ECONOMIC INVESTIGATION OF MASONRY CONSTRUCTION
FOR WIND RESISTANT DESIGN

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ABSTRACT

Research shows that the vulnerability of new residential construction to wind damage can be mitigated by incorporating wind resistant construction methods. This thesis estimates the additional structural cost for incorporating wind resistant construction methods into conventional residential construction. For this purpose, a rectangular masonry house is designed using the provisions in CABO One and Two Family Dwelling Code, South Florida Building Code, and ACI 530-92. This thesis compares the structural requirements and structural costs of these designs.
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NOTATIONS

\( A = \) cross-sectional area of element
\( A_s = \) area of tension reinforcement, in.\(^2\)
\( A'_s = \) area of compression reinforcement, in.\(^2\)
\( A_v = \) cross-sectional area of shear reinforcement, in.\(^2\)
\( C = \) compression force, lb
\( d = \) distance from extreme compression fiber to centroid of tension reinforcement, in.
\( d' = \) distance from extreme compression fiber to centroid of compression reinforcement
\( E_g = \) modulus of elasticity of grout, psi
\( E_m = \) modulus of elasticity of masonry in compression, psi
\( f = \) calculated stress, psi
\( f_a = \) calculated compressive stress in masonry due to axial load only, psi
\( f_b = \) calculated compressive stress in masonry due to flexure only, psi
\( f_m = \) masonry compressive strength, psi
\( f'_m = \) specified compressive strength of masonry, psi
\( f_y = \) specified yield stress of steel for reinforcement and anchors, psi
\( F_a = \) allowable compressive stress due to axial load only, psi
\( F_b = \) allowable compressive stress due to flexure only, psi
\( F_s = \) allowable tensile or compressive stress in reinforcement, psi
\( F_v = \) allowable shear stress in masonry, psi
\( h = \) effective height of column, wall, or pilaster, in.
\( I = \) moment of inertia of masonry, in.\(^4\)
\( j = \) ratio of distance between centroid of flexural compressive forces and centroid of tensile forces to depth, \( d \)

\( k = \) ratio of the distance between the neutral axis and the extreme fiber in compression to the depth, \( d \)

\( M = \) maximum moment occurring simultaneously with design shear force \( V \) at the section under consideration, in.-lb

\( n = \) modular ratio of elasticity

\( P = \) design axial load, lb

\( r = \) radius of gyration, in.

\( s = \) spacing of reinforcement, in.

\( S = \) section modulus, in.\(^3\)

\( T = \) tension force, lb

\( t = \) nominal thickness of wall, or overall depth of member cross-section, in.

\( v = \) shear stress, psi

\( V = \) design shear force, lb
CHAPTER I
INTRODUCTION

Wind damage investigations have shown that residential construction in regions along the Atlantic and Gulf coasts is vulnerable to wind damage during hurricane events. The vulnerability of these structures results in the potential of large economic and personal losses should a hurricane strike a large population center. Nearly every year, one or more hurricanes strike Gulf and Atlantic coastal states and about 900 tornadoes are reported in the US. Annual damage associated with these wind events often exceed $3 billion (SBCCI, 1990). The vulnerability of new residential construction to wind damage can be mitigated by incorporating existing wind resistant construction methods.

The objective of this thesis is to compare the structural costs of conventional construction with the structural costs of wind resistant construction. For this purpose, provisions for conventional residential masonry construction are taken as that given in CABO One and Two Family Dwelling Code (The Council of American Building Officials, 1995) and provisions for wind resistant masonry construction are taken from South Florida Building Code (Board of County Commissioners, 1988), ASCE 7-93 (ASCE, 1993), SBC 94 (SBBCI, 1994), ACI 530-92 (ACI, 1992), and APA (APA, 1995). This report investigates both the differences in structural requirements and the cost differences (computed using Means Square Foot Costs (Means, 1995)) associated with these differences. The cost comparisons are limited to a typical rectangular house of masonry construction.

This thesis is divided into five chapters. Chapter I is the introduction to this thesis. Chapter II gives a review of the literature, including previous cost comparison studies, wind load provisions in standards and building codes, and the design procedures of masonry design. Chapter III discusses masonry design for conventional construction and wind resistant construction. Chapter IV compares the designs based on CABO One and Two Family Dwelling Code (referred herein as “CABO Code”), South Florida Building Code, and other codes and standards.
Code (SFBC), ACI 530-92 with ASCE 7-93, and ACI 530-92 with SBC 94. Chapter V provides the conclusions. Appendix A to D show the engineering data of the designs based on CABO Code, SFBC, ACI 530-92 with ASCE 7-93, and ACI 530-92 with SBC 94.
CHAPTER II
LITERATURE REVIEW

The literature review is composed of three sections. The first section describes the previous cost comparison studies. The second section presents the wind load provisions for ASCE 7-93 (ASCE, 1993) and SBC 94 (SBCCI, 1994) used in wind resistant design. The third section provides an overview of the masonry provisions used in ACI 530-92 (ACI, 1992) and *Reinforced Masonry Engineering Handbook* (Amerhrin, 1992).

**Previous Cost Comparison Studies**

Several cost comparison studies have been done previously. NAHB Research Center (1993) performed a cost comparison of SFBC (Board of County Commissioners, 1988), Dade County Ordinance 93-14 (Board of County Commissioners, 1993), and other special wind loading requirements. In their study, they compare the wind resistant requirements of various building codes and standards. They also compare the costs of the designs based on these codes. Crandell (1994) has a similar report comparing the engineering requirements and structural costs between ASCE7-88 and ASCE7-95. Lombard et al. (1995) have a comparison report evaluating the cost-effectiveness of new building code for windstorm resistant construction. These studies are described below.

**Comparison of Dade County Ordinance 93-14, including ASCE 7-88 versus SFBC**

NAHB Research Center (1993) performed a cost comparison of SFBC, Dade County Ordinance 93-14, including ASCE 7-88, and other special wind loading requirements.

Dade County Ordinance 93-14 was passed by Dade County Board of County Commissioners. It implements recommendations from the research of hurricanes and contains many changes to the wind load provisions of SFBC (NHAB Research Center, 1993).
The report first compares the wind load provisions of SFBC, Dade County Ordinance 93-14, ASCE 7-88 and other requirements. Then, costs related to the differences in code provisions are estimated using two typical concrete masonry houses.

The two houses are described in the report (NAHB Research Center, 1993) as follows:

Unit B, Amaretto Village: One-story, slab-on-grade foundation, 1387 square feet of living area, 275 square foot one-car garage, 1600 square feet of wall area, truss-framed gable roof.

Unit C, Kensington: Two-story, slab-on-grade foundation, 2,510 square feet of living area (1,035 on first floor; 1,475 on second floor), 460 square foot two-car garage, 2,840 square feet of wall area, truss-framed gable roof. (p. 31)

According to the report, the comparison of the provisions and the analysis of the selected houses show that the provisions in Dade County Ordinance 93-14 and ASCE 7-88 result in higher design loads, compared to the design loads of SFBC. These increased design loads require stronger roof fastener and foundation anchorage, stronger windows and doors frames, and stronger masonry or wood frame walls.

The cost comparison shows that the additional direct construction costs to comply with ASCE 7-88 and Dade County Ordinance 93-14 versus SFBC are estimated to be from $3,819 to $4,845 for one-story buildings and from $5,032 to $6,326 for two-story buildings.

Comparison of ASCE 7-88 versus ASCE 7-95

Crandell (1994) has a similar report comparing the wind speeds, wind pressures, and structural engineering requirements in ASCE 7-88 and ASCE 7-95. Three representative homes are used to estimate the cost impacts related to the differences of the requirements in ASCE 7-88 and ASCE 7-95.

The characteristics of three homes investigated are:

1. Case 1: 1-story, 32ft x 52ft, wood trusses gable roof, wood-frame wall.
2. Case 2: 2-story, 28ft x 44ft, wood trusses gable roof, wood-frame wall.


According to the report, ASCE 7-95 has a higher design wind speed, with respect to that in ASCE 7-88, which causes higher wind pressure on the walls and the roofs of the houses. As a result, there are substantial increases in the requirements of roof covering, roof-wall connection, and wall construction.

The cost estimation is based primarily on Means Square Foot Cost (Means, 1995); only the costs related to the changes of the provisions (ASCE 7-88 and ASCE 7-95) are addressed and costs of contractor make ups are not included. Cost comparison shows that additional costs to comply with ASCE 7-95 versus ASCE 7-88 are in the range of 5% to 9% of the structural cost for inland areas where design wind speed given in ASCE 7-88 is 70 mph and from 40% to 56% of the structural cost along hurricane coastline where design wind speed given in ASCE 7-88 is over 100 mph.

Comparison of the New Code versus the Current Code for Windstorm Resistant Construction along the Texas Coast

Lombard et al. (1995) have a comparison report evaluating the cost-effectiveness of the new building code for windstorm resistant construction. In this report, the current and new code are analyzed and the changes of structural requirements are determined. Several selected houses are used to estimated the cost differences between the new and current code.

The characteristics of these selected houses are:

1. Case 1: 1-story, 1,000 sq. ft., inland area, wood-frame construction.
2. Case 2: 1-story, 1,000 sq. ft., seaward area, wood-frame construction.
7. Case 7: 2-story, 3,000 sq. ft., inland area, wood-frame construction.
8. Case 8: 2-story, 3,000 sq. ft., seaward area, wood-frame construction.

The major changes of structural requirements of new code versus current code are the requirements of anchorage, lateral wall bracing, roof framing, roof decking, and roof covering. The costs relative to these changes are obtained from the *National Construction Estimator*. Labor and material fee are included. The report shows that the cost to implement the new code versus the current code is from about 2% to 5% of the structural cost depending on the size of the house.

**Building Codes and Standards**

There are several building codes and standards that are currently used in the US, including ASCE 7-93, SBC 94, SFBC and CABO Code. The wind load provisions in these building codes and standards are given below. The masonry design provisions are given in the next section.

**ASCE 7: Minimum Design Loads for Buildings & Other Structures**

ASCE 7: Minimum Design Loads for Building and Other Structures provides minimum load requirements for the design of buildings and other structures. Section 6 of ASCE 7 gives provisions to determine minimum wind loads.

**Basic Wind Speed and Velocity Pressure**

ASCE 7-93 provides the Basic Wind Speed Map (shown in Figure 2.1) to determine the design wind speed for the US. ASCE 7 also gives the formula to calculate the velocity pressure \( q_z \):

\[
q_z = 0.00256K_z(IV)^2,
\]

where \( K_z \) is the velocity pressure exposure coefficient, \( I \) is the importance factor, and \( V \) is the basic wind speed. \( K_z \) is given in Table 2.1 and \( I \) is given in Table 2.2.
Gust Response Factors

Gust Response Factors take into account the loading effects due to the turbulence of wind. In some cases, gust response factors are combined with pressure coefficients (ASCE, 1993).

For Main Wind-Force Resisting System (MWFRS), the gust response factor $G_h$ can be obtained from Table 2.3 at the building mean roof height $h$. For internal pressures and Components and Cladding (C&C) pressures, the gust response factor is combined with the pressure coefficient to give a combined pressure and gust factor, $GC_p$.

Pressure Coefficients

Pressure coefficients take into account the effects of the shape and size of the building. External pressure coefficients and internal pressure coefficients are determined separately.

External Pressure Coefficients. For MWFRS, external pressure coefficients $C_p$ are given in Figure 2.2; for C&C, combined gust response and external pressure coefficients $GC_p$ are given in Figure 2.3.

Internal Pressure Coefficients. The combined gust response factor and internal pressure coefficients, $GC_{pi}$, are given in Table 2.4.

Design Wind Pressure

Design wind pressure can be determined using the appropriate equation given in Table 2.5.

Standard Building Code

SBC is written by the Southern Building Code Congress International Incorporation (SBCCI, 1994). Section 1606 of SBC 94 gives provisions to determine wind load.
Basic Wind Speed and Velocity Pressure

Similar to ASCE 7-93, SBC 94 provides the Basic Wind Speed Map to determine the basic wind speed for a 50-year Mean Recurrence Interval wind in the US. SBC 94 also gives similar formula to calculate the velocity pressure $q$:

$$q = 0.00256V^2 \left( \frac{H}{33} \right)^{2/7},$$

where $V$ is the basic wind speed, $H$ is mean height of roof above ground.

Pressure Coefficients

For MWFRS, the pressure coefficients, $GC_p$, are given in Figures 2.4 and 2.5; for C&C, the pressure coefficients are given in Figure 2.6.

Design Wind Pressure

Design wind pressure $p$ is:

$$p = qGC_p I,$$

where $q$ is the velocity pressure, $GC_p$ is the pressure coefficient, and $I$ is the use factor, which is given in Table 2.6.

South Florida Building Code

SFBC is written by the Board of County Commissioners of Dade County, Florida and it is used in the South Florida area. SFBC requires that all buildings and structures be designed and constructed against wind load. The design wind speed is 120 mph.

SFBC uses a formulation similar to the one in SBC 94 for calculating velocity pressure $p$:

$$p = 0.00256V^2 (H/30)^{2/7},$$

where $V = 120$ mph, and $H$ is the height of the house.

SFBC also gives Shape Factors and Use Factors which multiply the wind load based on use and geometric shape of the buildings.

CABO Code gives the Basic Wind Speed Map (similar to the one in ASCE 7-93) to determine the design wind speed and a Design Wind Loads Table to determine the design wind load. CABO Code requires that exterior walls be designed in accordance with wind resistant practice when design pressures are higher than 30 psf.

Masonry Design

Masonry construction is a house, building or other structure built with bricks or blocks. Usually, a masonry construction consists of four types of materials: brick or block, mortar, grout, and reinforcing steel bar. The design provisions for masonry are discussed below.

Materials

Bricks and blocks are the units to make masonry walls. Bricks are usually made from clay and blocks are made from concrete. In this thesis, only Concrete Masonry Unit (CMU) construction is considered.

Mortar is a plastic mixture of cement, lime, sand and water used as the bonding agent that holds the individual masonry unit and connectors together to act as a complete assembly. Three types of mortar are available as Types M, S and N. Type M mortar is used for masonry construction under high compressive or lateral loads; Type S mortar is used for masonry construction requiring high flexural bond strength, and under compressive or lateral loads; Type N mortar is for general use in masonry construction (Amerhin, 1992).
Grout is also a mixture consisting of cement, sand, pea gravel and water. Grout is similar to mortar but it has higher slump than mortar. Grout is used primarily in reinforced masonry construction to bond reinforcing steel bars to the masonry.

Reinforcing steel bars (rebar) improve the performance of masonry walls. Concrete blocks are good material resisting compression force but they work poorly resisting tension force, thus, steel bars are grouted into the masonry walls to resist the tension force. In this thesis, all masonry walls are assumed to be reinforced.

Analysis and Design

The masonry design includes shear design, flexure design, and diaphragm design.

Shear design has two parts: shear design for out-of-plane shear and shear design for in-plane shear. Shear design for out-of-plane shear is the analysis and design against the shear forces due to out-of-plane forces. Vertical reinforcement may be required to resist these shear forces. Shear design for in-plane shear is the analysis and design against the shear forces due to in-plane forces. These shear forces may cause different layers of the wall to separate from each other. Vertical rebar may be needed to resist these shear forces.

Flexural design is the analysis and design against the out-of-plane forces and axial forces. It is presented as combined axial and flexural design where the axial force can be a compressive force, tensile force or a zero force. Vertical rebar are often needed to resist these forces.

Diaphragm design determines the size of the bond beam, reinforcement in the bond beam, thickness of roof cladding, and the fasteners of roof cladding. Diaphragm design also checks the deflection of bond beam.

In this thesis, masonry design follows ACI 530-92 (ACI, 1992), Masonry Designers’ Guide (Matthys, 1993), and Reinforced Masonry Engineering Handbook (Amerhrin, 1992). The following are the details of masonry design.
Shear Design

Shear design includes two parts: shear design for out-of-plane shear and shear design for in-plane shear. Figure 2.7 is the flow chart for shear design. ACI 530-92 provides the equations to calculate the allowable shear stress in Section 7.5.2.2 and 7.5.2.3.

Allowable Shear Stress Specified in ACI530-92

Section 7.5.2.2. Section 7.5.2.2 specifies the allowable shear stress when no reinforcement is provided for shear in masonry wall. When designing for out-of-plane shear, the allowable shear stress is:

\[ F_v = \sqrt{f_m} \text{ and } F_v \leq 50 \text{psi}, \quad (2.5) \]

where \( f_m \) is the specified compressive strength of masonry;

when designing for in-plane shear, the allowable shear stress is:

i) \[ \frac{M}{Vd} < 1, \quad F_v = \frac{1}{3}[4 - \frac{M}{Vd}]\sqrt{f_m} \text{ and } F_v \leq 80 - 45 \frac{M}{Vd}, \quad \text{or} \]

ii) \[ \frac{M}{Vd} \geq 1, \quad F_v = \sqrt{f_m} \text{ and } F_v \leq 35 \text{psi}, \quad (2.7) \]

where \( V \) is the design shear force, \( M \) is the moment occurring simultaneously with \( V \), and \( d \) is the distance from extreme compression fiber to centroid of tension reinforcement.

Section 7.5.2.3. Section 7.5.2.3 specifies the allowable shear stress when reinforcement is provided for shear in masonry wall. When designing for out-of-plane shear, the allowable shear stress is:

\[ F_v = 3\sqrt{f_m} \text{ and } F_v \leq 150 \text{psi}; \quad (2.8) \]

when designing for in-plane shear, the allowable shear stress is:

i) \[ \frac{M}{Vd} < 1, \quad F_v = \frac{1}{2}[4 - \frac{M}{Vd}]\sqrt{f_m} \text{ and } F_v \leq 120 - 45 \frac{M}{Vd}, \quad \text{or} \]

ii) \[ \frac{M}{Vd} \geq 1, \quad F_v = 1.5\sqrt{f_m} \text{ and } F_v \leq 75 \text{psi}. \quad (2.10) \]
Allowable shear stress $F_v$ in Section 7.5.2.2 and 7.5.2.3 can be increased by 33% when designing against wind load.

Shear Design for Out-of-plane Shear

Shear design for out-of-plane shear is the design against shear forces due out-of-plane forces. Figure 2.8 shows the out-of-plane shear forces.

The following are the basic steps for out-of-plane shear design (Amerhin, 1992):

1. Calculate shear force per foot of wall:
   \[ \nu = \frac{V}{l}, \]  
   where $V$ is the total shear force on a design pier, and $l$ is the width of a design pier;

2. Calculate the shear stress:
   \[ f_v = \frac{\nu}{(b)(j)(d)}, \]  
   where $b$ is the width of design section, $b=12$ inches, $j=0.9$, and
   \[ d = \frac{T_u - 0.375}{2}, \]  
   where $T_u$ is the nominal thickness of masonry block;

3. Calculate allowable shear stress:
   When no reinforcement is provided for shear design, use Eq. 2.5 to calculate the allowable shear stress. When reinforcement is provided for shear design, use Eq. 2.8 to calculate the allowable shear stress;

4. Provide vertical rebar for shear if necessary:
   \[ A_v = \frac{V(12 \text{ in./ft})}{F_s d} \text{ in}^2 / \text{ft}, \]  
   where $F_s$ is the allowable stress in reinforcement. $F_s=24,000$ psi for Grade 60 Steel.
Shear Design for In-plane Shear

Shear design for in-plane shear is the design against shear forces due to in-plane shear forces. The in-plane shear forces are distributed to the piers by their relative stiffness. Figure 2.9 shows the in-plane shear forces.

The following are the basic steps for the design (Amerhin, 1992):

1. Calculate relative stiffness of piers (assume piers are fixed at both ends):

   Total shear and moment deflection for a fixed-fixed pier is:
   \[ \Delta_F = \frac{P}{E_m t} \left[ \frac{h^3}{l} + 3 \frac{h}{l} \right], \tag{2.14} \]

   where \( h \) and \( l \) are the height and width of a pier, \( P \) is the design axial load, \( t \) is the thickness of wall, and \( E_m \) is the modulus of elasticity of masonry.

   The relative stiffness is \( \frac{1}{\Delta_F} \);

2. Distribute shear force to piers by their relative stiffness:

   \[ V_i = \frac{1}{\sum \frac{1}{\Delta_{Fi}}} \cdot \frac{1}{\Delta_{Fi}} V, \tag{2.15} \]

   where \( V_i \) is the shear force distributed to pier \( i \);

3. Calculate allowable shear stress:

   When no reinforcement is provided for shear design, use Eq. 2.6 and 2.7 to calculate the allowable shear stress. When reinforcement is provided for shear design, use Eq. 2.9 and 2.10 to calculate the allowable shear stress;

4. Calculate shear stresses in piers:

   \[ f_{wi} = \frac{V_i}{(b)(j)(d)}, \tag{2.16} \]

   where \( V \) is the distributed shear force, \( b = T_u - 0.375 \), \( j = 0.92 \), and \( d = l - 6\text{in} \);
5. Provide reinforcement for shear (in² / ft) if necessary:

\[ A_y = \frac{V(12 \text{ in} / \text{ft})}{F_s d}, \tag{2.17} \]

where \( F_s \) is the allowable stress in reinforcement, and \( d = l - 6 \text{in} \).

\( F_s = 24,000 \text{ psi for Grade 60 Steel.} \)

Combined Axial and Flexural Design

Flexural design is the analysis and design against combined out-of-plane bending and in-plane axial forces. Figure 2.10 shows the moments and loads used for this design and Figure 2.11 is the flow chart for flexural design.

An iterative method which can be used for axial tension or compression as well as no axial force is used in combined axial and flexural design. The following are the basic steps for flexural design (Amerhin, 1992).

**Case I. \( kd < \) Thickness of face shell**

When compression block height, \( kd \), is less than thickness of face shell, \( t_{f1} \), sum moments about the center line of the wall (assume rebar is placed at the center of the wall),

\[ C(d - \frac{1}{3}kd) - M = 0, \tag{2.18} \]

\[ T = C - P, \tag{2.19} \]

\[ C = \frac{1}{2} f_m kdb_2, \tag{2.20} \]

where \( C \) is the compression force in masonry, \( T \) is the tension force in the rebar, \( M \) is the design moment, \( P \) is the design axial force, and \( b_2 \) is the width of design section.

Substituting Eq. 2.20 into Eq. 2.18 and rearranging the equation,
\[ \frac{1}{6} f_m b_2 (kd)^2 - \frac{1}{2} f_m b_2 d (kd) + M = 0 . \] 

(2.21)

Solve for \( kd \),

\[ kd = \frac{-b - \sqrt{b^2 - 4ac}}{2a} , \text{ where} \]

\[ a = \frac{1}{6} f_m b_2 , \]

\[ b = -\frac{1}{2} f_m b_2 d , \]

\[ c = M . \]

(2.22)

In the equations above, \( f_m \) is the actual stress in the masonry unit. The following are the equations to determine the maximum value of the \( f_m \) for a given wall pier.

\[ f_m = f_a + f_b , \]

(2.23)

where \( f_a \) is the stress due to the axial force,

\[ f_a = \frac{P}{A} , \text{ and} \]

\( f_b \) is the stress related to the bending moment,

\[ f_b = F_b \left( 1 - \frac{f_a}{F_a} \right) , \text{ where} \]

\[ F_a = \begin{cases} \frac{1}{4} f_m' [1 - \left( \frac{h}{140r} \right)^2 ] & (h/r \leq 99) \\ \frac{1}{4} f_m' \left( \frac{70r}{h} \right)^2 & (h/r > 99) \end{cases} , \text{ and} \]

\[ F_b = \frac{1}{3} f_m , \text{ where} \ F_a \ \text{is the allowable axial compression stress of masonry (assume masonry resists compression stress only),} \ F_b \ \text{is allowable axial forces related to bending,} \]

\( h \) is the height of the wall, and \( r \) is radius of gyration.
Check the value of $kd$. If $kd \geq d$, we need to increase the strength of masonry, $f_m'$, or the thickness of the masonry block, $T^*$; if $kd > t_f$, we need to recalculate $kd$ as Case II shown below; if $kd \leq t_f$, calculate reinforcement by following equations:

$$C = \frac{1}{2} f_m kd b_2,$$  \hspace{1cm} (2.24)  

$$T = C - P,$$  \hspace{1cm} (2.25)  

$$k = \frac{kd}{d},$$  \hspace{1cm} (2.26)  

$$f_s = \left(\frac{1 - k}{k}\right)n f_m,$$  \hspace{1cm} (2.27)  

where $n$ is the modular ratio, ratio of modulus of elasticity of steel to modulus of elasticity of masonry.

If $f_s$ is less than allowable stress in rebar $F_s$, provide reinforcement as $A_s = \frac{T}{f_s}$; if $f_s$ exceeds $F_s$, decrease $f_m$ and recalculate $kd$.

**Case II. $kd > $ Thickness of face shell**

When compression block height is larger than thickness of face shell, neglect the compression force of web $C_2$ and sum moments about the center line of wall,

$$C_1(d - \frac{1}{2} t_f) - M = 0,$$  \hspace{1cm} (2.28)  

$$T = C_1 - P,$$  \hspace{1cm} (2.29)  

$$C_1 = \frac{1}{2} (f_m + f_m') t_f b_2,$$  \hspace{1cm} (2.30)  

where $f_m = f_a + f_b$, and $f_m = (1 - \frac{t_f}{kd}) f_m$.

Substituting the third equation into the first equation and rearranging the equation:
\[ kd = \frac{t_{fs}}{2 - \frac{2M}{f_m t_{fs} b_2 (d - \frac{1}{2} t_{fs})}}. \]  \hspace{1cm} (2.31)

Check the value of \( kd \). If \( kd \geq d \), we need to increase \( f_m \) or \( T_v \); if \( kd < d \), calculate reinforcement by following equations:

\[ C_1 = \frac{1}{2} (2 - \frac{t_{fs}}{kd}) f_m t_{fs} b_2, \]  \hspace{1cm} (2.32)
\[ T = C_1 - P, \]  \hspace{1cm} (2.33)
\[ k = \frac{kd}{d}, \]  \hspace{1cm} (2.34)
\[ f_s = (\frac{1-k}{k}) n f_m, \]  \hspace{1cm} (2.35)

If \( f_s \) is less than allowable stress in the rebar \( F_s \), provide reinforcement as
\[ A_s = \frac{T}{f_s}; \] if \( f_s \) exceeds \( F_s \), decrease \( f_m \) and recalculate \( kd \).

**Diaphragm Design**

For masonry construction, the diaphragm consists of horizontal bond beams and roof or floor system. The diaphragm is assumed to act as a simply supported beam between the shear walls. When lateral forces act on the diaphragm, it deflects in beam action where the bond beams act as the flange of beam and the roof or floor system acts as the web of beam. The effective width of flange is assumed as three times of wall thickness on either side of the floor or roof system (neglecting the effect of reinforcement). Figure 2.12 shows the beam action of diaphragm when lateral forces act on it and Figure 2.13 is the flow chart for diaphragm design.

As the diaphragm deflects, the lateral loads are transmitted to the side shear walls. The deflection of diaphragm must be under a limit, usually 0.007 times the height of the
house (Amerhrin, 1992). The deflection causes tension and compression forces in the bond beams. Adequate reinforcement must be provided to resist these forces.

Diaphragm design includes the design of bond beam, deflection checking of the diaphragm, and design of roof cladding and fasteners. The roof cladding thickness and fastener are determined by shear load on diaphragm. Diaphragm design in this thesis does not account for several items including the design for dead and live loads, the design of roof truss system, the nailing pattern, and the layout of the diaphragm.

The following are the basic steps for diaphragm design (Amerhrin, 1992):

1. Calculate the moment due the lateral wind load which comes from the roof and walls:
   \[ M = \frac{1}{8}wl^2, \quad (2.36) \]
   where \( w \) is the lateral wind load and \( l \) is the length of house;

2. Calculate the tension and compression forces in bond beams:
   \[ T = \frac{M}{b}, \quad (2.37) \]
   where \( b \) is the width of house;

3. Provide reinforcement for tension or compression forces:
   \[ A_s = \frac{T}{F_s} \quad \text{or} \quad A_s = \frac{C}{F_s}, \quad (2.38) \]
   where \( F_s \) is the allowable stress in rebar.

As stated above, the deflection of diaphragm must be under 0.007 times the height of house. The following steps are used to check the deflection:

1. Effective width of diaphragm flange = \( 3T_u \), where \( T_u \) is the thickness of masonry wall;

2. Area of flange, \( A = T_u \times \) Effective width; \quad (2.39)

3. Moment of Inertia, \( I = 2A\left(\frac{b}{2}\right)^2 \); \quad (2.40)
4. Consider the diaphragm a simply supported beam, the deflection is:

\[ \Delta = \frac{5wL^4}{384E_mI}, \]  

(2.41)

if the deflection is larger than the limitation, we need to increase the width of the bond beam, and the thickness of the masonry block since the thickness of the masonry block is usually equal to the width of the bond beam.

The design of roof cladding and fastener follows APA (APA, 1995). The design used in this work accounts for the shear load acting on the roof diaphragm, the nailing pattern, and the layout of the diaphragm but does not account for several items including the design for dead and live loads, and the design of roof truss system.
Special wind region

Values are fastest-mile speeds at 33 ft (10m) above ground (or exposure category C) and are associated with an annual probability of 0.02. Linear interpolation between wind speed contours is acceptable. Caution in the use of wind speed contours in mountainous regions of Alaska is advised.

Figure 2.1 Basic Wind Speed

Wall Pressure Coefficients, $C_p$

<table>
<thead>
<tr>
<th>Surface</th>
<th>$L/H$</th>
<th>$C_p$</th>
<th>For use with</th>
</tr>
</thead>
<tbody>
<tr>
<td>Windward w.all</td>
<td>All values</td>
<td>-0.8</td>
<td>$q_z$</td>
</tr>
<tr>
<td></td>
<td>0-1</td>
<td>0.5</td>
<td></td>
</tr>
<tr>
<td>Leeward wall</td>
<td>2</td>
<td>-0.3</td>
<td>$q_h$</td>
</tr>
<tr>
<td></td>
<td>&gt;4</td>
<td>-0.2</td>
<td></td>
</tr>
<tr>
<td>Side walls</td>
<td>All values</td>
<td>-0.7</td>
<td>$q_h$</td>
</tr>
</tbody>
</table>

Roof Pressure Coefficients, $C_p$, for Use with $q_h$

<table>
<thead>
<tr>
<th>Wind direction to ridge</th>
<th>$h/L$</th>
<th>0</th>
<th>10-15</th>
<th>20</th>
<th>30</th>
<th>40</th>
<th>50</th>
<th>&gt;60</th>
<th>Leeward</th>
</tr>
</thead>
<tbody>
<tr>
<td>Normal</td>
<td></td>
<td>-0.7</td>
<td>0.2</td>
<td>0.2</td>
<td>0.3</td>
<td>0.4</td>
<td>0.5</td>
<td>0.018</td>
<td>-0.7</td>
</tr>
<tr>
<td></td>
<td>0.5</td>
<td>-0.7</td>
<td>-0.9</td>
<td>-0.75</td>
<td>-0.2</td>
<td>0.3</td>
<td>0.5</td>
<td>0.018</td>
<td>values</td>
</tr>
<tr>
<td></td>
<td>1.0</td>
<td>-0.7</td>
<td>-0.9</td>
<td>-0.75</td>
<td>-0.2</td>
<td>0.3</td>
<td>0.5</td>
<td>0.018</td>
<td>of $h/L$</td>
</tr>
<tr>
<td></td>
<td>&gt;1.5</td>
<td>-0.7</td>
<td>-0.9</td>
<td>-0.9</td>
<td>-0.35</td>
<td>0.2</td>
<td>0.018</td>
<td>and $\theta$</td>
<td></td>
</tr>
<tr>
<td>Parallel to ridge</td>
<td></td>
<td></td>
<td>-0.7</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>-0.7</td>
</tr>
<tr>
<td></td>
<td>$h/B$ or $h/L$</td>
<td>&lt; 2.5</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>$h/B$ or $h/L$</td>
<td>&gt; 2.5</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>-0.8</td>
</tr>
</tbody>
</table>

*Both values of $C_p$ shall be used in assessing load effects.

NOTES:
1. Refer to Table 10 for arched roofs.
2. For flexible buildings and structures, use appropriate $C_p$ as determined by rational analysis.
3. Plus and minus signs signify pressures acting toward and away from the surfaces, respectively.
4. Linear interpolation may be used for values of $\theta$, $h/L$, and $L/B$ ratios other than shown.
5. Notation:
   - $h$: Height above ground, in feet
   - $h_m$: Mean roof height, in feet, except that eave height may be used for $\theta < 10$ degrees
   - $q_h$: Velocity pressure, in pounds-force per square foot, evaluated at respective height
   - $G$: Gust response factor
   - $L$: Horizontal dimension of building, in feet, measured normal to wind direction
   - $B$: Horizontal dimension of building, in feet, measured parallel to wind direction
   - $\theta$: Roof slope from horizontal, in degrees

Figure 2.2

External Pressure Coefficients for Average Loads on MWFRS

Notes:
(1) The vertical scale denotes GC\textsubscript{p} to be used with q\textsubscript{b} based on Exposure C.
(2) The horizontal scale denotes the tributary area A, in square feet.
(3) External pressure coefficients for walls may be reduced by 10% when \( \theta \leq 10 \) degrees.
(4) If a parapet equal to or higher than 3 ft is provided around the perimeter of roof \( \theta \leq 10 \) degrees, zone 3 may be treated as zone 2.
(5) Plus and minus signs signify pressures acting toward and away from the surfaces, respectively.
(6) Each component shall be designed for maximum positive and negative pressures.
(7) Notation: \( a \): 10\% of minimum width or 0.4h, whichever is smaller, but not less than either 4\% of minimum width or 3 feet; h: mean roof height, in feet, except that eave height may be used when \( \theta \leq 10 \) degrees; and \( \theta \) roof slope from horizontal, in degrees.

Figure 2.3
External Pressure Coefficients, \( GC_p \), for C&C for Building
with Mean Roof Height Less than or Equal to 60 Feet

Figure 1606.2B1
Application of Coefficients for Primary Structural Systems Providing Resistance in Transverse Direction
(Positive sign indicates inward acting pressure)

Figure 1606.2B2
Application of Coefficients for Primary Structural Systems Providing Resistance In Longitudinal Direction
(Positive sign indicates inward acting pressure)

Figure 2.4
Application of Coefficients for Primary Structural Systems

### Providing Resistance In Transverse Direction

<table>
<thead>
<tr>
<th>Roof Angle Angle</th>
<th>Notes</th>
<th>End Zone Coefficients</th>
<th>Interior Zone Coefficients</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>1E</td>
<td>2E</td>
</tr>
<tr>
<td><strong>Enclosed Building</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>$0 &lt; a \leq 10^\circ$</td>
<td>2</td>
<td>+ 50</td>
<td>-1.4</td>
</tr>
<tr>
<td></td>
<td>3</td>
<td>+ 90</td>
<td>-1.0</td>
</tr>
<tr>
<td>$20^\circ &lt; a \leq 30^\circ$</td>
<td>2</td>
<td>+ 70</td>
<td>-1.4</td>
</tr>
<tr>
<td></td>
<td>3</td>
<td>+ 1.1</td>
<td>-1.0</td>
</tr>
<tr>
<td>$30^\circ &lt; a \leq 45^\circ$</td>
<td>2</td>
<td>+ 70</td>
<td>-1.0</td>
</tr>
<tr>
<td></td>
<td>3</td>
<td>+ 1.1</td>
<td>-60</td>
</tr>
<tr>
<td><strong>Partially Enclosed</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>$0 &lt; a \leq 10^\circ$</td>
<td>2</td>
<td>+ 50</td>
<td>-1.4</td>
</tr>
<tr>
<td></td>
<td>3</td>
<td>+ 90</td>
<td>-1.0</td>
</tr>
<tr>
<td>$20^\circ &lt; a \leq 30^\circ$</td>
<td>2</td>
<td>+ 70</td>
<td>-1.4</td>
</tr>
<tr>
<td></td>
<td>3</td>
<td>+ 1.1</td>
<td>-1.0</td>
</tr>
<tr>
<td>$30^\circ &lt; a \leq 45^\circ$</td>
<td>2</td>
<td>+ 70</td>
<td>-1.0</td>
</tr>
<tr>
<td></td>
<td>3</td>
<td>+ 1.1</td>
<td>-60</td>
</tr>
<tr>
<td><strong>Completely Open</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>$0 &lt; a \leq 10^\circ$</td>
<td>2</td>
<td>+ 50</td>
<td>-1.4</td>
</tr>
<tr>
<td></td>
<td>3</td>
<td>+ 90</td>
<td>-1.0</td>
</tr>
<tr>
<td>$20^\circ &lt; a \leq 30^\circ$</td>
<td>2</td>
<td>+ 70</td>
<td>-1.4</td>
</tr>
<tr>
<td></td>
<td>3</td>
<td>+ 1.1</td>
<td>-1.0</td>
</tr>
</tbody>
</table>

### Providing Resistance In Longitudinal Direction (All Roof Angles)

<table>
<thead>
<tr>
<th>Building Classification</th>
<th>Notes</th>
<th>End Zone Coefficients</th>
<th>Interior Zone Coefficients</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>2E</td>
<td>3E</td>
</tr>
<tr>
<td><strong>Enclosed</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>$8.9$</td>
<td>-1.42</td>
<td>-0.80</td>
<td>+0.50</td>
</tr>
<tr>
<td>$8.8$</td>
<td>-1.00</td>
<td>-0.40</td>
<td>+0.90</td>
</tr>
<tr>
<td><strong>Partially Enclosed</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>$8.5$</td>
<td>-1.80</td>
<td>-1.20</td>
<td>+0.10</td>
</tr>
<tr>
<td><strong>Completely Open</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>$8.1$</td>
<td>-0.90</td>
<td>-0.30</td>
<td>+1.00</td>
</tr>
<tr>
<td>$6.1$</td>
<td>-0.70</td>
<td>-0.70</td>
<td>+1.80</td>
</tr>
<tr>
<td><strong>Open</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>$7.1$</td>
<td>-0.30</td>
<td>-0.60</td>
<td>+1.80</td>
</tr>
</tbody>
</table>

Figure 2.5

Pressure Coefficients for MWFRS

Figure 1606.2C
Wall Coefficients, GCp, Components and Cladding
(Enclosed Buildings)

Figure 1606.2D
Roof Coefficients, GCp, Components and Cladding
(Enclosed Buildings)
$0^\circ \leq \alpha \leq 10^\circ$

Figure 2.6
Pressure Coefficients for C&C

Figure 2.6 (Continued)

In-plane

Calculate Relative Stiffness of Piers

Distribute Shear by Relative Stiffness

Out-of-plane

Calculate Shear due to the out-of-plane forces

Calculate Shear Stress: \( f_v \)

\( f_v > F_v \) ?

No

Shear Requirement Satisfied without reinforcement

Yes

\( f_v > F_v \) ?

Rebar provided for shear

No

Provide Shear reinforcement

Yes

Reproportion and Redesign

Shear Requirement Satisfied with reinforcement

End

Figure 2.7
Flow Chart for Shear Design
Figure 2.8
Out-of-plane Shear Forces

Figure 2.9
In-plane Shear Forces
Figure 2.10

Loads and Moments on Wall
Calculate $kd$ (Case I)
($kd > t_{fs}$)

No

Yes

Calculate $kd$ (Case II)
($kd > t_{fs}$)

No

Yes

$fs = \frac{(1-k)}{k} \cdot \left( \frac{E_s}{E_m} \right) \cdot f_m$

No

Yes

$fs < F_s$?

No
decrease $f_m$

Yes

$As = \frac{T}{fs}$

End

Figure 2.11
Flow Chart for Flexural Design
Figure 2.12
Compression and Tension in Bond Beam
Bond Beam Design
Provide reinforcement in bond beams

Check deflection

deflection < limit?

Yes

End

No

increase thickness of wall and width of bond beam

Figure 2.13
Flow Chart for Diaphragm Design
### Table 2.1
Velocity Pressure Exposure Coefficient, $K_z$

<table>
<thead>
<tr>
<th>Height above ground level, $Z$ (feet)</th>
<th>Exposure A</th>
<th>Exposure B</th>
<th>Exposure C</th>
<th>Exposure D</th>
</tr>
</thead>
<tbody>
<tr>
<td>0 -15</td>
<td>0.12</td>
<td>0.37</td>
<td>0.80</td>
<td>1.20</td>
</tr>
<tr>
<td>20</td>
<td>0.15</td>
<td>0.42</td>
<td>0.87</td>
<td>1.27</td>
</tr>
<tr>
<td>25</td>
<td>0.17</td>
<td>0.46</td>
<td>0.93</td>
<td>1.32</td>
</tr>
<tr>
<td>30</td>
<td>0.19</td>
<td>0.50</td>
<td>0.98</td>
<td>1.37</td>
</tr>
<tr>
<td>40</td>
<td>0.23</td>
<td>0.57</td>
<td>1.06</td>
<td>1.46</td>
</tr>
<tr>
<td>50</td>
<td>0.27</td>
<td>0.63</td>
<td>1.13</td>
<td>1.52</td>
</tr>
</tbody>
</table>


### Table 2.2
Importance Factor, $I$

<table>
<thead>
<tr>
<th>Category</th>
<th>$I$</th>
</tr>
</thead>
<tbody>
<tr>
<td>100 miles from hurricane oceanline and in other areas</td>
<td>1.00</td>
</tr>
<tr>
<td>At Hurricane Oceanline</td>
<td>1.05</td>
</tr>
</tbody>
</table>

Table 2.3
Gust Response Factors, $G_h$ and $G_z$

<table>
<thead>
<tr>
<th>Height above ground level, $Z$ (feet)</th>
<th>$G_h$ and $G_z$</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Exposure A</td>
</tr>
<tr>
<td>0 - 15</td>
<td>2.36</td>
</tr>
<tr>
<td>20</td>
<td>2.20</td>
</tr>
<tr>
<td>25</td>
<td>2.09</td>
</tr>
<tr>
<td>30</td>
<td>2.01</td>
</tr>
<tr>
<td>40</td>
<td>1.88</td>
</tr>
<tr>
<td>50</td>
<td>1.79</td>
</tr>
</tbody>
</table>

Table 2.4
Internal Pressure Coefficients for Buildings $GC_{pi}$

<table>
<thead>
<tr>
<th>Condition</th>
<th>$GC_{pi}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Condition I</td>
<td>+ 0.25; -0.25</td>
</tr>
<tr>
<td>Condition II</td>
<td>+ 0.75; - 0.25</td>
</tr>
</tbody>
</table>

- Condition I: All conditions except as noted under condition II.
- Condition II: Buildings in which both of the following are met:
  1. Percentage of openings in one wall exceeds the sum of the percentages of openings in the remaining walls and roof surfaces by 5% or more, and,
  2. Percentage of openings in any one of the remaining walls or roof do not exceed 20%.


Table 2.5
Design Wind Pressures and Forces

<table>
<thead>
<tr>
<th>Design wind loading</th>
<th>Building ( h &lt; 60ft )</th>
</tr>
</thead>
<tbody>
<tr>
<td>Main wind-force resisting systems</td>
<td>$p = qG_h C_p - q_h (GC_{pi})$</td>
</tr>
<tr>
<td>Components and cladding</td>
<td>$p = q_h (GC_p - GC_{pi})$</td>
</tr>
</tbody>
</table>

Note: $q = q_z$ for windward wall;
$q = q_h$ for leeward wall, side wall and roof, where $h$ is evaluated at mean roof height.

Table 2.6
Use Factors for Buildings and Other Structures

<table>
<thead>
<tr>
<th>Nature of Occupancy</th>
<th>Use Factor</th>
</tr>
</thead>
<tbody>
<tr>
<td>All buildings and structures except those list below</td>
<td>1.0</td>
</tr>
<tr>
<td>Buildings and structures where the occupant load is 300 or more in any one room.</td>
<td>1.15</td>
</tr>
<tr>
<td>Buildings and structures designated as essential facilities, including, but not limited to:</td>
<td>1.15</td>
</tr>
<tr>
<td>1. Hospital and other medical facilities having surgery or emergency treatment areas</td>
<td></td>
</tr>
<tr>
<td>2. Fire or rescue and police stations</td>
<td></td>
</tr>
<tr>
<td>3. Primary communication facilities and disaster operation centers</td>
<td></td>
</tr>
<tr>
<td>4. Power stations and other utilities required in an emergency</td>
<td></td>
</tr>
<tr>
<td>Buildings and structures that represent a low hazard to human life in the event of failure, such as agricultural buildings, certain temporary facilities, and minor storage facilities</td>
<td>1.15</td>
</tr>
</tbody>
</table>

CHAPTER III

MASSONEY DESIGN FOR CONVENTIONAL CONSTRUCTION 
AND WIND RESISTANT CONSTRUCTION

In order to perform a cost investigation, a 30 ft by 50 ft rectangular house with 
several typical openings is designed using the masonry provisions in ACI 530-92 (ACI, 
1992) and wind load provisions from ASCE 7-93 (ASCE, 1993) or SBC 94 (SBCCI, 
1994). It is also designed follows the provisions in CABO Code (The Council of 
American Building Officials, 1995) and SFBC (Board of County Commissioners, 1988), 
respectively.

Roof cladding design follows the provisions of APA (APA, 1995) to determine the 
thickness, grade of the roof cladding, and the connections for roof diaphragm. No design 
for detail connections are address in this thesis.

This chapter gives summary of masonry designs based on several provisions. The 
comparison of structural requirements and structural costs of these designs are discussed 
in the next chapter.

ACI 530-92 with ASCE 7-93 or SBC 94

ACI 530-92 is “Building Code Requirements for Masonry Structures.” The design 
using the provisions in ACI 530-92 represents wind resistant construction in the US. In 
order to perform a cost investigation of wind resistant construction, the selected house is 
designed following the provisions in ACI 530-92 for 70 mph, 90 mph, and 110 mph wind 
speed.

Wind loads for the design are calculated following the wind load provisions in 
ASCE 7-93 or SBC 94. Main Wind Force Resist System loads are used to compute loads 
for the shear walls, and the associated in-plane shear for the roof-to-wall connections, and 
wall-to-foundation connectors. Component and Cladding loads are used to compute the 
design loads for the roof deck, roof truss, and transverse loads on walls.
Appendix A shows the engineering data and calculations of the masonry design based on ACI 530-92 with ASCE 7-93 (70 mph case). Appendices D and E are the summary of masonry design based on ACI 530-92.

**CABO One and Two Family Dwelling Code**

The CABO Code is a commonly used building code. The design based on the CABO Code is assumed to represent conventional construction in the US. In order to perform a cost investigation of conventional construction, the selected house is designed using the provisions in CABO Code for 70 mph, 90 mph, and 110 mph wind speed.

According to the CABO Code, for exposure C classification, no detail design for wind load is required unless the design wind speed is higher than 110 mph. So, in 70 mph and 90 mph cases, the design of the selected house follows the minimum provisions specified in the CABO Code. For the 110 mph case, the wind load is given in the Design Wind Load Table in the CABO Code, and the design against wind load follows the procedures in ACI 530-92. Appendix B shows the summary of engineering data and calculations.

**South Florida Building Code**

SFBC is a building code used in South Florida area. The design based on SFBC represents wind resistant construction in the South Florida area. In order to perform a cost investigation of this wind resistant construction, the selected house is designed using the minimum provisions in SFBC. The design wind speed is 120 mph required by SFBC. Appendix C shows the engineering data of the design.
CHAPTER IV
COMPARISON OF ENGINEERING REQUIREMENTS
AND STRUCTURAL COSTS

A 30 ft by 50 ft rectangular house with several typical openings (shown in Figure 4.1 and 4.2) is investigated to make the comparison of engineering requirements and structural costs. The design using the provisions in ACI 530-92 (ACI, 1992) is compared to the structural requirements in CABO Code (The Council of American Building Officials, 1995) and SFBC (Board of County Commissioners, 1988). The difference in structural requirements is determined. Means Square Foot Costs (Means, 1995) is used to estimate the difference of costs.

Assumptions and Limitations

The following are the assumptions and limitations of the design method used in this thesis:

1. For this comparison, it is assumed that the roof framing is spaced at 24 inches in center.
2. The only design load applied on the house is wind load. The dead load and other live load are neglected.
3. The effects of openings on the design in terms of special details around these openings are neglected.

Comparison of Engineering Requirements

Tables 4.1 through 4.3 summarize the engineering requirements required by CABO Code, SFBC, and the results of structural design using the provisions in ACI 530-92. Three representative wind speeds (70, 90, and 110 mph) are selected for investigation. According to ASCE 7-93 (ASCE, 1993) wind speed map, 70 mph
represents the wind speed in inland areas of the US, 90 mph represents Atlantic and Gulf Coastal areas, and 110 mph represents extremely high wind speed area in South Florida.

Masonry Unit

CABO Code uses 6 inches CMU and SFBC uses 8 inches CMU. The design using ACI 530-92 uses 6 inches CMU in all cases (70 mph, 90 and 110 mph cases).

Wall Construction

CABO Code requires #4@48” horizontal and #4@24” vertical rebar in walls. In the 70 mph case, the design based on ACI 530-92 requires #4@48” vertical rebar. In 90 mph and 110 mph cases, more vertical rebar are required, especially in the short wall of the rectangular house. This is due principally to the longer span between the foundation and the roof for the short wall.

SFBC uses 8 inch by 12 inch tie columns with 4-#5 rebar at the corners, adjacent to some openings, and at intervals in the walls instead of providing reinforcement in the masonry. The maximum spacing of tie columns is 20 feet.

The comparison of wall construction shows that CABO Code provides appropriate reinforcement in masonry walls for the 70 and 90 mph cases. However, in the 110 mph case, the design based on CABO Code has insufficient vertical rebar in the masonry walls when compared to the design using ACI 530-92 with the ASCE 7-93 wind load provisions. This is principally due to the lower design pressures specified in the CABO Code as compared to ASCE 7-93 for the 110 mph case.

Bond Beam

The design based on CABO Code and ACI 530-92 use 6 inches by 8 inches bond beams in all cases (70 mph, 90 mph, and 110 mph cases). SFBC uses stronger bond beams: 8 inches by 12 inches with 4-#5 rebars. These bond beams work together with tie columns to frame the wall panels so no reinforcement is required in the wall panels.
Roof Cladding

CABO Code uses 3/8 inch cladding and SFBC uses 15/32 inch cladding. The design based on APA (APA, 1995) which is used for wind resistant design associated with the ASCE 7-93 and SBC 94 requires 5/16 to 3/8 inch cladding in 70 mph area, 3/8 inch cladding in 90 mph area, and 3/8 to 15/32 inch cladding in 110 mph area.

Cost Comparison

The cost investigation is based primarily on Means Square Foot Costs (Means, 1995). In this study, only the costs related to the structure and the changes relative to the codes are addressed. Materials and labor are included in the costs but no area adjustment factor is used. The cost comparison is shown in Table 4.4 and Figure 4.3.

In order to make a comparison of the total building cost, we assume that the non-structural building cost is 4 times of the structural cost of the design based on the CABO Code and non-structural building costs are the same for the designs based on CABO Code, SFBC, and ACI 530-92. Appendix F shows the basic cost data from Means Square Foot Costs and the calculations of structural costs and estimated total building costs.

SFBC versus CABO Code

The estimated structural cost for the design based on CABO Code is approximately $8,200, while the cost for the design based on SFBC is about $12,000. The difference in code requirements results in an additional structural cost of $3,800 (46% of the structural cost, 9% of the total building cost).

The cost difference is due to the higher design wind load in SFBC, which causes the design based on SFBC to use larger size masonry units and a stronger bond beam. Moreover, the design based on SFBC requires tie columns instead of reinforcement in masonry walls, which costs more money.
ACI 530-92 versus CABO Code

The estimated structural costs of the design based on ACI 530-92 are from $8,205 to $8,343 in 70 mph and 90 mph area, while the costs are from $8,631 to $8,907 in 110 mph area. In 70 mph and 90 mph area, the additional structural cost for the houses built following the CABO Code to meet the requirements of ACI 530-92, ASCE 7-93, and SBC 94 is less than $140 (1.7% of the structural cost, 0.3% of the total building cost); in 110 mph area, the additional costs range from $426 to $702 (5 to 9% of the total structural cost, 1% to 2% of the total building cost).

The cost difference in 110 mph area is due to the lesser wind resistant requirements in the CABO Code as compared with ASCE 7-93 with ACI 530-92. The CABO Code only requires reinforcement of #4 @ 24 inches in masonry wall.

SFBC versus ACI 530-92

In 110 mph cases, the estimated costs of the construction designed using ACI 530-92 is from $8,631 to $8,907. This is less than the cost of the design based on SFBC, which is $12,010.

ACI 530-92 with ASCE 7-93 versus ACI 530 -92 with SBC 94

In 70 mph and 90 mph cases, the cost of the house designed using ACI 530 with ASCE 7-93 is almost the same as that designed using ACI 530-92 with SBC 94 (SBCCI, 1994). However, in 110 mph cases, the cost of the house is almost $300 (3% of the structural cost, 0.6% of the total building cost) higher when designed using ACI 530-92 with ASCE 7-93 instead of ACI 530-92 with SBC 94.
North Elevation  1:100

East Elevation  1:100

South Elevation  1:100

Figure 4.2

Elevations of the investigated House
Figure 4.3
Structural Cost for Concrete Masonry House
Table 4.1
Summary of Engineering Requirements for Concrete Masonry House
(Inland Area, 70 mph Fastest Mile Wind Speed)

<table>
<thead>
<tr>
<th>Criteria Component</th>
<th>CABO One &amp; Two Family Dwelling Code</th>
<th>South Florida Building Code</th>
<th>ACI530-92 (Wind Loads from ASCE 7-93)</th>
<th>ACI530-92 (Wind Loads from SBC 94)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Masonry Unit</td>
<td>6&quot; CMU f m = 1500psi</td>
<td>N/A</td>
<td>6&quot; CMU f m = 1500psi</td>
<td>6&quot; CMU f m = 1500psi</td>
</tr>
<tr>
<td>Reinforcement</td>
<td>Horizontal: #4@48&quot; Vertical: #4@24&quot;</td>
<td>N/A</td>
<td>Vertical: #4@48&quot;</td>
<td>Vertical: #4@48&quot;</td>
</tr>
<tr>
<td>Wall</td>
<td>Tie Column</td>
<td>N/A</td>
<td>N/A</td>
<td>N/A</td>
</tr>
<tr>
<td>Bond Beam</td>
<td>6&quot; x 8&quot; 2 #4 rebars</td>
<td>N/A</td>
<td>6&quot; x 8&quot; 2 #4 rebars</td>
<td>6&quot; x 8&quot; 2 #4 rebars</td>
</tr>
<tr>
<td>Fastener</td>
<td>8d nails @ 6&quot;</td>
<td>N/A</td>
<td>8d nails @ 6&quot;</td>
<td>6d nails @ 6&quot;</td>
</tr>
</tbody>
</table>

* Design based on minimum requirements
**120 mph design wind speed
<table>
<thead>
<tr>
<th>Criteria</th>
<th>CABO One &amp; Two Family Dwelling Code</th>
<th>South Florida Building Code</th>
<th>ACI530-92 (Wind Loads from ASCE 7-93)</th>
<th>ACI530-92 (Wind Loads from SBC 94)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Masonry Unit</td>
<td>6&quot; CMU ( f_m = 1500 \text{psi} )</td>
<td>N/A</td>
<td>6&quot; CMU ( f_m = 1500 \text{psi} )</td>
<td>6&quot; CMU ( f_m = 1500 \text{psi} )</td>
</tr>
<tr>
<td>Reinforcement</td>
<td>Horizontal: #48&quot; Vertical: #4824&quot;</td>
<td>N/A</td>
<td>Vertical: #48&quot; Vertical: #4832&quot;</td>
<td>Vertical: #48&quot; Vertical: #4832&quot;</td>
</tr>
<tr>
<td>Tie Column</td>
<td>N/A</td>
<td>N/A</td>
<td>N/A</td>
<td>N/A</td>
</tr>
<tr>
<td>Bond Beam</td>
<td>6&quot; x 8&quot;</td>
<td>N/A</td>
<td>6&quot; x 8&quot;</td>
<td>6&quot; x 8&quot;</td>
</tr>
<tr>
<td></td>
<td>2 #4 rebars</td>
<td></td>
<td>2 #4 rebars</td>
<td>2 #4 rebars</td>
</tr>
<tr>
<td>Fastener</td>
<td>8d nails @ 6&quot;</td>
<td>N/A</td>
<td>8d nails @ 6&quot;</td>
<td>8d nails @ 6&quot;</td>
</tr>
</tbody>
</table>

* Design based on minimum requirements
** 120 mph design wind speed
### Table 4.3
Summary of Engineering Requirements for Concrete Masonry House
(�astal Area, 110 mph Fastest Mile Wind Speed)

<table>
<thead>
<tr>
<th>Component</th>
<th>CABO One &amp; Two Family Dwelling Code</th>
<th>South Florida Building Code</th>
<th>ACI530-92 (Wind Loads from ASCE 7-93)</th>
<th>ACI530-92 (Wind Loads from SBC 94)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Long Wall</td>
<td>Short Wall</td>
<td>Long Wall</td>
<td>Short Wall</td>
</tr>
<tr>
<td>Masonry Unit</td>
<td>6'' CMU $f_m = 1500$psi</td>
<td>8'' CMU $f_m = 1500$psi</td>
<td>6'' CMU $f_m = 1500$psi</td>
<td>6'' CMU $f_m = 1500$psi</td>
</tr>
<tr>
<td>Reinforcement</td>
<td>N/A</td>
<td>Vertical: #4824(^{\circ})</td>
<td>Vertical: #4832(^{\circ})</td>
<td>Vertical: #4824(^{\circ})</td>
</tr>
<tr>
<td>Wall</td>
<td></td>
<td>N/A</td>
<td>N/A</td>
<td>N/A</td>
</tr>
<tr>
<td>Tie Column</td>
<td>8'' x 12'' $#4$ #5 rebars</td>
<td>N/A</td>
<td>N/A</td>
<td>N/A</td>
</tr>
<tr>
<td>Bond Beam</td>
<td>6'' x 8'' $#4$ rebars</td>
<td>8'' x 12'' $#4$ rebars</td>
<td>6'' x 8'' $#4$ rebars</td>
<td>6'' x 8'' $#4$ rebars</td>
</tr>
<tr>
<td>Fastener</td>
<td>8d nails @ 6''</td>
<td>8d nails @ 6''</td>
<td>8d nails @ 6''</td>
<td>8d nails @ 6''</td>
</tr>
</tbody>
</table>

* 120 mph design wind speed
Table 4.4
Structural Cost for Concrete Masonry House

<table>
<thead>
<tr>
<th>Criteria Component</th>
<th>CABO One &amp; Two Family Dwelling Code</th>
<th>South Florida Building Code</th>
<th>ACI530-92 (Wind Loads from ASCE 7-93)</th>
<th>ACI530-92 (Wind Loads from SBC 94)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>70 mph</td>
<td>90 mph</td>
<td>110 mph</td>
<td>70 mph</td>
</tr>
<tr>
<td>Masonry Wall</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>($ per sq. ft)</td>
<td>$5.50</td>
<td>$5.50</td>
<td>$5.41</td>
<td>$5.50</td>
</tr>
<tr>
<td>Tie Column</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>(88 ft)</td>
<td>N/A</td>
<td>N/A</td>
<td>$36.60</td>
<td>N/A</td>
</tr>
<tr>
<td>Bond Beam</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Subtotal</td>
<td>$7321</td>
<td>$7321</td>
<td>$10939</td>
<td>$7321</td>
</tr>
<tr>
<td>Wall (625 sq ft + 576 sq ft)</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Cladding &amp; Fastener</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>($ per sq. ft)</td>
<td>$0.52</td>
<td>$0.52</td>
<td>$0.63</td>
<td>$0.52</td>
</tr>
<tr>
<td>Subtotal</td>
<td>$884</td>
<td>$884</td>
<td>$1071</td>
<td>$884</td>
</tr>
<tr>
<td>Total</td>
<td>$8205</td>
<td>$8205</td>
<td>$12010</td>
<td>$8205</td>
</tr>
</tbody>
</table>

Cost increase based on CABO Code: 0% 0% 46% 0% 2% 9% 0% 2% 5%
CHAPTER V
CONCLUSION

Engineering Requirements

The comparison of engineering requirements shows that:

1. The house designed based on CABO Code (The Council of American Building Officials, 1995), which represents many conventional houses in the US, do not meet wind resistant requirements when wind speed is 110 mph as compared to those designed using ACI 530-92 (ACI, 1992) and ASCE 7-93 (ASCE, 1993) wind provisions. In 70 mph and 90 mph cases, the design based on CABO Code is close to the design using ACI 530-92, which represents wind resistant design. However, when wind speed increases to 110 mph, there may be insufficient vertical rebar for masonry walls.

2. The house designed based on SFBC (Board of County Commissioners, 1988), which represents wind resistant houses in South Florida, has a different method of wall construction (uses tie columns and tie beams) compared to the method in CABO Code and ACI 530-92. As a result, the structural cost of the design based on SFBC is much higher.

3. Two standards are used to calculate wind pressures on the masonry walls using the design based on ACI 530-92. When using ASCE 7-93, the design requires more rebar in masonry walls, thicker roof cladding, and more roof fasteners compared to the design using SBC 94. The difference is due to the higher wind pressures when using ASCE 7-93 instead of SBC 94.
Structural Cost

The cost comparison shows that:

1. When wind speed is 70mph or 90mph, the additional costs for conventional houses, which are based on CABO Code, to comply with the wind resistant requirements in ASCE 7-93 and SBC 94, in conjunction with ACI 530-92, are less than 2 percent of the structural cost (0.3% of the total building cost); when wind speed increases to 110mph, the additional costs range from 5 to 9 percent of the structural cost (1 to 2 percent of the total building cost).

2. The additional costs for conventional houses to comply with SFBC is about 46 percent of the structural cost (9 percent of the total building cost).

3. In South Florida area, the structural cost of houses designed using ACI 530-92 is less than the houses designed using SFBC.

Finally, the incorporation of wind resistant technology into the new construction increases about 1 to 46 percent of the structural cost (0.3 to 9 percent of the total building cost), depending on where the houses are built and which wind resistant requirements are used.
REFERENCES

ACI. **Building Code Requirements for Masonry Structures.** Detroit, MI: American Concrete Institute, 1992.


Board of County Commissioners. **The Board of County Commissioners of Dade County, Ordinance 93-14: Ordinance Implementing Recommendations of the Grand Jury and Hurricane Task Force.** Miami, FL: Board of County Commissioners, 1993.


Lombard, P., Stubbs, N., & Perry, D. **Estimate of Costs to Implement the New Building Code for Windstorm Resistant Construction along the Texas Coast.** College Station, TX: Texas A&M University, 1995.


NAHB Research Center. **Comparison of South Florida Building Code and Dade County Ordinance 93-14, including ASCE 7-88 and other Special Wind Loading Requirements.** Upper Marlboro, MD: NAHB Research Center, 1993.

APPENDIX A
ENGINEERING DATA OF MASONRY DESIGN BASED ON
ACI 530-92 WITH ASCE 7 - 93 (70 MPH CASE)

This appendix shows the detail calculations of the masonry design for a 30 ft by 50 ft rectangular house (shown in Figure A.1) based on ACI 530-92 with ASCE 7-93 (70 mph case). It includes shear design and flexural design for Wall BC and Wall AB, roof diaphragm design and roof cladding design. The masonry designs for all other cases follow the same design procedure described here.

Wall BC

In-Plane Shear Design

Shear design for in-plane shear is more critical in Wall AB. See design for Wall AB.

Out-of-Plane Shear Design

Height of design pier : \( h = 8' + (3' - 9") = 9.875' \) USE \( h = 10' \)

Width of design pier : \( 6T_u = 6 \times 6" = 36" = 3' \)

Wind pressure (Residential construction, Exposure C):

\[ q_h = q_z = (0.00256)k_z(JV)^2 \]

\[ = (0.00256)(0.8)(1.0 \times 70)^2 \]

\[ = 10.04 \text{psf} \]

For C&C: \( p = q_h(GC_p - GC_{PF}) \) (Table 2.5)

\( TRIB, AREA = 3' \times 10' = 30 \text{ft}^2, GC_p = -1.4 \) (Figure 2.3)

\( GC_{PF} = 0.25 \) (Table 2.4)

\( P = 10.04(-1.4 - 0.25) = -16.6 \text{psf} \)
Shear and bend moment

Assume the wind load on the window adjacent to the design pier transfers to design pier 2 as two concentrated load (load 3); assume the wind load on the wall below and above window transfer to the design pier as distributed load (load 2); load 1 is the uniform wind load applied on the design pier.

$LOAD_1 = (16.6 \text{psf})(3 \text{ft}) = 49.8 \text{plf}$

$V_1 = V_2 = \frac{1}{2}(49.8)(10) = 249 \text{lb}$

$M_{(z=3')} = \frac{1}{2}(49.8)(3)(10 - 3) = 524 \text{ft} \cdot \text{lb}$

$LOAD_2 = (16.6 \text{psf})(3 \text{ft}) / 2 = 24.9 \text{lb/ft}$

$V_1 = \frac{(24.9)(3)(2 \cdot 10 - 3) + (24.9) \cdot 3^2}{2(10)} = 74.7 \text{lb}$

$M_{(z=3')} = (74.7)(3) - \frac{(24.9)(3)}{2}(2 \cdot 3 - 3) = 112.1 \text{ft} \cdot \text{lb}$

$LOAD_3 = (16.6 \text{psf})(3 \text{ft})(4 \text{ft}) / 4 = 49.8 \text{lb}$

$V_1 = \frac{49.8}{10}(10 - 3 + 3) = 49.8 \text{lb}$

$M_{(z=3')} = 49.8 \times 3 = 149.4 \text{ft} \cdot \text{lb}$

$V_1 = 249 + 74.7 + 49.8 = 373.5 \text{lb}$
Design out-of-plane shear = $373.5\,lb\,/\,3\,ft = 124.5\,lb\,/\,ft$

$$M = 523 + 112.1 + 149.4 = 784.5\,ft\,-\,lb$$

$$f_v = \frac{v}{bd} = \frac{124.5}{(12)(0.9)}\left(\frac{6 - 0.375}{2}\right) = 4.1\,psi$$

(2.12)

$$F_v = \sqrt{1500} = 38.73\,psi < 50\,psi$$

increased by 33%, $F_v = 1.33 \times 38.73 = 51.5 \gg 4.1\,psi$ (O.K.)

Flexural Design

Design wind load: $P = 16.6\,psf$

Bending moment: $M = 523 + 112.1 + 149.4 = 785\,ft\,-\,lb$

Uplift force: $0\,plf$

$$f_a = 0.00\,psi$$

$$r = 2.33, \quad \frac{h}{r} = \frac{10(12)}{2.33} = 51.5 < 99$$

$$F_a = \frac{1}{4} f_m \left[ 1 - \left(\frac{h}{140r}\right)^2 \right] = 324.25\,psi \times 1.33 = 431.25\,psi$$

$$F_b = \frac{1}{3} f_m = 500\,psi \times 1.33 = 665\,psi$$

$$f_b = F_b(1 - \frac{f_a}{F_a}) = 665(1 - 0) = 665\,psi$$

$$f_m = f_a + f_b = 665\,psi$$

$$a = \frac{1}{6} f_m b_2 = \frac{1}{6} (665)(3)(12) = 3990$$

$$b = -\frac{1}{2} f_m b_2 d = -\frac{1}{2} (665)(3)(12)\left(\frac{6 - 0.375}{2}\right) = -33666$$

$$c = M = 785 \times 12 = 9420$$

$$kd = \frac{-b - \sqrt{b^2 - 4ac}}{2a} = 0.29\,in < 1.00\,in$$

$$k = 0.10$$

$$f_s = (\frac{1-k}{k})n f_m = \frac{1-0.10}{0.10}\frac{29}{1.92}(665) = 90398\,psi > 32000\,psi$$
decrease $f_m$

try $f_m = 400 \text{psi}$

$$a = \frac{1}{6} f_m b_2 = \frac{1}{6} (400)(3)(12) = 2400$$

$$b = \frac{1}{2} f_m b_2 d = \frac{1}{2} (400)(3)(12)\left(6 - \frac{0.375}{2}\right) = -20250$$

$$c = 9420$$

$$kd = 0.49 \text{in}$$

$$k = 0.17$$

$$f_s = \frac{1 - 0.17}{0.17} \frac{29}{1.92} (400) = 28286 < 32000 \text{psi} \quad \text{O.K.}$$

$$A_s = \frac{C - P}{f_s} = \frac{1}{2} \frac{f_m kd b_2 - P}{f_s} = \frac{1}{2} (400)(0.49)(3)(12) - 0$$

$$0.12 \text{in}^2 / 3 = 0.04 \text{in}^2 / \text{ft}$$
Wall AB
In-Plane Shear Design

Wind Pressure for MWFRS

Windward:

\[ P = q G C_p - q_h (G C_{pT}) \]
\[ = 10.04(1.32)(0.8) - 10.04(-0.25) \]
\[ = 13.1 \text{psf} \] (Table 2.5)

Leeward:

\[ P = 10.04(1.32)(-0.36) - 10.04(-0.25) \]
\[ = -2.3 \text{psf} \]

Shear load on wall AB \((\text{HEIGHT} = 8 + \frac{3.75}{2} = 9.88 = 10)\):

\[ V = (13.1 + 2.3) \frac{10}{2} = 1155 \text{lb} \]

There are 5 piers in wall AB; design for pier 2:

\[ \Delta F_i = \frac{P}{E_m l} \left[ \left(\frac{h}{l}\right)^3 + 3 \left(\frac{h}{l}\right) \right] = \left[ \left(\frac{4}{5.3}\right)^3 + 3 \left(\frac{4}{5.3}\right) \right] \frac{P}{E_m l} \]
\[ = 2.67 \frac{P}{E_m l} \] (2.14)

\(\Delta F_i\) for other piers are listed below:

<table>
<thead>
<tr>
<th>PIER</th>
<th>(\text{h(ft)})</th>
<th>(\text{l(ft)})</th>
<th>(\text{h/l})</th>
<th>(\Delta F)</th>
<th>(V_i(\text{lb}))</th>
<th>(f_{vi}(\text{psi}))</th>
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<tbody>
<tr>
<td>1</td>
<td>7</td>
<td>2.67</td>
<td>2.62</td>
<td>25.84 (\frac{P}{E_m l})</td>
<td>31</td>
<td>0.23</td>
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<td>2</td>
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<td>301</td>
<td>1.00</td>
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<tr>
<td>3</td>
<td>4</td>
<td>3</td>
<td>1.33</td>
<td>6.34 (\frac{P}{E_m l})</td>
<td>127</td>
<td>0.82</td>
</tr>
</tbody>
</table>
Out-of-Plane Shear Design

Shear design for out-of-plane shear is more critical in Wall BC. See design for Wall BC.

Combined Axial and Flexure Design

Wall Pressure:

Trib. Area: $8' \times 5.33' = 42.6 \text{ft}^2$, $GC_p = -1.4$  \hspace{1cm} \text{(FIG 2.3)}

$P = 10.04(-1.4 - 0.25) = -16.6 \text{psf}$  \hspace{1cm} \text{(Table 2.5)}

Roof Pressure:

Trib. Area: $5.33' \times \frac{7.5'}{\cos 14'} = 42 \text{ft}^2$, $GC_p = -1.2$

$P = 10.04(-1.2 - 0.25) = -14.6 \text{psf}$

Bending moment and uplift force:
M = 918ft-lb (use the same method in the design of Wall BC)

Uplift force:

\[ P = -14.6 \left( \frac{15}{2} + 1.5 \right) = -131 \text{ plf} \]

\[ f_n = \frac{P}{A} = \frac{-131 \text{ plf}}{24 \text{ in}^2 / \text{ft}} = -5.46 \text{ psi} \]

(Area of 6" CMU Blcok)

\[ r = 2.33 \text{ in}, \quad \frac{h}{r} = \frac{8(12)}{2.33} = 41.2 < 99 \]

\[ F_a = \frac{1}{4} f_m \left[ 1 - \left( \frac{h}{140r} \right)^2 \right] = 342.5 \text{ psi} \times 1.33 = 455.56 \text{ psi} \]

\[ F_b = \frac{1}{3} f_m = 500 \text{ psi} \times 1.33 = 665 \text{ psi} \]

\[ f_b = F_b \left( 1 - \frac{f_a}{F_a} \right) = 665 \left( 1 - \frac{-5.46}{455.56} \right) = 692.97 \text{ psi} \]

\[ f_m = f_a + f_b = -5.46 + 692.97 = 667.51 \text{ psi} \]

\[ a = \frac{1}{6} f_m b_2 = \frac{1}{6} (667.51)(5.33 \text{ ft})(12 \text{ in} / \text{ft}) = 7115.66 \]

\[ b = -\frac{1}{2} f_m b_2 d = -\frac{1}{2} (667.51)(5.33)(12 \text{ in} / \text{ft}) \left( \frac{6 - 0.375}{2} \right) = -60038.35 \]

\[ c = m = 918 \text{ ft-lb} \cdot (12 \text{ in} / \text{ft}) = 110 \text{ lb} \]

\[ kd = \frac{-b - \sqrt{b^2 - 4ac}}{2a} = 0.188 \text{ in} < t_{fs} = 1.00 \text{ in} \]

\[ k = \frac{kd}{d} = \frac{0.188}{(6 - 0.375)} = 0.067 \]

\[ f_s = \left( \frac{1 - k}{k} \right) n f_m = \frac{1 - 0.067}{0.067} \frac{29}{1.92} (667.51) = 140398 \text{ psi} > \frac{4}{3} \times 24000 = 32000 \text{ psi} \]

if \( f_s > F_s \), need to decrease \( f_m \)
Try \( f_m = 350 \text{psi} \)
\[
a = \frac{1}{6} (350)(5.33)(12) = 3731
\]
\[
b = -\frac{1}{2} (350)(5.33)(12)(\frac{6 - 0.375}{2}) = -31480
\]
\[c = 11016\]
\[
k_d = \frac{-b - \sqrt{b^2 - 4ac}}{2a} = 0.37\text{in} < t_{fs} = 1.00\text{in}
\]
\[
k = 0.13
\]
\[
f_s = \frac{1 - 0.13}{0.13} \frac{29}{1.92} (350) = 35379 \text{psi} > 32000 \text{psi} \quad \text{N.G.}
\]

Try \( f_m = 333 \text{psi} \)
\[
a = 3550 \text{psi}
\]
\[
b = -29951
\]
\[c = 11016\]
\[
k_d = \frac{-b - \sqrt{b^2 - 4ac}}{2a} = 0.385\text{in} < f_{is}
\]
\[
k = 0.137
\]
\[
f_s = \frac{1 - 0.137}{0.137} \frac{29}{1.92} (333) = 31683 \text{psi} < 32000 \text{psi}
\]
\[
A_s = \frac{C - P}{f_s} = \frac{1}{2} \frac{f_m k d b_2 - P}{f_s} = \frac{1}{2} \frac{(333)(0.385)(5.33)(12) - (-131)(5.33)}{31683} = 0.151\text{in}^2
\]
\[
\frac{0.151\text{in}^2}{5.33} = 0.028\text{in}^2 / \text{ft}
\]
Diaphragm Design

Wind load (MWFRS): windward wall: 13.1 psf
                     leeward wall  -2.3 psf

\[ w = (13.1 + 2.3) \text{ psf} \cdot 8 \text{ ft} / 2 \]
\[ = 62 \text{ plf} \]
\[ M = \frac{1}{8} (62 \text{ plf}) (50 \text{ ft})^2 = 19375 \text{ ft} - \text{lb} \]
\[ T = \frac{19375 \text{ ft} - \text{lb}}{30 \text{ ft}} = 646 \text{ lb} \]
\[ A_s = \frac{646 \text{ lb}}{32000 \text{ psi}} = 0.02 \text{ in}^2 \]

Area of flange, \( A = 6 \times 3(6) = 108 \text{ in}^2 \)

Moment of Inertia, \( I = 2A\left(\frac{b}{2}\right)^2 = 2(108)(\frac{30 \times 12}{2})^2 = 6998400 \text{ in}^4 \)
\[ \Delta = \frac{5wl^4}{384EI} = \frac{5 \times 62 \text{ plf}}{384(1125000)(6998400)} = 0.001107 \]
\[ \Delta_{allow} = 0.007h = 0.007(8 \text{ ft})(12 \text{ in} / \text{ ft}) = 0.672 \text{ in} > 0.001 \text{ in} \quad \text{O.K.} \]

Roof Cladding Design

Wind load (MWFRS): windward wall: 13.1 psf
                     leeward wall  -2.3 psf

Shear at end wall:
\[ V = (13.1 + 2.3) \times 8 \times 50 / 2 / 30 / 2 \]
\[ = 52 \text{ plf} \]

Use Table 1 in APA

Roof Cladding 3/8" APA Structural plywood

Fastener: 8d@6in' o.c. (nail)
Figure A.1

Design Piers in the Investigated House
APPENDIX B
MASONRY DESIGN BASED ON CABO CODE

70 mph and 90 mph cases

In 70 mph and 90 mph cases, no design for wind load is required, so the design follows the minimum provisions in CABO Code (The Council of American Building Officials, 1995).

Masonry Units

Section 604.2 of CABO Code requires minimum thickness of masonry walls shall not be less than 6 inches for houses not greater than 9 feet. Use 6 inches CMU, $f'_m = 1,500$ psi.

Reinforcement in Masonry Wall

Section 604.11 of CABO Code (The Council of American Building Officials, 1995) requires that “the minimum area of reinforcement shall not be less than 0.002 times the gross cross-sectional area of wall, not more than two-thirds of which may be used in either direction.” (p. 73)

Gross cross-sectional of 6 inches CMU is $A = 67.5$ in$^2$/ft,

So, total reinforcement $A_s = 0.002A = 0.002(67.5) = 0.135$ in$^2$/ft,

Put one-third of total reinforcement in horizontal direction and the rest in vertical direction. The horizontal reinforcement is $A_h = (\frac{1}{3})A_s = 0.045$ in$^2$/ft (# 4 @ 48 inches)

and the vertical reinforcement is $A_v = (\frac{2}{3})A_s = 0.090$ in$^2$/ft (# 4 @ 24 inches).
Bond Beam

According to provisions in CABO Code and wind resistant practice, the bond beam is 6 inch by 8 inch with 2 # 4 rebars.

Roof Cladding

According to Table 503.2.1.1a of CABO Code, the minimum thickness of plywood roof cladding is 3/8 inch (assume rafters are spaced at 24 inches). According to Table 602.3a of CABO Code, the fastener of roof cladding to roof framing shall be 8d nails spacing at 6 / 12 inches at edge / intermediate.
110 mph case

In 110 mph cases, design for wind load is required, the design loads are given in Design Wind Load Table: wall pressure is 35 psf and roof uplift pressure is 38 psf. Masonry design follows the provisions in ACI 530-92 (ACI, 1992).

Shear Design for Out-of-plane Shear

Design Pier: Design Pier in Wall BC (shown in Figure A.1)
Design Wind Speed: 110 mph
Design Wind Pressure: 35 psf
Maximum Out-of-plane Shear: \( V = 262.5 \text{ lb. / ft.} \)
Thickness of CMU: \( T_u = 6 \text{ inches} \)
Strength of Masonry: \( f_m' = 1,500 \text{ psi} \)
Out-of-plane shear stress: \( f_v = 8.6 \text{ psi} \)
Allowable shear stress without reinforcement: \( F_v = 51.5 \text{ psi} \)

\( F_v \gg f_v \), so no reinforcement for out-of-plane shear required.
Shear Design for In-plane Shear

Design Pier: Design Pier in Wall AB (shown in Figure A.1)
Design Wind Speed: 110 mph

Design Wind Pressure: 35 psf
Shear Load in Wall AB: \( V = 5250 \text{ lb.} \)
Thickness of CMU: \( T_u = 6 \text{ inches} \)
Strength of Masonry: \( f_m = 1,500 \text{ psi} \)
Shear Forces distributed to the Design Pier: \( V_{pier} = 1,363 \text{ lb.} \)
Shear stress in the design pier: \( f_v = 4.5 \text{ psi} \)
Allowable shear stress without reinforcement: \( F_v = 62.2 \text{ psi} \)

\[ F_v \gg f_v, \text{ so no reinforcement for in-plane shear required.} \]
Flexural Design

Design Pier: Design Pier in Wall AB (shown in Figure A.1)
Design Wind Speed: 110 mph

Design Wind Pressure: 35 psf
Design Uplift Force: - 342 plf
Maximum Bending Moment: \( M = 1940 \text{ ft-lb} \)
Thickness of CMU: \( T_u = 6 \text{ inches} \)
Strength of Masonry: \( f_m = 1,500 \text{ psi} \)

Vertical reinforcement for flexural: \( A_s = 0.06 \text{ in}^2 / \text{ ft}, \# 4 @ 32 \text{ inches} \)

Design Pier: Design Pier in Wall BC (shown in Figure A.1)
Design Wind Speed: 110 mph

Design Wind Pressure: 35 psf
Design Uplift Force: 0 plf
Maximum Bending Moment: \( M = 1654 \text{ ft-lb} \)
Thickness of CMU: \( T_u = 6 \text{ inches} \)
Strength of Masonry: \( f_m = 1,500 \text{ psi} \)

Vertical reinforcement for flexural: \( A_s = 0.08 \text{ in}^2 / \text{ ft}, \# 4 @ 24 \text{ inches} \)
Diaphragm Design

Design Wind Speed: 110 mph

Design Wind Pressure: 35 psf
Lateral wind load: $w_{roof} = 280$ plf

Thickness of CMU: $T_u = 6$ inches

Strength of Masonry: $f'_m = 1,500$ psi

Bond Beam: 6 in x 8 in, $A_s = 0.09$ in$^2$, 2 # 4

Deflection of Bond Beam: 0.005 in

Deflection Limit: 0.672 in OK
Roof Cladding Design

Design Wind Speed: 110 mph

Design Wind Pressure: 35 psf
Shear in Diaphragm in End Wall: 116 plf
Thickness of CMU: $T_w = 6$ inches
Strength of Masonry: $f'_m = 1,500$ psi

Roof Cladding: 3/8 inch APA Structural I Plywood, rafters spaced at 24 inches

Fastener of roof cladding to roof framing: 8d @ 6 inches o.c.
The design of the selected house follows the minimum provisions in SFBC (Board of County Commissioners, 1988).

**Masonry Units**

Section 2704.2 of SFBC requires minimum thickness of masonry walls shall not be less than 8 inches. Use 8 inches CMU, $f'_m = 1,500$ psi.

**Wall Construction**

Section 2704.2 of SFBC shows the minimum requirements of wall construction. According to the wall construction provisions, use 11 tie columns (8 inches by 12 inches with 4 #5 rebars) in the masonry wall.

**Bond Beam**

According to provisions in Section 2704.2 of SFBC, the bond beam is 8 inches by 12 inches with 4 #5 rebars.

**Roof Cladding**

According to Section 2909.2 of SFBC, the minimum thickness of plywood roof cladding is 15/32 inch (assume rafters space at 24 inches). The fastener of roof cladding to roof framing shall be 8d nails spacing at 6 inches at edge.
The masonry design based on ACI 530-92 designs the critical wall piers of the selected house (shown in Figure A.1). The following are the engineering data.

**70 mph cases**

Shear Design for Out-of-plane Shear

Design Pier: Design Pier in Wall BC (shown in Figure A.1)
Design Wind Speed: 70 mph

Design Wind Pressure: 16.6 psf
Maximum Out-of-plane Shear: \( V = 124.5 \text{ lb. / ft.} \)
Thickness of CMU: \( T_u = 6 \text{ inches} \)
Strength of Masonry: \( f_m = 1,500 \text{ psi} \)
Out-of-plane shear stress: \( f_v = 4.1 \text{ psi} \)
Allowable shear stress without reinforcement: \( F_v = 51.5 \text{ psi} \)

\( F_v \gg f_v \), so no reinforcement for out-of-plane shear required.
Shear Design for In-plane Shear

Design Pier: Design Pier in Wall AB (shown in Figure A.1)
Design Wind Speed: 70 mph

Design Wind Pressure: windward: 13.1 psf; leeward: -2.3 psf
Shear Load in Wall AB: $V = 1155$ lb.
Thickness of CMU: $T_u = 6$ inches
Strength of Masonry: $f_m = 1,500$ psi
Shear Forces distribute to the Design Pier: $V_{pier} = 301$ lb.
Shear stress in the design pier: $f_v = 1.0$ psi
Allowable shear stress without reinforcement: $F_y = 62.2$ psi

$F_y >> f_v$, so no reinforcement for in-plane shear required.
Flexural Design

Design Pier: Design Pier in Wall AB (shown in Figure A.1)

Design Wind Speed: 70 mph

Design Wind Pressure: wall: 16.6 psf; roof: -14.6 psf

Design Uplift Force: -131 plf

Maximum Bending Moment: \( M = 918 \text{ ft-lb} \)

Thickness of CMU: \( T_u = 6 \text{ inches} \)

Strength of Masonry: \( f_m' = 1,500 \text{ psi} \)

Vertical reinforcement for flexural: \( A_s = 0.03 \text{ in}^2 / \text{ ft}, \# 4 @ 48 \text{ inches} \)

Design Pier: Design Pier in Wall BC (shown in Figure A.1)

Design Wind Speed: 70 mph

Design Wind Pressure: 16.6 psf

Design Uplift Force: 0 plf

Maximum Bending Moment: \( M = 785 \text{ ft-lb} \)

Thickness of CMU: \( T_u = 6 \text{ inches} \)

Strength of Masonry: \( f_m' = 1,500 \text{ psi} \)

Vertical reinforcement for flexural: \( A_s = 0.04 \text{ in}^2 / \text{ ft}, \# 4 @ 48 \text{ inches} \)
Diaphragm Design

Design Wind Speed: 70 mph

Design Wind Pressure: windward: 13.1 psf; leeward: -2.3 psf
Later wind load: \( w_{\text{roof}} = 62 \text{ plf} \)

Thickness of CMU: \( T_u = 6 \text{ inches} \)

Strength of Masonry: \( f'_m = 1,500 \text{ psi} \)

Bond Beam: 6 in x 8 in, \( A_s = 0.02 \text{ in}^2, 2 \# 4 \)

Deflection of Bond Beam: 0.001 in
Deflection Limit: 0.672 in OK
Roof Cladding Design

Design Wind Speed: 70 mph

Design Wind Pressure: windward: 13.1 psf; leeward: -2.3 psf

Shear in Diaphragm in End Wall: 52 plf

Thickness of CMU: $T_u = 6$ inches

Strength of Masonry: $f'_m = 1,500$ psi

Roof Cladding: 3/8 inch APA Structural I Plywood

Fastener of roof cladding to roof framing: 8d @ 6 inches o.c.
90 mph cases
Shear Design for Out-of-plane Shear

Design Pier: Design Pier in Wall BC (shown in Figure A.1)
Design Wind Speed: 90 mph

Design Wind Pressure: 27.4 psf
Maximum Out-of-plane Shear: $V = 205.5 \text{ lb. / ft.}$
Thickness of CMU: $T_u = 6 \text{ inches}$
Strength of Masonry: $f_m' = 1,500 \text{ psi}$
Out-of-plane shear stress: $f_v = 6.8 \text{ psi}$
Allowable shear stress without reinforcement: $F_y = 51.5 \text{ psi}$

$F_y >> f_v$, so no reinforcement for out-of-plane shear required.
Shear Design for In-plane Shear

Design Pier: Design Pier in Wall AB (shown in Figure A.1)
Design Wind Speed: 90 mph

Design Wind Pressure: windward: 21.7 psf; leeward: - 3.7 psf
Shear Load in Wall AB: $V = 1905$ lb.
Thickness of CMU: $T_u = 6$ inches
Strength of Masonry: $f_m' = 1,500$ psi

Shear Forces distribute to the Design Pier: $V_{pier} = 495$ lb.
Shear stress in the design pier: $f_v = 1.65$ psi
Allowable shear stress without reinforcement: $F_y = 62.2$ psi

$F_y >> f_v$, so no reinforcement for in-plane shear required.
Flexural Design

Design Pier: Design Pier in Wall AB (shown in Figure A.1)
Design Wind Speed: 90 mph

Design Wind Pressure: wall: 27.4 psf; roof: -24.1 psf
Design Uplift Force: -217 plf
Maximum Bending Moment: $M = 1515$ ft-lb
Thickness of CMU: $T_u = 6$ inches
Strength of Masonry: $f_m = 1500$ psi

Vertical reinforcement for flexural: $A_s = 0.05 \text{ in}^2 / \text{ft}, \# 4 @ 48$ inches

Design Pier: Design Pier in Wall BC (shown in Figure A.1)
Design Wind Speed: 90 mph

Design Wind Pressure: 27.4 psf
Design Uplift Force: 0 plf
Maximum Bending Moment: $M = 1296$ ft-lb
Thickness of CMU: $T_u = 6$ inches
Strength of Masonry: $f_m = 1500$ psi

Vertical reinforcement for flexural: $A_s = 0.07 \text{ in}^2 / \text{ft}, \# 4 @ 32$ inches
Diaphragm Design

Design Wind Speed: 90 mph

Design Wind Pressure:
- Windward: 21.7 psf; Leeward: -3.7 psf

Later wind load:
- $w_{\text{roof}} = 102$ plf

Thickness of CMU: $T_u = 6$ inches

Strength of Masonry: $f'_m = 1500$ psi

Bond Beam:
- 6 in x 8 in, $A_s = 0.03$ in$^2$, 2 # 4

Deflection of Bond Beam: 0.002 in

Deflection Limit: 0.672 in OK
Roof Cladding Design

Design Wind Speed: 90 mph

Design Wind Pressure:
- windward: 21.7 psf
- leeward: -3.7 psf

Shear in Diaphragm in End Wall: 85 plf

Thickness of CMU: $T_u = 6$ inches

Strength of Masonry: $f_m' = 1,500$ psi

Roof Cladding: 3/8 inch APA Structural I Plywood

Fastener of roof cladding to roof framing: 8d @ 6 inches o.c.
110 mph cases
Shear Design for Out-of-plane Shear

Design Pier: Design Pier in Wall BC (shown in Figure A.1)
Design Wind Speed: 110 mph

Design Wind Pressure: 41.0 psf
Maximum Out-of-plane Shear: $V = 307$ lb. / ft.
Thickness of CMU: $T_u = 6$ inches
Strength of Masonry: $f_m = 1,500$ psi
Out-of-plane shear stress: $f_v = 10.1$ psi
Allowable shear stress without reinforcement: $F_v = 51.5$ psi

$F_v \gg f_v$, so no reinforcement for out-of-plane shear required.
Shear Design for In-plane Shear

Design Pier: Design Pier in Wall AB (shown in Figure A.1)
Design Wind Speed: 110 mph

Design Wind Pressure: windward: 32.4 psf; leeward: - 5.6 psf
Shear Load in Wall AB: $V = 2850$ lb.
Thickness of CMU: $T_u = 6$ inches
Strength of Masonry: $f_m = 1,500$ psi

Shear Forces distribute to the Design Pier: $V_{pier} = 740$ lb.
Shear stress in the design pier: $f_v = 2.5$ psi
Allowable shear stress without reinforcement: $F_y = 62.2$ psi

$F_y >> f_v$, so no reinforcement for in-plane shear required.
Flexural Design

Design Pier:  
Design Wind Speed: 110 mph

Design Wind Pressure: 
Design Uplift Force: 
Maximum Bending Moment: 
Thickness of CMU: 
Strength of Masonry: 

Vertical reinforcement for flexural:  

Design Pier in Wall AB (shown in Figure A.1)

Design Pier:  
Design Wind Speed: 110 mph

Design Wind Pressure: 41.0 psf  
Design Uplift Force: 0 plf  
Maximum Bending Moment: \( M = 1939 \text{ ft-lb} \)  
Thickness of CMU: \( T_u = 6 \text{ inches} \)  
Strength of Masonry: \( f_m = 1,500 \text{ psi} \)  

Vertical reinforcement for flexural:  

Design Pier in Wall BC (shown in Figure A.1)
Diaphragm Design

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<th>Parameter</th>
<th>Value</th>
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<tbody>
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</tr>
<tr>
<td>Design Wind Pressure:</td>
<td>windward: 32.4 psf; leeward: -5.6 psf</td>
</tr>
<tr>
<td>Later wind load:</td>
<td>$w_{roof} = 152$ plf</td>
</tr>
<tr>
<td>Thickness of CMU:</td>
<td>$T_u = 6$ inches</td>
</tr>
<tr>
<td>Strength of Masonry:</td>
<td>$f'_m = 1,500$ psi</td>
</tr>
<tr>
<td>Bond Beam:</td>
<td>6 in x 8 in, $A_s = 0.05$ in$^2$, 2 # 4</td>
</tr>
<tr>
<td>Deflection of Bond Beam:</td>
<td>0.003 in</td>
</tr>
<tr>
<td>Deflection Limit:</td>
<td>0.672 in</td>
</tr>
</tbody>
</table>
Roof Cladding Design

Design Wind Speed: 110 mph

Design Wind Pressure: windward: 21.7 psf; leeward: -2.3 psf

Shear in Diaphragm in End Wall: 127 plf

Thickness of CMU: $T_u = 6$ inches

Strength of Masonry: $f'_m = 1,500$ psi

Roof Cladding: 3/8 inch APA Structural I Plywood

Fastener of roof cladding to roof framing: 8d @ 6 inches o.c.
APPENDIX E
MASSONRY DESIGN BASED ON ACI 530-92 WITH SBC 94

70 mph cases
Shear Design for Out-of-plane Shear

Design Pier: Design Pier in Wall BC (shown in Figure A.1)
Design Wind Speed: 70 mph
Design Wind Pressure: 14.2 psf
Maximum Out-of-plane Shear: \( V = 107 \text{ lb. / ft.} \)
Thickness of CMU: \( T_u = 6 \text{ inches} \)
Strength of Masonry: \( f_m = 1,500 \text{ psi} \)
Out-of-plane shear stress: \( f_v = 3.6 \text{ psi} \)
Allowable shear stress without reinforcement: \( F_v = 51.5 \text{ psi} \)

\( F_v >> f_v \), so no reinforcement for out-of-plane shear required.
### Shear Design for In-plane Shear

**Design Pier:**
Design Pier in Wall AB (shown in Figure A.1)

**Design Wind Speed:**
70 mph

**Design Wind Pressure:**
windward: 8.0 psf; leeward: -3.0 psf

**Shear Load in Wall AB:**
\[ V = 825 \text{ lb.} \]

**Thickness of CMU:**
\[ T_u = 6 \text{ inches} \]

**Strength of Masonry:**
\[ f'_m = 1,500 \text{ psi} \]

**Shear Forces distribute to the Design Pier:**
\[ V_{pier} = 214 \text{ lb.} \]

**Shear stress in the design pier:**
\[ f_v = 0.7 \text{ psi} \]

**Allowable shear stress without reinforcement:**
\[ F_v = 62.2 \text{ psi} \]

\[ F_v >> f_v, \text{ so no reinforcement for in-plane shear required.} \]
## Flexural Design

<table>
<thead>
<tr>
<th>Design Pier:</th>
<th>Design Pier in Wall AB (shown in Figure A.1)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Design Wind Speed:</td>
<td>70 mph</td>
</tr>
<tr>
<td>Design Wind Pressure:</td>
<td>wall: 14.2 psf; roof: - 14.2 psf</td>
</tr>
<tr>
<td>Design Uplift Force:</td>
<td>- 128 plf</td>
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<tr>
<td>Maximum Bending Moment:</td>
<td>$M = 787$ ft-lb</td>
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<tr>
<td>Thickness of CMU:</td>
<td>$T_u = 6$ inches</td>
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<tr>
<td>Strength of Masonry:</td>
<td>$f_m = 1,500$ psi</td>
</tr>
<tr>
<td>Vertical reinforcement for flexural:</td>
<td>$A_s = 0.02$ in$^2$/ft, # 4 @ 48 inches</td>
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</tbody>
</table>

<table>
<thead>
<tr>
<th>Design Pier:</th>
<th>Design Pier in Wall BC (shown in Figure A.1)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Design Wind Speed:</td>
<td>70 mph</td>
</tr>
<tr>
<td>Design Wind Pressure:</td>
<td>14.2 psf</td>
</tr>
<tr>
<td>Design Uplift Force:</td>
<td>0 plf</td>
</tr>
<tr>
<td>Maximum Bending Moment:</td>
<td>$M = 671$ ft-lb</td>
</tr>
<tr>
<td>Thickness of CMU:</td>
<td>$T_u = 6$ inches</td>
</tr>
<tr>
<td>Strength of Masonry:</td>
<td>$f_m = 1,500$ psi</td>
</tr>
<tr>
<td>Vertical reinforcement for flexural:</td>
<td>$A_s = 0.03$ in$^2$/ft, # 4 @ 48 inches</td>
</tr>
</tbody>
</table>
Diaphragm Design

Design Wind Speed: 70 mph

Design Wind Pressure: windward: 8 psf; leeward: -3 psf

Later wind load: \( w_{\text{roof}} = 44 \text{ plf} \)

Thickness of CMU: \( T_u = 6 \text{ inches} \)

Strength of Masonry: \( f_m' = 1,500 \text{ psi} \)

Bond Beam: 6 in x 8 in, \( A_s = 0.014 \text{ in}^2, 2 \# 4 \)

Deflection of Bond Beam: 0.0008 in

Deflection Limit: 0.672 in OK
Roof Cladding Design

Design Wind Speed: 70 mph

Design Wind Pressure: windward: 8 psf; leeward: -3 psf

Shear in Diaphragm in End Wall: 37 plf

Thickness of CMU: $T_u = 6$ inches

Strength of Masonry: $f_m' = 1,500$ psi

Roof Cladding: 5/16 inch APA Structural I Plywood

Fastener of roof cladding to roof framing: 6d @ 6 inches o.c.
90 mph cases

Shear Design for Out-of-plane Shear

Design Pier: Design Pier in Wall BC (shown in Figure A.1)

Design Wind Speed: 90 mph

Design Wind Pressure: 23.6 psf

Maximum Out-of-plane Shear: \( V = 178 \text{ lb.} / \text{ft.} \)

Thickness of CMU: \( T_u = 6 \text{ inches} \)

Strength of Masonry: \( f_m = 1,500 \text{ psi} \)

Out-of-plane shear stress: \( f_v = 5.9 \text{ psi} \)

Allowable shear stress without reinforcement: \( F_v = 51.5 \text{ psi} \)

\( F_v \gg f_v \), so no reinforcement for out-of-plane shear required.
Shear Design for In-plane Shear

Design Pier: Design Pier in Wall AB (shown in Figure A.1)
Design Wind Speed: 90 mph

Design Wind Pressure: windward: 13.2 psf; leeward: - 4.4 psf
Shear Load in Wall AB: \( V = 1320 \text{ lb.} \)
Thickness of CMU: \( T_u = 6 \text{ inches} \)
Strength of Masonry: \( f_m = 1500 \text{ psi} \)
Shear Forces distribute to the Design Pier: \( V_{\text{pier}} = 343 \text{ lb.} \)
Shear stress in the design pier: \( f_v = 1.1 \text{ psi} \)
Allowable shear stress without reinforcement: \( F_v = 62.2 \text{ psi} \)

\( F_v >> f_v \), so no reinforcement for in-plane shear required.
Flexural Design

Design Pier: Design Pier in Wall AB (shown in Figure A.1)
Design Wind Speed: 90 mph

Design Wind Pressure: wall: 23.6 psf; roof: - 23.6 psf
Design Uplift Force: - 212 plf
Maximum Bending Moment: $M = 1305$ ft-lb
Thickness of CMU: $T_u = 6$ inches
Strength of Masonry: $f_m' = 1,500$ psi

Vertical reinforcement for flexural: $A_s = 0.04 \text{ in}^2 / \text{ft}, \# \ 4 @ 48 \text{ inches}$

Design Pier: Design Pier in Wall BC (shown in Figure A.1)
Design Wind Speed: 90 mph

Design Wind Pressure: 23.6 psf
Design Uplift Force: 0 plf
Maximum Bending Moment: $M = 1116$ ft-lb
Thickness of CMU: $T_u = 6$ inches
Strength of Masonry: $f_m' = 1,500$ psi

Vertical reinforcement for flexural: $A_s = 0.06 \text{ in}^2 / \text{ft}, \# \ 4 @ 32 \text{ inches}$
Diaphragm Design

Design Wind Speed: 90 mph

Design Wind Pressure: windward: 13.2 psf; leeward: - 4.4 psf

Later wind load: $w_{roof} = 70$ plf

Thickness of CMU: $T_u = 6$ inches

Strength of Masonry: $f_m' = 1,500$ psi

Bond Beam: 6 in x 8 in, $A_s = 0.02$ in$^2$, 2 # 4

Deflection of Bond Beam: 0.001 in

Deflection Limit: 0.672 in OK
Roof Cladding Design

Design Wind Speed: 90 mph

Design Wind Pressure:
- windward: 13.2 psf; roof: -4.4 psf

Shear in Diaphragm in End Wall: 58 plf

Thickness of CMU: $T_u = 6$ inches

Strength of Masonry: $f_m' = 1,500$ psi

Roof Cladding: 3/8 inch APA Structural I Plywood

Fastener of roof cladding to roof framing: 8d @ 6 inches o.c.
110 mph cases

Shear Design for Out-of-plane Shear

Design Pier: Design Pier in Wall BC (shown in Figure A.1)
Design Wind Speed: 110 mph

Design Wind Pressure: 35.2 psf
Maximum Out-of-plane Shear: \( V = 264 \text{ lb. / ft.} \)
Thickness of CMU: \( T_u = 6 \text{ inches} \)
Strength of Masonry: \( f_m = 1,500 \text{ psi} \)
Out-of-plane shear stress: \( f_v = 8.9 \text{ psi} \)
Allowable shear stress without reinforcement: \( F_v = 51.5 \text{ psi} \)

\( F_v >> f_v \), so no reinforcement for out-of-plane shear required.
Shear Design for In-plane Shear

Design Pier: Design Pier in Wall AB (shown in Figure A.1)
Design Wind Speed: 110 mph

Design Wind Pressure: windward: 19.8 psf; leeward: - 6.6 psf
Shear Load in Wall AB: $V = 1980$ lb.
Thickness of CMU: $T_u = 6$ inches
Strength of Masonry: $f_m' = 1,500$ psi
Shear Forces distribute to the Design Pier: $V_{pier} = 514$ lb.
Shear stress in the design pier: $f_v = 1.7$ psi
Allowable shear stress without reinforcement: $F_r = 62.2$ psi

$F_r >> f_v$, so no reinforcement for in-plane shear required.
Flexural Design

Design Pier: Design Pier in Wall AB (shown in Figure A.1)
Design Wind Speed: 110 mph

Design Wind Pressure: wall: 35.2 psf; roof: -35.2 psf
Design Uplift Force: -317 plf
Maximum Bending Moment: $M = 1946 \text{ ft-lb}$
Thickness of CMU: $T_u = 6$ inches
Strength of Masonry: $f_{m'} = 1,500 \text{ psi}$

Vertical reinforcement for flexural: $A_s = 0.06 \text{ in}^2 / \text{ ft}, \# 4 @ 32$ inches

Design Pier: Design Pier in Wall BC (shown in Figure A.1)
Design Wind Speed: 110 mph

Design Wind Pressure: 35.2 psf
Design Uplift Force: 0 plf
Maximum Bending Moment: $M = 1664 \text{ ft-lb}$
Thickness of CMU: $T_u = 6$ inches
Strength of Masonry: $f_{m'} = 1,500 \text{ psi}$

Vertical reinforcement for flexural: $A_s = 0.08 \text{ in}^2 / \text{ ft}, \# 4 @ 24$ inches
## Diaphragm Design

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Design Wind Speed</td>
<td>110 mph</td>
</tr>
<tr>
<td>Design Wind Pressure</td>
<td>windward: 19.8 psf; roof: -6.6 psf</td>
</tr>
<tr>
<td>Later wind load</td>
<td>$w_{\text{roof}} = 105 \text{ plf}$</td>
</tr>
<tr>
<td>Thickness of CMU</td>
<td>$T_u = 6 \text{ inches}$</td>
</tr>
<tr>
<td>Strength of Masonry</td>
<td>$f'_m = 1,500 \text{ psi}$</td>
</tr>
<tr>
<td>Bond Beam</td>
<td>6 in x 8 in, $A_s = 0.03 \text{ in}^2$, 2 # 4</td>
</tr>
<tr>
<td>Deflection of Bond Beam</td>
<td>0.002 in</td>
</tr>
<tr>
<td>Deflection Limit</td>
<td>0.672 in, OK</td>
</tr>
</tbody>
</table>
Roof Cladding Design

Design Wind Speed: 110 mph

Design Wind Pressure: windward: 19.8 psf; leeward: -6.6 psf

Shear in Diaphragm in End Wall: 88 plf

Thickness of CMU: $T_u = 6$ inches

Strength of Masonry: $f'_m = 1,500$ psi

Roof Cladding: 3/8 inch APA Structural I Plywood

Fastener of roof cladding to roof framing: 8d @ 6 inches o.c.
APPENDIX F
COST COMPARISON DATA

Basic Cost Data from Means Square Foot Data

Masonry Wall

6 inches CMU with no reinforcement: $5.02 /sq. ft.
6 inches CMU with #4@48 inches reinforcement: $5.50 /sq. ft.
6 inches CMU with #4@32 inches reinforcement: $5.74 /sq. ft.
6 inches CMU with #4@24 inches reinforcement: $5.98 /sq. ft.
6 inches CMU with #4@16 inches reinforcement: $6.46 /sq. ft.
8 inches CMU with no reinforcement: $5.41 /sq. ft.

Tie Column

8 inches by 12 inches reinforced column: $36.60 /ln. ft.

Bond Beam

6 inches by 8 inches bond beam, 2-#4 bars: $4.47 /ln. ft.
8 inches by 12 inches bond beam, 4-#5 bars: $7.63 /ln. ft.

Roof Cladding

5/16 inch plywood cladding on roof: $0.50 /sq. ft.
3/8 inch plywood cladding on roof: $0.52 /sq. ft.
15/32 inch plywood cladding on roof: $0.63 /sq. ft.

Size of the Building Components

The selected house is 30 ft by 50ft rectangular construction with 3:12 wood-frame roof. Excluding the openings, the area of the long walls is 625 square feet, and the area of short walls is 576 square feet. Assume that the reinforcement for the wall piers under and above the openings are the same as the reinforcement of the wall piers between openings. The roof area is 1700 square feet.
The length of bond beam is 160 ft.
For the design based on SFBC, there are 11 tie columns in the masonry wall, so the total length of tie columns is 88 ft.

**Cost Comparison**

<table>
<thead>
<tr>
<th>CABO Code</th>
<th>Masonry Wall:</th>
<th>Bond Beam:</th>
<th>Subtotal:</th>
</tr>
</thead>
<tbody>
<tr>
<td>SFBC</td>
<td>$5.50 \times 1201 = $6606</td>
<td>$4.47 \times 160 = $715</td>
<td>$7321</td>
</tr>
<tr>
<td>SFBC</td>
<td>$5.41 \times 1201 = $6497</td>
<td>$36.60 \times 88 = $3221</td>
<td>$10939</td>
</tr>
<tr>
<td>SFBC</td>
<td>$0.63 \times 1700 = $1071</td>
<td>$0.52 \times 1700 = $884</td>
<td>$1071</td>
</tr>
<tr>
<td>SFBC</td>
<td>$5.50 \times 1201 = $6606</td>
<td>$4.47 \times 160 = $715</td>
<td>$7321</td>
</tr>
</tbody>
</table>

**ACI 530-92 with ASCE 7-93 (70 mph)**

| Masonry Wall: | $5.50 \times 1201 = $6606 |
| Bond Beam:    | $4.47 \times 160 = $715 |
| Subtotal:     | $7321 |
Roof Cladding & Fastener: \$0.52 \times 1700 = \$884
Subtotal: \$884
Total Structural Cost: \$8205
Non-structural Cost: \$32820
Total Building Cost: \$41025

**ACI 530-92 with ASCE 7-93 (90 mph)**

Masonry Wall: \$5.50 \times 625 + \$5.74 \times 576 = \$6744
Bond Beam: \$4.47 \times 160 = \$715
Subtotal: \$7459
Roof Cladding & Fastener: \$0.52 \times 1700 = \$884
Subtotal: \$884
Total Structural Cost: \$8343
Non-structural Cost: \$32820
Total Building Cost: \$41163

**ACI 530-92 with ASCE 7-93 (110 mph)**

Masonry Wall: \$5.74 \times 625 + \$6.46 \times 576 = \$7308
Bond Beam: \$4.47 \times 160 = \$715
Subtotal: \$8023
Roof Cladding & Fastener: \$0.52 \times 1700 = \$884
Subtotal: \$884
Total Structural Cost: \$8907
Non-structural Cost: \$32820
Total Building Cost: \$41727

**ACI 530-92 with SBC 94 (70 mph)**

Masonry Wall: \$5.50 \times 1201 = \$6606
Bond Beam: \$4.47 \times 160 = \$715
Subtotal: \$7321
Roof Cladding & Fastener: \$0.50 \times 1700 = \$850
ACI 530-92 with SBC 94 (90 mph)

Masonry Wall: \[5.50 \times 625 + 5.74 \times 576 = 6744\] 
Bond Beam: \[4.47 \times 160 = 715\] 
Subtotal: \[7459\] 
Roof Cladding & Fastener: \[0.52 \times 1700 = 884\] 
Subtotal: \[884\] 
Total Structural Cost: \[8343\] 
Non-structural Cost: \[32820\] 
Total Building Cost: \[41163\]

ACI 530-92 with SBC 94 (110 mph)

Masonry Wall: \[5.74 \times 625 + 5.98 \times 576 = 7032\] 
Bond Beam: \[4.47 \times 160 = 715\] 
Subtotal: \[7747\] 
Roof Cladding & Fastener: \[0.52 \times 1700 = 884\] 
Subtotal: \[884\] 
Total Structural Cost: \[8631\] 
Non-structural Cost: \[32820\] 
Total Building Cost: \[41451\]

Subtotal: \$850\] 
Total Structural Cost: \$8171\] 
Non-structural Cost: \$32820\] 
Total Building Cost: \$40991\]
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_____________________________  ____________________
Student's Signature             Date

Disagree (Permission is not granted.)

_____________________________  ____________________
Student's Signature             Date