Eric Alwine – Structural Option
George Read Hall – The University of Delaware
Dr. Boothby
Technical Assignment #1
October 5, 2005

Executive Summary:

This report is a structural analysis of George Read Hall, located in Newark, Delaware. The report includes a description of the structural system and design codes as well as an analysis of the lateral forces and typical floor elements.

George Read Hall is a new dormitory constructed on the campus of The University of Delaware. At 129,000 square feet, it is the largest of the new buildings being constructed to replace the existing Pencader residential complex. It has a unique architectural shape, which could affect the way the lateral loads are distributed.

The exterior structural system of George Read Hall consists of cold formed metal stud bearing walls. At the second floor, the interior support is comprised primarily of beams and columns. The second through fifth floor interior support changes to metal stud bearing walls. The roof is supported by light gauge metal trusses.

A typical floor spot check was performed on the Hambro composite floor system and a typical beam under the second floor corridor. The results of these spot checks showed that the existing elements are slightly overdesigned. This could possibly be attributed to higher superimposed dead loads in the initial design. The joists size may have been increased to leave more room for mechanical equipment. A more in depth discussion of the discrepancies is provided in this report.

In addition to the floor spot check, simplified lateral analyses were performed to determine the forces induced by wind and seismic forces. These lateral analyses were done using ASCE 7-98. The wind forces seem to be lower than expected. The seismic forces determined in this report are higher than the forces determined in the original design. The original design represents wind as the controlling lateral design force; however, this report shows that seismic is the controlling lateral design force. The differences in the seismic design begin with the design factors from the ASCE manual. It is possible that a different version of ASCE was used for the original design. A more specific reason for these differences is unknown at this time. Further investigation will be performed at a later time.

Also included in this report are other structural issues that will need to be addressed with further investigation, including footing capacities, basement wall lateral pressures, and exterior wall deflection.

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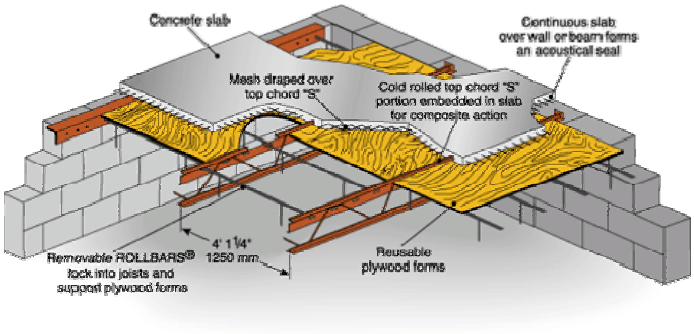
Introduction:

George Read Hall is a 129,000 square foot, five story residential building for students at The University of Delaware in Newark, Delaware. The building is one of three buildings being constructed in the new complex to replace the existing Pencader residential complex. The building's "U" shape makes it unique. The shape could affect the distribution of lateral forces. For the purposes of this report, the lateral loads will be distributed by the typical bay sizes. Typical floors are comprised of residential spaces on either side of a corridor running down the center of the building.

Overall Structural System:

The floor of George Read Hall is composed of a Hambro composite floor system. The floor system uses 14" deep steel joists with a $2^{3}/_{4}$" concrete slab. The joists are spaced at $4'1^{1}/_{4}$" on center. The figure on the right illustrates a typical Hambro floor system. The plywood is only used as formwork for the concrete slab and is removed when the concrete has reached its full compressive strength. The figure also shows masonry bearing walls; however, George Read Hall uses cold formed metal stud bearing walls.



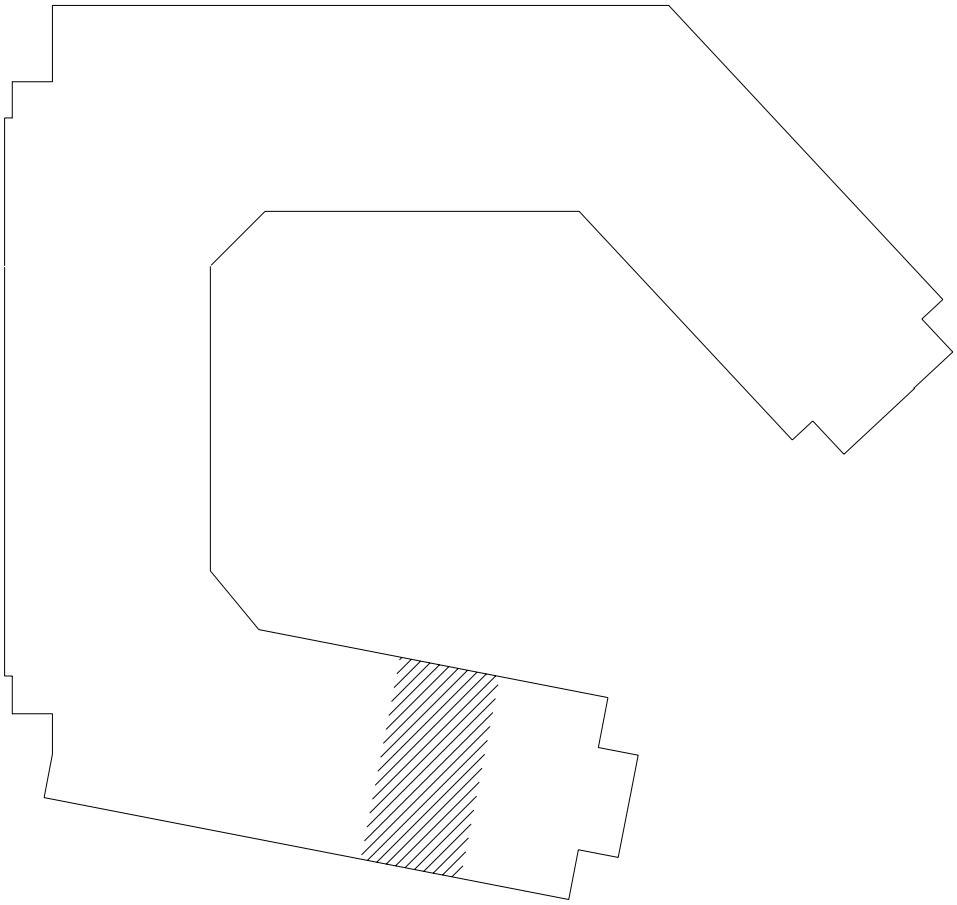
Bearing walls are 16 gauge, 50 ksi cold formed metal studs. The first floor is supported with 3-6" studs @ 16" on center. A typical bay is 26'-8" x 23'-6". Interior first floor framing consists of wide flange beams of various sizes. The second floor metal stud framing consists primarily of 3-6" studs @ 16" on center. Framing under the second floor hallway is wide flange beams, with the typical size being a W14x53. These interior hallway beams are located on each side of the 6'-0" wide hallway. A typical second floor bay showing these beams is shown on page 5. The third through fifth floor framing is very similar. The third floor bearing walls consist mainly of 2-6" studs @ 16" on center. The fourth and fifth floor bearing walls are built with 1-6"

stud @ 16" on center. The interior beams are replaced by metal stud bearing walls under the hallway in the third trhough fifth floor framing. Roof framing on George Read Hall consists of prefabricated light gauge metal trusses at a maximum of 4'-0" on center with $1^1/_2$" 22 gauge galvanized metal deck. The roof trusses span 54' with two intermediate supports located 23'-6" from each exterior wall.

The foundation is comprised of a combination of continuous and spread footings. The continuous footings range from 3'-0" wide to 7'-0" wide and are 1'0" deep and are reinforced with continuous #5 bars. Fifteen different sizes of spread footings are used ranging in size from 3'-0" wide x 3'-0" wide x 1'-0" deep to 10'-0" wide x 10'-0" wide x 2'-3" deep. These spread footings carry the concentrated loads from the interior columns. Reinforcing bars for the spread footings consist of #5 bars or #6 bars. The footings were designed with a soil bearing capacity of 4000 psf. Basement walls are 1'-4" thick with #4@12 both ways in both faces. The basement floor of George Read Hall is a 5" thick slab on grade with 6x6-W1.4 x W1.4 welded wire mesh. Slab control joints are located so that there is a maximum of 40 feet in length along any one side with a maximum uninterrupted concrete area of 1200 square feet.

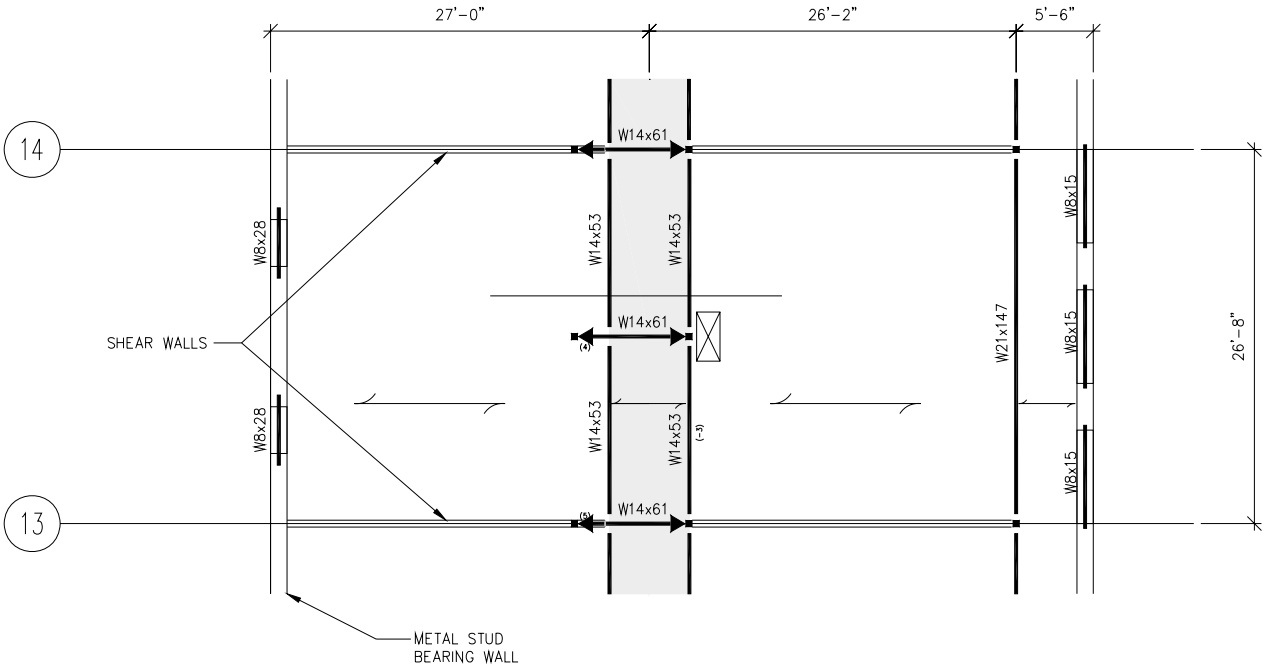
The lateral force resisting system of George Read Hall is x-braced shear walls. The shear walls are located along typical bay lines. first floor shear walls consist of x-bracing using 2-$4^1/_2$" metal straps. The second and third floor shear walls are x-braced walls of 2-4" metal straps. Fourth and fifth floor shear walls are 2-3" x-braced metal straps.

The building footprint and typical bay are shown below. The hatched area on the diagram represents the typical bay, which is shown in more detail on page 3.

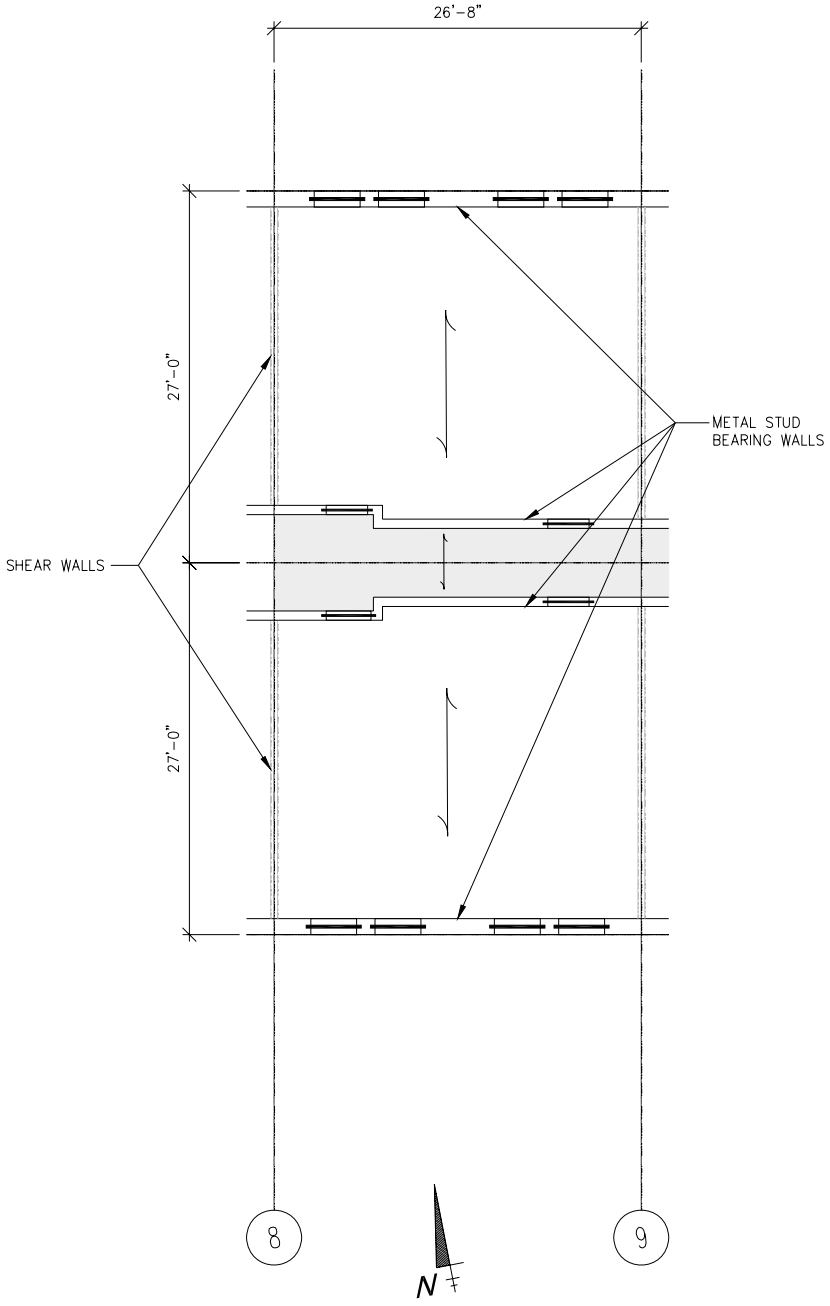


BUILDING FOOTPRINT

The shaded area represents the corridor area where the live load is increased to 100 psf. The span arrows show the direction of the steel joist framing in the Hambro floor system.



2ND FLOOR BAY SHOWING INTERIOR BEAMS



Typical 3rd through 5th Floor Bay

Design Codes and Code Requirements:

- IBC 2000 – International Building Code
- ACI 301 – Specification for Structural Concrete
- ACI 318 – Building Code Requirements for Structural Concrete
- CRSI Manual of Standard Practice
- AISC Manual of steel construction, Allowable Stress Design
- ASCE 7-98 – Minimum Design Loads for buildings
- AISI – American Iron and Steel Institute

Material Strengths:

Concrete:

- Normal weight, 4000 PSI
- Reinforcing bars – ASTM A 615/A 615M, Grade 60
- Welded wire mesh – ASTM A 185

Concrete Masonry Units:

- Normal weight, 1900 PSI
- Joint reinforcement- ASTM A-153, Grade B

Structural Steel:

- Steel shapes, plates, and bars – ASTM A992, Grade 50
- Cold-formed structural tubing – ASTM A 500, Grade B
- Steel pipe – ASTM A 53, Type E, Grade B
- Anchor Bolts – ASTM A307 Grade A
- High-Strength bolts – ASTM A325
- Steel joists – Fy = 50 KSI
- Cold Formed Metal Studs – Fy = 50 KSI

Gravity Design Loads:

Dead Loads:

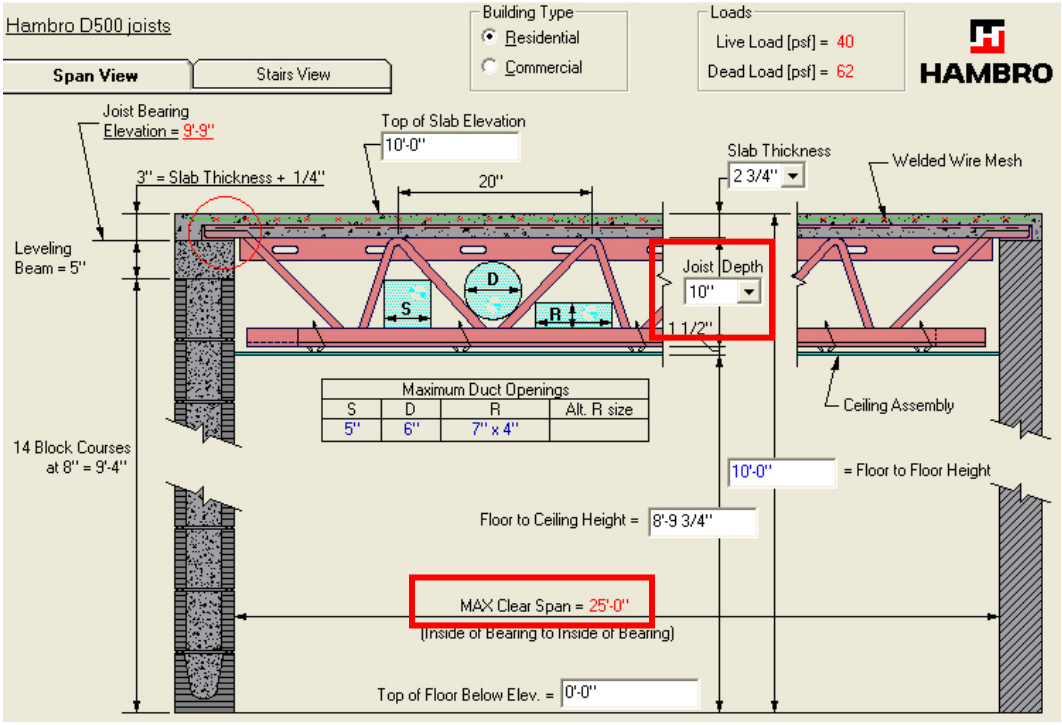
- Weight of slab = (2.75 in) x (150 pcf) x (1/12 in/ft) = 35 psf
- Weight of joists = 2 psf
- Miscellaneous dead load = 5 psf
- Superimposed dead load = 20 psf
- Total dead load = 35 + 2 + 5 + 20 = 62 psf

Live Loads:

- Roof – 20 PSF
- Ground snow load – 20 PSF
- Living spaces – 40 PSF
- Corridors, Lounges, Stairs – 100 PSF

Floor Spot Check:

Since all floors are primarily the same occupancy use, a typical floor spot check can be done at any floor. The typical floor system is a Hambro composite floor system using steel bar joists working compositely with a concrete slab. For the check, a design aid provided by Hambro was used. The results can be seen below. The worst case span of the floor system is 24'-0" in the living spaces.



The results of the design aid show that a floor system using 10" joists and a $2^3/_4$" slab can span 25'-0" under the appropriate design loads. The existing floor system utilizes 14" joists with a $2^3/_4$" slab. These differences could be due to several different things. One possible reason for the difference could be that a higher superimposed dead load or miscellaneous dead load was used in the initial design. Another possible reason might be the need for larger openings in the joists for mechanical pipes or equipment.

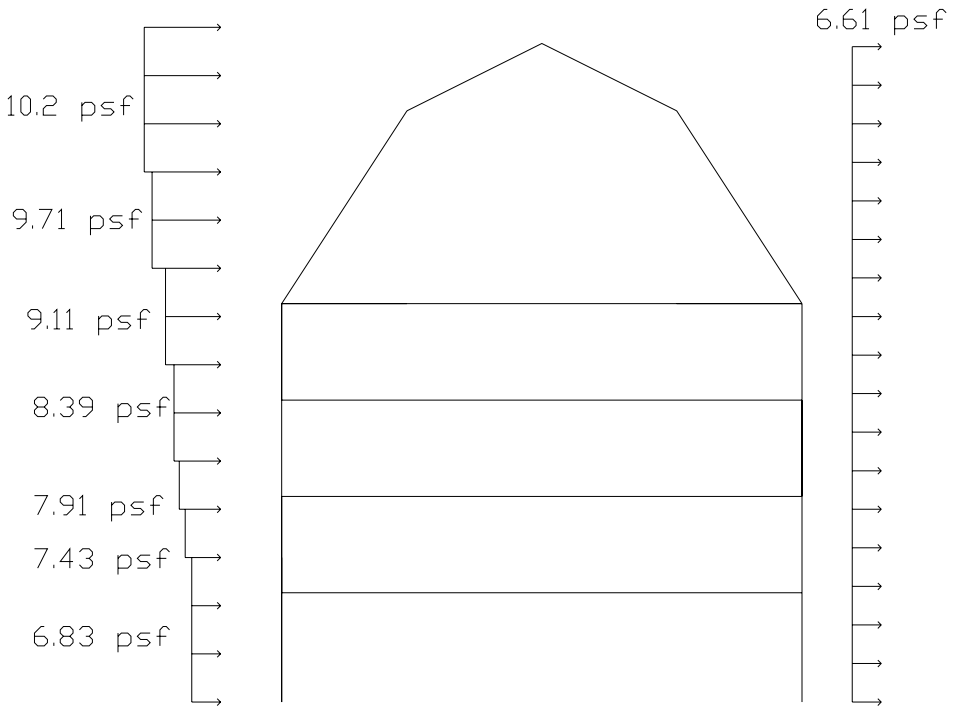
In addition to the floor system check, a spot check was also performed on a typical second floor beam at the corridor, using the allowable stress design method. Contributing loads came from the roof and four floors above. The second floor framing uses interior beams and columns because of different occupancy use on the first floor. The load from above is transferred down through metal stud bearing walls on the third through fifth floors. The total load was determined to be 7.142 k/ft over a length of 13'-3". It was determined that the existing W14x53 is ok; however, a W14x43 could be used instead. Once again, this difference could be attributed to discrepancies in superimposed dead loads. The deflectioni criteria could also have an effect on the result. the calculations in Appendix B were done using a deflection criterion of L/360. It is possible that a more stringent criterion was used. Detailed calculations can be found in Appendix B.

Wind Design Loads:

Wind loads were calculated using IBC 2000 and ASCE 7-98 and the following design factors:

$V = 90$ mph
$I = 1.0$
$K_{ZT} = 1.0$
$K_D = 0.85$

The building is category II with an exposure category B. The wind loading diagram is shown below, along with a table summarizing the story forces, story shears and overturning moment. After seeing the results in the table, the story forces seem to be low. further investigation will be performed at a later time to determine more accurate results. Detailed calculations can be found in appendix C.



Story	Story Force (kips)	Story Shear (kips)	Moment (ft-kips)
5	2.10	-	86.1
4	4.02	2.10	124.62
3	3.83	6.12	80.43
2	3.78	9.95	41.58
Base	-	13.73	332.73

Seismic Design Loads:

Seismic loads were also calculated using IBC 2000 and ASCE 7-98. It was determined from ASCE 7-98 that the simplified analysis procedure cannot be used because it is a light gauge building over three stories. Therefore, the equivalent lateral force method was used. The building is seismically classified as site D, design category A, and use group I. the following factors were used in the design:

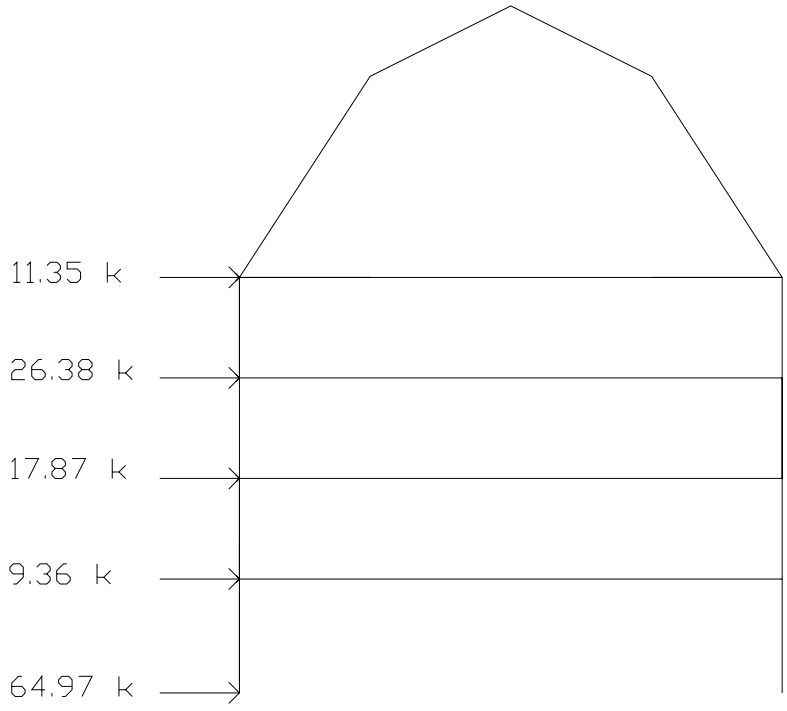
$S_S = 0.225$
$S_1 = 0.07$
$R = 3$
$I = 1.0$
$F_A = 1.6$
$F_V = 2.4$

$S_{MS} = 1.6(0.225) = 0.36$
$S_{M1} = 2.4(0.07) = 0.168$

$S_{DS} = {}^{2}/_{3}(0.36) = 0.24$
$S_{D1} = {}^{2}/_{3}(0.168) = 0.112$

The table below shows the force at each level in the right most column, with the total base shear being the sum of these story forces.

Level	w_x	h_x	$w_x h_x^{1.0}$	C_{vx}	F_x
5	50.4	41	2066.4	0.174747	11.35331
4	154.9	31	4801.9	0.406077	26.38282
3	154.9	21	3252.9	0.275084	17.87223
2	154.9	11	1703.9	0.144092	9.361645
Base	154.9	-	-	-	64.97
			11825.1	1	



These numbers are larger than the numbers determined by the structural engineer on the job. The difference began with S_s and S_1 determined to be higher than that specified on the drawings. In addition, F_A and F_V were also determined to be higher values. A specific reason for this difference is unknown at this time. More detailed analysis and calculations will be done to determine the reason for the difference. Detailed seismic calculations can be found in Appendix D.

Shear Wall Check:

After performing the wind and seismic analyses, it can easily be seen that the lateral design control is the seismic forces. The maximum shear force is 26.38 kips at the fourth floor. The shear resisting element at this level is an X-braced shear wall with 2-4" metal straps. A simplified method was used to determine the force in each strap. The results of this check showed that the existing straps are ok. Since this

is a very simplified method of analysis, it is possible that the shear wall is subjected to higher loads than modeled. More in depth calculations will be performed to determine a more precise analysis. The simplified calculation is shown in Appendix E.

Other Structural Issues:

Several other structural issues must be addressed in more detail in other reports. First is to determine whether or not the continuous and spread footings are sufficiently designed with respect to the soil bearing capacity. A second issue is if the basement walls will be affected as a result of being subjected to lateral soil loads. A final concern is the impact of wind force on the exterior wall; it must be designed to the proper deflection criteria so that the bricks are not damaged in any movement caused by wind.

APPENDIX

APPENDIX A:

CALCULATION OF ROOF SNOW LOAD:

$S = 0.7C_eC_tC_sP_gI_s$

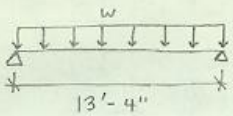
$C_e = 1.0$
$C_t = 1.0$
$C_s = 1.0$ ➔ THE ROOF HAS TWO DIFFERENCE SLOPES, BUT 1.0 IS CONSERVATIVE
$P_g = 20$ PSF
$I_s = 1.0$

$S = 0.7(1.0)(1.0)(1.0)(20)(1.0) = 14$ PSF

APPENDIX B:

Spot Check Typical Beam at Second Floor

- W14 x 53



Floor

LL = 40(0.5)(23.5) + 100(0.5)(5.67) = 753.5 lb/ft

DL = 62(0.5)(23.5) + 62(0.5)(5.67) = 904.3 lb/ft

Roof

LL = 20(0.5)(23.5) + 20(0.5)(5.67) = 291.7 lb/ft

DL = 15(0.5)(23.5) + 15(0.5)(5.67) = 218.8 lb/ft

$W_{TOTAL} = (291.7 + 218.8) + 4(753.5 + 904.3) = 7.142$ k/ft

$M_{max} = \frac{wl^2}{8} = \frac{7.142(13.33)^2}{8} = 158.6$ k·ft

$S_{req} = \frac{M}{F_b} = \frac{158.6(12)}{0.66(50)} = 57.6$ in^3

- $\frac{L}{360}$ Deflection Criteria

$\frac{13.33(12)}{360} = \frac{5(7.142)(13.33)^4 1728}{384(29000) I_{req.}}$

$I_{req} = 393.75$ in^4

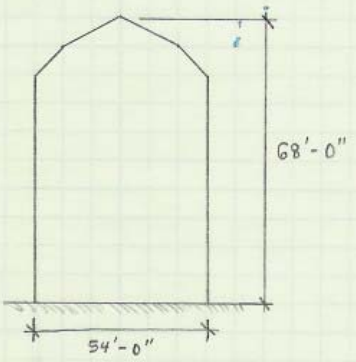
$S_{act} = 77.8$ in^3

$I_{act} = 541$ in^4

∴ W14 x 53 IS OK, COULD USE W14 x 43

APPENDIX C:

Wind Load Analysis:



V = 90 mph
I = 1.0
K_{zt} = 1.0
K_d = 0.85

$q_z = 0.00256 K_{zt} K_d V^2 I K_z$

$q_z = 0.00256(1)(0.85)(90)^2(1) K_z = 17.63 K_z$

$q_h = \frac{68-60}{70-60}(0.89-0.85)(17.63) + 17.63(0.85) = 15.55$ psf

External Pressure Coefficients

Windward → $C_p = 0.8$
Leeward → $L/B < 1$ → $C_p = -0.5$

Gust Factor Effect

$f = \frac{1}{C_t h^x} = \frac{1}{0.02(68)^{0.75}} = 2.11$ Hz. ⇒ Rigid

G = 0.85

Wind Pressures

Windward = $q_z C_p G = 17.63 K_z (0.8)(0.85) = 11.99 K_z$

Leeward = $q_h C_p G = 15.55(-0.5)(0.85) = -6.61$ psf

Z	K_z	Pressure
0-15	0.57	6.83
20	0.62	7.43
25	0.66	7.91
30	0.70	8.39
40	0.76	9.11
50	0.81	9.71
60	0.85	10.2

Story	Forces	Story Shear	Moment
1	-	13.73 k	86.1 k.ft
2	3.78 k	9.95 k	124.62 k.ft
3	3.83 k	6.12 k	80.43 k.ft
4	4.02 k	2.10 k	41.58 k.ft
5	2.10 k	-	332.73 k.ft

APPENDIX D:

Seismic Analysis

- Too tall for simplified method
- Use Equivalent Lateral Force Method

$S_s = 0.225$
$R = 3$ → Not specifically detailed for seismic
$I = 1.0$
$S_1 = 0.07$
$F_a = 1.6$
$F_v = 2.4$ } Site Class D

$S_{MS} = F_a S_s = 1.6(0.225) = 0.36$
$S_{M1} = F_v S_1 = 2.4(0.07) = 0.168$

$S_{DS} = \frac{2}{3} S_{MS} = \frac{2}{3}(0.36) = 0.24$
$S_{D1} = \frac{2}{3} S_{M1} = \frac{2}{3}(0.168) = 0.112$

Base Shear

$V = C_s W$

$W_{roof} = 26.67(54)(15) + 26.67(54)(20) = 50.4^k$
$W_{floor} = 26.67(54)(62) + 26.67(54)(40) + 15(10)(2)(26.67) = 154.9^k$
$W = 50.4 + 5(154.9) = 824.9^k$

$T = C_t h_n^x = 0.02(68)^{0.75} = 0.474\ s$

$C_s = \frac{S_{DS}}{R/I} = \frac{0.24}{(3/1)} = 0.08$

$C_{s,max} = \frac{S_{D1}}{T(R/I)} = \frac{0.112}{0.474(3/1)} = 0.079 \Rightarrow$ controls

$V = 0.079(824.9) = 64.97^k$

Vertical Distribution of Forces

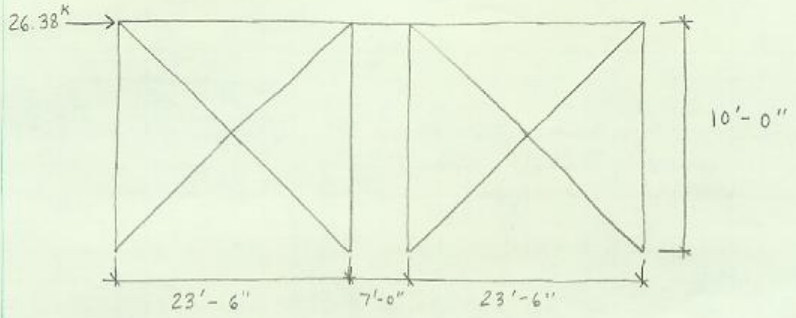
$F_x = C_{vx} V;\ C_{vx} = \frac{w_x h_x^k}{\Sigma w_i h_i^k}$

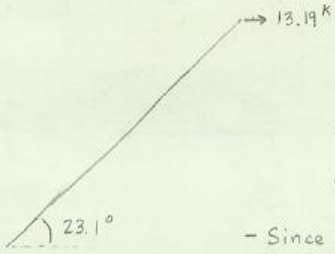
$k = 1$

— See Table for forces

APPENDIX E:

Shear Wall Check





$T = \frac{13.19}{\cos 23.1} = 14.34^K$

Area of Strap = $4(0.0598) = 0.2392\ in^2$

$F_y = 50\ ksi$

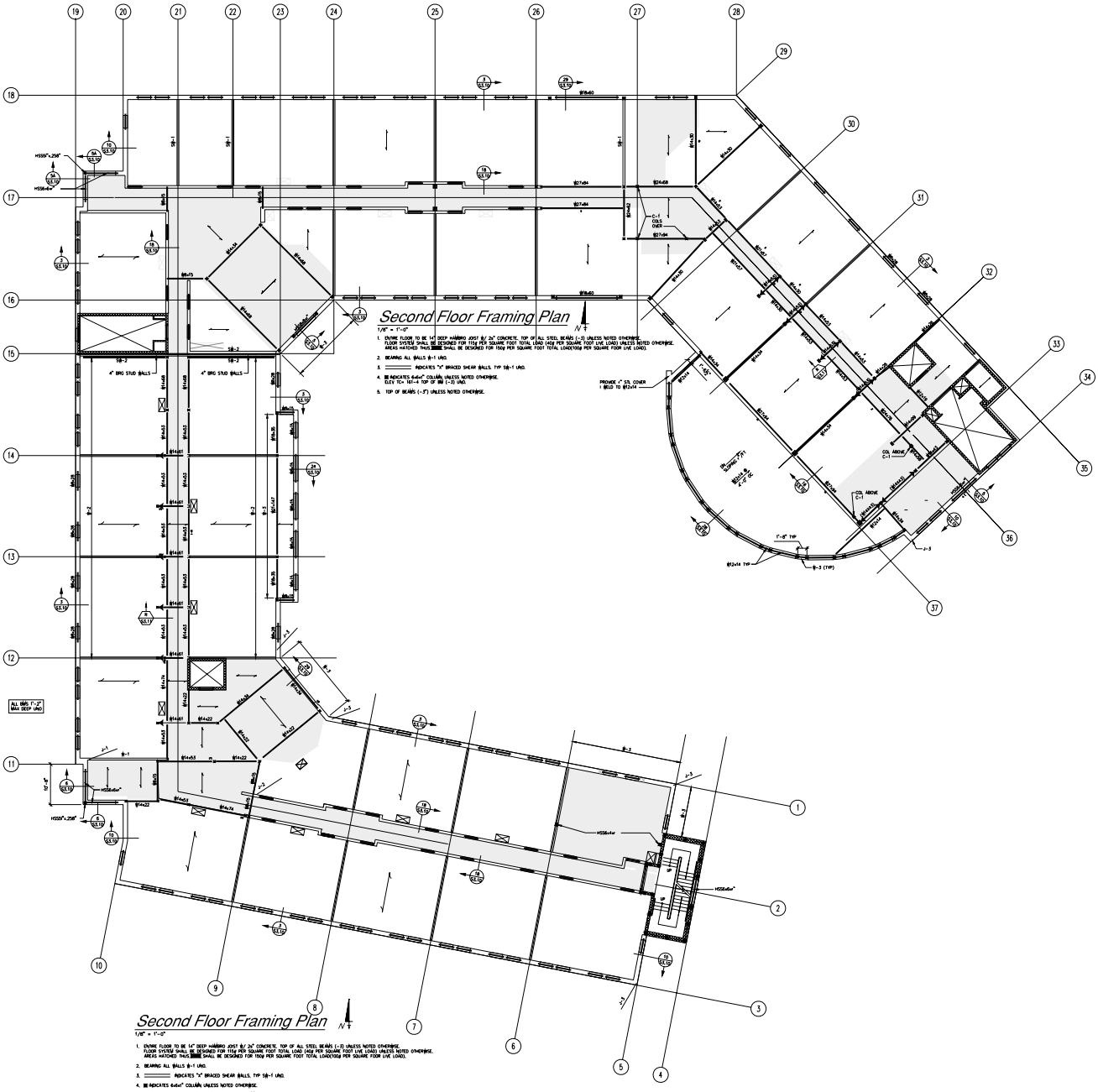
- Since there are two straps, $A = 2(0.2392) = 0.4784\ in^2$

$T_{allow} = 50(0.4784) = 23.92^K$

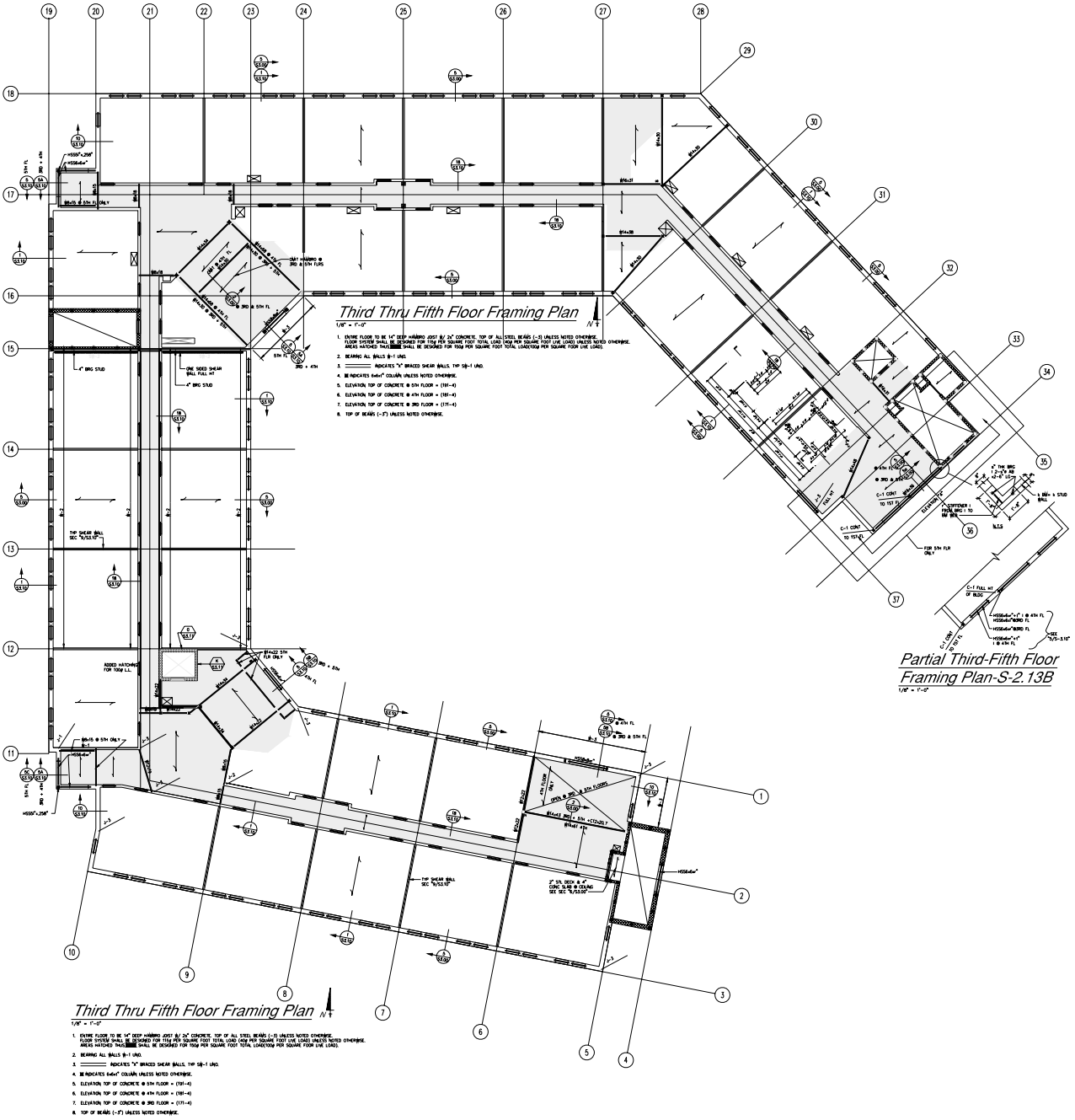
∴ 2 - 4" STRAPS ARE OK

APPENDIX F:

SECOND FLOOR FRAMING PLAN



THIRD THROUGH FIFTH FLOOR PLAN



Building Section