ERIC ALWINE – STRUCTURAL OPTION
GEORGE READ HALL – THE UNIVERSITY OF DELAWARE
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TECHNICAL ASSIGNMENT #3
NOVEMBER 21, 2005

EXECUTIVE SUMMARY:

THIS REPORT IS AN ANALYSIS OF THE EXISTING LATERAL FORCE RESISTING SYSTEM. INCLUDED IN THE REPORT ARE SEISMIC AND WIND ANALYSES TO DETERMINE THE CRITICAL LOAD CONDITION, FOLLOWED BY SUBSEQUENT CHECKS OF THE EXISTING SYSTEM FOR STRENGTH, DRIFT, AND OVERTURNING.

GEORGE READ HALL IS A FIVE STORY RESIDENTIAL DORMITORY LOCATED ON THE CAMPUS OF THE UNIVERSITY OF DELAWARE. IT RISES 68 FEET HIGH AND ENCOMPASSES APPROXIMATELY 129,000 SQUARE FEET. THE BEARING WALLS CONSIST OF METAL STUD FRAMING. THE BUILDING UTILIZES A HAMBRO COMPOSITE FLOOR SYSTEM WITH 14" OPEN WEB STEEL JOISTS AND A $2^3/_4$" CONCRETE SLAB. THE "U" SHAPE OF THE BUILDING MAKES IT UNIQUE WHEN DESIGNING THE LATERAL FORCE RESISTING SYSTEM. LATERAL LOADS CAN NOT BE APPLIED AND DISTRIBUTED IN THE SAME MANNER AS IN A RECTANGULAR BUILDING. ADDITIONALLY, THE CENTER OF RIGIDITY AND THE CENTER OF MASS DO NOT COINCIDE, RESULTING IN TORSIONAL SHEAR.

THE LATERAL FORCE RESISTING SYSTEM OF GEORGE READ HALL CONSISTS OF X-BRACED SHEAR WALLS. AT THE FIFTH AND FOURTH FLOOR, THE SHEAR WALLS ARE CONSTRUCTED WITH 2-3" STRAPS. 2-4" STRAPS ARE USED ON THE THIRD AND SECOND FLOOR, AND $2\text{-}4^1/_2$" STRAPS ARE USED ON THE FIRST FLOOR. ALL STRAPS ARE 16 GAUGE, 50 KSI.

AFTER APPLYING AND DISTRIBUTING THE LATERAL LOADS, IT WAS DETERMINED THAT THE SEISMIC FORCES CONTROL THE DESIGN. THE WORST CASE SHEAR WALL LOADING WAS MODELED USING RAM ADVANSE TO DETERMINE THE STRENGTH CAPACITY AND DRIFTS. AFTER ANALYZING THE STRUCTURE IN RAM ADVANSE, IT COULD EASILY BE SEEN THAT THE STRAPS ARE UNDERDESIGNED TO CARRY THE INTENDED LOADING. ALSO, BECAUSE OF THIS, THE DRIFT OF THE BUILDING EXCEEDS THE ALLOWABLE LIMITS FOR EACH FLOOR.

THE REASON FOR THE MEMBERS BEING UNDERDESIGNED CAN BE ATTRIBUTED TO MUCH HIGHER CALCULATED SEISMIC FORCES IN THIS REPORT THAN IN THE ORIGINAL DESIGN. ADDITIONALLY, THE SEISMIC LOADS WERE DISTRIBUTED TO THE SHEAR WALLS DIFFERENTLY THAN IN THE ORIGINAL DESIGN. IN THE ORIGINAL DESIGN, THE SEISMIC FORCES WERE MOST LIKELY DISTRIBUTED BY TRIBUTARY WIDTH TO THE WALLS; HOWEVER, THIS IS NOT THE APPROPRIATE METHOD FOR NON-FLEXIBLE BUILDINGS.

<u>Introduction</u>:

George Read Hall is a five story, 68 foot high residential dormitory on the campus of the University of Delaware. The 129,000 square foot building is the largest of three new dormitories being built on the campus to replace the existing complex.

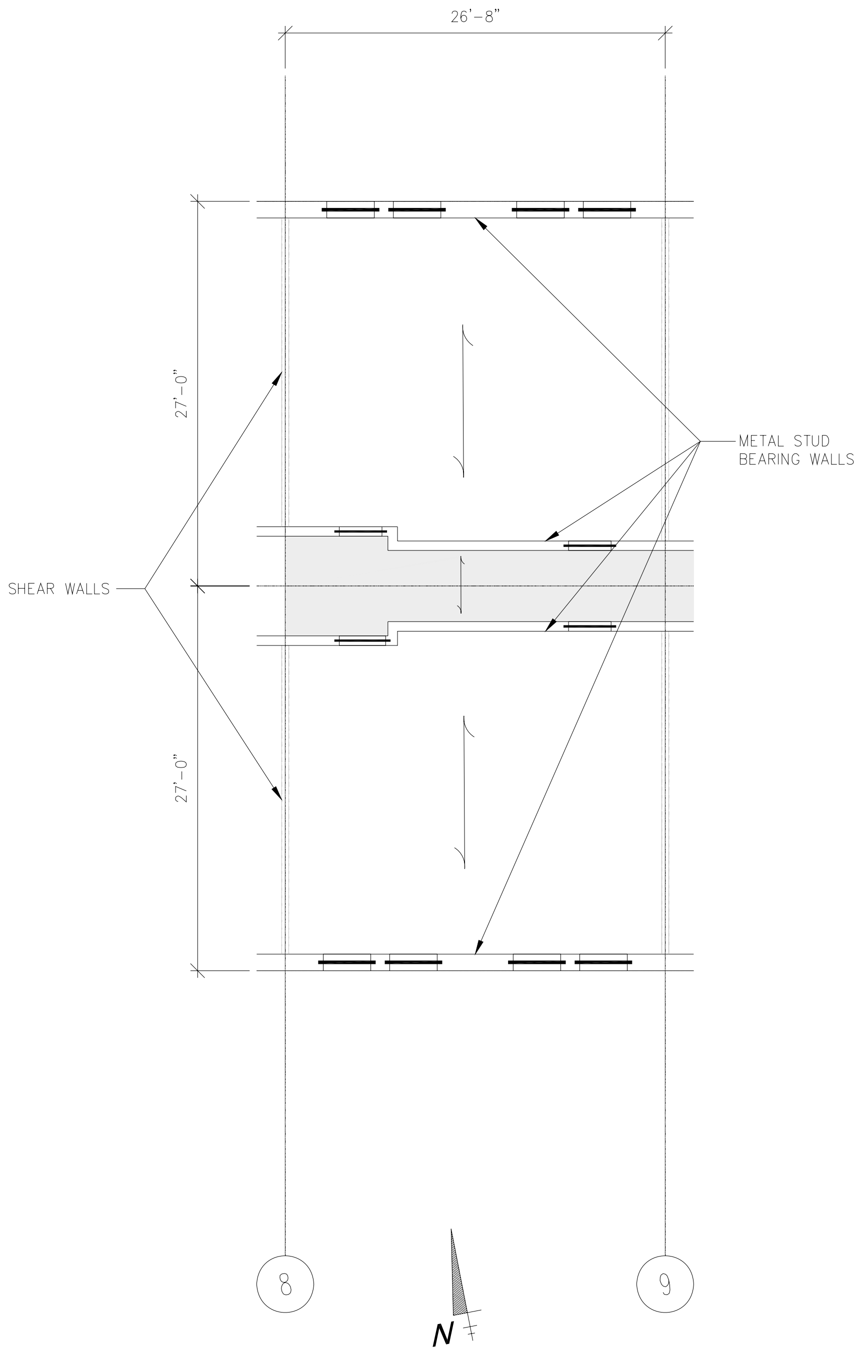
The superstructure of George Read Hall consists of metal stud bearing walls. The floor is comprised of a Hambro composite floor sytem of open web steel joists acting compositely with a $2^{3}/_{4}$” concrete slab. The roof framing is light gauge, prefabricated metal trusses. The building is supported on exterior continuous footings and interior spread footings.

The shape of the building makes it very unique. Its “U” shape makes the design of the structure different than more regularly shaped buildings. This is especially important in the design of the lateral resisting system. The center of mass and center of rigidity do not coincide as they might in a symmetrical building.

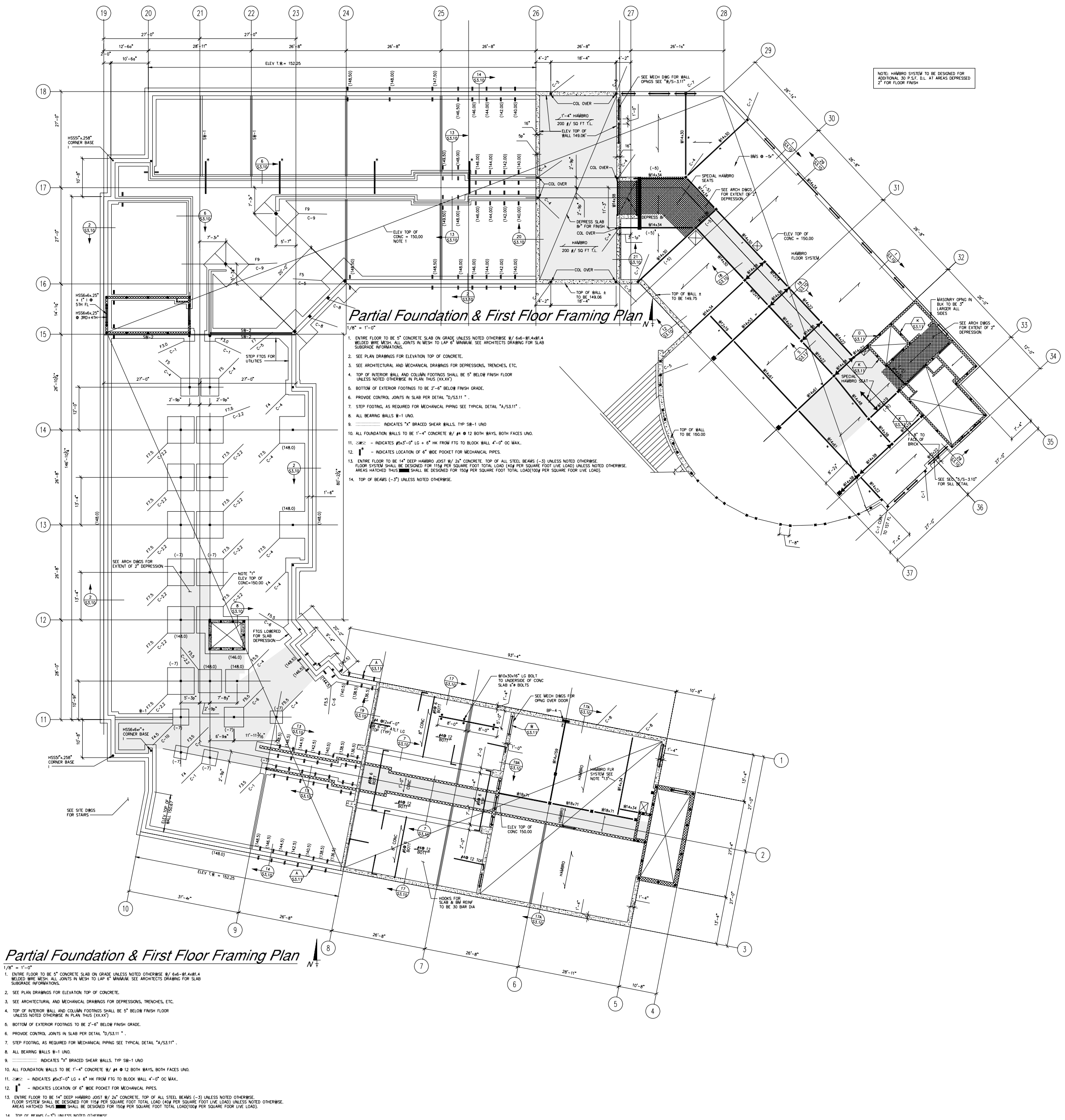
The typical bay is shown below indicating the location of the shear walls within the typical bay. In addition, the floor plans are also shown below.

included in this report is a description of the existing system, determination of the design control, an explanation of the distribution of forces, spot checks, and a conclusion of the results.

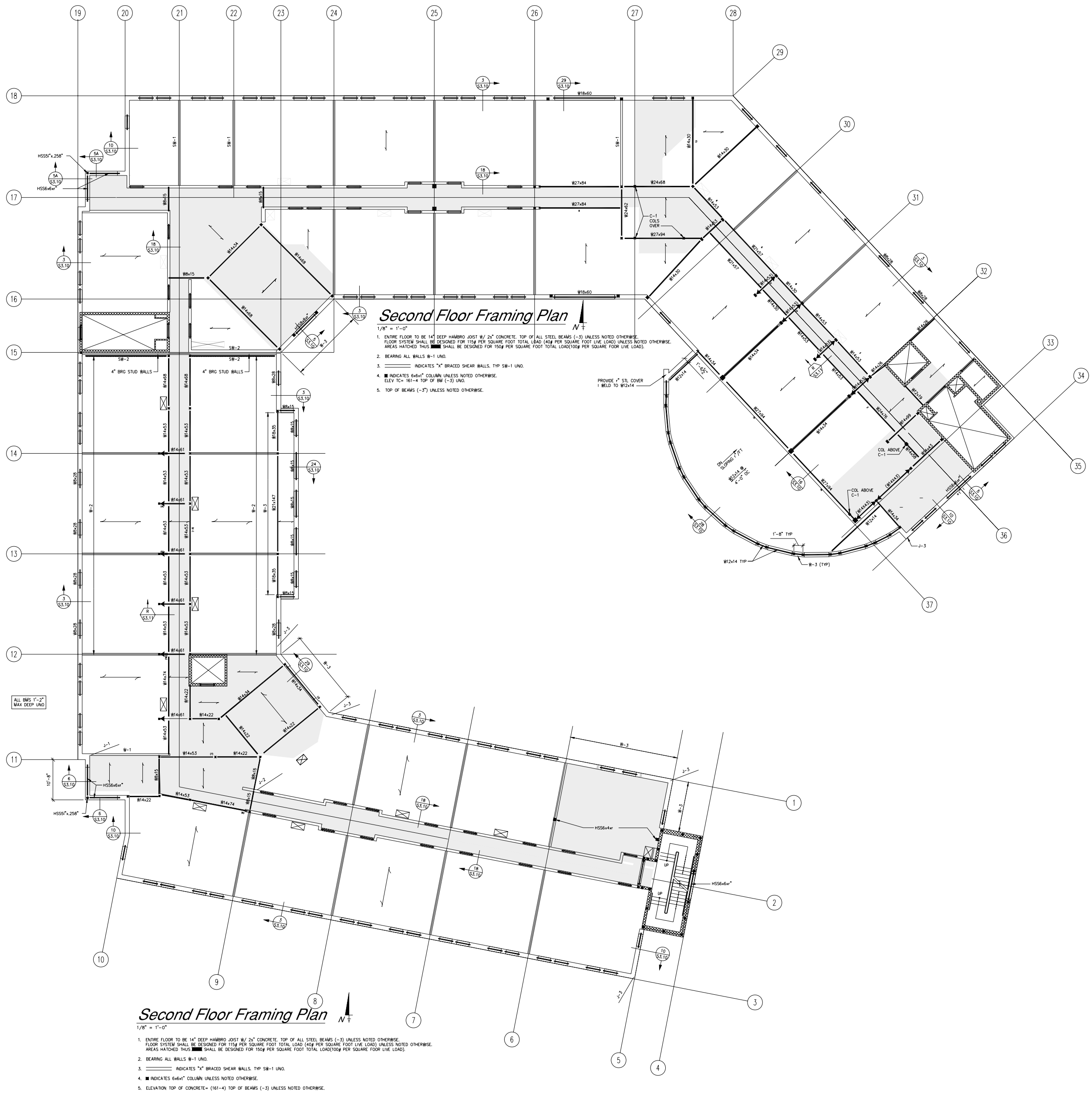
Typical Bay:



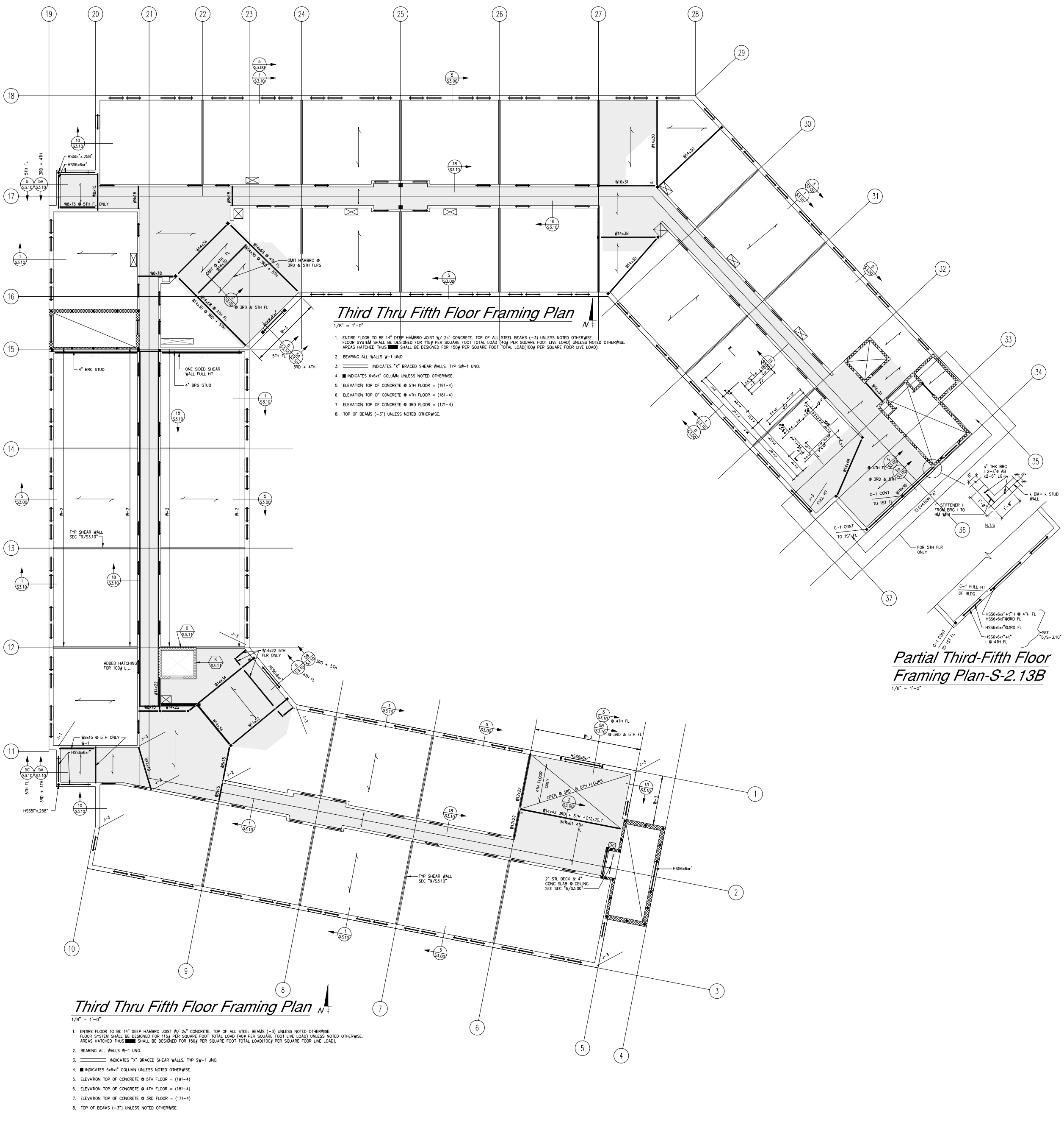
First Floor Plan:



Second Floor Plan:

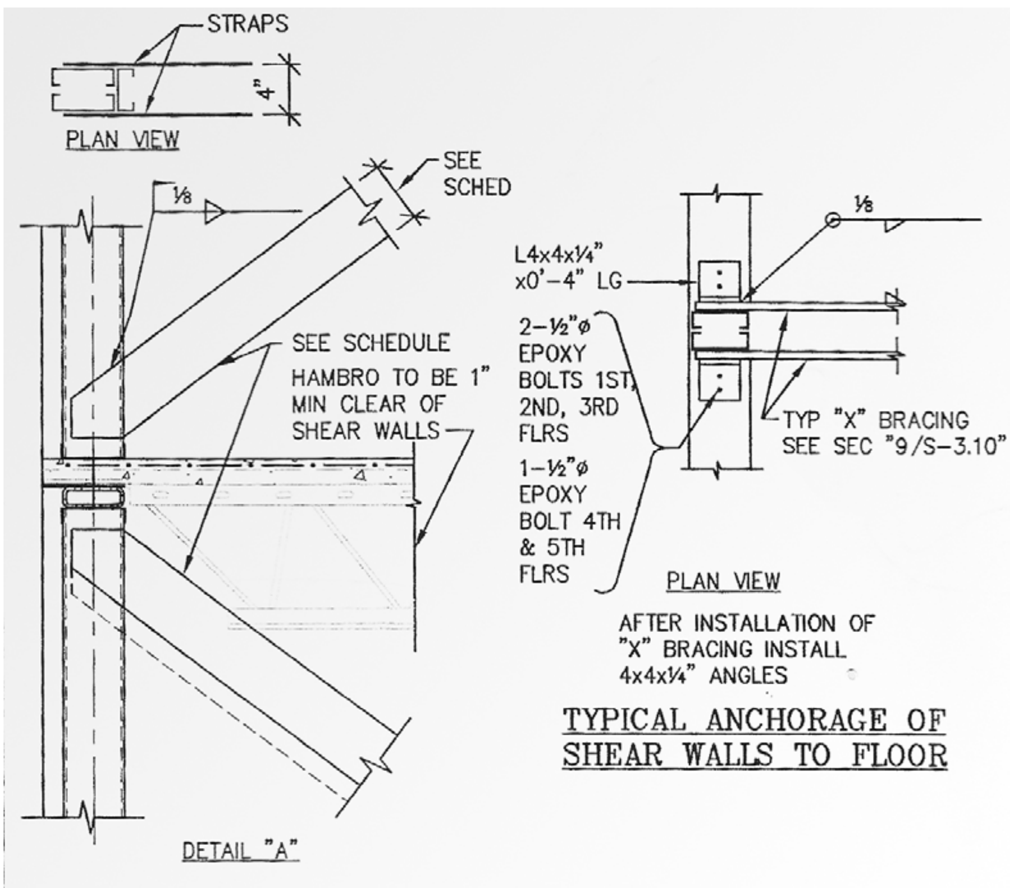


THIRD THROUGH FIFTH FLOOR PLAN:



Existing System:

The existing lateral resisting system consists of X-braced shear walls. The walls are cold formed metal studs with 16 gauge, 50 ksi metal straps. Shear walls are located on each side of the double loaded corridor. The typical distance between walls is 26'-8". At the fifth and fourth floors, the shear walls are constructed with 2-3" straps. 2-4" straps are used on the third and second floor, and 2-4 1/2" straps are used on the first floor. The reason for this is because the shear forces are higher at the base of the building. The typical shear wall details are shown below.



As seen on the details, the vertical edge members of the shear walls are metal studs. The straps are welded to the vertical studs with a 1/8" thick fillet weld. This shear wall system acts virtually as a vertical cantilevered truss.

Lateral Design Control:

The lateral resisting system for this building is controlled by the seismic forces. This can be seen by comparing the forces on a typical bay. The wind forces created a base shear of 27 kips, while the seismic forces created a base shear of 29 kips. This seismic force in turn also creates a greater torsional moment due to the irregular shape of the building. This is a simplified method used for comparison.

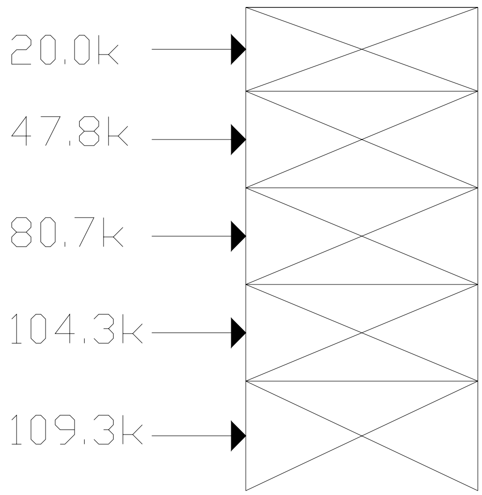
Distribution of Lateral Forces:

A more accurate analysis of the seismic forces takes into account the entire weight of the structure. It was assumed in this analysis that the building acts as a rigid diaphragm. A more in depth analysis to determine the rigidity of the structure may be conducted for the end of semester proposal. The total base shear was calculated using the total building weight, calculated by floor area. The resulting base shear was calculated to be 836.8 kips. The story forces were then calculated by distributing the loads according to the height and weight of each floor. The direct shear forces were determined, according to rigidity, by applying these story forces to the building in both directions.

Level	w_x	h_x	$w_x h_x^{1.0}$	C_{vx}	F_x	Shear
Roof	959.6	50	47980	0.162875	136.2937	
5	2796.7	41.333	115596	0.392407	328.3661	136.2937
4	2796.7	31.333	87629	0.297469	248.922	328.3661
3	2796.7	21.333	59662	0.202531	169.478	577.2881
2	2796.7	11.333	31695	0.107593	90.03393	746.7661
Base	2796.7	-	-	-	836.8	836.8
			294582	1		

Each wall represents a portion of the rigidity in a certain direction. This proportion is multiplied by the story force to determine the direct shear in each member. The center of rigidity was calculated using the stiffnesses of each shear wall. The center of mass was also determined. When compared, an eccentricity of 15’ results. This eccentricity was used to determine the torsional moment. The torsional shear forces were determined and added to the direct shear forces to find the total shear force in the shear walls. The torsional shear was distributed according to the rigidity of each wall and the perpendicular

distance from the center of rigidity. The worst case loading on the shear walls is shown in the diagram below. The total shear loads on all walls at all floors can be seen in the appendix.



The wind forces were distributed by tributary width onto the shear wall. The direct and torsional shear forces on each member were not calculated because the seismic forces are the design control.

The loads are transferred from the straps into the metal stud walls and the floor slab. The slab has a much higher relative stiffness than the straps, which also allows the forces to be distributed to the entire width of the building. A concern in this transfer of forces is the strength of the welded connection of the strap to the studs. Even with an all around weld, the weld can not develop the required force to appropriately transfer the load.

Strength Check:

After determining the critical loads, the typical X-braced shear wall was modeled using RAM Advanse. Upon applying the critical loads, it was determined that the existing members are significantly overstressed. The reason for this is because the seismic forces determined in this analysis are much higher than those determined in the existing design. In addition, it was determined in the original design that wind was the lateral design control. The difference in the seismic forces starts with higher determined spectral accelerations. Also, the analysis procedure in

this report may be more in depth than the original design. However, the loads in this report seem to be high. Further investigation into the distribution of the seismic forces will be performed and included in the end of semester proposal.

Drift Check:

In addition to a strength check, the drift was also determined and compared to allowable values. The total building drift was determined to be 14.1”. The largest interstory drift was on the first floor with a value of 4.49”. This far exceeds the allowable drift of 0.23”. The allowable drift is calculated from ASCE 7 table 9.5.2.8.

$$\Delta_{allow} = 0.02h_x$$

all of the story drifts exceed the allowable limits put forth by the above equation. This is directly correlated to the loads applied and the load resisting elements. Therefore, since the loads seem to be high and the members are underdesigned, it makes sense that the drift is exceeded.

Overturning:

The lateral forces on the structure can cause problems with the footings trying to prevent overturning. The overturning moment is calculated below.

OM = (109.3)(5.67) + (104.3)(16.33) + (80.7)(26.33) + (47.8)(36.33) + 20(45.67) = 7098 k-ft

Weight of structure = 5(26.67)(54)(62) + 26.67(54)(15) = 468.1 k

Resisting Moment = 468.1(27) = 12,638 k-ft. > 7098

Therefore, the dead weight of the building can resist the overturning moment caused by the seismic forces. Additionally, the footings are not required to resist the overturning moment in addition to the gravity loads.

Conclusions:

In conclusion, the lateral resisting system of George Read Hall is not sufficiently designed to resist the applied calculated loads. The x-braced straps can not resist the seismic loads and allow the building to drift too much. Additionally, the welded connections at the ends of the straps can not appropriately transfer the load into the studs.

The dead weight of the structure is sufficient to resist the overturning moments caused by the seismic forces. Therefore, the footings are not required to help resist the overturning moment, and are controlled in design by the compressive forces of the weight above.

Appendix

Appendix A:

Seismic Analysis

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

$S_s = 0.225$
$R = 3 \rightarrow$ Not specifically detailed for seismic
$I = 1.0$
$S_1 = 0.07$
$\left.\begin{array}{l} F_a = 1.2 \\ F_v = 1.7 \end{array}\right\}$ Site Class C

$S_{MS} = F_a S_s = 1.2(0.225) = 0.27$
$S_{M1} = F_v S_1 = 1.7(0.07) = 0.119$

$S_{DS} = \frac{2}{3} S_{MS} = \frac{2}{3}(0.27) = 0.18$
$S_{D1} = \frac{2}{3} S_{M1} = \frac{2}{3}(0.119) = 0.079$

Base Shear

$V = C_s W$

$A_{floor} = 27{,}418.5\ ft^2$
$A_{roof} = 27{,}418.5\ ft^2$
$W = 5(27{,}418.5(102)) + 27{,}418.5(35) = 14{,}943.1^k$

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

$C_s = \frac{S_{DS}}{R/I} = \frac{0.18}{(3/1)} = 0.06$

$C_{s,max} = \frac{S_{D1}}{T(R/I)} = \frac{0.079}{0.474(3/1)} = 0.056 \Rightarrow$ controls

$V = 0.056(14{,}943.1) = 836.8^k$

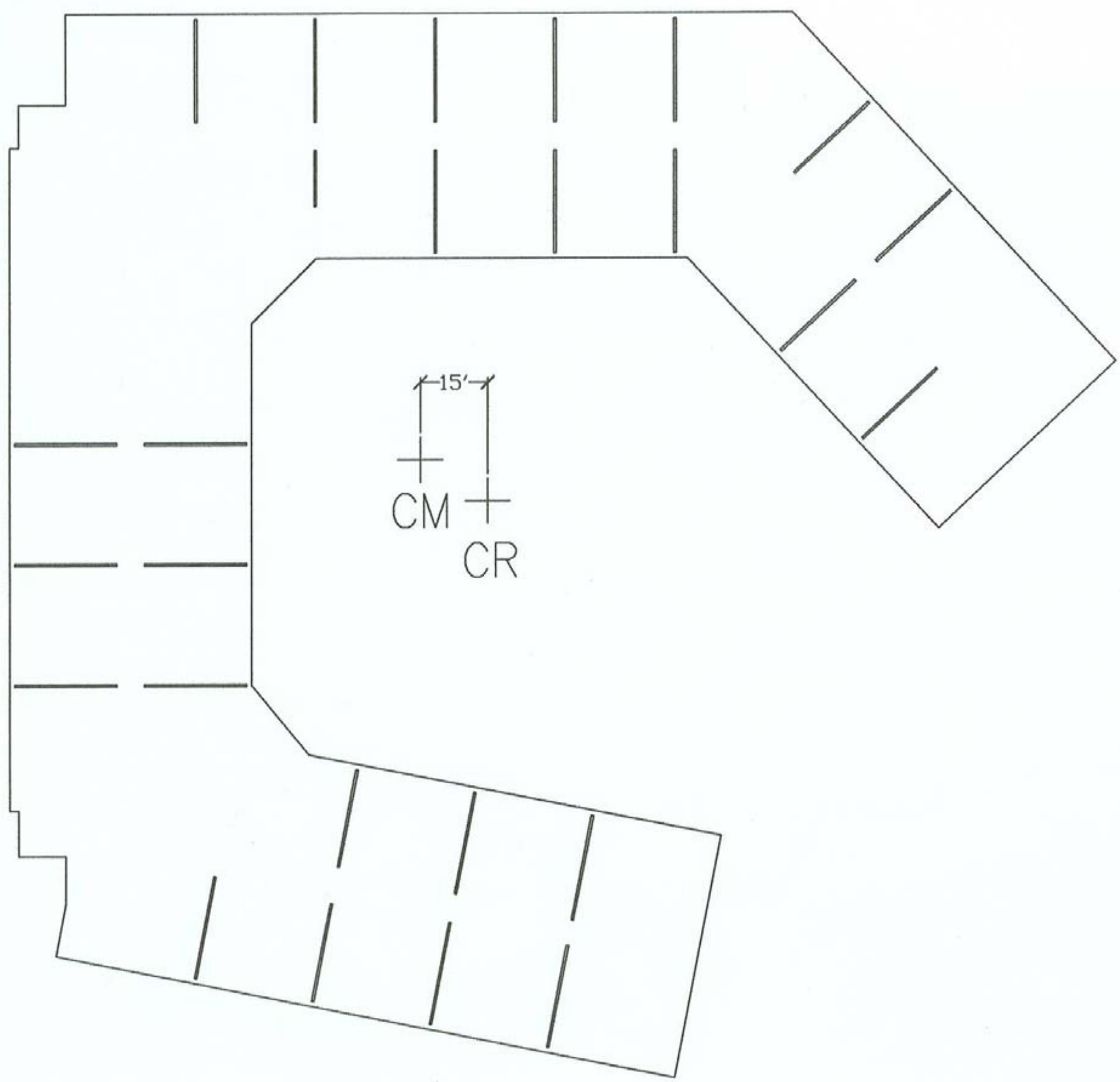
Vertical Distribution of Forces

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

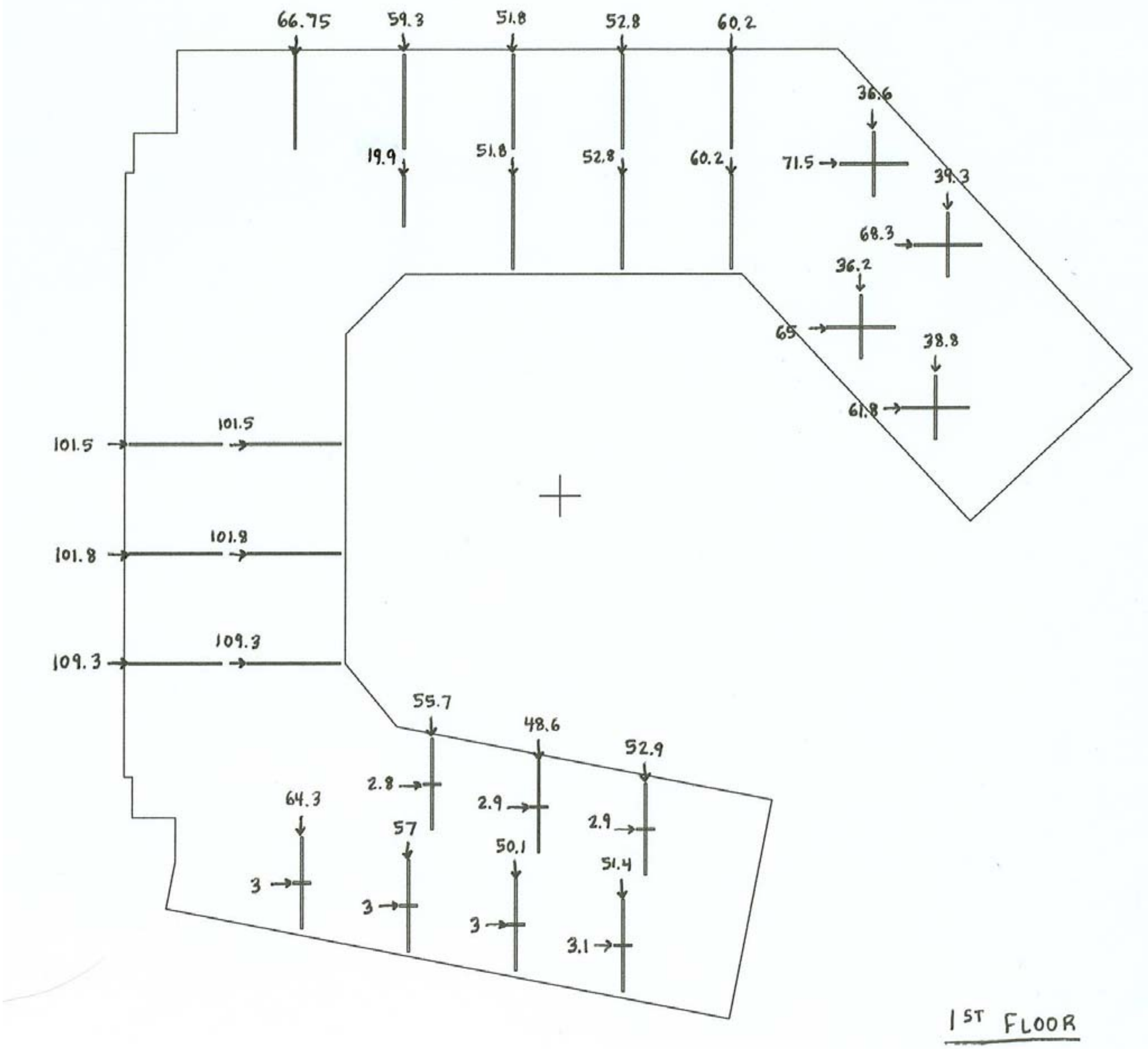
$k = 1$

– See Table for forces

Appendix B:



APPENDIX C: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60.2
19.9
51.8
52.8
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36.6
71.5
39.3
68.3
36.2
65
38.8
61.8
101.5
101.5
101.8
101.8
109.3
109.3
55.7
48.6
52.9
2.8
2.9
2.9
64.3
57
50.1
51.4
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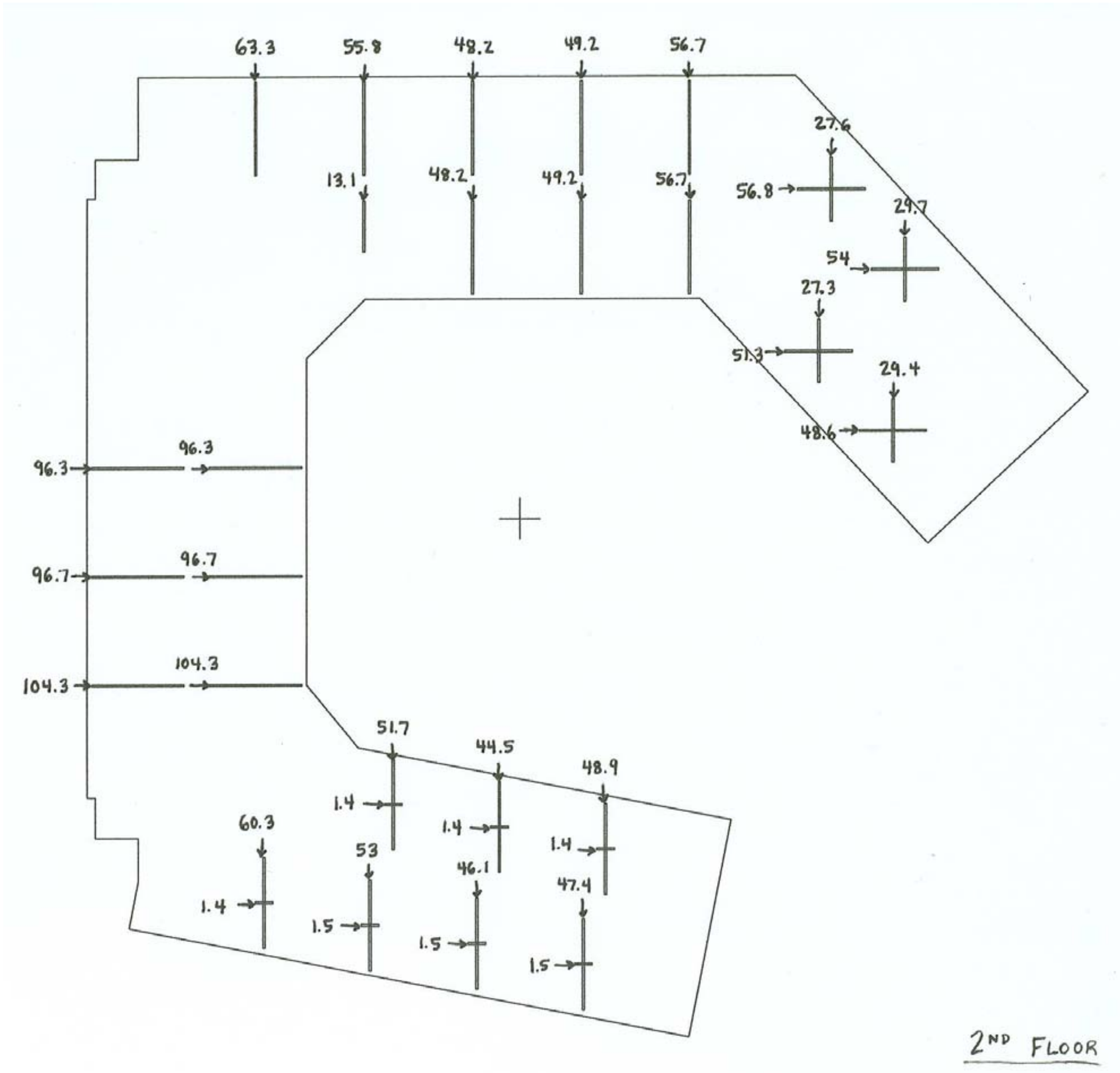
3
3
3.1
1ST FLOOR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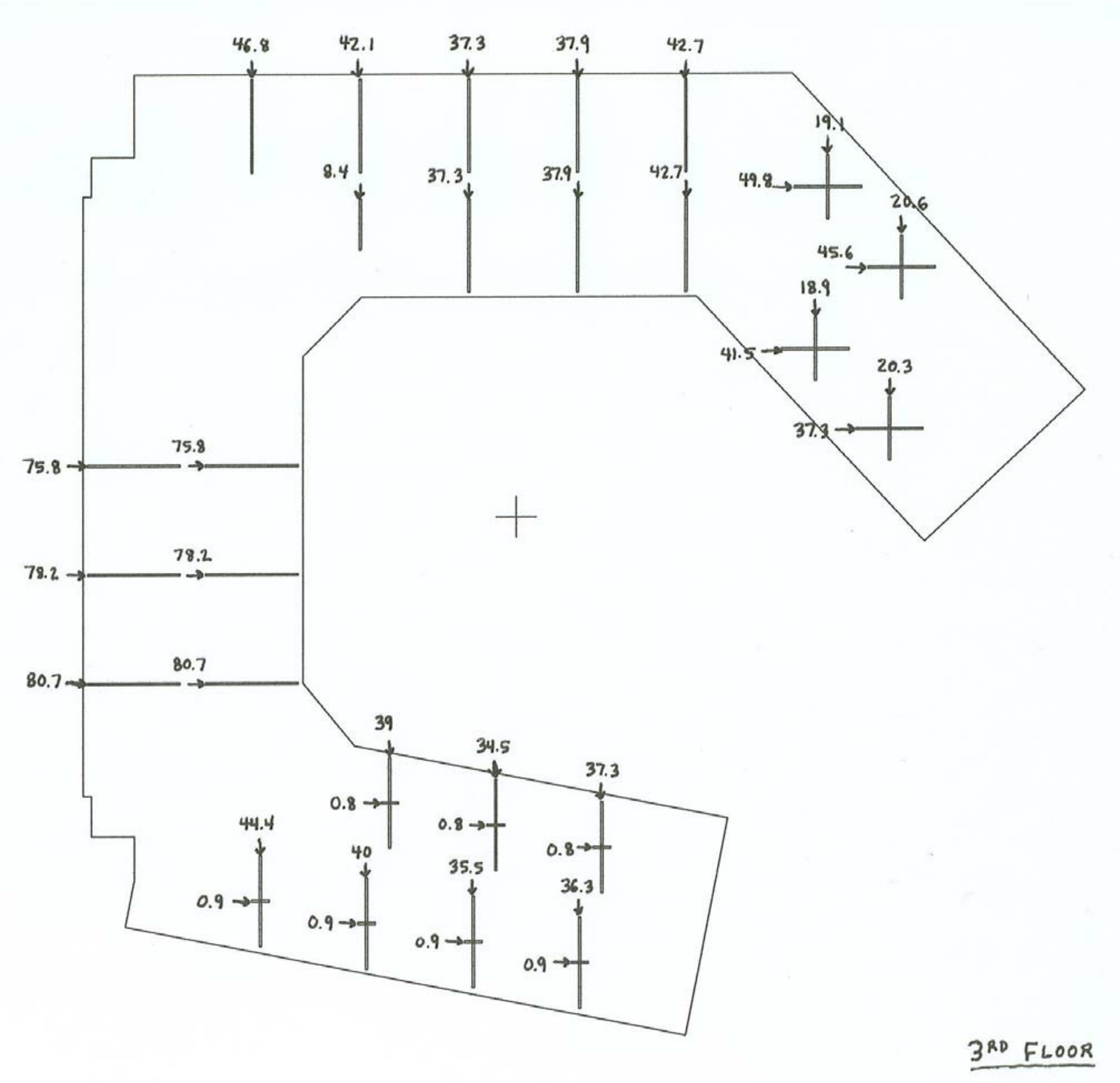
55.8
48.2
49.2
56.7
13.1
48.2
49.2
56.7
27.6
56.8
29.7
54
27.3
51.3
29.4
48.6
96.3
96.3
96.7
96.7
104.3
104.3
51.7
44.5
48.9
1.4
1.4
1.4
60.3
53
46.1
47.4
1.4
1.5
1.5
1.5
2ND FLOOR


46.8
42.1
37.3
37.9
42.7
8.4
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37.9
42.7
19.1
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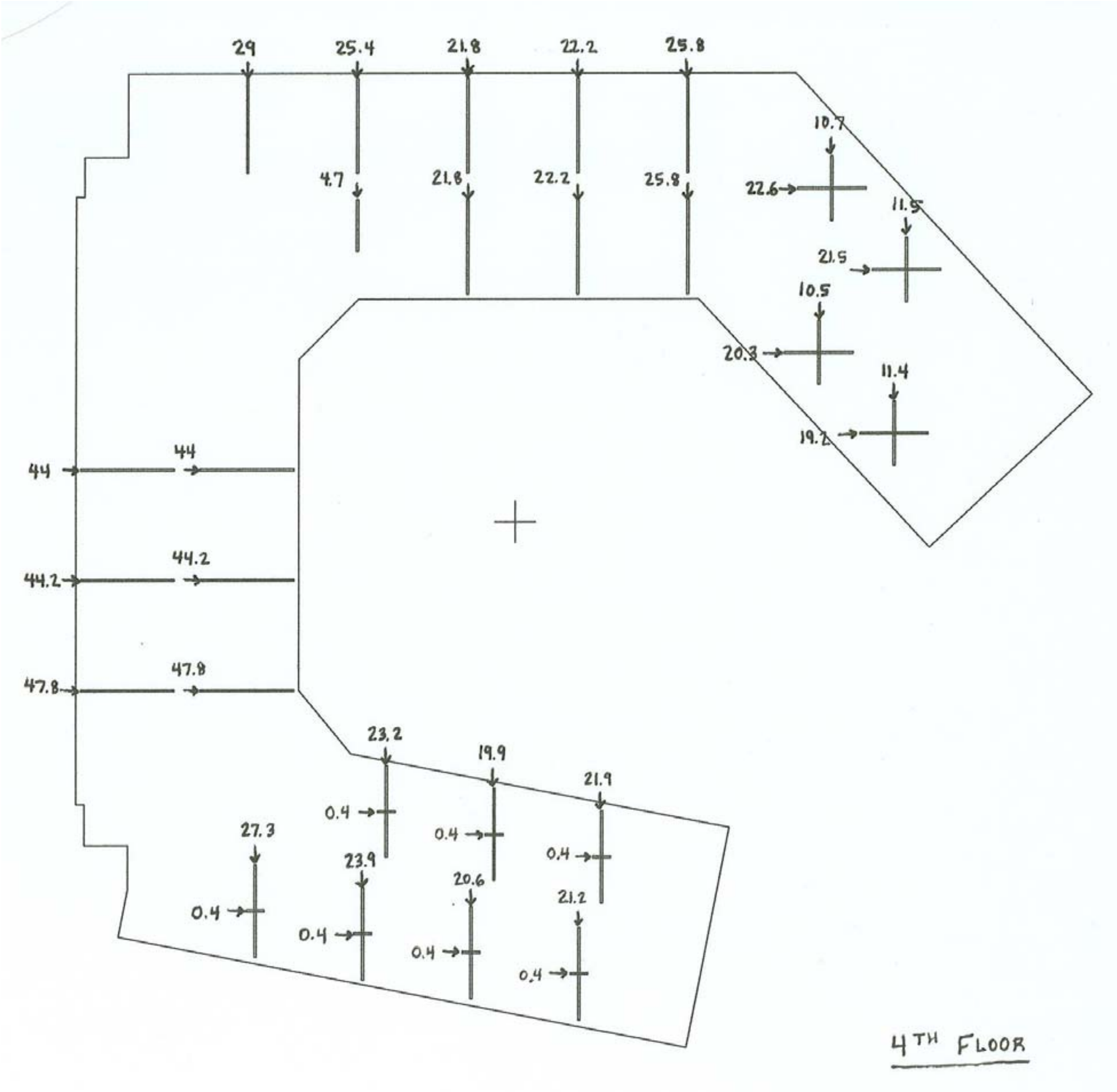
18.9
41.5
20.3
37.3
75.8
75.8
78.2
78.2
80.7
80.7
39
34.5
37.3
0.8
0.8
0.8
44.4
40
35.5
36.3
0.9
0.9
0.9
0.9
3RD FLOOR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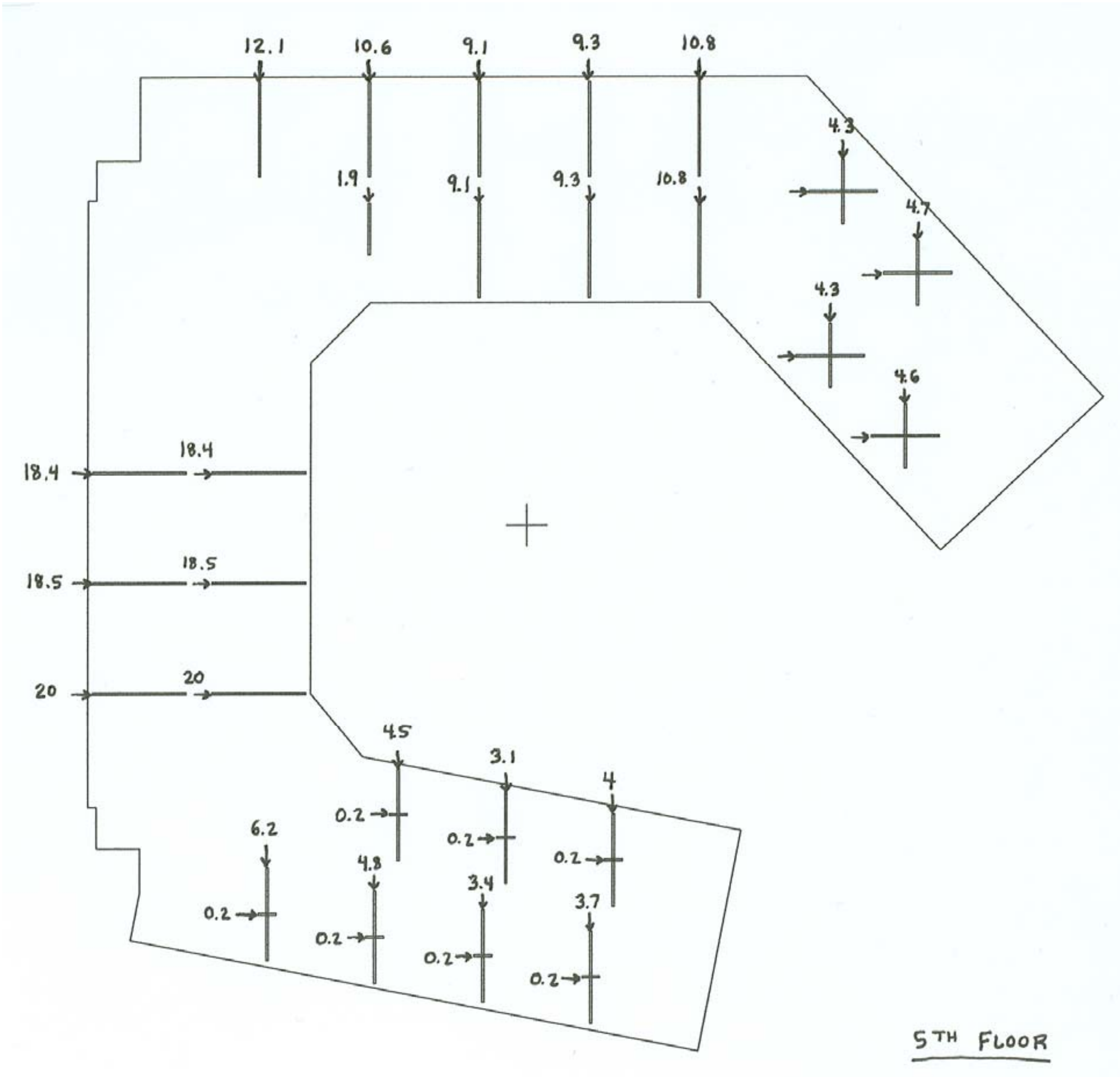
10.5
20.3
11.4
19.2
44
44
44.2
44.2
47.8
47.8
23.2
19.9
21.9
0.4
0.4
0.4
27.3
23.9
20.6
21.2
0.4
0.4
0.4
0.4
4TH FLOOR


12.1
10.6
9.1
9.3
10.8
1.9
9.1
9.3
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4.3
4.7
4.3
4.6
18.4
18.4
18.5
18.5
20
20
4.5
3.1
4
0.2
0.2
0.2
6.2
4.8
3.4
3.7
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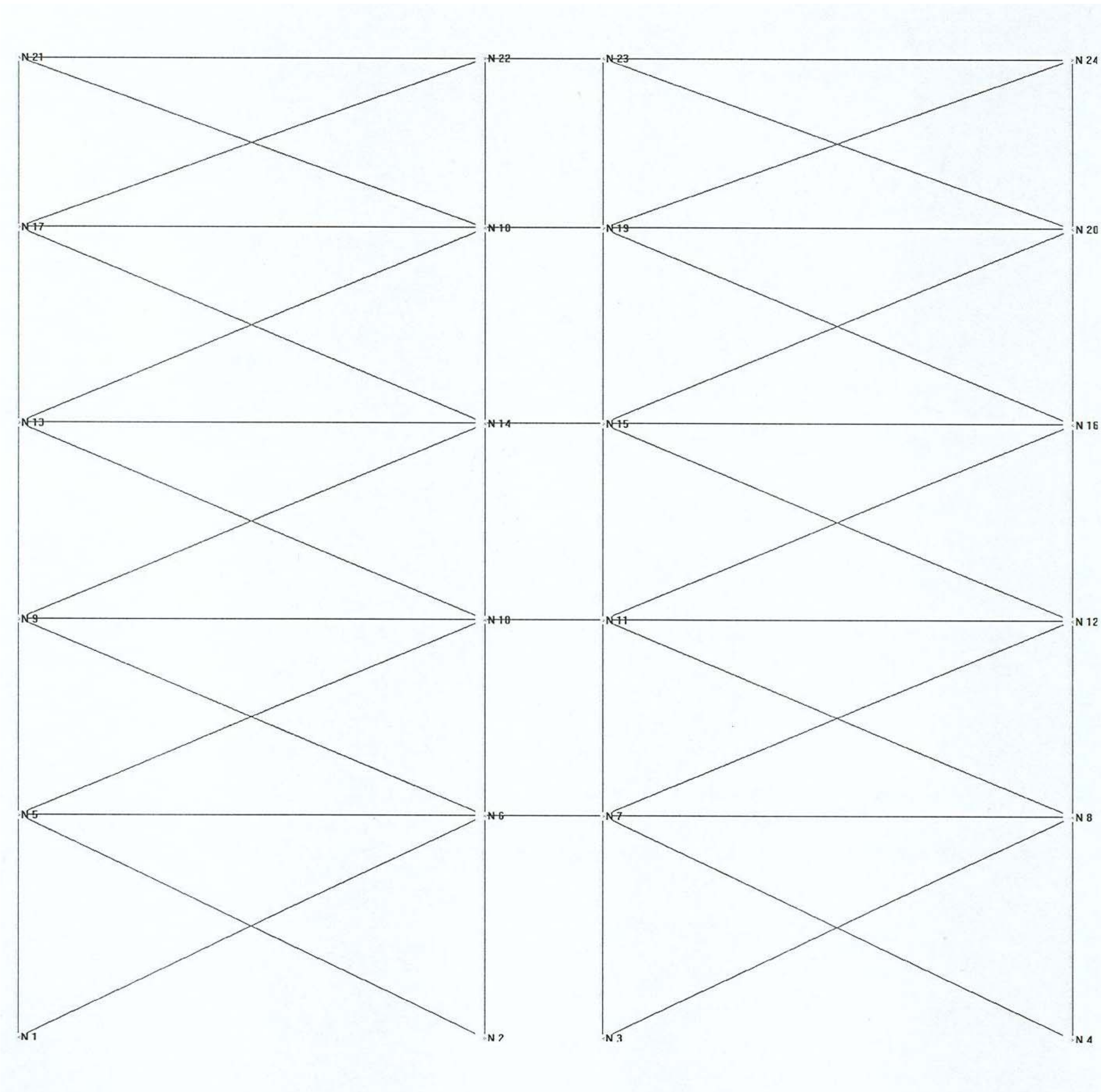
0.2
0.2
5TH FLOOR

Appendix D: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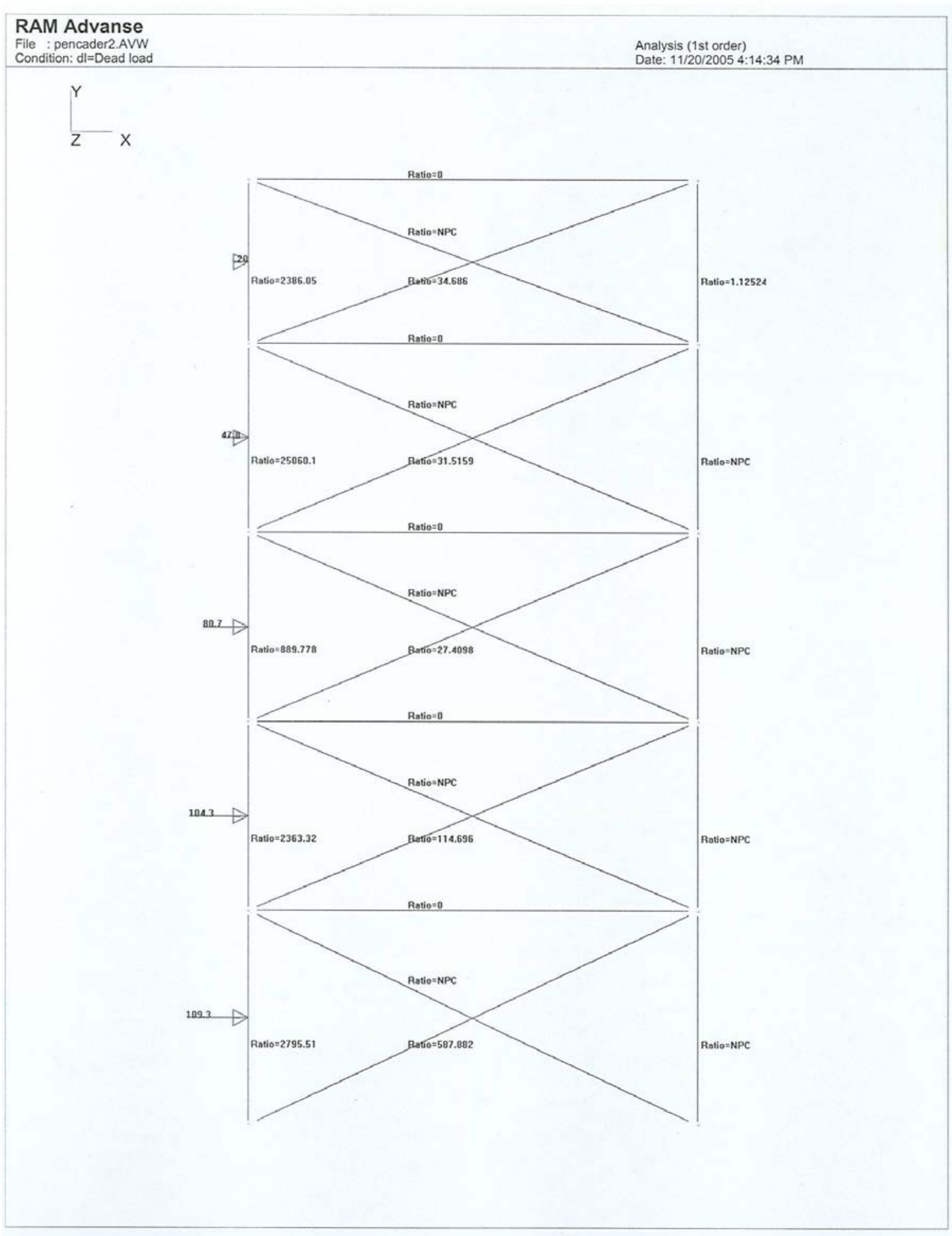
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RAM Advanse

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Analysis Results

Translations

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2	0.00000	0.00000	0.00000	0.00000	0.00000	-0.05725
3	0.00000	0.00000	0.00000	0.00000	0.00000	-0.03092
4	0.00000	0.00000	0.00000	0.00000	0.00000	-0.03087
5	4.49041	1.01013	0.00000	0.00000	0.00000	0.27198
6	4.47484	-1.01697	0.00000	0.00000	0.00000	-0.00058
7	4.45016	1.01373	0.00000	0.00000	0.00000	-0.03050
8	4.44498	-1.01329	0.00000	0.00000	0.00000	-0.03049
9	8.02600	1.42863	0.00000	0.00000	0.00000	-0.05125
10	8.01530	-1.43258	0.00000	0.00000	0.00000	-0.03920
11	7.99843	1.43464	0.00000	0.00000	0.00000	-0.02520
12	7.99490	-1.43365	0.00000	0.00000	0.00000	-0.02522
13	10.68069	1.59378	0.00000	0.00000	0.00000	0.12283
14	10.67291	-1.59720	0.00000	0.00000	0.00000	-0.01493
15	10.66047	1.60164	0.00000	0.00000	0.00000	-0.01953
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