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A Case Study on the Design of Safe Spaces in Hospitals Vulnerable to Tornadoes in Central US

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A CASE STUDY ON THE DESIGN OF SAFE SPACES IN HOSPITALS VULNERABLE TO TORNADOES IN CENTRAL US.

by

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A CASE STUDY ON THE DESIGN OF SAFE SPACES IN HOSPITALS VULNERABLE TO TORNADOES IN CENTRAL US.

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In the wake of recent disasters happening around the world such as the earthquake in Italy (January 2017); hurricanes in the United States (U.S.) (September 2016 and 2017); and compounding disasters in Haiti (September 2010 and 2016); to our best knowledge, never has the world seen the need to work on preemptive rather than reactionary measures to address this issue. Tornadoes are natural hazards that commonly occur in mid-western and central states of the U.S. Tornadoes, like all natural hazards, are very destructive and result in massive destruction to building structures, causing billions of dollars in damage and claim many lives. Healthcare facilities in general are vulnerable to disasters and the safety of patients, health workers as well as those who come in to seek shelter should be a priority. This study assessed disaster management measures instituted by hospitals. Thus, the study examined building structure vulnerabilities and the design of safe spaces in hospitals within central U.S. Objectives that guided the work involved identifying the impact of tornadoes in hospitals and assessing the structural design of safe spaces. St. John's Regional Medical Center, now Mercy Hospital in Joplin, was used as a case study as a point of comparison pointing out structural performance from the 2011 event. The study revealed that incorporating construction materials outlined by FEMA and designing safe zones according to high-winds capacity is vital for reducing vulnerability to disasters in healthcare facilities. Findings led to a proposed structural design of an interior hallway/corridor safe space for healthcare facilities.

Keywords: *Disaster management, safe spaces, structural design, tornado, vulnerability*

DEDICATION

This Research is dedicated to:

My parents Very Rev. and Mrs. Ampaw-Asiedu; to my siblings Desmond, Mercy and Mary for their selfless commitment, dedication and encouragement throughout my education.

I love you all very much.

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LIST OF ABBREVIATIONS

ABBREVIATION	**EXPLANATION**
ACI	American Concrete Institute
ASCE	American Society of Civil Engineers
C&C	Component and Cladding
CMU	Concrete Masonry Unit
EF	Enhanced Fujita Scale
FEMA	Federal Emergency Management Agency
HICS	Hospital Incident Command System
IBC	International Building Code
ICC	International Code Council
MWFRS	Main Wind Force Resisting System
NIST	National Institute of Standard and Technology
NOAA	National Oceanic and Atmospheric Administration
O/C	On Center
SJRMC	St. John’s Regional Medical Center
USGS	United States Geological Survey

CHAPTER ONE

INTRODUCTION

1.0 BACKGROUND

Tornadoes are natural hazards that mostly affect mid-western and central states in the United States (US) – Iowa, Oklahoma, Nebraska, Missouri, Minnesota and Kansas (Kenward & Raja, 2014; Pereira, 2016). Tornadoes, like all natural hazards such as hurricanes, earthquakes, floods and others, are very devastating and result in massive destruction to homes, property and infrastructure, and cause fatalities (mostly from flying debris) (Kenward & Raja, 2014; E-School Today, 2016).

Health care facilities in general are vulnerable to disasters. The safety of patients, health workers as well as those who come in to seek shelter should be a priority (Pan America Health Organization [PAHO], 2000; Department of Communicable Disease, World Health Organization [WHO], 2008). A study by Grey and Hebert (2007) indicate that, in the event of a disaster, hospitals or health care facilities are supposed to continue functioning (PAHO, 2000). In addition, several studies (Schultz et al., 2003; Kaji & Lewis, 2006; Mehta, 2006) suggest that a disaster response plan is a requisite for every hospital in the US as required by the Joint Commission for Accreditation of Healthcare Organization (JCAHO).

It is also worthy to mention that, the building structures that is, the structural (load-bearing system) and non-structural (architectural elements and installations) of hospitals/healthcare buildings are also vulnerable in the event of tornadoes (PAHO, 2000; Schultz et al., 2003; Department of Communicable Disease, WHO, 2008). The following

six elements are the key vulnerability indicators of a building during disasters (Schultz et al, 2003; Department of Communicable Disease, WHO, 2008):

1. Location
2. Type of Disaster
3. Design materials and construction
4. Type of housing
5. Shape of building
6. Orientation

A case study on Birmingham Nursing and Rehabilitation Center, La Rocca, Greenbiriar, revealed that in the event of the Tuscaloosa Tornado, these health facilities did not have safe spaces or safe rooms leading to patient injuries. A safe room is defined by the Federal Emergency Management (FEMA) as “an interior room, or hallway, a space within a building or an entirely separate building designed and constructed to provide near-absolute life-safety protection for its occupants from tornadoes or hurricanes” (FEMA P-361,2015; FEMA P-453, 2006; FEMA P-320,2008).

This study will therefore assess disaster management measures put in place by hospitals and health care facilities in the US.

1.1 PROBLEM STATEMENT AND JUSTIFICATION

During the Enhanced Fujita-scale (EF-5) Joplin tornado in 2011, the city of Joplin, Missouri was struck by the tornado, as well as smaller communities and rural areas between the two cities wrecked homes, infrastructure, and public facilities. Many lives were claimed and the resulting damage was in billions of dollars. The storm destroyed many vital institutions; and St John’s Regional Medical Center, now Mercy

Hospital, was not left out. In the process, 14 patients lost their lives (Levitan et al, 2011). Consequently, the building was demolished and reconstructed. Mercy Hospital now boasts of a tornado-proof hospital with safe zones and reinforced walls and ceilings that can resist an EF-5 Tornado (Katz, 2013). Furthermore, hospitals are supposed to function in the event of a disaster or an emergency (PAHO, 2000); however, during Hurricane Irma, in September 2017, eight (8) patients died in a nursing home in Florida. This has raised concerns about the safety of health care facilities with respect to disasters (Reynolds & Spencer, 2017).

The question that comes to mind following the foregoing is "Do hospitals in Central US have safe zones during disasters?" It is therefore important to look into ways by which research can help to assess the design of safe spaces within hospitals in Central US.

1.2 RESEARCH OBJECTIVES

The sole aim of the research is to examine building structure vulnerabilities and the design of safe spaces in hospitals in Central US.

In line with the aforementioned aim, the research questions that will guide the study are:

1. What are the impacts of tornadoes in hospitals in Central US?
2. What are the design specifications or requirements of safe spaces in hospitals?
3. What recommendations can be made?

The tasks include:

1. To identify the impacts of tornadoes in hospitals in Central US.
2. To identify the building structure vulnerabilities and the design specifications or requirements of safe spaces in hospitals.
3. To assess the structural design/specifications of safe spaces.
4. To recommend an appropriate safe space design for hospitals in Central US.

1.3 DEFINITION OF TERMS

1. **Vulnerable populations**: Vulnerable populations are populations, which are unable, or do not have the means or predisposition to evacuate from areas of impending storm. (Diaz et al, 2013; FEMA P-361, 2015).
2. **Tornadoes**: Tornadoes are very common in the US with an annual average of 1,200 (Philips et al, 2012). Tornadoes have measured wind speeds of 125 m s^{-1} to feasibly 140 m s^{-1} and they double up as the most violent of atmospheric storms. (Davie-Jones, 2001).
3. **Safe Spaces:** A safe space is a space within a building "designed and constructed to provide near-absolute or absolute life-safety protection for its occupants from tornadoes and hurricanes (FEMA P-361, 2015)

CHAPTER TWO

LITERATURE REVIEW

2.0 INTRODUCTION

This chapter presents theoretical underpinnings for the research and explores the various concepts pertaining to the study. The chapter starts by giving an overview of tornado activities in the US; as well as review the relevant works that are published and unpublished on the subject matter. The concluding part of the chapter highlights the design and construction of safe spaces to provide a background for the study; taking into account best practices.

2.1 CENTRAL US

The study location is Central US. Figure 2.1 shows the map of US highlighting the states that constitute Central US, which comprises of West North Central, East North Central, West South Central and East South Central.

Figure 2. 1: Map of US showing Central US states.

2.2 TORNADO ACTIVITIES IN THE US

Tornado activities in the US from 1950 to 2004 (Figure 2.2) and a Tornado risk map (Figure 2.3) were looked at side by side.

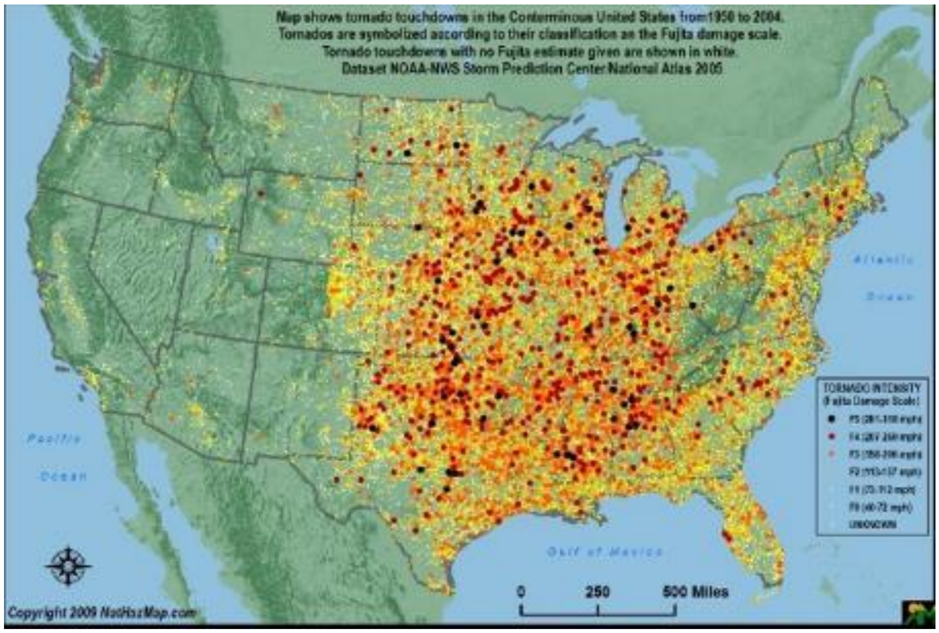

Figure 2. 2: Tornado activities in the US from 1950-2014
(ESRI & NOAA, 2009).

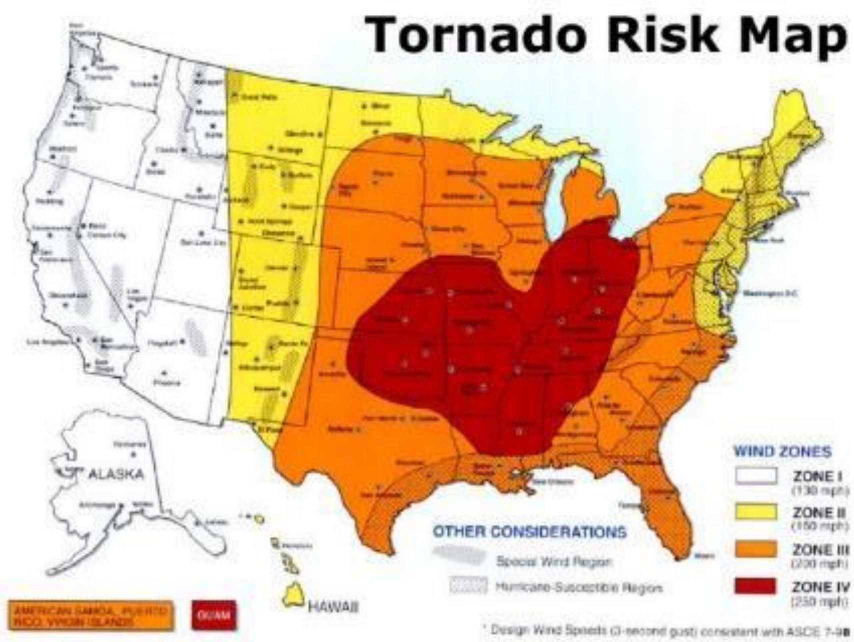

Figure 2. 3: Tornado Risk Map
(Strange, 2014).

From the maps (Figure 2.2 and 2.3 (Zone IV), Kansas, Alabama, Mississippi, Tennessee, Arkansas, Oklahoma, Missouri, Kentucky, Ohio and Indiana are in the high-risk areas and the healthcare facilities or hospitals in these areas will be of high importance with regards to recommendations for safe spaces.

2.3 IMPACT OF TORNADOES ON HOSPITALS

The effects of tornadoes on the built environment as against wind scale were also examined and Figure 2.4 shows the tornado scale with expected damages. According to National Institute of Standards and Technology [NIST] (2013), from 1950-2011, 68% of all tornado facilities were caused by tornadoes EF-3 and greater. Due to the study by NIST (2013), EF-3 to EF-5 were used to identify the impacts of tornadoes in hospitals in Central US.

Figure 2. 4: Tornado scale showing EF rating and expected damage (Rose, 2016).

Regarding the impact of tornadoes on the high-risk areas, assessment was based on the cost of structural damage, number of tornadoes and total fatalities and injuries as against the tornado scales of EF-3 to EF-5. Table 2.1 to Table 2.10 illustrate the impact of tornadoes in the High-risk States (Leitz, 2005).

Table 2. 1: Impact of Tornadoes in Oklahoma
(Modified from Leitz, 2005).

SCALE	DATE	NUMBER OF TORNADOES	TOTAL FATALITIES	TOTAL INJURIES	COST OF STRUCTURAL DAMAGE(S) IN US$
EF 3	1950-2015	193	256	1206	800 million -1 billion
EF 4	1950-2013	56	130	5285	1-2 billion
EF 5	1955-2013	8	256	2286	100-200 million

Table 2. 2: Impact of Tornadoes in Missouri
(Modified from Leitz, 2005).

SCALE	DATE	NUMBER OF TORNADOES	TOTAL FATALITIES	TOTAL INJURIES	COST OF STRUCTURAL DAMAGE(S) IN US$
EF 3	1950-2015	106	74	1206	400-600 million
EF 4	1952-2011	39	130	2006	300-500 million
EF 5	1957-2011	2	360	2507	30-50 million

Figures 2.5 and 2.6 however give examples of the impact of tornadoes on the building structural systems of health care facilities. Figure 2.5 also illustrates the impact of Tornado (Moore Tornado which was an EF-5 in 2013) on the structural system of Moore Medical Center, a healthcare facility in Oklahoma. Figure 2.6, however, shows the impact of Joplin tornado (EF-5) on St John’s Regional Medical Center (SJMRC).

Figure 2. 5: Impact of the Moore Tornado on Moore Medical Center
(Wilson, 2013).

Figure 2. 6: Impact of Joplin Tornado on SJRMC
(Katz, 2013).

Table 2. 3: Impact of Tornadoes in Alabama
(Modified from Leitz, 2005).

SCALE	**DATE**	**NUMBER OF TORNADOES**	**TOTAL FATALITIES**	**TOTAL INJURIES**	**COST OF STRUCTURAL DAMAGE(S) IN US$**
EF 3	1950-2014	136	93	2726	100-200 million
EF 4	1952-2011	35	343	5053	800 million -1 billion
EF 5	1957-2011	9	376	2436	200-300 million

Table 2. 4: Impact of Tornadoes in Kansas
(Modified from Leitz, 2005).

SCALE	DATE	NUMBER OF TORNADOES	TOTAL FATALITIES	TOTAL INJURIES	COST OF STRUCTURAL DAMAGE(S) IN US$
EF 3	1950-2015	179	32	638	200-400 million
EF 4	1950-2013	40	65	967	200-350 million
EF 5	1955-2007	8	270	2008	1-2 billion

Table 2. 5: Impact of Tornadoes in Mississippi
(Modified from Leitz, 2005).

SCALE	DATE	NUMBER OF TORNADOES	TOTAL FATALITIES	TOTAL INJURIES	COST OF STRUCTURAL DAMAGE(S) IN US$
EF 3	1950-2015	107	93	1516	300-400 million
EF 4	1952-2015	28	262	3572	200-300 million
EF 5	1953-2011	5	227	1959	300-500 million

Table 2. 6: Impact of Tornadoes in Tennessee
(Modified from Leitz, 2005).

SCALE	DATE	NUMBER OF TORNADOES	TOTAL FATALITIES	TOTAL INJURIES	COST OF STRUCTURAL DAMAGE(S) IN US$
EF 3	1952-2015	91	153	2726	500-800 million
EF 4	1952-2015	32	295	3487	200-400 million
EF 5	1974-2011	3	163	561	3000

Table 2. 7: Impact of Tornadoes in Indiana
(Modified from Leitz, 2005).

SCALE	DATE	NUMBER OF TORNADOES	TOTAL FATALITIES	TOTAL INJURIES	COST OF STRUCTURAL DAMAGE(S) IN US$
EF 3	1951-2013	94	83	1234	300-500 million
EF 4	1956-2012	30	249	3964	800 million - 3 billion
EF 5	1974	3	71	836	500-700 million

Table 2. 8: Impact of Tornadoes in Kentucky
(Modified from Leitz, 2005).

SCALE	DATE	NUMBER OF TORNADOES	TOTAL FATALITIES	TOTAL INJURIES	COST OF STRUCTURAL DAMAGE(S) IN US$
EF 3	1951-2013	93	1508	1234	300-400 million
EF 4	1964-2012	78	1613	3964	200 - 300 million
EF 5	1974	65	750	836	100-250 million

Table 2. 9: Impact of Tornadoes in Ohio
(Modified from Leitz, 2005).

SCALE	DATE	NUMBER OF TORNADOES	TOTAL FATALITIES	TOTAL INJURIES	COST OF STRUCTURAL DAMAGE(S) IN US$
EF 3	1950-2014	38	93	1104	1- 3 billion
EF 4	1952-2010	144	345	2376	900 million - 1 billion
EF 5	1968-1985	100	376	2913	2 - 4 billion

Table 2. 10: Impact of Tornadoes in Arkansas
(Modified from Leitz, 2005).

SCALE	DATE	NUMBER OF TORNADOES	TOTAL FATALITIES	TOTAL INJURIES	COST OF STRUCTURAL DAMAGE(S) IN US$
EF 3	1950-2011	157	114	2019	300-400 million
EF 4	1952-2014	28	294	2908	400 - 600 million

It is worth mentioning that, In November 2017 an emergency preparedness rule was passed by the federal register which required all Medicare and Medicaid participating providers to comply with this rule. Health care providers are to satisfy the requirement of Risk Assessment and Emergency planning. In addition, this takes care of hazards that are likely in the geographical area, Lost of either all or portions of the building structure and loss of supplies. This rule will help to ensure adequate planning for

both man-made and natural disasters and help reduce building structure vulnerabilities because hospitals will need to have these requirements for certification (CMS, 2017).

2.3.1 CASE STUDY OF ST JOHN'S REGIONAL MEDICAL CENTER (SJRMC)

1. **Events**

An EF-5 tornado destroyed St John's Regional Medical Center in Joplin in May 2011. The storm blew out all the windows of the building, and portions of the roof were pulled off and the infrastructure was severely damaged. Generators were destroyed and so were communications equipment (Hector & Hewitt,2013; Beatty et al, 2015). During the storm, 183 patients were in the hospital. There were patients in critical care, emergency rooms, labor rooms as well as surgical rooms. Three collection points were used for evacuation, namely: the East Side, West Side and Conference Center (Figure 2.7). The methods of evacuation employed included ambulatory and wheel chairs, mattresses, doors, medical sleds and triage. Critical patients were transferred to other hospitals. Incident command systems were used (Beatty et al, 2015).

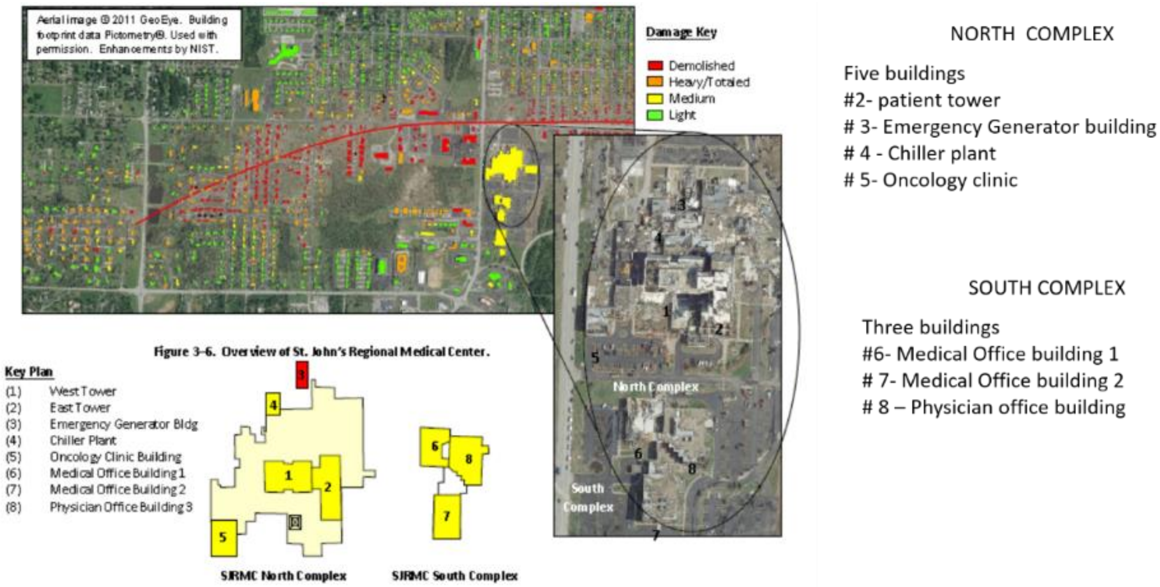

Figure 2. 7: Layout of SJRMC
(Levitan et al, 2011).

2. Tornado impact on building structural systems

Hospitals are categorized as *Risk Category IV* - “essential facilities” and are defined as “buildings and other structures that are intended to remain operational in the event of extreme environmental loading from flood, wind, snow, or earthquakes” (ASCE 7-10).

Building codes are important for structural design and prior to the 2011 Joplin Tornado; the City of Joplin adopted a building code through Ordinance No. 2008–068

- 2006 ICC International Building Code (IBC),
- 2006 ICC International Residential Code for One– and Two–Family Dwellings (IRC) (Levitan et al, 2011).

This code informed the design of St. John’s Regional Medical Center. Figure 2.7 shows the layout of SJMRC. The medical center was divided into North Complex and South Complex. The North complex consisted of five buildings, namely: The West Tower, East

Tower, Emergency Generator Building, chiller plant and Oncology clinic. The South Complex, however, was made up of three buildings including the medical office buildings and the physician office building. All the buildings were studied intensively and informed the proposed safe space design. The study of St John's Regional Medical Center was mainly a building structural analysis based on Building codes, Design Wind Speeds, Main Wind Force Resisting System (MWFRS), Floor system, and Component and Cladding (C&C).

Table 2.11 illustrates the design information of the West Tower. There was no structural damage; that is, damage to the lateral load system and gravity load system (MWFRS). However, the building's Component and Cladding system (C&C), which consist of vertical glass windows, were damaged. Additionally, unreinforced Concrete Masonry Units (CMU) collapsed. Interior partitions and HVAC equipment were damaged as well. The damage to the West Tower can be seen in Figure 2.8 (Levitan et al, 2011). It should be noted that the basic wind speed that affected buildings in the North Complex was 170+/-20mph (EF-4) and based on today's standards, the wind speed would have been 120mph.

Figure 2. 8: Damage to the West Tower
(Levitan et al, 2011).

Table 2. 11: Design information for West Tower
(Modified from Levitan et al, 2011).

BUILDING CODE	**DESIGN WIND SPEED**	**MWFRS**	**FLOOR SYSTEM**	**C&C**
1960 BOCA BBC	70 mph or 85 mph in 3 second gusts	Cast in place reinforced concrete with a mean roof height of 86.7 ft.	Reinforced concrete (RC) waffle slab floor.	Single story curtain wall panels made from aluminum framing and resistant glass window on 5th floor

Table 2.12 illustrates the design information of the East Tower. There was no structural damage; that is, damage to the lateral load system and gravity load system (MWFRS). However, the building's Component and Cladding system (C&C) which consist of glass curtain wall was damaged. Additionally, interior partitions and HVAC equipment were also damaged. The damage to the East Tower can be seen in Figure 2.9 (Levitan et al, 2011).

Figure 2. 9: Damage to the East Tower
(Levitan et al, 2011).

Table 2. 12: Design information for East Tower
(Modified from Levitan et al, 2011).

BUILDING CODE	DESIGN WIND SPEED	MWFRS	FLOOR SYSTEM	C&C
1984 BOCA B/NBC	70 mph or 85 mph in 3 second gusts	Nine story with moment connections and steel cross bracing.	Composite concrete-steel deck floor	Single story curtain wall panels made from aluminum framing and dual pane insulated glass glazing and precast concrete column

Table 2.13 shows the design information of the Emergency Generator Building. There was structural damage, thus, the lateral load system and gravity load system (MWFRS) failed. Roof joist disconnected from the CMU and due to the wind uplift pressure, lateral bracing for exterior CMU failed. The damage to the East Tower is shown in Figure 2.10 (Levitan et al, 2011).

Figure 2. 10: Damage to the Emergency Generator Building
(Levitan et al, 2011).

Table 2. 13: Design information for Emergency Generator Building
(Modified from Levitan et al, 2011).

BUILDING CODE	DESIGN WIND SPEED	MWFRS	FLOOR SYSTEM	C&C
BOCA National Code/1990	70 mph or 85 mph in 3 second gusts	• 14.5 mean roof height • Partially grouted , lightly reinforced, single-wythe CMU exterior walls (12in)	5 in thick RC slab on grade	• Envelope: 12 in thick CMU • 1in thick insulation on exterior walls

Table 2.14 illustrates the design information of the Chiller plant. There was no structural damage to the lateral load system and gravity load system which was a steel frame structure. However, there was damage to the building envelope and this damage was from the debris impact and wind pressure. In Addition, mechanical equipment were destroyed. Figure 2.11 shows the damage to the Chiller plant (Levitan et al, 2011).

Figure 2. 11: Damage to the Chiller plant
(Levitan et al, 2011)

Table 2. 14: Design information for Chiller plant
(Modified from Levitan et al, 2011).

BUILDING CODE	DESIGN WIND SPEED	MWFRS	FLOOR SYSTEM	C&C
1984 BOCA B/NBC	70 mph or 85 mph in 3 second gusts	• Steel frame (W-shape beams and columns connected with bolts • Column casted into foundation	5 in thick RC slab on grade	• Envelope: partially grouted CMU

Table 2.15 shows the design information of the Oncology Clinic. There was no structural damage to the lateral load resisting system and gravity load floor system, which was a steel moment frame structure. However, there was damage to the building envelope which consisted of insulated glass panels and the window systems were also damaged. In Addition, the interior of the building was severely damaged (Levitan et al, 2011). Figure 2.12 illustrates the damage to the Oncology Clinic.

Figure 2. 12: Damage to the Oncology Clinic
(Levitan et al, 2011).

Table 2. 15: Damage to Oncology Clinic
(Modified from Levitan et al, 2011).

BUILDING CODE	DESIGN WIND SPEED	MWFRS	FLOOR SYSTEM	C&C
BOCA National Code/1987	Not specified	• Moment connections between members	• Composite concrete-steel deck floor	• Envelope: single story curtain wall windows made of aluminum framing, insulated glass panel • Precast concrete arch. col

Table 2.16 shows the design information of Medical Building 1. There was no structural damage to the lateral load resisting system (steel frame) and gravity load floor system (Reinforced concrete on a steel deck). However, there was damage to the building envelope and a large portion of the roof deck made up of trusses were not damaged. In addition, the interior of the building was severely damaged (Levitan et al, 2011). Figure 2.13 shows the damage to the Medical building 1. The basic wind speed that affected buildings in the South Complex was 120+/-20mph (EF-2 to EF-3) and based on today standards, the wind speed would have been 120mph.

Table 2. 16: Damage to Medical Office 1
(Modified from Levitan et al, 2011).

BUILDING CODE	DESIGN WIND SPEED	MWFRS	FLOOR SYSTEM	C&C
Not specified	Not specified	• Steel frame (W-shape beams and columns connected with bolts • Column casted into foundation	• Composite concrete-steel deck floor	• Envelope: four types of curtain wall • Glass panels with aluminum framing • Brick veneer

Figure 2. 13: Damage to the Medical Building 1
(Levitan et al, 2011).

Table 2.17 shows the design information of Medical Building 2. There was no structural damage to the lateral load resisting system (steel frame) and gravity load floor system (Reinforced concrete on a steel deck). Nevertheless, there was damage to the building envelope and there was damage to steel roof deck. In addition, the interior of the

building was severely damaged (Levitan et al, 2011). Figure 2.14 shows the damage to the Medical building 2.

Table 2. 17: Damage to Medical Office 2
(Modified from Levitan et al, 2011).

BUILDING CODE	DESIGN WIND SPEED	MWFRS	FLOOR SYSTEM	C&C
BOCA National building code/1990	70 mph or 85 mph in 3 second gusts	• Steel frame (W-shape beams and columns connected with bolts • Steel K-braces for lateral loads	• Composite concrete-steel deck floor	• Envelope: four types of curtain wall • Glass panels with aluminum framing • Brick veneer • Steel roof deck

Figure 2. 14: Damage to the Medical Building 2
(Levitan et al, 2011).

Table 2.18 shows the design information of Physician Office Building. There was no structural damage to the lateral load resisting system (steel frame) and gravity load floor system (reinforced concrete on a steel deck). Yet, there was damage to the building envelope and there was damage to steel roof deck as well as the building's curtain wall system. Additionally, the interior of the building was severely damaged (Levitan et al, 2011). Figure 2.15 shows the damage to the Physician Office Building.

Table 2. 18: Damage to Physician Office Building
(Modified from Levitan et al, 2011).

BUILDING CODE	**DESIGN WIND SPEED**	**MWFRS**	**FLOOR SYSTEM**	**C&C**
BOCA National building code/1987	70 mph or 85 mph in 3 second gusts	• Steel frame (W-shape beams and columns connected with bolts	• Composite concrete-steel deck floor	• Envelope: Glass panels with aluminum framing • Architectural metal insulated panels • Brick veneer curtain wall

Figure 2. 15: Damage to the Physician office building
(Levitan et al, 2011).

3. New building construction after the tornado impact

After the storm, the hospital further put up a new building structure which can withstand up to EF-5 tornado and serves as a safe haven should a tornado strike (Figure 2.16). According to Hector & Hewitt,2013 and Beatty et al, 2015, The construction materials for the new design consist of a concrete (precast concrete) shell for the building, high-impact laminated glass that can withstand windspeeds of up to 250 mph for critical areas, barrier storm doors and fortified safe zones with reinforced concrete walls and ceilings on each floor. The design also includes a 450 ft underground tunnel for a central utility plant which will keep the hospital running after a natural hazard hits.

Figure 2. 16: New Hospital after the tornado
(Beatty et al, 2015).

From the intensive study of St. John's Regional Medical Center, the proposed safe space design should have a very resilient lateral and gravity system (MWFRS) and the C&C should be able to withstand extreme winds. Proposed design should also take into consideration applicable codes as well as roof to wall connection which will prevent the roof from tearing up from the building structure. In addition, storm doors should be

incorporated to help prevent wind-borne debris impact affecting people housed in the safe space.

2.4 DESIGN AND CONSTRUCTION OF SAFE SPACES

In order to assess the building structure vulnerabilities and the specifications or requirements of safe spaces in hospitals, the Hospital Incident Command System (HICS) was studied. During emergencies, hospitals either:

1. Transfer patients to a bigger hospital,
2. Evacuate the building after several training and exercises on how to evacuate the facility,
3. Put shelter in place for the community or
4. Use triage to sort out patients for treatments (Schultz et al, 2003; FEMA P-453, 2006).

2.4.1 VULNERABILITIES

Hospitals are vulnerable to disasters. Patients and Health workers are also vulnerable. The most vulnerable populations during disasters in hospitals are children, elderly with chronic diseases, bedridden patients and pregnant women (PAHO, 2003; Iserson & Moskop, 2007; Allen et al, 2007). Furthermore, the hospital building structure itself is also vulnerable. Building structures that are load-bearing systems and non-structural building system can adversely be affected (Schultz et al, 2003). However, it should be noted that, “vulnerability is not static and that vulnerability may reduce by evacuating the

region, evacuation to a local shelter, or to a sheltering in place within a "prepared shelter'" (Diaz et al, 2013).

2.4.2 TORNADO DAMAGE ON A BUILDING STRUCTURE

As already mentioned, the building structure is vulnerable during a disaster. According to FEMA P-431, 2009 tornado damage to a building is due to:

1. Debris impact
2. Differences in atmospheric pressure and
3. Wind-induced force

The effects of wind on building surface is such that, it creates an outward and inward-acting pressure. During tornadoes, however, most buildings fail due to suction pressure from the combination of internal pressure and outward pull and this causes the walls to pull outwards causing failure. Figure 2.17 explains the effect of wind on an enclosed building.

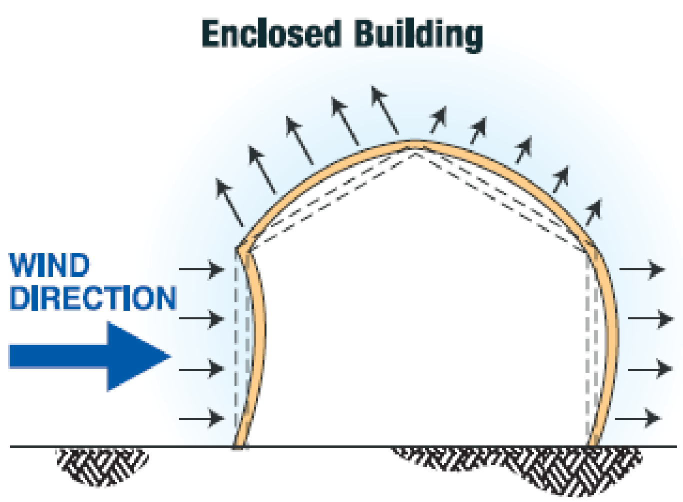

Figure 2. 17: Effects of wind on an enclosed building
(FEMA P-431, 2009).

2.4.3 SAFE SPACES

Safe rooms are to be strictly designed in line with FEMA P-361 standards with the main goal of protecting people against fatalities and injuries. Hence, it is always best to assume that the site for the building is within the higher tornado zone. The importance of safe spaces in hospitals cannot be ignored, since it allows for quick recovery for patients in addition to healthcare workers.

The design consideration for a safe room includes maximum occupancy time of 2 hours, 5 square feet per person (5ft^2/p) who is standing or seated, 10 square feet (10 ft^2/p) for wheel chair, 30 square feet (30 ft^2/p) for medical bed users. Emergency provisions such as water, communications equipment and supplies should be provided in the safe space. Safe rooms can also be multi-use safe rooms such as cafeterias, hallways bathrooms, surgical rooms (FEMA P-453, 2006; FEMA P-320, 2008; FEMA P-361, 2015). This study will employ the interior corridor/ hallway as the safe space. The cost of a safe room is also dependent on the location, design, whether it is new or a retrofit, and the design wind speed (FEMA P-361).

2.4.4 STRUCTURAL DESIGN OF SAFE SPACES

Structural design of safe rooms is based on International Code Council (ICC) 500 and American Society of Civil Engineers (ASCE) 7 standards; specifically, the latest version. The design parameters, however, are based on:

1. Single-use versus multi-use,
2. Design complexity,
3. Safe room design wind speed,
4. Safe room debris impact resistance design criteria,

5. Foundation,
6. Resistance to large wind-borne debris loads, and
7. Resistance to seismic loads (FEMA P-320, 2008).

This study will employ the Safe Room Design Wind Speed Parameter which includes designing the safe space to resist missile impact loads (ICC 500, 2014).

Some construction materials that are mostly used for safe rooms are Concrete Masonry Unit (CMU), Precast Concrete, Reinforced Concrete, Reinforced Masonry, Insulated Concrete Forms, etc. (FEMA P-320, 2008).

For the structural design, however, connections, floors, roof system, foundations, doors and windows should be looked at.

2.4.5 RELATED WORKS ON TORNADO RESISTANCE DESIGN

Below (Figure 2.18) shows a case study of a school community safe room in South East Kansas. It was designed be multi-used and shelter students and staff but not for the general public. The safe space houses 730 people. The design is constructed with a fully grouted, reinforced concrete masonry unit walls, and a reinforced concrete roof slab on composite metal deck that is supported by steel beams (Figure. 2.18). FEMA P-361 and ICC 500 documents for tornado community safe room were met. A wind speed of 250mph was used. The final design of the safe room is made up of design of connections, slabs and foundations. From the designed, the following were deduced and this informed the proposed design (FEMA, 2016).

1. **Connections:** Connections prove vital during tornado hazards. This is because they help transfer loads, and hence, should be strong enough to prevent deformation. A deficiency

in the connections will lead to structural damage of the safe room and loss of life. Connections used in the school community safe room were screws, steel bolts, welds, steel stuffs. Size and number depend on the wind pressure acting on it.

2. **Slabs (floor):** Slabs were be 3.5 inches thick and have steel reinforcement of a #4 minimum and a minimum spacing of 18 inches on center.

3. **Foundations:** Reinforcement bars go all the way from the walls to the foundation. Figure 2.18 illustrates a section through one of the safe rooms in the school community safe room in South East Kansas (FEMA, 2016).

FEMA recommends that the design wind speed for a safe space should be 250 mph regardless of location. In addition, from the case study, 250 mph was employed.

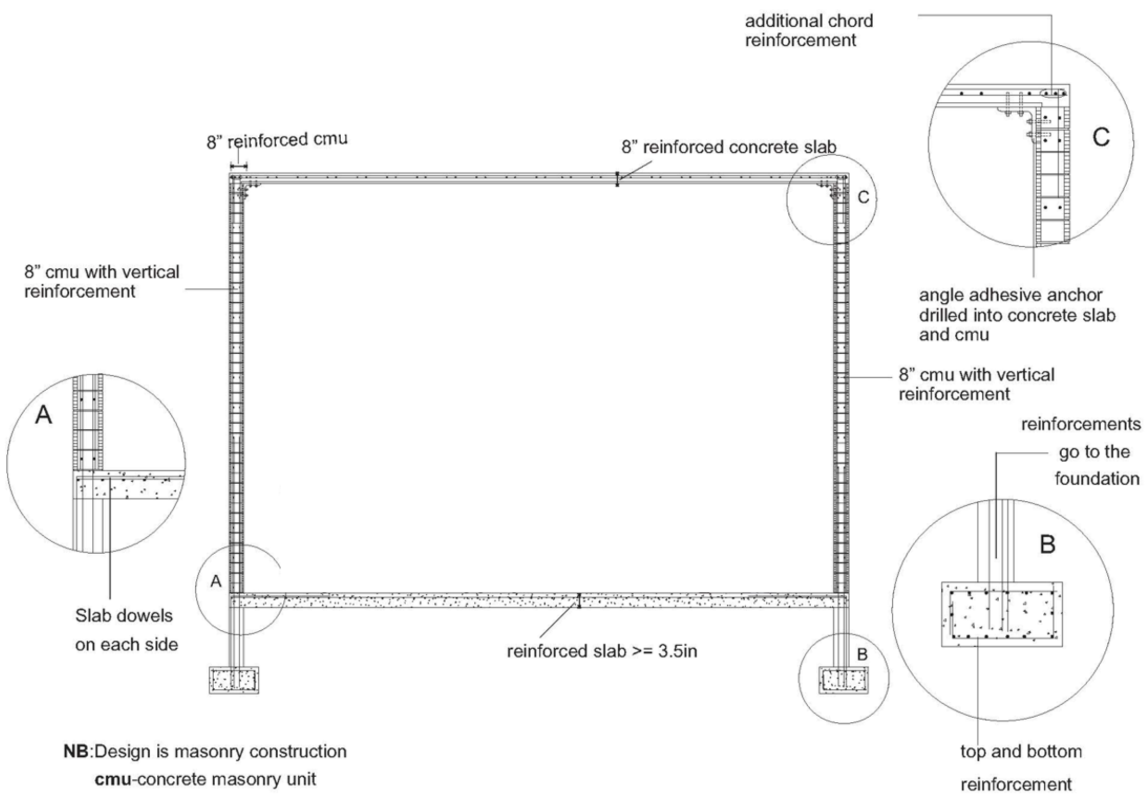

Figure 2. 18: Typical safe space design
(Modified from FEMA, 2016).

Other tornado resistant works were also studied. From FEMA, 2002 book on "Protecting School children from tornadoes"; the book highlights disaster mitigation measures put in place for students and staff to be safe during tornado in Kansas. It gives statistics on death, injuries as well as tornado damage and this information was helpful to know because it affected the criteria for safe spaces in schools. Also, Komarowski & Deming, 2000 in their article on "Safe room also highlights the use of different construction material that is light-weight construction to withstand tornado or high winds. These cases were study and proved vital in the proposed design of safe spaces.

The importance of safe spaces should not be ignored. The main purpose is to protect from death or injury. Internal safe spaces must be designed to receive design wind pressures and potential wind-borne debris impacts that are applicable to stand-alone ones. In effect, it should be assumed that the surrounding structure would not provide any shield or protection to the safe room (FEMA P-320, 2008).

2.5 WIND DESIGN SPECIFICATIONS

The American Society of Civil Engineers (ASCE 7-10) building code is used for most wind load calculations. Table 2.19 shows the steps for wind load calculations in ASCE 7-10 (Chapter 27). Figure 2.22 shows the wind speed map used for basic wind speed according to FEMA. The basic calculations for the wind loads are Velocity Pressure (q_z), Figure 2.19, Pressure on MWFRS for buildings (p) Figure 2.20, and Pressure on C&C and Attachments (p) Figure 2.21. Furthermore, The International Code Council (ICC) recommends exposure category C be used for safe spaces.

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

where:

q_z	=	velocity pressure (psf) calculated at height z above ground
K_z	=	velocity pressure exposure coefficient at height z above ground
K_{zt}	=	topographic factor
K_d	=	directionality factor = 1.0
V	=	safe room design wind speed (mph) (from Figure B3-1 or B3-2)

Figure 2. 19: Velocity Pressure Equation
(FEMA P-320, 2008).

$p = qGC_p - q_i\,(GC_{pi})$

where:

p	=	pressure (psf)
q	=	q_z for windward wall calculated at height z above ground
q	=	q_n for roof surfaces and all other walls
G	=	gust effect
C_p	=	external pressure coefficients
q_i	=	q_h = velocity pressure calculated at mean roof height
GC_{pi}	=	internal pressure coefficients

Figure 2. 20: Pressure on MWFRS on buildings
(FEMA P-320, 2008).

$p = q_h\,[(GC_p) - (GC_{pi})]$

where:

p	=	pressure (psf)
q_h	=	velocity pressure calculated at mean roof height
GC_p	=	external pressure coefficients
GC_{pi}	=	internal pressure coefficients

Figure 2. 21: Pressure on C&C and attachments
(FEMA P-320, 2008).

Table 2. 19: Steps for Wind load Calculations
(ASCE 7-10).

Table 27.2-1 Steps to Determine MWFRS Wind Loads for Enclosed, Partially Enclosed and Open Buildings of All Heights	
Step 1:	Determine risk category of building or other structure, see Table 1.4-1
Step 2:	Determine the basic wind speed, *V*, for the applicable risk category, see Figure 26.5-1A, B or C
Step 3:	Determine wind load parameters: ➢ Wind directionality factor, *Kd*, see Section 26.6 and Table 26.6-1 ➢ Exposure category, see Section 26.7 ➢ Topographic factor, *Kzt*, see Section 26.8 and Table 26.8-1 ➢ Gust Effect Factor, *G*, see Section 26.9 ➢ Enclosure classification, see Section 26.10 ➢ Internal pressure coefficient, (*GCpi*), see Section 26.11 and Table 26.11-1
Step 4:	Determine velocity pressure exposure coefficient, *Kz* or *Kh*, see Table 27.3-1
Step 5:	Determine velocity pressure *qz* or *qh* Eq. 27.3-1
Step 6:	Determine external pressure coefficient, *Cp* or *CN* ➢ Fig. 27.4-1 for walls and flat, gable, hip, monoslope or mansard roofs ➢ Fig. 27.4-2 for domed roofs ➢ Fig. 27.4-3 for arched roofs ➢ Fig. 27.4-4 for monoslope roof, open building ➢ Fig. 27.4-5 for pitched roof, open building ➢ Fig. 27.4-6 for troughed roof, open building ➢ Fig. 27.4-7 for along-ridge/valley wind load case
Step 7:	Calculate wind pressure, *p*, on each building surface ➢ Eq. 27.4-1 for rigid buildings ➢ Eq. 27.4-2 for flexible buildings ➢ Eq. 27.4-3 for open buildings

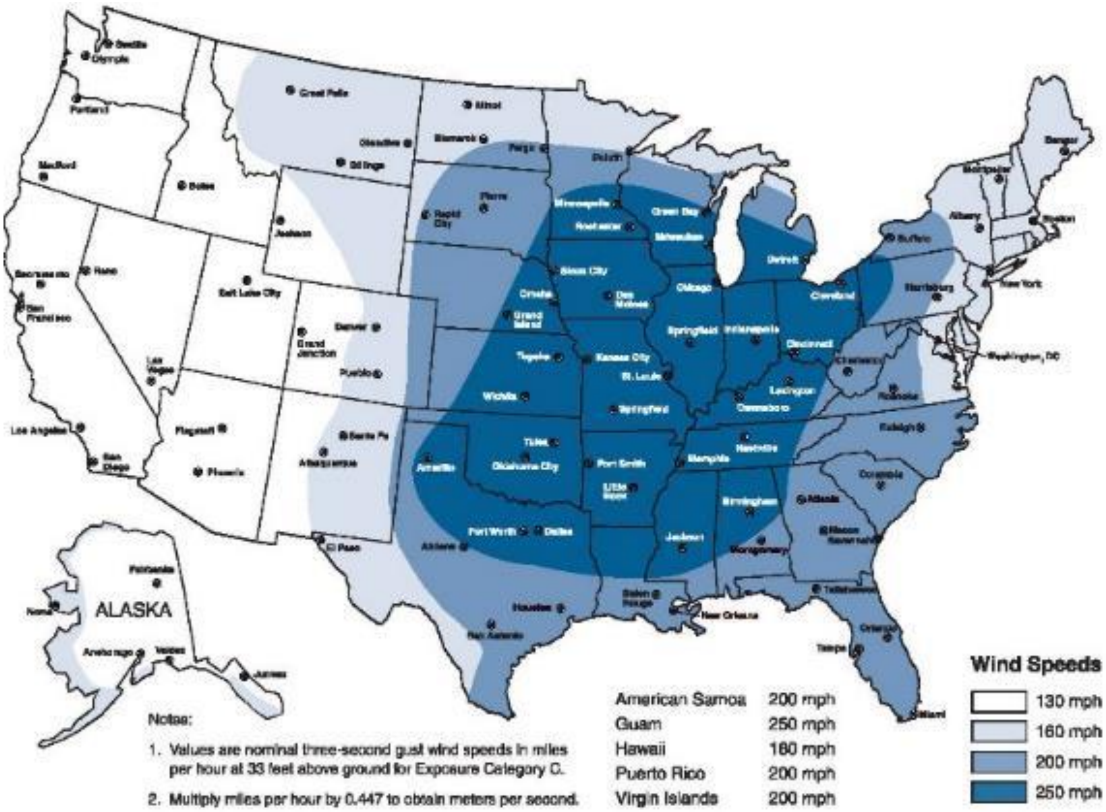

Figure 2. 22: Wind Speed map
(FEMA P-320, 2008).

2.6 SUMMARY

This chapter explored tornado activities in the US, the impact of tornadoes on hospitals, as well as the design and construction of safe spaces. St John’s Regional Medical Center was studied intensively based on building structural damages after the Joplin tornado in 2011. Other tornado resistant designs were also studied to inform the proposed safe space design. The chapter ended with a detailed explanation of ASCE 7-10’s wind design specifications. The next chapter explores the research methodology employed in achieving the proposed design of a safe space for healthcare facilities in Central US.

CHAPTER THREE

RESEARCH METHODOLOGY

3.0 INTRODUCTION

This chapter sets out to elucidate the main methods and processes which guided this research. The beginning of the chapter outlines the research methods and the research location and setting for the study. The second part looks into the architectural design for the proposed safe space design. The concluding part of this chapter looks at the structural design inputs for the study.

3.1 RESEARCH METHODS

The methodology for this research involves a precedent study of St John's Regional Medical Center now Mercy Joplin Hospital, pointing out the structural design and building codes employed. Additionally, there was a schematic design of the proposed safe space hospital design using Autodesk REVIT software. The next step involved the design development phase that consists of design of the structural system of the proposed hospital based on the precedent study. The building structure design includes the calculation of all gravity loads and lateral loads. The design is further analyzed using RAM software, a Structural Engineering Software with the linear static analysis method. Results were further verified with hand calculations.

3.1.1 RESEARCH LOCATION AND SETTING

The study location for the research was Joplin, Missouri based on the precedent study

conducted. The proposed design is located in Joplin to serve as a means of comparison to the precedent study of St. John’s Regional Medical center. Joplin is located in Missouri, 37°5′3″N 94°30′47″W and in Jasper and Newton County (Figure 3.1). According to the 2010 population census, Joplin has a population of 50,150 people, 20,680 households and 12,212 families. Given the building location, all loads such as seismic, snow and wind loads will employ the City of Joplin’s Standards.

MAP OF USA SHOWING MISSOURI

MAP SHOWING JOPLIN

Figure 3. 1: Map showing the location of Joplin

3.2 DESIGN

3.2.1 ARCHITECTURAL DESIGN

The design is a 60 ft. by 100 ft. hospital with a total height of 45 ft. Floor to floor dimension is 10 ft. Table 3.1 illustrates this.

Table 3. 1: Dimension of proposed hospital design

(Ampaw-Asiedu & Norton, 2017).

SPECIFICATIONS	MAGNITUDE
Height of Building	45 ft.
Number of Floors	4 floors
Height from Floor to Floor	10 ft.
Length of Building	100 ft.
Breadth of Building	60 ft.

Figures 3.2, 3.3 and 3.4 show a typical floor plan, the floor plan showing the safe space and the plan of the safe space, respectively. A safe space of an interior corridor/ hallway is employed for the design (Figure 3.3 and Figure 3.4). The dimensions are 6.5 ft. × 60 ft. Furthermore, Figure 3.6 and 3.7 illustrate the sections through the safe spaces and the whole building. Accessibility to the safe space by vulnerable populations is highlighted on the second level of the building (Figure 3.5).

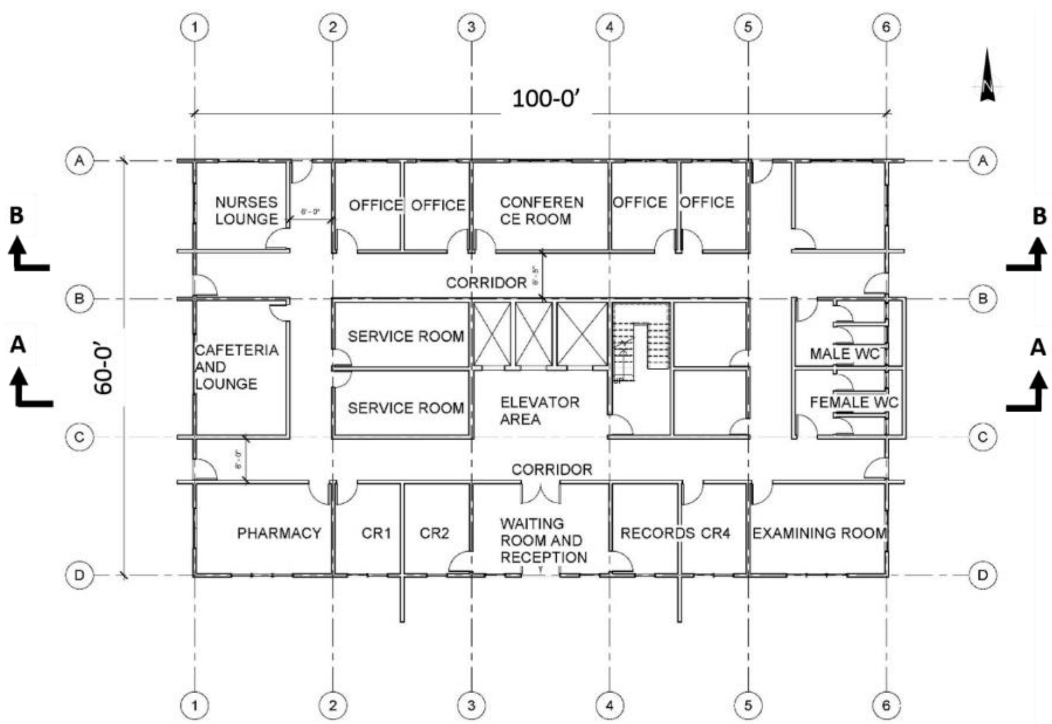

Figure 3. 2: Typical Floor plan
(Ampaw-Asiedu & Norton, 2017).

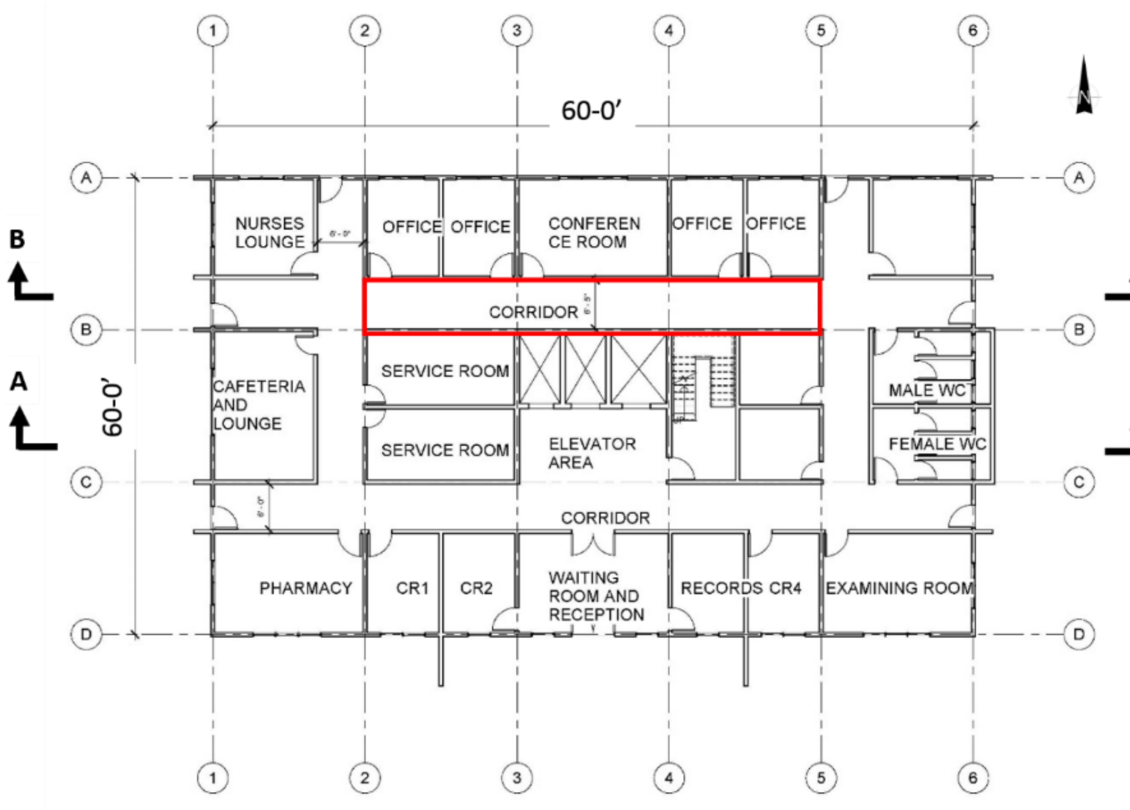

Figure 3. 3: Plan showing safe space
(Ampaw-Asiedu & Norton, 2017).

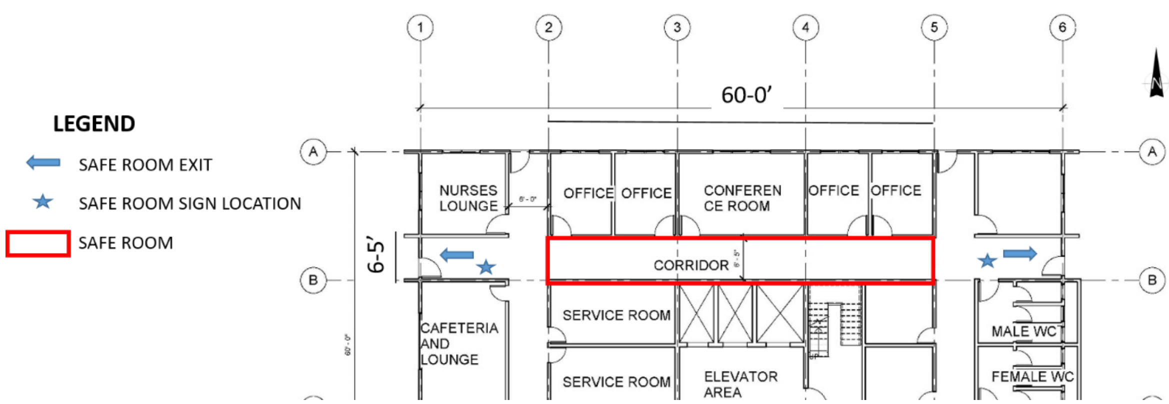

Figure 3. 4: Plan showing safe space
(Ampaw-Asiedu & Norton, 2017).

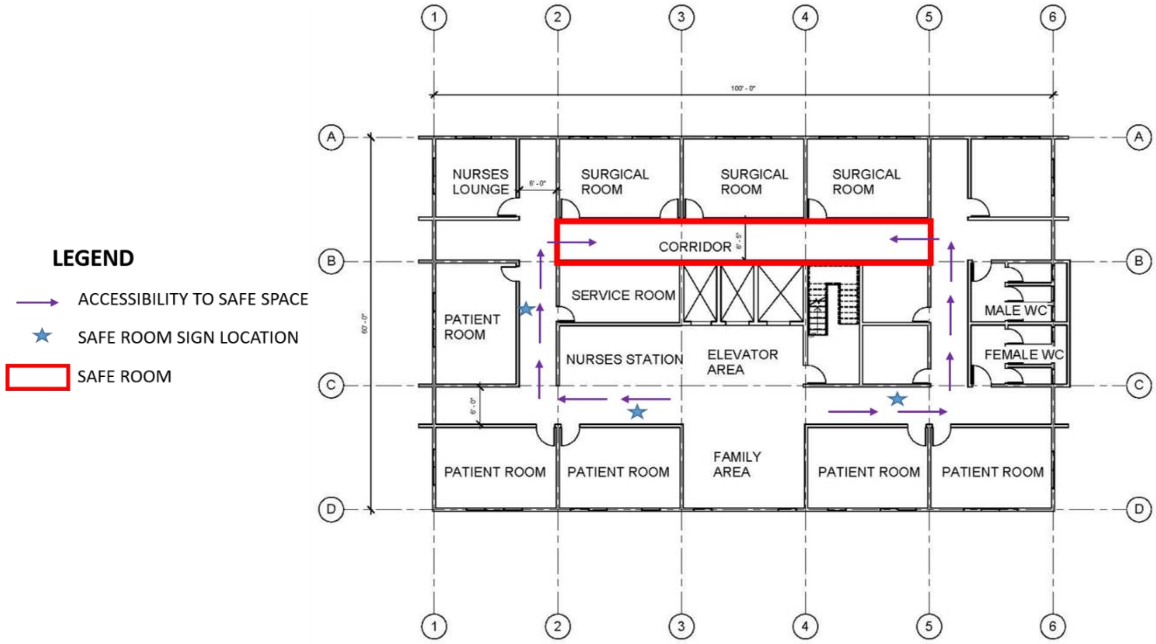

Figure 3. 5: Plan showing accessibility to safe space (Ampaw-Asiedu & Norton, 2017).

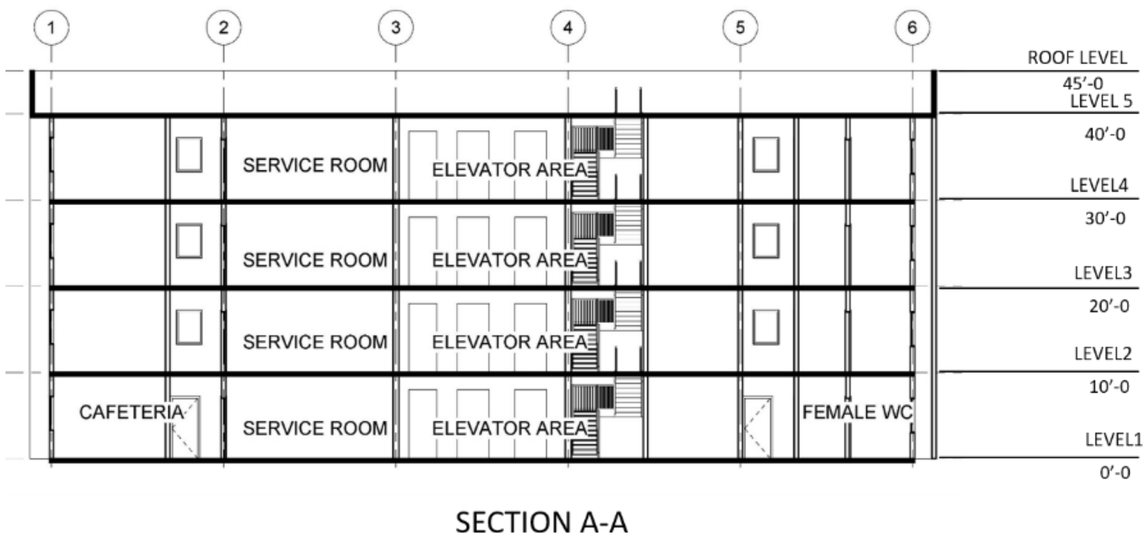

Figure 3. 6: Section through the whole building (Ampaw-Asiedu & Norton, 2017).

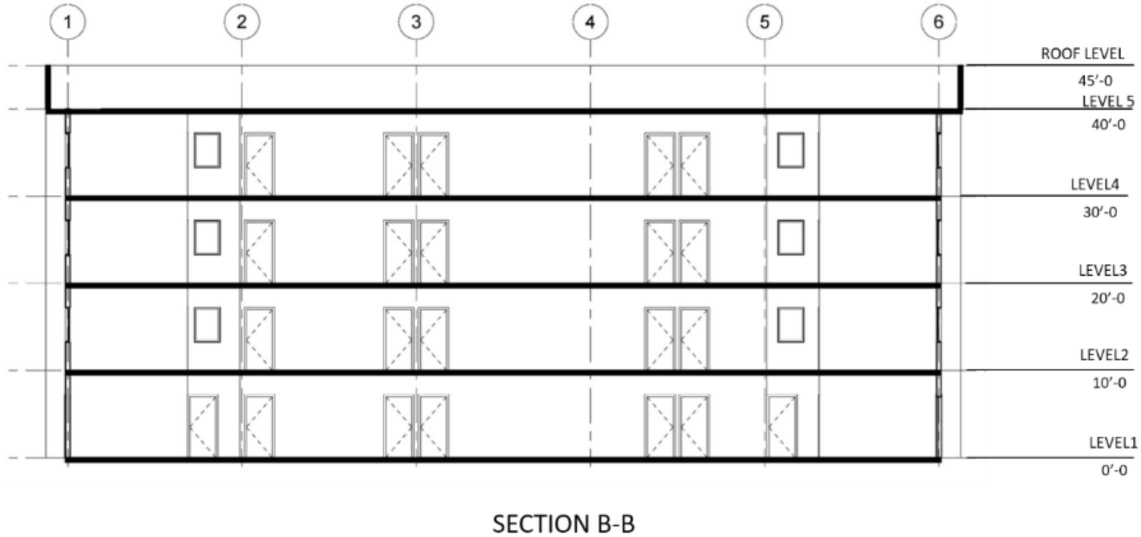

Figure 3. 7: Section through the safe space
(Ampaw-Asiedu & Norton, 2017).

3.2.2 STRUCTURAL DESIGN INFORMATION

1. APPLICABLE CODES AND STANDARDS

The governing codes used are as follows:

- American Society of Civil Engineers (ASCE) 7-10,
- American Concrete Institute (ACI) 318-11
- International Building Code (IBC) 2012
- Federal Emergency Management Agency (FEMA) P-361,2015

2. BUILDING LOADS

LOAD COMBINATIONS FOR STRENGTH DESIGN

Per ASCE 7-10, 2.3.2, the load combinations that employed are:

1. 1.4D
2. 1.2D +1.6L+0.5(Lr or S or R)

3. 1.2D + 1.6 (Lr or S or R) + (L or 0.5W)
4. 1.2D + 1.0W + L +0.5 (Lr or S or R)
5. 1.2D+1.0E+ L+0.2S
6. 0.9D+1.0W
7. 0.9 + 1.0E

D=Dead load, **Lr**= Roof live load, **L**=Live load. **R**= Rain load, **S**= Snow load and **W**=wind load

GRAVITY LOADS

- **Dead loads**

For dead loads, Table C-31 in ASCE 7-10 was used for calculations based on design components. Table 3.2 shows the estimated dead load calculations.

Table 3. 2: Estimated dead load calculations
(Ampaw-Asiedu & Norton, 2017).

ESTIMATED DEAD LOADS	
COMPONENTS	LOAD
Self-weight	As calculated
Exterior Cladding	20 psf
Roof load	25 psf
Mechanical Equipment	10 psf

- **Live loads**

For live loads, Chapter 4, Table 4.1 in ASCE 7-10 was used for calculations based on building occupancy. Table 3.3 illustrates the estimated live load calculations. Roof live load was 20 psf and 120 psf was used for the whole building.

Table 3. 3: Estimated live load calculations
(Ampaw-Asiedu & Norton, 2017).

ESTIMATED LIVE LOAD	
COMPONENTS	LOAD (psf)
Corridor and Entire building	120psf
Roof	20psf
MEP	250psf

- **Snow loads**

ASCE 7-10 (Chapter 7) was used for snow load calculations (Pf). The building employed a flat roof for the design since the roof will house the hospital's mechanical equipment. The ground snow load, Pg, for Joplin, Missouri is 19 psf. The total snow load for the hospital is 19.31 psf, which was even less than live load. Hence, snow load would not be the controlling case for gravity loads analysis, as roof live load is greater. Snow load hand calculation is shown in Appendix A.

Ground snow load as determined, based on Figure 7-1 (Pg)......= **19psf**

Exposure factor based on Table 7-2 and Category C (Ce).........= 1.1

Thermal factor based on Table 7-3 (Ct).....................................= 1.0

Importance factor based on Table 7-4 and exposure IV (I).........= 1.2

Flat roof: $\boldsymbol{p_f} = (Ce) \times (Ct) \times (I)(p_g)$
$\boldsymbol{p_f} = (0.7) \times (1.1) \times (1) \times (1.2) \times (19) = \boldsymbol{19.31psf}$

LATERAL LOADS

- **Seismic Provisions**

For seismic provisions, the risk category is Category IV, since it is a hospital (Critical facility) based on ASCE 7-10 standards in the review literature. The US Geological Survey (USGS) site (USGS, 2017) was used for calculations for Spectral accelerations at 1-second

periods (SD_1) and Spectral accelerations at 1 short periods (SD_S) (Appendix B). Hand Calculation (Appendix C) was used to verify results from USGS site. The hospital is in Seismic Design Category A based on SD_S and buildings assigned to Seismic Design Category A need only comply with the requirements of Section 1.4. Non-structural components are exempt from seismic design requirements. Based on SD_1 however, the proposed safe space is in Seismic Design Class C. This implies that, the safe space is in Seismic Design Category C and ASCE 7-10 recommends the use of an Ordinary reinforced concrete shear wall as the seismic Force-resisting system.

- **Wind loads**

In order to determine lateral loads to which the building structure is exposed to, Chapters 26, 27 and 30 of ASCE 7-10 were used. After a thorough review of these chapters, the wind input parameters were determined and are displayed in Table 3.4 and 3.5. These inputs are used to calculate pressure and base shear values to be applied to the Main Wind Force Resisting System (MWFRS) for the entire structure and the safe space design. The same input parameters are used to find components and cladding wind pressures. The basic wind speed used was 250 mph per FEMA-recommended best practices; the internal pressure coefficient is taken as 0.55 for the safe room.

Table 3. 4: Wind load parameters for whole building
(Ampaw-Asiedu & Norton, 2017).

IMPUT PARAMETERS FOR THE WHOLE BUILDING	
Risk Category	IV
Basic Wind speed, V (mph)	250
Wind Directionality Factor, Kd	1.00
Exposure Category	C
Topographic Factor, Kzt	1.00
Gust Factor, G	0.85
Enclosure Classification	Enclosed

Internal Pressure Coefficient, Gcpi	+/- 0.18

Table 3. 5: Wind load parameters for safe space
(Ampaw-Asiedu & Norton, 2017).

IMPUT PARAMETERS FOR SAFE SPACE	
Risk Category	IV
Basic Wind speed, V (mph)	250
Wind Directionality Factor, Kd	1.00
Exposure Category	C
Topographic Factor, Kzt	1.00
Gust Factor, G	0.85
Enclosure Classification	Enclosed
Internal Pressure Coefficient, Gcpi	+/- 0.55

3. STRUCTURAL SYSTEM

The whole building's structural system for construction is reinforced concrete post and beam. It consists of concrete gravity beams, columns, and a lateral load resisting system which doubles up to be the safe space is made up of a reinforced concrete shear wall. The exterior of the building will consist of a cast-in-place concrete.

4. CHECKS

The design required four checks namely:

- Axial/Flexural Check
- Bending moment
- Shear Check
- Serviceability criteria (Deflection)
- Roof and Floor Framing
  1. Maximum live load deflection ………………………………..L/360
  2. Maximum live load plus dead load deflection………………...L/240
- Elevator Framing

1. Elevator machine supports……………………………………L/1666
2. Elevator guide rail supports ……………………………………l/8”

- Lateral drift of the building

1. Wind loads………………………………………………………H/400
2. Seismic loads……………………………………………………as required by ASCE 7-10

CHAPTER FOUR

FINDINGS AND ANALYSIS

4.0 INTRODUCTION

This chapter presents the data and analysis from the RAM structural analysis software. Results are discussed according to the research questions. The main questions guiding research are:

1. What are the design specifications or requirements of safe spaces in hospitals?
2. What recommendations can be made?

The findings are mostly presented in the form of images from the RAM software and scanned images of hand calculations to verify results; Final proposed safe space design is also presented.

4.1 RESULTS AND DISCUSSIONS

- BUILDING LOADS (GRAVITY LOADS)

After running an analysis with RAM structural analysis software, the total dead load for each floor of the whole building is illustrated in Table 4.1. The highest dead load was on the first floor which came out to be 8634.75 Kips. Total dead loads for the safe space was also highest on the first floor.

Table 4. 1: Total dead loads on each floor
(Ampaw-Asiedu & Norton, 2017).

FLOORS	TOTAL DEAD LOAD (KIPS)
ROOF	1727.35
FOURTH	3452.7
THIRD	5180.05
SECOND	6907.4
FIRST	8634.75

- **BUILDING LOADS (LATERAL LOADS)**

1. **Wind loads**

Wind input parameters were calculated with an Excel sheet (Figure 4.1) and illustrated with reference to a section through the safe space (Figure 4.2). Wind loads were divided into MWFRS and wall C&C. Results from buildings C&C is illustrated in Figure 4.3. The wind parameters were also further applied to the safe space in the RAM software and the results were compared to excel sheet calculations. From the results, RAM analysis showed an approximate 1% difference with the Excel sheet calculations. Therefore, the RAM software results are used for the Total base shear. The total base shear for the safe space is 1484.10 Kips (Figure 4.4). Results on the safe space's building story shear from RAM analysis software can be found in Appendix D.

Wind load Results (E-W)						
Surface	z (ft.)	Kz	qz (psf)	Cp	p = Net Design Press. (psf) (w/ +GCpi)	(w/ -GCpi)
Windward Wall	0	0.85	115.45	0.80	52.32	104.69
	15.00	0.85	115.45	0.80	52.32	104.69
	20.00	0.90	122.66	0.80	57.22	109.59
	25.00	0.95	128.56	0.80	61.23	113.61
	30.00	0.98	133.59	0.80	64.65	117.03
	35.00	1.01	137.99	0.80	67.65	120.02
	40.00	1.04	141.93	0.80	70.32	122.70
For z = hr:	45.00	1.07	145.49	0.80	72.75	125.12
For z = he:	45.00	1.07	145.49	0.80	72.75	125.12
For z = h:	45.00	1.07	145.49	0.80	72.75	125.12
Leeward Wall	All	-	-	-0.37	-71.53	-19.16
Side Walls	All	-	-	-0.70	-112.75	-60.38
Roof (zone #1) cond. 1	-	-	-	-0.90	-137.49	-85.11
Roof (zone #1) cond. 2	-	-	-	-0.18	-48.45	3.93
Roof (zone #2) cond. 1	-	-	-	-0.90	-137.49	-85.11
Roof (zone #2) cond. 2	-	-	-	-0.18	-48.45	3.93
Roof (zone #3) cond. 1	-	-	-	-0.50	-88.02	-35.65
Roof (zone #3) cond. 2	-	-	-	-0.18	-48.45	3.93
Roof (zone #4) cond. 1	-	-	-	-0.30	-63.29	-10.91
Roof (zone #4) cond. 2	-	-	-	-0.18	-48.45	3.93

Figure 4. 1: Wind load results from Excel sheet calculations
(Ampaw-Asiedu & Norton, 2017).

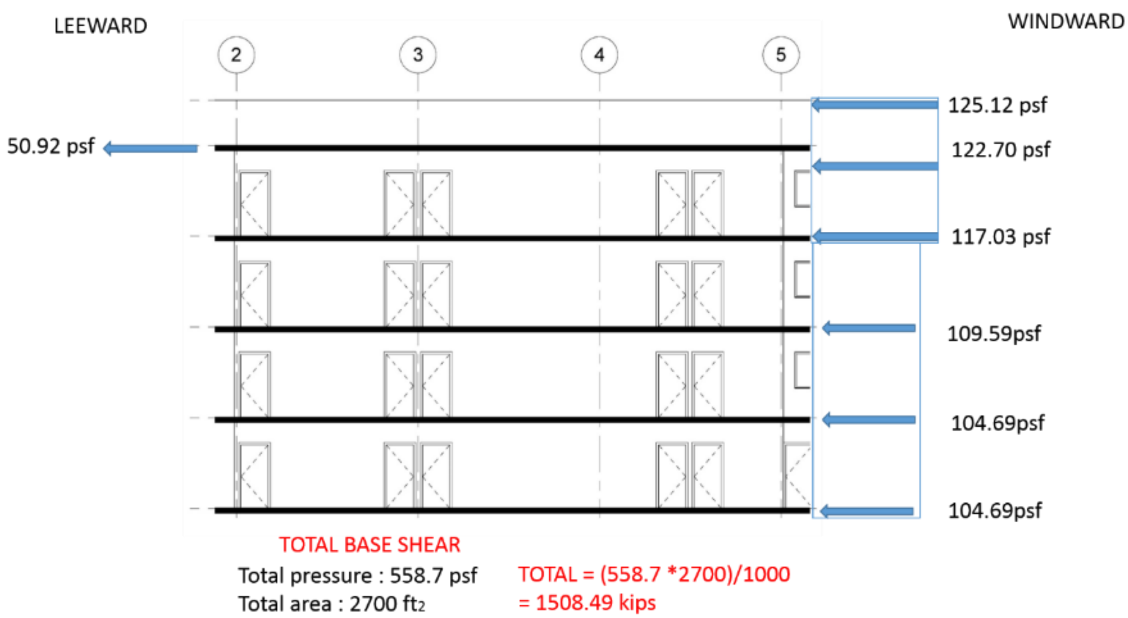

Figure 4. 2: Safe Space's building story shear based on excel sheet
(Ampaw-Asiedu & Norton, 2017).

Wind Load Tabulation for Wall Components & Cladding							
Component	z (ft.)	Kh	qh (psf)	p = Net Design Pressures (psf)			
				Zone 4 (+)	Zone 4 (-)	Zone 5 (+)	Zone 5 (-)
Wall	0	1.07	145.49	117.85	-130.94	117.85	-130.94
	15.00	1.07	145.49	117.85	-130.94	117.85	-130.94
	20.00	1.07	145.49	117.85	-130.94	117.85	-130.94
	25.00	1.07	145.49	117.85	-130.94	117.85	-130.94
	30.00	1.07	145.49	117.85	-130.94	117.85	-130.94
	35.00	1.07	145.49	117.85	-130.94	117.85	-130.94
	40.00	1.07	145.49	117.85	-130.94	117.85	-130.94
For z = hr:	45.00	1.07	145.49	117.85	-130.94	117.85	-130.94
For z = he:	45.00	1.07	145.49	117.85	-130.94	117.85	-130.94
For z = h:	45.00	1.07	145.49	117.85	-130.94	117.85	-130.94

Figure 4. 3: Wind loads of wall Components and Cladding (Ampaw-Asiedu & Norton, 2017).

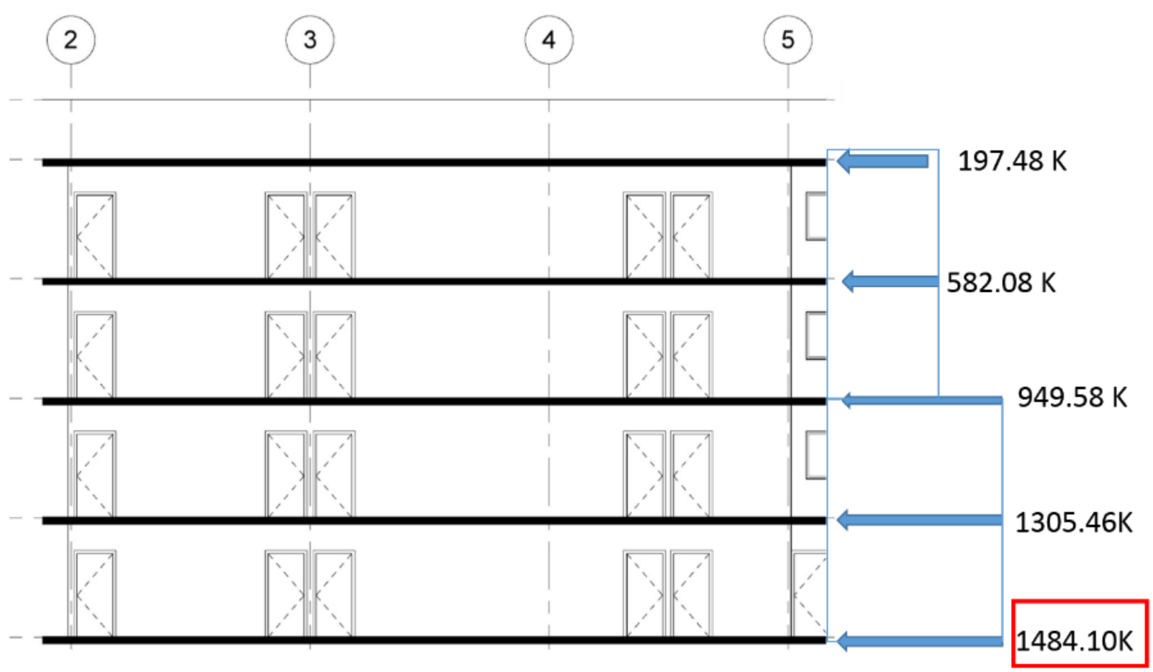

Figure 4. 4: Total base shear from RAM analysis software (Ampaw-Asiedu & Norton, 2017).

2. **Seismic loads**

Based on the seismic calculations on Appendix C, the proposed facility was found to be in Seismic Design Class C. According to ASCE 7-10, an ordinary reinforced concrete shear wall can be used as a Seismic Force Resisting System. Total base shear for the seismic load is 286.58 Kips. Figure 4.5 illustrates the seismic story shear. Appendix E shows results from RAM structural analysis software and Appendix F illustrates Seismic load Calculations.

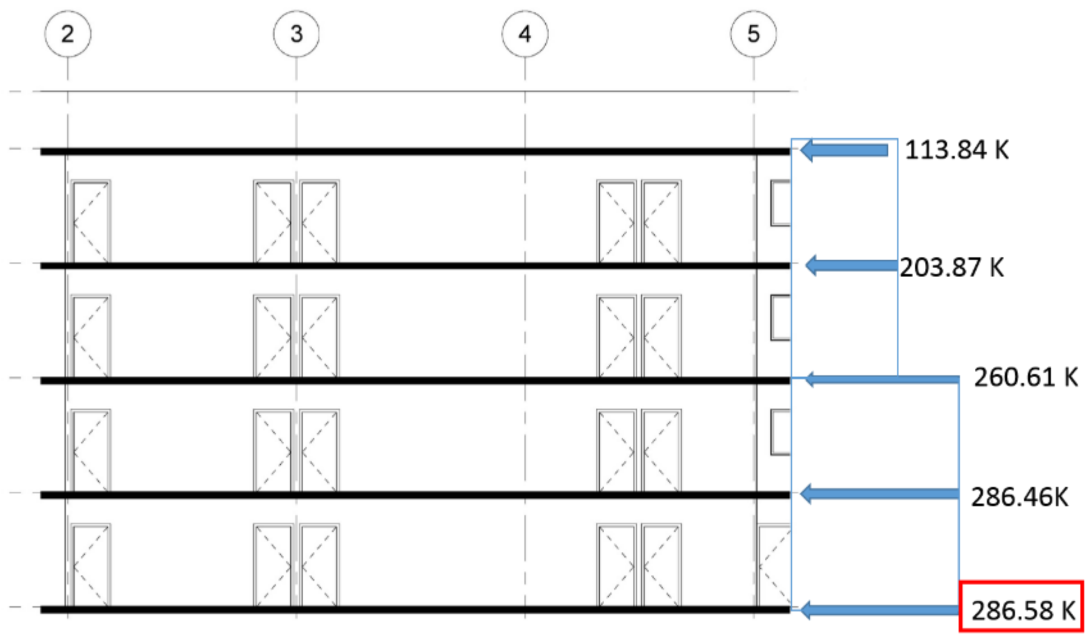

Figure 4. 5: Total Seismic Base Shear.
(Ampaw-Asiedu & Norton, 2017).

- **GRAVITY SYSTEM**

Gravity system for the whole building is a concrete beam and column. Figures 4.6 and 4.7 illustrate the three-dimensional (3-D) view and the plan view of the gravity system.

1. Concrete Columns

Column properties are in Table 4. 2 and Appendix G. Cracked properties are bending (0.7), Axial (1.0) and Torsion (1.00).

Table 4. 2: Column Properties
(Ampaw-Asiedu & Norton, 2017).

Concrete Type	**F'c (ksi)**	**Concrete Weight (pcf)**	**Concrete Modulus (ksi)**	**Fy longitudinal & Transverse (ksi)**	**Reinforcement Modulus (Ksi)**
Normal weight concrete	4.00	145	3645	ASTM A615, Grade 60	29000

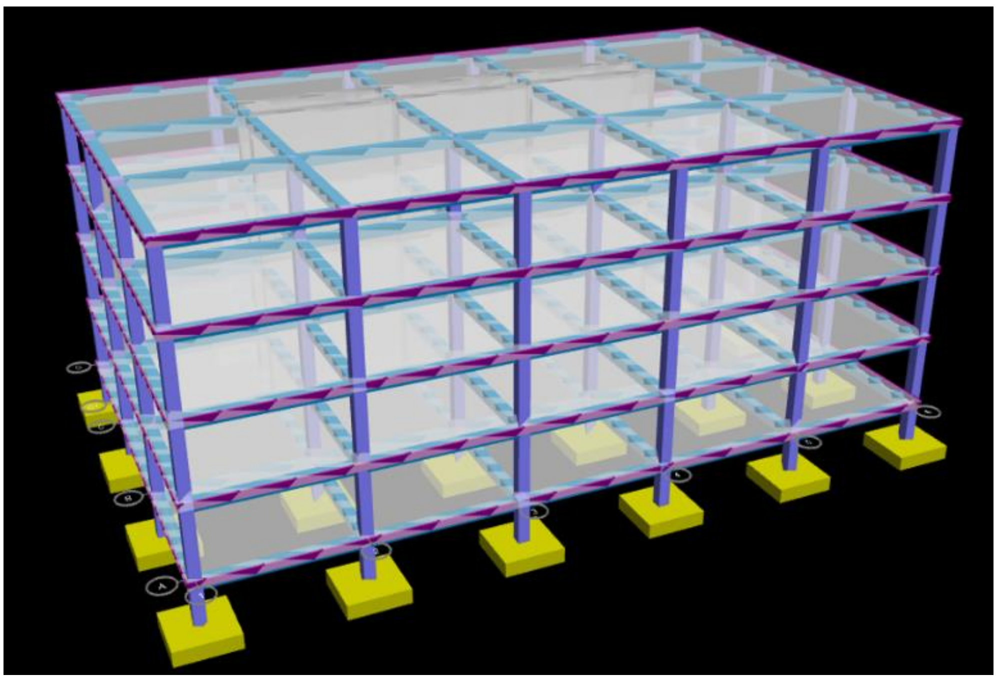

Figure 4. 6: 3-D of gravity system
(Ampaw-Asiedu & Norton, 2017).

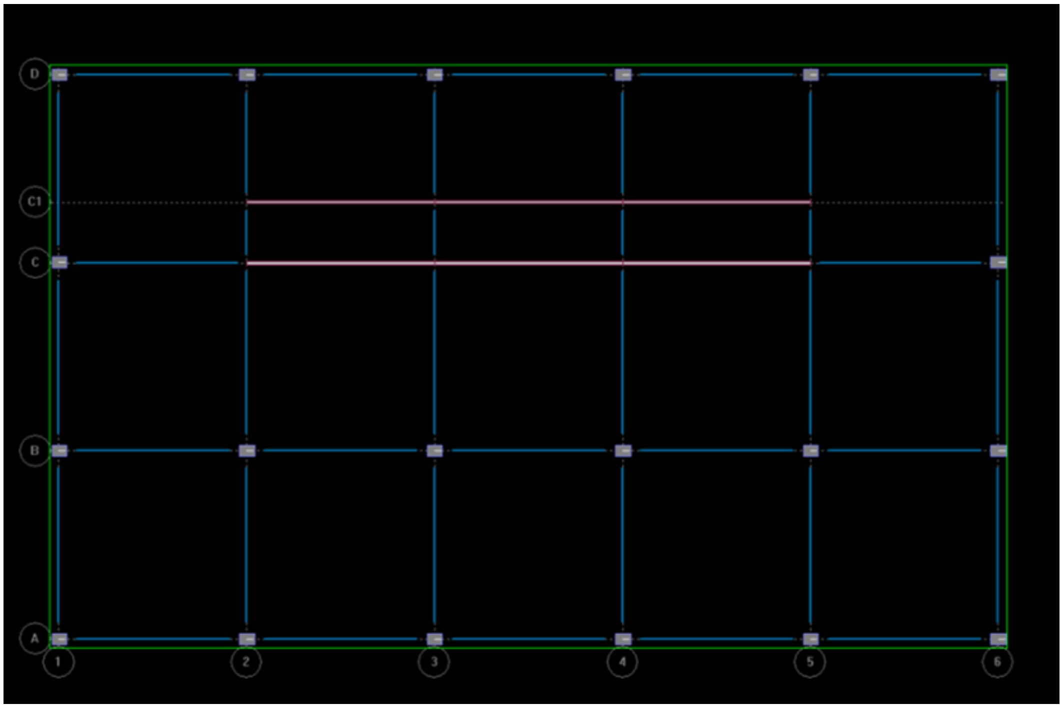

Figure 4. 7: Plan view of gravity system
(Ampaw-Asiedu & Norton, 2017).

Different column sizes where experimented based on the capacity. Column sizes 12in x 12 in, 14in x 14in and 16in x16in failed or did not work based on the capacity of the building. However, column size 18in x 18in was perfect for the design. A column stability check was further performed and none of the columns failed (Figure 4.8). For example, on level 4, the column number (3) on Grid location 1-C had longitudinal reinforcement of 12#5 and transverse reinforcement of #4@ 9' and area of steel reinforcement (As) is 3.72in^2. The standard for axial force, øPn ≥ Pu, was met, as was that for shear øVn ≥Vu. Therefore, the design is okay thus, the nominal is greater than the ultimate. A drawing is provided in Appendix F to illustrate this example. Column design satisfied all the checks, that is, deflection, moment, axial/Flexural and shear. Appendix G shows a typical column design on each floor.

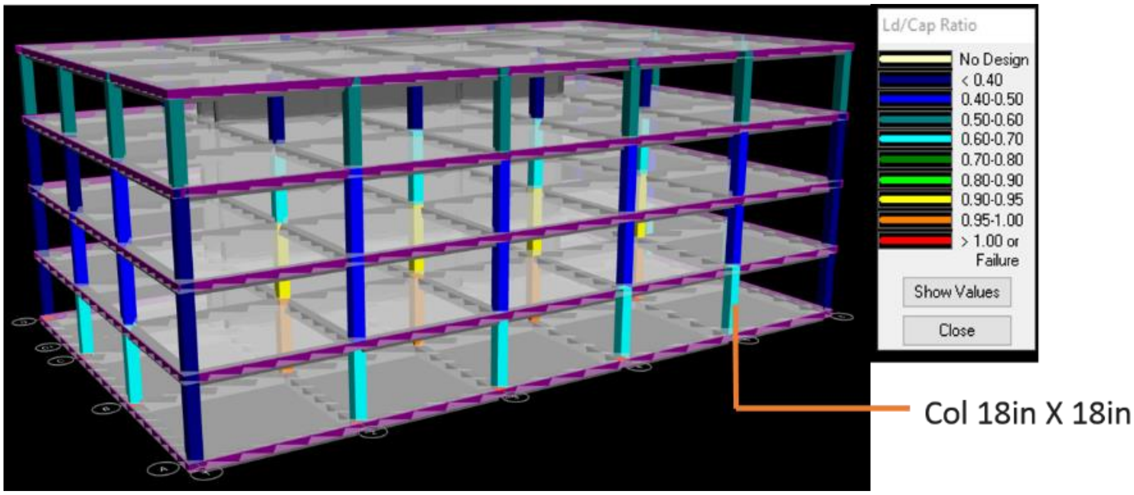

Figure 4. 8: Column Stability Check
(Ampaw-Asiedu & Norton, 2017).

2. Concrete beams and slab

F’c of the concrete beam is 4000 psi. Different beam sizes where tested based on the capacity. Beam sizes 12 in x12 in and 14in x 14in were run in the RAM analysis software and failed. However, beam size 16in x 16in was good for the facility. Beam design satisfied deflection, shear and moment checks. A 12in-thick reinforced concrete slab is used for all the floors excluding the roof level. The roof level has a slab of 16in-thick reinforced concrete because it houses the mechanical equipment. Figure 4.9 illustrates the beam and slab for the facility. A sample concrete beam detail can be found in Appendix H.

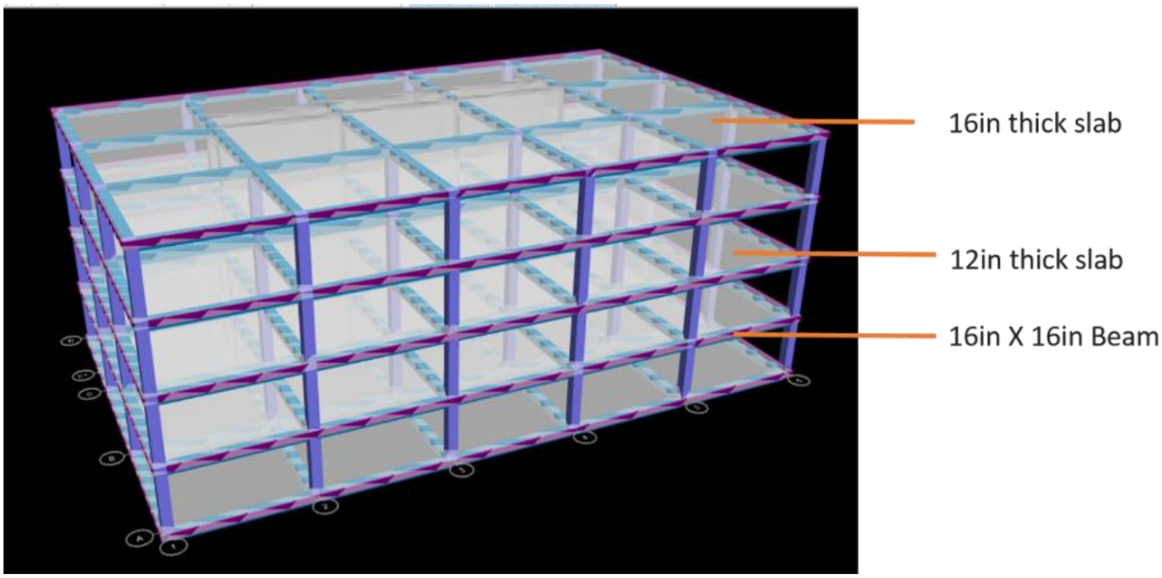

Figure 4. 9 : Beam size and slab thickness
(Ampaw-Asiedu & Norton, 2017).

- **LATERAL SYSTEM**

The lateral system for the building which doubles up to be the safe space is a reinforced concrete shear wall, which is designed as the main lateral load resisting system of the facility. The concrete shear wall also acts as a basic seismic force resisting system. Shear wall has f'c of 4000 psi and Fy of 60 ksi. The reinforced concrete shear wall has a thickness of 16in (Appendix I) after the thickness of 10in, 12in, 14in and 15in failed (Appendix I). All the levels of the concrete shear wall were checked for Axial/Flexural, Shear, Reinforcement; and all the levels passed the test. Figure 4.10 shows the reinforcement distribution. In addition, Appendix I also shows the Axial/Flexural, shear and reinforcing for the first level, which has the highest total dead load. Hand calculations (Appendix J) were used for shear wall verification for the bar size utilized. Flexural and shear strength were checked and both conditions were satisfied based on ACI 318-11. Finally, the 16in thick structural wall and #18 bars for both vertical and

horizontal reinforcement has adequate moment and shear strength and the reinforcement satisfies ACI code requirements.

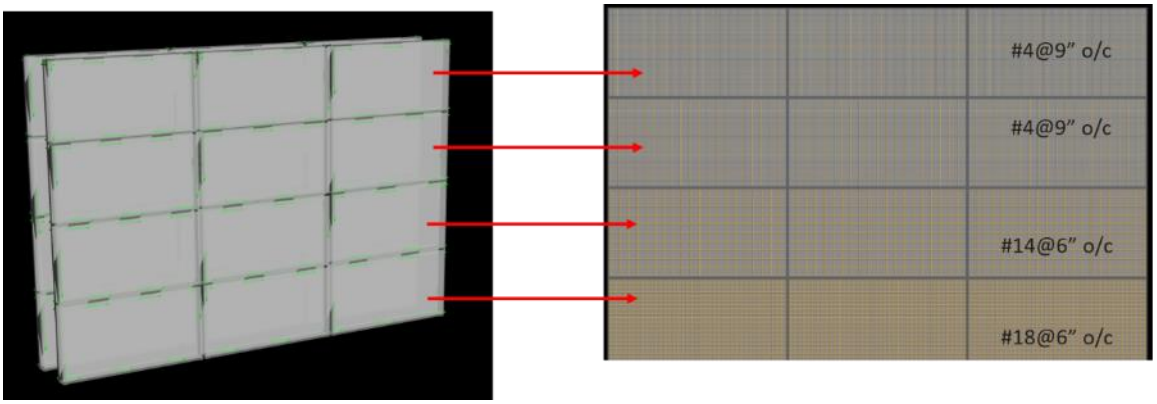

Figure 4. 10: Reinforcement distribution
(Ampaw-Asiedu & Norton, 2017).

- **CONNECTIONS**

Roof components and cladding were also checked and Figure 4.11 shows roof uplift pressure acting on the roof in red. Excel sheet was used for the calculations of the roof components and cladding (Figure 4.11).

Wind Load Tabulation for Roof Components & Cladding							
Component	z	Kh	qh	p = Net Design Pressures (psf)			
	(ft.)		(psf)	Zone 1,2,3 (+)	Zone 1 (-)	Zone 2 (-)	Zone 3 (-)
0	0	1.07	145.49	55.29	-157.13	-186.23	-186.23
	15.00	1.07	145.49	55.29	-157.13	-186.23	-186.23
	20.00	1.07	145.49	55.29	-157.13	-186.23	-186.23
	25.00	1.07	145.49	55.29	-157.13	-186.23	-186.23
	30.00	1.07	145.49	55.29	-157.13	-186.23	-186.23
	35.00	1.07	145.49	55.29	-157.13	-186.23	-186.23
	40.00	1.07	145.49	55.29	-157.13	-186.23	-186.23
For z = hr:	45.00	1.07	145.49	55.29	-157.13	-186.23	-186.23
For z = he:	45.00	1.07	145.49	55.29	-157.13	-186.23	-186.23
For z = h:	45.00	1.07	145.49	55.29	-157.13	-186.23	-186.23

Figure 4. 11: Excel sheet calculations on roof components and cladding
(Ampaw-Asiedu & Norton, 2017).

The roof to wall connection of the proposed safe space design is made up of a dowel bar of #4@6" lapped with #18 vertical bars (Figure 4.12).

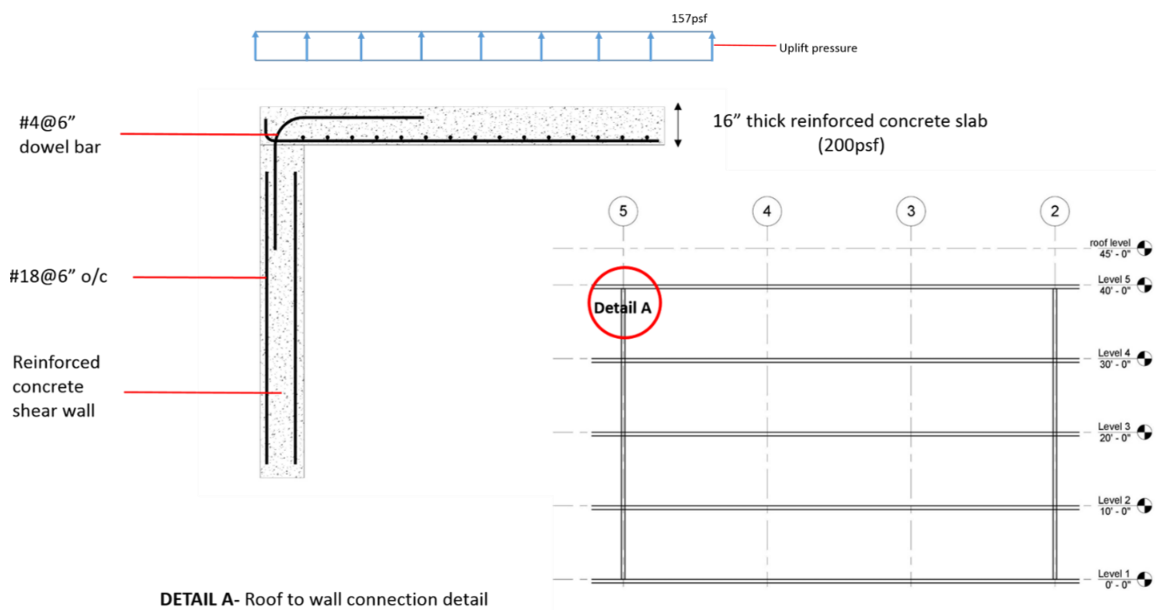

Figure 4. 12: Roof to wall connection detail
(Ampaw-Asiedu & Norton, 2017).

4.3 FINAL DESIGN

Main Wind Force Resisting System (MWFRS) consist of a gravity system made up of beams and columns (Figure 4.13). The lateral system for the building is a concrete shear wall, which is designed as the main lateral load resisting system of the facility and doubles up as the proposed safe space (Figure 4.14). These reinforced concrete shear walls were designed around elevators and at the back corridors with FEMA rated doors (Figure 4.15). The proposed safe space design is therefore an interior corridor made up of a reinforced concrete shear wall with FEMA rated doors and a roof connection of #4@6" dowel bar lapped with #18 vertical bars (Figure 4.12). The highest story displacement was found about the y-axis (Appendix K), and the thickness of the concrete shear wall

was selected as 16 in and verified with hand calculations (Appendix J). The shear wall also acts as the Basic Seismic Force resisting system. Table 4 illustrates the design summary of the facility.

Table 4. 3: Design summary of the safe space
(Ampaw-Asiedu & Norton, 2017).

MWFRS	FLOOR SYSTEM	C& C
Cast in place reinforced concrete wall with a 16in reinforced concrete shear wall acting as a lateral resisting system and an interior safe space. Gravity system made up of 18in x 18in columns and 16in x 16in beams.	12in thick Reinforced concrete slab for all floors except the roof which uses 16in.	Glass panels with Aluminum framing and a roof connection of #4@6" dowel bar lapped with #18 vertical bars.

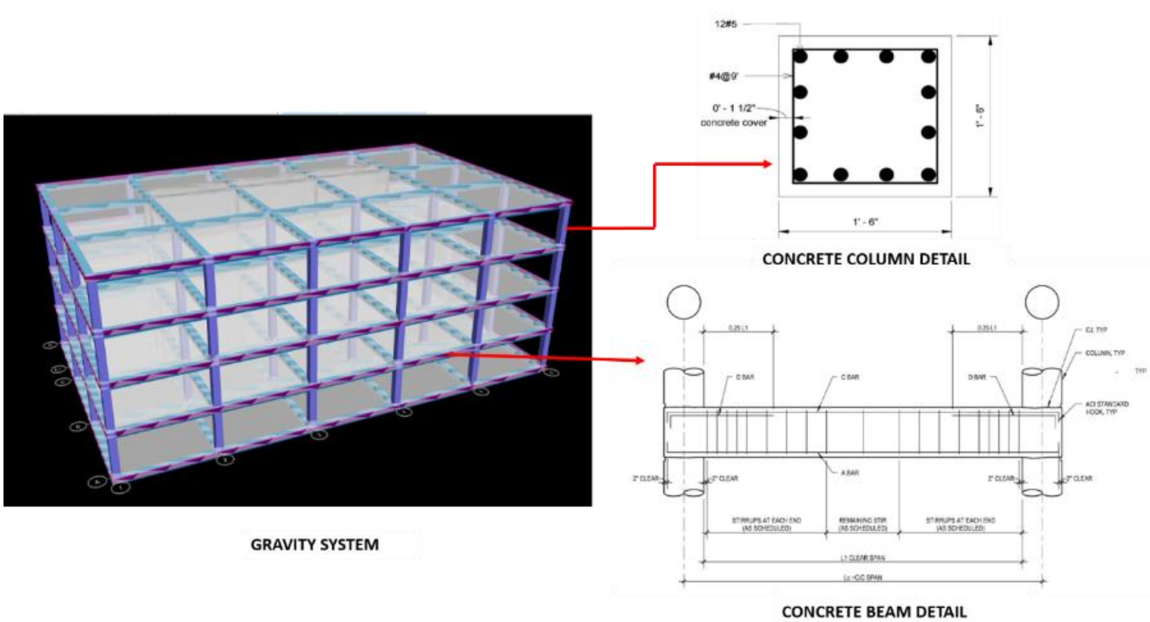

Figure 4. 13: Gravity system of the Safe space
(Ampaw-Asiedu & Norton, 2017).

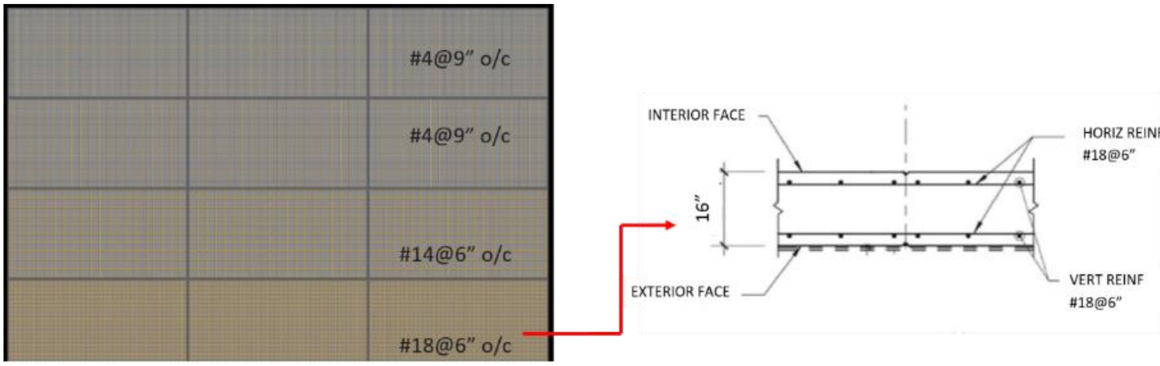

Figure 4. 14: Lateral System of the Safe space
(Ampaw-Asiedu & Norton, 2017).

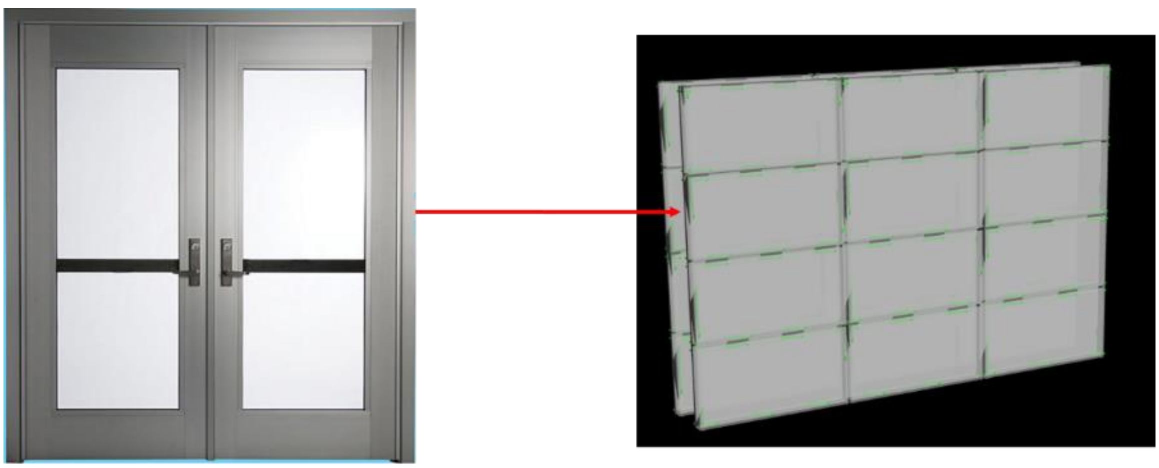

Figure 4. 15: FEMA rated door used for the safe space
(Ampaw-Asiedu & Norton, 2017).

CHAPTER FIVE

CONCLUSION AND RECOMMENDATIONS

5.0 INTRODUCTION

After a thorough literature review on the structural design of safe spaces in hospitals vulnerable to tornadoes, the next step is to recommend effective ways to reduce the vulnerability of hospital building structures to disasters. This chapter therefore highlights the major findings the study revealed after data was analyzed. The results will be used as a recommendation for hospitals in tornado-prone areas.

5.1 LIMITATIONS OF THE STUDY

This thesis has the following limitations,

1. Hazards considered are based on geographical area.
2. Calculations on wind design, snow and seismic considerations are site specific.
3. These parameters will however change based on location and what hazards are likely in that area.

5.2 PROPOSAL BASED ON MAJOR FINDINGS

From the outcome of the case study of St John's Regional Medical Center, the review of FEMA P-361 and the wind design specification requirement revealed some structural propositions significant for the design of safe spaces, one of which included the spatial configuration of the safe spaces. The proposed design employed an interior corridor /hallway as the safe space. In addition, the study revealed that, the new building

incorporated construction materials outlined by FEMA and the safe zones were designed according to high-winds capacity. Furthermore, incorporation of FEMA rated doors, designing the MWFRS, Component and Cladding (C&C), roof to wall connection, floor system as well as choosing an appropriate construction material is key for the overall design of such a facility. Hence, due attention was given these areas in the proposed design of safe spaces. Finally, the proposed safe space which is an interior corridor would help keep vulnerable patients safe during an impending storm.

In addition, based on the literature review on the "Emergency Preparedness rule", this thesis can contribute by providing information (design of a safe space in hospitals) to prevent building loss during a particular hazard. The proposed safe space is designed to remain functional in the event of a disaster hence; the design will satisfy the requirements for risk assessment and emergency planning. Finally, having this safe space design as part of the emergency preparedness plan will help meet Health care providers under Medicare and Medicaid requirement for certification.

5.3 RECOMMENDATIONS

Recommendations are based on the findings of the study. It is anticipated that these recommendations would help reduce the vulnerability of hospital structure, patients, and healthcare workers to disasters. The issues raised immediately point to a number of recommendations and actions that should be considered. These include:

1. Safe spaces in hospitals should be designed according to FEMA standards and should be incorporated in hospitals which have shelter in place to reduce loss of lives of those who are in the hospital or those who come to seek shelter.
2. Safe spaces should be designed with high winds. FEMA recommends 250mph and the safe space must resist missile impact loads.
3. Existing facilities without any disaster mitigation measures or without safe spaces should be retrofitted to house safe spaces because they can accommodate occupants and insulate them against near-absolute or absolute life safety threats for about 2 hours. Also, having safe spaces in a hospital reduce the vulnerability of people housed in the facility.
4. In the design of a safe space, special attention should be given to the lateral load resisting systems of the building since it proved vital in making the safe space resistant to the tornado impact.
5. In addition, FEMA-rated doors should be added to protect people housed in the safe space from wind-borne debris impact. Interior safe spaces should be designed as a standalone safe space in order to assume the worst-case scenario possible.
6. Roof to wall connection should also be designed to prevent the roof from tearing off the building structure.

5.4 FUTURE WORK

Future work should consider:

1. Looking into other design parameters for the design of safe spaces such as comparing a single safe space to a multi-use safe space, cost and construction of safe spaces and debris impact resistant design criteria.
2. Expanding the scope of design to other facilities such as nursing homes, schools and other critical facilities.
3. Extending the design of safe spaces to other disaster types such as hurricanes, Earthquakes and floods.
4. Making use of other construction materials such as masonry or steel for safe space design.

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LIST OF APPENDICES

APPENDIX A: SNOW LOAD CALCULATION

LUCY AMPAW-ASIEDU THESIS

SNOW LOADS

ASCE 7-10 (Chapter 7)

Flat roof snow load, P_f

$$P_f = 0.7\,C_e\,C_t\,I_s\,P_g$$

$P_g = 19$ psf

C_e = Based on exposure category C and enclosed

$C_e = 1.1$

$C_t = 1.0$

$I_s = 1.2$

$$\therefore P_f = 0.7(1.0)(1.1)(1.2)(19)$$

$$= \boxed{19.31 \text{ psf}}$$

APPENDIX B: SEISMIC INFORMATION

USGS Design Maps Summary Report

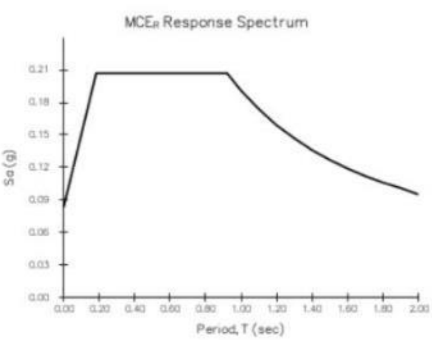
User–Specified Input

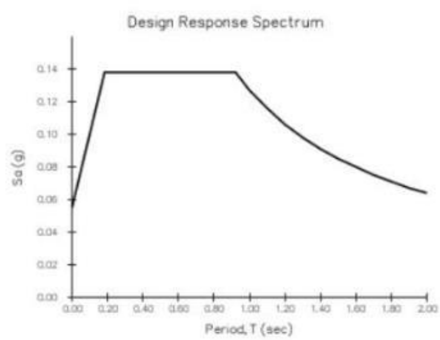
Report Title	JOPLIN Fri February 23, 2018 23:21:07 UTC
Building Code Reference Document	ASCE 7-10 Standard (which utilizes USGS hazard data available in 2008)
Site Coordinates	37.08476°N, 94.5134°W
Site Soil Classification	Site Class D – "Stiff Soil"
Risk Category	IV (e.g. essential facilities)

USGS–Provided Output

$S_S =$ 0.129 g	$S_{MS} =$ 0.207 g	$S_{DS} =$ 0.138 g
$S_1 =$ 0.080 g	$S_{M1} =$ 0.191 g	$S_{D1} =$ 0.127 g

For information on how the SS and S1 values above have been calculated from probabilistic (risk-targeted) and deterministic ground motions in the direction of maximum horizontal response, please return to the application and select the "2009 NEHRP" building code reference document.





For PGA_M, T_L, C_{RS}, and C_{R1} values, please view the detailed report.

APPENDIX C: SEISMIC LOAD CALCULATIONS

LUCY AMPAW-ASIEDU THESIS

SEISMIC PROVISIONS

1) Risk Category IV

2) Site Class D

3) Using http://earthquake.usgs.gov//hazards/designmaps/.

$S_s \Rightarrow 0.129g$

$S_1 \Rightarrow 0.080g$

4) $S_{ms} = F_a S_s$
$= 1.6(0.129g)$
$= 0.207g$

$S_{m1} = F_v S_1$
$= 2.4(0.080g)$
$= 0.191g$

5) $S_{DS} = \left(\frac{2}{3}\right) S_{ms} = \frac{2}{3}(0.207) = \boxed{0.138g}$

$S_{D1} = \left(\frac{2}{3}\right) S_{m1} = \frac{2}{3}(0.191) = \boxed{0.127g}$

6) Important Factor I_e
Since the risk category is $IV = I_e = 1.50$

7) Seismic Design Category
$\Rightarrow$ Based on short periods
• $S_{DS} < 0.167 \Rightarrow$ Seismic Design Category Class A

$\Rightarrow$ Based on 1-S periods $\Rightarrow$ Seismic Design Category
$S_{D1} > 0.067$ Class C

1/2

APPENDIX C: SEISMIC LOAD CALCULATIONS

LUCY AMPAW-ASIEDU THESIS

s) Building Frame Systems.

⇒ Seismic Design Category A

Buildings assigned to Seismic Design Category A need only comply with the requirements of section 1.4. Non-structural components are exempt from seismic design requirements

⇒ Seismic Design Category C

Use Ordinary reinforced concrete shear walls as Seismic Force-Resisting System.

APPENDIX D: WIND STORY SHEARS

Building Story Shears

RAM Frame 15.04.00.000 | Page 2/8
DataBase: WHOLEsafe space new | 02/17/18 15:50:21

FOURTH FLOOR	0.00	-0.00	-0.00	-0.00
THIRD	0.00	0.00	-0.00	-0.00
SECOND	0.00	0.00	-0.00	-0.00
FIRST	0.00	0.00	-0.01	-0.01

Load Case: W1 WIND Wind_ASCE710_1_X

Level	Diaph. #	Shear-X kips	Shear-Y kips
ROOF	1	44.40	0.00
FOURTH FLOOR	1	130.61	-0.00
THIRD	1	212.55	-0.00
SECOND	1	291.58	-0.00
FIRST	1	331.23	-0.00

Summary - Total Story Shears

Level	Shear-X kips	Change-X kips	Shear-Y kips	Change-Y kips
ROOF	44.40	44.40	0.00	0.00
FOURTH FLOOR	130.61	86.21	-0.00	-0.00
THIRD	212.55	81.94	-0.00	-0.00
SECOND	291.58	79.03	-0.00	-0.00
FIRST	331.23	39.64	-0.00	-0.00

Load Case: W2 WIND Wind_ASCE710_1_Y

Level	Diaph. #	Shear-X kips	Shear-Y kips
ROOF	1	0.00	197.48
FOURTH FLOOR	1	0.00	582.08
THIRD	1	-0.00	949.58
SECOND	1	-0.00	1305.46
FIRST	1	0.00	1484.10

Summary - Total Story Shears

Level	Shear-X kips	Change-X kips	Shear-Y kips	Change-Y kips
ROOF	0.00	0.00	197.48	197.48
FOURTH FLOOR	0.00	-0.00	582.08	384.60
THIRD	-0.00	-0.00	949.58	367.50
SECOND	-0.00	0.00	1305.46	355.88
FIRST	0.00	0.00	1484.10	178.65

Load Case: W3 WIND Wind_ASCE710_2_X+E

Level	Diaph. #	Shear-X kips	Shear-Y kips
ROOF	1	33.30	0.00

APPENDIX E: SEISMIC STORY SHEARS

Building Story Shears

RAM Frame 15.04.00.000 Page 7/8
DataBase: WHOLEsafe space new 02/17/18 15:50:21

FOURTH FLOOR	1	197.01	-0.00
THIRD	1	256.45	-0.00
SECOND	1	286.31	-0.00
FIRST	1	286.58	-0.00

Summary - Total Story Shears

Level	Shear-X kips	Change-X kips	Shear-Y kips	Change-Y kips
ROOF	108.01	108.01	-0.00	-0.00
FOURTH FLOOR	197.01	89.00	-0.00	-0.00
THIRD	256.45	59.43	-0.00	-0.00
SECOND	286.31	29.86	-0.00	-0.00
FIRST	286.58	0.27	-0.00	-0.00

Load Case: E2 SEISMIC EQ_ASCE710_X_-E_F

Level	Diaph. #	Shear-X kips	Shear-Y kips
ROOF	1	108.01	-0.00
FOURTH FLOOR	1	197.01	-0.00
THIRD	1	256.45	-0.00
SECOND	1	286.31	-0.00
FIRST	1	286.58	-0.00

Summary - Total Story Shears

Level	Shear-X kips	Change-X kips	Shear-Y kips	Change-Y kips
ROOF	108.01	108.01	-0.00	-0.00
FOURTH FLOOR	197.01	89.00	-0.00	-0.00
THIRD	256.45	59.43	-0.00	-0.00
SECOND	286.31	29.86	-0.00	-0.00
FIRST	286.58	0.27	-0.00	-0.00

Load Case: E3 SEISMIC EQ_ASCE710_Y_+E_F

Level	Diaph. #	Shear-X kips	Shear-Y kips
ROOF	1	0.00	113.84
FOURTH FLOOR	1	-0.00	203.87
THIRD	1	-0.00	260.61
SECOND	1	-0.00	286.46
FIRST	1	0.00	286.58

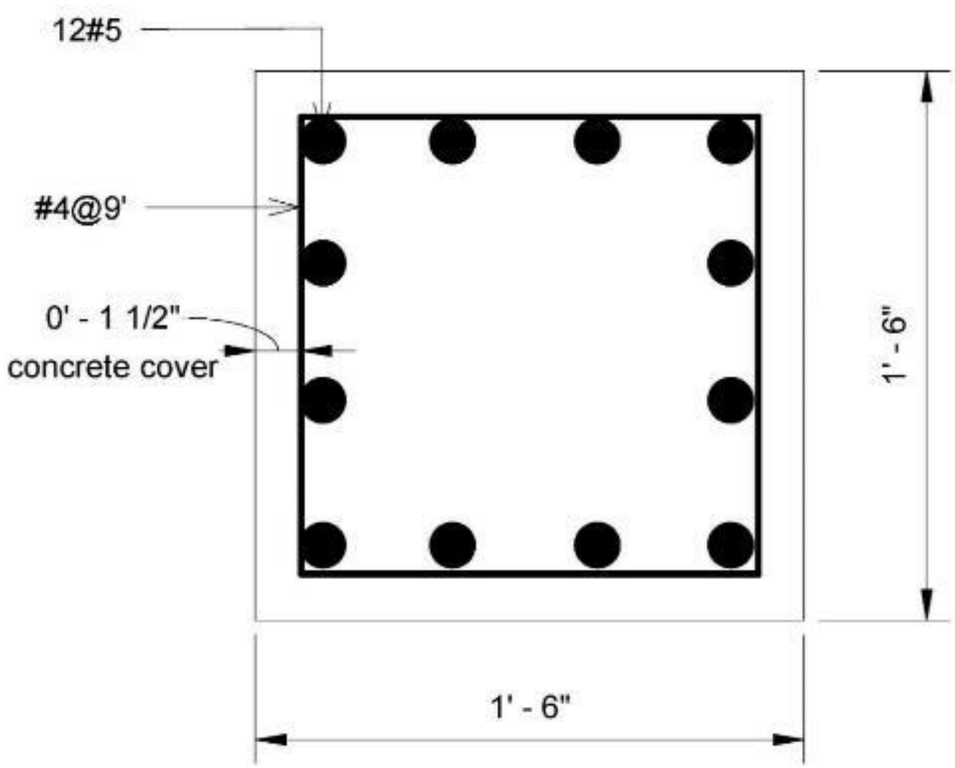
Summary - Total Story Shears

Level	Shear-X kips	Change-X kips	Shear-Y kips	Change-Y kips
ROOF	0.00	0.00	113.84	113.84

APPENDIX F: SEISMIC LOAD CALCULATIONS

FLOORS	TOTAL DEAD LOAD (KIPS)	TOTAL LATERAL FORCE (KIPS)
ROOF	1727.35	113.84
FOURTH	3452.7	203.87
THIRD	5180.05	260.61
SECOND	6907.4	286.46
FIRST	8634.75	286.58

APPENDIX G: CONCRETE COLUMN DESIGN



APPENDIX G: CONCRETE COLUMN DESIGN

Concrete Column Design

RAM Concrete Column v15.04.00.000	Page 20/120
Database: WHOLEsafe space new	02/23/18 15:02:53
Building Code: IBC	Concrete Code: ACI 318-11

COLUMN INFORMATION:

Level	ROOF		
Column Number:	1	Grid Location:	(1-A)
Size:	**COLUMN**	Depth x Width (in)	18.00x18.00
Reinforcement			
Longitudinal:	**12-#5 (4 x 2)**	As (in^2)	3.72 (1.15%)
Transverse:	**#4@ 9.0" 0'-0"-10'-0"**		
Confinement	Tie	Clear Cover (in)	1.50
Shear Legs Major	2	Shear Legs Minor	2

Longitudinal Bars Max Tension Stress Ratio: 1.00

MATERIAL PROPERTIES:

f'c (ksi):	4.00	fy Long (ksi):	60.00
fct (ksi):	0.00	fyt Trans (ksi):	60.00
Conc. Weight (pcf):	145.00	Conc. Type:	NWC
Conc. Modulus (ksi):	3644.15	Reinf. Modulus (ksi):	29000.00

DESIGN PARAMETERS:

	Major	Minor
Unbraced Length (ft)	8.67	8.67
K	2.84	2.84
Braced Against Sidesway	No	No

LONGITUDINAL REINFORCEMENT:

Controlling Load Combination: (100) 1.200 D + 1.600 Lp

Axial		Load (kip)	53.95
Moment	Top	Major(kip-ft)	52.89
		Minor(kip-ft)	52.04
Moment	Bottom	Major(kip-ft)	-19.29
		Minor(kip-ft)	-19.16

Calculated Parameters (Angle = 44.54 degrees): Ld/Cap = 0.52

0.81 Pn(kip):	53.95		
0.81 Mn Major(kip-ft):	101.28	0.81 Mn Minor(kip-ft):	99.65

	Major	Minor
Kl/r	56.87	56.87
Slender	Yes	Yes
10.13.5: lu/r > limit	No	No

TRANSVERSE REINFORCEMENT:

Controlling Load Combination: (2) 1.200 D + 1.600 Lp

	LC	Vu (kip)	Vc (kip)	Vs (kip)	φ	φ (Vc + Vs) (kip)	Ld/Cap
1 Major:	2	10.17	38.69	41.83	0.75	60.39	0.17
1 Minor:	2	10.04	38.69	41.83	0.75	60.39	0.17

TORSION CAPACITY:

0.75 Tn (kip-ft)	0.00	Tu (kip-ft)	0.00

APPENDIX G: CONCRETE COLUMN DESIGN

Concrete Column Design

RAM Concrete Column v15.04.00.000
Database: WHOLEsafe space new
Building Code: IBC

Page 33/120
02/23/18 15:02:53
Concrete Code: ACI 318-11

COLUMN INFORMATION:

Level	FOURTH FLOOR		
Column Number:	9	Grid Location:	(3-A)
Size:	**COLUMN**	Depth x Width (in)	18.00x18.00
Reinforcement			
Longitudinal:	**12-#5 (4 x 2)**	As (in²)	3.72 (1.15%)
Transverse:	**#4@ 9.0" 0'-0"-10'-0"**		
Confinement	Tie	Clear Cover (in)	1.50
Shear Legs Major	2	Shear Legs Minor	2

Longitudinal Bars Max Tension Stress Ratio: 0.79

MATERIAL PROPERTIES:

f'c (ksi):	4.00	fy Long (ksi):	60.00
fct (ksi):	0.00	fyt Trans (ksi):	60.00
Conc. Weight (pcf):	145.00	Conc. Type:	NWC
Conc. Modulus (ksi):	3644.15	Reinf. Modulus (ksi):	29000.00

DESIGN PARAMETERS:

	Major	Minor
Unbraced Length (ft)	8.67	8.67
K	2.48	3.35
Braced Against Sidesway	No	No

LONGITUDINAL REINFORCEMENT:

Controlling Load Combination: (2) 1.200 D + 1.600 Lp

Axial		Load (kip)	117.35
Moment	Top	Major(kip-ft)	0.33
		Minor(kip-ft)	30.28
Moment	Bottom	Major(kip-ft)	-0.48
		Minor(kip-ft)	-58.36

Calculated Parameters (Angle = 89.53 degrees): Ld/Cap = 0.40

0.65 Pn(kip):	117.35		
0.65 Mn Major(kip-ft):	1.19	0.65 Mn Minor(kip-ft):	144.80

	Major	Minor
Kl/r	49.54	67.14
Slender	Yes	Yes
10.13.5: lu/r > limit	No	No

TRANSVERSE REINFORCEMENT:

Controlling Load Combination: (2) 1.200 D + 1.600 Lp

	LC	Vu (kip)	Vc (kip)	Vs (kip)	ϕ	ϕ (Vc + Vs) (kip)	Ld/Cap
1 Major:	2	0.13	46.98	41.83	0.75	66.61	0.00
1 Minor:	2	13.29	46.98	41.83	0.75	66.61	0.20

TORSION CAPACITY:

0.75 Tn (kip-ft)	0.00	Tu (kip-ft)	0.00

APPENDIX G: CONCRETE COLUMN DESIGN

Concrete Column Design

RAM Concrete Column v15.04.00.000 — Page 43/120
Database: WHOLEsafe space new — 02/23/18 15:02:53
Building Code: IBC — Concrete Code: ACI 318-11

RAM Structural System
Bentley

COLUMN INFORMATION:

Level	THIRD		
Column Number:	22	Grid Location:	(6-B)
Size:	**COLUMN**	Depth x Width (in)	18.00x18.00
Reinforcement			
Longitudinal:	**12-#5 (4 x 2)**	As (in^2)	3.72 (1.15%)
Transverse:	**#4@ 9.0" 0'-0"-10'-0"**		
Confinement	Tie	Clear Cover (in)	1.50
Shear Legs Major	2	Shear Legs Minor	2

Longitudinal Bars Max Tension Stress Ratio: 0.40

MATERIAL PROPERTIES:

f'c (ksi):	4.00	fy Long (ksi):	60.00
fct (ksi):	0.00	fyt Trans (ksi):	60.00
Conc. Weight (pcf):	145.00	Conc. Type:	NWC
Conc. Modulus (ksi):	3644.15	Reinf. Modulus (ksi):	29000.00

DESIGN PARAMETERS:

	Major	Minor
Unbraced Length (ft)	8.67	8.67
K	3.35	2.48
Braced Against Sidesway	No	No

LONGITUDINAL REINFORCEMENT:

Controlling Load Combination: (100) 1.200 D + 1.600 Lp

Axial		Load (kip)	329.86
Moment	Top	Major(kip-ft)	-32.52
		Minor(kip-ft)	-1.82
Moment	Bottom	Major(kip-ft)	62.18
		Minor(kip-ft)	3.47

Calculated Parameters (Angle = 3.19 degrees): Ld/Cap = 0.48

0.65 Pn(kip):	329.86		
0.65 Mn Major(kip-ft):	182.42	0.65 Mn Minor(kip-ft):	10.17
	Major	Minor	
Kl/r	67.14	49.54	
Slender	Yes	Yes	
10.13.5: lu/r > limit	No	No	

TRANSVERSE REINFORCEMENT:

Controlling Load Combination: (2) 1.200 D + 1.600 Lp

	LC	Vu (kip)	Vc (kip)	Vs (kip)	ϕ	ϕ (Vc + Vs) (kip)	Ld/Cap
1 Major:	2	14.21	53.90	41.83	0.75	71.80	0.20
1 Minor:	2	0.79	53.90	41.83	0.75	71.80	0.01

TORSION CAPACITY:

0.75 Tn (kip-ft)	0.00	Tu (kip-ft)	0.00

APPENDIX G: CONCRETE COLUMN DESIGN

Concrete Column Design

RAM Structural System
Bentley

RAM Concrete Column v15.04.00.000	Page 65/120
Database: WHOLEsafe space new	02/23/18 15:02:53
Building Code: IBC	Concrete Code: ACI 318-11

COLUMN INFORMATION:

Level: SECONDResolution: SECOND
Column Number: 29 — Grid Location: (5-D)
Size: **COLUMN** — Depth x Width (in): 18.00x18.00

Reinforcement

Longitudinal: **12-#5 (4 x 2)** — As (in^2): 3.72 (1.15%)
Transverse: **#4@ 9.0" 0'-0"-10'-0"**
Confinement: Tie — Clear Cover (in): 1.50
Shear Legs Major: 2 — Shear Legs Minor: 2
Longitudinal Bars Max Tension Stress Ratio: 0.00

MATERIAL PROPERTIES:

f'c (ksi):	4.00	fy Long (ksi):	60.00
fct (ksi):	0.00	fyt Trans (ksi):	60.00
Conc. Weight (pcf):	145.00	Conc. Type:	NWC
Conc. Modulus (ksi):	3644.15	Reinf. Modulus (ksi):	29000.00

DESIGN PARAMETERS:

	Major	Minor
Unbraced Length (ft)	8.67	8.67
K	3.71	4.16
Braced Against Sidesway	No	No

LONGITUDINAL REINFORCEMENT:

Controlling Load Combination: (100) 1.200 D + 1.600 Lp

Axial		Load (kip)	318.31
Moment	Top	Major(kip-ft)	9.51
		Minor(kip-ft)	-21.76
Moment	Bottom	Major(kip-ft)	-0.48
		Minor(kip-ft)	1.58

Calculated Parameters (Angle = 66.40 degrees): Ld/Cap = 0.47

0.65 Pn(kip): 318.31
0.65 Mn Major(kip-ft): 67.07 — 0.65 Mn Minor(kip-ft): 153.53

	Major	Minor
Kl/r	74.25	83.24
Slender	Yes	Yes
10.13.5: lu/r > limit	No	No

TRANSVERSE REINFORCEMENT:

Controlling Load Combination: (2) 1.200 D + 1.600 Lp

	LC	Vu (kip)	Vc (kip)	Vs (kip)	ϕ	ϕ (Vc + Vs) (kip)	Ld/Cap
1 Major:	2	1.20	53.26	41.83	0.75	71.32	0.02
1 Minor:	2	2.85	53.26	41.83	0.75	71.32	0.04

TORSION CAPACITY:

0.75 Tn (kip-ft): 0.00 — Tu (kip-ft): 0.00

APPENDIX G: CONCRETE COLUMN DESIGN

Concrete Column Design

RAM Structural System
Bentley

RAM Concrete Column v15.04.00.000	Page 99/120
Database: WHOLEsafe space new	02/23/18 15:02:53
Building Code: IBC	Concrete Code: ACI 318-11

COLUMN INFORMATION:

Level	FIRST		
Column Number:	13	Grid Location:	(4-A)
Size:	**COLUMN**	Depth x Width (in)	18.00x18.00
Reinforcement			
Longitudinal:	**12-#7 (4 x 2)**	As (in²)	7.20 (2.22%)
Transverse:	**#4@ 3.0" 0'-0"-0'-1"**		
Confinement	Tie	Clear Cover (in)	1.50
Shear Legs Major	2	Shear Legs Minor	2

Longitudinal Bars Max Tension Stress Ratio: 0.37

MATERIAL PROPERTIES:

f'c (ksi):	4.00	fy Long (ksi):	60.00
fct (ksi):	0.00	fyt Trans (ksi):	60.00
Conc. Weight (pcf):	145.00	Conc. Type:	NWC
Conc. Modulus (ksi):	3644.15	Reinf. Modulus (ksi):	29000.00

DESIGN PARAMETERS:

	Major	Minor
Unbraced Length (ft)	0.01	0.01
K	2.34	2.35
Braced Against Sidesway	No	No

LONGITUDINAL REINFORCEMENT:

Controlling Load Combination: (2) 1.200 D + 1.600 Lp

Axial		Load (kip)	518.45
Moment	Top	Major(kip-ft)	-0.67
		Minor(kip-ft)	189.83
Moment	Bottom	Major(kip-ft)	-0.66
		Minor(kip-ft)	187.27

Calculated Parameters (Angle = 89.80 degrees): Ld/Cap = 0.96

0.65 Pn(kip):	518.45		
0.65 Mn Major(kip-ft):	0.70	0.65 Mn Minor(kip-ft):	198.38

	Major	Minor
Kl/r	0.05	0.05
Slender	No	No
10.13.5: lu/r > limit	No	No

TRANSVERSE REINFORCEMENT:

Controlling Load Combination: (100) 1.200 D + 1.600 Lp

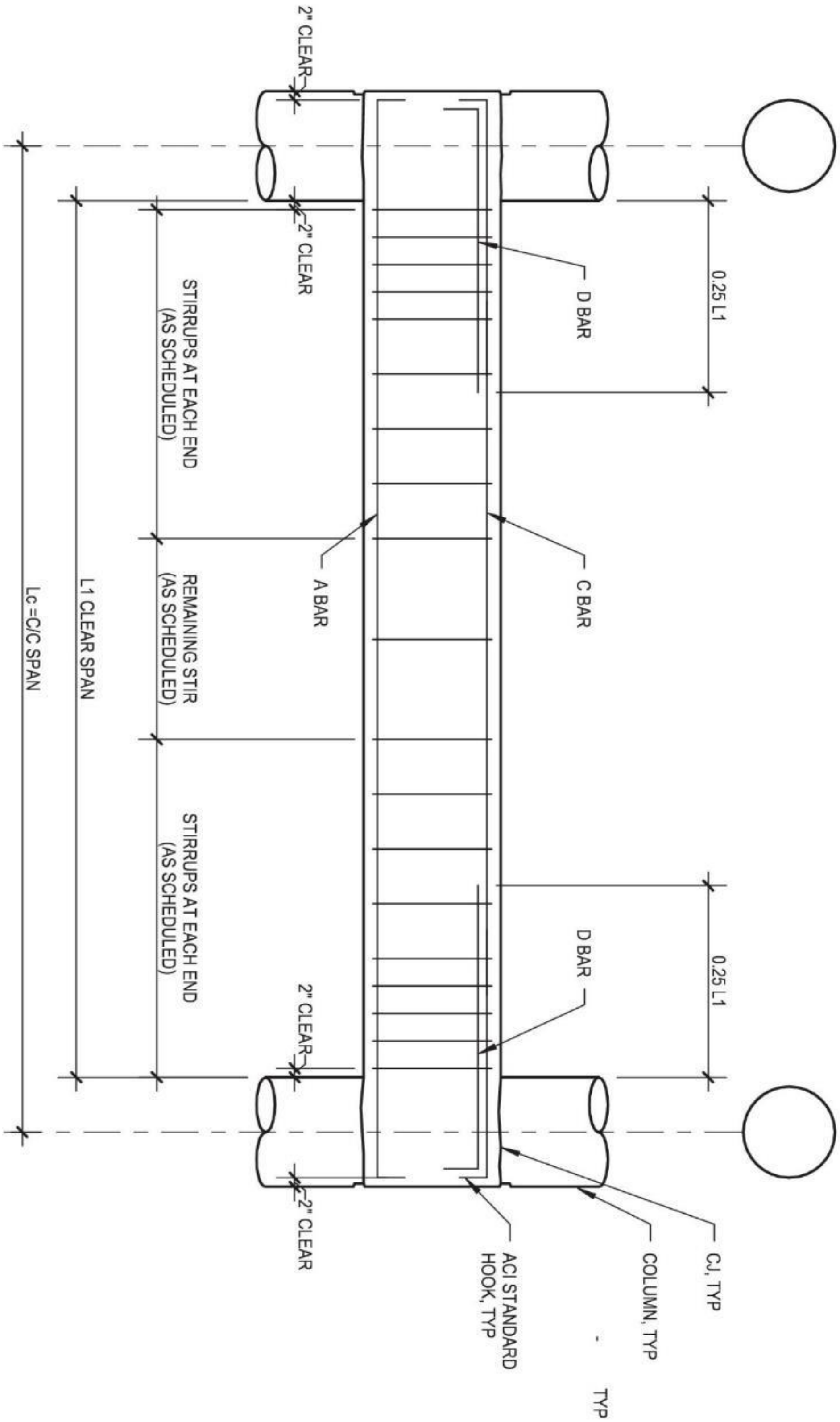
	LC	Vu (kip)	Vc (kip)	Vs (kip)	ϕ	ϕ (Vc + Vs) (kip)	Ld/Cap
1 Major:	100	38.11	51.60	124.50	0.75	132.07	0.29
1 Minor:	100	10327.92	51.60	124.50	0.75	132.07	78.20

Design Warning:

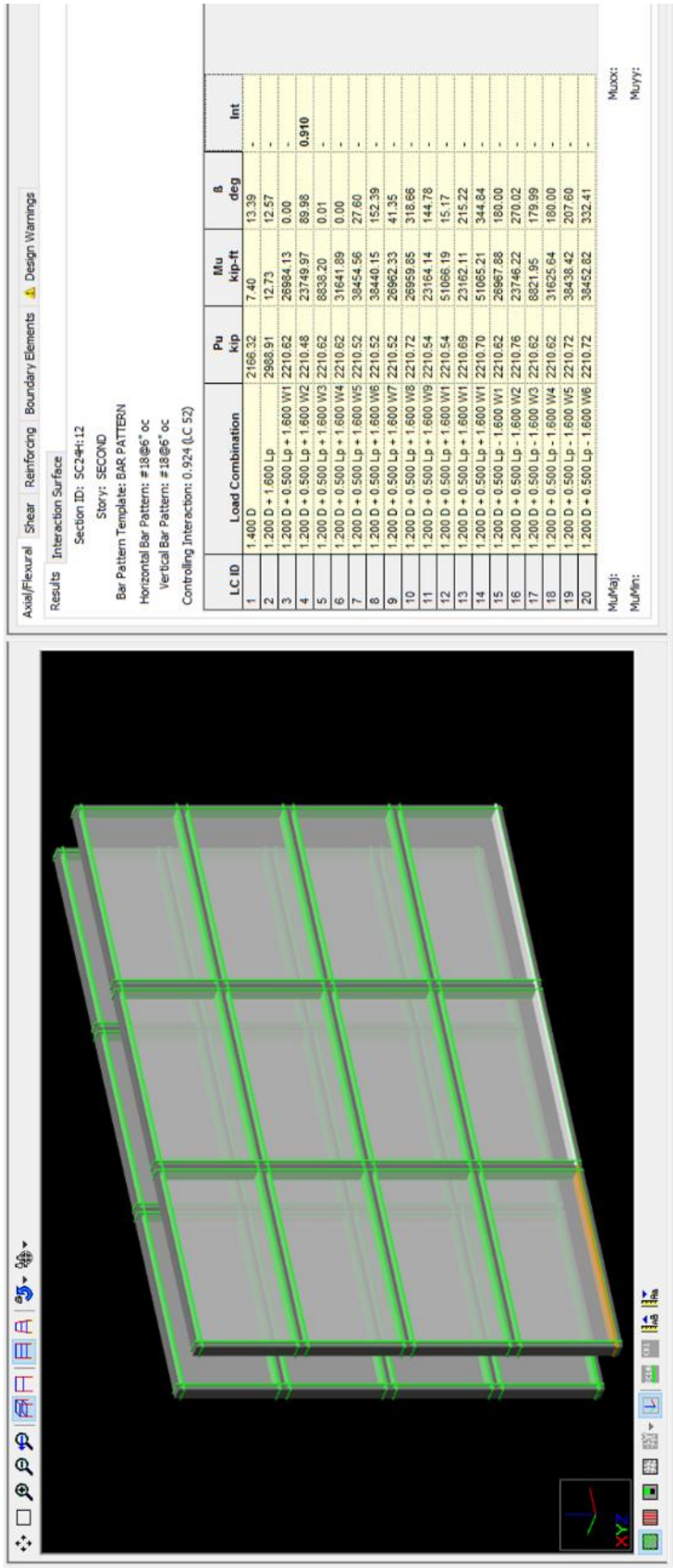
Segment # 1:

No acceptable bar sets could be found to resist shear - Increase number of shear legs or transverse bar size in pattern group

APPENDIX H: SAMPLE CONCRETE BEAM DETAIL



APPENDIX I: CONCRETE SHEAR WALL



Axial/Flexural | Shear | Reinforcing | Boundary Elements | Design Warnings

Results | Interaction Surface

Section ID: SC24H:12

Story: SECOND

Bar Pattern Template: BAR PATTERN

Horizontal Bar Pattern: #18@6" oc

Vertical Bar Pattern: #18@6" oc

Controlling Interaction: 0.924 (LC 52)

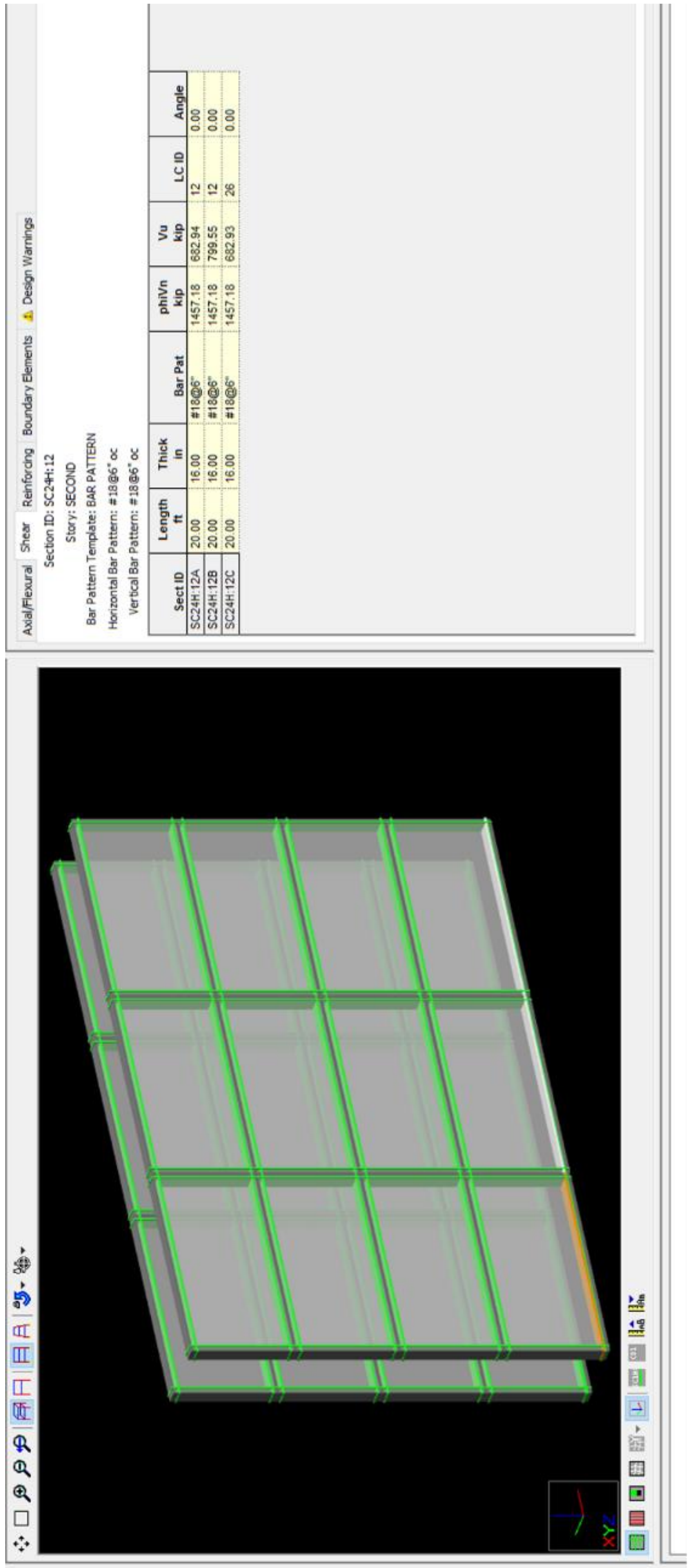
LC ID	Load Combination	Pu kip	Mu kip-ft	ß deg	Int
1	1.400 D	2166.32	7.40	13.39	-
2	1.200 D + 1.600 Lp	2988.91	12.73	12.57	-
3	1.200 D + 0.500 Lp + 1.600 W1	2210.62	26984.13	0.00	-
4	1.200 D + 0.500 Lp + 1.600 W2	2210.48	23749.97	89.98	0.910
5	1.200 D + 0.500 Lp + 1.600 W3	2210.62	8838.20	0.01	-
6	1.200 D + 0.500 Lp + 1.600 W4	2210.62	31641.89	0.00	-
7	1.200 D + 0.500 Lp + 1.600 W5	2210.52	38454.56	27.60	-
8	1.200 D + 0.500 Lp + 1.600 W6	2210.52	38440.15	152.39	-
9	1.200 D + 0.500 Lp + 1.600 W7	2210.52	26962.33	41.35	-
10	1.200 D + 0.500 Lp + 1.600 W8	2210.72	26959.85	318.66	-
11	1.200 D + 0.500 Lp + 1.600 W9	2210.54	23164.14	144.78	-
12	1.200 D + 0.500 Lp + 1.600 W1	2210.54	51066.19	15.17	-
13	1.200 D + 0.500 Lp + 1.600 W1	2210.69	23162.11	215.22	-
14	1.200 D + 0.500 Lp + 1.600 W1	2210.70	51065.21	344.84	-
15	1.200 D + 0.500 Lp - 1.600 W1	2210.62	26967.88	180.00	-
16	1.200 D + 0.500 Lp - 1.600 W2	2210.76	23746.22	270.02	-
17	1.200 D + 0.500 Lp - 1.600 W3	2210.62	8821.95	179.99	-
18	1.200 D + 0.500 Lp - 1.600 W4	2210.62	31625.64	180.00	-
19	1.200 D + 0.500 Lp - 1.600 W5	2210.72	38438.42	207.60	-
20	1.200 D + 0.500 Lp - 1.600 W6	2210.72	38452.82	332.41	-

MuMaj: Muxx:

MuMin: Muyy:

AXIAL / FLEXURAL CHECK

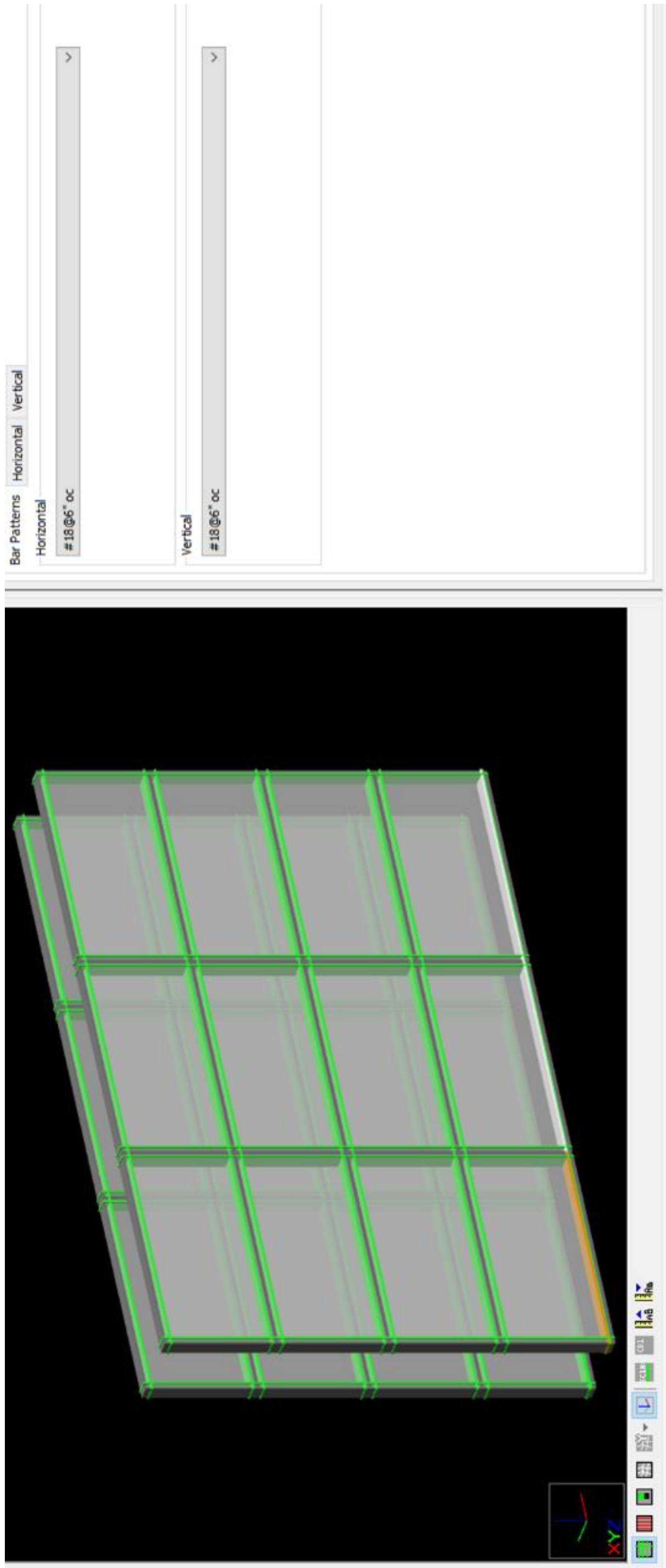
APPENDIX I: CONCRETE SHEAR WALL



Sect ID	Length ft	Thick in	Bar Pat	phiVn kip	Vu kip	LC ID	Angle
SC24H:12A	20.00	16.00	#18@6"	1457.18	682.94	12	0.00
SC24H:12B	20.00	16.00	#18@6"	1457.18	799.55	12	0.00
SC24H:12C	20.00	16.00	#18@6"	1457.18	682.93	26	0.00

SHEAR CHECK

APPENDIX I: CONCRETE SHEAR WALL



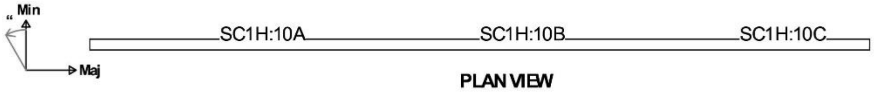
REINFORCEMENT

APPENDIX I: CONCRETE SHEAR WALL

Section Cut Design Summary

RAM Concrete Shearwall 15.00.00.000 | Page 9/144
Database: NEW RAM MODEL | 01/12/18 11:58:02
Design Code: ACI 318-11

Section Cut ID: SC1H:10 (Horizontal)
Story: FIRST
Ag = 7200 in2 Imaj = 311039972 in4 Imin = 60000 in4
Major Axis Orientation: 0.00 degrees (CCW from global X-axis)
Wall Design Group: 1
Design Status: FAILS



PLAN VIEW

Axial/Flexural Results:
Interaction: 1.004 NG
Pu = 1913.45 kips phiPn = 1906.56 kips
Mu = 8692.4 kip-ft at Beta = -90.0 deg CCW from Major axis
Controlling Load Combo: 1.200 D + 0.500 Lp - 1.600 W2 (LC 16)
Code Ref: 10.3.7

Shear Results:
Segment SC1H:10A:
Length = 20.00 ft Thick = 10.00 in f'c = 4000 psi fy = 60 ksi
Vert Bar Pat: #18@6" oc Horiz Bar Pat: #18@6" oc
Vu = 498.7 kip phiVn = 910.7 kip OK
Controlling Load Combo: 1.200 D + 0.500 Lp + 1.600 W12 (LC 14)
Code Ref: 14.2.3 & 11.9.5

Segment SC1H:10B:
Length = 20.00 ft Thick = 10.00 in f'c = 4000 psi fy = 60 ksi
Vert Bar Pat: #18@6" oc Horiz Bar Pat: #18@6" oc
Vu = 590.9 kip phiVn = 910.7 kip OK
Controlling Load Combo: 1.200 D + 0.500 Lp + 1.600 W10 (LC 12)
Code Ref: 14.2.3 & 11.9.5

Segment SC1H:10C:
Length = 20.00 ft Thick = 10.00 in f'c = 4000 psi fy = 60 ksi
Vert Bar Pat: #18@6" oc Horiz Bar Pat: #18@6" oc
Vu = 498.6 kip phiVn = 910.7 kip OK
Controlling Load Combo: 1.200 D + 0.500 Lp - 1.600 W10 (LC 24)
Code Ref: 14.2.3 & 11.9.5

Reinforcement Checks:
Min Vert Reinf Ratio: Limit: 0.250% Actual: 13.336 % (11.9.9.4) OK

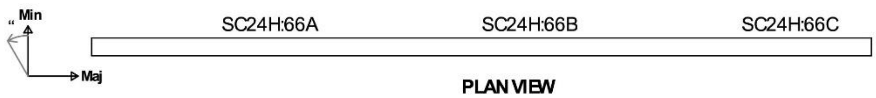
Segment SC1H:10A:

APPENDIX I: CONCRETE SHEAR WALL

Section Cut Design Summary

RAM Concrete Shearwall 15.04.00.000
Database: WHOLEsafe space new 02/23/18 15:44:11
Design Code: ACI 318-11

Section Cut ID:	SC24H:66 (Horizontal)		
Story:	ROOF		
Ag = 11520 in2	Imaj = 497663956 in4	Imin = 245760 in4	
Major Axis Orientation:	0.00 degrees (CCW from global X-axis)		
Wall Design Group:	24		
Design Status:	PASS		



Axial/Flexural Results:

Interaction:	0.034	OK	
Pu =	749.89 kips	phiPn =	21765.87 kips
Mu =	5.3 kip-ft	at Beta =	-83.7 deg CCW from Major axis
Controlling Load Combo:	1.200 D + 1.600 Lp (LC 2)		
Code Ref:	10.3.7		

Shear Results:

Segment SC24H:66A:

Length = 20.00 ft	Thick = 16.00 in	f'c = 4000 psi	fy = 60 ksi	
Vert Bar Pat: #4@6" oc	Horiz Bar Pat: #4@9" oc			
Vu =	206.6 kip	phiVn =	668.4 kip	OK
Controlling Load Combo:	1.200 D + 0.500 Lp + 1.400 E2 (LC 76)			
Code Ref:	14.2.3 & 11.9.5			

Segment SC24H:66B:

Length = 20.00 ft	Thick = 16.00 in	f'c = 4000 psi	fy = 60 ksi	
Vert Bar Pat: #4@6" oc	Horiz Bar Pat: #4@9" oc			
Vu =	231.2 kip	phiVn =	668.4 kip	OK
Controlling Load Combo:	1.200 D + 0.500 Lp + 1.400 E2 (LC 76)			
Code Ref:	14.2.3 & 11.9.5			

Segment SC24H:66C:

Length = 20.00 ft	Thick = 16.00 in	f'c = 4000 psi	fy = 60 ksi	
Vert Bar Pat: #4@6" oc	Horiz Bar Pat: #4@9" oc			
Vu =	206.5 kip	phiVn =	668.4 kip	OK
Controlling Load Combo:	1.200 D + 0.500 Lp - 1.400 E2 (LC 80)			
Code Ref:	14.2.3 & 11.9.5			

Reinforcement Checks:

Min Vert Reinf Ratio: Limit: 0.250% Actual: 0.412% (11.9.9.4) OK
Segment SC24H:66A:
Max Vert Bar Spacing Limit: 18.00 in Actual: 6.00 in (11.9.9.5) OK

APPENDIX J: SHEAR WALL VERIFICATION

LUCY AMPAW-ASIEDU THESIS

SHEAR WALL VERIFICATIONS

Wall thickness = 16in

f'_c = 4000 psi

#18 @ 6in o/c for both vertical & horizontal reinforcement

Grade 60 $f_y = f_{yt}$ = 60ksi

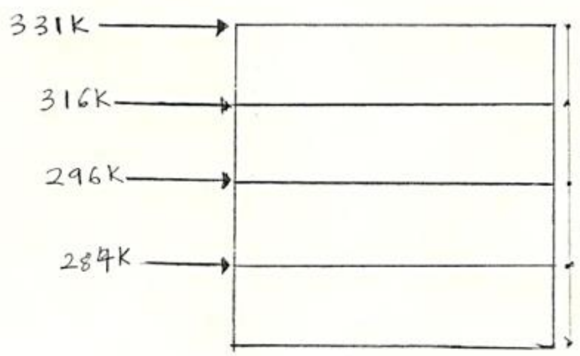
Length = 20ft (240in).

Initial check of wall reinforcement

ACI code section 11.6.2

$$\rho_t = \frac{A_{v,horiz}}{h s_2} = \frac{2 \times 2.25}{16in \times 6in} = 0.047$$ OK

$$\rho_l = \frac{2 \times 2.25}{16in \times 6in} = 0.047$$ OK.



$$M_{base} = (331 \times 40) + (316 \times 30) + (296 \times 20) + (284 \times 10)$$

$$= 31,480.\text{Kip-ft}$$

Factored Moment

$$M_u(base) = 1.0\,(31,480) = 31480\ \text{Kip-ft}$$

APPENDIX J: SHEAR WALL VERIFICATION

LUCY AMPAW-ASIEDU THESIS

$P_u = 2210.76$ kip

for f'c concrete = $\beta_1 = 0.85$

$$\omega = \rho l \frac{f_y}{f'c} = 0.047 \times \frac{60}{4}$$

$$= \underline{0.705}$$

$$a = \frac{P_u}{h l_w f'c} = \frac{2210.76}{16in \times 240 \times 4} = \underline{0.144}$$

$$c = \left(\frac{a + \omega}{0.85 \times 0.85 + 2\omega}\right) = \frac{0.144 + 0.705}{0.85 \times 0.85 + 2(0.705}$$

$$= \underline{0.398}$$

$d = \simeq 0.8 l_w$ (20ft) = 192 in

$c \simeq < 0.375 d \Rightarrow$ tension controlled

$\therefore \phi = 0.9$

$$A_{st} = 2_{Ab} \frac{l_w}{s_1} = 2 \times 2.25 \times \frac{240}{6} = \underline{178.4 in^2}$$

$$T = A_{st} f_y \left(\frac{l_w - c}{l_w}\right)$$

$$= 178.4 \times 60 \left(\frac{240 - 0.398}{240}\right)$$

$$= \underline{10,686.2 \text{ k}}$$

$$M_n = T\left(\frac{l_w}{2}\right) + N_u\left(\frac{L_w - c}{2}\right)$$

$$= 10,686\left(\frac{240}{2}\right) + 2210\left(\frac{240 - 0.398}{240}\right)$$

$$= \underline{128,923 \text{ Kip-ft}}$$

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APPENDIX J: SHEAR WALL VERIFICATION

LUCY AMPAW-ASIEDU THESIS

$\phi M_n = 0.9 \times 128{,}923$
$= 116{,}030$ Kip-ft.

but $M_u = 51{,}066$ Kip-ft

$116{,}030$ Kip-ft $> 51{,}066$ Kip-ft

$\phi M_n > M_u$

∴ Flexural strength is OK

Check Shear Strength

$V_u(base) = 331 + 316 + 296 + 284$
$= \underline{1227\,k}$

$$V_c = 3.3 \times \sqrt{f'c}\, h d + \frac{N_u d}{4 l_w}$$

$$V_c = \left[0.6 \times \sqrt{f'c} + \frac{l_w\left(1.25 \times \sqrt{f'c} + 0.2\frac{N_u}{l_w h}\right)}{\frac{M_u}{V_u} - \frac{l_w}{2}}\right] h d$$

lesser of the two

$$V_c = 3.3 \times \sqrt{f'c}\, h d + \frac{N_u d}{4 l_w}$$
$$= 3.3(1)\sqrt{4000} \times 16 \times 192 + \frac{2{,}210{,}000 \times 192}{4(240)}$$

$= \underline{1083\,k}$

$$M_u(\text{critical sec}) = M_u(base) - V_u(base)\frac{l_w}{2}$$
$$= 31480 - 1227\left(\frac{20}{2}\right)$$
$= 19{,}210$ Kip-ft

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APPENDIX J: SHEAR WALL VERIFICATION

LUCY AMPAW-ASIEDU THESIS

$$\frac{M_u}{V_u} = \frac{19210}{1227} = 15.6$$

$$V_c \left[0.6\sqrt{4000} + \frac{240\left(1.25\sqrt{4000} + 0.2\frac{(2210,00)}{16 \times 24}\right)}{5.6} \right] 16 \times 192$$

$= \underline{2567k}$

$\therefore V_c = 1083k$

$\phi = 0.75$

$\phi V_c = 0.75 \times 1083 = \underline{812.25k}$

$V_c < V_u$ so V_s is required for provided horizontal / transverse reinforcement

But $\phi V_n = 1457k$ and $V_u = 1227k$

$\therefore \phi V_n > V_u$

$\therefore$ Shear strength is ok

ok

<u>Check reinforcements limits</u>

$0.5\phi V_c < V_u$

$0.5(812.25) < 1227$

$406.125 < 1227$

ok.

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APPENDIX K: DRIFT LIMIT CALCULATION

Floors	Height (ft)	Displacement about x-axis (in)	Displacement about y-axis (in)	Maximum Displacement (in)	Status
ROOF	10	0.193	0.307	0.32	OK
FOURTH	10	0.192	0.311	0.32	OK
THIRD	10	0.192	0.310	0.32	OK
SECOND	10	0.160	0.207	0.32	OK
FIRST	10	0.128	0.19	0.32	OK