Structural Design of Insulating Concrete Form Walls in Residential Construction

Prepared by NAHB Research Center, Inc. Upper Marboro, Maryland
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Insulating concrete forms are hollow blocks, planks, or panels that can be constructed of rigid foam plastic insulation, a composite of cement and foam insulation, a composite of cement and wood chips, or other suitable insulating material that has the ability to act as forms for cast-in-place concrete walls. The forms typically remain in place after the concrete is cured to provide added insulation and are not considered to add to the structural capacity of the wall.

Although insulating concrete forms (ICFs) have been used successfully and extensively in Europe for over 60 years, it is only in the last five years that the system has become more widely used in the United States. The resistance to ICFs in the United States is waning as builders and homeowners become more familiar with the product and its capabilities. Some of the capabilities include the inherent strength of concrete construction in resisting high winds from hurricanes and tornadoes, the energy efficiency provided by insulating forms, the ability to reduce outside noise providing a quiet home, and the natural resistance to damage caused by termites. All of these attributes lead to a more durable form of construction. Recent unpredictable fluctuations in lumber prices and problems cited with the quality of lumber have also provided a major stimulus for the increased use of ICFs in light residential and commercial construction.

Even though ICFs have existed in the United States for almost 20 years, much of the resistance to ICFs in the United States is due to a lack of efficient design guidelines for residential structural concrete. *Structural Design of Insulating Concrete Form Walls in Residential Construction* presents a cost-effective and practical design procedure for residential buildings that reduces the cost of construction without sacrificing reliability, energy efficiency, or durability.
This document (PCA R&D Serial No. 2164) was prepared by the NAHB Research Center under sponsorship of the Portland Cement Association (PCA). The contents of this paper reflect the views of the authors, who are responsible for the facts and accuracy of the data presented. The contents do not necessarily reflect the views of the Portland Cement Association.

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# Table of Contents

<table>
<thead>
<tr>
<th>Section</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>FOREWORD</td>
<td>iii</td>
</tr>
<tr>
<td>ACKNOWLEDGMENTS</td>
<td>v</td>
</tr>
<tr>
<td>INTRODUCTION</td>
<td>I-1</td>
</tr>
<tr>
<td>CHAPTER 1 - ICF DESIGN PROCEDURE</td>
<td></td>
</tr>
<tr>
<td>1.1 Application And Limitations</td>
<td>1</td>
</tr>
<tr>
<td>1.2 Structural Reinforced Concrete Walls</td>
<td>2</td>
</tr>
<tr>
<td>1.3 Structural Plain Concrete Walls</td>
<td>10</td>
</tr>
<tr>
<td>1.4 Lintels</td>
<td>15</td>
</tr>
<tr>
<td>1.5 Footing Connections</td>
<td>19</td>
</tr>
<tr>
<td>1.6 Roof Connection: Bolted Sill Plate</td>
<td>24</td>
</tr>
<tr>
<td>1.7 Roof Connection: Strap</td>
<td>28</td>
</tr>
<tr>
<td>1.8 Floor Connection: Ledger</td>
<td>31</td>
</tr>
<tr>
<td>1.9 Floor Connection: Direct Bearing</td>
<td>35</td>
</tr>
<tr>
<td>1.10 Floor Connection: Pocket</td>
<td>36</td>
</tr>
<tr>
<td>1.11 References</td>
<td>38</td>
</tr>
<tr>
<td>CHAPTER 2 - ICF DESIGN EXAMPLE</td>
<td>41</td>
</tr>
<tr>
<td>2.1 Problem Statement</td>
<td>41</td>
</tr>
<tr>
<td>2.2 Determine Nominal And Factored Loads</td>
<td>46</td>
</tr>
<tr>
<td>2.3 Design Second-Story Wall</td>
<td>51</td>
</tr>
<tr>
<td>2.4 Design First-Story Wall</td>
<td>57</td>
</tr>
<tr>
<td>2.5 Design Foundation Wall</td>
<td>64</td>
</tr>
<tr>
<td>2.6 Design Second-Story Lintel</td>
<td>69</td>
</tr>
<tr>
<td>2.7 Design First-Story Lintel</td>
<td>72</td>
</tr>
<tr>
<td>2.8 Design Footing Connection</td>
<td>75</td>
</tr>
<tr>
<td>2.9 Design Roof Connection: Bolted Sill Plate</td>
<td>77</td>
</tr>
<tr>
<td>2.10 Design Roof Connection: Strap</td>
<td>82</td>
</tr>
<tr>
<td>2.11 Design Floor Connection: Ledger</td>
<td>84</td>
</tr>
<tr>
<td>2.12 Design Floor Connection: Pocket</td>
<td>91</td>
</tr>
</tbody>
</table>
# Table of Contents

**APPENDIX A - BEAM DIAGRAMS WITH TYPICAL LOADING CONDITIONS** ........................................... 93

**APPENDIX B - PROPERTIES OF GEOMETRIC SECTIONS** ............................................................... 97

**APPENDIX C - MOMENT MAGNIFIERS** ......................................................................................... 101

**APPENDIX D - INTERACTION DIAGRAMS FOR STRUCTURAL REINFORCED CONCRETE WALLS** .......................................................................................................................... 131

**APPENDIX E - INTERACTION DIAGRAMS FOR STRUCTURAL PLAIN CONCRETE WALLS** ........... 165

**APPENDIX F - CONVERSION FACTORS** ....................................................................................... 183

**APPENDIX G - WEIGHTS OF COMMON BUILDING MATERIALS** ..................................................... 187

**APPENDIX H - STANDARD REINFORCING BAR DATA** ............................................................... 191

**APPENDIX I - SYMBOLS** ........................................................................................................... 195

**INDEX** ......................................................................................................................................... 201
List of Figures

<table>
<thead>
<tr>
<th>Figure</th>
<th>Description</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>1-1</td>
<td>ICF Wall System Classification</td>
<td>2</td>
</tr>
<tr>
<td>1-2</td>
<td>ICF Wall System Types</td>
<td>3</td>
</tr>
<tr>
<td>1-3</td>
<td>Design Variables Defined For Perpendicular Shear Calculations For Structural Reinforced Concrete Walls</td>
<td>5</td>
</tr>
<tr>
<td>1-4</td>
<td>Design Variables Defined For Parallel (In-Plane) Shear Calculations For Structural Reinforced Concrete Walls</td>
<td>12</td>
</tr>
<tr>
<td>1-5</td>
<td>Design Variables Defined For Lintel Bending</td>
<td>16</td>
</tr>
<tr>
<td>1-6</td>
<td>Design Variables Defined For Lintel Shear</td>
<td>18</td>
</tr>
<tr>
<td>1-7</td>
<td>Loaded Area Of Footing For Bearing Strength</td>
<td>19</td>
</tr>
<tr>
<td>1-8</td>
<td>Design Variables Defined For Dowels In Wall-Footing Interface</td>
<td>20</td>
</tr>
<tr>
<td>1-9</td>
<td>Footing With Key</td>
<td>23</td>
</tr>
<tr>
<td>1-10</td>
<td>Forces On Bolt For Bolted Sill Plate Roof Connection</td>
<td>25</td>
</tr>
<tr>
<td>1-11</td>
<td>Assumed Cone Shear Failure Surface For Bolted Sill Plate Roof Connection</td>
<td>26</td>
</tr>
<tr>
<td>1-12</td>
<td>Design Variables Defined For Bolted Sill Plate Bearing On Icf Wall</td>
<td>27</td>
</tr>
<tr>
<td>1-13</td>
<td>Strap Connection</td>
<td>28</td>
</tr>
<tr>
<td>1-14</td>
<td>Assumed Cone Shear Failure Surface For Strap Connection</td>
<td>29</td>
</tr>
<tr>
<td>1-15</td>
<td>Roof Bearing Directly On Icf Wall</td>
<td>30</td>
</tr>
<tr>
<td>1-16</td>
<td>Side-Bearing Ledger Connection</td>
<td>31</td>
</tr>
<tr>
<td>1-17</td>
<td>Assumed Shear-Friction Failure Surface For Ledger Connection</td>
<td>32</td>
</tr>
<tr>
<td>1-18</td>
<td>Assumed Cone Shear Failure Surface For Ledger Connection</td>
<td>33</td>
</tr>
<tr>
<td>1-19</td>
<td>Direct Bearing Floor Connection</td>
<td>35</td>
</tr>
<tr>
<td>1-20</td>
<td>Pocket Connection</td>
<td>36</td>
</tr>
<tr>
<td>1-21</td>
<td>Variables Defined For Bearing Strength For Pocket Connection</td>
<td>37</td>
</tr>
<tr>
<td>2-1</td>
<td>Design Loads</td>
<td>41</td>
</tr>
<tr>
<td>2-2</td>
<td>Exterior Elevations</td>
<td>43</td>
</tr>
<tr>
<td>2-3</td>
<td>Floor Plans</td>
<td>44</td>
</tr>
<tr>
<td>2-4</td>
<td>Floor Plans And Building Section</td>
<td>45</td>
</tr>
<tr>
<td>2-5</td>
<td>South Wall Section And Distribution Of Gravity Loads</td>
<td>47</td>
</tr>
<tr>
<td>2-6</td>
<td>Parallel Shear Loads Due To Wind</td>
<td>48</td>
</tr>
<tr>
<td>2-7</td>
<td>Nominal Load Summary</td>
<td>49</td>
</tr>
<tr>
<td>2-8</td>
<td>Factored Load Summary</td>
<td>50</td>
</tr>
<tr>
<td>2-9</td>
<td>6-Inch (152 mm) Waffle-Grid ICF Structural Plain Concrete Interaction Diagram</td>
<td>52</td>
</tr>
<tr>
<td>2-10</td>
<td>6-Inch (152 mm) Waffle-Grid ICF Structural Reinforced Concrete Interaction Diagram</td>
<td>55</td>
</tr>
<tr>
<td>2-11</td>
<td>6-Inch (152 mm) Waffle-Grid ICF Structural Plain Concrete Interaction Diagram</td>
<td>59</td>
</tr>
<tr>
<td>2-12</td>
<td>6-Inch (152 mm) Waffle-Grid ICF Structural Reinforced Concrete Interaction Diagram</td>
<td>61</td>
</tr>
<tr>
<td>2-13</td>
<td>8-Inch (203 mm) Waffle-Grid ICF Structural Reinforced Concrete Interaction Diagram</td>
<td>67</td>
</tr>
</tbody>
</table>
List of Figures

Figure 2-14 Lintel Above Master Bedroom Window .........................................................69
Figure 2-15 Lintel Above Family Room Door .................................................................72
Figure 2-16 Footing ........................................................................................................75
Figure 2-17 Roof Diaphragm Shear ..............................................................................78
Figure 2-18 Wind Suction Pressure On Ledger Board Connection ...............................85
Figure 2-19 Ledger Board Bolt Placement .....................................................................90
Figure A-1 Uniform Load, Simple Span ..........................................................................94
Figure A-2 Eccentric Point Loads, Simple Span ..............................................................94
Figure A-3 Partial Triangular Load, Simple Span .............................................................95
Figure A-4 Load Uniformly Increasing To Center, Simple Span ......................................95
Figure A-5 Uniform Load, Fixed-End Simple Span ..........................................................96
Figure B-1 Rectangle, Axis Of Moments Through Center ...............................................98
Figure B-2 Rectangle, Axis Of Moments On Base ............................................................98
Figure B-3 Circle, Axis Of Moments Through Center .....................................................98
Figure B-4 Ellipse, Axis Of Moments Through Center ...................................................99
Figure B-5 Rounded Rectangle, Axis Of Moments Through Center ...............................99
Figure C-1 Dimensions Used For Moment Magnifier Tables ........................................101
Figure D-1 Interaction Diagram For Structural Reinforced Concrete Walls ................132
Figure D-2 Dimensions Used For Interaction Diagrams For Structural Reinforced Concrete Walls ..............................................................132
Figure E-1 Dimensions Used For Interaction Diagrams For Structural Plain Concrete Walls .................................................................................166
Figure E-2 Variables Defined For Interaction Diagrams For Structural Plain Walls ..........167
Introduction

Structural Design of Insulating Concrete Form Walls in Residential Construction was developed as a guideline for the design of single- and multi-unit residential structures using insulating concrete form (ICF) wall systems. The objective of this design guide is to employ the technology efficiently by assisting designers, code officials, and others with limited exposure to concrete design. It provides a step-by-step method to design homes using insulating concrete form wall systems and demonstrates the design procedure in a comprehensive design example. Design aids in the form of graphs, charts, and tables are provided to assist designers.

All ICF systems are typically categorized with respect to the form of the ICF unit itself and the resulting form of the concrete wall once it has cured. There are three types of ICF forms: (1) panel, (2) plank, and (3) block. The differences among the ICF form types are their size and attachment requirements. The different form types exist mainly for ease of installation based on use, available resources, and builder preferences and do not necessarily affect the structural capacity of the wall.

There are also three categories of ICF systems based on the resulting form of the concrete wall. From a structural design standpoint, it is only the shape of the concrete inside the form, not the type of the ICF form, that is of importance. The shape of the concrete wall may be better understood by visualizing the form stripped away from the concrete, thereby exposing it to view. The three categories of ICF wall types are (1) flat, (2) grid, and (3) post-and-beam. The grid wall type is further categorized into (2a) waffle-grid and (2b) screen-grid wall systems. Refer to Figure 1-1 for a classification of currently available ICF manufacturers’ wall systems and Figure 1-2 for graphical representations of the ICF wall system types based on the definitions below. These definitions are provided solely to ensure that the design procedure in this document is applied to the ICF wall systems as the authors intended.

A flat ICF wall system is a solid concrete wall of uniform thickness.

The waffle-grid ICF wall system is a concrete wall composed of closely spaced vertical (maximum 12 inches (305 mm) on center) and horizontal (maximum 16 inches (406 mm) on center) concrete members with concrete webs (approximately 2 inches (51 mm) thick) in between the members. The

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1In some publications and manufacturers’ literature, the waffle-grid may be referred to as an uninterrupted-grid and the screen-grid may be referred to as an interrupted-grid or post-and-beam system.
thicker vertical and horizontal concrete members and the thinner concrete webs create the appearance of a breakfast waffle made of concrete “batter”.

The screen-grid ICF wall system is similar to a waffle-grid ICF wall system without concrete webs in between the vertical and horizontal members. The thicker vertical and horizontal concrete members and the voids in between create the appearance of a window screen made of thick concrete “wire”. For the design procedures described herein, screen-grid ICF wall systems have horizontal and vertical cores spaced a maximum of 12 inches (305 mm) on center. There are some screen-grid ICF wall systems with cores spaced farther than 12 inches (305 mm) on center that may be analyzed in a similar manner; however, this document does not address these systems.

The post-and-beam ICF wall system has vertical and/or horizontal concrete members spaced farther than 12 inches (305 mm) on center; therefore, the post-and-beam ICF wall system resembles a concrete frame rather than a monolithic concrete construction (i.e. flat or grid wall). Given that post-and-beam ICF wall systems require a different engineering analysis than flat and grid systems per ACI 318-95, the design method for post-and-beam systems is not included in this design guide.

| Manufacturer | ICF Wall Systems | | | | Manufacturer | ICF Wall Systems | | | |
|--------------|-----------------|-----------------|-----------------|-------------------------------|-----------------|-----------------|-----------------|-------------------------------|-----------------|-----------------|-----------------|-------------------------------|-----------------|-----------------|
|              | Flat            | Waffle-Grid     | Screen-Grid     | Post-and-Beam               | Flat            | Waffle-Grid     | Screen-Grid     | Post-and-Beam               | Flat            | Waffle-Grid     | Screen-Grid     | Post-and-Beam               | Flat            | Waffle-Grid     | Screen-Grid     | Post-and-Beam               |
| A-10 Insulated Forms | | | | | AAS Building Systems | BlueMaxx | | | | | | | | | | | | |
| Aircraft Insulated Building Systems | | | | | American Insulated Building Systems | Snap-Form | | | | | | | | | | | | |
| American Insulated Building Systems | | | | | American Insulated Building Systems | SmartBlock Waffle | | | | | | | | | | | | |
| American Polystyrene Forms | | | | | American Polystyrene Forms | Polystyrene | | | | | | | | | | | | |
| Anikoma USA, Inc | | | | | ConstuWall | ConstuWall | | | | | | | | | | | | |
| ConstuWall | | | | | | | | | | | | | | | | | | |
| Energy Lock, Inc | | | | | Ener-Grid | | | | | | | | | | | | |
| Featherlite, Inc | | | | | Energy Lock | | | | | | | | | | | | |
| FoamFill Systems, LLC | Foam Form | | | | GreenForm Worldwide Corp | GreenBlock | | | | | | | | | | | | |
| Ice Block Building Systems | ICE Block | | | | K-Block Building Systems | K-Block | | | | | | | | | | | | |
| K & B Associates, Inc | | | | | | | | | | | | | | | | | | |
| K & B Associates, Inc | | | | | | | | | | | | | | | | | | |
| Keene, Inc | | | | | | | | | | | | | | | | | | |

*Due to the rapid growth in the ICF industry, ICF manufacturers may exist that do not appear in Figure I-1. There is no intention to preclude any ICF manufacturer. ICF manufacturers that do not appear in Figure I-1 may be classified according to the given ICF system definitions.

*There are some post-and-beam ICF wall manufacturers that can vary the spacing of the vertical and horizontal concrete members rather easily. When the spacing of these systems is altered to coincide with the definition of the screen-grid ICF wall system, these systems may be designed in accordance with the screen-grid ICF wall system design provisions herein.
Much of the design procedure described herein is based on the American Concrete Institute's *Building Code Requirements for Structural Concrete (ACI 318-95)*. References made to requirements from ACI 318-95 are presented by stating the document and section in a compatible format (e.g. ACI 10.12.1).
Figure 1-2  ICF Wall System Types
Chapter 1

ICF Design Procedure

1.1 APPLICATION AND LIMITATIONS

The design procedure presented in this publication addresses the design of flat, waffle-grid, and screen-grid ICF wall systems used specifically in single- and multi-unit residential construction. The differences among the ICF wall types affect their structural capacity and the methods with which they are structurally analyzed.

Prior to implementing the recommended design procedures described herein, care must be taken to accurately define and analyze the ICF wall system according to the definitions and categories described in the introduction. These definitions are provided solely to ensure that the design procedure in this document is applied to the ICF wall systems as the authors intended. Refer to Figure I-1 for a classification of currently available ICF manufacturers’ wall systems based on the definitions in the introduction.

This publication assumes that the user is familiar with load analysis on residential structures, strength-based design procedures, the ACI 318 code, and basic engineering mechanics. This publication also assumes that a user in high seismic regions is very familiar with any special detailing requirements in high seismic regions; therefore, the simplified design procedure presented in this publication does not include any special detailing required for concrete construction in high seismic regions. The simplified design procedure presented in this publication is not intended to substitute in any way for the recommendations of any ICF manufacturer or accepted engineering practice in general. The manufacturer’s recommendations and accepted engineering practices always take precedence over any material presented herein.
1.2 STRUCTURAL REINFORCED CONCRETE WALLS

The design of structural reinforced concrete walls is governed by ACI 318 Chapter 14, “Walls”. ICF wall geometry and loading conditions often do not satisfy the limitations of the empirical design method specified in ACI 14.5; therefore, the design procedure below provides a more flexible approach by which walls are designed as compression members in accordance with ACI 14.4. Although not discussed in detail here, walls may be designed in accordance with ACI 14.5 using the empirical design method if the following limiting conditions are satisfied:

- The wall cross-section is solid.
- The resultant of all axial loads acts within the middle one-third of the wall thickness.
- The wall thickness is not less than 4 inches (102 mm) for above-grade walls or 7.5 inches (191 mm) for basement walls.
- The wall thickness is not less than 1/25 of the supporting wall height or length of the wall, whichever is smaller.

The required minimum horizontal and vertical reinforcement ratio specified in ACI 14.3 for structural reinforced concrete walls has been relaxed in the following design approach based on test data under the provision of ACI 14.2.7, which states

“Quantity of reinforcement and limits of thickness required by 14.3 and 14.5 shall be permitted to be waived where structural analysis shows adequate strength and stability.”

In addition, there is evidence to show that the minimum wall thickness requirements, particularly for basement walls, may be conservative for many residential design conditions. The One- and Two-Family Dwelling Code permits basement walls of 5.5 inch (140 mm) thickness when the height of unbalanced fill is less than a prescribed maximum. Analysis will confirm this practice depending on the lateral soil loads present.

1.2.1 Select Trial Wall Section and Properties

Select an ICF wall system type (flat, waffle-grid, or screen-grid), a trial wall thickness for each story, and a trial vertical reinforcement and spacing. In addition, select a trial compressive strength for the concrete and a yield strength for the steel reinforcement. The selection of a particular ICF wall system type may be an issue of personal preference, cost, availability, and other concerns.

1.2.2 Determine Nominal and Factored Loads

Determine the loads acting on all structural concrete walls in accordance with the applicable provisions of the locally approved building code and recognized principles of engineering

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mechanics. Determine the critical factored axial load and moment for each applicable ACI load case listed in ACI 9.2.

For above-grade walls, applicable ACI load cases are generally

(1) 1.4 Dead + 1.7 Live
(2) 0.75 (1.4 Dead + 1.7 Live + 1.7 Wind)
(3) 0.9 Dead + 1.3 Wind
(4) 0.75 (1.4 Dead + 1.7 Live + 1.87 Seismic)
(5) 0.9 Dead + 1.43 Seismic

For below-grade walls, applicable ACI load cases are generally

(1) 1.4 Dead + 1.7 Live
(2) 0.75 (1.4 Dead + 1.7 Live + 1.7 Earth)
(3) 0.9 Dead + 1.7 Earth

ACI Load Case (1) rarely governs design, and ACI Load Cases (4) and (5) rarely govern design unless the structure is situated in regions of high seismic risk. To simplify calculations further, each wall story may be conservatively assumed to act as a simple span with each end pinned. Appendix A contains basic load diagrams and equations to assist in calculating typical loading conditions encountered in residential design. Refer to Chapter 2, “ICF Design Example” for examples on how to calculate loads.

![Design Variables Defined for Perpendicular Shear Calculations for Structural Reinforced Concrete Walls](image)

**Figure 1-1 Design Variables Defined for Perpendicular Shear Calculations for Structural Reinforced Concrete Walls**

1.2.3 **Check Perpendicular Shear**

The following equations are taken from ACI 11.10 to check perpendicular wall shear. Even though unreinforced vertical cores and webs are often neglected when calculating perpendicular shear, perpendicular shear rarely governs in residential concrete wall design. Dimensions are often simplified for waffle- and screen-grid wall systems that have complex cross-sectional geometries.
Refer to Figure 1-1 for the design variables used in determining perpendicular shear for various ICF wall types. Although shear reinforcement is permitted in ACI 11.5, the use of stirrups in thin ICF walls is difficult to install and should be avoided. If greater shear capacity is required, increasing the thickness of the wall, increasing the compressive strength of the concrete, or using vertical reinforcement to resist shear forces by the shear-friction method (ACI 11.7) is suggested in lieu of using stirrups.

\[
V_u \leq \phi V_n \\
V_n = V_c + V_s \\
V_c = 2\sqrt{f_c'b_wd} \\
V_s = \frac{A_v f_y d}{s} \leq 8\sqrt{f_c'b_wd} \text{ when } V_u > \phi V_c \\
A_v = \frac{(V_u - \phi V_c)s}{f_y d} \geq \frac{50b_w s}{f_y} \text{ for vertical stirrups} \\
s \leq \text{ minimum of } \left\{ \frac{d/2}{24''} \right\} \text{ when } V_s < 4\sqrt{f_c'b_wd} \\
s \leq \text{ minimum of } \left\{ \frac{d/4}{12''} \right\}
\]

**SHEAR-FRICTION METHOD**

\[
V_u \leq \phi V_n \\
V_n = A_v f_y f_y \leq 0.2 f_c' A_c \text{ and } \leq 800 A_c \\
A_c = b_w h
\]

where:

- \( \lambda \): Correction factor related to unit weight of concrete = 1.0 for normal weight concrete per ACI 11.7.4
- \( \mu \): Coefficient of friction per ACI 11.7.4
  - Concrete placed monolithically ........................................... 1.4\( \lambda \)
  - Concrete placed against hardened concrete with surface intentionally roughened \( \frac{3}{8} \text{ inch} (6.4 \text{ mm}) \) ........................................... 1.0\( \lambda \)
  - Concrete placed against hardened concrete not intentionally roughened ........................................... 0.6\( \lambda \)
  - Concrete anchored to as-rolled structural steel by headed studs or by reinforcing bars ........................................... 0.7\( \lambda \)
- \( \phi \): Strength reduction factor = 0.85 per ACI 9.3.2
- \( A_c \): Area of concrete section resisting shear transfer \( \text{ inch}^2 \)
- \( A_v \): Area of shear vertical reinforcement within distance, \( s \) \( \text{ inch}^2 \)
- \( A_{vr} \): Area of shear-transfer vertical reinforcement \( \text{ inch}^2 \)
- \( b_w \): Web width, Refer to Figure 1-1 \( \text{ inch} \)
- \( d \): Distance from extreme compression fiber to centroid of longitudinal tension reinforcement, Refer to Figure 1-1 \( \text{ inch} \)
- \( f_c' \): Specified compressive strength of concrete \( \psi \)
- \( f_y \): Specified yield strength of shear reinforcement \( \leq 60,000 \text{ psi} \) per ACI 11.5.2 and 11.7.6 \( \psi \)
- \( h \): Concrete wall thickness, Refer to Figure 1-1 \( \text{ inch} \)
- \( s \): Spacing of shear reinforcement per ACI 11.5.4 \( \text{ inch} \)
ICF Design Procedure: Structural Reinforced Concrete Walls

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<thead>
<tr>
<th>Variable</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>$V_C$</td>
<td>Nominal shear strength of concrete per ACI 11.3.1.1 lb</td>
</tr>
<tr>
<td>$V_n$</td>
<td>Nominal shear strength per ACI 11.1.1 or ACI 11.7.4 lb</td>
</tr>
<tr>
<td>$V_u$</td>
<td>Factored shear force at section lb</td>
</tr>
<tr>
<td>$V_s$</td>
<td>Nominal shear strength of steel reinforcement per ACI 11.5.6, assume $V_s = 0$ when $V_u &lt; \phi V_C$ lb</td>
</tr>
</tbody>
</table>

for ease of ICF construction

**Figure 1-2** Design Variables Defined for Parallel (In-Plane) Shear Calculations for Structural Reinforced Concrete Walls

### 1.2.4 Check Parallel (In-Plane) Shear

The following equations are taken from ACI 11.10 to check parallel wall shear. All vertical cores, both reinforced and unreinforced, are often assumed to resist parallel wall shear. Dimensions are often simplified for waffle- and screen-grid wall systems that have complex cross-sectional geometries. Design variables for determining parallel shear for various ICF wall types are illustrated in Figure 1-2. The level of parallel shear encountered in residential concrete construction typically does not require the use of shear reinforcement unless the wall is constructed with a large number of openings or is in an area with large lateral loads from wind or seismic forces. If shear reinforcement is required, the use of vertical and horizontal steel reinforcement may be used to increase the shear capacity of the wall.

\[
\begin{align*}
V_u & \leq \phi V_n \\
V_n &= V_c + V_s \\
V_s &= \frac{A_y f_y d}{s_2} \quad \text{when} \quad V_u > \phi V_c \\
d &= 0.81 l_w \\
V_c &= 2 \sqrt{f_c' h d}
\end{align*}
\]

where:

- $\phi$: Strength reduction factor = 0.85 per ACI 9.3.2 dimensionless
- $A_y$: Area of horizontal shear reinforcement within a distance, $s_2$, and distance, $d$ per ACI 11.10 inch²
- $f_c'$: Specified compressive strength of concrete psi
- $f_y$: Specified yield strength of shear reinforcement psi
- $h$: Concrete wall thickness, Refer to Figure 1-2 inch
1.2.5 Sway Determination

Determine whether the wall is part of a non-sway or sway frame by comparing the total lateral stiffness of the compression member to that of the bracing elements. A compression member may be assumed braced if it is located in a story in which bracing elements provide resistance against large lateral deflections so as not to affect the column strength substantially. Most homes built with flat, waffle-grid, or screen-grid ICF wall systems on all four sides can be categorized as non-sway frames provided reasonable limits on the amount of wall openings are met.

1.2.6 Determine Slenderness

ACI 10.10.2 allows an approximation method for walls with a slenderness ratio, $k_i \frac{h}{r}$, of 100 or less to account for slenderness effects in a wall. Walls with a slenderness ratio, $k_i \frac{h}{r}$, greater than 100 require a second-order analysis in accordance with ACI 10.10.1. The approximation method described in detail here is referred to as the moment magnifier method and is ideal for typical residential-scale construction with slenderness ratios less than 100.

1.2.6.1 Non-Sway Frames (ACI 10.12)

If the following condition is satisfied, the designer may ignore slenderness and go to Section 1.2.7.

$$\frac{k_i h}{r} < 34 - 12 \frac{M_1}{M_2}$$

where:

- $k$: Effective length factor ≤ 1.0; for most residential construction, $k = 1.0$ if the wall is tied to the footing, floors, and roof
- $l_u$: Unsupported length of compression member
- $M_1/M_2$: Ratio of smaller factored end moment to larger factored end moment ≥ -0.5
- $M_1$: Smaller factored end moment, very often assumed to be 0
- $M_2$: Larger factored end moment
- $r$: Radius of gyration of cross-section per ACI 10.11.2, Refer to Appendix B

1.2.6.2 Sway Frames (ACI 10.13)

For residential construction, the following condition is rarely satisfied; therefore, slenderness must be accounted for in a sway frame analysis using the magnified moment method described in Section 1.2.7. If the following condition is satisfied, the designer may ignore slenderness and go to Section 1.2.7.
1.2.7 Determine Magnified Moment

1.2.7.1 Non-Sway Frames

Tables are provided in Appendix C to determine the moment magnifier for flat, waffle-grid, and screen-grid walls. Unreinforced vertical cores and webs are often not assumed to resist moments and axial loads experienced by the wall. To use the moment magnifier tables in Appendix C, calculate the following variables:

\[
\begin{align*}
\beta_d &= \frac{M_2}{P_u} \\
M_{2,min} &= P_u(0.6 + 0.03h) \\
\rho &= \beta_d = \frac{P_{u,dead}}{P_u} \\
\rho &= \frac{A_s}{A_g}
\end{align*}
\]

where:

- \( h \): Concrete wall thickness; Refer to Figure 1-2, inch
- \( k \): Effective length factor \( \geq 1.0 \) per ACI 10.13.1, dimensionless
- \( l_u \): Unsupported length of compression member, inch
- \( r \): Radius of gyration of cross-section per ACI 10.11.2, Refer to Appendix B, inch

\[ \approx 0.3h \text{ for rectangular members or } \approx 0.25d \text{ for circular compression members} \]

Using the calculated values for \( e, P_u, \) and \( \beta_d \) and the initial assumed value for \( \rho \) of 0.0012, select the value of the moment magnifier, \( \delta_{M2} \), from the moment magnifier tables in Appendix C for the given wall height, ICF wall system type, wall thickness, and concrete compressive strength. For wall types that do not meet the minimum dimensions on which the Appendix C tables are based, calculate the moment magnifier using the equations in Appendix C, “Non-Sway Frames”.

Determine the magnified moment using the following equation:
ICF Design Procedure: Structural Reinforced Concrete Walls

\[
M_{ns} = \left\{ \begin{array}{ll}
\delta_{ns} M_2 & \\
\delta_{ns} M_{2,min} & 
\end{array} \right. \text{ whichever is greater}
\]

where:
- \( \delta_{ns} \) Moment magnifier from Appendix C, dimensionless
- \( M_2 \) Larger factored end moment, in-lb
- \( M_{2,min} \) Minimum value of \( M_2 \), in-lb
- \( M_{ns} \) Magnified factored moment of a non-sway frame, in-lb

1.2.7.2 Sway Frames

Moment magnifier tables for sway frames do not appear in this document. Determine the magnified moment, \( \delta_{ns} \), using the equations in Appendix C, “Sway Frames”.

1.2.8 Determine Reinforcement

To determine if the wall section is adequately reinforced, plot the magnified moment and the corresponding total factored axial load from Section 1.2.7 on an interaction diagram. Partial interaction diagrams can be found in Appendix D for most residential applications. The reinforcement plot line that lies below and to the right of the plotted point is the minimum vertical reinforcement required for the given wall section. If the plotted point lies directly on a reinforcement plot line, select that line for the minimum vertical reinforcement. Refer to Appendix D for more information on interaction diagrams and the equations used to construct complete interaction diagrams.

Tests have shown that horizontal and vertical reinforcement spacing limited to 8 times the wall thickness or 48 inches (1.2 m) results in good performance;\(^5\) therefore, it is suggested that the designer limit the vertical and horizontal reinforcement spacing to 8 times the wall thickness, not to exceed 48 inches (1.2 m).

Per ACI 14.3.7, the designer is required to provide additional reinforcement around all window and door openings to distribute loads; however, the requirement for two \#5 bars around openings may be excessive for residential loading and a smaller amount of reinforcement may be used around openings when justified by structural analysis. Refer to Section 2.4.7.2 for an example on how to determine reinforcement required around wall openings by structural analysis.

1.2.9 Check Deflection

ACI 318 does not limit wall deflection specifically; however, since many interior and exterior finishes applied to an ICF wall are susceptible to damage by large wall deflections, a conservative deflection limit of \( L/360 \) for live service loads and \( L/240 \) for total service loads is suggested for above-grade walls. For below-grade walls, a conservative deflection limit of \( L/240 \) for live service loads is suggested since earth loads are immediate and are not expected to change with time. These

deflection limits are conservative suggestions; deflection limits should be specified by the designer based on the finishes being used. When using the moment magnifier concept, it is recommended that the calculated moment magnification factor be applied to the service load moments used in conducting the deflection calculations.

To calculate wall deflection at service load levels, effective section properties of the assumed cracked concrete section must be established. According to test results,\(^6\) calculating deflection using \(0.1E_{ci}k\) was found to be conservative but more accurate than calculating deflection based on the cracking moment per ACI 9.5.2.3.

If service load deflections are found to be unacceptable, the designer may either increase the wall thickness or increase the quantity of vertical reinforcement. For most ICF wall configurations and residential loading conditions, however, satisfying service load deflection limits should not be a limiting condition.

1.3 STRUCTURAL PLAIN CONCRETE WALLS

Structural plain concrete walls are concrete walls that either have no reinforcement or less than the minimum amount specified by ACI for reinforced concrete. Structural plain concrete walls are designed according to the provisions of ACI 318 Chapter 22, which limits the use of plain concrete walls to members provided with continuous vertical support throughout the member’s length. ACI 22.3 further requires contraction and isolation joints to provide flexural discontinuity and to control cracking. ICF walls may be exempt from the required contraction joints because random cracking due to creep, shrinkage, and temperature is assumed to have a negligible impact on the structural integrity of the wall. Some nominal level of reinforcement to control crack width may be included at the designer’s discretion.

Limitations on the use of structural plain concrete are listed in ACI 22.6.6 and include the following:

- The horizontal length of wall to be considered effective for each vertical concentrated load is not greater than the distance between loads, or width of bearing plus 4 times the wall thickness.
- The wall thickness is not less than 1/25 of the supporting wall height or length of the wall, whichever is smaller.
- The wall thickness is not less than 5.5 inches (140 mm) for above-grade walls or 7.5 inches (190 mm) for basement walls.
- Walls are braced against lateral translation.
- Window and door openings require not less than two #5 bars around the openings extending not less than 24 inches (610 mm) beyond the corners of the openings.

There is evidence to show that the requirement for two #5 bars around openings may be excessive for residential loading. The Standard Building Code and National Building Code have clauses modifying this requirement to one #4 bar provided that vertical bars span continuously from support to support and horizontal bars extend a minimum of 24 inches (610 mm) beyond the opening. Likewise, there is evidence to show that minimum wall thickness requirements, particularly for basements, may be conservative for many residential conditions. The One- and Two-Family Dwelling Code permits unreinforced basement walls of 5.5-inch (140 mm) thickness when the height of unbalanced fill is less than a prescribed maximum. Analysis will confirm this practice depending on the lateral soil loads present.

ICF wall geometry and loading conditions often do not satisfy the limitations of the empirical design method specified in ACI 22.6.5; therefore, the design procedure below provides a more flexible approach by which walls are designed as compression members in accordance with ACI 22.5. Although not discussed in detail here, walls may be designed in accordance with ACI 22.6.5 using the empirical design method if the following additional limiting conditions are satisfied:

- The wall cross-section is solid.
• The resultant of all axial loads acts within the middle one-third of the wall thickness.

1.3.1 Select Trial Wall Section and Properties

Select an ICF wall system type (flat, waffle-grid, or screen-grid), a trial wall thickness for each wall story, and a trial concrete compressive strength. The selection of a particular ICF wall system type may be an issue of personal preference, cost, availability, or other concerns.

1.3.2 Determine Nominal and Factored Loads

Determine the loads acting on all structural concrete walls in accordance with the applicable provisions of the locally approved building code and recognized principles of engineering mechanics. Determine the critical factored axial load and moment for each applicable ACI load case listed in ACI 9.2.

For above-grade walls, applicable ACI load cases are generally

(1) 1.4 Dead + 1.7 Live
(2) 0.75 (1.4 Dead + 1.7 Live + 1.7 Wind)
(3) 0.9 Dead + 1.3 Wind
(4) 0.75 (1.4 Dead + 1.7 Live + 1.87 Seismic)
(5) 0.9 Dead + 1.43 Seismic

For below-grade walls, applicable ACI load cases are generally

(1) 1.4 Dead + 1.7 Live
(2) 0.75 (1.4 Dead + 1.7 Live + 1.7 Earth)
(3) 0.9 Dead + 1.7 Earth

ACI Load Case (1) rarely governs design, and ACI Load Cases (4) and (5) rarely govern design unless the structure is situated in regions of high seismic risk. To simplify calculations further, each wall story may be conservatively assumed to act as a simple span with pinned ends. Appendix A contains basic load diagrams and equations to assist in calculating typical loading conditions encountered in residential design. Refer to Chapter 2, “ICF Design Example”, for examples on how to calculate loads.

1.3.3 Check Perpendicular Shear

The equations below are taken from ACI 22.5.4 to check perpendicular wall shear. Greater shear capacity may be obtained by increasing the thickness of the wall, increasing the compressive strength of the concrete, or adding shear reinforcement. Dimensions are often simplified for waffle- and screen-grid wall systems that have complex cross-sectional geometries. Refer to Figure 1-3 for the design variables used in determining perpendicular shear for various ICF wall types. Refer to Section 1.2, “Structural Reinforced Concrete Walls”, if shear reinforcement is required.
$V_u \leq \phi V_n$

$V_n = \frac{4}{3} \sqrt{f_c' bh}$

where:

- $\phi$ Strength reduction factor = 0.65 per ACI 9.3.5 
- $b$ Width of concrete member, Refer to Figure 1-3 
- $h$ Concrete wall thickness, Refer to Figure 1-3 
- $f_c'$ Specified compressive strength of concrete 
- $V_n$ Nominal shear strength at section per ACI 22.5.4 for normal weight concrete 
- $V_u$ Factored shear force at section

**Figure 1-3** Design Variables Defined for Perpendicular Shear Calculations for Structural Plain Concrete Walls

**Figure 1-4** Design Variables Defined for Parallel (In-Plane) Shear Calculations for Structural Plain Concrete Walls
1.3.4 Check Parallel (In-Plane) Shear

The equations below are taken from ACI 22.5.4 to check parallel wall shear. Design variables for determining parallel shear for various ICF wall types are illustrated in Figure 1-4. The level of parallel shear encountered in residential concrete construction typically does not require the use of shear reinforcement unless the wall is constructed with a large number of openings or is in an area with large lateral loads from wind or seismic forces. Greater shear capacity may be obtained by increasing the thickness of the wall, increasing the compressive strength of the concrete, reducing the number of openings in the walls, or adding shear reinforcement. Refer to Section 1.2, “Structural Reinforced Concrete Walls”, if shear reinforcement is required.

\[ V_u \leq \phi V_n \]
\[ V_n = \frac{4}{3} \sqrt{f_c' bh} \]

where:

<table>
<thead>
<tr>
<th>Symbol</th>
<th>Description</th>
<th>Unit</th>
</tr>
</thead>
<tbody>
<tr>
<td>( \phi )</td>
<td>Strength reduction factor</td>
<td>dimensionless</td>
</tr>
<tr>
<td>( b )</td>
<td>Thickness of concrete member</td>
<td>inch</td>
</tr>
<tr>
<td>( h )</td>
<td>Width of concrete member</td>
<td>inch</td>
</tr>
<tr>
<td>( f_c' )</td>
<td>Specified compressive strength of concrete</td>
<td>psi</td>
</tr>
<tr>
<td>( V_n )</td>
<td>Nominal shear strength at section per ACI 22.5.4 for normal weight concrete</td>
<td>lb</td>
</tr>
<tr>
<td>( V_u )</td>
<td>Factored shear force at section</td>
<td>lb</td>
</tr>
</tbody>
</table>

1.3.5 Check Compression and Tension

To determine if the wall section is capable of resisting the design loads, plot the factored moment and the corresponding total factored axial load from Section 1.3.2 on an interaction diagram for structural plain concrete. The minimum factored moment required for design by ACI 22.6.3 is

\[ M_{u,min} = 0.1hP_u \]

where:

<table>
<thead>
<tr>
<th>Symbol</th>
<th>Description</th>
<th>Unit</th>
</tr>
</thead>
<tbody>
<tr>
<td>( h )</td>
<td>Concrete wall thickness, Refer to Figure 1-3</td>
<td>inch</td>
</tr>
<tr>
<td>( P_u )</td>
<td>Factored total axial load</td>
<td>lb</td>
</tr>
<tr>
<td>( M_{u,min} )</td>
<td>Minimum factored bending moment</td>
<td>in-lb</td>
</tr>
</tbody>
</table>

Interaction diagrams can be found in Appendix E for most residential applications. If the plotted point lies within the lower tension boundary, the upper compression boundary for the given wall height, and the reference axes, the wall section is adequate. Refer to Appendix E for more information on interaction diagrams and the equations used to construct them.

If the wall section is not adequate, repeat Sections 1.3.1 through 1.3.5 with an increased wall thickness or increased concrete compressive strength. Alternatively, design the wall in accordance with Section 1.2, “Structural Reinforced Concrete Walls”, using steel reinforcement to obtain the strength required.
1.3.6 Check Deflection

ACI 318 does not limit wall deflection specifically; however, since many interior and exterior finishes applied to an ICF wall are susceptible to damage by large wall deflections, a conservative deflection limit of \( L/360 \) for live service loads and \( L/240 \) for total service loads is suggested for above-grade walls. For below-grade walls, a conservative deflection limit of \( L/240 \) for service live loads is suggested since earth loads are immediate and are not expected to change with time. These deflection limits are conservative suggestions; deflection limits should be specified by the designer based on the finishes being used.

To calculate wall deflection at service load levels, effective section properties of the assumed uncracked concrete section are based on \( E_cI_g \).

If service load deflections are found to be unacceptable, the designer may either increase the wall thickness or add vertical reinforcement. For most ICF wall configurations and residential loading conditions, however, satisfying service load deflection limits should not be a limiting condition.

1.3.7 Determine Reinforcement

Although the wall is designed as a structural plain concrete wall, a nominal amount of reinforcement is typically specified. Tests have shown that horizontal reinforcement spacing limited to 8 times the wall thickness or 48 inches (1.2 m) results in good performance; therefore, it is suggested that the designer limit the horizontal reinforcement spacing to 8 times the wall thickness, not to exceed 48 inches (1.2 m).

Per ACI 22.6.6.5, the designer is required to provide two #5 bars around all window and door opening; however, this may be excessive for residential loading. The Standard Building Code and National Building Code have clauses modifying this requirement to one #4 bar provided that vertical bars span continuously from support to support and horizontal bars extend a minimum of 24 inches (610 mm) beyond the opening. In addition, one continuous #4 bar at the top of the wall is suggested. Lintels and narrower sections of wall near openings may require reinforcement in accordance with Section 1.2, “Structural Reinforced Concrete Walls”.

---

1.4 LINTELS

Lintels are concrete beams typically placed above doors and windows to support the floor, roof, and/or wall above. Lintel design is governed by the provisions of ACI Chapter 10, “Flexure and Axial Loads”, and Chapter 11, “Shear and Torsion”.

1.4.1 Select Trial Lintel Section and Properties

Select a lintel depth, thickness, area of steel reinforcement for shear and bending, and a reinforcement yield strength.

1.4.2 Determine Nominal and Factored Loads

Determine loads acting on lintels in accordance with the applicable provisions of the locally approved building code and recognized principles of engineering mechanics. Determine the critical factored load and moment for each applicable ACI load case listed in ACI 9.2. For lintels, the applicable ACI load case is generally

1.4 Dead + 1.7 Live

Each lintel is conservatively assumed to act as a simple span with each end pinned to simplify calculations. To some degree, the lintel may behave like a fixed-end beam; however, if such a model is assumed, the lintel should also be reinforced near the top. If the lintel is assumed to act as a fixed-end beam, sufficient embedment of the top and bottom reinforcement beyond each side of the opening should be provided to fully develop a moment resisting end in the lintel. Appendix A contains basic load diagrams and equations to assist in calculating typical structural loads. Refer to Chapter 2, “ICF Design Example”, for examples on how to calculate loads.

1.4.3 Check Deflection

Windows and doors are susceptible to damage by large lintel deflections; therefore, a conservative deflection limit of \( L/480 \) for service dead loads and sustained live loads is suggested. This limit is very conservative when the installation of the window and door components is properly detailed to allow for significant lintel deflection; deflection limits should be specified by the designer for given applications.

To calculate lintel deflection at service load levels, effective section properties of the assumed cracked concrete section must be established. According to test results,\(^8\) deflection calculated using \( 0.1E_{cr}L_g \) was found to be conservative but more accurate than deflection based on the cracking moment per ACI 9.5.2.3.

---

If service load deflections are found to be unacceptable, the designer may either increase the lintel depth, increase the quantity of reinforcement, or use a modified lintel.

### 1.4.4 Check Nominal Moment Strength

ICF lintels are designed for bending using the equations below in accordance with ACI Chapter 10, “Flexure and Axial Loads”. The width of the compression face varies for each type of ICF wall system; refer to Figure 1-5 for the appropriate width dimension.

\[
\frac{\phi}{\phi} M_n \leq M_u
\]

\[
\phi M_n = \phi A_s f_y \left( d - \frac{a}{2} \right)
\]

\[
a = \frac{A_s f_y}{0.85 f_c' b}
\]

where:

- $\phi$: Strength reduction factor = 0.9 for flexure per ACI 9.3.2 (dimensionless)
- $a$: Depth of equivalent rectangular stress block (inch)
- $A_s$: Area of tensile reinforcement (inch$^2$)
- $b$: Width of compression face of member, Refer to Figure 1-5 (inch)
- $d$: Distance from extreme compression fiber to centroid of tensile reinforcement, Refer to Figure 1-5 (inch)
- $f_c'$: Specified compressive strength of concrete (psi)
- $f_y$: Specified yield strength of reinforcement (psi)
- $M_n$: Nominal moment strength (in-lb)
- $M_u$: Factored moment at section (in-lb)

If greater bending capacity is required, increasing the lintel depth, the yield strength of the reinforcement, or the area of reinforcement is suggested. The lintel thickness is often limited to the thickness of the wall in which it is placed. The lintel depth is also often limited by the floor-to-floor height and the vertical placement of the opening in the wall based on aesthetic, functional, or economic reasons. For waffle- and screen-grid wall systems, it may be necessary to use special lintel blocks or modified standard blocks to create solid cross-sections for longer spans where the...
Lintel depth cannot be increased. In many cases, increasing the amount of bottom reinforcement is the most economical solution. However, care must be taken to avoid overcrowding of the reinforcement. When possible, one reinforcing bar of the appropriate size should be specified. Using more than two bars in a small area can be detrimental if there is not enough space between the bars or between the bars and the wall for concrete to flow around the bars. ACI 10.3.3 limits the amount of reinforcement to 0.75 of the reinforcement ratio, \( \rho_b \), that would produce balance strain conditions for the section under flexure without axial load.

### 1.4.5 Check Nominal Shear Strength

ICF lintels are designed for shear resulting from wall, roof, and floor loads above using the equations below in accordance with ACI Chapter 11, “Shear and Torsion”. The web width varies for each type of ICF wall system; refer to Figure 1-6 for the appropriate web width dimension.

\[
V_u \leq \phi V_n \\
V_n = V_c + V_s \\
V_c = 2 \sqrt{f_{c'} b_w d} \\
V_s = \frac{A_s f_y d}{s} \leq 8 \sqrt{f_{c'} b_w d} \text{ when } V_u > \phi V_c \\
A_{s, \text{min}} = \frac{50 b_w s}{f_y} \text{ when } V_u > \frac{\phi V_c}{2} \\
s \leq \text{minimum of } \left\{ \frac{d/2}{24''} \right\} \text{ when } V_s > 4 \sqrt{f_{c'} b_w d} \\
\]

where:

- \( \phi \): Strength reduction factor = 0.85 for flexure per ACI 9.3.2, dimensionless
- \( A_v \): Area of shear reinforcement within distance \( s \), inch\(^2\)
- \( A_{s, \text{min}} \): Minimum area of shear reinforcement
- \( b_w \): Web width, Refer to Figure 1-6, inch
- \( d \): Distance from extreme compression fiber to centroid of longitudinal tension reinforcement, inch
- \( f_{c'} \): Specified compressive strength of concrete, psi
- \( f_y \): Specified yield strength of shear reinforcement, psi
- \( s \): Spacing of shear reinforcement per ACI 11.5.4, inch
- \( V_n \): Nominal shear strength, lb
- \( V_c \): Nominal shear strength provided by concrete, lb
- \( V_s \): Nominal shear strength provided by shear reinforcement, \( V_s = 0 \) if no stirrups used, lb
- \( V_u \): Factored shear force, lb

If greater shear capacity is required, increasing the lintel thickness, the lintel depth, the yield strength of the reinforcement, or the area of reinforcement is suggested. Often the lintel thickness is limited to the thickness of the wall in which it is placed. The lintel depth is also often limited by the floor-to-floor height and the vertical placement of the opening in the wall based on aesthetic, structural considerations.
functional, or economic reasons. For waffle- and screen-grid wall systems, it may be necessary to use special lintel blocks or modified standard blocks to create solid cross-sections for longer spans where the lintel depth cannot be increased. Recent unpublished testing\(^9\) has provided data showing that predicted shear capacity provided by the concrete, \(V_c\), is very conservative. Future testing may conclude that stirrups are not required for ICF lintels used for short spans in residential construction.

---

1.5 FOOTING CONNECTIONS

Footing connections transmit axial and shear loads from the wall to the footing below. Wall-to-footing connections for residential construction are constructed in one of the following three ways:

- No vertical reinforcement or key
- Key only
- Dowels only

1.5.1 Check Bearing Strength of Footing

Determine whether the bearing strength of the concrete is adequate per ACI 10.17. If the bearing strength is adequate, skip Sections 1.5.2 and 1.5.3. Bearing strength is typically sufficient for residential construction; however, use dowels if additional bearing strength is required.

\[ B_c \leq \phi 0.85 f'_c A_1 \]

When the supporting surface is wider on all sides than the loaded area, the design bearing strength on the loaded area is permitted to be

\[ B_c \leq \phi 0.85 f'_c A_1 \sqrt{\frac{A_2}{A_1}} \text{ where } \frac{A_2}{A_1} \leq 2 \]

where:

- \( \phi \) Strength reduction factor = 0.7 per ACI 9.3.2  
- \( A_1 \) Loaded area of concrete; Refer to Figure 1-7  
- \( A_2 \) Projected loaded area of concrete; Refer to Figure 1-7  
- \( B_c \) Bearing strength of concrete  
- \( f'_c \) Specified compressive strength of concrete

![Figure 1-7 Loaded Area of Footing for Bearing Strength](image)
1.5.2 Use Dowels to Increase Bearing Capacity

If additional bearing strength is required as determined in Section 1.5.1, dowels are placed across the wall-footing interface. The development length of the dowel into the footing and into the wall is calculated per ACI 10.17. Refer to Figure 1-8 for typical dowel placement.

\[
B \leq \phi (B_c + B_s) \\
B_c = 0.85f_c' A_3 \\
B_s = A_s f_y
\]

Where the supporting surface is wider on all sides than the loaded area, the design bearing strength on the loaded area is permitted to be

\[
B \leq \phi (B_c + B_s) \sqrt{\frac{A_2}{A_1}} \quad \text{where} \quad \sqrt{\frac{A_2}{A_1}} \leq 2 \\
B_c = 0.85f_c' A_1 \\
B_s = A_s f_y
\]

where:

- $\phi$: Strength reduction factor = 0.7 per ACI 9.3.2
- $A_1$: Loaded area of concrete; Refer to Figure 1-7
- $A_2$: Projected loaded area of concrete; Refer to Figure 1-7
- $A_s$: Area of bearing reinforcement
- $B$: Design bearing strength
- $B_c$: Bearing strength of concrete
- $B_s$: Bearing strength of reinforcement
- $f_c'$: Specified compressive strength of concrete
- $f_y$: Specified yield strength of bearing reinforcement

$\phi$, $A_1$, $A_2$, $A_s$, $B_c$, $B_s$, $f_c'$, $f_y$ are dimensionless, inch$^2$, inch', inch', lb, lb, psi, psi.

Figure 1-8 Design Variables Defined for Dowels in Wall-Footing Interface
1.5.3 Determine Development Length of Dowels for Bearing Capacity

If dowels are used to increase the bearing capacity of the concrete, use the following equations, taken from ACI 12.3, to determine the minimum development length required:

\[
I_{db} = \frac{0.02d_b f_y}{\sqrt{f_c'}} \geq 0.0003d_b f_y
\]

\[
I_d = I_{db} \left( \frac{A_s,req'd}{A_s,provided} \right) \geq 8
\]

where:
- \( A_s \) Area of bearing reinforcement, inch²
- \( d_b \) Diameter of bearing reinforcement bar, inch
- \( f_c' \) Specified compressive strength of concrete, psi
- \( f_y \) Specified yield strength of bearing reinforcement, psi
- \( I_d \) Development length of bearing reinforcement, Refer to Figure 1-8, inch
- \( I_{db} \) Basic development length of bearing reinforcement, inch

1.5.4 Check Shear Transfer

Shear forces existing at the base of the wall may require a key or dowels to transfer the shear forces from the wall to the footing. The following equations are taken from ACI 11.7, “Shear-Friction Method”, to develop shear resistance by using vertical reinforcement (dowels) across the wall-footing interface. If a key is preferred to transfer shear forces from the wall to the footing instead of dowels, skip this Section and go to Section 1.5.6.

\[
V_u \leq \phi V_n
\]

\[
V_n = A_{of} f_y \mu \leq 0.2 f_c' A_c \text{ and } \leq 800 A_c
\]

\[
A_{of} = \frac{V_u}{\phi f_y \mu}
\]

where:
- \( \lambda \) Correction factor related to unit weight on concrete =1.0 for normal weight concrete per ACI 11.7.4, dimensionless
- \( \mu \) Coefficient of friction per ACI 11.7.4:
  - Concrete placed monolithically: \( \mu = 1.4 \lambda \)
  - Concrete placed against hardened concrete with surface intentionally roughened \( \frac{1}{4} \) inch (6.4 mm): \( \mu = 1.0 \lambda \)
  - Concrete placed against hardened concrete not intentionally roughened: \( \mu = 0.6 \lambda \)
  - Concrete anchored to as-rolled structural steel by headed studs or by reinforcing bars: \( \mu = 0.7 \lambda \)
- \( \phi \) Strength reduction factor = 0.85 for shear per ACI 9.3.2, dimensionless
- \( A_c \) Area of concrete section resisting shear transfer, inch²
- \( A_{of} \) Area of shear-transfer reinforcement, inch²
- \( f_c' \) Specified compressive strength of concrete, psi
- \( f_y \) Specified yield strength of reinforcement \( \leq 60,000 \) psi
- \( V_n \) Nominal shear strength per ACI 11.7.4, lb
- \( V_u \) Factored shear force at section, lb
1.5.5 Determine Development Length of Dowels for Shear Transfer

If dowels are used to transfer shear forces from the base of the wall to the footing, use the following equations to determine the minimum development length required. Refer to Figure 1-8 for typical dowel placement.

If dowels are required for both bearing and shear force transfer, the size and development length of the dowels only need to be adequate for the more severe of the horizontal or vertical load transfer conditions.

Standard Hooks (ACI 12.5)

\[ l_{hb} = \frac{1200d_b}{\sqrt{f_c}} \text{ with } f_y = 60,000 \text{ psi} \]

\[ l_{dh} = \text{maximum of} \begin{cases} l_{hb} & \xi \geq 8d_b \\ \geq 6" & \end{cases} \]

\[ \xi = \frac{f_y}{60,000} \]

\[ \omega = \frac{A_{s,req'd}}{A_{s, provided}} \]

Deformed Bars (ACI 12.2)

\[ l_{db} = \left( \frac{3f_y}{40\sqrt{f_c}} \right) \left( \frac{\alpha \beta \gamma \lambda}{c + K_{TR}} \right) \]

\[ 2.5 \geq \left( \frac{c + K_{TR}}{d_b} \right) \]

\[ l_d = l_{db} \left( \frac{A_{s,req'd}}{A_{s, provided}} \right) \geq 12" \]

where:

- \( \alpha \) Reinforcement location factor per ACI 12.2.4
  - Horizontal reinforcement placed so that more than 12 inches (305 mm) of fresh concrete is cast in the member below the development length ......................... 1.3
  - Other reinforcement ........................................... 1.0

- \( \beta \) Coating factor = 1.0 for uncoated reinforcement, per ACI 12.2.4

- \( \gamma \) Reinforcement size factor per ACI 12.2.4
  - Bars No. 6 and smaller ...................................... 0.8
  - Bars No. 7 and larger ..................................... 1.0

- \( \lambda \) Concrete type factor = 1.0 for normal weight concrete per ACI 12.2.4

- \( \omega \) Excess reinforcement factor

- \( \xi \) Reinforcement yield strength factor

- \( \psi \) Concrete side cover factor = 0.7 per ACI 12.5.3
1.5.6 Use Key to Provide Adequate Shear Transfer

A key may be used in lieu of dowels. In residential construction, a key is often formed using a 2x4 wood board with chamfered edges. Refer to Figure 1-9 for a footing with a key; although a flat wall is depicted in Figure 1-9, a key may also be used with a waffle-grid or screen-grid ICF wall system. Shear resistance developed by the key is computed using the following equations:

\[
V_u \leq \phi V_n \\
V_n = \frac{4}{3} f'c b h
\]

where:

- \( \phi \) Strength reduction factor =0.85 for shear per ACI 9.3.2
- \( b \) Shear width of section, Refer to Figure 1-9
- \( f'c \) Specified compressive strength of concrete
- \( h \) Shear height of section, Refer to Figure 1-9
- \( V_n \) Nominal shear strength at section per ACI 22.5.4
- \( V_u \) Factored shear force at section

---

**Figure 1-9 Footing with Key**
1.6 ROOF CONNECTION: BOLTED SILL PLATE

Roofs for residential-scale construction may be constructed of a variety of materials; however, the design procedure described herein assumes a wood-framed roof structure bearing on the ICF walls below. Where appropriate, the following design procedure should be altered by the designer for roof structures other than wood-framed.

1.6.1 Determine Design Loads

Determine axial loads, shear loads, wind loads, and uplift loads in accordance with the applicable provisions of the locally approved building code and recognized principles of engineering mechanics. Refer to Chapter 2, “ICF Design Example”, for examples on how to calculate loads.

1.6.2 Assume Connection Spacing and Size

Select a trial bolt diameter and spacing and sill plate size, grade, and species. Refer to “Part 4 Connections” in the American Institute of Steel Construction’s (AISC) Manual of Steel Construction for engineering data on bolts and the American Forest and Paper Association’s (AF&PA) Design Values for Wood Construction for wood data.

1.6.3 Check Shear in Bolt

Determine whether the bolt diameter and spacing is adequate to resist shear loads calculated in Section 1.6.1 using the equations below. If greater shear capacity in the bolt is required, increase the bolt diameter or reduce the bolt spacing. In residential construction, a 5/8-inch (16 mm) diameter anchor bolt is typically used for sill plate connections to concrete.

\[
\begin{align*}
  f_v & \leq F_v \\
  f_v &= \frac{V}{A_b} \\
  A_b &= \frac{\pi d^2}{4}
\end{align*}
\]

where:

- \( A_b \) Area of bolt, \( \text{inch}^2 \)
- \( d \) Threaded shank diameter of the bolt, \( \text{inch} \)
- \( f_v \) Allowable shear stress of bolt, Refer to AISC’s Manual of Steel Construction, \( \text{psi} \)
- \( V \) Shear force on bolt, Refer to Figure 1-10, \( \text{lb} \)

1.6.4 Check Tension in Bolt Due to Uplift and Shear-Friction

Determine whether the bolt diameter and spacing is adequate to resist tensile loads calculated in Section 1.6.1 using the equations below. If greater tensile capacity in the bolt is required, increase the bolt diameter or reduce the bolt spacing.
where:

- $f_t \leq F_t$
- $f_t = \frac{T}{A_b}$
- $T = T_{\text{uplift}} + \frac{V}{\mu}$
- $A_b = \frac{\pi d^2}{4}$

$\mu$  Coefficient of friction, assume $\mu = 0.6$  dimensionless

$A_b$  Area of bolt  inch$^2$

d  Diameter of bolt  inch

$f_t$  Actual tensile stress of bolt due to uplift and shear-friction  psi

$F_t$  Allowable tensile stress of bolt, Refer to AISC’s *Manual of Steel Construction*  psi

$T$  Tensile force on bolt due to uplift and shear-friction  lb

$T_{\text{uplift}}$  Tensile force on bolt due to uplift, calculated in Section 1.6.1  lb

$V$  Shear force on bolt, Refer to Figure 1-10  lb

1.6.5 **Check Tension in Concrete (Anchorage Capacity)**

Use the following equations to determine whether the concrete shear area of each bolt is sufficient to resist pull-out from the ICF wall due to uplift forces and shear-friction:

\[
V_u \leq \phi V_c
\]

\[
V_c = 4A_v \sqrt{f_c}
\]

\[
A_v = \text{minimum of } \left\{ \frac{\pi b^2}{\pi h^2} \right\}
\]

where:

- $\phi$  Strength reduction factor = 0.85 for shear per ACI 9.3.2  dimensionless

*Figure 1-10 Forces on Bolt for Bolted Sill Plate Roof Connection*
ICF Design Procedure: Roof Connections

$A_v$ Shear area of concrete right circular cone  
$f_c'$ Specified compressive strength of concrete  
$h$ Concrete wall thickness, Refer to Figure 1-11  
$l_b$ Embedment length of bolt, Refer to Figure 1-11  
$V_c$ Nominal shear strength provided by concrete  
$V_u$ Factored shear force in concrete due to uplift and shear-friction  
(Nominal shear force = tensile force on bolt due to uplift and shear-friction, Refer to Section 1.6.4)

![Figure 1-11 Assumed Cone Shear Failure Surface for Bolted Sill Plate Roof Connection](image)

1.6.6 Check Bending, Bearing, and Shear in Sill Plate

A sill plate is commonly installed on top of concrete walls to provide a fastening surface for a wood-framed roof. Determine if the spacing of bolts is adequate to prevent overstresses in the sill plate due to bending, bearing, and shear. In low wind and seismic conditions in the United States, it is common practice to use 1/2-inch to 7-inch diameter j-bolts spaced at 6 feet on center and within 12 inches of the end of, or a joint in, the sill plate. For sill plates constructed of lumber, refer to AF&PA’s Design Values for Wood Construction for allowable bending, bearing, and shear stresses. Refer to Chapter 2, “ICF Design Example”, for detailed calculations.

1.6.7 Check Bearing Strength of ICF Wall

Determine whether the bearing strength of the concrete is adequate per ACI 10.17. The bearing strength of the concrete is typically adequate for loads encountered in residential construction.

$$B_c \leq \phi \frac{0.85 f_c'}{A_1}$$

When the supporting surface is wider on all sides than the loaded area, the design bearing strength on the loaded area is permitted to be

$$B_c \leq \phi \frac{0.85 f_c'}{A_1} \sqrt{\frac{A_2}{A_1}} \text{ where } \sqrt{\frac{A_2}{A_1}} \leq 2$$
where:

<table>
<thead>
<tr>
<th>Symbol</th>
<th>Description</th>
<th>Unit</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\phi$</td>
<td>Strength reduction factor</td>
<td>dimensionless</td>
</tr>
<tr>
<td>$A_I$</td>
<td>Loaded area of concrete; Refer to Figure 1-12</td>
<td>inch$^2$</td>
</tr>
<tr>
<td>$A_2$</td>
<td>Projected loaded area of concrete; Refer to ACI 10.17</td>
<td>inch$^2$</td>
</tr>
<tr>
<td>$B_c$</td>
<td>Bearing strength of concrete</td>
<td>lb</td>
</tr>
<tr>
<td>$f_c'$</td>
<td>Specified compressive strength of concrete</td>
<td>psi</td>
</tr>
</tbody>
</table>

**Figure 1-12** Design Variables Defined for Bolted Sill Plate Bearing on ICF Wall
1.7 ROOF CONNECTION: STRAP

Roofs for residential-scale construction may be constructed of a variety of materials; however, the design procedure described herein assumes a wood-framed roof structure bearing directly on the ICF walls below. Where appropriate, the following design procedure should be altered by the designer for roof structures other than wood-framed.

A strap connection consists of roof trusses or rafters that bear directly on the ICF wall and are held in place with a tie-down strap or bracket. This type of construction is more common in areas subject to hurricane force winds. In lower wind and seismic regions of the United States, it is common practice to toe nail the rafter or truss to a wood sill plate as shown in Figure 1-12. Refer to Figure 1-13 for a typical strap connection detail.

1.7.1 Determine Design Loads

Determine axial loads, shear loads, and wind uplift in accordance with the applicable provisions of the locally approved building code and recognized principles of engineering mechanics. Refer to Chapter 2, “ICF Design Example”, for examples on how to calculate loads.

1.7.2 Assume Strap Connector Size

Each roof truss or rafter should be anchored to the concrete wall to resist uplift and lateral forces from wind loads. One end of the strap connector or bracket is embedded in the concrete and the other end is fastened by nails or bolts to the roof rafter or truss. Select from the manufacturer’s catalog, a trial strap connector that has sufficient capacity to resist the tensile loads calculated in Section 1.7.1.
1.7.3 Check Tension in Concrete (Anchorage Capacity)

In some cases, the manufacturer’s strap data is based on specific embedment and concrete wall thickness requirements. If these conditions are met, the calculation of tension in the concrete may not be necessary as it is inherent to the rated capacity of the connector.

If the manufacturer’s strap data does not specify embedment or wall thickness requirements, use the following equations to determine whether the concrete shear area of each strap is sufficient to resist pull-out from the ICF wall due to uplift forces:

\[ V_u \leq \phi V_c \]
\[ V_c = 4A_v \sqrt{f'_c} \]
\[ A_v = \text{minimum of} \left\{ \frac{\pi l_b^2}{\pi h^2} \right\} \]

where:
- \( \phi \) = Strength reduction factor = 0.85 for shear per ACI 9.3.2
- \( A_v \) = Shear area of concrete right circular cone
- \( f'_c \) = Specified compressive strength of concrete
- \( h \) = Concrete wall thickness, Refer to Figure 1-14
- \( l_b \) = Embedment length of strap, Refer to Figure 1-14
- \( V_c \) = Nominal shear strength provided by concrete
- \( V_u \) = Factored shear force in concrete due to uplift

Figure 1-14 Assumed Cone Shear Failure Surface for Strap Connection

1.7.4 Check Bearing Strength of ICF Wall

Determine whether the bearing strength of the concrete is adequate per ACI 10.17. The bearing strength of the concrete is typically adequate for residential construction.

\[ B_c \leq \phi 0.85 f'_c A_1 \]
When the supporting surface is wider on all sides than the loaded area, the design bearing strength on the loaded area is permitted to be

\[ B_c \leq \phi 0.85 f'_c \cdot A_1 \sqrt{\frac{A_2}{A_1}} \text{ where } \sqrt{\frac{A_2}{A_1}} \leq 2 \]

where:

- \( \phi \) Strength reduction factor = 0.7 per ACI 9.3.2
- \( A_1 \) Loaded area of concrete; Refer to Figure 1-15
- \( A_2 \) Projected loaded area of concrete; Refer to ACI 10.17
- \( B_c \) Bearing strength of concrete
- \( f'_c \) Specified compressive strength of concrete

\( f'_c \) is in psi, \( A_1 \) and \( A_2 \) are in inch\(^2\), and \( B_c \) is in lb.

Figure 1-15 Roof Bearing Directly on ICF Wall
1.8 FLOOR CONNECTION: LEDGER

Floors for residential-scale construction may be constructed of a variety of materials; however, the design procedure described herein assumes a wood-framed floor structure. Where appropriate, the following design procedures should be altered by the designer for floor structures other than wood-framed.

The ledger connection is very common in ICF construction. It consists of a 2x- or 3x-wood ledger board bolted to the side of an ICF wall with j-bolts. The floor joists are hung from the ledger board with joist hangers. Refer to Figure 1-16 for a typical side-bearing ledger connection.

The designer should recognize the importance of the ledger connection because the floor live and dead loads are transferred to the wall through a series of connections which is not typical of platform wood-framed construction. In high wind or seismic regions, straps may be needed. A strap is not shown in Figure 1-16; however, if a strap is required, one end of the strap connector is embedded in the concrete and the other end is fastened to the bottom edge of the floor joist by nails or bolts. The use of a strap may not be desired because it may interfere with the ceiling finish attachment. Perimeter edge nailing of the subflooring to the ledger may provide sufficient lateral restraint of the wall in lieu of using a strap.

1.8.1 Determine Design Loads

Determine axial loads, shear loads, and bending moments at the floor-wall connection in accordance with the applicable provisions of the locally approved building code and recognized principles of engineering mechanics. Refer to Chapter 2, “ICF Design Example”, for examples on how to calculate loads.
1.8.2 Assume Connection Spacing and Size

Select a trial bolt diameter, bolt spacing, nominal ledger board size, grade, and species. Refer to “Part 8: Bolts” in AF&PA’s National Design Specifications for Wood Construction for engineering data on bolts and AF&PA’s Design Values for Wood Construction for wood data.

1.8.3 Check Shear-Friction in Concrete

Shear forces from the floor joist must be transferred to the concrete wall. The following equations are taken from ACI 11.7, “Shear-Friction Method”, to develop shear resistance by using j-bolts across the concrete wall-ledger interface:

\[ V_u \leq \phi V_n \]
\[ V_n = A_{cf} f_y \mu \leq 0.2 f_c' A_c \quad \text{and} \quad \leq 800 A_c \]
\[ A_{cf} = \frac{V_n}{\phi f_y \mu} \]
\[ A_c = \pi \left( \frac{d}{2} \right)^2 \]

where:

- \( \lambda \): Correction factor related to unit weight on concrete, \( \lambda = 1.0 \) for normal weight concrete per ACI 11.7.4
- \( \mu \): Coefficient of friction per ACI 11.7.4
  - Concrete placed monolithically .................................................. 1.4\( \lambda \)
  - Concrete placed against hardened concrete with surface intentionally roughened \( \frac{1}{4} \) inch (6.4 mm) ........................................ 1.0\( \lambda \)
  - Concrete placed against hardened concrete not intentionally roughened ........................................... 0.6\( \lambda \)
  - Concrete anchored to as-rolled structural steel by headed studs or by reinforcing bars ........................................... 0.7\( \lambda \)
- \( \phi \): Strength reduction factor, =0.85 for shear per ACI 9.3.2
- \( A_c \): Area of concrete section resisting shear transfer

\( f_c' \): Compressive strength of concrete
\( f_y \): Yield strength of reinforcement
ICF Design Procedure: Floor Connections

### 1.8.4 Check Tension in Concrete (Anchorage Capacity)

Use the following equations to determine whether the concrete shear area of each bolt is sufficient to resist pull-out from the ICF wall due to wind suction pressure on the wall, lateral (out-of-plane) seismic forces on the wall, and prying action from eccentric floor loads:

\[
V_u \leq \phi V_c
\]

\[
V_c = 4 A_v \sqrt{f_c'}
\]

\[
A_v = \text{minimum of } \left\{ \frac{\pi (2l_{be})^2}{\pi l_b^2} \right\}
\]

where:

- \( \phi \) = Strength reduction factor = 0.85 for shear per ACI 9.3.2, dimensionless
- \( A_v \) = Shear area of concrete right circular cone, inch²
- \( f_c' \) = Specified compressive strength of concrete, psi
- \( l_b \) = Embedment length of bolt, Refer to Figure 1-18, inch
- \( l_{be} \) = Distance from bolt to nearest edge of concrete ledge, Refer to Figure 1-18, inch
- \( V_c \) = Nominal tensile strength provided by concrete, lb
- \( V_u \) = Tensile force on bolt, lb

![Figure 1-18 Assumed Cone Shear Failure Surface for Ledger Connection](image)

Structural Design of Insulating Concrete Form Walls in Residential Construction
1.8.5 Check Tension in Bolt

Determine whether the bolt diameter and spacing is adequate to resist tensile loads due to wind suction pressure on the wall, lateral (out-of-plane) seismic forces on the wall, and prying action from eccentric floor loads using the equations below. If greater tensile capacity in the bolt is required, increase the bolt diameter or reduce the bolt spacing.

\[ f_t \leq F_t \]
\[ f_t = \frac{T}{A_b} \]
\[ A_b = \frac{\pi d^2}{4} \]

where:

- \( A_b \): Area of bolt (inch\(^2\))
- \( d \): Diameter of bolt (inch)
- \( f_t \): Actual tensile stress of bolt (psi)
- \( F_t \): Allowable tensile stress of bolt (psi)
- \( T \): Tensile force on bolt (lb)

1.8.6 Check Shear in Bolt

Determine whether the bolt diameter and spacing is adequate to resist shear loads calculated in Section 1.8.1 using the equations below. If greater shear capacity in the bolt is required, increase the bolt diameter or reduce the bolt spacing.

\[ f_v \leq F_v \]
\[ f_v = \frac{V}{A_b} \]
\[ A_b \approx \frac{\pi d^2}{4} \]

where:

- \( A_b \): Area of bolt (inch\(^2\))
- \( d \): Diameter of bolt (inch)
- \( f_v \): Actual shear stress of bolt (psi)
- \( F_v \): Allowable shear stress of bolt, Refer to AF&PA’s *National Design Specifications for Wood Design* (psi)
- \( V \): Shear force on bolt (lb)

1.8.7 Check Bending, Bearing, and Shear in Ledger Board

Determine if the spacing of bolts is adequate to prevent overstresses in the ledger board due to bending, bearing, and shear. For ledger boards constructed of lumber, refer to AF&PA’s *Design Values for Wood Construction* and “Part 8: Bolts” in AF&PA’s *National Design Specifications for Wood Construction*. Refer to Chapter 2, “ICF Design Example”, for detailed calculations.
1.9 FLOOR CONNECTION: DIRECT BEARING

Floors for residential-scale construction may be constructed of a variety of materials; however, the design procedure described herein assumes a wood-framed floor structure. Where appropriate, the following design procedures should be altered by the designer for floor structures other than wood-framed.

The direct bearing connection consists of a wood sill plate bolted to the top of an ICF wall; floor joists bear on, and are fastened to, the sill plate. This floor connection is usually used for the ground floor on a below-grade ICF wall that has wood-framed construction above or for the first floor on a first-story ICF wall that has a wood-framed second story constructed above. Refer to Figure 1-19 for a direct bearing connection detail. The design procedure is similar to roof connection design and is not repeated here; refer to Section 1.6, “Roof Connections: Bolted Sill Plate”.

![Figure 1-19 Direct Bearing Floor Connection](image-url)
1.10 FLOOR CONNECTION: POCKET

Floors for residential-scale construction may be constructed of a variety of materials; however, the design procedure described herein assumes a wood-framed floor structure. Where appropriate, the following design procedures should be altered by the designer for floor structures other than wood-framed.

The pocket connection consists of a void or “pocket” in the ICF wall formed by placing a wood block in the wall cavity before pouring the concrete. Pockets for waffle- and screen-grid wall systems must be placed at the centerline of a vertical core. Once the concrete has set, the wood block is removed and the floor joist is placed in the pocket. The floor joist may be required to be fire cut by the locally approved building code to allow the floor joist to fall free from the wall in the event of a fire. Some manufacturers of ICFs provide special metal brackets in lieu of using wood blocks to create “pockets” in the ICF wall. Alternatively, the floor joists can be embedded in the forms before the concrete is placed, wrapped in building paper, and cast into the concrete. Refer to Figure 1-20 for a pocket connection detail. Care must be taken to insure that the use of a pocket does not adversely affect the strength of the wall since the pocket connections may interfere with the continuity of the vertical reinforcement in the wall. The minimum pocket depth is typically 3 to 4 inches (76 to 102 mm) to allow for adequate bearing for the floor joist. Where adequate bearing cannot be achieved due to wall thickness; a pocket connection cannot be used and the designer should refer to Section 1.9 for a ledger board connection.

1.10.1 Determine Design Loads

Determine axial loads, shear loads, and bending moments in accordance with the applicable provisions of the locally approved building code and recognized principles of engineering mechanics. Refer to Chapter 2, “ICF Design Example”, for examples on how to calculate loads.

![Figure 1-20 Pocket Connection](image)
1.10.2 Assume Strap Connector Size

Each floor joist should be anchored to the concrete wall to prevent pull-out due to lateral wall movement from the forces expected in high wind or high seismic regions. One end of the strap connector is embedded in the concrete and the other end is fastened by nails or bolts to the floor joist. Select from the manufacturer’s catalog, a trial strap connector that has sufficient capacity to resist the loads calculated in Section 1.10.1.

Refer to a wood connector manufacturer’s catalog for engineering data on strap connectors, appropriate number and spacing of fasteners, and recommended embedment lengths. Also, refer to the strap connector manufacturer’s test data and minimum concrete compressive strength requirements.

1.10.3 Check Bearing Strength of ICF Wall

Determine whether the bearing strength of the concrete is adequate per ACI 10.17.

\[ B_c \leq \phi 0.85 f_{c'} A_1 \]

where:
- \( \phi \) Strength reduction factor = 0.7 per ACI 9.3.2
- \( A_1 \) Loaded area of concrete; Refer to Figure 1-21
- \( B_c \) Bearing strength of concrete
- \( f_{c'} \) Specified compressive strength of concrete

1.10.4 Check Bearing and Shear in Floor Joist

For floor joists constructed of lumber, refer to AF&PA’s Design Values for Wood Construction for allowable bending, bearing, and shear stresses. Refer to Chapter 2, “ICF Design Example”, for detailed calculations.

![Diagram of ICF Wall Connections](image)

* Figure 1-21 Variables Defined for Bearing Strength for Pocket Connection
1.11 REFERENCES

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*Building Code Requirements for Structural Concrete and Commentary, ACI Standard 318-95*, American Concrete Institute, Farmington Hills, Mich., 1995. (Available from PCA as LT125.)

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Ghosh, S.K., David A. Fanella and Basile G. Rabbat.

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Roller, John.

Southern Building Code Congress International.

Structural Design of Insulating Concrete Form Walls in Residential Construction
Chapter 2

ICF DESIGN EXAMPLE

2.1 PROBLEM STATEMENT

Figure 2-1 contains the pre-determined design loads for a hypothetical geographic area where the house illustrated in Figures 2-2 through 2-4 is situated. Figures 2-2 through 2-4 depict a two-story house with a basement for which we will conduct engineering calculations in accordance with the design procedures described in Chapter 1, "ICF Design Procedure".

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<th>Given Loading Conditions</th>
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<td><strong>Dead Loads:</strong></td>
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<td>Ground Floor</td>
</tr>
<tr>
<td>First Floor</td>
</tr>
<tr>
<td>Roof &amp; Ceiling</td>
</tr>
<tr>
<td>6&quot; ICF Waffle-Grid Wall</td>
</tr>
<tr>
<td>8&quot; ICF Waffle-Grid Wall</td>
</tr>
<tr>
<td><strong>Live Loads:</strong></td>
</tr>
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</tr>
<tr>
<td>First Floor</td>
</tr>
<tr>
<td>Roof</td>
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<td>Attic</td>
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<tr>
<td><strong>Equivalent fluid density:</strong></td>
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<tr>
<td><strong>Wind Load:</strong></td>
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<tr>
<td>+/- 21.0 psf (1.01 kPa)</td>
</tr>
<tr>
<td><strong>Wind Uplift Load:</strong></td>
</tr>
<tr>
<td>19.0 psf (910. Pa)</td>
</tr>
<tr>
<td><strong>Seismic Zone:</strong></td>
</tr>
<tr>
<td>1.0</td>
</tr>
</tbody>
</table>

*Figure 2-1 Design Loads*
Although the following design example assumes the house is constructed with a waffle-grid ICF wall system, the design example may be followed for a flat ICF wall system by simply designing the walls on a per lineal foot basis instead of per vertical core and substituting the dimensions of the flat ICF wall for the waffle-grid ICF wall. Refer to the figures in Chapter 1, “ICF Design Procedure”, for the correct dimensions to use when making substitutions. The design example may also be followed for an ICF wall system which supports a light-framed wall system above by calculating the axial load experienced by the ICF wall below based on the weight of the light-framed materials above. Refer to Appendix G for the weights of typical light-framed walls.

The house is situated in a low-risk seismic area; therefore, the following design example does not show seismic calculations since the design seismic loads are substantially lower than the design wind loads induced on the structure. However, in higher seismic zones, special consideration of seismic forces and detailing will likely be necessary especially since the dead load of ICF walls is much higher than light-framed walls.

There are some building codes and standards that calculate wind pressure based on the fastest mile wind speed and some that calculate wind pressure based on a 3-second gust. Note that only the resulting hypothetical wind pressure is given in Figure 2-1 without reference to wind speed or exposure rating to simplify load calculations. In addition, the given wind pressure states that the inward acting wind pressure is equivalent to the outward acting wind pressure to further simplify calculations. The designer should be aware that inward and outward acting wind pressures are rarely equivalent on a building structure and should be calculated in accordance with the locally approved building code and recognized principles of engineering mechanics for a given wind speed and exposure rating.
ICF Design Example: Problem Statement

Figure 2-2 Exterior Elevations

Structural Design of Insulating Concrete Form Walls in Residential Construction
Figure 2-3  Floor Plans
2.2 DETERMINE NOMINAL AND FACTORED LOADS

The loads calculated below are based on a 6-inch (152 mm) waffle-grid trial wall section for above-grade walls and an 8-inch (203 mm) waffle-grid trial wall section for below-grade walls.

The following nominal axial loads and moments are determined for the south wall at Section A where the loads will be the greatest; assume that the wall has no openings. Figure 2-5 illustrates the distribution of axial loads throughout the south wall section and notes the values of eccentricity at the roof, floors, and walls needed to calculate the moments. The eccentricities calculated in Figure 2-5 are based on the actual thickness of the vertical core; refer to the core wall sections in Figure 2-5 for the correct dimensions. Figure 2-6 illustrates the distribution of parallel shear loads from wind, and Figures 2-7 and 2-8 summarize the nominal and factored loads calculated in this Section.

### Dead Loads

<table>
<thead>
<tr>
<th>Section</th>
<th>Formula</th>
<th>lb/lf (kN/m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Ground Floor</td>
<td>$0.5(18.5 , \text{ft})(100 , \text{psf})$ = 93 plf</td>
<td>[1.4 kN/m]</td>
</tr>
<tr>
<td>First Floor</td>
<td>$0.5(18.5 , \text{ft})(100 , \text{psf})$ = 93 plf</td>
<td>[1.4 kN/m]</td>
</tr>
<tr>
<td>Roof and Ceiling</td>
<td>$0.5(32.7 , \text{ft})(12.0 , \text{psf})$ = 196 plf</td>
<td>[2.9 kN/m]</td>
</tr>
<tr>
<td>Second-Story Wall</td>
<td>$0.5(8.5 , \text{ft})(550 , \text{psf})$ = 234 plf @ mid-height</td>
<td>[3.4 kN/m]</td>
</tr>
<tr>
<td>First-Story Wall</td>
<td>$0.5(9.0 , \text{ft})(550 , \text{psf})$ = 248 plf @ mid-height</td>
<td>[3.6 kN/m]</td>
</tr>
<tr>
<td>Foundation Wall</td>
<td>$0.5(8.5 , \text{ft})(750 , \text{psf})$ = 319 plf @ mid-height</td>
<td>[4.7 kN/m]</td>
</tr>
</tbody>
</table>

### Live Loads

<table>
<thead>
<tr>
<th>Section</th>
<th>Formula</th>
<th>lb/lf (kN/m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Ground Floor</td>
<td>$0.5(18.5 , \text{ft})(400 , \text{psf})$ = 370 plf</td>
<td>[5.4 kN/m]</td>
</tr>
<tr>
<td>First Floor</td>
<td>$0.5(18.5 , \text{ft})(300 , \text{psf})$ = 278 plf</td>
<td>[4.1 kN/m]</td>
</tr>
<tr>
<td>Roof and Ceiling</td>
<td>$0.5(32.7 , \text{ft})(350 , \text{psf})$ = 572 plf</td>
<td>[8.3 kN/m]</td>
</tr>
<tr>
<td>Attic</td>
<td>$0.5(32.7 , \text{ft})(100 , \text{psf})$ = 163 plf</td>
<td>[2.4 kN/m]</td>
</tr>
</tbody>
</table>

### Dead Load Moments

<table>
<thead>
<tr>
<th>Section</th>
<th>Formula</th>
<th>in-lb/lf (N-m/m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Second-Story Wall @ top</td>
<td>$(196 , \text{plf})(0 , \text{in}) = 0 , \text{in-lb/lf}$</td>
<td>[0 N-m/m]</td>
</tr>
<tr>
<td>First-Story Wall @ top</td>
<td>$(196 , \text{plf} + 2(234 , \text{plf}) (0 , \text{in}) + (93 , \text{plf})(-4.6 , \text{in}) = -428 , \text{in-lb/lf}$</td>
<td>[-159 N-m/m]</td>
</tr>
<tr>
<td>Foundation Wall @ top</td>
<td>$(196 , \text{plf} + 2(234 , \text{plf}) + 93 , \text{plf} + 2(248 , \text{plf})(0.9 , \text{in}) + (93 , \text{plf})(-5.5 , \text{in}) = 616 , \text{in-lb/lf}$</td>
<td>[228 N-m/m]</td>
</tr>
</tbody>
</table>

### Live Load Moments

<table>
<thead>
<tr>
<th>Section</th>
<th>Formula</th>
<th>in-lb/lf (N-m/m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Second-Story Wall @ top</td>
<td>$(572 , \text{plf} + 163 , \text{plf})(0 , \text{in}) = 0 , \text{in-lb/lf}$</td>
<td>[0 N-m/m]</td>
</tr>
<tr>
<td>First-Story Wall @ top</td>
<td>$(572 , \text{plf} + 163 , \text{plf})(0 , \text{in}) + (278 , \text{plf})(-4.6 , \text{in}) = -1,279 , \text{in-lb/lf}$</td>
<td>[-474 N-m/m]</td>
</tr>
<tr>
<td>Foundation Wall @ top</td>
<td>$(572 , \text{plf} + 163 , \text{plf} + 278 , \text{plf})(0.9 , \text{in}) + (370 , \text{plf})(-5.5 , \text{in}) = -1,123 , \text{in-lb/lf}$</td>
<td>[-416 N-m/m]</td>
</tr>
</tbody>
</table>
ICF Design Example: Loads

Vertical Core @ 12" o.c. 6" Waffle-Grid

Horizontal Core @ 16" o.c. 6" Waffle-Grid

Figure 2-5 South Wall Section and Distribution of Gravity Loads
Figure 2-6 Parallel Shear Loads Due to Wind
ICF Design Example: Loads

Wind Moments
Refer to Figure A-1 for equations and Figure 2-7 for values.

Earth Moments
Refer to Figure A-3 for equations and Figure 2-7 for values.

Perpendicular Shear

Second-Story Wall
Refer to Figure A-1 for equations and Figure 2-7 for values.

First-Story Wall
Refer to Figure A-1 for equations and Figure 2-7 for values.

Foundation Wall
Refer to Figure A-3 for equations and Figure 2-7 for values.

Parallel Shear: Refer to Figure 2-6 for the parallel shear load distribution. Parallel shear loads are calculated based on a simple model that applies a horizontal projection of positive pressure on the windward wall and roof with no suction pressure applied on the leeward side of the building. This simple and conservative model is used for this design example; however, the designer should calculate parallel shear in accordance with the locally approved building code.

Second-Story Wall: The second-story wall design is based on the north wall because it experiences the highest shear loads per lineal foot resulting from the design wind loads.

\[ V_{\text{parallel}} = F_2 = 0.5(32.7 \text{ ft})(21.0 \text{ psf}) \left( \frac{11 \text{ ft}}{2} + \frac{8.5 \text{ ft}}{2} \right) = 3,348 \text{ lb} \]  

[14.9 kN]

First-Story Wall: The first-story wall design is based on the west wall because it experiences the highest shear loads per lineal foot resulting from the design wind loads.

\[ V_{\text{parallel}} = F_1 + F_2 = 0.5(385 \text{ ft})(21.0 \text{ psf}) \left( \frac{9 \text{ ft}}{2} + \frac{8.5 \text{ ft} + 11 \text{ ft}}{2} \right) = 9,702 \text{ lb} \]  

[43.2 kN]

Foundation Wall: Refer to Figure 2-6 for the parallel shear load distribution. Parallel shear on the foundation wall should be neglected since the walls are restrained by the soil lateral pressure on three or more sides.

\[ V_{\text{parallel}} = F_0 = 0 \]  

[0 kN]

![NOMINAL STRUCTURAL LOAD SUMMARY](image)

*Values listed for wind are magnitudes only and therefore are non-directional.*

**Figure 2-7 Nominal Load Summary**
Figure 2-7 is a summary sheet of the greatest loading conditions described above. To determine the moments at various locations in each wall story, refer to Figure A-2. To determine the axial loads at various locations in each wall story, refer to Figure 2-5. Figure 2-8 is a summary sheet of the ACI factored loads for each wall story. The values listed in Figure 2-8 are determined by substituting the values from Figure 2-7 into the equations listed in the left column of Figure 2-8.

### Second Story Wall

<table>
<thead>
<tr>
<th>ACI 318 Load Cases</th>
<th>Vertical Location within Wall</th>
<th>Dead Loads</th>
<th>Live Loads</th>
<th>Wind Loads</th>
<th>Total Loads</th>
</tr>
</thead>
<tbody>
<tr>
<td>1. $U = 1.4D + 1.7L$</td>
<td>Top</td>
<td>1,055</td>
<td>1,725</td>
<td>2,750</td>
<td>2,500</td>
</tr>
<tr>
<td></td>
<td>Mid</td>
<td>1,407</td>
<td>1,725</td>
<td>2,750</td>
<td>2,500</td>
</tr>
<tr>
<td></td>
<td>Bottom</td>
<td>1,754</td>
<td>1,725</td>
<td>2,750</td>
<td>2,500</td>
</tr>
<tr>
<td>2. $U = 0.75(1.4D + 1.7L) + 1.7W$</td>
<td>Top</td>
<td>795</td>
<td>1,725</td>
<td>2,750</td>
<td>1,525</td>
</tr>
<tr>
<td></td>
<td>Mid</td>
<td>1,055</td>
<td>1,260</td>
<td>2,520</td>
<td>1,525</td>
</tr>
<tr>
<td></td>
<td>Bottom</td>
<td>1,316</td>
<td>1,260</td>
<td>2,520</td>
<td>1,525</td>
</tr>
<tr>
<td>3. $U = 0.9D + 1.3W$</td>
<td>Top</td>
<td>681</td>
<td>1,316</td>
<td>2,632</td>
<td>1,240</td>
</tr>
<tr>
<td></td>
<td>Mid</td>
<td>905</td>
<td>1,316</td>
<td>2,632</td>
<td>1,240</td>
</tr>
<tr>
<td></td>
<td>Bottom</td>
<td>1,128</td>
<td>1,316</td>
<td>2,632</td>
<td>1,240</td>
</tr>
</tbody>
</table>

### Foundation Wall

<table>
<thead>
<tr>
<th>ACI 318 Load Cases</th>
<th>Vertical Location within Wall</th>
<th>Dead Loads</th>
<th>Live Loads</th>
<th>Earth Loads</th>
<th>Total Loads</th>
</tr>
</thead>
<tbody>
<tr>
<td>1. $U = 1.4D + 1.7L$</td>
<td>Top</td>
<td>1,894</td>
<td>2,331</td>
<td>2,331</td>
<td>2,331</td>
</tr>
<tr>
<td></td>
<td>Mid</td>
<td>2,331</td>
<td>2,331</td>
<td>2,331</td>
<td>2,331</td>
</tr>
<tr>
<td></td>
<td>x = 3.43 ft Bottom</td>
<td>2,331</td>
<td>2,331</td>
<td>2,331</td>
<td>2,331</td>
</tr>
<tr>
<td>2. $U = 1.4D + 1.7L + 1.7W$</td>
<td>Top</td>
<td>1,894</td>
<td>2,331</td>
<td>2,331</td>
<td>2,331</td>
</tr>
<tr>
<td></td>
<td>Mid</td>
<td>2,331</td>
<td>2,331</td>
<td>2,331</td>
<td>2,331</td>
</tr>
<tr>
<td></td>
<td>x = 3.43 ft Bottom</td>
<td>2,331</td>
<td>2,331</td>
<td>2,331</td>
<td>2,331</td>
</tr>
<tr>
<td>3. $U = 0.9D + 1.3W$</td>
<td>Top</td>
<td>1,219</td>
<td>1,499</td>
<td>1,499</td>
<td>1,499</td>
</tr>
<tr>
<td></td>
<td>Mid</td>
<td>1,731</td>
<td>1,731</td>
<td>1,731</td>
<td>1,731</td>
</tr>
</tbody>
</table>

**Notes:**
- **Dead Load** (D) is for gravity loads and **Live Load** (L) is for loads due to occupancy.
- **Wind Load** (W) consists of wind forces and pressure.
- **Earth Load** (E) consists of soil and ground loads.
- **Total Load** (T) is the combination of all loads.

![Figure 2-8 Factored Load Summary](image-url)
2.3 DESIGN SECOND-STORY WALL

2.3.1 Select Trial Wall Section and Properties

Try a structural plain concrete 6-inch (152 mm) waffle-grid wall system and assume the concrete compressive strength is 3,000 psi (21 MPa). Refer to Figure 2-5 for the appropriate dimensions of the waffle-grid wall section. The following example follows the design procedure described in Section 1.3 for structural plain concrete walls.

2.3.2 Determine Nominal and Factored Loads

Refer to Figures 2-7 and 2-8 for a summary of nominal and factored loads acting on the second-story wall.

2.3.3 Check Perpendicular Shear

According to Figure 2-8, the critical factored perpendicular shear load, $V_{u}$, experienced by the second-story wall occurs at the bottom of the wall story due to ACI Load Case (3).

$$V_u = 116 \text{ plf} \left( \frac{\text{ft}}{1 \text{ vertical core}} \right) = 116 \text{ lb/ post}$$

$$\phi V_n = 0.65 \left( \frac{4}{3} \right) \sqrt{3,000 \text{ psi}} (6.25 \text{ in})(5 \text{ in}) = 1,483 \text{ lb/ post}$$

$$V_u \leq \phi V_n \quad \text{OK}$$

2.3.4 Check Parallel (In-Plane) Shear

According to Figure 2-8, the critical factored parallel shear load, $V_{u}$, experienced by the second-story wall occurs due to ACI Load Case (3). Recall that we calculated the parallel shear on the north wall since it experiences the highest loads per lineal foot of solid wall. Refer to Figure 2-6 for the north wall elevation. Note that the critical factored parallel shear load is divided by the total length of the north wall minus any openings in the wall since this is the portion of the wall available to resist shear. While this method has the advantage of being simple, the designer should use caution when the solid wall segments become too narrow. The designer should consider neglecting any narrow wall segments when calculating parallel shear resistance, particularly when much longer and stiffer segments exist in the same wall line.

$$V_u = \frac{4,352 \text{ lb}}{(38.5 \text{ ft} - (6 \text{ ft} + 9 \text{ ft} + 6 \text{ ft}))} \left( \frac{\text{ft}}{1 \text{ vertical core}} \right) = 249 \text{ lb/ post}$$

$$\phi V_n = 0.65 \left( \frac{4}{3} \right) \sqrt{3,000 \text{ psi}} (5 \text{ in})(6.25 \text{ in}) = 1,483 \text{ lb/ post}$$

$$V_u \leq \phi V_n \quad \text{OK}$$
2.3.5 Check Compression and Tension

According to Figure 2-8, the critical maximum total moment, $M_u$, experienced by the second-story wall occurs at mid-height due to ACI Load Case (3). The corresponding total factored axial load, $P_u$, is also taken at the mid-height of the second-story wall based on ACI Load Case (3).

$$M_u = 2,959 \text{ in-lb} / \text{ft} \left( \frac{\text{ft}}{1 \text{ vertical core}} \right) = 2,959 \text{ in-lb} / \text{post}$$

$$= 0.25 \text{ ft} - \text{kip} / \text{post} \quad \text{GOVERNS}$$

$$P_u = 387 \text{ plf} \left( \frac{\text{ft}}{1 \text{vertical core}} \right) = 387 \text{ lb} / \text{post} = 0.39 \text{ kip} / \text{post}$$

$$M_{u,\min} = 0.1(5 \text{ in})(387 \text{ plf}) \left( \frac{\text{ft}}{1 \text{ vertical core}} \right) = 194 \text{ in-lb} / \text{post}$$

$$= 0.02 \text{ ft} - \text{kip} / \text{post}$$

Plot $M_u$ and $P_u$ on the interaction diagram for a 6-inch (152 mm) waffle-grid wall. The interaction diagram can be found in Appendix E; however, the interaction diagram is reproduced in Figure 2-9 with the location of the plotted point illustrated. The plotted point lies within the lower tension boundary, the upper compression boundary for an 8.5-foot (2.6 m) wall height, and the reference axes; therefore, a 6-inch (152 mm) waffle-grid structural plain concrete wall is sufficient for the given loading conditions.

Figure 2-9 6-inch (152 mm) Waffle-Grid ICF Structural Plain Concrete Interaction Diagram
2.3.6 Check Deflection

Assume a gypsum board interior finish exposed to view. The deflection calculations below are based on suggestions made in Section 1.3.6 for service live loads taking only wind load into account. Refer to Figure A-1 for the maximum deflection equation.

\[
\Delta_{\text{actual}} = \frac{5(210 \text{ psf})(1 \text{ ft})(8.5 \text{ ft})^4 \left(\frac{1.728 \text{ in}^3}{\text{ft}^3}\right)}{384(3,122,019 \text{ psi}) \left(\frac{6.25 \text{ in}}{5 \text{ in}}\right)^3} = 0.012 \text{ in} \quad [0.3 \text{ mm}]
\]

\[
\Delta_{\text{allowable}} = \frac{12 \text{ in}}{360} = 0.28 \text{ in} \quad [7 \text{ mm}]
\]

\[
\Delta_{\text{actual}} \leq \Delta_{\text{allowable}} \quad OK
\]

2.3.7 Determine Reinforcement

A nominal amount of reinforcement is required in a structural plain concrete wall. Based on the suggestions of Section 1.3.6, the following reinforcement will be installed:

2.3.7.1 Horizontal Reinforcement

\(8(5\text{ in}) = 40 \text{ in} \quad \text{GOVERNS} \quad [1 \text{ m}]\)

\(\text{max} = 48 \text{ in} \quad [1.2 \text{ m}]\)

\(\text{horizontal core spacing} = 16 \text{ in} \quad \text{space reinforcement} 32 \text{ inches on center} \quad [0.8 \text{ m}]\)

\(32 \text{ in} \leq 40 \text{ in} \quad \text{OK} \quad [0.8 \text{ m} < 1 \text{ m}]\)

Install one Grade 40 (276 MPa), minimum #3 bar at 32 inches (813 mm) on center. At least one continuous horizontal reinforcement bar should be placed within the top 12 inches (305 mm) of the wall story.

2.3.7.2 Reinforcement Around Openings

Recall in Section 2.2 that the loads were determined assuming that the wall had no openings. Wall openings create greater loads on the vertical core adjacent to the opening; therefore, check to determine if the reinforcement around openings suggested in Section 1.3.7 is sufficient.

Check the vertical core adjacent to the master bedroom window in the north wall since this is the largest opening in the second-story wall at Section A as shown in Figures 2-3 and 2-4. The designer may use the same design procedure as follows to check the required reinforcement around other openings in the second-story wall; however, calculations for other openings in the
ICF Design Example: Structural Plain Concrete Wall

Second-story wall are not included here. The following example follows the design procedures described in Section 1.2 for structural reinforced concrete walls.

(a) Determine Factored Loads

Refer to Section 2.2 for moments calculated and refer to Figure 2-14 in Section 2.6 for lintel loads transmitted to the vertical core.

\[ P_u = 1.4(251 \text{ plf}) + 1.7(735 \text{ plf}) \left( \frac{6.5 \text{ ft}}{2} \right) = 5,203 \text{ lb} \]  \[ [23.1 \text{ kN}] \]

\[ M_u = 2,959 \text{ in} - \text{lb} \]  \[ [334 \text{ N-m}] \]

(b) Determine Slenderness

Note that the unbraced length of the vertical core is assumed to be the height of the window opening.

\[ \frac{(1)(5 \text{ ft}) \left( \frac{12 \text{ in}}{\text{ft}} \right)}{0.3(5 \text{ in})} = 40 \quad : \quad \text{slender} \]

(c) Determine Magnified Moment

Use the equations in Appendix C for non-sway frames since the tables in Appendix C do not include heights less than 8 feet (2.4 m). Try one #4 bar in the vertical core. Refer to Section 2.2 for moments calculated and refer to part (a) of this section for the factored axial load.

\[ M_u = 2,959 \text{ in} - \text{lb} \]  \[ [338 \text{ N-m}] \]

\[ P_u = 5,203 \text{ lb} \]  \[ [23.1 \text{ kN}] \]

\[ P_{u,\text{dead}} = 1.4(251 \text{ plf}) \left( \frac{6.5 \text{ ft}}{2} \right) = 1,142 \text{ lb} \]  \[ [5.1 \text{ kN}] \]

\[ M_{u,\text{min}} = 5,203 \text{ lb}(0.6 + 0.03(5 \text{ in})) = 3,902 \text{ in} - \text{lb} \quad \text{GOVERS} \]  \[ [441 \text{ N-m}] \]

\[ e = \frac{3,902 \text{ lb}}{5,203 \text{ lb}} = 0.75 \text{ in} \]  \[ [19 \text{ mm}] \]

\[ \beta_d = \frac{1.142 \text{ lb}}{5,203 \text{ lb}} = 0.22 \]

\[ \rho = \frac{0.20 \text{ in}^2}{(5 \text{ in})(6.25 \text{ in})} = 0.0064 \]

\[ E_c = 57,000\sqrt{3,000 \text{ psi}} = 3,122,019 \text{ psi} \]  \[ [21.5 \text{ GPa}] \]

\[ \beta = 0.9 + 0.5(0.22)^2 - 12(0.0064) = 0.85 \quad \text{use} \ 1.0 \]
ICF Design Example: Structural Plain Concrete Wall

\[ EI = \frac{0.4(3,122,019 \text{ psi}) \left( \frac{6.25 \text{ in}(5 \text{ in})^3}{12} \right)}{1.0} = 81,302,578 \text{ psi} \]

\[ C_m = 1.0 \]

\[ P_c = \frac{\pi^2 (81,302,578 \text{ psi})}{[1(5 \text{ ft})(\frac{12 \text{ in}}{\text{ft}})]^2} = 222,896 \text{ lb} \]

\[ \delta_{ns} = \frac{1}{1 - \frac{1}{0.75(222,896 \text{ lb})}} = 1.03 \]

\[ M_{ns} = 1.03(3,902 \text{ in} - \text{lb}) = 4,019 \text{ in} - \text{lb} = 0.3 \text{ ft - kip} \]

\[ [560.6 \text{ GPa}] \]

\[ [991.5 \text{ kN}] \]

\[ [450 \text{ N-m}] \]

Figure 2-10 6-inch (152 mm) Waffle-Grid ICF Structural Reinforced Concrete Interaction Diagram
(d) Determine Reinforcement

Plot $M_{ns}$ and $P_U$ from (c) on the interaction diagram for a 6-inch (152 mm) waffle-grid wall with $f_{c'} = 3,000$ psi (21 MPa) and $f_y = 40,000$ psi (276 MPa). The interaction diagram can be found in Appendix D; however, the interaction diagram is reproduced in Figure 2-10 with the location of the plotted point illustrated. The plotted point lies well above and to the left of the reinforcement line for $\rho = 0.0064$; therefore, one #4 bar is sufficient for this opening.

Based on the calculations above and assuming that other wall openings in the second-story wall yield similar results although not shown here, use the following reinforcement for openings in the second-story wall.

For openings less than 2 feet (0.6 m) in width:

No reinforcement by inspection

For openings 2 to 4 feet (0.6 to 1.2 m) in width:

One #4 bar at bottom and top of opening extending 24 inches (610 mm) beyond each side of the opening

Plus One #4 bar placed vertically on each side of the opening spanning the wall story height

Plus Horizontal reinforcement at top of opening as required by lintel design.
2.4 DESIGN FIRST-STORY WALL

2.4.1 Select Trial Wall Section and Properties

Try a structural plain concrete 6-inch (152 mm) waffle-grid wall system and assume the concrete compressive strength is 3,000 psi (21 MPa). Refer to Figure 2-5 for the appropriate dimensions of the waffle-grid wall section. The following example follows the design procedure described in Section 1.3 for structural plain concrete walls.

2.4.2 Determine Nominal and Factored Loads

Refer to Figures 2-7 and 2-8 for a summary of nominal and factored loads acting on the first-story wall.

2.4.3 Check Perpendicular Shear

According to Figure 2-8, the critical factored perpendicular shear load, \(V_u\), experienced by the first-story wall occurs at the bottom of the wall story due to ACI Load Case (3).

\[
V_u = 124 \text{ plf} \left( \frac{ft}{1 \text{ vertical core}} \right) = 124 \text{ lb / post} \quad [552 \text{ N/post}]
\]

\[
\phi V_n = 0.65 \left( \frac{4}{3} \right) \sqrt{3,000 \text{ psi} (6.25 \text{ in})(5 \text{ in})} = 1,483 \text{ lb / post} \quad [6.6 \text{ kN/post}]
\]

\[
V_u \leq \phi V_n \quad \text{OK}
\]

2.4.4 Check Parallel (In-Plane) Shear

According to Figure 2-8, the critical factored parallel shear load, \(V_u\), for the first-story wall occurs due to ACI Load Case (3). Recall that we calculated the parallel shear on the west wall since it experiences the highest loads per lineal foot of solid wall. Refer to Figure 2-6 for the west wall elevation. Note that the critical factored parallel shear load is divided by the total length of the west wall minus any openings in the wall since this is the portion of the wall available to resist shear.

\[
V_u = \frac{12,613 \text{ lb}}{(32.7 \text{ ft} - (9 \text{ ft} + 3 \text{ ft} + 4 \text{ ft})}) \left( \frac{ft}{1 \text{ vertical core}} \right) = 755 \text{ lb / post} \quad [3.4 \text{ kN/post}]
\]

\[
\phi V_n = 0.65 \left( \frac{4}{3} \right) \sqrt{3,000 \text{ psi} (5 \text{ in})(6.25 \text{ in})} = 1,483 \text{ lb / post} \quad [6.6 \text{ kN/post}]
\]

\[
V_u \leq \phi V_n \quad \text{OK}
\]
2.4.5 Check Compression and Tension

According to Figure 2-8, the critical maximum total moment, $M_u$, experienced by the first-story wall occurs at mid-height due to ACI Load Case (2). The corresponding total factored axial load, $P_u$, is also taken at the mid-height of the second-story wall based on ACI Load Case (2).

$$M_u = 4,295 \text{ in}-\text{lb} / \text{ft} \left( \frac{\text{ft} \text{ vertical core}}{\text{vertical core}} \right) = 4,295 \text{ in}-\text{lb} / \text{post}$$

$$P_u = 2,347 \text{ plf} \left( \frac{\text{ft} \text{ vertical core}}{\text{vertical core}} \right) = 2,347 \text{ lb/post} = 2.35 \text{ kip/post}$$

$$M_{u,min} = 0.1(5\text{ in})(2,347 \text{ plf}) \left( \frac{\text{ft}}{\text{vertical core}} \right) = 1,174 \text{ in}-\text{lb} / \text{post}$$

$$= 0.10 \text{ ft-kip/post}$$

Plot $M_u$ and $P_u$ on the interaction diagram for a 6-inch (152 mm) waffle-grid wall. The interaction diagram can be found in Appendix E; however, the interaction diagram is reproduced in Figure 2-11 with the location of the plotted point illustrated. The plotted point lies within the lower tension boundary, the upper compression boundary for a 9-foot (2.7 m) wall height, and the reference axes; therefore, a 6-inch (152 mm) waffle-grid structural plain concrete wall is sufficient for the given loading conditions.
2.4.6 Check Deflection

Assume a gypsum board interior finish exposed to view. The following deflection calculations are based on suggestions made in Section 1.3.6 for service live loads taking only wind loads into account. Refer to Figure A-1 for the maximum deflection equation.

\[
\Delta_{\text{actual}} = \frac{5(21.0 \text{ psf})(1 \text{ ft})(9 \text{ ft})^4 \left(\frac{1,728 \text{ in}^3}{\text{ft}^3}\right)}{384(3,122,019 \text{ psi}) \left(\frac{6.25 \text{ in}}{5 \text{ in}}\right) \left(\frac{12 \text{ in}}{\text{ft}}\right) \left(\frac{1}{360}\right)} = 0.02 \text{ in} \quad [0.5 \text{ mm}]
\]

\[
\Delta_{\text{allowable}} = \frac{(9 \text{ ft})(12 \text{ in})}{360} = 0.3 \text{ in} \quad [8 \text{ mm}]
\]

\[\Delta_{\text{actual}} \leq \Delta_{\text{allowable}} \quad \text{OK}\]
2.4.7 Determine Reinforcement

A nominal amount of reinforcement is required in a structural plain concrete wall. Based on the suggestions of Section 1.3.7, the following reinforcement will be installed:

### 2.4.7.1 Horizontal Reinforcement

- \( 8(5\text{ in}) = 40\text{ in} \) \( \text{GOVERNS} \) \( [1\text{ m}] \)
- \( \text{max} = 48\text{ in} \) \( [1.2\text{ m}] \)
- horizontal core spacing = 16 in :: space reinforcement 32 inches on center \( [0.8\text{ m}] \)
- 32 in \( \leq 40\text{ in} \) \( \text{OK} \) \( [0.8\text{ m} < 1\text{ m}] \)

Install one Grade 40 (276 MPa), minimum #3 bar at 32 inches (813 mm) on center. At least one continuous horizontal reinforcement bar should be placed within the top 12 inches (305 mm) of the wall story.

### 2.4.7.2 Reinforcement Around Openings

Recall in Section 2.2, that the loads were determined assuming that the wall had no openings. Wall openings create greater loads on the vertical core adjacent to the opening; therefore, check to determine if the reinforcement around openings suggested in Section 1.3.7 is sufficient.

Check the vertical core adjacent to the family room door in the north wall since this is the largest opening in the first-story wall at Section A as shown in Figures 2-3 and 2-4. The designer may use the same design procedure as follows to check the required reinforcement around other openings in the first-story wall; however, calculations for other openings in the first-story wall are not included here. The following example follows the design procedures described in Section 1.2 for structural reinforced concrete walls.

(a) Determine Factored Loads

Refer to Section 2.2 for moments calculated and refer to Figure 2-15 in Section 2.7 for lintel loads transmitted to the vertical core.

\[
\begin{align*}
P_u &= \left[1.4(254\text{ plf}) + 1.7(213\text{ plf})\right]\left(\frac{9.5\text{ ft}}{2}\right) = 3,409\text{ lb} [15.2\text{ kN}] \\
M_u &= 4,295\text{ in} - \text{lb} [485.3\text{ N-m}]
\end{align*}
\]
(b) Determine Slenderness

Note that the unbraced length of the vertical core is assumed to be the height of the door opening.

\[
\frac{(1)(7 \text{ ft})(12 \text{ in})}{0.3(5 \text{ in})} = 56 \quad \therefore \text{slender}
\]

(c) Determine Magnified Moment

Use the equations in Appendix C for non-sway frames since the tables in Appendix C do not include heights less than 8 feet (2.4 m). Try one #4 bar in the vertical core.

\[
M_u = 4,295 \text{ in} - \text{lb} \quad \text{GOVERNS}
\]

\[
P_u = 3,409 \text{ lb}
\]

\[
P_{x,\text{dead}} = 1.4(254 \text{ plf})(\frac{9.5 \text{ ft}}{2}) = 1,689 \text{ lb}
\]

\[
M_{u,\text{min}} = 3,409 \text{ lb}(0.6 + 0.03(5 \text{ in})) = 2,557 \text{ in} - \text{lb}
\]

\[
e = \frac{4,295 \text{ lb}}{3,409 \text{ lb}} = 1.25 \quad \text{in}
\]

\[
\beta_d = \frac{1,689 \text{ lb}}{3,409 \text{ lb}} = 0.5
\]

\[
\rho = \frac{0.20 \text{ in}^2}{(5 \text{ in})(6.25 \text{ in})} = 0.0064
\]

\[
E_c = 57,000\sqrt{3,000 \text{ psi}} = 3,122,019 \text{ psi}
\]

\[
\beta = 0.9 + 0.5(0.5)^2 - 12(0.0064) = 0.95 \quad \therefore \text{use 1.0}
\]

\[
0.4(3,122,019 \text{ psi}) \left( \frac{6.25 \text{ in}(5 \text{ in})^3}{12} \right) = 81,302,578 \text{ psi}
\]

\[
C_e = 1.0
\]

\[
P_c = \frac{\pi^2(81,302,578 \text{ psi})}{1(7 \text{ ft})(12 \text{ in})} = 113,722 \text{ lb}
\]

\[
\delta_{\text{ns}} = \frac{1}{3,409 \text{ lb}} = 0.75(113,722 \text{ lb}) = 1.04
\]

\[
M_{\text{ns}} = 1.04(4,295 \text{ in} - \text{lb}) = 4,467 \text{ in} - \text{lb} = 0.37 \text{ ft} - \text{kip}
\]
(d) Determine Reinforcement

Plot $M_{ns}$ and $P_u$ from (c) on the interaction diagram for a 6-inch (152 mm) waffle-grid wall with $f_c = 3,000$ psi (21 MPa) and $f_y = 40,000$ psi (276 MPa). The interaction diagram can be found in Appendix D; however, the interaction diagram is reproduced in Figure 2-12 with the location of the plotted point illustrated. The plotted point lies well above and to the left of the reinforcement line for $\rho = 0.0064$; therefore, one #4 bar is sufficient for this opening.

Based on the calculation above and assuming that other wall openings in the first-story wall yield similar results although not shown here, use the following reinforcement for openings in the first-story wall.

For openings less than 2 feet (0.6 m) in width: No reinforcement by inspection

For openings 2 to 4 feet (0.6 to 1.2 m) in width: One #4 bar at bottom and top of opening extending 24 inches (610 mm) beyond each side of the opening
For openings greater than 4 feet (1.2 m) in width:

- One #4 bar at bottom of opening extending 24 inches (610 mm) beyond each side of the opening
- One #4 bar placed vertically on each side of the opening spanning the wall story height
- Horizontal reinforcement at top of opening as required by lintel design.
2.5 DESIGN FOUNDATION WALL

2.5.1 Select Trial Wall Section and Properties

Try a structural reinforced concrete 8-inch (203 mm) waffle-grid wall system and assume the concrete compressive strength is 3,000 psi (21 MPa) and the yield strength of reinforcement is 40,000 psi (276 MPa). Refer to Figure 2-5 for the appropriate dimensions of the waffle-grid wall section. Try a vertical reinforcement spacing at 24 inches (610 mm) on center. The following example follows the design procedure described in Section 1.2 for structural reinforced concrete walls.

2.5.2 Determine Nominal and Factored Loads

Refer to Figures 2-7 and 2-8 for a summary of nominal and factored loads acting on the foundation wall.

2.5.3 Check Perpendicular Shear

According to Figure 2-8, the critical factored perpendicular shear load, $V_u$, experienced by the foundation wall occurs at the bottom of the wall story due to ACI Load Case (2) or (3). Note that only the reinforced vertical cores are assumed to resist perpendicular wall shear.

$$V_u = \left(1,013 \text{ psf}\right) \left(\frac{2 \text{ ft}}{\text{reinforced vertical core}}\right) = 2,026 \text{ lb / post}$$ [9.0 kN/post]

$$V_s = 0 \quad \text{assume no stirrups}$$

$$\phi V_n = 0.85(2)\sqrt{3,000 \text{ psi}(7 \text{ in})\left(\frac{7 \text{ in}}{2}\right)} = 2,281 \text{ lb / post}$$ [10.1 kN/post]

$$V_u \leq \phi V_n \quad \text{OK}$$

2.5.4 Check Parallel (In-Plane) Shear

The basement walls are constrained against lateral loads by the passive pressure of the soil and soil-wall friction. Parallel shear on the basement wall can be neglected by design inspection.

2.5.5 Sway Determination

This home is determined to be a non-sway structure by observation since all four walls are constructed and braced with an ICF bearing wall system.

2.5.6 Determine Slenderness

To determine whether the wall is slender, the radius of gyration is approximated as $0.3h$ for rectangular compression members and $k$ is assumed to be 1.0 since the wall is tied to the footing below and the floor above.
The slenderness ratio is larger than 34 but less than 100; therefore, the wall is considered slender and may be designed using the moment magnifier method in lieu of second-order analysis.

### 2.5.7 Determine Magnified Moment

When designing foundation walls, it is often recommended that backfilling occur after the first-story above-grade walls are constructed and at least seven days after the concrete is poured for the foundation walls. However, the designer should be aware that backfilling typically occurs three days after pouring the concrete for the foundation walls on a residential construction job site. To prevent foundation wall failure during construction due to early backfilling, the contractor should adequately brace the foundation walls. In certain situations, the designer may also elect to design the foundation wall for construction conditions assuming minimal bracing as discussed in Section 2.5.7.1. The designer may skip Section 2.5.7.1 if backfilling occurs as specified or the contractor provides adequate temporary bracing.

#### 2.5.7.1 Determine Magnified Moment for Construction Conditions

*Design load considered = factored dead load + factored earth load*

To use the tables in Appendix C, the values for the variables below are calculated using the factored loads from Figure 2-8. Note that only the reinforced vertical cores are assumed to resist moments and axial loads experienced by the wall.

\[
M_u = \left( \frac{19,014 \text{ in-lb}}{\text{ft}} \right) \left( \frac{2 \text{ ft}}{1 \text{ reinforced vertical core}} \right) = 38,028 \text{ in-lb/post} \quad [4.3 \text{ kN-m/post}]
\]

\[
P_u = (1,731 \text{ plf}) \left( \frac{2 \text{ ft}}{1 \text{ reinforced vertical core}} \right) = 3,462 \text{ lb/post} = 3.5 \text{ kip/post} \quad [15.4 \text{ kN/post}]
\]

\[
P_{u,\text{dead}} = P_u
\]

\[
M_{u,\text{min}} = 3,462 \text{ lb/post} \left[ 0.6 + 0.03(7 \text{ in}) \right] = 2,804 \text{ in-lb/post} \quad [317 \text{ kN-m/post}]
\]

\[
e = \frac{38,028 \text{ in-lb/post}}{3,462 \text{ lb/post}} = 11 \text{ in}
\]

\[
\beta_d = \frac{3,462 \text{ lb/post}}{3,462 \text{ lb/post}} = 1.0
\]

\[
\rho = 0.0014 \quad \text{assumed}
\]
Using the Non-Sway Moment Magnifier Table found in Appendix C for an 8-inch (203 mm) waffle-grid wall with $f_c' = 3,000$ psi (21 MPa) and a wall height of 8.5 feet (2.6 m), the moment magnifier, $\delta_{NS}$, is approximately 1.12. The magnified moment is

$$M_{ns} = 1.12 \left( \frac{32 \text{ ft} - \text{kip}}{\text{post}} \right) = 3.6 \text{ ft} - \text{kip} / \text{post}$$

### 2.5.7.2 Determine Magnified Moment for Final Conditions

**Design load considered** = factored dead load + factored live load + factored earth load

To use the tables in Appendix C, the values for the variables below are calculated using the factored loads from Figure 2-8. Note that only the reinforced vertical cores are assumed to resist moments and axial loads experienced by the wall.

$$M_u = \left( \frac{18,369 \text{ in} - \text{lb}}{\text{ft}} \right) \left( \frac{2 \text{ ft}}{1 \text{ reinforced vertical core}} \right) = 36,738 \text{ in} - \text{lb} / \text{post}$$

$$P_u = \left( \frac{5.043 \text{ plf}}{1 \text{ reinforced vertical core}} \right) = 10,086 \text{ lb} / \text{post} = 10 \text{ kip} / \text{post}$$

$$P_{u,\text{dead}} = \left( \frac{2.692 \text{ plf}}{1 \text{ reinforced vertical core}} \right) = 5,384 \text{ lb} / \text{post} = 5.4 \text{ kip} / \text{post}$$

$$M_{u,\text{min}} = 10,086 \text{ lb} / \text{post} (0.6 + 0.03(7 \text{ in})) = 8,170 \text{ in} - \text{lb} / \text{post}$$

$$e = \frac{36,738 \text{ in} - \text{lb} / \text{post}}{10,086 \text{ lb} / \text{post}} = 3.6 \text{ in}$$

$$\beta_d = \frac{5,384 \text{ lb} / \text{post}}{10,086 \text{ lb} / \text{post}} = 0.5$$

$$\rho = 0.0014 \text{ assumed}$$

Using the Non-Sway Moment Magnifier Table found in Appendix C for an 8-inch (203 mm) waffle-grid wall with $f_c' = 3,000$ psi (21 MPa) and a wall height of 8.5 feet (2.6 m), the moment magnifier, $\delta_{NS}$, is approximately 1.32. The magnified moment is

$$M_{ns} = 1.32 \left( \frac{31 \text{ ft} - \text{kip}}{\text{post}} \right) = 4 \text{ ft} - \text{kip} / \text{post}$$

### 2.5.8 Determine Reinforcement

Plot $M_{NS}$ and $P_u$ from Sections 2.5.7.1 and 2.5.7.2 on the interaction diagram found in Appendix D for an 8-inch (203 mm) waffle-grid wall with $f_c' = 3,000$ psi (21 MPa) and $f_y = 40,000$ psi (276 MPa). The interaction diagram can be found in Appendix D; however, the interaction diagram is reproduced in Figure 2-13 with the location of the plotted points illustrated. The plotted points lie
well above and to the left of the reinforcement lines for \( \rho = 0.0063 \) and \( \rho = 0.0082 \). Vertical reinforcement required in the foundation wall is

\[
\begin{align*}
\text{therefore use:} & \quad \text{One \#4 bar at 6 inches (152 mm) on center (} \rho = 0.0082 \text{) for the construction condition} \\
\text{or use:} & \quad \text{One \#5 bar at 12 inches (305 mm) on center (} \rho = 0.0063 \text{) for the final condition}
\end{align*}
\]

Based on the suggestions of Section 1.2.8, the following reinforcement will be installed in addition to the vertical reinforcement specified above:

### 2.5.8.1 Horizontal Reinforcement

\[
8(7 \text{ in}) = 56 \text{ in} \quad [1.4 \text{ m}]
\]

\[
\text{max} = 48 \text{ in} \quad \text{GOVERNS} \quad [1.2 \text{ m}]
\]

---

**Figure 2-13** 8-Inch (203 mm) Waffle-Grid Structural Reinforced Concrete Interaction Diagram

2.5.8.1 Horizontal Reinforcement

\[
8(7 \text{ in}) = 56 \text{ in} \quad [1.4 \text{ m}]
\]

\[
\text{max} = 48 \text{ in} \quad \text{GOVERNS} \quad [1.2 \text{ m}]
\]

---

**Figure 2-13** 8-Inch (203 mm) Waffle-Grid Structural Reinforced Concrete Interaction Diagram
Install one Grade 40 (276 Mpa), minimum #3 bar at 48 inches (1.2 mm) on center. At least one continuous horizontal reinforcement bar should be placed within the top 12 inches (305 mm) of the wall story.

2.5.8.2 Reinforcement Around Openings

Recall in Section 2.2. that the loads were determined assuming that the wall had no openings. Wall openings create greater loads on the vertical core adjacent to the opening; however, since the openings in the basement wall are small and do not occur in the north or south walls where the heaviest loads occur, the following reinforcement is installed by observation.

<table>
<thead>
<tr>
<th>Width of Opening</th>
<th>Reinforcement Required</th>
</tr>
</thead>
<tbody>
<tr>
<td>Less than 2 ft (0.6 m)</td>
<td>No reinforcement by inspection</td>
</tr>
<tr>
<td>2 to 4 ft (0.6 to 1.2 m)</td>
<td>One #4 bar at bottom and top of opening extending 24 inches (610 mm) beyond each side of the opening</td>
</tr>
<tr>
<td>Greater than 4 ft (1.2 m)</td>
<td>One #4 bar at bottom of opening extending 24 inches (610 mm) beyond each side of the opening, plus one #4 bar placed vertically on each side of the opening spanning the wall story height, plus horizontal reinforcement at top of opening as required by lintel design</td>
</tr>
</tbody>
</table>

2.5.9 Check Deflection

Assume a gypsum board interior finish exposed to view. The following deflection calculations are based on the suggestions made in Section 1.2.9 for service live loads taking only earth loads into account. Note that the service load moment is multiplied by the moment magnification factor as suggested. Assume the earth load acts on the entire wall height for simplicity when calculating the maximum deflection.

\[
\Delta_{\text{maximum}} = \frac{0.01304(1.32)(0.5)(300 \text{ pcf})(1 \text{ ft})(8.5 \text{ ft})^5}{(0.1)(3,122,019 \text{ psi})} \left( \frac{7 \text{ in}}{12} \right) = 0.31 \text{ in} \quad [8 \text{ mm}]
\]

\[
\Delta_{\text{allowable}} = \frac{(8.5 \text{ ft})(12 \text{ in})}{240} = 0.43 \text{ in} \quad [11 \text{ mm}]
\]

\[
\Delta_{\text{actual}} \leq \Delta_{\text{allowable}} \quad \text{OK}
\]
2.6 DESIGN SECOND-STORY LINTEL

Design the lintel above the master bedroom window in the north wall (refer to Figures 2-2 through 2-4) since this is the largest span in the second-story wall with the greatest loads and assume the lintel has 6 inches (152 mm) of bearing on each side of the opening. The following example follows the design procedure described in Section 1.4 for lintels.

2.6.1 Select Trial Lintel Section and Properties

Assume the lintel depth is 12 inches (305 mm) and constructed of a 6-inch (152 mm) waffle-grid wall form. The concrete compressive strength is assumed to be 3,000 psi (21 MPa) and the yield strength of reinforcement is assumed to be 40,000 psi (276 MPa). Refer to Figure 2-14 for the lintel cross-section. The geometric properties of the trial lintel cross-section are calculated below assuming the section is unreinforced.

\[ NA_y = \frac{\sum A_y}{\sum A} = \frac{(5 \text{ in})(4 \text{ in})(10 \text{ in})+(5 \text{ in})(3 \text{ in})(15 \text{ in})+(2 \text{ in})(5 \text{ in})(5.5 \text{ in})}{(5 \text{ in})(4 \text{ in})+(5 \text{ in})(3 \text{ in})+(2 \text{ in})(5 \text{ in})} = 6.17 \text{ in} \quad [157 \text{ mm}] \]

\[ I_g = \frac{bh^3}{12} + Ad^2 = \frac{(5 \text{ in})(4 \text{ in})^3}{12} + (5 \text{ in})(4 \text{ in})(3.83 \text{ in})^2 + \frac{(5 \text{ in})(3 \text{ in})^3}{12} + (5 \text{ in})(3 \text{ in})(4.67 \text{ in})^2 + \frac{(2 \text{ in})(5 \text{ in})^3}{12} + (2 \text{ in})(5 \text{ in})(0.67 \text{ in})^2 = 684 \text{ in}^4 \quad [2.8 \text{ dm}^4] \]

![Figure 2-14 Lintel above Master Bedroom Window](image)

2.6.2 Determine Nominal and Factored Loads

Refer to Figure 2-14 for the nominal lintel loading diagram. The loads in Figure 2-14 are based on the calculations in Section 2.2. Factored loads will be determined in the sections below as required in accordance with Section 1.4.2. The trial lintel section is 12 inches (305 mm) deep and is constructed of a 6-inch (152 mm) waffle-grid wall form; therefore, the lintel dead weight is 55 plf (803 N/m).
2.6.3 Check Deflection

For purposes of calculating deflection, assume the lintel supports 100 percent of the dead load and assume 33 percent of the live load is sustained. In addition, assume that the maximum deflection limit for the window below is L/360. Refer to Figure A-1 for the maximum deflection equation.

\[
\Delta_{allowable} = \frac{6.5 \text{ ft} \left( \frac{12 \text{ in}}{\text{ft}} \right)}{360} = 0.22 \text{ in} \quad [5.6 \text{ mm}]
\]

\[
\Delta_{actual} = \frac{5(251 \text{ plf} + (0.33)(735 \text{ plf}) \left(\frac{6.5 \text{ ft}^4}{\text{ft}^3}\right) \left(\frac{1,728 \text{ in}^3}{\text{ft}^3}\right)}{384(0.1)(3,122,019 \text{ psi})(684 \text{ in}^4)} = 0.09 \text{ in} \quad [2.2 \text{ mm}]
\]

\[\Delta_{actual} \leq \Delta_{allowable} \quad \text{OK}\]

2.6.4 Check Nominal Moment Strength

The nominal moment strength of the lintel is calculated based on the ACI Load Case in Section 1.4.2. Refer to Figure 2-14 for the nominal loads acting on the lintel and for the lintel dimensions. Assume the lintel is reinforced with one #5 bar \(A_s = 0.31 \text{ in}^2 (200 \text{ mm}^2)\).

\[M_u = \frac{\left[1.4(251 \text{ plf}) + 1.7(735 \text{ plf})\right](6.5 \text{ ft})^2}{8} = 8,455 \text{ ft-lb} = 101,460 \text{ in-lb} \quad [11.5 \text{ kN-m}]
\]

\[a = \frac{0.31 \text{ in}^2 \left(40,000 \text{ psi}\right)}{0.85 \left[3,000 \text{ psi}\right] \left(5 \text{ in}\right)} = 0.97 \text{ in} \quad [24.6 \text{ mm}]
\]

\[\phi M_n = 0.9 \left(0.31 \text{ in}^2 \left(40,000 \text{ psi}\right)\right) \left(12 \text{ in} - 1.5 \text{ in cover} - 0.375 \text{ in stirrup} - \frac{0.97 \text{ in}}{2}\right) = 107,582 \text{ in-lb} \quad [12 \text{ kN-m}]
\]

\[M_u \leq \phi M_n \quad \text{OK}\]

2.6.5 Check Nominal Shear Strength

The nominal shear strength of the lintel is calculated based on the ACI Load Case in Section 1.4.2. Refer to Figure 2-14 for the nominal loads acting on the lintel and for the lintel dimensions.

\[V_u = \left[1.4(251 \text{ plf}) + 1.7(735 \text{ plf})\right] \frac{6.5 \text{ ft}}{2} = 5,203 \text{ lb} \quad [23.1 \text{ kN}]
\]

\[\phi V_c = (0.85)(2) \sqrt{3,000 \text{ psi} \left(2 \text{ in}\right) \left(10.13 \text{ in}\right)} = 1,886 \text{ lb} \quad [8.4 \text{ kN}]
\]

\[V_u \geq \frac{\phi V_c}{2} \quad \therefore \text{ use stirrups}\]

Try one #3 stirrup 6 inches (152 mm) on center (two per vertical core)

\[A_{v,min} = \frac{50 \left(2 \text{ in}\right) \left(6 \text{ in}\right)}{40,000 \text{ psi}} = 0.02 \text{ in}^2 \quad [12.9 \text{ mm}^2]\]
ICF Design Example: Lintels

\[ \phi V_s = \frac{0.85(0.11 \text{ in}^2)(40,000 \text{ psi})(10.13 \text{ in})}{6 \text{ in}} = 6.311 \text{ lb} \quad \longrightarrow \quad \text{GOVERNS} \quad [28 \text{ kN}] \]

\[ \phi V_{s,\text{max}} = 0.85(8)\sqrt{3,000 \text{ psi}(2 \text{ in})(10.13 \text{ in})} = 7,542 \text{ lb} \]

\[ \phi V_n = (1,886 \text{ lb} + 6,311 \text{ lb}) = 8,197 \text{ lb} \]

\[ V_u \leq \phi V_n \quad \text{OK} \quad \text{[33.5 kN]} \]

\[ V_{u} \geq \phi V_{n} \quad \text{[36.5 kN]} \]


2.7 DESIGN FIRST-STORY LINTEL

Design the lintel above the family room door in the north wall since it is the largest span in the first-story wall and assume the lintel has 6 inches (152 mm) of bearing on each side of the opening. The following example follows the design procedure described in Section 1.4 for lintels.

2.7.1 Select Trial Lintel Section and Properties

Assume the lintel is 16 inches (406 mm) deep and is constructed of a 6-inch (152 mm) waffle-grid wall form. The concrete compressive strength is assumed to be 3,000 psi (21 MPa) and the yield strength of reinforcement is assumed to be 40,000 psi (276 MPa). Refer to Figure 2-15 for the lintel cross-section. The geometric properties of the trial lintel section are calculated as follows assuming the section is unreinforced:

\[
NA_x = \frac{\Sigma A y}{\Sigma A} = \frac{(5 \text{ in})(4 \text{ in})(14 \text{ in}) + (5 \text{ in})(3 \text{ in})(15 \text{ in}) + (2 \text{ in})(9 \text{ in})(7.5 \text{ in})}{(5 \text{ in})(4 \text{ in}) + (5 \text{ in})(3 \text{ in}) + (2 \text{ in})(9 \text{ in})} = 8.25 \text{ in} \quad [210 \text{ mm}]
\]

\[
I_e = \frac{bh^3}{12} + Ad^2 = \frac{(5\text{ in})(4\text{ in})^3}{12} + (5\text{ in})(4\text{ in})(6.06 \text{ in})^2 + \frac{(5\text{ in})(3\text{ in})^3}{12} + (5\text{ in})(3\text{ in})(6.44 \text{ in})^2 + \frac{(2\text{ in})(5\text{ in})(0.75)}{12} = 1,421 \text{ in}^4 \quad [5.9 \text{ dm}^4]
\]

2.7.2 Determine Nominal and Factored Loads

Refer to Figure 2-15 for nominal lintel loading diagram. The loads in Figure 2-15 are based on the calculations in Section 2.2. Factored loads will be determined in the following sections as required in accordance with Section 1.4.2. The trial lintel section is 16 inches (406 mm) deep and is constructed of a 6-inch (152 mm) waffle-grid wall form; therefore, the lintel dead weight is 73 plf (1.07 kN/m).

![Figure 2-15 Lintel above Family Room Door](image-url)
2.7.3 Check Deflection

For purposes of calculating deflection, assume that the lintel supports 100 percent of the dead load and assume 33 percent of the live load is sustained. In addition, assume that the maximum deflection limit for the French doors below is \( L/360 \). Refer to Figure A-1 for the maximum deflection equation.

\[
\Delta_{\text{allowable}} = \frac{9.5 \text{ ft} \left( \frac{12 \text{ in}}{\text{ft}} \right)}{360} = 0.32 \text{ in} \quad [8 \text{ mm}]
\]

\[
\Delta_{\text{actual}} = \frac{5(254 \text{ plf} + (0.33)213 \text{ plf}) (9.5 \text{ ft})^4}{384(0.1)(3,122,019 \text{ psi})(1,421 \text{ in}^4)} \left( \frac{1.728 \text{ in}^3}{\text{ft}^3} \right) = 0.19 \text{ in} \quad [4.8 \text{ mm}]
\]

\[ \Delta_{\text{actual}} \leq \Delta_{\text{allowable}} \quad \text{OK} \]

2.7.4 Check Nominal Moment Strength

\[
M_u = \frac{[1.4(254 \text{ plf}) + 1.7(213 \text{ plf})] (9.5 \text{ ft})^2}{8} = 8,097 \text{ ft}-\text{lb} = 97,164 \text{ in}-\text{lb} \quad [11 \text{ kN-m}]
\]

\[
d = \left( \frac{0.31 \text{ in}^2}{0.85(3,000 \text{ psi})/5 \text{ in}} \right) = 0.97 \text{ in} \quad [24.6 \text{ mm}]
\]

\[
\phi M_u = 0.9 \left( 0.31 \text{ in}^2 \right) \left( \frac{(40,000 \text{ psi})(16 \text{ in} - 1.5 \text{ in cover} - 0.375 \text{ in stirrup}) - 0.97 \text{ in}}{2} \right) = 152,222 \text{ in}-\text{lb} \quad [17.2 \text{ kN-m}]
\]

\[
M_u \leq \phi M_u \quad \text{OK}
\]

2.7.5 Check Nominal Shear Strength

The nominal shear strength of the lintel is calculated based on the ACI Load Case in Section 1.4.2. Refer to Figure 2-15 for the nominal loads acting on the lintel and for the lintel dimensions.

\[
V_u = \left[1.4(254 \text{ plf}) + 1.7(213 \text{ plf}) \right) \frac{9.5 \text{ ft}}{2} = 3,409 \text{ lb} \quad [15.2 \text{ kN}]
\]

\[
\phi V_u = 0.85(2)(40,000 \text{ psi})(2 \text{ in})(14.13 \text{ in}) = 2,630 \text{ lb} \quad [11.7 \text{ kN}]
\]

\[
V_u \geq \phi V_c \quad \text{use stirrups}
\]

\[
A_{v,\text{min}} = \frac{50(2 \text{ in})(6 \text{ in})}{40,000 \text{ psi}} = 0.02 \text{ in}^2 \quad [12.9 \text{ mm}^2]
\]

\[
\phi V_s = \frac{0.85(0.11 \text{ in}^2)(40,000 \text{ psi})(14.13 \text{ in})}{6} = 8,805 \text{ lb} \quad \text{GOVERNS} \quad [39.2 \text{ kN}]
\]
ICF Design Example: Lintels

\[
\phi V_{u,\text{max}} = 0.85(8)\sqrt{3,000 \text{ psi} (2 \text{ in})(14.13 \text{ in})} = 10,522 \text{ lb} \\
\phi V_n = (2,630 \text{ lb} + 8,805 \text{ lb}) = 11,435 \text{ lb} \\
V_u \leq \phi V_n \quad \text{OK}
\]

[46.8 kN]

[50.9 kN]
2.8 DESIGN FOOTING CONNECTION

The following example follows the design procedure described in Section 1.5 for footing connections. Design the footing connection for the south wall since it experiences the highest loads and assume the footing is 16 inches (406 mm) wide by 12 inches (305 mm) deep due to soil conditions.

2.8.1 Check Bearing Strength of Footing

Refer to Figure 2-5 for nominal loads acting on the footing and refer to Figure 2-16 for the dimensions used for the footing connection calculations.

\[
B_{c,\text{actual}} = 1.4 \left( \frac{ft}{1 \text{ vertical core}} \right) \left( 2(234 \text{ plf} + 248 \text{ plf} + 319 \text{ plf} + 93 \text{ plf}) + 196 \text{ plf} \right) + 1.7 \left( \frac{ft}{1 \text{ vertical core}} \right) \left( 370 \text{ plf} + 278 \text{ plf} + 163 \text{ plf} + 572 \text{ plf} \right) = 5,129 \text{ lb} \quad [22.8 \text{ kN}]
\]

\[
B_{c,\text{allowable}} = 0.7(0.85)(3,000 \text{ psi})(49 \text{ in}^2)(2.0) = 174,930 \text{ lb} \quad [778 \text{ kN}]
\]

\[B_{c,\text{actual}} \leq B_{c,\text{allowable}} \quad \text{OK} \quad \therefore \text{no dowels required for bearing}\]

\[\text{Figure 2-16 Footing}\]

2.8.2 Check Shear Transfer

According to Figure 2-8, the critical factored perpendicular shear load, \(V_u\), occurring at the bottom of the wall is due to ACI Load Case (2) or (3). Assume the coefficient of friction is 0.6 and one Grade 40 (276 MPa), #3 bar (\(A_v = 0.11 \text{ in}^2 (71 \text{ mm}^2)\)) is spaced every 2 feet (0.6 m) on center. Note that only the reinforced vertical cores are assumed to transfer shear.

\[
V_u = 1,013 \text{ plf} \left( \frac{2 \text{ ft}}{1 \text{ reinforced vertical core}} \right) = 2,026 \text{ lb/ post} \quad [9.0 \text{ kN/post}]
\]
2.8.3 Determine Development Length of Dowels for Shear Transfer

\[ l_{hb} = \frac{1200(0.375 \text{ in})}{\sqrt{3000}} = 8.2 \text{ in} \]

\[ l_{db} = \max\left(\left[\frac{(8.2 \text{ in})(40,000 \text{ psi})(0.7)}{60,000 \text{ psi}}\right](0.10 \text{ in}^2 / \text{ post})\right) = 3.5 \text{ in} \]

\[ l_{db} = \max(3 \text{ in}, 6 \text{ in}) = 6 \text{ in} \]

\[ l_{db} = 6 \text{ in} \]

\[ l_{db} \geq 6 \text{ in} \]
2.9 DESIGN ROOF CONNECTION: BOLTED SILL PLATE

The following example follows the design procedure described in Section 1.6 for bolted sill plate roof connections. Design the roof connection for the south or north wall and refer to Figure 2-17 for the building plan and building sections.

2.9.1 Determine Design Loads

Refer to Figure 2-1 for the given wind and uplift pressures on the structure and neglect the negative pressure acting on the leeward wall. The calculations for unit shear perpendicular to the grain for the sill plate conservatively assume that the roof diaphragm is more flexible than the concrete walls below. Therefore, the shear load experienced by the bolts in the sill plate is the result, $V_b$, of the wind load on the vertical projection of the roof area. If the designer has reason to assume that the roof diaphragm is more stiff than the concrete walls below (not shown in the example below), the shear load experienced by the bolts in the sill plate would be the result of the wind load acting on the top half of the second-story wall since the roof diaphragm is assumed to rigidly support the top of the wall. In reality, the actual model is somewhere between these two extremes.

$$V_a = (0.5)(210 \text{ psf})(0.5)(11 \text{ ft})(32.7 \text{ ft}) = 1,888 \text{ lb} \quad [8.4 \text{kN}]$$

$$v_a = \frac{1,888 \text{ lb}}{38.5 \text{ ft}} = 49 \text{ pfl} \quad [715 \text{ N/m}]$$

$$V_b = (0.5)(210 \text{ psf})(11 \text{ ft})(38.5 \text{ ft}) = 4,447 \text{ lb} \quad [19.8 \text{kN}]$$

$$v_b = \frac{4,447 \text{ lb}}{38.5 \text{ ft}} = 116 \text{ pfl} \quad [1.7 \text{kN/m}]$$

2.9.2 Assume Connection Spacing and Size

Try a $\frac{1}{2}$-inch (13 mm) diameter ASTM A36 anchor bolt ($A_b=0.196 \text{ in}^2 \ (126.5 \text{ mm}^2)$), 6 inches (152 mm) long and spaced 4 feet (1.2 m) on center. Assume the washers are 1-1/4 inches (32 mm) in diameter. According to Part 4, “Connections”, in ASD/AISC’s *Manual of Steel Construction*, the allowable tensile strength of the bolt, $F_t$, the ultimate tensile strength of the bolt, $F_u$, and the allowable shear strength of the bolt, $F_v$, are as follows:

$$F_u = 58,000 \text{ psi} \quad [400 \text{ MPa}]$$

$$F_t = 19,100 \text{ psi} \quad [132 \text{ MPa}]$$

$$F_t = \sqrt{44^2 - 4.39 f_v^2} \text{ for combined shear and tension}$$

$$F_t = \sqrt{57.2^2 - 4.39 f_v^2} \text{ for combined shear and tension due to wind or seismic}$$

$$F_v = (0.17)(58,000 \text{ psi}) = 9,860 \text{ psi for threads included in shear plane} \quad [68 \text{ MPa}]$$

$$F_v = (0.22)(58,000 \text{ psi}) = 12,760 \text{ psi for threads excluded from shear plane} \quad [88 \text{ MPa}]$$
Assume the sill plate is constructed of a No.2 grade Douglas Fir-Larch 2x8 with the following properties as taken from Table 4A in AF&PA’s Design Values for Wood Construction.

\[
F_b = 900 \text{ psi} \\
F_b' = (1.15)(1.6)(12)(900 \text{ psi}) = 1,987 \text{ psi modified for flat use, wind load duration, and size} \\
F_{cL} = 625 \text{ psi} \\
F_{cL}' = \left(\frac{125 + 0.375}{0.375}\right)(625 \text{ psi}) = 813 \text{ psi modified for small bearing area} \\
F_c = 1,350 \text{ psi} \\
F_c' = (1.05)(1.6)(1,350 \text{ psi}) = 2,268 \text{ psi modified for size and wind load duration} \\
S_{xy} = 2.719 \text{ in}^3 \\
h = 1.5 \text{ in} \\
b = 7.25 \text{ in}
\]
2.9.3 Check Shear in Bolt

Refer to Section 2.9.1 for the unit shear in the south wall sill plate and refer to Section 2.9.2 for the allowable bolt shear assuming the threads are included in the shear plane.

\[ F_v = 9,860 \text{ psi} \]
\[ f_{vb} = \frac{49 \text{ pfs}(4 \text{ ft bolt spacing})}{0.196 \text{ in}^2} = 1,000 \text{ psi} \]
\[ f_{vb} = \frac{116 \text{ pfs}(4 \text{ ft bolt spacing})}{0.196 \text{ in}^2} = 2,367 \text{ psi} \]
\[ f_v \leq F_v \quad \text{OK} \]

\[ f_{vb} = 9,860 \text{ psi} \quad [68 \text{ MPa}] \]
\[ f_{vb} = 1,000 \text{ psi} \quad [6.9 \text{ MPa}] \]
\[ f_{vb} = 2,367 \text{ psi} \quad [16.3 \text{ MPa}] \]

2.9.4 Check Tension in Bolt Due to Uplift and Shear-Friction

Refer to Section 2.9.1 for the calculated unit shear in the south wall sill plate and refer to Section 2.9.2 for the allowable tensile load on the bolt for combined shear and tension. Note that the total tensile load on the bolt is the difference between the factored uplift load and the factored dead load on the roof structure plus the factored tensile load due to shear friction.

\[ F_{tu} = \sqrt{572^2 - 4.39(1 \text{ kip})^2} = 57.2 \text{ ksi} = 57,200 \text{ psi} \quad [394 \text{ MPa}] \]
\[ F_{tb} = \sqrt{572^2 - 4.39(2.4 \text{ kip})^2} = 57 \text{ ksi} = 57,000 \text{ psi} \quad [393 \text{ MPa}] \]
\[ T_a = ((1.3)19 \text{ psf} - (0.9)12 \text{ psf}) \left( \frac{32.7 \text{ ft}}{2} \right)(4 \text{ ft bolt spacing}) \]
\[ + \frac{(1.3)49 \text{ pfs}(4 \text{ ft bolt spacing})}{0.6} = 1,334 \text{ lb} \]
\[ T_b = ((1.3)19 \text{ psf} - (0.9)12 \text{ psf}) \left( \frac{32.7 \text{ ft}}{2} \right)(4 \text{ ft bolt spacing}) \]
\[ + \frac{(1.3)116 \text{ pfs}(4 \text{ ft bolt spacing})}{0.6} = 1,914 \text{ lb} \]
\[ f_{ta} = \frac{1,334 \text{ lb}}{0.196 \text{ in}^2} = 6,806 \text{ psi} \quad [46.9 \text{ MPa}] \]
\[ f_{tb} = \frac{1,914 \text{ lb}}{0.196 \text{ in}^2} = 9,765 \text{ psi} \quad [67.3 \text{ MPa}] \]
\[ f_t \leq F_t \quad \text{OK} \]

2.9.5 Check Tension in Concrete (Anchorage Capacity)

Note that the tension in the concrete, \( V_{tu} \), is equivalent to the total tensile load on the bolt.

\[ V_{ua} = 1,334 \text{ lb} \quad [5.9 \text{ kN}] \]
\[ V_{ub} = 1,914 \text{ lb} \quad [8.5 \text{ kN}] \]
ICF Design Example: Roof Connections

\[
A_v = \text{minimum of} \left\{ \frac{\pi (6 \text{ in})^2}{\pi (5 \text{ in})} = 113 \text{ in}^2 \right\} = 78.5 \text{ in}^2
\]

\[
\phi V_c = 0.85(4)(78.5 \text{ in}^2)\sqrt{5,000 \text{ psi}} = 14,619 \text{ lb}
\]

\[
V_u = \phi V_c \quad \text{OK}
\]

2.9.6 Check Bending, Bearing, and Shear in Sill Plate

**Bending:** Refer to Section 2.9.2 for the sill plate’s allowable bending stress about its weak axis. Note that the allowable bending stress is modified by the flat use factor, the wind load duration factor, and the size factor per AF&PA’s  *National Design Specification for Wood Construction*. Also note that the design loads are not factored since AF&PA’s  *National Design Specification for Wood Construction* is an allowable stress design method in which the allowable stresses already include a safety factor.

\[
F_b = 1,987 \text{ psi} \quad \text{[13.7 MPa]}
\]

\[
M = \frac{(19 \text{ psf} - 12 \text{ psf})(\frac{32.7 \text{ ft}}{2})(4 \text{ ft bolt spacing})^2}{8} = 229 \text{ ft - lb} = 2,748 \text{ in - lb}
\]

\[
S_{xy} = 2.719 \text{ in}^3 \quad \text{[44.6 cm}^3\text{]}
\]

\[
f_b = \frac{2,748 \text{ in - lb}}{2.719 \text{ in}^3} = 1,011 \text{ psi}
\]

\[
f_b \leq F_b' \quad \text{OK}
\]

**Bearing:** Determine if the 1-1/4 inch (32 mm) diameter washers are large enough to withstand the uplift forces acting on the sill plate without failing the wood by compression at the washer. Refer to Section 2.9.2 for the sill plate’s allowable compressive stress perpendicular to the grain. Note that the allowable compressive stress is modified by the small bearing factor per AF&PA’s  *National Design Specification for Wood Construction*. Also note that the design loads are not factored since AF&PA’s  *National Design Specification for Wood Construction* is an allowable stress design method in which the allowable stresses already include a safety factor.

\[
F_{c\perp} = 813 \text{ psi} \quad \text{[5.6 MPa]}
\]

\[
T = (19 \text{ psf} - 12 \text{ psf})(\frac{32.7 \text{ ft}}{2})(4 \text{ ft bolt spacing}) = 458 \text{ lb}
\]

\[
A_{\text{washer}} = 1.25 \text{ in}^2 - 0.5 \text{ in}^2 = 1.03 \text{ in}^2
\]

\[
f_{c\perp} = \frac{458 \text{ lb}}{1.03 \text{ in}^2} = 445 \text{ psi}
\]

\[
f_{c\perp} \leq F_{c\perp} \quad \text{OK}
\]

**Shear Compression Parallel to the Grain at the Bolt Hole:** Refer to Section 2.9.2 for the sill plate’s allowable compression parallel to the grain and refer to Section 2.9.1 for unit shear in the

---

Structural Design of Insulating Concrete Form Walls in Residential Construction
sill plate. Note that the design loads are not factored since AF&PA’s *National Design Specification for Wood Construction* is an allowable stress design method in which the allowable stresses already include a safety factor.

\[
F_c' = 2,268 \text{ psi} \quad [15.6 \text{ MPa}]
\]

\[
f_c = \frac{(49 \text{ plf})(4 \text{ ft bolt spacing})}{(15 \text{ in})(0.5 \text{ in})} = 261 \text{ psi} \quad [1.8 \text{ MPa}]
\]

\[
f_c' \leq F_c' \quad \text{OK}
\]

**Shear Compression Perpendicular to the Grain at the Bolt Hole:** Refer to Section 2.9.2 for the sill plate’s allowable compression perpendicular to the grain and refer to Section 2.9.1 for the unit shear in the sill plate. Note that the design loads are not factored since AF&PA’s *National Design Specification for Wood Construction* is an allowable stress design method in which the allowable stresses already include a safety factor.

\[
F_{c_{\perp}}' = 813 \text{ psi} \quad [5.6 \text{ MPa}]
\]

\[
f_{c_{\perp}} = \frac{(116 \text{ plf})(4 \text{ ft bolt spacing})}{(15 \text{ in})(0.5 \text{ in})} = 619 \text{ psi} \quad [4.3 \text{ MPa}]
\]

\[
f_{c_{\perp}}' \leq F_{c_{\perp}}' \quad \text{OK}
\]

Determine if the actual bolt spacings, as controlled by wood compression parallel to the grain, are greater than those required by AF&PA’s *National Design Specification for Wood Construction*.

<table>
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<th>Distance</th>
<th>Required</th>
<th>Actual</th>
</tr>
</thead>
<tbody>
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<td>End distance</td>
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</tr>
<tr>
<td></td>
<td>[305 mm]</td>
<td></td>
</tr>
<tr>
<td>Edge distance</td>
<td>4(0.5 in) = 2 in ≤ \frac{7.25 in}{2} = 3.6 in</td>
<td>OK</td>
</tr>
<tr>
<td></td>
<td>[91 mm]</td>
<td></td>
</tr>
<tr>
<td>Spacing between bolts</td>
<td>3(0.5 in) = 1.5 in ≤ 48 in</td>
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</tr>
<tr>
<td></td>
<td>[38 mm &lt; 1.2 m]</td>
<td></td>
</tr>
</tbody>
</table>

**2.9.7 Check Bearing Strength of ICF Wall**

The bearing strength of the ICF wall is determined using the nominal loads shown in Figure 2-5 and calculated in Section 2.2. Note that the roof truss spacing is 2 feet (0.6 m) on center.

\[
B_{c,\text{actual}} = 1.4(2 \text{ ft})(196 \text{ plf}) + 1.7(2 \text{ ft})(163 \text{ plf} + 572 \text{ plf}) = 3,048 \text{ lb}
\]

\[
B_{c,\text{allowable}} = 0.7(0.85)(3,000 \text{ psi})(4.5 \text{ in})(7.25 \text{ in}) = 58,236 \text{ lb}
\]

\[
B_{c,\text{actual}} \leq B_{c,\text{allowable}} \quad \text{OK}
\]
2.10 DESIGN ROOF CONNECTION: STRAP

The following example follows the design procedure described in Section 1.7 for strap connections. Design the strap connection for the south or north wall and refer to Figure 2-17 for the building plan and sections.

2.10.1 Determine Design Loads

Note that the total tensile load on the strap is the difference between the factored uplift load and the factored dead load on the roof structure.

\[ T_d = \left( (1.3)19 \text{ psf} - (0.9)12 \text{ psf} \right) \left( \frac{32.7 \text{ ft}}{2} \right) (2 \text{ ft strap spacing}) = 455 \text{ lb} \quad [2 \text{ kN}] \]

2.10.2 Assume Strap Connector Size

Per a strap connector manufacturer’s literature, try an 18 gauge coiled strap tie cut to a 20 inch (508 mm) length. Embedment length is 4 inches (102 mm) and fastened to the truss with eight 10d fasteners. Refer to Section 2.9.1 for previously calculated shear loads.

- \( T_{\text{actual}} = 455 \text{ lb} \)
- \( T_{\text{strap}} = 1,160 \text{ lb rated capacity from strap manufacturer} \)
- \( T_{\text{actual}} \leq T_{\text{strap}} \) OK
- \( V_{\text{actual}} = (116 \text{ psf})(2 \text{ ft strap spacing}) = 232 \text{ lb} \) \[ 1 \text{ kN} \]
- \( V_{\text{strap}} = 300 \text{ lb rated capacity from strap manufacturer} \)
- \( V_{\text{actual}} \leq V_{\text{strap}} \) OK

2.10.3 Check Tension in Concrete (Anchorage Capacity)

Note that the tension in the concrete, \( V_w \), is equivalent to the total uplift load on the strap.

- \( V_{\text{actual}} = 455 \text{ lb} \) \[ 2 \text{ kN} \]
- \( A_v = \text{minimum of} \ \left\{ \frac{\pi (4 \text{ in})^2}{\pi (5 \text{ in})} = 50.3 \text{ in}^2 \right\} = 50.3 \text{ in}^2 \) \[ 325 \text{ cm}^2 \]
- \( \phi V_c = 0.85(4)(50.3 \text{ in}^2)\sqrt{3000 \text{ psi}} = 9,367 \text{ lb} \) \[ 41.6 \text{ kN} \]
- \( V_{\text{actual}} \leq \phi V_c \) OK

2.10.4 Check Bearing Strength of ICF Wall

The bearing strength of the ICF wall is determined using the nominal loads shown in Figure 2-5 and calculated in Section 2.2. Note that the roof truss spacing is 2 feet (0.6 m) on center.
ICF Design Example: Roof Connections

\[ B_{c,\text{actual}} = 1.4(2 \, \text{ft})(196 \, \text{plf}) + 1.7(2 \, \text{ft})(163 \, \text{plf} + 572 \, \text{plf}) = 3,048 \, \text{lb} \]

\[ B_{c,\text{allowable}} = 0.7(0.85)(3,000 \, \text{psi})(1.5 \, \text{in})(5 \, \text{in}) = 13,388 \, \text{lb} \]

\[ B_{c,\text{actual}} \leq B_{c,\text{allowable}} \quad \text{OK} \]

\[ 13.6 \, \text{kN} \]

\[ 59.6 \, \text{kN} \]
2.11 DESIGN FLOOR CONNECTION: LEDGER

The following example follows the design procedure described in Section 1.8 for ledger connections. Design the ledger connection for the south wall where the highest loads occur. The designer should recognize the importance of the ledger connection because the floor live and dead loads are transferred to the wall through a series of connections which is not typical of platform wood-framed construction.

2.11.1 Determine Design Loads

Refer to Figure 2-5 and Section 2.2 for the nominal loads acting on the ledger connection. Note that the floor joist spacing is 16 inches (406 mm) on center. The designer should be aware that a waffle-grid ICF wall has a vertical core spacing of 12 inches on center. The bolts for a ledger connection must coincide with the centerline of the vertical cores. To eliminate construction conflicts due to the different joist and vertical core spacing, the floor joist spacing is increased to 24 inches on center in this example. Assume that the floor joists are sized properly for the additional loads based on the new 24-inch joist spacing.

\[
V_{DL} = 93 \text{ plf} \quad \text{[1.4 kN/m]}
\]
\[
V_{LL} = 278 \text{ plf} \quad \text{[4.1 kN/m]}
\]
\[
V = 93 \text{ plf} + 278 \text{ plf} = 371 \text{ plf} \quad \text{[5.4 kN/m]}
\]
\[
V_u = 1.4(93 \text{ plf}) + 1.7(278 \text{ plf}) = 603 \text{ plf} \quad \text{[8.8 kN/m]}
\]

2.11.2 Assume Connection Spacing and Size

Try a 5/8-inch (16 mm) diameter ASTM A36 anchor bolt \((A_b=0.306 \text{ in}^2 (197 \text{ mm}^2))\), 6 inches (152 mm) long and spaced 12 inches (305 mm) on center. Assume the washer is 1-3/8 inches (35 mm) in diameter. According to Part 4, “Connections”, in ASD/AISC’s Manual of Steel Construction, the allowable tensile strength of the bolt, \(F_t\), the ultimate tensile strength of the bolt, \(F_u\), and the allowable shear strength of the bolt, \(F_v\), are as follows:

\[
F_u = 58,000 \text{ psi} \quad \text{[400 MPa]}
\]
\[
F_t = 19,100 \text{ psi} \quad \text{[132 MPa]}
\]
\[
F_t = \sqrt{44^2 - 4.39 f_v^2} \quad \text{for combined shear and tension}
\]
\[
F_t = \sqrt{57.2^2 - 4.39 f_v^2} \quad \text{for combined shear and tension due to wind or seismic}
\]
\[
F_v = (0.17)(58,000 \text{ psi}) = 9,860 \text{ psi for threads included in shear plane} \quad \text{[68 MPa]}
\]
\[
F_v = (0.22)(58,000 \text{ psi}) = 12,760 \text{ psi for threads excluded from shear plane} \quad \text{[88 MPa]}
\]

The following allowable bolt design values for single shear are taken from Table 8.2E in AF&PA’s National Design Specification for Wood Construction assuming a bending strength, \(F_y\), of 36,000 psi (248 MPa).
ICF Design Example: Floor Connections

$$Z_{\text{allowable}} = \frac{520 \text{ lb}}{\text{bolt}} \left( \frac{12 \text{ in bolt spacing}}{12 \text{ in} / \text{ft}} \right) = 520 \text{ lb}$$

$$Z'_{\text{allowable}} = (1.0)520 = 520 \text{ lb \ modified \ for \ live \ load \ duration}$$

[2.3 kN]

Assume the ledger board is a No. 1 grade, Douglas Fir-Larch 2x12 with the following properties as taken from Table 4A in AF&PA’s Design Values for Wood Construction.

- $$F_b = 1,200 \text{ psi}$$ \hspace{1cm} \[8.3 \text{ MPa}\]
- $$F_b' = 12(1.6)(1.0)(1,200 \text{ psi}) = 2,304 \text{ psi \ modified \ for \ flat \ use, \ wind \ load \ duration, \ and \ size}$$ \[10.2 \text{ MPa}\]
- $$F_p' = (1.0)(1.0)(1,200 \text{ psi}) = 1,200 \text{ psi \ modified \ for \ live \ load \ duration \ and \ size}$$ \[8.3 \text{ MPa}\]
- $$F_{ct} = 625 \text{ psi}$$ \[4.3 \text{ MPa}\]
- $$F_{cl} = \left( \frac{1375 + 0.375}{1.375} \right)(625 \text{ psi}) = 813 \text{ psi \ modified \ for \ small \ bearing \ area}$$ \[5.6 \text{ MPa}\]
- $$F_v = 95 \text{ psi}$$ \[655 \text{ kPa}\]
- $$F_v' = (2)95 \text{ psi} = 190 \text{ psi \ modified \ for \ no \ splits}$$ \[1.3 \text{ MPa}\]
- $$S_{xy} = 4.219 \text{ in}^3$$ \[69 \text{ cm}^3\]
- $$S_{xc} = 31.64 \text{ in}^3$$ \[518.5 \text{ cm}^3\]
- $$h = 1.5 \text{ in}$$\hspace{1cm} \[38 \text{ mm}\]
- $$b = 11.25 \text{ in}$$\hspace{1cm} \[286 \text{ mm}\]

2.11.3 Check Shear-Friction in Concrete

Refer to Section 2.11.1 for the factored shear force at the section. Assume an 8-inch (203 mm) diameter hole is cut into the form around the bolt to allow installation.

$$V_u = \frac{603 \text{ pbf (12 in bolt spacing)}}{0.6} \left( \frac{12 \text{ in} / \text{ft}}{12 \text{ in} / \text{ft}} \right) = 1,005 \text{ lb}$$ \[4.5 \text{ kN}\]

$$A_c = \pi (4 \text{ in})^2 = 50.3 \text{ in}^2$$ \[325 \text{ cm}^2\]

$$V_n = (0.306 \text{ in}^2)(36,000 \text{ psi})(0.6) = 6,610 \text{ lb} \quad \leftarrow \text{GOVERNS}$$ \[29.4 \text{ kN}\]

$$V_{n,max} = 0.2(3,000 \text{ psi})(50.3 \text{ in}^2) = 30,180 \text{ lb}$$ \[134.2 \text{ kN}\]

$$\phi V_n = 0.85(6,610 \text{ lb}) = 5,619 \text{ lb}$$ \[25 \text{ kN}\]

$$V_u \leq \phi V_n \quad \text{OK}$$

2.11.4 Check Tension in Concrete (Anchorage Capacity)

Assume an 8-inch (203 mm) diameter hole is cut into the form around the bolt to allow installation. Assume the coefficient of friction is 0.6. Note that the tension in the concrete, $$V_{tu}$$, is equivalent to the total shear load on the ledger board as calculated in Section 2.11.1 divided by the coefficient of friction plus the tension resulting from wind suction pressure. Refer to Figure
Design Example: Floor Connections

2-18 for wind suction pressure acting on the connection. Note that the tension load is factored for ACI load case, \( U = 0.75(1.4 \text{ Dead} + 1.7 \text{ Live} + 1.7 \text{ Wind}) \).

\[
V_v = 0.75 \left( \frac{603 \text{ plf}}{0.6} \right) + (1.7)(210 \text{ psf}) \left( \frac{9 \text{ ft}}{2} + \frac{8.5 \text{ ft}}{2} \right) \left( \frac{12 \text{ in bolt spacing}}{12\text{in} / \text{ft}} \right) = 988 \text{ lb} \quad [4.4 \text{ kN}]
\]

\[
A_v = \text{minimum of} \left\{ \frac{\pi(6\text{in})^2}{\pi(8\text{in})^2} = 113\text{in}^2 \right\} = 113\text{in}^2 \quad [325 \text{ cm}^2]
\]

\[
\phi V_c = 0.85(4)(113\text{in}^2)\sqrt{3,000 \text{ psi}} = 21,044\text{lb} \quad [93.6 \text{ kN}]
\]

\[
V_v \leq \phi V_c \quad \text{OK}
\]

**Figure 2-18 Wind Suction Pressure on Ledger Board Connection**

2.11.5 Check Tension in Bolt Due to Shear-Friction and Wind Suction Pressure

Refer to Section 2.11.2 for the allowable tensile strength, \( F_t \), of the bolt and Section 2.11.4 for the factored tensile load on the bolt. Note that the factored tensile load on the bolt is equivalent to \( V_v \) in Section 2.11.4.

\[
F_t = 19,100 \text{ psi} \quad [303.4 \text{ MPa}]
\]

\[
T = 0.75 \left( \frac{603 \text{ plf}}{0.6} \right) + (1.7)(210 \text{ psf}) \left( \frac{9 \text{ ft}}{2} + \frac{8.5 \text{ ft}}{2} \right) \left( \frac{12 \text{ in bolt spacing}}{12\text{in} / \text{ft}} \right) = 988 \text{ lb} \quad [4.4 \text{ kN}]
\]

\[
f_t = \frac{988 \text{ lb}}{0.306 \text{ in}^2} = 3,229 \text{ psi} \quad [22.3 \text{ MPa}]
\]
2.11.6 Check Shear in Bolt

Refer to Section 2.11.2 for the allowable bolt shear and refer to Section 2.11.1 for nominal shear resulting from floor loads.

\[
Z_{\text{allowable}} = 520 \text{ lb} \quad [2.3 \text{ kN}]
\]

\[
Z_{\text{actual}} = (371 \text{ plf})(\frac{12 \text{ in bolt spacing}}{12 \text{ in/ft}}) = 371 \text{ lb} \quad [1.7 \text{ kN}]
\]

\[
Z_{\text{actual}} \leq Z_{\text{allowable}} \quad \text{OK}
\]

2.11.7 Check Bending, Bearing, and Shear in Ledger Board

**Bending About Strong Axis:** Refer to Section 2.11.2 for the allowable bending stress for the ledger board. Note that the design loads are not factored since AF&PA’s *National Design Specification for Wood Construction* is an allowable stress design method in which the allowable stresses already include a safety factor.

\[
F_b = 1,200 \text{ psi} \quad [8.3 \text{ MPa}]
\]

\[
M = \frac{P_l}{4} = \frac{(371 \text{ plf})(2 \text{ ft joist spacing})(12 \text{ in bolt spacing})}{4} = 186 \text{ ft-lb} = 2,232 \text{ in-lb} \quad [252 \text{ N-m}]
\]

\[
S_{xx} = 31.64 \text{ in}^3 \quad [518.5 \text{ cm}^3]
\]

\[
f_b = \frac{2,232 \text{ in-lb}}{31.64 \text{ in}^3} = 70.5 \text{ psi} \quad [486 \text{ kPa}]
\]

\[
f_b \leq F_b \quad \text{OK}
\]

**Bending About Weak Axis Due to Wind Suction Pressure:** Refer to Section 2.11.2 for the allowable bending stress for the ledger board. Note that the allowable bending stress is modified by the flat use, wind load duration, and size factors per AF&PA’s *National Design Specification for Wood Construction*. Also note that the design loads are not factored since AF&PA’s *National Design Specification for Wood Construction* is an allowable stress design method in which the allowable stresses already include a safety factor.

\[
F'_b = 2,304 \text{ psi} \quad [8.3 \text{ MPa}]
\]

\[
w = (21.0 \text{ psf})(\frac{9 \text{ ft}}{2} + \frac{8.5 \text{ ft}}{2}) = 184 \text{ plf} \quad [2.7 \text{ kN/m}]
\]

\[
M = \frac{(184 \text{ plf})(12 \text{ in bolt spacing})^2}{8} = 23 \text{ ft-lb} = 276 \text{ in-lb} \quad [3.2 \text{ kN-m}]
\]

\[
S_{yy} = 4.219 \text{ in}^3 \quad [69.1 \text{ cm}^3]
\]
ICF Design Example: Floor Connections

$$f_b = \frac{276 \text{ in}-lb}{4.219 \text{ in}^3} = 65 \text{ psi}$$

$$f_b \leq f_b^* \text{ OK}$$

**Bearing At Washer Due to Weak Axis Bending:** Determine if the 1-3/8 inch diameter washers are large enough to withstand the tensile forces acting on the ledger board without failing by compression. Refer to Section 2.11.2 for the ledger board’s allowable compressive stress perpendicular to the grain. Note that the allowable compressive stress is modified by the small bearing factor per AF&PA’s National Design Specifications for Wood Construction. Also note that the design loads are not factored since AF&PA’s National Design Specifications for Wood Construction is an allowable stress design method in which the allowable stresses already include a safety factor. A load acting perpendicular to the grain resulting from wind suction pressure is assumed.

$$F_{cl} = 813 \text{ psi}$$

$$T = (210 \text{ psi}) \left( \frac{9 \text{ ft}}{2} + \frac{8.5 \text{ ft}}{2} \right) \left( \frac{12 \text{ in bolt spacing}}{12 \text{ in/ft}} \right) = 184 \text{ lb}$$

$$A_{washer} = \pi \left( \frac{1.375 \text{ in}}{2} \right)^2 - \pi \left( \frac{0.625 \text{ in}}{2} \right)^2 = 1.18 \text{ in}^2$$

$$f_{cl} = \frac{184 \text{ lb}}{1.18 \text{ in}^2} = 156 \text{ psi}$$

$$f_{cl} \leq F_{cl} \text{ OK}$$

In areas of high wind and high seismic risk, it is imperative that the floor diaphragm be properly attached to prevent the floor diaphragm and framing from pulling away from the ledger and wall. The designer may either use a strap connector at the bottom of the floor joist to properly anchor the floor to the wall or the designer may require a closer nail spacing along the exterior perimeter of the subfloor sheathing into the ledger. (Refer to Figure 2-18). Since a strap connector may interfere with the ceiling finish attachment, the following calculations are based on the closer perimeter nail spacing alternative. Assume 8d nails are used to attach the 3/4-inch subfloor sheathing to the ledger. Refer to Table 12.3A in AF&PA’s National Design Specifications for Wood Construction for nail design values and UBC-97 for the seismic coefficient for seismic zone 1.

$$Z = 72 \text{ lb / nail}$$

$$Z' = (1.6)(72 \text{ lb / nail}) = 115 \text{ lb / nail modified for wind/seismic load duration}$$

$$V = (210 \text{ psi}) \left( \frac{9 \text{ ft}}{2} + \frac{8.5 \text{ ft}}{2} \right) = 184 \text{ plf} \quad \text{for wind conditions}$$

$$W_{wall} = (55 \text{ psi}) \left( \frac{9 \text{ ft}}{2} + \frac{8.5 \text{ ft}}{2} \right) = 481 \text{ plf}$$

$$V = \frac{3C_a}{R} W = \frac{3(0.07)}{3.5} (481 \text{ plf}) = 29 \text{ plf} \quad \text{for seismic conditions}$$

$$\text{GOVERNS}$$

$$Z = 72 \text{ lb / nail} \quad [320 \text{ N/nail}]$$

$$Z' = (1.6)(72 \text{ lb / nail}) = 115 \text{ lb / nail modified for wind/seismic load duration} \quad [512 \text{ N/nail}]$$

$$V = (210 \text{ psi}) \left( \frac{9 \text{ ft}}{2} + \frac{8.5 \text{ ft}}{2} \right) = 184 \text{ plf} \quad \text{for wind conditions} \quad [2.7 \text{ kN/m}]$$

$$W_{wall} = (55 \text{ psi}) \left( \frac{9 \text{ ft}}{2} + \frac{8.5 \text{ ft}}{2} \right) = 481 \text{ plf} \quad [7 \text{ kN/m}]$$

$$V = \frac{3C_a}{R} W = \frac{3(0.07)}{3.5} (481 \text{ plf}) = 29 \text{ plf} \quad \text{for seismic conditions} \quad [423 \text{ N/m}]$$
ICF Design Example: Floor Connections

\[
\text{spacing} = \left( \frac{184 \text{ plf}}{115 \text{ lb/}} \right) = 1.6 \text{ ft} \approx 2 \text{ ft} \quad [610 \text{ mm}]
\]

\[\therefore \text{ use one 8d nail @ 2 \text{ ft} (610 mm) on center along exterior perimeter edge of subfloor sheathing}\]

Shear Compression at Bolt Hole: Already accounted for when using the bolt design value, \( Z \), tables in AF&PA’s *National Design Specification for Wood Construction*; refer to Section 2.11.6.

Shear Parallel to the Grain: Refer to Section 2.11.2 for the ledger board’s allowable shear parallel to the grain. Note that the design loads are not factored since the AF&PA’s *National Design Specification for Wood Construction* is an allowable stress design method in which the allowable stresses already include a safety factor.

\[
\begin{align*}
F_v' &= 190 \text{ psi} \\
\frac{f_v}{2bd} &= \frac{3(371 \text{ plf})(2 \text{ ft joist spacing})}{2(15 \text{ in})(11.25 \text{ in})} = 66 \text{ psi} \\
f_v &\leq F_v' \quad \text{OK}
\end{align*}
\]

Determine if actual bolt spacings are greater than those required by AF&PA’s *National Design Specification for Wood Construction*.

<p>| | | |</p>
<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>End distance</strong></td>
<td>12 in</td>
<td>OK</td>
</tr>
<tr>
<td><strong>Edge distance</strong></td>
<td>4(0.625 in) = 2.5 in \leq \frac{11.25 \text{ in}}{2} = 5.63 in</td>
<td>OK</td>
</tr>
<tr>
<td><strong>Spacing between bolts</strong></td>
<td>3(0.625 in) = 1.9 in \leq 12 in</td>
<td>OK</td>
</tr>
</tbody>
</table>

Although one 5/8-inch bolt at 12 inches on center is adequate, a better connection would be to use two 5/8-inch bolts at 24 inches on center. Not only does this alleviate spacing conflicts during construction between the floor joist spacing and the bolt spacing in this particular example, but it adds another level of safety against ledger board failure due to poor quality in lumber, poor installation, or inadequate design assumptions. Using two bolts instead of one bolt, moves the bolts out of the neutral axis of the ledger board providing greater resistance to bending about the weak axis and shear parallel to the grain. For ledger boards which do not allow for two bolts to be installed as shown in Figure 2-19, the bolts may be staggered to produce a similar result.

Vertical Spacing between bolts \( 3(0.625 \text{ in}) = 1.9 \text{ in} \leq 6.25 \text{ in} \quad \text{OK} \quad [48 \text{ mm} \times 159 \text{ mm}] \)

*Please note that this design example (Section 2.11) should be repeated using the new bolt spacing to verify that the bolt size and new spacing is adequate for the design loads; however, the calculations for the new bolt spacing are not shown here.*
Figure 2-19  Ledger Board Bolt Placement
2.12 DESIGN FLOOR CONNECTION: POCKET

The following example follows the design procedure described in Section 1.10 for pocket connections. Design the pocket connection for the south wall where the highest loads occur. Assume that the floor is wood-framed; therefore, blocking between joists at the perimeter of the floor is required to ensure proper floor diaphragm strength to resist and distribute lateral loads resulting from wind or seismic forces.

2.12.1 Determine Design Loads

Refer to Figure 2-5 and Section 2.2 for the nominal loads acting on the pocket connection. Note that the floor joist spacing is 16 inches (406 mm) on center. The designer should be aware that a waffle-grid ICF wall has a vertical core spacing of 12 inches on center. The bolts for a ledger connection must coincide with the centerline of the vertical cores. To eliminate construction conflicts due to the different joist and vertical core spacing, the floor joist spacing is increased to 24 inches on center in this example. Assume that the floor joists are sized properly for the additional loads based on the new 24-inch spacing. Assuming wind suction on the walls, the following tensile load occurs at the pocket connection location.

\[
\begin{align*}
V_{DL} &= 93 \text{ plf} & [1.6 \text{kN/m}] \\
V_{LL} &= 278 \text{ plf} & [4 \text{kN/m}] \\
V &= 93 \text{ plf} + 278 \text{ plf} = 371 \text{ plf} & [15.4 \text{kN/m}] \\
V_u &= 1.4(93 \text{ plf}) + 1.7(278 \text{ plf}) = 603 \text{ plf} & [8.8 \text{kN/m}] \\
T_{\text{actual}} &= 21.0 \text{plf} \left( \frac{8.5 \text{ ft}}{2} + \frac{9 \text{ ft}}{2} \right) \left( \frac{24 \text{ in strap spacing}}{12 \text{ in/ft}} \right) = 368 \text{ lb} & [1.6 \text{kN}] \\
\end{align*}
\]

2.12.2 Assume Strap Connector Size

Per a strap connector manufacturer’s literature, try an 18 gauge coiled strap, 12 inches (305 mm) long cut to length. Embedment length is 4 inches (102 mm) and fastened to the joist with four 10d fasteners when the concrete compressive strength is 3,000 psi (21 MPa).

\[
\begin{align*}
T_{\text{actual}} &= 368 \text{ lb} & [1.6 \text{kN}] \\
T_{\text{strap}} &= 580 \text{ lb \ rated capacity of the strap} & [2.6 \text{kN}] \\
T_{\text{actual}} &\leq T_{\text{strap}} & \text{OK} \\
\end{align*}
\]

Assume the floor joist is a No. 1 grade, Douglas Fir-Larch 2x12 with the following properties as taken from AF&PA’s *Design Values for Wood Construction*.

\[
\begin{align*}
F_{c1} &= 625 \text{ psi} & [4.3 \text{ MPa}] \\
F_{c1} &= \left( \frac{4 + 0.375}{4} \right)(625 \text{ psi}) = 688 \text{ psi \ modified for small bearing area} & [4.7 \text{ MPa}] \\
F_v &= 95 \text{ psi} & [655 \text{ MPa}] \\
F_v &= (2)(95 \text{ psi}) = 190 \text{ psi \ modified for no splits} & [1.3 \text{ MPa}] \\
\end{align*}
\]
2.12.3 Check Bearing Strength of ICF Wall

The bearing strength of the ICF wall is determined using the nominal loads in Section 2.12.1. Note that the joist spacing is 24 inches (610 mm) on center. Assume the joists have 4 inches (102 mm) minimum bearing at each support.

\[ B_{c,\text{actual}} = 1.4 \left( \frac{24 \text{ in strap spacing}}{12 \text{ in/ft}} \right) (93 \text{ plf}) + 1.7 \left( \frac{24 \text{ in strap spacing}}{12 \text{ in/ft}} \right) (278 \text{ plf}) = 1,206 \text{ lb} \]  
\[ B_{c,\text{allowable}} = 0.7 (0.85) (3,000 \text{ psi}) (1.5 \text{ in})(4 \text{ in}) = 10,710 \text{ lb} \]
\[ B_{c,\text{actual}} \leq B_{c,\text{allowable}} \text{ OK} \]

2.12.4 Check Bearing and Shear in Floor Joist

**Bearing:** Determine if the joists have adequate bearing area to resist floor loads. Refer to Section 2.12.2 for the allowable compressive stress perpendicular to the grain and to Section 2.12.1 for the floor loads. Note that the allowable compressive stress is modified by the small bearing factor per AF&PA’s *National Design Specifications for Wood Construction*. Also note that the design loads are not factored since AF&PA’s *National Design Specifications for Wood Construction* is an allowable stress design method in which the allowable stresses already include a safety factor.

\[ F_{cl} = 688 \text{ psi} \]  
\[ v = 371 \text{ plf} \left( \frac{24 \text{ in joist spacing}}{12 \text{ in/ft}} \right) = 742 \text{ lb} \]  
\[ A_{\text{bearing}} = (15 \text{ in})(4 \text{ in}) = 6 \text{ in}^2 \]  
\[ f_{cl} = \frac{742 \text{ lb}}{6 \text{ in}^2} = 124 \text{ psi} \]  
\[ f_{cl} \leq F_{cl} \text{ OK} \]

**Shear Parallel to Grain:** Refer to Section 2.12.2 for the allowable parallel shear for the joist. Note that the design loads are not factored since AF&PA’s *National Design Specification for Wood Construction* is an allowable stress design method in which the allowable stresses already include a safety factor.

\[ F_{sv} = 190 \text{ psi} \]  
\[ f_v = \frac{3v}{2bd} = \frac{3 (371 \text{ plf}) \left( \frac{24 \text{ in joist spacing}}{12 \text{ in/ft}} \right)}{2(1.5 \text{ in})(11.25 \text{ in})} = 66 \text{ psi} \]  
\[ f_v \leq F_{sv} \text{ OK} \]
APPENDIX A

BEAM DIAGRAMS WITH TYPICAL LOADING CONDITIONS
**Typical Loading Conditions**

\[
\begin{align*}
V_{\text{max}} &= \frac{ql}{2} \\
V_x &= q \left( \frac{l}{2} - x \right) \\
M_{\text{max}} &= \frac{ql^2}{8} \\
M_x &= \frac{q}{2} (l - x) \\
\Delta_{\text{max, center}} &= \frac{S q l^4}{384E I} \\
\Delta_x &= -\frac{q x}{24E I} \left( b^3 - 2b x^2 + x^3 \right)
\end{align*}
\]

**Figure A-1 Uniform Load, Simple Span**

**Figure A-2 Eccentric Point Loads, Simple Span**

\[
\begin{align*}
V_{\text{max}} &= V_x = 0 \\
M_{\text{max}} &= |M_2| - |M_1| \quad \text{where} \quad |M_2| > |M_1| \\
M_{\text{max}} &= |M_1| - |M_2| \quad \text{where} \quad |M_1| > |M_2| \\
M_2 &= P_2 x_2 \\
M_1 &= P_1 x_1 \\
M_x &= \frac{M_{\text{max}}}{l} \left( \frac{x}{l} \right)
\end{align*}
\]
Typical Loading Conditions

\[
V_{\text{top}} = \frac{qL^3}{6L}
\]

\[
V_{\text{bottom}} = \frac{qL^2}{2} - \frac{qL^3}{6L}
\]

\[
V_x = qLx - \frac{q}{2} - \frac{qL^3}{6L}
\]

\[
V_x = V_{\text{top}} \text{ where } x \geq L
\]

\[
M_x = \frac{qL^2}{2} + \frac{q}{6} \left( x + V_{\text{bottom}}x \right) \text{ where } x < L
\]

\[
M_x = \frac{qL^3}{6L} \left( L - x \right) \text{ where } x \geq L
\]

\[
x_{M_{\max}} = \frac{qL^2}{2} \left( 1 \right) \text{ where } x < L
\]

\[
x_{M_{\max}} = \frac{qL}{3} \left( 2 - x \right) \text{ where } x < L
\]

\[
x_{M_{\max}} = \frac{-qL}{3} \left( 2 - x \right) \text{ where } x < L
\]

Figure A-3  Partial Triangular Load, Simple Span

Figure A-4  Load Uniformly Increasing to Center, Simple Span
Typical Loading Conditions

\[ V_{\text{max}} = \frac{ql}{2} \]
\[ V_x = q\left(\frac{l}{2} - x\right) \]
\[ M_{\text{max}} = \frac{ql^2}{12} \]
\[ M_1 = \frac{ql^2}{24} \]
\[ M_x = \frac{q}{12}\left(6lx - l^2 - 6x^2\right) \]
\[ \Delta_{\text{max center}} = \frac{ql^4}{384EI} \]
\[ \Delta_x = \frac{qx^2}{24EI}\left(l - x\right)^2 \]

Figure A-5 Uniform Load, Fixed-End Simple Span
APPENDIX B

PROPERTIES OF GEOMETRIC SECTIONS
Properties of Geometric Sections

Figure B-1 Rectangle, Axis of Moments through Center

\[ A = bh \]
\[ c = \frac{h}{2} \]
\[ I = \frac{1}{12} bh^3 \]
\[ S = \frac{1}{6} bh^2 \]
\[ r = \frac{h}{\sqrt{12}} \]

Figure B-2 Rectangle, Axis of Moments on Base

\[ A = bh \]
\[ c = h \]
\[ I = \frac{1}{3} bh^3 \]
\[ S = \frac{1}{6} bh^2 \]
\[ r = \frac{h}{\sqrt{3}} \]

Figure B-3 Circle, Axis of Moments through Center

\[ A = \frac{1}{4} \pi d^2 = \pi R^2 \]
\[ c = \frac{d}{2} = R \]
\[ I = \frac{\pi d^4}{64} = \frac{\pi R^4}{4} \]
\[ S = \frac{\pi d^3}{32} = \frac{\pi R^3}{4} \]
\[ r = \frac{d}{4} = \frac{R}{2} \]
Properties of Geometric Sections

**Figure B-4** Ellipse, Axis of Moments through Center

\[
A = \frac{\pi bh}{4}
\]

\[
c = \frac{h}{2}
\]

\[
I = \frac{\pi bh^3}{64}
\]

\[
S = \frac{\pi bh^2}{32}
\]

\[
r = \frac{h}{4}
\]

**Figure B-5** Rounded Rectangle, Axis of Moments through Center

\[
A = bh - 4a^2 \left(1 - \frac{\pi}{4}\right)
\]

\[
c = \frac{h}{2}
\]

\[
I = \frac{bh^3}{12} - 4 \left[a^4 \left(\frac{1}{3} - \frac{\pi}{16} - \frac{1}{36(1 - \frac{\pi}{4})}\right) + a^2 \left(1 - \frac{\pi}{4}\right) \left(c - a + \frac{a}{6(1 - \frac{\pi}{4})}\right)^2\right]
\]

\[
S = \frac{I}{c}
\]

\[
r = \frac{I}{A}
\]
The moment magnifier method is an approximation method used to account for slenderness in a structural reinforced concrete wall. ACI 10.10.2 allows the moment magnifier method to be used in lieu of second-order analysis for walls with a slenderness ratio less than or equal to 100. The tables in this appendix contain the moment magnifier for non-sway frames constructed of ICF flat, waffle-, or screen-grid walls with slenderness ratios less than or equal to 100. To generate moment magnifier tables for use with as many ICF manufacturers as possible, the tables are based on an equivalent rectangular section with the following dimensions:

### Non-Sway Moment Magnifier Tables

<table>
<thead>
<tr>
<th>ICF Wall Type</th>
<th>Nominal Thickness (inch)</th>
<th>Minimum Equivalent Thickness (h) (inch)</th>
<th>Minimum Equivalent Width (b) (inch)</th>
<th>Vertical Core Spacing (inch)</th>
</tr>
</thead>
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<td>5.5</td>
<td>12.0</td>
<td>N.A.</td>
</tr>
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<td>9.5</td>
<td>12.0</td>
<td>N.A.</td>
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<td>Waffle-Grid</td>
<td>6</td>
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<td>12.0</td>
<td>12</td>
</tr>
<tr>
<td></td>
<td>8</td>
<td>7.0</td>
<td>12.0</td>
<td>12</td>
</tr>
<tr>
<td>Screen-Grid</td>
<td>6</td>
<td>5.5</td>
<td>12.0</td>
<td>12</td>
</tr>
</tbody>
</table>

Figure C-1 Dimensions used for Moment Magnifier Tables
The moment magnifier tables were also generated using the equations contained in this appendix and are based on the following assumptions:

\[ \begin{align*}
M_1 &= 0 \\
C_m &= 1.0 \\
w_c &= 150 \\
\rho &= 0.0012
\end{align*} \]

where:

| \( \rho \) | Ratio of vertical reinforcement area to gross concrete area | dimensionless |
| \( C_m \) | Factor relating moment diagram to equivalent uniform moment diagram | dimensionless |
| \( M_l \) | Smaller factored end moment | in-lb |
| \( w_c \) | Weight of concrete | pcf |

The reinforcement ratio was determined to have a minor impact on the moment magnifier values; therefore, a conservative value of \( \rho \) is used to generate the moment magnifier tables. The moment magnifiers listed in the following tables are also limited to a maximum value of 4.0 to ensure stability. Interpolation between the wall height, \( e \), \( \beta_d \), and the total factored axial load is permissible when using the tables in this appendix.

As a result of generating moment diagrams representative of current ICF products, some design efficiency has been sacrificed. In addition, moment magnifier tables have not been generated for ICF wall types constructed in sway frames since most residential structures meet the non-sway criteria. Designers may either use the moment magnifier tables found in this appendix or calculate the moment magnifier using the following equations.

### C.1 Non-Sway Frames (ACI 10.12)

The equations below are taken from ACI 10.12 with one exception. The equation for \( EI \), as listed in ACI 10.12.3, is applicable for wall sections that contain a double layer of reinforcement. Since ICFs contain only one layer of reinforcement, the equation for \( EI \) listed below is used instead.\(^{10}\)

\[ \begin{align*}
M_{wt} &= \delta_{m} M_2 \\
\delta_{m} &= \frac{C_m}{1 - \left(\frac{P_u}{0.75 P_c}\right)} \geq 1.0 \\
P_c &= \frac{\pi^2 EI}{(kl_u)^2} \\
C_m &= 0.6 + 0.4 \left(\frac{M_1}{M_2}\right) \geq 0.4
\end{align*} \]

or

\( C_m = 1.0 \) for members with transverse loads between supports

\[
M_{2,\text{min}} = P_u (0.6 + 0.03h)
\]

\[
E I = \frac{0.4 E_c I_g}{\beta} \geq \frac{E_c I_g (0.5 - \frac{\rho}{h})}{\beta} \geq \frac{0.1 E_c I_g}{\beta}
\]

\[
e = \frac{M_2}{P_u}
\]

\[
\beta = 0.9 + 0.5 \beta_d^2 - 12 \rho \geq 1.0
\]

\[
\rho = \frac{A_t}{A_g}
\]

\[
\beta_d = \frac{P_{u,\text{dead}}}{P_u}
\]

\[
E_c = 57,000 \sqrt[4]{f_c^2} \text{ or } w_c^{15.33} \sqrt[4]{f_c^2}
\]

where:

- \( \beta \): As defined in the given equation
- \( \beta_d \): Ratio of dead axial load to total axial load
- \( \delta_{ns} \): Moment magnification factor for non-sway frames
- \( \rho \): Ratio of vertical reinforcement area to gross concrete area
- \( A_g \): Gross concrete area
- \( A_v \): Area of vertical steel reinforcement
- \( C_m \): Factor relating moment diagram to equivalent uniform moment diagram
- \( e \): Overall eccentricity of axial load in the wall
- \( E_c \): Modulus of elasticity of concrete per ACI 8.5.1
- \( f_{c'} \): Specified compressive strength of concrete
- \( h \): Wall thickness, Refer to Figure 1-1
- \( I_g \): Moment of inertia of gross concrete section
- \( k \): Effective length factor \( \leq 1.0 \). For most residential construction, \( k = 1.0 \) if the wall is tied to the footing, floors, and roof.
- \( l_u \): Unsupported length of compression member
- \( M_1/M_2 \): Ratio of smaller factored end moment to larger factored end moment \( \geq -0.5 \)
- \( M_1 \): Smaller factored end moment
- \( M_2 \): Larger factored end moment
- \( M_{2,\text{min}} \): Minimum value of \( M_2 \)
- \( M_{ns} \): Magnified factored moment to be used for designing compression members
- \( P_c \): Critical buckling load
- \( P_u \): Factored total axial load
- \( P_{u,\text{dead}} \): Factored axial dead load
- \( w_c \): Weight of concrete

C.2 Sway Frames (ACI 10.13)

With one exception, the following equations are taken from ACI 10.13.4.3. The equation for \( EI \), as listed in ACI 10.12.3, is applicable for wall sections that contain a double layer of reinforcement.
Since ICFs contain only one layer of reinforcement, the equation for $EI$ listed below is used instead.

$$M_s = M_{ns} + \delta_s M_2$$

$$\delta_s = 1.0 \leq \frac{1}{1 - \frac{\sum P_u}{0.75 \sum P_c}} \leq 2.5$$

$$P_c = \frac{\pi^2 EI}{(kl_u)^2}$$

$$\epsilon = \frac{M_2}{P_u}$$

$$\beta_d = \frac{P_{u,dead}}{P_u}$$

$$EI = \frac{0.4E_c I_g}{\beta} \geq \frac{E_c I_g (0.5 - \frac{\rho}{h})}{\beta} \geq \frac{0.1E_c I_g}{\beta}$$

$$\beta = 0.9 + 0.5\beta_d^2 - 12\rho \geq 1.0$$

$$\rho = \frac{A_s}{A_g}$$

$$E_c = 57,000 \sqrt{f_{c'}} \text{ or } w_c^{15/33} \sqrt{f_{c'}}$$

where:

- $\beta$: As defined in the given equation
- $\beta_d$: Ratio of dead axial load to total axial load
- $\delta_s$: Moment magnification factor for sway frames
- $\rho$: Ratio of vertical reinforcement area to gross concrete area
- $b$: Wall width, Refer to Figure 1-1
- $E_c$: Modulus of elasticity of concrete
- $EI$: Flexural stiffness of compression member
- $f_{c'}$: Specified compressive strength of concrete
- $h$: Wall thickness, Refer to Figure 1-1
- $I_g$: Moment of inertia of gross concrete section
- $l_u$: Unsupported length of compression member
- $k$: Effective length factor $\geq 1.0$
- $M_2$: Larger factored end moment
- $M_{2,min}$: Minimum value of $M_2$
- $M_S$: Magnified factored moment to be used for designing compression members for sway frames
- $M_{NS}$: Magnified factored moment to be used for designing compression members for non-sway frames
- $P_u$: Factored axial load
- $P_{u,dead}$: Factored axial dead load
- $\sum P_c$: Summation for all sway-resisting columns in a story

---

Moment Magnifiers

\[ \Sigma P_u \quad \text{Summation for all the vertical loads in a story} \quad \text{lb} \\
\]

\[ w_c \quad \text{Weight of concrete} \quad \text{pcf} \]

Check the following equation for each individual compression member in a sway frame:

\[ \frac{l_u}{r} \geq \frac{35}{\sqrt{f_c' \cdot A_g}} \]

where:

- \( A_g \): Gross area of concrete, \text{inch}^2
- \( f_c' \): Specified compressive strength of the concrete, psi
- \( l_u \): Unsupported length of compression member, inch
- \( h \): Wall thickness, Refer to Figure 1-1
- \( P_u \): Factored axial load, lb
- \( r \): Radius of gyration of cross-section per ACI 10.11.2, inch
  - \( \approx 0.3h \) for rectangular members or \( \approx 0.25d \) for circular compression members

If the above equation is true for an individual compression member, it shall be designed in accordance with “Non-Sway Frames”, with the following substitutions made to the variables per ACI 10.13:

\[ M_1 = M_{1\text{m}} + \delta_i M_{1\text{i}} \], often assumed to be 0
\[ M_2 = M_{2\text{m}} + \delta_i M_{2\text{i}} \]
## Moment Magnifiers

### Non-Sway Moment Magnifier for 4" Flat Walls

\( f_c = 3000 \text{ psi} \)

| Magnifier | 1 | 2 | 3 | 4 | 5 | 6 | 7 | 8 | 9 | 10 | 11 | 12 | 13 | 14 | 15 | 16 | 17 | 18 | 19 | 20 | 21 | 22 | 23 | 24 | 25 |
| 0.60       | 1 | 2 | 3 | 4 | 5 | 6 | 7 | 8 | 9 | 10 | 11 | 12 | 13 | 14 | 15 | 16 | 17 | 18 | 19 | 20 | 21 | 22 | 23 | 24 | 25 |
| 1.00       | 0.97 | 0.92 | 0.86 | 0.80 | 0.74 | 0.68 | 0.63 | 0.58 | 0.53 | 0.48 | 0.43 | 0.38 | 0.34 | 0.30 | 0.26 | 0.23 | 0.20 | 0.17 | 0.14 | 0.11 | 0.08 |
| 0.80       | 0.99 | 0.97 | 0.94 | 0.91 | 0.88 | 0.85 | 0.82 | 0.79 | 0.76 | 0.73 | 0.70 | 0.67 | 0.64 | 0.62 | 0.59 | 0.57 | 0.55 | 0.53 | 0.51 | 0.49 | 0.47 | 0.45 |
| 0.60       | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 |
| 0.40       | 0.80 | 0.80 | 0.80 | 0.80 | 0.80 | 0.80 | 0.80 | 0.80 | 0.80 | 0.80 | 0.80 | 0.80 | 0.80 | 0.80 | 0.80 | 0.80 | 0.80 | 0.80 | 0.80 | 0.80 | 0.80 | 0.80 |
| 0.20       | 0.60 | 0.60 | 0.60 | 0.60 | 0.60 | 0.60 | 0.60 | 0.60 | 0.60 | 0.60 | 0.60 | 0.60 | 0.60 | 0.60 | 0.60 | 0.60 | 0.60 | 0.60 | 0.60 | 0.60 | 0.60 | 0.60 |
| 0.10       | 0.40 | 0.40 | 0.40 | 0.40 | 0.40 | 0.40 | 0.40 | 0.40 | 0.40 | 0.40 | 0.40 | 0.40 | 0.40 | 0.40 | 0.40 | 0.40 | 0.40 | 0.40 | 0.40 | 0.40 | 0.40 | 0.40 |
| 0.05       | 0.20 | 0.20 | 0.20 | 0.20 | 0.20 | 0.20 | 0.20 | 0.20 | 0.20 | 0.20 | 0.20 | 0.20 | 0.20 | 0.20 | 0.20 | 0.20 | 0.20 | 0.20 | 0.20 | 0.20 | 0.20 | 0.20 |
| 0.00       | 0.10 | 0.10 | 0.10 | 0.10 | 0.10 | 0.10 | 0.10 | 0.10 | 0.10 | 0.10 | 0.10 | 0.10 | 0.10 | 0.10 | 0.10 | 0.10 | 0.10 | 0.10 | 0.10 | 0.10 | 0.10 | 0.10 |

*Total Factored Axial Load (kip)*
Non-Sway Moment Magnifier for 4" Flat Walls

$f_c = 4000$ psf

| $w$ (in) | $P_N$ | 1 | 2 | 3 | 4 | 5 | 6 | 7 | 8 | 9 | 10 | 11 | 12 | 13 | 14 | 15 | 16 | 17 | 18 | 19 | 20 | 21 | 22 | 23 | 24 | 25 |
|---------|-------|---|---|---|---|---|---|---|---|---|----|----|----|----|----|----|----|----|----|----|----|----|----|----|----|----|----|
| 0.40    | 1.02  | 1.04 | 1.07 | 1.10 | 1.13 | 1.17 | 1.20 | 1.23 | 1.26 | 1.30 | 1.35 | 1.40 | 1.46 | 1.53 | 1.60 | 1.67 | 1.75 | 1.83 | 1.92 | 2.00 | 2.09 |
| 0.60    | 1.03  | 1.06 | 1.09 | 1.13 | 1.17 | 1.21 | 1.25 | 1.30 | 1.35 | 1.41 | 1.48 | 1.55 | 1.63 | 1.71 | 1.80 | 1.89 | 1.99 | 2.10 | 2.20 | 2.33 | 2.50 | 2.70 |
| 0.80    | 1.04  | 1.08 | 1.12 | 1.16 | 1.21 | 1.26 | 1.32 | 1.38 | 1.45 | 1.53 | 1.63 | 1.73 | 1.85 | 1.99 | 2.15 | 2.30 | 2.48 | 2.68 | 2.93 | 3.23 | 3.63 |
| 1.00    | 1.05  | 1.10 | 1.15 | 1.20 | 1.25 | 1.31 | 1.38 | 1.46 | 1.55 | 1.65 | 1.77 | 1.90 | 2.06 | 2.24 | 2.45 | 2.70 | 3.00 | 3.36 | 3.82 | 4.39 | 5.16 |

Moments in kip-ft

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Structural Design of Insulating Concrete Form Walls in Residential Construction
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**Non-Sway Moment Magnifier**

for 6" Flat Walls

\[ f' = 3000 \text{ psi} \]

**Structural Design of Insulating Concrete Form Walls in Residential Construction**

108
## Non-Sway Moment Magnifier for 6" Flat Walls

\( f'_{ce} = 3000 \text{ psi} \)

| \( \phi_0 \) | \( \phi_2 \) | 2 | 4 | 6 | 8 | 10 | 12 | 14 | 16 | 18 | 20 | 22 | 24 | 26 | 28 | 30 | 32 | 34 | 36 | 38 | 40 | 42 | 44 | 46 | 48 | 50 |
| <0.60    | <0.40    | 1.04| 1.08| 1.13| 1.18| 1.23| 1.29| 1.36| 1.43| 1.51| 1.60| 1.70| 1.82| 1.95| 2.11| 2.29| 2.51| 2.77| 3.09| 3.49|    |
| 0.60     | 0.40     | 1.04| 1.09| 1.14| 1.19| 1.25| 1.32| 1.39| 1.47| 1.56| 1.67| 1.79| 1.92| 2.06| 2.27| 2.50| 2.78| 3.13| 3.55|    |
| 0.80     | 0.40     | 1.05| 1.10| 1.16| 1.22| 1.29| 1.37| 1.46| 1.57| 1.68| 1.83| 1.99| 2.19| 2.40| 2.73| 3.13| 3.55|    |
| 1.00     | 0.40     | 1.05| 1.12| 1.18| 1.26| 1.35| 1.45| 1.57| 1.71| 1.88| 2.09| 2.34| 2.68| 3.09| 3.68|    |
| 1.00     | >0.40    | 1.05| 1.10| 1.16| 1.23| 1.30| 1.38| 1.48| 1.59| 1.71| 1.89| 2.03| 2.24| 2.50| 2.82| 3.25| 3.62|    |
| 0.80     | 0.40     | 1.05| 1.11| 1.17| 1.24| 1.33| 1.42| 1.52| 1.65| 1.79| 1.97| 2.18| 2.44| 2.77| 3.21| 3.61|    |
| 0.80     | >0.40    | 1.06| 1.13| 1.20| 1.29| 1.39| 1.50| 1.64| 1.80| 2.00| 2.25| 2.54| 3.01| 3.61|    |
| 1.00     | 0.40     | 1.07| 1.15| 1.24| 1.34| 1.47| 1.62| 1.81| 2.05| 2.36| 2.77| 3.37|    |
| 1.00     | >0.40    | 1.06| 1.14| 1.22| 1.31| 1.43| 1.56| 1.72| 1.92| 2.17| 2.49| 2.62| 3.54|    |
| 1.40     | <0.40    | 1.07| 1.15| 1.24| 1.34| 1.47| 1.62| 1.81| 2.04| 2.35| 2.76| 3.35|    |
| 1.40     | >0.40    | 1.08| 1.17| 1.28| 1.41| 1.56| 1.76| 2.02| 2.36| 2.65| 3.59|    |
| 1.00     | 0.40     | 1.06| 1.20| 1.33| 1.50| 1.71| 1.99| 2.38| 2.97| 3.54|    |
| 1.00     | >0.40    | 1.09| 1.20| 1.34| 1.52| 1.74| 2.04| 2.47| 2.12|    |
| 0.80     | 0.40     | 1.10| 1.22| 1.37| 1.57| 1.83| 2.19| 2.73| 3.53|    |
| 0.80     | >0.40    | 1.11| 1.26| 1.44| 1.69| 2.05| 2.50| 3.45|    |
| 1.00     | 0.40     | 1.13| 1.31| 1.55| 1.89| 2.43| 3.41|    |
| 1.00     | >0.40    | 1.17| 1.42| 1.79| 2.42| 3.76|    |
| 0.60     | 0.40     | 1.19| 1.45| 1.88| 2.67|    |
| 0.60     | >0.40    | 1.22| 1.55| 2.13| 3.43|    |
| 1.00     | 0.40     | 1.26| 1.69| 2.57|    |
### Non-Sway Moment Magnifier
#### for 6" Flat Walls

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#### Total Factored Axial Load (kip)

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#### Structural Design of Insulating Concrete Form Walls in Residential Construction
### Non-Sway Moment Magnifier for 6" Flat Walls

\( f_y = 4000 \text{ psi} \)

#### Table 1: Total Factored Axial Load (ips)

| \( d \) (in) | \( f_y \) | 2  | 4  | 6  | 8  | 10 | 12 | 14 | 16 | 18 | 20 | 22 | 24 | 26 | 28 | 30 | 32 | 34 | 36 | 38 | 40 | 42 | 44 | 46 | 48 | 50 |
|-------------|---------|-----|-----|-----|-----|-----|-----|-----|-----|-----|-----|-----|-----|-----|-----|-----|-----|-----|-----|-----|-----|-----|-----|-----|-----|-----|-----|
| <0.80       |         |     |     |     |     |     |     |     |     |     |     |     |     |     |     |     |     |     |     |     |     |     |     |     |     |     |     |
| 0.60        |         |     |     |     |     |     |     |     |     |     |     |     |     |     |     |     |     |     |     |     |     |     |     |     |     |     |     |
| 0.80        |         |     |     |     |     |     |     |     |     |     |     |     |     |     |     |     |     |     |     |     |     |     |     |     |     |     |     |
| 1.00        |         |     |     |     |     |     |     |     |     |     |     |     |     |     |     |     |     |     |     |     |     |     |     |     |     |     |     |
| 1.20        |         |     |     |     |     |     |     |     |     |     |     |     |     |     |     |     |     |     |     |     |     |     |     |     |     |     |     |
| 1.40        |         |     |     |     |     |     |     |     |     |     |     |     |     |     |     |     |     |     |     |     |     |     |     |     |     |     |     |
| 1.60        |         |     |     |     |     |     |     |     |     |     |     |     |     |     |     |     |     |     |     |     |     |     |     |     |     |     |     |
| 1.80        |         |     |     |     |     |     |     |     |     |     |     |     |     |     |     |     |     |     |     |     |     |     |     |     |     |     |     |
| >2.00       |         |     |     |     |     |     |     |     |     |     |     |     |     |     |     |     |     |     |     |     |     |     |     |     |     |     |     |

#### Structural Design of Insulating Concrete Form Walls in Residential Construction

111
## Non-Sway Moment Magnifier for 8" Flat Walls

For $F_r = 3000$ psi

### Table 1: Total Factored Axial Load (kips)

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<tr>
<th>$L_r$ (in)</th>
<th>$L_x$</th>
<th>2 4 6 8 10</th>
<th>12 14 16 18 20</th>
<th>Total Factored Axial Load (kips)</th>
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<tbody>
<tr>
<td>0.00</td>
<td>1.00</td>
<td>1.01 1.02 1.03 1.04 1.05</td>
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<td>1.05 1.06 1.08 1.10 1.12</td>
<td>1.14 1.16 1.18 1.20 1.22</td>
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<td>1.00</td>
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<td>1.02 1.03 1.05 1.07 1.09</td>
<td>1.10 1.12 1.14 1.16 1.18</td>
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<td>1.00 1.01 1.03 1.05 1.07</td>
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<td>0.99 1.00 1.02 1.04 1.06</td>
<td>0.96 0.98 1.00 1.02 1.04</td>
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</table>

### Table 2: Structural Design of Insulating Concrete Form Walls in Residential Construction

- Material: Insulating Concrete Form Walls
- Wall Thickness: 8" flat walls
- Load Factor: $F_r = 3000$ psi
- Dynamic Equations:
  - Magnification Factor: $M_f$
  - Total Factored Axial Load: $P_f = M_f \times P_{\text{dead}}$

---

### Notes:

- $P_{\text{dead}}$: Dead load
- $P_{\text{live}}$: Live load
- $P_{\text{wind}}$: Wind load
- $M_f$: Magnification factor

---

**Source:** Table 1, Table 2. Non-Sway Moment Magnifier for 8" Flat Walls. Structural Design of Insulating Concrete Form Walls in Residential Construction. 2023.
### Structural Design of Insulating Concrete Form Walls in Residential Construction

#### Non-Sway Moment Magnifier for 8" Flat Walls

| \( f_{c} \) (psi) | 2 | 4 | 6 | 8 | 10 | 12 | 14 | 16 | 18 | 20 | 22 | 24 | 26 | 28 | 30 | 32 | 34 | 36 | 38 | 40 | 42 | 44 | 46 | 48 | 50 |
| 3000             | 1.0 | 0.01 | 1.02 | 1.03 | 1.04 | 1.05 | 1.06 | 1.07 | 1.08 | 1.09 | 1.10 | 1.11 | 1.12 | 1.13 | 1.14 | 1.15 | 1.16 | 1.17 |
| 5000             | 0.25 | 1.01 | 1.02 | 1.03 | 1.04 | 1.05 | 1.06 | 1.07 | 1.08 | 1.09 | 1.10 | 1.11 | 1.12 | 1.13 | 1.14 | 1.15 | 1.16 | 1.17 |
| 7000             | 0.50 | 1.01 | 1.02 | 1.03 | 1.04 | 1.05 | 1.06 | 1.07 | 1.08 | 1.09 | 1.10 | 1.11 | 1.12 | 1.13 | 1.14 | 1.15 | 1.16 | 1.17 |
| 9000             | 0.75 | 1.01 | 1.02 | 1.03 | 1.04 | 1.05 | 1.06 | 1.07 | 1.08 | 1.09 | 1.10 | 1.11 | 1.12 | 1.13 | 1.14 | 1.15 | 1.16 | 1.17 |
| 11000            | 1.00 | 1.01 | 1.02 | 1.03 | 1.04 | 1.05 | 1.06 | 1.07 | 1.08 | 1.09 | 1.10 | 1.11 | 1.12 | 1.13 | 1.14 | 1.15 | 1.16 | 1.17 |

#### Moment Magnifiers

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<th>44</th>
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</tr>
</tbody>
</table>

#### Total Factored Axial Load (kips)

| \( f_{c} \) (psi) | 2 | 4 | 6 | 8 | 10 | 12 | 14 | 16 | 18 | 20 | 22 | 24 | 26 | 28 | 30 | 32 | 34 | 36 | 38 | 40 | 42 | 44 | 46 | 48 | 50 |
| 3000             | 1.0 | 0.01 | 1.02 | 1.03 | 1.04 | 1.05 | 1.06 | 1.07 | 1.08 | 1.09 | 1.10 | 1.11 | 1.12 | 1.13 | 1.14 | 1.15 | 1.16 | 1.17 |
| 5000             | 0.25 | 1.01 | 1.02 | 1.03 | 1.04 | 1.05 | 1.06 | 1.07 | 1.08 | 1.09 | 1.10 | 1.11 | 1.12 | 1.13 | 1.14 | 1.15 | 1.16 | 1.17 |
| 7000             | 0.50 | 1.01 | 1.02 | 1.03 | 1.04 | 1.05 | 1.06 | 1.07 | 1.08 | 1.09 | 1.10 | 1.11 | 1.12 | 1.13 | 1.14 | 1.15 | 1.16 | 1.17 |
| 9000             | 0.75 | 1.01 | 1.02 | 1.03 | 1.04 | 1.05 | 1.06 | 1.07 | 1.08 | 1.09 | 1.10 | 1.11 | 1.12 | 1.13 | 1.14 | 1.15 | 1.16 | 1.17 |
| 11000            | 1.00 | 1.01 | 1.02 | 1.03 | 1.04 | 1.05 | 1.06 | 1.07 | 1.08 | 1.09 | 1.10 | 1.11 | 1.12 | 1.13 | 1.14 | 1.15 | 1.16 | 1.17 |
### Non-Sway Moment Magnifier for 8° Flat Walls

**Factor:** $F_m = 3000$ psi

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### Table: Total Factored Axial Load (kips)

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<th>(y_{F_m} = 2.40)</th>
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### Structural Design of Insulating Concrete Form Walls in Residential Construction

Moment Magnifiers

- **Non-Sway Moment Magnifier**
- **8° Flat Walls**
- **Factor:** $F_m = 3000$ psi

- **Total Factored Axial Load (kips)**

- **Table:**

  - | \(y_{F_m} = 2.00\) |
  - | \(y_{F_m} = 2.40\) |
  - | \(y_{F_m} = 2.80\) |
  - | \(y_{F_m} = 3.20\) |
  - | \(y_{F_m} = 3.60\) |
  - | \(y_{F_m} = 4.00\) |
  - | \(y_{F_m} = 4.40\) |
  - | \(y_{F_m} = 4.80\) |

- **Legend:**
  - \(q\) (ksi) refers to the applied axial load.
  - \(y_{F_m}\) represents the factored moment magnifier.

- **Data Overview:**
  - Data includes total factored axial loads for various moments and applied axial loads, facilitating the design process for insulating concrete form walls in residential construction.
## Non-Sway Moment Magnifier for 8" Flat Walls

\[ f_y = 3000 \text{ psi} \]

| \( a \) (in) | \( f_a \) | 2 | 4 | 6 | 8 | 10 | 12 | 14 | 16 | 18 | 20 | 22 | 24 | 26 | 28 | 30 | 32 | 34 | 36 | 38 | 40 | 42 | 44 | 46 | 48 | 50 |
| <0.80   | <0.40   | 1.03 | 1.06 | 1.10 | 1.14 | 1.18 | 1.22 | 1.27 | 1.32 | 1.37 | 1.43 | 1.49 | 1.56 | 1.64 | 1.73 | 1.82 | 1.93 | 2.04 | 2.18 | 2.33 | 2.51 | 2.71 | 2.95 | 3.24 | 3.58 |
| 0.80    | 0.88    | 1.03 | 1.07 | 1.11 | 1.15 | 1.19 | 1.24 | 1.29 | 1.34 | 1.40 | 1.47 | 1.54 | 1.62 | 1.71 | 1.81 | 1.92 | 2.05 | 2.19 | 2.36 | 2.59 | 2.78 | 3.05 | 3.33 | 3.63 |
| 0.90    | 1.04    | 1.08 | 1.12 | 1.17 | 1.22 | 1.28 | 1.34 | 1.41 | 1.48 | 1.57 | 1.68 | 1.77 | 1.89 | 2.03 | 2.19 | 2.38 | 2.60 | 2.87 | 3.21 | 3.63 |
| 1.00    | 1.04    | 1.08 | 1.11 | 1.14 | 1.19 | 1.23 | 1.28 | 1.34 | 1.42 | 1.50 | 1.60 | 1.71 | 1.84 | 2.00 | 2.18 | 2.40 | 2.66 | 2.89 | 3.42 | 3.95 |

| 1.20   | <0.40   | 1.04 | 1.07 | 1.12 | 1.18 | 1.23 | 1.28 | 1.34 | 1.42 | 1.50 | 1.60 | 1.71 | 1.84 | 2.00 | 2.18 | 2.40 | 2.66 | 2.89 | 3.42 | 3.95 |
| 0.80   | 1.04 | 1.07 | 1.12 | 1.17 | 1.23 | 1.29 | 1.35 | 1.42 | 1.50 | 1.59 | 1.69 | 1.80 | 1.93 | 2.08 | 2.25 | 2.45 | 2.70 | 3.00 | 3.37 | 3.66 |
| 0.90   | 1.04 | 1.09 | 1.14 | 1.20 | 1.27 | 1.34 | 1.42 | 1.50 | 1.61 | 1.72 | 1.85 | 2.01 | 2.20 | 2.42 | 2.69 | 3.03 | 3.48 |
| 1.00   | 1.06 | 1.11 | 1.17 | 1.24 | 1.32 | 1.41 | 1.51 | 1.63 | 1.76 | 1.93 | 2.13 | 2.37 | 2.67 | 3.07 | 3.60 |

| 1.60   | <0.40   | 1.04 | 1.07 | 1.14 | 1.20 | 1.28 | 1.33 | 1.41 | 1.49 | 1.59 | 1.70 | 1.83 | 1.98 | 2.15 | 2.36 | 2.62 | 2.84 | 3.34 | 3.88 |
| 0.80   | 1.05 | 1.10 | 1.15 | 1.21 | 1.28 | 1.38 | 1.44 | 1.54 | 1.65 | 1.78 | 1.93 | 2.11 | 2.33 | 2.60 | 2.93 | 3.36 | 3.89 |
| 0.90   | 1.05 | 1.11 | 1.18 | 1.25 | 1.33 | 1.42 | 1.53 | 1.66 | 1.81 | 1.99 | 2.21 | 2.48 | 2.82 | 3.29 | 3.63 |
| 1.00   | 1.05 | 1.13 | 1.21 | 1.30 | 1.40 | 1.53 | 1.67 | 1.84 | 2.06 | 2.32 | 2.59 | 2.88 |

| 2.00   | <0.40   | 1.05 | 1.11 | 1.18 | 1.25 | 1.33 | 1.42 | 1.51 | 1.61 | 1.76 | 1.94 | 2.17 | 2.48 | 2.84 | 3.35 |
| 0.80   | 1.07 | 1.14 | 1.22 | 1.32 | 1.44 | 1.58 | 1.75 | 1.96 | 2.22 | 2.57 | 3.04 | 3.74 |
| 0.90   | 1.08 | 1.16 | 1.27 | 1.39 | 1.54 | 1.75 | 1.97 | 2.26 | 2.71 | 3.35 |
| 1.00   | 1.08 | 1.18 | 1.29 | 1.43 | 1.62 | 1.83 | 2.15 | 2.56 | 3.19 | 3.93 |

| 4.00   | <0.40   | 1.07 | 1.15 | 1.25 | 1.36 | 1.49 | 1.65 | 1.85 | 2.11 | 2.44 | 2.91 | 3.62 |
| 0.80   | 1.08 | 1.16 | 1.27 | 1.39 | 1.54 | 1.72 | 1.96 | 2.27 | 2.70 | 3.32 |
| 0.90   | 1.09 | 1.18 | 1.31 | 1.46 | 1.65 | 1.90 | 2.24 | 2.73 | 3.45 |
| 1.00   | 1.10 | 1.22 | 1.38 | 1.57 | 1.83 | 2.06 | 2.75 | 3.67 |

| 8.00   | <0.40   | 1.10 | 1.23 | 1.39 | 1.60 | 1.87 | 2.27 | 2.78 | 3.44 |
| 0.80   | 1.11 | 1.25 | 1.42 | 1.65 | 1.99 | 2.46 | 3.20 |
| 0.90   | 1.13 | 1.29 | 1.51 | 1.82 | 2.28 | 3.07 |
| 1.00   | 1.15 | 1.35 | 1.63 | 2.07 | 2.63 |

| >8.00   | <0.40   | 1.13 | 1.31 | 1.55 | 1.90 | 2.44 | 3.44 |
| 0.80   | 1.14 | 1.34 | 1.61 | 2.01 | 2.70 |
| 0.90   | 1.17 | 1.40 | 1.75 | 2.32 | 3.48 |
| 1.00   | 1.20 | 1.48 | 1.97 | 2.60 |

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**Structural Design of Insulating Concrete Form Walls in Residential Construction**

115
| Load (kips) | L/32 | 2 | 4 | 6 | 8 | 10 | 12 | 14 | 16 | 18 | 20 | 22 | 24 | 26 | 28 | 30 | 32 | 34 | 36 | 38 | 40 | 42 | 44 | 46 | 48 | 50 |
| 0.80       | 1.00 | 1.01 | 1.02 | 1.03 | 1.04 | 1.05 | 1.06 | 1.07 | 1.08 | 1.09 | 1.10 | 1.11 | 1.12 | 1.13 | 1.14 | 1.15 | 1.16 | 1.17 | 1.18 | 1.19 | 1.20 | 1.21 | 1.22 | 1.23 | 1.24 | 1.25 | 1.26 |
| 0.80       | 1.01 | 1.02 | 1.03 | 1.04 | 1.05 | 1.06 | 1.07 | 1.08 | 1.09 | 1.10 | 1.11 | 1.12 | 1.13 | 1.14 | 1.15 | 1.16 | 1.17 | 1.18 | 1.19 | 1.20 | 1.21 | 1.22 | 1.23 | 1.24 | 1.25 | 1.26 | 1.27 |
| 0.80       | 1.02 | 1.03 | 1.04 | 1.05 | 1.06 | 1.07 | 1.08 | 1.09 | 1.10 | 1.11 | 1.12 | 1.13 | 1.14 | 1.15 | 1.16 | 1.17 | 1.18 | 1.19 | 1.20 | 1.21 | 1.22 | 1.23 | 1.24 | 1.25 | 1.26 | 1.27 | 1.28 |
| 0.80       | 1.03 | 1.04 | 1.05 | 1.06 | 1.07 | 1.08 | 1.09 | 1.10 | 1.11 | 1.12 | 1.13 | 1.14 | 1.15 | 1.16 | 1.17 | 1.18 | 1.19 | 1.20 | 1.21 | 1.22 | 1.23 | 1.24 | 1.25 | 1.26 | 1.27 | 1.28 | 1.29 |
| 0.80       | 1.04 | 1.05 | 1.06 | 1.07 | 1.08 | 1.09 | 1.10 | 1.11 | 1.12 | 1.13 | 1.14 | 1.15 | 1.16 | 1.17 | 1.18 | 1.19 | 1.20 | 1.21 | 1.22 | 1.23 | 1.24 | 1.25 | 1.26 | 1.27 | 1.28 | 1.29 | 1.30 |

Non-Sway Moment Magnifier for 8" Flat Walls, \( f_c = 4000 \text{ psi} \)

Total Factored Axial Load (kips)

| Load (kips) | 2 | 4 | 6 | 8 | 10 | 12 | 14 | 16 | 18 | 20 | 22 | 24 | 26 | 28 | 30 | 32 | 34 | 36 | 38 | 40 | 42 | 44 | 46 | 48 | 50 |
| 0.80       | 1.04 | 1.05 | 1.06 | 1.07 | 1.08 | 1.09 | 1.10 | 1.11 | 1.12 | 1.13 | 1.14 | 1.15 | 1.16 | 1.17 | 1.18 | 1.19 | 1.20 | 1.21 | 1.22 | 1.23 | 1.24 | 1.25 | 1.26 | 1.27 | 1.28 | 1.29 |
| 0.80       | 1.05 | 1.06 | 1.07 | 1.08 | 1.09 | 1.10 | 1.11 | 1.12 | 1.13 | 1.14 | 1.15 | 1.16 | 1.17 | 1.18 | 1.19 | 1.20 | 1.21 | 1.22 | 1.23 | 1.24 | 1.25 | 1.26 | 1.27 | 1.28 | 1.29 | 1.30 |
| 0.80       | 1.06 | 1.07 | 1.08 | 1.09 | 1.10 | 1.11 | 1.12 | 1.13 | 1.14 | 1.15 | 1.16 | 1.17 | 1.18 | 1.19 | 1.20 | 1.21 | 1.22 | 1.23 | 1.24 | 1.25 | 1.26 | 1.27 | 1.28 | 1.29 | 1.30 | 1.31 |
| 0.80       | 1.07 | 1.08 | 1.09 | 1.10 | 1.11 | 1.12 | 1.13 | 1.14 | 1.15 | 1.16 | 1.17 | 1.18 | 1.19 | 1.20 | 1.21 | 1.22 | 1.23 | 1.24 | 1.25 | 1.26 | 1.27 | 1.28 | 1.29 | 1.30 | 1.31 | 1.32 |
| 0.80       | 1.08 | 1.09 | 1.10 | 1.11 | 1.12 | 1.13 | 1.14 | 1.15 | 1.16 | 1.17 | 1.18 | 1.19 | 1.20 | 1.21 | 1.22 | 1.23 | 1.24 | 1.25 | 1.26 | 1.27 | 1.28 | 1.29 | 1.30 | 1.31 | 1.32 | 1.33 |
| 0.80       | 1.09 | 1.10 | 1.11 | 1.12 | 1.13 | 1.14 | 1.15 | 1.16 | 1.17 | 1.18 | 1.19 | 1.20 | 1.21 | 1.22 | 1.23 | 1.24 | 1.25 | 1.26 | 1.27 | 1.28 | 1.29 | 1.30 | 1.31 | 1.32 | 1.33 | 1.34 |

Structural Design of Insulating Concrete Form Walls in Residential Construction
### Non-Sway Moment Magnifier

**for
6" Flat Walls**

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### Structural Design of Insulating Concrete Form Walls in Residential Construction

- Moment Magnifiers

#### Table 1: Non-Sway Moment Magnifier for 6" Flat Walls

| 8 in | 10 in | 12 in | 14 in | 16 in | 18 in | 20 in | 22 in | 24 in | 26 in | 28 in | 30 in | 32 in | 34 in | 36 in | 38 in | 40 in | 42 in | 44 in | 46 in | 50 in |
|------|------|------|------|------|------|------|------|------|------|------|------|------|------|------|------|------|------|------|------|------|------|
| 1.0  | 1.0  | 1.0  | 1.0  | 1.0  | 1.0  | 1.0  | 1.0  | 1.0  | 1.0  | 1.0  | 1.0  | 1.0  | 1.0  | 1.0  | 1.0  | 1.0  | 1.0  | 1.0  | 1.0  | 1.0  |
| 1.5  | 1.5  | 1.5  | 1.5  | 1.5  | 1.5  | 1.5  | 1.5  | 1.5  | 1.5  | 1.5  | 1.5  | 1.5  | 1.5  | 1.5  | 1.5  | 1.5  | 1.5  | 1.5  | 1.5  | 1.5  |
| 2.0  | 2.0  | 2.0  | 2.0  | 2.0  | 2.0  | 2.0  | 2.0  | 2.0  | 2.0  | 2.0  | 2.0  | 2.0  | 2.0  | 2.0  | 2.0  | 2.0  | 2.0  | 2.0  | 2.0  | 2.0  |
| 2.5  | 2.5  | 2.5  | 2.5  | 2.5  | 2.5  | 2.5  | 2.5  | 2.5  | 2.5  | 2.5  | 2.5  | 2.5  | 2.5  | 2.5  | 2.5  | 2.5  | 2.5  | 2.5  | 2.5  | 2.5  |
| 3.0  | 3.0  | 3.0  | 3.0  | 3.0  | 3.0  | 3.0  | 3.0  | 3.0  | 3.0  | 3.0  | 3.0  | 3.0  | 3.0  | 3.0  | 3.0  | 3.0  | 3.0  | 3.0  | 3.0  | 3.0  |
| 3.5  | 3.5  | 3.5  | 3.5  | 3.5  | 3.5  | 3.5  | 3.5  | 3.5  | 3.5  | 3.5  | 3.5  | 3.5  | 3.5  | 3.5  | 3.5  | 3.5  | 3.5  | 3.5  | 3.5  | 3.5  |
| 4.0  | 4.0  | 4.0  | 4.0  | 4.0  | 4.0  | 4.0  | 4.0  | 4.0  | 4.0  | 4.0  | 4.0  | 4.0  | 4.0  | 4.0  | 4.0  | 4.0  | 4.0  | 4.0  | 4.0  | 4.0  |
| 4.5  | 4.5  | 4.5  | 4.5  | 4.5  | 4.5  | 4.5  | 4.5  | 4.5  | 4.5  | 4.5  | 4.5  | 4.5  | 4.5  | 4.5  | 4.5  | 4.5  | 4.5  | 4.5  | 4.5  | 4.5  |
| 5.0  | 5.0  | 5.0  | 5.0  | 5.0  | 5.0  | 5.0  | 5.0  | 5.0  | 5.0  | 5.0  | 5.0  | 5.0  | 5.0  | 5.0  | 5.0  | 5.0  | 5.0  | 5.0  | 5.0  | 5.0  |
### Non-Sway Moment Magnifier for 8" Flat Walls

Total Factored Axial Load (kip)

| a (ft) | 2 | 4 | 6 | 8 | 10 | 12 | 14 | 16 | 18 | 20 | 22 | 24 | 26 | 28 | 30 | 32 | 34 | 36 | 38 | 40 | 42 | 44 | 46 | 48 | 50 |
|-------|---|---|---|---|----|----|----|----|----|----|----|----|----|----|----|----|----|----|----|----|----|----|----|----|----|----|----|----|----|
| 0.00  | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 |
| 0.02  | 1.04 | 1.05 | 1.06 | 1.07 | 1.08 | 1.09 | 1.10 | 1.11 | 1.12 | 1.13 | 1.14 | 1.15 | 1.16 | 1.17 | 1.18 | 1.19 | 1.20 | 1.21 | 1.22 | 1.23 | 1.24 | 1.25 | 1.26 | 1.27 | 1.28 | 1.29 | 1.30 |
| 0.04  | 1.08 | 1.09 | 1.10 | 1.11 | 1.12 | 1.13 | 1.14 | 1.15 | 1.16 | 1.17 | 1.18 | 1.19 | 1.20 | 1.21 | 1.22 | 1.23 | 1.24 | 1.25 | 1.26 | 1.27 | 1.28 | 1.29 | 1.30 | 1.31 | 1.32 | 1.33 | 1.34 |
| 0.06  | 1.12 | 1.13 | 1.14 | 1.15 | 1.16 | 1.17 | 1.18 | 1.19 | 1.20 | 1.21 | 1.22 | 1.23 | 1.24 | 1.25 | 1.26 | 1.27 | 1.28 | 1.29 | 1.30 | 1.31 | 1.32 | 1.33 | 1.34 | 1.35 | 1.36 | 1.37 | 1.38 |
| 0.08  | 1.16 | 1.17 | 1.18 | 1.19 | 1.20 | 1.21 | 1.22 | 1.23 | 1.24 | 1.25 | 1.26 | 1.27 | 1.28 | 1.29 | 1.30 | 1.31 | 1.32 | 1.33 | 1.34 | 1.35 | 1.36 | 1.37 | 1.38 | 1.39 | 1.40 | 1.41 | 1.42 |
| 0.10  | 1.20 | 1.21 | 1.22 | 1.23 | 1.24 | 1.25 | 1.26 | 1.27 | 1.28 | 1.29 | 1.30 | 1.31 | 1.32 | 1.33 | 1.34 | 1.35 | 1.36 | 1.37 | 1.38 | 1.39 | 1.40 | 1.41 | 1.42 | 1.43 | 1.44 | 1.45 | 1.46 |

Moment Magnifiers

Structural Design of Insulating Concrete Form Walls in Residential Construction
# Moment Magnifiers

## Non-Sway Moment Magnifier for 8" Flat Walls

| \( \gamma_0 = 4000 \) psi |

| \( \gamma_0 \) | 2 | 4 | 6 | 8 | 10 | 12 | 14 | 16 | 18 | 20 | 22 | 24 | 26 | 28 | 30 | 32 | 34 | 36 | 38 | 40 | 42 | 44 | 46 | 48 | 50 |
| 1.00 - 1.25 | 1.03 | 1.05 | 1.07 | 1.10 | 1.14 | 1.18 | 1.22 | 1.26 | 1.31 | 1.37 | 1.43 | 1.50 | 1.57 | 1.64 | 1.71 | 1.78 | 1.86 | 1.94 | 2.02 | 2.10 | 2.19 | 2.29 | 2.39 | 2.49 | 2.59 |
| 1.26 - 1.50 | 1.03 | 1.05 | 1.07 | 1.10 | 1.14 | 1.18 | 1.22 | 1.26 | 1.31 | 1.37 | 1.43 | 1.50 | 1.57 | 1.64 | 1.71 | 1.78 | 1.86 | 1.94 | 2.02 | 2.10 | 2.19 | 2.29 | 2.39 | 2.49 | 2.59 |
| 1.51 - 1.75 | 1.03 | 1.05 | 1.07 | 1.10 | 1.14 | 1.18 | 1.22 | 1.26 | 1.31 | 1.37 | 1.43 | 1.50 | 1.57 | 1.64 | 1.71 | 1.78 | 1.86 | 1.94 | 2.02 | 2.10 | 2.19 | 2.29 | 2.39 | 2.49 | 2.59 |
| 1.76 - 2.00 | 1.03 | 1.05 | 1.07 | 1.10 | 1.14 | 1.18 | 1.22 | 1.26 | 1.31 | 1.37 | 1.43 | 1.50 | 1.57 | 1.64 | 1.71 | 1.78 | 1.86 | 1.94 | 2.02 | 2.10 | 2.19 | 2.29 | 2.39 | 2.49 | 2.59 |
| 2.01 - 2.25 | 1.03 | 1.05 | 1.07 | 1.10 | 1.14 | 1.18 | 1.22 | 1.26 | 1.31 | 1.37 | 1.43 | 1.50 | 1.57 | 1.64 | 1.71 | 1.78 | 1.86 | 1.94 | 2.02 | 2.10 | 2.19 | 2.29 | 2.39 | 2.49 | 2.59 |
| 2.26 - 2.50 | 1.03 | 1.05 | 1.07 | 1.10 | 1.14 | 1.18 | 1.22 | 1.26 | 1.31 | 1.37 | 1.43 | 1.50 | 1.57 | 1.64 | 1.71 | 1.78 | 1.86 | 1.94 | 2.02 | 2.10 | 2.19 | 2.29 | 2.39 | 2.49 | 2.59 |
| 2.51 - 2.75 | 1.03 | 1.05 | 1.07 | 1.10 | 1.14 | 1.18 | 1.22 | 1.26 | 1.31 | 1.37 | 1.43 | 1.50 | 1.57 | 1.64 | 1.71 | 1.78 | 1.86 | 1.94 | 2.02 | 2.10 | 2.19 | 2.29 | 2.39 | 2.49 | 2.59 |
| 2.76 - 3.00 | 1.03 | 1.05 | 1.07 | 1.10 | 1.14 | 1.18 | 1.22 | 1.26 | 1.31 | 1.37 | 1.43 | 1.50 | 1.57 | 1.64 | 1.71 | 1.78 | 1.86 | 1.94 | 2.02 | 2.10 | 2.19 | 2.29 | 2.39 | 2.49 | 2.59 |

**Structural Design of Insulating Concrete Form Walls in Residential Construction**
### Non-Sway Moment Magnifier

**for 10" Flat Walls**

**f<sub>c</sub> = 3000 psi**

| a (in) | l<sub>1</sub> | 8 | 12 | 15 | 20 | 24 | 28 | 32 | 36 | 40 | 44 | 48 | 52 | 56 | 60 | 64 | 72 | 80 | 88 | 96 | 100 |
|--------|--------------|---|----|----|----|----|----|----|----|----|----|----|----|----|----|----|----|----|----|----|----|----|----|
| 1.80   | 0.00         | 1.00 | 1.01 | 1.02 | 1.03 | 1.04 | 1.05 | 1.06 | 1.07 | 1.08 | 1.09 | 1.10 | 1.11 | 1.12 | 1.13 | 1.14 | 1.15 | 1.16 | 1.17 | 1.18 | 1.19 |
| 1.40   | 0.00         | 1.00 | 1.01 | 1.02 | 1.03 | 1.04 | 1.05 | 1.06 | 1.07 | 1.08 | 1.09 | 1.10 | 1.11 | 1.12 | 1.13 | 1.14 | 1.15 | 1.16 | 1.17 | 1.18 | 1.19 |
| 1.00   | 0.00         | 1.00 | 1.01 | 1.02 | 1.03 | 1.04 | 1.05 | 1.06 | 1.07 | 1.08 | 1.09 | 1.10 | 1.11 | 1.12 | 1.13 | 1.14 | 1.15 | 1.16 | 1.17 | 1.18 | 1.19 |
| 0.60   | 0.00         | 1.00 | 1.01 | 1.02 | 1.03 | 1.04 | 1.05 | 1.06 | 1.07 | 1.08 | 1.09 | 1.10 | 1.11 | 1.12 | 1.13 | 1.14 | 1.15 | 1.16 | 1.17 | 1.18 | 1.19 |
| 0.00   | 0.00         | 1.00 | 1.01 | 1.02 | 1.03 | 1.04 | 1.05 | 1.06 | 1.07 | 1.08 | 1.09 | 1.10 | 1.11 | 1.12 | 1.13 | 1.14 | 1.15 | 1.16 | 1.17 | 1.18 | 1.19 |

### Total Factored Axial Load (kips)

| a (in) | l<sub>1</sub> | 8 | 12 | 15 | 20 | 24 | 28 | 32 | 36 | 40 | 44 | 48 | 52 | 56 | 60 | 64 | 72 | 80 | 88 | 96 | 100 |
|--------|--------------|---|----|----|----|----|----|----|----|----|----|----|----|----|----|----|----|----|----|----|----|----|
| 1.80   | 0.00         | 1.00 | 1.01 | 1.02 | 1.03 | 1.04 | 1.05 | 1.06 | 1.07 | 1.08 | 1.09 | 1.10 | 1.11 | 1.12 | 1.13 | 1.14 | 1.15 | 1.16 | 1.17 | 1.18 | 1.19 |
| 1.40   | 0.00         | 1.00 | 1.01 | 1.02 | 1.03 | 1.04 | 1.05 | 1.06 | 1.07 | 1.08 | 1.09 | 1.10 | 1.11 | 1.12 | 1.13 | 1.14 | 1.15 | 1.16 | 1.17 | 1.18 | 1.19 |
| 1.00   | 0.00         | 1.00 | 1.01 | 1.02 | 1.03 | 1.04 | 1.05 | 1.06 | 1.07 | 1.08 | 1.09 | 1.10 | 1.11 | 1.12 | 1.13 | 1.14 | 1.15 | 1.16 | 1.17 | 1.18 | 1.19 |
| 0.60   | 0.00         | 1.00 | 1.01 | 1.02 | 1.03 | 1.04 | 1.05 | 1.06 | 1.07 | 1.08 | 1.09 | 1.10 | 1.11 | 1.12 | 1.13 | 1.14 | 1.15 | 1.16 | 1.17 | 1.18 | 1.19 |
| 0.00   | 0.00         | 1.00 | 1.01 | 1.02 | 1.03 | 1.04 | 1.05 | 1.06 | 1.07 | 1.08 | 1.09 | 1.10 | 1.11 | 1.12 | 1.13 | 1.14 | 1.15 | 1.16 | 1.17 | 1.18 | 1.19 |
Non-Sway Moment Magnifier
for 10^4 Flat Walls
\( f_{dy} = 4000 \text{ psi} \)

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Structural Design of Insulating Concrete Form Walls in Residential Construction
### Non-Sway Moment Magnifier

for 6" Waffle-Grid Walls

$P_s = 3000$ psi

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### Total Factored Axial Load (kips)

*Note: Values are approximate.*
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<td>6-inch Wallfrid</td>
<td>123</td>
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<tr>
<td>Non-Stray Moment Magnifiers</td>
<td>9000 psi</td>
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Structural Design of Insulating Concrete Form Walls in Residential Construction
### 8" Waffle-Grid Walls

**f_e = 3000 psi**

<table>
<thead>
<tr>
<th>L / (in)</th>
<th>L_e</th>
<th>2 4 6 8 10 12</th>
<th>Total Factored Axial Load (kip)</th>
<th>Total Factored Axial Load (kip)</th>
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<td>1.06 1.10 1.14 1.17 1.21</td>
<td>1.06 1.10 1.14 1.17 1.21</td>
</tr>
</tbody>
</table>

*Note: The table and graph represent the Total Factored Axial Load (kip) for Waffle-Grid Walls. The values are given in kilo-pounds (kip) and correspond to different lengths (L) and effective lengths (L_e) of the walls.*

**Design Considerations:**
- **Compression Resistance:** The design should account for the compression resistance of the concrete and the reinforcing steel.
- **Tension Resistance:** The tension resistance should also be considered to ensure the structural integrity of the wall.
- **Load and Distance:** The load and distance from the wall will affect the total factored axial load.

### Structural Design of Insulating Concrete Form Walls in Residential Construction

- **Material Selection:** Choose appropriate materials for the walls to ensure durability and energy efficiency.
- **Geometric Design:** Optimize the geometric design to minimize load and distance effects.
- **Insulation:** Incorporate insulation to reduce heat loss and improve energy performance.
### Non-Sway Moment Magnifier

**for 8" Waffle-Grid Walls**  
\( f_{y} = 36000 \) psi

#### Total Factored Axial Load (kips)

| \( a \) (in) | \( I_a \) | 2 | 4 | 6 | 8 | 10 | 12 | 14 | 16 | 18 | 20 | 22 | 24 | 26 | 28 | 30 | 32 | 34 | 36 | 38 | 40 | 42 | 44 | 46 | 48 | 50 |
| \(<0.80\)  | \(<0.40\) | 1.02 | 1.05 | 1.07 | 1.10 | 1.13 | 1.16 | 1.19 | 1.23 | 1.26 | 1.30 | 1.34 | 1.39 | 1.48 | 1.54 | 1.64 | 1.73 | 1.87 | 1.99 | 2.14 | 2.26 | 2.39 | 2.53 | 2.67 | 2.82 | 3.00 | 3.19 |
| 0.60 | 1.03 | 1.05 | 1.08 | 1.11 | 1.14 | 1.17 | 1.21 | 1.25 | 1.29 | 1.34 | 1.39 | 1.48 | 1.57 | 1.66 | 1.77 | 1.89 | 2.05 | 2.23 | 2.44 | 2.65 | 2.89 | 3.15 | 3.44 | 3.77 | 4.13 | 4.53 |
| 0.80 | 1.03 | 1.06 | 1.09 | 1.13 | 1.17 | 1.21 | 1.26 | 1.30 | 1.34 | 1.39 | 1.44 | 1.51 | 1.60 | 1.71 | 1.84 | 2.01 | 2.22 | 2.47 | 2.75 | 3.08 | 3.44 | 3.84 | 4.30 | 5.03 | 6.02 |
| 1.00 | 1.03 | 1.07 | 1.11 | 1.15 | 1.19 | 1.24 | 1.29 | 1.34 | 1.39 | 1.45 | 1.51 | 1.58 | 1.65 | 1.74 | 1.84 | 2.01 | 2.27 | 2.59 | 3.08 | 3.68 | 4.38 | 5.31 | 6.50 | 8.03 |

#### Total Factored Axial Load (kips)  

| \( a \) (in) | \( I_a \) | 2 | 4 | 6 | 8 | 10 | 12 | 14 | 16 | 18 | 20 | 22 | 24 | 26 | 28 | 30 | 32 | 34 | 36 | 38 | 40 | 42 | 44 | 46 | 48 | 50 |
| \(<0.80\)  | \(<0.40\) | 1.03 | 1.07 | 1.11 | 1.15 | 1.20 | 1.24 | 1.28 | 1.33 | 1.38 | 1.44 | 1.50 | 1.56 | 1.65 | 1.76 | 1.88 | 2.00 | 2.14 | 2.30 | 2.48 | 2.72 | 3.02 | 3.41 | 4.01 | 5.03 | 7.00 |
| 0.60 | 1.04 | 1.08 | 1.13 | 1.18 | 1.23 | 1.28 | 1.33 | 1.39 | 1.45 | 1.51 | 1.58 | 1.66 | 1.76 | 1.88 | 2.01 | 2.16 | 2.33 | 2.53 | 2.80 | 3.23 | 4.00 | 5.03 | 7.00 | 9.30 |
| 0.80 | 1.05 | 1.10 | 1.15 | 1.21 | 1.27 | 1.33 | 1.39 | 1.46 | 1.53 | 1.61 | 1.70 | 1.80 | 1.92 | 2.05 | 2.19 | 2.34 | 2.50 | 2.78 | 3.21 | 4.00 | 5.03 | 7.00 | 9.30 |
| 1.00 | 1.06 | 1.12 | 1.18 | 1.25 | 1.32 | 1.39 | 1.47 | 1.55 | 1.64 | 1.74 | 1.85 | 2.00 | 2.17 | 2.35 | 2.55 | 2.80 | 3.21 | 4.00 | 5.03 | 7.00 | 9.30 |

#### Structural Design of Insulating Concrete Form Walls in Residential Construction
Non-Sway Moment Magnifier
for
8" Waffle-Grid Walls

\( f_{c} = 4000 \text{ psi} \)

<table>
<thead>
<tr>
<th>( \theta ) (in)</th>
<th>( \Delta )</th>
<th>Total Factored Axial Load (kip)</th>
</tr>
</thead>
<tbody>
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<tr>
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<td>0.00</td>
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</tbody>
</table>

Structural Design of Insulating Concrete Form Walls in Residential Construction
| \( f_0 \) | \( f_0 \) | 2 | 4 | 6 | 8 | 10 | 12 | 14 | 16 | 18 | 20 | 22 | 24 | 26 | 28 | 30 | 32 | 34 | 36 | 38 | 40 | 42 | 44 | 46 | 48 | 50 |
| 0.40 | 0.40 | 1.02 | 1.04 | 1.06 | 1.09 | 1.11 | 1.14 | 1.16 | 1.18 | 1.22 | 1.25 | 1.28 | 1.32 | 1.35 | 1.39 | 1.43 | 1.48 | 1.52 | 1.57 | 1.62 | 1.67 | 1.73 | 1.80 | 1.86 | 1.94 | 2.01 |
| 0.80 | 0.40 | 1.02 | 1.04 | 1.07 | 1.10 | 1.13 | 1.16 | 1.19 | 1.22 | 1.26 | 1.29 | 1.33 | 1.38 | 1.43 | 1.49 | 1.54 | 1.61 | 1.68 | 1.74 | 1.81 | 1.89 | 1.97 | 2.04 | 2.12 | 2.20 | 2.29 | 2.38 |
| 0.80 | 0.80 | 1.02 | 1.05 | 1.08 | 1.11 | 1.14 | 1.17 | 1.20 | 1.24 | 1.28 | 1.32 | 1.36 | 1.41 | 1.46 | 1.51 | 1.57 | 1.63 | 1.68 | 1.75 | 1.81 | 1.87 | 1.94 | 2.01 | 2.09 | 2.17 | 2.25 | 2.34 |
| 1.00 | 1.06 | 1.09 | 1.13 | 1.16 | 1.20 | 1.24 | 1.29 | 1.34 | 1.39 | 1.44 | 1.49 | 1.55 | 1.61 | 1.67 | 1.73 | 1.81 | 1.89 | 1.97 | 2.04 | 2.12 | 2.20 | 2.29 | 2.38 | 2.47 | 2.57 | 2.68 | 2.79 |

Structural Design of Insulating Concrete Form Walls in Residential Construction
### Non-Sway Moment Magnifier for 6” Screen-Grid Walls

\( f_e = 3000 \text{ psi} \)

| \( \phi \) | \( \beta_\phi \) | 2 | 4 | 6 | 8 | 10 | 12 | 14 | 16 | 18 | 20 | 22 | 24 | 26 | 28 | 30 | 32 | 34 | 36 | 38 | 40 | 42 | 44 | 46 | 48 | 50 |
| \(<0.60\) | \(<0.40\) | 1.03 | 1.06 | 1.09 | 1.12 | 1.15 | 1.19 | 1.23 | 1.27 | 1.32 | 1.37 | 1.42 | 1.47 | 1.53 | 1.60 | 1.67 | 1.75 | 1.83 | 1.93 | 2.03 | 2.15 | 2.28 | 2.43 | 2.60 | 2.79 | 3.02 |
| \(0.60\) | \(1.06\) | 1.09 | 1.12 | 1.15 | 1.18 | 1.22 | 1.26 | 1.30 | 1.35 | 1.40 | 1.46 | 1.52 | 1.59 | 1.66 | 1.75 | 1.84 | 1.94 | 2.05 | 2.16 | 2.33 | 2.49 | 2.68 | 2.91 | 3.17 | 3.48 |
| \(0.80\) | \(1.07\) | 1.11 | 1.15 | 1.19 | 1.24 | 1.29 | 1.35 | 1.41 | 1.48 | 1.55 | 1.63 | 1.72 | 1.82 | 1.94 | 2.07 | 2.18 | 2.31 | 2.45 | 2.60 | 2.82 | 3.10 | 3.45 | 3.87 |
| \(1.00\) | \(1.08\) | 1.13 | 1.17 | 1.22 | 1.29 | 1.36 | 1.44 | 1.52 | 1.62 | 1.73 | 1.86 | 2.02 | 2.18 | 2.36 | 2.55 | 2.76 | 2.98 | 3.23 | 3.50 | 3.81 | 4.15 | 4.52 | 4.92 | 5.34 |
| \(1.20\) | \(1.09\) | 1.14 | 1.19 | 1.25 | 1.32 | 1.40 | 1.49 | 1.58 | 1.70 | 1.84 | 2.02 | 2.22 | 2.44 | 2.68 | 2.95 | 3.25 | 3.58 | 3.94 | 4.34 | 4.79 | 5.28 | 5.83 | 6.43 |
| \(\geq2.2\) | \(2.40\) | 1.12 | 1.26 | 1.46 | 1.72 | 2.10 | 2.69 | 3.70 |
| \(2.60\) | \(0.60\) | 1.13 | 1.29 | 1.50 | 1.80 | 2.26 | 3.02 |
| \(0.80\) | \(0.80\) | 1.14 | 1.34 | 1.61 | 2.02 | 2.71 |
| \(1.00\) | \(1.00\) | 1.17 | 1.41 | 1.77 | 2.38 | 3.63 |

| \( \phi \) | \( \beta_\phi \) | 2 | 4 | 6 | 8 | 10 | 12 | 14 | 16 | 18 | 20 | 22 | 24 | 26 | 28 | 30 | 32 | 34 | 36 | 38 | 40 | 42 | 44 | 46 | 48 | 50 |
| \(<0.60\) | \(<0.40\) | 1.04 | 1.09 | 1.14 | 1.20 | 1.26 | 1.33 | 1.41 | 1.50 | 1.60 | 1.72 | 1.85 | 2.01 | 2.19 | 2.41 | 2.68 | 3.02 | 3.46 |
| \(0.60\) | \(1.05\) | 1.10 | 1.15 | 1.22 | 1.29 | 1.37 | 1.45 | 1.55 | 1.67 | 1.80 | 1.96 | 2.15 | 2.38 | 2.66 | 3.01 | 3.48 |
| \(0.80\) | \(1.05\) | 1.11 | 1.18 | 1.25 | 1.34 | 1.43 | 1.55 | 1.68 | 1.83 | 2.02 | 2.24 | 2.53 | 2.90 | 3.40 |
| \(1.00\) | \(1.06\) | 1.13 | 1.21 | 1.30 | 1.41 | 1.53 | 1.68 | 1.86 | 2.09 | 2.38 | 2.76 | 3.28 |
| \(1.20\) | \(1.07\) | 1.15 | 1.25 | 1.36 | 1.50 | 1.67 | 1.87 | 2.14 | 2.40 | 2.76 | 3.28 |
| \(\geq2.2\) | \(2.40\) | 1.17 | 1.41 | 1.77 | 2.38 | 3.63 |

### Total Factored Axial Load (kip)

\( P_{\text{factored}} = \phi \beta_\phi P_{\text{axial}} \)

Where:
- \( \phi \) is the load factor
- \( \beta_\phi \) is the magnification factor
- \( P_{\text{axial}} \) is the axial load

Note: The values in the table represent the total factored axial load for different load factors and magnification factors. The table is used to calculate the total axial load considering the load factor and the magnification factor for the given wall dimensions and concrete strength.
## Non-Sway Moment Magnifier for 6" Screen-Grid Walls

### Total Factored Axial Load (kip)

| \( \theta_0 \) | \( P_0 \) | 2 | 4 | 6 | 8 | 10 | 12 | 14 | 16 | 18 | 20 | 22 | 24 | 26 | 28 | 30 | 32 | 34 | 36 | 38 | 40 | 42 | 44 | 46 | 48 | 50 |
| 0.15       | 1.00    | 1.31 | 1.63 | 1.95 | 2.24 | 2.53 | 2.82 | 3.12 | 3.41 | 3.70 | 4.05 | 4.39 | 4.72 | 5.05 | 5.37 | 5.69 | 6.01 | 6.33 | 6.66 | 6.98 | 7.31 | 7.64 | 7.97 | 8.30 |
| 0.30       | 1.00    | 1.53 | 1.94 | 2.34 | 2.73 | 3.12 | 3.51 | 3.90 | 4.29 | 4.68 | 5.07 | 5.47 | 5.86 | 6.25 | 6.64 | 7.03 | 7.42 | 7.81 | 8.21 | 8.60 | 8.99 | 9.39 | 9.78 | 10.18 |
| 0.45       | 1.00    | 1.73 | 2.13 | 2.52 | 2.92 | 3.31 | 3.70 | 4.09 | 4.48 | 4.88 | 5.27 | 5.66 | 6.05 | 6.44 | 6.83 | 7.22 | 7.61 | 8.00 | 8.39 | 8.78 | 9.17 | 9.56 | 9.95 | 10.34 |
| 0.60       | 1.00    | 1.91 | 2.30 | 2.69 | 3.08 | 3.47 | 3.86 | 4.25 | 4.64 | 5.03 | 5.42 | 5.81 | 6.19 | 6.58 | 6.96 | 7.35 | 7.73 | 8.12 | 8.50 | 8.89 | 9.27 | 9.66 | 10.04 | 10.42 |
| 0.75       | 1.00    | 2.04 | 2.42 | 2.81 | 3.20 | 3.59 | 3.97 | 4.36 | 4.74 | 5.13 | 5.52 | 5.90 | 6.28 | 6.66 | 7.04 | 7.42 | 7.79 | 8.17 | 8.55 | 8.92 | 9.29 | 9.67 | 10.05 | 10.42 |
| 0.90       | 1.00    | 2.12 | 2.50 | 2.87 | 3.25 | 3.62 | 4.00 | 4.37 | 4.75 | 5.13 | 5.51 | 5.89 | 6.26 | 6.64 | 7.02 | 7.39 | 7.77 | 8.15 | 8.52 | 8.89 | 9.26 | 9.64 | 10.01 | 10.38 |
| 1.05       | 1.00    | 2.19 | 2.57 | 2.93 | 3.30 | 3.66 | 4.02 | 4.39 | 4.75 | 5.12 | 5.49 | 5.86 | 6.23 | 6.60 | 6.97 | 7.34 | 7.70 | 8.07 | 8.44 | 8.80 | 9.17 | 9.54 | 9.90 | 10.27 |
| 1.20       | 1.00    | 2.22 | 2.60 | 2.95 | 3.31 | 3.67 | 4.03 | 4.39 | 4.75 | 5.12 | 5.48 | 5.85 | 6.21 | 6.58 | 6.94 | 7.31 | 7.67 | 8.04 | 8.40 | 8.76 | 9.13 | 9.50 | 9.86 | 10.23 |
| 1.35       | 1.00    | 2.25 | 2.63 | 2.97 | 3.33 | 3.68 | 4.04 | 4.39 | 4.75 | 5.11 | 5.47 | 5.83 | 6.19 | 6.55 | 6.91 | 7.27 | 7.63 | 8.00 | 8.36 | 8.72 | 9.09 | 9.45 | 9.81 | 10.18 |
| 1.50       | 1.00    | 2.27 | 2.65 | 2.98 | 3.34 | 3.69 | 4.04 | 4.39 | 4.74 | 5.10 | 5.46 | 5.82 | 6.18 | 6.54 | 6.89 | 7.25 | 7.61 | 7.98 | 8.34 | 8.70 | 9.06 | 9.42 | 9.78 | 10.14 |

### Footnotes

- Moment Magnifiers
- For 6" Screen-Grid Walls
- \( P_0 = 4000 \text{ psi} \)
Interaction diagrams represent the relationship of the combined effects of axial load and bending moment on a concrete wall and are used as design aids to assist the designer with the selection of reinforcement. Points located within the interaction curve and the reference axes represent a combination of axial load and bending moment that the wall can support. The interaction diagram can be approximately constructed by connecting five basic points with straight lines (refer to Figure D-1).

1. Pure compression with zero bending moment (concrete compression failure)
2. Stress in reinforcement closest to the tension face $= 0$ (concrete compression failure)
3. Stress in reinforcement closest to the tension face $= 0.5$ times the yield stress (concrete compression failure)
4. Stress in reinforcement closest to the tension face $= \text{yield stress}$ (balanced concrete compression failure and reinforcement tensile yielding)
5. Pure bending with zero axial load (under-reinforced with ductile reinforcement tensile failure)

Due to the low structural loads that exist in residential-scale structures, the interaction diagrams in Appendix D represent only the portion of the curve bounded by points (4) and (5). The
Interaction Diagrams for Structural Reinforced Concrete Walls

reinforcement ratios are calculated in Appendix H to assist the designer and are based on the gross area of the concrete equivalent rectangular section. To construct interaction diagrams representing as many ICF manufacturers as possible, the interaction diagrams were determined using the dimensions shown in Figure D-2; the reinforcement is assumed to be placed in the center of the wall.

<table>
<thead>
<tr>
<th>ICF Wall Type</th>
<th>Nominal Thickness (inch)</th>
<th>Minimum Equivalent Thickness (h) (inch)</th>
<th>Minimum Equivalent Thickness (h) (mm)</th>
<th>Minimum Equivalent Width (b) (inch)</th>
<th>Minimum Equivalent Width (b) (mm)</th>
<th>Vertical Core Spacing (inch)</th>
<th>Vertical Core Spacing (mm)</th>
</tr>
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<tbody>
<tr>
<td>Flat</td>
<td>4</td>
<td>101.6</td>
<td>3.5</td>
<td>88.9</td>
<td>12.0</td>
<td>304.8</td>
<td>N.A.</td>
</tr>
<tr>
<td></td>
<td>6</td>
<td>152.4</td>
<td>5.5</td>
<td>139.7</td>
<td>12.0</td>
<td>304.8</td>
<td>N.A.</td>
</tr>
<tr>
<td></td>
<td>8</td>
<td>203.2</td>
<td>7.5</td>
<td>190.5</td>
<td>12.0</td>
<td>304.8</td>
<td>N.A.</td>
</tr>
<tr>
<td></td>
<td>10</td>
<td>254.0</td>
<td>9.5</td>
<td>241.3</td>
<td>12.0</td>
<td>304.8</td>
<td>N.A.</td>
</tr>
<tr>
<td>Waffle-Grid</td>
<td>6</td>
<td>152.4</td>
<td>5.0</td>
<td>127.0</td>
<td>6.25</td>
<td>158.8</td>
<td>12.0</td>
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<td>203.2</td>
<td>7.0</td>
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<td>5.5</td>
<td>139.7</td>
<td>5.5</td>
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<td>12.0</td>
</tr>
</tbody>
</table>

As a result of constructing interaction diagrams representative of current ICF manufacturers, some design efficiency has been sacrificed, particularly in the screen-grid wall type because of the variety of dimensions available. Therefore, the designer may wish to construct custom interaction diagrams to obtain a more efficient design. The coordinates of each point on the interaction diagram may be computed using the following equations:
Point 1: Pure Compression

\[ \phi P_n = \phi (C_c \cdot C_s) \]
\[ \phi P_{n,\text{max}} = 0.8 \phi P_n \]
\[ \phi M_n = 0 \]
\[ C_c = 0.85 f'_c \left( A_g - A_s \right) \]
\[ C_s = A_s f_y \]

Point 2: Stress in Reinforcement = 0

\[ \phi P_n = \phi (C_c) \]
\[ \phi M_n = \phi C_c \left( d - \frac{a}{2} \right) \]
\[ C_c = 0.85 a b f'_c \]
\[ a = \beta c \]
\[ c = d \]

Point 3: Stress in Reinforcement = 0.5\( f'_c \)

\[ \phi P_n = \phi (C_c - T_s) \]
\[ \phi M_n = \phi C_c \left( d - \frac{a}{2} \right) \]
\[ T_s = \tau_s (0.5 f'_c) \]
\[ a = \beta c \]
\[ c = \left( \frac{\frac{\varepsilon_c}{\varepsilon_c + 0.5\varepsilon_s}}{d} \right) \]
\[ \varepsilon_y = \frac{f'_y}{E_s} \]
\[ \varepsilon_c = 0.903 \]
Point 4: Balanced Condition

\[ \phi P_x = \phi (C_c - T_s) \]
\[ \phi M_n = \phi C_c (d - a/2) \]
\[ T_s = A_s f_y \]
\[ C_c = 0.85 a b f_y \]
\[ a = \beta c \]
\[ c = \left( \frac{\varepsilon_c}{\varepsilon_c + \varepsilon_y} \right) d \]
\[ \varepsilon_y = \frac{f_y}{E_s} \]
\[ \varepsilon_c = 0.003 \]

Point 5: Pure Bending

\[ \phi P_{ec} = 0 \]
\[ \phi M_n = \phi A_s f_y \left( d - \frac{a}{2} \right) \]
\[ a = \frac{A_s f_y}{0.85 f_c b} \]

where:
- \( \beta \) Factor 0.85 for \( f_{c'} \leq 4000 \) psi per ACI 10.2.7.3
- \( \varepsilon_c \) Strain in concrete
- \( \varepsilon_y \) Yield strain of tensile reinforcement
- \( \phi \) Strength reduction factor 0.7 for combined axial load and flexure per ACI 9.3.2 and 0.9 for pure flexure per ACI 9.3.2
- \( a \) Depth of equivalent rectangular stress block per ACI 10.2.7.1
- \( A_g \) Gross area of concrete section
- \( A_s \) Area of tensile reinforcement
- \( b \) Width of compression face. Refer to Figure 1-3
- \( c \) Distance from extreme compression fiber to neutral axis
- \( f_c \) Compression force in concrete
- \( f_y \) Compression force in reinforcement
- \( d \) Distance from extreme compression fiber to centroid of tension reinforcement
- \( E_s \) Modulus of elasticity of reinforcement 29,000,000 psi per ACI 8.5.2
- \( f_{c'} \) Specified compressive strength of concrete
- \( f_y \) Specified yield strength of reinforcement
- \( h \) Thickness of concrete wall. Refer to Figure 1-1
<table>
<thead>
<tr>
<th>Symbol</th>
<th>Description</th>
<th>Unit</th>
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<td>Nominal flexural strength</td>
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<tr>
<td>$P_n$</td>
<td>Nominal axial load strength</td>
<td>lb</td>
</tr>
<tr>
<td>$P_{n,max}$</td>
<td>Maximum nominal axial load strength</td>
<td>lb</td>
</tr>
<tr>
<td>$T_s$</td>
<td>Tensile force in reinforcement</td>
<td>lb</td>
</tr>
</tbody>
</table>
Interaction Diagrams for Structural Reinforced Concrete Walls

4" Flat Wall (f_c = 3 ksi, f_y = 60 ksi)

\( (\sigma_d) \|\beta_d \phi \)

\( \phi M_{in} \) (ft-kip)
Interaction Diagrams for Structural Reinforced Concrete Walls

4" Flat Wall (f_c = 4 ksi, f_y = 60 ksi)

Structural Design of Insulating Concrete Form Walls in Residential Construction
Interaction Diagrams for Structural Reinforced Concrete Walls

6" Flat Wall (f_c = 3 ksi, f_y = 40 ksi)

(dp) u, f

φN_0 (fs=kip)

50 35 30 25 20 15 10 5 0

6.0 5.0 4.0 3.0 2.0 1.0 0.0
Interaction Diagrams for Structural Reinforced Concrete Walls

6" Flat Wall ($f_c = 3$ ksi, $f_y = 60$ ksi)
6" Flat Wall (f'_c = 4 ksi, f_y = 40 ksi)
Interaction Diagrams for Structural Reinforced Concrete Walls

8" Flat Wall ($f_c = 3$ ksi, $f_y = 60$ ksi)
Interaction Diagrams for Structural Reinforced Concrete Walls

8" Flat Wall ($f_c = 4$ ksi, $f_y = 60$ ksi)

$(drx) u_{xy}$

$\phi M_n$ (k-ft)

Structural Design of Insulating Concrete Form Walls in Residential Construction
Interaction Diagrams for Structural Reinforced Concrete Walls

10" Flat Wall (C = 3 ksi, f_y = 40 ksi)
10" Flat Wall ($f'_c = 3$ ksi, $f_y = 60$ ksi)
Interaction Diagrams for Structural Reinforced Concrete Walls

10" Flat Wall (fc = 4 ksi, fy = 40 ksi)

(dp)¹dφ

Structural Design of Insulating Concrete Form Walls in Residential Construction
Interaction Diagrams for Structural Reinforced Concrete Walls

10" Flat Wall (C = 4 ksi, f_y = 60 ksi)
Interaction Diagrams for Structural Reinforced Concrete Walls

6" Waffle Grid Wall (f'_c = 3 ksi, f_y = 40 ksi)
6" Waffle Grid Wall ($f_c = 3$ ksi, $f_y = 60$ ksi)

Interaction Diagrams for Structural Reinforced Concrete Walls

Structural Design of Insulating Concrete Form Walls in Residential Construction
Interaction Diagrams for Structural Reinforced Concrete Walls

6" Waffle Grid Wall (f_c = 4 ksi, f_y = 40 ksi)
Interaction Diagrams for Structural Reinforced Concrete Walls

8" Waffle Grid Wall ($f_c = 3$ ksi, $f_y = 60$ ksi)

Structural Design of Insulating Concrete Form Walls in Residential Construction
Interaction Diagrams for Structural Reinforced Concrete Walls

8" Waffle Grid Wall (f_c = 4 ksi, f_y = 40ksi)
Interaction Diagrams for Structural Reinforced Concrete Walls

8" Waffle Grid Wall (f_c = 4 ksi, f_y = 60 ksi)
6" Screen Grid Wall ($f_c = 3$ ksi, $f_y = 40$ ksi)
6" Screen Grid Wall (t = 3 ksi, $f_y = 60$ ksi)
Interaction Diagrams for Structural Reinforced Concrete Walls

6" Screen Grid Wall (E = 4 ksi, f = 60 ksi)

Structural Design of Insulating Concrete Form Walls in Residential Construction
Interaction diagrams represent the relationship of the combined effects of axial load and bending moment on a concrete wall and are used as design aids to assist the designer in determining the wall's structural adequacy. Points located within the interaction curve for a given wall height and the reference axes represent a combination of axial load and bending moment that the wall can support. To construct interaction diagrams representing as many ICF manufacturers as possible, the interaction diagrams were determined using the dimensions shown in Figure E-1.

### Non-Sway Moment Magnifier Tables

<table>
<thead>
<tr>
<th>ICF Wall Type</th>
<th>Nominal Thickness</th>
<th>Minimum Equivalent Thickness (h)</th>
<th>Minimum Equivalent Width (b)</th>
<th>Vertical Core Spacing</th>
</tr>
</thead>
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<tr>
<td></td>
<td>(inch)</td>
<td>(inch)</td>
<td>(inch)</td>
<td>(inch)</td>
</tr>
<tr>
<td>Flat</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
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<td>5.0</td>
<td>127.0</td>
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<tr>
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<td>Screen-Grid</td>
<td>6</td>
<td>152.4</td>
<td>5.5</td>
<td>139.7</td>
</tr>
</tbody>
</table>

**Figure E-1 Dimensions Used for Interaction Diagrams for Structural Plain Concrete Walls**

As a result of constructing interaction diagrams representative of current ICF manufacturers, some design efficiency has been sacrificed, particularly in the screen-grid wall type because of the variety of dimensions available. The designer may wish to construct custom interaction diagrams to obtain a more efficient design. The interaction diagram is constructed using the following two linear equations in accordance with ACI 22.5.3:

\[
\frac{P_n}{A_g} + \frac{M_u}{\phi M_n} \leq 1 \quad \text{on the compression face}
\]

\[
\frac{M_u}{S} - \frac{P_n}{A_g} \leq 5\phi \sqrt{f_c} \quad \text{on the tension face}
\]

\[
P_n = 0.6 f_c \left[1 - \left(\frac{l_c}{32 h}\right)^2\right] A_g
\]

\[
M_n = 0.85 f_c \cdot S
\]

\[
M_{u, min} = 0.1 h P_u
\]

where:

- $\phi$ = Strength reduction factor = 0.65 per ACI 9.3.5
- $A_g$ = Gross area of concrete section
- $f_c$ = Specified compressive strength of concrete
- $h$ = Thickness of wall, Refer to Figure E-2
- $l_c$ = Vertical distance between supports
- $M_n$ = Nominal moment strength at section
- $M_u$ = Factored moment at section
- $M_{u, min}$ = Minimum factored moment per ACI 22.6.3
Interaction Diagrams for Structural Plain Concrete Walls

\[ P_n \quad \text{Nominal axial load strength at section} \quad \text{lb} \]
\[ P_u \quad \text{Factored axial load at section} \quad \text{lb} \]
\[ S \quad \text{Elastic section modulus of section} \quad \text{inch}^3 \]

Figure E-2 Variables Defined for Interaction Diagrams for Structural Plain Walls
Interaction Diagrams for Structural Plain Concrete Walls

4" Flat Wall ($f_c = 4$ ksi)

$d_{EF}$ vs. $d_{PL}$

Structural Design of Insulating Concrete Form Walls in Residential Construction
Interaction Diagrams for Structural Plain Concrete Walls

6" Flat Wall ($f_c = 3$ ksi)

(dps) $H e f l e t$
Interaction Diagrams for Structural Plain Concrete Walls

6" Flat Wall ($f_c = 4$ ksi)

\[ \phi M_n (\text{ft-kip}) \]

\[ (dpt)^{4/3} \phi \]

---

Structural Design of Insulating Concrete Form Walls in Residential Construction

171
8" Flat Wall ($f'_c = 4$ ksi)

Graph showing interaction between $P_n$ (kip) and $M_n$ (ft-kip) for different wall heights.
Interaction Diagrams for Structural Plain Concrete Walls

10" Flat Wall ($C_c = 3$ ksi)

$(d_{pl}) u \cdot d \phi$

$\Phi M_n$ (k-ft)

8 feet, 10 feet, 12 feet, 14 feet, 16 feet, 18 feet, 20 feet

Structural Design of Insulating Concrete Form Walls in Residential Construction
Interaction Diagrams for Structural Plain Concrete Walls

6" Waffle-Grid Wall (f'c = 3 ksi)

\( \phi M_n \) (ft-kip)

\( (d_H) u_d \phi \)

8 Feet

10 Feet

Structural Design of Insulating Concrete Form Walls in Residential Construction
Interaction Diagrams for Structural Plain Concrete Walls

8" Waffle-Grid Wall ($f_c = 3$ ksi)

$\Phi M_n$ (ft-kip)

$(dpp)^n d \Phi$

Structural Design of Insulating Concrete Form Walls in Residential Construction
Interaction Diagrams for Structural Plain Concrete Walls

8" Waffle-Grid Wall (f_c = 4 ksi)

(dpa)^1/3 \phi

Structural Design of Insulating Concrete Form Walls in Residential Construction 179
Interaction Diagrams for Structural Plain Concrete Walls

6" Screen Grid Wall (f_c = 3 ksi)

(\text{dpr}) \text{ U}_d \phi

\phi \text{ M}_{\text{r}} (\text{ft-lb})

Structural Design of Insulating Concrete Form Walls in Residential Construction
APPENDIX F

CONVERSION FACTORS
<table>
<thead>
<tr>
<th>Angle</th>
<th>Degree</th>
<th>deg</th>
<th>1.745329 E-02</th>
<th>Radian</th>
<th>rad</th>
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<tr>
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<tr>
<td>Bending-Moment</td>
<td>Inch-Pound per Linear Inch</td>
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<td>Newton-Meter per Meter</td>
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1 Pascal = 1000 N/m²

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<td>G</td>
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<td>10⁵</td>
<td>10⁴</td>
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<td>10¹</td>
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Structural Design of Insulating Concrete Form Walls in Residential Construction
## Conversion Factors

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1 Pascal = 1000 N/m²

### SI PREFIXES

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<th>Tera</th>
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### SI PREFIXES

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Structural Design of Insulating Concrete Form Walls in Residential Construction
APPENDIX G

WEIGHTS OF COMMON BUILDING MATERIALS
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<td>Gypsum Sheathing, 1&quot;</td>
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<td>Wood Shingles</td>
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<td>Wood Roof Framing</td>
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Structural Design of Insulating Concrete Form Walls in Residential Construction
## Standard Reinforcing Bar Data

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<th>Diameter (In.)</th>
<th>Diameter (mm)</th>
<th>Cross-Sectional Area (In²)</th>
<th>Cross-Sectional Area (mm²)</th>
<th>Unit Weight (lb/ft)</th>
<th>Unit Weight (kg/m)</th>
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### Reinforcement Ratios

#### ICF Wall Type

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<td>6&quot; Flat (152 mm)</td>
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#### Reinforcement Ratios

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<td>6&quot; Flat (152 mm)</td>
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Structural Design of Insulating Concrete Form Walls in Residential Construction 193
APPENDIX I

SYMBOLS
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<td>( \Delta_{\text{actual}} )</td>
<td>Actual deflection</td>
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<td>Allowable deflection</td>
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<td>( \Delta_{\text{max}} )</td>
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<td>Moment magnification factor for non-sway frames</td>
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<td>( \Delta_x )</td>
<td>Deflection at distance ( x )</td>
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<td>( \varepsilon_c )</td>
<td>Strain in concrete</td>
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<td>( \varepsilon_y )</td>
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<td>( \phi )</td>
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<td>( \gamma )</td>
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<td>( \lambda )</td>
<td>Correction factor related to unit weight of concrete or concrete type factor</td>
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<td>( \mu )</td>
<td>Coefficient of friction</td>
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<td>Bearing strength of reinforcement</td>
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<td>Reinforcement spacing or concrete cover</td>
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<td>( C_c )</td>
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<td>lb</td>
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<tr>
<td>( C_m )</td>
<td>Factor relating moment diagram to equivalent uniform moment diagram</td>
<td>dimensionless</td>
</tr>
<tr>
<td>( C_s )</td>
<td>Compression force in reinforcement</td>
<td>lb</td>
</tr>
<tr>
<td>( d )</td>
<td>Distance from extreme compression fiber to centroid of</td>
<td>inch</td>
</tr>
<tr>
<td>( d_b )</td>
<td>Diameter of reinforcement</td>
<td>inch</td>
</tr>
<tr>
<td>( e )</td>
<td>Eccentricity of axial load</td>
<td>inch</td>
</tr>
<tr>
<td>( E_c )</td>
<td>Modulus of elasticity of concrete</td>
<td>psi</td>
</tr>
</tbody>
</table>

Structural Design of Insulating Concrete Form Walls in Residential Construction
Symbols

<table>
<thead>
<tr>
<th>Symbol</th>
<th>Description</th>
<th>Unit</th>
</tr>
</thead>
<tbody>
<tr>
<td>$EI$</td>
<td>Flexural stiffness of compression member</td>
<td>psi</td>
</tr>
<tr>
<td>$E_s$</td>
<td>Modules of elasticity of reinforcement</td>
<td>psi</td>
</tr>
<tr>
<td>$f_b$</td>
<td>Actual bending stress</td>
<td>psi</td>
</tr>
<tr>
<td>$F_b$</td>
<td>Allowable bending stress</td>
<td>psi</td>
</tr>
<tr>
<td>$f_{c_t}$</td>
<td>Actual compressive stress perpendicular to the grain</td>
<td>psi</td>
</tr>
<tr>
<td>$F_{c_t}$</td>
<td>Allowable compressive stress perpendicular to the grain</td>
<td>psi</td>
</tr>
<tr>
<td>$f_c$</td>
<td>Specified compressive strength of concrete</td>
<td>psi</td>
</tr>
<tr>
<td>$f_t$</td>
<td>Actual tensile stress</td>
<td>psi</td>
</tr>
<tr>
<td>$F_t$</td>
<td>Allowable tensile stress</td>
<td>psi</td>
</tr>
<tr>
<td>$f_y$</td>
<td>Actual shear stress</td>
<td>psi</td>
</tr>
<tr>
<td>$F_y$</td>
<td>Allowable shear stress</td>
<td>psi</td>
</tr>
<tr>
<td>$f_v$</td>
<td>Specified yield strength of steel reinforcement</td>
<td>psi</td>
</tr>
<tr>
<td>$h$</td>
<td>Thickness</td>
<td>inch</td>
</tr>
<tr>
<td>$I_g$</td>
<td>Moment of inertia of gross concrete section</td>
<td>inch$^4$</td>
</tr>
<tr>
<td>$k$</td>
<td>Effective length factor</td>
<td>dimensionless</td>
</tr>
<tr>
<td>$K_{TR}$</td>
<td>Transverse reinforcement index</td>
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<tr>
<td>$l_b$</td>
<td>Embedment length</td>
<td>inch</td>
</tr>
<tr>
<td>$l_{be}$</td>
<td>Distance from fastener to nearest edge of concrete ledge</td>
<td>inch</td>
</tr>
<tr>
<td>$l_d$</td>
<td>Development length of reinforcement bar</td>
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</tr>
<tr>
<td>$l_{db}$</td>
<td>Basic development length of reinforcement bar</td>
<td>inch</td>
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<tr>
<td>$l_{dh}$</td>
<td>Development length of reinforcement hook</td>
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</tr>
<tr>
<td>$l_{hb}$</td>
<td>Basic development length of reinforcement hook</td>
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</tr>
<tr>
<td>$l_u$</td>
<td>Unsupported length</td>
<td>inch</td>
</tr>
<tr>
<td>$l_w$</td>
<td>Length of wall</td>
<td>inch</td>
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<tr>
<td>$M_1$</td>
<td>Smaller factored end moment</td>
<td>inch-lbs</td>
</tr>
<tr>
<td>$M_2$</td>
<td>Larger factored end moment</td>
<td>inch-lbs</td>
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<tr>
<td>$M_{2,min}$</td>
<td>Minimum permissible value of $M_2$</td>
<td>in-lb</td>
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<tr>
<td>$M_n$</td>
<td>Nominal moment strength</td>
<td>in-lb</td>
</tr>
<tr>
<td>$M_{ns}$</td>
<td>Magnified factored moment to be used for designing compression</td>
<td>in-lb</td>
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<tr>
<td>$M_S$</td>
<td>Magnified factored moment to be used for designing compression</td>
<td>in-lb</td>
</tr>
<tr>
<td>$M_u$</td>
<td>Factored bending moment</td>
<td>in-lb</td>
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<tr>
<td>$M_{u, min}$</td>
<td>Minimum factored bending moment</td>
<td>inch-lb</td>
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<tr>
<td>$NAX$</td>
<td>Neutral axis about the x axis</td>
<td>inch</td>
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<tr>
<td>$P_c$</td>
<td>Critical buckling load</td>
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<tr>
<td>$P_u$</td>
<td>Factored axial load</td>
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</tr>
<tr>
<td>$P_{u, dead}$</td>
<td>Factored axial dead load</td>
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</tr>
<tr>
<td>$\Sigma P_c$</td>
<td>Summation for all sway-resisting columns</td>
<td>lb</td>
</tr>
<tr>
<td>$\Sigma P_u$</td>
<td>Summation for all the vertical loads</td>
<td>lb</td>
</tr>
<tr>
<td>$r$</td>
<td>Radius of gyration of cross-section</td>
<td>inch</td>
</tr>
<tr>
<td>$s$</td>
<td>Spacing of reinforcement</td>
<td>inch</td>
</tr>
<tr>
<td>$s_2$</td>
<td>Spacing of reinforcement</td>
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<tr>
<td>$S_{xx}$</td>
<td>Section modulus of section about the x-x axis</td>
<td>inch$^3$</td>
</tr>
<tr>
<td>$S_{yy}$</td>
<td>Section modulus of section about the y-y axis</td>
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<tr>
<td>$T$</td>
<td>Tensile force</td>
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<tr>
<td>$T_{uplift}$</td>
<td>Tensile force due to uplift</td>
<td>lb</td>
</tr>
<tr>
<td>Symbol</td>
<td>Description</td>
<td>Unit</td>
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<td>--------</td>
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<tr>
<td>$V$</td>
<td>Shear force</td>
<td>lb</td>
</tr>
<tr>
<td>$V_C$</td>
<td>Nominal shear strength of concrete</td>
<td>lb</td>
</tr>
<tr>
<td>$V_n$</td>
<td>Nominal shear strength</td>
<td>lb</td>
</tr>
<tr>
<td>$V_S$</td>
<td>Nominal shear strength provided by shear reinforcement</td>
<td>lb</td>
</tr>
<tr>
<td>$V_U$</td>
<td>Factored shear force</td>
<td>lb</td>
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<tr>
<td>$w_C$</td>
<td>Weight of concrete</td>
<td>pcf</td>
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<tr>
<td>$Z_{actual}$</td>
<td>Actual bolt shear</td>
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</tr>
<tr>
<td>$Z_{allowable}$</td>
<td>Allowable bolt shear</td>
<td>lb</td>
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</tbody>
</table>
axial load ........................................ 2, 7, 8, 10, 11, 13, 17,
.............................................. 24, 28, 31, 36, 42, 46,
.............................................. 50, 52, 54, 57, 64, 65,
.............................................. 102, 103, 104, 105, 131,
.............................................. 134, 135, 166, 167, 197, 198
bending moments .................................. 10, 19, 20, 21, 22, 24,
.............................................. 26, 28, 29, 30, 31, 34,
.............................................. 35, 36, 37, 63, 68,
.............................................. 71, 79, 80, 81, 87, 91, 197
bearing strength .................................. 19, 20, 26, 29, 30, 37,
.............................................. 80, 81, 91, 197
bending moment ................................... 31, 36
block ............................................. 1, 16, 36, 134, 197
bolt .............................................. 24, 25, 26, 32, 33, 34,
.............................................. 76, 78, 80, 83, 84, 85,
.............................................. 86, 88, 197, 199
cavity ............................................ 36
coefficient of friction .................................. 74, 84
core ............................................ 2, 4, 6, 10, 13, 16, 17,
.............................................. 52, 58, 79, 80, 87, 103,
.............................................. 104, 105, 131, 134, 197, 198
corative strength .................................. 2, 4, 5, 7, 11, 12, 13, 16,
.............................................. 17, 19, 20, 21, 23, 26, 27, 29,
.............................................. 30, 33, 37, 51, 57, 63, 68, 71,
.............................................. 90, 103, 104, 105, 134, 166, 198
cement type ..................................... 10
contraction ....................................... 10
contraction joint ................................ 35
core ............................................ 36, 42, 46, 53, 54, 59, 60, 67, 83, 90
derign ......................................... iii, v, 1, 2, 3, 10, 11, 13, 15, 19,
.............................................. 20, 24, 28, 30, 31, 35, 36,
.............................................. 41, 42, 49, 51, 53, 57, 59,
.............................................. 63, 64, 68, 71, 74, 76, 79,
.............................................. 80, 81, 83, 86, 87, 88,
.............................................. 90, 91, 102, 131, 132, 166
development length ................................ 20, 21, 22, 23, 198
dowels ........................................... 19, 20, 21, 22, 23
eccentricity ..................................... 7, 46, 103, 104
embdenent length ..................................... 37
empirical design method .......................... 2, 10
flat ........................................... 1, 2, 6, 7, 11, 23, 42, 79, 86, 101
floor ............................................ 15, 16, 17, 31, 32, 33, 34,
.............................................. 35, 36, 37, 83, 86, 87, 88, 90, 91
floor connection .................................. 35
floor joist ....................................... 31, 32, 35, 36, 37, 83, 87, 88, 90
footing ......................................... 6, 19, 20, 21, 22, 23, 74, 103
gird .............................................. 1, 2, 1, 2, 3, 5, 6, 7,
.............................................. 11, 16, 17, 23, 36, 42, 46,
.............................................. 51, 52, 56, 57, 58, 61, 63,
.............................................. 64, 65, 68, 71, 83, 90,
.............................................. 101, 132, 166
horizontal ....................................... 1, 2, 5, 8, 10, 14, 22, 49, 53, 59, 67
isolation joint .................................. 10
j-bolt ............................................ 26, 31, 32
key .............................................. 19, 21, 23
ledger ........................................... 31, 32, 34, 36, 83, 84, 85, 86, 87, 88, 90
load cases ...................................... 3, 11
location factor ................................... 22
moment .......................................... 2, 6, 7, 8, 9, 11, 13, 15, 16,
.............................................. 52, 57, 64, 65, 67, 69, 101, 102,
.............................................. 103, 104, 105, 131, 166, 197, 198
moment magnifier .................................. 6, 7, 8, 64, 65, 101, 102
moment strength .................................. 16, 69, 166, 198
National Building Code .......................... 10, 14, 38
openings ........................................ 5, 6, 8, 10, 13, 14, 46,
.............................................. 51, 53, 56, 57, 59, 62, 67
panel ............................................. I-1
parallel shear ................................... 5, 12, 46, 49, 51, 57, 91
perpendicular shear .................................. 3, 11, 51, 57, 63, 74
plank ............................................ 1-1
pocket connection ................................ 36, 90
post-and-beam .................................. I-1, 1-2
radius of gyration .................................. 63
reinforcement ................................... 2, 3, 4, 5, 7, 8, 9,
.............................................. 10, 11, 13, 14, 15, 16, 17, 19,
.............................................. 20, 21, 22, 23, 33, 53, 56, 59,
.............................................. 61, 62, 63, 65, 66, 67, 68, 71,
Index

reinforcement ratio ........................................ 2, 17, 102, 132
roof .............................................................. 6, 15, 17, 24, 26, 28, 35,
............................................................ 46, 49, 76, 78, 80, 81, 103
screen ...................................................... 1, 2, 3, 5, 6, 7, 11, 16, 17,
.............................................................. 23, 36, 101, 132, 166
section modulus ............................................ 167
shear .................................................. 3, 4, 5, 11, 12, 13, 15, 17, 19,
............................................................ 21, 22, 23, 24, 25, 26, 28, 29,
............................................................ 31, 32, 33, 34, 36, 46, 49,
............................................................ 51, 57, 63, 69, 72, 74, 76, 78,
............................................................ 80, 81, 83, 84, 85, 86, 88, 91,
............................................................ 197, 198, 199
shear strength ........................................... 4, 5, 12, 13, 17, 23, 26, 29,
............................................................ 33, 69, 72, 76, 83, 198, 199
side cover factor ........................................... 22, 197
sill plate .............................................. 24, 26, 28, 35, 76, 77, 78, 79, 80
size factor .................................................. 22, 79, 86, 197
slenderness .................................................. 6, 64, 101
spacing ...................................................... 2, 8, 14, 24, 26, 32, 34, 37,
............................................................ 63, 80, 81, 83, 87, 88, 90, 91, 197
stirrups ...................................................... 3, 17, 18
strap ......................................................... 28, 29, 31, 37, 81, 87, 90
structural plain concrete .................. 10, 13, 14, 51, 52,
........................................................... 53, 57, 58, 59
structural reinforced concrete ............. 2, 53, 59, 63, 101
sway ...................................................... 6, 7, 8, 54, 60, 63, 101, 102,
........................................................... 103, 104, 105, 197, 198
tensile capacity ........................................... 24, 34
type factor ................................................... 22, 197
uplift ...................................................... 24, 25, 26, 28, 29, 76, 78, 79, 81
waffle ....................................................... 1, 2, 3, 5, 6, 7, 11, 16, 17,
........................................................... 23, 36, 42, 46, 51, 52, 56, 57, 58,
........................................................... 61, 63, 64, 65, 68, 71, 83, 90, 101
web width ................................................... 17

202 Structural Design of Insulating Concrete Form Walls in Residential Construction