# Handley McDonald

# Technical Report 3


Claude Moore Medical Education Building

Faculty Advisor: Kevin Parfitt

Handley McDonald

# TABLE OF CONTENTS

# EXECUTIVE SUMMARY

The Claude Moore Medical Education center is the newest addition to the University of Virginia's health and medical sciences program. The project itself is meant to push the department and the school forward into the future with new labs, new techniques, and a new space to learn. The 58,000 square foot building achieves this perfectly by providing state of the art mock labs and outpatient care, as well as appealing architecture that is meant to make the students feel welcome.

The third technical report is a summary and design check of the lateral resisting system of the building, in reference to both wind and seismic design. It includes a summary of how the system is put together, the distribution of forces, and several calculations to prove the accuracy of the original design.

A simplified lateral model of the structure was made in ETABS to help understand how the global system works together, as opposed to simply looking at individual parts. While the model itself provides an excellent visual, there is a serious inaccuracy within the digital model that gave completely unreasonable results, and therefore could not be compared to hand calculations. This situation will require much more attention as the project moves forward into next semester.

As for calculations, ASCE 7-10 guidelines were used in deriving wind loads and seismic loads on the structure. These loads were distributed across the system in accordance with each element's individual stiffness, and torsional effects were accounted for. To derive the stiffness of the lateral elements, a 1 kip load was applied to the top of each, and the displacement was calculated to come up with a final stiffness. To save time, models of the individual moment frames were made in ETABS, and given the same treatment, however results here were clearly inaccurate as well, yielding a stiffness of just 27 k/in in the central moment frame. As a result, the torsional effects of the lateral loads appear larger than what seems reasonable. Once again, this will receive much more attention in upcoming projects, but for now, it is a work in progress.

# BUILDING INFORMATION

Claude Moore Medical Education Building

58,000 sq. ft.

Type B and A-3 mixed occupancy

6 total levels, 4 above grade

| | |
|---|---|
| OWNER | University Of Virginia \| 575 Alderman Rd Charlottesville, VA |
| ARCHITECT | CO Architects \| 5055 Wilshire Blvd Los Angeles, CA |
| ASSOCIATE ARCH | Train and Partners Architects \| 1218 E Market Street Charlottesville, VA |
| BUILDER | Barton Malow Construction \| 100 Tenth Street NE #100 Charlottesville, VA |
| STRUCTURAL ENG | Nolen Frisa Associates \| 103 Homestead Dr Forest, VA |
| M.E.P. ENG | Bard, Rao& Thomas \| 311 Arsenal St Watertown, MA |
| CIVIL ENG | RMF Engineering \| 217 5th St, N.E. #2 Charlottesville, VA |
| LANDSCAPE ARCH | Dirtworks, PC \| 200 Park Avenue South New York, NY |
| GEOTECH ENG | Schnabel Engineering South \| 2020 Avon Court, #15 Charlottesville, VA |
| AUDIOVISUAL | The Sextant Group \| 730 River Avenue #600 Pittsburgh, PA |

The Claude Moore Medical Education Building was constructed on the University of Virginia's Health System campus, where they are centralizing all of their medical facilities, both educational and practical. Completed in August of 2010, just in time for classes, the new building was to represent a huge leap forward in medical technologies, and demonstrate the new, hands on teaching facilities of the University.

*Fig. 1*

*The third floor Lecture hall can seat 117 students, and provides a traditional learning environment.*

This new style of teaching the medical students is represented best in the Learning Center (shown in fig. 2), a large, round room meant to encourage group oriented learning, as opposed to the traditional lecture hall classrooms. Below this learning center, are state of the art mock medical facilities, to provide hands on training in a controlled environment, and with trained "patients." In addition, it will also include a traditional lecture hall, administrative offices, and student lounge.

*Fig. 2*

*The Learning Center provides a high-tech and group oriented learning space, where students can collaborate with the teacher, as well as each other.*

Exceeding the University's environmental building policy, the Claude Moore building received a LEED silver certification due to a number of environmentally friendly systems. These systems include efficient HVAC equipment, a cool roof design, and several water reduction strategies that help to reduce the amount of runoff from the building.

The entire project cost $40 million, and greatly adds to the effort of condensing the medical facilities of the University.

# STRUCTURAL SYSTEM OVERVIEW

The Claude Moore Medical Education Building is a four level structure. The main structure is a composite deck system, composed of steel beams, columns, and a concrete slab on metal floor decking. This system rests on a foundation of drilled concrete piers that continue approximately 25'below grade into bedrock. In several aspects of the design, the large circular section of the building that contains the lecture hall and Learning Center, are distinguished from the typical structural design, and is referred to as the "drum."

## FOUNDATION

The foundation for the Medical Education Building primarily consists of 18" drilled piers. These piers are made of 4000 psi, normal weight concrete, and go 2' into the bedrock underneath the site. This decision was made based on the geotechnical report done by Schnabel Engineering South in 2006. Because of the large column loads, and limited space between this site and the adjacent buildings, a deep foundation had to be used.

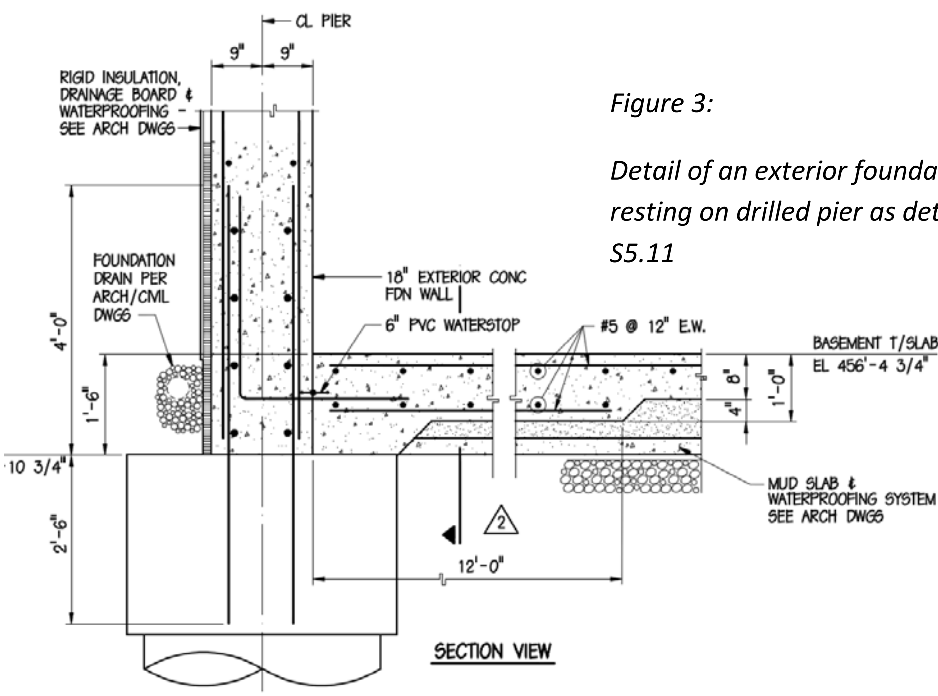



*Figure 3:*

*Detail of an exterior foundation wall resting on drilled pier as detailed in S5.11*

The basement level foundation walls are made of 18" thick cast in place concrete, reinforced with both vertical and horizontal reinforcement. These walls rest on the same centerline as the drilled piers below and connect to a 12" thick slab on grade system that includes a mud slab, and waterproofing.

## FLOOR SYSTEM

The ground level is made up of an 8" thick concrete slab on grade, with reinforcing in both directions. Below this slab is a mud slab and a waterproofing system, to help stabilize and protect the slab. On each of the floors above, there is a composite metal deck with lightweight concrete, laid in thicknesses of 4.5" and 5.5" (including deck thickness). All metal decking was used in conjunction with composite steel beams, and welded shear studs. All ends were built with a minimum of 1.5" overlay, and end joints lapped at least 2". The beam and girder system here is relatively light, with most wide flanges ranging from W18 to W24, and 10 to 40 pounds per linear foot. Due to the minimal amount of space, and difficulty of the structural system, there is not really any typical bay type; however the rectangular layout fits into the drum section with minimal interruption.


*Figure 4: Installation of lecture hall structure*

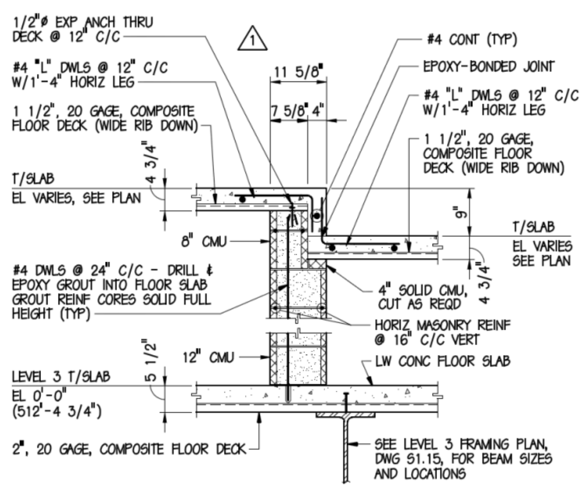



*Figure 5: Detail of lecture hall floors, as noted in S5.22*

For the lecture hall, 8" grout filled CMU was used to support the stepped composite floor deck. This slab is a 4.75" thick slab, and the circular CMU walls rest on a 5.5" composite floor deck. This part of the building has a much larger substructure of wide flanges, most of which are greater than 150 pounds per linear foot. There is no typical bay type for this section of the floor structure either.

## FRAMING SYSTEM

All of the framing for the Claude Moore Building was done with steel wide flanges. The beams, as previously mentioned, unfortunately do not follow much of a typical plan for size or spacing, but one should note that very minimal deviations were made as far as fitting the structure of the drum area into the rectangular structure of the rest of the building. A framing plan for reference is located in Appendix D. The columns are mostly W12 to W24 wide flanges; however the weights and spacings vary greatly within that. Because of the irregularity in the framing system, several transfer girders were necessary to allow for the change in structure from floor to floor. Most of these transfers happen below the first floor, and allow for the load to move from the main structure to the structure below grade.

## LATERAL RESISTING SYSTEM

The lateral resisting system for this project is mostly made up of moment frames. Originally, the intent was to use only moment frames, with limited X-bracing to react with the curtain wall system. Changes were made, however, when the owner and architect modified the design, and limited the space available such that other options had to be considered. As a result, the system is a hybrid of moment frames, X-bracing, and shear walls.

The bays that include X-bracing are shown below in blue. The east wall braces are made of HSS 4x4x3/16 sections, and the south wall employs the same type of section, ranging from HSS 7x5x1/4 to HSS 9x5x1/2. The loads applied to these systems are transferred to the cast in place concrete foundation wall below, using a bolted base plate connection. In addition to these braced frames, two 14' long 12" CMU shear walls (red) were added at the plan southwest and southeast corners of the building. These walls help for shear in the north-south direction, and transfer their loads directly to the basement foundation below. The moment frames are given in green, and generally fill out the middle of the structure. These frames are also steel wide flanges, with bolted and welded connections to transfer lateral load directly to the foundation below.

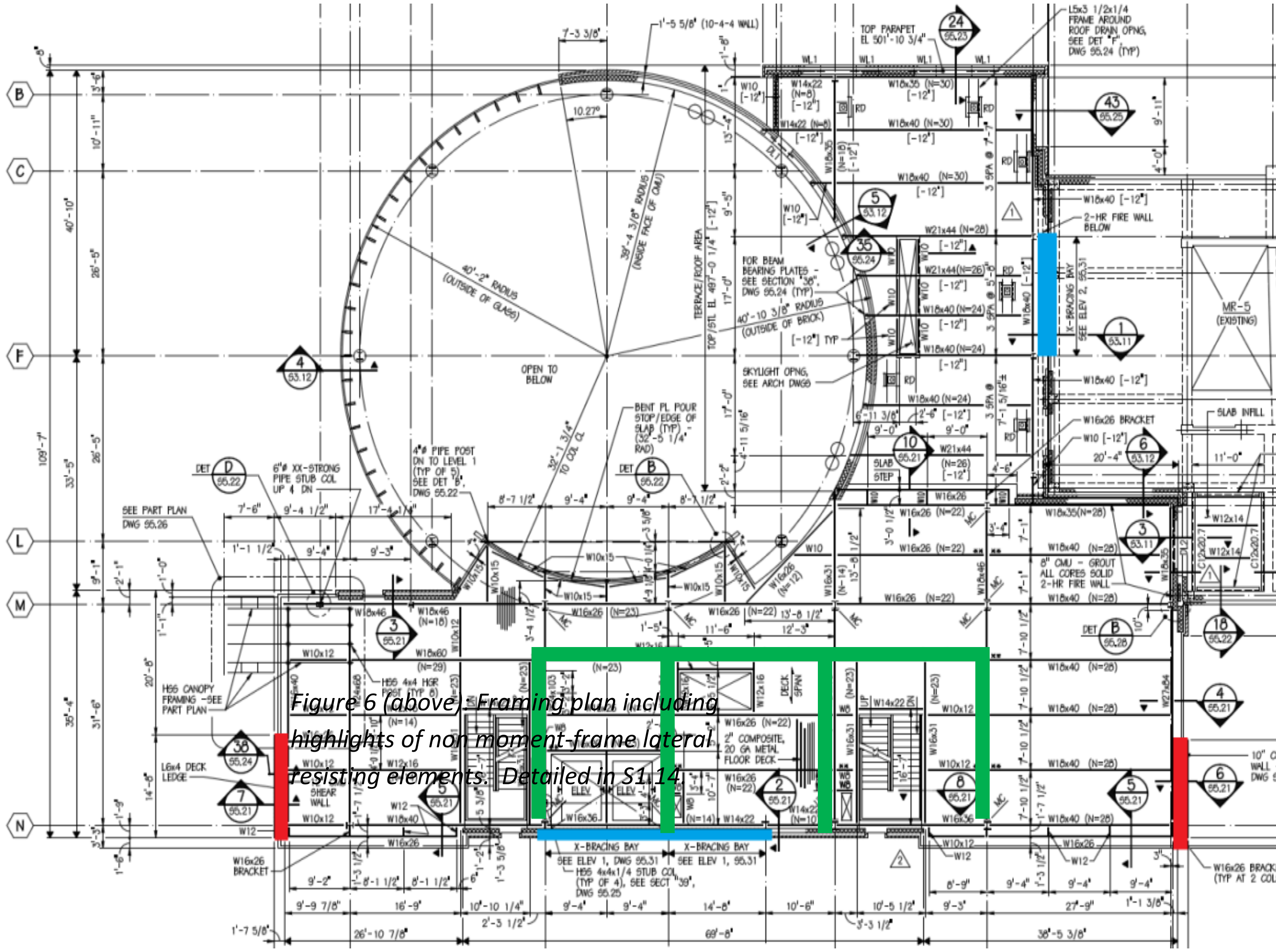


*Figure 6 (above): Framing plan including highlights of non moment-frame lateral resisting elements. Detailed in S1.14*

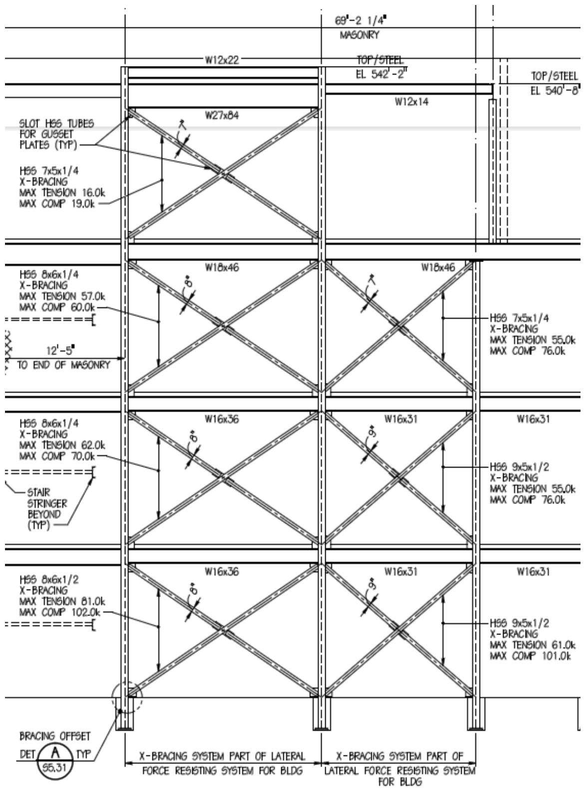
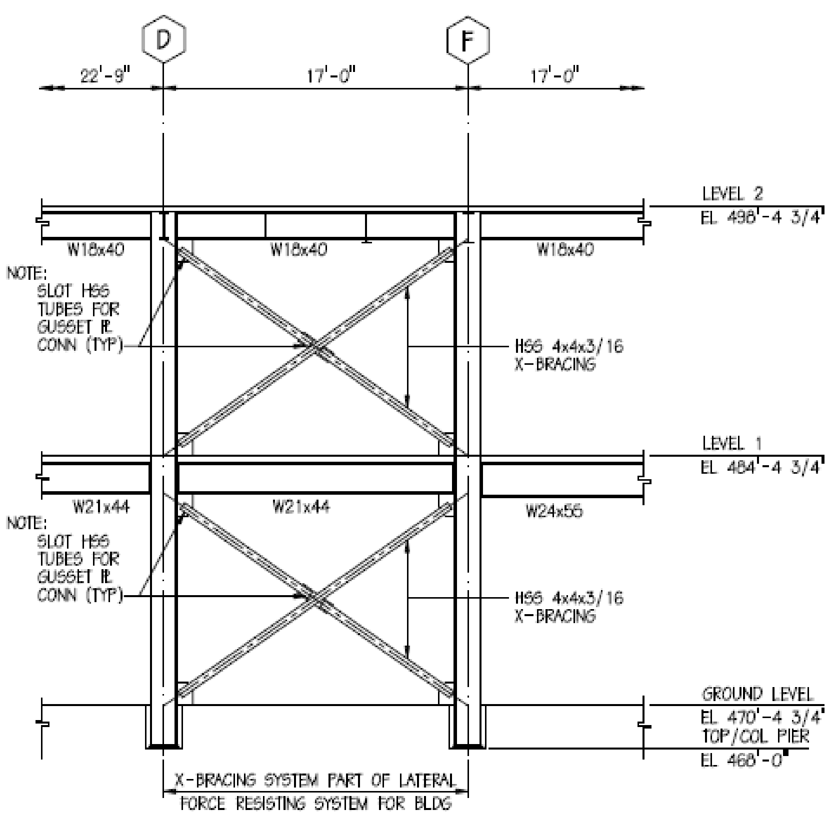



*Figure 7 (left): Elevation of X-bracing between column lines 3 and 5.9 as detailed in S5.31.*

*Figure 8 (below): Elevation of X-bracing between column lines D and F, as detailed in S5.31*



## DESIGN CODES

According to sheets S0.11 and A0.02, the following major code regulations were applied to this project:

- IBC 2003 with VA amendments (Virginia Uniform Statewide Building Code)
- IFC 2003 with VA amendments (Virginia Statewide Fire Prevention Code)
- IMC 2003 International Mechanical Code
- IPC 2001 International Plumbing Code
- ANSI/ASME A17.1 Safety Code for Elevators and Escalators
- Local ordinances and amendments to all of the above codes
- ACI 318-02 Structural Concrete Building Code
- AISC Manual of Steel Construction, 9th edition
- ASCE 5-02, 6-02 Code Requirements and Specifications for Masonry Structures
- ASCE 7-02 Minimum Design Loads for Buildings

These code standards vary from the ones used in this report, and from the ones that will be used in future reports. These differences will result in variations between the report results, and the results used in the building design.

## MATERIALS USED

The following is a breakdown of the structural materials used throughout the building as taken from S0.11

| | STEEL | |
|---|---|---|
| **Use** | **Class** | **Strength** |
| W Sections | ASTM A992 GR 50 | 50000 psi |
| Channels, Angles, & Plates | ASTM A36 | 36000 psi |
| Hollow Structural Sections | ASTM A500 GR B | 46000 psi |
| Steel Pipe Section | ASTM A53 GR B Type E or S | 35000 psi |
| Structural Bolts | ASTM A325 and A490 | n/s |
| Welding Electrodes | -- | E70xx |
| Anchor Bolts | ASTM F1554 GR 36 | 36000 psi |
| Headed Shear Studs for Composite Beams | ASTM A108 | 60000 psi<br>Designed for 11.4k per stud |

**CONCRETE**

| Use | Class | Strength |
|---|---|---|
| Slab on grade, cast in place walls & foundations | Normal Weight (Assume 150 $lb/ft^{3}$) | 4000 psi |
| Elevated Floor Slabs | Light Weight (Assume 100 $lb/ft^{3}$) | 4000 psi |
| Reinforcing Steel | ASTM A615 GR 60 | Fy=60000 psi |
| Welded Wire Fabric | ASTM A185 | Fy=60000 psi |

| MASONRY | | |
|---|---|---|
| **Use** | **Class** | **Strength** |
| Lightweight CMU | ASTM C90 GR N-1 | f'm=1500 psi |
| Mortar for CMU | ASTM C270 Type S | f'c=1800 psi |
| Structural Grout | ASTM C476 | f'c=2500 psi |
| Vertical Reinforcement | ASTM A615 GR 60 | fy=60000 psi |
| Horizontal Joint Reinforcement | ASTM A82 w/ galvanizing per ASTM A 153 class B-2 | n/s |

| | SOILS |
|---|---|
| **Use** | **Strength** |
| Bearing Capacity | 3000 psf standard bearing case |
| Bedrock Bearing | 50 ksf for drilled piers |
| Disintegrated Rock Bearing | 25 ksf for drilled piers |
| Side Friction | 2 ksf for elevation below 450' above sea level |

## GRAVITY LOADS

| USE | LOAD |
|---|---|
| **Roof live load** | 30psf unreduced |
| **Assembly and large lecture halls** | 100psf |
| **Terrace level roof at 2nd level** | 100psf |
| **Stairs, corridors, lobbies, and exitways** | 100psf |
| **Classrooms and training rooms** | 100psf* |
| **Offices and conference rooms** | 100psf* |
| **File storage** | 250psf |
| **Mechanical equipment room (penthouse)** | 150psf or equipment weight |
| **Slab on grade at basement level** | 200psf |
| **All other floor areas** | 100psf* |

*Indicates areas designed for greater load than code minimum. These greater loads allow for flexibility in future use of the space.

# LATERAL LOADS

Several hand calculations were made pertaining to the amount of lateral load on the building. An estimation of wind load was performed using a simplified building shape, and a seismic load calculation was done using the MWFRS method.

# WIND LOADS

A simplified shape of the building was used to derive wind loads acting on it in both the N-S and E-W directions. A sketch of this shape can be found in Appendix A. ASCE 7-10 guidelines were used to derive wind pressures, base shear, and overturning moment. The results are shown below, and supporting calculations can be found in Appendix A. The wind forces for the E-W direction proved to be the controlling factor in the lateral design, with a total of 626k and an overturning moment of 6534 ft k.

| Wind Pressures (North South Direction) | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|
| Type | Level | Height/Distance (ft.) | Kz | qz/qh | Wind Pressure (psf) | Internal Pressure( psf) | | Net Pressure (psf) | |
| | | | | | | (+)Gcpi | (-)Gcpi | (+)Gcpi | (-)Gcpi |
| Windward Walls | 0 | 0 | 0.57 | 20.54 | 13.64 | 17.84 | -17.84 | 31.48 | -4.20 |
| | 1 | 14 | 0.57 | 20.54 | 13.64 | 17.84 | -17.84 | 31.48 | -4.20 |
| | 2 | 28 | 0.68 | 24.65 | 16.37 | 17.84 | -17.84 | 34.20 | -1.47 |
| | 3 | 42 | 0.77 | 27.75 | 18.42 | 17.84 | -17.84 | 36.26 | 0.59 |
| | 4 | 56 | 0.83 | 30.05 | 19.96 | 17.84 | -17.84 | 37.79 | 2.12 |
| | 5 | 72.67 | 0.90 | 32.43 | 21.53 | 17.84 | -17.84 | 39.37 | 3.70 |
| Leeward Walls | ALL | ALL | 0.90 | -19.08 | 7.92 | 17.84 | -17.84 | 25.75 | -9.92 |
| Side Walls | ALL | ALL | 0.90 | -26.71 | 15.52 | 17.84 | -17.84 | 33.35 | -2.32 |
| Roof | | 0-h/2 | 0.90 | -34.34 | 25.65 | 17.84 | -17.84 | 43.49 | 7.81 |
| | | h/2-h | 0.90 | -34.3388 | 25.65 | 17.84 | -17.84 | 43.49 | 7.81 |
| | | 2h | 0.90 | -19.0771 | 7.92 | 17.84 | -17.84 | 25.75 | -9.92 |

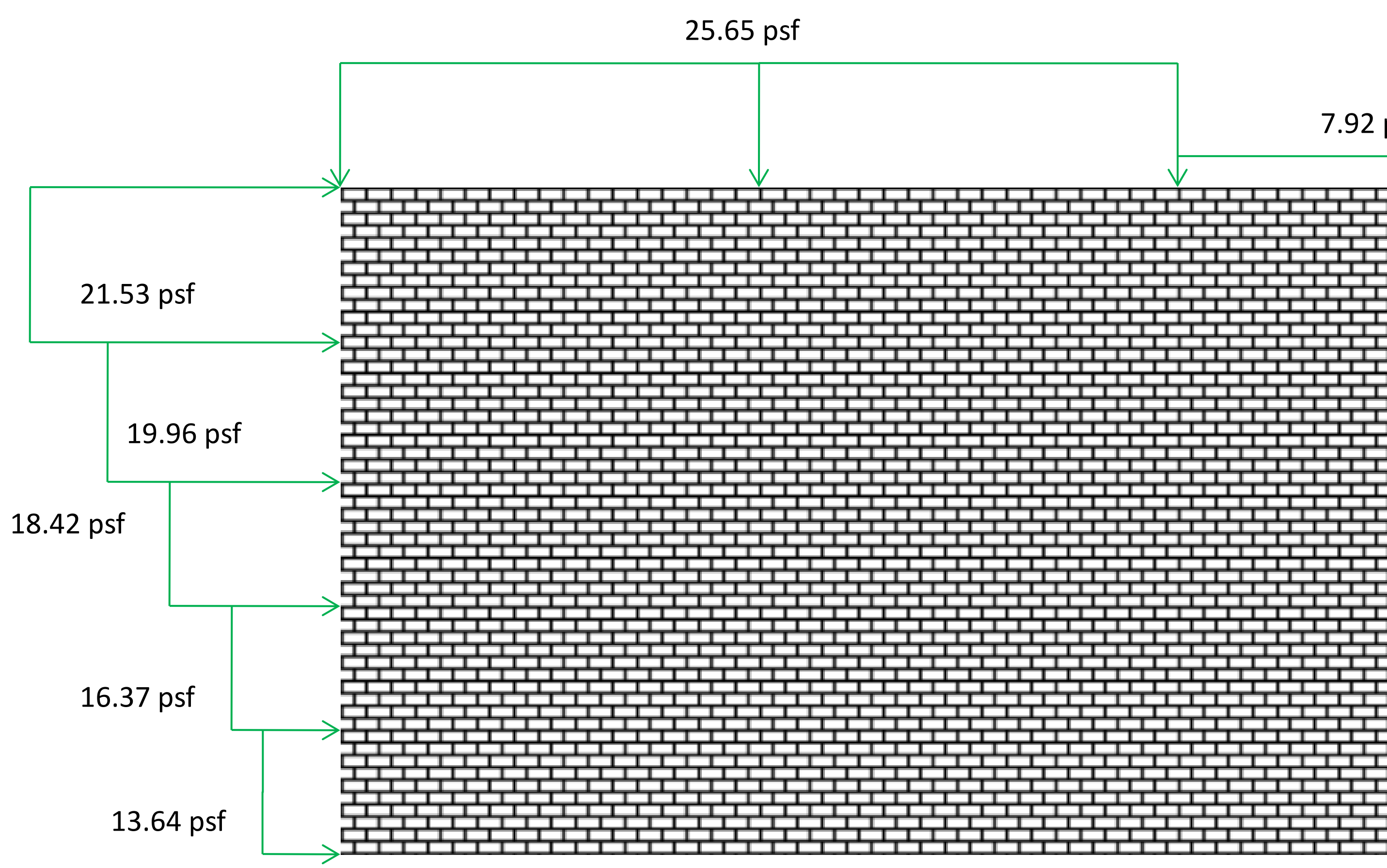



*Fig. 9 N-S wind pressures*

| Wind Forces (North South Direction) | | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| | | Trib Area Below | | Trib area above | | | | |
| Level | Elevation (ft) | Height (ft) | Area (sq ft) | Height (ft) | Area(sq ft) | Story Force (k) | Story Shear (k) | Overturning Moment (ft k) |
| 0 | 0 | | 0.00 | 7.00 | 901.81 | 12.30 | 162.22 | 0.00 |
| 1 | 14 | 7.00 | 901.81 | 7.00 | 901.81 | 24.60 | 149.92 | 344.38 |
| 2 | 28 | 7.00 | 901.81 | 7.00 | 901.81 | 29.52 | 125.32 | 826.51 |
| 3 | 42 | 7.00 | 901.81 | 7.00 | 901.81 | 33.23 | 95.80 | 1395.64 |
| 4 | 56 | 7.00 | 901.81 | 8.34 | 1074.44 | 39.44 | 62.57 | 2208.43 |
| 5 | 73 | 8.34 | 1074.44 | | 0.00 | 23.14 | 23.14 | 1681.39 |
| | | | | | | | Total Base Shear = | 618.97 |
| | | | | | | | Total Overturning Moment = | 6456.34 |

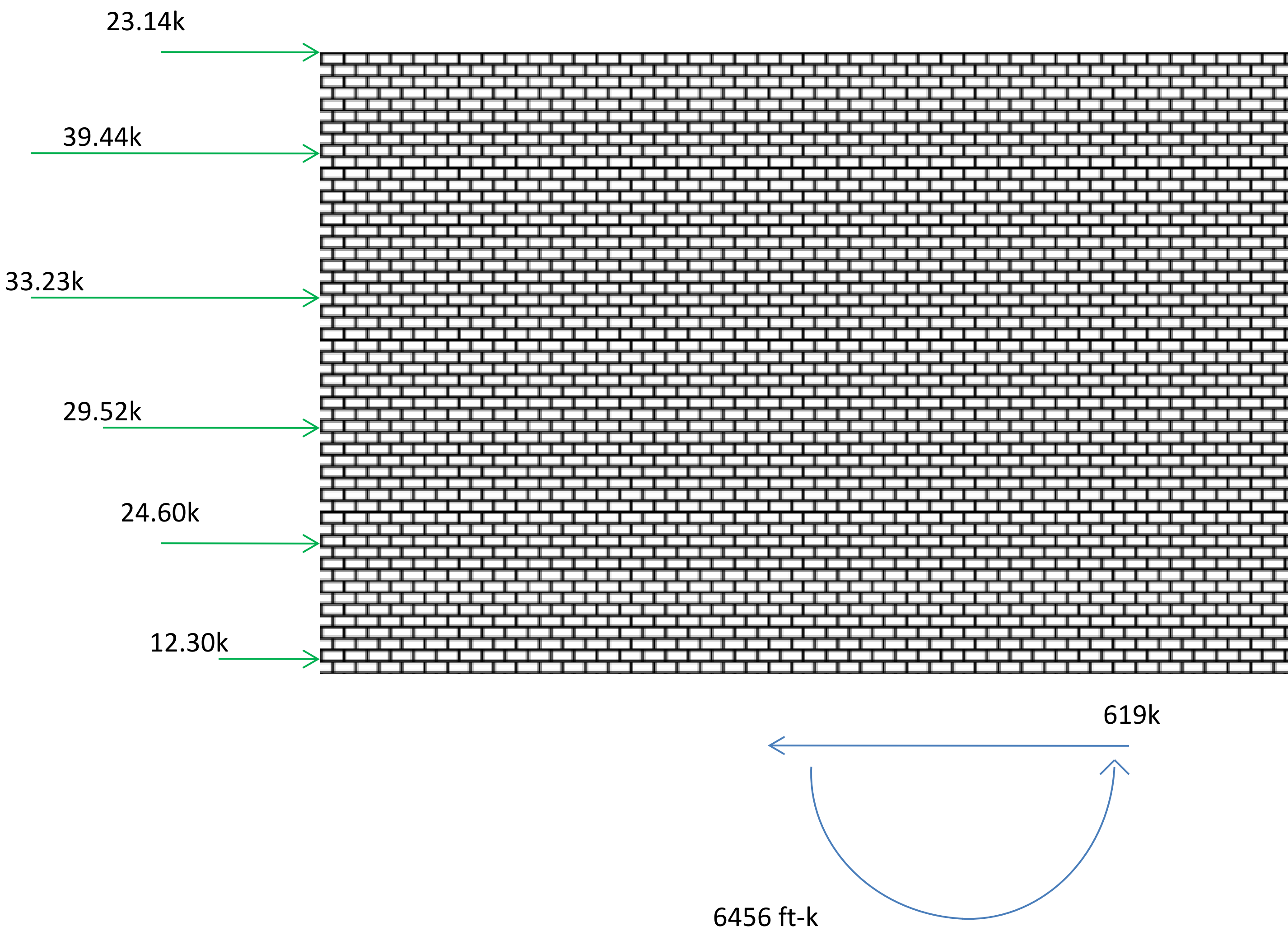



*Fig. 10 N-S Base Shear and Overturning Moment*

| Wind Pressures (East West Direction) | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|
| Type | Level | Height/Distance (ft) | Kz | qz/qh | Wind Pressure (psf) | Internal Pressure (psf) | | Net Pressure (psf) | |
| | | | | | | (+)Gcpi | (-)Gcpi | (+)Gcpi | (-)Gcpi |
| Windward Walls | 0 | 0 | 0.57 | 20.54 | 13.80 | 17.84 | -17.84 | 31.64 | -4.03 |
| | 1 | 14 | 0.57 | 20.54 | 13.80 | 17.84 | -17.84 | 31.64 | -4.03 |
| | 2 | 28 | 0.68 | 24.65 | 16.56 | 17.84 | -17.84 | 34.40 | -1.27 |
| | 3 | 42 | 0.77 | 27.75 | 18.65 | 17.84 | -17.84 | 36.48 | 0.81 |
| | 4 | 56 | 0.83 | 30.05 | 20.20 | 17.84 | -17.84 | 38.03 | 2.36 |
| | 5 | 72.67 | 0.90 | 32.43 | 21.79 | 17.84 | -17.84 | 39.63 | 3.96 |
| Leeward Walls | ALL | ALL | 0.90 | -17.55 | 6.87 | 17.84 | -17.84 | 24.71 | -10.97 |
| Side Walls | ALL | ALL | 0.90 | -26.71 | 15.70 | 17.84 | -17.84 | 33.54 | -2.13 |
| Roof | | 0-h/2 | 0.90 | -34.34 | 25.96 | 17.84 | -17.84 | 43.80 | 8.12 |
| | | h/2-h | 0.90 | -34.34 | 25.96 | 17.84 | -17.84 | 43.80 | 8.12 |
| | | 2h | 0.90 | -19.08 | 8.01 | 17.84 | -17.84 | 25.85 | -9.82 |

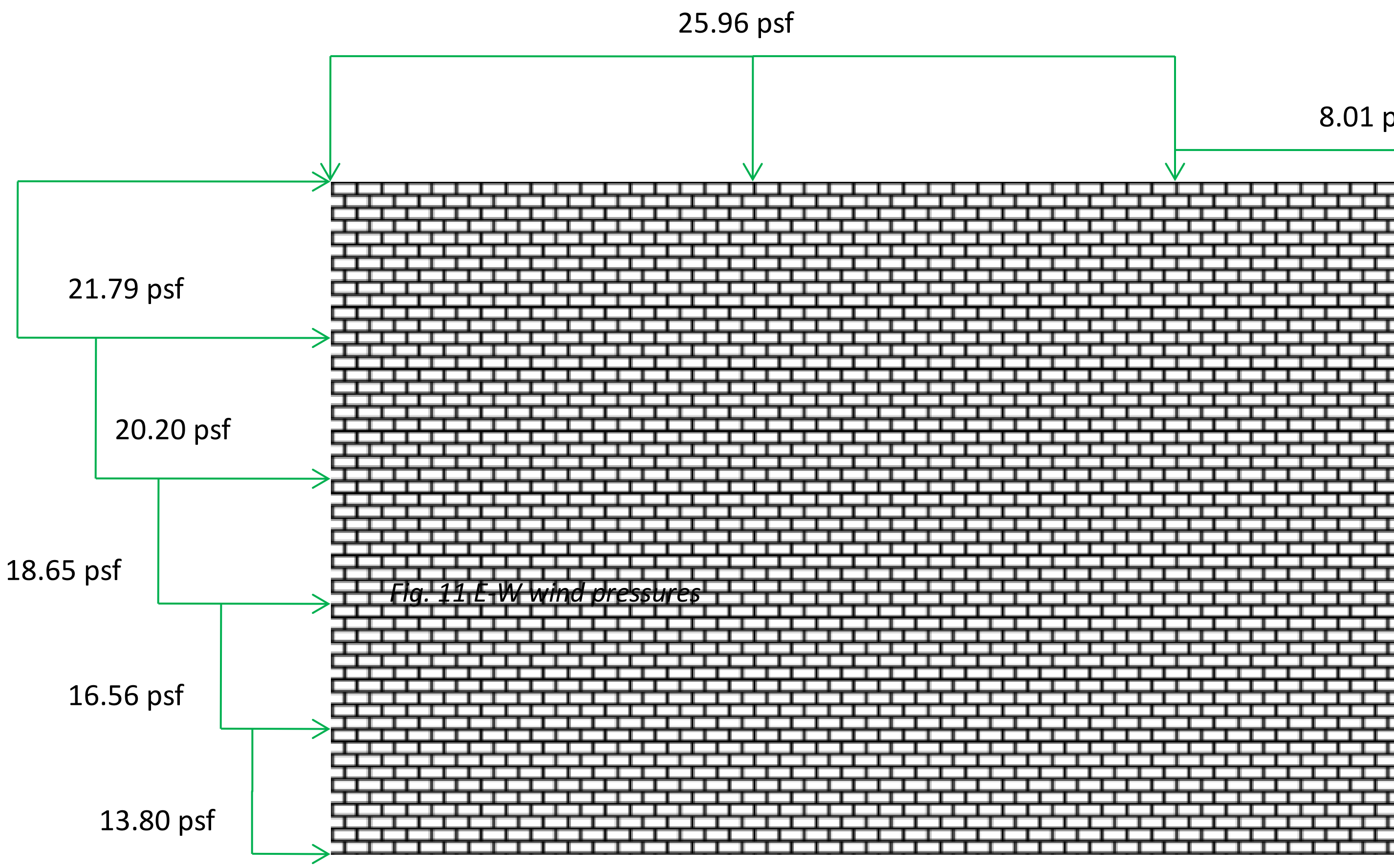



*Fig. 11 E-W wind pressures*

| Wind Forces (East West Direction) | | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| | | Trib Area Below | | Trib area above | | | | |
| Level | Elevation (ft) | Height (ft) | Area(sq ft) | Height (ft) | Area (sq ft) | Story Force (k) | Story Shear (k) | Overturning Moment (ft k) |
| 0 | 0 | | 0.00 | 7.00 | 901.81 | 12.45 | 164.17 | 0.00 |
| 1 | 14 | 7.00 | 901.81 | 7.00 | 901.81 | 24.89 | 151.73 | 348.53 |
| 2 | 28 | 7.00 | 901.81 | 7.00 | 901.81 | 29.87 | 126.83 | 836.46 |
| 3 | 42 | 7.00 | 901.81 | 7.00 | 901.81 | 33.63 | 96.96 | 1412.45 |
| 4 | 56 | 7.00 | 901.81 | 8.34 | 1074.44 | 39.91 | 63.33 | 2235.04 |
| 5 | 73 | 8.34 | 1074.44 | | 0.00 | 23.42 | 23.42 | 1701.65 |
| | | | | | | | Total Base Shear = | 626.43 |
| | | | | | | | Total Overturning Moment = | 6534.13 |

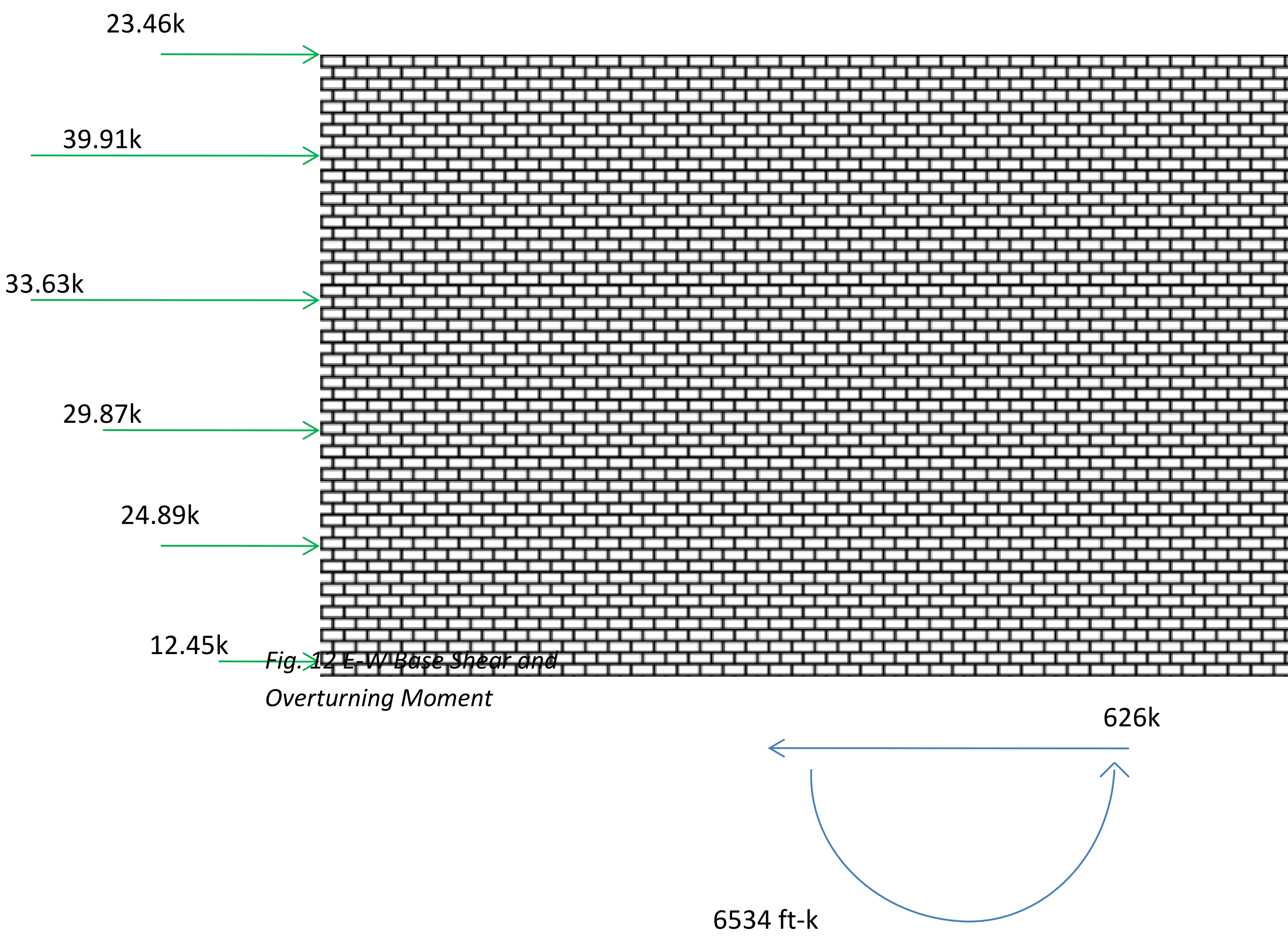



*Fig. 12 E-W Base Shear and Overturning Moment*

# SEISMIC FORCES

A derivation of the seismic loads on the structure was performed using the methods outlined in ASCE 7-05. The results are shown below, and the process can be found in Appendix C. The peculiar spikes in story force are attributed to the variation in floor weight. The drum, while it is four stories tall, only has two floor in it, and therefore these two floor are far heavier than other floors in the building, and carry more lateral seismic force. After the analysis, a total of 412 k base shear and 29581 ft k overturning moment were the result.

| Seismic Forces | | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| Floor | Area (sq ft) | Weight (k) | Height (ft) | wxhx^k | Cvx | Story Force (k) | Story Shear (k) | Overturning Moment (ft k) |
| 0 | 12971.42 | 1894.48 | 0.00 | 0.00 | 0.00 | 0.00 | 412.20 | 0.00 |
| 1 | 10619.29 | 1550.95 | 14.00 | 137727.04 | 0.39 | 160.02 | 412.20 | 5770.85 |
| 2 | 4058.15 | 592.69 | 28.00 | 44051.86 | 0.12 | 51.18 | 252.19 | 7061.21 |
| 3 | 7540.06 | 1101.23 | 42.00 | 91787.85 | 0.26 | 106.64 | 201.00 | 8442.19 |
| 4 | 4058.15 | 592.69 | 56.00 | 44051.86 | 0.12 | 51.18 | 94.36 | 5284.22 |
| 5 | 7540.06 | 513.48 | 70.00 | 37164.66 | 0.10 | 43.18 | 43.18 | 3022.57 |
| | | | | | | | Total Base Shear = | 412.20 |
| | | | | | | | Total Moment = | 29581.05 |

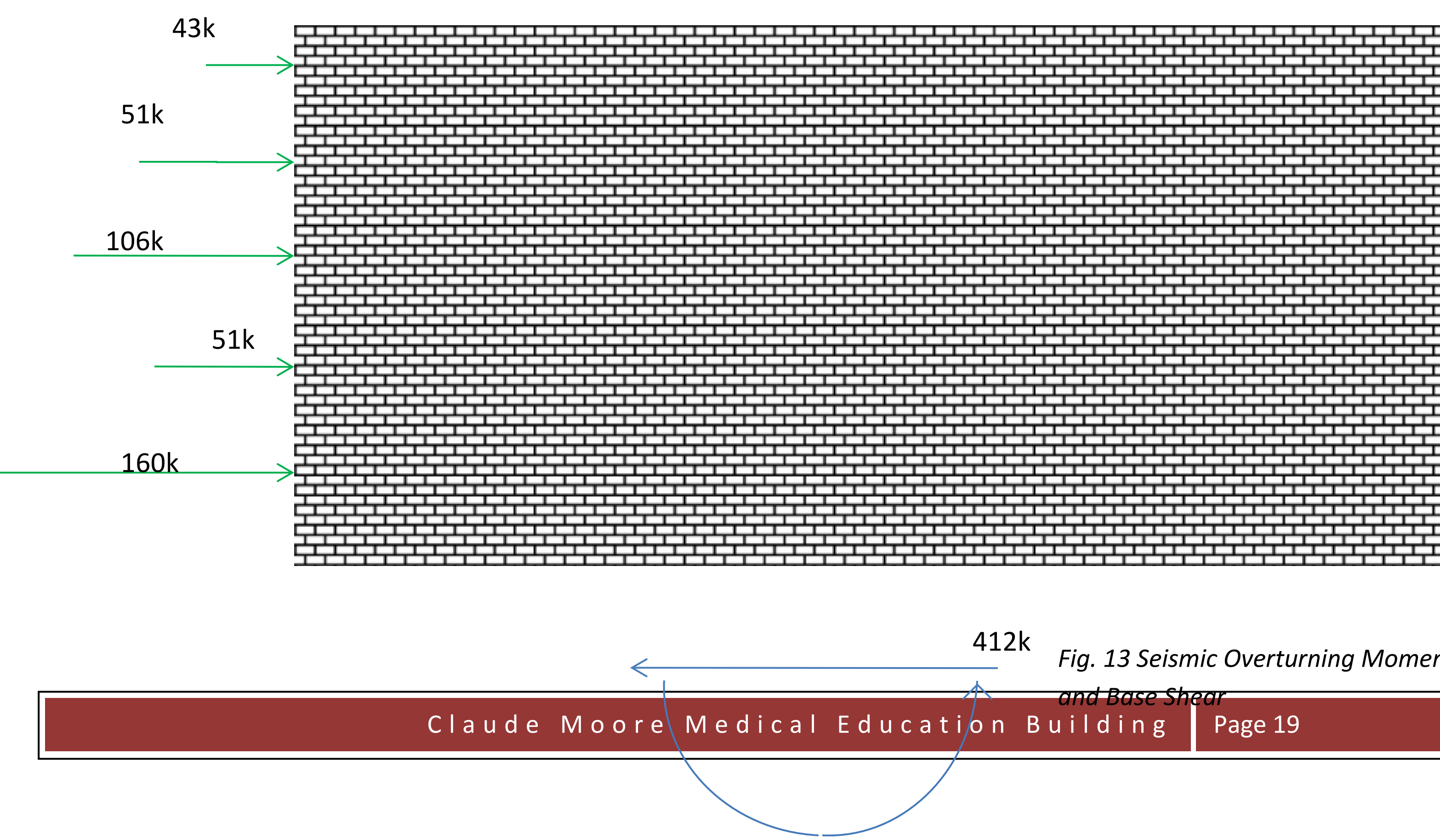


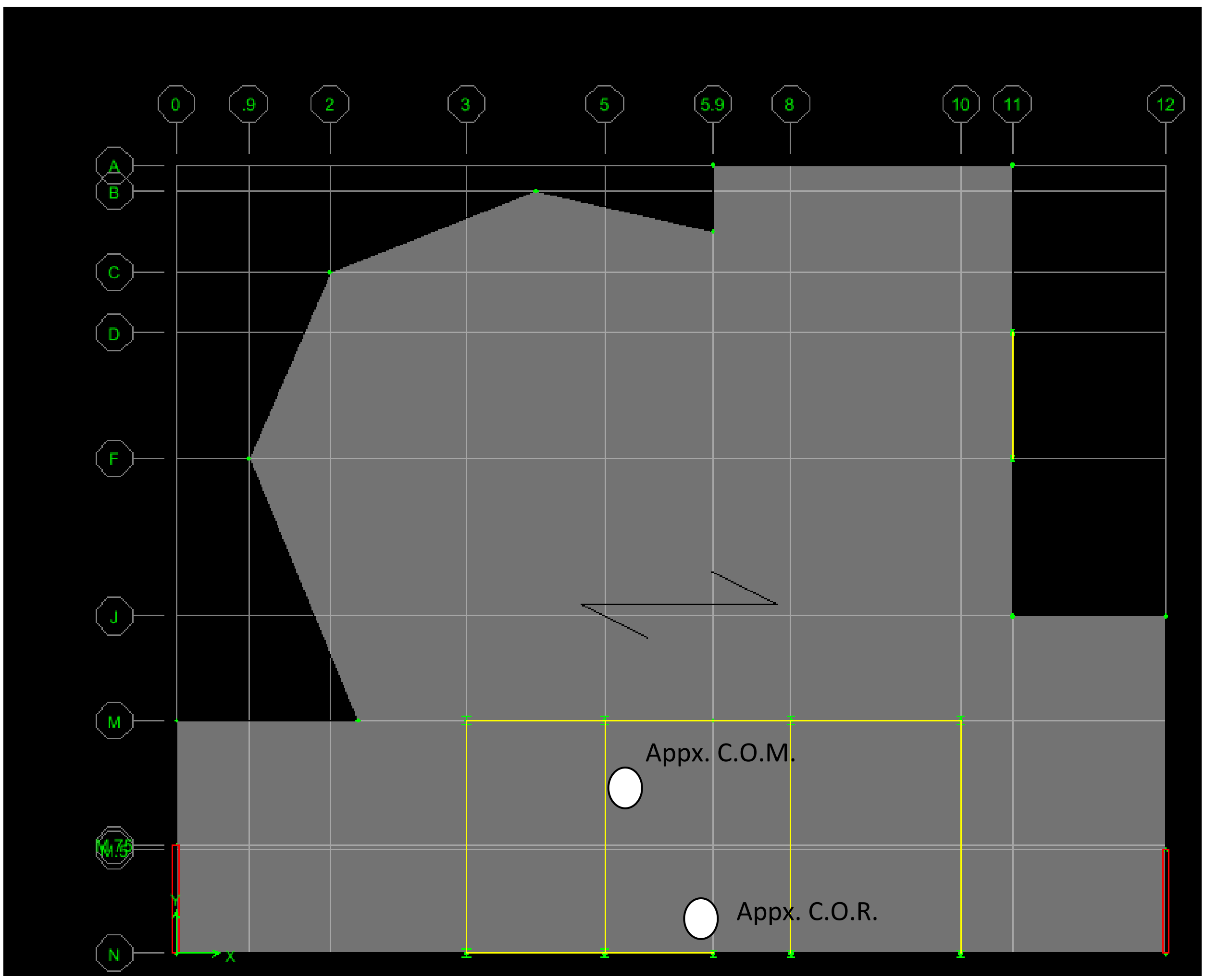

*Fig. 13 Seismic Overturning Moment and Base Shear*

29581 ft-k

## LATERAL SYSTEMS MODEL

As mentioned previously, many errors occurred in the making of a lateral systems model in ETABS, however a visual representation is always helpful, and is given below.

*Fig. 14 Plan view of building*




As you can see, by the calculations derived, there is a very large torsional effect in both the N-S and E-W directions caused by the offset between the centers of mass and rigidity.

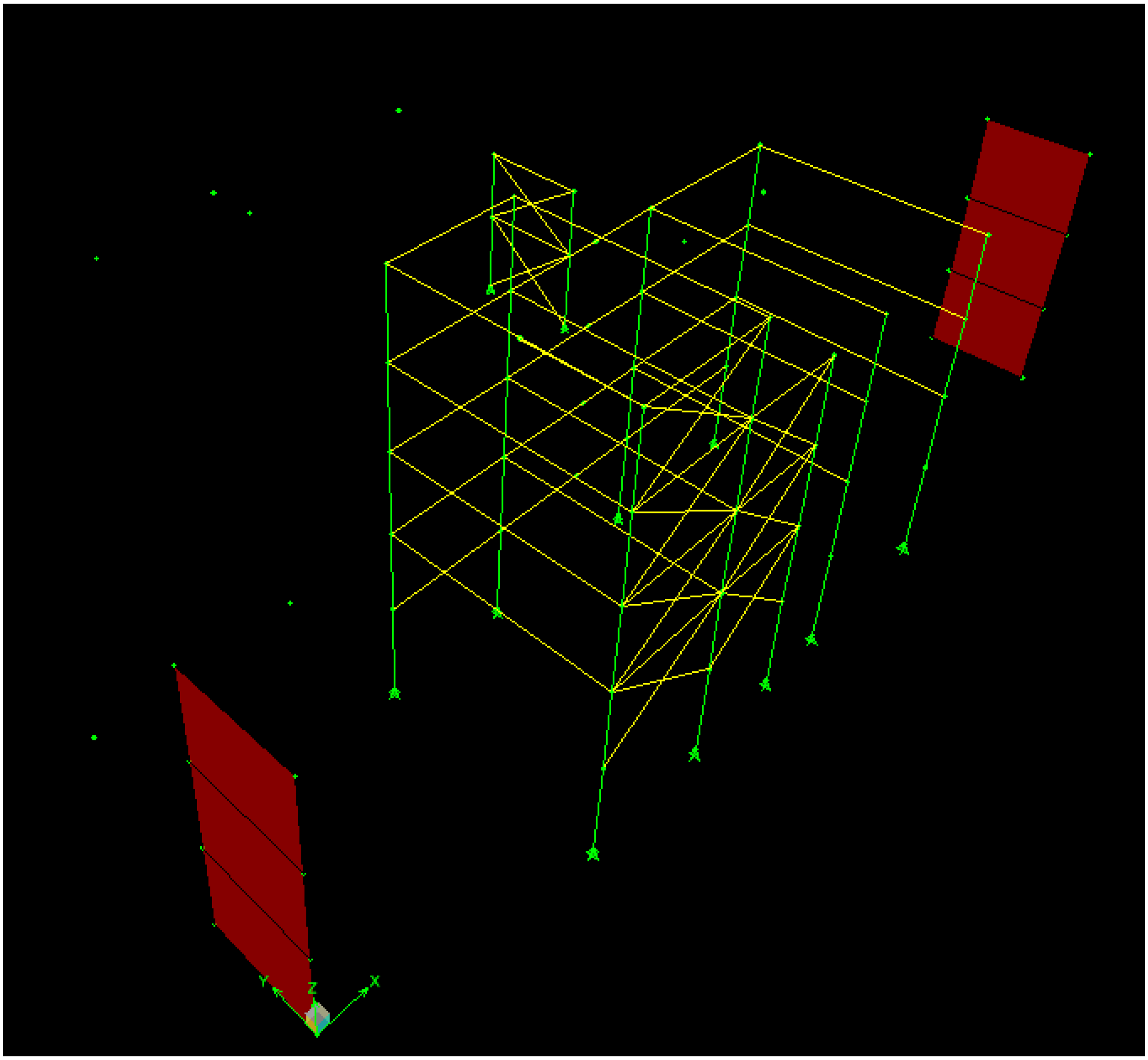



*Fig. 15 3D view of Lateral System*

| | Story | Item | Load | Point | X | Y | Z | DriftX | DriftY |
|---|---|---|---|---|---|---|---|---|---|
| ▶ | PENTHOUSE | Max Drift X | DEAD | 14 | 471.875 | 378.000 | 576.000 | 0.001313 | |
| | PENTHOUSE | Max Drift Y | DEAD | 38 | 471.875 | 0.000 | 576.000 | | 0.005354 |
| | PENTHOUSE | Max Drift X | LIVE | 39 | 695.875 | 0.000 | 576.000 | 0.000000 | |
| | PENTHOUSE | Max Drift Y | LIVE | 39 | 695.875 | 0.000 | 576.000 | | 0.000000 |
| | LEVEL 3 | Max Drift X | DEAD | 61 | 0.000 | 0.000 | 432.000 | 73028641.96 | |
| | LEVEL 3 | Max Drift Y | DEAD | 65 | 1606.875 | 168.000 | 432.000 | | 35935494.71 |
| | LEVEL 3 | Max Drift X | LIVE | 65 | 1606.875 | 168.000 | 432.000 | 0.000000 | |
| | LEVEL 3 | Max Drift Y | LIVE | 65 | 1606.875 | 168.000 | 432.000 | | 0.000000 |
| | LEVEL 2 | Max Drift X | DEAD | 63 | 0.000 | 176.000 | 288.000 | 73028641.97 | |
| | LEVEL 2 | Max Drift Y | DEAD | 65 | 1606.875 | 168.000 | 288.000 | | 35935494.71 |
| | LEVEL 2 | Max Drift X | LIVE | 65 | 1606.875 | 168.000 | 288.000 | 0.000000 | |
| | LEVEL 2 | Max Drift Y | LIVE | 65 | 1606.875 | 168.000 | 288.000 | | 0.000000 |
| | LEVEL 1 | Max Drift X | DEAD | 59 | 1357.875 | 1008.000 | 144.000 | 476606876.4 | |
| | LEVEL 1 | Max Drift Y | DEAD | 64 | 1606.875 | 0.000 | 144.000 | | 35935494.71 |
| | LEVEL 1 | Max Drift X | LIVE | 65 | 1606.875 | 168.000 | 144.000 | 0.000000 | |
| | LEVEL 1 | Max Drift Y | LIVE | 65 | 1606.875 | 168.000 | 144.000 | | 0.000000 |
| | GROUND | Max Drift X | DEAD | 59 | 1357.875 | 1008.000 | 0.000 | 408520180.1 | |
| | GROUND | Max Drift Y | DEAD | 38 | 471.875 | 0.000 | 0.000 | | 0.409137 |
| | GROUND | Max Drift X | LIVE | 60 | 1357.875 | 804.000 | 0.000 | 0.000000 | |
| | GROUND | Max Drift Y | LIVE | 60 | 1357.875 | 804.000 | 0.000 | | 0.000000 |

As you can clearly see from the nonsensical data displayed above for story drift results, there is clearly an error in the model. I am unsure as to whether it is a joint fixity issue, or perhaps there are interactions occurring that should not be. Either way, story drift data is simply unavailable until next semester's project begins. However, since the building is already completed, and has not failed yet, it is safe to assume it has met that criteria. Below are tables outlining the distribution of forces along the N-S and E-W directions respectively. These numbers were derived by hand calculations, using individual rigidity of the lateral elements to analyze how the load would travel across the system. Frame 8, one of the moment frames, appears to be under an immense torsional shear, due to its position near both the center of mass and the center of rigidity. This will most likely be the cause of error in the model as well.

| N-S Seismic Distribution | | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| Element | Rigidity | Load | e | d | $kd^2$ | Direct Shear | Torsional Shear | Total Shear |
| 1 | 137 | 160 | 17 | 62 | 526628 | 53.73 | 43.87 | 97.60 |
| 2 | 100 | 160 | 17 | 76 | 577600 | 39.22 | 35.79 | 75.01 |
| 5 | 67 | 160 | 17 | 56 | 210112 | 26.27 | 48.57 | 74.85 |
| 7 | 25 | 160 | 17 | 19.4 | 9409 | 9.80 | 140.21 | 150.01 |
| 8 | 25 | 160 | 17 | 1 | 25 | 9.80 | 2720.00 | 2729.80 |
| 9 | 28 | 160 | 17 | 24.5 | 16807 | 10.98 | 111.02 | 122.00 |
| 10 | 26 | 160 | 17 | 138 | 495144 | 10.20 | 19.71 | 29.91 |
| SUM | 408 | | | | | | | |

| E-W Seismic Distribution | | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| Element | Rigidity | Load | e | d | $kd^2$ | Direct Shear | Torsional Shear | Total Shear |
| 3 | 68 | 160 | 8 | 5 | 1700 | 57.87 | 256.00 | 313.87 |
| 4 | 93 | 160 | 8 | 5 | 2325 | 79.15 | 256.00 | 335.15 |
| 6 | 27 | 160 | 8 | 31.5 | 26790.75 | 22.98 | 40.63 | 63.61 |
| SUM | 188 | | | | | | | |

## SPOT CHECKS

Basic hand calculations were done on several members of the braced frames to show that they were adequately designed to carry the load. The bottom tension chord, and bottom left column were chosen out of frames 3 and 5 and both were well within acceptable ranges of load to capacity. The internal forces of each of the members analyzed was derived from my rigidity calculations, and adapted accordingly. These calculations can be found in Appendix C.

## CONCLUSION

In conclusion, I was unable to learn as much as was necessary for this report, due to unknown errors in the 3d model. The hand calculations showed that most of the N-S resistance was given by the CMU shear walls at each corner of the building, and that the E-W lateral motion was mainly being taken care of by the two braced frames at the plan south side of the building.

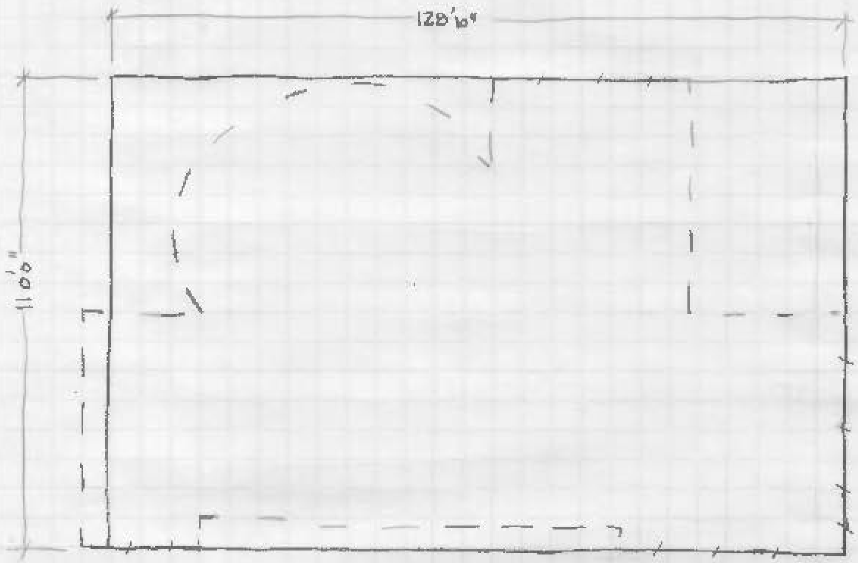
A strong understanding of each individual element was gained, however, through the hand calculations and spot checks. These exercises helped to idealize how the load is distributed throughout each element in the structure, and where potential problems may occur.

While it is simple enough to assume the numbers are correct and move on, a proposal involving adjusting the lateral system will most likely be pursued in the future.

# APPENDIX A

HANDLEY McDONALD TECH 3 WIND LOAD CALC

SIMPLIFICATION OF BUILDING SHAPE:




* NOT DRAWN TO SCALE

ACTUAL SHAPE
IDEALIZED SHAPE
OVERLAP

ALL BUILDING HEIGHT TAKEN AS MAXIMUM EXPOSED HEIGHT OF 72'8"

AMPAD

HANDLEY MCDONALD | TECH 3 | WIND LOAD CALC

MAIN WIND FORCE RESISTING SYSTEM (MWRFS)

RISK CATEGORY: III (SIGNIFICANT RISK)
BASIC WIND SPEED (V): 120 MPH, 3-SECOND GUST
(DESIGN USED 90 MPH, * SO.11)
$K_d = .85$ TABLE 26.6-1
EXPOSURE CATEGORY: B
$K_{zt} = 1.0$ GENERALLY HOMOGENEOUS TOPOGRAPHY
$GC_{pi} = \pm .55$
ENCLOSURE PARTIALLY ENCLOSED
$I_{wind} = 1.15$
'$K_z$' = INTERPOLATION 72' 8" HEIGHT

$K_z \rightarrow$ VARIES BY HEIGHT BASED ON TABLE 27.3-1
SEE SPREADSHEET

$$G = .925\left(\frac{1+1.7\,g_Q I_{\bar z} Q}{1+1.7\,g_v I_{\bar z}}\right) \quad ; \; \bar z = .6h = .6(72.67) = 43.6'$$

$c = .3$

$g_Q = g_v = 3.4$

$$I_{\bar z} = c\left(\frac{33}{\bar z}\right)^{1/6} = .3\left(\frac{33}{43.6}\right)^{1/6} = .287$$

$$Q_{N\text{-}S} = \sqrt{\frac{1}{1+.63\left(\frac{B_{N\text{-}S}+h}{L_{\bar z}}\right)^{.63}}} \qquad Q_{E\text{-}W} = \sqrt{\frac{1}{1+.63\left(\frac{B_{E\text{-}W}+h}{L_{\bar z}}\right)^{.63}}}$$

$$L_{\bar z} = \ell\left(\frac{\bar z}{33}\right)^{\bar\epsilon}$$

$$L_{\bar z} = 320\left(\frac{30}{33}\right)^{1/3}$$

$$L_{\bar z} = 309.99$$

$$Q_{N\text{-}S} = \sqrt{\frac{1}{1+.63\left(\frac{128.83+72.67}{309.99}\right)}} \qquad Q_{E\text{-}W} = \sqrt{\frac{1}{1+.63\left(\frac{110+72.67}{309.99}\right)}}$$

$Q_{N\text{-}S} = .84$ $\qquad$ $Q_{E\text{-}W} = .85$

$$G_{N\text{-}S} = .925\left(\frac{1+1.7(3.4)(.287)(.84)}{1+1.7(3.4)(.287)}\right) \qquad G_{E\text{-}W} = .925\left(\frac{1+1.7(3.4)(.287)(.85)}{1+1.7(3.4)(.287)}\right)$$

$G_{N\text{-}S} = .83$ $\qquad$ $G_{E\text{-}W} = .84$

$q_z \rightarrow$ VARIES BY $K_z$

$$q_z = .00256\,K_z K_{zt} K_d V^2 I \qquad \text{EQ } 27.3\text{-}1$$

HANDLEY McDONALD | TECH 3 | WIND LOAD CALC

$C_p$ : By FIG 27.4-1

WINDWARD

N-S, $C_p = .8$, ALL CASES

E-W, $C_p = .8$, ALL CASES

LEEWARD

N-S

$\frac{L}{B} = {}^{110}/_{128.83} = .85$

$0 \le L/B \le 1.0$, $C_p = -.5$

E-W

$\frac{L}{B} = \frac{128.83}{110} = 1.17$

INTERPOLATION:

$C_p = -.466$

SIDE

N-S, $C_p = -.7$ ALL CASES

E-W, $C_p = -.7$ ALL CASES

ROOF

| | | |
|---|---|---|
| N-S | $0 - h/2$ | $C_p = -.9, -.18$ |
| | $h/2 - h$ | $C_p = -.9, -.18$ |
| | $h - 2h$ | $C_p = -.5, -.18$ |
| E-W | $0 - h/2$ | $C_p = -.9, -.18$ |
| | $h/2 - h$ | $C_p = -.9, -.18$ |
| | $h - 2h$ | $C_p = -.5, -.18$ |
| | $> 2h$ | $C_p = -.3, -.18$ |

AMPAD

# APPENDIX B

| Seismic Forces | | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| Floor | Area (sq ft) | Weight (k) | Height (ft) | wxhx^k | Cvx | Story Force (k) | Story Shear (k) | Overturning Moment (ft k) |
| 0 | 12971.42 | 1894.48 | 0.00 | 0.00 | 0.00 | 0.00 | 412.20 | 0.00 |
| 1 | 10619.29 | 1550.95 | 14.00 | 137727.04 | 0.39 | 160.02 | 412.20 | 5770.85 |
| 2 | 4058.15 | 592.69 | 28.00 | 44051.86 | 0.12 | 51.18 | 252.19 | 7061.21 |
| 3 | 7540.06 | 1101.23 | 42.00 | 91787.85 | 0.26 | 106.64 | 201.00 | 8442.19 |
| 4 | 4058.15 | 592.69 | 56.00 | 44051.86 | 0.12 | 51.18 | 94.36 | 5284.22 |
| 5 | 7540.06 | 513.48 | 70.00 | 37164.66 | 0.10 | 43.18 | 43.18 | 3022.57 |
| | | | | | | | Total Base Shear = | 412.20 |
| | | | | | | | Total Moment = | 29581.05 |

HANDLEY MCDONALD | TECH 3 | SEISMIC LOADS

LOCATION: CHARLOTTESVILLE, VA

$S_s = 16\%$ $\quad S_1 = 5.1\%$ (TABLES 22-1, 22-2)

SITE CLASS D ASSUMED (TABLE 20.3-1)

$S_{MS} = F_a S_s = 1.6(.16) = .256$

$S_{M1} = F_v S_1 = 2.4(.051) = .1224$ (TABLES 11.4-1, 11.4-2)

$S_{DS} = 2/3\, S_{MS} = 2/3(.256) = .171$

$S_{D1} = 2/3\, S_{M1} = 2/3(.1224) = .082$

OCCUPANCY CATEGORY III, $I = 1.25$ (TABLE 11.5-1)

SEISMIC DESIGN CATEGORY → B (TABLE 11.6-1)

→ B (TABLE 11.6-2)

$R = 3.25$ (TABLE 12.2-1) FOR CONCENTRIC STEEL BRACED FRAMES

$T = C_t h_n^x$ $.8$ ; $C_t = .028$ (TABLE 12.8-2)

$T = .028(72.67)$ $\quad x = .8$

$T = .863$

$C_s = \dfrac{S_{DS}}{R/I} = \dfrac{.171}{3.25/1.25} = .066$

$T_L = 8s$

$T \le T_L, \quad C_s < \dfrac{S_{D1}}{(R/I)T}$

$C_s < \dfrac{.171}{(3.25/1.25)(.863)} = .07$ OK

$C_s \geq .01$ OK

ESTIMATE BUILDING WEIGHT:

$W_{floor} \approx 45$ psf

$W_{BM} \approx$ 10x10 SECTION, 2 W18x40 BMS

$W_{BM} \approx \dfrac{40(10)(2)}{10} = 80$ psf

$w_{col} \approx \dfrac{14'(79(6)+58(2)+40+72+46+ [illegible] + 106)}{178.83'(106.75')}$ *USING COL SCHED

$W_{col} \approx 2$ psf

$W_{TOT} \approx (45+80+2)1.15 = 146.05$ psf ALL FLOORS

HANDLEY MCDONALD TECH 3 SEISMIC LOADS

$W_{roof}$ = 45psf + 23.1psf = 68.1psf

T = .863, K FOUND BY LINEAR INTERPOLATION
K = 1.183

$C_{vx} = \frac{w_x h_x^k}{\sum w_i h_i^k}$ → VALUES TABULATED IN SPREADSHEET

# APPENDIX C

| Internal Forces: Frame 5 | | | | | |
|---|---|---|---|---|---|
| Member | u | L | A | E | Disp. |
| AC | 1.64 | 168 | 11.7 | 29000 | 0.001331721 |
| BD | 0.814 | 168 | 11.7 | 29000 | 0.000328076 |
| AD | 1.29 | 264.3 | 2.58 | 29000 | 0.005878397 |
| BC | 0 | 264.3 | 2.58 | 29000 | 0 |
| CD | 1 | 204 | 13 | 29000 | 0.000541114 |
| ED | 1.29 | 264.3 | 2.58 | 29000 | 0.005878397 |
| CF | 0 | 264.3 | 2.58 | 29000 | 0 |
| CE | 0.814 | 168 | 11.7 | 29000 | 0.000328076 |
| DF | 0 | 168 | 11.7 | 29000 | 0 |
| EF | 1 | 204 | 11.8 | 29000 | 0.000596143 |
| SUM | 0.014882 | | | K | 67.19562154 |

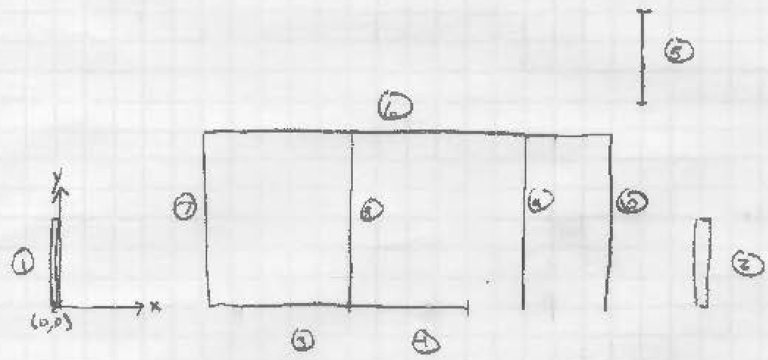
| Internal Forces: Frame 4 | | | | | |
|---|---|---|---|---|---|
| Member | u | L | A | E | Disp. |
| AC | 2.86 | 168 | 44.7 | 29000 | 0.00106 |
| BD | 1.9 | 168 | 17 | 29000 | 0.00123 |
| AD | 0 | 243.3 | 11.6 | 29000 | 0 |
| BC | 1.38 | 243.3 | 11.6 | 29000 | 0.001377 |
| CD | 1 | 176 | 9.13 | 29000 | 0.000665 |
| CF | 0 | 243.6 | 11.6 | 29000 | 0 |
| ED | 1.38 | 243.6 | 11.6 | 29000 | 0.001379 |
| EC | 1.86 | 168 | 44.7 | 29000 | 0.000448 |
| FD | 0.95 | 168 | 17 | 29000 | 0.000308 |
| EF | 1 | 176 | 9.13 | 29000 | 0.000665 |
| EH | 0 | 243.6 | 5.24 | 29000 | 0 |
| FG | 1.38 | 243.6 | 5.24 | 29000 | 0.003053 |
| EG | 0.86 | 168 | 44.7 | 29000 | 9.59E-05 |
| FH | 0 | 168 | 17 | 29000 | 0 |
| GH | 1 | 176 | 13.5 | 29000 | 0.00045 |
| SUM | 0.01073 | | | K | 93.19423 |

| Internal Forces: Frame 3 | | | | | |
|---|---|---|---|---|---|
| Member | u | L | A | E | Disp. |
| AC | 3 | 168 | 44.7 | 29000 | 0.001166397 |
| BD | 2.25 | 168 | 44.7 | 29000 | 0.000656098 |
| AD | 0 | 280 | 11.6 | 29000 | 0 |
| BC | 1.25 | 280 | 11.6 | 29000 | 0.001300535 |
| CD | 1 | 224 | 10.6 | 29000 | 0.000728692 |
| CF | 0 | 280 | 6.17 | 29000 | 0 |
| ED | 1.25 | 280 | 6.17 | 29000 | 0.00244509 |
| CE | 2.25 | 168 | 44.7 | 29000 | 0.000656098 |
| FD | 1.5 | 168 | 44.7 | 29000 | 0.000291599 |
| EF | 1 | 224 | 10.6 | 29000 | 0.000728692 |
| EH | 0 | 280 | 6.17 | 29000 | 0 |
| FG | 1.25 | 280 | 6.17 | 29000 | 0.00244509 |
| EG | 1.5 | 168 | 44.7 | 29000 | 0.000291599 |
| FH | 0.75 | 168 | 44.7 | 29000 | 7.28998E-05 |
| GH | 1 | 224 | 13.5 | 29000 | 0.000572158 |
| GJ | 0 | 280 | 5.24 | 29000 | 0 |
| IH | 1.25 | 280 | 5.24 | 29000 | 0.002879047 |
| GI | 0.75 | 168 | 21.1 | 29000 | 0.000154437 |
| HJ | 0 | 168 | 21.1 | 29000 | 0 |
| IJ | 1 | 224 | 24.7 | 29000 | 0.000312718 |
| SUM | 0.014701 | | | K | 68.02188125 |

HANDLEY MCDONALD | TECH 3 | LAT DISTRIBUTION

LATERAL SYSTEM:

1,2 → CMU SHEAR WALLS, DIFFERING THK
3-5 → CROSS BRACED FRAMES
6 → MOMENT FRAME




SHEAR WALLS

$$R = \frac{Et}{4(h/L)^3 + 3(h/L)}$$

① - 12" CMU

$$E = 33w^{1.5}\sqrt{f'_c}$$
$$E = 33(89)^{1.5}\sqrt{1800}$$
$$E = 1175 \text{ ksi}$$

* 89 PCF BASED ON PROPIETARY DATA, ACTUAL WEIGHT NOT GIVEN

$$R_1 = \frac{1175(12")}{4(504/176)^3 + 3(504/176)} = 137 \text{ k/in}$$

② - 10" CMU

$$R_2 = \frac{1175(10)}{4(504/168)^3 + 3(504/168)} = 100 \text{ k/in}$$

HANDLEY MCDONALD | TECH 3 | LAT DISTRIBUTION

3 - $K = 68\ ^{k}/_{in}$
4 - $K = 93\ ^{k}/_{in}$
5 - $K = 67\ ^{k}/_{in}$
6 - $K = 27\ ^{k}/_{in}$ ??
7 - $K = 25\ ^{k}/_{in}$
8 - $K = 25\ ^{k}/_{in}$
9 - $K = 28\ ^{k}/_{in}$
10 - $K = 26\ ^{k}/_{in}$

- COLLECTED FROM COMPUTER MODEL
- MORE ANALYSIS REQUIRED
- CLEARLY INCORRECT

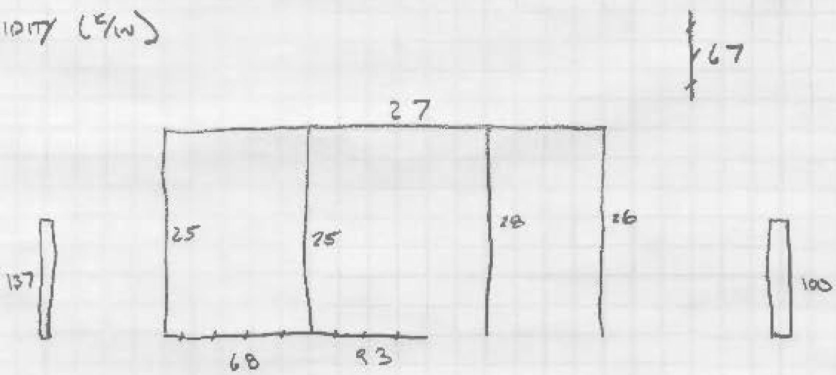
CENTER OF RIGIDITY

$$\bar{x}_r = \frac{137(0) + 100(138.125) + 67(110) + 25(29.5) + 25(48) + 28(92) + 26(110)}{137 + 100 + 67 + 25 + 25 + 28 + 26}$$

$\bar{x}_r = 70'$ FROM PLAN SW

$$\bar{y}_r = \frac{68(0) + 93(0) + 27(31.5)}{68 + 93 + 27} = 5'$$ ?? FROM PLAN SW

RIGIDITY (K/IN)



HANDLEY McDONALD | TECH 3 | LAT DISTRIBUTION

CENTER OF MASS

WALL WEIGHTS, AS NOTED IN STRUCTURAL PLANS

① - $w = 89\,pcf\,(1')(14.67')(14'(3)) = \underline{54.8^k}$

② - $w = 89\,pcf\,(10/12)(14')(14'(3)) = \underline{43.6^k}$

③ - $w = 42\,psf(2)(23') + 23\,psf\,(4)(23) + 19\,psf\,(2)(23) + 16(18.67)(2) + 18(18.67) + 24(18.67) = \underline{6.3^k}$

④ - $w = 42(4)(20) + 19(2)(20) + 16(14.67)(2) + 18(14.67) = \underline{4.9^k}$

⑤ - $w = 9.42(4)(22) + 21(17) + 18(17) = \underline{1.5^k}$

⑥ - $w = 18(66.8') + 16(2)(48') + 21(18.67) + 2(16)(18.67) + 18(25.16') + 16(23') = \underline{4.6^k}$

⑦, ⑧ - $w = 2(24)(30.25) + 18(30.25) + 30(30.25) = \underline{2.9^k}$

⑨, ⑩ - $w = W_{7,8} - 30(30.25) = \underline{2^k}$

$$\bar{y}_m = \frac{4.6(74.4) + \ldots}{4.6 + 6.3 + 4.9} = \underline{22'}$$

$$\bar{x}_m = \frac{1.5(110) + 2.9(41') + 2.9(60') + 2(85') + 2(108') + 43.6(138)}{1.5 + 2.9 + 2.9 + 2 + 2 + 43.6 + 54.8} = 62'$$

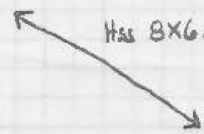
* HEAVY TORSION CREATED IN BOTH DIRECTIONS

C.O.M. = (62, 22)  C.O.R. = (70, 5)

# APPENDIX D

Handley McDonald | Tech 3 | Spot Check

Frame 3: Bottom Tension Chord



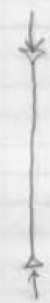

$V_{BT} = 313^k$

Based on previous member forces when a $1^k$ force was applied:

$V = 1(313)$

$T_u = 313(1.25) = 391^k \leq 480^k$ ok

Bottom Column:




$P_u = 3(313) = 939$

$\phi P_n = 1640^k$ ok

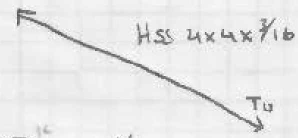
$F_e = \dfrac{\pi^2 E}{\left(\frac{KL}{r}\right)^2} = \dfrac{\pi^2(29000)}{94^2} = 32.4 \text{ ksi}$

$F_{cr} = .877(32.4) = 28.4 \text{ ksi}$

$\phi P_n = .9(28.4)(44.7) = 1170^k$ ok

Handley McDonald | Tech 3 | Spot Check

FRAME 5: BOTTOM TENSION CHORD




$V_{TOT} = 74.85$

$T_U = 74.85(1.39) = 97^k \leq 107^k$ OK

BOTTOM COLUMN


$P_U = 1.64(74.85) = 123^k$

$\phi P_n = 321^k$ OK

$$F_e = \frac{\pi^2 E}{\left(\frac{KL}{r}\right)^2} = \frac{\pi^2(29000)}{(107)^2} = 25 \text{ ksi}$$

$F_{cr} = .877(25) = 22 \text{ ksi}$

$\phi P_n = .9(22)(11.7) = 231^k$ OK

# APPENDIX E