### Table 11

<table>
<thead>
<tr>
<th>Roof Slope</th>
<th>Fastest Mile Wind Speed (MPH)</th>
<th>80</th>
<th>90</th>
<th>100</th>
<th>110</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>W_{ref}</td>
<td>W_{l}</td>
<td>W_{cr}</td>
<td>W_{cr}</td>
</tr>
<tr>
<td>h</td>
<td></td>
<td>105</td>
<td>130</td>
<td>169</td>
<td>205</td>
</tr>
<tr>
<td>3</td>
<td></td>
<td>106</td>
<td>132</td>
<td>169</td>
<td>205</td>
</tr>
<tr>
<td>5</td>
<td></td>
<td>130</td>
<td>153</td>
<td>190</td>
<td>227</td>
</tr>
<tr>
<td>6</td>
<td></td>
<td>141</td>
<td>164</td>
<td>201</td>
<td>238</td>
</tr>
<tr>
<td>7</td>
<td></td>
<td>152</td>
<td>175</td>
<td>212</td>
<td>250</td>
</tr>
<tr>
<td>8</td>
<td></td>
<td>163</td>
<td>186</td>
<td>223</td>
<td>261</td>
</tr>
<tr>
<td>10</td>
<td></td>
<td>187</td>
<td>210</td>
<td>247</td>
<td>285</td>
</tr>
<tr>
<td>12</td>
<td></td>
<td>213</td>
<td>236</td>
<td>273</td>
<td>311</td>
</tr>
</tbody>
</table>

1. Table based on ASCE 7-93 wind requirements for MWFRS.
2. H₁ = 9FT based on 8FT floor to ceiling weight. If 10FT floor to ceiling height, multiply Table values by 1.25.
3. L = 60 FT was used for Tabled values.
This companion manual has been written for the sole purpose of mitigating storm damage from high wind conditions in residential light frame construction. The original manual, *Windstorm Mitigation Manual for Light Frame Construction*, explained the wind resistance philosophy and revealed typical wind related problems. This manual emphasizes structural planning principles that are essential to minimizing the cost of wind resistant wood frame construction. It also introduces tables and charts to assist architects, contractors and owners in developing a wind resistant system for generic house construction. The manual has taken a graphics approach to illustrate technically difficult concepts and engineering design techniques. The companion manual is written for the layperson, rather than the academic; the contractor rather than the engineer.

The information contained in chapter 1 is intended to establish the difference between tornado and straight-line winds. It explains a comprehensive wind design program initiated by the Building Safety Division of the City of Urbana, IL. The program was intended to educate the public, the residential construction industry and the design professionals within the community about the importance of wind resistant wood framed construction. It charts the cost of wind resistant construction for two distinctly different categories of home.

Chapter 2 reviews the structural planning issues for resistance to wind uplift which is the tension chain from roof to foundation. Two techniques are emphasized: total structural panel approach and total metal connector approach.

Chapter 3 reviews the structural planning issues related to the stability of the residence subjected to wind load, overturning and sliding. It introduces the concept of a diaphragm, a shear wall and a drag strut. This Chapter illustrates and explains the behavior of each stability issue and why each presents a problem in construction. The Chapter tells how to design to resist the effects of overturning and sliding. It conceptually reviews two examples to further clarify the stability concepts.

Chapter 4 develops a detailed wind uplift design procedure with illustrations. The procedure is for a generic house design. A numerical example is worked to aid familiarity with the procedure, and employs use of Appendix A Tables. The Tables work with this chapter to minimize much of the actual mathematics.

Chapter 5 develops a detailed wind stability design procedure, again with illustrations, to explain the Tables found in Appendix B. A numerical example is worked in total, incorporating all of the tables of Appendix B.
Selected references are found at the end of each Chapter, and nomenclature for use of the Appendices is also found at the end of Chapters 4 and 5.

Incorporation of the concepts, structural planning advice, design procedures and worked examples permit owners, architects, building officials and contractors to become conversant with wind resistant design. The use of this manual provides confidence to perform the wind design procedures by following the examples included. This manual will enhance the structural capability of the residence and provide a greater chance of mitigating structural damage from straight-line winds. The cost to incorporate these details is in the range of 3%-5% of the cost of new residential construction. A small price to pay for the potential benefit of security and safety.

A “quick start” for readers wanting to go right to the design solutions would be to read Chapters 2 and 3, and then move into Chapters 4 and 5 to use the design procedures and the tables from both Appendices.
Acknowledgements

This Manual was prepared through the Building Research Council (BRC) of the School of Architecture at the University of Illinois at Urbana/Champaign under contract to and sponsored by the Illinois Emergency Management Agency (IEMA), State Farm Insurance (SFI) and the Simpson Strong-Tie Company, Inc.

Special thanks for counsel and review of the draft versions of this manual are extended to Jan Horton, State Hazard Mitigation Officer for IEMA, Larry Sanders, Federal Hazard Mitigation Officer for FEMA, Phil Line, Staff Engineer for the American Forest & Paper Association and Randall Shackelford, P.E. and research Engineer for Simpson Strong-Tie Co., Inc.

Special appreciation is extended to the Staff at the BRC for responding to equipment and material needs, mailing and contract coordination. Particular recognition should go to Selah Peterson, graphic designer, for her dedicated work in converting my sketches into beautiful computer generated hard-line drawings, and for continuing to teach me how to edit and produce my own drawings. Special thanks for coordination of the contract go to Bill Rose, research architect.

I must also thank Joseph R. Hetzel, P.E., and Technical Director of the Door & Access Systems Manufacturers Association International (DASMA) for providing the garage door wind Tables.

Thanks go to all the individuals who reviewed drafts of this document.

David Wickersheimer, S.E., Architect, Professor of Architecture,
President of Wickersheimer Engineers. Inc.
December 31, 1999
# Table of Contents

**Forward** .......................................................... i
**Acknowledgements** ........................................ iii
**List of Illustrations** .......................................... viii
  **Figures** ....................................................... viii
  **Tables** ........................................................ xii
**List of Acronyms** ............................................. xiv

## Chapter 1 Wind Basics Revisited

1. Introduction ........................................................................... 1
2. Straight-line Winds vs. Tornadoes ........................................ 1
   - Straight-line Winds ...................................................... 1
   - Tornado Winds .................................................................. 4
   - Reasonable Expectations of Wind Resistant Design ........... 5
   - Wind Resistance Construction Program .......................... 5
   - Wind Speed Map: A Footnote ........................................ 8
3. Conclusions ........................................................................ 13
4. Selected References .......................................................... 14

## Chapter 2 Wind Structural Planning Basics—The Tension Chain

1. Introduction ........................................................................... 15
2. Wind Loads for Design ..................................................... 15
3. The Tension Chain – Structural Planning Basics ................. 19
   - Walls Sheathed with Insulation Board (non-structural) in areas without Openings ................. 20
     - The Roof to the Double Top Plate and Stud ............. 23
     - The Wall Stud to the Bottom Plate and then to the Foundation ........................................ 27
     - Second Floor Plane with Stud Walls Above and Below ..................................................... 30
   - Walls Sheathed with Structural Panels in areas without Openings ......................................... 33
     - The Structural Sheathing of the Wall Framing Elements ................................................... 33
     - Connection of the Roof Truss/Rafter to the Structural Wall Plane ................................... 35
     - Connection of the Structural Wall Plane to the Foundation ............................................. 35
Chapter 3 Wind Structural Planning Basics—
The Stability System

Introduction ................................................................. 59
The Stability System Defined ............................................ 59
Stability Concepts Used in Structural Planning
  to Resist Sliding and Overturning .................................... 63
    • Foundation Walls .................................................... 63
    • Garage Walls .......................................................... 65
    • Garage Doors .......................................................... 66
    • Drag Struts .............................................................. 72
Shear wall Design .......................................................... 83
  • Traditional Shear Wall Design Method ..................... 83
  • Perforated Shear Wall Empirical Method ............... 86
    • Shear Walls Containing Garage Doors .................... 88
    • Placement of Doors and Windows in
      Shear Walls .......................................................... 91
    • Avoid Wall Offsets .................................................. 92
• Shear Walls that are Bearing Walls ........................................ 95
• End Wall Design ...................................................................... 98
  • Gable Roof ........................................................................ 98
  • Hip Roof ........................................................................... 98
Stability Applications .............................................................. 101
• Structural Planning Applications .......................................... 101
  • Example 1 ........................................................................ 101
  • Example 2 ........................................................................ 111
Conclusions ............................................................................. 135
Post Script .............................................................................. 135
Selected References .................................................................. 136

Chapter 4 Numerical Examples - Uplift Chain Resistance 137

Introduction ............................................................................. 137
One Story Residences ........................................................... 137
• General Design Procedure .................................................... 137
Example 1 - Structural Wood Panel Option ....................... 145
Example 2 - All Metal Connector Solution ......................... 169
Two Story Residences ............................................................ 179
  • Notes ............................................................................... 179
Conclusions ............................................................................. 181
Nomenclature ........................................................................... 182
Selected references ............................................................... 184

Chapter 5 Numerical Examples - The Stability System 185

Introduction ............................................................................. 185
Overview of the Stability System ............................................ 185
One Story Residences ............................................................ 186
• General Design Procedure - Expanded Version ............... 186
  • Flat Roof Diaphragms ...................................................... 195
  • Gable Roof Diaphragms .................................................. 197
  • First Floor Diaphragms .................................................... 199
  • Wind Load to End Walls Below the Triangular Gable End .. 206
  • Wind Load to the Triangular Gable Portion of Windward End Wall ................................................. 208
  • Wind Load to the Triangular Portion of the Leeward Gable ................................................................. 209
  • Roof Diaphragm Behavior .............................................. 211
  • First Floor Diaphragm Behavior ...................................... 213
• Traditional Shear Wall Method .......................................... 217
• Perforated Shear Wall Empirical Method ........................ 221
Appendix A – Uplift Chain Design Tables

Introduction ......................................................... A-1
Uplift Tables 1 through 17 .................................... A-2 to A-18

Appendix B – Stability System Design Tables

Introduction ......................................................... B-1
Stability Tables 1 through 31 ................................. B-2 to B-33
List of Illustrations

Figures

1.1 Basic Wind Speed Map .................................................. 2
1.2 ASCE 7-95 Wind Speed Map (part 1) .............................. 9
1.3 ASCE 7-95 Wind Speed Map (part 2) .............................. 10
1.4a ASCE 7-93 & 95 Comparison ........................................ 11
1.4b ASCE 7-93 & 95 Geographic Location Comparison .......... 12
2.1a Components that receive wind directly ........................... 16
2.1b Additional components that receive wind directly .............. 17
2.2 Basic Building Planes ................................................... 19
2.3a The Links of the Tension Chain – Ext. Wall .................... 20
2.3b Uplift Capacity – Connector Calculations ....................... 22
2.4 Elevations of Stud Wall/Construction Options ................. 23
2.5 Roof and Stud Framing Alignment – Metal Connector Options ... 24
2.6a Truss/Rafter to Double Top Plate Connectors ................... 25
2.6b Double Top Plate to Stud Connections ........................... 26
2.7a Stud to First Floor Connections ...................................... 28
2.7b Stud to Foundation Connections ................................. 29
2.8a Second to First Floor Stud Connection –
          Alignment between Studs ........................................ 30
2.8b Second to First Floor Stud Connection –
          Non-alignment between Studs ................................. 31
2.8c Double Capacity of Connectors (Shown in Plan Views) ....... 32
2.9 Total Use of Plywood or OSB Board to Resist Uplift .......... 34
2.10 Anchor Bolt Problems ............................................... 36
2.11 Shims, Non-level Top of Foundation ............................. 37
2.12a Anchorage to Foundation-Options ............................... 38
2.12b Alternate Mud Sill Anchor ......................................... 39
2.13 Unique Approach to Sheathing Stud Wall ....................... 41
2.14a Roof to Wall Detail (cathedral ceiling) .......................... 42
2.14b Roof to Wall Detail (2nd story) ................................... 42
2.15 Roof to Wall Detail .................................................... 43
2.16 Variable Interior Ceiling Heights ................................... 44
2.17 Garage Stud Wall ....................................................... 45
2.18 Typical Openings in Exterior Wall ................................. 46
2.19 Exploded View of Uplift at Opening in Exterior Wall ......... 47
2.20 Typical Porch and Support Posts ................................... 49
2.21 Exploded Side View of Porch – Uplift Chain Defined ........... 50
2.22 Typical Post Cap to Beam Connections for Uplift Resistance .... 51
2.23 Typical Post Base Connections for Uplift ......................... 52
2.24 Cantilevered Second Floors ........................................... 53
2.25 Several Insulating Concrete Form Types ......................................... 54
2.26 Typical Concrete/CMU Truss Connectors ....................................... 55
3.1 Stability-Wind Directions for Structural Analysis ............................. 60
3.2 Basic Stability System Wind Perpendicular to
Long Dimension (L) ........................................................................ 61
3.3 Basic Stability System Wind Perpendicular to
Short Dimension (B) ....................................................................... 62
3.4 Concept 1 – Foundation Wall Below Shear Wall ............................. 64
3.5a Closed and Open Garage Wind Magnitudes ................................... 65
3.5b External Pressure Coefficients for Components and
Cladding and Buildings with Mean Roof Height Less than
or Equal to 60 Feet ........................................................................ 67
3.5c Generic Garage Door Jamb Detail .................................................. 69
3.5d Generic Garage Door Sections for Figure 3.5c ............................... 70
3.5e Garage Door Load Transfer to Wood Frame .................................... 71
3.6a Diaphragm Load Transfer in a One Story Residence ..................... 73
3.6b Drag Strut Force Distribution To Shear Wall and Foundation .......... 74
3.6c Alternate Drag Strut Connections to Shear Wall ......................... 76
3.7a Section at Interior Offset Shear Walls (within same plane) ............ 77
3.7b Section Through Shear Walls ..................................................... 79
3.7c Side Elevation of Shear Wall ........................................................ 80
3.8 Concept 1-Drag Strut ..................................................................... 82
3.9a Traditional Approach to Shear Walls with Openings .................... 84
3.9b Equilibrium of a Shear Wall ........................................................ 85
3.9c Perforated Shear Wall Method .................................................... 86
3.10 Improved Garage Front Wall Shear Resistance ............................... 89
3.11 Steel Frame Hidden Within Garage Wall ........................................ 90
3.12 Elevation Planning ........................................................................ 91
3.13 Partial Foundation Plan Views ...................................................... 92
3.14 Partial Foundation Plan ................................................................. 94
3.15 Partial Plans with Offsets ............................................................. 95
3.16 Bearing Wall Used as Shear Wall .................................................. 96
3.17 Non-bearing Wall as a Shear Wall ................................................. 97
3.18a Stability-Wind on Gable and Hip Roof End Walls ....................... 99
3.18b Stability – Wind on Two-Story Space End Wall ......................... 100
3.19 Plans for Example #1 .................................................................. 102
3.20 Structural Planes of Resistance .................................................... 103
3.21a Transverse Shear Plane A .......................................................... 104
3.21b Transverse Shear Plane A Perforated Shear Wall
Empirical Method ............................................................................ 105
3.22a Transverse Shear Plane B .......................................................... 105
3.22b Transverse Shear Plane B Perforated Shear Wall
Empirical Method ............................................................................ 106
3.23 Structural Planes of Resistance .................................................... 107
3.24a Longitudinal Shear Plane A ....................................................... 108
4.11c Summary of Final Net Uplift Forces at Foundation, A.B. Spacing & Minimum Trench Footing Sizes ........................................... 162
4.13 Garage Tension – Chain @ Door .................................................. 164
4.14. Tie-Down Anchor Type Without Bolts to the Studs ........................ 165
4.15 Entry Column – Tension Chain .................................................. 167
4.16 Stud/Truss Relationships .............................................................. 171
4.17 Acceptable Metal Connector Arrangements Between Stud & Double Top Plate ............................................................... 173
4.18 Gable End Stud Wall Elevation Location of Metal Connectors ....... 177
4.19 Wind Uplift Design – 2 Stories .................................................... 180
5.1 Diaphragm Recommendations from SBC & UBC .......................... 188
5.2 Diaphragm Continuity @Ridge ..................................................... 189
5.3 Dimension Lumber Grading Stamp Definitions ............................. 191
5.4 Structural Panel Grade Stamp Definition ....................................... 192
5.5 Wind Load to Diaphragms Procedure for 1-Story Residence Wind Perpendicular to Ridge .................................................. 194
5.6 Flat Roof Diaphragm Behavior ..................................................... 196
5.7 Sloping Roof Diaphragms ............................................................. 198
5.8 Exploded View of One Story Diaphragms and Shear Wall ............. 200
5.9 Blocked Diaphragm Nail Pattern & Nomenclature ........................ 204
5.10 Unblocked Diaphragm Nail Pattern & Nomenclature .................... 205
5.11 Roof/Ceiling Diaphragm Loads-Wind Parallel to Long Dimension “L”-One Story .................................................. 207
5.12 1st Floor Diaphragm Loads-Wind Parallel to Long Dimension “L”-One Story .................................................. 210
5.13 Roof Diaphragm Wind Parallel to Ridge – One Story ..................... 212
5.14 Exploded View of One Story Diaphragms and Shear Wall ............. 214
5.15 Traditional Shear Wall ............................................................... 219
5.16 Perforated Shear Wall ................................................................. 222
5.17 East/West Direction for Roof Diaphragms and Shear Walls .......... 234
5.18 North/South Direction for Roof Diaphragms and Shear Walls ........ 236
5.19 Perspective of One Story Residence Diaphragm A & AA Reactions and Shears .................................................. 239
5.20 Perspective of One Story Residence Diaphragm B Reaction and Shear .................................................. 241
5.21 Perspective of One Story Residence Diaphragm C Reaction and Shear .................................................. 242
5.22 Perspective of One Story Residence Diaphragm D Reaction and Shear .................................................. 243
5.23 Perspective of One Story Residence Floor Diaphragm Reaction and Shear .................................................. 246
5.24 Perspective of One Story Residence Floor Diaphragm Reaction and Shear .................................................. 247
5.25 Shear Wall A – Method Comparison ............................................. 250
5.26 Shear Wall B – Method Comparison ............................................. 251
5.27 Shear Wall C – Method Comparison ............................................. 252
5.28 Shear Wall D – Method Comparison ............................................ 253
5.29 Shear Plane E – Method Comparison
Steel Rigid Frame Required ........................................................... 254
5.30 Shear Wall F – Method Comparison ............................................. 255
5.31 Shear Wall G – Method Comparison ............................................ 256
5.32 Shear Wall H – Method Comparison ............................................ 257
5.33 Perspective of One Story Residence –
Ceiling Diaphragm – Stability for Gable End Walls ....................... 278
5.34 Perspective of One Story Residence –
No Ceiling Diaphragm – Use Diagonal Braces
For Stability of Gable End Walls ...................................................... 280

Tables
1.1 Cost Comparison .............................................................................. 7
2.1 Component and Cladding Uplift on Roof Trusses (in psf) ................ 18
3.1 DASMA Garage Door Wind Load Guide ........................................ 68
A.1 Structural Panel Axial Tension Allowable Capacity ....................... A-2
A.2 Lateral Allowable Capacity per Nail (lbs.) ..................................... A-3
A.3 Uplift Requirements for a Single Story Residence-20’ .................... A-4
A.4 Uplift Requirements for a Single Story Residence-24’ .................... A-5
A.5 Uplift Requirements for a Single Story Residence-28’ .................... A-6
A.6 Uplift Requirements for a Single Story Residence-32’ .................... A-7
A.7 Uplift Requirements for Top Story of a Two Story Residence-20’ .. A-8
A.8 Uplift Requirements for Top Story of a Two Story Residence-24’ .. A-9
A.9 Uplift Requirements for Top Story of a Two Story Residence-28’ A-10
A.10 Uplift Requirements for Top Story of a Two Story Residence-32’ A-11
A.11 Floor Framing Dead Load ........................................................... A-12
A.12 Wood Stud Wall Weights ........................................................... A-13
A.13 Crawl Space Foundation Wall Uplift Resistance ......................... A-14
A.14 Basement Foundation Wall Uplift Resistance .............................. A-15
A.15 Vertical Anchor Capacity for Foundation Walls ....................... A-16
A.16 Gable Ends-Average Height .................................................... A-17
A.17 Uplift Resistance of Trench Footing/Crawl Space ....................... A-18
B.1 Wind Perpendicular to Roof Ridge-One Story-20’ ...................... B-2
B.2 Wind Perpendicular to Roof Ridge-One Story -24’ ...................... B-3
B.3 Wind Perpendicular to Roof Ridge-One Story -28’ ................. B-4
B.4 Wind Perpendicular to Roof Ridge-One Story -32’ .................... B-5
B.5 Wind Perpendicular to Roof Ridge-Two Story -20’ .................... B-6
B.6 Wind Perpendicular to Roof Ridge-Two Story -24’ ................. B-7
B.7 Wind Perpendicular to Roof Ridge-Two Story -28’ .................... B-8
B.8 Wind Perpendicular to Roof Ridge-Two Story -32’ .................... B-9
B.9 Gable End Wall-Wind Parallel to Ridge of Roof-One Story-20’ ... B-10
B.10 Gable End Wall-Wind Parallel to Ridge of Roof-One Story-24’ ... B-11
B.11 Gable End Wall-Wind Parallel to Ridge of Roof-One Story-28’ ... B-12
B.12 Gable End Wall-Wind Parallel to Ridge of Roof-One Story-32'... B-13
B.13 Gable End Wall-Wind Parallel to Ridge of Roof-Two Story-20'... B-14
B.14 Gable End Wall-Wind Parallel to Ridge of Roof-Two Story-24'... B-15
B.15 Gable End Wall-Wind Parallel to Ridge of Roof-Two Story-28'... B-16
B.16 Gable End Wall-Wind Parallel to Ridge of Roof-Two Story-32'... B-17
B.17 Gable End Wall Bracing Load if Ceiling/Attic
Floor/Discrete Braces-Wind Parallel to Roof Ridge-
One Story-20'................................................................. B-18
B.18 Gable End Wall Bracing Load if Ceiling/Attic
Floor/Discrete Braces-Wind Parallel to Roof Ridge-
One Story-24'................................................................. B-19
B.19 Gable End Wall Bracing Load if Ceiling/Attic
Floor/Discrete Braces-Wind Parallel to Roof Ridge-
One Story-28'................................................................. B-20
B.20 Gable End Wall Bracing Load if Ceiling/Attic
Floor/Discrete Braces-Wind Parallel to Roof Ridge-
Two Story-20'............................................................... B-21
B.21 Gable End Wall Bracing Load if Ceiling/Attic
Floor/Discrete Braces-Wind Parallel to Roof Ridge-
Two Story-24'............................................................... B-22
B.22 Gable End Wall Bracing Load if Ceiling/Attic
Floor/Discrete Braces-Wind Parallel to Roof Ridge-
Two Story-28'............................................................... B-23
B.23 Gable End Wall Bracing Load if Ceiling/Attic
Floor/Discrete Braces-Wind Parallel to Roof Ridge-
Two Story-32'............................................................... B-24
B.24 Gable End Wall Bracing Load if Ceiling/Attic
Floor/Discrete Braces-Wind Parallel to Roof Ridge-
Two Story-32'............................................................... B-25
B.25 Recommended Uniform Roof Live Loads for APA Rated
Sheathing and APA Rated Sturdi-I-Floor with Long
Dimension Perpendicular to Support..................................... B-26
B.26 APA Panel Subflooring (APA RATED SHEATHING)................. B-27
B.27 Recommended Shear (pounds per foot) for Horizontal APA Panel
Diaphragms for Wind or Seismic Loading............................... B-28
B.27 Notes Continued............................................................. B-29
B.28 Recommended Shear (PLF) for APA Panel Shear Walls
for Wind Loading............................................................ B-30
B.29 Allowable Shear for Wind or Seismic Forces in
Pounds per Foot for Vertical Diaphragms of Lath and
Plaster or Gypsum Board Frame Wall Assemblies.................... B-31
B.30 Specific Gravity for Solid Sawn Lumber............................... B-32
B.31 Truss Reactions............................................................... B-33

xiii
List of Acronyms

AFPA  American Forest Products Association
AF&PA  American Forest & Paper Association, American Wood Council
         (Same organization as above, but name revised)
ANSI  American National Standards Institute
APA   The Engineered Wood Association
         (Formerly the American Plywood Association)
ASCE  American Society of Civil Engineers
ASTM  American Society for Testing and Materials
BOCA  Building Officials & Code Administrators
CABO  Council of American Building Officials
DASMA Door and Access Systems Manufacturers Association
FEMA  Federal Emergency Management Agency
ICBO  International Conference of Building Officials
IEMA  Illinois Emergency Management Agency
LVL   Laminated Veneer Lumber
MWFRS Main Wind-Force Resisting System
NDS   National Design Specification
OSB   Oriented Strand Board
SBC   Southern Building Code
SBCCI Southern Building Code Congress International
UBC   Uniform Building Code
WMM   Windstorm Mitigation Manual for Light Frame Construction, Aug. '97
Chapter 1 - Wind Basics Revisited

Introduction

This Companion Manual complements the Windstorm Mitigation Manual for Light Frame Construction, published in August of 1997. References made to the 1997 Manual occur throughout this text, and are referred to by the acronym WMM in Italic. There is no attempt to duplicate material found in the WMM. The two Manuals together allow decisions to be made about desired level of wind resistance, the selection of the wind forces to achieve the desired level of wind resistance, and the details which can resist those design forces.

This Companion Manual provides basic structural guidance in planning a home’s layout and overall form to maximize the wind resistance properties of the construction, while minimizing the cost to implement the details required to achieve the selected level of wind resistance.

Tables and Charts are provided in the Companion Manual to aid in the selection of design uplift and stability forces, followed by the allowable capacities of specific recommended details. The appropriate choice of detail is one that provides an allowable capacity equal to or greater than the design force. This will assist contractors in establishing construction details and the cost to implement these details. Thus, the owner will know what part of the overall cost is attributable to their desired level of wind resistance.

Straight-line Winds vs. Tornadoes

Straight-line Winds

Straight-line winds are the typical winds described in Chapter 1 of the WMM. These straight-line winds form the probabilistic basis of the maximum wind speed isobar map used for wind design across the United States. The map of Figure 1.1 was in general usage up to 1996 [1.1], and is still used in the 1996 BOCA National Building Code. It is based on a fastest mile wind concept. Data was collected from 129 weather stations and has been statistically analyzed for a 50-year return period. Tornadoes have not been considered in developing the basic wind speed distributions.
Figure 1.1 – Basic Wind Speed Map

Example:
To illustrate the use of the map, consider a 50-year wind in Champaign, IL. The map shows a 70-mph maximum wind speed. This means that Champaign has experienced or is expected to experience a wind speed greater than 70 mph on the average of once in 50 years. The probability of the wind speed exceeding 70 mph in Champaign within a given year is 2%, while the probability that this wind speed will be equaled or exceeded in 50 years is 64%[1.2].

It would seem that the scenario outlined in the example above, implies that the design wind speed for Champaign has a high probability of being exceeded within the assumed 50-year life of a building. However, buildings designed and constructed in accordance with current building codes, assuming reasonable standards of workmanship, have reserve strength to resist higher wind speeds than the design wind speed. The use of wood load duration factors, and factors of safety that establish an allowable stress level, would increase the actual exposure of the building to approximately a 500 year time period. The probability that the 70 mph wind speed will be equaled or exceeded at least once in 500 years is 100%. The inclusion of normal safety factors dramatically improves the chances
for a structure to resist higher loads without collapse or even damage. Also, wind design assumes wind applied perpendicular to the longer surfaces of the building, which most often produces the worst possible wind loading case [1.3].

Why then do wind speeds, even less than the design wind speed, sometimes cause damage to buildings, or even collapse?

(1) Connections are generally the weakest link in the wind resistance system of any light frame construction. This was explained in the Chapters of the WMM. The studs, joists, rafters or trusses seldom fail as individual members subjected to bending, shear and axial stresses. If a member fails, it is usually after a connection fails. Connections are the essence of adequate straight-line wind resistance. This is not to imply that the breach of a garage door, window or patio door by the wind cannot be a catalyst for member damage. It does not imply that projectiles cannot also cause severe damage to structural members. These topics will be treated in depth in the following chapters.

The straight-line wind speed map of Figure 1.1 is shown in all three Model Building Codes: the 1996 Basic Building Code (BOCA), the 1994 Uniform Building Code (ICBO) and the 1995 Standard Building Code (SBCC). Each Code dominates in a particular region of the United States, and some States actually use a mixture of the model codes. These model codes are revised on a 3-year basis to reflect new knowledge, or clarify existing provisions. Although these three codes have their own specific provisions for wind resistance, they also reference the wind provisions of the ASCE 7 Standard Minimum Design Loads for Buildings and Other Structures [1.4] as an alternate procedure. The provisions of the BOCA Code and the ASCE 7 may start with different procedures, but they arrive at the same wind pressures and suctions used for design in most cases.

(2) Compliance with the minimum requirements of the accepted building code should imply adequate wind resistance with respect to the use of nails as defined therein. Yet residential construction techniques cited in Chapter 2 through 4 of the WMM clearly illustrated the potential inadequacies of platform framing. These inadequacies involve the type of exterior sheathing and associated nailing of that sheathing to the studs, plates, bandboards, mudsills, rafters and trusses. “Accepted” construction practices that inadvertently introduce weak links into the tension uplift chain include:

1) the use of nails for stud walls in conjunction with non-structural sheathing;
2) the use of nails for stud walls in conjunction with structural sheathing that does not completely tie the wood framed skeleton together; and
3) the methods by which the wood framing is anchored to the foundation and the amount of uplift resistance provided by that foundation.

The first two practices can produce withdrawal loading on nails that is a direct violation of all three model codes, while the third practice leads to separation of the home’s superstructure from its foundation. Chapter 7 of the WMM provides options to improve the strength of light frame construction and overcome the three issues described above.
The three model codes are performance based and use similar language for structural
design in wood construction. For example the 1996 BOCA Code states as follows:

2303.1 Structural design: All structural wood members and connections shall be
of sufficient size or capacity to carry all design loads as required by Chapter 16 [Structural
Loads] without exceeding the allowable design values specified in AFPA NDS [National

2305.2 Fastening: The quantity and size of fasteners connecting wood frame
members together and sheathing materials to wood frame members shall not be less than
that specified in Table 2305.2. [The Table establishes a minimum connection required for
any framing condition. A portion of the Table related to the exterior wall tension chain is
shown in Chapter 4 of the WMM].

Neither of the above provisions mention non-structural rigid-board (pink or blue)
insulation as the likely "sheathing" that will cover the majority of the residence and create
"weak links" in the uplift tension chain. The first BOCA provision above references the
1991 NDS for allowable stresses in members and the allowable loads on fasteners.

Part XII: Nails and Spikes in the NDS states:

12.2.2-Withdrawal from End Grain
Nails and spikes shall not be loaded in withdrawal from end grain of wood.

It becomes necessary for connection design to be done by an engineer when non­
structural sheathing is used, since the Tables seemingly only provide minimum fasteners
for conditions where structural wood sheathing is employed. The other two model codes
either states the above NDS provision in their own words or reference the NDS.

Tornado Winds

Every year approximately one thousand tornadoes touch down in the United States. Only
a small percentage actually strike occupied buildings; however, every year a number of
people are killed or injured and an extensive amount of property damage results [1.5].
It has been stated in an article in the Journal of the Structural Division of the ASCE that
"Tornadoes are the most violent of the weather phenomenon with which engineers must
contend. To develop designs for tornadic loading, it is necessary to establish limits on
ground level wind speeds that can occur in tornadoes"[1.6]. Wind Engineering by Henry
Liu states "Tornado wind speed probability is based on fastest-gust speed rather than on
fastest mile straight-line wind." Using this approach, it can be stated that "50% of
tornadoes have maximum gust speeds lower than 100 mph, and only 10% of tornadoes
have maximum speeds higher than 150 mph" [1.7]. These facts are re-phrased in the
ASCE's Minimum Design Loads for Buildings and Other Structures Commentary as
follows: “It is recognized that tornadic wind speeds have a significantly lower probability of occurrence at a point than the probability for basic wind speeds. In addition, it is found that in approximately one-half of the recorded tornadoes, gust speeds are less than the gust speeds” associated with basic wind speeds [1.8]. This adds credibility to the goal to improve straight-line wind load resistance in residential construction, since “straight-line winds cause far more cumulative damage than tornadoes” [1.9]. It is feasible and economical to build straight-line wind resistant structures to withstand higher wind speeds than probability concepts would anticipate.

As mentioned in the WMM, it is not economically feasible to build entire tornado resistant residences. It would be necessary to build underground structures or totally reinforced concrete structures to accomplish such resistance.

Current research leans toward construction of a “safe room”, meaning a small interior room of reinforced masonry, concrete or steel walls with a concrete top [1.10]. One such safe room for occupants was proven effective in Tulsa, Oklahoma during the May 3, 1999 Category 4 tornado. The entire house was destroyed while the safe room remained. Owners will need to decide if the “safe room” alone, or in combination with straight-line windstorm resistance detailing is the desired approach for safety and also for minimizing construction damage.

**Reasonable Expectations of Wind Resistant Design**

Employing the wind resistant design details recommended in both Manuals has potential advantages for both the owner and the contractor.

The owner should generally benefit from:
- Improved overall safety
- Reduced damage to the residence
- Less chance of being displaced from the residence
- Less loss of personal property

The contractor should benefit from:
- Improved quality control
- Fewer call backs
- Enhanced reputation

**Wind Resistant Construction Program**

The Cities of Urbana, Decatur and Ogden Illinois experienced a tornado on April 19, 1996. The damage was devastating to all three communities; however, after a review of the repetitive failure mechanisms by engineers, FEMA/DEMA officials, and building inspectors, it was clear that much of the damage could have been reduced or even
eliminated in many cases. After the publication of Mitigating Storm Damage in Light Frame Construction and the public presentations that followed in all three cities, the major question was “how much will it cost to rebuild with wind resistant construction techniques”?

The Building Safety Division of the City of Urbana proposed a program of increased awareness by the residential construction industry and the general public. A voluntary use of wind resistant techniques by local contractors and homeowners was promoted. The proposal received a grant in 1997 from the Federal Emergency Management Agency’s (FEMA) Hazard Mitigation Grant Program which was administered by the Illinois Emergency Management Agency’s (IEMA). The Illinois Department of Commerce and Community Affairs and the State Farm Fire and Casualty Company awarded funds to the City of Urbana.

The program consisted of several components:

- Presentations to builders and code officials of the basic chapters in the WMM concerning the mechanics of wind, the basics of the structural behavior of nails, the idiosyncracies of platform framing, the wind tension chain, wind stability, wind resistant design techniques applied to new construction.
- Application of wind resistant construction techniques on two new homes and one retrofit project. These homes would serve as demonstration homes, and their techniques videotaped.
- Development of a video that focuses on the effects of wind on homes, evaluate present building performance, recommend alternative construction techniques for improved wind resistance, and provide a bibliography and resource list for further information.
- Initiate a permit rebate program, which offers an incentive to builders and potential homeowners to utilize a 90-mph minimum wind resistant design to construct their home in Urbana.

All three homes have been completed and the contractors have provided a cost breakdown of material and labor increases that were a result of incorporating wind resistant design techniques following the WMM. Table 1.1 summarizes basic information about the homes in the study, and the associated costs.

The two new homes represented both ends of the design spectrum: (1) a modest one-story home with approximately 1300 square feet of usable living space and a cost of $70,000; (2) a custom home with a variety of spatial features, numerous amenities and a price of $405,000. The retrofit house was a modest one-story home that was completely gutted and converted into a contemporary home with a partial second floor addition that cost $160,000.

All three homes were engineered to structurally handle a 90-mph straight-line wind. It is clear from the data in Table 1.1 that the more expensive the home, the less impact the cost of including wind resistant construction has on the overall cost of the home. This
makes perfect sense on a percentage basis. The $70,000 home included approximately $5,600 for material, labor and engineering for the added wind resistance. The dollar amount does not seem high, but it becomes an almost 8% cost increase. The $405,000 home added almost $14,600, but this only amounted to 3.6% of the total cost of the house. The retrofit house is unique. It only had a crawl space of concrete block foundation walls that provided very little mass to resist the addition of a second story wind condition. Twelve cubic yards of concrete were added to provide overturning stability. The added cost for the engineering and construction amounted to $12,900 of the $160,000 total. One can assume from this small study that wind resistant construction will cost between 5% and 8% of the construction cost when engineering is included. It should be noted that all three homes had installed 90 mph rated garage doors and associated hardware. This added only approximately $200 to each house. Obviously, this is a small price to pay given the observed damages attributable to houses that had their garage doors breached.

### Table 1.1 - Cost Comparison

<table>
<thead>
<tr>
<th>Construction Cost</th>
<th>New</th>
<th>New</th>
<th>Retrofit</th>
</tr>
</thead>
<tbody>
<tr>
<td>Description</td>
<td>$70,000</td>
<td>$405,000</td>
<td>$160,000</td>
</tr>
<tr>
<td>1 story, crawl, flat bottom gable roof trusses, 2 car garage, conform insulation forms + construction foundation</td>
<td>1 story, partial flat bottom gable trusses, (2) 1 car garages, 5 fireplaces, partial cathedral ceiling, full basement, concrete cast in place</td>
<td>1 story, crawl, flat bottom gable trusses, concrete block foundation, partial 2 story</td>
<td></td>
</tr>
<tr>
<td>Square Feet</td>
<td>1308 (w/o garage)</td>
<td>1st floor 2975</td>
<td>2nd floor 4390</td>
</tr>
<tr>
<td></td>
<td>2nd floor 1415</td>
<td>w/o garage</td>
<td>2nd floor 564</td>
</tr>
<tr>
<td>Cost/Square Ft</td>
<td>$53.50</td>
<td>$92.25</td>
<td>$69.70</td>
</tr>
<tr>
<td>Wind Resistant Construction Costs</td>
<td>OSB sheathing, additional A.B.s, footing &amp; foundation, wall concrete, metal storm connectors, garage door upgrade</td>
<td>Metal wind connectors, fabricated steel for drag struts, extra reinforcement, epoxy anchors, upgrade foundation to all poured, garage door upgrade</td>
<td>Shear walls for added 2nd floor, 12 cubic yards of concrete, tie-downs in crawl space, OSB board, metal connectors, steel beam drag strut, garage door upgrade</td>
</tr>
<tr>
<td>Materials</td>
<td>$1942</td>
<td>$6122</td>
<td></td>
</tr>
<tr>
<td>Labor</td>
<td>$2816</td>
<td>$3620</td>
<td></td>
</tr>
<tr>
<td>Total</td>
<td>$4758</td>
<td>$9742</td>
<td>$7100</td>
</tr>
<tr>
<td>Cost/Square Ft</td>
<td>$3.64 (6.8%)</td>
<td>$2.22 (2.4%)</td>
<td>$3.09 (4.4%)</td>
</tr>
<tr>
<td>Engineering</td>
<td>$640</td>
<td>$4000</td>
<td>$4000</td>
</tr>
<tr>
<td>Observation visits</td>
<td>$160</td>
<td>$800</td>
<td>$800</td>
</tr>
<tr>
<td>Total</td>
<td>$800</td>
<td>$4800</td>
<td>$4800</td>
</tr>
<tr>
<td>Cost/Square Ft</td>
<td>$0.61 (1.14%)</td>
<td>$1.10 (1.2%)</td>
<td>$2.10 (3%)</td>
</tr>
<tr>
<td>% Const, Cost Eng. + Const.</td>
<td>7.9%</td>
<td>3.6%</td>
<td>7.4%</td>
</tr>
</tbody>
</table>
Wind Speed Map: A Footnote

The 1995 Edition of the ASCE-7 Minimum Design loads for Buildings and Other Structures [1.11], has introduced a new concept for wind speed determination: a “three-second gust”, rather than the “fastest mile” philosophy that has been in place since 1972. The basic wind speed has been redefined as the peak gust that is recorded on instruments with various response characteristics and thus are averaged over three seconds. Data has been collected at 485 weather stations for a minimum of five years. The new map as shown in Figure 1.2 and 1.3, as well as the map of Figure 1.1, similarly reflect an exposure category C and a 50-year mean recurrence interval. The new map is divided into counties to more easily determine where contours lie. Neither map is representative of wind speeds at which structural failures are expected to occur. As stated earlier in this Chapter, employed factors of safety leads to structural resistances and corresponding wind loads that are substantially higher than the speeds shown on the maps. Note that 85 and 90 mph cover the majority of the USA, except for the Gulf and Atlantic Coast Hurricane areas.
Alaska Note:
For coastal areas and islands, use nearest contour.

Figure 1.2 – ASCE 7-95 Wind Speed Map (part 1)
\textbf{Notes:}  
1. Values are 3-second gust speeds in miles per hour (m/s) at 33 ft (10m) above ground for Exposure C category and are associated with an annual probability of 0.02.  
2. Linear interpolation between wind speed contours is permitted.  
3. Islands and coastal areas shall use wind speed contour of coastal area.  
4. Mountainous terrain, gorges, ocean promontories, and special wind regions shall be examined for unusual wind conditions.

\textit{Figure 1.3 – ASCE 7-95 Wind Speed Map (part 2)}
A comparison of wind loads for one and two story structures was discussed in an article from the Journal of the American Institute of Steel Construction:

"It is difficult to make a simple and direct comparison between the MWFRS loads found in the new Standard for buildings less than 60 feet in height and those in the 1993 version. ...Limited comparison based on base shear suggests a rather substantial decrease in the wind loads ...........

[1.12]

One comparison between the fastest mile concept and the three-second gust concept would be to select a 100 mph wind speed and perform the calculations for main wind-force resistance system wind pressures and also for a single roof truss uplift based on components and cladding pressures. Figure 1.4a illustrates the results, which are significantly different for the two concepts. Note the base shear of 374 plf and the uplift of 485 plf is larger for the fastest mile concept of the ASCE 7-93 and BOCA 1996 documents.

Figure 1.4a - ASCE 7-93 & 95 Comparison
Figure 1.4b compares the required wind speed for Springfield, Illinois based on the maps for a fastest mile concept and a three-second gust concept. This may be a more meaningful comparison, since it really shows what the differences are for a given geographic location that requires 70 mph using a fastest mile concept versus 90 mph required wind speed for a three-second gust concept. Note that the three-second gust concept produces the larger base shear for MWFRS and the larger uplift for a single truss component and cladding analysis.
Based on the variability of the above two comparisons, and given that the WMM used the fastest mile concept in all of its discussions, this manual will continue to use the wind speed map of Figure 1.1 (ASCE 7-93).

Conclusions

It is clear the design of buildings to resist wind loading is based on the concept of probability. Structures are designed to provide a specific degree of safety against high winds, determined by the probability of occurrence of wind speeds exceeding the design value. It is also clear that straight-line wind and associated gusting have a greater probability of causing damage to light frame construction than do tornado winds. Again, it should be emphasized this does not imply that homes can withstand the severe swirling and upward acting winds of the major tornadoes seen on television. It does suggest that..."if buildings and houses are designed using the wind load requirements referenced in the model codes and constructed properly, they should be able to resist as many as two-thirds of the tornadoes"[1.13].

Connections and structural sheathing continuity are the most important factors for wind resistant construction. These factors will be emphasized throughout this Companion Manual.
Selected References


1.3 Liu, p. 37.

1.4 ASCE 7-93.

1.5 Tornado Project, www.tornadoproject.com


1.7 Liu, p. 36.

1.8 ASCE 7-95, Commentary, p. 156.

1.9 Liu, p. 37.


Chapter 2 - Wind Structural Planning Basics – The Tension Chain

Introduction

Chapter 2 and Chapter 3 are intended to assist homeowners, contractors and architects in the basic requirements for designing wind resistant light frame structures, as it relates to architectural house planning. There are structural “wind resistance” concepts that, if considered during the design of a house floor plan and its elevations, will reduce the cost impact of a wind resistance system. Many times the mere shifting of a wall a foot or two one way or the other will significantly improve the structural design for wind resistance, without greatly impacting the house layout. Shifting a window or door may contribute to improved wind resistance. Aligning specific first and second floor walls may greatly improve wind resistance. Alignment of the foundation with the first and second floor walls will improve wind resistance. These are just a few of the items that might make inclusion of wind resistant design affordable, and provide the expectations described in Chapter 1.

This Chapter includes a structural planning discussion of the wind uplift chain of resistance for solid walls, walls with openings, porches, second floor cantilevers, and insulating concrete forms for cast-in-place concrete construction. Also, this Chapter will explore ideas that contractors have used, after reading the WMM, simplifying wind resistant detailing for their operation, and reducing time and material in the process.

Chapter 4 will use the information of this Chapter to present numerical examples of uplift resistance design and design tables for general application related to the tension chain.

Wind Loads for Design

There are two categories of design wind loads on buildings and other structures: (1) Components and Cladding wind forces; and (2) Main Wind-Force Resisting System wind forces.

Category one: Components and Cladding wind loads are applied to individual structural elements that receive wind force directly, such as a fastener, a stud, a roof truss, a rafter or a sheet of structural sheathing. Figures 2.1a and 2.1b illustrate a few examples. A stud receives wind over an area defined by its height and by its spacing. Thus, an 8 foot high stud spaced at 16 inches on center receives wind based on an area A = 8' * (16/12)' = 10.67 square feet. A truss spanning 32 feet and spaced 24 inches on center receives wind
over a projected area \( A = 32' \times 2' = 64 \) square feet, while a sheet of plywood receives wind over an area \( A = 8' \times 4' = 32 \) square feet. It becomes apparent that all of these areas are very small compared to wind against an entire elevation 8 feet tall by 60 feet long, where \( A = 8' \times 60' = 480 \) square feet.

The probability of a larger wind force hitting a small-localized area is greater than over a very large area. Thus, components are designed for higher localized wind forces, than the building as a total. The same logic applies to wind gusts hitting these same elements. Also, ridges and eaves receive higher wind pressures (positive or negative) than the main part of the roof, and this is reflected in all the building codes. **Note:** Negative pressure is many times referred to as “suction”, and both terms will be used interchangeably in this
Manual. Technically, a negative pressure acts away from a surface, while a positive pressure acts against a surface.

The slope of the roof influences the magnitude of the uplift force for design. Using Components and Cladding wind pressures (both positive and negative) for a 90-mph wind speed produces the wind uplift comparison shown in Table 2.1. Eave and ridge locations, defined by a dimension “a”, receive a higher negative pressure or suction, and the central parts of the roof surfaces, defined by the dimension “b” receive smaller negative pressures.
It is clear for a typical floor plan of 32' x 60' that roof slope influences uplift design suctions in two basic ranges: from a 3 in 12 to a 6 in 12 slope. A 25% decrease in uplift design suction occurs for slopes between 7 in 12 to 12 in 12. The eave and ridge locations receive higher wind suction values for an “a” distance that is calculated from a formula based on the building dimensions. Overhangs would be part of the “a” dimension and part of this higher design suction. This particular roof slope influence is only applicable to the design uplift suctions and should not be considered applicable to the lateral design wind pressures (both positive and negative) for the main wind-force resistance system.

Table 2.1. Component and Cladding Uplift on Roof Trusses (in psf)
Category two: Main Wind-Force Resistance System (MWFRS) wind loads are applied to elements of the building that do not receive wind force directly, but participate in the lateral load resistance stability (overturning and sliding) of the building. These elements receive wind load from structural floor and roof planes called diaphragms. Examples in Category two include plywood or OSB board sheathing (commonly referred to in the building codes as structural panels) used as shear walls, which lie in planes parallel to the wind direction. See Figure 3 in Chapter 5 of the WMM.

Note: The term Structural panels will be used interchangeably with plywood or OSB sheathing.

The tension chain issues of uplift resistance will be discussed in this Chapter. The illustrations and text of Chapters 4 and 7 of the WMM will be referenced. The stability issues of Overturning and Sliding will follow in Chapter 3. The illustrations and text of Chapters 5 and 7 of the WMM will be referenced.

The Tension Chain – Structural Planning Basics

The superstructure of a house must be thought of in terms of planes. There are roof, wall and floor planes, as shown in Figure 2.2.
Generally, the exterior wall planes are the major part of the tension chain that link together the roof and foundation. The wood frame of the superstructure provides only a small amount of dead load resistance to the wind uplift, thus the weight of the foundation is the most important "ballast" to resist wind uplift on the superstructure. Essentially, the wall's connection to the foundation is the final link in the tension chain that must hold down the superstructure against uplift.

**Walls Sheathed with Insulation Board (non-structural) in areas without Openings**

Figure 2.3a illustrates a section of a typical exterior loadbearing wall with the entire skeleton framing elements in the tension chain separated.
The sheathing is typically non-structural insulation board. The uplift from the Components and Cladding wind category is shown across the roof. All the uplift arrows between elements are indicated. Since the tension chain of resistance is dependent on properly installed metal connectors, when non-structural insulation board is used, a one or two story residence will require connections at three basic locations:

1. The roof to the double top plate and stud;
2. the wall studs to the bottom plate;
   a. include the influence of floor plane band boards for one or two stories;
3. the wall to the foundation;
   a. include the influence of floor plane band boards for one or two stories;
   b. walls that bear directly on the foundation, such as garages.

These three connections will be discussed separately and in considerable detail.

**Warning:** The use of metal connectors depends on filling all the holes in the connector with the manufacturer’s required size, length and type of nail. If properly installed the published rated structural capacity of the connector can be relied upon. Too often an inadequate nail size or type is used, or an insufficient number of nails are installed. This practice results in connectors that cannot provide the required structural capacity, and thus creates a new weak link in the tension chain. Contractors should always keep the manufacturer’s catalog handy for reference. Use of connectors for conditions not specifically addressed in the manufacturer’s catalog are prohibited.

Figure 2.3b clarifies the procedure used by one manufacturer (Simpson Strong-Tie) to arrive at their catalog [2.1 and 2.2] of allowable values in tension uplift. Assume application of connector “D” to resist an uplift value of 330 lbs. between the roof rafter and (bypassing the double top plate) the stud wall. The allowable tension capacity of this connector is determined mathematically from the 1997 Edition of the NDS for Wood Construction [2.3], referenced by all three model codes, and a load test to find the ultimate capacity at failure. The ultimate test value is divided by a factor of safety of 3.0 to arrive at a comparable allowable capacity for the entire group of (5) 8-d nails. The smaller of: (1) the equation derived value; and (2) the reduced ultimate value establishes the allowable capacity. The allowable capacity of 347 lbs. is greater than the required tension capacity and therefore this connector is usable. The connection would not provide the necessary tension resistance if: (1) less than 5 nails were installed to the rafter and to the stud; (2) nails with smaller shear capacity were installed; (3) the connector was less than 18 gage; or (4) the species of wood used had a different specific gravity than 0.5.
Allowable Capacity found by:

1. NDS Nail Spec---Capacity/nail in shear = 93.9 lb
   a. modify for load duration (wind = 1.33 or 1.6)
   b. modify for penetration (gage of metal)
   c. modify for wood species density
      allowable capacity = \( 93.9 \times 1.33 \times 0.92 \times 5 = 575 \text{ lb} \)

2. load test
   ultimate capacity = 1040 lbs.
   for Factor of Safety = 3
   allowable capacity per connector = \( \frac{1040}{3} = 347 \text{ lb} \)

3. smaller value controls between 1 and 2
   Therefore: allowable uplift capacity = 347 lb

Figure 2.3b Uplift Capacity - Connector Calculations
The Roof to the Double Top Plate and Stud

The roof may be framed with trusses or rafters at a particular spacing. The spacing of trusses is typically 24" on center and rafters are usually spaced 16" or 24" on center. Either of these roof elements will receive the uplift wind force and are the first elements to be tied down. The resistance chain starts with the roof element and it must be connected to the double top plate of the wall with metal connectors.

Selection of the appropriate metal connector depends on the spacing of the wall studs and their alignment with the roof framing. Figure 2.4 shows the three stud wall construction options.

Figure 2.4 Elevations of Stud Wall/Construction Options
The top illustration “A” is the most common condition, where alignment has been planned into the framing design. Since trusses are common today, and usually spaced at 24” on center, every fourth stud will align with the roof trusses, creating a 4’-0” regular continuous path for tension connections. Situation “B”, where the roof and wall framing is all at 16” on center, is very rare today. This pattern uses more lumber and the framing layout takes more time. Situation “C”, where there is no intentional alignment in the framing design and the roof framing and wall studs only occasionally align, is the most common house framing. This saves a contractor time in layout but complicates the tension chain of resistance connections.

The nails used for the tension chain must be loaded “laterally” and avoid the use of nails in “withdrawal” or “toe-nailed” as defined in Chapter 2 of the WMM. Illustration of a variety of metal connectors used to attach the trusses or dimension lumber rafters to the stud wall is shown in Figure 2.5, as they apply to the condition of alignment of studs and roof framing.
It should be noted that when alignment exists, the double top plate could be avoided in the tension chain. A direct connection between a roof truss or a rafter and the wall stud has the advantage of fewer nails and a tension chain with fewer links. Connector types D, L and K were shown in Chapter 7 of the WMM and are repeated here, while types T and U are more elaborate connectors.

Non-alignment of wall and roof framing members involves an extra connector and therefore means more material and labor cost for the contractor. Illustration of a variety of metal connectors used to attach the trusses or dimension lumber rafters to the double top plate is shown in Figure 2.6a, as they apply to the condition of non-alignment of studs and roof framing. Connectors G, H and J were shown in Chapter 7 of the WMM and are repeated here, while connector V is another option. It is important that the connector is nailed to both plates of the double top plate. Thus, this connection becomes the first link in the tension chain of resistance.
Illustrations of metal connectors used to attach the double top plate to the stud are shown in Figure 2.6b. Connectors B and F were shown in Chapter 7 of the WMM and are repeated here, while connector W is another option. Each of these connectors adds another link to the tension chain of resistance, and now has the tension down the length of the stud.
The Wall Stud to the Bottom Plate and then to the Foundation

This connection in the tension chain of resistance is dependent on what material lies below the bottom plate. If a one story residence is used as a starting point, there may be:

1. A first floor sub-floor, bandboard and mudsill, followed by a crawl space or basement foundation;
2. A concrete or masonry foundation for a garage slab-on-grade, or;
3. A concrete or masonry foundation for a tri-level home, where the basement slab is built a half level into the ground.

Each of these three possible conditions will now be discussed separately:

When condition #1 is confronted, the tension chain usually requires two connections. The first is the stud to the bandboard, bypassing the bottom plate. Illustrations of metal connectors used to attach the stud to the bandboard are shown in Figure 2.7a. Connectors E, Q and R were shown in Chapter 7 of the WMM and are repeated here. Connector E merely passes the tension chain on to the bandboard, requiring connector R to be the link between the bandboard and the mudsill. From there the anchor bolts transfer the tension into the concrete to complete the chain of resistance. Connector Q and R together accomplish the same thing. Connector X is a continuous strap that ties the stud to the band board and is also imbedded into the cast-in-place concrete. This single tension chain link accomplishes several connections in one, but has the disadvantage of needing to be properly set in the concrete prior to the wall placement. One must know where the stud will be located for this connection to work and requires extreme care in placement. Connector Y is another type of strap that must be set in the concrete and wrapped around the bandboard. Again care in placement and alignment is required to make this connector function properly. This connector, in combination with either connector E or Q, which connects the bandboard to the studs, completes the tension chain of resistance. These later two options (Y and X) provide considerable tension strength and should be considered when wind uplift values become high. They are also more expensive options.

When condition #2 or #3 is confronted, the easiest connector to install is type Z, illustrated in Figure 2.7b. Here, nails attach the connector to the stud, and an expansion anchor or epoxy anchor is drilled into the concrete or grouted masonry foundation (passing through the bottom plate/mudsill) to complete the tension chain of resistance. This approach is easier than anchor bolts, which need to be straight and properly located to align with the bolt hole in the connector. Connector X in Figure 2.7b is similar to connector X in Figure 2.7a, except it is shorter in length.
Figure 2.7a: Stud to First Floor Connections
Figure 2.7b Stud to Foundation Connections
Second Floor Plane with Stud Walls Above and Below

When a two story portion of residence exists, the alignment of the studs in the walls above and below will influence the selection of a metal connector to continue the tension chain. Figure 2.8a illustrates two connector options when the studs above and below the second floor align. Connector type \( P \) is a continuous strap that nails all the second floor framing components between the studs, as well as the studs themselves. This is an excellent approach that is reasonably economical and easy to install. Connector \( N \) actually connects only the stud above and below the second floor framing components. This is another reasonable option.

![Figure 2.8a- Second to first Floor Stud Connection-Alignment between Studs](image)
Figure 2.8b illustrates stud non-alignment above and below the second floor plane. Two connectors are now required and the splitting strength of the bandboard is now a factor. This will usually require a #1 grade Douglas Fir or Southern Pine bandboard for maximum - perpendicular to grain - tensile strength.
**Note:** It is possible to double connectors between the roof and the wall when the required uplift capacity exceeds the allowable capacity of a single connector. Certain guidelines must be followed [2.4] to avoid splitting the rafter or truss. These guidelines are illustrated in Figure 2.8c. Case "A" would not allow doubling connectors, since splitting of the 1-1/2" wide truss or rafter is likely. Case "B" separates the two connectors by the width of the double top plate to avoid splitting when 1-1/2" width roof members are used. Case "C" allows doubling connectors but requires the roof member to be at least 2-1/2" wide. This condition is less likely to split the roof member.

![Diagram showing double capacity of connectors](link-to-diagram)

**Figure 2.8c - Double Capacity of Connectors**
*(Shown in Plan Views)*
Walls Sheathed with Structural Panels in areas without Openings

The discussion has so far involved the skeleton framing of roof and wall with the exterior sheathing being a non-structural insulation board. There are two other choices:

1. Structural panels (plywood or OSB) as sheathing;
2. Insulation board with calculable structural capacity.

Both of these options will now be discussed in detail.

The Structural Sheathing of the Wall Framing Elements

When a residence is totally sheathed with structural panels in a manner to assure vertical continuity, the need for metal connectors is significantly reduced. There will only be a need for a metal connector at the roof truss or rafter to the wall and the wall to the foundation. This is a very economical approach to the structural requirements of the tension chain, with only a minimal reduction in the insulation value of the wall. This issue was discussed in Chapter 1.

The essential criteria is for the structural panels to become a continuous plane from the top of the double top plate, across the second and first floor construction and down to the bottom of the mud sill, as shown in Figure 2.9. This figure has been slightly modified from Figure 10 of Chapter 7 in the *WMM*. 
Metal connector required between truss and double top plate.

Metal connector required between truss and double top plate.

Preferred butt splice location. Blocking required.

Nail each panel to the blocking.

Plywood or OSB sheathing.

Nail to all members.

Anchor bolt to foundation.

Foundation weight is the uplift resistance.

Recommended to splice sheathing along studs and provide continuity at floor construction.

Possible butt splice location (not preferred).

Figure 2.9- Total Use of Plywood or OSB Board to Resist Uplift.
The first modification is that the 2x4 wood blocking at a typical splice has been rotated 90 degrees, so that the 3-1/2" width of the 2x4 is flat against the sheathing. This is done for several positive reasons:

1. To provide more room for nailing, avoiding nails on a slant and splits in the wood;
2. To provide for batt insulation to pass the blocking and merely be compressed in that region; and
3. To provide room for electrical wiring to pass vertically through the wall.

The second modification deals with splices at the bandboard of the second floor level. Although it is structurally feasible for a splice to be made at mid-height of the bandboard, it is even stronger to place the splice within the length of the studs above and/or below the floor construction. This avoids the potential of splitting the bandboard perpendicular to the horizontal direction of its grain, and places the splice along the studs, which have their grain parallel to the uplift force.

**Connection of the Roof Truss/Rafter to the Structural Wall Plane**

The structural sheathing starts from the top of the double top plate and runs continuous for the full length of the wall. Connections between roof truss/rafter to the wall no longer depend on the position of the studs in relation to the roof framing. Thus, the metal connector options of Figures 2.5, 2.6a and 2.6b are all possible and the selection merely depends on the magnitude of the uplift force ($R_{up}$) found in Chapter 4. Additional style of metal connectors for this purpose can be found on Figure 5 of Chapter 7 in the WMM. Every roof truss/rafter must be metal connected to the structural sheathed wall.

**Connection of the Structural Wall Plane to the Foundation**

The transfer of uplift force from the sheathing to the foundation is rather uniformly distributed along the mudsill by closely spaced nails into the mudsill. The mudsill is the means of transfer of this uniform uplift to the anchor bolts. The anchor bolts in turn have to transfer a concentrated uplift force into the concrete or grout of the foundation, which is the final link in the chain of resistance. The wind uplift is thus resisted by the weight of the foundation.

Two mudsill conditions that exist are:

1. When the floor joists bear on the wall subject to uplift; or
2. When the floor joists run parallel to the wall subject to uplift.

When the floor joists bear on the mudsill more dead load is present to help reduce the uplift on the mudsill. The joists also help to stiffen the mudsill and avoid bending and shear. The only failure mode likely is that of direct shear of the mudsill at the small round washer. This is illustrated in Figure 19c of Chapter 4 in the WMM.
When the floor construction runs parallel to the mudsill, the mudsill has to resist bending, shear and pullout. The weakest stress, usually shear, will result in the failure mode referenced in Figure 21b of Chapter 4 in the WMM. Square washers are recommended to reduce the tendency for the mudsill to shear and to spread the bearing stress over a larger area. It is recommended that 3 x 3 x 9/64 square washers be used with the anchor bolts at a spacing that satisfies both:

1. The requirement of backfill pressure against a basement wall (Table 3 in Chapter 4 of the WMM); or
2. The required wind uplift on the mudsill and anchor bolts.

There are somewhat problematic field conditions that can be alleviated by the use of the square washers that contain a diagonal slot. Figure 2.10 illustrates several anchor bolt conditions that have been seen far too often by this author.
Anchor bolts are many times placed in the concrete so they will not be centered on a mudsill or bottom plate. Figure 2.10a shows an off center anchor bolt in an oversized hole, where the round washer barely develops any bearing against the wood fibers. A square washer with a slotted hole not only adds bearing but covers most of the mudsill and avoids bending of the mudsill. Should the anchor bolt placement actually miss the mudsill or bottom plate, as shown in Figure 2.10b, the situation worsens. Now the washer squashes the wood fibers and will never produce a tight bearing without further damaging the wood. Again, a square washer will reduce the severity of this situation, but in this case it may extend over the edge of the wood. Note the illustration of thin foam between the concrete and the mudsill. When the top of the concrete is rough and uneven, this foam will try to fill the void and prevent air infiltration. Unfortunately, when the roughness is extensive daylight is visible between the wood and concrete.

Figure 2.10 - Off Center Anchor Bolt

Figure 2.11 illustrates a sloping (non-level) top of foundation wall where wood shims are used to return the stud wall for a garage or mudsill for a first floor to a straight horizontal plane. This is one of the more common, and most time consuming, conditions to correct. It creates an open link in the tension chain for wind uplift resistance. Plus, this condition is typically hidden behind the drywall and unknown to the owners. Figure 2.11 (C) illustrates that when the shim height is too great the anchor bolt cannot project above the wood to install a washer and nut. Figure 2.11 (B) indicates a notched mudsill or bottom plate to allow placement of the washer and nut over the anchor bolt. Here the shear capacity of the wood is severely reduced and so is the uplift resistance. Figure 2.11 (A) is
an unconscionable omission of the nut and washer, even though sufficient anchor bolt length projects above the wood.

Metal straps that act as a belt embedded in the concrete or concrete block grouted cores may substitute for anchor bolts. They have sufficient structural capacity, as long as the strap ends are properly lapped. Figure 9 of Chapter 7 in the WMM illustrates these mudsill to foundation anchorage options. It is recommended to start with the anchor bolt-spacing requirements for the anticipated backfill condition and test whether this also satisfies the uplift requirement on the anchor bolt or strap.

Due to typographic errors in the dimensions of the square washer of Figure 9 in Chapter 7 of the WMM, and the need to clarify its appearance, it is reproduced here as Figure 2.12a.

![Diagram of anchorage to foundation options]

Figure 2.12a - Anchorage to Foundation-Options
Figure 2.12b illustrates an alternate mud sill anchor, using a split-strap connector. This anchor either spreads the straps into a “Y” shape based on the width of the mudsill, and wraps around the mudsill and is nailed to it; or the connector extends out of the hole drilled in the mudsill and the straps are bent down and nailed to it. The former is preferred for cast-in-place concrete, while the later is preferred for use in concrete block cores. **Incorrect use** of this split-strap connector is common, since it is usually bent at 90 degrees to the top of the concrete or block and then wrapped around the mudsill and nailed as illustrated. This misinstallation application reduces its allowable tension capacity and results in significant movement before it will resist any tension uplift.
Walls Sheathed with Insulation Board with Calculable Structural Capacity – with or without Openings

Structural insulation board sheathing comes in various thicknesses from 0.078", 0.115", and 0.137", depending on the shear strength required. It comes in sheets 4'x 8' and the outside is covered with foil. The sheets can be lapped or wrapped around corners by bending. Nails or staples are commonly used to attach the sheathing to the wood skeleton. Thus, the entire house is sheathed with this material. Continuity is essential, just as it is for OSB or plywood structural panels. Too often this author has seen it stopped at floor planes and infill pieces inserted along the bandboard. This reduces the system to isolated pieces and destroys the tension chain of resistance. Staples or nails many times miss the studs behind, or are driven with a gun that forces them to gouge into the panel or even penetrate completely through the panel. Obviously the gun pressure must be calibrated for such a thin sheathing. Although the sheathing has its merits, it requires care in installation.

Exterior/Interior Walls Sheathed with Gypsum Board – with or without Openings

When fire ratings are required for apartment buildings, townhouses or condominiums a contractor may select exterior “X” grade Gypsum sheathing, fastened with nails, staples or screws. The Uniform Building Code (UBC) assigns allowable shear values for use as shear walls, but not for direct tension uplift. This topic will be discussed in Chapter 3.

Creative Tension Chain Ideas Used by Contractors

The natural creative ideas from contractors to minimize labor and material, while still achieving the required strength of the tension chain, are many. A few of these ideas catalogued under the specific categories used above follow.

The Structural Sheathing of the Wall Components

The creative input of one contractor has led to the idea illustrated in Figure 2.13 for the placement of structural sheathing while constructing the entire length of a wall elevation. The top structural sheet of OSB board is placed horizontal at the top of the wall, while it is still lying flat on the first floor plane. The entire wall becomes a stable wall for lifting and placement. Note that the top plate of the double top plate is included at this stage. The remainder of the sheathing is placed from grade, after the wall is positioned vertically and braced. This sheathing is placed with its grain oriented vertically. This approach avoids ladders and results in fewer accidents. This same approach can be done with plywood. Regardless of structural sheathing choice, the sheets must be cut resulting in some waste since the wall height of the sheathed exceeds 8 feet. Reference pages 37-38 and Figure 11 in Chapter 4 of the WMM. When finished, the structural sheathing covers the wall from the top of the double top plate to the bottom of the mudsill in a continuous
tension plane. The blocking at panel free edges is not shown for clarity, but is then installed.

The Roof Truss/Rafter to the Structural Wall Plane

Ideas to eliminate metal connectors between roof truss/rafter and the structural wall plane can be unique. The extension of the structural sheathing to the top of "I" joist as detailed in Figure 2.14a and 2.14b, represents a one-story residence and a two-story residence respectively. The one story solution creates a cathedral ceiling and detailing elsewhere must prevent the horizontal outward movement associated with cathedral ceilings. A 2x6 wood blocking and a sufficient number of nails are required to resist the design uplift and tie the rafter to the wall sheathing. This method is preferred by some contractors to eliminate the time that is required to install the significant number of metal connectors when "I" joists will be used for the roof framing. Overhangs, if desired, can then be prefabricated and attached directly to the wall sheathing. The illustration shows a dotted overhang. The same approach to the tension chain could be accomplished with dimension lumber rafters. A squash type dimension lumber side plate would need to be appropriately nailed to the side of the rafter, again with its grain oriented vertically. This
would allow nailing the structural panel into side grain rather than end grain of the rafter itself.

Figure 2.14a - Roof to Wall Detail (cathedral ceiling)

Figure 2.14b - Roof to Wall Detail (2nd story)
A similar concept is possible with roof trusses that have extended verticals at their bearing ends, as shown in Figure 2.15. Again, a sufficient number of nails are required to resist the design uplift and tie the truss to the wall’s structural panels.
The use of roof trusses with an extended vertical is not very common in conventional residential construction, but has a particular function in custom homes as shown in Figure 2.16. It is possible to maintain a constant roof height for a home, while varying the interior spaces. For example, a taller (9 foot ceiling in this case) cathedral space in one location, while having an 8 foot flat ceiling in other spaces with a second story or an attic above, might utilize all three details shown in Figures 2.12a and 2.12b and 2.13. Again, this is only one approach to the variable ceiling height issue. Note that the “I” joist rafters are tied together at the ridge with solid “L” shaped plywood gussets to provide some continuity and moment resistance to the outward movement at the walls of the cathedral space.

![Diagram of Variable Interior Ceiling Heights](image)

Figure 2.16 - Variable Interior Ceiling Heights
The Wall Sheathing to the Foundation

One contractor’s approach to dealing with the large washers at the garage walls, was to place a first bottom plate on top of the foundation wall without recesses at the anchor bolt locations. He then used these as a template for a second bottom plate that then became the bottom plate for construction of the stud wall. Thus, the full strength of the 2x4 mudsill was available and there was no interference from the anchor bolts for the layout of the studs. Figure 2.17 illustrates this detail.

![Diagram](image)

Figure 2.17 - Garage Stud Wall
**Uplift Recommendations for Sections of Walls with Openings**

The above discussion applied to exterior solid wall areas, where no doors or windows are present. Openings interrupt the transfer of uplift force through the sheathing. The uplift force suddenly concentrates at each side of every opening, transferring from the header over the opening. The larger the opening, the larger the tension uplift force to be transferred to the foundation. The span of the wood header ($H_l$) over the opening is the key to determining the magnitude of the concentrated force to be transferred to the studs and then to the foundation. It is clear that normally the garage door opening will be the largest concentrated force to be transferred to the foundation. Figure 2.18 illustrates typical openings.

![Diagram showing uplift forces and connections](image)

**Figure 2.18 - Typical Openings in Exterior Wall**

Figure 2.19 is a dissected view of the components that surround an opening, and the design wind suction reactions at each roof truss or rafter. The arrows show the concentration of forces transferring around the opening. The important connections are as follows:

1. The roof trusses or rafters to the header, bypassing the double top plate;
2. the header bearing ends to the studs; and
3. the studs to the foundation, bypassing the first floor construction.
Figure 2.19 - Exploded View of Uplift at Opening in Exterior Wall
These three connections will be discussed separately.

A longer metal connector or a twist strap is required to reach the wood header and to transfer each roof truss reaction to the header. It is also possible for the structural sheathing to transfer this suction to the header, but that will depend on the magnitude of the truss uplift values.

**Header to Studs at Ends of the Opening**

The header tries to bend upward and lift off the jack studs. Thus, the header transfers an uplift end reaction ($R_{u}$) to the jack and full-length studs, typically by means of metal straps. It is true that the structural wood sheathing may be able to assist or transfer all of the reaction from the header. It is not desirable to concentrate too much force into the sheathing and depend on a group of nails to be positioned just right. Two car garage door openings support the longest headers. If the header supports the bearing ends of roof trusses, the entire 16-foot header span is subjected to uplift. This will most likely be the longest single span around the exterior of the residence, and will produce the largest uplift force to be transferred to the foundation.

**Studs to the Foundation**

The uplift force ($R_{u}$) is transferred from the studs to the foundation by means of hold-downs, such as found in Figure 14 of Chapter 7 in the WMM. These hold-downs use a rod that is drilled through the subfloor and bottom stud wall plate. The rod then goes past the bandboard and is drilled through the mudsill into the foundation. These rods are typically epoxied into the concrete foundation or grouted block cell, rather than placed during the foundation concrete or grout placement.

All the connectors will be larger or longer to accommodate more nails for garage door openings. The hold-downs will be a larger diameter rod embedded deeper into the foundation and more foundation weight will need to be engaged.

**Covered Entry Porches or Patios**

There is an infinite variety of possible entry-porch and patio forms and proportions that are linked to residences. A roof section is generally provided over them for weather protection. These roofs are many times open on three sides and are therefore supported at the open-end corners by posts.

Wind can move under as well as over these roofs creating positive pressure on the underside and negative pressure (suction) on the topside. The pressure pushes up on the underside of the open roof and the suction pulls up on the top surface of the roof. Thus, these two wind actions combine to produce considerable uplift on the roof.
The ASCE 7-95 wind provisions refer to this as an “open structure” and mathematically treats this condition differently than enclosed buildings to determine the uplift design force. It would be impossible to treat every porch or patio condition, but a typical porch will suffice to describe the approach to the tension chain of resistance to uplift. Figure 2.20 illustrates such an entry porch. The porch roof is attached directly to the roof of the residence, and is usually composed of rafters in combination with roof trusses as shown. The portion of the porch roof that projects is supported by beams that are in turn supported by corner posts that bear on a concrete porch that is supported by a foundation below grade. The projection of the roof beyond the house \( B_p \) is an important dimension. The structural wood posts are usually a pressure treated (wolmanized) 4x4 or 6x6, depending on strength or aesthetic considerations. This will avoid rotting of the wood due to exposure to moisture.

![Diagram of a typical porch and support posts](image-url)

*Figure 2.20 - Typical Porch and Support Posts*
The load flow from wind uplift follows a typical pattern from roof to foundation, and the following connections compose the tension chain:

1. Roof trusses or rafters to the projecting beams
2. Beams to the wall of the residence and the corner post
3. Corner post to the porch concrete
4. Porch concrete to the foundation

Figure 2.21 separates all the porch components that require connections to complete the tension chain. These connections will be discussed separately.

![Diagram of Porch Uplift Chain](image-url)
Roof Truss or Rafter to the projecting beam

Once the design wind uplift reaction is determined, metal connectors can be selected to tie the rafters or trusses to the wood beam. These connectors would be very much like those found in Figure 5 of Chapter 7 in the WMM.

Beam to the corner post and the wall of the residence

The uplift reaction at the ends of the beam is \( R_p \). This is the design uplift force that must be transferred from the beam to the post. Many metal connector options exist, depending on the magnitude of \( R_p \). Several examples are shown in Figure 2.22. Several depend on nails for load transfer, while some depend on bolts. Bolted connectors will have bolt head, nut and washers projecting from the post. If the post is clad with finish wood, shim material will be required. This architectural treatment will make the size of the column much bigger than structurally required.

Figure 2.22 - Typical Post Cap to Beam Connections for Uplift Resistance
Corner post to the porch concrete

There is a variety of column post bases that are capable of transferring tension uplift to the porch concrete. Several such metal connectors are illustrated in Figure 2.23. Notice again, that either nails or bolts, or a combination of both are used to connect to the wood post. It is assumed that either the connector is set in the concrete as the porch is poured, or that an epoxy anchor is drilled and set after the concrete cures. Most contractors prefer the later detail, as setting the metal base requires exact placement or the post will not be straight.

Figure 2.23 - Typical Post Base Connections for Uplift
Porch Concrete to the Foundation

It is most likely that sufficient weight of concrete exists in the porch construction and its reinforced connection to the foundation that the tension chain is complete.

Cantilevered Second Floor Planes

Cantilevered second floor planes are fairly common. Figure 2.24 (A) illustrates when the floor and walls above and below have their framing in alignment. Metal connectors can be used throughout to accommodate the tension chain of wind resistance. Structural sheathing is another alternative even if the framing aligns. Figure 2.24 (B) illustrates when the wall and floor framing does not align. Structural sheathing is the likely choice here; however, a metal connector is still required to tie the second floor to the lower wall plane.
Insulating Concrete Forms

There are numerous types of insulating forms used to create reinforced cast-in-place wall systems [2.5]. A few of these are illustrated in Figure 2.25. Regardless of the system employed, the basic concept is to use 2 layers of rigid insulation separated by ties that hold horizontal and vertical reinforcement in place and maintain their verticality while concrete is poured between the insulation layers. The end result is a box of insulated reinforced concrete walls that can have other finish materials applied to the insulation inside and out. The forms can be used for crawl spaces, full basements or even two story walls. They are becoming popular for two story condominiums, where they are used as separation firewalls between condo units. Ledger boards are expansion bolted, epoxy anchored or anchor bolted at the floor levels. Metal joist hangers attached to the ledger boards then support the wood floor level framing.

Significant weight is provided in these walls, whether for crawl, basement or two story applications. The uplift chain of resistance is easily provided by this system. Wood roof trusses or rafters are installed atop the two story walls to complete the roof enclosure of the wall system. This truss/rafter connection to the concrete wall is the only metal plate connection that must be provided to start the tension chain resistance from the roof. Reinforced concrete block walls can also be built to provide the walls of the box. Similar details are incorporated to tie the roof to the block wall. Both of these wall types and representative roof truss/rafter details are shown in Figure 2.26. A potential problem, when the rigid foam insulation comes in contact with the earth, is the potential for increased termite infestation. There is no definitive explanation for this surprising statement. The CABO Code now requires rigid foam insulation to start a minimum of 6 inches above grade. Insulating concrete forms can be used for foundation walls if non-combustible
materials, such as steel, or treated lumber are used for the wood framing mudsills of the superstructure. This is not an issue when the insulating concrete forms extend from the footing to the roof. No access to wood is provided for the termites [2.6]. This issue is beyond the scope of this Manual and the references at the end of this chapter should be consulted.

Figure 2.26 - Typical Concrete/CMU Truss Connectors
Tension Chain Summary

It is clear that all the exterior walls, plus the roof and foundation components of a residence participate in the tension chain of resistance. This resistance can be totally accomplished with metal connectors or by a combination of roof and foundation metal connections linked to a wall that is structurally sheathed continuously from the top of the double top plate to the bottom of the mudsill. This sheathing can be accomplished with either Structural panels.

The total metal connected tension chain of resistance can incorporate the typical types of non-structural exterior 1-inch rigid insulation board. Yet the stability of the residence will require some structural sheathing to resist overturning and sliding.

The use of continuous exterior OSB or Plywood can be used to develop the total tension chain of resistance. Super batt insulation rolls inside the walls can make up for most of the lost R-value. However, it should be kept in mind that the real insulation loss is not at the solid wall locations, but at the seals around openings where infiltration of air is the main loss of R-value. It might be of interest to use 2x6 stud walls instead of 2x4 stud walls to accommodate more insulation, however this may not provide an R-value worth the increase in stud and insulation cost [2.7]. This issue is beyond the scope of this Manual and the references at the end of the chapter provide further reading.

Sufficient foundation weight is required to complete the tension chain of resistance and hold down the house. This weight is easily provided when basements are used, but less weight is available when crawl spaces or slabs on grade are used. Obviously, unique situations found mainly in custom homes require use of internal anchorage techniques not covered here. The input of a structural engineer is recommended when homes become that complex.
Chapter 2  Wind Structural Planning Basics – The Tension Chain

Selected References

**Chapter 3 - Wind Structural Planning Basics**

**The Stability System**

**Introduction**

The stability issues of **Overturning and Sliding** comprise the last of the structural design issues for the lateral load resistance system. These topics will comprise the emphasis of this chapter. This is without a doubt the most complicated, but most important issue for the planner of a residential floor plan and its elevations. The Chapter will begin with basic concepts and then gradually apply these concepts in the structural planning of several residences of varying floor plan complexity.

*Main Wind-Force Resistance System* wind loads, as described in the “Wind Loads For Design” section of Chapter 2, are applied to the entire exterior planes of buildings for Overturning and Sliding resistance design. The term “diaphragm” for structural use of floor and roof planes and the term “drag strut” will be fully explained in this chapter. See Figure 3 in Chapter 5 of the WMM. The illustrations and text of Chapters 5 and 7 of the WMM will be used to supplement the information and ideas presented in this chapter.

**The Stability System Defined**

The stability of the total residence involves its resistance to **overturning** and **sliding** as a total building. The wind loads used for the stability design are those of the *Main Wind-Force Resistance System*, described earlier. Wind pressures, both positive and negative, hit large tributary surface areas perpendicular to the wind direction. These surfaces transfer the wind pressures and suctions to the roof and floor planes and then to the vertical wall planes parallel to the wind direction. Now the horizontal components of the wind applied to the building are to be resisted; as shown in Figures 10 and 11 of Chapter 1 in the WMM along with all the figures and text of Chapter 5 in the WMM. Wind resistance for stability is generally an engineering review in two primary directions:

(a) Perpendicular to the long dimension of the building; and  
(b) perpendicular to the short direction of the building.

These are illustrated in Figure 3.1.
Just as for the issue of uplift and the tension chain of resistance, the building again must be thought of as composed of planes: wall, roof and floor planes. Since the primary wind force is now horizontal, specific planes must be selected to perform the resistance function.

Figure 3.2 has exploded a simple two-story rectangular residence. For the moment assume there are no openings in the exterior walls, thus all the planes are solid. A wind force perpendicular to the long dimension of the building means that the largest surface area is exposed to the wind. The largest sliding and overturning forces generally occur when wind is applied perpendicular to the long dimension of the building.

Returning to Figure 3.2, the windward walls and windward sloping roof plane receive the wind positive pressure, while the leeward walls and leeward sloping roof plane receive the negative pressure (suction). The wind forces against these wall plane and sloping roof plane surfaces, perpendicular to the wind direction, push and pull at the roof and floor planes, which are called “diaphragms”.

Diaphragms behave like deep beams with their “web” being analogous to the floor plane or roof plane. Diaphragms therefore are assumed to behave like beams laid on their side, sending their wind generated reactions to the end walls, called “shear walls”.

Figure 3.1 - Stability-Wind Directions for Structural Analysis

(a) Wind Perpendicular to long dimension of building

(b) Wind Perpendicular to short dimension of building
The walls that are physically connected to the diaphragms receive their share of the reaction according to a distribution based on tributary area. Tributary area is measured as half the distance to the other shear walls multiplied by half the height of the wall. Once the wind reaction is transferred to the shear walls, the connection of the shear wall to the diaphragm forces the reaction to move down the wall in shear. Thus the name “shear wall”. The shear is resisted by the material of the shear wall and it continues down the wall to the foundation. These shear walls must resist the tendency to slide and the tendency to overturn. This is accomplished by proper connection and anchorage to the mass of the foundation below the shear walls. The sliding is resisted by foundation anchor bolts spread along the length of the bottom of the shear walls. The overturning is resisted by anchorage to the foundation wall at the ends of the shear walls. Again, reference should be made to Figures 3 and 4 of Chapter 5 in the WMM.
Figure 3.3 has exploded the same simple two-story rectangular residence shown in Figure 3.2. Again, for the moment assume there are no openings, and all the planes are solid. The wind force is now perpendicular to the short dimension of the building. This means that the smaller surface area is now exposed to the wind. **A smaller sliding and overturning force generally occurs when wind is applied perpendicular to the short dimension of the building.** Returning to Figure 3.3, the windward end walls and windward gable end wall receive the positive wind pressure, while the leeward walls and leeward gable end wall receive the negative wind pressure (suction). The wind forces against those surfaces push and pull against the roof and floor planes, again called the “diaphragms”. These diaphragms are now extremely deep beams, sending their wind reactions to the side walls, which now become the “shear walls”. These long shear walls transfer the smaller buildup of horizontal force in their plane in shear. They become the stability resistance system for wind perpendicular to the short direction of the residence. These long shear walls must resist the tendency to slide and the tendency to overturn. This is accomplished by connection and anchorage to the mass of the foundation. The sliding is resisted by the foundation wall anchor bolts spread along the length of the shear walls. The overturning is resisted by anchorage to the foundation wall at the ends of the shear walls. It is obvious, as solid very long walls, they generally have no trouble providing the required stability.
This very basic example, when looked at in two mutually perpendicular directions, illustrates what commonly must be reviewed on every residence – most not as simple as the basic rectangle. However, it will serve to point out some very important structural planning concepts applicable to developing the stability system of any residence.

**Stability Concepts Used in Structural Planning to Resist Sliding and Overturning**

**Foundation Walls**

1. It is most desirable to have foundation walls under the superstructure walls selected as the shear walls for stabilizing the residence in each analysis direction.

   This concept is intended to provide sufficient foundation mass (weight) under the shear walls to be able to resist the overturning tie-down forces and use the foundation anchor bolts to resist the sliding. When the house proportions of length to width are less than 4, only exterior end walls are required for shear walls and therefore this concept is automatically provided.

   When the house proportions of length to width exceeds 4, an interior wall must be incorporated as a shear wall to reduce the proportions between shear walls back to 4 or less. Since an interior shear wall should also have a foundation below it, it is best to select a superstructure wall where a foundation change between a basement and a crawlspace takes place below a potential shear wall, or a basement and a slab-on-grade takes place below a potential shear wall. These concepts are illustrated in Figure 3.4.

   Figure 3.4(a) and detail Figures 9 and 11 in Chapter 7 of the WMM shows the typical sliding and overturning that occurs between the shear wall and foundation at a garage condition. Figure 3.4(b) illustrates either a basement or crawl space condition, where a floor plane separates the shear wall from the foundation. Longer anchor rods are needed for the overturning tie-down anchors and again anchor bolts are used to resist the sliding, which is really the uniform shear along the base of the shear wall. Figures 9 and 14 in Chapter 7 of the WMM illustrate typical construction details.
a. At Exterior Garage Wall

b. At Exterior Foundation Wall-Basement or Crawl Space

Figure 3.4 - Concept 1-Foundation Wall Below Shear Wall
Garage Walls

2. Garage walls should always be used as tension chain walls to resist uplift forces and also as shear walls to provide the stability to resist overturning and sliding forces.

Garage walls, except for the wall that contains the door opening, are an excellent choice for shear walls. Garage walls are particularly vulnerable in strong winds after the garage door has been breached. The tension chain to resist the increased uplift will require additional fastener requirements. These same walls will need to perform double duty and provide stability as shear walls for the increased horizontal wind forces. Luckily garage walls generally have foundations beneath the walls and one merely needs to check the foundation dead weight for its ability to complete the uplift tension chain resistance and the stability (sliding and overturning) resistance. So, whenever possible garage walls should be sheathed with structural panels and used as shear walls. Figures 6 and 7 in Chapter 1 of the WMM refer to the generic external and internal wind forces generated on an enclosed and open garage respectively, while Figure 8 in Chapter 1 of the WMM shows the distorted garage envelope relative to the wall elevation that contains the breached door. Figure 3.5a numerically illustrates the wind magnitudes for a closed garage door and then an open garage door. The wind pressures are based on Main Wind-Force Resisting system (MWFRS) values at 90-mph wind speed.

![Diagram of enclosed and open garage wind magnitudes](image-url)
The garage is detached from the house and has dimensions 20' x 24' x 10' high and a 5 in 12 gable roof slope. The wind is coming towards the elevation that contains the garage door. It is clear when the garage door is open or breached that the wind enters the garage and produces an increased roof uplift resistance requirement of approximately 50%, and a leeward wall suction that increases stability requirements for sliding and overturning resistance by almost 100%.

**Garage Doors**

3. Garage Doors should be selected to resist component and cladding wind pressures based on the selected design wind speed. Perimeter framework and door anchors should also be designed for the selected wind speed.

A. Garage door selection

It has been stated above in item 2, that if a garage door is structurally damaged and permits a strong wind to enter the garage space, internal pressures subject the side walls and back wall to considerably higher pressures. Pressures against all internal surfaces will generally lead to the garage being lifted off its foundation, the roof being torn off or the walls being blown out. Further damage will occur to the residence if the garage is attached. Figure 8 in Chapter 1 of the WMM illustrates potential garage behavior.

The selection of a garage door, sufficiently reinforced and tested, to resist a particular design wind speed is very important to the structural integrity of the garage and the attached residence. The area of an attached single garage door (7' x 9') or double garage door (7' x 16') is usually quite small in comparison to the total area of the elevation that contains the door. Components and Cladding wind pressures as defined in the ASCE 7-93 [3.1], and referenced by all the major codes, are thus used to account for localized higher wind pressures than typically used for Main Wind-Force Resisting systems.

Figure 3.5b is taken from the ASCE 7-93 [3.2] to illustrate the variation in wind pressure near corners (region 5) and between corners (region 4) as discussed in detail at the beginning of Chapter 2. A sample two story residence is shown with dimensions and the garage door location. The door falls within both regions 4 and 5 as shown by the change in cross-hatching. Thus, 2' of the door falls in Region 5 and 14' falls in Region 4. Area averaging is used to arrive at a single negative and positive pressure value. A set of curves shown on the graph defines external pressure coefficients for both regions and for negative and positive pressure coefficients. Notice that the coefficients are largest for very small areas, such as garage doors. The 63 sq.ft. for a single garage door or the 112 sq.ft. for a double garage door have
coefficients that are much greater than 1.0, indicating that the positive and negative pressures for Components and Cladding will be greater than Main Wind-Force Resisting System pressures.

Note a. The vertical scale denotes $GC$, to be used, based on Exposure C.
Note b. The horizontal scale denotes the wind tributary area ($A$) in square feet.
Note c. External pressure coefficients for walls are permitted to be reduced by 10 percent where $\theta \geq 10$ degrees.
Note d. Plus and minus signs signify pressures acting toward and away from the surfaces, respectively.
Note e. Each component shall be designed for maximum positive and negative pressures.
Note f. Notation:
- $a = 10$ percent of minimum building width or $0.4h$, whichever is smaller, but not less than either 4 percent of the minimum building width or 3 feet.
- $h = \text{Mean roof height, in feet, except that the eave height is permitted to be utilized where } \theta \geq 10$ degrees.
- $\theta = \text{Roof slope from horizontal, in degrees.}$

Figure 3.5b - External Pressure Coefficients for Components and Cladding and Buildings with Mean Roof Height Less than or Equal to 60 Feet [3.2]

The “Door and Access Systems Manufacturers Association” (DASMA) has developed a Garage Door Wind Load Guide [3.3] that charts the Components and Cladding pressures on garage doors for wind speeds from 70 to 110 mph for all three model codes. The Guide covers one and two story structures with one or two car garages. Table 3.1 on the next page charts the positive and negative pressures in pounds per square foot based on the equations found in the 1996 BOCA Code [3.3] with accompanying notes, and use of Figure 3.5b.
Table 3.1

DASMA Garage Door Wind Load Guide
BOCA 1996 NBC

<table>
<thead>
<tr>
<th>MEAN ROOF HEIGHT</th>
<th>DOOR SIZE</th>
<th>70 MPH</th>
<th>80 MPH</th>
<th>90 MPH</th>
<th>100 MPH</th>
<th>110 MPH</th>
</tr>
</thead>
<tbody>
<tr>
<td>15 Feet Single</td>
<td>Single</td>
<td>14.7 psf*</td>
<td>19.2</td>
<td>24.2</td>
<td>29.9</td>
<td>36.2</td>
</tr>
<tr>
<td></td>
<td>9’ x 7’</td>
<td>-16.3</td>
<td>-21.2</td>
<td>-26.9</td>
<td>-33.2</td>
<td>-40.2</td>
</tr>
<tr>
<td></td>
<td>Double</td>
<td>14.1</td>
<td>18.4</td>
<td>23.3</td>
<td>28.7</td>
<td>34.8</td>
</tr>
<tr>
<td></td>
<td>16” x 7’</td>
<td>-15.3</td>
<td>-20.0</td>
<td>-25.3</td>
<td>-31.3</td>
<td>-37.8</td>
</tr>
<tr>
<td>25 Feet Double</td>
<td>Single</td>
<td>17.1</td>
<td>22.3</td>
<td>28.2</td>
<td>34.8</td>
<td>42.1</td>
</tr>
<tr>
<td></td>
<td>9’ x 7’</td>
<td>-18.9</td>
<td>-24.7</td>
<td>-31.3</td>
<td>-38.6</td>
<td>-46.7</td>
</tr>
<tr>
<td></td>
<td>Double</td>
<td>16.4</td>
<td>21.4</td>
<td>27.1</td>
<td>33.4</td>
<td>40.4</td>
</tr>
<tr>
<td></td>
<td>16” x 7’</td>
<td>-17.8</td>
<td>-23.3</td>
<td>-29.4</td>
<td>-36.4</td>
<td>-44.0</td>
</tr>
<tr>
<td>25 Feet Flat</td>
<td>Commercial</td>
<td>15.1</td>
<td>19.8</td>
<td>25.0</td>
<td>30.9</td>
<td>37.4</td>
</tr>
<tr>
<td></td>
<td>10’ x 10’</td>
<td>-16.6</td>
<td>-21.7</td>
<td>-27.5</td>
<td>-33.9</td>
<td>-41.1</td>
</tr>
</tbody>
</table>

*Chart represents calculated “design” pressures (in psf, pounds per square foot) for the listed code standard representing common installations.

The local building authority may require testing to verify product performance. DASMA suggests that this product testing, if required, be performed to ASTM-E330. Test conditions:

1. Product shall be tested to both negative and positive pressures. Product shall be installed to normal conditions (i.e. top rollers in track radius, proper counterbalancing, etc.).

2. Total test duration for each test direction shall be as follows:

52 seconds (70 MPH), 45 seconds (80 MPH), 40 seconds (90 MPH), 36 seconds (100 MPH), and 33 seconds (110 MPH) at design pressure. Pressure equal to 1.5 times the design pressure to be included for 10 seconds in each test.

Product successfully passes the test if the door remains safely operable through full travel of the door track and recovers at least 75% of its maximum deflection. Sound engineering principles may be used to interpolate or extrapolate test results to door sizes not specifically tested.

This guide is provided for reference purposes only. In all cases the local building authority is the sole and final determiner of the requirements and suitability of product.

Continued
Notes:

- Chart displays the design pressures calculated from the 1996 National Building Code (NBC) for a typical single story residential house, a typical double story residential house, and a typical commercial building.
- The pressure requirements decrease for larger sizes and increase for smaller sizes.
- Negative pressures assume door has 2 feet of width in building's end zone.
- Garage doors evaluated as Components and Cladding, Exposure C.
- Garage doors evaluated as attached to enclosed buildings with an Importance Factor of 1.0.

Owners, contractors, architects and engineers can select a garage door from a manufacturer based on the values contained in Table 3.1 and required compliance with ASTM-E330 test results supplied by the manufacturer. As wind design speeds increase from 70 to 110 mph, so does the type and gauge of door reinforcement and hardware, plus the size and number of fasteners between door tracks and perimeter dimension lumber. Figures 3.5c and 3.5d illustrate typical details for a generic 90-mph rated door. Shown in these two figures are (1) the parts of the door where panels hinge; (2) the lightgage metal tracks and rollers; and (3) the connections between tracks and the wood framing. Also shown are the panel rib stiffeners.

![Diagram of Generic Garage Door Jamb Detail](image-url)
B. Door anchorage and wood frame requirements

Figures 3.5c and 3.5d basically familiarize the user with door terminology and how door components relate and fit together. It is thus possible to discuss how wind loads flow and transfer between the lightgage metal parts and the opening wood framing.
Wind load transfers between door components as shown in Figure 3.5e in the exploded perspective.
Wind positive pressure (or negative pressure: suction) pushes or pulls on the door panels. Due to the rib reinforcement behind each panel the wind load is transferred to the rollers that bear within the vertical metal tracks. These tracks are anchored to the wood frame (jamb) by means of anchor brackets that are lag screwed to the wood studs adjacent to the track. The enlarged view of a single bracket, track and roller illustrates the load transfer when positive wind pressure is applied to the door. The lag screws will be in withdrawal from the wood when positive pressure is against the door panels. Since lag bolts (or lag screws as another name) are threaded the NDS provides allowable withdrawal load capacities per inch of threaded length [3.4].

The lag screws will be in compression when a negative wind pressure is applied to the door panels. This is when the largest loading is on the wood studs that act as the jamb on each side of the door. The doubled or tripled studs will be subjected to bending stress and will deflect outward. The strength and stiffness of the wood studs at the door jamb are important to the strength of the whole door. A numeric example will be detailed in Chapter 5.

A tie-down anchor into the foundation is shown adjacent to the jamb. This is a recommended location for tie-downs. If the garage door is breached by the wind, direct uplift will necessitate a tie-down. Should the length of the wall either side of the garage door opening be sufficient to act as a shear wall, the tie-down will also serve the purpose of overturning stability - besides providing uplift resistance as part of the tension chain.

**Drag Struts**

4. Drag struts are a useful engineering device to transfer wind load between shear walls. Specifically, shear walls that are not aligned between levels or shear walls that do not extend the full width of the floor or roof diaphragm.

This is an appropriate time to discuss the definition and use of drag struts, as shown in Figure 3.6a. Assume a wind direction from right to left as shown, producing wind positive pressure and wind negative pressure (suction) on the windward and leeward walls respectively. The roof diaphragm receives wind from half the height of the wall, producing illustrative values of 100 plf on the windward side of the diaphragm edge and 200 plf on the leeward side of the diaphragm edge for a total of 300 plf. This example uses two exterior shear walls and one interior shear wall with foundation walls below the three shear walls. The exterior shear walls “A” and “C” run the full width of the residence and receive the wind reaction directly from the diaphragm along their entire length. It is the interior shear wall “B” that does not receive the wind from the diaphragm directly, since it does not start from the windward side of the roof diaphragm and end at the leeward side of the roof diaphragm. It is most helpful to incorporate a beam in the plane of the roof
that will stiffen the diaphragm and force a distribution of wind to the shear wall “B”. The function of a drag strut is to assist in directing and transferring wind load to a shear wall.

To mathematically illustrate the distribution of shear from a diaphragm to the shear walls, the magnitudes of windward and leeward wind on the diaphragm will be used. **Wood diaphragms are considered flexible diaphragms and thus are assumed to distribute load to the shear walls based on tributary widths.** Tributary widths are defined as half the distance between adjacent shear walls. Starting with the exterior shear wall “A”, the total wind reaction at the wall will be a tributary width of half the distance to shear wall “B” multiplied by the total horizontal diaphragm wind load at the roof, i.e. $30'/2 \times (100\text{plf} + 200\text{plf}) = 4500$ pounds. Since shear wall “A” is 60 feet long, this means $4500 \text{ Lbs}/60' = 75 \text{ plf}$ of shear per lineal foot that must be transferred from the diaphragm and spread along the shear wall.
Connections could be closely spaced nails or more widely spaced metal connectors. Exterior shear wall “C” similarly receives a wind reaction from the diaphragm of 40'/2 x 300 plf = 6000 pounds. Since the shear wall is 40 feet long, this means 6000Lbs/40’ = 150 plf of shear per lineal foot that must be transferred from the diaphragm and spread along the shear wall. The interior shear wall “B” receives its reaction from a tributary width equal to half the distance to the shear wall “A” plus half the distance to the shear wall “C”.

Figure 3.6b is an enlarged view of the wall plane that contains shear wall “B”, the drag strut and the foundation wall below the shear wall.

The roof sheathing is the diaphragm that receives the uniform horizontal wind force of 300 plf. The shear wall “B” must carry the total wind reaction of (30'/2 + 40'/2) x 300 plf = 10,500 pounds. The shear wall “B” plus the drag strut length must transfer 10,500 Lbs/60’ = 175 plf of shear along the total length of 60 feet. The drag strut delivers 175 plf x 40’ = 7000 pounds to the shear wall and it must be connected to the shear wall to transfer that reaction. The shear to be transferred and spread along the length of the shear wall “B” is 10,500 lbs/20’ = 525 plf. This
is considerably more shear per foot than the exterior shear walls must receive and transfer from the diaphragm. **Interior shear walls generally carry more shear per foot because of their increased tributary width.**

The foundation wall that exists under shear wall “B” receives the 525plf and resists that shear by the anchor bolts. Assume 4 anchor bolts exist, thus each anchor bolt must resist 10,500 lbs./4 = 2625 lbs. The *sliding* of the foundation due to the 10,500 pounds must be resisted by friction between the concrete and the soil plus the passive soil block shown. The *overturning* tendency of the shear wall produces an uplift force at the end of the shear wall. This force is determined by taking moments about point “X”. The uplift value is \((12’ \times 10500 \text{ lbs.})/20’ = 6300 \text{ pounds of tension, conservatively ignoring the shear wall weight and any roof dead load.}

This example has the roof framing spanning parallel with the shear walls and very little roof gravity dead load is transferred to the shear walls. The tie-down anchor, near the end of the shear wall, must transfer the tension to the foundation wall. The weight of the concrete foundation must in turn resist the tie-down force from overturning the concrete shear wall. The basement wall tends to rotate (overturn) about the point “Y”. Summing moments about point “Y” produces 10’ x 20000 lbs. – 20’ x 6300 lbs. = -6280 ft-lbs. The negative answer proves there is sufficient dead load in the foundation to resist the overturning and the uplift. Various details are available to connect a drag strut to a shear wall.

Figure 3.6c illustrates two possible approaches when dimension lumber framing is used. The upper detail occurs when the drag strut beam extends continuous across the width of the roof and actually rests on top of the shear wall. The lower illustration is a detail when the drag strut beam stops, or is discontinuous, at the shear wall. It becomes essential to tie the two elements together and provide a tension resistance capability. Note the placement of an angle to support the gravity load carried by the reaction at the end of the drag strut beam. The tension straps resist the tension pull.

Conditions where foundations do not exist under shear walls, and/or first floor shear walls do not exist under second floor shear walls, are very complicated and uneconomical, although sometimes unavoidable. These conditions commonly occur at interior walls. Figure 3.7a illustrates both conditions occurring on the same two-story residence, and shows the need for drag struts at each level to complete the transfer of load between roof and floor diaphragms to the offset shear walls at each level. These shear walls lie in the same overall building plane, but not directly under one another. The horizontal wind reactions to each shear wall are calculated based on tributary area just as for the previous example, resulting in the concentrated wind reaction arrows shown at each level. The shear walls do not extend across the full width of the building at each level and it is required that drag struts force the wind reactions over to each of the shear walls.
Figure 3.6c-Alternate Drag Strut Connections to Shear Wall
Conditions where foundations do not exist under shear walls, and/or first floor shear walls do not exist under second floor shear walls, are very complicated and uneconomical, although sometimes unavoidable. These conditions commonly occur at interior walls. Figure 3.7a illustrates both conditions occurring on the same two-story residence, and shows the need for drag struts at each level to complete the transfer of load between roof and floor diaphragms to the offset shear walls at each level. These shear walls lie in the same overall building plane, but not directly under one another. The horizontal wind reactions to each shear wall are calculated based on tributary area just as for the previous example, resulting in the concentrated wind reaction arrows shown at each level. The shear walls do not extend across the full width of the building at each level and it is required that drag struts force the wind reactions over to each of the shear walls.

Figure 3.7a - Section at Interior Offset Shear Walls (within same plane)
The bottom chords of doubled trusses are used as the drag strut beam at the roof level to transfer the wind reaction from the roof plane diaphragm to the top of the second floor shear wall labeled “A”. Connections capable of making the horizontal shear transfer will require nails, lag screws or metal connectors.

The shear force at the bottom of the second floor shear wall adds to the second floor level horizontal wind reaction and must transfer the entire reaction uniformly along the second floor drag strut beam, which is a doubled “I” joist. The second floor drag strut joists sends its total shear force to the top of the first floor shear wall, labeled “B”. The drag strut must connect to the shear wall to transfer its total shear force and the shear wall “B” must transfer the total sum of all the wind reactions above it uniformly across its length. Thus, the shear force transferred to the first floor shear wall is larger than the shear force transferred to the second floor shear wall. Connections capable of making these horizontal shear transfers include nails, lag screws or metal connectors.

The horizontal shear force at the bottom of shear wall “B” must add to the wind reaction, and again transfer along the first floor doubled “I” joist drag strut uniformly in shear. This time a steel beam is attached to the doubled “I” joist and takes over as the drag strut to transfer the sum of all the wind reactions at the roof, second floor and first floor to the foundation wall “C”. This scenario completes the horizontal sliding resistance.

Figure 3.7b is a section through all of the shear walls and drag struts of Figure 3.7a with all the horizontal shear connections satisfied by the use of nails. This is followed by Figure 3.7c, which is a side elevation of shear walls “A” and “B” to help clarify the connections shown in the section of Figure 3.7b.
Figure 3.7b - Section Through Shear Walls
Wind Reaction
Bottom chords as drag strut

Shear Wall

Doubled roof truss

Tie-down anchor rod

Metal plate, washer, and nut

Wood blocking at rod location

Doubled floor joist drag strut beam

1st Floor

Tie-down anchor rod

Metal plate, washer, and nut

Steel beam drag strut

2nd Floor

Doubled floor joist drag strut beam
Each shear wall in Figure 3.7a also requires overturning tie-down anchors at the ends of each wall. These anchors resemble Figure 14 and Figure 16 of Chapter 7 in the WMM. The doubled joists and the steel beam are thus used in bending to accommodate the concentrated tie-down forces, combined with the axial tension or compression force for drag strut behavior. This double duty accounts for why the trusses and joists are doubled.

Figure 3.8 is a perspective of just the first floor construction and foundation; between the steel beam drag strut to the in-plane concrete foundation walls that are used as shear walls as shown in Figure 3.7a. The sequence of load transfer to the steel beam is described in detail above for Figures 3.7a, 3.7b and 3.7c. The horizontal shear transfer and the added components of wind load related to the first floor shear wall down to the concrete foundation can be summarized as described below:

(1) Place two floor-framing joists directly under the shear wall “B”;

(2) Connect shear wall “B” to the doubled floor joists below with lag screws, spikes or metal connectors to transfer the horizontal wind reactions similar to that shown in Figure 3.7b for nails only;

(3) Place a steel beam below the doubled joists to span between the in-plane foundation walls that run parallel to the shear wall. This step is not always necessary, particularly if the doubled wood joists are short enough in span to reach the foundation walls and adequate in strength to act as the drag strut and tie-down support;

(4) Connect the double joists (if a steel beam is used) to the steel beam, most likely with screws as shown in Figure 3.7b;

(5) Weld steel channels to the steel beam at each parallel foundation wall, and then use expansion bolts to tie the steel channels to the foundation wall;

(6) If the span between walls is too long, a steel pipe column and footing will also be required to reduce the steel beam span, as shown in Figure 3.8.
Figure 3.8 - Concept 1-Drag Strut
Shear Wall Design

5. Select the traditional shear wall analysis method or the perforated shear wall empirical method.

A. Traditional Shear Wall Analysis Method

Traditionally, only wall sections fully sheathed with structural panels were used as shear walls. Sections of wall containing door and window openings were excluded from consideration. Figure 3.9a illustrates numerous types of wall openings and their influence on shear wall appropriateness, based on the "traditional" method. Very small openings were ignored, if they did not interfere with the continuity of the shear wall; thus the entire wall could be used as a shear wall. Vertical, long and narrow openings in a wall split the potential shear wall into two separate but smaller shear wall segments. Large openings, such as doors, large windows, or obviously garage doors also split the wall into solid segments between these openings.

Structural panels above and below openings are not considered to contribute to the overall strength and stiffness of the wall. Solid planes of walls less than 4'-0" long are questionable for use as shear walls, since the overturning anchorage force becomes very large and economical to tie the shear wall to the foundation.

Refer to Figure 9 in Chapter 5 of the WMM for examples of relative tie-down forces dependent on shear wall proportions based on the traditional method. Note that if the shear wall becomes too narrow it will become more flexible and horizontally move and rotate much to easily. The traditional method uses tie-downs at the ends of each solid wall segment to resist overturning, as shown in Figure 3.9a.
Chapter 5 of the WMM gives a complete overview of “traditional” shear wall behavior and the necessary connections to resist sliding and overturning. The equilibrium of a shear wall and the resulting equations are of general use in the examples that will appear in Chapter 5. Figure 3.9b illustrates the forces applied to a shear wall, and develops the shear and overturning equations. It also illustrates, as did Chapter 5 in the WMM, that a square wall segment is most appropriate as a shear wall, since it cuts the overturning force in half and increases the wall stiffness by six times.
\[ V_{LB} = V_s^{plf} \cdot L^{ft} \] or \[ V_s^{plf} = \frac{V_{LB}}{L^{ft}} \]

sum moments about pivot:
\[ \Sigma M = 0 \]
\[ V \cdot H - F_{uplift} \cdot L \]
\[ V \cdot H = F_{uplift} \cdot L \]

\[ F_{uplift} = \frac{V \cdot H}{L} \]

if \( L = 4'\text{-}O \) & \( H = 8'\text{-}O \)
\[ F_{uplift} = 2V \]

sum moments about pivot:
\[ \Sigma M = 0 \]
\[ V \cdot H = F_{uplift} \cdot H \]

\[ F_{uplift} = V \]

if \( L = 8'\text{-}O \) & \( H = 8'\text{-}O \)

Panel 6 times as stiff as the 4' panel

Figure 3.9b - Equilibrium of a Shear Wall
B. Perforated Shear Wall Empirical Method

Considerable research has been dedicated to achieving more economy in shear wall design. Design optimization studies incorporating the structural panels above and below openings found that these normally ignored structural panels are a significant contributor to the overall wall shear strength and stiffness. This approach also required tie-downs only at the corners or ends of a shear wall elevation. A significant number of static and cyclic tests have verified the empirical equations of the perforated shear wall method [3.5].

Figure 3.9c illustrates the perforated empirical method, when compared to the shear wall examples in Figure 3.9a.
Note the significant reduction in anchor tie-downs, and the incorporation of the whole wall for shear wall performance. This method now appears in the *Standard Building Code* [3.6] and the *Wood Frame Construction Manual for One and Two Family Dwellings* [3.7]. The example in Chapter 5 will use both methods for comparison purposes.

The perforated shear wall design approach, based on finding the *required shear wall length*, uses the following steps:

1. Find the required length of shear wall sheathed with structural panels for the full height of the wall. This is based on the traditional method;

2. Use the empirical equations of the perforated shear wall method to find the *length adjustment factor* \( R_a \) to reduce the strength of the traditional fully sheathed shear wall (found in step “1”) and compensate for the presence of openings by increasing the shear wall length.

The perforated shear wall design approach, based on finding the *allowable shear strength* of a perforated wall length, uses the following steps:

1. Determine the allowable shear (plf) based on the structural panel thickness and nailing size and spacing from Table B.28 in Appendix B. Determine the total allowable shear load (lbs.) by multiplying the allowable shear by the length of fully sheathed segments of a given building exterior wall elevation. This is based on the traditional method;

2. Use the maximum opening height and the ratio of fully sheathed walls to the total length of wall with openings to find a *shear capacity adjustment factor*. Multiply the adjustment factor by the allowable shear found in step one and multiplied by the sum of solid structurally paneled wall segments to determine the reduced allowable shear strength (lbs.) of the perforated wall. This will account for the presence of openings and permit tie-down anchors to be placed only at the ends of the wall.

Notes:
- When designing for a given wind load, shear walls by the perforated shear wall method will be longer, but generally have a reduced number of overturning tie-downs compared to the traditional method’s solid-sheathed shear wall segments.

- Shear walls designed by the perforated shear wall method instead of the traditional method will have lower ultimate shear capacity and less stiffness.
Plywood or OSB sheathing is appropriate as the structural panels used on the exterior of the wall. Interior drywall can add to the allowable shear capacity of the wall.

The perforated shear wall method uses only two tie-down anchors located at each end of the elevation, rather than the traditional method that uses tie-downs at the ends of each solid wall segment. This method will generally prove to be more economical.

**Shear Walls Containing Garage Doors**

6. Provide more than 4 feet of solid shear wall on either side of an elevation that contains a garage door opening.

The unique case of a wall containing a garage door should be reviewed here. Two car garage doors are usually 16 feet long, and front garage wall widths are generally 20 feet long. It is clear that for the 20 foot front garage wall, only 2'-0" remains for symmetry of the elevation with respect to the door. These 2'-0" sections are extremely flexible, and will produce very large tie-down anchorage forces, if used as shear walls. Since garage foundation walls are approximately 4'-0" deep or less, to bear below the frost level of a given geographic location, there is usually insufficient foundation wall and footing weight to resist a large tie-down force.

Figure 3.10 illustrates a typical plan view of a garage that is attached to a residence.

**A. Traditional Approach**

Consider first that the entire garage projects in front of the house, as indicated by the black walls extending past the rear of the garage. For a wind direction "A" it would be advisable to use wall 2 as one shear wall and purposely increase the width of the garage to approximately 26 feet, rather than the typical 20 feet. This permits offsetting the garage door to the right as shown, so that a large segment of wall 4 is created that can work as a stiff shear wall. This creates a non-symmetrical front elevation of the garage. The additional space in the garage can be used for storage or a work shop area. This is an economical approach to creating a structural paneled shear wall at location 4. This approach is valid even if the house exterior wall is partially set back from the front of the garage as shown by position 4B. When the house front elevation is in alignment with the garage wall, the house wall 4A can become the shear wall. This direct alignment allows a return to a 20-foot wide garage elevation, although the additional garage storage space is then eliminated.
Sometimes it is easier to achieve lateral stability at the front elevation of a garage by means of a steel frame around the garage door. This approach would avoid using the 2'-0" wall segments on each side of the door as shear walls, and instead uses the walls to hide the columns of the steel frame. Figure 3.11 illustrates this concept for a situation when the house is set back from the front of the garage and there is no need for additional garage space. Either wide flange shapes or rectangular tubes are used for the steel frame. The steel beam acts as the garage door header beam. If the beam supports brick above the opening then a tube should be used to avoid twisting of the beam. The column base detail is shown for a tubular solution that fits within the 3-1/2" stud dimension. It is common for a structural engineer to assist an architect when steel frames are required.

B. Perforated Wall Method

Even if the empirical method is used, the solid structural paneling either side of the garage door should be at east 4'-0", when the garage wall is not in
alignment with the house wall. This wall dimension is necessary to achieve any acceptable movement control in the plane of the wall for wind blowing in direction A. If the minimum 4'-0" paneled wall is achieved on each side, then its possible that the entire wall plane can be used, as shown in Figure 3.9c. Acceptable sway control might then be achieved.

If the small door opening has structural paneling above shear wall 2, then the entire wall may be treated as a perforated shear wall. Anchored at each end of the elevation.

Wall 1 is likely to still perform as a shear wall despite the two window openings by the perforated wall method, when wind blows in the direction B. Structural paneling would be needed above and below the openings.

Figure 3.11 - Steel Frame Hidden Within Garage Wall
Placement of Doors and Windows in Shear Walls

7. Place doors and windows to maximize potential shear wall length.

Regardless of shear wall design method selected in item #4, the shear walls must have sufficient stiffness; that is, they control the amount of sideways movement of the residence subjected to horizontal wind load forces. The longer the shear walls the stiffer the shear walls, and the smaller will be their horizontal movement. Also, longer shear walls are less likely to overturn. Thus, the required tie-down force will be smaller and the anchorage detail will be more economical. It may not always be possible to shift the placement of windows and doors. However, if it is possible without disrupting the design of the room behind the wall, it can greatly influence the shear wall structural stiffness and the required tie-down force magnitude. This concept was presented in Chapter 5 and illustrated in Figure 9 of the WMM. Figure 3.12 continues the same relative stiffness and relative tie-down force proportions as found in the WMM, but with an emphasis on the position of window and door openings. Figure 3.12 specifically relates to the traditional shear wall method.
Wall “A” in Figure 3.12 is an elevation with a door and window placement that has been designed for one elevation of a residence. Notice that the 24-foot long wall ends up with two openings that break the wall into 3 short segments that could be used as shear walls. If it were possible to rearrange the two openings off to one side, as shown in elevation “B”, a significant increase in stiffness would occur. This would reduce the movement of the residence subjected to a horizontal wind reaction by 100%. This requires a design trade-off or a compromise in the visual expression to be made for the sake of creating a stiffer and more wind resistant home without increasing the cost. A similar example is the elevation of wall “C”, where three large windows are adjacent to one another, resulting in two potential shear wall segments. Again, if it is feasible to move the trio of windows to one end of the wall, as shown in Elevation “D”, the stiffness again doubles. Thus, the sideways movement under wind load would be cut in half. Tie-downs would be used at the ends of each full height structurally paneled segment. This improves the shear wall stiffness overall.

Use of the Perforated Shear Wall Method may allow elevations “A” and “C” not to have to change, but they will be more flexible. Tie-downs will be located at the ends of the elevation only.

Avoid Wall Offsets

8. Avoid short offsets over the length of exterior walls.

Offsets, as shown in the partial foundation plan in Figure 3.13a, may be considered an architectural feature for the interest of the superstructure elevations above. The elevations will add shadow lines and visual interest, plus it will avoid making a wall seem overly long.
These minor offsets, usually in the 1 to 3 foot range, do little to increase the size of an interior room and result in a foundation that is more expensive to install. Most contractors will agree that every exterior corner or re-entrant corner is an added cost. This information leads to the following conclusion: offsets of one to three feet mean very little to interior space, but do complicate and make the foundation more costly. Offsets should be at least four feet or more to influence interior space and make the added cost of the foundation worth considering, as shown in Figure 3.13c.

Structurally, short or long offsets will break a shear wall into two separate segments and reduce its overall stiffness as previously shown for wall openings in Figures 3.9a and 3.12. Since short offsets add cost for very little gain, avoid them to maximize wall length and thus maximize strength and stiffness as a potential shear wall as shown in Figure 3.13b. Architecturally the straight wall has less visual interest but structurally produces the best potential shear walls for stiffness and overturning resistance. Somewhere there must be a balance struck between design and structures, particularly as it will influence cost. Drag struts, although adding cost, may be used to force short offset walls to transfer shear as though both wall segments were in the same plane. This technique is less likely to be structurally effective for large offsets.

Unusual situations can occur in custom house designs where cathedral ceilings and two story spaces are part of the design. One such complicated design is shown in Figure 3.14. It involves a two-story residence with a cathedral ceiling for a two-story space, adjacent to two floors with a flat ceiling and roof trusses. The length of the house and the magnitude of the wind forces requires using an interior shear wall along with two exterior walls as shear walls. The interior shear wall is on one side of an open sided stair, that separates the cathedral space from the two-story spaces. Only the crosshatched wood stud wall runs from the roof down to the basement floor. The length of the shear wall is insufficient to transfer the build-up of wind shear at the basement level.

The solution is to engage a drag strut from the top of the basement wall to the garage foundation wall, which is in perfect alignment. Thus, the garage foundation wall is made to share the wind shear with the stair wall. The exterior foundation wall of the basement is offset by 16 inches, and would require a diagonal drag strut and significantly more engineering. Although it is easier to introduce a second drag strut and balance the shear, the offset makes this impractical. This is a complicated solution, which requires an engineering review.
Figure 3.14 - Partial Foundation Plan

Figure 3.15 illustrates another type of offset situation. A recessed side entry to a residence also breaks the potential exterior shear wall length into two pieces, even if a straight foundation wall exists below. Add to this the introduction of a masonry fireplace bounded by large windows on both sides. Now the rear exterior portion
of wall is unlikely to be of any shear wall value. This leaves the front portion of wall as the only exterior shear wall of use.

It is clear from this discussion that not all walls can be shear walls, and that such issues as openings or recesses, or even offsets will lessen the shear wall length and therefore its strength and stiffness.

**Figure 3.15 - Partial Plans with Offsets**

**Shear Walls that are Bearing Walls**

9. Exterior walls that are bearing walls are the best choice for shear walls.

The added dead load of floors and/or roofs framing into exterior walls improves their resistance against overturning and reduces or eliminates the anchorage requirements at the ends of the shear wall. A reduced anchor size and a reduction in the number of bolts lessen the cost of the shear wall. Figure 3.16 illustrates a portion of an exterior bearing wall of a two-story residence. The hip roof is framed with rafters that bear on the exterior shear wall. The first and second floor framing
joists bear on the exterior shear wall. Only a central portion of the total wall length can be utilized as a shear wall due to openings that are not shown. Drag strut members will be used to transfer the wind reaction to the shear wall. The entire dead load adds resistance to the overturning tendency of the shear wall.

Figure 3.16 - Bearing Wall Used as Shear Wall

Figure 3.17 illustrates a non-bearing wall used as a shear wall. Only a small portion of the roof dead load from the short rafters is added to the weight of the wall itself. The floor joists run parallel to the shear wall and do not add any dead load to the walls. Non-bearing walls will most likely require a greater anchorage at the ends of the shear wall. This requires more uplift resistance from the foundation concrete, longer anchorage rods and more bolts into the ends of the shear wall - ultimately more cost.
It is logical that the floor and roof framing should span parallel to the long direction of the residence, since the long direction receives a greater amount of wind and places larger wind reactions on the short direction shear walls, these shear walls will be loadbearing. However, it is most common that floor and roof framing spans perpendicular to the long direction of the residence, which makes the long exterior walls bearing walls.

Figure 3.17 - Non-bearing Wall as a Shear Wall
End Wall Design
10. End wall Design for Gable and Hip Roofs.

Figure 3.18a compares the end wall behavior for a single story gable roof (B) building and a single story hip roof (A) building.

Gable Roof

End stud walls of a gable roof, that create an interior cathedral ceiling, will be very flexible in platform framing. The double top plate location is where the weakness occurs. The failure mechanism at the double top plate when subjected to a wind load is shown in location “B” of Figure 3.18a.

Figure 3 in Chapter 6 of the WMM illustrates a potential solution utilizing 4x4 posts to stiffen the wall. This solution resembles balloon framing, as the 4x4’s extend continuous from the floor to the roof plane. Another solution is to introduce a 4-foot offset in plan near the highest portion of the gable. This will significantly stiffen the wall. Cathedral spaces are generally used for family rooms or living rooms. A design consideration is to introduce a fireplace in either of these types of spaces. Should either of these rooms contain an end wall, place the fireplace within the end wall. This technique will also stiffen the wall.

Figure 21 in Chapter 7 of the WMM illustrates a detail to stiffen the gable end wall at the double top plate when flat bottom roof trusses are used and a horizontal ceiling plane exists.

Hip Roof

A hip roof, utilizing rafters to create an interior cathedral space, presents a structural improvement to the end wall problem since the jack rafters provide a restraint at the wall’s double top plate. The stud wall will bend when subjected to a wind load, rather than rotate without bending at the double top plate mechanism. This is illustrated in “A” of Figure 3.18a.

A hip roof is the structurally preferred roof form for wind resistance. This is true since it avoids any additional bracing to stabilize the end walls. Gable frame end walls can be made to perform adequately if bracing, such as that shown in Figure 3.18b. The sheathed hip roof form also acts as a 3-dimensional shell and creates a stiff roof shape for wind resistance.
Figure 3.18a - Stability-Wind on Gable and Hip Roof End Walls
Two story spaces, that include an exterior end wall, present much the same problem of stability as shown in Figure 3.18a for gable roofs. Figure 3.18b illustrates this situation. Solutions are similar to those described above: offsets in plan, 4x4 posts, or if design warrants a fireplace.

Figure 3.18b - Stability - Wind on Two-Story Space End Wall
Stability Applications

It is valuable to apply the structural planning concepts developed in this chapter to several example problems to gain proficiency in the use of the concepts. This will help to solidify the approach to selection of diaphragm proportions, shear wall locations, tie-down locations and the need for drag struts.

Structural Planning Applications

Example 1: Figure 3.19 shows a first floor plan, roof plan and foundation plan for a basic one-story rectangular house, given the following:

Roof: 3 in 12 slope hip roof form, composed of trusses spaced 24 inches apart. The hips are a combination of flat top trusses and jack rafters.

Garage: Detached.

Foundation Type: Cast-in-place Basement; 8 inch thick walls; a central steel girder spans the length of the basement and is supported by three steel pipe columns spaced 20 feet apart.

Wood Stud Walls: Dimension lumber 2 x 4's at 16 inches on center.

Structural Planning Solution:

A. Wind Direction Perpendicular to the Long dimension of the Building

1. Begin by assessing the proportions of the overall rectangle. Do they exceed L/W of 4? In this case 80/32 = 2.5 and does not exceed 4. Thus the proportions of the diaphragm are within acceptable code prescribed limits. Knowing this, it is possible to proceed with the perimeter walls of the residence to achieve total lateral stability, without the need to employ interior walls. Drag struts may still be required within the shear walls, dependent on window and door locations.

2. Use imaginary planes as shown in Figure 3.20 to help visualize the important structural planes for wind resistance perpendicular to the long dimension of the building. Pass the planes through the exterior stud wall and foundation wall, the first floor and the sloping roof planes. This helps to define the roof diaphragms D and E, the floor diaphragm C and the exterior walls that lie in the planes A and B. The wind reactions are the dark arrows located at the intersection of the roof, floor and exterior wall planes.
Figure 3.19 - Plans for Example #1
3. The exterior walls, whole or in part, will be used as the shear walls 80 feet apart. The floor and roof diaphragms will act as deep horizontal or inclined beams respectively, and transfer wind load they receive to their supports (called wind reactions), which in this case are the short shear walls A and B. They are desirable shear walls, since they have foundation walls below. They can directly transfer shear force to the foundation walls to resist sliding and overturning. The foundation wall is also important because its concrete mass is excellent mass for resisting overturning.

4. Review the shear wall plane A shown in Figure 3.21a by the traditional shear wall method. Windows exist at each end of the elevation. Thus, a rather substantial solid wall segment (20 feet) is available in-between for a solid shear wall.
application. **Always use the maximum length of shear wall available** since it will reduce the overturning force and also allow the spacing of the anchor bolts to be further apart for the sliding force. Note again that a foundation wall exists below shear wall A. The shear wall being located in the center of the elevation does not receive the wind reaction directly. A drag strut is required to transfer the wind reaction to the shear wall. It is common to use the headers over window and door openings as the drag strut beam. Figure 3.21a illustrates the shear transfer between roof diaphragm and the shear wall when the wind reaction comes from the right side. The sliding and overturning anchorage forces are as shown, and the tension tie-down is at the right side of the shear wall. If the wind reaction comes from the left side all the arrows shown will reverse direction. Overturning anchorage is now required on the left side of the shear wall. This is explained to make the point that **wind can come from either direction and the overturning anchorage tie-down must be provided at both ends of the shear wall.**

![Figure 3.21a - Transverse Shear Plane A](image)

4A. Also, review the shear wall plane A shown in Figure 3.21b by the perforated shear wall empirical method. Assume structural sheathing exists above and below the windows at each end of the elevation. Now, the entire perforated wall (32 feet) is available as a shear wall. This method exactly conforms to the general rule to **always use the maximum length of shear wall available.** The perforated shear wall method produces the smallest overturning force and also allows the spacing of the anchor bolts to be further apart for the sliding force. Note again that a foundation wall exists below shear wall A. The shear wall being located across the entire elevation does receive the wind reaction directly. No drag strut is required to transfer the wind reaction to the shear wall.
5. Review shear wall plane B shown in Figure 3.22a by the traditional shear wall method. Two separate windows exist, leaving two 8-foot solid wall segments available as shear walls. Whether this is structurally adequate will depend on the wind load magnitudes, but most likely for a one-story residence the two-combined shear wall segments should be structurally sufficient for handling the overturning and sliding. Two drag struts will be required to transfer the wind reaction from the left to the two shear wall segments. It is assumed that both shear wall segments will be required to handle the wind reaction. When the wind reaction comes from the right the shear and anchorage arrows will reverse direction as described in step #4. This time only the drag struts left and right of the left shear wall segment are required to transfer the wind reaction between the two shear walls, since the right shear wall starts at the end of the wall. Note again the positive influence of a foundation wall existing below shear plane B.
Option: It could be possible to move either or both of the windows to create one large shear wall. The maximum shear wall length might become 16 feet. This is a design consideration that may be unworkable for the outside views from within the rooms, or it may be unsatisfactory for the aesthetics of the elevation.

5A. The shear plane B should also be looked at for economy using the perforated shear wall empirical method. Given that the left window is a corner window, and it is desired by the architect to omit all but the jamb, even with sheathing above and below the corner window there are no studs at the corner. Thus, the shear wall will run 24 feet, including the right window that must be structurally sheathed above and below. This approach eliminates two tie-down anchors. Figure 3.22b illustrates this solution to elevation plane B's shear wall design. A drag strut will be required over the left window for wind coming from the left.

![Diagram of shear wall design](image)

**Figure 3.22b - Transverse Shear Plane B Perforated Shear Wall Empirical Method**

6. The above 5 steps have been done without numbers to avoid complicating the structural planning process. Chapter 5 will review an example with numbers and use the selection Tables of Appendix B.
B. Wind Applied Parallel to the Long Dimension of the Building.

1. Begin by assessing the proportions of the overall rectangle. Do they exceed L/W of 4? In this case 32/80 = 0.4 and does not exceed 4. Thus the proportions of the diaphragm are well within acceptable code prescribed limits. Knowing this, it is possible to begin the process with the perimeter of the residence to achieve total lateral stability. Drag struts may still be required to transfer the wind reaction to the shear wall segments, depending on the location of door and window openings.

2. Use imaginary planes as shown in Figure 3.23 to help visualize the important structural planes for wind resistance parallel to the long dimension of the building. Pass the planes through the long exterior stud and foundation walls, the first floor and the sloping roof planes. This helps to define the roof diaphragms D and E, the floor diaphragm C and the shear wall planes A and B. The wind reactions are the dark arrows located at the intersection of the roof, floor and exterior wall planes.

3. The exterior walls, whole or in part, will be used as the shear wall planes 32 feet
apart. The floor and roof diaphragms will act as horizontal beams and transfer wind load they receive to their supports, which in this case are the long shear walls. They are desirable shear walls, since they have foundation walls below them, and can directly transfer shear force to the foundation walls to resist sliding. The foundation wall is also important because its concrete mass can be used for resisting overturning. Note that smaller wind reactions will occur in this direction since the surface area of the end walls is much smaller than the long wall surface areas.

4. Review shear wall plane A shown in Figure 3.24a using the traditional shear wall method. Several windows plus an entry door exist along the elevation. A rather substantial length (18 feet) is available for two major shear walls, if needed. These shear walls are located at the ends of the elevation, which is good to avoid twist of the building under a diagonal wind load. No drag struts will be required to transfer wind reactions to the shear walls since both shear walls start at the ends of the elevation. When the wind reaction comes from the left, the left shear wall is structurally activated. It is highly unlikely that both shear walls are required to handle the wind reaction from the left. When the wind reaction comes from the right the dotted arrows are activated at the right shear wall, without involving the left shear wall. Thus only one shear wall functions for a wind reaction in one particular direction. Should both shear walls be required, a long drag strut would be required to link the two shear walls. Note again that a foundation wall exists below the potential shear walls. There is no need to use the shorter wall segments as shear walls. The longer shear walls are more structurally efficient.

Figure 3.24a - Longitudinal Shear Plane A
4A. Review shear wall plane A shown in Figure 3.24b using the perforated shear wall method. Several windows plus an entry door exist along the elevation. Given the length of the wall, and the fact that it is only a one-story residence, it is unlikely that the whole wall will be required as a shear wall. However, if the entire wall is structurally sheathed, including above and below openings, then the entire wall can be a shear wall. Most likely there will be no uplift due to overturning and tie-downs won’t even be required.

Entire wall must be structural sheathed

Wind Direction
Left

Wind Direction
Right

Foundation wall below shear walls

Figure 3.24b - Longitudinal Shear Plane A Perforated Shear Wall Empirical Method

5. Review shear wall plane B shown in Figure 3.25a. Five window locations exist, leaving two 8-foot solid wall segments available for shear walls at the ends of the elevation, or the 16-foot long solid wall segment near the left side of the elevation available as a single shear wall. Whether the two shorter end wall segments should be used, or the longer more central wall segment should be used as the potential shear wall(s) will depend on the wind load magnitudes. Most likely for a one-story residence the two end segments will be structurally sufficient and most economical to handle the overturning and sliding.

Since these two shear walls exist at the corners of the elevation, the shear wall on the right side will directly receive the wind reaction (solid arrows in Figure 3.25a) that is coming from the right. This right shear wall will most likely be structurally adequate to handle all the wind from the right direction. Therefore, the opposite end shear wall is not needed for sliding or overturning resistance. If this is the case, no drag strut will be necessary to transfer half the wind reaction to the opposite shear wall. However, when the wind reaction comes from the left (the dotted arrows in Figure 3.25a) the left shear wall directly receives the wind and the right end shear wall is not required. Again, no drag strut is required to force sharing of the total wind reaction.
Should one decide to use the 16-foot long wall segment as the shear wall (dotted section in Figure 3.25a) all by itself, drag struts would be required to transfer the wind reaction from the right or from the left. Tie-down anchors will be required at the ends of each shear wall segment, regardless of option selected. Note again that a foundation wall exists below all of these shear walls.

It is unlikely, but if all three shear walls would be required to equally share the wind reaction from either direction, drag struts between the shear walls would be needed to force the transfer of the wind reaction between the three shear walls.

Figure 3.25a - Longitudinal Shear Plane B

5A. Review shear wall plane B shown in Figure 3.25b by the perforated shear wall empirical method. Structurally sheath above and below all five openings. The entire length of the elevation is now available as a shear wall. The main advantages include 1) no need for drag struts; 2) a reduction of tie-down anchors from 4 to 2 or most likely none. Since the entire wall is now structurally sheathed the tension uplift chain is also accounted for, and thus serves two purposes.
6. The above 5 steps have been done without numbers to avoid complicating the structural planning process. Chapter 5 will review this example with numbers and use the selection tables of Appendix A.

Example 2: This example is very complicated compared to the first example. It will test your stability understanding to the fullest. Figures 3.26 through 3.28 show a floor plan, foundation plan and roof plan for a one-story house with a set of intersecting gable roofs. The following information is given:

Roof: 6 in 12 slope hip roof form, composed of trusses spaced 24 inches apart. A cathedral ceiling exists with rafters at 16 inches on center, where shown on Figure 3.28. The roof intersections require jack rafters sitting on the trusses below. The structural roof-framing plan is shown in Figure 3.29, and the four elevations are shown in Figure 3.30. North is up on the plans.

Garage: Attached with a slab-on-grade.

Foundation Type: Cast-in-place Basement, 8 inch thick walls and a partial basement where shown on Figure 3.27. Two steel girder lines span the length of the basement and are supported by three steel pipe columns as shown in Figure 3.27. A fourth column is used to support the roof ridge beam that provides support for the vaulted ceiling framing.

Wood Stud Walls: Dimension lumber 2 x 4's at 16 inches on center.
Figure 3.26 - First Floor Plan
Figure 3.27 - Foundation Plan
Figure 3.28 - Roof Plan
Figure 3.29 - Roof Framing Plan
Structural Planning Solution:

A. Wind Direction Perpendicular to the Long dimension of the Building

1. Assessing the proportions of the overall rectangle is difficult since there is more than one rectangle and they are offset from each other. When more than one gable roof exists and they are at different elevations, each roof form will become a separate diaphragm. This example has three gable roof forms and thus three diaphragms will become activated. Do any of the three roof spans perpendicular to the direction of the wind exceed L/W of 4? In this case 33/27 = 1.22 and does not exceed 4. The two other smaller diaphragms will certainly be less than that so the proportions of the three diaphragms are within the acceptable code prescribed limits. Knowing this, it is possible to proceed with the perimeter walls that border each diaphragm and unfortunately any interior walls as well to achieve total lateral wind stability. Drag struts will most likely be required given the plan, roof and foundation layout.

2. Use imaginary planes as shown in Figure 3.31 and 3.32 below to help visualize the important structural planes for wind resistance perpendicular to the long dimension of the building for the main roof diaphragm #1. Pass the planes through the exterior stud wall and foundation wall, the first floor and the sloping roof planes.
The vertical planes help to define the gable ends of the roof diaphragm #1, the floor diaphragm and the elements within the exterior/interior walls that lie in the planes A and B. Refer back to the first floor plan and basement plan to see the location of the A and B planes in plan view.

Figure 3.31 - Shear Plane A
The vertical plane in Figures 3.31 and 3.32 cut through the wall sections shown in Figures 3.33 and 3.34 respectively. They show in detail what can be used for the stability system related to roof diaphragm #1. The wall section at plane A in Figure 3.33 reveals the need for extensive and irregular shaped drag strut members. It also shows that only a 6-foot long wood shear wall is available to handle the overturning and sliding from the right or left wind reaction. The anchorage tie-down force will be large, but there is a substantial foundation wall directly below to provide sufficient dead load resistance. The drag strut beams would probably be formed from doubling the rafters and use of the ridge beam, plus adequate connections to the wood shear wall. The traditional shear wall design method will be employed here. There is no benefit from the perforated shear wall empirical
method, since there is no sheathed segment to the left of the door opening. The stiffness of the shear wall would also not improve.

To improve the structural stability conditions one might consider moving the sliding doors to the north elevation and extending the patio deck. This would create a 12-foot shear wall, reduce the anchorage tie-down force and create a stiffer shear wall to reduce movement during a wind loading. There would be the elimination of the drag strut header over the door opening, but the most complicated drag struts would still remain. The architect for this example would not move the opening and it was built as shown.

Figure 3.33 - Building Section at Plane A

Figure 3.34 is the building section cut by the vertical plane B. This section cuts through the garage necessitating a 20-foot long drag strut. This drag strut would be a combination of the bottom chord of the roof truss and the creation of a beam perpendicular to the garage roof trusses. **Diagonal braces spaced at 4 feet apart are an important framing addition to transfer wind along the garage front elevation to the roof diaphragm over the garage.** Consider how flexible the front elevation will be given a 7' x 16' opening cut into the wall. The braces will help to keep the front elevation from imploding if subjected to a left wind direction. Even
with a high wind resistant garage door, the wall around it needs to be strong and stiff, and anchored to the foundation.

The exterior wall behind the garage is quite a substantial wood shear wall and has a foundation wall below it for sliding and overturning force transfer. The left wind reaction transfers along the drag strut to the wood shear wall, which in turn is anchored to the foundation wall below. The final resistance to the wind reaction is the friction along the bottom of the concrete footing and the passive soil pressure block on the leeward side of the foundation wall. Again, the traditional shear wall method is the logical choice, since the only part of the elevation that is useful as a shear wall is the exterior wall as shown.

![Diagram](image)

**Figure 3.34 - Building Section at Plane B**

4. Figure 3.35 illustrates the second set of vertical planes C and D cut at the extremities of the **roof diaphragm #2**. Note that this diaphragm is perpendicular to the main roof diaphragm #1. The wind direction is the same as for Figure 3.32.
5. The vertical planes C and D in Figure 3.35 cut through the wall sections shown in Figures 3.36a and 3.37 respectively. They show in detail what can be used for the stability system related to roof diaphragm #2.
Review the exterior wall cut by plane C shown in Figure 3.36a for potential shear walls using the traditional shear wall design method.

The windows and the fireplace break the wall into distinct segments. Only a 4-foot section either side of the fireplace is available to act as wood shear walls. The fireplace unit is a “zero clearance” type, and thus can be boxed in wood framing. It was stated earlier in this chapter that 4 feet should be considered the shortest shear wall to resist the stability issues of overturning and sliding. It is fortunate that two such walls exist to share the right or left wind reaction. A drag strut header over the windows will be required to transfer the left wind reaction to the left shear wall segment. It is assumed that both shear wall segments will be required to handle the wind reaction and thus the fireplace wood framing will have to be detailed to act as a drag strut. When the wind reaction comes from the right, the shear and anchorage arrows will be as shown in Figure 3.36a. This case sends the right wind reaction directly into the right shear wall. The fireplace construction, acting as the drag strut again will activate the left shear wall segment. Note again that a foundation wall exists below both shear walls. The final resistance to the wind
reaction is the friction along the bottom of the concrete footing and the passive soil pressure block on the leeward side of the foundation wall.

Review the exterior wall cut by plane C shown in Figure 3.36b for potential shear walls using the perforated shear wall empirical design method.

![Figure 3.36b - Building Exterior Wall at Plane C](image)

There is an opportunity to take advantage of the perforated wall method. Two windows exist near the left corner, but are separated by a small wall dimension. Structural sheathing above and below the windows will permit the length of wall between the fireplace and the left corner as one shear wall as shown in Figure 3.36b. Two tie-down anchors will likely be required at the ends of the wall. The right shear wall segment will still be treated by the traditional method as described above.

Option: It could be possible to move the right window to create a longer shear wall segment on the left side if stiffness is an issue. The window could be placed adjacent to the other dining room window on the north elevation. This is a
design consideration that may be unworkable for the outside views from within the rooms, or it may be unsatisfactory for the aesthetics of the elevation.

Review the vertical section wall cut by plane \( D \) shown in Figure 3.37 for shear wall potential by the *traditional shear wall design method*.

There are two interior wood stud walls that could be structurally sheathed on one side to create shear walls within the vertical plane \( D \). Both walls may not be needed as shear walls depending on the magnitude of the right or left wind reaction. It is permitted by most building codes to utilize drywall as shear resistant for shears less than 150 plf, but this author prefers to let drywall sheathing be a reserve capacity added to the safety factor, yet not relied on exclusively for the shear resistance of the house. It is common to add a layer of drywall over the structural wood sheathing for painting and finishing purposes. The major problem in this vertical wall section is that no foundation wall exists below either potential wood shear wall. Three possible structural design approaches can be considered for such a condition.

![Figure 3.37 - Building Section at Plane D](image)

a. The first approach is somewhat intuitive, but not necessarily conservative. It assumes that roof diaphragm #1 contains the vertical
plane D and will interactively pick up much of the wind reaction from plane D and send it into the diaphragm to its ends. The shear walls in vertical planes A and B will therefore receive a larger wind reaction. This means there is no need for either of the potential shear walls in plane D.

b. The second approach is somewhat conservative, but will add stiffness to the house at the vertical plane D. It ignores the roof diaphragm #1 and transfers the wind reaction to one of the potential wood shear walls. It then doubles the first floor joists under the shear wall, using it as a drag strut to transfer the overturning and sliding to a wood stud wall sheathed with structural panels in the basement. It is an architectural design decision whether to place this new wall under the left or right shear wall. Figure 3.37 shows both options. Either new shear wall should be anchored along the vertical edge that comes in contact with the foundation wall so as to activate its dead weight. The opposite end of the shear wall may require a footing below the slab to provide enough overturning uplift resistance.

c. The third approach inserts a steel beam under the shear wall(s) to act as a drag strut, but also as a bending member that receives the overturning forces by means of tie-down anchors to the steel. This eliminates the need for shear walls in the basement, but means that the ends of the steel beam must be anchored to the concrete walls to resist uplift.

Review the vertical section wall cut by plane D shown in Figure 3.37 for shear wall potential by the perforated shear wall empirical method.

There is no possibility to use the perforated shear wall empirical method. The segments have openings that traverse the full height of the wall plane.

6. Figure 3.38 illustrates the vertical planes E and F that define the extremities of roof diaphragm #3. Note that vertical plane E cuts through the roof diaphragm #1 much like vertical plane D above. It must be also noted that this time the garage and its two-door arrangement will influence our decisions.
Figure 3.38 - Shear Planes E & F

7. The vertical planes E and F in Figure 3.38 cut through the wall section and exterior wall shown in Figures 3.39a, 3.39b and 3.40 respectively. They show in detail what can be used for the stability system related to roof diaphragm #3.

a. Review the building section cut by vertical plane E illustrated in Figure 3.39a by the *traditional shear wall design method*.

Three potential interior stud walls can be used as shear walls. A similar discussion to the one for the building section cut by vertical plane D occurs similarly here. Thus, the discussion will be abbreviated.

The roof diaphragm #1 could be considered to interact with the interior potential shear walls under the main roof trusses. Thus, there is no need for any shear walls in this location. However, the exterior garage wall and the
piece of the interior wall that completes the garage wall in Figure 3.39a require special consideration. Chapter 1, Figures 7 and 8 of the WMM, illustrate when wind pressure is directed into the door opening. For that case, all the sides explode outward. The illustration also shows when the wind blows from the side opposite the garage door, all four sides implode. These two conditions produce wind pressures or suctions that are at least 50 to 100% greater than when the door is closed, as stated near the beginning of this chapter. It is recommended that all the garage walls be anchored with tie-downs, have shear walls on all four walls and provide for the increased uplift.

Figure 3.39a - Building Section at Plane E

Figure 3.39b shows the best choice of shear walls for the vertical section cut through plane E. It only utilizes the 20 foot long garage wall, which has a combination basement and slab-on-grade foundation condition underneath the garage wall length.

b. Review the vertical section wall cut by plane E shown in Figures 3.38 and 3.39b for shear wall potential by the perforated shear wall empirical method.
There is no possibility to use the perforated shear wall empirical method. The segments have openings that traverse the full height of the wall plane.

Review the exterior garage wall that lies in the vertical plane F shown in Figure 3.40 by the traditional shear wall design method.

This wall is the garage wall only. It has a foundation below that must provide the ballast against direct uplift and overturning, while friction between the bottom of the footing and the soil resists the sliding with an assist from the passive soil block on the leeward side of the garage. The wood stud wall is sheathed with structural sheathing to create the shear wall. Anchorage tie-downs at the ends of the wall handle the overturning. The anchor bolts spaced uniformly between bottom plate of the wall and the foundation resist the sliding.

Review the vertical section wall cut by plane F shown in Figure 3.40 for shear wall potential by the perforated shear wall empirical method.

There is nothing to be gained by the use of the perforated shear wall empirical method. The same design results.
B. Wind Direction Parallel to the Main Roof's Ridge

1. The approach to wind resistance for the stability issues of overturning and sliding will not be done in its entirety as was done for the wind direction perpendicular to the main roof ridge. The same explanations would be presented and much repetition would result. Only the issues related to the garage will be presented.

2. Figure 3.41 illustrates the vertical planes G and H that define the extremities of roof diaphragm #3 for the wind direction shown. Note that vertical plane G cuts through the roof diaphragm #1 much like vertical plane D above. It must be also noted that plane H includes the garage front elevation and its two-car door opening. Figures 3.42 and 3.43 illustrate the location of the two vertical planes in plan view for the first floor and the basement respectively.
Figure 3.41 - Shear Planes G & H
Figure 3.42 - Partial First Floor Plan
Figure 3.43 - Partial Foundation Plan
3a. Review the wall conditions that lie in the vertical plane G in Figure 3.44 by the traditional shear wall design method.

There is a substantial length of stud wall to act as a shear wall if the wind reaction can be transferred to it from the perpendicular roof trusses. The stud wall must be sheathed with structural panels. Dimension lumber blocking will be required between enough trusses to provide the requisite shear transfer to the top of the shear wall. This will be called a drag strut, and it must transfer the wind shear via metal connections to the shear wall. The shear wall must resist overturning and sliding to provide stability for this end of the garage. Tie-down anchorage is required at the ends of the wood shear wall. This anchorage is resisted by the concrete foundation directly below the stud wall. Sliding is resisted partly by the garage slab’s friction against the soil and partly by the bottom of the foundation walls in friction against the soil.

The garage walls that are perpendicular to the direction of the wind are the windward and leeward walls that receive suction or pressure and must bend to resist the magnitude of that wind. When wind blows into the garage front elevation, even with the garage door closed and secure, these same walls are considered sidewalls that will again bend in response to the suction they receive. All garage walls must be able to resist the pressures that result from wind in either wind direction, and for double that wind magnitude when the garage door is open.

---

Figure 3.44 - Building Section at Plane G
3b. Review the vertical section wall cut by plane F shown in Figure 3.45 for shear wall potential by the *perforated shear wall empirical method.*

![Diagram](image)

**Figure 3.45 - Building Section at Plane G**

4. Review the wall conditions that lie in the vertical plane H. The wood stud wall, within the vertical plane, contains the double garage door. This is a very flexible wall given that only two feet either side of the door is available as a potential shear wall. This is not an acceptable condition for stiffness, strength or overturning anchorage as illustrated in Figure 3.9 in this chapter. It is not likely that the garage can increase in size to apply the idea shown in Figure 3.10, nor will it be possible to modify the design of the house to provide additional length as shown in Figure 3.10, items 4A and 4B. The only acceptable solution is the most expensive solution: a steel frame hidden around the door opening, as depicted in Figure 3.11. A structural engineer will be required to design and detail that steel frame.

5. The above steps have been done without numbers to avoid complicating the structural planning process. Chapter 5 will review another example with numbers and use the selection Tables of Appendix B.
Conclusions

This chapter has elaborated on the structural planning aspects of accommodating the essential aspects of stability for the residence; overturning and sliding. It has offered ideas for the architect or developer to consider in designing homes to make them more cost effective for wind resistance. Two examples, one simple and one complex, applied the principles described herein to illustrate and clarify their application.

Emphasis was placed on the garage’s ability to provide adequate wind resistance, as this is a very vulnerable part of a residence in high wind conditions. The ability of the garage to withstand the pressures and suctions applied to it is instrumental in preventing the wind from attacking other adjacent parts of the house.

The two shear wall design methodologies were presented and discussed for both examples in this Chapter.

Post Script

Since this Manual was completed, new ideas have emerged with regard to stability for residential construction. The primary items include:

1. Prefabricated shear walls.

These engineered components will potentially reduce cost, assist compliance with the wind provisions of the model building codes, and reduce field labor to speed up construction. Five different companies have created different approaches to wind stability. The system names include CeeWal, Hardy Frame, Shear Max Panel, strong-Wall, and Z-Wall. Space does not permit including illustrations of these systems; however, their web sites are www.ceewal.com, www.hardyframe.com, www.shearmax.com, www.strongtie.com, and www.hawaii50.com/zwall respectively.

2. Drag-struts and tie-down anchors.

These engineered products are made from steel, are easy to install and avoid eccentricity between the tie-down and the double stud, where bending can occur. The manufacturer is Zone Four, website: www.zonefour.com.

3. Post-tensioned concrete block foundation walls

A technique intended to vertically anchor by post-tensioning, the concrete block foundation units without the need of grouting. Vertical steel tie rods run from the footing to the top of the block wall. Manufacturer is Dur-O-Wal at website: www.dur-o-wal.com.
Selected References


3.2 ASCE 7-93, p.17.


