Lauren Wilke
Structural Option
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Boyds Bear Country
Pigeon Forge, TN

Technical Report 3
Lateral System Analysis and Confirmation Design

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.

Boyds Bear Country is a building constructed on many materials. The lateral system of the building is no exception and this report investigates the specifics of this system.

Primarily, lateral loads are resisted by a series of 8 concentric steel braced frames, 4 in each direction of the structure. Five of theses eight frames also incorporate masonry piers and masonry walls within the lowest story. In both types of frame, lateral forces are finally transferred to the foundation of the building in both strip and spread footings.

Loads applied to the building in the redesign follow IBC 2003, ASCE 07-05 and Allowable Stress Design. The controlling load case for this design was of dead load, live load, and earthquake loads. This loading case, among others was investigated using RAM Structural System and Risa-3D.

The original design of the building did not consider the relative stiffness of frames and distributed lateral loads equally to each frame. The redesign of the structure considered relative stiffness differences in the frames as well as adjustments in loading as required by current codes. Under these changed conditions, the original design of the building met code requirements and industry standards as built, including a control value of H/400 for drift.

Overturning and torsion as created by the lateral loads on the building do not greatly influence its design. Overturning moments and induced couples are easily resisted by the spread footings below each frame, and are reduced by the weight of the structure and most notably the weight and strength of the masonry portions of the frames. Torsion on the building is negligible when compared to base shear and story shear created by the same loads.

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Table of Contents

Introduction to the Lateral System

Original drawings for Boyds Bear Country in Pigeon Forge, Tennessee call out two lateral systems, that of masonry shear walls and steel braced frames. In specific study of the design documents, it can be found that the primary lateral resistance system is concentric steel braced frames. A secondary masonry lateral resistance system can be found in 5 of the 8 braced frames in the building. These frames sit on either one or two masonry piers which are incorporated within reinforced block walls. All eight of these frames can be seen, highlighted in red, in figure 3.01.

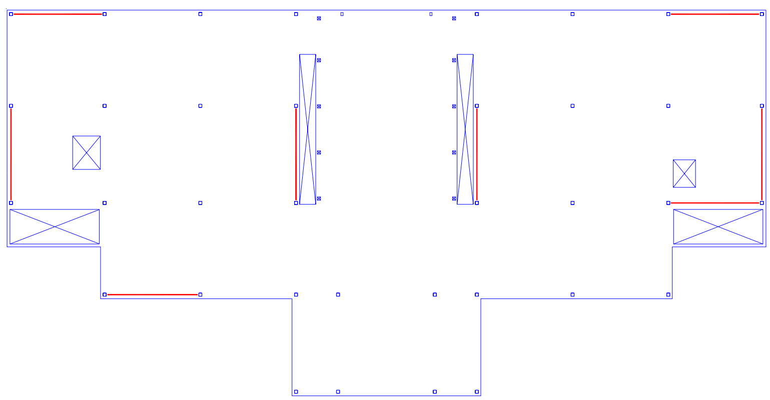

Figure 3.01: Lateral system plan

The floor system of composite steel beams, girders, and concrete deck acts as a diaphragm, transferring lateral forces to the frames at each of four elevated floor levels. Both wind and seismic forces are imparted as lateral loads on the structure, and through design calculations included in this report, it is found that seismic forces control the design of the structure.

Structural Design Theory

Design Theory and Applied Codes:

The building was originally designed in 2003 and 2004 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 to a new site and Southern building conditions; this is most prominent in the use of the 1999 Standard Building Code in 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).

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, and steel members were originally chosen using AISC Manual of Steel Construction, 9th Edition (1989).

For the purposes of this redesign, current codes are applied, including the 2003 International Building Code and ASCE 7-05. Both the original member selections and the current member selections are chosen using Allowable Stress Design requirements for uniformity.

Load combinations applied to the design are as follows:

Dead Load + Live Load + Earthquake Load
Dead Load + Earthquake Load
Dead Load + Live Load
Earthquake Load

Of which, the first controls, and is applied as so in the following calculations.

Building Loads

Gravity Loading:

Gravity loads are applied as required by current codes. Changes between present codes and those as applied to the building are not significant, and as a result, gravity loading is applied to the building as determined in previous technical reports. As lateral loads are the main concern of this report, more information concerning gravity loads may be found in Technical Report 1. More specific information can be delivered upon request.

Wind Loading: Please see initial calculations in appendix

Wind Loading Velocity:		**Kz**	**q_z [psf]**
Direct	Height [ft] ($q_z = 0.00256K_zK_{zt}K_dV^2I$)		
E-W	**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
E-W	**Leeward Wall** (all heights):	1.31	23.44

Wind Loading Pressure:		**q_z**	**p [psf]**
Direct	Height [ft] (p = qGp-qiGCpi)		
E-W	**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
E-W	**Leeward Wall** (all heights): (Cp = 0.5)	23.44	9.97
N-S	**Leeward Wall** (all heights): (Cp = 0.3)	23.44	5.98
E-W	**Roof:**		
	Windward (Cp = 0.2)	23.44	3.98
	Leeward (Cp = -0.6)	23.44	11.95
N-S	**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

Note: Values for heights above that of the overall roof height are included for application to cupolas which extend to approximately 125' from the lowest ground level.

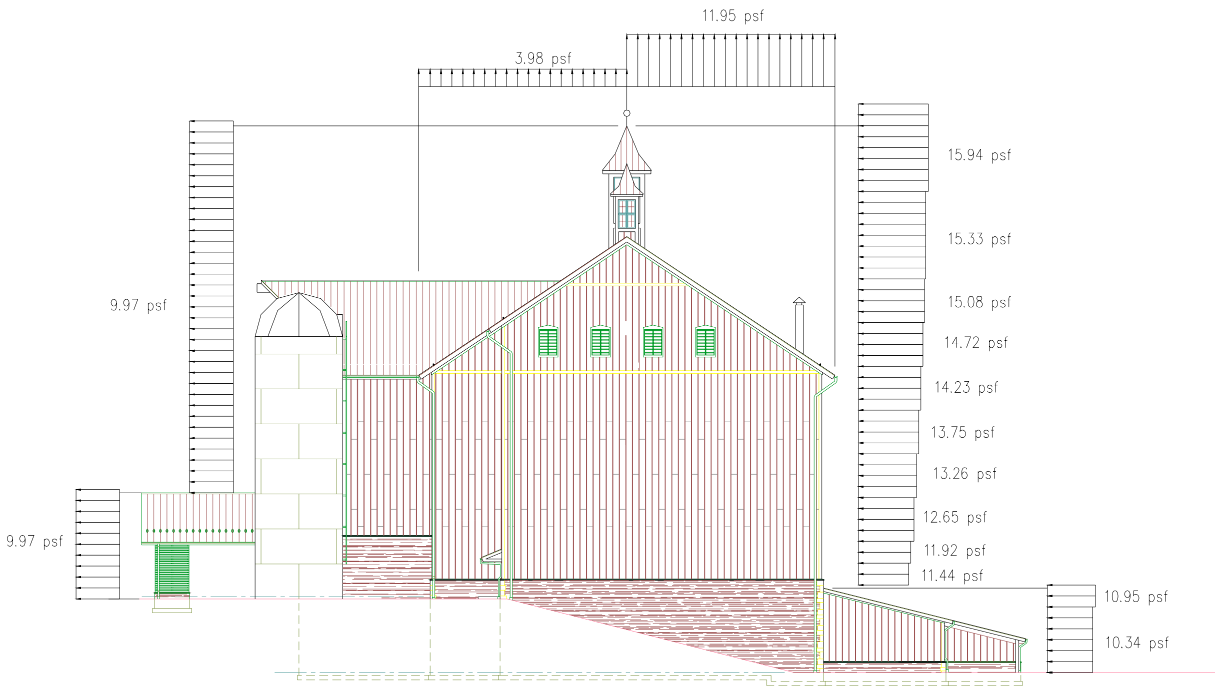

Figure 3.02: Wind Loading E-W Direction

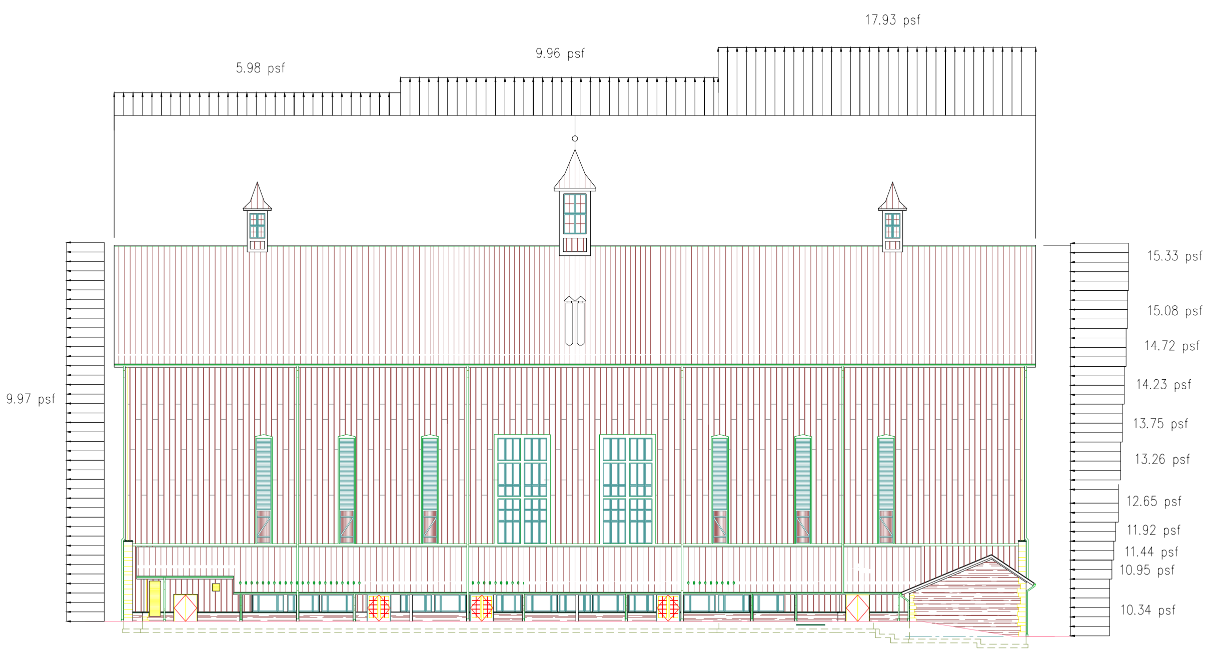

Figure 3.03: Wind Loading N-S Direction

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**

For initial comparison purposes, it is assumed that each frame takes an equal amount of the lateral load (one fourth per frame). More refined calculations, utilizing stiffness adjustments, are included in later portions of this report.

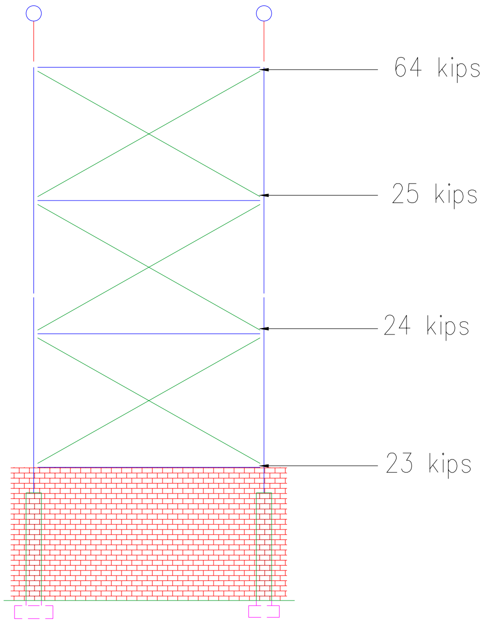

Figure 3.04:
E-W Wind Loading on Frame

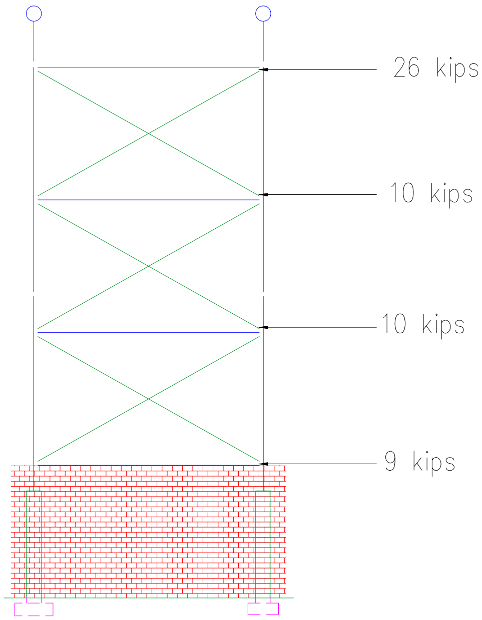

Figure 3.05:
N-S Wind Loading on Frame

Seismic Loading: Please see initial calculations in appendix

Note: Elevated floors are denoted as 1st-4th, while the partially exposed floor is denoted as the lower level.

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

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

Base Shear: $V = C_s W$
$= (0.0802)(8804\text{ k})$
$= 706.1\text{ k}$

Vertical Seismic Distribution:	$w_x h_x$	C_{vx}	**V [kips]**
1st floor	40586	0.126	88
2nd floor	70930	0.218	154
3rd floor	97624	0.301	212
4th floor	115633	0.356	251
Total (Base)	**324773**	**1.001**	**706**

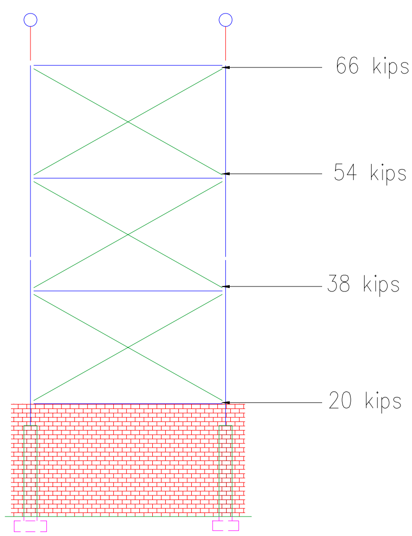

Figure 3.06: Siesmic Loading on Frame

In the same manor as was applied to wind loading, each frame is assumed to take an equal amount, one-fourth, of the lateral load for initial comparison purposes. More refined calculations are included in later portions of this report.

Based on these values, seismic loading will control the design of the lateral members within the building.

Lateral System Design

Basic Design of a Typical Braced Frame:

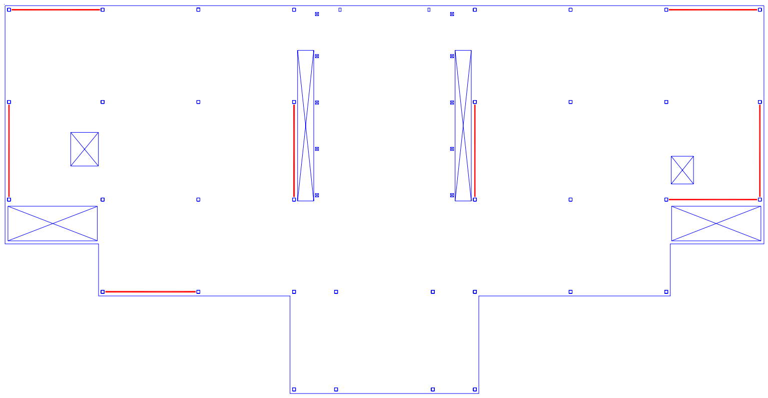

Figure 3.07: Lateral system plan

Each direction of lateral load resistance features 4 braced frames, some of which are supported on masonry piers. These 8 frames are shown in figure 3.07, highlighted in red. The original design of the building, although noting the presence of both materials, only considers the braced frames in the resistance of lateral forces.

For the purposes of this design and comparison, loads are considered to be delivered through the diaphragm to the concentric frames and directly transferred to the masonry piers, where present.

A diagram of a typical braced frame loaded with wind and seismic floor forces, evenly divided between frames, can be seen in figure 3.08.

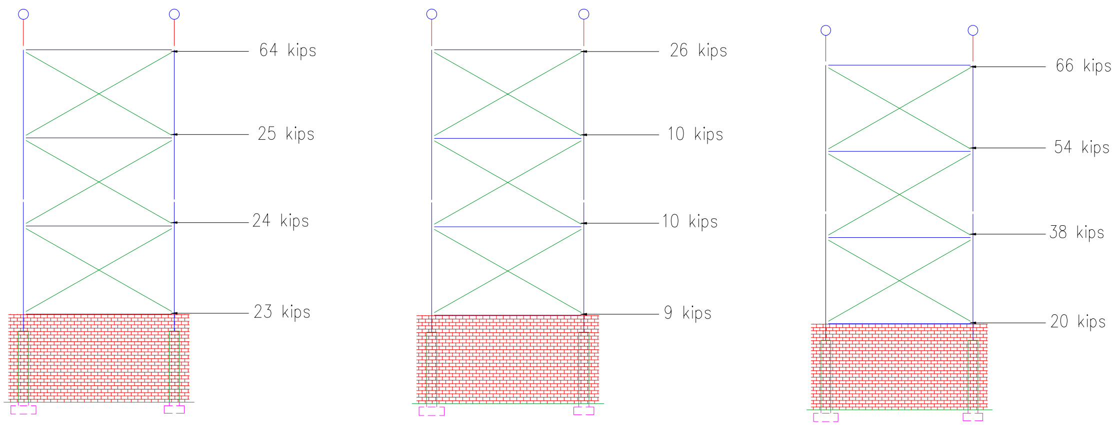

Figure 3.08:
Wind E-W Pattern **Wind N-S Pattern** **Seismic**

Seismic loading will control the lateral system design in both directions.

Relative Stiffness:

Original design of the individual frames was completed in Risa-3D. A model of each concentric frame was created and seismic loading was equally applied to the frames. Diagrams of these are included in the appendix along with rigidity calculations.

Each of these frames is then subjected to loading equally distributed to each frame and deflection is found through Risa-3D and displayed below.

Steel Frames	Original Deflection [in]	Relative k
E-W Direction		
B4 - C4	1.752	1.0
B7 - C7	1.752	1.0
B10 - C10	0.772	2.27
B1 - C1	0.595	2.94
N-S Direction		
A1 - A2	0.652	1.0
A9 - A10	0.446	1.46
C1 - C2	0.53	1.23
D8 - D9	0.511	1.28

Adjusted Lateral Loading:

Based on rigidity values, the loads carried by the individual frames are adjusted to match their respective rigidities. These values can be seen in the table and figures 3.09 and 3.10.

Steel Frames	Original Forces [k]	Adjusted Forces [k]
E-W Direction		
B4 - C4 & B7 - C7	66	36.6
	54	29.9
	38	21.1
	20	11.1
B10 - C10	66	83.1
	54	60.0
	38	47.9
	20	25.2
B1 - C1	66	107.7
	54	88.1
	38	62.0
	20	32.6
N-S Direction		
A1 - A2	66	53.1
	54	43.5
	38	30.6
	20	16.1
A9 - A10	66	77.6
	54	63.5
	38	44.7
	20	23.5
C1 - C2	66	65.3
	54	53.5
	38	37.6
	20	19.8
D8 - D9	66	68.0
	54	56.1
	38	39.2
	20	20.6

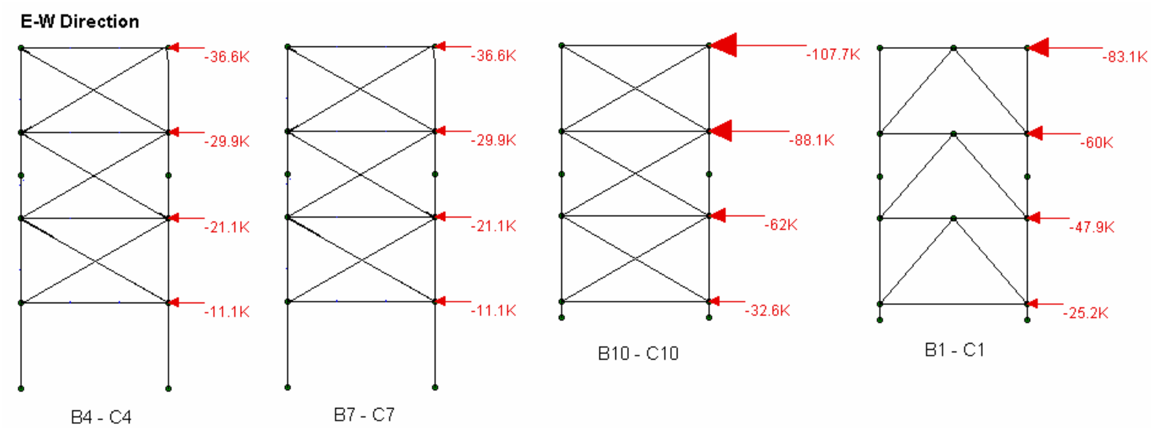

Figure 3.09: Adjusted Loading on E-W Frames

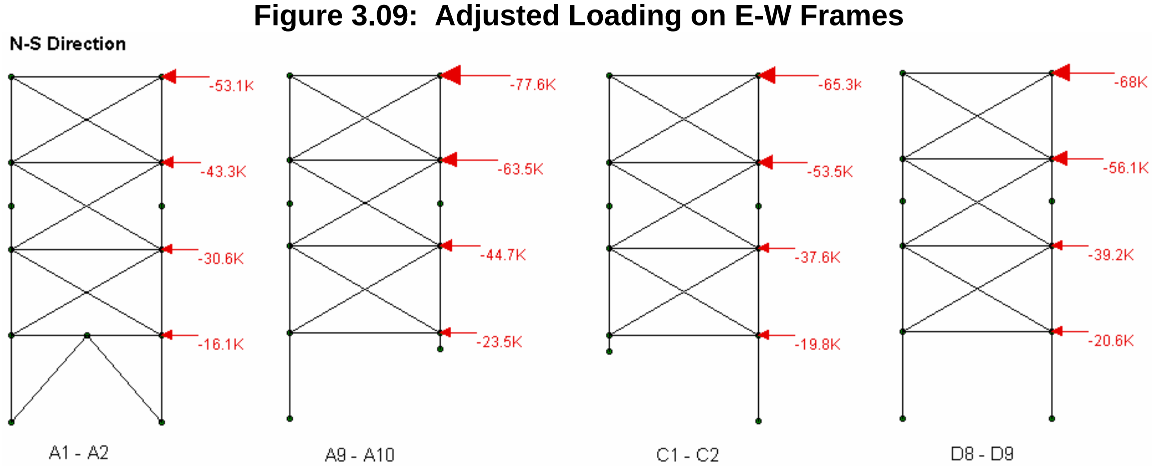

Figure 3.10: Adjusted Loading on N-S Frames

Frame Drift:

Deflections resulting from the adjusted loading can be seen in the table below and in figures 3.11 and 3.12.

Steel Frames	Adjusted Deflection [in]
E-W Direction	
B4 - C4	0.315
B7 - C7	0.315
B10 - C10	0.896
B1 - C1	1.014
N-S Direction	
A1 - A2	0.508
A9 - A10	0.533
C1 - C2	0.524
D8 - D9	0.529

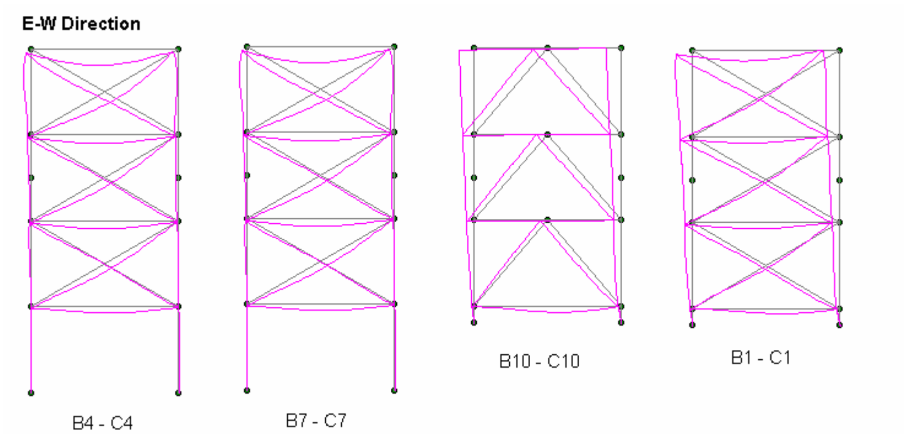

Figure 3.11: Adjusted Deflection of E-W Frames

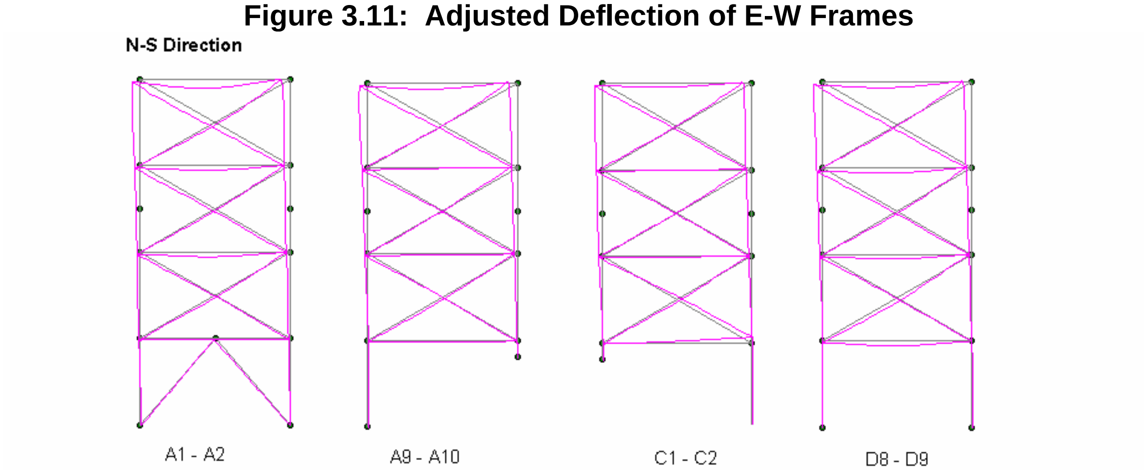

Figure 3.12: Adjusted Deflection of N-S Frames

Story Drift:

For a complete picture of how the braced frames will act as placed in the building, their coupling on masonry walls and piers must be considered. As both materials are designed to carry the resistance of lateral forces, and due to the lengthy embedment of steel in the masonry piers, it is assumed that the two portions are firmly connected and act as one member.

Specific calculations about the sections of masonry walls incorporated with the frames can be seen in the appendix. Deflections of this portion of the wall can conservatively be calculated at 0.748”. This value is greater than would actually occur in the building, as the walls incorporate masonry piers, and thus additional stiffness.

Story drift for the building is found by selecting the controlling drift of the lateral resisting system. In the case of Boyds Bear Country under seismic loads, the controlling deflection of a steel braced frame alone in the East-West direction is in frame B1-C1at 1.014” and in the North-South direction is in frame D8-D9 at 0.529”. The influence of added deflection due to the masonry mainly affects frames B10-C10 and B1-C1. This increases the East-West story deflection to nearly 1.762”. The standard control for lateral deflection is 1/400, or 2.08” over the full height of the frame. All values calculated under loading adjusted for relative stiffnesses are below this control value.

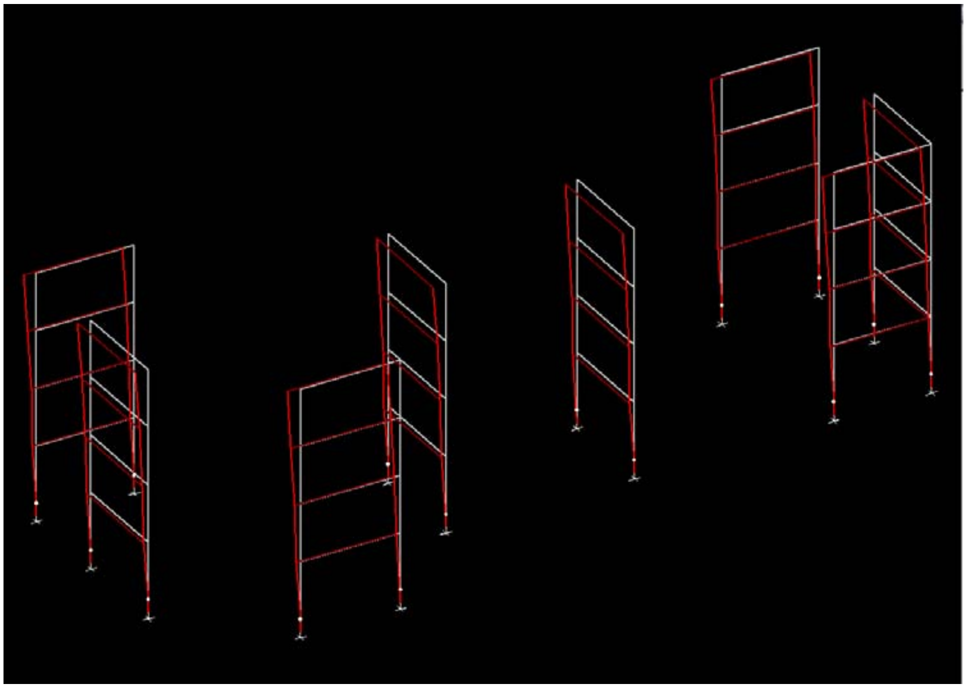

Figure 3.13: Adjusted Deflection of System in RAM

Comparison of Designs

In the original of the building, only the steel portion of the lateral system was considered in both lateral resistance and drift calculations. The load distribution to each individual frame is simply the determined seismic load split evenly (one fourth of the load to each). The frames selected from the original analysis were studied under the adjusted loads, and met code requirements and industry standards in strength and serviceability.

Torsion

Building torsion is calculated by applying the adjusted lateral loads through the center of stiffness. The values for center of stiffness can be found as calculated in the appendix, and a minimum eccentricity of 5% is used. The same eccentricity is used for each floor of the building, as all of the frames continue from the foundation to the fourth floor. Torsion on the building can be seen in the table below, for each floor and both directions.

Building Torsion:	x (N-S) [ft-k]	y (E-W) [ft-k]	Total [ft-k]
1st floor	1056	1320	2376
2nd floor	1848	2310	4158
3rd floor	2544	3180	5724
4th floor	3012	3765	6777
Total (Base)	**8,460**	**10,575**	**19,035**

These values are then distributed to the braced frames based on their distances from the center of stiffness. The resulting forces are negligible when compared to base shear and story shear created by the same seismic loads.

Overturning

Overturning moment on each frame is found by applying the adjusted lateral loads to each frame, and calculating the moment created by these forces at the based of each frame. For the calculations included here, only the adjusted seismic forces are considered; the gravity loading on the frames is not included.

Steel Frames	Overturning Moment [ft-k]	Induced Couple [k]
E-W Direction		
B4 - C4	4,554	152
B7 - C7	4,554	152
B10 - C10	10,966	366
B1 - C1	14,745	492
N-S Direction		
A1 - A2	7,264	242
A9 - A10	10,627	354
C1 - C2	8,945	298
D8 - D9	9,337	311

Diagrams of these couples on each frame can be found in figures 3.14 and 3.15.

These overturning moments are resisted by the foundation directly tied to the frames. The frames which experience the greatest amount of overturning are most frequently constructed with masonry piers and the masonry wall potion. The masonry used here increases the weight and the stiffness of the frame a great deal, and thus reduces the impact of the overturning moment on the frames. The foundations used in construction are a combination of a strip footing, found between the columns, and spread footings, found directly below the columns. Because of the additional area incorporated by the inclusion of spread footings, the overturning moments and their resulting couples, created by lateral loads, are absorbed.

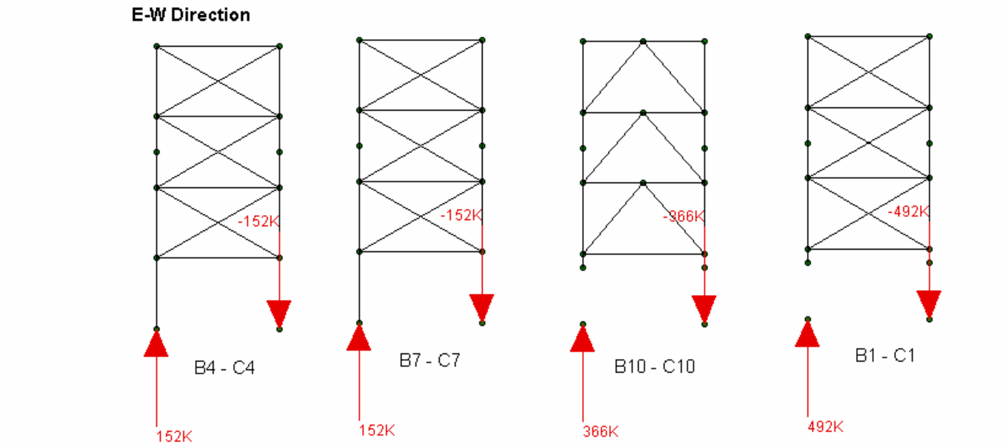

Figure 3.14: Couple Induced by Overturning of E-W Frames

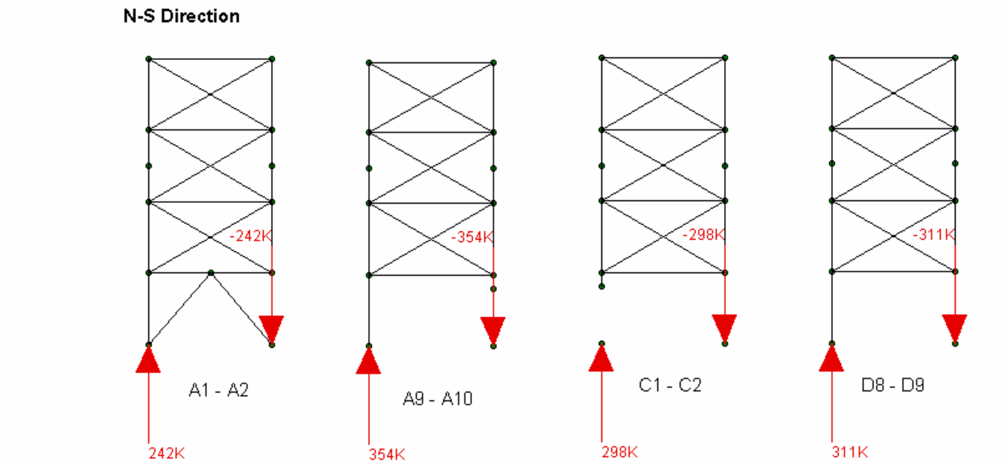

Figure 3.15: Couple Induced by Overturning of N-S Frames

Strength Check

Member strength was checked using RAM Structural System. Under loads adjusted to meet current codes, all members in general of the original design satisfy requirements. Many of the beams and girders as produced by RAM are undersized, but acceptable adjustments were made in the original design to include alterations in light storage areas, and specific item loading, such as seasonal decorations and large decorative timbers. Examples of these designs can be seen in figures 3.16, 3.17 and 3.18.

Verifications of these designs can be found in more detail within Technical Report 1, as all vertical loads remain unchanged from this information. Adjustments in lateral loading can be seen in calculations included within the appendix.

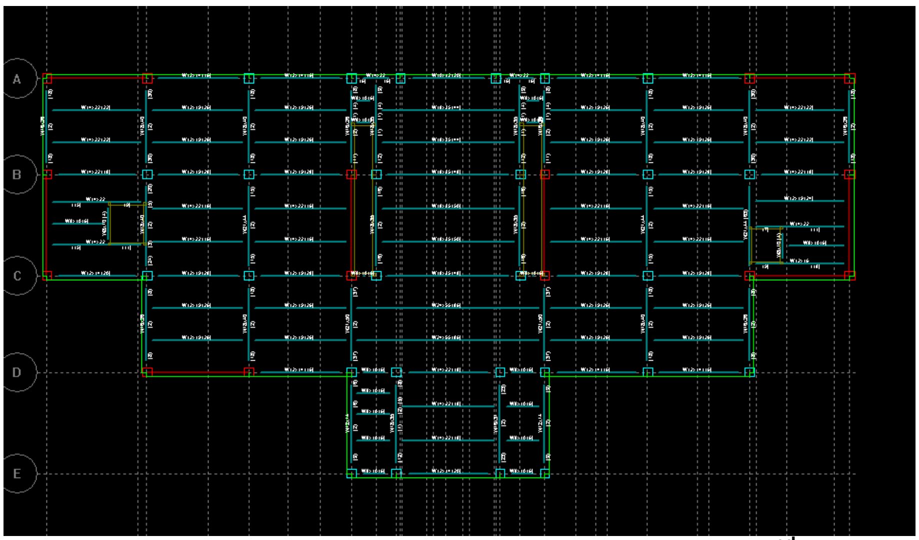

Figure 3.16: Beam / Girder Strength Check in RAM (2nd floor)

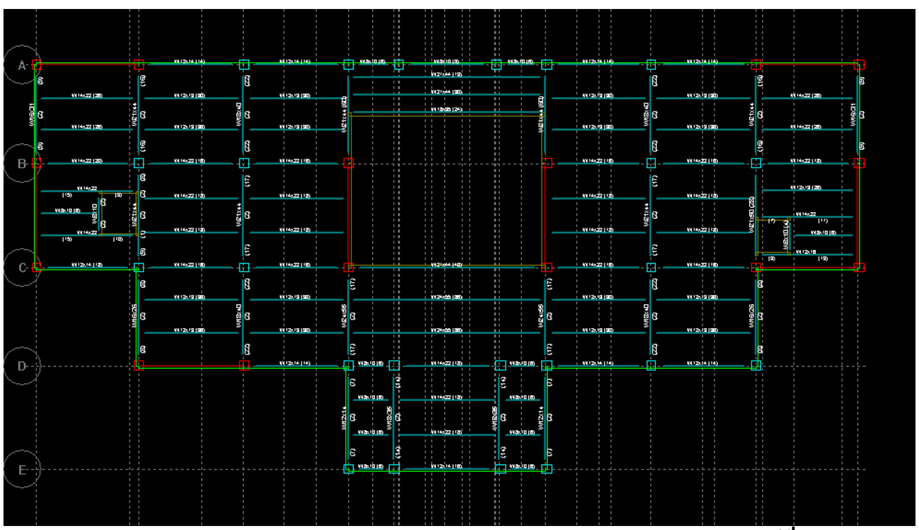

Figure 3.17: Beam / Girder Strength Check in RAM (2nd floor)

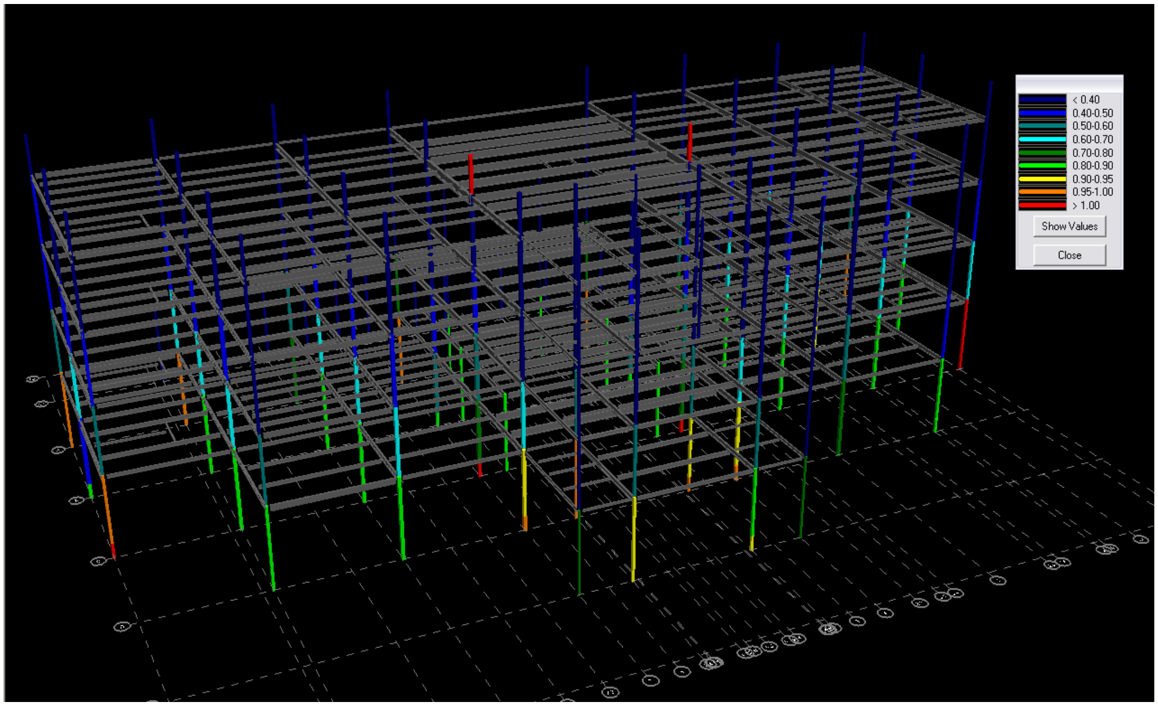

Figure 3.18: Column Strength Check in RAM

Conclusions

Boyds Bear Country, in Pigeon Forge, Tennessee, is a building constructed on many materials. The lateral system of the building is no exception. Primarily, lateral loads are resisted by a series of 8 concentric steel braced frames, 4 in each direction of the structure. Five of theses eight frames also incorporate masonry piers and masonry walls within the lowest story. In both types of frame, lateral forces are finally transferred to the foundation of the building in both strip and spread footings.

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Overturning and torsion as created by the lateral loads on the building do not greatly influence its design. Overturning moments and induced couples are easily resisted by the spread footings below each frame, and are reduced by the weight of the structure and most notably the weight and strength of the masonry portions of the frames. Torsion on the building is negligible when compared to base shear and story shear created by the same loads.

PENN STATE UNIVERSITY

CLASS: AE 481W
DATE: 11-21-06
ASSIGNMENT: TECH REPORT #3
LOADING REVISION / UPDATE
PAGE: ____ of ____

SEISMIC LOADING - DESIGNED W/ ASCE 7-05.

FROM usgs.gov @ ZIP 37862

SITE CLASS C.

$S_{MS} = 0.624\ g$

$S_{M1} = 0.199\ g$

[SECTION 11.4.4]

$S_{DS} = 2/3\, S_{MS} = (2/3)(0.624g) = 0.416\ g$

$S_{D1} = 2/3\, S_{M1} = (2/3)(0.199g) = 0.133\ g$

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

$C_T = 0.02$ FOR SHEARWALLS

$h_n = 75.4'$

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

$T_0 = 0.2 S_{D1}/S_{DS}$

$= (0.2)(0.133)/(0.416) = 0.064$

$T_S = S_{D1}/S_{DS} = (0.133)/(0.416) = 0.32$

$T_L =$ N/A

$S_a = S_{D1}/T$

$= (0.133)/(0.51) = 0.261$

[TABLE 11.6-2] & [TABLE 11.6-1]

OCCUPANCY CATEGORY C.

ORDINARY STEEL BRACED FRAMES $R = 3\tfrac{1}{4}$ (CONCENTRIC)

ORDINARY REINFORCED MASONRY SHEARWALLS. $R = 2$

$$C_s = \frac{S_{DS}}{(R/I)} = \frac{(0.416g)}{(3\tfrac{1}{4})/1.0} = 0.128$$

$0.128 \geq 0.01$ OK

$$C_s \leq \frac{S_{D1}}{T(R/I)} = \frac{(0.133g)}{(0.51)(3.25)/1.0} = 0.0802 \leftarrow \text{CONTROLS.}$$

$V = C_s W = (0.0802)(8804k) = 706\ k.$

& SEE SUPPLIMENTAL INFORMATION FOR BUILDING WEIGHTS & DISTRIBUTION.

REVISION OF LOADS TO IBC 2003 & ASCE 7-05.

SNOW LOADING - DESIGNED W/ IBC 2003 & ASCE 7-05.

[ASCE 7-05 SECTION 7.0]

[FIGURE 7.1]

$p_g = 15$ 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]

$p_s = C_s p_f$ $C_s = 0.53$

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

$= 5.6$ psf

[SECTION 7.6.1.]

UNBALANCED SNOW LOADS FOR HIP & GABLE ROOFS.

$p_u = (0.3)(p_s)$

$= (0.3)(5.6\text{psf})$

$= 1.68$ psf

$p_b = p_s = 5.6$psf

[SECTION 7.7.1]

DRIFTS ON LOWER ROOF

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

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

$= 16$ psf < 30 psf OK

FOR ROOF STRUCTURE @ COLUMN A TO COLUMN A.2.

SAME AS 7-95

$h_d = 2.76'$ $h_b = 0.66'$ $h_c = 79.34'$

$p_d = h_d \gamma$

$= (2.8')(16\text{pcf})$

$= 45$ psf

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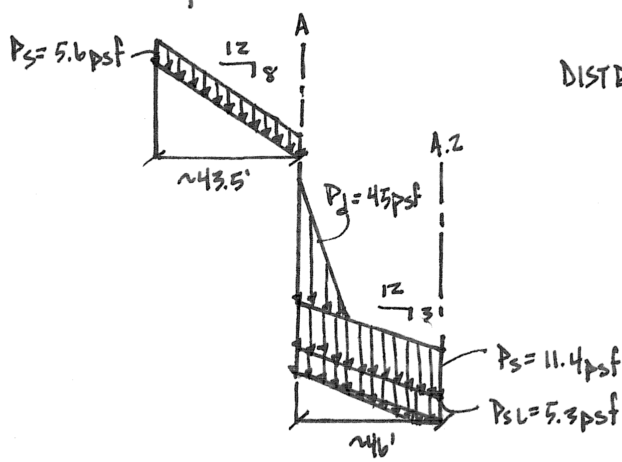
CLASS: AE 481 W
DATE: 11-21-06
ASSIGNMENT: TECH REPORT #3
LOADING REVISION / UPDATE.
PAGE: ____ of ____

SNOW LOADING - DESIGNED WITH IBC 2003 & ASCE 7-05.

[SECTION 7.9]

SLIDING SNOW:



STRIP ON UPPER ROOF:

$W = (5.6\text{psf})(1')(43.5') = 243.6 \text{ lbs}$

DISTRIBUTE ON LOWER ROOF:

$P_{sL} = \dfrac{(243.6 \text{ lbs})}{(1)(46.0')} = 5.30 \text{ psf}$

$P_s = C_s P_f = (0.85)(10.5\text{psf}) = 11.4 \text{ psf}.$

TOTAL LOAD ON LOWER ROOF:

$P_T = P_{sL} + P_s + \cancel{P_d}$
$= 5.3 \text{ psf} + 11.4 \text{ psf}$
$= 16.7 \text{ psf}$

IMPACT FACTOR:

$P_T = (1.5)(16.7 \text{ psf})$
$= 19.4$ psf OVER LOWER ROOF SPAN. — BE SURE TO INCLUDE DRIFT.

WIND LOADS
CALCULATIONS PER ASCE 7-05 SECTION 6, Main Windforce Resisting System - Method 2

DATE: 11/21/2006
ENGINEER: Lew

PROJECT NAME: Boyds Bear Country - Pigeon Forge TN
DESCRIPTION: AE 481W - Tech Report #3

DESIGN INPUT

						ASCE	
General Info:							
Building Height (Height of Interest)			z	100 ft			
Mean Roof Height (Eave height if roof slope < 10 deg.)			h	84 ft			
Basic Wind Speed			V	90 mph		6.5.4	Figure 6-1
Wind Directionality Factor			Kd	0.85		6.5.4.4	Table 6-4
Importance Factor			I	1		6.5.5	Table 6-1
Exposure Category				C		6.5.6	
Velocity Pressure Exp. Coeff. (hgt. of interest)			Kz	1.21		6.5.6.6	Table 6-3
Velocity Pressure Exp. Coeff. (mean roof height)			Kh	1.21		6.5.6.6	Table 6-3
Topographic Factor			Kzt	1		6.5.7	Eq. 6-3
Gust Effect Factor			G	0.85		6.5.8	
Internal Pressure Coeff.	(Input both values)		GCpi	0.18		6.5.11.1	Figure 6-5
			GCpi	-0.18		6.5.11.1	Figure 6-5
External Pressure Coeff.	(All bldgs)	Windward	Cp	0.8		6.5.11.2	Figure 6-6
		Leeward	Cp	-0.5		6.5.11.2	Figure 6-6
		Sidewalls	Cp	-0.7		6.5.11.2	Figure 6-6
		Roof (1)	Cp	-0.2		6.5.11.2	Figure 6-6
		Roof (2)	Cp	0.2		6.5.11.2	Figure 6-6
					Region		
External Pressure Coeff.	(Low-Rise Buildings)		GCpf		1	6.5.11.2	Figure 6-10
			GCpf		2	6.5.11.2	Figure 6-10
			GCpf		3	6.5.11.2	Figure 6-10
			GCpf		4	6.5.11.2	Figure 6-10
			GCpf		5&6	6.5.11.2	Figure 6-10
			GCpf		1E	6.5.11.2	Figure 6-10
			GCpf		2E	6.5.11.2	Figure 6-10
			GCpf		3E	6.5.11.2	Figure 6-10
			GCpf		4E	6.5.11.2	Figure 6-10

ANALYSIS

$q_z = 0.00256 K_z K_{zt} K_d V^2 I$	21.33 PSF	6.5.10	Eq. 6-15
$q_h = 0.00256 K_h K_{zt} K_d V^2 I$	21.33 PSF	6.5.10	Eq. 6-15

Rigid Buildings of all Heights:			
$P_W = q_z G C_p - q_h(GC_{pi})$	18.34 PSF	6.5.12.2	Eq. 6-17
$P_L = q_h G C_p - q_h(GC_{pi})$	-12.90 PSF		
$P_S = q_h G C_p - q_h(GC_{pi})$	-16.53 PSF		
$P_{R1} = q_h G C_p - q_h(GC_{pi})$	-7.46 PSF		
$P_{R2} = q_h G C_p - q_h(GC_{pi})$	7.46 PSF		
$P_{TOT} = q_z G C_p + q_h G C_p$	23.57 PSF		

Low-Rise Buildings:		Region		
$P = q_h[(GC_{pf})-(GC_{pi})]$	N/A		6.5.12.2	Eq. 6-18
$P = q_h[(GC_{pf})-(GC_{pi})]$	N/A			
$P = q_h[(GC_{pf})-(GC_{pi})]$	N/A			
$P = q_h[(GC_{pf})-(GC_{pi})]$	N/A			
$P = q_h[(GC_{pf})-(GC_{pi})]$	N/A			
$P = q_h[(GC_{pf})-(GC_{pi})]$	N/A			
$P = q_h[(GC_{pf})-(GC_{pi})]$	N/A			
$P = q_h[(GC_{pf})-(GC_{pi})]$	N/A			
$P = q_h[(GC_{pf})-(GC_{pi})]$	N/A			

ae
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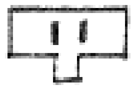
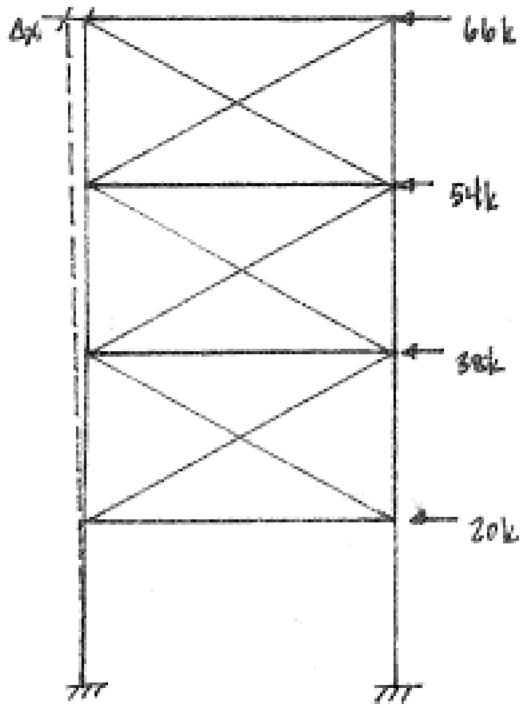
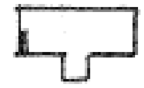
CLASS: AE 481 W
DATE: 11-21-06
ASSIGNMENT: TECH REPORT #3
BRACED FRAME STIFFNESSES E-W
PAGE: ____ of ____

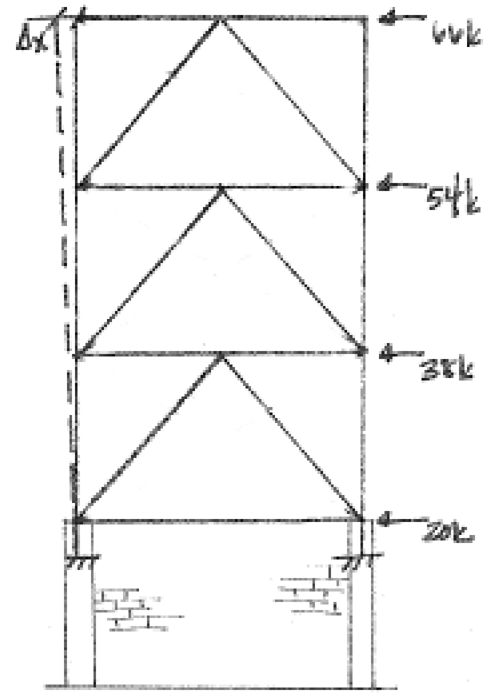
FRAME ALONG B4-C4 & B7-C7





FRAME ALONG B10-C10





UNDER LOADS EVENLY DISTRIBUTED TO FRAMES.

DEFLECTION AS FOUND BY RISA 3D [AVAILABLE UPON REQUEST]

$\Delta_x = 1.752''$ | $\Delta_x = 0.722''$

RELATIVE STIFFNESS BASED ON DEFLECTION.

$k = 1.752'' / 1.752'' = 1.0$ | $k = 1.752'' / 0.722'' = 2.27$

ADJUSTED LOADING BASED ON RELATIVE STIFFNESSES.

66k → 36.6k	66k → 83.1k
54k → 29.9k	54k → 60.0k
38k → 21.1k	38k → 47.9k
20k → 11.1k.	20k → 25.2k

ADJUSTED DEFLECTION AS FOUND BY RISA 3D.

$\Delta_x = 0.315''$ | $\Delta_x = 0.896''$

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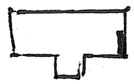
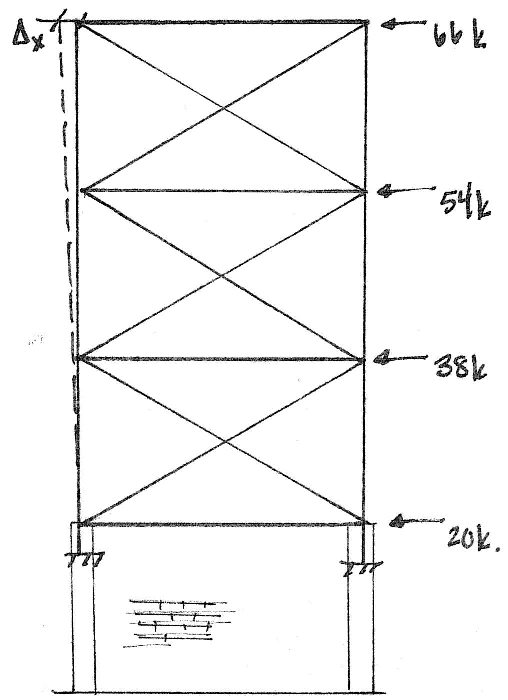
CLASS: AE 481 W
DATE: 11-21-06
ASSIGNMENT: TECH REPORT #3.
BRACED FRAME STIFFNESS E-W CONT.
PAGE: ____ **of** ____

FRAME ALONG B1-C1. 



$\Delta_x = 0.595''$

$k = 1.752'' / 0.595''$
$= 2.94$

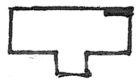
66k → 107.7k
54k → 881.k
38k → 62.0k
20k → 32.6k.

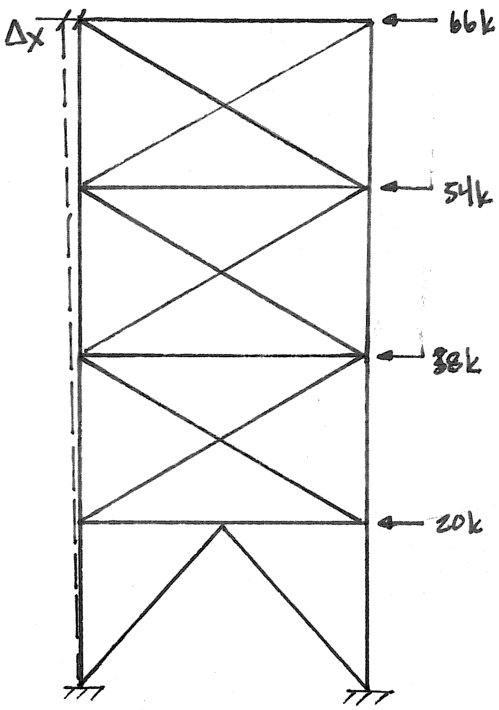
$\Delta_x = 1.014''$

PENN STATE UNIVERSITY

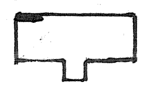
CLASS: AE 481W
DATE: 11-21-06
ASSIGNMENT: TECH REPORT #3
BRACED FRAME STIFFNESSES N-S
PAGE: ____ of ____

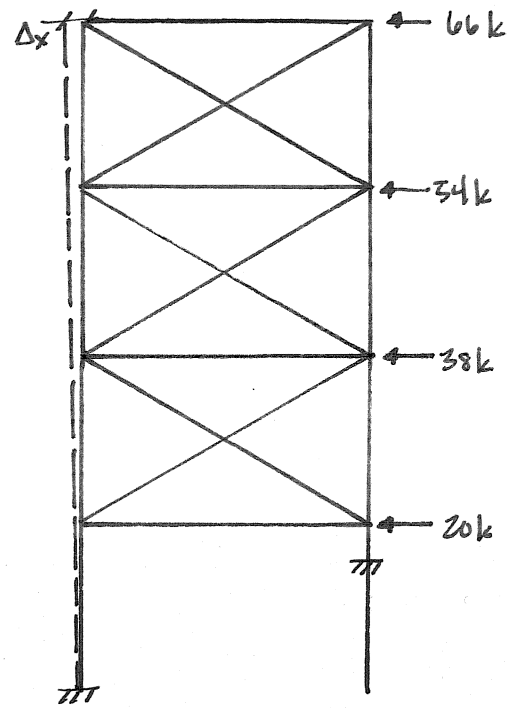
FRAME A1-A2.





FRAME A9-A10





DRAWN W/ LOADS EVENLY
DISTRIBUTED TO EACH FRAME.

$\Delta x = 0.652"$

ΔX VALUES AS FOUND BY RISA3D.
[AVAILABLE UPON REQUEST]

$\Delta x = 0.446"$

RELATIVE STIFNESSES
BASED ON DEFLECTION

$k =$
$= 1.0$

$k = (0.652"/0.446")$
$= 1.46$

ADJUSTED LOADING
BASED ON STIFFNESSES

66k → 53.1
54k → 43.5
38k → 30.6
20k → 16.1

66k → 77.6k
54k → 63.5
38k → 44.5
20k → 23.5

ADJUSTED Δ VALUE BY RISA3D.

$\Delta x = 0.308"$
$\leq 2.08"$ OK

$\Delta x = 0.533"$ $\leq 2.08"$ OK.

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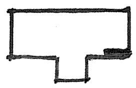
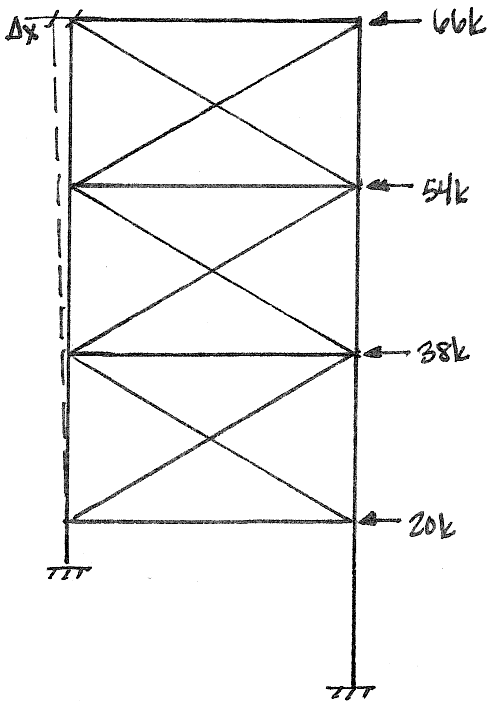
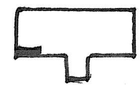
CLASS: AE 481W
DATE: 11-21-06
ASSIGNMENT: TECH REPORT #3
BRACED FRAME STIFFNESSES. CONT. N-S.
PAGE: ____ of ____

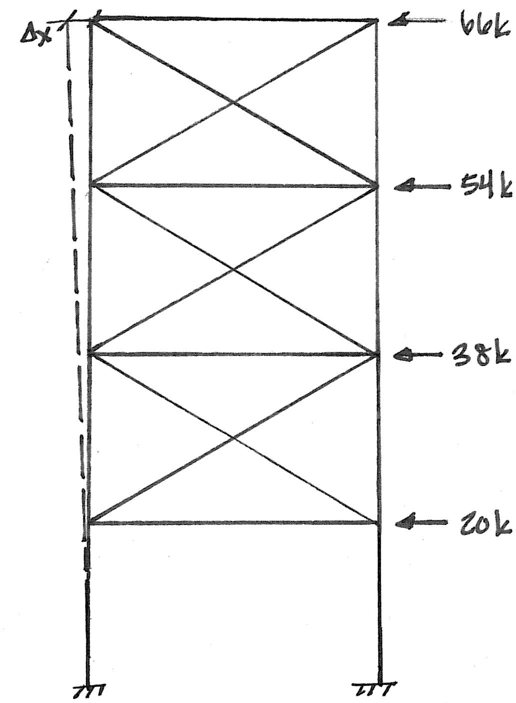
FRAME C1-C2.





FRAME D8-D9





DRAWN W/ LOADS EVENLY DISTRIBUTED TO EACH FRAME

Δx = 0.53" — Δx VALUES AS FOUND BY RISA 3D [AVAILABLE UPON REQUEST] — Δx = 0.511"

k = — RELATIVE STIFFNESSES BASED ON DEFLECTION — k =

= 1.23   = 1.28

ADJUSTED LOADING BASED ON STIFFNESSES.

66k →	65.3	66k →	68.0
54k →	53.3	54k →	56.1
38k →	37.6	38k →	39.2
20k →	19.8	20k →	20.6

Δx = 0.524" ≤ 2.08" OK — ADJUSTED Δ VALUE BY RISA 3D. — Δx = 0.529" ≤ 2.08" OK.

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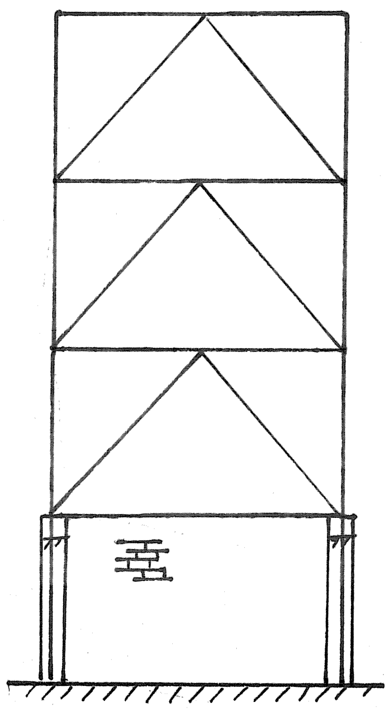
CLASS: AE 481W

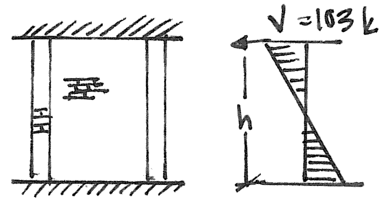
DATE: ~~AE~~ 11-21-06

ASSIGNMENT: TECH REPORT #3
MASONRY SHEAR WALLS

PAGE: _____ of _____

MASONRY SHEAR WALL ALONG B10-C10.





$M = \frac{1}{2} h V$

$= (\frac{1}{2})(17.3')(103k)$

$= 891\ 'k$

W/Ø REIN:

$F_{vm} = \frac{1}{3}\left[4 - M/Vd\right](f'_m)^{1/2}$

$= \frac{1}{3}\left[4 - \frac{(891'k)}{(103k)(30')}\right](1500\ psi)^{1/2} = 47.9\ psi$

$F_{vm} = 80 - 45\left[M/d\right]$

$= 80 - 45\left[(891'k)/\frac{(30')}{(103k)}\right] = 67.02\ psi$ USE 47.9 psi.

$f_{vm} = \frac{V}{td} = \frac{103k}{(1')(30')} = 3.43\ ksf = 23.8\ psi \le 47.9\ psi$

OK

$A_v = \frac{V_s}{F_s d} = \frac{(103k)(8")}{(60\ ksi)(30')(12"/')} = 1.15\ in^2$

↳ MIN #10. = 1.23 in²

$\Delta = \frac{h^2 F_b}{0.01 E_m t} = \frac{((17.3')\cancel{(12"/')})^2 (0.3)(1500psi)}{(0.01)(1000)(1500psi)(12")}$

$= \cancel{108"}\ 0.748"$

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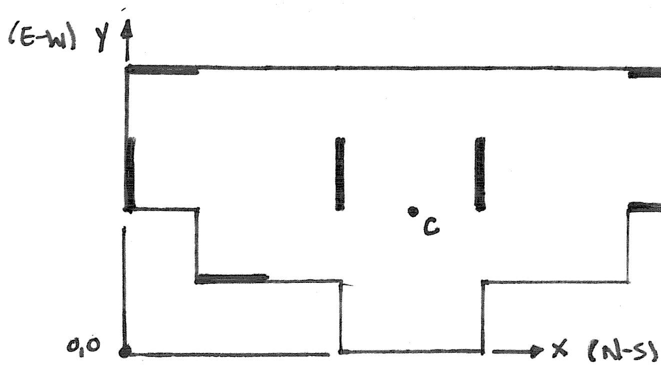
CLASS: AE 481 W
DATE: 11-21-06
ASSIGNMENT: TECH REPORT #3
CENTER OF STIFFNESS
PAGE: ____ of ____

CENTER OF STIFFNESS.



ALL BRACED FRAMES ARE SAME STIFFNESS IN SIMILAR DIRECTION.

|—| = 30'

X-DIR = 15' + 45' + (2)(225') = 510'   510'/4 = 127.5'   e = 240'/2 - 127.5' = 7.5' →

Y-DIR = (4)(75') = 300'   300'/4 = 75'   e = 120'/2 - 75' = 15' ↑

% ECC MINIMUM : (240')(0.05) = 12' ← CONTROLS
(120')(0.05) = 6'   USE 15' CONTROL.

BUILDING TORSION :

	X (N-S)	Y (E-W)
1ST FLOOR	(12')(88k) = 1056'k	(15')(88k) = 1320'k
2ND FLOOR	(12')(154k) = 1848'k	(15')(154k) = 2310'k
3RD FLOOR	(12')(212k) = 2544'k	(15')(212k) = 3180'k
4TH FLOOR	(12')(231k) = 3012'k	(15')(251k) = 3765'k.