.