Case Study in Timber

adapted from <u>Simplified Design of Wood Structures</u>, James Ambrose, 5th ed.

<u>Building description</u>

The building is a one-story building intended for commercial occupancy. Figure 16.1 presents a building plan, partial elevation, section and elevation of the perimeter shear walls. Light wood framing (assuming the fire resistance requirements have been met) will be used.

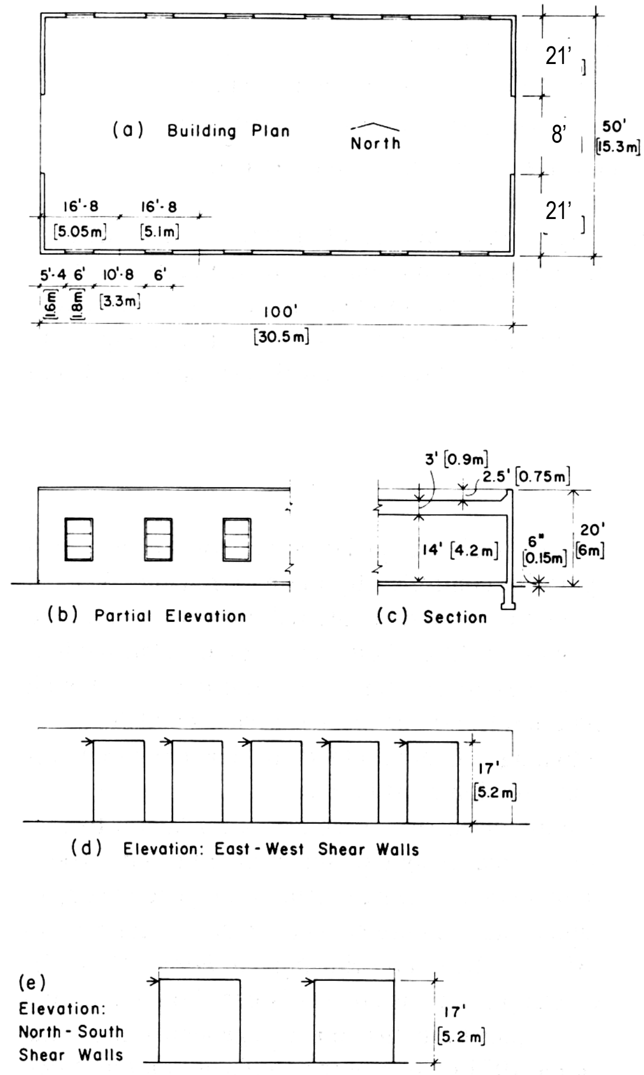


Figure 16.1 Building One, general form.

<u>Loads</u>

Live Loads:
Roof: 20 lb/ft^2 (0.96 kPa)

Wind: critical at 20 lb/ft^2 (0.96 kPa) on vertical exterior surfaces.

Dead Loads:
Roofing & deck: 7.5 lb/ft^2 (0.36 kPa)
Ceiling joists, ceiling & fixtures:
6.5 lb/ft^2 (0.31 kPa)
<u>Total: 14 lb/ft^2 (0.67 kPa)</u>

<u>Materials</u>

Wood framing of Douglas fir-larch, structural grades No. 1 & 2 having a density of 32 lb/ft^3, and AITC glulam timber.

<u>Structural Elements/Plan</u>

If the interior partition walls are arranged as in Figure 16.3a, there are options on the arrangement of the roof structure. We will analyze case 16.3b consisting of roof deck and rafters, stud walls, continuous (two span) beams, and columns.

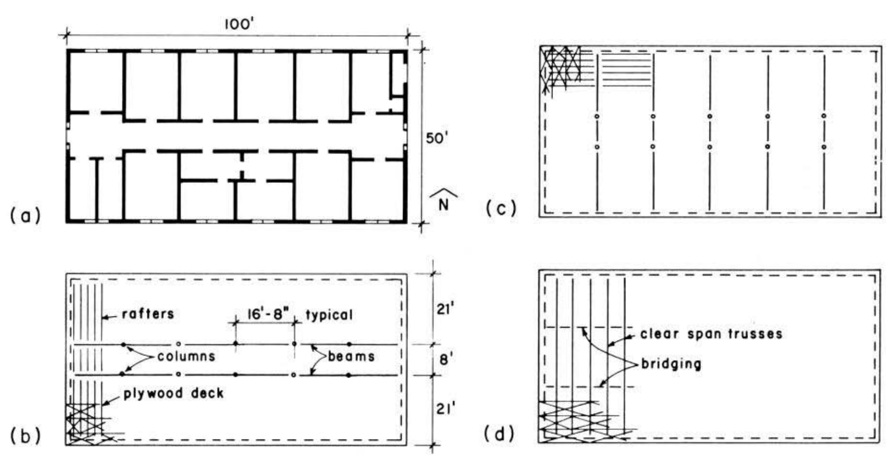


Figure 16.3 Developed plan for interior partitioning and alternatives for the roof framing.

Decking & Rafters:

The standard size of plywood or structural deck panel is 4 ft x 8 ft. The typical orientation is with the long direction with the face grain perpendicular to the rafters or floor joists. (See cross hatching in Figure 16.3.) Typical joist and rafter spacings are 12 in., 16 in., and 24 in. on center. If we use 16 in. on center, the total distributed roof loads (with allowable stress design) with an assumed self weight of 4 lb/ft is:

$$w = (20\ \text{lb/ft}^2 + 14\ \text{lb/ft}^2)\cdot 16\ \text{in}/12\ \text{in/ft} + 4\ \text{lb/ft} = 49.3\ \text{lb/ft}$$

$$M_{max} = \frac{wL^2}{8} = \frac{49.3^{\text{lb}/\text{ft}}(21^{\text{ft}})^2}{8} = 2718^{\text{lb=ft}}$$

Tabular allowable stresses for No. 2 Douglas fir-larch, 2”-4” thick and 2” to 4” wide are:

$$F_{b-single} = 875^{psi},\ F_v = 95^{psi},\ F_{c\perp} = 625^{psi},\ F_c = 1300^{psi},\ E = 1{,}600{,}000^{psi}$$

The load duration for roof loads, C_D = 1.25. The repetitive member factor, C_r = 1.15, applies and the adjusted allowed stress for a fully braced 2x is:

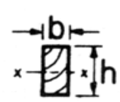
$$F_b' = C_D C_r F_b = (1.25)(1.15)(875\ \text{psi}) = 1258\ \text{psi}$$

The required section modulus is

$$S_{req'd} \ge \frac{M}{F_b'} = \frac{2718^{\text{lb}-\text{ft}}\cdot 12^{\text{in}/\text{ft}}}{1258^{psi}} = 25.9\ \text{in}^3$$

A 2x12 will work if the deflection is limited to allowable for the building code. (This tends to govern for floors. Shear stress should also be checked).

SECTION PROPERTIES
JOISTS AND BEAMS



Nominal Size In Inches b h	Surfaced Size In Inches For Design b h	Area (A) $A = bh$ (In²)	Section Modulus (S) $S = \frac{bh^2}{6}$ (In³)	Moment of Inertia (I) $I = \frac{bh^3}{12}$ (In⁴)	Board Feet Per Linear Foot of Piece
2 x 2	1.5 x 1.5	2.25	0.562	0.422	0.33
2 x 3	1.5 x 2.5	3.75	1.56	1.95	0.50
2 x 4	1.5 x 3.5	5.25	3.06	5.36	0.67
2 x 5	1.5 x 4.5	6.75	5.06	11.39	.83
2 x 6	1.5 x 5.5	8.25	7.56	20.80	1.00
2 x 8	1.5 x 7.25	10.88	13.14	47.63	1.33
2 x 10	1.5 x 9.25	13.88	21.39	98.93	1.67
2 x 12	1.5 x 11.25	16.88	31.64	177.98	2.00
2 x 14	1.5 x 13.25	19.88	43.89	290.78	2.33
3 x 3	2.5 x 2.5	6.25	2.60	3.26	0.75
3 x 4	2.5 x 3.5	8.75	5.10	8.93	1.00
3 x 5	2.5 x 4.5	11.25	8.44	18.98	1.25
3 x 6	2.5 x 5.5	13.75	12.60	34.66	1.50
3 x 8	2.5 x 7.25	18.12	21.90	79.39	2.00
3 x 10	2.5 x 9.25	23.12	35.65	164.89	2.50
3 x 12	2.5 x 11.25	28.12	52.73	296.63	3.00
3 x 14	2.5 x 13.25	33.12	73.15	484.63	3.50
3 x 16	2.5 x 15.25	38.12	96.90	738.87	4.00

Continuous Beams:

The distributed load, including an estimated self weight of 11 lb/ft (about a 6 in x 12 in section) of a glulam beam can be found from:

rafter distributed load:

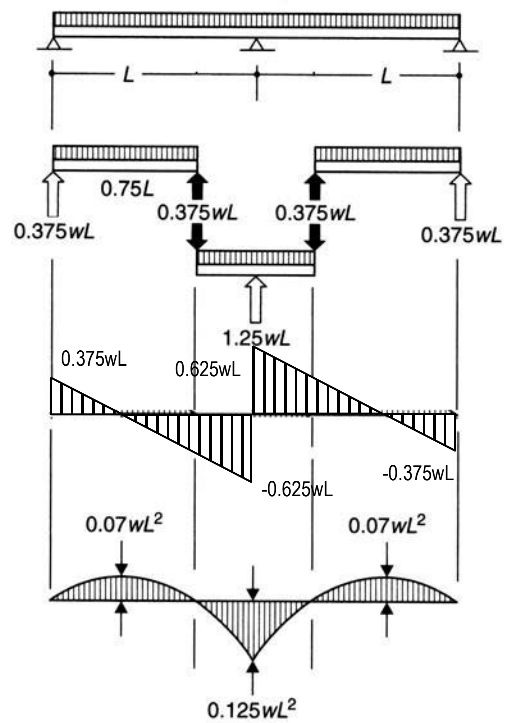
$$\frac{\gamma \cdot A \cdot trib.\ width}{rafter\ spacing} = \frac{(32\ {}^{lb}\!/_{ft^3})(16.88 in^2)(21\,{}^{ft}\!/_2 + 8\,{}^{ft}\!/_2)}{16in} \cdot \left(\frac{1ft}{12in}\right)^2 \cdot \frac{12in}{ft} = 40.8\ {}^{lb}\!/_{ft}$$

roof load:

$(20\ lb/ft^2 + 14\ lb/ft^2)\cdot(21ft/2+8ft/2) = 493\ ^{lb/ft}$

total distributed load:

w = 40.8 lb/ft + 493 lb/ft + 11 lb/ft = 545 lb/ft



The maximum positive moment is $0.07wL^2$ and the maximum negative moment over the support is $0.125wL^2$, where L is the length of one span. $V_{max} = 0.625wL$. (These values come from a beam diagram.)

$M_{max} = 0.125(545\ lb/ft)(16.67\ ft)^2 = 18{,}931\ ^{lb\text{-}ft}$

$V_{max} = 0.625(545\ lb/ft)(16.67\ ft) = 5678\ lb$

$F'_b = C_D F_b = (1.25)(2400\ psi) = 3000\ psi$

$$S_{req'd} \geq \frac{M}{F'_b} = \frac{18931^{lb-ft}}{3000^{psi}} \cdot 12\ {}^{in}\!/_{ft} = 75.7\ in^3$$

From SECTION PROPERTIES/STANDARD SIZES, the 5 ⅛ " x 10.5" is adequate, although a 3 ⅛ " x 13.5" could be evaluated.

DEPTH, d in.	AREA, A in.²	MODIFIED SECTION MODULUS, S_c in.³	MOMENT OF INERTIA, I in.⁴
3⅛" WIDTH			
6.0	18.8	18.8	56
7.5	23.4	29.3	110
9.0	28.1	42.2	190
10.5	32.8	57.4	302
12.0	37.5	75.0	450
13.5	42.2	93.7	641
15.0	46.9	114.3	879
16.5	51.6	136.9	1,170
18.0	56.3	161.3	1,519
19.5	60.9	187.6	1,931
21.0	65.6	215.8	2,412
22.5	70.3	245.9	2,966
24.0	75.0	277.8	3,600
5⅛" WIDTH			
7.5	38.4	48.0	180
9.0	46.1	69.2	311
10.5	53.8	94.2	494
12.0	61.5	123.0	738
13.5	69.2	153.6	1,051
15.0	76.9	187.5	1,441
16.5	84.6	224.5	1,919
18.0	92.3	264.6	2,491
19.5	99.9	307.7	3,167
21.0	107.6	354.0	3,955
22.5	115.3	403.2	4,865
24.0	123.0	455.5	5,904
25.5	130.7	510.8	7,082
27.0	138.4	569.0	8,406
28.5	146.1	630.2	9,887
30.0	153.8	694.3	11,531
31.5	161.4	761.4	13,349
33.0	169.1	831.3	15,348
34.5	176.8	904.1	17,538
36.0	184.5	979.8	19,926
6¾" WIDTH			
12.0	81.0	162.0	972
13.5	91.1	202.4	1,384
15.0	101.3	246.9	1,898
16.5	111.4	295.6	2,527
18.0	121.5	348.4	3,280
19.5	131.6	405.3	4,171
21.0	141.8	466.2	5,209
22.5	151.9	531.1	6,407

TABLE DF-25
DOUGLAS FIR - LARCH

THE AMERICAN INSTITUTE OF TIMBER CONSTRUCTION

Structural Glued Laminated Timber

ROOF BEAMS
CONSTRUCTION LOAD

F_b	F_v	E	C_D	Deflection limit
2400 psi	240 psi	1.8 million psi	1.25	Span / 180 for TOTAL LOAD

Simple Span Beams
For Preliminary Design Purposes
Lamination thickness: 1.500 in.

BEAM SIZE		BEAM	BEAM CAPACITY, UNIFORM LOAD w, plf																
Width	Depth	WEIGHT	SPAN, ft																
b, in.	d, in.	plf	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24
3 1/8	6	4.6	586 D	412 D	300 D	225 D	174 D	137 D	109 D	89 D	--	--	--	--	--	--	--	--	--
3 1/8	7 1/2	5.7	916 B	723 B	586 D	440 D	339 D	267 D	214 D	174 D	143 D	119 D	100 D	--	--	--	--	--	--
3 1/8	9	6.8	1318 B	1042 B	844 B	697 B	586 D	461 D	369 D	300 D	247 D	206 D	174 D	148 D	127 D	109 D	95 D	--	--
3 1/8	10 1/2	8.0	1794 B	1418 B	1148 B	949 B	798 B	680 B	586 D	476 D	393 D	327 D	276 D	234 D	201 D	174 D	151 D	132 D	116 D
3 1/8	12	9.1	2344 B	1852 B	1500 B	1240 B	1042 B	888 B	765 B	667 B	586 D	488 D	412 D	350 D	300 D	259 D	225 D	197 D	174 D
3 1/8	13 1/2	10.3	2935 S	2344 B	1898 B	1569 B	1318 B	1123 B	969 B	844 B	742 B	657 B	586 D	498 D	427 D	369 D	321 D	281 D	247 D
3 1/8	15	11.4	3000 *	2885 S	2344 B	1937 B	1628 B	1387 B	1196 B	1042 B	916 B	811 B	723 B	649 B	586 D	506 D	440 D	385 D	339 D
3 1/8	16 1/2	12.5	3000 *	3000 *	2836 B	2344 B	1969 B	1678 B	1447 B	1260 B	1108 B	981 B	875 B	786 B	709 B	643 B	586 D	513 D	451 D
3 1/8	18	13.7	3000 *	3000 *	3000 *	2789 B	2344 B	1997 B	1722 B	1500 B	1318 B	1168 B	1042 B	935 B	844 B	765 B	697 B	638 B	583 B
3 1/8	19 1/2	14.8	3000 *	3000 *	3000 *	3000 *	2751 B	2344 B	2021 B	1760 B	1547 B	1371 B	1223 B	1097 B	990 B	898 B	815 B	743 B	679 B
5 1/8	6	7.5	961 D	675 D	492 D	370 D	285 D	224 D	179 D	146 D	--	--	--	--	--	--	--	--	--
5 1/8	7 1/2	9.3	1501 B	1186 B	961 D	722 D	556 D	437 D	350 D	285 D	235 D	196 D	165 D	--	--	--	--	--	--
5 1/8	9	11.2	2162 B	1708 B	1384 B	1144 B	961 D	756 D	605 D	492 D	405 D	338 D	285 D	242 D	208 D	179 D	156 D	--	--
5 1/8	10 1/2	13.1	2943 B	2325 B	1883 B	1557 B	1308 B	1114 B	961 D	781 D	644 D	537 D	452 D	384 D	330 D	285 D	248 D	217 D	191 D
5 1/8	12	14.9	3844 B	3037 B	2460 B	2033 B	1708 B	1456 B	1255 B	1093 B	961 D	801 D	675 D	574 D	492 D	425 D	370 D	323 D	285 D
5 1/8	13 1/2	16.8	4813 S	3844 B	3113 B	2573 B	2162 B	1842 B	1588 B	1384 B	1216 B	1077 B	961 D	817 D	701 D	605 D	526 D	461 D	405 D
5 1/8	15	18.7	5591 S	4731 S	3844 B	3177 B	2669 B	2274 B	1961 B	1708 B	1501 B	1328 B	1178 B	1052 B	944 B	830 D	722 D	632 D	556 D
5 1/8	16 1/2	20.6	6000 *	5412 S	4651 B	3844 B	3230 B	2752 B	2373 B	2067 B	1808 B	1592 B	1412 B	1261 B	1132 B	1022 B	926 B	841 D	740 D
5 1/8	18	22.4	6000 *	6000 *	5271 S	4574 B	3844 B	3275 B	2824 B	2443 B	2133 B	1878 B	1666 B	1487 B	1335 B	1205 B	1093 B	996 B	911 B
5 1/8	19 1/2	24.3	6000 *	6000 *	5922 S	5158 S	4511 B	3841 B	3288 B	2844 B	2484 B	2187 B	1940 B	1731 B	1555 B	1403 B	1273 B	1159 B	1060 B
5 1/8	21	26.2	6000 *	6000 *	6000 *	5740 S	5065 S	4422 B	3785 B	3274 B	2859 B	2518 B	2233 B	1993 B	1790 B	1615 B	1465 B	1334 B	1220 B
5 1/8	22 1/2	28.0	6000 *	6000 *	6000 *	6000 *	5591 S	4986 S	4315 B	3733 B	3260 B	2870 B	2546 B	2272 B	2040 B	1842 B	1670 B	1521 B	1391 B
5 1/8	24	29.9	6000 *	6000 *	6000 *	6000 *	6000 *	5467 S	4878 B	4220 B	3685 B	3245 B	2878 B	2569 B	2306 B	2082 B	1888 B	1720 B	1573 B
5 1/8	25 1/2	31.8	6000 *	6000 *	6000 *	6000 *	6000 *	5974 S	5362 S	4735 B	4135 B	3641 B	3229 B	2882 B	2588 B	2336 B	2119 B	1930 B	1765 B
6 3/4	6	9.8	1266 D	889 D	648 D	487 D	375 D	295 D	236 D	192 D	--	--	--	--	--	--	--	--	--
6 3/4	7 1/2	12.3	1978 B	1563 B	1266 D	951 D	732 D	576 D	461 D	375 D	309 D	258 D	217 D	--	--	--	--	--	--
6 3/4	9	14.8	2848 B	2250 B	1823 B	1506 B	1266 D	995 D	797 D	648 D	534 D	445 D	375 D	319 D	273 D	236 D	205 D	--	--
6 3/4	10 1/2	17.2	3876 B	3063 B	2481 B	2050 B	1723 B	1468 B	1266 D	1029 D	848 D	707 D	595 D	506 D	434 D	375 D	326 D	285 D	251 D
6 3/4	12	19.7	5063 B	4000 B	3240 B	2678 B	2250 B	1917 B	1653 B	1440 B	1265 B	1055 D	889 D	756 D	648 D	560 D	487 D	426 D	375 D
6 3/4	13 1/2	22.1	6339 S	5063 B	4101 B	3389 B	2848 B	2426 B	2092 B	1812 B	1583 B	1393 B	1236 B	1076 D	923 D	797 D	693 D	607 D	534 D
6 3/4	15	24.6	7364 S	6231 S	5063 B	4184 B	3516 B	2990 B	2559 B	2214 B	1933 B	1702 B	1510 B	1348 B	1210 B	1092 B	951 D	832 D	732 D
6 3/4	16 1/2	27.1	8000 *	7128 S	6126 B	5063 B	4239 B	3583 B	3067 B	2653 B	2317 B	2040 B	1809 B	1615 B	1450 B	1309 B	1187 B	1081 B	975 D
6 3/4	18	29.5	8000 *	8000 *	6943 S	6004 B	5001 B	4228 B	3618 B	3130 B	2734 B	2407 B	2135 B	1905 B	1711 B	1544 B	1401 B	1276 B	1167 B
6 3/4	19 1/2	32.0	8000 *	8000 *	7800 S	6794 S	5823 B	4922 B	4213 B	3644 B	3183 B	2802 B	2485 B	2218 B	1992 B	1798 B	1631 B	1485 B	1358 B
6 3/4	21	34.5	8000 *	8000 *	8000 *	7560 S	6671 S	5666 B	4850 B	4196 B	3664 B	3226 B	2861 B	2554 B	2293 B	2070 B	1877 B	1710 B	1564 B
6 3/4	22 1/2	36.9	8000 *	8000 *	8000 *	8000 *	7364 S	6460 B	5529 B	4783 B	4177 B	3678 B	3262 B	2912 B	2614 B	2360 B	2140 B	1949 B	1783 B
6 3/4	24	39.4	8000 *	8000 *	8000 *	8000 *	8000 *	7200 S	6250 B	5407 B	4722 B	4157 B	3687 B	3291 B	2955 B	2667 B	2419 B	2204 B	2015 B
6 3/4	25 1/2	41.8	8000 *	8000 *	8000 *	8000 *	8000 *	7869 S	7013 B	6067 B	5298 B	4665 B	4137 B	3693 B	3316 B	2993 B	2715 B	2473 B	2261 B
6 3/4	27	44.3	8000 *	8000 *	8000 *	8000 *	8000 *	8000 *	7674 S	6763 B	5906 B	5200 B	4612 B	4117 B	3696 B	3337 B	3026 B	2756 B	2521 B
6 3/4	28 1/2	46.8	8000 *	8000 *	8000 *	8000 *	8000 *	8000 *	8000 *	7495 B	6545 B	5763 B	5111 B	4562 B	4096 B	3697 B	3353 B	3054 B	2793 B

TABLE SPECIFICATIONS: This table applies to straight, simply supported glued laminated timber beams under dry conditions of use. Beams must be laterally supported at the top along the length of the beam and at the top and bottom at the ends. The load carrying capacities tabulated are for total load including the weight of the member.

BEAM WEIGHT: 35.0 pounds per cubic foot was used to determine beam weight per lineal foot shown in the table.

DESIGN VALUE MODIFICATIONS: The allowable stress in bending , Fb , has been adjusted by the AITC volume factor, CV .

For determination of load carrying capacities governed by shear, loads within a distance "d" (the depth of the beam) from the ends have been neglected.

DEFLECTION LIMITS: For roof beams, deflection is limited to span /180 for total load.

CONTROLLING VALUES: Values marked with a D are controlled by deflection, B are bending controlled, and S are shear controlled.

SPAN: Span is defined as the length from centerline to centerline of bearing. This span is the length used in standard engineering equations to calculate deflection, bending and shear.

* The values have been limited to reasonable capacities. Engineering calculations may allow for greater capacities.

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The self weight should be determined to compare to the assumption. Table DF-25 indicates the self weight is 13 lb/ft, and that size at our span is controlled by deflection (I for Δ=L/180), but this chart is for *simply supported beams* and $\Delta_{max} = \frac{5wL^4}{384EI}$.

The maximum deflection for a two span beam can be found with $\Delta_{max} = \frac{wL^4}{185EI}$, which is only 0.415x the deflection of a simply supported span.

For sawn lumber, a 6x14 would be required from the comparison chart.

Equivalent Glulam Sections for Dimension Lumber/Timber Beams

Sawn[4] Sections Nominal Size	Roof Beams[1, 2]: Select Structural, Douglas Fir/Larch	Select Structural, Southern Pine	No. 1, Douglas Fir/Larch	No. 1, Southern Pine
3x8	3 1/8x6	3x6 7/8	3 1/8x6	3x5 1/2
3x10	3 1/8x7 1/2	3x8 1/4	3 1/8x6	3x6 7/8
3x12	3 1/8x9	3x9 5/8	3 1/8x7 1/2	3x8 1/4
3x14	3 1/8x9	3x11	3 1/8x7 1/2	3x9 5/8
4x6	3 1/8x6	3x6 7/8	3 1/8x6	3x5 1/2
4x8	3 1/8x7 1/2	3x8 1/4	3 1/8x6	3x6 7/8
4x10	3 1/8x9	3x11	3 1/8x7 1/2	3x8 1/4
4x12	3 1/8x10 1/2	3x12 3/8	3 1/8x9	3x9 5/8
4x14	3 1/8x12	3x13 3/4	3 1/8x10 1/2	3x11
4x16	3 1/8x13 1/2	3x15 1/8	3 1/8x10 1/2	3x12 3/8
6x8	5 1/8x7 1/2	5x6 7/8	5 1/8x7 1/2	5x6 7/8
6x10	5 1/8x9	5x8 1/4	5 1/8x7 1/2	5x8 1/4
6x12	5 1/8x10 1/2	5x9 5/8	5 1/8x9	5x9 5/8
6x14	5 1/8x12	5x12 3/8	5 1/8x10 1/2	5x11
6x16	5 1/8x13 1/2	5x13 3/4	5 1/8x12	5x12 3/8
6x18	5 1/8x15	5x15 1/8	5 1/8x13 1/2	5x13 3/4
6x20	5 1/8x18	5x16 1/2	5 1/8x16 1/2	5x15 1/8

Evaluate shear strength:

$$F'_v = C_D F_v = (1.25)240 \text{ psi} = 300 \text{ psi}$$

$$f_v = \frac{3V}{2A} = \frac{3(5678lb)}{2(53.8in^2)} = 158psi$$

which is less than the allowable of 300 psi (OK).

Stud Walls & Columns:

Building codes dictate the maximum height for slenderness (10 ft typical), and the spacing of wall studs depending on what they support (roof, roof and one floor, roof and two floors). Structural design focuses on shear wall behavior.

The interior column load is:

$$P = 1.25wL = 1.25(545 \text{ lb/ft} + 2 \text{ lb/ft of extra beam self weight})(16.67 \text{ ft}) = 11.4^{kips}$$

For a 10 ft braced column height, choose a 6 x 6.

TABLE 10.1 Safe Loads for Wood Columns[a]

Column Section: Nominal Size	Area (in.²)	Unbraced Length (ft): 6	8	10	12	14	16	18	20	22	24	26
4 × 4	12.25	11.1	7.28	4.94	3.50	2.63						
4 × 6	19.25	17.4	11.4	7.76	5.51	4.14						
4 × 8	25.375	22.9	15.1	10.2	7.26	6.46						
6 × 6	30.25	27.6	24.8	20.9	16.9	13.4	10.7	8.71	7.17	6.53		
6 × 8	41.25	37.6	33.9	28.5	23.1	18.3	14.6	11.9	9.78	8.91		
6 × 10	52.25	47.6	43.0	36.1	29.2	23.1	18.5	15.0	13.4	11.3		
8 × 8	56.25	54.0	51.5	48.1	43.5	38.0	32.3	27.4	23.1	19.7	16.9	14.6
8 × 10	71.25	68.4	65.3	61.0	55.1	48.1	41.0	34.7	29.3	24.9	21.4	18.4
8 × 12	86.25	82.8	79.0	73.8	66.7	58.2	49.6	42.0	35.4	30.2	26.0	22.3
10 × 10	90.25	88.4	85.9	83.0	79.0	73.6	67.0	60.0	52.9	46.4	40.4	35.5
10 × 12	109.25	107	104	100	95.6	89.1	81.2	72.6	64.0	56.1	48.9	42.9
10 × 14	128.25	126	122	118	112	105	95.3	85.3	75.1	65.9	57.5	50.4
12 × 12	132.25	130	128	125	122	117	111	104	95.6	86.9	78.3	70.2
14 × 14	182.25	180	178	176	172	168	163	156	148	139	129	119
16 × 16	240.25	238	236	234	230	226	222	216	208	200	190	179

[a]Load capacity in kips for solid-sawn sections of No. 1 grade Douglas fir–larch under normal moisture and load duration conditions.

Wind Design:

Diaphragms are categorized as flexible or rigid and must resist lateral forces in both transverse and longitudinal directions. A diaphragm is made up of a shear-resisting element (sheathing) and boundary members called *chords* and *collectors (struts or drag struts)*. The chords are designed to carry the moment in the diaphragm. The collectors are designed to transmit the horizontal reactions to the shear walls. The structural behavior is often compared to that of a steel I section on its side (Figure 15.6).

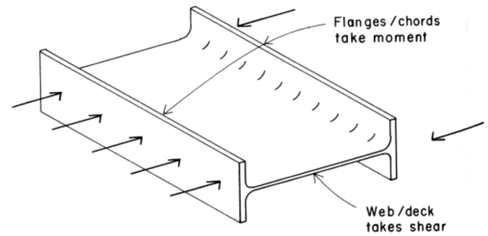


Figure 15.6 Flanged and webbed beam analogy for a horizontal, wood-framed diaphragm.

Tables in building codes for combinations of plywood grade, common nail size, plywood thickness, how the panels are arrayed and if blocking is used provide allowable shear in pounds per foot.

Consideration of lateral wind loads will be presented, but uplift on the roof must be accounted for with anchorage if the live load exceeds the downward gravity loads.

TABLE 23-II-H—ALLOWABLE SHEAR IN POUNDS PER FOOT FOR HORIZONTAL WOOD STRUCTURAL PANEL DIAPHRAGMS WITH FRAMING OF DOUGLAS FIR-LARCH OR SOUTHERN PINE[1]

PANEL GRADE	COMMON NAIL SIZE	MINIMUM NAIL PENETRATION IN FRAMING (inches)	MINIMUM NOMINAL PANEL THICKNESS (inches)	MINIMUM NOMINAL WIDTH OF FRAMING MEMBER (inches)	BLOCKED DIAPHRAGMS: Nail spacing (in.) at diaphragm boundaries (all cases), at continuous panel edges parallel to load (Cases 3 and 4) and at all panel edges (Cases 5 and 6) × 25.4 for mm				UNBLOCKED DIAPHRAGMS: Nails spaced 6" (152 mm) max. at supported edges	
					6	4	2 1/2[2]	2[2]	Case 1 (No unblocked edges or continuous joints parallel to load)	All other configurations (Cases 2, 3, 4, 5 and 6)
					Nail spacing (in.) at other panel edges × 25.4 for mm					
					6	6	4	3		
		× 25.4 for mm			× 0.0146 for N/mm					
Structural 1	6d	1 1/4	5/16	2	185	250	375	420	165	125
				3	210	280	420	475	185	140
	8d	1 1/2	3/8	2	270	360	530	600	240	180
				3	300	400	600	675	265	200
	10d[3]	1 5/8	15/32	2	320	425	640	730	285	215
				3	360	480	720	820	320	240
C-D, C-C, Sheathing, and other grades covered in UBC Standard 23-2 or 23-3	6d	1 1/4	5/16	2	170	225	335	380	150	110
				3	190	250	380	430	170	125
			3/8	2	185	250	375	420	165	125
				3	210	280	420	475	185	140
	8d	1 1/2	3/8	2	240	320	480	545	215	160
				3	270	360	540	610	240	180
			7/16	2	255	340	505	575	230	170
				3	285	380	570	645	255	190
			15/32	2	270	360	530	600	240	180
				3	300	400	600	675	265	200
	10d[3]	1 5/8	15/32	2	290	385	575	655	255	190
				3	325	430	650	735	290	215
			19/32	2	320	425	640	730	285	215
				3	360	480	720	820	320	240

[1]These values are for short-time loads due to wind or earthquake and must be reduced 25 percent for normal loading. Space nails 12 inches (305 mm) on center along intermediate framing members.

Allowable shear values for nails in framing members of other species set forth in Division III, Part III, shall be calculated for all other grades by multiplying the shear capacities for nails in Structural I by the following factors: 0.82 for species with specific gravity greater than or equal to 0.42 but less than 0.49, and 0.65 for species with a specific gravity less than 0.42.

[2]Framing at adjoining panel edges shall be 3-inch (76 mm) nominal or wider and nails shall be staggered where nails are spaced 2 inches (51 mm) or 2 1/2 inches (64 mm) on center.

[3]Framing at adjoining panel edges shall be 3-inch (76 mm) nominal or wider and nails shall be staggered where 10d nails having penetration into framing of more than 1 5/8 inches (41 mm) are spaced 3 inches (76 mm) or less on center.

<u>North-South</u>

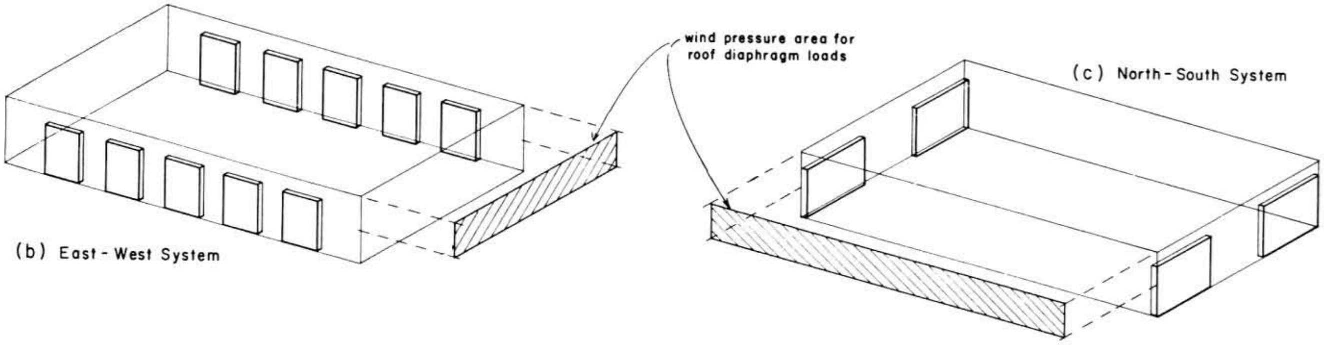
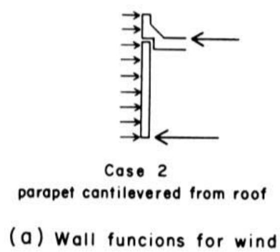


Figure 16.5 Building One, wall functions and wind pressure development.



The tributary height for the wall and parapet is 17.5 ft/2 + 2.5 ft = 11.25 ft

The distributed lateral wind load = (20 lb/ft^2)11.25ft = 225 lb/ft

The total lateral wind load = (225 lb/ft)(100 ft) = 22,500 lb

The end reactions to the lateral load = 22,500 lb/2 = 11,250 lb

The *unit shear* (or distributed shear) in the **diaphragm** = 11,250 lb/(50 ft) = 225 lb/ft;

so a roof deck can be chosen that has an allowable shear > 225 lb/ft.

Knowing that ½ in decking is the minimum for a membrane-type roof, we use table 23-II-H to select $\frac{15}{32}$ in. sheathing with 2 x framing and 8d nails at 6 in. at all panel edges and a blocked diaphragm having an allowable shear in pounds per foot of 270 lb/ft.

The moment of the "deep beam" is used to determine the force in the top and bottom chords as show in Figure 16.6 which is 5.62 kips.

The *unit shear* in the two **shear walls** of 21 ft each = 11,250 lb/(2·21 ft) = 268 lb/ft;

so a stud wall can be chosen that has an allowable shear > 268 lb/ft.

Using table 23-II-I-1, $\frac{3}{8}$ in. plywood sheathing with 6d nails at 4 in. at all panel edges directly applied to framing (not over gypsum sheathing) has an allowable shear in pounds per foot of 300 lb/ft.

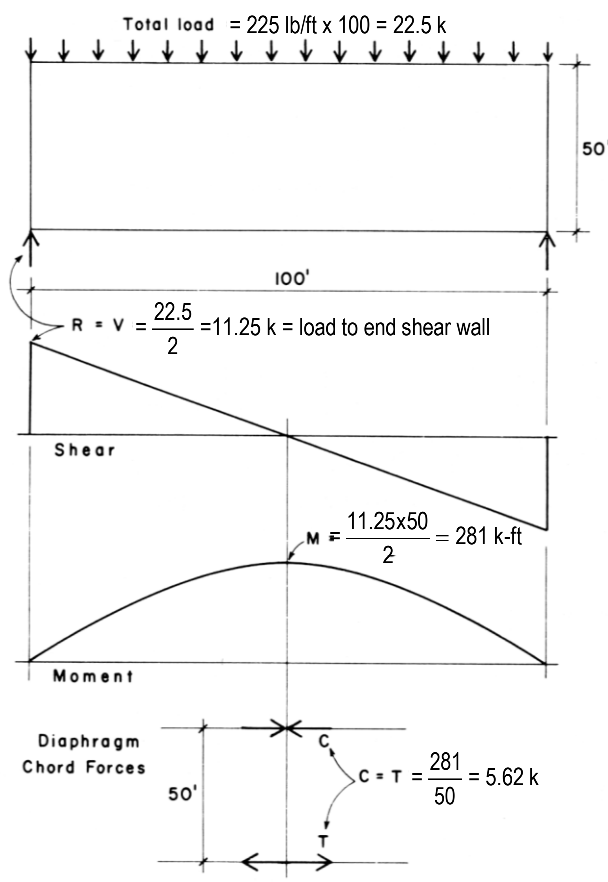


Figure 16.6 Spanning functions of the roof diaphragm.

TABLE 23-II-I-1—ALLOWABLE SHEAR FOR WIND OR SEISMIC FORCES IN POUNDS PER FOOT FOR WOOD STRUCTURAL PANEL SHEAR WALLS WITH FRAMING OF DOUGLAS FIR-LARCH OR SOUTHERN PINE[1,2,3]

PANEL GRADE	MINIMUM NOMINAL PANEL THICKNESS (inches)	MINIMUM NAIL PENETRATION IN FRAMING (inches)	PANELS APPLIED DIRECTLY TO FRAMING: Nail Size (Common or Galvanized Box)[5]	Nail Spacing at Panel Edges (in.) 6	4	3	2	PANELS APPLIED OVER 1/2-INCH (13 mm) OR 5/8-INCH (16 mm) GYPSUM SHEATHING: Nail Size (Common or Galvanized Box)[5]	Nail Spacing at Panel Edges (in.) 6	4	3	2
	× 25.4 for mm			× 25.4 for mm; × 0.0146 for N/mm					× 25.4 for mm; × 0.0146 for N/mm			
Structural I	5/16	1 1/4	6d	200	300	390	510	8d	200	300	390	510
	3/8	1 1/2	8d	230[4]	360[4]	460[4]	610[4]	10d	280	430	550	730
	7/16			255[4]	395[4]	505[4]	670[4]					
	15/32			280	430	550	730					
	15/32	1 5/8	10d	340	510	665	870	—	—	—	—	—
C-D, C-C Sheathing, plywood panel siding and other grades covered in UBC Standard 23-2 or 23-3	5/16	1 1/4	6d	180	270	350	450	8d	180	270	350	450
	3/8			200	300	390	510		200	300	390	510
	3/8	1 1/2	8d	220[4]	320[4]	410[4]	530[4]	10d	260	380	490	640
	7/16			240[4]	350[4]	450[4]	585[4]					
	15/32			260	380	490	640					
	15/32	1 5/8	10d	310	460	600	770	—	—	—	—	—
	19/32			340	510	665	870					
			Nail Size (Galvanized Casing)					Nail Size (Galvanized Casing)				
Plywood panel siding in grades covered in UBC Standard 23-2	5/16	1 1/4	6d	140	210	275	360	8d	140	210	275	360
	3/8	1 1/2	8d	160	240	310	410	10d	160	240	310	410

[1]All panel edges backed with 2-inch (51 mm) nominal or wider framing. Panels installed either horizontally or vertically. Space nails at 6 inches (152 mm) on center along intermediate framing members for 3/8-inch (9.5 mm) and 7/16-inch (11 mm) panels installed on studs spaced 24 inches (610 mm) on center and 12 inches (305 mm) on center for other conditions and panel thicknesses. These values are for short-time loads due to wind or earthquake and must be reduced 25 percent for normal loading.

Allowable shear values for nails in framing members of other species set forth in Division III, Part III, shall be calculated for all other grades by multiplying the shear capacities for nails in Structural I by the following factors: 0.82 for species with specific gravity greater than or equal to 0.42 but less than 0.49, and 0.65 for species with a specific gravity less than 0.42.

[2]Where panels are applied on both faces of a wall and nail spacing is less than 6 inches (152 mm) on center on either side, panel joints shall be offset to fall on different framing members or framing shall be 3-inch (76 mm) nominal or thicker and nails on each side shall be staggered.

[3]Where allowable shear values exceed 350 pounds per foot (5.11 N/mm), foundation sill plates and all framing members receiving edge nailing from abutting panels shall not be less than a single 3-inch (76 mm) nominal member. Nails shall be staggered.

[4]The values for 3/8-inch (9.5 mm) and 7/16-inch (11 mm) panels applied direct to framing may be increased to values shown for 15/32-inch (12 mm) panels, provided studs are spaced a maximum of 16 inches (406 mm) on center or panels are applied with long dimension across studs.

[5]Galvanized nails shall be hot-dipped or tumbled.

Wall overturning must be considered from the shear and compared to the resisting moment from gravity loads and proper anchorage must be provided to keep the wall from sliding off the foundation. Referring to Figure 16.7:

$V = 11.25\text{ k}/ 2 = 5.625\text{ k}$

Roof dead load is determined from a tributary area of half a rafter spacing width, one rafter, and the wall length

roof DL = $(14\text{ lb/ft}^2 \cdot 16\text{ in}/12\text{ in/ft}/2 + 4\text{ lb/ft})21\text{ ft} = 280\text{ lb}$

Wall dead load can be determined with the material weights for stud walls, sheathing, gypsum board and wood shingles:

wall DL= $(2\text{ lb/ft}^2 + 3\text{ lb/ft}^2 + 5\text{ lb/ft}^2 + 2\text{ lb/ft}^2)(21\text{ ft})(17\text{ ft}) = 4.3\text{ k}$

overturning moment = $(5.625\text{k})(17\text{ ft}) = 95.6\text{ k-ft}$

resisting moment = $(4.6\text{ k})(21\text{ ft})/2 = 48.4\text{ k-ft}$

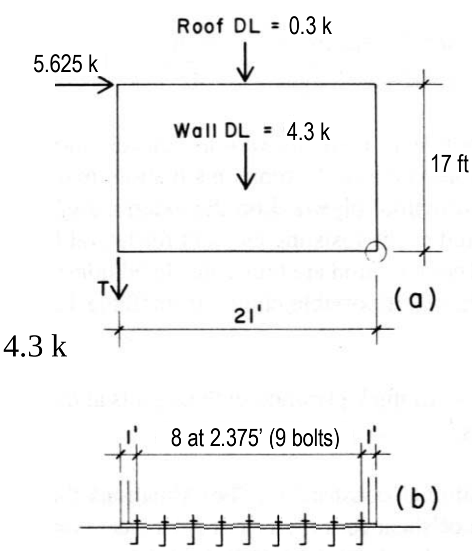


Figure 16.7

Table 17-13.
Weights of Building Materials

Materials	Weight lb per sq ft	Materials	Weight lb per sq ft
CEILINGS		**PARTITIONS**	
Channel suspended system	1	Clay tile	
Lathing and plastering	See Partitions	3 in.	17
Acoustical fiber tile	1	4 in.	18
		6 in.	28
		8 in.	34
		10 in.	40
FLOORS		Gypsum block	
Steel deck	See Manufacturer	2 in.	$9\frac{1}{2}$
		3 in.	$10\frac{1}{2}$
Concrete-Reinforced 1 in.		4 in.	$12\frac{1}{2}$
Stone	$12\frac{1}{2}$	5 in.	14
Slag	$11\frac{1}{2}$	6 in.	$18\frac{1}{2}$
Lightweight	6 to 10	Wood studs 2×4	
		12–16 in. o.c.	2
Concrete-Plain 1 in.		Steel partitions	4
Stone	12	Plaster 1 in.	
Slag	11	Cement	10
Lightweight	3 to 9	Gypsum	5
		Lathing	
Fills 1 inch		Metal	$\frac{1}{2}$
Gypsum	6	Gypsum board $\frac{1}{2}$ in.	2
Sand	8		
Cinders	4		
Finishes			
Terrazzo 1 in.	13		
Ceramic or Quarry Tile $\frac{3}{4}$-in.	10	**WALLS**	
Linoleum $\frac{1}{4}$-in.	1	Brick	
Mastic $\frac{3}{4}$-in.	9	4 in.	40
Hardwood $\frac{7}{8}$-in.	4	8 in.	80
Softwood $\frac{3}{4}$-in.	$2\frac{1}{2}$	12 in.	120
		Hollow concrete block	
		(Heavy aggregate)	
ROOFS		4 in.	30
Copper or tin	1	6 in.	43
Corrugated steel	See Manufactuer	8 in.	55
3-ply ready roofing	1	$12\frac{1}{2}$ in.	80
3-ply felt and gravel	$5\frac{1}{2}$	Hollow concrete block	
5-ply felt and gravel	6	(Light aggregate)	
		4 in.	21
Shingles		6 in.	30
Wood	2	8 in.	38
Asphalt	3	12 in.	55
Clay tile	9 to 14	Clay tile (Load bearing)	
Slate $\frac{1}{4}$ in.	10	4 in.	25
		6 in.	30
Sheathing		8 in.	33
Wood $\frac{3}{4}$ in.	3	12 in.	45
Gypsum 1 in.	4	Stone 4 in.	55
		Glass block 4 in.	18
Insulation 1 in.		Window, Glass, Frame, & Sash	8
Loose	$\frac{1}{2}$	Curtain walls	See Manufacturer
Poured	2	Structural glass 1 in.	15
Rigid	$1\frac{1}{2}$	Corrugated Cement Asbestos $\frac{1}{4}$ in.	3

For weights of other materials used in building construction, see Table 17-12.

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The resisting moment is not enough to compensate for the overturning moment. We like the factor of safety for overturning to be 1.5, and there *is no safety* in this case, which means we must provide a tie down in tension (T). The L shape of the corner will help some resisting overturning, as well as the glulam beam reaction.

For equilibrium of moments (positive = negative)

$$SF = \frac{M_{resist}}{M_{overturning}} \geq 1.5$$

T(21ft) + 48.4 k-ft = (95.6 k-ft)1.5; $T_{req'd}$ = 4.52 k

The shear must be resisted, and the code minimum bolting usually consists of ½ in. diameter bolts at 1 ft from the wall ends and at a maximum of 6 ft on center for the remainder of the wall length. If design for wind loading allows us to increase the allowable stress by 1/3, the number of bolts from single shear in a 2" sill plate parallel to the grain will be:

$(1.33)(480 \text{ lb/bolt})(n) \geq 5{,}625$ lb

$n \geq 8.8$ bolts

Use 9 bolts, spaced at 2.375 ft

(see next page for description of design value symbols)

TABLE 11.1 Bolt Design Values for Wood Joints with Douglas Fir–Larch (lb/bolt)

THICKNESS			DOUGLAS FIR-LARCH					
MAIN MEMBER	SIDE MEMBER	BOLT DIAMETER	SINGLE SHEAR			DOUBLE SHEAR		
t_m inches	t_s inches	D inches	$Z_{\parallel}$ lbs.	$Z_{s\perp}$ lbs.	$Z_{m\perp}$ lbs.	$Z_{\parallel}$ lbs.	$Z_{s\perp}$ lbs.	$Z_{m\perp}$ lbs.
1-1/2	1-1/2	1/2	480	300	300	1050	730	470
		5/8	600	360	360	1310	1040	530
		3/4	720	420	420	1580	1170	590
		7/8	850	470	470	1840	1260	630
		1	970	530	530	2100	1350	680

$Z_{||}$ = nominal lateral design value for single bolt in connection with all wood members loaded parallel to grain

$Z_{s\perp}$ = nominal lateral design value for single bolt in wood-to-wood connection with main member loaded parallel to grain and side member loaded perpendicular to grain

$Z_{m\perp}$ = nominal lateral design value for single bolt in wood-to-wood connection with main member loaded parallel to grain and side member loaded perpendicular to grain and side member loaded parallel to grain

East-West

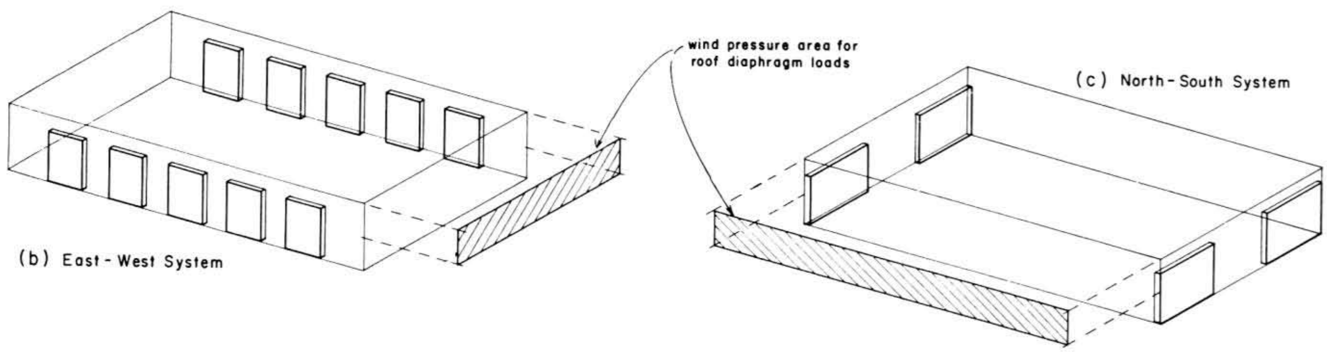

Figure 16.5 Building One, wall functions and wind pressure development.

The tributary height for the wall and parapet and the distributed lateral wind load are the same as in the North-South direction.

The total lateral wind load = (225 lb/ft)(50 ft) = 11,250 lb

The end reactions to the lateral load = 11,250 lb/2 = 5,625 lb

The *unit shear* (or distributed shear) in the **diaphragm** = 5,625 lb/(100 ft) = 56.25 lb/ft.

It is convenient to use the diaphragm structural panel construction chosen in the North-South direction with a capacity of 270 lb/ft.

The *unit shear* (or distributed shear) in the five **shear walls** of 10.67 ft each:

= 5,625 lb/(5·10.67 ft) = 105 lb/ft.

It is convenient to use the shear wall structural panel construction chosen in the North-South direction with a capacity of 300 lb/ft.