Considerations for Out-of-Plane Wall and Uplift Design
Overview

• Uplift
• Wall Design
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
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

These connections are not required for uplift but may be required to transfer shear loads.

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
Uplift Resistance: Direct Load Path

Important to detail uplift restraint connectors to provide direct load path
Uplift Wind Loads

Truss/Rafter to Top Plate Connection

What happens to the uplift load after this?
Uplift Wind Loads

- Truss or rafter at 2'-0" o.c.
- Rod
- Wide Spacing
- Stud 16" o.c.
- Roof sheathing not shown for clarity
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
Uplift: MWFRS or C&C?

Consider member part of MWFRS if:
• Tributary Area $> 700\text{ft}^2$ 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 = L²/3
Effective Wind Area Example

Trib. $A = (44)(2) = 88 \text{ ft}^2$

$EWA = \left(\frac{44^2}{3}\right) = 645 \text{ ft}^2$
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:

\[ L = 65 \text{ ft}, \quad h/L = 80/65 = 1.23 \]

\[ C_p = 1.3, \ -0.18 \]

---

#### ASCE 7-10 Fig. 27.4-1

<table>
<thead>
<tr>
<th>Wind Direction</th>
<th>Windward</th>
<th>Leeward</th>
<th>Angle, $\theta$ (degrees)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>$h/L$</td>
<td>10</td>
<td>15</td>
<td>20</td>
</tr>
<tr>
<td>Normal to ridge for $\theta \geq 10^\circ$</td>
<td>-0.7</td>
<td>-0.18</td>
<td>-0.5</td>
</tr>
<tr>
<td>Normal to ridge for $\theta \leq 10^\circ$ and Parallel to ridge for all $\theta$</td>
<td>-0.9</td>
<td>-0.18</td>
<td>-0.7</td>
</tr>
<tr>
<td>Normal to ridge for $\theta &lt; 10^\circ$ and Parallel to ridge for all $\theta$</td>
<td>-1.3**</td>
<td>-1.0</td>
<td>-1.0</td>
</tr>
</tbody>
</table>

*Value is provided for interpolation purposes.

**Value can be reduced linearly with area over which it is applicable as follows:

<table>
<thead>
<tr>
<th>Area (sq ft)</th>
<th>Reduction Factor</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\leq 100$ (9.3 sq m)</td>
<td>1.0</td>
</tr>
<tr>
<td>$250$ (23.2 sq m)</td>
<td>0.9</td>
</tr>
<tr>
<td>$\geq 1000$ (92.9 sq m)</td>
<td>0.8</td>
</tr>
</tbody>
</table>
MWFRS - Running the numbers

- \( GC_p : (0.85)(-1.3) = 1.105 \) (26.9.4 & Fig. 27.4-1)
- \( GC_{pi} : \pm 0.18 \) (Table 26.11-1)
- \( q_h = 0.00256K_zK_{zt}K_dV^2 \)
  - \( K_z : 0.93 \) – Table 27.3-1
  - \( K_{zt} : 1.00 \) - Figure 26.8-1
  - \( K_d : 0.85 \) - Table 26.6-1
  - \( V_u : 115 \text{ mph} \)
- \( q_h = 26.8 \text{ psf} \)
- \( p = (26.8 \text{ psf})(-1.105+(-0.18)) = 34.4 \text{ psf} \)
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
  - $p_{oh} = (26.8 \text{ psf})(-0.8)(0.85) = 18.2 \text{ psf}$
  - $p_{ext} = (26.8 \text{ psf})(-1.105) = 29.6 \text{ psf}$
  - $p_{oh \, net} = 18.2 + 29.6 = 47.8 \text{ psf}$

Per ASCE 7-10 section 27.4.4
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 = $0.6(95.6 \text{ plf}(2\text{ ft.}) + 68.8 \text{ plf}*20\text{ ft}/2) = 528 \text{ lbs}$
Dead Load = $0.6((2+20/2)*10\text{ psf}*2\text{ ft}) = 144 \text{ lbs}$

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

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 = \frac{H^2}{3} = \frac{22^2}{3} = 161\text{ft}^2

GC_p = -1.1 \text{ 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
• $p = (26.8 \text{ psf})(-1.1+(-0.18)) = 34.3$ psf
C&C - Roof Overhang per section 30.10

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

$EWA = 2*2 = 4 \text{ sf}$

$GC_p = -1.7$
C&C - Determining the Uplift Load

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

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

Net Uplift at Left Support = 615 lbs - 144 lbs = 471 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
Wind Uplift Requirements: Insurance

Some insurance companies require a building’s roof to be rated for wind uplift resistance.

Some designers and owners aren’t aware that there are wood assemblies which meet FM uplift requirements.

UL and Factory Mutual (FM) have done testing & research on roof uplift assemblies.

UL assigns ratings based on max. uplift resistance allowed for an assembly in psf, such as Class 30, 60, 90.

FM requires a factor of safety of 2, i.e. FM Uplift Rating of 90 required for roofs with 45 psf uplift forces.
Wind Uplift Requirements: Insurance

### TABLE 1

<table>
<thead>
<tr>
<th>Maximum Wind Velocity (mph, 3 second gust)</th>
<th>Roof Corner Uplift Design Pressure (psf)</th>
<th>Required FM Uplift Rating</th>
</tr>
</thead>
<tbody>
<tr>
<td>85</td>
<td>33</td>
<td>75</td>
</tr>
<tr>
<td>90</td>
<td>37</td>
<td>75</td>
</tr>
<tr>
<td>100</td>
<td>45</td>
<td>90</td>
</tr>
</tbody>
</table>

### FIGURE 1

**UL CLASS 90 (NM519)**

- Two-ply sheets (UL Type G1 asphalt glass fiber mat, 10 lb nominal) heat-mopped with surface flood coat
- 2' nominal Douglas-fir or southern pine framing spaced 24" o.c. maximum
- 16-ga. x 7/8"-long coated staples spaced 4" o.c. typ.
- 8d common deformed shank nails (0.131" x 2-1/2"), spaced 6" o.c. at panel ends and 12" o.c. at interior supports
- Base sheet (UL Type G2 asphalt glass fiber mat, 20 lb nominal)
- Plywood marked PS 1
- Plywood face grain direction

### FIGURE 5

**FM 75 WITH ARMA ROOF COVERING**

- Deformed-shank nails (0.135" x 2-1/8") spaced 6" o.c. at panel ends and edges and 12" o.c. at interior supports
- All panel edges supported
- Minimum 1-1/2" net thickness No. 2 Douglas-fir or southern pine framing, or equivalent
- 15/32" 5-ply plywood or OSB 32/16 RATED SHEATHING

(a) Design in accordance with local building code requirements for roof loads and anchorage. All framing must be minimum net thickness of 1-1/2 inches No. 2 Douglas-fir or southern pine or equivalent. For wood I-Joists, follow manufacturer's recommendations for minimum nail spacing.

(b) Panel strength axis across supports for direct-to-support spacing as shown. To install panels with strength axis parallel to supports spaced 24" o.c., as in panelized roof systems, see minimum panel requirements listed in Table 2.

Source: APA Form G310
Wind Uplift: Solar Panels

Resources:
DSA Form IR 16-8
SEAOC PV2
Overview

• Uplift
• Wall Design
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
Loads into WSP

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

SDPWS Table 3.2.1

<table>
<thead>
<tr>
<th>Sheathing Type (Sheathing Grades, C-C, C-D, C-C Plugged, OSB)</th>
<th>Span Rating or Grade</th>
<th>Minimum Thickness (in.)</th>
<th>Strength Axis</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>Perpendicular to Supports</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Maximum Stud Spacing (in.)</td>
</tr>
<tr>
<td></td>
<td>24/0</td>
<td>3/8</td>
<td>24</td>
</tr>
<tr>
<td></td>
<td>24/16</td>
<td>7/16</td>
<td>24</td>
</tr>
<tr>
<td></td>
<td>32/16</td>
<td>15/32</td>
<td>24</td>
</tr>
<tr>
<td></td>
<td>40/20</td>
<td>19/32</td>
<td>24</td>
</tr>
<tr>
<td></td>
<td>48/24</td>
<td>23/32</td>
<td>24</td>
</tr>
</tbody>
</table>

For ASD Capacity: Divide Nominal Capacity by 1.6
For LRFD Capacity: Multiply Nominal Capacity by 0.85
Determining Unbraced Length

- What is the unbraced length, $l_u$?
- Strong & weak axis
Gypsum & Weak Axis Buckling

NDS Commentary:
“Experience has shown that any code allowed thickness of gypsum board, hardwood plywood, or other interior finish adequately fastened directly to studs will provide adequate lateral support of the stud across its thickness irrespective of the type or thickness of exterior sheathing and/or finish used.”
Intermediate Wall Stud Blocking
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
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 1 for Stud Design

Strength Check as a Vertical Load Supporting element:
• Apply Vertical Dead, Live, Roof and/or Snow Loads
• Apply out-of-plane lateral loads
  • MWFRS wind loads (ASCE 7-10 Chapter 27 or 28)
  • Seismic wall forces (ASCE 7-10 12.11.1)
• Combined Bending & Axial Load Check per AWC NDS 3.9
• Use standard load combinations
  • IBC Section 1605 or
  • ASCE 7-10 Chapter 2

Design Tip: Bottom plate crushing may govern over Stud Capacity
Design Considerations

Slenderness Limits (NDS 2015 3.7.1.4)
Max Effective Unbraced Length = 50d, d = depth in inches
Max of 75d during construction

1½” depth
6’-3” max unbraced length.
9’-4” during construction.

3½” (2x4) Max Height: 14’-7”
5½” (2x6) Max Height: 22’-11”
7¼” (2x8) Max Height: 30’-2”

Stud or column can be braced against buckling in this direction by sheathing.

Stud or column is not braced against buckling in this direction by sheathing.
Strength Check 2 for Stud Design

Strength Check for Components & Cladding Wind Loads

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

Design Tip: Be aware of ASCE 7 Definition of Effective Wind Area to decrease the required C&C wind load

EFFECTIVE WIND AREA, $A$: The area used to determine ($GC_p$). For component and cladding elements, the effective wind area in Figs. 30.4-1 through 30.4-7, 30.5-1, 30.6-1, and 30.8-1 through 30.8-3 is the span length multiplied by an effective width that need not be less than one-third the span length. For cladding fasteners, the effective wind area shall not be greater than the area that is tributary to an individual fastener.
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 referenced from IBC 2015 allows application of Wall Stud Repetitive Factor to Stud STIFFNESS. See SDPWS 3.1.1

<table>
<thead>
<tr>
<th>Stud Size</th>
<th>System Factor</th>
</tr>
</thead>
<tbody>
<tr>
<td>2x4</td>
<td>1.50</td>
</tr>
<tr>
<td>2x6</td>
<td>1.35</td>
</tr>
<tr>
<td>2x8</td>
<td>1.25</td>
</tr>
<tr>
<td>2x10</td>
<td>1.20</td>
</tr>
<tr>
<td>2x12</td>
<td>1.15</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*

<table>
<thead>
<tr>
<th>CONSTRUCTION</th>
<th>$L$</th>
<th>$S$ or $W'$</th>
</tr>
</thead>
<tbody>
<tr>
<td>With plaster or stucco finishes</td>
<td>—</td>
<td>$l/360$</td>
</tr>
<tr>
<td>With other brittle finishes</td>
<td>—</td>
<td>$l/240$</td>
</tr>
<tr>
<td>With flexible finishes</td>
<td>—</td>
<td>$l/120$</td>
</tr>
</tbody>
</table>

*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 $V_{ULT}$ (ASCE 7-10) OR 0.70 when using $V_{ASD}$ (ASCE 7-05 and earlier) for deflection*
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 1604.3 DEFLECTION LIMITS

<table>
<thead>
<tr>
<th>CONSTRUCTION</th>
<th>( L )</th>
<th>( S ) or ( W )</th>
<th>( D + L )</th>
</tr>
</thead>
<tbody>
<tr>
<td>Roof members: e</td>
<td>//360</td>
<td>//360</td>
<td>//240</td>
</tr>
<tr>
<td>Supporting plaster or stucco ceiling</td>
<td>//240</td>
<td>//240</td>
<td>//180</td>
</tr>
<tr>
<td>Supporting nonplaster ceiling</td>
<td>//180</td>
<td>//180</td>
<td>//120</td>
</tr>
<tr>
<td>Not supporting ceiling</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Floor members</td>
<td>//360</td>
<td>–</td>
<td>//240</td>
</tr>
<tr>
<td>Exterior walls and interior partitions:</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>With plaster or stucco finishes</td>
<td>–</td>
<td>//360</td>
<td>–</td>
</tr>
<tr>
<td>With other brittle finishes</td>
<td>–</td>
<td>//240</td>
<td>–</td>
</tr>
<tr>
<td>With flexible finishes</td>
<td>–</td>
<td>//120</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 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

\[ W_{DL} = (11\text{ft})(20\text{psf} + 30\text{psf}) + (26\text{ft})(18\text{psf}) = 1018 \text{ plf} \]
\[ W_{RL} = (11\text{ft})(20\text{psf}) = 220 \text{ plf} \]
\[ W_{LL} = (11\text{ft})(65\text{psf}) = 715 \text{ plf} \]

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**

MWFRS – Part 2: Wind Loads – Walls

<table>
<thead>
<tr>
<th>V(mph)</th>
<th>110</th>
<th>115</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>0.5</td>
<td>1</td>
</tr>
<tr>
<td>h(ft.), L/B</td>
<td></td>
<td></td>
</tr>
<tr>
<td>30</td>
<td>28.5</td>
<td>28.5</td>
</tr>
<tr>
<td>20</td>
<td>26.9</td>
<td>26.9</td>
</tr>
<tr>
<td>15</td>
<td>25.2</td>
<td>25.2</td>
</tr>
</tbody>
</table>
Wall Stud Design: **Strength Check 1**

2x6 DF #2 Studs @ 16” o.c. OK for Strength Check 1

**Nominal Size:** (1) 2 x 6

**Species:** Douglas Fir-Larch

**Grade:** No.2

**Height (H):** 13 ft - 0 in

**Unbraced Length (I1):** 13 ft - 0 in

**Unbraced Length (I2):** 2 ft - 0 in

**P = 2310 lb = DL + FL**

**w = 22.8 plf = Wind**

**lu = 13 ft - 0 in**

---

**Set Duration Factors**

1. **C_D = 1.00 (P)**
2. **1.60 (P+w)**
3. **(1.3/2) fb (psi) = 497 < 1346 = Fb**
4. **(fc / Fc)^2 + fb / [F'b (1 - fc / Fce)] = 0.95 < 1.00 OK**

**Set Effective-Length Factor**

1. **K = 1.00**

**Set Deflection Limit**

1. **Δ / H = 120**

**Mid-H Deflection due to w, Δ (inch) = 0.85 < H / 120 OK**
Wall Stud Design: **Strength Check 2**

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 = \( h^2 / 3 \) = 56 ft\(^2\)

Zone 4:

\( GC_{pf} = -0.97 \)
\( GC_{pi} = -0.18 \) (Table 26.11-1)

Zone 5:

\( GC_{pf} = -1.1 \)

ASCE 7-10 Figure 30.4-1
Running the numbers – Zone 4

- $G_{cpf}: 0.97$ (Figure 30.4-1)
- $G_{pei}: 0.18$ (Table 26.11-1)
- $q_h = 0.00256 K_z K_{zt} K_d V^2$
  - $K_h: 0.98$ - 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

** Dimension Lumber **
Nominal Size : 2 x 6
Species = Douglas Fir-Larch

Sill Plate Nominal Size : 2 x 6
Species or Symbol = Douglas Fir-Larch

Grade = No.2

Bearings at < 3” of Sill End? No

Set Duration Factors
CD = 1.00 (P)

Set Effective-Length Factor
K = 1.00

Set Deflection Limit
Δ / H = 360

P = 1357 lb = DL + FL
w = 23.7 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) = 516 < 1346 = Fb

Mid-H Deflection due to w, Δ (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)
- $q_h = 0.00256K_zK_{zt}K_d V^2$
  - $K_h : 0.98$ - 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$(1.1+0.18) = 33$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

** 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

Height (H) = 13 ft - 0 in
Unbraced Length (I₁) = 13 ft - 0 in
Unbraced Length (I₂) = 2 ft - 0 in

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
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?
Wood Framed Stair/Elevator Shaft Walls
SPICE 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.

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

**Intermediate Stair Landing**

**When Stair Shaft Wall is Exterior Wall**
### Wall Framing at Shafts

How far can just wall plates span at shaft stud breaks? Requires no joints in these plates:

<table>
<thead>
<tr>
<th></th>
<th>8’ Tall Walls</th>
<th>10’ Tall Walls</th>
<th>12’ Tall Walls</th>
</tr>
</thead>
<tbody>
<tr>
<td>2-2x4</td>
<td>7’-10”B / 6’-4”D</td>
<td>7’-0”B / 5’-10”D</td>
<td>6’-4”B / 5’-6”D</td>
</tr>
<tr>
<td>3-2x4</td>
<td>10’-3”B / 7’-3”D</td>
<td>9’-2”B / 6’-8”D</td>
<td>8’-4”B / 6’-4”D</td>
</tr>
<tr>
<td>2-2x6</td>
<td>11’-5”B / 9’-11”D</td>
<td>10’-3”B / 9’-2”D</td>
<td>9’-4”B / 8’-8”D</td>
</tr>
<tr>
<td>3-2x6</td>
<td>15’-0”B / 11’-4”D</td>
<td>13’-5”B / 10’-6”D</td>
<td>12’-3”B / 9’-11”D</td>
</tr>
</tbody>
</table>

B – span controlled by bending  
D – span controlled by deflection

**Assumptions:**  
DF # 2  
L/360 Deflection Criteria  
18 psf C&C (bending) 12.6 psf C&C (deflection)
Wall Framing at Shafts

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
Wall Framing at Shafts

Floor Plan

CORRIDOR

OCCUPIED FLOOR

DN

RIM SPAN

RIM SPAN
Wall Framing at Shafts

*Connections are Key*
Exterior Wall Plate Elevations Shifting Down to Intermediate Landing Elevation

- Eliminates Hinge Effect
- Avoids Interference with Landing Windows

Intermediate Stair Landing

2x10 Ledger Fastened to Each Wall Stud with (3) 16d Nails

LU210 Face Mount Hanger

When Stair Shaft Wall is Exterior Wall
Questions?

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