T.C. WILLIAMS HIGH SCHOOL

ALEXANDRIA, VA

CHRISTOPHER B. DEKER

STRUCTURAL OPTION

TECHNICAL REPORT #1

05 OCTOBER 2007

FACULTY CONSULTANT: PROF PARFITT

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Executive Summary

The purpose of this technical report is to research and analyze the existing conditions and structural procedures used in the design of T.C. Williams High School in Alexandria, VA. The primary codes used in this report are IBC 2006, and ASCE 7-05. The school was originally designed with the Virginia State Building Code, which at the time referenced IBC 2000, and ASCE 7-99.

Building Description

T.C. Williams is a 3 Story 461,000 SF high school in Alexandria, VA, designed to accommodate 2,500 students. It was both designed architecturally and engineered by Moseley Architects, and later constructed by Hensel Phelps. Construction was completed during the Summer of 2007, and later opened in the Fall of 2007.

The building utilizes a composite slab with decking on steel frame construction. Due to the large size of the school, it was separated into six different 'buildings'. All together these six buildings have 4 different lateral resisting systems, the most common being Steel Concentrically Braced Frames. The others include Steel Moment Resisting Frames, and both Ordinary and Intermediate Masonry Shear Walls.

The original design of the school was done using ASD, while this technical report focuses on the design using LRFD. Due to both the difference in design methods, and the difference in building codes used, small discrepancies between my calculations and those of the engineer are expected. In no way does this report make the claim that any of the designer's approaches, assumptions, calculations, or resulting designs are incorrect or unsuitable.

Structural System Overview

Roof System

The typical flat roof system on T.C. Williams High School consists primarily of a Thermoplastic Polyolefin (TPO) Membrane system on 1½" 22 gauge steel roof deck, supported by K-Series Steel Joists which are typically spaced 5' O.C. The typical sloped roofing system is similar to the flat roofing system except instead of the TPO Membrane system there is a standing seam metal roof.

The typical roofing system over larger span areas such as the gymnasium and the auditorium consist of 3" 20 gauge steel roof deck, supported by DLH Steel Joists typically spaced 12' O.C.

Floor System

The typical floor is a composite system consisting of a 3" concrete slab on 1½" 18 gauge steel composite deck, supported by Steel Beams typically spaced 8' O.C. The concrete slab is made of Normal Weight Concrete (145 PCF) and has a minimum 28 day compressive strength (F'c) of 4000 PSI. The most typical Steel Beam is a W18x35 spanning a maximum of 34' with steel studs spaced at 12" O.C., but the range of steel beams may vary greatly depending on specific room requirements; generally ranging anywhere from a W16x26 to a W21x44. The steel studs creating the composite action are ¾" in diameter and 3½" long.

Foundation

All main building foundations are constructed on subgrade soils improved by the installation of a 'Geopier Rammed Aggregate Pier Soil Reinforcement' system and are designed to bear on strata capable of sustaining a minimum bearing pressure of 6,000 PSF. The slab on grade consists of Normal Weight Concrete (145 PCF) and has a minimum 28 day compressive strength (F'c) of 3,500 PSI. The slab is 4" thick and is reinforced with 6x6-W1.4xW1.4 WWF at mid depth. All spread and strip footings consist of Normal Weight Concrete (145 PCF) and have a minimum 28 day compressive strength (F'c) of 3,000 PSI.

LATERAL SYSTEM

The T.C. Williams High School is separated into 6 different "buildings" through the use of 'Fire Walls'. Both classroom towers are laterally supported with ordinary steel concentrically braced frames in both the N-S and E-W directions. The 3 story area connecting the 2 three story classroom towers is laterally supported with ordinary steel moment frames in both the N-S and E-W directions. The gymnasium and auditorium areas are supported by intermediate reinforced masonry shear walls, in all directions. The rest of the building which includes the area between the gymnasium and auditorium sections is laterally supported by ordinary reinforced masonry shear walls, in all directions.

COLUMNS

The columns are the primary gravity resisting member of the building. They consist of Grade 50 ASTM A992 wide flange shapes, grade 46 ASTM A500 rectangular HSS shapes, and grade 42 ASTM A500 round HSS shapes. The wide flange shapes generally run from a W10x49 to a W10x68, and is the primary support for most of the building. The Round HSS shapes are found connecting the two classroom wings and under the green roof, they generally run from a HSS12.750x.375 to a HSS16x.500.

CODES

ORIGINAL DESIGN CODES:

Virginia State Building Code (VUSBC), 2000 Edition

International Building Code (IBC), 2000 Edition

American Society of Civil Engineers (ASCE-7), 1999 Edition

Building Code Requirements for Structural Concrete (ACI 318-95)

Standard Specifications for Structural Concrete (ACI 301-96)

AISC Code of Standard Practice for Steel Buildings, 2000 Edition

AISC Specification for Structural Steel Buildings, Allowable Stress Design and Plastic Design, 1989 Edition

THESIS DESIGN CODES:

International Building Code (IBC), 2006 Edition

American Society of Civil Engineers (ASCE-7), 2005 Edition

AISC Steel Construction Manual, LRFD, 13^{th} Edition

THESIS DEFLECTION CRITERIA:

TOTAL = L / 240

LIVE = L / 360

CONSTRUCTION = L / 360

MASONRY WALLS = L / 600

LOADS

TYPICAL ROOF DEAD LOAD	THESIS DESIGN
TPO Membrane / S.S. metal Roof	3 psf
4"-6" Rigid Insulation	2.5 psf
$1^1/_2$" - 3" Galvanized Steel Deck	2 psf
K-Series Steel Joists	3.5 psf
Ceiling Finishes	5 psf
Mechanical / Electrical	6.5 psf
Sprinklers	2.5 psf
TOTAL	25 psf

TYPICAL FLOOR DEAD LOAD	THESIS DESIGN
3" NWC Slab (145 pcf)	38 psf
18 gauge $1^1/_2$" Composite Deck	3 psf
Steel Beams	5 psf
Ceiling Finishes	5 psf
Mechanical / Electrical	6.5 psf
Sprinklers	2.5 psf
TOTAL	60 psf

TYPICAL ROOF LIVE LOAD	THESIS DESIGN	CODE REFERENCE
Minimum Roof LL	20 psf	ASCE 7-05 Section 4.9.1
Ground Snow Load (Pg)	25 psf	IBC Figure 1608.2
Importance Category III	Is = 1.10	IBC Section 1604.5
Exposure Factor	Ce = 1.0	IBC Table 1608.3.1
Thermal Factor	Ct = 1.0	IBC Table 1608.3.2
Flat Roof Snow Load	19.25 psf + Drift	IBC Section 1608.3
Drift	Varies	ASCE 7-05 Section 7.7

FLOOR LIVE LOADS	THESIS DESIGN	ORIGINAL DESIGN	ASCE 7-05 MIN VALUE
Classroom	50 psf	50 psf	40 psf
First Floor Corridor	100 psf	100 psf	100 psf
Above First Floor Corridor	80 psf	80 psf	80 psf
Offices	50 psf	50 psf	50 psf
Light' Storage	125 psf	125 psf	125 psf
Mechanical	150 psf	150 psf	n/a
Green Roof	100 psf	100 psf	n/a
Library Stacks	150 psf	150 psf	150 psf

Analyses And Conclusions

Seismic

Normally seismic forces wouldn't be much of a problem in Alexandria, VA, but due to extremely poor soil conditions, and small R values for the lateral resisting elements, seismic proved to be a much bigger factor than at first thought. These R values differ greatly from what the engineer had originally intended. When this building was designed, there were much more lenient design codes for seismic, but since then the newer versions of the code such as ASCE 7-05 have greatly reduced these R-Values.

Using the equivalent lateral force method, I obtained a base shear of 488 kips for building A, and 246 kips for building E. I believe these two 'buildings' best represent the others. When calculating the weight of the building I included 100% of the buildings dead load, along with 25% of storage rooms, and 100% of equipment operating weight if available. Included in the Dead load was, interior CMU partitions, exterior walls, and the dead load weight of the floor system.

Wind

After examining the seismic results of building A, I knew wind wasn't going to be a huge factor. I knew seismic would govern in the N-S direction of building A, but completed a wind analysis to see how wind would compare to seismic in the E-W direction.

Using Method 2 'The Analytical Procedure', for building A, I obtained a base shear of 88 kips in the N-S direction, which I originally expected due to the shape of the building. In the E-W direction I obtained a base shear of 244 kips. For building B, I obtained a base shear of 173 kips in the N-S direction, and 147 kips in the E-W direction.

Seismic vs Wind

Comparing seismic design base shear of building 'A' (488 kips) to wind design base shear (244 kips), gives a clear idea that seismic has always governed. However, comparing seismic design base shear of building 'E' (246 kips) to wind design base shear (173 kips) lends more interesting results. From inspection it's difficult to tell what originally governed the lateral design. The building may have been controlled by wind, but due to the newer codes with stricter seismic designs the seismic design base shear will govern.

SEISMIC ANALYSIS

SEISMIC ANALYSIS	THESIS DESIGN	CODE
Analysis Procedure	Equivalent Lateral Force Procedure	ASCE 7 Section 12.8
Importance Category	III	ASCE 7 Table 1-1
Importance Factor (I_E)	1.25	ASCE 7 Table 11.5-1
Seismic Category	II	ASCE 7 Section 11.6
Site Class	D	IBC Table 1613.5.2
Spectral Acceleration for Short Periods (Ss)	15.30%	IBC Figure 1615 (1)
Spectral Acceleration for 1 Second Periods (S1)	5.00%	IBC Figure 1615 (2)
Site Coefficient, Fa	1.6	ASCE 7 Table 11.4-1
Site Coefficient, Fv	2.4	ASCE 7 Table 11.4-2
S_{MS}	0.2448	ASCE 7 Section 11.4.3
S_{M1}	0.12	ASCE 7 Section 11.4.3
S_{DS}	0.1632	ASCE 7 Section 11.4.4
S_{D1}	0.08	ASCE 7 Section 11.4.4
Seismic Design Category	B	ASCE 7 Table 11.6-1,2
Structural System - Building 'A'	Ordinary Steel Concentrically Braced Frames	ASCE 7 Table 12.2-1
Structural System - Building 'B'	Ordinary Steel Concentrically Braced Frames	ASCE 7 Table 12.2-1
Structural System - Building 'C'	Ordinary Steel Moment Frames	ASCE 7 Table 12.2-1
Structural System - Building 'D'	Ordinary Reinforced Masonry Shear Walls	ASCE 7 Table 12.2-1
Structural System - Building 'E'	Intermediate Reinforced Masonry Shear Walls	ASCE 7 Table 12.2-1
Structural System - Building 'F'	Intermediate Reinforced Masonry Shear Walls	ASCE 7 Table 12.2-1
R Factor - Building 'A'	3.25	ASCE 7 Table 12.2-1
R Factor - Building 'B'	3.25	ASCE 7 Table 12.2-1
R Factor - Building 'C'	3.5	ASCE 7 Table 12.2-1
R Factor - Building 'D'	2.0	ASCE 7 Table 12.2-1
R Factor - Building 'E'	3.5	ASCE 7 Table 12.2-1
R Factor - Building 'F'	3.5	ASCE 7 Table 12.2-1
Deflection Modification Factor - Building 'A'	3.25	ASCE 7 Table 12.2-1
Deflection Modification Factor - Building 'B'	3.25	ASCE 7 Table 12.2-1
Deflection Modification Factor - Building 'C'	3.0	ASCE 7 Table 12.2-1
Deflection Modification Factor - Building 'D'	1.75	ASCE 7 Table 12.2-1
Deflection Modification Factor - Building 'E'	2.25	ASCE 7 Table 12.2-1
Deflection Modification Factor - Building 'F'	2.25	ASCE 7 Table 12.2-1

Seismic Design | TC Williams High School

Importance Cat. III ~ $I_E = 1.25$
Seismic Cat. II ~ SUG = II
Site Class ~ D

Building Height = 50'

Alexandria, VA 22302

$S_s = 15.3\%$ $F_a = 1.6$
$S_1 = 5.0\%$ $F_v = 2.4$

$S_{ms} = F_a S_s$
$= (1.6)(0.153) = 0.2448$

$S_{m1} = F_v S_1$
$= (2.4)(0.05) = 0.120$

$S_{DS} = \frac{2}{3} S_{ms}$
$= (\frac{2}{3})(0.2448) = 0.1632$

$S_{D1} = \frac{2}{3} S_{m1}$
$= \frac{2}{3}(0.120) = 0.080$

For $S_{DS} = 0.1632$ & SUG = II ⇒ SDC = A
For $S_{D1} = 0.80$ & SUG = II ⇒ SDC = B → controls

Seismic Design Catagorie : SDC = B

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a) Load Bearing Masonry Walls w/ intermediate reinforced masonry shear walls

R = 3.5 - BUILDING 'E & F'

b) Building Frame System w/ ordinary steel concentrically braced frames

R = 3.25 - BUILDING 'A & B'

c) Building Frame system w/ ordinary reinforced masonry shear walls

R = 2.0 - BUILDING 'D'

d) Moment Resisting Frame system w/ ordinary steel moment frames

R = 3.5 - BUILDING 'C'

SDC = B

$S_{DS} = 0.1632$ $S_{D1} = 0.080$

a) $R = 3.5$
b) $R = 3.25$
c) $R = 2.0$
d) $R = 3.5$

Determination of T

$S_{D1} = 0.080 \leq 0.1 \Rightarrow C_u = 1.7$

$T_a = C_T \cdot h_n^x$

$T_a = 0.02\,(h_n)^{0.75}$

$T = C_u T_a = 1.7\,(0.02\,(h_n)^{0.75})$

$C_s = S_{DS}/(R/I)\ \{\leq S_{D1}/(R/I)T,\ \geq 0.044\,S_{DS}\,I.\}$

a) $C_s = 0.1632/\left(\frac{3.5}{1.25}\right) = 0.058$

$h_n = 36'$

$T = 1.7(0.02(36)^{0.75}) = 0.50\ s$

$C_s \leq S_{D1}/((R/I)\cdot T) = 0.08/\left(\left(\frac{3.5}{1.25}\right)\cdot 0.50\right) = \underline{0.057}$ → controls

$C_s \geq 0.044\,S_{DS}\,I = 0.044\,(0.1632)(1.25) = 0.009$

$\boxed{C_s = 0.057}$ - BUILDING E & F

b) $C_s = S_{DS}/(R/I) = 0.1632/\left(\frac{3.25}{1.25}\right) = 0.063$

$h_n = 45'$

$T = 1.7\,(0.02(45)^{0.75}) = 0.59\ s$

$C_s \leq S_{D1}/(R/I)\cdot T = 0.08/(3.25/1.25)\times 0.59 = \underline{0.052}$ → controls

$C_s \geq 0.044\,S_{DS}\,I = 0.009$

$\boxed{C_s = 0.052}$ - BUILDING A & B

c) $C_s = S_{DS}/(R/I) = 0.1632/(\frac{2}{1.25}) = 0.102$

$h_n = 45'$

$T = 1.7(0.02(45)^{0.75}) = 0.59\ s$

$C_s \le S_{D1}/(\frac{R}{I}) \times T = 0.08/(\frac{2}{1.25}) \times 0.59 = \underline{0.085}$ → controls

$C_s \geq 0.009$

$\boxed{C_s = 0.085}$ - BUILDING 'D'

d) $C_s = 0.1632/(\frac{3.5}{1.25}) = 0.058$

$h_n = 45'$

$T = 0.59\ s$

$C_s \le 0.08/(\frac{3.5}{1.25}) \times 0.59 = \underline{0.048}$ → controls

$C_s \geq 0.009$

$\boxed{C_s = 0.048}$ - BUILDING 'C'

BUILDING WEIGHT, W

BUILDING A

FLOOR 2

FLOOR DL = 60 psf (38,000 SF) = 2,280 K
WALL LOAD = 35 psf (10'8" × 1700') = 635 K
25% STORAGE = 125 psf (1340 SF)(0.25) = 42 K
Ext. WALLS = 100 psf (450' × 15') = 675 K
Int. FIRE WALLS = 54 psf (330' × 15') = 267 K

3,900 K

FLOOR 3

FLOOR DL = 2280 K
WALL LOAD = 635 K
25% STORAGE = 125 psf (620 SF)(0.25) = 19.4 K
Ext. WALLS = 675 K
INT FIREWALLS = 267 K

3880 K

ROOF

ROOF DL = 25 psf (38,000 SF) = 950 K
WALL LOAD = 35 psf (3'2" × 1700') = 188.5 K
EXT WALLS = 100 psf (7.5' × 450') = 337.5 K
INT WALLS = 54 psf (7.5' × 330') = 133.5 K

1,610 K

TOTAL WEIGHT = 9,390 K

$C_s = 0.052$

$V = C_s \times W$

$\boxed{V = 488\ K}$

BUILDING WEIGHT, W

BUILDING E

FLOOR 2

FLOOR DL = 40,500 SF × 25 psf = 1,012.5^k
11,000 SF × 60 psf = 660 K

Mechanical = 2,400 SF × 150 psf = 360^k

WALL LOAD BELOW = 3'2" × 34' × 50 × 35 psf = 188^k

WALL LOAD ABOVE = 7'6" × 34' × 10 × 35 psf = 89^k

Ext. WALL BELOW = (282 + 287 + 325) × 7.5' × 100 psf = 670.5^k

Ext WALL ABOVE = 420 × 7.5' × 100 psf = 315^k

INTERIOR FIRE WALL = 325' × 15' × 54 psf = 263^k

3,560^k

FLOOR 3 (ROOF)

FLOOR DL = 11,000 × 25 psf = 275^k

WALL LOW BELOW = 3'2" × 34' × 10 × 35 psf = 37.7^k

Ext. WALL BELOW = 420 × 7.5' × 100 psf = 315^k

INTERIOR FIRE WALL = 325 × 7.5' × 54 psf = 131.6^k

760^k

TOTAL WEIGHT = 4320 K

$C_s = 0.057$

$V = C_s \times W$

V = 246 K

Seismic Summary

Weight, W

Total DL

25% Storage LL (if available)

Partition Loads (if available)

Equipment Operating Weight (if available)

20% Flat Roof Snow Load *if* $P_f > 30$ psf

Base Shear (Building 'A')

V = Cs * W

Cs = 0.052

W = 9,390 kips

V = 0.052 * 9,390 k = 488 kips

Base Shear (Building 'E')

V = Cs * W

Cs = 0.057

W = 4,320 kips

V = 0.057 * 4,320 k = 246 kips

Wind Analysis

Wind Analysis	Thesis Design	Code
Importance Category	III	ASCE 7 Table 1-1
Importance Factor, I_w	1.15	ASCE 7 Table 11.5-1
Basic Wind Speed, V	90mph	ASCE 7 Figure 6-1C
Directionality Factor, K_d	0.85*	ASCE 7 Table 6-4
Exposure Category	B	ASCE 7 Section 6.5.6.3
Topographic Factor, K_{zt}	1.0	ASCE 7 Figure 6-4
Gust Factor, G	0.85	ASCE 7 Section 6.5.8
Resonant Response Factor	1.0	ASCE 7 3.5.8.2
Mean Roof Height	45'	n/a
Enclosure Classification	Enclosed	ASCE 7 Section 6.5.9
Internal Pressure Coefficient, GC_{pi}	±0.18	ASCE Figure 6.5
Reduction Factor, R_i	1.0	ASCE 7 Section 6.5.11.1.1

*K_d is only permitted to be used in combination with load cases

External Pressure Coefficients, Cp	
Windward	0.8
E-W Leeward - Building 'A'	-0.5
N-S Leeward - Building 'A'	-0.2875
E-W Leeward - Building 'E'	-0.472
N-S Leeward - Building 'E'	-0.5
Side Wall	-0.7

z	k_z	q_z
0-15	0.57	13.59
30	0.7	16.69
45	0.785	18.72

PRESSURE				
N-S Building A				
WINDWARD		LEEWARD		TOTAL
h (ft)	P (psf)	h (ft)	P (psf)	-
0-15	12.6	0-15	-1.2	13.8
30	14.7	30	-1.2	15.9
45	16.1	45	-1.2	17.3
E-W Building A				
WINDWARD		LEEWARD		TOTAL
h (ft)	P (psf)	h (ft)	P (psf)	-
0-15	12.6	0-15	-4.6	17.2
30	14.7	30	-4.6	19.3
45	16.1	45	-4.6	20.7

PRESSURE				
N-S Building E				
WINDWARD		LEEWARD		TOTAL
h (ft)	P (psf)	h (ft)	P (psf)	-
0-15	12.6	0-15	-4.6	17.2
30	14.7	30	-4.6	19.3
E-W Building E				
WINDWARD		LEEWARD		TOTAL
h (ft)	P (psf)	h (ft)	P (psf)	-
0-15	12.6	0-15	-4.1	16.7
30	14.7	30	-4.1	18.8

DESIGN BASE SHEAR

BUILDING A

V = AVG WIND OVER AREA OF WALL

$V_{A,N\text{-}S}$ = (13.8 psf * 15’ * 130’)+((15.9+13.8) / 2 psf * 15’ * 130’)+((17.3+15.9) / 2 psf * 15’ * 130’)

$V_{A,N\text{-}S}$ = 88 kips

$V_{A,E\text{-}W}$ = (17.2 psf * 15’ * 293’)+((17.2+19.3) / 2 psf * 15’ * 293’)+((19.3+20.7) / 2 psf * 15’ * 293’)

$V_{A,E\text{-}W}$ = 244 kips

$V_{E,N\text{-}S}$ = (17.2 psf * 15’ * 325’)+((19.3+17.2) / 2 psf * 15’ * 325’)

$V_{E,N\text{-}S}$ = 173 kips

$V_{E,E\text{-}W}$ = (16.7 psf * 15’ * 285’)+((18.8+16.7) / 2 psf * 15’ * 285’)

$V_{E,E\text{-}W}$ = 147 kips

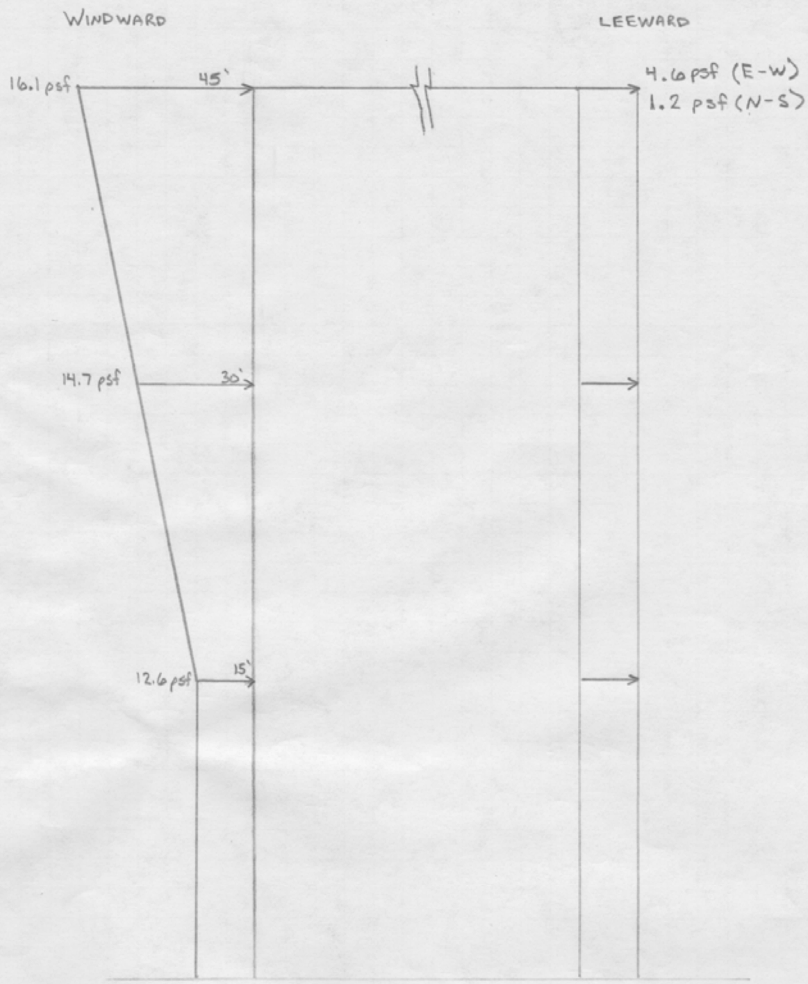
Wind Pressure, E - W

WINDWARD

$P = q_z G C_p - q_h(-G C_{pi})$

LEEWARD

$P = q_h G C_p - q_h(-G C_{pi})$

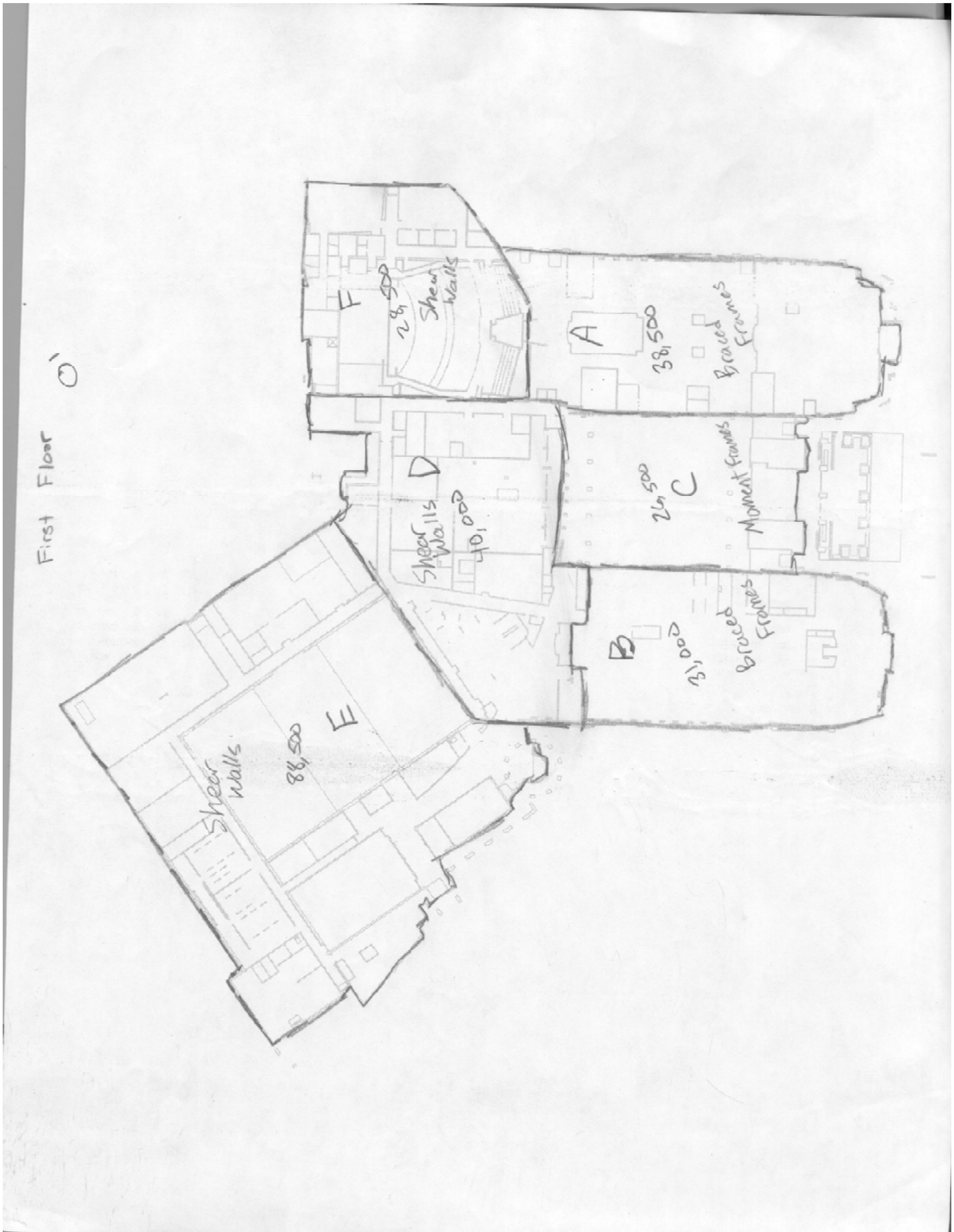


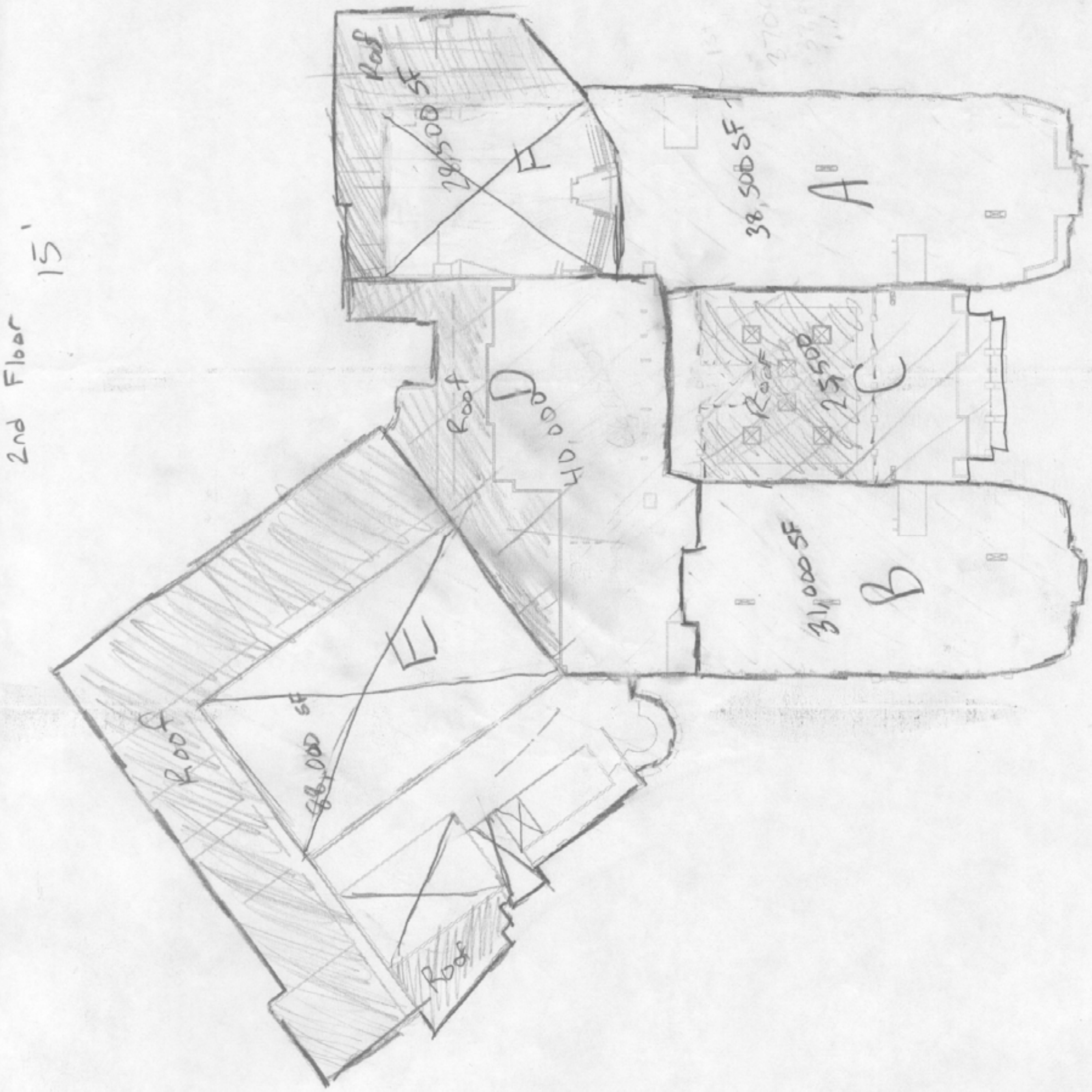
NOTE: DIST ON WINDWARD SIDE IS NOT LINEAR

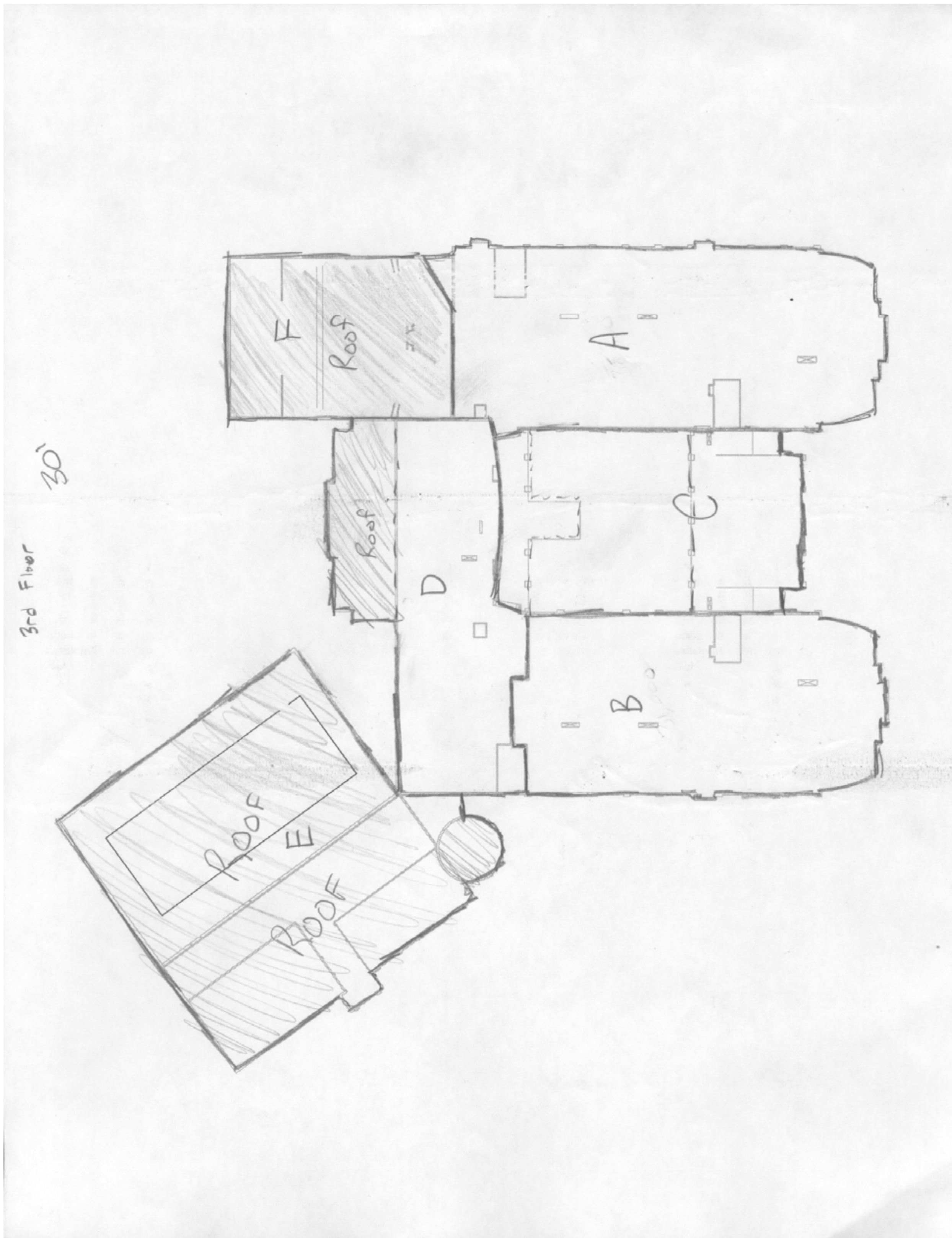
BASE SHEAR:

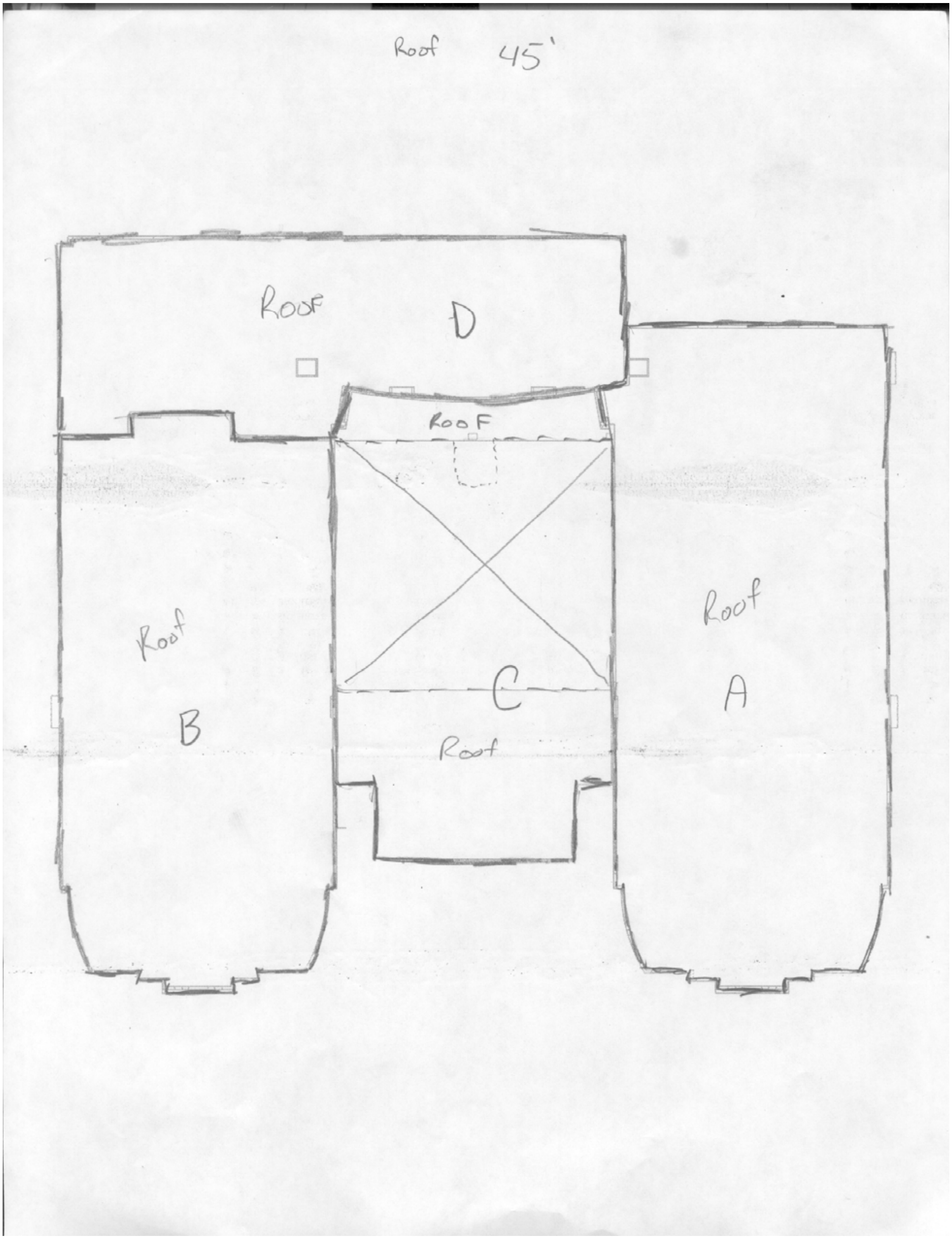
N-S	E-W
V = 88 k	V = 244 k

APPENDICES


First Floor
0'
F
29,500
Shear Walls
A
38,500
Braced Frames
D
Sheer Walls
40,000
C
26,500
Moment Frames
B
31,000
Braced Frames
E
88,500
Sheer Walls


2nd Floor
15'
Roof
28,500 SF
F
38,500 SF
A
Roof
D
40,000
Roof
C
31,000 SF
B
E
68,000 SF
Roof
Roof


3rd Floor
30'
F
Roof
A
C
D
Roof
B
ROOF
E
ROOF


Roof
45'
Roof
D
Roof
Roof
Roof
C
A
B
Roof

Spot Checks

After completing the calculations, I found a few discrepancies between my designs and that of the engineers. These discrepancies could happen for a number of reasons. For example; difference in the design method used (LRFD vs. ASD), Code changes from 2000 – 2006, simplified assumptions, and the need to standardize a design to make the construction process easier. In no way do these calculations try to make the claim that any of the designer's approaches, assumptions, calculations, or resulting designs are incorrect or unsuitable.

Composite Beam, B1

When designing a typical beam I was surprised to have a nearly equal design as that of the engineer. The major difference was I found the beam to only require 18 studs, while the engineer designed for studs spaced 12" O.C. which equates to 34 studs. Looking over the other designs in the building I was able to observe that nearly all of the composite beams had studs spaced 12"O.C., which could be one of the possible reasons for the overdesign. The main control factor in the design was for the criteria of the Masonry walls supported by the beam. They require a Live Load + wall weight Deflection of L/600, to prevent unwanted cracking.

Composite Girder, G1

When designing a typical girder I found my first major discrepancy between my design and that of the engineers. Through the use of the codes used in this tech report I found a required girder size of W21x44 with 16 studs. The engineer had designed the girder to be a W21x50 with studs spaced 12" O.C. which would provide 23 studs. Again the main controlling factor in the design was the criteria of the masonry wall deflection of L/600.

Column, C1

When designing a typical column I again found a small discrepancy between my design and that of the engineers, but again that was to be expected. I found a W10x49 to be sufficient to carry the loads while the engineer called for a W10x54, which would be the next size up.

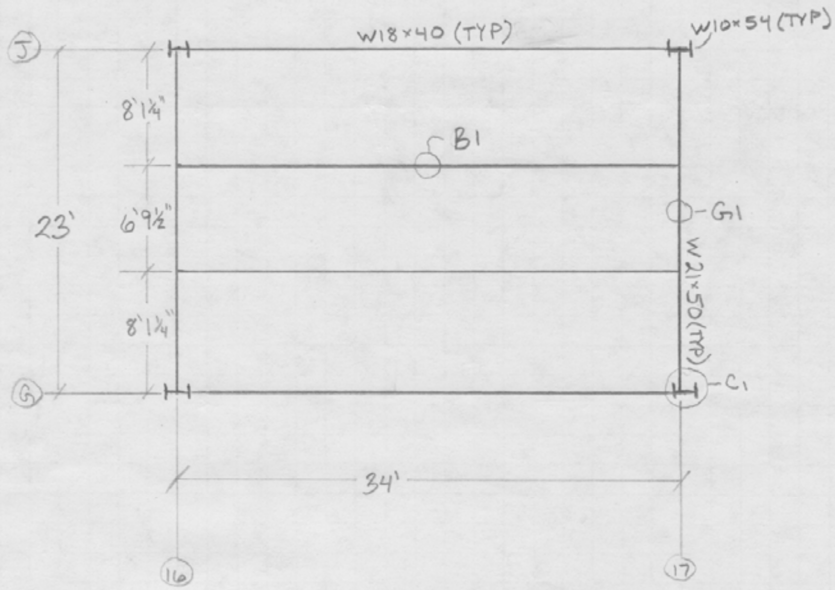
Conclusion

Given the results of the few spot checks done, I'm able to easily state that the engineer's designs are slightly more conservative than my own, be it due to code, or standardizing of materials. However, all of these designs where relatively close to those of my own, therefore I'm able to conclude that the Engineer's designs nearly match my own, which is what I was hoping to examine in this tech report.

SPOT CHECK

TYP. BAY IN BUILDING A

SECOND FLOOR



LOADING: OFFICE AREA w/ PARTITIONS

LL: 50 psf OFFICE
20 psf PARTITIONS
70 psf TOTAL

DL: 60 psf TOTAL

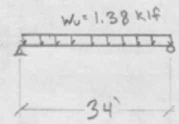
FACTORED LOAD (LRFD)

$w_u = 1.2\ D + 1.6\ L$

$w_u = 184$ psf

BEAM 1

$f_y = 50$ ksi
$f'_c = 4$ ksi

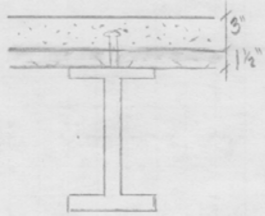


W = 184 psf
TRIB WIDTH = 7.5'

$W_u = 1380$ plf = 1.38 klf

$M_u = \frac{W_u L^2}{8} = \frac{1.38(34')^2}{8}$

$M_u = 199.4'^k$



1 weak stud / rib
$f'_c = 4$ ksi
¾" DIAMETER SHEAR STUD
3½" LENGTH
→ $Q_n = 17.2^k$

$b_{eff} = \frac{34'}{4} = 8.5' = 102"$

$= 6'9½" = 81.5"$ ← Controls

Assume a = 1"

$\frac{a}{2} = (4½) - (½") = 4"$ Y2 = 4"

① FIND $I_{req.}$ TO LIMIT DEFLECTION TO L/600 (REQ'D DEFL FOR MASONRY)

L/600 = 0.68" $W_{LL} = 70$ psf

SOLVING $\left(\frac{5}{384}\right)\left(\frac{0.525(34)^4(1728)}{29{,}000\ x}\right) = 0.68"$ ⇒ $x = 800\ in^4$
-Min req.d I_{LB}

② FIND $I_{req.}$ TO LIMIT CONST. DL TO L/360

L/360 = 1.13" $W_{const} = 38$ psf $W_{LL} = 20$ psf $W_u = [1.2(38) + 1.6(20)] \times 7.5'$
$W_u = 0.582$ klf

SOLVING $\left(\frac{5}{384}\right)\left(\frac{0.582(34)^4(1728)}{29{,}000\ x}\right) = 1.13"$ ⇒ $x = 534\ in^4$
-Min req'd I (bear beam)

TRY
W18×40 ⑦
$\Sigma Q_n = 147^k$
$\phi M_n = 411'^k > M_u = 199'^k$ ok
$I_{BB} = 612\ in^4 > I_{BB\ req} = 534\ in^4$ ok
$I_{LB} = 1010\ in^4 > I_{LB\ req} = 800\ in^4$ ok

$\Sigma Q_n = 147^k$ @ $Q_n = 17.2^k$
STUDS = 9 per side
TOT # STUDS = 18

Check 'a'

$a = \frac{147^k}{0.85(4{,}000)(81.5")} = 0.53" < 1"$ ok

$Y_2 = 4.5 - \frac{0.53}{2} = 4.24"$ ok

-USE W18×40 w/ 18 STUDS ✓ SPECIFIED W18×40 w/ 34 STUDS

SPOT CHECK – GIRDER

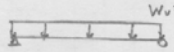
GIRDER 1

$l/600$ (REQ'D DEFL FOR MASONRY) = 0.46" – LIVE + MASONRY
$l/360$ – TOTAL = 0.77"

FACTORED LOAD ON BEAM = 1.38 klf
POINT LOAD ON GIRDER FROM BEAM B1 = 1.38 × 17' = **23.46ᴷ**

FIND: LOAD FROM CORRIDOR ON GIRDER

CORRIDOR BM



$W_u = 1.2D + 1.6L = 200$ psf
× 7.5'
1500 plf

⇒ LOAD ON GIRDER **8.25ᴷ**

TRIB WIDTH = 5.5'
LL = 80 psf
DL = 60 psf

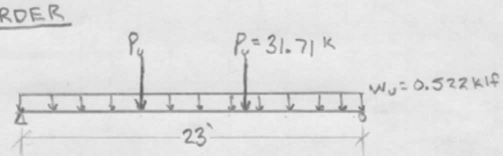
CORRIDOR WALL (6" CMU)

10'8" × 35 psf = 373 plf = **0.373 klf**

Factored (1.4D): $w_u = (0.373)(1.4) =$ **0.522 klf**

GIRDER

LL = 70 psf × 17' × 7.5' + 80 psf × 5.5' × 7.5'
$P_{LL} = 1.6(LL) = 19.56^k$



$M_{max} = Pa + \frac{w_u l^2}{8} = 31.71^k(8'1\frac{1}{4}") + \frac{0.522(23)^2}{8}$

$M_u = 292'k$

LIVE

$\Delta_{max} = \frac{Pa}{24EI}(3l^2 - 4a^2) + \frac{5wl^4}{384EI} = \frac{1902}{696{,}000\,I}(228528 - 37{,}830) + \left(\frac{5}{384}\right)\left(\frac{.522(23)^4(1728)}{29{,}000\,I}\right)$
$= 0.46"$

SOLVING FOR I

$I = 1380\ in^4 \sim I_{LB}$

CONST

DL = 38 psf (slab) + 5 psf Beam
= 43 psf
LL = 20 psf

TRIB AREA = (17' + 5.5') × (7.5') = 169 SF

$P_u = 1.2D + 1.6L$
= 84 psf
× 169 SF
14.1 K

$\Delta_{max} = \frac{Pa}{24EI}(3l^2 - 4a^2) = \frac{(14.1)(8'1\frac{1}{4}")}{24(29000)\,I}(3(23\times12)^2 - 4(8\times12+1.25)^2)$
$= 0.77"$

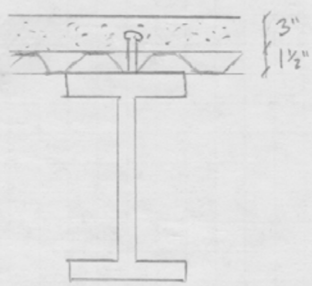
I = 488 – Bare BM

GIRDER (CONT)

$M_u = 292'k$
$I_{LB} = 1380 in^4$
$I_{BB} = 488 in^4$

$f_y = 50$ ksi
$f'c = 4$ ksi

$w_r / h_r = 6"/1.5" = 4 \geq 1.5$
1 WEAK STUD / RIB
$f'c = 4$ KSI
3/4" DIAMETER SHEAR STUD
3½ LENGTH
→ $Q_n = 21.5^k$



$b_{eff} = \frac{23'}{4} = 5.75' = 69"$ ← controls

ASSUME $a = 1"$

$(4½") - (½") = 4"$ $Y_2 = 4"$

TRY

	ΣQn	# STUDS	TOTAL WEIGHT
W18×40 (3)	430	40	1320
W21×44 (7)	162	16	1172

W21×44
$\Sigma Q_n = 162$
$\phi M_n = 504'k \geq 292'k$ ok
$I_{BB} = 843 in^4 \geq 488 in^4$ ok
$I_{LB} = 1380 in^4 \geq 1380 in^4$ ok

$\Sigma Q_n = 162^k$ @ $Q_n = 21.5^k$
STUDS = 8 / SIDE
TOT # STUDS = 16

CHECK 'a'

$a = \frac{162k}{0.85(4000)(69")} = 0.69" \leq 1"$ ok

$Y_2 = 4.5 - \frac{0.69}{2} = 4.16"$ ok

- USE W21×44 w/ 16 STUDS
- SPECIFIED W21×50 w/ 23 STUDS

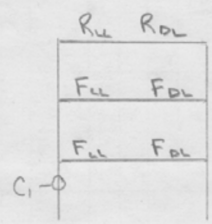
COLUMN C1

- FLOOR 2, INTERIOR COLUMN $K_{LL} = 4$

LL OFFICE + PART. = 70 PSF
LL CORRIDOR = 80 PSF
> 72.5 PSF

TRIB AREA:
OFFICE: 23' × 17' = 391 ft^2 ≈ 75%
CORRIDOR: 23' × 5.5' = 126.5 ft^2 ≈ 25%
TOTAL = 517.5 SF

INFLUANCE AREA, AI = 2070 SF



$A_t = 2(517.5) = 1035$
$K_{LL} = 4$

$L = L_o\left(0.25 + \frac{15}{\sqrt{K_{LL} A_t}}\right)$

$L = L_o(0.48) > 0.4\,L_o$ ok ✓

$R_{LL} = 20$ PSF $(517.5) = 10.35^k$
$R_{DL} = 25$ PSF $(517.5) = 12.94^k$

$F_{LL} = 72.5$ PSF $(517.5) = 37.52$ k $(0.48) = 18$ k
$F_{DL} = 60$ PSF $(517.5) = 31.05$ k

$P_u = 1.2D + 1.6L + 0.5L_R$

$P_u = 1.2(12.94^k + (2)\,31.05^k) + 1.6(18^k(2)) + 0.5(10.35^k)$

$P_u = 388^k$

EFFECTIVE LENGTH = 15'

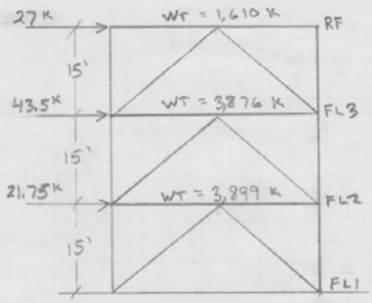
W10×49
$\phi P_n = 450^k > P_u = 388^k$ ok ✓

- USE W10×49
- SPECIFIED W10×54

SPOT CHECK - BRACED FRAME

BRACE 1

BASE SHEAR, V = 369 K
RIGID DIAPHRAM
4 EQUAL STIFFNESS BRACED FRAMES
1/4 LOAD TO EACH FRAME



VERTICAL DIST OF SEISMIC FORCES

$$F_x = C_{vx} \cdot V$$

$$C_{vx} = \frac{w_x h_x^k}{\sum_{i=1}^{n} w_i h_i^k} \; ; \quad K = 1 \text{ for } T \leq 0.5 \text{ sec.}$$

$$C_{RF} = \frac{(1{,}610^k)(45')}{(1{,}610)(45') + (3{,}876)(30') + (3{,}899)(15')} = 0.293$$

$$C_{FL3} = \frac{(3{,}876^k)(30')}{(1{,}610)(45) + (3{,}876)(30') + (3{,}899)(15')} = 0.470$$

$$C_{FL2} = \frac{(3{,}899^k)(15')}{(1{,}610)(45) + (3{,}876)(30') + (3{,}899)(15')} = 0.237$$

$F_{RF} = (0.293)(369^k)$
$F_{RF} = 108^k$

$F_{FL3} = (0.470)(369^k)$
$F_{FL3} = 174^k$

$F_{FL2} = (0.237)(369^k)$
$F_{FL2} = 87^k$

FORCES ON FRAME 1

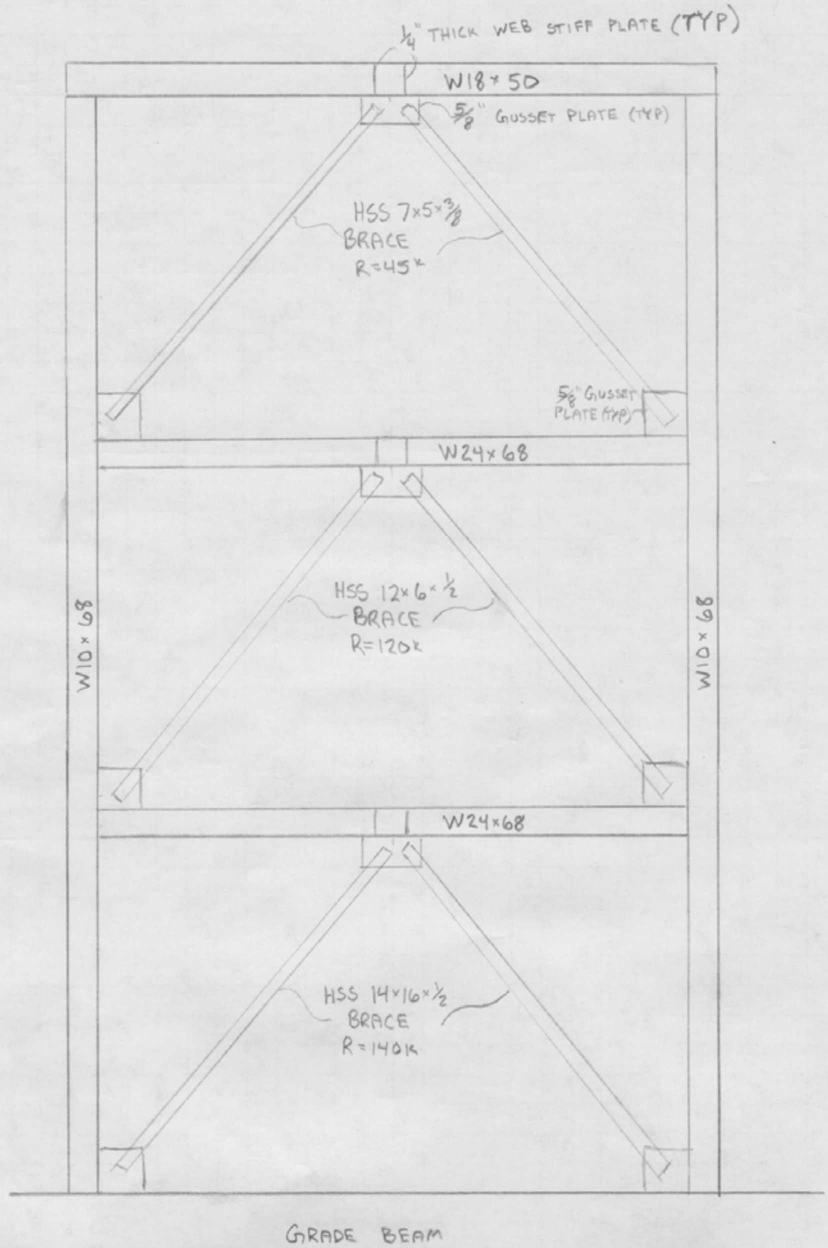
$F_{RF} = 108^k (1/4) = 27^k$

$F_{FL3} = 174^k (1/4) = 43.5^k$

$F_{FL2} = 87^k (1/4) = 21.75^k$

SPOT CHECK - BRACED FRAME

BRACE 1 DETAILS



SCALE: NOT TO SCALE

ADDITIONAL PICTURES