North Shore at Canton

Baltimore, MD

Technical Report 3

Lateral System Analysis

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11/21/05

Executive Summary

North Shore at Canton is a four story town home structure built on top of a concrete pier in Baltimore harbor. For the purposes of this report, the building can be considered as three different structures. The concrete pier will make up the foundation, the first floor is a rigid steel frame, which is pinned to the pier, and the second, third and fourth floors are comprised of a bearing/shear wall system, made up of steel studs and gypsum sheathing. The floor system for the first floor is made up of pre-cast hollow core planks, which are welded to the steel frame, and a 2-1/2" topping of concrete. The second, third, and fourth floor systems are made up of pre-engineered floor trusses topped with ¾" OSB, which rest on the bearing/shear wall.

An analysis was made to determine the capacity of the lateral force resisting system. Both wind and seismic loads were calculated and compared based on the Allowable Stress Design equations as given in the IBC 2003. The loads were distributed to the building based on tributary area, the shear walls were analyzed and then the steel frame, the forces on the frame were to come from the lateral load at that level, as well as the resulting base shear and resulting moment that are caused from the three stories of shear walls that sit on top of it. The loads then transferred from the columns into the pier bents. The shear capacity of a typical shear wall was considered for this report, as well as story drift, overturning moment, also the steel frame was analyzed using a computer model. The pier structure was assumed to be able to carry the loads transferred from the columns.

There are two span directions for the shear walls, and it was determined that the short spanning shear walls could carry the story forces, while the long span could not. Over-turning moment was not a factor, as the dead load of the structure provided enough resisting moment against the story force moment. Also Lateral drift was not an issue as the building deflection was within limitation, however it should be noted that some calculations were based on assumptions and should be verified.

Introduction

North Shore at Canton is a four story town home structure, built over a pier in Baltimore harbor. The pier structure is supported by concrete piles, which also make up the pier bents. The bents span 60’, are spaced at 25’, and are tied together with a cast in place concrete slab. The top of the pier level is also the first floor of the town home structure. The first floor structure is comprised of a rigid steel frame topped with hollow core pre-cast planks and a thin layer of concrete. The steel frame consists of W 12x96 columns, 9’ in height, that line up over the pier bents. The columns are to be considered pinned to the pier structure. The steel beams that make up the rigid frame range from W 24x62, to W 24x78. The second, third, and fourth floors are made up of 25’x30’ bays, with a floor height of 10’. The top three floors have bearing/shear walls that are spaced 25’ apart and line up directly over the steel columns and pier bents. There is also a shear wall which spans the length of the building, 250’. The walls are comprised of steel studs and use gypsum board as the diaphragm. The floor systems of the top three floors are made up of pre-engineered wood floor trusses spaced 16” on center.

This report will analyze the lateral force resisting elements of the building. The loads considered for this analysis were wind and seismic and were derived from the IBC 2003; however it should be noted that the building was designed under the BOCA 1996 code where wind loads usually controlled the design of the lateral system. The report will also consist of, but is not limited to, the load distribution to the lateral system, the load capacity of a typical shear wall, overturning moment, and lateral building deflection. It should be noted, for the purposes of this report, that the top of the pier will be considered ground level, and all lateral loading transferred into the pier structure will be resisted by diagonal piles.

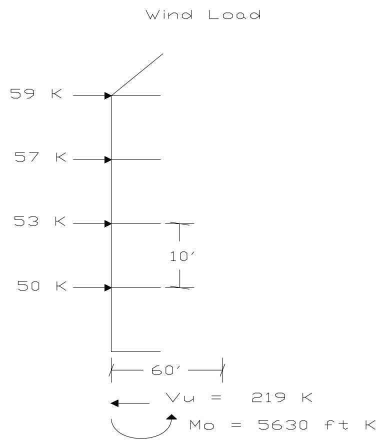
Loads and Load Cases

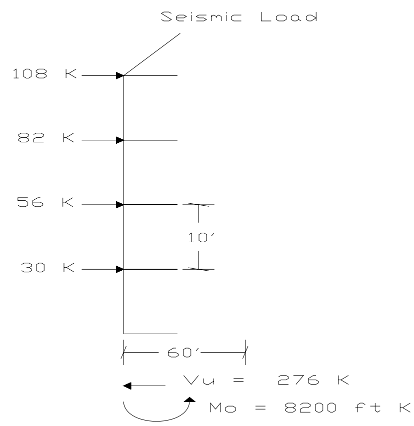
The loads used when the building was designed were derived from the BOCA 1996 codes. The BOCA ‘96 code has some downfalls; one is that the wind load predominately controlled over seismic load in most places, another is that the use of lateral sheathing was not of big concern. The load used for the lateral design of the building structure, was a wind load with an approximate pressure of 14 psf.

The loads used for the purpose of this report were derived for the IBC 2003, and both wind and seismic loads were considered. Wind and seismic calculations are located in Appendix A. The resulting story forces are located on the next page.

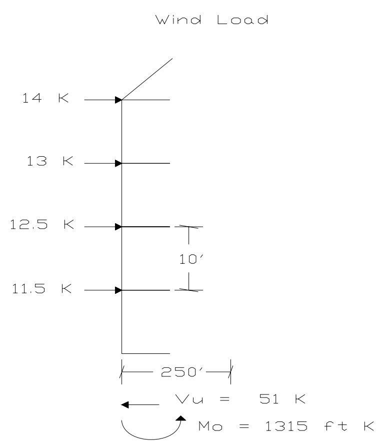
Un-factored Story Forces:

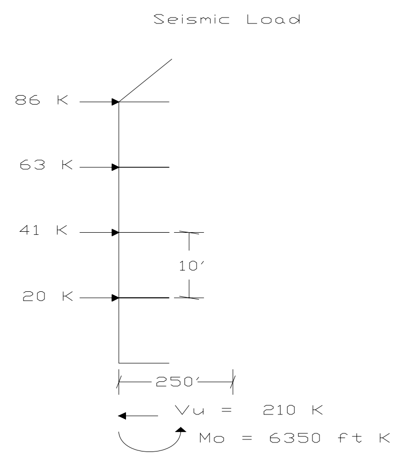
Short Span





Long Span





Load Cases:

Since the shear walls use gypsum as the diaphragm, the load cases used will be derived from the Allowable Stress Design equations. Both types of loading were considered in each span direction, based on the ASD equation:

D + (W or 0.7E) + L + (Lr or S or R) (IBC 2003 eq: 16-10)

Only D + (W or 0.7E) will be considered for the purposes of this report.

When both loads are put into this equation, the seismic loads still control in both span directions. Loading comparisons are located in Appendix A.

Distribution

Lateral loads will be distributed to each member based on tributary width. This assumption was based on the fact that the shear walls have an equal stiffness, and are laid out in an equal and symmetric manner. A diagram of the shear wall layout is located in Appendix A. Also the rigid steel frame which makes up the first floor has both a symmetrical layout and equal stiffness, which also backs up my assumption. It should also be noted that each floor system should be considered a rigid diaphragm, so the load would transfer entirely into the lateral elements.

Analysis and Member Checks

For the purposes of this report, the building will be considered as three separate structural systems; three stories of shear walls, one level of a rigid steel frame, and the concrete pier. First the top three floor of the building will be analyzed as a simple shear wall system. Since the shear wall sit on top of the concrete and hollow core planks, it will be assumed that the base shear and resulting moment will be resisted by the steel frame that supports it. The frame will have a story force acting on it as well. Since the steel columns are pinned to the pier, moments will not transfer into the concrete bents, only lateral and axial forces.

The shear wall analysis method used, was obtained from the IBC 2003. The allowable shear for the diaphragm member of the shear wall came from table 2306.4.5. For the short span, ½" gypsum board with nail spacing of 6" around the edges and 12" intermediate was the information used to obtain the table value. A value of 90 plf was noted, and since the walls are to be considered double sheathed the value was increased to 180 plf. For the long spanning shear walls there were some differences in the sheathing used. On the exterior bays a sheathing of 15/32" Plywood was used, while the interior units used the ½" gypsum board. The long spanning shear walls are to be considered double sheathed, and nail spacing is provided in Appendix A.

Allowable Shear Values (for single sided sheathing):

½" Gypsum Board	90 plf	IBC 2003 Table 2306.4.5
15/32" Plywood	170plf	IBC 2003 Table 2306.4.1

These values were adjusted and compared to the tributary story shear at each level. The shear walls spanning the short direction were adequate to carry the load\; however the shear in the long span direction, while it could be carried by the plywood, the plywood would need four times as much sheathing to carry the load correctly.

The tributary base shears and resulting moments from the top three floor, were distributed into the steel frame that they sit on. This was modeled in a computer program. Complete results are located in Appendix A.

Story Drift:

The story drift calculations were based on the equation given in the IBC 2003, for shear wall deflection. Some assumptions need to be noted about this analysis; first the shear wall deflection equation is based on two parts, deflection of the chord member and deflection of the diaphragm member. Since the chord members are made up from cold rolled steel studs, the area of the members and the modulus of elasticity will be adjusted accordingly. Also since the diaphragm is made up of gypsum board, some material properties have to be assumed. All information used for the assumption was obtained from USG, and is available upon request.

Story	Deflection
1	0.1”
2	0.098”
3	0.151”
4	0.205”
Total	0.554”

The overall building drift was based on H / 480, with H = height (in), and is equal to 1.2” > 0.554” therefore lateral drift is ok. However these deflection calculations were based on assumptions made and can not be permitted until proper information is verified.

Drift calculations are located in Appendix A.

Over-turning Moment:

It was found that the dead load for the structure was more than adequate to resist the over turning moment caused by the story shear. A representation of this is located in Appendix A.

Conclusion

The purpose of this report was to gain a better understanding of the lateral system of the building, the transfer of the loads, and how these factors affect items such as building drift. Most values calculated were with in acceptable values, though it should be noted that these values were based on some important assumptions that should be verified before they are used for any practical purposes. Also it should be noted that the pier structure is assumed to be capable of supporting the loads that are transferred through the steel columns. Since some of the lateral elements could not support the loads given and since the building was designed from a previous code, more analysis, and possible redesign, will be required.

Appendix A

Load Calculations: Wind Loads

		Resultant pressure		Story Forces	
		N-S	E-W	N-S	E-W
Pressure Coefficients, C_p	Windward	0.8	0.8		
	Leeward	-0.5	-0.35		
	Gust Factor Effect, G	0.859	0.886		

q_z (lb/ft^2)	q_h (lb/ft^2)	$q_zC_pG - q_hC_pG$ (lb/ft^2)	$q_zC_pG - q_hC_pG$ (lb/ft^2)	(kips)	(kips)
-	-			-	-
-	-			-	-
22.208	10.000	19.558	18.887	58.67	13.60
21.292	10.000	18.928	18.237	56.78	13.13
19.600	10.000	17.765	17.037	53.29	12.27
18.154	10.000	16.772	16.013	50.31	11.53
18.154	10.000	16.772	16.013		

Load Calculations: Seismic Loads

PROJECT TITLE :	North Shore at Canton	Date Designed
CLIENT :	Architectural Engineering Department	Date Checked
TITLE:	EQUIVALENT LATERAL FORCE METHOD	Page:
Designed By :	Beau Menard	
Checked By :		

REFERENCE	CALCULATION					OUTPUT
	1. Introduction					
ASCE 7-02	These calculation sheets serve to determine the Seismic Design Category and calculate Seismic Design Storey shear using the Equivalent Lateral Force Procedure as outlined in Section 9 of ASCE 7-02 "*Minimum Design Loads for Buildings and Other Structures*".					
	2. Seismic Design Parameters					
	Building Location :			Scranton, Pennsylvania		
	Number of Stories :	N		4		
	Inter-story Height	h_s		12 ft		
	Building Height :	h_n		48 ft		
Table 9.1.3 & Table 1.1	Seismic Use Group :	I		I	(Office)	
	Occupany Importance Factor :			1.00		
Table 9.4.1.2.1	Site Classification :			D	(Assumed stiff soil)	
Figure 9.4.1.1a	0.2s Acceleration :	S_S		0.18 g-s		
Figure 9.4.1.1b	1s Acceleration :	S_1		0.06 g-s		
Table 9.4.1.2.4a	Site Class Factor :	F_a		1.20		
Table 9.4.1.2.4b	Site Class Factor :	F_v		1.70		
	Adjusted Accelerations :	S_{MS}	$= F_a S_S$	0.216 g-s		
		S_{M1}	$= F_v S_1$	0.102 g-s		
	Design Spectral Response Accelerations :	S_{DS}	$= (2/3)S_{MS}$	0.144 g-s		S_{DS} = 0.144 g-s
		S_{D1}	$= (2/3)S_{M1}$	0.068 g-s		S_{D1} = 0.068 g-s
Table 9.4.2.1a & Table 9.4.2.1b	Seismic Design Category :			C	Equivalent Lateral Load Method can be used	Seismic Design Cateorgy is C
	3. Equivalent Lateral Force Procedure (9.5.3)					
	a. Seismic Base Shear Coefficient (9.5.3.2)					
	i.N-S Direction				Assumed light framed walls shear panels	
Table 9.5.2.2	Response Modification Factor :	R_{N-S}		2		R_{N-S} = 2
Equation 9.5.3.2.1-1	Seismic Response Coefficient :	$C_{s, N-S}$	$= S_{DS}/(R_{N-S}/I)$	0.072		
Table 9.5.5.3.2		$C_{T, N-S}$		0.02		
Table 9.5.5.3.2		x		0.75		
	Approximate Period of Structure :	T_{N-S}	$= C_{T, N-S} h_n^x$	0.36		T_{N-S} = 0.36 s
	but Seismic Response Coefficient need not be					
Equation 9.5.3.2.1-2	greater than	$C_{S\ max, N-S}$	$S_{D1}/T(R_{N-S}/I)$	0.093		
Equation 9.5.3.2.1-3	and	$C_{S\ min}$	$= 0.044 I S_{DS}$	0.0063		
	Therefore, the Seismic Response Coefficient ($C_{s, N-S}$) used is			0.072		$C_{s, N-S}$ = 0.093
	ii.E-W Direction				Assumed light framed walls shear panels	
Table 9.5.2.2	Response Modification Factor :	R_{E-W}		2		R_{E-W} = 2
Equation 9.5.3.2.1-1	Seismic Response Coefficient :	$C_{s, E-W}$	$= S_{DS}/(R_{E-W}/I)$	0.072		
Table 9.5.5.3.2		$C_{T, E-W}$		0.028		
Table 9.5.5.3.2		x		0.80		
	Approximate Period of Structure :	T_{E-W}	$= C_{T, E-W} h_n^x$	0.62		T_{E-W} = 0.62 s
	but Seismic Response Coefficient need not be					
Equation 9.5.3.2.1-2	greater than	$C_{S\ max, E-W}$	$S_{D1}/T(R_{E-W}/I)$	0.055		
Equation 9.5.3.2.1-3	and	$C_{S\ min}$	$= 0.044 I S_{DS}$	0.0063		
	Therefore, the Seismic Response Coefficient ($C_{s, E-W}$) used is			0.055		$C_{s, E-W}$ = 0.055
Table 2.2-1 "*Basic Loading Criteria*" of Design Report	**b. Loading Characteristics**					
	i. Roof :					
	Dead					
	Membrane			1.0 psf		
	Rigid Insulation			2.0 psf		
	Metal Roof Deck			2.0 psf		
	Roof Framing			20.0 psf		
	Drywall ceilding 0.5"			5.0 psf		
	M&E Services			5.0 psf		
	TOTAL q_{roof} :			35 psf of roof area		q_{roof} = 35 psf
	ii. All other Floors :					
	Dead					
	Flooring			1.0 psf		
	Open Web Floor Joists			16.0 psf		
	3/2" OSB Sheathing			4.0 psf		
	Structural Steel Studs w/ 1/2" Gyp.Sheathing			10.0 psf		
	0.5" Drywall Ceiling			5.0 psf		
	M&E Services			5.0 psf		
	Live					
As required in 9.5.3.2	Moveable Partition			20.0 psf		
	TOTAL q_{floor} :			61.0 psf of floor area		q_{floor} = 61.0 psf
	iii. Perimeter Wall:					
	Dead					
	Brick Curtain Wall, q_{wall} :			10.0 psf		q_{wall} = 10.0 psf
	iv. Snow Load:					
	Snow					
	Snow, q_{snow} :			20.0 psf		q_{snow} = 20.0 psf

Load Calculations: Seismic Loads

Checked By :

REFERENCE	CALCULATION	OUTPUT

Building Width : W = 60.0 ft

Building Length : L = 250.0 ft

Gross Roof or Floor Area: $A = W \times L =$ 15,000.0 sq. ft

Total weight of roof, $w_{roof} = A \times (q_{roof} + q_{snow}) + 2(W + L)0.5h_x q_{wall} =$ 862 kips

Total weight of each floor, $w_{per\ floor} = A \times q_{floor} + 2(W + L)h_x q_{wall} =$ 989 kips

$w_{floors} = (N-1)w_{per\ floor}$ 2,968 kips

Total Building Weight, $W = w_{roof} + w_{floors} =$ **3,830** kips — Output: $W =$ **3,830** kips

Equation 9.5.3.2-1 — Hence Seismic Base Shear, $V_{N\text{-}S} = C_{s,N\text{-}S}W =$ **276** kips — Output: $V_{N\text{-}S} =$ **276** kips

Hence Seismic Base Shear, $V_{E\text{-}W} = C_{s,E\text{-}W}W =$ **210** kips — Output: $V_{E\text{-}W} =$ **210** kips

c. Vertical Distribution of Seismic Forces (9.5.3.4)

The distribution of lateral forces over the height of the building is shown in Table 1 and 2 below.

Equation 9.5.3.4-2 — Exponent $k_{N\text{-}S} = 1 + (T_{N\text{-}S} - 0.5)/(2.5 - 0.5) =$ **0.932**

Table 1 : Vertical Distribution of Seismic Forces (N-S)

	Level, x	w_x	h_x	$w_x h_x^k$	C_{vx}	F_x	V_x	M_x
		(kips)	(ft)			(kips)	(kips)	(ft-kips)
		862	0	-	0.000	-		-
		989	0	-	0.000	-	-	-
		989	48	36,551	0.390	108	-	5,164
	4	989	36	27,952	0.298	82	108	2,962
	3	989	24	19,153	0.204	56	190	1,353
	2	989	12	10,036	0.107	30	246	355
Ground	1						276	
		Σ = 5809		Σ = 93691	Σ = 1.000	Σ = 276		Σ = 9834

Exponent $k_{N\text{-}S} = 1 + (T_{N\text{-}S} - 0.5)/(2.5 - 0.5) =$ **1.060**

Table 2 : Vertical Distribution of Seismic Forces (E-W)

where $C_{vx} = w_x h_x^k / \Sigma_{all\ levels}(w_x h_x^k)$

$F_x = C_{vx}V$

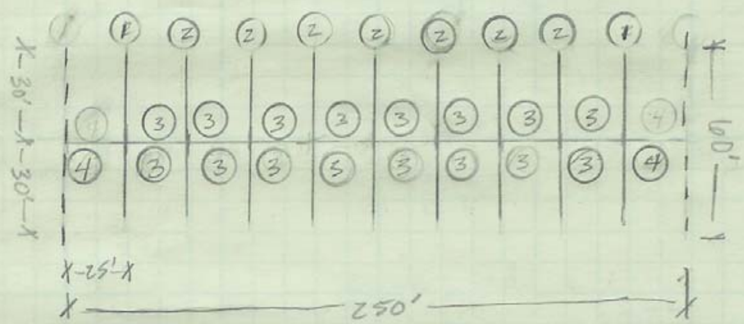
	Level, x	w_x	h_x	$w_x h_x^k$	C_{vx}	F_x	V_x	M_x
		(kips)	(ft)			(kips)	(kips)	(ft-kips)
		862	0	-	0.000	-		-
		989	0	-	0.000	-	-	-
		989	48	59,869	0.409	86	-	4,123
	4	989	36	44,135	0.301	63	86	2,279
	3	989	24	28,718	0.196	41	149	989
	2	989	12	13,776	0.094	20	190	237
Ground	1						210	
		Σ = 5809		Σ = 146498	Σ = 1.000	Σ = 210		Σ = 7628

Load Analysis: Shear walls

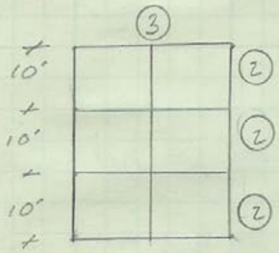
North Shore | Shear wall

	Sheathing (Both sides)	Chords	Studs	Sheathing attachment	Spacing int	Spacing edge	Chord Anchor
1	1/2" Gyp	2-JW16	JW16 @16" OC	#6 TEK screw	12	6	Simpson 2-LS22
2	1/2" Gyp	2-JW16	JW16 @16" OC	#6 TEK screws	12	6	2-LS22
3	1/2" Gyp	2-JW16	JW16 @16" OC	#6 TEK screws	12	6	None
4	15/32 PLY	2-JW16	JW16 @16" OC	#10 TEK screws	12	6	2-LS22

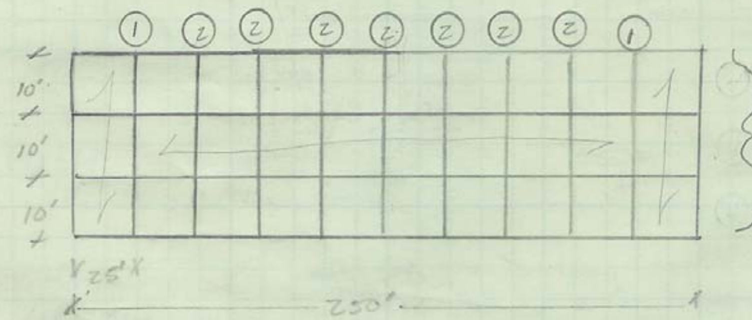
Studs 3 1/2" @ interior
6" @ exterior



Plan view
floors 2, 3, 4



Elevation



4 exterior wall each end each level

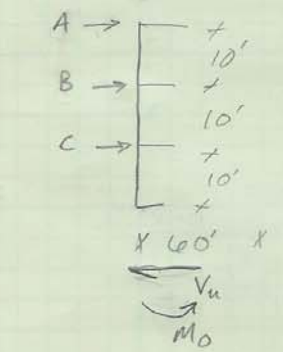
3 interior wall each level

Load Analysis: Shear walls

North Shore | lateral loading | 2

Story Forces

(EQ 16-10 IBC 2003) D + (0.7E or 1.0W) Based on ASD



Wind (1.0)

A = 58.67
B = 56.78
C = 53.29

Base shear & Resulting M_o

$V_u = 168.74^k$

$M_o = \Sigma F h = 3428.6^{ft}$

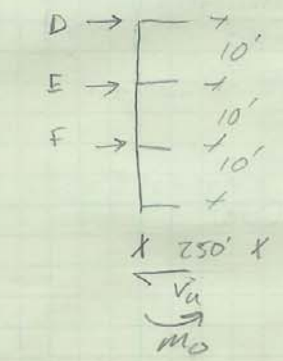
Seismic (0.7)

A = 106 k (0.7) = 74.2
B = 85 k (0.7) = 59.5
C = 56 k (0.7) = 39.2

Base Shear & M_o

$V_u = 172.9$ k

$M_o = 3808$ ft k



Wind (1.0)

$D = 13.6^k$
$E = 13.13^k$
$F = 12.2^k$

Base shear & M_o

$V_u = 39.1$ k

$M_o = 792.6$ ft k

Seismic (0.7)

$D = 86k (0.7) = 60.2^k$
$E = 63k (0.7) = 44.1^k$
$F = 41 k (0.7) = 28.7^k$

Base Shear & M_o

$V_u = 133^k$

$M_o = 2975$ ft k

Since all the shear walls, spanning 60' have equivalent stiffnesses, a load distribution to each wall will be based on tributary area.

Wall #	Trib width	Tributary force (k) A	B	C	
1	37.5'	11.1	8.9	5.9	Seismic
2	25'	7.4	5.9	3.9	Seismic
3 & 4	60'	60.2	44.1	28.7	Seismic
		D	E	F	

$$\text{force} = \frac{\text{Story force}}{\perp(\text{length})} \cdot \text{trib width}$$

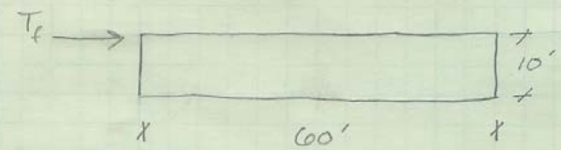
$$\text{ie} = \frac{74.2}{250'} \cdot 25' = 7.42^k$$

Seismic loads control for both building spans.

the wall spanning the length of the building (250') will be assumed to take the full load, equivalent to a tributary width of the build width (60')

Load Analysis: Shear walls

North Shore | Lateral Analysis



Wall Shear

$D_f = \frac{\text{Trib force}}{L}$

i.e. $= \frac{11.1^k}{60'} = 0.185\ k/ft$
$= 185\ lb/ft$

Trib Force (k)

Wall	A	B	C	
1	11.1	8.9	5.9	Seismic
2	7.4	5.9	3.9	Seismic

D_f (lb/ft)

Wall	A	B	C
1	185	148	98
2	123	98	65

exterior shear walls
ok 185 (lb/ft) close 180 (lb/ft)

Interior shear walls ok

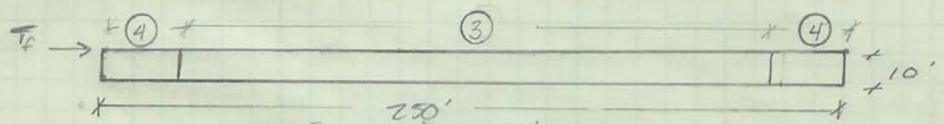
$T_{Mo} = \Sigma T_f (H) =$ (1) 570 ft k
(2) 379 ft k

(IBC 2003) Table 2306.4.5

Gypsum Board 1/2"
Blocked
(nail spacing) 6/12

Allowable Shear Value
90 plf

∴ double sheathing ≃ 180 plf



wall	Trib Force (k) D	E	F	D_f (lb/ft) D	E	F
3 & 4	60.2	44.1	28.7	240	176	115

(Seismic)

$T_{Mo} = 4250$ ft k

the table used from the IBC is just an estimate, it is intended for wood framing w/ nails instead of screws. Assume story shear is distributed through the entire length of the wall (250')

(IBC 2003) Table 2306.4.1

Plywood 15/32"
(assume penetration) 1"
Direct to Framing
fastener space 6"

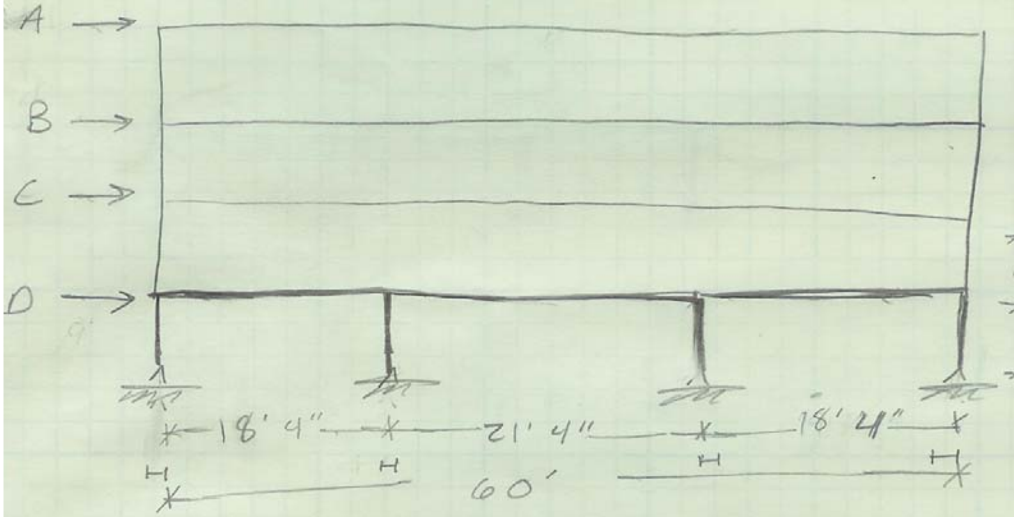
Allowable Shear
170 lb/ft

exterior
Assume Double Sheathing ≃ 340 lb/ft

interior
Assume (Double wall)
double sheathing ≃ 180 plf

Load Analysis: Shear walls

North Shore | Lateral Analysis



shear wall	columns	ext beam	center beam
1	W12x96	W24x62	W24x84
2	W12x96	W24x62	W24x76

columns pinned @ Base

Assume: the top 3 stories act independent and are attached to the hollow core planks & concrete, which acts as a diaphragm for the steel frame.

the base shear from the 3 levels above will act in the opposite direction of the story force from the lateral load.

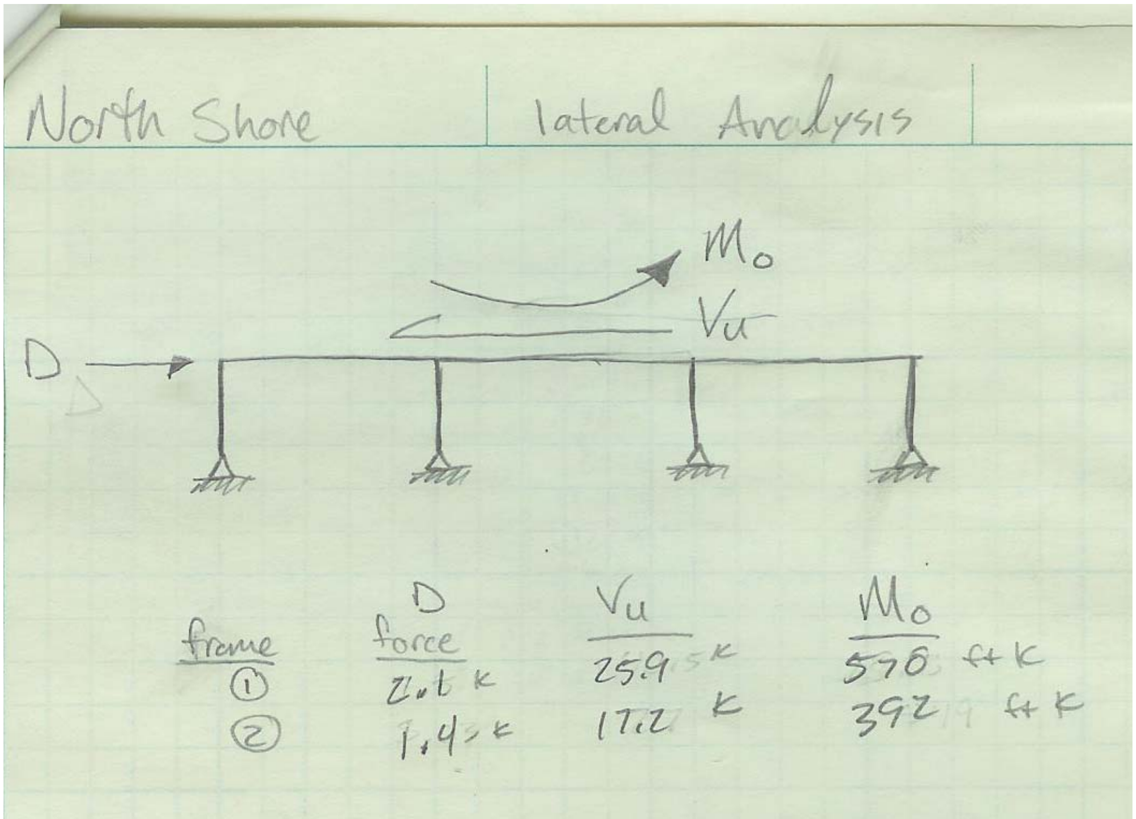
there will also be a resulting moment from each story force

columns are oriented w/ strong axis resisting this moment.

Story force $D = 0.7(20^k) = 14^k$

frame	lateral force	Base shear	resulting Moment
1	$\frac{14}{250'} \cdot 37.5' = 2.1^k$	$11.1 + 8.9 + 5.9 = 25.9^k$	570 ft k
2	$\frac{14}{250} \cdot 25' = 1.4^k$	17.2^k	379 ft k

Load Analysis: Steel Frame



Load Analysis: Steel Frame

North Shore

VisualAnalysis 4.00.EDU Report
Company: Penn State University   **Engineer:** Menard   **Billing:** For Educational Purposes Only
File: C:\Documents and Settings\bam301\Desktop\thesis\steel frame analysis.vap

Nodes

Node	X ft	Y ft	Fix DX	Fix DY	Fix RZ
N1	0.00	0.00	Yes	Yes	No
N2	18.33	0.00	"	"	"
N3	39.66	0.00	"	"	"
N4	58.00	0.00	"	"	"
N5	58.00	9.00	No	No	"
N6	39.66	9.00	"	"	"
N7	18.33	9.00	"	"	"
N8	0.00	9.00	"	"	"

Spring Elements

This item is empty. Check the selection state, or report properties.

Member Elements

Member	Section	Material	Length ft	Weight lb	Theta deg
M1	W12x96	Steel	9.00	863.63	0.00
M2	"	"	9.00	863.63	0.00
M3	"	"	9.00	863.63	0.00
M4	"	"	9.00	863.63	0.00
M5	W24x62	"	18.34	1135.81	0.00
M6	W24x76	"	21.33	1625.82	0.00
M7	W24x62	"	18.33	1135.19	0.00

Section Properties

Category	Section	Ax in^2	Iz in^4	Sy+ in^3	Sy- in^3
AISC Sha	W12x96	28.20	833.00	131.00	131.16
"	W24x62	18.20	1550.00	131.00	130.17
"	W24x76	22.40	2100.00	176.00	175.17

Material Properties

Material	Strength Ksi	Elasticity Ksi	Poisson	Density lb/ft^3	Therm. in/in/d
Steel	-NA-	29000.00	0.30	490.00	0.00

Plate Elements

This item is empty. Check the selection state, or report properties.

Load Analysis: Steel Frame

Member Extreme Results

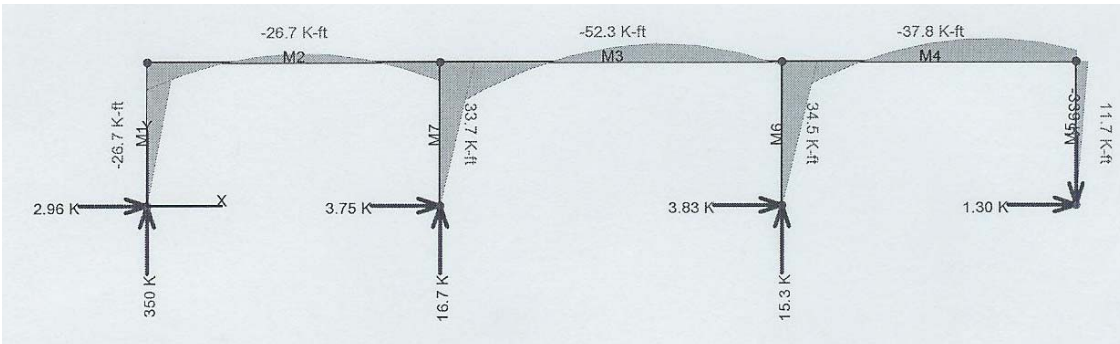
Member	Fx(lc) K	Fy(lc) K	Mz(lc) K-ft	fc max(lc) Ksi	fc min(lc) Ksi	Dx(lc) in	Dy(lc) in
M1	**-780.83(3)**	-12.93(6)	-116.38(6)	**-27.69(3)**	**-34.53(6)**	-0.10(3)	-0.11(8)
"	-3.28(2)	2.89(8)	25.98(8)	4.44(2)	-0.12(2)	0.00(2)	0.43(6)
M2	-37.79(6)	**-25.09(6)**	-225.82(6)	-1.34(6)	-22.00(6)	-0.00(6)	-0.12(8)
"	2.06(8)	10.72(8)	96.50(8)	19.35(6)	0.07(8)	0.00(8)	0.44(6)
M3	-21.19(3)	-17.66(8)	-158.96(8)	-0.75(3)	-14.96(8)	-0.00(3)	-0.12(8)
"	-1.44(2)	8.72(2)	78.47(2)	14.14(8)	-0.05(2)	0.00(2)	0.46(6)
M4	18.08(2)	-19.80(6)	-178.21(6)	0.64(2)	-9.38(2)	0.00(2)	-0.13(8)
"	**764.94(3)**	12.84(8)	115.57(8)	**40.41(6)**	**27.13(3)**	0.10(3)	**0.48(6)**
M5	-47.70(6)	18.08(2)	-245.11(8)	-2.20(6)	-34.38(6)	**-0.12(8)**	**-0.15(6)**
"	-2.04(8)	784.20(3)	**346.75(6)**	29.35(6)	-0.12(8)	**0.48(6)**	0.14(8)
M6	-50.60(6)	16.64(2)	**-340.86(6)**	-2.17(6)	-25.61(6)	-0.12(8)	-0.15(8)
"	15.62(8)	**785.40(3)**	309.63(8)	21.91(8)	0.69(8)	0.46(6)	0.19(6)
M7	-25.51(6)	3.28(2)	-35.56(8)	-1.38(6)	-12.06(6)	-0.11(8)	-0.03(2)
"	4.90(8)	780.83(3)	116.38(6)	9.33(6)	0.16(8)	0.44(6)	0.10(3)

Nodal Reactions

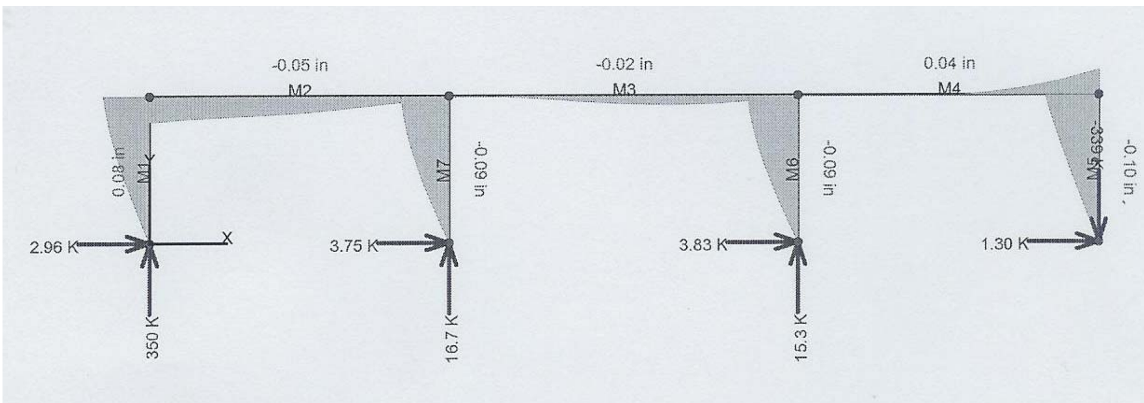
Node	Load Case	FX K	FY K	MZ K-ft
N1	Dead loads	4.7829	557.733	-NA-
"	LRFD A4-1	6.6961	**780.827**	-NA-
"	LRFD A4-2a	5.7395	669.280	-NA-
"	LRFD A4-3b	10.1652	671.904	-NA-
"	LRFD A4-4a	12.9312	673.543	-NA-
"	LRFD A4-6a	11.4963	506.223	-NA-
"	LRFD A4-6b	-2.8871	497.697	-NA-
"	LRFD A4-6c	4.3046	501.960	-NA-
"	Wind loads	5.5321	3.2794	-NA-
N2	Dead loads	6.8424	17.0166	-NA-
"	LRFD A4-1	9.5794	23.8232	-NA-
"	LRFD A4-2a	8.2109	20.4199	-NA-
"	LRFD A4-3b	18.5990	31.1116	-NA-
"	LRFD A4-4a	**25.0915**	37.7939	-NA-
"	LRFD A4-6a	23.0387	32.6889	-NA-
"	LRFD A4-6b	-10.722	-2.0591	-NA-
"	LRFD A4-6c	6.1582	15.3149	-NA-
"	Wind loads	12.9850	13.3646	-NA-
N3	Dead loads	7.0304	15.1376	-NA-
"	LRFD A4-1	9.8425	21.1926	-NA-
"	LRFD A4-2a	8.4365	18.1651	-NA-
"	LRFD A4-3b	1.4612	19.3147	-NA-
"	LRFD A4-4a	-2.8984	20.0331	-NA-
"	LRFD A4-6a	-5.0075	15.4919	-NA-
"	LRFD A4-6b	17.6622	11.7558	-NA-
"	LRFD A4-6c	6.3273	13.6238	-NA-
"	Wind loads	-8.7191	1.4369	-NA-
N4	Dead loads	3.3143	-546.38	-NA-
"	LRFD A4-1	4.6400	**-764.94**	-NA-
"	LRFD A4-2a	3.9771	-655.66	-NA-
"	LRFD A4-3b	13.7147	-670.13	-NA-
"	LRFD A4-4a	19.8007	-679.17	-NA-
"	LRFD A4-6a	18.8064	-515.25	-NA-
"	LRFD A4-6b	**-12.840**	-468.24	-NA-
"	LRFD A4-6c	2.9828	-491.74	-NA-
"	Wind loads	12.1720	-18.080	-NA-

Load Analysis: Steel Frame

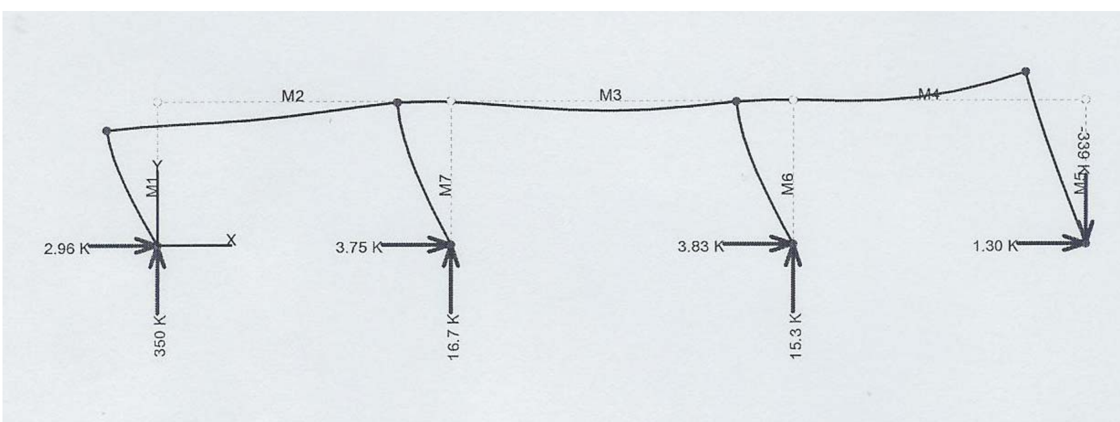
Member Moments:



Local Member Displacement:



Deflected Shape:



Load Analysis: Story Drift

North Shore | Story drift

Shear wall deflection (IBC 2003 - 2305.3.2)

$$\Delta = \frac{8vh^3}{EAb} + \frac{vh}{Gt} + 0.75he_n + d_a$$

A = Area of cross section of chord members (in^2)

b = wall width (ft)

d_a = deflection due to anchor detail (in)

E = Mod of Elasticity of the chord element (psi)

e_n = Deformation of mechanically fastened connectors (in)

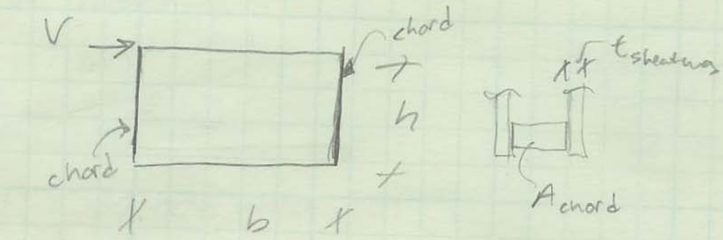
G = Mod of Rigidity of the sheathing element (psi)

h = height of wall (ft)

t = thickness of sheathing (in)

V = Design shear @ top of wall (lbs/ft)

Δ = deflection (in)



Load Analysis: Story Drift

North Shore | Drift Calc (2)

chord member - (2) JW-16 ($3\frac{1}{2} \times 1\frac{1}{2}$) metal studs

$E = 30{,}000{,}000$ psi (assumed cold roled section)

$A = 2(0.262) = 0.524\ in^2$ (Marino/WARE Light weight Framing)

$h_{wall} = 10' = 120''$

Sheathing - (2) $\frac{1}{2}''$ gypsum Board

$t = 2(\frac{1}{2}'') = 1''$

$G = \frac{E}{2(1+\gamma)}$ $\quad$ $\frac{EI}{(\text{in width})} = 2650\ \frac{lb\,in^2}{\text{in of width}}$ (Gypsum Association)

assume width of wall = 60' = 720 in

$b = 60'$

$EI = 2650(720) = 1908000\ lb\,in^2$

assume $\gamma = 0.2$ $\quad$ assume $I = \frac{(\frac{1}{2}'')(44'')^3}{12} = 114.3\ in^4$ (ASTM 14" full piece)

$G = \frac{16693}{2(1+0.2)} = 6955$ psi

$E = \frac{1908000\ lb\,in^2}{114.3\ in^4} = 16693$ psi

$A = 0.524\ in^2$

$b = 60'$

$d_a = 0$

$E = 30\ 000\ 000$ PSI

$e_n = 0$

$G = 6955$ psi

$t = 1''$

V = varies based on height an unfactored wind load will be used in this analyss.

deflection will be primaly based on chord & diaphragm deflection so the variables d_a & e_n will be assumed 0

Load Analysis: Story Drift

North Shore — Drift Calc

unfactored loads

level	F: Wall Force, Interior (25' trib width)	V ($F/60$)
4	8.6 k	143 lb/ft
3	6.3 k	105 lb/ft
2	4.1 k	68 lb/ft

Individual drift

level	Δ
4	$\frac{8(97)(10)^3}{(3000000)(0.524)(60)} + \frac{(143)(10)}{(6955)(1)} = 0.00082'' + 0.205\text{ in}$
3	$\frac{8(95)10^3}{(30\,000\,000)(0.524)(60)} + \frac{(105)(10)}{(6955)(1)} = 0.00081'' + 0.151''$
2	$\frac{8(89)(10^3)}{(30\,000\,000)(0.524)(60)} + \frac{(68)(10)}{(6955)(1)} = 0.00075 + 0.098\text{ in}$

chord deflection — diaphragm deflection

level 1 calculation were modeled using a computer

total drift

level 1 - 0.1"
level 2 - 0.198"
level 3 - 0.349"
level 4 - 0.554"

total building drift

$\Delta = 0.554''$

building height

$48' \approx 576$

Base total building drift on $\frac{h}{480}$

$\frac{576}{480} = 1.2''$

$0.554 < 1.2$

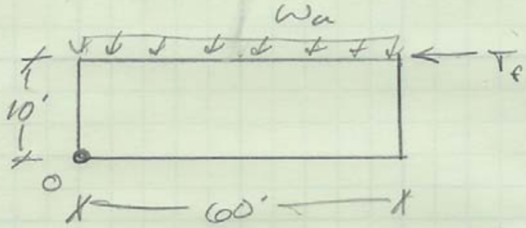
$\boxed{ok}$

It should be noted that these calculation are based on assumptions made about the diaphragm material properties. Since no clear information was given about the properties of gypsum board these results cannot be used properly. The equation to calculate the drift is correct, and, if the proper information is given, can be used to calculate the story drift.

Load Analysis: Over-turning Moment

Over turning Moment

DL = 61 psf
25' trib width
$w_a = 1.525\ ^{k}/_{ft}$



Average
Tributary force
$T_f \triangleq 10\ k$

Moment @ O from T_f

$10^{k}(10') = 100$ ft k

Moment @ O from Wa

$(1.525\ ^{k}/_{ft})(60')(38') = 2745$ ok

∴ the dead load is more than adequate to resist the over turning moment caused from the lateral forces.