Lauren Wilke
Structural Option
Advisor: M.K. Parfitt

Boyds Bear Country
Pigeon Forge, TN

Technical Report 1
Structural Concepts / Structural Existing Conditions Report

Executive Summary:

Boyds Bear Country, located in Pigeon Forge, Tennessee, is designed as a multi-functional space and tourist attraction for Boyds Collections Ltd. The 112,620 square foot building houses three floors of retail space with multiple cashier and information desks, warehouse storage, a loading dock, a full sized restaurant, food court, ice cream parlor, special events areas, and offices.

In analyzing structural systems of Boyds Bear Country it is quickly apparent that special considerations will have to be made to accommodate the use of multiple materials and unique applied loads in the structure. It implements structural steel, cold rolled steel, concrete slabs, cast-in-place concrete walls and foundations, reinforced concrete block, wooden trusses, and wooden members. Later studies of the building will require more in-depth research as to the effects of incorporating such a great number of materials into one building.

This report investigates the existing structural conditions and design concepts of the building, as well as redesigns of typical members within the building. A brief summary of these systems is as follows:

The main structural system of the building is composed of a steel grid supporting composite slabs. Wooden trusses support the roof, covered in plywood sheathing. Lateral resistance is provided by concentric braced frames and masonic shear walls. Foundations consist of shallow footings and piers, built as a mixture of cast-in-place concrete and masonry. No expansion joints are present in the structure.

The original design was performed using the 1999 Standard Building Code and its respective related codes. Loadings were determined using ASCE 7-95 and ASCE 7-98 and original member selections were chosen using AISC Manual of Steel Construction, Allowable Stress Design, 9th Edition (1989).

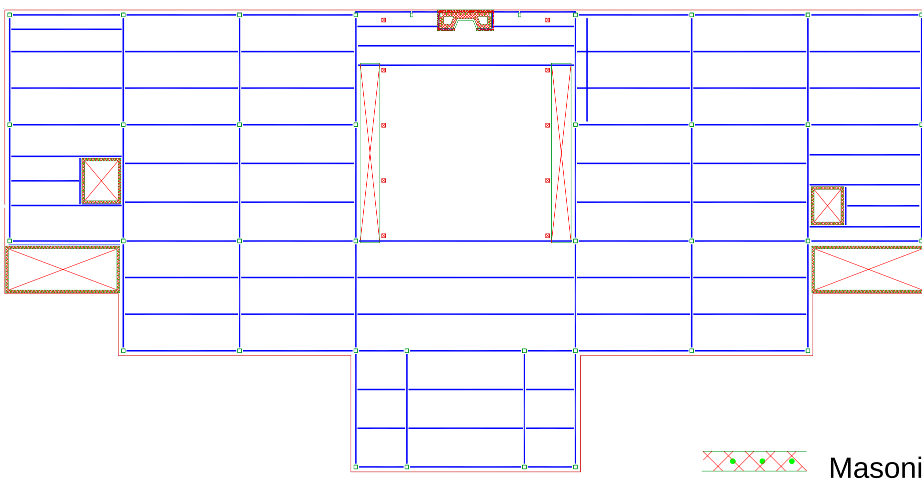
A redesign of typical members used similar loading requirements and a selection of members from AISC Steel Construction Manual, 13th Edition (2005). The design of these typical members resulted in similar selections to those used in the original design and construction. Variations in design can be attributed to variations in modeling of loading conditions and simplification of design, such as the influence of connections and block shear.

Introduction to the Structural System:

Framing Layouts

The structural framing at Boyds Bear Country is primarily a steel grid with wooden roof trusses. The main structural grid is made of standard steel shapes, the yield strengths of which are listed below:

Structural Steel Shapes	Type	[ksi]
Wide Flanges and WTs	ASTM A992	50
Pipe – Type S, Grade B	ASTM A53	35
Tube – Grade B	ASTM A500	46
Plates	ASTM A36	36
Other Shapes	ASTM A36	36
¾" Diameter High Strength Bolts	ASTM A325	n/a



Typical Framing Plan

Typical bays measure ~30'x30' square with W16 beams, framing to W24 girders, which connect to steel tube columns. This framing grid varies around stairwells, elevators, and the front façade projection of the building.

Deeper members are located within the center bay, which features spans of up to 60'. On the central floors, this center bay becomes and atrium flanked on either side by large escalators with primarily decorative wooden framing.

Wooden framing is located in other areas of the building as structural support, primarily in exterior seating areas. Structural joists, girders, and posts are typically designed as No. 2 Southern Pine. All roof framing consists of wooden trusses spaced 2' on center and were manufactured off site of primarily 2x4 No. 2 Southern Pine.

Exterior walls on the ground floor are primarily concrete block, ranging from 8" to 16" thick; both common concrete blocks (1500 psi) and high strength Ivany blocks (2800 psi) are used. These blocks are also used in the construction of walls surrounding stairwells, mechanical rooms, and elevator shafts, some of which act as shear walls in the building. Interior walls throughout, and exterior walls on the upper floors, are cold-formed steel framing sheathed in plywood and gypsum board.

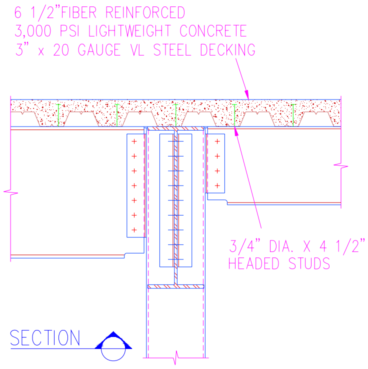
Light gauge steel framing with plywood sheathing[1]

Roof trusses of 2x4 No.2 Southern Pine spaced at 2' OC.[1]

Structural Slabs

Elevated slabs in the building are composite construction. All elevated slabs are supported on 3" x 20 gauge Type VL galvanized steel decking, and the slabs of the main structure are composed of 6½" thick, monofilament synthetic polypropylene fiber reinforced, 3,000 psi lightweight concrete. The slab of the Northeast pavilion / mechanical area is composed of 5½" of normal weight concrete. Secondary reinforcing consists of 6x6-W2.0xW2.0 welded wire mesh in both types of concrete.

Interior floor slabs on grade are 4" thick, monofilament synthetic polypropylene fiber reinforced, 3,000 psi non-air-entrained concrete; with 6x6 W2.0x2.0 WWF on 4" of dense gravel aggregate. Exterior slabs on grade are typically 5" thick, 4,000 psi air-entrained concrete; with 6x6 W2.0x2.0 WWF on 6" of dense graded aggregate.



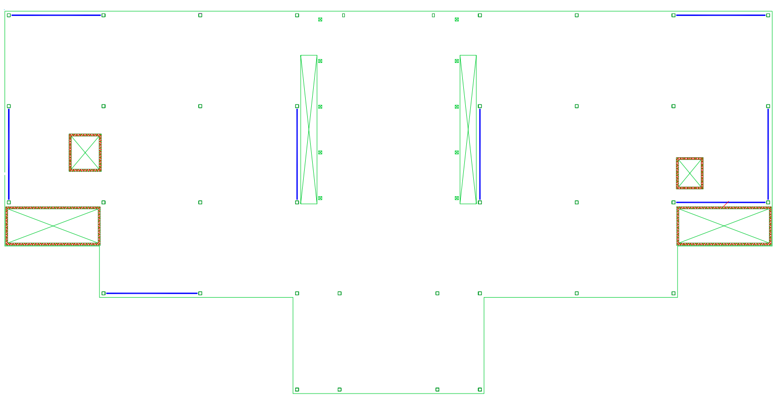
Cross-section of typical slab

Composite slab as placed during construction[1]

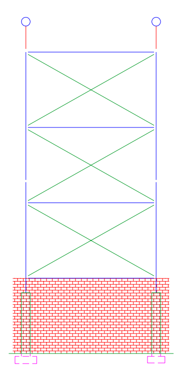
Lateral Resisting System

The lateral resisting system of the building is again a combination of systems. Both concentric braced frames and concrete shear walls were utilized in the structure to absorb wind and seismic loading.

Braced frames are located as shown below, highlighted in blue. These are designed such that each frame in the direction shares and equal amount of the lateral load. Each braced frame is steel with bolted connections, and are surrounded in masonry at the lower level. This wall, made of 2,800 psi Ivany block, incorporates masonry piers to transfer loads to the earth.



Lateral system plan



Typical braced frame elevation

Concrete shear walls can be found highlighted above in red. These shear walls are reinforced masonry and generally run the full height of the building, located around areas of vertical transportation. They are designed as a secondary lateral system, mainly supporting the loads created by the special areas of elevators and stairwells.

Frame with double angle cross-bracing[1]

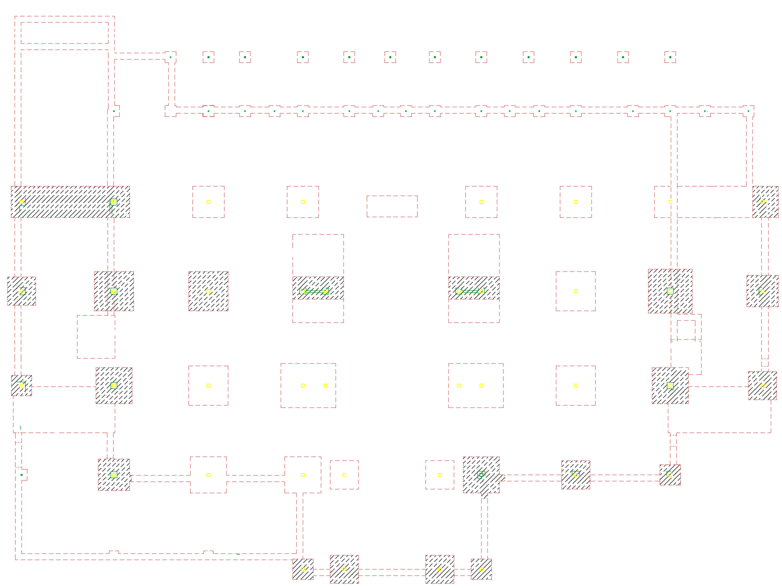
Frame with tube chevron bracing[1]

Foundation System

Foundations of the building consist of shallow footings and piers. All wall footings are simple thickened slabs measuring 2'-0" wide and 1'-0" thick. Column footings extend to a maximum of 3'-0" thick. Piers are located scattered through the foundation, mainly located underneath columns adjacent to bathrooms and mechanical areas.

Foundations of the building are designed with a bearing pressure of 3,000 psf based on geotechnical investigations of the site. Typically, exterior footings extend to 3' below finished grade, to account for frost depths.

Footings and most piers consist of 3,000 psi cast in place concrete with reinforcing billet steel of ASTM A615, grade 60, with class B splices. Masonry piers in the building are constructed of Ivany block. Footings which have a pier located underneath the column are highlighted below.



Expansion Joints

Vertical control joints are located at intersections of reinforced and un-reinforced walls. Control joints in the floors slabs are required at 15'-0" OC in each direction. Control joints are not readily visible in the finished structure as they are covered with hardwood flooring and wooden column covers.

The main building itself, designed as a steel structure with a maximum length of 240', does not feature an expansion joint.

Structural Design Theory

Codes

As Boyds Bear Country was designed in 2003 and 2004, as a follow-up to a similar building in Gettysburg, PA, older southern based codes were used. The codes selected for use in the original design of the building are as follows:

- 1999 Standard Building Code
- 1999 Standard Fire Code
- 1997 Standard Plumbing, Mechanical, and Gas Code
- 2002 National Electric Code
- 2000 International Building Code
- ANSI 1998
- National Design Specifications for Wood Construction

Design Theory and Structural Concepts

The building was designed using older codes as it was a follow-up building to a prototype built in Gettysburg, PA. Changes were made to the original to Pennsylvania design to adapt the building to a new site and Southern building conditions; this is most prominent in the use of the 1999 Standard Building Code its design as opposed to the International Building Code. The auxiliary systems of the building are designed using their respective codes most closely related to the applied Standard Building Code (ie, the 1999 Standard Fire Code and the 1997 Standard Plumbing, Mechanical, and Gas Code). All of these may be seen listed above.

The original structural design references both ASCE 7-95 and ASCE 7-98 within its calculations in addition to the requirements of the Standard Building Code. All of these calculations were completed using Allowable Stress Design methods. Steel members were originally chosen using the requirements prescribed in AISC Manual of Steel Construction, 9th Edition (1989).

The general structural design of the building consists of a loading pattern that transfers as follows:

- Roof loads through plywood sheathing to wooden trusses to walls to foundation.
- Floor loads through composite slabs to steel beams to steel girders to steel columns to foundation.
- Lateral loads through composite floor diaphragm to steel braced frames to masonic piers.

Loading

As the building is home to multiple use spaces, it is also home to multiple loading conditions. Below is a list of those used in the original design of the building as required by the Standard Design Code 1999.

Loading conditions as listed in plans:

Design Roof Loads:		**[psf]**
Minimum Roof Live Load		20
Roof Dead Load (Including structure weight)		20
Ground Snow Load	Pg	15
Flat Roof Snow Load	Pf	15.0
Snow Exposure Factor	Ce	1.0
Snow Load Importance Factor	I	1.0
Thermal Factor	Ct	1.0

Design Wind Loads:		
Basic Wind Speed	V	90 mph
Wind Exposure Category		C
Wind Importance Factor		1.0
Internal Pressure Coefficient	GCpi	± 0.18
Wind Design Pressure		24 psf
Net Uplift Load		10 psf

Design Seismic Loads:		
Seismic Hazard Exposure Group		1
Peak Velocity-Rated Acceleration	Av	0.15
Peak Acceleration	Aa	0.15
Seismic Performance Category		C
Soil Profile Type		S3
Seismic Resisting System: Reinforced Masonry Shear Walls and Concentrically Braced Frames		
Response Modification Factor	R	4.5
Deflection Amplification Factor	Cd	4
Analysis Procedure Used: Equivalent Lateral Force Procedure		

Gravity Loads:

Design Floor Dead Loads:	[psf]
Composite Floor Slab 5½" with 3" Deck	50
Beams / Columns	8
Flooring Finish	1
Ceiling Finish	1
Mechanical / Electrical	5
Total	**65**

Design Roof Dead Loads:	[psf]
Wooden trusses	15
Sheathing and finish	5
Total	**20**

Additional Design Dead Loads:		Unit
Escalators (each)	30	kips
Stairs	100	psf
Elevator	100	psf
Decorative Timbers (each)	10	kips
Fireplace (ground to 2nd floor) (3rd and 4th floors)	150 75	pcf pcf
Exterior Light Gauge Walls	10	psf

Design Floor Live Loads:	[psf]
Retail Areas	100
Office Areas	100
Stairs, including landings, platforms, and exits	100
Light Storage Area	125

More information on specific loading and specific structural systems may be found in later design checks.

Special Load Design Conditions:

Special consideration was taken for additional systems in the original structural design.

This most notably includes the additional loading requirements of Christmas decorations to be hung within the building for several months of the year.

Additional loading was also supplied by the decorative wooden timbers used in the structure. These are spread through several areas of the building but are considered to have the largest impact on the first floor surrounding the atrium. At this point 8 wooden columns are included which run nearly the full height of the building. Each of these timbers is estimated to weigh approximately 10 kips.

The gas fireplace which serves as a focal point on several floors is considered mainly to carry its own weight. It is however considered as a factor in the seismic loading of the building, contributing an additional 470 kips to the weight of the structure.

The escalators which flank either side of the center atrium also impose additional loads to the structure. They are considered to bear on the floor directly below them and are carried primarily by the steel structure adjacent to them. Each escalator measures approximately 45' long and 20' high, and is considered to weigh 30 kips.

Stairwells and elevators are designed as would be in any typical building at 100 psf.

Design Check of Typical Structural Elements:

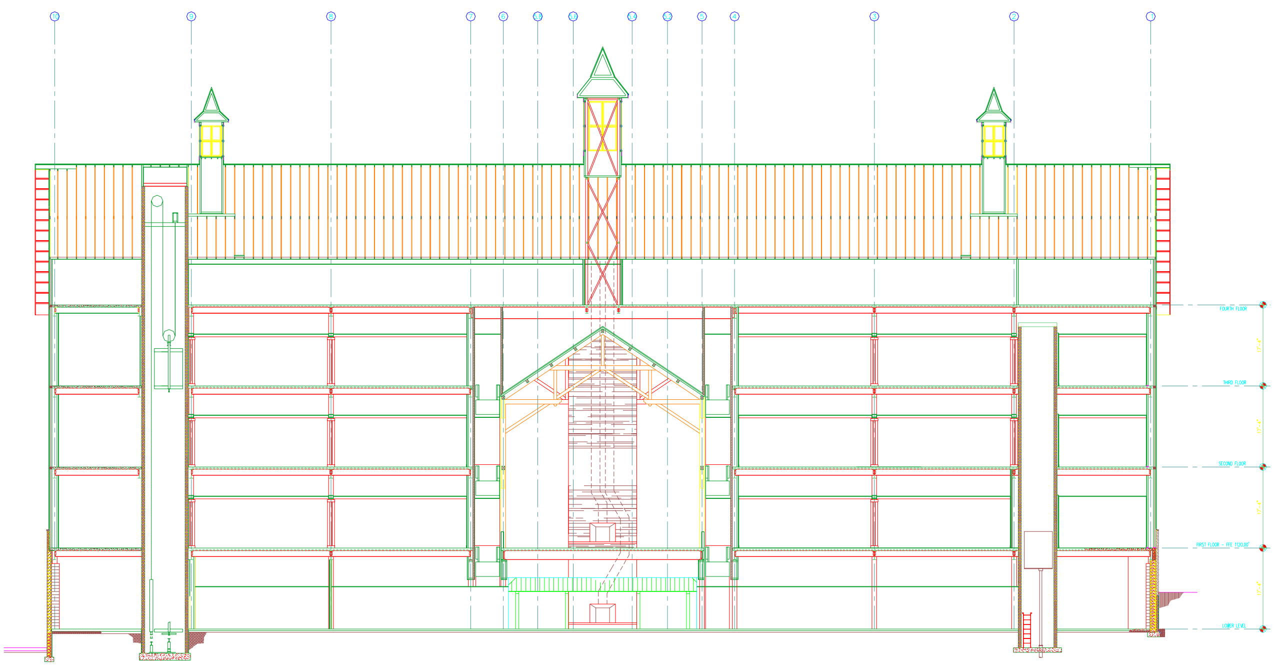
Please note that a full version of these calculations may be found in the appendix to this report.

A design check of a typical beam was completed, resulting in the same selection of W16x31's as used in the original design. The new calculation uses loads equal to those in the original, however differs by using the composite construction tables of AISC Steel Construction Manual, 13^{th} Edition (2005), as opposed to the AISC Manual of Steel Construction, 9th Edition (1989). Both designs use Allowable Stress Design.

A design check of a typical girder resulted in a selection similar to the girders used in construction. The first design performed used a point load at the placement of beams on the girder. This resulted in an overall moment of 742.5 'k, and by using the composite construction tables as with the typical beam design, resulted in the selection of a W24x84. A second typical design was then performed in an attempt to match the selected beam more closely to the original design of a W24x30. This second design was performed with loading modeled as a distributed load, meaning that the live and dead loads of the tributary floor area were simply applied to the girder evenly. This design resulted in the selection of a W24x30 girder, the same shape chosen for construction.

A typical column design also yielded a result similar to typical columns used in the building's construction. The initial column selection was one size larger than the one chosen for construction; an HSS14x14x5/8 as opposed to the HSS12x12x5/8's selected for construction. The difference in member selection can most likely be attributed to an alteration in kL, or effective column height. When the effective height of the column was reduced from 17.3' to 16', the same size column was selected as used in construction. This lower effective length can be attributed to the depth of floor slabs and connections.

The design of the cross braces results in the selection of smaller members than used in construction. This can be accounted for as there was no added consideration applied to the design for connection strength and issues such as block shear. The design selected a pair of 4x4x¾ angles, where as in construction LL4x4x5/8's and LL6x6x5/8's were used.



Building Section

Wind Loading Calculations – Please see initial calculations in appendix

Wind Loading Velocity:		Kz	q_z [psf]
Dir	Height [ft] ($q_z = 0.00256K_zK_{zt}K_dV^2I$)		
1:	**Windward Wall:**		
	0-15	0.85	15.21
	20	0.90	16.10
	25	0.94	16.82
	30	0.98	17.53
	40	1.04	18.61
	50	1.09	19.50
	60	1.13	20.22
	70	1.17	20.93
	80	1.21	21.65
	90	1.24	22.18
	100	1.26	22.54
	120	1.31	23.44
1:	**Leeward Wall** (all heights):	1.31	23.44

Wind Loading Windward Wall:		q_z	p [psf]
Dir	Height [ft] (p = qGp-qiGCpi)		
1:	**Windward Wall:** (Cp = 0.8)		
	0-15	15.21	10.34
	20	16.10	10.95
	25	16.82	11.44
	30	17.53	11.92
	40	18.61	12.65
	50	19.50	13.26
	60	20.22	13.75
	70	20.93	14.23
	80	21.65	14.722
	90	22.18	15.08
	100	22.54	15.33
	120	23.44	15.94
1:	**Leeward Wall** (all heights)**:** (Cp = 0.5)	23.44	9.97
2:	**Leeward Wall** (all heights)**:** (Cp = 0.3)	23.44	5.98
1:	**Roof:**		
	Windward (Cp = 0.2)	23.44	3.98
	Leeward (Cp = -0.6)	23.44	11.95
2:	**Roof:**		
	0-84’ (Cp = -0.9)	23.44	17.93
	84’-168’ (Cp = -.05)	23.44	9.96
	168’-240’ (Cp = -0.3)	23.44	5.98

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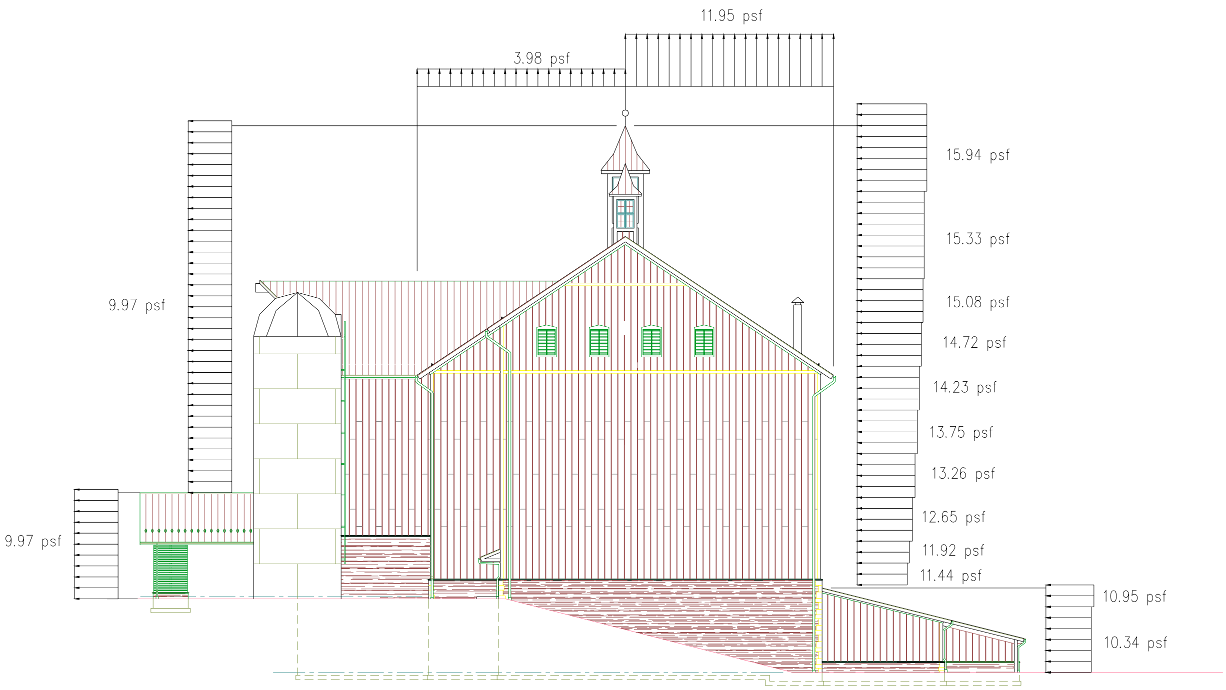
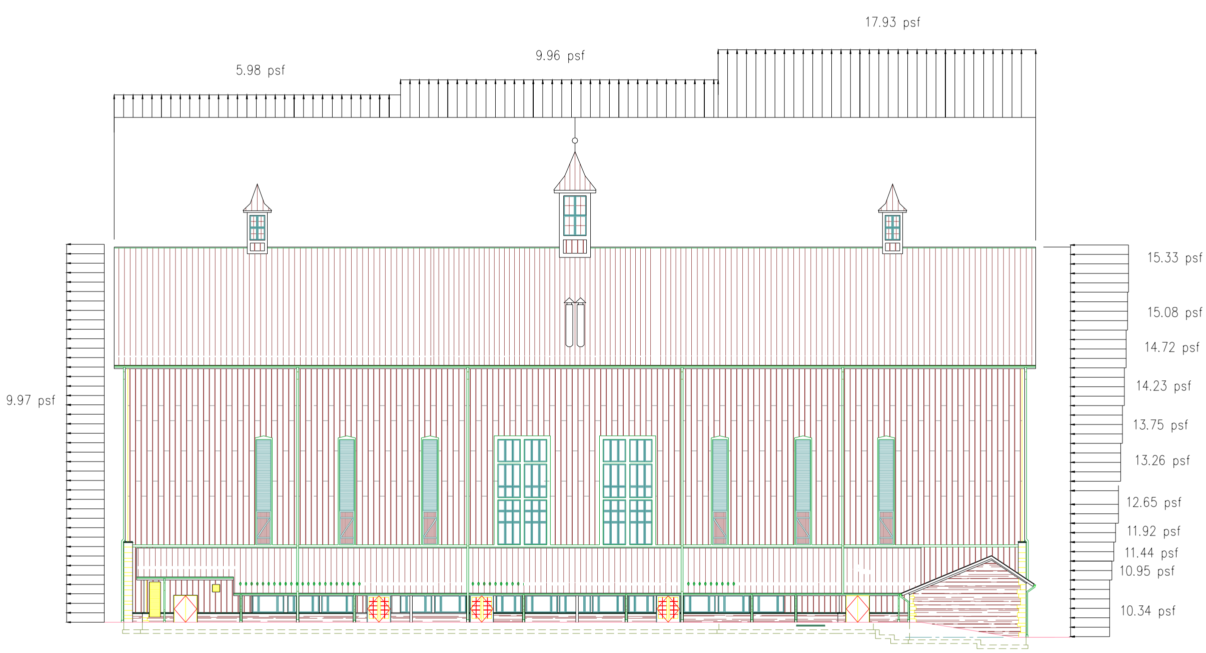


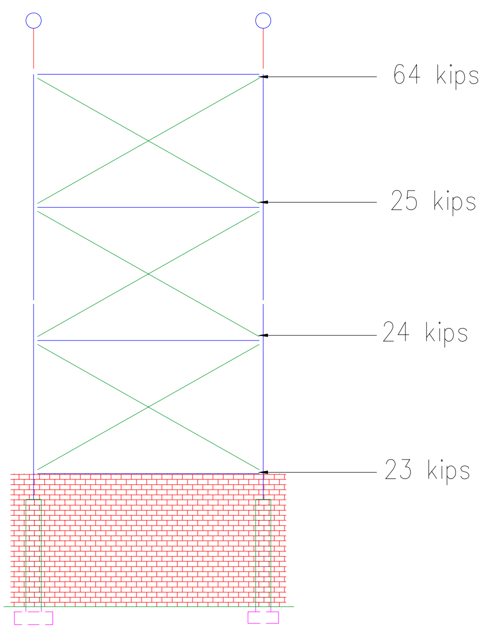
Loading Direction 1



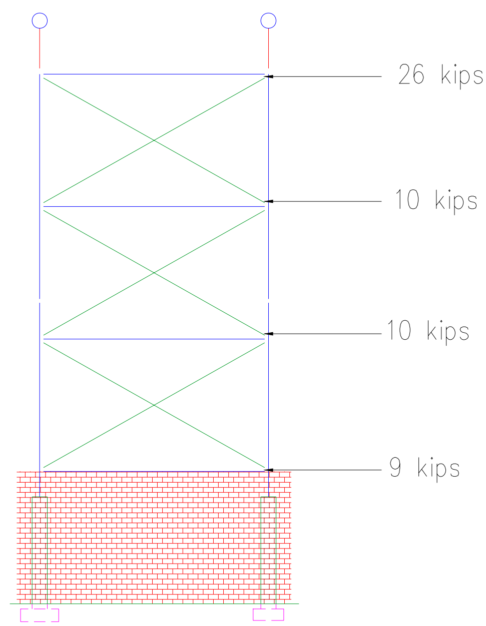
Loading Direction 2

Vertical Wind Distribution:	Area [sf]	V [kips]
Loading Pattern 1		
1st floor	4325	92
2nd floor	4325	97
3rd floor	4325	101
4th floor	7700	256
Total (Base)	**25000**	**546**
Loading Pattern 2		
1st floor	1730	37
2nd floor	1730	39
3rd floor	1730	40
4th floor	3080	102
Total (Base)	**10000**	**218**

It is assumed that each braced frame carries an equal portion of the lateral load applied in the direction of the braced frame. The effects of shear walls are neglected.



Wind loading pattern 1



Wind loading pattern 2

Seismic Loading Calculations – Please see initial calculations in appendix

Note: Elevated floors are denoted as 1^{st}-4^{th}, while the partially exposed floor is denoted as the ground floor.

Building Dead Loads:	**w [psf]**	**Area [sf]**	**Load [k]**
- **Floor**			
1^{st} floor	65	20886	1358
2^{nd} floor	65	18182	1182
3^{rd} floor	65	18182	1182
4^{th} floor	65	16527	1057
- **Partitions**			
1^{st} floor	20	20886	418
2^{nd} floor	20	18182	364
3^{rd} floor	20	18182	364
4^{th} floor	20	16527	331
- **Roof**	20	24285	486
- **Escalator** (30 k each)			
1^{st} floor	-	-	60
2^{nd} floor	-	-	60
3^{rd} floor	-	-	60
- **Stairwell**			
1^{st} floor	100	706	71
2^{nd} floor	100	706	71
3^{rd} floor	100	706	71
4^{th} floor	100	706	71
- **Elevator**			
1^{st} floor	100	1384	138
2^{nd} floor	100	1384	138
3^{rd} floor	100	1384	138
4^{th} floor	100	1384	138
- **Timber Posts** (10 k each)			
1^{st} floor	-	-	80
- **Fireplace**			
1^{st} floor	2595	78	202
2^{nd} floor	2595	78	202
3^{rd} floor	75	468	33
4^{th} floor	75	468	33
- **Exterior Walls**			
1^{st} floor	10	12387	124
2^{nd} floor	10	12387	124
3^{rd} floor	10	12387	124
4^{th} floor	10	12387	124
		floor area	weight
- **Totals**		**107514**	**8804**

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Note: Calculated floor area is ~5000 sf less than the total area. This can be accounted for in the additional floor area within the ground floor for storage and seating.

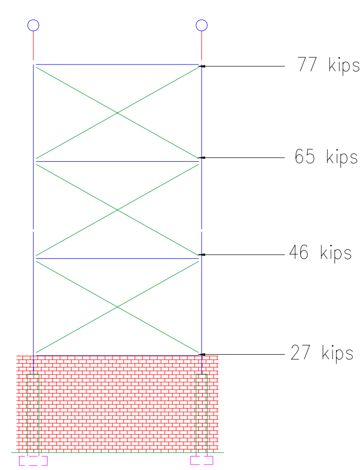
Building Live Load by Floor:		w [psf]	Area [sf]	Load [k]
-	**Floor** @ 25%			
	1st floor	25	20886	523
	2nd floor	25	18182	455
	3rd floor	25	18182	455
	4th floor	25	16527	414
-	**Roof** @ 20%	1.12	24285	38
-	**Totals**			**1885**

Building Weight by Floor with Live Load:	[kips]
1st floor	2869
2nd floor	2505
3rd floor	2336
4th floor	2085
Roof	524
Total	**10319**

Base Shear: V = CsW
= (0.083)(10319 k)
= 856.5 k

Vertical Seismic Distribution:	w_xh_x	C_{vx}	V [kips]
1st floor	49634	0.124	106
2nd floor	86673	0.216	185
3rd floor	121238	0.302	259
4th floor	144282	0.359	307
Total (Base)	**401827**	**1.001**	**857**

Note: The seismic loading will control the design of the braced frames, as it induces higher loads than the wind loading.



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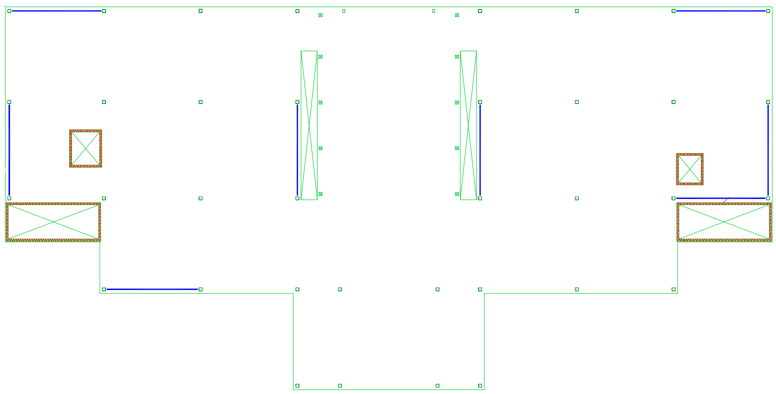
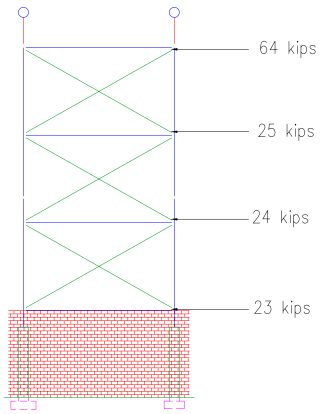
Design of a Typical Braced Frame:



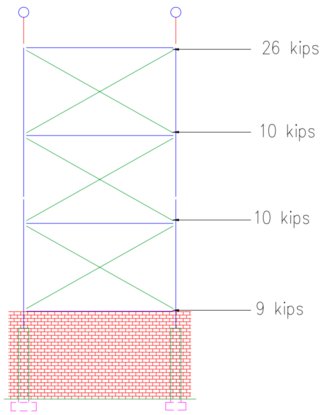
Lateral system plan

Each direction of lateral load resistance features 4 braced frames as well as concrete shear walls. For the requirements of this design, only the resistance of the braced frames will be determined, resulting in a conservative outcome. It will be assumed that each of the four frames will carry an equal portion of the load as they were all constructed in a similar fashion and for simplicity.

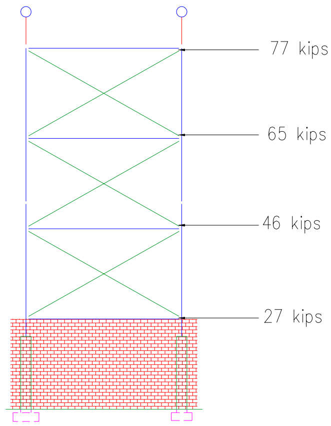
A diagram of a typical braced frame loaded with wind and seismic floor forces follows:



Wind pattern 1



Wind pattern 2



Seismic

Seismic loading will control design in all directions.

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Conclusions:

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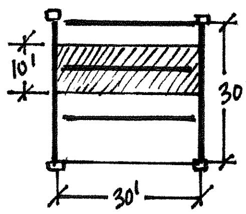
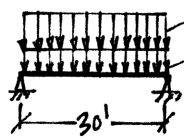
Citation Note: [1] All photographs c/o Kinsley Construction.
[2] All photographs by Lauren Wilke.

PENN STATE UNIVERSITY

CLASS: AE 481 W
DATE: 10-03-06
ASSIGNMENT: TECH. REPORT #1
TYPICAL BEAM DESIGN.
PAGE: ____ of ____

TYPICAL FLOOR BEAM DESIGN.





100 psf LL. = 1000 plf LL
65 psf DL = 650 plf DL

1650 plf TL.

$V = \frac{wl}{2} = \frac{(1650\ plf)(30')}{2} = 24750\ plf = 24.75\ klf.$

$M = \frac{wl^2}{8} = \frac{(1650\ plf)(30')^2}{8} = 185625\ plf = 185.63\ 'k.$

$Y_{CON} = 5.5''$ (OVERALL HEIGHT OF CONCRETE/DECK SYSTEM)

ASSUME $a = 1''$

$Y_2 = Y_{CON} - a/2 = 5.5'' - 1''/2 = 5''$

USING STEEL CONSTRUCTION MANUAL, 13th EDITION, ASD.

[TABLE 3-19.]

@ $Y_2 = 5''$ TRY W14x34 $M = 197\ 'k \geq 186\ 'k$ OK

PNA = 7

$\Sigma Q_n = 111\ k$

$b_{eff} = 1/4$ SPAN $= (1/4)(30') = 7.5'$ ← CONTROLS USE $b_{eff} = 7.5' = 90''$

$= $ SPACING $= (10') = 10'$

$a = \frac{\Sigma Q_n}{0.85 f'_c b} = \frac{111\ k}{(0.85)(3\ ksi)(90'')} = 0.484'' < 1.0''$ OK.

TRY W16x31 $M = 197\ 'k \geq 186\ 'k$ OK.

PNA = 7

$\Sigma Q_n = 114\ k$

$a = \frac{114k}{(0.85)(3ksi)(90'')} = 0.497'' < 1.0''$ OK.

[TABLE 3-21]

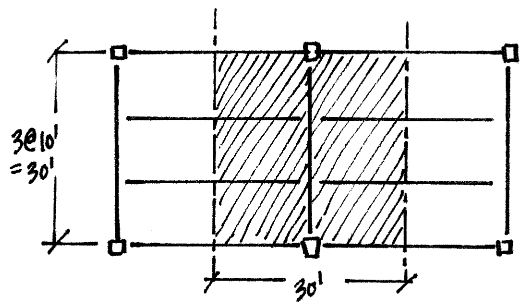
DECK ⊥, 1 WEAK STUD PER RIB, 3/4" ϕ, LIGHTWEIGHT CONCRETE, $f'_c = 3000$ psi.

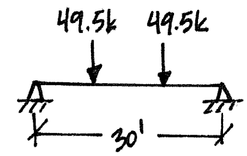
$Q_n = 17.2k$

W14x34: $\frac{\Sigma Q_n}{Q_n} = \frac{111\ k}{17.2k} = 6.43 \rightarrow 7$ W16x31: $\frac{\Sigma Q_n}{Q_n} = \frac{114k}{17.2k} = 6.63 \rightarrow 7$

$\boxed{\text{USE W16x31 W/ 14 STUDS}}$ *SAME AS ORIGINAL DESIGN!

TYPICAL GIRDER DESIGN.





$P = (2)(V_B)$
$= (2)(24.75k)$
$= 49.5k$

$V = \frac{P}{l}(l - a + b) = \frac{(49.5k)(30' - 10' + 10')}{30'} = 49.5k$

$M = \frac{Pl}{2} = \frac{(49.5k)(30')}{2} = 742.5'k.$

$Y_{con} = 5.5"$
ASSUME $a = 1"$
$Y_2 = Y_{con} - a/2 = 5.5" - 1"/2 = 5"$

USING STEEL CONSTRUCTION MANUAL, 13th EDITION, ASD.
[TABLE 3-19]

@$Y_2 = 5"$ TRY W24x68 $M = 777'k \geq 742.5'k$ OK
PNA = 3
$\Sigma Q_n = 741k$

$b_{eff} = 1/4$ SPAN $= (1/4)(30') = 7.5'$ ← CONTROLS
= SPACING = 30'
USE $b_{eff} = 7.5' = 90"$

$a = \frac{\Sigma Q_n}{0.85 f'_c b} = \frac{741k}{(.85)(3ksi)(90")} = 3.23" \not\leq 1.0"$ FAILS.

ASSUME $a = 3"$
$Y_2 = Y_{con} - a/2 = 5.5" - 3"/2 = 4"$
@$Y_2 = 4"$ ~~TRY~~
ASSUME $a = 4"$
$Y_2 = Y_{con} - a/2 = 5.5" - 4"/2 = 3.5"$
@$Y_2 = 3.5"$ TRY W24x68 $M = 769'k \geq 742.5'k$
PNA = TFL
$\Sigma Q_n = 1000k$

$a = \frac{1000k}{(0.85)(3ksi)(90")} = 4.36" \not\leq 4.0"$ FAILS.

TYPICAL GIRDER DESIGN CONT.

TRY W24x76 M = 743 k ≥ 742.5 k OK. $Y_2 = 5"$
PNA = 6
$\Sigma Q_n = 393$ k

$$a = \frac{393k}{(0.85)(3ksi)(90")} = 1.71" \nleq 1.0" \text{ FAILS.}$$

TRY W24x84 M = 756k ≥ 742.5k OK $Y_2 = 4"$
PNA = 7
$\Sigma Q_n = \cancel{303}\ 309k$

$$a = \frac{309k}{(0.85)(3ksi)(90")} = 1.35" \leq 2.0" \text{ OK}$$

[TABLE 3-21]
DECK 11, $\frac{w_r}{h_r} = \frac{4.75"}{3"} = 1.58$, 3/4"$\phi$, LIGHTWEIGHT CONCRETE, $f'_c = 3000$ psi
$Q_n = 17.1$ k
W24x84: $\frac{\Sigma Q_n}{Q_n} = \frac{309k}{17.1k} = 18.07 \rightarrow 19$

$\boxed{\text{USE W24x84 w/ 38 3/4" STUDS.}}$

NOTE: DIFFERS FROM ORIGINAL.
SEE ALTERNATE METHOD ON NEXT PAGE.

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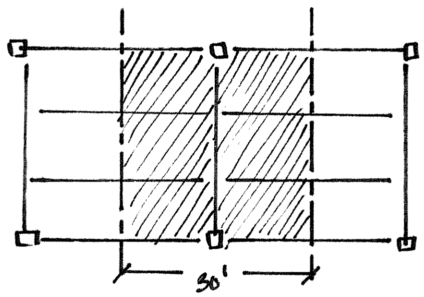
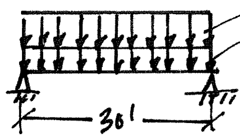
TYPICAL GIRDER DESIGN CONT.





100 psf LL = 3000 psf LL
65 psf DL = 1950 psf DL
4950 psf TL.

$V = \frac{wl}{2} = \frac{(4.95klf)(30')}{2} = 74.25\ k$

$M = \frac{wl^2}{8} = \frac{(4.95klf)(30')^2}{8} = 557\ 'k.$

$Y_{CON} = 5.5"$
ASSUME $a = 1"$
$Y_2 = 5.5" - 1"/2 = 5"$

[TABLE 3-19.]

@ $Y_2 = 5"$ TRY W24×68. $M = 614'k \geq 557'k$ OK
$PNA = 7$
$\Sigma Q_n = 251\ k$

$a = \frac{251k}{(0.85)(3ksi)(90")} = 1.09"$

@ $Y_2 = 4.5"$ $M = 608'k \geq 557'k$ OK
$PNA = 7$
$\Sigma Q_n = 251\ k$
$a = 1.09" \ngeq 2"$ OK

[TABLE 3-21]

$Q_n = 17.1k$ $\frac{251k}{17.1k} = 14.67 \rightarrow 15$ STUDS.

$\boxed{\text{USE W24×68 W/ 30 3/4" } \phi \text{ STUDS.}}$

*SAME AS ORIGINAL DESIGN!

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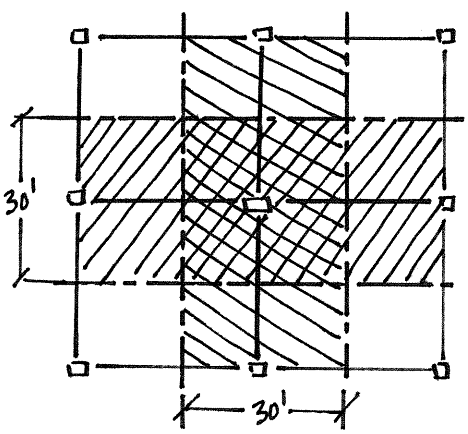
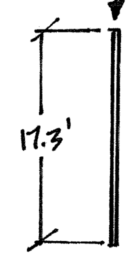
TYPICAL COLUMN DESIGN.





P = (30')(30')(100 psf LL + 65 psf DL)
= 148.5k / floor.
(30')(30')(20 psf LL + 20 psf DL)
= 36k / roof.
@ GROUND LEVEL:
$\Sigma P = 36k + (4)(148.5k)$
$= 630k$

ALLOWABLE COMPRESSION =
(0.6)(46 ksi) = 27.6 ksi ← CONTROL
(0.5)(58 ksi) = 29 ksi

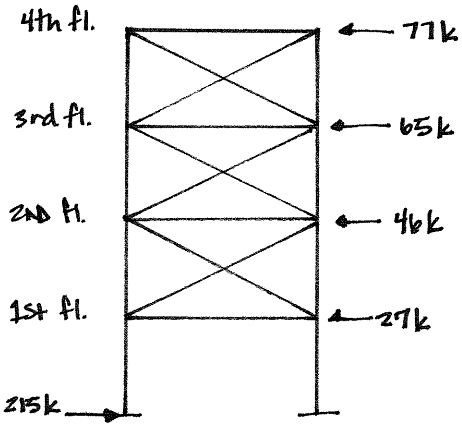
$A_{RQD} = \frac{P}{\sigma} = \frac{630k}{27.6k} = 22.8\ in^2$

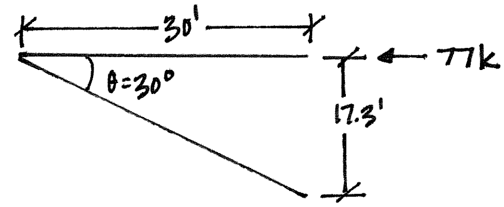
[TBL 4-4]

- TRY HSS 14x14x5/8 — $A = 30.3\ in^2 > 22.8\ in^2$ OK; $P = 760k$ @ 17' > 630 k OK
- TRY HSS 12x12x5/8 — $A = 25.7\ in^2 > 22.8\ in^2$ OK; $P = 630k$ @ 16' = 630 k OK

CHANGE IN KL DUE TO DEPTH OF FLOOR SLAB & CONNECTION LENGTH.

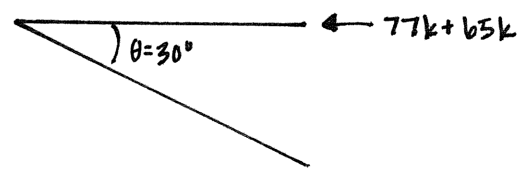
TYPICAL BRACED FRAME DESIGN.



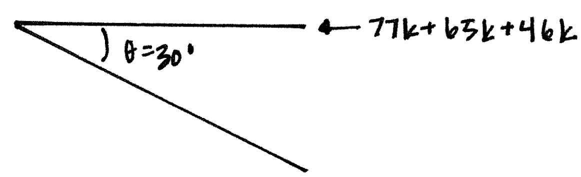


BRACING MEMBER:

$P = \frac{77k}{\cos 30°} = 88.9k.$



$P = \frac{(77k+65k)}{\cos 30°} = 164\ k$



$P = \frac{(77k+65k+46k)}{\cos 30°} = 217.1k$

DESIGN BRACING BASED ON
MAXIMUM TENSION LOAD = 217 k.

ALLOWABLE TENSION =

$(0.6)(36ksi) = 21.6ksi$ ← CONTROL.

$(0.5)(58ksi) = 29\ ksi$

$A_{REQD} = \frac{P}{\sigma} = \frac{217k}{21.6ksi} = 10.05\ in^2$

[TBL. 5-?]

TRY LL 3½x3½x½ $A = (2)(3.25 in^2)$
$= 6.5 in^2 \ngtr 6.03 in^2$ FAILS.

TRY LL 4x3x½ $A = (2)(3.25 in^2)$
$= 6.5 in^2 \ngtr 6.03 in^2$ FAILS.

[TBL. 5-8]

TRY LL 4x4x3/4 $A = 10.9\ in^2 \geq 10.05\ in^2$ OK
$P = 233\ k \geq 217.1\ k$ OK

$\boxed{\text{USE LL 4x4x3/4 BRACES}}$

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SNOW LOADING – DESIGNED W/ SBC 1999 & ASCE 7-95.

[ASCE 7-95 SECTION 7.0]

[FIGURE 7.2]

$P_g = 15$ psf

[SECTION 7.3]

$P_f = 0.7 C_e C_t I P_g$

$= (0.7)(1.0)(1.0)(1.0)(15\text{ psf})$

$= 10.5$ psf

[SECTION 7.4] [FIGURE 7-2a] WARM ROOF, $\theta = 33.7°$, UNOBSTRUCTED...

$P_s = C_s P_f$ $\quad C_s = 0.53$

$= (0.53)(10.5\text{psf})$

$= 5.6$ psf

[SECTION 7.6.1]

UNBALANCED SNOW LOADS FOR HIP & GABLE ROOFS.

$P_u = \frac{1.3 P_s}{C_e}$

$= \frac{(1.3)(5.6\text{psf})}{1.0} =$

$= 7.3$ psf

$P_b = P_s = 10.5$ psf

[SECTION 7.7.2]

DRIFTS ON LOWER ROOF

$\gamma = 0.13 p_g + 14 < 30$ psf

$= (0.13)(15\text{psf}) + 14$

$= 16$ psf < 30 psf OK.

FOR ROOF STRUCTURE @ COLUMN A TO COLUMN A.2

$l_{UPPER} = 87'$ COLUMN D TO A

$l_{LOWER} = 46'$ COLUMN A TO A-2.

$h = 80'$ PEAK TO PEAK.

LEEWARD DRIFT

[FIGURE 7.9]

$h_d = 0.43 \sqrt[3]{l_u}\ \sqrt[4]{P_g + 10'} - 1.5$

$= (0.43)\sqrt[3]{87'}\ \sqrt[4]{15\text{psf} + 10\text{psf}'} - 1.5$

$= 2.76'$

$h_b = \frac{P_f}{\gamma} = \frac{10.5\text{ psf}}{16\text{pcf}} = 0.66'$

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SNOW LOADING – DESIGNED W/ SBC 1999 CONT'

$h_c = h - h_b = 80' - 0.66' = 79.34'$

WINDWARD DRIFT.

$h_d = 0.5 h_d$ $\quad l_{LOWER} = 46'$

$= (0.5)(1.8')$

$= 0.9'$ DNC.

$w = 4 h_d$ $\quad [h_d > h_c]$

$= (4)(2.8')$

$= 11.2'$

$p_d = h_d \gamma$

$= (2.8')(16\ pcf)$

$= 45\ psf$

[SECTION 7.9]

SLIDING SNOW:

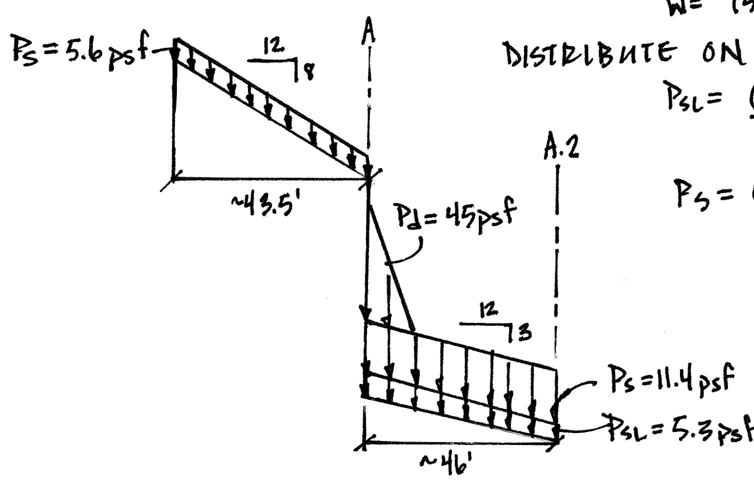
STRIP ON UPPER ROOF:

$W = (5.6\,psf)(1')(43.5') = 243.6\ lbs$

DISTRIBUTE ON LOWER ROOF:

$P_{SL} = \dfrac{(243.6\,lbs)}{(1')(46')} = 5.30\ psf$

$P_S = C_S P_f = (0.85)(10.5\,psf) = 11.4\ psf.$



TOTAL LOAD ON LOWER ROOF

$P_T = P_{SL} + P_S + \cancel{P_d}$

$= 5.3\ psf + 11.4\ psf$

$= 16.7\ psf$

IMPACT FACTOR:

$P_T = (1.5)(16.7\ psf)$

$= 19.4\ psf$ OVER LOWER ROOF SPAN.

BE SURE TO INCLUDE DRIFT.

WIND LOADING - DESIGNED W/ SBC 1999.

[SBC 1606.1.1] WIND FORCES ON EVERY BUILDING OR STRUCTURE SHALL BE DETERMINED BY THE PROVISIONS OF ASCE 7.

[ASCE 7-98 SECTION 6.0]

$V = 90$ MPH

$K_d = 0.85$

$I = 1.0$

EXPOSURE CATEGORY = C

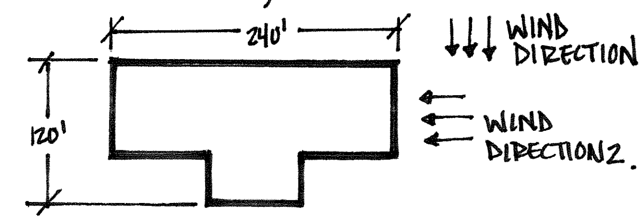
$h = 84'-0"$ (MEAN ROOF HEIGHT)

$\theta = 33.7°$

$K_h = 1.22$ (@ MEAN ROOF HEIGHT) FOR OTHERS SEE NEXT PAGE.

$G = 0.85$

OVERALL BUILDING DIMENSIONS:



1: $\frac{L}{B} = \frac{120'}{240'} = 0.5$ 2: $\frac{L}{B} = \frac{240'}{120'} = 2.0$

$\frac{h}{L} = \frac{84'}{120'} = 0.70$ $\frac{h}{L} = \frac{84'}{240} = 0.35$

[FIGURE 6-6]

$C_p = 0.8$ WINDWARD WALL

1: $C_p = -0.5$ LEEWARD WALL

2: $C_p = -0.3$ LEEWARD WALL

$C_p = -0.7$ SIDE WALL

1: $C_p = \pm 0.2$ ROOF

1: $C_p = -0.6$ LEEWARD ROOF

$GC_{pi} = \pm 0.18$

$C_f = 1.2$.

K_{zt} = TOPOGRAPHIC FACTOR
$= (1 + K_1 K_2 K_3)^2$

2: $C_p = -0.9$ ROOF 0-84'
$= -0.5$ ROOF 84'-168'
$= -0.3$ ROOF 168'-240'

[FIGURE 6-2]

CONDITION = 2-D ESCARPMENT

$\frac{H}{L_h} = \frac{19.5'}{75'} = 0.26 \rightarrow K_1 = 0.22$

$\frac{x}{L_h} = \frac{0'}{75'} = 0 \rightarrow K_2 = 1.0$

$\frac{z}{L_h} = \frac{103.5'}{75'} = 1.38 \rightarrow K_3 = 0.034$

} DATA PROVIDED BY CIVIL ENG.

$K_{zt} = [1 + (0.22)(1.0)(0.034)]^2 = 1.015$

$q_z = 0.00256 K_z K_{zt} K_d V^2 I$
$= (0.00256)(1.22)(1.015)(0.85)(90 \text{ MPH})^2(1.0)$
$= 21.8$ psf. @ MEAN ROOF HEIGHT

SEE OTHER ATTACHED.

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WIND LOADING - DESIGNED W/ SBC 1999 CONT'.

$$\left.\begin{aligned} P_{WINDWARD} &= qGC_p - q_i(GC_{pi}) \\ &= (21.8\ psf)(0.85)(0.8) - 0 \\ &= 14.8\ psf \\ P_{LEEWARD} &= (21.8\ psf)(0.85)(-0.5) - 0 \\ &= -9.27\ psf \end{aligned}\right\} \text{@ MEAN ROOF HEIGHT.}$$

$$P_{TOTAL} = 14.8\ psf + 9.27\ psf$$

$$= 24.07\ psf \quad \text{(AT MEAN ROOF HEIGHT).}$$

$$F = (24.07\ psf)(240')(97') = 560\ k$$

APPROX. VALUE @ ~~LARGEST~~ FACE.

SEISMIC LOADING - DESIGNED WITH SBC 1999.

[TABLE 1607.66]

EXPOSURE GROUP = I

[FIGURE 1607.1.5A]

$A_v = 0.15$

$A_a = 0.15$

[TABLE 1607.1.8]

PERFORMANCE CATEGORY = C

[TABLE 1607.3.1]

SITE COEFFICIENT, $S_s = 1.5$

[TABLE 1607.3.3]

$R = 4\frac{1}{2}$ FOR MASONRY SHEAR WALLS ← USE CONSERVATIVELY.

$R = 3\frac{1}{4}$ FOR CONCENTRIC BRACED FRAMES DNC.

[SECTION 1607.4]

EQUIVALENT LATERAL FORCE PROCEDURE.

BUILDING PERIOD:

$T = C_T h_n^{3/4}$

$C_T = 0.02$ FOR SHEARWALLS

$h_n = 75.4'$

$T = (0.02)(75.4')^{3/4} = 0.51$

SEISMIC COEFFICIENT:

$C_s = \frac{1.2 A_v S}{R T^{2/3}} = \frac{(1.2)(0.15)(1.5)}{(4.5)(0.51)^{2/3}} = 0.094$ DNC.

$C_s < \frac{2.5 A_a}{R} = \frac{(2.5)(0.15)}{(4.5)} = 0.083$ ← CONTROLS.

BASE SHEAR:

$V = C_s W$

$= (0.083)(10{,}319\ k)$

$= 856.5k$

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SEISMIC LOADING - DESIGNED W/ SBC 1999 CONT'.

W = TOTAL LOAD OF BUILDING:

FLOOR = 65 psf
1ST FLOOR = (65 psf)(20886 sf) = 1358 k
2ND FLOOR = (65 psf)(18182 sf) = 1182 k
3RD FLOOR = (65 psf)(18182 sf) = 1182 k
4TH FLOOR = (65 psf)(16257 sf) = 1057 k

ROOF = 20 psf
ROOF = (20 psf)(24275 sf) = 486 k.

ESCALATORS = 30k EACH
TOTAL ESCALATOR LOAD = (6)(30k) = 180k.

STAIRWELL = 100 psf
EACH FLOOR = (353 sf)(2)(100 psf) = [71k]
TOTAL STAIRWELL = (4)(71k) = 284k.

TIMBER FRAMING = 10 k EACH
TOTAL TIMBER LOAD = (8)(10k) = 80k.

FIREPLACE 150 pcf / 75 psf
1ST & 2ND FL = (150 pcf)(14.7')(5.3')(17.3') = 202k
= 202k
3RD & 4TH FL = (75 psf)(14.7'+5.3'+5.3')(17.3') = 33k
= 33k.

EXTERIOR WALLS = 10 psf
TOTAL EXT. WALL = (10 psf)(716')(69') = 494 k.

ELEVATOR = 100 psf
TOTAL ELEVATOR = (100 psf) = 552k
(80')(69')

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SEISMIC LOADING - DESIGNED W/ SBC 1999. CONT.

$C_{vx} = \dfrac{W_x h_x^k}{\sum_1 w_i h_i^k}$

$k = 1$

$T = 0.51$

$h = 17.3'$

$\sum_1 W_x = 10319\ k$

$\sum_1 h_x = 69.2'$

$\sum_1 W_x h_x = 714075\ 'k$

SEE SPREAD SHEET
FOR DISTRIBUTION.