EAST LYME
BUILDING DEPARTMENT

HIGH WIND WOOD FRAME CONSTRUCTION MANUAL

HANDOUT

(Add) **APPENDIX M – BASIC WIND SPEED**

Municipality	Basic Wind Speed (3 sec. gust)	Municipality	Basic Wind Speed (3 sec. gust)	Municipality	Basic Wind Speed (3 sec. gust)
Andover	100	East Lyme	110/120[1]	New Britain	90
Ansonia	100	Easton	100	New Canaan	100
Ashford	100	East Windsor	90	New Fairfield	90
Avon	90	Ellington	90	New Hartford	90
Barkhamsted	90	Enfield	90	New Haven	110
Beacon Falls	100	Essex	110	Newington	100
Berlin	100	Fairfield	100/110[2]	New London	120
Bethany	100	Farmington	90	New Milford	90
Bethel	90	Franklin	100	Newtown	90
Bethlehem	90	Glastonbury	100	Norfolk	90
Bloomfield	90	Goshen	90	North Branford	110
Bolton	100	Granby	90	North Canaan	90
Bozrah	110	Greenwich	100	North Haven	100/110[2]
Branford	110	Griswold	110	North Stonington	110
Bridgeport	110	Groton	120	Norwalk	100/110[3]
Bridgewater	90	Guilford	110	Norwich	110
Bristol	90	Haddam	110	Old Lyme	110/120[1]
Brookfield	90	Hamden	100/110[2]	Old Saybrook	110
Brooklyn	100	Hampton	100	Orange	110
Burlington	90	Hartford	90	Oxford	100
Canaan	90	Hartland	90	Plainfield	100
Canterbury	100	Harwinton	90	Plainville	90
Canton	90	Hebron	100	Plymouth	90
Chaplin	100	Kent	90	Pomfret	100
Cheshire	100	Killingly	100	Portland	100
Chester	110	Killingworth	110	Preston	110
Clinton	110	Lebanon	100	Prospect	100
Colchester	100	Ledyard	110	Putnam	100
Colebrook	90	Lisbon	110	Redding	100
Columbia	100	Litchfield	90	Ridgefield	90
Cornwall	90	Lyme	110	Rocky Hill	100
Coventry	100	Madison	110	Roxbury	90
Cromwell	100	Manchester	100	Salem	110
Danbury	90	Mansfield	100	Salisbury	90
Darien	100	Marlborough	100	Scotland	100
Deep River	110	Meriden	100	Seymour	100
Derby	100	Middlebury	90	Sharon	90
Durham	100	Middlefield	100	Shelton	100/110[4]
Eastford	100	Middletown	100	Sherman	90
East Granby	90	Milford	110	Simsbury	90
East Haddam	110	Monroe	100	Somers	90
East Hampton	100	Montville	110	Southbury	90
East Hartford	90	Morris	90	Southington	100
East Haven	110	Naugatuck	100	South Windsor	90

Municipality	Basic Wind Speed (3 sec. gust)
Sprague	100
Stafford	90
Stamford	100
Sterling	100
Stonington	110/120[5]
Stratford	110
Suffield	90
Thomaston	90
Thompson	100
Tolland	100
Torrington	90
Trumbull	100/110[2]
Union	90
Vernon	100
Voluntown	110
Wallingford	100
Warren	90
Washington	90
Waterbury	90
Waterford	110/120[1]
Watertown	90
Westbrook	110
West Hartford	90
West Haven	110
Weston	100
Westport	100/110[2]
Wethersfield	100
Willington	100
Wilton	100
Winchester	90
Windham	100
Windsor	90
Windsor Locks	90
Wolcott	90
Woodbridge	100
Woodbury	90
Woodstock	100

Footnotes:
1. Areas south of I-95 = 120 mph; areas north of I-95 = 110 mph
2. Areas south of Rt. 15 = 110 mph; areas north of Rt. 15 = 100 mph
3. Areas south of I-95 = 110 mph; areas north of I-95 = 100 mph
4. Areas east of Rt. 8 = 110 mph; areas west of Rt. 8 = 100 mph
5. Areas south of Rt. 184 = 120 mph; areas north of Rt. 184 = 110 mph

HANDOUT EXPLAINING THE HIGH WIND (110, 115 & 120 MPH) CONNECTION AND SHEATHING REQUIREMENTS OF THE WOOD FRAME CONSTRUCTION MANUAL-2001

Disclaimer: The purpose of this handout is to attempt to explain the various connections and sheathing requirements in the WFCM-2001, one of the design criteria referenced by Section R301.2.1.1 of the 2003 International Residential Code for use in municipalities where the wind speed as designated in Appendix M equals or exceeds 110 mph. (Note that when using the WFCM for towns in the 115 mph wind zone, follow the requirements for 110 mph.) This handout is not meant to take the place of the referenced standard and must be used in conjunction with the standard. When discrepancies exist between this handout and the standard or the code, the language of the standard or code prevails. When following the prescriptive design provisions of Chapter 3 in the standard it is not necessary to engage the services of a design professional as long as the building falls within the prescriptive design limitations of Section 3.1.3, although that is always a code-compliant alternative.

Note that connections must be provided to resist lateral **(L)**, shear **(S)**, uplift **(U)** and overturning **(O)** forces. In addition the standard requires several special **(SP)** connections. Where a specific connection is required to resist more than one type of force, the connection selected must be capable of resisting the combined loads. See Handout Drawings 1 through 5 at the back of this package for a description of the case study as well as a graphic representation of the locations of the various connections and evaluations that the standard requires.

Design parameters: 110 mph wind speed; Exposure B (for exposure C, follow same method but use Appendix A tables); 32' roof span; 10/12 roof pitch; 2 story on basement (note that even though this is considered a two-story building by the IRC, for the purposes of the WFCM it is considered a three-story since the attic is under a roof pitch that exceeds 6:12 – see F. 3.1a); 8' wall heights at house, 9' at garage; Hem-Fir #2 framing (G = 0.43) @ 16" O.C.; building is fully sheathed with 7/16" wood structural panels.

Item 1 – Roof assembly connections (L & S), 3.2.1.1: Roof framing connections must comply with T. 3.1 nailing schedule which tells you the number and size of nails used to connect the various elements of the roof assembly.

Item 2 – Ridge connection (SP), 3.2.5.1: Ridge straps or collar ties are required. Choice: either provide ridge straps that will resist forces per T. 3.6 or the prescriptive solutions of T. 3.6A as modified by Footnote 4 for ridge strap spacing **or** provide 1x6 or 2x4 collar ties in the upper third of the rafter attached per Footnote 6 to T. 3.6A.
Example: For ridge straps, go into T. 3.6 with 32' roof span, 110 mph and 10/12 pitch, select connector that will resist 259 lbs. x spacing of straps: for strap at each 16" O.C. rafter: 259 x 1.33 = 344.5 lbs. For straps @ 24" O.C.: 259 x 2.00 = 518 lbs. **or** use T. 3.6A for prescriptive solution: use 1 ¼ inch strap with number of nails calculated per Footnote 4. For straps 16" O.C. use (3 x 1.33 = 4) 4 - 8d common or 10d box nails in each end. For straps @ 24" O.C. use (3 x 2.00 = 6) 6 - 8d common or 10d box nails in each end.
Alternative: Use 1x6 or 2x4 collar ties with the same number of 10d common or 12d box nails in each end determined above based on the spacing of the collar ties (see Footnote 6).

Item 3 – Rafter to ceiling joist connection (L & S), 3.2.1.1: This is actually one of the connection evaluations required by Item 1. Rafters and ceiling joists must be connected to each other in accordance with T.3.9 **or** the prescriptive method of T. 3.9A.
Example: Find prescriptive solution from T. 3.9A: Enter Table with 30 psf Ground Snow Load, 16" O.C. framing, 10/12 pitch and 32' Roof Span (for simplicity use 9/12 and 36', although interpolation is allowed). Use 5 - 16d common or 40d box nails per heel joint connection. Note that this table assumes that the ceiling joist is at the plate – if raised off the plate the number of nails must be increased per Footnote 6.

Item 4 – Roof assembly to wall assembly, 3.2.1.2 and 3.2.2.1 (L, S & U): Applies to rafter and ceiling joist assembly and to roof trusses (for trusses, the truss drawing may show the uplift, lateral & shear loads to be resisted). Roof assemblies must be connected to the top plate for shear & lateral loads and to the studs below for uplift per T. 3.4 or the corresponding prescriptive tables (if the roof assembly is attached only to the top plate for uplift, then the top plate must be attached to the studs below per T. 3.4 or T. 3.4B). Lateral loads are loads acting perpendicular to the wall; shear loads are loads acting parallel to the wall caused by the wind blowing against the wall perpendicular to the wall being evaluated; uplift loads are loads trying to lift the roof assembly off the wall. Note that for this item these loads do not apply to gable end walls, only to walls that support the roof assembly. See Figure 3.7a for required ceiling bracing of gable end walls to resist lateral and shear loads.
Example: Per T. 3.4 select a proprietary connector that will resist a lateral load (perpendicular to the wall), a shear load (parallel to the wall) and an uplift load on the walls upon which the roof assembly bears:
LATERAL: Enter T. 3.4 with 110 mph, 16" O.C. and 32' roof; req'd. L value is 176 lbs.
SHEAR: When the wind blows parallel to the ridge against the 32' gable end wall it is the 44' wall that resists it. Enter T. 3.4 with same criteria as lateral – req'd. S value = 78R where R for wind parallel to the ridge = Width/Length so S=78 x 32/44 = 56.73
UPLIFT: Enter T. 3.4 with same criteria as lateral – req'd. U value = 336
Thus, you must select a connector that will resist an uplift load of 336 lbs., a lateral load of 176 lbs. and a shear load of 56.73 lbs. Be sure to evaluate the connectors chosen for simultaneous loads. Again, for the gable wall see F. 3.7a.

Or, use alternative prescriptive method............................
Example: LATERAL & SHEAR: enter T 3.4A with 110 mph, 8' wall height and 16" O.C. framing: use 3 - 8d common or 10d box toenails to connect rafter/clg. jst. or truss to top plate (max. 2 nails per side for 2x4 plate, 3 per side for 2x6 plate) **and...............**
UPLIFT: enter T. 3.4B with 110 mph, 16" O.C. framing and 32' roof span – select 1 ¼" x 20 gage strap with 3 - 8d common or 10d box nails each end. Remember, these straps must attach to the studs below. Note: per Footnote 3, increase by 1 nail if there is no ceiling or if the ceiling is not connected to the rafters.
Use these connections on the 44' wall upon which the roof assembly bears – See F. 3.7a for gable end walls

Item 5 – Jack rafters (if applicable), 3.2.5.2 (SP): Connect jack rafters to wall assembly per 3.2.2.1 (same as Item 4) and to hip per T. 3.6 or T. 3.6A (same as Item 2).

Item 6 – Rake (gable end) overhang, 3.2.5.3 (SP): See T. 3.4C for gable end overhangs for rafter/clg. jst. construction and F. 2.1h for prescriptive requirements for truss roof gable end overhangs.
Example for rafter/clg. jst.: Enter T. 3.4C with 110 mph and outlookers spaced 16" O.C. Each outlooker must be connected to the studs below with a connection capable of resisting 417 lbs. uplift load. This uplift load resistance must be continued through each wall to wall connection and through the wall to foundation connection. Note that gable walls only need be designed to resist the uplift from the gable overhang. If there is no gable overhang, there is no uplift on the gable wall.

Item 7 – Top & bottom plate to wall studs, 3.2.1.3 (L, see Item 4 for U): See T. 3.5 or the prescriptive solutions of T. 3.5A for connections of top and bottom plates to wall studs.
Example: Using T. 3.5, enter table with 110 mph and 8' wall height. Find value of 111 lbs./ft. See Footnote 3: If you will install a connector at each stud (16" O.C.), multiply 111 x 1.33 = 147.63, pick connector for top & bottom of each stud to resist 148 lb. lateral load. If you install connectors every 4', multiply 111 x 4 = 444, pick connector for 4' O.C. to resist 444 lbs.

Or, use alternative prescriptive method...................
Example: Enter T. 3.5A with 110 mph, 16" O.C. and 8' wall height. Provide 2 - 16d common or 40 d box nail per stud for plate to stud connection (normal construction practice should take care of this). Note that you must also consult T. 3.1 for other wall nailing requirements.

Item 8 – Wall assembly to wall assembly, 3.2.2.2 (U): Upper story walls must be connected to lower story walls to resist uplift in accordance with T. 3.4 or the prescriptive solutions of T. 3.4B (See Figure 2.2c). When upper story studs and lower story studs do not line up, they must each be connected to the box or band joist in accordance with T. 3.4.
Example: Note that we already evaluated this in Item 4 for the roof to wall uplift connection and came up with 336 lbs. of uplift. In accordance with Footnote 4 to T. 3.4 we can reduce this value by 60 plf for each full wall above. Since we have one full wall above this connection and since our framing (and thus our connection) is 16" O.C., our reduction is 60 x 1.33 = 80 lbs, so each connector must be sized for 336 – 80 = 256 lbs. of uplift.

Or use alternative prescriptive method......................
Example: Enter T. 3.4B with 110 mph, 16" O.C. framing and 32' roof span: select a 1 ¼" x 20 gage strap with 3 - 8d common or 10d box nails in each end.

Or, For story to story wall assembly connections, a **third option** is possible that requires no connectors or straps. See Drawing #6 at the back of this handout. If you run your 48 inch wide wood structural panel sheathing horizontally, centered on the floor assembly, block all panel edges and nail the sheathing 6" OC, you will end up with 4 - 8d common nails in each wall above and below the floor. Per T. 6A, for Hem-fir with G=0.43 an 8d common nail through ½ inch material (the sheathing is actually 7/16) into the stud has a shear capacity of 98 lbs. 98 lbs. x 4 nails = 392 lbs. which is OK to resist the required 256 lbs. of uplift. Obviously if you install the sheathing vertically and position it to get a minimum of 4 - 8d common nails in each wall above and below the floor, that works too.

Item 9 – Wall bottom plate to floor assembly, 3.2.1.4 (L & S): Per T. 3.1, 2 - 16d common or box nails are required per foot through the bottom plate into the floor joists or band joist (box). Also see Footnotes 1 & 2 for applicable increases or decreases.

Item 10 – Floor framing component nailing, 3.2.1.5 (L & S): See T. 3.1 for nailing schedule of floor framing components.

Item 11 – Floor assembly to wall assembly or sill plate, 3.2.1.6 (L & S): See T. 3.1 for nailing schedule for lateral and shear connections for floor joists to sill, top plate or girder (4 - 8d common or 10 d box nails per joist); band joist (box) to joist (3 - 16d common or 4 - 16d box nails per joist); and band joist (box) to sill or top plate (2 - 16d common or 3 - 16d box nails per foot). See F. 3.7b for required floor bracing when the floor joists run parallel to the gable wall, and Section 3.3.5 for required blocking.

Item 12 – Wall assembly to foundation, 3.2.2.3 (U): First floor wall studs shall be connected to the foundation, sill plate or bottom plate in accordance with T. 3.2 or the prescriptive requirements of T. 3.4B to resist uplift (see Figure 3.2 a-c).
Example: Enter T. 3.2 with 110 mph, 3 stories and 32' roof span, using the sill plate to foundation requirements since our example has a basement. The connector chosen must resist 72 pounds per lineal foot of uplift within 8' of the corners and, in accordance with Footnote 1, 72 x 0.75 = 54 plf where not within 8' of corners. Since this load is in pounds per linear foot, you must multiply the load by the appropriate factor based on spacing of the connectors (1.33 for 16" O.C., 2.00 for 24" O.C., etc.), so if I put a connector at each stud 16" O.C., that connector must resist 72 x 1.33 = 96 lbs. of uplift within 8' of the corners and 54 x 1.33 = 72 lbs. of uplift elsewhere. Note that this load is lower than the uplift connections required at the roof (Item 4) and the second floor to the first floor (Item 8), presumably due to the cumulative weight of the building resisting uplift.

Or, use alternative prescriptive method............................
Example: Enter T. 3.4B with 110 mph, 16" OC framing and 32' roof span. Use 1 ¼" x 20 gage strap with 3 - 8d common or 10d box nails in each end (standard offers no explanation for why the prescriptive solution doesn't take weight of building into account). If this path is chosen see 3.2.2.3 for options regarding embedding the strap in the foundation or lapping the strap under the sill and increasing the number of anchor bolts (see explanation at Item 13 alternative prescriptive method below).

Note that the requirements of Item 13 must also be met for connection Item 12.........

Item 13 – Wall assembly or sill plate to foundation, 3.2.1.7 (L & S): Sill plates (bsmt. or crawl) or wall bottom plates (slab on grade) shall be anchored to the foundation to resist lateral and shear loads from wind in accordance with T. 3.2 or the prescriptive solutions of T. 3.2A for sill plate to foundation or T. 3.2B for bottom plate to foundation. Section 3.2.1.7 also includes minimum anchor bolt requirements.
Example: Enter T. 3.2 with 110 mph, 3 stories and 32' roof span, using the sill plate to foundation requirements since our example has a basement. We see that the foundation requirements of the 2003 IRC will take care of the lateral loads (not true for wall bottom plates when using slab-on-grade construction) but we must select a connector for shear load. Again, from T. 3.2, working from the Sill Plate to Foundation portion of the table since our

example has a basement, we see that the shear load to be resisted is 345R, where R= L/W for the 32' wall (wind perpendicular to the ridge) and R=W/L for the 44' wall (wind parallel to the ridge). Thus,
for the 32' wall – 345 x L/W = 345 x 44/32 = 474 plf, and
for the 44' wall – 345 x W/L = 345 x 32/44 = 251 plf,
so the 32' wall needs shear connectors capable of resisting 474 plf in addition to the uplift resistance required by Item 12, and the 44' wall needs shear connectors capable of resisting 251 plf in addition to the uplift resistance required by Item 12. Keep in mind that the number of anchor bolts can never be less than what is required by Section 3.2.1.7.

Or, use alternative prescriptive method ....
Example: Enter T. 3.2A with 110 mph, 3 story. For the number of anchor bolts in the 32' wall, you enter the table with the 44' dimension, since that is the dimension of the building that is perpendicular to the 32' wall and is "catching" the wind that produces the shear. A 40' wall requires 14 - ½" bolts or 10 - 5/8" bolts and a 50' wall requires 17 - ½" bolts or 12 - 5/8" bolts, so after interpolation we see that for a 44' wall we use either 15 - ½" bolts or 11 - 5/8" bolts. Remember that the bolts must also meet the prescriptive requirements of Section 3.2.1.7. For the number of anchor bolts in the 44' wall you enter the table with the 32' dimension; so the 44' wall must have either 11 - ½" bolts or 8 - 5/8" bolts, and also must meet the prescriptive requirements of Section 3.2.1.7. Remember you also have to resist the uplift of Item 12 – the anchor bolts that resist shear can also be used to resist uplift as long as a 3" square washer is used at each bolt, the anchor bolt spacing meets the maximums allowed by T. 3.2C (39" O.C. within 8' of corners and 45" O.C. elsewhere) and the bottom plate is attached to the rest of the building in accordance with Item 12.

Item 14a – Header (or girder) to stud at wall openings (L&U), 3.2.5.4.1 – Header to stud connections shall be in accordance with T. 3.7. Uplift is based on roof span as well as the width of the opening but lateral is a function of the size of the opening only. Enter the table with 110 mph, a header span of 3' (midway between the tabular values of 2' and 4', so multiply the 2' values by 1.5) and a 32' roof span (which is 33% greater than the tabular values for 24', so multiply those values by 1.33). Note that the tabular values are for framing within 8' of the building corners. Tabular values for areas not within 8' of building corners may be reduced by multiplying by 0.75 for uplift and 0.92 for lateral.
UPLIFT: 202 x 1.5 = 303 x 1.33 = 403 lbs. within 8' of corners, 403 x 0.75 = 302 elsewhere
LATERAL: 132 x 1.5 = 198 x 1.33 = 263 lbs. within 8' of corners, 263 x 0.92 = 242 elsewhere
So, you need to select a connector that will resist both 403 lbs. of uplift and 263 lbs. of lateral force within 8' of the corners and 302 lbs. of uplift and 242 lbs. of lateral force elsewhere. Unfortunately the code offers no specific prescriptive solution, but you can go to T. 7A for common nails or T. 7B for box nails and investigate the appropriate combination of toe nails through the header into the stud to resist lateral forces (for 16d common @ 59 lbs/nail = 5 at each end) and nails through the plywood into the header and adjacent framing (must be a single piece of plywood to make this connection) to resist uplift (for 8d common through ½ inch material @ 66 lbs/nail = 6 at each end). These nailing amounts are for within 8' of corners – take appropriate reductions elsewhere. Note that conventional framing practices will take care of the connections at most of the smaller openings in the wall.

Item 14b – Window sill plate to stud (L), 3.2.5.4.1 – Window sill plate to stud connection shall be in accordance with T. 3.8. Enter table with 110 mph and select value for 3' opening that is 1.5 times the 2' tabular value: 132 x 1.5 = 198 lbs. within 8' of the corner and 198 x 0.92 = 182 lbs. elsewhere. Again, either select a proprietary connector resisting this lateral force or go to T. 7A or T. 7B and select the appropriate number of toenails (for 16d common @ 59 lbs/nail = 198/59 = 4 each end when within 8' of corners which will probably split a single sill but would be OK when a double sill is utilized and the two sills are securely fastened to each other). Again, reduce the load and thus the number of nails required when not within 8' of corners (198 x 0.92 = 182/59 = 3 nails each end).

Item 15 – Top and bottom plates to full height studs alongside window openings (SP), 3.2.5.4.2 – The first step in this evaluation is to select the number of full height studs req'd on each side of the opening in accordance with T. 3.23C. **Example:** 16" O.C. wall stud spacing, 3' header span = 1.5, so use 2 full height studs each side of the opening. (note: reductions possible per T. 3.23D).
The second step is to determine loads that a connector must resist per T. 3.5 **or** to determine the prescriptive solution per T. 3.5A. **Example:** Enter T. 3.5 with 110 mph and 8' wall height. Lateral force is 111 lbs. of lateral force to be resisted at full height studs on either side of opening within 8' of the corner. Per Footnote 1, we can reduce this by multiplying by 0.92 for openings not within 8' of the corner = 111 x 0.92 = 102 lbs. So, we can either choose a proprietary connector for each side of the opening that will resist 111 lbs. of lateral force within 8 feet of the corner and one that will resist 102 lbs. of lateral force elsewhere **or** we can follow the prescriptive solutions of T. 3.5A: **Example:** Enter T. 3.5A with 110 mph, 16" O.C. framing and 8' wall height – each of the two full height studs on either side of the opening must be connected to the top and bottom plates of the wall with a minimum of 2 - 16d common or 40d box nail – once again, normal framing practices will fulfill this requirement – the only difference is the addition of another full height stud alongside the opening.

Note: prior to evaluating Item 16, you might want to review Item 20 to get a feel for segmented shearwalls and perforated shearwalls.

Item 16 – Hold-downs (O), 3.4.4.2.3 – Hold-downs are required in shearwall segments and in perforated shearwalls to resist the overturning force of the wind. In our example we need them to connect the second floor walls to the first floor walls and the first floor walls to the foundation. You need one at each end of a segmented shearwall segment **or** one at each end of each perforated shearwall. When shearwalls or shearwall segments meet at a corner and the corner framing is fastened together to resist uplift (see F. 3.8 a-b), a single hold-down at the corner is sufficient. Hold-down capacity is determined in accordance with T. 3.17F based on wall height only. Per Footnote 2 to T. 3.17F, required hold-down capacities must be summed from stories above.
Example: Enter T. 3.17F with 8' high wall. We see that the second story needs a hold-down for wind that will resist 3488 lbs. The first story needs a hold-down that will resist its own 3488 lbs. as well as the 3488 lbs. from the second story = 3488 + 3488 = 6976 lbs. Remember that you will need a hold-down at each story at the end of each shearwall segment for Type I walls or for each perforated shearwall for Type II walls (see Item 20).

Item 17 – Roof sheathing nailing, 3.2.4.1 – Roof sheathing must be nailed in accordance with T. 3.10 using 8d common or 10d box nails.
Example: Enter T. 3.10 with 110 mph and 16" O.C. framing. We see that in both the interior zone and the perimeter edge zone, we need to nail the sheathing 6" O.C. at the panel edges

and 12" O.C. in the field. However, since our framing lumber, Hem-fir, has a G of 0.43, according to Footnote 2, we must nail the field of panels in the perimeter edge zone (within 4' of all edges of the roof and within 4' of the roof peak) at 6" O.C. Roof sheathing material must comply with T. 3.12A (min 3/8" thick, which also meets IRC req.).

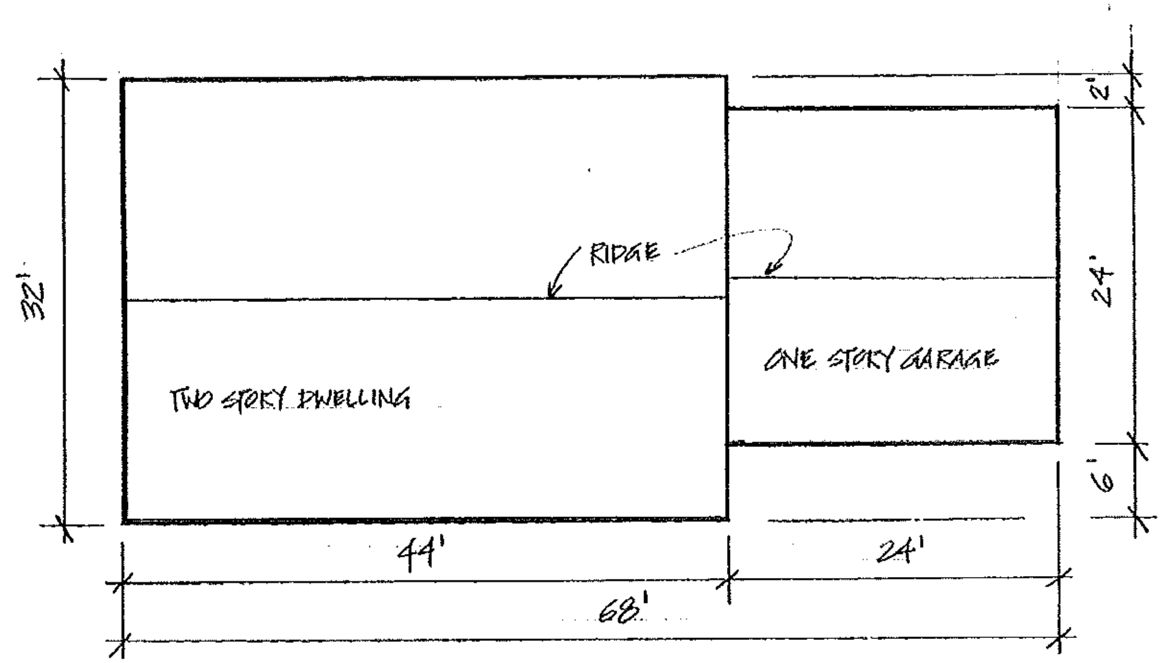
Item 18 – Wall sheathing nailing, 3.2.4.2 – Wall sheathing must be nailed in accordance with T. 3.11 using 8d common or 10d box nails.
Example: Enter T. 3.11 with 110 mph and 16" O.C. framing. We see that in both the interior and 4 foot edge zone, wall sheathing must be nailed 6" O.C. at panel edges and 12" O.C. in the field – again, normal construction practice for those using 8d common or 10d box nails.

Item 19 – Floor sheathing nailing, 3.2.4.3 – Floor sheathing shall be attached with min. 8d common nails 6" O.C. at panel edges and 12" O.C. in the field – again, normal construction practice (watch size of nails).

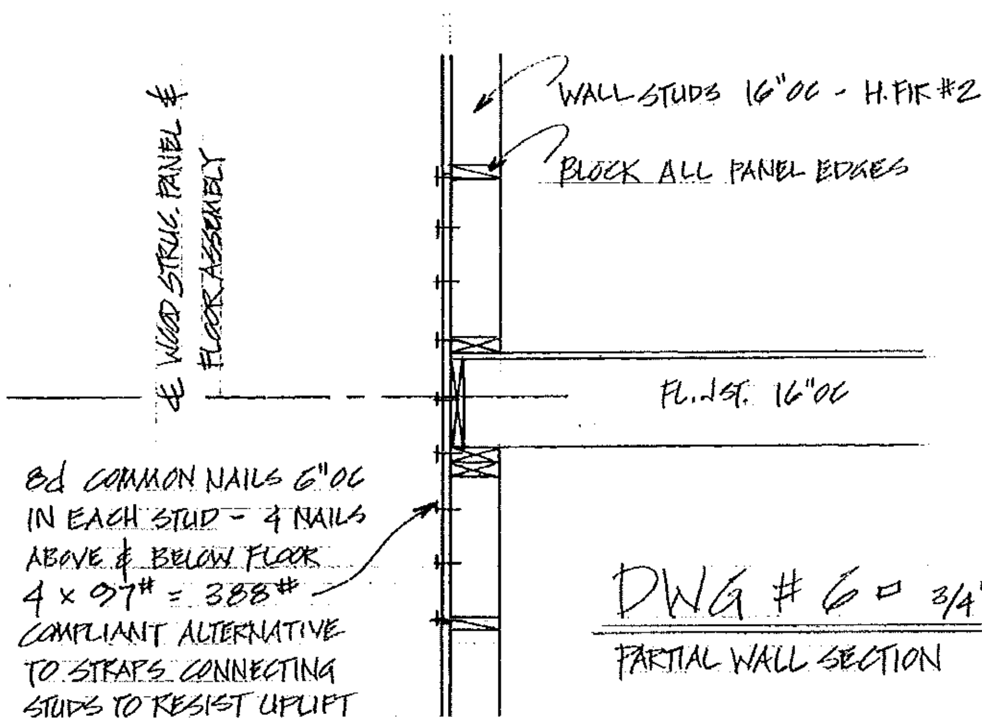
Item 20 – Exterior shearwalls, 3.4.4.2 - You have a choice of designing with segmented shearwalls (each solidly sheathed piece of wall between openings that meets the aspect ratios of T. 3.17D can be considered a shearwall segment, but each segment must have its own hold-downs), **or** designing with a perforated shearwall (treat the entire wall as a unit and provide hold-downs at each end of the wall – you get credit for each fully sheathed portion of the wall between openings that does not exceed the maximum aspect ratio). The aspect ratio is the ratio of the fully sheathed (no openings) portion of the wall height to its width. For our example which has 7/16" plywood sheathing outside and ½" gyp. bd. inside, the maximum allowable aspect ratio is 3 ½:1. For an 8' high wall that is 5' wide, the aspect ratio is 8:5, which is 1.6:1, considerably lower than the maximum allowed. See Handout Drawing #5 at the back of this package for additional information for our examples. Note that all exterior walls must be evaluated, but in reality in most cases if you evaluate the "worst case" wall and it passes, you should be OK. Remember that due to the roof pitch our example is considered a three-story building, so you have to drop down a section in each table.
SEGMENTED SHEAR WALLS (TYPE I), 3.4.4.2a: Segmented shearwalls must be in accordance with T. 3.17A and T. 3.17B. For our example we will assume 7/16" wood structural panels fastened in accordance with Item 18 as exterior sheathing and ½" gyp. bd. on the interior, fastened with 5d cooler nails 7" O.C. at panel edges and 10" O.C. in the field. Required lengths of shearwall segments must be multiplied by factors in T. 3.17D when other sheathing or nailing patterns are used. When evaluating shearwalls each story of each wall must be evaluated individually. Each wall is evaluated based on the length of the wall perpendicular to it, since that is the wall "catching" the wind causing the shear.
Segmented Example – Wall A: Since Wall A is a gable wall we use T. 3.17A. Enter table with 110 mph and building sidewall length of 44' (interpolate values between 40' and 50').
Wall A-1 (wall beneath roof, ceiling & 1 floor): Min. length shearwall segment required is approx. 16.5'. Since we have 17 feet between windows, that is our shearwall segment and we need hold-downs at each end of that segment.
Wall A-2 (wall beneath roof, ceiling and 2 floors): Min. length shearwall segment required is approx. 26'. We have a 17' segment between the windows and two 4'-6" segments at each corner for a total of 26'. OK if we put hold-downs at each of the three segments.
Segmented Example – Wall B: Since Wall B is parallel to the ridge (supports the roof), we use T. 3.17B. We enter the table with 110 mph and building endwall length of 32' (again, the 44' long wall is resisting the shear force caused by the wind blowing on the 32' wall).

Wall B-1 (wall beneath roof , ceiling & 1 floor): Min. length shearwall segment is 8.9'. Since we have segments of 5' between openings we need 2 segments and each segment must have its own hold-downs.
Wall B-2 (wall beneath roof, ceiling and 2 floors): Min. length shearwall segment is 13.5', so we would need three 5' segments, each with its own hold-downs (keeping in mind that two of the segments must be below the segments in Wall B-1 so the second floor hold-downs are continuous to the foundation).
***Or use.....*PERFORATED SHEARWALLS (TYPE II), 3.4.4.2.2:** You multiply the segmented shearwall length requirements found as above by the appropriate full-height sheathing length adjustment factors in T. 3.17E. For this table you need wall height, height of tallest opening and percent of full height sheathing (without openings) on the wall.
Perforated Example – Wall A:
Wall A-1: 8' wall, maximum opening height of 5'-2", total wall length = 32', length of full height sheathing = 26'. Percent of full height sheathing = 26/32 = 81%. Multiply the Type I (segmented) segment length by the Type II Length Increase Factor to find the required length of full height in the Type II (perforated) wall based on the maximum opening height in that wall: 16.5' x 1.11 = 18.3' req'd. OK since we have a total of 26 feet of fully sheathed wall without openings. Need hold-downs at each end of the wall.
Wall A-2: Same process is used since all of the same dimensions apply in our example: 26 x 1.11 = 28.9'. Since we only have 26 feet, we need to eliminate a window to increase the length of fully sheathed wall to 29' and we need hold-downs at each end of the wall.
Perforated Example – Wall B:
Wall B-1: 8' wall, maximum opening height of 5'-2", total wall length = 44', length of full height sheathing = 29' = 65% (29/44). Required length of full height sheathing = 8.9' x 1.22 (midway between 1.25 for 60% and 1.18 for 70%) = 10.8'. OK since we have 29'.
Wall B-2: 8' wall, maximum opening height of 6'-8", percent full height sheathing same as Wall B-1 = 65% Required length of full height sheathing = 13.5 x 1.27 (again, midway between 60% and 70% values) = 17.1'. Again OK since we have 29'.
So.............. All walls evaluated in our example may be treated as segmented shearwalls **or** perforated shearwalls. But, if we choose segmented shear walls we need multiple sets of hold-downs in Walls A-2, B-1 and B-2, whereas if we choose perforated shearwalls we only need one set of hold-downs in each wall, but we must eliminate a window in wall A-2. My choice would be perforated shearwalls since it reduces the number of required hold-downs, especially if I nail the corners properly so I only need one hold-down at each corner. It generally makes sense to treat all walls fully sheathed with wood structural panels as a perforated shear wall. The capacity of the hold-downs has already been evaluated at Item 16.

Item 21 – Short walls on sides of large openings: Walls adjacent to garage door openings, represent a special problem since the aspect ration (ratio of height of wall to width of wall) will generally be greater than that allowed by T. 3.17D. Therefore, the design must be altered to provide wider walls on each side of the garage door opening **or** you must use proprietary prefabricated shearwall panels installed in accordance with manufacturer's installation instructions on both sides of the garage door. Note that such panels may also be used in other walls where large openings and narrow segments of full-height sheathing are desired.
Finally, THE END! Remember, this handout is not the code or the standard; it is simply one man's attempt to make sense of it all. Good Luck..............................

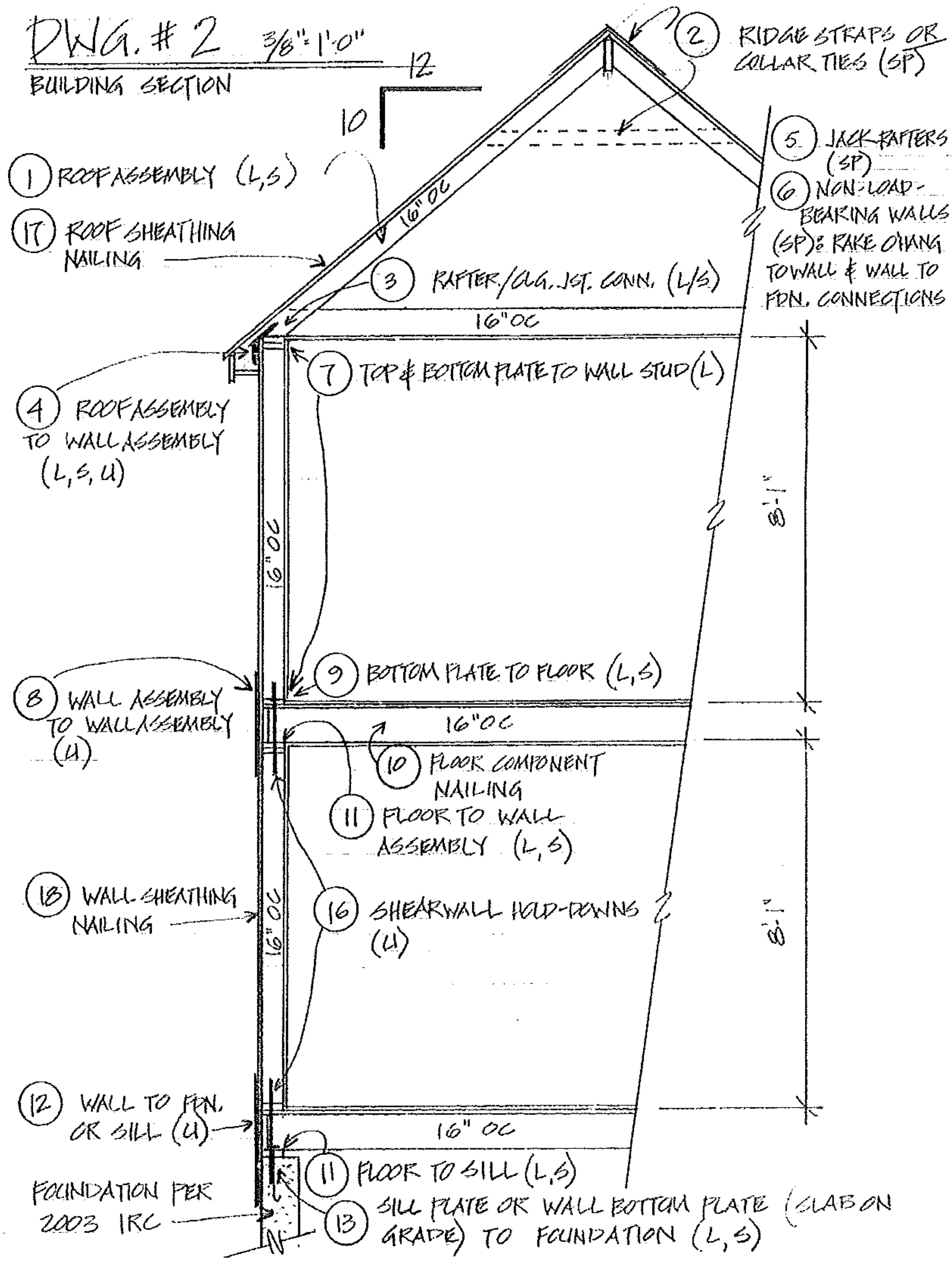


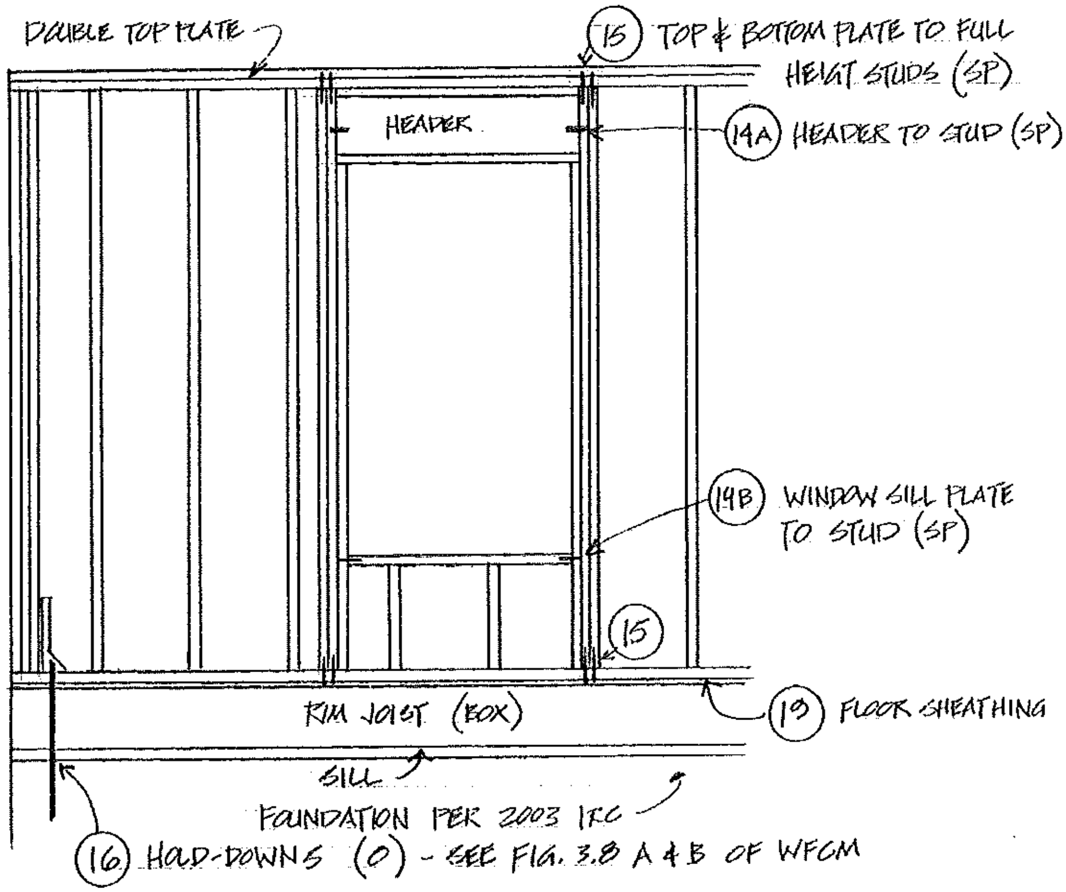
DRAWING #1 □ ROOF PLAN 3/32 = 1'-0"



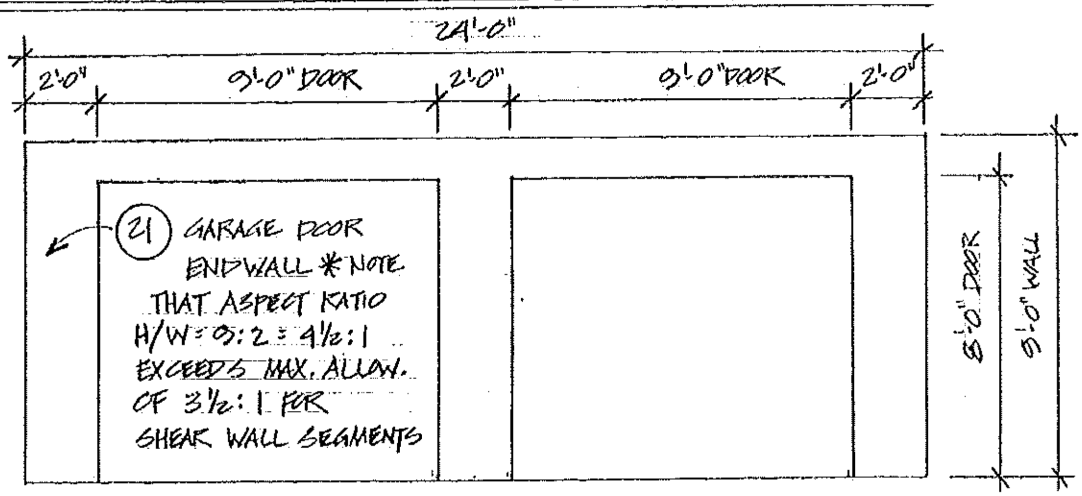
DWG # 6 □ 3/4" = 1'-0"

PARTIAL WALL SECTION

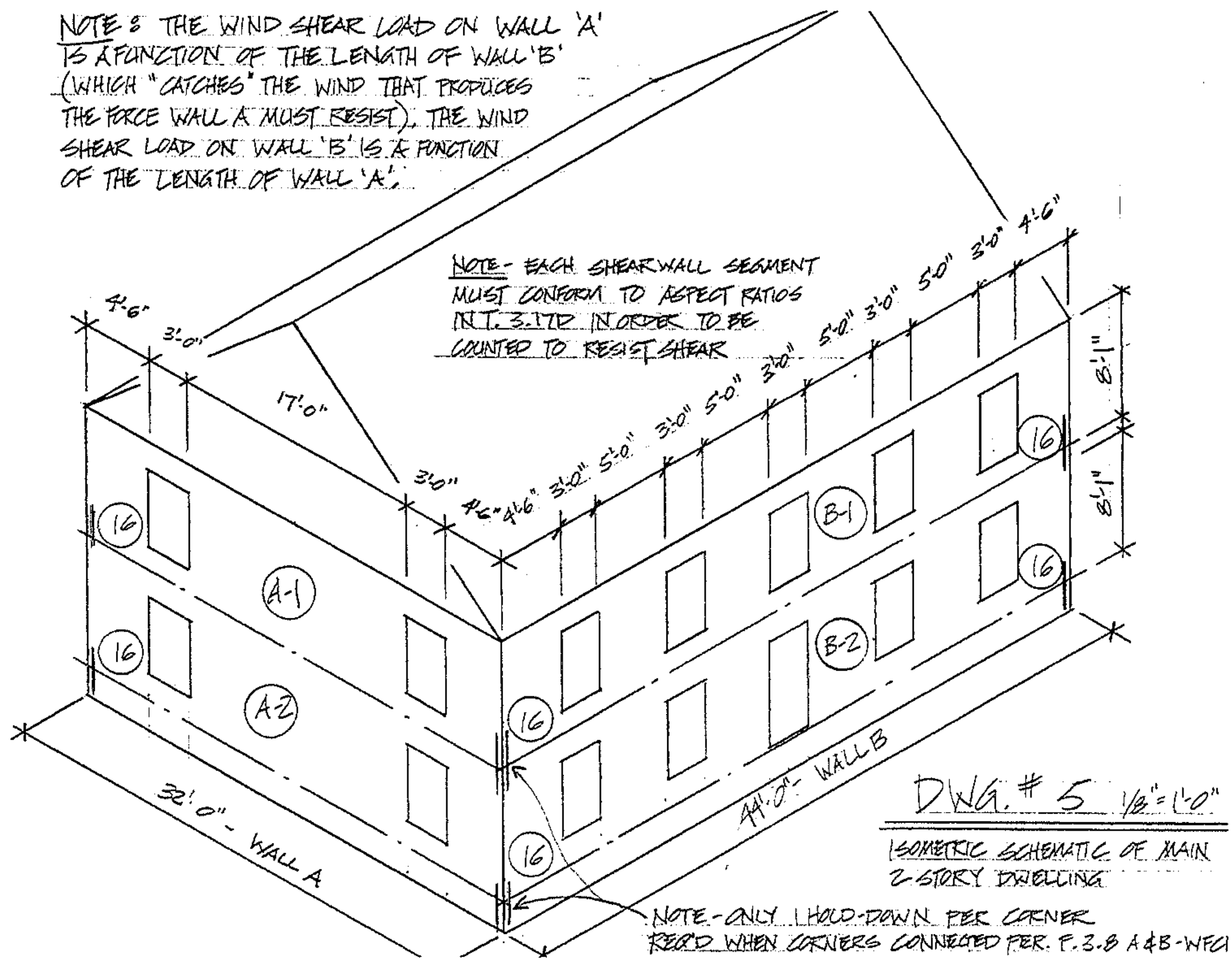

DWG. # 2 3/8" = 1'-0"
BUILDING SECTION
12
10
1 ROOF ASSEMBLY (L, S)
17 ROOF SHEATHING NAILING
16" OC
2 RIDGE STRAPS OR COLLAR TIES (SP)
5 JACK RAFTERS (SP)
6 NON-LOAD-BEARING WALLS (SP) & RAKE O'HANG TO WALL & WALL TO FDN. CONNECTIONS
3 RAFTER/CLG. JST. CONN. (L/S)
16" OC
4 ROOF ASSEMBLY TO WALL ASSEMBLY (L, S, U)
7 TOP & BOTTOM PLATE TO WALL STUD (L)
8'-1"
16" OC
8 WALL ASSEMBLY TO WALL ASSEMBLY (U)
9 BOTTOM PLATE TO FLOOR (L, S)
16" OC
10 FLOOR COMPONENT NAILING
11 FLOOR TO WALL ASSEMBLY (L, S)
18 WALL SHEATHING NAILING
16" OC
16 SHEARWALL HOLD-DOWNS (U)
8'-1"
12 WALL TO FDN. OR SILL (U)
16" OC
11 FLOOR TO SILL (L, S)
FOUNDATION PER 2003 IRC
13 SILL PLATE OR WALL BOTTOM PLATE (SLAB ON GRADE) TO FOUNDATION (L, S)



DRAWING #3 @ 1/2" = 1'-0" - PORTION OF WALL 'A-2'



DRAWING #4 @ 1/4" = 1'-0" - PORTION OF GARAGE ENDWALL


NOTE: THE WIND SHEAR LOAD ON WALL 'A' IS A FUNCTION OF THE LENGTH OF WALL 'B' (WHICH "CATCHES" THE WIND THAT PRODUCES THE FORCE WALL A MUST RESIST). THE WIND SHEAR LOAD ON WALL 'B' IS A FUNCTION OF THE LENGTH OF WALL 'A'.
NOTE- EACH SHEARWALL SEGMENT MUST CONFORM TO ASPECT RATIOS IN T. 3.17D IN ORDER TO BE COUNTED TO RESIST SHEAR
4'-6"
3'-0"
17'-0"
3'-0"
4'-6"
4'-6"
3'-0"
5'-0"
3'-0"
5'-0"
3'-0"
5'-0"
3'-0"
5'-0"
3'-0"
4'-6"
8'-1"
8'-1"
16
A-1
A-2
B-1
B-2
32'-0" - WALL A
44'-0" - WALL B
NOTE - ONLY 1 HOLD-DOWN PER CORNER REQ'D WHEN CORNERS CONNECTED PER. F.3.8 A & B - WFCI
DWG. # 5 1/8" = 1'-0"
ISOMETRIC SCHEMATIC OF MAIN 2-STORY DWELLING