MONONGALIA GENERAL HOSPITAL

1200 J.D. ANDERSON DRIVE, MORGANTOWN, WEST VIRGINIA

LATERAL SYSTEM ANALYSIS AND CONFIRMATION DESIGN REPORT

TECH THREE

THE PENNSYLVANIA STATE UNIVERSITY
DEPARTMENT OF ARCHITECTURAL ENGINEERING
SENIOR THESIS 2008-2009

SUBMITTED: NOVEMBER 21, 2008

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Executive Summary

Purpose

The Lateral System Analysis and Confirmation Design report (Tech 3) discusses the lateral system of the Monongalia General Hospital. The columns and shear walls have been analyzed against the lateral forces (wind and seismic) found in Tech 1. The Hospital was modeled using ETABS based on the structural drawings provided by the Atlantic Engineering Services dated December 2005.

Building Description

The Monongalia General Hospital is a 405,994 square feet hospital located in Morgantown, West Virginia. The building project includes a 280,000 square feet addition as well as a 60,000 square feet renovation to the existing structure. The building envelope is a brick façade tied to structural concrete walls with openings for punch windows and curtain wall systems. Aluminum curtain wall systems can be seen all around the Hospital, oriented around lobbies and other major openings on plan. The system consists of insulated tempered spandrel glass framed by aluminum mullions which is tied into the concrete structural system. The main structural system of the Hospital consists of two-way flat slab supported by columns that follow a typical grid and edge beams located in the perimeter of each floor. The loads carried by the columns are transferred to the foundations. The lateral loads are resisted by twelve shear walls of varying height and width located in various portions of the building.

Lateral System Analysis

The Monongalia General Hospital's lateral load resisting system was found to be a shear wall system, a different finding from the assumptions from earlier technical reports. The shear wall was analyzed against lateral loads under two main conditions: a shear wall by itself and as a system. Under these conditions drift of the shear walls and the building as whole, and the strength of the shear wall was studied. From the analyses conducted, the lateral force resisting system has proved to be adequate to resist the lateral forces.

Monongalia General Hospital

1200 J.D. Anderson Drive
Morgantown, WV

Lateral System Analysis and Confirmation Design Report

Introduction

The Lateral System Analysis and Confirmation Design report (Tech 3) discusses the lateral system of the Monongalia General Hospital. The columns and shear walls have been analyzed against the lateral forces (wind and seismic) found in Tech 1. The Hospital was modeled using ETABS based on the structural drawings provided by the Atlantic Engineering Services dated December 2005.

The Monongalia General Hospital

The Monongalia General Hospital is located on 1200 J.D. Anderson Drive, West Virginia (Photograph 2 for aerial view, Photograph 3 for façade). The current project the Hospital is going through is a 340,000 square foot expansion and renovation named the Hazel Ruby McQuain Tower, this new addition will provide more various facilities and departments to the Hospital. The construction started on June of 2006 and is scheduled to be completed on May of 2009 with a design-build contract with a guaranteed maximum price set at an estimated $69,000,000 by the Turner Construction Company. The Tower has been designed by Freeman White, Inc. from North Carolina and the structure designed by Atlantic Engineering Services from Pittsburgh. (See Appendix A for Project Team Directory)

The Monongalia General Hospital's plan can be divided into four different quads, A, B, C, and D (Figure 1). The first floor of the Monongalia General Hospital occupies 92,086 square feet and houses a boiler/chiller room, electrical rooms, doctors' offices, labs, nurse stations, storage spaces, and a dining space equipped with a food services kitchen. The second floor follows a similar layout but provides more space for examination rooms as well as a gift shop and café on the southern face of Quad A. The third floor mainly consists of patient rooms with the central part of the plan dedicated to operation rooms. The third floor has a reduced square footage compared to those of the floors below with an area of 80,882 square feet; the western section of Quad D does not continue up to the third floor as patient room spaces but provides housing for two air handling units. The fourth floor sees an even less square footage on plan at 53,833 square feet, with the western section of Quad D no longer existing at this elevation. This floor only houses private patient rooms, each equipped with a private toilet and shower. The square footage of the fourth floor continues up to the fifth, housing more private patient rooms as well as a Labor, Delivery, Recovery, and Postpartum (LDRP) rooms in Quad B and C. The sixth floor sees nearly a fifty percent reduction in square footage from the fifth floor with only Quads B and C serving rooms for private patients. The rooftop at Quad A is located at this elevation and houses five air handling units. Acoustic ceiling systems are utilized on each floor to provide acoustic insulation. The rooftop of the Monongalia General Hospital is used primarily to house mechanical equipment. Two different types of roof systems are utilized: an adhered roof system and a ballasted roof system. The ballasted roof system is only present on the rooftop of Quad A and all other roofs utilize the adhered roof system. (Refer to Figure 2 for building cross section)

The exterior façade of the Monongalia General Hospital is a brick façade tied to 8” structural concrete walls with openings for punch windows and curtain wall systems. Windows are typically aluminum punch window units and located where there are offices and patient rooms, located on the third floor and up. Aluminum curtain wall systems can be seen all around the Hospital, oriented around lobbies and other major openings on plan (Photograph 1 and 3). The system consists of insulated tempered spandrel glass framed by aluminum mullions which is tied into the concrete structural system. Two inch rigid insulation is provided all around the building for insulation.

Structural System

Introduction

The primary structure of the Monongalia General Hospital is reinforced concrete with several composite floor systems present in parts of the building where appropriate (i.e. canopy/wall junctions, canopy fascia, etc.). The concrete used for the Hospital ranges from 3000 pounds per square inch (psi) to 5000 psi depending on its use. All concrete, as specified by ASTM C150; is normal weight concrete with a minimum weight of 144 pounds per cubic foot, and the reinforcement used are all ASTM A615 – Grade 60 steel reinforcement bars.

Foundation and Columns

Concrete foundations are placed below every column located at a minimum depth of 3’-6” below grade and utilize 3000 psi cast in place concrete. The columns that transfer the loads to these foundations are all 24 inches by 24 inches utilizing 5000 psi cast in place concrete. A total of 100 columns are present in the structure ranging in height from 11’-6” (supports one floor) to the full height of the building 58’-5”. There are six columns in the structure in which the column’s material changes from concrete to steel. These columns support the canopy in Quad A as well as used as corner columns for the stair towers.

Slabs

The slab on grades are 5” thick normal weight concrete and the slabs used in floors above are two-way flat plate slabs that utilizes 4000 psi normal weight concrete and are used as the primary floor system with the exception of a few in Quad C where an emergency energy plant is present: a composite concrete-steel floor system is used. The two way slab system is 8 inches thick and transfers its load to the columns and concrete edge beams present in the perimeter of each floor.

Beams

The beams are all variable in size although the dominant cross section is an 18 inch by 24 inch beam usually spanning 27' from column to column. Like the columns, the concrete used for the beams are 5000 psi normal weight concrete framed in by the two way slabs. As mentioned earlier, beams in this Hospital are all edge beams with an exception around openings in plan for elevator shafts, stairs, as well as for the energy plant located in the northern part of Quad C.

Shear Walls

There are twelve lateral force resisting shear walls present in the Hospital (Figure 3). All of these are variable sizes ranging in height and width, the most representative shear wall being a 52'-9-1/8" x 70' wall with two sets of eight #5 bars used at each floor level.

Building Design Loads

Gravity Loads

For the structural analysis, gravity loads were determined as per ASCE 7-05, AISC 13th Edition, IBC 2006, and other relevant publications. The construction documents were also referenced to provide a better perception of code compliant loads. On the following page is a table listing the loads by type and material.

Floor Loads			
Type	*Material/Occupancy*	*Load*	*Reference*
Dead Load	Normal Weight Concrete	145 PCF	Drawing G1-2
	Steel	Per shape	AISC 13th Edition
	Brick Masonry	40 PSF	MSJC
	Partitions	20 PSF	Drawing G1-2
	Superimposed	10 PSF	*
Live Load	Public Areas	100 PSF	IBC 2006
	Lobbies	100 PSF	IBC 2006
	Corridors (1st Floor)	100 PSF	IBC 2006
	Corridors (Above 1F)	80 PSF	IBC 2006
	Operation Rooms	60 PSF	Drawing G1-2
	Patient Rooms	40 PSF	Drawing G1-2
	Mechanical	150 PSF	Drawing G1-2
	Stairs	100 PSF	Drawing G1-2
Roof Loads			
Dead Load	Normal Weight Concrete	145 PCF	Drawing G1-2
	Steel	Per shape	AISC 13th Edition
	Brick Masonry	40 PSF	MSJC
	Superimposed	10 PSF	**
Live Load	Roof Live Load	20 PSF	Drawing G1-2
	Mechanical	150 PSF	Drawing G1-2
Snow Load	Flat Roof Load	24 PSF	ASCE 7-08
Rain Load	Rain Load	21 PSF	ASCE 7-08

*Includes electrical and telecommunications wiring, ductwork, drop ceiling
**Includes ballasting, waterproofing, insulation

Snow drift loads were to be considered as a loading condition as per ASCE 7-08 however this type of loading was determined to be beyond the scope of this report and therefore neglected and will be discussed in future reports.

Lateral Loads

Lateral loads were calculated as per ASCE 7-08. Although the building is only six stories high, these loads must be considered as a design issue. The wind loads were calculated by referencing parameters from ASCE 7-08, IBC 2006, and the United States Geological Service under the analytical method:

- Basic Wind Speed 90 mph
- Direction Factor 0.85
- Occupancy Category IV
- Importance Factor 1.15

- Exposure Category	B
- Topographic Factor	1
- Gust Effect Factor	0.85
- Fundamental Frequency	6.43 (Rigid Structure)
- Peak Factor	3.4
- Enclosure	Enclosed

The above listed parameters were used to calculate the wind load in pounds per square feet for the different surfaces of the Hospital:

Wind Loads				
	North to South Wind Pressure		***East to West Wind Pressure***	
	Height (ft)	*Pressure (PSF)*	*Height*	*Pressure (PSF)*
Windward	0-15	7.9	0-15	7.9
	20	8.5	20	8.5
	25	8.9	25	8.9
	30	9.6	30	9.6
	40	10.5	40	10.5
	50	11.2	50	11.2
	60	11.3	60	11.3
	70	11.3	70	11.3
Leeward	All	-8.3	All	-7.9
	Base Shear (kips)	**362.3**	*Base Shear*	**362.3**
	Overturning Moment (k-ft)	**47875.4**	*Overturning Moment (k-ft)*	**47875.4**
Roof	Windward to 90°	-12.7	Windward to 90°	-12.7
	90°-180°	-7.0	90°-180°	-7.0
	180° to Leeward	-4.2	180° to Leeward	-4.2

(Refer to Figure 4and 5 for Wind Loading Diagram)

The seismic loads were also calculated in a similar fashion, by referencing the aforementioned publications, the following parameters were used:

- Occupancy Category	IV
- Importance Factor	1.5
- Seismic Category	A
- Site Class	C
- Spectral Acceleration, Short Period	0.133
- Spectral Acceleration, 1 Second	0.052
- Site Coefficient, F_a	1.2
- Site Coefficient, F_v	1.7
- R-Factor	5.0

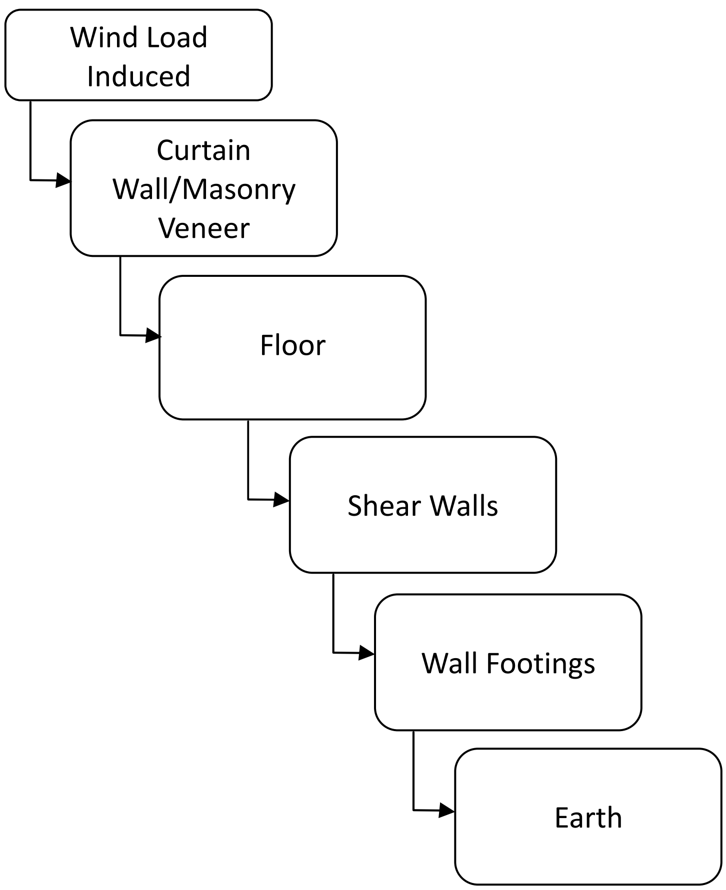
These parameters were used under the equivalent lateral force procedure to calculate the base shear of the building as well as the force acting at each floor level:

Seismic Loads		
Floor	*Height (ft)*	*F_x (kips)*
1	0	314.83
2	12	340.39
3	24	389.23
4	35.5	278.90
5	47	367.52
6	58.5	455.63
Roof	70	314.83
Seismic Base Shear (kips)		**1543.78**
Overturning Moment (k-ft)		**33854.8**

(Refer to Figure 6 for Seismic Loading Diagram)

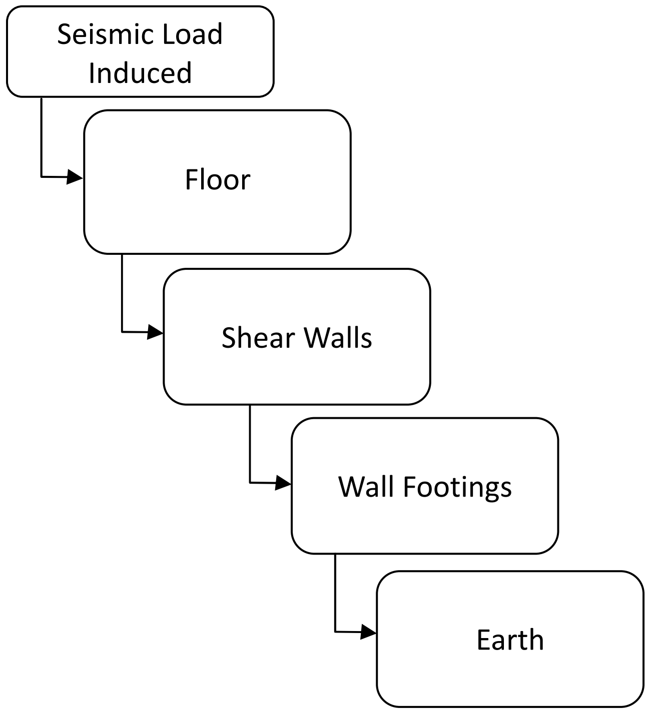
Load Path

Wind



The load induced by the wind will travel through the curtain wall or the masonry veneer, travel through the floor. The load will then follow the stiffness to the shear walls (stiffest structural member of the Hospital), and the load is directly transferred downwards to the footings and then to the earth.

Seismic



The seismic load will travel through the floor and follow the stiffness to the shear wall. The shear wall will directly transfer its load to its base, the wall footings and then release the load to the earth.

Lateral System Analysis

Through the analysis using ETABS, the shear wall system was the dominating lateral load resisting system in the Hospital as opposed to the initial speculation that the lateral load resisting system was a combination of moment frames and shear walls. There are a total of twelve shear walls present in the Hospital (see Figure 3) divided into three groups of four placed around openings for elevators. Two major types of analyses were undertaken in the writing of this report: Shear walls alone loaded with wind and seismic forces, and the building as a whole (including the columns and a fictitious floor) utilizing the following load cases:

1. 1.4D
2. 1.2D + 1.6L
3. 1.2D + 1.6L + 0.5L_r
4. 1.2D + 1.0L
5. 1.2D + 1.6L_r +1.0L
6. 1.2D + 1.6L_r + 0.8W
7. 1.2D + 0.8W
8. 1.2D + 1.0E
9. 1.2D + 1.0L + 0.5L_r + 1.6W

Upon analyzing the results provided by the computer model, the dominating load case was case 6. This load case was used for the remainder of the lateral load resisting system study.

Columns

As mentioned, columns were found to have no contribution to the resisting of lateral forces. The building modeled on ETABS was initially analyzed under lateral loads only. Through this, only the shear walls have been found to be resisting the lateral loads.

Shear Walls

As mentioned above, the shear wall is the main lateral load resisting system in the Hospital. The shear wall was analyzed under two main different conditions. The first to be discussed is the shear wall analyzed as a standalone structure as shown in Figures 7 to 9. The shear walls have been modeled as 12" thick meshed membranes. The shear walls were loaded with a unit force to study the rigidity and the drift for the East-West (X) direction and the North-South (Y) direction. The following table summarizes the findings:

Shear Wall Group 1 (X Direction)

Story	*Drift*
STORY5	0.000001
STORY4	0.000001
STORY3	0.000001
STORY2	0
STORY1	0

DISPLACEMENT	*RIGIDITY*
0.000003	333333.3333

Shear Wall Group 1 (Y Direction)

Story	*Drift*
STORY5	0.000003
STORY4	0.000002
STORY3	0.000002
STORY2	0.000001
STORY1	0.000001

DISPLACEMENT	*RIGIDITY*
0.000009	111111.1111

Shear Wall Group 2 (X Direction)

Story	*Drift*
STORY6	0.000002
STORY5	0.000002
STORY4	0.000002
STORY3	0.000001
STORY2	0.000001
STORY1	0

DISPLACEMENT	*RIGIDITY*
0.000008	125000

Shear Wall Group 2 (Y Direction)

Story	*Drift*
STORY6	0.000001
STORY5	0.000001
STORY4	0.000001
STORY3	0.000001
STORY2	0
STORY1	0

DISPLACEMENT	*RIGIDITY*
0.000004	250000

Shear Wall Group 3 (X Direction)

Story	*Drift*
STORY6	0.000001
STORY5	0
STORY4	0
STORY3	0
STORY2	0
STORY1	0

DISPLACEMENT	*RIGIDITY*
0.000001	1000000

Shear Wall Group 3 (Y Direction)

Story	*Drift*
STORY6	0.000002
STORY5	0.000002
STORY4	0.000001
STORY3	0.000001
STORY2	0.000001
STORY1	0.000001

DISPLACEMENT	*RIGIDITY*
0.000007	142857.1429

From these results, another table was created to calculate the relative stiffness and the proportioned wind and seismic loads:

Relative Stiffness and Proportioned Loads (X Direction)

SW #	*STORY*	*Total P (kips)*		*RELATIVE STIFFNESS*	*Element P (kips)*	
		WIND	*SEISMIC*		*WIND*	*SEISMIC*
SW1	2	52	405	0.23	12	93
	3	56	296		13	68
	4	61	391		14	89
	5	66	484		15	111
	6	71	337		16	77

SW #	*STORY*	*Total P (kips)*		*RELATIVE STIFFNESS*	*Element P (kips)*	
		WIND	*SEISMIC*		*WIND*	*SEISMIC*
SW2	2	52	405	0.09	4	35
	3	56	296		5	25
	4	61	391		5	33
	5	66	484		6	42
	6	71	337		6	29

SW #	*STORY*	*Total P (kips)*		*RELATIVE STIFFNESS*	*Element P (kips)*	
		WIND	*SEISMIC*		*WIND*	*SEISMIC*
SW3	2	52	405	0.69	36	278
	3	56	296		39	203
	4	61	391		42	268
	5	66	484		45	332
	6	71	337		48	231

Relative Stiffness and Proportioned Loads (Y Direction)

SW #	*STORY*	*Total P (kips)*		*RELATIVE STIFFNESS*	*Element P (kips)*	
		WIND	*SEISMIC*		*WIND*	*SEISMIC*
SW1	2	47	405	0.22	10	89
	3	51	296		11	65
	4	55	391		12	86
	5	60	484		13	107
	6	64	337		14	74

SW #	*STORY*	*Total P (kips)*		*RELATIVE STIFFNESS*	*Element P (kips)*	
		WIND	*SEISMIC*		*WIND*	*SEISMIC*
SW2	2	47	405	0.50	23	201
	3	51	296		25	147
	4	55	391		28	194
	5	60	484		30	240
	6	64	337		32	167

SW #	*STORY*	*Total P (kips)*		*RELATIVE STIFFNESS*	*Element P (kips)*	
		WIND	*SEISMIC*		*WIND*	*SEISMIC*
SW3	2	47	405	0.28	13	115
	3	51	296		15	84
	4	55	391		16	111
	5	60	484		17	137
	6	64	337		18	96

Shear Walls in the Building Structure

Following the shear wall analysis, the shear walls were modeled along with the rest of the building (see Figure 10) and analyzed for story drifts under the aforementioned load combination. The following assumptions were made during the creation of the model:

- Columns were all modeled as 24” x 24” columns despite the presence of six W12x40 columns.
- Beams were all modeled as 24” x 18” beams despite the presence of beams ranging in size from 24” x 18” to 32” x 36”. This modeling assumption was based on the majority of the beams being 24” x 18”.
- The floors were modeled as mass-less and weightless meshed semi-rigid diaphragms. As such, the openings on the floors such as elevator shafts and stair cases surrounded by the shear walls have been neglected.

The model yielded the following story drifts:

Building Story Drift (X Direction)

STORY	*DRIFT*	Δ_a
STORY6	0.051285	0.75
STORY5	0.13926	0.635
STORY4	0.131417	0.52
STORY3	0.103549	0.405
STORY2	0.045122	0.285
STORY1	0	0

Building Story Drift (Y Direction)

STORY	*DRIFT*	Δ_a
STORY6	0.00748	0.75
STORY5	0.007118	0.635
STORY4	0.007757	0.52
STORY3	0.006903	0.405
STORY2	0.003511	0.285
STORY1	0	0

Under the aforementioned loading condition and the modeling assumption for the analysis conducted, the Hospital is well within the allowable drift limits.

From the same analysis, a shear wall was randomly selected for a strength analysis. The base shear and moment were taken from the model. Refer to Appendix E for calculations.

Shear Wall Design and Analysis

From the full structural lateral load analysis, a shear wall was randomly picked for strength analysis basing all design calculations on the ACI 318-08 (note the structural drawings are dated in 2005/2006 and the author assumes the previous version of ACI 318 was used for the existing design).

From ETABS, the loading on the shear wall of interest was found to be 5900 k-ft of overturning moment and 630 kips of base shear. The design assumed that no boundary elements were required and also neglected secondary effects (see Appendix E for design calculations). The design is as follows:

Comparison of Shear Wall Design

Design	*Hypothetical*	*Existing*
Horizontal Shear Reinforcement	#4 @ 6" o.c.	#4 @ 12" o.c.
Vertical Shear Reinforcement	#4 @7" o.c.	#4 @ 12" o.c.
Flexural Reinforcement	(7) #8	(8) #5

From the assumptions and the analysis, the hypothetical design differs from the existing design. The greatest discrepancies in between the two are the flexural reinforcement. From the table above, the hypothetical design seems extremely conservative in its approach. This could be the result of assumptions made during the construction of the computer model. The greatest difference could arise from the difference of loading locations. The program by default assumes the loading of lateral forces directly to the center of mass or the center of rigidity (see Figure 11) depending on the type of lateral loads. Also, the floor was modeled as a semi-rigid diaphragm. In such case, the load path is almost neglected and the shear wall is almost directly loaded by the lateral loads.

Conclusion

The Monongalia General Hospital's lateral load resisting system was found to be a shear wall system, a different finding from the assumptions from earlier technical reports. The shear wall was analyzed against lateral loads under two main conditions: a shear wall by itself and as a system. Under these conditions drift of the shear walls and the building as whole, and the strength of the shear wall was studied. From the analyses conducted, the lateral force resisting system has proved to be adequate to resist the lateral forces.

MONONGALIA GENERAL HOSPITAL

LATERAL SYSTEM ANALYSIS
AND CONFIRMATION DESIGN

APPENDIX A

PROJECT TEAM

Owner	**Monongalia General Hospital** 1200 J.D. Anderson Dr. Morgantown, WV 26505	Phone: 304-598-7690 Fax: 304-598-7693 Website: http://www.monhealthsys.org/
Architect and Interiors	**Freeman White, Inc.** 8025 Arrowbridge Blvd. Charlotte, NC 28273-5665	Phone: 704-523-2230 Fax: 704-523-2235 Website: http://www.freemanwhite.com/
Civil Engineer	**Alpha Associates, Inc.** 209 Prairie Ave. Morgantown, WV 26502	Phone: 304-296-8216 Fax: 304-296-8216 Website: http://www.alphaaec.com/
Construction Manager	**Turner Construction Company** Two PNC Plaza, 620 Liberty Ave., 27th Floor Pittsburgh, PA 15222-2719	Phone: 412-255-5400 Fax: 412-255-0249 Website: http://www.turnerconstruction.com/
Geotechnical and Environmental Consultant	**Potesta Engineers and Environmental Consultants** 125 Lakeview Drive Morgantown, WV 26508	Phone: 304-225-2245 Fax: 304-225-2246 Website: http://www.potesta.com/
Mechanical, Electrical, and Plumbing	**Freeman White, Inc.** 2300 Rexwoods Dr., Suite 300 Raleigh, NC 27607	Phone: 919-782-0699 Fax: 919-783-0139 Website: http://www.freemanwhite.com/
Structural Engineer	**Atlantic Engineering Services** 650 Smithfield St., Suite 1200 Pittsburgh, PA 15222	Phone: 412-338-9000 Fax: 412-338-0051 Website: http://www.aespj.com/

MONONGALIA GENERAL HOSPITAL

LATERAL SYSTEM ANALYSIS
AND CONFIRMATION DESIGN

APPENDIX B

FIGURES

Figure 1: Hospital Divided in Four Quads

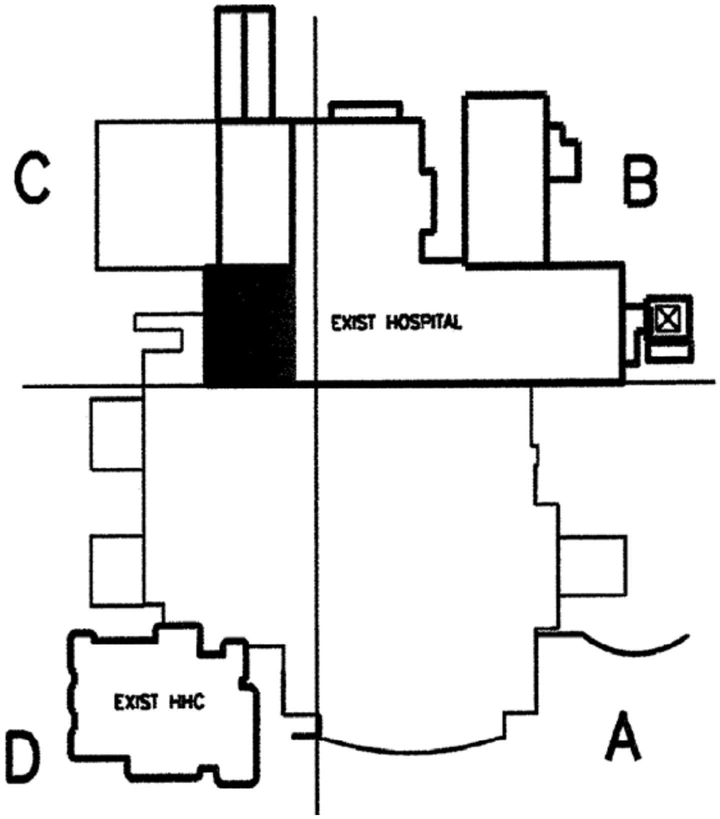
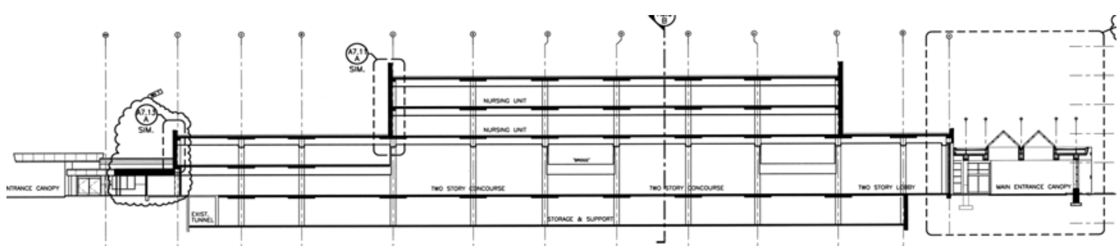
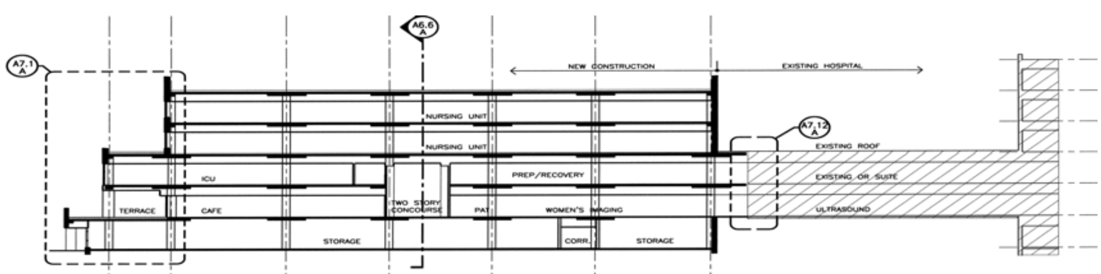


Figure 2: Cross Section of the Monongalia General Hospital



West Section



South Section

Figure 3: Location of Shear Walls (Colored in blue)

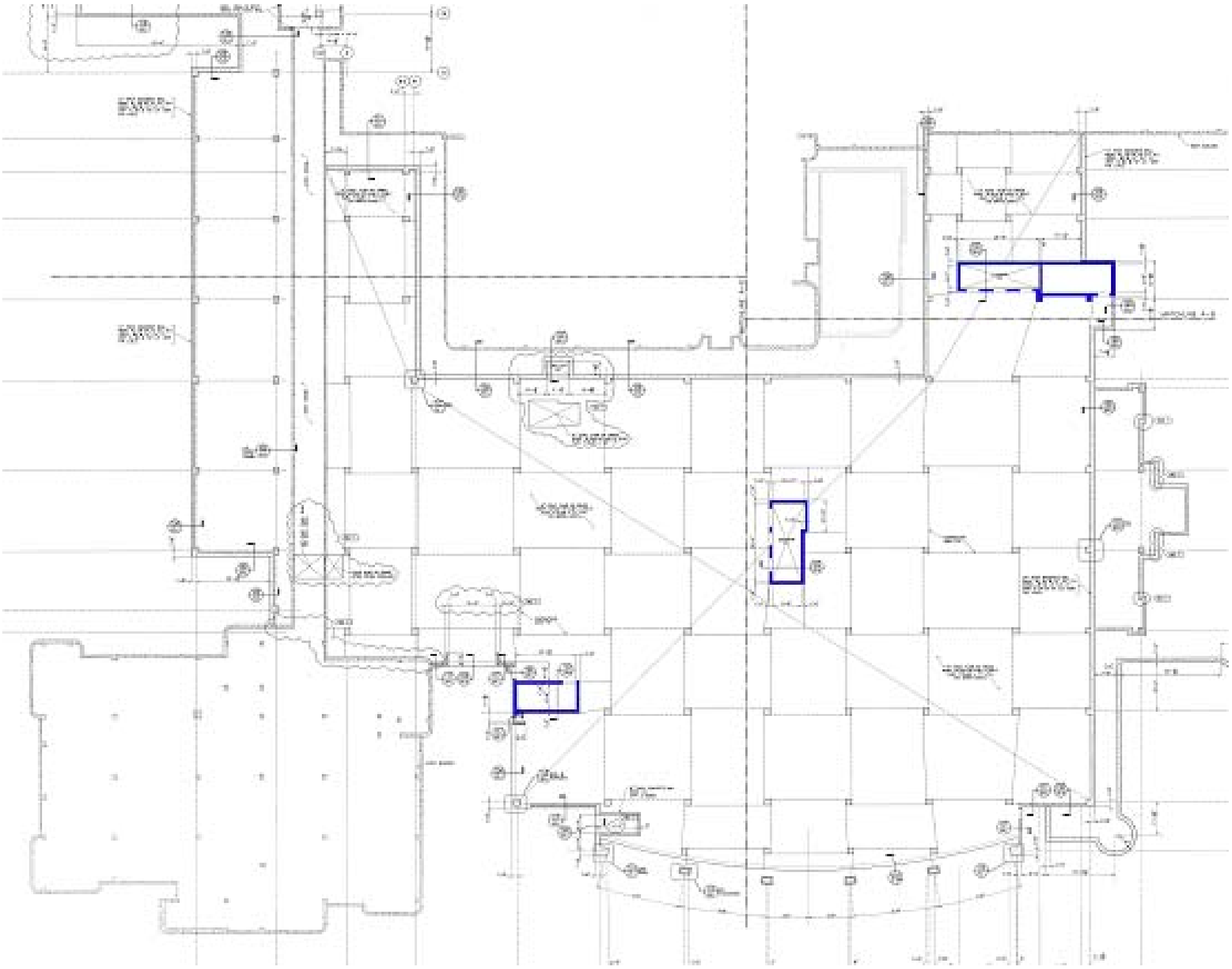


Figure 4: Wind Loading – North to South

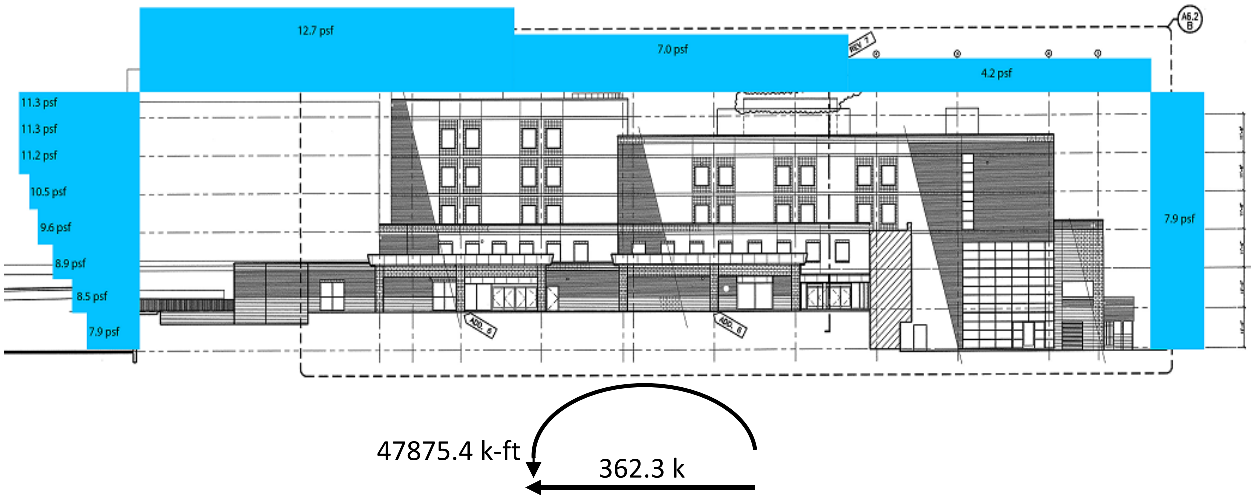


Figure 5: Wind Loading – East to West

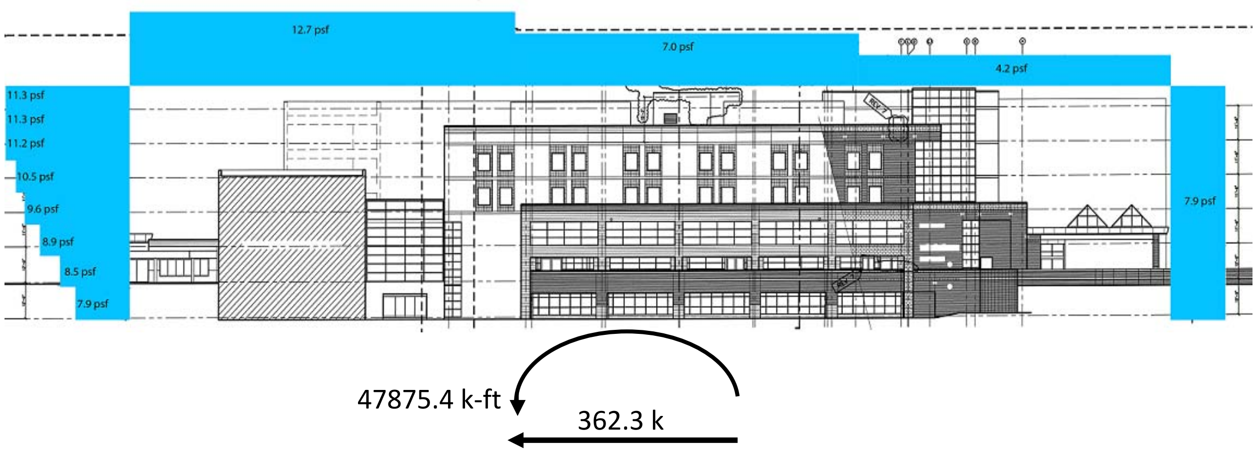


Figure 6: Seismic Loading

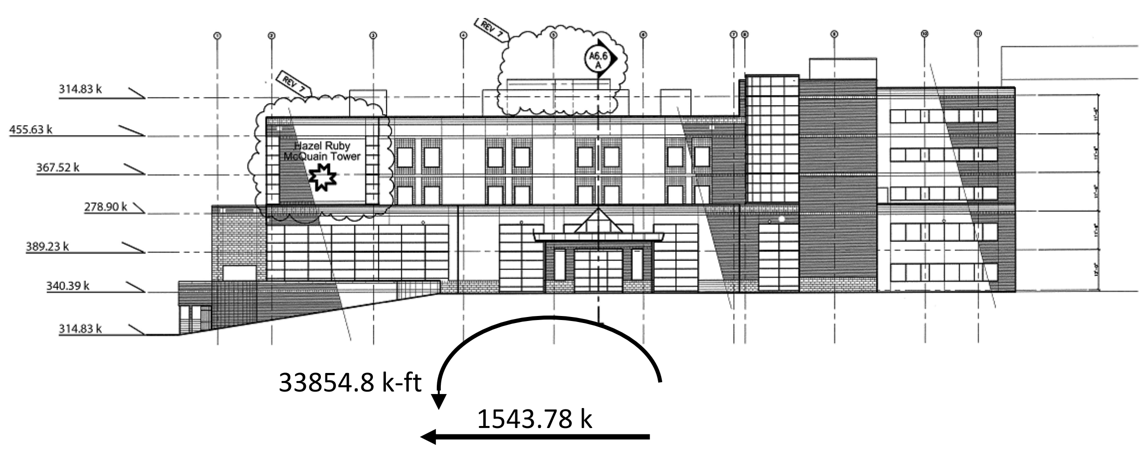


Figure 7: Shear Wall Group 1 Loaded in X Direction

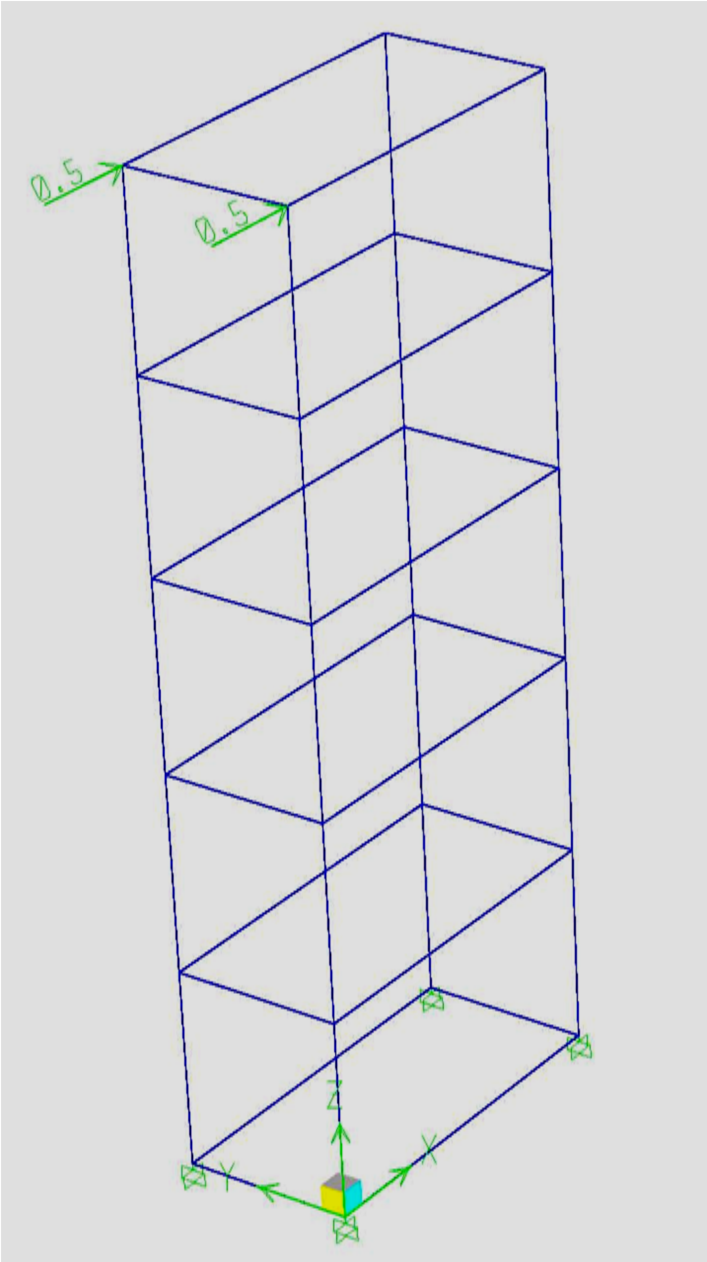


Figure 8: Shear wall Group 2 Loaded in X Direction

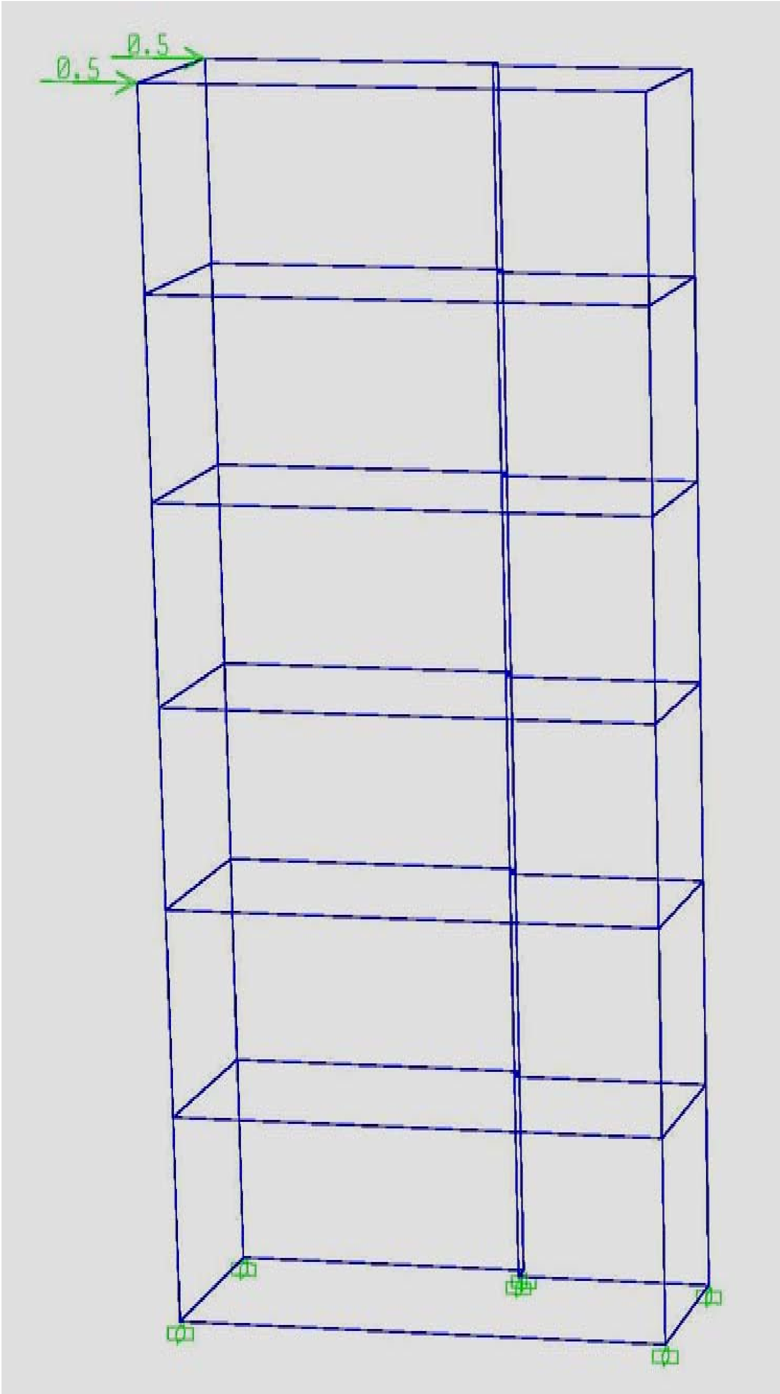


Figure 9: Shear Wall Group 3 Loaded in Y Direction

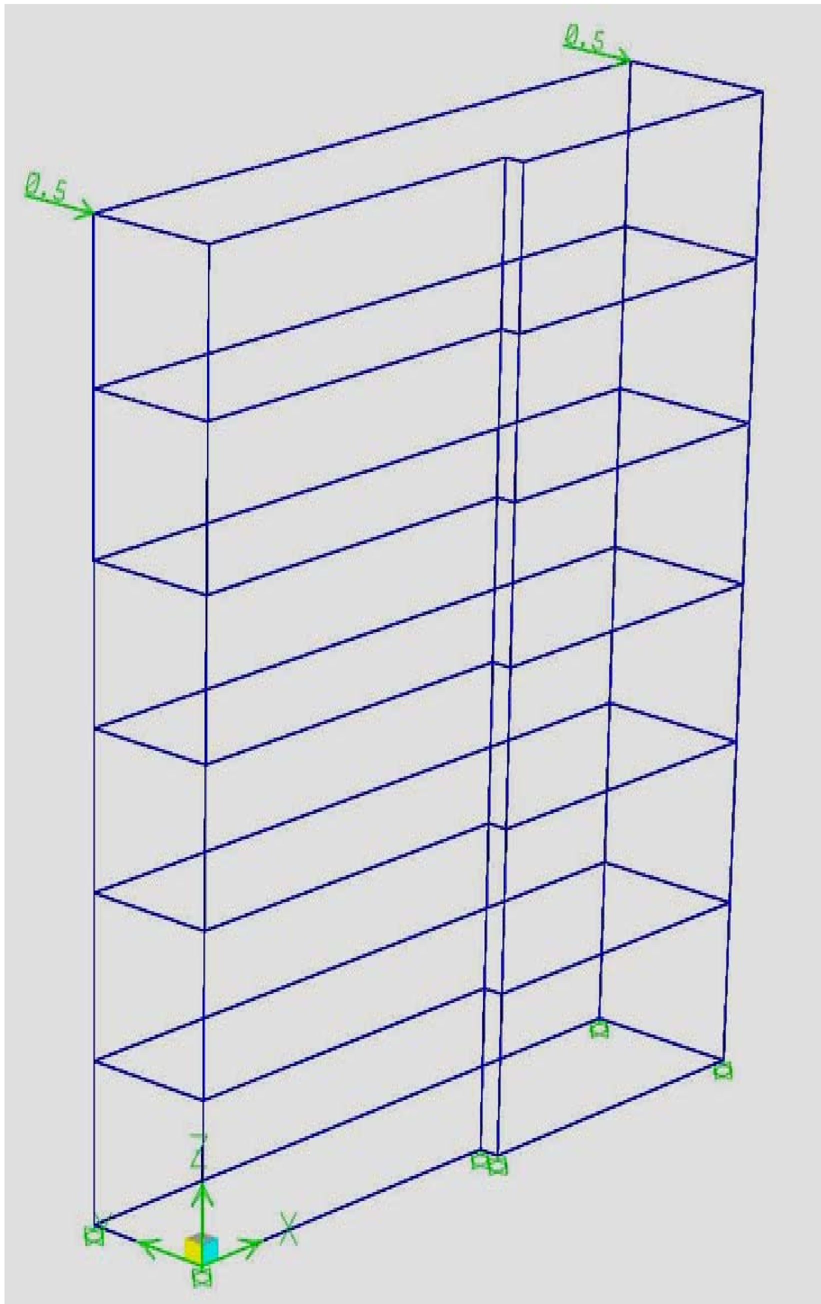


Figure 10: Monongalia General Hospital ETABS Model

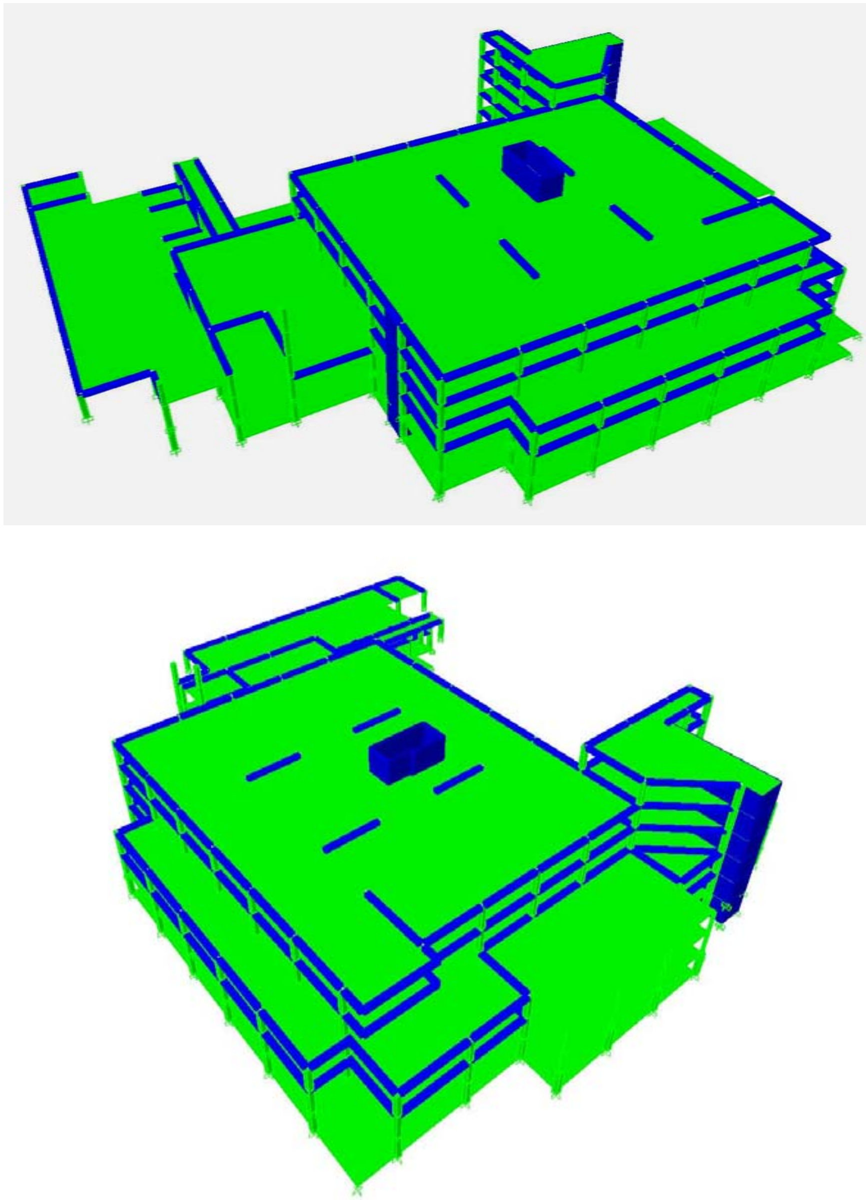
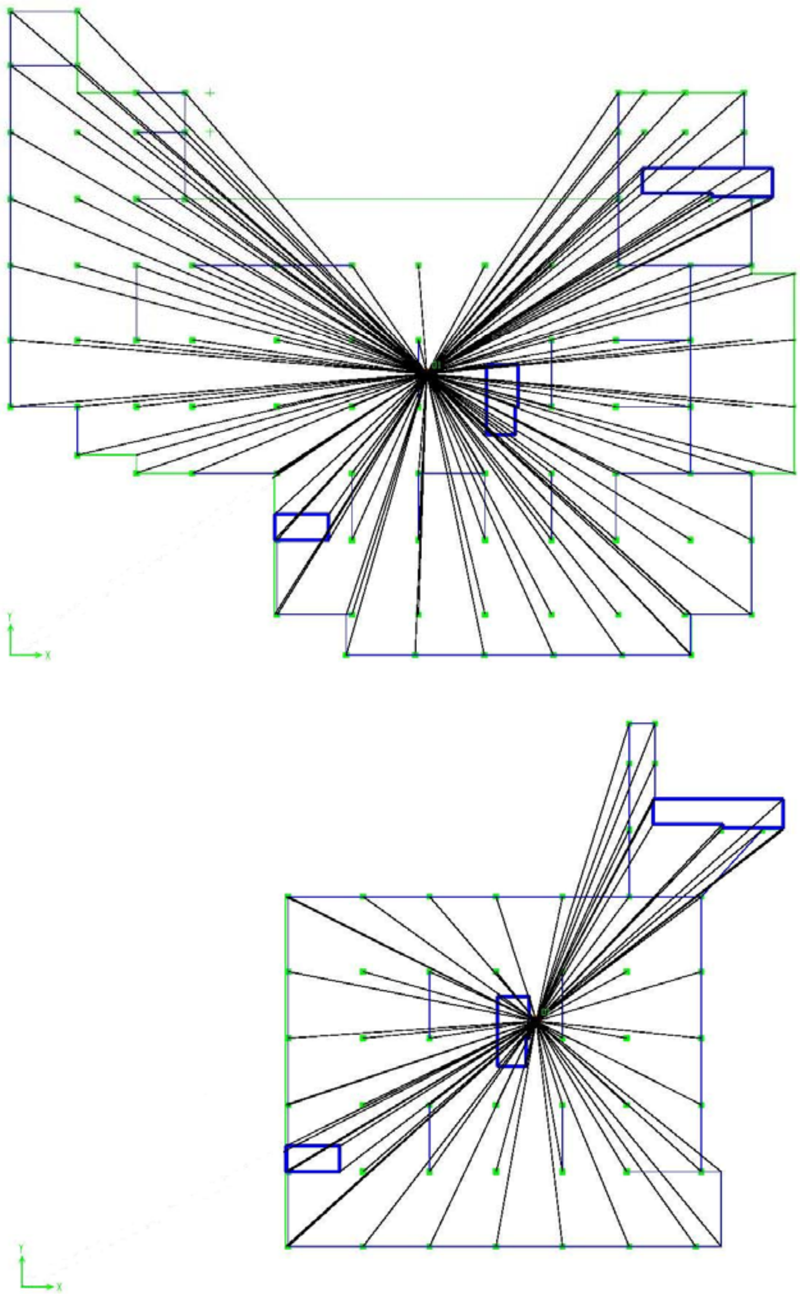


Figure 11: Floor Diaphragm Extent (Top: Story 1; Bottom: Story 5)



MONONGALIA GENERAL HOSPITAL

LATERAL SYSTEM ANALYSIS
AND CONFIRMATION DESIGN

APPENDIX C

PHOTOGRAPHS

Photograph 1: View from South-East

Photograph 2: Aerial Photo of the Monongalia General Hospital

Photograph 3: View from South-East showing the brick façade and curtain walls

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APPENDIX D

CODES

Type	Designed with	Analyzed with
Building	IBC 2000	IBC 2006
Structural	IBC 2003	IBC 2006
Plumbing	IPC 2000	-
Mechanical	IMC 2000	-
Electrical	NFPA 1999	-
Fire Safety	WV Fire Code 2002	-
Accessibility	ADA 1994	-
Energy	IEGC 2000	-
Fuel Gas	IFGC 2000	-
Sprinkler	NFPA 13	-

Construction Type: 1-A

Primary Occupancy: Institutional I-2

At the point of the project design phase, the building codes that were effective in Morgantown, WV are the ones listed above under the "Designed with" column. Today, the city of Morgantown has adopted the latest codes and ordinances.

The shearwalls were designed as per ACI 318-08.

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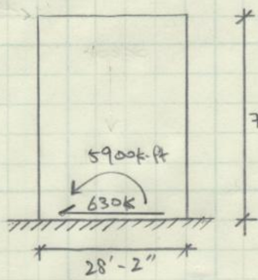
APPENDIX E

CALCULATIONS

STRENGTH CHECK: SHEAR WALL



$f'_c = 5000 psi$
$f_y = 60000 psi$

$h = 12"$

* ASSUME: NO BOUNDARY ELEMENT REQUIRED.
LOAD DUE TO WIND (CONTROLLING)
GRAVITY CAN BE NEGLECTED.

- SHEAR DESIGN.

- CHECK PERMITTED SHEAR

$V_u \le \phi V_n = \phi 10 \sqrt{f'_c}\, h \cdot d$

$\Rightarrow d = 0.8 l_w$

$= 0.8(28.167')(12"/ft)$

$\therefore d = 270"$

$630k \overset{?}{\le} (0.75)(10)\sqrt{5000 psi}\,(12")(270") = 1718k$ (GOOD)

- SHEAR STRENGTH BY Vc

$V_{c1} = 2\sqrt{f'_c}\, h \cdot d$

$= 2\sqrt{5000 psi}\,(12")(270")$

$\therefore V_{c1} = 458k$

$V_{c2} = 3.3\sqrt{f'_c} \cdot h \cdot d$

$= 3.3\sqrt{5000 psi}\,(12")(270")$

$\therefore V_{c2} = 756k.$

$V_{c3} = \left[0.6\sqrt{f'_c} + \dfrac{l_w 1.25\sqrt{f'_c}}{M_u/V_u - l_w/2}\right] \cdot h \cdot d$

$\Rightarrow$ CRITICAL SECTION

$$a \le \begin{cases} l_w/2 = 28.167'/2 = 14.1' & \text{* CRITICAL} \\ h_w/2 = 75'/2 = 37.5' \end{cases}$$

→ CONT'D.

⇒ CRITICAL SECTION

$M_U = V_U(h_w - \frac{l_w}{2})$

$= V_U(75' - 14.1')(12''/ft)$

$\therefore M_U = 731 V_U$

$V_{c_3} = \left[0.6\sqrt{5000 psi} + \frac{(28.167')(12''/ft)(1.25)\sqrt{5000psi}}{731 V_U/V_U - (28.167')(12''/ft)/2}\right](12'')(270'')$

$\therefore V_{c_3} = 310k$ * CONTROLS

- REQUIRED HORIZONTAL SHEAR REINFORCEMENT

- CHECK:

$V_U \overset{?}{>} \frac{1}{2}\phi V_C$

$630k > \frac{1}{2}(0.75)(310k) = 116k$

$\Rightarrow V_U \le \phi V_N = \phi(V_C + V_S)$

$630k \le (0.75)(310k + V_S)$

$\therefore V_S = 530k.$

- REQUIRED STEEL REINFORCEMENT

$\frac{A_V}{S} = \frac{V_S}{f_y \cdot d}$

$= \frac{530k}{(60ksi)(270'')}$

$\therefore \frac{A_V}{S} = 0.0327$

⇒ TRY (1) #4

$S = \frac{0.2 in^2}{0.0327}$

$\therefore S = 6''$

→ CONT'D.

- CHECK MINIMUM AND MAXIMUM REQUIREMENTS

$\rho_t = \frac{A_v}{s \cdot h}$

$\frac{0.2 in^2}{(6'')(12'')}$

$\therefore \rho_t = 0.00277 > 0.0025$ (GOOD)

$S_{MAX} \leq \begin{cases} l_w/5 = (28.167')(12''/ft)/5 = 67.6'' \\ 3h = (3)(12'') = 36'' \\ 18'' \text{ * CONTROLS} \end{cases}$

$S_{MAX} = 18'' > 6''$ (GOOD)

$\therefore$ **USE #4 @ 6" o.c.** (EXISTING USES #4 @ 12" o.c)

- REQUIRED VERTICAL SHEAR REINFORCEMENT

$\rho_l = \frac{A_v}{sh} \geq 0.0025 + 0.5(2.5 - h_w/l_w)(\rho_t - 0.0025)$

$\rho_l = 0.0025 + 0.5(2.5 - 75'/28.167')(0.00277 - 0.0025)$

$\therefore \rho_l = 0.0025 = \frac{A_v}{sh}$

=> TRY #4

$s = \frac{0.2 in^2}{(0.0025)(12'')}$

$\therefore s = 7''$

- CHECK MINIMUM AND MAXIMUM SPACING

$S_{MAX} \leq \begin{cases} l_w/3 = (28.167')(12''/ft)/3 = 112'' \\ 3h = 36'' \\ 18'' \text{ * CONTROLS} \end{cases}$

$\therefore S_{MAX} = 18'' > 7''$ (GOOD)

$\therefore$ **USE #4 @ 7" o.C.** (EXISTING USES #4 @ 12" o.c.)

→ CONT'D.

- FLEXURE DESIGN
  - CHECK AREA OF STEEL IN TENSION

$M_u = V_u h_w = \phi M_n = \phi A_s f_y jd$

$\Rightarrow jd = 0.9d$

$= 0.9(270'')$

$\therefore jd = 243''$

$(5900k \cdot ft)(12''/ft) = (0.9)A_s(60ksi)(243'')$

$\therefore A_s = 5.4 in^2$

  - CHECK CRITICAL SECTION

$C = T$

$0.85 f'_c h \cdot a = A_s f_y$

$0.85(5ksi)(12'')a = (5.4in^2)(60ksi)$

$\therefore a = 6.35''$

$jd = d - a/2$

$= 270'' - (6.35''/2)$

$\therefore jd = 267''$

  - REQUIRED STEEL REINFORCEMENT

$M_u = \phi A_s f_y jd$

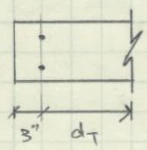
$(5900k \cdot ft)(12''/ft) = (0.9)A_s(60ksi)(267'')$

$\therefore A_s = 4.9 in^2$

$\therefore$ **USE (7) #8 ; $A_s = 5.5 in^2$** (EXISTING USES (8) #5)

→ CONT'D.

- CHECK TENSION CONTROL



$$d_T = (28.167')(12''/ft) - 3'' = 335''$$

$$C = T$$

$$0.85\,(5\,ksi)(12'')\,a = (5.5\,in^2)(60\,ksi)$$

$$\therefore a = 6.47''$$

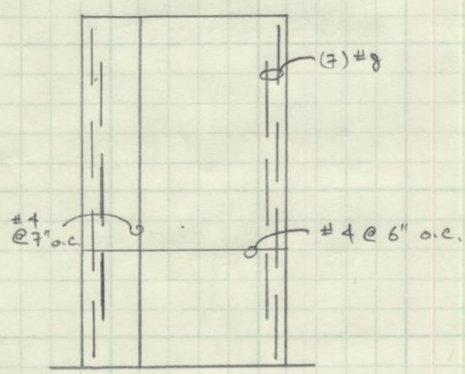
$$c = \frac{a}{\beta_1} = \frac{6.47''}{0.85}$$

$$\therefore c = 7.61''$$

$$\varepsilon_T = \varepsilon_u\left(\frac{d_T - c}{c}\right) = 0.003\left(\frac{335'' - 7.61''}{7.61''}\right)$$

$$\therefore \varepsilon_T = 0.129 > 0.005 \quad \therefore \text{TENSION CONTROLLED}$$

- DETAILING



* END OF ANALYSIS *

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APPENDIX F

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References

The following resources were utilized or considered in the writing of this report.

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  - S2-0
  - S2-4AD Bottom
  - S2-4AD Top
  - S3-0
  - S3-1
  - S4-0
  - S4-1
  - S4-4
  - S8-0
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Photographs

- Photograph 1 and 3 taken by the Turner Construction Company.

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