Designing Wood Frame Structures For High Winds

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SEAMASS Meeting 10-26-16
Overview

• Wind Loads and Code Changes
• Uplift
• Wall Design
• Diaphragms
• Shearwalls
Wind Loads

Wind loads acting on buildings are modeled as uniform surface loads. Wind loads can create both positive and negative loads (inwards and outwards loads) on building surfaces and create three different loading conditions:

- Uplift
- Racking/overturning
- Sliding/shear
Massachusetts Building Code

Massachusetts 8th Edition Building Code

The Official Website of the Executive Office of Public Safety and Security

Public Safety


Home  >  Consumer Protection & Business Licensing  >  License Type by Business Area  >  Construction Supervisor License  >  8th Edition Base Code

8th Edition Base Code

The 8th edition of the Base Code is comprised of the International Building Code 2009 (IBC), several companion I-codes and a separate package with Massachusetts amendments to the I-codes. The IBC and its companion codes are sold by the International Code Council www.iccsafe.org and the Massachusetts amendments package is sold by the State House Bookstore (617-727-2834).

Key references needed to properly use all chapters of the Base Code are listed in the table below.
Updates to Draft 9th Edition of the Building Code approved by BBRS on 1/12/2016

The purpose of this notice is to provide stakeholders with the status of the draft 9th Edition of the MA State Building Code. As of January 12, 2016, the Board of Building Regulations and Standards (“BBRS”) completed an effort, which has lasted over two (2) years, by approving a final draft of the 9th Edition of the MA State Building Code (“780 CMR”). The proposed new code is based upon the 2015 I-Codes, as published by the International Code Council (“ICC”). This effort is a testament to the hard work performed by a large group of volunteers, sister agencies, and professional organizations who assisted the BBRS through participation in Technical Advisory Committees and presentations for the BBRS at public meetings.
Massachusetts Building Code

Massachusetts 9th Edition Building Code

IBC 2015

ASCE 7-10
Massachusetts Amendments

1609.3 Replace the first paragraph with the following:

1609.3 Ultimate Wind Speed The ultimate design wind speed, $V_{ult}$ in mph, shall be determined in accordance with Table 1604.11.

### TABLE 1604.11 SNOW LOADS, WIND SPEEDS, AND SEISMIC PARAMETERS

<table>
<thead>
<tr>
<th>City/Town</th>
<th>Ground Snow Load, $P_g$ (psf)</th>
<th>Minimum Flat Roof Snow Load, $P_{f1}$ (psf)</th>
<th>Risk Category I</th>
<th>Risk Category II</th>
<th>Risk Category III or IV</th>
<th>$S_a$</th>
<th>$S_l$</th>
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<td>35</td>
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<td>122</td>
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<td>0.059</td>
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<td>Adams$^2$</td>
<td>60</td>
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<td>105</td>
<td>115</td>
<td>120</td>
<td>0.172</td>
<td>0.069</td>
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<td>0.065</td>
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<td>118</td>
<td>125</td>
<td>0.172</td>
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Wind Speed By Location Software

Search Results

Query Date: Thu Oct 08 2015
Latitude: 35.2271
Longitude: -80.8431

ASCE 7-10 Windspeeds (3-sec peak gust in mph):
- Risk Category I: 105
- Risk Category II: 115
- Risk Category III-IV: 120
- MRI** 10-Year: 76
- MRI** 25-Year: 84
- MRI** 50-Year: 90
- MRI** 100-Year: 96

ASCE 7-05 Windspeed:
- 90 (3-sec peak gust in mph)

ASCE 7-93 Windspeed:
- 71 (fastest mile in mph)

windspeed.atcouncil.org
Wind Code Changes

The main changes in wind loads from ASCE 7-05 to ASCE 7-10 are:

- Base wind loads are Ultimate rather than ASD
- Occupancy/Importance factor built into Wind Speed Maps rather than included in equations
- Introduced inclusion of Exposure D in Hurricane Prone Regions
- Revised triggers for Hurricane Prone Regions and Wind Borne Debris Regions
Calculating Wind Loads

- **ASCE 7-05**
  - Chpt. 6: Contained All Provisions

- **ASCE 7-10**
  - Chpt. 26: General Requirements
  - Chpt. 27: MWFRS – Directional
  - Chpt. 28: MWFRS – Enveloped
  - Chpt. 29: Other Structures
  - Chpt. 30: Components & Cladding
  - Appendices
## Importance Factor, $I$ (Wind Loads)

### Table 6-1

<table>
<thead>
<tr>
<th>Category</th>
<th>Non-Hurricane Prone Regions and Hurricane Prone Regions with $V = 85-100$ mph and Alaska</th>
<th>Hurricane Prone Regions with $V &gt; 100$ mph</th>
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<tbody>
<tr>
<td>I</td>
<td>0.87</td>
<td>0.77</td>
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<td>II</td>
<td>1.00</td>
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<td>IV</td>
<td>1.15</td>
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</table>

**Note:**
1. The building and structure classification categories are listed in Table 1-1.
Determine Basic Wind Speed, V mph

Basic Wind Speeds for Occupancy Category II Buildings and Other Structures

<table>
<thead>
<tr>
<th>Location</th>
<th>Vmph (m/s)</th>
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<tbody>
<tr>
<td>Guam</td>
<td>195 (87)</td>
</tr>
<tr>
<td>Virgin Islands</td>
<td>165 (74)</td>
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<tr>
<td>American Samoa</td>
<td>160 (72)</td>
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<tr>
<td>Hawaii (Special Wind Region Statewide)</td>
<td>130 (58)</td>
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</table>

Source: ASCE 7-10

Per ASCE 7-10 Fig. 26.5-1A
Determine Basic Wind Speed, V

- **ASCE 7-05**
  - ASD Loads
  - 90 mph per fig. 6-1

- **ASCE 7-10** (figures incorporate importance factor)
  - Ultimate Loads
  - 115 mph per figure 26.5-1A for RK II
  - 120 mph per figure 26.5-1B for RK III & IV
  - 105 mph per figure 26.5-1C for RK I

Note: RK = Risk Category

Image Source: SK Ghosh Associates
Basic Wind Speed: Probabilities

• ASCE 7-05
  - Wind Speeds based on 50 year return period

• ASCE 7-10
  - RK I based on 300 year return period (15% probability of exceedance in 50 Years)
  - RK II based on 700 year return period (7% in 50 years)
  - RK III & IV based on 1,700 year return period (3% in 50 years)
Comparing ASCE 7-05 to ASCE 7-10:

Load Combinations:

7. \(0.6D + W\) (ASCE 7-05)
7. \(0.6D + 0.6W\) (ASCE 7-10)

3 Second Wind Speed:

90 mph (ASCE 7-05)

115 mph \(\times \sqrt{0.6} = 89\) mph (ASCE 7-10)

Final load on building is very similar for inland locations.
Example: *Boston* Basic Wind Speeds

8\textsuperscript{th} Edition Mass Code (ASCE 7-05)

\[ V_{\text{ASD}} = 105 \text{ mph} \]

9\textsuperscript{th} Edition Mass Code (ASCE 7-10)

\[ V_{\text{ULT}} = 128 \text{ mph (RK II)} \]

\[ V_{\text{ASD}} = (128)(\sqrt{0.6}) = 99 \text{ mph} \]
ASCE 7-05 to 7-10 Comparison

So, wind loads per ASCE 7-10 are similar to or slightly lower than those per ASCE 7-05?

Yes....and No

ASCE 7-10 re-introduced the possibility of having exposure D in hurricane prone regions
Hurricane Prone Regions

Boston is by definition in a hurricane prone region: Hurricane prone region: Atlantic Ocean and Gulf of Mexico coasts where RK II basic wind speed > 115 mph (ASCE 7-10 26.2)

<table>
<thead>
<tr>
<th>Location</th>
<th>Risk Category</th>
<th>ASCE 7-10 Design Wind Speed (MPH)</th>
<th>ASCE 7-10 [A] Exp C Velocity Pressure (psf)</th>
<th>ASCE 7-10 [B] Exp D Velocity Pressure (psf)</th>
<th>ASCE 7-05 Design Wind Speed (MPH)</th>
<th>ASCE 7-05 [C] Exp C Velocity Pressure (psf)</th>
<th>Ratio [A] [C]</th>
<th>Ratio [B] [C]</th>
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</thead>
<tbody>
<tr>
<td>Boston, MA</td>
<td>II</td>
<td>128</td>
<td>35.7</td>
<td>42.1</td>
<td>105</td>
<td>38.4</td>
<td>0.93</td>
<td>1.10</td>
</tr>
</tbody>
</table>
Running the Numbers: Velocity Pressure

• $q_z = 0.00256K_z K_{zt} K_d V^2$
  
  - $q_z = \text{velocity pressure (psf)}$
  
  - $K_z$ – Exposure coefficient, Table 30.3-1 (7-05 Table 6-3)
  
  - $K_{zt}$ – Topographic factor, Figure 26.8-1 (7-05 Figure 6-4)
  
  - $K_d$ – Directionality factor, Table 26.6-1 (7-05 Table 6-4)
Wind Loads Types

2 Types of Wind Loads

• **MWFRS – Main Wind Force Resisting System**
  An assemblage of structural elements assigned to provide support and stability for the overall structure. The system generally receives wind loading from more than one surface. Eg. Shearwalls, diaphragms

• **C&C – Components & Cladding**
  Elements of the building envelope that do not qualify as part of the MWFRS. Eg. Wall studs
Two Methods of Calculating MWFRS loads:

- **Envelope**: Pressure coefficients represent “pseudo” loading that envelope the desired moment, shear... Limited to low-rise

- **Directional**: Pressure coefficients reflect wind loading on each surface as a function of wind direction
How to decide which method to use:

**Envelope**: ASCE 7-10 Chapter 28

- Part 1: Can be used for all regular-shaped enclosed & partially enclosed buildings with mean roof height \( \leq 60 \text{ ft} \)

- Part 2 (Simplified): Can be used for all regular-shaped, enclosed, simple diaphragm buildings with mean roof height \( \leq 60 \text{ ft} \)
MWFRS Method Options

How to decide which method to use:

**Directional**: ASCE 7-10 Chapter 27

- **Part 1**: Can be used for all regular-shaped buildings
- **Part 2** (Simplified): Can be used for all regular-shaped, enclosed, simple diaphragm buildings with mean roof height \( \leq 160 \text{ ft} \)
MWFRS Method Options

- ASCE 7-10 MWFRS Options
  - Directional Method, CH 27
  - Envelope Method CH 28

Part 1:
- Enclosed, Partially Enclosed, Open Buildings
- All Heights

Part 2:
- Enclosed, Simple Diaphragm Buildings with h ≤ 160 ft

Part 1:
- Enclosed & Partially Enclosed Buildings with h ≤ 60 ft

Part 2:
- Enclosed, Simple Diaphragm Buildings with h ≤ 60 ft

Note: Wind Tunnel Procedure (ASCE 7-10 Chpt 31) can also be used
Comparison of methods to calculate MWFRS ($GC_{pf}$)

Example: Flat Roof, 30’ x 60’ Building:

Ch. 27 Directional

- Windward Wall (0.8)
- Leeward Walls (-0.3)
- Determine Gust Effect ($G$) = 0.85
- For MWFRS $GC_{pf} = (1.1)(0.85) = 0.935$

Ch. 28 Enveloped

- Limited to Low-Rise ($h \leq 60'$)
- Windward Wall (0.4)
- Leeward Wall (-0.29)
- For MWFRS $GC_{pf} = 0.69$

35% difference in loading not accounting for end zones.
Benificial to use the envelope method when its limitations are met.

MWFRS Method Options

ASCE 7-10 Fig. C28.4-1
Minimum Wind Loads

For both the Directional & Envelope Methods, consider minimum wind loads:

ASCE 7-10 Sections 27.1.5 & 28.4.4:

Wind Loads for MWFRS in an enclosed or partially enclosed building shall not be less than:
- 16 psf (ultimate or ~10 psf ASD) for walls
- 8 psf (ultimate or ~5 psf ASD) for roofs

Wall and roof loads shall be applied simultaneously. The design wind force for open buildings shall be not less than 16 psf ultimate (open building provisions apply only to Directional Method).
Building Enclosure

Accounts for degree to which wind forces can enter and exit a structure, creating varying amounts of internal wind pressure

3 building enclosure classifications:

Open, Partially Enclosed, and Enclosed
Internal Pressure Coefficient – Table 26.11-1

+- 0.18 - Enclosed
+- 0.55 – Partially Enclosed
Running the Numbers: Design Wind Pressure

• \( p = q_h[(G C_p) - (G C_{pi})] \)
  - \( p \) = Design wind pressure (psf)
  - \( q_h \) = velocity pressure (psf)
  - \( G C_p \): External pressure coefficient
    - Figures 27.4-1, 28.4-1, 30.4-1

Note: Figure 27.4-1 also requires Gust effect factor (G) per section 26.9

• \( G C_{pi} \): Internal pressure coefficient, Table 26.11-1 (7-05 Figure 6-5)
## Design Wind Pressure Tables

**ASCE 7-10**

### Table 27.6-1
MWFRS – Part 2: Wind Loads – Walls
Exposure B

<table>
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<th>V(mph)</th>
<th>110</th>
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IBC’s Alternate All-Heights Method

IBC Section 1609.6 provides an alternative to the Directional Wind Load Procedure in ASCE 7

Alternate All-Heights Method

Limitations such as:
• Building Height ≤ 75 ft
• Building Height/Width ≤ 4
• Building has simple diaphragm
• Others (IBC 1609.6.1)

\[ P_{\text{net}} = 0.00256V^2K_zC_{\text{net}}K_{zt} \]
IBC’s Alternate All-Heights Method

\[ P_{\text{net}} = 0.00256V^2K_zC_{\text{net}}K_{zt} \]

- \( V \) = Basic wind speed (ASCE 7)
- \( K_z \) = Exposure coefficient (ASCE 7)
- \( K_{zt} \) = Topographic factor (ASCE 7)
- \( C_{\text{net}} \) = Net-pressure coefficient (IBC Table 1609.6.2)
## IBC’s Alternate All-Heights Method

### IBC Table 1609.6.2

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<td>+ Internal pressure</td>
<td>- Internal pressure</td>
<td>+ Internal pressure</td>
<td>- Internal pressure</td>
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<tr>
<td>Walls:</td>
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<tr>
<td>Windward wall</td>
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<td>Leeward wall</td>
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<td>Sidewall</td>
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<tr>
<td>Parapet wall</td>
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<tr>
<td>Windward</td>
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<tr>
<td>Leeward</td>
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<tr>
<td>Roofs:</td>
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<tr>
<td>Wind perpendicular to ridge</td>
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<tr>
<td>Leeward roof or flat roof</td>
<td></td>
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<td>Windward roof slopes:</td>
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<tr>
<td>Slope &lt; 2:12 (10°)</td>
<td>Condition 1</td>
<td>-1.09</td>
<td>-0.79</td>
<td>-1.41</td>
<td>-0.47</td>
<td></td>
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<tr>
<td></td>
<td>Condition 2</td>
<td>-0.28</td>
<td>0.02</td>
<td>-0.60</td>
<td>0.34</td>
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<tr>
<td>Slope = 4:12 (18°)</td>
<td>Condition 1</td>
<td>-0.73</td>
<td>-0.42</td>
<td>-1.04</td>
<td>-0.11</td>
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<td></td>
<td>Condition 2</td>
<td>-0.05</td>
<td>0.25</td>
<td>-0.37</td>
<td>0.57</td>
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<tr>
<td>Slope = 5:12 (23°)</td>
<td>Condition 1</td>
<td>-0.58</td>
<td>-0.28</td>
<td>-0.90</td>
<td>0.04</td>
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<td>Condition 2</td>
<td>0.03</td>
<td>0.34</td>
<td>-0.29</td>
<td>0.65</td>
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<tr>
<td>Slope = 6:12 (27°)</td>
<td>Condition 1</td>
<td>-0.47</td>
<td>-0.16</td>
<td>-0.78</td>
<td>0.15</td>
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<tr>
<td></td>
<td>Condition 2</td>
<td>0.06</td>
<td>0.37</td>
<td>-0.25</td>
<td>0.68</td>
<td></td>
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<tr>
<td>Slope = 7:12 (30°)</td>
<td>Condition 1</td>
<td>-0.37</td>
<td>-0.06</td>
<td>-0.68</td>
<td>0.25</td>
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<td></td>
</tr>
<tr>
<td></td>
<td>Condition 2</td>
<td>0.07</td>
<td>0.37</td>
<td>-0.25</td>
<td>0.69</td>
<td></td>
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</tr>
<tr>
<td>Slope = 9:12 (37°)</td>
<td>Condition 1</td>
<td>-0.27</td>
<td>0.04</td>
<td>-0.58</td>
<td>0.35</td>
<td></td>
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<td></td>
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<td>0.14</td>
<td>0.44</td>
<td>-0.18</td>
<td>0.76</td>
<td></td>
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<tr>
<td>Slope = 12:12 (45°)</td>
<td>Condition 1</td>
<td>-0.14</td>
<td>0.44</td>
<td>-0.48</td>
<td>0.76</td>
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<tr>
<td>Wind parallel to ridge and flat roofs</td>
<td></td>
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</tbody>
</table>
Wind Borne Debris Regions

Per ASCE 7-10, section 26.2, Wind Borne Debris regions are Areas within hurricane-prone regions where impact protection is required for glazed openings (buildings in Risk Category I are exempt – ASCE 26.10.3 & IBC 1609.1.2)

Protection of glazed openings is required (ASCE 7 26.10.3):

* Within 1 mile of the coastal mean high water line where the basic wind speed is equal to or greater than 130 mph,

* In areas where the basic wind speed is equal to or greater than 140 mph

* Other exemptions, testing requirements given in ASCE 7-10, section 26.10.3
Failed openings can change a structure from enclosed to partially enclosed, significantly increasing wind forces.
Overview

• Calculating Wind Loads
• Uplift
• Wall Design
• Diaphragms
• Shearwalls
Uplift Wind Loads

Uplift – Outward (suction) force acting on roof

Load path - roof to foundation required unless dead load is greater than uplift
Uplift Loads

Source: strongtie.com
Methods to Resist Uplift Loads

- Mechanical connectors (straps, hurricane ties, screws, threaded rods)
- Sheathing
- Dead Loads

Source: strongtie.com
Uplift Resistance: Mechanical Connectors

Continuous Load Path To Resist Uplift Forces

- Roof to Wall Connection
  - Roof member to top plate connections
  - Top plate to stud connections

- Upper Wall to Lower Wall Connection
  - This connection is not required for a single story home.

- Lower Wall to Foundation Connection
  - Stud to sill plate connections
  - Sill plate to foundation connections

Source: IIBHS
Uplift Resistance: Wall Sheathing

- When joints, fasteners are considered, can use sheathing to resist uplift
- SDPWS Section 4.4
Uplift Resistance: Wall Sheathing

SDPWS Figure 4J
Using Dead Load to Resist Uplift

Dead load from above (Wall, Floor, Roof) can be used to resist some or all uplift forces, depending on magnitude.

Load Combinations of ASCE 7-10: **06.D + 0.6W**

Source: Strongtie
What happens to the uplift load after this?
Uplift: MWFRS or C&C?

Consider member part of MWFRS if:

- Tributary Area > 700ft$^2$ per ASCE 7-10 30.2.3
- Load coming from more than one surface per ASCE 7-10 26.2
Uplift: MWFRS or C&C?

AWC’s WFCM commentary C1.1.2 states that MWFRS is used for all uplift conditions:

The rationale for using MWFRS loads for computing the uplift of roof assemblies recognizes that the spatial and temporal pressure fluctuations that cause the higher coefficients for components and cladding are effectively averaged by wind effects on different roof surfaces.
Uplift: MWFRS or C&C?

ASCE 7-10 26.2 commentary provides some discussion on uplift & MWFRS vs. C&C.

Components receive wind loads directly or from cladding and transfer the load to the MWFRS. Examples of components include fasteners, purlins, girts, studs, roof decking, and roof trusses. Components can be part of the MWFRS when they act as shear walls or roof diaphragms, but they may also be loaded as individual components.
Effective Wind Area

For wind design, tributary area does not necessarily = effective wind area

Effective Wind Area (EWA) - Two cases:
• Area of building surface contributing to force being considered (tributary area)
• Long and narrow area (wall studs, roof trusses): width of effective area may be taken as 1/3 length; increases effective area, decreases load (per ASCE 7-10 section 26.2 commentary); EWA = $L^2/3$
Effective Wind Area Example

Trib. A = (44)(2) = 88 ft²

EWA = 44²/3 = 645 ft²
Uplift Example Calculation

- Roof Framing Rafter
- 20’ Span
- 2’ Spacing
- 2’ Overhang
- 115 mph Exposure B
- Roof H = 80 ft
- 65’x220’

Photo credit: Matt Todd & PB Architects
MWFRS - External Pressure Coefficient

Look at wind acting on building’s long side:
$L = 65\text{ ft}$, $h/L = 80/65 = 1.23$

$C_p = -1.3, -0.18$

ASCE 7-10 Fig. 27.4-1
MWFRS - Running the numbers

• GC_p: (0.85)(-1.3) = 1.105 (26.9.4 & Fig. 27.4-1)
• GC_{pi}: ±0.18 (Table 26.11-1)
• q_h = 0.00256 K_z K_{zt} K_d V^2
  ▪ K_z : 0.93 – Table 27.3-1
  ▪ K_{zt} : 1.00 - Figure 26.8-1
  ▪ K_d : 0.85 - Table 26.6-1
  ▪ V_u: 115 mph
• q_h = 26.8 psf
• p = (26.8 psf)(-1.105+(-0.18)) = 34.4 psf
MWFRS - Roof Overhang per section 27.4.4

• For Overhangs: ASCE 7 27.4.4 – use $C_p = 0.8$ on underside of overhang, use same top pressures calculated for typ. roof

• $p_{oh} = (26.8 \text{ psf})(-0.8)(0.85) = 18.2 \text{ psf}$

• $p_{ext} = (26.8 \text{ psf})(-1.105) = 29.6 \text{ psf}$

• $p_{oh\ net} = 18.2 + 29.6 = 47.8 \text{ psf}$
MWRFS - Determining the Uplift Load

• \( p = (34.4 \text{ psf})(2\text{ ft}) = 68.8 \text{ plf} \)
• \( p_{oh} = (47.8 \text{ psf})(2\text{ ft}) = 95.6 \text{ plf} \)

Uplift = 0.6(95.6 \text{ plf}(2\text{ ft.}) + 68.8 \text{ plf}\times20\text{ ft}/2) = 528 \text{ lbs}

Dead Load = 0.6((2+20/2)\times10\text{ psf}\times2\text{ ft}) = 144 \text{ lbs}

Net Uplift at Left Support = 528 \text{ lbs} - 144 \text{ lbs} = 384 \text{ lbs}

Note: It is common practice to use 2 sets of dead loads: highest potential dead loads for gravity, lowest potential dead loads for uplift
C&C - External Pressure Coefficient

3 zones with differing wind loads:

1: Field
2: Perimeter
3: Salient corners

\[ a = \text{smaller of 10\% of least horizontal dimension or 0.4h, but not less than either 4\% of least horizontal dimension of 3 ft} \]
EWA = $H^2/3 = 22^2/3 = 161\text{ft}^2$

$GC_p = -1.1$ FOR INTERIOR
C&C - Running the numbers – Zone 2

- $G_{C_p}$: -1.1 (Figure 30.4-2A)
- $G_{C_{pi}}$: ±0.18 (Table 26.11-1)
- $q_h = 0.00256K_zK_{zt}K_dV^2$
  - $K_z$: 0.93 - Table 30.3-1
  - $K_{zt}$: 1.00 - Figure 26.8-1
  - $K_d$: 0.85 - Table 26.6-1
  - $V_u$: 115 mph
- $q_h = 26.8$ psf
- $p = (26.8$ psf$)(-1.1+(-0.18)) = 34.3$ psf
C&C - Roof Overhang per section 30.10

- For Overhangs Figures 30.4-2A & 30.10-1 are utilized
- \( p_{oh} = 26.8 \text{ psf} \times (1.7 + 0.18) = 50.4 \text{ psf} \)
- \( p_s = p_w = 34.3 \text{ psf} \)
- \( p_{oh\, net} = 50.4 + 34.3 = 84.7 \text{ psf} \)

\[ EWA = 2 \times 2 = 4 \text{ sf} \]
\[ GC_p = -1.7 \]

Per ASCE 7-10 Fig. 30.10-1

ASCE 7-10 Fig. 30.4-2A
C&C - Determining the Uplift Load

- \( p = (34.3 \text{ psf})(2\text{ft}) = 68.6 \text{ plf} \)
- \( p_{oh} = (84.7 \text{ psf})(2\text{ft}) = 169.4 \text{ plf} \)

Uplift = 0.6(169.4 \text{ plf}(2\text{ft.}) + 68.6 \text{ plf} \times 20\text{ft}/2) = 615 \text{ lbs}
Dead Load = 0.6((2+20/2) \times 10\text{psf} \times 2\text{ft}) = 144 \text{ lbs}

Net Uplift at Left Support = 615 \text{ lbs} - 144 \text{ lbs} = 471 \text{ lbs}

Note: It is common practice to use 2 sets of dead loads: highest potential dead loads for gravity, lowest potential dead loads for uplift.
Determining the Uplift Load

- 384 lbs MWFRS OR
- 471 lbs C&C

@ ea. rafter
Roof Framing: Compression Edge Bracing

- Bending causes compression in one edge of member
- Roof sheathing braces compression flange of roof joists
Roof Framing: Compression Edge Bracing

• What about Uplift? Need full depth blocking/bridging or bottom chord bracing
Overview

• Calculating Wind Loads
• Uplift
• Wall Design
• Diaphragms
• Shearwalls
Wind Loads

Uniform surface wind loads generally increase with building height

If wind loads vary with building height, common to use higher wind load over a single story or building
Wall Design Considerations

- Panels
- L/d Ratio
- Unbraced Length
- Wall Veneer
- Wind only loading C&C
- Design Properties
- Hinges
Wood Frame Design

National Design Specification (NDS): Provides design procedures and reference design values used in the structural design of wood framing members and connections.

IBC: References National Design Specification (NDS) for design of wood construction.
Loads into WSP

Wind loads are transferred to wall framing studs through wood structural panels (sheathing)

SDPWS Table 3.2.1

### Table 3.2.1 Nominal Uniform Load Capacities (psf) for Wall Sheathing Resisting Out-of-Plane Wind Loads

<table>
<thead>
<tr>
<th>Sheathing Type 3 (Sheathing Grades, C-C, C-D, C-C Plugged, OSB) 4</th>
<th>Span Rating or Grade</th>
<th>Minimum Thickness (in.)</th>
<th>Perpendicular to Supports</th>
<th>Parallel to Supports</th>
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</thead>
<tbody>
<tr>
<td></td>
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<td></td>
<td>Maximum Stud Spacing (in.)</td>
<td>Actual Stud Spacing (in.)</td>
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<td>12</td>
<td>16</td>
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<tr>
<td>Wood Structural Panels</td>
<td>24/0</td>
<td>3/8</td>
<td>24</td>
<td>425</td>
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<td>24/16</td>
<td>7/16</td>
<td>24</td>
<td>540</td>
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<tr>
<td></td>
<td>32/16</td>
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<td>24</td>
<td>625</td>
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<td>40/20</td>
<td>19/32</td>
<td>24</td>
<td>955</td>
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<tr>
<td></td>
<td>48/24</td>
<td>23/32</td>
<td>24</td>
<td>1160</td>
</tr>
</tbody>
</table>

For ASD Capacity: Divide Nominal Capacity by 1.6
For LRFD Capacity: Multiply Nominal Capacity by 0.85
Calculating Deflection – IBC Table 1604.3

For $\Delta$ of most brittle finishes use $l/240$

For C&C pressures a 30% load reduction is allowed for $\Delta$ only (IBC Table 1604.3 footnote f)

<table>
<thead>
<tr>
<th>CONSTRUCTION</th>
<th>$L$</th>
<th>$S$ or $W^f$</th>
<th>$D + L^{d, g}$</th>
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<tbody>
<tr>
<td>Roof members: e</td>
<td>$l/360$</td>
<td>$l/360$</td>
<td>$l/240$</td>
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<tr>
<td>Supporting plaster or stucco ceiling</td>
<td>$l/240$</td>
<td>$l/240$</td>
<td>$l/180$</td>
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<tr>
<td>Supporting nonplaster ceiling</td>
<td>$l/180$</td>
<td>$l/180$</td>
<td>$l/120$</td>
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<tr>
<td>Not supporting ceiling</td>
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<td></td>
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<tr>
<td>Floor members</td>
<td>$l/360$</td>
<td>—</td>
<td>$l/240$</td>
</tr>
<tr>
<td>Exterior walls and interior partitions:</td>
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<td>$l/360$</td>
<td></td>
</tr>
<tr>
<td>With plaster or stucco finishes</td>
<td></td>
<td>$l/240$</td>
<td></td>
</tr>
<tr>
<td>With other brittle finishes</td>
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<td>$l/120$</td>
<td></td>
</tr>
<tr>
<td>With flexible finishes</td>
<td></td>
<td>—</td>
<td></td>
</tr>
</tbody>
</table>

f. The wind load is permitted to be taken as 0.42 times the "component and cladding" loads for the purpose of determining deflection limits herein.
Wood Studs with Brick Veneer - Deflection

IBC Table 1604.3: min. wall deflection with brittle finishes = L/240

Brick Industry Association recommends much stricter limits

Structure Magazine May 2008 article, Harold Sprague

BIA Tech Note 28
Wall Design: MWFRS or C&C?
3 Step Process: Exterior Wall Design

- **Strength Check 1:**
  Gravity (axial) + Main Wind Force Loads

- **Strength Check 2:**
  Full Components and Cladding Wind Loads, No Axial (or minimal axial)

- **Deflection Check:**
  Reduced Components and Cladding Wind Loads
Wall Design Considerations

For other design issues see the article:

• Considerations in Wind Design of Wood Structures
• Free download from AWC available at:
  

CONSIDERATIONS IN WIND DESIGN OF WOOD STRUCTURES

Bradford K. Douglas, P.E.
Brian R. Weeks, P.E.

Proper design of wood structures to resist high wind loads requires the correct use of wind load provisions and member design properties. A thorough understanding of the interaction between wind loads and material properties is important in the design process.

There are varying wind load provisions in local, state and model building codes currently used in the United States. Most of these provisions are based on wind engineering research conducted over the last 50 years. Proposals to change current code provisions are the result of interpretations of new state-of-the-art wind engineering research.
Strength Check 2 for Stud Design

Strength Check for Components & Cladding Winds

- No axial loading
- C&C transverse Wind loads only
- Check stud bending and shear.

**Design Tip:** For bending stress check, be aware of Repetitive Use factor $C_r$ of NDS and Wall Stud Repetitive Member Factor of SDPWS 3.1.1. Change in SDPWS 2015 allows application of Wall Stud Repetitive Factor to Stud STIFFNESS. See SDPWS 3.1.1

<table>
<thead>
<tr>
<th>Table 3.1.1.1 Wall Stud Repetitive Member Factors</th>
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<tbody>
<tr>
<td>Stud Size</td>
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</tr>
<tr>
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<td>2x6</td>
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<tr>
<td>2x8</td>
</tr>
<tr>
<td>2x10</td>
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<tr>
<td>2x12</td>
</tr>
</tbody>
</table>
Deflection Check for Stud Design

Deflection Check for Components and Cladding Winds

• Check out-of-plane deflection to IBC Table 1604.3 or other more stringent requirements.

Note: This check often governs tall walls

Design Tip: Read all the footnotes! IBC Table 1604.3 footnote f allows the following C&C Wind load reduction:

Multiply calculated C&C Wind Loads by 0.42 when using \( V_{\text{ULT}} \) (ASCE 7-10) OR 0.70 when using \( V_{\text{ASD}} \) (ASCE 7-05 and earlier) for deflection
Wall Stud Design Aid

Western Wood Products Association (WWPA) Design Suite:
Example: Office Building Wall Studs

2 Story Building
13’ tall wood framed walls.
Assume studs 16” o.c.
110 mph Exposure C
Least Horizontal Dim. = 90ft
Wall Stud Design: **Strength Check 1**

Gravity Loads:

Roof Dead Load = 20 psf;  
Floor Dead Load = 30 psf

Roof Live Load = 20 psf;  
Floor Live Load = 65 psf

Wall Dead Load = 18 psf;  
Wall Deflection = L/360

Roof & Floor Tributary Width = (22 ft)(0.5) = 11 ft

Wall Tributary Width = 13 ft + 13 ft = 26 ft

\[ W_{DL} = (11ft)(20\text{psf} + 30\text{psf}) + (26ft)(18\text{psf}) = 1018 \text{ plf} \]

\[ W_{RL} = (11ft)(20\text{psf}) = 220 \text{ plf} \]

\[ W_{LL} = (11ft)(65\text{psf}) = 715 \text{ plf} \]

Controlling Load Combo: D + L = 1018 + 715 = 1733 plf
Wall Stud Design: **Strength Check 1**

Gravity Loads:

Axial Load Per Stud = (1733 plf)(1.333 ft) = **2310 lb**

Bottom plate crushing: 2310/(1.5”*5.5”) = 280 psi < 625 psi: OK

MWFRS Wind Loads:

ULT. = 28.5 psf; ASD = (28.5psf)(0.6) = **17.1 psf**  ASCE Table 27.6-1

![Table 27.6-1](image)

MWFRS – Part 2: Wind Loads – Walls

<table>
<thead>
<tr>
<th>V(mph)</th>
<th>110</th>
<th>115</th>
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<tbody>
<tr>
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<td>0.5</td>
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<tr>
<td>h(ft.), L/B</td>
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<td>22.6</td>
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<tr>
<td>30</td>
<td>26.2</td>
<td>26.2</td>
</tr>
<tr>
<td>20</td>
<td>25.8</td>
<td>25.8</td>
</tr>
<tr>
<td>15</td>
<td>25.2</td>
<td>25.2</td>
</tr>
</tbody>
</table>
2x6 DF # 2 Studs @ 16” o.c. OK for Strength Check 1

** Dimension Lumber **

Nominal Size :  (1)  2 x 6  
Species = Douglas Fir-Larch  
Grade = No.2

Sill Plate Nominal Size :  2 x 6  
Species or Symbol = Douglas Fir-Larch  
Grade = No.2

** Dimension Lumber **

Bearing at < 3” of Sill End?  No

Height (H) = 13 ft - 0 in
Unbraced Length (l1) = 13 ft - 0 in
Unbraced Length (l2) = 2 ft - 0 in

Setup

(pressed-down buttons are selected)

Repetitive Use ?  No  Yes
Incised for PT ?  No  Yes
Flat Use :  No  Yes
Moisture Content :  <19%  >19%
Temperature (° F) :  <100  100~125  125~150

<table>
<thead>
<tr>
<th>Set Duration Factors</th>
<th>CD</th>
<th>1.00 (P)</th>
</tr>
</thead>
<tbody>
<tr>
<td>CD</td>
<td>1.60 (P+w)</td>
<td></td>
</tr>
<tr>
<td>(1.3/2) fb (psi)</td>
<td>497</td>
<td>&lt; 1346 = Fb</td>
</tr>
</tbody>
</table>

for P only, fc (psi) = 280 < 533 = Fc //  
for P + w, fc (psi) = 280 < 558 = Fc //  

( (fc / Fc)^2 + fb / [F'b (1 - fc / Fce)] ) = 0.95 < 1.00 OK  
Mid-H Deflection due to w, Δ (inch) = 0.85 < H / 120 OK
C&C Wind Loads: ASCE 7 Fig. 30.4-1

\[ a = \text{Lesser of:} \]

- 10% least horizontal dimension (LHD) \( 90' \times 0.1 = 9' \)
- \( 0.4h = 0.4 \times 26 = 10.4' \)

But not less than:

- \( 0.04 \text{ LHD} = 3.6' \) or 3'

Use \( a = 9' \) for zone 5
Strength Check 2: C&C Wind Loads

Wall studs are 13’ long

EWA = \( \frac{h^2}{3} = 56 \text{ ft}^2 \)

Zone 4:

\( G_{C_{pf}} = -0.97 \)

\( G_{C_{pi}} = -0.18 \) (Table 26.11-1)

Zone 5:

\( G_{C_{pf}} = -1.1 \)
Running the numbers – Zone 4

- $GC_{pf}: 0.97$ (Figure 30.4-1)
- $GC_{pi}: 0.18$ (Table 26.11-1)
- $q_h = 0.00256K_zK_{zt}K_d V^2$
  - $K_h : 0.98$ - Table 30.3-1
  - $K_{zt} : 1.00$ - Figure 26.8-1
  - $K_d : 0.85$ - Table 26.6-1
  - $V: 110$ mph
- $q_h = 25.8$psf
- $p = 25.8$psf$(0.97+0.18) = 29.7$psf
- $0.6W = 0.6(29.7) = 17.8$psf
## Strength Check 2 & Deflection Check (Zone 4)

2x6 DF # 2 Studs @ 16” o.c. OK for Strength Check 2 & Deflection Check

### Member Details
- **Nominal Size:** (1) 2 x 6
- **Species:** Douglas Fir-Larch
- **Grade:** No.2

### Design Calculations
- **Height:** 13 ft 0 in
- **Unbraced Length (L1):** 13 ft 0 in
- **Unbraced Length (L2):** 2 ft 0 in

### Load Conditions
- **P:** 1357 lb = DL + FL
- **w:** 23.7 plf = Wind
- **lu:** 13 ft 0 in

### Section Properties
- **Post/Stud:**
  - **b:** 1.5 in
  - **d:** 5.5 in
- **Sill PL:
  - **b:** 1.5 in
  - **d:** 5.5 in
- **Beam Stability:**
  - **CL:** 1.00
  - **CF:** 1.30
  - **fu:** 1.00
  - **i:** 1.00
- **Flat Use:**
  - **Cfu:** 1.00
  - **Ct:** 1.00
  - **K:** 1.00
  - **D:** 1.60

### Stability Calculations
- **Adjusted (P):**
  - **Fb:** 516 psi
  - **Fc:** < 1346 psi
- **Adjusted (P+w):**
  - **Fb:** 516 psi
  - **Fc:** < 1346 psi

### Moment of Inertia
- **I:** 20.8 in^4

### Temperature (°F)
- **fc (psi):**
  - **For P:** 164 psi
  - **For P + w:** 164 psi

### Reference
- **900 1350 1600 00000 625 580000**
- **Column Stability (P):**
  - **CP:** 0.36
- **Column Stability (P+w):**
  - **CPw:** 0.23

### Designed on
- April 12, 2016

### Developed by
- Forum Engineers

### How to Enter Data
- ASD Method
- Yes
- No
- Yes
- No
- Yes
- No
- Yes
- No
- Yes
- No

### Effective-Depth Factor
- **Cp:** 1.00 (P)
- **K:** 1.00
- **D:** 1.60 (P+w)

### Deflection Limit
- **f_c / f_c:** 2 + fb / [Fb (1 - fc / Fce)] = 0.62 < 1.00 OK
- **Mid-H Deflection due to w, ∆ (inch):** 0.32 < H / 360 OK
Running the numbers – Zone 5

- \( GC_p : 1.1 \) (Figure 30.4-1)
- \( GC_{pi} : 0.18 \) (Table 26.11-1)
- \( q_h = 0.00256K_zK_{zt}K_dV^2 \)
  - \( K_h : 0.98 \) - Table 30.3-1
  - \( K_{zt} : 1.00 \) - Figure 26.8-1
  - \( K_d : 0.85 \) - Table 26.6-1
  - \( V : 110 \) mph
- \( q_h = 25.8 \) psf
- \( p = 25.8 \) psf \((1.1+0.18) = 33 \) psf
- \( 0.6W = 0.6(33) = 19.8 \) psf
Strength Check 2 & Deflection Check (Zone 5)

2x6 DF # 2 Studs @ 16” o.c. OK for Strength Check 2 & Deflection Check

Member #

Location:

Sits on Sill Plate?

Yes

Nominal Size:

(1)

2 x 6

Species =

Douglas Fir-Larch

Grade =

No.2

Species or Symbol =

Douglas Fir-Larch

Grade =

No.2

Bearing at < 3” of Sill End?

No

P = 1357 lb = DL + FL

w = 26.4 plf = Wind

lu = 13 ft - 0 in

for P only, fc (psi) = 164 < 533 = Fc //

for P + w, fc (psi) = 164 < 558 = Fc //

(1.3/2) fb (psi) = 575 < 1346 = Fb

Mid-H Deflection due to w, ∆ (inch) = 0.36 < H / 360 OK

Section Properties

Post/Stud

Sill PL

breadth (b) = 1.5 in

depth (d) = 5.5 in

Bending Comp // E

Area (A) = 8.3 in^2

Wet Service

C M = 1.00

Temperature (°F)

for P + w, fc (psi) = 164 < 558 = Fc //

C t = 1.00

Moment of Inertial (I) = 20.8 in^4

Beam Stability

C L = 1.00

Size

C F = 1.30

Flat Use

C fu = 1.00

Incising

C i = 1.00

R = 1.15

Reference

900 1350 1600000 625 580000

Column Stability (P)

C P = N/A

Adjusted (P) 533 1600000 781 580000

Column Stability (P+w)

C Pw = N/A

Adjusted (P+w) 1346 558 1600000 781 580000

Bearing Area

C b = N/A

1485

2152.8 2376

Adjustment Factors

Set Duration Factors

Set Effective-Length Factor

Set Deflection Limit

For data entry, enter values in the table below.
Gable End Wall Hinge
Gable End Bracing Details

Gable end wall and roof framing may require cross bracing.
If no openings in gable end wall exist, can design studs to span from floor/foundation to roof (varying stud heights). May require closer stud spacings at taller portions of wall.
Gable End Walls with Openings
Gable End Walls with Openings
- Often gable end walls are locations of large windows
- Horizontally spanning member in plane of wall breaks stud length, provides allowable opening
Dropped Headers: Out of Plane Braced?
When Stair Shaft Wall is Exterior Wall

Intermediate Stair Landing

Wall Plates at Typical Floor Elevation – Creates Potential “Hinge”

Splice Exterior Wall Studs at Stairwells at Standard Floor Elevations. 2x6 Plates shall be continuous for the full length of the stair opening and shall extend 2'-0" beyond the opening. Overlap plates at corner and nail to each plate to the plate below with (4) 10d nails.

2x10 Ledger fasted to each wall stud with (3) 16d nails

LU210 Face Mount Hanger
Stair Exterior Wall Detail

**Stair Shaft Side**

**Exterior Side**

Consider “Hinge” at wall plates for out-of-plane wind & seismic loads due to lack of adjacent floor:

- Install additional member (rim) to span horizontally
- Options include solid sawn lumber (4x or 6x), glulam, PSL
- If multi-ply member, unique design considerations
Exterior Wall Plate Elevations Shifted Down to Intermediate Landing Elevation

- Eliminates Hinge Effect
- Avoids Interference with Landing Windows

Intermediate Stair Landing

When Stair Shaft Wall is Exterior Wall Wall
Overview

- Calculating Wind Loads
- Uplift
- Wall Design
- Diaphragms
- Shearwalls
Diaphragm Design
Wind Load Distribution to Diaphragm

- Wind into Diaphragms
- Wind surface loads on walls
Wind Load Paths

WIND INTO DIAPHRAGMS AS UNIFORM LINEAR LOADS
Wind Load Paths

DIAPHRAGMS SPAN BETWEEN SHEARWALLS

WIND INTO SHEARWALLS AS CONCENTRATED LOADS
Stud to Diaphragm

DIAPHRAGM SHEATHING

FLOOR JOIST

Floor/Roof framing perpendicular to walls

WIND LOAD
Stud to Diaphragm

- **DIAPHRAGM SHEATHING**
- **WIND LOAD**
- **FLOOR JOIST BLOCKING**

Floor/Roof framing parallel to walls (add blocking)
Unblocked Diaphragm
Blocked Diaphragm
Wood Frame Lateral Design

SDPWS: Provides capacities of most wood-framed vertical and horizontal lateral force resisting systems

IBC: References Special Design Provisions for Wind & Seismic (SDPWS) for capacities of most wood framed lateral systems. IBC provides capacity of stapled WSP and gypsum shear walls
Example: Retail Restaurant

Assume Basic Wind Speed = 115 mph Ultimate

Exposure B

Diaphragm Design
  • Capacity

Shearwall Design
  • Conventional
  • Force Transfer Around Opening
  • Perforated Shearwall
Retail Restaurant – Diaphragm Design

Critical Shearwall at front of building

Check Diaphragm for wind loads on 84’ wall
## Diaphragm Aspect Ratios

### SDPWS TABLE 4.2.4

<table>
<thead>
<tr>
<th>TYPE</th>
<th>MAXIMUM LENGTH/WIDTH RATIO</th>
</tr>
</thead>
<tbody>
<tr>
<td>Wood structural panel, unblocked</td>
<td>3:1</td>
</tr>
<tr>
<td>Wood structural panel, blocked</td>
<td>4:1</td>
</tr>
<tr>
<td>Single-layer straight lumber sheathing</td>
<td>2:1</td>
</tr>
<tr>
<td>Single-layer diagonal lumber sheathing</td>
<td>3:1</td>
</tr>
<tr>
<td>Double-layer diagonal lumber sheathing</td>
<td>4:1</td>
</tr>
</tbody>
</table>

For an 84 x 34 diaphragm the aspect ratio is 2.5 < 3. Diaphragm aspect ratio is OK.
Calculating MWFRS Wind Loads

Calculate wind pressure using Directional Method (ASCE 7 Chpt 27)

\[ p = q_h[(G_{C_{pf}})-(G_{C_{pi}})] \]

\[ q_h = 0.00256 \times 0.57 \times 1.0 \times 0.85 \times 115^2 \times 1 = 16.4 \text{ psf} \]

\[ G_{C_{pf}} = 0.85 \times [0.8 - (-0.3)] = 0.935 \]

\[ G_{C_{pi}} = 0.18 - 0.18 = 0 \]

\[ p = (16.4 \text{ psf})(0.935) = 15.34 \text{ psf} \]

\[ 0.6 \times W = 0.6 \times 15.34 = 9.2 \text{ psf on walls} \]

Use min 9.6 psf per ASCE 27.1.5
Section 27.4.5: $P_p = q(GC_{pn})$

$GC_{pn} = 1.5$ Windward parapet, -1.0 Leeward parapet

Windward Parapet $GC_{pf}$ is 1.5: $16.4 \times 1.5 \times 0.6 = 14.76$ psf

Leeward Parapet $GC_{pf}$ is 1.0: $16.4 \times 1.0 \times 0.6 = 9.84$ psf

Net Parapet = $14.76 + 9.84 = 24.6$ psf

At parapets windward and leeward pressures occur on each parapet.
Retail Restaurant – Diaphragm Design

\[ P = (9.6 \text{psf} \times (5' + 3') + (24.6) \times 3') \times (84'/2) = 6,325 \text{ lb} \]

\[ \nu_{\text{diaphragm}} = \frac{6,325 \text{ lb}}{34'} = 186 \text{ plf} \]
Diaphragm Types

CASE 1 DIAPHRAGM
• Higher Shear Values
• Panels perpendicular to floor framing for improved performance

CASES 2-6 May be preferred for low shear demand where changing framing direction helps
• HVAC runs
• Fire Blocking/Draft Stopping

N-S  4x8 sheathing  Roof Trusses
Diaphragm Types

SDPWS Tables 4.2A & B
Diaphragm Capacity - SDPWS Chpt 4

Table 4.2A Nominal Unit Shear Capacities for Wood-Frame Diaphragms

<table>
<thead>
<tr>
<th>Sheathing Grade</th>
<th>Common Nail Size</th>
<th>Minimum Fastener Penetration in Framing Member or Blocking (in.)</th>
<th>Minimum Nominal Panel Thickness (in.)</th>
<th>Minimum Nominal Width of Nailed Face at Adjoining Panel Edges and Boundaries (in.)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Structural I</td>
<td>6d</td>
<td>1-1/4</td>
<td>5/16</td>
<td>32</td>
</tr>
<tr>
<td></td>
<td>8d</td>
<td>1-3/8</td>
<td>3/8</td>
<td>3</td>
</tr>
<tr>
<td></td>
<td>10d</td>
<td>1-1/2</td>
<td>15/32</td>
<td>3</td>
</tr>
<tr>
<td>Structural I</td>
<td>6d</td>
<td>1-1/4</td>
<td>5/16</td>
<td>3</td>
</tr>
<tr>
<td></td>
<td>8d</td>
<td>1-3/8</td>
<td>3/8</td>
<td>3</td>
</tr>
<tr>
<td></td>
<td>10d</td>
<td>1-1/2</td>
<td>15/32</td>
<td>3</td>
</tr>
</tbody>
</table>

- Capacities are Nominal: Modify by ASD reduction factor of 2, Modify by LRFD multiplication factor of 0.8
- Capacity is reduced for species with Specific Gravity < 0.5
- For Spruce Pine Fir multiply by 0.92
Diaphragm Capacity: SDPWS Table 4.2C

<table>
<thead>
<tr>
<th>PANEL GRADE</th>
<th>COMMON NAIL SIZE OR STAPLE² LENGTH AND GAGE</th>
<th>MINIMUM FASTENER PENETRATION IN FRAMING</th>
<th>MINIMUM NOMINAL WIDTH OF FRAMING MEMBERS AT ADJOINING PANEL EDGES AND BOUNDARIES⁸</th>
<th>NAIL SPACING AT ALL PANEL EDGES</th>
<th>Case 1 (No unblocked edges or continuous joints parallel to load)</th>
<th>All other configurations (Cases 2, 3, 4, 5 and 6)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sheathing &amp; single floor</td>
<td>8d (2½ “ x 0.131”)</td>
<td>1 3/8”</td>
<td>2 IN.</td>
<td>6 IN.</td>
<td>460 (Seismic) 645 (Wind)</td>
<td>340 (Seismic) 475 (Wind)</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>7/16”</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>3IN.</td>
<td>6 IN.</td>
<td>510 (Seismic) 715 (Wind)</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>380 (Seismic) 530 (Wind)</td>
<td></td>
</tr>
</tbody>
</table>

Capacity is reduced for species with Specific Gravity < 0.5.
For Spruce Pine Fir multiply by 0.92

**Capacity** = (645 plf)(0.92)/2 = 297 plf

297 plf > 186 plf, diaphragm is adequate with sheathing & fastening as shown above
Multi-Story Wind Design

Source: WoodWorks Five-Story Wood-Frame Structure over Podium Slab Design Example
Diaphragm Modeling Methods

Possible Shear Wall Layouts

Robust Diaphragm Aspect Ratio

Not using all shared walls for Shear
Diaphragm Modeling Methods

Possible Shear Wall Layouts

Robust Diaphragm Aspect Ratio

But maybe not much wall available on exterior
Rigid or Flexible Diaphragm?

**Light Frame Wood Diaphragms often default to Flexible Diaphragms**

**Code Basis: ASCE 7-10 26.2 Definitions (Wind)**

*Diaphragms constructed of wood structural panels are permitted to be idealized as flexible*

**Code Basis: ASCE 7-10 12.3.1.1 (Seismic)**

*Diaphragms constructed of untopped steel decking or wood structural panels are permitted to be idealized as flexible if any of the following conditions exist:*

[...]

**c. In structures of light-frame construction where all of the following conditions are met:**

1. *Topping of concrete or similar materials is not placed over wood structural panel diaphragms except for nonstructural topping no greater than 1 1/2 in. thick.*
2. *Each line of vertical elements of the seismic force resisting system complies with the allowable story drift of Table 12.12-1.*
Hypothetical Flexible Diaphragm Distribution

Changing wall construction does NOT impact load to wall line

Large portion of load on little wall

Area tributary to corridor wall line

Area tributary to exterior wall line

Hypothetical Flexible Diaphragm Distribution
Hypothetical Rigid Diaphragm Distribution

Longer, stiffer walls receive more load.

Changing wall construction impacts load to wall line.

Narrow, flexible walls receive less load.

Diaphragm assumed to be rigid body.

Hypothetical Rigid Diaphragm Distribution
Can a Rigid Diaphragm be Justified?

ASCE 7-10 12.3.1.3 (Seismic)

[Diaphragms] are permitted to be idealized as flexible where the computed maximum in-plane deflection of the diaphragm under lateral load is more than two times the average story drift of adjoining vertical elements of the seismic force-resisting system of the associated story under equivalent tributary lateral load as shown in Fig. 12.3-1.

IBC 2012 Chapter 2 Definition (Wind & Seismic)

A diaphragm is rigid for the purpose of distribution of story shear and torsional moment when the lateral deformation of the diaphragm is less than or equal to two times the average story drift.
Rigid Diaphragm Analysis

Some Advantages of Rigid Diaphragm

• More load (plf) to longer interior/corridor walls
• Less load (plf) to narrow walls where overturning restraint is tougher
• Can tune loads to walls and wall lines by changing stiffness of walls

Some Disadvantages of Rigid Diaphragm

• Considerations of torsional loading necessary
• More complicated calculations to distribute load to shear walls
• May underestimate “Real” loads to narrow exterior walls
• Justification of rigid assumption
Two More Diaphragm Approaches

Semi-Rigid Diaphragm Analysis

- Neither idealized flexible nor idealized rigid
- Explicit modeling of diaphragm deformations with shear wall deformations to distribute lateral loads
- Not easy.

Enveloping Method

- Idealized as BOTH flexible and rigid.
- Individual components designed for worst case from each approach
- Been around a while, officially recognized in the 2015 SDPWS
Possible Shear Wall Layouts

The Cantilever Diaphragm Option
Possible Shear Wall Layouts

Robust Aspect Ratio but only supported on 3 sides...

Typical Unit
Cantilevered Diaphragms in SDPWS 2008

Open Front Structure

Cantilever Diaphragm

AWC SDPWS 2008 Figure 4A

AWC SDPWS 2008 Figure 4B
Cantilevered Diaphragms in SDPWS 2008

Open Front Structure

SDPWS 4.2.5.1.1

- $L \leq 25$ ft
- $L/W \leq 1$, one story
- $\leq 2/3$, multi-story

Exception: Where calculations show the diaphragm deflections can be tolerated, the length, $L$, can be increased to $L/W \leq 1.5$ for WSP sheathed diaphragms.
Cantilevered Diaphragms in SDPWS 2008

Cantilevered Diaphragm

SDPWS 4.2.5.2

Lc ≤ 25 ft

Lc/W ≤ 2/3
Possible Shear Wall Layouts

Open Front Structure or Cantilevered Diaphragm?
Cantilevered Diaphragms in SDPWS 2015

Open Front Structure with a Cantilevered Diaphragm

AWC SDPWS 2015 Figure 4A
Open Front Structure & Cantilevered Diaphragms in SDPWS 2015

Cantilevered Diaphragm

SDPWS 4.2.5.2
\( L'/W' \leq 1.5 \)

When Torsionally Irregular
\( L'/W' \leq 1, \text{ one story} \)
\( 2/3, \text{ multi-story} \)
\( L' \leq 35 \text{ ft} \)

Provided diaphragms modelled as rigid or semi-rigid and for seismic, the story drift at each edge of the structure within allowable story drift of ASCE 7. Story drifts include torsion and accidental torsional loads and deformations of the diaphragm.
Small Openings in Diaphragms

Accounting for openings in shear panels (diaphragms and shear walls) is a code requirement (IBC 2305.1.1)

No code path for checking minimum size opening limit (other than prescriptive design – IBC 2308.4.4.1 & 2308.7.6.1)

Do you need to account for a 12” square opening in a diaphragm?

Small Openings in Diaphragms

FPInnovations method for checking small holes in diaphragms:

Recommend running an analysis of the opening’s effects on the diaphragm unless the following conditions are met.

3. It is strongly recommended that analysis for a diaphragm with an opening should be carried out except where all four of the following items are satisfied:

a. Opening depth no greater than 15% of diaphragm depth;

b. Opening length no greater than 15% of diaphragm length;

c. Distance from diaphragm edge to the nearest opening edge is a minimum of 3 times the larger opening dimension; and

d. The diaphragm portion between opening and diaphragm edge satisfies the maximum aspect ratio requirement.
Overview

- Calculating Wind Loads
- Uplift
- Wall Design
- Diaphragms
- Shearwalls
Shearwall Functions

Wind Loads create shear (sliding) and racking forces on a structure

- Sliding resisted by shearwall base anchorage
- Racking resisted by shear panel & fasteners
Shear Wall Components: Wall Framing

- Strut/collector
- Wall Framing (Studs)
- Wall Top Plates
- Wall Sole Plate
- Blocking Between Studs at All Panel Edges

Note: Can use “un-blocked” wall but capacities can be significantly lower: SDPWS 4.3.3
**Shear Wall Components: WSP & Fasteners**

**Boundary Nailing** – Typ. 2” – 6” o.c.

- Field or Intermediate Nailing – Typ. 12” o.c.

**Sheathing Panels**
- OSB or Plywood

**Field or Intermediate Nailing**: Attaches panel to intermediate wall framing (studs, not along panel edges)

**Boundary Nailing**: Attaches all 4 edges of every panel to wall framing (studs, blocking, top & sole plates)
Panel Fasteners
Shearwalls - Overturning

Due to cantilever nature of shearwalls, overturning forces are also generated.

Overturning forces are resisted by tension/compression couple – tension portion resisted by dead loads and hold down anchors.
Shearwall - Cantilever Member

Reaction from diaphragm

Collector

Deflected shape

Chords

Sheathing

Tension edge

Compression edge

FDN

\[ v = \frac{R}{b} \]

\[ M = Rb \]

\[ M = Rh \]
Shear Wall Components: Base Anchorage, End Posts & Hold Downs

- **Strut/collector**

- **Sole Plate Uniform Anchorage**: Transfers shear from wall sole plate to floor/wall or foundation below.

- **Wall End Posts & Hold Down**: Transfers vertical tension & compression forces to floor/wall or foundation below.

**Wall End Posts** (Sized for Tension & Compression)

**Sole Plate Uniform Anchorage** (Nails, Screws, Anchor Bolts)
Shear Wall Holdown Options

Strap Holdown Installation
6+ kip story to story capacities

Standard Holdown Installation
13+ kip capacities

Continuous Rod Tiedown Systems
100+ kip capacities
20+ kips/level
Threaded Rod Tie Down w/Take Up Device

Source: Strongtie

Source: hardyframe.com
Threaded Rod Tie Down w/o Take Up Device
<table>
<thead>
<tr>
<th>Material</th>
<th>Aspect Ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>Wood structural panels, blocked</td>
<td>3½:1¹</td>
</tr>
<tr>
<td>Wood structural panels, unblocked</td>
<td>2:1</td>
</tr>
<tr>
<td>Diagonal sheathing, single</td>
<td>2:1</td>
</tr>
<tr>
<td>Structural Fiberboard</td>
<td>3½:1³</td>
</tr>
<tr>
<td>Gypsum board, portland cement plaster</td>
<td>2:1²</td>
</tr>
</tbody>
</table>

1. For WSP shear walls with AR > 2:1, multiply shear wall capacity by 1.25 - 0.125h/bs
WSP Shearwall Capacity

• Capacities listed in AWC’s Special Design Provisions for Wind and Seismic (SDPWS)
• Sheathed shear walls most common. Can also use horizontal and diagonal board sheathing, gypsum panels, fiberboard, lath and plaster, and others
• Blocked shear walls most common. SDPWS has reduction factors for unblocked shear walls
• Capacities are given as nominal: must be adjusted by a reduction or resistance factor to determine allowable unit shear capacity (ASD) or factored unit shear resistance (LRFD)
Shearwall Capacity - SDPWS Chpt 4

Table 4.3A Nominal Unit Shear Capacities for Wood-Frame Shear Walls\(^1,3,6,7\)

<table>
<thead>
<tr>
<th>Sheathing Material</th>
<th>Minimum Nominal Panel Thickness (in.)</th>
<th>Minimum Fastener Penetration in Framing Member or Blocking (in.)</th>
<th>Fastener Type &amp; Size</th>
<th>Panel Edge Fastener Spacing (in.)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Seismic</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Panel Edge Fastener Spacing (in.)</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>6</td>
</tr>
<tr>
<td>Wood Structural Panels - Structural (^1,3,5)</td>
<td>5/16</td>
<td>1-1/4</td>
<td>6d</td>
<td>OSB</td>
</tr>
<tr>
<td>Wood Structural Panels - Sheathing (^4,5)</td>
<td>3/8</td>
<td>1-1/4</td>
<td>6d</td>
<td>OSB</td>
</tr>
<tr>
<td>Plywood Siding</td>
<td>5/16</td>
<td>1-1/4</td>
<td>6d</td>
<td>OSB</td>
</tr>
<tr>
<td>Particleboard Sheathing - (M-S &quot;Exterior Glue&quot; and M-2 &quot;Exterior Glue&quot;)</td>
<td>3/8</td>
<td>1-3/8</td>
<td>6d</td>
<td>OSB</td>
</tr>
<tr>
<td>Structural Fiberboard Sheathing</td>
<td>1/2</td>
<td>5/8</td>
<td>6d</td>
<td>OSB</td>
</tr>
</tbody>
</table>

A SEISMIC

<table>
<thead>
<tr>
<th>v₀ (plf)</th>
<th>G₀ (kips/in.)</th>
<th>v₄ (plf)</th>
<th>G₄ (kips/in.)</th>
<th>v₃ (plf)</th>
<th>G₃ (kips/in.)</th>
<th>v₂ (plf)</th>
<th>G₂ (kips/in.)</th>
</tr>
</thead>
<tbody>
<tr>
<td>400</td>
<td>13</td>
<td>110</td>
<td>16</td>
<td>1020</td>
<td>35</td>
<td>1430</td>
<td>2435</td>
</tr>
<tr>
<td>460</td>
<td>19</td>
<td>14</td>
<td>1120</td>
<td>37</td>
<td>1400</td>
<td>2054</td>
<td>2435</td>
</tr>
<tr>
<td>510</td>
<td>16</td>
<td>13</td>
<td>1340</td>
<td>24</td>
<td>1775</td>
<td>2054</td>
<td>2435</td>
</tr>
<tr>
<td>560</td>
<td>16</td>
<td>14</td>
<td>1480</td>
<td>24</td>
<td>1900</td>
<td>2054</td>
<td>2435</td>
</tr>
<tr>
<td>680</td>
<td>22</td>
<td>16</td>
<td>1740</td>
<td>24</td>
<td>1860</td>
<td>2054</td>
<td>2435</td>
</tr>
</tbody>
</table>

B WIND

<table>
<thead>
<tr>
<th>v₀ (plf)</th>
<th>v₄ (plf)</th>
<th>v₃ (plf)</th>
<th>v₂ (plf)</th>
</tr>
</thead>
<tbody>
<tr>
<td>560</td>
<td>840</td>
<td>1090</td>
<td>1430</td>
</tr>
<tr>
<td>645</td>
<td>1010</td>
<td>1290</td>
<td>1710</td>
</tr>
<tr>
<td>715</td>
<td>1105</td>
<td>1415</td>
<td>1875</td>
</tr>
<tr>
<td>785</td>
<td>1205</td>
<td>1540</td>
<td>2045</td>
</tr>
<tr>
<td>950</td>
<td>1430</td>
<td>1860</td>
<td>2435</td>
</tr>
</tbody>
</table>

Notes:
\(^1\) For use with 2" x 6" framing members.
\(^2\) For use with 2" x 8" framing members.
\(^3\) Data are based on uplift tests.
\(^4\) Panel fastener spacing shall not exceed 8".
\(^5\) Panel fastener spacing shall not exceed 12".
\(^6\) Panel fastener spacing shall not exceed 16".
\(^7\) Panel fastener spacing shall not exceed 24".

This table provides nominal unit shear capacities for wood-frame shear walls, incorporating different sheathing materials and panel types, as well as considerations for seismic and wind loads.
Shearwall Capacity - SDPWS Chpt 4

<table>
<thead>
<tr>
<th>Sheathing Material</th>
<th>Minimum Nominal Panel Thickness (in.)</th>
<th>Minimum Fastener Penetration in Framing Member or Blocking (in.)</th>
<th>Nail (common or galvanized box)</th>
<th>Fastener Type &amp; Size</th>
<th>B WIND Panel Edge Fastener Spacing (in.)</th>
<th>6</th>
<th>4</th>
<th>3</th>
<th>2</th>
</tr>
</thead>
<tbody>
<tr>
<td>Wood Structural</td>
<td>5/16</td>
<td>1-1/4</td>
<td>6d</td>
<td></td>
<td></td>
<td>560</td>
<td>840</td>
<td>1090</td>
<td>1430</td>
</tr>
<tr>
<td>Panels - Structural</td>
<td>3/8</td>
<td>1-3/8</td>
<td>8d</td>
<td></td>
<td></td>
<td>645</td>
<td>1010</td>
<td>1290</td>
<td>1710</td>
</tr>
<tr>
<td>7/16</td>
<td>15/32</td>
<td>1-1/2</td>
<td>10d</td>
<td></td>
<td></td>
<td>715</td>
<td>1105</td>
<td>1415</td>
<td>1875</td>
</tr>
<tr>
<td>15/32</td>
<td>8d</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>785</td>
<td>1205</td>
<td>1540</td>
<td>2045</td>
</tr>
<tr>
<td>15/32</td>
<td>10d</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>950</td>
<td>1430</td>
<td>1860</td>
<td>2435</td>
</tr>
<tr>
<td>Wood Structural</td>
<td>5/16</td>
<td>1-1/4</td>
<td>6d</td>
<td></td>
<td></td>
<td>505</td>
<td>755</td>
<td>980</td>
<td>1260</td>
</tr>
<tr>
<td>Panels - Structural</td>
<td>3/8</td>
<td>1-3/8</td>
<td>8d</td>
<td></td>
<td></td>
<td>560</td>
<td>840</td>
<td>1090</td>
<td>1430</td>
</tr>
<tr>
<td>7/16</td>
<td>15/32</td>
<td>1-1/2</td>
<td>10d</td>
<td></td>
<td></td>
<td>615</td>
<td>895</td>
<td>1150</td>
<td>1485</td>
</tr>
<tr>
<td>15/32</td>
<td>8d</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>670</td>
<td>980</td>
<td>1260</td>
<td>1640</td>
</tr>
<tr>
<td>15/32</td>
<td>10d</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>730</td>
<td>1065</td>
<td>1370</td>
<td>1790</td>
</tr>
<tr>
<td>19/32</td>
<td>10d</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>870</td>
<td>1290</td>
<td>1680</td>
<td>2155</td>
</tr>
<tr>
<td>15/32</td>
<td>10d</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>950</td>
<td>1430</td>
<td>1860</td>
<td>2435</td>
</tr>
</tbody>
</table>
P = 6,325 lb – from diaphragm calcs using Directional Method

Let’s see what happens when we use Envelope Method to calculate MWFRS loads to front shearwall
Calculating MWFRS Wind Loads

Calculate wind pressure using Envelope Method (ASCE 7 Chpt 28)

\[ p = q_h [ (G_{C\text{pf}}) - (G_{C\text{pi}}) ] \]

\[ q_h = 0.00256 \times 0.70 \times 1.0 \times 0.85 \times 115^2 \times 1 = 20.14 \text{ psf} \]

\[ G_{C\text{pf}} (\text{Zones 1 & 4}) = 0.4 - (-0.29) = 0.69 \] (ASCE 7 Fig. 28.4-1)

\[ G_{C\text{pf}} (\text{Zones 1E & 4E}) = 0.61 - (-0.43) = 1.04 \] (ASCE 7 Fig. 28.4-1)

\[ G_{C\text{pi}} = 0.18 - 0.18 = 0 \]

\[ P_{1\&4} = (20.14 \text{ psf})(0.69) = 13.9 \text{ psf}; 0.6 \times W = 0.6 \times 13.9 = 8.3 \text{ psf walls typ.} \]

\[ P_{1E\&4E} = (20.14 \text{ psf})(1.04) = 20.9 \text{ psf}; 0.6 \times W = 0.6 \times 20.9 = 12.5 \text{ psf walls crnr} \]
Calculating MWFRS Wind Loads

\[ a = \text{Lesser of:} \]

- \( \text{10\% least horizontal dimension (LHD)} = 34' \times 0.1 = 3.4' \)
- \( 0.4h = 0.4 \times 13' = 5.2' \)

But not less than:

- \( 0.04 \text{ LHD} = 1.4' \) or \( 3' \)

\[ \text{Use } a = 3.4' \text{ for zones 1E & 4E} \]

\[ 2a = 3.4' \times 2 = 6.8' \]
Section 28.4.2: \( P_p = q(GC_{pn}) \)

\[ GC_{pn} = 1.5 \text{ Windward parapet, } -1.0 \text{ Leeward parapet} \]

Windward Parapet \( GC_{pf} \) is 1.5: \( 20.14 \times 1.5 \times 0.6 = 18.12 \text{ psf} \)

Leeward Parapet \( GC_{pf} \) is 1.0: \( 20.14 \times 1.0 \times 0.6 = 12.08 \text{ psf} \)

Net Parapet = 18.12 + 12.08 = 30.2 psf
Retail Restaurant – Shearwall Design

\[ P = (8.3 \text{ psf} \times (5' + 3') + (30.2) \times 3') \times (84' / 2) + ((12.5 \text{ psf} - 8.3 \text{ psf}) \times (5' + 3')) \times 6.8' \times (77.2' / 84') = 6,804 \text{ lb} \]

(for comparison: Directional method gave us 6,325 lb)
Directional vs. Envelope

One specific instance when envelope loads can be higher than directional: Velocity Exposure Coefficient, $K_h$

Building $H < 30$ ft, Exposure B

Directional – Table 27.3-1

Envelope – Table 28.3-1
Questions?

This concludes The American Institute of Architects Continuing Education Systems Course

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Visit www.woodworks.org for more educational materials, case studies, design examples, a project gallery, and more
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