BUILDING DESIGN EXAMPLE 1

Wisconsin Rapids (Wood County)

STRUCTURAL CALCULATIONS

Loadings – Wind 90 mph
Ground snow 50 psf
Floor live 80 psf or 2,000 lbs. concentrated load corridors for second floor
Floor live 50 psf or 2,000 lbs. concentrated load offices w/o heavy machines
Floor live 100 psf or 2,000 lbs. concentrated load slab on grade corridors
Partition live 20 psf in addition to the above loads
Dead load = 15 PSF wood floor system and 20 PSF roof system w/shingles

Using ASD design methods, thus load combinations are per Section 1605.3.1 requirements.

Seismic design category A due to location north of 4%g line in Figure 1615(2), thus wind load is more critical horizontal load design, as shown in this analysis.

Snow load design – Roof snow load is 50 PSF ground snow load (p_g) times adjustment factors *ASCE 7 section 7.4* [Equation 7-2] *shows the sloped roof snow load* (p_s) *as roof slope factor* (C_s) *times flat roof snow load. ASCE 7 section 7.3* [equation 7-1] *shows the flat roof snow load as the ground snow load times 0.7 times exposure factor times thermal factor times importance factor.*
$p_f = 0.7\ C_e\ C_t\ I\ p_g = 0.7(0.9)1.1(1.0)50$ PSF = 34.7 PSF *flat roof snow load*
$\mathbf{p_s} = C_s\ p_f = 1.0(34.7$ PSF) = **34.7 PSF** *(no slope factor reduction w/this pitch & shingles)*

Wood roof trusses will span 60' and have 2' overhang each side, **submitted separately** later as a structural component plan. Roof hip design assumes change in truss orientation at 20' from end walls, due to location needed to avoid excessive window headers. **Wood floor trusses**, mostly top chord bearing will also be **submitted separately** at a later date as structural component plan.

Roof load on north & south walls = (34.7 + 20)PSF x (60'/2 + 2') = 54.7(32) = 1750 PLF
Roof load on east & west walls = (34.7 + 20)PSF x (20'/2 +2') = 54.7(12) = 656 PLF
Concentrated load from hip girder truss = (54.7 PSF) x (60'/2 + 2') x (20'/2 + 2'/2) = 19,255 lbs.

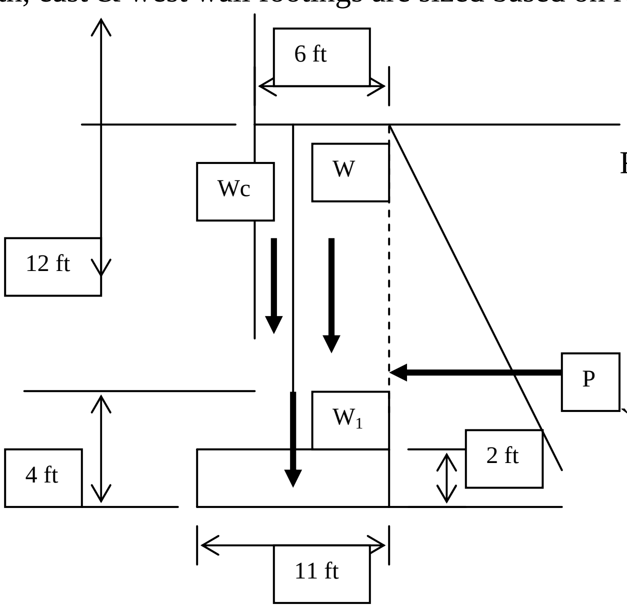
Floor loads on north & south exterior walls = [floor live + partition live + floor dead loads] x tributary width = (80 + 20 + 15)PSF x (30'/2) = 1725 PLF
Floor loads on center beam = (80 + 20 + 15)PSF x 60'/2 = 3450 PLF

Foundation loads – Interior footings = 3450 PLF (30'spacing) = 103,500 lbs. live load
South wall uniform (vertical only) = 1750 PLF + 1725 PLF + [10 PSF (22')]wall = 3695 PLF
North wall uniform = 1750 PLF + 1725 PLF + [10 PSF (10') + 150 PSF (12')]wall = 5375 PLF
East & west worst case uniform = 656 PLF + [10 PSF (10') + 150 PSF (12')]wall = 2556 PLF
Concentrated loads on exterior walls from headers and girder truss will be adequately spread by foundation wall, thus no special analysis is needed for them.

Interior spread footings – 103,500 pounds / 2000 PSF = 51.75 square feet (+ for ftg. dead load)
Use 8' x 8' x 1'3" concrete pads @ 64 square feet with #4 rebars at 12" on center both directions

South wall footing – 3695 PLF / 2000 SPF = 1.85 feet wide minimum Use a 4' deep continuous footing 12" thick by 2.5 foot wide with (3) #4 continuous rebar.

North, east & west wall footings are sized based on retaining wall requirement limitations below.



Retaining wall foundation –

Soil weight = 120 pcf,
Angle of internal friction = 35^0,
f'c = 3000 psf,
fy = 40,000 psi,
soil bearing capacity = 2000 psf

W = (120)(14)(5) = 8400 lbs

Ca = [(1 - sin35)/1 + sin 35) = 0.271

Ca(w) = (0.271)(120) = 32.5 pcf

P = (1/2)[(32.5)(16)](16)(1) = 4160 lbs

Wc = (150)(14)(1)(1) = 2100 lbs

W_1 = (150)(2)(11)(1) = 3300 lbs

Moment from heel = (8400)(2.5) + (2100)(5.5) + (3300)(5.5) = 50700 ft-lbs

Summation of weights = 8400 + 2100 + 3300 = 13800 lbs

Resultant from heel = 50700/13800 = 3.67 ft

Resisting moment = 13800(11 - 3.67) = 101,154 ft-lbs

Overturning moment = P(1/3)(16) = (4160)(1/3)(16) = 22187 ft-lbs

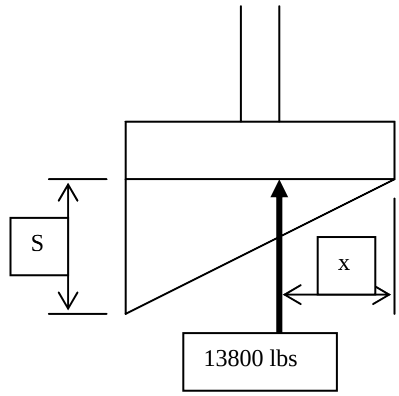
Factor of safety against overturning = (101154)/(22187) = 4.5 ***<u>Since the factor of safety is greater than 2 so far design is acceptable!!!!!!</u>***

x = [50700 + 22187]/13800 = 5.28 ft

Maximum soil pressure = S

(13800) = (1/2)(S)(11 - 5.28)(3)

S = 1608.4 psf ***<u>Which is less than 2000 Psf so the maximum soil bearing capacity Is not exceeded!!!!</u>***



Note that the design would continue at this point to do the calculations to determine the reinforcing required to resist the shear and moment values of the footing and the wall portion of the retaining wall.

<u>The retaining wall will have a maximum exposed height of 12 feet (worst condition). Retaining wall is 12 inches thick reinforced concrete with a 2 feet thick footing that is 11 feet in width. The calculations for the design of the concrete reinforcing of the wall and footing has not performed for this example as it would take a fair amount of time to do such.</u>

<u>Wind resistance design</u> – simple diaphragm type and using IBC 1609.6 Simplified Procedure

Winds from east, west, and north are adequate as taken by standard diaphragm construction. *Worst case is for wind from south, due to both more wall height and exposure C for that side.*

<u>Worst case</u> – As roof wind pressure values are negative, per Table 1609.6.2(1) footnote c, use 0 for roof and MWFRS for walls of 17.8 PSF in end zone of 12' and 11.9 PSF elsewhere, but for south side in exposure C the increase of 1.38 (interpolated value for 28' roof) is required.

<u>Upper half of second floor</u> – value at roof = 0# [due to negative values]   wall h = 10'/2 + 1'heel
Wall height x pressure x wall length = (10'/2 + 1' heel) [1.38(17.8 or 11.9)]PSF(12' or 156'/2)
6'[1.38(17.8 PSF)]12' + 6'[1.38(11.9 PSF)]78' = 1768.6# + 7685.5# = **9454.1# shear each end**

9454.1# / 60' roof width = **157.6 PLF** <u>roof shear</u> **unblocked diaphragm** <u>okay</u> < allowed 275.5 PLF *from Table 2306.3.1 with wind increase and group III lumber reduction* 1.4(240 PLF)0.82

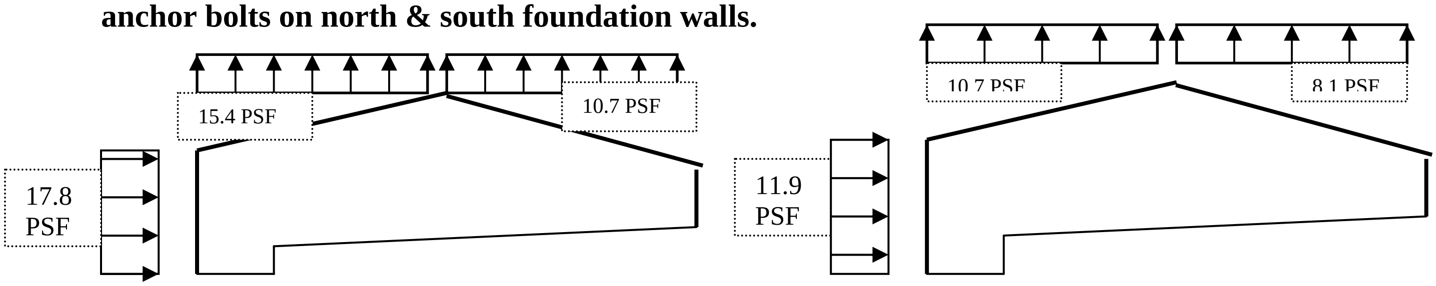
<u>West wall</u> 9454.1# / (60' – [2(3') + 3(4')]) = **225.1 PLF west wall shear** <u>okay</u> < allowed 292.7 PLF *from Table 2306.4.1 with wind increase and group III lumber reduction* 1.4(255 PLF)0.82

<u>East wall</u> 9454.1# / (60' – [8(4')]) = **337.6 PLF east wall shear** <u>okay</u> < allowed 453.5 PLF *from Table 2306.4.1 with wind increase, group III lumber reduction &* ***4" nailing*** 1.4(395 PLF)0.82

<u>Lower half of second floor</u> – Wall above 10'/2 + floor system depth 2' + wall below 10'/2 = 12'
12'[1.38(17.8 PSF)]12' + 12'[1.38(11.9 PSF)]156'/2 = 3537.2 + 15,371 = **18,908.2# shear** to each end via the floor system   *(conservative value shown, as leeward wall not same height)*

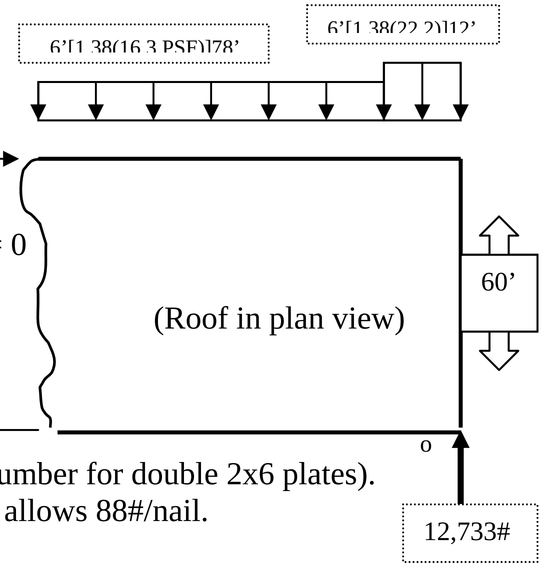
Floor system shear at ends = 18.908.2# / 60' = **315.1 PLF unblocked diaphragm** <u>okay</u> < 327.2 PLF allowed *from Table 2306.3.1 w/wind increase & group III lumber reduction* 1.4(285)0.82

East & west wall to foundation transfer – using the wall and floor systems to bottom plate and directly into the concrete walls only (conservative to ignore the wood portion). From above values each end 9454.1# + 18,908.2# = 28,362.3# load resisted over the worst case west end wall of only 44 feet. 28,362.3# / 44' = 644.6 PLF
Based on 5/8" diameter anchor bolts in Southern Pine wall plate parallel bolt value ('97 NDS Table 8.2E) with wind adjustment (1.6)930#/bolt = 1488# /bolt; spacing would be (1488# /bolt) / 644.6 PLF = 2.3 feet / bolt; thus **use 2'3" on center for 5/8" diameter anchor bolts on east & west foundation walls.**
Checking south wall anchor bolts would have (12'/2)[1.38(17.8 PSF)] = 147.4 PLF shear at end zone. With '97 NDS perpendicular bolt value of 1.6(560# /bolt) = 896# /bolt; spacing would be maximum (896# /bolt) / 147.4 PLF = 6.1 feet / bolt; thus use **4' on center for 5/8" diameter anchor bolts on north & south foundation walls.**



END ZONE   INTERIOR ZONE

Note that per this above calculation, the floor & roof trusses may have group III lumber species.



Chord forces – the roof chord force using free body diagram

$\Sigma M_o = 0$
$T(0') + 12{,}733\#(0') + 2205.8\#(6') + 10{,}527.2\#(51') – C(60') = 0$
$C = (13{,}234.8 + 536{,}887.2) / 60'$
$C = 9{,}168.7\#$
$\Sigma F_x = 0$
$C – T = 0 \quad C = T = 9{,}168.7\#$

Top plate splices on wall at roof truss bearing (assuming SPF lumber for double 2x6 plates). Using 3" x 0.135" gun nails (16d size) from NDS Table 12.3A allows 88#/nail.
9,168.7# / 2 (1.6) 88#/nail = 33 nails each side of a splice.
As that is excessive, design would best be switched to a **triple top plate** (or better lumber type). Staggering the splices, two members are always present, thus only 1/3 of load must be transferred on each side of a splice (& using the same 3" x 0.135" gun nails & SPF triple plates). 9,168.7# / 3 (1.6) 88#/nail = **22 nails each side of a splice**, that is 10 more than row 10 of Table 2304.9.1 would require for typical top plate nailing (having only 12 each side of a splice). **OK**

Floor system chord forces are calculated similarly.

C = [(2404.5# + 1003.5#)6' + (11,366.8# + 4843.8#)51'] / 60' = 14,119.8#

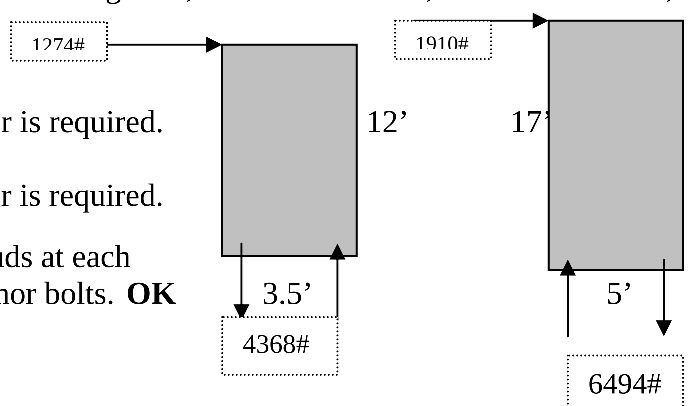
On south side use triple top plate as chord for this floor system bearing wall (similar to plate above). 14,119.8# / 3 (1.6) 88#/nail = **34 nails each side of a splice**, which is 22 more than row 10 of Table 2304.9.1 requirement for typical top plate nailing (with only 12 each side of splices).

On west side the floor system is attached to concrete foundation wall (acting as the chord) with ledger blocks, needing an attached value of 14,119.8# / 90' = 156.9 PLF to the concrete. If floor trusses are 2' on center, each ledger block needs 2' (156.9 PLF) = 313.8# anchor to the concrete. This ledger needs 1725 PLF vertical capacity for floor loading, thus will be adequate in shear. Anchor bolts are sized as 1725 PLF / 880#/bolt = 6" on center for 1" diameter anchor bolts in 3" ledger (from NDS Table 8.2E for 2.5" SPF lumber). **OK**

Anchorage from overturning of shear walls – West end has 4', 8', and 22' shear walls of a 12' tall wall height (with 3.5', 7.5', and 21' between end anchor bolts) and one 6' wide shear wall at 17' tall (use 5' between anchors). Dividing the total shear by the lengths of these wall gives loads of [12,733# / 40'] (4') = 1274# for 4' wall segment; 1910# for 6' wall; 2547# for 8' wall; and 7003# for 22' wall segment.
Analyzing two cases we see that:

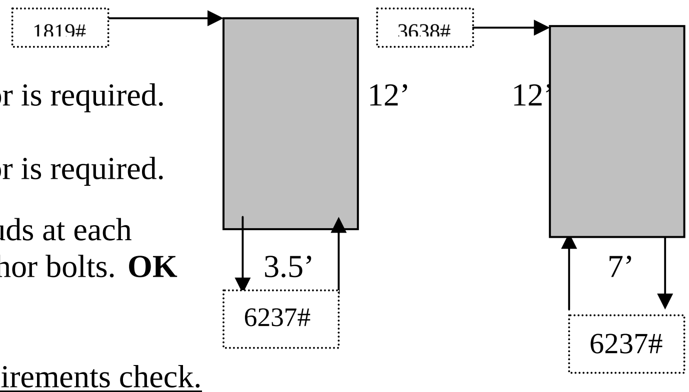
For 4' wall a minimum 4368# uplift anchor is required.
and
For 6' wall a minimum 6494# uplift anchor is required.

Use a **SIMPSON® HD8A** with double studs at each hold-down location and 7/8" diameter anchor bolts. **OK**



East end wall has an 8' and five 4' shear walls (between window openings) on upper level, having 3.5' between anchors. [Alternative hold down using IBC 2305.3.7.2 would require 79% reduction of allowable shear wall capacity (per Table 2305.3.7.2 for 5' high openings in 11' walls) calculated on page 4].

17' tall (use 5' between anchors). Dividing the total shear by the lengths of these wall gives loads of [12,733# / 28'] (4') = 1819# for 4' wall segment and 3638# for an 8' wall segment.



Analyzing two cases we see that:

For 4' wall a minimum 6237# uplift anchor is required.
and
For 8' wall a minimum 6237# uplift anchor is required.

Use a **SIMPSON® HD8A** with double studs at each hold-down location and 7/8" diameter anchor bolts. **OK**

<u>Component and cladding</u> <u>wind load requirements check.</u>

Roof and wall sheathing is in 4' x 8' sheets, thus if not interpolating values (round down) from IBC Table 1609.6.2.1(2), you must use the 20 ft^2 effective wind area values. Using the worst case for wind from the south, for exposure C increase of 1.38 to adjust the Table 1609.6.2.1(2) values for the design 90 mph wind, you get:

Roof center area pressure	(1.38)10 PSF = 13.8 PSF	or	suction (1.38)-16 PSF = -22.1 PSF
Roof edge area pressure	(1.38)10 PSF = 13.8 PSF	or	suction (1.38)-25.6 PSF = -35.3 PSF
Roof corner area pressure	(1.38)10 PSF = 13.8 PSF	or	suction (1.38)-25.6 PSF = -35.3 PSF
Wall center area pressure	(1.38)13.9 PSF = 19.2 PSF	or	suction (1.38)-15.1 PSF = -20.8 PSF
Wall end zone area pressure	(1.38)13.9 PSF = 19.2 PSF	or	suction (1.38)-18.2 PSF = -25.1 PSF

Worst case roof sheathing field nailing at 12" on center into trusses spaced two feet apart gives a total of 15 nails (3 rows of 5 nails) and edge nails at 6" on center both ends gives 2(9) = 18 nails; for a total of 33 nails per sheet (tributary area of 4'x8' = 32 ft^2) with eave & ridge suction of 35.3 PSF. Assuming a dead load of 2 PSF for asphalt shingles and 1.5 PSF for ½" plywood sheathing; (2 + 1.5) = 3.5 PSF may be deducted. 32(-35.3 + 3.5) = 1017.6# total uplift per sheet. Taking the uplift divided by the total number of nails 1017.6# / 33 nails = 30.8#/nail required.
Assume that roof trusses may be SPF lumber in top chord (specific gravity = 0.42) and using NDS Table 12.2A for 8d box nail withdrawal design values; we get only 18#/inch (2.5" – 0.5") = 36# / nail allowable. Allowable exceeds required holding value, standard nailing is adequate. **OK**

Note that if any open canopy or overhang were provided, then IBC Table 1609.6.2.1(3) uplift loading would require additional fastening for components (framing) and cladding (sheathing). If a 3' x 7' canopy over the doors on the south side were added uplift loading would be (from Table 1609.6.2.1(3) with exposure C adjustment) = 1.38(-40.5 PSF) = -55.9 PSF. Compare that to the old Comm 53.12(2) of only 30 PSF uplift!!!

HEADERS –

Exterior wall headers over 6' windows second floor – use **(3) 2x12's No.1 Douglas Fir-Larch** or equal F_b = 1000 psi(1.15 *LDF*)(c_r 1.15) = 1322.5 psi S = (3)31.64 in^3 = 94.92 in^3 M = $WL^2/8$
Worst case 1750 PLF (from previous page) on 6' span M = 1750 PLF$(6')^2/8$ = 7,875 ft-lbs
$f_b = M/S$ f_b = 7,875 ft-lbs[12"/1']/ 94.92 in^3 = 995.6 psi actual < 1322.5 psi allowable **OK**
$f_v = V/A$ = [1750 PLF (6'/2)]/(3)16.88 in^2 = 103.7 psi < 109.3 psi [= 95 psi (1.15 LDF)(C_H)] **OK**
$\Delta = 5wL^4/384EI$ = 5[1750 PLF]$(6')^4(12''/1')^3$/384(1,700 ksi)534 in^4 = 0.056 in << L/360 **OK**

Exterior wall headers over < 6' windows second floor – use **(3) 2x12's No.1 Douglas Fir-Larch** or equal *same as above* actual less than above 6' case < 1322.5 psi allowable **OK**

South first floor wall headers over 6' windows – use **(2) 1.75"x12" LVL** or equal F_b = 2600 psi
S = (2)42 in^3 = 84 in^3 I = (2)252 in^4 = 504 in^4 M = $WL^2/8$ $\Delta = 5wL^4/385EI$
3695 PLF (from previous page) on 6' span M = 3695 PLF$(6')^2/8$ = 16,627.5 ft-lbs
$f_b = M/S$ f_b = 16,627.5 ft-lbs[12"/1']/ 84 in^3 = 2375.4 psi actual < 2600 psi allowable **OK**
$f_v = V/A$ = [3695 PLF (6'/2)]/(2)21 in^2 = 263.9 psi < 285 psi [= 285 psi (1.0 LDF)(1.0 C_H)] **OK**
Δ = 5[3695 PLF]$(6')^4(12''/1')^3$/385(2,000 ksi)504 in^4 = 0.107 in = L/673 < L/360 **OK**

Center headers floor system – (previous page) 3450 PLF spans for 30'spacing using steel beams & columns to support wood truss floor system. M = $wL^2/8$ $\Delta = 5wL^4/384EI$
M = 3450 PLF$(30')^2/8$ = 388.1 Kip-ft Providing a positive connection at trusses for lateral bracing will provide 2' on center lateral support and limiting beam depth to 18" to allow for mechanical runs below would lead to using 50 ksi steel for economy sake. From Ninth Edition of AISC steel manual page 2-200 table the beam size W18x76 is most economical (or W16x89).
Δ = 5[3450 PLF]$(30')^4(12''/1')^3$/384(29,000 ksi)1330 in^4 = 1.63 in ≅ L/188 > L/360 ***no good***

This is not even close, so span must be cut to limit deflection (since span is taken to the 4^{th} power), thus add another column. [180' – 2' (exterior walls)] / 7 spans ≅ 25.5 feet on center.
M = 3450 PLF$(25.5')^2/8$ = 280.4 Kip-ft page 2-202 table beam size W18x60 is economical
Δ = 5[3450 PLF]$(25.5')^4(12''/1')^3$/384(29,000 ksi)984 in^4 = 1.15 in ≅ L/263 > L/360 ***no good***

This is not good enough, so span must be cut again to further limit deflection, thus adding two more columns. [180' – 2' (exterior walls)] / 8 spans ≅ 22.25 feet on center. *(change footing too)*
M = 3450 PLF$(22.25')^2/8$ = 213.5 Kip-ft page 2-202 table beam size W18x46 is economical
Δ = 5[3450 PLF]$(22.25')^4(12''/1')^3$/384(29,000 ksi)712 in^4 = 0.92 in ≅ L/290 > L/360 ***no good***
L/360 =22.25'[12"/1']/360= 0.742 = Δ solving backwards for required I = $5wL^4/384E\Delta$ = 884 in^4
Most economical 18" deep beam with I > 884 in^4 is **beam size W18x55** 50 ksi with I = 890 in^4
Δ = 5[3450 PLF]$(22.25')^4(12''/1')^3$/384(29,000 ksi)890 in^4 = 0.67 in ≅ L/399 < L/360 **OK**

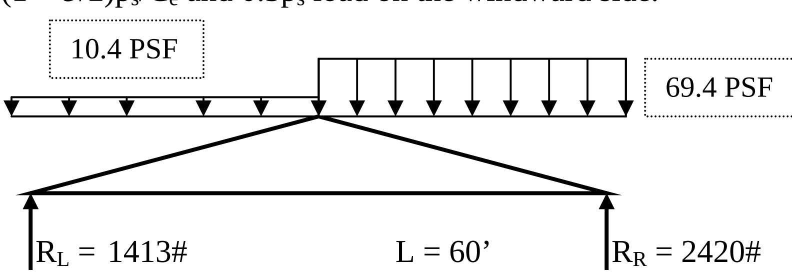
Revised **–** Interior footing loads = (3450 + 55) PLF (22.25'spacing) = 77,986 lbs. live load
Interior spread footings – 77,986 pounds / 2000 PSF = 38.99 square feet (+ for ftg. dead load)
Use **7'x 7'x 1'3" concrete pads** @ 49 sq.ft. **w/ #4 rebars at 12" on center** both directions **OK**

Interior columns – load = 78 kips 10.5 feet high (12' less beam depth) use steel tube column
Steel manual page 3-42 table column size **5 x 5 x ¼ square tube column** will support 82 kips @ 13' (used K = 1.2 to increase unbraced length from 10.5' to 13') **OK**

<u>Bearing plate for steel beam on top of steel column</u> – load transferred = 78 kips (or 39 kips/side)
Beam width is 7.5 inches = plate width Column cross section area is 4.59 in^2
W18x55 50 ksi from steel manual page 2-113 for allowable uniform loads for beams shows that
R = 60 k R_1 = 42.2 kips R_2 = 12.9 kips/in R_3 = 46.5 kips R_4 = 3.75 kips/in
Bearing length based on local web yielding = $(R – R_1)/ R_2$ = (60 – 42.2)/12.9 = 1.4 inches
Bearing length based on web crippling = $(R – R_3)/ R_4$ = (60 – 46.5)/3.75 = 3.6 inches - - controls
Actual bearing length is at least 2(3.6 inches) = 7.2 inches, but the need for room to place bolts and still have clearance from 5" x 5" column will require plate to be at least 3" + 5" + 3" = 11"
Plate thickness t = bottom flange thickness = 5/8" **top plate is 7.5" x 5/8" x 11"** **OK**
Assuming (4)3/4" diameter bolts to keep things in place (bearing alone handles the load)

<u>Column base plate</u> – area of plate must be large enough to permit anchor bolts, thus assume at least a 1.75" clearance to column & 1" to edge of plate on diagonal $2.75"/(2)^{½}$ = 1.94" each side plus column of 5", yielding 1.94" + 5" + 1.94" = 8.88" or 9" size. 9" x 9" = 81 in^2 actual plate
Checking A_2 = 78/(0.175)3 = 149 in2 finding A_1 as greater of A_1 = 5"(5") = 25 in^2 or
A_1 = 78/(0.7)3 = 37.2 in^2 or $A_1 = 1/149[78/(0.35)3]^2$ = 37.2 in^2
Δ = 0.5[0.9(5") – 0.8(5")] = 0.375" N $\cong$ $(37.2)^{½}$ + 0.375" = 6.5" $\cong$ 7" B = 37.2/7 = 5.3 $\cong$ 6"
f_p = 78/(7)6 = 1.9 ksi < $F_p = 0.35(3)[149/37.2]^{½}$ = 2.1 ksi
m = [7 – 0.95(5")]/2 = 1.125" n = [6" – 0.8(5")]/2 = 1" n' = $([5"(5")]^{½})/4$ = 1.25"
minimum thickness $t_p = 2(1.25")[1.9/36]^{½}$ = 0.57" use at least 5/8" thick base plate
Column base plate is 9" x 9" x 5/8" assuming (4)3/4" diameter anchor bolts

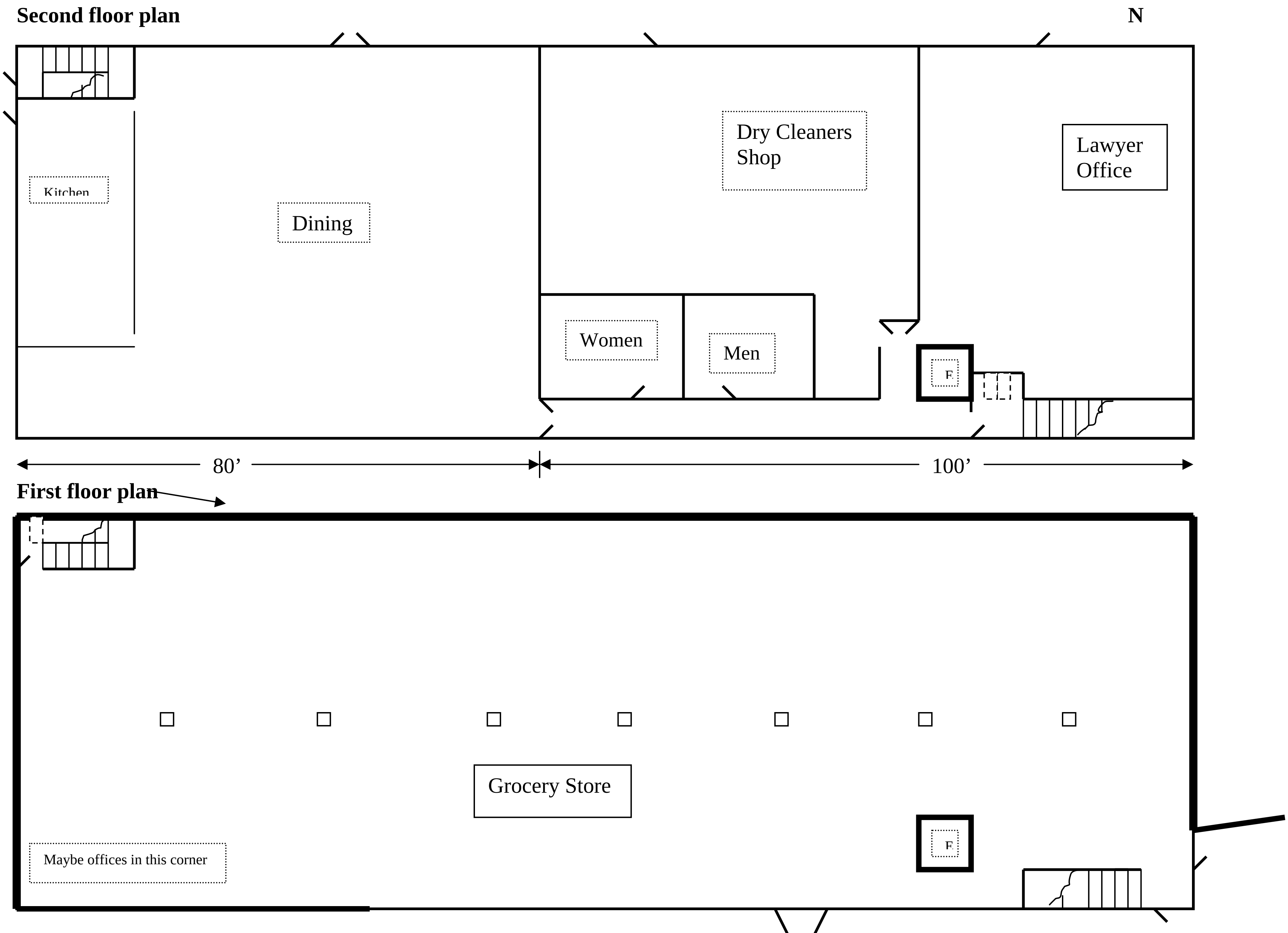
<u>Snow Drift Loading Calculations:</u> per ASCE 7-98 section 7.6.1 using p_f & p_s = 34.7 PSF for p_g = 50 PSF with roof pitch is 4:12 for a slope = 18.4^o and the eave to ridge distance W = 32 feet as L/W = 5.6 is > 4, then β = 1.0 and per Eq. 7-4 $\gamma = 0.13p_g + 14$ = 0.13(50) + 14 = 20.5 PCF roof load is based on slope if greater or less than $275(\beta)p_f/\gamma W$ = 275(1.0)34.7/20.5(32) = 14.5^o
As W > 20 ft and slope is > 14.5^o, then the structural shall be designed to resist an unbalanced snow load on the leeward side of $1.2(1 + \beta/2)p_s/C_e$ and $0.3p_s$ load on the windward side.
1.2(1 + ½)34.7/0.9 = 69.4 PSF
0.3(34.7 PSF) = 10.4 PSF
No windward load and only applying that load as shown will yield reactions shown which leads to revised header and revised footing calculations.



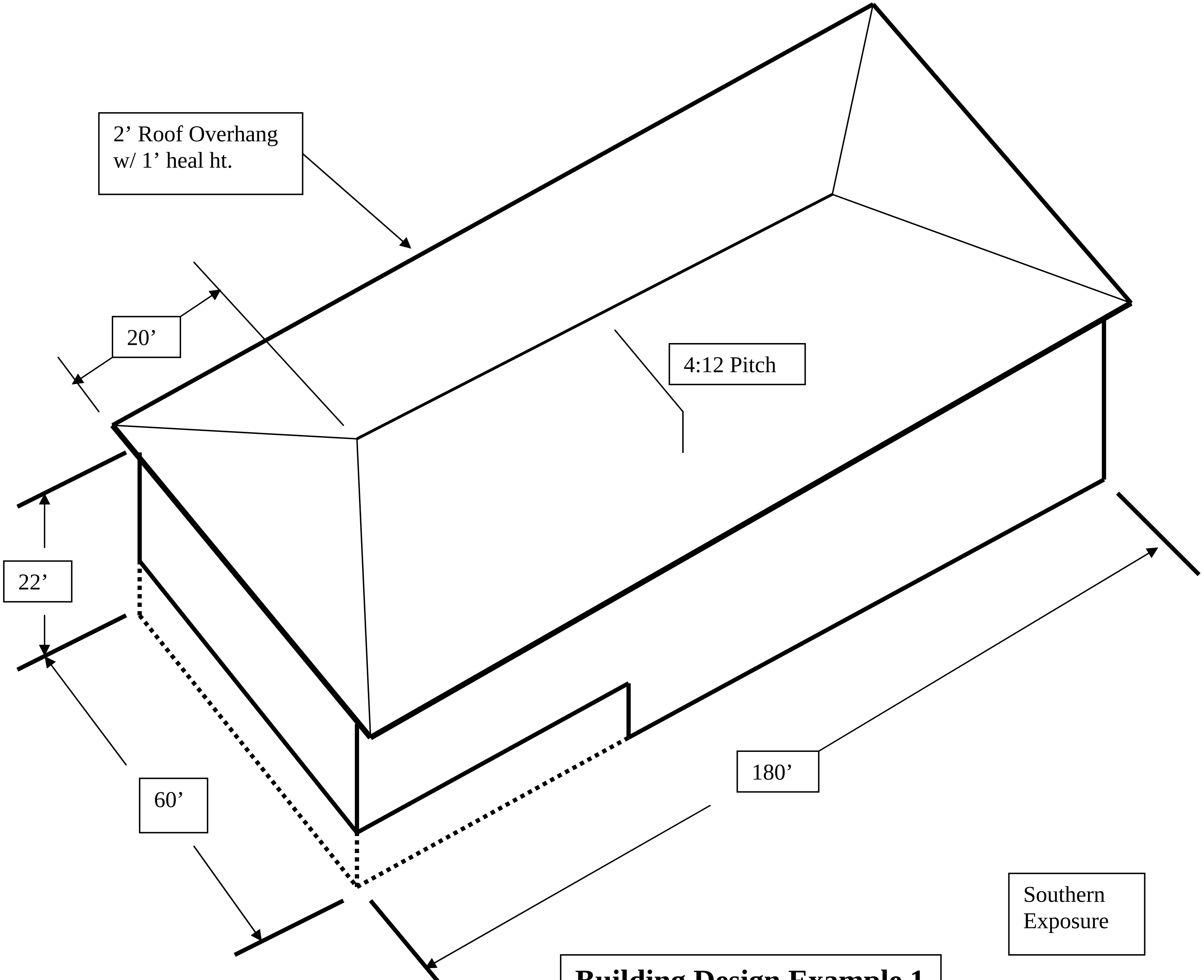
$\Sigma M_y = 0 = R_R(60') – [(10.4 + 20)32']14' – [(69.4 + 20)32']46'$, thus R_R = 2420.3 pounds
$\Sigma F_y = 0 = R_R + R_L – 32'(10.4 + 20 \text{ PSF}) – 32'(69.4 + 20 \text{ PSF})$, thus R_L = 1413.3 pounds

South wall uniform (vertical only) = 2420 PLF + 1725 PLF + [10 PSF (22')]wall = 4365 PLF
North wall uniform = 2420 PLF + 1725 PLF + [10 PSF (10') + 150 PSF (12')]wall = 6165 PLF
Thus from previous pages the footing sizes shown are still adequate with extra loads. **OK**

Window headers for second floor at f_b = 1376.7 psi is 4% **overstressed** and Δ = 0.078" or L/926.
First floor south window headers are 8% **overstressed** in bending & 9.4% **overstressed** in shear.



Building Design Example 1
Plan View



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