North Shore at Canton

Baltimore, MD

Technical Report 1

Structural Concepts

Beau Menard
Structural
Schneider
10/05/05

Executive Summary

North Shore at Canton is a 4 story town home and parking garage structure built on top of a pier in Baltimore harbor. The building is unique in the fact that it is built over the water. The first floor of the building is an enclosed parking level, from which the residence gain access to the town houses. The second, third, and fourth levels are comprised of the town house structure. The building is approximately 15,000 sq ft. per floor, with a total square footage of 60,000 sq ft.

This report examines the structural aspects of the garage and town homes, and briefly describes the pier structure. The parking level consists of steel columns and beams which make up a full moment frame, the primary columns are made up of W 12x96 while the beams are primarily W 18x60 and W 24x68, with only some slight variation. The beams have hollow core precast concrete planks, with embedded steel plates, welded to the top flange, the planks are topped with a layer of rigid insulation and a thin layer of concrete. The remaining stories structure is made up of a metal stud shear wall system, and the floor systems consist of open web wood trusses bearing on the shear walls. The roof system is comprised of pre-engineered wood roof trusses, that also bear on the shear walls.

An examination of the loads of the structure, dead, live and lateral, was also done. The design loads used were from BOCA 1996, while the loads in my analysis came from the ASCE 7-02. The dead loads were mainly comprised of the weight of the structure and the snow loads. The live loads that were uses were based on a residential structure. The lateral loads are made up from the wind and seismic loads.

A spot check of a typical beam located in the garage floor structure and a typical shear wall on the third floor are analyzed. The results of the spot check of the beam resulted in a slightly larger beam size, however this is most likely as a result of the assumptions made, and the fact that the loads have increases from the code used to design the structure. The shear loads have also been increased, from the previous code the building was exempt from seismic loading, and the shear force that resulted from my analysis was much greater than expected. An analysis with out a seismic lateral force was made and the resulting shear force was still with in the design limitation.

National Code Selection

BOCA 1996

The preliminary design of both the pier and town home structures, were based off of the loads given in the BOCA 1996; this includes live, snow and lateral loads, wind and seismic. However for the purposes of this report the International Building Code 2003, and the ASCE 7-02 will be used and compared to the loads given from the BOCA 1996 code.

Design Codes

- Concrete – ACI 318
- Precast Concrete – ACI 318
- Masonry – ACI 530-92, ASCE 5-92
- Structural Steel – AISC
- Metal Studs – AISI
- Wood – NDS

Live Loads

Floor Live Load:

BOCA 1996

The floor live loads indicated from the BOCA 1996 code, multifamily Residential, show a floor pressure of 40 pounds per square foot. With a partition load also being used, in the BOCA '96 the partition load varies from 5 psf on the first floor to 12 psf on the second and third floors. There is no live load reduction applied to this structure.

IBC 2003

The floor live loads indicated by the IBC 2003, for a multifamily Residential, also show a distributed floor pressure of 40 psf. The partition loading increased greatly, IBC 1607.5, since the live load does not exceed 80 psf, the partition load shall be at least a uniformly distributed load of 20 psf

Roof Live Load:

BOCA 1996

A roof live load of 30 psf was used in accordance to BOCA ’96.

IBC 2003

(IBC 1607.11)
A_t = 25’(span) * 2’(spacing) = 50 sqft < 200 sqft

R_1 = 1
F = 4 in rise per foot
R_2 = 1
L_r = 20* R_1 * R_2 = 20*1*1
L_r = 20 psf

Dead Load

Material load:

ASCE 7-02

Precast hollow core concrete planks = 105 pcf *(8/12) = 70 psf
2.5” concrete topping = 144 pcf * (2.5/12) = 30 psf
Rigid Insulation = 1.5 psf

Open web wood floor joist w/ ¾” OSB sheathing = 20 psf
Structural steel studs w/ ½” Gypsum sheathing = 10psf
Roof Trusses = 20psf
Misc. Roof = 15 psf
Structural Steel = (as noted on plans)

(assumed)
Mechanical = 10 psf
Electrical = 5 psf
Ceiling = 5 psf

Snow Load:

BOCA 1996

Importance factor (I)= 1
Ground snow load (P_g) = 20 psf
Snow exposure factor (C_e) = 0.7
Flat roof Snow load (P_f) = I* P_g* C_e=1*20*0.7
P_f = 14 psf

IBC 2003

Importance factor (I) = 1
Ground snow load (P_g) = 25 psf
Snow exposure factor (C_e) = 0.8(Category D fully exposed)\
Thermal factor (C_t) = 1
Flat roof Snow load (P_f) = I* P_g* C_t*C_e=1*25*0.9*1
P_f = 20 psf

Lateral Loads

Wind Load:

BOCA 1996

Wind speed (V) = 75 mph
Wind load importance factor (I) = 1.05
Wind exposure = D
Basic wind velocity pressure (P_v) = 14 psf

IBC 2003

Wind speed (V) = 90 mph
Wind load importance factor (I) = 1
Wind exposure = D
Mean roof height = 45' (low rise, simplified method used)
Height adjustment factor (λ) = 1.78
Simplified design wind preasure (P_{s30}) = 11.9
Simplified wind preasure (P_s) = P_{s30} * I * λ = 11.9 * 1.78 * 1
P_s = 22 psf (across whole structure)

A visual representation for the wind load is located in Appendix A.

Seismic Load:

BOCA 1996

Building as a whole is exempt as per section 1610.3.6.1

ASCE 7-02

The calculations and visual representation of the seismic load analysis is located in Appendix A. Some assumptions need to be noted, for the purposes of this report. The top of the pier level, the garage level, is to be considered top of grade. No seismic analysis will be done on the pier foundation, though it will be looked at more thoroughly in a later report. All levels are assumed to be the same, though the garage level is built with structural steel and is considered a moment frame, the building will be thought of as uniform, for the purposes of determining the load. The actual distribution of the lateral load will be looked at more thoroughly; after further consultation with the engineers. A spot check of the shear wall on the third level is located in Appendix A.

Framing and Lateral Description

The framing for the garage level consists of structural steel columns and beams, with full moment connections at column interface. The columns are comprised of W12x96, and are all 9' in height. The columns connect to the pier through base plates, which range from 14"x14"x3/4" to 18"x18"x1-1/4". The beams range from W 14x22 to W 24x68, and have spans ranging from, 18' to 25'. The garage floor, floor system, is the top of the pier. A framing plan and typical weld section are located in Appendix A.

The first floor, floor system, is made up of 8" hollow core precast concrete planks, with 3" of rigid insulation topped with 2.5" of concrete. The hollow core precast planks have embedded steel plates that are welded to the steel beams.

The first, second, and third floors were framed out using light gauge metal stud shear walls, with gypsum used as the diaphragm. The interior walls use 4" studs while the exterior walls us 6" studs. There are also 3 inch hollow steel tubes used to support steel beams or wood PSL, which ever specified by the plan, that support joist spanning in the perpendicular direction. General shear wall data is given in Appendix A.

The floor systems of the remaining levels uses open web pre-engineered wood joists, which bear on the shear walls, exterior joist have top chord bearing while interior joists are bottom chord bearing. Typical joist layout is located in Appendix A.

Foundation

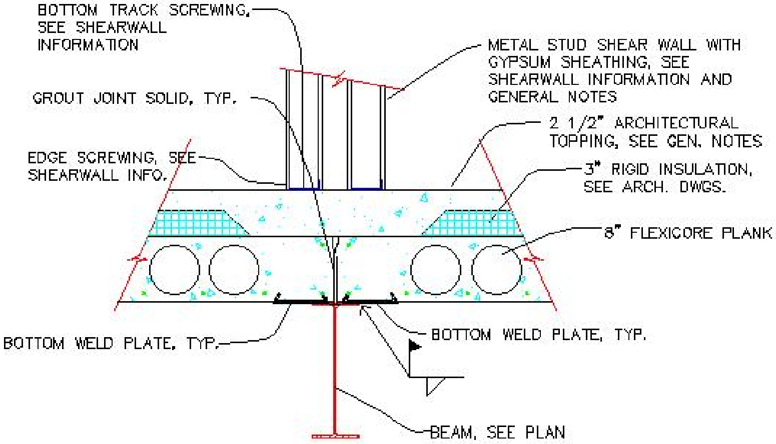
The foundation of the town home structure is the pier structure. The pier is a single span concrete structure, which replaced the old wood pier. The structure is relatively simple and is comprised of eight precast concrete bents. The bents consist of eight precast concrete piles. There is also a wooden walk area which is made up of wood piles and wood decking. A visual representation of a typical concrete bent with reinforcing is located in Appendix A.

Spot Check Analysis

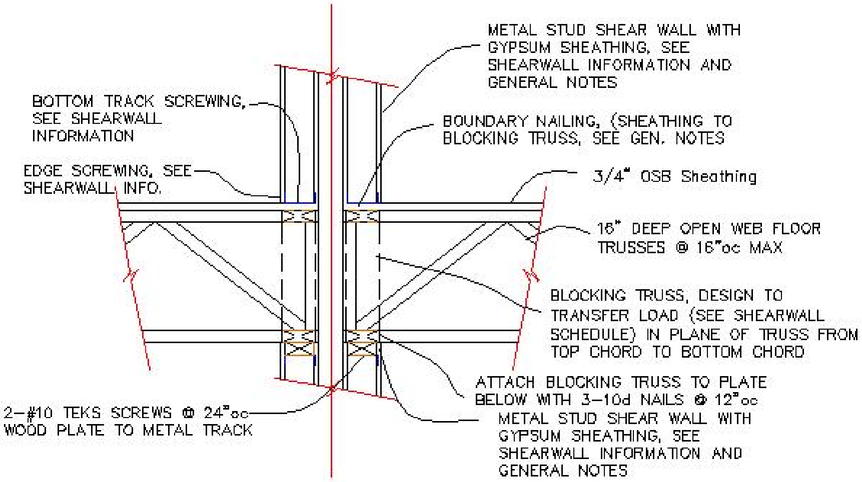
For the beam spot check, I chose to analyze a typical beam located along column line B in the moment frame of the garage level. All of the beams along this column line consist of W 18x60, and all of the columns consist of W 12x96. Since the frame is comprised of full moment connections, an indeterminate analysis had to be done. By the method of moment distribution, it was determined that the maximum negative moment controlled when sizing for the beam. After the analysis I found that I needed a slightly larger beam size to adequately support the loads designed for. The maximum negative moment was 644 foot kips, and from the AISC beam tables in chapter 5, I determined that with an un-braced length of 0 feet a W 14x99 could adequately handle the moment; with its capacity being 645 foot kips. However a more economical size is available, a W 24x68, which also has a higher moment capacity, 664 foot kips. Since the loads have increased since the building was designed it is easy to see why the size of the beam increased. Some assumptions have to be noted about the analysis of the beam. First the three stories above the garage level were assumed to be evenly distributed over the concrete slab. Second symmetry was used since the bays are identical in size, only half of the column line was looked at. Third, two load cases were applied to induce both maximum positive and negative moments in the beam. The calculations for the analysis are located in Appendix A.

For the lateral spot check, an analysis of the third level, along the width of the building was done. The shear wall consists of structural steel studs with sheathing of ½" gypsum board on both sides of the wall. The loads used in the analysis were determined from the seismic and wind calculations. Since the building was exempt from seismic loading under the design code, it is logical that the shear loading would increase and surpass the capacity of the shear wall. When only the wind load is applied the shear increases, because the values from the code increased, but the shear load is under the capacity of the wall and is there for adequate. Some assumptions need to be noted for the lateral analysis. First it is assumed that the shear wall is continuous through the width of the building. Second the building is to be considered flexible, and not rigid, so the distribution of the lateral loads comes from tributary width. The calculations for the lateral analysis are located in Appendix A.

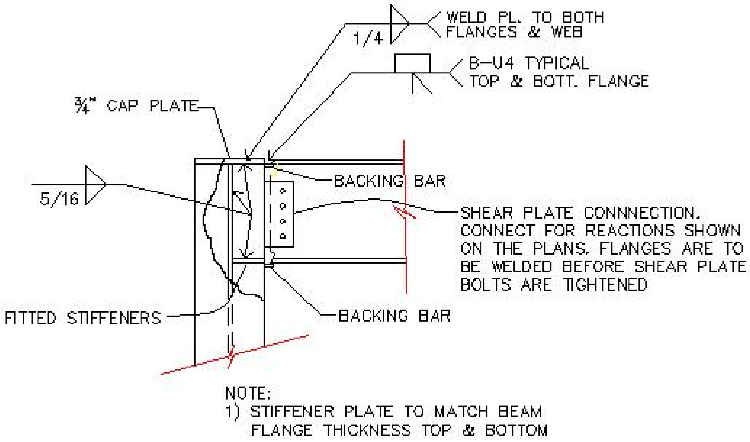
Appendix A



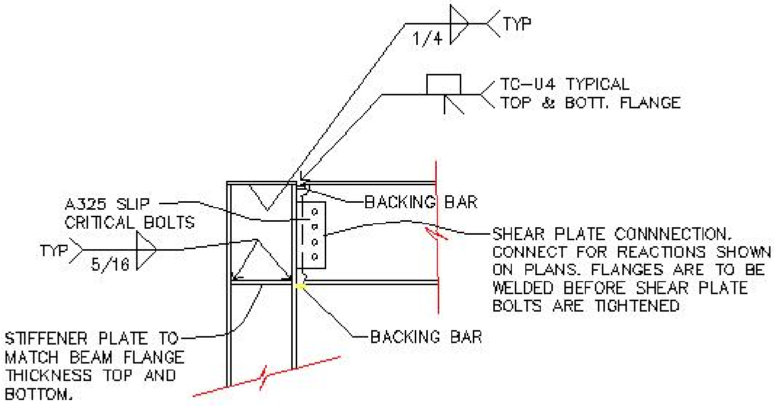
1st Floor
Floor Structure



2nd, 3rd, and 4th Floor
Floor Structure

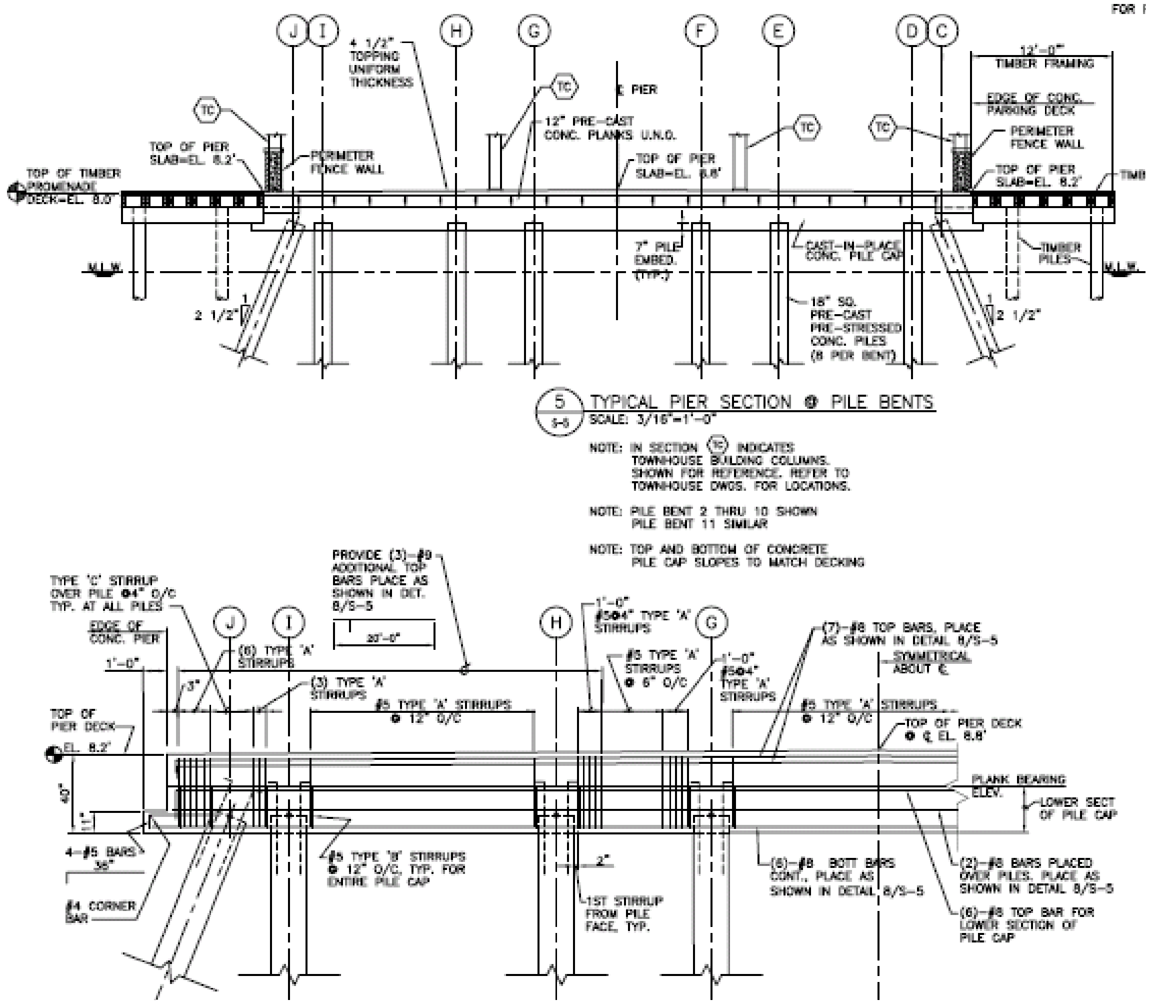


Typical Moment Connection 1st
floor frame.
Perpendicular to column Web



Typical Moment Connection
1st floor frame.
Parallel to column Web

Pier structure, typical concrete bent.



5 S-5 TYPICAL PIER SECTION @ PILE BENTS

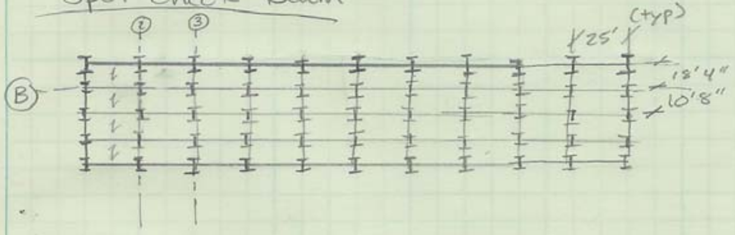
SCALE: 3/16"=1'-0"

NOTE: IN SECTION TC INDICATES TOWNHOUSE BUILDING COLUMNS. SHOWN FOR REFERENCE. REFER TO TOWNHOUSE DWGS. FOR LOCATIONS.

NOTE: PILE BENT 2 THRU 10 SHOWN PILE BENT 11 SIMILAR

NOTE: TOP AND BOTTOM OF CONCRETE PILE CAP SLOPES TO MATCH DECKING

Spot check Beam



Beam check along column line (B)   Column – (B) (2)

Columns – W12×96 (typ) $I_c = 833 \text{ in}^4$
Beams – W18×60 (typ) $I_g = 984 \text{ in}^4$

column height = 9'
Beam length = 25'   full lateral support from welded plates embeded in Precast Planks

load

assume weight of 3 stories above evenly distributed over slab

weight/level: live – 40 psf – LL
20 psf – Partition
60 psf

DEAD – 20 psf – Joist w/ sheating
10 psf shear walls
10 psf mech.
5 psf elec.
5 psf ceiling
12 psf roof/per floor
62 psf

3 Levels = 180 psf (LL), 186 psf (DL)

weight first floor: 60 psf – LL

Dead 20 psf snow
70 psf – Planks
30 psf – topping
1.5 psf – inslation
121.5 psf

$$W_u = 1.2(186 + 121.5)\text{psf} + 1.6(180 + 60)\text{psf}$$

$$W_u = 365 + 384 = 749 \text{ psf}$$

Beam check

trib width = $\frac{18'4'' + 10'8''}{2} = \frac{29'}{2} = 14.5'$

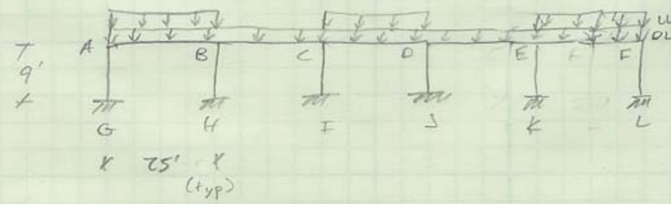
$W_u = 749$ psf

$w = 749$ psf $(14.5') + (80$ plf$) = 10.94$ K/ft total load

80 plf (Selfweight) Beam + 365 (14.5) = 5.37 K/ft DL

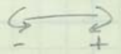
384 (14.5) = 5.57 K/ft LL

Since frame is 10 Bays long can analyze 5 Bay section from symmetry



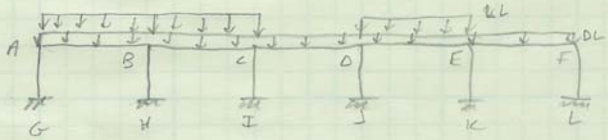
case I
Max (+) moment 1st span

FEM $\left(\frac{wL^2}{12}\right)$:



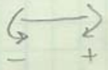
DL+LL
AB = - 570 ft k
BA = 570 ft k
CD = - 570 ft k
DC = 570 ft k
EF = - 570 ft k
FE = 570 ft k

DL
BC = - 280 ft k
CB = 280 ft k
DE = - 280 ft k
ED = 280 ft k



Case II
Max (-) moment @ B

FEM $\left(\frac{wL^2}{12}\right)$



DL+LL
AB = - 570 ft k
BA = 570 ft k
BC = - 570 ft k
CB = 570 ft k
DE = - 570 ft k
ED = 570 ft k

DL
CD = - 280 ft k
DC = 280 ft k
EF = - 280 ft k
FE = 280 ft k

Stiffness

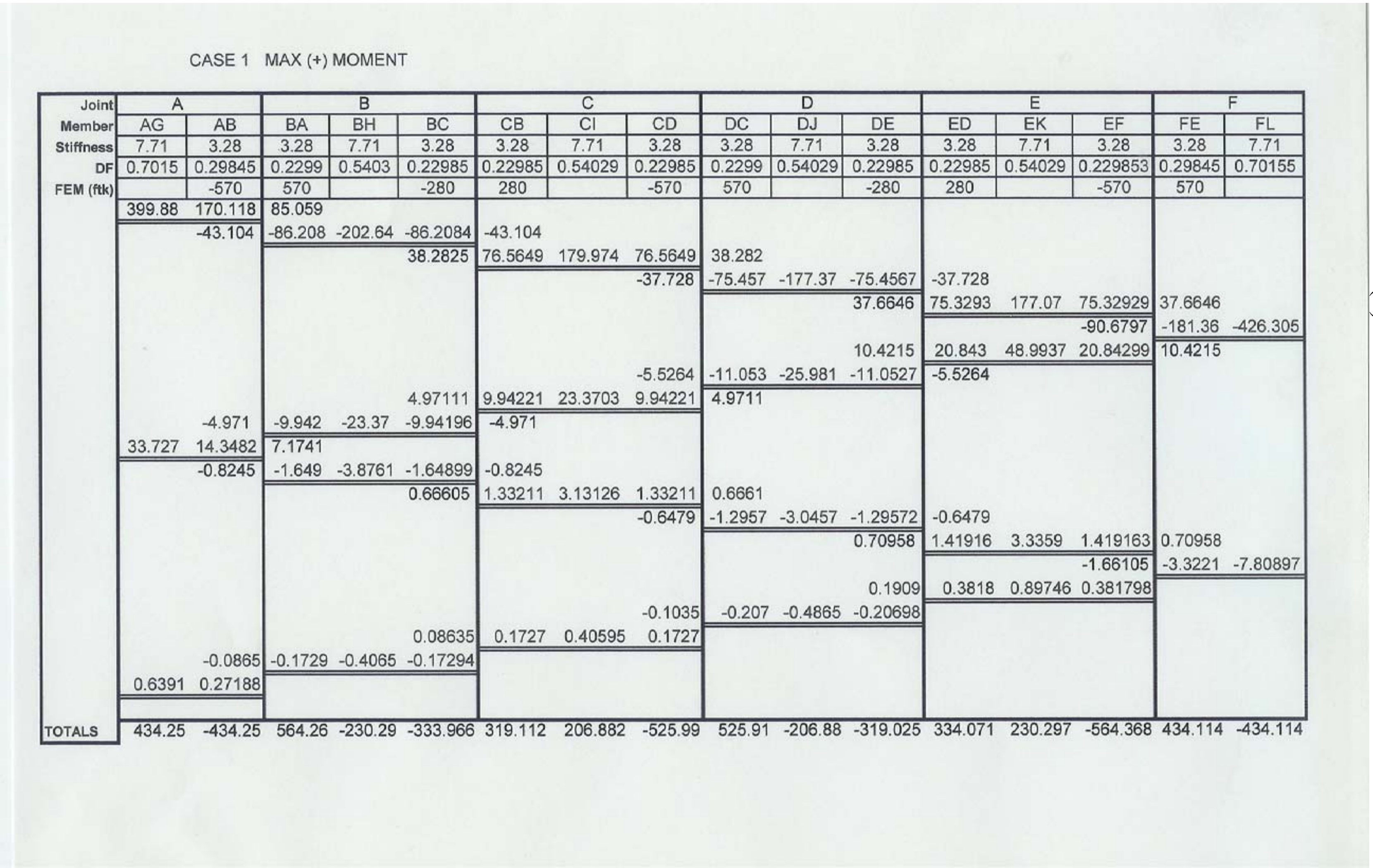
$K_C = I/L = \frac{833 \text{ in}^4}{9(12)}$

$K_C = 7.71 \text{ in}^3$

$K_B = I/L = \frac{984 \text{ in}^4}{25(12)}$

$K_B = 3.28 \text{ in}^3$

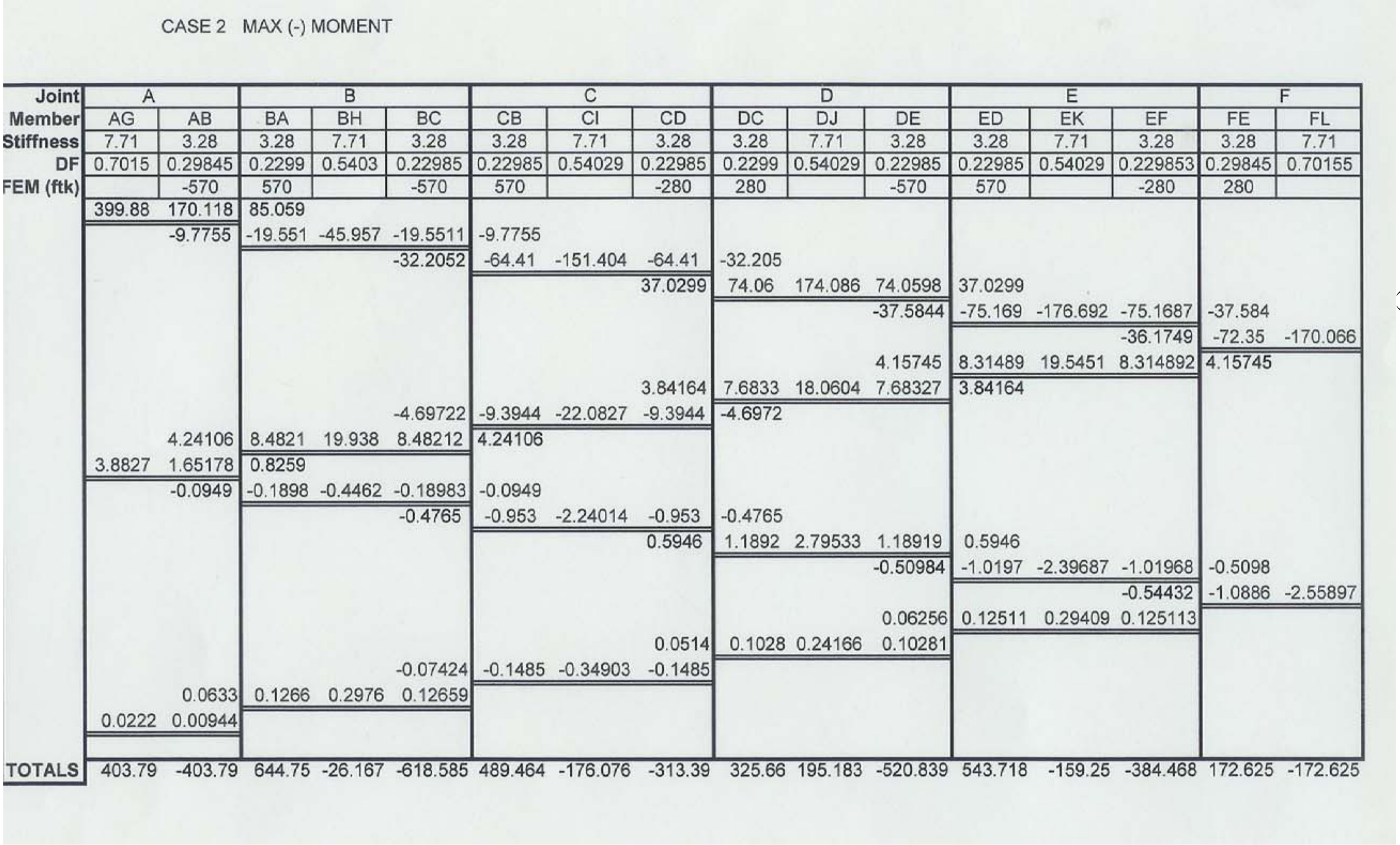
Moment distribution: max (+)



CASE 1 MAX (+) MOMENT

Joint	A		B			C			D			E			F	
Member	AG	AB	BA	BH	BC	CB	CI	CD	DC	DJ	DE	ED	EK	EF	FE	FL
Stiffness	7.71	3.28	3.28	7.71	3.28	3.28	7.71	3.28	3.28	7.71	3.28	3.28	7.71	3.28	3.28	7.71
DF	0.7015	0.29845	0.2299	0.5403	0.22985	0.22985	0.54029	0.22985	0.2299	0.54029	0.22985	0.22985	0.54029	0.229853	0.29845	0.70155
FEM (ftk)		-570	570		-280	280		-570	570		-280	280		-570	570	
	399.88	170.118	85.059													
		-43.104	-86.208	-202.64	-86.2084	-43.104										
					38.2825	76.5649	179.974	76.5649	38.282							
								-37.728	-75.457	-177.37	-75.4567	-37.728				
											37.6646	75.3293	177.07	75.32929	37.6646	
														-90.6797	-181.36	-426.305
											10.4215	20.843	48.9937	20.84299	10.4215	
								-5.5264	-11.053	-25.981	-11.0527	-5.5264				
					4.97111	9.94221	23.3703	9.94221	4.9711							
		-4.971	-9.942	-23.37	-9.94196	-4.971										
	33.727	14.3482	7.1741													
		-0.8245	-1.649	-3.8761	-1.64899	-0.8245										
					0.66605	1.33211	3.13126	1.33211	0.6661							
								-0.6479	-1.2957	-3.0457	-1.29572	-0.6479				
											0.70958	1.41916	3.3359	1.419163	0.70958	
														-1.66105	-3.3221	-7.80897
											0.1909	0.3818	0.89746	0.381798		
								-0.1035	-0.207	-0.4865	-0.20698					
					0.08635	0.1727	0.40595	0.1727								
		-0.0865	-0.1729	-0.4065	-0.17294											
	0.6391	0.27188														
TOTALS	434.25	-434.25	564.26	-230.29	-333.966	319.112	206.882	-525.99	525.91	-206.88	-319.025	334.071	230.297	-564.368	434.114	-434.114

Moment distribution: max (-)



CASE 2 MAX (-) MOMENT

Joint	A		B			C			D			E			F	
Member	AG	AB	BA	BH	BC	CB	CI	CD	DC	DJ	DE	ED	EK	EF	FE	FL
Stiffness	7.71	3.28	3.28	7.71	3.28	3.28	7.71	3.28	3.28	7.71	3.28	3.28	7.71	3.28	3.28	7.71
DF	0.7015	0.29845	0.2299	0.5403	0.22985	0.22985	0.54029	0.22985	0.2299	0.54029	0.22985	0.22985	0.54029	0.229853	0.29845	0.70155
FEM (ftk)		-570	570		-570	570		-280	280		-570	570		-280	280	
	399.88	170.118	85.059													
		-9.7755	-19.551	-45.957	-19.5511	-9.7755										
					-32.2052	-64.41	-151.404	-64.41	-32.205							
								37.0299	74.06	174.086	74.0598	37.0299				
											-37.5844	-75.169	-176.692	-75.1687	-37.584	
														-36.1749	-72.35	-170.066
											4.15745	8.31489	19.5451	8.314892	4.15745	
								3.84164	7.6833	18.0604	7.68327	3.84164				
					-4.69722	-9.3944	-22.0827	-9.3944	-4.6972							
		4.24106	8.4821	19.938	8.48212	4.24106										
	3.8827	1.65178	0.8259													
		-0.0949	-0.1898	-0.4462	-0.18983	-0.0949										
					-0.4765	-0.953	-2.24014	-0.953	-0.4765							
								0.5946	1.1892	2.79533	1.18919	0.5946				
											-0.50984	-1.0197	-2.39687	-1.01968	-0.5098	
														-0.54432	-1.0886	-2.55897
											0.06256	0.12511	0.29409	0.125113		
								0.0514	0.1028	0.24166	0.10281					
					-0.07424	-0.1485	-0.34903	-0.1485								
		0.0633	0.1266	0.2976	0.12659											
	0.0222	0.00944														
TOTALS	403.79	-403.79	644.75	-26.167	-618.585	489.464	-176.076	-313.39	325.66	195.183	-520.839	543.718	-159.25	-384.468	172.625	-172.625

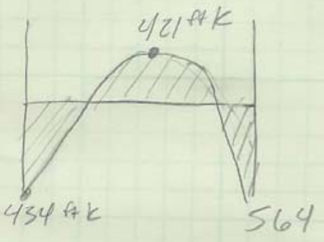
Beam check

From moment distribution

Max (+)

$$\frac{wL^2}{8} = \frac{10.94(25)^2}{8} = 855 \text{ ftk}$$

(-) end moment - (From moment distribution Case I)
-434



Max (+) = 855 - 434 = 421 ft k

Max (-)

Case II moment distribution
(-) BA = -645 kft
controls

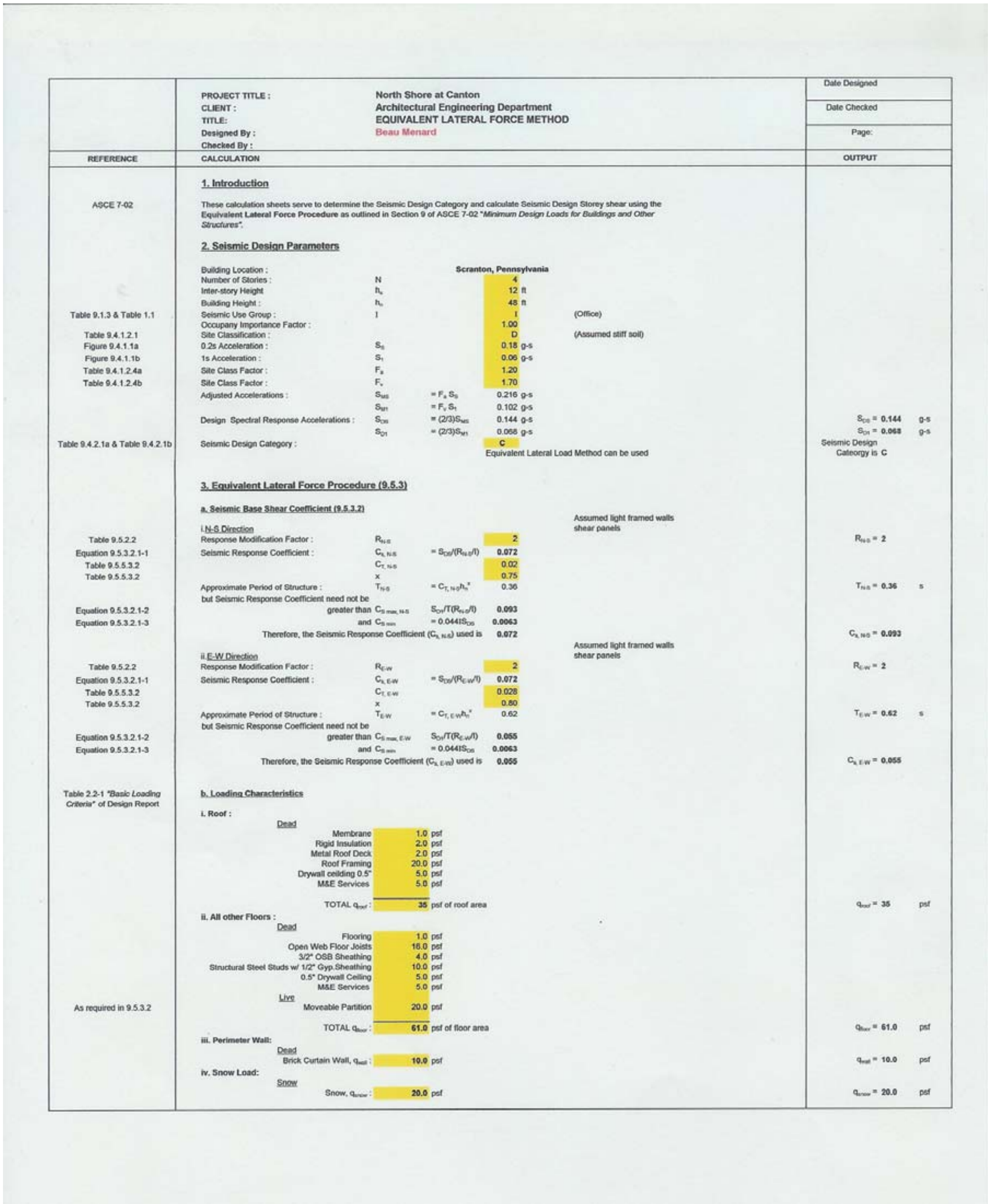
AISC pg 5-88

unbraced length = 0

W14x99 works
$\phi M_n = 646$ ft k

however
W24x68 more economical
$\phi M_n = 664$ ft k

Seismic Loading:

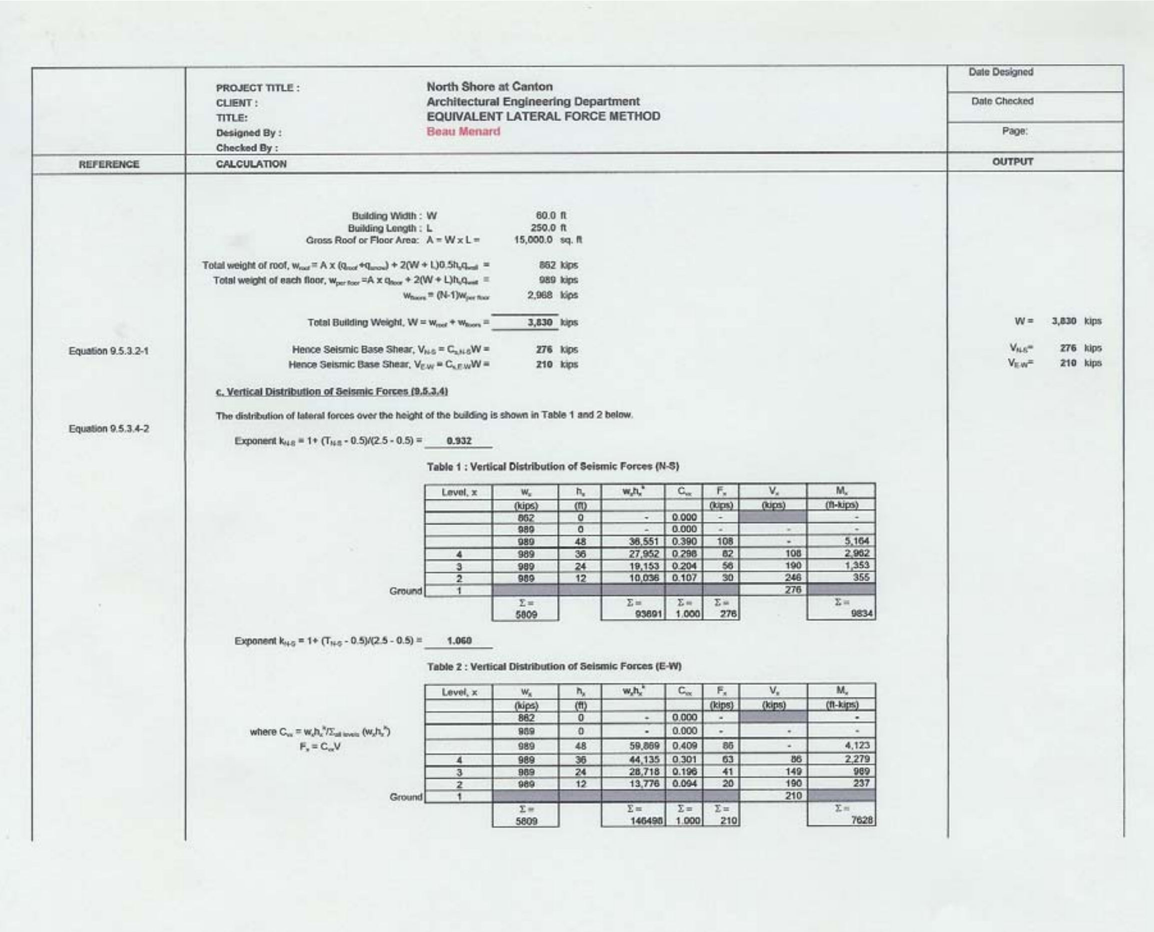


PROJECT TITLE :	North Shore at Canton		Date Designed
CLIENT :	Architectural Engineering Department		Date Checked
TITLE:	EQUIVALENT LATERAL FORCE METHOD		Page:
Designed By :	Beau Menard		
Checked By :			

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	$C_{S\,min}$	$= 0.044IS_{DS}$	**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	$C_{S\,min}$	$= 0.044IS_{DS}$	**0.0063**		
	Therefore, the Seismic Response Coefficient ($C_{s,E-W}$) used is			**0.055**		$C_{s,E-W}$ = **0.055**

REFERENCE	CALCULATION			OUTPUT
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

Seismic Results:



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

Building Width : W 60.0 ft

Building Length : L 250.0 ft

Gross Roof or Floor Area: A = W x 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 — 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 — $V_{N\text{-}S}$= **276** kips

Hence Seismic Base Shear, $V_{E\text{-}W} = C_{s,E\text{-}W}W$ = **210** kips — $V_{E\text{-}W}$= **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 $C_{vx} = w_x h_x^k / \Sigma_{all\ levels} (w_x h_x^k)$

$F_x = C_{vx} V$

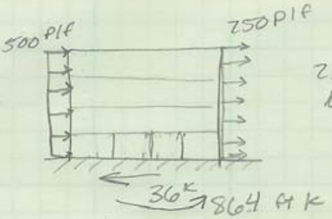
Level, x	w_x	h_x	$w_x h_x^k$	C_{vx}	F_x	V_x	M_x
	(kips)	(ft)			(kips)	(kips)	(ft-kips)
	862	0	-	0.000	-		-
	989	0	-	0.000	-	-	-
	989	48	59,869	0.409	86	-	4,123
4	989	36	44,135	0.301	63	86	2,279
3	989	24	28,718	0.196	41	149	989
2	989	12	13,776	0.094	20	190	237
Ground 1						210	
	Σ = 5809		Σ = 146498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

Lateral Distribution:

North Shore

lateral loading

Wind :

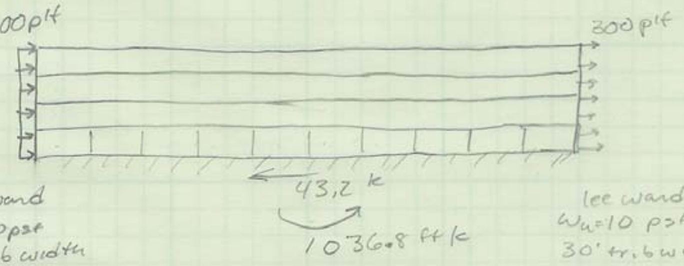


25' trib width
Wu = 20psf
wind ward

25' trib width
Wu = 10 psf
lee ward

IBC 1609.6.2.2.1

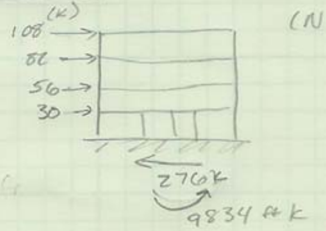
Typical story height – 12'



wind ward
Wu = 20psf
30' trib width

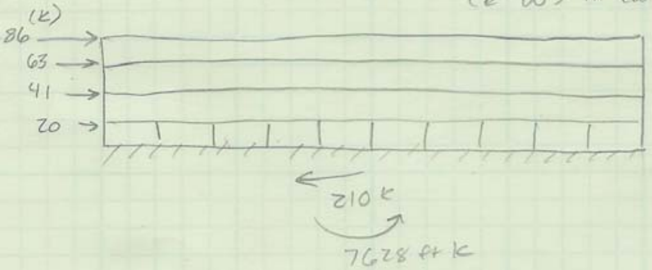
lee ward
Wu = 10 psf
30' trib width

Seismic – from seismic worksheet

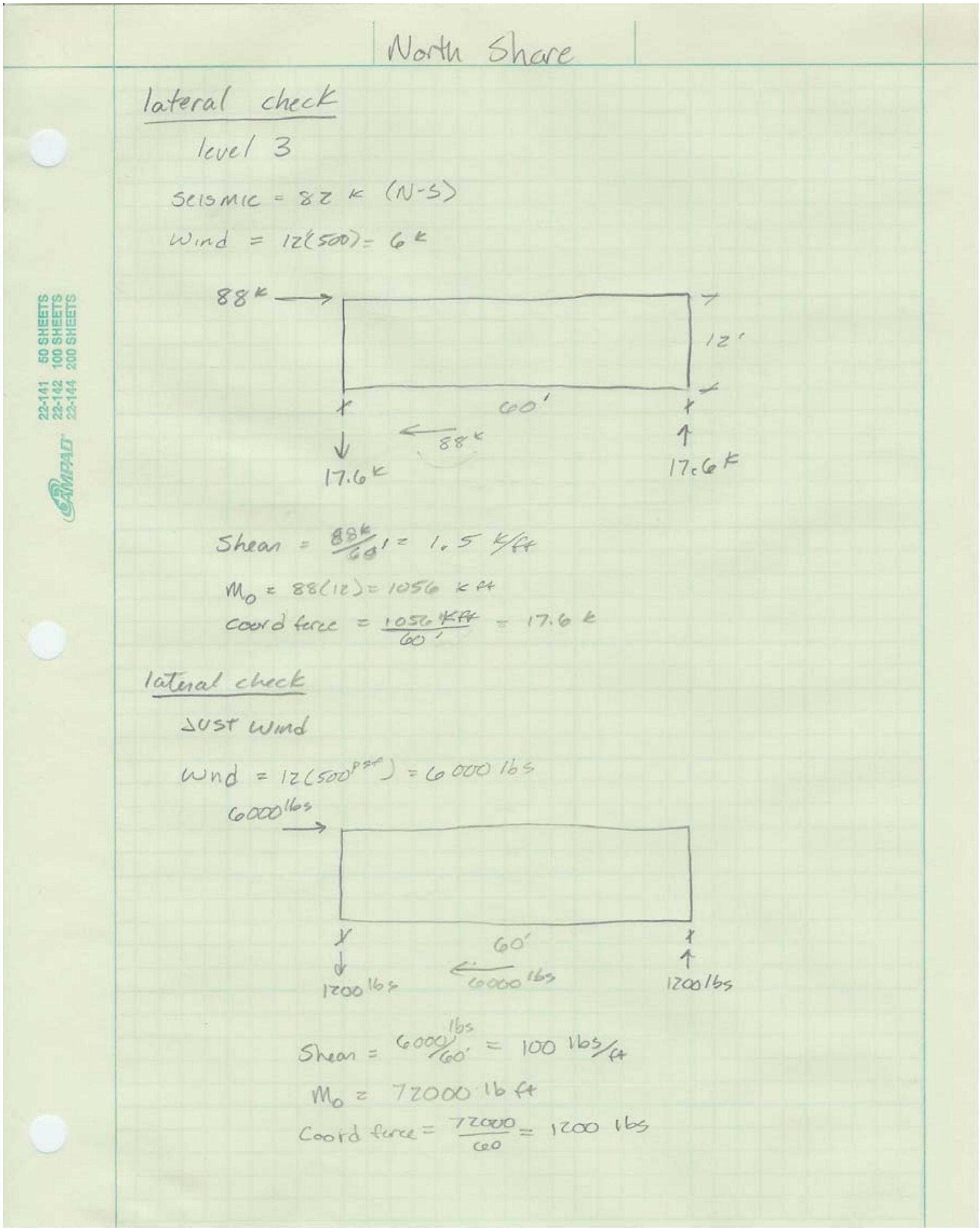


(N-S) in calculations

(E-W) in calculations



Lateral Check:



North Share

lateral check

level 3

seismic = 82^k (N-S)

Wind = 12(500) = 6^k

Shear = $88^k / 60' = 1.5$ k/ft

M_o = 88(12) = 1056 k ft

Coord force = $\frac{1056 \text{ kft}}{60'}$ = 17.6 k

lateral check

Just wind

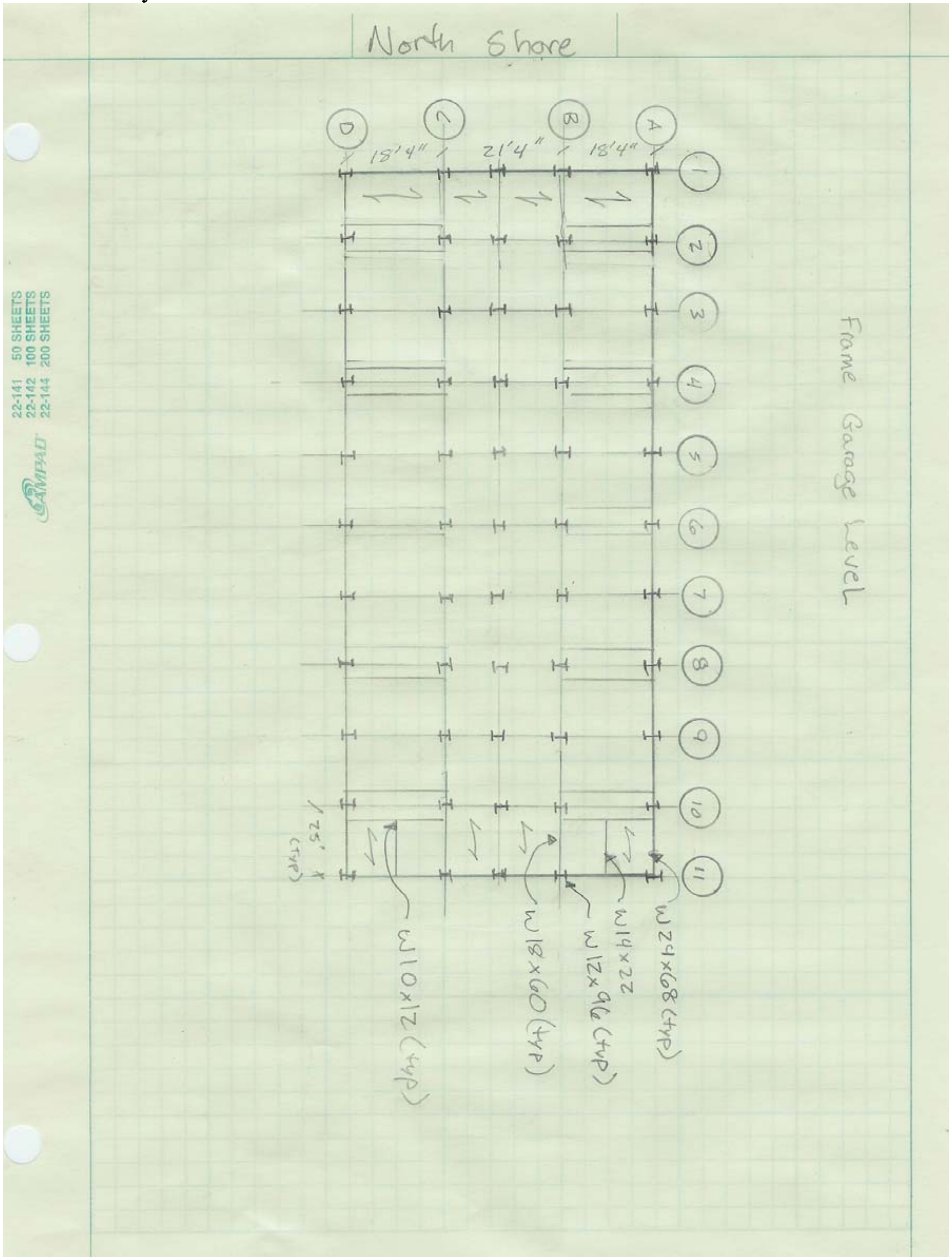
wind = 12(500 psf) = 6000 lbs

Shear = $6000^{\text{lbs}} / 60'$ = 100 lbs/ft

M_o = 72000 lb ft

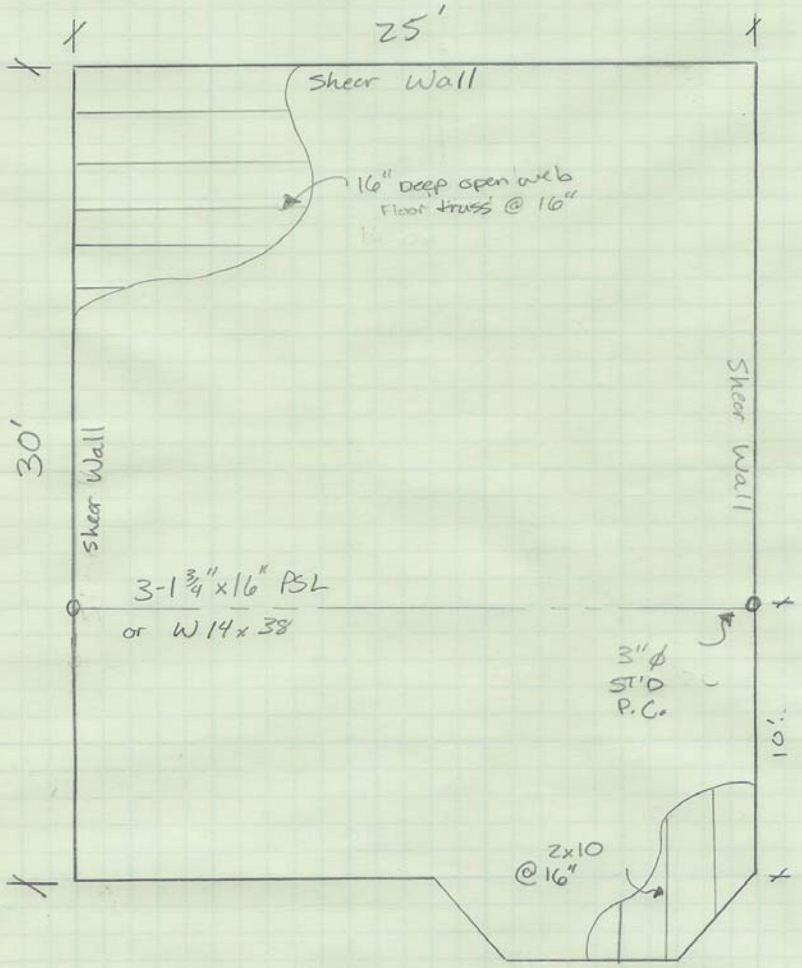
Coord force = $\frac{72000}{60}$ = 1200 lbs

Structural Layout:



North Shore

Typical Bay



Typical Shear Wall

sheathing	studs	chords	sheathing Attachment	Chord anchor
1/2" Gypsum (Both sides)	JW16 @ 16" O.C.	2-JW16	#6 TEKS Screws @ 6" o.c. (edge) @ 12" OC (field)	2-CS22 simpson straps

Shear truss design load ≃ 140 plf